US ARMY CORPS
OF ENGINEERS
NEW YORK DISTRICT

RARITAN BAY AND SANDY HOOK BAY, NEW JERSEY FEASIBILITY REPORT FOR HURRICANE AND STORM DAMAGE REDUCTION PORT MONMOUTH, NJ

VOLUME III
TECHNICAL APPENDICES

APPENDIX A- ENGINEERING AND DESIGN, APPENDIX B - BENEFITS,
APPENDIX C - QUANTITIES AND COSTS, APPENDIX D - BORROW AREA,
APPENDIX E - GEOTECHNICAL, APPENDIX F - INTERIOR DRAINAGE,
APPENDIX H - MONITORING, APPENDIX J - REAL ESTATE

June 2000

U.S. Army Corps of Engineers
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APPENDIX A

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RARITAN BAY AND SANDY HOOK BAY, NEW JERSEY
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PORT MONMOUTH COMBINED FLOOD CONTROL AND SHORE PROTECTION FEASIBILITY STUDY

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INTRODUCTION

Description of Project Area and Vicinity

A-1. The project area consists of approximately 6,000 feet of shoreline in Middletown Township, in Monmouth County, central New Jersey, extending along the shoreline of Sandy Hook Bay (Figure A-1). Pews Creek and Compton Creek are waterways that represent the western and eastern limits of the project area, respectively. The southern limit of the study area is the existing inland 15 foot NGVD contour line, which lies a short distance south of Route 36. The northern project area limit is the location of the closure depth contour within Sandy Hook Bay.

A-2. To the west the study area is East Keansburg and (at the shoreline) Ideal Beach. To the east is Belford, with Belford Harbor at the mouth of Compton Creek. From its 6,000 foot maximum east-west dimension at the shoreline, the width of Port Monmouth reaches a minimum of about 4,000 feet, as defined by the separation between the bordering creeks. The Port Monmouth project area extends approximately 8,000 feet inland. The shoreline consists of a small beach and dune system fronting an extensive low-lying marshland. The wetlands form the drainage basins of Pews and Compton Creeks, covering a combined area of 5.0 square miles and 8.5 river miles. To the west of Pews Creek is a levee with nominal crest elevation of 15 feet which runs south from the mouth of the creek for a distance of 6,000 feet. The levee is part of the Raritan Bay and Sandy Hook Bay Beach Erosion and Hurricane project for Keansburg.

A-3. Port Monmouth is composed of mostly residential structures. Small homes along the shoreline which were formerly summer residences have been converted to year-round homes. Many year-round homes are constructed within the marsh between the banks of Pews and Compton Creeks. Near the center of the Port Monmouth shoreline, at the end of Wilson Avenue, lies the historic "Spy House," which dates back to the Revolutionary War. A small pleasure boat marina is located near the entrance of Pews Creek and a large commercial marina known as Shoal Harbor is located at the mouth of Compton Creek. The commercial marina contains storage buildings and a fish market. The fish market is located on the west side of Shoal Harbor, on the site of a former fish processing factory.

A-4. Compared to other regions in the study area, Port Monmouth has undergone substantial changes since 1960. Many new residential homes have been constructed within the interior of the wetlands and improvements in recreational facilities have occurred.

A-5. A recent site inspection of the Port Monmouth area provided an indication of the amount of new development within the flood plain. It appears that almost every existing

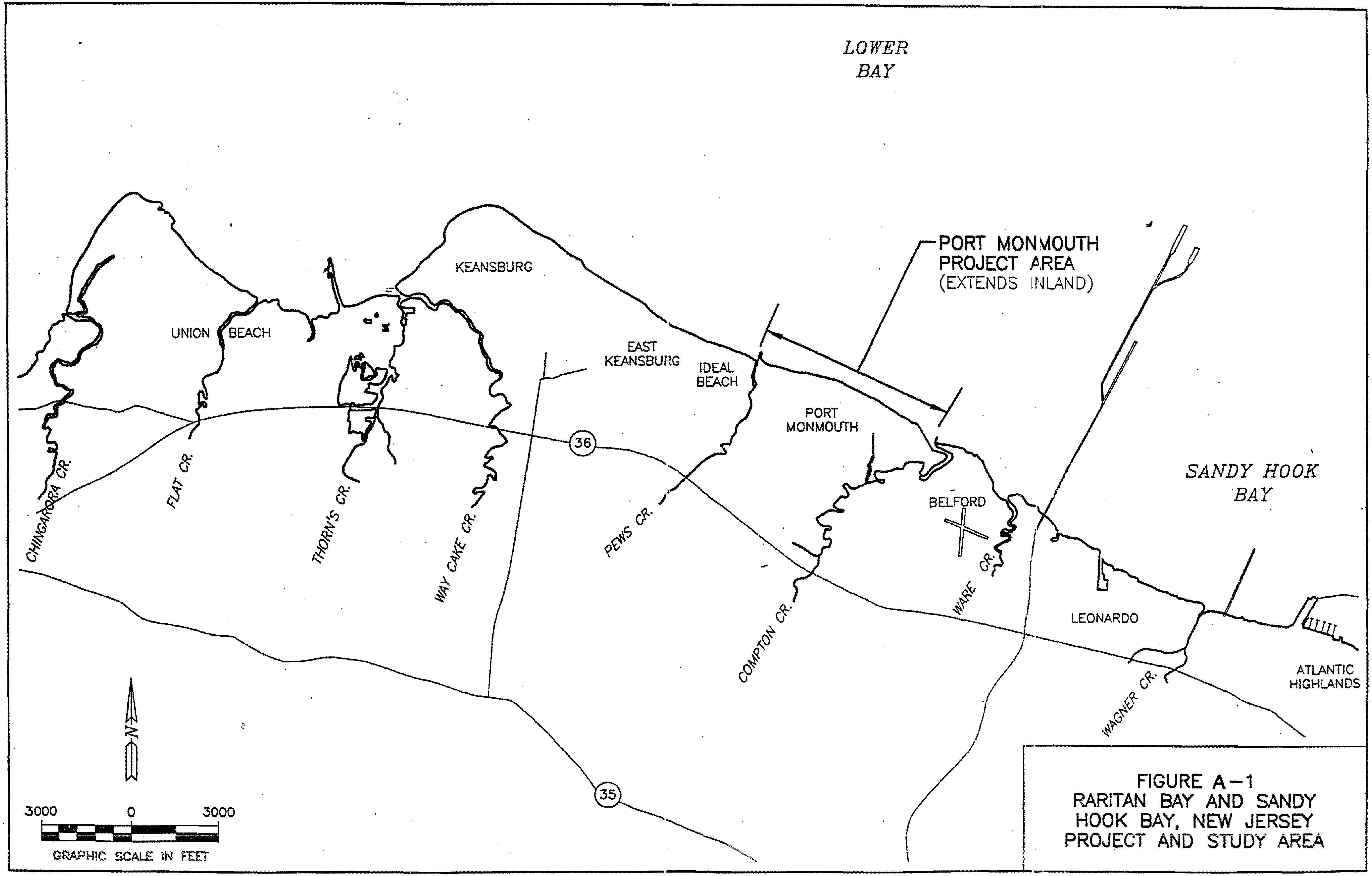


FIGURE A-1
RARITAN BAY AND SANDY HOOK BAY, NEW JERSEY
PROJECT AND STUDY AREA

buildable lot within the flood plain has been developed. A recreational fishing pier exists near the Spy House.

Description of the Study Area

A-6. To the west of Sandy Hook Bay lies Raritan Bay. To its north lies the Lower (New York) Bay and Long Island, New York. The eastern terminus of the bay is Sandy Hook, New Jersey, a low-lying peninsula separating the Atlantic Ocean from Sandy Hook Bay. The approach to the bay from the Atlantic Ocean is through a 6-mile wide opening between Sandy Hook and Rockaway Point, a spit of land located northeast of Sandy Hook at the southernmost extremity of Queens, New York.

A-7. The lands adjacent to Sandy Hook and Raritan Bays range from high bluffs well east and west of the Port Monmouth project area to low marshlands. These marshlands frequently become partially inundated by spring tides. Low narrow beaches front most of the area. A number of tidal creeks, such as Pews Creek and Compton Creek, intersect the shoreline. The region has a maritime climate with warm summers and moderate winters. Precipitation averages approximately 45 inches annually, with July and August being the wettest months. Snowfall, which can occur from October through May, averages almost 25 inches annually.

Shoreline Ownership

A-8. While continued development along the Port Monmouth shoreline is minimal, significant changes in ownership are underway. Monmouth County has acquired much of the land adjacent to the Spy House for the development of a County park and recreational area. This purchase will include the Port Monmouth shoreline in a park system extending along the shoreline of both Sandy Hook and Raritan Bays (Sardonia, 1996). Currently, seaward of Port Monmouth Road, a significant portion of the land is publicly held, and little development exists. Near Pews Creek a few small residences remain. Landward of Port Monmouth Road is a mixture of private development and undeveloped lands, with some commercial development.

Coastal History and Status of Project Area

A-9. The shoreline of Port Monmouth has been protected by a variety of measures, some of which continue to provide limited coastal protection. In 1966 a beach fill and dune project was conducted by the State of New Jersey and approximately 540,000 cubic yards was placed (NJ, 1966). These currently include an artificial dune system constructed by local interests. Dredged fill from Pews Creek has been placed on the beach in the vicinity of the historic "Spy House" near the center of Port Monmouth. The latest fill project occurred in or near the time 1992, involving the placement of approximately 20,000 cubic yards of sand. Most of this material has eroded. In recent years, four low profile wood sheet pile groins have been constructed near the middle of the Port Monmouth shoreline.

A-10. In 1960, the majority of the Sandy Hook and Raritan Bays was authorized for hurricane protection. While this resulted in improvements elsewhere in the study area, Port Monmouth did not receive improvements. The authorizing report, however, provides survey maps indicating shoreline improvements in place at that time. These maps indicate jetties in place at both creeks. Along the beach, a total of 12 timber groins were concentrated along the central to eastern length of shoreline, with a single groin approximately 300 feet from Pews Creek. The groins featured a typical length of 150 feet, described as being in fair to poor physical condition. Private, state, and municipal interests had constructed all of the groins prior to 1943.

A-11. Middletown Township and Monmouth County are presently developing plans to enhance and improve the Port Monmouth area. In addition to the planned shoreline park, the Monmouth County Planning Board has indicated in its 1987 Bayshore Waterfront Access Plan that the County could benefit from the revitalization of the historic Shoal Harbor district at the mouth of Compton Creek. Monmouth County has recently replaced the Pews Creek bridge. Finally, Port Monmouth Road, Church Street, and Broadway were recently raised by Monmouth County to a minimum elevation of +9.0 ft. NGVD so that emergency access is available to residents of Port Monmouth during severe flooding events.

A-12. As indicated by the amount of redevelopment planned for Port Monmouth there is great local interest in flood control and shore protection projects within the region. The Middletown Township Engineer has expressed interest in combining the road raising project with a possible hurricane/flood control project. Additionally, the County Park System and Bayshore Trail could be tied into a Federal protection project. Middletown Township has indicated a willingness to continue flood control protection investigations, and has proven to be a supportive sponsor of the East Keansburg Flood Control and Hurricane Protection project.

A-13. The creeks of the study area have also undergone a history of maintenance. Much of this has impacted the project area, or may contribute to its future protection measures. Details of this history follow:

A-14. Drainage Structures. Interior stormwater drainage systems are nonexistent along the Port Monmouth shoreline, limited to those within the interior wetlands. Twelve outfalls discharge into Compton Creek and six into Pews Creek from the low-lying marshland. Compton and Pews Creeks, however, receive a considerable amount of drainage from heavily developed upland areas. In total, Compton Creek provides drainage for 85 stormwater outfalls and Pews Creek accepts water from 29 drainage pipes. Detention basins have been constructed along Compton Creek to store stormwater runoff and reduce riverine flooding. Review of the Middletown Township Flood Insurance Study (FEMA) indicates that the 100 year fluvial flood stage for Compton Creek is 7.0 ft. NGVD, more than 4 feet below the 100 year tidal flood stage.

A-15. Pews Creek. This creek divides Port Monmouth and Keansburg and is maintained by local interests. Its mouth is dredged periodically for navigation. The navigation project consists of a channel to the Monmouth Cove Marina. A 350 foot long stone jetty stabilizes the east bank of the bay entrance. Parallel bulkheads approximately 425 feet long stabilize the channel from the bay shoreline to the marina. The inlet channel is approximately 60 feet wide through this reach. The marina entrance is located approximately 825 feet from the tip of the jetty. The marina size is approximately 115,000 square feet and borders the creek for approximately 500 feet (Harrington, 1994). Dredge depths of the project are 8 feet MLW for the channel and 6 feet MLW for the basin (Sardonia, 1996), which is designed to maintain 6 ft. MLW and 5 ft. MLW navigation depths, respectively.

A-16. In 1988, a reported 20,796 cubic yards of material was removed from Pews Creek or its basin (Table A-1). The dredged material was truck-hauled and placed on the Port Monmouth shoreline near the Spy House. In the early 1990's, Monmouth County assumed control of the marina and navigation project. The Monmouth County Recreation Department now maintains the project. Since 1992, the inlet and boat basin have been dredged four times. Quantities dredged from the inlet were estimated during the early dredging projects, but are measured more accurately since 1994.

A-17. The 1994 dredging project showed high volume largely because the dredge contractor exceeded the desired channel dredge volume of 8,000 cy by removing 12,900 cy. The 1994 project cost $25,000 for mobilization, and $9/cy for channel dredging and $10/cy of marina basin dredging.

A-18. Since 1988, approximately 58,300 cy (7,300 cy/yr) has been removed from the inlet, but not all of this material is suitable for beach disposal. Since 1992, approximately 26,300 cy (6,600 cy/yr) of coarse sand has been dredged from the inlet channel. Grain size measurements taken by the County in 1993 show the channel sand with a mean grain size of 0.29 mm. Sand dredged from the offshore portions of the channel is fine. Sand located seaward of the jetty tip has a mean grain size of 0.18 mm.

A-19. Dredge spoils are typically stockpiled east of the creek for later use or final disposal. The coarsest sand dredged from the channel is stockpiled adjacent to the bay shoreline east of the channel (Photo A-1). This material is used to support County projects, such as the 1995-96 construction on the Port Monmouth Road (Photo A-2). The fine material dredged from the basin is stockpiled directly east of the marina for later removal.

TABLE A-1
PEW'S CREEK & MONMOUTH COVE MARINA MAINTENANCE DREDGING QUANTITIES

YEAR	DREDGE VOLUME (CY)			NOTE
	CHANNEL	BASIN	TOTAL	
1988	?	?	20,796	(3)
1990-91	COUNTY ASSUMED CONTROL			
1992	5,000	2,000	7,000	(1) (4)
1993	5,000	2,000	7,000	(1) (4)
1994	12,900	5,130	18,030	(2) (4)
1995	1,929	2,073	4,002	(2) (4)
	1,500		1,500	(1) (4)
TOTAL	26,329	11,203	58,328	
	SHOALING RATE (CY/YR)			
	CHANNEL (SINCE 1988)	BASIN (SINCE 1988)	TOTAL (SINCE 1988)	
	3,761	1,600	5,362	

(1): ESTIMATE

(2): MEASURED

(3): USACE (1993)

(4): SARDONIA, 1996

Photo A-1
Sand stockpile at Port Monmouth adjacent to Pews Creek Marina.
September, 1995

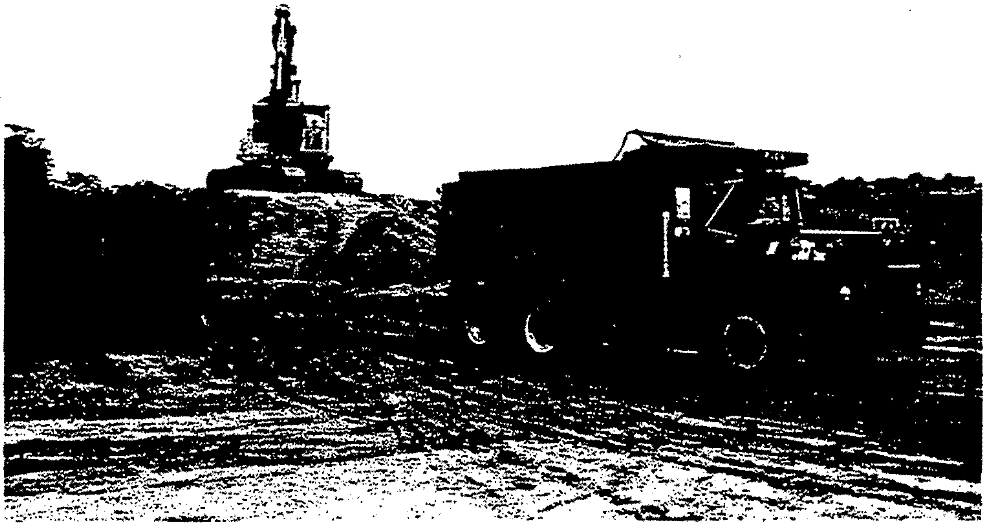


Photo A-2
Stockpile sand being utilized for road raising project.
September, 1995

A-20. The County has plans to improve navigation in the inlet. The high shoaling rate in the inlet is thought to be compounded by the location of the coarse sand disposal site (updrift) and the lack of a western jetty. The county plans to dispose of all future coarse sand on the west (Keansburg) side of the inlet. Plans also include the construction of a western jetty to diminish shoaling in the inlet. To improve navigation, inlet widening by approximately 25 feet is being considered (Sardonia, 1996).

Federal Navigation Projects

A-21. Port Monmouth lies near several Federal Navigation Projects. These projects are significant in their contribution to the local economy, their demands on Port Monmouth, and their potential supply of spoil material.

A-22. Compton Creek. This creek separates Belford and Port Monmouth and is a Federal navigation project. A rock rubble jetty stabilizes the east side of the inlet. A concrete seawall protects the south side of the east channel bank. The west side of the inlet is protected by a timber terminal groin along the shoreline, and a wooden bulkhead landward. In areas, the bulkhead is backed by rock rubble. The project depth inside the mouth of the creek is 8 feet mean low water (MLW) with a width of 75 feet. Outside of the mouth the project depth is 12 feet MLW with a width of 150 feet. The ocean channel extends 1.3 miles into Sandy Hook Bay.

A-23. Table A-2 delineates maintenance dredging quantities from Compton Creek. Dredging has been accomplished on 15 occasions, with intervals averaging 3.5 years. Since 1968 through 1990, the annual volume removed was 22,900 cubic yards. Thus, the maintenance of Compton Creek is more extensive than at Pews Creek.

A-24. Earle Naval Weapons Pier Channel. The U.S. Navy Pier is located 1.2 miles east of Port Monmouth and consists of a channel connecting its many berths to the Sandy Hook Channel. The berths and channel for the pier are undergoing major expansion in addition to periodic maintenance dredging. Recent dredging activity is as follows (Mahoney, 1996):

> 1989: A deep berth construction dredging project was undertaken. The dredged material was a combination of suitable and unsuitable beach quality material. The upper layers were fine sand, which were disposed at an offshore mud dump. Lower layers contained beach quality material, which was placed as beach nourishment on Sandy Hook.
>
> 1995: Maintenance dredging was conducted in the 35 foot berth area. Material was not beach quality and was disposed in the mud dump.

TABLE A-2
COMPTON CREEK MAINTENANCE DREDGING QUANTITIES
MONMOUTH COUNTY, N.J.
(CUBIC YARDS)

YEAR	DREDGED VOLUME	FILL LOCATION	FILL VOLUME
1937	155,998		
1941	61,575		
1945	62,281		
1947	32,555		
1950	19,588		
1951	63,536		
1956	10,400		
1957	170,531	BELFORD	100,000
1957	61,051		
1962	87,126		
1968	107,644		
1972	141,552		
1978	143,847		
1984	140,628	P. MONMOUTH	70,000
1990	78,500		

USACE, 1993

A-25. Lower Bay Navigation Channels. With the existing network of shipping channels throughout Lower Bay (including adjoining Raritan and Sandy Hook Bays), extensive maintenance dredging is conducted. The New York District of the Corps of Engineers has compiled statistics for the maintenance dredged material for the years 1980-1990. Throughout the bays, 13,750,000 cubic yards of material (6,462,500 cubic yards of which is identified as sand) was removed and placed on an offshore mud dump. Of this volume, 6,293,900 cubic yards (65% sand) was removed from Sandy Hook Bay.

Problem Identification

A-26. The primary problem encountered in the study area is coastal flooding associated with elevated water levels. Although nuisance flooding can occur during periods of high astronomical tides or minor storms, severe flooding damage results from northeasters and hurricanes. Due to Port Monmouth's location within the bay, the surge elevations during these severe storms can be extreme. The flooding generally results from the surge elevations propagating up the adjacent creeks and spreading over the adjacent marshes. Having flooded the marshes, there is minimal protection against prolonged and extensive inundation of the adjacent low-lying communities. This effect is compounded as storm drainage systems are blocked. By contrast, following the construction of a protective dune in 1966, flooding resulting from overtopping along the Port Monmouth shoreline has been minimal. However, due to ongoing shoreline recession, much of the dune's protective berm has been lost.

Authorization

A-27. This study of the Raritan Bay and Sandy Hook Bay shorefront areas was authorized by a resolution of the Committee of Public Works and Transportation of the U.S. House of Representatives adopted 1 August 1990, which states:

> *Resolved by the Committee on Public Works and Transportation of the United States House of Representatives, that the Board of Engineers for Rivers and Harbors is requested to review the report of the Chief of Engineers on Raritan Bay and Sandy Hook Bay, New Jersey, published as House Document 464, Eighty-seventh Congress, Second Session, and other pertinent reports, to determine the advisability of modifications to the recommendations contained therein to provide erosion control and storm damage prevention for the Raritan Bay and Sandy Hook Bay.*

Prior Federal Studies

A-28. Under the current Authorization a Reconnaissance Study was completed in March 1993. This study concluded that a 50-ft. wide sand berm at elevation +5 ft. NGVD backed by a dune with a 40-ft. wide crest at elevation +15 ft. NGVD with suitable advanced and continuing nourishment, and a levee approximately 10,000 feet in length with a crest width of 10 ft. at elevation +13 ft. NGVD, with suitable interior drainage structures, is an implementable plan.

A-29. Based on the Reconnaissance Study findings, the Army Corps of Engineers and the State of New Jersey entered into an agreement to perform a cost-shared Feasibility Study for the Port Monmouth area.

EXISTING CONDITIONS

A-30. Physical processes are analyzed primarily by review of existing published data. This includes data compiled by NOAA (1987) and the USACE Wave Information Study (Hubertz et al, 1993). Some analyses require interpolation of data available at locations adjacent to Port Monmouth. Wave data makes use of offshore WIS wave studies, as well as shallow water and fetch limited wave models. Bay surge levels utilize existing data and recent model studies.

A-31. Shoreline conditions and changes are based on available historic and recent 1995 survey data. When necessary, due to questionable accuracy of profile comparisons, volumetric change are calculated from historical shoreline changes. Similarly, evaluation of structures is based on historic and recent surveys, aerial photographs, mapping, and site inspections.

Tides and Datums

A-32. Tides at Port Monmouth are semi-diurnal and have a mean range of 4.6 feet and a spring range of 5.6 feet. Tide ranges for points in the study area are summarized in Table A-3. The maximum recorded storm water elevation at Port Monmouth was observed during Hurricane Donna. The water level reported was 9.9 feet NGVD on September 12, 1960. Recent storm water levels at Sandy Hook were 8.7 feet NGVD on December 11, 1992.

Currents

A-33. Currents in the project area are predominately tidal, with contribution from waves and creek discharges. Tidal currents 0.4 miles west of Sandy Hook tip in the channel have a measured average maximum of 2.0 knots on the flood tide and 1.6 knots on the ebb tide. In the Raritan Bay Reach Channel north of Keansburg, the maximum average flood and ebb currents are 0.6 knots and 0.4 knots respectively (NOAA, 1995). Bay currents in the vicinity of Port Monmouth are weaker, except under storm conditions.

A-34. The current in Pews Creek was measured on July 13, 1993 and was used to predict average current velocities in a study conducted by Stevens Institute of Technology (Harrington, 1994). The measurements were used to calibrate a model and calculate neap

TABLE A-3
TIDAL DATUMS NEAR PORT MONMOUTH
(FEET)

ELEVATIONS OF DATUMS	ATLANTIC HIGHLANDS MLW	ATLANTIC HIGHLANDS NGVD	WAY CAKE CREEK MLW	WAY CAKE CREEK NGVD	PORT MONMOUTH MLW	PORT MONMOUTH NGVD
MEAN HIGHER HIGH WATER (MHHW)	5.27	3.49	5.12	3.60	5.20	3.55
MEAN HIGH WATER (MHW)	4.93	3.15	4.79	3.27	4.86	3.21
MEAN TIDE LEVEL (MTL)	2.57	0.79	2.47	0.95	2.52	0.87
MEAN LOW WATER (MLW)	0.20	-1.58	0.16	-1.36	0.18	-1.47
MEAN LOWER LOW WATER (MLLW)	0.00	-1.78	0.00	-1.52	0.00	-1.65
NGVD	1.78	0.00	1.52	0.00	1.65	0.00

Values at Port Monmouth are interpolated from adjacent sites.

NOAA, 1987

and spring tidal current speeds. Under spring tidal conditions, the average peak flood and ebb currents were measured to be approximately 1.7 fps (1.0 knots) and 1.8 fps (1.1 knots) respectively at the critical cross section. Neap tide conditions produced flood and ebb current speeds of 0.5 fps and 0.4 fps. In addition, the investigation estimated the change in tidal elevations between the bay and the marina under neap and spring tidal conditions. Under both conditions, the range of tides were nearly identical at the bay and marina boundaries of the creek. The marina is located approximately 500 feet from the open bay.

Winds

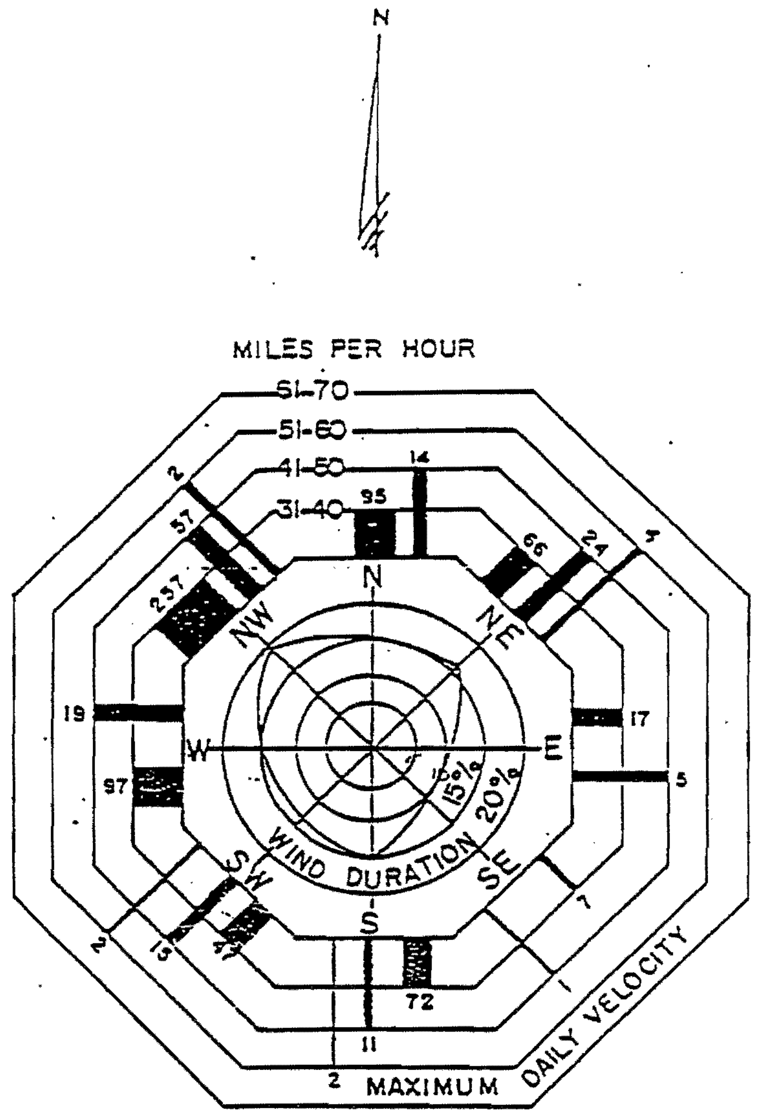
A-35. Wind data is not adequately available within the project limits, but is available for the Sandy Hook area, east of the project area. A wind rose was constructed based on data covering a 10-year period between 1924 and 1934. The wind rose for Sandy Hook is given in Figure A-2. The figure indicates that the prevailing winds are from the northwest, occurring 19 percent of the time. Winds from the north, northeast and south occur more than 15 percent of the time. Winds from the east and southeast occur approximately 10 percent of the time. The northeast accounts for most occurrences greater than 50 mph. The maximum sustained wind reported in the area was 78 mph measured at Long Branch, NJ on June 11, 1953 (Bruno, 1991).

A-36. Wind information is also available in the Wave Information Study data base (Hubertz et al, 1993). This data base provides hindcast winds at 3 hour intervals for the 1956-1975 period. The wind data represents the 10 minute averaged wind at 10 meters above open water. The winds for WIS station 73 will be used in this study, since it best represents the project area. Station 73 is located approximately 25 miles east of the Lower Bay entrance at latitude 40.5, longitude 73.5 in 18 meters (60 feet) of water.

A-37. The maximum wind velocity in WIS data base is 56 mph. Winds of such velocity are primarily oriented from the north. This maximum velocity and direction is similar to the Sandy Hook 1924-34 wind rose. The WIS data shows that speeds exceed 28 mph 6.7% of the time (Table A-4).

Waves

A-38. Detailed inshore wave statistics for the study area do not seem to be available. Open ocean wave statistics have been developed by the Coastal Engineering Research Center for the Atlantic Coast near the study area. However, no similar wave statistics have been developed for restricted waters of Raritan or Sandy Hook Bays. Insight into the region's wave climate can be gained by nearby Wave Information Study (WIS) statistics and some basic wave climate data developed in earlier Corps of Engineers (and other) reports.



THE WIDTHS OF THE BARS SHOWN ARE ONLY APPROXIMATELY PROPORTIONAL TO THE NUMBER OF OCCURENCES. THE NUMBERS AT THE ENDS OF THE BARS INDICATE THE ACTUAL NUMBER OF OCCURENCES.

FIGURE A-2

WIND DATA
WIND ROSE FOR
SANDY HOOK, NEW JERSEY
1924 – 1934

TABLE A-4
WIND STATISTICS
WIS STATION 73

WIND DIRECTION	EVENTS NUMBER	EVENTS PERCENT
337.5 -22.5	6,108	10.5%
22.5 -67.5	3,650	6.2%
67.5 -112.5	3,217	5.5%
112.5 -157.5	4,788	8.2%
157.5 -202.5	7,435	12.7%
202.5 -247.5	9,428	16.1%
247.5 -292.5	12,139	20.8%
292.5 -337.5	11,675	20.0%
TOTAL	58,440	100.0%

WIND SPEED (m/s)	WIND SPEED (MPH)	TIME DURATION OF WINDS EXCEEDING SPEED
20.0	44.7	0.2%
17.5	39.1	1.0%
15.0	33.6	1.9%
12.5	28.0	6.7%
10.0	22.4	13.5%
7.5	16.8	31.5%
5.0	11.2	56.0%
2.5	5.6	96.0%
0	0.0	100.0%

USACE, 1993

A-39. The WIS Phase III wave statistics for station 73 provides data in 18 meters (60 ft) of water approximately 25 miles east of the bay entrance between Sandy Hook and Rockaway Point. The statistics show a maximum wave height of 5.7 meters (18.7 ft) approach the bay entrance from the northeast quadrant. Hurricane waves are not included in this data base.

A-40. Shallow water effects limit the impact of such waves on Port Monmouth. The highest waves in the study area are generated by winds from the north and northeast quadrant, which is the direction of longest and deepest fetch for wave generation. Waters near the study area may also be affected by ocean swells from this direction, which enter the bays between Rockaway Point and Sandy Hook. Observations by experienced seafaring personnel report that swells as high as 15 feet occur between Sandy Hook and Rockaway Point. These swells are reduced to a height not in excess of 6 feet in the area of the Navy piers near Leonardo. Storm wave heights of about 5 feet have been reported at Atlantic Highlands (USACE, 1977).

A-41. A storm wave study was conducted for Old Bridge Township, New Jersey, which is located in the western reach of Raritan Bay (Bruno, 1991). The 100 year return sea and swell wave height were 8.0 ft and 7.9 ft. respectively. The site has a longer northeast fetch than Port Monmouth, and should have a larger wave height. In addition, the maximum wave height in the vicinity of Cheesequake Creek was estimated at 7 ft, with an associated period of 6.5 seconds. This represents a wave height in 15 feet of water.

A-42. To calculate shallow water effects on waves at Port Monmouth, six northeast quadrant wave sets from the WIS station 73 data base were selected for analysis (Table A-5). These waves are the largest for the direction indicated. By using linear refraction and the diffraction routine described in CETN-I-18 (March 1982), the wave height at Port Monmouth can be estimated. The largest offshore wave in this set is 18.7 feet (from 86 degrees), but refraction, shoaling and diffraction, reduce its height to 2.1 feet. Furthermore, waves from the east are greatly reduced in height because of the diffraction effects of the Sandy Hook peninsula.

A-43. At the other end of the quadrant, a wave from 29 degrees would be minimally reduced by diffraction and refraction, from 11.2 feet to 10.5 feet. However, a closer look at the geometry of the bay entrance shows a direction of 35 degrees from the study area to the opening between Sandy Hook and Rockaway Point, eliminating the possibility of waves incident from 29 degrees. The wave from 52 degrees is also minimally affected by diffraction on its approach into the bays, but for Port Monmouth the bay entrance is located further north, so such a wave would be limited by fetch.

A-44. This analysis suggests that at most a very narrow band of waves can approach Port Monmouth from the open ocean. Those offshore waves that can reach the study area are

TABLE A-5

WAVE SHOALING AND DIFFRACTION

REP. WAVE NO.	WAVE HEIGHT (FEET) OFFSHORE d=60'	SHOAL d=20'	PERIOD (SEC.)	DIR. (DEG) (fm north)	DIFFRACTED WAVE HT. (FEET)
1	11.2	10.8	6	29	10.5
2	11.2	10.8	6	58	7.5
3	10.2	10.5	7	76	3.7
4	10.5	11.8	9	79	3.3
5	18.7	11.0	12	86	2.1
6	15.1	13.8	10	94	1.7

OFFSHORE DATA FROM WIS (USACE, 1993)
SHOALING MODEL: CETN-I-18, MAR 82

severely affected by shoaling, refraction, and diffraction. Thus the wave climate at the study area is dominated by fetch limited wind-driven waves.

A-45. Fetch-Limited Wave Statistics. Having found that the wave climate at Port Monmouth is primarily wind driven and fetch limited, the wind data from WIS station 73 (USACE, 1993) is used to develop wave climate data for Port Monmouth. This analysis made use of four routines from the ACES (CERC, 1992) family of programs to develop the Port Monmouth wave statistics. These routines include: Windspeed Adjustment and Wave Growth; Wave Transformation: Linear Wave Theory; and Extreme Significant Wave Height Analysis.

A-46. The wind data was sorted by into blocks using the recommended sorting values of 2.5 m/s in velocity and 15 degrees in direction. For each block the average and maximum values of speed and direction were determined (Table A-6). This data set includes the highest on-shore wind speeds for the 20 year period from 1956-1975, excluding hurricanes. The data set was augmented with data from the 10 year Sandy Hook wave Rose to form the 30 year data set.

A-47. The average fetch distance and depth were determined at 10 degree intervals from a point at the center of the Port Monmouth Shoreline. NOS Chart No. 12327, New York Harbor (Aug 1995) was used for these measurements. The fetch lengths and depths were averaged over 20 degree arches. Effective depths were determined for each direction and major storm return interval by adding the storm surge level at chart datum to the chart depth.

A-48. The ACES Wave Growth routine was then used to calculate the wave direction, height and period for each wind block and selected storm water levels. A wave block from the northeast and northwest quadrant was selected for each storm interval. The results were adjusted to a common depth (18 feet) using the Linear Wave Transformation Routine from ACES. The final wave data set consisted of 20 waves which occurred during a 30 year interval, each with corresponding heights of the significant wave in 18 feet of water, the deep water wave and the breaking wave.

A-49. Wave frequency statistics were calculated using the ACES' Extreme Significant Wave Height routine. This routine provided the 2 year through 100 year return interval. The 1 and 200 year values were extrapolated using variables provided from the routine's output. The Weibull Distribution (k=2.00) had the best fit for all values and its results are displayed in Figure A-3 and Table A-7.

A-50. These results are in agreement with other observation from the project area. The 100 year return value (6.4 feet) compares well with the maximum values reported at Leonardo Naval Pier (6.0 feet) and Cheesequake Creek (7.0 feet).

TABLE A-6
SUMMARY OF HIGHEST WIS WIND DATA

DATE	RETURN IN 30 YRS	AVERAGE WIND VEL. (MPH)	MAXIMUM WIND VEL. (MPH)	AVERAGE DIR. (DEGREES)	DURATION (HOURS)
FEB 69	1	51.2	55.9	336.1	27
MAR 62	2	49.5	55.9	34.4	27
DEC 62	3	46.5	55.9	324.8	78
JAN 59	4	46.5	55.9	299.6	66
1924-34*	5		55.0	45.0	4
1924-34*	6		55.0	315.0	2
MAR 73	7	46.7	53.7	283.9	27
JAN 64	8	46.7	53.7	53.1	24
DEC 64	9	45.9	51.5	350.8	18
MAR 60	10	45.5	51.5	338.3	27
DEC 75	11	48.7	51.5	26.3	24
FEB 68	12	46.2	49.2	5.0	18
MAR 68	13	45.6	49.2	331.0	15
DEC 69	14	45.2	49.2	315.4	42
JAN 64	15	45.2	49.2	294.0	15
JAN 71	16	45.0	49.2	295.6	24
DEC 74	17	44.7	49.2	87.1	21
DEC 64	18	44.7	49.2	318.8	12
APR 67	19	44.5	49.2	359.4	24
JAN 69	20	44.1	49.2	296.5	30
DEC 75	21	43.8	49.2	284.0	15
JAN 63	22	43.1	49.2	304.3	21
DEC 56	23	42.8	47.0	308.6	21
APR 58	24	42.1	47.0	348.3	18
DEC 74	25	40.5	44.7	323.0	30
MAR 62	26	40.6	42.5	75.0	18
JAN 64	27	38.5	42.5	338.0	15
FEB 63	28	36.3	38.0	296.3	12

* FROM SANDY HOOK WIND ROSE 1924-1934.
USACE, 1993 (RECONNAISSANCE REPORT)

FOR WIS DATA, USED AVERAGE WIND FOR FULL DURATION
FOR SANDY HOOK, USED 1 HOUR WIND WITH 10 HOUR FINAL DURATION
USACE, 1993 (WIS REPORT 30)

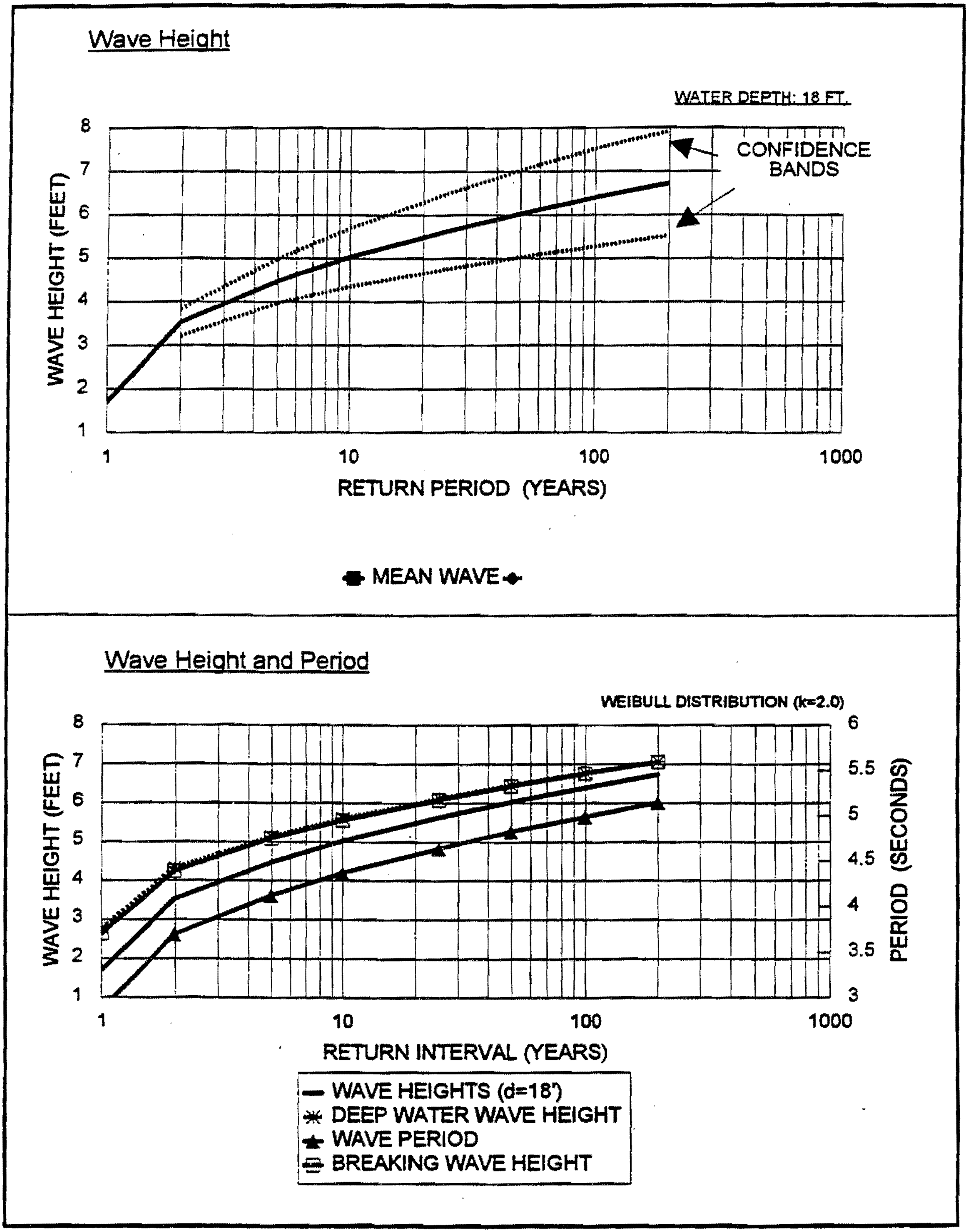


FIGURE A-3

STORM WAVE CHARACTERISTICS

TABLE A-7
STORM WAVE CHARACTERISTICS

RETURN PERIOD (YEARS)	HEIGHT (d=18') (FEET)	DEEP WATER HEIGHT (FEET)	BREAKING HEIGHT (FEET)	PERIOD (SEC.)
1	1.7	2.8	2.6	2.9
2	3.5	4.3	4.2	3.7
5	4.5	5.1	5.1	4.1
10	5.0	5.6	5.6	4.4
25	5.6	6.1	6.1	4.6
50	6.0	6.5	6.4	4.8
100	6.4	6.8	6.8	5.0
200	6.7	7.1	7.1	5.1

BASED ON FETCH LIMITED WAVE THEORY.

Bay Storm Stage

A-51. Flooding in the study area is typically caused by the combination of waves with storm-induced water levels and astronomical tide. The storm-induced water level is produced by several effects. Storm winds develop a shear force on the water surface causing an elevation of the water surface along the coast. Decreasing barometric pressure raises the water surface drawing water from adjacent higher pressure areas. Finally, storm waves raise the water level along the shore as water piles up along the coast, an effect called wave setup.

A-52. Two distinct classes of storms that affect the study area are northeasters (extratropical) and hurricanes (tropical). Northeasters, named after the predominant direction of winds, are large-scale low pressure disturbances which usually occur from November through March. The severity of a northeaster is not as great as that of a hurricane. Although wind gusts can reach hurricane strength in a very severe northeaster, sustained wind speeds are rarely greater than 50 knots. The flood damage caused by the typical northeaster is often more a function of its duration rather than its intensity, as the longer storms have more opportunity to destroy both natural and engineered flood protection features. Also, as northeasters typically last two to three days, it is possible for the storm to act during several periods of high astronomical tide. Hurricanes are a rarer occurrence in the study area than northeasters. By the time hurricanes approach the latitudes of the north New Jersey coast, they are usually in a state of energy loss and are beginning to decay into the category of tropical storm. The average period between hurricanes is about 5.7 years, or 0.175 hurricanes per year. Despite their infrequency and short duration, hurricanes have the potential to be devastating in the study area because of their high wind speed and high surge.

A-53. Stage-frequency curves relate the elevation of flood waters to the probability of recurring floods of equal or greater severity. A storm surge curve was developed by the Corps of Engineers Coastal Engineering Research Center in January 1996. The combined hurricane and northeaster curve was used for design purposes in this investigation. The combined storm stage in the study area is approximately 8.4 ft and 12.2 ft NGVD for a 10-year and 100-year return period event respectively (Figure A-4 and Table A-8).

A-54. Table A-8 also includes the predicted impact of the wave setup component on the bay storm stage. Wave setup values are calculated by applying the representative wave climate values to the predictions contained in the *Shore Protection Manual* (USACE, 1984) for monochromatic waves. Between the 2- and 250-year storm events, the predicted contribution of wave setup ranges from 0.6 to 1.0 feet.

A-55. Interior Flooding. The major creeks and waterways at Port Monmouth include Pews Creek and Compton Creek, which convey stormwater runoff from the interior sections of the study area into Sandy Hook Bay. During normal high tides and tidal surge

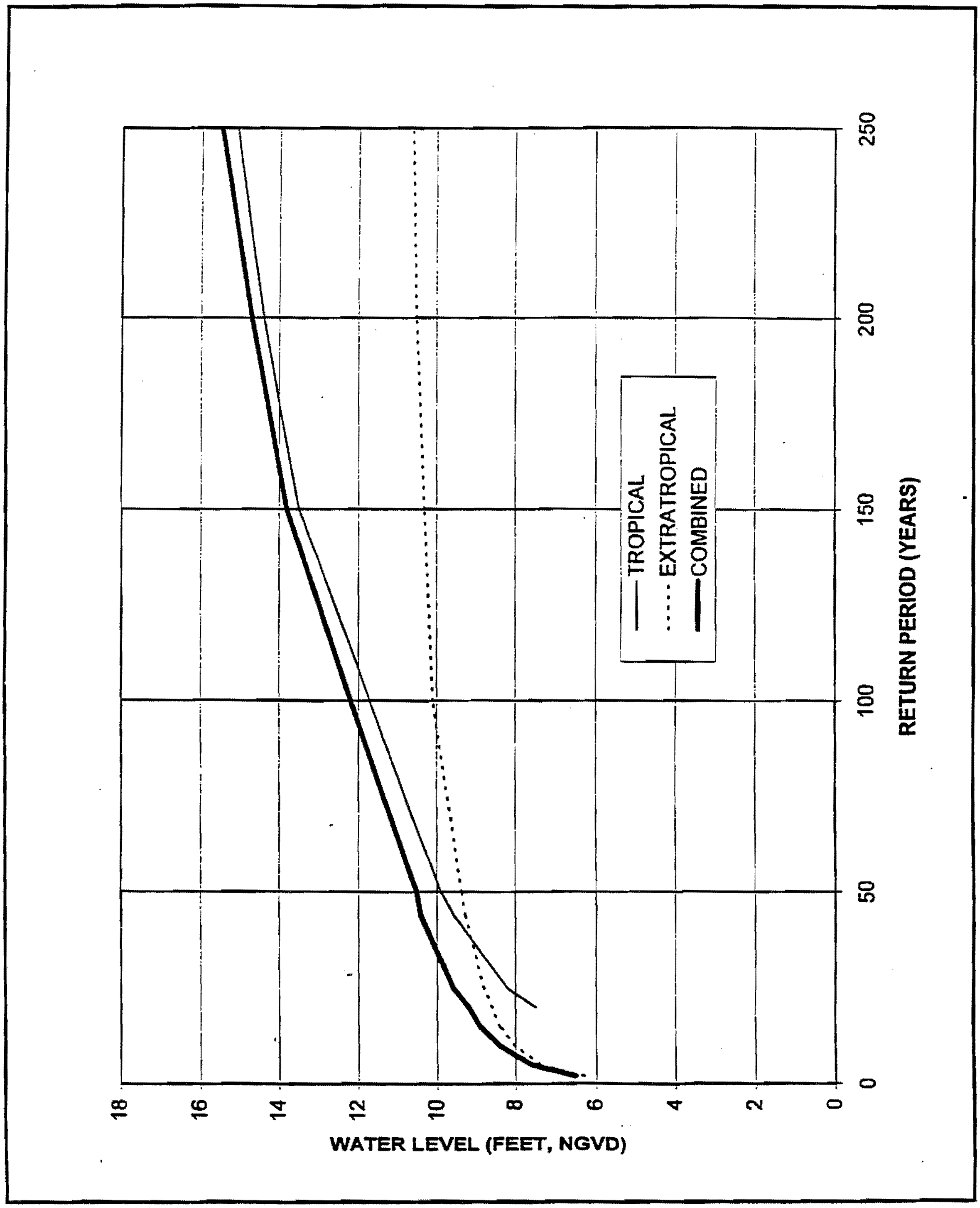


FIGURE A-4

COMBINED STAGE FREQUENCY CURVE

(USACE, 1996)

TABLE A-8
STAGE FREQUENCY (FEET, NGVD)
PORT MONMOUTH, NJ
SOUTH HARBOR NODE (LAT 40.48N, LONG 74.19W)

RETURN PERIOD (YEARS)	STORM TYPE					
	TROPICAL	EXTRA-TROPICAL	COMBINED			
			WATER LEVEL	CONFIDENCE LIMIT		WITH WAVE SETUP
				LOWER	UPPER	
	(1)	(1)	(1)	(1)	(1)	(3)
2	(2)	6.3	**6.5**	6.5	6.5	**7.1**
5	(2)	7.4	**7.6**	7.5	7.8	**8.4**
10	(2)	8.0	**8.4**	8.2	8.6	**9.3**
15	(2)	8.4	**8.9**	8.7	9.1	**9.8**
20	7.5	8.6	**9.2**	8.0	10.5	**10.1**
25	8.2	8.8	**9.6**	8.3	10.9	**10.5**
44	9.6	9.3	**10.4**	8.9	11.8	**11.3**
50	9.9	9.4	**10.5**	8.9	12.1	**11.5**
100	11.7	10.1	**12.2**	9.8	14.6	**13.2**
150	13.5	10.3	**13.8**	10.5	17.1	**14.8**
200	14.4	10.5	**14.7**	10.7	18.7	**15.7**
250	15.1	10.6	**15.5**	10.8	20.2	**16.5**

NOTES:

(1): MTL VALUES CALCULATED BY USACE (1996), CORRESPONDING VALUES WITH RESPECT TO NGVD CALCULATED USING CPE-CALCULATED DATUMS

(2): USACE (1996) PREDICTS NEGLIGIBLE SURGE DUE TO HURRICANES

(3): WAVE SETUP VALUES CALCULATED BY SHORE PROTECTION MANUAL (USACE, 1984) events, the lower interior drainage systems become submerged and inundated, reducing the flow carrying capacity of the drainage systems. Thus even in normal high tides, some of the low-lying sections of Port Monmouth are frequently flooded. During storm events, a combination of peak interior runoff flows coinciding with high tides and/or storm surge contributes to the flooding of the interior low lying sections throughout these areas.

Sea Level Rise

A-56. Sea level rise is a factor contributing to coastal erosion. Long term sea level rise due to the melting of polar ice caps has resulted in a general increase in sea level. Based on NOAA tide gauge readings between 1933 and 1986 at Sandy Hook, sea level has been increasing by an average of approximately 0.014 ft. per year. Tidal flooding is expected to increase in severity in direct relation to this 0.7 foot increase over a 50-year period.

Storms

A-57. Hurricane of 14 September 1944. This hurricane caused losses estimated at over $2,500,000 (1944 dollars) in the bayshore area. The storm reached Cape Hatteras on the morning of 14 September where a central barometric pressure of 27.88 inches and a wind speed of about 108 miles per hour were reported by the U.S. Weather Bureau. Toward evening the storm center moved close to the New Jersey coast about one hour after predicted high tide. Peak tide height reached 8.4 ft NGVD in the area from Highlands to Keyport and 12.0 inches of rain were recorded in New Brunswick.

A-58. Boardwalks and several homes in Port Monmouth were destroyed by waves. Tidal stages which exceeded bulkhead heights resulted in washed-out roads, walks and pavements along with the flooding of homes and hotels in Middletown Township.

A-59. In Union Beach, bay waters reached 500 to 1,000 feet inshore. Artificial dunes were damaged and almost the entire beach was washed away. Homes and stores were flooded and a section of streets and walks was washed out by wave action.

A-60. Flood waters reached points 1,500 to 2,000 feet inland in Keansburg. Waves destroyed the steamboat pier and the entire boardwalk with its amusement buildings and business properties. The waterfront street was washed out, damaging sewer and water lines, jetties and bulkheads. Nearly all frame buildings collapsed when struck by heavy debris. Twelve summer cottages were demolished. Many other homes were flooded and furnishings were damaged.

A-61. Extratropical Storm of 25 November 1950. This storm, which produced tides of 9.1 feet at Keyport, caused over $2,000,000 (1950 dollars) of damage in the Bayshore area. According to newspaper accounts, there were two deaths, one in Union Beach and another in Keansburg. Rainfall totals were approximately 2.5 inches. The storm center formed over eastern North Carolina and moved northward toward the study area. At New York City the winds, which were less severe than during the 1944 hurricane, attained an average hourly velocity of 47 miles per hour, peaking at 59 miles per hour from the south with gust velocities as high as 72 miles per hour. The accompanying tides in the New York Harbor area were about one to two feet above the previous maximums recorded during the 1944 hurricane.

A-62. From the standpoint of the effect of the winds on the tide, the following differences between the storm of 25 November 1950 and other hurricanes are of significance. Unlike the 1944 hurricane, the low pressure storm center of the November 1950 storm always lay well inland and passed about 80 miles west of the study area. During this storm strong easterly winds were blowing for 17 hours, causing a build-up of tide in Raritan Bay. In contrast, the easterly winds during the 1944 hurricane lasted only about 6 hours and were later counteracted by strong winds from the north and west, which tended to suppress the high tides in the bay.

A-63. Keansburg, where floodwaters extended almost a mile inland, was placed under martial law. Residents were evacuated from their homes with the aid of troops and equipment from Fort Monmouth. A section of the eastern end of the boardwalk for a distance of about 150 feet was washed away, and most of the beach concessions and amusement stands were destroyed or severely damaged. A number of homes and business establishments near the beach front were inundated and damaged.

A-64. About 200 people were evacuated in East Keansburg. At Leonardo, Atlantic Highlands and Highlands, boats and piers were damaged severely by tide and wave action in Sandy Hook Bay. The yacht basin in Atlantic Highlands was wrecked. The entire downtown section of Highlands was flooded resulting in the evacuation of the residents and heavy damage to many commercial establishments. Beach erosion was extensive, and many streets in the area were damaged.

A-65. In the Morgan Beach and Laurence Harbor area over 250 families were evacuated by rowboat. Approximately 50 homes were destroyed, with about 30 more badly damaged, and scores of others suffered minor damage to structures and furnishings. Residential damages were estimated at $500,000 (1950 price level). At Laurence Harbor, a new boardwalk was wrecked, beach concessions were destroyed, and a new casino was heavily damaged.

A-66. Damages to boats and boat facilities in the Cliffwood Beach-Keyport area were estimated at $150,000 (1950 price level). High tides in Matawan Creek inundated several roadways between Matawan and Cliffwood. Portions of State Highway No. 35 were also flooded and traffic was interrupted.

A-67. At Union Beach, one house was completely demolished and many others were damaged. About 55 families were evacuated. A 1200-foot section of roadway and curbing on Front Street, two bridges, the water plant, and numerous bulkheads experienced damage.

A-68. Extratropical Storm of 6-7 November 1953. This storm caused damage estimated at $1,630,000 (1953 dollars) with peak tides of 8.9 feet. The storm of 1953 originated in the Gulf coastal region and traveled easterly to a position offshore of the Georgia coast where it was deflected to a more northerly course. A high pressure area, centered over the Upper Great Lakes region, brought cold air into the southeastern portion of the country and tended to intensify the storm. The storm center, moving in a northerly direction, moved inland in the vicinity of New York City. Winds along the New Jersey coast exceeded 60 miles per hour. At Long Branch (Atlantic coast) the strongest wind was measured as 78 miles per hour from the east. Total rainfall was estimated at 1.25 inches.

A-69. At Keansburg a major portion of the borough was inundated, resulting in heavy damage to both commercial and residential properties. About 100 commercial establishments were flooded to depths ranging to more than 4 feet. Hundreds of residences were flooded above the first floor and about 25 houses were destroyed or severely damaged structurally. The beach was severely eroded and the bulkheading was destroyed at some locations and badly damaged at others. At East Keansburg residential damage was extensive. About 300 persons were evacuated from flooded homes.

A-70. Within East Keansburg and Port Monmouth damage to bulkheads and jetties was severe. The fish factory at Port Monmouth suspended operations for seven days due to damaged buildings, supplies and other property. Fifteen homes at Port Monmouth were damaged and seven were destroyed by water and wind.

A-71. Between Atlantic Highlands and Leonardo about 4,000 feet of bulkheading was destroyed. About 15 residences were inundated from 2 to 4 feet above the first floor and were structurally damaged by undercutting. Undermining of utility poles together with wind action cut off electric service over a wide area.

A-72. As a result of severe damages caused by this storm, the State Legislature of New Jersey organized the "Legislative Commission to Study Sea Storm Damage." The Commission found that direct damage to public property in the Bayshore area was $374,000 (1953 dollars).

A-73. Hurricane of 12 September 1960 (Donna). Tides produced by Hurricane Donna reached 8.9 feet at Morgan with a reported rainfall of 4.5 inches. Tidal damages were estimated at about $6,000,000 (1960 dollars).

A-74. On September 12 the "eye" of the storm, which passed near the study area while moving northerly at the time of predicted high tide, became elongated and extended from New York City to Montauk Point. U.S. Air Force radar operators at Montauk Point indicated that Hurricane Donna had separated into three "eyes" upon reaching this area. At Long Branch the highest gust recorded was 79 m.p.h. from the northeast and the minimum barometric pressure was 28.55 inches.

A-75. The Union Beach Borough was flooded by waters from Raritan Bay and Chingarora Flat, and East Creeks. Most of the stores along the shore suffered heavy damage. Two houses east of Flat Creek were totally destroyed and two others partially destroyed. About 100 people were evacuated when their homes were flooded. The embankment of the Central Railroad of New Jersey was washed out at several locations, resulting in a disruption of service in the study area.

A-76. At Keansburg, where flood waters came inland a mile from the shore, the greatest damages were sustained. Local officials estimated the damage to be in excess of $2,000,000 with damage to municipal property over $250,000 (1960 dollars). The New York District verified this approximation with an independent estimate of $2.7 million (1960 dollars). Water came over the beach and up Way Cake Creek as far as the railroad. State Police aid was required to prevent looting in the Borough.

A-77. The western portion of Middletown experienced severe damage. The beaches in East Keansburg were overtopped and many homes were damaged. Near Pews Creek, two homes were totally destroyed and the bridge was washed out. Over 400 persons were evacuated from homes in East Keansburg and Port Monmouth. In Port Monmouth and Belford, where a number of homes were severely damaged, looting prevention became a major police problem. In Leonardo, the jetties at the State marina were damaged and the homes along the shore suffered minor damage due to flooding.

A-78. At Atlantic Highlands and Highlands, boats and piers were severely damaged by the storm. In Highlands, water was 4 to 5 feet deep on the main street and a great number of stores and homes were flooded. Newspapers carried reports of raw sewage floating in the borough streets. A bulkhead recently constructed by the State was flanked by the tide and the street behind the bulkhead was washed out.

A-79. Northeaster of 6-8 March 1962. The storm of 6-8 March produced unusually high wind driven tides and very high waves which battered the shore for three successive days. Public and private damages consisted mainly of beach and dune erosion and damages to the bulkhead, seawalls, groins, boardwalks, buildings and roads along the New Jersey coast. Peak tides at Perth Amboy were 8.1 feet. Damage estimates for the entire Raritan Bay and Sandy Hook area were estimated to be $6,400,000 (1962 dollars).

A-80. On Tuesday, 6 March, the center of the large storm stopped its northward movement and became nearly stationary off Delaware, Maryland and the Virginia Capes. During this period, the storm seemed to develop at least two centers. The western center apparently developed a small westward loop over Chesapeake Bay, and the eastern center developed a loop southeastward into the Atlantic off the outer banks of North Carolina. This complex pattern of multiple centers persisted as the storm moved eastward out into the Atlantic on 7-9 March.

A-81. At Port Monmouth, the county bridge over Pews Creek was damaged and made unsafe for traffic. The beach was eroded and cottages on the beach were displaced and damaged by wave action. Roads were eroded and blocked by sand and several homes and schools were evacuated. Minor damage was reported at Leonardo, Atlantic Highlands and Belford. Considerable amounts of debris were deposited around residences and marinas. At Highlands, the business area was completely flooded and 60 percent of the residential area was flooded by five successive tides.

A-82. At Keansburg, bathing beaches west of Point Comfort were seriously eroded. Most of the decking was torn from the steamboat pier. Pavements, boardwalks, fences, curbs and roads in the area were damaged by wave action and a great part of the borough was subjected to tidal flooding. There was extensive cellar flooding at Union Beach. In the Union Beach-Keansburg area about 200 persons were evacuated.

A-83. At East Keansburg, beaches and dunes were eroded and reduced in elevation and width. Sand fences were destroyed, roads were eroded and streets near beaches were covered with sand. There was damage to buildings by both wave attack and flooding.

A-84. Northeaster of 12 March 1984. The storm of 12 March produced a mixture of snow, sleet, hail and hurricane force winds. A peak stage of +7.0 ft. MSL was reported at Keansburg.

A-85. Erosion of the beaches included dune escarpment in Port Monmouth with scarps measuring five to nine feet near Main Street. In Leonardo, erosion of the beaches and dune escarpment accompanied street and property flooding near Wagner Creek. Retaining walls were undermined by high water removing sand. Extensive beach erosion occurred east of the harbor to the harbor light.

A-86. Dune escarpment was measured at about two feet at Ideal Beach in East Keansburg. In East Keansburg, a 15-foot high berm was breached during the morning but shifting winds lessened the evening tide effects and the berm held. The parking lot adjacent to the berm was undermined, with sections destroyed. The dunes of the Federal Project were eroded causing four to five-foot scarps, however, the dunes were not overwashed. Beach erosion was severe and dune erosion was greatest at the west end of the Federal Project near Way Cake Creek. Flooding on low-lying streets was reported.

A-87. Three of six piers and a launching ramp were destroyed at the Atlantic Highlands municipal harbor. Flooding occurred near the banks of Many Mind Creek, where erosion and escarpment were pronounced. The east bank of the creek showed six to seven feet of escarpment. On the west bank portions of residential backyards were lost as the bank eroded.

A-88. Northeaster Storm of 11-12 December, 1992. The storm caused extensive flooding along the coastal communities of Raritan Bay and Sandy Hook Bay. Extensive wave and erosion damage was also reported. The high tide recorded in the bay was +9.8 ft. NGVD at Luppatatong Creek in Keyport.

A-89. As a result of this storm the entire study area was included in a disaster area declaration. Residents, businesses and public organizations were therefore eligible for aid under a variety of Federal disaster assistance programs. Major Federal programs include:

- Individual Financial Assistance (IFA) to provide emergency aid for temporary housing. Preliminary data indicates that within the study area 71 applicants received $127,250 in assistance.
- Individual Financial Grants (IFG) to provide assistance to eligible applicants in repairing uninsured damages. Preliminary data for the study area indicates 25 grants totaling $55,310 were issued.
- Small Business Administration (SBA) low interest loans to provide individual residents or businesses assistance in restoring properties. Within the study area 103 SBA applications were provided.
- Public Assistance provides Federal reimbursement of 75% for eligible public damage expenses. Review of the mid-February estimate of $1.2 million in public damages indicates that this preliminary figure does not include several major items such as damage to the public bulkhead in Union Beach. Significant upward revision of this estimate is considered likely.

A-90. The Port Monmouth side of Pews Creek had previously experienced one section of failed bulkhead before the storm. The storm caused that failure to become more severe, and caused another section to fail. The stockpile of dredge spoil from Pews Creek was severely eroded. The newly constructed fishing pier located approximately 1000 feet north of the Spy House Museum sustained approximately 20% damage. The dunes fronting the Spy House were completely destroyed. Dunes to the east and west of the museum were severely eroded with a remaining vertical scarp of approximately 15 feet. Severe flooding occurred throughout the community.

A-91. The Belford Seafood Cooperative experienced water levels of approximately one foot above the ground elevation in the buildings. Minimal damage was sustained.

A-92. The berm at Point Comfort in Keansburg experienced some scarping. Ideal Beach sustained severe dune erosion, leaving a vertical scarp of approximately 15 feet. The Keansburg side of Pews Creek experienced some failed sections of bulkhead near the end of Port Monmouth Road, and the seaward end of the bulkhead appears to be damaged. According to Mr. Bernard Moore of the NJDEP&E, the tide gates in the Way Cake Creek and Pews Creek and the pump stations functioned effectively to dramatically reduce interior flooding. Some sand was deposited in the channel behind the tide gate which may interfere with future functioning of the gate.

A-93. In Leonardo, a debris line was visible on the chain link fence near the mouth on the west side of the marina, evidence of a water line of more than 3 feet above the ground elevation. Low-lying homes adjacent to the marina suffered significant flood damage. The beach on the east side of the marina experienced severe erosion damage. The western dunes were completely destroyed. The remaining dunes were severely eroded leaving a vertical scarp of approximately 15 feet. A vertical loss of berm material, fronting the dunes, of approximately 2-3 feet was demonstrated by an increase in the exposed height of piles fronting the parking area (pre-storm exposed height of approx. 1 foot, post-storm exposed height of approx. 3-4 feet). The dunes near the intersection of Beach Ave. and N. Leonard Ave. were completely destroyed, and the underlying bulkhead exposed. The road at the intersection was undermined and washed out. The dunes near Conover Beacon were severely scarped. Severe erosion and scarping occurred, evidenced by the undermined and washed out road at the end of Brevant Street. To the east of Conover Beacon, numerous bulkheads and seawalls were destroyed. Upland properties were severely eroded, exposing structures to direct storm impacts.

A-94. At the recently constructed bulkhead along Front St. in Union Beach from Florence Ave. to Cedar Street, the majority of planks were missing and/or broken. The Bayshore Club restaurant on the bayside of Front St. sustained severe structural damage as the entire rear portion of the building collapsed. The water level inside the building on the wall farthest away from the ocean showed an elevation of approximately 8 inches above the ground floor elevation. The Sand Bar Inn restaurant on the bayside of Front St. sustained some damage to a newly constructed timber deck at the rear of the restaurant. Several homes behind this bulkhead between Pine St. and Cedar St. sustained structural damage. Some sections of the asphalt walkway along the bulkhead behind these houses were destroyed. The chain link fence along the walkway was completely destroyed. Along Front St. to the west of Florence Avenue, numerous seawalls and bulkheads were damaged or destroyed. Erosion behind these failed structures left upland buildings unprotected and resulted in some undermining and damage. Inland areas were subject to widespread tidal flooding with water above the main floor of many structures.

A-95. The two marinas in Keyport at the mouth of Matawan Creek sustained extensive damage. Some boats were displaced as far east as the Garden State Parkway; several boats were grounded near Route 35; several boats were sunk; and several docks were destroyed.

The Up The Creek restaurant on the creek side of West Front St. sustained severe structural damage caused by inundation. The marina at the mouth of the Luppatatong Creek sustained less extensive damages. Some of the boats were grounded or sunk. The bridge crossing the creek on West Front St. sustained some structural damage and was closed for a short time by Monmouth County officials.

A-96. The east bank of Whale Creek in Cliffwood Beach suffered sever scarping. Some of the dunes on the beach along Lakeshore Drive were completely destroyed, and the remaining dunes were severely scarped. A debris line was visible on the chain link fence surrounding the tennis courts on the creek side of Lakeshore Drive, evidence of a water level of approximately 4 feet above the ground elevation. The north end of the seawall experienced a few misplaced capstone units and scattered toe stone.

A-97. Two marinas are located on the southeast side of Cheesequake Creek in Laurence Harbor. The northern marina, Auback's Marina, experienced inundation of approximately 3-4 feet inside the building, a broken dock, and several sunken boats. The southern marina, Viking Marina, sustained damages of several grounded boats, several sunken and capsized boats. Gerrity's bar located on the oceanside of Route 35 experienced approximately 3-4 feet of interior inundation. The beach at Laurence Harbor at the southeast end of Shoreline Circle suffered a scarped and eroded berm. The low berm crest is presently located within 20 feet of a tot lot. An old bulkhead previously covered by sand was exposed by the storm. The scarping of the berm continued to the end of Bayview Avenue.

Geology

A-98. A comprehensive geologic description is provided in the Raritan Bay and Sandy Hook Bay Reconnaissance Study of 1993 (USACE, 1993). The following observations were reported:

> "The area lies within the Coastal Plain Province which forms the eastern margin of the State of New Jersey south of Perth Amboy. The plain has a gentle slope to the southeast, generally not exceeding 5 or 6 feet to the mile. The strike of outcrop of the various formations is in a northeast-southwest direction. Throughout the greater portion of the plain, the relief is insignificant and streams flow in open valleys that lie at only slightly lower levels than the broad flat surrounding area.
>
> The underlying bedrock is a crystalline rock with local infolded or infaulted Triassic sediments. The soils overlying bedrock are of considerable thickness exceeding several hundred feet, and are of the Upper Cretaceous and Tertiary Period. The oldest and therefore the deepest formation which rests unconformably on the bedrock is the Raritan (Magothy) formation.

It consists of dark lignitic sand and clay containing some glauconite at the top overlying light colored sands and clays. The marine origin of this formation was verified by numerous shells and fossils found in outcrops of this formation during explorations in the vicinity of Cheesequake Creek. A period of erosion followed the deposition of this formation.

The Mechantville and Woodbury clay formations overlie the Raritan formation discomformably. Both formations are a black, glauconitic, micaceous clay, the former being slightly more plastic and firmer than the latter. To the southeast of Waycake Creek, the upper formation, the Englishtown sand, outcrops at the surface along Creek Road, and extends southeastward to Highlands under the recent swamp deposits at Pews Creek. It reaches its maximum thickness at the Highlands where some of the beds have been cemented by iron oxide. This material overlies the Woodbury clay formation and it represents a period of emergence. The Englishtown sand consists of a white and yellow quartz sand, slightly micaceous.

With the final uplift of the land and withdrawal of the Cretaceous sea, streams established themselves across the emerging sea bottom. This ushered in the Cenozoic Era. Periods of submergence and emergence were the dominating geological force, but with the exception of a very shallow deposit of sand referred to as the Cape May formation, no other soil material from this era is found in the project area. The Cape May formation is an interglacial formation deposited by streams and overland deposition at the close of the last glacial period. The sea again invaded the area and created valleys which have been filling with recent swamp material and sediment.

Considering the age of the Cretaceous materials, estimated by geologists to be some 120 to 150 million years old and all the intervals of submergence and deposition, and emergence and erosion, one would expect these soils to be very firm on the basis that they have been subjected to relatively high prestresses. However, the clay materials were found to be nominally consolidated and very soft."

Littoral Materials

A-99. The most comprehensive study of material underlying the entire Lower Bay region is provided in "The Volume of Sand and Gravel Resources in the Lower Bay of New York Harbor" (Bokuniewicz and Fray, 1979). The results indicate that a very significant volume of sand exists in the bays, estimated to be over 3 billion cubic yards. This value, however, is highly speculative and is not limited to surficial material which would be convenient for excavation. Extending offshore about 1.5 miles from the Port Monmouth area lies a sandy area identified as Keansburg Sands. Further offshore and east, mud is identified as dominating the bed and extending as deep as -150 feet. Other areas of the bays feature mud or sand, of varied qualities. No hardbottom is reported.

A-100. The November, 1995, survey of the Port Monmouth shoreline (Rogers, 1995) included collecting and analyzing 42 surface sediment samples throughout the area, with the following technique:

> "Samples were mechanically separated to obtain a sample of approximately 400-500 grams. Samples were then air and/or oven dried. The dried samples were then put through a nest of sieves. The quantities of each sieve were weighed. Cumulative percents were computed and plotted against grain size as represented by the corresponding phi value. As set forth in the 1984 *Shore Protection Manual*, half phi values as well as phi 84 and phi 16 percentages were interpolated from the graphed data. The phi 84 and phi 16 values were then used to determine the mean and standard deviation. All samples were described based on the Wentworth Soils Classification System. Composite data was determined for the: above tidal (+12′, +6′), intertidal (+3′, 0′, -2′), below tidal (-6′, -10′), and total beach (+12′, +6′, +3′, 0′, -2′, -6′, -10′). Composites were also done for above tidal, intertidal, below tidal and total line, for each of the six lines. Analysis was done as described above for the individual samples."

A-101. Results are shown on Table A-9. Average values indicate strong similarity between sample lines. The average composite grain size for the dry beach and offshore is 0.35 and 0.41 mm respectively. The intertidal material is coarser, at 0.66 mm.

Coastal Structure Evaluation

A-102. Previous Survey Records. As improvement to the Port Monmouth shoreline has generally been small in scope, reports of detailed structure surveys are scarce. Comprehensive documentation of contemporary structure surveys is limited to the 1960 Interim Hurricane Study (USACE, 1960) and the 1993 Reconnaissance Report (USACE, 1993). Site inspections conducted for the 1993 report are amended with more recent inspections conducted in 1995 to constitute the existing conditions structure survey of this report. The 1960 report indicates the following:

> "Jetties have been constructed on both the east and west sides of the mouth of Pews Creek and there has been little evidence of accretion at either of these structures.

TABLE A-9

NOVEMBER 1995 SEDIMENT SAMPLE CHARACTERISTICS (ROGERS, INC. 1995)

	PL-210		PL-212		PL-214		PL-216		PL-218		PL-220		COMPOSITE	
ELEVATION (FEET, NGVD)	MEAN (mm)	PHI SORTING	MEAN (mm)	PHI SORTING	MEAN (mm)	PHI SORTING	MEAN (mm)	PHI SORTING	MEAN (mm)	PHI SORTING	MEAN (mm)	PHI SORTING	MEAN (mm)	PHI SORTING
-10	0.33	0.3	0.41	0.2	0.41	0.4	0.44	0.3	0.44	0.2	0.38	0.2		
-6	0.38	0.4	0.41	0.4	0.44	0.4	0.44	0.5	0.54	0.6	1.07	1.6		
-2	0.44	0.4	0.47	0.3	0.44	0.5	9.85	1.9	0.76	0.5	1.32	1.1		
0	0.44	0.4	0.47	0.3	0.38	0.3	1.32	1.6	0.66	0.5	1.41	1.3		
3	0.33	0.2	0.41	0.2	0.35	0.2	0.38	0.3	0.44	0.3	0.38	0.3		
6	0.38	0.3	0.35	0.1	0.31	0.2	0.41	0.2	0.41	0.3	0.41	0.3		
12	0.35	0.2	0.31	0.2	0.35	0.3	0.38	0.3	0.29	0.1	0.29	0.2		
ABOVE TIDAL	0.38	0.2	0.33	0.2	0.33	0.2	0.44	0.2	0.35	0.3	0.50	0.6	0.35	0.2
INTERTIDAL	0.38	0.3	0.47	0.3	0.41	0.2	2.00	2.2	0.62	0.6	0.87	1.2	0.66	0.9
BELOW TIDAL	0.35	0.4	0.41	0.3	0.44	0.3	0.44	0.4	0.47	0.5	0.50	0.6	0.41	0.5
TOTAL LINE	**0.35**	**0.3**	**0.41**	**0.2**	**0.38**	**0.3**	**1.32**	**1.8**	**0.50**	**0.5**	**0.54**	**0.7**	**0.41**	**0.7**

NOTE: PHI < 0.5 IS CONSIDERED WELL SORTED
PHI > 1.0 IS CONSIDERED POORLY SORTED

> From the mouth of Pews Creek to Compton Creek, there are 13 timber groins, six of which were constructed by the State and township between 1942 and 1943. The balance of the groins and about 1,600 feet of timber bulkhead were constructed by private parties prior to 1930. Private interests have constructed about 2,100 feet of timber and steel sheet pile bulkhead just west of the mouth of Compton Creek to protect the fish processing plant at that location and to stabilize the west shore at the mouth of the creek."

A-103. These groins were reported to feature top elevations varying from 2.7 to 8.2 feet MLW, ranging in condition from good to poor. Although the groins ranged in length from 60 to 265 feet, their effective sediment trapping was reported to be minimal.

A-104. Existing Conditions. Aerial and oblique photographs of the study area were analyzed to evaluate the effectiveness of existing coastal structures. Their effectiveness for erosion control and sand trapping was determined by comparing photographs from 1974, 1990 and 1995.

A-105. The shoreline of Port Monmouth is stabilized at each end by jetties on Compton Creek and Pews Creek. At both of these inlets a significant shoreline offset exists, with the Port Monmouth shoreline seaward of the adjacent beach in both cases. Therefore, jetties at both inlets appear to be effective in trapping a large fillet of sand, helping to maintain the Port Monmouth shoreline.

A-106. The jetties at Pews Creek have been recently improved and are in good condition. The west side of the creek features a wood bulkhead protecting the creek's interior, with a low wooden groin extending seaward as a jetty (Photo A-3). The southern terminus of the bulkhead is approximately 400 feet from the shoreline, landward from which the creek extends unprotected. Similarly, the wood bulkhead on the creek's east side extends a short distance landward, from which the marina's small craft docks extend. The wood bulkhead itself is in good shape, extending seaward to just beyond the shoreline. From this point extends the rock jetty which is in good shape and seems effective in trapping sand (Photo A-4).

A-107. Compton Creek, however, is more extensively protected on the side opposite from Port Monmouth, where an extensive jetty extends westward from the existing marshland, turning seaward at its end (Photo A-5). This jetty protects Belford Harbor from bay exposure due to Compton Creek's skewed orientation. This jetty appears to be solid and in adequate condition. It is constructed of rock and its landward and seaward sections are relatively high and moderate in elevation, respectively. The western jetty at Compton Creek is a low wooden structure of a construction similar to timber groins. The western side the protection extends into Belford Harbor with a wooden bulkhead of poor to fair condition. From the beach to the harbor, the bulkhead protecting the lands is particularly

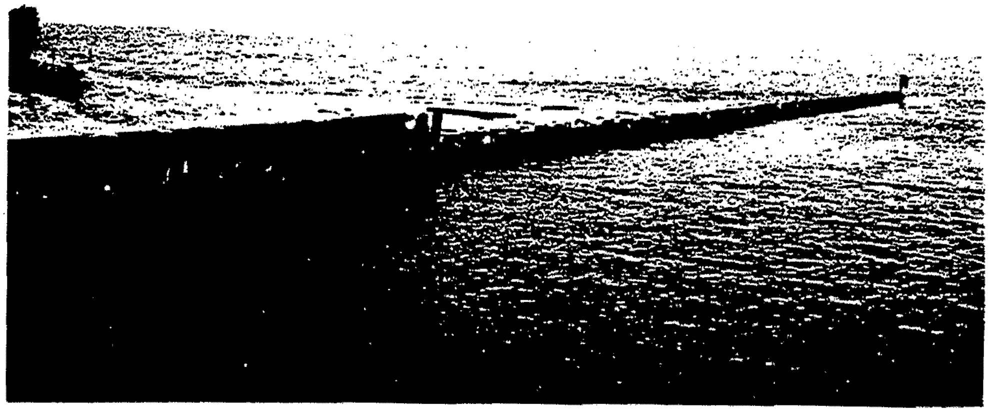


Photo A-3
West bulkhead/groin jetty at Pews Creek.
September, 1995

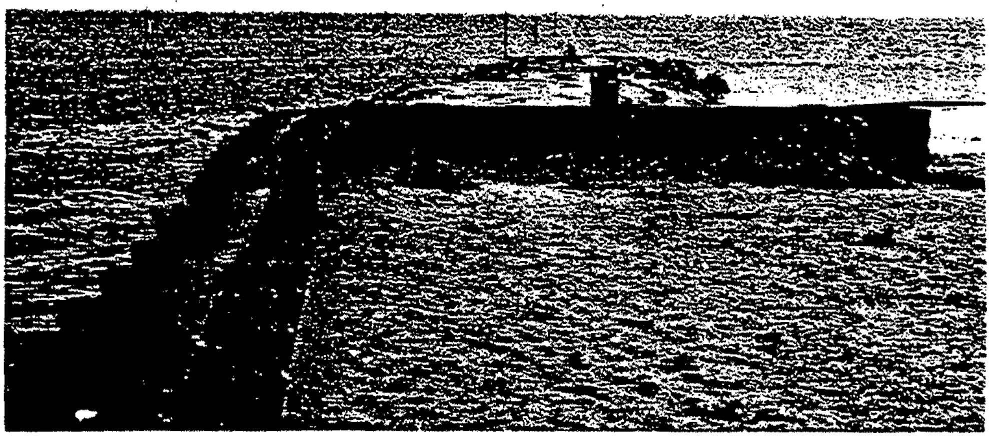


Photo A-4
East bulkhead/rock jetty at Pews Creek.
September, 1995

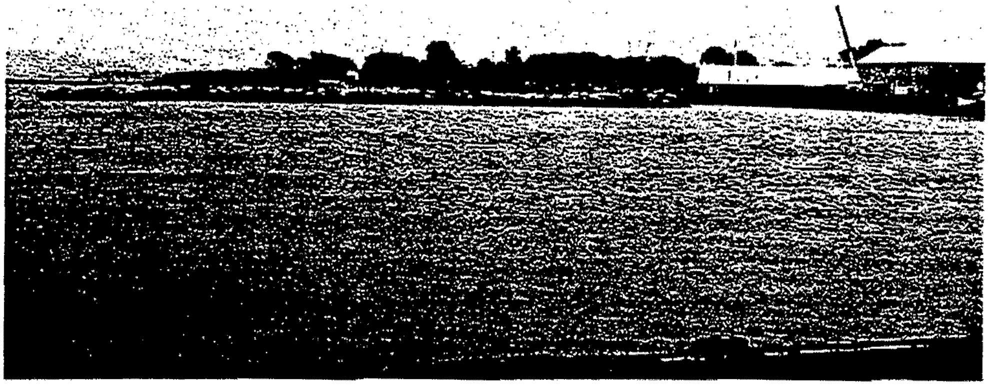


Photo A-5
Rock jetty on east side of Compton Creek.
September, 1995

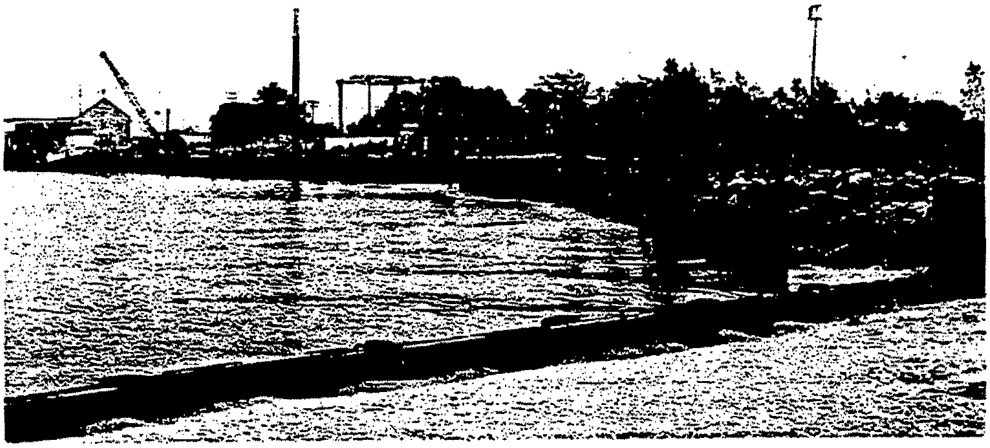


Photo A-6
Landward section of western wood groin jetty and bulkhead at Compton Creek.
September, 1995

dilapidated and nearly ineffective (Photo A-6). The jetty itself, however, appears to be fairly effective in trapping sand from transport into the inlet.

A-108. Cross-shore structures dominate the remainder of the Port Monmouth shoreline. Seven woodpile groins are present near the center of the study area, extending from east of the Spy House nearly to Compton Creek. The first three of these are in derelict condition, and not visible throughout the tide cycle. The easternmost four groins are low but relatively new and in good condition (Photo A-7).

A-109. The shoreline effect of the western groins is minimal. The eastern four, however, appear more effective in trapping sand. The central groin features a shoreline offset of approximately 40 feet (November 1995), with the beach west of the groin located seaward, indicating easterly transport. The remaining 3 eastern groins feature minimal shoreline offsets, but significant symmetrical shoreline bulging. This dynamic seems to indicate low net transport, as the gross transport is balanced. Further west, a new fishing pier extends into the bay near the location of the Spy House. The pier is porous in design and seems to have minimal effect on the shoreline or sediment transport. Finally, a derelict wall running 650 feet along the base of the dune near Compton Creek represents the only longshore structure (Photo A-8).

A-110. <u>Summary</u>. With the current approach of periodic maintenance fill of the Port Monmouth shoreline, it appears that the structural protection of the beach is adequate to marginally inadequate. Four of the area's groins offer protection against alongshore loss of material from the study area shoreline, which seems to present a nodal point of sediment transport. The terminal structures at the creeks also offer some stability to the shoreline, despite the fact that their conditions range from good to poor. In contrast with the groins, the Pews Creek terminal jetty is built of rock rubble, and can be expected to endure well with regular maintenance. The timber construction of the groins themselves (as well as portions of the terminal structures) is expected to have a shorter life span, despite their currently new appearance.

<u>Existing Beach Characteristics</u>

A-111. <u>Available Survey Data</u>. For analysis of the Port Monmouth shoreline and its changes, a limited set of recent and historical surveys is available. The most recent and most complete survey data available is that measured for this study (Rogers, 1995). This data includes 12 profile lines (identified as lines PL-210 to PL-221) extending from the seawardmost dune line to well offshore. Onshore measurements were conducted by rod and level, while offshore measurements were made with a fathometer. This survey was conducted in November, 1995, consecutive with topographic mapping (with 2 foot intervals) of the entire community.

Photo A-7
Wood groins along eastern shoreline.
September, 1995

Photo A-8
Derelict wood wall at base of dune at location of
old fish factory near Compton Creek.
September, 1995

A-112. A profile line survey was also conducted along Raritan and Sandy Hook Bays by the USACE in May-July, 1957 (USACE, 1960). The survey included historical profile lines 21 and 22 near the west and east extremities of Port Monmouth, at locations near 1995 profile lines PL-210 and PL-220, respectively.

A-113. Aerial photographs taken in 1959 complement the above period's shoreline data. Additional shoreline data (as well as offshore contour data) is available from the area's Reconnaissance Study of 1993 (USACE, 1993). This report includes shoreline position data for the years 1988, 1970-76, and 1836. Consecutive profile survey data for these years is not available.

A-114. Beach Dimensions. Dimensions of the Port Monmouth beach were taken from the November 1995 onshore and offshore survey (Table A-10). This survey was conducted on 12 profile lines (PL-210 - PL-221), the locations of which are shown on Figure A-5. Qualities of the beach that are consistent throughout the study area include the dramatic slope change from the steep beach and dune slopes to the nearly flat offshore slope. This slope change, occurring at a point identified as the beach toe, is found between elevations -2.4 and -3.9 feet NGVD. An additional feature consistent along the study area is the nearly complete lack of a beach berm. The beach extends from the shoreline along a nearly uniform slope to a point identified as the dune toe, from which the slope is more steep. Dune crest elevations are relatively high except in the area's easternmost length. The existing beach profiles are presented in Sub-Appendix A1.

Shoreline and Offshore Contour Changes

A-115. The November 1995 survey (Rogers, 1995) is incorporated with data presented in the recent Reconnaissance Report (USACE, 1993) in order to evaluate 1836-1995 Port Monmouth shoreline changes. The location of the shoreline in the years studied is indicated in Figure A-6 for the shoreline within the study area. All measurements of shoreline location and changes are measured at the mean high water contour. Shoreline changes for the various periods of study are presented in Table A-11 and Figure A-7.

A-116. Landward shoreline retreat seems the dominant trend, as three of the four time periods feature an almost exclusive shoreline loss. The period 1836-1957 experienced retreat over nearly the entire shoreline length, but particularly severe at Pews Creek and at the central length of the shoreline. The average shoreline retreat in the period was 132 feet, or -1.1 feet per year.

A-117. Shoreline data from the following period show the effect of a substantial beach fill and dune project that the State of New Jersey constructed in 1966. Shoreline width increases as great as 365 feet contributed to an average shoreline growth of 240 feet.

TABLE A-10
BEACH PROFILE CHARACTERISTICS
NOVEMBER, 1995

PROFILE NAME	BEACH WIDTH (FEET)	DUNE HEIGHT (FT, NGVD)		ELEVATION OF TOE OF BEACH (FT, NGVD)	SLOPE (1: ..)	
		CREST	TOE		ONSHORE	OFFSHORE
PL-210	87	17.2	9.0	-2.5	14.1	341
PL-211	94	19.9	9.8	-3.0	14.2	383
PL-212	83	23.0	8.8	-2.4	14.8	315
PL-213	78	15.7	7.5	-3.7	14.2	484
PL-214	76	17.4	7.6	-3.5	14.5	554
PL-215	44	17.1	6.7	-3.4	11.5	600
PL-216	60	18.0	7.1	-3.5	13.2	881
PL-217	50	19.3	6.4	-3.3	12.8	577
PL-218	51	18.0	7.6	-3.2	12.7	416
PL-219	65	16.4	7.0	0.0	14.2	279
PL-220	120	12.0	10.1	-3.5	28.7	514
PL-221	65	12.7	7.1	-3.9	20.6	750
AVERAGE	73	17.2	7.9	-3.0	15.5	508

NOTE:

TOE OF DUNE ALSO REPRESENTATIVE OF CREST OF BERM (WHICH FEATURES 0' WIDTH).
DUNE TOE IS TYPICALLY LOCATED WHERE SURVEYS INDICATE A SLOPE CHANGE.
BEACH TOE IS THE MOST DISTINCT NEARSHORE SLOPE CHANGE
BEACH WIDTH MEASURED BETWEEN THE DUNE TOE AND MHW CONTOURS.
ONSHORE SLOPE MEASURED BETWEEN BEACH TOE AND +10' NGVD.
OFFSHORE SLOPE MEASURED FROM BEACH TOE OUT 1,500 FEET.

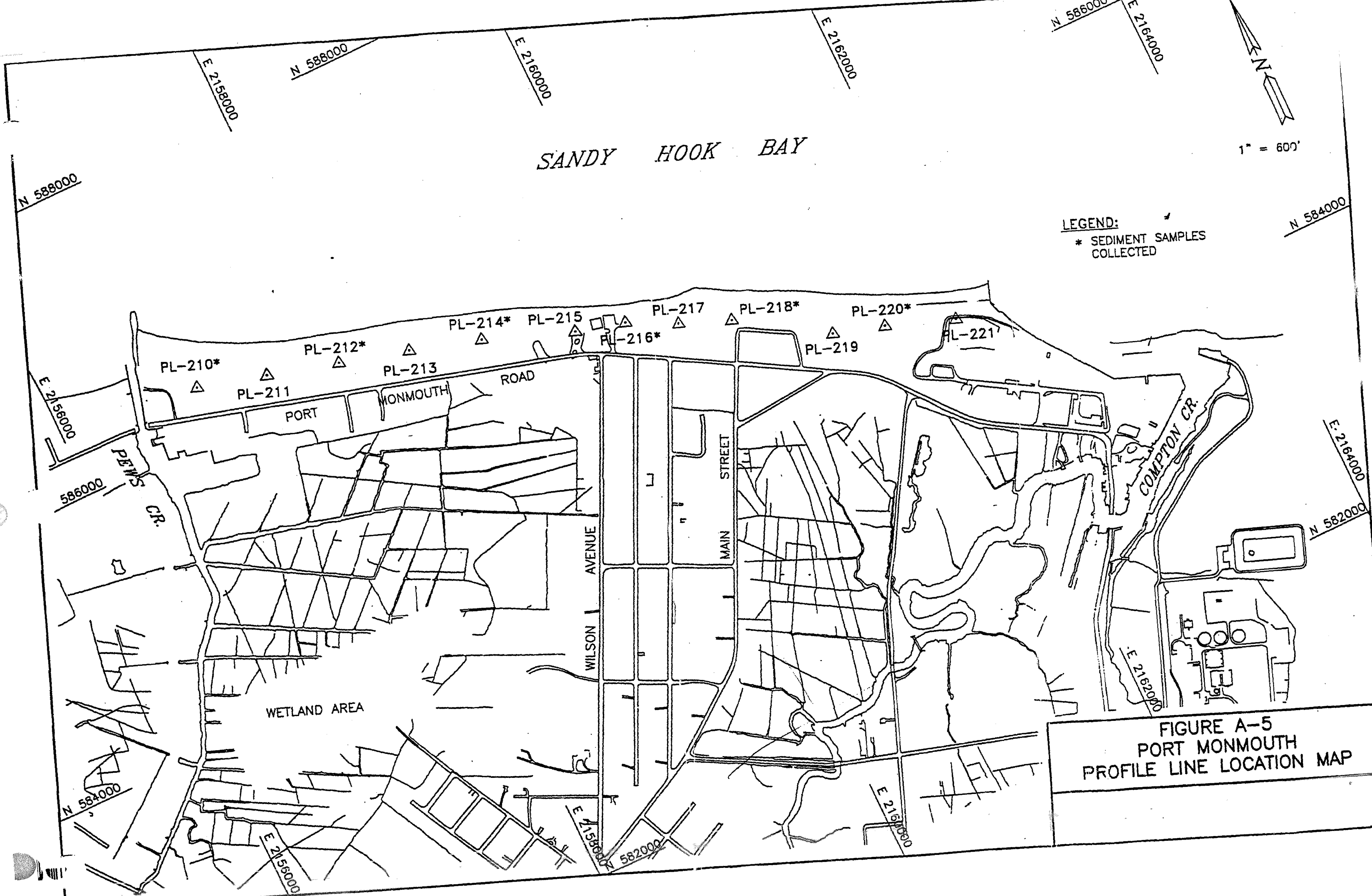
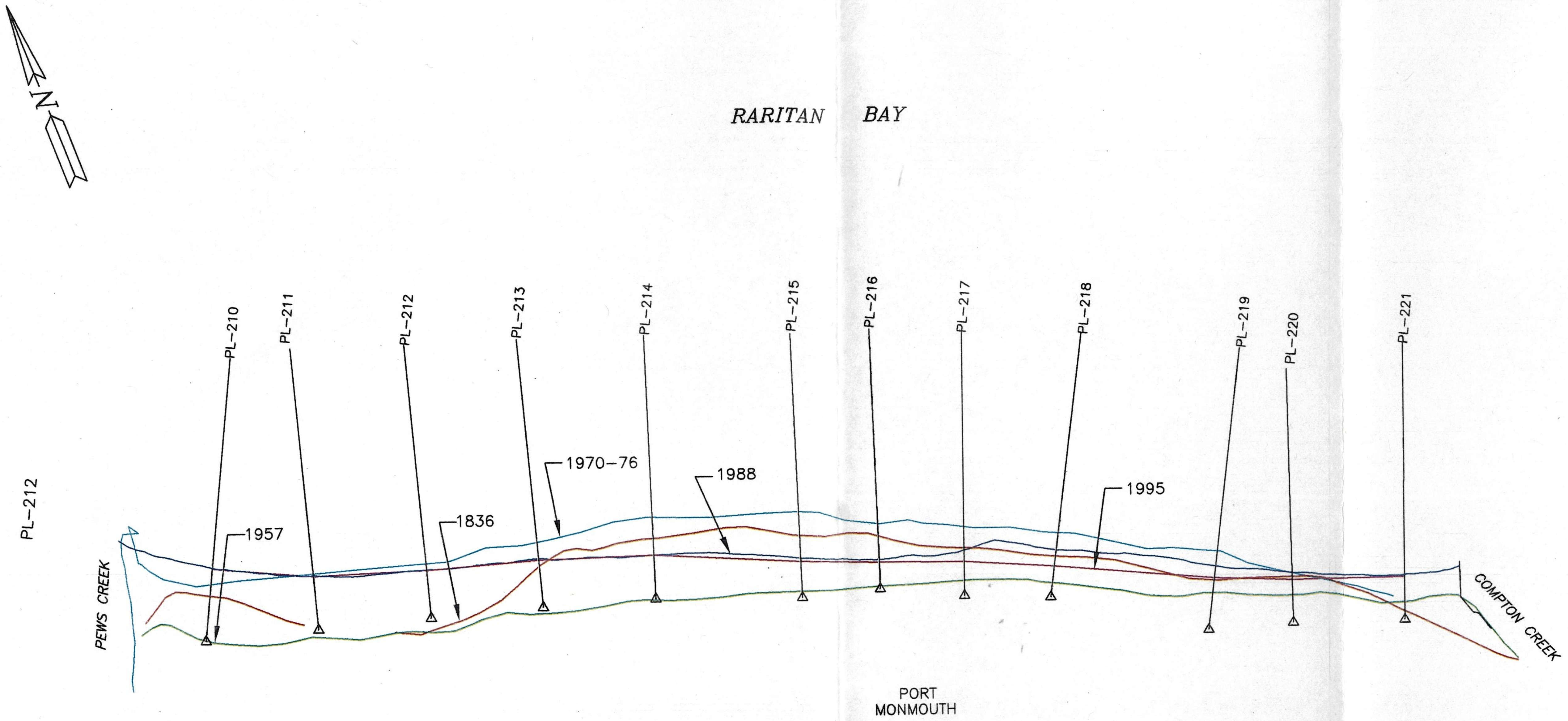


FIGURE A-5
PORT MONMOUTH
PROFILE LINE LOCATION MAP



LEGEND :

△ DENOTES PROFILE ORIGIN
— DENOTES NOVEMBER 1995 MEAN HIGH WATER LINE
— DENOTES 1988 MHW LINE (TAKEN FROM "TOWNSHIP OF MIDDLETOWN TAX MAPS"
— DENOTES 1970-76 MHW LINE (TAKEN FROM U.S.G.S. QUAD SHEETS)
— DENOTES 1957 MHW LINE (TAKEN FROM "BEACH EROSION AND HURRICANE SURVEY, RARITAN BAY AND SANDY HOOK BAY, N.J.; 30 NOVEMBER 1960; PLATES 5, 6, & 7)
— DENOTES 1836 MHW LINE (TAKEN FROM "BEACH EROSION AND HURRICANE SURVEY, RARITAN BAY AND SANDY HOOK BAY, N.J.; 30 NOVEMBER 1960; PLATES 5, 6, & 7)
— SUPPLEMENTAL TOPOGRAPHY (1940's & 1950's) TAKEN FROM U.S.G.S. QUAD SHEETS

500 0 500

GRAPHIC SCALE IN FEET

PORT MONMOUTH NEW JERSEY
HISTORICAL MHW
SHORELINE POSITIONS

FIGURE A-6

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TABLE A-11

MHW SHORELINE CHANGE AT PORT MONMOUTH, NEW JERSEY

PROFILE NAME	1836 TO 1957		1957 TO 1970/76		1970/76 TO 1988		1988 TO 1995	
	TOTAL (FEET)	ANNUAL (FT./YR.)	TOTAL (FEET)	ANNUAL (FT./YR.)	TOTAL (FEET)	ANNUAL (FT./YR.)	TOTAL (FEET)	ANNUAL (FT./YR.)
PL - 210	-198	-1.6	248	16.1	72	4.8	-2	-0.3
PL - 211	-51	-0.4	281	18.2	-7	-0.5	-6	-0.8
PL - 212	-10	-0.1	292	18.9	-18	-1.2	-1	-0.1
PL - 213	-209	-1.7	316	20.5	-72	-4.8	-6	-0.8
PL - 214	-274	-2.3	365	23.7	-166	-11.1	-1	-0.1
PL - 215	-246	-2.0	351	22.8	-207	-13.8	-13	-1.7
PL - 216	-220	-1.8	276	17.9	-159	-10.6	-13	-1.7
PL - 217	-143	-1.2	235	15.2	-105	-7.0	-48	-6.1
PL - 218	-118	-1.0	218	14.1	-73	-4.9	-71	-9.1
PL - 219	-58	-0.5	185	12.0	-66	-4.4	-52	-6.6
PL - 220	-82	-0.7	98	6.4	2	0.1	-31	-4.0
PL - 221	30	0.2	11	0.7	108	7.2	-8	-1.0
AVERAGE	**-132**	**-1.1**	**240**	**15.5**	**-58**	**-3.8**	**-21**	**-2.7**

NOTE:

1995 DATA FROM SURVEY BY ROGERS INC.
PRIOR DATA FROM USACE (1993)
1970/76 DATA ASSUMED TO REPRESENT 1973 CONDITIONS FOR RATE COMPUTATIONS.

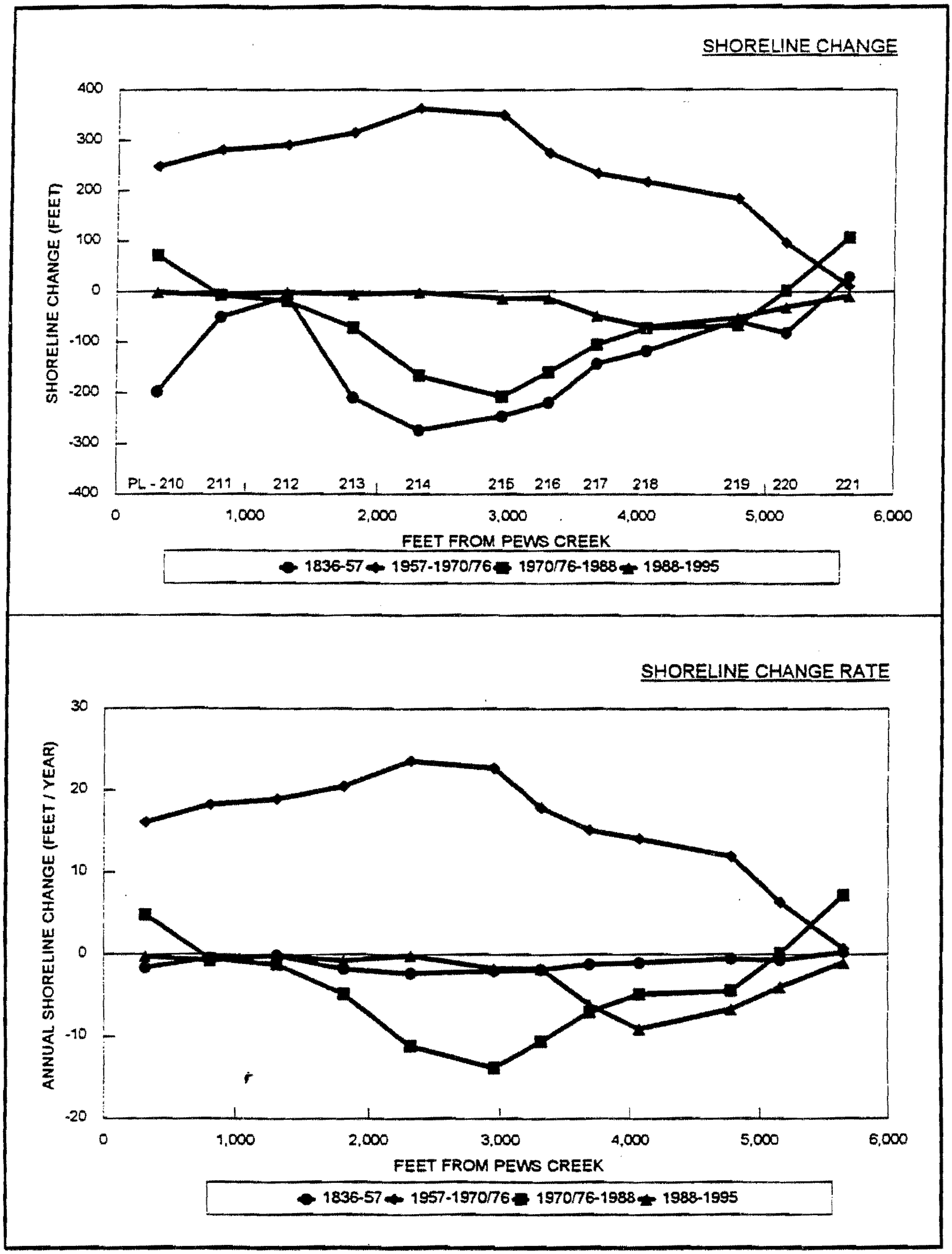


FIGURE A-7

SHORELINE CHANGES ALONG PORT MONMOUTH SHORELINE

A-118. The period was followed by shoreline recession. The years 1970/76 to 1988 experienced a return to the pre-project trend. Very similar shoreline retreat rates occurred throughout the regions except near the creeks, where the dry beach grew (Figure A-7). The most recent period, 1988 to 1995, has seen reduced losses, with some higher retreat in the eastern shoreline.

A-119. Generally, the natural trend of the beach is one of gradual recession. The central section of the study area seems to be particularly dynamic.

A-120. Changes in the offshore depth contours are illustrated in Figure A-8, calculated from the November 1995 survey and data presented by the USACE (1993). The -6 and -12 foot MLW contour locations were relatively stable from 1836 through 1954. The November 1995 -6 foot contour has migrated landward, indicating offshore deepening. This process is the most noticeable adjacent to Pews Creek where sand has apparently adjusted to fill in the borrow area from the 1966 beach fill project. This borrow area is located just seaward of the -6 foot contour. The dramatic appearance of offshore changes in Figure A-8 is in part due to the flat bathymetry.

Volumetric Changes

A-121. A variety of methods can be used to calculate the volumetric changes along a shoreline. Regardless of any method's inherent accuracy, all are sensitive to the abundance and accuracy of data used in the volumetric analysis. Thus, the selection of the best method of calculating volume changes requires careful inspection of available data. In many studies accurate onshore and offshore profile survey data exists for various times, and the comparison of the profiles can be used to directly compute the volumetric changes.

A-122. At Port Monmouth two profile lines, PL-210 and PL-220, closely match the origin location and azimuth of profile lines surveyed in 1957 (USACE, 1960), formerly identified as lines 21 and 22. Their profile comparison is shown in Figure A-9. Apart from known changes in the profiles in the intervening period due to beach and dune fill as well as due to the creation of an offshore borrow area, the profiles appear inconsistent with one another. The appearance of such bed elevations that vary consistently a great distance offshore is frequently interpreted as survey inaccuracy. Significant bed elevation changes are unexpected at depths beyond the closure depth, which is calculated to be -3.3 feet NGVD (see below). Therefore, significant evidence exists that one of these surveys, namely the 1957 survey, is significantly in error. Uncertainties that exist regarding the accuracy of 1957 survey include questions of the specific location of the profile origins.

A-123. In addition to appearing inaccurate, the 1957 profile survey at Port Monmouth is inadequate as it is limited to two profile lines near the creeks. Thus any volumetric analysis based on profile comparison with this survey would be particularly sensitive to

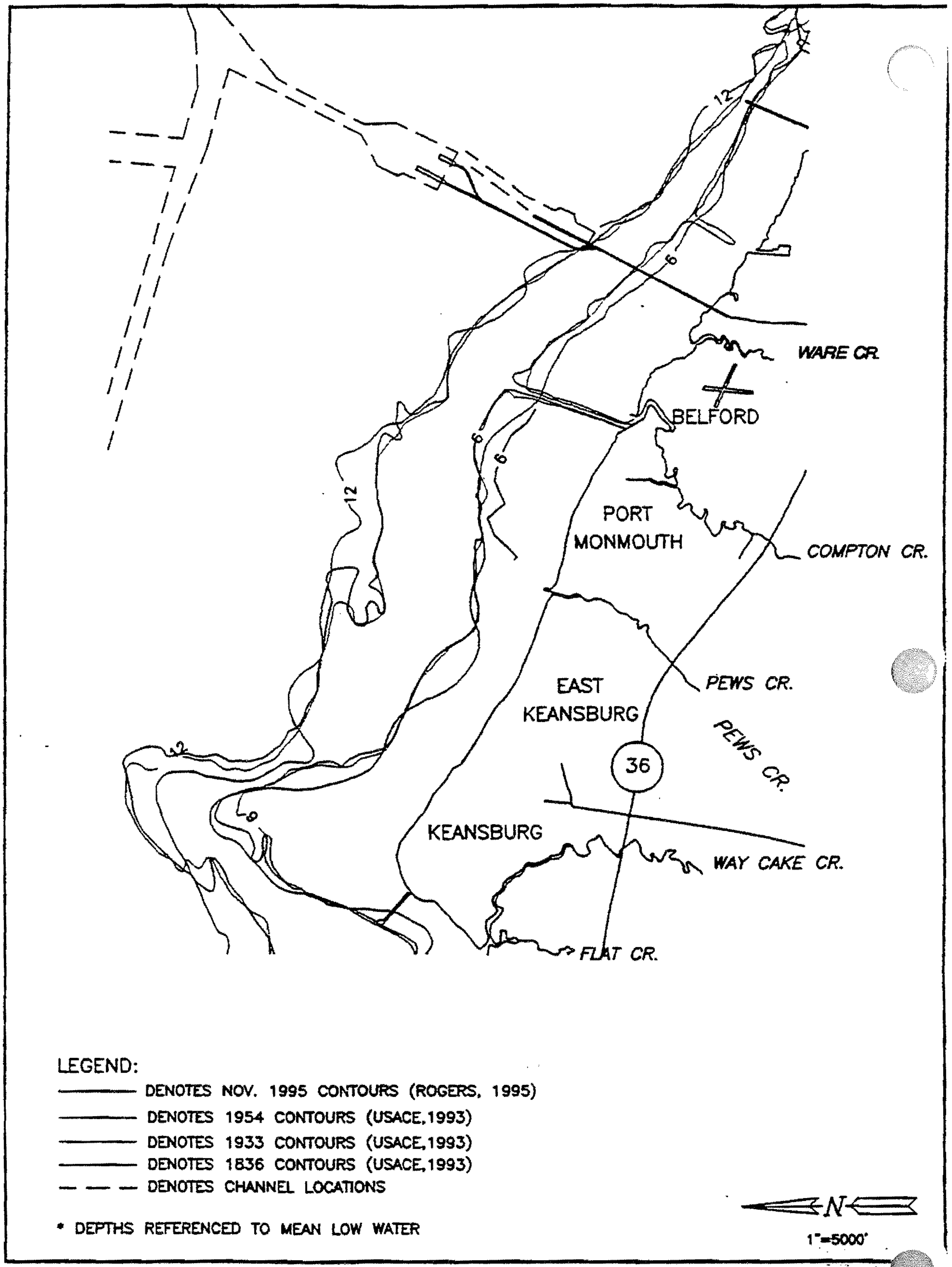


FIGURE 8

DEPTH CONTOUR CHANGES
NEAR PORT MONMOUTH
1836–1996

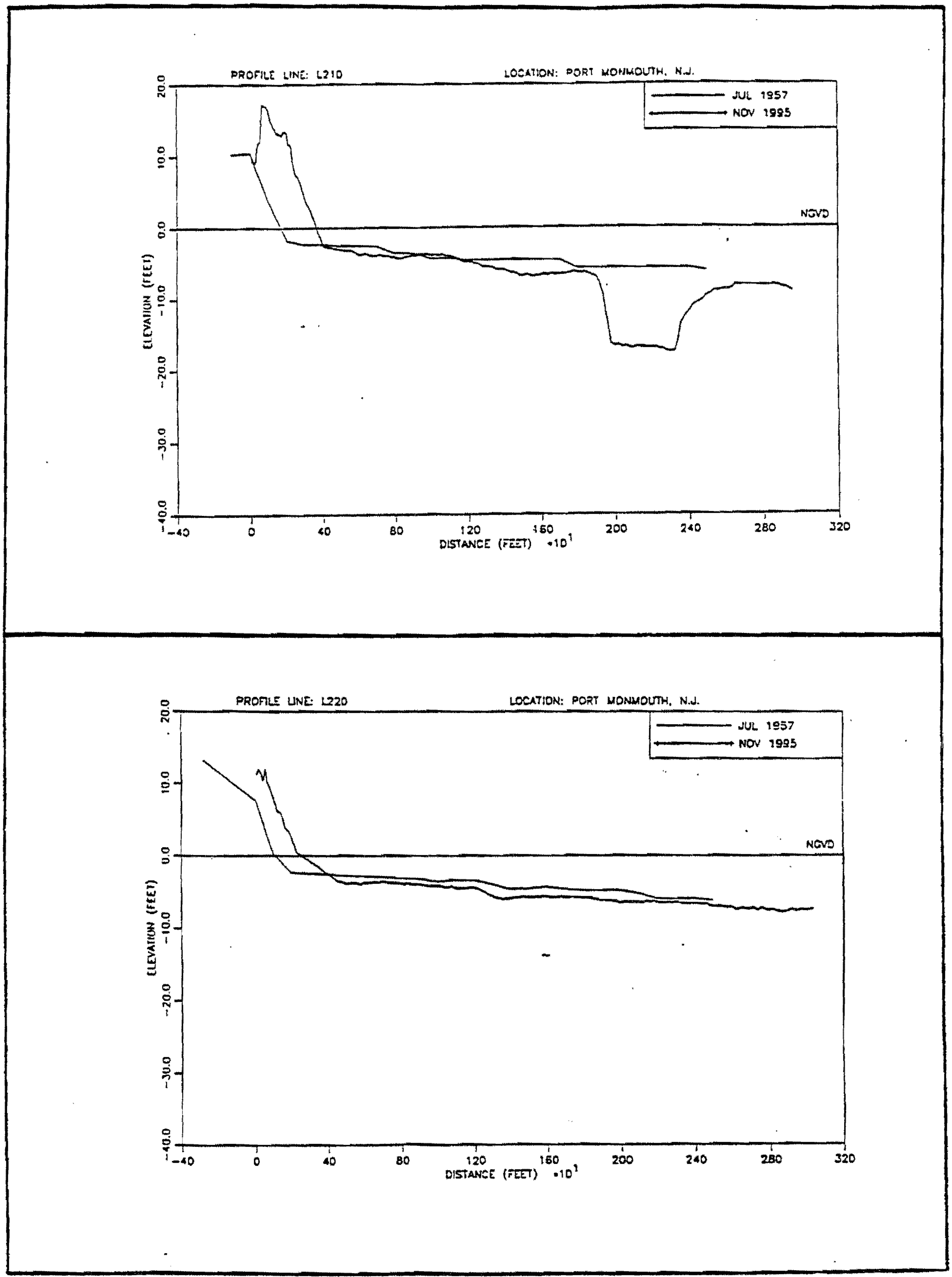


FIGURE A-9

SURVEYED PROFILE COMPARISONS

1957 AND 1995

highly localized effects, such as dunes constructed at these locations. For these reasons, an alternate method of calculating volumetric changes is applied.

A-124. Volumetric change data are calculated from changes in historical and recent shoreline locations. In addition to the recent survey's data (Rogers, November 1995), shoreline data is available for the years 1836, 1957, 1970, 1976 and 1988 (USACE, 1993). For the volumetric analysis it is assumed that the shoreline change occurred uniformly from the average berm crest to the contour representing the depth of closure.

A-125. The closure depth is calculated by the theory of Birkemeier (1985). In various length periods of measurement during the author's study, changes in the measured profile were observed to identify the deepest point to which the profiles showed a significant elevation difference. Seaward of this point, the profiles consistently featured matching elevations. For each of the periods of measurement, the elevation of this point (interpreted to be the effective closure depth during the study) was compared with the nearshore wave characteristics measured in the period. Correlating the data, the author presented a closure depth estimate for given incident wave height and period (presented as wave values exceeded only 12 hours per year).

A-126. For the Port Monmouth study area, inputting predicted fetch limited wave conditions (Table A-7,) into the predictive closure depth theory results in a closure depth contour value of -3.3 feet NGVD. Assuming that shoreline adjustments occur uniformly from the contour to the average 7.9 foot berm crest equivalent to the dune toe (see Table A-10), volumetric changes are calculated by multiplying the shoreline change in feet by 0.415 cubic yards per linear foot of beach.

A-127. The results indicate that the shoreline is prone to experience a mild volumetric erosion rate (Figure A-10, Table A-12), which is greatest in the area's central reach. From 1836 to 1957, 323,000 cubic yards (-2,700 c.y. annually) eroded from the project area. The next period, 1957 to 1970/76, saw the shoreline gain 592,000 cubic yards. This compares reasonably well with the 540,000 cubic yards estimated to have been installed in the 1966 beach fill project (NJ, 1966). From 1970/76 to 1988 the shoreline returned to its previous erosional trend, but with accretion near the creeks; total losses were 135,000 cubic yards, or -9,000 cubic yards per year. The next period, 1988 to 1995 has seen a trend toward reduced erosion, with losses totaling 51,000 cubic yards, or -6,500 cubic yards per year.

Sediment Budget

A-128. A sediment budget was developed for the Port Monmouth area to quantify the volume of sand transported within and into/out of the study area. The 1988 to 1995 time period was chosen for the analysis in order to represent the most recent period over which data is available.

RECENT VOLUMETRIC CHANGE RATE AT PORT MONMOUTH

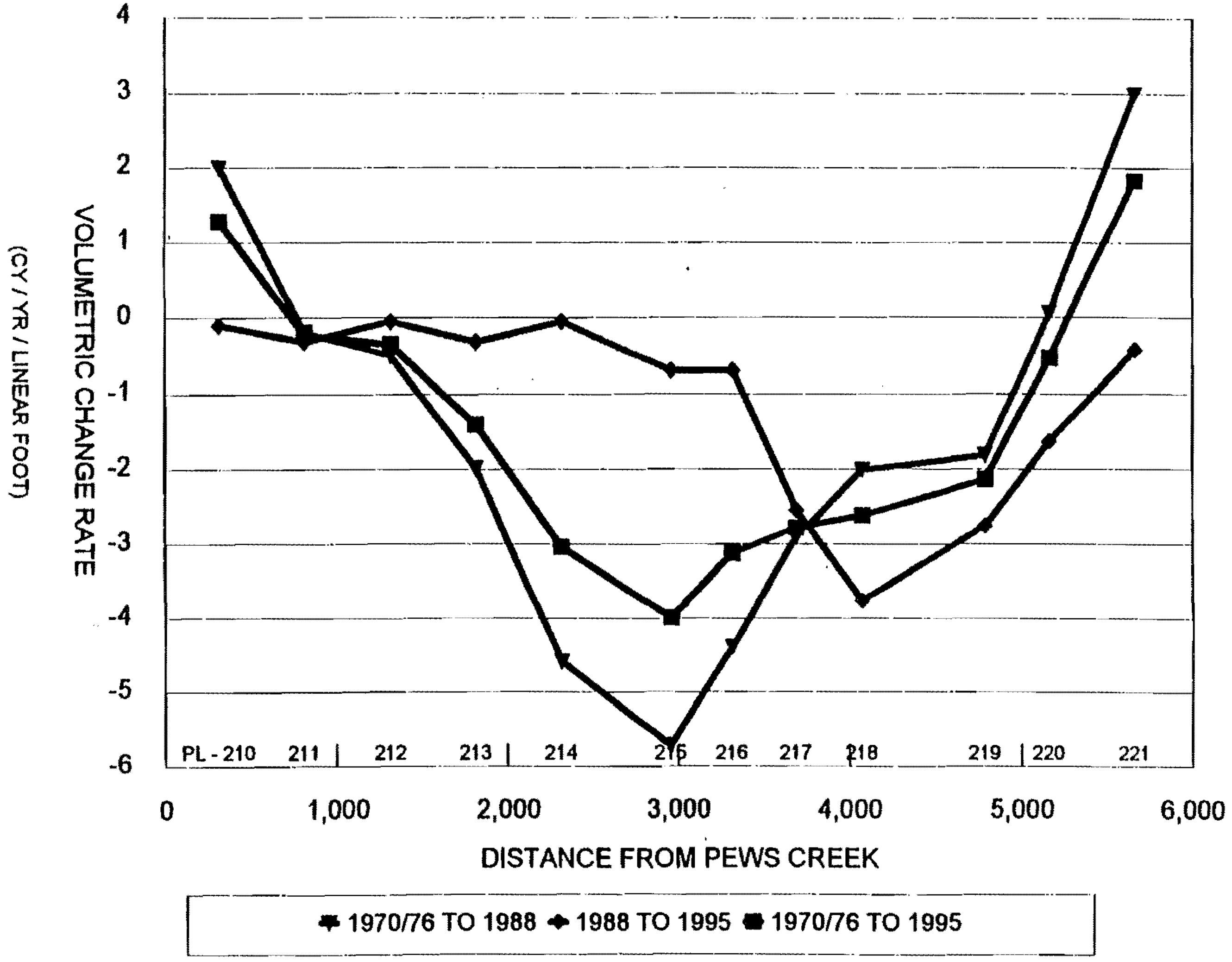


FIGURE A-10

TABLE A-12
VOLUMETRIC CHANGES
ALONG PORT MONMOU SHORELINE

LOCATION	TOTAL CHANGES (CY)			
	1836 TO 1957	1957 TO 1970/76	1970/76 TO 1988	1988 TO 1995
PL - 210	-46,405	58,124	16,875	-469
PL - 211	-10,578	58,281	-1,452	-1,244
PL - 212	-2,074	60,563	-3,733	-207
PL - 213	-43,353	65,548	-14,935	-1,245
PL - 214	-64,963	86,538	-39,357	-237
PL - 215	-51,084	72,889	-42,986	-2,700
PL - 216	-33,450	41,965	-24,175	-1,977
PL - 217	-22,445	36,885	-16,480	-7,534
PL - 218	-26,832	49,572	-16,600	-16,145
PL - 219	-13,111	41,818	-14,919	-11,754
PL - 220	-14,888	17,793	363	-5,628
PL - 221	6,222	2,281	22,400	-1,659
TOTAL:	**-322,961**	**592,257**	**-135,000**	**-50,799**
	ANNUAL CHANGES (CY/YR)			
PL - 210	-382	3,770	1,125	-60
PL - 211	-87	3,780	-97	-159
PL - 212	-17	3,928	-249	-26
PL - 213	-357	4,252	-996	-159
PL - 214	-534	5,613	-2,624	-30
PL - 215	-420	4,728	-2,866	-345
PL - 216	-275	2,722	-1,612	-252
PL - 217	-185	2,393	-1,099	-962
PL - 218	-221	3,215	-1,107	-2,061
PL - 219	-108	2,713	-995	-1,501
PL - 220	-122	1,154	24	-719
PL - 221	51	148	1,493	-212
TOTAL:	**-2,656**	**38,417**	**-9,000**	**-6,485**

NOTE:

1995 DATA FROM SURVEY BY ROGERS INC.

PRIOR DATA FROM USACE (1993)

QUAD MAP DATA (1970/76) ASSUMED TO REPRESENT 1973

1957 TO 1970/76 VOLUMETRIC CHANGES INCLUDE FILL FROM 1966 PROJECT

A-129. The Port Monmouth littoral cell under study extends from Pews Creek to Compton Creek, out to the depth of closure (-3.3 feet NGVD). At the creeks, the littoral cell includes the areas that undergo maintenance dredging, consisting of the entrance channel, areas immediately offshore, and the docking areas in close proximity to the bay.

A-130. Analysis of sand deposit patterns at Port Monmouth shoreline structures supports the conclusion that a sediment transport nodal point exists near the center of the area's shoreline. A western offset exists at most groins in the eastern half of the shoreline, as well as at Compton Creek. This indicates a net eastward flow of sand in the eastern segment of the Port Monmouth shoreline. At the shoreline's western end, the sand pattern adjacent to Pews Creek shows an eastern offset, indicating a net westward flow. For the sediment budget, the specific location of the nodal point is interpreted to be halfway between shoreline monuments PL-215 and PL-216. The shoreline is divided into individual beach segments based on the location of the nodal point (Figure A-11).

A-131. The longshore transport rate in the beach segments is based on the assumption that no net flow drifts into the Port Monmouth shoreline from the adjacent shorelines. Mechanical transport onto the shoreline from offshore, from the creeks, or from other sources did not occur in the period. In addition, data and observations do not support any conclusion of flow to or from regions offshore.

A-132. Therefore, the estimated transport rate out of each beach segment is dependent only on measured volumetric loss rates in the period. Interpretation of the period's shoreline changes indicates losses in the western and eastern segments of -800 and -5,700 cubic yards per year, respectively. These rates are assumed to be equivalent to the longshore transport rates from Port Monmouth into the respective creeks.

A-133. The maintenance dredging history of Pews Creek is detailed on Table A-1. The rate of sediment removal is assumed to be equal to the shoaling rate, and is presented as annual volumes. Following the initial dredging activity of record, in 1988, the average annual shoaling rate has been 5,400 cubic yards per year.

A-134. Table A-2 indicates the dredging history of Compton Creek. The annual dredging rate at that creek is also assumed to be reflective of the shoaling rate. Calculating the annual dredging rate over the longest period possible helps reduce inaccuracies that can result from sporadic dredging. Following the initial dredging in 1937, maintenance dredging had removed a total of 1,180,814 cubic yards through 1990. From these values, the annual shoaling rate can be calculated to be 22,300 cubic yards per year.

A-135. Some of the material shoaling in the region of the creeks drifts from the adjacent shoreline of Port Monmouth. The volume of sand that erodes from the western segment and drifts into Pews Creek is detailed above. Similarly, the volume that erodes from the eastern segment into Compton Creek is presented above. The remainder of the material

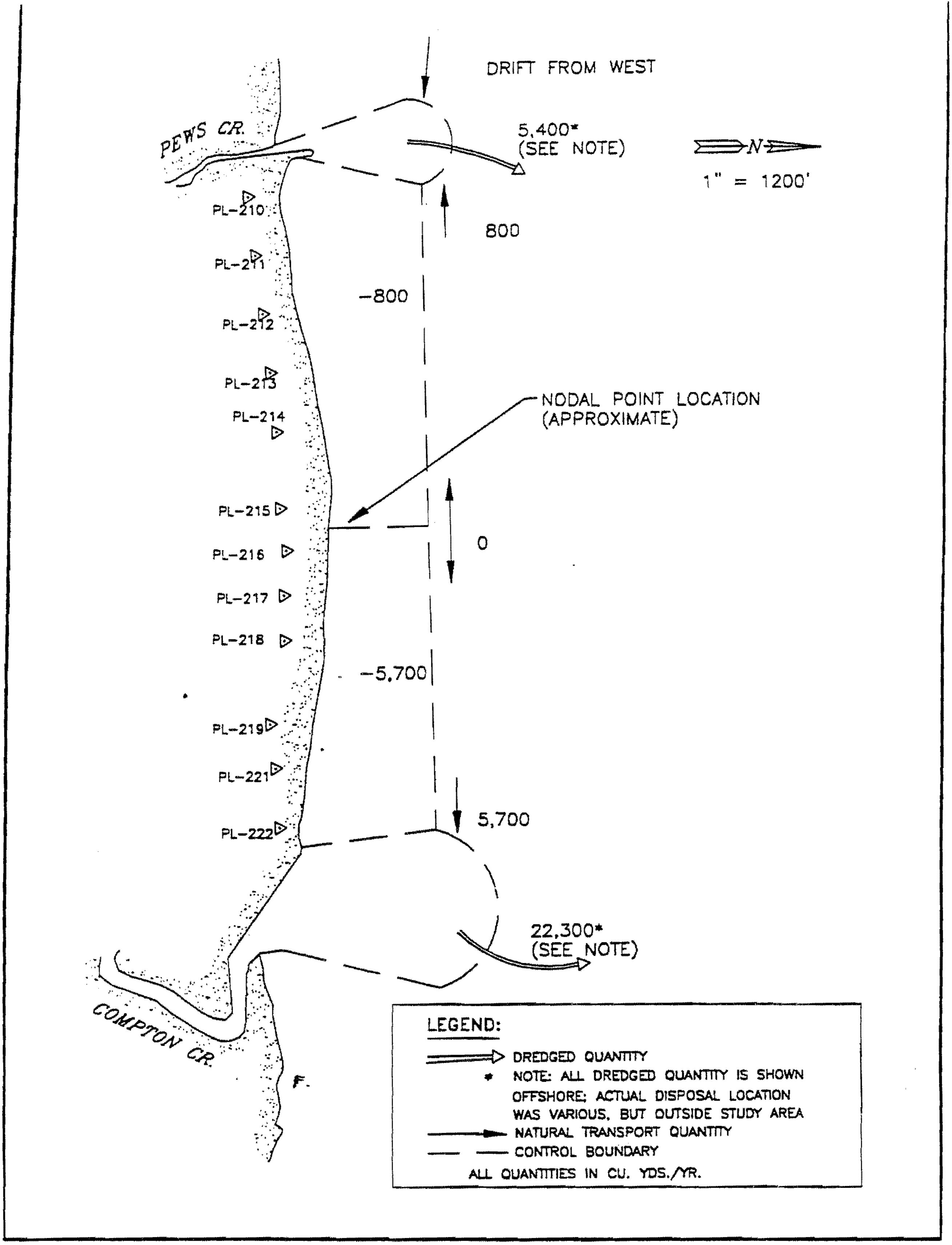


FIGURE A-11

SEDIMENT BUDGET
PORT MONMOUTH, NEW JERSEY
1988 – 1995: EXISTING CONDITIONS

that is calculated to shoal in the creek areas is assumed to drift from updrift reaches of the creeks and from the adjacent shorelines beyond Port Monmouth.

WITHOUT PROJECT CONDITIONS COASTAL PROCESSES

A-136. The benefits of an inundation control and shore protection project are primarily measured by the level of protection offered against storm damages. The anticipated level of storm damage with the project is compared to the damage level anticipated without the project, and the predicted reduction in damages is interpreted to be the anticipated benefits. This value is then compared with the project's costs. Therefore, with this approach it is imperative to obtain predicted damage values for the without project conditions that are as accurate as possible.

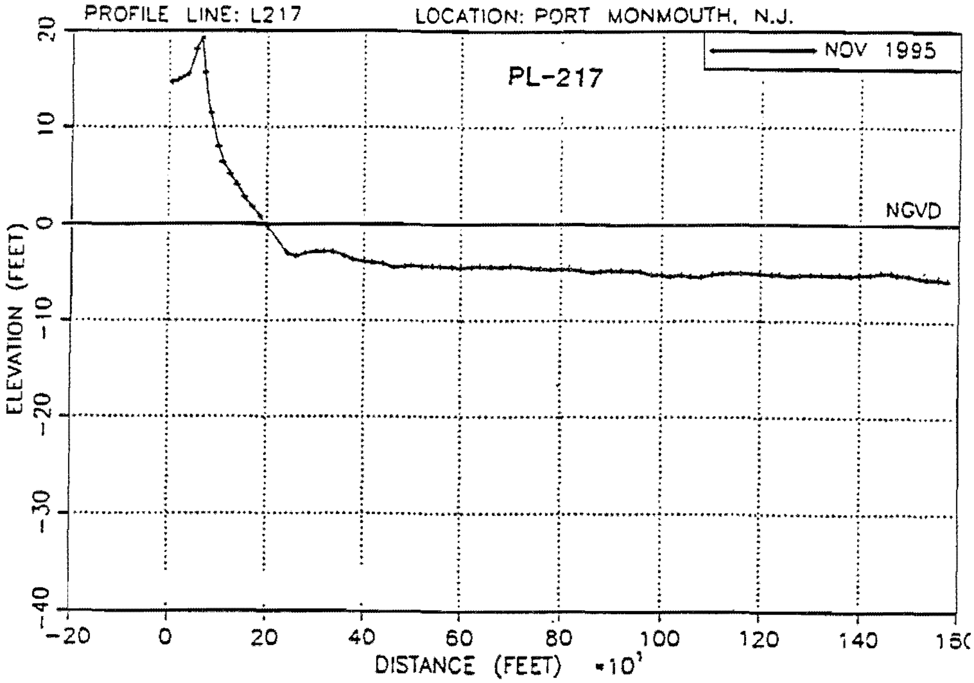
Existing Conditions Representative Profiles

A-137. Storm impacts (recession, runup, etc.) are particularly sensitive to certain beach features, such as slopes or elevations. As an attempt to address these sensitivities in coastal response to storms, two distinct profiles, PL-217 and PL-221 (Figure A-12) are selected for input into predictive models and equations. The variation in the two profiles' characteristics is fairly reflective of variations in the local profile dimensions. While PL-217 features a marginally higher dune elevation and steeper slopes, PL-221 features lower, flatter dimensions. These profiles are applied as input in the storm recession model in order to predict representative post-storm profiles. These post-storm profiles are then applied as input in the existing conditions runup and overtopping investigations.

Storm Induced Recession

A-138. The best analysis of storm-related erosion potential for coastal sites requires a long period of record over which the important storm parameters as well as the resultant storm erosion have been quantified. An alternative analysis is a numerical (computer) model capable of simulating the erosion effects of a particular set of storm parameters acting on a given beach configuration. For most locations, including Port Monmouth, the prototype information necessary for an historic-based recession analysis is unavailable. Therefore, a model was used in this study to evaluate storm-induced erosion.

A-139. The EDUNE dune erosion model by Kriebel (1989) was used in this study to evaluate the susceptibility of the existing coastal dunes to storm-induced erosion. The model assumes that for any given combination of wave and water level, there is a unique equilibrium beach profile which can be approximated by an exponential equation relating depth on the profile to distance from the shoreline. The input for this model includes the geometry of the dune profile and basic storm parameters of wave height, runup, elevation storm water level, and storm duration.



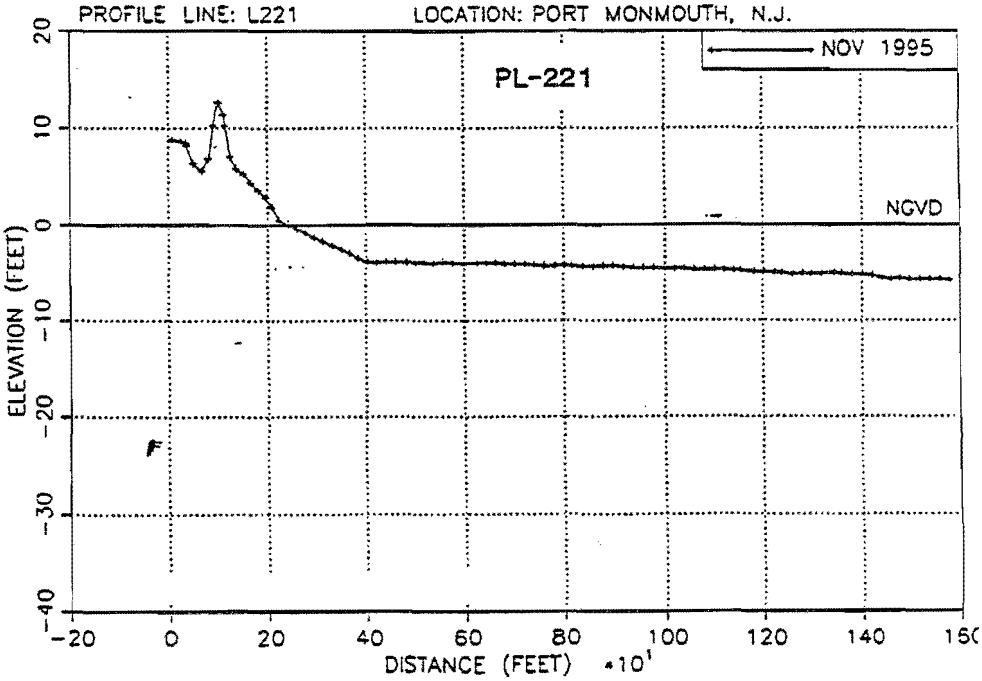


FIGURE A-12

REPRESENTATIVE BEACH PROFILES
EXISTING CONDITIONS

A-140. The EDUNE program models offshore slopes based upon equilibrium profiles (i.e., $Y = Ax^{2/3}$). The "A" coefficient can be evaluated from the beach sediment grain size. Recent sediment sampling indicates that the grain size at Port Monmouth averages 0.39 mm onshore and offshore (although material is more coarse in the narrow intertidal region), so this value was used to compute an A value of 0.212 $ft.^{1/3}$.

A-141. The derivation of other input parameters for the EDUNE model are as follows. The storm duration was assumed to be 18 hours, which is representative of the average between tropical and northeastern type storms. Post-storm dune slope is known to approach near vertical. In the absence of local data, Kriebel recommends using a steep slope on the order of 1:1, which was used for this study. The post storm beach slope value was 1:18, which is representative of the existing beach slope.

A-142. Storm recession effects were computed for the 2 representative profiles, PL-217 and PL-221. Storm conditions were input representing return intervals of 2, 5, 10, 25, 50, 100, and 200 years. Sample output profiles are presented in Figure A-13. The value used to quantify the storm recession for each of these cases is the recession extent, which is the distance between the existing 0' NGVD shoreline and the landward limit of the eroded portion of the profile. The resulting predicted values of recession extent, presented in Table A-13 and Figure A-14, range from 126 to 216 feet. Also shown is the remaining peak dune elevation predicted for each profile for each storm studied. Input values and output EDUNE profiles are presented in Sub-Appendix A2.

A-143. The runup value was computed on post-storm profiles output by EDUNE. This required several iteration steps in order to result in agreement between the runup value (used as EDUNE input) and the post-storm profile (with slopes used as runup theory input).

Wave Runup

A-144. Wave runup predictive values were determined for existing conditions at representative profiles PL-217 and PL-221. Wave runup values were calculated using predictive equations incorporated by the Dutch government for dike design (Pilarczyk, 1990). The runup equation is formulated to yield conservative estimates with a 2% exceedance level. The runup ($R_{2\%}$) is expressed as:

$$R_{2\%} = 0.7\ T_p \sqrt{gH_s}\ \tan\alpha$$

where T_p is the peak wave period, g is gravity, H_s is the offshore significant wave height, and $\tan\alpha$ is the beach slope.

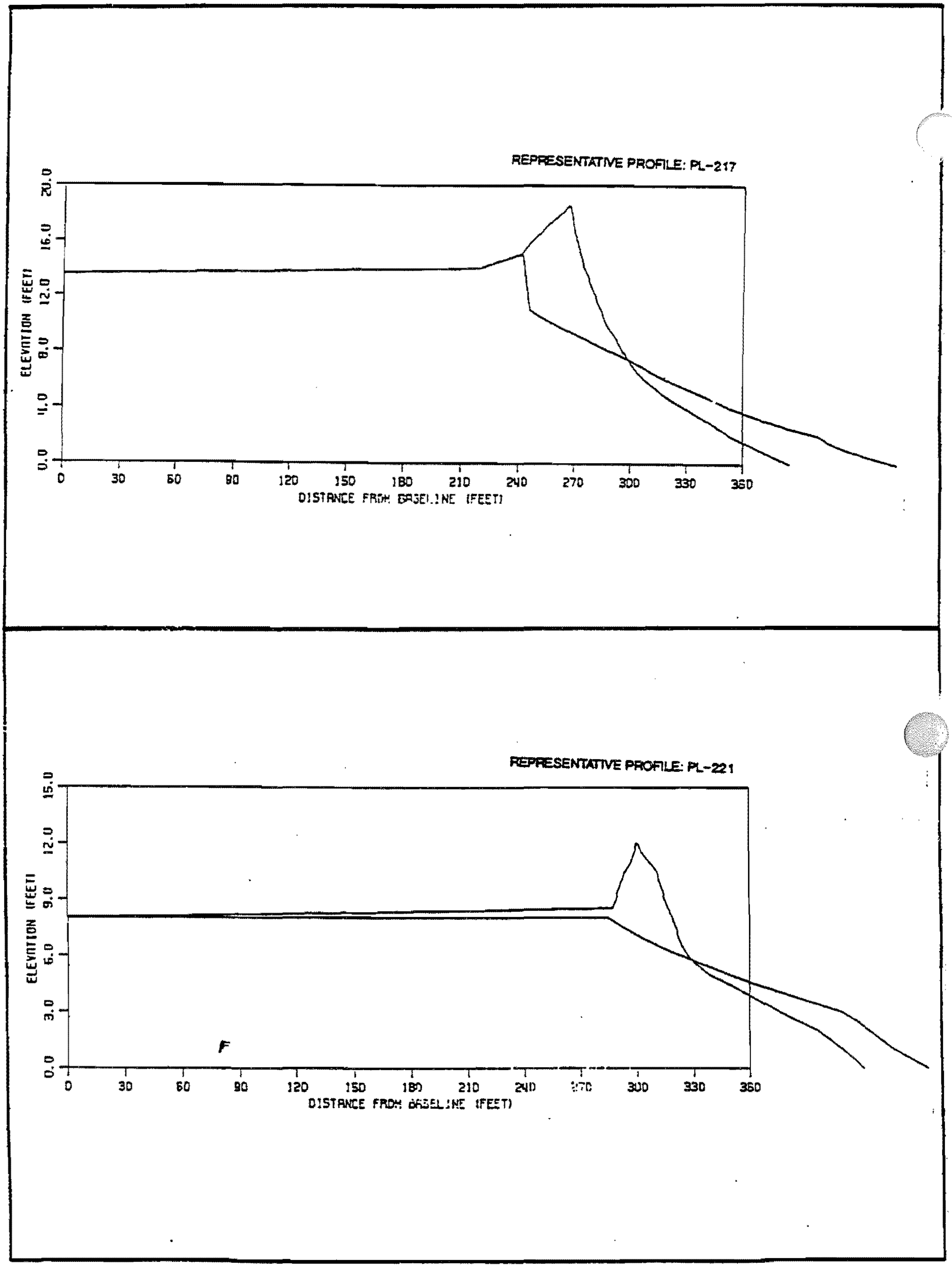


FIGURE A-13

TYPICAL POST-STORM PROFILES PREDICTED BY EDUNE

EXISTING CONDITIONS: 25 YEAR STORM

TABLE A-13
PREDICTED STORM RECESSION EXTENT
EXISTING CONDITIONS

STORM RETURN INTERVAL (YEARS)	PREDICTED RECESSION EXTENT (FEET)		REMAINING DUNE ELEVATION (FEET, NGVD)	
	PL - 217	PL - 221	PL - 217	PL - 221
2	126	136	19.4	12.9
5	139	156	17.9	9.4
10	146	158	17.4	8.9
25	156	160	15.9	8.9
50	167	170	14.9	8.9
100	200	181	14.4	8.9
200	216	192	14.4	8.9

NOTES:

RECESSION EXTENT REPRESENTS A MEASURE BETWEEN THE EXISTING 0' NGVD SHORELINE

AND THE LANDWARD EROSION LIMIT

EDUNE MODEL USED FOR PREDICTION

200 YEAR PL-221 DATA EXTRAPOLATED

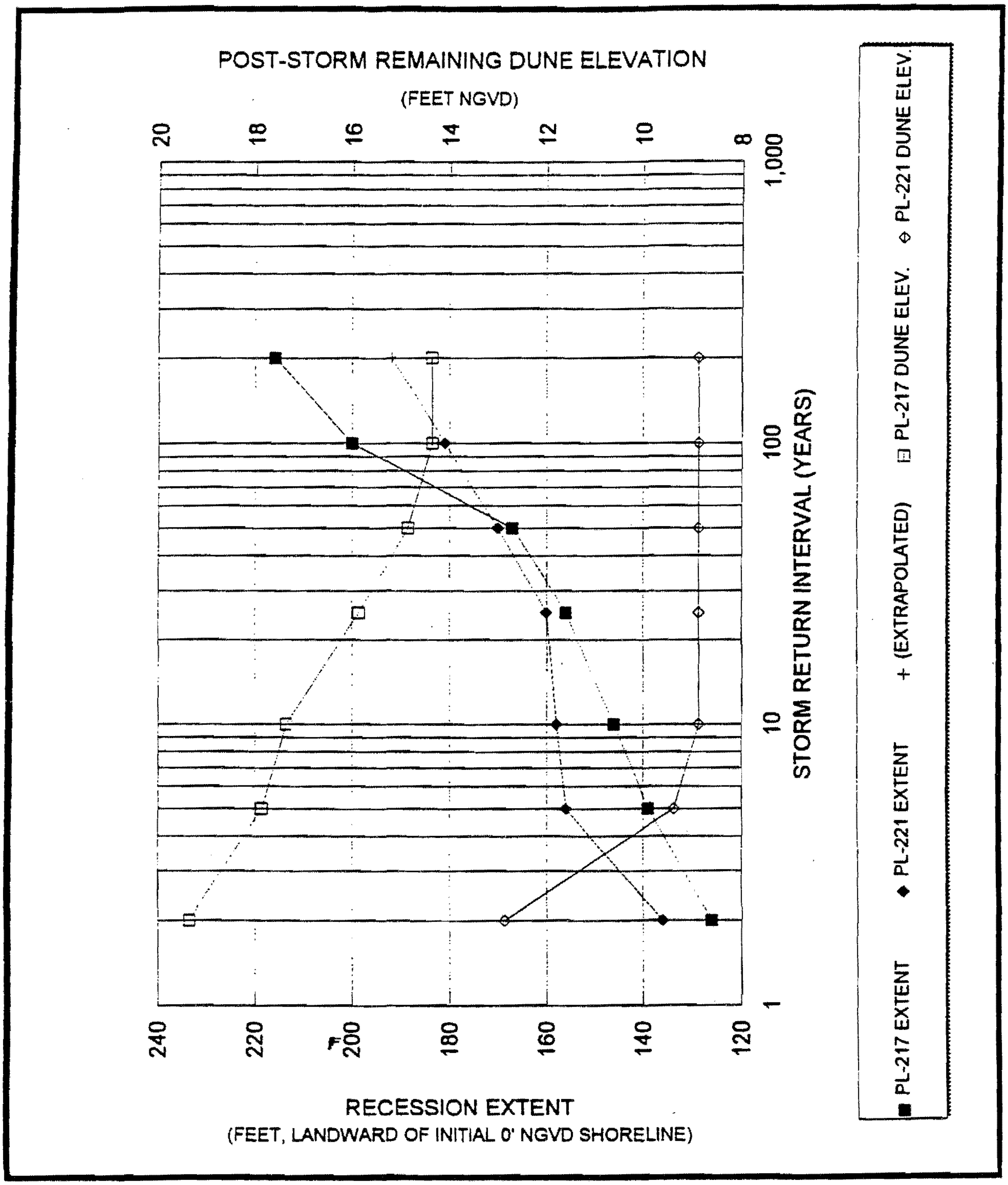


FIGURE A-14

PREDICTED STORM RECESSION ON EXISTING BERM
PORT MONMOUTH, NEW JERSEY

REPRESENTATIVE PROFILES (PL-217 AND PL-221) SURVEYED 11/95

A-145. This runup prediction model is based on flood dikes rather than complex beach systems. Port Monmouth, by contrast, is a low, narrow land mass which is not immediately backed by higher land. Without the higher elevations behind the beach, the composite slope method outlined in the *Shore Protection Manual* is considered inappropriate. Rather, the beach slope is evaluated using the guidance of the runup model. The beach slope value tan α, used is representative of onshore slopes measured from post-storm profiles output by EDUNE. As runup elevation is an input parameter for the EDUNE model, for each representative profile several iterations were required in order to result in a post-storm profile whose slope (input in the runup equation) resulted in a predicted runup value that (used as input in EDUNE) yielded a post-storm profile consistent with that above. The resulting post-storm slopes used as runup input were 1:20.9 and 1:32.9 for PL-217 and PL-221, respectively.

A-146. Wave and surge characteristics were equivalent to values described above. Finally, it is assumed that $T_p = T_s/0.9$, where T_s is the significant wave period.

A-147. Resulting runup predictions are shown in Table A-14. Wave runup elevations range from 0.9 to 2.7 feet above the still water level (SWL). Accounting for the storm surge associated with the storms, runup elevations are predicted to reach as high as 17.4 feet NGVD. Due to a steeper post-storm profile, runup elevations are marginally higher on Pl-217 than on PL-221. These predictions indicate that under certain storm conditions the wave runup at Port Monmouth can reach elevations that will overtop much of the existing dune.

Wave Overtopping

A-148. The primary source of storm induced coastal damage in Port Monmouth is flooding which has resulted primarily from creek overflow. In addition to creek overflow, the potential exists for the high storm surge associated with severe events to result in overtopping of the existing dune, due to its low elevation. Dune overtopping potentials are magnified by dune damage and lowering such as that predicted in the storm recession analysis of this study. The dune overtopping contribution to the total flood volumes could potentially be catastrophic due to storm surge levels in Raritan Bay.

A-149. As a result, it is important to quantify the overtopping potential. The resulting information calculated for the with project conditions could be used to conceive the design of coastal protection alternatives (dunes, seawalls, etc.), as well as to compare the alternatives. The overtopping evaluation for the without project conditions is less useful, as historic records exist which may be more accurate in evaluating the potential. The primary usefulness of the existing conditions overtopping evaluation will be in effectively calibrating the with project evaluation.

TABLE A-14

PREDICTED WAVE RUNUP AT REPRESENTATIVE PROFILES EXISTING CONDITIONS

STORM RETURN INTERVAL	WAVE RUNUP (SIGNIFICANT VALUES) (FEET)				STILL WATER LEVEL
	PL - 217		PL - 221		
	(ABOVE SWL)	(ABOVE NGVD)	(ABOVE SWL)	(ABOVE NGVD)	(NGVD)
2	1.3	7.8	0.9	7.4	6.5
5	1.8	9.5	1.3	8.9	7.6
10	2.0	10.4	1.4	9.8	8.4
25	2.1	11.7	1.4	11.0	9.6
50	2.1	12.6	1.5	12.0	10.5
100	2.6	14.8	1.8	14.0	12.2
200	2.7	17.4	1.9	16.6	14.7

NOTE:

CALCULATED ON POST-STOM PROFILES (AS PREDICTED BY THE EDUNE MODEL).
PREDICTED BY EQUATIONS OF PILARCZYK (1990)

A-150. Existing conditions wave overtopping values are based on the theory developed by Kobayashi, et al. (1996). The theory estimates the overtopping rate based on a number of normalized parameters used to characterize the profile dimensions and wave characteristics. The theory was selected after comparison with results derived by applying alternative theories developed by Pilarczyk (1990) and the USACE (ACES).

A-151. The theory is applied to the post-storm profiles resulting from the storm recession study of the design profiles at PL-217 and PL-221. The post-storm profiles are used as overtopping input because EDUNE results indicate that most dune lowering occurs early in a storm. The storm surge elevation (with respect to the dune elevation) and wave characteristics represent peak storm conditions. The improved conditions overtopping results are presented in Table A-15. Because wave overtopping models fail when the dune is completely overwashed, and because of the storm-induced dune lowering, many of the severe storms cannot be applied in the model.

Without Project Future Conditions

A-152. The without project future conditions at Port Monmouth are identified as: 1) continuing erosion of the shoreline, 2) impacts from future storm episodes, and 3) increased inundation potential due to dune loss.

A-153. Current trends indicate that ongoing erosion is likely to result in significant shoreline loss. The current shoreline trend (1988-1995) indicates an average loss of -2.7 feet annually. Long-term shoreline projections based on this value are compared with the current shoreline location in Table A-16, based on a 2002 construction date of the proposed project.

A-154. In addition to ongoing erosion, discrete storm episodes are anticipated to have a severe impact on the study area. As future storm induced recession episodes can be expected to occur along a shoreline that has already receded due to long term processes, the recession extent (as measured from the 1995 surveyed conditions) will be landward of the values indicated in Table A-13. Whereas natural processes can restore most or all of the losses resulting from storms, damage calculations must be made on the most extreme location of the recession extent, however temporary. Table A-16 and Figure A-15 indicate the estimated total future recession extent without the project, which is the EDUNE-predicted recession extent translated by the anticipated shoreline recession. The loss of this dune would result in increased inundation potential in the community of Port Monmouth.

A-155. Tidal flooding is expected to increase in severity in direct relation to the anticipated rise in relative sea level. With the loss of bayfront protective measures and the continuation of sea level rise, inundation damage can be expected to increase and occur more frequently in the future.

TABLE A-15
PREDICTED WAVE OVERTOPPING AT REPRESENTATIVE PROFILES
EXISTING CONDITIONS

STORM RETURN INTERVAL	WAVE OVERTOPPING RATE PER LINEAR FOOT ((C.F./S.)/L.F.)		TOTAL OVERTOPPING ((C.F./S.)
	PL-217	PL-221	
2	0.00	0.00	0
5	0.00	0.00	0
10	0.00	0.04	126
25	0.00	(1)	(1)
50	0.00	(1)	(1)
100	0.00	(1)	(1)
200	(1)	(1)	(1)

NOTE:

CALCULATED ON POST-STOM PROFILES (AS PREDICTED BY THE EDUNE MODEL).
PREDICTED BY EQUATIONS OF KOBAYASHI, et al. (1996).
TOTAL OVERTOPPING CALCULATED ON 6,000 FEET OF STUDY AREA SHORELINE LENGTH.
(1): THEORY INVALID WHEN SURGE ELEVATION EXCEEDS DUNE ELEVATION

F

TABLE A-16
PREDICTED WITHOUT PROJECT FUTURE TOTAL RECESSION MEASURED FROM THE 1995 0' NGVD SHORELINE

DATE (1)	LONG TERM RECESSION (FEET) (2)	PREDICTED RECESSION EXTENT CAUSED BY THE FOLLOWING STORM RETURN INTERVAL (FEET) (3)						
		2	5	10	25	50	100	200
2002	19	150	166	171	177	187	209	223
2012	46	177	193	198	204	214	236	250
2022	73	204	220	225	231	241	263	277
2032	100	231	247	252	258	268	290	304
2042	127	258	274	279	285	295	317	331
2052	154	285	301	306	312	322	344	358

NOTES:

(1) Future conditions based on anticipated 2002 construction date.

(2) Based on an assumed shoreline loss rate of 2.7 feet per year

(3) Long term recession and EDUNE predicted storm recession based on PL-217 and PL-221 average.

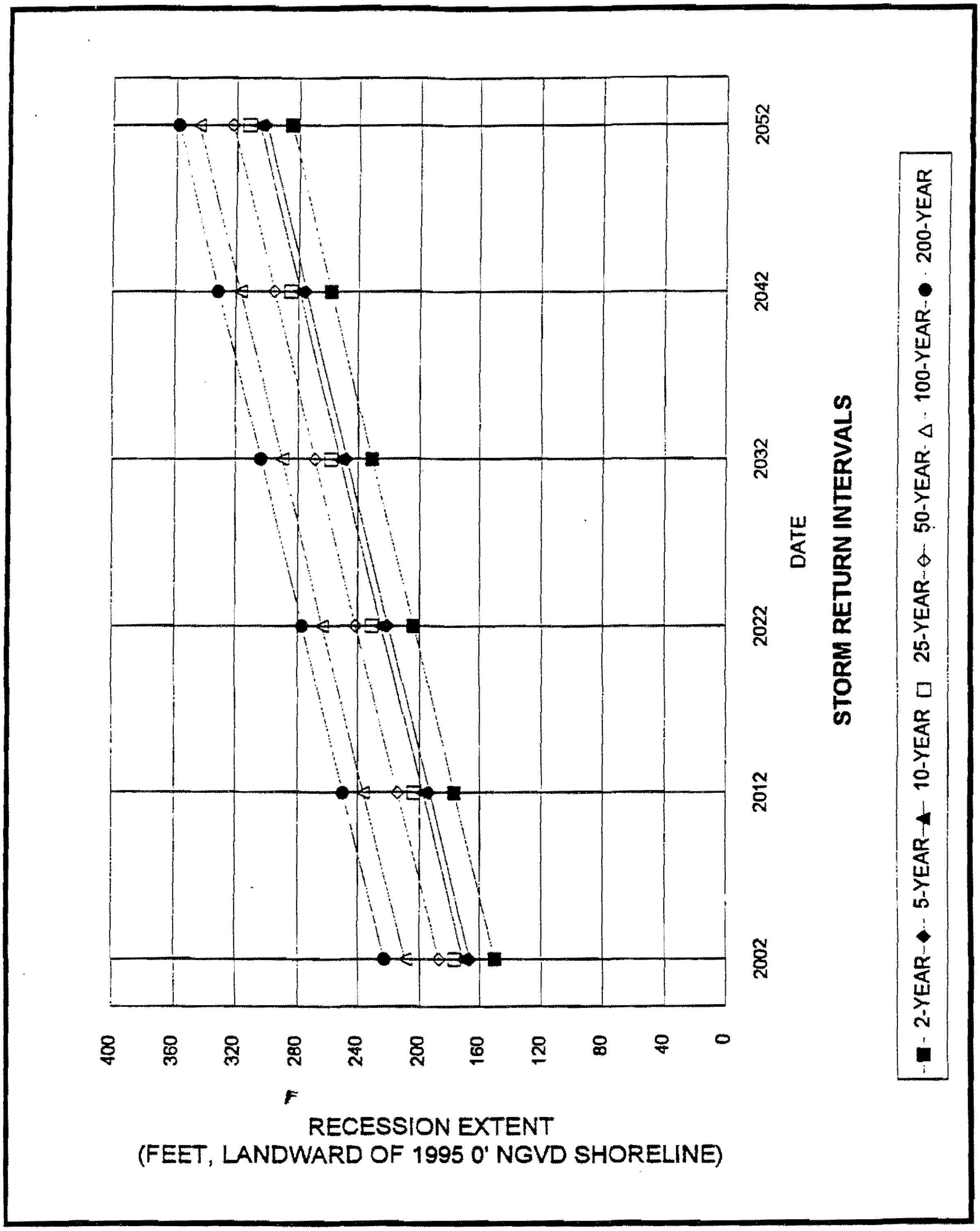


FIGURE A-15

PREDICTED TOTAL LONG TERM AND STORM INDUCED RECESSION WITHOUT PROJECT FUTURE CONDITIONS

A-156. Recently completed roadway and bridge improvements by Monmouth County will significantly improve access to the Port Monmouth area and partially address the frequent flooding of emergency access routes.

A-157. Based on plans presented in the Bayshore Waterfront Access Plan, additional recreation facilities are expected to be developed. In conjunction with this plan, a cluster of homes along Port Monmouth Road near Pews Creek will be removed. Potential activities are stated as:

> *Nature interpretation, boating, saltwater swimming, sunbathing, educational program in cooperation with fishing industry, wetlands preservation, active recreation.*

A-158. Finally, there are currently plans for a major mixed use development on the bayshore near the Belford fish co-op. Since the preliminary plans indicate structures will be located above the 100-year tide elevation, protection for this area was not considered.

DEVELOPMENT ALTERNATIVES

Description of the 1993 Reconnaissance Plan

A-159. The reconnaissance investigation led to the recommendation of a cost-effective plan for storm damage reduction and shore protection for further study. The coastal protection plan was developed to extend from Pews Creek approximately 1 mile east to Church Street, and featured a 50 ft. wide sand berm at elevation +5 ft. NGVD backed by a dune with a 40 ft. wide crest at elevation +15 NGVD. Along the marshes adjacent to both Pews and Compton Creeks, a total of 10,000 feet of levees were included, featuring a 10 ft. crest width at elevation +13 ft. NGVD. Suitable interior drainage structures were included.

Design Criteria

Design Storm

A-160. The feasibility level analysis focuses on the comparison of alternatives to provide flood protection against a storm with a 1% chance of being exceeded in any year (a 100 year storm). Based on modeling conducted at the Coastal Engineering Research Center (CERC), the flood stage associated with such a probability is 12.2 ft. NGVD. In order to ensure that a structural solution will reliably protect against a 100 year flood, the top of levee and floodwall structures has been set at 14.0 ft. NGVD, one foot above the mean 100 year flood elevation plus the 50 year anticipated rise in the sea level. The top of the bayshore dune has been set at 16 ft. NGVD, providing additional elevation to protect against wave runup, overtopping and sea level rise.

A-161. In order to optimize the project design, various levels of protection were applied in order to determine the cost and resulting benefits of a variety of design alternatives. One application investigated was designed to provide protection against a 25 year storm event. The design alternatives resulting from this lower level of protection feature a peak elevation of 13 ft. NGVD on the levees and floodwalls, and 15 ft. on the dune. Another application investigated was the "no flood potential" design, which was intended to guard against the upper confidence limit of all dynamics of the 100 year level storm. The design alternatives resulting from this higher level of protection feature a peak elevation of 15.2 ft. NGVD on the levees and floodwalls, and 17 ft. on the dune

A-162. Another storm event is considered to quantify the survivability of the advance fill. Since this material is merely intended to remain (relatively) stable during a 10 year renourishment interval, it would be less likely to encounter such a major storm. Therefore, the stability of the advance fill (only) is quantified by applying 15-year storm conditions, as during a 10 year period, the likelihood of encountering a storm of 15-year or greater intensity is 50%.

Beach and Dune Sections

A-163. Protection along the project's bay shoreline considered a dune/berm system with periodic nourishment and/or structures to stabilize the design. The design was developed based primarily on hurricane storm damage reduction features, and secondly on flood control features. Each alternative was sized to provide an approximately 100-year level of protection including allowances for anticipated sea level rise, wave runup and overtopping.

A-164. A preliminary dune crest width of 40 feet was selected for alternative development. This width was selected based on the adjacent project design constructed at Keansburg. A preliminary design dune elevation of 16 ft. NGVD was selected. This is similar to the average existing dune elevation of 17.0 ft. NGVD. By contrast, the 100-year storm surge with wave setup elevation is +13.2 ft. NGVD, and the peak wave runup for the design storm is approximately 2.2 ft. (existing conditions). The 16 ft. NGVD dune elevation would allow an initial safety margin of 0.6 ft., decreasing to approximately 0.0 ft. at the end of the 50-year project life due to sea level rise. A landward dune slope of 1 vertical on 5 horizontal was selected. A flatter slope of 1 vertical on 15 horizontal was selected for the seaward face of the dune. The dune section would be stabilized with dune grass and fencing. Dune vegetation would be protected from pedestrian damage by wood overwalks.

A-165. In order to preserve the integrity of the protective dune, a beach cross-section seaward of the dune was developed for preliminary select alternatives. The beach berm elevation of 5 ft. NGVD and width of 50 ft. were selected. A nearshore beach slope of 1 vertical on 15 horizontal was selected from the seaward edge of the +5 ft. berm to the intersection with the existing bottom (generally at -3 ft. NGVD). The dune and beach cross-section used for preliminary investigation is presented in Figure A-16.

Levee/Floodwall Sections

A-166. The earthen levee and floodwall design was developed in accordance with the published standards of the Office of the Chief of Engineers. The top of levee is 14.0 ft. NGVD. An impervious core material (cutoff wall) is recommended for the entire length of the levees. The impervious core will extend from the top of the levee to approximately five (5) feet below grade to prevent seepage through and under the levee. The levee top widths are 10.0 ft. in accordance with levee design standards. The proposed preliminary levee side slopes of 1V on 2.5H were selected due to stability concerns.

A-167. Floodwalls are vertically driven sheet pile I-type walls or timber pile foundation T-type floodwalls. The top of the floodwalls are at elevation 14.0 ft. NGVD. The floodwall top widths are 1.25 ft. for I-type walls and 1.5 ft. for T-type walls in accordance with floodwall design standards. The sheet pile extend approximately 5 to 10 feet below grade to control under seepage through and under the floodwall.

Closure Gates

A-168. The closure gates included in the alternatives use the design of gate type closures at the roadway crossings. These gates provide a 40 ft. opening. Gate closure structures were selected because of their functionality and ease of opening and closing. The closure gate design was developed in accordance with published standards of the Office of the Chief of Engineers.

A-169. A storm closure gate was designed for the levee crossing of Pews Creek (Alternate P1). When a flood event is imminent, the gate is to be closed and a bypass pump will divert Pews Creek flow into the Sandy Hook Bay. The upstream low-lying area will be utilized to capture and store the upland runoff. The bypass pumping elevations are discussed in detail in Appendix F-Interior Drainage. The design criteria for the pump station was in accordance with published standards of the Office of the Chief Engineer.

Interior Flood Control

A-170. Included with the selected alternative is an interior flood control design. Various means of alleviating flood damages as a result of interior runoff are considered. These include drainage ditches, "natural" ponding areas and drainage structures through the levees. The drainage structures, which consist of 18' RCP preliminary pipes through the levee with a flap gate and sluice gate, will allow for the flow of the interior stormwater runoff through the proposed levees. During high flood stages in Sandy Hook Bay the

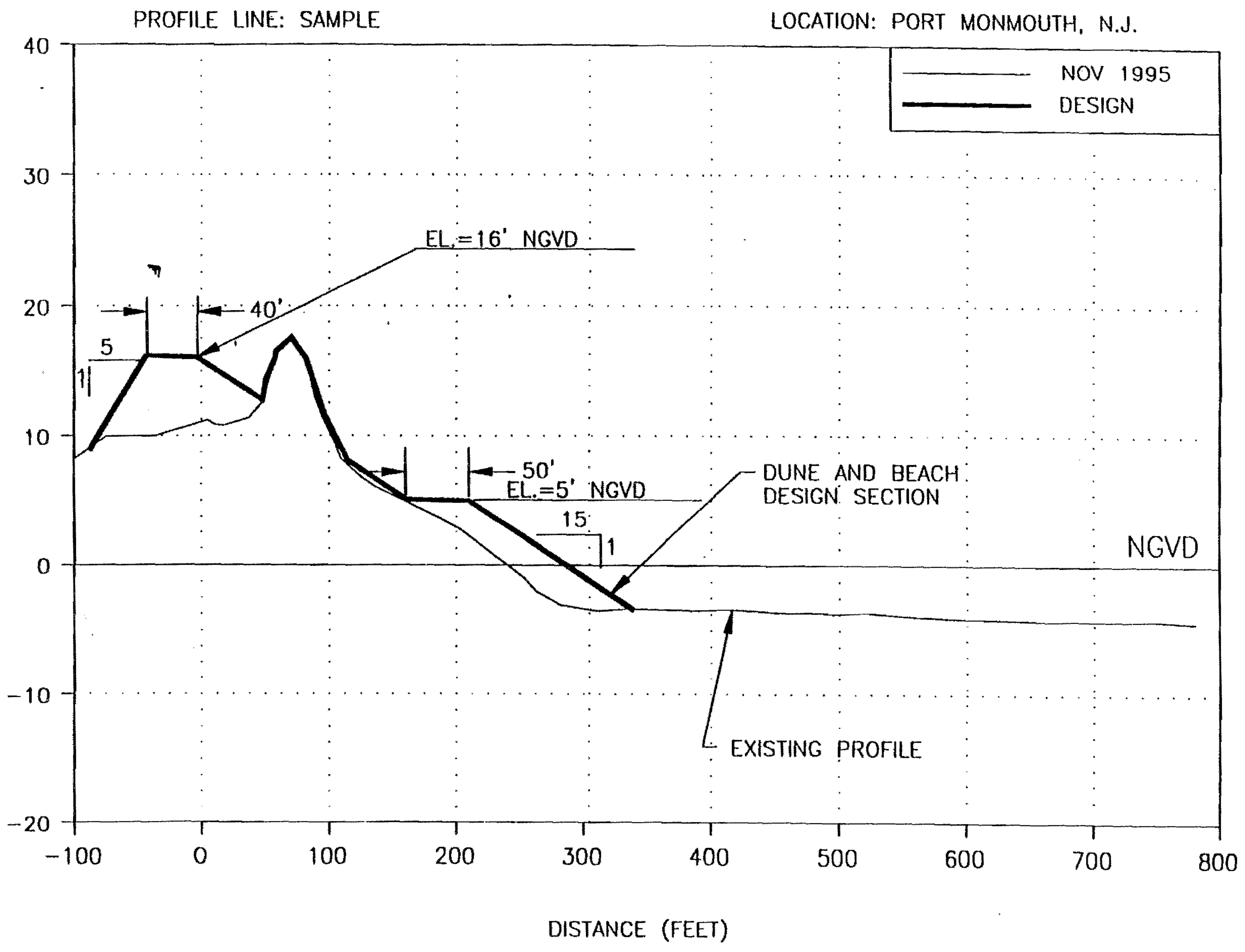


FIGURE A-16

TYPICAL BEACH & DUNE CROSS-SECTION

(BASED ON RECONNAISSANCE STUDY, USACE, 1993)

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interior stormwater runoff will be stored in the "natural" ponding areas. Minimal selective regrading may be required for the "natural" ponding areas to insure positive drainage.

A-171. Provisions will be made to handle runoff from all existing storm drains within the protected areas using a combination of the natural ponding areas behind the levees and the proposed drainage structures which penetrate through the levees. Continuous swales and ditches draining to the new drainage structures will be constructed where necessary to provide positive interior drainage during flood periods. The design interior runoff will be discharged into the creeks by gravity.

Non-Structural Features

A-172. The non-structural alternatives provide flood protection on a building-by-building basis. In some cases, providing flood protection to a handful of flood prone buildings is more cost effective than providing flood protection for entire reaches, where the majority of buildings are not prone to flood damage. The non-structural plan considered floodproofing, raising, ringwall, and buyouts as an alternative to structural features. There are a number of methods that can be used to protect a property, a building, and its contents from flood. The options that are available are dependent on a number of things such as the depth of the flood; the type of building; the presence of a basement or a crawl space; soil conditions; and the layout of a property. Flood protection measures range from very radical ones to those which require minimal physical changes. Flood protection measures considered for design of the selected plan include the following:

- Buyout - evacuating buildings from the floodplain;
- Raising - elevating the structure;
- Ringwalls - constructing various types of barriers to stop floodwaters from entering a building;
- Floodproofing - using techniques known as "dry floodproofing"; and/or, "wet floodproofing" where major utilities are protected while allowing the basement to flood.

A-173. The selection of protection methods for individual structures relies heavily on information provided in "Quantity and Cost Curves for Flood Control Measures, Passaic River Basin," December 1980. Since detailed assessments have not been performed for individual buildings, the proposed methods and costs are conceptual in nature.

Preliminary Alternative Design Layouts

A-174. During the preliminary analysis presented in this report, four different approaches to providing flood and storm protection were developed for each of the three project components (Bay Shorefront, Pews Creek and Compton Creek). When combined, these protection features represent a total of 64 structural alternative plans. In addition, three composite non-structural protection plans were developed. Thus, a total of 67 alternative plans for protection for Port Monmouth can be evaluated. The combination of shorefront alignment 1(S1) with Pews Creek alignment 4 (P4) and Compton Creek alignment 4 (C4) is comparable to the plan recommended at the Reconnaissance level updated to reflect new survey and hydraulic model results.

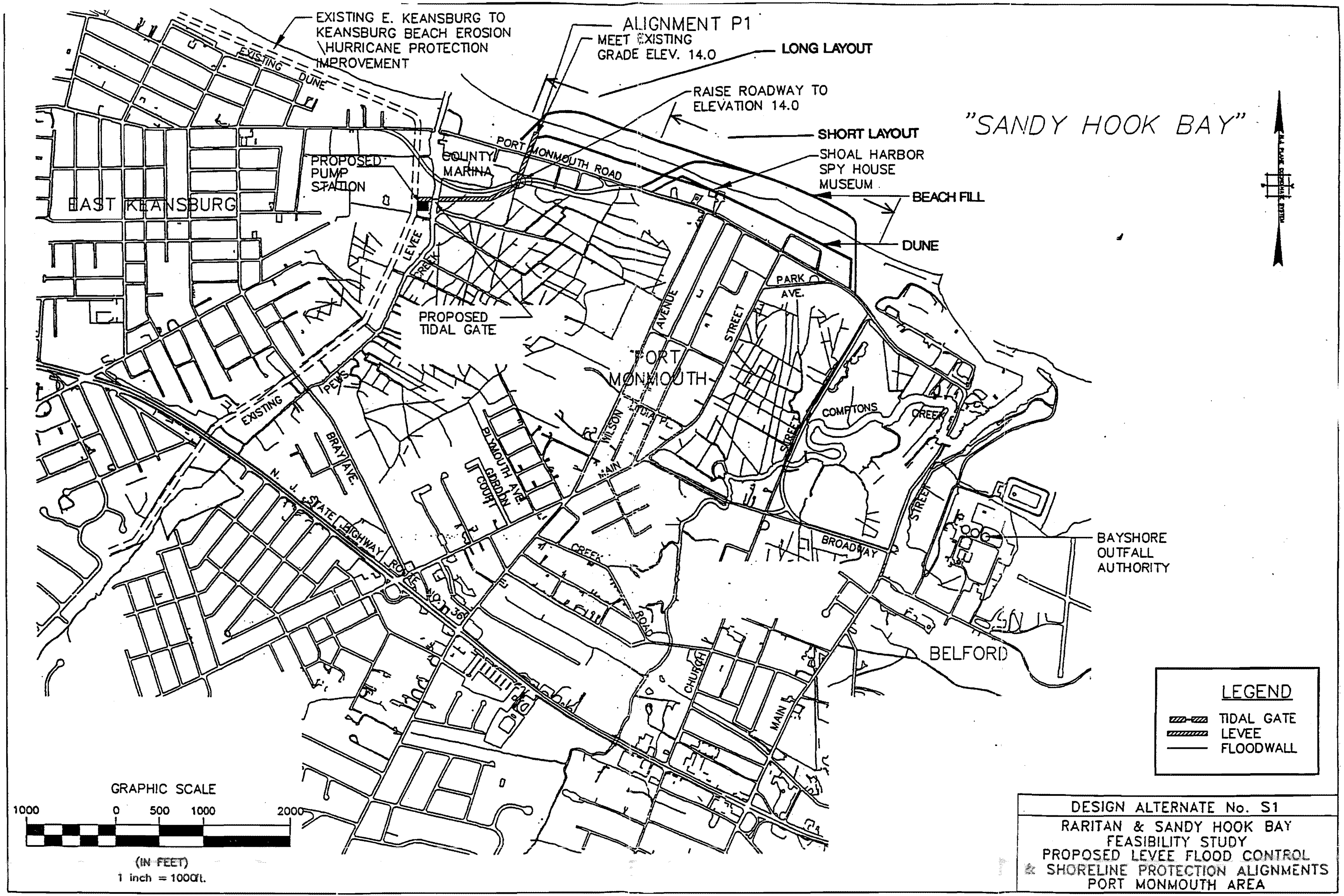
Shorefront Layouts

Alternative S1

A-175. Description. Alternative S1 considers construction of the beach and dune section with periodic nourishment. The eastern limit of the fill would be near the intersection of Park Avenue and Port Monmouth Road (profile PL-219) and would tie in with the Compton Creek levee alignments. The western limit of the fill would be approximately 1,200 ft. east of Pews Creek (profile PL-212) to tie into the Pews Creek levee alignment P1 for a total berm and dune length of 3,700 ft, not including taper sections. (long layout). The western limit of the fill section would be approximately 700 ft. west of Wilson Avenue (profile PL-214) for Pews Creek alignments 2 through 4 for a total length of 2,500 ft. (short layout, see Figure A-18). The newly constructed dune would be stabilized by vegetation. Three overwalks would be constructed for the longer layout and two for the shorter layout to minimize dune damage.

A-176. Cost. Construction of the 16-ft. NGVD dune and 5-ft. NGVD beach section would be accomplished by utilizing fill from an upland borrow source. The initial beach and dune fill volume is estimated at 136,900 cubic yards for the Pews Creek P1 levee alignment and 95,600 cubic yards for all other levee alignments. Advance nourishment for a 10-year period would be included in the initial project requiring 54,000 cubic yards for alignment P1 and 52,000 cubic yards for all other alignments. Subsequent nourishments would be accomplished by trucking material from an upland source every 2 years, requiring an estimated 10,600 cubic yards. The first cost is estimated at $3,721,000 for the longer fill project and $3,026,000 for the shorter project. Annual costs are $348,000 and $294,000 for the long short layouts, respectively.

A-177. Impact. The primary impact of Alternative S1 is the creation of a larger dune cross-section and dune footprint. The beachface will be widened but not beyond historical dimensions. Beach vegetation would be impacted during construction. Sand transport along the shoreline is expected to continue at rates nearly equal to historic rates. The amount of sand transported into Pews and Compton Creek is expected to remain unchanged.



DESIGN ALTERNATE No. S1

RARITAN & SANDY HOOK BAY
FEASIBILITY STUDY
PROPOSED LEVEE FLOOD CONTROL
& SHORELINE PROTECTION ALIGNMENTS
PORT MONMOUTH AREA

FIGURE A-18

Alternative S2

A-178. Description. The beach and dune section and layout would be identical to Alternative S1. However, to reduce erosion rates the beach fill would be stabilized by the construction of rock groins at the east and west limits of the fill (Figure A-19). The structures are estimated to reduce renourishment requirements by approximately 50%. The two terminal structures would be rubblemound groins. The groins would extend from the base of the dune to 100 ft. seaward of the toe of fill (Figure A-20).

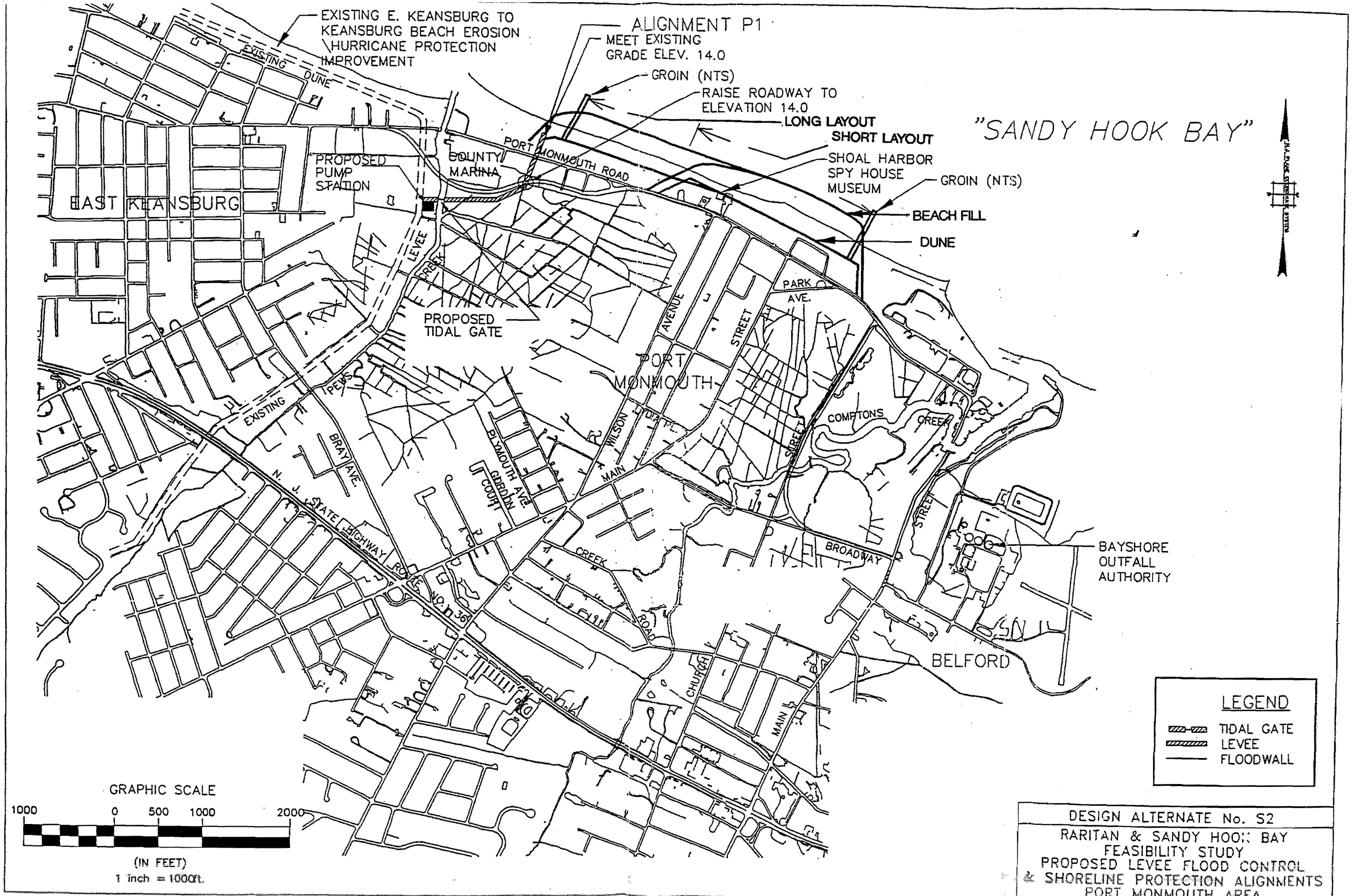
A-179. Cost. The beach and dune fill volume for the initial construction would be the same as Alternative S1. The advance fill quantity placed in the initial project would be reduced to 27,000 cubic yards for the Pews Creek levee alignment P1 and 26,000 cubic yards for all other alignments over a 10-year nourishment period, based on anticipated reduced erosion rates. Subsequently, nourishment operations would be based on trucking 5,300 cubic yards once every 2 years. The first cost is estimated at $3,745,000 for the longer project and $3,065,000 for the shorter project. Average annual costs are $327,000 and $274,000 for the long and short plans, respectively.

A-180. Impact. The primary impact of Alternative S2 is the creation of a larger beach and dune cross-section. Beach vegetation would be impacted during construction. The proposed terminal groins will help stabilize the fill and reduce beach erosion. This will result in a reduced amount of sand transported east into Compton Creek and west into Pews Creek.

Alternative S3

A-181. Description. Alternative S3 considers construction of a vertical concrete floodwall to protect the bayfront shoreline which would tie into the adjacent flood control structures. (Figure A-21). The structure would be constructed of steel sheet piles. The elevation of the floodwall would be set equal to the dune elevation detailed above. The wall would limit shoreward movement of the shoreline and minimize overtopping of floodwaters. A small beach would be maintained between the wall and the shoreline for seawall toe protection and recreational purposes. The existing shoreline would be allowed to recede to a 50 ft. design beach width at an elevation of +5 ft. NGVD fronting the seawall. Advanced fill would not be required during the initial construction project. As the shoreline retreated and approached a new equilibrium shoreline configuration, periodic nourishment would be required. Future nourishment would be accomplished by trucking an estimated 10,600 cubic yards of fill semi-annually as required.

A-182. Cost. An estimated 3,900 ft. of floodwall would be required for the Pews Creek levee alignment P1. Approximately 2,700 ft. of structure would be required for the other alignments. Based on average shoreline retreat rates, periodic nourishment would be required starting 25 years after the initial project. The first cost is estimated at $6,610,000



DESIGN ALTERNATE No. S2

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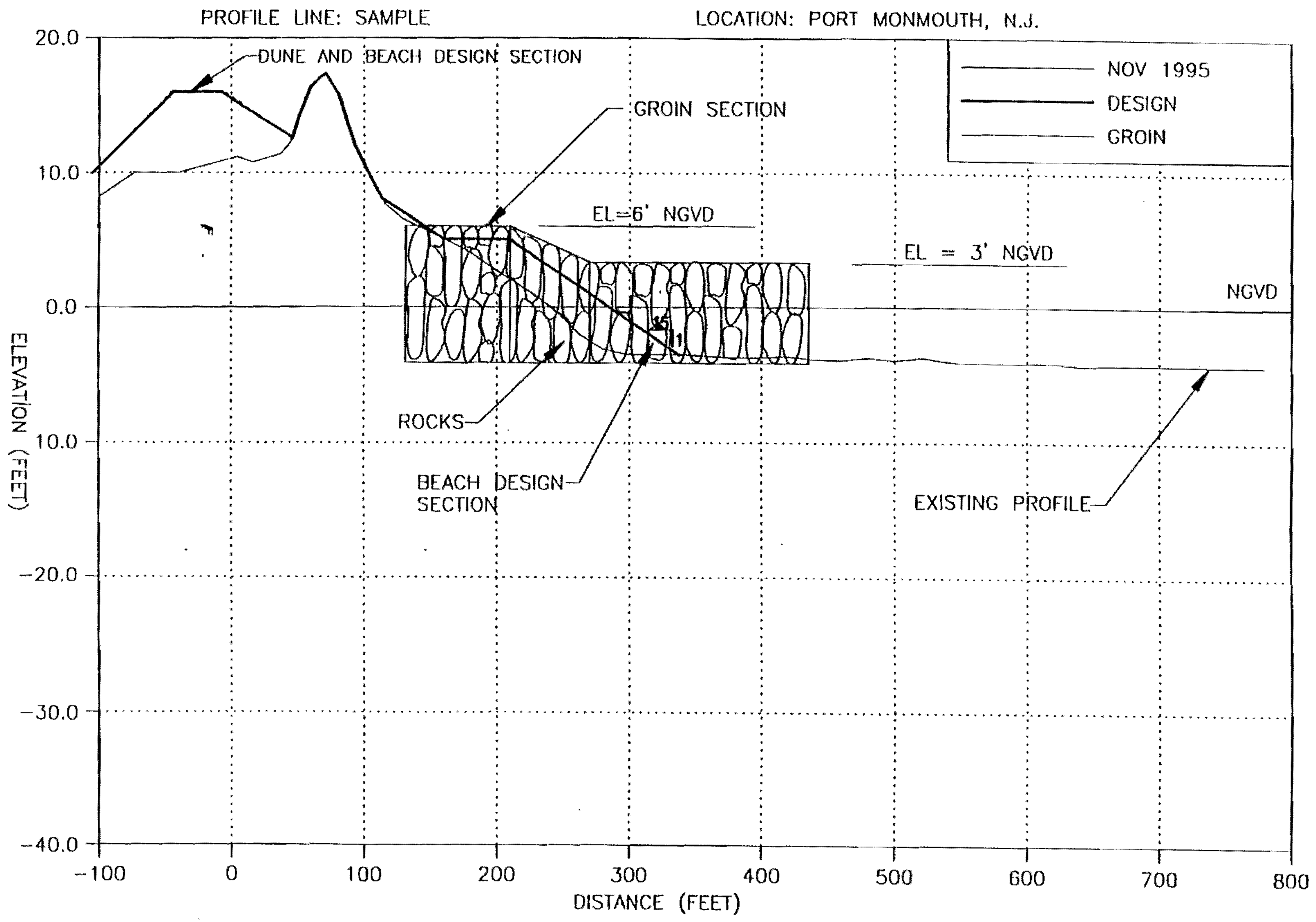
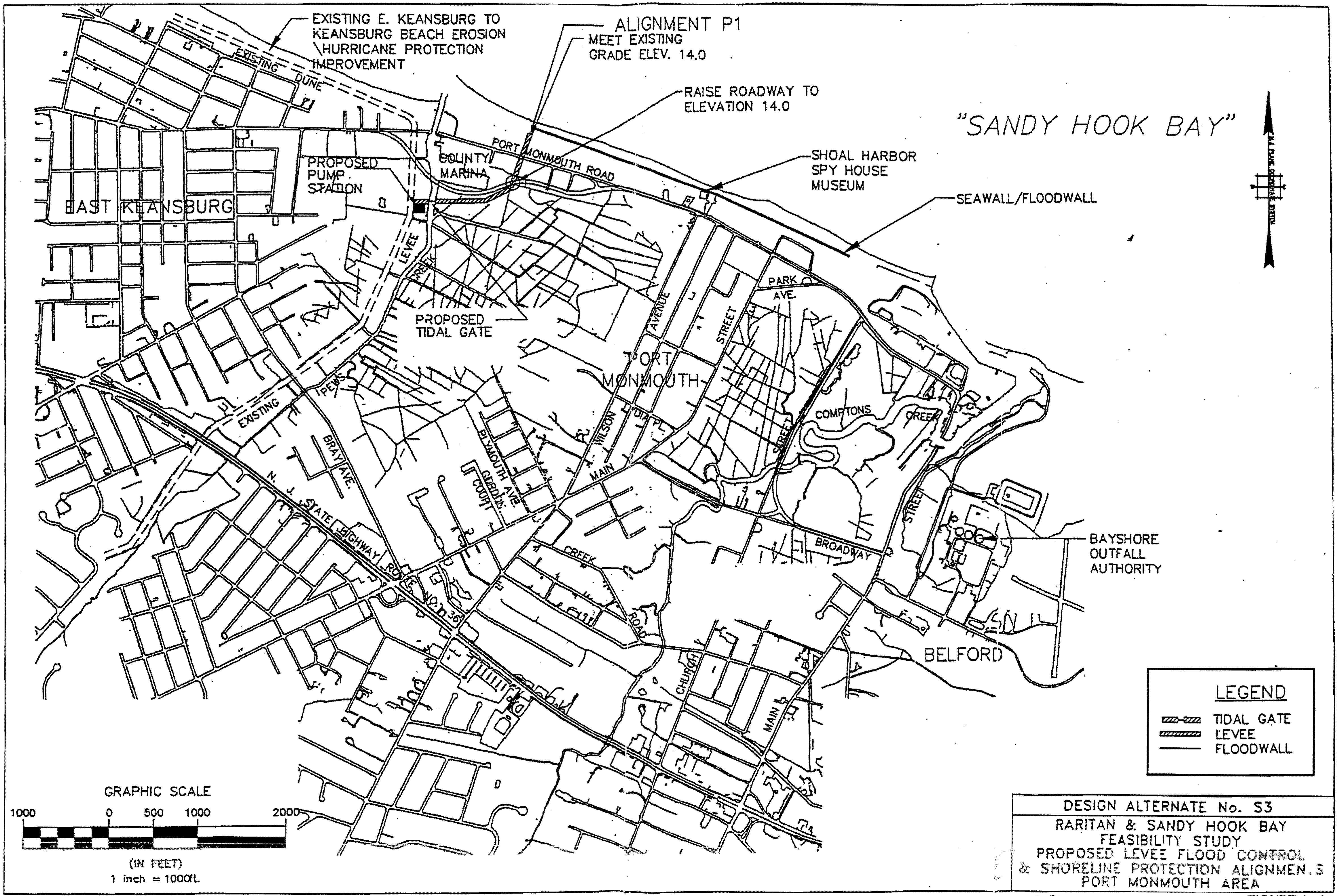
FIGURE A-19



TYPICAL GROIN CROSS-SECTION

FIGURE A-20



DESIGN ALTERNATE No. S3

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FIGURE A-21

for the longer project and $4,575,000 for the shorter project. The annual costs are $612,000 and $441,000 for the long and short layouts, respectively.

A-183. Impact. The alternative would allow loss of beach and dune areas. Construction impact would be the least of the considered shorefront alternatives. Alongshore sand transport to the adjacent inlets should be unaffected.

Alternative S4

A-184. Description. This alternative would take maximum advantage of previous real estate purchases along the bayfront shoreline. The proposed new +16 ft. NGVD dune would be reconstructed at a landward location except from the Spy House to the fishing pier. Coastal erosion protection and flood control at this historical site would be accomplished through the construction of a steel sheet pile seawall featuring the same elevation as the dune (Figure A-22). Flood control would be accomplished by the construction of a steel floodwall. The existing dune would not be removed, but would be allowed to erode in future years as long-term shoreline erosion continues. The new dune would be constructed far enough inland so that it would not be affected by long-term shoreline retreat prior to renourishment.

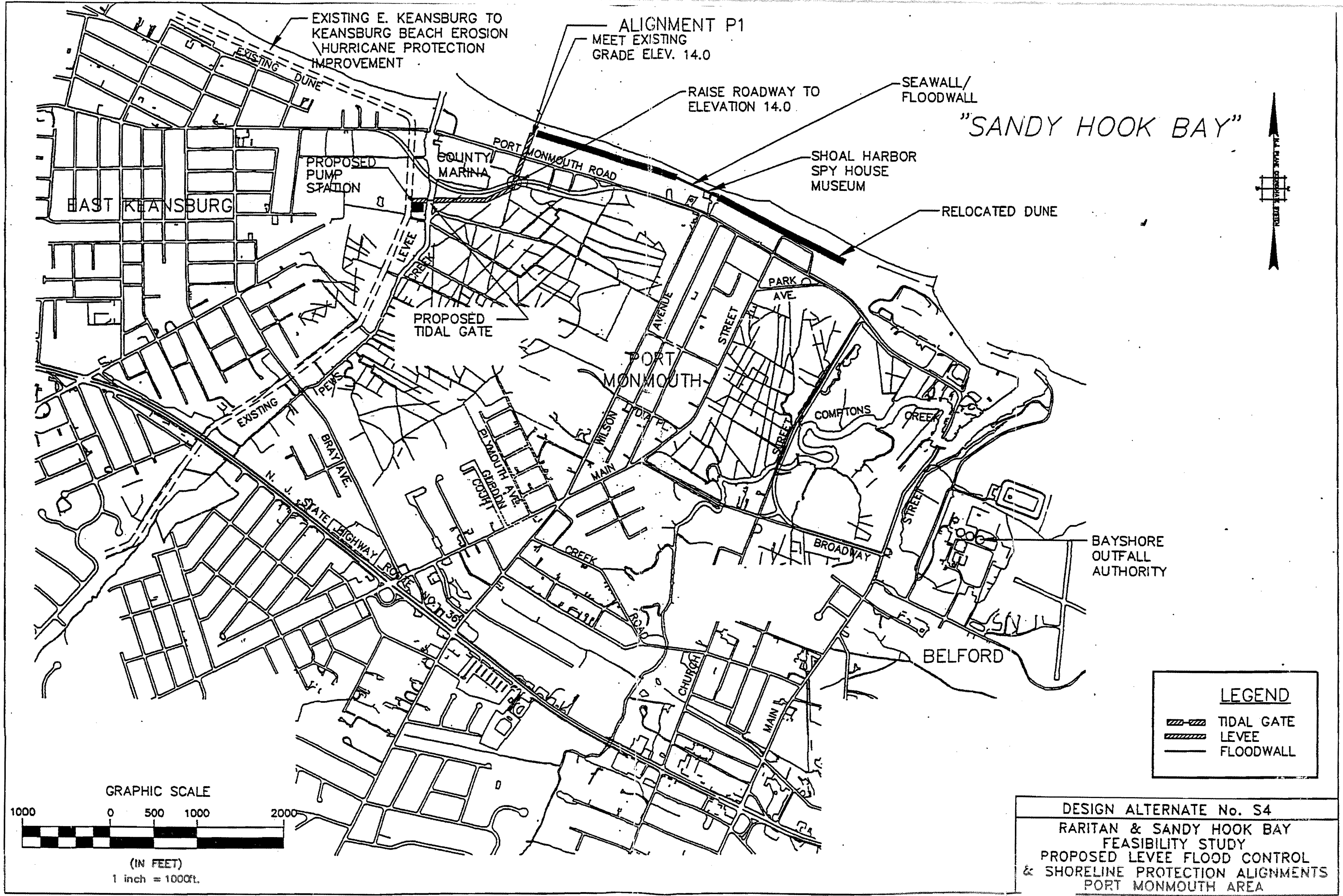
A-185. Cost. The rock revetment and steel floodwall would be approximately 600 ft. long. Approximately 1,800 to 3,100 ft. of new dune would be constructed depending on the final levee alignment at Pews Creek. The initial dune construction volume is estimated at 79,400 cubic yards for the longer fill plan and 31,400 cubic yards for the shorter layout. The preliminary first cost estimate for the longer layout is $3,076,000; $2,465,000 for the shorter plan. Preliminary annual cost estimates are $322,000 and $275,000 for the long and short projects, respectively.

A-186. Impacts. The alternative will create a new and larger dune cross-section. Dune vegetation would be impacted during construction. Beach erosion and sand transport into the adjacent inlets should be unaffected initially and the rate should decrease with time.

Pews Creek Layouts

Alignment P1

A-187. Description. Alignment P1, shown on Figure A-23, consists of an earth levee from the Keansburg Beach Erosion/Hurricane Protection Improvement Levee, southwest of the Monmouth Cove Marina, and proceeds east towards Pews Creek where a tidal gate and pump station which will span the creek to control flooding from tidal as well as fluvial flows. A combination of levee and floodwall continues from the crossing of the creek and proceeds easterly for about 700 ft. The alignment continues northerly crossing Port Monmouth Road. Port Monmouth Road will be raised where it intersects the proposed



DESIGN ALTERNATE No. S4

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FIGURE A-22

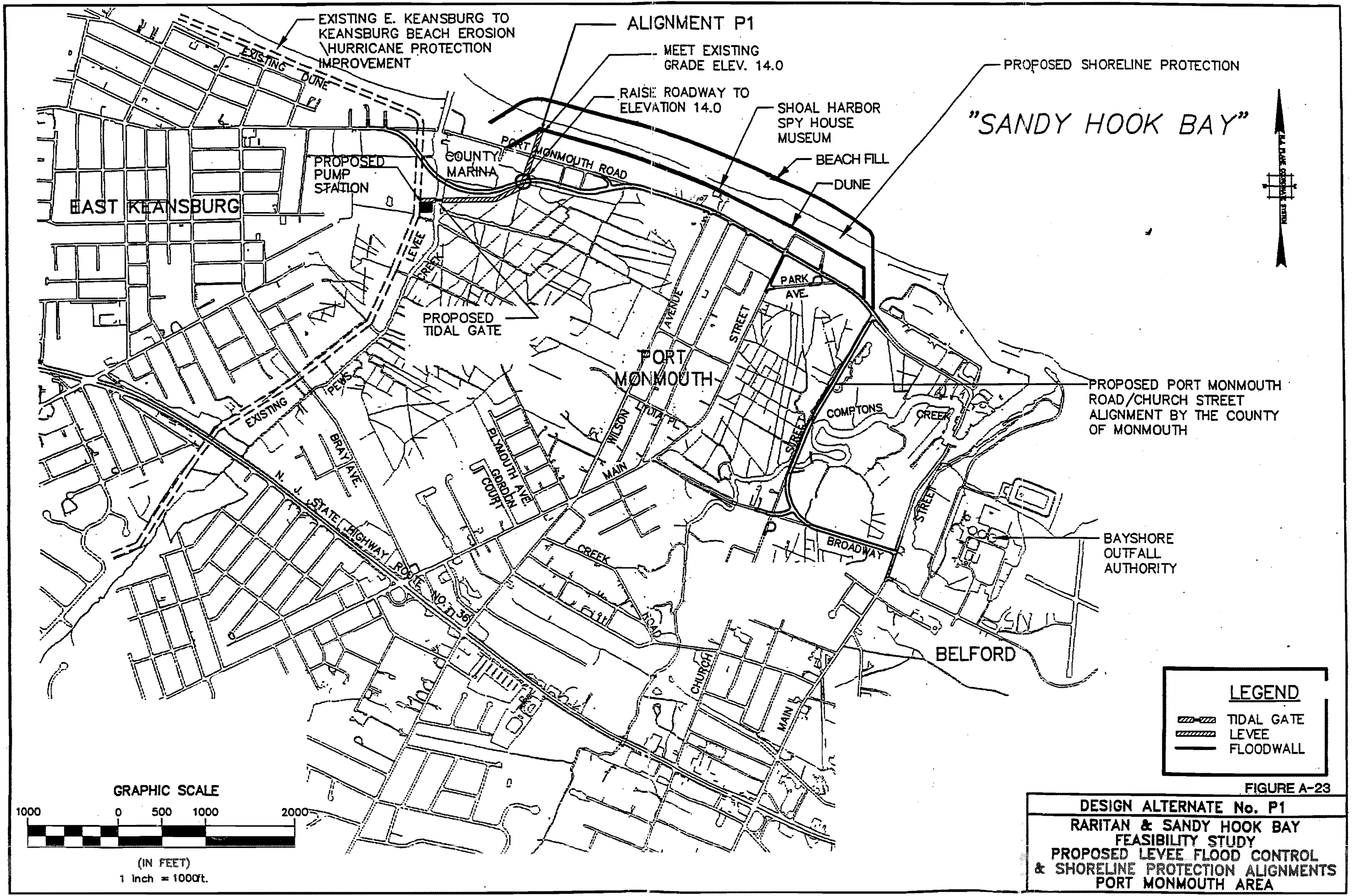

FIGURE A-23

DESIGN ALTERNATE No. P1

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levee in order to maintain a 14.0 ft. NGVD level of protection. The levee finally terminates at the dune along Sandy Hook Bay. A section of the Keansburg Beach Erosion/ Hurricane Protection Improvement Levee between the P1 alignment connection and the dunes along Sandy Hook Bay will be raised to achieve a 14.0 ft. NGVD design elevation in order to provide closure along the entire length of improvement.

A-188. This alignment consists of approximately 1,900 ft. of levee, a tidal gate and pump station. This alignment configuration will have minimal impact to existing wetland areas because the tidal gates will be employed only in cases of extreme high tides. Water levels landward of the tidal gates may reach critical heights due to a combination of upland fluvial flows and prolonged closure of the tidal gates.

A-189. A preliminary 500 cfs pump station was incorporated in the design in order to maintain water levels landward of the proposed tidal gates. The NED optimization verified that a 120 cfs pump station was most cost effective. A more detailed hydrologic design was later performed to finalize the proper pump size.

A-190. Costs. The preliminary cost for this alignment is approximately $5,327,000, including the tidal gates and pump station.

A-190a. Impacts. The levee footprint totals about 2 acres of impacted wetlands. This alternative also requires a tidal gate for which variable openings must be investigated to ensure no indirect effects to wetland hydrology.

Alternative P2

A-191. Description. Alignment P2 consists of an earth levee, much of which is located in the wetland area. As seen in Figure A-24, the alignment proceeds along the outer perimeter of the wetlands areas, thereby minimizing the interruption of tidal flow to the majority of the wetlands. The alignment starts at the 14.0 NGVD elevation near the intersection of Bray Avenue and Main Street and then proceeds in a northwest direction. The alignment continues along the eastern edge of the wetland area and ties into the 14.0 NGVD elevation of the dunes located along Sandy Hook Bay. A section of the Port Monmouth Road profile must be raised to elevation 14.0 NGVD at the proposed levee intersection. The levee is approximately 6,600 ft. long, but because it runs through tidal wetlands for a majority of its length, 24.78 acres of wetlands will be impacted. The area of wetlands which will be permanently disturbed due to the footprint of the levee is approximately 8.33 acres.

A-192. Cost. The first cost for this alignment is estimated to be $6,984,000.

A-193. Impact. The levee footprint totals 8.33 acres of impacted wetlands.

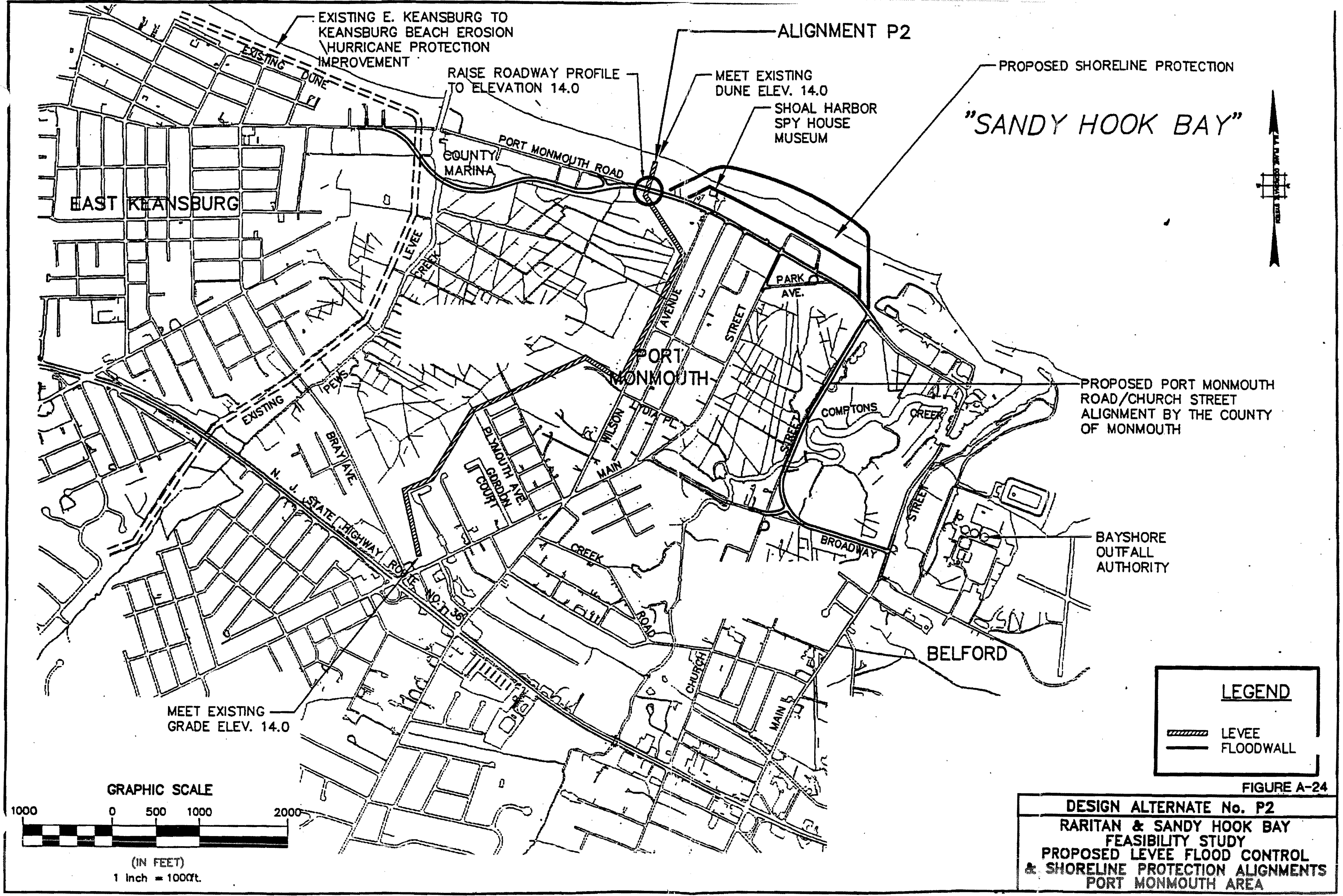

FIGURE A-24

DESIGN ALTERNATE No. P2

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PORT MONMOUTH AREA

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A-83

Alternative P3

A-194. Description. Alignment P3 consists of an earth levee and sheet pile floodwall with a concrete cap. The major design objective of this alternative was to avoid encroaching into the wetlands to minimize impacts. This alternative starts at the 14.0 NGVD elevation near the intersection of Bray Avenue and Main Street and proceeds northerly towards the wetlands area as a levee. (Figure A-25). The alignment continues in a northerly direction as a floodwall at the wetlands limit boundary. The flood wall alignment then proceeds northeasterly to the western terminus of Gordon Court where it continues to a levee. Near the western terminus of Plymouth Avenue the alignment continues as a floodwall and follows the wetland limit line. Approximately 10 properties on Lydia Place will be acquired as the alignment crosses Lydia Place. The alignment then changes to a levee between Lydia Place and Renfrew Place along the western side of Wilson Avenue where another eight properties will be acquired. The alignment then reverts back to a floodwall and proceeds northerly along the western edge of Wilson Avenue for about 1,300 ft. where it turns west along the upland side of the wetland limit line for another 200 ft. The alignment then changes to a levee following the wetland limit line to Port Monmouth Road. A section of Port Monmouth Road will have to be raised to elevation 14.0 NGVD in order to accommodate the levee crossing. The alignment proceeds north and terminates at the dunes along Sandy Hook Bay.

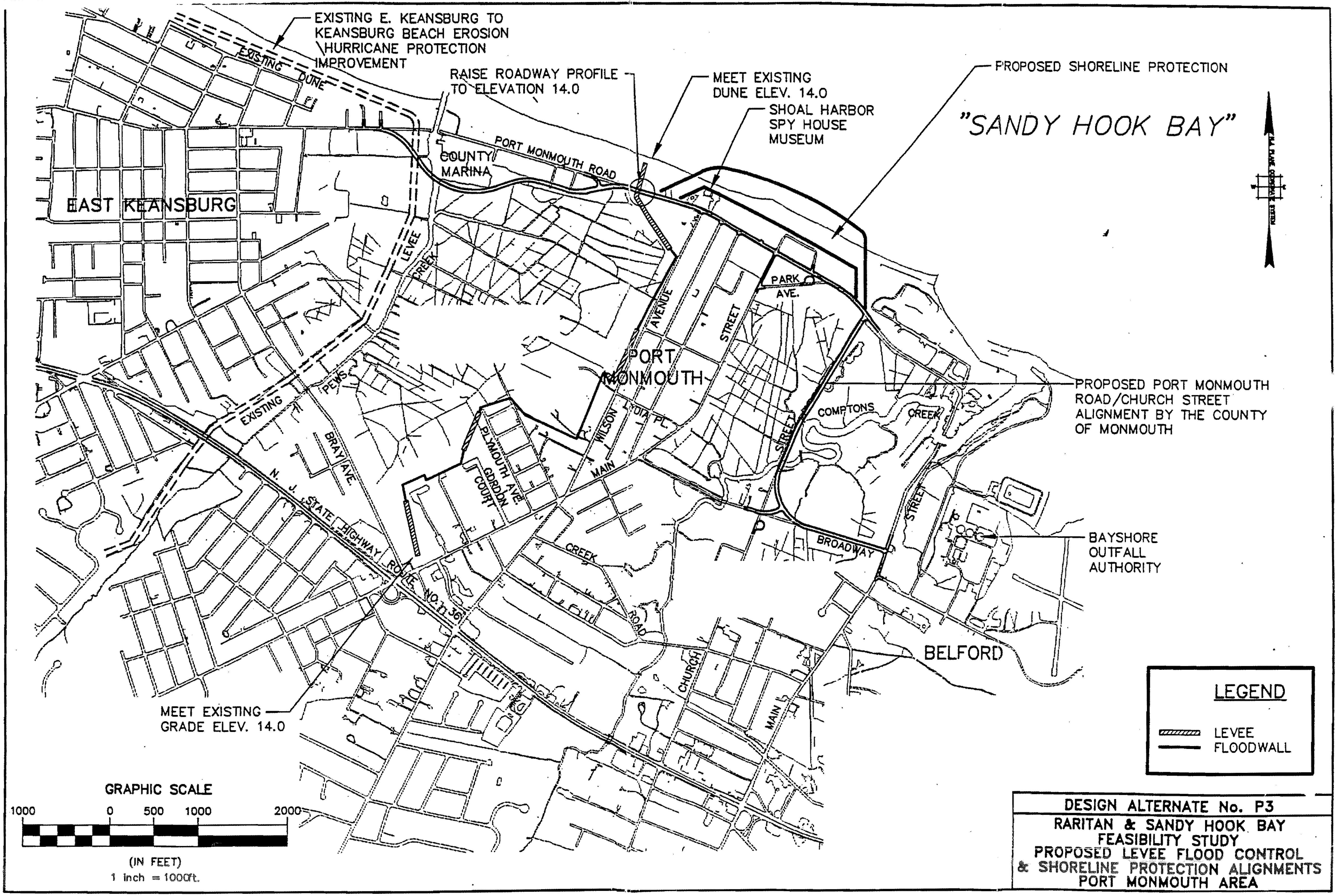
A-195. Costs. The cost for this alignment is $7,985,000.

A-196. Impacts. The alternative has no impact on wetlands.

Alternative P4

A-197. Description. Alignment P4 is a combination of alignments P2 and P3 which seeks to balance social and environmental impact. The alignment commences as a levee at the 14.0 NGVD elevation near the intersection of Bray Avenue and Main Street and proceeds in a northwesterly direction to the wetland limit line. (Figure A-26). Near the northern terminus of Shoal Harbor Court the alignment changes to a floodwall and follows the upland side of the wetland limit line in a northeasterly direction until it reaches the northern terminus of Gordon Court where it reverts back to a levee and proceeds in a northerly direction into the tidal wetlands. The route heads in a northerly direction along the eastern edge of the wetlands until it crosses Port Monmouth Road, a section of which will have to be raised in order to maintain the 14 ft. elevation for the alignment. The alignment then proceeds north for about 300 ft. and terminates at the dunes along Sandy Hook Bay.

A-198. Cost. The first cost for Alignment P4 is estimated to be $6,818,000.



DESIGN ALTERNATE No. P3

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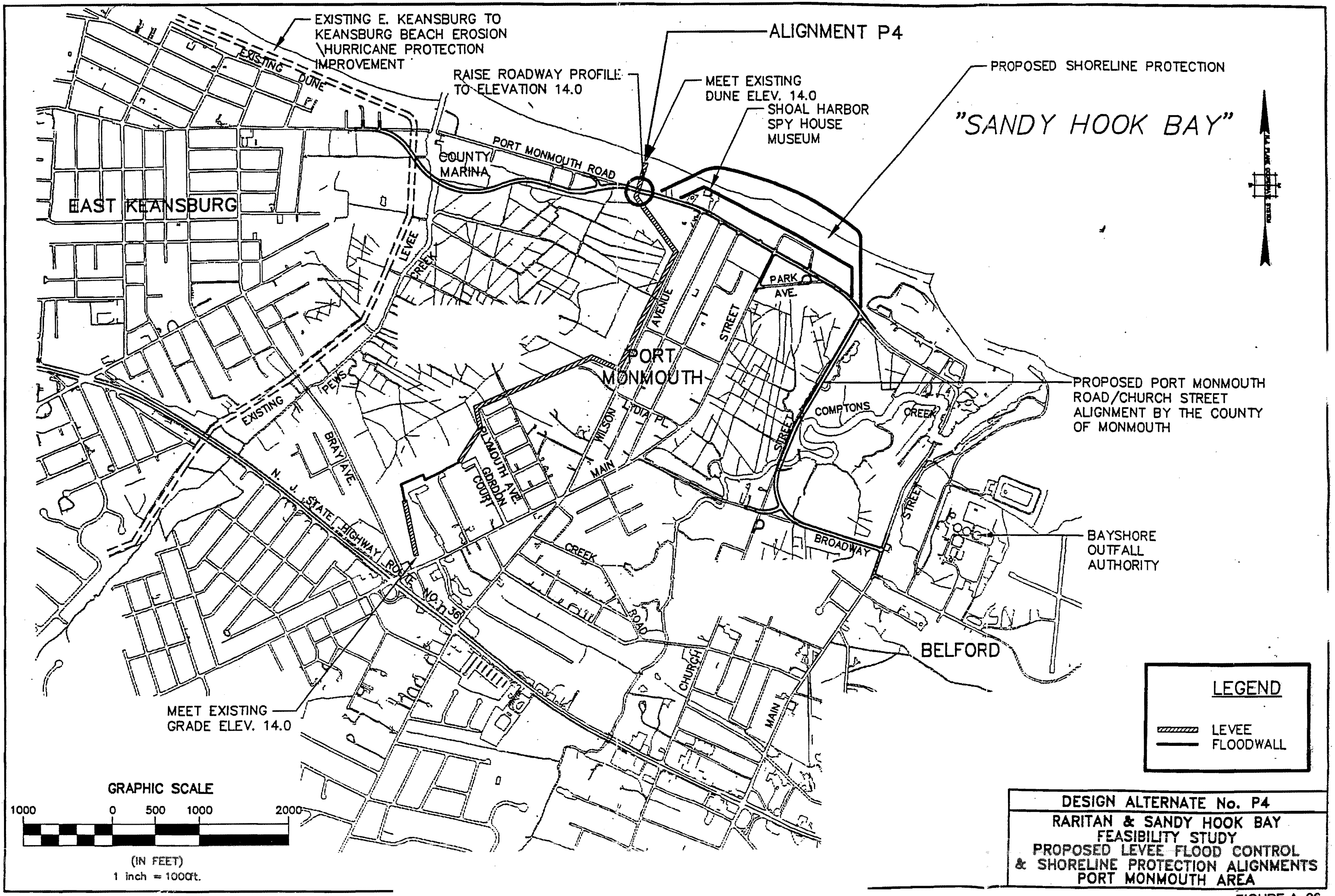
FIGURE A-25



DESIGN ALTERNATE No. P4

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FIGURE A-26

A-199. Impact. This option impacts 6.5 acres of wetland from the footprint of the floodwall/levee system.

Compton Creek Layouts

Alignment C1

A-200. Description. Alignment C1 is comprised of an earthen levee approximately 7,300 ft. long with two (2) closure gates along the roadway crossings (Figure A-27). This alignment is the shortest of the four alignment alternatives for Compton Creek and was developed using the most direct route of closure. The top of the levee is at elevation 14.0 NGVD. The levee commences at the intersection of Wilson Avenue and Route 36 and proceeds easterly along the properties on the south side of Willow Avenue. The levee section continues north and parallels the westerly side of Compton Creek, crosses Port Monmouth Road and terminates at the dunes along the northern limits of Port Monmouth along Sandy Hook Bay.

A-201. This alignment requires two closure gates, one across Campbell Road near the intersection of Creek Road and the other at Broadway where it intersects with Main Street. The alignment requires the roadway profile for Port Monmouth Road to be raised to elevation 14.0 NGVD where the levee alignment crosses. The alignment also requires a section of the Henry Hudson Trail be raised to elevation 14.0 ft. NGVD.

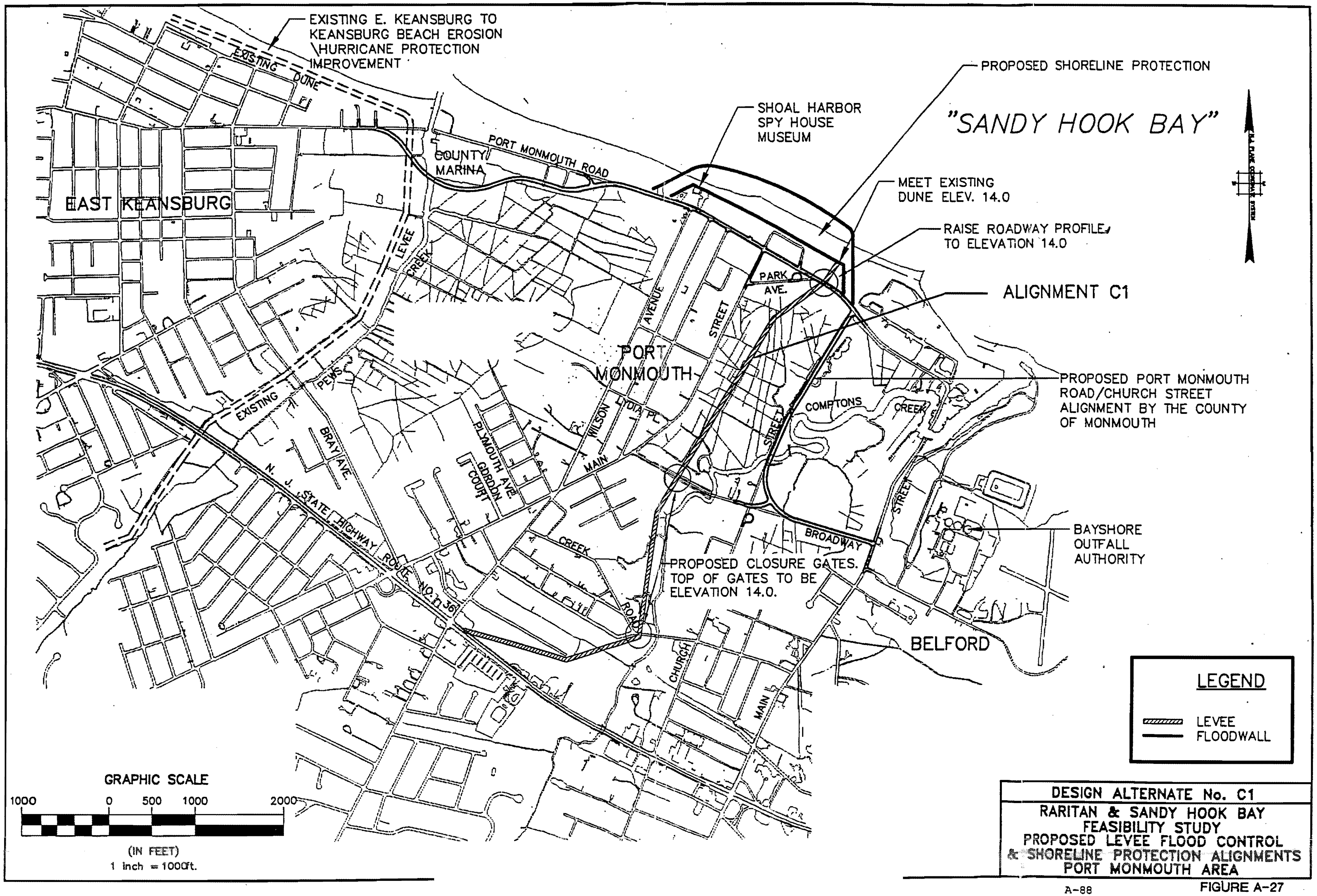
A-202. Costs. The total first cost of this alignment alternative is estimated to be $10,435,000.

A-203. Impacts. Approximately 10.07 acres of wetlands is impacted due to the levee footprint.

Alignment C2

A-204. Description. Alignment C2, as in levee Alignment C1, traverses tidal wetlands, however, is a more lengthy alternative than Alignment C1 with less impact to the wetlands. This alignment commences at elevation 14.0 NGVD near the intersection of Wilson Avenue and Route 36 and proceeds easterly along the properties on the south side of Willow Avenue. (Figure A-28). The levee proceeds northerly and follows the eastern edge of the developed (upland) area through the wetlands and eventually ties into the dunes along Sandy Hook Bay. This alternate levee alignment has an overall length of approximately 8,100 ft.

A-205. Alignment C2 requires the installation of two closure gates, one across Campbell Road near the intersection of Creek Road and the other at Broadway where it intersects with Main Street. This alignment requires a section of Port Monmouth Road to be raised



DESIGN ALTERNATE No. C1

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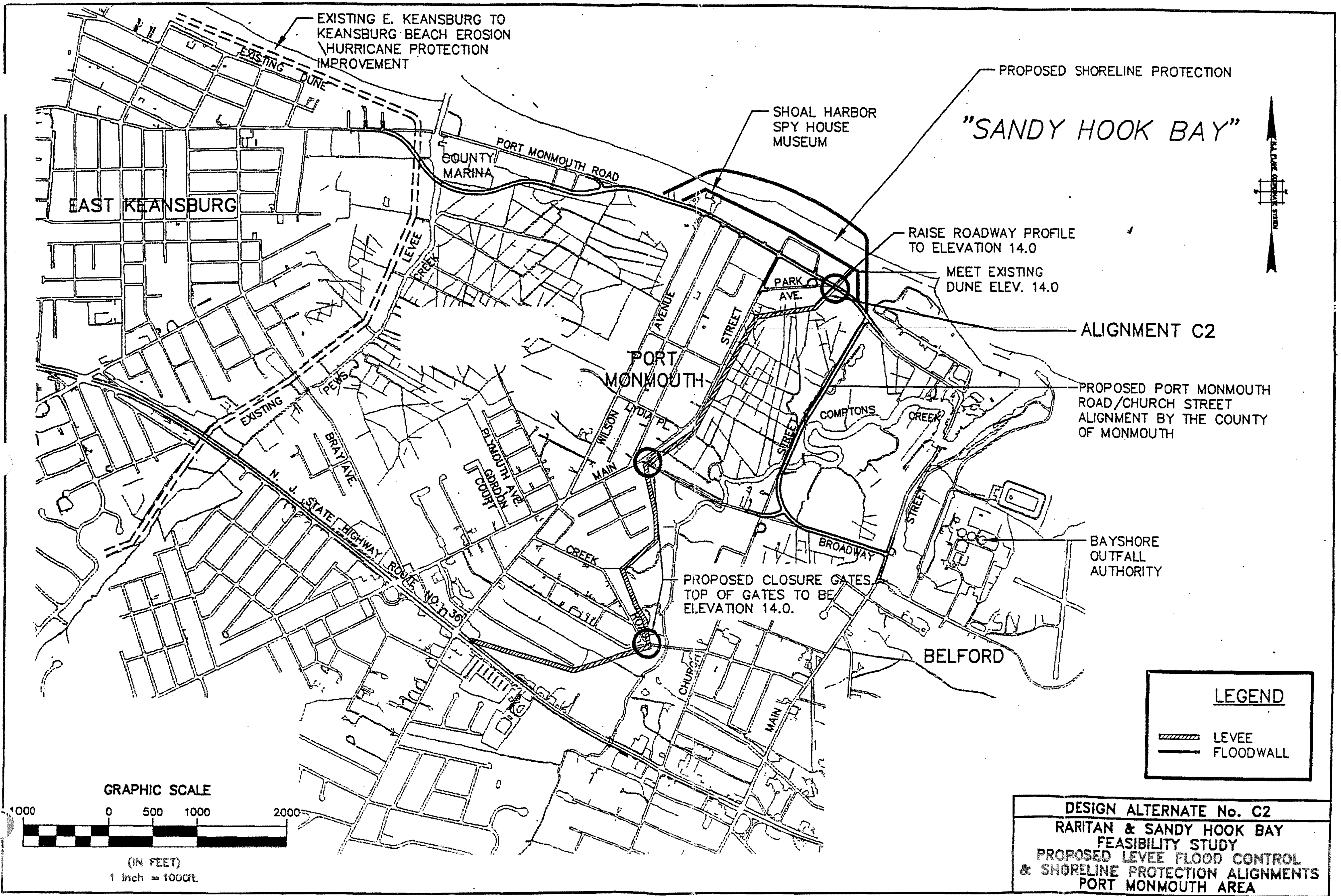
PORT MONMOUTH AREA

FIGURE A-27



DESIGN ALTERNATE No. C2

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to elevation 14.0 NGVD where the levee intersects it, as well as a section of the Henry Hudson Trail in order to accommodate the levee crossing.

A-206. Costs. The total first cost for this alignment alternative is $8,500,000.

A-207. Impacts. Alignment C2 is longer than the C1 levee alignment due to the fact that it conserves wetland area and thus has to follow a more circuitous path. Approximately 10.97 acres of wetlands is impacted due to the levee.

Alignment C3

A-208. Description. Alignment C3 is comprised of an earthen levee and floodwall with two closure gates. The main objective of this alternative was to avoid the wetland area and to minimize the impact to the environmentally sensitive areas. The levee alignment commences at the 14.0 ft. NGVD elevation near the intersection of Wilson Avenue and NJ State Highway Route 36 (Figure A-29). The levee alignment proceeds easterly along the rear yards of properties on the south side of Willow Avenue to the eastern terminus of Willow Avenue. The alignment proceeds northerly as a sheet pile floodwall with a concrete cap following the upland side of the wetlands to limit line. This alignment requires two closure gates, one across Campbell Road near the intersection of Creek Road and the other at Broadway where it intersects with Main Street. The alignment proceeds northerly as a floodwall along the eastern right-of-way of Main Street. The levee section continues from this point to Port Monmouth Road. Where the levee intersects with Port Monmouth Road, the road will have to be raised to elevation 14.0 NGVD. The alignment terminates at the dunes along Sandy Hook Bay. Approximately 30 parcels would require temporary or permanent easements.

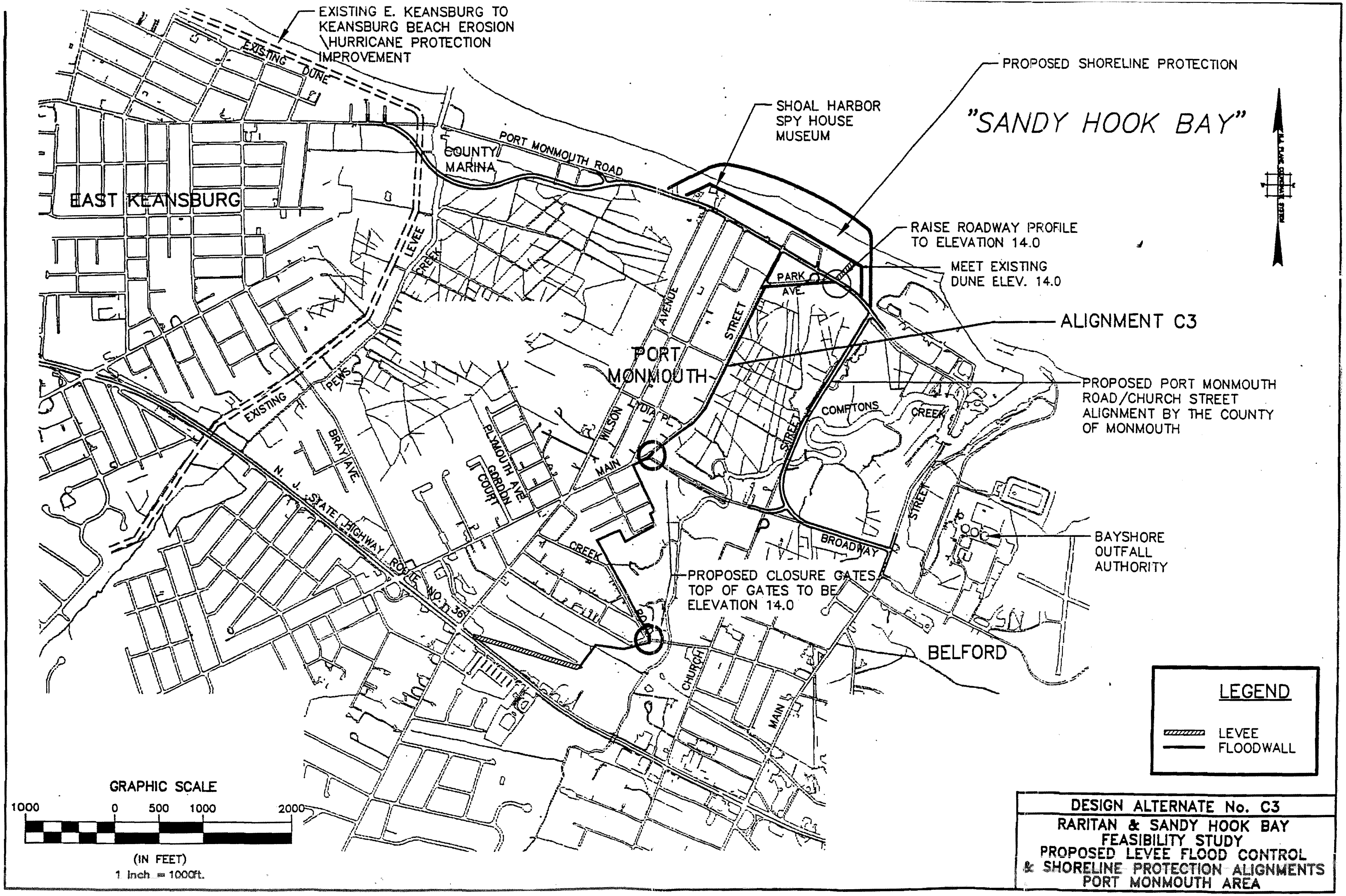
A-209. This alignment consists of 7,300 linear ft. of floodwall, two segments of levee totaling 1,500 linear feet, two closure gates, and elevating a section of the Port Monmouth roadway profile, as well as a section of the Henry Hudson Trail.

A-210. Costs. The first cost for this alignment alternative is estimated to be $10,193,000.

A-211. Impacts. It should be noted the proposed floodwall height will be between 6 and 10 ft. above existing grade. Consideration must be given to the visual aesthetics and desirability of a monolithic wall and its susceptibility to graffiti. Form liners may be necessary to reduce visual impacts.

Alignment C4

A-212. Description. Alignment C4 proceeds as a levee from the same location as the previous 3 alternatives and follows the C2 alignment. It crosses the Henry Hudson Trail (Figure A-30), where a section of the trail has to be elevated to 14.0 NGVD, it then



DESIGN ALTERNATE No. C3

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FEASIBILITY STUDY
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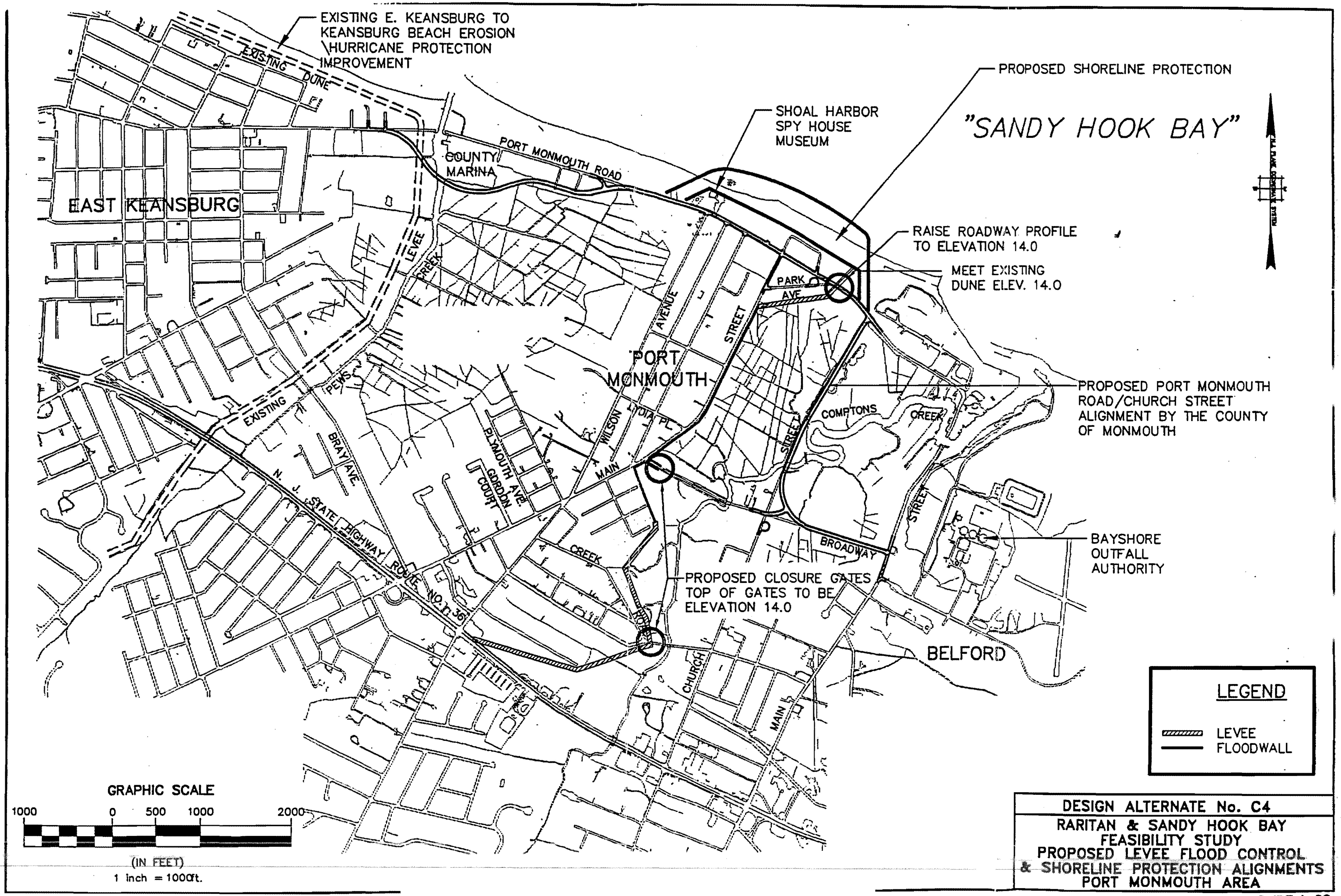
FIGURE A-29



DESIGN ALTERNATE No. C4

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PORT MONMOUTH AREA

FIGURE A-30

crosses Campbell Road where a closure gate is to be installed. The levee alignment proceeds northerly towards the eastern terminus of Pine Hurst Avenue. The alignment proceeds northwesterly to Main Street as a floodwall. A closure gate will be required across Broadway at the intersection of Main Street. The floodwall continues northerly along the eastern side of Main Street for approximately 2,100 ft. where it reverts to a levee and again follows the C2 alignment. The route then crosses Port Monmouth Road, which will have to be raised to elevation 14.0 NGVD. The alignment then terminates at the dunes along Sandy Hook Bay.

A-213. Costs. The first cost for this alignment alternative is $8,757,000.

A-214. Impact. This alignment is essentially a hybrid of Alignments C2 and C3 in that it includes a levee which runs through the coastal wetlands and also contains a floodwall which helps mitigate wetland impacts.

A-215. The C4 alignment footprint will impact approximately 6.41 acres of tidal wetlands.

Non-Structural Alternatives

Alternative N1

A-216. Description. Flood protection could be provided with minimal environmental impact through non-structural treatments. The first Non-Structural Alternative considered, designated N1, would provide protection to a stage of 14.0 ft. NGVD, the same level of protection as the structural alternatives. With Alternative N1, 883 buildings would require some form of non-structural protection. Preliminary assessments dictates 571 raisings, 232 floodproofings, and 12 ringwalls. would be required. Buyouts would be required at 67 residential and 1 commercial properties at which flood depths would exceed the physical limits of reliable floodproofing.

A-217. Among the protected buildings, the Shoal Harbor Museum building and a day care facility require the use of ringwalls. Both the fire station and rescue squad require floodproofing. The Bayshore Village apartment complex require a combination of floodproofing and raising treatments to various buildings. Shoal Harbor Live Lobster, the Seafood Corp., the Fish Co-op, and the county marina were excluded from the plan due to their unique configuration and shorefront access needs.

A-218. Costs. The cost of the non-structural plan protecting to 14.0 ft NGVD is estimated to be $60,600,000.

A-219. Impacts. Smaller scale construction to individual properties will minimize impacts to open spaces. Relocation of residents will create a significant social hardship and may prove to be entirely unfeasible. This plan would not provide complete protection since many locations, including the fire station, rescue squad and a daycare facility would remain inaccessible during the design storm. Accordingly this plan is not effective in eliminating threats to public safety.

Alternative N2

A-220. Description. Construction of non-structural flood protection against the 100 year event with only 1 ft. of freeboard would require protection to 12.8 ft. NGVD. Alternative N2 includes 681 buildings that would require non-structural protection. Preliminary assessment indicates 479 raisings, 157 floodproofings, and 7 ringwalls would be required. Buyouts would be required at 37 residentials and 1 commercial properties.

A-221. Alternative N2 differs from Alternative N1 in that the Shoal Harbor Museum property, the Rescue Squad, and part of the Bayshore Village development would not require flood protection.

A-222. Alternative N2 requires floodproofing for the fire station and the daycare facility. As with Alternative N1 the Shoal Harbor Live Lobster, the Seafood Corp., the Fish Co-op and the county marina would not be protected.

A-223. Costs. The costs for the non-structural plan N2, which protects against the 100 year one of foot of with freeboard is estimated to be $44,461,000.

A-224. Impacts. Relocation of residents creates a significant disruption and hardship and may prevent successful implementation of a non-structural plan. The fire station and the daycare facility would remain inaccessible during the design storm presenting a threat to public safety.

Alternative N3

A-225. Description. A third non-structural alternative was developed which would not require buyouts and relocation. Construction of non-structural flood protection against the 25 year event with 1 ft. freeboard would require protection to 10.2 ft. NGVD for 433 buildings. Preliminary assessment indicates 268 raisings, 161 floodproofings, and 4 ringwalls would be required.

A-226. As with Alternative N1 and Alternative N2 the Shoal Harbor Live Lobster, the Seafood Corp., the Fish Co-op and the marina will be excluded from the plan due to their operational needs.

A-227. Costs. The analysis for the non-structural plan to protect against the 25 year with freeboard event is calculated to be $21,120,000.

A-228. Impacts. While this plan would create minimal environmental impact and would not require relocation of any floodplain residents, the flood protection provided would be highly unreliable. The low level of design would result in greater than a 1 in 3 chance that the design storm would be exceeded at least once over any 10 year period. There is only a 13% chance that the design would be successful over the 50 year project life.

Summary and Comparison of Preliminary Alternatives

Comparison of Preliminary Alternatives

A-229. With the exception of non-structural alternative plan N3, all the plans represent technically feasible solutions. The most severe negative impacts identified in the analysis are the direct destruction of wetlands due to construction activities; the indirect impacts to wetlands due to changes in hydrology; and the disruption of community and personal lives due to buyouts and relocations.

A-230. In general, plans which minimize socially disruptive buyouts result in the most significant wetland disturbance, and conversely plans with the largest wetland disturbances tend to have the lowest implementation cost. This indicates that the decision as to the most desirable plan will represent a tradeoff of social, environmental and economic concerns, requiring input from the local sponsor and environmental review agencies.

PREFERRED PLAN

Plan Formulation Background

A-231. The overall plan has been formulated as four separate components for evaluation, all of which are necessary to provide protection from storm damage and flooding. Separate discussions are included of plans for the Raritan Bay shorefront, Pews Creek, & Compton Creek Each of these components has been formulated with consideration of avoiding or minimizing environmental impacts. The plan requires development of a fifth component, environmental mitigation, to meet planning requirements for completeness where total avoidance of impacts was not feasible.

A-232. The first planning effort was an investigation of the preliminary alternatives in the fall of 1996, which documented preliminary costs and impacts for an array of possible levee, storm gate, and non-structural plans. These preliminary results were coordinated with the Local Sponsor, New Jersey Department of Environmental Protection (NJDEP), which expressed several preferences for specific features. From a land use and environmental perspective the agency expressed a preference for a storm gate at Pews Creek which would minimize direct footprint impacts. From an operations and maintenance (O&M) perspective, however, there was significant concern that the use of a storm closure gate at Pews Creek could require a long term commitment to increase agency staff and may not be a supportable alternative. These concerns were taken into account in conducting the screening of plans for more detailed development.

A-233. For the Compton Creek segment of the project, the alignment identified as C2 was selected as the preferred alternative. This alignment minimized the impact of levees on the wetlands, without relying on extensive lengths of floodwalls. Extensive areas of floodwalls, such as proposed in alternatives C3 and C4, would be prohibitively expensive and would probably create a graffiti nuisance. Alternative C1 was not selected due to unacceptable levels of environmental disturbance.

A-234. The selections of the shorefront element attempted to reduce costs and to minimize or avoid future beachfill renourishments while providing the desired level of protection. One bayshore protection layout that was examined was the alignment featuring an upland dune layout, with a sheet pile floodwall protecting the Spy House property with minimal footprint (S4). However, in comments pertaining to the preliminary layout, the Monmouth County Board of Recreation Commissioners expressed opposition because the dune footprint conflicted with the local plan for the shoreline development. Since the parkland was purchased as dedicated recreation land with Green Acres funding, a change in the use of the land would require in kind replacement. Given the unique nature of the site, blending active recreation with interpretative historic facilities, such replacement was not viable. Therefore, the dune layout over upland features is not considered implementable. In order to maintain consistency with public usage of the shorefront, alternative S1 consisting of beach and dune fill was selected. Alternative S2 was not selected since the costs of S1 and S2 are similar. The S3 alignment is cost prohibitive for the same level of protection.

A-235. The screening of protection along Pews Creek attempted to minimize impacts to the environment without creating severe social impacts due to numerous structure acquisitions. Since the local sponsor indicated that they may not support the use of a closure gate at Pews Creek, alternative P1 was not selected for continued development. Alternative P2 was not selected due to excessive impacts to the tidal wetlands. While alternative P3 would avoid wetland impacts, it was not selected due to the need for numerous structure acquisitions. Alternative P4 was initially identified as the Pews Creek alternative which provided the best balance in minimizing environmental and social impacts.

A-236. The findings of the screening process were subsequently further coordinated with representatives of Middletown Township and various County agencies. In an effort to expand the geographic extent of coverage, Township officials indicated a clear preference for storm closure gates at Pews, and if possible, Compton Creeks. In response to the Local Sponsor's reluctance to make a long term commitment of State manpower for security and O&M for such closure gates, the Township officials suggested possible solutions. Currently the Town maintains a staff for the O&M of the nearby East Keansburg Storm Water Pump Station which could possibly service a station at Pews

Creek. In addition, by locating the gate adjacent to the Monmouth County marina vandalism and security concerns would be reduced. Based on the reduction of environmental impacts, the more inclusive protection, and the availability of local resources to support the maintenance of a closure facility, the Local Sponsor indicated a willingness to support a storm closure structure at Pews Creek.

A-237. Subsequent to the local coordination meeting additional economic investigations were undertaken to identify if gate structures could be supported as components of the National Economic Development (NED) Plan which normally establishes the limit for Federal cost sharing. Based on the preliminary screening information the initial estimate of total annual costs for a gate and pump station at Pews Creek indicated that the annualized cost of the gate alternative would be $223,000/year higher than the originally preferred Pews Creek Levee alignment (P4). This increase in cost compares to a preliminary estimate of a $280,000 increase in annual benefits (excluding residual interior damage) due to the protection of approximately 90 additional structures in the Bray Avenue area. Additional benefits to 121 structures upstream of the previously defined study area, which ended at Route 36, would total approximately $413,000 annually. Accordingly the gate at Pews Creek would yield $460,000 in annual net benefits and was considered as a possible element of the NED Plan.

A-238. Whereas the levee alignment P4 could also effect the tidal inundation patterns of wetlands located on the protected side of the levee, the use of a closure gate alternative at Pews Creek would reduce this effect as well as the extent of the permanent project footprint within the wetlands. The gate alignment and opening would be developed to allow tidal inundation of the wetlands to continue with minimal disruption of existing depths and frequency. The size and configuration of the gate required to maintain the existing tidal flow conditions would later be established as part of a 2-dimensional hydrodynamic modeling effort in this study.

A-239. Based on the request of Township officials to consider a more comprehensive protection alignment than the levee alignment (C2) favored in the initial screening, a levee/gate alignment extending further east over Compton Creek was examined. This preliminary levee/gate alignment would follow the new Port Monmouth Road, extending protection to the west bank of this creek. The alignment would provide protection to approximately 12 ft. NGVD, and would provide protection to 276 structures not protected by the proposed levee. Since structures in this area do not suffer significant damage as frequently as structures in other portions of the study area only limited additional protection would be provided by a gate on Compton Creek. Economic analysis indicated that the use of a storm closure gate and levee at elevation 12 ft. NGVD will only provide 20% and 50% reductions in equivalent annual damage in the additional reaches protected by the gate at Compton Creek. The preliminary estimate of damage reduction benefits for a gate protecting these areas totals $77,000 annually with the added annual cost exceeding these benefits. Therefore, a gate at Compton Creek was not considered an economically viable element and the levee alternative was selected.

A-240. In order to identify the plan which most fully satisfies the planning objectives, additional technical, economic, environmental and cultural resource analyses are required. These analyses will be limited to plans incorporating dune/beach fill improvements along Raritan Bay, combined with levees/floodwalls along Compton Creek and the storm closure gate at Pews Creek. These improvements, combined with necessary mitigation and interior drainage features, have been identified as the most efficient means to achieve the planning objectives. Following more detailed design of the levee, gate and drainage requirements, this decision was verified through a comparison of the resulting costs and benefits. This assessment indicated that the actual increase in cost for using the gate rather than the levee at Pews Creek would be approximately $335,000 annually. After adjusting for residual interior damages of nearly $50,000, the additional benefits of the gate alignment would total approximately $640,000. This assessment verified that the use of a closure gate at Pews Creek provides approximately $300,000 of annual benefits in excess of costs and represents the NED plan alignment.

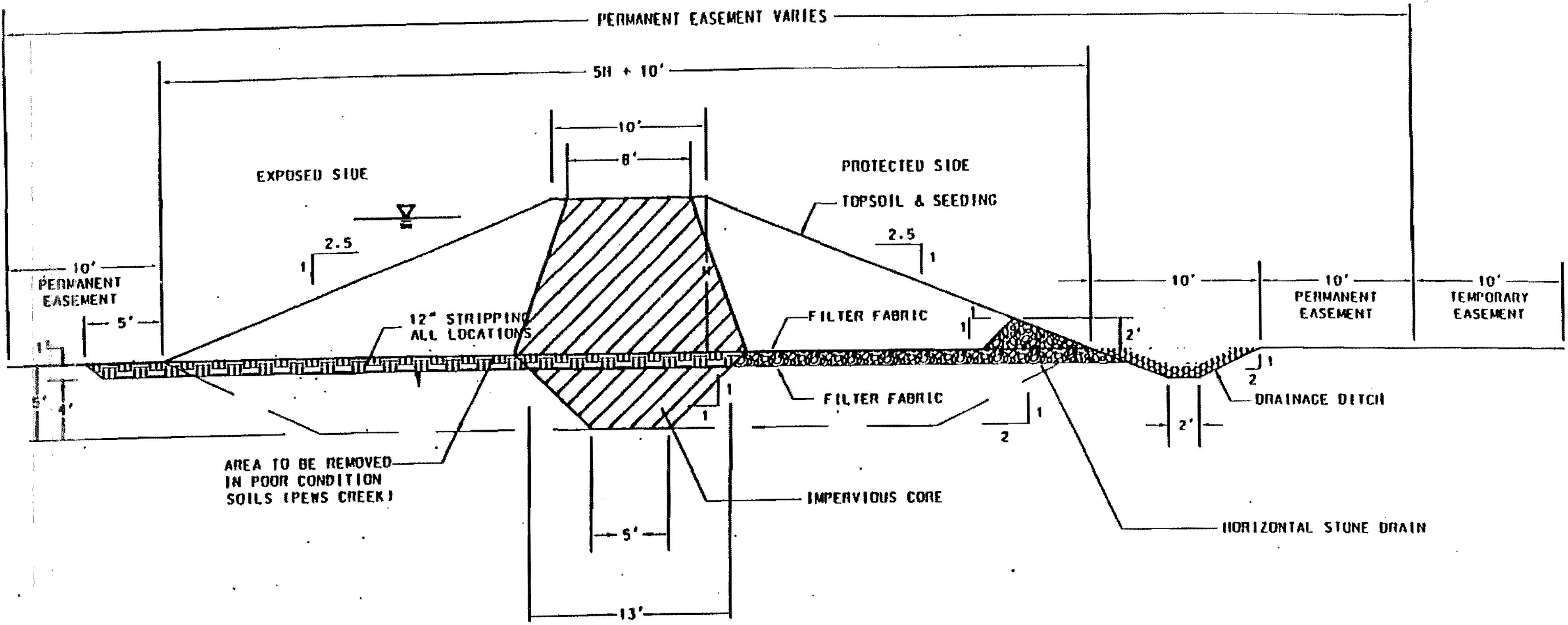
Design Detail

A-241. The storm damage protection system selected is comprised of levees, floodwalls, reconstructed dunes and beach, a storm gate pump station, street gates and a road raising. The alignment will span from State Highway 36 to Sandy Hook Bay then west along the shoreline to Port Monmouth Road and tie into the existing Keansburg levee by way of a storm gate. The following sections describe the facilities for each of the major plan components; Compton Creek, the exposed bay shoreline, and Pews Creek.

A-242. Design details for the Compton Creek and Pews floodwalls, road raising, and closures are presented in the following paragraphs. The Pews creek pump station and storm gate are also discussed. Levee design details are presented in Appendix E. Drainage design details are presented in Appendix F.

Compton Creek

A-243. Levee. The Compton Creek levee will provide flood control protection for a portion of the study area with 6,725 linear feet of earthen levee. The levee is designed in accordance with EM 1110-2-1913. The proposed levee will have a crest width of 10 feet and a design elevation of +14.0 NGVD with side slopes of 1V on 2.5 H. The levee alignment will basically traverse the edge of the wetlands delineation limit in an attempt to minimize adverse impacts to the wetlands area. An impervious core is provided along the entire length of levee. The impervious core will extend from the top of the levee to approximately 5 feet below the existing ground to prevent seepage under and through the foundation of the levee (Figure A-31). A 12" thick horizontal drainage layer is



TYPICAL LEVEE SECTION

Revision	Description	Date	Approved

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
NEW YORK, NEW YORK

Designed by:

Drawn by:

Checked by:

Reviewed by:

SECTION CHIEF

Concurred by:

PORT MONMOUTH COMBINED FLOOD CONTROL AND SHORE PROTECTION FEASIBILITY STUDY

TYPICAL LEVEE SECTION
PEWS CREEK
COMPTONS CREEK

Approval Recommended:

CHIEF, ENGINEERING DIVISION

Approval By:

SHEET REFERENCE NUMBER:

DATE:

SCALE: 1" = 10'

FIG.:

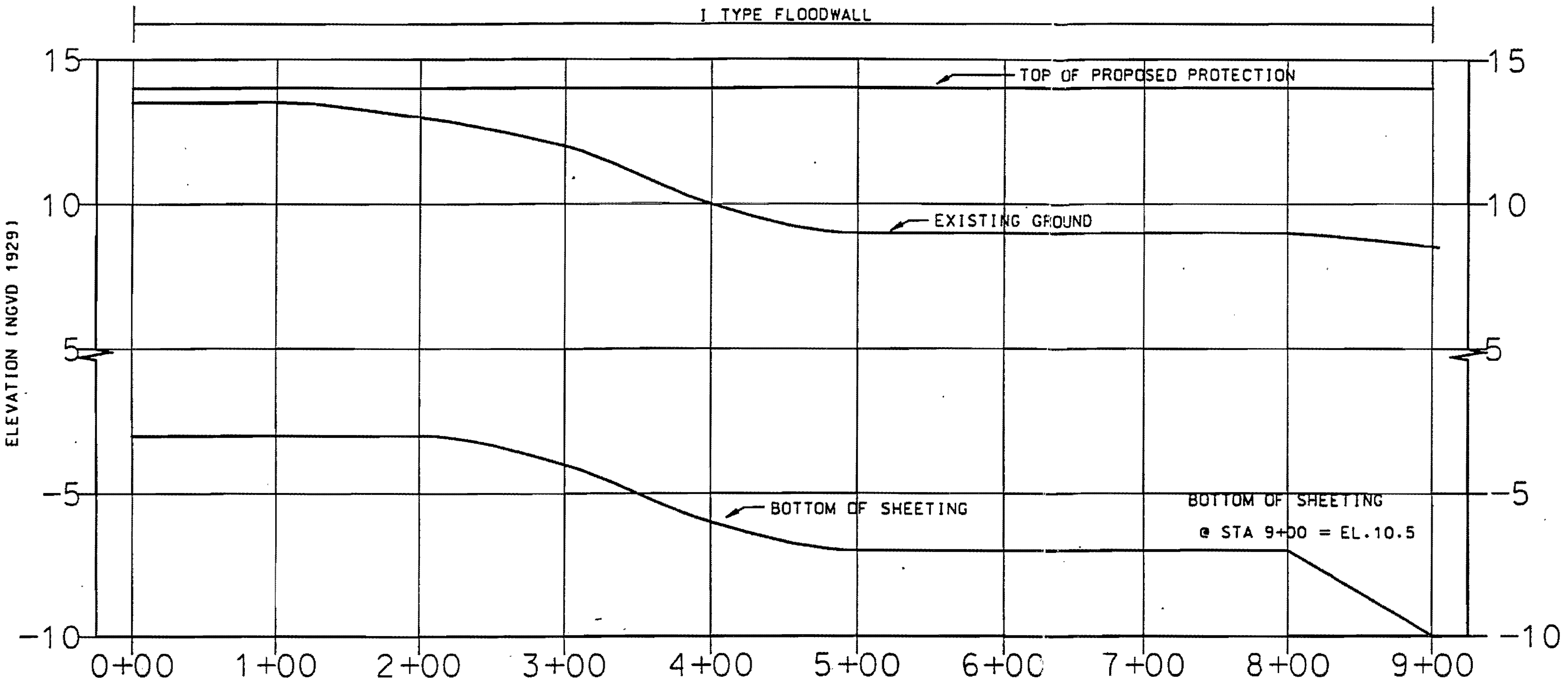
FIGURE A-31

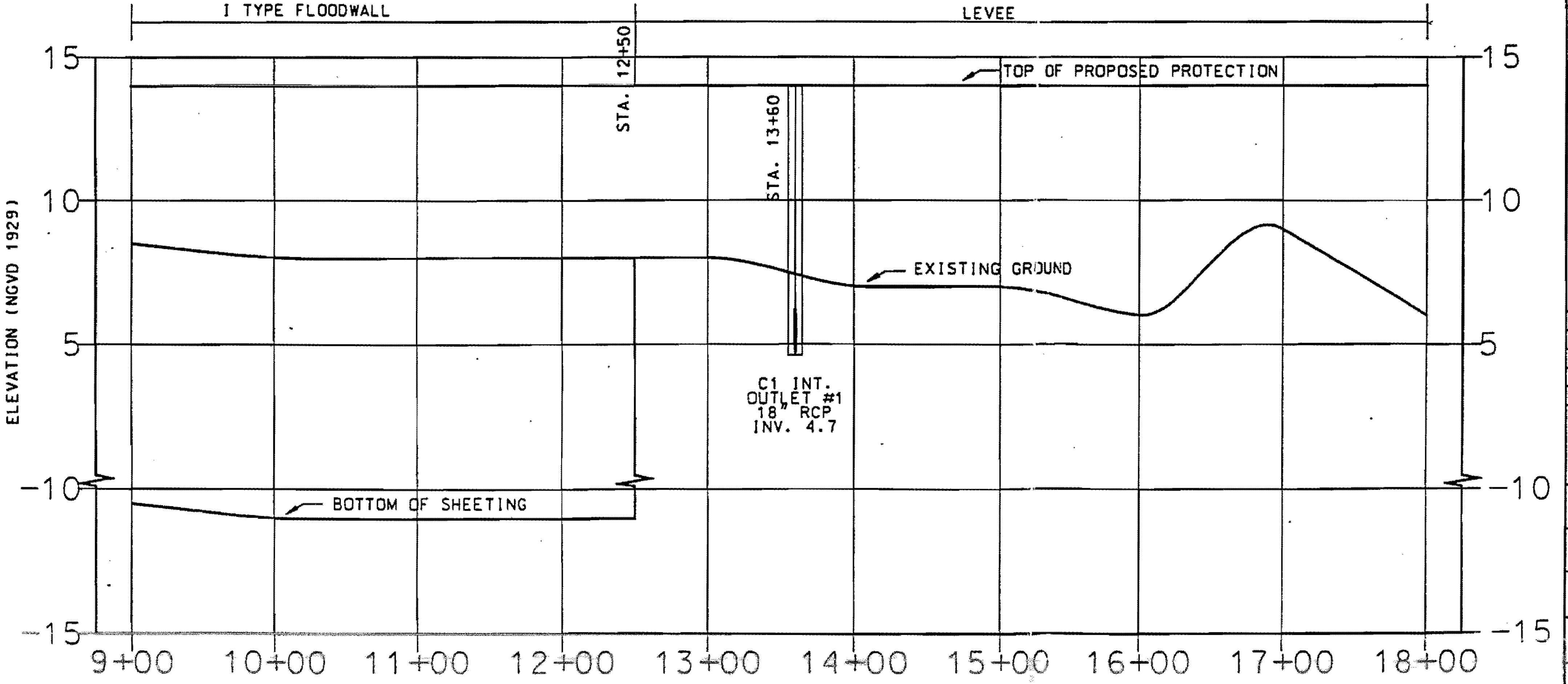
incorporated into the levee section from the impervious core to the upland toe of the levee. The drainage layer will protect the base of the embankment against high uplift pressures and to carry off seepage. Three (3) main drainage structures and eleven (11) interior drainage structures were incorporated along the levee alignment (Figures A-32 through A-38). A drainage ditch along the entire upland toe of the levee will collect interior runoff and direct flow to the drainage structure. One (1) road raising on Port Monmouth Road will be required along the alignment to provide closure of the line of protection.

A-244. Floodwall. The Compton Creek line of protection ends with approximately 1,250 linear feet of concrete cap sheet pile floodwall tying into existing ground approximately 250 feet east of Wilson Avenue just north of State Highway Route 36 (Figure A-39). The floodwall has been designed in accordance with EM 1110-2-2502. The floodwall follows the rear property lines of the existing homes fronting on Willow Avenue. The floodwall has a top width of 1.25 feet and a design elevation of 14.0 ft. NGVD. The concrete cap extends approximately five (5) feet below existing grade with a bottom width of 3.5 feet. The sheeting extends approximately seven to eight feet below grade to provide stability and to prevent seepage under and through the foundation of the wall. The use of a floodwall in this area is due to the limited area to construct a levee and to minimize property acquisition and easement widths.

A-245. Roadway Closures. The Compton Creek line of protection requires the installation of three (3) mitre gate roadway closures to maintain a closure elevation of +14.0 ft. NGVD. The roadway closure gates are designed in accordance with EM 1110-2-2705. The closure gate located at Campbell Avenue will have an opening of 40 feet and a height of 8.5 feet (Figure A-40). The closure gate located at Broadway has an opening of 40 feet and a height of 8.5 feet. The installation of the gate structures will require entire roadway resurfacing across the closure structure. The installation of piles, sheeting and tie downs will also be required at both locations. The closure gate at Port Monmouth Road at the County Marina has an opening of 40 feet and a height of 4 feet.

A-246. Road Raisings. A section of Port Monmouth Road, approximately 100 feet east of the former Park Avenue intersection, will require raising to provide closure to +14 ft. NGVD. The elevation of Port Monmouth Road at this location is between +8 to +9 ft. NGVD. A closure gate at this location was determined to be impractical due to the modest difference in elevation of the existing road and the design elevation, roadway width (40 feet) and the volume of traffic the road carries (collection road). The modest difference in existing elevation to proposed (approximately 5 to 6 feet) can be readily achieved by raising the road within the existing right-of-way (Figure A-40A). In order to raise the road to the design elevation a change in the road profile will be required resulting in road reconstruction for a distance of approximately 275 feet east and west of the levee crossing (Figure A-40B). The road reconstruction can be accommodated within the existing 60 foot wide right-of-way. As Port Monmouth is a County road, the specifics of this reconstruction must be coordinated with and approved by Monmouth County. No





HORIZ. 1" = 100'
VERT. 1" = 5'

REVISION	DESCRIPTION	DATE	APPROVED

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
NEW YORK, NEW YORK

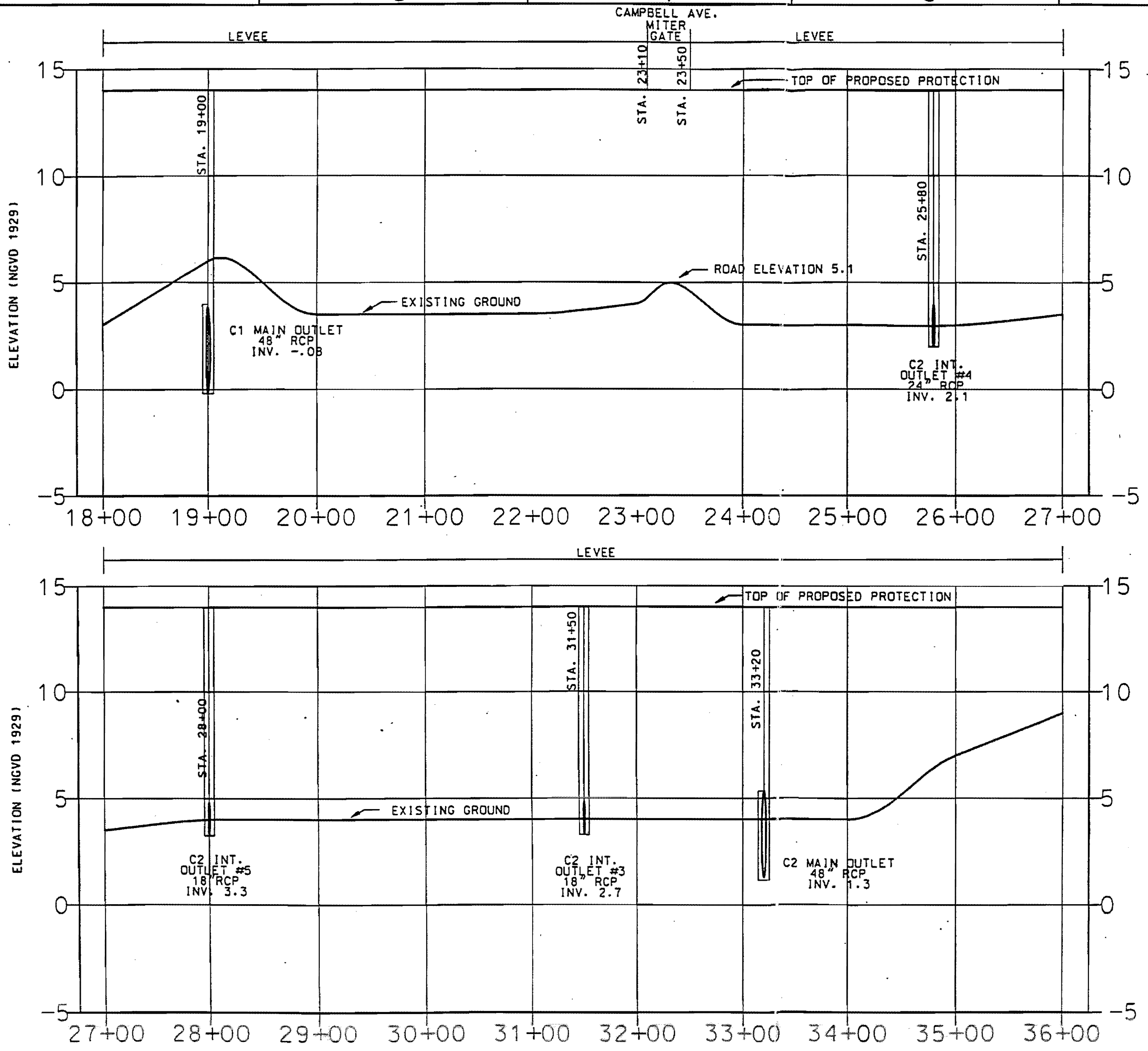
DESIGNED BY:
DRAWN BY:
CHECKED BY:
REVIEWED BY:
SECTION CHIEF
CONCURRED BY:
CHIEF, DESIGN BRANCH

PORT MONMOUTH COMBINED FLOOD CONTROL
AND SHORE PROTECTION FEASIBILITY STUDY

COMPTONS CREEK ALIGNMENT
FLOOD PROTECTION ELEVATION 14.0
STA. 0+00 TO 18+00

APPROVAL RECOMMENDED:
CHIEF, ENGINEERING DIVISION
APPROVAL BY:
COLONEL, C.E. DISTRICT ENGINEER

SHEET REFERENCE NUMBER:
DATE:
SCALE: AS SHOWN
FIG.: A-32



HORIZ. 1" = 100'
VERT. 1" = 5'

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NEW YORK, NEW YORK

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SECTION CHIEF

CONCURRED BY:
CHIEF, DESIGN BRANCH

PORT MONMOUTH COMBINED FLOOD CONTROL
AND SHORE PROTECTION FEASIBILITY STUDY

COMPTONS CREEK ALIGNMENT
FLOOD PROTECTION ELEVATION 14.0
STA. 18+00 TO 36+00

APPROVAL RECOMMENDED:
CHIEF, ENGINEERING DIVISION

APPROVAL BY:
COLONEL, C.E. DISTRICT ENGINEER

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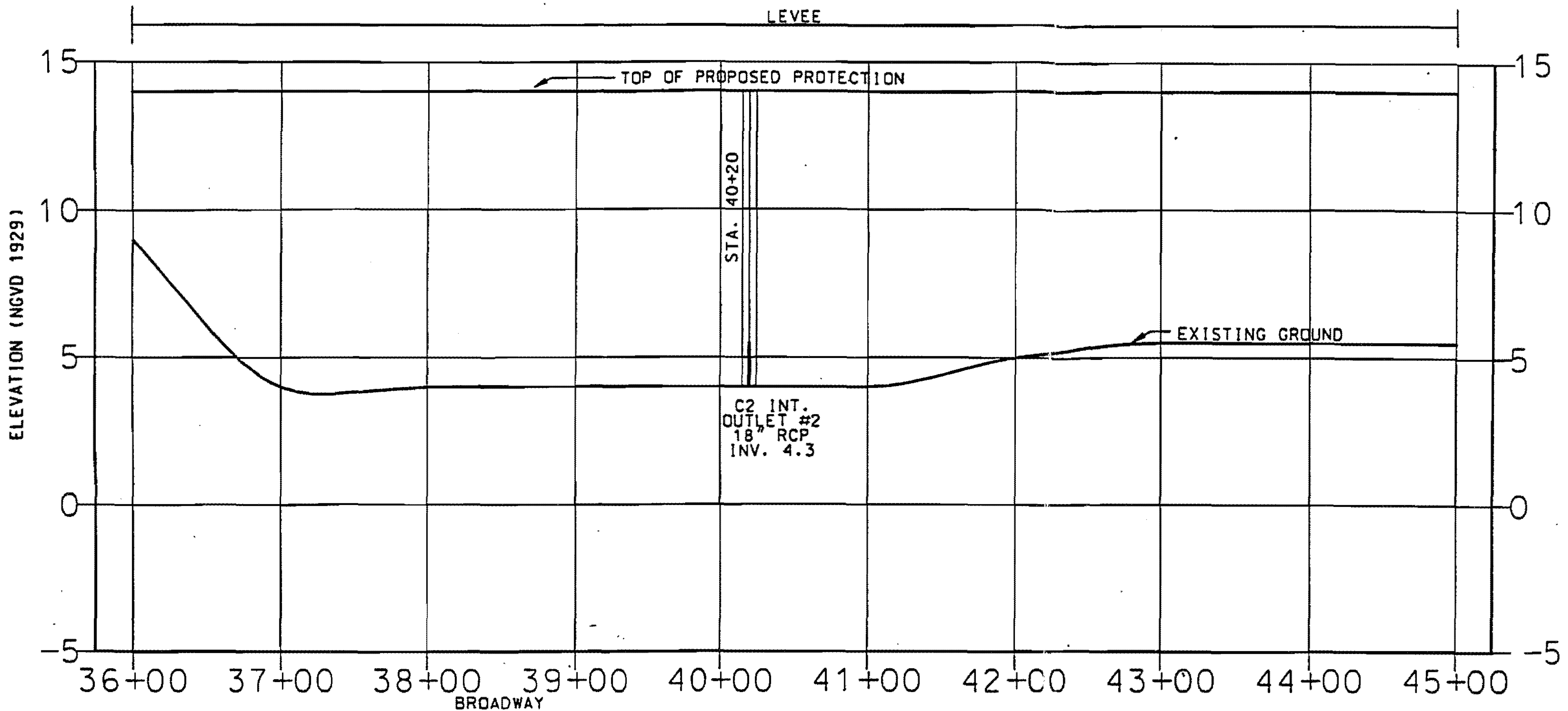
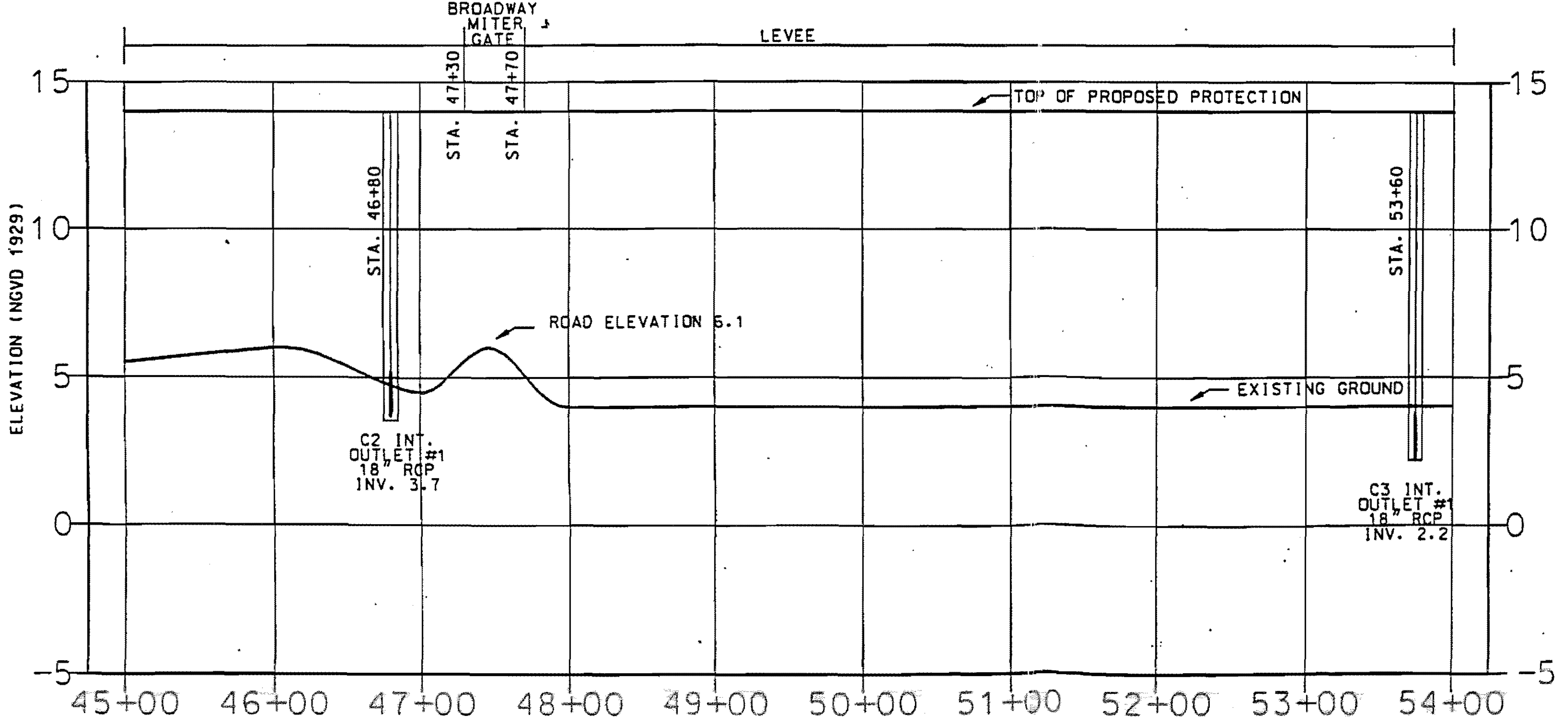
DATE:

SCALE: AS SHOWN

FIG.: A-33





HORIZ. 1" = 100'
VERT. 1" = 5'

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U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
NEW YORK, NEW YORK

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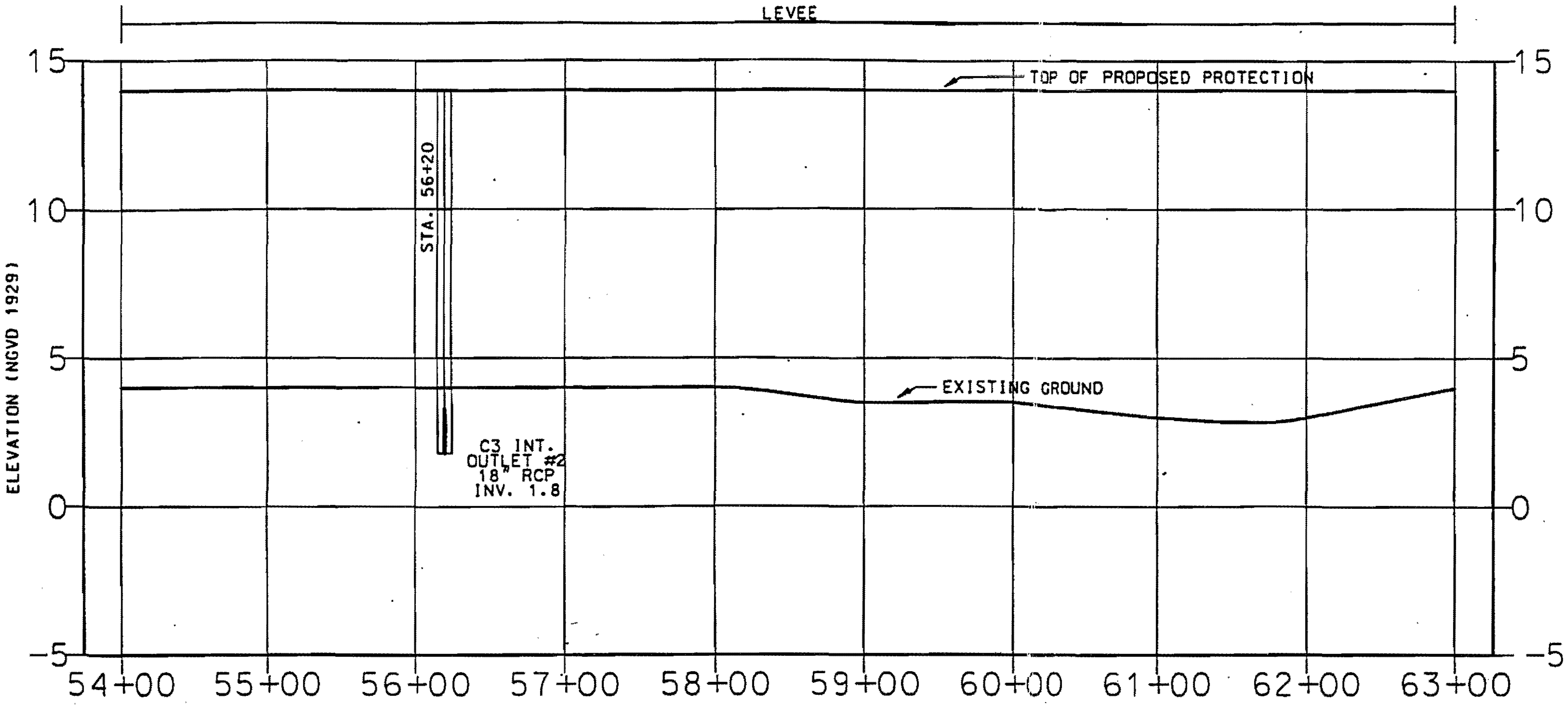
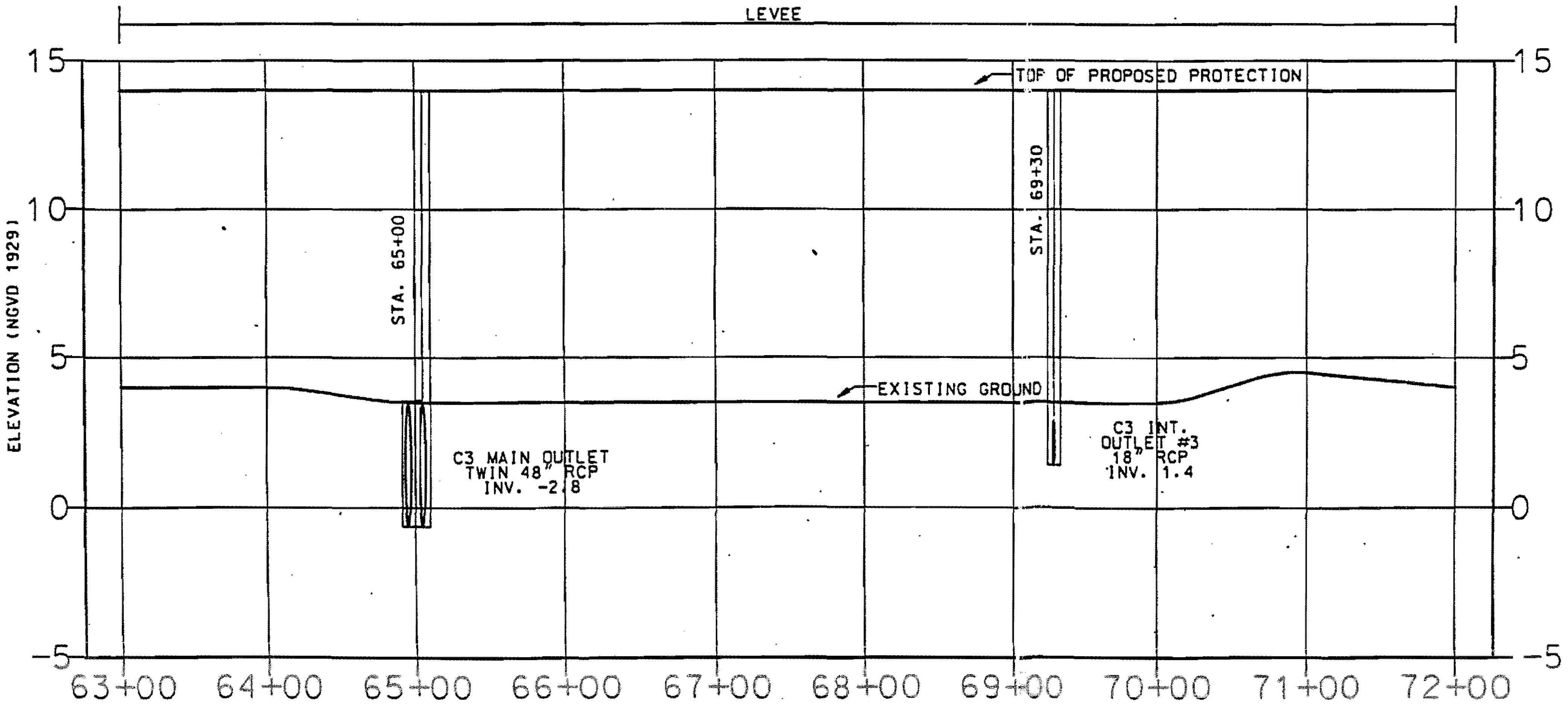
REVIEWED BY:
SECTION CHIEF
CONCURRED BY:
CHIEF, DESIGN BRANCH

PORT MONMOUTH COMBINED FLOOD CONTROL
AND SHORE PROTECTION FEASIBILITY STUDY

COMPTONS CREEK ALIGNMENT
FLOOD PROTECTION ELEVATION 14.0
STA. 36+00 TO 54+00

APPROVAL RECOMMENDED:
CHIEF, ENGINEERING DIVISION
APPROVAL BY:
COLONEL, C.E. DISTRICT ENGINEER
SHEET REFERENCE NUMBER:
DATE:
SCALE: AS SHOWN
FIG.: A-34





HORIZ. 1" = 100'
VERT. 1" = 5'

REVISION	DESCRIPTION	DATE	APPROVED

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
NEW YORK, NEW YORK

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REVIEWED BY:
SECTION CHIEF
CONCURRED BY:
CHIEF, DESIGN BRANCH

PORT MONMOUTH COMBINED FLOOD CONTROL
AND SHORE PROTECTION FEASIBILITY STUDY

COMPTONS CREEK ALIGNMENT
FLOOD PROTECTION ELEVATION 14.0
STA. 54+00 TO 72+00

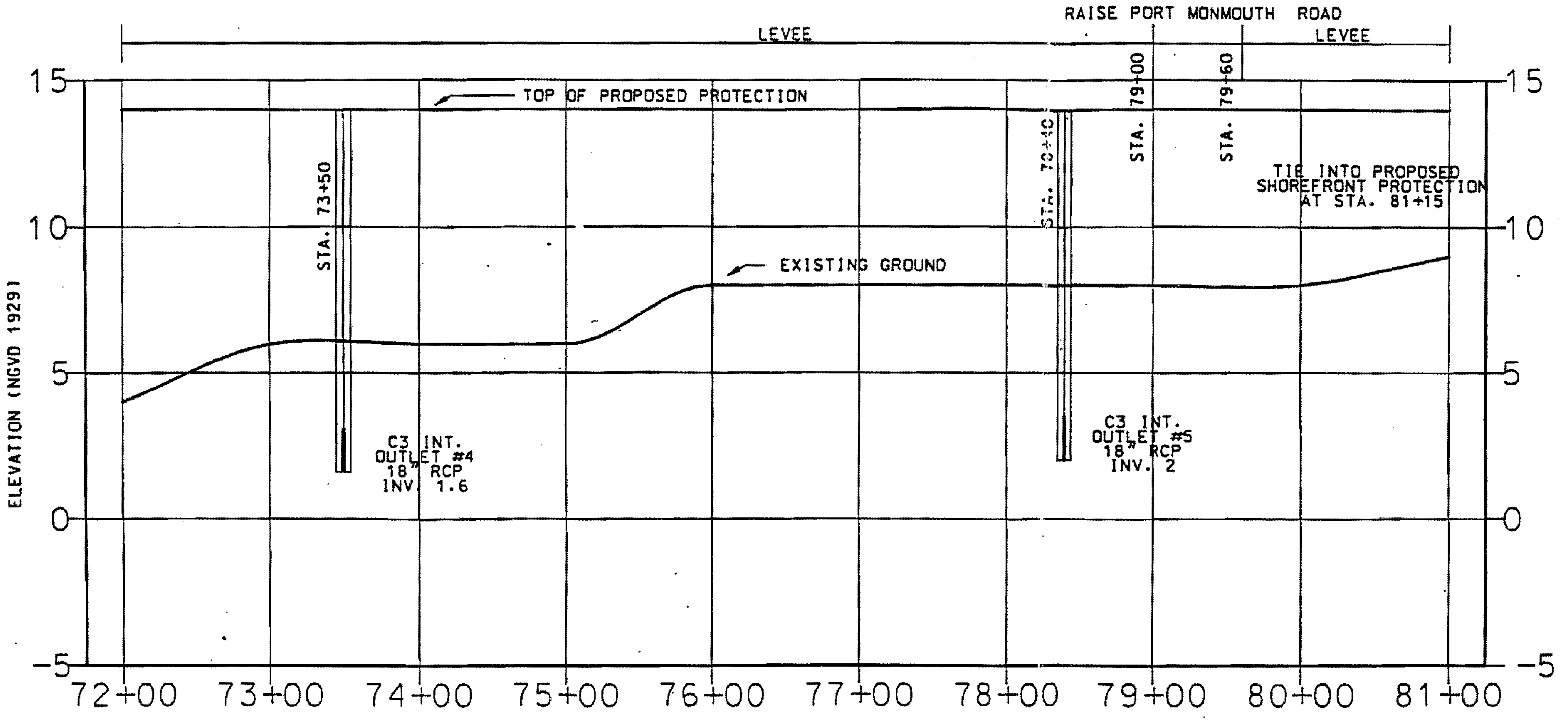
APPROVAL RECOMMENDED:
CHIEF, ENGINEERING DIVISION
APPROVAL BY:
COLONEL, C.E. DISTRICT ENGINEER

SHEET REFERENCE NUMBER:

DATE:

SCALE: AS SHOWN

FIG. A-35



HORIZ. 1" = 100'
VERT. 1" = 5'

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NEW YORK, NEW YORK

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CHECKED BY:

PORT MONMOUTH COMBINED FLOOD CONTROL
AND SHORE PROTECTION FEASIBILITY STUDY

COMPTONS CREEK ALIGNMENT
FLOOD PROTECTION ELEVATION 14.0
STA. 72+00 TO 81+00

REVIEWED BY:
SECTION CHIEF
CONCURRED BY:
CHIEF, DESIGN BRANCH
APPROVAL RECOMMENDED:
CHIEF, ENGINEERING DIVISION
APPROVAL BY:
COLONEL, C.E. DISTRICT ENGINEER
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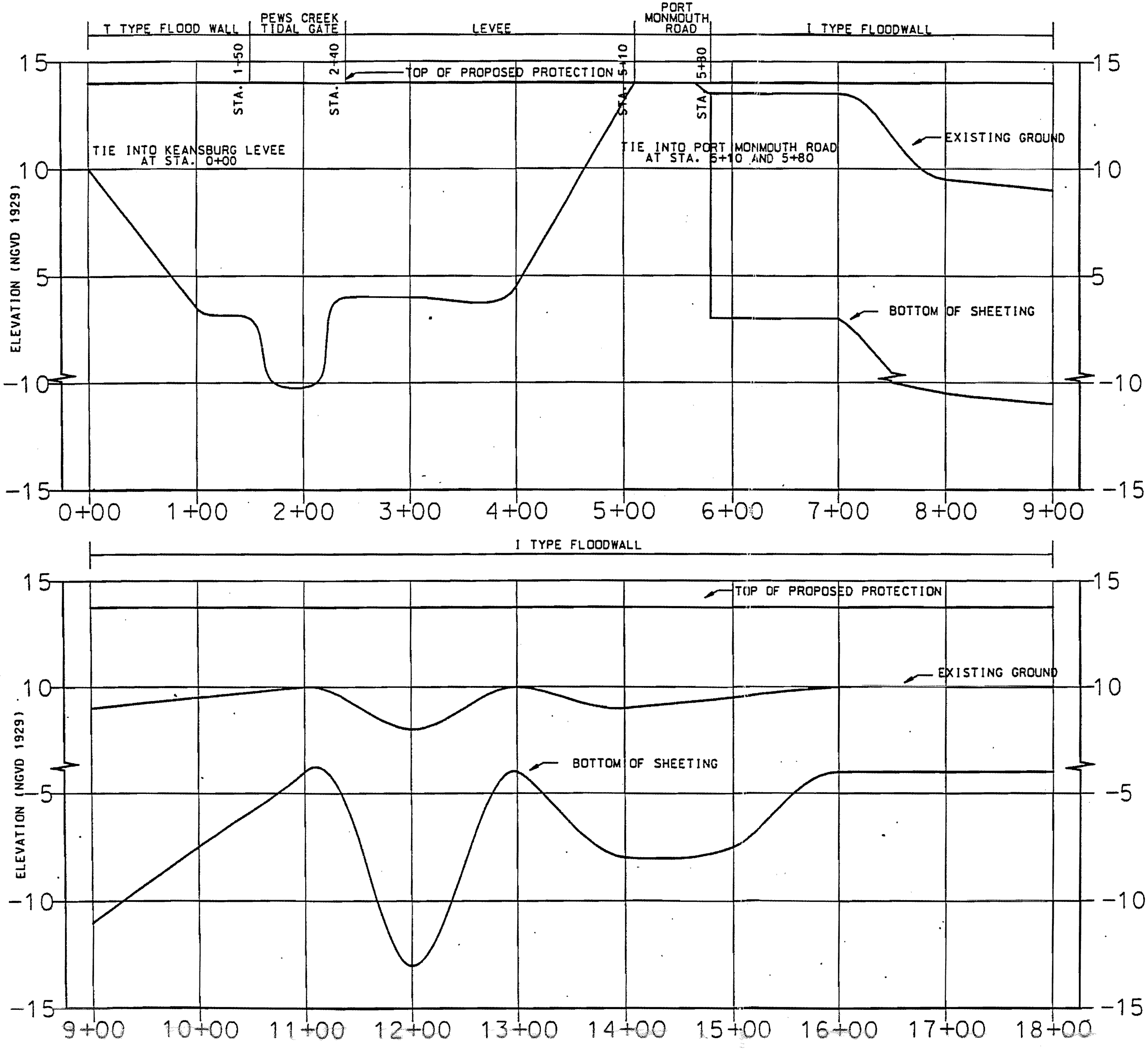
DATE:
SCALE: AS SHOWN
FIG.: A-36



HORIZ. 1" = 100'
VERT. 1" = 5'

REVISION	DESCRIPTION	DATE	APPROVED

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
NEW YORK, NEW YORK

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REVIEWED BY:
SECTION CHIEF
CONCURRED BY:
CHIEF, DESIGN BRANCH

PORT MONMOUTH COMBINED FLOOD CONTROL AND SHORE PROTECTION FEASIBILITY STUDY

PEWS CREEK ALIGNMENT
FLOOD PROTECTION ELEVATION 14.0
STA. 0+00 TO 18+00

APPROVAL RECOMMENDED:
CHIEF, ENGINEERING DIVISION
APPROVAL BY:
COLONEL, C.E. DISTRICT ENGINEER
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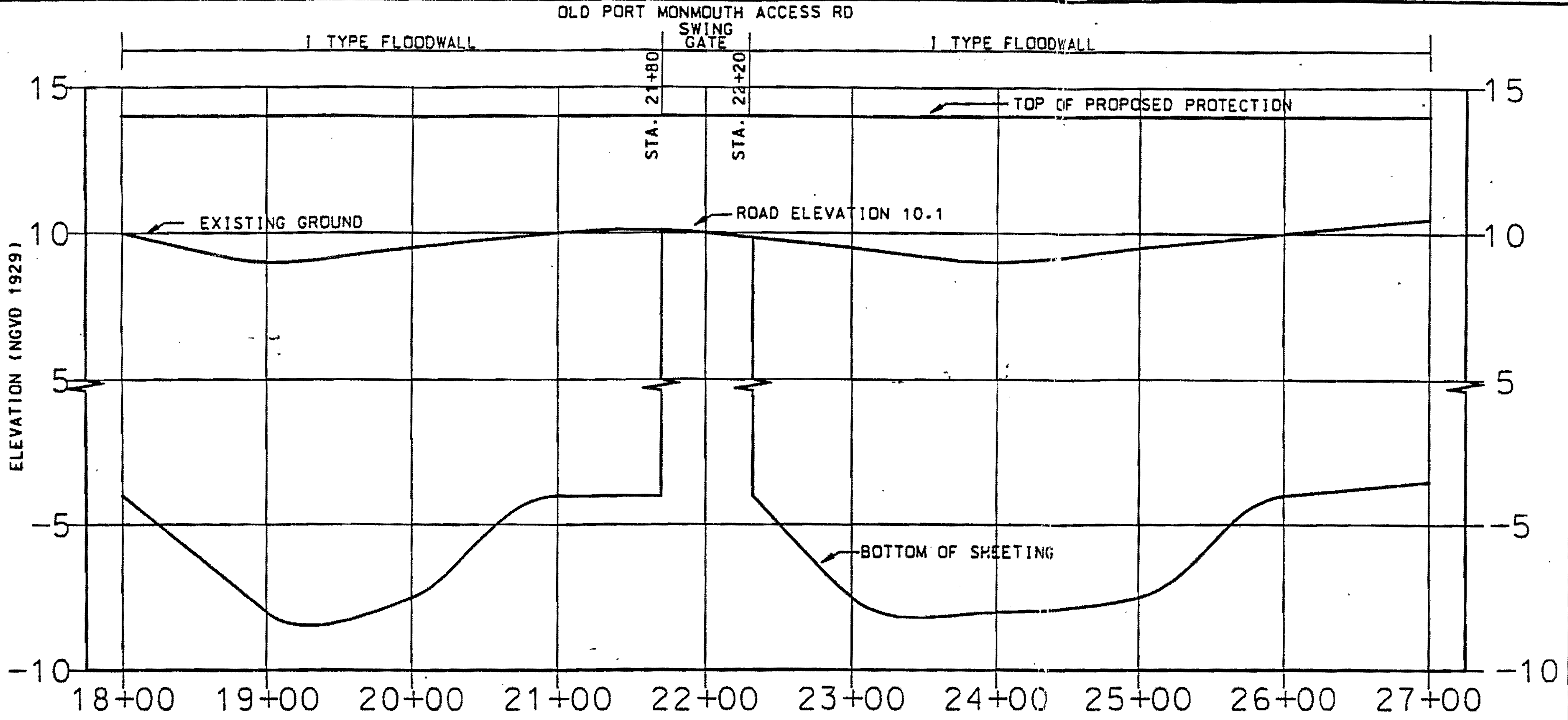
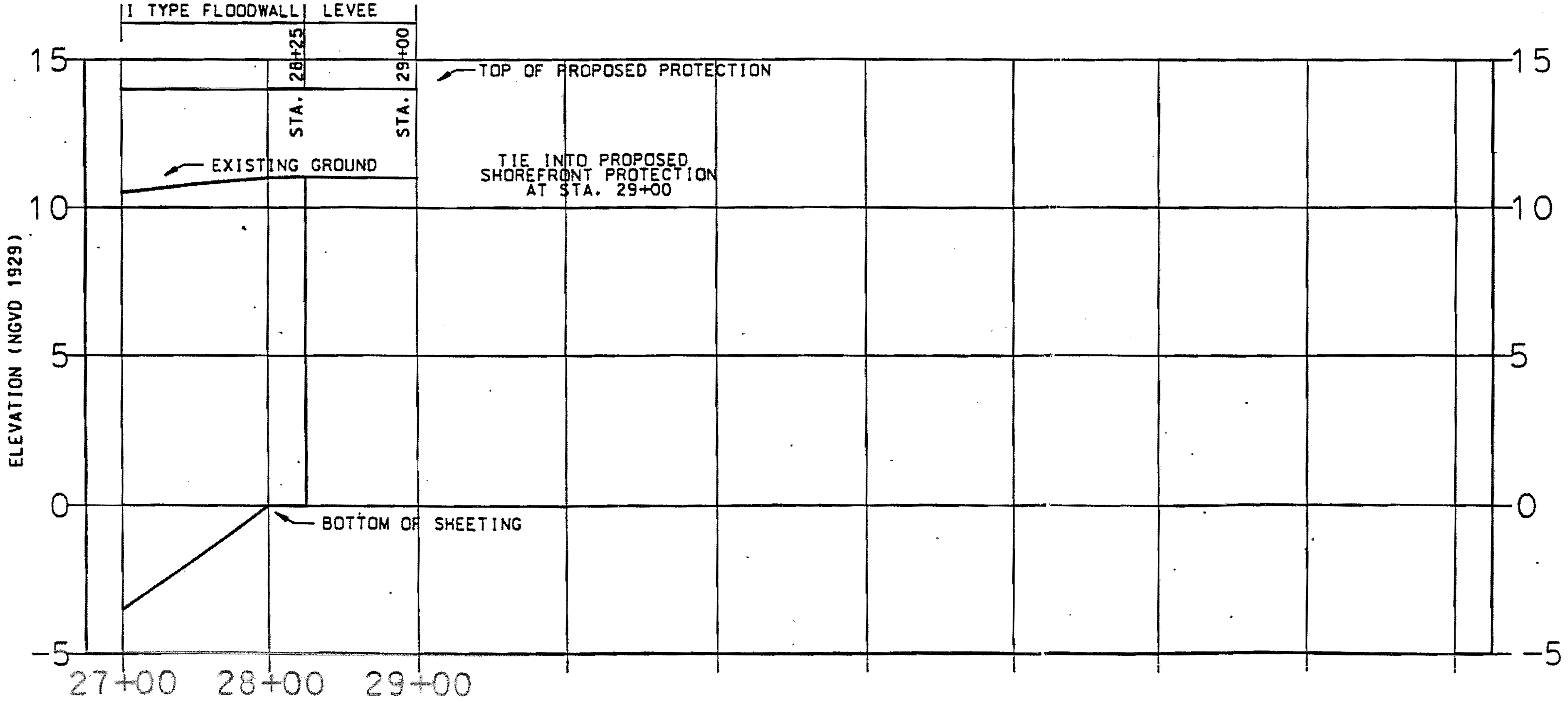
DATE:
SCALE: AS SHOWN
FIG.: A-37





HORIZ. 1" = 100'
VERT. 1" = 5'

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CORPS OF ENGINEERS
NEW YORK, NEW YORK

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REVIEWED BY:
SECTION CHIEF
CONCURRED BY:
CHIEF, DESIGN BRANCH

PORT MONMOUTH COMBINED FLOOD CONTROL
AND SHORE PROTECTION FEASIBILITY STUDY

PEWS CREEK ALIGNMENT
FLOOD PROTECTION ELEVATION 14.0
STA. 18+00 TO 29+00

APPROVAL RECOMMENDED:
CHIEF, ENGINEERING DIVISION
APPROVAL BY:
COLONEL, C.E. DISTRICT ENGINEER

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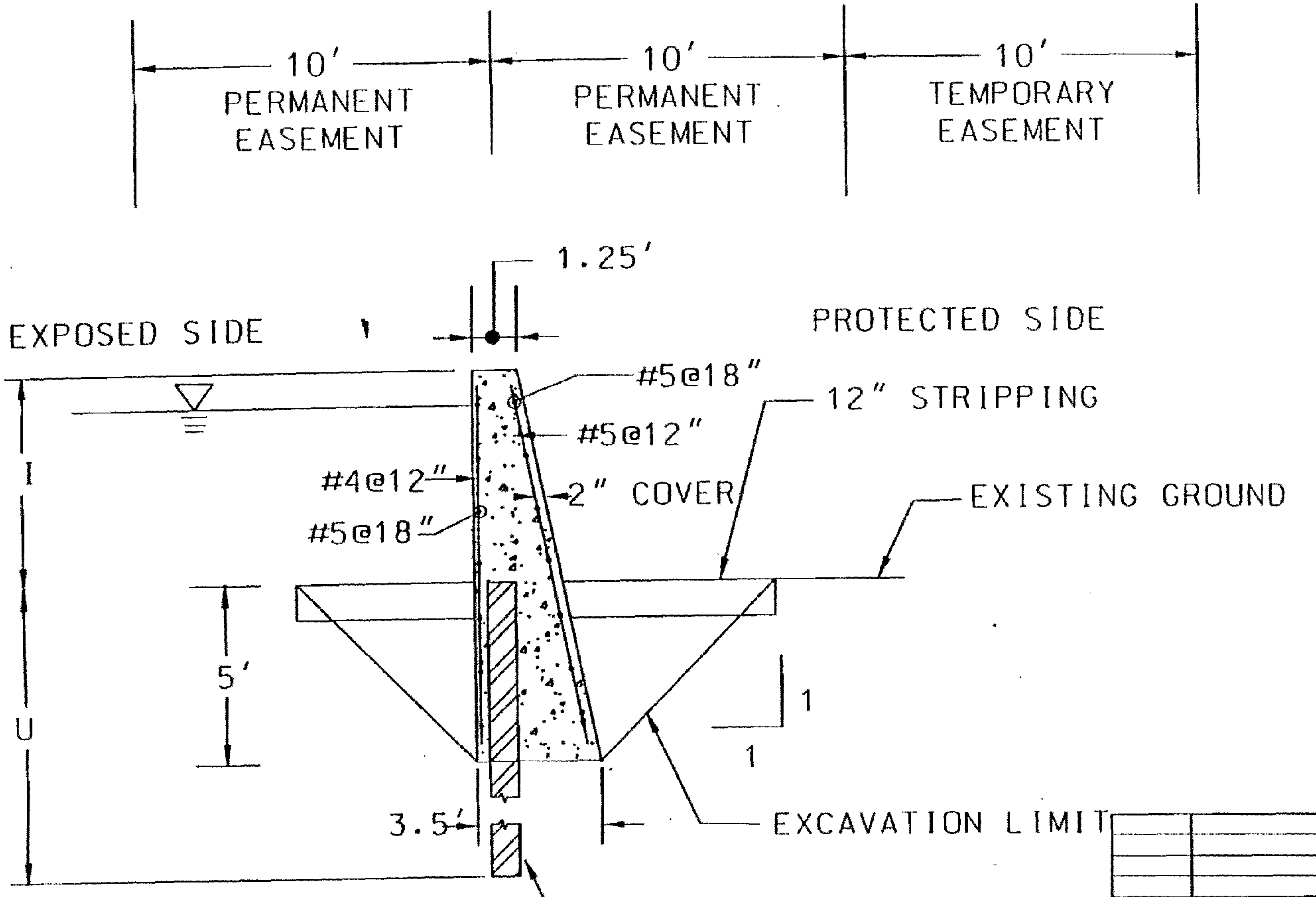
DATE:
SCALE: AS SHOWN
FIG.: A-38



TYPICAL FLOODWALL SECTION

I-S TYPE FLOODWALL

I - HEIGHT OF FLOODWALL ABOVE GRADE

U - LENGTH OF SHEET PILE

Revision	Description	Date	Approv

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
NEW YORK, NEW YORK

Designed by:
Drawn by:
Checked by:
Reviewed by:
SECTION CHIEF
Concurred by:

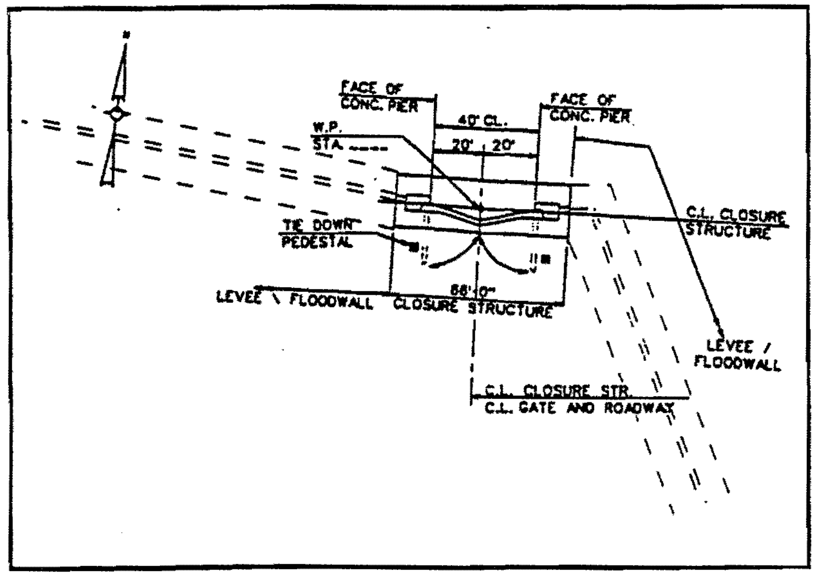
PORT MONMOUTH COMBINED FLOOD CONTROL
AND SHORE PROTECTION FEASIBILITY STUDY

TYPICAL FLOODWALL SECTION

Approval Recommended:
CHIEF, ENGINEERING DIVISION
Approval By:

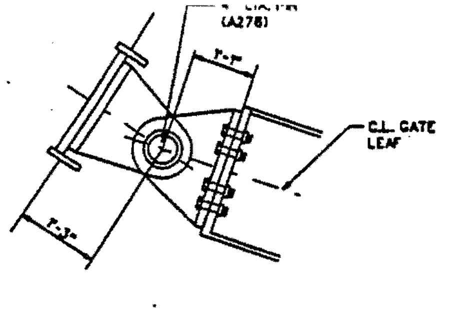
SHEET REFERENCE NUMBER:

DATE:
SCALE: 1" = 5'
FIG :



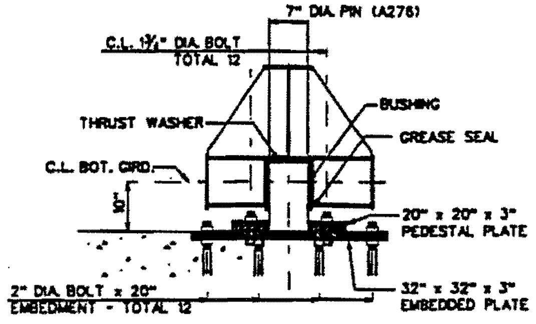
PLAN VIEW

0 10 20 30



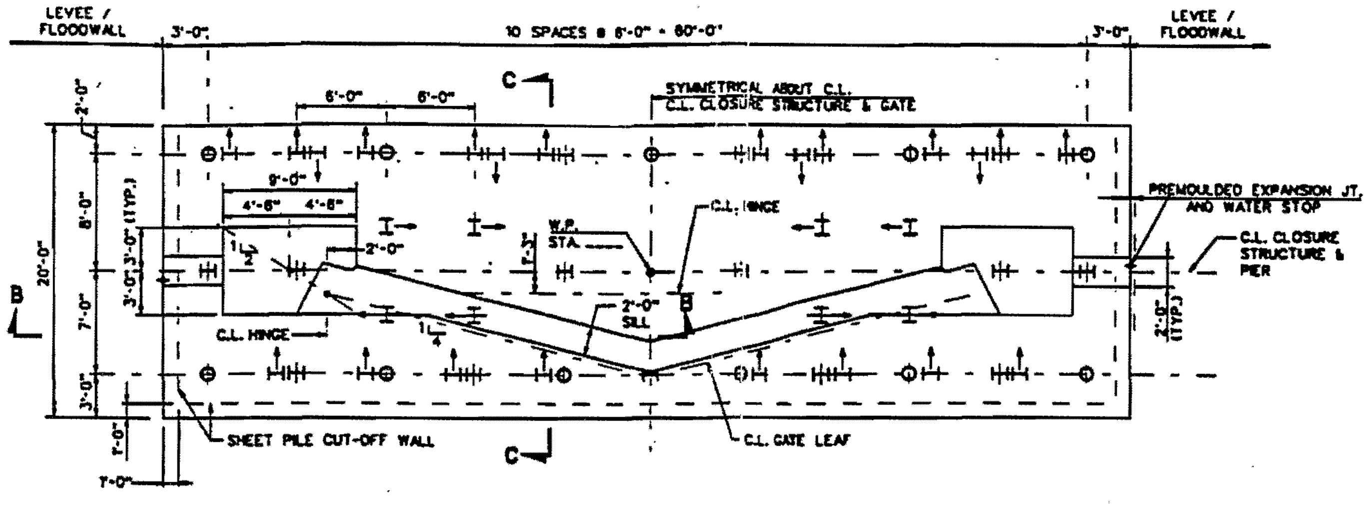
PLAN OF TOP HINGE

NOT TO SCALE



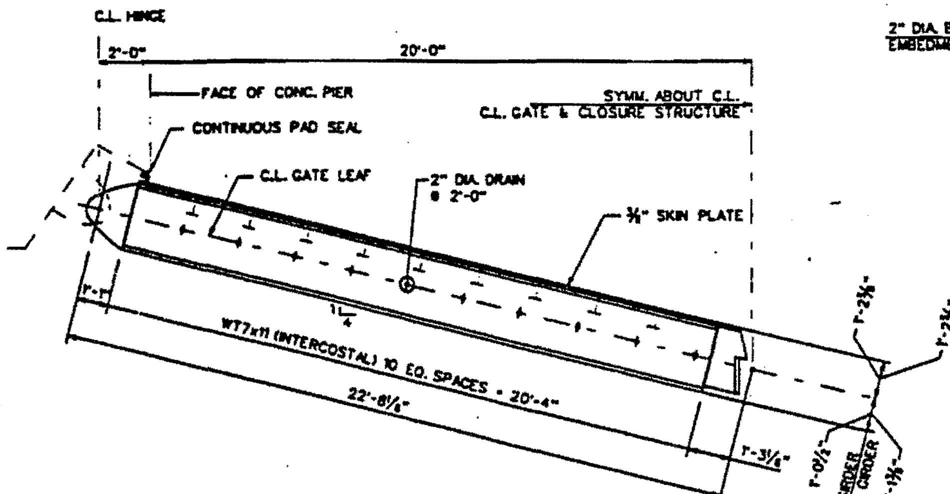
SECTION THRU BOTTOM HINGE

NOT TO SCALE



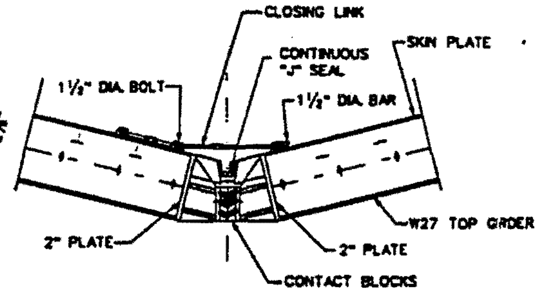
PLAN - SUBSTRUCTURE

0 2 4 6



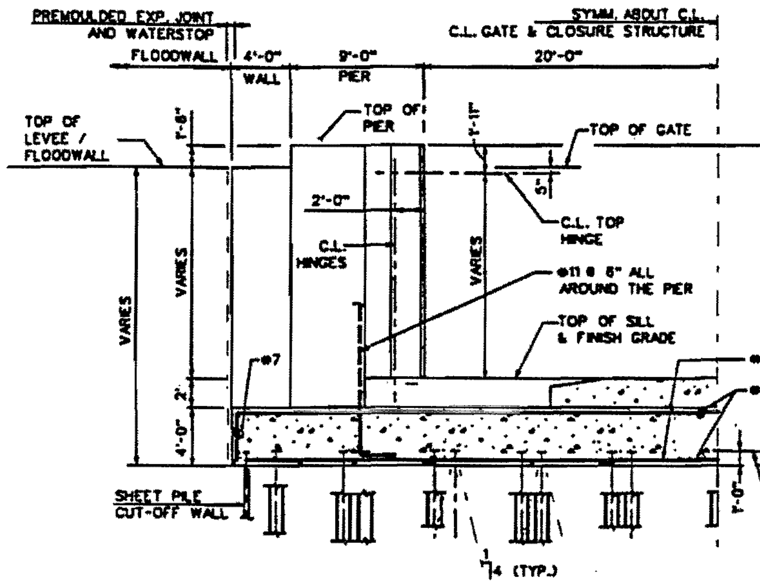
PLAN - GATE LEAF

0 1 2 3



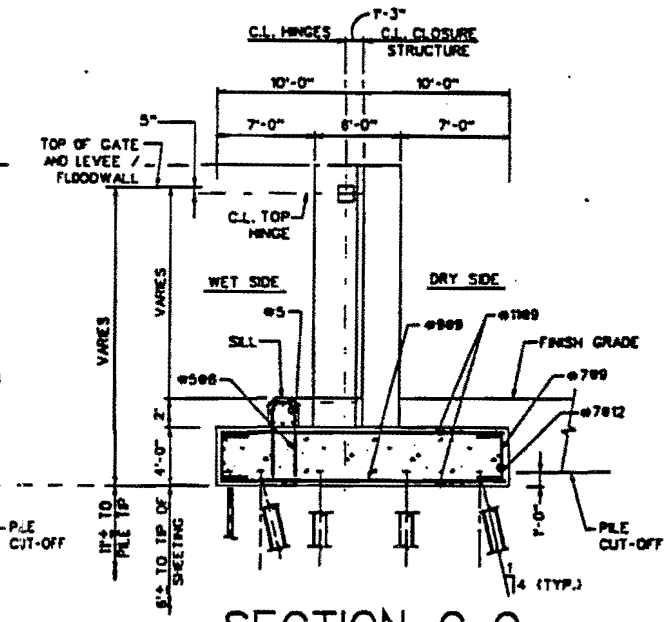
PLAN OF MITER DETAIL

NOT TO SCALE



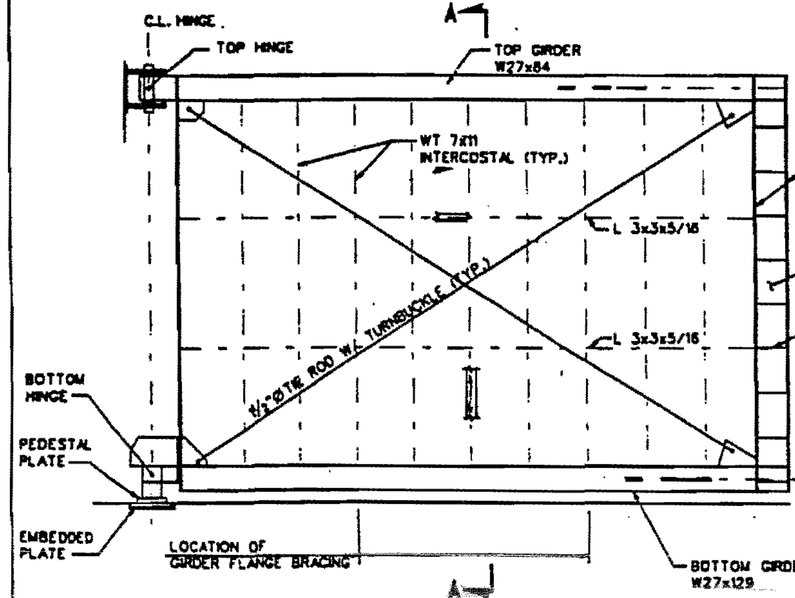
SECTION B-B

0 2 4 6



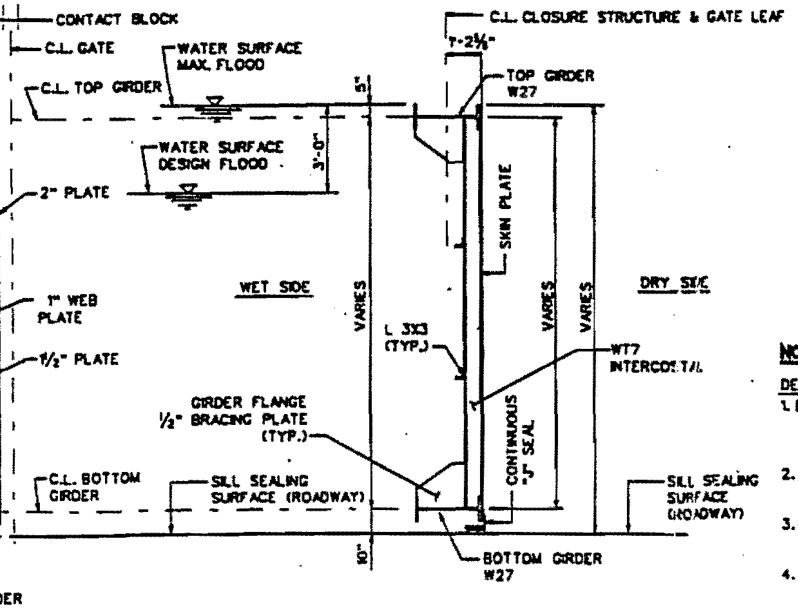
SECTION C-C

0 2 4 6



ELEVATION - GATE LEAF

0 1 2 3



SECTION A-A

0 1 2 3

LEGEND:

○ — ANCHORED PILE (ASTM A252 GRADE 2, 14"Ø x1/2" WALL)
90 TON TENSION AND COMPRESSION

I — PLUMB PILE (ASTM A36, HP 12x63)
90 TON COMPRESSION

I→ — BATTER PILE, BATTERED 1H:4V (ASTM A36, HP 12x63)
90 TON COMPRESSION

LOCATION	EL. 14.00 HEIGHT OF GATE
MARINA ACCESS DRIVE	4.0'
CAMPBELL AVENUE	9.0'
BROADWAY	8.0'

NOTES:

DESIGN CRITERIA:

1. EM 1110-2-2705, 31 MARCH 1994, "STRUCTURAL DESIGN OF CLOSURE STRUCTURE FOR LOCAL FLOOD PROTECTION PROJECTS"
2. EM 1110-2-2105, 31 MARCH 1993, "DESIGN OF HYDRAULIC STEEL STRUCTURES"
3. EM 1110-2-2502, 29 SEPT. 1989, "RETAINING AND FLOOD WALLS"
4. ACI 318 "BUILDING CODE REQUIREMENTS FOR REINFORCED CONCRETE"
5. MANUAL OF STEEL CONSTRUCTION: "ALLOWABLE STRESS DESIGN" 1989, AND "LOAD AND RESISTANCE FACTOR DESIGN" 1994

1. CONCRETE	f'_c=3,000 psi
2. REINFORCED STEEL	f_y =60,000 psi
3. STRUCTURAL STEEL (ASTM A36)	f_y =36,000 psi

HYDRAULIC CRITERIA AND LOAD CASES:

PER EM 1110-2-2705 "STRUCTURE DESIGN OF CLOSURE STRUCTURES FOR LOCAL FLOOD PROTECTION PROJECTS":

CASE I1. DESIGN FLOOD LOADINGS
CASE I2. MAXIMUM FLOOD LOADINGS
CASE I3. EARTHQUAKE LOADING
CASE I4. SHORT DURATION LOADINGS

Revision	Description	Date	Approved

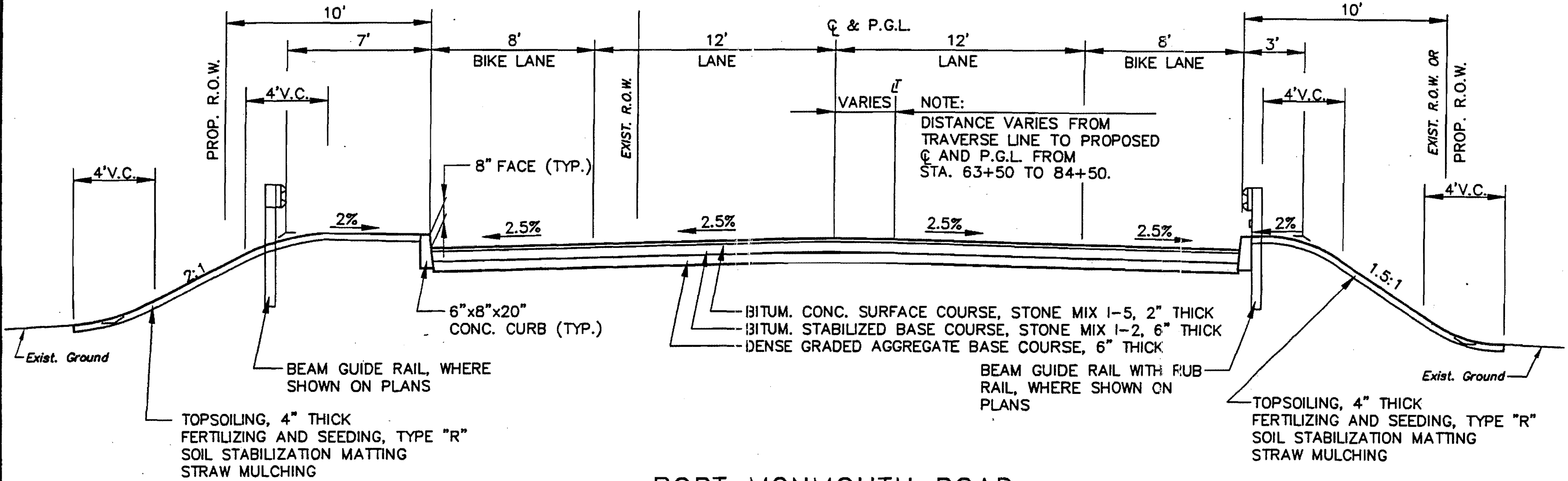
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
NEW YORK, NEW YORK

Designed by:
Drawn by:
Checked by:

PORT MONMOUTH COMBINED FLOOD CONTROL
AND SHORE PROTECTION FEASIBILITY STUDY

TYPICAL MITER GATE DETAILS

Reviewed by:
SECTION CHIEF
Approval Recommended:
CHIEF, ENGINEERING DIVISION
Concurred by:
Approval By:
SHEET REFERENCE NUMBER:
pg.
DATE:
SCALE: AS SHOWN
FIG : A-40



PORT MONMOUTH ROAD

Revision	Description	Date	Approved

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
NEW YORK, NEW YORK

Designed by:
Drawn by:
Checked by:
Reviewed by:
SECTION CHIEF
Concurred by:
CHIEF, DESIGN BRANCH

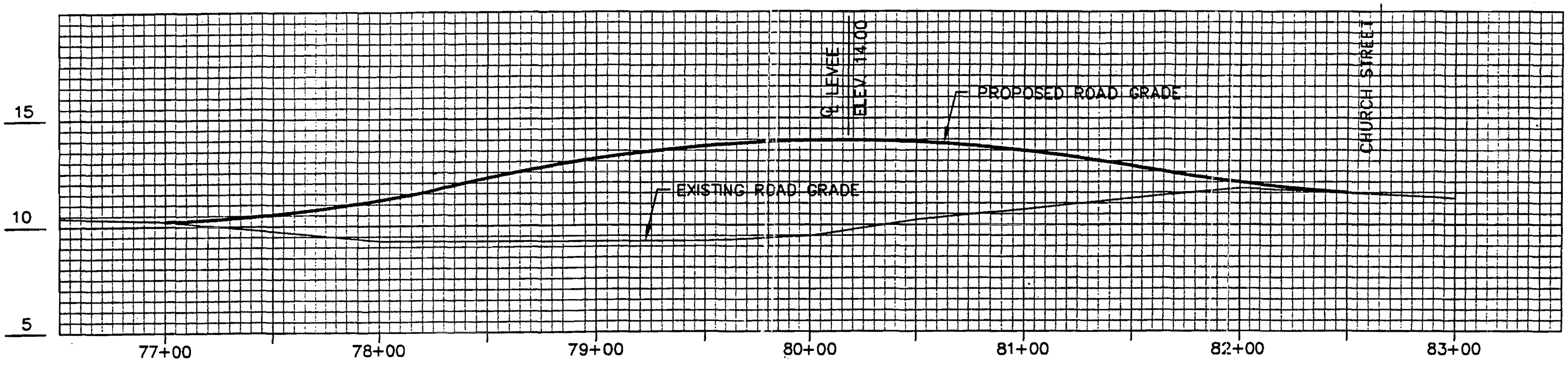
PORT MONMOUTH COMBINED FLOOD CONTROL
AND SHORE PROTECTION FEASIBILITY STUDY
PORT MONMOUTH ROAD TYPICAL SECTION

Approval Recommended:
CHIEF, ENGINEERING DIVISION
Approval By:
COLONEL, C.E. DISTRICT ENGINEER

SHEET REFERENCE NUMBER:

DATE:
SCALE: 1" =5'
FIG.: A 40A

page A-109 A



PROFILE – PORT MONMOUTH ROAD

SCALE: HORIZ. 1" = 50'
VERT. 1" = 5'

Revision	Description	Date	Approved

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
NEW YORK, NEW YORK

Designed by:

Drawn by:

Checked by:

PORT MONMOUTH COMBINED FLOOD CONTROL
AND SHORE PROTECTION FEASIBILITY STUDY
PORT MONMOUTH ROAD PROFILE

Reviewed by:
SECTION CHIEF

Concurred by:
CHIEF, DESIGN BRANCH

Approval Recommended:
CHIEF, ENGINEERING DIVISION

Approval By:
COLONEL, C.E. DISTRICT ENGINEER

SHEET REFERENCE NUMBER:

DATE:

SCALE: HOR. 1" =50'
VER. 1" = 5'

FIG.: A 40B

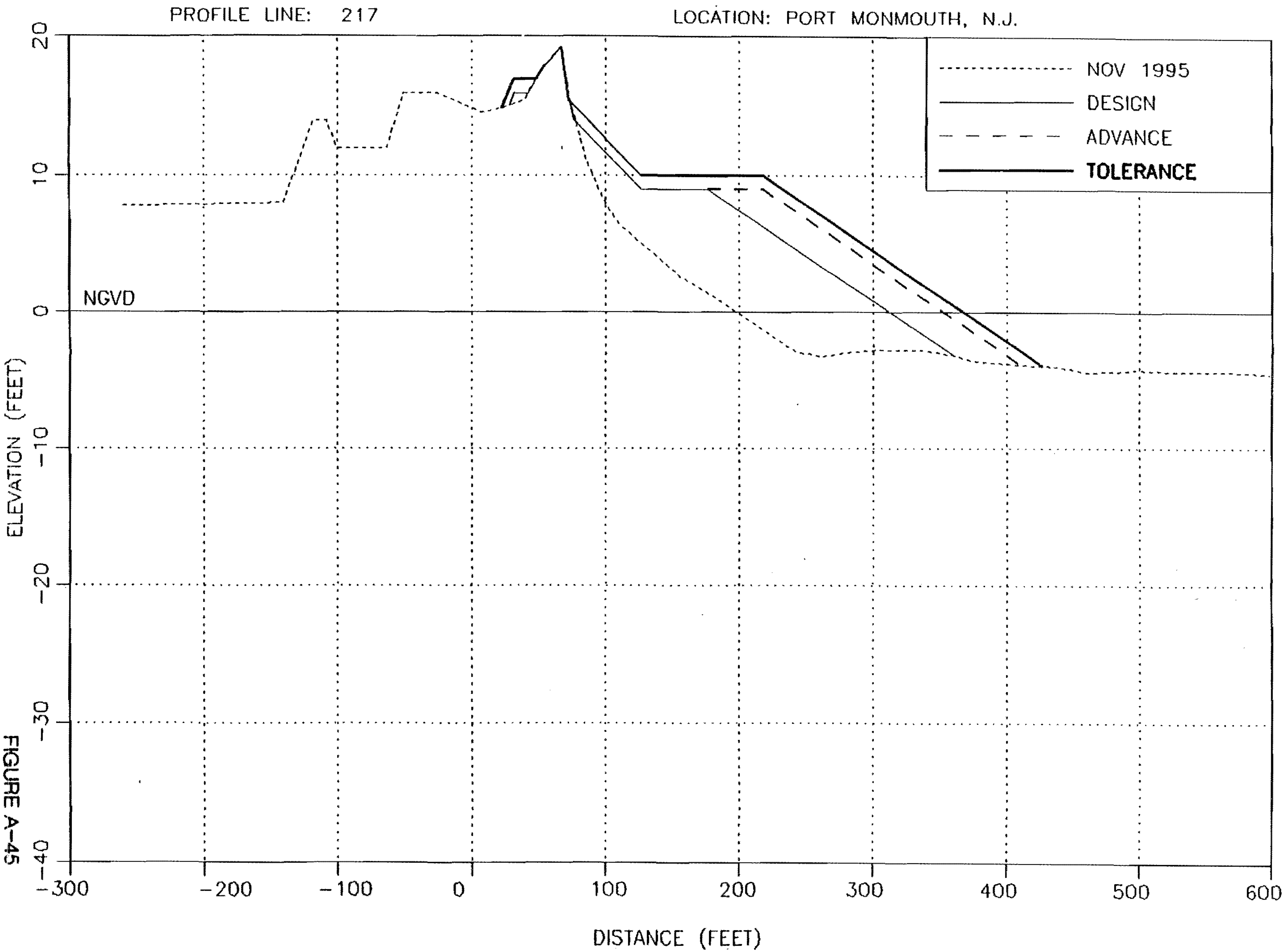
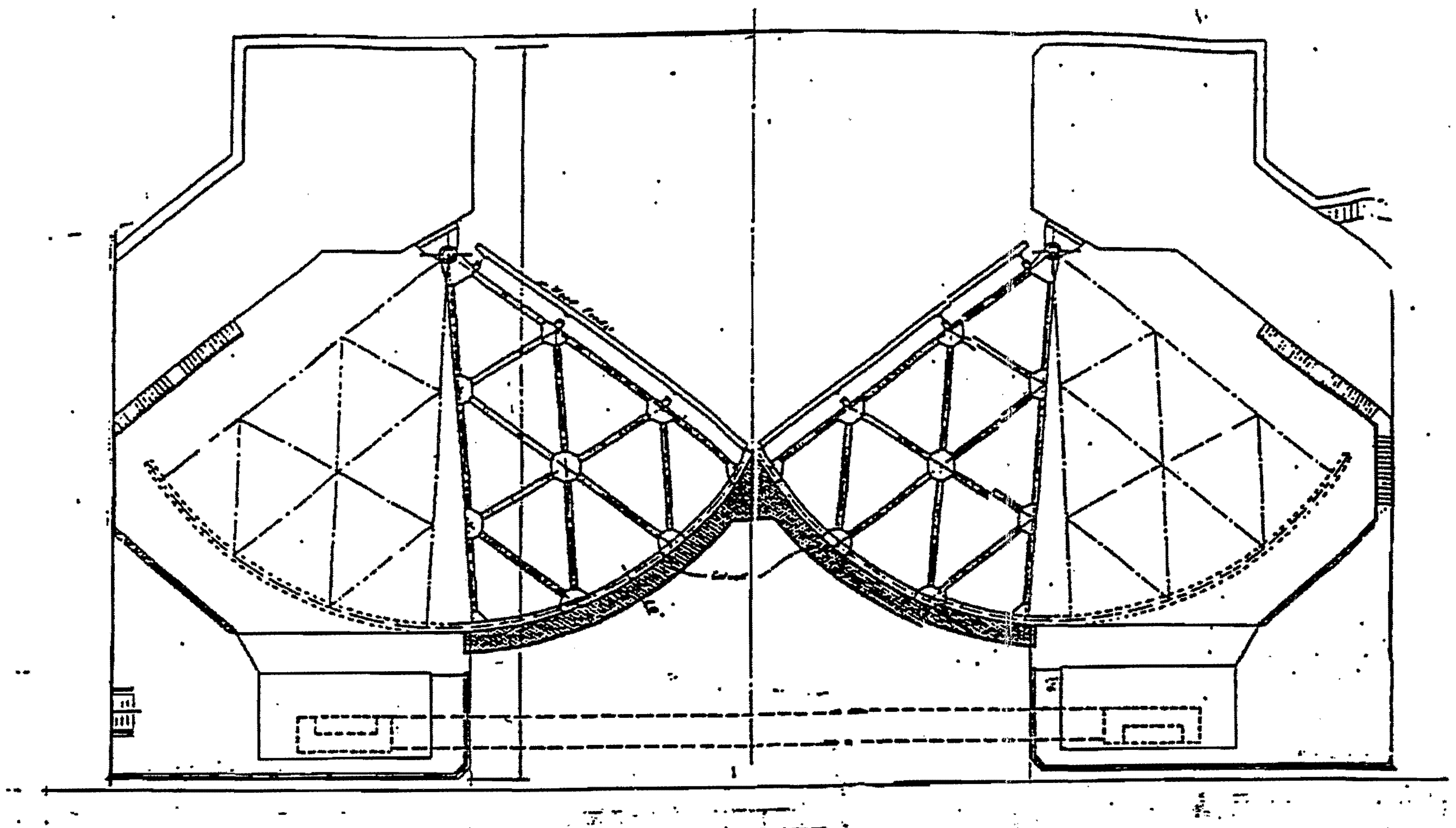
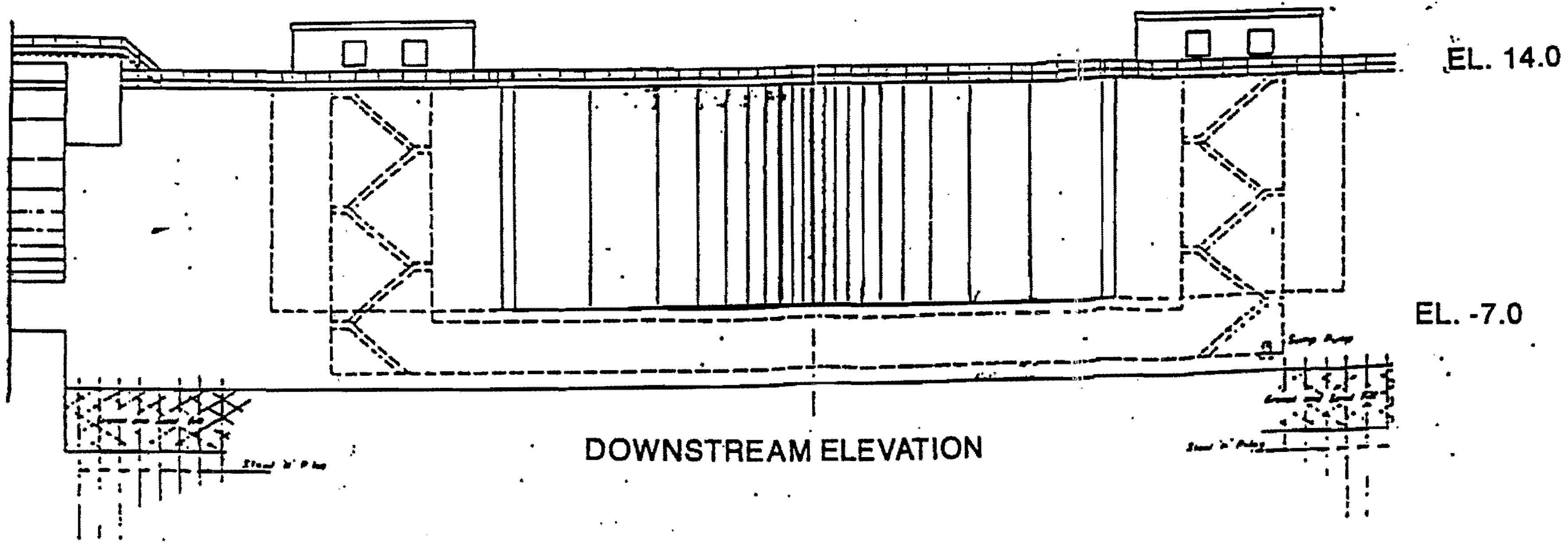

PROFILE LINE: 217
LOCATION: PORT MONMOUTH, N.J.
NOV 1995
DESIGN
ADVANCE
TOLERANCE
NGVD
ELEVATION (FEET)
20
10
0
-10
-20
-30
-40
DISTANCE (FEET)
-300
-200
-100
0
100
200
300
400
500
600

FIGURE A-45



40' GATE

PLAN



DOWNSTREAM ELEVATION

Revision	Description	Date	Approved

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
NEW YORK, NEW YORK

Designed by:

Drawn by:

Checked by:

Reviewed by:

SECTION CHIEF

Concurred by:

CHIEF, DESIGN BRANCH

PORT MONMOUTH COMBINED FLOOD CONTROL
AND SHORE PROTECTION FEASIBILITY STUDY

SECTOR GATE

Approval Recommended:

CHIEF, ENGINEERING DIVISION

Approved by:

COLONEL, C.E. DISTRICT ENGINEER

SHEET REFERENCE NUMBER:
page
A-122

DATE: JUNE, 1998

SCALE:

FIG.: A-49

Compton Creek

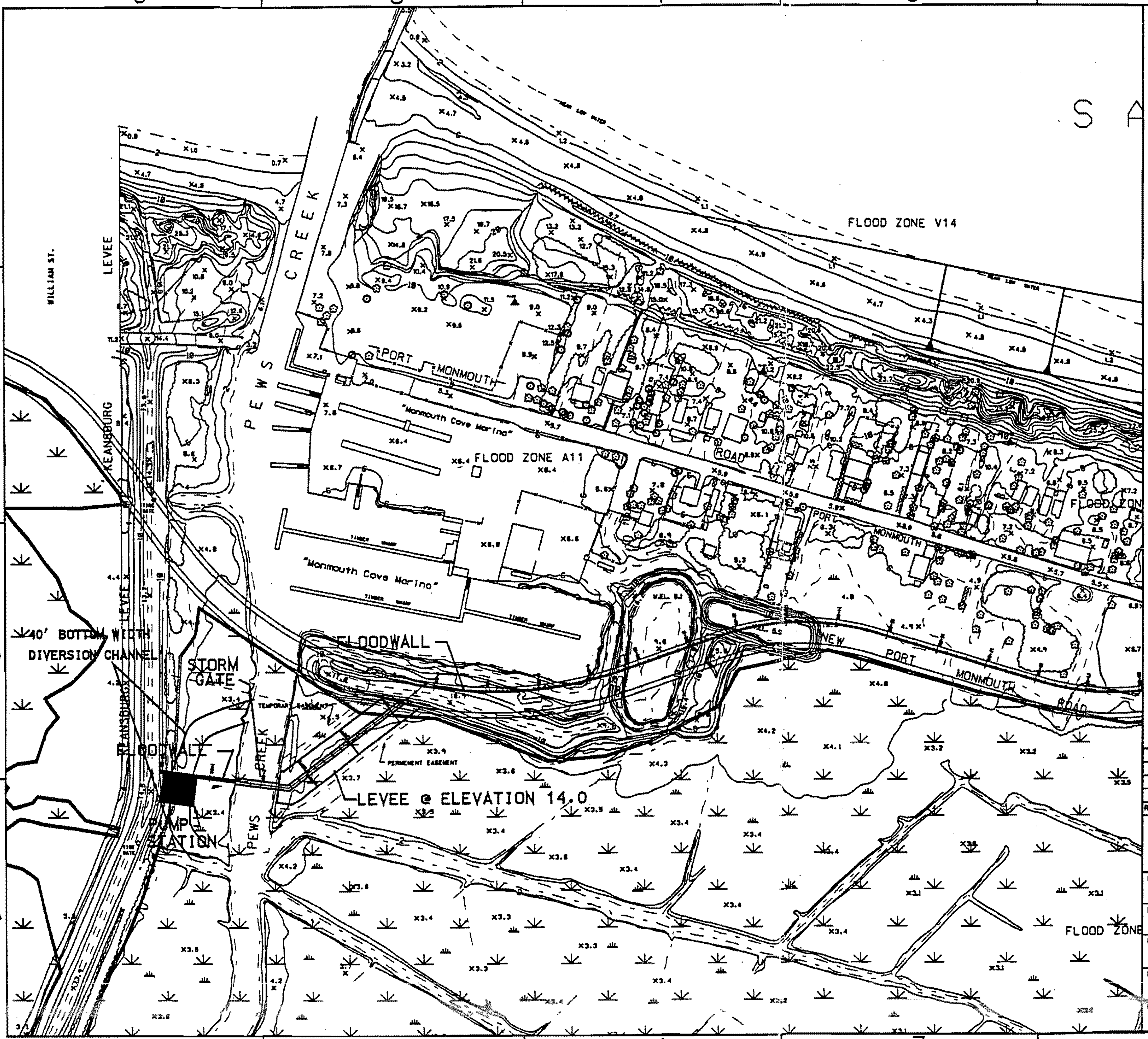
A-261. The alignment for flood protection from Compton Creek starts out as an I-type floodwall approximately 250 feet southeast of the intersection of Wilson Avenue and State Highway 36 and proceeds easterly along the rear property line of the homes fronting on Willow Avenue (Figures A-50 through A-56). This floodwall section will be approximately 1250 feet long and range from one-half to six feet above existing grade (Figures A-32 through A-38). The reason for an I-Type floodwall in this reach is to minimize property acquisition and easement widths.

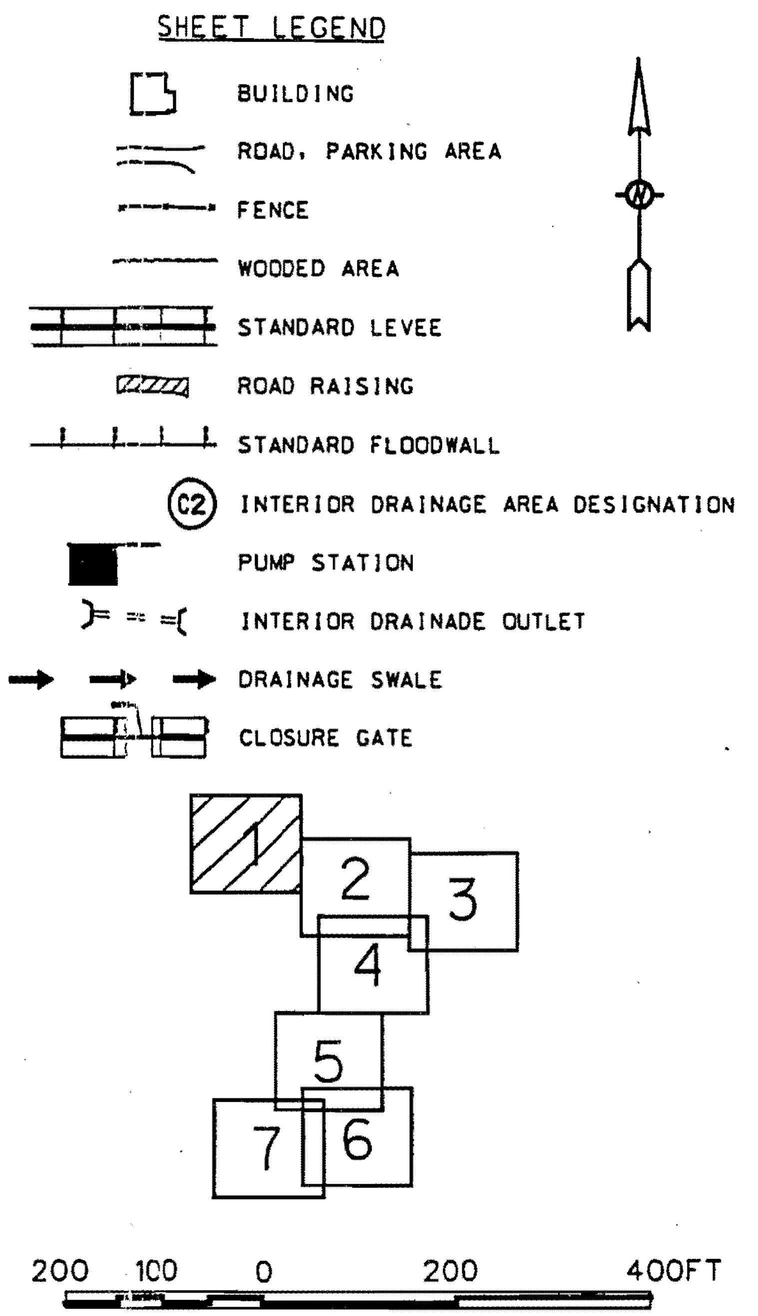
A-262. The alignment converts from an I-type floodwall to a levee and proceeds easterly for about 600 feet where it crosses an existing drainage ditch located between Campbell Avenue and Willow Avenue. The levee then turns north and approaches Campbell Avenue perpendicularly about 100 feet east of the intersection of Campbell Avenue and Creek Road. A mitre gate is proposed for the Campbell Avenue crossing. The gate will be approximately 40 feet wide and 8.5 feet high to provide flood protection to elevation 14 ft. NGVD. A gate is necessary at this location because the road could not be elevated to the design height while maintaining traffic design speeds.

A-263. The levee continues from the Campbell Avenue crossing in a northerly direction through the wetlands nearly parallel to Creek Road for approximately 1,100 feet. The levee height for this section varies between 5 feet and 11 feet above existing grade. The selected levee alignment has been positioned to minimize impact to the wetlands, as much as is practical by locating it close to the developed area immediately west of the alignment. The levee makes a turn towards the northeast, paralleling Woodstock Avenue for 400 feet then changes direction northward for 800 feet to meet Broadway about 100 feet east of the intersection of Main Street and Broadway.

A-264. A mitre gate is proposed to span across Broadway. The gate will be approximately 40 feet wide and 8 feet high to provide flood protection elevation to 14 ft. NGVD.

A-265. The alignment continues as a levee in a northeasterly direction paralleling Main Street for about 2,000 feet before it changes course and heads east for approximately 700 feet along the rear property lines of the homes which front on Park Avenue. The average levee height for this 2,700 foot section is about 10.5 feet above existing grade. The levee then proceeds northerly meeting Port Monmouth Road about 800 feet southeast of the intersection of Main Street and Port Monmouth Road. Port Monmouth Road will have to be elevated to the design elevation in the area where the levee meets the road. The levee picks up again at the north side of Port Monmouth Road and proceeds north towards the bayfront where it will tie into the design dune.





REVISION	DESCRIPTION	DATE	APPROVED

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
NEW YORK, NEW YORK

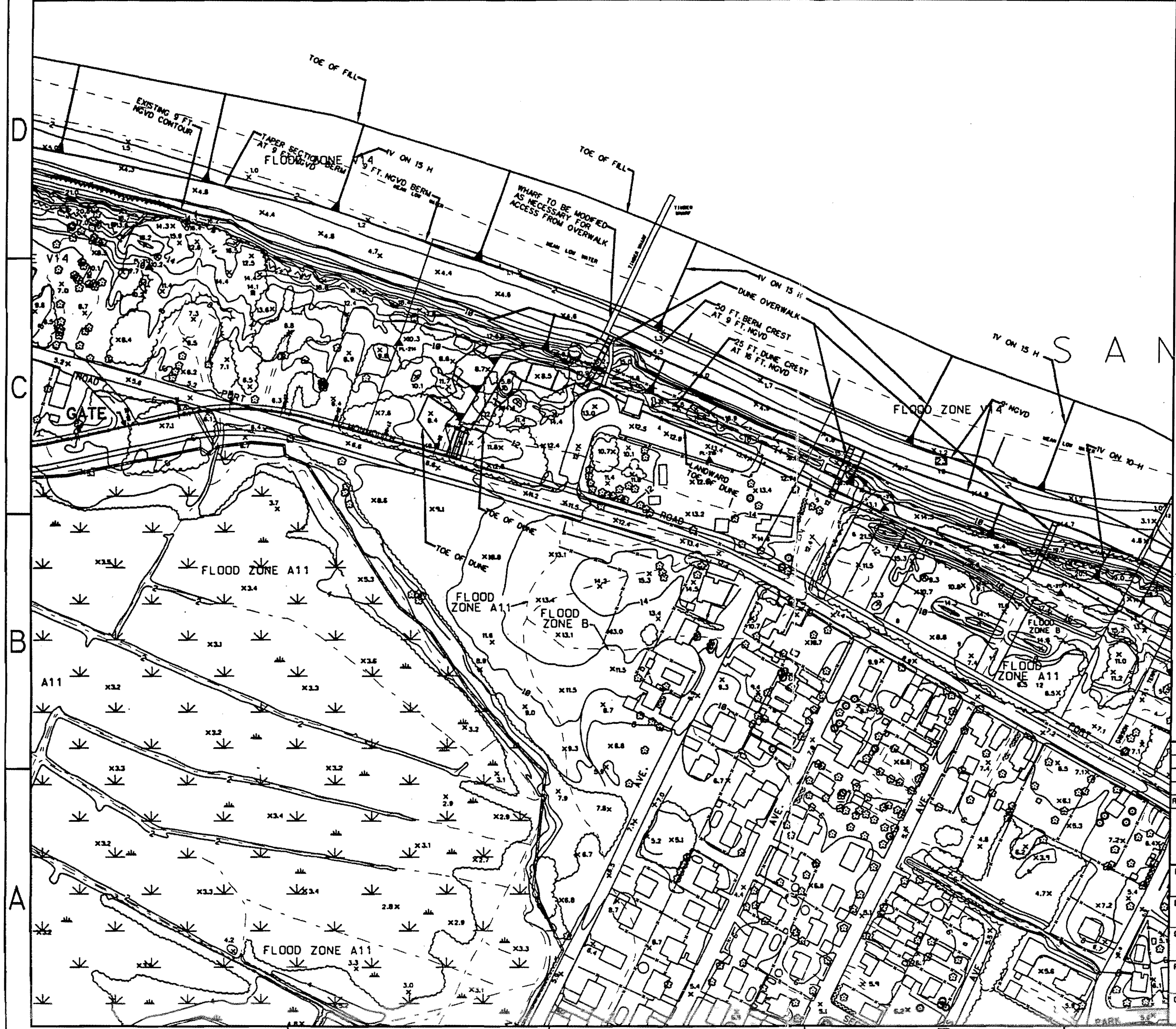
DESIONED BY:
DRAWN BY:
CHECKED BY:
REVIEWED BY:
SECTION CHIEF
CONCURRED BY:
CHIEF, DESIGN BRANCH

PORT MONMOUTH COMBINED FLOOD CONTROL
AND SHORE PROTECTION FEASIBILITY STUDY
LEVEL OF PROTECTION @ ELEVATION 14.0

APPROVAL RECOMMENDED:
CHIEF, ENGINEERING DIVISION
APPROVAL BY:
COLONEL, C.E. DISTRICT ENGINEER

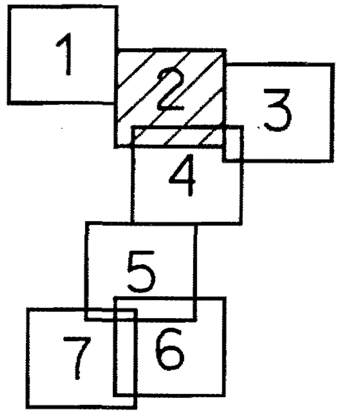
SHEET REFERENCE NUMBER: page A-124

DATE:
SCALE: 1" = 200'
FIG.: A-50



SHEET LEGEND

⌐	BUILDING
═	ROAD, PARKING AREA
—·—·—	FENCE
——	WOODED AREA
═╪═	STANDARD LEVEE
▨	ROAD RAISING
┴┴┴	STANDARD FLOODWALL
(C2)	INTERIOR DRAINAGE AREA DESIGNATION
■	PUMP STATION
}= = =[	INTERIOR DRAINADE OUTLET
→ → →	DRAINAGE SWALE
═╬═	CLOSURE GATE



200 100 0 200 400FT

REVISION	DESCRIPTION	DATE	APPROVED

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
NEW YORK, NEW YORK

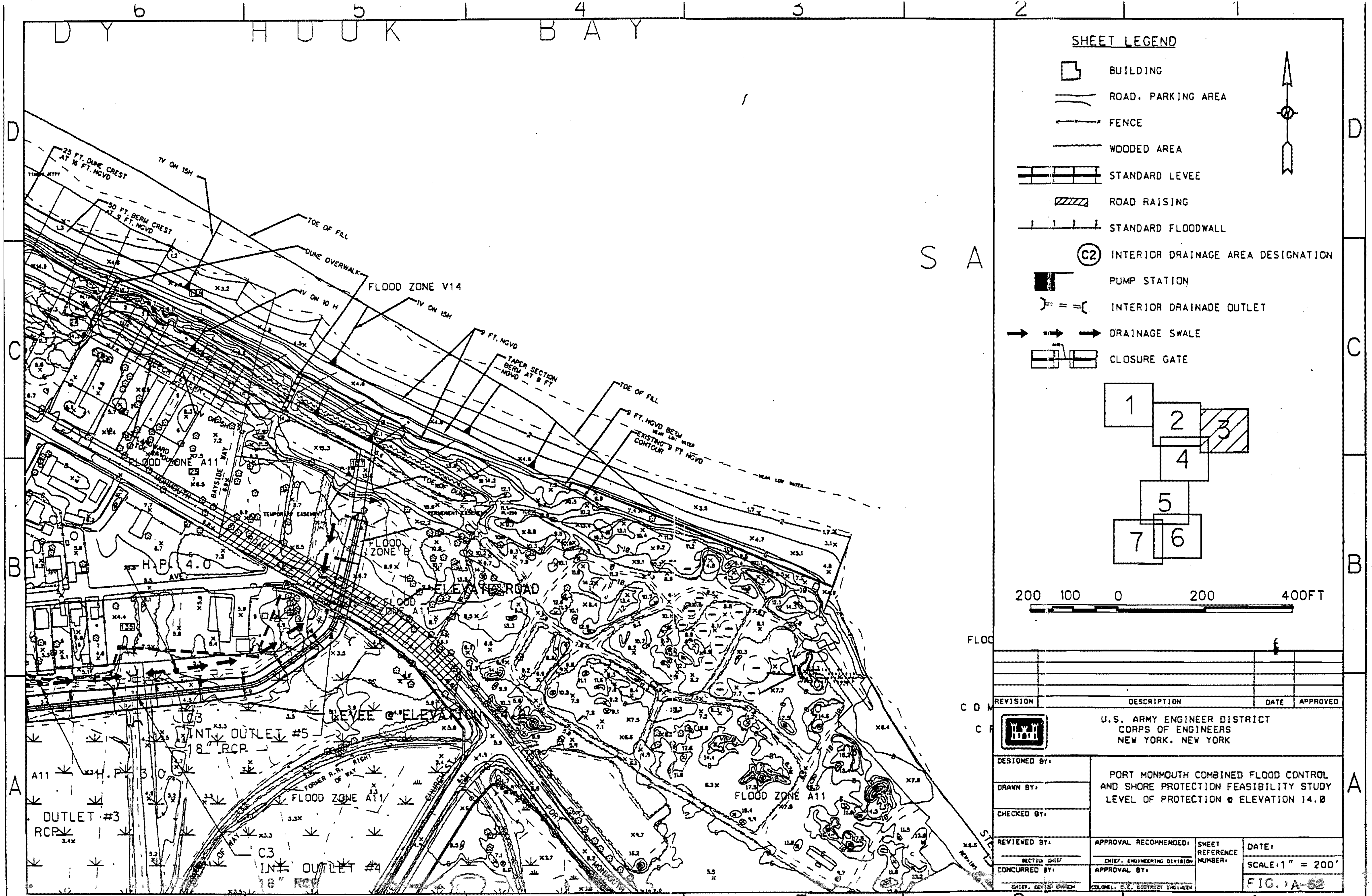
DESIGNED BY:
DRAWN BY:
CHECKED BY:
REVIEWED BY:
SECTION CHIEF
CONCURRED BY:
CHIEF, DESIGN BRANCH

PORT MONMOUTH COMBINED FLOOD CONTROL
AND SHORE PROTECTION FEASIBILITY STUDY
LEVEL OF PROTECTION @ ELEVATION 14.0

APPROVAL RECOMMENDED:
CHIEF, ENGINEERING DIVISION
APPROVAL BY:
COLONEL, C.E. DISTRICT ENGINEER

SHEET REFERENCE NUMBER:

DATE:
SCALE: 1" = 200'
FIG.: ~~A-51~~



REVISION	DESCRIPTION	DATE	APPROVED

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
NEW YORK, NEW YORK

DESIONED BY:

DRAWN BY:

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REVIEWED BY:
SECTION CHIEF

CONCURRED BY:
CHIEF, DESIGN BRANCH

PORT MONMOUTH COMBINED FLOOD CONTROL AND SHORE PROTECTION FEASIBILITY STUDY
LEVEL OF PROTECTION @ ELEVATION 14.0

APPROVAL RECOMMENDED:
CHIEF, ENGINEERING DIVISION

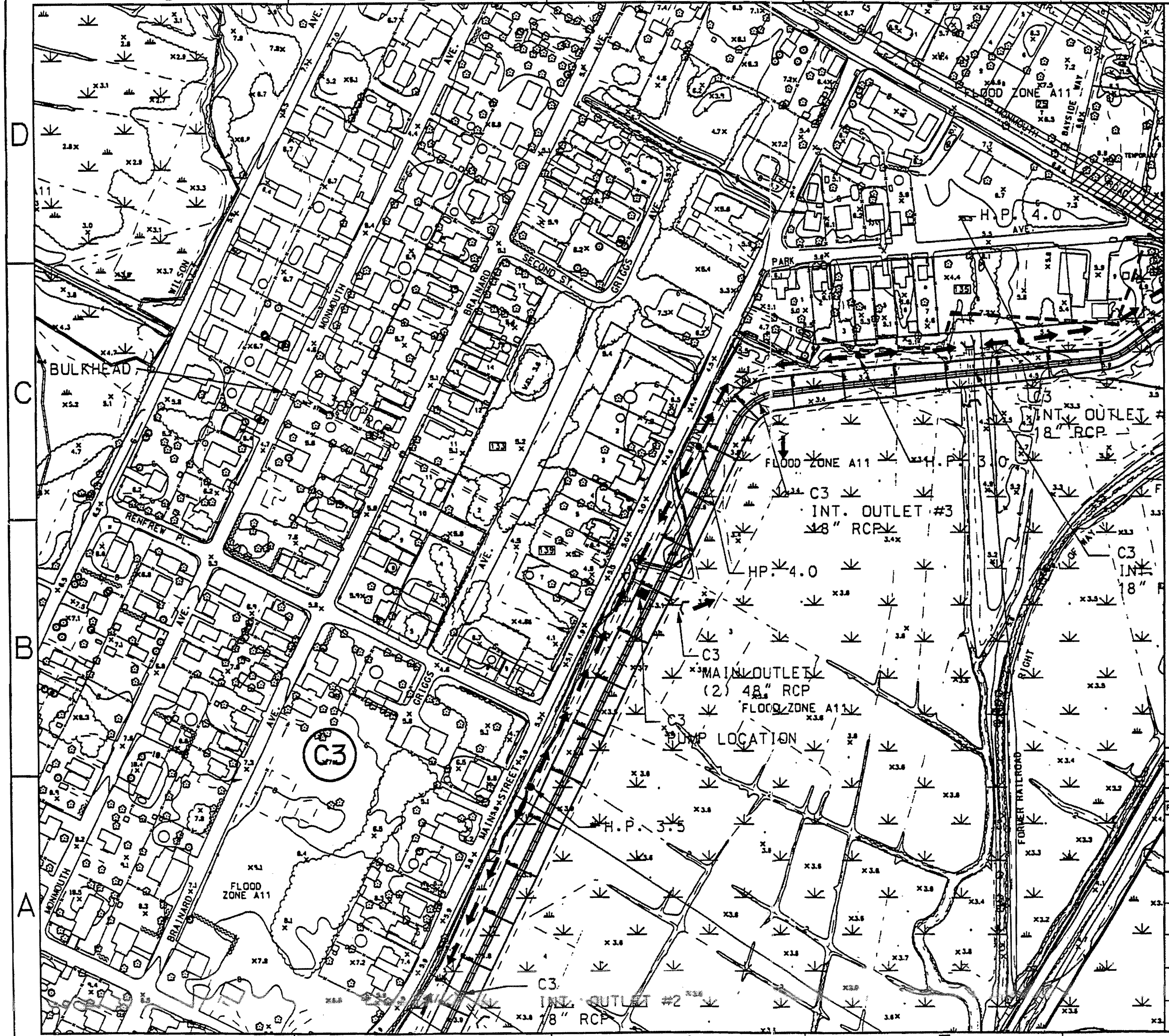
APPROVAL BY:
COLONEL, C.E. DISTRICT ENGINEER

SHEET REFERENCE NUMBER:

DATE:

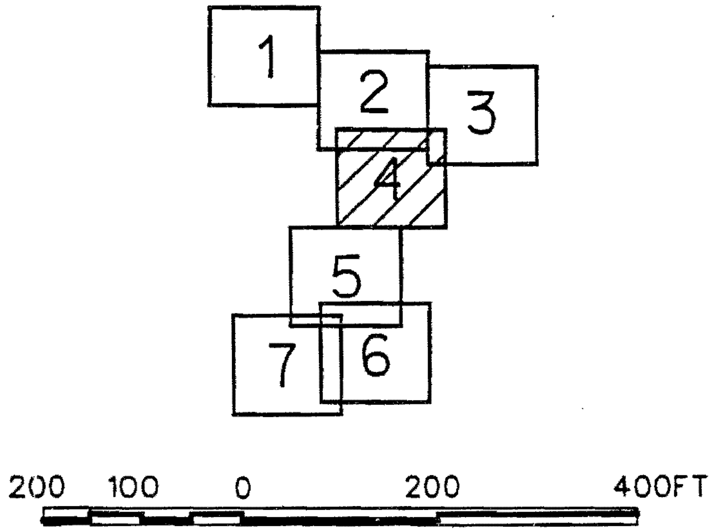
SCALE: 1" = 200'

FIG. : A-52



SHEET LEGEND

- BUILDING
- ROAD, PARKING AREA
- FENCE
- WOODED AREA
- STANDARD LEVEE
- ROAD RAISING
- STANDARD FLOODWALL
- (C2) INTERIOR DRAINAGE AREA DESIGNATION
- PUMP STATION
- INTERIOR DRAINADE OUTLET
- DRAINAGE SWALE
- CLOSURE GATE



REVISION	DESCRIPTION	DATE	APPROVED

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
NEW YORK, NEW YORK

DESIGNED BY:

DRAWN BY:

CHECKED BY:

REVIEWED BY:
SECTION CHIEF

CONCURRED BY:
CHIEF, DESIGN BRANCH

PORT MONMOUTH COMBINED FLOOD CONTROL
AND SHORE PROTECTION FEASIBILITY STUDY
LEVEL OF PROTECTION @ ELEVATION 14.0

APPROVAL RECOMMENDED:
CHIEF, ENGINEERING DIVISION

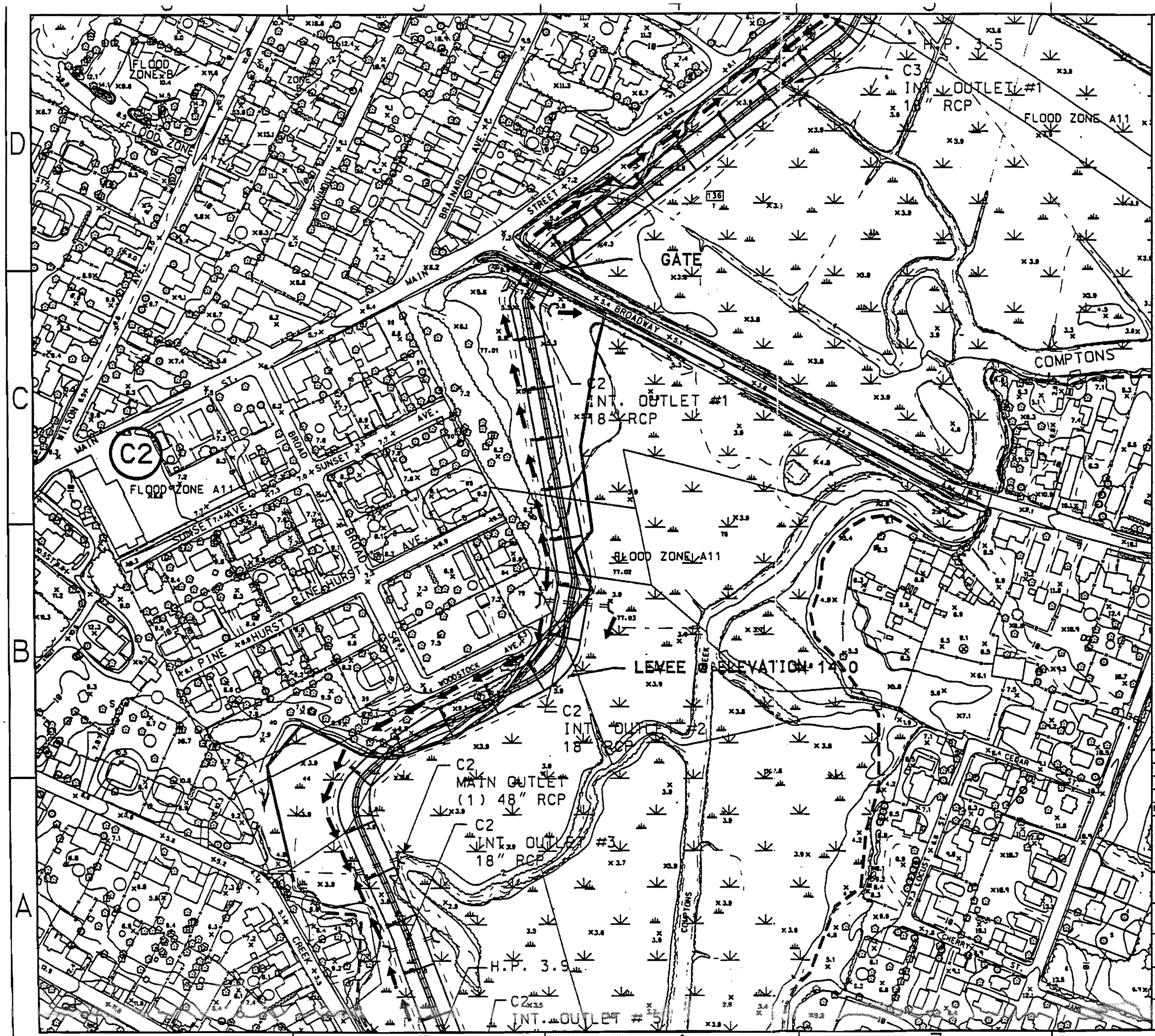
APPROVAL BY:
COLONEL, C.E. DISTRICT ENGINEER

SHEET REFERENCE NUMBER: page A-127

DATE:

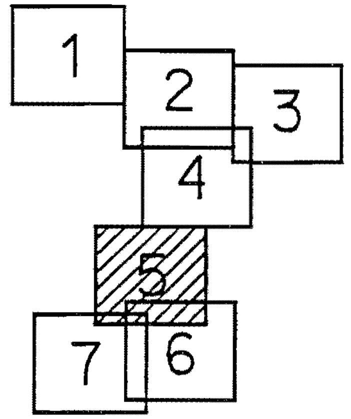
SCALE: 1" = 200'

FIG. : A-53



SHEET LEGEND

- BUILDING
- ROAD, PARKING AREA
- FENCE
- WOODED AREA
- STANDARD LEVEE
- ROAD RAISING
- STANDARD FLOODWALL
- Ⓒ2 INTERIOR DRAINAGE AREA DESIGNATION
- PUMP STATION
- INTERIOR DRAINADE OUTLET
- DRAINAGE SWALE
- CLOSURE GATE



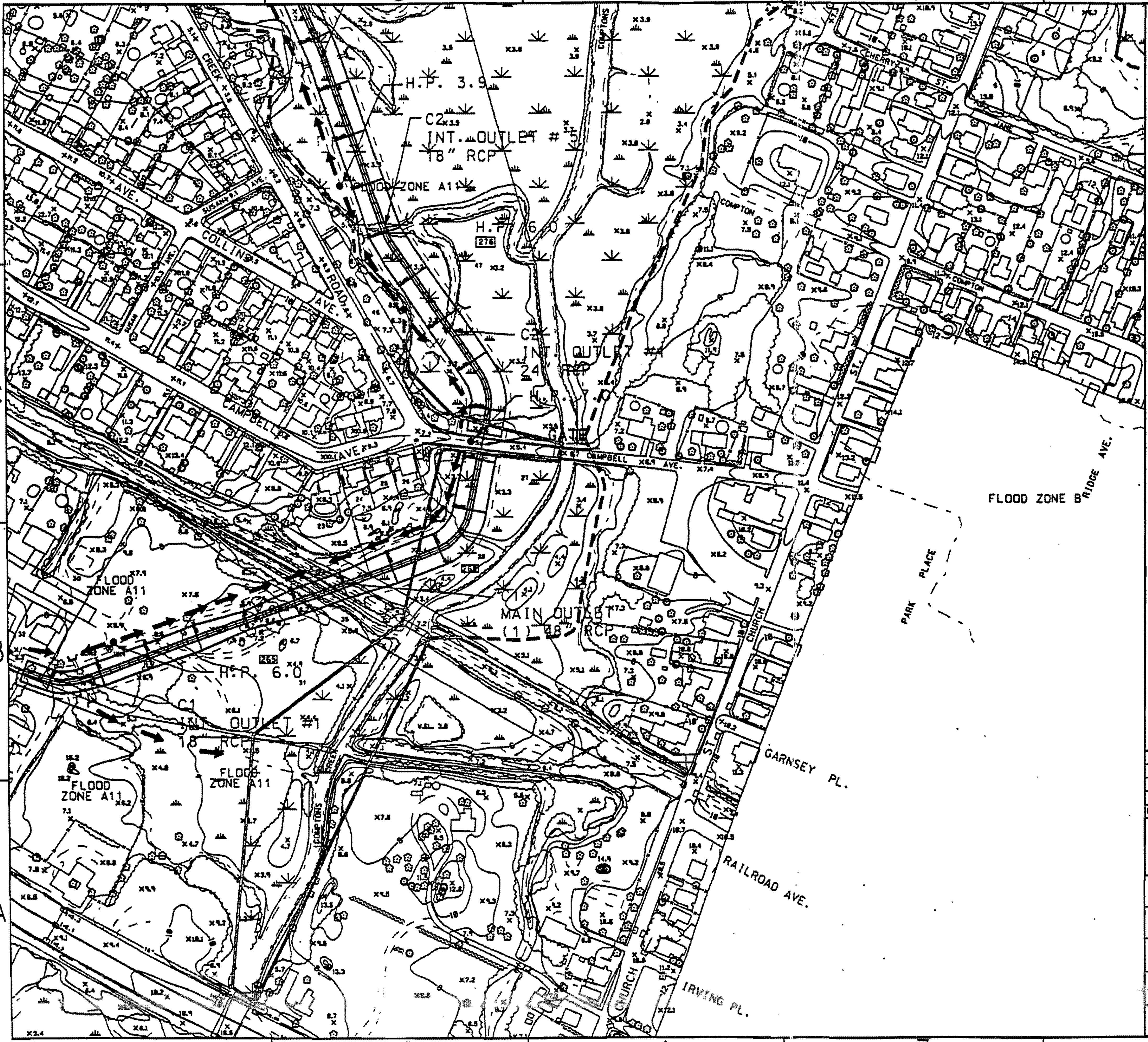
200 100 0 200 400FT

REVISION	DESCRIPTION	DATE	APPROVED

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
NEW YORK, NEW YORK

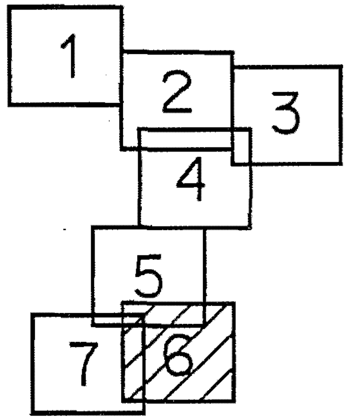
PORT MONMOUTH COMBINED FLOOD CONTROL
AND SHORE PROTECTION FEASIBILITY STUDY
LEVEL OF PROTECTION @ ELEVATION 14.0

DESIGNED BY:
DRAWN BY:
CHECKED BY:
REVIEWED BY:
SECTION CHIEF
CONCURRED BY:
CHIEF, DESIGN BRANCH
APPROVAL RECOMMENDED:
CHIEF, ENGINEERING DIVISION
APPROVAL BY:
COLONEL, C.E. DISTRICT ENGINEER
SHEET REFERENCE NUMBER:
DATE:
SCALE: 1" = 200'
FIG.: A-54



SHEET LEGEND

- BUILDING
- ROAD, PARKING AREA
- FENCE
- WOODED AREA
- STANDARD LEVEE
- ROAD RAISING
- STANDARD FLOODWALL
- (C2) INTERIOR DRAINAGE AREA DESIGNATION
- PUMP STATION
- INTERIOR DRAINADE OUTLET
- DRAINAGE SWALE
- CLOSURE GATE



200 100 0 200 400FT

REVISION	DESCRIPTION	DATE	APPROVED

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
NEW YORK, NEW YORK

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REVIEWED BY:
SECTION CHIEF

CONCURRED BY:
CHIEF, DESIGN BRANCH

PORT MONMOUTH COMBINED FLOOD CONTROL
AND SHORE PROTECTION FEASIBILITY STUDY
LEVEL OF PROTECTION @ ELEVATION 14.0

APPROVAL RECOMMENDED:
CHIEF, ENGINEERING DIVISION

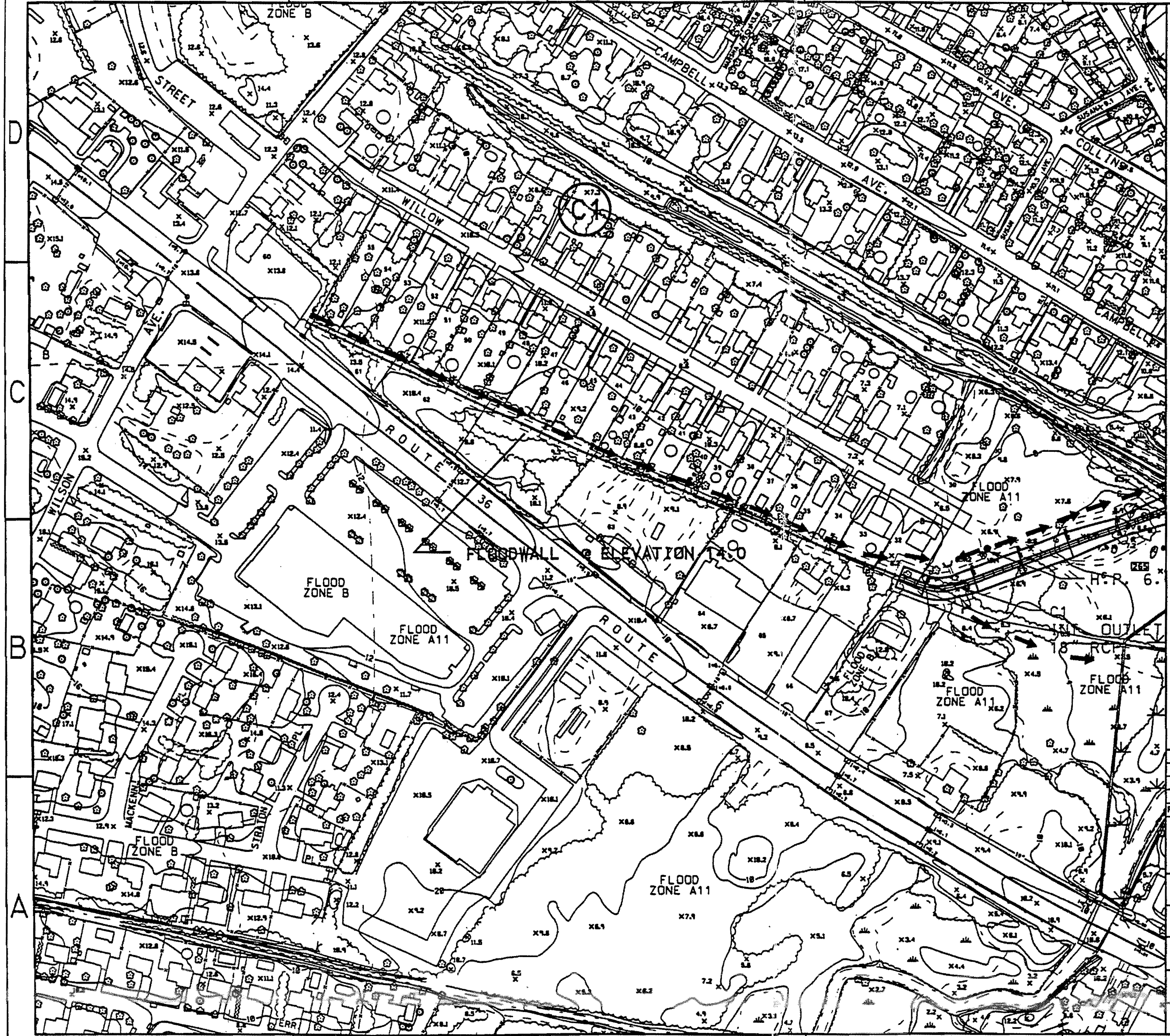
APPROVAL BY:
COLONEL, C.E. DISTRICT ENGINEER

SHEET REFERENCE NUMBER:

DATE:

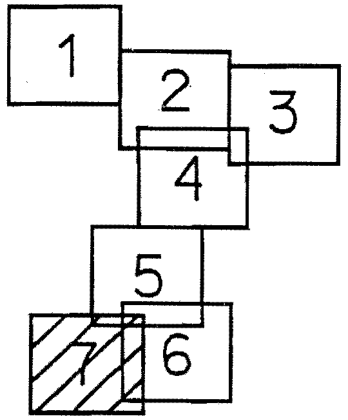
SCALE: 1" = 200'

FIG.: A-55



SHEET LEGEND

- BUILDING
- ROAD, PARKING AREA
- FENCE
- WOODED AREA
- STANDARD LEVEE
- ROAD RAISING
- STANDARD FLOODWALL
- (C2) INTERIOR DRAINAGE AREA DESIGNATION
- PUMP STATION
- INTERIOR DRAINADE OUTLET
- DRAINAGE SWALE
- CLOSURE GATE



200 100 0 200 400FT

REVISION	DESCRIPTION	DATE	APPROVED

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
NEW YORK, NEW YORK

PORT MONMOUTH COMBINED FLOOD CONTROL
AND SHORE PROTECTION FEASIBILITY STUDY
LEVEL OF PROTECTION @ ELEVATION 14.0

DESIGNED BY:
DRAWN BY:
CHECKED BY:
REVIEWED BY:
SECTION CHIEF
CONCURRED BY:
CHIEF, DESIGN BRANCH
APPROVAL RECOMMENDED:
CHIEF, ENGINEERING DIVISION
APPROVAL BY:
COLONEL, C.E. DISTRICT ENGINEER
SHEET REFERENCE NUMBER:
DATE:
SCALE: 1" = 200'
FIG. A-56

Bayshore

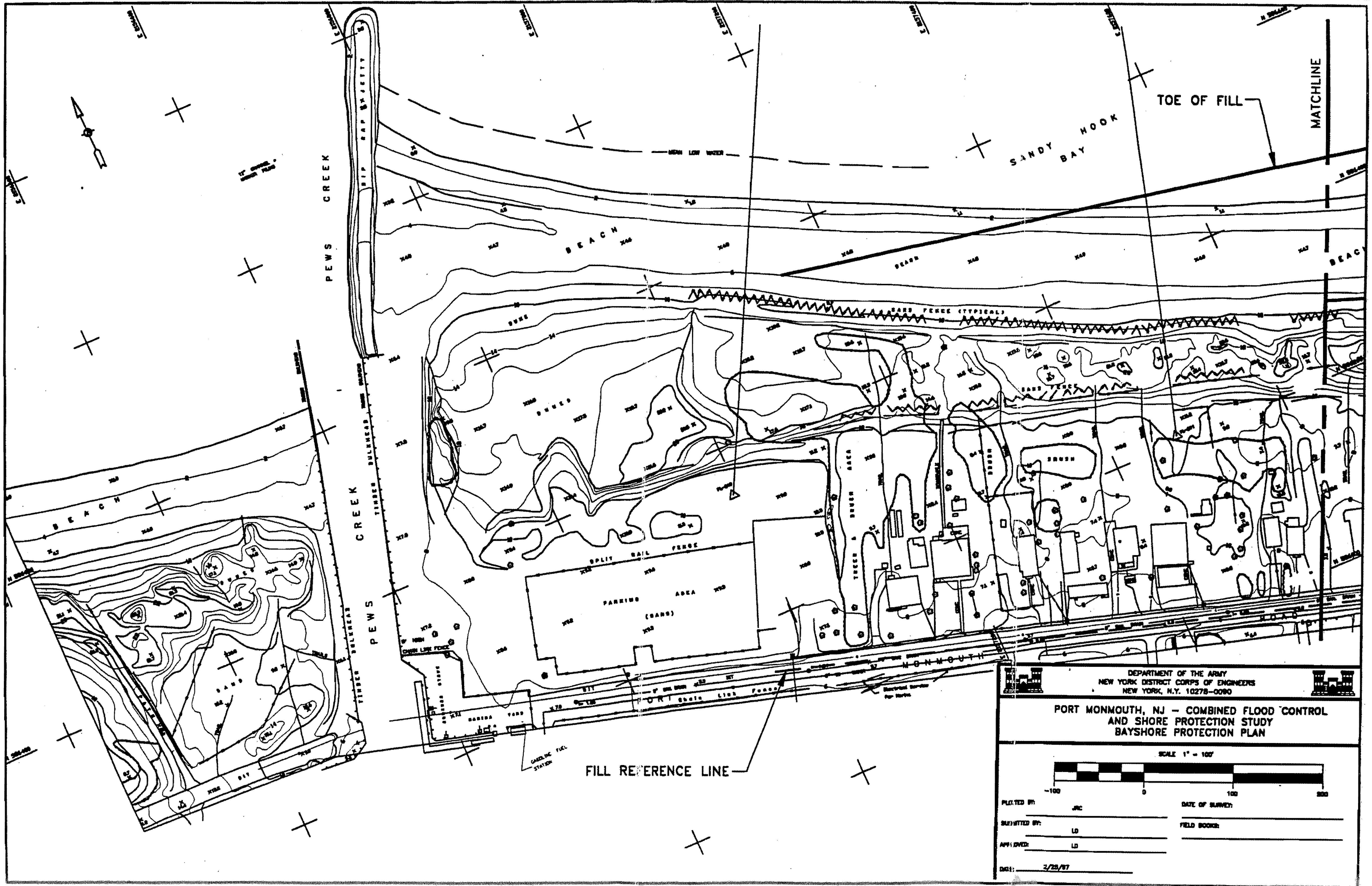
A-266. The selected plan for flood protection along the project's bay shoreline consists of a reconstructed dune to reinforce the existing dune (Figures A-57 through A-60). Dune integrity will be ensured by periodic nourishment beginning approximately 10 years after initial construction.

A-267. The eastern limit of the relocated dune ties into the Compton Creek levee alignment near the intersection of Park Avenue and Port Monmouth Road (beach profile PL-219). From its eastern terminus, the dune extends approximately 2,300 feet to the west, beyond the Spy House parking area. The dune layout is based on the location of the existing seaward-most dune crest, which is used as a project baseline. In order to minimize the fill volume, the design dune is laid out such that its seaward dune crest is placed 15 feet landward of the project baseline. This layout ensures that the design is generally placed seaward of the "in shore limit of fill" identified in the design for the 1966 dune construction project.

Pews Creek

A-268. From the terminus of the new dune approximately 700 feet northwest of the intersection of Wilson Avenue and Port Monmouth Road a levee section will span between the dune and Port Monmouth Road. This levee section is to abut the beginning of the proposed floodwall along the north side of Port Monmouth Road. This floodwall will be approximately four feet high at a design elevation of 14 ft. NGVD. The alignment continues westerly as a floodwall along the northern side of Port Monmouth Road for about 700 feet until it reaches the intersecting ramp to the Monmouth County Marina. A closure structure approximately 40 feet long by 4 feet high will bridge this gap. The alignment then continues as a floodwall westward along the north side of Port Monmouth Road for about 1,000 feet where it will meet a recently constructed steel bulkhead between Port Monmouth Road and the County Marina. The bulkhead is to be incorporated into an I-Type floodwall with the steel sheet pile already in place. This bulkhead will be retrofitted to accept the floodwall up to a point perpendicular to an area of the new Port Monmouth Road which is at or above the design height. A transition earthen section will be placed between the floodwall and the roadway to bridge the gap. Placement of the floodwall along the north side of Port Monmouth Road will allow the roadway to remain accessible during floods and provide access from East Keansburg to Port Monmouth or vice versa.

A-269. The alignment incorporates a section of Port Monmouth Road which is at or above the design height and connects to a proposed levee south of new Port Monmouth Road. The levee proceeds in a southwest direction through the wetlands for about 300 feet until it reaches the east bank of Pews Creek. A storm gate is to be constructed across Pews Creek about 300 feet south of the Pews Creek Bridge. The storm gate system will connect



DEPARTMENT OF THE ARMY
NEW YORK DISTRICT CORPS OF ENGINEERS
NEW YORK, N.Y. 10278-0090

PORT MONMOUTH, NJ – COMBINED FLOOD CONTROL
AND SHORE PROTECTION STUDY
BAYSHORE PROTECTION PLAN

SCALE 1" = 100'

-100 0 100 200

PLOTTED BY: JRC

DATE OF SURVEY:

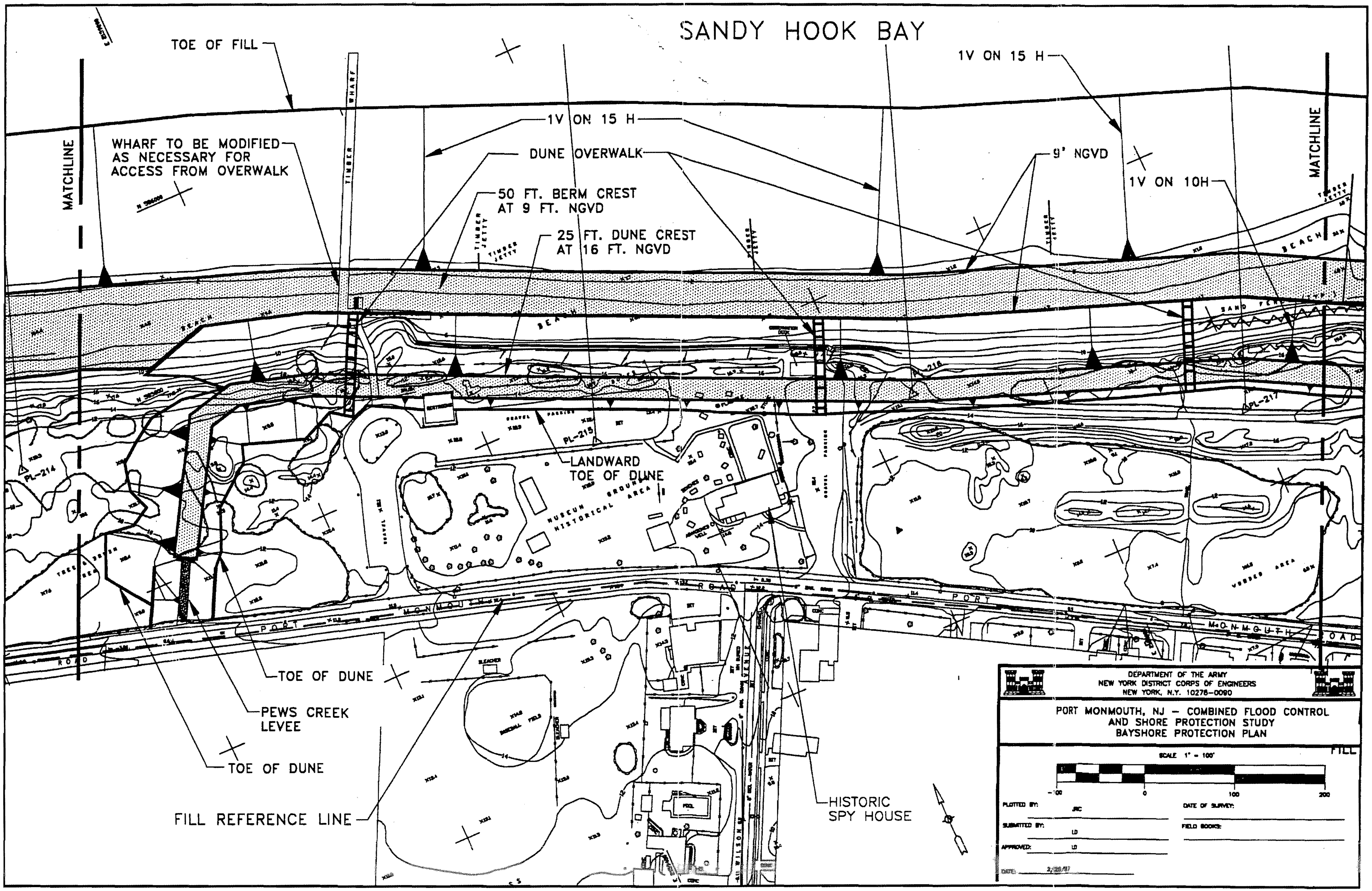
SUBMITTED BY: LD

FIELD BOOKS:

APPROVED: LD

DATE: 2/28/87

FIGURE A-57A



DEPARTMENT OF THE ARMY
NEW YORK DISTRICT CORPS OF ENGINEERS
NEW YORK, N.Y. 10278-0090

PORT MONMOUTH, NJ – COMBINED FLOOD CONTROL
AND SHORE PROTECTION STUDY
BAYSHORE PROTECTION PLAN

SCALE 1" = 100'

-100 0 100 200

PLOTTED BY: JRC

SUBMITTED BY: LD

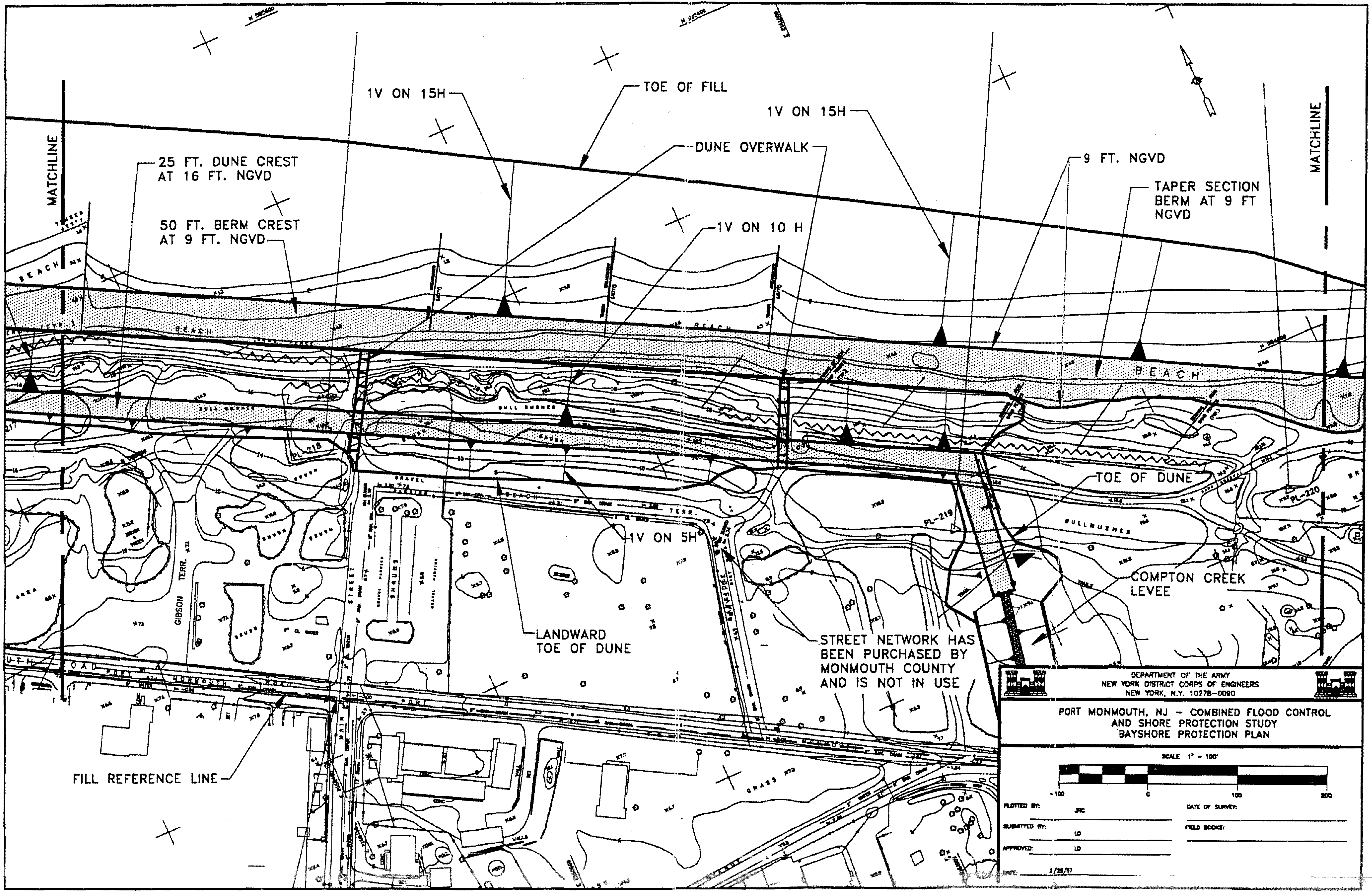
APPROVED: LD

DATE: 2/[illegible]/97

DATE OF SURVEY:

FIELD BOOKS:

FIGURE A-58



DEPARTMENT OF THE ARMY
NEW YORK DISTRICT CORPS OF ENGINEERS
NEW YORK, N.Y. 10278-0090

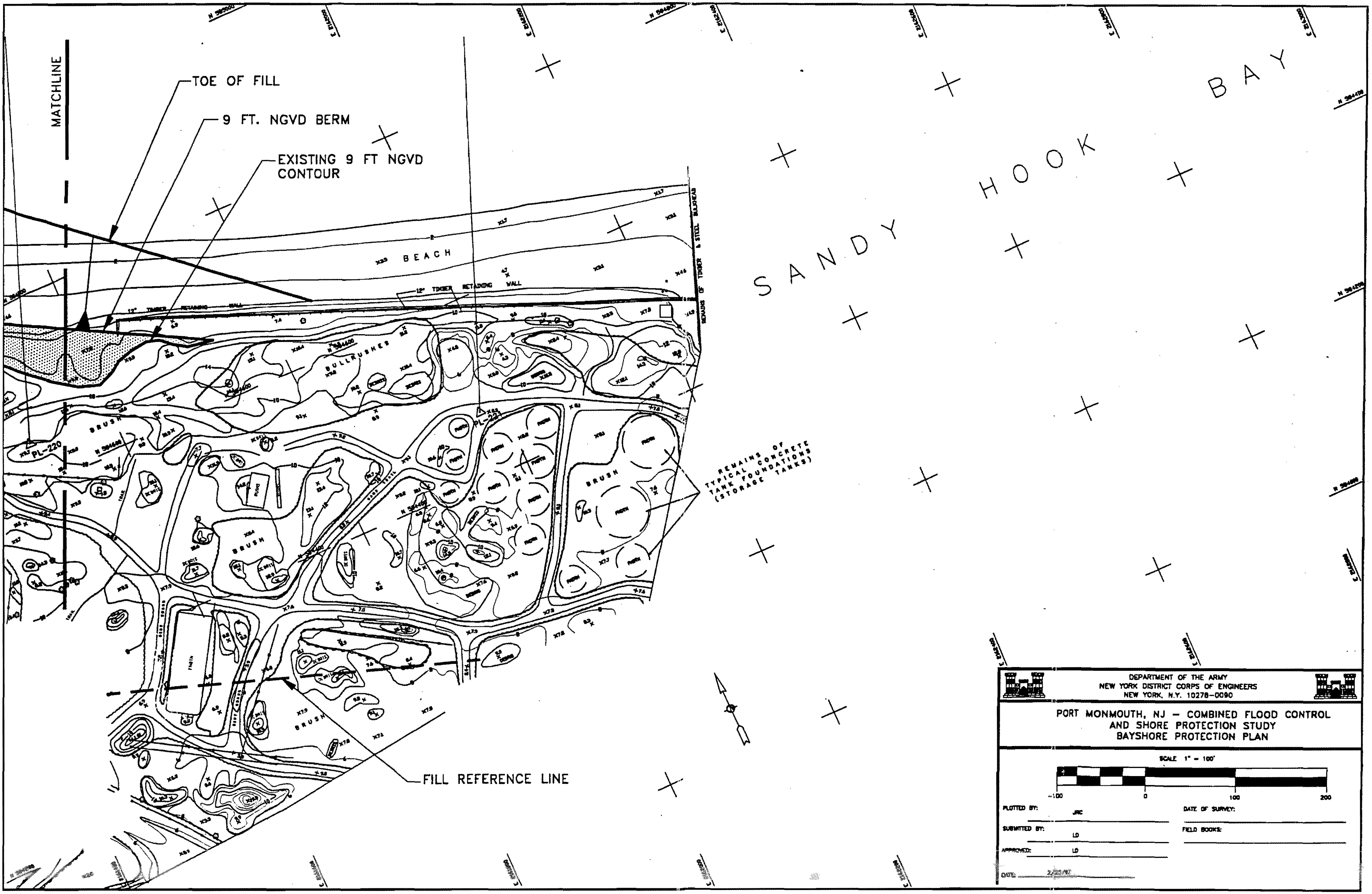
PORT MONMOUTH, NJ – COMBINED FLOOD CONTROL
AND SHORE PROTECTION STUDY
BAYSHORE PROTECTION PLAN

SCALE 1" = 100'

-100 0 100 200

PLOTTED BY: JRC
SUBMITTED BY: LD
APPROVED: LD
DATE: 2/25/97
DATE OF SURVEY:
FIELD BOOKS:

FIGURE A-59



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SUBMITTED BY: LD

APPROVED: LD

DATE: 2/25/97

DATE OF SURVEY:

FIELD BOOKS:

FIGURE A-60

to a proposed concrete wall on the west bank of Pews Creek. This concrete wall will run west for about 150 feet where it will connect with the existing East Keansburg levee. A storm gate is to be constructed across Pews Creek about 300 feet south of the Pews Creek Bridge. The sector gate size opening will be 40 feet wide. The storm gate will connect to a concrete pile supported T-wall on the east side of Pews Creek for about 150 feet where it will join the existing Keansburg levee.

Interior Drainage Areas

A-270. Pews Creek. A 120 cfs pump station will be located in direct vicinity of the Creek on the left bank for interior pumping conditions for the gate closed position. The 120 cfs pump station would be utilized along with 2 gravity outlets. The storm gate would have an opening size of about 40 feet. The gate would close at elevations between 5 and 5.5 ft. NGVD.

A-271. Compton Creek. Interior drainage facilities are required to safely store and discharge storm water runoff which collects on the protected side of dunes, berms, levees and floodwalls associated with flood control projects. For purposes of this project, an "interior drainage area" is a distinct land area which drains to one primary outlet location behind the proposed line-of-protection. The identification of such areas is complicated by the presence of anthropogenic features such as storm sewers which divert flow into or out of a drainage area. In some cases, otherwise distinct interior areas could become combined during rare storm events due to high ponding depths behind the line of protection works.

A-272. The interior drainage analysis is presented for Compton Creek. As noted above, the drainage facilities at the Pews Creek storm gate are considered integral to the line of protection. The proposed Compton Creek plan includes three interior drainage sub-basins (C1, C2, and C3) which parallel the line of protection works and are located along the west bank of Compton Creek. For the C3 area of 78 acres, a pump station of about 60 cfs was formulated. For a complete discussion of interior drainage design see Interior Drainage Appendix.

A-273. Drainage Area Delineation. Interior drainage basins were delineated on topographic maps with 2-foot contour intervals, dated April 1995. The "Middletown Township Master Drainage Plan", prepared by T&M Associates and local reports were also utilized. Mapping information was further supplemented by field inspections.

Area C1

A-274. Interior drainage area C1 is located along the left (west) bank of Compton Creek from south of Route 36 area near Chestnut Street to the north between Campbell and Collins Avenues. The area extends west beyond Wilson Avenue to Main Street in the New Street area. The interior drainage area of C1 is comprised of 47.65 acres of developed urban land, with minimal wetlands. The lowest buildings are located at elevation 7 ft. NGVD while Willow Street may start to flood at elevation 6.5 ft. NGVD.

Land Use	Area (ac)	% of Area
Residential	24.05	50.47
Open Space	13.83	29.03
Bldg/Rdwy	7.43	15.59
Wetland	2.34	4.91
Total	47.65	100.00

A-275. Review of the maps and field investigation revealed that drainage systems consisting of catch basins, manholes and storm drainage pipes, serve the Port Monmouth interior drainage area. There are three existing storm outfalls within sub-basin C1. The following is a summary of the existing storm drainage outlets.

EXISTING STORM DRAINAGE OUTLETS FOR DRAINAGE AREA C1

Outfall Designation	Outfall Location	Type	Size	Existing Inv. El. (ft.)
CC-11	Warsha Ave.	RCP	15"	5.3
CC-12	Wilson Ave (±250' S. of Campbell Ave.)	RCP	15"	5.3
CC-13	Willow Street	RCP	15"	4.1

A-276. The selected facility for sub-basin C1 has a primary outlet and secondary outlet as noted below. Both the primary and secondary outlets are being provided with a sluice gate and trash rack. The outlets will also be provided with flap gates to prevent tidal surges from entering the protected area. Ditches will be constructed along the landward side of the levee to direct runoff toward either the primary or secondary outlet.

FACILITY OUTLET STRUCTURES FOR DRAINAGE AREA C1

Outlet	Location	Size
Primary	300' N. of Willow Street	48" RCP
Secondary	100' S. of Willow Street	18" RCP

Area C2

A-277. Interior drainage area C2 is located along the left (west) bank of Compton Creek from sub-basin area C1 extending north just beyond Broadway. A segment of proposed levee and Wilson Avenue form the east and west boundaries of the interior drainage area. The interior drainage area of C2 totals 50.84 acres of predominantly residential development with limited wetlands areas. The lowest buildings are located at elevation 7 ft. NGVD while flooding of Creek Road and Main Street will start at 4.7 and 5.7 ft. NGVD, respectively.

Land Use	Area (ac)	% of Area
Residential	48.81	96.00
Wetland	2.03	4.00
Total	50.84	100.00

A-278. There are four existing storm outfalls from sub-basin C2. The following is a summary of the storm drainage outlets.

EXISTING STORM DRAINAGE OUTLETS FOR DRAINAGE AREA C2

Outfall Designation	Outfall Location	Type	Size	Existing Inv. El. (ft.)
CC-7	Main St./Broadway Intersection	RCP	18"	4.2
CC-8	Creek Rd. (±900' E. of Wilson Ave. Int.)	HRC	19"x30"	3.1
CC-10	Creek Rd. (± 150' S.E. of Collins Ave)	P	24"	2.5
CC-150	Wood Stock Ave. near Broad St.	RCP CIP	12"	NA

A-279. The selected facility for sub-basin C2 will have a primary outlet and five secondary outlets as noted below. Both the primary and secondary outlets are being provided with a flap gate, sluice gate, and trash rack. Drainage ditches will direct runoff along the protected side of the levee to a nearby outfall.

FACILITY OUTLET STRUCTURES FOR DRAINAGE AREA C2

Outlet	Location	Size
Primary	100' S. of Broad Street	48" RCP (Extension exist. CC-8 &CC-150)
Secondary #1	Broadway near Main St. Int.	18" RCP (Extension of exist. CC-7)
Secondary #2	150' S. of Pinehurst Ave.	18" RCP
Secondary #3	200' S. of Creek Road	18" RCP
Secondary #4	Near Creek Rd. Collins Ave Int.	24" RCP (Extension of exist. pipe CC-10)
Secondary #5	100' N. of Collins Ave.	18" RCP

Area C3

A-280. The C3 interior drainage area is also located on the left (west) bank of Compton Creek. Main Street and Wilson Avenue from the east and west boundaries of the area with the dune/berm forming the north boundary and C2 (just south of Lydia Place) forming the south boundary. The interior drainage area C3 is comprised of 78.74 acres, the majority of which is residential. The area near Monmouth Avenue is subject to some of the most frequent flooding in the area. Street elevations in this area are as low as 4.4 ft. NGVD. The lowest buildings are located at elevation 5 ft. NGVD.

Land Use	Area (ac)	% of Area
Residential	59.01	74.94
Open Space	19.73	25.06
Total	78.74	100.00

A-281. There are several existing storm drainage systems within sub-basin C3. Currently a significant portion of the area is drained to Pews Creek via a 36-inch diameter pipe which outlets at PC6. The plan is to redirect drainage to Compton Creek. Since the pipe drains a very low area, this will increase the zero damage elevation at Pews Creek, reduce the required frequency of storm gate closure, and increase the volume of storm water storage available on Pews Creek. The following is a summary of the storm drainage outlets.

EXISTING STORM DRAINAGE OUTLETS FOR DRAINAGE AREA C3

Outfall Designation	Outfall Location	Type	Size	Existing Inv. El. (ft.)
CC-1	±200' S. of Park Ave., ±400' E. of Park Ave. & Main St. Int.	CIP	15"	3.1
CC-2	Main St. (±250' S. of Park Ave. Int.)	CIP	12"	2.2
CC-6	Main St. & Lydia Pl. Int.	RCP	18"	NA
CC-128	Main St. (±400' S. of Renfrew Pl. Int.)	RCP	15"	2.5
PC-6	Wilson Ave. (±350' N. of Renfrew Pl. Int.)	RCP	36"	NA

A-282. The selected facility for sub-basin C3 has a primary outlet and five secondary outlets as noted below. Both the primary and secondary outlets are being provided with a flap gate, sluice gate, and trash rack. The 36-inch storm water diversion pipe that will direct flow to Compton Creek instead of Pews Creek will be approximately 750 feet long. Ditches are included on the protected side for the levee to direct runoff to primary or secondary outlets.

FACILITY OUTLET STRUCTURES FOR DRAINAGE AREA C3

Outlet	Location	Size
Primary	± 300' N of Renfrew St.	2x48" RCP
Secondary #1	Lydia Pl.	18" RCP (New extension of exist. CC-6)
Secondary #2	Main St. (±500' S. of Renfrew Pl. Int.)	18" RCP (New extension of exist. CC-128)
Secondary #3	Main St. (±250' S. of Park Ave. Int)	18" RCP (New extension of exist. CC-2)
Secondary #4	400' E. of Main St./Park Ave. Int.	18" RCP (Extension of exist. CC-1)
Secondary #5	Pt. Monmouth Rd., ±100' E. of Park Ave. & Pt Monmouth Rd. Int.	18" RCP

Additional facilities considered resulted in finding that a 60 cfs pump station would be justified for implementation at the C3 basin area because of high residual damages.

Bayshore Fill Volume

A-283. Design Fill Volume Requirements. The design dune profiles are compared with surveyed profile data to determine the required dune fill density. At any location along the dune the required dune fill density is estimated to be the density determined at the nearest profile line in the project area. For example, the required dune fill density at PL-218 is assumed to represent the dune fill requirements from halfway between profiles PL-217 and PL-218 to halfway between profiles PL-218 and PL-219. At the ends of the dune fill, the dune fill volume is estimated similarly. For example, the required dune fill density at PL-219 is applied to the dune length extending from halfway between PL-218 and PL-219 to the eastern terminus of the dune fill (as it meets the Compton Creek levee). The easternmost profile location in the project area is PL-215, so the fill density at this location is used to calculate the volume required to construct the dune to the Pews Creek levee.

A-284. In addition, because the design fill renourishment activity will alter the shoreline shape and potentially induce end losses, the design will include taper sections. The taper section fill volumes are calculated by assuming that the existing profile will be translated seaward along the taper section length from the berm elevation to the toe of fill (+9 to -3.3 ft. NGVD). The translation width of the taper section varies from its maximum (at the design berm) to the translation terminus. Table A-17 indicates the design fill volume for the +16 ft. dune elevation, while Table A-18 indicates the design fill volume for the

TABLE A-17
PORT MONMOUTH SHORELINE PROTECTION
DESIGN FILL VOLUME - 16 FT. DUNE HEIGHT

SURVEYED PROFILE	PROJECT LENGTH (FEET)	DESIGN FILL	
		DENSITY (YD³/FT)	VOLUME (YD³)
WEST TAPER	1,450	VARIES	32,142
PL-215	805	58.9	47,446
PL-216	366	56.1	20,517
PL-217	378	51.6	19,517
PL-218	548	39.9	21,853
PL-219	540	23.4	12,638
EAST TAPER	550	VARIES	7,700
DESIGN FILL	**4,637**		**161,812**
ADVANCE FILL			95,454
TOTAL FILL VOLUME			**257,266**

NOTE:

(1): DUNE LENGTH INCLUDES SHORE-NORMAL SEGMENT AT WEST AND EAST ENDS
WEST END FEATURES 175' OF TIE-BACK DUNE
EAST END FEATURES 160' OF TIE-BACK DUNE

(2): FILL DENSITY REQUIREMENTS ARE CALCULATED ON CLOSEST SURVEYED PROFILE IN PROJECT AREA

(3): FILL VOLUME BASED ON 16' DESIGN CREST

(4): ADVANCE FILL VOLUME INCLUDES VOLUME REQUIRED TO ACCOUNT FOR SEA LEVEL RISE

TABLE A-18
INITIAL BAYSHORE DESIGN FILL VOLUME VS. DUNE HEIGHT
PORT MONMOUTH, NJ

DUNE CREST HEIGHT (FT)	DESIGN FILL VOLUME (YD.^{3})	ADVANCE FILL VOLUME (YD.^{3})	TOLERANCE FILL VOLUME (YD.^{3})	SUBTOTAL (YD.^{3})	OVERFILL FACTOR	TOTAL FILL VOLUME (YD.^{3})
15	157,999	95,454	52,468	305,921	1.22	**373,223**
16	161,812	95,454	52,948	310,214	1.22	**378,461**
17	165,719	95,454	53,428	314,601	1.22	**383,813**

alternative dune elevations. Sub-Appendix A3 presents the fill layouts for the alternative project elevations.

A-285. Advance Fill. Advance fill volumes are calculated by assuming that historic erosional trends will continue. The majority of the shoreline protection lies within the eastern shoreline cell of the sediment budget. The eastern sediment budget cell extends 2,950 feet along the Port Monmouth shoreline, and currently experiences more severe erosion (-5,700 c.y./yr.) than the remainder of the shoreline. Use of the most severe erosion rate identified in the existing conditions sediment budget provides a conservative erosion rate. This conservative value (along with the project contingencies) accounts for uncertainties in the rate of project diffusion (end) loss. In the cell, the existing conditions sediment budget identifies an annual erosion rate of -1.93 cubic yards per linear foot of shoreline (-5,700 c.y./yr./2,950 l.f.). An additional volume is included to account for losses resulting from sea level rise. This volume is computed by applying the measured profile dimensions and the anticipated sea level rise in the Bruun Rule equation (Weggel, 1979), resulting in an additional 0.13 c.y./l.f. Using these values, the volumetric erosion rate in the shoreline project is approximated to be 9,500 c.y./yr., multiplying 2.06 (1.93+0.13) by 4,600 feet of project length. This number is further multiplied by the 10 year renourishment interval for an estimate of the total advance fill requirement. Assuming that the profile will equilibrate at its existing shape (i.e. assuming profile translation) between the design berm elevation and the closure depth, the advance fill will create an equilibrated additional shoreline width of about 41 feet. This layout exceeds not only the volumetric requirements, but the requirement resulting from long term shoreline changes, which average –2.7 ft./yr.

A-286. Tolerance Fill. Additional fill will be required during construction of the beach restoration project to provide for the design fill template tolerance. A 1-foot construction tolerance is typical for this level of analysis.

A-287. Total Initial Fill. The total initial project fill volume is the sum of the design fill, the advance fill and the tolerance fill over the nourishment cycle. Total initial fill volumes and nourishment fill volumes for the proposed storm protection project are presented in Table A-18. Based on available geotechnical data, an overfill factor of 1.22 was used for the dredged beach and dune fill.

A-288. Emergency Fill. Major rehabilitation or emergency fill costs are included as an additional annualized requirement to initial and nourishment construction to account for impacts to the design profile from major storm/hurricane events. The required volume to restore the design profile and the existing dune system plus advance nourishment was correlated to the anticipated beach loss from major storm events of various frequency levels, starting with the 10-year event. Major rehabilitation volumes are presented in Table A-19.

TABLE A-19
MAJOR REHABILITATION QUANTITIES

RETURN INTERVAL (YEARS)	FREQUENCY (EVENTS / YR.)	FREQUENCY INTERVAL	PERMANENT LOSS FACTOR	EROSION VOLUME (YD^3/FT)	EMERGENCY FILL (YD^3/FT)	AVERAGE EMERGENCY FILL (YD^3)	ANNUAL EMERGENCY FILL (YD^3/YR)
10	0.10		0.16	10.1	1.62		
		0.05				6,825	341
20	0.05		0.22	11.4	2.51		
		0.03				10,091	303
50	0.02		0.27	13.3	3.60		
		0.01				20,244	202
100	0.01		0.60	14.4	8.66		
		0.01				39,148	196
200	0.01		1.00	15.1	15.05		
	ANNUAL REHABILITATION VOLUME:						**1,042**

FILL LENGTH: 3302 FT.

NOTE: PERMANENT LOSS FACTOR IS THE PERCENT OF ERODED VOLUME PERMANENTLY LOST TO PROFILE.
PERMANENT LOSS FACTOR VALUES ARE BASED ON EXPERIENCE AT OCEAN CITY, MD.
EROSION VOLUME IS THE MAXIMUM LANDWARD OF A GIVEN PROFILE POSITION COMPUTED FROM EDUNE MODEL.
PERMANENT LOSS FACTOR TAKEN AS 0.60 FOR STORMS JUST PRIOR TO FAILURE EVENT TO ACCOUNT FOR DUNE OVERTOPPING.
PERMANENT LOSS FACTOR TAKEN AS 1.0 IF THE DUNE IS COMPROMISED BY STORM.

A-289. Renourishment. The renourishment volume required for beach maintenance is calculated in a manner similar to the initial advance fill volume. The annual volumetric loss of sediment along the entire project length is assumed to be 2.06 c.y./yr./l.f. This value is multiplied by 4,600 linear feet (the 2,300 ft. project length plus taper sections). The tolerance fill is included and the annual requirement is then multiplied by the 10 year renourishment interval, for a total renourishment requirement of 127,300 c.y. per maintenance cycle. Renourishment is expected to be accomplished from upland sources meeting material specifications, so the overfill ratio for renourishment is estimated to be 1.0.

IMPROVED CONDITIONS COASTAL PROCESSES

A-290. The improved (i.e., with project) condition is identified as the condition with the project design in place, and includes long-term renourishment and maintenance of the project features. The fill layout and renourishment have been designed to ensure the integrity of the design dune cross-section. However, coastal processes will continue to impact the project area shoreline. The improved conditions coastal processes of storm-induced erosion, dune failure, wave runup and overtopping were evaluated. The improved conditions were analyzed on a representative profile reflecting the offshore and upland topographies for the shoreline under project conditions. The following paragraphs describe the coastal processes which were used to estimate the improved condition benefits.

A-291. The methodologies used to investigate the improved conditions coastal processes are equivalent to those used in the existing conditions study with the following exceptions. Firstly, the representative profile represents conditions just prior to renourishment, with the design dune and berm in place.

A-292. The improved conditions representative profiles feature the design berm and dune extending to their landward and seaward toes of fill, beyond which the existing profile extends. The dune crest features a 25 foot width at +16.0 feet NGVD. The landward dune slope is 1V on 5H, and the seaward dune slope is 1V on 10H. The berm extends seaward 50 feet at the +9 foot NGVD elevation, beyond which the berm slope extends at 1V on 15H.

Storm-Induced Recession

A-293. The approach used to evaluate the improved conditions erosion potential was equivalent to that used in the existing conditions study. The EDUNE dune erosion model by Kriebel (1989) was used in this study to evaluate the susceptibility of the design berm and dune to storm-induced erosion. Input and output parameters are identical to those in the existing conditions storm recession study. In particular, it is assumed that the post-project representative grain size will not be changed from 0.39 mm. The input profile applied as input represents the minimum design conditions. To create the profile it was assumed that the entire volume of advance fill sand was eroded from the profiles. Therefore, the profiles are interpreted to represent conditions just prior to renourishment.

A-294. With-project storm recession effects were calculated for storms with return intervals of 2, 5, 10, 25, 50, 100, and 200 years. Sample input and output profiles are presented in Figure A-61. As in the existing conditions study, the value used to quantify the storm recession for each of these cases is the recession extent, which is the distance between the 0' NGVD shoreline in the design profile and the landward limit of the eroded portion of the profile. Values of recession extent are presented in Table A-20 and Figure A-62. Also shown is the remaining peak dune elevation predicted for each storm studied. Input values and output EDUNE profiles are presented in Sub-Appendix A2.

A-295. Featuring only the design dimensions, the input profile represents conditions just prior to renourishment. Not included are either the advance fill volume or the existing dune where it exceeds the design dimensions. Therefore, the predicted improved conditions storm recession values are considered conservative.

A-296. The subsequent runup value was computed on post-storm profiles output by EDUNE. This required several iteration steps in order to result in agreement between the runup value (used as EDUNE input) and the post-storm profile (the slopes of which are used as runup theory input).

Wave Runup

A-297. Wave runup values for improved conditions were calculated using the same approach as the existing conditions study. The input parameters were identical to the existing conditions study with the exception of tan α, the beach slope. The improved conditions beach slope value used is representative of onshore slopes measured from post-storm profiles output by EDUNE. As runup elevation is an input parameter for the EDUNE model, for each representative improved profile several iterations were required in order to result in a post-storm profile whose slope (input in the runup equation) resulted in a predicted runup value that (used as input in EDUNE) yielded a post-storm profile consistent with that above. The resulting post-storm slope using improved conditions as runup input was 1:18.0.

A-298. Improved conditions resulting runup predictions are shown in Table A-21. Predicted wave runup elevations range from 1.7 to 3.4 ft. above the still water level (SWL), slightly higher than the existing conditions wave runup values. Accounting for the storm surge associated with the storms, runup elevations are predicted to reach as high as 18.1 ft. NGVD for a 200 year event.

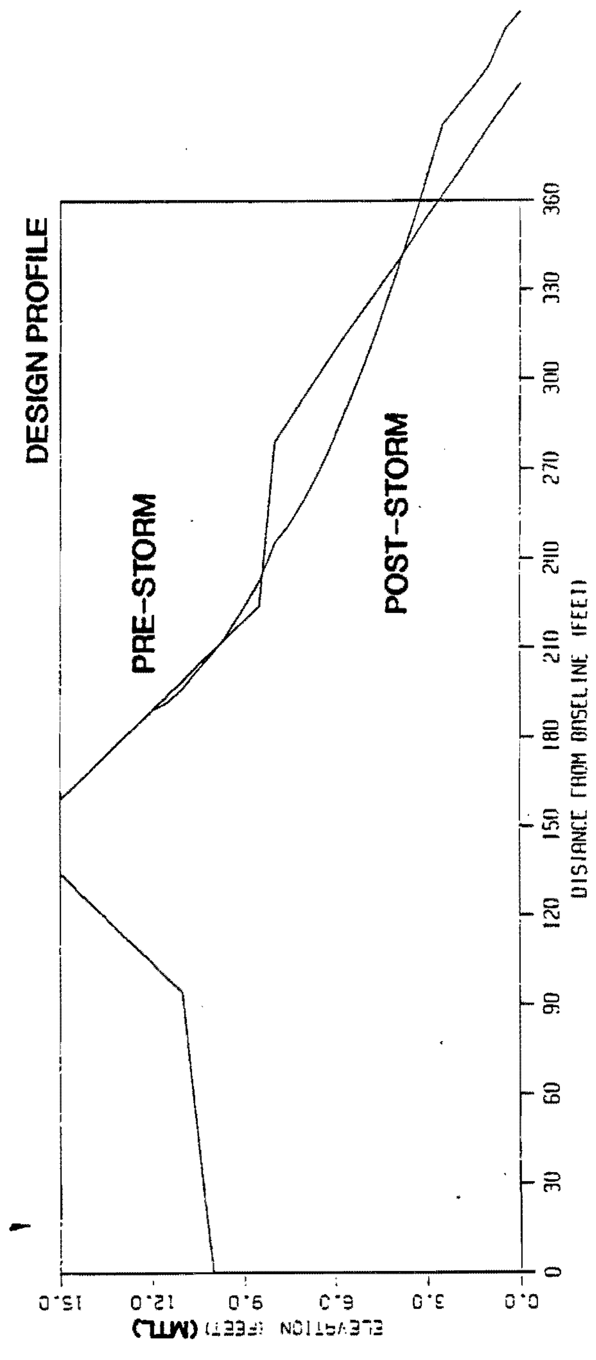

FIGURE A-61

TYPICAL POST-STORM PROFILE PREDICTED BY EDUNE

IMPROVED CONDITIONS: 25 YEAR STORM

TABLE A-20
PREDICTED STORM RECESSION EXTENT
IMPROVED CONDITIONS

STORM RETURN INTERVAL (YEARS)	PREDICTED RECESSION EXTENT (FEET)	REMAINING DUNE ELEVATION (FEET, NGVD)
2	141	15.9
5	198	15.9
10	209	15.9
25	221	15.9
50	239	15.9
100	265	15.9
200	288	14.9

RECESSION EXTENT REPRESENTS A MEASURE BETWEEN
THE EXISTING 0' NGVD SHORELINE AND THE LANDWARD EROSION LIMIT

EDUNE MODEL USED FOR PREDICTION

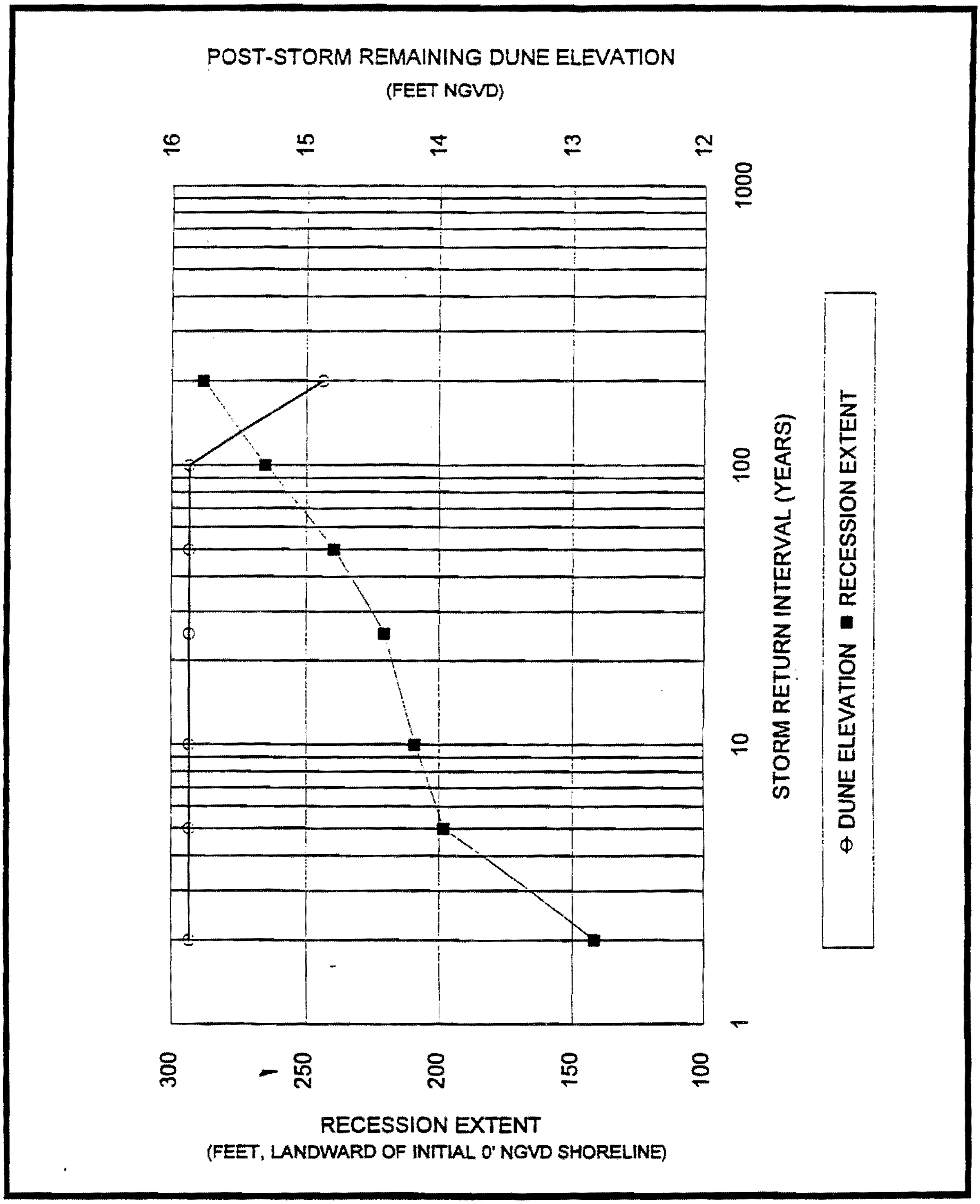

FIGURE A-62

PREDICTED STORM RECESSION ON IMPROVED BEACH PROFILE
PORT MONMOUTH, NEW JERSEY

TABLE A-21

PREDICTED WAVE RUNUP: IMPROVED CONDITIONS

STORM RETURN INTERVAL (YEARS)	WAVE RUNUP (FEET, 2% EXCEEDANCE) (ABOVE SWL)	(ABOVE NGVD)	STILL WATER LEVEL (NGVD)
2	1.7	8.2	6.5
5	2.3	10.0	7.6
10	2.5	10.9	8.4
25	2.6	12.2	9.6
50	2.7	13.2	10.5
100	3.3	15.5	12.2
200	3.4	18.1	14.7

NOTE:

CALCULATED ON POST-STOM PROFILES (AS PREDICTED BY THE EDUNE MODEL).
PREDICTED BY EQUATIONS OF PILARCZYK (1990)

Wave Overtopping

A-299. Improved conditions values of wave overtopping over the dune are based on the theory developed by Kobayashi, et al. (1996). Using an approach similar to that in the existing conditions study, the theory is applied to the post storm profile resulting from the storm recession study of the design profile. The storm surge elevation (with respect to the dune elevation) and wave characteristics represent peak storm conditions.

A-300. The improved conditions overtopping results are presented in Table A-22. Because 1) the integrity of the design dune is generally preserved in the EDUNE results, and 2) the elevation of the dune is established to exceed the surge elevation + the runup height, the overtopping rate over the dune is predicted to be low. The only significant dune overtopping predicted results from the 200 year level storm. Since the values represent peak overtopping values, over the majority of the storm's duration the overtopping rate is likely to be lower than the values in Table A-22.

Sediment Budget

A-301. The construction of coastal protection projects typically changes the local sediment transport patterns by altering the coastal structures, the local maintenance dredging behavior, the beach sediment characteristics, or the shoreline layout. Following such changes, the improved conditions sediment budget values must be determined in order to help quantify the potential success or impacts of the project design.

A-302. The improved conditions rates of volumetric change and sediment transport are predicted by estimating the influences of the project on the existing conditions sediment budget (Figure A-11). The resulting improved conditions sediment budget is presented in Figure A-63. The primary influence of the project is an increase in the volumetric losses from the beach caused by the altered shoreline layout. No shoreline model study was conducted to predict this anticipated loss, frequently referred to as diffusion loss, so historic volumetric losses are used to calculate future volumetric change rates. The existing conditions sediment budget indicates annual volumetric loss rates of 0.27 and 1.93 c.y./linear foot of beach in the bayfront's western and eastern segments respectively. Applying these values conservatively to predict the future anticipated erosion rate, the higher of these values is applied in the entire project area (including taper sections). Therefore, multiplying 1.93 by 4,600 feet of project length yields the annual loss from the project. Finally, an additional volume is included to account for losses due to sea level rise. Losses from beyond the project limits are assumed to continue as indicated in the sediment budget. The volumetric loss within each segment is calculated by adding the project loss to the background loss outside the project. Assuming the same nodal point, these values contribute to increasing the sediment transport rate into the creeks.

TABLE A-22
PREDICTED WAVE OVERTOPPING AT REPRESENTATIVE PROFILES
IMPROVED CONDITIONS

STORM RETURN INTERVAL	WAVE OVERTOPPING RATE PER LINEAR FOOT ((C.F./S.)/L.F.)	TOTAL OVERTOPPING ((C.F./S.)
2	0.00	0
5	0.00	0
10	0.00	0
25	0.00	0
50	0.00	0
100	0.00	0
200	0.87	1,992

NOTE:

CALCULATED ON POST-STOM PROFILES (AS PREDICTED BY THE EDUNE MODEL).

DUNE OVERTOPPING PREDICTED BY EQUATIONS OF KOBAYASHI, et al. (1996).

TOTAL OVERTOPPING CALCULATED ON 2,302 FEET SHORELINE PROJECT LENGTH.

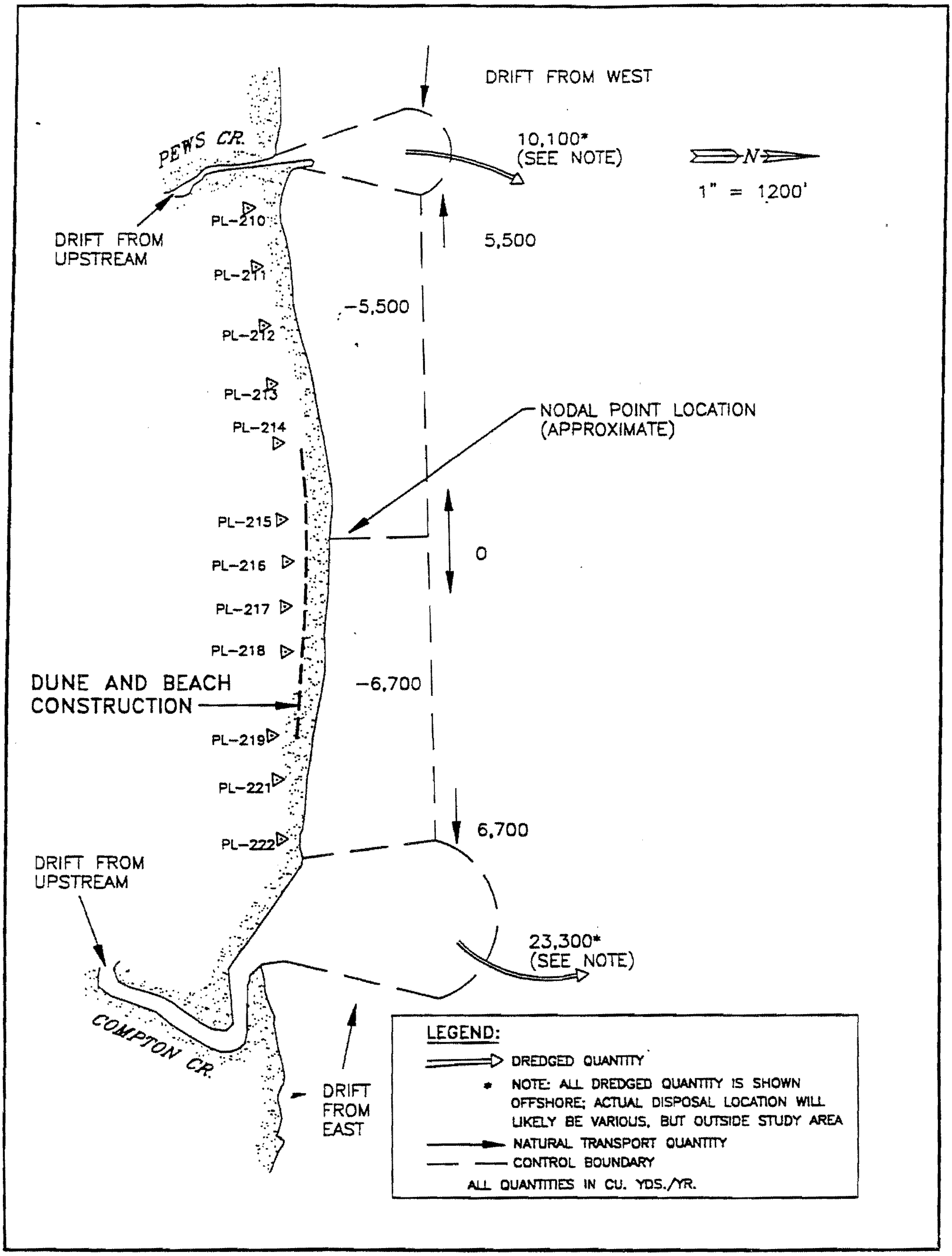

FIGURE A-63

SEDIMENT BUDGET
PORT MONMOUTH, NEW JERSEY
IMPROVED CONDITIONS

A-303. A secondary influence of the project on the sediment budget results from additional sediment losses due to fill sand incompatibility. To calculate this loss, the volume of advance fill applied every nourishment interval (which is calculated using the method described above) is multiplied by the overfill (1.15) ratio. The additional loss, attributed to the overfill, is computed as volumetric loss. While in some locations this material may adjust offshore, the flat profile at Port Monmouth indicates that the overfill adjustment may result in increased longshore transport. Therefore, this volume is included in the improved conditions sediment budget.

A-304. Similar to the existing conditions sediment budget study, the increased volumetric loss from Port Monmouth Beach will be assumed to contribute to the longshore sediment transport rate. Assuming that the beach's estimated nodal point location (halfway between monuments PL-215 and PL-216) will remain unchanged, anticipated beach losses west and east of this point are assumed to drift away from the point.

Impact on Adjacent Channels

A-305. One important effect will be an increase in the dredging requirements in Pews and Compton Creeks. Assuming no change in drift from other sources of creek shoaling, the total shoaling rate in the creeks is assumed to increase by the same volume as the increase in the shoreline volumetric erosion. Compared to existing conditions, the increased shoaling in Pews and Compton Creeks will be about 4,700 and 1,000 c.y./yr., respectively. Despite this increased shoaling, the improved conditions sediment budget assumes that, like the existing conditions, the beach placement of the dredged material will be minimal. This final assumption is based on the possibility that the shoal material will contain a significant volume of silty material whose origin was updrift in the creeks. While beach placement of spoil material from channel dredging is often advantageous, an increase in this practice may not be possible at Port Monmouth due to sediment incompatibility.

A-306. Storm Gate. A hydrodynamic analysis was conducted to evaluate the impact to astronomic tides for alternative storm gate configurations at Pews. Periodic tidal inundation of the estuary is important for the marsh to maintain itself.

A-307. The numerical modeling system used in this study is the US Army Corps of Engineers hydrodynamic RMA-2 model, which is part of the TABS-2 system. The TABS-2 system consists of pre- and post-processor utility codes and finite element two-dimensional depth-averaged computational programs for hydrodynamics (RMA-2), water quality (RMA-4), and sedimentation (SEDH). The finite element method provides a mean of obtaining an approximate solution to a system of governing equations by dividing the area of interest into smaller subareas called elements. Time-varying partial differential equations are transformed into finite element form and then solved in a global matrix system for the modeled area of interest. The solution is smooth across each element and continuous over the computational area. This modeling system is capable of simulating wetting and drying of marsh and intertidal area of the estuarine system. The version used in this study is called FASTTABS, which is the personal computer (PC) version of the main-frame based TABS-2.

A-308. The FASTTABS model described above requires that the study area be represented by a network of nodal points (i.e. points defined by coordinates in the horizontal plane and water depth) and elements (i.e. areas made up by connecting adjacent nodal points). Nodes can be connected to form 2-D (3 or 4 nodes) or 1-D elements (2 nodes). The resulting nodal/element network is commonly called a finite element mesh and provides a computerized representation of the area geometry and bathymetry.

A-309. The two most important aspects in laying out a finite element mesh are: (1) determining the level of detail necessary to adequately represent the study area and (2) determining the extent or coverage of the mesh. The models described above are numerically robust and capable of simulating tidal elevation flows, constituent transport, and sedimentation over a mesh with reasonable resolution. Accordingly, the bathymetric features of the estuary generally dictate the level of detail for the mesh. With regard to the present study, greater resolution were required both in the vicinity of the main channel of Pews Creek and in the principle side canals. Greater resolution was also required at the tidal gate to accurately define the impact of the structure.

A-310. Several additional factors guide decisions regarding the aerial extent of the mesh. The mesh should include all wetlands affected by and adjacent to Pews Creek in order to evaluate tidal inundation/exchange for each of the three storm gate alternatives. The mesh should also extend to the two tide gages used for monitoring/model calibration purposes. Additionally, the mesh should *utilize the* hydrographic soundings for Pews Creek. Based on above factors, the finite element mesh covers an area of approximately 200 acres. The northern boundary of the mesh is located at the inlet jetty of Pews Creek just north of Monmouth Cove Marina, and the southern boundary is located south of Highway 36 in Pews Creek. The western boundary is the Keansburg Levee, and the eastern boundary is the extent of tidal wetlands.

A-311. Quadrilateral and triangular 2-D elements were used to represent the estuarial system. As previously stated, a finer mesh was created in Pews Creek particularly in the vicinity of the proposed storm gate(s) and also in the principle side canals to accurately represent geometry changes. The side canals were slightly deepened to -2.5 ft-NGVD to prevent "ponding" *(areas of wet elements not connected)* in order to obtain model stability within the marsh areas. Higher resolution is achieved with a greater number of elements, however, additional computation effort is required. Differences between the existing condition or base line mesh and the three tidal gate alternative meshes are limited to the immediate vicinity of the proposed tidal gate.

A-312. The marsh porosity option in RMA-2 was used in order to increase model stability. The marsh porosity option facilitates a gradual transition between wet and dry states. The model was not stable when the other wetting/drying options were used because too many elements would dry at once. The hydrodynamic model was operated using measured tide gage data for existing conditions with a quarter hour time step for a 55 hour time period. The simulation was performed for a total of four tide cycles, which allowed for an initial "spin up" of the model. A few minor discrepancies exist between the simulations and measurements. For example, the simulated tide at Bray Street slightly lags and has a slightly lower range than the measured tide. Overall, however, the agreement between the simulated and recorded tides was judged to be satisfactory. Based on the hydrodynamic model results a 40 ft wide gate would have only minimal reduction on wetland inundation, with depths reduced by 0.06 ft. to 0.19 feet depending on the tidal condition and analysis location. A 20 ft. wide gate was investigated and rejected due to the more significant wetland impacts.

A-313. The Pews Creek Storm Gate selection incorporated reviews of Department of the Army Manuals and discussions with the New Orleans District, where gates are used extensively on navigation and flood control projects. Engineering Manual No. 1110-2-2703, "Engineering and Design, Lock Gates and Operating Equipment", was specifically reviewed for information on various gate types and associated advantages and disadvantages both in gate operation and construction. Based on a review of EM 1110-2-2703, it appeared that a Sector Gate would be the most appropriate gate type for use at the Pews Creek site. Though Sector gates have generally higher construction cost due to the need for larger recesses in the gate monolith, they have operational and maintenance advantages over other types of gates. Conversations with the New Orleans District confirmed that Sector Gates operate well under high sediment conditions. In addition, Sector gates can be closed under flow conditions which could be experienced under a storm surge. Other types of gates, such as miter gates, do not perform well under conditions that may generate a differential hydraulic head. Sector gates also provide maintenance advantages since they can be removed and replaced from the gate monolith in the wet. Other types of gates require cofferdaming and dewatering for gate removal and maintenance.

Project Performance

A-314. For a successful project, it is important that the project feature a reasonable certainty of surviving the project life. For coastal projects, this means that the project must be able to withstand storm conditions that can be reasonably expected to occur over the life of the project. For a 50 year project, statistical distributions indicate that there is a 50% likelihood that the project will be subjected to storms not exceeding a 73-year event. In order to maintain a reasonably low level of project risk, it should be demonstrated that the project can withstand a storm with severity equal to or greater than a 73 year level.

A-315. The closest storm investigated is the 100 year level storm. The storm recession investigation estimated the post-storm profile shape, and predicted that the peak dune elevation would be protected during a 100 year level storm. The wave overtopping investigation on the post-storm profiles verified that the dune would provide adequate protection against wave overtopping in such an event. Therefore, the design is adequate to withstand the most severe storm that is anticipated to occur over the project life, so predicted project performance is adequate.

Risk Analysis

A-316. In order to minimize the risk of project failure, the level of risk must be quantified. Due to the difficulties in quantifying potential storm impacts (recession, runup, storm surge, etc.), the primary source of risk in a coastal protection project stems from the uncertainty of these values. The risk of project failure is addressed by estimating the storm impacts resulting from upper limit values of the storm impacts. The risk analysis is limited to the 73 year storm level, because this level storm features a 50% likelihood of occurrence in the 50 year project life.

A-317. Because the storm surge elevation dominates the project performance and potential failure (due to the low wave climate), the risk analysis focuses on the storm surge elevation uncertainty. By interpolating the predicted surge elevations, the 73 year estimated elevation is calculated to be 11.4 ft. NGVD, with an upper limit of 13.5 ft. NGVD. The anticipated runup can be estimated to be 2.5 ft. by averaging the existing conditions runup with the improved conditions runup, assuming that at the time of the storm's occurrence the profile may continue to include some of the existing dune features. Finally, with the potential increased mean sea level ranging from 0 ft. at construction to 0.7 ft. at the end of the 50 year project, the value 0.4 ft. can be used to represent the average expected rise in sea level when the most severe storm impacts the project. The upper limit runup elevation can be calculated by adding the upper limit storm surge elevation (13.5 ft. NGVD) to the sea level rise (0.4 ft.) and runup (2.5 ft.) for an elevation of 16.4 ft. NGVD.

A-318. Assuming that the integrity of the dune is not violated by the 73-year level storm (as the EDUNE results indicated that the dune would withstand a 100-year level storm), this upper limit runup elevation exceeds the dune crest by 0.4 ft. Therefore, while a realistic potential exists for overtopping of the dune, the overtopping may not be significant. This conclusion is based on the small runup overtopping elevation and the short duration during which storms feature peak conditions.

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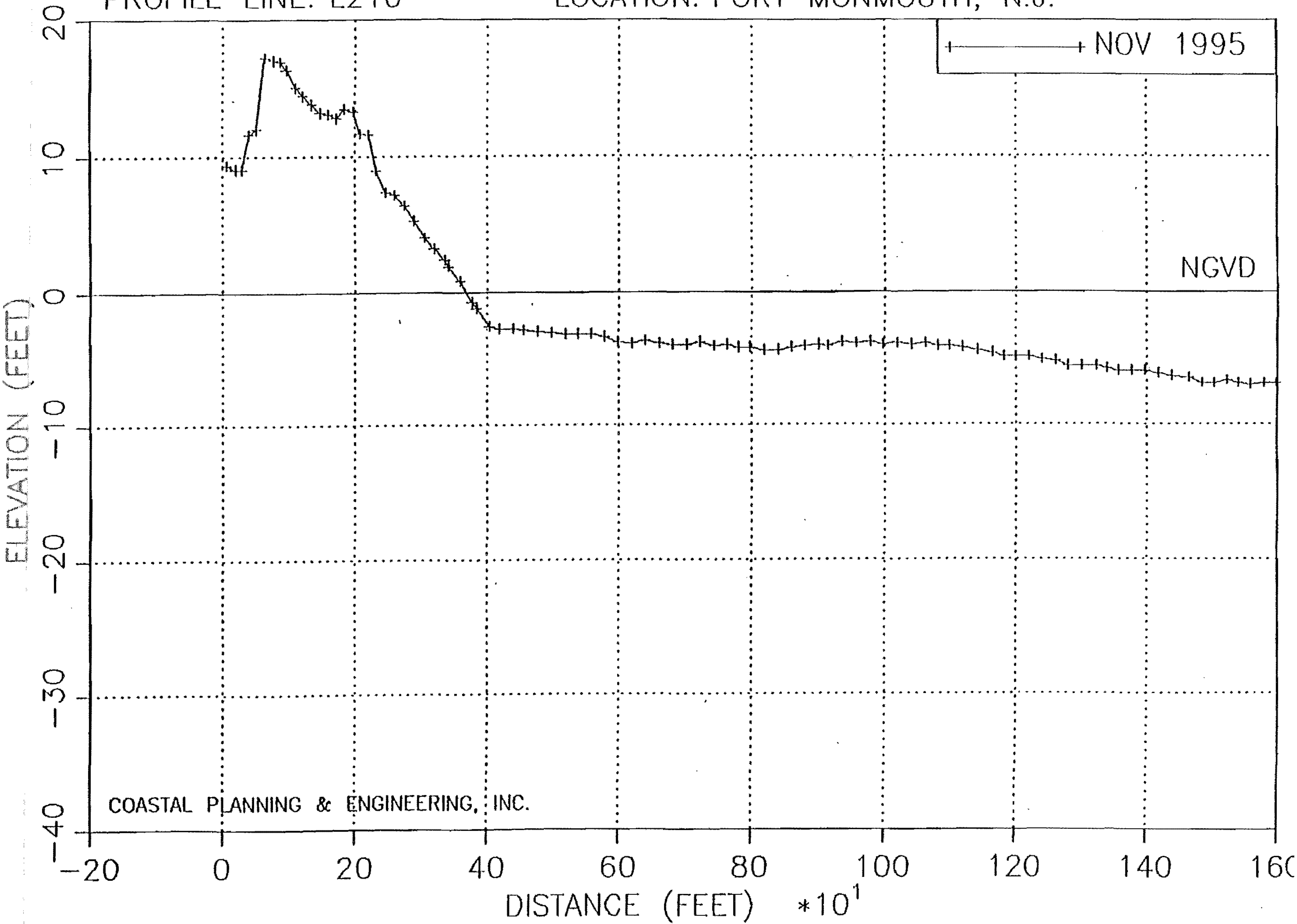
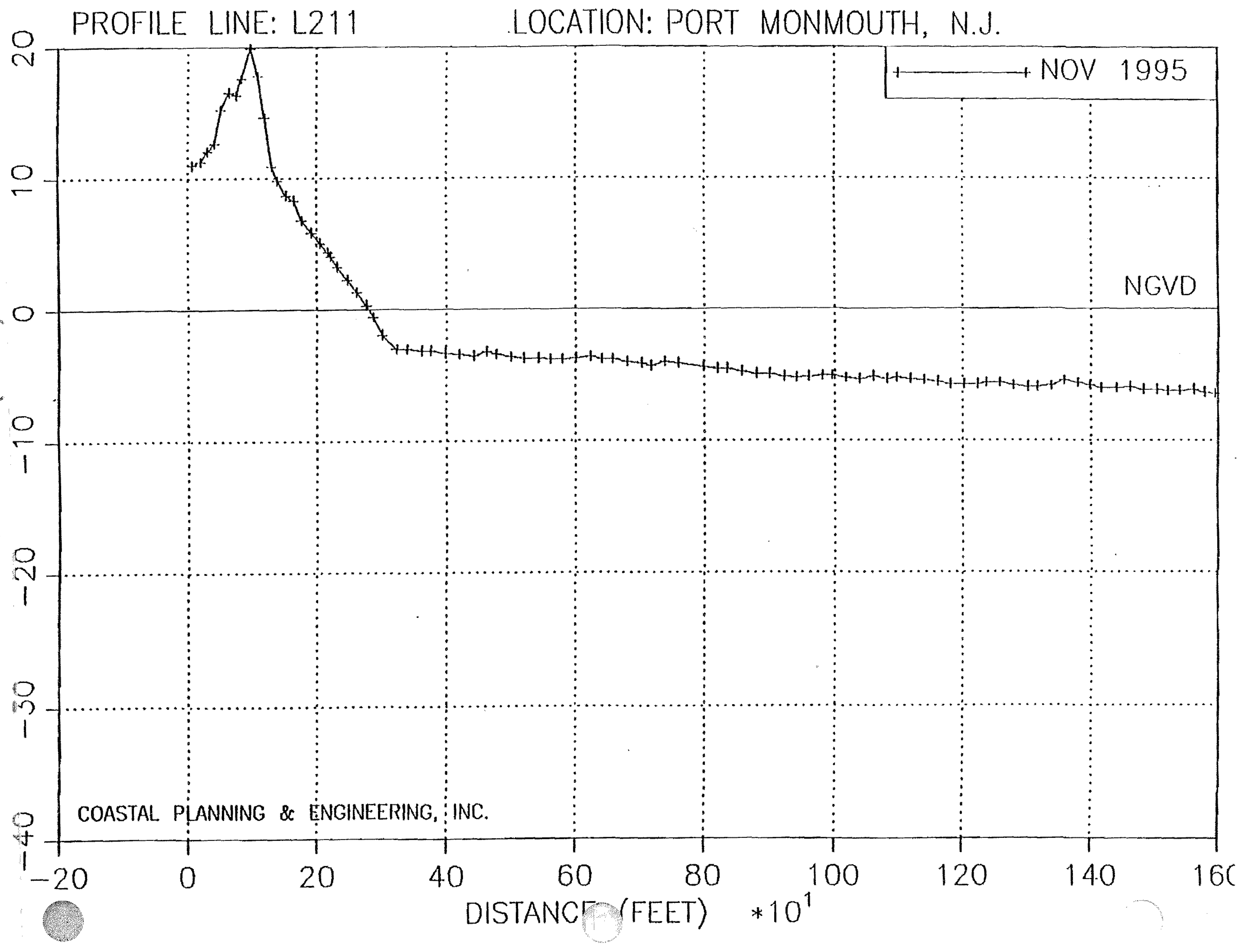
Sub-Appendix A1

Existing Conditions Beach Profiles

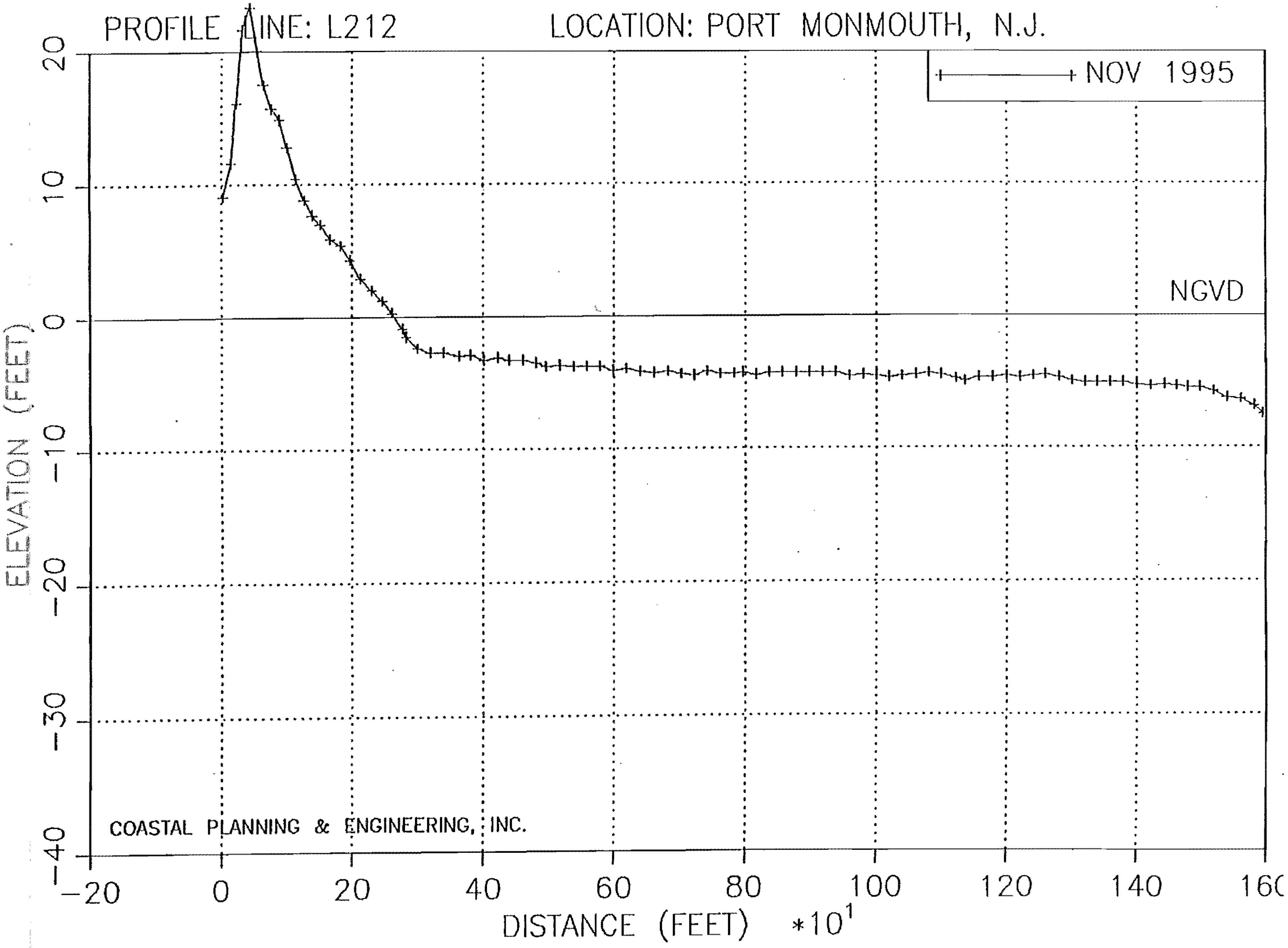
November 1995

PROFILE LINE: L210 LOCATION: PORT MONMOUTH, N.J.




PROFILE LINE: L211
LOCATION: PORT MONMOUTH, N.J.
NOV 1995
NGVD
ELEVATION (FEET)
20
10
0
-10
-20
-30
-40
DISTANCE (FEET) *10^1
-20
0
20
40
60
80
100
120
140
160
COASTAL PLANNING & ENGINEERING, INC.

A1-3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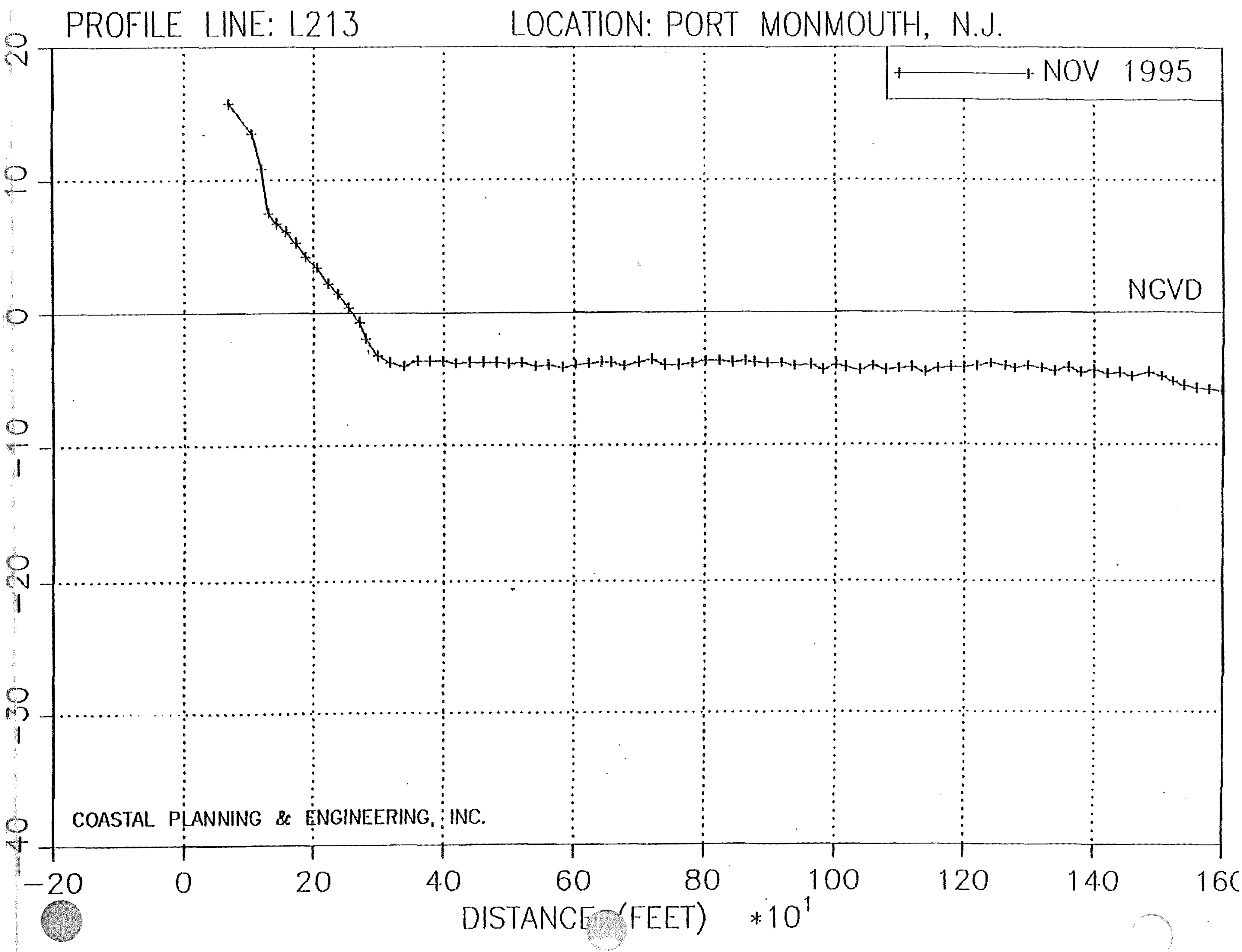
NOV 1995
NGVD
COASTAL PLANNING & ENGINEERING, INC.
ELEVATION (FEET)
20
10
0
-10
-20
-30
-40
-20
0
20
40
60
80
100
120
140
160
DISTANCE (FEET) $*10^1$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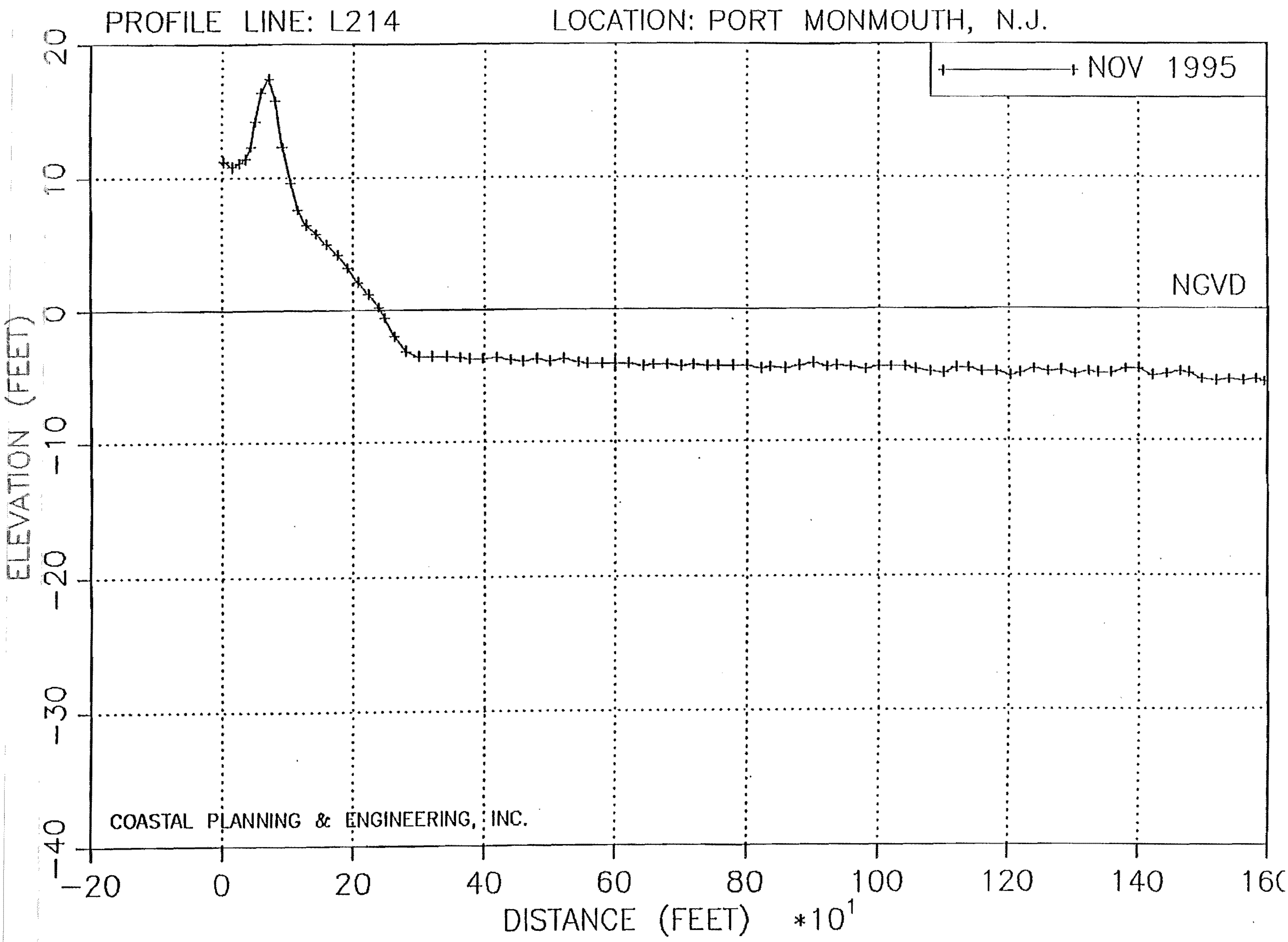
LOCATION: PORT MONMOUTH, N.J.
NOV 1995
NGVD
ELEVATION (FEET)
20
10
0
-10
-20
-30
-40
-20
0
20
40
60
80
100
120
140
160
DISTANCE (FEET) *10^1
COASTAL PLANNING & ENGINEERING, INC.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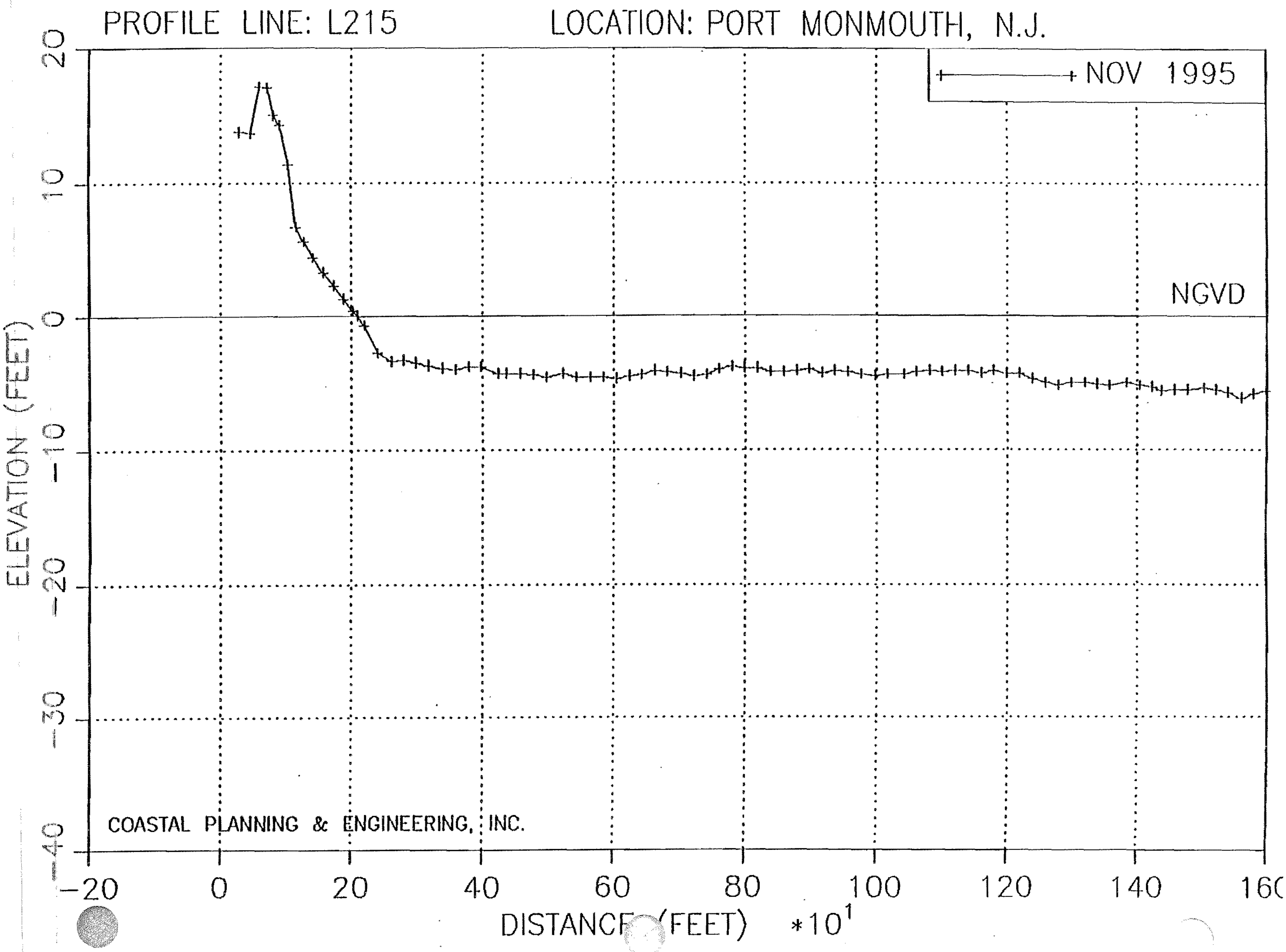
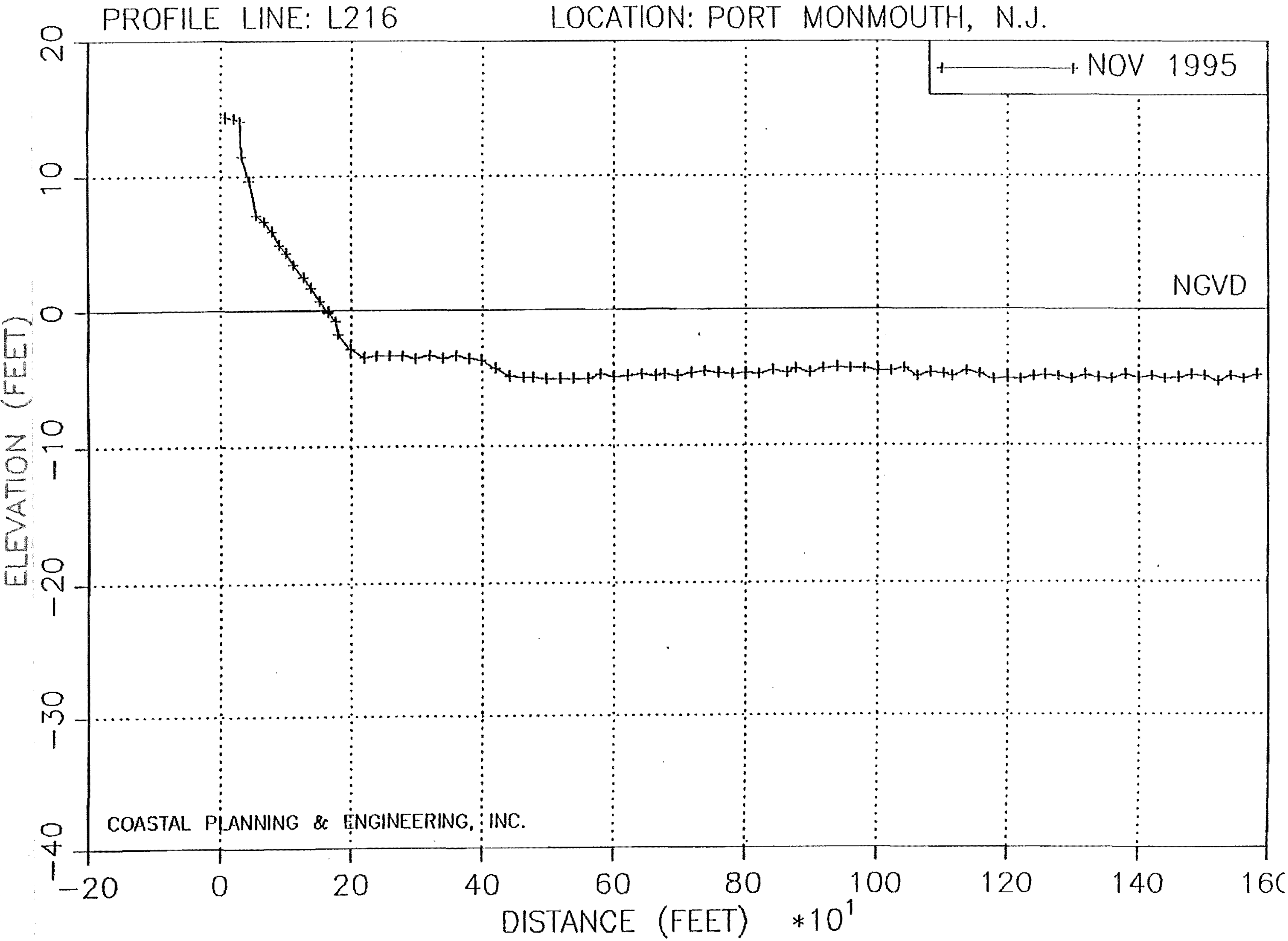
NOV 1995
NGVD
ELEVATION (FEET)
20
10
0
-10
-20
-30
-40
-20
0
20
40
60
80
100
120
140
160
DISTANCE (FEET) *10^1
COASTAL PLANNING & ENGINEERING, INC.


PROFILE LINE: L215
LOCATION: PORT MONMOUTH, N.J.
NOV 1995
NGVD
COASTAL PLANNING & ENGINEERING, INC.
ELEVATION (FEET)
20
10
0
-10
-20
-30
-40
DISTANCE (FEET) *10^1
-20
0
20
40
60
80
100
120
140
16(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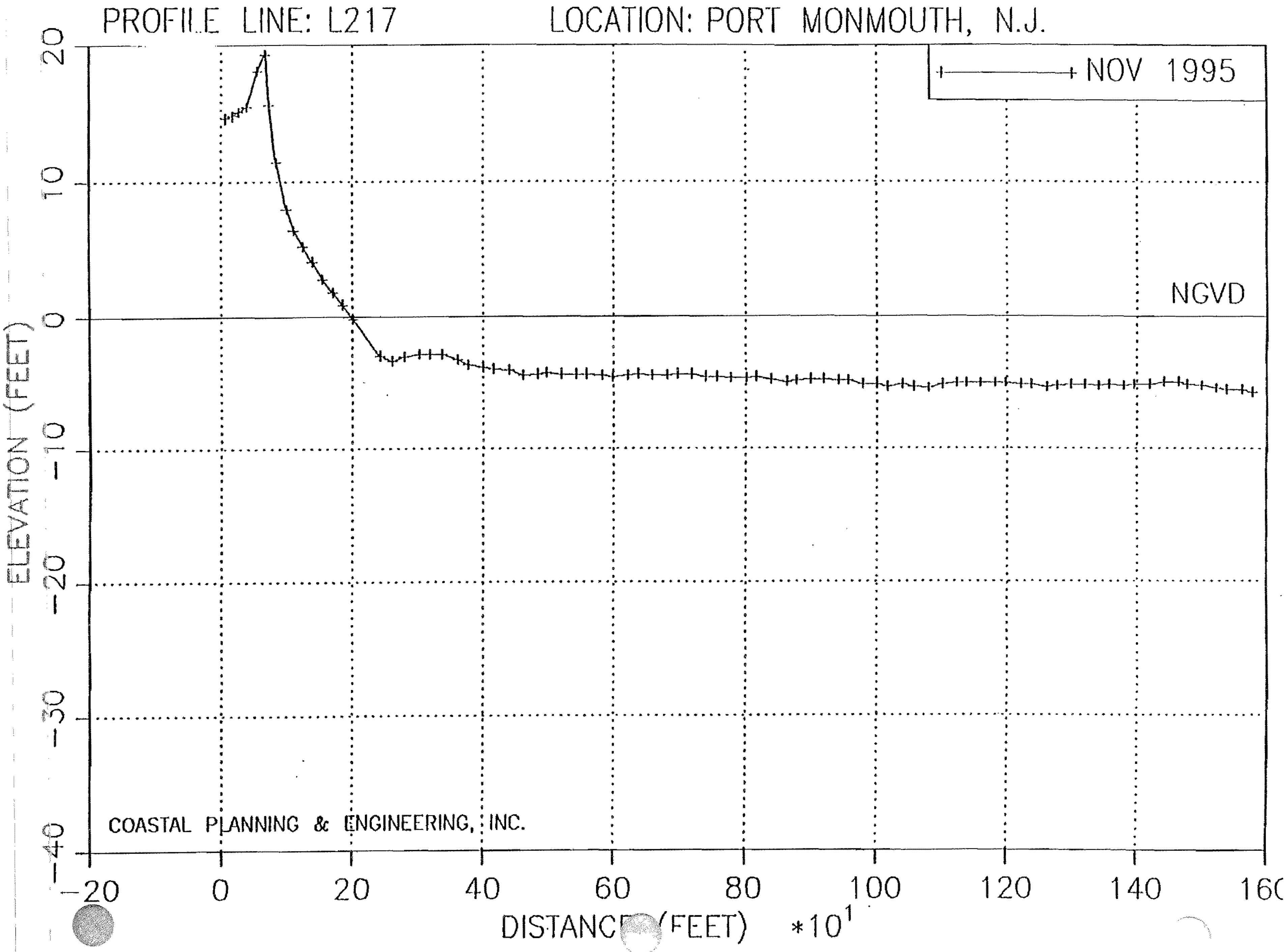
NOV 1995
NGVD
ELEVATION (FEET)
20
10
0
-10
-20
-30
-40
DISTANCE (FEET) *10^1
-20
0
20
40
60
80
100
120
140
160
COASTAL PLANNING & ENGINEERING, INC.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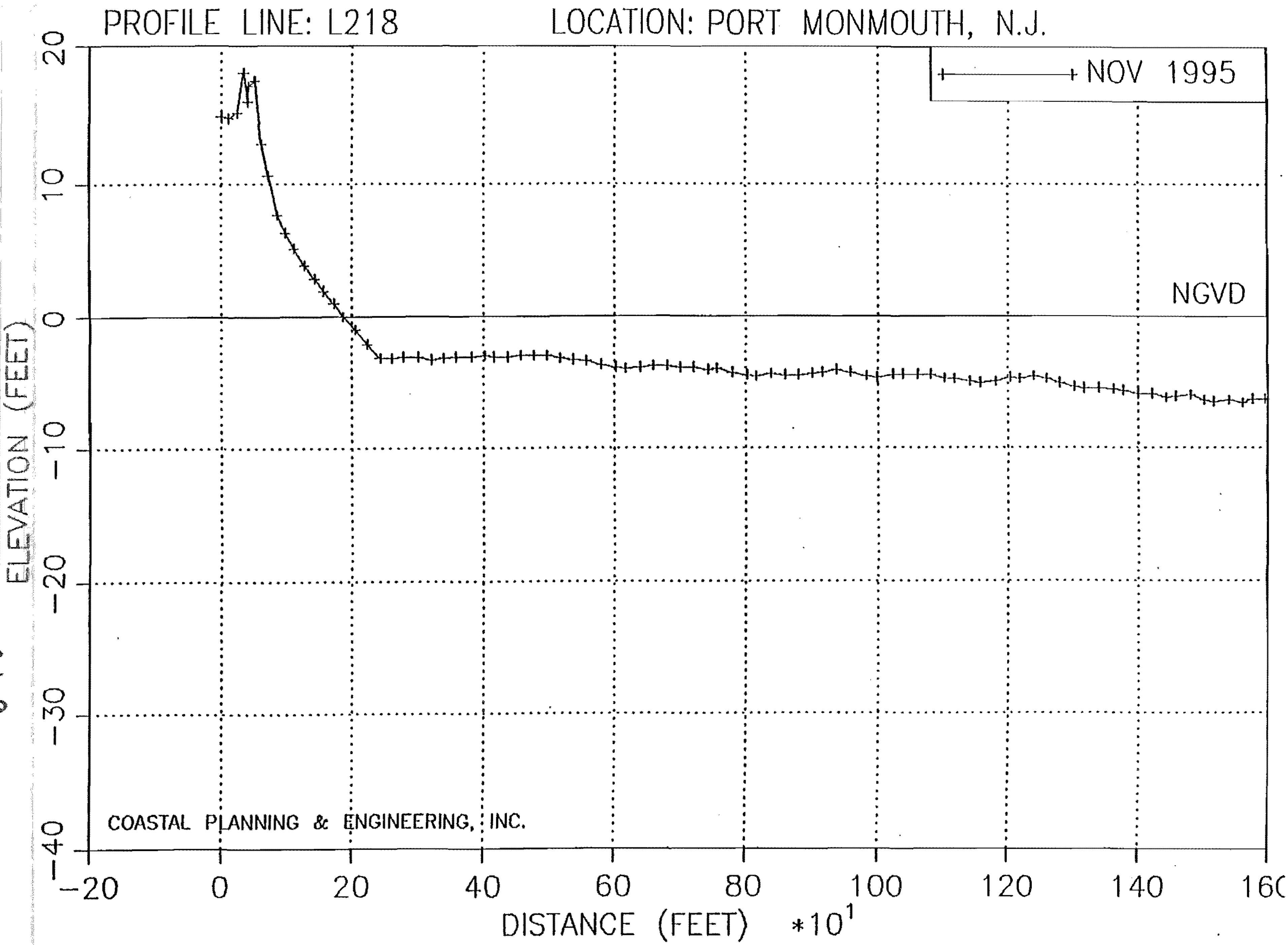
LOCATION: PORT MONMOUTH, N.J.
NOV 1995
NGVD
ELEVATION (FEET)
20
10
0
-10
-20
-30
-40
-20
0
20
40
60
80
100
120
140
160
DISTANCE (FEET) *10¹
COASTAL PLANNING & ENGINEERING, INC.

A1-B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LOCATION: PORT MONMOUTH, N.J.
NOV 1995
NGVD
ELEVATION (FEET)
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10
0
-10
-20
-30
-40
-20
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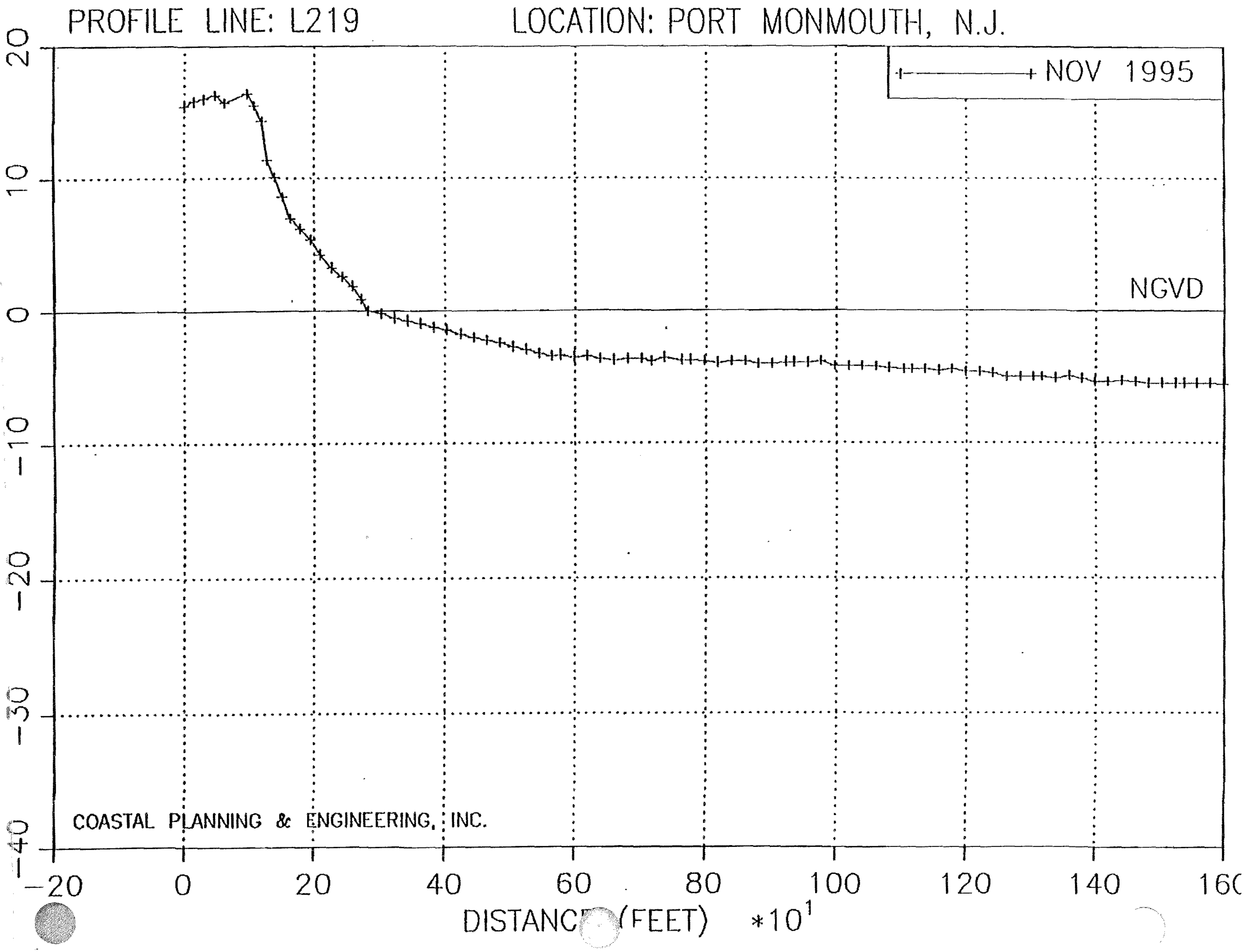
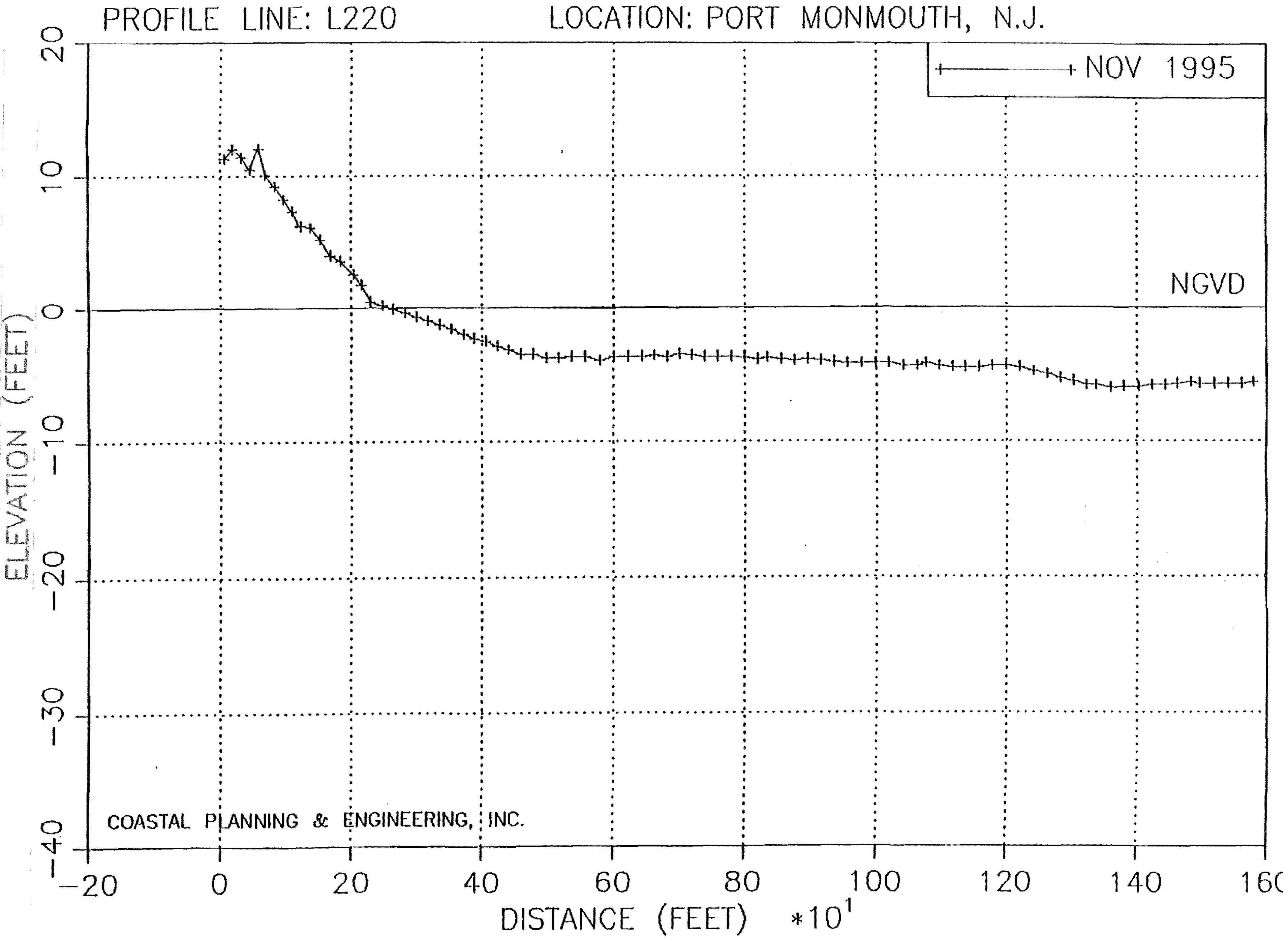
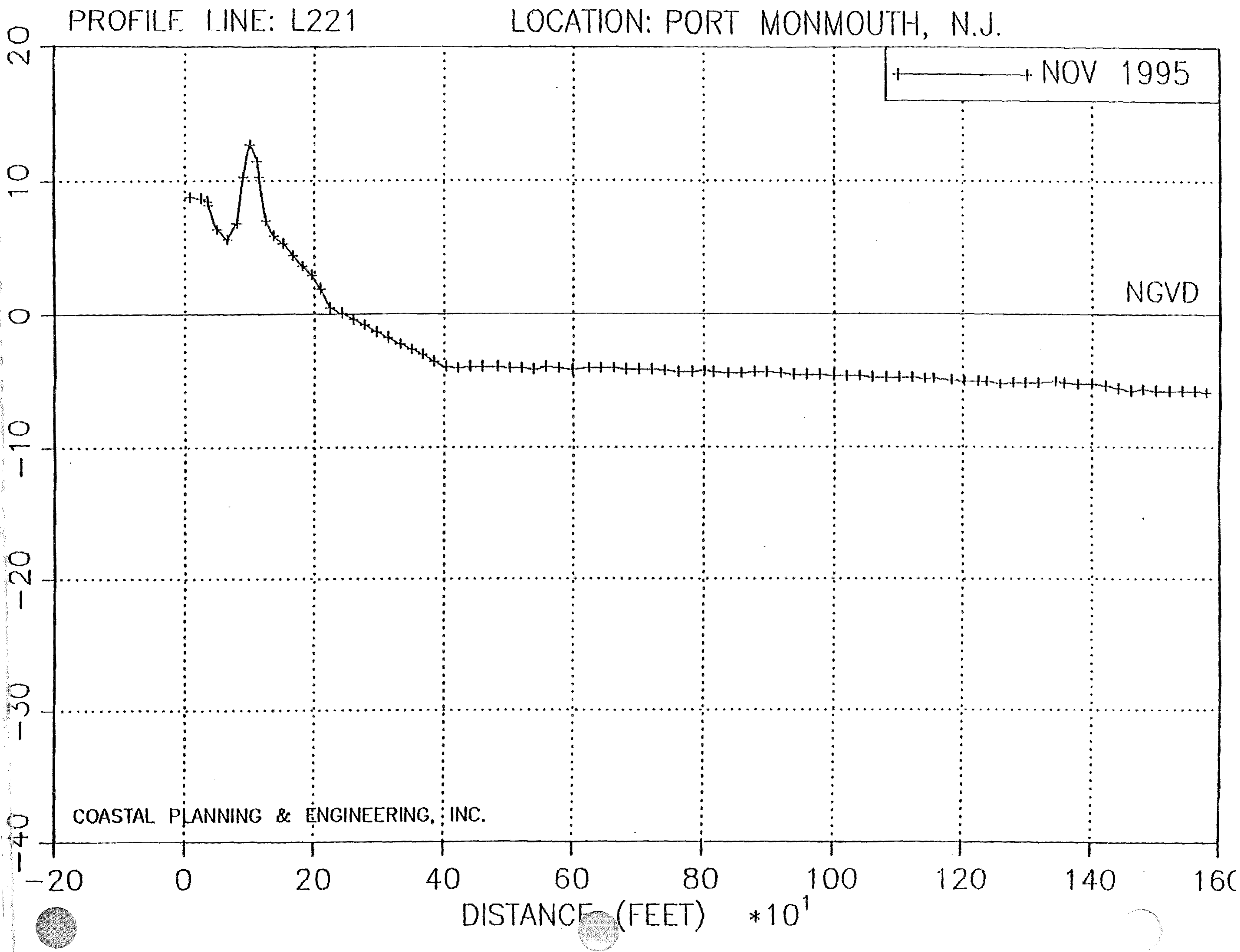
20
40
60
80
100
120
140
160
DISTANCE (FEET) *10¹
COASTAL PLANNING & ENGINEERING, INC.


PROFILE LINE: L219
LOCATION: PORT MONMOUTH, N.J.
NOV 1995
NGVD
ELEVATION (FEET)
20
10
0
−10
−20
−30
−40
DISTANCE (FEET) *10¹
−20
0
20
40
60
80
100
120
140
160
COASTAL PLANNING & ENGINEERING, INC.

A1-10



PROFILE LINE: L220 LOCATION: PORT MONMOUTH, N.J.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NOV 1995
NGVD
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0
−10
−20
−30
−40
COASTAL PLANNING & ENGINEERING, INC.
−20
0
20
40
60
80
100
120
140
160
DISTANCE (FEET) $*10^1$

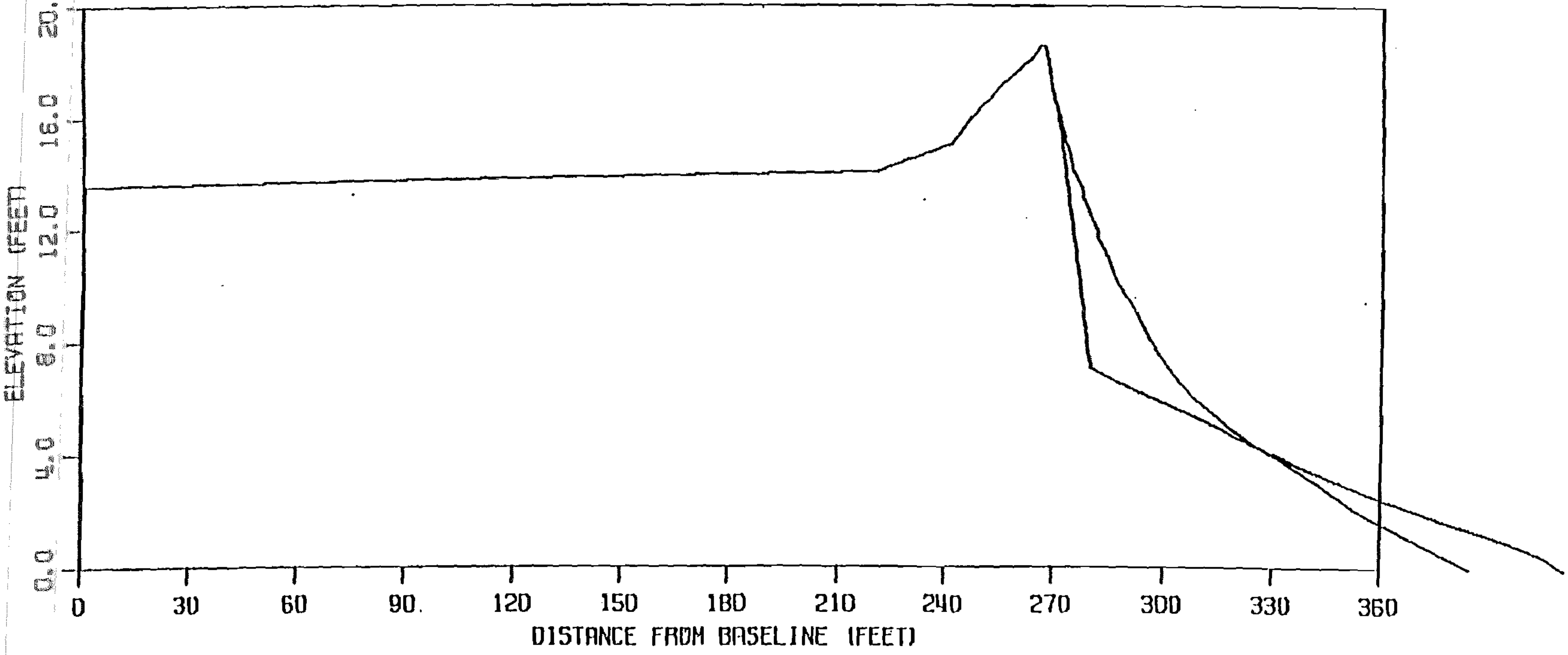
Sub-Appendix A2

Storm Recession - Existing and Improved Conditions

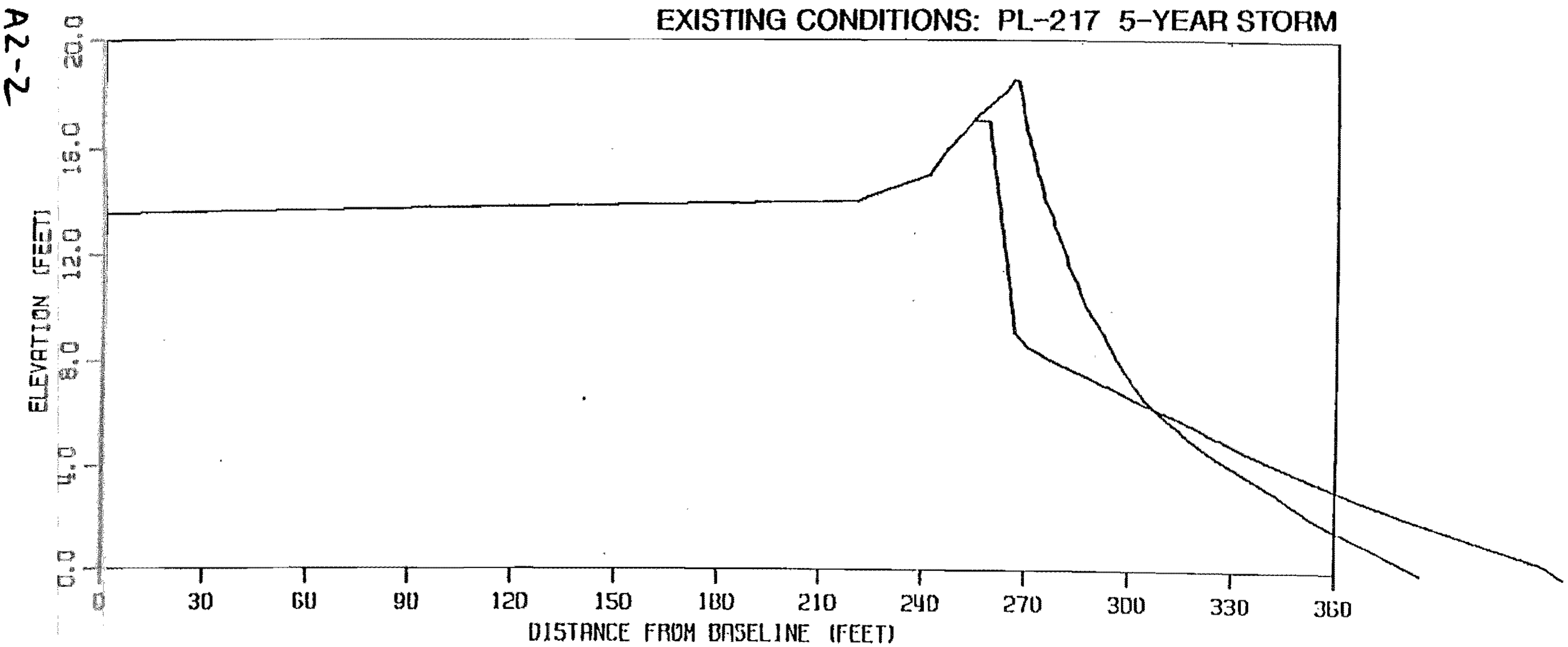
Input and Output Profiles

A2

EXISTING CONDITIONS: PL-217 2-YEAR STORM

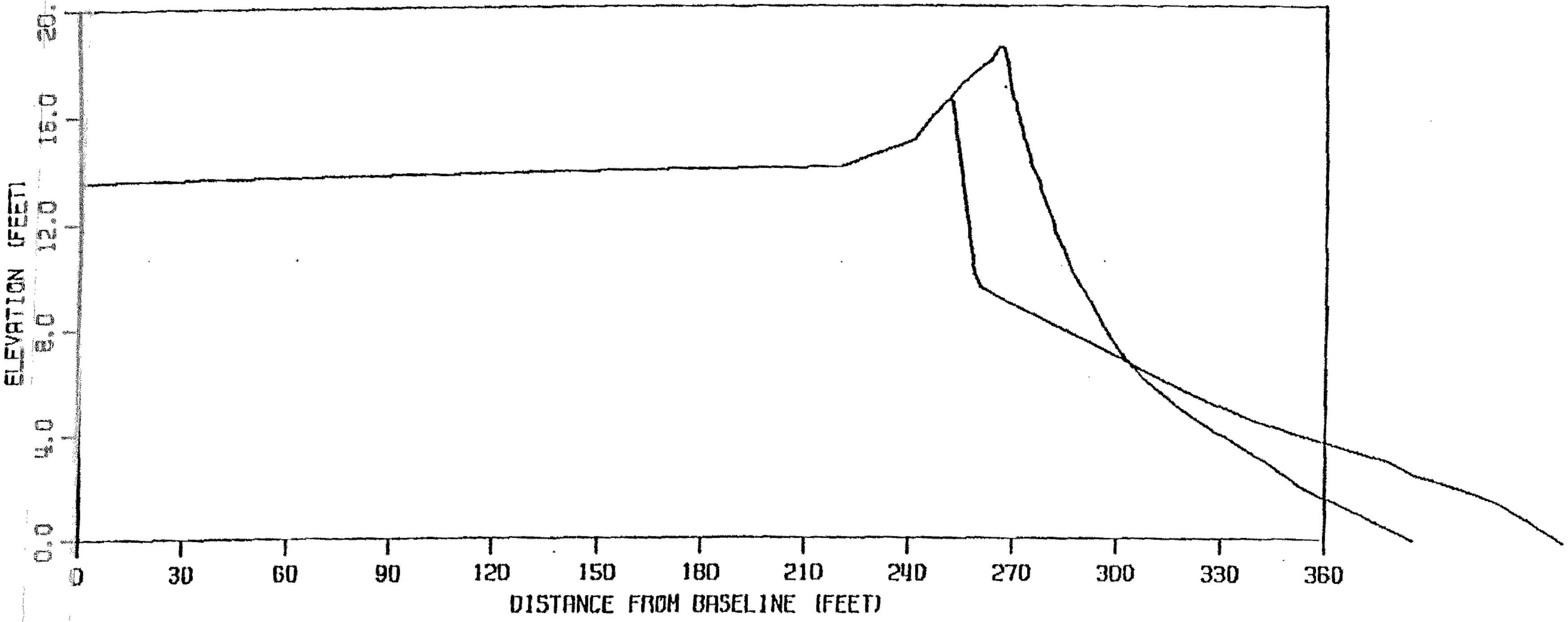


AZ-1



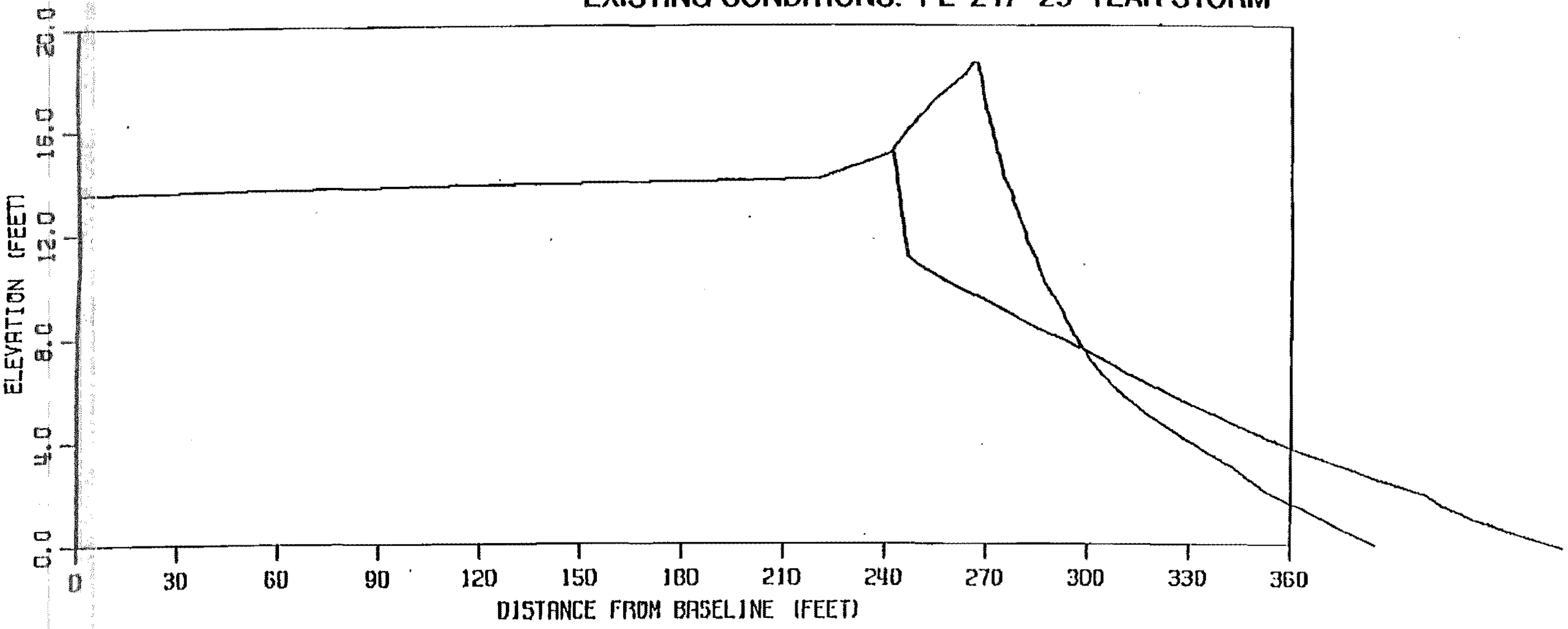
EXISTING CONDITIONS: PL-217 5-YEAR STORM

EXISTING CONDITIONS: PL-217 10-YEAR STORM

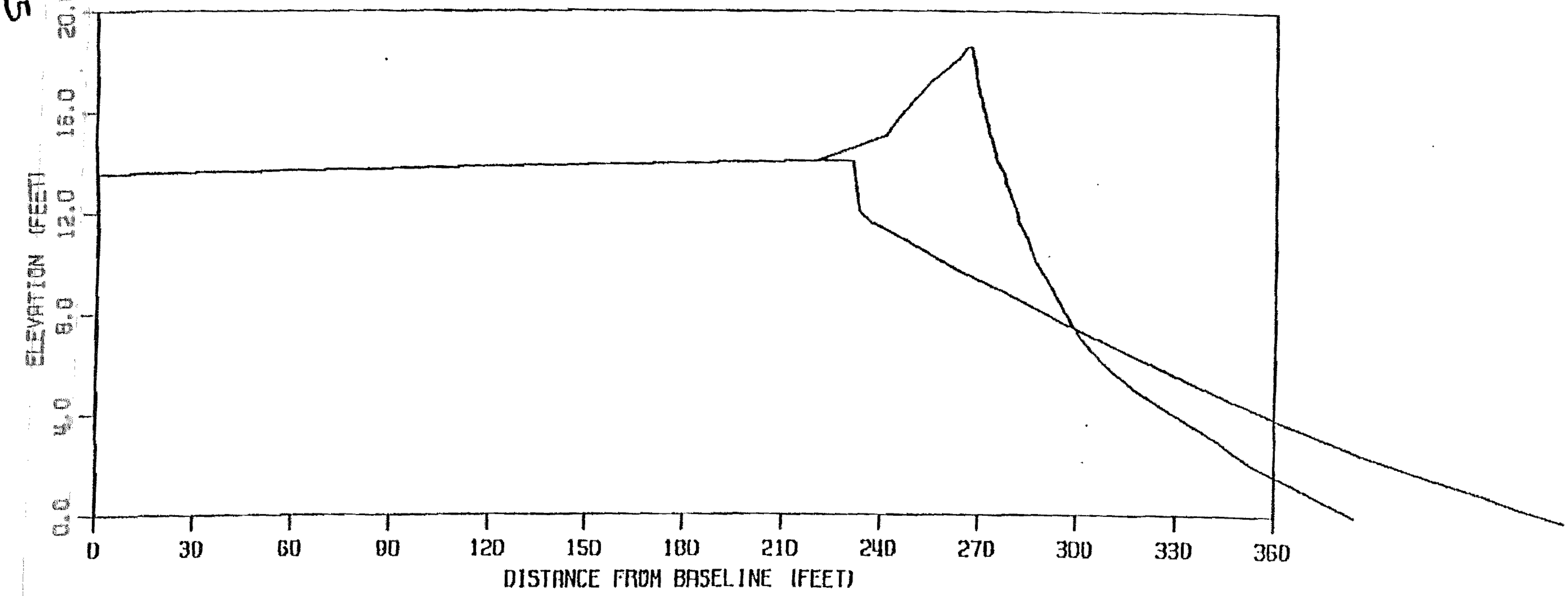


A2-3

EXISTING CONDITIONS: PL-217 25-YEAR STORM



EXISTING CONDITIONS: PL-217 50-YEAR STORM



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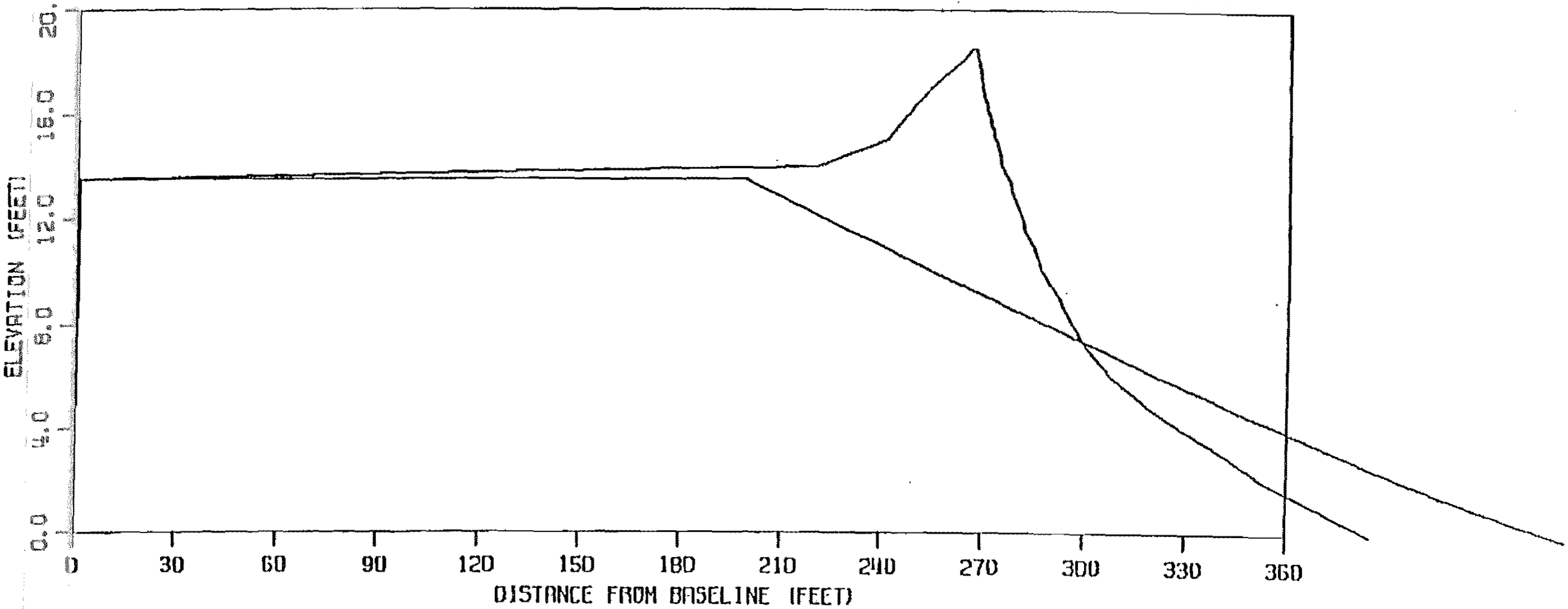

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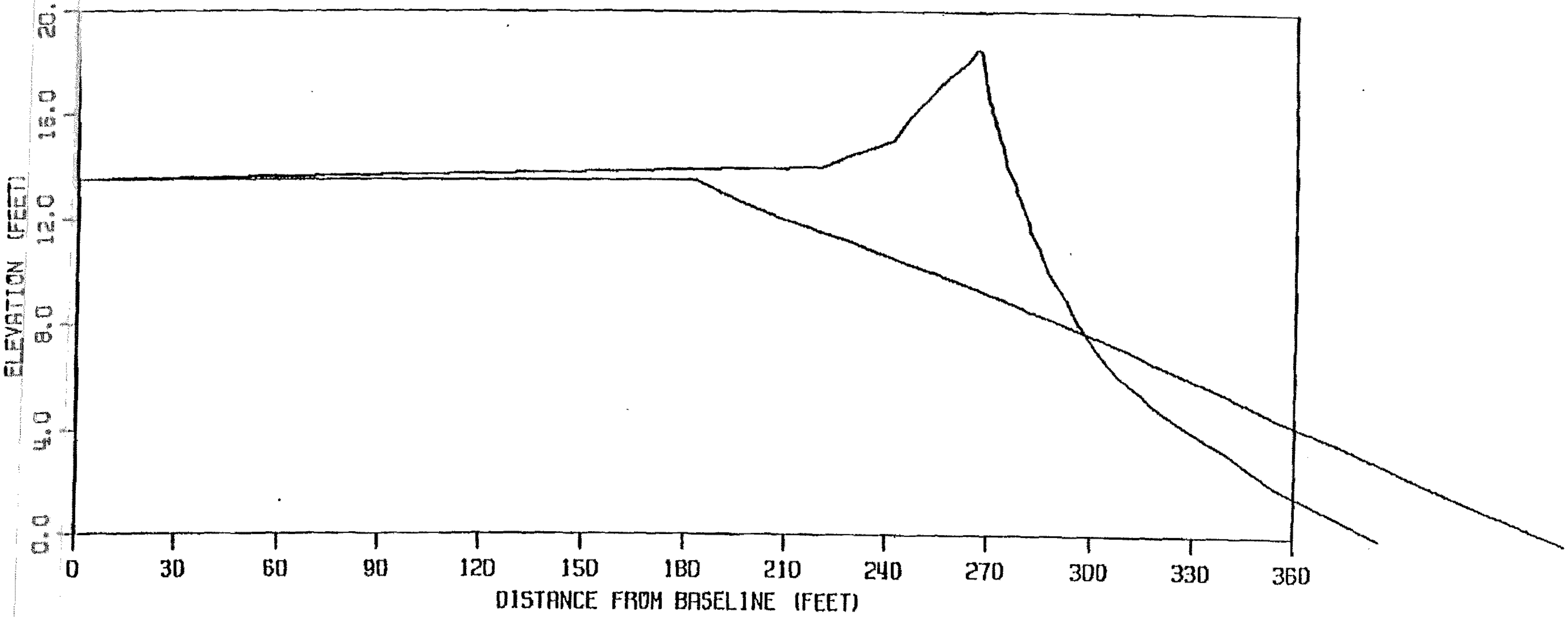
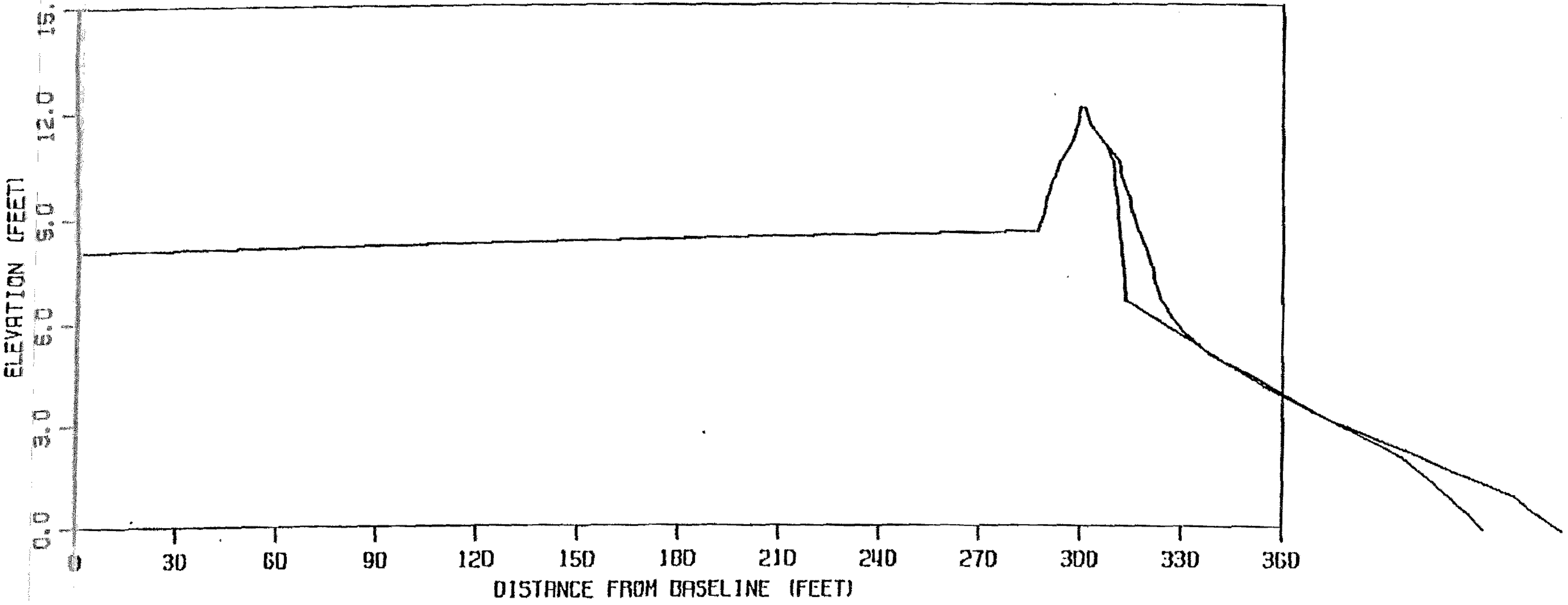
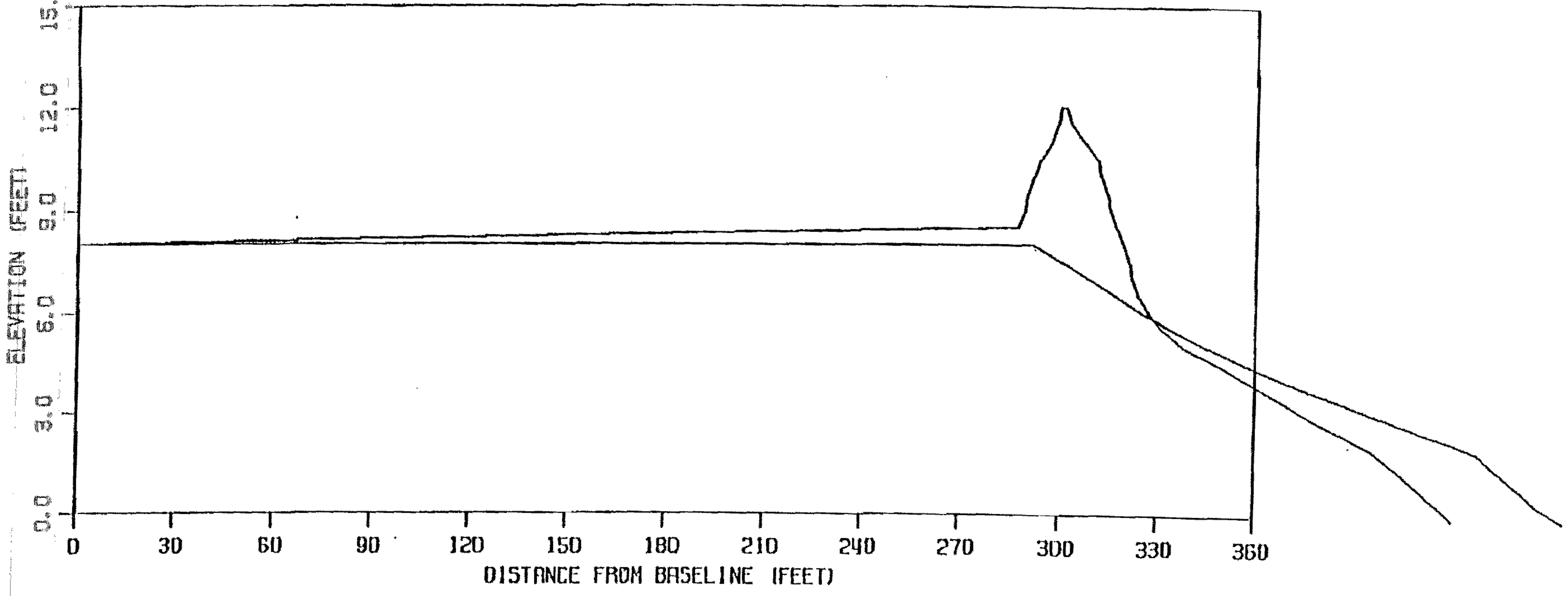


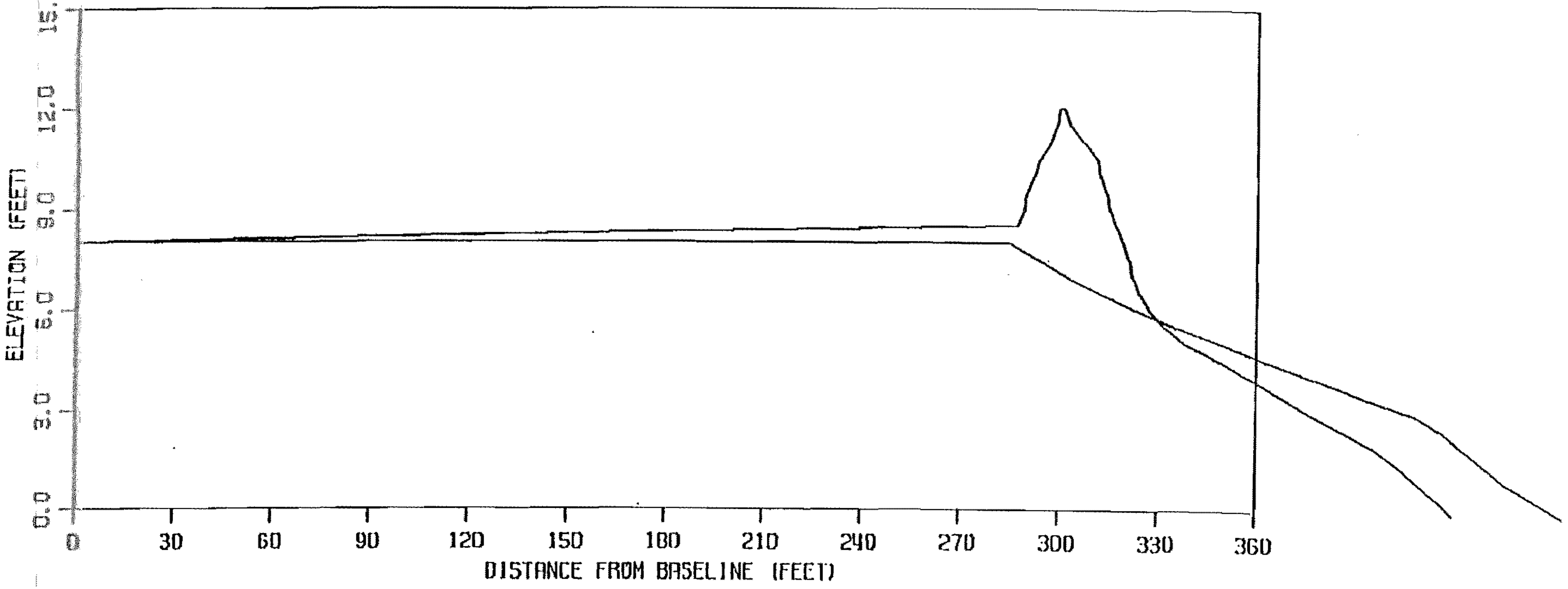


EXISTING CONDITIONS: PL-221 2-YEAR STORM

EXISTING CONDITIONS: PL-221 10-YEAR STORM

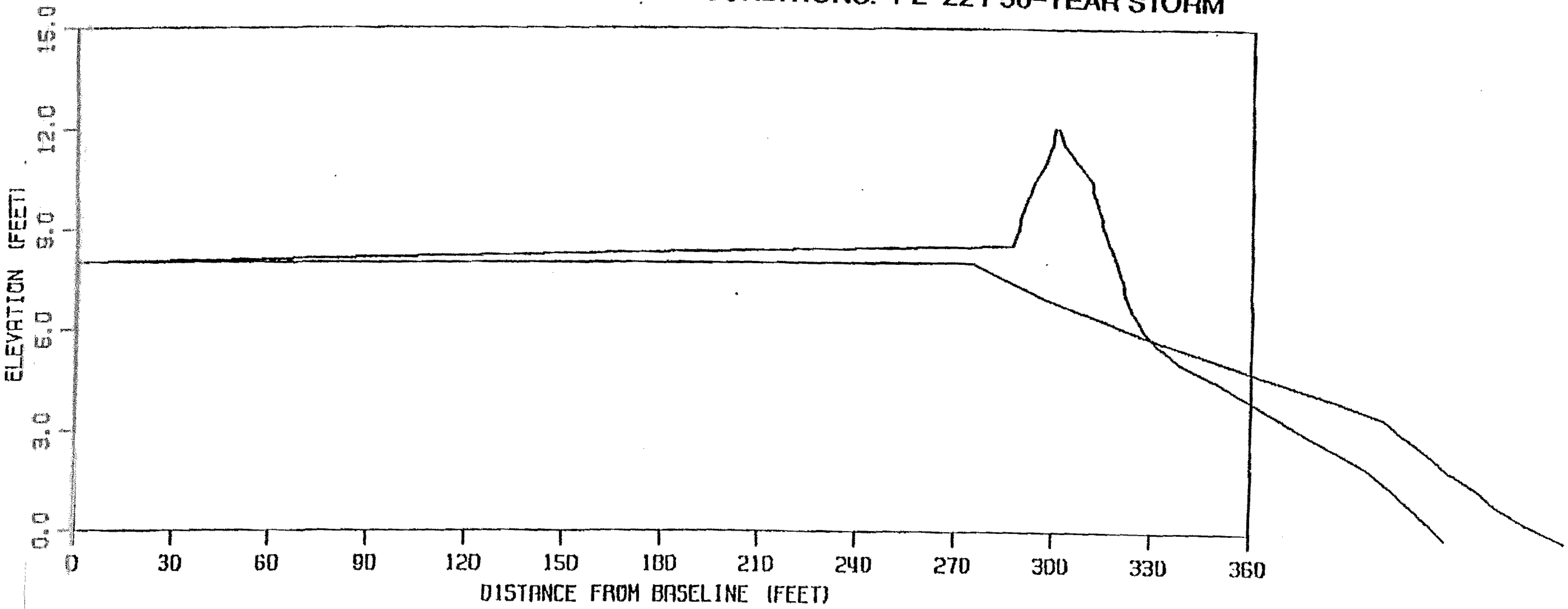


EXISTING CONDITIONS: PL-221 25-YEAR STORM

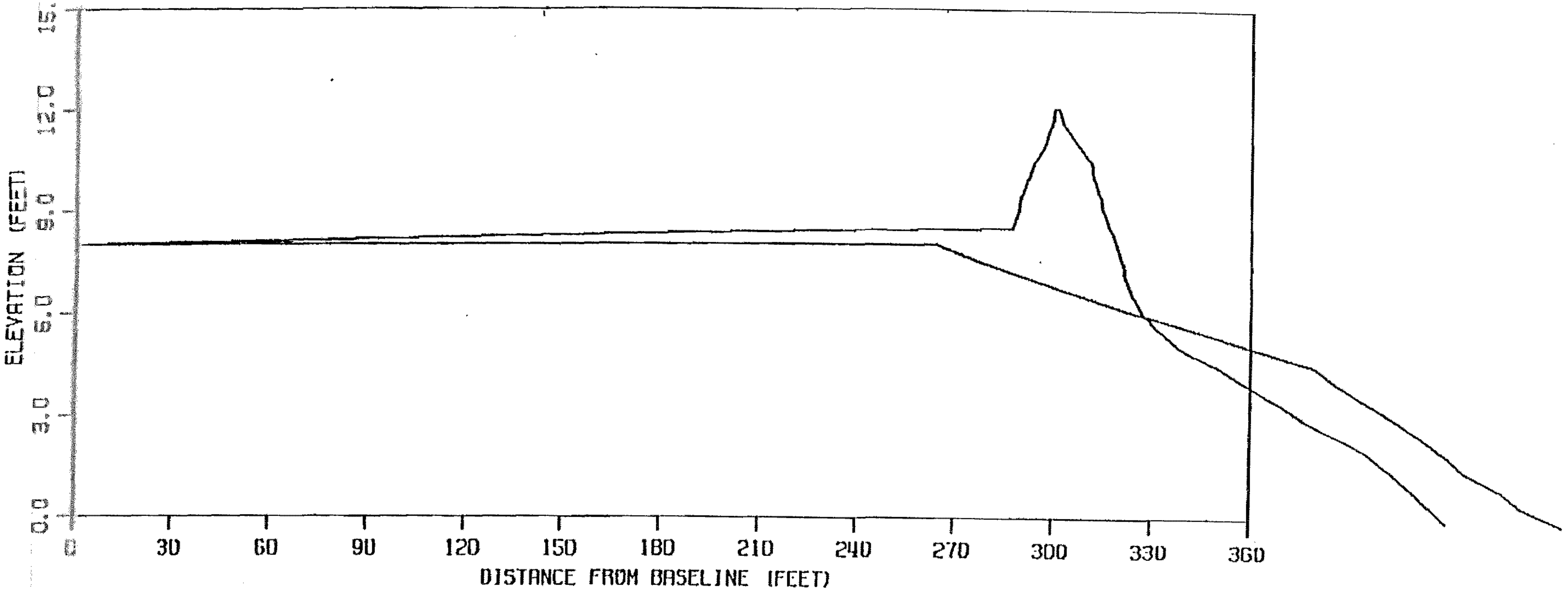


N2-11

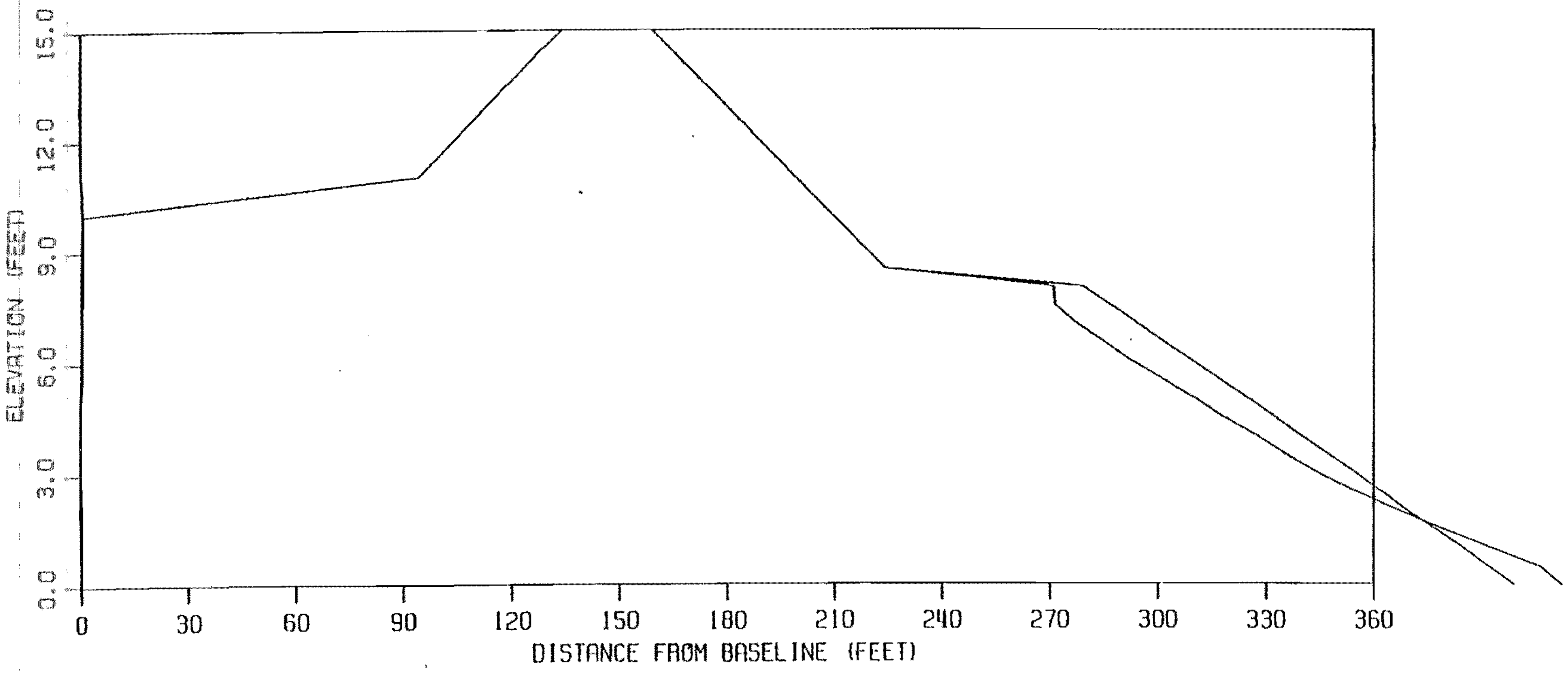
EXISTING CONDITIONS: PL-221 50-YEAR STORM



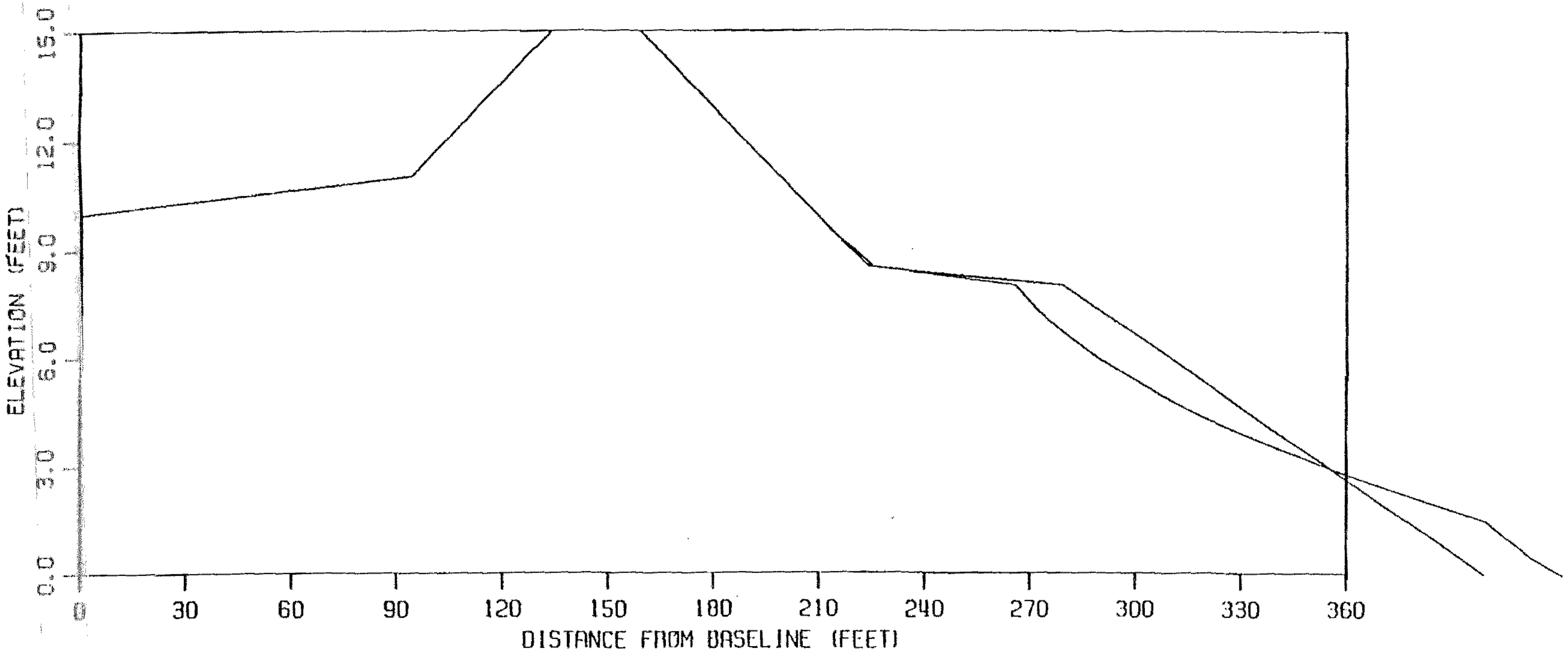
EXISTING CONDITIONS: PL-221 100-YEAR STORM



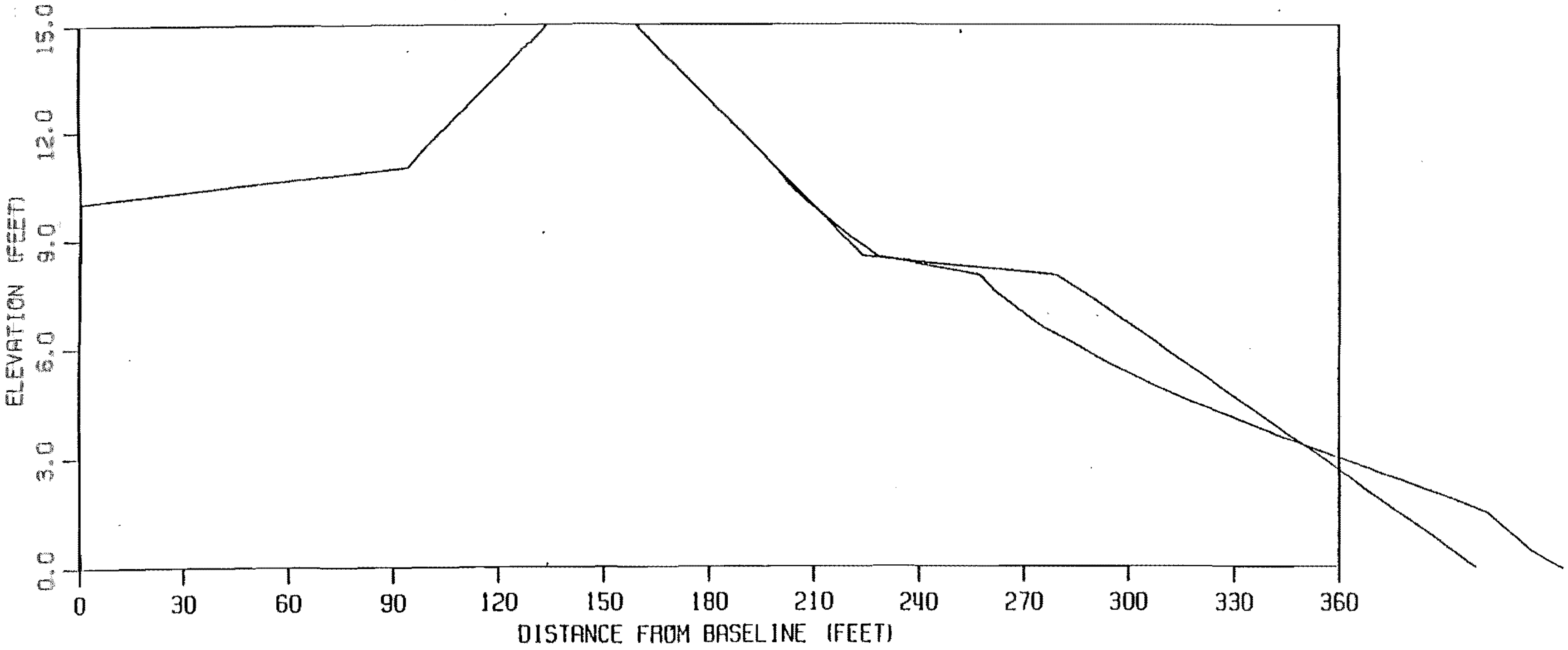
Improved Conditions
2-year storm



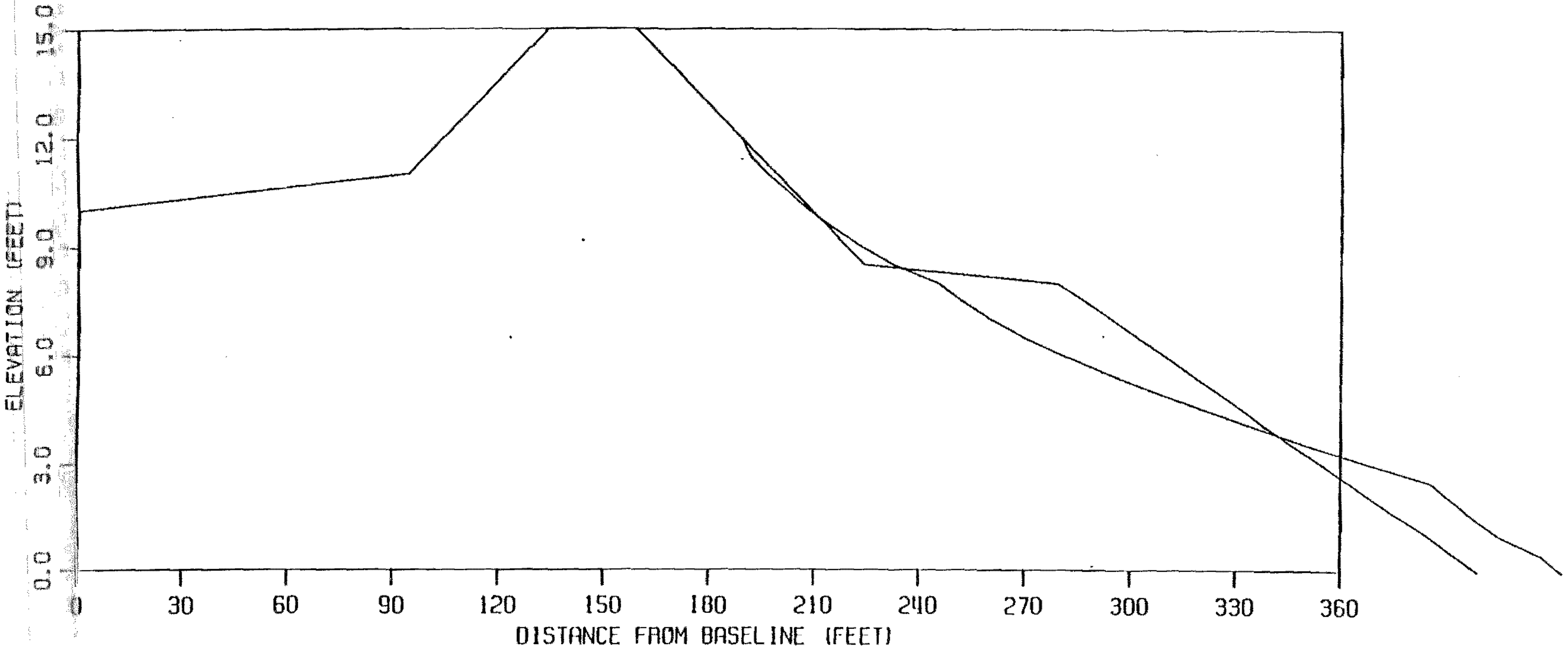
Improved Conditions
5-year storm



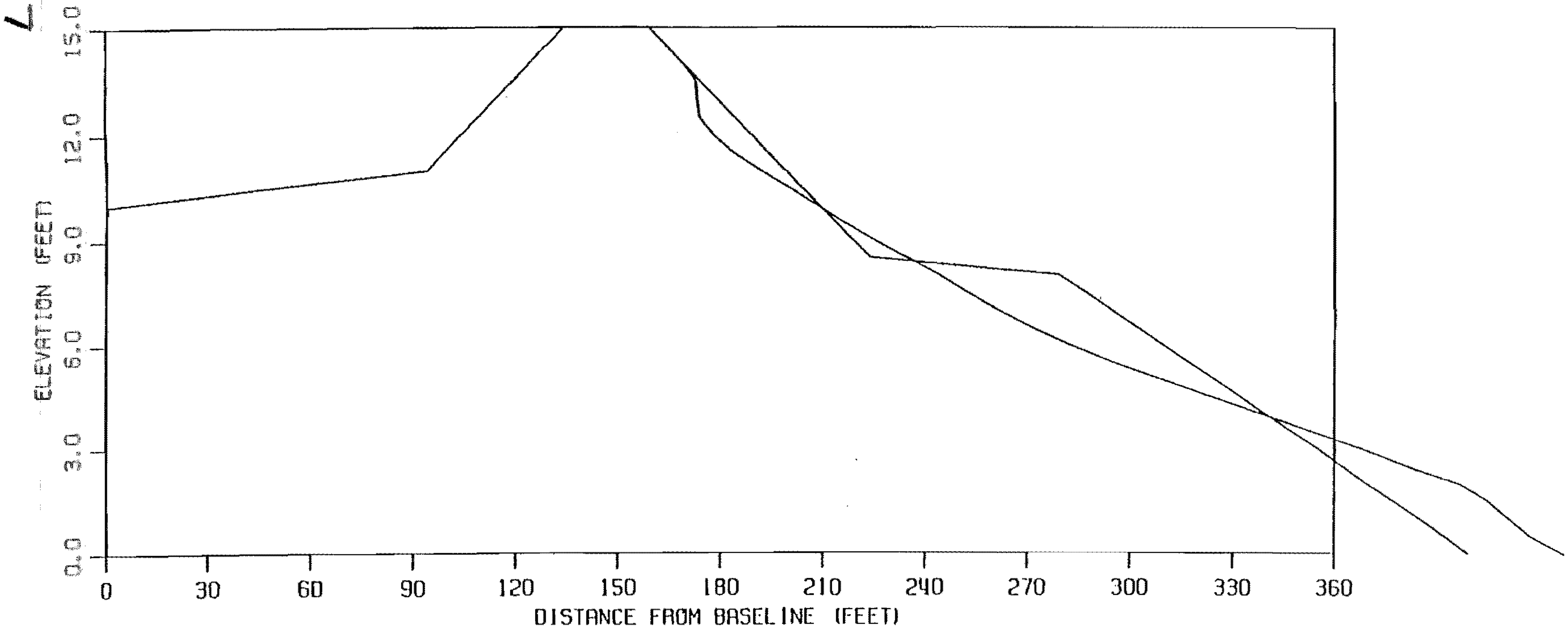
Improved Conditions
10-year storm



Improved Conditions
25-year storm

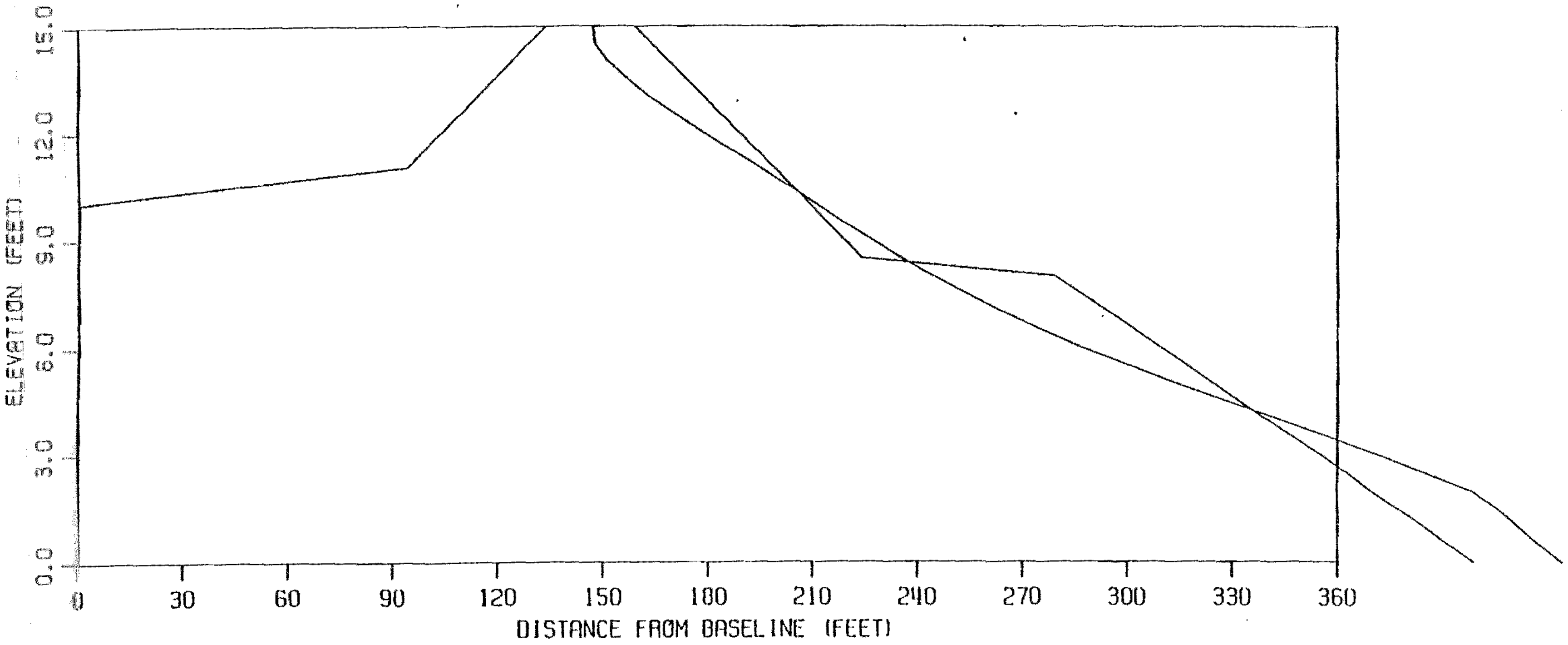


Improved Conditions
50-year storm

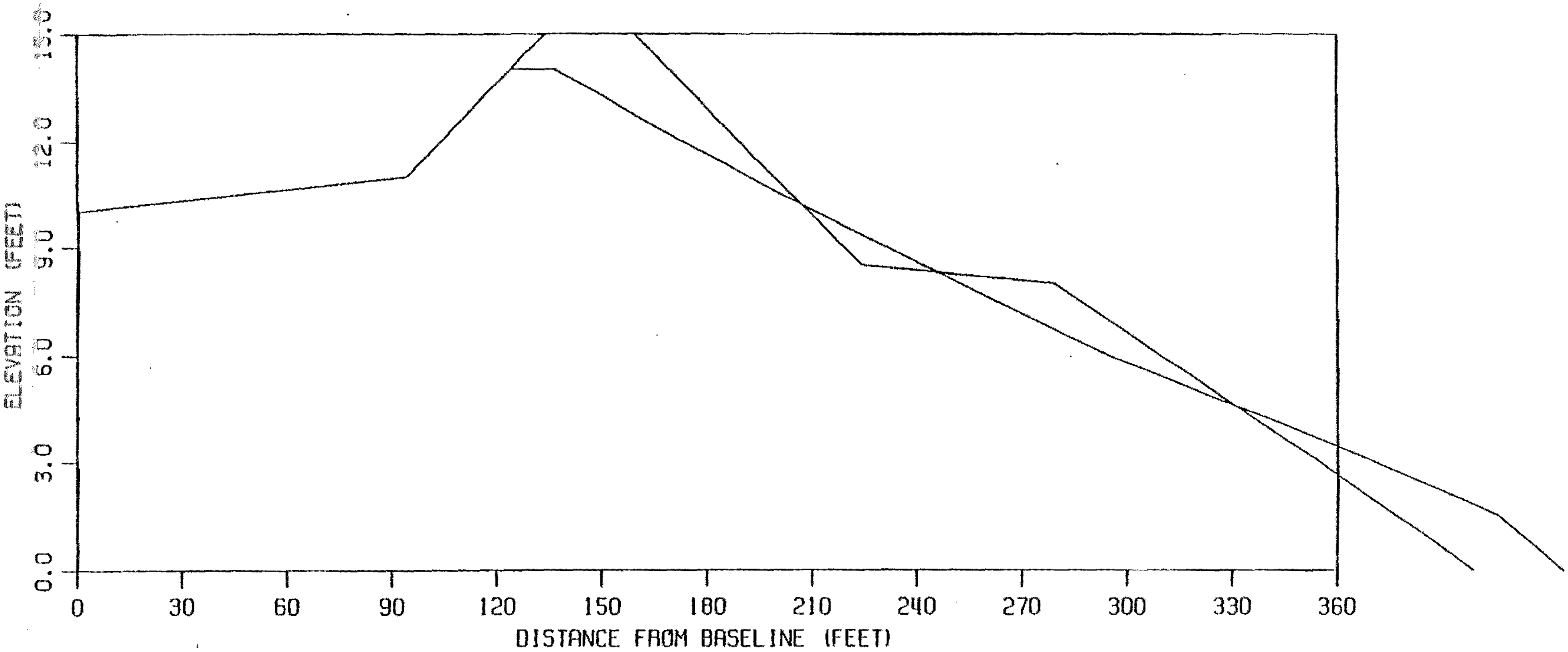


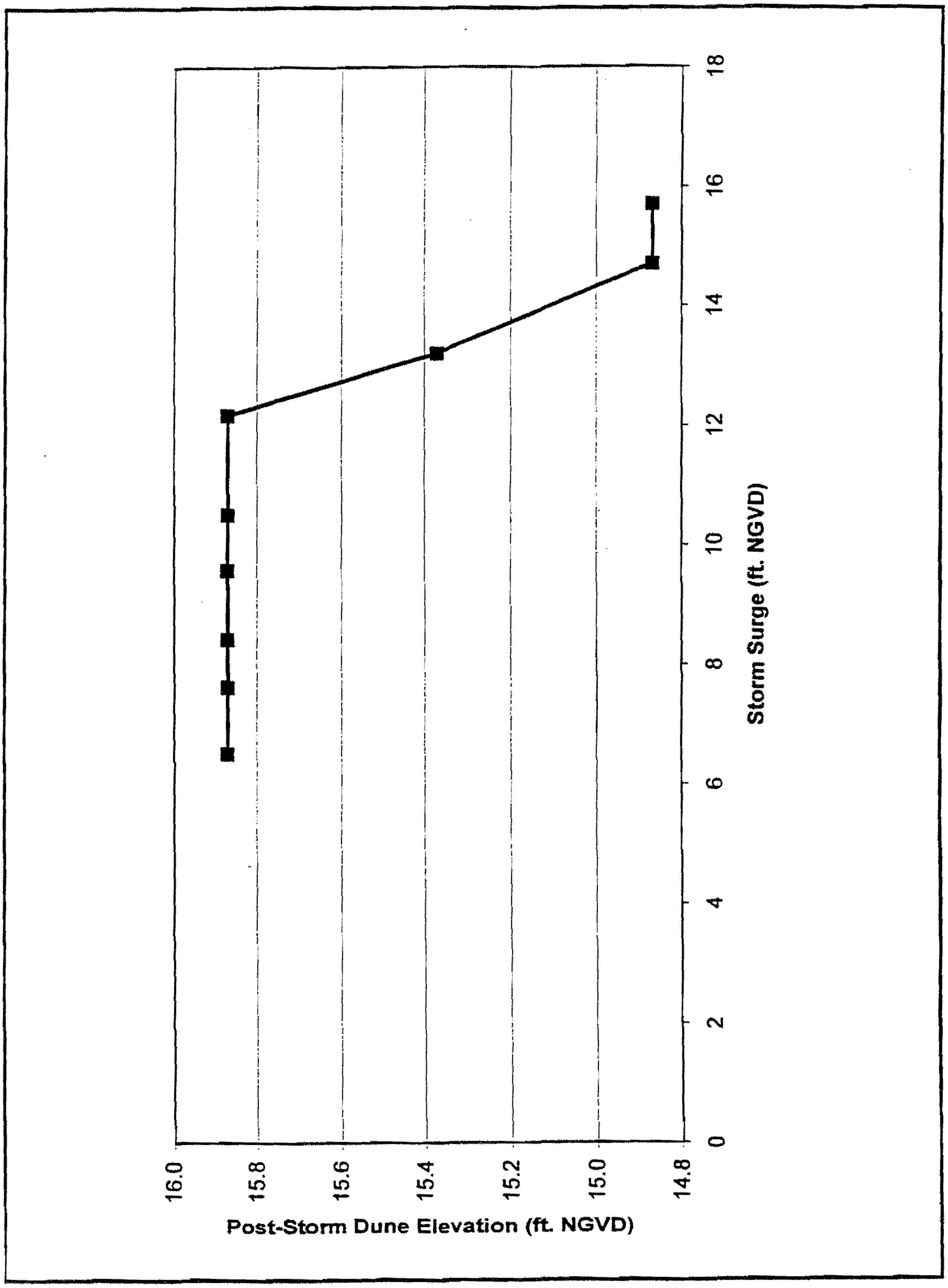
AZ-17

Improved Conditions
100 year level storm



Improved Conditions
200 year level storm.





Post-Storm Dune Elevations and Storm Surge

Predicted by EDUNE on Post-Storm Profiles

Sub-Appendix A3

Coastal Engineering Evaluation
Sample Calculations, Input and Output Parameters

Storm Recession
Wave Runup
Wave Overtopping

Sub-Appendix A3

Coastal Engineering Evaluation
Sample Calculations, Input and Output Parameters

Storm Recession
Wave Runup
Wave Overtopping

```
EDUNE OUTPUT PROFILES
PL-221    2-YEAR STORM    EXISTING CONDITIONS
                                  4.5     6.6     8.8    10.9    13.0      9.8
                                  4.5    16.6    29.5    28.9    28.0     31.9
  -12.0   300.0   301.0   301.0   301.0   301.0   301.0   301.0   301.0
  -11.5   299.0   303.0   303.0   303.0   303.0   303.0   303.0   303.0
  -11.0   297.0   307.0   307.0   307.0   307.0   307.0   307.0   307.0
  -10.5   294.0   311.0   311.0   311.0   309.8   309.3   309.3   309.3
  -10.0   292.0   312.0   312.0   312.0   310.3   309.8   309.8   309.8
   -9.5   290.0   314.0   314.0   314.0   310.8   310.3   310.3   310.3
   -9.0   289.0   315.0   315.0   315.0   311.3   310.8   310.8   310.8
   -8.5   287.0   317.0   317.0   317.0   311.8   311.3   311.3   311.3
   -8.0      .0   319.0   319.0   319.0   312.3   311.8   311.8   311.8
   -7.5      .0   321.0   321.0   321.0   312.8   312.3   312.3   312.3
   -7.0      .0   322.0   322.0   322.0   313.3   312.8   312.8   312.8
   -6.5      .0   324.0   324.0   324.0   313.8   313.7   313.7   313.3
   -6.0      .0   327.0   327.0   324.9   322.2   323.4   323.4   321.9
   -5.5      .0   332.0   332.0   328.1   330.6   331.9   333.7   330.4
   -5.0      .0   338.0   338.0   335.0   339.1   340.4   343.6   338.9
   -4.5      .0   349.0   349.0   345.1   347.5   348.7   353.5   347.4
   -4.0      .0   358.0   357.8   353.8   355.7   356.8   362.5   355.9
   -3.5      .0   367.0   366.6   362.7   365.5   365.4   370.7   365.7
   -3.0      .0   376.0   375.0   371.7   376.4   375.2   378.9   376.9
   -2.5      .0   386.0   383.2   381.6   388.5   386.4   387.4   389.2
   -2.0      .0   396.0   391.3   393.0   401.8   398.7   396.3   402.2
   -1.5      .0   403.0   403.8   405.5   416.4   411.8   405.9   415.4
   -1.0      .0   409.0   416.2   419.2   425.4   425.8   416.6   428.9
    -.5      .0   415.0   426.8   434.4   434.4   440.2   428.9   435.2
     .0      .0   420.0   438.4   443.7   443.7   446.5   442.3   443.7
     .5      .0   430.0   451.0   454.8   454.8   454.8   456.3   454.8
    1.0      .0   450.0   464.8   467.6   467.6   467.6   469.8   467.6
    1.5      .0   470.0   479.9   479.9   479.9   479.9   483.4   479.9
    2.0      .0   489.0   490.6   490.6   490.6   490.6   490.6   490.6
    2.5      .0   510.0   501.1   501.1   501.1   501.1   501.1   501.1
    3.0      .0   528.0   511.3   511.3   511.3   511.3   511.3   511.3
    3.5      .0   550.0   525.7   525.7   525.7   525.7   525.7   525.7
    4.0      .0   571.0   544.6   544.6   544.6   544.6   544.6   544.6
    4.5      .0   570.0   570.0   570.0   570.0   570.0   570.0   570.0
    5.0      .0   881.0   881.0   881.0   881.0   881.0   881.0   881.0
    5.5      .0  1240.0  1240.0  1240.0  1240.0  1240.0  1240.0  1240.0
    6.0      .0  1478.0  1478.0  1478.0  1478.0  1478.0  1478.0  1478.0
    6.5      .0  1680.0  1680.0  1680.0  1680.0  1680.0  1680.0  1680.0
    7.0      .0  2336.0  2336.0  2336.0  2336.0  2336.0  2336.0  2336.0
    7.5      .0  2496.0  2496.0  2496.0  2496.0  2496.0  2496.0  2496.0
    8.0      .0  3082.0  3082.0  3082.0  3082.0  3082.0  3082.0  3082.0
    8.5      .0  3860.0  3860.0  3860.0  3860.0  3860.0  3860.0  3860.0
    9.0      .0  4018.0  4018.0  4018.0  4018.0  4018.0  4018.0  4018.0
    9.5      .0  4041.2  4041.2  4041.2  4041.2  4041.2  4041.2  4041.2
   10.0      .0  4065.1  4065.1  4065.1  4065.1  4065.1  4065.1  4065.1
   10.5      .0  4089.5  4089.5  4089.5  4089.5  4089.5  4089.5  4089.5
   11.0      .0  4114.6  4114.6  4114.6  4114.6  4114.6  4114.6  4114.6
   11.5      .0  4140.2  4140.2  4140.2  4140.2  4140.2  4140.2  4140.2
   12.0      .0  4166.3  4166.3  4166.3  4166.3  4166.3  4166.3  4166.3
   12.5      .0  4193.1  4193.1  4193.1  4193.1  4193.1  4193.1  4193.1
   13.0      .0  4220.3  4220.3  4220.3  4220.3  4220.3  4220.3  4220.3
   13.5      .0  4248.1  4248.1  4248.1  4248.1  4248.1  4248.1  4248.1
   14.0      .0  4276.5  4276.5  4276.5  4276.5  4276.5  4276.5  4276.5
   14.5      .0  4305.3  4305.3  4305.3  4305.3  4305.3  4305.3  4305.3
   15.0      .0  4334.6  4334.6  4334.6  4334.6  4334.6  4334.6  4334.6
   15.5      .0  4364.4  4364.4  4364.4  4364.4  4364.4  4364.4  4364.4
```

+11ft. MT
did not r

+10.5 ft. M
contour reced
to 309.3

Interpolating
NGVO (-0.87f
contour is at
station 445

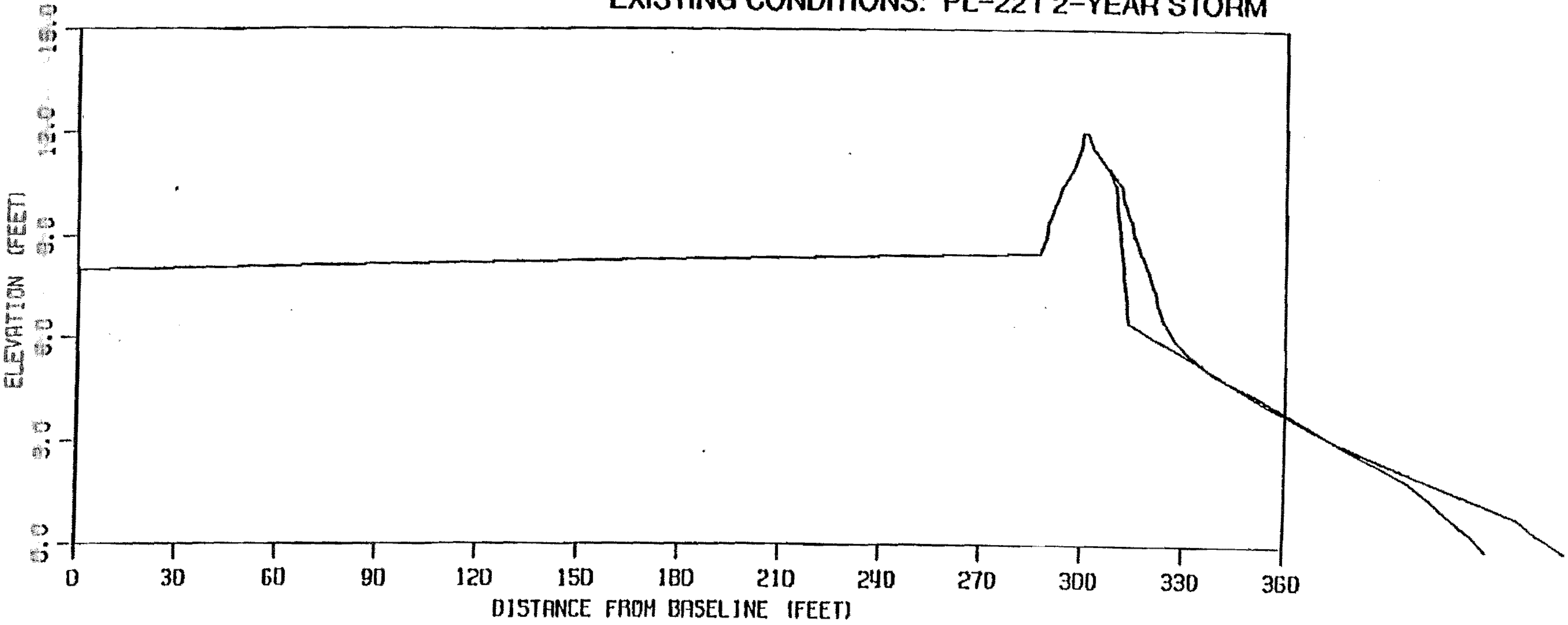
Recession Dist
from 0' NGV

445-309=13
consistent
Table 2

3-2

A3-3

EXISTING CONDITIONS: PL-221 2-YEAR STORM



```
EDUNE OUTPUT PROFILES
PL-221    100-YEAR STORM   EXISTING CONDITIONS
                                  4.6      6.7      8.8     11.0     13.1      9.7
                                 27.6    117.0    137.7    135.3    110.9    138.1
   -12.0    300.0    301.0    301.0    300.0    300.0    300.0    300.0    300.0
   -11.5    299.0    303.0    303.0    299.0    299.0    299.0    299.0    299.0
   -11.0    297.0    307.0    304.0    297.0    297.0    297.0    297.0    297.0
   -10.5    294.0    311.0    304.5    294.0    294.0    294.0    294.0    294.0
   -10.0    292.0    312.0    305.0    292.0    292.0    292.0    292.0    292.0
    -9.5    290.0    314.0    305.5    290.0    290.0    290.0    290.0    290.0
    -9.0    289.0    315.0    306.0    289.0    289.0    289.0    289.0    289.0
    -8.5    287.0    317.0    306.5    287.0    287.0    287.0    287.0    287.0
    -8.0       .0    319.0    311.9    280.5    263.7    270.5    286.7    264.1
    -7.5       .0    321.0    317.2    291.1    278.0    281.0    296.0    277.9
    -7.0       .0    322.0    322.6    301.9    293.1    293.4    304.6    292.8
    -6.5       .0    324.0    327.9    312.9    309.0    307.3    313.4    308.7
    -6.0       .0    327.0    333.3    324.4    325.8    322.2    322.5    325.3
    -5.5       .0    332.0    338.6    336.5    343.5    337.8    332.2    342.4
    -5.0       .0    338.0    344.0    349.3    362.0    353.7    342.2    360.7
    -4.5       .0    349.0    349.3    363.1    376.2    369.5    352.7    379.4
    -4.0       .0    358.0    357.3    378.4    386.9    384.8    363.6    386.9
    -3.5       .0    367.0    365.8    395.6    395.6    400.7    376.0    395.6
    -3.0       .0    376.0    374.9    404.9    404.9    409.4    389.2    404.9
    -2.5       .0    386.0    385.1    412.8    412.8    415.7    402.8    412.8
    -2.0       .0    396.0    396.3    419.2    419.2    421.9    416.5    419.2
    -1.5       .0    403.0    408.5    425.4    425.4    428.2    430.1    425.4
    -1.0       .0    409.0    421.9    436.2    436.2    436.2    443.8    436.2
     -.5       .0    415.0    436.7    443.5    443.5    443.5    457.5    443.5
      .0       .0    420.0    453.6    456.5    456.5    456.5    469.9    456.5
      .5       .0    430.0    467.4    467.4    467.4    467.4    484.3    467.4
     1.0       .0    450.0    482.1    482.1    482.1    482.1    493.1    482.1
     1.5       .0    470.0    494.7    494.7    494.7    494.7    499.3    494.7
     2.0       .0    489.0    510.3    510.3    510.3    510.3    510.3    510.3
     2.5       .0    510.0    525.5    525.5    525.5    525.5    525.5    525.5
     3.0       .0    528.0    544.5    544.5    544.5    544.5    544.5    544.5
     3.5       .0    550.0    568.0    568.0    568.0    568.0    568.0    568.0
     4.0       .0    571.0    605.1    605.1    605.1    605.1    605.1    605.1
     4.5       .0    570.0    648.0    648.0    648.0    648.0    648.0    648.0
     5.0       .0    881.0    819.7    819.7    819.7    819.7    819.7    819.7
     5.5       .0   1240.0   1184.7   1184.7   1184.7   1184.7   1184.7   1184.7
     6.0       .0   1478.0   1364.5   1364.5   1364.5   1364.5   1364.5   1364.5
     6.5       .0   1680.0   1594.2   1594.2   1594.2   1594.2   1594.2   1594.2
     7.0       .0   2336.0   2336.0   2336.0   2336.0   2336.0   2336.0   2336.0
     7.5       .0   2496.0   2496.0   2496.0   2496.0   2496.0   2496.0   2496.0
     8.0       .0   3082.0   3082.0   3082.0   3082.0   3082.0   3082.0   3082.0
     8.5       .0   3860.0   3860.0   3860.0   3860.0   3860.0   3860.0   3860.0
     9.0       .0   4018.0   4018.0   4018.0   4018.0   4018.0   4018.0   4018.0
     9.5       .0   4041.2   4041.2   4041.2   4041.2   4041.2   4041.2   4041.2
    10.0       .0   4065.1   4065.1   4065.1   4065.1   4065.1   4065.1   4065.1
    10.5       .0   4089.5   4089.5   4089.5   4089.5   4089.5   4089.5   4089.5
    11.0       .0   4114.6   4114.6   4114.6   4114.6   4114.6   4114.6   4114.6
    11.5       .0   4140.2   4140.2   4140.2   4140.2   4140.2   4140.2   4140.2
    12.0       .0   4166.3   4166.3   4166.3   4166.3   4166.3   4166.3   4166.3
    12.5       .0   4193.1   4193.1   4193.1   4193.1   4193.1   4193.1   4193.1
    13.0       .0   4220.3   4220.3   4220.3   4220.3   4220.3   4220.3   4220.3
    13.5       .0   4248.1   4248.1   4248.1   4248.1   4248.1   4248.1   4248.1
    14.0       .0   4276.5   4276.5   4276.5   4276.5   4276.5   4276.5   4276.5
    14.5       .0   4305.3   4305.3   4305.3   4305.3   4305.3   4305.3   4305.3
    15.0       .0   4334.6   4334.6   4334.6   4334.6   4334.6   4334.6   4334.6
    15.5       .0   4364.4   4364.4   4364.4   4364.4   4364.4   4364.4   4364.4
```

This means these contours don't exist on the post storm profile

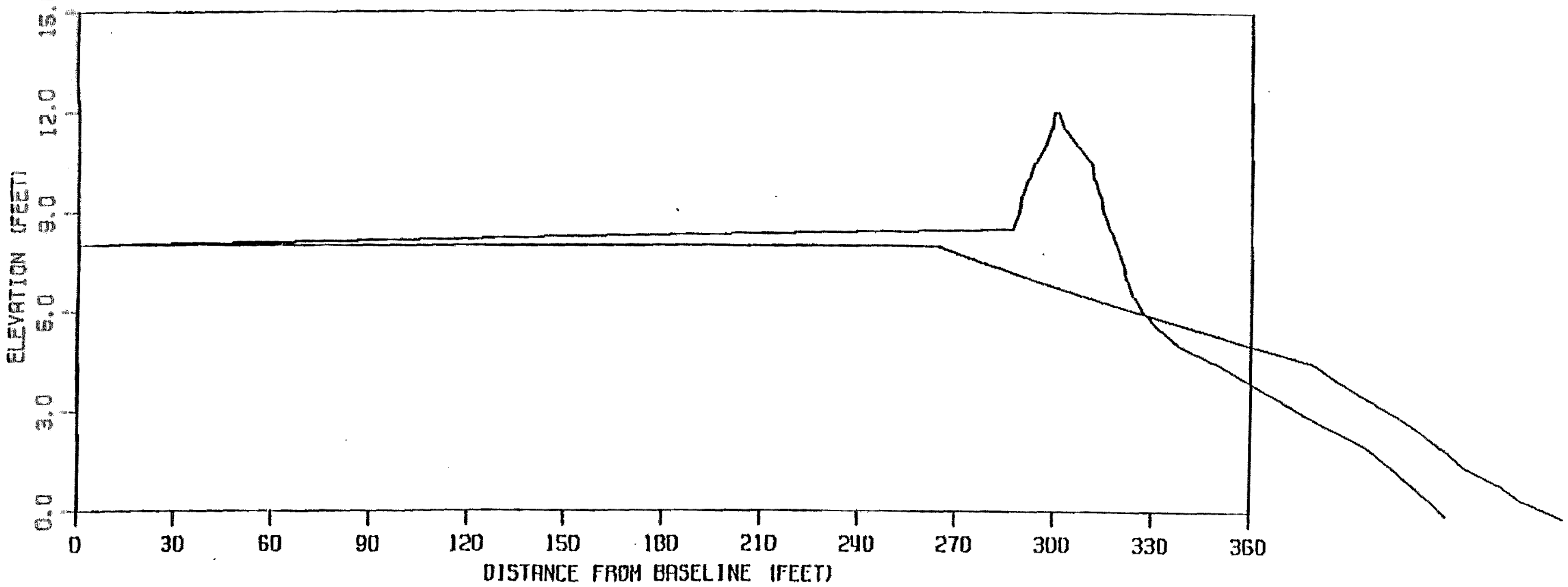
this is the landward-most recession

∴ recession from NGVD contour is 445-264=181 ft.

NGVD ∴ 0.87ft.

13-4

EXISTING CONDITIONS: PL-221 100-YEAR STORM



Sample Calculations theory of Pilarczyk et al (1990)

Wave Runup - Existing conditions (25-year storm) PC-217

$\boxed{R_{2\%} = 0.7\, T_p \sqrt{g H_s}\ \tan\alpha}$

$T_s = 4.6$ ft. assume $T_p = T_s / 0.9$ $T_p = 5.0$

$g = 32.2\ ^{f}/_{s^2}$

$H_s = 5.6$

Slope is iterated with EDUNE. The slope is measured from the beach toe to the runup limit.

The initial slope is 20.9

input into EDUNE ⇒ slope = 25.2 Iteration 2

Iteration 3 ⇒ Slope is 22.7

Iteration 4 ⇒ Slope is 22.8

$\tan\alpha = \frac{1}{22.8}$

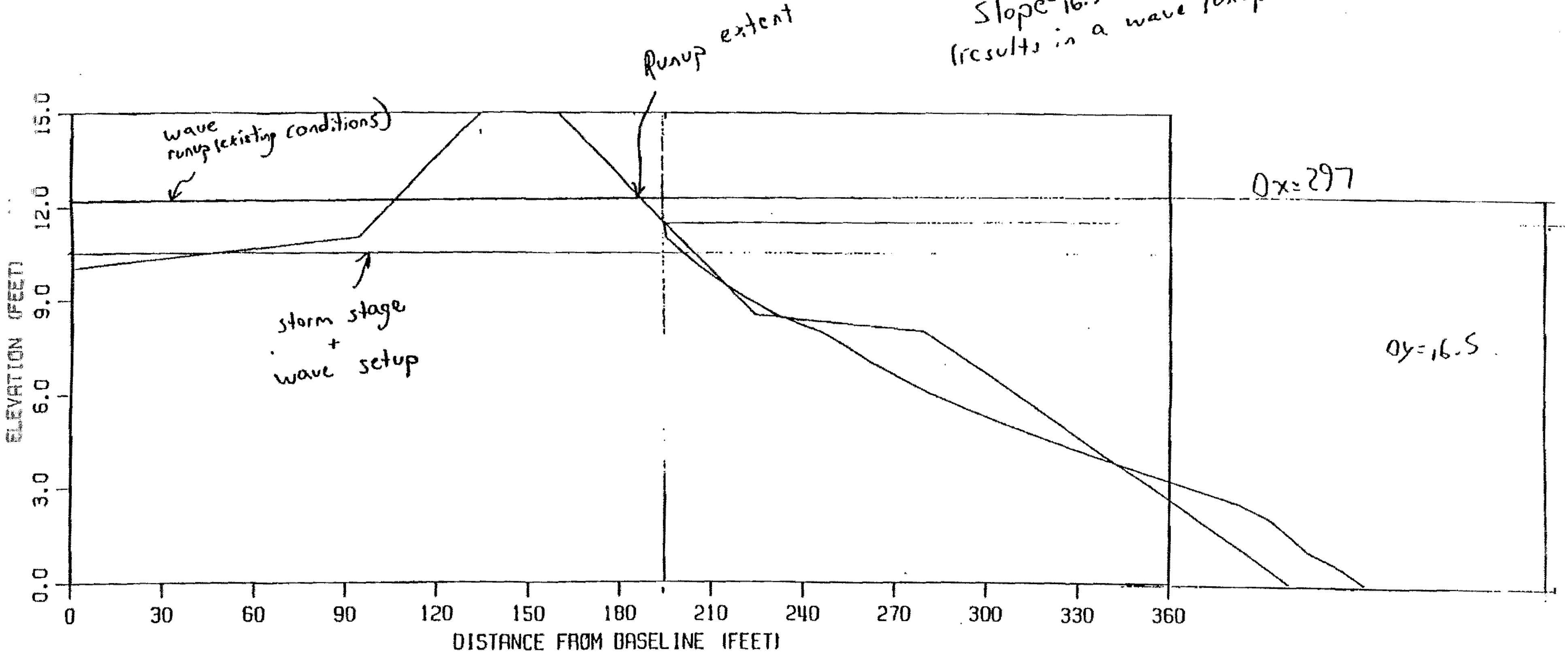
$\therefore R_{2\%} = 0.7(5.0)\sqrt{32.2(5.6)}\ / 22.8$

$R_{2\%} = 2.0$ ft. above SWL

A3-7

Improved Conditions -25 year Storm
Composite slope for Runup Iteration
Iteration 1

Slope = $\frac{297}{16.5}$ = 17.8
(results in a wave runup value of 2.6 ft.)



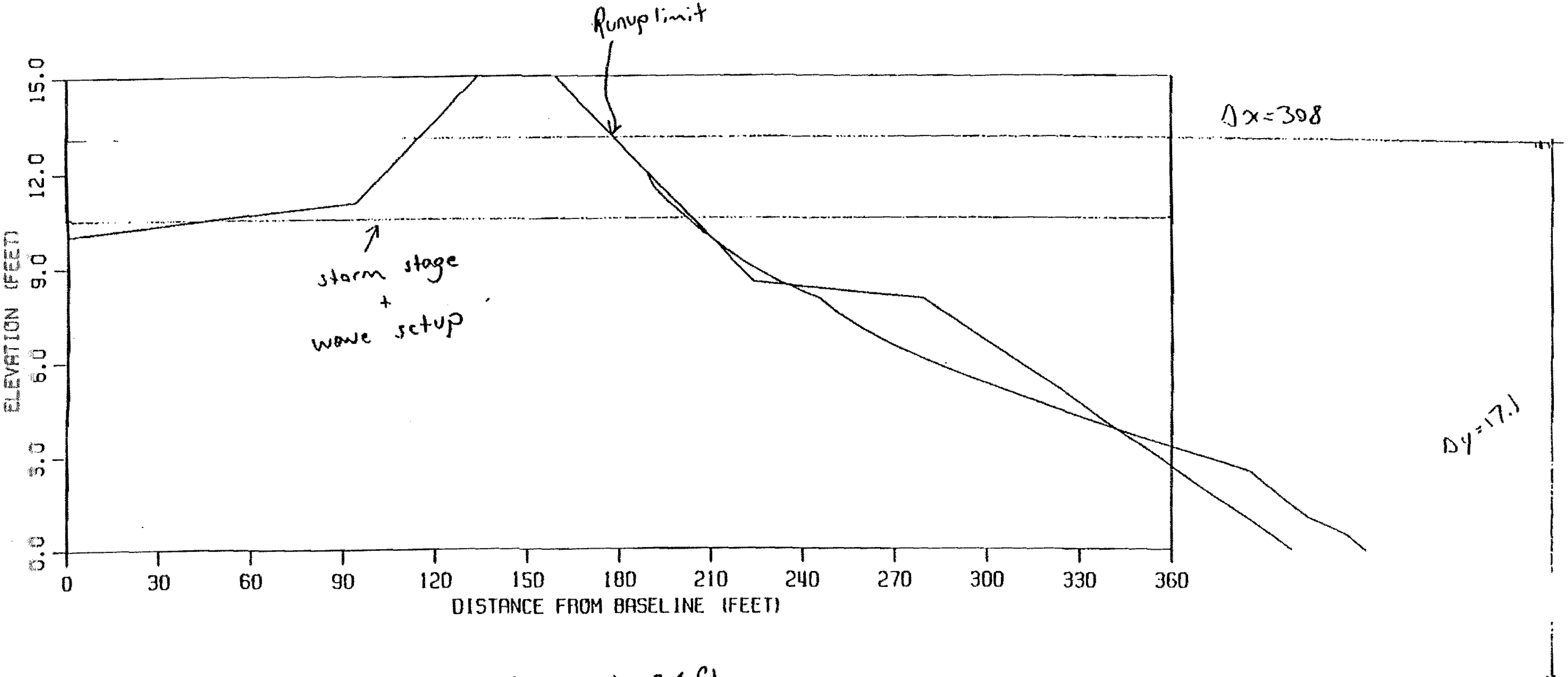
	PL-217	PL-221	Avg
25 year wave Runup	2.1	1.4	1.8

extent of profile
Change (from offshore
post-storm plots)
assumed to be brr

A3-B

Improved Conditions -25 year storm
Composite Slope for Runup Iteration
Iteration 2

slope $= \frac{308}{17.1} = 18.0$
(results in a wave runup value of 2.6 ft.



Wave runup (from iteration #1) = 2.6 ft.

extent of profile adjustment (assumed represent extent [illegible]aking zone)

Sample Calculation Kobayashi et al. Theory page 1 of 2

Dune overtopping: PL-221 (existing conditions) (2-year storm)

---

step 1) See enclosed paper for full detail on theory

dune - breaking distance

$\boxed{B_s = 200}$ (see profiles, dune to -3.3 ft, contour)

Dune height = 12 ft.) = 6.36 ft above surge

$\boxed{H_c = 6.36\text{ft}}$

from fetch-limited wave theory

$\boxed{H_s = 3.52}$ (defined in paper as both a depth and a wave height)

therefore, equivalent uniform slope $\boxed{m_o = \frac{H_c + H_s}{B_s}} = \frac{6.36 + 3.52}{200}$ $\boxed{m_o = 0.04}$

From linear wave theory of above wave, $T_s = 3.7$ sec

assuming $T_{2\%} = \frac{T_s}{0.9}$ $T_{2\%} = 4.0$ sec assuming $T_p = T_{2\%}$ $\boxed{T_p = 4.0\text{ sec}}$

Similarity parameter $\boxed{\xi_o = m_o \left(\frac{2\pi H_s}{g T_p^2} \right)^{-0.5}} = 0.044 \left(\frac{2 \cdot 3.142 \cdot 3.52}{32.2 \cdot (4.0)^2} \right)^{-0.5}$ $\boxed{\xi_o =}$

is $\xi_o < 2$ yes $\boxed{✓}$

---

step 2) water depth d_t: assume $\boxed{d_t = \frac{H_s}{0.78}} = \frac{3.52}{0.78}$ $\boxed{d_t = 4.51}$

$\boxed{\gamma_h = 1 - 0.03 \left(4 - \frac{d_t}{H_s} \right)^2} = 1 - 0.03 \left(4 - \frac{4.51}{3.52} \right)^2 = 0.78$

normalized crest height $\boxed{R_b = \frac{1}{\gamma_h \xi_o} \frac{H_c}{H_s}} = \frac{1}{0.78 \cdot 0.74} \frac{6.36}{3.52}$ $\boxed{R_b = 9.74}$

"acceptable" range $0.3 < R_b < 2$

for 2- and 5- year storms, $R_b > 2$, assumed to indicate negligible overto

for 10 year storm, R_b is acceptable

for 25, 50, 100 and 200 year storms, $R_b < 0.3$ - theory fails when complete overtopping occurs.

JOB NO.	SUBJECT:	DATE:	SHEET NO.
	A3-9	MADE BY:	
		CHECKED BY:	

page 2 of

Sample Calculation Kobayashi et al. Theory

Dune Overtopping: PL-221 (existing conditions) (2-year storm)

Step 3) $\boxed{Q_b = 0.06\, e^{(-5.2 R_b)}} = -0.06\, e^{(-5.2(9.74))}$ $\boxed{Q_b = -6.05 \times 10^{-24}}$

$$\boxed{Q_1 = \frac{Q_b\, \xi_0 \sqrt{g H_s^3}}{\sqrt{m_0}}} = \frac{(-6.05 \times 10^{-24})(0.24)\sqrt{32.2(3.52^3)}}{\sqrt{0.049}}$$

$$\boxed{Q_1 = -7.78 \times 10^{-22} \frac{ft^3}{ft \cdot shoreline}}$$

effective length = 3,000 ft.

$Q_f = Q_1 \cdot 3{,}000$ $\boxed{Q_f = -2.33 \times 10^{-18} ft^3}$

which is added to overtopping of dune represented by PL-217 ~~which also has an effective~~ length of 3,000 ft.

the values become more significant with larger storms

WAVE REFLECTION AND OVERWASH OF DUNES (KOBAYASHI, ET AL., 1996)
RED: KOBAYASHI THEORY VARIABLES
STEP 1) IS SIMILARITY PARAMETER EPSILONo IN RANGE (<2)? YES
STEP 2) IS NORMALIZED CREST HEIGHT Rb IN RANGE (0.3<Rb<2)? NO
STEP 3) IS CALCULATED OVERTOPPING CONSISTENT WITH HISTORICAL OBSERVATIONS? NO
NOTE: THE PILARCZYK AND ACES THEORY VALUES DEPEND ON THESE VALUES!!!

	DUNE ELEVATIONS (MTL-BEFORE STORM)		DUNE TO BREAKING POINT DISTANCE	
	PL217	PL221	Bs PL217	Bs PL221
	18.43	11.83	170	200
PERCENT DUNE REDUCTION	100.00%	100.00%		

STORM RETURN INTERVAL	PL217	PL221	PL217	PL221	SURGE (MTL)	Hc PL217	Hc PL221	WAVE HEIGHT Hs	mo PL217	mo PL221
	DUNE ELEVATIONS (MTL-AFTER STORM)		DUNE ELEVATIONS (MTL-DURING STORM)			DUNE HEIGHT ABOVE SURGE			EQUIVALENT UNIFORM SLOPE	
2	18.5	12	18.5	12	5.6	12.86	6.36	3.52	0.096	0.049
5	17	8.5	17	8.5	6.8	10.25	1.75	4.47	0.087	0.031
10	16.5	8	16.5	8	7.6	8.95	0.45	5.02	0.082	0.027
25	15	8	15	8	9.6	5.4	-1.6	5.62	0.065	0.020
50	14	8	14	8	9.6	4.36	-1.64	6.02	0.061	0.022
100	13.5	8	13.5	8	11.3	2.2	-3.3	6.39	0.051	0.015
200	13.5	8	13.5	8	13.8	-0.31	-5.81	6.73	0.038	0.005

Step 1

assume $H_s/d_t=0.78$

Step 2

STORM RETURN INTERVAL	PEAK PERIOD Tp	EPSILONo PL217	EPSILONo PL221	WATER DEPTH dt	GAMMAh PL217	GAMMAh PL221	Rb PL217	Rb PL221
		SIMILARITY PARAMETER			UNNAMED PARAMETER		NORMALIZED CREST HEIGHT	
2	4.00	0.47	0.24	4.51	0.78	0.78	10.09	9.74
5	5.00	0.46	0.17	5.73	0.78	0.78	6.35	3.02
10	5.00	0.42	0.14	6.44	0.78	0.78	5.52	0.83
25	5.00	0.31	0.10	7.21	0.78	0.78	3.99	-3.81
50	5.00	0.28	0.10	7.72	0.78	0.78	3.30	-3.46
100	6.00	0.27	0.08	8.19	0.78	0.78	1.63	-7.99
200	6.00	0.20	0.02	8.63	0.78	0.78	-0.30	-46.03

EFFECTIVE SHORELINE LENGTH (FT.)

ERR	ERR
3000	3000

step3

STORM RETURN INTERVAL	Qb PL217	Qb PL221	Q PL217	Q PL221	ERR ERR	ERR ERR
	NORMALIZED OVERTOPPING RATE		OVERTOPPING PER FOOT (CF/S)		TOTAL OVERTOPPING (CF/S)	
2	0.00	0.00	0	0	0	0
5	0.00	0.00	0	0	0	0
10	0.00	0.00	0	0	0	125
25	0.00	24263796.48	0	########	0	########
50	0.00	3994035.96	0	228,542,384	0	########
100	0.00	########	0	########	4	########
200	0.28	########	29	########	86,022	########

Verification Study of a Dune Erosion Model

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INTRODUCTION

ACCURATE ESTIMATES of beach and dune erosion on the open coast are required for a variety of regulatory and design purposes. For example, the 1979 Beach and Shore Protection Act in the State of Florida requires that coastal construction control lines be established "so as to define that portion of the beach-dune system which is subject to severe fluctuations based on a 100-year storm surge."[1] At present, however, any method of predicting dune erosion during a severe storm must be considered a "best guess" since extensive verification has not been possible due to the lack of accurate field data. Likewise, all methods are, by necessity, highly simplified and attempt to represent only the most significant features of one of nature's most violent events.

In this paper, a numerical model,[2,3] developed to estimate the rate and extent of dune erosion, is reviewed with emphasis on recent model calibration and verification to determine the accuracy of erosion estimates.[4] The paper also illustrates the need for additional field measurements of pre- and post-storm beach profiles to provide the basis for more complete model verification.

DESCRIPTION OF DUNE EROSION MODEL

The numerical model is based on a finite-difference solution of a simplified set of governing equations, using the time-histories of the storm surge and breaking wave heights as forcing and off shore boundary conditions, respectively. The model assumes that for a given water level and breaking wave height, the beach profile will evolve toward its most stable or equilibrium form; however, this evolution, may require a long time relative to the length of time over which the forcing conditions persist. The ultimate profile form, representing a dynamic equilibrium with no additional net onshore or offshore sediment transport, has been found to be well approximated by a monotonic curve of the form:

$$h = Ax^{2/3} \qquad (1)$$

where h is the depth at some distance x seaward of the shoreline. The parameter A governs the steepness of the profile and has been related to a unique value of the wave energy dissipation per unit volume, D_*, which exists everywhere in the surf zone when the profile is in equilibrium.[5] A least-squares fit of Equation 1 to over 700 beach profiles has resulted in empirical relationships between A (or D_*) and sediment size and fall velocity.[5,6,7]

During a severe storm, the elevated water levels permit waves to break closer to shore, reducing the volume of water over which a given incident wave energy is dissipated as depicted in Figure 1. This causes the actual energy dissipation per unit volume to be greater than the equilibrium value, D_*, at all points across the surf zone. The profile will respond to the increased energy dissipation through a redistribution of sand. Over time, the profile evolves back toward equilibrium through erosion of the beach face and deposition offshore near the breakpoint. Equilibri rarely attained, however, since the forcing cond are usually maintained for a relatively short time.

Based on these concepts, and using shallow wa linear wave theory, the offshore sediment transport rate, Q, may be approximated according to the excess energy dissipation per unit volume at each depth contour across the surf zone as:

$$Q = K(D - D_*) \qquad (2)$$

In Equation 2, the coefficient, K, is the only unknown and must be determined empirically based on a comparison of numerical results to observed profile response. Time-dependent profile response is then determined by an implicit finite-difference solution of the equation for continuity of sand in the onshore-offshore direction:

$$\frac{\partial x}{\partial t} = -\frac{\partial Q}{\partial h} \qquad (3)$$

As a review of the solution method, the entire profile is represented by a series of elevation contours wit uniform spacing, Δh, and with the distance x definec from a baseline to the center of each contour. Since I and Q vary with water depth and local bottom slope the rate of change of each discrete contour positior differs from adjacent contours. The solution proceeds by establishing the water level and wave height on th given profile form at the beginning of each tim The energy dissipation per unit water volume the sediment transport rate, Q, are then calcu the active part of the surf zone between the offsho limit at the breakpoint and the onshore limit. T numerical solution is in the form of a tridiagonal matrix that relates the contour change of three adjacent contours to the local forcing conditions. The system is then solved by the so-called double sweep procedure to determine the change in contour position. Δx, over the time step.

A preliminary verification of the model was carried out in a simulation of the time-dependent dune erosion associated with Hurricane Eloise in Bay and Walton Counties in Florida.[2-4] It was found that the model reasonably predicted the magnitude of average storm-induced erosion as predicted eroded volumes compared favorably with observed county-wide average eroded volumes.[3] Dune steepening during erosion was not realistically simulated. however. so predicted recession of individual elevation contours did not agree as closely with observed values.

While results of the initial model application were encouraging and showed the utility of the method, this verification was considered preliminary for several reasons. First, the model was not uniquely calibrated and instead an erosion rate constant K determined by Moore[7] based on a different numerical scheme was used. Second, the model assumed that the onshore limit of transport was always at either the berm or dune crest rather than at a realistic wave runup limi In addition. the beach and dune slopes were maintained at their initial values during erosion. Thes restrictions prevented formation of an erosion scar above the wave runup limit as is clearly evident i nature. Third, the input storm surge for Hurrican Eloise. estimated using a one-dimensional Bathystrophic storm surge model. was somewhat too larg and had a longer duration than more recent estimate using a full two-dimensional storm surge model indicate.[11] Finally, the preliminary verification was pe fomed on a single average pre-storm profile from th two-county area and results were compared to the ave age erosion characteristics of the two-county area.[3] N attempt was made to simulate the response of ind vidual profiles to consider local effects of pre-stor profile forms or variation of the storm surge over th two-county area.

After reviewing 110 measured pre- and post-stor profiles for Walton County, it is evident that there great varibility in pre-storm profile forms and in stor erosion response of individual profiles. In fact, over 2 profiles show a net gain of sand between the two ava able surveys, from October 1973 to October 1975 (th 1975 post-storm surveys were made 2-4 weeks afte Hurricane Eloise). The inclusion of these profiles, an others which showed little erosion. skews average er sion statistics such that published county-wide avera erosion values are lower than the actual erosion expe enced by most of the profiles. in some cases by mo than a factor of two. Variability in profile response ca be attributed in part to three sources of uncertain 1) pre-storm profile modification between Octobe 1973 and Hurricane Eloise, 2) localized longshor transport effects during the storm, causing areas local accretion and accentuated erosion, and 3) no uniform beach recovery between the storm and th post-storm surveys.

In order to obtain more realistic estimates of actu storm-induced erosion, the original erosion mode has been modified to include many effects not pre ously represented.[4] Specifically, provisions are made the updated model to include a realistic wave run limit with formation of an erosion scarp and du steepening to a near vertical slope. These changes pe mit a more realistic post-storm profile form such th horizontal recession of individual elevation contou agrees more closely with nature. Along with the changes, a more complete verification of the mod is made using actual pre- and post-storm beach profil from the Hurricane Eloise data set. The revised mod is first calibrated based on a large-scale laborato experiment of Saville[10] and is then recalibrated usi a reference profile, line R-41, from the Walton Cou data set. Profile R-41 is used as a calibration standa since it was also used in two other dune erosi verification studies.[11,12] Finally, the calibrated mo

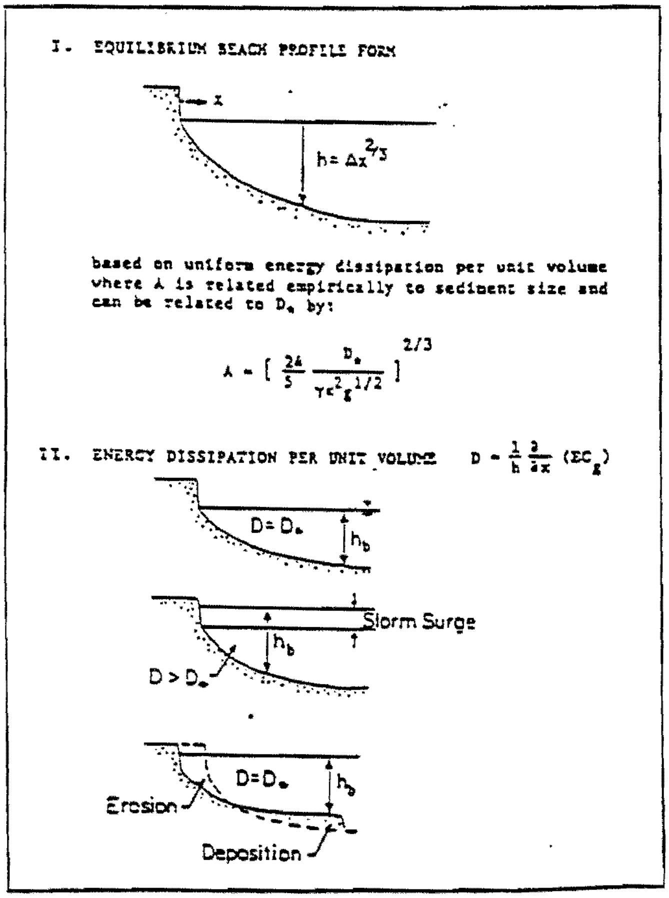

Fig. 1 Basic Concepts for Erosion Model

A3-13

is used to hindcast erosion for an additional 20 profiles from the Walton County data set in an effort to test model sensitivity and bias introduced in the calibration process.

CALIBRATION – SAVILLE'S LABORATORY EXPERIMENT

The modified erosion model is tested and calibrated based on numerical simulations of one of Saville's[10] large-scale laboratory experiments. Saville's experiments provide a useful data set for testing the numerical erosion model because they were conducted at approximately full-scale and because detailed measurements were made throughout the profile development from which time-dependent erosion characteristics, including beach slope steepening, can be tested. Initial conditions for Saville's test at prototype scale are:

Beach Slope	1:15
Water Depth at toe of slope	14 feet
Berm Crest above still water level	6 feet
Median sand diameter	0.22 mm
Wave height	5.5 feet
Wave period	11.33 sec.
Breaking depth	6 feet

For calibration of the numerical erosion model, all physical parameters are specified so that the transport coefficient, K, remains as the only free parameter to be determined. Required physical parameters include the initial profile form (1:15 slope), the characteristic A parameter, the equilibrium beach slopes, runup distance, and breaking depth. Based on Saville's final equilibrium profile, the equilibrium beach face slope is approximately 1:5 and the vertical runup distance above the still water level is 4 feet. Seaward of the breakpoint, the offshore slope is approximated by a uniform slope of 1:15. The scaling parameter, A, is then determined by a least-squares fit of the $Ax^{2/3}$ profile form to the observed profile after 40 hours and the best-fit A value is found to be 0.160 $ft.^{1/3}$

Calibration of the erosion model is accomplished by a series of simulations in which separate values of K are used to simulate Saville's profile development while all other parameters are held constant. The eroded volume at any time is determined as the cumulative volume of material displaced between the initial profile and the profile at the current time. The mean squared error between the predicted and observed eroded volumes is then obtained at various times.

Because all other parameters are held constant and are best-fit values obtained from Saville's final profile, it is expected that a distinct best-fit K may be obtained. In Figure 2, results from the calibration test series are summarized with the mean squared error plotted for eight K values tested. The five curves shown correspond to the error curves obtained after 10, 20, 30 40, and 50 hours of simulation since, it is desired to mine an overall K which provides best agreement the duration of the experiment not just after eq rium is attained at 40 to 50 hours. After 10 hours, the best-fit K is 0.004 to 0.0045 ft^4/lb while from 20 to 50 hours, minima of the error curves occur between 0.0045 and 0.005 ft^4/lb. In general, the broad troughs of the mean squared error curves indicate that varying K by plus or minus 10 percent is not critical and will give similar results for erosion estimates. Based on these results, an overall value of K = 0.0045 ft^4/lb is adopted which seems to give near minimum error over all time scales of interest. The observed and predicted cumulative erosion curves are shown in Figure 3 based on the selected best-fit value of K. Figure 4 shows a comparison of the predicted and measured profiles for the calibration run; note that the numerical model predicts a smooth monontonic profile form out to the breaking depth and does not attempt to predict bar formation.

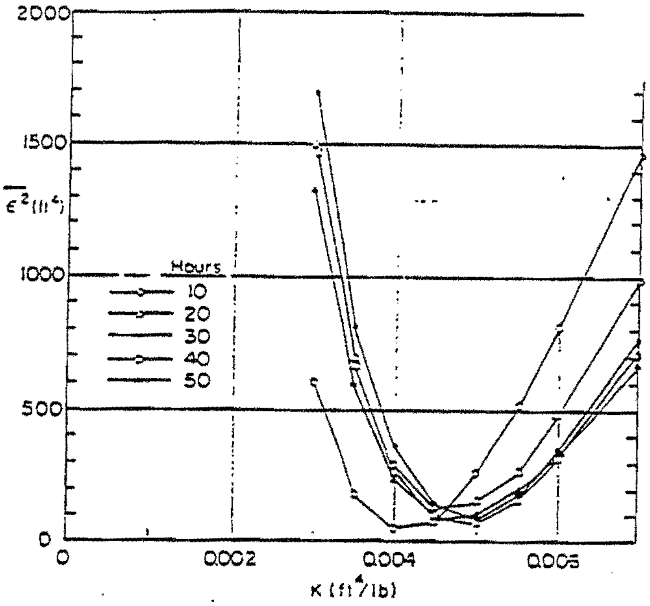

Fig. 2 Mean Square Error of Volume Eroded as a Function of Sediment Transport Coefficient K for Model Simulation of Saville's experiment.

CALIBRATION – HURRICANE ELOISE FIELD DATA

Hurricane Eloise made landfall just east of Walton County, Florida, on September 23, 2975 and was a rapidly moving storm which, while lasting less than 20 hours, produced estimated peak water lev between 8 and 10 feet. Although no open coast surge measurements are available, numerical esti s by Dean and Chiu[8] range from 8.4 feet at the western end of Walton County to 9.6 feet at the eastern end of

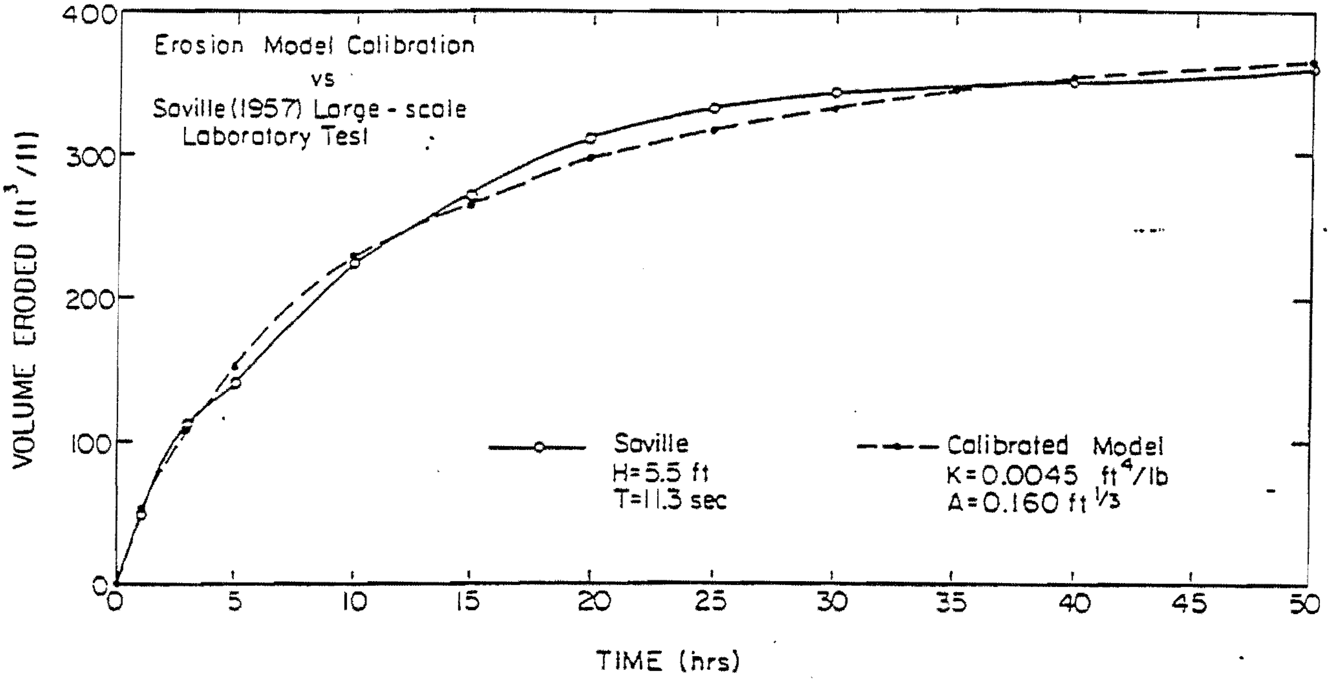

Fig. 3 Comparison of Cumulative Erosion for Calibrated Model and Saville's Laboratory Experiment.

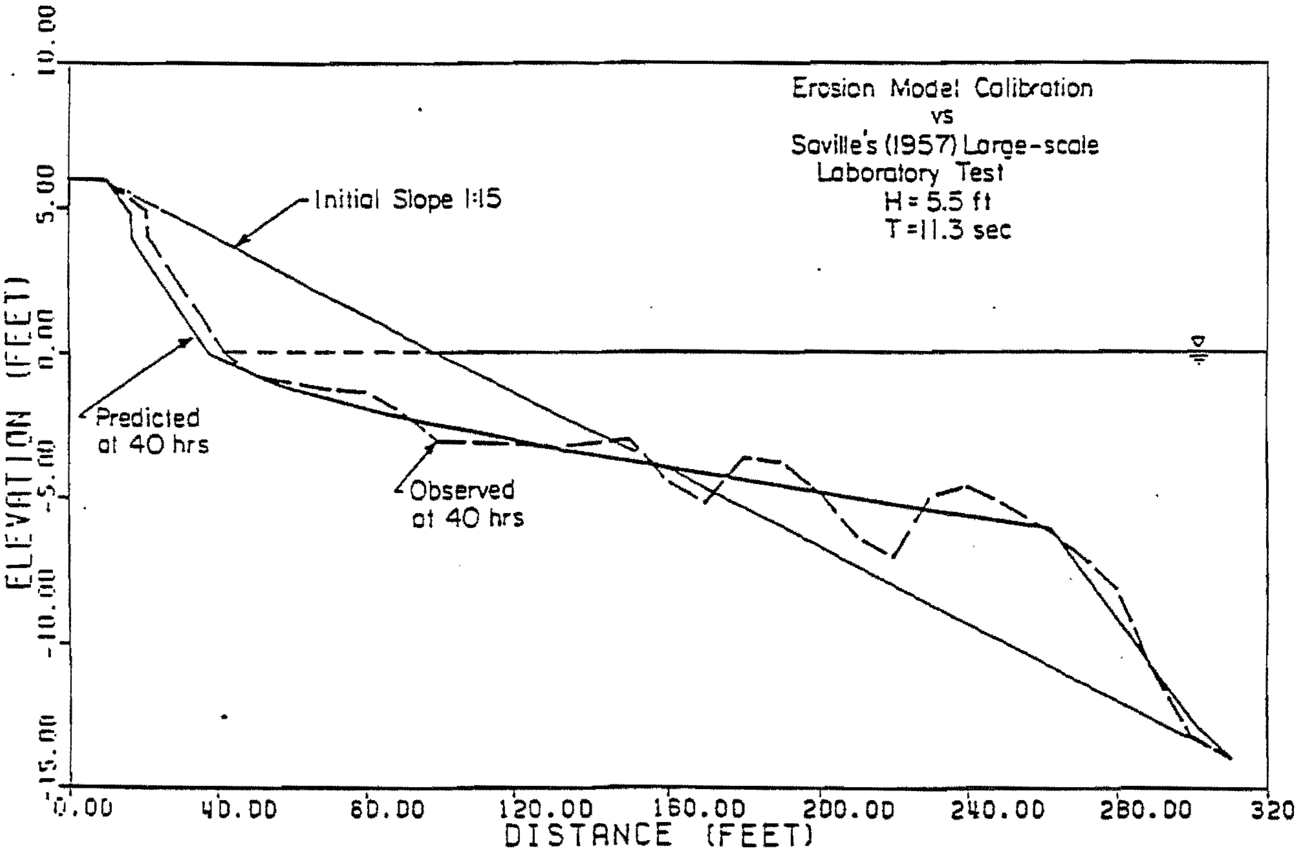

Fig. 4 Comparison of Profile Forms for Calibrated Model and Saville's Laboratory Experiment.

A3-15

the county. The predicted storm surge hydrograph near the western end of Walton County is shown in Figure 5. The storm surge elevations have been interpolated by Dean and Chiu for all 110 profiles in Walton County and the surge hydrograph in Figure 5 is multiplied by a constant to give the hydrograph for each site. For profile R-41, the surge hydrograph is multiplied by 1.083 at all times. Significant wave heights recorded during the peak of the storm are 10 to 14 feet with a dominant period of 11 seconds. For this study, a breaking wave height of 12 feet is used to obtain an estimate of the offshore limit of sediment deposition. Offshore profiles forms for pre-storm conditions are assumed to be characterized by an $Ax^{2/3}$ profile. The scaling parameter A, based on the effective grain size of 0.262 mm. is 0.184 ft.$^{1/3}$

Pre-storm profiles for the Walton County area are taken from the October 1973 survey of the area by the Florida Department of Natural Resources. Post-storm surveys were conducted within 2-4 weeks after the storm in October 1975. Due to the timing of these surveys, the "observed" erosion associated with Hurricane Eloise may be contaminated by two effects. First, although Walton County is a low wave energy coastline, it is probable that some modification of the pre-storm profile occurred in the two years between October 1973 and September 1975. These effects may include natural erosion or accretion of the shoreface, possible modification of the dunes by wind-blown sand, or construction activities. Second, in the 2-4 weeks after the storm, some recovery of the shoreface certainly occurred. It has been suggested that on average 50 ft^3/ft had returned to the beach face above mean sea level prior to the post-storm survey.[5]

Two previous studies of beach/dune erosion have used profile R-41 from Walton County, for model verification. Hughes and Chiu[11] selected R-41 as a representative profile to be used in verification of small-scale laboratory simulations of dune erosion. Vellinga[15] also used R-41 as part of continuing verification Dutch computational method for predicted dun. sion due to severe storms. Since profile R-41 has become, in some respects, the standard reference file from the Hurricane Eloise data set, it seems reasonable to calibrate the numerical erosion model based on this profile. In this study however, the erosion model is also tested against 20 additional profiles from the Walton County data set to determine possible bias associated with using R-41 as a benchmark for calibration/verification.

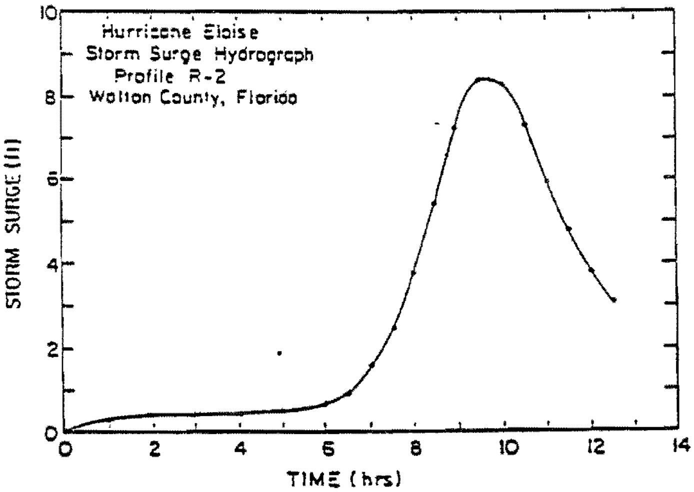

Fig. 5 Hurricane Eloise Storm Surge Hydrograph Estimated by Dean and Chiu (1984).

The pre- and post-storm profiles for range R-41 are shown in Figure 6. This profile is fairly representative of the Walton County area in that it has well-developed dunes, and a narrow berm with a berm crest elevation of 5 to 6 feet. The post-storm profile is typical in that a distinct break-in-slope occurs between the dune scarp and the flattened beach at about 10 feet. The rebuilt post-storm berm is also clearly evident. The computed eroded volume of about 400 ft^3/ft above NGVD is slightly above average for the entire Walton County data set. Accounting for the post-storm berm, actual eroded volumes may be closer to 425 ft^3/ft.

Input to the numerical model consists of the actual pre-storm profile for R-41 between the dune crest at 26 feet and mean sea level, taken to be at 0 feet NGVD. The offshore profile is established according to $0.184\ x^{2/3}$. Additional required input consists of th storm slopes of the dune scarp as well as an eff runup height.

Due to the uncertainties involved in the post-storm eroded volumes, calibration of the erosion model using Profile R-41 is obtained in a more subjective manner than in the previous calibration against Saville's data. In this case, since the estimate of total eroded volume (400 ft^3/ft) may not exactly represent the actual eroded volume, and since the time-history of erosion is not available, the mean squared error curves as a function of K cannot be developed. Instead, in Figure 7 predicted maximum volumetric erosion is plotted against the different values of K used.

Based on the 400 ft^3/ft estimate of total observed eroded volume, the best prediction is obtained using a value of K = 0.0044 ft^4/lb. Within 10 percent error bands, the best K values range between 0.0038 and 0.0052 ft^4/lb. This range is similar to that suggested by the calibration based on Saville's profile and the previous best estimate of K = 0.0045 ft^4/lb seems equally appropriate for profile R-41. This surprising agreement between the two calibration runs is fortuitous and should not be taken as an absolute indication that K = 0.0045 ft^4/lb is the "correct" or universally valid constant for the proposed erosion model. Most like agreement is the result of assumptions made simulation concerning input parameters. For ex based on the estimated eroded volume of 425 ft^3/f

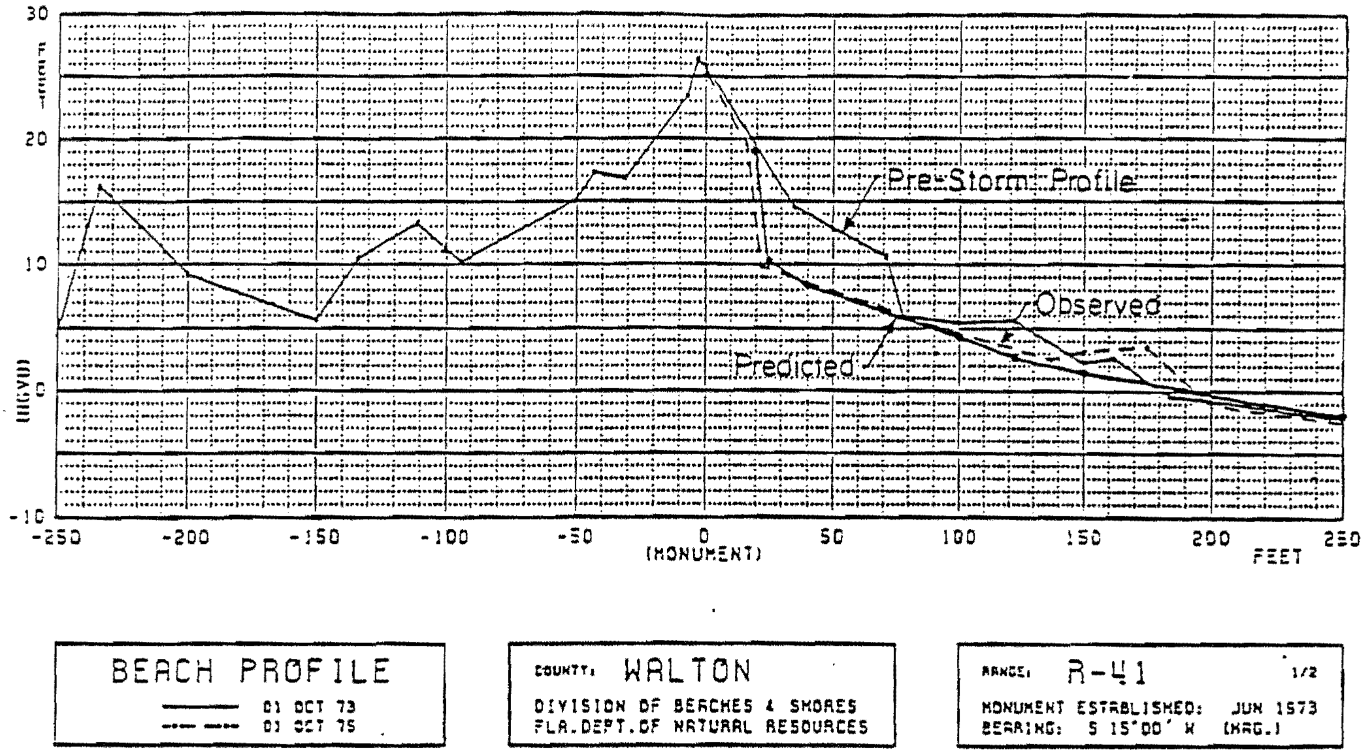

Fig. 6 Comparison of Pre- and Post- Storm Beach Profiles, R-41 Walton County, Florida as Measured by Florida Department of Natural Resources; also, Predicted Post-Storm Profile from Model Calibration.

which accounts for the effect of post-storm recovery, the best value of K is 0.0048 ft^4/lb.

As a sensitivity test, erosion estimates for R-41 are obtained using various wave height scenarios. In Figure 8, the eroded volumes obtained by using a wave height that is constant over the duration of the storm surge are compared to estimates obtained by applying a variable wave height. Variable wave heights are scaled from 3 feet to the maximum height shown according to the ratio of the storm surge level at each time step to the peak surge level. Results of this test indicate that a variation in the constant wave height of ±20 percent from the 12 foot height used in calibration produces less than a 5 percent change in eroded volume. Likewise, use of a variable wave height tends to decrease the erosion estimate by less than 10 percent for the range of wave heights of interest. If calibration had been carried out using a variable wave height, a slightly larger value of K would have been required. Due to the small differences between predictions, all model calibration, verification, and application is performed with a constant wave height.

In Figure 6, the predicted post-storm profile is compared to the observed post-storm profile form for $K = 0.0045$ ft^4/lb. There is good agreement between predicted and observed profiles from the base of the dune scarp across the shoreface to the post-storm berms. The major differences between the predicte and observed profiles are: 1) at the post-storm berr which cannot be simulated by the model and 2) in th predicted position of the dune scarp, which is about to 4 feet seaward of its actual positon. In this case, slightly larger value of K would also provide the bes agreement with the observed dune scarp location.

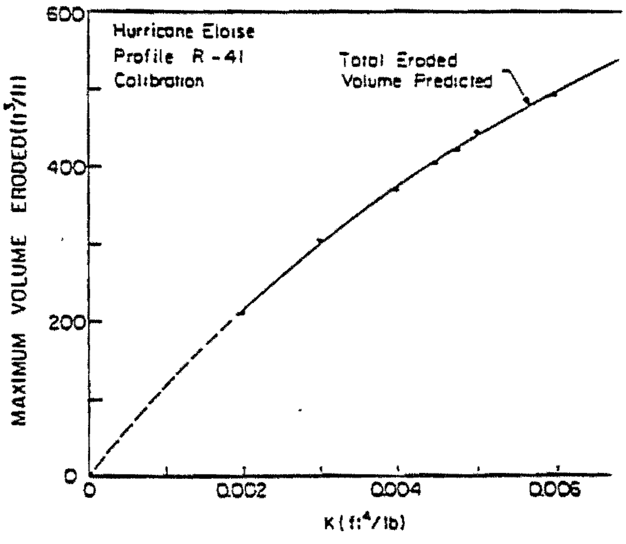

Fig. 7 Predicted Volume Eroded as a Function of Sedim Transport Coefficient K for Profile R-41.

VERIFICATION USING ADDITIONAL HURRICANE ELOISE FIELD PROFILES

Numerical simulations are also carried out on an additonal 20 profiles from the Hurricane Eloise data set. These profiles are selected to be representative of the 80 to 90 eroded profiles in the data set which exhibit erosion, and include cases with both large eroded volumes or in the case of low dunes, of large horizontal contour recession. The profiles are not the most severely eroded in the data set. The selected profiles cover the entire length of the Walton County shoreline and several groups of closely spaced profiles are chosen to indicate the natural variability that may exist between adjacent profiles. All tests are made with $K = 0.0045$ ft^4/lb as established by calibration tests. Input in each case consists of idealized pre-storm profiles, i.e. described by a dune crest position, dune face slope, berm crest position, and beach face slope. Offshore profiles are all simulated initially by $A = 0.184$ $ft.^{1/3}$ Storm surge hydrographs are identical in form to Figure 5 but modified by a multiplicative constant for each profile.[8] Other input variables include the observed post-storm dune slope and runup distance as determined for each profile. For this verification study, these values are obtained from the post-storm profiles; however, in application of the model as a predictive tool, these values would be selected based on previously observed dune erosion conditions or other available empirical data.

In Figure 9, predicted eroded volumes are compared to observed values for the 20 profiles and for the calibration profile R-41. The diagonal line representing perfect agreement between predicted and observed values, falls only through profile R-41. There is considerable scatter in the data points about the line of complete agreement; however, there seems to be little bias as the numerical model overpredicts erosion in 11 cases and underpredicts in 9 cases. In 5 cases, predictions are within 10 percent of the observed values; in 17 cases predictions are within 25 percent of observed values; and all 20 cases are within a 40 percent margin of error.

Errors outside the 25 percent range may be attributed to several factors. For profile R-8, the model underpredicts erosion substantially. However, the large eroded volume at R-8 appears to be a local anomaly; adjacent profile R-9, located 1,000 feet east, shows about 300 ft^3/ft of erosion while adjacent profile R-7 located 1,000 feet west shows net accretion. For profiles R-15, and R-123 the model overpredicts erosion by 25 to 30 percent. The explanation for this seems to be that these are among the steepest profiles simulated and the numerical scheme does tend to predict greater erosion in areas of steep berm or dune slopes.[2] Again, however, local longshore effects may play a considerable role; profile R-122 just 1,000 feet west of R-123,

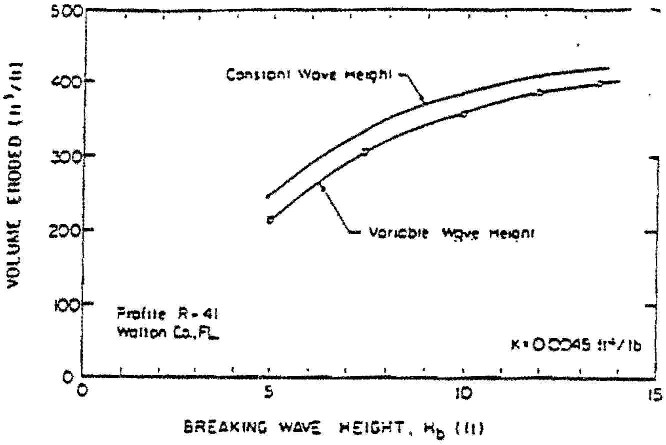

Fig. 8 Sensitivity of Predicted Volume Eroded to Wave Height Description.

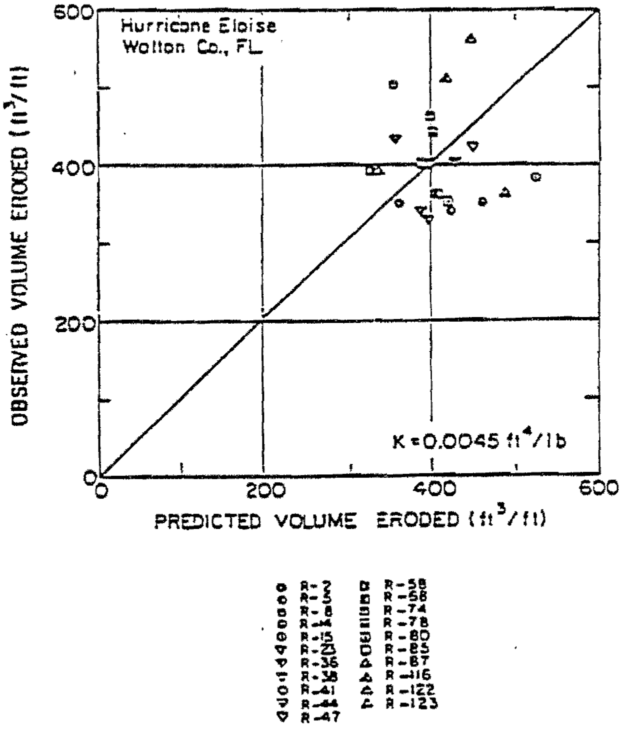

Fig. 9 Comparison of Predicted to Observed Erosion for 20 Beach Profiles from Walton County, Florida.

shows the greatest observed erosion of the 21 profiles and is underpredicted by the numerical model by about 25 percent.

Results are somewhat biased in the prediction of dune recession. Of the 20 tests, the location of the dune scarp is correctly predicted on 4 profiles. The predicted dune scarp is too far landward on 5 profiles and is underpredicted, or too far seaward, on 11 profiles. On average, the position of the dune scarp was underpredicted by 5.4 feet. Extreme estimates range from an underprediction of 18 feet (out of a total observed recession of about 55 feet for a 33 percent error) to an overestimate of 5 feet (out of a total observed recession of 36 feet for a 14 percent error). Based on these results, dune scarp location might be better predicted with a slightly larger K value. Examples of numerical results, selected to show examples of accurate numerical prediction as well as over-and under-prediction, are presented in Figure 10.

SUMMARY AND CONCLUSIONS

The modified numerical erosion model seems to be well-calibrated and verified for application to other areas. The first calibration, against Saville's large-scale laboratory profile, provides some indication of the validity of the model for predicting time-dependent profile development under controlled conditions. The second calibration, against profile R-41 from the Hurricane Eloise data set, is less conclusive in terms of precise numerical calibration. However, both calibrations indicate a best-fit K value of approximately 0.0045 ft^4/lb for prototype scale dune erosion. Certainly, the agreement between the calibration test of R-41 and the Saville calibration should be viewed, in part, as a fortuitous correlation. This conclusion is supported by the comparison of numerical predictions to observed erosion for an additional 20 profiles from the Hurricane Eloise data set where the best-fit K = 0.0045 ft^4/lb gives agree-

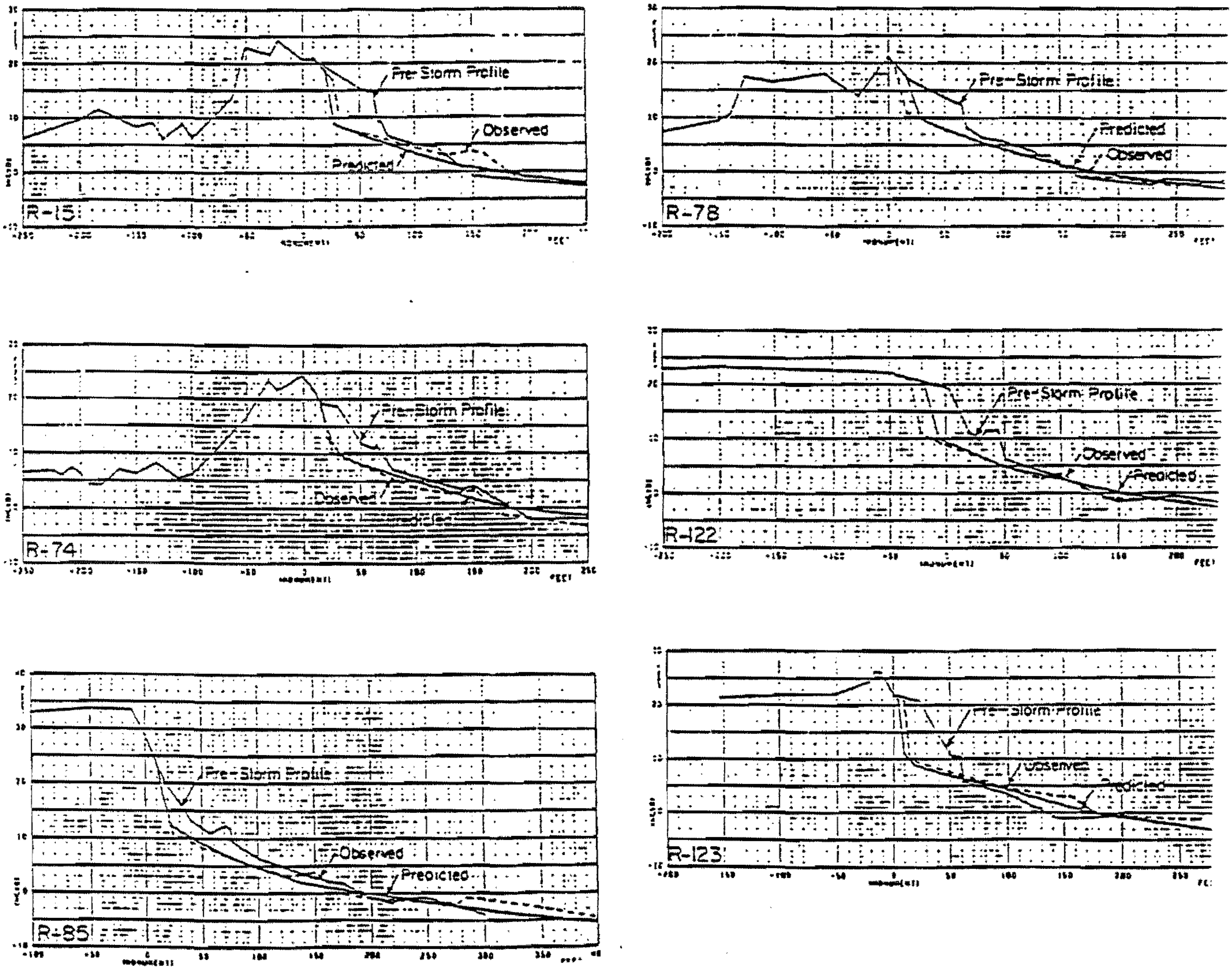

Fig. 10 Examples of Predicted and Measured Beach Profiles Walton County, Florida.

ment of eroded volumes to within a 25 percent error on 17 profiles. Larger errors on the other 3 profiles are mainly attributed to localized longshore or steepness effects. A further conclusion reached is that beach slope changes and dune scarps are reasonably simulated by the model. However a slightly larger value of K would yield a better estimate of the dune scarp location.

For application to other dune erosion predictions, the following guidelines are recommended:

(1) For the modified erosion model, K = 0.0045 ft^4/lb should be used to obtain average dune erosion characteristics.

2) Actual dune erosion is highly variable due to a number of natural factors. Numerical predictions are also sensitive to some parameters such as very steep slope. Therefore, all erosion volume estimates should be considered average estimates with probable errors of around 25 percent and possibly larger.

3) While individual contour recession predictions are much better than those previously obtained, estimates of dune recession should also be considered to have probable error limits of 5 to 20 feet, or 25 percent, and perhaps more. The model also is somewhat biased, however, and tends to underpredict dune recession based on the profiles tested.

Continued verification studies are required to provide further confidence in model accuracy. Since Hurricane Eloise was a short-duration storm, verification for slow-moving hurricanes or other storm systems would be helpful. As indicated by the Hurricane Eloise data set, accurate and representative field measurements are difficult to obtain. Available pre-storm profiles typically do not represent actual beach conditions immediately preceding the storm. Likewise, post-storm measurements typically are contaminated by post-storm beach recovery. Furthermore, individual beach profiles, even those located a few hundred feet apart, may show great variability in erosion characteristics.

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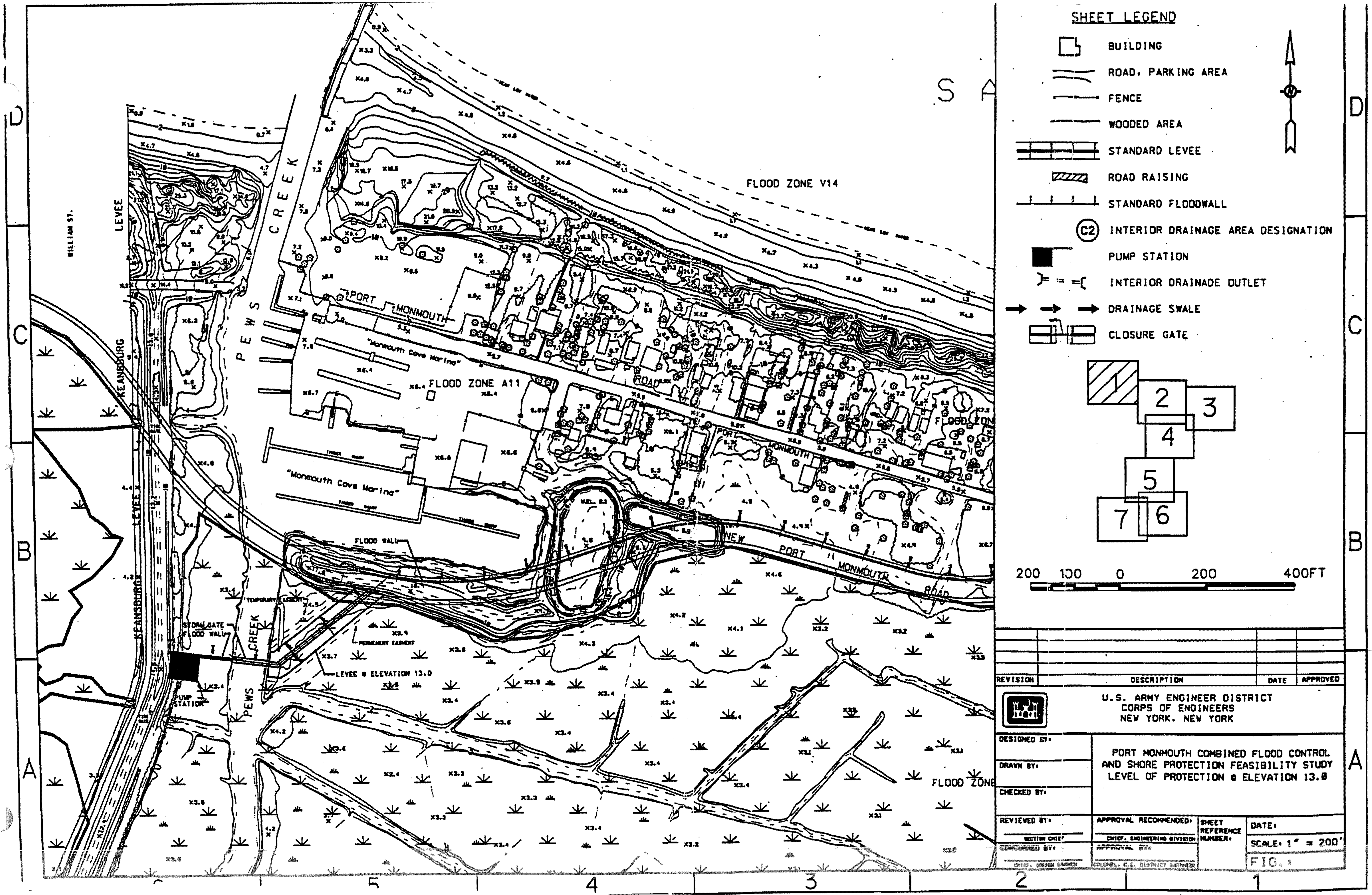
Sub-Appendix A4

13 Ft. Elevation Levee Alignment Plan View
15.2 Ft. Elevation Levee Alignment Plan View

A4



REVISION	DESCRIPTION	DATE	APPROVED

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
NEW YORK, NEW YORK

DESIGNED BY:

DRAWN BY:

CHECKED BY:

PORT MONMOUTH COMBINED FLOOD CONTROL
AND SHORE PROTECTION FEASIBILITY STUDY
LEVEL OF PROTECTION @ ELEVATION 13.0

REVIEWED BY:
SECTION CHIEF

CONCURRED BY:
CHIEF, DESIGN BRANCH

APPROVAL RECOMMENDED:
CHIEF, ENGINEERING DIVISION

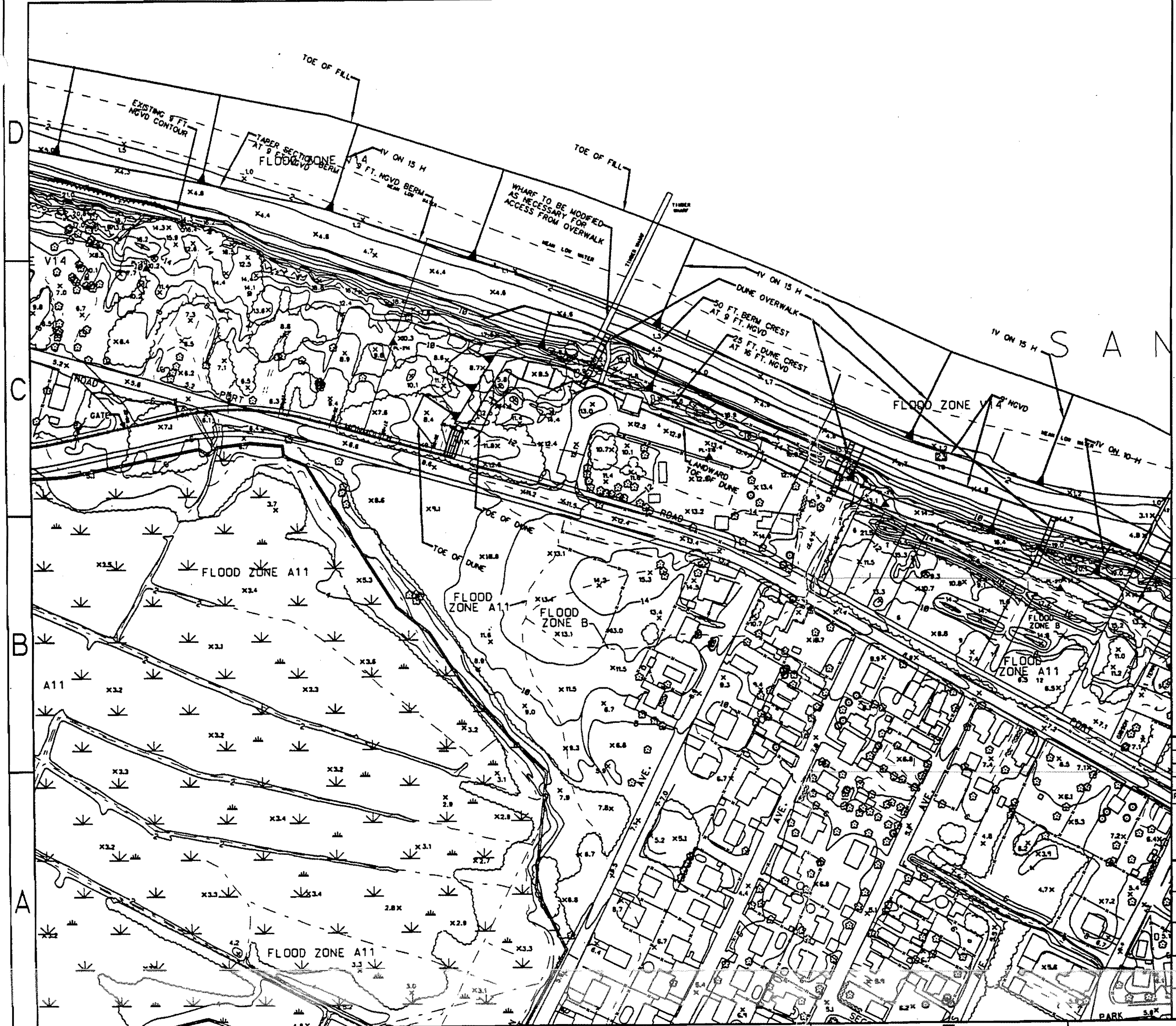
APPROVAL BY:
COLONEL, C.E. DISTRICT ENGINEER

SHEET REFERENCE NUMBER:

DATE:

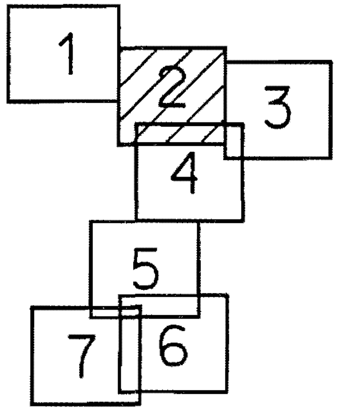
SCALE: 1" = 200'

FIG. 1



SHEET LEGEND

- BUILDING
- ROAD, PARKING AREA
- FENCE
- WOODED AREA
- STANDARD LEVEE
- ROAD RAISING
- STANDARD FLOODWALL
- (C2) INTERIOR DRAINAGE AREA DESIGNATION
- PUMP STATION
- INTERIOR DRAINADE OUTLET
- DRAINAGE SWALE
- CLOSURE GATE



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PORT MONMOUTH COMBINED FLOOD CONTROL
AND SHORE PROTECTION FEASIBILITY STUDY
LEVEL OF PROTECTION @ ELEVATION 13.0

REVIEWED BY:
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CONCURRED BY:
CHIEF, DESIGN BRANCH

APPROVAL RECOMMENDED:
CHIEF, ENGINEERING DIVISION

APPROVAL BY:
COLONEL, C.E. DISTRICT ENGINEER

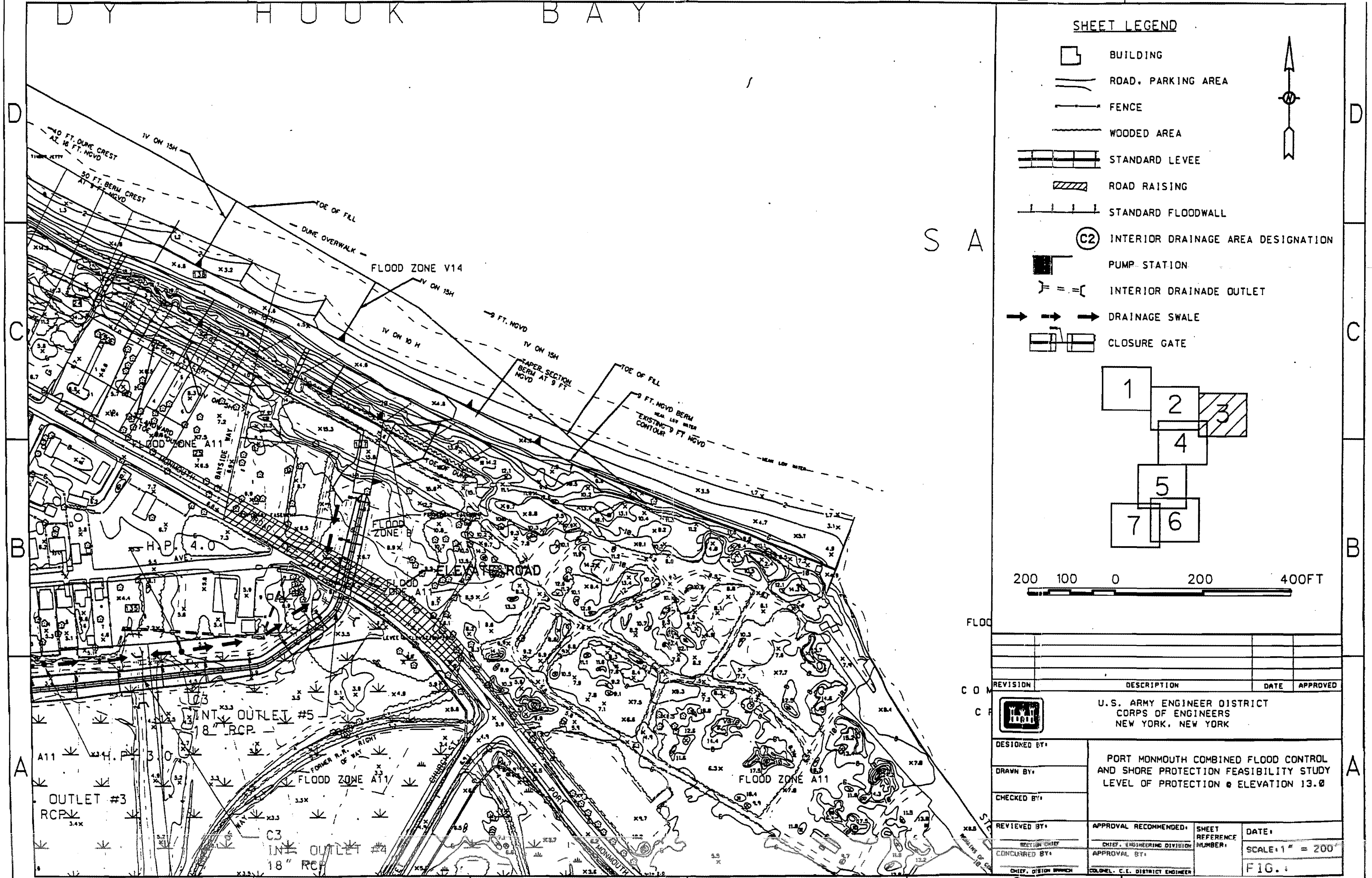
SHEET REFERENCE NUMBER:

DATE:

SCALE: 1" = 200'

FIG.:

A4-2



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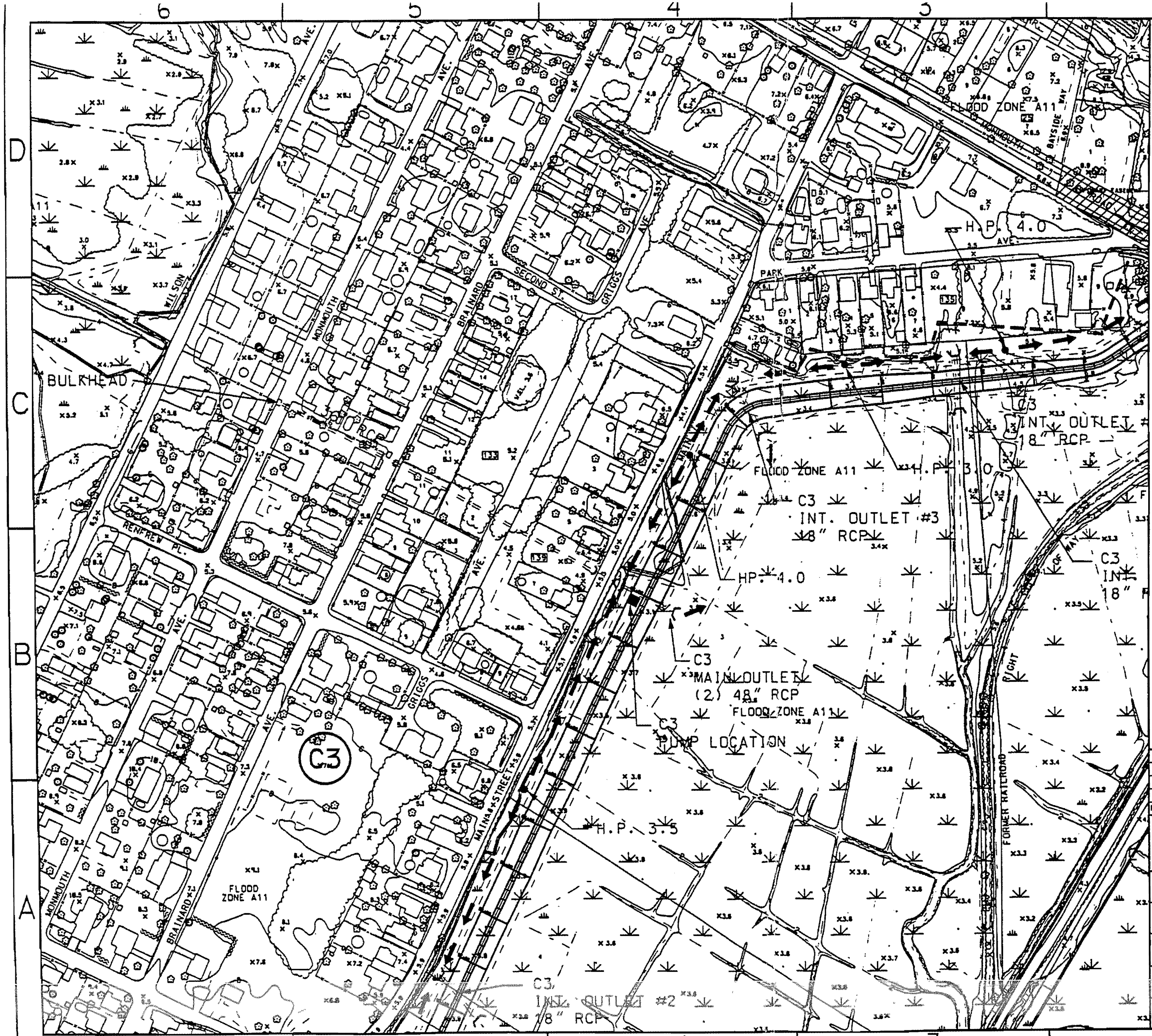
SHEET REFERENCE NUMBER:

DATE:

SCALE: 1" = 200'

FIG.:

A4-3



SHEET LEGEND

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- WOODED AREA
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- STANDARD FLOODWALL
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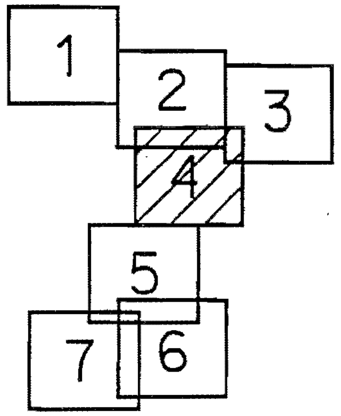

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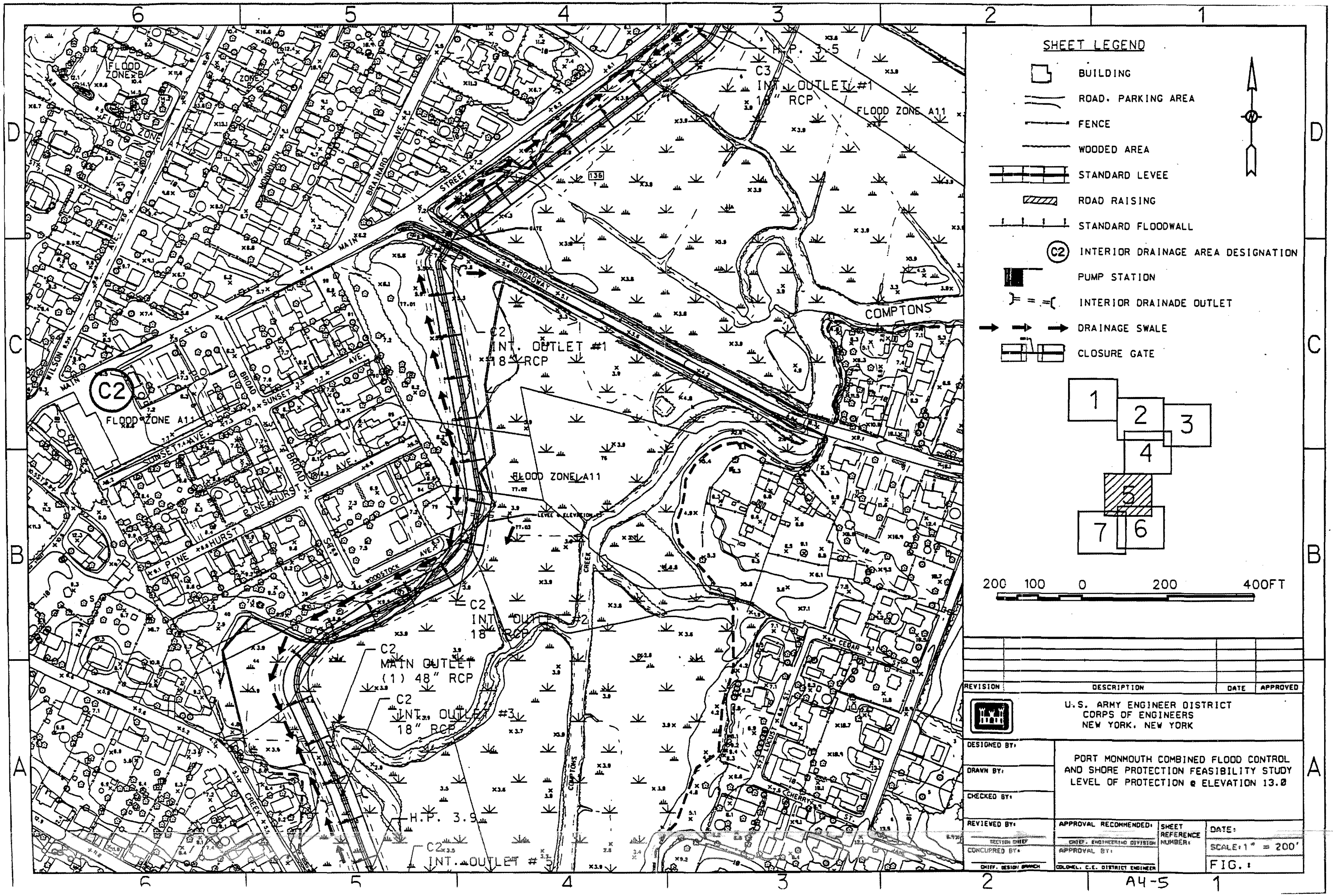
SHEET REFERENCE NUMBER:

DATE:
SCALE: 1" = 200'
FIG.:

A4-4



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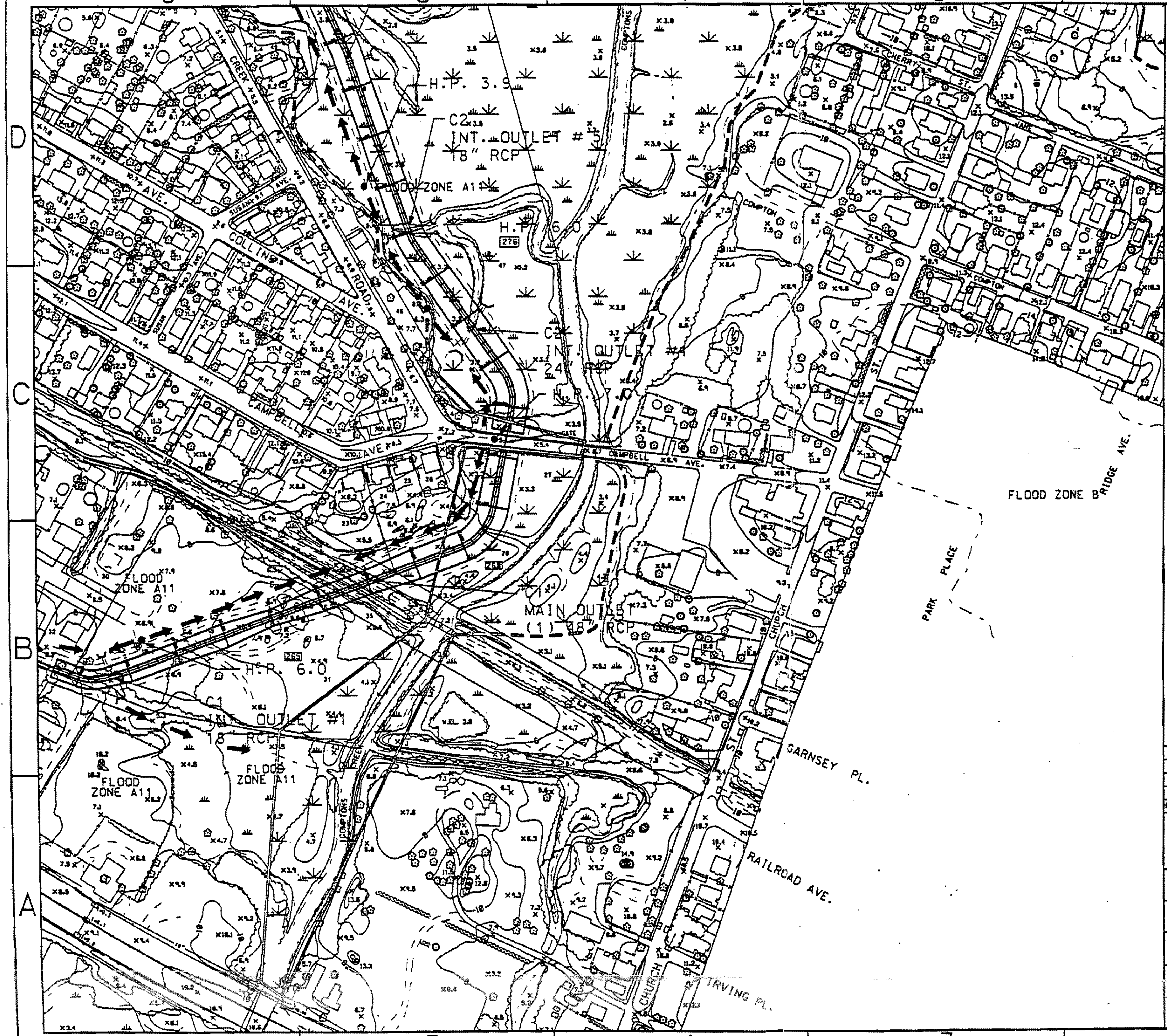
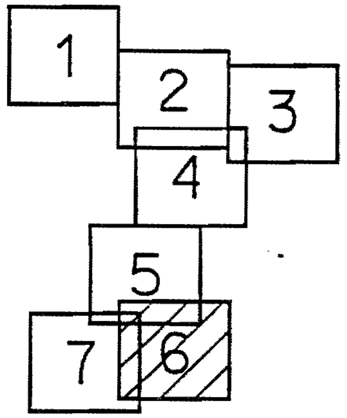
SCALE: 1" = 200'

FIG.:



SHEET LEGEND

- BUILDING
- ROAD, PARKING AREA
- FENCE
- WOODED AREA
- STANDARD LEVEE
- ROAD RAISING
- STANDARD FLOODWALL
- (C2) INTERIOR DRAINAGE AREA DESIGNATION
- PUMP STATION
- INTERIOR DRAINADE OUTLET
- DRAINAGE SWALE
- CLOSURE GATE



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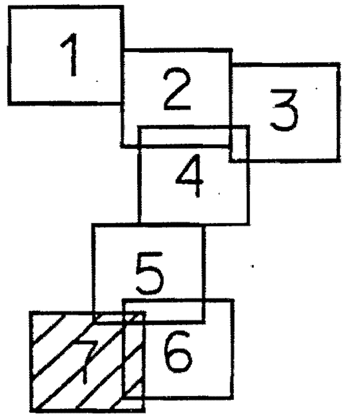
SCALE: 1" = 200'

FIG.:

SHEET LEGEND

⬜	BUILDING
═	ROAD, PARKING AREA
—·—	FENCE
----	WOODED AREA
═╪═	STANDARD LEVEE
▨	ROAD RAISING
┴┴┴	STANDARD FLOODWALL
(C2)	INTERIOR DRAINAGE AREA DESIGNATION
■	PUMP STATION
⊱ = =(	INTERIOR DRAINADE OUTLET
→ →	DRAINAGE SWALE
⊟	CLOSURE GATE



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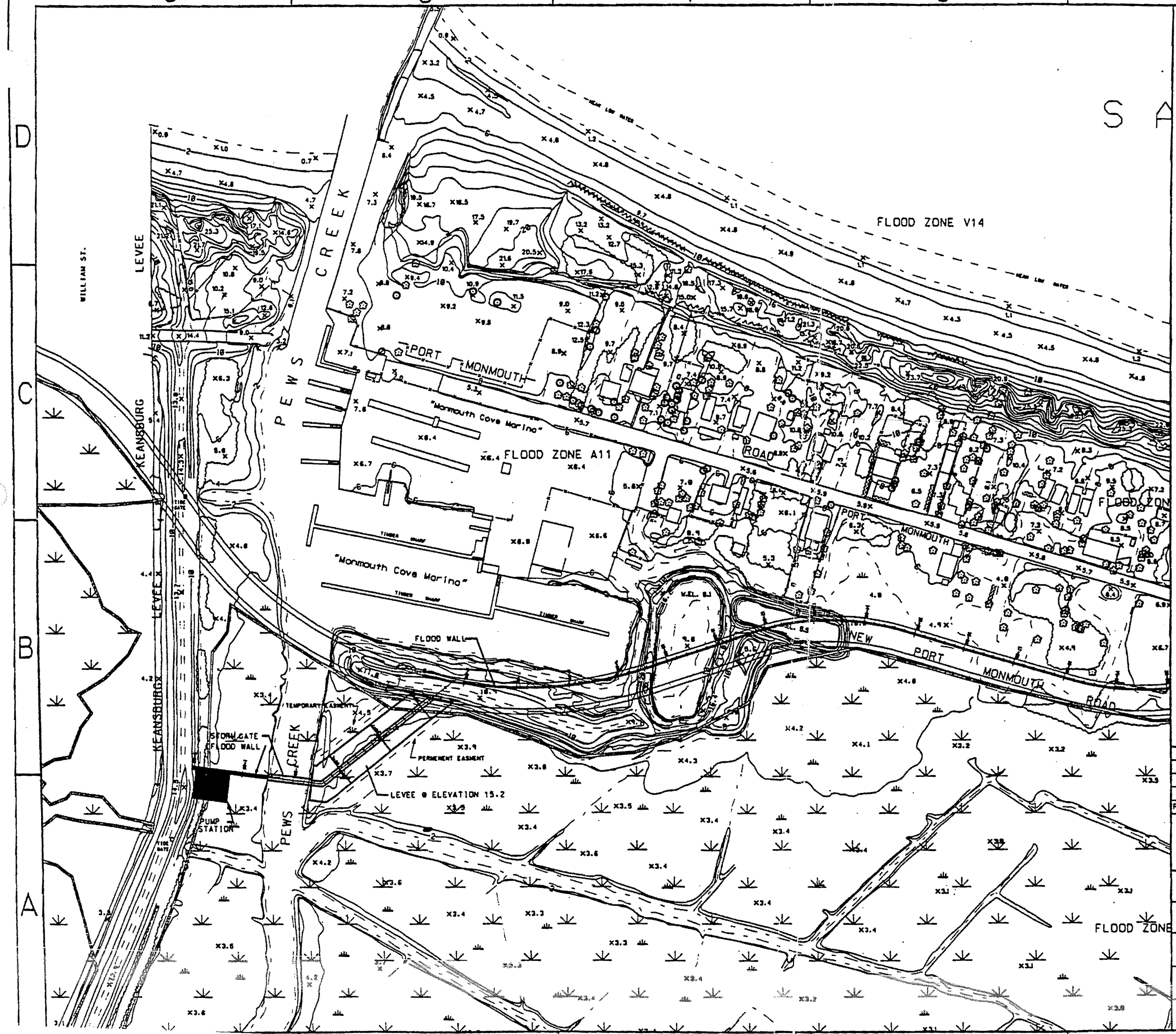
SCALE: 1" = 200'

FIG. :

A4-7



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PORT MONMOUTH COMBINED FLOOD CONTROL
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LEVEL OF PROTECTION @ ELEVATION 15.2

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DATE:

SCALE: 1" = 200'

FIG.:

A4-8

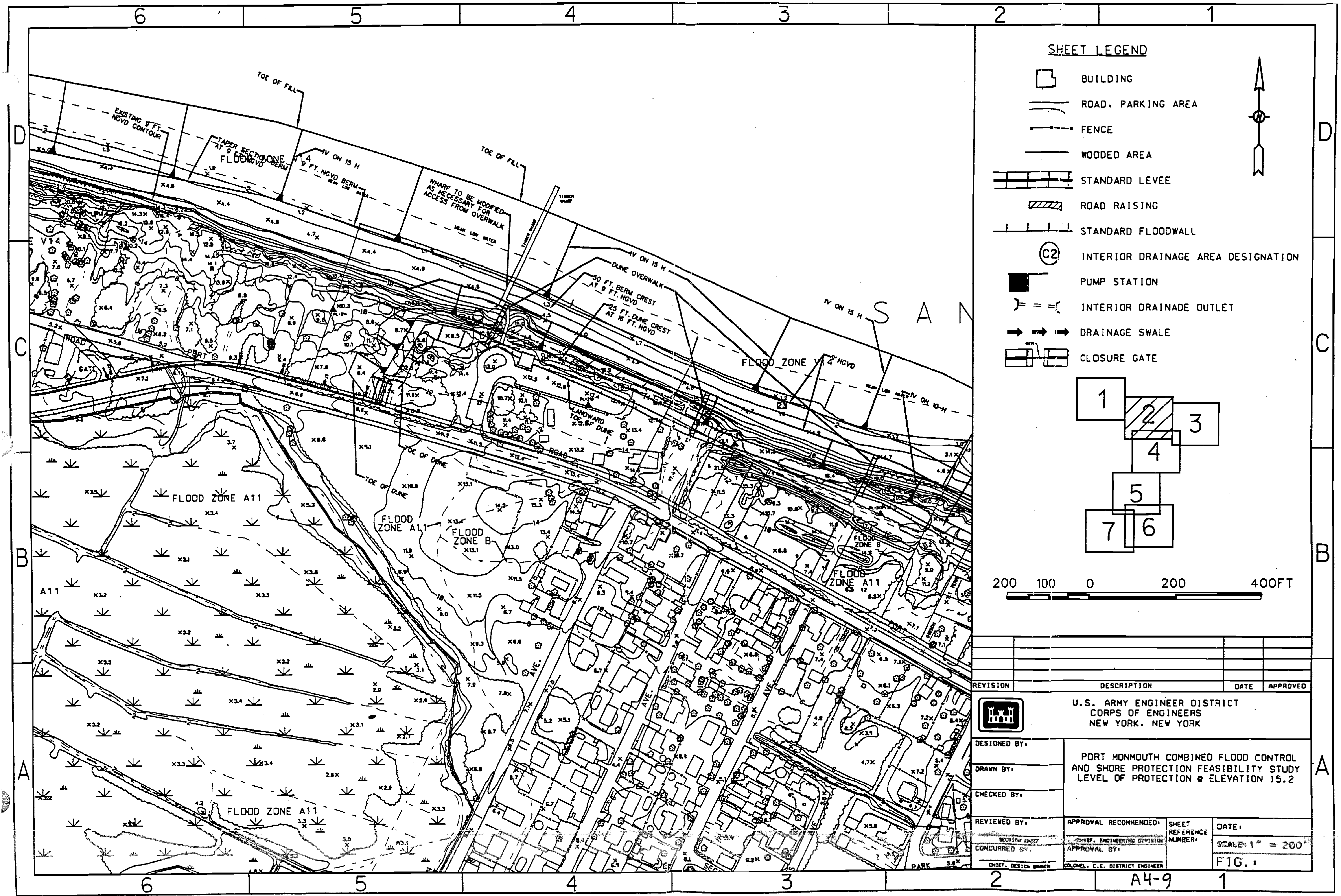

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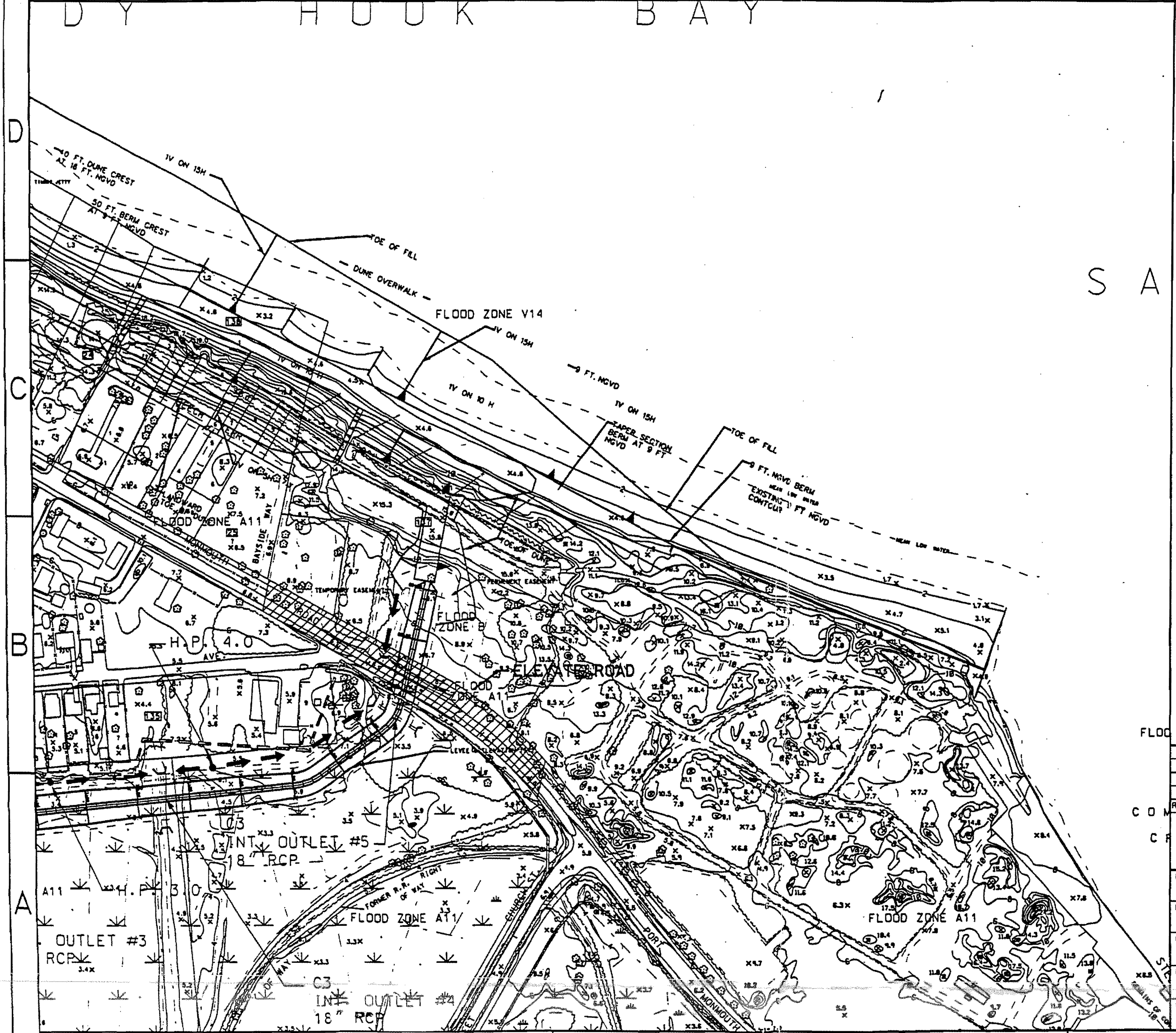
FIG.:



SHEET LEGEND

- ⌂ BUILDING
- ═ ROAD, PARKING AREA
- —·— FENCE
- ⁓ WOODED AREA
- ╪ STANDARD LEVEE
- ▨ ROAD RAISING
- ┴ STANDARD FLOODWALL
- Ⓒ2 INTERIOR DRAINAGE AREA DESIGNATION
- ■ PUMP STATION
- }= =(INTERIOR DRAINADE OUTLET
- → DRAINAGE SWALE
- ⊞ CLOSURE GATE

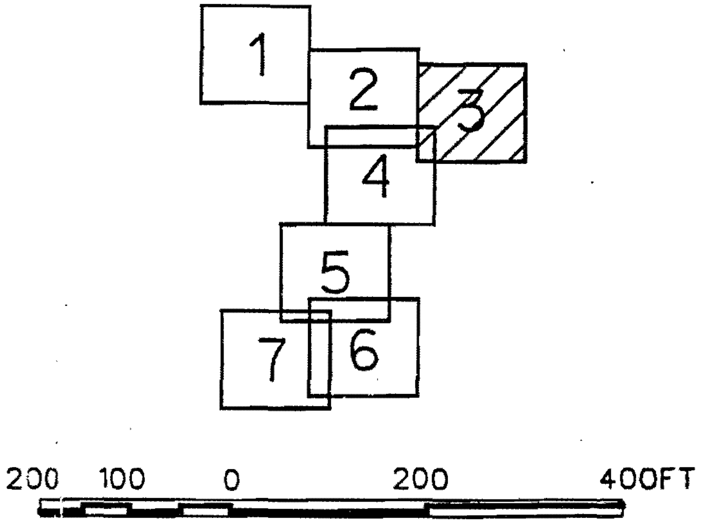


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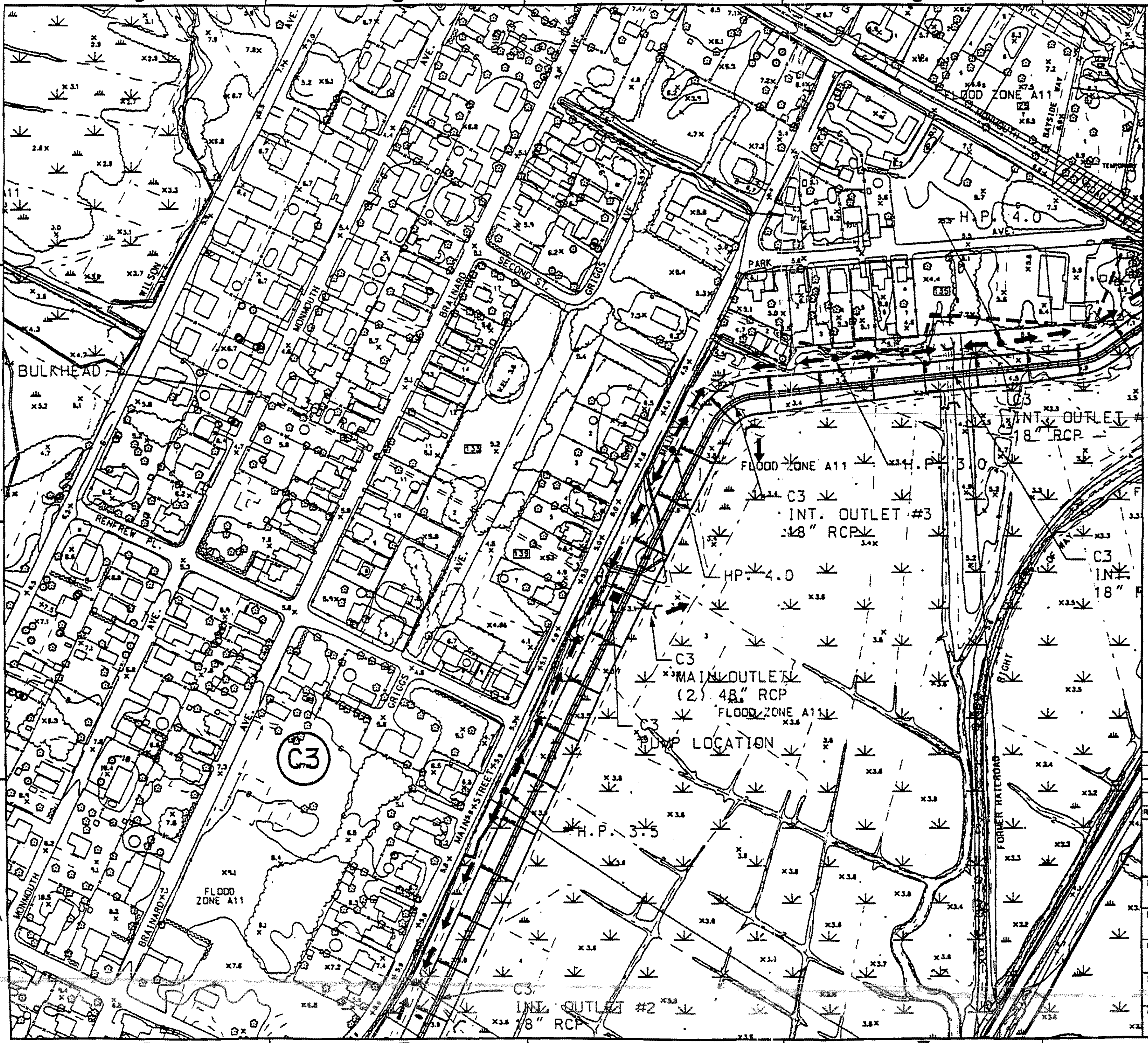
FIG.:

A4-10



SHEET LEGEND

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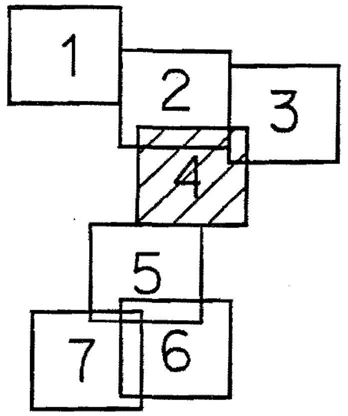

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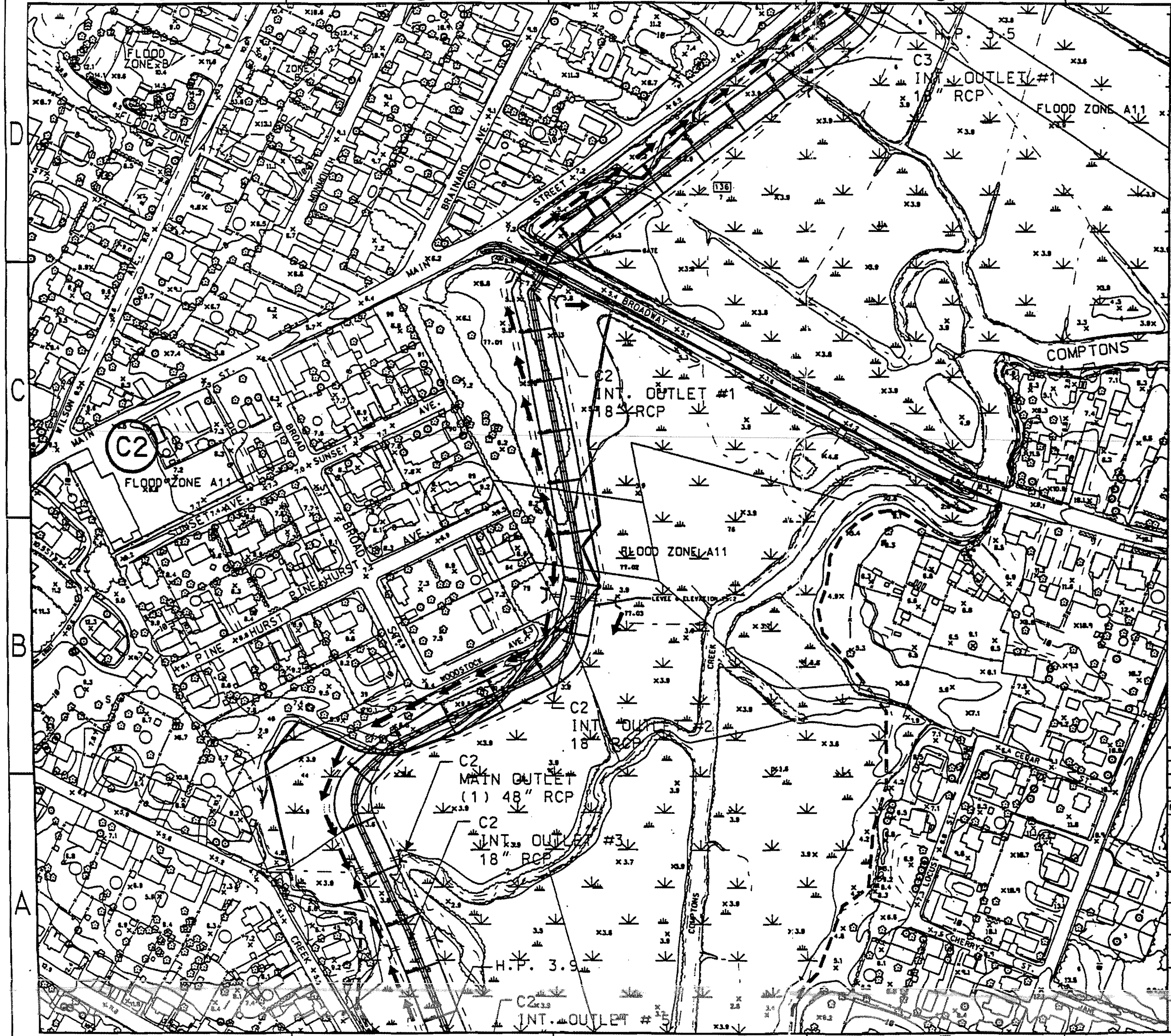
SHEET REFERENCE NUMBER:

DATE:
SCALE: 1" = 200'
FIG.:



SHEET LEGEND

	BUILDING
	ROAD, PARKING AREA
	FENCE
	WOODED AREA
	STANDARD LEVEE
▨	ROAD RAISING
	STANDARD FLOODWALL
(C2)	INTERIOR DRAINAGE AREA DESIGNATION
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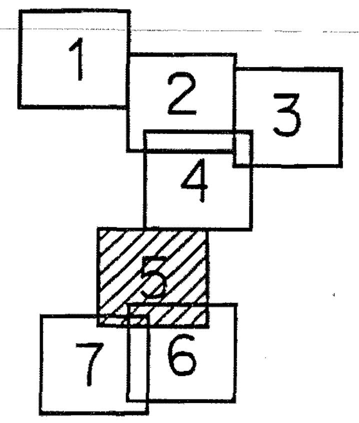

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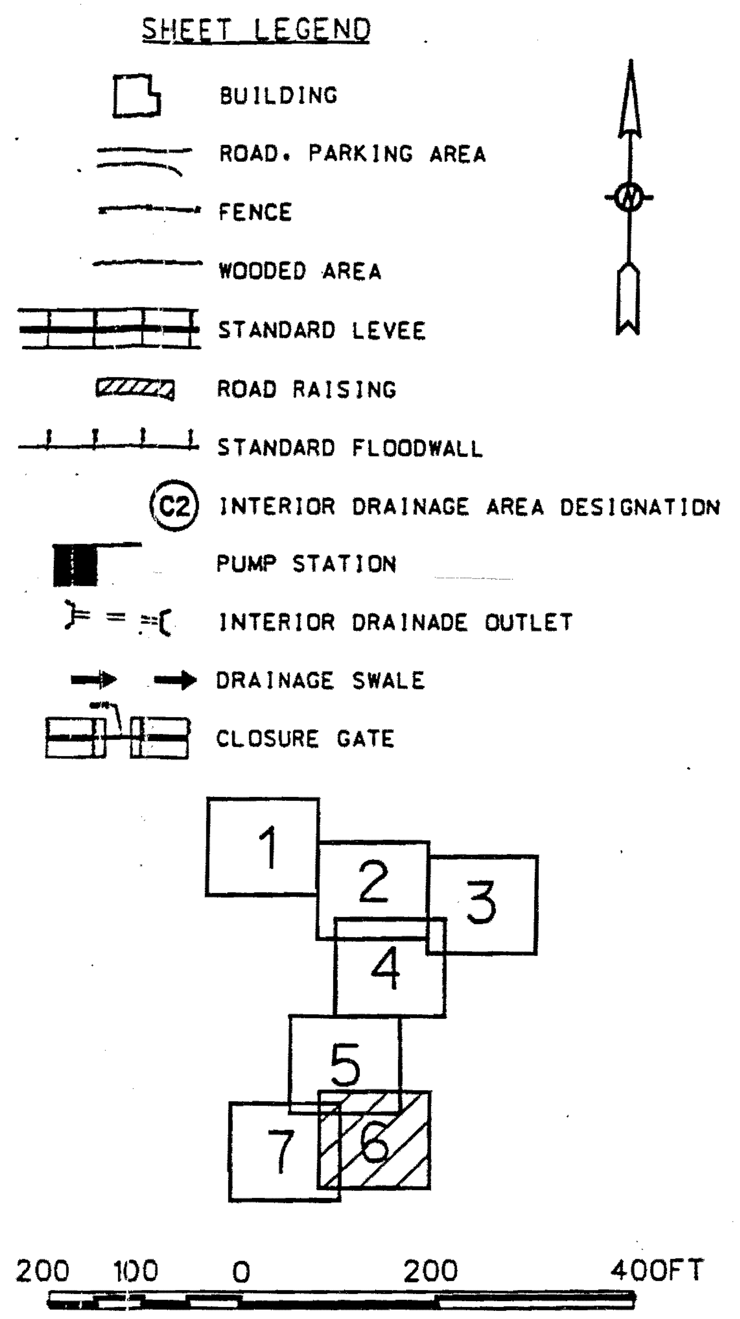
SCALE: 1" = 200'

FIG.:

A4-12 1



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SECTION CHIEF

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SCALE: 1" = 200'

FIG.:

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CHIEF, DESIGN BRANCH

APPROVAL BY:
COLONEL, C.E. DISTRICT ENGINEER

SHEET LEGEND

- BUILDING
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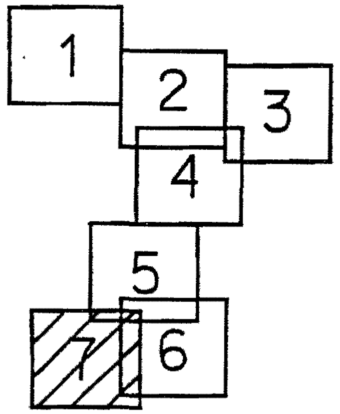

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FIG.:

JANUARY 2000

PORT MONMOUTH COMBINED FLOOD CONTROL AND SHORE PROTECTION FEASIBILITY STUDY

APPENDIX B

BENEFITS

page B

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INTRODUCTION

Purpose

B1. This Appendix presents the benefits and associated analysis procedures used in the determination of the economic viability for federal participation in shore protection and tidal flood control.

B2. Benefits were calculated for the plan which was anticipated to be the most implementable with respect to local support, survivability and storm protection criteria. Alternatives were screened for relative cost-effectiveness based on the level of without project damages, and preliminary estimates of benefits and costs.

Benefit Types

B3. Benefits to be derived from the plan of improvement include:

1. Reduced inundation damage to structures
2. Reduced public emergency costs
3. Reduced maintenance of the existing beach
4. Reduced Federal Insurance Administrative costs
5. Enhanced recreation use

Conditions

B4. Estimates of monetary benefits are based on May 1998 price levels and a 50-year period of analysis, and reflect the economic condition of the flood plain as of 1997. The base year for the proposed project is 2002. All calculations utilize the fiscal year 2000 discount rate of 6-5/8%.

Exclusions

B5. Benefits due to reduced traffic delays were not anticipated to be significant and were therefore not included.

DESCRIPTION OF THE STUDY AREA

Location

B6. The area of study described in this appendix is contained within Port Monmouth, a bayshore area located in northern Middletown Township, Monmouth County. The northern border of Port Monmouth is defined by the Raritan Bay while the southern border is designated by New Jersey State Route 36. The eastern and western borders are denoted by Compton Creek and Pews Creek, respectively (see Figure B-1). The economic assessment includes a limited area on the right bank of Compton Creek considered necessary to adequately evaluate alternative storm protection solutions. A number of homes located along Pews Creek upstream of Route 36 were not included in the initial study area definition. Local coordination efforts revealed that many homes in this area are subject to significant flooding, with some residents reporting over $60,000 damage in the December 1992 storm. This area was subsequently incorporated into the study as damage reach P4.

Accessibility

B7. The Study Area is convenient to major population centers through a network of modern highways. The Garden State Parkway and Route 9 run northward to New York State and southward to Cape May, New Jersey. Route 287 extends westward beyond Middlesex County and the New Jersey Turnpike provides additional north-south access. Direct access from these major corridors to the Bayshore is provided by Route 35 and Route 36. The communities are also serviced by the shore line of New Jersey Transit which provides passenger rail access to Newark and New York City, and by ferry service to downtown Manhattan.

B8. Significant improvements in access to Port Monmouth were recently completed. These include reconstruction of Church Street and Port Monmouth Road and construction of relocated bridges over Pews and Compton Creeks. These roadways were rebuilt to a minimum elevation of +9 NGVD, improving emergency access during all but the most severe storms. Primary routes from Route 36 to the shorefront are Wilson Avenue and Main Street.

Population

B9. Population in Monmouth County increased by 219,000 persons between 1960 and 1990. While this represents a 65% increase in 30 years, the recent trend shows a reduced growth rate, going downward from 38% between 1960 and 1970 to 9% between 1970 and 1980. Census data for 1990 indicates a continued growth of 10% since 1980 with an increase of 50,000 people, suggesting a stabilization of the growth rate over the last twenty years. Population data for New Jersey, Monmouth County and adjacent counties is shown in Table B-1. Table B-2 provides a summary of population data for the Middletown Township and adjacent communities.

Land Use and Economy

B10. The majority of land in the immediate project area contains residential development with commercial development concentrated along Route 36. The Belford fish co-op at the mouth of Compton Creek represents an important regional commercial resource.

B11. Historically, the Bayshore played a role as a market and distribution center for the agricultural goods produced on the fertile soils of the County's interior. Later the Bayshore's local commercial resources were developed. These included shellfish, clay (used in brick and tile manufacturing) and the waterfront as a tourist attraction.

B12. The economy of Monmouth County has undergone extensive growth in recent years with much of the development concentrated along the major transportation routes. The majority of non-residential development has been for office and research facilities, probably due to the availability of comparatively inexpensive land with good access to the Northern New Jersey - New York City markets. Economic development within the Township of Middletown has been extremely strong in recent years. An extensive expansion of the AT&T business campus is currently underway, and there have been numerous smaller commercial and residential developments within the Port Monmouth section of Middletown. New development has generally been limited to large public projects, including the reconstruction of Port Monmouth Road and Church Street, replacement of the Port Monmouth road bridge, modification of the Pews Creek Channel, and the acquisition and development of a county park along the Bayshore. The majority of development within Port Monmouth is more than 25 years old, and was constructed prior to implementation of the Flood Insurance Program and adoption of the associated Flood Plain Management Regulations.

TABLE B-1
POPULATION AND PROJECTIONS OF FUTURE POPULATIONS
MONMOUTH AND SURROUNDING COUNTIES

	Population 1960	Census 1970	Census 1980	Census 1990	1995	2000	2005	2010	2015	2020
New Jersey State	6,066,782	7,168,164	7,365,011	7,730,188	8,154,000	8,450,000	8,685,200	8,895,700	9,042,900	9,179,200
Middlesex County	433,856	583,813	595,893	671,780	690,600	726,600	760,800	791,800	819,900	846,000
Monmouth County	334,401	461,849	503,173	553,124	608,400	623,700	639,300	655,300	663,900	675,200
Ocean County	108,241	208,470	346,038	433,203	449,600	484,400	515,800	545,900	572,300	594,300

Source: Projection, NEW JERSEY DEPARTMENT OF LABOR, DIVISION OF PLANNING AND RESEARCH, OFFIC
OF DEMOGRAPHIC AND ECONOMIC ANALYSIS, except Monmouth County 1990 through 2010 projection
which are based on 1990 State Development and Redevelopment Plan

TABLE B-2
POPULATION SUMMARY DATA BY MUNICIPALITY

		RESIDENT POPULATION 1960-1990				POPULATION DENSITY 1960-1990			
MUNICIPALITY	**AREA (sq.mi.)**	**1960**	**1970**	**1980**	**1990**	**1960**	**1970**	**1980**	**1990**
South Amboy City	1.55	8,422	9,338	8,322	7,863	5,433.5	6,024.5	5,369.0	5,072.9
Sayreville Borough	16.13	22,553	32,508	29,969	34,986	1,398.2	2,015.4	1,858.0	2,169.0
Aberdeen Township *	5.56	7,359	17,680	17,235	17,038	1,323.6	3,179.9	3,099.8	3,064.4
Keport Borough	1.41	6,440	7,205	7,413	7,586	4,567.4	5,109.9	5,257.4	5,380.1
Union Beach Borough	1.87	5,862	6,472	6,354	6,156	3,134.8	3,461.0	3,397.9	3,292.0
Hazlet Township **	5.63	15,334	22,239	23,013	21,976	2,723.6	3,950.1	4,087.6	3,903.4
Keansburg Borough	1.07	6,854	9,720	10,613	11,069	6,405.6	9,084.1	9,918.7	10,344.9
Middletown Township	41.11	39,675	54,623	62,574	68,183	965.1	1,328.7	1,522.1	1,658.6
Port Monmouth	1.3	+	+	+	3,558	+	+	+	2,736.9
Belford	3.0	+	+	+	4,151	+	+	+	1,383.7
Leonardo	0.6	+	+	+	3,788	+	+	+	6,313.3
Atlantic Highlands Borough	1.24	4,119	5,102	4,950	4,629	3,321.8	4,114.5	3,991.9	3,733.1
Highlands Borough	0.77	3,536	3,916	5,187	4,849	4,592.2	5,085.7	6,736.4	6,297.4
Total Project Area ***	76.34	120,154	168,803	175,630	195,832	1,573.9	2,211.2	2,337.3	2,414.7

\+ Data not available.

* Matawan Township name changed to Aberdeen Township, 1978.

** Raritan Township name changed to Hazlet Township, 1967.

*** Since additional population data is not available, all of Middletown Township is considered.

Source: Population from U.S. Decennial Census.
Area Square Miles based on 1990 Census of Population and Housing.

DESCRIPTION OF THE PROBLEM

B13. Extratropical storms, northeasters, and hurricanes historically impact the Raritan and Sandy Hook Bayshore areas. These storms produce tides and waves that cause extensive flooding and erosion to the study area. The shoreline composition has been greatly altered with time. Storm induced erosion has removed much of the beachfront and has expedited deterioration of any existing coastal protection and drainage structures. In addition to physical alterations, tidal surges often block existing storm drainage systems, resulting in several areas experiencing prolonged and extensive flooding.

B14. Storms impacting the area include the September 14, 1944 hurricane, extratropical storms of November 25, 1950 and November 6-7, 1953, Hurricane Donna (1960), March 6-8, 1962 Northeaster, March 12, 1984 Northeaster, and the December 11, 1992 Northeaster. These storms resulted in transportation problems such as damaged roads and bridges; damage or destruction of shoreline structures such as dunes, jetties, and the damage and destruction of homes and commercial properties. Overall these problems have resulted in extensive damage to upland properties, loss of life, numerous evacuations during storms, and a significant constraint to commerce and regional economic development. Several of the roads and homes in the community are subject to chronic flooding during periods of high astronomic tides.

B15. The Port Monmouth area experiences damage from tidal inundation from the waters of Pews and Compton Creek. During even moderate storms, tidal flood waters enter the creeks and quickly spread over the broad low-lying flood plain from both the east and the west. A tidal stage of ten feet MSL results in flooding so severe that most residents north of Route 36 are stranded. Extensive damage to hundreds of structures has been recorded in the Port Monmouth area during such storms.

WITHOUT PROJECT FUTURE CONDITIONS

B16. The without project future conditions at Port Monmouth is identified as continuing erosion of the shoreline with periodic placement of sand to maintain the beach and dune.

B17. Tidal flooding is expected to increase in severity in direct relation to the anticipated rise in relative sea level. Based on long-term trends measured at the Sandy Hook Gage, a rate of 0.014 foot per year increase is anticipated, resulting in a 0.7 foot increase over the analysis period.

B18. Monmouth County is currently developing a recreation facility along the Bayshore. In conjunction with this plan, as structures between Port Monmouth Road and the shorefront have or will be removed as part of the recreation program, the County has acquired the marina at Pews Creek. There are extensive efforts underway to improve navigation and reduce future sedimentation. These efforts include widening the inlet, constructing a west jetty and raising the east jetty and existing bulkheads. Potential activities as stated in the Bayshore Waterfront Access Plan are:

> *Nature interpretation, boating, saltwater swimming, sunbathing, educational program in cooperation with fishing industry, wetlands preservation, and active recreation.*

B19. A ferry terminal providing high speed access to New York City is currently under development.

EXTENT AND SCOPE OF ALTERNATIVES

General

B20. In order to address the tidal flooding problem in Port Monmouth, each alternative considered must provide flood protection from Pews Creek, Compton Creek and Raritan Bay. Alternative protection features were formulated independently for each flooding source. Preliminary cost and benefit estimates for each alternative were then combined in a matrix format to determine the preferred protection alignment. Each alternative considered was initially sized to provide flood protection against a storm with a 1% chance of being equaled or exceeded in any year (100-year storm) including allowances for anticipated sea level rise. Based on modeling conducted at the Coastal Engineering Research Center (CERC), the flood stage associated with such a probability is approximately 12.2 ft. NGVD. The standard deviation based on Empirical Simulation Techniques (EST) was estimated to be ±2.2 ft. In order to ensure that a structural solution may reliably protect against a 100-year flood, the top of levee and flood wall structures was set at 14.0 ft. NGVD. This represents approximately 0.8 standard deviation, above the mean 100-year flood elevation under existing conditions, providing approximately a 71% reliability during a 100-year event. When sea level rise over the project life is considered, there is approximately a 65% reliability during a 100-year event.

B21. After identifying which if any alternative alignments are implementable with consideration of economic, environmental and institutional constraints, a preferred plan was identified for NED optimization.

Identification of NED Alignment

B22. The overall plan has been formulated as four separate components for evaluation, all of which are necessary to provide protection from storm damage and flooding. Separate discussions are included of plans for the Raritan Bay shorefront, Pews Creek, Compton Creek and Interior Drainage. Each of these components has been formulated with consideration of avoiding or minimizing environmental impacts. The plan requires development of a fifth component, environmental mitigation, to meet planning requirements for completeness where total avoidance of impacts was not feasible.

B23. The first planning effort was an investigation of the preliminary alternatives in the fall of 1996, which documented preliminary costs and impacts for an array of possible levee, storm gate, and non-structural plans. These preliminary results were coordinated with the Local Sponsor, New Jersey Department of Environmental Protection (NJDEP), which expressed several preferences for specific features. From a land use and environmental perspective the agency expressed a preference for a storm gate at Pews Creek which would minimize direct footprint impacts. From an operations and maintenance (O&M) perspective, however, there was significant concern that the use of a storm closure gate at Pews Creek could require a long term commitment to increase agency staff and may not be a supportable alternative. These concerns were taken into account in conducting the screening of plans for more detailed development.

B24. For the Compton Creek segment of the project, the alignment identified as C2 was selected as the preferred alternative. This alignment minimized the impact of levees on the wetlands, without relying on extensive lengths of floodwalls. Extensive areas of floodwalls, such as proposed in alternatives C3 and C4, would be prohibitively expensive and would probably create a graffiti nuisance. Alternative C1 was not selected due to unacceptable levels of environmental disturbance.

B25. The selections of the shorefront element attempted to reduce costs and to minimize or avoid future beachfill renourishments while providing the desired level of protection. One bayshore protection layout that was examined was the alignment featuring an upland dune layout, with a sheet pile floodwall protecting the Spy House property with minimal footprint (S4). However, in comments pertaining to the preliminary layout, the Monmouth County Board of Recreation Commissioners expressed opposition because the dune footprint conflicted with the local plan for the shoreline development. Since the parkland was purchased as dedicated recreation land with Green Acres funding, a change in the use of the land would require in kind replacement. Given the unique nature of the site, blending active recreation with interpretative historic facilities, such replacement was not viable as no other available shorefront land exists. Therefore, the dune layout over upland features is not considered implementable. In order to maintain consistency with public usage of the shorefront, alternative S1 consisting of beach and dune fill was selected. Alternatives S2 and S3 were not selected since they are more costly than S1.

B26. The screening of protection along Pews Creek attempted to minimize impacts to the environment without creating severe social impacts due to numerous structure acquisitions. Since the local sponsor indicated that they may not support the use of a closure gate at Pews Creek, alternative P1 was not selected for continued development. Alternative P2 was not selected due to excessive impacts to the tidal wetlands. While alternative P3 would avoid wetland impacts, it was not selected due to the need for numerous structure acquisitions. Alternative P4 was initially identified as the Pews Creek alternative which provided the best balance in minimizing environmental and social impacts.

B27. The findings of the screening process were subsequently further coordinated with representatives of Middletown Township and various County agencies. In an effort to expand the geographic extent of coverage, Township officials indicated a clear preference for storm closure gates at Pews, and if possible, Compton Creeks. In response to the Local Sponsor's reluctance to make a long term commitment of State manpower for security and O&M for such closure gates, the Township officials suggested possible solutions. Currently the Town maintains a staff for the O&M of the nearby East Keansburg Storm Water Pump Station which could possibly service a station at Pews Creek. In addition, by locating the gate adjacent to the Monmouth County marina vandalism and security concerns would be reduced. Based on the reduction of environmental impacts, the more inclusive protection, and the availability of local resources to support the maintenance of a closure facility, the Local Sponsor indicated a willingness to support a storm closure structure at Pews Creek.

B28. Subsequent to the local coordination meeting additional economic investigations were undertaken to identify if gate structures could be supported as components of the National Economic Development (NED) Plan which normally establishes the limit for Federal cost sharing. Based on the preliminary screening information the initial estimate of total annual costs for a gate and pump station at Pews Creek indicated that the annualized cost of the gate alternative would be $223,000/year higher than the originally preferred Pews Creek Levee alignment (P4). This increase in cost compares to a preliminary estimate of a $280,000 increase in annual benefits (excluding residual interior damage) due to the protection of approximately 90 additional structures in the Bray Avenue area. Additional benefits to 121 structures upstream of the previously defined study area, which ended at Route 36, would total approximately $413,000 annually. Accordingly the gate at Pews Creek would yield $460,000 in annual net benefits and was considered as a possible element of the NED Plan.

B29. Whereas the levee alignment P4 could also effect the tidal inundation patterns of wetlands located on the protected side of the levee, the use of a closure gate alternative at Pews Creek would reduce this effect as well as the extent of the permanent project footprint within the wetlands. The gate alignment and opening would be developed to allow tidal inundation of the wetlands to continue with minimal disruption of existing depths and frequency. The size and configuration of the gate required to maintain the existing tidal flow conditions would later be established as part of a 2-dimensional hydro-dynamic modeling effort in this study.

B30. Based on the request of Township officials to consider a more comprehensive protection alignment than the levee alignment (C2) favored in the initial screening, a levee/gate alignment extending further east over Compton Creek was examined. This preliminary levee/gate alignment would follow the new Port Monmouth Road, extending protection to the west bank of this creek. The alignment would provide protection to approximately 12 ft. NGVD, and would provide protection to 276 structures not protected by the proposed levee. Since structures in this area do not suffer significant damage as frequently as structures in other portions of the study area only limited additional protection would be provided by a gate on Compton Creek. Economic analysis indicated that the use of a storm closure gate and levee at elevation 12 ft. NGVD will only provide 20% and 50% reductions in equivalent annual damage in the additional reaches protected by the gate at Compton Creek. The preliminary estimate of damage reduction benefits for a gate protecting these areas totals $77,000 annually with the added annual cost exceeding these benefits. Therefore, a gate at Compton Creek was not considered an economically viable element and the levee alternative was selected.

FLOOD DAMAGE

General

B31. The analysis of flood damage utilized the following basic steps:

- Inventory flood plain development
- Estimate depreciated replacement costs
- Assign generalized damage functions
- Assign evaluation reaches
- Calculate aggregated stage vs. damage relationships.

B32. The flood damage calculations were performed using Version 1 of the HEC-FDA Flood damage Reduction Analysis computer program. This program applies Monte Carlo Simulation to calculate expected damage values while explicitly accounting for uncertainty in the input data.

B33. Under current Corps guidance, risk and uncertainty must be incorporated in flood damage reduction studies. The following areas of uncertainty were incorporated into the calculation of flood damage:

- first floor stage
- structure value
- content-to-structure value ratio
- other-to-structure value ratio

B34. Based on EM 1110-2-1619 Table 6-5, the first floor elevation standard deviation is approximately 0.6 foot when using topographic mapping with 2-ft contour intervals.

B35. The coefficient of variation in structure value was estimated to be 10% for a building valued at $100,000. This is equivalent to a standard deviation of $10,000. The other-to-structure value standard deviation is also assumed to be $10,000.

B36. EM 1110-2-1619 suggests that in lieu of better site-specific information, content-to-structure value ratios based on large samples of Flood Insurance Administration (FIA) claims records can be used (Table 6-4 presented in EM 1110-2-1619). An approximate average of the standard deviation is 0.25.

Conditions

B37. The base year for this economic evaluation is 2002. Since the period of analysis is determined to be 50 years, damages were evaluated for the period 2002-2051 using the fiscal year 2000 interest rate of 6-5/8%.

Inventory Methodology

B38. To accomplish the benefit analysis, the initial consideration was the development of a structural data base to assist in predicting flood damages. The structural data base was generated through a survey of the structures adjacent to the project area. The building data was obtained through a windshield survey of the area using topographic mapping with a scale of 1" = 100' with a 2-foot contour interval. The structure inventory was conducted in the summer of 1996 with a field update in the summer of 1997. Table B-3 indicates the type of physical characteristics obtained for the building inventory. Table B-4 provides a summary of flood plain structures. Along Pews Creek upstream of Route 36, the inventory was limited to categorizing structures by type and elevation, and identifying the typical structure value and foundation height.

TABLE B-3
PHYSICAL CHARACTERISTICS OBTAINED FOR BUILDING INVENTORY

1) Type- Residential, Commercial, etc.
2) Town
3) Location ID
  3a) Creek
  3b) Bank
  3c) Reach
4) Map Number
5) Structure ID
6) Structure Size
7) Stories
8) Usage
9) Basement/Foundation Type
10) Ground Elevation (NGVD)
11) Main Floor Height
12) Low Opening
13) Number of Garage Openings
14) Exterior Materials
15) Units on First Floor
16) Total Units
17) Number of Buildings
18) Quality
19) Owner Operator
20) Condition/Depreciation

B39. The data collected was used to categorize the structure population into groups having common physical features. Data pertaining to structure usage, size and stories assisted in the stratification of the building population. For each building, data was also gathered pertaining to its damage potential including ground and main floor elevations, lowest opening, size, construction material, and the condition as related to structure value depreciation.

Structure Values

B40. The value of each building in the flood plain was calculated using standard building cost estimating procedures from Means & Marshall & Swift. This analysis combines the physical characteristics obtained in the inventory with standard unit prices per square foot. Depreciation was then calculated based on the observed type and condition of each structure.

TABLE B-4 STRUCTURE POPULATION

	USAGE \ REACH	CL1	CL2	CL3	CR1	CR2	CR3	PR1	PR2	PR3	PR4*	Total
Non-Residential Structures	21 - Art Gallery							4				4
	22 - Auto Sales					2						2
	23 - Auto Service		1			2			1			4
	24 - Bank								1			1
	25 - Bar					2			1			3
	26 - Bathhouse								1			1
	27 - Church		1				1					2
	28 - Clothing Store						1		1			2
	30 - Diner		1			1		1	4			7
	31 - Drug Store								1			1
	32 - Dry Cleaning								1			1
	33 - Food Store			1					1	1		3
	34 - Funeral Home					1						1
	35 - Hair Salon								2			2
	36 - Hardware			1		1			1	1		4
	40 - Jewelers								1			1
	41 - Liquors			1								1
	42 - Marina				2			1				3
	44 - Office					1	1		2			4
	45 - Office Warehouse	1										1
	47 - Restaurant				1							1
	49 - Small Retail			3					4			7
	51 - Vacant	5		1	1	1			2			10
	71 - Food/ Kindred Prods	4										4
	77 - Printing/ Publishing					1						1
	82 - Electrical					1						1
	86 - Light Industry					1						1
	150 - Garage			1					1			2
	160 - Parking Lot											0
	201 - Fire House					1			1			2
	202 - Storage Garage					2						2
	206 - Schools						1					1
	207 - Rescue Squad								1			1
	208 - Library								1			1
	209 - Post Office								1			1
	210 - General Storage			1								1
	Subtotal Non-Residential	10	3	9	4	17	4	6	29	2	4	88
Residential Structures	1 - Colonial		61	48	5	93	23	3	75	6		314
	2 - Cape Cod		55	63	1	61	6	3	81	21		291
	3 - Ranch		92	54	1	38	9	1	68	25		288
	4 - Split Level		2			1			7	3		13
	5 - BiLevel		16	6	3	8	2	1	14	5		55
	6 - Raised Ranch		6	8		6		1	10	1		32
	7 - Bungalow		14		1	2		7	5			29
	9 - Mobile Home							2				2
	10 - 2-Family		2			4			2			8
	11 - Duplex					2		2	2			6
	12 - Multi-Family		2						2	1		5
	13 - Garden Apartment		3						12			15
	Subtotal Residential	0	253	179	11	215	40	20	278	62	117	1,175
	TOTAL	10	256	188	15	232	44	26	307	64	121	1,263

* Detailed distribution of structure types not available for Reach PR4.

Reach Selection

B41. To assist in determining the impacts of plans of protection, spatial economic reaches corresponding to anticipated limits of protection were defined. This procedure yielded the ten (10) reaches shown in Figure B-2.

Description of Damage Functions

B42. Generalized damage functions for structure damage, content damage and other damage were applied to the residential and non-residential structures. All of the damage functions used for this investigation were developed from on site surveys conducted during the Passaic River Study. The damage functions reflect damages as a percent of structural value over a full range of water depths and were applied on a structure by structure basis to determine damages at one foot increments of flood stage.

Stage vs. Damage

B43. Based on the type, usage and size of each structure inventoried, damage was calculated relative to the main floor of the structure. Using structure and ground elevation data these depth vs. damage relationships were converted to corresponding stage (NGVD) vs. damage relationships. Damages for individual structures at various stages were aggregated according to structure type (residential vs. non-residential) and location (reach). Resulting stage vs. damage curves are presented in Figures B-3 through B-12. Tabulated stage vs. damage data are presented in Tables B-5 through B-13. It should be noted that the stage damage curves reflect the impact of uncertainty in ground elevations.

Damage Verification

B44. The Federal Emergency Management Agency (FEMA) was contacted regarding damages from the December 1992 storm. Data provided indicates that 176 requests for disaster assistance were received from the Port Monmouth area. This compares well with the data utilized to calculate damage which indicates that 166 structures would be flooded above the main floor during this storm.

B45. Due to the lack of comparative historic damage estimates specific to the Port Monmouth area, the calculated damages were evaluated for reasonableness. The annual without project damage for residential structures was calculated to be $2,750 per structure. Given the extremely frequent flooding in this area, this level of annual damage per structure is considered reasonable.

Sea Level Rise

B46. Sea level rise is a significant factor in contributing to coastal erosion and tidal flooding. Based on NOAA tide gauge readings at Sandy Hook, sea level has been increasing at an average of approximately 0.014 foot per year and will result in an approximately 0.7 foot increase in tidal stages to the end of the project life. In future years this will result in more frequent and higher stages of flooding.

TABLE B-5
REACH CL1
ELEV. (NGVD) VS. DAMAGE SUMMARY
(IN THOUSANDS OF DOLLARS)

	Damages ($1,000)						
Stage	Residential	Apartment	Commercial	Industrial	Municipal	Emergency	Total
3	0.00	0.00	0.97	0.00	0.00	0.01	**0.98**
3.5	0.00	0.00	3.80	0.00	0.00	0.05	**3.85**
4	0.00	0.00	9.60	1.97	0.00	0.15	**11.72**
4.5	0.00	0.00	18.18	10.04	0.00	0.36	**28.57**
5	0.00	0.00	28.16	29.48	0.00	0.72	**58.35**
5.5	0.00	0.00	37.35	66.21	0.00	1.29	**104.85**
6	0.00	0.00	46.11	120.78	0.00	2.37	**169.26**
6.5	0.00	0.00	55.28	191.16	0.00	3.37	**249.81**
7	0.00	0.00	70.27	272.31	0.00	4.57	**347.15**
7.5	0.00	0.00	90.67	360.05	0.00	5.92	**456.64**
8	0.00	0.00	116.79	451.35	0.00	7.39	**575.53**
8.5	0.00	0.00	143.80	545.00	0.00	8.89	**697.69**
9	0.00	0.00	172.95	638.98	0.00	10.44	**822.37**
9.5	0.00	0.00	211.61	728.94	0.00	12.05	**952.60**
10	0.00	0.00	261.80	810.72	0.00	13.69	**1086.21**
10.5	0.00	0.00	315.77	879.65	0.00	15.22	**1210.64**
11	0.00	0.00	361.30	935.92	0.00	16.50	**1313.72**
11.5	0.00	0.00	396.49	982.17	0.00	17.52	**1396.17**
12	0.00	0.00	422.86	1022.33	0.00	18.35	**1463.54**
12.5	0.00	0.00	444.31	1059.10	0.00	19.07	**1522.49**
13	0.00	0.00	464.37	1093.47	0.00	19.76	**1577.60**
13.5	0.00	0.00	483.72	1124.30	0.00	20.39	**1628.41**
14	0.00	0.00	502.32	1150.60	0.00	20.95	**1673.87**
14.5	0.00	0.00	519.77	1171.23	0.00	21.42	**1712.42**
15	0.00	0.00	535.98	1187.00	0.00	21.82	**1744.81**
15.5	0.00	0.00	550.82	1198.80	0.00	22.16	**1771.77**
16	0.00	0.00	564.96	1208.35	0.00	22.45	**1795.75**
16.5	0.00	0.00	578.76	1216.08	0.00	22.72	**1817.56**
17	0.00	0.00	592.57	1222.30	0.00	22.97	**1837.83**
17.5	0.00	0.00	606.61	1226.63	0.00	23.20	**1856.44**
18	0.00	0.00	621.18	1229.36	0.00	23.42	**1873.96**

TABLE B-6
REACH CL2
ELEV. (NGVD) VS. DAMAGE SUMMARY
(IN THOUSANDS OF DOLLARS)

	Damages ($1,000)						
Stage	Residential	Apartment	Commercial	Industrial	Municipal	Emergency	Total
3	0.00	0.00	0.00	0.00	0.00	0.00	**0.00**
3.5	0.00	0.00	0.00	0.00	0.00	0.00	**0.00**
4	8.14	0.00	0.00	0.00	0.00	0.11	**8.25**
4.5	24.62	0.00	0.00	0.00	0.00	0.74	**25.35**
5	102.44	0.00	0.00	0.00	0.00	1.71	**104.16**
5.5	232.87	0.00	0.00	0.00	0.00	3.64	**236.51**
6	481.95	0.00	2.08	0.00	0.00	7.21	**491.24**
6.5	806.53	0.00	5.79	0.00	0.00	11.31	**823.62**
7	1283.33	1.78	13.19	0.00	0.00	17.39	**1315.69**
7.5	1849.93	5.00	24.31	0.00	0.00	24.65	**1903.89**
8	2617.03	11.46	39.39	0.00	0.00	34.51	**2702.38**
8.5	3515.85	21.18	56.78	0.00	0.00	46.08	**3639.88**
9	4601.82	33.70	78.00	0.00	9.45	60.20	**4783.15**
9.5	5767.60	46.72	103.67	0.00	36.40	75.59	**6029.98**
10	7075.95	60.04	134.31	0.00	90.09	93.17	**7453.56**
10.5	8444.57	71.09	170.81	0.00	165.42	111.81	**8963.70**
11	9879.89	79.34	210.82	0.00	247.69	131.38	**10549.13**
11.5	11340.90	85.54	250.30	0.00	313.95	151.04	**12141.72**
12	12877.08	90.86	286.19	0.00	364.24	171.39	**13789.76**
12.5	14462.03	95.82	317.25	0.00	402.02	192.12	**15469.24**
13	16051.89	100.64	344.03	0.00	434.24	212.79	**17143.59**
13.5	17548.10	105.21	366.95	0.00	464.13	232.21	**18716.59**
14	18917.28	109.43	387.13	0.00	493.14	250.00	**20156.97**
14.5	20151.37	113.16	404.20	0.00	520.14	266.02	**21454.89**
15	21283.20	116.55	418.00	0.00	544.02	280.68	**22642.46**
15.5	22323.85	119.72	427.85	0.00	563.79	294.10	**23729.30**
16	23297.86	122.83	434.75	0.00	580.35	306.61	**24742.39**
16.5	24199.00	125.92	439.93	0.00	595.02	318.16	**25678.03**
17	25030.76	129.10	444.73	0.00	609.18	328.83	**26542.61**
17.5	25785.50	132.67	449.55	0.00	623.22	338.54	**27329.48**
18	26478.00	136.82	454.49	0.00	637.31	347.49	**28054.10**

TABLE B-7
REACH CL3
ELEV. (NGVD) VS. DAMAGE SUMMARY
(IN THOUSANDS OF DOLLARS)

				Damages ($1,000)			
Stage	Residential	Apartment	Commercial	Industrial	Municipal	Emergency	**Total**
3	0.00	0.00	0.00	0.00	0.00	0.00	**0.00**
3.5	0.00	0.00	0.00	0.00	0.00	0.00	**0.00**
4	1.97	0.00	0.00	0.00	0.00	0.02	**1.99**
4.5	5.58	0.00	0.00	0.00	0.00	0.07	**5.65**
5	13.83	0.00	0.00	0.00	0.00	0.45	**14.29**
5.5	25.36	0.00	0.00	0.00	0.00	0.89	**26.26**
6	45.99	0.00	0.00	0.00	0.00	1.16	**47.15**
6.5	74.23	0.00	0.00	0.00	0.00	1.79	**76.02**
7	120.90	0.00	1.97	0.00	0.03	2.40	**125.29**
7.5	183.49	0.00	8.54	0.00	0.14	3.26	**195.43**
8	273.75	0.00	29.72	0.00	0.33	4.66	**308.46**
8.5	379.00	0.00	74.34	0.00	0.60	6.53	**460.48**
9	526.47	0.00	152.23	0.00	0.90	9.36	**688.98**
9.5	704.92	0.00	262.80	0.00	1.15	12.97	**981.84**
10	958.93	0.00	399.74	0.00	1.33	17.86	**1377.86**
10.5	1266.89	0.00	541.55	0.00	1.46	23.49	**1833.39**
11	1696.49	0.00	673.12	0.00	1.58	30.50	**2401.68**
11.5	2210.70	0.00	784.16	0.00	1.69	38.32	**3034.88**
12	2865.68	0.00	877.52	0.00	1.80	47.68	**3792.67**
12.5	3594.46	0.00	957.86	0.00	1.89	57.79	**4612.00**
13	4425.86	0.00	1032.22	0.00	1.99	69.12	**5529.19**
13.5	5288.52	0.00	1100.23	0.00	2.05	80.75	**6471.56**
14	6236.89	0.00	1162.83	0.00	2.11	93.39	**7495.22**
14.5	7228.23	0.00	1218.82	0.00	2.17	106.47	**8555.68**
15	8249.15	0.00	1270.12	0.00	2.22	119.89	**9641.38**
15.5	9219.57	0.00	1317.75	0.00	2.27	132.60	**10672.19**
16	10136.35	0.00	1364.32	0.00	2.32	144.65	**11647.64**
16.5	10980.84	0.00	1408.95	0.00	2.38	155.77	**12547.94**
17	11767.31	0.00	1451.15	0.00	2.43	166.12	**13387.02**
17.5	12484.38	0.00	1490.53	0.00	2.48	175.58	**14152.97**
18	13136.26	0.00	1527.55	0.00	2.55	184.19	**14850.55**

TABLE B-8
REACH CR1
ELEV. (NGVD) VS. DAMAGE SUMMARY
(IN THOUSANDS OF DOLLARS)

	Damages ($1,000)						
Stage	Residential	Apartment	Commercial	Industrial	Municipal	Emergency	**Total**
3	0.00	0.00	0.00	0.00	0.00	0.00	**0.00**
3.5	0.00	0.00	0.00	0.00	0.00	0.00	**0.00**
4	0.00	0.00	0.00	0.00	0.00	0.00	**0.00**
4.5	0.00	0.00	0.00	0.00	0.00	0.00	**0.00**
5	0.65	0.00	0.00	0.00	0.00	0.01	**0.66**
5.5	1.90	0.00	0.00	0.00	0.00	0.31	**2.21**
6	5.11	0.00	0.52	0.00	0.00	0.36	**5.99**
6.5	9.80	0.00	1.96	0.00	0.00	0.43	**12.19**
7	22.27	0.00	7.92	0.00	0.00	0.66	**30.86**
7.5	41.29	0.00	17.65	0.00	0.00	1.02	**59.96**
8	73.76	0.00	37.69	0.00	0.00	1.67	**113.12**
8.5	114.29	0.00	74.53	0.00	0.00	2.64	**191.46**
9	165.86	0.00	143.97	0.00	0.00	4.16	**313.98**
9.5	226.00	0.00	239.56	0.00	0.00	6.10	**471.66**
10	305.44	0.00	339.60	0.00	0.00	8.34	**653.38**
10.5	400.05	0.00	427.84	0.00	0.00	10.64	**838.53**
11	513.99	0.00	496.13	0.00	0.00	12.91	**1023.03**
11.5	631.37	0.00	548.67	0.00	0.00	15.03	**1195.07**
12	760.31	0.00	593.53	0.00	0.00	17.21	**1371.05**
12.5	891.30	0.00	634.48	0.00	0.00	19.36	**1545.14**
13	1031.48	0.00	673.67	0.00	0.00	21.60	**1726.74**
13.5	1173.12	0.00	709.58	0.00	0.00	23.82	**1906.52**
14	1336.48	0.00	740.29	0.00	0.00	26.25	**2103.02**
14.5	1517.14	0.00	764.02	0.00	0.00	28.80	**2309.96**
15	1719.02	0.00	782.56	0.00	0.00	31.55	**2533.13**
15.5	1922.05	0.00	797.99	0.00	0.00	34.28	**2754.33**
16	2119.09	0.00	812.66	0.00	0.00	36.94	**2968.68**
16.5	2304.98	0.00	827.11	0.00	0.00	39.44	**3171.54**
17	2493.55	0.00	841.72	0.00	0.00	41.97	**3377.24**
17.5	2687.07	0.00	856.85	0.00	0.00	44.58	**3588.51**
18	2884.38	0.00	873.13	0.00	0.00	47.26	**3804.77**

TABLE B-9
REACH CR2
ELEV. (NGVD) VS. DAMAGE SUMMARY
(IN THOUSANDS OF DOLLARS)

	Damages ($1,000)						
Stage	Residential	Apartment	Commercial	Industrial	Municipal	Emergency	Total
3	0.00	0.00	0.00	0.00	0.00	0.00	**0.00**
3.5	0.00	0.00	0.00	0.00	0.00	0.00	**0.00**
4	0.00	0.00	0.00	0.00	0.00	0.00	**0.00**
4.5	0.00	0.00	0.00	0.00	0.00	0.00	**0.00**
5	0.00	0.00	0.00	0.00	0.00	0.00	**0.00**
5.5	0.00	0.00	0.00	0.00	0.00	0.00	**0.00**
6	0.00	0.00	0.00	0.00	0.00	0.00	**0.00**
6.5	0.00	0.00	0.00	0.00	0.00	0.00	**0.00**
7	0.00	0.00	0.00	0.00	0.00	0.00	**0.00**
7.5	0.00	0.00	0.00	0.00	0.00	0.00	**0.00**
8	3.70	0.00	0.00	0.00	0.00	0.04	**3.75**
8.5	10.84	0.00	0.00	0.00	0.00	0.14	**10.97**
9	62.52	0.00	0.00	0.00	0.00	0.78	**63.30**
9.5	154.73	0.00	0.00	0.00	0.00	1.94	**156.66**
10	342.74	0.00	4.76	1.43	0.08	4.37	**353.38**
10.5	593.78	0.00	17.11	8.33	0.25	7.74	**627.21**
11	979.86	0.00	53.63	25.10	3.86	13.28	**1075.73**
11.5	1455.51	0.00	120.10	56.82	13.81	20.58	**1666.81**
12	2129.44	0.00	217.88	102.66	33.32	31.04	**2514.34**
12.5	2939.17	0.00	335.66	159.57	60.53	43.69	**3538.62**
13	3941.80	0.00	462.00	220.85	90.26	58.93	**4773.84**
13.5	5023.72	0.00	576.32	281.58	114.32	74.94	**6070.88**
14	6248.62	0.00	679.69	339.09	132.62	92.50	**7492.51**
14.5	7531.69	0.00	776.19	392.92	146.36	110.59	**8957.75**
15	8851.51	0.00	880.61	443.49	158.09	129.18	**10462.88**
15.5	10117.30	0.00	1006.60	489.29	168.97	147.28	**11929.44**
16	11369.86	0.00	1159.70	529.00	179.53	165.48	**13403.57**
16.5	12589.39	0.00	1331.70	560.43	189.36	183.38	**14854.26**
17	13757.03	0.00	1503.59	584.76	198.06	200.54	**16243.98**
17.5	14826.60	0.00	1653.59	603.65	205.26	216.11	**17505.21**
18	15791.53	0.00	1778.63	620.09	211.28	230.02	**18631.55**

TABLE B-10
REACH CR3
ELEV. (NGVD) VS. DAMAGE SUMMARY
(IN THOUSANDS OF DOLLARS)

	Damages ($1,000)						
Stage	**Residential**	**Apartment**	**Commercial**	**Industrial**	**Municipal**	**Emergency**	**Total**
3	0.00	0.00	0.00	0.00	0.00	0.00	**0.00**
3.5	0.00	0.00	0.00	0.00	0.00	0.00	**0.00**
4	0.00	0.00	0.00	0.00	0.00	0.00	**0.00**
4.5	0.00	0.00	0.00	0.00	0.00	0.00	**0.00**
5	0.00	0.00	0.00	0.00	0.00	0.00	**0.00**
5.5	0.00	0.00	0.00	0.00	0.00	0.00	**0.00**
6	0.13	0.00	0.13	0.00	0.00	0.00	**0.25**
6.5	0.68	0.00	0.52	0.00	0.00	0.01	**1.21**
7	10.73	0.00	2.28	0.00	0.00	0.16	**13.17**
7.5	29.96	0.00	5.73	0.00	0.00	0.44	**36.14**
8	71.88	0.00	12.37	0.00	1.71	1.07	**87.04**
8.5	129.04	0.00	21.12	0.00	4.76	1.94	**156.85**
9	226.32	0.00	38.06	0.00	11.48	3.45	**279.31**
9.5	352.94	0.00	65.25	0.00	23.75	5.52	**447.46**
10	537.15	0.00	110.62	0.00	42.35	8.63	**698.75**
10.5	761.76	0.00	168.60	0.00	65.39	12.45	**1008.21**
11	1044.18	0.00	235.49	0.00	90.88	17.14	**1387.69**
11.5	1351.21	0.00	308.70	0.00	113.51	22.17	**1795.59**
12	1713.43	0.00	401.18	0.00	131.18	28.07	**2273.87**
12.5	2105.24	0.00	518.03	0.00	144.79	34.60	**2802.65**
13	2509.03	0.00	649.99	0.00	156.39	41.44	**3356.85**
13.5	2894.94	0.00	779.78	0.00	167.15	48.02	**3889.90**
14	3283.33	0.00	895.60	0.00	177.60	54.45	**4410.98**
14.5	3669.90	0.00	995.29	0.00	187.32	60.66	**4913.17**
15	4043.99	0.00	1083.12	0.00	195.92	66.54	**5389.57**
15.5	4384.02	0.00	1162.62	0.00	203.05	71.87	**5821.56**
16	4682.48	0.00	1236.59	0.00	209.00	76.60	**6204.67**
16.5	4938.08	0.00	1301.81	0.00	214.29	80.68	**6534.86**
17	5161.00	0.00	1355.53	0.00	219.38	84.20	**6820.12**
17.5	5357.11	0.00	1394.68	0.00	224.44	87.20	**7063.43**
18	5535.51	0.00	1422.66	0.00	229.53	89.85	**7277.55**

TABLE B-11
REACH PR1
ELEV. (NGVD) VS. DAMAGE SUMMARY
(IN THOUSANDS OF DOLLARS)

	Damages ($1,000)						
Stage	Residential	Apartment	Commercial	Industrial	Municipal	Emergency	Total
3	0.00	0.00	0.00	0.00	0.00	0.00	**0.00**
3.5	0.00	0.00	0.00	0.00	0.00	0.00	**0.00**
4	0.00	0.00	0.00	0.00	0.00	0.00	**0.00**
4.5	0.00	0.00	0.00	0.00	0.00	0.00	**0.00**
5	2.68	0.00	0.00	0.00	0.00	0.03	**2.71**
5.5	8.84	0.00	0.00	0.00	0.00	0.40	**9.24**
6	21.09	0.00	2.66	0.00	0.00	0.58	**24.33**
6.5	37.18	0.00	10.13	0.00	0.00	0.87	**48.18**
7	58.30	0.00	25.08	0.00	0.00	1.33	**84.71**
7.5	80.09	0.00	46.09	0.00	0.00	1.86	**128.04**
8	112.89	0.00	69.25	0.00	0.00	2.56	**184.70**
8.5	153.30	0.00	88.44	0.00	0.00	3.30	**245.04**
9	203.47	0.00	103.42	0.00	0.00	4.12	**311.01**
9.5	251.89	0.00	114.52	0.00	0.00	4.86	**371.27**
10	299.17	0.00	123.56	0.00	0.00	5.57	**428.29**
10.5	344.15	0.00	131.49	0.00	0.00	6.23	**481.87**
11	397.24	0.00	139.66	0.00	0.00	7.00	**543.89**
11.5	459.48	0.00	148.77	0.00	0.00	7.89	**616.15**
12	525.33	0.00	170.26	0.00	0.00	8.98	**704.57**
12.5	584.70	0.00	217.83	0.00	0.00	10.32	**812.85**
13	634.20	0.00	306.94	0.00	0.00	12.05	**953.19**
13.5	676.31	0.00	441.59	0.00	0.00	14.25	**1132.15**
14	716.39	0.00	606.48	0.00	0.00	16.82	**1339.69**
14.5	759.38	0.00	772.34	0.00	0.00	19.43	**1551.14**
15	807.32	0.00	923.04	0.00	0.00	21.91	**1752.26**
15.5	858.54	0.00	1049.82	0.00	0.00	24.14	**1932.50**
16	908.66	0.00	1157.78	0.00	0.00	26.11	**2092.55**
16.5	953.00	0.00	1253.30	0.00	0.00	27.87	**2234.16**
17	991.09	0.00	1342.74	0.00	0.00	29.46	**2363.29**
17.5	1025.01	0.00	1423.60	0.00	0.00	30.89	**2479.49**
18	1056.67	0.00	1493.48	0.00	0.00	32.16	**2582.31**

TABLE B-12
REACH PR2
ELEV. (NGVD) VS. DAMAGE SUMMARY
(IN THOUSANDS OF DOLLARS)

	Damages ($1,000)						
Stage	Residential	Apartment	Commercial	Industrial	Municipal	Emergency	Total
3	0.00	0.00	0.00	0.00	0.00	0.00	**0.00**
3.5	0.00	0.00	0.00	0.00	0.00	0.00	**0.00**
4	1.59	0.00	0.00	0.00	0.00	0.02	**1.59**
4.5	4.61	0.00	0.00	0.00	0.00	0.05	**4.61**
5	10.16	0.00	0.00	0.00	0.00	0.41	**10.16**
5.5	16.80	0.00	0.00	0.00	0.00	0.49	**16.80**
6	31.62	0.00	0.00	0.00	0.00	0.83	**31.62**
6.5	53.96	0.00	0.00	0.00	0.00	1.10	**53.96**
7	117.67	0.00	0.08	0.00	0.00	1.90	**117.75**
7.5	216.74	0.00	0.33	0.00	0.00	3.15	**217.07**
8	398.21	0.00	0.81	0.00	0.00	5.42	**399.02**
8.5	633.74	0.00	1.49	0.00	0.00	8.37	**635.23**
9	989.28	0.00	2.23	0.00	0.00	12.82	**991.51**
9.5	1427.61	0.00	2.83	0.00	0.00	18.32	**1430.44**
10	2024.84	14.33	4.58	0.00	0.00	25.97	**2043.74**
10.5	2723.98	40.25	9.23	0.00	0.00	35.09	**2773.45**
11	3567.43	101.74	18.52	0.00	1.14	46.54	**3688.82**
11.5	4466.83	197.24	32.50	0.00	3.17	59.18	**4699.74**
12	5461.18	339.24	54.35	0.00	9.10	73.73	**5863.87**
12.5	6487.73	508.06	78.99	0.00	19.89	89.11	**7094.68**
13	7570.11	709.60	113.36	0.00	39.32	105.83	**8432.39**
13.5	8651.21	910.56	159.34	0.00	67.56	122.79	**9788.66**
14	9756.53	1110.12	224.14	0.00	105.80	140.39	**11196.59**
14.5	10844.75	1291.90	310.39	0.00	150.54	157.91	**12597.58**
15	11929.73	1463.83	421.42	0.00	200.89	175.63	**14015.87**
15.5	12968.89	1620.61	547.67	0.00	251.66	192.79	**15388.82**
16	13959.78	1766.83	677.36	0.00	301.36	209.24	**16705.33**
16.5	14879.28	1898.23	798.44	0.00	346.24	224.45	**17922.19**
17	15754.55	2017.98	905.08	0.00	384.52	238.71	**19062.12**
17.5	16589.91	2123.24	997.21	0.00	415.41	252.01	**20125.78**
18	17383.45	2216.28	1079.16	0.00	442.00	264.44	**21120.89**

TABLE B-13
REACH PR3
ELEV. (NGVD) VS. DAMAGE SUMMARY
(IN THOUSANDS OF DOLLARS)

	Damages ($1,000)						
Stage	Residential	Apartment	Commercial	Industrial	Municipal	Emergency	**Total**
3	0.00	0.00	0.00	0.00	0.00	0.00	**0.00**
3.5	0.00	0.00	0.00	0.00	0.00	0.00	**0.00**
4	0.00	0.00	0.00	0.00	0.00	0.00	**0.00**
4.5	0.00	0.00	0.00	0.00	0.00	0.00	**0.00**
5	0.00	0.00	0.00	0.00	0.00	0.00	**0.00**
5.5	0.00	0.00	0.00	0.00	0.00	0.28	**0.28**
6	8.10	0.00	0.00	0.00	0.00	0.39	**8.49**
6.5	22.59	0.00	0.00	0.00	0.00	0.57	**23.15**
7	82.14	0.00	0.00	0.00	0.00	1.32	**83.46**
7.5	183.48	0.00	0.00	0.00	0.00	2.58	**186.06**
8	377.18	0.00	1.98	0.00	0.00	5.03	**384.19**
8.5	631.39	0.00	5.48	0.00	0.00	8.25	**645.12**
9	972.48	0.00	13.27	0.00	0.00	12.60	**998.35**
9.5	1354.38	0.00	27.56	0.00	0.00	17.56	**1399.50**
10	1833.15	0.00	50.16	0.00	0.00	23.83	**1907.14**
10.5	2369.36	0.00	80.30	0.00	0.00	30.91	**2480.56**
11	2998.62	0.00	116.21	0.00	0.00	39.22	**3154.05**
11.5	3653.33	0.00	152.14	0.00	0.00	47.86	**3853.33**
12	4331.98	0.00	192.40	0.00	0.00	56.84	**4581.22**
12.5	4986.06	0.00	238.63	0.00	0.00	65.59	**5290.28**
13	5647.68	0.00	306.95	0.00	0.00	74.71	**6029.35**
13.5	6297.07	0.00	396.05	0.00	0.00	83.95	**6777.07**
14	6947.24	0.00	513.90	0.00	0.00	93.54	**7554.69**
14.5	7564.37	0.00	665.95	0.00	0.00	103.16	**8333.48**
15	8139.11	0.00	863.67	0.00	0.00	112.82	**9115.60**
15.5	8657.67	0.00	1100.22	0.00	0.00	122.25	**9880.14**
16	9141.63	0.00	1356.35	0.00	0.00	131.51	**10629.49**
16.5	9605.68	0.00	1597.01	0.00	0.00	140.32	**11343.00**
17	10050.79	0.00	1803.71	0.00	0.00	148.47	**12002.96**
17.5	10451.58	0.00	1976.77	0.00	0.00	155.64	**12583.99**
18	10804.49	0.00	2128.29	0.00	0.00	161.94	**13094.73**

AVERAGE ANNUAL DAMAGES

General

B47. The stage vs. damage data were combined with the stage vs. frequency data using the Hydrologic Engineering Center's Next Generation Flood Damage Analysis computer program (NexGen HEC-FDA). The NexGen HEC-FDA program quantifies uncertainty in discharge-frequency, stage-discharge, and stage-damage functions and incorporates it into economic and performance analyses of alternatives. The process applies a procedure (Monte Carlo Simulation) that computes the expected value of damage while accounting for uncertainty in the basic value.

B48. The HEC-FDA program presents results for expected annual damages and equivalent annual damages. The impacts of sea level rise were incorporated by increasing the end of project stages in the stage vs. frequency curve by the projected rate of sea level rise, approximately 0.7 feet. The stage frequency curve without sea level rise is shown in Figure B-13.

Uncertainty

B49. As previously stated, risk and uncertainty must be incorporated in flood damage reduction studies. The following areas of uncertainty were incorporated into the HEC-FDA program:

- stage frequency
- first floor stage
- structure value
- content-to-structure value ratio
- other -to-structure value ratio

B50. Uncertainty in stage frequency was initially determined by the Waterways Experiment Station (WES) using Empirical Simulation Techniques (EST). The HEC-FDA program, however, requires that uncertainty be calculated using order statistics and equivalent record lengths. An equivalent record length of 150 years was selected using trial and error procedures in order to best replicate the standard deviations provided by WES. Table B-14 presents a summary of the impact of equivalent record length on stage frequency uncertainty.

TABLE B-14
UNCERTAINTY ON STAGE FREQUENCY

Storm	Modif. EST Stage	EST Standard Deviation	Standard Deviations for Various Order Statistic Equivalent Record Lengths*					
			9 yr	15 yr	50 yr	100 yr	150 yr	200 yr
2 yr	6.5	0	0.46	0.37	0.2	0.14	0.11	0.1
5 yr	7.6	0.15	0.56	0.55	0.31	0.22	0.18	0.16
10 yr	8.4	0.18	0.64	0.64	0.53	0.36	0.29	0.24
25 yr	9.6	1.32	0.76	0.79	0.73	0.76	0.68	0.54
50 yr	10.5	1.57	0.84	0.9	0.84	0.88	1.12	1.08
100 yr	12.2	2.4	1.01	1.11	1.07	1.11	1.43	1.38
250 yr	15.5	4.7	1.32	1.52	1.49	1.57	2.04	1.96

* Standard deviations determined using HEC-FDA graphical stage frequency procedure.

Damages

B51. Average annual damages for base year conditions are presented in Table B-15, while future year 2050 damages are shown in Table B-16. Equivalent annual damages over the projects 50-year period at a 6-5/8% discount rate are shown in Table B-17. The summary HEC-FDA outputs are located in Sub-Appendix B1.

Public Emergency Costs

B52. The cost of providing additional public services during storms was analyzed based on data provided by Middletown Township.

TABLE B-15
SUMMARY OF EXISTING CONDITION/BASE YEAR ANNUAL DAMAGE BY DAMAGE CATEGORIES & DAMAGE REACHES

Damage Reach	Damage Categories						Total
	Residential	Apartment	Commercial	Industrial	Municipal	Public Emergency	
PR1	$71,150	$0	$36,960	$0	$0	$1,500	$109,610
PR2	$365,660	$17,620	$6,110	$0	$1,670	$4,800	$395,870
PR3	$288,790	$0	$14,860	$0	$0	$3,670	$307,330
PR4	$406,090	$0	$12,220	$0	$0	$0	$418,320
Subtotal Pews Creek	$1,131,700	$17,620	$70,150	$0	$1,670	$9,970	$1,231,120
CR1	$59,960	$0	$37,140	$0	$0	$1,330	$98,440
CR2	$124,150	$0	$12,210	$5,070	$1,750	$1,590	$144,780
CR3	$92,790	$0	$21,410	$0	$5,010	$1,350	$120,560
CL1	$0	$0	$80,870	$254,990	$0	$4,150	$340,020
CL2	$1,599,740	$7,250	$23,560	$0	$11,630	$20,190	$1,662,360
CL3	$246,900	$0	$42,740	$0	$170	$3,980	$293,790
Subtotal Compton Creek	$2,123,550	$7,250	$217,950	$260,060	$18,560	$32,590	$2,659,950
TOTAL	**$3,255,000**	**$24,870**	**$288,100**	**$260,060**	**$20,230**	**$42,560**	**$3,891,080**

TABLE B-16
SUMMARY OF FUTURE CONDITION (YEAR 2050) ANNUAL DAMAGE BY DAMAGE CATEGORIES & DAMAGE REACHES

	Damage Categories						
Damage Reach	**Residential**	**Apartment**	**Commercial**	**Industrial**	**Municipal**	**Public Emergency**	**Total**
PR1	$104,530	$0	$56,160	$0	$0	$2,140	**$162,810**
PR2	$564,740	$23,480	$8,110	$0	$2,140	$7,210	**$605,690**
PR3	$465,890	$0	$20,880	$0	$0	$5,810	**$492,570**
PR4	$608,010	$0	$18,090	$0	$0	$0	**$626,100**
Subtotal Pews Creek	$1,743,170	$23,480	$103,220	$0	$2,140	$15,160	**$1,887,180**
CR1	$92,570	$0	$62,330	$0	$0	$2,010	**$156,910**
CR2	$171,560	$0	$16,190	$6,710	$2,300	$2,170	**$198,930**
CR3	$144,190	$0	$32,280	$0	$7,990	$2,090	**$186,540**
CL1	$0	$0	$107,070	$357,400	$0	$5,730	**$470,140**
CL2	$2,376,610	$12,390	$37,670	$0	$17,730	$29,890	**$2,474,290**
CL3	$360,450	$0	$70,920	$0	$300	$5,700	**$437,370**
Subtotal Compton Creek	$3,145,380	$12,390	$326,450	$364,100	$28,310	$47,600	**$3,924,230**
TOTAL	**$4,888,550**	**$35,870**	**$429,670**	**$364,100**	**$30,450**	**$62,760**	**$5,811,410**

TABLE B-17
SUMMARY OF EQUIVALENT ANNUAL DAMAGE
(50-YEAR PERIOD, 6-5/8% DISCOUNT RATE)
BY DAMAGE CATEGORIES & DAMAGE REACHES

Damage Reach	Damage Categories						Total
	Residential	Apartment	Commercial	Industrial	Municipal	Public Emergency	
PR1	$80,180	$0	$42,150	$0	$0	$1,670	**$124,000**
PR2	$419,510	$19,200	$6,650	$0	$1,800	$5,450	**$452,620**
PR3	$336,690	$0	$16,490	$0	$0	$4,250	**$357,430**
PR4	$460,710	$0	$13,810	$0	$0	$0	**$474,520**
Subtotal Pews Creek	$1,297,090	$19,200	$79,100	$0	$1,800	$11,380	**$1,408,570**
CR1	$68,780	$0	$43,960	$0	$0	$1,520	**$114,250**
CR2	$136,980	$0	$13,290	$5,510	$1,900	$1,750	**$159,430**
CR3	$106,700	$0	$24,350	$0	$5,810	$1,550	**$138,410**
CL1	$0	$0	$87,960	$282,690	$0	$4,580	**$375,230**
CL2	$1,809,870	$8,640	$27,380	$0	$13,280	$22,810	**$1,881,970**
CL3	$277,610	$0	$50,370	$0	$200	$4,440	**$332,630**
Subtotal Compton Creek	$2,399,940	$8,640	$247,290	$288,200	$21,200	$36,650	**$3,001,910**
TOTAL	**$3,697,030**	**$27,840**	**$326,390**	**$288,200**	**$23,000**	**$48,030**	**$4,410,480**

STORM DAMAGE REDUCTION BENEFITS

Methodology and Assumptions

B53. Benefits from the proposed plan of improvement were estimated by comparing damages with and without the proposed project under existing and future conditions.

Limits of Local Protection

B54. Although flood damages were calculated for ten (10) reaches representing the maximum area considered for protection, the preferred plan provides flood protection for only the five reaches with the most significant annual damages. The benefits presented in this section reflect the selected alternative which provides local flood protection of reaches PR2, PR3 and PR4 (Pews Creek, Right Bank, Reach 2, 3 & 4) and CL2 and CL3 (Compton Creek, Left Bank, Reach 2 & 3).

Storm Damage With Plans

B55. Residual damage from storm surges overtopping the levee/floodwall and dune line of protection was calculated for each plan using the same data and procedures as the without project condition. For each reach the appropriate levee elevation was used to determine whether storm surges would impact the protected area.

Residual Interior Damage

B56. In addition to potential damage from storm surges overtopping the levees and floodwalls, runoff from rainfall in the interior of the protected area may also cause damages. Interior stage vs. frequency data was analyzed for three locations on Compton Creek and for Pews Creek. Interior flood protection alternatives were formulated independent from the line of protection as described in Appendix F, Interior Drainage. A variety of interior facilities at each location were evaluated for hydrologic and economic impacts. The economic assessments utilized the same data and analysis procedures previously described. The smallest, or minimum, facility analyzed at each location consists of gravity outlets and natural pond. The costs and residual damages for the selected interior features are presented in Table B-18. Details of the interior drainage plan formulation are presented in Appendix F, Interior Drainage.

Reduced Flood Insurance Administrative Costs

B57. The Township of Middletown participates in the regular program of Flood Insurance. Field investigations have identified 862 structures below the regulatory Base Flood Elevation (BFE) which would be protected by the selected alternative. As a result of project implementation, these structures would no longer be required to maintain Flood Insurance Policies as a condition of their federally backed mortgages. Therefore, based on the current administrative overhead cost of $121 per policy, an annual savings of $104,300 is anticipated.

Summary of Flood Damage Reduction Benefits

B58. Flood damage reduction benefits were calculated based on comparison of annual damages under the with- and without-project conditions. Annual benefits for various design levels are presented in Table B-19.

REDUCED MAINTENANCE BENEFITS

B59. As stated under the without project future conditions, it is anticipated that in the absence of a Federal Project, the Township of Middletown or Monmouth County will continue to conduct periodic beach nourishment operations along the Port Monmouth shoreline. These operations have previously utilized material dredged from the Pews Creek navigation channel to replenish areas of significant dune erosion. The dredging of Pews Creek is normally performed on an ongoing as needed basis, with materials stockpiled near the mouth of the creek. This material has since been trucked and placed as for the construction of Port Monmouth Road. With the changes to structures at the inlet, it is expected that suitable nourishment material will now have to be purchased and trucked to the site. Based on the estimated long term erosion rate of five thousand cubic yards per year and a unit cost of $15 per cubic yard to truck and spread the sand, the average annual maintenance costs are estimated at $75,000. Since the selected alternative incorporates future periodic nourishment as a design feature, the current dune maintenance operations will not be required, providing an annual benefit of $75,000.

TABLE B-18
SUMMARY OF SELECTED INTERIOR FEATURE COSTS & BENEFITS
(1998 PRICE LEVELS)

Creek		Interior Drainage Facility	Total Annual Cost*	Equivalent Annual Damage	Annual Damage Reduction	Incremental Cost Above Minimum Facility	Annual Net Benefit
Pews Creek		120 cfs Pump Station	$182,650	$47,750	$271,400	$166,350	$105,050
Compton Creek	C1	Minimum Facility	$14,800	$8,800	$0	$0	$0
	C2	Minimum Facility	$31,800	$14,980	$0	$0	$0
	C3	60 cfs Pump Station	$179,860	$20,210	$145,330	$126,560	$18,770

*Pump station costs include minimum facility cost.

B60. Project construction will increase sediment transport resulting in increased dredging rates for Compton and Pews Creeks. The estimated annual NED cost for the dredging has been estimated and is included in the project O&M costs. The material dredged from Pews Creek is scheduled to be placed at Keansburg to reduce the maintenance of the Keansburg dune. Since this material will be used in place of trucked material, an additional savings of $70,000 will result. A combined benefit of $145,000 ($70,000 + $75,000) will result from the reduced maintenance within the project area and the increased availability of dredged material for maintenance of the Keansburg dune and beach.

TABLE B-19
FLOOD DAMAGE REDUCTION BENEFITS ADJUSTED FOR INTERIOR FLOODING
(1998 PRICE LEVEL, 50-YEAR PERIOD, INTEREST RATE - 6-5/8%, IN THOUSANDS)

Line of Protection Design Elevation	13 FT. NGVD	14 FT. NGVD	15.2 FT. NGVD
Residential	$2,762,020	$2,871,620	$2,972,320
Non-residential	$83,730	$91,540.	$99,040
Subtotal	$2,845,750	$2,963,160	$3,071,360
Reduced Public Emergency Costs	$30,660	$31,900	$33,040
Reduced Flood Insurance Administrative Costs	$0	$0	$104,300
Total Flood Damage Reduction Benefits	$2,876,420	$2,995,070	$3,208,700
Minus Residual Interior			
Compton Creek	$43,990	$43,990	$43,990
Pews Creek	$47,750	$47,750	$47,750
Net Damage Reduction	$2,784,680	$2,903,330	$3,116,960

RECREATION

General Description

B61. The considered shorefront improvements would improve the recreational elements of the Port Monmouth area by restoring the eroded, inadequate beach by adding fill and stabilizing the dunes. The wider beach will be better able to accommodate the general public visiting the beach, and along with the stable, uniform dune system will enhance the esthetic quality of the area. In addition, the County Parks Department has indicated a desire to utilize the levee easements to connect the waterfront park to the Henry Hudson Trail, a bayshore region hiking trail located about a mile inland from the Raritan Bay waterfront. With two of the greatest attractions to the bayshore region being recreational boating and fishing, current plans are not anticipated to greatly increase or decrease the performance of these activities.

Without Project Conditions

B62. Based on the pace of ongoing property acquisitions, it is anticipated that the County will complete the purchase of privately owned structures and create a regional county park providing adequate public facilities prior to the base year of the proposed project. Presently, beach access is provided at three parking areas along the shore.

With Project Conditions

B63. With the implementation of the project, the beach will act in conjunction with the County Park to provide an attractive recreational environment. The project includes widening and enhancing the beach and the dune line. The County has also indicated their intent to expand the existing beach parking facilities and the desire to connect the beach facilities to the Henry Hudson Trail via the levee system.

Beach Attendance

B64. Since current beach attendance data is not readily available and is not considered a reliable indicator of future without project attendance, data from nearby areas was used to aid in the formulation of Port Monmouth beach attendance. Using information from site inspections and data from previous surveys performed from Sandy Hook to Manasquan Inlet, an estimated beach attendance was developed by approximating the number of area residents and visitors, and the frequency of attendance.

B65. Based on field surveys of structures, there are approximately five hundred residences within reasonable walking distance to the beach, with an estimated three residents per household. Review of the extensive Contingent Valuation Method (CVM) surveys conducted at the nearby beaches from Sandy Hook to Manasquan Inlet indicates that the typical local beach user visits the beach sixteen to twenty-six times a year. For this analysis it is assumed that the immediate Port Monmouth populace will frequent the beach twenty times a year (estimate corrected for trip bias). Therefore, on a yearly average the local residents account for 30,000 (500 homes x 3 people/home x 20 visits/person = 30,000 visits) beach visits.

B66. In addition to the local residents, the Port Monmouth beach area provides a public recreation resource within reasonable travel of millions of potential users. Attendance by day visitors was estimated based on the number of parking spaces available, the turnover rate of the spaces, and length of recreational season. Discussions with the County recreation department indicate that they plan on providing approximately 200 parking spaces at three newly constructed parking lots. Based on the previous studies, it was approximated that each space is occupied two times per day with four people per car during peak days (weekends and holidays). Off-peak days are estimated at fifty percent of peak days. Thus the visiting public is estimated as follows:

30 peak days x 2 car/space/day x 4 people/car x 200 spaces = 48,000

60 off-peak x 2 car/space/day x 4 people/car x 200 spaces x 50%= <u>48,000</u>

96,000

Therefore, there are approximately 96,000 visits to the beach from non-local residents.

B67. Combining the area residents and the visiting public, it is estimated that the without-project annual use of the county beach is 126,000 visits to the beach. By comparison the Monmouth County beach park attendance for Seven President's Park was 177,449 visits in 1985. Therefore, the estimated 126,000 visits to a county park at Port Monmouth is considered reasonable.

B68. Although the with-project condition will improve the recreational experience and enhance the visual aspects of the area, there is no evidence to support any significant increase in attendance.

In addition, it is not anticipated that the improvements will draw attendance from other nearby beaches. However, the with-project condition will influence the general recreation experience, improving the quality and value.

Unit Day Value

B69. The Unit Day Value (UDV) method has been used to determine a selected value under a with- and without-project scenario. This value was then applied to the annual use and the difference between the with- and without-project condition results in an estimate of the recreation benefits. For the purpose of this study only the general recreation activities are impacted by the implementation of the project. With the use of guidelines established in the ER 1105-2-100, December 28, 1990 (Revised 30 September 1997), points were assigned to various criteria under a with- and without-project condition. These points were then converted to dollar values which were applied to the attendance data described above.

B70. The general recreation points were determined under a with- (Table B-21) and without-project (Table B-20) condition. The potential to link the storefront areas to the Henry Hudson Trail via the levee easements will provide a significant increase in opportunities for hiking and biking and significantly improve accessibility to the trail system. The completion of the project will also influence the carrying capacity and the environmental criteria (See Tables B-20 & B-21). Points were assigned to each criteria and summed. The total points for the with- and without-project conditions are 38 and 29, respectively.

B71. Once the total points for the with- and without-project condition were determined, the points were converted into dollar values in accordance with standard tables of general recreational values. The dollar values calculated for the with- and without-project condition are $5.33 and $3.81.

TABLE B-20
WITHOUT PROJECT UNIT DAY VALUE (UDV) ANALYSIS

Criteria *	Factors Affecting Point Values	Points Assigned
(a) Recreational experience Maximum Points: 30	The planned park development in the Port Monmouth Area champions several general activities such as saltwater swimming, picnicking, sunbathing, boating, fishing, nature interpretation, wetlands preservation, and educational programs. In addition to the numerous water dependant activities there is a relatively unique and high quality cultural experience available. Listed on the State and National Registers of Historic Places since 1974 is the Seabrook House, more commonly known as the Spy House. This early colonial structure functions as a museum bringing New Jersey history to its community and visitors. In association with the numerous articles contained within the museum itself, the grounds of the museum feature an assortment of historical objects relating to the proximity to the New York Harbor.	13
(b) Availability of opportunity Maximum Points: 18	With many local waterfront parks or beaches along the Bayshore and numerous nearby ocean beaches, availability of opportunity is moderately high. However, each community boasts its own unique resources and style. This site offers gentle offshore slopes and calm waters for recreational swimming.	2
(c) Carrying Capacity Maximum Points: 14	The waterfront park and eroded public beach provide moderate space for the general public as well as an opportunity for water-oriented activities such as swimming, boating, fishing and other waterfront activities.	3
(d) Accessibility Maximum Points: 18	Access via Route 36, the only road that connects all of the Bayshore communities, is often impeded by traffic congestion resulting from the high volume of visitors to Sandy Hook. The roads within the site are in good condition with recent improvements to Port Monmouth road improving the site. There are limited parking facilities in the area providing access to the Henry Hudson Trail.	6
(e) Environmental Maximum Points: 20	Due to the nature of waterfront parks and beaches in general, the esthetics quality is reasonably good. Although the moderate erosion of the beach limits the esthetics value, several scenic resources are available including spacious wetlands, various wildlife, open areas, and a view of the Manhattan Skyline. Although perception of the water quality tends to be negative due to the proximity of NYC and Raritan River drainage, bay waters generally exceed the standards required for swimming.	5
Total Points Calculated UDV	-	29 $3.81

* Criteria and point values are determined as ER 1105-2-100, Table 6-29; Guidelines for Assigning Points for General Recreation.

TABLE B-21
WITH PROJECT UNIT DAY VALUE (UDV) ANALYSIS

Criteria	Factors Affecting Point Values	Points Assigned
(a) Recreational experience Maximum Points: 30	The project will provide hiking trails along the levees, linking the beach area to the Henry Hudson Trail. This will provide additional activities including hiking, jogging and birdwatching from elevated levee areas.	18
(b) Availability of opportunity Maximum Points: 18	No significant change in access to the beach with implementation of project. Linking the project to the Henry Hudson Trail will significantly improve access to the trail system.	2
(c) Carrying Capacity Maximum Points: 14	With the implementation of the project and an addition of more public beach, the waterfront park and public beach will provide ample space for the general public for picnicking and sunbathing as well as extensive opportunity for water-oriented activities such as swimming, boating, fishing and other waterfront activities. Due to the gradual slopes and gentle waters, the beaches will be conducive to family outings.	8
(d) Accessibility Maximum Points: 18	No significant change in access to the beach with implementation of project. Linking the project to the Henry Hudson Trail will significantly improve access to the trail system.	10
(e) Environmental Maximum Points: 20	The implementation of the project will enhance the esthetics value of the waterfront area. The exposed old wooden groins will be buried in the fill and the dunes will be increased which will create a more natural environment. The waterfront area sustains numerous scenic resources including wide beaches with vegetated, protective dunes, spacious wetlands, various wildlife, open areas, and a view of the Manhattan Skyline. Although perception of the water quality tends to be negative due to the proximity of New York City and Raritan River drainage, bay waters generally exceed the standards required for swimming.	10
Total Points Calculated UDV		48 $5.33

Benefits

B72. The dollar values determined in the UDV analysis are applied to the annual use data under the with and without-project scenario. The difference between the two estimated recreation values results in the recreation benefits for the project. Since the with- and without-project attendance are equal, all benefits are derived from the increased value per visit. Under the without project condition, the recreation value is 126,000 visits at $3.81 per visit, or $480,060. Under the with-project condition, the recreation experience is enhanced to $5.33 per visit, which results in an annual recreation value of $671,580. Therefore, the recreation benefits resulting from implementation of the project are approximately $191,500, the difference between the with- and without-project conditions.

SUMMARY OF BENEFITS

B73. The total benefits for the selected alternative are presented in Table B-22.

TABLE B-22
SUMMARY OF BENEFITS
(1998 PRICE LEVEL, 50-YEAR PERIOD, INTEREST RATE 6-5/8%)

Line of Protection Design Elevation	13 FT. NGVD	14 FT. NGVD	15.2 FT. NGVD
Net Damage Reduction Benefits*	$2,784,680	$2,903,330	$3,116,960
Reduced Maintenance	$145,000	$145,000	$145,000
Recreation	$191,500	$191,500	$191,500
Total Benefits	$3,121,180	$3,239,830	$3,453,460

*Excludes residual interior damage.

IDENTIFICATION OF THE NED PLAN

Description of the Plan Alignment

B74. General. The preferred storm damage protection system is comprised of levees, floodwalls, seawalls, relocated dunes, storm gates and pump stations. The alignment will span from State Highway 36 to Sandy Hook Bay then east along the shoreline to Port Monmouth Road and tie into the existing Keansburg levee by way of a storm gate. The preferred plan alignment is shown graphically in the main text.

Interior Drainage Facility Optimization

Formulation Overview

B75. The determination of interior facilities is being conducted using guidance from EM 1110-2-1413, dated January 15, 1987. The strategy outlined under this guidance follows the premise that interior facilities will be planned and evaluated separately from the line of protection, and should provide adequate drainage at least equal to that of the existing infrastructure. This initial plan represents the minimum interior facilities required to implement the line of protection plan. In order to minimize the environmental impact of these facilities, the outlet pipes discharge to existing drainage ditches where possible. Three primary and eleven secondary outlet pipes are required for minimum facility for Compton Creek. Two 48-inch diameter outlet pipes are required for Pews Creek.

B76. The minimum facility plan is the starting point against which additional interior facilities are compared. The benefits accrued from other alternative plans are attributable to the reduction in the residual flooding and damages which would have remained under the minimum facility condition. For an alternative facility to be justified and become a component of the NED plan, it must be implementable and reasonably maximize benefits vs. the additional cost required for its construction, operation, and maintenance. Plan alternatives to be examined include the use of excavated ponds, pump stations and the use of pump stations in conjunction with ponds. The following is a brief summary of the interior drainage plans for Pews Creek and Compton Creek.

Pews Creek Interior Drainage

B77. General. The Line of Protection works include the construction of a storm gate across Pews Creek, about 300 feet upstream of the recently completed New Port Monmouth Road bridge.

B78. Minimum Facility. Minimum facility consists of two 48-inch diameter pipes through the floodwall located between the storm gate and existing Keansburg Levee just west of Pews Creek. The diversion channel constructed during installation of the storm gate will be utilized as the inlet and outlet channel for the pipes. Each pipe will be equipped with a flap gate. No ditch is provided along the levee toe as it directly abuts the marshes of Pews Creek. Thus, there are no secondary outlets. The minimum facilities are described below:

B79. Additional Facilities Considered. Further analyses investigated the use of additional facilities in addition to the minimum facility. No ponding alternatives were considered since the extensive low-lying wetlands area along Pews Creek behind the line of protection offer significant storage capacity. Pump stations, however, were considered as a means of displacing accumulated surface runoff from the interior watershed.

B80. Pump station sizes of 60, 100, 120, 150 and 180 cfs were evaluated. The most cost-effective interior facility at this location was identified as a pump station with a total capacity of 120 cfs. The annual interior damage reduced (NED benefits) for this alternative exceeded the annual cost by approximately $105,000.

Compton Creek Interior Drainage

Area C1

B81. Interior drainage area C1 is located along the left (west) bank of Compton Creek from south of Route 36 area near Chestnut Street to the north between Campbell and Collins Avenues. The area extends west beyond Wilson Avenue to Main Street in the New Street area. The interior drainage area of C1 is comprised of 47.65 acres of developed urban land, with minimal wetlands. The lowest buildings are located at elevation 7 ft. NGVD while Willow Street may start to flood at elevation 6.5 ft. NGVD. The Minimum Facility consisting of natural storage and gravity outlets was selected as the most cost-effective alternative.

Area C2

B82. Interior drainage area C2 is located along the left (west) bank of Compton Creek from sub-basin area C1 extending north just beyond Broadway. A segment of proposed levee and Wilson Avenue form the east and west boundaries of the interior drainage area. The interior drainage area of C2 totals 50.84 acres of predominantly residential development with limited wetlands areas. The lowest buildings are located at elevation 7 ft. NGVD while flooding of Creek Road and Main Street will start at 4.7 and 5.7 ft. NGVD, respectively. Again, the Minimum facility was selected as the most cost-effective.

Area C3

B83. The C3 interior drainage area is also located on the left (west) bank of Compton Creek. Main Street and Wilson Avenue from the east and west boundaries of the area with the dune/berm forming the north boundary and C2 (just south of Lydia Place) forming the south boundary. The interior drainage area C3 is comprised of 78.74 acres, the majority of which is residential. The area near Monmouth Avenue is subject to some of the most frequent flooding in the area. Street elevations in this area are as low as 4.4 ft. NGVD. The lowest buildings are located at elevation 5 ft. NGVD. A 60-cfs pump station was identified as the most cost-effective method to eliminate this frequent flooding.

Line of Protection Optimization

B84. The selected line of protection alignment was evaluated at different design levels to establish the optimum NED Plan. In general, the alignments for each design level are similar except that the highest level considered, 15.2 ft. NGVD, would require a 9-inch raising of a portion of Route 36 and low floodwall (2.5-3 ft.) along the entrance road to the A&P, tying into high ground at Wilson Avenue.

Annual Costs

B85. Annual NED costs were calculated using the current discount rate of 6-5/8%. The costs include all expenses necessary to implement, maintain and operate the improvements over the 50-year period of analysis. Details of the cost estimate are presented in Appendix C.

Benefit-Cost Comparison

B86. Benefits and costs, including the selected interior facilities, were compared as shown in Table B-23. The 14 ft. NGVD design elevation was selected as the NED Plan since it provides the maximum net benefits in excess of costs.

TABLE B-23
SUMMARY OF PLAN ECONOMICS
(1998 PRICE LEVEL, 50-YEAR PERIOD, 6-5/8% INT. RATE)

Line of Protection Design Elevation	13 FT. NGVD	14 FT. NGVD	15.2 FT. NGVD
Annual Benefits	$3,121,180	$3,239,830	$3,453,460
Annual Costs	$2,579,524	$2,667,490	$2,916,308
Net Excess Benefits	$541,656	$572,340	$537,152
BCR	1.2	1.2	1.2

Residual Damage

B87. With the proposed plan in place, the study area will remain subject to flood damage from several sources. For reaches outside the proposed line of protection, damage will remain as presented in Tables B-15 through B-17. This includes the right bank of Compton Creek (Reaches CR1, CR2 and CR3), the left bank of Compton Creek at the mouth of the stream (Reach CL1), and the right bank of Pews Creek at its mouth (Reach PR1).

B88. Within the line of protection, residual damage may occur due to either ponding of interior runoff or overtopping of the levee during extreme events. Interior damage, based on the anticipated depth and frequency of flooding, is expected to average $91,740 on an equivalent annual basis.

B89. Residual damage, due to tidal storms which overtop the line of protection, is summarized in Table B-24. Future increases in damage are due to the projected rise in sea level.

Uncertainty

B90. In order to evaluate the impact of potential uncertainty in flood damages, the uncertainty in benefit estimates was analyzed to evaluate the impact of possible outcomes on the BCR. As seen in Table B-25, there is a 75% chance that the BCR is greater than 1.08 and a 25% chance that it is greater than 1.35.

TABLE B-24
RESIDUAL TIDAL STORM DAMAGE
(1998 PRICE LEVEL, 50-YEAR PERIOD, 6-5/8% INT.)

Reach	Base Year	Future (2050)	Equivalent Annual
P2	$111,750	$139,730	$119,320
P3	$70,140	$87,580	$74,860
P4	$54,500	$67,880	$58,120
C12	$158,920	$197,750	$169,420
C13	$77,160	$96,480	$82,390
Total	$472,470	$589,420	$504,110

TABLE B-25
BENEFIT UNCERTAINTY
(1998 PRICE LEVEL, 50-YEAR PERIOD, 6-5/8% INT.)

	Expected Value	75th Percentile	50th Percentile	25th Percentile
Annual Benefits	$3,239,830	$3,594,240	$3,227,340	$2,884,930
Annual Costs	$2,667,490	$2,667,490	$2,667,490	$2,667,490
Net Annual Benefits	$572,340	$926,750	$559,850	$217,440
BCR	1.21	1.35	1.21	1.08

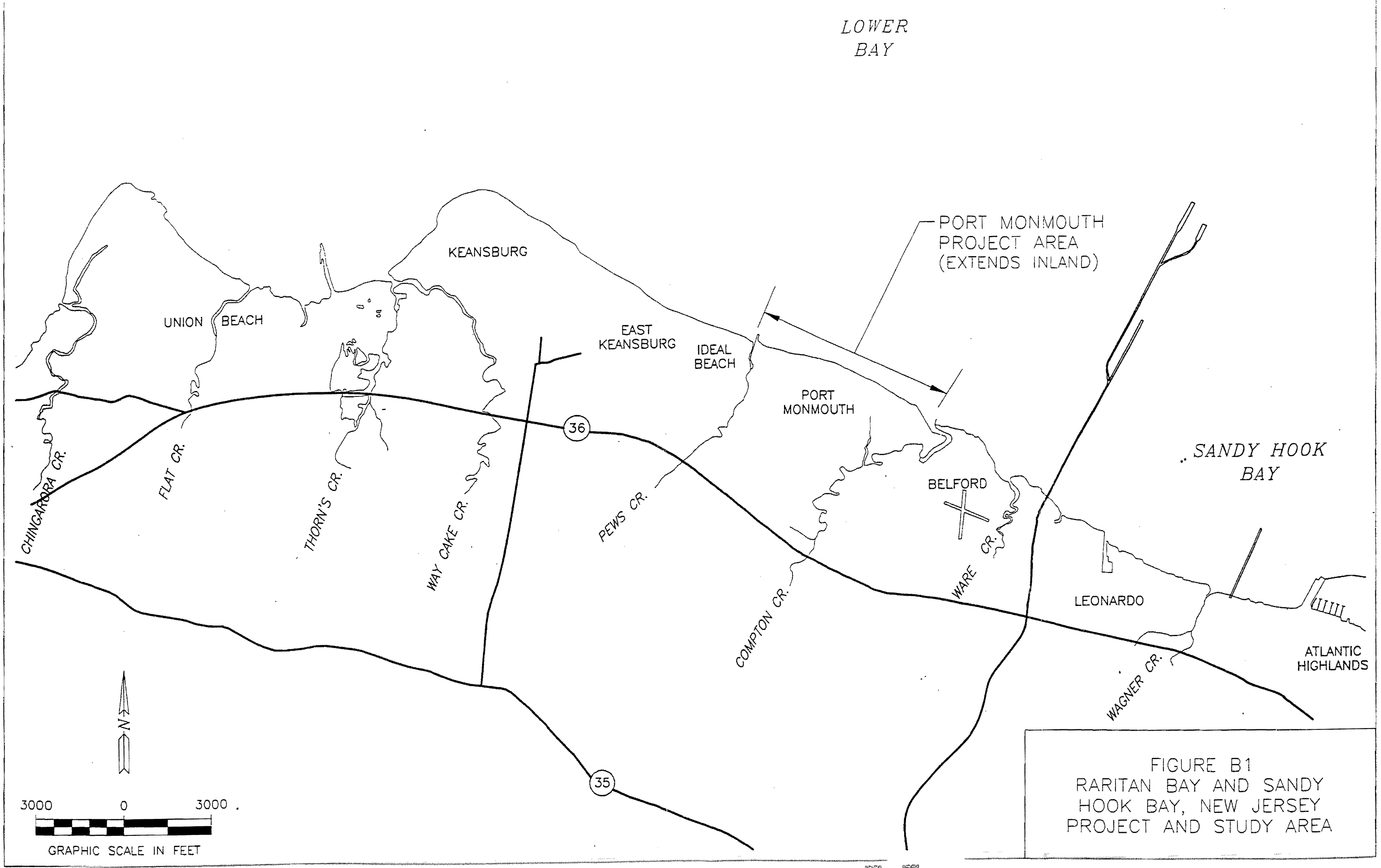
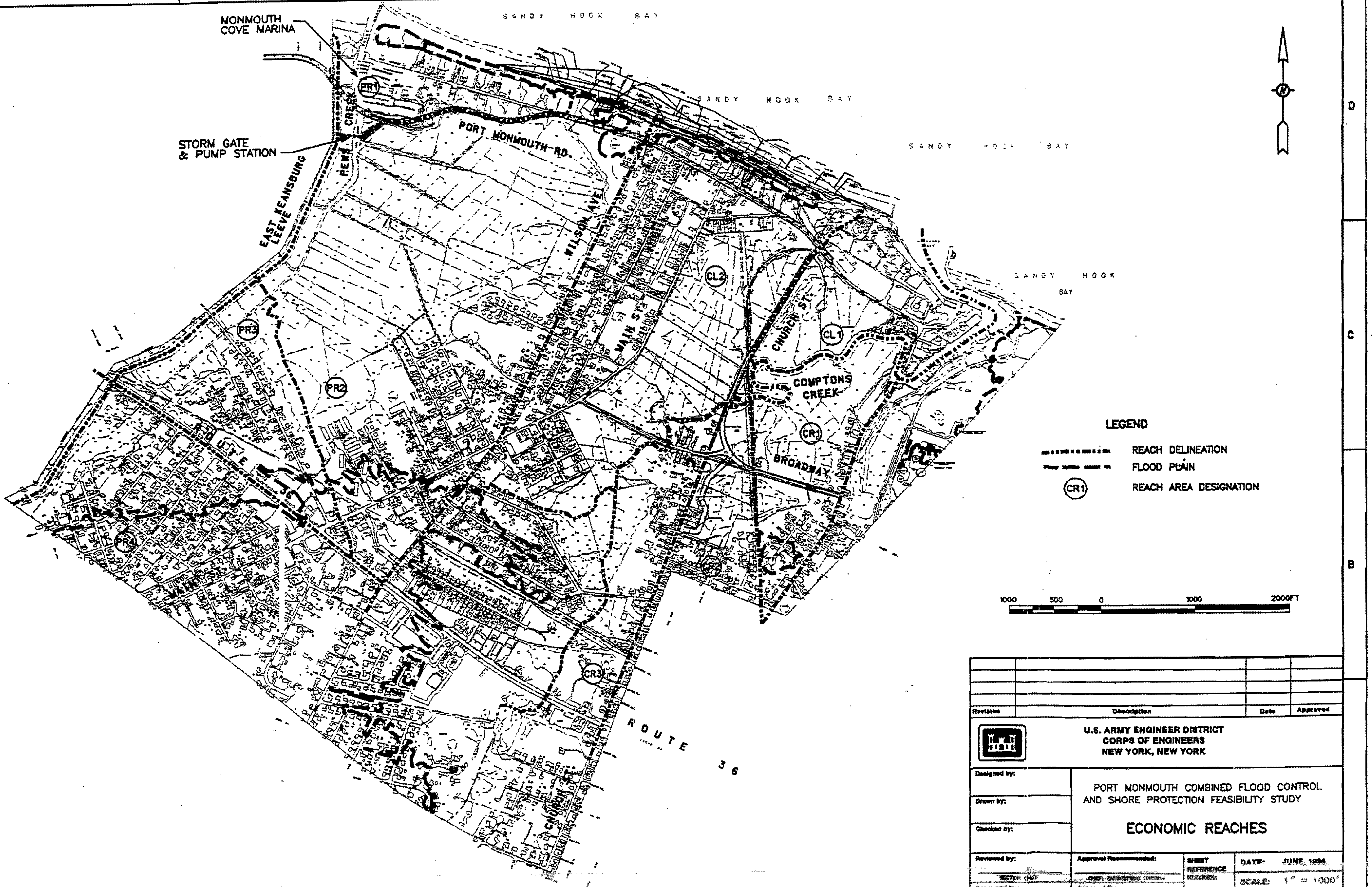


FIGURE B1
RARITAN BAY AND SANDY
HOOK BAY, NEW JERSEY
PROJECT AND STUDY AREA



Revision	Description	Date	Approved

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
NEW YORK, NEW YORK

PORT MONMOUTH COMBINED FLOOD CONTROL
AND SHORE PROTECTION FEASIBILITY STUDY

ECONOMIC REACHES

Designed by:

Drawn by:

Checked by:

Reviewed by:

SECTION CHIEF

Concurred by:

CHIEF, DESIGN BRANCH

Approval Recommended:

CHIEF, ENGINEERING DIVISION

Approval By:

COLONEL, C.E. DISTRICT ENGINEER

SHEET REFERENCE NUMBER:

DATE: JUNE, 1998

SCALE: 1" = 1000'

FIG.: B-2

Port Monmouth, NJ
Stage Damage Reach CL1

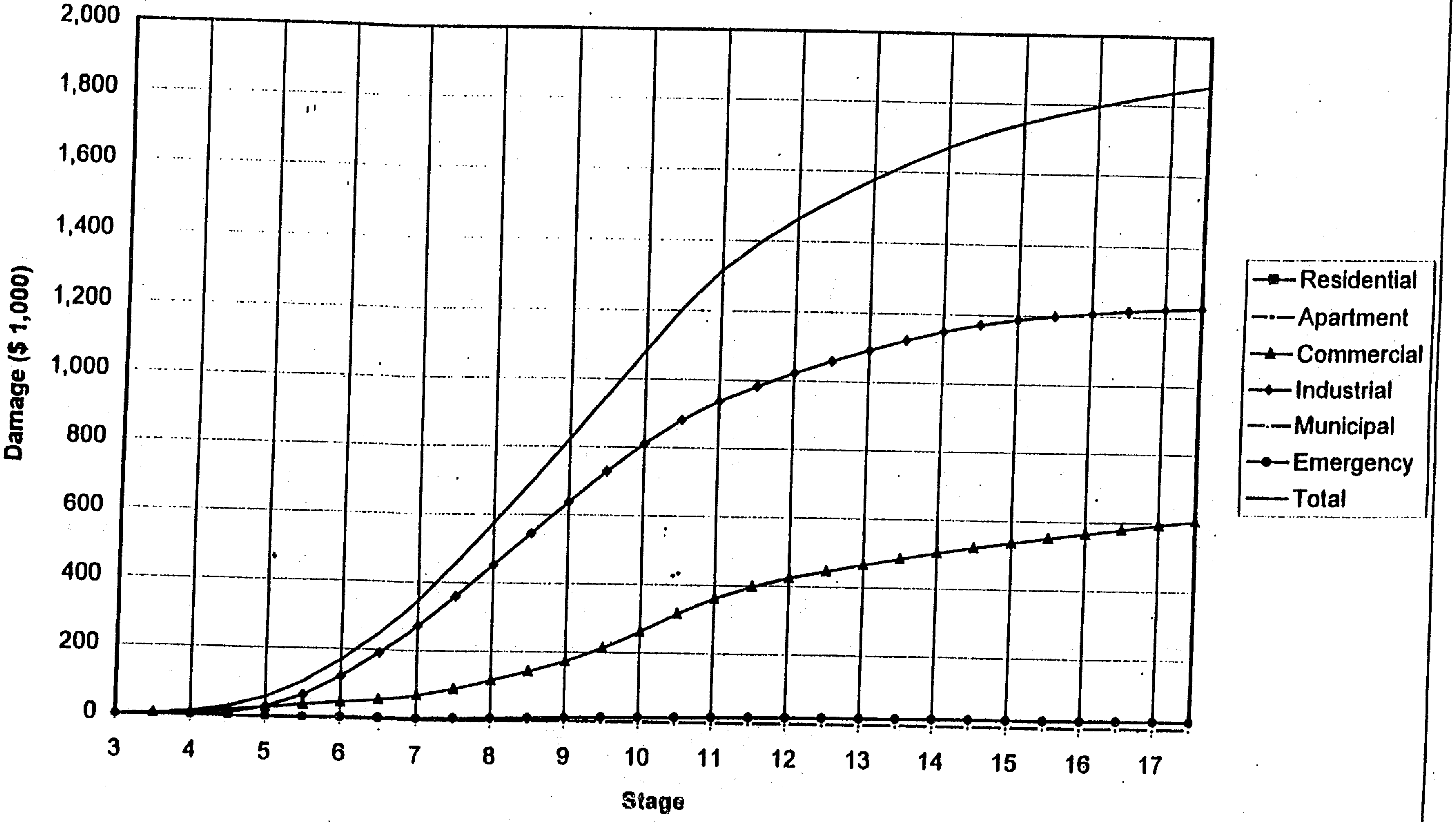
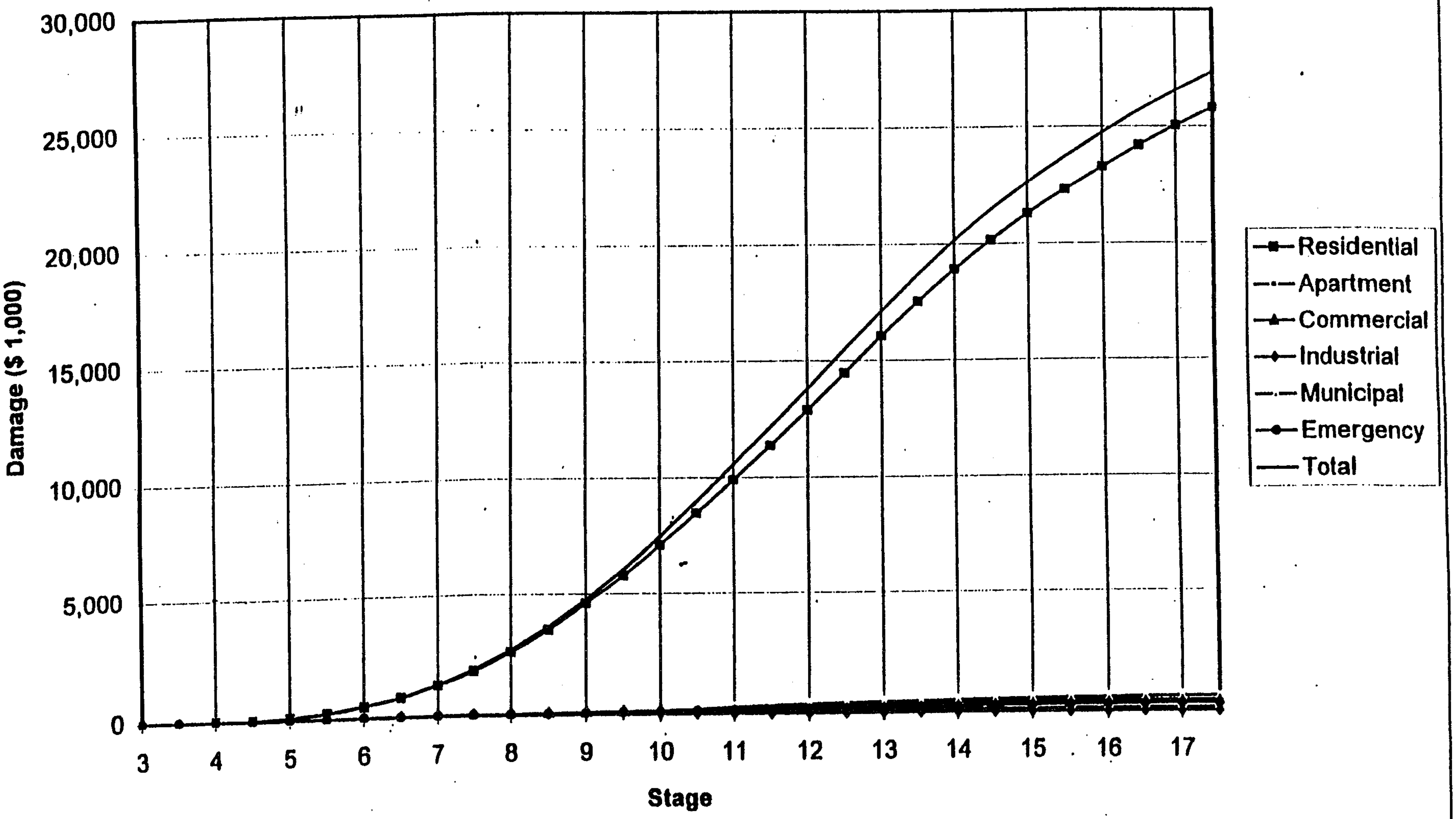


Figure B-3

Port Monmouth, NJ
Stage Damage Reach CL2



B-4

Port Monmouth, NJ
Stage Damage Reach CL3

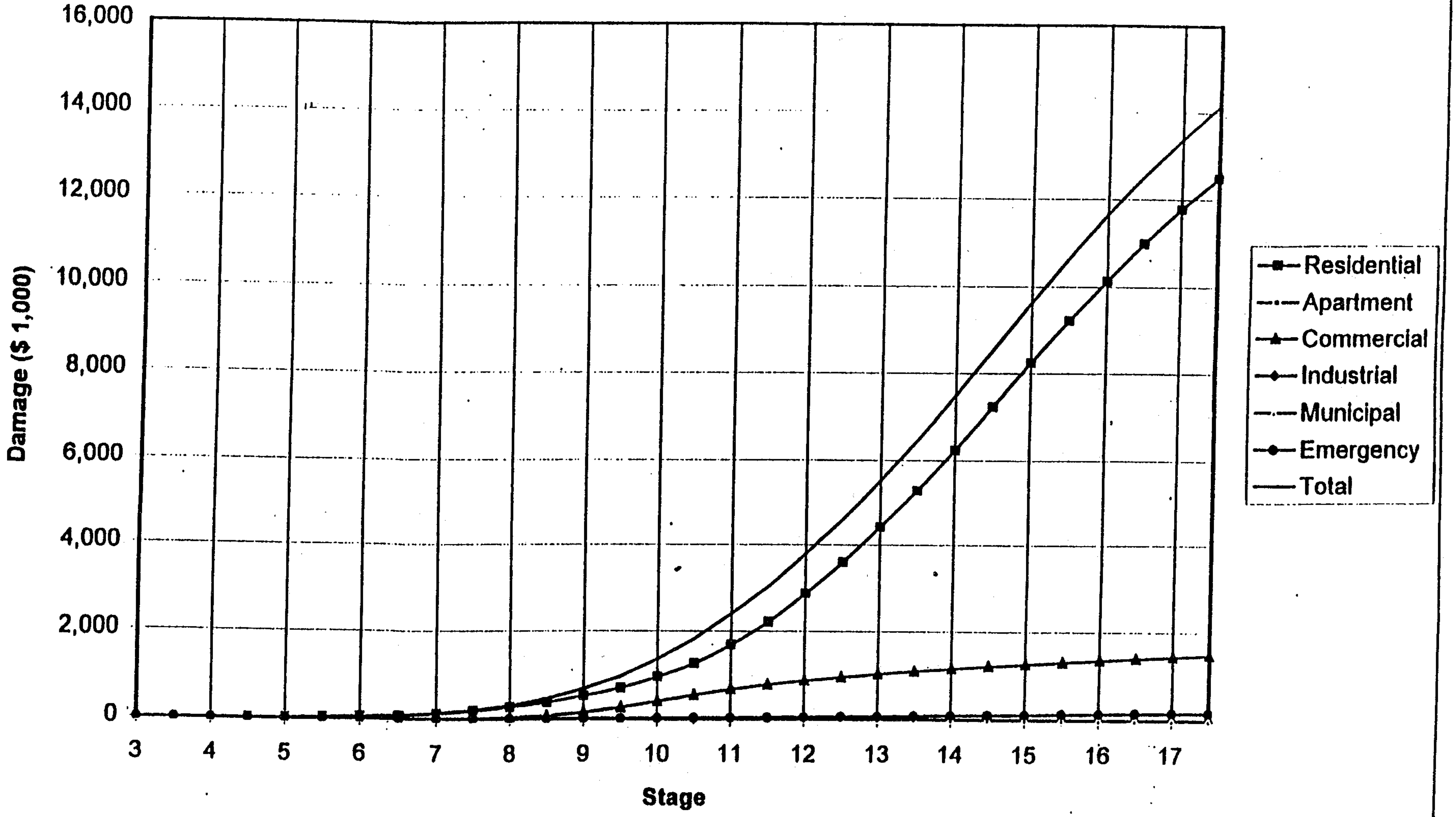
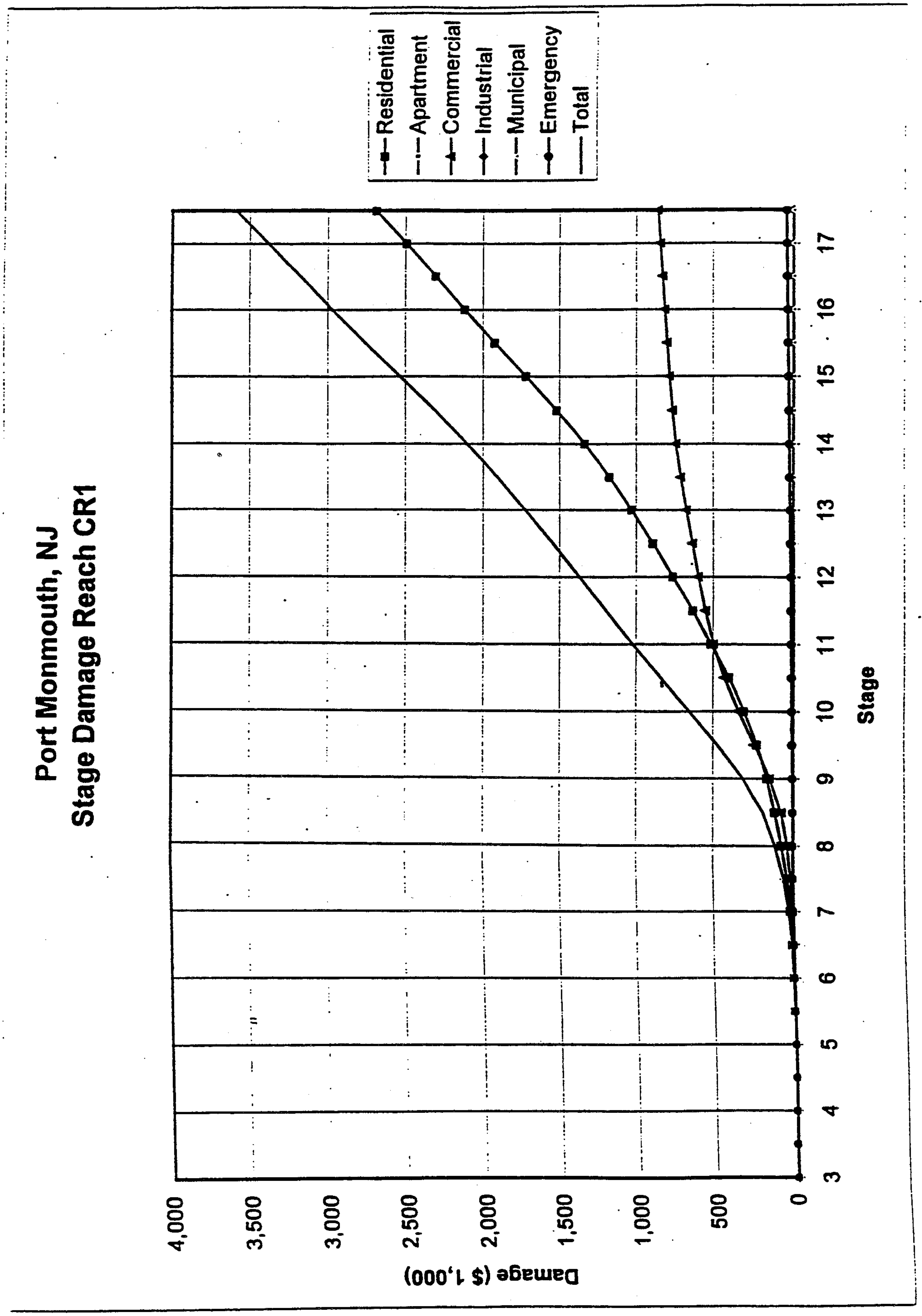


Figure B-5

Figure B-6


Port Monmouth, NJ
Stage Damage Reach CR1
Damage ($ 1,000)
4,000
3,500
3,000
2,500
2,000
1,500
1,000
500
0
Stage
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
Residential
Apartment
Commercial
Industrial
Municipal
Emergency
Total

Port Monmouth, NJ
Stage Damage Reach CR2

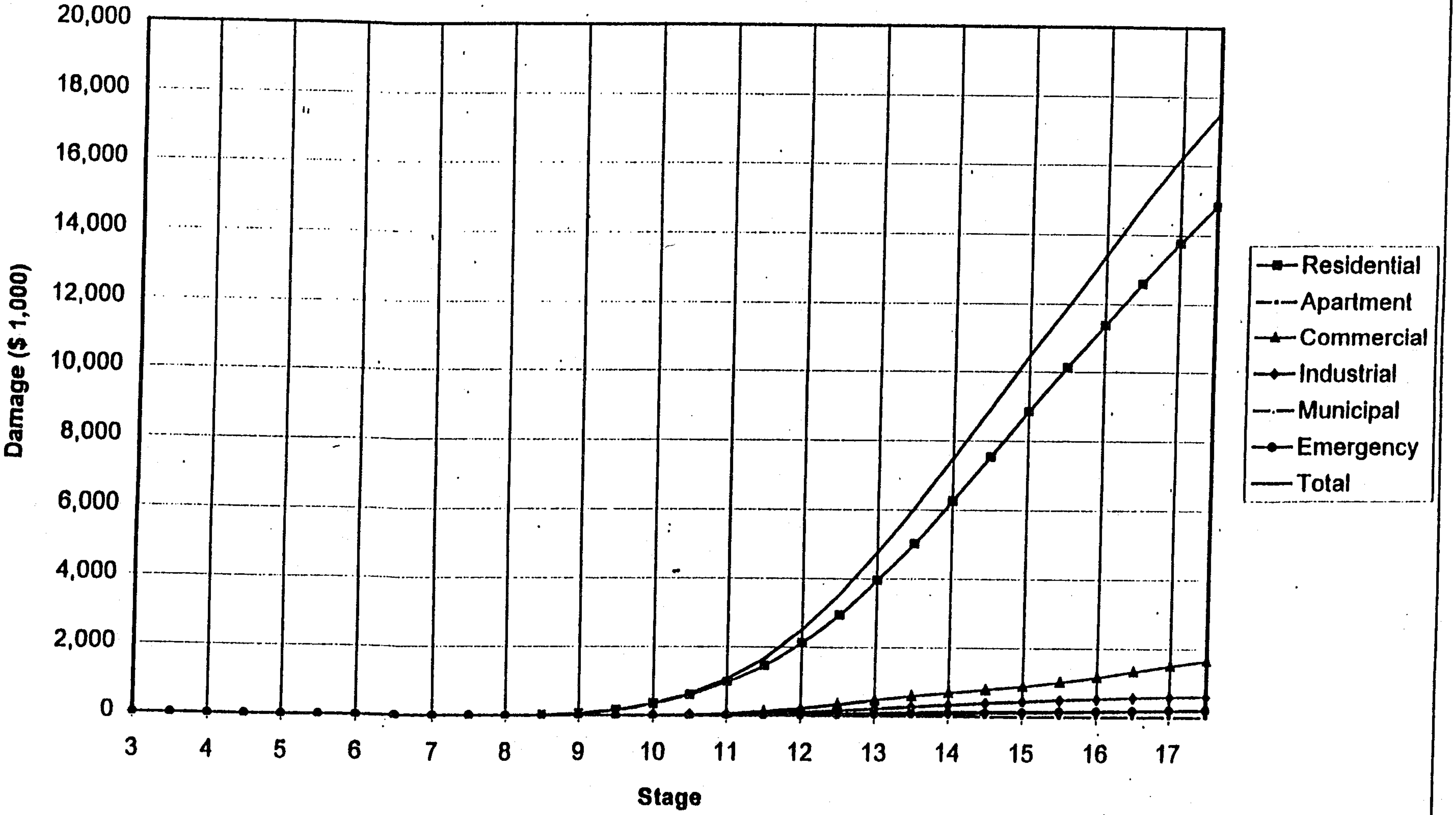
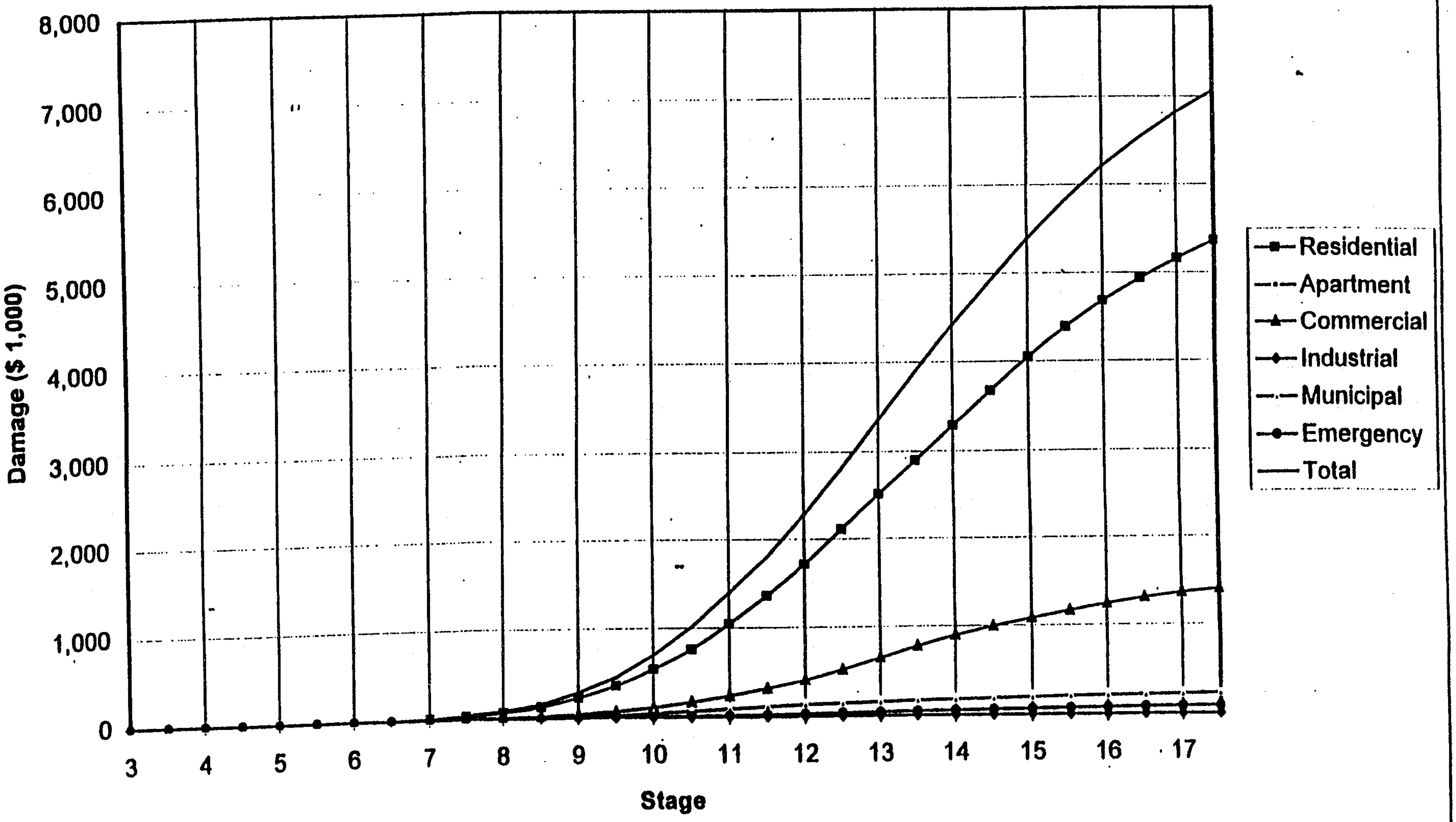


Figure B-7

Port Monmouth, NJ
Stage Damage Reach CR3



e B-8

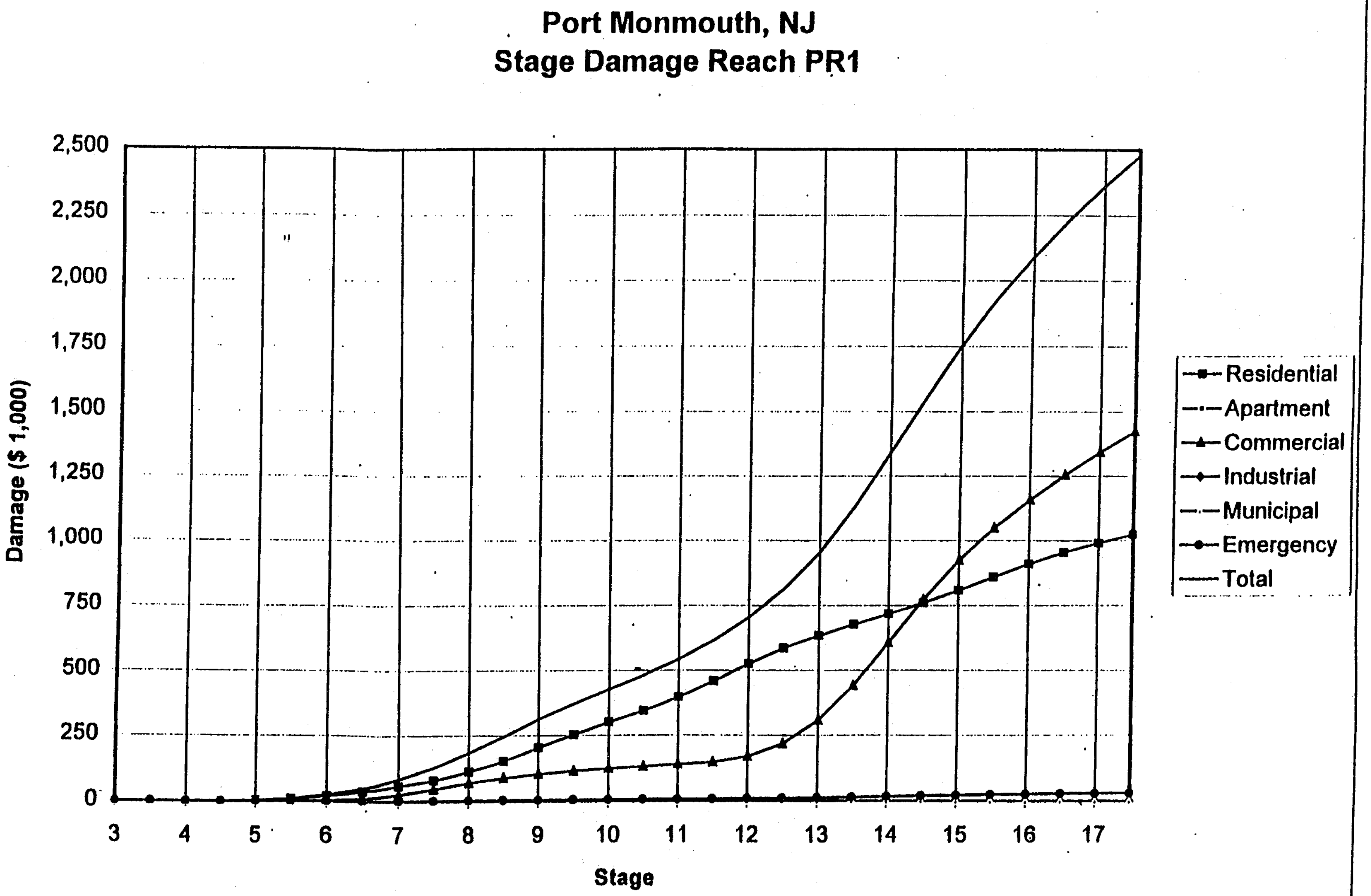
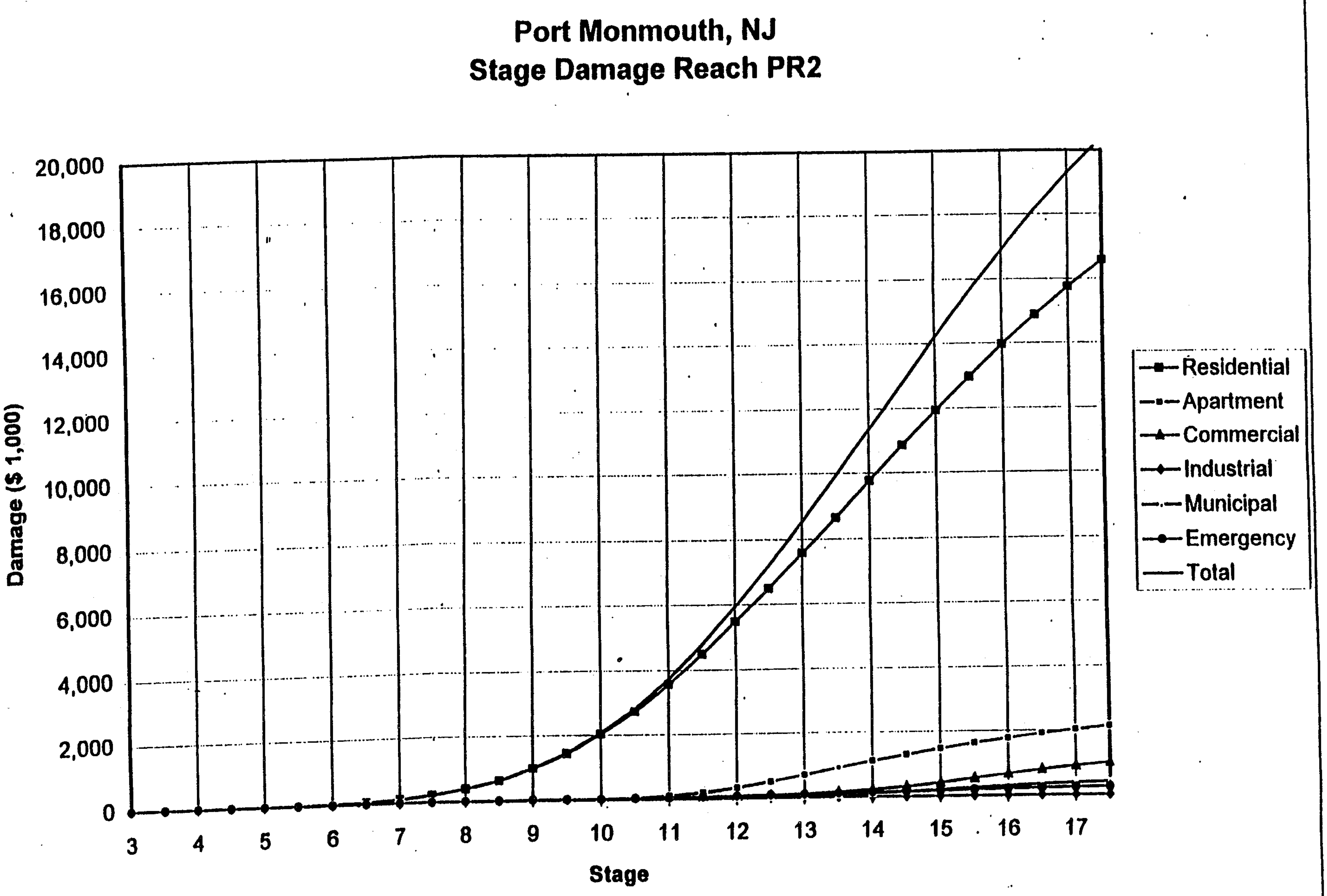

Port Monmouth, NJ
Stage Damage Reach PR1
Damage ($ 1,000)
2,500
2,250
2,000
1,750
1,500
1,250
1,000
750
500
250
0
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
Stage
Residential
Apartment
Commercial
Industrial
Municipal
Emergency
Total

Figure B-9


Port Monmouth, NJ
Stage Damage Reach PR2
Damage ($ 1,000)
20,000
18,000
16,000
14,000
12,000
10,000
8,000
6,000
4,000
2,000
0
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
Stage
Residential
Apartment
Commercial
Industrial
Municipal
Emergency
Total

F. B-10

Port Monmouth, NJ
Stage Damage Reach PR3

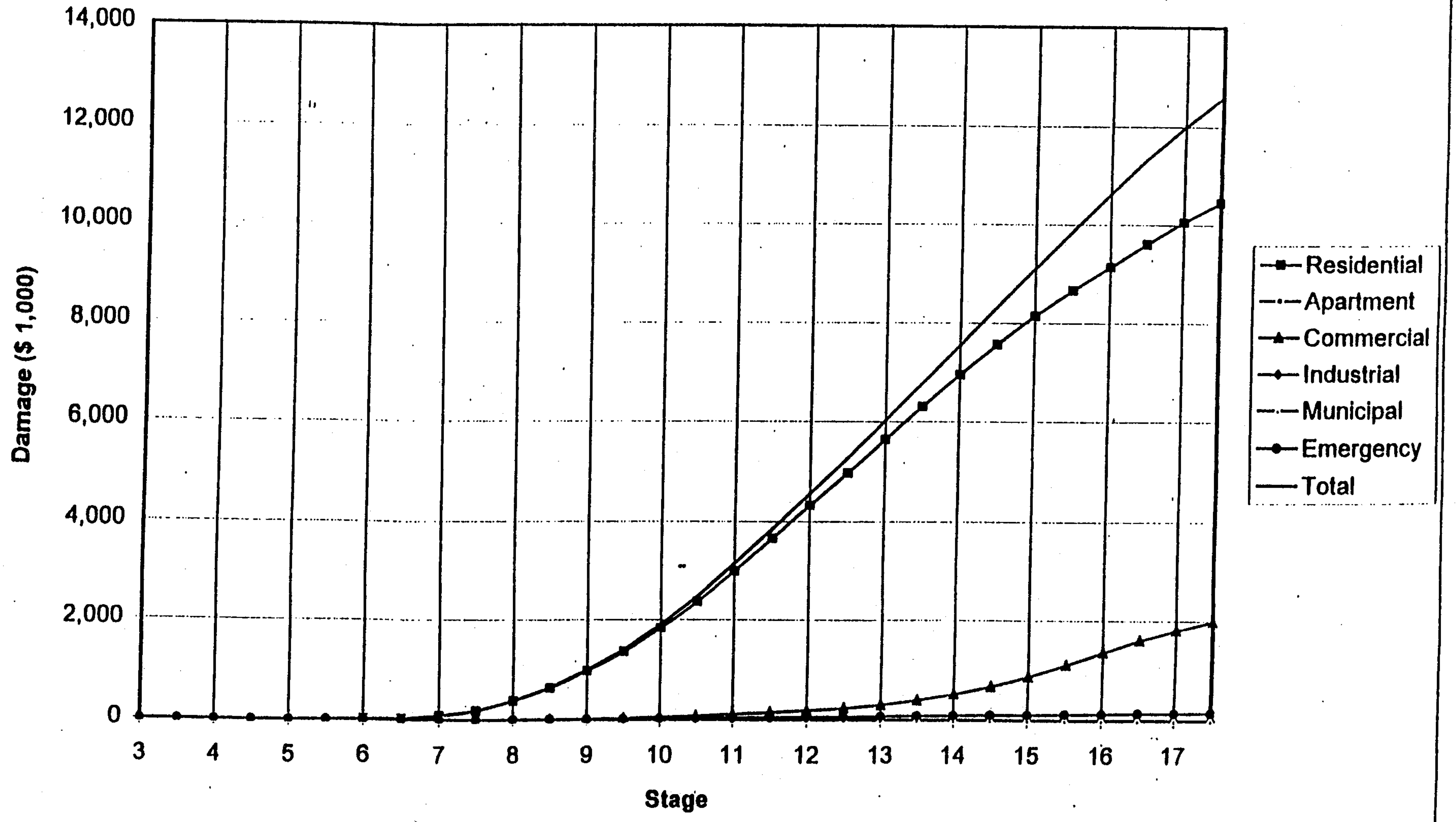


Figure B-11

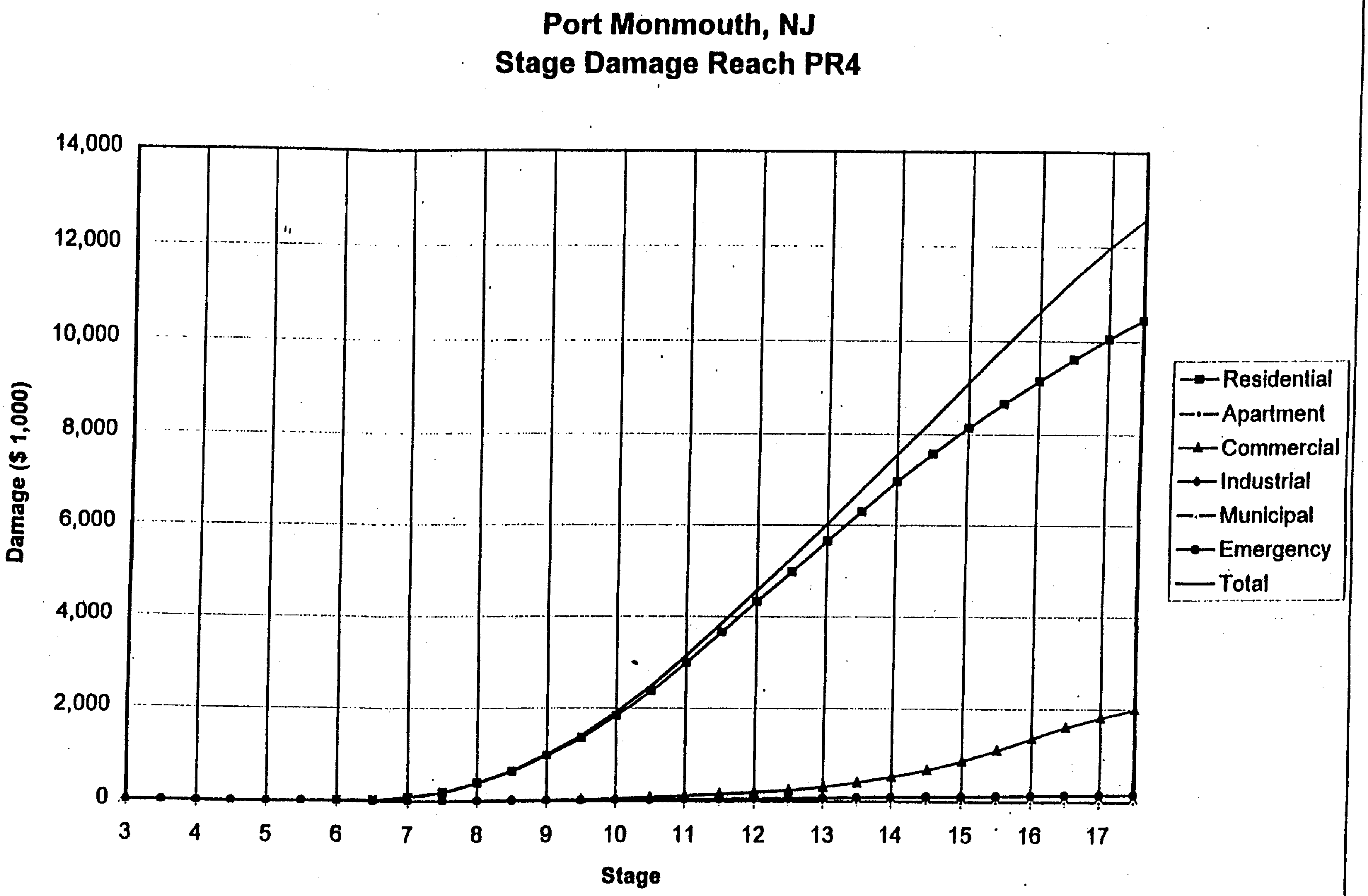

Port Monmouth, NJ
Stage Damage Reach PR4
Damage ($ 1,000)
14,000
12,000
10,000
8,000
6,000
4,000
2,000
0
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
Stage
Residential
Apartment
Commercial
Industrial
Municipal
Emergency
Total

Figure B-12

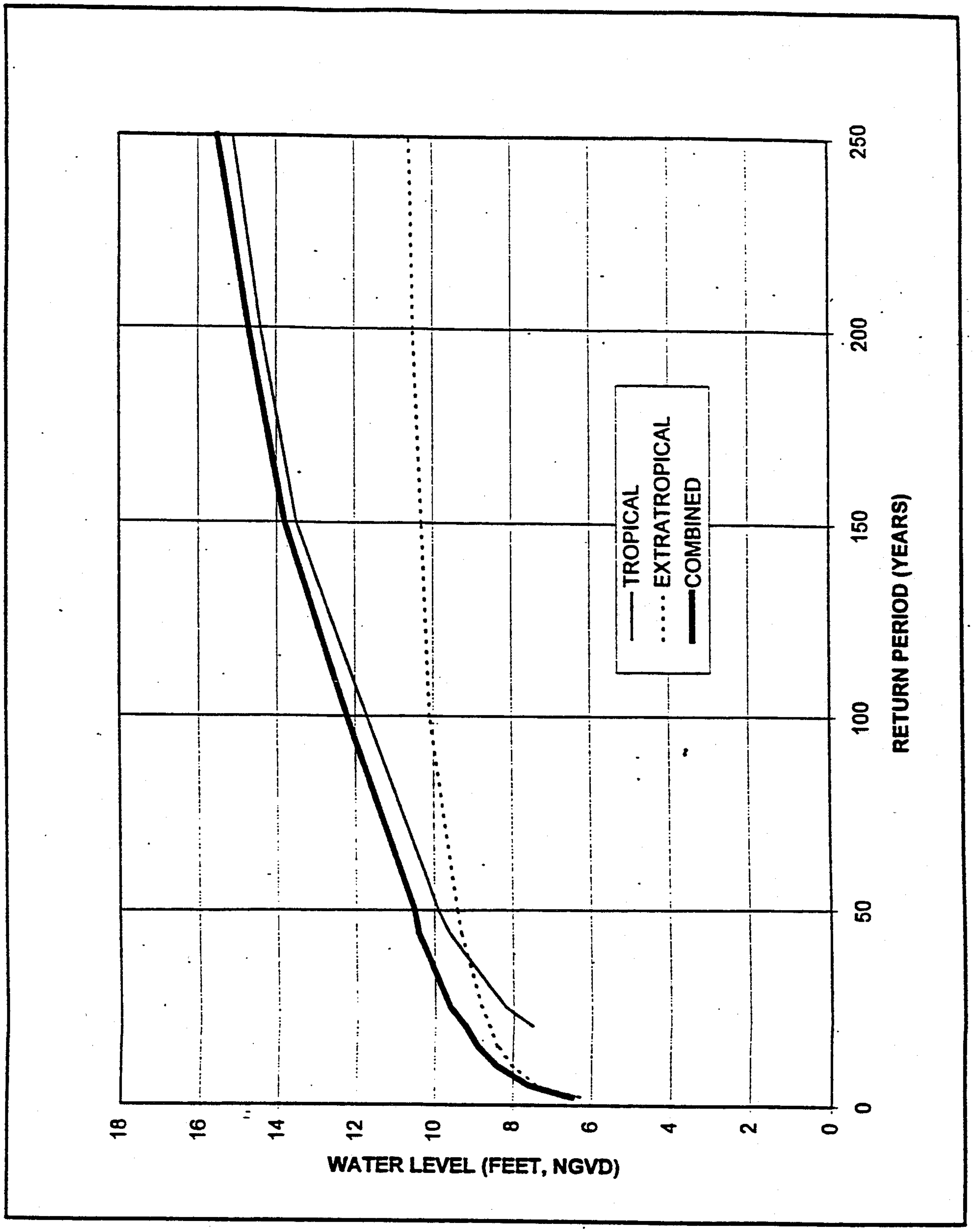


FIGURE B-13

COMBINED STAGE FREQUENCY CURVE

(USACE, 1996)

Sub-Appendix B1

Annual Damage Summaries

PORT MONMOUTH, NJ FEASIBILITY

Expected Annual Damage by Damage Categories and Damage Reaches
for the LEVEE 14/GATE (Levee 14 and Pews Gate Alternative) Plan and Analysis Year 2002
(Damage in $1,000's)
Plan was calculated with Uncertainty

Stream Name	Stream Description	Damage Reach Name	Damage Reach Description	APT
COMPTONS CREEK	East of study area	CR1	COMPTONS CREEK REACH 1 RIGHT BANK	0.00
		CL1	COMPTONS CREEK REACH 1 LEFT BANK	0.00
		CR2	COMPTONS CREEK REACH 2 RIGHT BANK	0.00
		CL2	COMPTONS CREEK REACH 2 LEFT BANK	0.81
		CR3	COMPTONS CREEK REACH 3 RIGHT BANK	0.00
		CL3	COMPTONS CREEK REACH 3 LEFT BANK	0.00
	Total for stream: COMPTONS CREEK			0.81
PEWS CREEK	West of study area	PR1	PEWS CREEK REACH 1	0.00
		PR2	PEWS CREEK REACH 2	11.63
		PR3	PEWS CREEK REACH 3 - Bray Ave. Area	0.00
		PR4	Upstream of Route 36	0.00
	Total for stream: PEWS CREEK			11.63

PORT MONMOUTH, NJ FEASIBILITY
Expected Annual Damage Reduced and Distributed
for the LEVEE 14/GATE (Levee 14 and Pews Gate Alternative) Plan and
Analysis Year 2002
(Damage in $1,000's)
Plan was calculated with Uncertainty

ted Annual Damage		Probability Damage Reduced Exceeds Indicated Values		
Total With Project	Damage Reduced	.75	.50	.25
98.44	0.00	0.00	0.00	0.00
340.02	0.00	0.00	0.00	0.00
144.78	0.00	0.00	0.00	0.00
158.92	1503.44	1348.42	1500.94	1657.95
120.56	0.00	0.00	0.00	0.00
77.16	218.63	180.31	212.98	250.31
939.87	1720.07	1528.73	1713.92	1908.27
109.61	0.00	0.00	0.00	0.00
111.75	284.12	232.43	279.52	332.85
70.14	237.19	193.84	234.88	278.78
54.50	363.82	319.29	361.79	407.79
346.00	885.12	1059.64	1190.27	1333.50

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PORT MONMOUTH, NJ FEASIBILITY
Expected Annual Damage by Damage Categories and Damage Reaches for the LEVEE 14/GATE (Levee 14 and Pews Gate Alternative) Plan and Analysis Year 2002
(Damage in $1,000's)
Plan was calculated with Uncertainty

Damage Categories						
COM	IND	MUN	RES	UTL	XEMG	Total
37.14	0.00	0.00	59.96	0.00	1.33	98.44
80.87	254.99	0.00	0.00	0.00	4.15	340.02
12.21	5.07	1.75	124.15	0.00	1.59	144.78
2.74	0,00	3.74	149.78	0.00	1.84	158.92
21.41	0.00	5.01	92.79	0.00	1.35	120.56
8.81	0.00	0.01	67.47	0.00	0.87	77.16
163.19	260.06	10.52	494.16	0.00	11.13	939.87
36.96	0.00	0.00	71.15	0.00	1.50	109.61
4.79	0.00	1.40	92.68	0.00	1.28	111.75
9.61	0.00	0.00	59.74	0.00	0.79	70.14
4.88	0.00	0.00	49.62	0.00	0.00	54.50
56.24	0.00	1.40	273.18	0.00	3.54	346.00

PORT MONMOUTH, NJ FEASIBILITY
Expected Annual Damage Reduced and Distributed for the LEVEE 14/GATE (Levee 14 and Pews Gate Alternative) Plan and
Analysis Year 2002
(Damage in $1,000's)
Plan was calculated with Uncertainty

Stream Name	Stream Description	Damage Reach Name	Damage Reach Description	Exp
				Total Without Project
COMPTONS CREEK	East of study area	CR1	COMPTONS CREEK REACH 1 RIGHT BANK	98.44
		CL1	COMPTONS CREEK REACH 1 LEFT BANK	340.02
		CR2	COMPTONS CREEK REACH 2 RIGHT BANK	144.78
		CL2	COMPTONS CREEK REACH 2 LEFT BANK	1662.36
		CR3	COMPTONS CREEK REACH 3 RIGHT BANK	120.56
		CL3	COMPTONS CREEK REACH 3 LEFT BANK	293.79
	Total for stream: COMPTONS CREEK			2659.95
PEWS CREEK	West of study area	PR1	PEWS CREEK REACH 1	109.81
		PR2	PEWS CREEK REACH 2	395.87
		PR3	PEWS CREEK REACH 3 - Bray Ave. Area	307.33
		PR4	Upstream of Route 36	418.32
	Total for stream: PEWS CREEK			1231.12

PORT MONMOUTH, NJ FEASIBILITY
Expected Annual Damage Reduced and Distributed
for the LEVEE 14/GATE (Levee 14 and Pews Gate Alternative) Plan and
Analysis Year 2050
(Damage in $1,000's)
Plan was calculated with Uncertainty

Stream Name	Stream Description	Damage Reach Name	Damage Reach Description	Exp
				Total Without Project
COMPTONS CREEK	East of study area	CR1	COMPTONS CREEK REACH 1 RIGHT BANK	156.91
		CL1	COMPTONS CREEK REACH 1 LEFT BANK	470.19
		CR2	COMPTONS CREEK REACH 2 RIGHT BANK	198.93
		CL2	COMPTONS CREEK REACH 2 LEFT BANK	2474.29
		CR3	COMPTONS CREEK REACH 3 RIGHT BANK	186.54
		CL3	COMPTONS CREEK REACH 3 LEFT BANK	437.37
	Total for stream: COMPTONS CREEK			3924.23
PEWS CREEK	West of study area	PR1	PEWS CREEK REACH 1	162.81
		PR2	PEWS CREEK REACH 2	605.69
		PR3	PEWS CREEK REACH 3 - Bray Ave. Area	492.57
		PR4	Upstream of Route 36	626.10
	Total for stream: PEWS CREEK			1887.18

PORT MONMOUTH, NJ FEASIBILITY

Expected Annual Damage Reduced and Distributed
for the LEVEE 14/GATE (Levee 14 and Pews Gate Alternative) Plan and
Analysis Year 2050
(Damage in $1,000's)
Plan was calculated with Uncertainty

ted Annual Damage		Probability Damage Reduced Exceeds Indicated Values		
Total With Project	Damage Reduced	.75	.50	.25
156.91	0.00	0.00	0.00	0.00
470.19	0.00	0.00	0.00	0.00
198.93	0.00	0.00	0.00	0.00
197.75	2276.54	2089.34	2277.43	2476.79
186.54	0.00	0.00	0.00	0.00
96.48	340.89	291.62	338.79	391.52
1306.81	2617.42	2380.96	2616.22	2868.31
162.81	0.00	0.00	0.00	0.00
139.73	465.96	396.18	463.21	537.33
87.58	404.99	347.19	403.98	465.35
67.88	558.22	502.66	557.72	616.11
458.00	1429.18	1778.65	1957.53	2151.42

PORT MONMOUTH, NJ FEASIBILITY
Equivalent Annual Damage by Damage Categories and Damage Reaches for the LEVEE 14/GATE (Levee 14 and Pews Gate Alternative) plan
(Damage in $1,000's)

Discount Rate: 6.625
Analysis Period: 50 Years
Plan was calculated with Uncertainty

Stream Name	Stream Description	Damage Reach Name	Damage Reach Description	APT
COMPTONS CREEK	East of study area	CR1	COMPTONS CREEK REACH 1 RIGHT BANK	0.00
		CL1	COMPTONS CREEK REACH 1 LEFT BANK	0.00
		CR2	COMPTONS CREEK REACH 2 RIGHT BANK	0.00
		CL2	COMPTONS CREEK REACH 2 LEFT BANK	0.87
		CR3	COMPTONS CREEK REACH 3 RIGHT BANK	0.00
		CL3	COMPTONS CREEK REACH 3 LEFT BANK	0.00
	Total for stream: COMPTONS CREEK			0.87
PEWS CREEK	West of study area	PR1	PEWS CREEK REACH 1	0.00
		PR2	PEWS CREEK REACH 2	12.42
		PR3	PEWS CREEK REACH 3 - Bray Ave. Area	0.00
		PR4	Upstream of Route 36	0.00
	Total for stream: PEWS CREEK			12.42

PORT MONMOUTH, NJ FEASIBILITY
Equivalent Annual Damage by Damage Categories and Damage Reaches for the LEVEE 14/GATE (Levee 14 and Pews Gate Alternative) plan
(Damage in $1,000's)

Discount Rate: 6.625
Analysis Period: 50 Years
Plan was calculated with Uncertainty

Equivalent Annual Damage or Damage Categories						Total Damage
COM	IND	MUN	RES	UTL	XEMG	
43.96	0.00	0.00	68.78	0.00	1.52	114.25
87.96	282.69	0.00	0.00	0.00	4.58	375.23
13.29	5.51	1.90	136.98	0.00	1.75	159.43
2.92	0.00	3.99	159.68	0.00	1.96	169.42
24.35	0.00	5.81	106.70	0.00	1.55	138.41
9.39	0.00	0.02	72.06	0.00	0.92	82.39
181.86	288.20	11.72	544.19	0.00	12.27	1039.12
42.15	0.00	0.00	80.18	0.00	1.67	124.00
5.13	0.00	1.50	98.94	0.00	1.34	119.32
10.29	0.00	0.00	63.73	0.00	0.84	74.86
5.20	0.00	0.00	52.92	0.00	0.00	58.12
62.76	0.00	1.50	295.76	0.00	3.85	376.30

PORT MONMOUTH, NJ FEASIBILITY Project Performance
by Damage Reaches for the LEVEE 14/GATE
(Levee 14 and Pews Gate Alternative) plan for Analysis Year 2002
(Stages in ft.)
Plan was calculated with Uncertainty

Without Project Base Year Performance Target Criteria:
Event Exceedance Probability = 0.01
Residual Damage = 5.00 %

Stream Name	Stream Description	Damage Reach Name	Damage Reach Description	Target Stage	Target Annual Exc Probab
					Median
COMPTONS CREEK	East of study area	CR1	COMPTONS CREEK REACH 1 RIGHT BANK	7.49	0.2240
		CL1	COMPTONS CREEK REACH 1 LEFT BANK	5.14	0.9190
		CR2	COMPTONS CREEK REACH 2 RIGHT BANK	9.29	0.0470
		CL2	COMPTONS CREEK REACH 2 LEFT BANK	levee	0.0060
		CR3	COMPTONS CREEK REACH 3 RIGHT BANK	8.19	0.1220
		CL3	COMPTONS CREEK REACH 3 LEFT BANK	levee	0.0060
PEWS CREEK	West of study area	PR1	PEWS CREEK REACH 1	6.22	0.6120
		PR2	PEWS CREEK REACH 2	levee	0.0060
		PR3	PEWS CREEK REACH 3 - Bray Ave. Area	levee	0.0060
		PR4	Upstream of Route 36	levee	0.0060
				0.00	0.0000

PORT MONMOUTH, NJ FEASIBILITY Project Performance
by Damage Reaches for the LEVEE 14/GATE
(Levee 14 and Pews Gate Alternative) plan for Analysis Year 2002
(Stages in ft.)
Plan was calculated with Uncertainty

Without Project Base Year Performance Target Criteria:
Event Exceedance Probability = 0.01
Residual Damage = 5.00 %

age edance ity	Long-Term Risk (years)			Conditional Non-Exceedance Probability by Events					
Expected	10	25	50	10%	4%	2%	1%	.4%	.2%
0.2230	0.9198	0.9982	1.0000	0.0000	0.0004	0.0004	0.0000	0.0000	0.0000
0.9170	1.0000	1.0000	1.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
0.0470	0.3806	0.6980	0.9088	0.9974	0.3276	0.1437	0.0234	0.0021	0.0000
0.0060	0.0620	0.1478	0.2737	0.9974	0.9974	0.9974	0.8932	0.2318	0.1020
0.1200	0.7204	0.9587	0.9983	0.2366	0.0223	0.0210	0.0038	0.0000	0.0000
0.0060	0.0620	0.1478	0.2737	0.9974	0.9974	0.9974	0.8932	0.2318	0.1020
0.6110	0.9999	1.0000	1.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
0.0060	0.0620	0.1478	0.2737	0.9974	0.9974	0.9974	0.8932	0.2318	0.1020
0.0060	0.0620	0.1478	0.2737	0.9974	0.9974	0.9974	0.8932	0.2318	0.1020
0.0060	0.0620	0.1478	0.2737	0.9974	0.9974	0.9974	0.8932	0.2318	0.1020
0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

June 2000

RARITAN BAY AND SANDY HOOK BAY, NEW JERSEY
FEASIBILITY REPORT
FOR
HURRICANE AND STORM DAMAGE REDUCTION
PORT MONMOUTH, NEW JERSEY

APPENDIX C

QUANTITIES AND COST

page C

JUNE 2000

RARITAN BAY AND SANDY HOOK BAY, NEW JERSEY FEASIBILITY REPORT FOR HURRICANE AND STORM DAMAGE REDUCTION PORT MONMOUTH, NEW JERSEY

QUANTITIES AND COST

APPENDIX C

U.S. Army Corps of Engineers
New York District

Table Of Contents

Table Of Contents (Continued)

Page No.

LIST OF TABLES

LIST OF ATTACHMENTS

ATTACHMENT C-1

INTRODUCTION

General

C1. This support document outlines the development of, and contains the first costs for the Port Monmouth Flood Control Project. Methods for deriving the costs of various project elements of the recommended plan are discussed.

C2. The overall cost estimate is comprised of three (3) individual construction elements, namely, Raritan Bay shoreline, Pews Creek and Comptons Creek. A summary sheet reflecting costs reported to feasibility level is shown in Attachment C-1 at the end of this support document.

Basis of Estimates

C3. All estimates are based on May 1998 price levels for labor, materials, and equipment and 1996 topographic surveys. The work quantities for the considered plan of improvement for Pews Creek and Comptons Creek have been developed from the detailed plans shown in the Feasibility Report, as well as detailed design data reflected in accompanying support documents.

C4. The quantities for the Raritan Bay shoreline fill volumes were computed as follows and are presented in Table C-1:

> Dune Fill Volume Estimate. The 1995 beach profile survey was used as existing conditions, forming the basis for the dune volume estimate. The design dune cross-section was superimposed on each of the existing beach profiles. Fill

volume estimates for the dune and the east and west fill transition tapers at heights of 15, 16 and 17 feet were computed and are shown in Table C-1.

Advance Fill Volume. In addition to the design dune, an additional sacrificial quantity of sand will be placed on the beach to compensate for the anticipated erosion rate to maintain the design dune. Advance fill was calculated assuming a continuation of historical erosion trends. The annual rate of 1.93 cy/ft was applied over the dune and taper length of the project to calculate advance fill needs (Table C-1). Advance fill for initial construction included sufficient quantity for the time between the initial construction and construction of the first 10-year renourishment cycle.

Tolerance Fill. Additional fill will be required during construction of the dune and berm to provide for the construction template tolerance of the initial placement volume. A 1 foot construction tolerance is typical. Fill tolerance requirements for each dune height are presented in Table C-1.

Total Initial Fill Volume. The total initial project fill volume is the sum of the design dune fill, the advance fill over the first nourishment cycle, and the tolerance fill. The total fill requirement for the project is the sum above plus overfill. Based on the proposed construction method, an overfill factor of 1.22 was used. The total initial fill volume for the 16 ft. dune height as well as two other heights are presented in Table C-1.

Borrow Areas. In initial construction the borrow area is the Sea Bright borrow area approximately 12 miles from the project site. Renourishment borrow will come from an upland source such as South Amboy, which is approximately 10 miles from the project site.

TABLE C-1
INITIAL BAYSHORE DESIGN FILL VOLUME VS. DUNE HEIGHT
PORT MONMOUTH, NJ

DUNE CREST HEIGHT (FT.)	DESIGN FILL VOLUME (CY.)	ADVANCE FILL VOLUME (CY.)	TOLERANCE FILL VOLUME (CY.)	SUBTOTAL (CY.)	OVERFILL FACTOR	TOTAL FILL VOLUME (CY.)
15	157,999	95,454	52,468	305,921	1.22	373,223
16	161,812	95,454	52,948	310,214	1.22	378,461
17	165,719	95,454	53,428	314,601	1.22	383,813

Future Renourishment Fill. The volume of fill required for each 10-year renourishment project was calculated in a similar manner to the initial advance fill. The annual erosion rate of 1.93 cy/yr/lf of beach was multiplied by the dune plus taper length. An additional volume of sand was added to compensate for sea level rise. Tolerance fill was also included.

Major Rehabilitation Fill. Major rehabilitation/emergency fill volumes are included as an additional responsibility regarding nourishment fill to account for impacts to the design profile from major storm/hurricane events. The requirement for restoration of the design dune profile volume plus advance nourishment was correlated to the anticipated beach loss from major storm events of various frequency levels.

Work Breakdown Structure

C5. The estimate was compiled using MCACES Gold and patterned after the Civil Works Template as a model. The estimate makes use of all six reporting levels available in the following format:

Level 1	Construction Element	One of five major account codes used to estimate the total project cost.
Level 2	Sub-element/Segment	An individual segment of construction activity comprising one or more categories of work or features (cost accounts)
Level 3	Feature	A sub-component of a major type of work (cost accounts)
Level 4-6	Sub-Feature, Bid Item, and Assembly	Increasingly detailed levels of descriptions and estimating dependent on the information and design level developed for the Feasibility Report

Project Description

C6. The project is located in Middletown Township, Monmouth County, New Jersey. The Recommended Plan which is fully described in the Feasibility Report, can be defined according to the major construction elements and sub-elements or segments within three distinct areas of the Port Monmouth area. The three areas of the project are designated as the Raritan Bay Shoreline, Pews Creek, and Comptons Creek. The location of each of the sub-areas within the project area are shown in Figure 1 of the Main Report and are described in the following sections.

Raritan Bay Shoreline

C7. The selected plan for protection along the project's bay shoreline consists of a reconstructed dune, berm and beach section with periodic nourishment. The design dune incorporates and enhances the existing dune to the extent possible to provide required protection to minimize project impacts.

The eastern limit of the relocated dune ties into the Comptons Creek levee alignment near the intersection of Park Avenue and Port Monmouth Road. The dune extends approximately 2,640 feet to the west to tie into the Pews Creek levee.

A dune crest width of 25 feet was adopted for the selected plan. This width was selected based on dune survivability after the design storm event. The design dune optimized elevation is 16 ft. NGVD. The selected landward dune slope is 1 vertical on 5 horizontal whereas the bayside dune face has a flatter slope of 1 vertical on 10 horizontal. A 50 foot wide berm section will be constructed seaward of the dune at an elevation of +9 feet NGVD. The beach will then slope at 1 vertical on 15 horizontal to the existing bottom. The dune section and beach will be created from hydraulically placed fill from the Sea Bright borrow area. The dune section will be stabilized with dune grass and fencing. Wood overwalks will be provided for access and to protect dune vegetation from pedestrian damage.

Pews Creek

C8. From the terminus of the new dune approximately 700 feet northwest of the intersection of Wilson Avenue and Port Monmouth Road a 65 foot long levee section will span between the dune and Port Monmouth Road. This levee section is to abut the beginning of the proposed floodwall along the north side of Port Monmouth Road. This floodwall will be approximately four feet high at a design elevation of 14 ft. NGVD. The alignment continues westerly as a floodwall along the northern side of Port Monmouth Road for about 700 feet until it reaches the interse ıg ramp to the Monmouth County Marina. A mitre closure gate approximately 40 feet long by 4 feet high will bridge this gap. The alignment then continues as a floodwall westward along the north side of Port Monmouth Road for about 1,500 feet where it will tie into the northern side road embankment at elevation +14 NGVD. Placement of the floodwall along the north side of

Port Monmouth Road will allow the roadway to remain accessible during floods and provide access from Keansburg to Port Monmouth or vice versa.

The alignment incorporates a section of Port Monmouth Road which is at or above the design height and connects to a proposed levee south of "new" Port Monmouth Road. The levee proceeds in a southwest direction through the wetlands for about 300 feet until it reaches the east bank of Pews Creek. A 40 foot wide sector gate is to be constructed across Pews Creek about 300 feet south of the Pews Creek Bridge. This storm gate system will connect to a proposed concrete wall on the west bank of Pews Creek. This concrete wall will run west for about 150 feet where it will connect with the existing Keansburg levee.

A 120 cfs pump station is an integral part of this flood protection system and will be located adjacent to the east side of the Keansburg levee and will incorporate the concrete wall between the Keansburg levee and the storm gate at its north wall.

Comptons Creek

C9. The alignment for flood protection from Comptons Creek starts out as I-wall 250 feet southeast of the intersection of Wilson Avenue and State Highway 36 and proceeds easterly along the rear property line of the homes fronting on Willow Avenue. The wall spans approximately 1,250 feet in length.

This floodwall section ranges from one to six feet above existing grade.

The alignment converts from an I-type floodwall to a levee and proceeds easterly for about 600 feet where it crosses an existing drainage ditch located between Campbell Avenue and Willow Avenue. The levee then turns north and approaches Campbell

Avenue perpendicularly about 100 feet east of the intersection of Campbell Avenue and Creek Road. A mitre closure gate is proposed for the Campbell Avenue crossing. The gate will be approximately 40 feet wide and 9 feet high to provide flood protection to elevation 14 ft. NGVD.

The levee continues from the Campbell Avenue crossing in a northerly direction through the wetlands nearly parallel to Creek Road for approximately 1,100 feet. The levee height for this section varies between 4.5 feet and 10.5 feet above existing grade. The levee makes a turn towards the northeast, paralleling Woodstock Avenue for 400 feet then changes direction northward for 800 feet to meet Broadway about 100 feet east of the intersection of Main Street and Broadway.

A mitre closure gate is proposed to span across Broadway. The gate will be approximately 40 feet wide and 8 feet high to provide flood protection elevation to 14 ft. NGVD.

The alignment continues as a levee in a northeasterly direction paralleling Main Street for about 2,000 feet before it changes course and heads east for approximately 700 feet along the rear property lines of the homes which front on Park Avenue. The average levee height for this 2,700 foot section is about 10.5 feet above existing grade. The levee then proceeds northerly meeting Port Monmouth Road about 800 feet southeast of the intersection of Main Street and Port Monmouth Road. Port Monmouth Road will have to be elevated to the design elevation in the area where the levee meets the road. The levee picks up again at the north side of Port Monmouth Road and proceeds north approximately 150 feet towards the dunes where it will tie into the bayfront dune.

Formulation of Project First Costs

First Costs

C10. First costs include the charges arising from the construction of each individual construction element, as well as the costs of contingencies, engineering, design, supervision and administration. The detailed estimates include such items as: lands, relocations, levees, floodwalls, road closure gates, storm sector gate with pump station, interior drainage pump station, beach replenishment, raising of roads, and wetlands mitigation costs. Given in Attachment C1 are the MCACES Gold estimate's title, table of contents, and summary pages for the recommended plan of protection. A summary of the project costs for the recommended plan of protection for the complete project is shown in Table C-2. Table C2-A provides the Fully Funded Costs for the recommended plan escalated to the midpoint of construction, September 2003.

Table C-3 provides detailed quantities and first cost estimates for the recommended plan (i.e. levee/floodwall 14.0 feet NGVD; dune 16.0 feet NGVD). Tables C4 and C5 provided detailed quantities and first cost estimates for the other two alternative level of protection plans for the three levels of protection analyzed.

TABLE C-2
SUMMARY OF PROJECT FIRST COSTS
MAY, 1998 PRICE LEVEL

Account	Project Total
01 Lands and Damages	$ 946,000
02 Relocations	$ 452,000
06 Fish & Wildlife Mitigation	$ 1,706,000
11 Levees & Floodwalls	$12,492,000
13 Pumping Plant	$ 2,869,000
15 Floodway Control Diversion Structures	$ 2,498,000
17 Beach Replenishment	$ 3,810,000
30 Planning, Engineering & Design	$ 4,850,000
31 Construction Management	$ 1,760,000
TOTAL	**$31,383,000**

TABLE C-2A
FULLY FUNDED COSTS

Project First Costs						Fully Funded Estimate			
Current MCASES Estimate Prepared: 6/98 Effective Pricing Level 5/98 Account						Feature Mid Point: 9/2003			
		Cost ($)	Contingency ($)	Contingency (%)	Total	CWCCIS	Cost ($)	Contingency ($)	Total
02	Relocations	$393,000.00	$59,000.00	15%	$452,000	1.176	$462,168.00	$69,384.00	$531,552
06	Fish & Wildlife Mitigation	$1,550,000.00	$156,000.00	10%	$1,706,000	1.171	$1,815,050.00	$182,676.00	$1,997,726
11	Levees & Floodwalls	$10,995,000.00	$1,497,000.00	15%	$12,492,000	1.167	$12,831,165.00	$1,746,999.00	$14,578,164
13	Pumping Plant	$2,494,000.00	$374,000.00	15%	$2,869,000	1.153	$2,875,582.00	$431,222.00	$3,306,804
15	Floodway Control Diversion Structures	$2,119,000.00	$379,000.00	15%	$2,498,000	1.171	$2,481,349.00	$443,809.00	$2,925,158
17	Beach Replenishment	$3,313,000.00	$497,000.00	15%	$3,810,000	1.183	$3,919,279.00	$587,951.00	$4,507,230
	Total Construction Cost	$20,864,000.00	$2,962,000.00		$23,827,000		$24,384,593.00	$3,462,041.00	$27,846,634
01	Lands and Damages	$757,000.00	$189,000.00	25%	$946,000	1.230	$931,110.00	$232,470.00	$1,163,580
30	Engineering & Design	$4,400,000.00	$450,000.00	10.2%	$4,850,000	1.230	$5,412,000.00	$553,500.00	$5,965,500
31	Construction Management	$1,537,000.00	$223,000.00	14.5%	$1,760,000	1.230	$1,890,510.00	$274,290.00	$2,164,800
	Total Project Cost	$27,558,000.00	$3,824,000.00		$31,383,000		$32,618,213.00	$4,522,301.00	$37,140,514

TABLE C3 - RECOMMENDED PLAN
TOTAL FIRST COST - PORT MONMOUTH REACH
RARITAN BAY & SANDY HOOK BAY COMBINED FLOOD CONTROL & SHORE PROTECTION STUDY
LEVEE/FLOODWALL ELEVATION 14.0; DUNE ELEVATION 16.0

ACCOUNT CODE	DESCRIPTION	AMOUNT	CONTINGENCY AMOUNT	CONT. %	TOTAL
01	LANDS AND DAMAGES				
01.03	Real Estate Costs - Local	$672,000	$168,000	25%	
01.04	Real Estate Costs - Administration	$85,000	$21,000	25%	
	SUBTOTAL	$757,000			
	CONTINGENCY		$189,000		
	LANDS AND DAMAGES TOTAL				$946,000
02	RELOCATIONS				
02.01	Raise Port Monmouth Road				
02.01.01	Site Work	$22,000	$3,000	15%	
02.01.13	Traffic Control	$9,000	$1,000	15%	
02.01.19	Construct Approaches to Subgrade	$52,000	$8,000	15%	
02.01.39	Road Surfacing	$106,000	$16,000	15%	
02.01.99	Associated General Items	$4,000	$1,000	15%	
02.02	Drainage				
02.02.01	36" Outlet Diversion Pipe	$119,000	$18,000	15%	
02.02.39	Road Surfacing	$6,000	$1,000	15%	
02.03	Port Monmouth Pier Extension	$75,000	$11,000	15%	
	SUBTOTAL	$393,000			
	CONTINGENCY		$59,000		
02	RELOCATIONS TOTAL				$452,000
06	FISH AND WILDLIFE FACILITIES				
06.03	Wildlife Facilities & Sanctuaries				
06.03.06	Wetland Mitigation	$1,535,000	$154,000	10%	
06.03.07	Wetland Monitoring	$15,000	$2,000	16%	
	SUBTOTAL	$1,550,000			
	CONTINGENCY		$156,000		
06	FISH AND WILDLIFE TOTAL				$1,706,000

TABLE C3 - RECOMMENDED PLAN (continued)
TOTAL FIRST COST - PORT MONMOUTH REACH
RARITAN BAY & SANDY HOOK BAY COMBINED FLOOD CONTROL & SHORE PROTECTION STUDY
LEVEE/FLOODWALL ELEVATION 14.0; DUNE ELEVATION 16.0

ACCOUNT CODE	DESCRIPTION	AMOUNT	CONTINGENCY AMOUNT	CONT. %	TOTAL
11	LEVEES AND FLOODWALLS				
11.01	Levees				
11.01.01	Mob, Demob & Preparatory Work				
11.01.01.01	Mob. & Demob.	$124,000	$19,000	15%	
11.01.01.02	Erosion & Sediment Controls	$19,000	$2,000	10%	
11.01.01.04	Temp. Access Road	$20,000	$2,000	10%	
11.01.05	Site Work				
11.01.05.01	Clear & Grub	$220,000	$22,000	10%	
11.01.05.02	Excavation Common	$246,000	$25,000	10%	
11.01.05.04	Embankment Common	$1,228,000	$123,000	10%	
11.01.05.05	Riprap / Ditch Lining	$233,000	$23,000	10%	
11.01.05.06	Embankment Impervious	$1,014,000	$101,000	10%	
11.01.05.07	Embankment Pervious	$158,000	$16,000	10%	
11.01.05.08	Stripping	$140,000	$14,000	10%	
11.01.05.09	Topsoil and Seeding	$424,000	$42,000	10%	
11.01.05.15	Fabric (Geotextile)	$123,000	$12,000	10%	
11.01.06	Storm Gate				
11.01.06.01	Storm Gate Structure @ Pews Creek	$3,914,000	$783,000	20%	
11.02	Floodwalls				
11.02.02	Site Work				
11.02.02.01	Excavation @ Base	$28,000	$3,000	10%	
11.02.02.02	Steel Sheeting	$1,576,000	$158,000	10%	
11.02.02.03	Timber Piles	$78,000	$8,000	10%	
11.02.02.04	Dewatering	$22,000	$2,000	10%	
11.02.03	Floodwall Construction				
11.02.03.01	Formwork	$334,000	$33,000	10%	
11.02.03.02	Reinforcing Steel	$516,000	$52,000	10%	
11.02.03.03	Place Concrete	$577,000	$58,000	10%	
	SUBTOTAL	$10,994,000			
	CONTINGENCY		$1,497,000		
11	TOTAL LEVEES AND FLOODWALLS:				$12,492,000

TABLE C3 - RECOMMENDED PLAN (continued)
TOTAL FIRST COST - PORT MONMOUTH REACH
RARITAN BAY & SANDY HOOK BAY COMBINED FLOOD CONTROL & SHORE PROTECTION STUDY
LEVEE/FLOODWALL ELEVATION 14.0; DUNE ELEVATION 16.0

ACCOUNT CODE	DESCRIPTION	AMOUNT	CONTINGENCY AMOUNT	CONT. %	TOTAL
13	PUMPING PLANT				
13.01	Comptons Creek (C3) 60 CFS	$1,127,000	$169,000	15%	
13.02	Pews Creek 120 CFS Pump Station	$1,367,000	$205,000	15%	
	SUBTOTAL	$2,494,000			
	CONTINGENCY		$374,000		
13	TOTAL PUMPING PLANT:				$2,869,000
15	FLOODWAY CONTROL DIVERSION STRUCTURES				
15.01	Road Closure Gates				
15.01.01	4' x 40' Closure Gate (Marina Road)	$242,000	$48,000	20%	
15.01.02	9' x 40' Closure Gate (Campbell Avenue)	$546,000	$109,000	20%	
15.01.03	8' x 40' Closure Gate (Broadway)	$485,000	$97,000	20%	
15.02	Drainage Structures				
15.02.00	Site Work				
15.02.00.02	Excavation	$15,000	$0		
15.02.00.10	Seeding	$11,000	$2,000	15%	
15.02.03	Outlet Pipes				
15.02.03.03	Reinforced Concrete Pipe	$47,000	$7,000	15%	
15.02.04	Drainage Structures				
15.02.04.01	Drainage Inlet and Outlet Structures	$400,000	$60,000	15%	
15.02.05	Metals				
15.02.05.06	Gates and Trash Racks	$368,000	$55,000	15%	
15.02.39	Road Surfacing				
15.02.39.01	Restore Road Surface	$5,000	$1,000	15%	
	SUBTOTAL	$2,119,000			
	CONTINGENCY		$379,000		
15	TOTAL FLOODWAY CONTROL DIVERSION STRUCTURES				$2,498,000

TABLE C3 - RECOMMENDED PLAN (continued)
TOTAL FIRST COST - PORT MONMOUTH REACH
RARITAN BAY & SANDY HOOK BAY COMBINED FLOOD CONTROL & SHORE PROTECTION STUDY
LEVEE/FLOODWALL ELEVATION 14.0; DUNE ELEVATION 16.0

ACCOUNT CODE	DESCRIPTION	AMOUNT	CONTINGENCY AMOUNT	CONT. %	TOTAL
17	Beach Replenishment				
17.00	Beach Replenishment				
17.00.01	Mob, Demob for Dredging	$450,000	$68,000	15%	
17.00.02	Mob, Demob for Associated Work	$23,000	$3,000	15%	
17.00.16	Pipeline Dredging				
17.00.16.01	Dredging & Placement	$2,706,000	$406,000	15%	
17.00.99	Associated General Items				
17.00.99.01	Dune Grass	$47,000	$7,000	15%	
17.00.99.02	Sand Fence	$18,000	$3,000	15%	
17.00.99.03	Dune Overwalk	$70,000	$10,000	15%	
	SUBTOTAL	$3,313,000			
	CONTINGENCY		$497,000		
17	TOTAL BEACH REPLENISHMENT:				$3,810,000
	Sub-Total Construction Cost				$24,773,000
30	Engineering & Design	$4,400,000	$450,000	10.2%	$4,850,000
31	Construction Management	$1,537,000	$223,000	14.5%	$1,760,000
	TOTAL PROJECT FIRST COST				$31,383,000

TABLE C4
TOTAL FIRST COST - PORT MONMOUTH REACH
RARITAN BAY & SANDY HOOK BAY COMBINED FLOOD CONTROL & SHORE PROTECTION STUDY
LEVEE/FLOODWALL ELEVATION 13.0;DUNE ELEVATION 15.0

DESCRIPTION	AMOUNT	CONTINGENCY AMOUNT	CONT. %	TOTAL
LANDS AND DAMAGES				
Pre-Authorization Planning				
Develop Aquisition Schedule				
Permanent Easement	$478,054	$119,514	25%	
Temporary Easement	$37,763	$9,441	25%	
Severance Damages	$38,440	$9,610	25%	
All Others				
Mitigation Aquisition	$58,880	$14,720	25%	
Enhancement Aquisition	$0	$0	25%	
Survey, Appraisal & Admin.	$85,000	$21,250	25%	
SUBTOTAL	$698,137			
CONTINGENCY		$174,534		
LANDS AND DAMAGES TOTAL				$873,000
RELOCATIONS				
Roads, Construction Activity				
Mob & Demob	$3,700	$555	15%	
Road Raising				
Port Monmouth Road Raising	$175,500	$26,325	15%	
Maintenance & Protection of Traffic	$7,500	$1,125	15%	
36" Outlet Diversion Pipe	$117,906	$17,686	15%	
Port Monmouth Peir Extension	$75,000	$11,250	15%	
SUBTOTAL	$379,606			
CONTINGENCY		$56,941		
RELOCATIONS TOTAL				$437,000
FISH AND WILDLIFE FACILITIES				
Wildlife Facilities				
Habitat & Feeding Facilities				
Wetland Mitigation Site Work				
Construction	$412,854	$41,285	10%	
Excavation	$675,738	$67,574	10%	
Disposal	$299,492	$29,949	10%	
Plantings	$76,800	$7,680	10%	
Erosion and Sediment Control	$70,294	$7,029	10%	
Wetland Monitoring	$15,000	$2,400	16%	
SUBTOTAL	$1,550,178			
CONTINGENCY		$155,918		
FISH AND WILDLIFE TOTAL				$1,706,000

TABLE C4 (CONTINUED)
TOTAL FIRST COST - PORT MONMOUTH REACH
RARITAN BAY & SANDY HOOK BAY COMBINED FLOOD CONTROL & SHORE PROTECTION STUDY
LEVEE/FLOODWALL ELEVATION 13.0;DUNE ELEVATION 15.0

DESCRIPTION	AMOUNT	CONTINGENCY AMOUNT	CONT. %	TOTAL
LEVEES AND FLOODWALLS				
Levees				
Mob, Demob & Preparatory Work	$83,720	$12,558	15%	
Drainage				
Drainage Structures				
Primary Outlet NO.C1	$88,299	$13,245	15%	
Secondary Outlet NO.C1-1	$33,324	$4,999	15%	
Primary Outlet NO.C2	$91,507	$13,726	15%	
Secondary Outlet NO.C2-1	$32,047	$4,807	15%	
Secondary Outlet NO.C2-2	$30,508	$4,576	15%	
Secondary Outlet NO.C2-3	$31,363	$4,704	15%	
Secondary Outlet NO.C2-4	$35,851	$5,378	15%	
Secondary Outlet NO.C2-5	$31,255	$4,688	15%	
Primary Outlet NO.C3	$162,021	$24,303	15%	
Secondary Outlet NO.C3-1	$32,151	$4,823	15%	
Secondary Outlet NO.C3-2	$32,215	$4,832	15%	
Secondary Outlet NO.C3-3	$33,407	$5,011	15%	
Secondary Outlet NO.C3-4	$32,981	$4,947	15%	
Secondary Outlet NO.C3-5	$32,172	$4,826	15%	
36" Outlet Port Monmouth Road	$19,838	$2,976	15%	
Pews Creek Primary Outlet	$128,489	$19,273	15%	

TABLE C4 (CONTINUED)
TOTAL FIRST COST - PORT MONMOUTH REACH
RARITAN BAY & SANDY HOOK BAY COMBINED FLOOD CONTROL & SHORE PROTECTION STUDY
LEVEE/FLOODWALL ELEVATION 13.0;DUNE ELEVATION 15.0

DESCRIPTION	AMOUNT	CONTINGENCY AMOUNT	CONT. %	TOTAL
Concrete				
Concrete	$762,447	$76,245	10%	
Reinforcement	$432,666	$43,267	10%	
Sheet Pile (steel)	$1,334,855	$133,486	10%	
Support Piles	$78,329	$7,833	10%	
Pump Stations				
Comptons Creek (C3) 60 CFS	$1,127,085	$169,063	15%	
Pews Creek 120 CFS Pump Station	$1,367,360	$205,104	15%	
Storm Gate Structure				
40' x 20' Storm Gate Structure	$3,728,000	$745,600	20%	
Permanent Access Roads & Parking				
Road Closure Gates				
4' x 40' Closure Gate (Marina Road)	$181,926	$36,385	20%	
9' x 40' Closure Gate (Campbell Avenue)	$485,136	$97,027	20%	
8' x 40' Closure Gate (Broadway)	$424,494	$84,899	20%	
Associated General Items				
Site Work				
Excavation, Stripping	$129,137	$12,914	10%	
Excavation Common	$153,611	$15,361	10%	
Embankment Fill, Common	$990,770	$99,077	10%	
Embankment Fill, Impervious	$938,070	$93,807	10%	
Embankment Fill, pervious	$144,997	$14,500	10%	
Filter Fabric	$107,881	$10,788	10%	
Topsoil & Seeding	$395,351	$39,535	10%	
Clearing & Grubbing	$212,109	$21,211	10%	
Ditch Lining	$229,246	$22,925	10%	
Maintenance & Portection of traffic	$77,500	$7,750	10%	
Dewatering	$130,240	$13,024	10%	
SUBTOTAL	$14,362,358			
CONTINGENCY		$2,089,471		
TOTAL LEVEES AND FLOODWALLS:				$16,451,829

TABLE C4 (CONTINUED)
TOTAL FIRST COST - PORT MONMOUTH REACH
RARITAN BAY & SANDY HOOK BAY COMBINED FLOOD CONTROL & SHORE PROTECTION STUDY
LEVEE/FLOODWALL ELEVATION 13.0;DUNE ELEVATION 15.0

DESCRIPTION	AMOUNT	CONTINGENCY AMOUNT	CONT. %	TOTAL
Beach Replenishment				
Beach Replenishment				
Mob, Demob for Dredging	$450,000	$67,500	15%	
Mob, Demob for Trucking	$0	$0	15%	
Mob, Demob for Associated Work	$23,000	$3,450	15%	
Pipeline Dredging				
Dredging & Placement	$2,668,544	$400,282	15%	
Trucking & Placement	$0	$0	15%	
Associated General Items				
Dune Grass	$40,870	$6,131	15%	
Sand Fence	$17,802	$2,670	15%	
Dune Overwalk	$69,500	$10,425	15%	
SUBTOTAL	$3,269,716			
CONTINGENCY		$490,457		
TOTAL BEACH REPLENISHMENT:				$3,760,174
Sub-Total Constru n Cost				$23,228,003
Engineering & Design	$4,400,000	$450,000	10.2%	$4,850,000
Construction Management	$1,449,940	$210,241	14.5%	$1,660,182
TOTAL PROJECT FIRST COST				$29,738,185

TABLE C5
TOTAL FIRST COST - PORT MONMOUTH REACH
RARITAN BAY & SANDY HOOK BAY COMBINED FLOOD CONTROL & SHORE PROTECTION STUDY
LEVEE/FLOODWALL ELEVATION 15.2; DUNE ELEVATION 17.0

DESCRIPTION	AMOUNT	CONT. %	TOTAL
LANDS AND DAMAGES			
Pre-Authorization Planning			
Develop Aquisition Schedule			
Permanent Easement	$545,731	25%	
Temporary Easement	$40,612	25%	
Severance Damages	$50,737	25%	
All Others			
Mitigation Aquisition	$58,880	25%	
Enhancement Aquisition	$0	25%	
Survey, Appraisal & Admin.	$85,000	25%	
SUBTOTAL	$780,960		
CONTINGENCY			
LANDS AND DAMAGES TOTAL			$976,000
RELOCATIONS			
Roads, Construction Activity			
Mob & Demob	$7,400	15%	
Road Raising			
Port Monmouth Road Raising	$201,500	15%	
N.J State Highway # 36	$360,000	15%	
Maintenance & Protection of Traffic	$15,000	15%	
36" Outlet Diversion Pipe	$117,906	15%	
Port Monmouth Pier Extension	$75,000	15%	
SUBTOTAL	$776,806		
CONTINGENCY			
RELOCATIONS TOTAL			$893,000
FISH AND WILDLIFE FACILITIES			
Wildlife Facilities			
Habitat & Feeding Facilities			
Wetland Mitigation Site Work			
Construction	$412,854.00	10%	
Excavation	$675,738.00	10%	
Disposal	$299,492.00	10%	
Plantings	$76,800.00	10%	
Erosion and Sediment Control	$70,294.00	10%	
Wetland Monitoring	$15,000	16%	
SUBTOTAL	$1,550,178		
CONTINGENCY			
FISH AND WILDLIFE TOTAL			$1,706,000

TABLE C5 (CONTINUED)
TOTAL FIRST COST - PORT MONMOUTH REACH
RARITAN BAY & SANDY HOOK BAY COMBINED FLOOD CONTROL & SHORE PROTECTION STUDY
LEVEE/FLOODWALL ELEVATION 15.2; DUNE ELEVATION 17.0

DESCRIPTION	AMOUNT	CONT. %	TOTAL
LEVEES AND FLOODWALLS			
Levees			
Mob, Demob & Preparatory Work	$83,720	15%	
Drainage			
Drainage Structures			
Primary Outlet NO.C1	$88,299	15%	
Secondary Outlet NO.C1-1	$33,324	15%	
Primary Outlet NO.C2	$91,507	15%	
Secondary Outlet NO.C2-1	$32,047	15%	
Secondary Outlet NO.C2-2	$30,508	15%	
Secondary Outlet NO.C2-3	$31,363	15%	
Secondary Outlet NO.C2-4	$35,851	15%	
Secondary Outlet NO.C2-5	$31,255	15%	
Primary Outlet NO.C3	$162,021	15%	
Secondary Outlet NO.C3-1	$32,151	15%	
Secondary Outlet NO.C3-2	$32,215	15%	
Secondary Outlet NO.C3-3	$33,407	15%	
Secondary Outlet NO.C3-4	$32,981	15%	
Secondary Outlet NO.C3-5	$32,172	15%	
36" Outlet Port Monmouth Road	$19,838	15%	
Pews Creek Primary Outlet	$128,489	15%	

TABLE C5 (CONTINUED)
TOTAL FIRST COST - PORT MONMOUTH REACH
RARITAN BAY & SANDY HOOK BAY COMBINED FLOOD CONTROL & SHORE PROTECTION STUDY
LEVEE/FLOODWALL ELEVATION 15.2; DUNE ELEVATION 17.0

DESCRIPTION	AMOUNT	CONT. %	TOTAL
Concrete			
Concrete	$1,112,869	10%	
Reinforcement	$631,522	10%	
Sheet Pile (steel)	$2,190,931	10%	
Support Piles	$78,329	10%	
Pump Stations			
2-30 CFS Pump Station	$1,127,085	15%	
120 CFS Pump Station	$1,367,360	15%	
Storm Gate Structure			
40' x 22' Storm Gate Structure	$4,138,080	20%	
Permanent Access Roads & Parking			
Road Closure Gates			
4' x 40' Closure Gate (Marina Road)	$315,338	20%	
9' x 40' Closure Gate (Campbell Avenue)	$618,548	20%	
8' x 40' Closure Gate (Broadway)	$557,906	20%	
Associated General Items			
Site Work			
Excavation, Stripping	$152,087	10%	
Excavation Common	$168,913	10%	
Embankment Fill, Common	$1,544,724	10%	
Embankment Fill, Impervious	$1,103,220	10%	
Embankment Fill, pervious	$174,670	10%	
Filter Fabric	$139,261	10%	
Topsoil & Seeding	$459,875	10%	
Clearing & Grubbing	$232,102	10%	
Ditch Lining	$245,005	10%	
Maintenance & Protection of Traffic	$77,500	10%	
Dewatering	$141,579	10%	
SUBTOTAL	$17,508,053		
CONTINGENCY			
TOTAL LEVEES AND FLOODWALLS:			$19,993,125

TABLE C5 (CONTINUED)
TOTAL FIRST COST - PORT MONMOUTH REACH
RARITAN BAY & SANDY HOOK BAY COMBINED FLOOD CONTROL & SHORE PROTECTION STUDY
LEVEE/FLOODWALL ELEVATION 15.2; DUNE ELEVATION 17.0

DESCRIPTION	AMOUNT	CONT. %	TOTAL
Beach Replenishment			
Beach Replenishment			
Mob, Demob for Dredging	$450,000	15%	
Mob, Demob for Trucking	$0	15%	
Mob, Demob for Associated Work	$23,000	15%	
Pipeline Dredging			
Dredging & Placement	$2,744,263	15%	
Trucking & Placement	$0	15%	
Associated General Items			
Dune Grass	$52,930	15%	
Sand Fence	$17,802	15%	
Dune Overwalk	$69,500	15%	
SUBTOTAL	$3,357,495		
CONTINGENCY			
TOTAL BEACH REPLENISHMENT:			$3,861,119
Sub-Total Construction Cost			$27,429,244
Engineering & Design	$4,400,000	10.2%	$4,850,000
Construction Management	$1,701,089	14.5%	$1,947,747
TOTAL PROJECT FIRST COST			$34,226,991

C11. **Unit Costs** Unit costs for material and equipment were developed and based on: the Unit Price Book (UPB) associated with MCACES, current New Jersey DOT bid unit costs (adjusted appropriately for the size of the project, construction period, inflation and profit), actual costs and productions on projects and construction similar in nature, contact with manufacturers, dealers, distributors, and material suppliers in the vicinity of the proposed project. Current labor rates for the northern New Jersey area and cost estimating judgement based on experience.

C12. **Cost Curves** Cost curves based on the detailed design development for the Green Brook Flood Project for similar quantity features were utilized to develop estimates for the interior drainage pump stations. A generalized cost curve, depicting cost versus pumping capacity, was developed based on similar designs for the Green Brook Flood Control Project. This curve was adjusted for this project and used to estimate the costs of the interior pump stations on the project.

C13. **Lump Sum Items** Based on experience, certain items of cost such as mobilization and demobilization and maintenance and protection of traffic were assigned a "lump sum" cost. These items were estimated in this way due to the multiplicity of activities required to accomplish each of these items.

C14. **Market Research** To accurately estimate unit prices for individual work items, manufacturers, distributors, vendors and suppliers, and state agencies were contacted for price information on materials and types of construction. When more than one source of information or price quote was obtained for a single item, the average cost was calculated and used in the MCACES estimate.

C15. **Labor Rates** The labor rates for the estimate were taken from the prevailing Davis-Bacon wage rates for the State of New Jersey for building, heavy, and highway construction as detailed in General Decision Number NJ 960003. The wage rate data was received in detail, listed by counties, and published on June 19, 1998. Wage rates were reviewed and averages were calculated for use in the development of the project area MCACES for Monmouth County for each trade listing. These average labor and fringe benefit costs were input into the MCACES system in the labor rates database PORTMON.

C16. **Contingencies** As stated in ER 1110-2-1302 (31 Mar 94), the goal in contingency development is to identify the uncertainty associated with an item of work or task, forecast the risk/cost relationship, and assign a value to this task that would limit the cost risk to an acceptable degree of confidence. Consideration has been given to the level of detail available at the current stage of planning for which this cost estimate has been prepared.

C17. Contingencies vary throughout the estimate. They increase where the level of design detail is not as complete. Consideration must be given to the details available at each stage of planning, design, or construction for which a cost estimate is being prepared. Based on the current level of design development for the project, the following general contingency factors (%) were used.

- Lands and damages 25% - Final design for major real estate requirements.
- Relocations 15% - Road raising based on preliminary detailed design, final design will be accomplished in plans and specs stage.
- Storm sector gate 20% - No site specific design; cost based on detailed design of similar gate and size used elsewhere (New Orleans).
- Pump Stations 15% - No site specific design, but cost based on a detailed design of pump stations with similar capacities used for the Green Brook Flood Control Project.

- Beach Replenishment 15% - Final design developed, but subject to quantity increases due to moderate erosion that is anticipated to occur between survey and construction.
- Interior Drainage 15% - Cost based on preliminary detailed design, final detailed design will be accomplished in the Design Memo stage to follow.
- Levees and Floodwalls 10% - Final detailed design. Very little change anticipated in plans and specs stage.
- Fish & Wildlife mitigation 10% - Final detailed design. Very little change anticipated in plans and specs stage.

Estimates of Project Features

C18. **Lands & Damages** In order to construct the proposed flood damage reduction plan, the non-Federal sponsor will be required to provide lands and easements and rights-of-way. The extent and value of the lands required for project implementation are outlined in Appendix J – Real Estate.

C19. **Fish and Wildlife Mitigation** Environmental impacts are addressed by mitigation plans which compensate for losses associated with project implementation. A conceptual mitigation plan has been developed to identify sites, which could be used to fully replace lost habitat. This conceptual plan forms the basis of the mitigation component of the project cost. Fish and Wildlife issues associated with the project are fully outlined in the Draft Environmental Impact Statement.

C20. **Levees and Floodwalls** Levees and floodwalls comprise the majority of the line of protection in the Comptons Creek and Pews Creek areas and represent one of the most significant construction features of the project.

C21. **Levees** The estimate for the construction of the levees was approached from the viewpoint of heavy earthwork operations characterized by large dozers, loaders, and rollers.

C22. Productivity considerations were based on the relative placement area configuration of the embankments, compaction, placement criteria, and the distance of truck-delivered borrow from off-site and project stockpiles, access, haul roads, entrances, proximity to streams and construction easements.

C23. The inspection trenches were assumed to extend the full length of each levee with consideration given to intrusion of ground water into these trenches. A contingency was allocated for remediating possible unsuitable material after proof-rolling the levee subgrade. It has been assumed that 10% of the excavated material from the inspection trench will be unsuitable for reuse and must be trucked away. However, the topsoil from stripping operations, as well as the balance of the excavated material from the inspection trench, can be reused and stockpiled near enough to the levees to generally not warrant trucking.

C24. The levees will be constructed with a core section composed of impervious soils to restrict seepage through and beneath the levee under surge conditions. The core section will be 8 feet wide and will extend from the top of the levee to a depth of approximately 5 feet below grade for the entire length of the levee. In order to control seepage velocities and preserve the structural integrity of the levee, a 12-inch thick stone blanket at grade surrounded by geotextile will be constructed from the impervious core to the landward toe of the levee. In this fashion seepage under the levee will be collected and directed to the drainage ditch along the toe of the levee in a controlled fashion. The impervious core will consist of uniform silts (<200 sieve) and supplied from local borrow pits (within 20 miles of site). The stone blanket will consist of one inch diameter clean stone supplied from local quarries (within 20 miles of site).

The Pews Creek levee will be constructed early in the 2-year construction period to allow for anticipated foundation settlement to reach its maximum prior to project completion. This will allow for providing a stable required crest elevation.

C25. **Floodwalls** There are two types of floodwalls on the project: Type A walls are typical cast-in-place concrete cantilever retaining walls with timber pile foundation supports (T-walls), and Type B walls consist of a steel sheet pile PU6 foundation and a cast-in-place concrete topping wall (I-wall). Approximately 150 linear feet of Type A wall will be constructed adjacent to the storm sector gate at Pews Creek. Approximately 1,250 linear feet of Type B floodwalls are to be constructed in areas which are basically too narrow to allow the construction of levees along Comptons Creek. In addition, approximately 2,205 linear feet Type B walls are also proposed along the north side of the new Port Monmouth Road from Pews Creek east to a transition levee at the beginning of the dune.

C26. Construction access for the Type A wall at Pews Creek will be adjacent to the existing levee in East Keansburg from an access ramp off the New Port Monmouth Road on the west side of Pews Creek. The wall will be built during the construction of the sector gate/pump station facility.

C27. Construction access to the Type B floodwall sites along Comptons Creek vary. For construction along Port Monmouth Road, access and work will be from the road right-of-way, however, allowing for continuation of traffic flow. The cost of providing traffic control measures has been added to the cost estimate.

C28. Following the installation of steel sheeting for the Type B floodwalls, backhoes will excavate for the wall foundations. The excavated material will be removed to an adjacent location for drying and possible reuse as backfill.

C29. Reinforcing steel and formwork will be required for both types of walls. The walls will be backfilled using loaders and compaction will be performed by hand-operated tamping machines, as the tight site will not permit larger equipment to be used.

C30. **Storm Sector (Tidal) Gate** The Pews Creek Storm Gate selection incorporated reviews of Department of the Army Manuals and discussions with the New Orleans District, where gates are used extensively on navigation and flood control projects. Engineering Manual No. 1110-2-2703, "Engineering and Design, Lock Gates and Operating Equipment", was specifically reviewed for information on various gate types and associated advantages and disadvantages both in gate operation and construction. Based on a review of EM 1110-2-2703, it appeared that a Sector Gate would be the most appropriate gate type for use at the Pews Creek site. Though Sector gates have generally higher construction cost due to the need for larger recesses in the gate monolith, they have operational and maintenance advantages over other types of gates. Conversations with the New Orleans District confirmed that Sector Gates operate well under high sediment conditions since they divert sediment away from the gate during closing and opening operations. In addition, Sector gates can be closed under flow conditions, which could be experienced under a storm surge. Other types of gates, such as mitre gates, do not perform well under conditions that may generate a differential hydraulic head. Sector gates also provide maintenance advantages since they can be removed and replaced from the gate monolith in the wet. Other types of gates require cofferdaming and dewatering for gate removal and maintenance.

The estimated cost for the Pews Creek Sector Gate is based on a review and analysis of a Bid Abstract for a similar sector gate known as the Larose Floodgate in the Lafourche Parish, Louisiana. The Larose Floodgate has a total opening width of 56 feet and a total height of 20 feet. These gate dimensions are similar to the anticipated dimensions for the Pews Creek Gate (opening width of 40 feet and total height of 20 to 22 feet). The similarity in the scope of gate construction allows an extrapolation of the Larose Floodgate construction bids to estimate the cost of the Pews Creek gate.

Bid Abstracts for the Larose Floodgate included costs for bid items covering the following aspects of the gate construction:

1. Mobilization and Demobilization
2. Clearing and Grubbing
3. Excavation
4. Random, compacted and impervious backfill
5. Cofferdam
6. Piles – Timber and Steel H-Piles
7. Sheet piling
8. Dewatering
9. Fertilizer & Seed
10. Rip Rap
11. Fenders, Guide Walls and Dolphins
12. Pile Testing
13. Reinforced Concrete
14. Sector Gates
15. Gate Operating Machinery
16. Electrical Work/Generating Unit
17. Cathodic Gate Protection
18. Control Houses

The government estimate and three construction bids were reviewed for applicability to the Pews Creek Gate. Construction items estimated for the Larose Floodgate which are applicable to the Pews Creek Gate construction totaled $3.1 million. This construction estimate equates to a cost of roughly $2,770 per square foot of gate area. This per square foot cost was increased by a factor of 1.4 to adjust for differences in price level between the current period and the date of the Larose Floodgate bids (1983). The square foot cost was also increased by a factor of 1.2 to adjust for differences in labor costs between Louisiana and New Jersey. Application of these factors results in an estimated gate cost of $4,660 per square foot of gate area. This translates to an estimated Pews Creek gate construction cost total of $3,914,000 for the recommended height of 14 ft. NGVD. A 20% contingency has been added to this cost to compensate for design issues not fully defined at this point in time.

Cost estimates for the three considered levels of protection are shown in Table C-6.

TABLE C-6
PORT MONMOUTH FLOOD CONTROL PROJECT
STORM GATE COST DEVELOPMENT
ADJUSTMENTS TO BID ABSTRACTS FROM LAROUSE FLOODGATE, GRAND ISLE LA.

	Bidder Total Services		
Bid Amount	$ 3,744,000		
Deduct 123,000 ft. sq. Conc cell mats	$ 492,000		
Deduct Temporary Fenders	$ 145,000		
Bid Adjusted to project requirements	$ 3,107,000		
Unit Price per ft. sq.	$ 2,774		
Time Adjustment (40%)	$ 1,110		
Location Adjustment (20%)	$ 777		
Adjusted Unit Price per ft. sq.	$ 4,661		
SAY	$ 4,660		
Estimate Cost of Pews Creek Gate			
Pews Creek Gate Costs	Size (sq. ft.)	Unit Price	Gate Cost
Elevation 13 NGVD (40x20)	800	$ 4,660	$ 3,728,000
Elevation 14 NGVD (40x21)	840	$ 4,660	$ 3,914,400
Elevation 15.2 NGVD (40x22.2)	888	$ 4,660	$ 4,138,080

The detailed design of the sector gate will be accomplished in the Feature Design Memorandum Phase.

C31. **Beach Replenishment** The Bayshore plan includes a dune and beach berm cross-section to provide the needed level of protection. The dune and beach will be constructed along a 2,640 ft. length of shoreline. The total fill length is 4,640 ft. including the fill transition sections at the east and west ends of the project. The initial beach fill operation will also include a volume of necessary sand to compensate for the erosion expected to occur over the ten years prior to the first renourishment fill operation. The total initial beach fill volume, including overfill and tolerance fill is estimated at 378,500 c.y. for the selected plan.

C32. **Road Raising** Approximately 550 LF of Port Monmouth Road will be raised from elevation +9.75 NGVD to the design elevation of +14 NGVD where the Comptons Creek levee intersects the road.

C33. **Closure Structures** There are three mitre gate road closure structures utilized in the project. A mitre gate closure consists of swinging gates on hinge pins that seals an opening in a floodwall or a levee. The sequence and methods of construction described in a composite fashion as follows:

- Mobilize – mobilization costs include construction fencing, temporary facilities, supervision, and start-up costs.

- Install traffic controls/detour road – The estimate was priced according to a phased installation to allow for maintenance of traffic, however, the installation of a detour (temporary roadway if necessary) around the work site in order to allow for complete installation in one phase may be prudent. These structures are not well suited for phased construction due to the complex and relatively small foundation area.

- Install erosion/sediment controls – consisting of silt fence around all disturbed areas.

- Demolition – removal and disposal of existing roadway/paving at the closure structure location.

- Piling, sheeting and tie-down installation – installation of piles, sheeting, and tie downs including one pile load test. Anchor piles were estimated for the closure structures with one tie-down per anchored pile.

- Excavate/Haul Spoils – spoils from the closure structure excavations were assumed 100% usable. No tipping fees were included, only transportation to another site within the project. No temporary sheeting was included due to the shallow depth of excavation. Hand excavation was included for excavating near pile locations.

- Concrete construction – it was estimated that the contractor would overdig the grade beam and pile caps and, therefore, require forming at these locations.

- Backfill – included at the perimeter of the grade beams/pile caps.

- Install gate(s) – gates were estimated on a per ton structural steel fabrication basis with allowance for galvanizing all materials. Sealing and operating equipment was estimated by weight and unit, based on available detail. Installation of all embedded items is included in concrete forming and pouring costs.

- Pavement repair and marking – pavement repair at the swinging-gate type structure includes the entire roadway surface across the closure structure. Allowance was made for standard pavement marking and permanent signs.

- Install guide rails – it has been assumed that some type of channeling/guiding device will be required in the immediate vicinity of the closure structures. One hundred feet (100') of guide rail per side has been allowed at each approach to the structure.

- Restore site – once the modifications are completed the site is restored by spreading topsoil, fertilizing, seeding, and removing siltation controls and fencing.

- Demobilize – costs for removing temporary facilities and equipment and materials are included.

- Road closure gate costs were developed from detailed designs and costs developed for the Green Brook Flood Control Project of similar size openings and height.

C34. **Interior Drainage** Work classified as Interior Drainage consists of pumping stations, drainage inlets, pipes and manholes necessary to permit drainage from areas behind levees and floodwalls to drain to the exterior side of the line of protection. This work will generally be done just prior, during, or concurrent with levee construction in the area.

C35. **Pumping Stations** The construction of two interior pump stations is needed to maintain interior flood elevations to an acceptable level during severe storm events. The stations assist in the protection of interior drainage areas and develop a 60 cfs capacity for Comptons Creek and a 120 cfs capacity for Pews Creek. The pump station cost curves were developed from prior projects which had progressed sufficiently in design to allow the development of a comprehensive estimate. Final design will be accomplished in the subsequent Feature Design Memorandum Phase. The Port Monmouth pump stations share similar design features. Their construction and estimate are described as follows:

C36. Generally high ground water levels in the pump station area will prohibit open-cut excavation. Steel sheet cofferdams with structural steel walers and pipe struts are used to facilitate the 20 ft (approx.) deep excavations. Infiltration of water is expected and dewatering by sump pumps or well points are assumed to handle the inflow adequately. This effluent is assumed to be silt laden (particularly after initial excavation) and an allowance for a sediment trap/basin has been made to handle such water.

C37. The slab/ground elevation differential in conjunction with the cofferdam geometry make direct chute placement of foundation slab concrete impractical and placement by pump truck is assumed. The walls, beams and suspended slabs will proceed next and will be labor intensive with the considerable number of mechanical and electrical items to follow. It is anticipated that each pump will be delivered to the jobsite in approximately 3 major pieces. Consideration has been made for assembly and testing time for the pumps and the other mechanical and electrical systems included in the pump stations.

C38. **Main Outlet Pipe and Structures** These outlets consist of large (48" dia.), multiple-pipe drainage structures running transversely through the levees. There are three main outlet pipe and structures along the Comptons Creek levee. These structures include inlet and outlet headwalls, control manholes, motor-controlled sluice gates, trash racks and AD gates. It is assumed that all manhole and outlet structures will be cast in place. Pews Creek gravity outfall consists of two (2) 48" diameter outlet pipes through the floodwall with sluice gates and flap gates.

C39. **Secondary Outlet Pipe and Structures** These are generally small pipes (24" dia.) running transversely through the levees at regular intervals, and include associated flood control structures consisting of inlet and outlet headwalls, control manholes, flood gates, and trash racks. It is assumed that construction of these structures will be sequenced in combination with adjoining levee construction. There are eleven (11) interior outlet pipe and structures along the Comptons Creek levee.

C40. **Drainage Swale Excavation** This is excavation necessary to provide a paved ditch along the toe of the protected side of the levee to direct interior runoff to the outlet pipes and structures.

It is assumed that most of the material from this excavation (90%) is suitable for further use. Approximately 10% must be hauled from the site and disposed of.

Estimates of Additional Costs

C41. **Planning, Engineering and Design** Costs were developed for all activities associated with the planning, engineering, and design effort. These costs include the preparation of a Feature Design Memorandum for Interior Drainage, pre-construction monitoring, and plans and specifications for each construction contract and engineering support through project construction.

C42. **Construction Management** Costs were developed for all construction management activities from pre-award requirements through final contract closeout. This cost was based on approximately 8% of the direct construction cost.

C43. **Interest During Construction** Interest during construction (IDC) is the cost of construction money invested before the beginning of the period of economic analysis and before the accumulation of benefits by the project. IDC costs have been added to the project cost to determine the investment costs. Average annual costs were determined based on investment costs which include IDC. Interest during construction was considered for a 24-month construction period at 6-5/8%.

C44. Planning Guidance Notebook (EP 1105-2-45, Paragraph 2-6, page 2-2) states that costs incurred during the construction period should be increased by adding compound interest at the applicable project discount rate from the date the expenditures are made to the beginning of the period of analysis (base year). For the purposes of this study, construction expenditures were assumed to occur in monthly increments.

Annual Charges

Period of Analysis

C45. It is estimated that the major features of the project consisting of levees, floodwalls, closure structures, pump stations, dune and beachfill would have a useful life expectancy of 50 years.

Interest and Amortization

C46. The interest rate used in converting investment costs to equivalent annual costs is the rate set by the Water Resources Council for the evaluation of Federal Government water resources projects. This rate has been set at 6.5/8% for FY2000.

Amortization is the financial or economic process of recovering an investment in a project. The amortization period is the period of time assumed or selected for economic recovery of the net investment in a project. When combined, interest and amortization become the capital recovery factor which, when applied to project costs, will result in the annual cost of the project investment.

C47. **Monitoring** Shore protection projects are periodically monitored to assist in the planning of volumetric requirements of nourishment operations. The monitoring at Port Monmouth includes beach profile surveys with sediment sampling and a report every other year, and aerial photographs every year. The coastal monitoring cost is estimated to average approximately $51,100 annually over the 50-year period.

C48. It has been determined that additional monitoring will be necessary for the first three years following construction to establish environmental impacts. The costs are estimated $42,100 annually. Details of monitoring requirements are outlined in Appendix H – Monitoring.

C49. **Renourishment** Periodic renourishment is required to protect the integrity of the design dune and beach from the effects of long-term erosion and sea level rise. Renourishment at Port Monmouth will occur every 10 years over the period of analysis. Because of the fill limited quantity of 127,345 c.y., renourishment operations will be performed by trucking fill from an upland source. The annualized maintenance cost is estimated to be $169,000 over the period of analysis, see Table C-7. In addition to periodic renourishment, emergency beachfill may be necessary after significant coastal storms. The annual emergency beach fill cost calculations are shown in Table C-8.

TABLE C-7
ANNUAL COST ESTIMATE
PORT MONMOUTH, NJ – BEACH NOURISHMENT
(16 Ft. Dune Elevation Plan)

YEAR	FUTURE WORK	PRESENT WORTH FACTOR	PRESENT WORTH
0	$0	1.00000	$0
10	$2,384,136	0.52651	$1,255,280
20	$2,384,136	0.27722	$660,922
30	$2,384,136	0.14596	$347,984
40	$2,384,136	0.07685	$183,218
Sum of Present Worths			$2,447,404
Total Amount Cost			**$168,978**

TABLE C-8
ANNUAL EMERGENCY FILL COST CALCULATION

RETURN INTERVAL (YRS)	FREQUENCY (EVENTS/YR)	FREQUENCY INTERVAL	PERMANENT LOSS FACTOR	EROSION VOLUME (YD/FT)	EMERGENCY FILL (YD/FT)	EMERGENCY FILL COST ($/FT)	AVRAGE EMERGENCY FILL COST	AVERAGE EMERGENCY FILL COST
10	0.10		0.16	10.1	1.62	$32.32		
		0.05					$198,567	$9,928
20	0.05		0.22	11.4	2.51	$50.16		
		0.03					$278,911	$8,367
50	0.02		0.27	13.3	3.59	$71.82		
		0.01					$528,365	$5,284
100	0.01		0.60	14.4	8.64	$172.80		
		0.01					$1,148,856	$5,744
200	0.01		1.00	15.1	15.05	$301.00		
TOTAL ANNUAL COST								**$29,324**

FILL LENGTH: 3,302 FT.

NOTE: COST BASED ON $20/CY FOR TRUCKED EMERGENCY FILL, $25,000 MOB/DEMOB, NO OVERFILL, 12% E&D AND S&E, AND 10% CONTINGENCY EROSION VOLUME BASED ON EDUNE RESULTS FOR DESIGN PROFILE.

PERMANENT LOSS FACTOR TAKEN AS 0.60 FOR STORMS JUST PRIOR TO FAILURE EVENT TO ACCOUNT FOR DUNE OVERTOPPING. PERMANENT LOSS FACTOR TAKEN AS 1.0 IF THE DUNE IS COMPROMISED BY STORM. AVERAGE EMERGENCY FILL INCLUDES $154,332 FOR THE REPLACEMENT OF ASSOCIATED GENERAL ITEMS.

C50. **Rehabilitation** Significant portions of the overall project's components such as levees, beach berm, dune and drainage facilities are subject to damage from storms exceeding the design levels. The cost of repair after various flood events was weighted by their expected probability of occurrence to determine average annual rehabilitation costs.

C51. **Major Levee Rehabilitation Costs** During some extreme storm events, overtopping of the line of protection may result in significant damage to the levee and associated facilities. The cost of restoring or rehabilitating the project features after such an event has been evaluated and included in the economic assessment. The primary features subject to damage during an extreme event are; the levee earthwork, drainage outlets within the levees, and electrical/mechanical equipment at the storm gate and pump stations. Based on investigations conducted during the Green Brook Study, damage to the levee earthwork was assumed to be 70% of the initial cost. Repairs to interior drainage outlets within the levee were estimated to be equal to the initial construction cost of approximately $700,000. Repairs to pump stations were estimated to be $250,000 per station, approximately twice the cost of mechanical equipment, while repair of the storm gate was estimated to cost $500,000. The frequency of such repairs has been evaluated based on the expected frequency of overtopping determined from the HEC FDA flood damage simulations. The expected frequency of overtopping, which incorporates the impact of flood stage uncertainty, was determined to be 0.9% annually for the 13 ft. NGVD levee, 0.6% annually for the 14 ft. NGVD levee, and 0.5% annually for the 15.2 ft. NGVD levee. The major rehabilitation costs for the levee and associated facilities are in Table C-9.

TABLE C-9
MAJOR REHAB COST
PORT MONMOUTH LEVEE/DRAINAGE

Plan/Levee Elevation	Earthwork Cost	Earthwork Damage	Drainage Repair	Pump Station Repair	Gate Repair	Total Repair (Including 10% Conting, 7% E&D, 7% SA)	Probability	Annual Rehab Cost
13	3550000	2485000	700000	500000	500000	5247990	0.009	47200
14	3950000	2765000	700000	500000	500000	5599110	0.006	33600
15.2	4550000	3185000	700000	500000	500000	6125790	0.005	30600

Operation & Maintenance, and Replacements

C52. Charges attributed to the operation and maintenance (O&M) of the flood control project consist of annualized replacement costs, repair anticipated energy charges, and labor charges for the care and cleaning of project facilities. Project components requiring routine care include; the storm gate, levees and floodwalls, interior drainage closure and manhole structures, pump stations, beach dune grass and sand fence.

C53. The major mechanical equipment within the storm gate and the interior drainage pump stations have anticipated life expectancies of 30 years. The cost of periodic equipment replacement has been estimated, annualized over the 50-year period of analysis and incorporated into the O&M charge. In addition, electric power requirements based on the anticipated frequency of pump station and storm gate operation have been added to the project' annual operation charge.

C54. The selected plan is predicted to increase shoaling in Pews Creek by an estimated 4,700 c.y./yr. Shoaling in Comptons Creek is predicted to increase by 1,000 c.y./yr. This will increase the maintenance dredging requirements in the two creeks. The average annual channel maintenance cost is estimated to increase $40,000/yr.

Annual Costs

C55. Annual Costs. The annual charges include the annualized first cost with interest during construction, annualized renourishments at 10-year intervals, major rehabilitation costs on an annualized basis to restore the design beach profile after major storm events between nourishment operations, annual operations and maintenance costs of the flood control improvements and annualized monitoring cost. Annual project costs for the recommended plan (i.e. 14 foot NGVD levee/floodwall; 16 foot NGVD dune) are presented in Table C-10. Annual costs for two alternative levels of protection are presented in Tables C-11 and C-12.

C56. A summary of annual costs is presented in Table C-13.

TABLE C10 - RECOMMENDED PLAN
TOTAL ANNUAL COST - PORT MONMOUTH REACH
RARITAN BAY & SANDY HOOK BAY COMBINED FLOOD CONTROL & SHORE PROTECTION STUDY
LEVEE/FLOODWALL ELEVATION 14.0; DUNE ELEVATION 16.0

Total First Cost	$31,383,000
Interest During Construction (a)	$3,654,000
Total Investment Cost	$35,037,000

Annualized Investment Cost (b)	$2,419,100
Annualized Renourishment (c)	$169,000
Annual Federal Inspection Cost	$3,000
Annualized Emergency Beach Fill Costs	$29,300
Annual Dune Maintenance	$5,600
Annual Levee\Floodwall Maintenance (d)	$24,600
Potential Increase in Channel Maintenance, Dredging	$40,000
Monitoring	$103,200
Annual Rehabilitation Costs	$33,600
Interior Drainage O&M and Replacement Equipment	$17,300
Area No.C3 Pump Station O&M	$17,760
Storm Gate O&M	$34,100
Pews Creek Pump Station O&M	$34,450
Subtotal O&M	$133,810
Total Annual Cost	$2,931,010

(a) i = 6 5/8% for all funds expended

(b) For 50 yr.period of analysis.
(c) 127000 c.y. every 10 years for the 50 year period of analysis; @15/cy; $25,000 mob/demob; 10% contingency; 12% E&D, S&A.
(d) Based on $2.29 / l.f.

TABLE C11
TOTAL ANNUAL COST - PORT MONMOUTH REACH
RARITAN BAY & SANDY HOOK BAY COMBINED FLOOD CONTROL & SHORE PROTECTION STUDY
LEVEE/FLOODWALL ELEVATION 13.0;DUNE ELEVATION 15.0

Total First Cost	$29,738,185
Interest During Construction (a)	$3,530,000
Total Investment Cost	$33,268,185

Annualized Investment Cost (b)	$2,296,957
Annualized Renourishment (c)	$166,000
Annual Federal Inspection Cost	$3,000
Annualized Emergency Beach Fill Costs	$48,600
Annual Dune Maintenance	$5,600
Annual Levee\Floodwall Maintenance (d)	$24,600
Potential Increase in Channel Maintenance, Dredging	$40,000
Monitoring	$103,200
Annual Rehabilitation Costs	$47,200
Interior Drainage O&M and Replacement Equipment	$17,300
Area No.C3 Pump Station O&M	$17,760
Tidal Gate O&M	$34,100
Pews Creek Pump Station O&M	$34,450
Subtotal O&M	$133,810
Total Annual Cost	$2,838,767

(a) i = 6 5/8% for all funds expended

(c) For 50 yr.period of analysis
(d) 127000 c.y. every 10 year for the 50 year period of analysis; @$15/cy; $25,000 Mob and Demob; 10% Contingency; 12% E&D, S&A.
(e) Based on $2.29 / l.f.

BCR Interest Rev June 00.xls

TABLE C12
TOTAL ANNUAL COST - PORT MONMOUTH REACH
RARITAN BAY & SANDY HOOK BAY COMBINED FLOOD CONTROL & SHORE PROTECTION STUDY
LEVEE/FLOODWALL ELEVATION 15.2; DUNE ELEVATION 17.0

Total First Cost	$34,226,991
Interest During Construction (a)	$4,372,000
Total Investment Cost	$38,598,991

Annualized Investment Cost (b)	$2,665,015
Annualized Renourishment (c)	$166,000
Annual Federal Inspection Cost	$3,000
Annualized Emergency Beach Fill Costs	$28,800
Annual Dune Maintenance	$5,600
Annual Levee\Floodwall Maintenance (d)	$26,800
Potential Increase in Channel Maintenance, Dredging	$40,000
Monitoring	$103,200
Annual Rehabilitation Costs	$30,600
Area No.C3 Pump Station O&M	$17,760
Interior Drainage O&M and Replacement Equipment	$17,300
Storm Gate O&M	$34,100
Pews Creek Pump Station O&M	$34,450
Subtotal O&M	$136,010
Total Annual Cost	$3,172,625

(a) i = 6 5/8% for all funds expended

(b) i = For 50 yr.period of analysis
(c) i = 127000 c.y. every 10 years for the 50 year period; @ $15/cy; $25,000 mob and demob; 10% contingency; 12% E&D, S&A.
(d) i = Based on $2.29 / l.f.

TABLE C-13
ESTIMATED ANNUAL CHARGES OF THE RECOMMENDED PLAN

	TOTAL
FIRST COSTS	$31,383,000
INVESTMENT COSTS	
Interest During Construction	$ 3,654,000
TOTAL INVESTMENT	**$35,037,000**
ANNUAL COSTS	
Interest and Amortization	$2,419,100
Monitoring	$ 103,200
Federal Inspection Cost	$ 3,000
Renourishment	$ 169,000
Rehabilitation	$ 33,600
Potential Increase in Channel Maitenance	$ 40,000
Operation & Maintenance and Replacements	$ 133,810
TOTAL ANNUAL COST	**$2,931,010**

Cost Sharing Responsibilities

General

C56. The basic requirements for the Federal and non-Federal sharing of responsibilities in the construction, operation, and maintenance of Federal water resources projects are set forth in the Water Resources Development Act (WRDA) of 1986 (PL 99-662).

Cost Apportionment

C57. The Water Resources Development Act of 1986, Section 103, which sets forth cost sharing for hurricane and storm damage reduction projects, states that non-Federal interests must operate, maintain, and rehabilitate the project; must provide lands, easements, rights-of-way, relocations, and disposal areas (LERRD). The non-Federal share of the project cost is limited to 35% of the first costs and 50% of those costs assigned to periodic nourishment.

C58. The Federal share of the project's total first cost is $20,398,950. This represents 65% of the total.

C59. The non-Federal share of the estimated total first cost of the proposed project is $10,984,050. The non-Federal cost consists of a number of components including lands, easements, rights-of-way, relocations, and a cash contribution. The non-Federal share represents 35% of the total project first costs. A breakdown of the Federal and non-Federal cost share is shown in Table C-14 – Cost Apportionment.

TABLE 14
COST APPORTIONMENT

COST SHARING	FEDERAL SHARE	NON-FEDERAL SHARE	TOTAL
Cash Contribution	$20,398,950	$9,586,050	$29,985,000
Real Estate Lands & Damages	---	$946,000	$946,000
Relocations (Walkovers, Accessways)	---	$452,000	$452,000
Total First Cost	$20,398,950	$10,984,050	$31,383,000
Periodic Nourishment Cost (per cycle)	$1,192,050	$1,192,050	$2,384,100
Annualized Nourishment Cost, Scheduled Emergency, Post-Storm Rehabilitation	$99,150	$99,150	$198,300
Annual Dune Maintenance	--	$5,600	$5,600
Annual Federal Inspection Costs	$3,000	--	$3,000
Monitoring, Environmental	$33,865	$18,235	$52,100
Monitoring, Coastal	$25,550	$25,550	$51,100
Rehabilitation of Storm Damage Reduction Features	--	$33,600	$33,600
Operations and Maintenance of Levee, Floodwall & Drainage Features	--	$128,210	$128,210
Potential Increase in Channel Maintenance Dredging	--	$40,000	$40,000
Total Annual Nourishment, O&M, and Rehabilitation Costs	$161,550	$350,350	$511,900

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Mon 26 Jun 2000                  Tri-Service Automated Cost Engineering System (TRACES)                      TIME 13:50:22
Eff. Date  05/01/98     PROJECT PORTMN:   Port Monmouth Feasibility Study - Raritan & Sandy Hook Bay
                                     Port Monmouth - Draft Estimate (Non-FDM Area)                       SUMMARY PAGE    1
                               ** PROJECT OWNER SUMMARY - Element (Rounded to 1000's) **

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                                                    QUANTITY UOM    CONTRACT  CONTINGN  ESCALATN  TOTAL COST   UNIT COST
-----------------------------------------------------------------------------------------------------------------------

  01  Pews Creek & Comptons Creek                       1.00 EA   27,558,000 3,825,000         0  31,383,000 31382958.16
                                                                  ----------- --------- --------- -----------
 TOTAL Port Monmouth Feasibility Study                  1.00 EA   27,558,000 3,825,000         0  31,383,000 31382958.16
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                                                    QUANTITY UOM    CONTRACT  CONTINGN  ESCALATN  TOTAL COST   UNIT COS
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  01  Pews Creek & Comptons Creek

  01_1D  Flood Control & Shore Protection                 1.00 EA   27,558,000 3,825,000         0  31,383,000 31382958.16
                                                                   ----------- --------- --------- -----------
   TOTAL Pews Creek & Comptons Creek                      1.00 EA   27,558,000 3,825,000         0  31,383,000 31382958.16
                                                                   ----------- --------- --------- -----------
   TOTAL Port Monmouth Feasibility Study                  1.00 EA   27,558,000 3,825,000         0  31,383,000 31382958.16
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                                                QUANTITY UOM    CONTRACT  CONTINGN  ESCALATN  TOTAL COST  UNIT COST
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 01  Pews Creek & Comptons Creek

 01_1D  Flood Control & Shore Protection

 01_1D.01  Lands and Damages                                       757,000   189,000         0     946,000
 01_1D.02  Relocations                                             393,000    59,000         0     452,000
 01_1D.06  Fish & Wildlife Facilities                            1,550,000   156,000         0   1,706,000
 01_1D.11  Levees and Floodwalls                                10,995,000 1,497,000         0  12,492,000
 01_1D.13  Pumping Plant                                         2,494,000   374,000         0   2,869,000
 01_1D.15  Floodway Control-Diversion Struc                      2,119,000   379,000         0   2,498,000
 01_1D.17  Beach Replenishment                                   3,313,000   497,000         0   3,810,000
 01_1D.30  Planning, Engineering and Design                      4,400,000   450,000         0   4,850,000
 01_1D.31  Construction Management                               1,537,000   223,000         0   1,760,000
                                                               ----------- --------- --------- -----------
     TOTAL Flood Control & Shore Protection        1.00 EA    27,558,000 3,825,000         0  31,383,000 31382958.16
                                                               ----------- --------- --------- -----------
     TOTAL Pews Creek & Comptons Creek             1.00 EA    27,558,000 3,825,000         0  31,383,000 31382958.16
                                                               ----------- --------- --------- -----------
     TOTAL Port Monmouth Feasibility Study         1.00 EA    27,558,000 3,825,000         0  31,383,000 31382958.16
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                                                          QUANTITY UOM   CONTRACT  CONTINGN  ESCALATN  TOTAL COST  UNIT COS
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  01  Pews Creek & Comptons Creek

  01_1D  Flood Control & Shore Protection

  01_1D.01  Lands and Damages

  01_1D.01.03  Real Estate Costs - Local                                   672,000   168,000         0     840,000
  01_1D.01.04  Real Estate Costs - Admin.                                   85,000    21,000         0     106,000
                                                                       ----------- --------- --------- -----------
        TOTAL Lands and Damages                                            757,000   189,000         0     946,000


  01_1D.02  Relocations

  01_1D.02.01  Roads                                                       193,000    29,000         0     222,000
  01_1D.02.02  Drainage                                                    125,000    19,000         0     144,000
  01_1D.02.03  Port_Monmouth Pier Extension                                 75,000    11,000         0      86,000
                                                                       ----------- --------- --------- -----------
        TOTAL Relocations                                                  393,000    59,000         0     452,000


  01_1D.06  Fish & Wildlife Facilities

  01_1D.06.03  Wildlife Facilities &Sanctuaries                          1,550,000   156,000         0   1,706,000
                                                                       ----------- --------- --------- -----------
        TOTAL Fish & Wildlife Facilities                                 1,550,000   156,000         0   1,706,000


  01_1D.11  Levees and Floodwalls

  01_1D.11.01  Levees                                     7070.00 LF     7,863,000 1,184,000         0   9,047,000     1279.68
  01_1D.11.02  Floodwalls                                 3605.00 LF     3,131,000   313,000         0   3,445,000      955.51
                                                                       ----------- --------- --------- -----------
        TOTAL Levees and Floodwalls                                     10,995,000 1,497,000         0  12,492,000


  01_1D.13  Pumping Plant

  01_1D.13.01  Comptons Creek (C3) 60 cfs                                1,127,000   169,000         0   1,296,000
  01_1D.13.02  Pews Creek 120cfs Pump Station                            1,367,000   205,000         0   1,572,000
                                                                       ----------- --------- --------- -----------
        TOTAL Pumping Plant                                              2,494,000   374,000         0   2,869,000


  01_1D.15  Floodway Control-Diversion Struc

  01_1D.15.01  Roadway Closure Structure                                 1,273,000   255,000         0   1,528,000
  01_1D.15.02  Interior Drainage                                           845,000   125,000         0     970,000
                                                                       ----------- --------- --------- -----------
        TOTAL Floodway Control-Diversion Struc                           2,119,000   379,000         0   2,498,000
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	QUANTITY	UOM	CONTRACT	CONTINGN	ESCALATN	TOTAL COST	UNIT COST
01_1D.17 Beach Replenishment							
01_1D.17.00 Beach Replenishment			3,313,000	497,000	0	3,810,000	
TOTAL Beach Replenishment			3,313,000	497,000	0	3,810,000	
01_1D.30 Planning, Engineering and Design			4,400,000	450,000	0	4,850,000	
01_1D.31 Construction Management			1,537,000	223,000	0	1,760,000	
TOTAL Flood Control & Shore Protection	1.00	EA	27,558,000	3,825,000	0	31,383,000	31382958.16
TOTAL Pews Creek & Comptons Creek	1.00	EA	27,558,000	3,825,000	0	31,383,000	31382958.16
TOTAL Port Monmouth Feasibility Study	1.00	EA	27,558,000	3,825,000	0	31,383,000	31382958.16

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                                                              QUANTITY UOM    CONTRACT  CONTINGN  ESCALATN  TOTAL COST   UNIT COST
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  01  Pews Creek & Comptons Creek

  01_1D  Flood Control & Shore Protection

  01_1D.01  Lands and Damages

  01_1D.01.03  Real Estate Costs - Local                                        672,000   168,000         0    840,000
  01_1D.01.04  Real Estate Costs - Admin.                                        85,000    21,000         0    106,000
                                                                            ----------- --------- --------- ----------
           TOTAL Lands and Damages                                          757,000   189,000         0    946,000


  01_1D.02  Relocations

  01_1D.02.01  Roads

  01_1D.02.01_001  Roads - Raise Port Monmouth Rd                           193,000    29,000         0    222,000
                                                                            ----------- --------- --------- ----------
           TOTAL Roads                                                      193,000    29,000         0    222,000


  01_1D.02.02  Drainage

  01_1D.02.02_ 01  36" Diversion Pipe                                       119,000    18,000         0    137,000
  01_1D.02.02_ 39  Road Surfacing                                             6,000     1,000         0      7,000
                                                                            ----------- --------- --------- ----------
           TOTAL Drainage                                                   125,000    19,000         0    144,000

  01_1D.02.03  Port Monmouth Pier Extension                                  75,000    11,000         0     86,000
                                                                            ----------- --------- --------- ----------
           TOTAL Relocations                                                393,000    59,000         0    452,000


  01_1D.06  Fish & Wildlife Facilities

  01_1D.06.03  Wildlife Facilities &Sanctuaries

  01_1D.06.03_ 99  General Associated Items                               1,550,000   156,000         0  1,706,000
                                                                            ----------- --------- --------- ----------
           TOTAL Wildlife Facilities &Sanctuaries                         1,550,000   156,000         0  1,706,000
                                                                            ----------- --------- --------- ----------
           TOTAL Fish & Wildlife Facilities                               1,550,000   156,000         0  1,706,000


  01_1D.11  Levees and Floodwalls

  01_1D.11.01  Levees

  01_1D.11.01_ 01  Mob, Demob & Preparatory Work                            163,000    23,000         0    185,000
  01_1D.11.01_ 05  Site Work                                              3,787,000   379,000         0  4,166,000
```

	QUANTITY UOM	CONTRACT	CONTINGN	ESCALATN	TOTAL COST	UNIT COST
01_1D.11.01_ 06 Storm Gate Structure		3,914,000	783,000	0	4,696,000	
TOTAL Levees	7070.00 LF	7,863,000	1,184,000	0	9,047,000	1279.68
01_1D.11.02 Floodwalls						
01_1D.11.02_ 02 Site Work		1,705,000	170,000	0	1,875,000	
01_1D.11.02_ 03 Construct Floodwall		1,427,000	143,000	0	1,569,000	
TOTAL Floodwalls	3605.00 LF	3,131,000	313,000	0	3,445,000	955.51
TOTAL Levees and Floodwalls		10,995,000	1,497,000	0	12,492,000	
01_1D.13 Pumping Plant						
01_1D.13.01 Comptons Creek (C3) 60 cfs		1,127,000	169,000	0	1,296,000	
01_1D.13.02 Pews Creek 120cfs Pump Station		1,367,000	205,000	0	1,572,000	
TOTAL Pumping Plant		2,494,000	374,000	0	2,869,000	
01_1D.15 Floodway Control-Diversion Struc						
01_1D.15.01 Roadway Closure Structure						
01_1D.15.01_ 01 Marina Road		243,000	49,000	0	291,000	
01_1D.15.01_ 02 Campbell Avenue		546,000	109,000	0	655,000	
01_1D.15.01_ 03 Broadway		485,000	97,000	0	582,000	
TOTAL Roadway Closure Structure		1,273,000	255,000	0	1,528,000	
01_1D.15.02 Interior Drainage						
01_1D.15.02_ 00 Sitework		26,000	2,000	0	27,000	
01_1D.15.02_ 03 Interim Outlet Pipe		47,000	7,000	0	54,000	
01_1D.15.02_ 04 Drainage Structure & Pipe Incids		400,000	60,000	0	460,000	
01_1D.15.02_ 05 Metals		368,000	55,000	0	424,000	
01_1D.15.02_ 39 Road Surfacing		5,000	1,000	0	5,000	
TOTAL Interior Drainage		845,000	125,000	0	970,000	
TOTAL Floodway Control-Diversion Struc		2,119,000	379,000	0	2,498,000	
01_1D.17 Beach Replenishment						
01_1D.17.00 Beach Replenishment						

** PROJECT OWNER SUMMARY - Bid Item (Rounded to 1000's) **

		QUANTITY	UOM	CONTRACT	CONTINGN	ESCALATN	TOTAL COST	UNIT COST
01_1D.17.00_ 01	Mob,Demob for Dredging			450,000	68,000	0	518,000	
01_1D.17.00_ 02	Mob,Demob for Associated Work			23,000	3,000	0	26,000	
01_1D.17.00_ 16	Pipeline Dredging			2,706,000	406,000	0	3,112,000	
01_1D.17.00_ 99	Associated General Items			134,000	20,000	0	154,000	
	TOTAL Beach Replenishment			3,313,000	497,000	0	3,810,000	
	TOTAL Beach Replenishment			3,313,000	497,000	0	3,810,000	
01_1D.30	Planning, Engineering and Design			4,400,000	450,000	0	4,850,000	
01_1D.31	Construction Management			1,537,000	223,000	0	1,760,000	
	TOTAL Flood Control & Shore Protection	1.00	EA	27,558,000	3,825,000	0	31,383,000	31382958.16
	TOTAL Pews Creek & Comptons Creek	1.00	EA	27,558,000	3,825,000	0	31,383,000	31382958.16
	TOTAL Port Monmouth Feasibility Study	1.00	EA	27,558,000	3,825,000	0	31,383,000	31382958.16

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                                                    QUANTITY UOM   CONTRACT  CONTINGN  ESCALATN  TOTAL COST  UNIT COST
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   01  Pews Creek & Comptons Creek

   01_1D  Flood Control & Shore Protection

   01_1D.01  Lands and Damages

   01_1D.01.03  Real Estate Costs - Local                         672,000   168,000         0     840,000
   01_1D.01.04  Real Estate Costs - Admin.                         85,000    21,000         0     106,000
                                                              ----------- --------- --------- -----------
                TOTAL Lands and Damages                           757,000   189,000         0     946,000


   01_1D.02  Relocations

   01_1D.02.01  Roads

   01_1D.02.01_001  Roads - Raise Port Monmouth Rd

   01_1D.02.01_001_ 01  Site Work                                  22,000     3,000         0      26,000
   01_1D.02.01_001_ 13  Traffic Control                             9,000     1,000         0      10,000
   01_1D.02.01_001_ 19  Construct Approaches to Subgrade           52,000     8,000         0      60,000
   01_1D.02.01_001_ 39  Road Surfacing                            106,000    16,000         0     122,000
   01_1D.02.01_001_ 99  Associated General Items                    4,000     1,000         0       4,000
                                                              ----------- --------- --------- -----------
                TOTAL Roads - Raise Port Monmouth Rd              193,000    29,000         0     222,000
                                                              ----------- --------- --------- -----------
                TOTAL Roads                                       193,000    29,000         0     222,000


   01_1D.02.02  Drainage

   01_1D.02.02_ 01  36" Diversion Pipe                            119,000    18,000         0     137,000
   01_1D.02.02_ 39  Road Surfacing                                  6,000     1,000         0       7,000
                                                              ----------- --------- --------- -----------
                TOTAL Drainage                                    125,000    19,000         0     144,000

   01_1D.02.03  Port Monmouth Pier Extension                       75,000    11,000         0      86,000
                                                              ----------- --------- --------- -----------
                TOTAL Relocations                                 393,000    59,000         0     452,000


   01_1D.06  Fish & Wildlife Facilities

   01_1D.06.03  Wildlife Facilities &Sanctuaries

   01_1D.06.03_ 99  General Associated Items

   01_1D.06.03_ 99_ 06  Wetland Mitigation                      1,535,000   154,000         0   1,689,000
   01_1D.06.03_ 99_ 07  Wetland Monitoring                         15,000     2,000         0      17,000
                                                              ----------- --------- --------- -----------
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	QUANTITY	UOM	CONTRACT	CONTINGN	ESCALATN	TOTAL COST	UNIT COST
TOTAL General Associated Items			1,550,000	156,000	0	1,706,000	
TOTAL Wildlife Facilities &Sanctuaries			1,550,000	156,000	0	1,706,000	
TOTAL Fish & Wildlife Facilities			1,550,000	156,000	0	1,706,000	
01_1D.11 Levees and Floodwalls							
01_1D.11.01 Levees							
01_1D.11.01_ 01 Mob, Demob & Preparatory Work							
01_1D.11.01_ 01_ 01 Mob & Demob			124,000	19,000	0	143,000	
01_1D.11.01_ 01_ 02 Erosion & Sediment Controls			19,000	2,000	0	21,000	
01_1D.11.01_ 01_ 04 Temp Access Road			20,000	2,000	0	22,000	
TOTAL Mob, Demob & Preparatory Work			163,000	23,000	0	185,000	
01_1D.11.01_ 05 Site Work							
01_1D.11.01_ 05_ 01 Clear & Grub	23.70	ACR	220,000	22,000	0	242,000	10221.
01_1D.11.01_ 05_ 02 Excavation, Common	15762.30	CY	246,000	25,000	0	270,000	17.
01_1D.11.01_ 05_ 04 Embankment, Common	63194.80	CY	1,228,000	123,000	0	1,351,000	21.38
01_1D.11.01_ 05_ 05 Riprap/Ditch Lining	3953.70	CY	233,000	23,000	0	257,000	64.95
01_1D.11.01_ 05_ 06 Embankment, Impervious	40582.00	CY	1,014,000	101,000	0	1,116,000	27.49
01_1D.11.01_ 05_ 07 Embankment, pervious	7681.00	CY	158,000	16,000	0	174,000	22.64
01_1D.11.01_ 05_ 09 Stripping	20339.17	CY	140,000	14,000	0	154,000	7.56
01_1D.11.01_ 05_ 10 Topsoil and Seeding			424,000	42,000	0	467,000	
01_1D.11.01_ 05_ 15 Fabric			123,000	12,000	0	135,000	
TOTAL Site Work			3,787,000	379,000	0	4,166,000	
01_1D.11.01_ 06 Storm Gate Structure							
01_1D.11.01_ 06_ 01 40' x 21' Storm Gate Structure			3,914,000	783,000	0	4,696,000	
TOTAL Storm Gate Structure			3,914,000	783,000	0	4,696,000	
TOTAL Levees	7070.00	LF	7,863,000	1,184,000	0	9,047,000	1279.68
01_1D.11.02 Floodwalls							
01_1D.11.02_ 02 Site Work							
01_1D.11.02_ 02_001 Site Work			1,705,000	170,000	0	1,875,000	
TOTAL Site Work			1,705,000	170,000	0	1,875,000	

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                                                        QUANTITY UOM    CONTRACT  CONTINGN  ESCALATN  TOTAL COST  UNIT COST
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  01_1D.11.02_ 03  Construct Floodwall

  01_1D.11.02_ 03_031  Formwork                                          334,000    33,000         0     367,000
  01_1D.11.02_ 03_032  Reinforcing Steel                                 516,000    52,000         0     568,000
  01_1D.11.02_ 03_033  Place Concrete                   4305.00 CY       577,000    58,000         0     634,000     147.33
                                                                     ----------- --------- --------- -----------
               TOTAL Construct Floodwall                               1,427,000   143,000         0   1,569,000
                                                                     ----------- --------- --------- -----------
               TOTAL Floodwalls                         3605.00 LF     3,131,000   313,000         0   3,445,000     955.51
                                                                     ----------- --------- --------- -----------
               TOTAL Levees and Floodwalls                            10,995,000 1,497,000         0  12,492,000


  01_1D.13  Pumping Plant

  01_1D.13.01  Comptons Creek (C3) 60 cfs                              1,127,000   169,000         0   1,296,000
  01_1D.13.02  Pews Creek 120cfs Pump Station                          1,367,000   205,000         0   1,572,000
                                                                     ----------- --------- --------- -----------
               TOTAL Pumping Plant                                     2,494,000   374,000         0   2,869,000


  01_1D.15  Floodway Control-Diversion Struc

   1_1D.15.01  Roadway Closure Structure

  01_1D.15.01_ 01  Marina Road

  01_1D.15.01_ 01_ 01  4'x40' Closure Gate(Marina Rd.)    160.00 SF      243,000    49,000         0     291,000    1819.26
                                                                     ----------- --------- --------- -----------
               TOTAL Marina Road                                         243,000    49,000         0     291,000


  01_1D.15.01_ 02  Campbell Avenue

  01_1D.15.01_ 02_ 01  9'x40' Closure Gate                360.00 SF      546,000   109,000         0     655,000    1819.26
                                                                     ----------- --------- --------- -----------
               TOTAL Campbell Avenue                                     546,000   109,000         0     655,000


  01_1D.15.01_ 03  Broadway

  01_1D.15.01_ 03_ 01  8'x40' Closure Gate                320.00 SF      485,000    97,000         0     582,000    1819.26
                                                                     ----------- --------- --------- -----------
               TOTAL Broadway                                            485,000    97,000         0     582,000
                                                                     ----------- --------- --------- -----------
               TOTAL Roadway Closure Structure                         1,273,000   255,000         0   1,528,000


  01_1D.15.02  Interior Drainage

  01_1D.15.02_ 00  Sitework
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	QUANTITY UOM	CONTRACT	CONTINGN	ESCALATN	TOTAL COST	UNIT COST
01_1D.15.02_ 00_ 02 Excavation		15,000	0	0	15,000	
01_1D.15.02_ 00_ 10 Grass Seeding		11,000	2,000	0	12,000	
TOTAL Sitework		26,000	2,000	0	27,000	
01_1D.15.02_ 03 Interim Outlet Pipe						
01_1D.15.02_ 03_ 03 RCP Pipe		47,000	7,000	0	54,000	
TOTAL Interim Outlet Pipe		47,000	7,000	0	54,000	
01_1D.15.02_ 04 Drainage Structure & Pipe Incids		400,000	60,000	0	460,000	
01_1D.15.02_ 05 Metals						
01_1D.15.02_ 05_ 06 Gates and Trash Racks		368,000	55,000	0	424,000	
TOTAL Metals		368,000	55,000	0	424,000	
01_1D.15.02_ 39 Road Surfacing		5,000	1,000	0	5,000	
TOTAL Interior Drainage		845,000	125,000	0	970,000	
TOTAL Floodway Control-Diversion Struc		2,119,000	379,000	0	2,498,000	
01_1D.17 Beach Replenishment						
01_1D.17.00 Beach Replenishment						
01_1D.17.00_ 01 Mob,Demob for Dredging		450,000	68,000	0	518,000	
01_1D.17.00_ 02 Mob,Demob for Associated Work		23,000	3,000	0	26,000	
01_1D.17.00_ 16 Pipeline Dredging						
01_1D.17.00_ 16_ 01 Dredging & Placement	378461.00 CY	2,706,000	406,000	0	3,112,000	8.22
TOTAL Pipeline Dredging		2,706,000	406,000	0	3,112,000	
01_1D.17.00_ 99 Associated General Items						
01_1D.17.00_ 99_ 01 Dune Grass	7.00 ACR	47,000	7,000	0	54,000	7705.00
01_1D.17.00_ 99_ 02 Sand Fence	2300.00 LF	18,000	3,000	0	20,000	8.90
01_1D.17.00_ 99_ 03 Dune Overwalk	5.00 EA	70,000	10,000	0	80,000	15985.00
TOTAL Associated General Items		134,000	20,000	0	154,000	
TOTAL Beach Replenishment		3,313,000	497,000	0	3,810,000	

		QUANTITY	UOM	CONTRACT	CONTINGN	ESCALATN	TOTAL COST	UNIT COST
	TOTAL Beach Replenishment			3,313,000	497,000	0	3,810,000	
01_1D.30	Planning, Engineering and Design			4,400,000	450,000	0	4,850,000	
01_1D.31	Construction Management			1,537,000	223,000	0	1,760,000	
	TOTAL Flood Control & Shore Protection	1.00	EA	27,558,000	3,825,000	0	31,383,000	31382958.16
	TOTAL Pews Creek & Comptons Creek	1.00	EA	27,558,000	3,825,000	0	31,383,000	31382958.16
	TOTAL Port Monmouth Feasibility Study	1.00	EA	27,558,000	3,825,000	0	31,383,000	31382958.16

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                                       QUANTITY UOM      DIRECT  FIELD OH  HOME OFC    PROFIT      BOND  TOTAL COST  UNIT COS
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01  Pews Creek & Comptons Creek            1.00 EA  25,086,000 1,199,000   324,000   856,000    94,000  27,558,000 27558172.56
                                                    ----------- --------- --------- --------- --------- -----------
    Port Monmouth Feasibility Study        1.00 EA  25,086,000 1,199,000   324,000   856,000    94,000  27,558,000 27558172.56

  Contingency                                                                                             3,825,000
                                                                                                         -----------
    SUBTOTAL                                                                                             31,383,000
  Escalation                                                                                                      0
                                                                                                         -----------
    TOTAL INCL OWNER COSTS                                                                               31,383,000
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Tri-Service Automated Cost Engineering System (TRACES)
PROJECT PORTMN: Port Monmouth Feasibility Study - Raritan & Sandy Hook Bay
Port Monmouth - Draft Estimate (Non-FDM Area)
** PROJECT INDIRECT SUMMARY - Sub Elem (Rounded to 1000's) **

	QUANTITY	UOM	DIRECT	FIELD OH	HOME OFC	PROFIT	BOND	TOTAL COST	UNIT COST
01 Pews Creek & Comptons Creek									
01_1D Flood Control & Shore Protect	1.00	EA	25,086,000	1,199,000	324,000	856,000	94,000	27,558,000	27558172.56
TOTAL Pews Creek & Comptons Creek	1.00	EA	25,086,000	1,199,000	324,000	856,000	94,000	27,558,000	27558172.56
TOTAL Port Monmouth Feasibility Stu	1.00	EA	25,086,000	1,199,000	324,000	856,000	94,000	27,558,000	27558172.56
Contingency								3,825,000	
SUBTOTAL								31,383,000	
Escalation								0	
TOTAL INCL OWNER COSTS								31,383,000	

** PROJECT INDIRECT SUMMARY - Feature (Rounded to 1000's) **

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                                          QUANTITY UOM      DIRECT  FIELD OH  HOME OFC    PROFIT      BOND  TOTAL COST   UNIT COST
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01  Pews Creek & Comptons Creek

01_1D  Flood Control & Shore Protect

01_1D.01  Lands and Damages                              757,000         0         0         0         0     757,000
01_1D.02  Relocations                                    344,000    24,000     6,000    17,000     2,000     393,000
01_1D.06  Fish & Wildlife Facilities                   1,550,000         0         0         0         0   1,550,000
01_1D.11  Levees and Floodwalls                        8,742,000 1,092,000   295,000   780,000    85,000  10,995,000
01_1D.13  Pumping Plant                                2,494,000         0         0         0         0   2,494,000
01_1D.15  Floodway Control-Diversion                   1,949,000    83,000    22,000    59,000     6,000   2,119,000
01_1D.17  Beach Replenishment                          3,313,000         0         0         0         0   3,313,000
01_1D.30  Planning, Engineering and                    4,400,000         0         0         0         0   4,400,000
01_1D.31  Construction Management                      1,537,000         0         0         0         0   1,537,000
                                                     ----------- --------- --------- --------- --------- -----------
    TOTAL Flood Control & Shore Prot      1.00 EA   25,086,000 1,199,000   324,000   856,000    94,000  27,558,000 27558172.56
                                                     ----------- --------- --------- --------- --------- -----------
    TOTAL Pews Creek & Comptons Cree      1.00 EA   25,086,000 1,199,000   324,000   856,000    94,000  27,558,000 27558172.56
                                                     ----------- --------- --------- --------- --------- -----------
    TOTAL Port Monmouth Feasibility       1.00 EA   25,086,000 1,199,000   324,000   856,000    94,000  27,558,000 27558172.56

 Contingency                                                                                            3,825,000
                                                                                                      -----------
    SUBTOTAL                                                                                           31,383,000
 Escalation                                                                                                     0
                                                                                                      -----------
    TOTAL INCL OWNER COSTS                                                                             31,383,000
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	QUANTITY	UOM	DIRECT	FIELD OH	HOME OFC	PROFIT	BOND	TOTAL COST	UNIT COST
01 Pews Creek & Comptons Creek									
01_1D Flood Control & Shore Protect									
01_1D.01 Lands and Damages									
01_1D.01.03 Real Estate Costs - Loc			672,000	0	0	0	0	672,000	
01_1D.01.04 Real Estate Costs - Adm			85,000	0	0	0	0	85,000	
TOTAL Lands and Damages			757,000	0	0	0	0	757,000	
01_1D.02 Relocations									
01_1D.02.01 Roads			170,000	11,000	3,000	8,000	1,000	193,000	
01_1D.02.02 Drainage			99,000	12,000	3,000	9,000	1,000	125,000	
01_1D.02.03 Port Monmouth Pier Exte			75,000	0	0	0	0	75,000	
TOTAL Relocations			344,000	24,000	6,000	17,000	2,000	393,000	
D.06 Fish & Wildlife Facilities									
01_1D.06.03 Wildlife Facilities &Sa			1,550,000	0	0	0	0	1,550,000	
TOTAL Fish & Wildlife Facilit			1,550,000	0	0	0	0	1,550,000	
01_1D.11 Levees and Floodwalls									
01_1D.11.01 Levees	7070.00	LF	6,252,000	781,000	211,000	558,000	61,000	7,863,000	1112.22
01_1D.11.02 Floodwalls	3605.00	LF	2,490,000	311,000	84,000	222,000	24,000	3,131,000	868.65
TOTAL Levees and Floodwalls			8,742,000	1,092,000	295,000	780,000	85,000	10,995,000	
01_1D.13 Pumping Plant									
01_1D.13.01 Comptons Creek (C3) 60			1,127,000	0	0	0	0	1,127,000	
01_1D.13.02 Pews Creek 120cfs Pump			1,367,000	0	0	0	0	1,367,000	
TOTAL Pumping Plant			2,494,000	0	0	0	0	2,494,000	
01_1D.15 Floodway Control-Diversion									
01_1D.15.01 Roadway Closure Structu			1,273,000	0	0	0	0	1,273,000	
1D.15.02 Interior Drainage			675,000	83,000	22,000	59,000	6,000	845,000	
TOTAL Floodway Control-Divers			1,949,000	83,000	22,000	59,000	6,000	2,119,000	

** PROJECT INDIRECT SUMMARY - Sub Feat (Rounded to 1000's) **

	QUANTITY UOM	DIRECT	FIELD OH	HOME OFC	PROFIT	BOND	TOTAL COST	UNIT COST
01_1D.17 Beach Replenishment								
01_1D.17.00 Beach Replenishment		3,313,000	0	0	0	0	3,313,000	
TOTAL Beach Replenishment		3,313,000	0	0	0	0	3,313,000	
01_1D.30 Planning, Engineering and		4,400,000	0	0	0	0	4,400,000	
01_1D.31 Construction Management		1,537,000	0	0	0	0	1,537,000	
TOTAL Flood Control & Shore P	1.00 EA	25,086,000	1,199,000	324,000	856,000	94,000	27,558,000	27558172.56
TOTAL Pews Creek & Comptons C	1.00 EA	25,086,000	1,199,000	324,000	856,000	94,000	27,558,000	27558172.56
TOTAL Port Monmouth Feasibili	1.00 EA	25,086,000	1,199,000	324,000	856,000	94,000	27,558,000	27558172.56
Contingency							3,825,000	
SUBTOTAL							31,383,000	
Escalation							0	
TOTAL INCL OWNER COSTS							31,383,000	

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                                              QUANTITY UOM     DIRECT  FIELD OH  HOME OFC    PROFIT      BOND  TOTAL COST  UNIT COST
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01  Pews Creek & Comptons Creek

01_1D  Flood Control & Shore Protect

01_1D.01  Lands and Damages

01_1D.01.03  Real Estate Costs - Loc                        672,000         0         0         0         0     672,000
01_1D.01.04  Real Estate Costs - Adm                         85,000         0         0         0         0      85,000
                                                        ----------- --------- --------- --------- --------- -----------
           TOTAL Lands and Damages                          757,000         0         0         0         0     757,000

01_1D.02  Relocations

01_1D.02.01  Roads

01_1D.02.01_001  Roads - Raise Port                         170,000    11,000     3,000     8,000     1,000     193,000
                                                        ----------- --------- --------- --------- --------- -----------
           TOTAL Roads                                      170,000    11,000     3,000     8,000     1,000     193,000

   1D.02.02  Drainage

01_1D.02.02_ 01  36" Diversion Pipe                          95,000    12,000     3,000     8,000     1,000     119,000
01_1D.02.02_ 39  Road Surfacing                               5,000     1,000         0         0         0       6,000
                                                        ----------- --------- --------- --------- --------- -----------
           TOTAL Drainage                                    99,000    12,000     3,000     9,000     1,000     125,000

01_1D.02.03  Port Monmouth Pier Exte                         75,000         0         0         0         0      75,000
                                                        ----------- --------- --------- --------- --------- -----------
           TOTAL Relocations                                344,000    24,000     6,000    17,000     2,000     393,000

01_1D.06  Fish & Wildlife Facilities

01_1D.06.03  Wildlife Facilities &Sa

01_1D.06.03_ 99  General Associated                       1,550,000         0         0         0         0   1,550,000
                                                        ----------- --------- --------- --------- --------- -----------
           TOTAL Wildlife Facilities                      1,550,000         0         0         0         0   1,550,000
                                                        ----------- --------- --------- --------- --------- -----------
           TOTAL Fish & Wildlife Fac                      1,550,000         0         0         0         0   1,550,000

01_1D.11  Levees and Floodwalls

01_1D.11.01  Levees

    D.11.01_ 01  Mob, Demob & Prepar                        130,000    16,000     4,000    12,000     1,000     163,000
  _1D.11.01_ 05  Site Work                                3,011,000   376,000   102,000   269,000    29,000   3,787,000
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                                       QUANTITY UOM      DIRECT  FIELD OH  HOME OFC    PROFIT      BOND  TOTAL COST  UNIT COS
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01_1D.11.01_ 06  Storm Gate Structur                  3,112,000   389,000   105,000   278,000    30,000   3,914,000
                                                    ----------- --------- --------- --------- --------- -----------
          TOTAL Levees                  7070.00 LF    6,252,000   781,000   211,000   558,000    61,000   7,863,000     1112.22


01_1D.11.02  Floodwalls

01_1D.11.02_ 02  Site Work                            1,356,000   169,000    46,000   121,000    13,000   1,705,000
01_1D.11.02_ 03  Construct Floodwall                  1,134,000   142,000    38,000   101,000    11,000   1,427,000
                                                    ----------- --------- --------- --------- --------- -----------
          TOTAL Floodwalls              3605.00 LF    2,490,000   311,000    84,000   222,000    24,000   3,131,000      868.65
                                                    ----------- --------- --------- --------- --------- -----------
          TOTAL Levees and Floodwal                   8,742,000 1,092,000   295,000   780,000    85,000  10,995,000


01_1D.13  Pumping Plant

01_1D.13.01  Comptons Creek (C3) 60                   1,127,000         0         0         0         0   1,127,000
01_1D.13.02  Pews Creek 120cfs Pump                   1,367,000         0         0         0         0   1,367,000
                                                    ----------- --------- --------- --------- --------- -----------
          TOTAL Pumping Plant                         2,494,000         0         0         0         0   2,494,000


01_1D.15  Floodway Control-Diversion

01_1D.15.01  Roadway Closure Structu

01_1D.15.01_ 01  Marina Road                            243,000         0         0         0         0     243,000
01_1D.15.01_ 02  Campbell Avenue                        546,000         0         0         0         0     546,000
01_1D.15.01_ 03  Broadway                               485,000         0         0         0         0     485,000
                                                    ----------- --------- --------- --------- --------- -----------
          TOTAL Roadway Closure Str                   1,273,000         0         0         0         0   1,273,000


01_1D.15.02  Interior Drainage

01_1D.15.02_ 00  Sitework                                23,000     1,000         0     1,000         0      26,000
01_1D.15.02_ 03  Interim Outlet Pipe                     37,000     5,000     1,000     3,000         0      47,000
01_1D.15.02_ 04  Drainage Structure                     318,000    40,000    11,000    28,000     3,000     400,000
01_1D.15.02_ 05  Metals                                 293,000    37,000    10,000    26,000     3,000     368,000
01_1D.15.02_ 39  Road Surfacing                           4,000         0         0         0         0       5,000
                                                    ----------- --------- --------- --------- --------- -----------
          TOTAL Interior Drainage                       675,000    83,000    22,000    59,000     6,000     845,000
                                                    ----------- --------- --------- --------- --------- -----------
          TOTAL Floodway Control-Di                   1,949,000    83,000    22,000    59,000     6,000   2,119,000


01_1D.17  Beach Replenishment

01_1D.17.00  Beach Replenishment
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Mon 26 Jun 2000
Eff. Date 05/01/98

Tri-Service Automated Cost Engineering System (TRACES)
PROJECT PORTMN: Port Monmouth Feasibility Study - Raritan & Sandy Hook Bay
Port Monmouth - Draft Estimate (Non-FDM Area)
** PROJECT INDIRECT SUMMARY - Bid Item (Rounded to 1000's) **

		QUANTITY	UOM	DIRECT	FIELD OH	HOME OFC	PROFIT	BOND	TOTAL COST	UNIT COST
01_1D.17.00_ 01	Mob,Demob for Dredg			450,000	0	0	0	0	450,000	
01_1D.17.00_ 02	Mob,Demob for Assoc			23,000	0	0	0	0	23,000	
01_1D.17.00_ 16	Pipeline Dredging			2,706,000	0	0	0	0	2,706,000	
01_1D.17.00_ 99	Associated General			134,000	0	0	0	0	134,000	
	TOTAL Beach Replenishment			3,313,000	0	0	0	0	3,313,000	
	TOTAL Beach Replenishment			3,313,000	0	0	0	0	3,313,000	
01_1D.30	Planning, Engineering and			4,400,000	0	0	0	0	4,400,000	
01_1D.31	Construction Management			1,537,000	0	0	0	0	1,537,000	
	TOTAL Flood Control & Sho	1.00	EA	25,086,000	1,199,000	324,000	856,000	94,000	27,558,000	27558172.56
	TOTAL Pews Creek & Compto	1.00	EA	25,086,000	1,199,000	324,000	856,000	94,000	27,558,000	27558172.56
	TOTAL Port Monmouth Feasi	1.00	EA	25,086,000	1,199,000	324,000	856,000	94,000	27,558,000	27558172.56
	Contingency								3,825,000	
	SUBTOTAL								31,383,000	
	Escalation								0	
	TOTAL INCL OWNER COSTS								31,383,000	

** PROJECT INDIRECT SUMMARY - Assembly (Rounded to 1000's) **

	QUANTITY	UOM	DIRECT	FIELD OH	HOME OFC	PROFIT	BOND	TOTAL COST	UNIT COST
01 Pews Creek & Comptons Creek									
01_1D Flood Control & Shore Protect									
01_1D.01 Lands and Damages									
01_1D.01.03 Real Estate Costs - Loc			672,000	0	0	0	0	672,000	
01_1D.01.04 Real Estate Costs - Adm			85,000	0	0	0	0	85,000	
TOTAL Lands and Damag			757,000	0	0	0	0	757,000	
01_1D.02 Relocations									
01_1D.02.01 Roads									
01_1D.02.01_001 Roads - Raise Port									
01_1D.02.01_001_ 01 Site Work			22,000	0	0	0	0	22,000	
01_1D.02.01_001_ 13 Traffic Control			7,000	1,000	0	1,000	0	9,000	
01_1D.02.01_001_ 19 Construct Appro			52,000	0	0	0	0	52,000	
01_1D.02.01_001_ 39 Road Surfacing			84,000	11,000	3,000	8,000	1,000	106,000	
01_1D.02.01_001_ 99 Associated Gene			4,000	0	0	0	0	4,000	
TOTAL Roads - Raise P			170,000	11,000	3,000	8,000	1,000	193,000	
TOTAL Roads			170,000	11,000	3,000	8,000	1,000	193,000	
01_1D.02.02 Drainage									
01_1D.02.02_ 01 36" Diversion Pipe			95,000	12,000	3,000	8,000	1,000	119,000	
01_1D.02.02_ 39 Road Surfacing			5,000	1,000	0	0	0	6,000	
TOTAL Drainage			99,000	12,000	3,000	9,000	1,000	125,000	
01_1D.02.03 Port Monmouth Pier Exte			75,000	0	0	0	0	75,000	
TOTAL Relocations			344,000	24,000	6,000	17,000	2,000	393,000	
01_1D.06 Fish & Wildlife Facilities									
01_1D.06.03 Wildlife Facilities &Sa									
01_1D.06.03_ 99 General Associated									
01_1D.06.03_ 99_ 06 Wetland Mitigat			1,535,000	0	0	0	0	1,535,000	
01_1D.06.03_ 99_ 07 Wetland Monitor			15,000	0	0	0	0	15,000	

	QUANTITY	UOM	DIRECT	FIELD OH	HOME OFC	PROFIT	BOND	TOTAL COST	UNIT COST
TOTAL General Associa			1,550,000	0	0	0	0	1,550,000	
TOTAL Wildlife Facili			1,550,000	0	0	0	0	1,550,000	
TOTAL Fish & Wildlife			1,550,000	0	0	0	0	1,550,000	
01_1D.11 Levees and Floodwalls									
01_1D.11.01 Levees									
01_1D.11.01_ 01 Mob, Demob & Prepar									
01_1D.11.01_ 01_ 01 Mob & Demob			99,000	12,000	3,000	9,000	1,000	124,000	
01_1D.11.01_ 01_ 02 Erosion & Sedim			15,000	2,000	1,000	1,000	0	19,000	
01_1D.11.01_ 01_ 04 Temp Access Roa			16,000	2,000	1,000	1,000	0	20,000	
TOTAL Mob, Demob & Pr			130,000	16,000	4,000	12,000	1,000	163,000	
01_1D.11.01_ 05 Site Work									
D.11.01_ 05_ 01 Clear & Grub	23.70	ACR	175,000	22,000	6,000	16,000	2,000	220,000	9292.28
D.11.01_ 05_ 02 Excavation, Com	15762.30	CY	195,000	24,000	7,000	17,000	2,000	246,000	15.58
01_1D.11.01_ 05_ 04 Embankment, Com	63194.80	CY	977,000	122,000	33,000	87,000	10,000	1,228,000	19.44
01_1D.11.01_ 05_ 05 Riprap/Ditch Li	3953.70	CY	186,000	23,000	6,000	17,000	2,000	233,000	59.05
01_1D.11.01_ 05_ 06 Embankment, Imp	40582.00	CY	807,000	101,000	27,000	72,000	8,000	1,014,000	25.00
01_1D.11.01_ 05_ 07 Embankment, per	7681.00	CY	126,000	16,000	4,000	11,000	1,000	158,000	20.58
01_1D.11.01_ 05_ 09 Stripping	20339.17	CY	111,000	14,000	4,000	10,000	1,000	140,000	6.87
01_1D.11.01_ 05_ 10 Topsoil and See			337,000	42,000	11,000	30,000	3,000	424,000	
01_1D.11.01_ 05_ 15 Fabric			97,000	12,000	3,000	9,000	1,000	123,000	
TOTAL Site Work			3,011,000	376,000	102,000	269,000	29,000	3,787,000	
01_1D.11.01_ 06 Storm Gate Structur									
01_1D.11.01_ 06_ 01 40' x 21' Storm			3,112,000	389,000	105,000	278,000	30,000	3,914,000	
TOTAL Storm Gate Stru			3,112,000	389,000	105,000	278,000	30,000	3,914,000	
TOTAL Levees	7070.00	LF	6,252,000	781,000	211,000	558,000	61,000	7,863,000	1112.22
01_1D.11.02 Floodwalls									
01_1D.11.02_ 02 Site Work									
D.11.02_ 02_001 Site Work			1,356,000	169,000	46,000	121,000	13,000	1,705,000	
TOTAL Site Work			1,356,000	169,000	46,000	121,000	13,000	1,705,000	

	QUANTITY UOM	DIRECT	FIELD OH	HOME OFC	PROFIT	BOND	TOTAL COST	UNIT COST
01_1D.11.02_ 03 Construct Floodwall								
01_1D.11.02_ 03_031 Formwork		265,000	33,000	9,000	24,000	3,000	334,000	
01_1D.11.02_ 03_032 Reinforcing Ste		410,000	51,000	14,000	37,000	4,000	516,000	
01_1D.11.02_ 03_033 Place Concrete	4305.00 CY	458,000	57,000	15,000	41,000	4,000	577,000	133.94
TOTAL Construct Flood		1,134,000	142,000	38,000	101,000	11,000	1,427,000	
TOTAL Floodwalls	3605.00 LF	2,490,000	311,000	84,000	222,000	24,000	3,131,000	868.65
TOTAL Levees and Floo		8,742,000	1,092,000	295,000	780,000	85,000	10,995,000	
01_1D.13 Pumping Plant								
01_1D.13.01 Comptons Creek (C3) 60		1,127,000	0	0	0	0	1,127,000	
01_1D.13.02 Pews Creek 120cfs Pump		1,367,000	0	0	0	0	1,367,000	
TOTAL Pumping Plant		2,494,000	0	0	0	0	2,494,000	
01_1D.15 Floodway Control-Diversion								
01_1D.15.01 Roadway Closure Structu								
01_1D.15.01_ 01 Marina Road								
01_1D.15.01_ 01_ 01 4'x40' Closure	160.00 SF	243,000	0	0	0	0	243,000	1516.05
TOTAL Marina Road		243,000	0	0	0	0	243,000	
01_1D.15.01_ 02 Campbell Avenue								
01_1D.15.01_ 02_ 01 9'x40' Closure	360.00 SF	546,000	0	0	0	0	546,000	1516.05
TOTAL Campbell Avenue		546,000	0	0	0	0	546,000	
01_1D.15.01_ 03 Broadway								
01_1D.15.01_ 03_ 01 8'x40' Closure	320.00 SF	485,000	0	0	0	0	485,000	1516.05
TOTAL Broadway		485,000	0	0	0	0	485,000	
TOTAL Roadway Closure		1,273,000	0	0	0	0	1,273,000	
01_1D.15.02 Interior Drainage								
01_1D.15.02_ 00 Sitework								

	QUANTITY	UOM	DIRECT	FIELD OH	HOME OFC	PROFIT	BOND	TOTAL COST	UNIT COST
01_1D.15.02_ 00_ 02 Excavation			15,000	0	0	0	0	15,000	
01_1D.15.02_ 00_ 10 Grass Seeding			8,000	1,000	0	1,000	0	11,000	
TOTAL Sitework			23,000	1,000	0	1,000	0	26,000	
01_1D.15.02_ 03 Interim Outlet Pipe									
01_1D.15.02_ 03_ 03 RCP Pipe			37,000	5,000	1,000	3,000	0	47,000	
TOTAL Interim Outlet			37,000	5,000	1,000	3,000	0	47,000	
01_1D.15.02_ 04 Drainage Structure			318,000	40,000	11,000	28,000	3,000	400,000	
01_1D.15.02_ 05 Metals									
01_1D.15.02_ 05_ 06 Gates and Trash			293,000	37,000	10,000	26,000	3,000	368,000	
TOTAL Metals			293,000	37,000	10,000	26,000	3,000	368,000	
01_1D.15.02_ 39 Road Surfacing			4,000	0	0	0	0	5,000	
TOTAL Interior Draina			675,000	83,000	22,000	59,000	6,000	845,000	
TOTAL Floodway Contro			1,949,000	83,000	22,000	59,000	6,000	2,119,000	
01_1D.17 Beach Replenishment									
01_1D.17.00 Beach Replenishment									
01_1D.17.00_ 01 Mob,Demob for Dredg			450,000	0	0	0	0	450,000	
01_1D.17.00_ 02 Mob,Demob for Assoc			23,000	0	0	0	0	23,000	
01_1D.17.00_ 16 Pipeline Dredging									
01_1D.17.00_ 16_ 01 Dredging & Plac	378461.00	CY	2,706,000	0	0	0	0	2,706,000	7.15
TOTAL Pipeline Dredgi			2,706,000	0	0	0	0	2,706,000	
01_1D.17.00_ 99 Associated General									
01_1D.17.00_ 99_ 01 Dune Grass	7.00	ACR	47,000	0	0	0	0	47,000	6700.00
01_1D.17.00_ 99_ 02 Sand Fence	2300.00	LF	18,000	0	0	0	0	18,000	7.74
01_1D.17.00_ 99_ 03 Dune Overwalk	5.00	EA	70,000	0	0	0	0	70,000	13900.00
TOTAL Associated Gene			134,000	0	0	0	0	134,000	
TOTAL Beach Replenish			3,313,000	0	0	0	0	3,313,000	

** PROJECT INDIRECT SUMMARY - Assembly (Rounded to 1000's) **

		QUANTITY	UOM	DIRECT	FIELD OH	HOME OFC	PROFIT	BOND	TOTAL COST	UNIT COS
	TOTAL Beach Replenish			3,313,000	0	0	0	0	3,313,000	
01_1D.30	Planning, Engineering and			4,400,000	0	0	0	0	4,400,000	
01_1D.31	Construction Management			1,537,000	0	0	0	0	1,537,000	
	TOTAL Flood Control &	1.00	EA	25,086,000	1,199,000	324,000	856,000	94,000	27,558,000	27558172.56
	TOTAL Pews Creek & Co	1.00	EA	25,086,000	1,199,000	324,000	856,000	94,000	27,558,000	27558172.56
	TOTAL Port Monmouth F	1.00	EA	25,086,000	1,199,000	324,000	856,000	94,000	27,558,000	27558172.56
	Contingency								3,825,000	
	SUBTOTAL								31,383,000	
	Escalation								0	
	TOTAL INCL OWNER COSTS								31,383,000	

FEBRUARY 2000

RARITAN BAY AND SANDY HOOK BAY, NEW JERSEY
INTERIM FEASIBILITY REPORT
FOR
HURRICANE AND STORM DAMAGE REDUCTION

APPENDIX D

BORROW AREA

page D

APPENDIX D - BORROW AREA APPENDIX

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PROJECT LOCATION AND DESCRIPTION

D-1. The project area consists of approximately 7,000 feet of shoreline in Monmouth County in central New Jersey, extending along the shoreline of Sandy Hook Bay (Figure D-1). Pews Creek and Compton Creek are waterways that represent the western and eastern limits of the project area, respectively. The southern limit of the project area is the existing inland 20-foot NGVD contour line, which lies a short distance south of Route 36. The northern project area limit is the location of the closure depth contour within Sandy Hook Bay. The recommended plan for the Port Monmouth area includes a dune and beach fill section along the Raritan Bay shoreline. This Appendix summarizes efforts to identify and recommend a suitable source of borrow material for initial project construction and beach renourishment over the 50-year project life.

D-2. Pertinent borrow area site information including borrow material grain sizes, potential borrow quantities and distances to the beach fill site were gathered for comparison. Consideration was given to the mode of transport of the borrow material and to the environmental restrictions affecting the dredging and transport of the material.

GEOLOGY

D-3. The following geologic background is presented in the 1960 Raritan Bay and Sandy Hook Bay Cooperative Beach Erosion Control and Interim Hurricane Study (USACE, 1960)(D1).

D-4. "The study area lies within the Coastal Plain Province, which forms the eastern margin of the State of New Jersey. Its surface has a gentle slope to the southeast, generally not exceeding 5 or 6 feet to the mile. The moderate elevation of the Coastal Plain, which rises to 400 feet in some areas but is generally lower than 200 feet, has prevented the streams from cutting valleys of any considerable depth. Throughout the greater portion of the plain, the relief is insignificant and the streams flow in open valleys that lie at only slightly lower levels than the broad, flat divides. The surface of the plain extends eastward with the same gentle slope beneath the Atlantic Ocean for about 100 miles to the end of the continental shelf, where the depth is approximately 100 fathoms. At this point, the ocean bottom drops abruptly to greater depths.

D-5. The Cretaceous Period. The Cretaceous Period resulted in many successive sedimentary formations, each of which were subject to erosion, deposition, submergence and emergence. Realizing that all of New Jersey's geomorphology was determined by weathering and its associated agents, this geological period had great influence on the study area. The resulting Cretaceous formations are composed of unconsolidated sand, clay and green-sand marl (glauconite), which dip 25 to 60 feet per mile to the southeast and have a thickness in places of 500 to 1,000 feet. The sediments rest on a sloping formation of deep seated hard

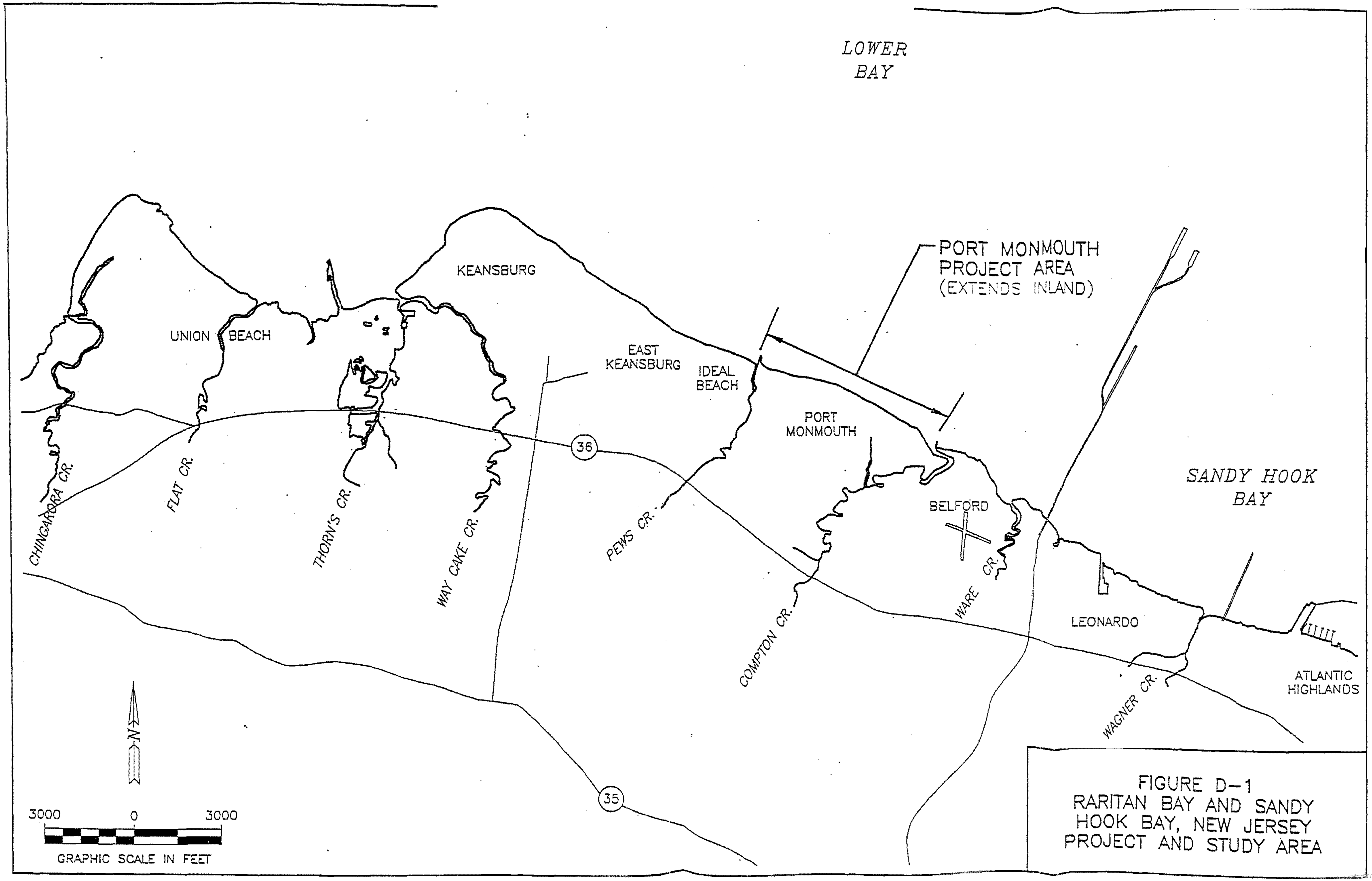


FIGURE D-1
RARITAN BAY AND SANDY HOOK BAY, NEW JERSEY
PROJECT AND STUDY AREA

rocks. The present surface features were most recently determined during the glacial Pleistocene Period and by subsequent erosion.

D-6. Subsurface Geology. The subsurface geology of the Coastal Plain has been determined by study and correlation of well logs and by interpretation of seismic profiles. The Coastal Plain consists of Cretaceous to Recent sediments lapping on the basement material which is composed of crystalline rock with locally infolded or infaulted Triassic sediments. The basement surface slopes at about 75 feet per mile, reaching a depth of more than 6,000 feet near the coast. A semi-consolidated sedimentary formation, varying in thickness to a maximum of about 13,000 feet, rests upon the basement material. An unconsolidated formation, which overlies the semi-consolidated material, consists of approximately equal thickness of the Upper Cretaceous and Tertiary sediments. The Cretaceous sediments are of prime importance in Raritan and Sandy Hook Bays area. The maximum thickness of the sediments (both Cretaceous and Tertiary) is about 4,800 feet near the edge of the continental shelf."

NATIVE BEACH CHARACTERISTICS

D-7. In November 1995, Rogers Surveying, Inc. analyzed sediment samples collected during the 1995 survey of Port Monmouth. Sediment samples were obtained at +12, +6, +3, 0, -2, -6 and -10 feet NGVD elevation (Rogers, 1995)(D2). The samples were collected at profile lines PL-210, Pl-212, PL-214, PL-216, PL-218, and PL-220 (Figure D-2). The results of this analysis are included in Table D-1. The average above tidal mean grain size was 0.39 mm. The sample intertidal composite reported an average mean grain size of 0.43 mm, and the mean grain size for the below tidal composite is 0.55 mm. The total beach composite has a mean grain size of 0.41mm and a phi sorting of 0.7, which is classified as medium sand on the Wentworth Classification system.

D-8. The total beach composite calculated by Rogers Surveying was not used due to the nature of the material. At profile lines PL-216 and PL-218, the material contains pebble and gravel size material that was used in computing the composite grain size. Therefore, a coarser composite mean grain size was produced. After a detailed analysis, it was determined to use samples collected along PL-214 as the most representative material on Port Monmouth Beach. Samples collected at -2.0,0.0,3.0,6.0, and 12.0 feet NGVD were averaged to compute a composite. This composite has a mean grain size of 0.37 mm and a phi sorting of 0.30. The PL-214 beach model was used to compare potential borrow materials to determine their suitability for beach fill.

D-9. In 1960, the Raritan Bay and Sandy Hook Bay Cooperative Beach Erosion Control and Interim Hurricane Study (1960)(D1) conducted sediment sampling directly offshore from Port Monmouth. The results from this analysis are presented in Table D-2Samples were taken at

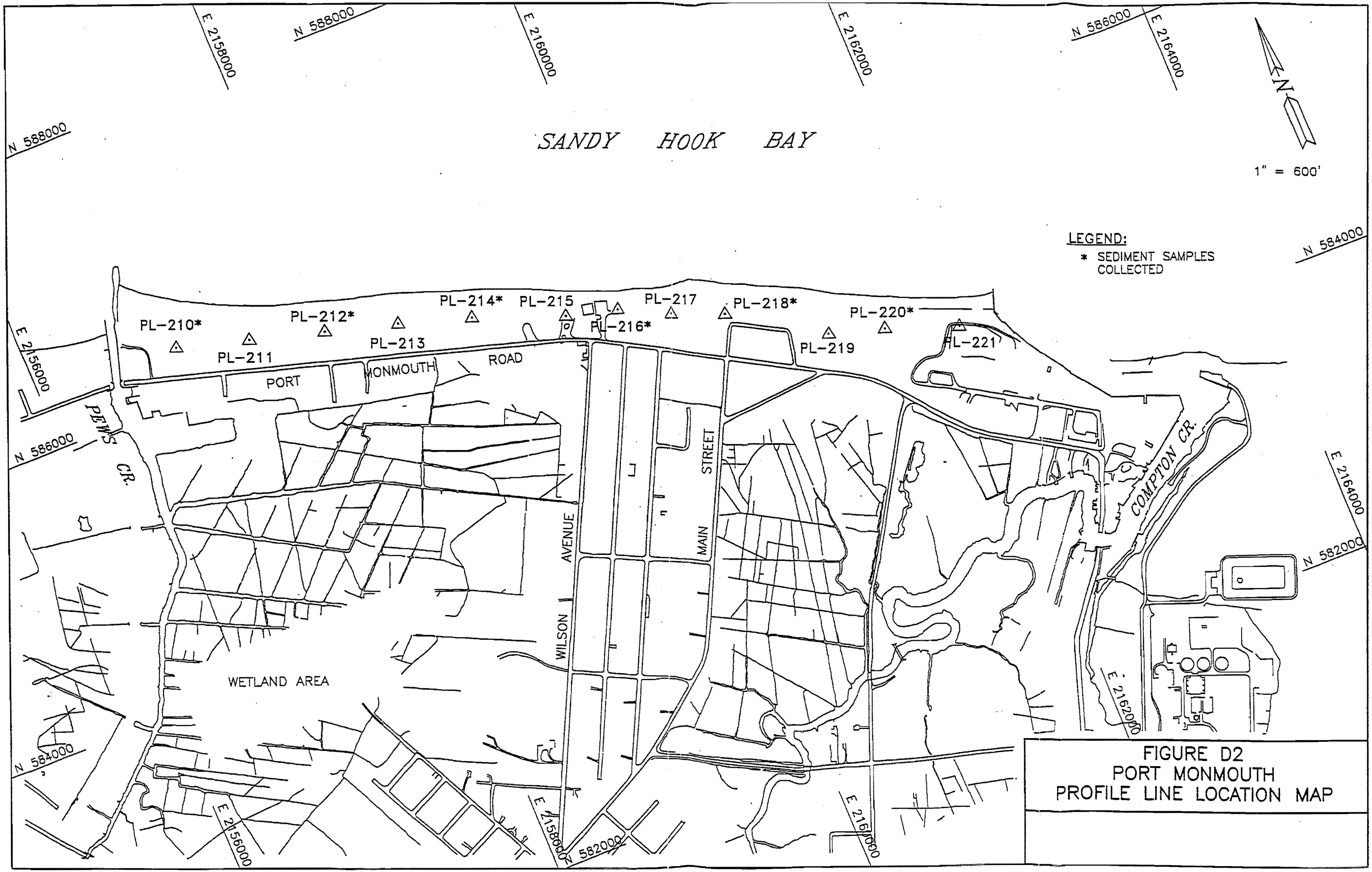


FIGURE D2
PORT MONMOUTH
PROFILE LINE LOCATION MAP

TABLE D-1

NOVEMBER 1995 SEDIMENT SAMPLE CHARACTERISTICS

(ROGERS, INC. 1995) (D2)

	PL-210		PL-212		PL-214		PL-216		PL-218		PL-220		COMPOSITE	
COORDINATES	2156994 E	506346 N	2157955 E	586070 N	2158913 E	5057840 N	2159059 E	505459 N	2160547 E	585185 N	2161504 E	504644 N		
ELEVATION (FEET, NGVD)	MEAN (mm)	PHI SORTING	MEAN (mm)	PHI SORTING	MEAN (mm)	PHI SORTING	MEAN (mm)	PHI SORTING	MEAN (mm)	PHI SORTING	MEAN (mm)	PHI SORTING	MEAN (mm)	PHI SORTING
-10	0.33	0.3	0.41	0.2	0.41	0.4	0.44	0.3	0.44	0.2	0.38	0.2		
-6	0.38	0.4	0.41	0.4	0.44	0.4	0.44	0.5	0.54	0.6	1.07	1.6		
-2	0.44	0.4	0.47	0.3	0.44	0.5	9.05	1.9	0.76	0.5	1.32	1.1		
0	0.44	0.4	0.47	0.3	0.38	0.3	1.32	1.6	0.66	0.5	1.41	1.3		
3	0.33	0.2	0.41	0.2	0.35	0.2	0.38	0.3	0.44	0.3	0.38	0.3		
6	0.38	0.3	0.35	0.1	0.31	0.2	0.41	0.2	0.41	0.3	0.41	0.3		
12	0.35	0.2	0.31	0.2	0.35	0.3	0.38	0.3	0.29	0.1	0.29	0.2		
ABOVE TIDAL	0.38	0.2	0.33	0.2	0.33	0.2	0.44	0.2	0.35	0.3	0.50	0.6	0.35	0.2
INTERTIDAL	0.38	0.3	0.47	0.3	0.41	0.2	2.00	2.2	0.62	0.6	0.87	1.2	0.66	0.9
BELOW TIDAL	0.35	0.4	0.41	0.3	0.44	0.3	0.44	0.4	0.47	0.5	0.50	0.6	0.41	0.5
TOTAL LINE	0.35	0.3	0.41	0.2	0.38	0.3	1.32	1.8	0.50	0.5	0.54	0.7	0.41	0.7

NOTE: PHI < 0.5 IS CONSIDERED WELL SORTED
PHI > 1.0 IS CONSIDERED POORLY SORTED

TABLE D-2

HISTORICAL SEDIMENT DATA AT PORT MONMOUTH

LOCATION	MEDIAN (mm)	PHI SORTING	SKEWNESS	SOURCE
MHW	0.31	1.35	1.08	(USACE, 1960)
MLW	0.95	1.51	1.24	(USACE, 1960)
1,500 - 4,000 FT OFFSHORE (1966 BORROW AREA)	0.28			(NJ, 1966)
~ 1/2 MILE OFFSHORE (DEPTH = 4.1 FT)	0.28	1.30	1.09	(USACE, 1960)
~ 1 MILE OFFSHORE (DEPTH = 12.5 FT)	0.28	1.20	1.05	(USACE, 1960)
AVERAGE	0.42	1.34	1.12	

NOTE:

(USACE, 1960) REPRESENTS AVERAGE VALUES OF LOCAL SAMPLES

(NJ, 1966) VALUES REPRESENT INTERPRETATION OF CORE PRESENTATIONS

MHW, MLW and approximately 2,500 and 6,000 feet from the Port Monmouth shoreline. A comparison indicates that the offshore samples were finer than the onshore samples.

BORROW AREA INVESTIGATION METHODOLOGY

D-10. The primary objective of the borrow area investigation was to identify and delineate sources of sand borrow material in the waters of Sandy Hook Bay and Raritan Bay, for use as beach and dune fill material for the Port Monmouth Shore Protection Project. The criteria a borrow source must satisfy, to be considered as a usable source include; acceptable Ra and Rj factors, contain sufficient volume and located within a reasonable distance from the project shoreline. Figure D-3 shows the locations of potential offshore and navigational channel borrow areas.

D-11. Methods to determine the volume and quality of borrow sediments for the Port Monmouth project included a combination of field and laboratory techniques, as well as utilization of previously existing data. For this study, field investigations included taking approximately 35 miles of seismic data, and obtaining fifteen new 20-ft. long offshore vibracore samples. Laboratory and office procedures included detailed analysis of 1995 vibracores, including vibracore logs, grain-size computation of samples, and compatibility analysis as described in the *Shore Protection Manual* (1984)(D3) between the native beach composite and the offshore core samples. The CERC ACES program (USACE, 1992)(D4) was used to calculate the overfill factor (Ra) and renourishment factor (Rj) for both the 1995 samples, and for core samples taken in prior studies. Computer assisted mapping techniques were used for a quantitative presentation of results and recommendations.

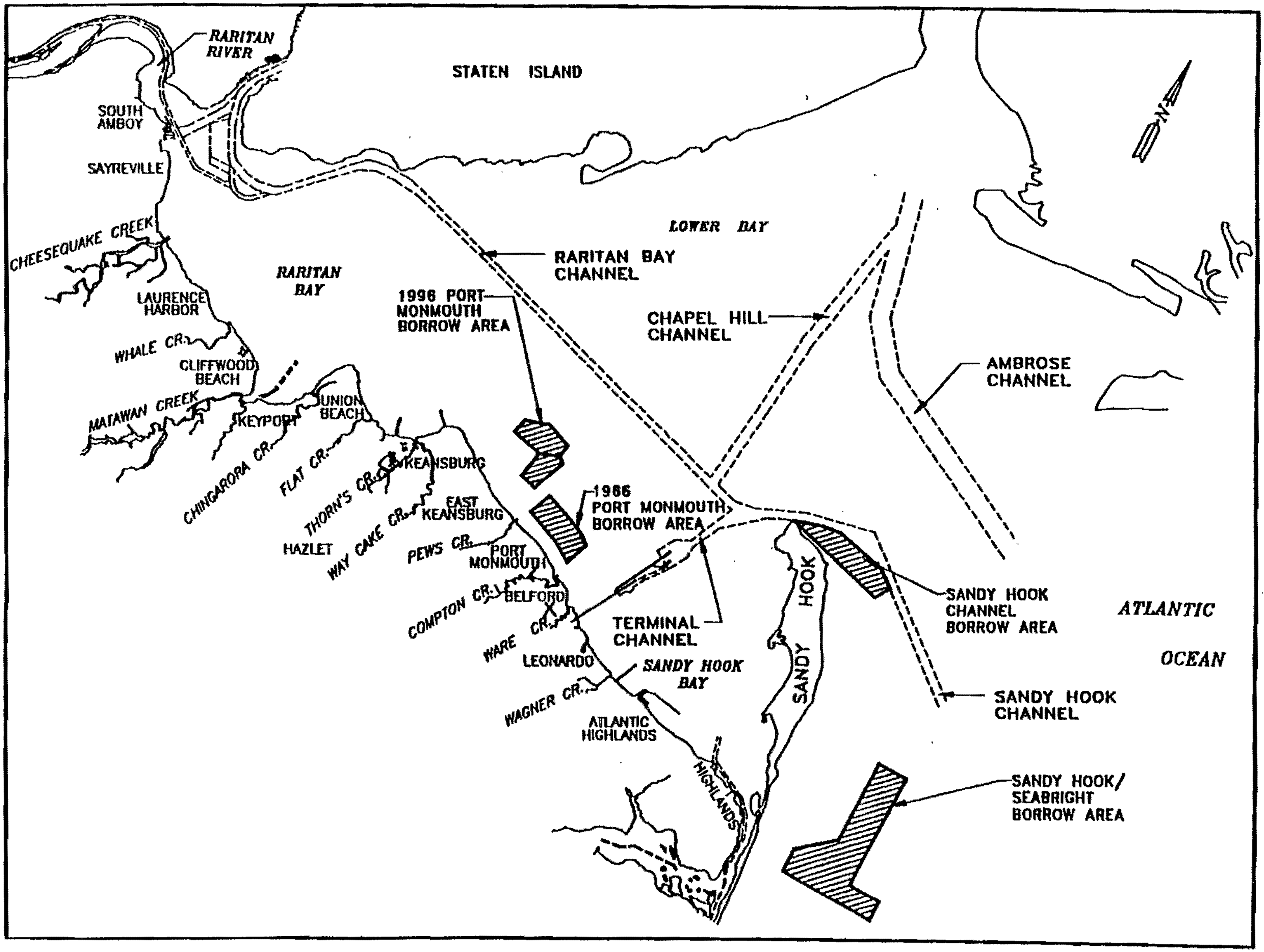
SUITABILITY CRITERIA

D-12. The suitability of sediments from potential borrow areas considered as a source of supply for the construction of Port Monmouth shore protection project were evaluated. The CERC ACES program was used to calculate the overfill factor, Ra and the Renourishment factor, Rj. The suitability criteria is divided into three categories: suitable, marginal, and unsuitable. The ranges for these criteria are presented below:

Sediment Suitability Criteria for Port Monmouth, New Jersey

Ra	Classification	Rj
1.0-1.1	Suitable	0-1.0
1.1-1.3	Marginal	1.0-1.1
>1.3	Unsuitable	>1.1

FIGURE D-3



LOCATION OF NAVIGATION CHANNELS AND BORROW AREAS

PORT MONMOUTH FEASIBILITY STUDY

D-13. The overfill factor, Ra, predicts the amount of material required to produce, after natural beach processes, one cubic yard of beach material which will have a mean grain size similar or greater than the native beach sand. The renourishment factor, Rj is a measure of the stability of the placed borrow material relative to the native sand. A Rj value equal to or less than one is most desirable for this project. However, due to the characteristics of the material available the Ra range of $1.1 < Ra < 1.2$ was utilized as suitable.

D-14. Additional criteria include sufficient volume of sand available, cost-effective constructability considerations including type of plant needed for mining and haul distance, and an absence of negative environmental impacts. The required volume used in this evaluation is approximately 350,000 cy needed for initial construction. Although a potential borrow source can meet the Ra and Rj criteria, if the source does not meet all other criteria, it can be considered unsuitable or marginal.

Evaluation of Suitability Criteria

D-15. Although the suitability criteria presented above may be the most widely used to evaluate fill material compatibility, its Ra and Rj factors have their limitations in quantifying the suitability of fill material. Assumptions must be made concerning the native and fill characteristics, these include: (1) that the native sediment is considered the most stable for the environment in which it occurs (2) sorting of borrow material by coastal processes will achieve a similar grain size distribution as the native beach, given time. The second of these assumptions is an important factor in Port Monmouth. The profiles in Port Monmouth do not exhibit typical beach profile characteristics, with their flat offshore slopes. Using the Ra and Rj factors could possibly disregard suitable material. The material used in the 1966 Port Monmouth dune project had a mean grain size of 0.28 mm. As indicated in the Engineering & Design Appendix, this material has been relatively stable. This material is much finer than the beach model composite grain size of 0.37 mm.

ENVIRONMENTAL RESTRICTIONS

D-16. In addition to borrow area considerations of material supply and characteristics, impacts to critical environmental habitats and fishing grounds pose a potential limitation to the borrow area selection. Within the waters of Sandy Hook Bay and Raritan Bay thrive populations of a variety of fishes, mollusks, and crustaceans, supporting an industry of recreational and commercial fisheries.

D-17. Surveys of the commercial fishermen contribute to an inventory of the habitats of many benthic species in the waters of Sandy Hook Bay. The delineation of these various habitat areas is presented in a report by the New Jersey Department of Environmental Protection (NJDEP, 1988)(D5). Several species of mollusks, crustaceans and fishes reside near Port

Monmouth. Within approximately 2,000 feet from shore, eels are harvested in pots (Figure D-4). Landward from a distance varying between 3,000 to 10,500 feet from shore, the sand and sand/mud bottom provides habitat for soft clams (Figure D-5).

D-18. Another principal habitat area extends into Sandy Hook Bay from approximately 9,000 feet seaward of the Port Monmouth beach. This area features beds of mussels, oysters, and blue crabs. Furthermore, this entire habitat area is said to feature hard clam populations, with the eastern portion of the area providing particularly high hard clam densities. The deeper waters in this area also yield lobster harvests. Finally, additional portions of this habitat area are identified as featuring soft clam beds of some significance. Environmental restrictions are further described in the Phase 1 Borrow Area Report of this Feasibility Study (CPE/URS, 1996)(D6).

SUMMARY OF POTENTIAL BORROW SOURCE INVESTIGATIONS

D-19. Several studies have been conducted in the Port Monmouth area since 1960 in support of federal, state and local flood control, shore protection and navigation projects (USACE, 1960)(D1). Historical borrow sources were analyzed and determined to be unsuitable. A summary of the sources analyzed is presented in Table D-3. In 1996, Alpine Ocean Seismic Surveys conducted a geophysical investigation in Raritan Bay as part of the present feasibility study. The intention of this investigation was to find suitable material for Port Monmouth Shore Protection Project. In addition to the Raritan Bay borrow area, three vibracores were taken near Sandy Hook Channel under the same contract in 1997. These investigations consisted of a seismic survey and vibracore analysis.

Offshore Sources

Historical Offshore Investigations

D-20. 1966 Port Monmouth Borrow Area An area 1,500 feet directly offshore of Port Monmouth's was examined for textural characteristics in 1966 (Figure D-3). This area was used to extract approximately 543,000 cubic yards of fill in the late 1960's (NJ, 1966)(D7). The material was placed along 5,100 feet of shoreline in the vicinity of Port Monmouth. This borrow area was reported to contain approximately 1 million cubic yards of sand, a volume of 460,000 cubic yards of beach quality sand may remain in the area. However, reuse of this borrow site is considered infeasible due in part to shallow water depths and environmental regulations enacted since the 1960's that restrict dredging in shallow nearshore areas (CPE/URS. 1996)(D6). Environmental regulations indicate that dredging is prohibited in areas of shellfish habitat. Therefore, this borrow area can be considered unsuitable due to environmental restrictions.

PORT MONMOUTH FEASIBILITY REPORT
FISHERY RESOURCES
PORT MONMOUTH FEASIBILITY STUDY

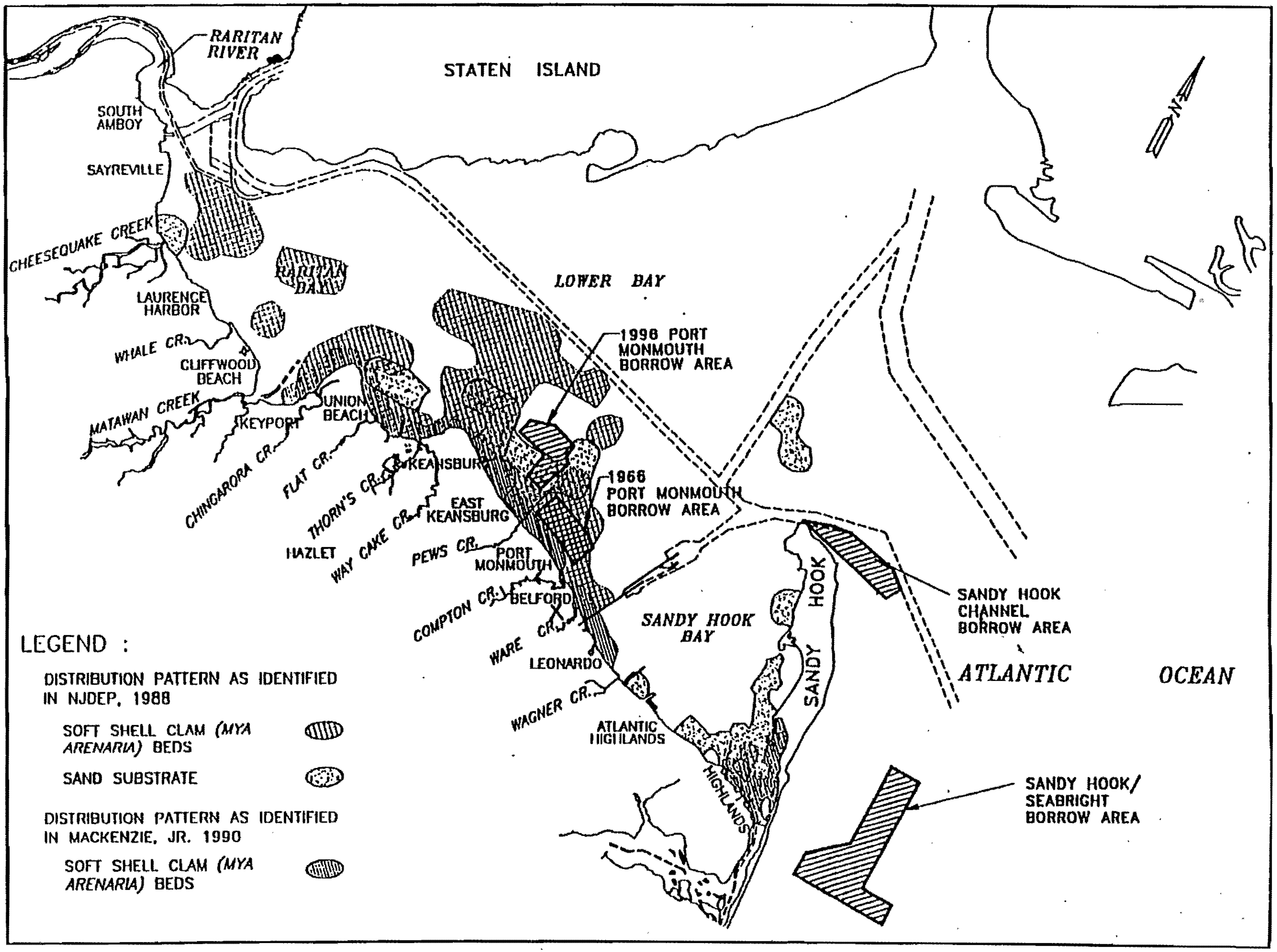


FIGURE D-4

PORT MONMOUTH
FEASIBILITY REPORT
FISHERY RESOURCES
PORT MONMOUTH FEASIBILITY STUDY

FIGURE D-5

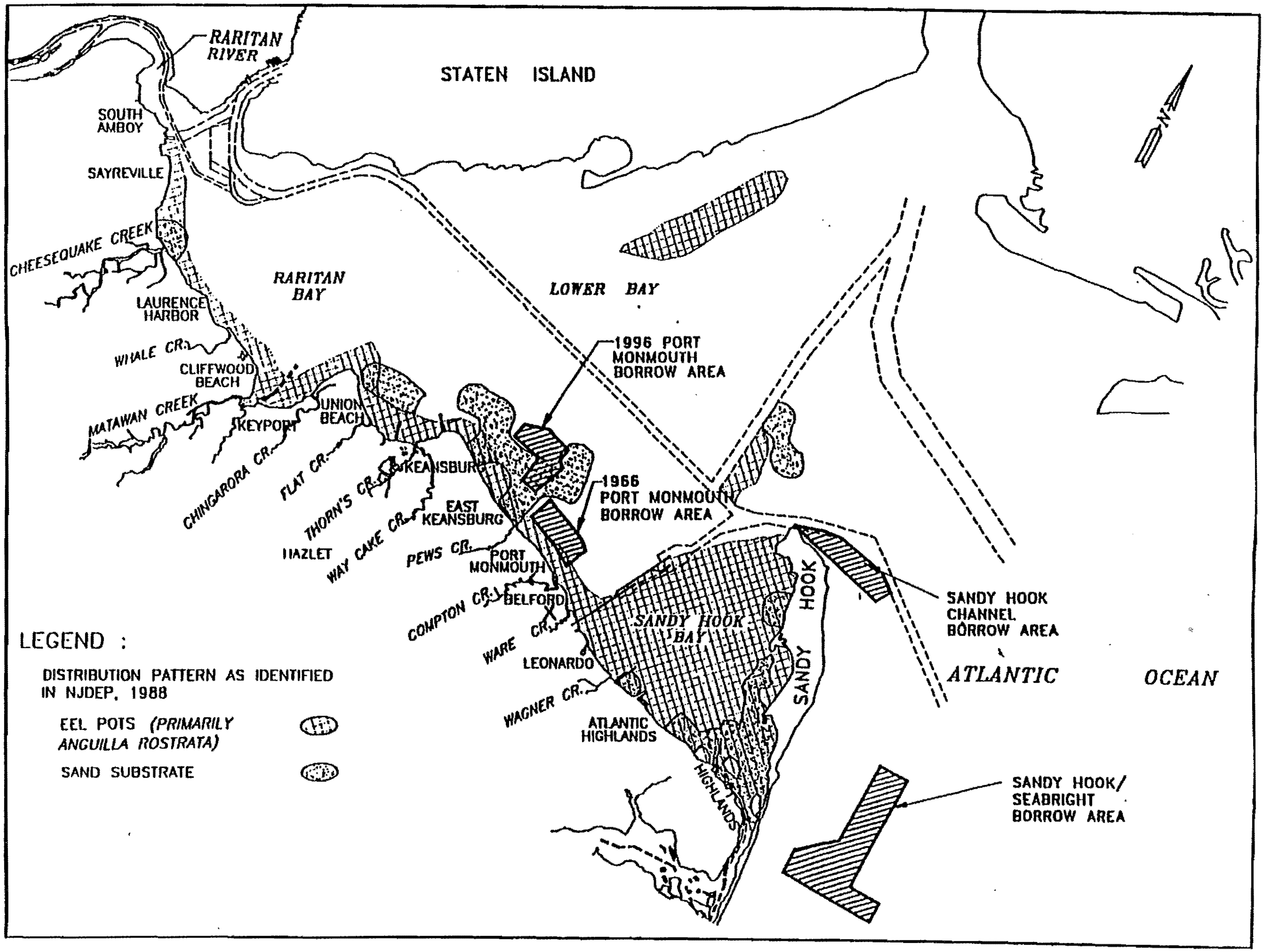


TABLE D-3

BORROW SOURCES INVESTIGATED FOR PORT MONMOUTH

	MEAN GRAIN SIZE (mm)	PHI SORTING	ESTIMATED VOLUME (CY)	OVERFILL FACTOR Ra	RENOURISHMENT FACTOR Rj	SIGNIFICANT FISHERIES IMPACTED	SUITABILITY
SANDY HOOK CHANNEL							MATERIAL IN CHANNEL IS MARGINAL
1983 CORES	0.67	0.96	ASSUMED	1.12	0.08	M,C	LOCATED 5-6 MILES FROM PORT MONMOUTH
1997 CORES	0.39	0.48	ADEQUATE	1.18	0.39		COULD POSSIBLE REDUCE STRESS ON CHANNEL DREDGING
AMBOY AGGREGATE	0.61	1.75	ASSUMED ADEQUATE	1.35	0.00	NONE	SAMPLE PROVIDED UNSUITABLE, HOWEVER SAND SUPPLIER CAN PRODUCE A MATERIAL THAT CAN PROVIDE AN OVERFILL RATIO OF 1.0
SANDY HOOK / SEA BRIGHT FILL BORROW AREA	0.60	1.21	< 54,500,000	1.22	0.00		MATERIAL IS MARGINAL LOCATED 10-11 MILES FROM PORT MONMOUTH
1996 PORT MONMOUTH BORROW AREA	0.61	1.60	296,000	1.37	0.00	F	ALPINE\CPE INVESTIGATED 1996 UNSITABLE DUE TO HIGH OVERFILL FACTOR
1966 PORT MONMOUTH BORROW	0.28	?	>400,000			M,F	USED IN 1966 BEACH FILL SHALLOW WATER DEPTH LIMITS USE THEREFORE CAN BE CONSIDERED UNSUITABLE DUE TO CONSTRUCTABILITY SOFT CLAM & EEL HABITAT
1954 EAST KEANSBURG BORROW AREA	0.24	1.23	?	2.47	0.00	M,F	UNSUITABLE DUE TO HIGH OVERFILL RATIO AND ENVIRONMENTAL RESTRICTIONS
PEWS CREEK	0.29	0.32	20,000	7.01	3.01	M,F	VOLUME INDICATES GREATEST QUANTITY REMOVED SINCE 1988 UNSUITABLE TO TO INSUFFICIENT VOLUME AND HIGH OVERFILL FACTOR DATA FROM USACE (1993)
COMPTON CREEK	FINE (79% SAND IN OUTER CHANNEL) (36% SAND IN INNER CHANNEL)	?	5,000 TO 10,000 CY/YR			M,F	UNSUITABLE DUE TO HIGH SILT CONTENT AND INSUFFICIENT VOLUME SOFT CLAM & EEL HABITAT DATA FROM 1953 DREDGING AND USACE (1993)

NOTES:

FISHERIES RANGE DATA PROVIDED BY NJDEP (1988)
C: CRUSTACEAN HABITAT (i.e. BLUE CRAB, LOBSTER)
M: MOLLUSK HABITAT (i.e. HARD & SOFT CLAMS, MUSSELS, OYSTERS)
F: FISH HABITAT (i.e. FLOUNDER, BLUEFISH, STRIPED BASS, EEL)
PM: PORT MONMOUTH
OVERFILL AND RENOURISHMENT FACTORS BASED ON ACES CALCULATIONS

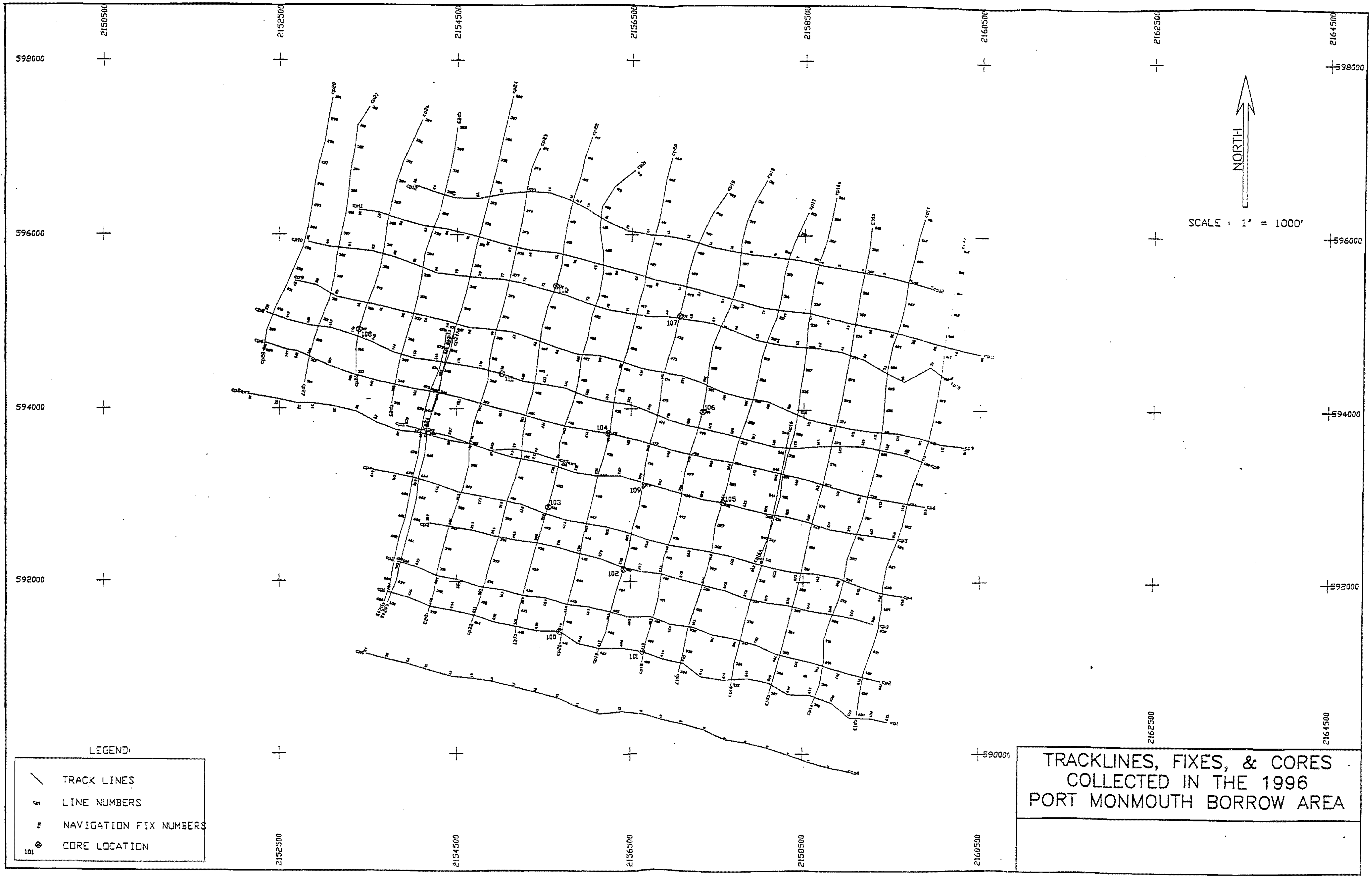
D-21. East Keansburg Borrow Area A borrow area similar to the 1966 Port Monmouth Borrow Area was utilized in 1954 to nourish the beach at East Keansburg, west of Port Monmouth. The particular layout of the borrow area is unknown, but it appears in the 1960 beach profile survey of Sandy Hook Bay. That survey indicates the cross-shore width of the borrow area to be approximately 300 feet, excavated to a depth of about -13 feet MLW. As this borrow area lies approximately 1,500 feet offshore, the surrounding depths are approximately -3 to -4 feet MLW. The longshore length of the borrow area, as well as its dredged and remaining sediment volumes are unknown, along with specific measurements of the material composition. Like the 1966 Port Monmouth borrow area, the 1954 East Keansburg borrow area lies in water currently identified as habitat of soft clams, eels, and other fishes. Therefore this borrow area can also be considered unsuitable due to a high overfill factor and environmental restrictions.

D-22. Sandy Hook/Sea Bright Borrow Area As part of a 1989 investigation, the New York District (USACE,1989)(D8) conducted 31 vibracore samples within the 7 sub-areas of the borrow area just east of Sandy Hook. The sand was shown to feature a mean grain size of 0.60 mm and a phi sorting of 1.22 Within sub-area 7, the largest and closest to Port Monmouth, the sediment characteristics are similar.

Recent Offshore Investigation

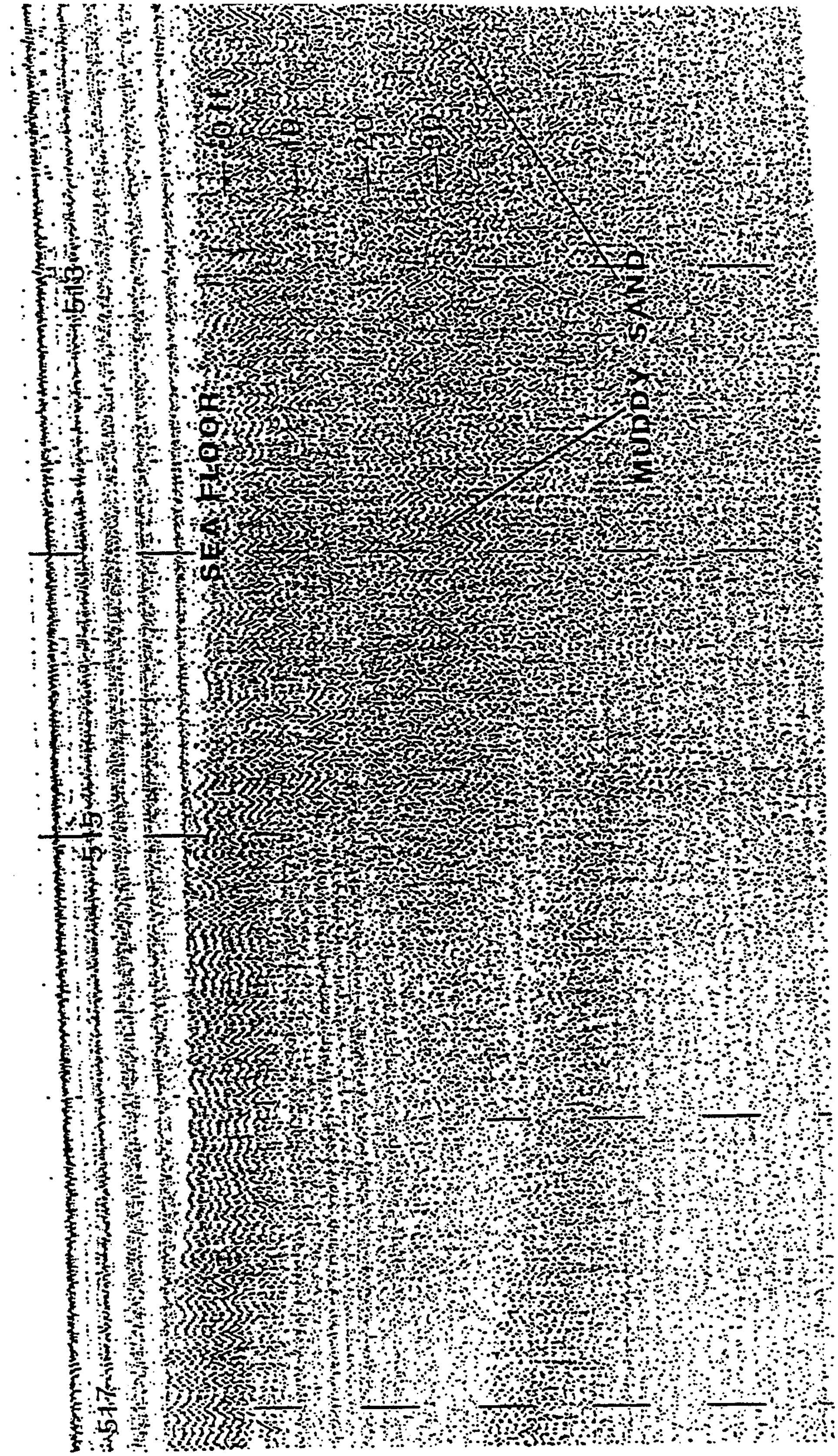
D-23. 1996 Port Monmouth Borrow Area Thirty-five miles of high-resolution seismic data was collected during the Port Monmouth Borrow Area Investigation Phase 2 (Figure D-6). Seismic records were acquired using ORE 3.5 kHz sub-bottom profiler for high resolution of near-surface sediments and an EG&G Boomer for acoustic penetration into deeper sedimentary units (Alpine, 1996)(D9). Alpine reported that the borrow area consisted predominantly of fine sand, muddy fine sand and clay deposits (Alpine, 1996)(D9). Analysis of seismic records and vibracore material indicates that potential borrow sediments within the upper 20 to 25 feet of the inner continental shelf have been strongly influenced by the presence of older geologic features in the project area. Four of eleven Vibracores penetrated clay units that are probably members of the Cretaceous Formations that have been described from the New Jersey Mainland (Alpine, 1996)(D9). Seismic records and vibracores show that bands of near-surface sediments across the north side and south side of the survey area are either muddy sand or clay (Figure D-7). Sediments in the central portion of the survey area are low in mud content, but generally contain more than 70% fine sand.

D-24. A model 271B Alpine Pneumatic Vibracorer configured to collect cores 20-feet in length, was used to retrieve eleven cores from the borrow area (Figure D-6)(Alpine, 1996)(D9). The cores were split and inspected for lithology. The core material was described



TRACKLINES, FIXES, & CORES COLLECTED IN THE 1996 PORT MONMOUTH BORROW AREA

FIGURE D-6



(ALPINE, 1996)

FIGURE D-7

SIGNATURE OF MUDDY SAND ALONG
LINE 17 OF THE 3.5kHz SEISMIC RECORDS
ALTERNATE ADJACENT BORROW AREA

according to ASTM standard. Individual sand samples from each core were analyzed for grain size distribution using mechanical sieving methods according to ASTM standards and classified on the Unified Soils Classification required by ASTM (Alpine, 1996)(D9). In, addition a composite sample of each core was compiled and analyzed for grain size distribution (Sub-Appendix D-1).

D-25. To determine the quality of the sands in the borrow area a comparison of samples from the eleven vibracores with the composite beach model of the Port Monmouth Beach was performed. The results indicate that the material from the 1996 Port Monmouth Borrow Area is likely to perform poorly (Table D-4). Overfill factors of composite samples from 8 cores are greater than 2.0. Overfill factors for individual samples from these cores range from 1.4 to more than 20. Renourishment factors for most of these samples were less than 1.0 in comparison to the beach model (Alpine, 1996)(D9). Based on the analysis of the sedimentology and stratigraphy of the borrow area, as well as the compatibility of borrow area samples with the Port Monmouth composite beach samples, the sediments within the Port Monmouth Borrow Area can be considered unsuitable. CP101 appears in Table D-4 to be suitable however, this is based on two samples one of which is mud and the other is a combination of gravel and pebbles, therefore making the composite appear as if it is compatible.

Navigational Channel Sources

D-26. As Port Monmouth lies near a number of navigation channels, dredged material from these channels provides another potential source of fill sand, featuring cost effectiveness due to combining of efforts. An investigation into these sources can indicate the feasibility of such an approach, as well as the quality of the sediment. Following is an inventory of some of the navigation projects that were investigated as potential sources of beach fill material. Table D-5 presents the potential navigational channel borrow area sediment characteristics. Ambrose Channel is a potential navigational channel borrow area, but due to an existing sand mining permitting arrangement, this material is included in the upland sand source section of this report.

Historical Navigational Channel Investigation

D-27. Pews Creek Pews Creek and the Monmouth Cove Marina located at the west end of Port Monmouth are maintained for navigation by the Monmouth County Park System. Since 1992, the creek has been dredged annually. Sands dredged from the navigation project can be grouped into three categories. The marina basin generally contains fine material unsuitable for beach nourishment. When dredged, basin material is stockpiled east of the marina. Sand dredged from the bulkheaded channel connecting the marina to the bay are generally the coarsest (USACE,1993)(D10). Grab samples from three locations taken in December 1993 showed a mean grain size of 0.29 mm (Table D-5). The navigation channel extends into the

Table D-4
Analysis of Cores Taken as Part of the Port Monmouth Feasibility Study in the 1996 Port Monmouth Borrow Area By Alpine Ocean Survey , 1996

Composite Core #	Northing	Easting	Mean Grain Size (mm)	Phi Sorting	Overfill Factor*	Suitability	Renourishment Factor *	Suitability
CP100	591423.3	2155678.8	0.34	1.57	1.77	Unsuitable	0.00	Suitable
CP101	591182.4	2156645.7	0.95	2.32	1.29	Marginal	0.00	Suitable
CP102	592141.0	2156418.4	0.61	1.80	1.37	Unsuitable	0.00	Suitable
CP103	592864.5	2155542.9	0.28	0.91	2.10	Unsuitable	0.06	Suitable
CP104	593718.4	2156229.2	0.18	0.75	7.58	Unsuitable	2.32	Unsuitable
CP105	592923.2	2157573.4	0.21	0.80	3.96	Unsuitable	0.72	Suitable
CP106	593971.5	2157340.0	0.22	1.15	2.77	Unsuitable	0.01	Suitable
CP108	594909.6	2153400.0	0.25	0.85	2.57	Unsuitable	0.20	Suitable
CP109	593114.4	2156649.4	0.20	0.96	3.66	Unsuitable	0.19	Suitable
CP110	595415.4	2155623.0	0.25	1.02	2.42	Unsuitable	0.03	Suitable
CP111	594406.1	2155015.7	0.22	0.74	3.83	Unsuitable	0.96	Suitable

* Based on ACES Calculations

TABLE D-5
SEDIMENT CHARACTERISTICS OF ADJACENT NAVIGATION CHANNELS

CORE NO.	COORDINATES (NORTHING,EASTING)	MEAN GRAIN SIZE (mm)	PHI SORTING	CORE LENGTH (feet)	OVERFILL FACTOR Ra	SUITABLITY	RENOURISHMENT FACTOR RJ	SUITABLITY
1964 BAY ENTRANCE SAND SEARCH (WILLIAMS AND DUANE, 1974)								
64-1	NA	0.40	0.87	10	1.39	Unsuitable	0.12	Suitable
64-3	NA	0.28	1.03	7	2.07	Unsuitable	0.02	Unsuitable
64-4	NA	0.40	1.09	5.7	1.48	Unsuitable	0.00	Suitable
1983 and 1997 SANDY HOOK CHANNEL SOUTH BOUNDARY (ALPINE, 1987, 1997)								
83-11	600196.0,2180473.1	0.90	1.37	28	1.12	Marginal	0.00	Suitable
83-12	602390.7,2182749.0	0.43	0.54	24	1.11	Marginal	0.16	Marginal
83-13	601946.9,2193434.8	0.28	0.74	24	2.16	Unsuitable	0.30	Unsuitable
97-11A	600221.7,2182980.2	0.42	0.52	17	1.11	Marginal	0.20	Suitable
97-11B	600633.1,2184225.1	0.37	0.46	18	1.21	Marginal	0.51	Suitable
97-12A	600717.1,2187129.8	0.37	0.47	15.5	1.22	Marginal	0.48	Suitable
1993 PEWS CREEK (SARDONIA,1996)								
93-1 TO 93-3 (1)	NA	0.29	1.45	SURFACE	1.98	Unsuitable	0.00	Suitable
93-4 TO 93-5 (2)	NA	0.18	1.69	SURFACE	2.92	Unsuitable	0.00	Suitable
AMBOY AGGREGATES RAW SAND (AMBROSE CHANNEL)								
UPPER LIMIT	NA	0.61	1.75	UPLAND	1.35	Unsuitable	0.00	Suitable
AVERAGE	NA	0.44	1.49	UPLAND	1.50	Unsuitable	0.00	Suitable
LOWER LIMIT	NA	0.31	0.93	UPLAND	1.83	Unsuitable	0.03	Unsuitable

NOTES
(1) INLET CHANNEL
(2) OFFSHORE CHANNEL
(3) OVERFILL AND RENOURISHMENT FACTORS BASED ON ACES CALCULATIONS

bay beyond the east jetty. The sand in this portion of the project area has a mean grain size of 0.18 mm.

D-28. Material dredged from Pews Creek has been used to nourish Port Monmouth beach in the past. In 1988, 20,800 c.y. were placed near the Spy House from Pews Creek dredging. Recently the sand has supported other local needs. The County is planning to place future dredge spoil to the west at Keansburg. The County may decide to work out an arrangement to support the Corps flood control project (Sardinio, 1996)(D11). This source can be considered unsuitable due to insufficient the volume of acceptable sand dredged from the creek.

D-29. Compton Creek The Federal project is located at the-eastern limit of Port Monmouth and consists of a channel 12 ft. deep and 150 ft. wide from Sandy Hook Bay through the mouth of Compton Creek. Compton Creek and Belford Harbor have a distinct shoaling problem. Sand shoals frequently at the seaward end of the outer channel and near the jetty at the west side of the creek's mouth. Observations of local fishermen indicate that silt travels down Compton Creek and quickly fills boat berths if they are not frequently used. East bank erosion adds to the siltation. The greatest shoaling rate occurs in the inner channel.

D-30. Sediment samples taken prior to the 1983 maintenance dredging of the navigational channel show inner harbor sections to contain mostly silt and clay. Sand content averages 36% in these samples, while the outer channel samples average 79% sand (USACE,1993)(D10). This is in contrast with the shoreline at Port Monmouth, which is mostly sand, with a mean grain size of 0.41 mm (Table D-1). This disparity in sediment sizes indicates that shoaling due to littoral drift from Port Monmouth is low. Rather, silts and clays from Compton Creek dominate shoaling in the inner harbor region, while sand dominates the offshore channel sections. Therefore, this material would not be compatible with the material on Port Monmouth and can be considered unsuitable.

D-31. ICONS Study Vibracores and sub-bottom seismic records acquired during the Inner Continental Shelf and Sediment Structures (ICONS) Study (Williams and Duane, 1974)(D12) indicate medium sand 0.28 to 0.4 mm in mean grain size and a 5 to 10 feet thick layer near the entrance of Raritan Bay. However, this sedimentologic information is based on only three vibracores and can be considered unsuitable due to high overfill factors.

Recent Navigation Channel Investigation

D-32. Sandy Hook Channel Borrow Area The northern most reach of Sandy Hook channel lies adjacent to the peninsula terminus. The channel and its southern bank are the recipients of significant quantities of beach quality sand transported from the beaches to the south. The potential exist for a borrow area to be identified along the southern edge of this channel (just north of Sandy Hook) that could provide sand for Port Monmouth. Alpine Ocean Seismic Survey (1987)(D13) investigated sediment characteristics along the south boundary of Sandy

Hook channel, approximately six miles from Port Monmouth. Sub-bottom sands represented by composite samples from three cores ranged from 0.28 to 0.90 mm in diameter. The advantages of using this location would include high material volume and quality, and reduced stress on channel maintenance. Disadvantages would include distance from Port Monmouth shoreline and sensitivity of removal so near to a major shipping channel.

D-33. In the late 1980's, one million cubic yards were dredged from this channel and placed on Sandy Hook. Prior to this, a sand search was conducted along the south boundary of the Sandy Hook Channel in 1983 (Figure D-8) to identify sand sources for the Sandy Hook shoreline. Borings 11,12, and 13 were taken along a line from the Sandy Hook tip to a point approximately 1.5 miles directly east (Ocean Seismic Surveys, Inc. 1983)(D14). Coarse sand was found at Boring 11, and is among the most compatible found from past investigations (Table D-5). This area is fed from alongshore drift that moves north along the Atlantic shoreline. A volume could not be computed due to the lack of the vibracore logs.

D-34. Due to the compatibility of the prior sand investigations, three additional vibracores were recommended in the Port Monmouth, Identification and Delineation of Sand Borrow Area Phase 1 report. These cores, labeled 11A, 11B, and 12A on Figure D-8, were taken in October 1997. The mean grain size of the samples collected for each core is 0.42 mm, 0.37mm, and 0.37mm respectively (Table D-5). As presented in Table D-5, this material is considered suitable with an average overfill factor of 1.15 (83-13 is not included in this average). Vibracore 83-11 is the westernmost sample and is not included in the Sandy Hook Channel Borrow Area.

D-35. Sandy Hook Channel is currently being considered for a maintenance dredging arrangement similar to the one that exists for Ambrose Channel (Rosamilia, 1998)(D15). Ambrose Channel is maintained cost free to the federal government through an arrangement with a local aggregate supplier. If this dredging arrangement is made the sand may be available as an upland sand source from Amboy Aggregates.

Upland Sources

D-36. Truck hauled fill from upland sources is an alternative to fill dredged from offshore locations or navigation channels. For small fill requirements the use of upland sources can provide the more cost-effective approach, as the costs associated with the mobilization and operation of dredge equipment is eliminated. However, with large projects the operational expenses of the heavy transport equipment is often prohibitive, particularly when the material is stockpiled a great distance from the project site.

D-37. Additional quantities of upland sand are likely available. In 1991 the New Jersey Department of Environmental Protection conducted an "Inventory of Active and Abandoned Sand and Gravel Mining Operations in New Jersey" (Bell et al., 1991)(D16). This inventory

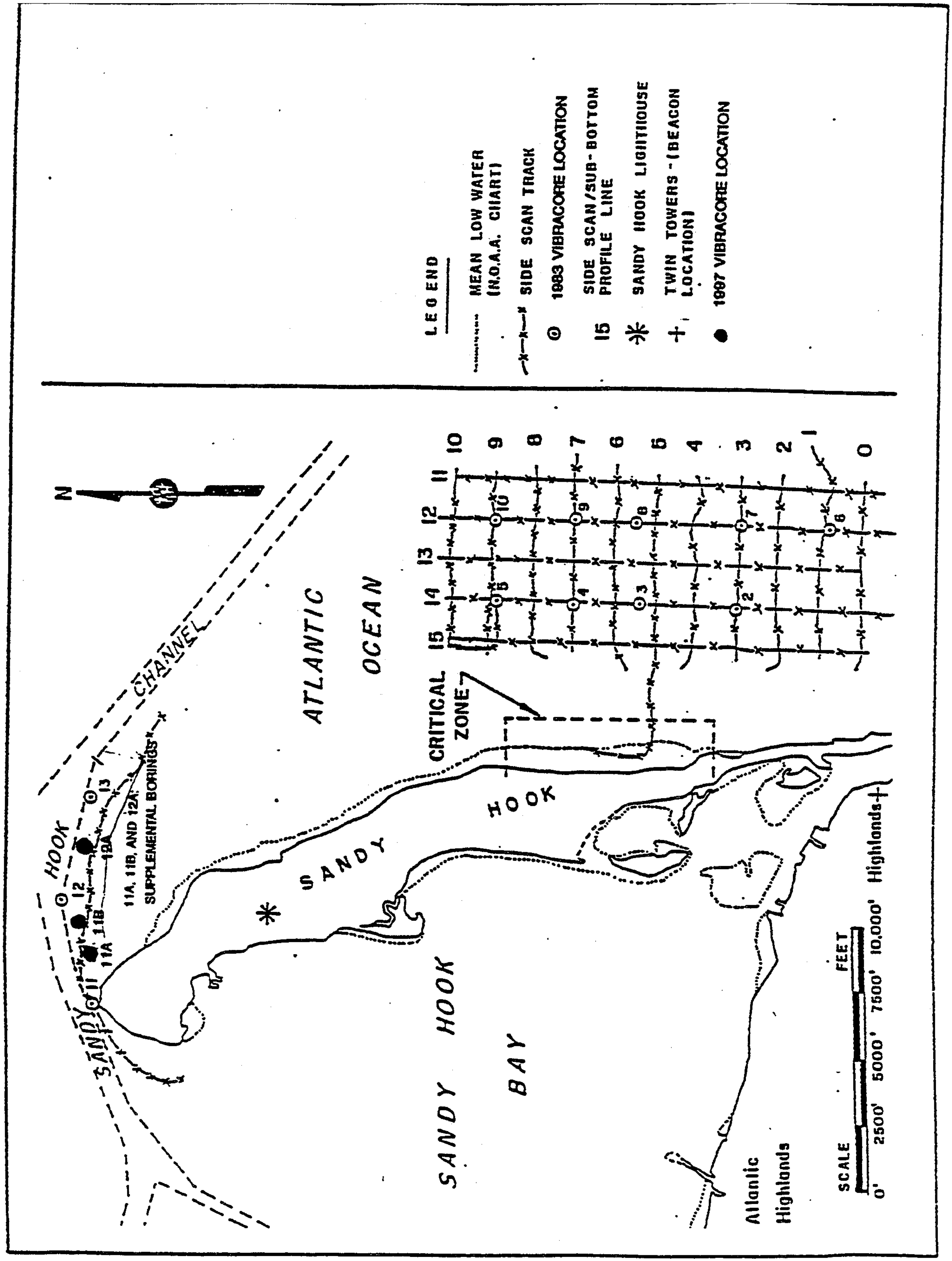


FIGURE D-8

LOCATION MAP OF 1983 SEISMIC LINES AND 1983,1997 VIBRACORE SITES

PORT MONMOUTH FEASIBILITY STUDY

identified 89 stockpiles of material in Monmouth County, along with their dates of operation. Many of these are listed as presently active. Descriptions of their stockpiled materials quality and quantity are also included, but these are vague and non-specific.

D-38. Amboy Aggregate The company is one of the largest suppliers of aggregate in the United States. One of its largest sources of sand and gravel is the Ambrose Channel. This arrangement not only provides Amboy Aggregate with a commercial source of sand, but provides a synergistic benefit in maintaining the Federal navigation project dredged at a savings of approximately $3.5 million dollars per year (Degener, 1996)(D17). Amboy Aggregate uses a hopper dredge to mine the sand and has a production rate of approximately 10,000 cy/day. It currently mines approximately two million cubic yards of sand per year from Ambrose Channel. The company is seeking to expand its cooperative dredging effort to Sandy Hook Channel (Degener, 1996)(D17).

D-39. Amboy Aggregate has a large processing plant in South Amboy, NJ, and is capable of sorting dredged material into the gradations needed by the construction industry. The characteristics of the dredged sand are as shown in Figure D-9. The mean grain size varies from 0.31 to 0.61 mm with an average sorting coefficient of 1.49.

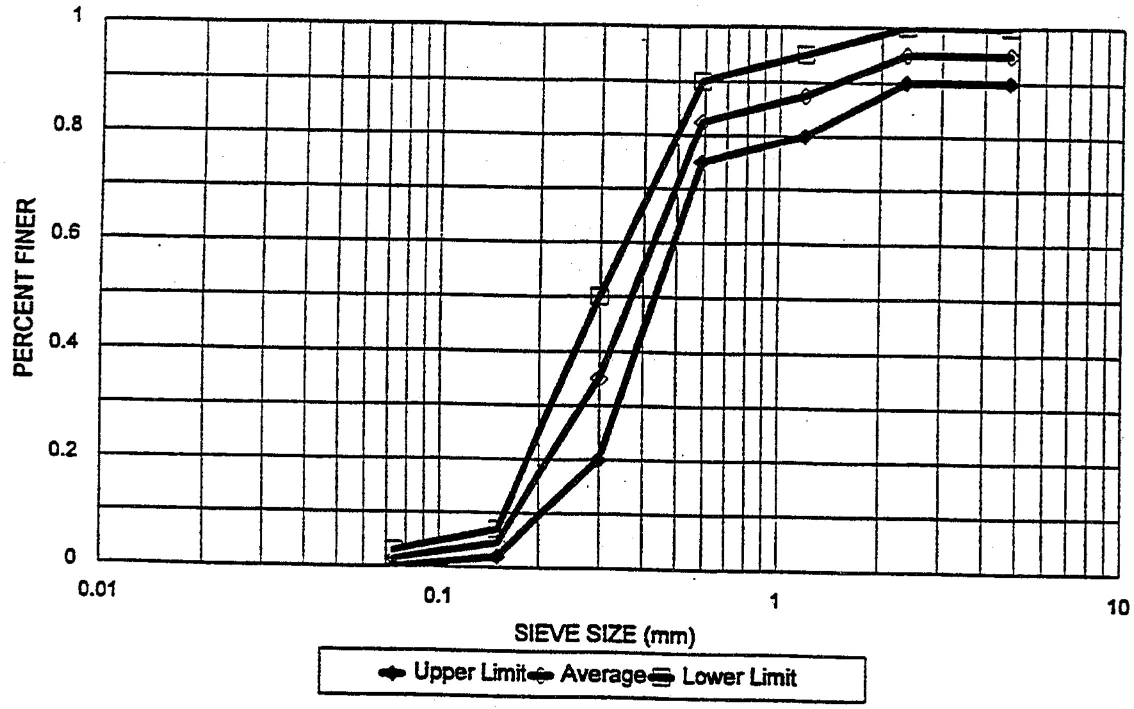
D-40. This material could be utilized to construct dune or beach fill at Port Monmouth. For example, Union Beach located on Raritan Bay approximately 4 miles west of Port Monmouth, recently utilized an upland fill source. In 1996, 75,000 cubic yards of sand from Amboy Aggregate was placed on Union Beach, as part of a shore protection project by the State of New Jersey. A sand specification was produced by the engineering and construction element of the New Jersey Department of Environmental Protection. The sand specification required 90% of material to pass the #20 sieve (0.84 mm) and be retained by the #40 sieve (0.42 mm). The unit cost of the sand for the project was $10.15. The sand was trucked in over a 5-month period, from March to August.

D-41. R. W. Vogel This company has a sand pit in Stafford Township in southern Ocean County. The material from this pit was used to nourish Harvey Cedars on Long Beach Island in 1995-96. The material contains between 2 and 8% silts and clays (Garafolo, 1996)(D18). R.W. Vogel was recommended for use in the Keansburg Shore Protection Project.

D-42. Muccio The company provides sand from pits in Wall Township, NJ. They recently bid on the Union Beach fill project.

CONCLUSIONS

D-43. Based on the modified suitability criteria and potential environmental impacts, the selected material would be the sand within the Sea Bright borrow area. The overfill factor is



GRAIN SIZE			PASSING LIMITS		
Sieve	(mm)	(Phi)	Upper	Average	Lower
3/4"	19.05	-4.25	95%	98%	100%
#4	4.75	-2.25	90%	95%	100%
#8	2.38	-1.25	90%	95%	100%
#16	1.19	-0.25	80%	88%	95%
#30	0.59	0.76	75%	83%	90%
#50	0.30	1.74	20%	35%	50%
#100	0.15	2.74	2%	5%	7%
#200	0.074	3.76	0%	2%	3%
MOMENT METHOD CHARACTERISTICS					
Mean (Phi)			0.70	1.20	1.69
Mean (mm)			0.61	0.44	0.31
Sorting (Phi)			1.75	1.49	0.93
Median (Phi)			1.20	1.42	1.74
Median (mm)			0.43	0.37	0.30

MATERIAL IS STOCKPILED AND DEWATERED, NOT IN-SITU

FIGURE D-9

AMBOY AGGREGATE SAND CHARACTERISTICS

MATERIAL REMOVED FROM AMBROSE CHANNEL

estimated as 1.22. The volume contained in this borrow area is estimated at 54.5 million cubic yards. With an initial fill volume of approximately 378,500 cubic yards, this borrow area is a satisfactory source of fill. The approximate limits of the Sea Bright Borrow Area are shown in Figure D-5. A detailed investigation of the Sandy Hook/Sea Bright Borrow Area should be conducted during the Plans and Specification phase of this project. Due to the substantial mobilization costs associated with dredging, Amboy Aggregates is recommended for renourishment cycles as a result of the lower fill volumes needed. Recent experience at Union Beach demonstrates the practical application of a trucked upland fill source for smaller projects in the Raritan Bay Area. The unit price per cubic yard for the Union Beach Project was $10.15. Therefore, an estimate of $15.00 per cubic yard would be a conservative estimate.

D-44. Although many sources were identified, most were eliminated due to their suitability. The suitability requirements included sufficient volume, constructability, acceptable Ra and Rj factors and environmental restrictions. Examples of these include Pews Creek and Compton Creek and their shoals. The sand volume is small in these areas because of low tidal flow and low longshore transport. Therefore, in addition to reports of high silt content (especially within Compton Creek), these areas are inappropriate for borrow sources due to poor sand quality and volumes contained.

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SUB-APPENDIX D-1

Vibracore Logs
Grain Size Distribution Curves

DRILLING LOG DIVISION	INSTALLATION SHEET 1 OF 1
1. PROJECT Pt. Monmouth	10. SIZE AND TYPE OF BIT 4" Vibracore
2. LOCATION (*Coordinates or Station*) 2155678.8 E 591423.3 N	11. DATUM FOR ELEVATION SHOWN (*TBM or MSL*) MLW
3. DRILLING AGENCY Alpine Ocean Seismic Survey, Inc.	12. MANUFACTURER'S DESIGNATION OF DRILL Alpine Pneumatic Vibracore
4. HOLE NO. (*As shown on drawing title and file number*) CP100	13. TOTAL NO. OF OVERBURDEN SAMPLES TAKEN disturbed: undisturbed:
5. NAME OF DRILLER Alpine Ocean Seismic Survey, Inc.	14. TOTAL NUMBER OF CORE BOXES
	15. ELEVATION GROUND WATER
6. DIRECTION OF HOLE ☒ VERTICAL ☐ INCLINED	16. DATE HOLE STARTED 5/16/96 COMPLETED 5/16/96
7. WATER DEPTH	17. ELEVATION TOP OF HOLE -16 ft
8. DEPTH DRILLED INTO ROCK	18. TOTAL CORE RECOVERY FOR BORING
9. TOTAL DEPTH OF HOLE 10.0'	19. SIGNATURE OF GEOLOGIST G. Zarillo

ELEV.	DEPTH	LEGEND	CLASSIFICATION OF MATERIALS (Description)	CORE REC %	SAMPLE NUMBER	REMARKS
-16.0	0.0					
-17.2	1.2		Dark gray, fine, muddy sand, gravel and pebbles. (SM)	100	0.5	
-26.0	10.0	▨	Dark gray, inorganic clay, medium plasticity. (CL)	100 100		7.1'–7.2' Consolidate light-brown clay inclusion
			End at 10.0'			

0 2.5 5 7.5 10 12.5 15 17.5 20 22.5

D1-1

ENG FORM 1836 PREVIOUS EDITIONS ARE OBSOLETE.

PROJECT

HOLE NUMBER

DRILLING LOG

DIVISION

INSTALLATION

SHEET 1 OF 1

1. PROJECT Pt. Monmouth
2. LOCATION (*Coordinates or Station*) 2156645.7 E 591182.4 N
3. DRILLING AGENCY Alpine Ocean Seismic Survey, Inc.
4. HOLE NO. (*As shown on drawing title and file number*) CP101
5. NAME OF DRILLER Alpine Ocean Seismic Survey, Inc.
6. DIRECTION OF HOLE ☒ VERTICAL ☐ INCLINED
7. WATER DEPTH
8. DEPTH DRILLED INTO ROCK
9. TOTAL DEPTH OF HOLE 9.9'
10. SIZE AND TYPE OF BIT 4" Vibracore
11. DATUM FOR ELEVATION SHOWN (*TBM or MSL*) MLW
12. MANUFACTURER'S DESIGNATION OF DRILL Alpine Pneumatic Vibracore
13. TOTAL NO. OF OVERBURDEN SAMPLES TAKEN disturbed: undisturbed:
14. TOTAL NUMBER OF CORE BOXES
15. ELEVATION GROUND WATER
16. DATE HOLE STARTED 5/16/96 COMPLETED 5/16/96
17. ELEVATION TOP OF HOLE -12.5 ft
18. TOTAL CORE RECOVERY FOR BORING
19. SIGNATURE OF GEOLOGIST G. Zarillo

ELEV.	DEPTH	LEGEND	CLASSIFICATION OF MATERIALS (Description)	CORE REC %	SAMPLE NUMBER	REMARKS
-12.5	0.0					
-13.5	1.0		Gray–brown, fine sand, some mud, scattered pebbles. (SM)	100	0.5	
-16.2	3.7		Medium-fine sand, gravel and pebbles, iron-stained. (SW)		3.0	
-22.4	9.9	▨	Dark gray, inorganic clay, medium plasticity. (CL)	100		6.4' Consolidated mud inclusion
			End at 9.9'			

0
2.5
5
7.5
10
12.5
15
17.5
20
22.5

D1-2

ENG FORM 1836 PREVIOUS EDITIONS ARE OBSOLETE.

PROJECT

HOLE NUMBER

DRILLING LOG

DIVISION	INSTALLATION SHEET 1 OF 1
1. PROJECT Pt. Monmouth	10. SIZE AND TYPE OF BIT 4" Vibracore
2. LOCATION (*Coordinates or Station*) 2156418.4 E 592141 N	11. DATUM FOR ELEVATION SHOWN (*TBM or MSL*) MLW
3. DRILLING AGENCY Alpine Ocean Seismic Survey, Inc.	12. MANUFACTURER'S DESIGNATION OF DRILL Alpine Pneumatic Vibracore
4. HOLE NO. (*As shown on drawing title and file number*) CP102	13. TOTAL NO. OF OVERBURDEN SAMPLES TAKEN disturbed: undisturbed:
5. NAME OF DRILLER Alpine Ocean Seismic Survey, Inc.	14. TOTAL NUMBER OF CORE BOXES 15. ELEVATION GROUND WATER
6. DIRECTION OF HOLE ☒ VERTICAL ☐ INCLINED	16. DATE HOLE STARTED 5/15/96 COMPLETED 5/15/96
7. WATER DEPTH	17. ELEVATION TOP OF HOLE -16.0 ft
8. DEPTH DRILLED INTO ROCK	18. TOTAL CORE RECOVERY FOR BORING
9. TOTAL DEPTH OF HOLE 17.1'	19. SIGNATURE OF GEOLOGIST G. Zarillo

ELEV.	DEPTH	LEGEND	CLASSIFICATION OF MATERIALS (Description)	CORE REC %	SAMPLE NUMBER	REMARKS
-16.0	0.0					
-17.3	1.3	░	Tan, medium-fine sand. (SP)	100	0.5	
-17.7	1.7	░	Tan gravel, medium sand, pebbles. (GW)			
-22.5	6.5	░	Tan–brown, medium-fine sand, scattered quartz pebbles. (SP)	100	5.0	
-25.8	9.9	░	Iron-stained, fine sand, gravel, pebbles, small cobbles. (SP)			
-28.5	12.5	░	Iron-stained, fine sand, gravel, pebbles, small cobbles. (SP)		10.0	
-33.1	17.1	▨	Dark gray, inorganic clay, medium pasticity. (CL)	100		16.0'–17.1' Stringers of semiconsolidated brown clay
			End at 17.1'			

D1-3

ENG FORM 1836 PREVIOUS EDITIONS ARE OBSOLETE.

PROJECT

HOLE NUMBER

DRILLING LOG

DIVISION	
INSTALLATION	SHEET 1 OF 1
1. PROJECT	Pt. Monmouth
2. LOCATION (Coordinates or Station)	2155642.9 E 592864.5 N
3. DRILLING AGENCY	Alpine Ocean Seismic Survey, Inc.
4. HOLE NO. (As shown on drawing title and file number)	CP103
5. NAME OF DRILLER	Alpine Ocean Seismic Survey, Inc.
6. DIRECTION OF HOLE	☒ VERTICAL ☐ INCLINED
7. WATER DEPTH	
8. DEPTH DRILLED INTO ROCK	
9. TOTAL DEPTH OF HOLE	19.5'
10. SIZE AND TYPE OF BIT	4" Vibracore
11. DATUM FOR ELEVATION SHOWN (TBM or MSL)	MLW
12. MANUFACTURER'S DESIGNATION OF DRILL	Alpine Pneumatic Vibracore
13. TOTAL NO. OF OVERBURDEN SAMPLES TAKEN	disturbed: undisturbed:
14. TOTAL NUMBER OF CORE BOXES	
15. ELEVATION GROUND WATER	
16. DATE HOLE	STARTED 5/15/96 COMPLETED 5/15/96
17. ELEVATION TOP OF HOLE	-16.0 ft
18. TOTAL CORE RECOVERY FOR BORING	
19. SIGNATURE OF GEOLOGIST	G. Zarillo

ELEV.	DEPTH	LEGEND	CLASSIFICATION OF MATERIALS (Description)	CORE REC %	SAMPLE NUMBER	REMARKS
-16.0	0.0					
-16.8	0.8	∴	Tan, fine sand. (SP)	100	0.5	0.7' Clay ball
-20.2	4.2	∴	Tan–brown, fine sand, scattered pebbles and shell fragments. (SP)			
-26.1	10.1	∴	Medium-fine, iron-stained sand, scattered dark organic material, heavy minerals. (SP)	100	4.5 9.5	
-28.4	12.4	∴	Iron-stained, medium-fine sand, trace gravel. (SP)			
-32.2	16.2	○	Iron-stained, medium-fine sand, gravel and pebbles. (SW)	100	14.0	
-35.2	19.5	▨	Dark gray, inorganic clay, stringers of hard brown clay. (CL)			
			End at 19.5'			

D1-4

ENG FORM 1836 PREVIOUS EDITIONS ARE OBSOLETE.

PROJECT Pt. Monmouth

HOLE NUMBER CP103

DRILLING LOG	DIVISION	INSTALLATION	SHEET 1 OF 1

1. PROJECT	Pt. Monmouth
2. LOCATION (Coordinates or Station)	2156229.2 E 593718.4 N
3. DRILLING AGENCY	Alpine Ocean Seismic Survey, Inc.
4. HOLE NO. (As shown on drawing title and file number)	CP104
5. NAME OF DRILLER	Alpine Ocean Seismic Survey, Inc.
6. DIRECTION OF HOLE	☒ VERTICAL ☐ INCLINED
7. WATER DEPTH	
8. DEPTH DRILLED INTO ROCK	
9. TOTAL DEPTH OF HOLE	18.5'
10. SIZE AND TYPE OF BIT	4" Vibracore
11. DATUM FOR ELEVATION SHOWN (TBM or MSL)	MLW
12. MANUFACTURER'S DESIGNATION OF DRILL	Alpine Pneumatic Vibracore
13. TOTAL NO. OF OVERBURDEN SAMPLES TAKEN	disturbed: undisturbed:
14. TOTAL NUMBER OF CORE BOXES	
15. ELEVATION GROUND WATER	
16. DATE HOLE	STARTED 5/15/96 COMPLETED 5/15/96
17. ELEVATION TOP OF HOLE	-14.5 ft
18. TOTAL CORE RECOVERY FOR BORING	
19. SIGNATURE OF GEOLOGIST	G. Zarillo

ELEV.	DEPTH	LEGEND	CLASSIFICATION OF MATERIALS (Description)	CORE REC %	SAMPLE NUMBER	REMARKS	
-14.5	0.0						0
			Gray, fine sand. (SP)	100	0.5		
							2.5
					4.5		5
-22.1	7.6						7.5
			Fine, iron-stained sand. (SP)	100	9.0	7.6'–11.5' Thin layers of coarse sand interbedded with medium sand	
							10
-26.0	11.5						
			Brown–orange, fine sand, iron-stained, scattered pebbles. (SP)	100			12.5
-29.2	14.7						
			Fine sand, iron-stained, scattered shell fragments. (SP)		16.0		15
							17.5
-33.0	18.5						
			End at 18.5'				
							20
						D1-5	
							22.5

ENG FORM 1836 MAR 71 PREVIOUS EDITIONS ARE OBSOLETE.

PROJECT Pt. Monmouth

HOLE NUMBER

DRILLING LOG

DIVISION

INSTALLATION

SHEET 1 OF 1

1. PROJECT Pt. Monmouth

2. LOCATION (*Coordinates or Station*) 2157573.4 E 592923.2 N

3. DRILLING AGENCY Alpine Ocean Seismic Survey, Inc.

4. HOLE NO. (*As shown on drawing title and file number*) CP105

5. NAME OF DRILLER Alpine Ocean Seismic Survey, Inc.

6. DIRECTION OF HOLE ☒ VERTICAL ☐ INCLINED

7. WATER DEPTH

8. DEPTH DRILLED INTO ROCK

9. TOTAL DEPTH OF HOLE 20.0'

10. SIZE AND TYPE OF BIT 4" Vibracore

11. DATUM FOR ELEVATION SHOWN (*TBM or MSL*) MLW

12. MANUFACTURER'S DESIGNATION OF DRILL Alpine Pneumatic Vibracore

13. TOTAL NO. OF OVERBURDEN SAMPLES TAKEN disturbed: undisturbed:

14. TOTAL NUMBER OF CORE BOXES

15. ELEVATION GROUND WATER

16. DATE HOLE STARTED 5/16/96 COMPLETED 5/16/96

17. ELEVATION TOP OF HOLE -15.0 ft

18. TOTAL CORE RECOVERY FOR BORING

19. SIGNATURE OF GEOLOGIST G. Zarillo

ELEV.	DEPTH	LEGEND	CLASSIFICATION OF MATERIALS (Description)	CORE REC %	SAMPLE NUMBER	REMARKS
-15.0	0.0					
-16.8	1.8	░	Light brown–gray, fine sand, few quartz pebbles, shell fragments. (SP)	100	0.5	
-20.8	5.8	░	Gray, fine sand, few shell fragments. (SP)		4.0	4.7'–5.7' Some mud
-26.2	11.2	░	Medium-fine sand, iron-stained. (SP)	100	8.0	5.8'–11.2' Thin layers of very coarse, sandy gravel
-35.0	20.0	░	Fine sand, iron-stained, few shell fragments. (SP)	100	16.0	
			End at 20.0'			

0 2.5 5 7.5 10 12.5 15 17.5 20 22.5

D1-6

ENG FORM 1836 PREVIOUS EDITIONS ARE OBSOLETE.

PROJECT

HOLE NUMBER CP105

DRILLING LOG

DIVISION

INSTALLATION

SHEET 1 OF 1

1. PROJECT Pt. Monmouth
2. LOCATION (*Coordinates or Station*) 2157340.0 E 593971.5 N
3. DRILLING AGENCY Alpine Ocean Seismic Survey, Inc.
4. HOLE NO. (*As shown on drawing title and file number*) CP106
5. NAME OF DRILLER Alpine Ocean Seismic Survey, Inc.
6. DIRECTION OF HOLE ☒ VERTICAL ☐ INCLINED
7. WATER DEPTH
8. DEPTH DRILLED INTO ROCK
9. TOTAL DEPTH OF HOLE 16.7'
10. SIZE AND TYPE OF BIT 4" Vibracore
11. DATUM FOR ELEVATION SHOWN (*TBM or MSL*) MLW
12. MANUFACTURER'S DESIGNATION OF DRILL Alpine Pneumatic Vibracore
13. TOTAL NO. OF OVERBURDEN SAMPLES TAKEN disturbed: undisturbed:
14. TOTAL NUMBER OF CORE BOXES
15. ELEVATION GROUND WATER
16. DATE HOLE STARTED 5/15/96 COMPLETED 5/15/96
17. ELEVATION TOP OF HOLE -14.7 ft
18. TOTAL CORE RECOVERY FOR BORING
19. SIGNATURE OF GEOLOGIST G. Zarillo

ELEV.	DEPTH	LEGEND	CLASSIFICATION OF MATERIALS (Description)	CORE REC %	SAMPLE NUMBER	REMARKS
-14.7	0.0					
-15.9	1.2	▨	Brown, medium-fine sand, shell fragments. (SP)	100	0.5	
-19.6	4.9	▨	Gray, medium-coarse sand. (SP)		4.0	
-22.9	8.2	▨	Dark gray, fine sand, some mud. (SP)			5.0'–8.0' Strong smell of decaying organic material
-25.0	10.3	▨	Gray, medium-fine sand, shell fragments, few quartz pebbles. (SP)	100	9.0	
-31.4	16.7	▨	Fine, iron-stained sand. (SP)	100	15.0	
			End at 16.7'			

0 2.5 5 7.5 10 12.5 15 17.5 20 22.5

D1-7

ENG FORM 1836 PREVIOUS EDITIONS ARE OBSOLETE. PROJECT HOLE NUMBER

DRILLING LOG

DIVISION	
INSTALLATION	
SHEET	1 OF 1
1. PROJECT	Pt. Monmouth
2. LOCATION (Coordinates or Station)	2153400 E 594909.8 N
3. DRILLING AGENCY	Alpine Ocean Seismic Survey, Inc.
4. HOLE NO. (As shown on drawing title and file number)	CP108
5. NAME OF DRILLER	Alpine Ocean Seismic Survey, Inc.
6. DIRECTION OF HOLE	☒ VERTICAL ☐ INCLINED
7. WATER DEPTH	
8. DEPTH DRILLED INTO ROCK	
9. TOTAL DEPTH OF HOLE	19.1'
10. SIZE AND TYPE OF BIT	4" Vibracore
11. DATUM FOR ELEVATION SHOWN (TBM or MSL)	MLW
12. MANUFACTURER'S DESIGNATION OF DRILL	Alpine Pneumatic Vibracore
13. TOTAL NO. OF OVERBURDEN SAMPLES TAKEN	disturbed: undisturbed:
14. TOTAL NUMBER OF CORE BOXES	
15. ELEVATION GROUND WATER	
16. DATE HOLE	STARTED 5/16/96 COMPLETED 5/16/96
17. ELEVATION TOP OF HOLE	-14.4 ft
18. TOTAL CORE RECOVERY FOR BORING	
19. SIGNATURE OF GEOLOGIST	G. Zarillo

ELEV.	DEPTH	LEGEND	CLASSIFICATION OF MATERIALS (Description)	CORE REC %	SAMPLE NUMBER	REMARKS
-14.4	0.0					
-18.0	3.7		Dark gray, fine sand, shells, shell fragments, some mud. (SM)	100	0.5	
-25.7	11.3		Medium-fine sand, iron-stained, scattered pebbles, and shell fragments. (SP)	100	4.0 9.0	9.1' Layer of pebbles
-29.2	14.8		Tan–brown, medium-fine sand, few pebbles and shell fragments. (SP)			
-33.1	18.7		Medium-fine sand, iron-stained. (SP)	100	15.0	
-33.2	18.8		Gravely pebbles, coarse sand. (GW)			
			End at 18.8'			

0 2.5 5 7.5 10 12.5 15 17.5 20 22.5

D1-8

ENG FORM 1836 PREVIOUS EDITIONS ARE OBSOLETE. PROJECT HOLE NUMBER

DRILLING LOG

DIVISION

INSTALLATION

SHEET 1
OF 1

1. PROJECT Pt. Monmouth
2. LOCATION (*Coordinates or Station*) 2156649.4 E 593114.4 N
3. DRILLING AGENCY Alpine Ocean Seismic Survey, Inc.
4. HOLE NO. (*As shown on drawing title and file number*) CP109
5. NAME OF DRILLER Alpine Ocean Seismic Survey, Inc.
6. DIRECTION OF HOLE ☒ VERTICAL ☐ INCLINED
7. WATER DEPTH
8. DEPTH DRILLED INTO ROCK
9. TOTAL DEPTH OF HOLE 17.6'
10. SIZE AND TYPE OF BIT 4" Vibracore
11. DATUM FOR ELEVATION SHOWN (*TBM or MSL*) MLW
12. MANUFACTURER'S DESIGNATION OF DRILL Alpine Pneumatic Vibracore
13. TOTAL NO. OF OVERBURDEN SAMPLES TAKEN disturbed: undisturbed:
14. TOTAL NUMBER OF CORE BOXES
15. ELEVATION GROUND WATER
16. DATE HOLE STARTED 5/16/96 COMPLETED 5/16/96
17. ELEVATION TOP OF HOLE -15.0 ft
18. TOTAL CORE RECOVERY FOR BORING
19. SIGNATURE OF GEOLOGIST G. Zarillo

ELEV.	DEPTH	LEGEND	CLASSIFICATION OF MATERIALS (Description)	CORE REC %	SAMPLE NUMBER	REMARKS
-15.0	0.0					
-18.5	3.5		Gray, fine sand, shell fragments, few quartz pebbles. (SP)	100	0.5 3.0	
-25.0	10.0		Medium-fine sand, iron-stained. (SP)	100	8.0	
-30.2	15.2		Medium-fine sand, thin layers of coarse sand, iron-stained. (SP)			13.5'–13.7' Sand blackened by organic material
-32.6	17.6		Fine sand, iron-stained, few shell fragments, some mud. (SM) or SP? as original	100	16.0	
			End at 17.6'			

0
2.5
5
7.5
10
12.5
15
17.5
20
22.5

D1-9

ENG FORM 1836 PREVIOUS EDITIONS ARE OBSOLETE. PROJECT HOLE NUMBER

DRILLING LOG

DIVISION

INSTALLATION

SHEET 1 OF 1

1. PROJECT Pt. Monmouth
2. LOCATION (*Coordinates or Station*) 2155623 E
595415.4 N
3. DRILLING AGENCY Alpine Ocean Seismic Survey, Inc.
4. HOLE NO. (*As shown on drawing title and file number*) CP110
5. NAME OF DRILLER Alpine Ocean Seismic Survey, Inc.
6. DIRECTION OF HOLE ☒ VERTICAL ☐ INCLINED
7. WATER DEPTH
8. DEPTH DRILLED INTO ROCK
9. TOTAL DEPTH OF HOLE 17.0'
10. SIZE AND TYPE OF BIT 4" Vibracore
11. DATUM FOR ELEVATION SHOWN (*TBM or MSL*) MLW
12. MANUFACTURER'S DESIGNATION OF DRILL Alpine Pneumatic Vibracore
13. TOTAL NO. OF OVERBURDEN SAMPLES TAKEN disturbed: undisturbed:
14. TOTAL NUMBER OF CORE BOXES
15. ELEVATION GROUND WATER
16. DATE HOLE STARTED 5/15/96 COMPLETED 5/15/96
17. ELEVATION TOP OF HOLE -17.8 ft
18. TOTAL CORE RECOVERY FOR BORING
19. SIGNATURE OF GEOLOGIST G. Zarillo

ELEV.	DEPTH	LEGEND	CLASSIFICATION OF MATERIALS (Description)	CORE REC %	SAMPLE NUMBER	REMARKS
-17.8	0.0					
-24.4	6.6	▨	Dark-gray, inorganic clay, medium plasticity. (CL)	100	0.5	
-26.9	9.1	⁘	Dark gray, muddy, fine sand. (SM)		8.0	6.6'–17.0' Composite sample
-28.5	10.7	⁘	Medium-fine sand, few quartz pebbles, iron-stained. (SP)	100		
-32.9	15.1	⁘	Fine sand, iron-stained, black organic material. (SP)		12.0	10.7'–15.1' Bands of blackened sand from organic material
-34.8	17.0	⁘	Fine sand, iron-stained. (SP)	100	16.0	Sand blackened by organic material near bottom
			End at 17.0'			

0
2.5
5
7.5
10
12.5
15
17.5
20
22.5

D1-10

DRILLING LOG

DIVISION

INSTALLATION

SHEET 1
OF 1

1. PROJECT Pt. Monmouth

2. LOCATION (*Coordinates or Station*) 2155015.7 E
594406.1 N

3. DRILLING AGENCY Alpine Ocean Seismic Survey, Inc.

4. HOLE NO. (*As shown on drawing title and file number*) CP111

5. NAME OF DRILLER Alpine Ocean Seismic Survey, Inc.

6. DIRECTION OF HOLE
☒ VERTICAL ☐ INCLINED

7. WATER DEPTH

8. DEPTH DRILLED INTO ROCK

9. TOTAL DEPTH OF HOLE 18.7'

10. SIZE AND TYPE OF BIT 4" Vibracore

11. DATUM FOR ELEVATION SHOWN (*TBM or MSL*) MLW

12. MANUFACTURER'S DESIGNATION OF DRILL
Alpine Pneumatic Vibracore

13. TOTAL NO. OF OVERBURDEN SAMPLES TAKEN
disturbed: undisturbed:

14. TOTAL NUMBER OF CORE BOXES

15. ELEVATION GROUND WATER

16. DATE HOLE STARTED 5/16/96 COMPLETED 5/16/96

17. ELEVATION TOP OF HOLE -13.6 ft

18. TOTAL CORE RECOVERY FOR BORING

19. SIGNATURE OF GEOLOGIST G. Zarillo

ELEV.	DEPTH	LEGEND	CLASSIFICATION OF MATERIALS (Description)	CORE REC %	SAMPLE NUMBER	REMARKS
-13.6	0.0					
-16.4	2.8		Brown, fine sand, some mud content. (SP)	100	0.5	
-20.2	6.6		Gray, fine, muddy sand. (SM)		5.0	
-24.4	9.8		Gray–brown, medium-fine sand, some mud, few shell fragments and small pebbles. (SP)	100	8.0	
-25.0	11.4		Dark gray, muddy, fine sand. (SM)		10.0	
-29.1	15.5		Gray, fine sand. (SP)			
-32.3	18.7		Fine sand, iron-stained, pebbles. (SP)	100	16.0	
			End at 18.7'			

0
2.5
5
7.5
10
12.5
15
17.5
20
22.5

ENG FORM 1836 PREVIOUS EDITIONS ARE OBSOLETE. PROJECT

VIBRACORE LOG

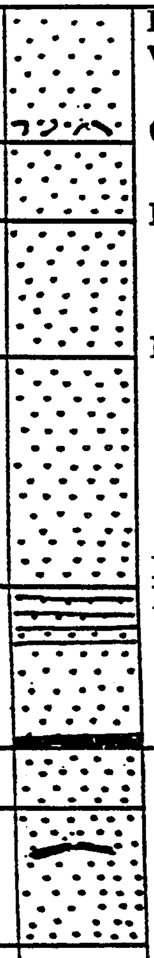
Project: Port Monmouth Feasibility Study **Area:** Sandy Hook

Core No. 11A **Run No.** 1 **Date:** 10/17/97

Grid Position: **Easting** 2182980.2 **Northing** 600221.7

Geographic Position: **Lat.** 40° 28' 45.387" **Long.** 74° 00' 32.066"

Water Depth		Vibration Time		Core Depth	
Uncorrected:	9.8	Stop:	16:03	Penetration:	19.42
Tide:	+0.25	Start:	15:59	Recovery:	17.0
= Corrected:	10.0	= Elapsed:	4 min.		



0	Elev. -10.0	Depth	Legend	Classification	Sample No.	Remarks
1				Light Brown/Grey Fine Sand (SP) with Shell Fragments		
2	-12.5	2.5		Clam Shells		
3						
4	-14.0	4.0		Light Grey Fine to Medium Sand (SP)		
5						
6	-16.5	6.5		Brownish/Grey Fine Sand (SP)	1	6' - 6.5'
7						
8						
9						
10	-20.5	10.5		Dark Grey - Streaks of Black Organics - Fine to Medium Sand		
11						
12						
13	-23.3	13.3		Black Organics 13.3' - 13.4'		
14	-24.5	14.5			2	14 - 14.5'
15				15.7' Wood Very Dark Gray Fine to Medium Sand		
16						
17	-27.0	17.0				
18						
19						
20						Composite 0-17.0'

ALPINE OCEAN SEISMIC SURVEY, INC.

D1-12

VIBRACORE LOG

Project: Port Monmouth Feasibility Study **Area:** Sandy Hook

Core No. 11B **Run No.** 1 & 2 **Date:** 10/17/97

Grid Position: **Easting** 2184225.1 **Northing** 600633.1

Geographic Position: **Lat.** 40° 28' 49.360" **Long.** 74° 00' 15..917"

Water Depth		**Vibration Time**		**Core Depth**	
Uncorrected:	19.8	**Stop:**	14:59	**Penetration:**	16.25 (1),19.7 (2)
Tide:	+0.8	**Start:**	14:55	**Recovery:**	13.6 (1)
= Corrected:	20.6	**= Elapsed:**	4 min.		13.3-18.0 (2)

0	Elev. -20.6	Depth	Legend	Classification	Sample No.	Remarks
1			⁘	Light Brown Fine to Medium Sand (SP)		
2			⁘			
3			⁘		1	2.0' - 2.5'
4	-24.6	4.0	≡	Dark Gray/Black Fine to Medium Sand.Organics Wood Fragments		
	-25.0	4.4				
5			⁘			
6			⁘			
7			⁘	Brownish/ Dark Gray Fine to Medium Sand (SP)		
8			⁘			
9			⁘			
10			⁘			
11			⁘			
12			⁘			
13	-33.6	13.0	⁘	Dark Brown Sand	2	12.0' - 12.5'
14			⁘			
15			⁘			
16	-36.6	16.0	⁘	Dark Brownish Gray Fine to Medium Sand (SP) with shells, lenses of Black Orgsnics		
17			⁘ ≡		3	16.0' - 16.5'
18	-38.0	18.0	≡			
19						Composite Runs 1 & 2 0 - 18.0'
20						

VIBRACORE LOG

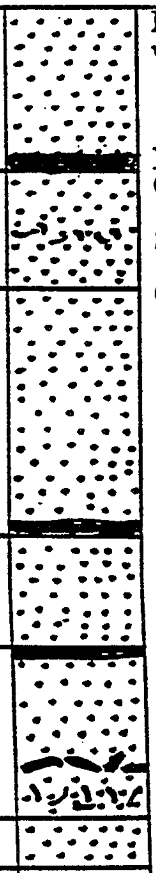
Project: Port Monmouth Feasibility Study **Area:** Sandy Hook

Core No. 12A **Run No.** 1 **Date:** 10/17/97

Grid Position: **Easting** 2187129.8 **Northing** 600717.1

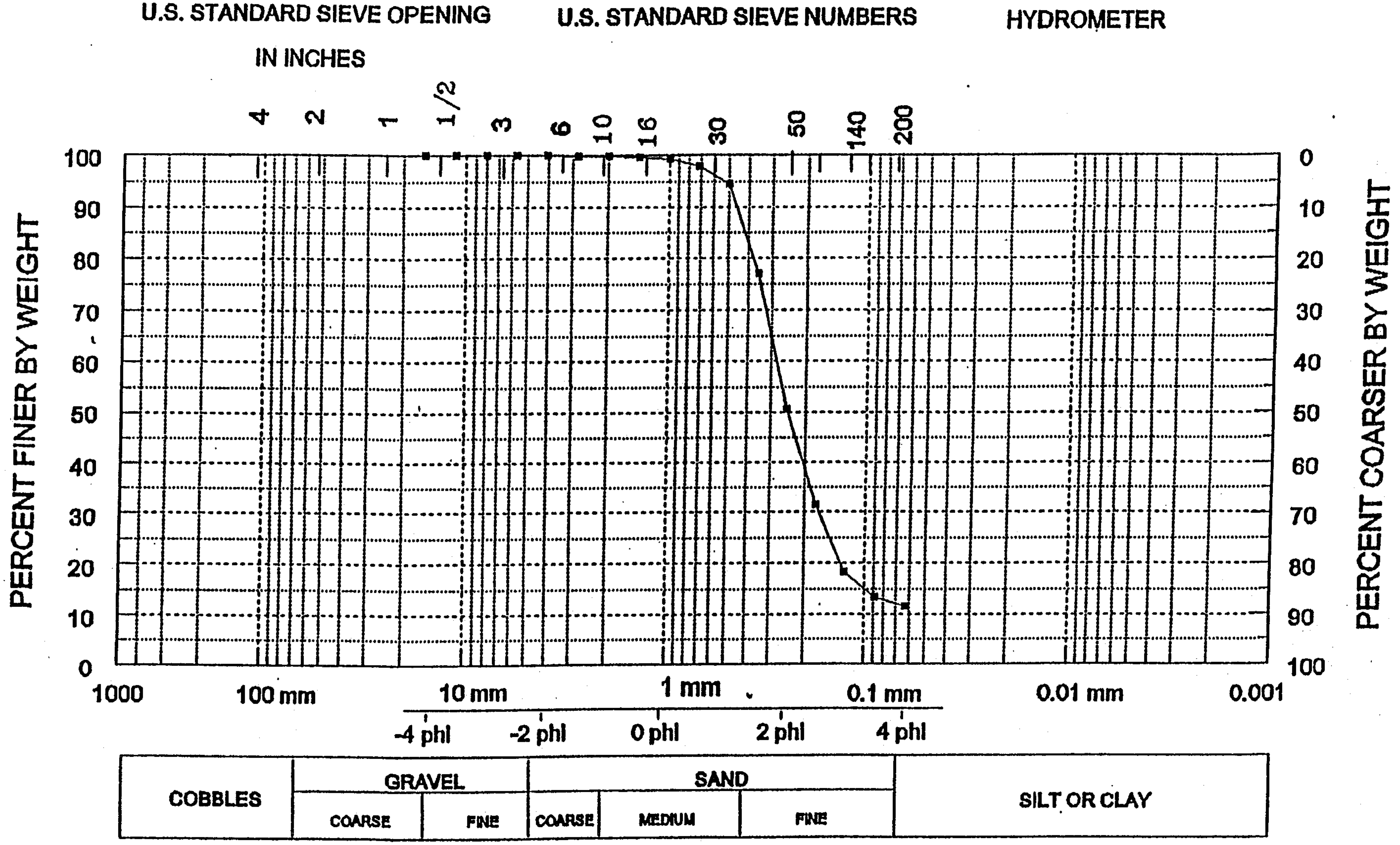
Geographic Position: **Lat.** 40° 28' 49.973" **Long.** 73° 59' 38.321"

Water Depth		**Vibration Time**		**Core Depth**	
Uncorrected:	33.8	**Stop:**	11:59	**Penetration:**	19.6
Tide:	-2.2	**Start:**	11:56	**Recovery:**	15.5
= Corrected:	31.6	**= Elapsed:**	3 min.		

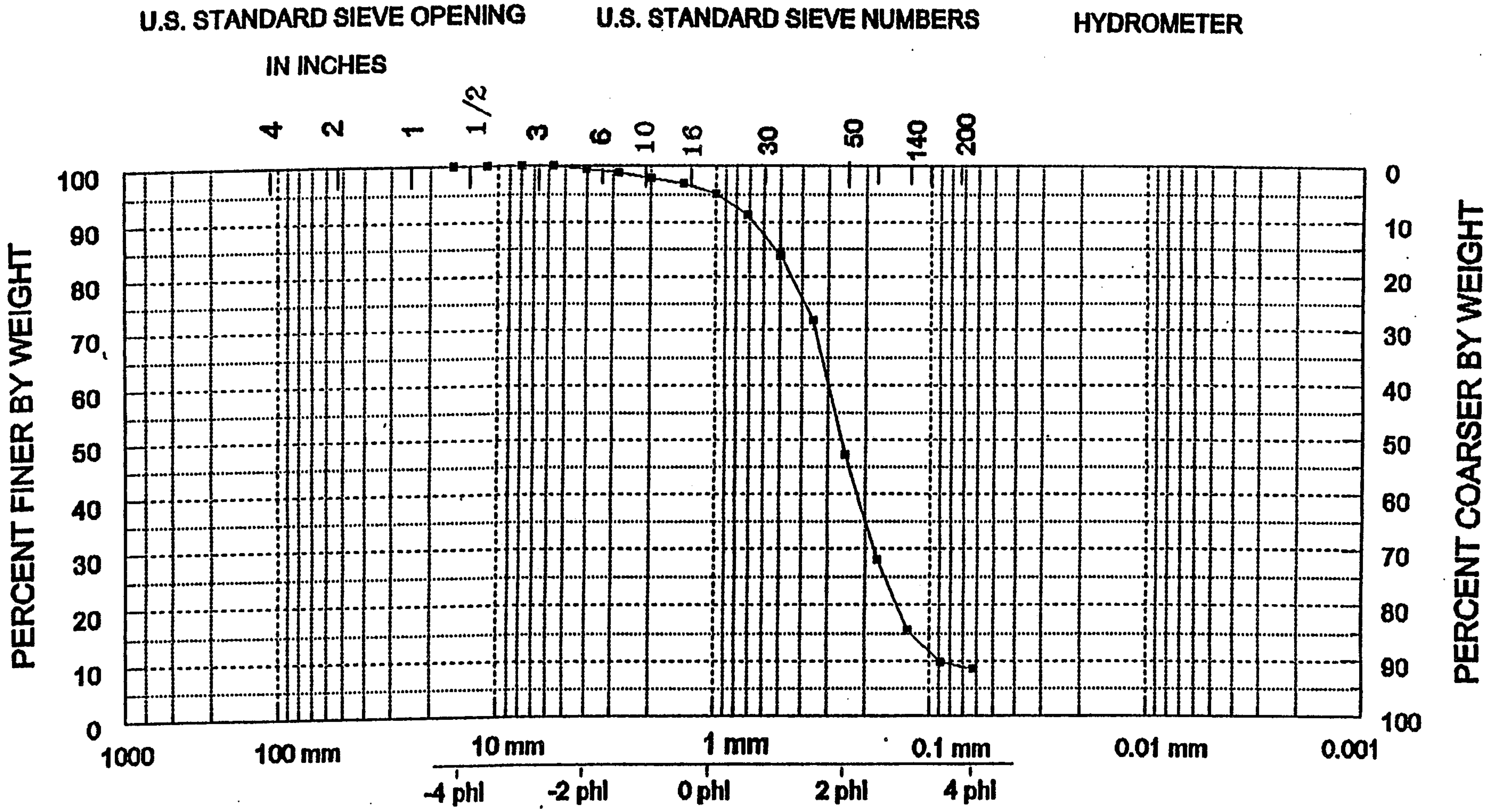


0	Elev. -31.6	Depth	Legend	Classification	Sample No.	Remarks
1				Light Brown/Gray Fine - Medium Sand with Shell Hash (SP)		
2					1	1.5 - 1.8'
3	-34.6	3.0		Black Organic Layer Gray, Fine to Medium Sand (SP)		
4				Shells		
5	-36.6	5.0		Grading to Brown Below 5'		
6						
7					2	7.0 - 7.5'
8						
9	-41.2	9.6		9.6 to 9.8' Black Silty Organic Layer Brown/Gray Fine to Medium Sand (SP)		
10						
11	-43.3	11.7		11.7 - 11.8 Black Organics Gray Fine to Medium Sand (SP)		
12						
13					3	12.5 - 13.0'
14	-46.1	14.5		13.9 - 14.3 Wood Fragments * 14.5 Dark Gray Sand with Shells		* Fragment of Dressed Lumber 1 1/4"x7/8"
15	-47.1	15.5				
16						
17						
18						
19						
20						Composite 0 - 15.5'

ALPINE OCEAN SEISMIC SURVEY, INC.



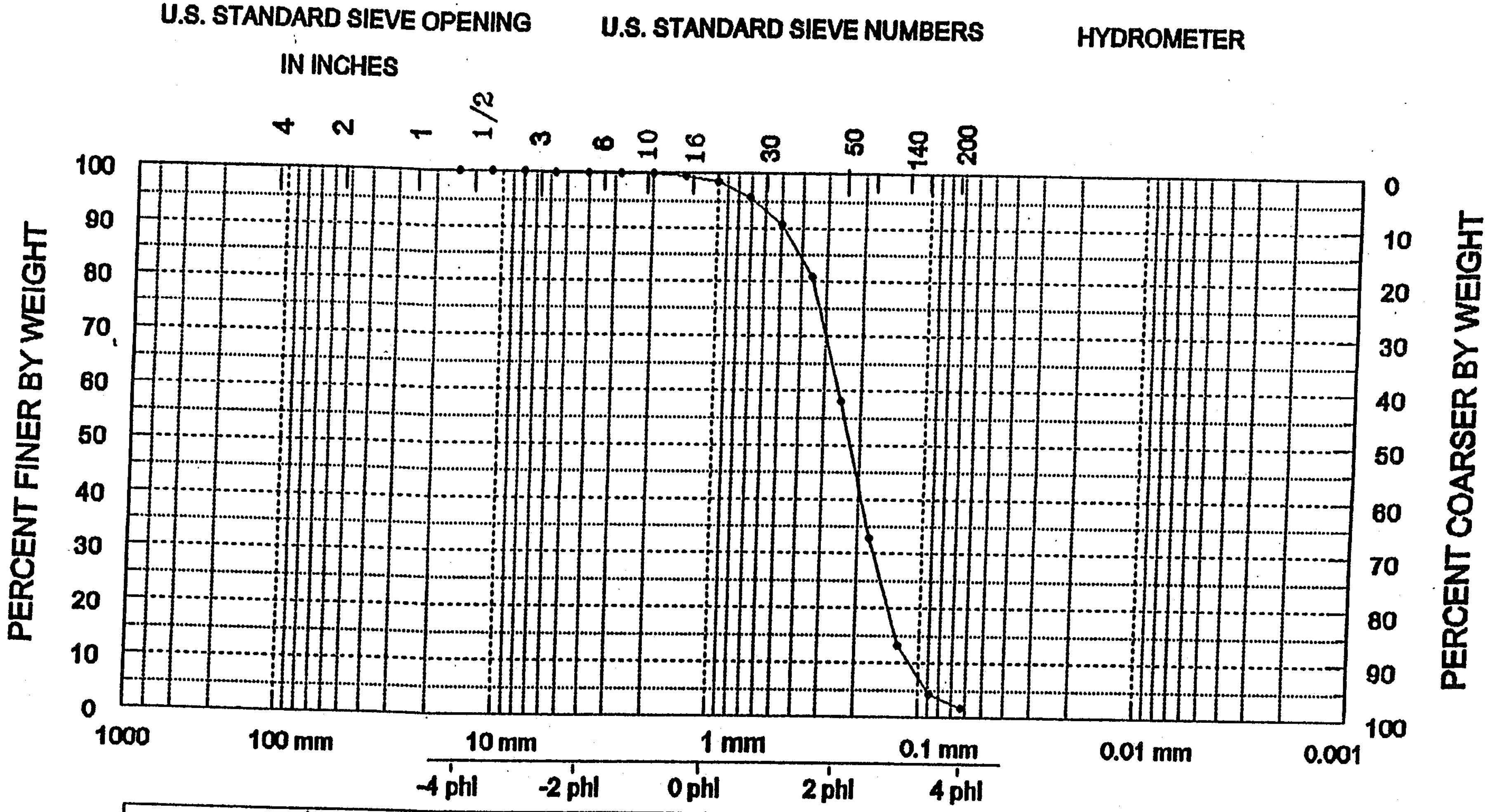
SAMPLE NO.	ELEV.	CLASSIFICATION	PROJECT Sand Borrow Area
CP111-Comp		Fine sand SP	AREA Pt. Monmouth, NJ
			BORING NO. CP111
			DATE June 25, 1996



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

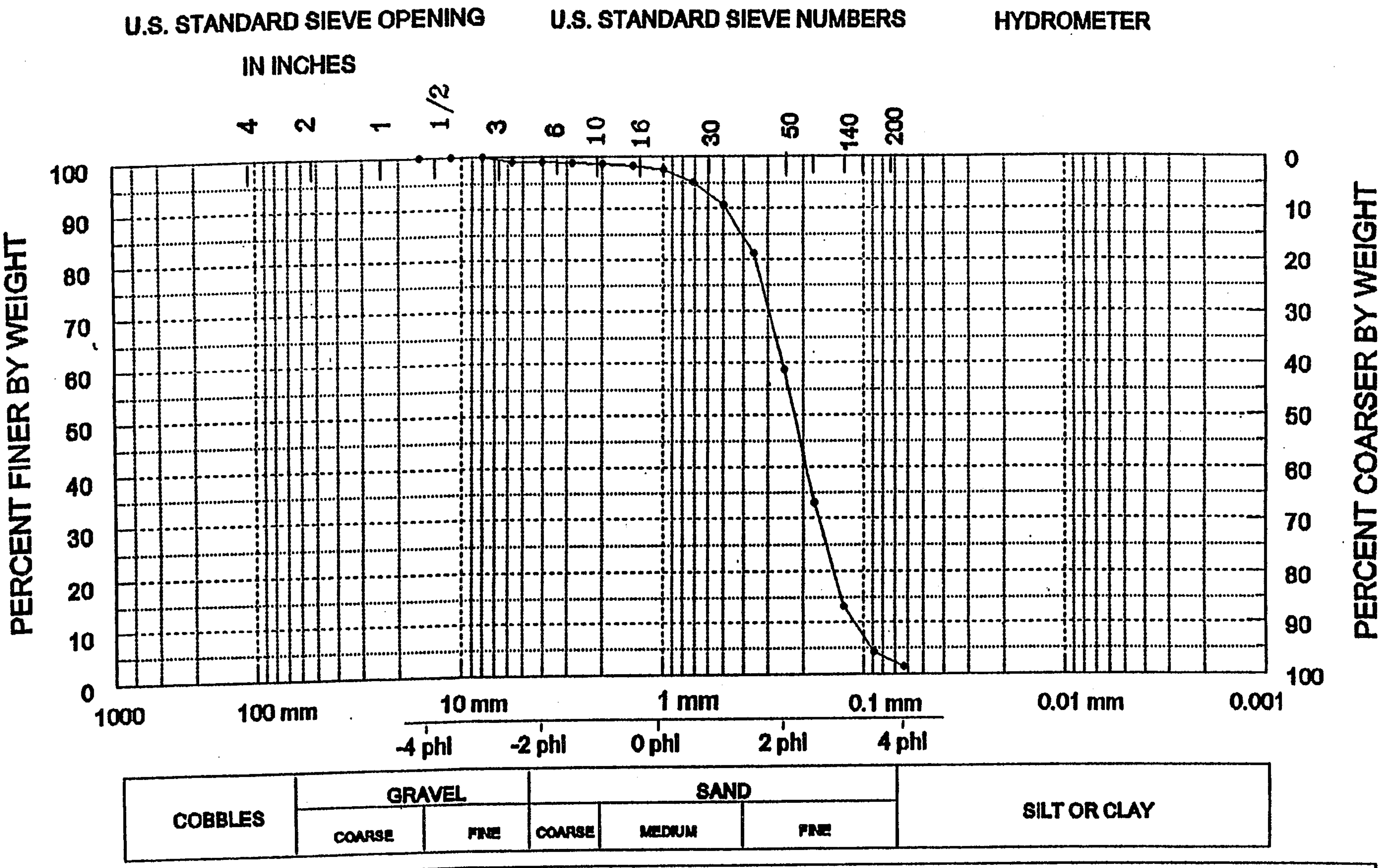
SAMPLE NO.	ELEV.	CLASSIFICATION	PROJECT Sand Borrow Area
CP110-Comp		Fine sand SP	AREA Pt. Monmouth, NJ
			BORING NO. CP110
			DATE June 25, 1996

D1-16



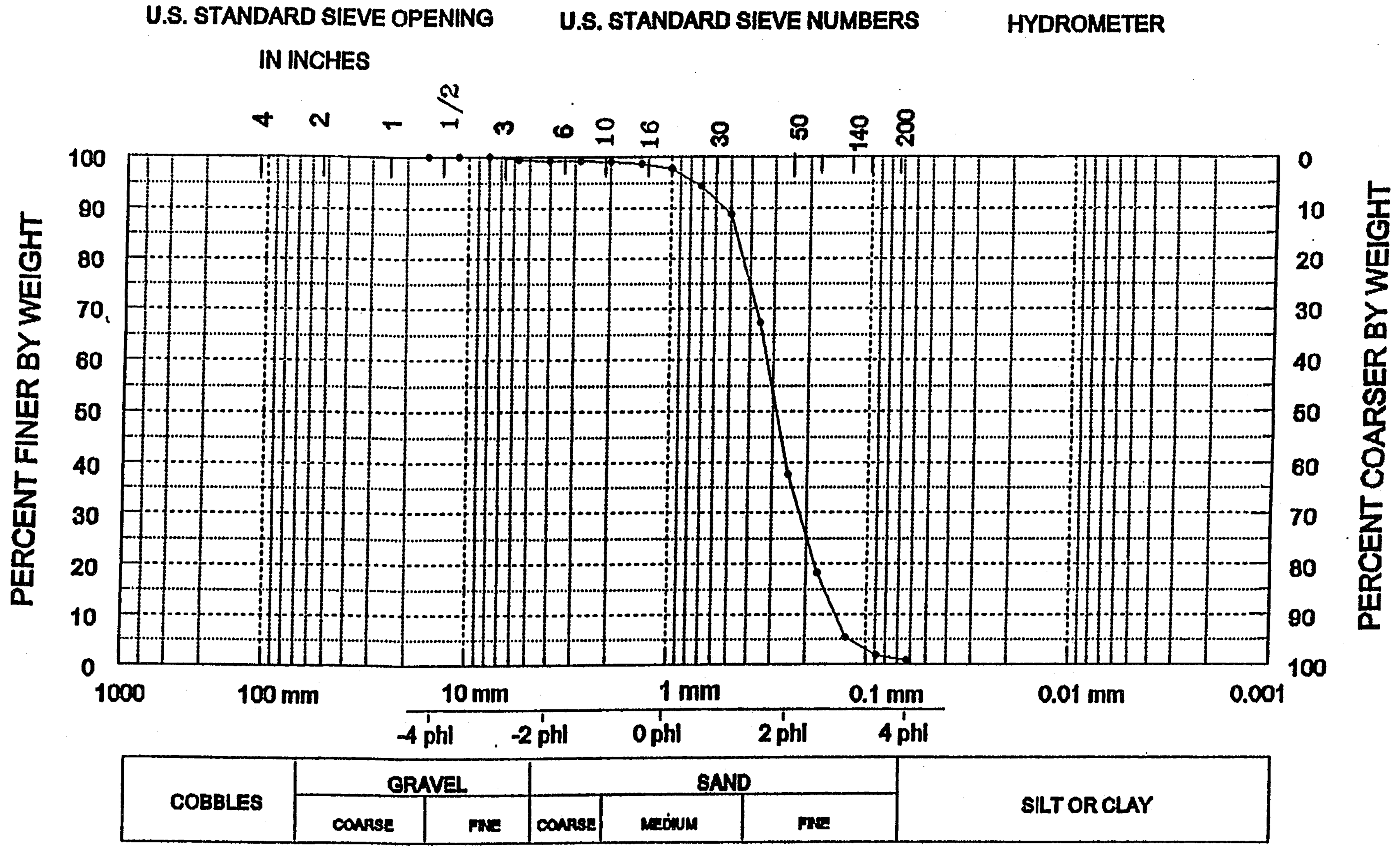
COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

SAMPLE NO.	ELEV.	CLASSIFICATION	PROJECT Sand Borrow Area
CP 109-CompR		Fine sand SP	AREA Pt. Monmouth, NJ
			BORING NO. CP109
			DATE June 25, 1996

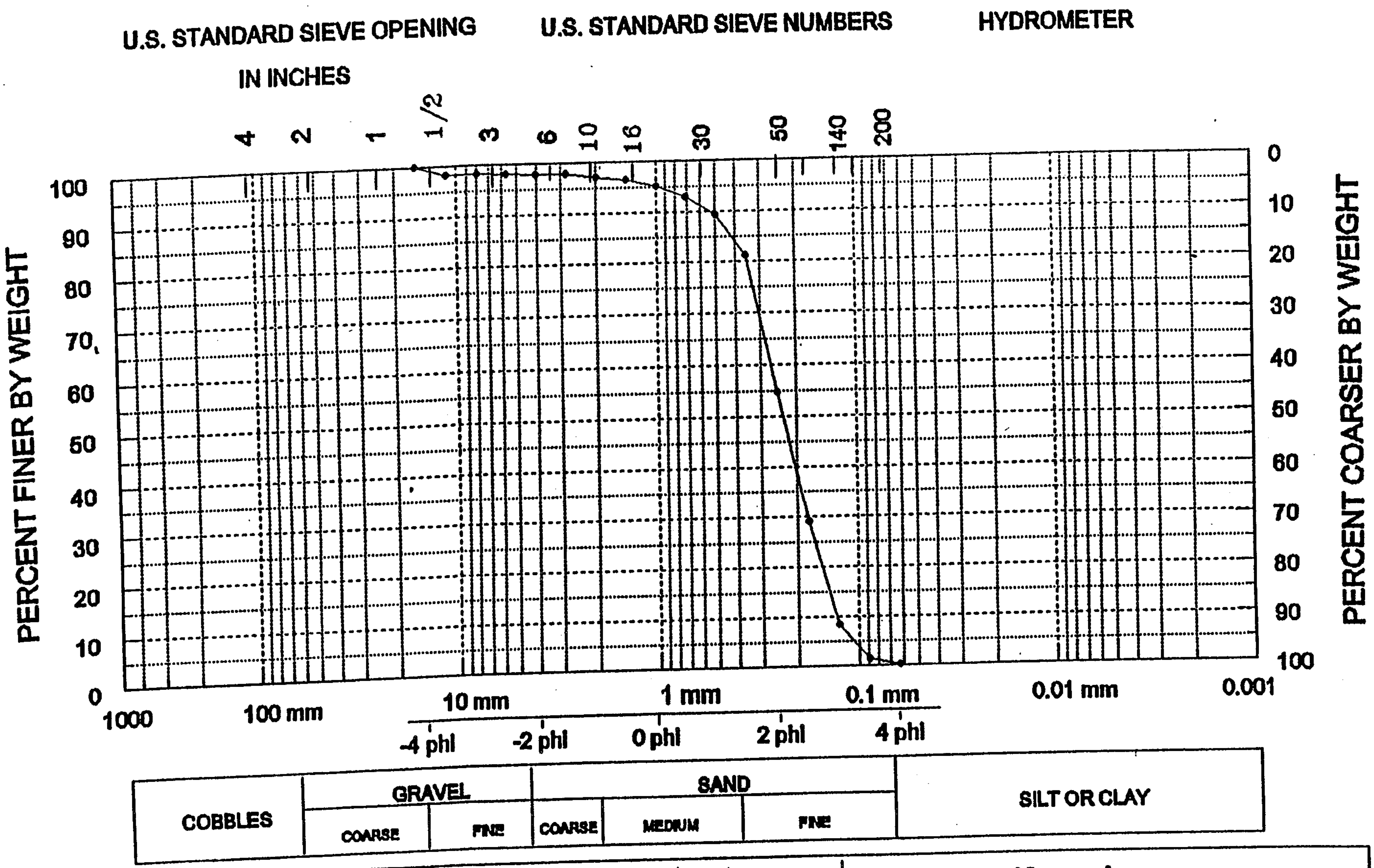


SAMPLE NO.	ELEV.	CLASSIFICATION	PROJECT Sand Borrow Area
CP 109-Comp		Fine sand SP	AREA Pt. Monmouth, NJ
			BORING NO. CP109
			DATE June 25, 1996

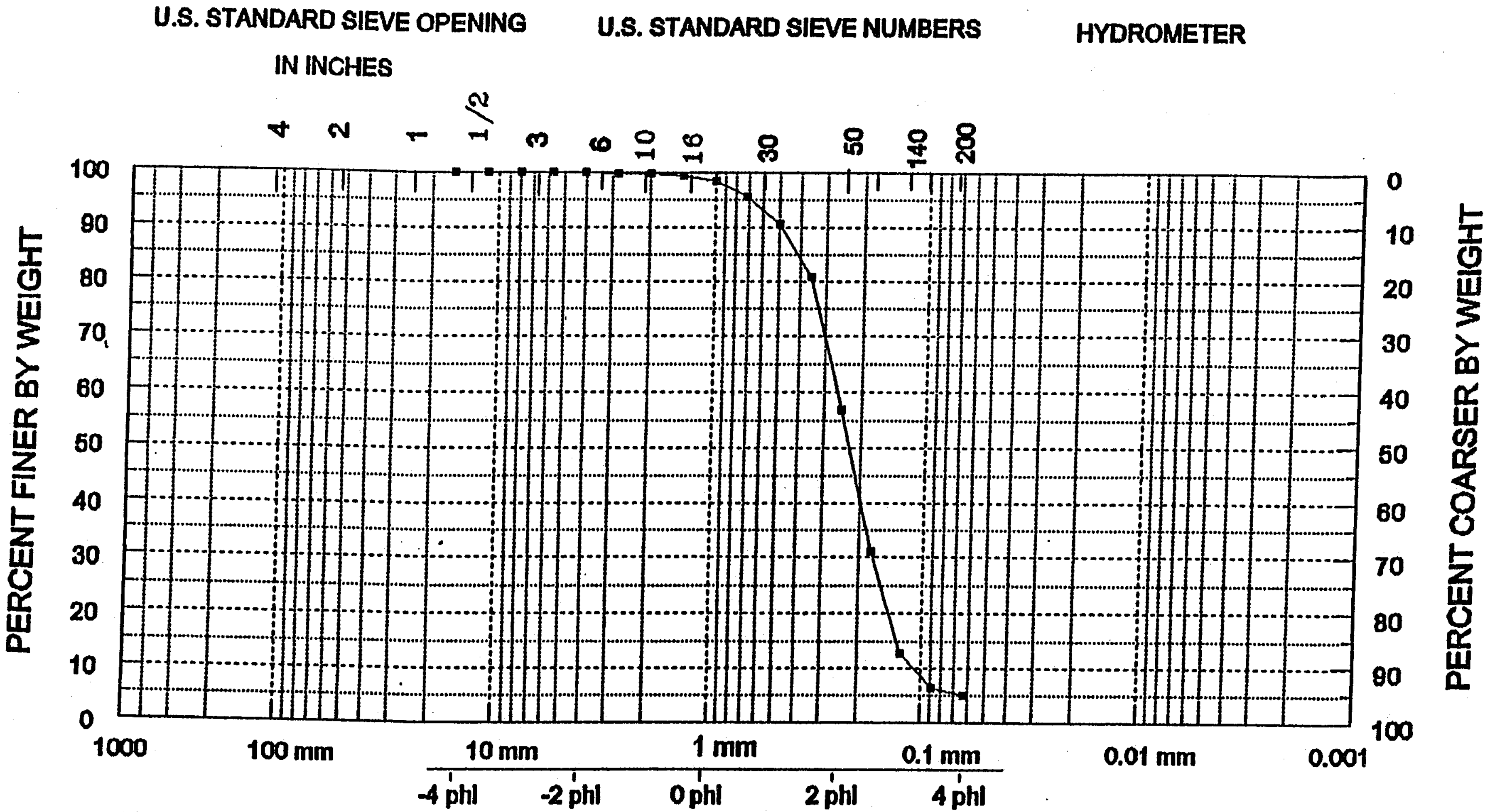
D1-18



SAMPLE NO.	ELEV.	CLASSIFICATION	PROJECT Sand Borrow Area
CP 108-Comp		Fine sand SP	AREA Pt. Monmouth, NJ
			BORING NO. CP108
			DATE June 25, 1996

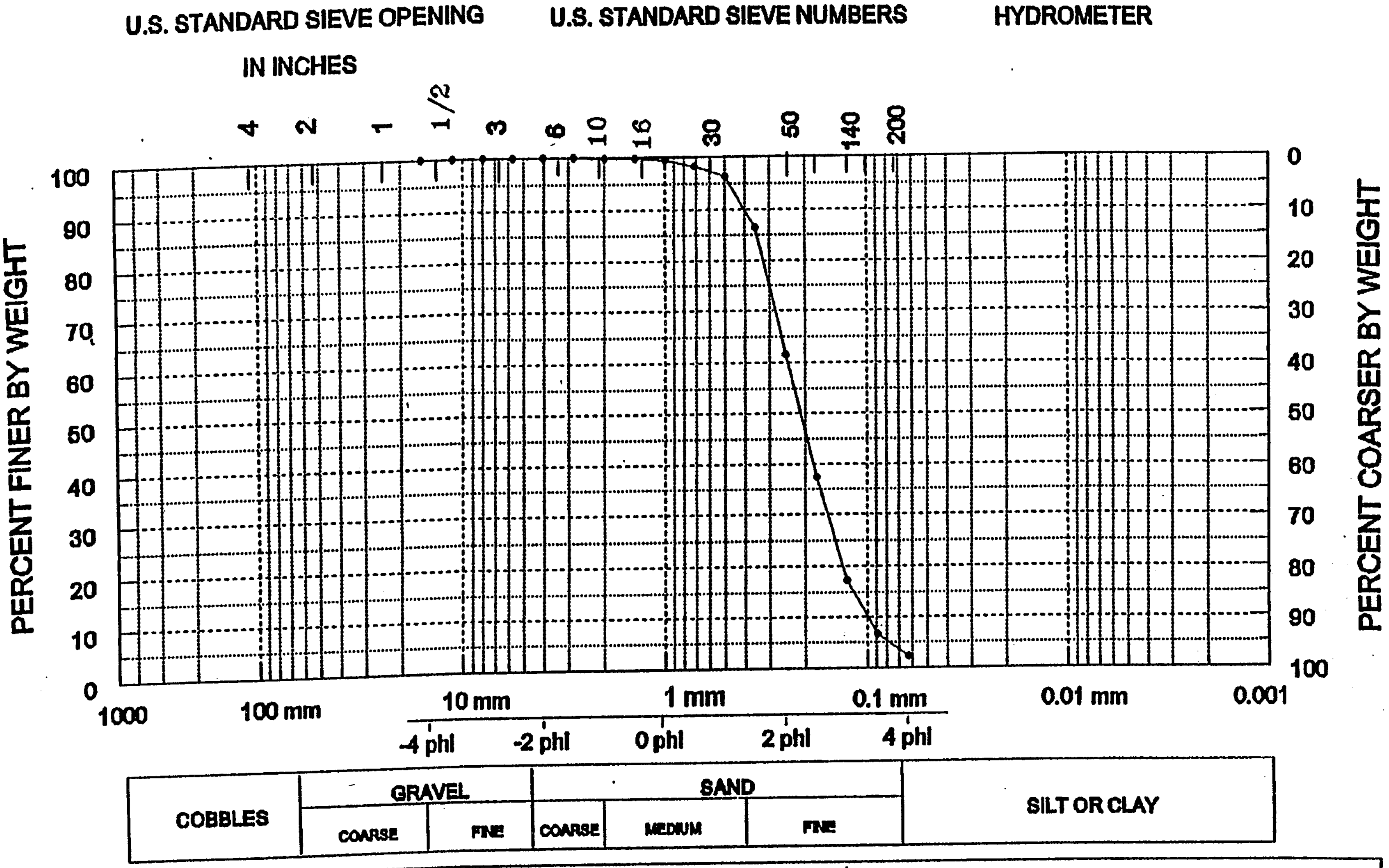


SAMPLE NO.	ELEV.	CLASSIFICATION	PROJECT Sand Borrow Area
CP 106-Comp		Fine sand SP	AREA Pt. Monmouth, NJ
			BORING NO. CP106
			DATE June 25, 1996

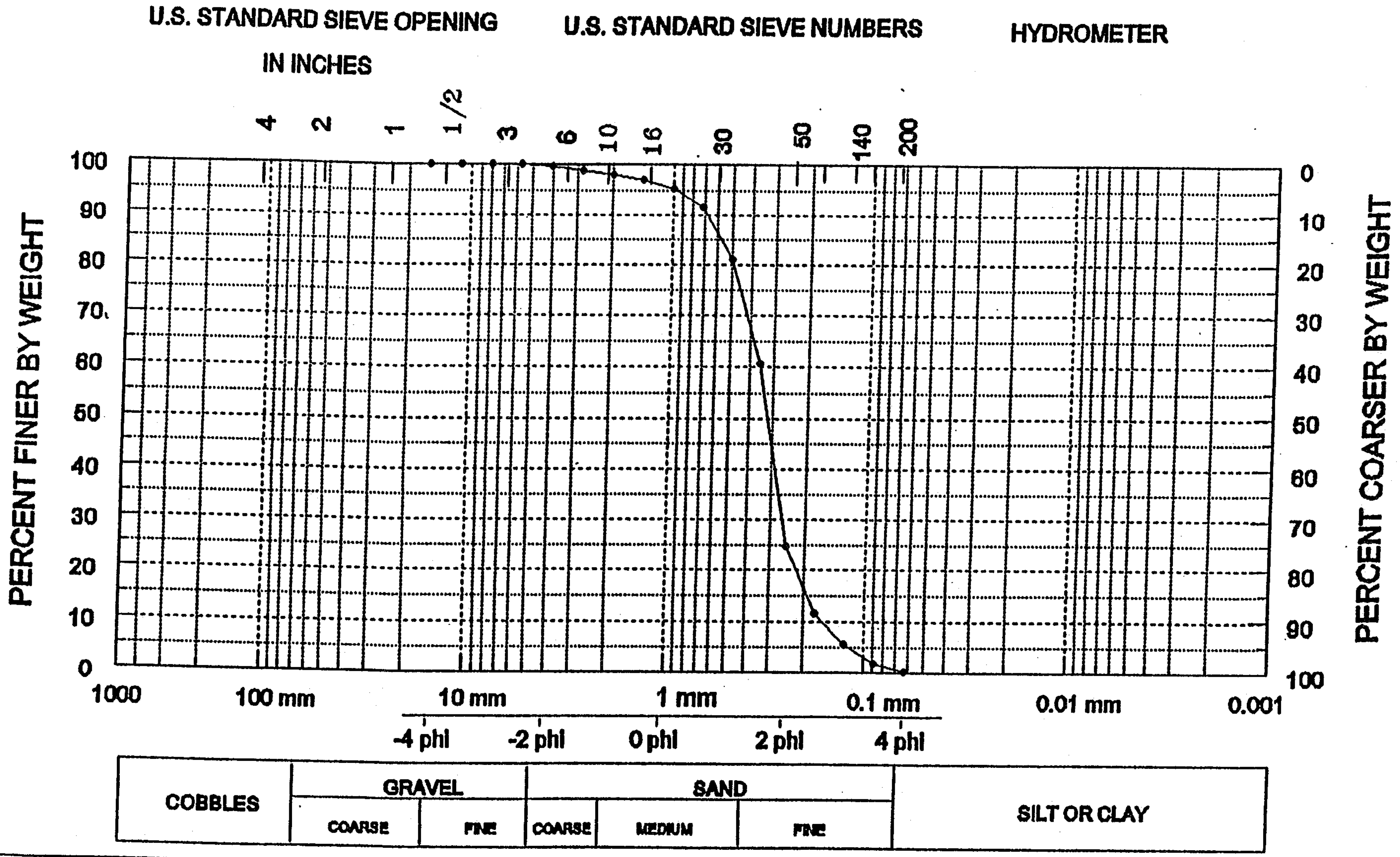


COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

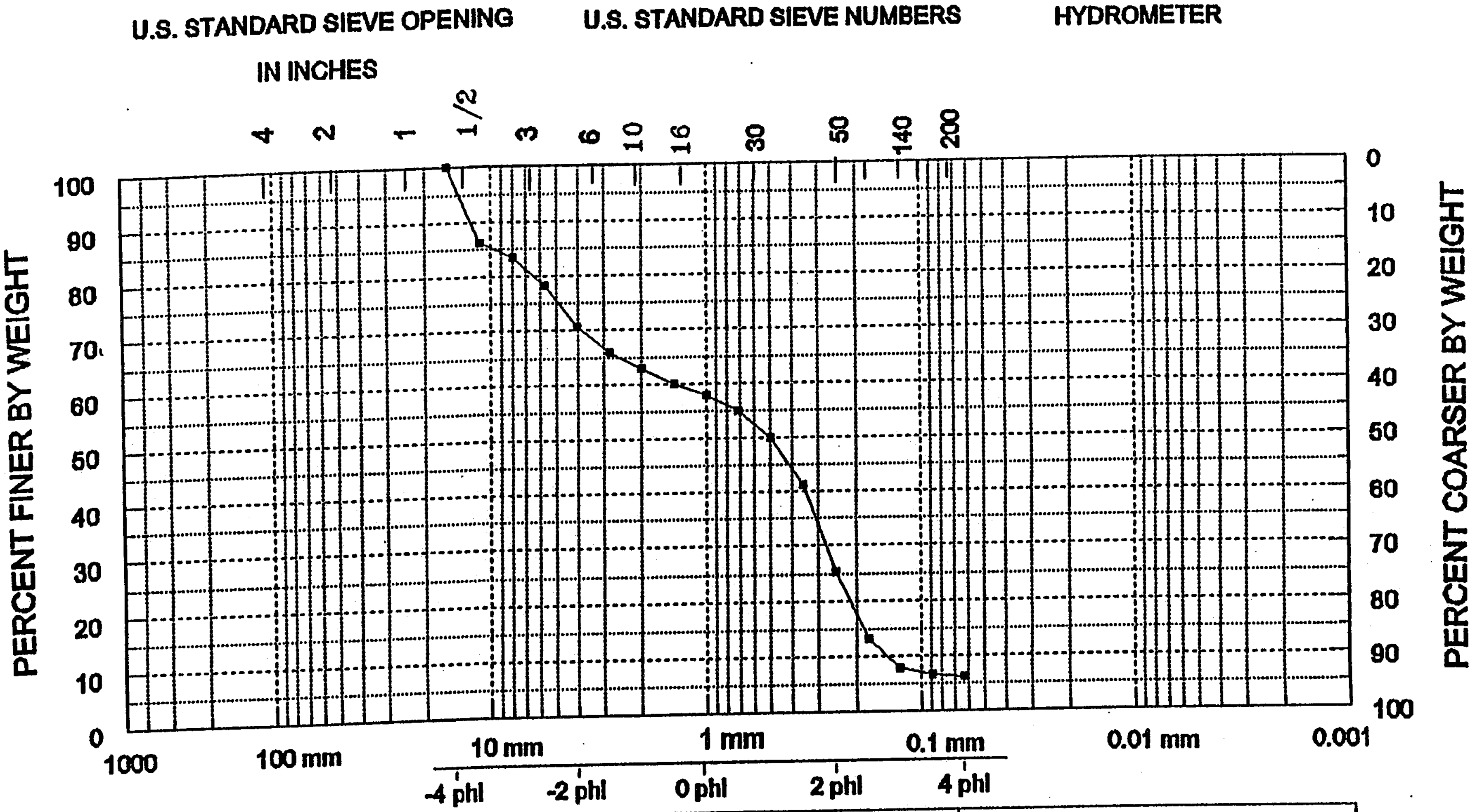
SAMPLE NO.	ELEV.	CLASSIFICATION	PROJECT Sand Borrow Area
CP105-Comp		Fine sand SP	AREA Pt. Monmouth, NJ
			BORING NO. CP105
			DATE June 25, 1996



SAMPLE NO.	ELEV.	CLASSIFICATION	PROJECT Sand Borrow Area
CP 104-Comp		Fine sand SP	AREA Pt. Monmouth, NJ
			BORING NO. CP104
			DATE June 25, 1996

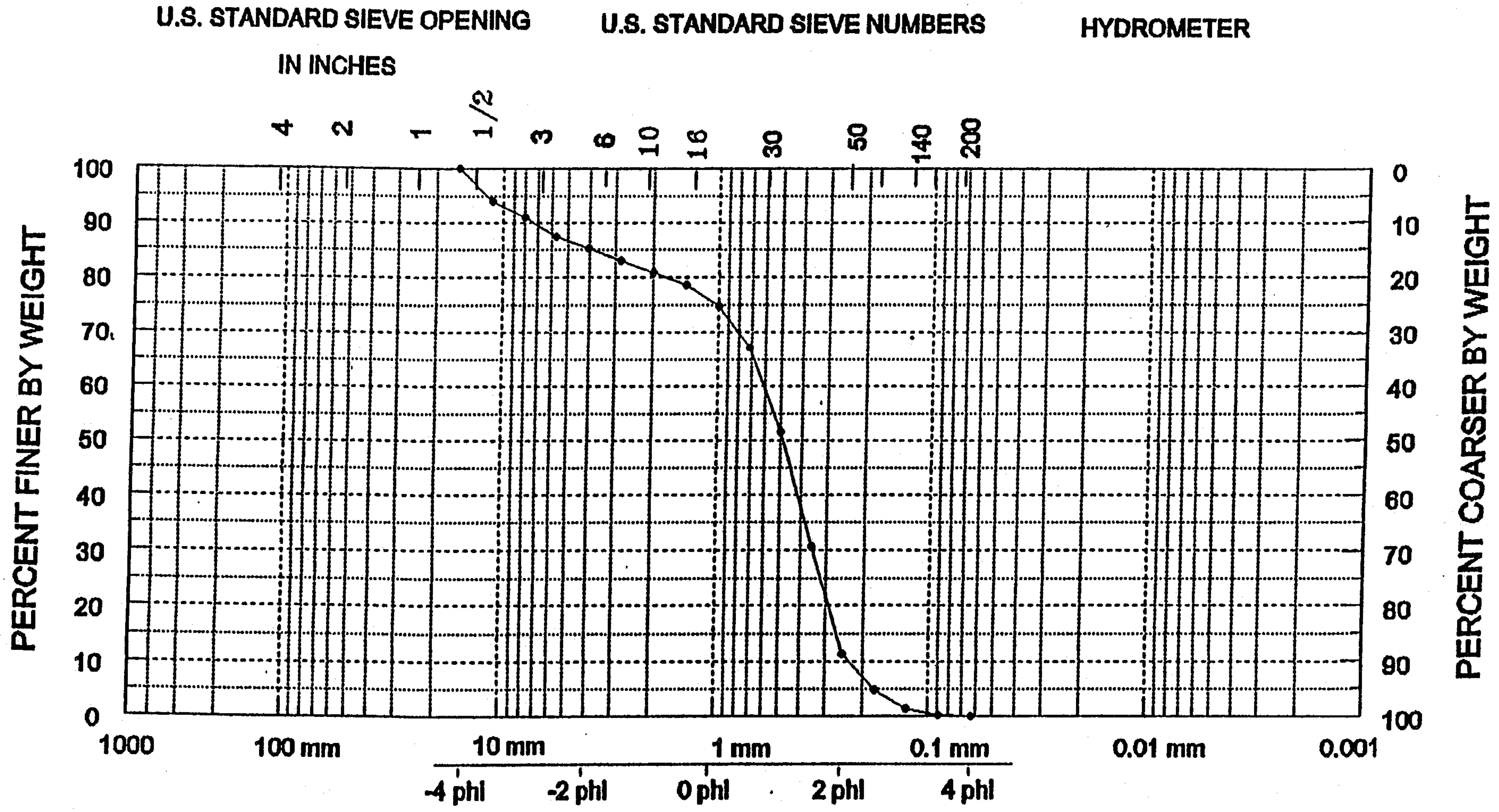


SAMPLE NO.	ELEV.	CLASSIFICATION	PROJECT Sand Borrow Area
CP 103-Comp		Fine sand	AREA Pt. Monmouth, NJ
			BORING NO. CP103
			DATE June 25, 1996



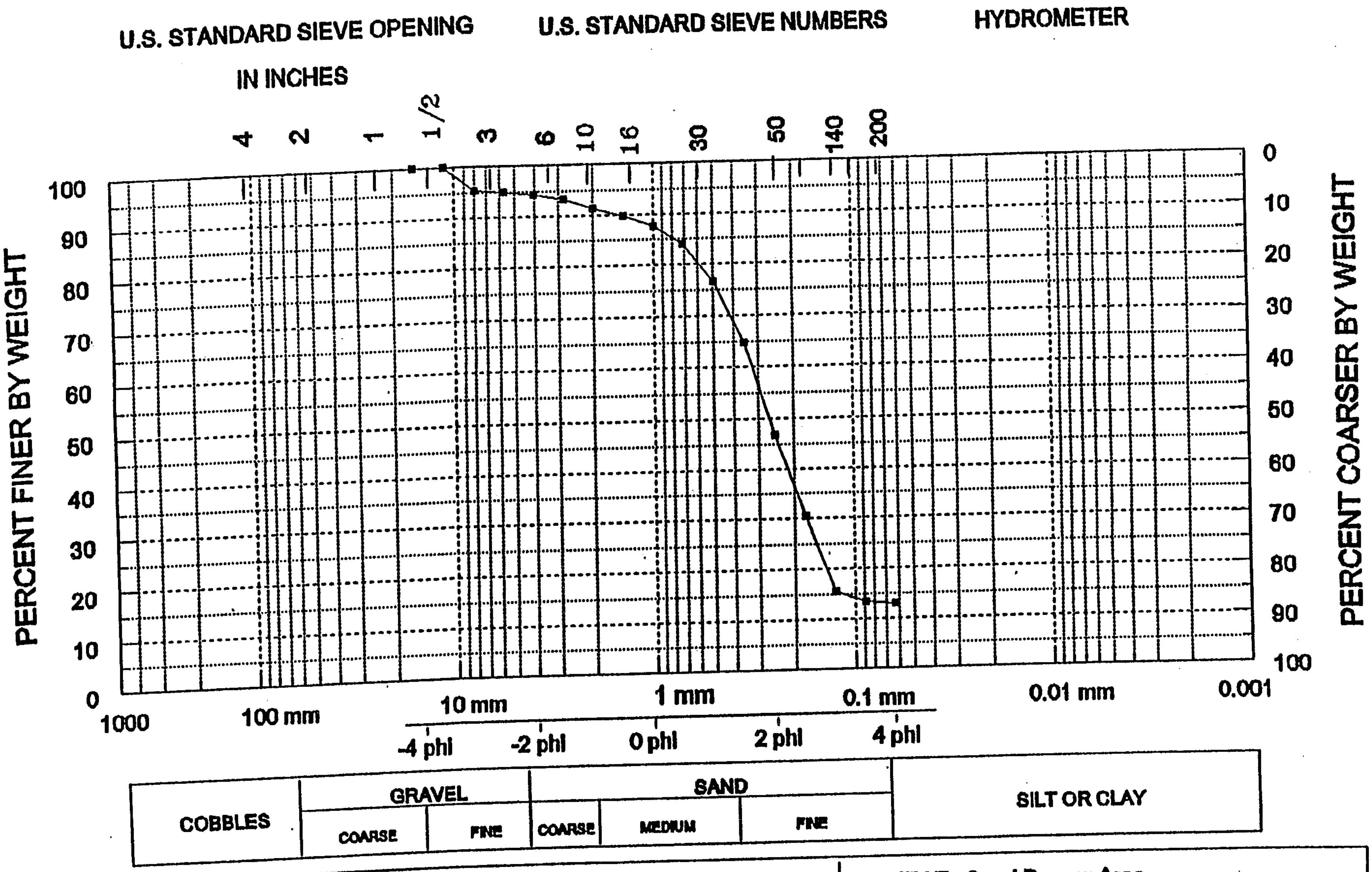
COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

SAMPLE NO.	ELEV.	CLASSIFICATION	PROJECT Sand Borrow Area
CP101-Comp		Gravel and Sand SP	AREA Pt. Monmouth, NJ
			BORING NO. CP101
			DATE June 25, 1996

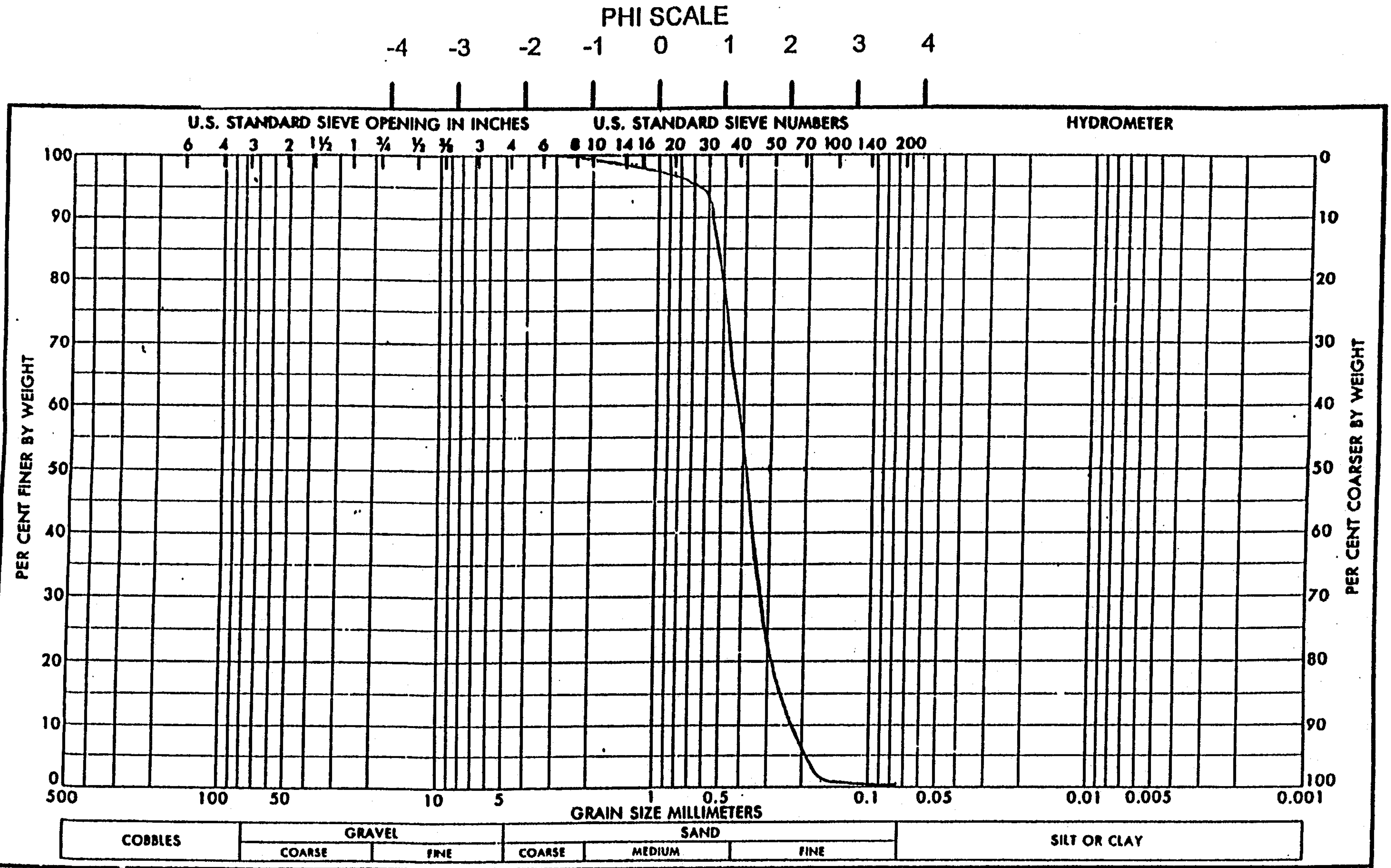


COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

SAMPLE NO.	ELEV.	CLASSIFICATION	PROJECT Sand Borrow Area
CP 102-Comp		Medium to fine sand SP	AREA Pt. Monmouth, NJ
			BORING NO. CP102
			DATE June 25, 1996



SAMPLE NO.	ELEV.	CLASSIFICATION	PROJECT Sand Borrow Area
CP100-Comp		Fine sand SP	AREA Pt. Monmouth, NJ
			BORING NO. CP100
			DATE June 25, 1996



SAMPLE NO.	ELEV OR DEPTH	CLASSIFICATION	NAT W%	LL	PL	PI
Composite 12A		Gray fine/medium Sand (SP)				

PROJECT O.S.S.

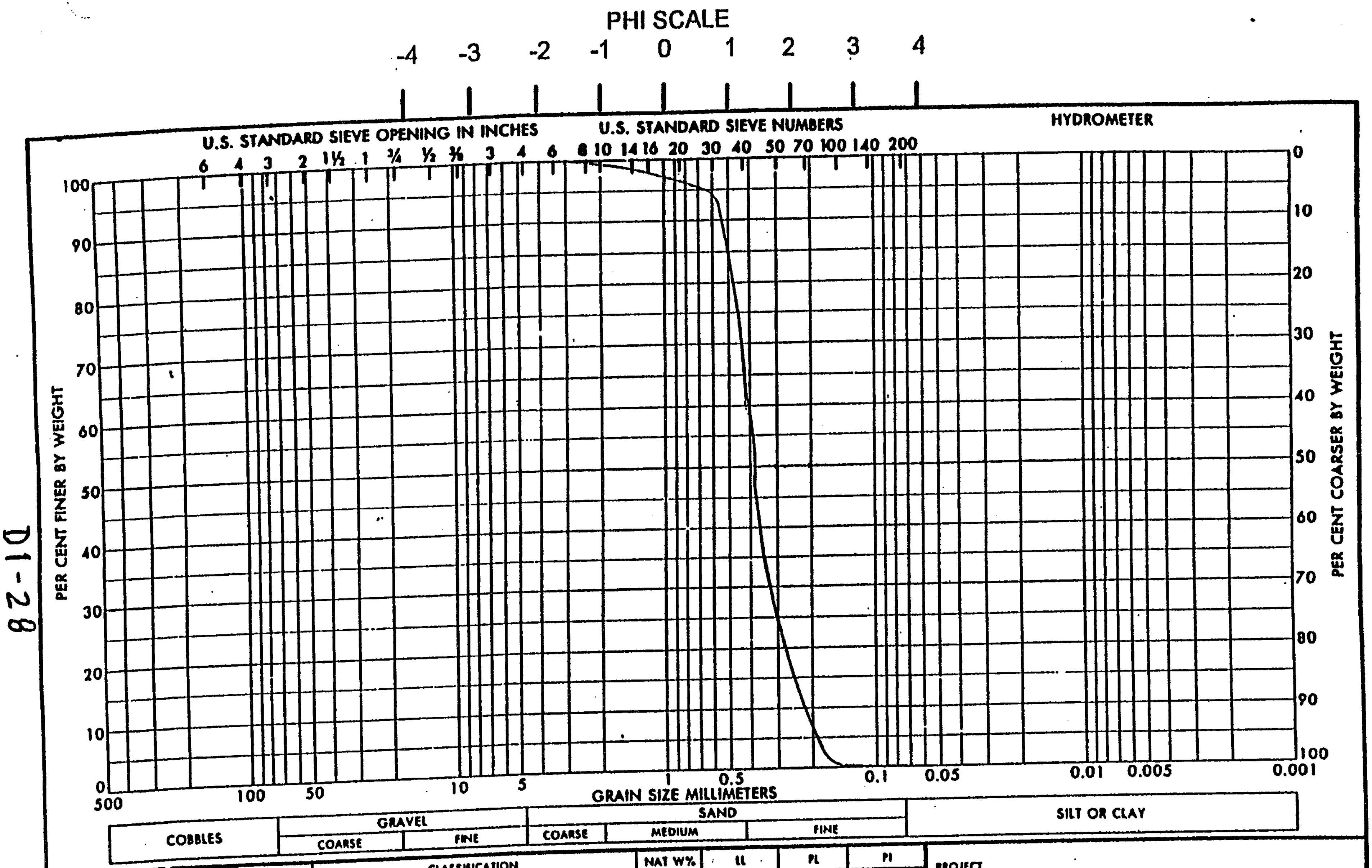
AREA Sandy Hook

BORING NO.

DATE December 18, 1997

GRADATION CURVES

JOHNSON SOILS ENGINEERING LABORATORY



D1-28

SAMPLE NO.	ELEV OR DEPTH	CLASSIFICATION	NAT W%	LL	PL	PI
Composite		Gray fine/medium Sand (SP)				

GRADATION CURVES

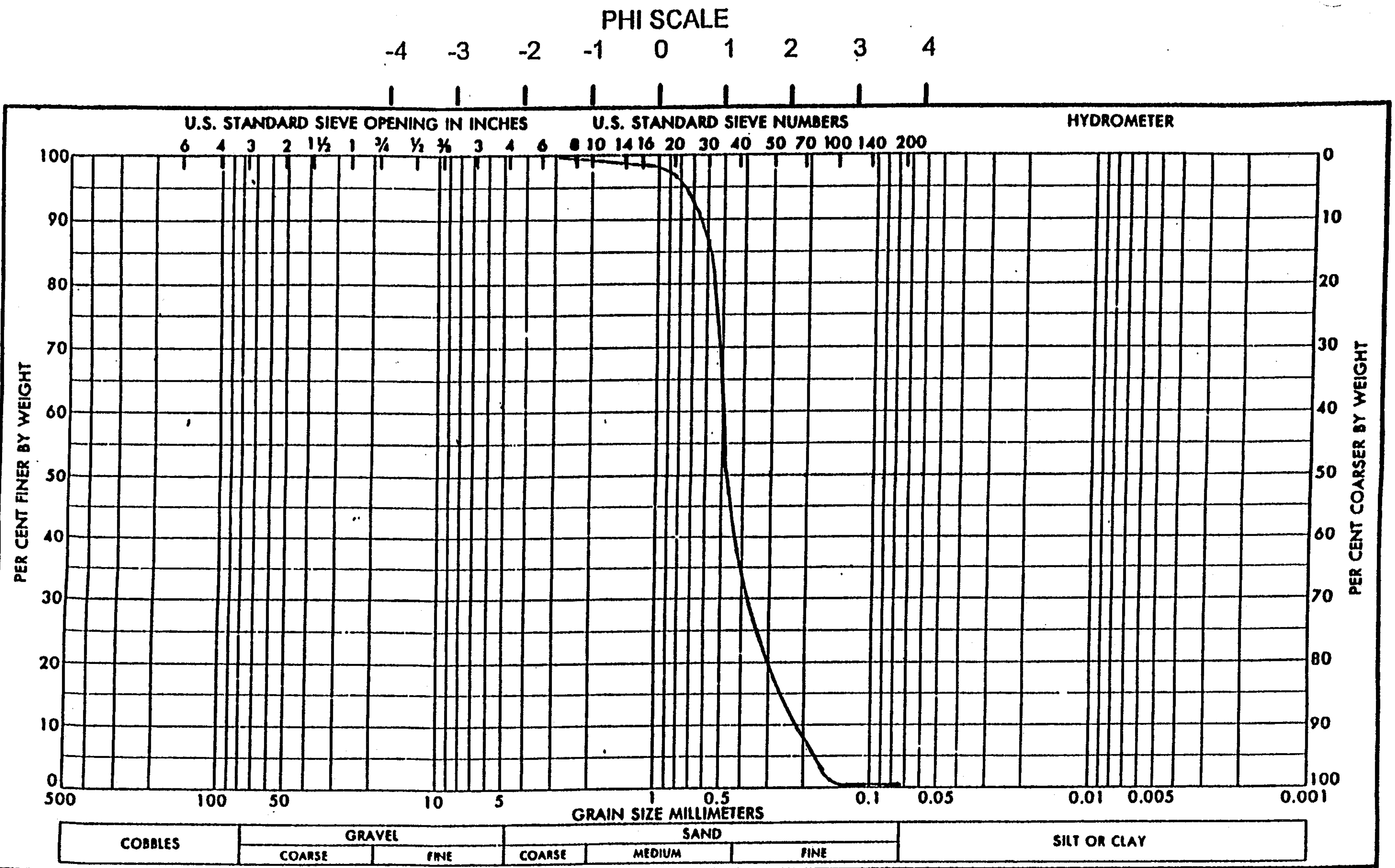
PROJECT O.S.S.

AREA Sandy Hook

BORING NO. 11B

DATE December 18, 1997

JOHNSON SOILS ENGINEERING LABORA .Y



SAMPLE NO.	ELEV OR DEPTH	CLASSIFICATION	NAT W%	LL	PL	PI
Composite		Gray medium/fine Sand (SP)				

GRADATION CURVES

PROJECT O.S.S.

AREA Sandy Hook

BORING NO. 11A

DATE December 18, 1997

JOHNSON SOILS ENGINEERING LABORATORY

FEBRUARY 2000

RARITAN BAY AND SANDY HOOK BAY, NEW JERSEY
FEASIBILITY REPORT
FOR
HURRICANE AND
STORM DAMAGE REDUCTION
PORT MONMOUTH, NJ

APPENDIX E

FOUNDATION

E

FEBRUARY 2000

RARITAN BAY AND SANDY HOOK BAY, NEW JERSEY
FEASIBILITY REPORT
FOR
HURRICANE AND
STORM DAMAGE REDUCTION
PORT MONMOUTH, NJ

GEOTECHNICAL

APPENDIX E

U.S. Army Corps of Engineers
New York District

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Geotechnical Draft Appendix
Port Monmouth Feasibility Study

TABLE OF CONTENTS

Introduction

E1. The proposed Port Monmouth Flood Control system includes the construction of approximately 7,070 feet of earthen levees and 3,650 feet of T and I Type Floodwalls. The proposed system is separated into two sections. The first section will begin at the existing Keansburg levee and run east adjacent to the newly constructed Port Monmouth Road, where it will cross the ramp to old Port Monmouth Road with a floodgate and head north to terminate at the proposed shore protection along the Sandy Hook Bay. The other section will start at the eastern end of the proposed shore protection and run south, parallel to Main Street. At the intersection of Main Street and Broadway a flood gate will be provided and the levees will continue southeast behind the existing residences on Creek Road where it will then turn west and terminate at Route 36.

E2. Seepage and slope stability analyses assumed a homogeneous earth fill consisting of a silty fine sand. This material was evaluated as proposed fill because soils exhibiting this type of gradation are readily available in this region. Seepage analyses also included the use of an impermeable core in the center of the levee to provide a seepage cutoff. A typical levee section is provided in Figure 2.

E3. In areas that lack sufficient room to construct an earthen levee, T and I Type Floodwalls have been analyzed. Sections along relocated Port Monmouth Road and adjacent to Route 36 will require the use of steel sheet piling driven to depths to develop acceptable factors of safety and provide both a seepage and surface water barrier. Analysis of these walls indicate that the I-Type wall is useable in areas where water heights above existing ground are 10 feet or less. T-Type walls are recommended for water heights that exceed 10 feet. Typical wall sections are shown in Figures 3 and 4.

Subsurface Investigation

E4. Subsurface soil information was obtained from two soil investigation programs. A subsurface investigation program performed during 1990 for the relocation of Port Monmouth Road, provided six (6) borings, BP-3, BP-4, B-8, B-9, D-11 & D-12 in the vicinity of proposed flood control project. In addition to this information in 1997, the Corps of Engineers took ten borings to provide a generalized soil profile along the proposed levee/wall route. Borings PM-P1, PM-P2, and PM-11 were taken along the relocated Port Monmouth Road and PM-1 thru PM-6, and PM-8 adjacent to Main Street. Boring location plans and geologic sections are included in Sub-Appendix A of this report.

E5. A laboratory testing program was also performed to provide design parameters for the various analyses. Testing included grain size analysis, Atterberg Limits, Consolidation and CU Triaxial Shear testing with pore pressure measurements. The intention of this program was to provide guidelines for the preliminary analyses. In addition to this testing, laboratory test data was also available from the Port Monmouth Road relocation project which is immediately adjacent to part of the proposed flood control system (See Laboratory Testing, Sub-Appendix D).

Geology

E6. The soils within the project limits belong to two separate geologic associations (see Soil Survey Map). The majority of the soils are mapped as MTM in the Rutgers University Engineering Soil Survey of New Jersey, Report No. 19, Monmouth County. These soils are found mainly along the area of relocated Port Monmouth Road. They consist of a marine tidal marsh, with a soil profile that is made up of a decomposed organic mat underlain at varying depths by organic sand, silt, clayey silt, and clay. The nature of this material will require a detailed subsurface investigation prior to the construction of the various flood control components.

E7. The remainder of the project area resides on soils mapped as AM-23pi . These soils are found south of the proposed shore protection to Route 36 along the proposed levee alignment. They consist of sand and silty sands which generally become coarser with depth and are underlain by stratified deposits of silt and silty fine sands.

Flood Control Analyses

E8. The following table summarizes the various analyses of the proposed flood control project:

Along New Port Monmouth Road	Design Type	Boring Used
Connection to Keansburg Levee	T Type Wall	Boring PM-P1
Sta. 0+00 to Sta. 4+50	Levee	Boring BP-3/BP-4
Sta. 8+50 to Sta. 11+00	I Type Wall	Boring B-9
Sta. 11+00 to Sta. 12+50	I Type Wall	"
Sta. 12+50 to Sta. 29+00	I Type	"
Sta. 30+00 to Shore Protection	Levee	Boring PM-11
Main Street Toward Route 36		
Shore Protection to Sta. 12+00	Levee	Boring PM-8
Sta. 12+00 to Sta. 22+00	Levee	Boring PM-6
Sta. 22+00 to Sta. 42+50	Levee	Boring PM-3/PM-4
Sta. 42+50 to Sta. 48+50	Levee	Boring PM-2
Sta. 48+50 to Sta. 71+00	Levee	Boring PM-1
Sta. 71+00 to Route 36	I Type Wall	Boring PM-1

Refer to the boring location plans in Appendix A for stationing.

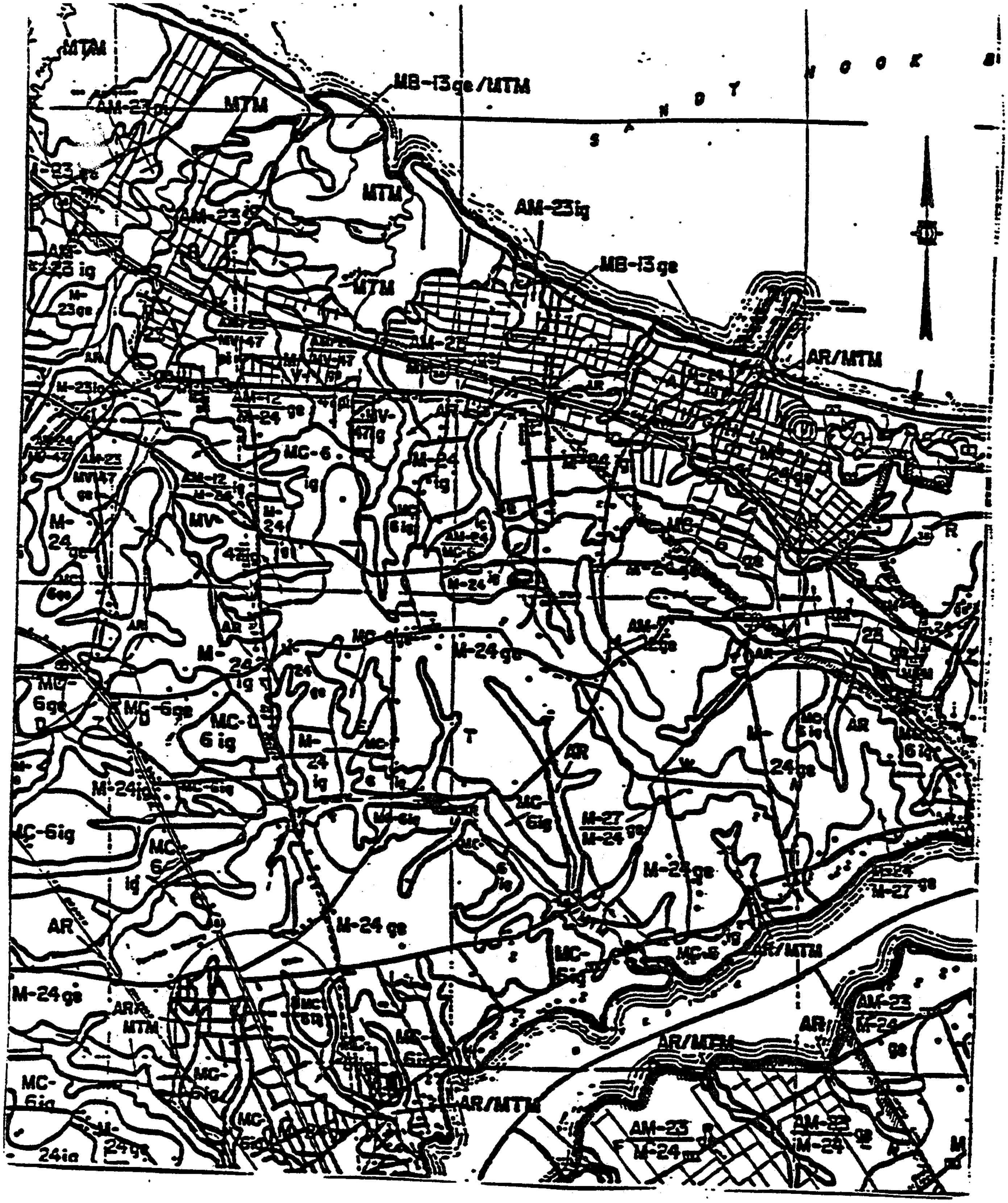


FIGURE 1
SOIL SURVEY MAP
PORT MONMOUTH FLOOD CONTROL
FEASIBILITY STUDY

Levee Analyses

E9. The proposed levees were assumed to be constructed with a silty fine sand utilizing an impermeable core as a seepage cutoff and a horizontal drain for control of exit gradients. The proposed levee areas were subdivided into seven (7) sections (listed previously) based on the soil boring(s) taken within the limits of each section. The levee sections were analyzed for seepage and slope stability during flood elevations of 13.0, 14.0 and 15.2. In addition, settlement analyses were performed to evaluate the short term effects of increased overburden on underlying compressible deposits. The seepage analyses were performed utilizing a finite element analysis program (BOSS SEEP2D) assuming a steady state flow condition. Slope stability analyses were performed on selected critical sections for both steady state seepage and rapid drawdown utilizing PCSTABLE5, a program which allows the phreatic surface for various conditions to be included in the analysis.

E10. Additional assumptions included in these analyses are the removal of five (5) feet of surficial organic soil believed to exist in the area of Pews Creek and general removal of one (1) foot of in-situ material to eliminate vegetation. The removal of the material in the Pews Creek area will minimize settlements and help to increase slope stability factors of safety to acceptable values. A summary of analysis results can be found in Table 1 and the individual analyses are included in Sub-Appendix B of this report.

Seepage

E11. Earthen levees are subject to seepage and uplift pressures as a result of hydrostatic head differentials developed during the design flood condition. Excessive seepage and uplift pressures can lead to slope failure, piping, and heave at the toe of the land side slope. The seepage analysis (flownet) provides a picture of how pressures are dissipated and the path of the flow. It is also used to calculate escape gradients at the exit face as well as define the phreatic surface used in the stability analysis.

E12. The seepage analyses considered a steady state flow condition with flood waters on the unprotected side at the top of the levee, and water at existing ground on the protected side. The levee fill was assigned a permeability of 1000 ft/yr with underlying soils permeability varying based on soil type. The analyses also included an impermeable core located at the center of the embankments and a horizontal drain consisting of uniform medium sand wrapped in a geotextile filter fabric. Exit gradients were calculated from the computer generated flow nets with values varying from .09 to .33. These values are lower than the range of critical gradients of 0.8 to 1.0, and within the range of acceptable values. The units of permeability utilized in these analyses were feet per year resulting in a calculated flow rate, as specified above each flow net, in cubic feet per year (per foot of levee).

Slope Stability

E13. Initial slope stability analyses were performed on the protected side of the levee sections assuming a steady-state seepage flow and side slopes of 2H:1V. Results of these analyses are summarized as follows:

	Flood Elevation		
Boring No.	13.0	14.0	15.2
PM1	1.435	1.42	1.50
PM2	1.33	1.32	1.44
PM3	1.13	1.26	1.29
PM6	1.45	1.46	1.52
PM8	1.47	1.38	1.74
BP3	1.45	1.45	1.45
PM11	1.45	1.45	1.45

E14. Minimum stability factor of safety requirements for earth levees in this application are 1.5 for steady state seepage and 1.0 for the rapid drawdown condition ("Engineering and Design Stability of Earth and Rockfill Dams" - EM 1110-2-1902, 1970). Stability analyses for the rapid drawdown condition with 2H:1V side slopes generated safety factors of approximately 0.7

E15. Based on the results of these initial analyses, the use of 2.5H:1V side slopes was evaluated to improve stability factors of safety. It is believed that this alternative would increase factors of safety for stability to acceptable levels with the least economic impact.

E16. The 2.5H:1V levees were evaluated for two (2) separate cases, Case 1 and Case 2. A summary of assumptions and conditions for these two cases are as follows:

CASE 1. Seepage as a result of design flood conditions was used to develop the critical stability factor of safety for the protected side slope. This condition was modeled with flood waters at three (3) specified design elevations and water at existing ground on the protected side. The phreatic surface used in the analysis was established from the corresponding seepage analysis.

CASE 2. Rapid drawdown of flood water from three (3) specified elevations was used to evaluate slope stability on the unprotected side. A rapid drawdown of flood water may cause an excess pore pressure in the saturated soils, thereby reducing the soils' strength and resulting in a reduced factor of safety.

E17. Surface sloughing may occur during sudden drawdown on the unprotected side slope. This condition is not critical to the levee function and diminishes with growth of slope vegetation, but may require repair. These failures may be prevented by reinforcing the unprotected side slope.

E18. The soil properties assumed for the levee fill and used in the analyses are as follows:

Moist unit weight, γ_t = 115 pcf
Saturated unit weight, γ_{sat} = 125 pcf
Angle of internal friction, $\phi = 35°$
Coefficient of Permeability = 1000ft/yr (.001 cm/sec)

Impermeable Core Material:

Moist unit weight = 115pcf
Saturated unit weight = 120pcf
Cohesion = 1000psf
Coefficient of Permeability = 50ft/yr (.00005 cm/sec)

Horizontal Drain Material:

Coefficient of Permeability = 100,000ft/yr (0.1 cm/sec)

Settlement

E19. Settlement analyses of the proposed levees were limited to sections founded on fine grained plastic soils. Settlements of levees placed on sands are expected to be rapid and occur progressively as the levees are constructed. Plastic soils are found in borings PM-P1, PM-P2, BP-3 and PM-1. Settlements in these areas were based on the increased effective stresses resulting from embankment placement to an elevation of 15.2. Consolidation data provided by the Army Corps of Engineers as well as testing performed for the Port Monmouth Road relocation were utilized to evaluate settlement magnitude and duration. Based on a preliminary analysis, settlements are anticipated to range from 4 to 9" over a period of 1 to 2 years in the vicinity of Pews Creek and PM-1, respectively.

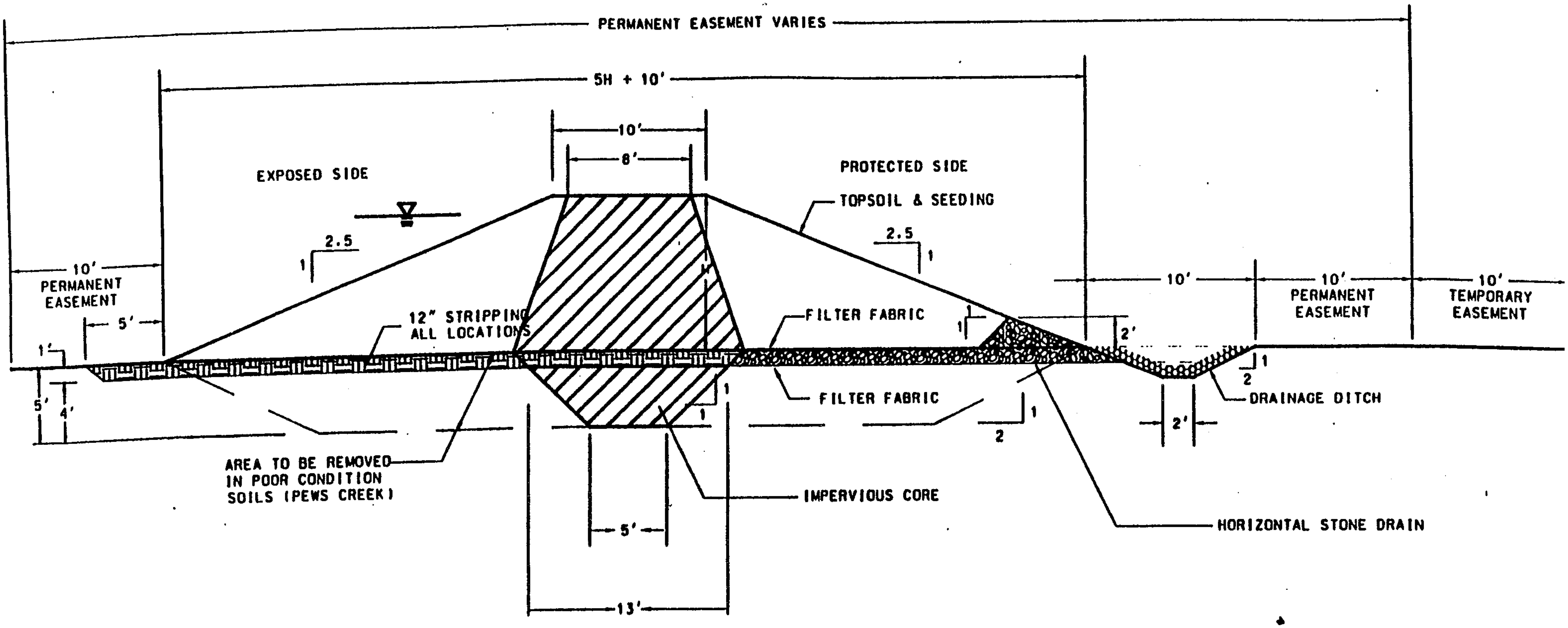
I Type Flood Walls

E20. The I Type flood walls consist of steel sheeting driven to a specified depth and capped with concrete which transitions to a concrete wall above the existing ground line. These walls are to be considered only in areas where the anticipated flood elevation is less than 10 feet above existing ground. The walls were analyzed to determine the required embedment length of sheeting needed to resist hydrostatic pressure developed during maximum flood conditions. Existing subsurface information in the vicinity of relocated Port Monmouth Road indicate that the proposed sheet piles will be driven through impermeable stratum providing a cut off from seepage. Currently, there is approximately 1600 linear feet of proposed I-wall where no boring data is available. Additional subsurface investigation along these walls is required to verify assumptions used in the analysis. The I-Type walls adjacent to Route 36 are founded in permeable sands. Seepage analyses, included in Appendix B, reveal exit gradients lower than critical values. A typical I-wall section can be found in Figure 3 and a summary of required sheeting lengths in Table 2. Cantilever sheet calculations are provided in Sub-Appendix C.

T Type Flood Walls

E21. The T Type flood walls will be required at the connection to the existing Keansburg Levee for flood elevations of 14 and 15.2. These walls are required where anticipated maximum flood heights will exceed 10' above existing ground elevations.

E22. The T-type flood walls adjacent to the Keansburg Levee will be supported on piles as the soils adjacent to Pews Creek consist of soft, organic, compressible silts and clays. Existing subsurface information indicates a 22' long 12" timber pile with an 8" tip will develop an allowable bearing capacity in excess of 19 tsf. The proposed footing will require three rows of timber piles, the first two rows will be spaced at four (4) feet center to center and driven on a 1:4 batter. The third or back row can be driven vertical at twelve (12) foot center to center. Preliminary flownet analysis using existing subsurface information indicate seepage below these walls will be negligible as the soils in this area are expected to have a relatively low permeability. Wall sections showing the proposed scheme are included in Figure 4 and design calculations are included in Sub-Appendix C.



TYPICAL LEVEE SECTION

Revision	Description	Date	Approv

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
NEW YORK, NEW YORK

Designed by:

Drawn by:

Checked by:

PORT MONMOUTH COMBINED FLOOD CONTROL
AND SHORE PROTECTION FEASIBILITY STUDY

TYPICAL LEVEE SECTION
PEWS CREEK
COMPTONS CREEK

Reviewed by:
SECTION CHIEF
Concurred by:
CHIEF, DESIGN BRANCH

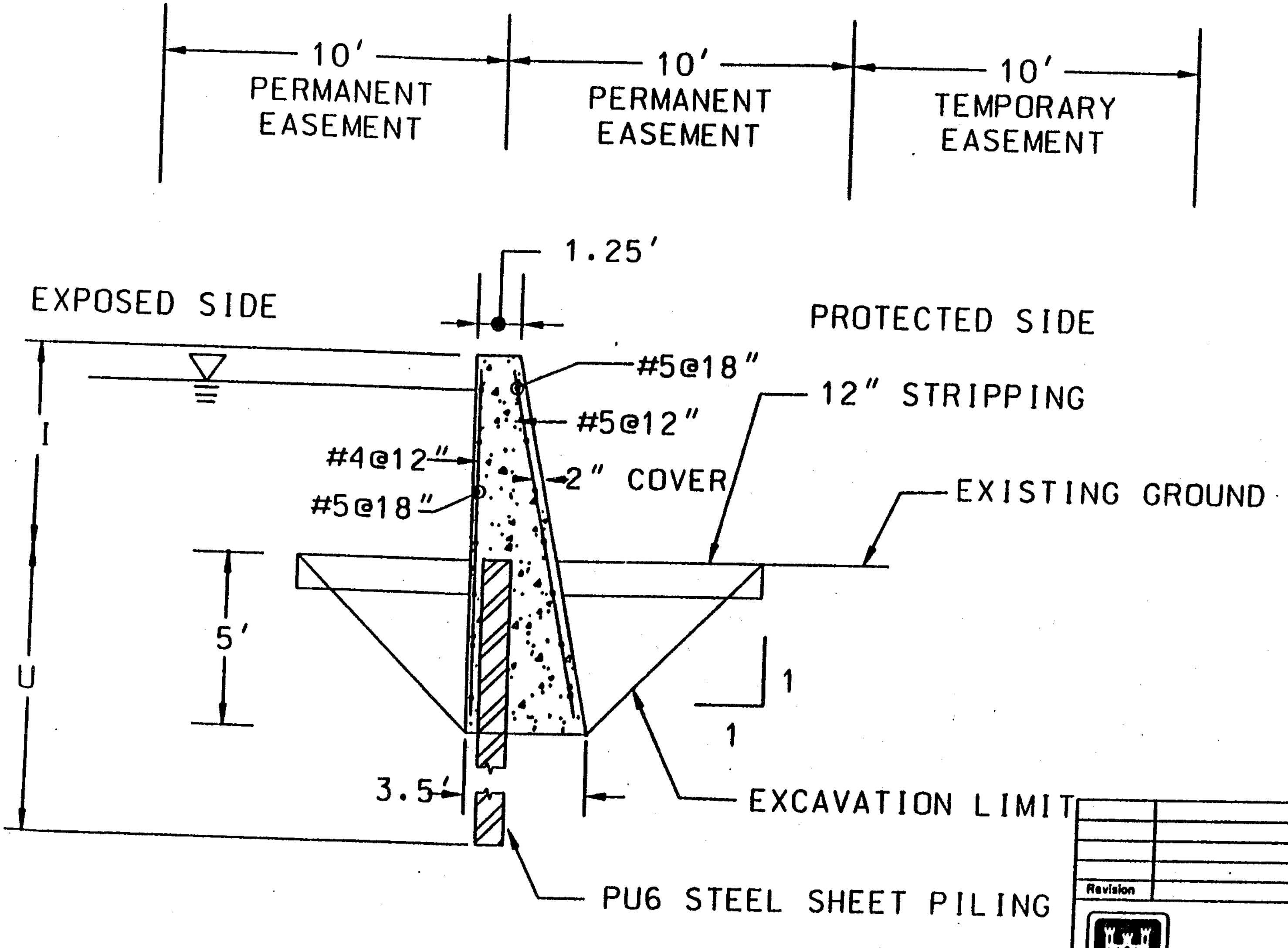
Approval Recommended:
CHIEF, ENGINEERING DIVISION
Approval By:
COLONEL, C.E. DISTRICT ENGINEER

SHEET REFERENCE NUM

DATE:

SCALE: 1" = 10'

FIG.: 2



TYPICAL FLOODWALL SECTION

I-S TYPE FLOODWA

I - HEIGHT OF FLOODW
ABOVE GRADE

U - LENGTH OF SHEET PI

Revision	Description	Date	Approv

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
NEW YORK, NEW YORK

Designed by:

Drawn by:

Checked by:

PORT MONMOUTH COMBINED FLOOD CONTROL
AND SHORE PROTECTION FEASIBILITY STUDY

TYPICAL FLOODWALL SECTION

Reviewed by:
SECTION CHIEF

Concurred by:
CHIEF, DESIGN BRANCH

Approval Recommended:
CHIEF, ENGINEERING DIVISION

Approval By:
COLONEL, C.E. DISTRICT ENGINEER

SHEET REFERENCE NUMBER:

DATE:

SCALE: 1" = 5'

FIG.: 3

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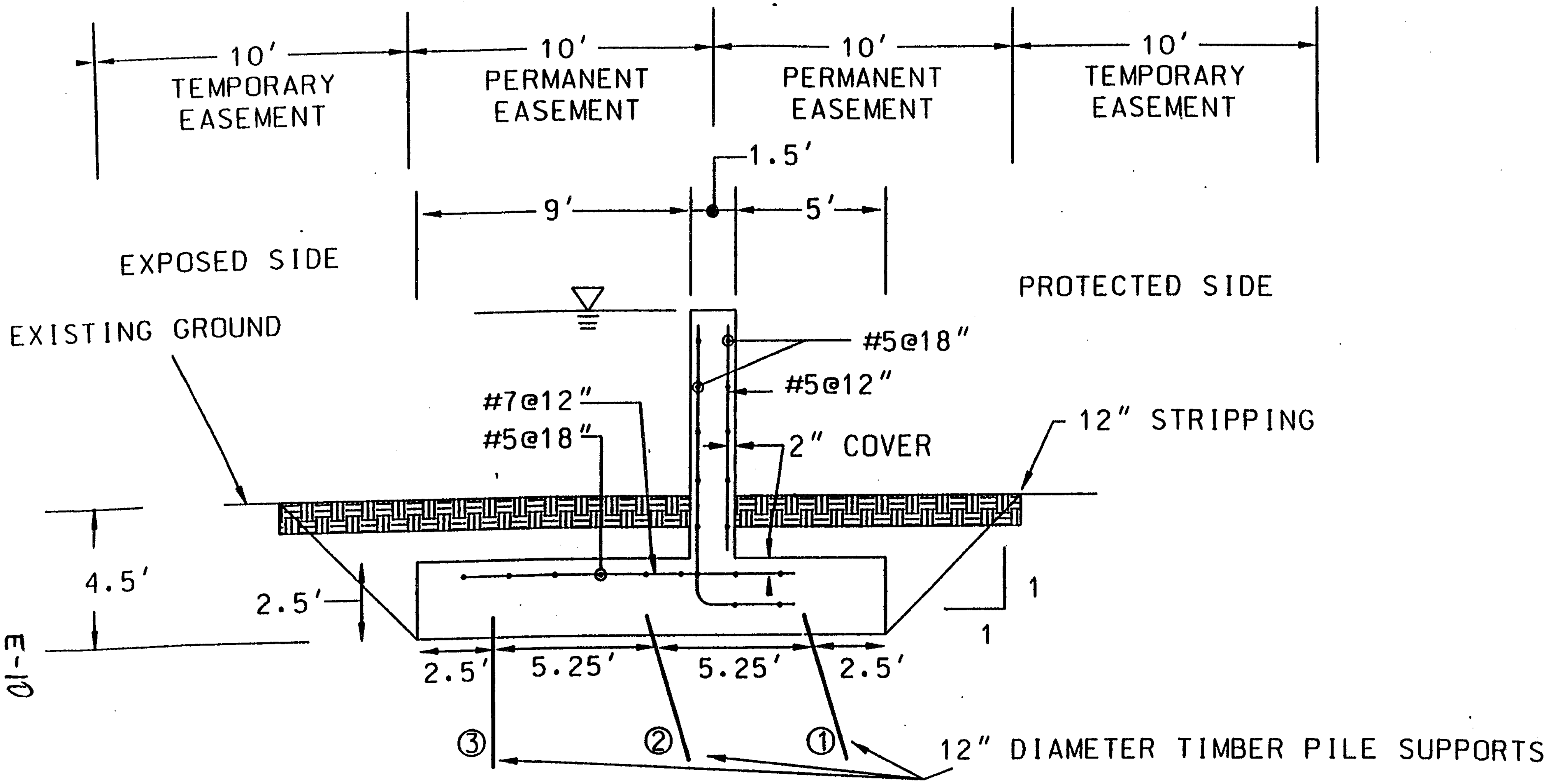


T-TYPE FLOOD WALL A

WOOD PILES 23' LONG

ROW	SPACING
①	4'
②	4'
③	12'

Revision	Description	Date	Approv

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
NEW YORK, NEW YORK

Designed by:

Drawn by:

Checked by:

Reviewed by:
SECTION CHIEF

Concurred by:
CHIEF, DESIGN BRANCH

PORT MONMOUTH COMBINED FLOOD CONTROL
AND SHORE PROTECTION FEASIBILITY STUDY

TYPICAL FLOODWALL SECTION
T-TYPE FLOODWALL A

Approval Recommended:
CHIEF, ENGINEERING DIVISION

Approval By:
COLONEL, C.E. DISTRICT ENGINEER

SHEET REFERENCE NUMB

DATE:

SCALE: 1" = 5'

FIG.: 4

Levee Section	Flood Elevation	Slope Stability Factor of Safety Seepage FSmin = 1.5	 Drawdown FSmin = 1.0	Seepage Analysis i(exit)	Anticipated Settlement
Along New Port Monmouth Road					
Sta. 0+00 to Sta. 4+50	Elevation 13.0	1.678	0.996	0.13	4"
	Elevation 14.0	1.648	0.955	0.15	
	Elevation 15.2	1.596	0.949	0.15	
Sta. 30+00 to Sta. 32+50	Elevation 13.0	1.538	1.025	0.20	1"
	Elevation 14.0	1.654	.990	0.28	
	Elevation 15.2	1.390	.974	0.28	
Main Street Toward Route 36					
Sta. 1+00 to Sta. 12+00	Elevation 13.0	1.767	0.998	0.20	
	Elevation 14.0	1.870	0.957	0.13	
	Elevation 15.2	1.746	0.958	0.13	
Sta. 12+00 to Sta. 22+00	Elevation 13.0	1.804	0.998	0.28	
	Elevation 14.0	1.817	0.957	0.17	
	Elevation 15.2	1.795	0.958	0.22	
Sta. 22+00 to Sta. 42+50	Elevation 13.0	1.459	0.980	0.33	
	Elevation 14.0	1.504	0.950	0.28	
	Elevation 15.2	1.584	0.950	0.28	
Sta. 42+50 to Sta. 48+50	Elevation 13.0	1.590	0.998	0.31	
	Elevation 14.0	1.579	0.957	0.22	
	Elevation 15.2	1.668	0.958	0.31	
Sta. 48+50 to Sta. 71+00	Elevation 13.0	1.714	0.998	0.14	9"
	Elevation 14.0	1.686	0.957	0.09	
	Elevation 15.2	1.876	1.06	0.10	

Port Monmouth Flood Control Feasibility Study

Summary of Levee Analyses TABLE 1

LOCATION	Estimated Length of Sheeting
Along New Port Monmouth Road	
At Keansburg Connection	
Elevation 13.0	23'
Elevation 14.0	T-Type Wall A
Elevation 15.2	T-Type Wall A
Sta. 8+50 to Sta. 11+00	
Elevation 13.0	20'
Elevation 14.0	24'
Elevation 15.2	28'
Sta. 11+00 to Sta. 12+50	
Elevation 13.0	20'
Elevation 14.0	24'
Elevation 15.2	28'
Sta. 12+50 to Sta. 29+00	
Elevation 13.0	17'
Elevation 14.0	21'
Elevation 15.2	25'
Main Street Toward Route 36	
Sta. 71+00 to Sta. 83+50	
Elevation 13.0	16'
Elevation 14.0	19'
Elevation 15.2	22'

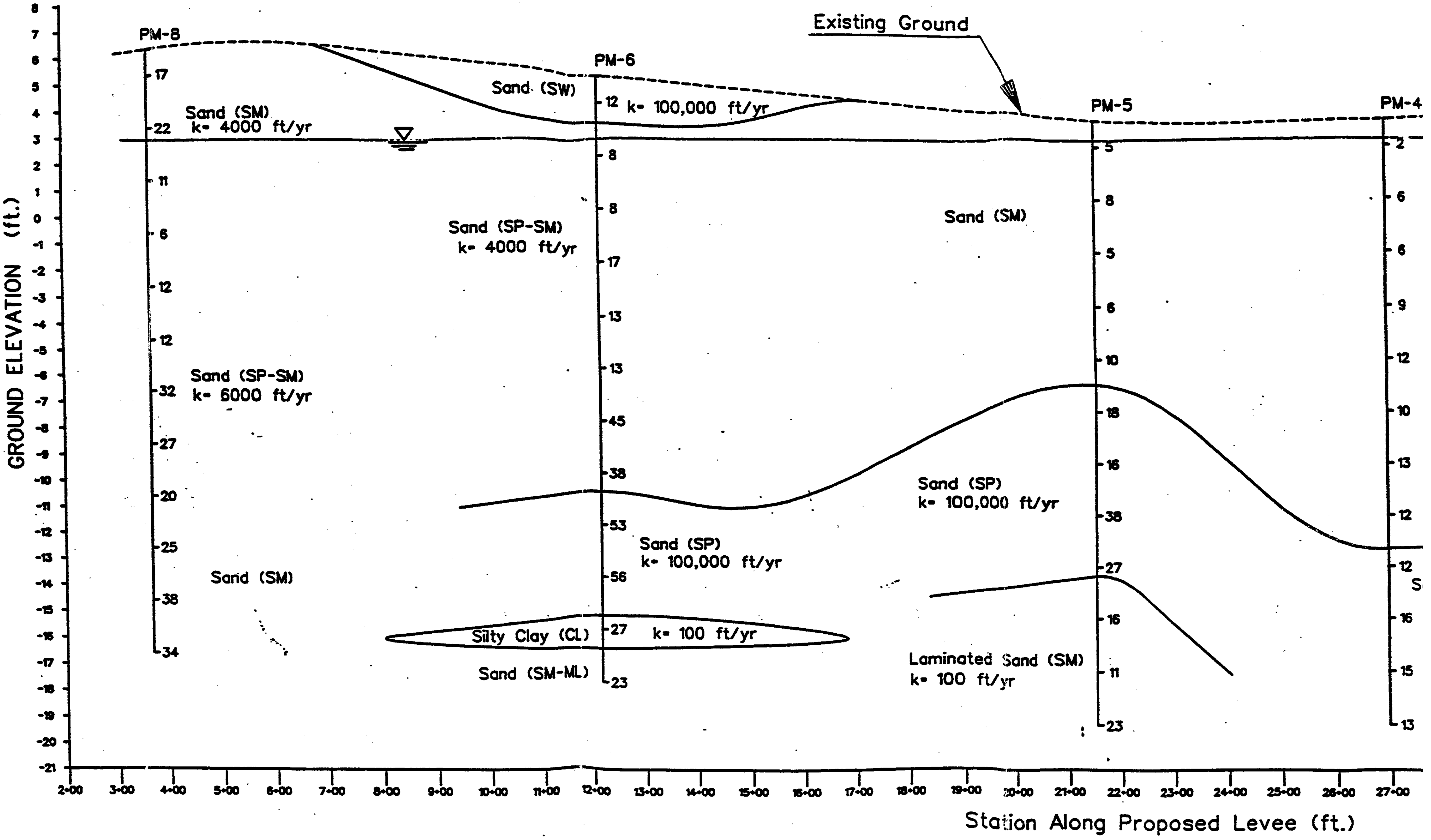
TABLE 2

Port Monmouth Flood Control
Feasibility Study

Sheeting Depths TABLE 2

APPENDIX A
BORING LOCATION PLANS
GEOLOGIC SECTIONS

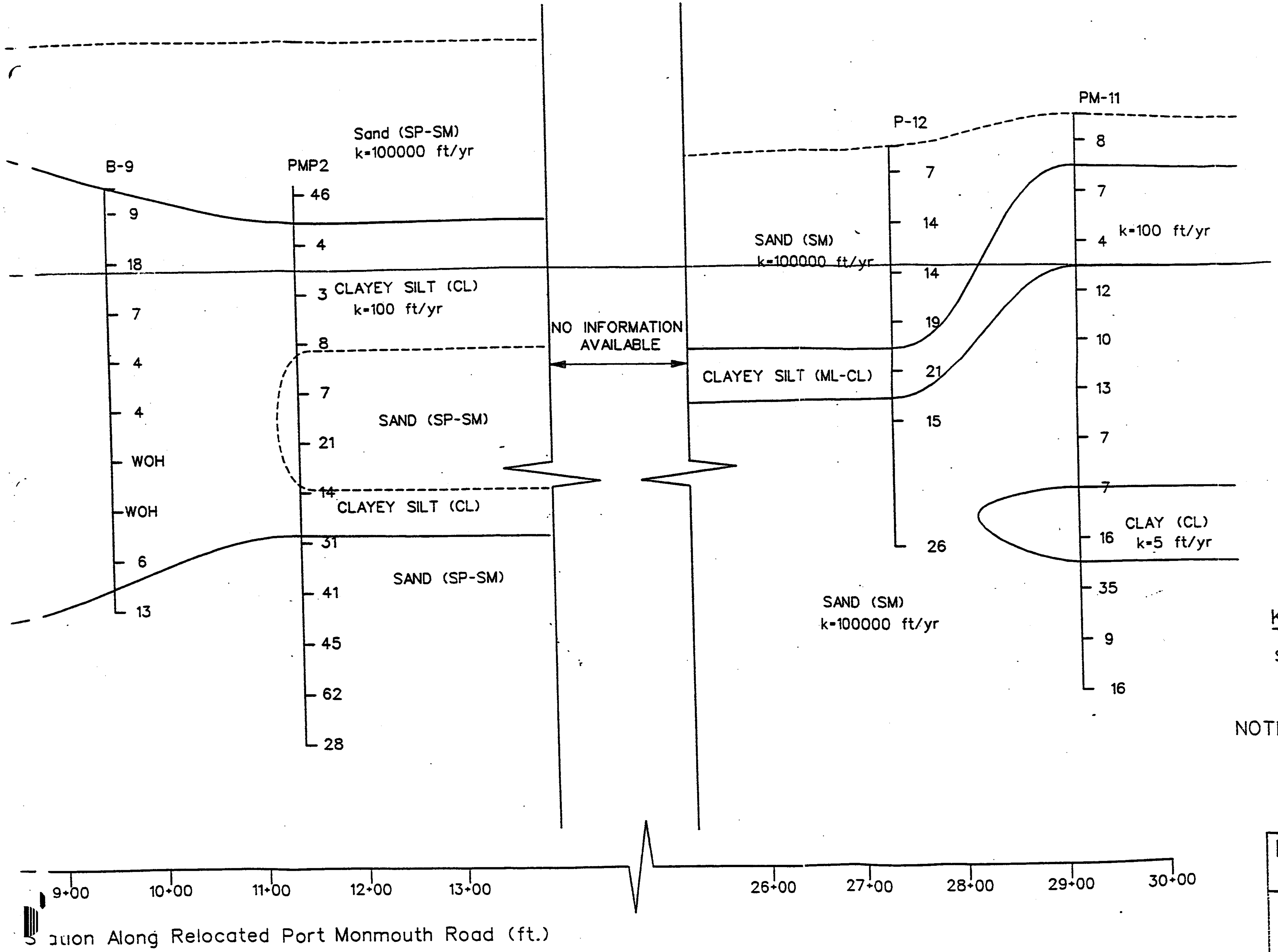
EA



KEY

NOTE:
Permeability values approximated from soil classification.

EA-1



Scale: 1"=4 '' V
1"=100' H

KEY:

Sand (SP) - Soil Type (Unified Classification)
14 - SPT N value

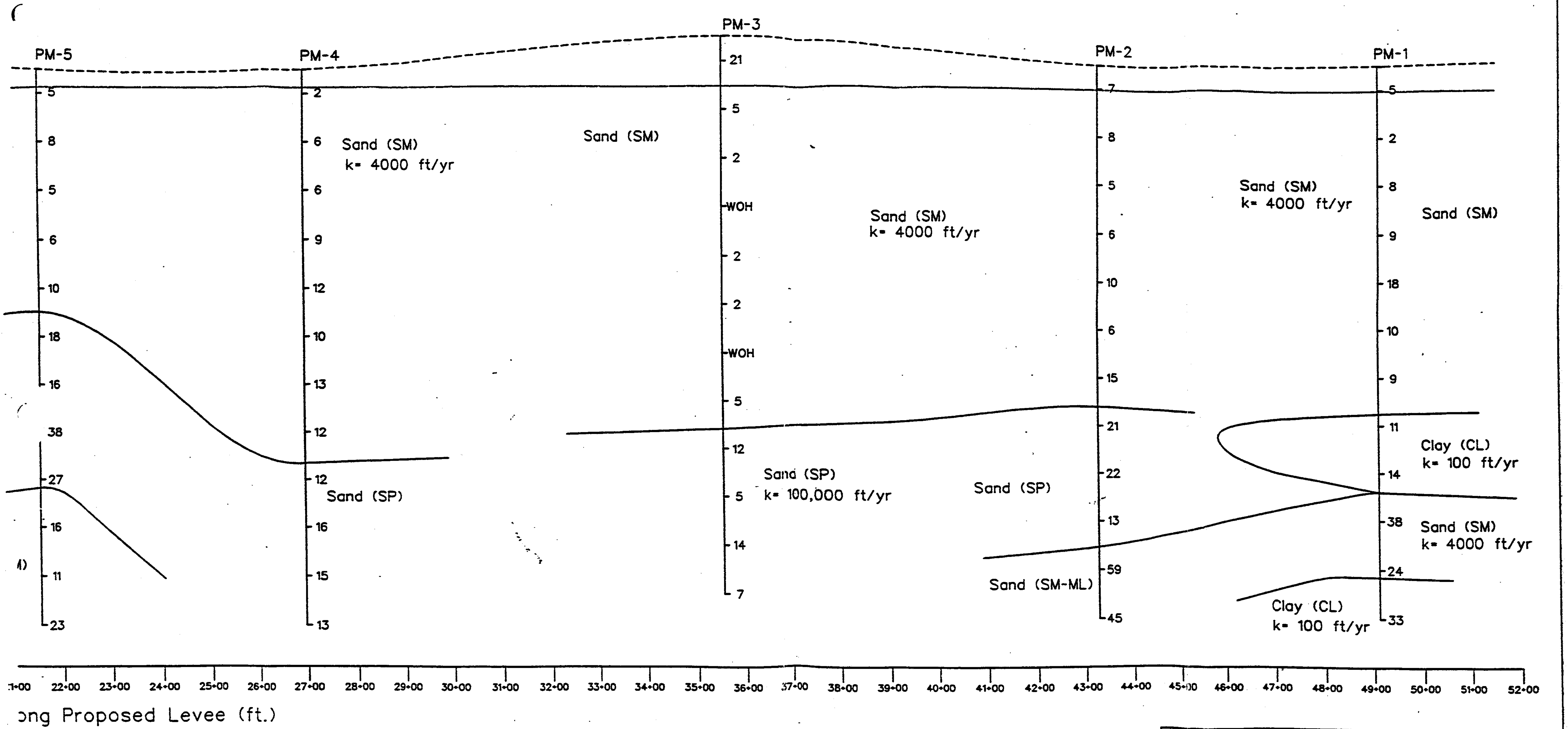
NOTE:
Permeability values approximate
from soil classification.

Port Monmouth Flood Contr
Feasibility Study

Geologic Section
Relocated Port Monmouth Road
Sta. 1+00 to Sta. 30+00

EA-2


PROPOSED SHORE PROTECTION
PROPOSED LEVEE
PM-8
PM-6
PM-5
PM-4
PM-3
FLOOD ZONE B
FLOOD ZONE A11
FLOOD ZONE A11
FLOOD ZONE A11
FLOOD ZONE A11
FORMER RAILROAD RIGHT OF WAY
COMPTONS CREEK
MAIN
GRIGGS AVE.
BRAINARD AVE.
6+00
8+00
10+00
12+00
14+00
16+00
18+00
20+00
22+00
24+00
26+00
26+00
30+00
32+00
34+00
38+00



KEY:

Sand (SP) - Soil Type (Unified Classification)

14 - SPT N value

Scale:
1"=4 ' V
1"=200' H

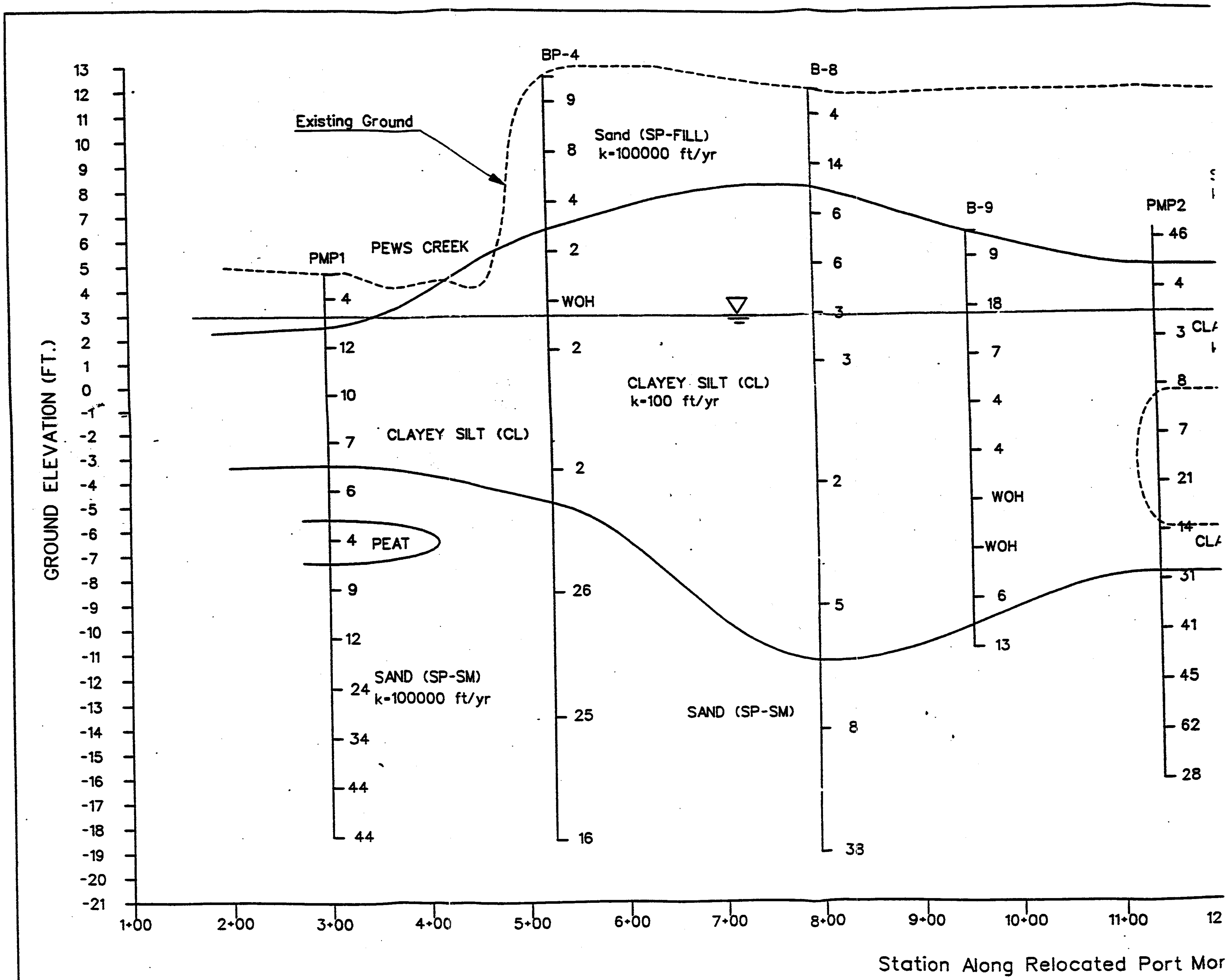
Port Monmouth Flood Control Feasibility Study

Geologic Section

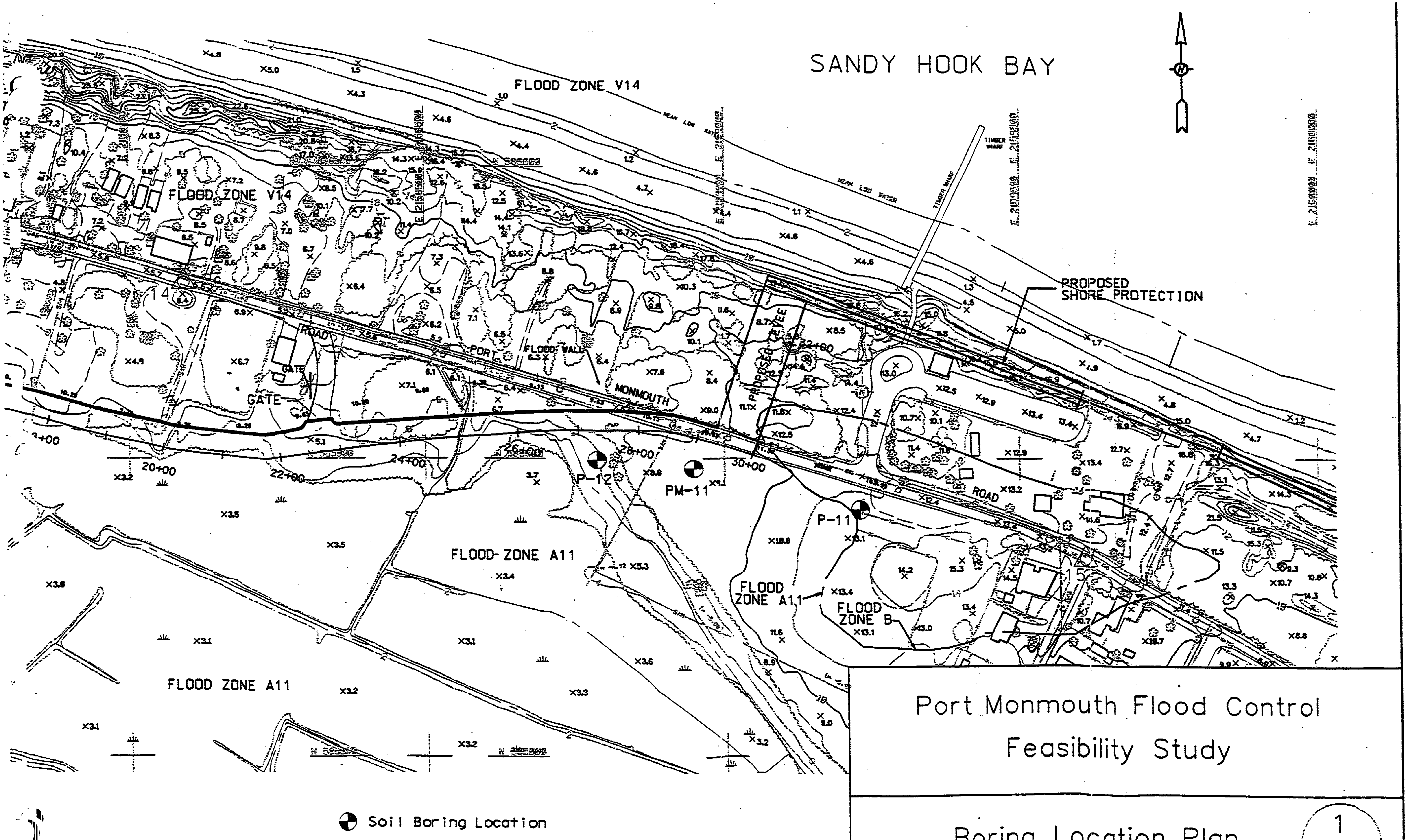
Along Main Street Toward Route 36
Sta. 2+00 to Sta. 52+00

2/2

EA-4


GROUND ELEVATION (FT.)
13
12
11
10
9
8
7
6
5
4
3
2
1
0
-1
-2
-3
-4
-5
-6
-7
-8
-9
-10
-11
-12
-13
-14
-15
-16
-17
-18
-19
-20
-21
1+00
2+00
3+00
4+00
5+00
6+00
7+00
8+00
9+00
10+00
11+00
12
Station Along Relocated Port Mor
Existing Ground
PEWS CREEK
PMP1
BP-4
B-8
B-9
PMP2
Sand (SP-FILL)
k=100000 ft/yr
CLAYEY SILT (CL)
k=100 ft/yr
CLAYEY SILT (CL)
PEAT
SAND (SP-SM)
k=100000 ft/yr
SAND (SP-SM)
CLA
CLA
4
12
10
7
6
4
9
12
24
34
44
44
9
8
4
2
WOH
2
2
26
25
16
4
14
6
6
3
3
2
5
8
38
9
18
7
4
4
WOH
WOH
6
13
46
4
3
8
7
21
14
31
41
45
62
28

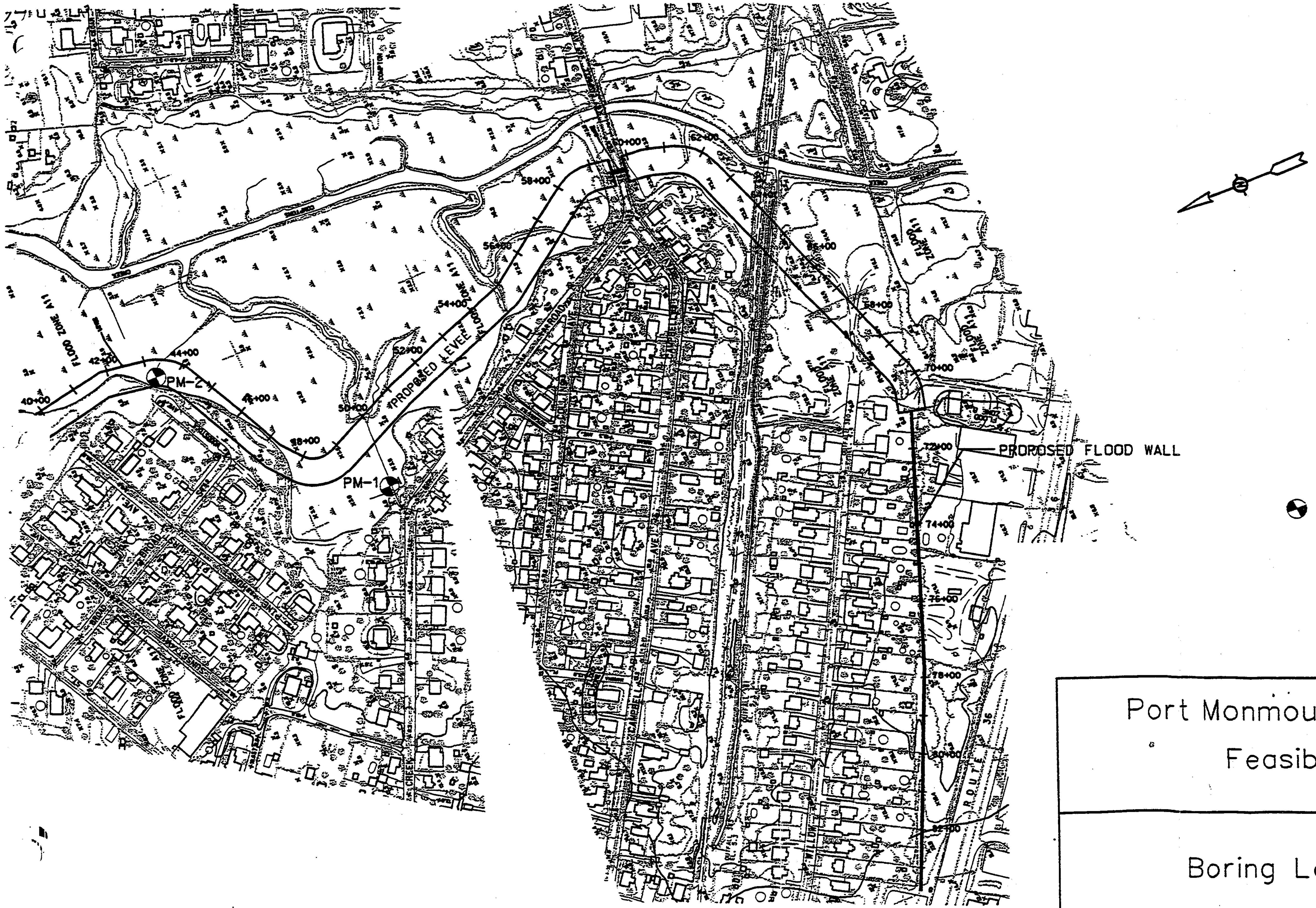
EA-5



Port Monmouth Flood Control Feasibility Study

Boring Location Plan

1/2

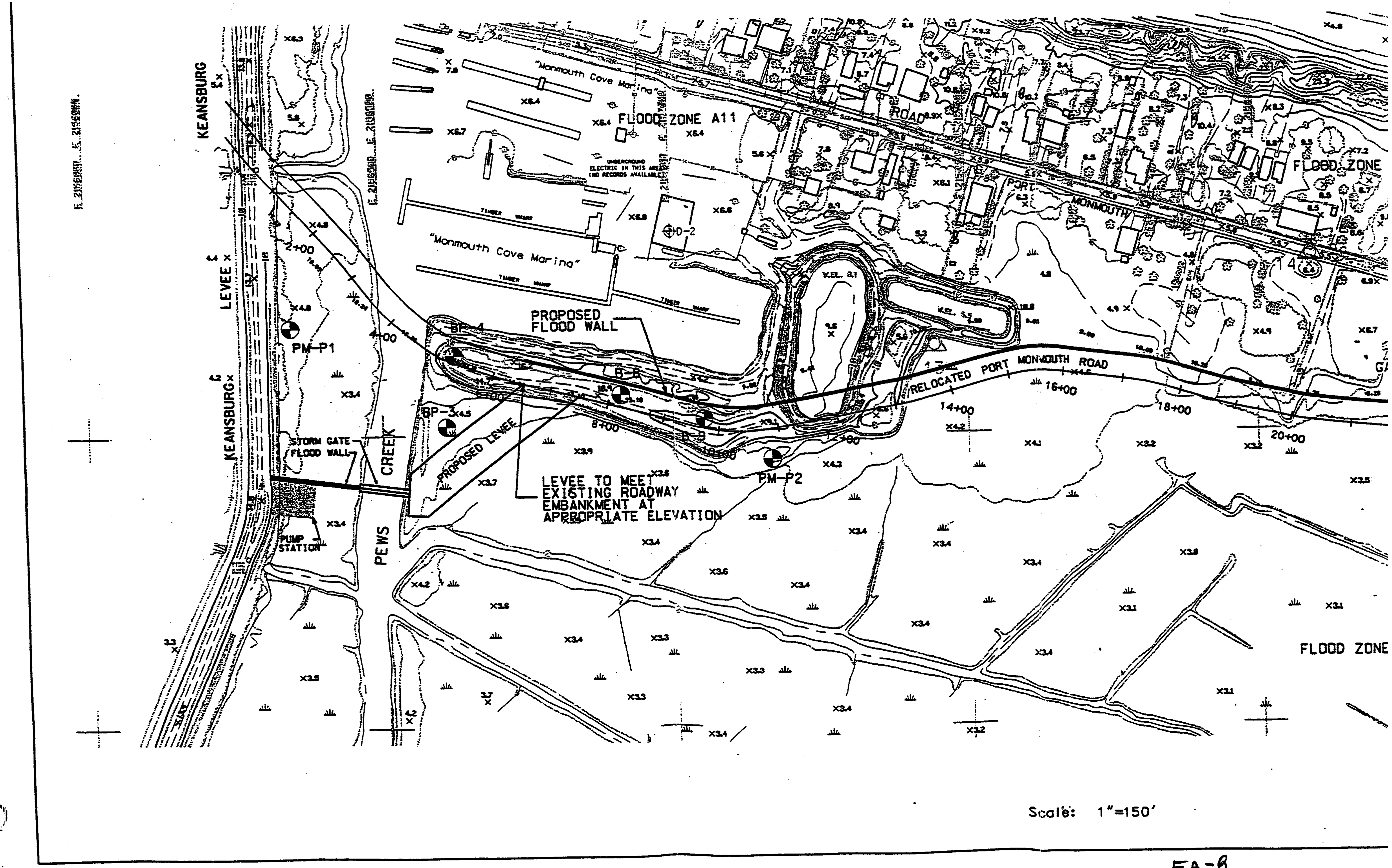


Soil Boring Location

Scale: 1"=250'

Port Monmouth Flood Control Feasibility Study

Boring Location Plan


KEANSBURG
LEVEE
KEANSBURG
"Monmouth Cove Marina"
"Monmouth Cove Marina"
TIMBER WHARF
FLOOD ZONE A11
UNDERGROUND ELECTRIC IN THIS AREA (NO RECORDS AVAILABLE)
D-2
ROAD
PORT MONMOUTH
FLOOD ZONE
PM-P1
BP-4
BP-3
B-8
B-9
PM-P2
W.EL. 3.1
PROPOSED FLOOD WALL
PROPOSED LEVEE
LEVEE TO MEET EXISTING ROADWAY EMBANKMENT AT APPROPRIATE ELEVATION
RELOCATED PORT MONMOUTH ROAD
STORM GATE
FLOOD WALL
PUMP STATION
PEWS CREEK
FLOOD ZONE
2+00
4+00
6+00
8+00
10+00
12+00
14+00
16+00
18+00
20+00
Scale: 1"=150'

APPENDIX B

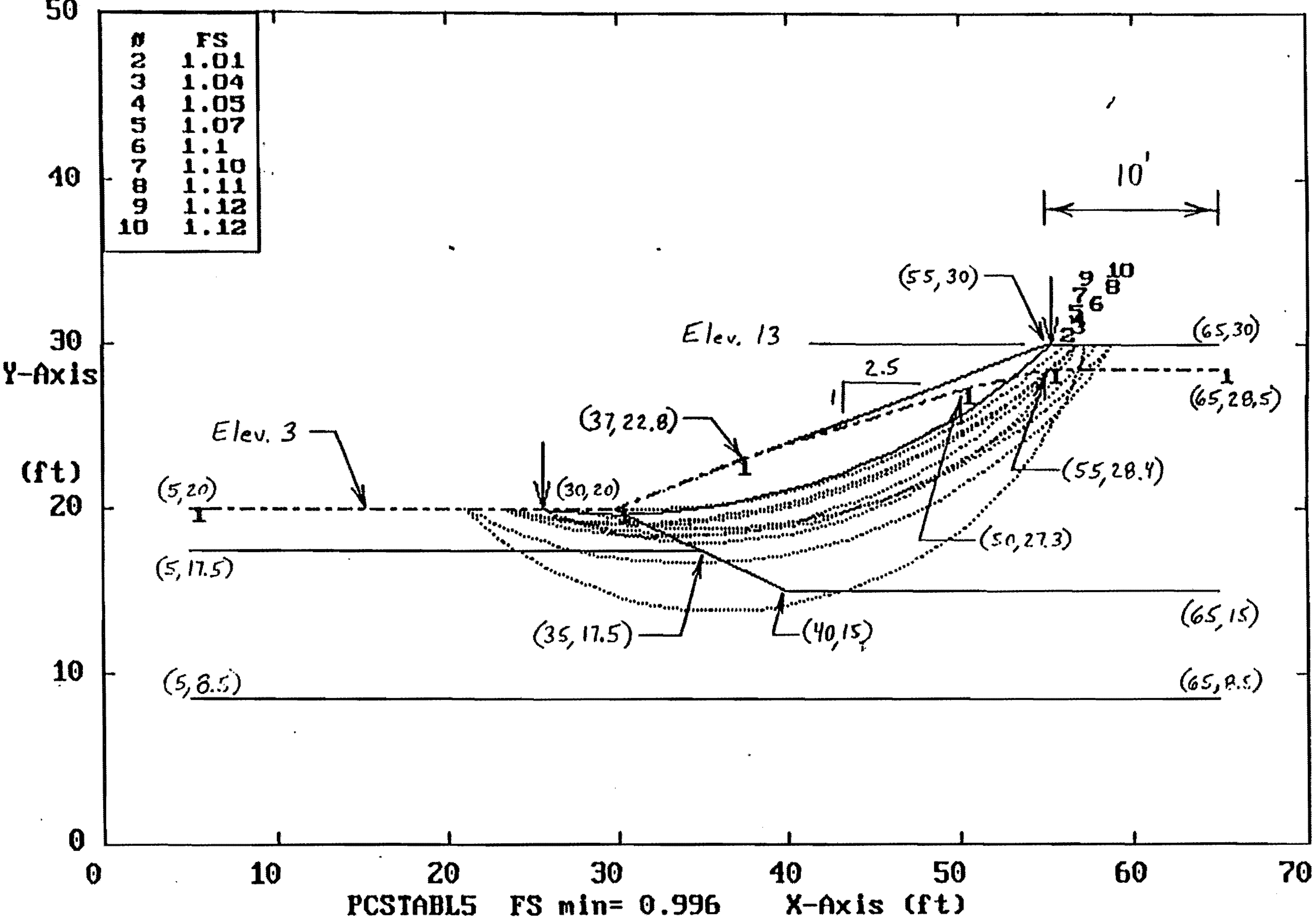
LEVEE ANALYSES

SEEPAGE
SLOPE STABILITY
SETTLEMENT

EB

PORT MONMOUTH FLOOD CONTROL FEASIBILITY STUDY

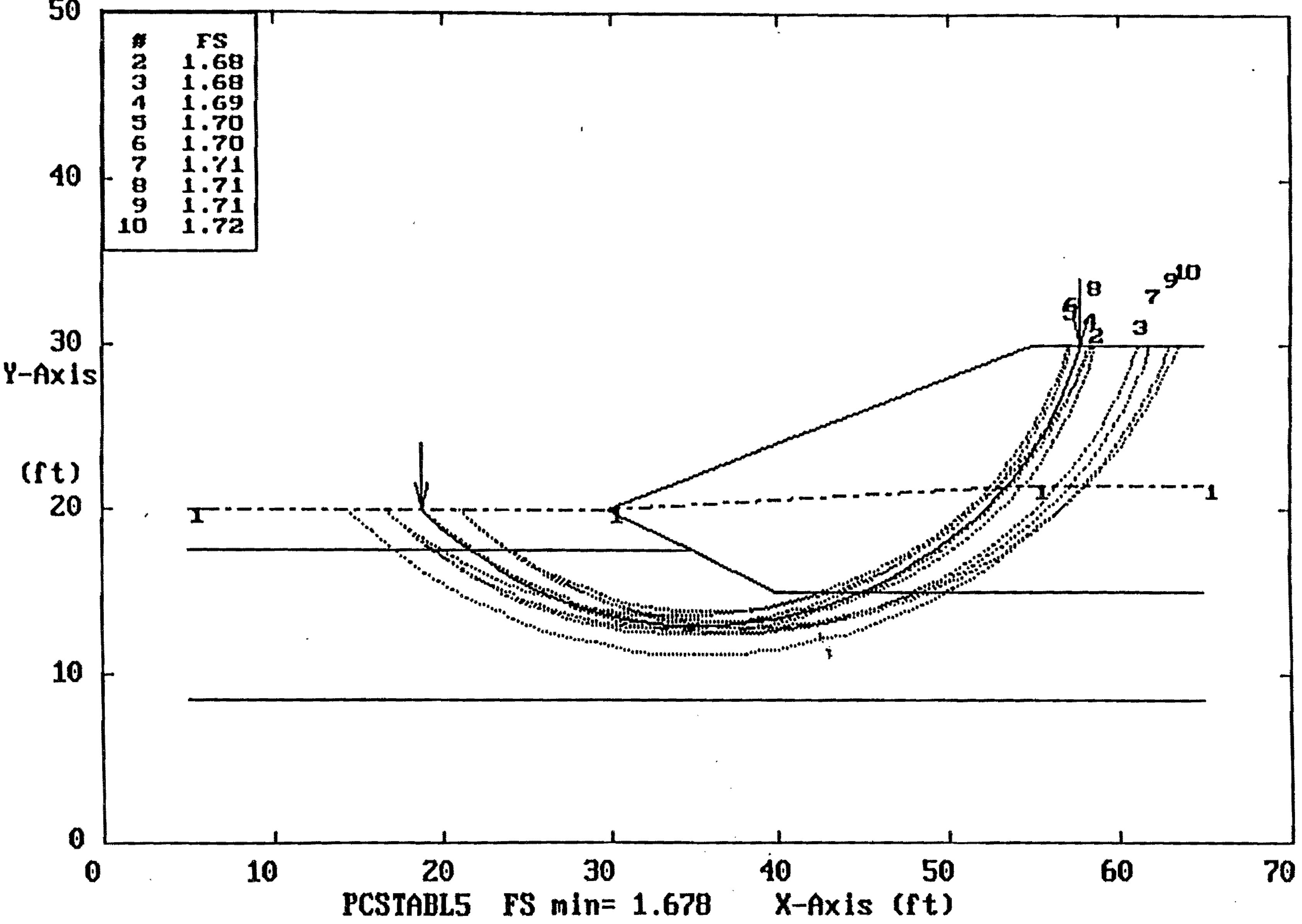
Ten Most Critical. A:BP3D_13.PLT By: EJS 01-12-98 3:04pm



EB-1

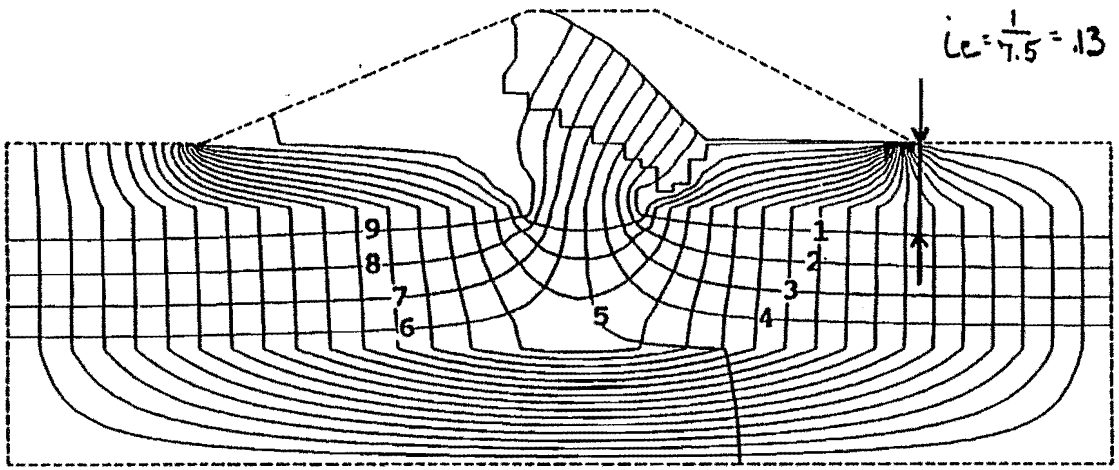
PORT MONMOUTH FLOOD CONTROL FEASIBILITY STUDY

Ten Most Critical. A:BP3_13.PLT By: EJS 01-15-98 2:18pm



EB-2

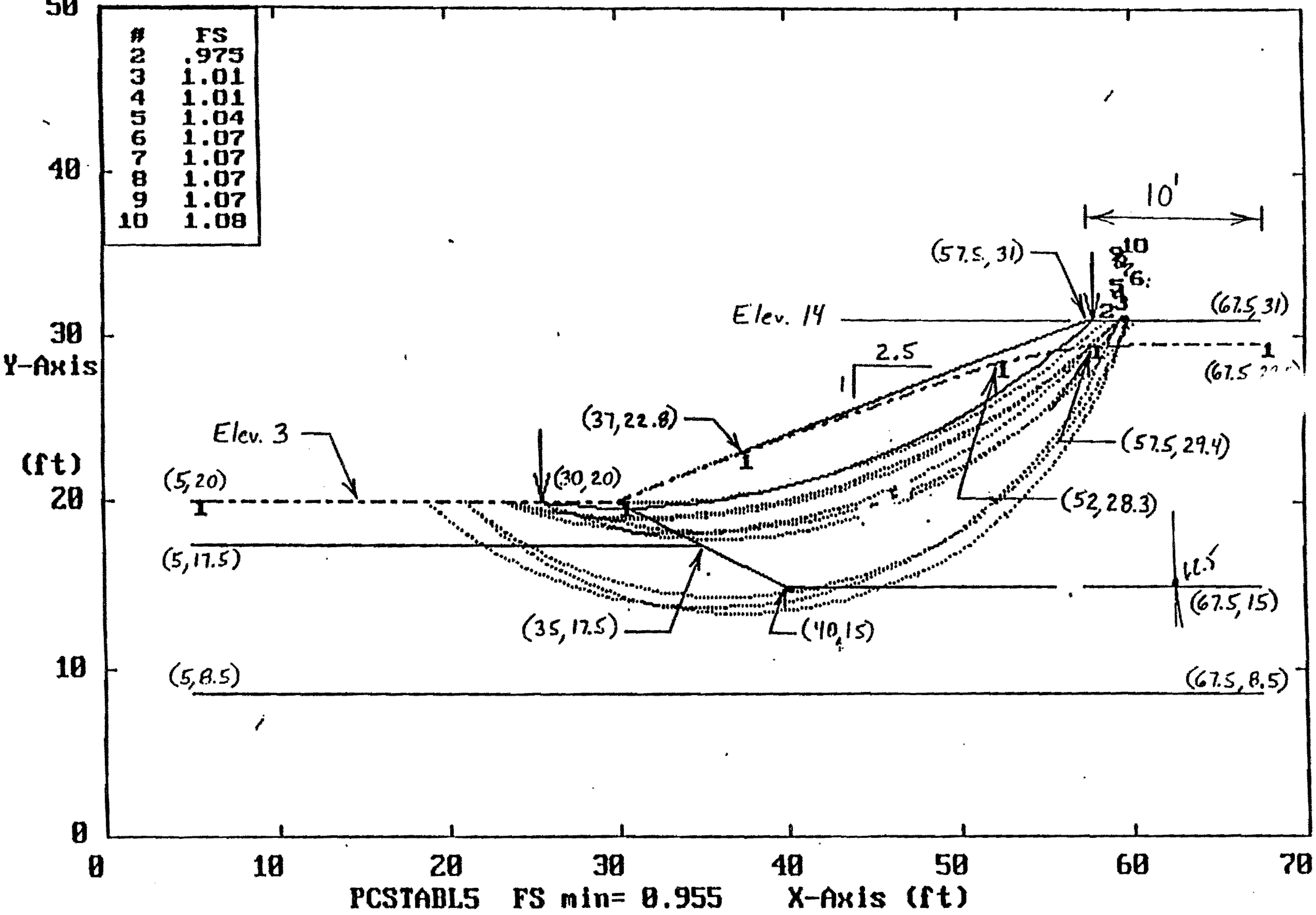
Sta. 0+00 to Sta. 4+50, El. 13 , BP-4
Total Flowrate = 19020.000



$i_e = \frac{1}{7.5} = .13$

EB-3

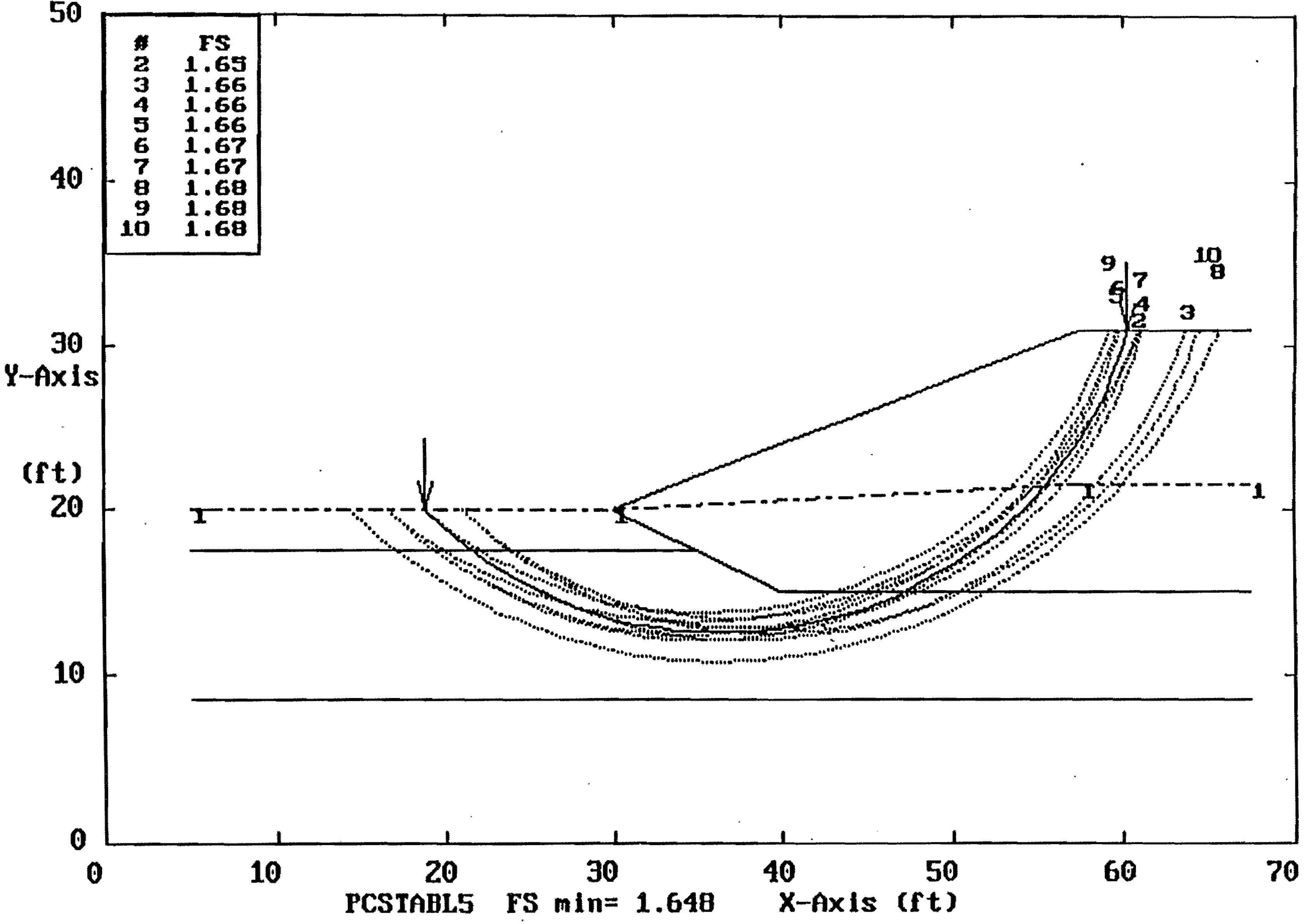
PORT MONMOUTH FLOOD CONTROL FEASIBILITY STUDY
Ten Most Critical. A:BP3D_14.PLT By: EJS 01-12-98 4:09pm



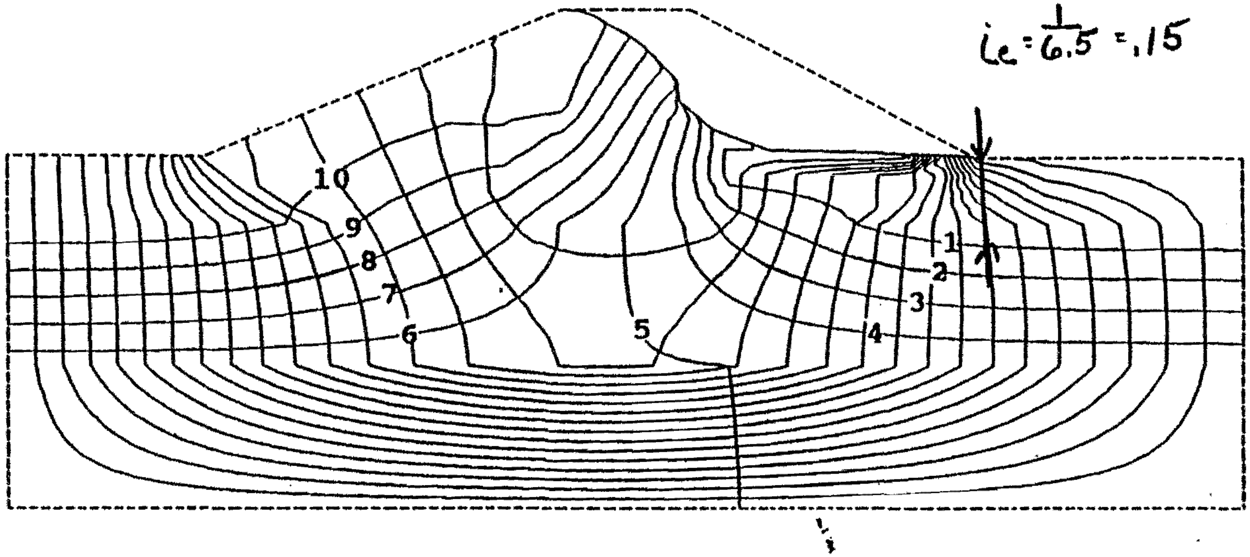
EB-4

EB-5

PORT MONMOUTH FLOOD CONTROL FEASIBILITY STUDY
Ten Most Critical. A:BP3_14.PLT By: EJS 01-15-98 2:20pm



Sta. 0+00 to Sta. 4+50, El. 14 - BP-4
Total Flowrate = 16480.000

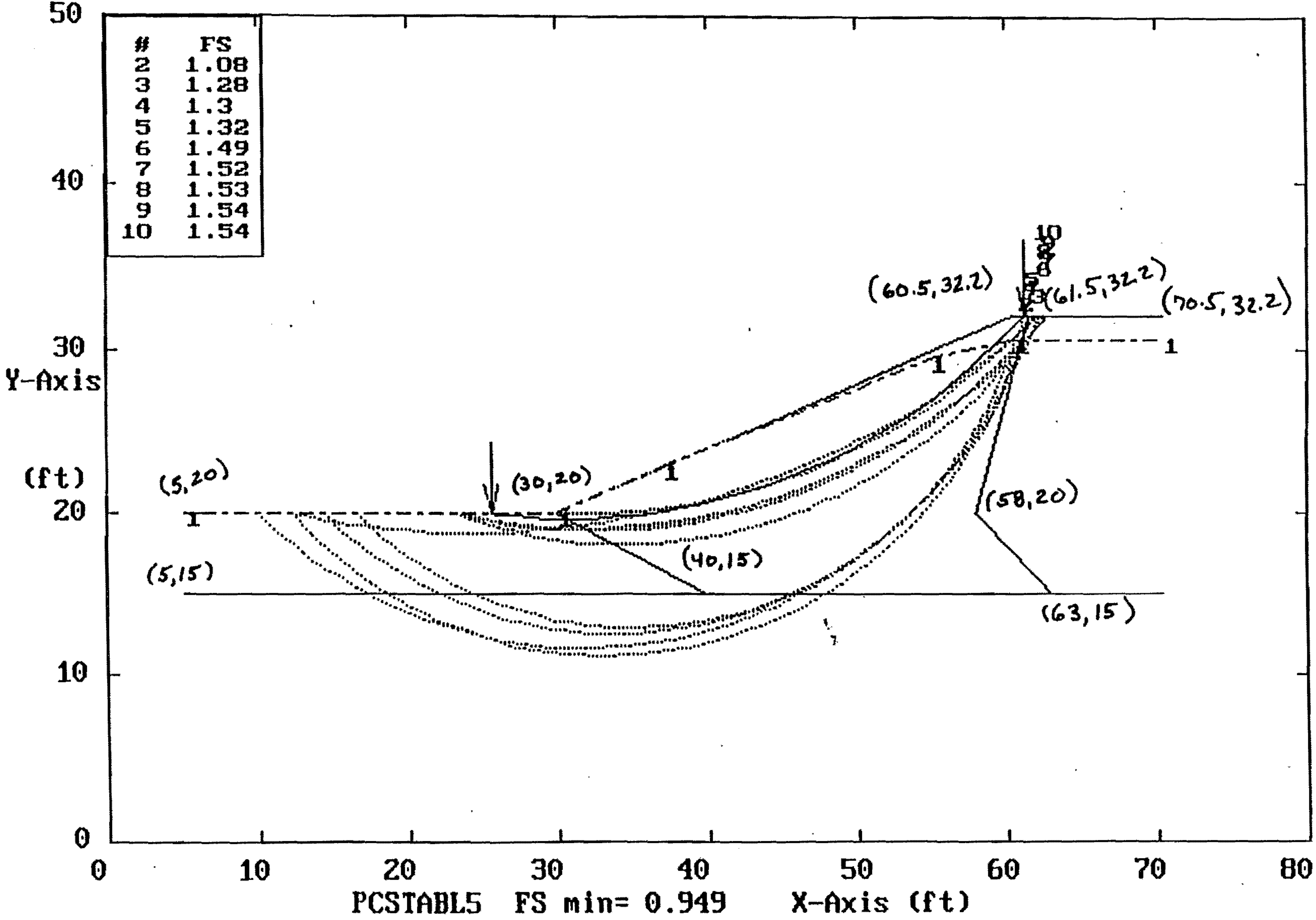


(1 inch = 15.00)

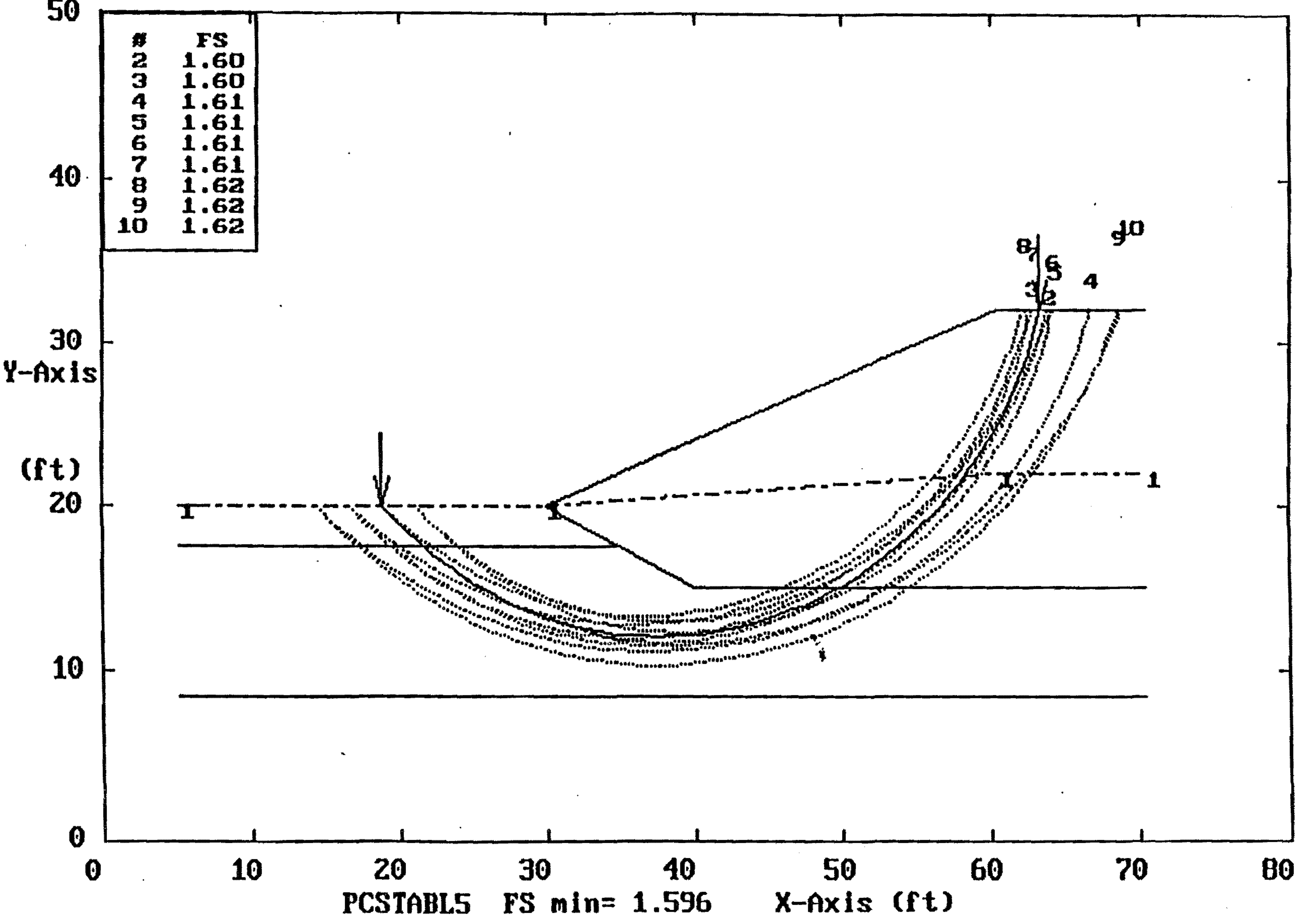
EB-6

EB-7

PORT MONMOUTH FLOOD CONTROL FEASIBILITY STUDY
Ten Most Critical. A:BP3D_15.PLT By: jtm 06-02-98 11:28am



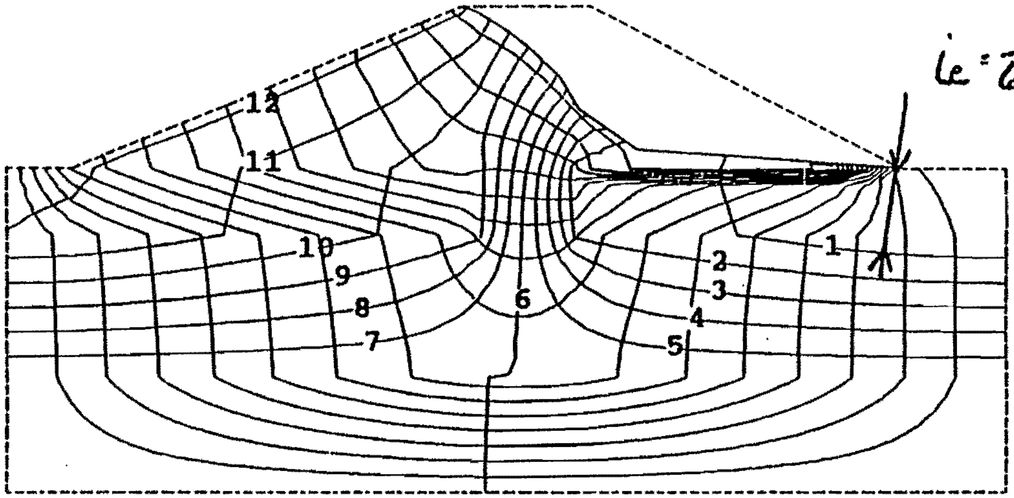
PORT MONMOUTH FLOOD CONTROL FEASIBILITY STUDY
Ten Most Critical. A:BP3_15.PLT By: EJS 01-15-98 2:21pm



PCSTABL5 FS min= 1.596

EB-8

Sta. 0+00 to Sta. 4+50, El. 15.2 BP-4
Total Flowrate = 15320.000



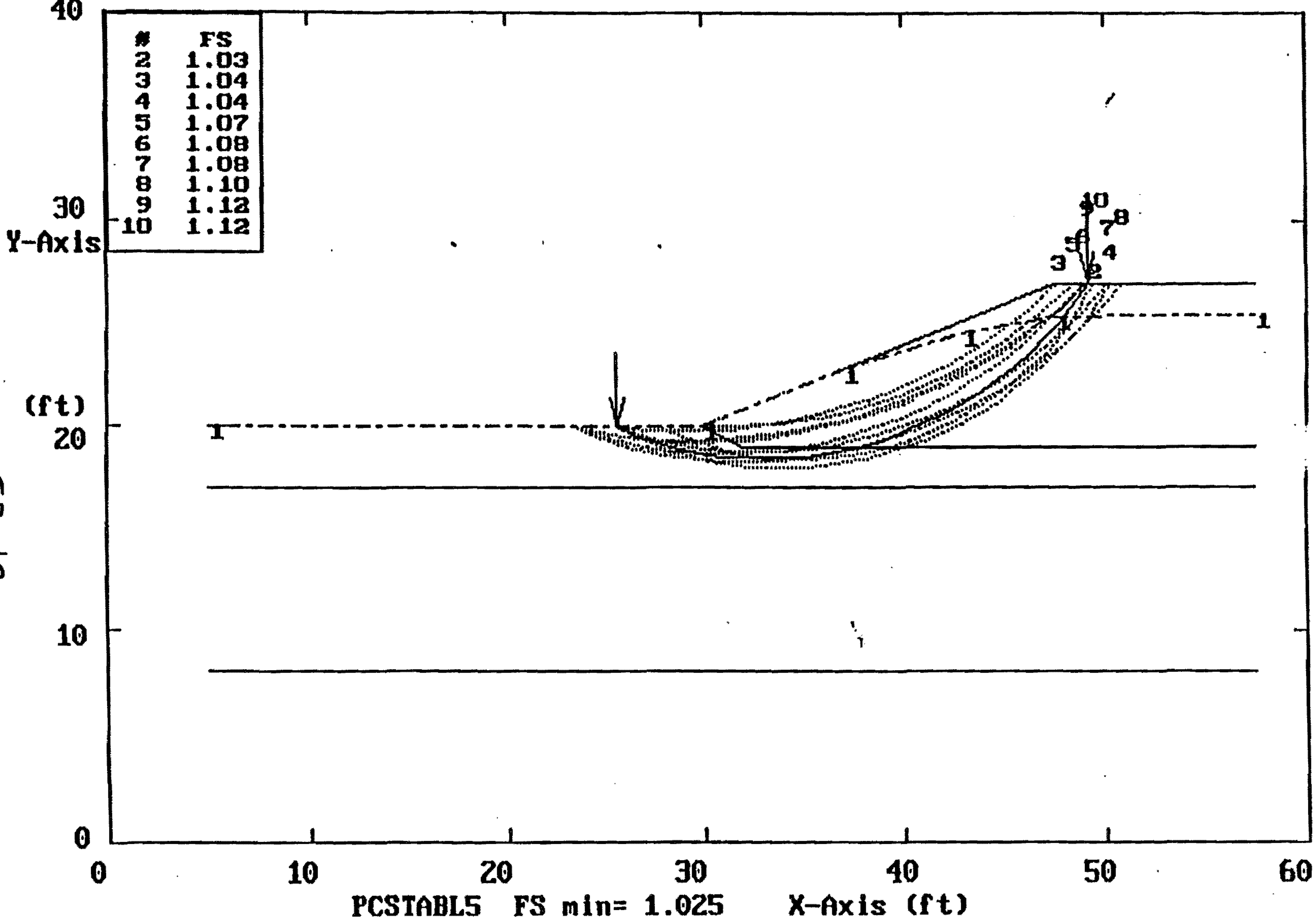
$i_e = \frac{1}{6.5} = .15$

(1 inch = 15.00)

PORT MONMOUTH FLOOD CONTROL FEASIBILITY STUDY

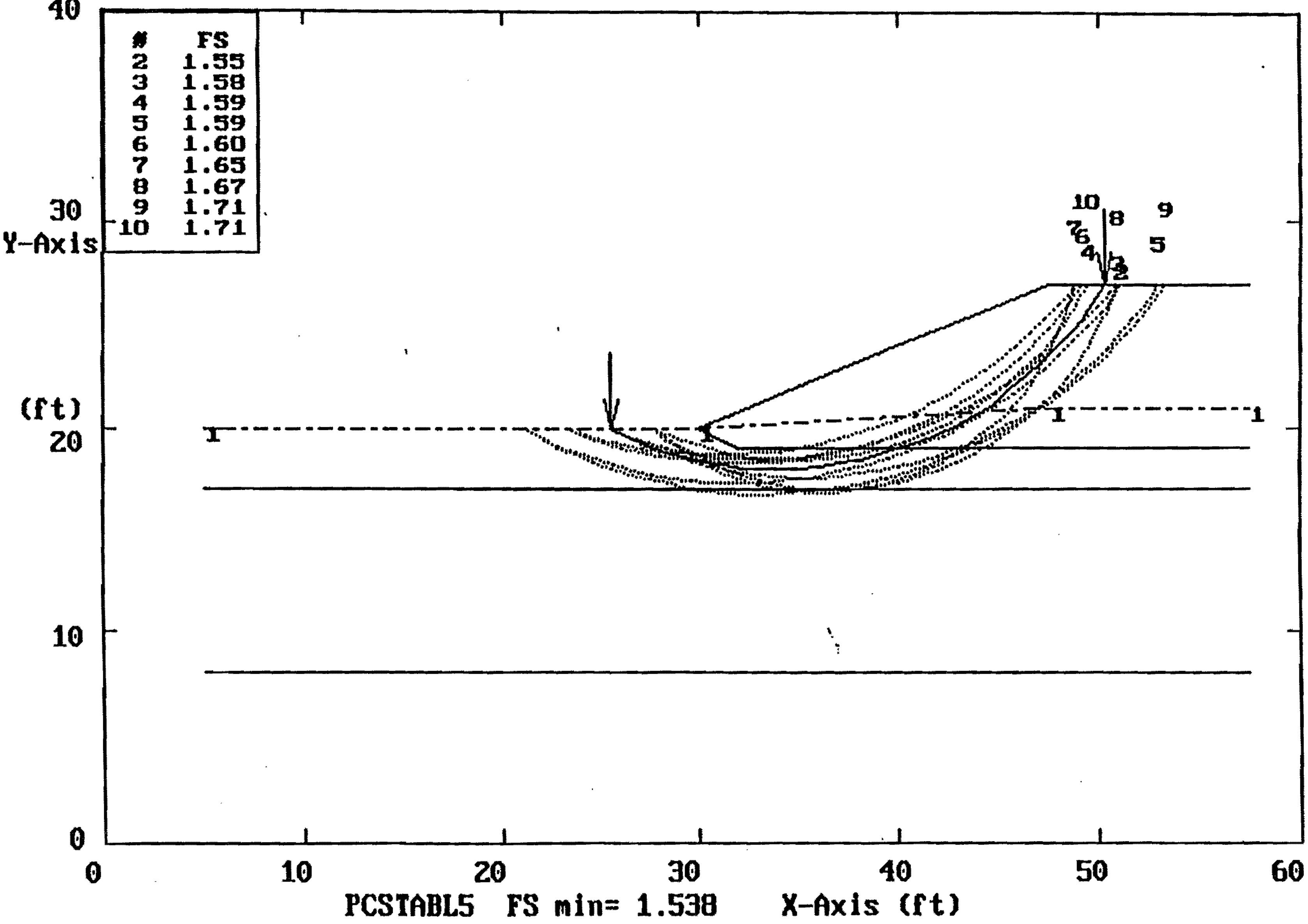
Ten Most Critical. A:PM11D_13.PLT By: EJS 02-04-98 9:03pm

EB-10



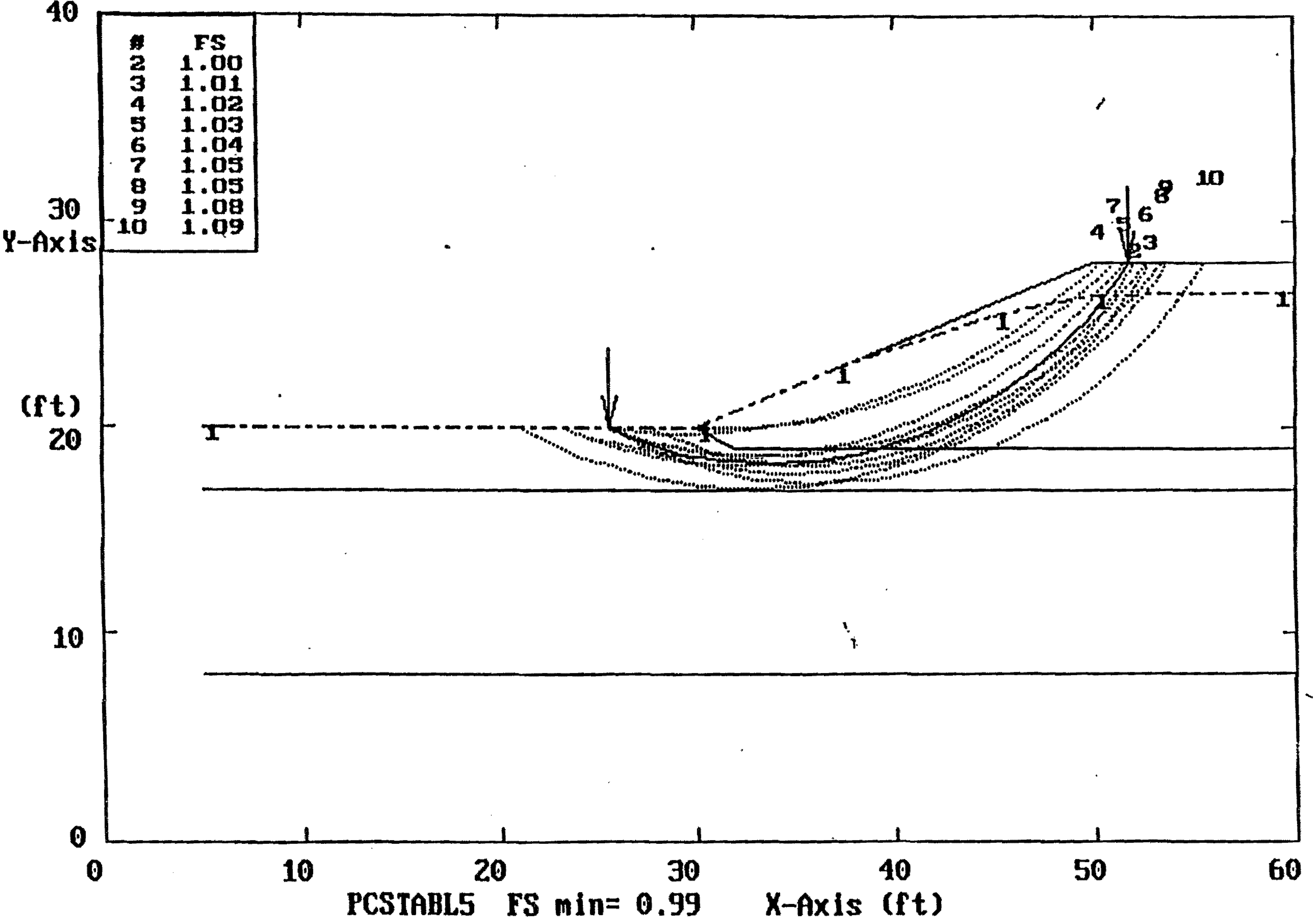
PORT MONMOUTH FLOOD CONTROL FEASIBILITY STUDY
Ten Most Critical. A:PM11_13.PLT By: EJS 02-04-98 7:22pm

EB-11



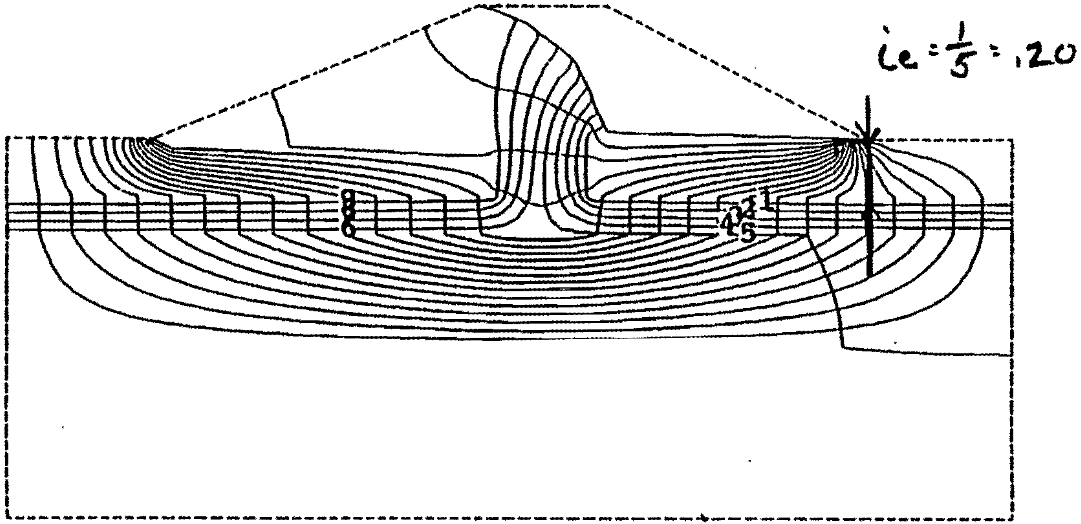
PCSTABL5 FS min= 1.538 X-Axis (ft)

PORT MONMOUTH FLOOD CONTROL FEASIBILITY STUDY
Ten Most Critical. A:PM11D_14.PLT By: EJS 02-04-98 9:06pm



EB-12

Sta. 30+00 to Sta. 32+50, El. 13 , PM-11
Total Flowrate = 17740.000

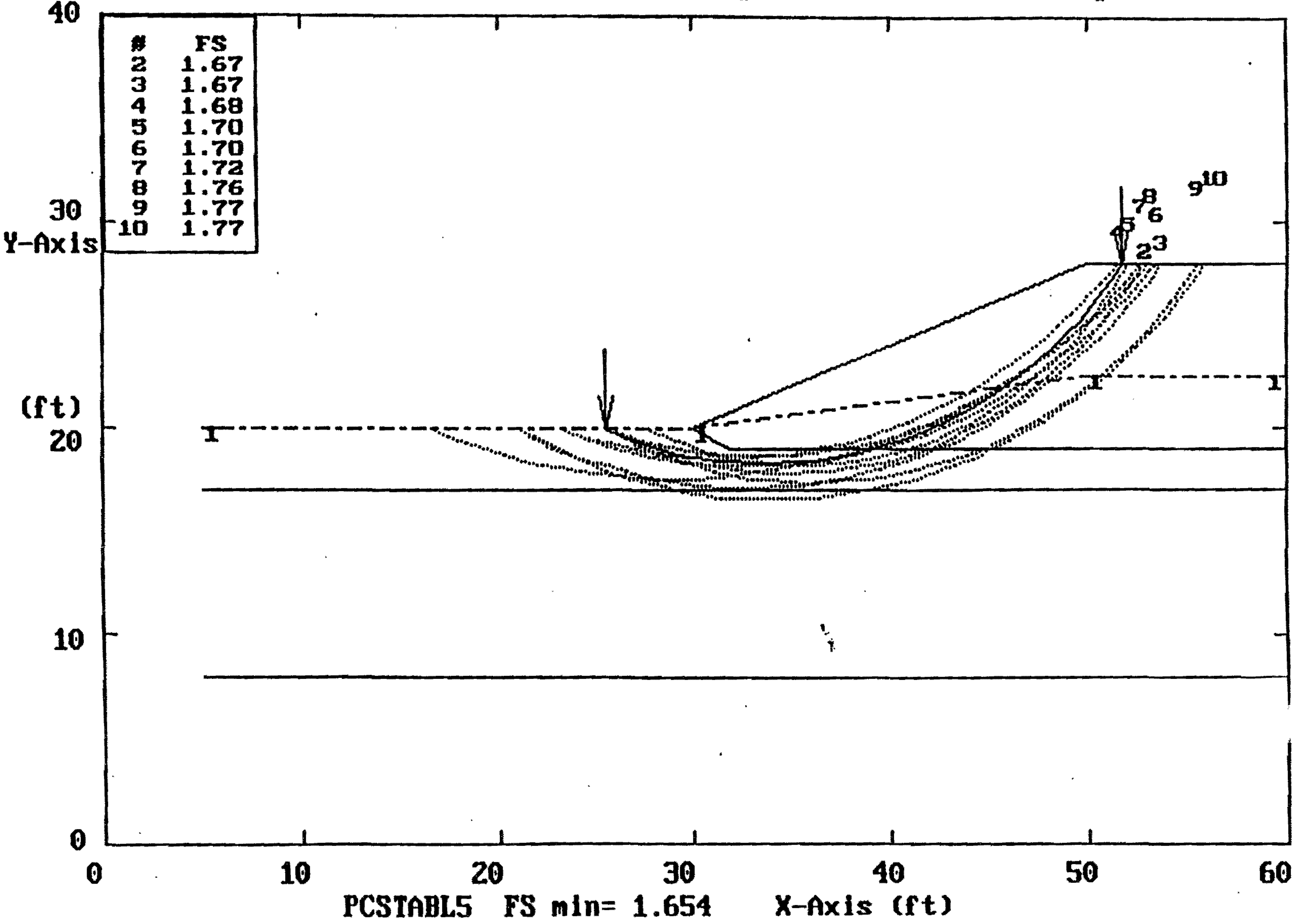


(1 inch = 15.00)

EB-13

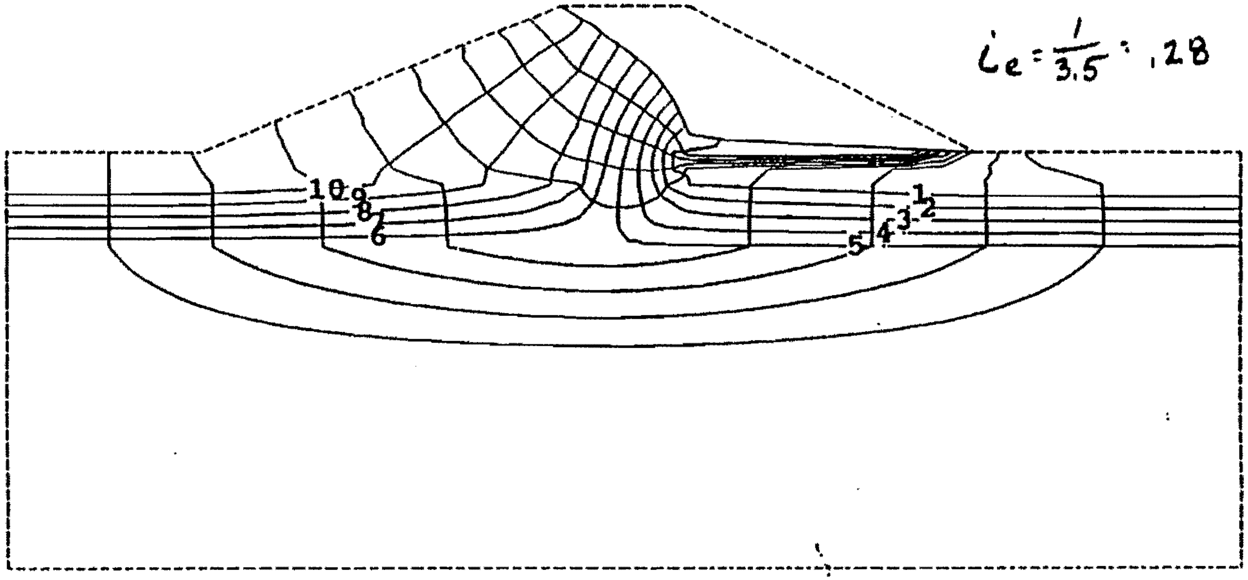
PORT MONMOUTH FLOOD CONTROL FEASIBILITY STUDY
Ten Most Critical. A:PM11_14.PLT By: EJS 02-04-98 7:26pm

EB-14



Sta. 30+00 to Sta. 32+50, El. 14 - PM-11
Total Flowrate = 8977.000

$i_e = \frac{1}{3.5}$: .28



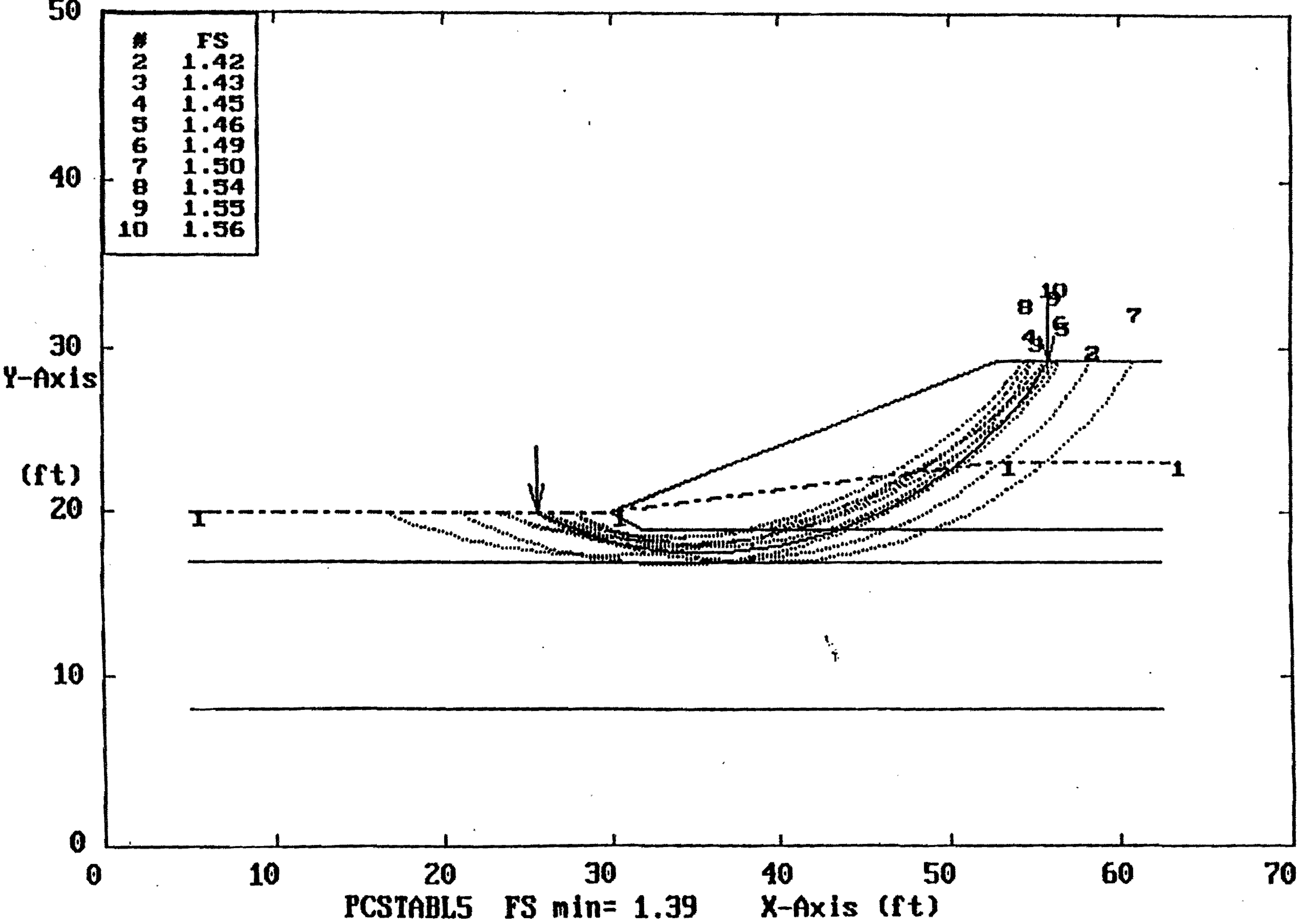
(1 inch = 15.00)

EB-15

PORT MONMOUTH FLOOD CONTROL FEASIBILITY STUDY

Ten Most Critical. A:PM11_15.PLT By: EJS 02-04-98 7:38pm

EB-16

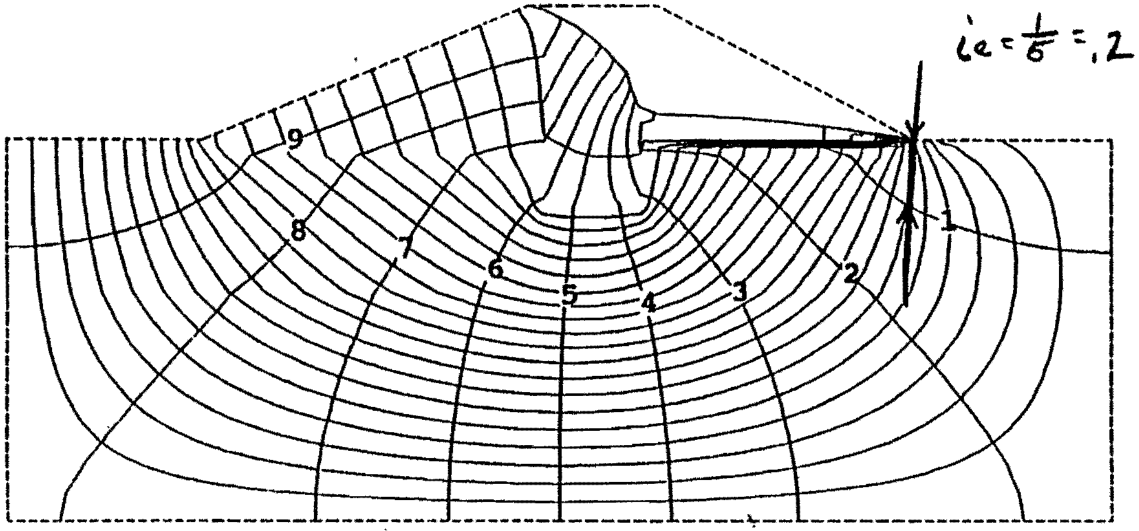


#	FS
2	1.42
3	1.43
4	1.45
5	1.46
6	1.49
7	1.50
8	1.54
9	1.55
10	1.56

PCSTABL5 FS min= 1.39

EB-17

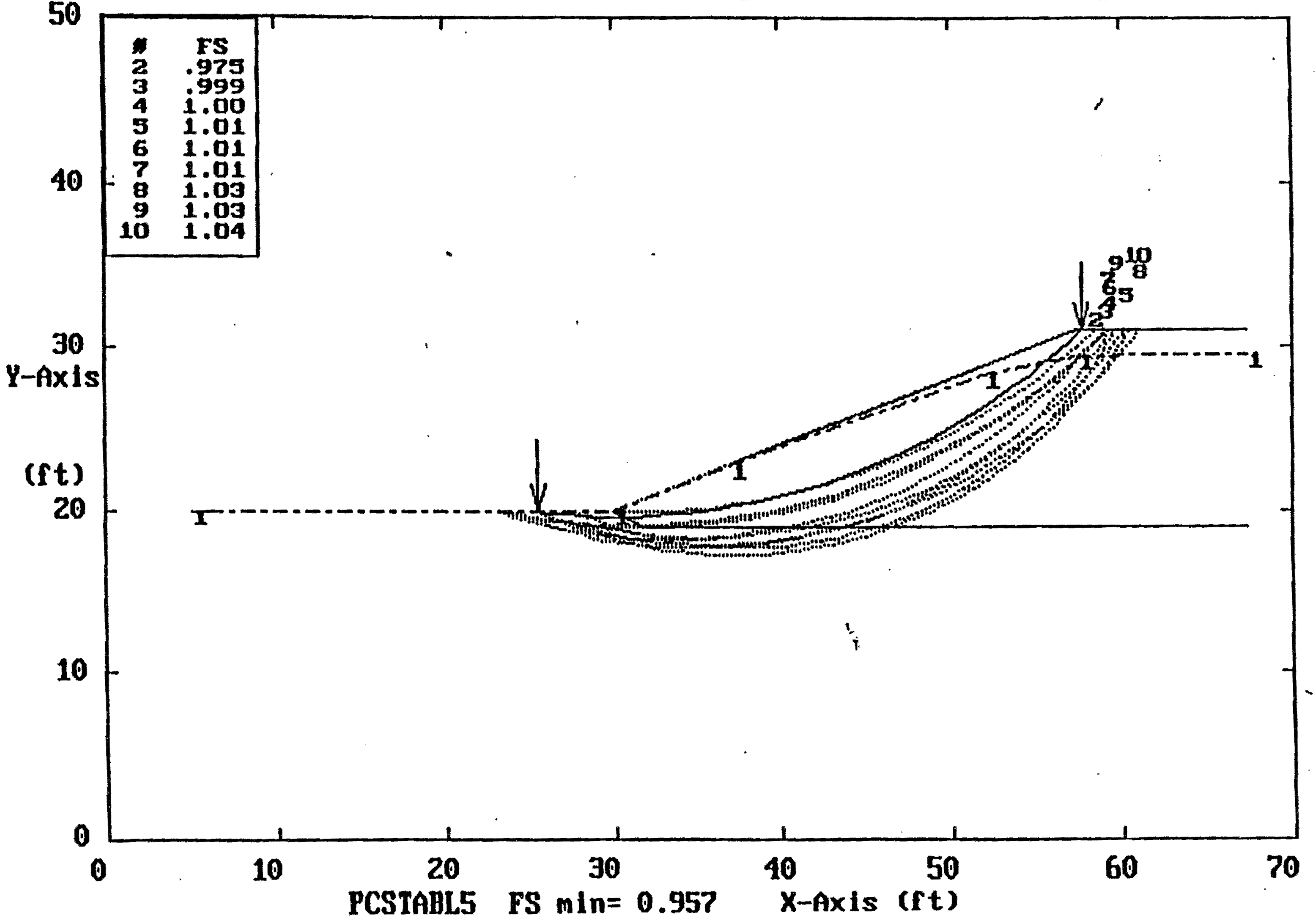
Sta. 0+00 to Sta. 12+00, El. 13 , PM-8
Total Flowrate = 20250.000



$i_e = \frac{1}{5} = .2$

(1 inch = 15.00)

PORT MONMOUTH FLOOD CONTROL FEASIBILITY STUDY
Ten Most Critical. A:PM8D_14.PLT By: EJS 02-04-98 6:55pm

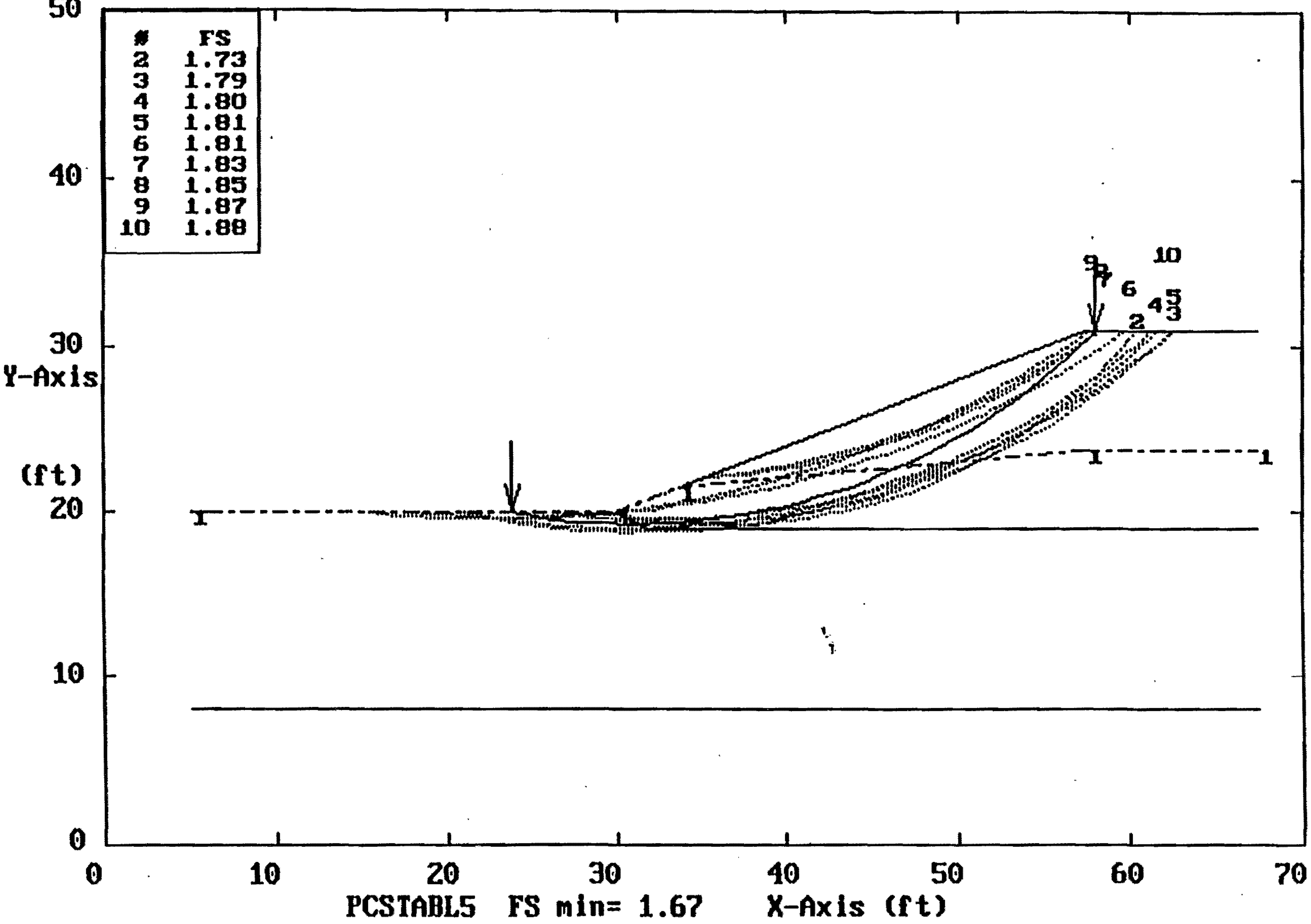


EB-18

EB-19

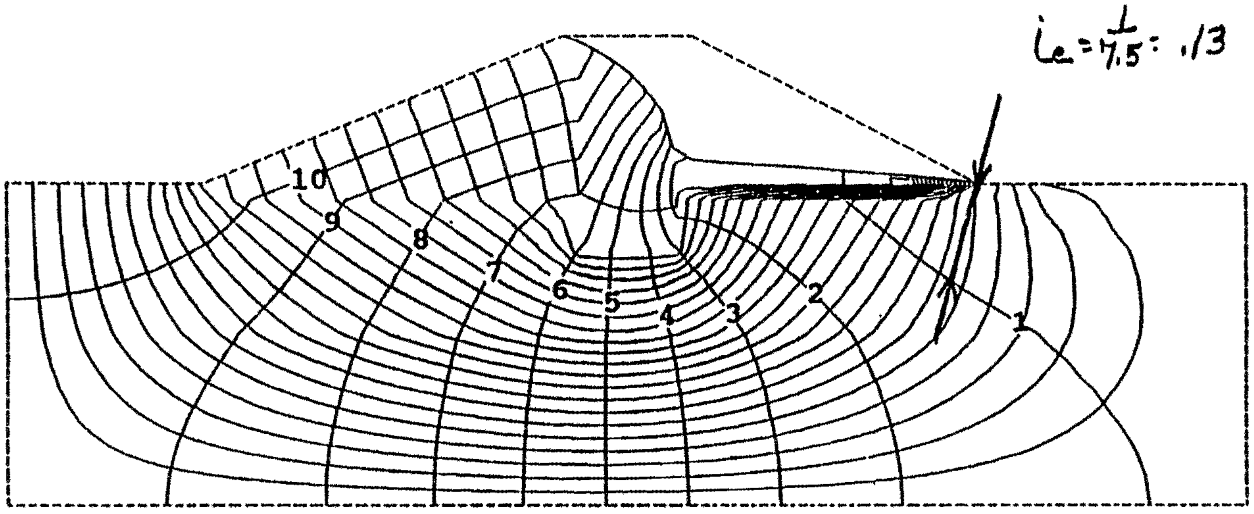
PORT MONMOUTH FLOOD CONTROL FEASIBILITY STUDY

Ten Most Critical. A:PM8_14.PLT By: EJS 02-04-98 8:17pm



EB-20

Sta. 0+00 to Sta. 12+00, El. 14 - PM-8
Total Flowrate = 21630.000

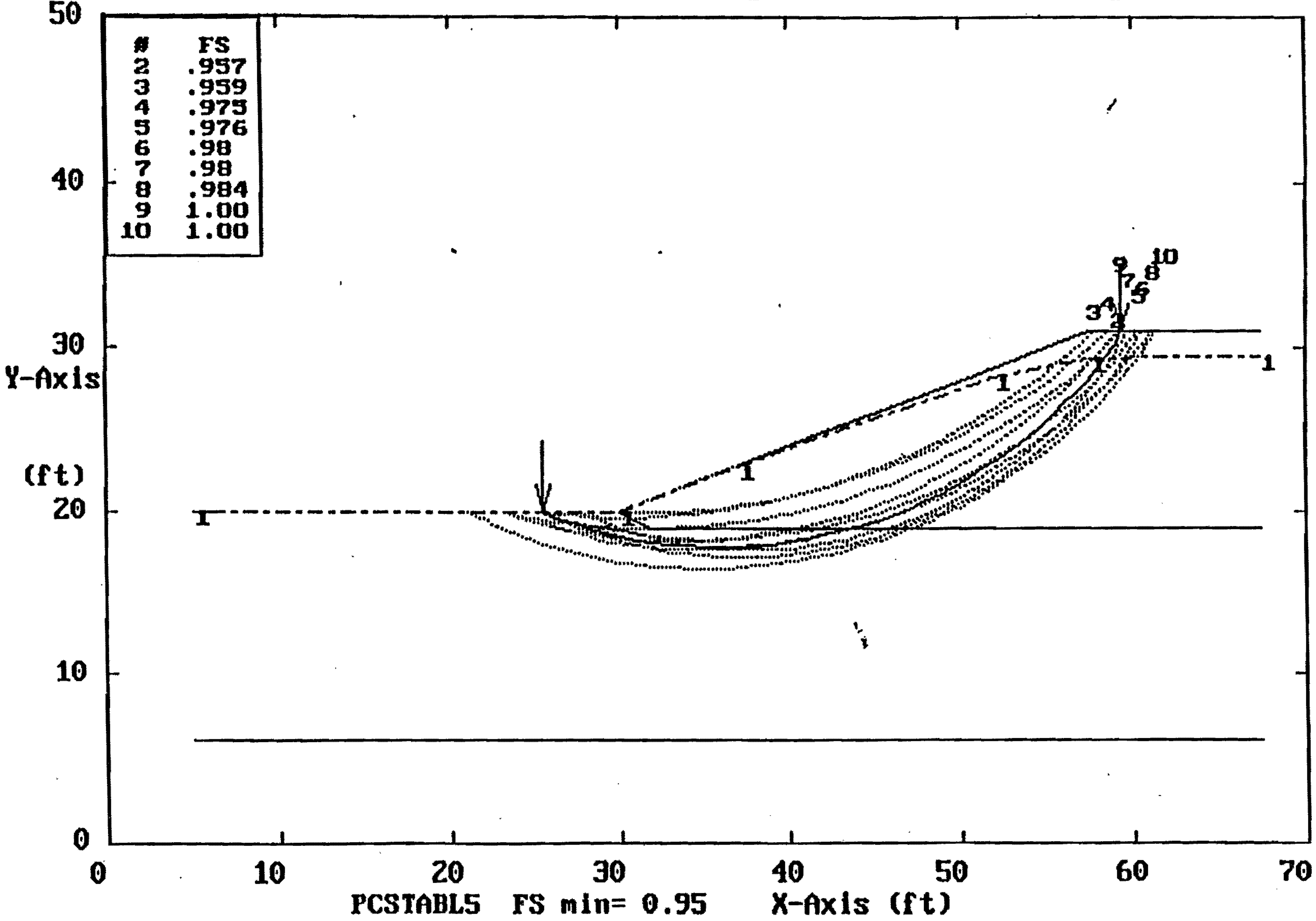


$i_e = \frac{1}{7.5} = .13$

(1 inch = 15.00)

PORT MONMOUTH FLOOD CONTROL FEASIBILITY STUDY

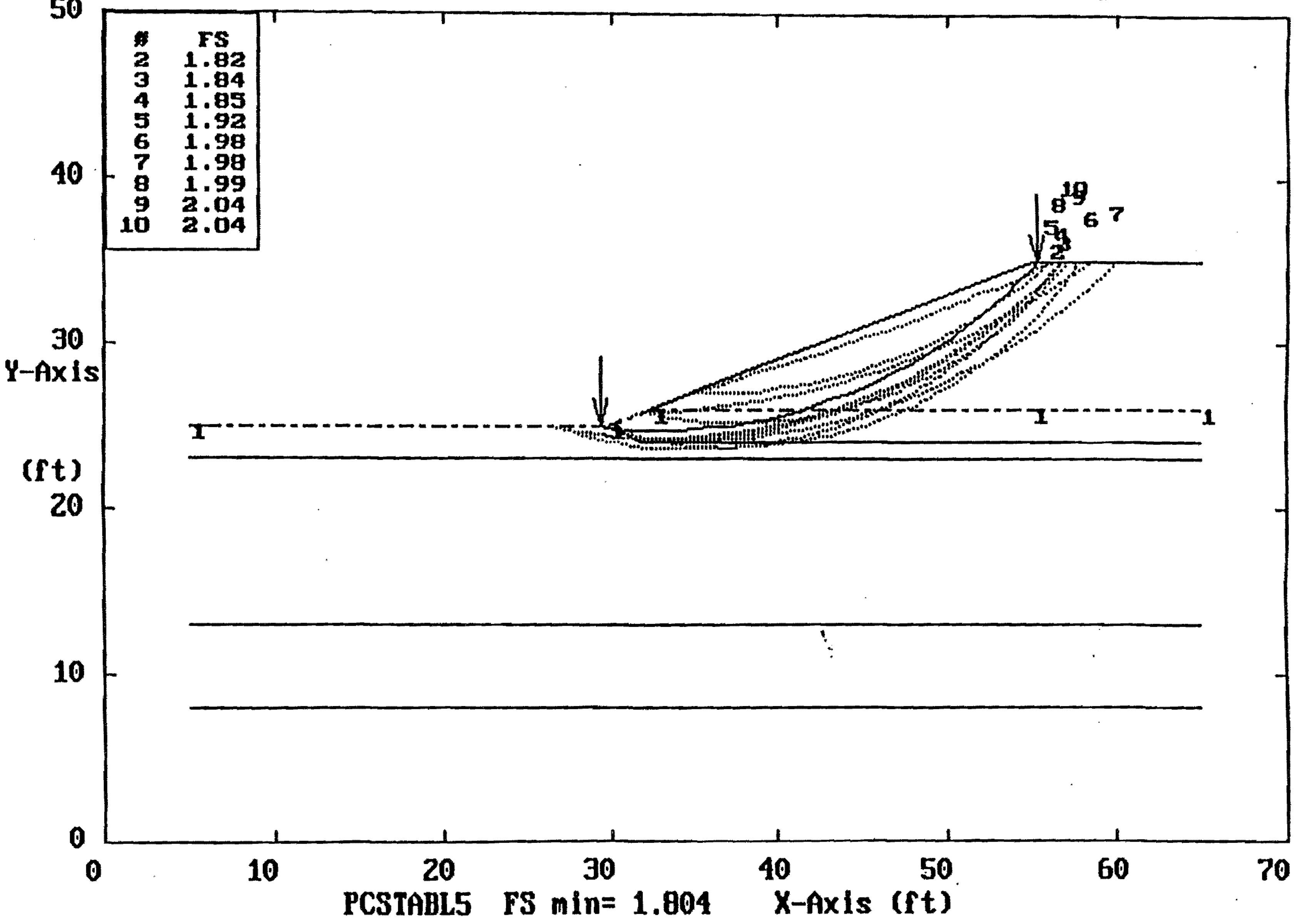
Ten Most Critical. A:PM3D_14.PLT By: EJS 02-04-98 6:36pm



EB-21

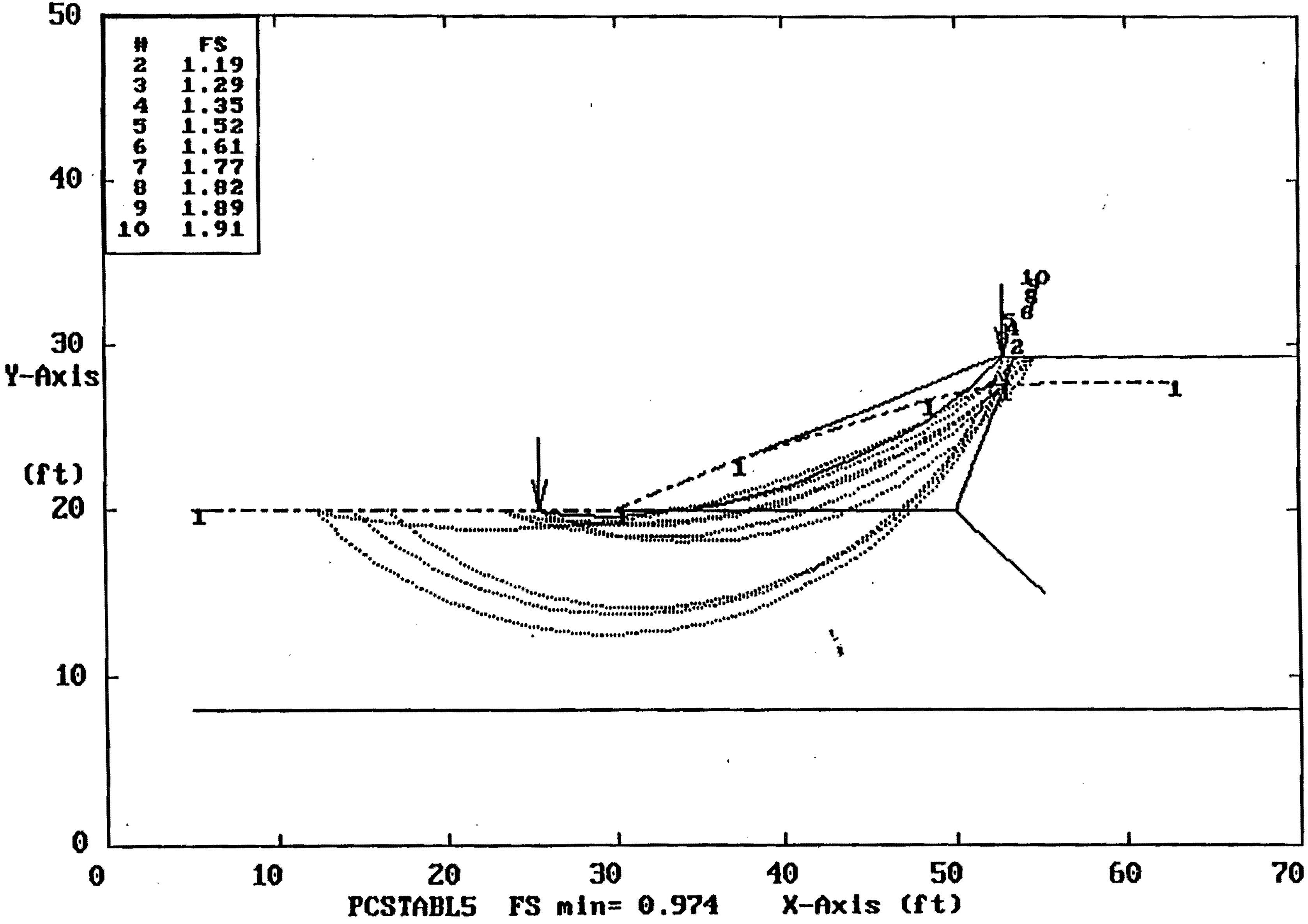
PORT MONMOUTH FLOOD CONTROL FEASIBILITY STUDY

Ten Most Critical. A:PM6_13.PLT By: EJS 02-04-98 8:08pm



EB-22

PORT MONMOUTH FLOOD CONTROL FEASIBILITY STUDY
Ten Most Critical. A:PM11D_15.PLT By: JTM 06-02-98 3:31pm

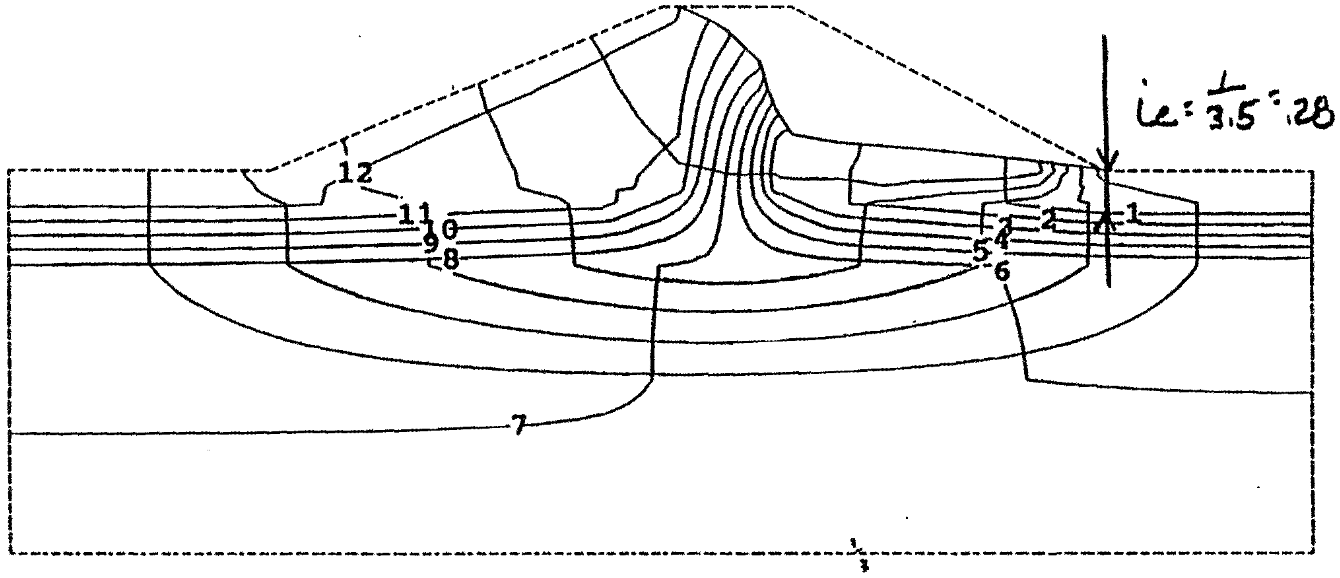


PCSTABL5 FS min= 0.974

EB-23

EB-24

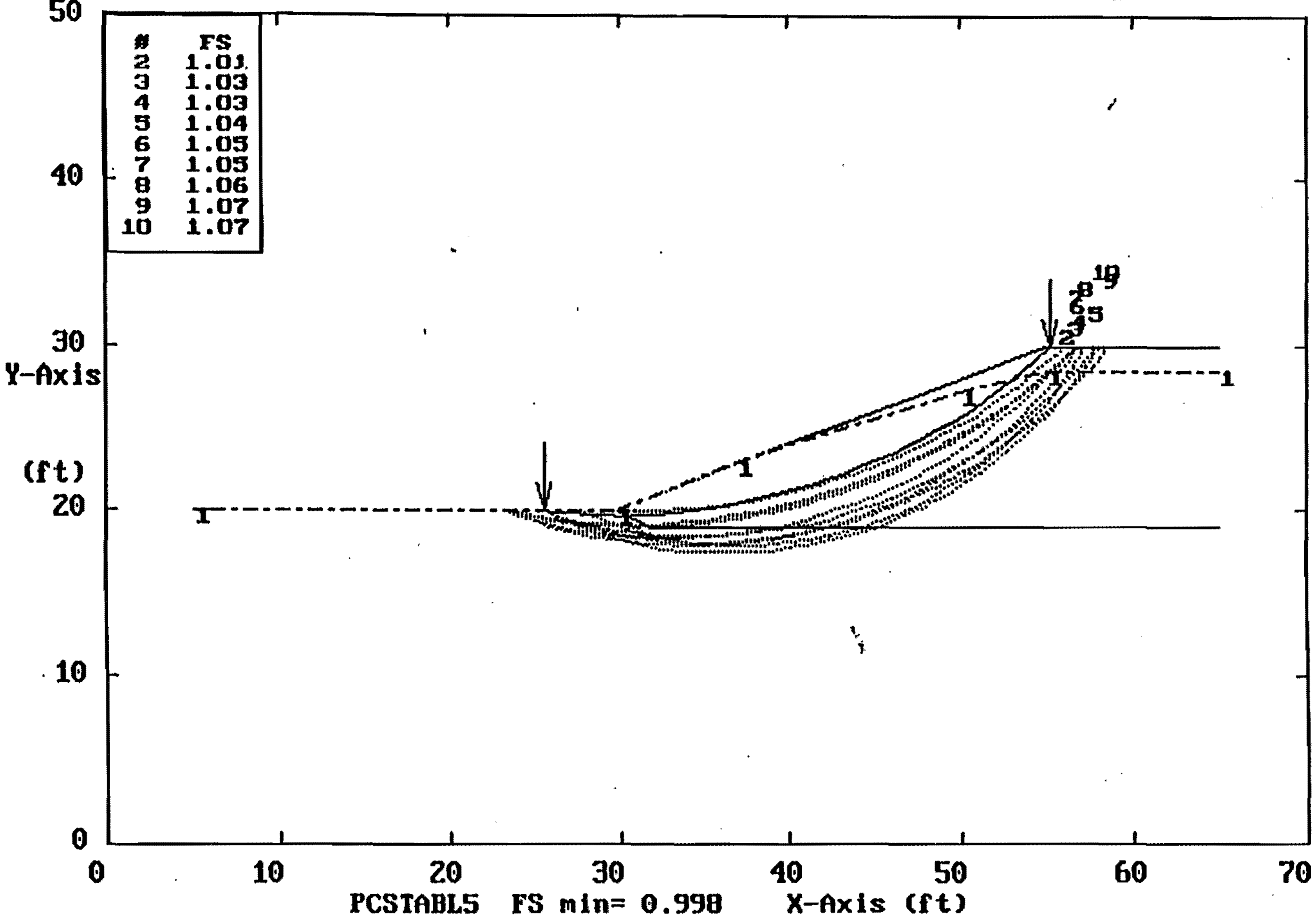
Sta. 30+00 to Sta. 32+50, El. 15.2 PM-11
Total Flowrate = 5821.000



(1 inch = 15.00)

PORT MONMOUTH FLOOD CONTROL FEASIBILITY STUDY

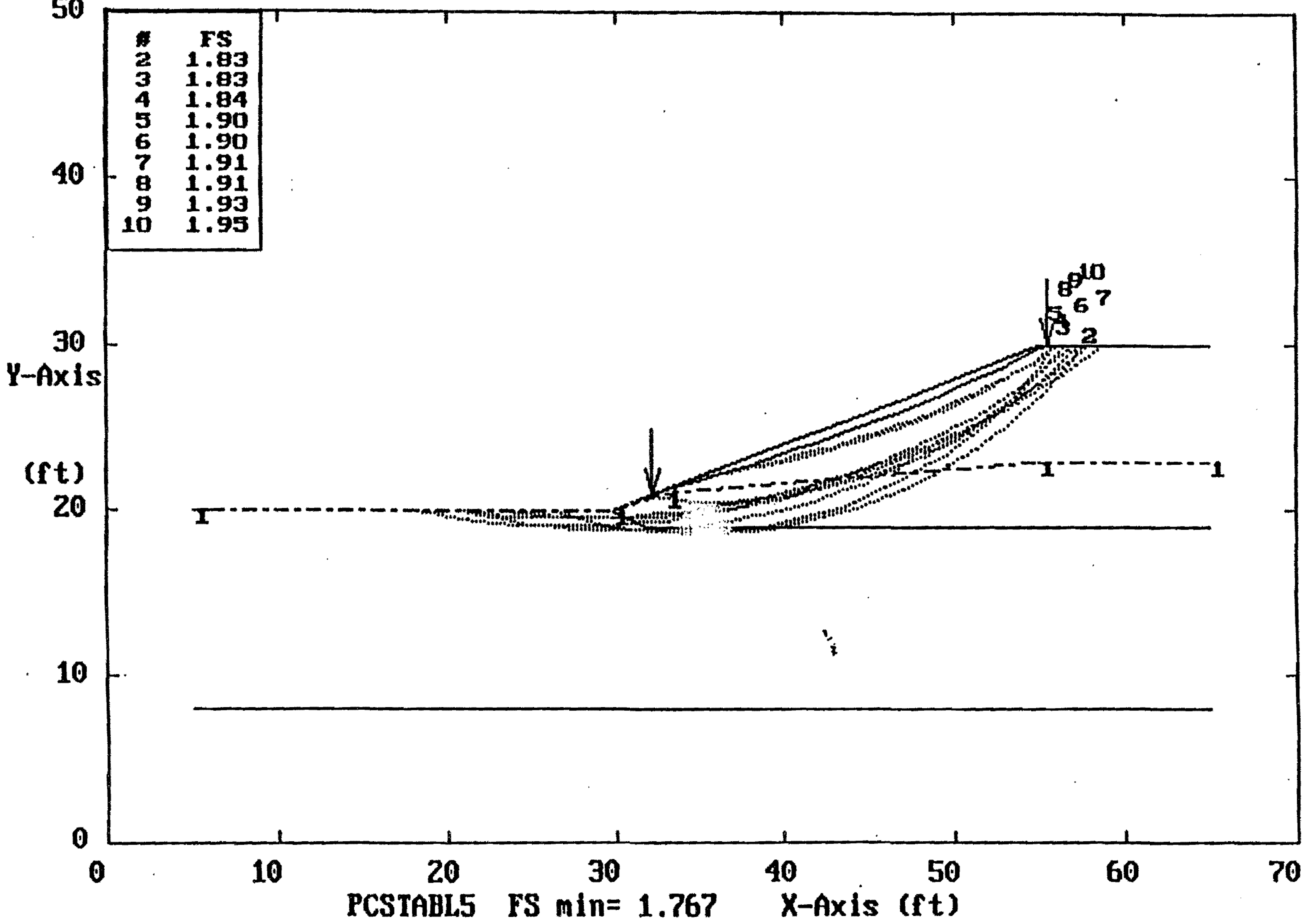
Ten Most Critical. A:PM8D_13.PLT By: EJS 02-04-98 6:54pm



EB-25

PORT MONMOUTH FLOOD CONTROL FEASIBILITY STUDY

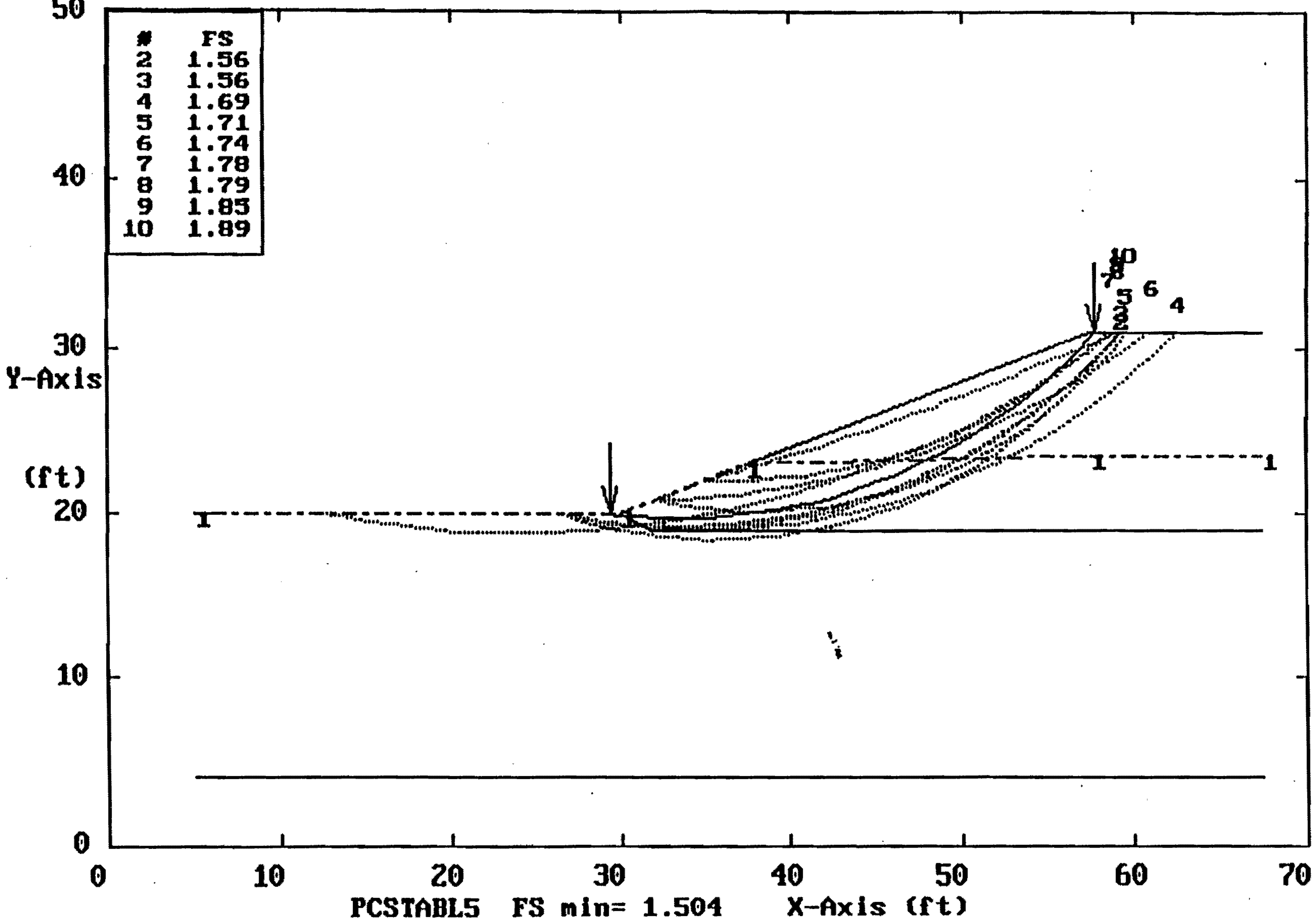
Ten Most Critical. A:PM8_13.PLT By: EJS 02-04-98 8:00pm



EB-26

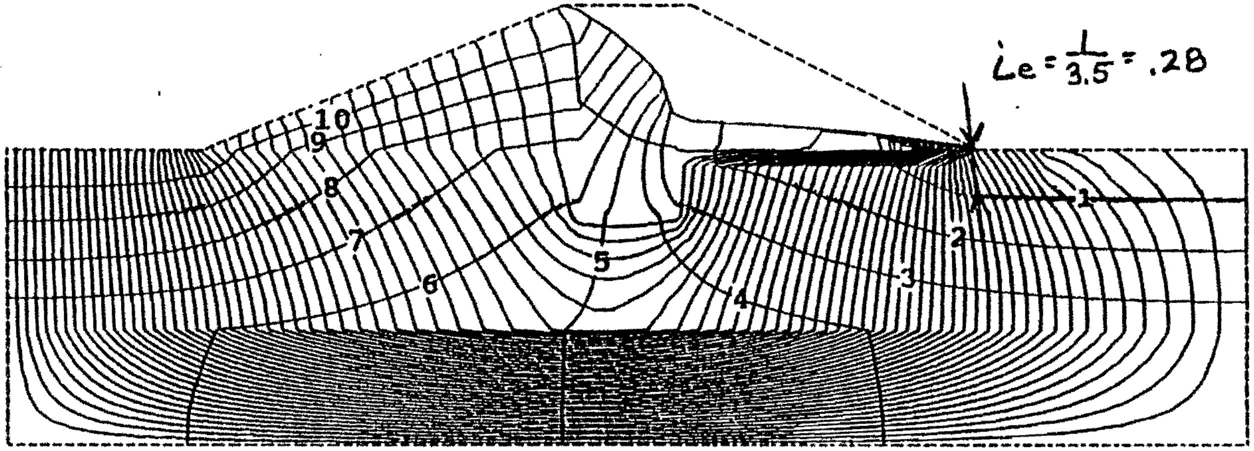
PORT MONMOUTH FLOOD CONTROL FEASIBILITY STUDY

Ten Most Critical. A:PM3_14.PLT By: EJS 02-04-98 8:25pm



EB-27

Sta. 22+00 to Sta. 42+50, El. 14 - PM-3
Total Flowrate = 45200.000



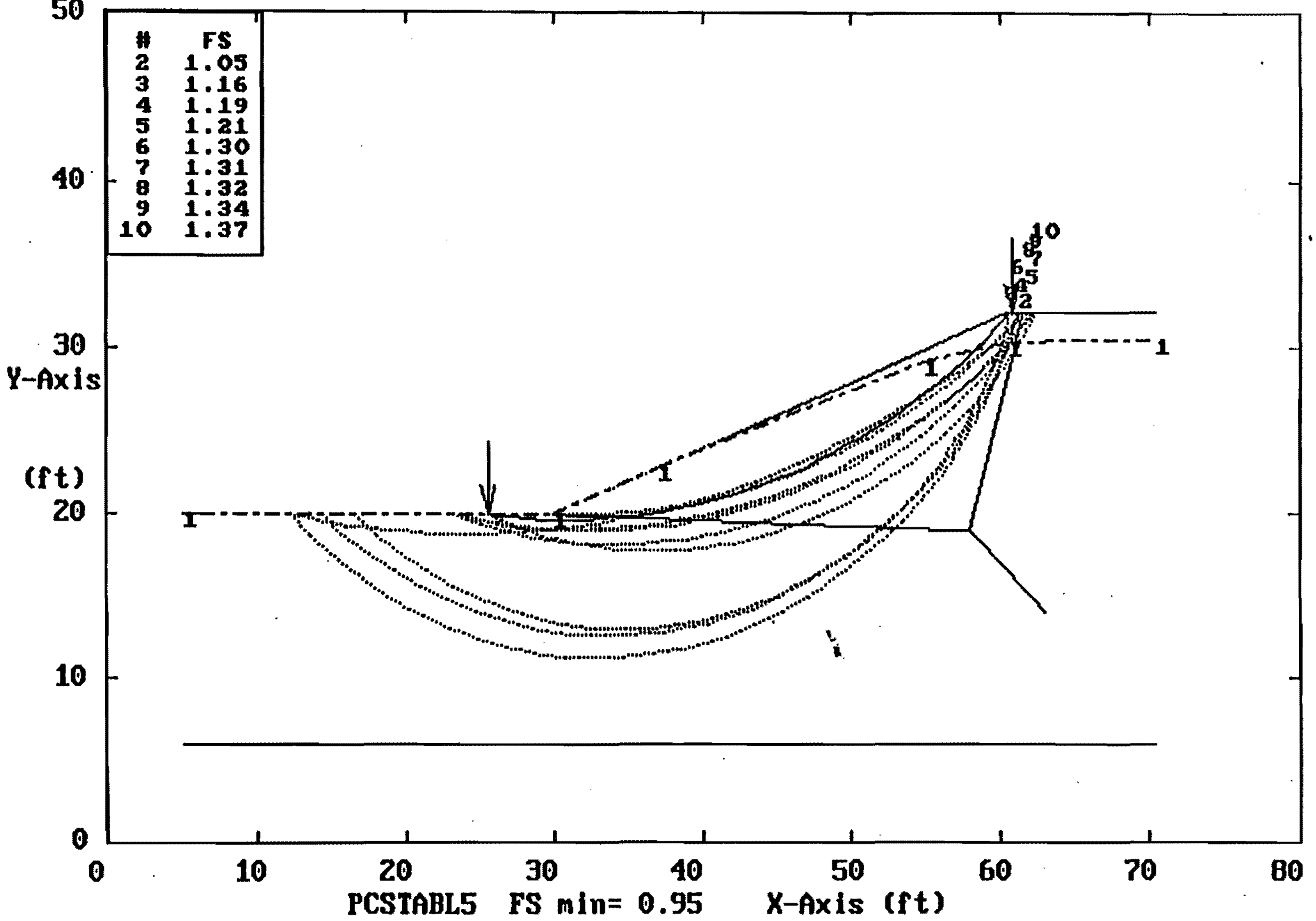
$i_e = \frac{L}{3.5} = .28$

(1 inch = 15.00)

EB-28

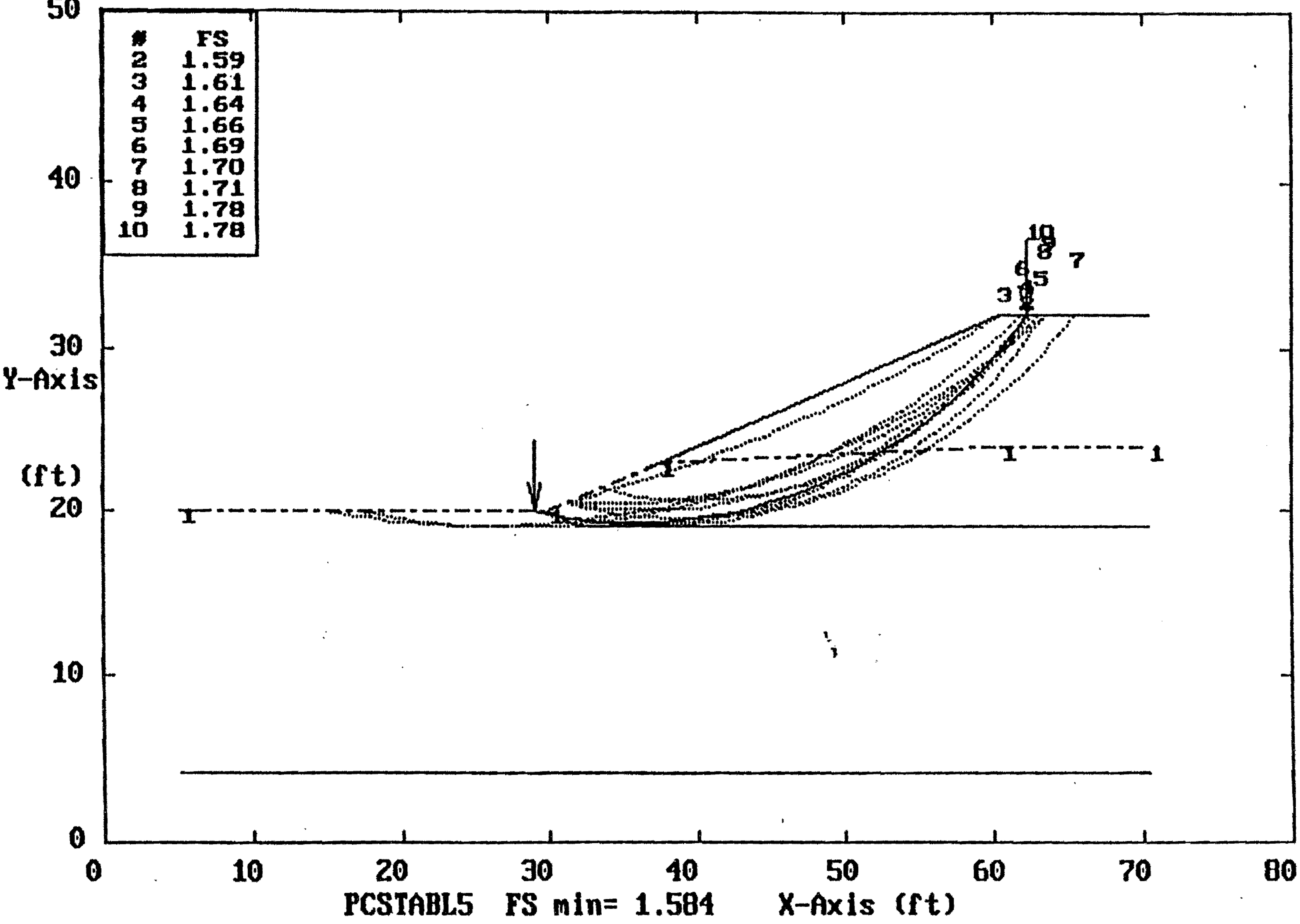
PORT MONMOUTH FLOOD CONTROL FEASIBILITY STUDY
Ten Most Critical. A:PM3D_15.PLT By: JTM 06-02-98 5:29pm

EB-29



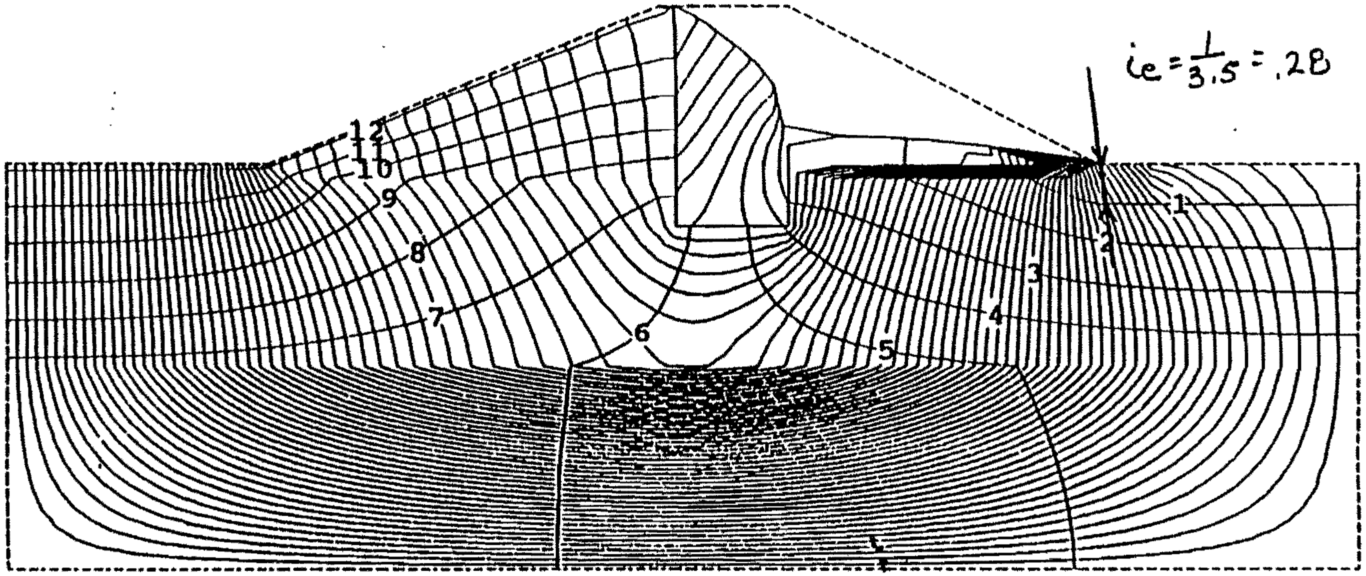
PORT MONMOUTH FLOOD CONTROL FEASIBILITY STUDY

Ten Most Critical. A:PM3_15.PLT By: EJS 02-04-98 7:41pm



EB-30

Sta. 22+00 to Sta. 42+50, El. 15.2 PM-3
Total Flowrate = 51010.000

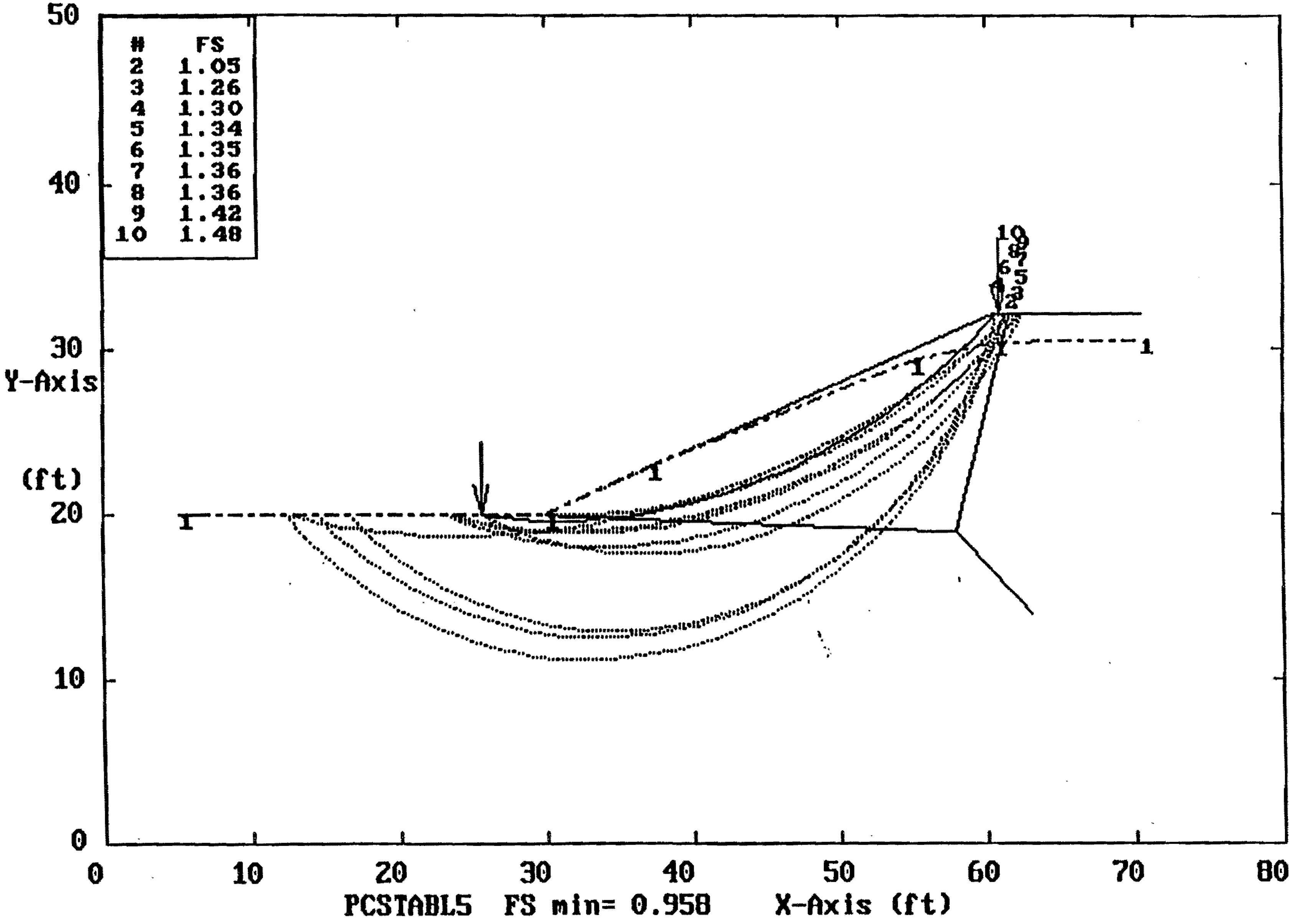


(1 inch = 15.00)

EB-31

PORT MONMOUTH FLOOD CONTROL FEASIBILITY STUDY

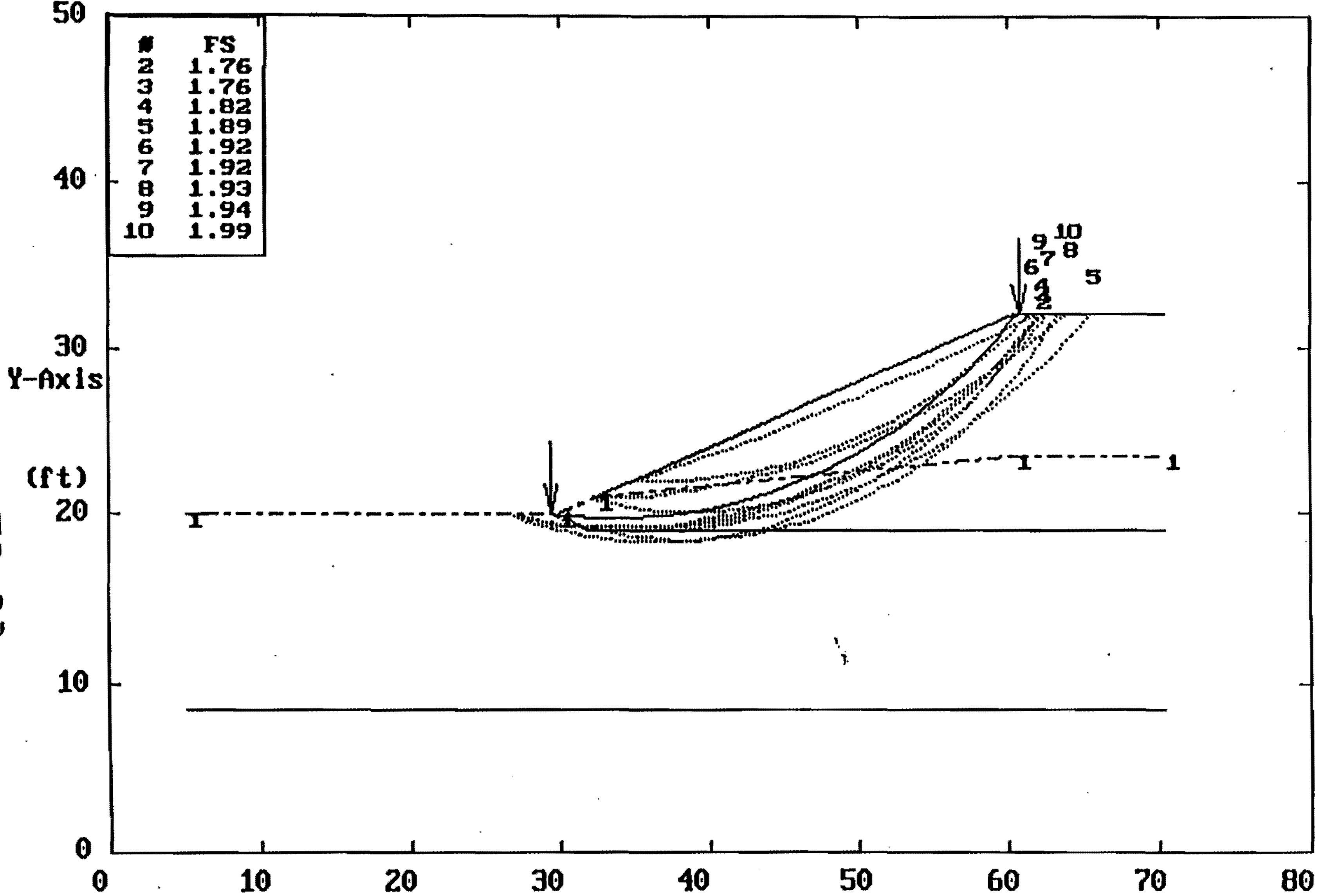
Ten Most Critical. A:PM8D_15.PLT By: JTM 06-02-98 4:43pm



EB-32

PORT MONMOUTH FLOOD CONTROL FEASIBILITY STUDY
Ten Most Critical. A:PM8_15.PLT By: EJS 02-04-98 8:18pm

EB-33

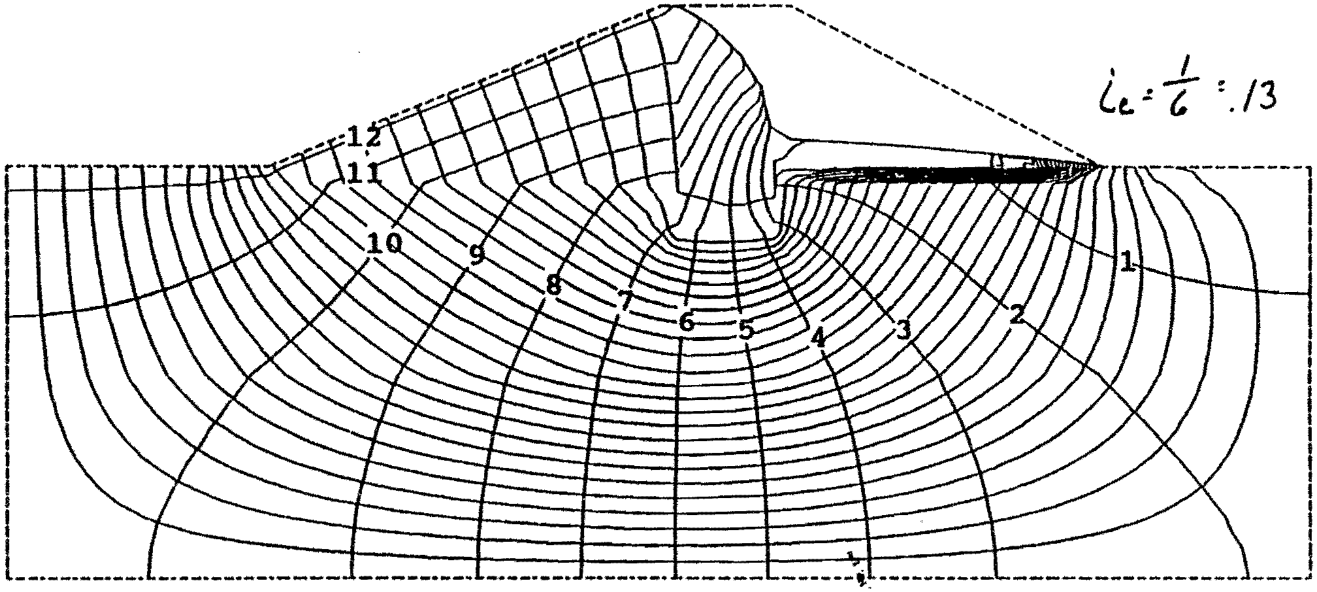


#	FS
2	1.76
3	1.76
4	1.82
5	1.89
6	1.92
7	1.92
8	1.93
9	1.94
10	1.99

PCSTABL5 FS min= 1.746 X-Axis (ft)

EB-34

Sta. 0+00 to Sta. 12+00, El. 15.2 PM-8
Total Flowrate = 25400.000

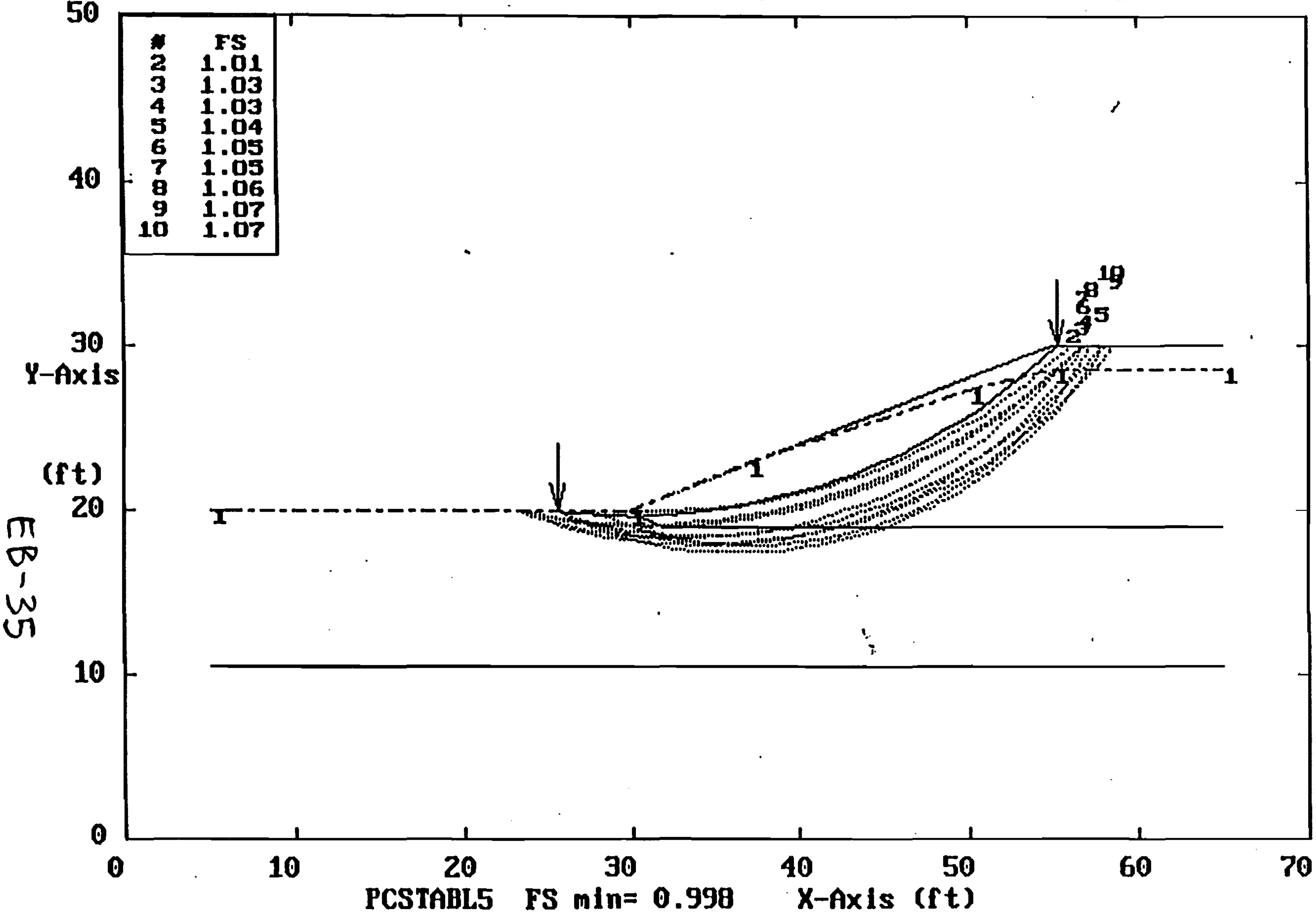


$i_e = \frac{1}{6} = .13$

(1 inch = 15.00)

PORT MONMOUTH FLOOD CONTROL FEASIBILITY STUDY

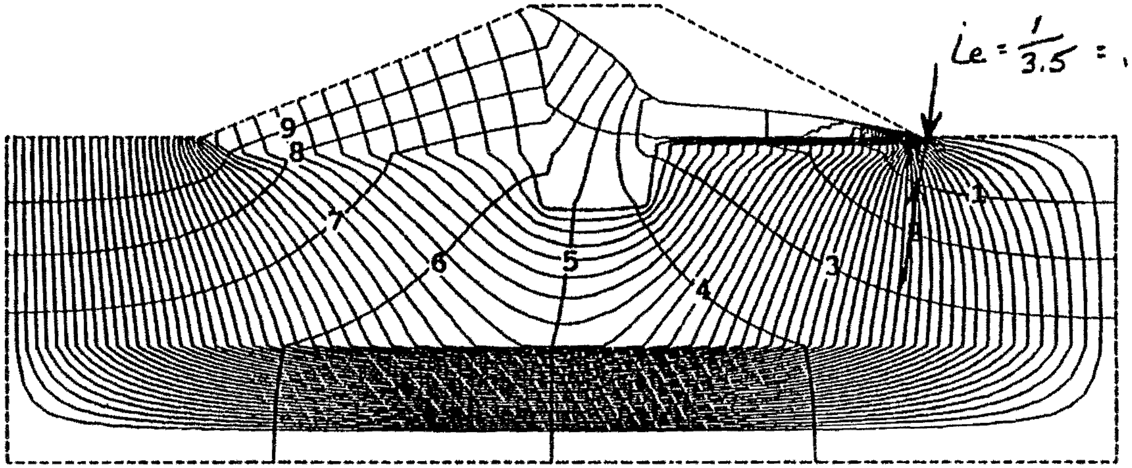
Ten Most Critical. A:PM6D_13.PLT By: EJS 02-04-98 6:54pm



EB-35

EB-36

Sta. 12+00 to Sta. 22+00, El. 13 , PM-6
Total Flowrate = 44390.000

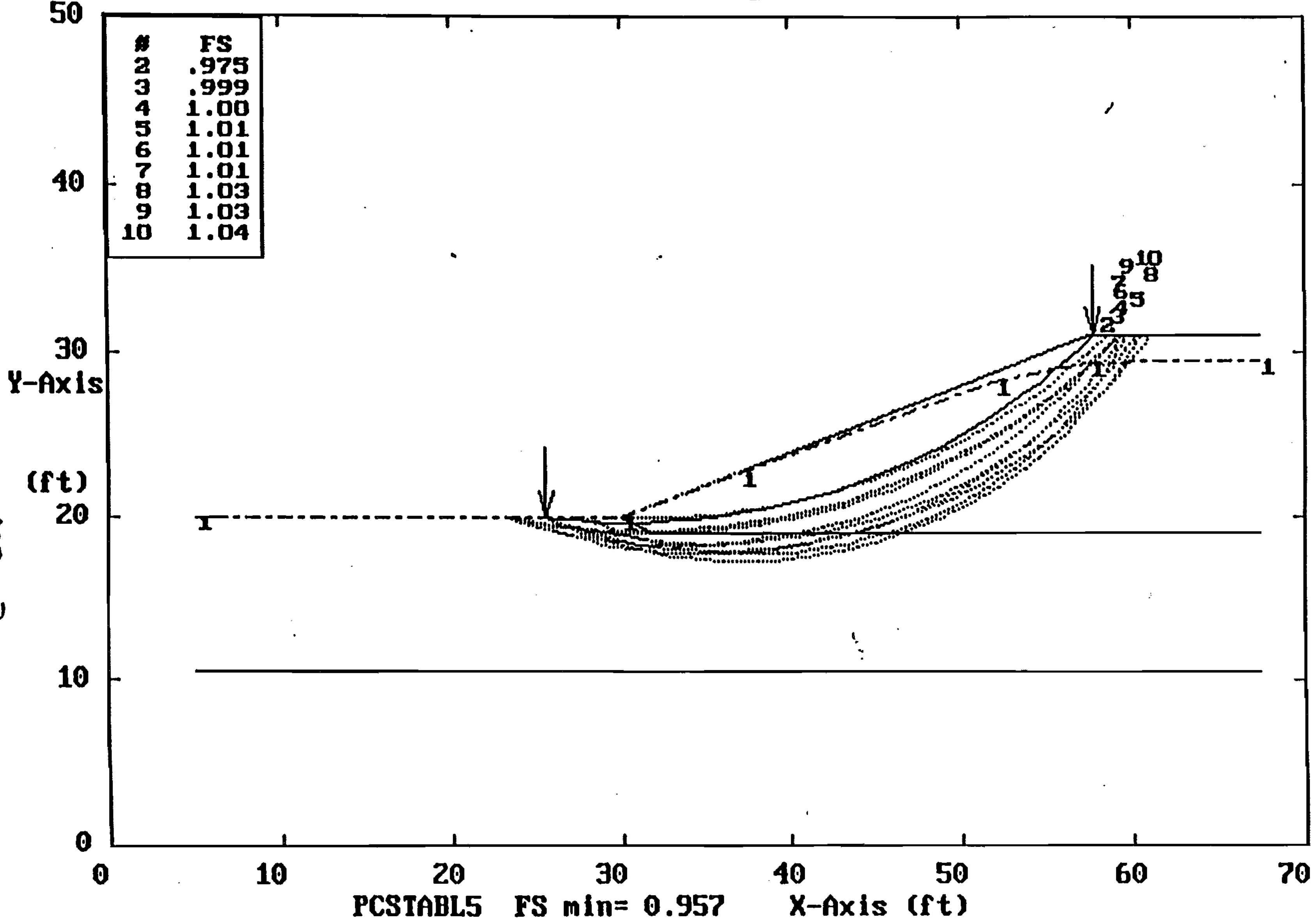


$i_e = \frac{1}{3.5} = .28$

(1 inch = 15.00)

PORT MONMOUTH FLOOD CONTROL FEASIBILITY STUDY

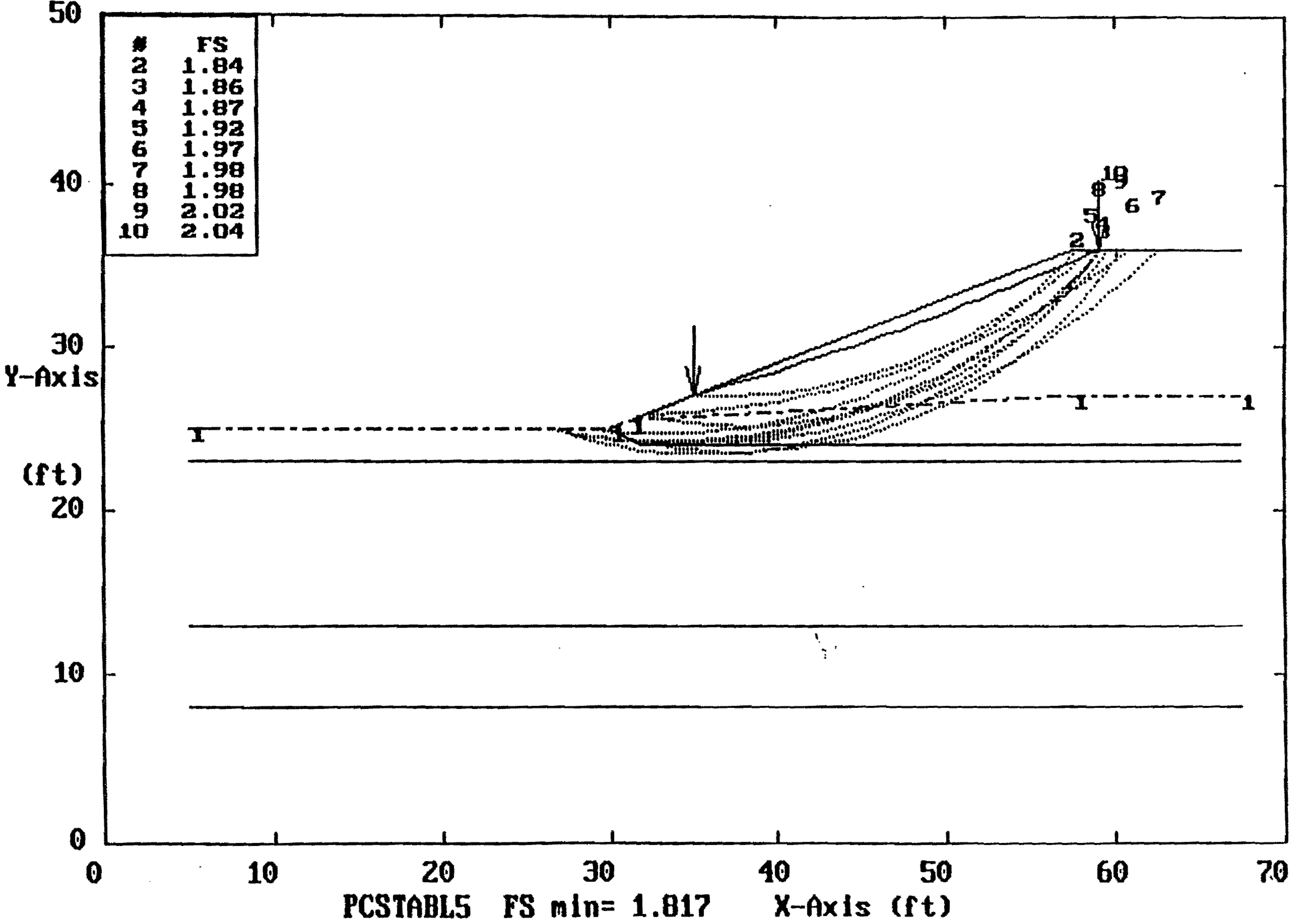
Ten Most Critical. A:PM6D_14.PLT By: EJS 02-04-98 6:38pm



EB-37

PORT MONMOUTH FLOOD CONTROL FEASIBILITY STUDY

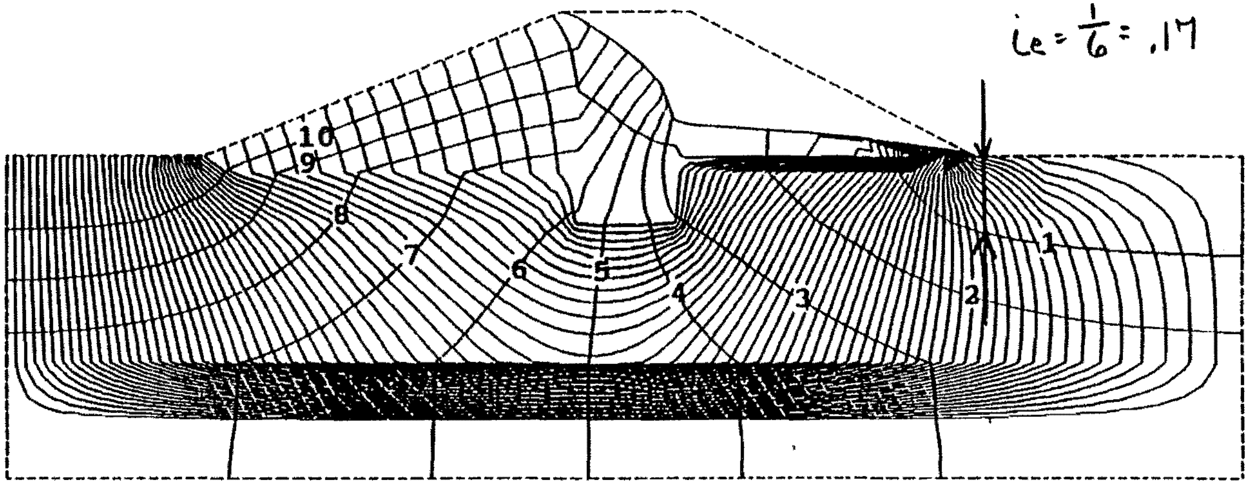
Ten Most Critical. A:PM6_14.PLT By: EJS 02-04-98 8:12pm



EB-38

EB-39

Sta. 12+00 to Sta. 22+00, El. 14 - PM-6
Total Flowrate = 51300.000

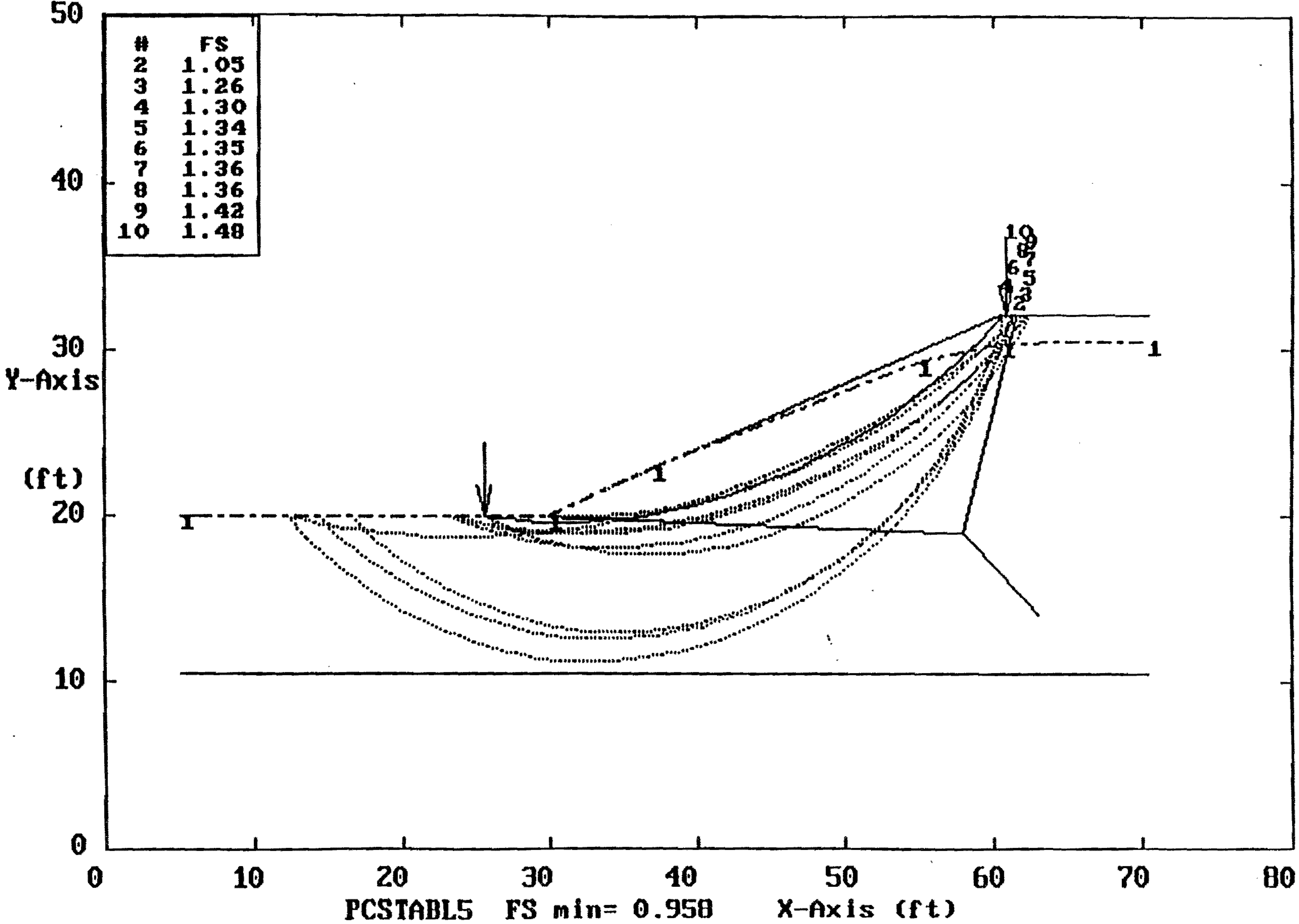


$i_e = \frac{1}{6} = .17$

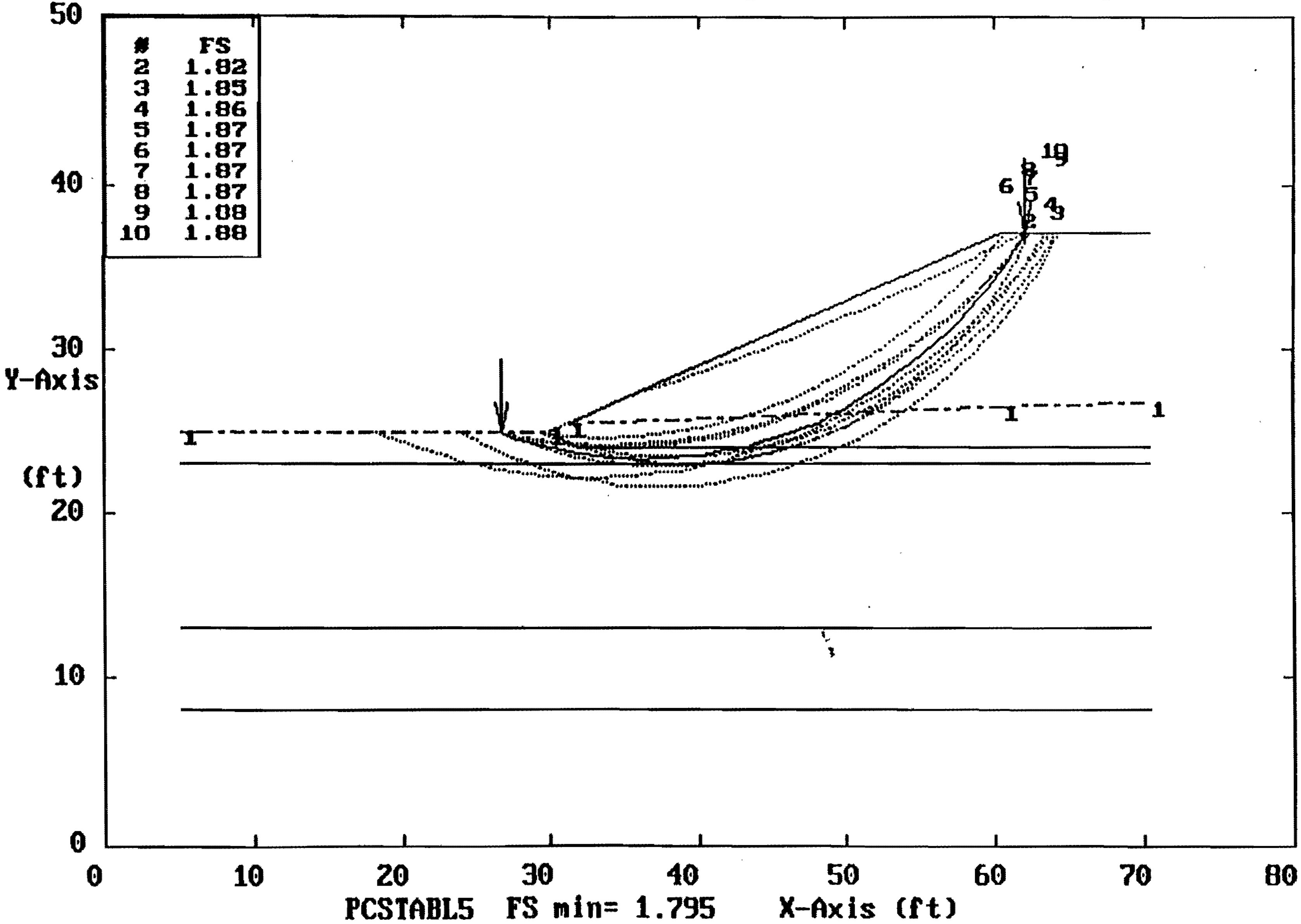
(1 inch = 15.00)

PORT MONMOUTH FLOOD CONTROL FEASIBILITY STUDY
Ten Most Critical. A:PM6D_15.PLT By: JTM 06-03-98 9:48am

EB-40



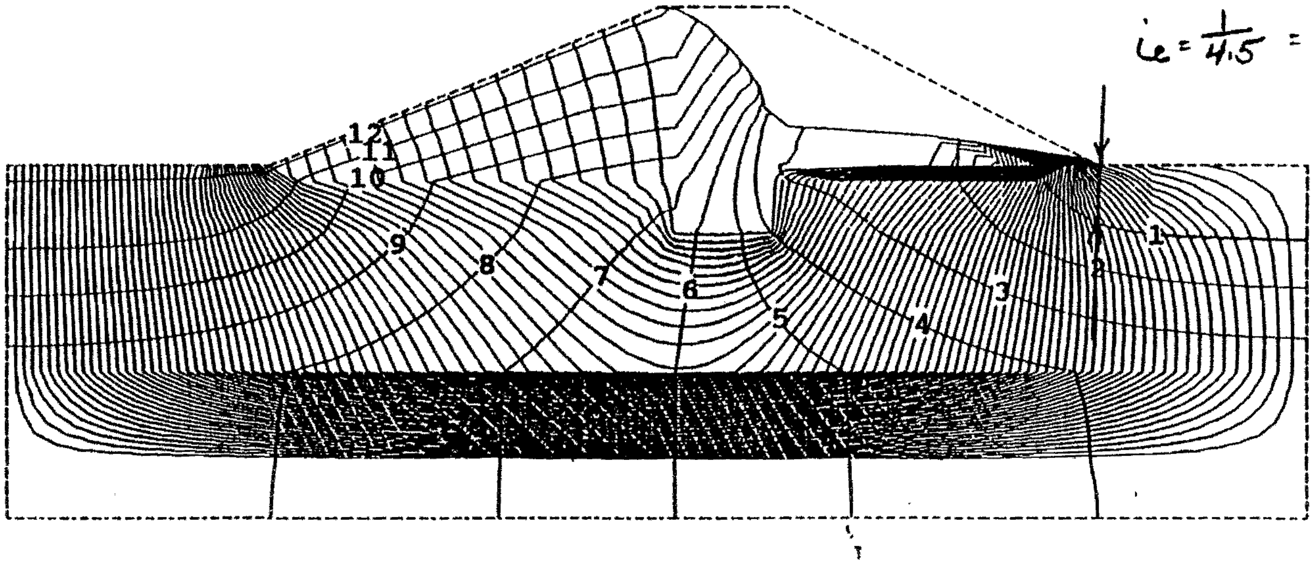
PORT MONMOUTH FLOOD CONTROL FEASIBILITY STUDY
Ten Most Critical. A:PM6_15.PLT By: EJS 02-04-98 8:14pm



EB-41

EB-42

Sta. 12+00 to Sta. 22+00, El. 15.2 PM-6
Total Flowrate = 62900.000



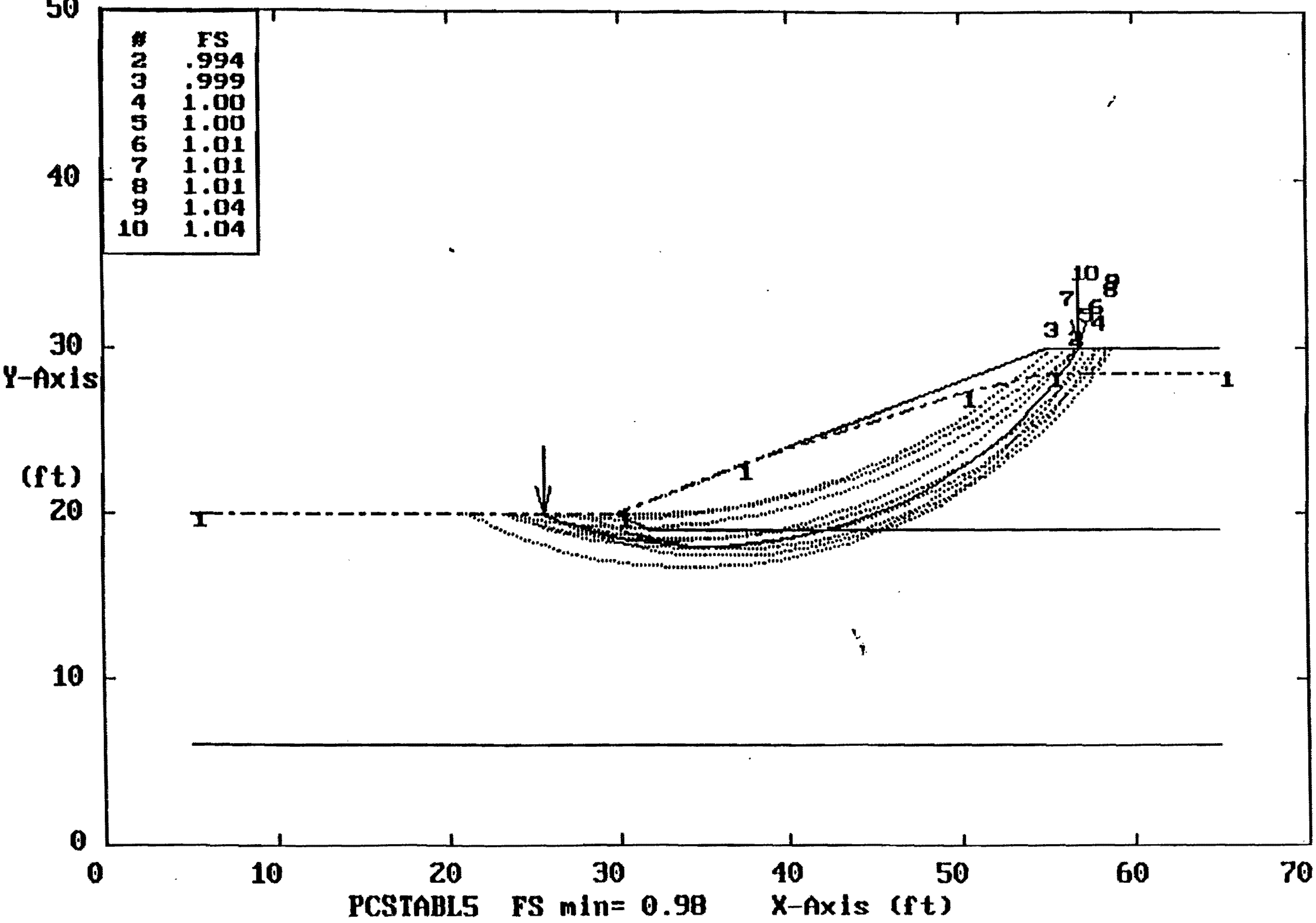
$i_e = \frac{1}{4.5} = .22$

(1 inch = 15.00)

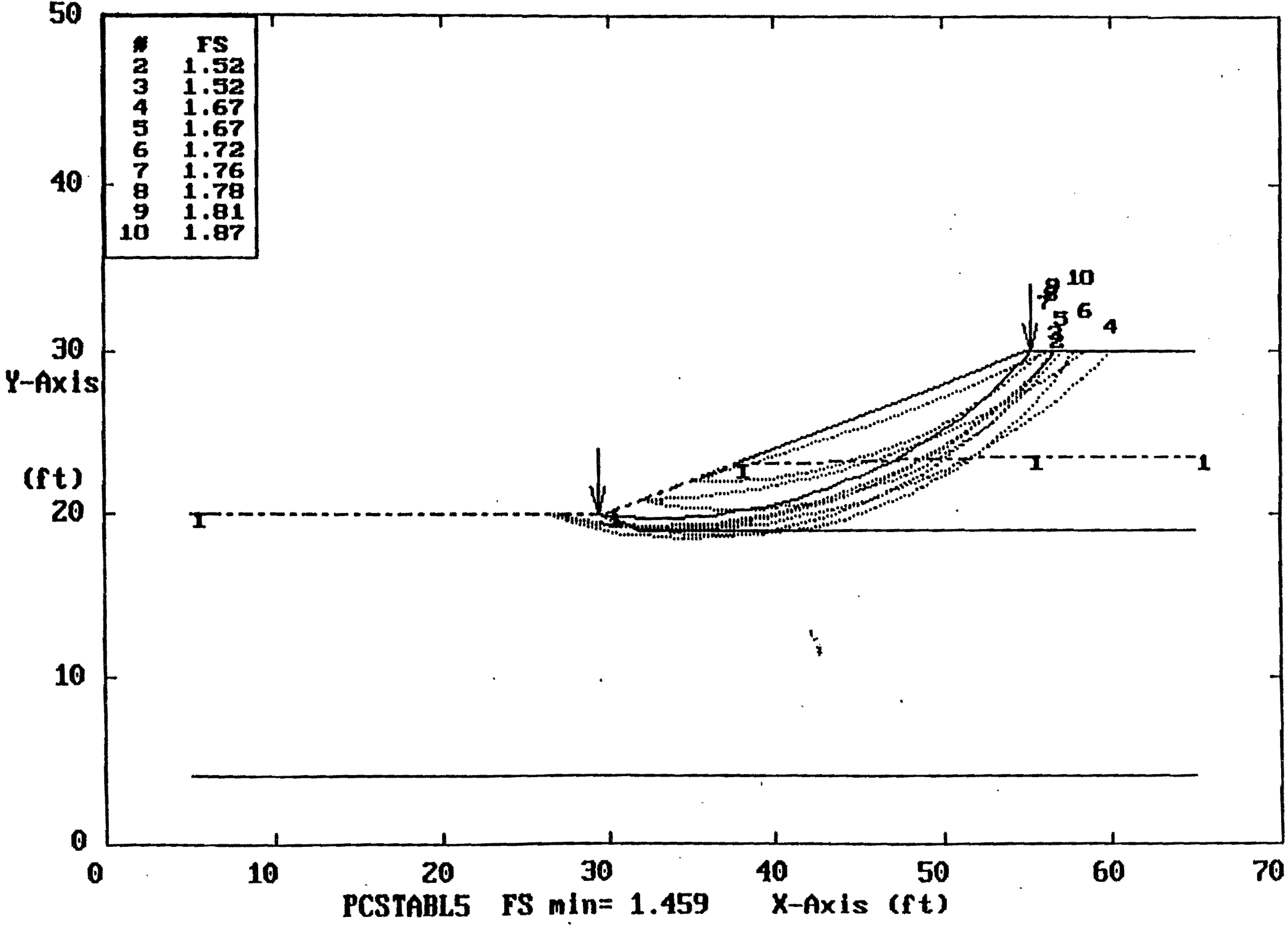
PORT MONMOUTH FLOOD CONTROL FEASIBILITY STUDY

Ten Most Critical. A:PM3D_13.PLT By: EJS 02-04-98 6:53pm

EB-43

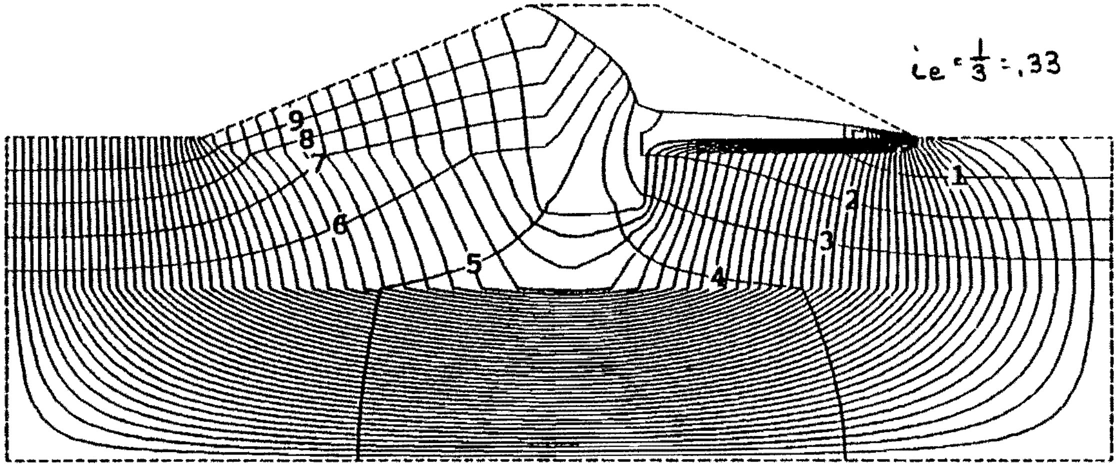


PORT MONMOUTH FLOOD CONTROL FEASIBILITY STUDY
Ten Most Critical. A:PM3_13.PLT By: EJS 02-04-98 8:24pm



EB-45

Sta. 22+00 to Sta. 42+50, El. 13 , PM-3
Total Flowrate = 43890.000

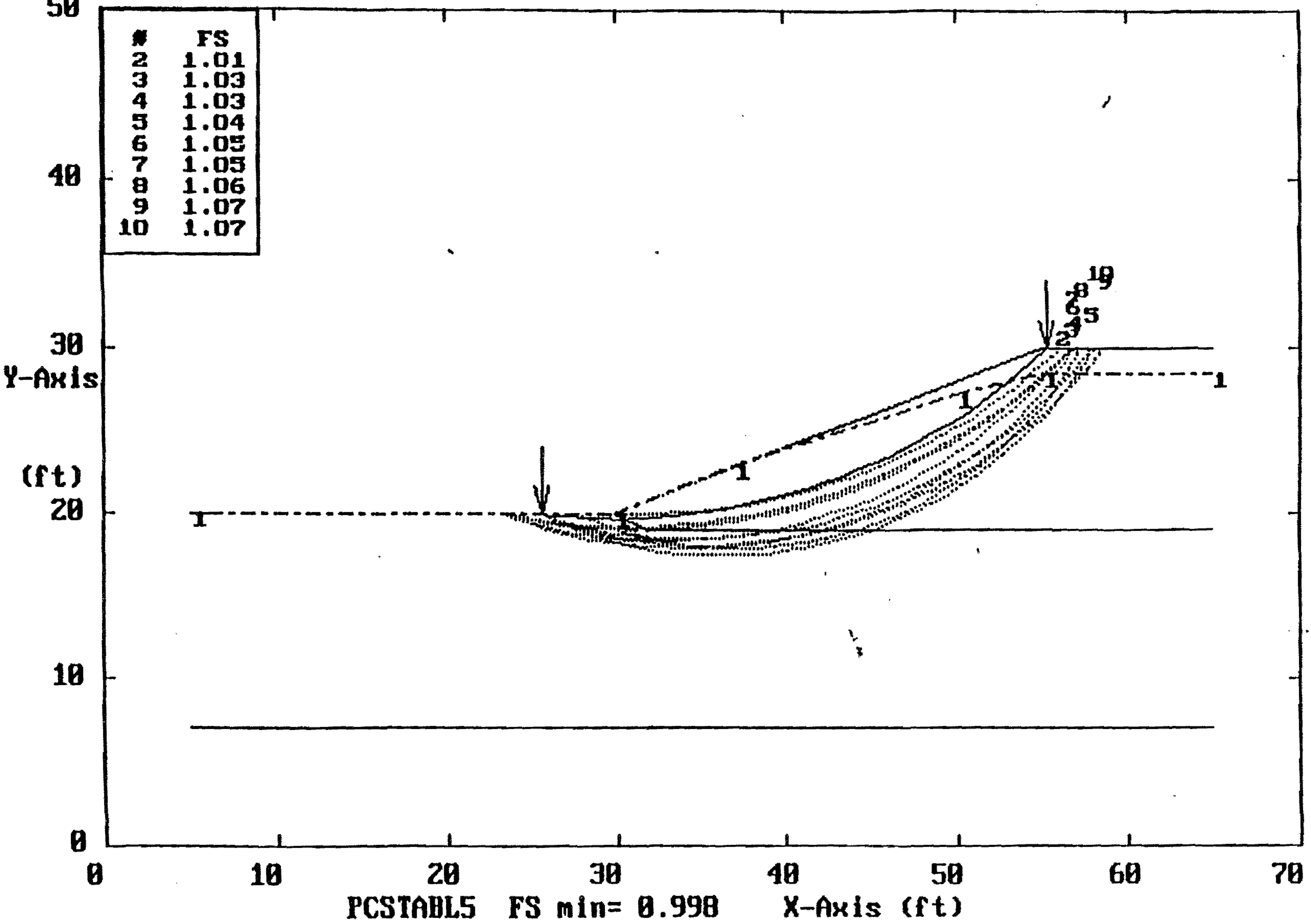


ie = $\frac{1}{3}$ = .33

(1 inch = 15.00)

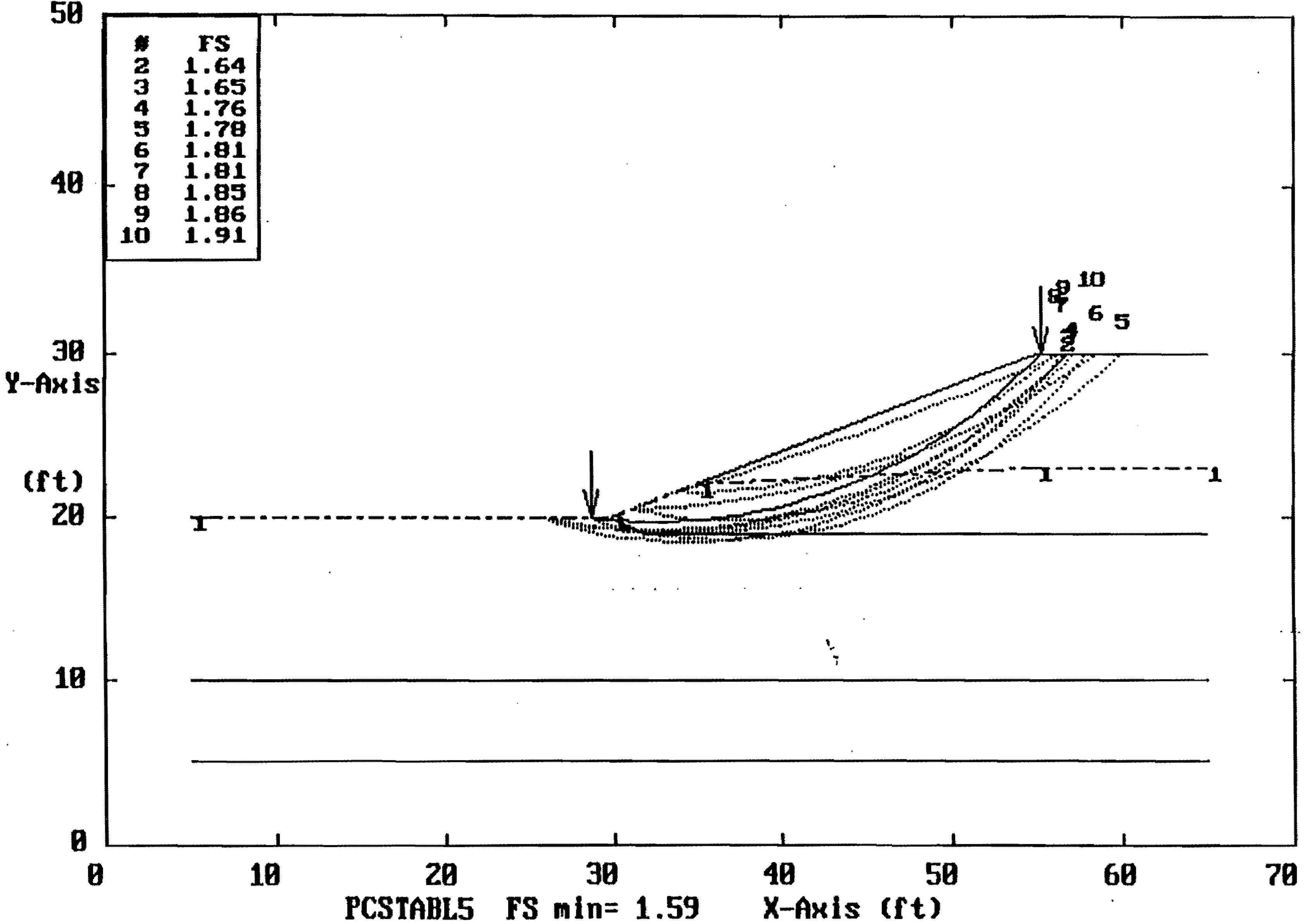
PORT MONMOUTH FLOOD CONTROL FEASIBILITY STUDY

Ten Most Critical. A:PM2D_13.PLT By: EJS 02-04-98 5:57pm

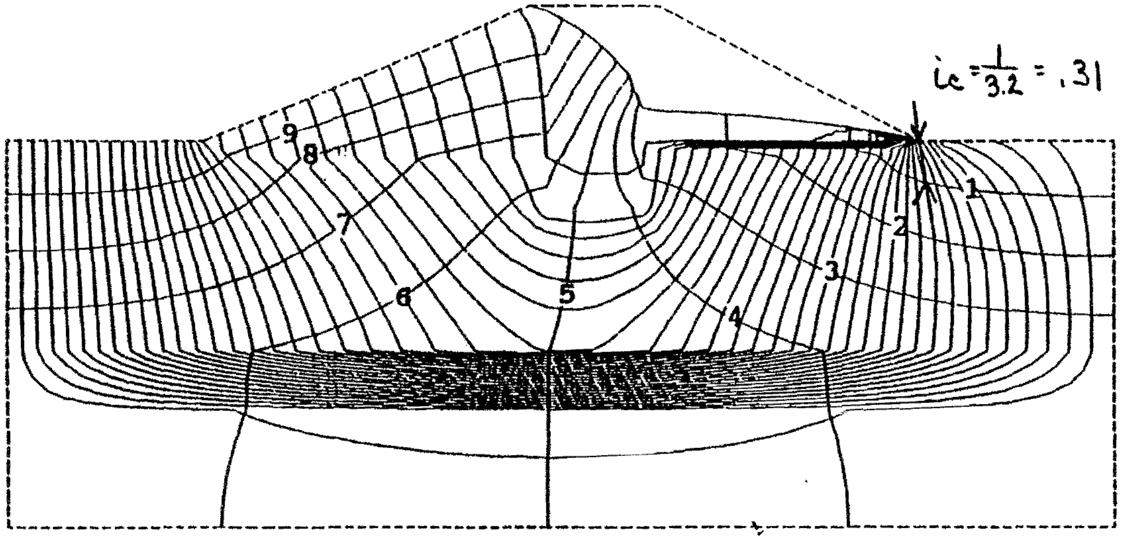


PORT MONMOUTH FLOOD CONTROL FEASIBILITY STUDY

Ten Most Critical. A:PM2_13.PLT By: EJS 02-04-98 5:53pm



Sta. 42+50 to Sta. 48+50, El. 13 , PM-2
Total Flowrate = 30280.000



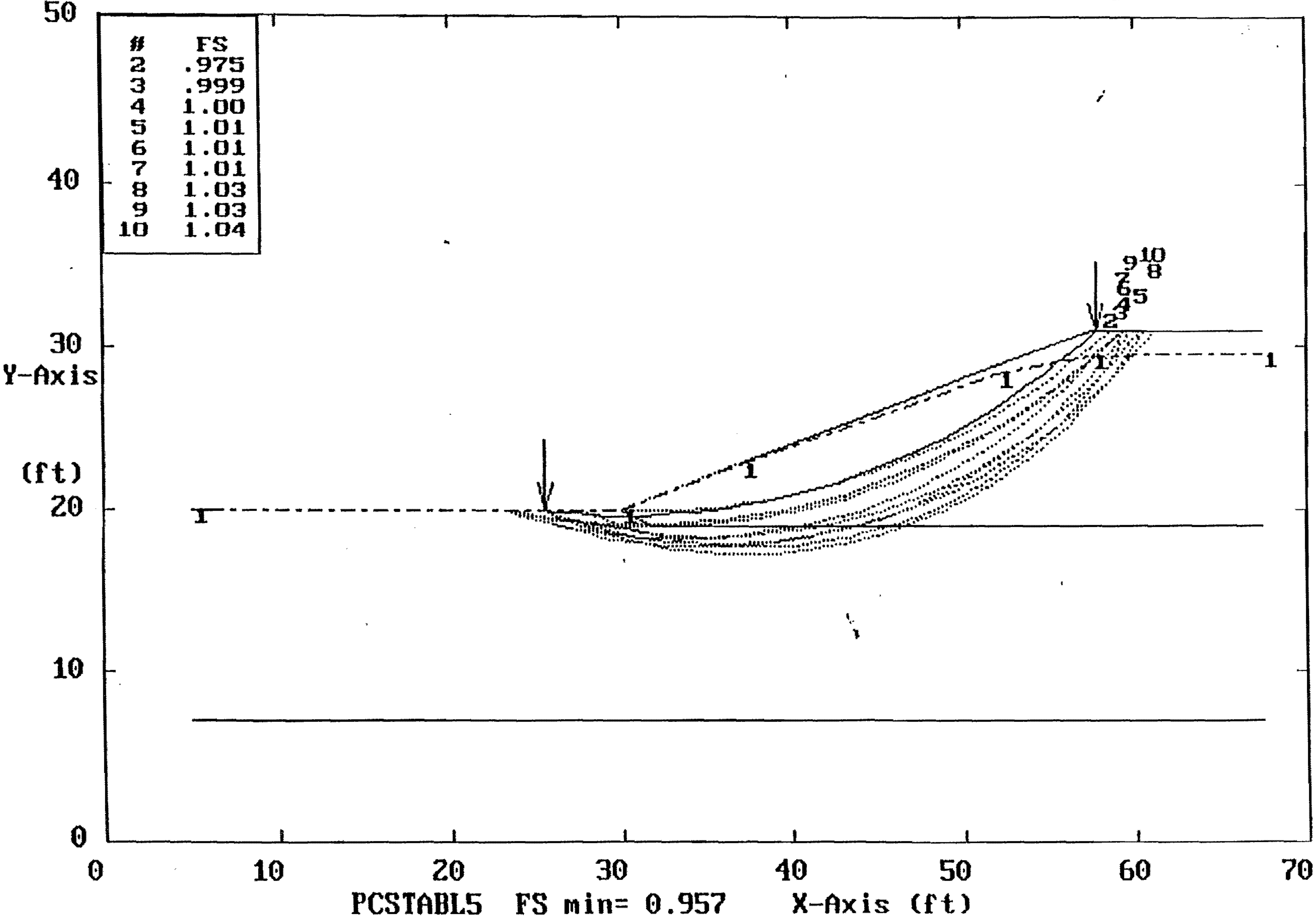
$i_c = \frac{1}{3.2} = .31$

(1 inch = 15.00)

EB-48

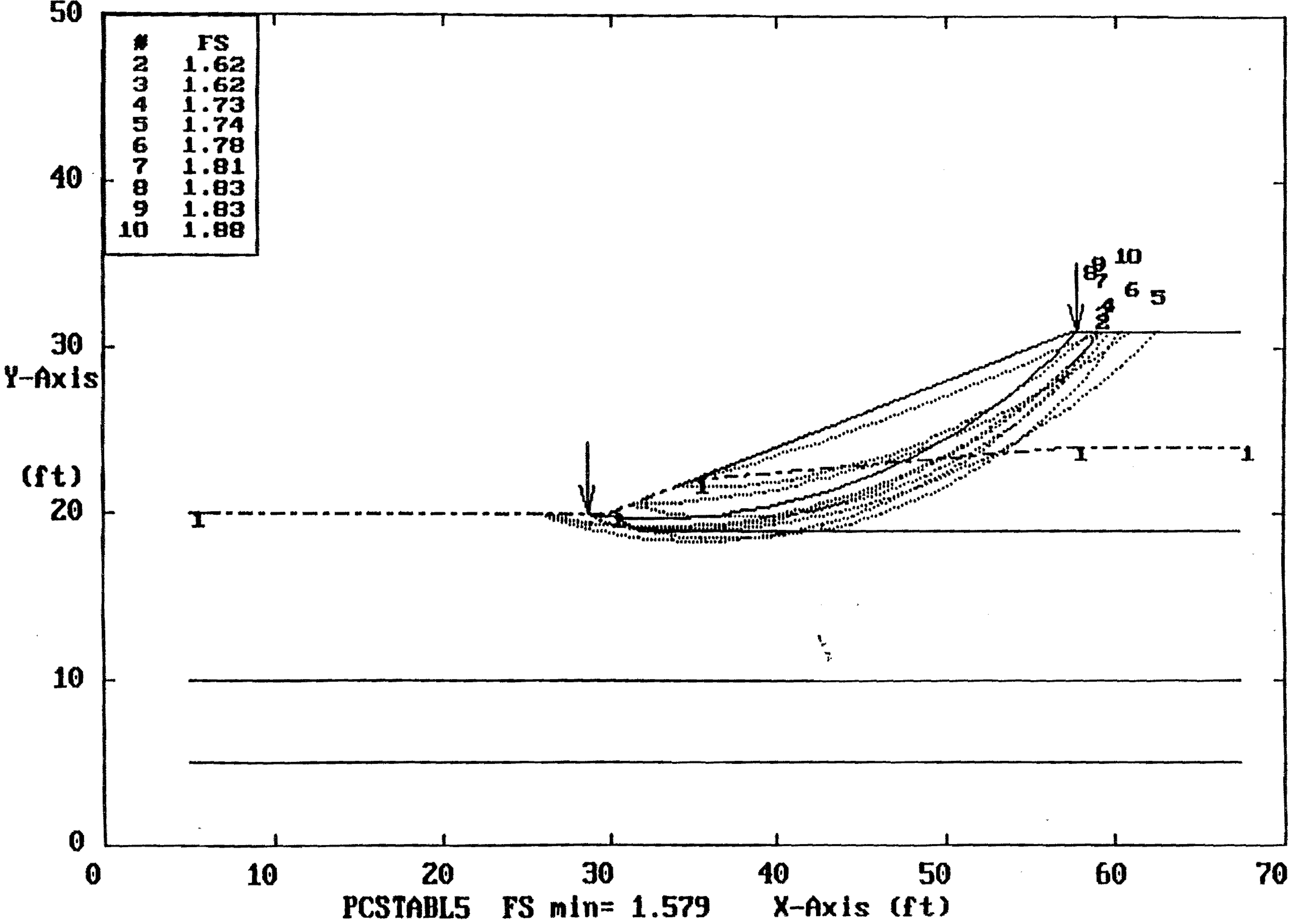
PORT MONMOUTH FLOOD CONTROL FEASIBILITY STUDY

Ten Most Critical. A:PM2D_14.PLT By: EJS 02-04-98 6:12pm



PORT MONMOUTH FLOOD CONTROL FEASIBILITY STUDY

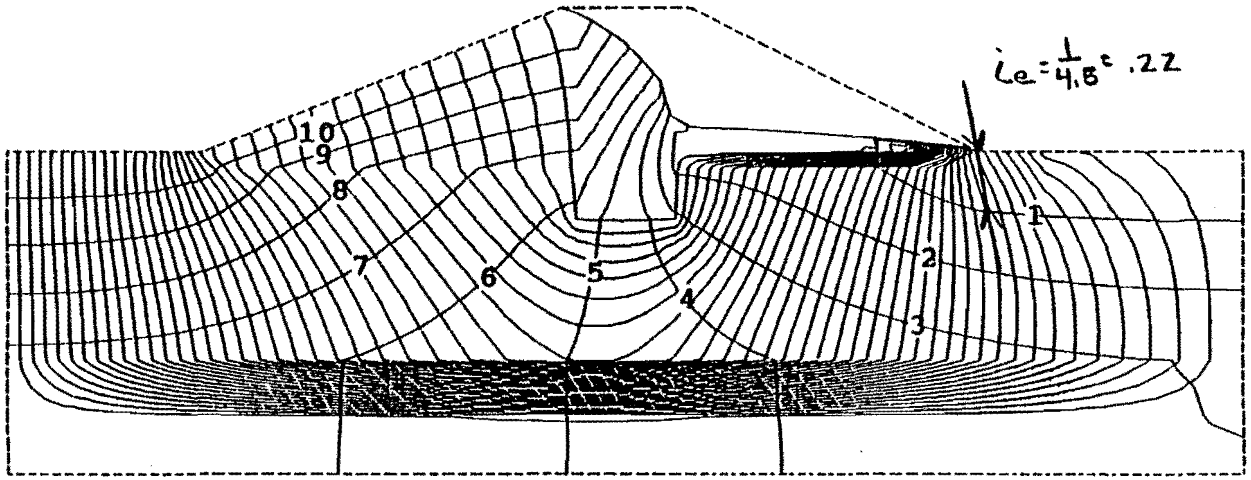
Ten Most Critical. A:PM2_14.PLT By: EJS 02-04-98 8:22pm



EB-50

EB-51

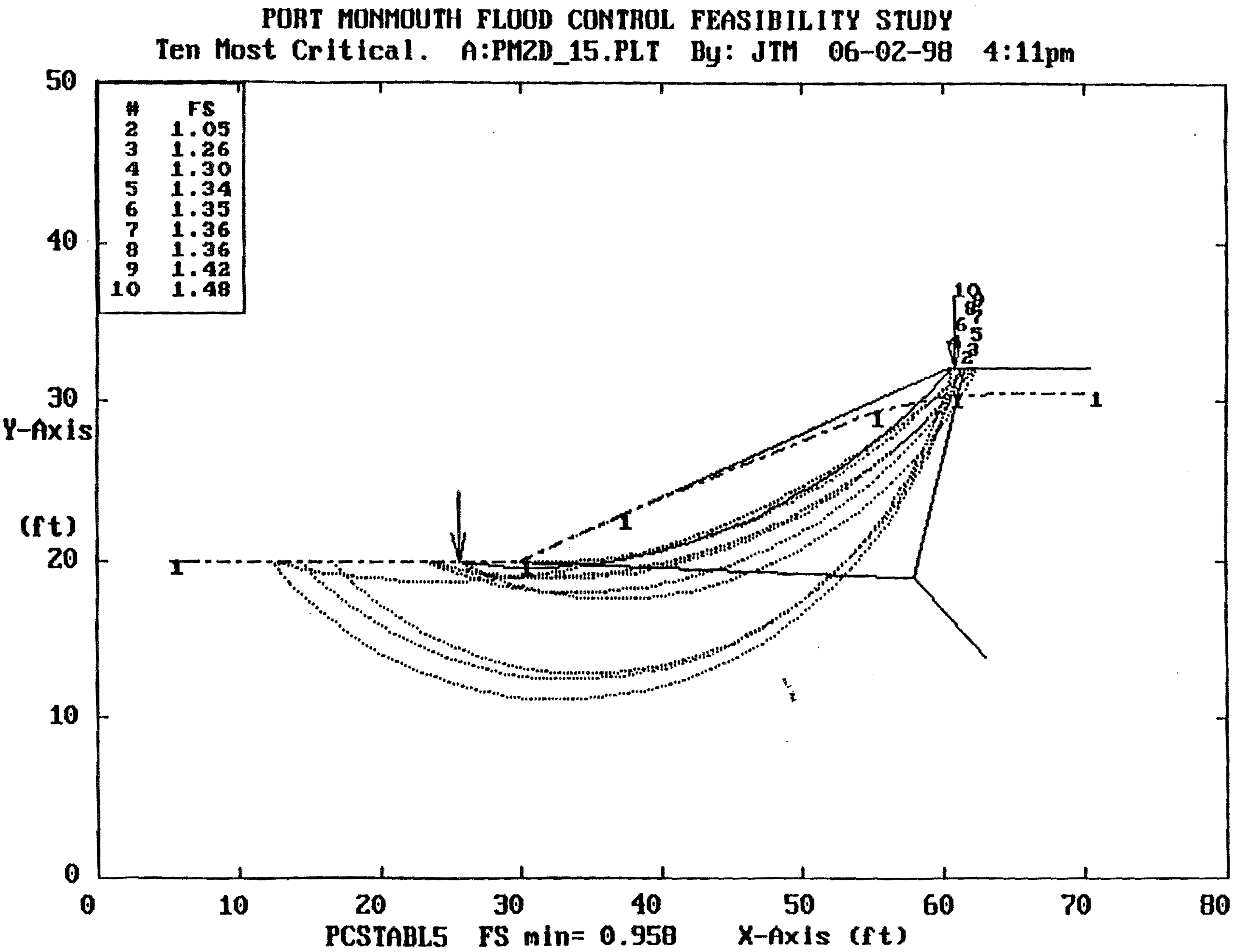
Sta. 42+50 to Sta. 48+50, El. 14 - PM-2
Total Flowrate = 36570.000



$i_e = \frac{1}{4.8} = .22$

(1 inch = 15.00)

EB-5Z



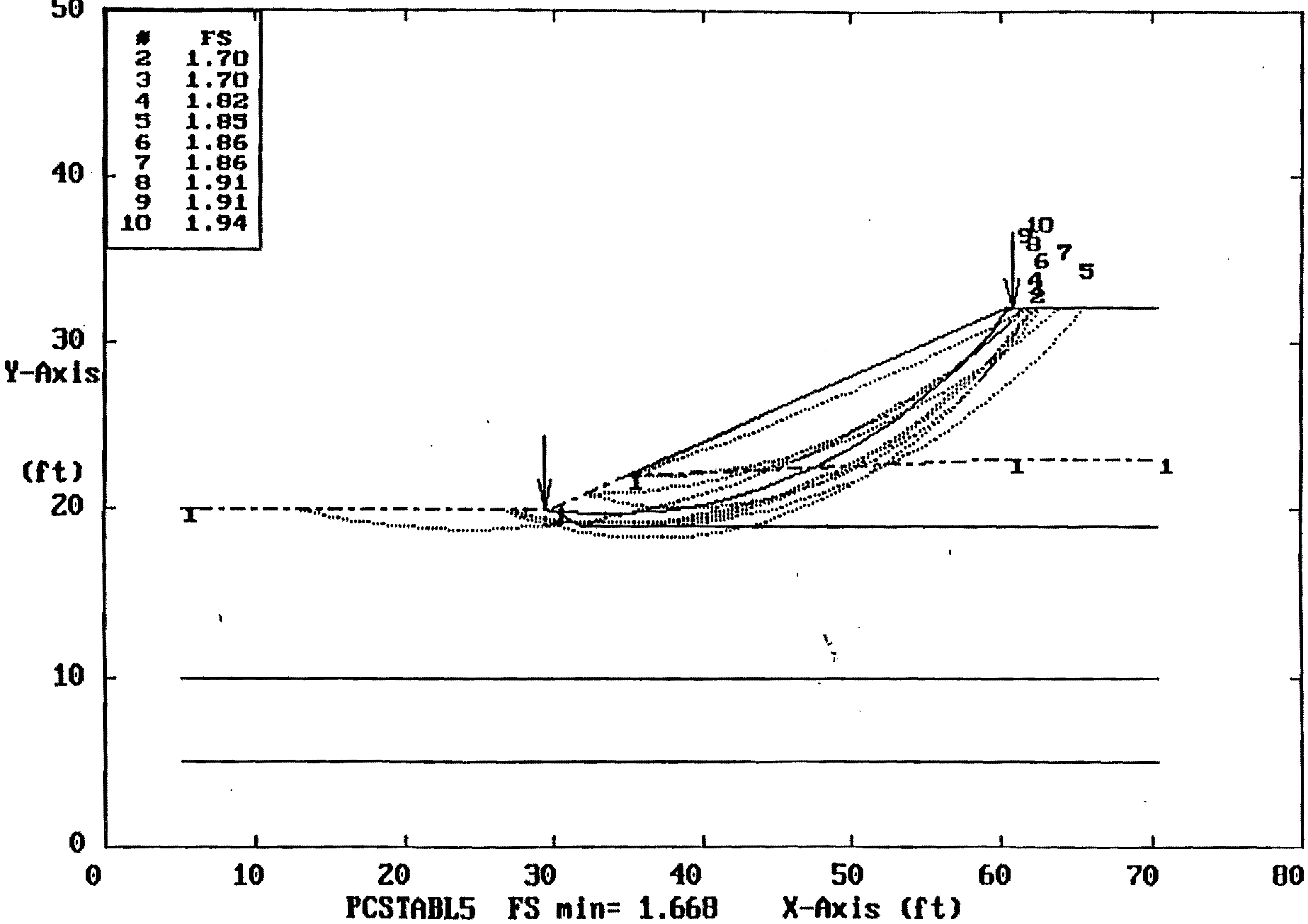
PORT MONMOUTH FLOOD CONTROL FEASIBILITY STUDY

Ten Most Critical. A:PM2D_15.PLT By: JTM 06-02-98 4:11pm

PCSTABL5 FS min= 0.958

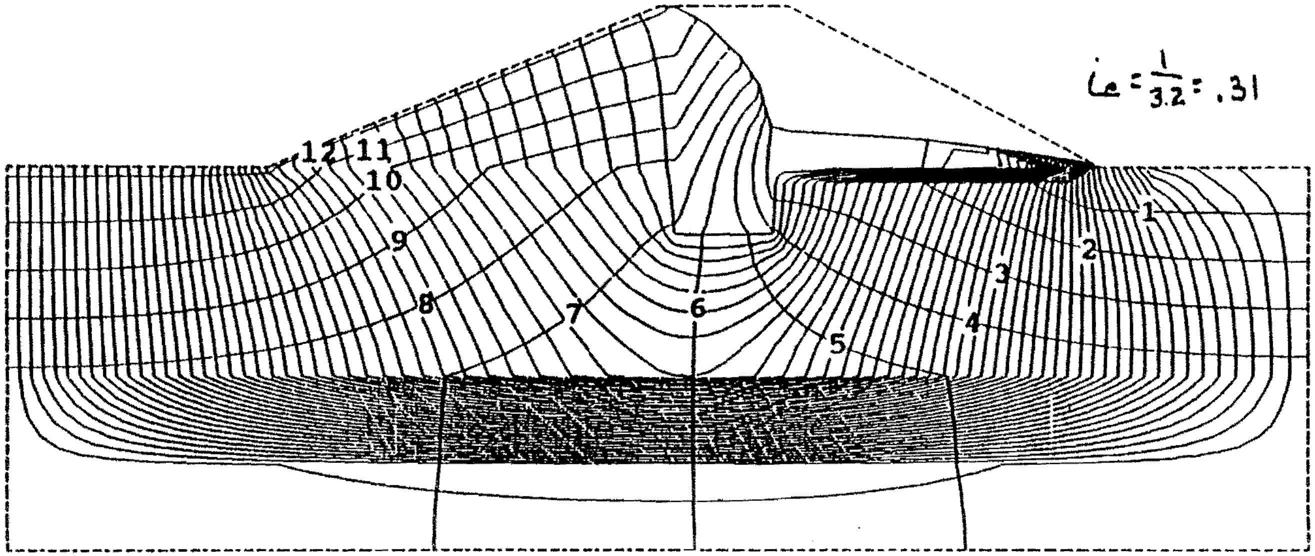
PORT MONMOUTH FLOOD CONTROL FEASIBILITY STUDY
Ten Most Critical. A:PM2_15.PLT By: EJS 02-04-98 8:23pm

EB-53



EB-54

Sta. 42+50 to Sta. 48+50, El. 15.2 PM-2
Total Flowrate = 45630.000

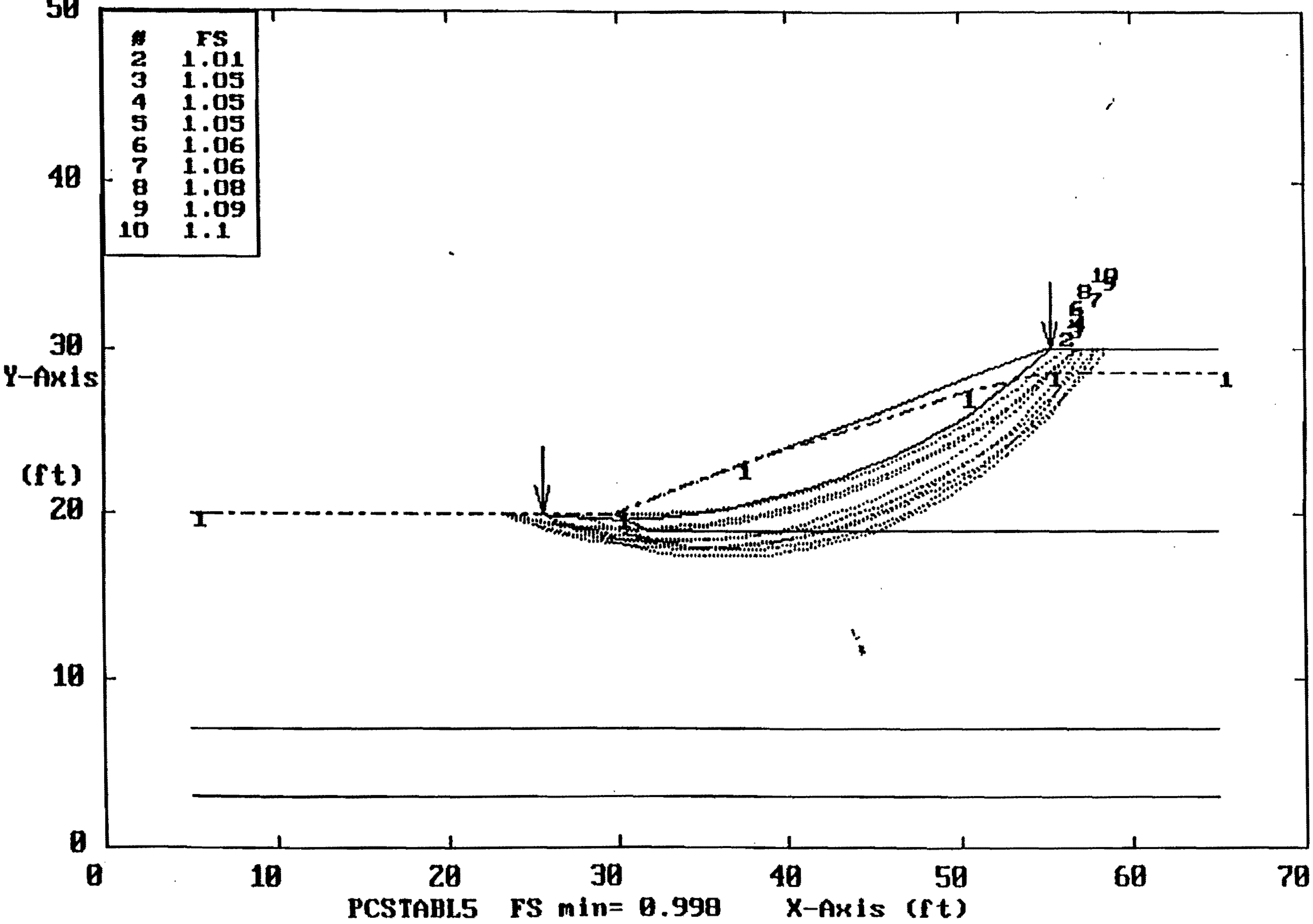


$i_e = \frac{1}{3.2} = .31$

(1 inch = 15.00)

PORT MONMOUTH FLOOD CONTROL FEASIBILITY STUDY

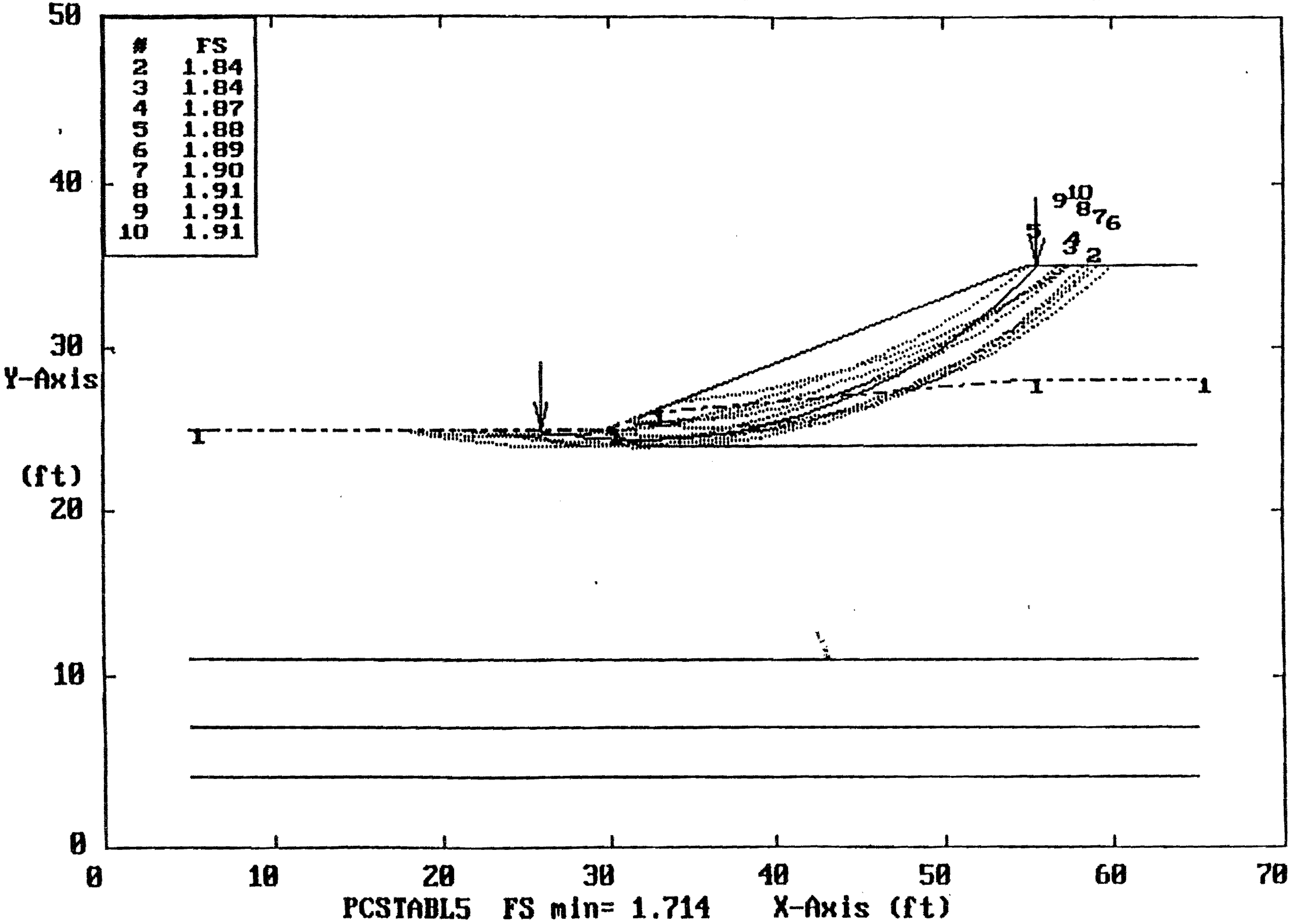
Ten Most Critical. A:PM1D_13.PLT By: EJS 02-04-98 5:47pm



EB-55

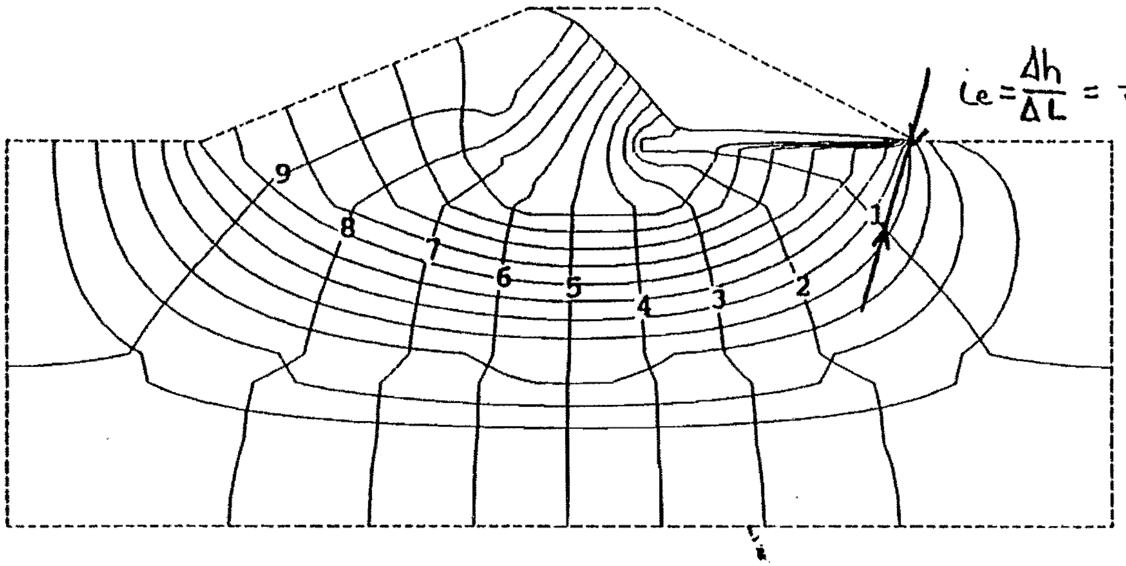
PORT MONMOUTH FLOOD CONTROL FEASIBILITY STUDY

Ten Most Critical. A:PM1_13.PLT By: EJS 02-04-98 5:43pm



EB-56

Sta. 48+50 to Sta. 71+00, El. 13 , PM-1
Total Flowrate = 11570.000



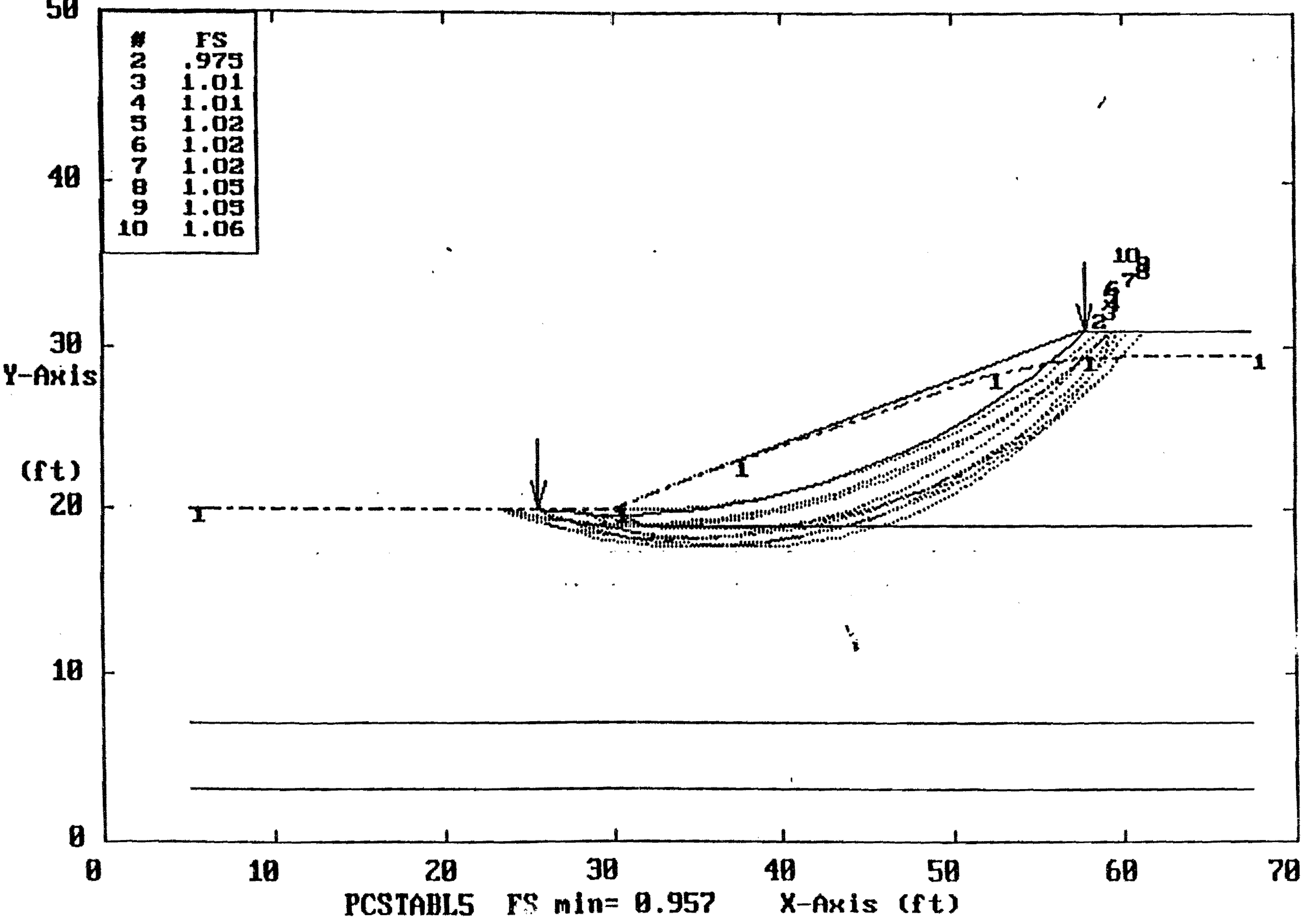
$i_e = \frac{\Delta h}{\Delta L} = \frac{1}{7} = .14$

(1 inch = 15.00)

EB-57

PORT MONMOUTH FLOOD CONTROL FEASIBILITY STUDY

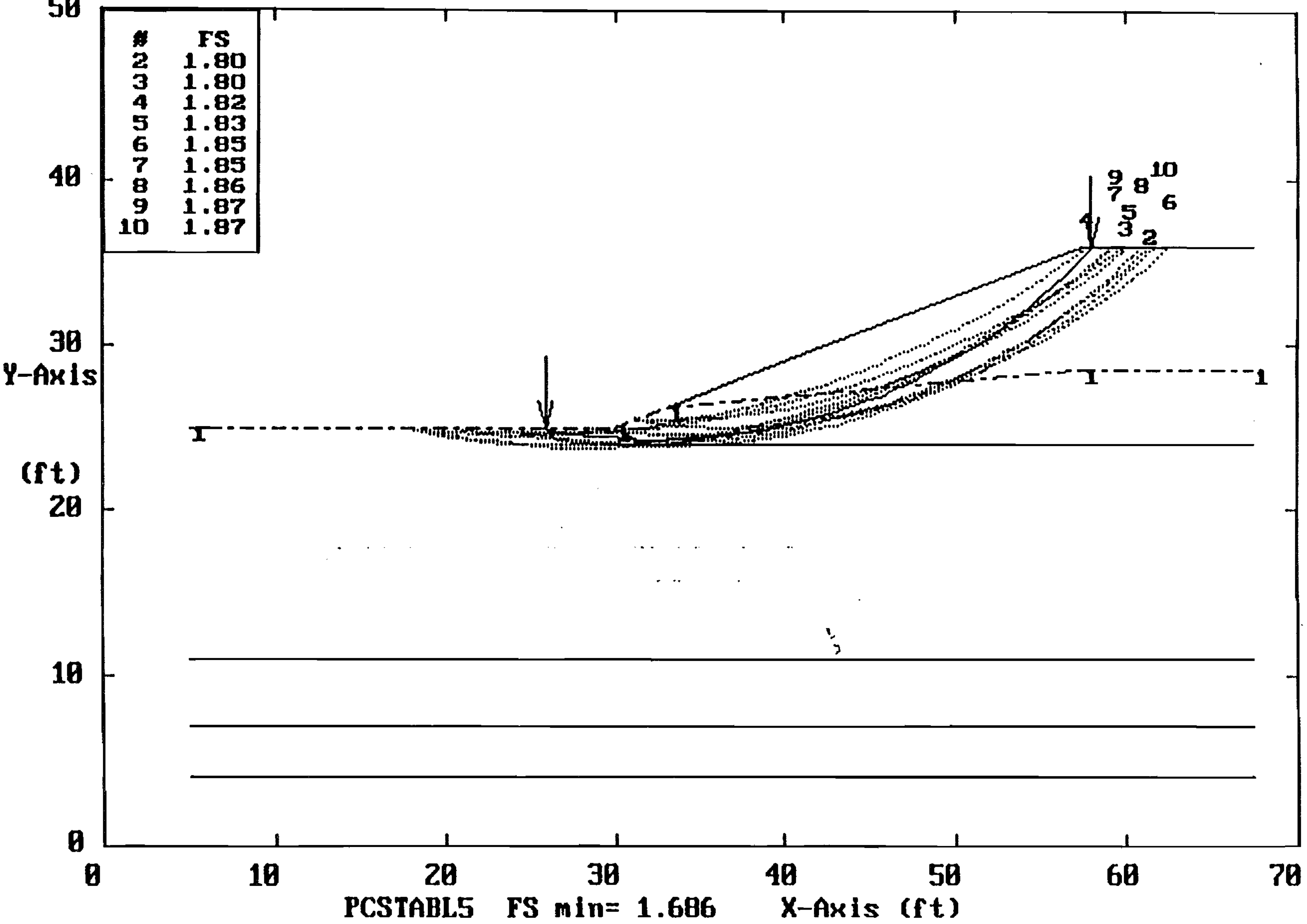
Ten Most Critical. A:PM1D_14.PLT By: EJS 02-04-98 5:22pm



#	FS
2	.975
3	1.01
4	1.01
5	1.02
6	1.02
7	1.02
8	1.05
9	1.05
10	1.06

EB-58

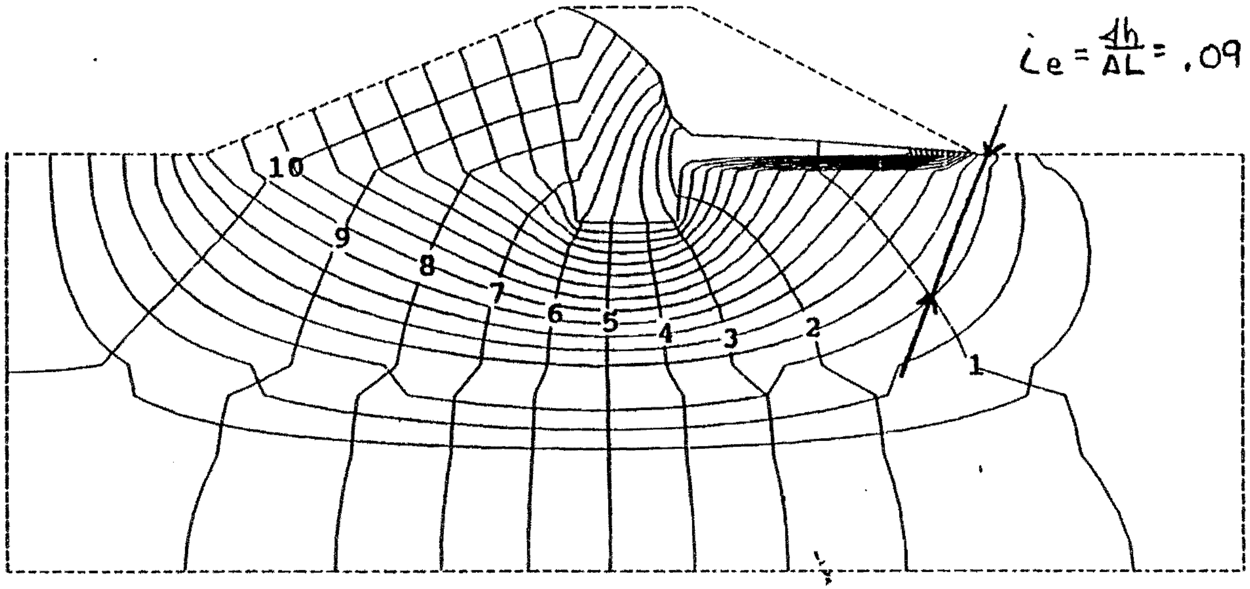
PORT MONMOUTH FLOOD CONTROL FEASIBILITY STUDY
Ten Most Critical. A:PM1_14.PLT By: EJS 02-04-98 5:30pm



EB-59

EB-60

Sta. 48+50 to Sta. 71+00, El. 14 - PM-1
Total Flowrate = 16730.000



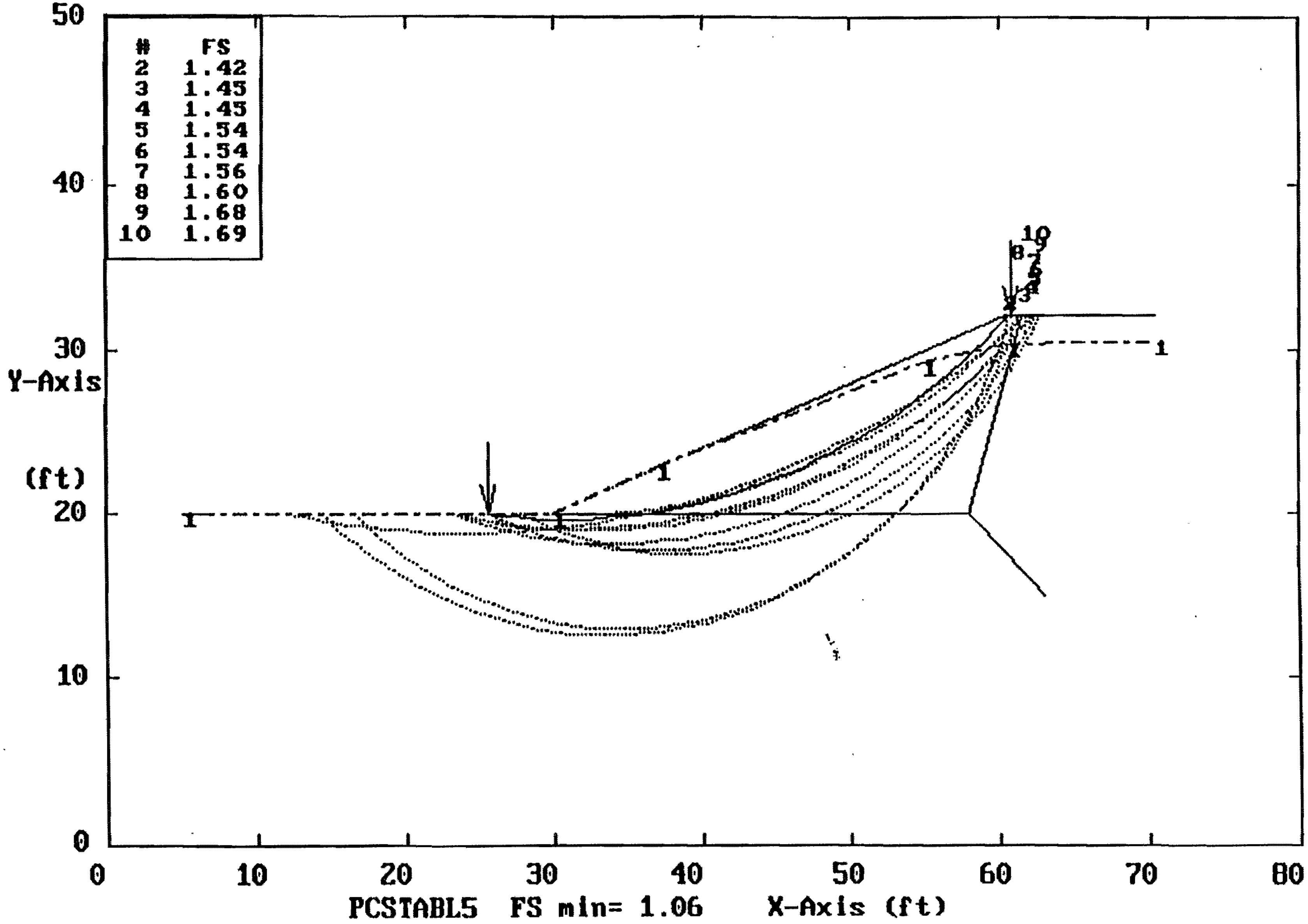
$i_e = \frac{\Delta h}{\Delta L} = .09$

(1 inch = 15.00)

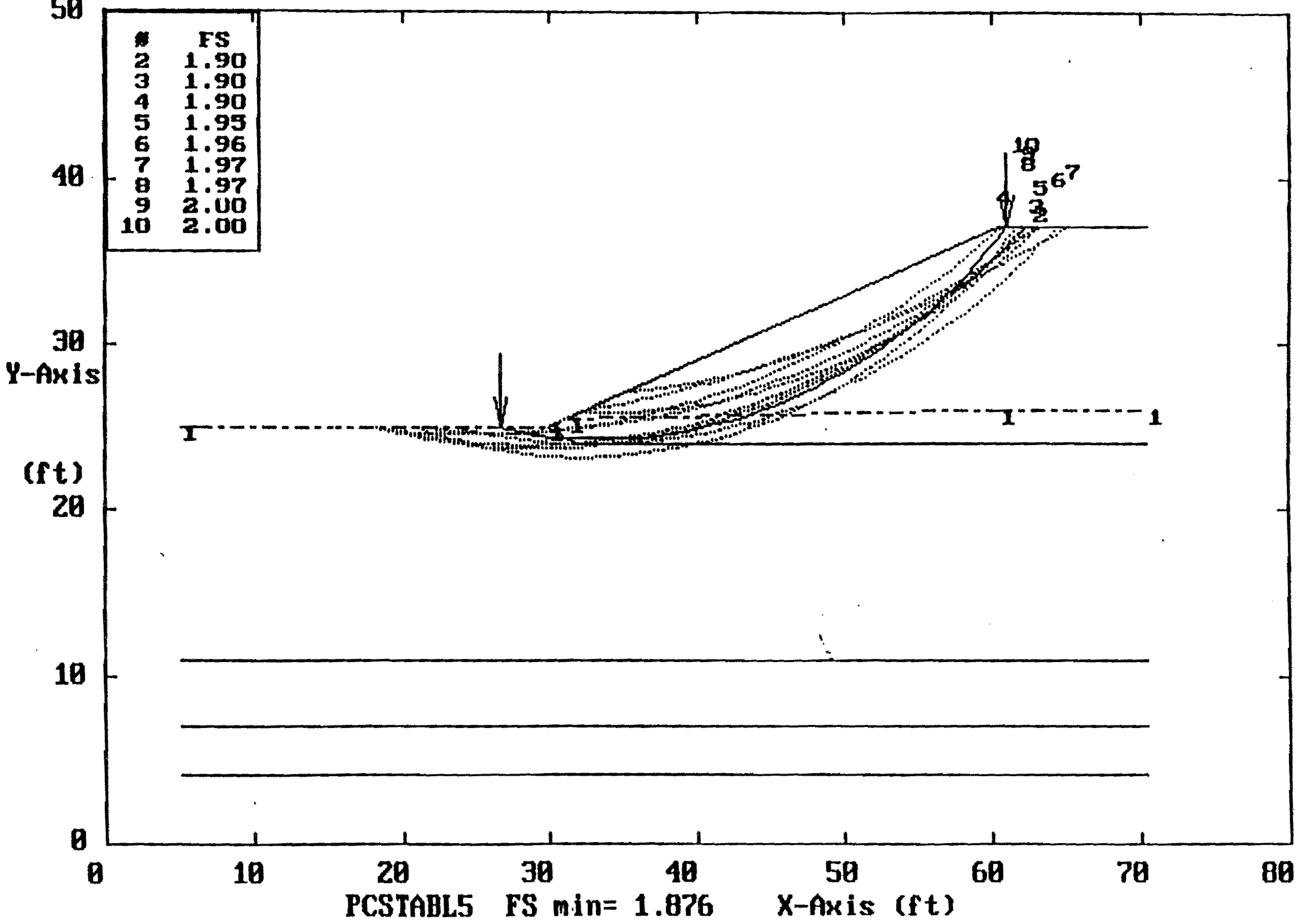
PORT MONMOUTH FLOOD CONTROL FEASIBILITY STUDY

Ten Most Critical. A:PM1D_15.PLT By: JTM 06-02-98 3:41pm

EB-61

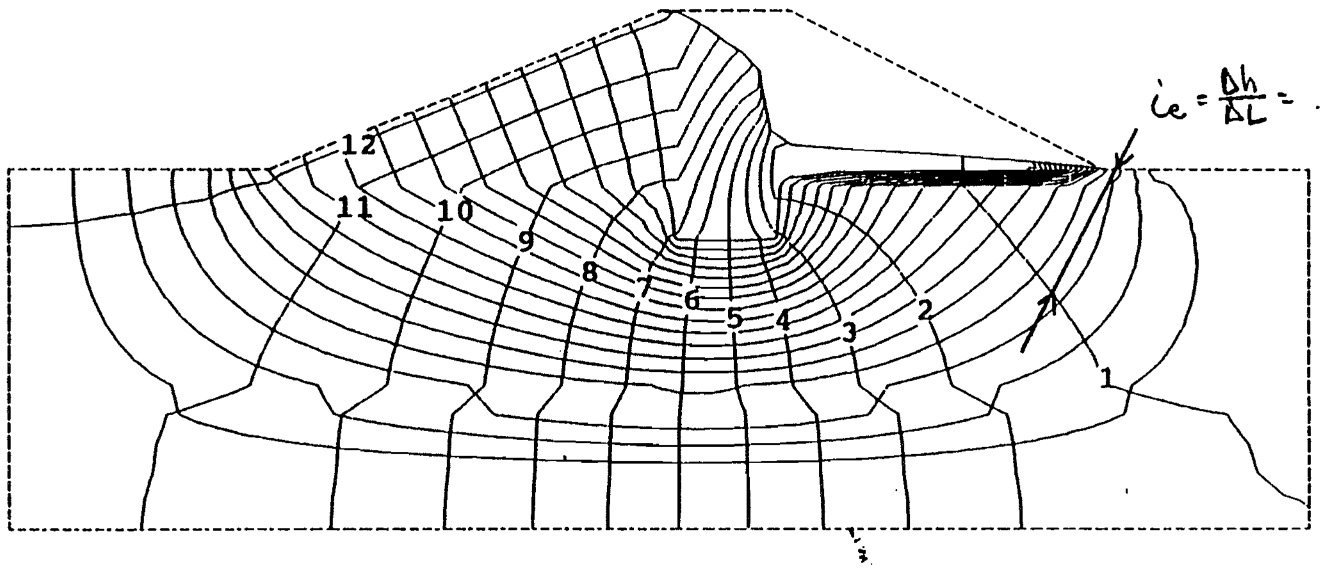


PORT MONMOUTH FLOOD CONTROL FEASIBILITY STUDY
Ten Most Critical. A:PM1_15.PLT By: EJS 02-04-98 5:05pm



EB-63

Sta. 48+50 to Sta. 71+00, El. 15.2 , PM-1
Total Flowrate = 17850.000

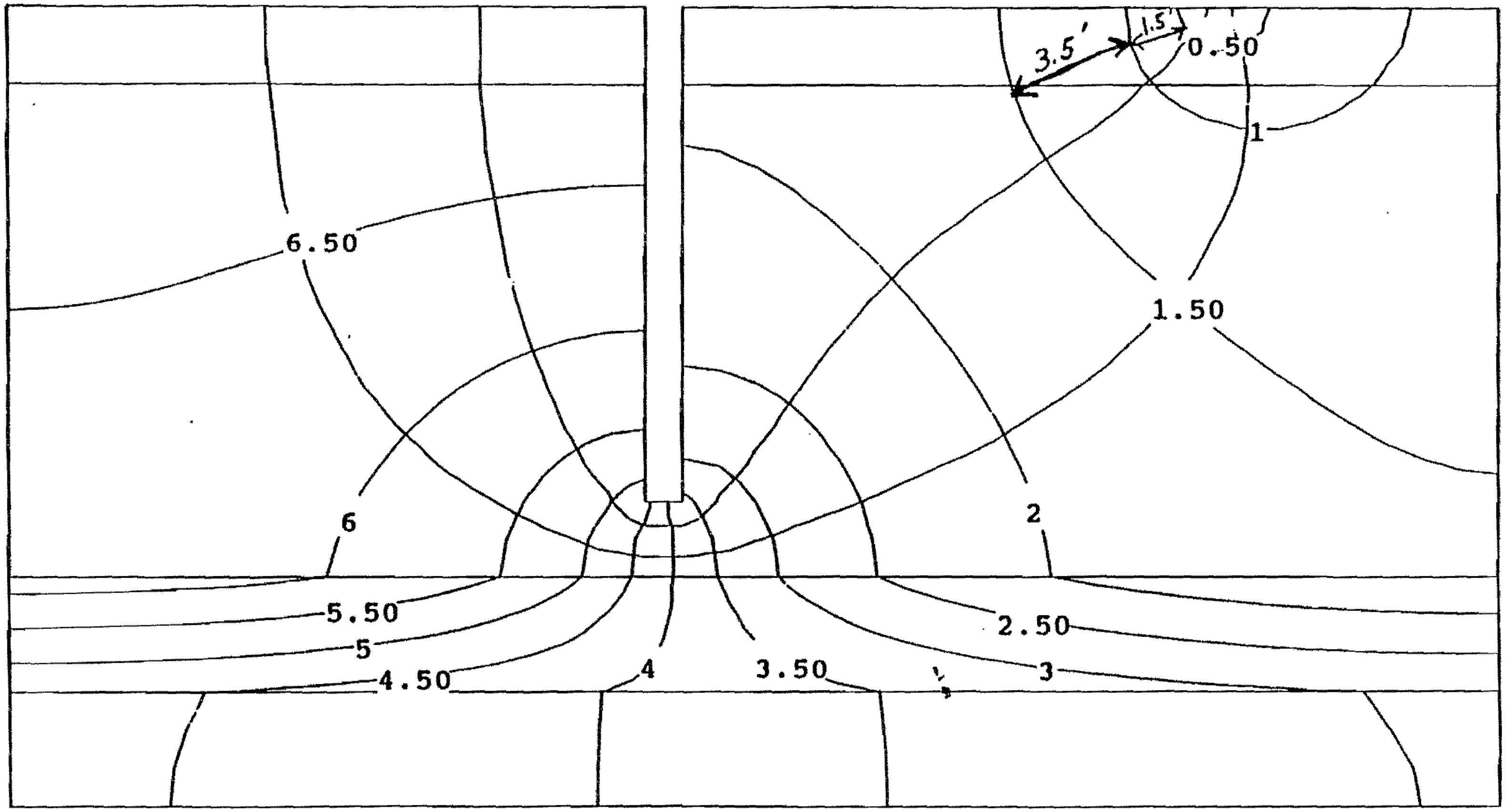


$i_e = \frac{\Delta h}{\Delta L} = .1$

(1 inch = 15.00)

Sta. 71+00 to Route 36, Flood Wall, PM1, El. 13

Total Flowrate = 5809.000



(1 inch = 5.00)

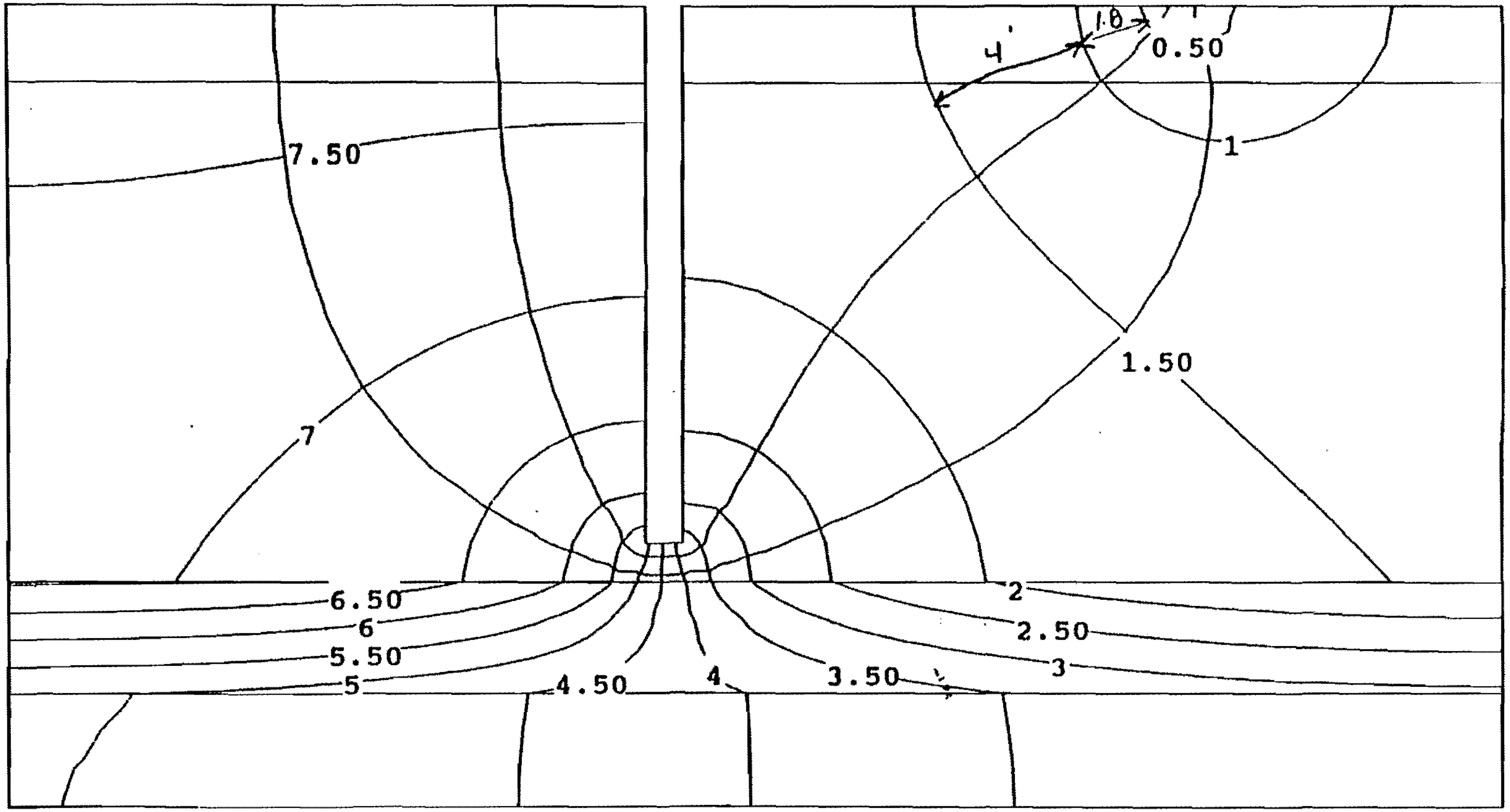
$i_{e_1} : \frac{\Delta h}{L} = \frac{.5}{3.5} = .14$

$i_{e_2} = \frac{.5}{1.5} = .33$

EB-65

Sta. 71+00 to Route 36, Flood Wall, PM1, El. 14

Total Flowrate = 5530.000



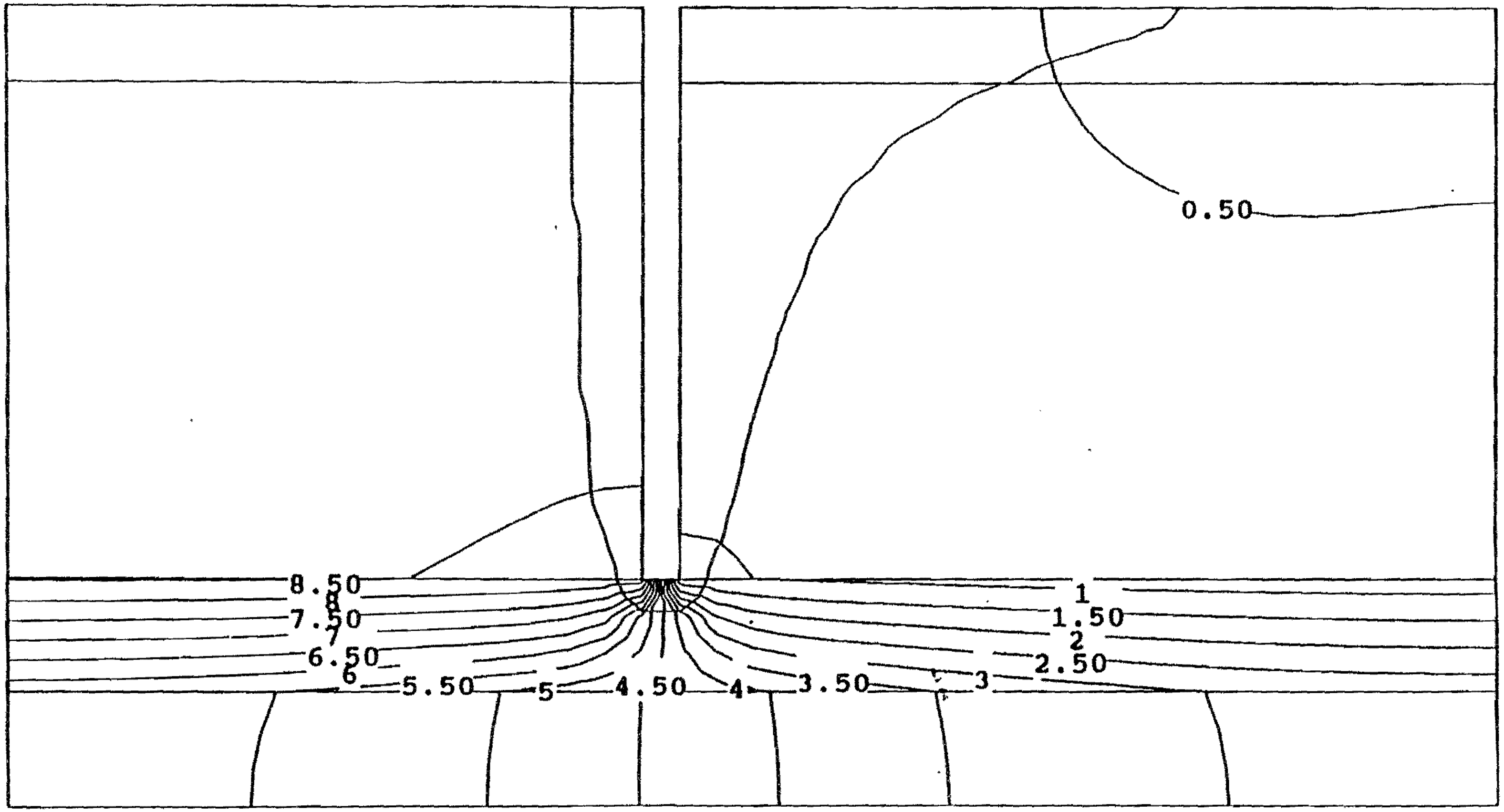
(1 inch = 5.00)

$i_{e_1} = \frac{.5}{4'} = .125$

$i_{e_2} = \frac{.5}{1.8} = .28$

Sta. 71+00 to Route 36, Flood Wall, PM1, El. 15.2

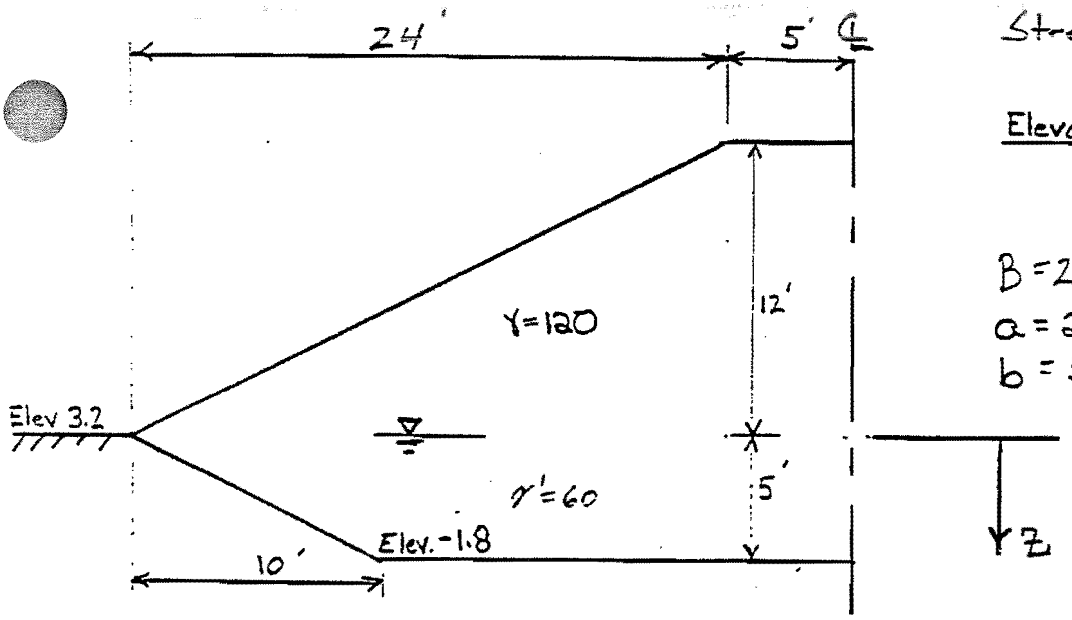
Total Flowrate = 2269.000



(1 inch = 5.00)

iex - negligible

JOB Port Monmouth Levees NOTES BY JTM DATE 12-5-97 PAGE NO. 1
Stress Increase CHECK BY EJS DATE 12-5-97



Stress Increase

Elevation 15.2

B = 2 × 29 = 58
a = 24'
b = 5'

1.) Calculate change in stress from El 3.2 to El -1.8 from excavation & backfill
Existing In-Situ γ' = 50
Fill γ' = 60

60(5) - 50(5) = 50psf = .025 tsf (P_2)

Unit Load of embankment = 12(120) = 1440 psf = .72 tsf (P_1)

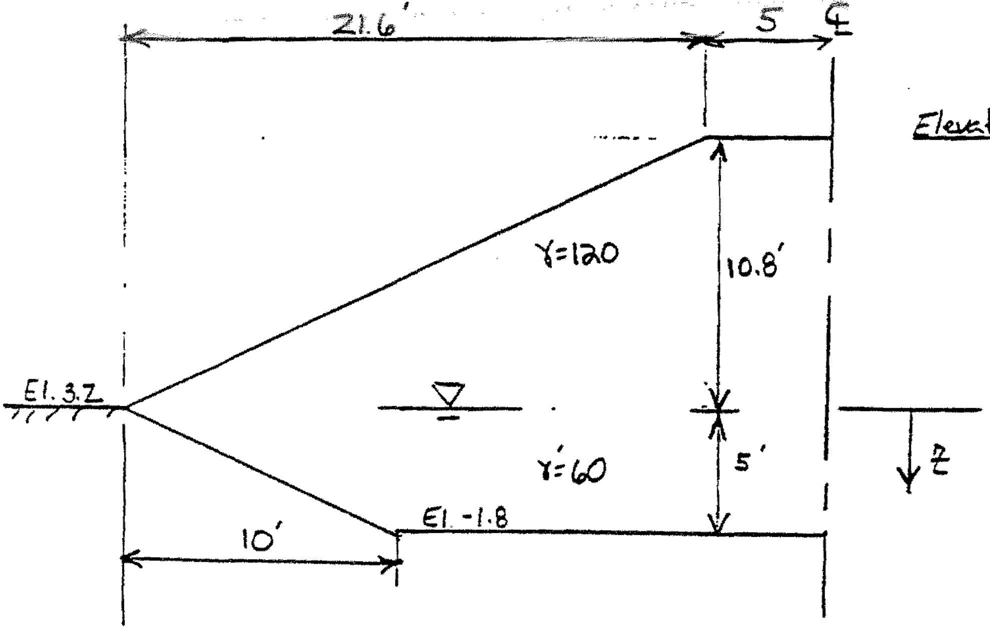
Z (ft.)	$\frac{a}{Z}$	$\frac{b}{Z}$	$\frac{Z'}{B_{strip}}$	I_E	$2I_E$	I_{strip}	$\sigma_Z = 2I_E P_1 + I_{strip} P_2$	$\Delta \sigma_Z$ tsf
5	4.8	1	1	.485	.97	1	.97(.72)+1(.025)	.72 ✓
9	2.7	.56	.07	.455	.91	1	.91(.72)+1(.025)	.68 ✓
13	1.8	.38	.14	.413	.83	1	.83(.72)+1(.025)	.62 ✓
17	1.4	.29	.21	.380	.76	.95	.76(.72)+.95(.025)	.57 ✓
21	1.1	.24	.28	.338	.68	.9	.68(.72)+.9(.025)	.51 ✓
25	.96	.20	.34	.320	.64	.85	.64(.72)+.85(.025)	.48 ✓

See DM7.1 p167, p170
* For Z/B_{STRIP} $Z' = Z - 5$

$\sigma \Delta Z_{w/o}$ 5' Exc. →

$\Delta \sigma_Z$(No Ex.)
.70
.65
.60
.56
.49
.46

EB-67



Stress Increase

Elevation 14.0

$B = 26.6' \times 2 = 53.2'$
$a = 21.6'$
$b = 5'$

1.) See page 1 P_2 (strip) = .025 tsf ✓

2.) $P_1 = 10.8(120) = 1296$ psf = .648 tsf say .65 tsf ✓

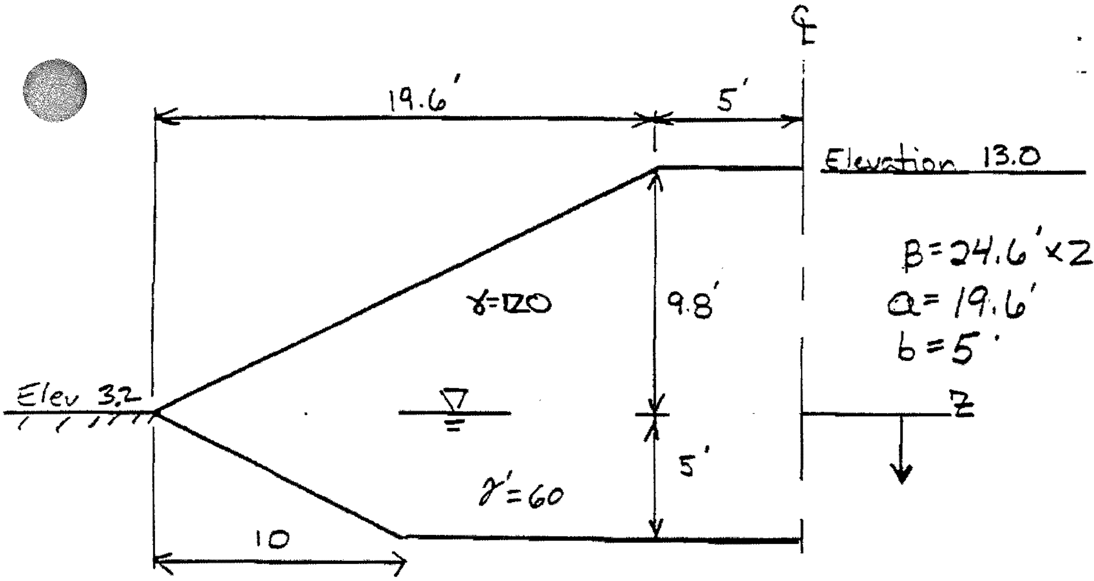
z (ft.)	$\frac{a}{z}$	$\frac{b}{z}$	$\frac{z'}{B_{strip}}$	I_E	$2I_E$	I_{strip}	$\sigma_z = 2I_E P_1 + I_{strip} P_2$	σ_z tsf
5	4.3	1	0	.485	.97	1	.97(.65)+1(.025)	.65 ✓
9	2.4	.56	.075	.450	.90	1 (.95)	.90(.65)+1(.025)	.61 ✓
13	1.7	.38	.15	.410	.82	.90	.83(.65)+ $\overset{.95}{.90}$(.025)	.56 ✓
17	1.3	.29	.22	.355	.71	.92	.71(.65)+.92(.025)	.48 ✓
21	1.0	.24	.30	.338	.68	.88	.68(.65)+.88(.025)	.46 ✓
25	.86	.20	.37	.306	.61	.85	.61(.65)+.85(.025)	.42
	✓	✓	✓	✓	✓			

See DM7.1 p167, p170
* For z'/B_{STRIP} $z' = z - 5$

JOB Port Monmouth Levees NOTES BY JTM DATE 12/5/97 PAGE NO 3

Stress Increase CHECK BY EJS DATE 12-5-97

Stress Increase



B = 24.6' × 2 = 49.2'
a = 19.6'
b = 5'

1.) See page 1. $P_2 = .025$ ✓

2.) $P_1 = 9.8(120) = 1176 = .588$ tsf ⇒ .59 tsf ✓

Z (ft.)	$\frac{a}{Z}$	$\frac{b}{Z}$	$\frac{Z'}{B_{STRIP}}$ *	I_E	$2I_E$	I_{STRIP}	$\sigma_Z = 2I_E P_1 + I_{STRIP} P_2$	σ_Z tsf
5	3.92	1	–	.48	.96	1	.96(.59) + 1(.025)	.59
9	2.18	.56	.08	.445	.89	1	.89(.59) + 1(.025)	.55
13	1.51	.38	.16	.400	.80	.95	.80(.59) + .95(.025)	~~.49~~
17	1.15	.29	.24	.360 ~~.350~~	.72 ~~.70~~	.90	.70(.59) + .9(.025)	.44
21	.93	.24	.32	.335	.67	.85	.67(.59) + .85(.025)	.42
25	.78	.20	.41	.295	.59	.80	.59(.59) + .80(.025)	.37

✓ ✓ ✓

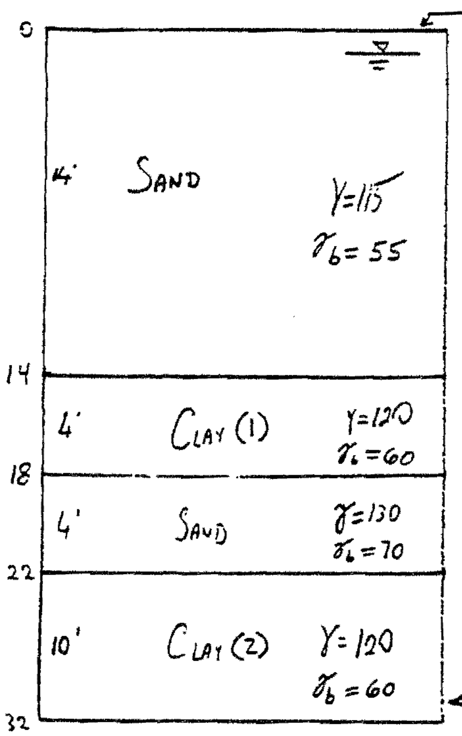
See DM 7.1 p167, 170 ✓

* Z'/Bstrip - Z' = Z - 5'

EB-69

SETTLEMENT

PM 1



start @ Elev. 3.0
GWT @ -1.0

σ_{max} = .83 tsf (max. past preconsol. pressure)
(See BP-3 Consolid)

say settlement of SAND strata complete at end of construction ∴ only settlement due to consolidation of CLAY will occur after construction.

bottom CLAY layer assumed to be 10' thick
boring ended at 3' CLAY layer

Depth Layer	$\bar{\sigma}_o$ mid tsf	$\bar{\sigma}\Delta_z$ tsf	$\bar{\sigma}_f$ tsf	e_o	H_{max} layer
14-18 (1)	0.475	.56	1.035	1.76	4'
22-32 (2)	0.825	.46	1.285	1.75	10'

$$s = C_s \frac{H_o}{1+e_o} \log \frac{\bar{\sigma}_{max}}{\bar{\sigma}_o} + \frac{C_c H_o}{1+e_o} \log \frac{\bar{\sigma}_f}{\bar{\sigma}_{max}}$$

	H	$\frac{H}{1+e_o}$	$\frac{\bar{\sigma}_{MAX}}{\bar{\sigma}_o}$	C_s	C_c	$\frac{\sigma_f}{\sigma_{max}}$	S_s	S_c
Layer 1	4	1.44	1.75	.029	.764	1.26	.010'	.11
Layer 2	10	3.64	1.0	.029	.764	1.57	0	.55'

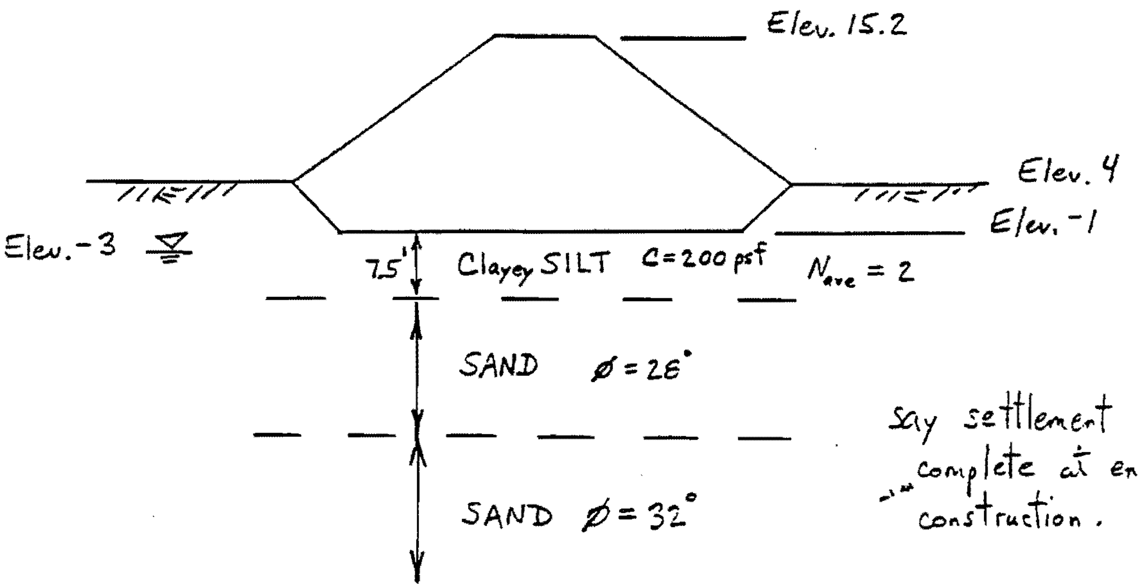
7.9"

$$T_{90} = \frac{c_v t_{90}}{H^2_{dr.}} \qquad C_v = .0011\ ^{in^2/min}\ (PM\text{-}2A) \times 60\ ^{min/hr} \times 24\ ^{hr/d} = 1.584\ in^2/d$$

$$t_{90} = \frac{T_{90} H^2}{C_v} = \frac{.848(2\times12)^2}{1.584} = 308 \text{ days or } 10 \text{ months}$$

JOB #1968-01 Fort Monmouth NOTES BY EJS DATE 1-14-98 PAGE NO. 5
CHECK BY 4r DATE 1-19-98

Settlement @ BP-3 Levee



say settlement of SAND complete at end of levee construction.

$\sigma_{max\ precons.} = .83\ tsf\ (P_c)$

$e_o = 1.77$, only 1 layer, midpt. @ 3.75' below levee. (z = 8.75')

using BP=3 consol. data (conservative) : $C_s = 0.029$

$(P_o)\ \bar{\sigma}_o$ @ midpt. $= (7' \cdot 115\ pcf) + (1.75' \cdot 55\ pcf) = 900\ psf = 0.45\ tsf$

$(\Delta P)\ \Delta\sigma = 0.69\ tsf$ (see Pg. 1 of Stress Increase calc's)

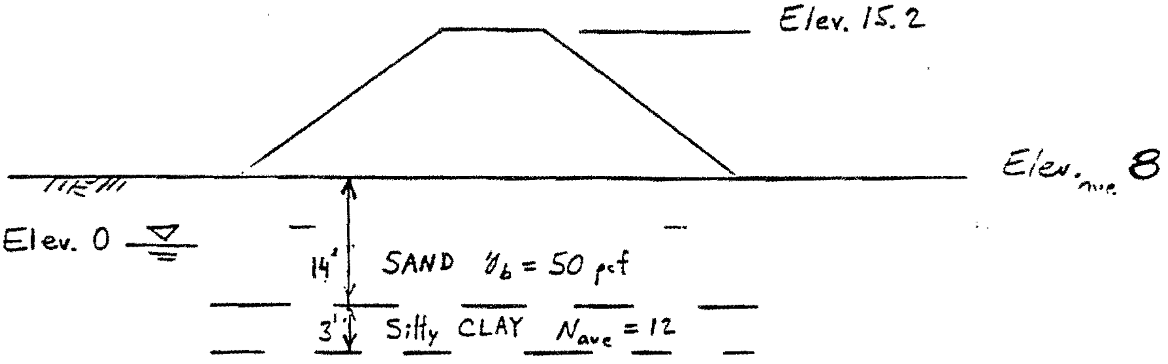
$$S_c = \frac{C_s H_c}{1+e_o} \log \frac{P_c}{P_o} + \frac{C_c H_c}{1+e_o} \log \frac{P_o + \Delta P_{AV}}{P_c} \ldots$$

	H	$\frac{H}{1+e_o}$	$\frac{P_c}{P_o}$	C_s	C_c	$\frac{P_o+\Delta P}{P_c}$	S_s	S_c
Layer 1	7.5	2.7	1.84	.029	.764	1.37	.02	.28'

Settlement - 3.6", Say 4"

$$t_{90} = \frac{.848\,(3.5 \times 12)^2}{1.584} \simeq 2.6\ yrs$$

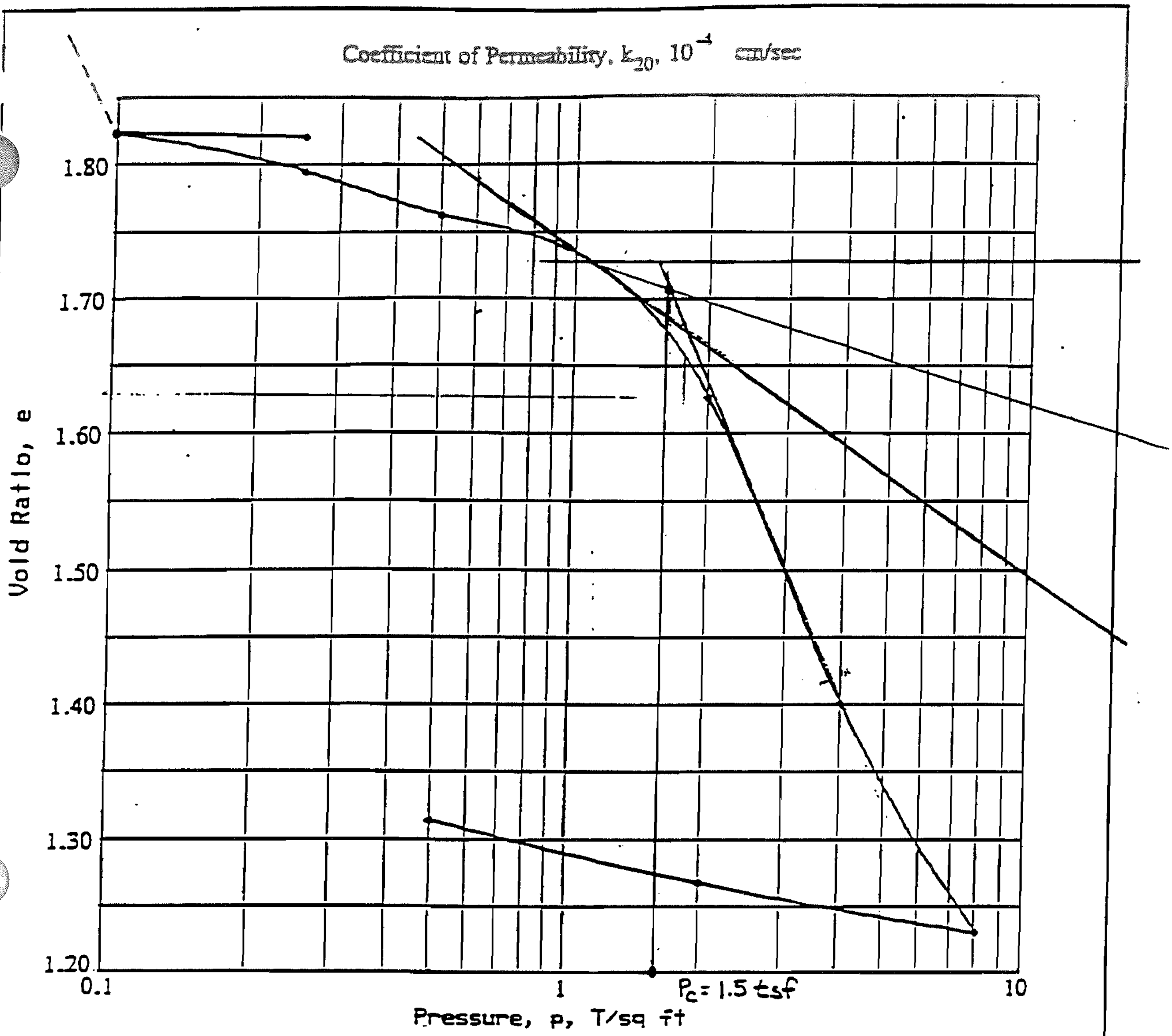
Settlement @ PM-11 Levee



as compared to settlement at BP-3 levee, this section has:

less levee load
half the compressible soil
compressible strata situated deeper

∴ say settlement due to consolidation ~ 1"



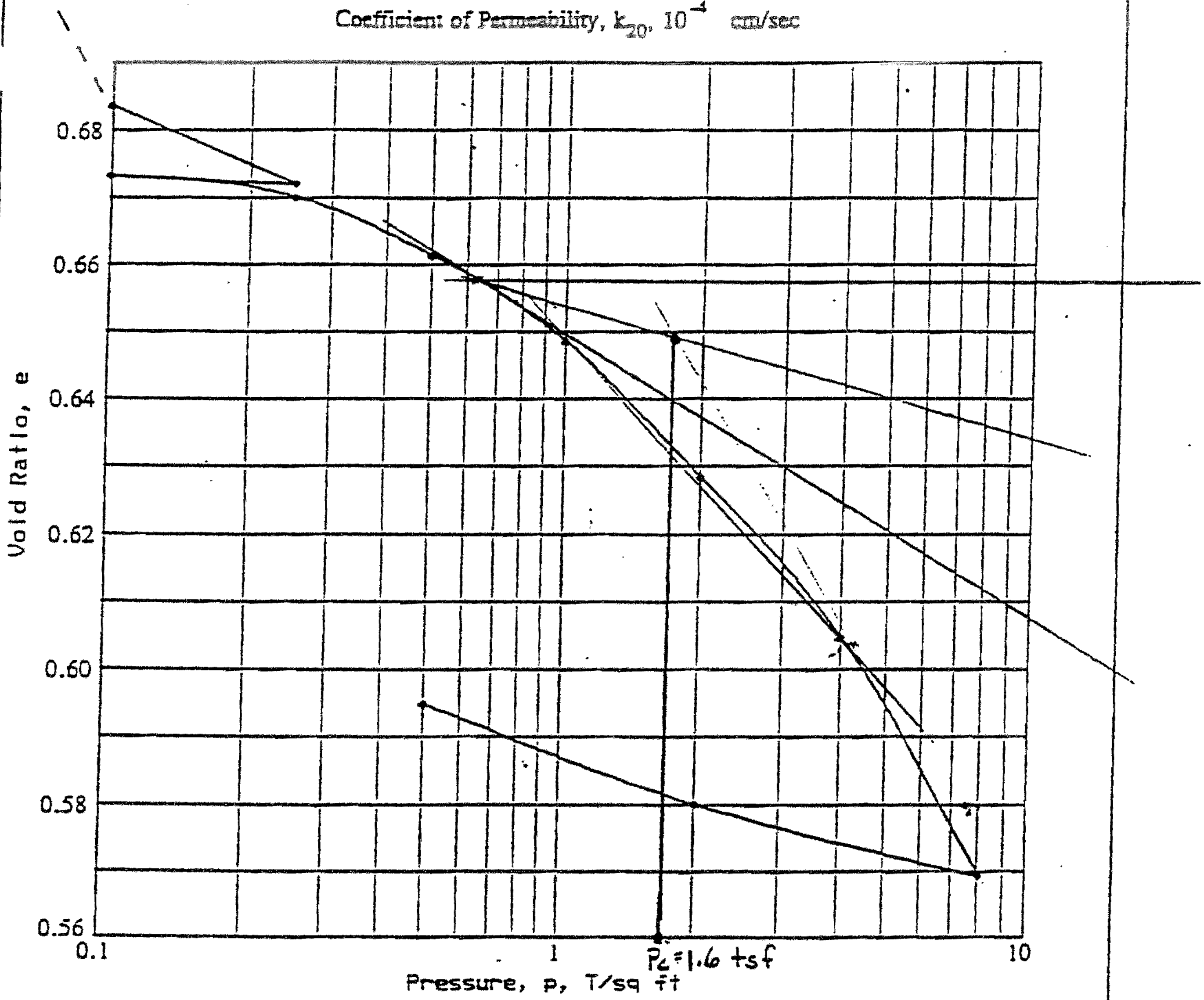
Type of Specimen UNDISTURBED		Before Test			After Test		
Diam. 2.490 in.	Height 1.000 in.	Water Content	w_o	74.8 %	w_f	49.0 %	
Overburden Pressure, p_o T/sq ft		Void Ratio	e_o	2.035	e_f	1.314	
Preconsol. Pressure, p_c T/sq ft		Saturation	S_o	98.4 %	S_f	100.0 %	
Compression Index, C_c		Dry Density	γ_d	55.1 lb/ft^3			
Classification GRAY, FAT CLAY (CH), WITH A TRACE OF SAND AND MICA.							
LL 116	G_s 2.68	Project PORT MONMOUTH NJ					
PL 40	D_{10}						
Remarks 1. SPECIMEN FLOODED			Lab No. 264/814				
THROUGHOUT TEST. 2. TIME CURVES		Boring No. ~~PM-2A~~ PM-P2A	Sample No.				
URNISHED FOR LOADS OF 0.5, 2.0		Depth (ft) 9.0-11.5 ✓	Date 08/01/97				
4.0 AND 8.0 TSF.		CONSOLIDATION TEST REPORT					

$$Cc = \frac{1.63 - 1.40}{\log\left(\frac{4}{2}\right)} = .764$$

$$Cs = \frac{1.314 - 1.26}{-\log\left(\frac{2}{.5}\right)} = ,0897$$

EB-73

REQUSITION: W31XNJ71



Type of Specimen	UNDISTURBED	Before Test			After Test	
Diam. 2.490 in.	Height 1.000 in.	Water Content	w_o	25.2 %	w_f	21.4 %
Overburden Pressure, p_o	T/sq ft	Void-Ratio	e_o	0.700	e_f	0.595
Preconsol. Pressure, p_c	T/sq ft	Saturation	S_o	100.0 %	S_f	100.0 %
Compression Index, C_c		Dry Density	γ_d	102.1 lb/ft³		

Classification GRAY, FAT CLAY (CH), WITH SOME SAND SIZES AND MICA.

LL	50	G_s	2.78	Project PORT MONMOUTH, NJ
PL	23	D_{10}		

Remarks 1. SPECIMEN FLOODED		Lab No. 264/813
THROUGHOUT TEST. 2. TIME CURVES	Boring No. ~~PM-1A~~ PM-P1A	Sample No.
RNISHED FOR LOADS OF 1.0,	Depth (ft) 5.0-7.0 ✓	Date 07/31/97
2.0, 4.0 AND 8.0 TSF.		

CONSOLIDATION TEST REPORT

$C_c = \frac{.605 - .570}{\log(\frac{8}{4})} = .116$

$C_s = \frac{.595 - .580}{\log(2/.5)} = .0249$

EB-74

SUMMARY OF LABORATORY TEST RESULTS

PROJECT: Port Monmouth Road 90L016A **DATE:** 8/90

BORING & SAMPLE NUMBER	DEPTH	CLASSIFICATION	NATURAL WATER CONTENT %	ATTERBERG LIMITS		UNCONFINED COMPRESSION		UNIT DRY WEIGHT PCF	SPECIFIC GRAVITY	Vane Shear PSF	COMPACTION	GRAIN SIZE	CONSOLI-DATION	TRIAXIAL	C.B.R.	pH	ORG NIC
				LIQUID LIMIT	PLASTIC LIMIT	STRESS TSF	STRAIN %										
P-3 S-7	12'-14'	Dark Grey Organic SILT with fibers	80	113	50			52	2.25	685			*	*			7
P-4 S-9	20'-22'	Green Brown mf SAND, trace Silt	13					93									
P-5 S-5	8'-10'	Dark Grey Organic SILT with fibers	97	93	46			45	2.20	220			*				8
-16	1'-3'	Brown mf SAND, little Silt	19					90									

EB-75

* SEE TEST CURVES

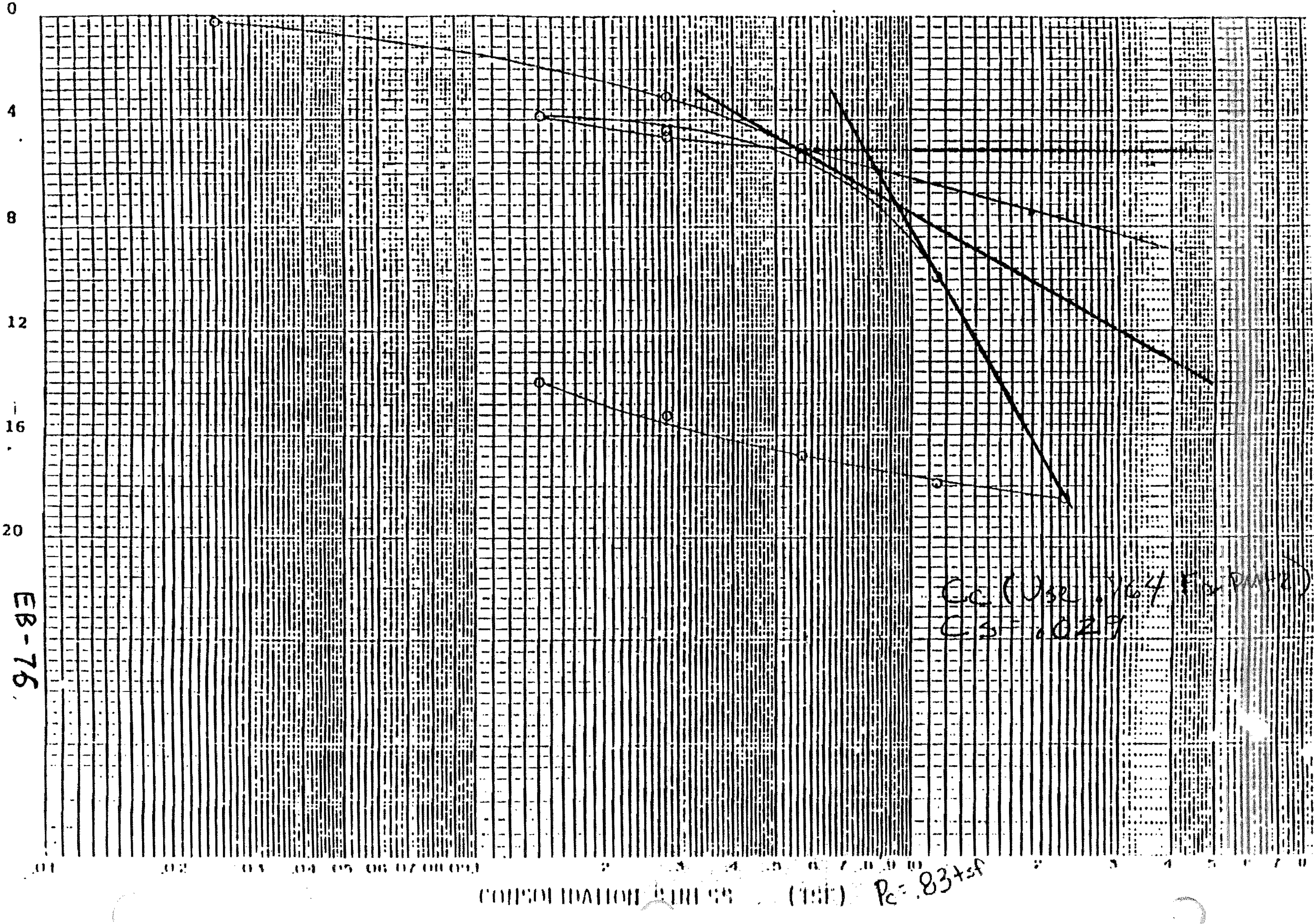
PROJECT: Port Monmouth Road

DESCRIPTION: Dark organic SILT, with fibers.

LOCATION:

INITIAL HEIGHT 1 INCH

CONSOLIDATI TES
COMPRESSION CURV



Cc (Use .164 for [illegible])
Cs .027

Pc = .83 tsf

EB-76.

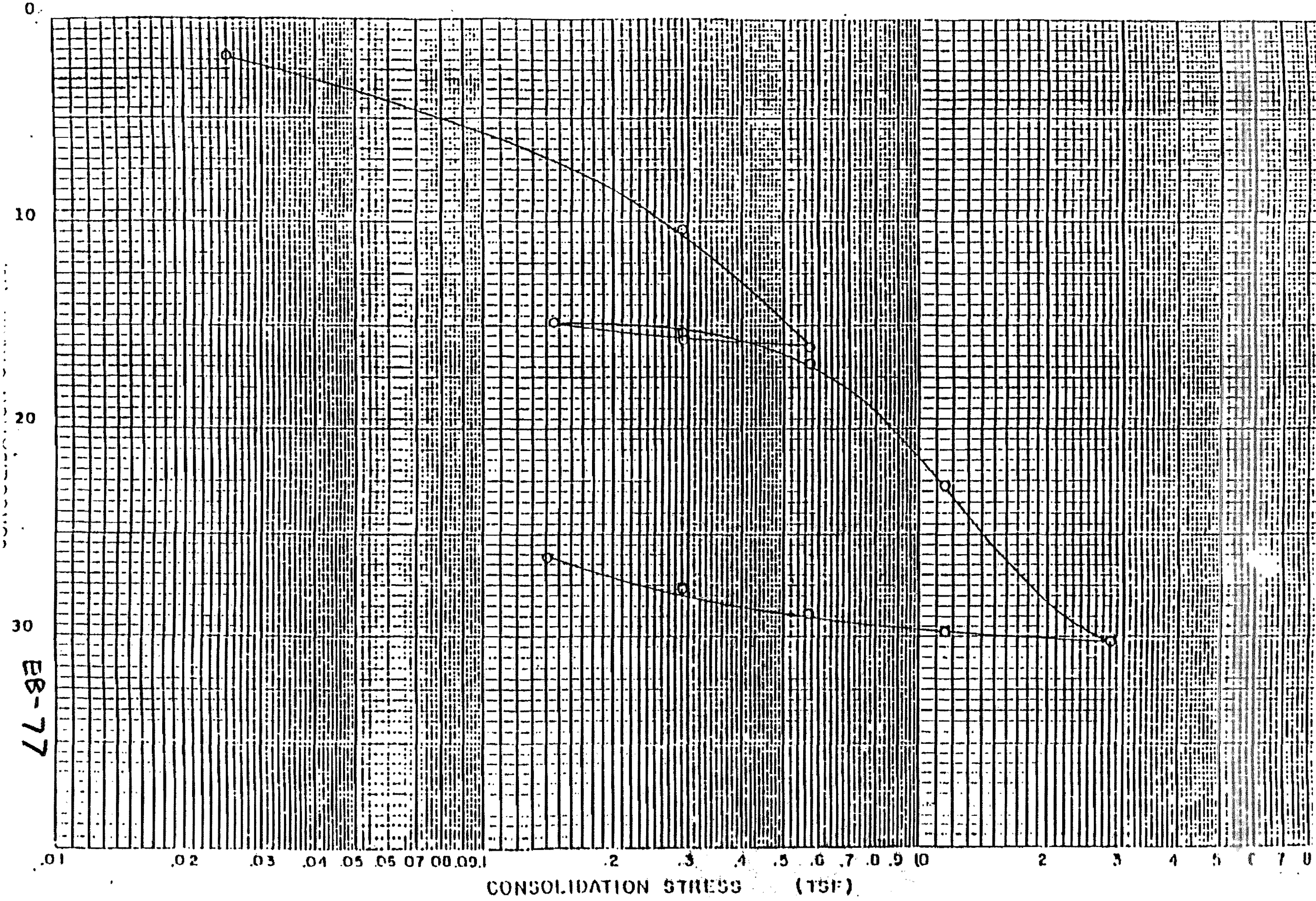
PROJECT Port Monmouth Road

LOCATION

DESCRIPTION Dar' y organic SILT, w ers.

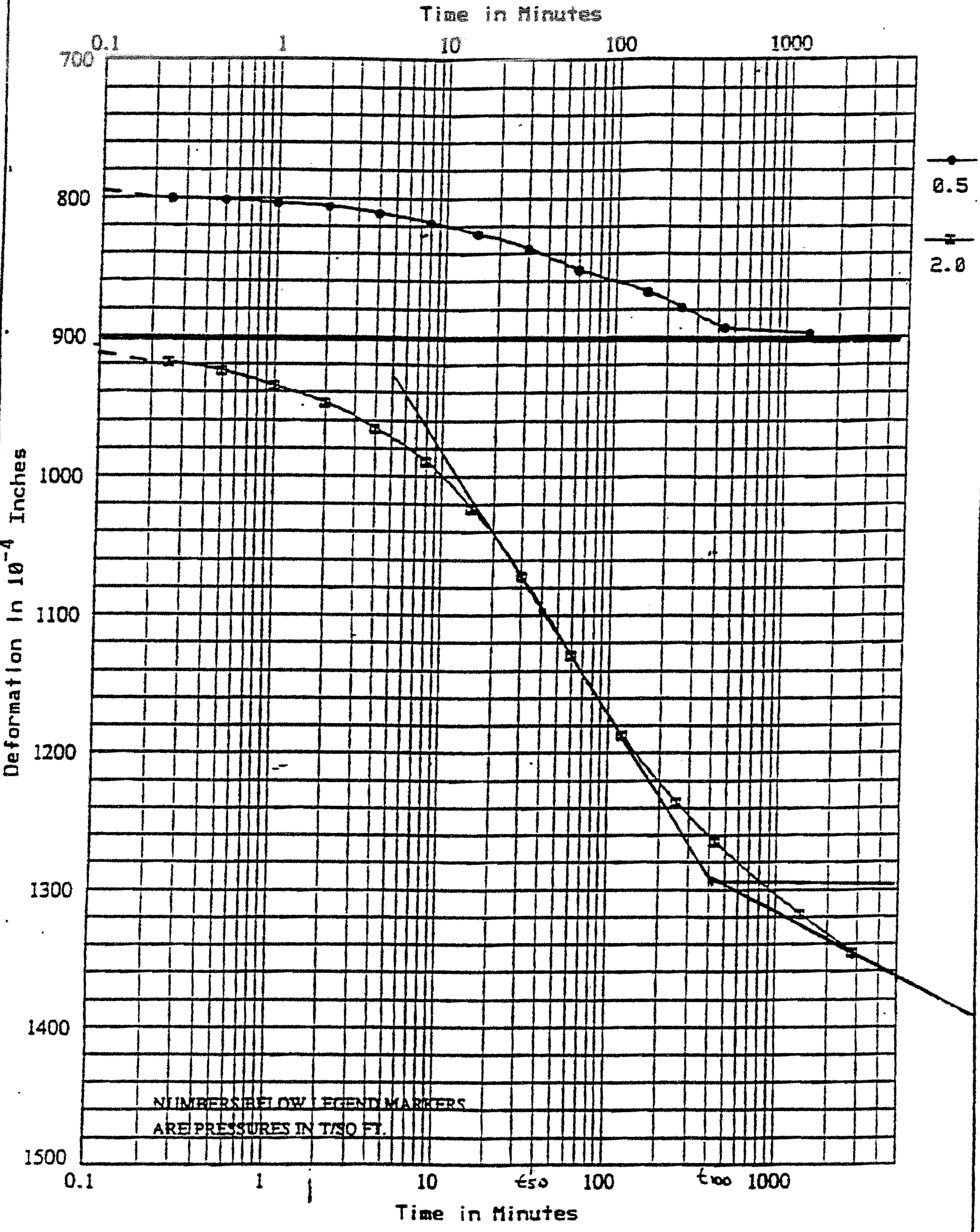
INITIAL HEIGHT 1 INCH

CONSOLIDAT TES
COMPRESS URV



EB-77

WORK ORDER: 8275
REQUSITION: W31XNJ71694



ENGINEERS, 611 SOUTH COBB DRIVE, MARIETTA, GA

Project PORT MONMOUTH, NJ			
Area			Lab No. 264/814
Boring No. PM-2A	Sample No.	Depth (ft) 9.0-11.5	Date 08/01/97

CONSOLIDATION TEST - TIME CURVES

1- .1122 - .8878/2 = .4439 = H

EB-78

APPENDIX C
FLOOD WALLS

page EC

```
PILE      Rev: 9208      GOODKIND & O'DEA, INC  1998.06.03  10:53:13  PAGE:     1
roject:  1968                           Designed By:  JTM                05/31/98
r   :  port.in                          Checked  By:  XXX                05/31/98
      PORT MONMOUTH PMP1 El. 13
    -------------------<< CANTILEVER SHEET PILING >>----------------------

                    * * * INPUT VERIFICATION * * *

                                  DREDGE DEPTH
               N        DELTA        (PLAG)        SURCHARGE
                        (DEG)         (FT)         (LB/FT2)
               4.        .00          9.6              .0


                  COHESION  DENSITY   LAYER    ----- COEFFICIENTS -------
 LAYER     PHI      (C)     (GAMMA)  DEPTH(H)    Ka         Kp          Ka
          (DEG)  (LB/FT2)  (LB/FT3)   (FT)                            DREDGE
   1        .0       .0      64.4      9.6    1.00000    1.00000    1.00000
   2        .0    750.0      50.0     17.6    1.00000*   1.00000*   1.00000*
   3      30.0       .0      55.0     24.6     .33333*   3.00000*    .33333*
   4      33.0       .0      55.0     40.0     .29480*   3.39212*    .29480*

                     [*]  Values not input. Calculated internally.


                          * * * RESULTS * * *

                         PRESSURE DIAGRAM DATA

              ---- PRESSURE -----                     DEPTH
                TOP         BOTTOM         HEIGHT    BELOW GND
             (LB/FT2)      (LB/FT2)         (FT)        (FT)
                  .00        618.24         9.600       9.600
             -1500.00      -1879.50         7.590      17.190

                       PRESSURE DIAGRAM HEIGHTS

Distance (X)     from PT of zero pressure to pile tip:      7.5900 (FT)
Distance (Z)     from PT Z                to pile tip:      4.5038 (FT)
Distance (ZPRIM) from intersection PT I   to pile tip:      2.7093 (FT)
Distance (ZCONJ) from intersection PT I   to PT Z     :     1.7944 (FT)

           Depth of pile tip below ground            :     17.190    (FT)

           Pressure at pile tip                      :    2497.74    (LB/FT2)

           Pressure at point Z                       :   -1654.31    (LB/FT2)

           Depth of point of zero shear below ground:      11.52     (FT)

           Moment at point of zero shear             :   12370.09    (FT-LB)
```

Keansburg

Depth below ground = 1.3(17.2) = 22.4' Say 23'

EC-1

```
ILE      Rev: 9208      GOODKIND & O'DEA, INC  1998.06.03  12:32:07  PAGE:    1
oject:  1968                               Designed By:  JTM               05/31/98
  :    port.in                             Checked  By:  XXX               05/31/98
      PORT MONMOUTH PMP1 El. 14
    --------------------<< CANTILEVER SHEET PILING >>---------------------------

                    * * * INPUT VERIFICATION * * *

                                  DREDGE DEPTH
               N        DELTA        (PLAG)        SURCHARGE
                        (DEG)         (FT)         (LB/FT2)
               4.         .00          5.8               .0


                COHESION  DENSITY   LAYER     ----- COEFFICIENTS -------
  LAYER     PHI    (C)     (GAMMA)  DEPTH(H)    Ka          Kp          Ka
           (DEG) (LB/FT2)  (LB/FT3)   (FT)                            DREDGE
    1        .0      .0      64.4      5.8   1.00000     1.00000     1.00000
    2      29.0      .0      55.0      9.8    .34697*    2.88206*     .34697*
    3      32.0      .0      55.0     19.8    .30726*    3.25459*     .30726*
    4        .0   *****      50.0     40.0   1.00000*    1.00000*    1.00000*

                    [*]  Values not input. Calculated internally.


                          * * * RESULTS * * *

                        PRESSURE DIAGRAM DATA

            ---- PRESSURE -----                        DEPTH
              TOP          BOTTOM         HEIGHT     BELOW GND
            (LB/FT2)      (LB/FT2)         (FT)         (FT)
                 .00            .00          .000         .000
                 .00         373.52         5.800        5.800
              129.60            .00          .930        6.730
                 .00        -428.12         3.070        9.800
             -716.01       -4384.39         4.495       14.295

                       PRESSURE DIAGRAM HEIGHTS

Distance (X)      from PT of zero pressure to pile tip:       7.5650 (FT)
Distance (Z)      from PT Z                to pile tip:       3.1150 (FT)
Distance (ZPRIM)  from intersection PT I   to pile tip:       1.8406 (FT)
Distance (ZCONJ)  from intersection PT I   to PT Z    :       1.2744 (FT)

           Depth of pile tip below ground             :     14.295   (FT)

           Pressure at pile tip                       :    2660.24   (LB/FT2)

           Pressure at point Z                        :   -1841.96   (LB/FT2)

           Depth of point of zero shear below ground:        10.32   (FT)

           Moment at point of zero shear              :    6113.48   (FT-LB)
```

Sta 71+00 to Sta. 83+50

Depth below Ground = 1.3 (14.3) = 18.6 Say 19'

EC-2

```
PILE     Rev: 9208      GOODKIND & O'DEA, INC  1998.06.03  12:27:18  PAGE:    1
roject:  1968                         Designed By:  JTM                05/31/98
n  :     port.in                      Checked  By:  XXX                05/31/98
   :     PORT MONMOUTH PMP1 El. 15.2
    --------------------<< CANTILEVER SHEET PILING >>-----------------------

                        * * * INPUT VERIFICATION * * *

                                     DREDGE DEPTH
                 N        DELTA         (PLAG)          SURCHARGE
                          (DEG)          (FT)           (LB/FT2)
                 4.         .00           7.0                  .0


                    COHESION   DENSITY   LAYER     ----- COEFFICIENTS -------
  LAYER      PHI      (C)      (GAMMA)  DEPTH(H)    Ka         Kp          Ka
            (DEG)  (LB/FT2)   (LB/FT3)   (FT)                           DREDGE
    1          .0       .0       64.4     7.0    1.00000    1.00000    1.00000
    2        29.0       .0       55.0    11.0     .34697*   2.88206*    .34697*
    3        32.0       .0       55.0    21.0     .30726*   3.25459*    .30726*
    4          .0    *****       50.0    40.0    1.00000*   1.00000*   1.00000*

                        [*]  Values not input. Calculated internally.


                              * * * RESULTS * * *

                            PRESSURE DIAGRAM DATA

                ---- PRESSURE -----                       DEPTH
                  TOP          BOTTOM          HEIGHT    BELOW GND
                (LB/FT2)      (LB/FT2)          (FT)        (FT)
                    .00            .00          .000        .000
                    .00         450.80         7.000       7.000
                 156.42            .00         1.122       8.122
                    .00        -401.30         2.878      11.000
                -716.01       -4931.97         5.747      16.747

                         PRESSURE DIAGRAM HEIGHTS

 Distance (X)     from PT of zero pressure to pile tip:      8.6250 (FT)
 Distance (Z)     from PT Z                to pile tip:      3.7625 (FT)
 Distance (ZPRIM) from intersection PT I   to pile tip:      2.2168 (FT)
 Distance (ZCONJ) from intersection PT I   to PT Z    :      1.5457 (FT)

             Depth of pile tip below ground          :     16.747    (FT)

             Pressure at pile tip                    :    3114.76    (LB/FT2)

             Pressure at point Z                     :   -2171.74    (LB/FT2)

             Depth of point of zero shear below ground:     12.00    (FT)

             Moment at point of zero shear           :   10364.57    (FT-LB)
```

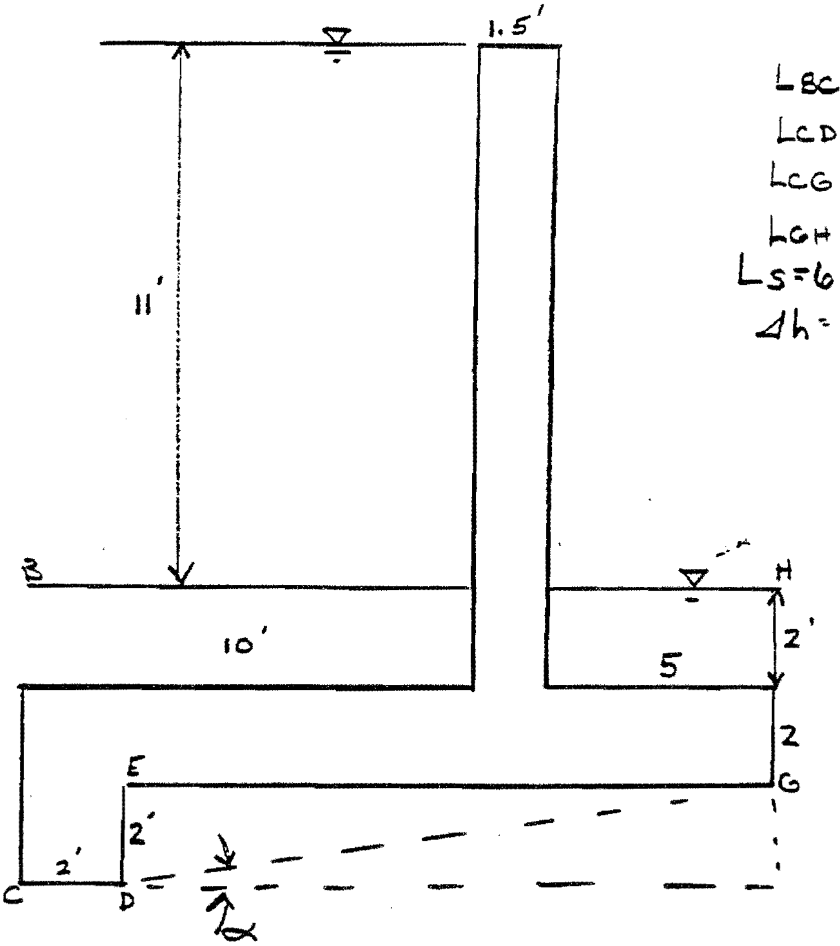
Sta. 71+00 to 83+50

Depth below ground: 1.3(16.7') = 21.7' Say 22'

EC-3

JOB Port Monmouth Flood Control NOTES BY JTM DATE 6/2/98 PAGE NO. 1
CHECK BY ETS DATE 6-4-98

T-Type Wall - Elev. 15.2, Sta 12+50 to Sta 29+00.



$L_{BC} = 6'$
$L_{CD} = 2'$
$L_{CG} = \sqrt{2^2+16.5^2} = 16.6$ ✓
$L_{GH} = 4'$
$L_S = 6 + 16.6 + 4 = 26.6$
$\Delta h = 11$ ✓

Seepage Path BCDGH

$U_C = \left[17 - 11\frac{6}{26.6}\right].064 = .93$ ksf ✓

$U_G = \left[15 - 11\,\frac{6+16.6}{26.6}\right].064 = .36$ ksf ✓

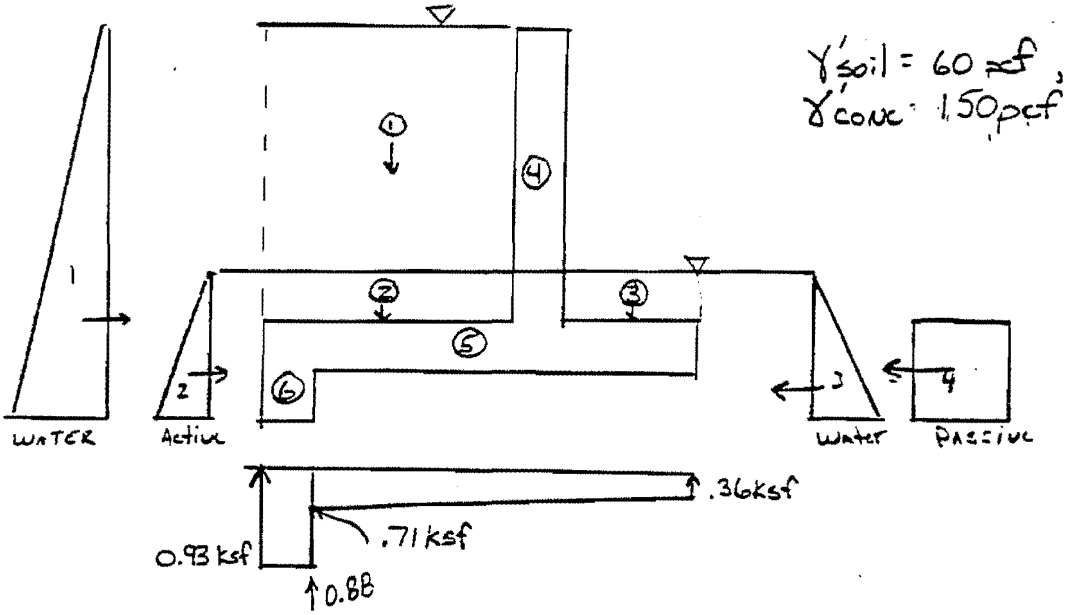
Head Loss:

$CG = 11\,\frac{16.6}{26.6} = 6.86'$, $\underset{\text{CONC SURF.}}{L_S} = 2+2+14.5 = 18.5'$ ✓

@ D : Head Loss $= \frac{2}{18.5}(6.86) = .74$, $U_D = \left[17 - 11\frac{6}{26.6} - .74\right].0644 = .88$ ksf ✓

E : $H_{Loss} = \frac{4}{18.5}(6.86) = 1.48$, $U_E = \left[15 - 11\,6/26.6 - 1.48\right].0644 = .71$ ksf ✓

JOB Port Monmouth Flood Control NOTES BY JTM DATE 6/1/98 PAGE NO. 2
CHECK BY ETS DATE 6/4/98



γ'_{soil} = 60 pcf, ϕ = 32°, ka = .30
γ'_{conc} = 150 pcf

					k-ft
ΣV	① .064 (11×10)	= 7.08 k	@	11.5'	81.42
	② .06 (10×2)	1.2 k	@	11.5'	13.8
	③ .06 (5×2)	0.6 k	@	2.5'	1.5
	④ .15 (1.5×13)	2.9 k	@	5.75'	16.6
	⑤ .15 (2×16.5)	4.9 k	@	8.25'	40.4
	⑥ .15 (2^2)	0.60 k	@	15.5'	9.3
Uplift:					
	.36 (14.5)	−5.2 k	@	7.25'	−37.7
	(.71−.36)(14.5) ½	−2.5 k	@	9.6'	−24.0
	(.90)(2)	−1.8 k	@	15.5'	−27.9
		ΣV = 7.78			73.42
ΣH	1 .0644 (17^2) .5	−9.3 k	@	3.6'	−33.48
	2 .3 (6^2) .06 (.5)	−.3 k	@	0'	
	3 .0644 (6^2) .5	+1.2 k	@	0'	
	4 Pass Req.	+8.4 k	@	0'	

ΣM = 39.94 k-ft

$$\bar{x} = \frac{39.94}{7.78} = 5.13$$

Resultant Ratio = $\frac{5.13}{16.5}$ = .31 ∴ 97% Compression
(EM1110-2-2502 Fig 4-4) Say O.K.

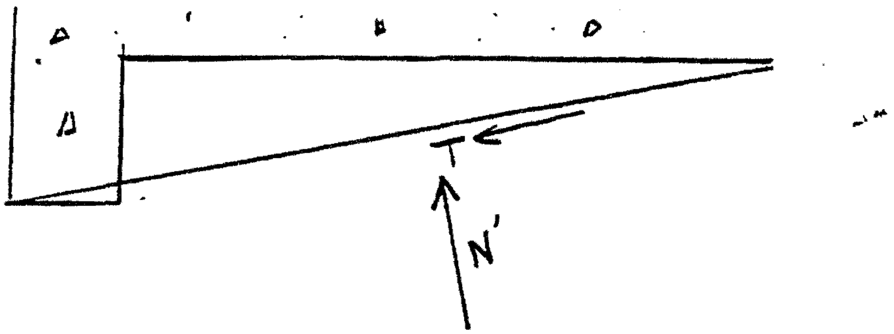
EC-5

Sliding

wht. of soil DEG = $\frac{1}{2}(2)(14.5).06 = .87$ k @ 9.7'

$\Sigma V = 17.3\text{ k} + .87 = 18.15\text{ k}$
$\Sigma H = 9.3\text{ k} - 1.2\text{ k} + .3\text{ k} = 8.4\text{ k}$

$N' = \Sigma V \cos\alpha + \Sigma H \sin\alpha$ $\alpha \Rightarrow \tan^{-1}\alpha = \frac{2}{16.5}$
$T = \Sigma H \cos\alpha - \Sigma V \sin\alpha$ $\alpha = 6.9^\circ$



$N = 18.15 \cos 6.9^\circ + 8.4 \sin 6.9^\circ = 18.97$, Say 19 ✓
$T = 8.4 \cos 6.9^\circ - 18.15 \sin 6.9^\circ = 6.15$ ✓

$$T \leq \frac{N' \tan\phi}{F.S.}$$

F.S. = 1.91 O.K. ✓

Bearing Capacity

$q_{ult} = \gamma D N_q + \frac{\gamma B}{2} N_\gamma$, $q_{NET} = \gamma D(N_q - 1) + \frac{\gamma B}{2} N_\gamma$

for $\phi = 32^\circ$, $N_q = 24$, $N_\gamma = 22$ $\gamma' = .06$, $D = 4$

$q_{NET} = .06(4)(24-1) + \frac{1}{2}\,.06(16.5)22 = 16.4$ ksf ✓

$q_{NET_{Allow}} = 16.4/3 = 5.5$ ksf ✓ Settlement Controls

$D_f/B \leq .25$, $B = 16.5'$ $q_{NET_{Allow}}$ $S = 2.25$ tsf ✓
$N_{AVG} \sim 20$ (Peck, Hanson, thornburn 1974)

T-Type Wall @ Keansburg

****

6

ABUTMENT AND RETAINING WALL

33021

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EC-7

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1                        ABUTMENT AND RETAINING WALL                      330
216

 PROGRAM P4354040                                                  06/05/98 15
:39
 VERSION 5.1                  LAST UPDATED 12/10/96          DOCUMENTATION 12
/96

 INPUT: LEVE1.DAT

 PROJECT RETAINING LEVEE FOR JM
 BY RL


                            ANALYSIS OF
                     ABUTMENT WITHOUT BACKWALL
```

EC-8

```
                       TYPE OF FOOTING : ON PILES

                                                                EQUIV FLUID
 E  GN                    FTG      PILE             PILE           PRESSURE
 ETHOD    A OR D    TYPE  TYPE       D    EMBEDDED  ROWS   R OR S   DRY    WET
  SL        A        4     2       12.0      Y       3              64.4   64.4

                       ALLOW SOIL   WATER  TOP FTG    WATER   TOP WALL   LIVE
OEFF OF   BACKFILL  PRESS OR AXIAL LEVEL  TO TOP     LEVEL    TO TOP    LOAD
RICTION     SLOPE     PILE CAPAC    BACK  EMBANK     FRONT   BACKFILL   SURCH
                         60.00      0.00   12.00

 TOP FTG   F'C                                      LATERAL
O ROCK    BACK    F'C    F'C   REBAR   PILE   PILE  PILE       REBAR OVR   80%
URFACE    WALL    STEM   FTG   GRADE  BATTER   OPT  CAPAC  Kv   DES  STR   RULE
         3500.   3500.  6000.                        2.0              Y
   TOP OF
BACK)WALL                                         MAX
O TOP OF     TOP       PROJECTION          MAX    FTG
 FOOTING   THICKNESS   TOE   HEEL  T OR H  PROJ  WIDTH    H1     H2     H3
4
 12.00       1.50      3.00  3.50

                             FRONT            HEIGHT
                            FACE   BRIDGE      OF
                            TO DL   SEAT   BACKWALL  BACK    FOOTING   PILE  F

      W2    W3    BW1   BW2    REACT   WIDTH    BATTER  BATTER THICKNESS COST  C
S
                                                      12.    2.50

                                           LONG
                     WIND   WIND          FORCE
 DL     LL     WIND    ON     ON   UPWARD  FROM    CENTR  TEMP
EACT   REACT   ON LL  SUPER   SUB   WIND    LL     FORCE  FORCE

                               BACKWALL             ALLOW
   PARAPET OR EXTERNAL        LIVE LOAD  SEISMIC    PILE
HORZ    DIST    VERT    DIST   VERT   HORZ   LOAD    UPLIFT

              DISTANCE          PERCENT
OW    PILE   BETWEEN    PILE      ROW
O    BATTER    ROWS    SPACING  BATTERED
1      3.0     1.50     4.00    100.0
2      3.0     3.00     4.00    100.0
3      0.0     3.00    12.00      0.0
***   NOTE   ***
EBAR GRADE WAS NOT ENTERED.  GRADE 60 BARS ASSUMED.
               REBAR SPACINGS (in):
                       6.
                       9.
```

EC-9

```
                         12.
                         15.
                         18.
   REBAR COVERS (in):   (C.G. OF BAR TO OUTER FACE)
          STEM                         FOOTING
  BACK          FRONT           TOP           BOTTOM
VERT  HORIZ   VERT  HORIZ  LONG   TRANS  LONG    TRANS
3.50   4.50   3.50   2.50  4.50    3.50  5.50     4.50
               BACK WALL -- SEVERE
EXPOSURES:    STEM ------- SEVERE
              FOOTING ---- NORMAL
 MINIMUM AREA OF STEEL PER FOOT: 0.125 SQUARE INCHES

**  NOTE  ***
HE REBARS IN THE BOTTOM OF THE FOOTING
RE BELOW THE TOP OF THE PILES
 ***  NOTE  ***
OR THE TEMPORARY CONSTRUCTION LOADING CONDITION (DEFINED AS GROUP T) -
ILL HEIGHT TO ABUTMENT SEAT, LIVE LOAD SURCHARGE, BUOYANCY AND WIND ON
UBSTRUCTURE IF APPLICABLE:
ILE CAPACITY HAS BEEN INCREASED BY A FACTOR OF 1.25.
 ***  NOTE  ***
OR PILE ROWS WITH BOTH VERTICAL AND BATTERED PILES,
ILE LOADS ARE FOR THE MAXIMUM CONDITION (AXIAL LOAD IN BATTERED PILES).
STABILITY ANALYSIS (SERVICE LOADS) - WITH VERTICAL COMPONENT OF LL SURCHARG


AASHTO          BAL.             O.T.    FRONT ROW   PILE LOAD     LATERAL  F.
.
ROUP   SUM V  MOMENT    SUM H  MOMENT   TO RESULT  FRONT   BACK   RESISTANCE O.
.
 1     12.03   47.66     6.77   32.72     1.24      30.58   4.35      7.28   1.
5
 2     12.03   47.66     6.77   32.72     1.24      30.58   4.35      9.10   1.
5
 3     12.03   47.66     6.77   32.72     1.24      30.58   4.35      9.10   1.
5
 4     12.03   47.66     6.77   32.72     1.24      30.58   4.35      9.10   1.
5
 5     12.03   47.66     6.77   32.72     1.24      30.58   4.35     10.19   1.
5
 6     12.03   47.66     6.77   32.72     1.24      30.58   4.35     10.19   1.

 T     12.03   47.66     6.77   32.72     1.24      30.58   4.35      9.10   1.
 TABILITY ANALYSIS (SERVICE LOADS) - WITHOUT VERTICAL COMPONENT OF LL SURCH
 GE

ASHTO           BAL.             O.T.    FRONT ROW   PILE LOAD     LATERAL   F
```

EC-10

```
 /P  SUM V   MOMENT    SUM H  MOMENT  TO RESULT  FRONT  BACK   RESISTANCE O.
  1  12.03   47.66     6.77   32.72     1.24     30.58   4.35       7.28  1.
46
  2  12.03   47.66     6.77   32.72     1.24     30.58   4.35       9.10  1.
46
  3  12.03   47.66     6.77   32.72     1.24     30.58   4.35       9.10  1.
46
  4  12.03   47.66     6.77   32.72     1.24     30.58   4.35       9.10  1.
46
  5  12.03   47.66     6.77   32.72     1.24     30.58   4.35      10.19  1.
46
  6  12.03   47.66     6.77   32.72     1.24     30.58   4.35      10.19  1.
46
  T  12.03   47.66     6.77   32.72     1.24     30.58   4.35       9.10  1.
46

 FOOTING ANALYSIS (SERVICE LOADS) - WITH VERTICAL COMPONENT OF LL SURCHARGE
 AASHTO      TOE      TOE         HEEL     HEEL
 GROUP      MOMENT    SHEAR @    MOMENT    SHEAR @
   1          9.52    -0.32 D      9.56     5.55 F
   2          9.52    -0.32 D      9.56     5.55 F
   3          9.52    -0.32 D      9.56     5.55 F
   4          9.52    -0.32 D      9.56     5.55 F
   5          9.52    -0.32 D      9.56     5.55 F
   6          9.52    -0.32 D      9.56     5.55 F
   T          9.52    -0.32 D      9.56     5.55 F

 FOOTING ANALYSIS (SERVICE LOADS) - WITHOUT VERTICAL COMPONENT OF LL SURCHAR
GE

 AASHTO      TOE      TOE         HEEL     HEEL
 GROUP      MOMENT    SHEAR @    MOMENT    SHEAR @
   1          9.52    -0.32 D      9.56     5.55 F
   2          9.52    -0.32 D      9.56     5.55 F
   3          9.52    -0.32 D      9.56     5.55 F
   4          9.52    -0.32 D      9.56     5.55 F
   5          9.52    -0.32 D      9.56     5.55 F
   6          9.52    -0.32 D      9.56     5.55 F
   T          9.52    -0.32 D      9.56     5.55 F

 FOOTING DESIGN - WITH AND WITHOUT VERTICAL COMPONENT OF LL SURCHARGE

 FOOTING   FOOTING                                EFFECTIVE DEPTH
 WIDTH    THICKNESS    TOE PROJ    HEEL PROJ        TOE     HEEL
  9.00      2.50         3.00        3.50          2.125   2.208

 SHEAR STR   ALLOWABLE   SHEAR STR   ALLOWABLE   TRANS REINFORCEMENT   LONGITUDIN
AL
    TOE      SHEAR TOE      HEEL     SHEAR HEEL  TOE-BOT     HEEL-TOP   REINFORCEM
ENT
```

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```
 0.000      0.074      0.017      0.074     0.26     0.31          0.27

LONGITUDINAL MOMENT CALCULATIONS - WITH VERTICAL COMPONENT OF LL SURCHARGE
GROUP  1 GOVERNS

MAX UNIFORM LOAD (w)            =   1.337 K/FT
MAX UNIFORM SERVICE LOAD (w) =   1.337 K/FT
MAX PILE SPACING                =   12.00 FEET
MAX LONGITUDINAL MOMENT         =    9.63 K-FT
MAX LONG. MOMENT (SERVICE)      =    9.63 K-FT

                                  PILE PATTERN

                  ROW      BATTER      DISTANCE     SPACING    % BATTERED
                   1      3.0 ON 12      1.50         4.00        100.0
                   2      3.0 ON 12      4.50         4.00        100.0
                   3        VERT         7.50        12.00          0.0

       C.G. OF PILES  3.64 FT. FROM TOE    I OF PILES PER FOOT  2.571
       PILE DENSITY 0.5833 PILES PER FOOT

DESIGN OF STEM SECTION AT  3.00 FT FROM TOP

AASHTO GROUP (SERVICE)     1       2       3       4       5       6       T
MOMENT                   0.30    0.30    0.30    0.30    0.30    0.30    0.30
  IAL FORCE              0.78    0.78    0.78    0.78    0.78    0.78    0.78
  EAR                    0.29    0.29    0.29    0.29    0.29    0.29    0.29

 SECTION     SHEAR    ALLOWABLE  BACK FACE
THICKNESS    STRESS     SHEAR      REINF
  1.750      0.001     0.056      0.125

DESIGN OF STEM SECTION AT  6.00 FT FROM TOP

AASHTO GROUP (SERVICE)     1       2       3       4       5       6       T
MOMENT                   2.36    2.36    2.36    2.36    2.36    2.36    2.36
AXIAL FORCE              1.75    1.75    1.75    1.75    1.75    1.75    1.75
SHEAR                    1.16    1.16    1.16    1.16    1.16    1.16    1.16

 SECTION     SHEAR    ALLOWABLE  BACK FACE
THICKNESS    STRESS     SHEAR      REINF
  2.000      0.005     0.056      0.125

DESIGN OF STEM SECTION AT  9.00 FT FROM TOP

AASHTO GROUP (SERVICE)     1       2       3       4       5       6       T
MOMENT                   7.91    7.91    7.91    7.91    7.91    7.91    7.91
AXIAL FORCE              2.94    2.94    2.94    2.94    2.94    2.94    2.94
SHEAR                    2.61    2.61    2.61    2.61    2.61    2.61    2.61

 SECTION     SHEAR    ALLOWABLE  BACK FACE
```

```
  CKNESS     STRESS      SHEAR       REINF
  2.250       0.009      0.056       0.142

DESIGN OF STEM SECTION AT 12.00 FT FROM TOP

AASHTO GROUP (SERVICE)    1      2      3      4      5      6      T
MOMENT                  18.71  18.71  18.71  18.71  18.71  18.71  18.71
AXIAL FORCE              4.32   4.32   4.32   4.32   4.32   4.32   4.32
SHEAR                    4.64   4.64   4.64   4.64   4.64   4.64   4.64

 SECTION     SHEAR    ALLOWABLE  BACK FACE
THICKNESS    STRESS     SHEAR      REINF
  2.500      0.015      0.056      0.365
1

                  SUMMARY OF STEEL DESIGN
                  -----------------------


                       FOOTING DESIGN

              Top longitudinal reinforcement:

     REBAR      REBAR       ACTUAL
    SPACING     SIZE          As
     (in)                 (sq in/ft)
      6           5         0.620
      9           5         0.413
     12           5         0.310
     15           6         0.352
     18           7         0.400

             Bottom longitudinal reinforcement:

     REBAR      REBAR       ACTUAL
    SPACING     SIZE          As
     (in)                 (sq in/ft)
      6           5         0.620
      9           5         0.413
     12           5         0.310
     15           6         0.352
     18           6         0.293

              Top transverse reinforcement:

     REBAR      REBAR       ACTUAL
    SPACING     SIZE          As
     (in)                 (sq in/ft)
      6           5         0.620
      9           5         0.413
```

12	5	0.310
15	6	0.352
18	7	0.400

Bottom transverse reinforcement:

REBAR SPACING (in)	REBAR SIZE	ACTUAL As (sq in/ft)
6	5	0.620
9	5	0.413
12	5	0.310
15	6	0.352
18	6	0.293

1

STEM DESIGN

Vertical reinforcement on back face:

REBAR SPACING (in)	REBAR SIZE	ACTUAL As (sq in/ft)
At Section #3, 3.00 feet from the top:		
6	4	0.400
9	4	0.267
12	4	0.200
15	5	0.248
18	5	0.207
At Section #4, 6.00 feet from the top:		
6	4	0.400
9	4	0.267
12	4	0.200
15	5	0.248
18	5	0.207
At Section #5, 9.00 feet from the top:		
6	4	0.400
9	4	0.267
12	4	0.200
15	5	0.248
18	5	0.207
At Section #6, 12.00 feet from the top:		
6	5	0.620
9	5	0.413
12	6	0.440
15	7	0.480
18	7	0.400

EC-14

```
1CPILE    Rev: 9208     GOODKIND & O'DEA, INC  1998.06.03  12:09:56  PAGE:  1
0Project:  1968                            Designed By:  JTM             05/31/98
 Input:  port.in                           Checked  By:  XXX             05/31/98
  le:  PORT MONMOUTH PMP1 El. 13
   ----------------------<< CANTILEVER SHEET PILING >>-------------------------

                        * * * INPUT VERIFICATION * * *

                                   DREDGE DEPTH
               N         DELTA        (PLAG)        SURCHARGE
                         (DEG)         (FT)         (LB/FT2)
               2.          .00          8.0              .0


                  COHESION  DENSITY  LAYER     ----- COEFFICIENTS -------
  LAYER     PHI      (C)    (GAMMA)  DEPTH(H)    Ka        Kp         Ka
           (DEG)  (LB/FT2)  (LB/FT3)   (FT)                          DREDGE
    1        .0       .0      64.4      8.0    1.00000   1.00000   1.00000
    2      30.0       .0      55.0     40.0     .33333*  3.00000*   .33333*

                     [*]  Values not input. Calculated internally.


                            * * * RESULTS * * *

                         PRESSURE DIAGRAM DATA

               ---- PRESSURE -----                       DEPTH
                 TOP          BOTTOM         HEIGHT    BELOW GND
               (LB/FT2)      (LB/FT2)         (FT)        (FT)
                   .00         515.20         8.000       8.000
                171.73            .00         1.171       9.171
                   .00       -1779.80        12.135      21.306

                       PRESSURE DIAGRAM HEIGHTS

 Distance (X)     from PT of zero pressure to pile tip:     12.1350 (FT)
 Distance (Z)     from PT Z                to pile tip:      3.2725 (FT)
 Distance (ZPRIM) from intersection PT I   to pile tip:      2.3858 (FT)
 Distance (ZCONJ) from intersection PT I   to PT Z    :       .8867 (FT)

           Depth of pile tip below ground             :     21.306    (FT)

           Pressure at pile tip                       :   3497.13    (LB/FT2)

           Pressure at point Z                        :  -1299.83    (LB/FT2)

           Depth of point of zero shear below ground:       14.60    (FT)

           Moment at point of zero shear              :   15809.42    (FT-LB)
1
```

Sta. 12+50 to Sta. 29+00

Depth below ground = 1.2(21.3) = 25.5' Say 26

EC-15

```
1CPILE      Rev: 9208       GOODKIND & O'DEA, INC  1998.06.03  11:59:00  PAGE:
0Project:  1968                            Designed By:  JTM              05/31/
  put:   port.in                           Checked  By:  XXX              05/31/
  le:   PORT MONMOUTH PMP1 El. 14
 -----------------------<< CANTILEVER SHEET PILING >>----------------------

                      * * * INPUT VERIFICATION * * *

                                   DREDGE DEPTH
                 N        DELTA       (PLAG)       SURCHARGE
                          (DEG)        (FT)        (LB/FT2)
                 2.        .00         9.0            .0


                  COHESION  DENSITY   LAYER    ----- COEFFICIENTS -------
 LAYER     PHI       (C)    (GAMMA)  DEPTH(H)    Ka        Kp         Ka
          (DEG)   (LB/FT2)  (LB/FT3)   (FT)                          DREDGE
   1        .0        .0      64.4      9.0    1.00000   1.00000   1.00000
   2      30.0        .0      55.0     40.0     .33333*  3.00000*   .33333*

                    [*]  Values not input. Calculated internally.


                          * * * RESULTS * * *

                        PRESSURE DIAGRAM DATA

                ---- PRESSURE -----                    DEPTH
                  TOP         BOTTOM       HEIGHT    BELOW GND
                (LB/FT2)     (LB/FT2)       (FT)        (FT)
                    .00        579.60       9.000       9.000
                 193.20           .00       1.317      10.317
                    .00      -2002.00      13.650      23.967

                      PRESSURE DIAGRAM HEIGHTS

 Distance (X)      from PT of zero pressure to pile tip:     13.6500 (FT)
 Distance (Z)      from PT Z                to pile tip:      3.6812 (FT)
 Distance (ZPRIM)  from intersection PT I   to pile tip:      2.6838 (FT)
 Distance (ZCONJ)  from intersection PT I   to PT Z    :       .9974 (FT)

          Depth of pile tip below ground             :      23.967    (FT)

          Pressure at pile tip                       :     3934.00    (LB/FT2

          Pressure at point Z                        :    -1462.08    (LB/FT2

          Depth of point of zero shear below ground:         16.42    (FT)

          Moment at point of zero shear              :    22509.90    (FT-LB)
1
```

Sta. 12+50 to Sta. 29+00

Depth below ground: 1.2(23.9) = 28.7'

Say 29'

EC-16

```
ILE     Rev: 9208      GOODKIND & O'DEA, INC  1998.06.05  14:51:50  PAGE:      1
oject:   1968                      Designed By:  JTM                  05/31/98
---t:    port.in                   Checked  By:  XXX                  05/31/98
   :     PORT MONMOUTH PM El. 13
   -------------------<< CANTILEVER SHEET PILING >>-------------------------

                    * * * INPUT VERIFICATION * * *

                                       DREDGE DEPTH
                N        DELTA            (PLAG)        SURCHARGE
                         (DEG)             (FT)          (LB/FT2)
                3.        .00              5.0               .0


                 COHESION  DENSITY   LAYER     ----- COEFFICIENTS --------
LAYER      PHI      (C)     (GAMMA)  DEPTH(H)    Ka         Kp          Ka
          (DEG)  (LB/FT2)  (LB/FT3)   (FT)                           DREDGE
  1         .0       .0      64.4      5.0   1.00000    1.00000    1.00000
  2       24.0       .0      55.0     21.0    .42173*   2.37118*    .42173*
  3       30.0       .0      55.0     50.0    .33333*   3.00000*    .33333*

                      [*]  Values not input. Calculated internally.


                           * * * RESULTS * * *

                         PRESSURE DIAGRAM DATA

               ---- PRESSURE -----                      DEPTH
                TOP          BOTTOM        HEIGHT     BELOW GND
             (LB/FT2)       (LB/FT2)        (FT)        (FT)
                  .00         322.00        5.000       5.000
               135.80            .00        1.267       6.267
                  .00        -977.31        9.115      15.382

                       PRESSURE DIAGRAM HEIGHTS

Distance (X)     from PT of zero pressure to pile tip:      9.1150 (FT)
Distance (Z)     from PT Z                to pile tip:      2.4962 (FT)
Distance (ZPRIM) from intersection PT I   to pile tip:      1.8113 (FT)
Distance (ZCONJ) from intersection PT I   to PT Z    :       .6850 (FT)

           Depth of pile tip below ground           :     15.382    (FT)

           Pressure at pile tip                     :    1876.63    (LB/FT2)

           Pressure at point Z                      :    -709.66    (LB/FT2)

           Depth of point of zero shear below ground:      10.34    (FT)

           Moment at point of zero shear            :    4855.41    (FT-LB)
```

Sta 8+50 to 12+50

1.3(15.4) = 20'

EC-17

```
PILE     Rev: 9208      GOODKIND & O'DEA, INC  1998.06.05  14:53:47  PAGE:     1
roject:  1968                              Designed By:  JTM               05/31/98
   :    port.in                             Checked  By:  XXX               05/31/98
        PORT MONMOUTH PM El. 14
    -----------  -------<< CANTILEVER SHEET PILING >>---------------------

                     * * * INPUT VERIFICATION * * *

                                     DREDGE DEPTH
                N        DELTA          (PLAG)        SURCHARGE
                         (DEG)           (FT)         (LB/FT2)
                3.         .00           6.0               .0

                      COHESION  DENSITY   LAYER     ----- COEFFICIENTS -------
   LAYER      PHI       (C)     (GAMMA)  DEPTH(H)    Ka        Kp          Ka
             (DEG)   (LB/FT2)  (LB/FT3)   (FT)                            DREDGE
     1         .0        .0       64.4     6.0   1.00000   1.00000   1.00000
     2       24.0        .0       55.0    22.0    .42173*  2.37118*    .42173*
     3       30.0        .0       55.0    50.0    .33333*  3.00000*    .33333*

                       [*]  Values not input. Calculated internally.

                            * * * RESULTS * * *

                            PRESSURE DIAGRAM DATA

                ---- PRESSURE -----                    DEPTH
                  TOP          BOTTOM       HEIGHT    BELOW GND
               (LB/FT2)      (LB/FT2)        (FT)       (FT)
                    .00        386.40        6.000      6.000
                 162.96           .00        1.520      7.520
                    .00      -1172.99       10.940     18.460

                         PRESSURE DIAGRAM HEIGHTS

  Distance (X)      from PT of zero pressure to pile tip:     10.9400  (FT)
  Distance (Z)      from PT Z                to pile tip:      2.9962  (FT)
  Distance (ZPRIM)  from intersection PT I   to pile tip:      2.1741  (FT)
  Distance (ZCONJ)  from intersection PT I   to PT Z     :      .8222  (FT)

            Depth of pile tip below ground            :     18.460   (FT)

            Pressure at pile tip                      :   2252.17    (LB/FT2)

            Pressure at point Z                       :   -851.73    (LB/FT2)

            Depth of point of zero shear below ground:     12.41    (FT)

            Moment at point of zero shear             :   8390.16    (FT-LB)
```

Sta. 8+50 to 12+50

1.3(18.4) = 23.9, 24'

EC-1B

```
PILE     Rev: 9208     GOODKIND & O'DEA, INC  1998.06.05  14:56:01  PAGE:    1
roject:  1968                      Designed By:  JTM                 05/31/98
       port.in                     Checked  By:  XXX                 05/31/98
    :  PORT MONMOUTH PM El. 15.2
    -------------------<< CANTILEVER SHEET PILING >>----------------------

                    * * * INPUT VERIFICATION * * *

                                DREDGE DEPTH
               N        DELTA      (PLAG)       SURCHARGE
                        (DEG)       (FT)        (LB/FT2)
               3.        .00         7.2             .0

                COHESION  DENSITY   LAYER     ----- COEFFICIENTS -------
  LAYER    PHI     (C)    (GAMMA)  DEPTH(H)    Ka        Kp         Ka
          (DEG) (LB/FT2) (LB/FT3)   (FT)                         DREDGE
    1       .0      .0     64.4      7.2    1.00000   1.00000   1.00000
    2     24.0      .0     55.0     23.2     .42173*  2.37118*   .42173*
    3     30.0      .0     55.0     50.0     .33333*  3.00000*   .33333*

                     [*]  Values not input. Calculated internally.


                          * * * RESULTS * * *

                        PRESSURE DIAGRAM DATA

              ---- PRESSURE -----                    DEPTH
                TOP         BOTTOM       HEIGHT    BELOW GND
             (LB/FT2)     (LB/FT2)        (FT)        (FT)
                  .00       463.68        7.200       7.200
               195.55          .00        1.824       9.024
                  .00     -1407.26       13.125      22.149

                     PRESSURE DIAGRAM HEIGHTS

 Distance (X)     from PT of zero pressure to pile tip:     13.1250  (FT)
 Distance (Z)     from PT Z                to pile tip:      3.5950  (FT)
 Distance (ZPRIM) from intersection PT I   to pile tip:      2.6086  (FT)
 Distance (ZCONJ) from intersection PT I   to PT Z    :       .9864  (FT)

            Depth of pile tip below ground              :     22.149   (FT)

            Pressure at pile tip                        :    2702.28   (LB/FT2)

            Pressure at point Z                         :   -1021.81   (LB/FT2)

            Depth of point of zero shear below ground:         14.89   (FT)

            Moment at point of zero shear               :   14498.19   (FT-LB)
```

Sta 8+50 to 12+50

1.3(22) = 28.65 : Say 28

EC-19

```
PILE    Rev: 9208      GOODKIND & O'DEA, INC  1998.06.03  12:37:13  PAGE:    1
roject:  1968                              Designed By:  JTM              05/31/98
     :  port.in                            Checked  By:  XXX              05/31/98
     :  PORT MONMOUTH PMP1 El. 13
   ====================<< CANTILEVER SHEET PILING >>------------------------

                     * * * INPUT VERIFICATION * * *

                                  DREDGE DEPTH
            N         DELTA          (PLAG)         SURCHARGE
                      (DEG)           (FT)          (LB/FT2)
            4.          .00           4.8               .0


                   COHESION   DENSITY   LAYER     ----- COEFFICIENTS -------
 LAYER      PHI       (C)     (GAMMA)  DEPTH(H)    Ka          Kp          Ka
           (DEG)   (LB/FT2)  (LB/FT3)   (FT)                             DREDGE
   1         .0        .0       64.4      4.8   1.00000     1.00000     1.00000
   2       29.0        .0       55.0      8.8    .34697*    2.88206*     .34697*
   3       32.0        .0       55.0     18.8    .30726*    3.25459*     .30726*
   4         .0     *****       50.0     40.0   1.00000*    1.00000*    1.00000*

                   [*]  Values not input. Calculated internally.


                        * * * RESULTS * * *

                       PRESSURE DIAGRAM DATA

            ---- PRESSURE -----                        DEPTH
             TOP          BOTTOM         HEIGHT      BELOW GND
           (LB/FT2)      (LB/FT2)         (FT)          (FT)
               .00            .00          .000          .000
               .00         309.12         4.800         4.800
            107.26            .00          .769         5.569
               .00        -450.46         3.231         8.800
           -716.01       -3791.13         3.414        12.214

                     PRESSURE DIAGRAM HEIGHTS

Distance (X)      from PT of zero pressure to pile tip:      6.6450  (FT)
Distance (Z)      from PT Z                to pile tip:      2.5175  (FT)
Distance (ZPRIM)  from intersection PT I   to pile tip:      1.5078  (FT)
Distance (ZCONJ)  from intersection PT I   to PT Z    :      1.0097  (FT)

          Depth of pile tip below ground            :      12.214    (FT)

          Pressure at pile tip                      :    2275.53     (LB/FT2)

          Pressure at point Z                       :   -1523.69     (LB/FT2)

          Depth of point of zero shear below ground:        8.87     (FT)

          Moment at point of zero shear             :    3527.46     (FT-LB)
```

Sta 71+00 to 83+50

Depth below ground = 1.3(12.2) = 15.8 say 16'

EC-20

OPTIMUM REBAR DESIGN

Top longitudinal reinforcement:

Spacing	Rebar
12.	5
18.	7
15.	6
9.	5
6.	5

Bottom longitudinal reinforcement:

Spacing	Rebar
18.	6
12.	5
15.	6
9.	5
6.	5

Top transverse reinforcement:

Spacing	Rebar
18.	7
12.	5
15.	6
9.	5
6.	5

Bottom transverse reinforcement:

Spacing	Rebar
18.	6
12.	5
15.	6
9.	5
6.	5

Here is the design of the vertical reinforcement in the back face of the stem. For this design, the spacing and rebar size are the same for the entire height of the wall.

Vertical reinforcement on back of stem:

Spacing	Rebar
18.	7
9.	5
12.	6
15.	7
6.	5

EL DESIGNS LISTED FROM MOST DESIRABLE TO LEAST OPTIMAL

```
--- ULTIMATE STATIC PILE CAPACITY/Federal Highway Administration --------+
     Nordlund (1963, 1979) and Tomlinson (1979, 1980) methods

P   ect Name      : Port Monmouth        Client          : T&M Associates
Fi  Name          : Twall.spl            Project Manager : mrr
Date              :  6/ 5/98             Computed by     : jtm

Depth of Top of Pile =   0.00 ft.        Pile length          =  22.00 ft.
Depth to Water Table =   0.00 ft.
Diameter of pile tip =   8.00 in.
Type of Pile         = Timber Pile
Taper of Pile        =     0.47

                         SKIN FRICTION CONTRIBUTION

Layer    Soil          Thickness    Effective   Internal   N-SPT      Pile
         Type                        Stress     Friction            Perimeter
                         (ft)         (psf)      Angle                (ft)

  1   Cohesionless       3.00         78.90     31.76*    15.86*      3.15
  2   Cohesive           9.00        394.50      ---       --         2.84
  3   Cohesionless      10.00        919.20     33.57*    21.91*      2.35

Layer    Soil        Undrained Shear  Adhesion   Pile    Sliding      Skin
         Type           Strength                 Taper   Friction  Resistance
                          (psf)                           Angle      (Kips)

      Cohesionless        --          --------   0.47     20.12        0.78
      Cohesive          800.00        800.00    0.47     -----        20.46
      Cohesionless        --          --------   0.47     16.36        19.64

                                        Total Side Friction    :      40.89
                       POINT RESISTANCE CONTRIBUTION

Effective      Internal      SPT      Pile End      Bearing        End Bearing
Stress at      Friction      Value      Area        Capacity       Resistance
Pile Tip        Angle                               Factor
 (psf)                                 (ft*ft)        Nq              (Kips)

 1207.20        39.54*      43.07       0.35        148.54            46.48

                              Limiting End Bearing Resistance :   133.05

                              Ultimate Static Pile Capacity   :    87.37
---- Hit arrow keys to display next screen. <F8> Print. <F10> Main Menu -----+
```

F.S. = 2.25 (EM 1110-2-2906)

$q_{all} = \frac{87.37}{2.25} = 38.8k = 19.4$ tons

EC-23

APPENDIX D
BORING LOGS
LABORATORY TESTING

page ED

PMP1

El. 4.0

DEPTH	BLOWS/0.5'	SAMPLE #	SOIL SYMBOL	SOIL AND ROCK CLASSIFICATION UNIFIED SOIL CLASSIFICATION	REMARKS
1	1	1	░	Tan to brown, fine SAND, trace Silt. (SP - FILL)	
	2		░		
2	2		░		
	3		░		
3	4	2	░		
	5		░		
4	7		‖	Greenish-brown, organic SILT, some Sand. (CL)	
	9		‖		
5	4	3	▨	Reddish-brown, Silty CLAY, little, fine Sand. (CH)	
	4		▨		
6	E		▨		
	8		▨		
7	3	4	▨	Greenish-grey	
	3		▨		
8	4		▨		
	9		▨		
9	2	5	‖	Tan to brown, medium SAND, trace Silt and Gravel, (SP-SM)	
	3		‖		
10	3		‖		
	5		‖		
11	2	⑥	☰	Brown, fibrous PEAT. (PT)	
	2		☰		
12	2		☰		
	3		☰		
13	4	7	‖	Gray-green, fine, glauconitic SAND, little Silt. (SM)	
	4		‖		
14	5		‖		
	7		‖		
15	4	8	‖		
	5		‖		
16	7		‖		
	5		‖		
17	5	9	‖	Gray-green, fine to medium SAND, trace Silt. (SP-SM)	
	9		‖		
18	15		‖		
	14		‖		
19	10	10	░	Gray-green to brown, fine to medium SAND, trace Silt. (SP)	
	15		░		
20	19		░		
	25		░		
21	9	11	░		
	17		░		
22	27		░		
	32		░		
23	14	12	░	Brown, fine to coarse grained, trace Silt and Gravel.	
	23		░		
24	21		░		
	24		░		

ED-1

oject #. MCTT-0033.00010-S11

oject Name: Port Monmouth – Near Marina

T&M ASSOCIATES

Elev. 4.5

Sheet 1 of 2
Boring # BP-3
Location Sta. 31+55
Offset 4 Ft. Right

r Water ____ ft w/ ____ casing out on ____	Date Started:	7/16/90	Ground Elev: ~~+10.9~~
. of Water 8 ft w/ all casing out on ____	Finished:	7/16/90	Grnd Wtr Elev: -2.9

eight of Hammer:	Hammer Fall on:	Casing	O.D. ____	I.D. ____
asing ____ lbs. Sampler 140 lbs.	Sampler: 30"	Sampler	O.D. 2"	I.D. 1-3/8"
nside Length of Sampler 24 in.	Casing: Auger	Coupling	O.D. ____	I.D. ____

pth low rface	Casing Blows Per Ft.	Sample # Depths Below Surface, ft.	Blows Per 6" on Sampler 0-6"	6-12"	12-18	18-24	Profile Change Depth	Soil Identification / Remarks
0	H	S-1	1	1	2	3		S-1 Yellowish grey & brown SILT, and med.-fine
	O	0-2'						Sand, trace Fibers
	L	S-2	2	2	3	4		S-2 Rust & grey med.-fine SAND, and Silt
	L	2'-4'						
5	O	S-3	4	5	5	6		S-3 Black & grey Organic Clayey SILT, trace
	W	4'-6'						fine Sand
		S-4	1	1	1	1		S-4 Black & Brown Clayey SILT, trace med.-
	S	6'-8'						fine Sand
	T	S-5	WOH	1	1	1		S-5 Brown & grey Organic Clayey SILT and
10	E	8'-10'						Fibers (peat)
	M	S-6	WOH	WOH	WOH	1		S-6 Grey CLAY & SILT, some Fibers (peat)
		10'-12'						
	A	S-7						S-7 Shelby Tube
	U							
	G							
	E	S-8	4	4	5	6		S-8 Light grey fine SAND, trace Silt
	R	15'-17'						Bottom: 6" Brown med.-fine SAND,
								trace Silt
20								
		S-9	1	1	1	1		S-9 Greenish brown med.-fine SAND, little Silt
		20'-22'						Bottom: Greenish brown med.-fine SAND,
								little fine Gravel, trace Silt
25								
		S-10	10	12	16	22		S-10 Brown med.-fine SAND, race Silt
		25'-27'						
30								

ils Engineer:	Michael J. Guerriero	Contractor:	Testwell Craig Test Boring Co.
illing Inspector:	Michael Steiner	Driller:	Tom Ward

Visual Identification Terms Used

	PLASTIC SOILS	
Clayey SILT	slight PI	Thread 1/4"
LT & CLAY	low PI	Thread 1/8"
AY & SILT	medium PI	Thread 1/16"
Silty CLAY	high PI	Thread 1/32"

GRANULAR SOILS	
GRAVEL	-Coarse 1"-3"
	-Med. 3/8"-1"
	-Fine .078"-3/8"
SAND	-Coarse .023"-.078"
	-Med. .01"-.023"
	-Fine .003"-.01"
SILT	-.003" and finer

PROPORTIONS USED

trace = 1%-10%
little = 10%-20%
some = 20%-35%
and = 35%-50%

ED-2

either the
boring is
located on [illegible]
incorrectly,
or the elev.
on log is wron

roject #: MCTY-0033.00010-S11
roject Name: Port Monmouth – Near Marina

T&M ASSOCIATES

Sheet 2 of 2
Boring # BP-3
Location Sta. 31+55
Offset 14 Ft. Right

Water ____ ft w/ ____ casing out on ____
epth of Water 8 ft w/ all casing out on ____
Date Started: 7/16/90
Finished: 7/16/90
Ground Elev: +10.9
Grnd Wtr Elev: -2.9

/eight of Hammer:
asing ____ lbs. Sampler 140 lbs.
Inside Length of Sampler 24 in.
Hammer Fall on:
Sampler: 30"
Casing: Auger

Casing	O.D. ____	I.D. ____
Sampler	O.D. 2"	I.D. 1-3/8"
Coupling	O.D. ____	I.D. ____

epth elow urface	Casing Blows Per Ft.	Sample # Depths Below Surface, ft.	Blows Per 6" on Sampler				Profile Change Depth	Soil Identification / Remarks
			0–6"	6–12"	12–18	18–24		
30	H	S-11	4	4	4	7		S-11 Same as S-9
	O	30'-32'						
	L							
	L	S-12						
35	O	35'-37'	7	8	9	10		S-12 Same as S-9
	W							
	S							
	T	S-13	7	8	9	10		S-13 Same as S-9
40	E	40'-42'						
	M							
	A							
	U							
	G							
	E							
	R							
50								
55								
60								

Soils Engineer: Michael J. Guerriero
Drilling Inspector: Michael Steiner
Contractor: Testwell Craig Test Boring Co.
Driller: Tom Ward

Visual Identification Terms Used

PLASTIC SOILS		
Clayey SILT	slight PI	Thread 1/4"
SILT & CLAY	low PI	Thread 1/8"
CLAY & SILT	medium PI	Thread 1/16"
Silty CLAY	high PI	Thread 1/32"

GRANULAR SOILS	
GRAVEL	–Coarse 1"–3"
	–Med. 3/8"–1"
	–Fine .078"–3/8"
SAND	–Coarse .023"–.078"
	–Med. .01"–.023"
	–Fine .003"–.01"
SILT	– .003" and finer

PROPORTIONS USED
trace = 1%–10%
little = 10%–20%
some = 20%–35%
and = 35%–50%

ED-3

roject #: MCTY-0033.00010-S11

roject Name: Port Monmouth – Near Marina

T&M ASSOCIATES

Sheet 1 of 2

Boring # BP-4

Location Sta. 31+41

Offset 24 Ft. Left

of Water ____ ft w/ ____ casing out on ____

. of Water 9.5 ft w/ all casing out on ____

Date Started: 7/11/90

Finished: 7/11/90

Ground Elev: +10.8

Grnd Wtr Elev: +1.

eight of Hammer:

asing ____ lbs. Sampler 140 lbs.

Inside Length of Sampler in.

Hammer Fall on:

Sampler: 30"

Casing: Auger

Casing O.D. ____ I.D. ____

Sampler O.D. 2" I.D. 1-3/8"

Coupling O.D. ____ I.D. ____

epth elow rface	Casing Blows Per Ft.	Sample # Depths Below Surface, ft.	Blows Per 6" on Sampler				Profile Change Depth	Soil Identification / Remarks	
			0–6"	6–12"	12–18	18–24			
0	H	S-1	3	4	5	6		S-1	Black & brown med.-fine SAND, trace Silt,
	O	0-2'							debris (weeds)
	L	S-2	3	4	4	5		S-2	Brown med.-fine SAND, trace Silt, trace
	L	2'-4'							fine Gravel
5	O	S-3	1	2	2	2		S-3	Dark greenish grey med.-fine SAND, and
	W	4'-6'							Silt, debris
		S-4	1	1	1	1		S-4	Top: Same as S-3
	S	6'-8'							Bottom: Black med.-fine SAND and Silt, and
	T	S-5	WOH	WOH	WOH	WOH			peat
10	E	8'-10'						S-5	Dark grey Clayey SILT, little med-fine
	M	S-6	1	1	1	1			Sand, little Fibers
		10'-12'						S-6	Dark grey CLAY & SILT, little Fibers
	A								
	U								
	G								
	E	S-7	1	1	1	2		S-7	Dark grey Clayey SILT, 2" ler.s, med.-fine
	R	15'-17'							Sand, some Silt at about 15'-6"
20									
		Shelby							
		Tube (20-22)						S-8	Shelby Tube
		S-9	8	10	16	19		S-9	Green and brown med.-fine SAND, trace
									Silt, trace fine Gravel
25									
		S-10							
		25'-27'	7	11	14	18		S-10	Same as S-8
30									

s Engineer: Michael J. Guerriero

Contractor: Testwell Craig Test Boring Co.

ling Inspector: Michael Steiner

Driller: Tom Ward

Visual Identification Terms Used

PLASTIC SOILS		
Clayey SILT	slight PI	Thread 1/4"
SILT & CLAY	low PI	Thread 1/8"
Y & SILT	medium PI	Thread 1/16"
Silty CLAY	high PI	Thread 1/32"

GRANULAR SOILS	
GRAVEL	–Coarse 1"–3"
	–Med. 3/8"–1"
	–Fine .078"–3/8"
SAND	–Coarse .023"–.078"
	–Med. .01"–.023"
	–Fine .003"–.01"
SILT	– .003" and finer

PROPORTIONS USED

trace = 1%–10%

little = 10%–20%

some = 20%–35%

and = 35%–50%

ED-4

ɔject #: MCTY-0033.00010-S11
ɔject Name: Port Monmouth – Near Marina

T&M ASSOCIATES

Sheet 2 of 2
Boring # BP-4
Loca Sta. Sta. 15+68
Offset 24 Ft. Left

.f Water ____ ft w/ ____ casing out on ____
Water 9.5 ft w/ all casing out on ____
ight Hammer:
sing ____ lbs. Sampler 140 lbs.
side Length of Sampler in.

Date Started: 7/11/90
Finished: 7/11/90

Ground Elev: +10.8
Grnd Wtr Elev: +1.3

Hammer Fall on:
Sampler: 30"
Casing: Auger

Casing	O.D. ____	I.D. ____
Sampler	O.D. 2"	I.D. 1-3/8"
Coupling	O.D. ____	I.D. ____

h v ce	Casing Blows Per Ft.	Sample # Depths Below Surface, ft.	Blows Per 6" on Sampler 0–6"	6–12"	12–18	18–24	Profile * Change Depth	Soil Identification / Remarks
)	H	S-11	5	8	8	8		S-11 Same as S-8
	O	30'-32'						
	L							
	L							
5	O	S-12	5	8	9	10		S-12 Same as S-8
	W	35-37'						
	S							
	T	S-13	7	7	11	11		S-13 Same as S-8
	E	40'-42'						
	M							
	A							
	J							
	R							

:ngineer: Michael J. Guerriero
Inspector: Michael Steiner
Contractor: Testwell Craig Test Boring Co.
Driller: Tom Ward

Visual Identification Terms Used

PLASTIC SOILS

Clayey SILT	slight PI	Thread 1/4"
T & CLAY	low PI	Thread 1/8"
SILT	medium PI	Thread 1/16"
Silty CLAY	high PI	Thread 1/32"

GRANULAR SOILS

GRAVEL	–Coarse 1"–3"
	–Med. 3/8"–1"
	–Fine .078"–3/8"
SAND	–Coarse .023"–.078"
	–Med. .01"–.023"
	–Fine .003"–.01"
SILT	– .003" and finer

PROPORTIONS USED

trace – 1%–10%
little – 10%–20%
some – 20%–35%
and – 35%–50%

ED-5

ASSOCIATES. P.A.

PARLIN EAST ONE • 3741 BORDENTOWN AVENUE • PARLIN, N.J. 08859

PORT MONMOUTH, NJ

PROJECT NO. 88B013

LOCATION SEE PLAN

OFFSET 4 ste. 22+3.

EPTH OF WATER ______ FT. W/ ______ FT. CASING OUT ON ______

EPTH OF WATER 10.0 FT. W/ ALL CASING OUT ON 6-14-88

DATE STARTED 6-14-88

DATE FINISHED 6-14-88

GROUND ELEV. +12.2

GROUND WATER ELEV. +

WEIGHT OF HAMMER:

CASING ______ LBS SAMPLER 140 LBS

INSIDE LENGTH OF SAMPLER: 24 IN.

CASING: O.D. ______ I.D. ______

SAMPLER: O.D. 2" I.D. 1-3/8"

COUPLING: O.D. ______ I.D. ______

HAMMER FALL ON:

CASING ______

SAMPLER 30"

DEPTH BELOW SURFACE	CASING BLOWS PER FOOT	SAMPLE NUMBER / DEPTHS BELOW SURFACE, FT.	BLOWS PER 6" ON SAMPLER 0-6"	6-12"	12-18"	18-24"	PROFILE CHANGE DEPTH / ELEV.	IDENTIFICATION OF SOILS / REMARKS
0	H	S-1	1	1	3	4		S-1: Light Brown cm$^+$f SAND, trace$^-$ Silt.
	O	0'-2'						
	L	S-2	7	7	7	5		S-2: Same as S-1.
	L	2'-4'						
	O	S-3	3	3	3	5		S-3: Dark Grey Silty CLAY, and$^+$ cf Sand.
	W	4'-6'						
		S-4	4	3	3	3		S-4: Dark Grey Silty CLAY, some$^-$ cf Sand.
	S	6'-8'						
	T	S-5	2	1	2	1		S-5: Dark Grey Silty CLAY, littl mf Sand.
10	E	8'-10'						
	M	S-6	1	3	50/6"			S-6: Same as S-5 (wood in tip).
		10'-13.5'						
	A							
	U							
	G							
	E	S-7	1	1	1	2		S-7: Dark Grey Silty CLAY, trace mf Sand.
	R	15'-17'						
20		S-8	1	2	3	3		S-8: Same as S-7 (organic).
		20'-22'						
		S-9	2	3	5	8		S-9: TOP: Same as S-7. BOT: Brown cm$^+$f SAND, trace$^+$ f Gravel, trace$^-$ Silt.
		25'-27'						
30'		S-10	12	17	21	28		S-10: Dark Brown and Brown cm$^+$f SAND, trace$^+$ f Gravel, trace$^-$ Silt.
		30'-32'					32.0	
								END OF BORING
40								

Soils Engineer: J.B. HELLER

Contractor: TESTWELL CRAIG TEST BORING COMPANY

ing Inspector: S.J. KOMSKI

Driller: S. BURNS

VISUAL IDENTIFICATION TERMS USED ED-6

	Clayey soils	At Ball Moisture
Clayey SILT	slight PI	Thread 1/4
SILT & CLAY	low PI.	Thread 1/8"
CLAY & SILT	medium PI	Thread 1/16"
Silty CLAY	high PI	Thread 1/32"

Relative Density (Dr) of granular soils	
Very loose	0-15 %
Loose	15-35%
Medium	35-65%
Dense	65-85%

Consistency of Clayey soils		
soft	(S)	0.1-0.5 tsf
firm	(F)	0.5-1.0 tsf
med. hard	(MH)	1.0-2.0 tsf
hard	(H)	2.0-4.0 tsf

Proportions used
trace = 1-10%
little = 10-20%
some = 20-35%
and = 35-50%

ASSOCIATES, PA

PARLIN EAST ONE · 3141 BORDENTOWN AVENUE · PARLIN, N.J. 08859

PORT MONMOUTH, NJ

PROJECT NO. 88E013

LOCATION SEE PLAN

OFFSET [illegible]

EPTH OF WATER _____ FT. W/ _____ FT. CASING OUT ON _____

EPTH OF WATER _____ FT. W/ ALL CASING OUT ON _____

DATE STARTED 6-14-88

DATE FINISHED 6-14-88

GROUND ELEV. +6.4'

GROUND WATER ELEV. _____

GHT OF HAMMER:

CASING _____ LBS SAMPLER 140 LBS

INSIDE LENGTH OF SAMPLER: 24 IN.

CASING: O.D. _____ I.D. _____

SAMPLER: O.D. 2" I.D. 1-3/8"

COUPLING: O.D. _____ I.D. _____

HAMMER FALL ON:

CASING _____

SAMPLER 30"

DEPTH BELOW SURFACE	CASING BLOWS PER FOOT	SAMPLE NUMBER / DEPTHS BELOW SURFACE, FT.	BLOWS PER 6" ON SAMPLER				PROFILE CHANGE DEPTH ELEV.
			0-6"	6-12"	12-18"	18-24"	
0	H	S-1	4	4	5	5	
	O	0'-2'				-	
	L	S-2	3	8	10	6	
	L	2'-4'					
	O	S-3	6	4	3	2	
	W	4'-6'					
		S-4	2	2	2	2	
	S	6'-8'					
	T	S-5	2	2	2	2	
10	E	8'-10'					
	M	S-6	1	WOH	WOH	1	
		10'-12'					
	A	S-7	WOH	WOH	WOH	1	
	U	12'-14'					
	G	S-8	WOH	2	4	3	
	E	14'-16'					
	R	S-9	4	6	7	8	
		16'-18'					18.0'
20							
30							
40							

IDENTIFICATION OF SOILS / REMARKS

S-1: Dark Grey and Brown Clayey SI little$^-$ mf Sand.

S-2: TOP: Same as S-1.
BOT: Dark Grey, Dark Brown and Brown cm$^+$f SAND, little f Gravel, little$^-$ Clayey Silt.

S-3: Dark Grey and Dark Brown Silty CLAY, trace$^-$ cf Sand.

S-4: Dark Grey Silty CLAY, trace cf Sand.

S-5: Same as S-4.

S-6: Dark Grey cm$^+$f SAND, little$^+$ Clay Silt.

S-7: TOP: Same as S-6.
BOT: Greenish Brown cm$^+$f SAND, little$^-$ Silt.

S-8: Same as S-7 (BOT).

S-9: Brown cm$^+$f SAND, trace Silt.

END OF BORING

Soils Engineer: J.B. HELLER

Contractor: TESTWELL CRAIG TEST BORING COMPANY

ling Inspector: S.J. KOMSKI

Driller: S. BURNS

VISUAL IDENTIFICATION TERMS USED

ED-7

	Clayey soils	At Ball Moisture	Relative Density (Dr) of granular soils		Consistency of Clayey soils			Proportions used
Clayey SILT	slight PI.	Thread 1/4"	Very loose	0-15 %	soft	(S)	0.1-0.5 tsf	trace = 1-10%
SILT & CLAY	low PI.	Thread 1/8"	Loose	15-35%	firm	(F)	0.5-1.0 tsf	little = 10-20%
CLAY & SILT	medium PI	Thread 1/16"	Medium	35-65%	med. hard	(MH)	1.0-2.0 tsf	some = 20-35%
Silty CLAY	high PI	Thread 1/32"	Dense	65-85%	[illegible]	[illegible]	[illegible]	[illegible]

PMP2

E1.4

DEPTH	BLOWS/0.5'	SAMPLE #	SOIL SYMBOL	SOIL AND ROCK CLASSIFICATION UNIFIED SOIL CLASSIFICATION	REMARKS
6	3	1		Brown, fine to medium SAND, trace Silt. (SP-SM)	[illegible]
	6				
2	40				
	-				
3	1	2		Dark gray to brown CLAY, slightly organic, fibrous. (CH)	[illegible]
	2				
4	2				
	2				
5	1	3			
	1			Tube	
6	2				
	2				
7	2	4			
	3				
8	5			Dark greenish-brown, fine to medium SAND, some Silt. (SM)	
	6				
9	2	⑤			
	3				
10	4				
	4				
11	5	6		Dark grey-green, fine to medium SAND, trace Silt. (SP-SM)	[illegible]
	8				
12	13				
	15				
13	5	7		Dark grey-green, fine SAND, little to some Silt. (SM)	
	7				
14	7			Dark greenish-brown, Silty CLAY. (CH)	
	12				
15	10	8		Dark grey-green, fine to medium SAND, trace Silt. (SP-SM)	
	15				
16	16				
	21				
17	8	⑨			
	19				
18	22			Medium grained, trace Gravel, trace Shell Fragments.	
	25				
19	11	10		Greenish-brown, medium SAND, trace Silt. (SP)	
	21				
20	24				
	29				
21	17	11			
	27				
22	35				
	29				
23	9	⑫		Greenish-brown, fine SAND, little Silt with red-brown stringers. (SM)	
	14				
24	14				
	7				

ED-8

Project #: MCTY-0033.00010-S11
Project Name: Port Monmouth
Church Street

T&M ASSOCIATES

Sheet 1 of 1
Boring # P-12
Location Sta. 54+34
Offset 46 Ft. Right

Depth of Water ____ ft w/ ____ casing out on ____
Depth of Water 6 ft w/ all casing out on ____
Date Started: 6/21/90
Finished:
Ground Elev: +7.6
Grnd Wtr Elev: +1.6

Weight of Hammer:
Casing ____ lbs. Sampler 140 lbs.
Inside Length of Sampler 24 in.
Hammer Fall on:
Sampler: 30"
Casing: Auger
Casing O.D. ____ I.D. ____
Sampler O.D. 2" I.D. 1-3/8"
Coupling O.D. ____ I.D. ____

Depth Below Surface	Casing Blows Per Ft.	Sample # Depths Below Surface, ft.	Blows Per 6" on Sampler 0-6"	6-12"	12-18	18-24	Profile Change Depth	Soil Identification / Remarks	
0	H	S-1	3	3	4	3		S-1	Dark brown fine SAND, little SILT, little
	O	0-2'							fine Gravel
	L	S-2	3	6	8	4		S-2	Top: Dark brown fine SAND, trace Silt
	L	2'-4'							Bottom: Brown med.-fine SAND, little Silt
5	O	S-3	3	6	8	12		S-3	Reddish brown fine SAND, little Silt
	W	4'-6'							
		S-4	12	10	9	9		S-4	Brown med.-fine SAND, little Silt, trace
	S	6'-8'							med.-fine Gravel
	T	S-5	5	7	14	17		S-5	Brown & Grey SILT & CLAY, lenses of fine
10	E	8'-10'							Sand
	M	S-6	2	5	10	13		S-6	Brown, fine SAND, little Silt
		10'-12'							
	A								
	U								
15	G								
	E	S-7	6	10	16	18		S-7	Light brown fine SAND, trace Silt
	R	15'-17'							46-46-1/2 Grey Clayey SILT
									46-1/2-47 Grey fine SAND, trace Silt
20									
25									
30									

Soils Engineer: Michael J. Guerriero
Drilling Inspector: Michael Steiner
Contractor: Testwell Craig Test Boring Co.
Driller: Stephan Burns

Visual Identification Terms Used

PLASTIC SOILS

Clayey SILT	slight PI	Thread 1/4"
SILT & CLAY	low PI	Thread 1/8"
CLAY & SILT	medium PI	Thread 1/16"
Silty CLAY	high PI	Thread 1/32"

GRANULAR SOILS

GRAVEL	-Coarse 1"-3"
	-Med. 3/8"-1"
	-Fine .078"-3/8"
SAND	-Coarse .023"-.078"
	-Med. .01"-.023"
	-Fine .003"-.01"
SILT	- .003" and finer

PROPORTIONS USED

trace = 1%-10%
little = 10%-20%
some = 20%-35%
and = 35%-50%

ED-9

El. 9

PW-11

DEPTH	BLOWS/0.5'	SAMPLE #	SOIL SYMBOL	SOIL AND ROCK CLASSIFICATION UNIFIED SOIL CLASSIFICATION	REMARKS
					[illegible]
1	1 4	1		Brown, fine SAND, some Silt, with Clayey Silt layers. (SM)	
2	4 3				▼
3	41 3	2		Brown, Clayey SILT, little Sand, trace Gravel. (ML-CL)	
4	4 5				
5	2 2	3			
6	2 6				
7	4 6	4		Brown SAND, some Silt, trace Gravel. (SM)	
8	6 7				
9	3 5	5		Gray, fine SAND, little Silt with thin brown to gray Silty Clay layers. (SM)	
10	5 10				
11	5 6	6			
12	7 11				
13	3 3	7		Yellow-brown SAND, little Silt. (SM)	
14	4 6				
15	2 3	8		Yellow-brown, Silty CLAY. (CL)	
16	4 7			Dark gray to black CLAY. (CL)	
17	5 6	9			
18	10 15				
19	8 14	10		Yellow-brown, fine to medium SAND, little Silt with Silt and Clay layers. (SM)	
20	21 23				
21	3 3	11			
22	6 8				
23	5 6	12			
24	10 11				

6
7
4
16

ED-10

PM-1

El. 3,9

DEPTH	BLOWS/0.5'	SAMPLE #	SOIL SYMBOL	SOIL AND ROCK CLASSIFICATION UNIFIED SOIL CLASSIFICATION	REMARKS
1	1 2	1		Dark brown, very fine SAND, some Silt. (SM)	
2	3 3				
3	2 1	2			
4	1 1				
5	4 4	3		Brown, fine SAND, little Silt. (SM)	
6	4 4				
7	2 3	4		Trace gravel.	
8	6 4				
9	3 8	5		Dark gray, fine SAND, little Silt. (SM)	
10	12 12				
11	5 5	6			
12	3 6				
13	4 4	7			
14	5 9				
15	5 7	⑧		Gray to brown CLAY. (CL)	
16	8 10				
17	4 5	9			
18	8 12				
19	4 16	10		Tan, fine to medium SAND, trace Silt. (SP)	
20	22 25				
21	11 12	11			
22	17 15			Dark gray, slightly organic, Silty CLAY. (CL)	
23	8 16	12			
24	17 23				

ED-11

PW-2

El 5.3

DEPTH	BLOWS/0.5'	SAMPLE #	SOIL SYMBOL	SOIL AND ROCK CLASSIFICATION UNIFIED SOIL CLASSIFICATION	REMARKS
1	1 2	1		Brown, fine SAND, trace Silt, trace rootlets, trace gravel. (SP)	
2	5 5				
3	5 4	2			
4	4 4			No organics, moist to wet.	▽
5	1 2	3		Light brown, wet.	
6	3 4				
7	3 3	4			
8	3 6				
9	5 5	5			
10	5 5				
11	2 2	6		Light brown, fine to medium SAND, trace to little Silt, wet. (SP-SM)	
12	4 5				
13	4 6	7		Light brown, fine to medium SAND, trace gravel, trace Silt, wet. (SP)	
14	9 9				
15	6 9	8		Light brown to yellowbrown with reddish mottling, fine to medium SAND, trace to little Silt, possible glauconite, trace highly weathered rock fragments. - (SP-SM)	
16	12 12				
17	5 10	9			
18	12 16				
19	5 9	10			
20	9 16				
21	9 24	11		Yellowish tan, fine SAND, trace Silt, wet, slightly laminated. (SP)	
22	35 40				
23	11 18	12		At 22.5', orange with trace to little Silt.	
24	27 33				

ED-12

PW-3

El 4.5

DEPTH	BLOWS/0.5'	SAMPLE #	SOIL SYMBOL	SOIL AND ROCK CLASSIFICATION UNIFIED SOIL CLASSIFICATION	REMARKS
1	2 11	1		Brown, fine SAND, some Silt. (SM)	
2	10 11				
3	7 3	②		Grey-green. (SM-SC)	
4	2 7				
5	1 1	3		Trace gravel.	
5	1 1				
7	0 0	④			
8	0 0				
9	1 1	5			
10	1 1				
11	2 1	6			
12	1 1				
13	0 0	⑦			
14	0 0				
15	2 2	8		Grey-green, fine SAND, little Silt. (SM)	
16	3 4				
17	2 5	9		Grey-green, fine to medium SAND, trace to little Silt. (SM-SP)	
18	7 5				
15	4 2	10			
20	3 5				
21	3 3	11		Grey-green, fine to coarse SAND, trace Silt and Gravel. (SP)	
22	11 5				
23	3 3	12			
24	4 6				

ED-13

PW-4

El 3.8

DEPTH	BLOWS/0.5'	SAMPLE #	SOIL SYMBOL	SOIL AND ROCK CLASSIFICATION UNIFIED SOIL CLASSIFICATION	REMARKS
1	1 1	1		Brown, very fine SAND, some Silt. (SM)	
2	1 1				
3	2 3	2			
4	3 5				
5	2 3	3		Gray-green, very fine to fine SAND, trace to little Silt. (SP-SM)	
6	3 5				
7	3 4	4			
8	3 7				
9	5 6	5		Yellow-brown, fine to medium SAND, trace Silt. (SP-SM)	
10	6 8				
11	3 4	6			
12	6 7				
13	4 5	7		Yellow-brown, fine SAND, little Silt. (SM)	
14	7 3				
15	2 4	⑧		Gray-green, very fine to fine SAND, trace Silt. (SP-SM)	
16	3 14				
17	5 6	9		Trace coarser red-brown lamination.	
18	6 10				
19	4 6	10			
20	13 14			Dark gray SILT and SAND layer from 18.5' to 18.8'	
21	5 7	11		Tan and Red-brown (orange), medium SAND, trace Silt. (SP)	
22	8 4				
23	2 5	12			
24	8 17			Little to some Gravel.	

ED-14

PM-5

El. 3.75

DEPTH	BLOWS/0.5'	SAMPLE #	SOIL SYMBOL	SOIL AND ROCK CLASSIFICATION UNIFIED SOIL CLASSIFICATION	REMARKS
1	1 1	1		Tan to gray-green, very fine to fine SAND, little Silt. (SM)	▼
2	4 6				
3	2 3	2			
4	5 6			Gray-green, fine SAND, trace Silt. (SP-SM)	
5	5 2	3			
6	3 3				
7	2 3	4			
8	3 5				
9	3 3	5			
10	5 8				
11	7 7	6		Tan to red-brown, fine to medium SAND. (SP)	
12	11 12				
13	5 7	7			
14	5 14				
15	14 16	⑧		Red-brown, fine to medium SAND, trace Silt and Gravel.	
16	22 24				
17	11 13	9			
18	14 9			Gray and black, laminated micaceous, very fine SAND. (SM)	≡
19	6 6	10			
20	10 14				
21	3 5	11			
22	6 14				
23	5 8	12			
24	15 18				

ED-15

PM-6

El. 5.0

DEPTH	BLOWS/0.5'	SAMPLE #	SOIL SYMBOL	SOIL AND ROCK CLASSIFICATION UNIFIED SOIL CLASSIFICATION	REMARKS
1	5 6	1		Light gray SAND and GRAVEL, trace Silt. (SW-GW)	
2	6 7				
3	3 4	2		Brown, fine SAND, trace Silt. (SP-SM)	
4	4 8				
5	4 4	3		Trace Gravel.	
6	4 6				
7	11 7	4			
8	10 12				
9	4 5	5			
10	8 6				
11	5 5	6		Greenish-brown.	
12	8 10				
13	6 18	7		Greenish-brown, fine to medium SAND. (SP)	
14	27 30				
15	12 18	8		Greenish-brown to reddish-brown, fine SAND, trace Silt. (SP-SM)	
16	20 34				
17	3 13	9			
18	34 22				
19	17 24	10		Reddish-brown, fine to coarse SAND, little Gravel, trace Silt. (SP)	
20	32 31				
21	3 12	11		Reddish-brown, fine to coarse SAND, little Silt. (SM)	
22	15 13			Black, stiff Silty CLAY. (CL)	
23	9 8	12		Tan to brown, very fine SAND and SILT. (SM-ML)	
24	15 22				

ED-16

PM-8

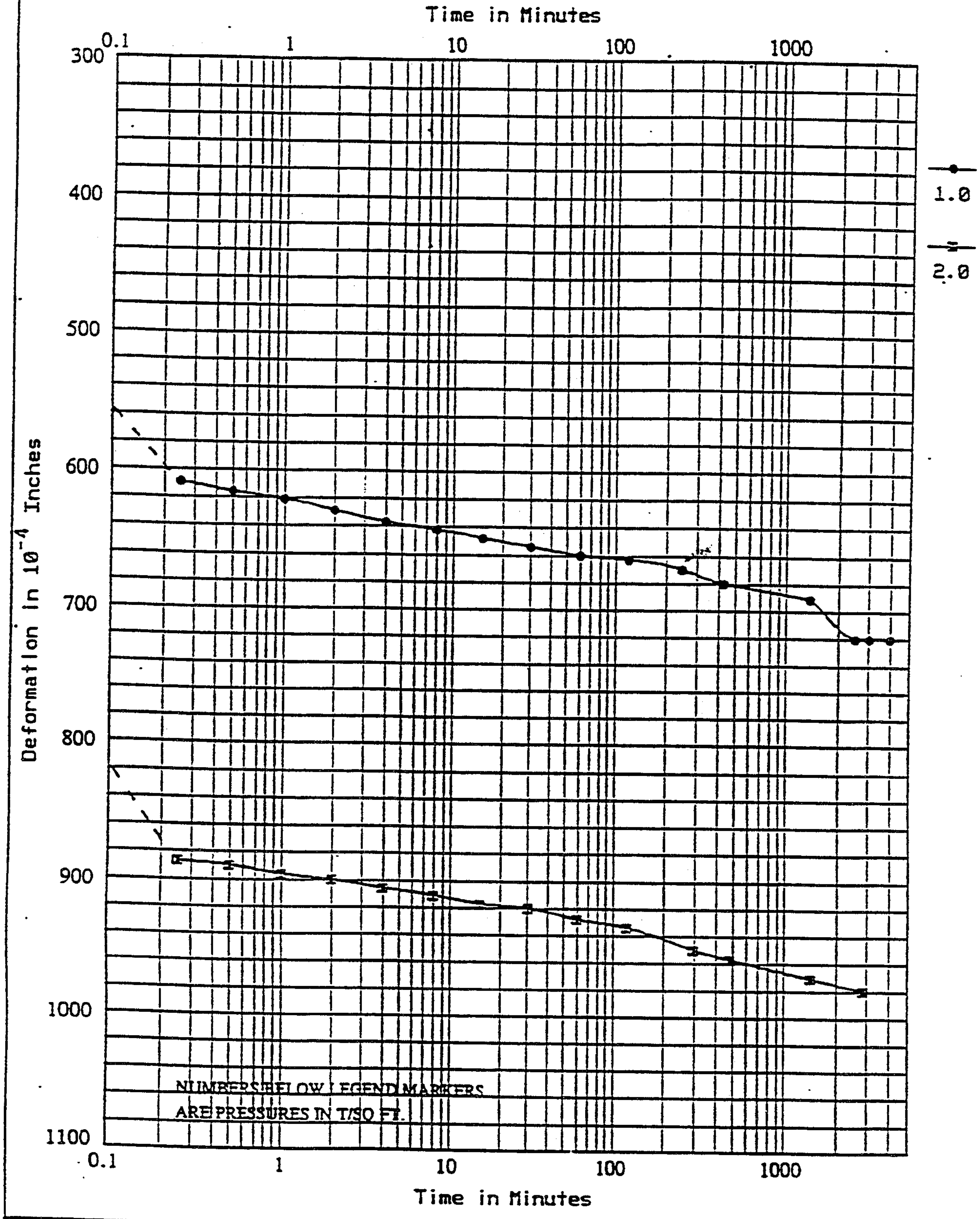
El. 6

DEPTH	BLOWS/0.5'	SAMPLE #	SOIL SYMBOL	SOIL AND ROCK CLASSIFICATION UNIFIED SOIL CLASSIFICATION	REMARKS
1	4 7	1		Brown, fine SAND, little to some Silt. (SM)	
2	10 14				
3	3 3	②		Little Silt.	
4	13 14				
5	6 5	3			▽
6	6 8			Greenish-brown, fine SAND, trace Silt. (SP-SM)	
7	3 3	④			
8	3 4				
9	3 5	5		Trace Gravel from 8.0' to 12.0'.	
10	7 8				
11	3 4	6			
12	8 5				
13	5 14	7		Brown, fine to medium SAND, trace Silt.	
14	18 19				
15	7 15	8			
16	12 14				
17	8 8	9			
18	12 14				
19	5 11	10			
20	14 15				
21	11 18	11			
22	20 27				
23	7 14	12			
24	20 25				

ED-17

WORK ORDER: 0275
REQUSITION: W31XNJ71694138

DEF ENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORA
COR. .F ENGINEERS, 611 SOUTH COBB DRIVE, MARIETTA, GA 30066



Project PORT MONMOUTH, NJ

Area

Lab No. 264/815

Boring No. PM-12

Sample No.

Depth (ft) 13.0-15.5

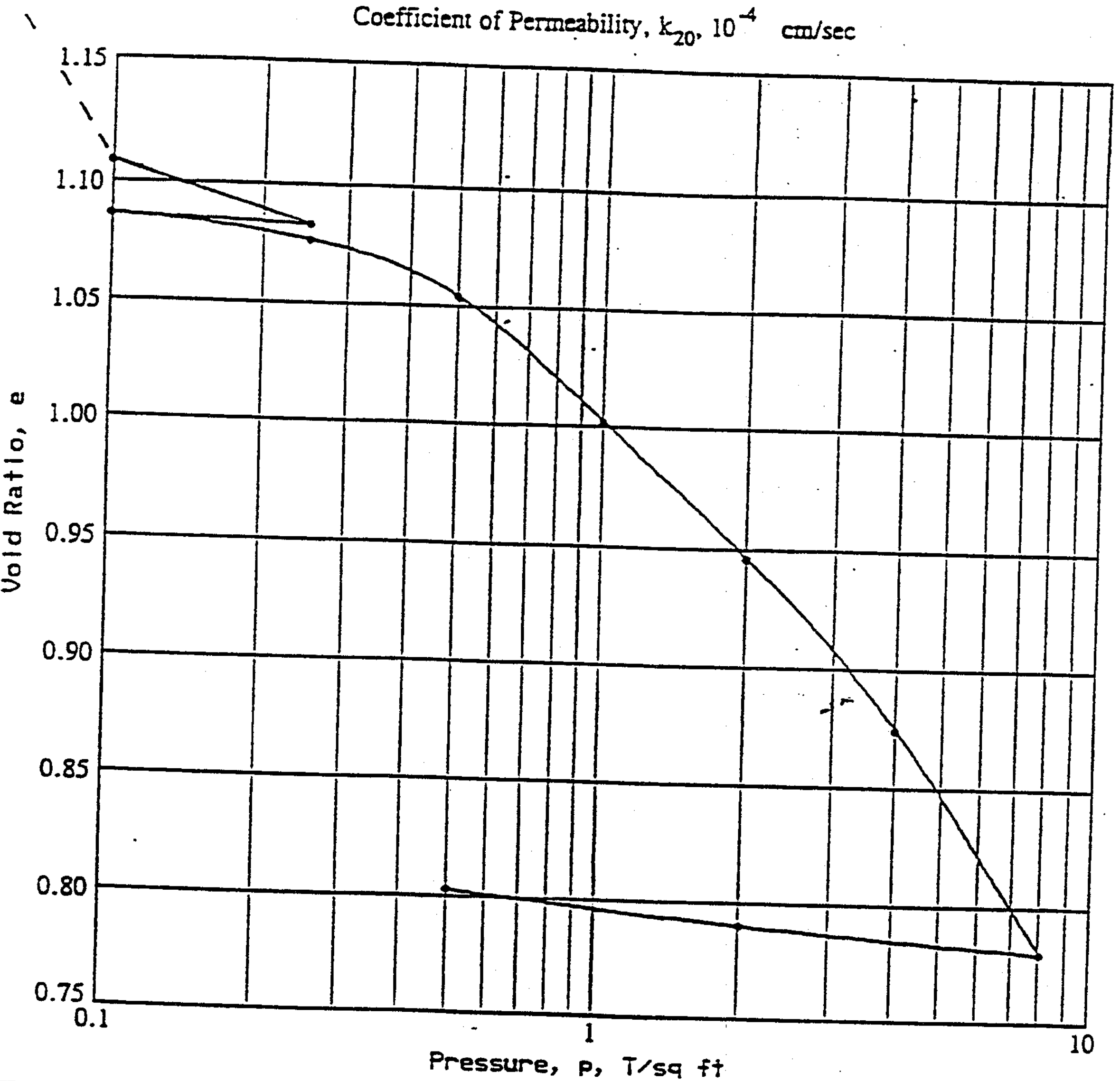
Date 08/04/97

CONSOLIDATION TEST - TIME CURVES

ED-18

WORK ORDER: 0275
REQUSITION: W31XNJ71

ENGINEERS, 611 SOUTH COBB DRIVE, MARIETTA, G



Type of Specimen UNDISTURBED			Before Test			After Test	
Diam. 2.500 in.	Height 1.000 in.	Water Content	w_o	42.3 %	w_f	32.3 %	
Overburden Pressure, p_o	T/sq ft	Void Ratio	e_o	1.158	e_f	0.803	
Preconsol. Pressure, p_c	T/sq ft	Saturation	S_o	90.9 %	S_f	100.0 %	
Compression Index, C_c		Dry Density	γ_d	72.0 lb/ft^3			

Classification GRAY, SILTY SAND (SM), WITH A TRACE OF MICA.

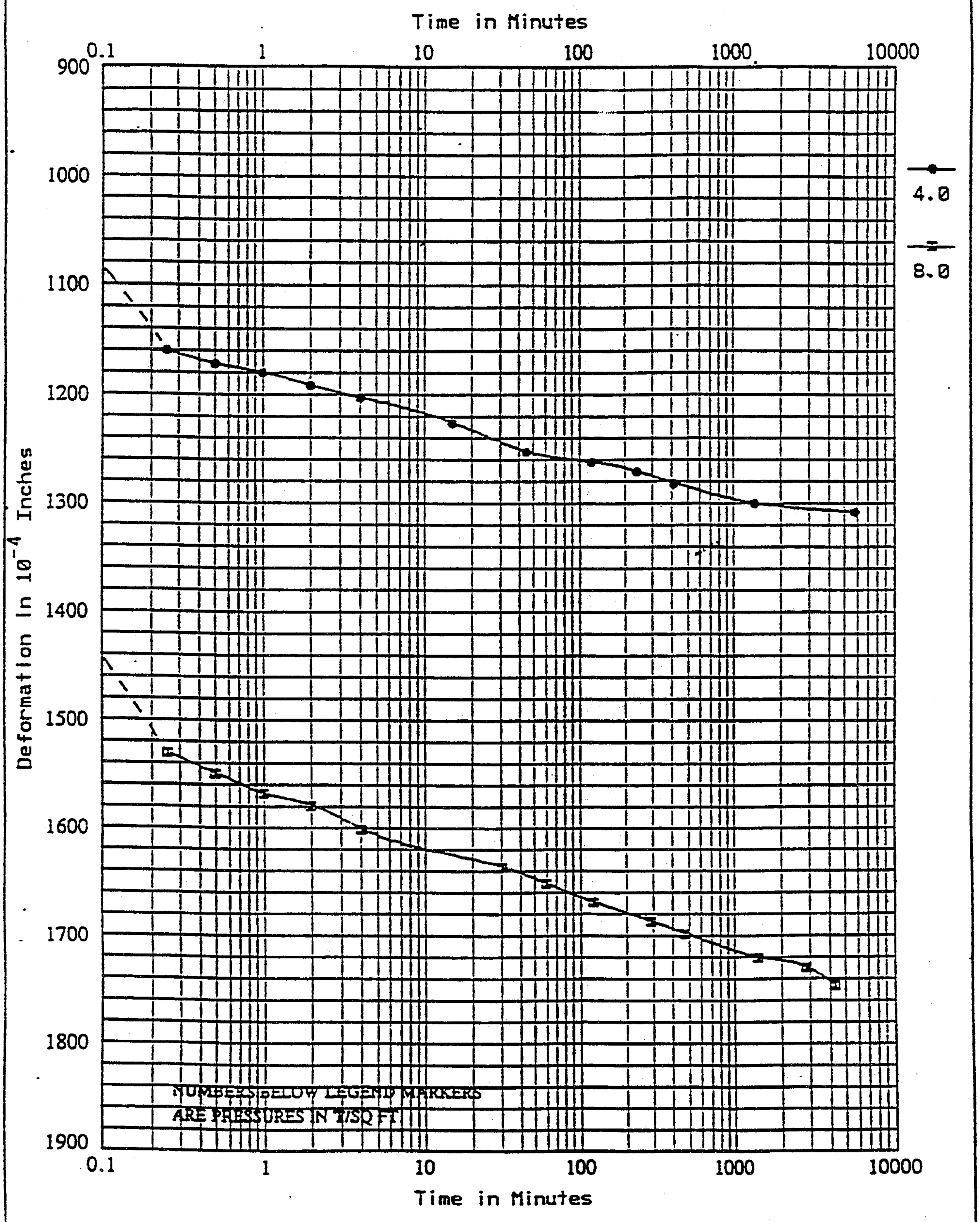
LL		G_s	2.49	Project PORT MONMOUTH, NJ
PL		D_{10}		

Remarks 1. SPECIMEN FLOODED		Lab No. 264/815
THROUGHOUT TEST. 2. TIME CURVES	Boring No. PM-12	Sample No.
FURNISHED FOR LOADS OF 1.0,	Depth (ft) 13.0-15.5	Date 08/04/97
2.0, 4.0 & 8.0 TSF.	CONSOLIDATION TEST REPORT	

ED-19

WORK ORDER: 0275
REQUISITION: W31XNJ71694

SOUTH ATLANTIC DIVISION LABORAT
ENGINEERS, 611 SOUTH COBB DRIVE, MARIETTA, GA
CORI



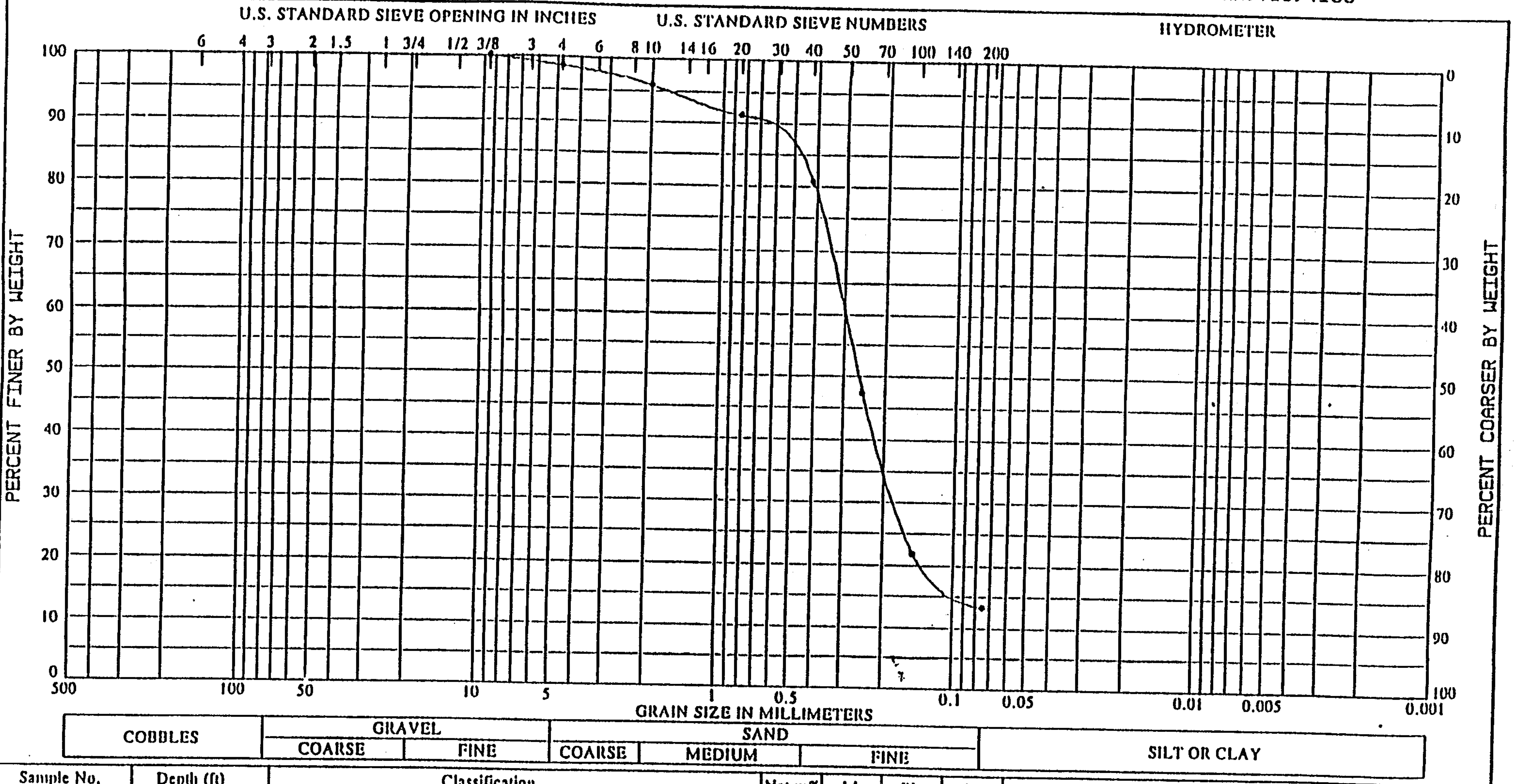
Project PORT MONMOUTH, NJ			
Area		Lab No. 264/815	
Boring No. PM-12	Sample No.	Depth (ft) 13.0-15.5	Date 08/04/97

CONSOLIDATION TEST - TIME CURVES

ED-20

DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE, MARIETTA, GA. 30060

WORK ORDER: 8275
REQUISITION: W31XNJ71694138



Sample No.	Depth (ft)	Classification	Nat w%	LL	PL	PI
	2.0 - 4.0	CLASSIFICATION NOT REQUESTED. (SP-SM)	16.2			

Project PORT MONMOUTH, NJ

Lab No. 264/816

Boring No. NW-1

Date 08/06/97

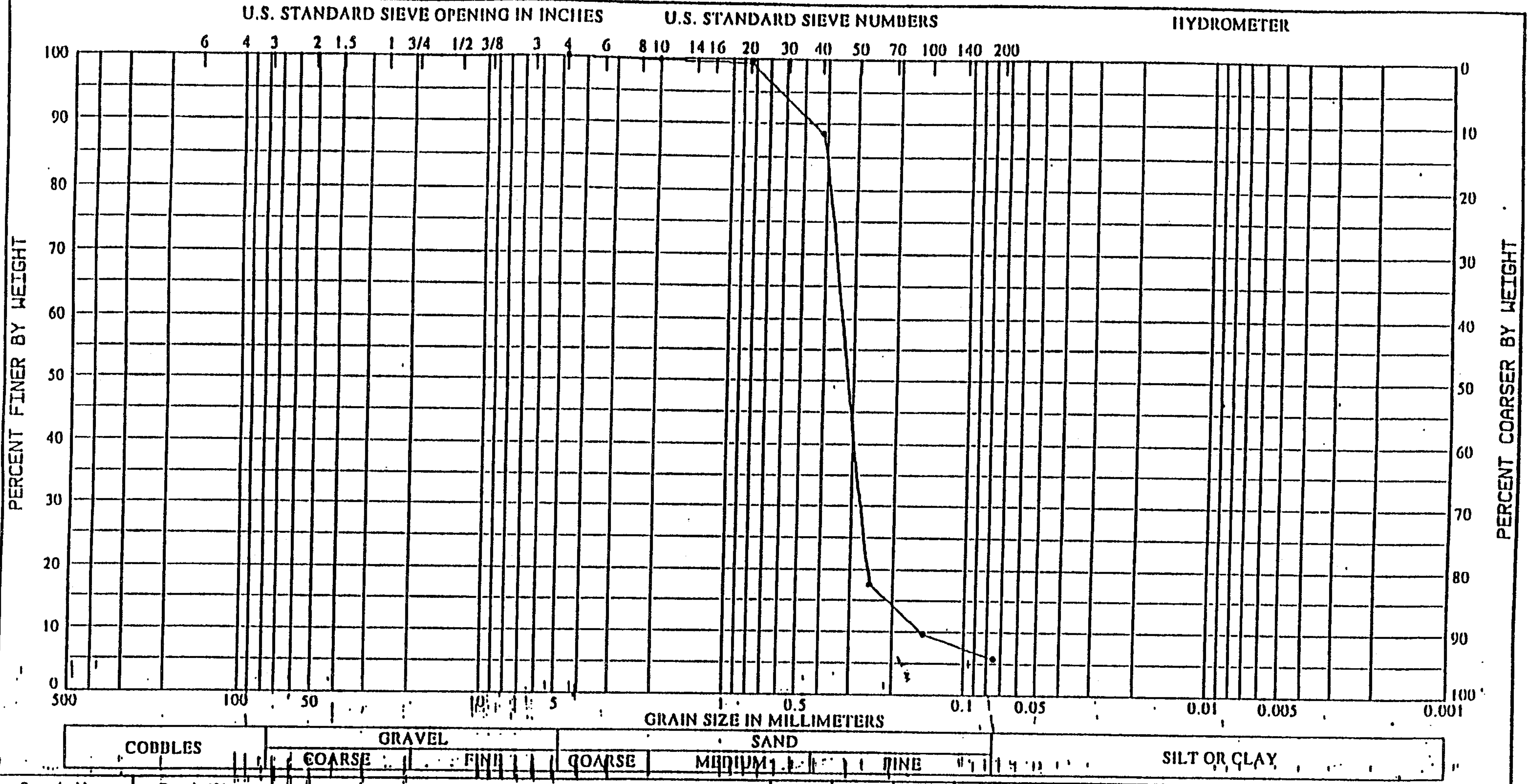
GRADATION CURVES

ED-21

DEPAR ENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE, MARIETTA, GA. 30060

WORK ORDER: 8275
REQUISITION: W31XNJ71694138

ED-22



Sample No.	Depth (ft)	Classification	Nat w%	LL	PL	PI
	20.0 - 22.0	CLASSIFICATION NOT REQUESTED. (SP)	20.8			

Project PORT MONMOUTH, NJ

Lab No. 264/817

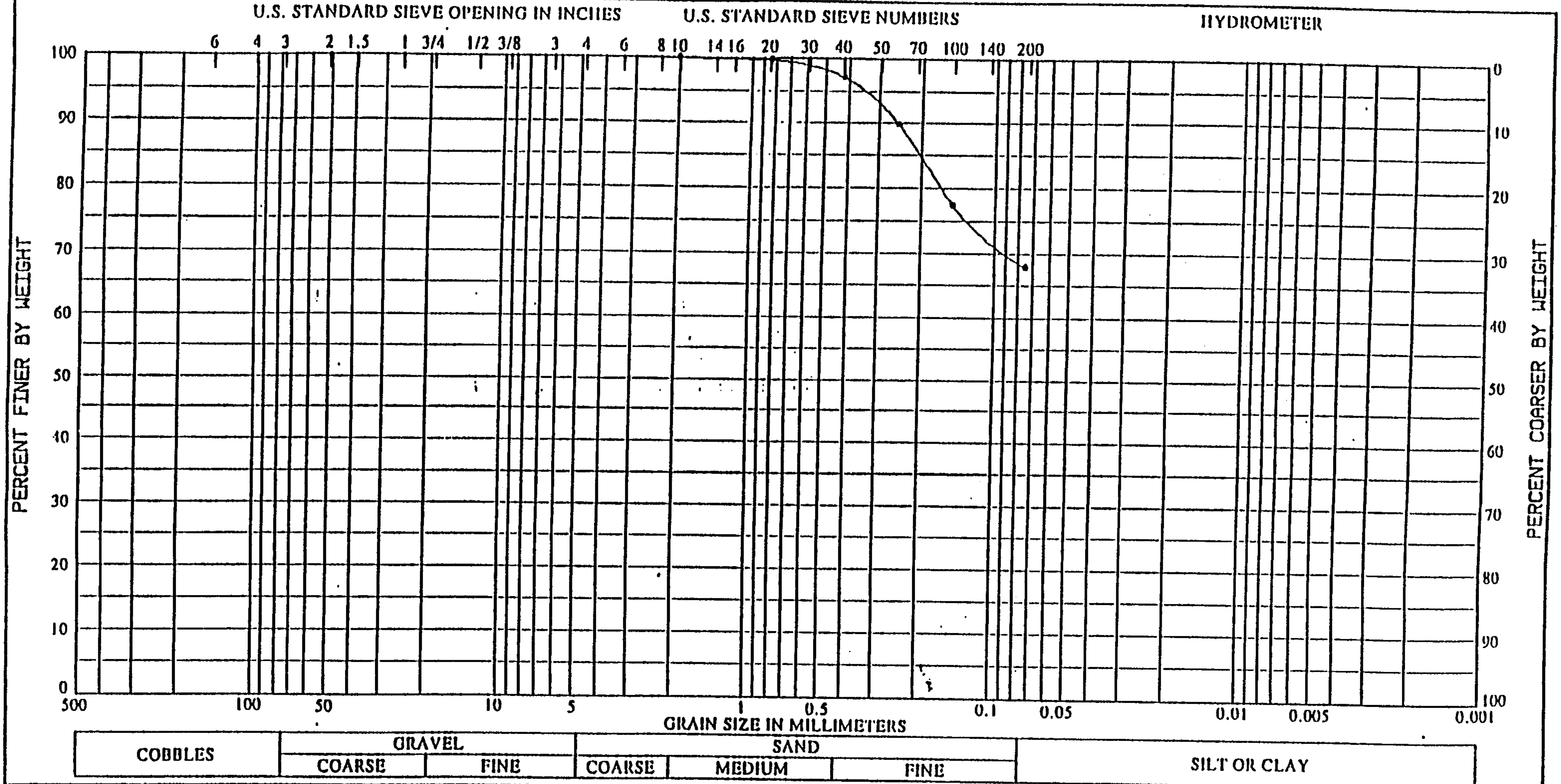
Boring No. NW-1

Date 08/06/97

GRADATION CURVES

DEPAR. .NT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE, MARIETTA, GA. 30060

WORK ORDER: 8275
REQUISITION: W31XNJ71694138



Sample No.	Depth (ft)	Classification	Nat w%	LL	PL	PI
	14.0 - 16.0	GRAY, SILTY LEAN CLAY (CL), WITH SOME SAND SIZES, MICACEOUS	26.0	30	22	8

Project PORT MONMOUTH, NJ

Lab No. 264/818

Boring No. PM-1

Date 08/06/97

GRADATION CURVES

ED-23

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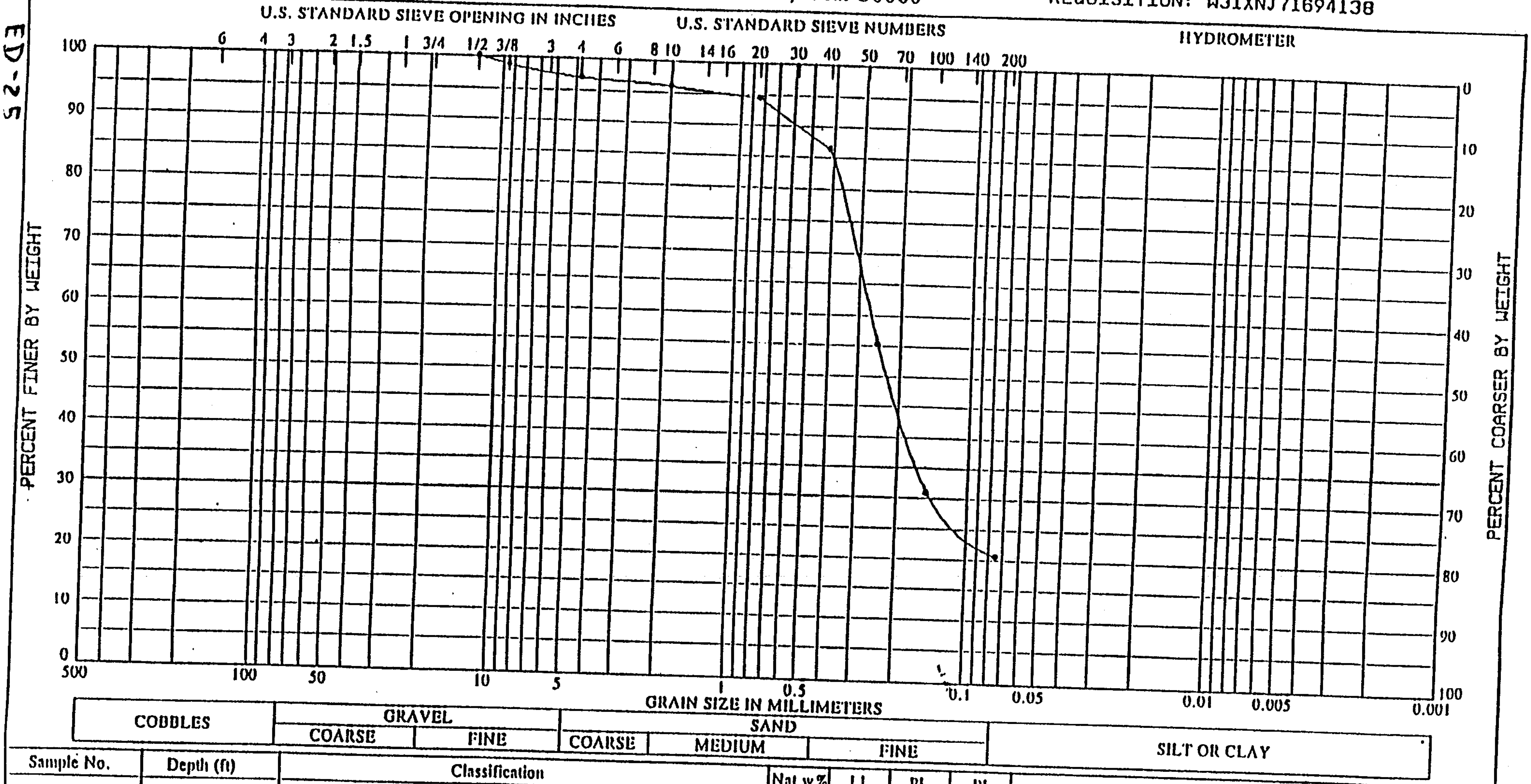
DEPART OF THE ARMY, SOUTH ATLANTIC DIVISIO ORATORY
CORPS OF NGINEERS, 611 SOUTH COBB DRIVE, MARIETTA, GA. 30060

WORK ORDER: 8275
REQUISITION: W31XNJ71694138

ED-25



Sample No.	Depth (ft)	Classification	Nat w%	LL	PL	PI
	2.0 - 4.0	CLASSIFICATION NOT REQUESTED. (SM)	17.1			

Project PORT MONMOUTH, NJ

Lab No. 264/820

Boring No. PM-3

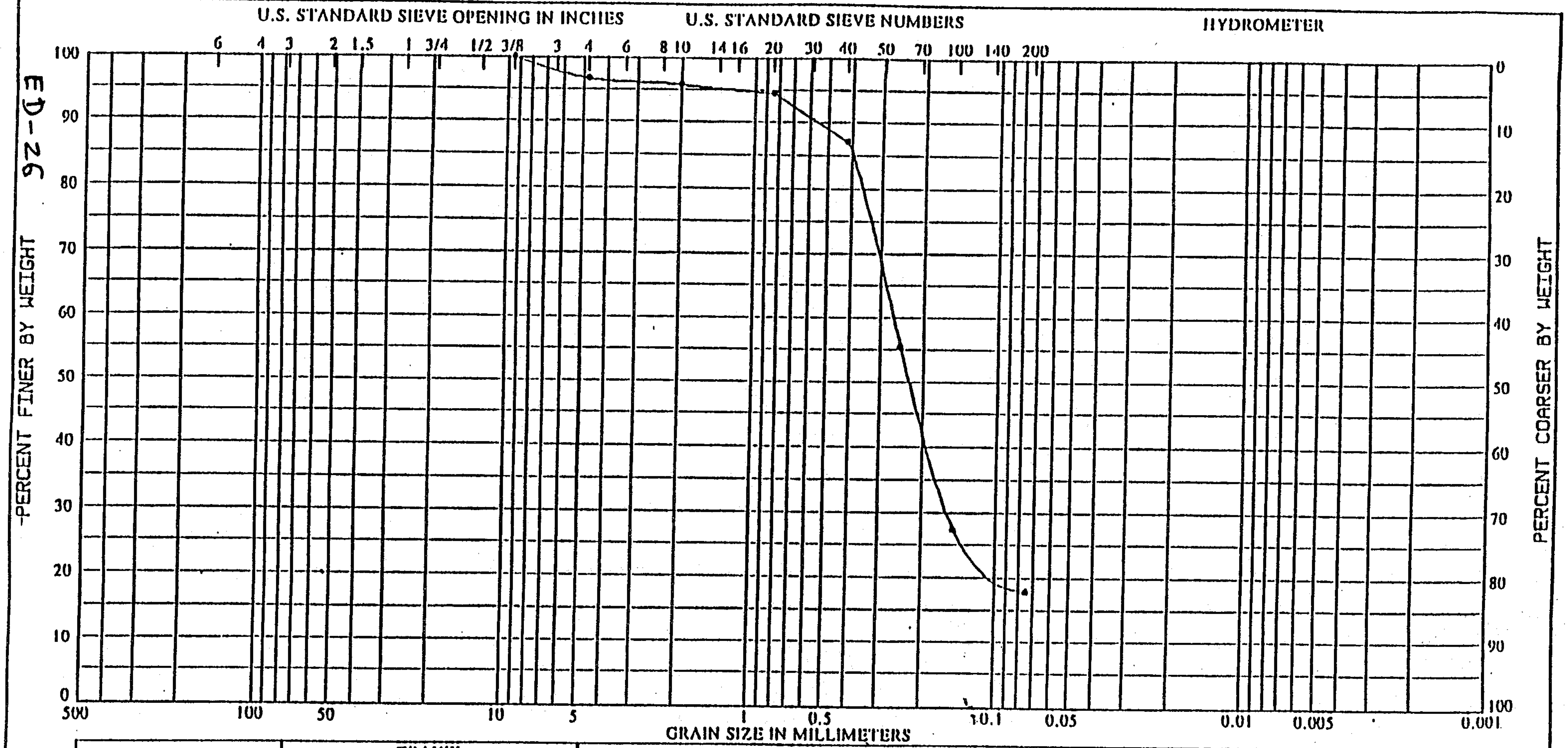
Date 08/06/97

GRADATION CURVES

DEPART T OF THE ARMY, SOUTH ATLANTIC DIVISION L RATORY
CORPS O GINEERS, 611 SOUTH COBB DRIVE, MARIETTA, GA. 30060

WORK ORDER: 0275
REQUISITION: W31XNJ71694130

ED-26



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Sample No.	Depth (ft)	Classification	Nat w%	LL	PL	PI
	6.0 - 8.0	CLASSIFICATION NOT REQUESTED. (SM)	22.3			

Project PORT MONMOUTH, NJ

Lab No. 264/021

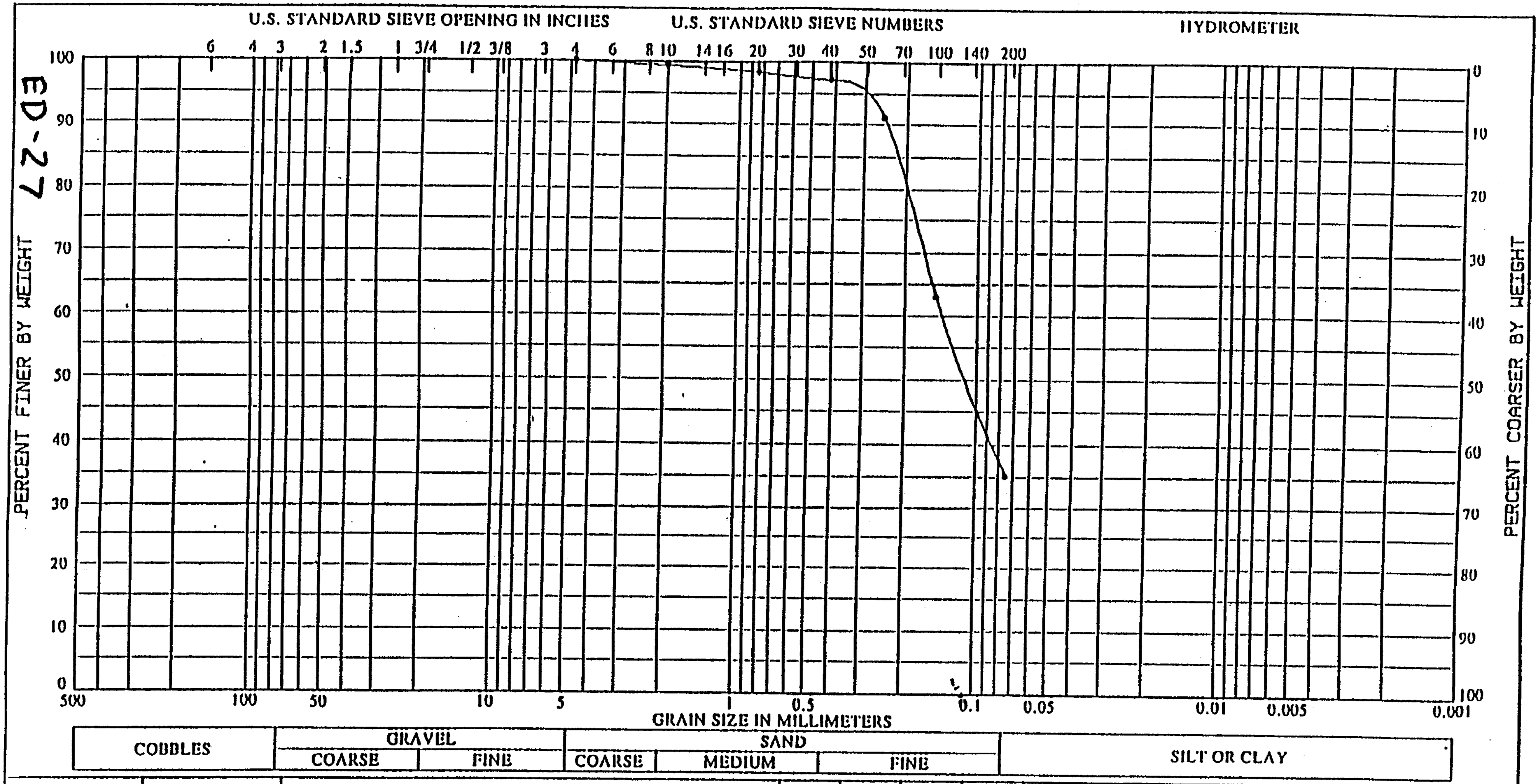
Boring No. PM-3

Date 08/06/97

GRADATION CURVES

DEPARTM' OF THE ARMY, SOUTH ATLANTIC DIVISION L. ƆRATORY
CORPS OF .GINEERS, 611 SOUTH COBB DRIVE, MARIETTA, GA. 30060

WORK ORDER: 0275
REQUISITION: W31XNJ71694138



ED-27

Sample No.	Depth (ft)	Classification	Nat w%	LL	PL	PI
	12.0 - 14.0	CLASSIFICATION NOT REQUESTED. (SM)	30.1			

Project PORT MONMOUTH, NJ

Lab No. 264/822

Boring No. PM-3

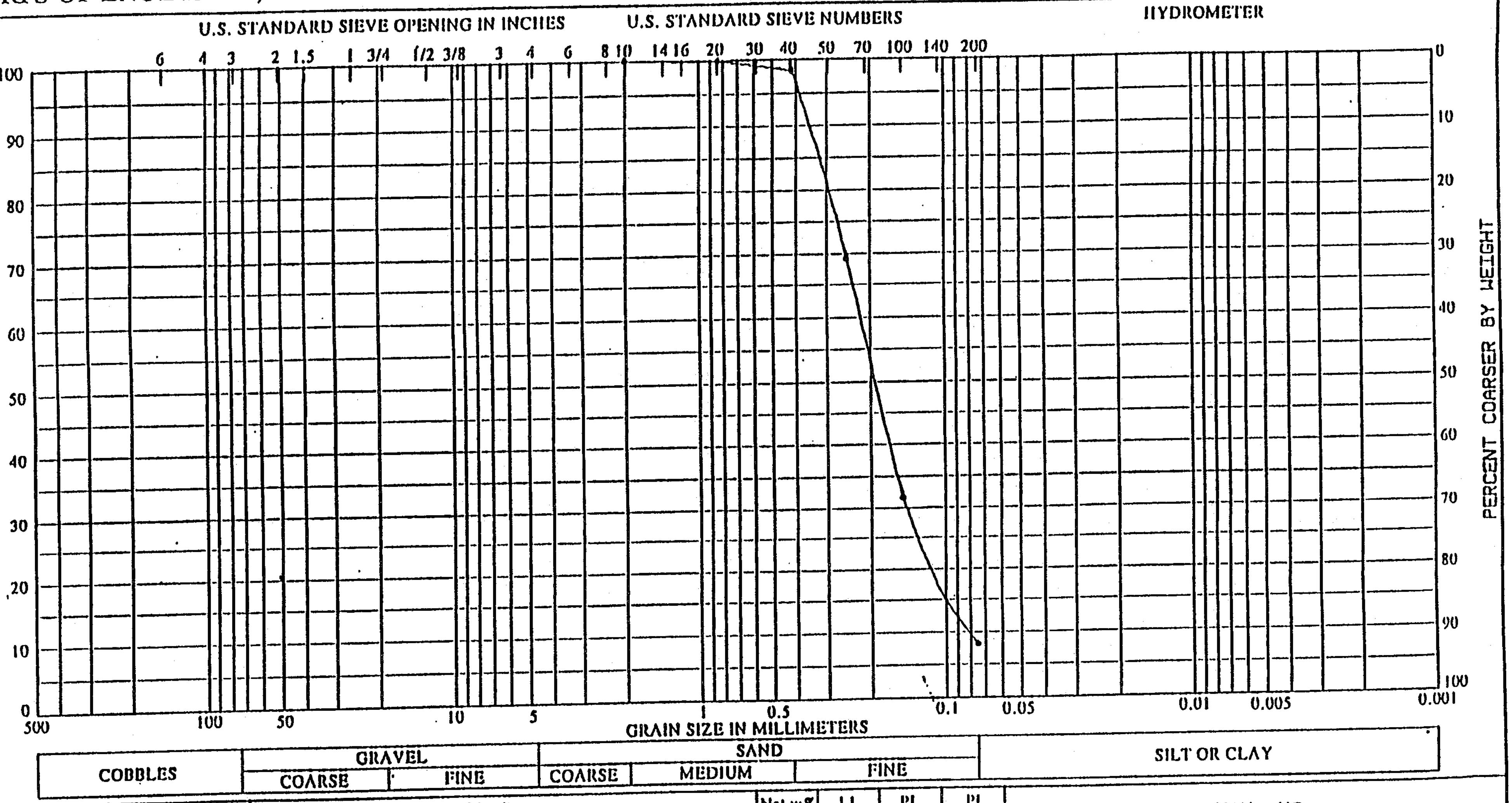
Date 08/06/97

GRADATION CURVES

DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE, MARIETTA, GA. 30060

WORK ORDER: 8275
REQUISITION: W31XNJ71694138

ED-28



Sample No.	Depth (ft)	Classification	Nat w%	LL	PL	PI
	14.0 - 16.0	CLASSIFICATION NOT REQUESTED. (SP-SM)	25.5			

Project PORT MONMOUTH, NJ

Lab No. 264/823

Boring No. PM-4

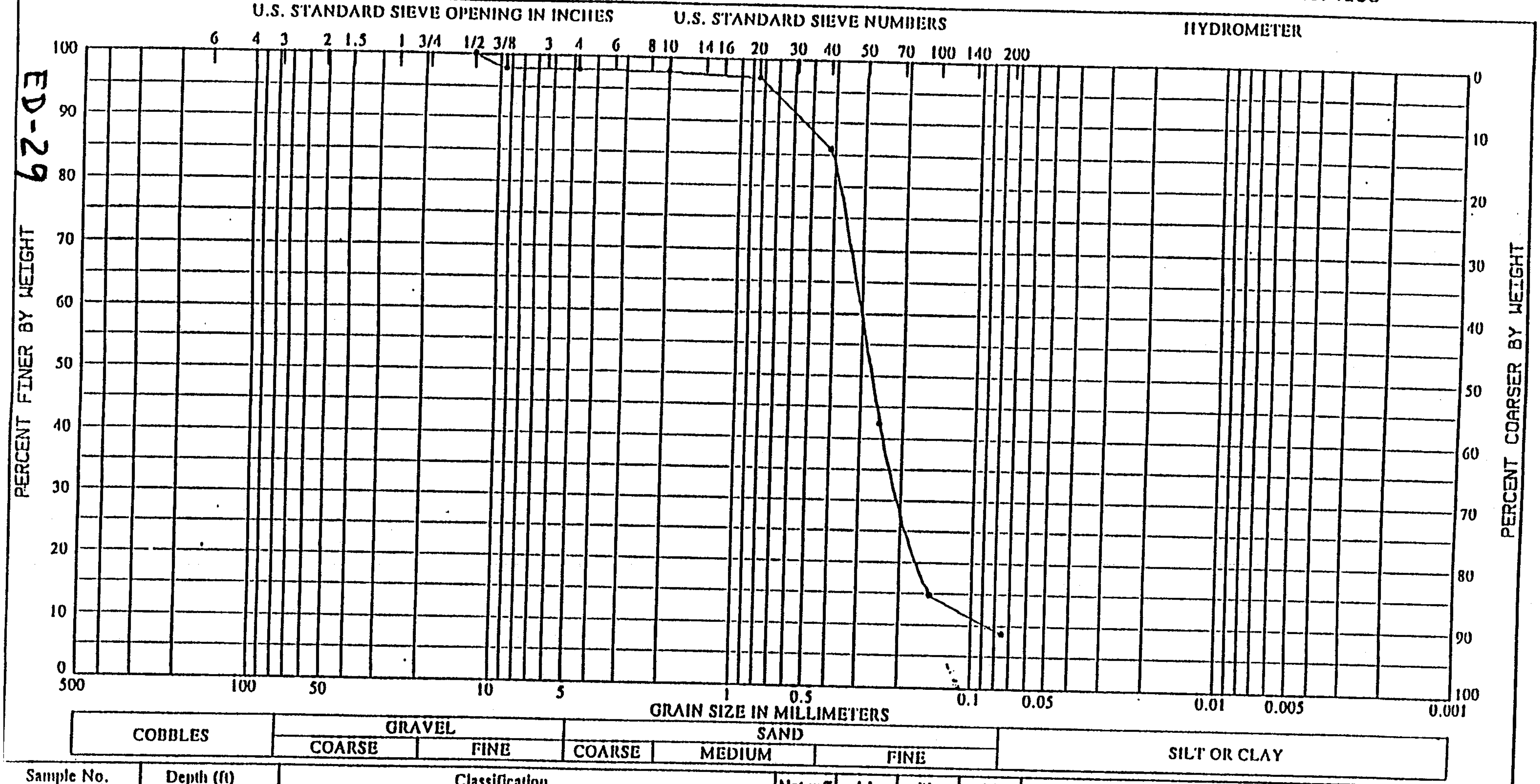
Date 08/06/97

GRADATION CURVES

DEPARTM OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE, MARIETTA, GA. 30060

WORK ORDER: 8275
REQUISITION: W31XNJ71694138

ED-29



Sample No.	Depth (ft)	Classification	Nat w%	LL	PL	PI
	14.0 - 16.0	CLASSIFICATION NOT REQUESTED. (SP-SM)	20.1			

Project PORT MONMOUTH, NJ

Lab No. 264/824

Boring No. PM-5

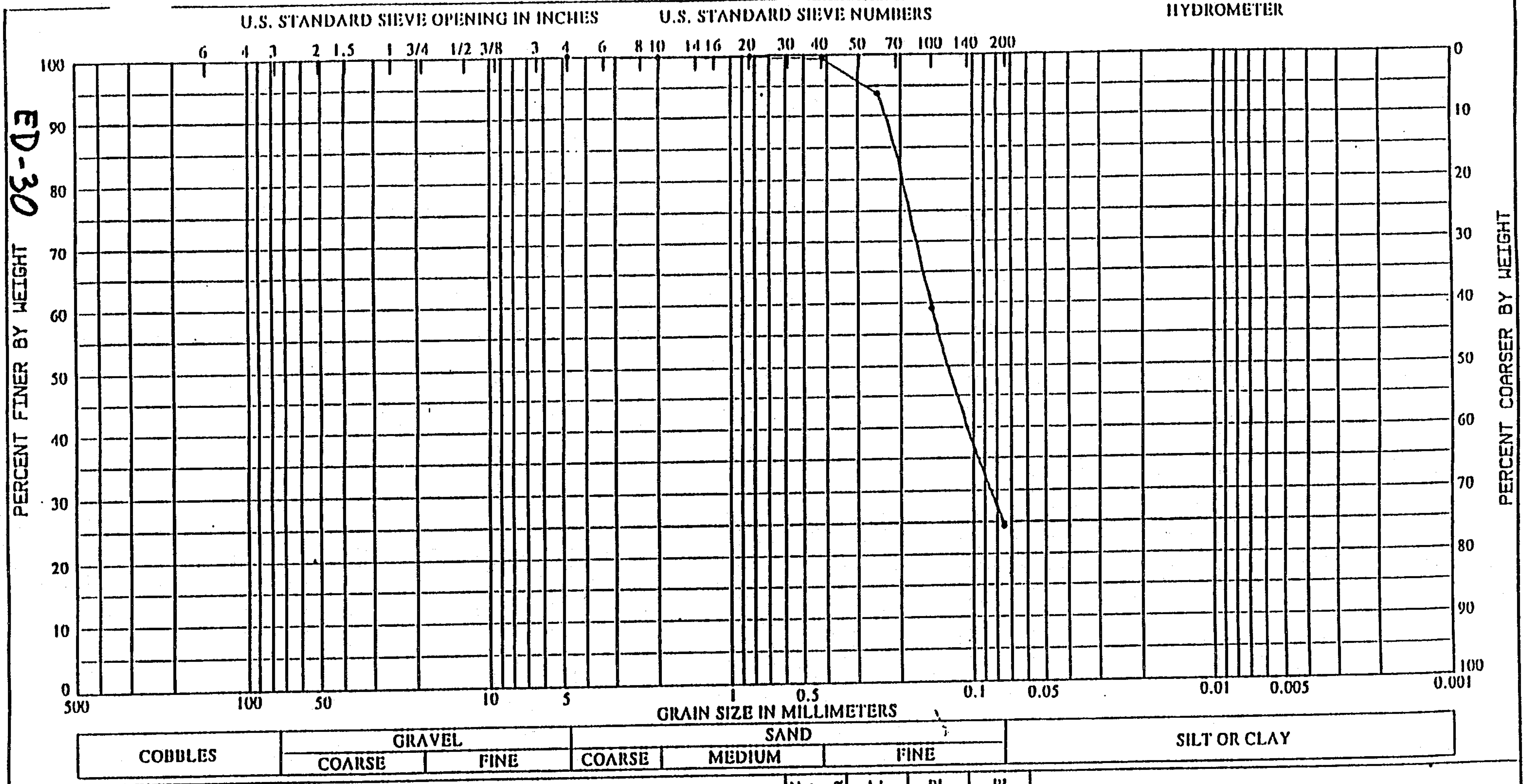
Date 08/06/97

GRADATION CURVES

NT OF THE ARMY, SOUTH ATLANTIC DIVISION ORATORY
CORPS C NGINEERS, 611 SOUTH COBB DRIVE, MARIETT . 30060

WORK ORDER: 0275
REQUISITION: W31XNJ71694130

ED-30



Sample No.	Depth (ft)	Classification	Nat w%	LL	PL	PI
	10.0 - 12.0	CLASSIFICATION NOT REQUESTED. (SM)	21.3			

Project PORT MONMOUTH, NJ

Lab No. 264/825

Boring No. PM-6

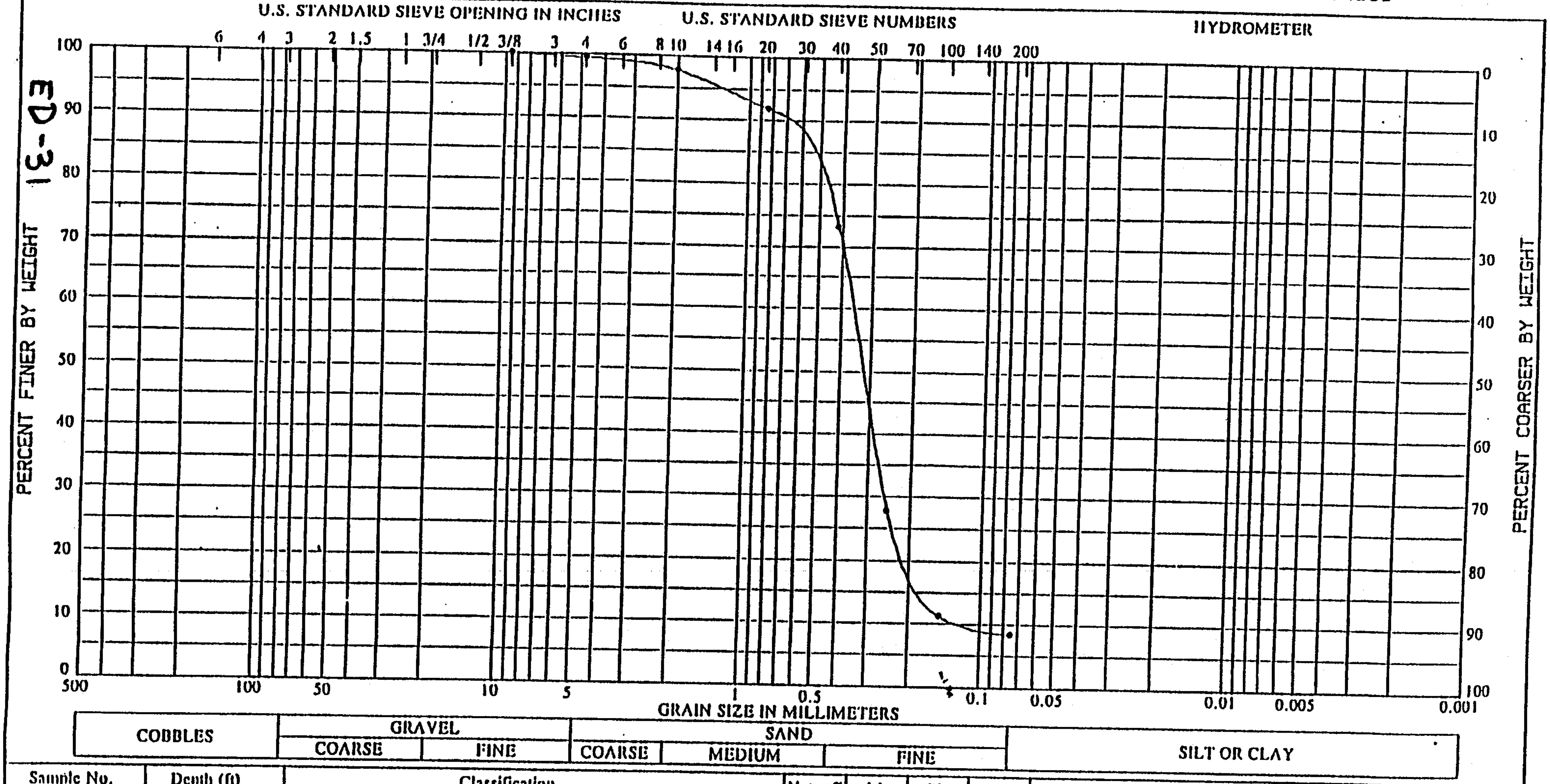
Date 08/06/97

GRADATION CURVES

DEPART T OF THE ARMY, SOUTH ATLANTIC DIVISION L ORATORY
CORPS O NGINEERS, 611 SOUTH COBB DRIVE, MARIETTA, GA. 30060

WORK ORDER: 8275
REQUISITION: W31XNJ71694138

ED-31



Sample No.	Depth (ft)	Classification	Nat w%	LL	PL	PI
	2.0 - 4.0	CLASSIFICATION NOT REQUESTED. (SP-SM)	9.6			

Project PORT MONMOUTH, NJ

Lab No. 264/826

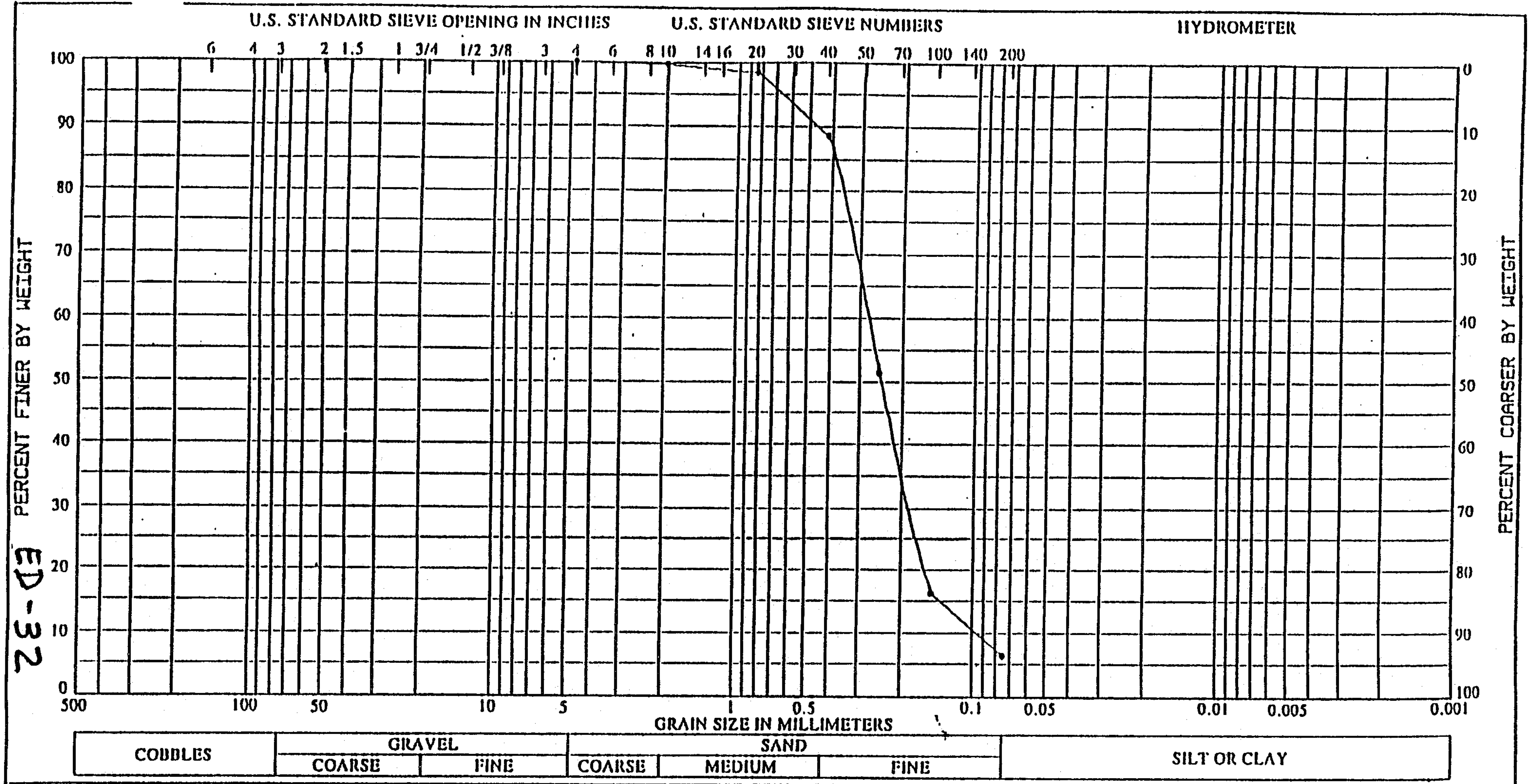
Boring No. PM-8

Date 08/06/97

GRADATION CURVES

DEPARTT 'T OF THE ARMY, SOUTH ATLANTIC DIVISION)RATORY
CORPS O 'GINEERS, 611 SOUTH COBB DRIVE, MARIETTA, .. 30060

WORK ORDER: 8275
REQUISITION: W31XNJ71694138



ED-32

Sample No.	Depth (ft)	Classification	Nat w%	LL	PL	PI
	6.0 - 8.0	CLASSIFICATION NOT REQUESTED. (SP)	25.9			

Project PORT MONMOUTH, NJ

Lab No. 264/827

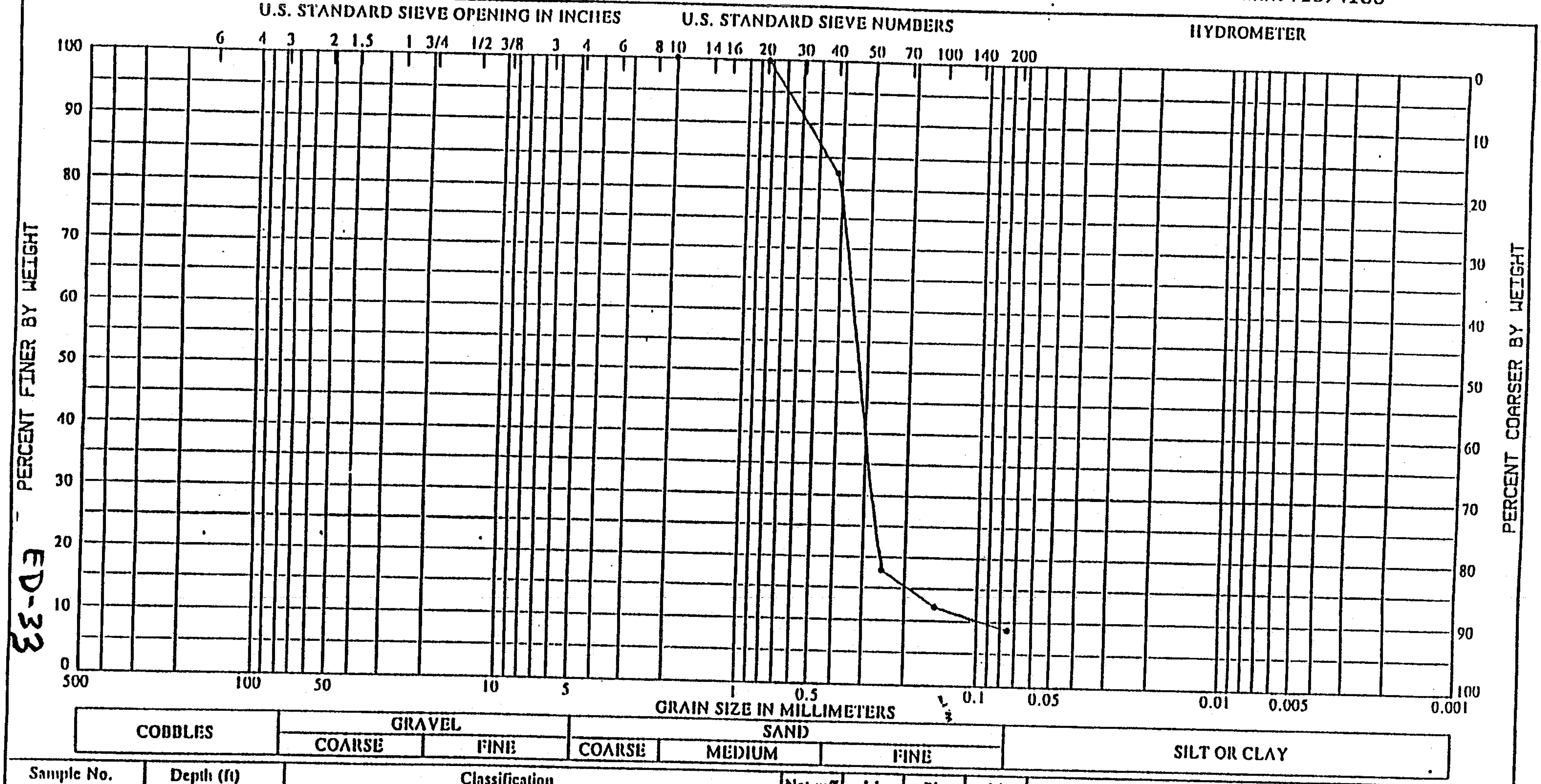
Boring No. PM-8

Date 08/06/97

GRADATION CURVES

DEPART OF THE ARMY, SOUTH ATLANTIC DIVISIO RATORY
CORPS GINEERS, 611 SOUTH COBB DRIVE, MARIET , . 30060

WORK ORDER: 8275
REQUISITION: W31XNJ71694138



ED-33

Sample No.	Depth (ft)	Classification	Nat w%	LL	PL	PI
	12.0 - 14.0	CLASSIFICATION NOT REQUESTED. (SP-SM)	19.9			

Project PORT MONMOUTH, NJ

Lab No. 264/828

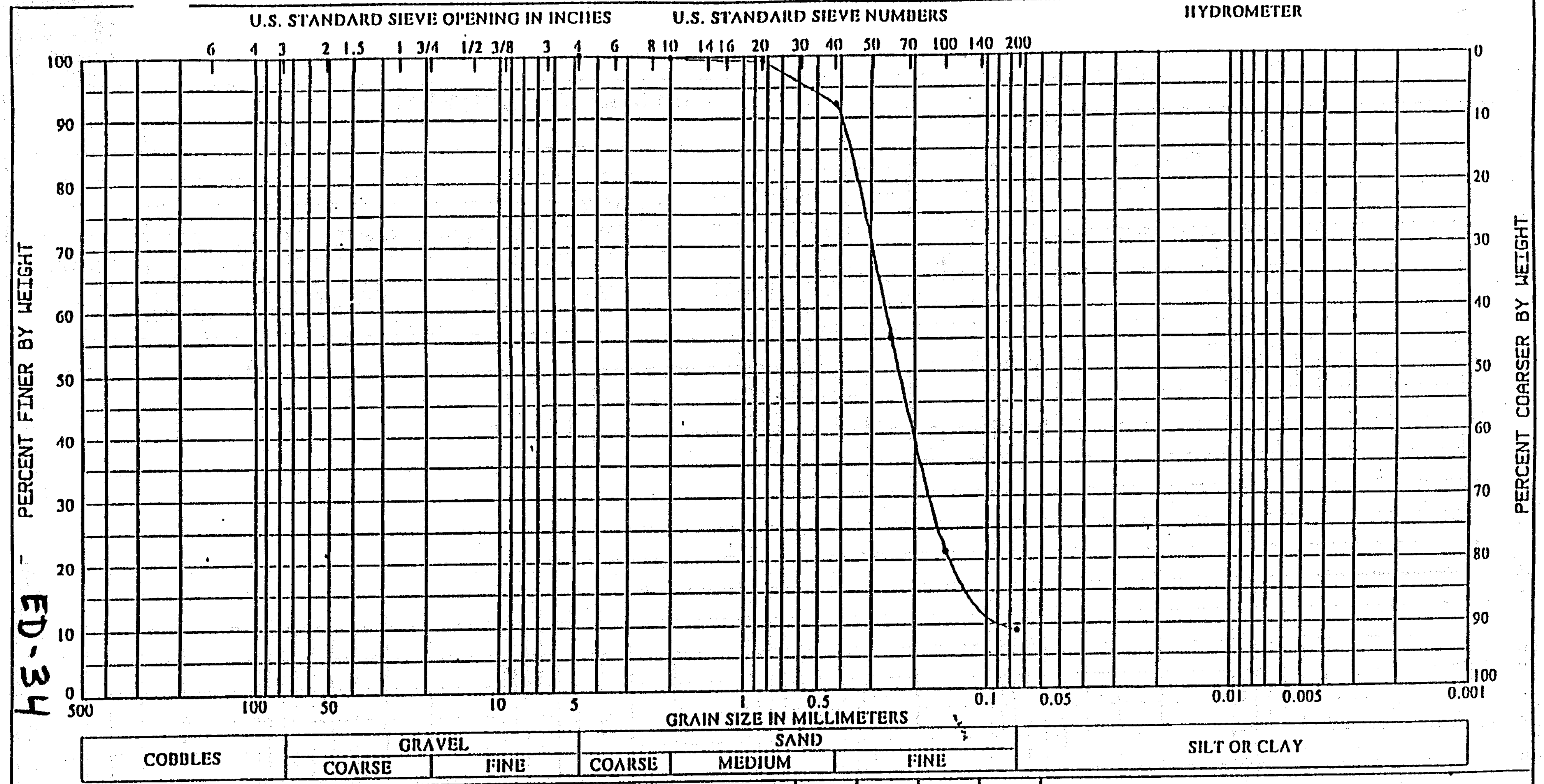
Boring No. PM-13

Date 08/06/97

GRADATION CURVES

DEPARTM 'T OF THE ARMY, SOUTH ATLANTIC DIVISION 'RATORY
CORPS O GINEERS, 611 SOUTH COBB DRIVE, MARIETTA, . 30060

WORK ORDER: 8275
REQUISITION: W31XNJ71694138



ED-34

Sample No.	Depth (ft)	Classification	Nat w%	LL	PL	PI
	4.0 - 6.0	CLASSIFICATION NOT REQUESTED. (SP-SM)	21.3			

Project PORT MONMOUTH, NJ

Lab No. 264/829

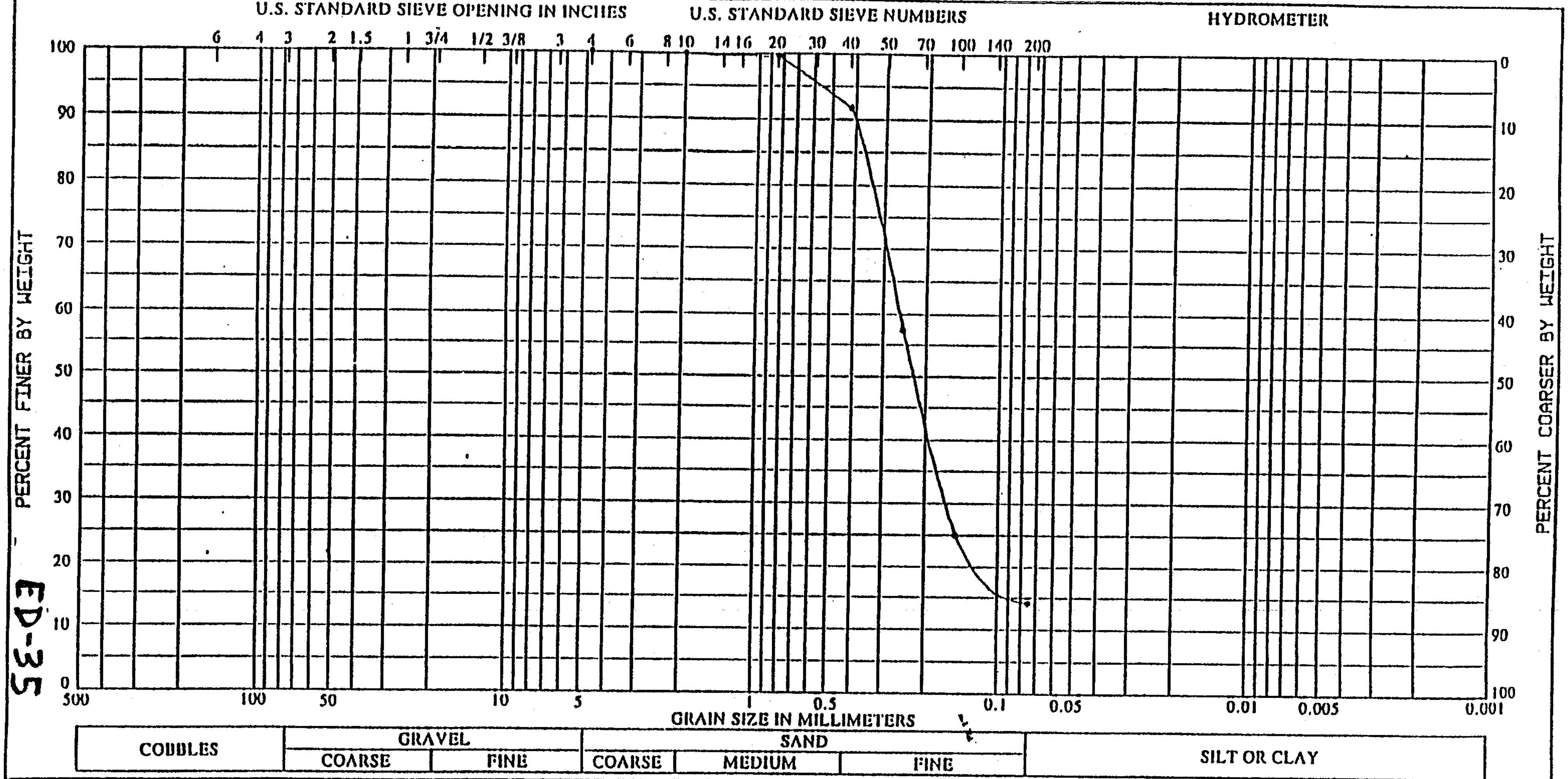
Boring No. PM-14A

Date 08/06/97

GRADATION CURVES

DEPARTI T OF THE ARMY, SOUTH ATLANTIC DIVISIO RATORY
CORPS O GINEERS, 611 SOUTH COBB DRIVE, MARIETT .. 30060

WORK ORDER: 8275
REQUISITION: W31XNJ71694138



ED-35

Sample No.	Depth (ft)	Classification	Nat w%	LL	PL	PI
	4.0 - 6.0	CLASSIFICATION NOT REQUESTED. (SM)	21.3			

Project PORT MONMOUTH, NJ

Lab No. 264/830

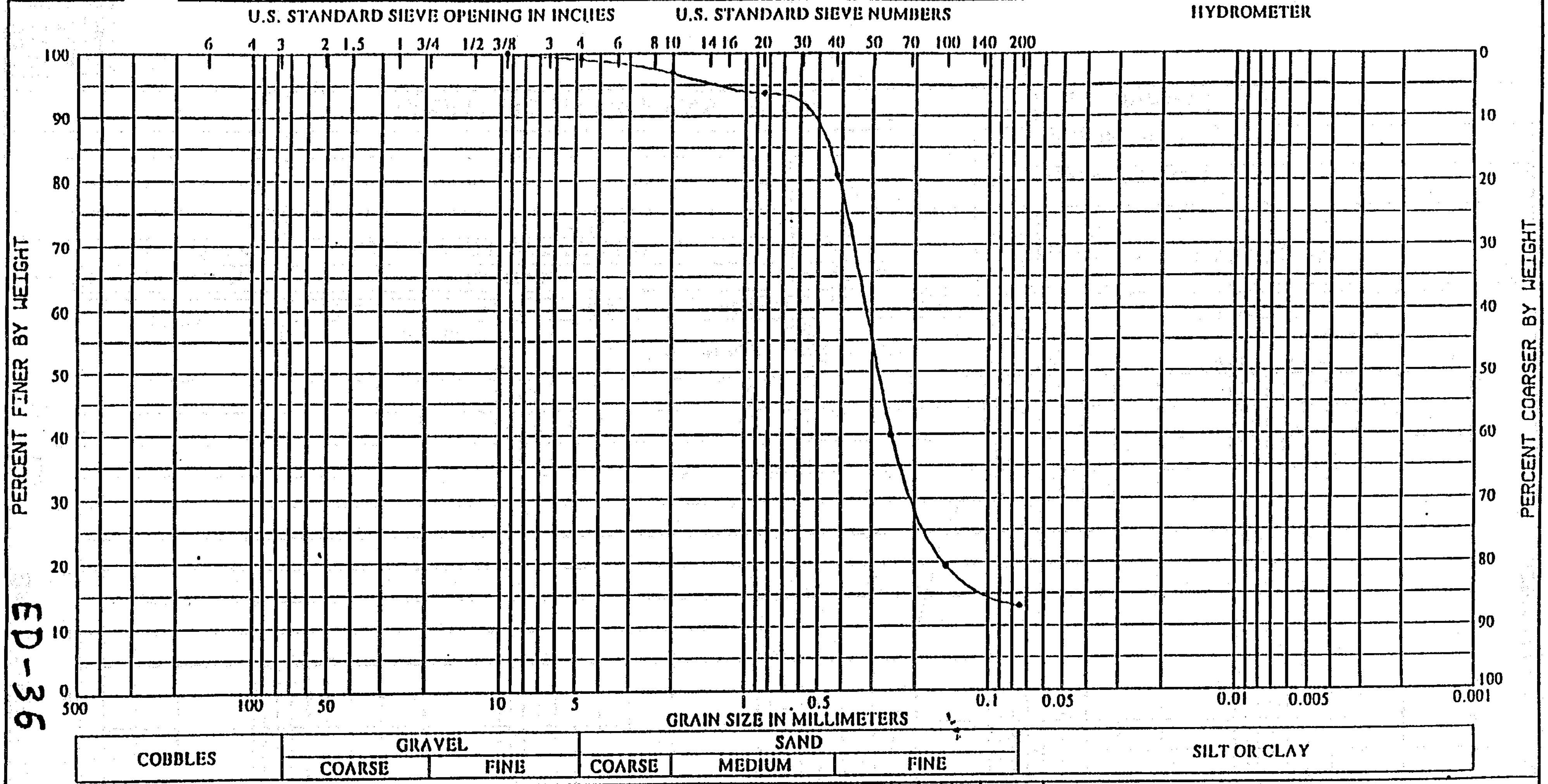
Boring No. PM-15

Date 08/06/97

GRADATION CURVES

DEPARTM NT OF THE ARMY, SOUTH ATLANTIC DIVISION ORATORY
CORPS C GINEERS, 611 SOUTH COBB DRIVE, MARIETTA, . 30060

WORK ORDER: 0275
REQUISITION: W31XNJ71694138



ED-36

Sample No.	Depth (ft)	Classification	Nat w%	LL	PL	PI
	20.0 - 22.0	CLASSIFICATION NOT REQUESTED. (SP-SM)	21.5			

Project PORT MONMOUTH, NJ

Lab No. 264/831

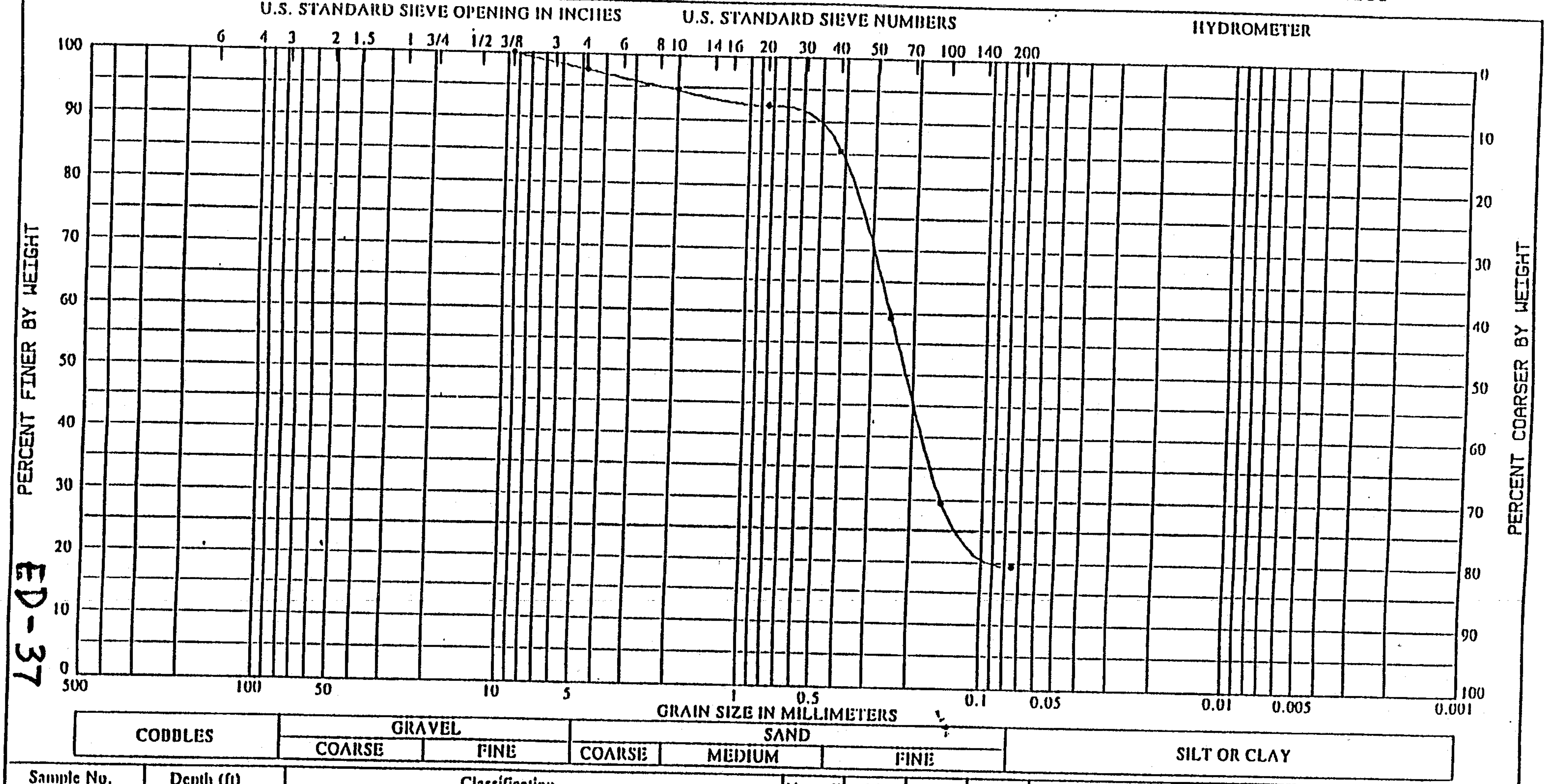
Boring No. PM-15

Date 08/06/97

GRADATION CURVES

DEPART T OF THE ARMY, SOUTH ATLANTIC DIVISI ORATORY
CORPS GINEERS, 611 SOUTH COBB DRIVE, MARIET .. 30060

WORK ORDER: 0275
REQUISITION: W31XNJ71694130



ED-37

Sample No.	Depth (ft)	Classification	Nat w%	LL	PL	PI
3	4.0 - 6.0	CLASSIFICATION NOT REQUESTED. (SM)	16.1			

Project PORT MONMOUTH, NJ

Lab No. 264/832

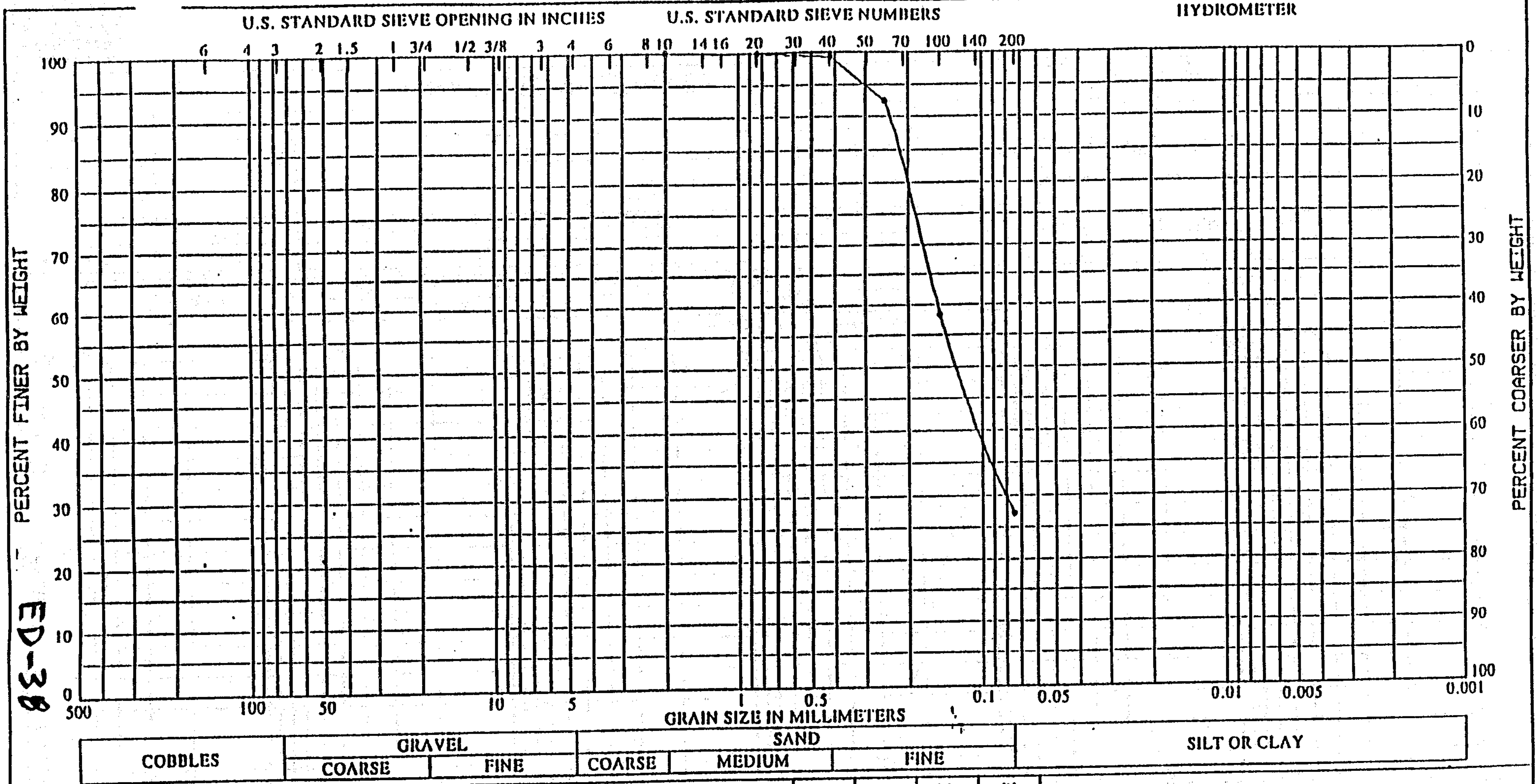
Boring No. PM-16

Date 08/06/97

GRADATION CURVES

DEPARTM ' OF THE ARMY, SOUTH ATLANTIC DIVISION RATORY
CORPS OI GINEERS, 611 SOUTH COBB DRIVE, MARIETTA, . 30060

WORK ORDER: 0275
REQUISITION: W31XNJ71694138



ED-38

Sample No.	Depth (ft)	Classification	Nat w%	LL	PL	PI
	4.0 - 6.0	CLASSIFICATION NOT REQUESTED. (SM)	12.5			

Project PORT MONMOUTH, NJ

Lab No. 264/834

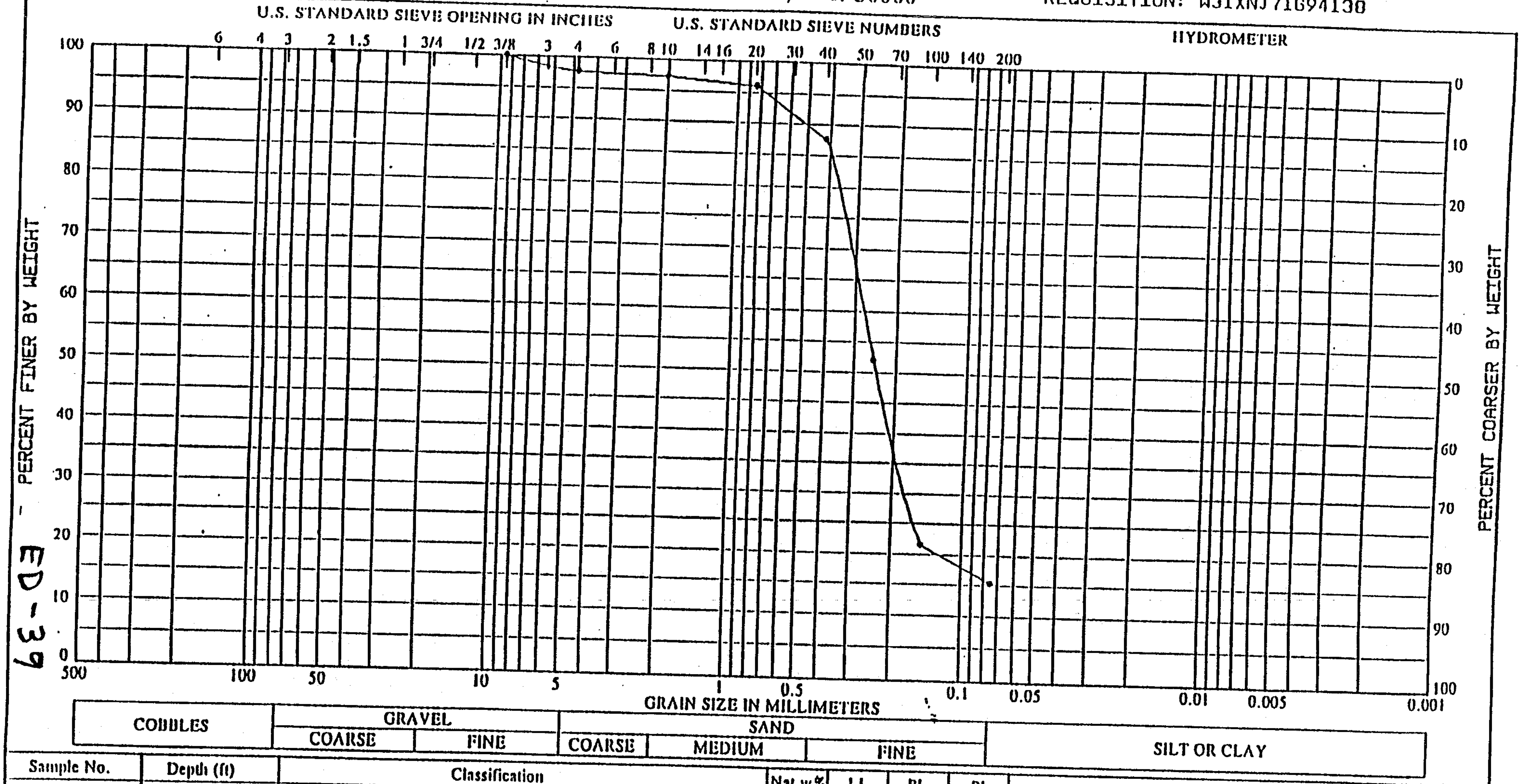
Boring No. PM-17

Date 08/06/97

GRADATION CURVES

DEPARTM " OF THE ARMY, SOUTH ATLANTIC DIVISIO RATORY
CORPS (GINEERS, 611 SOUTH COBB DRIVE, MARIET .. 30060

WORK ORDER: 0275
REQUISITION: W31XNJ71694130



ED-39

Sample No.	Depth (ft)	Classification	Nat w%	LL	PL	PI
	8.0 - 10.0	CLASSIFICATION NOT REQUESTED. (SM)				

Project PORT MONMOUTH, NJ

Lab No. 264/837

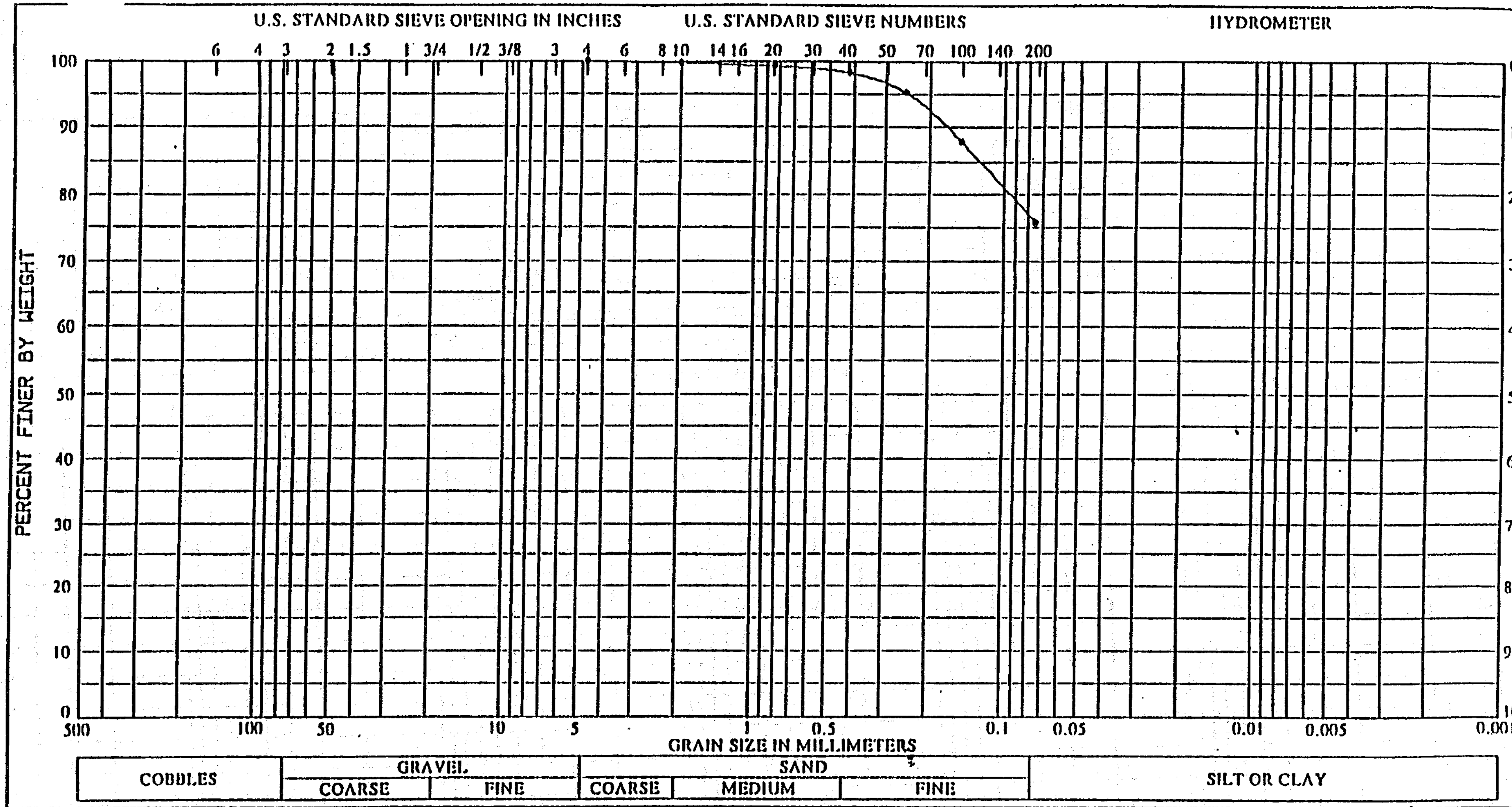
Boring No. PMP-2

Date 08/06/97

GRADATION CURVES

DEP 'TMENT OF THE ARMY, SOUTH ATLANTIC DIVI LABORATORY
COI)F ENGINEERS, 611 SOUTH COBB DRIVE, MARI. ., GA. 30060

WORK ORDER: 8275
REQUISITION: W31XNJ71694136



Sample No.	Depth (ft)	Classification	Nat w%	LL	PL	PI
	5.0 - 7.0	GRAY, FAT CLAY (CH), WITH SOME SAND SIZES AND A TRACE OF MICA.		50	23	27

Project PORT MONMOUTH, NJ

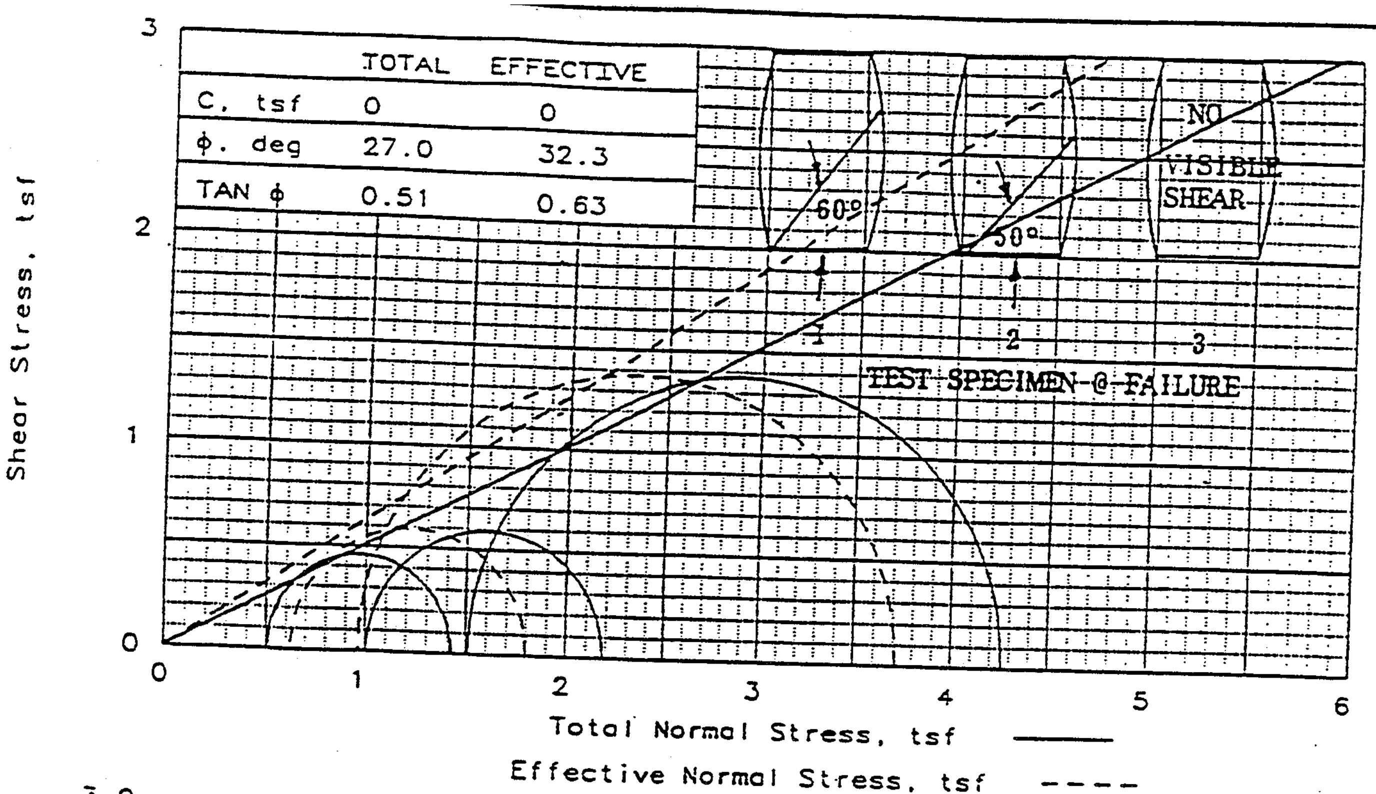
Lab No. 264/813

Boring No. PM-1A

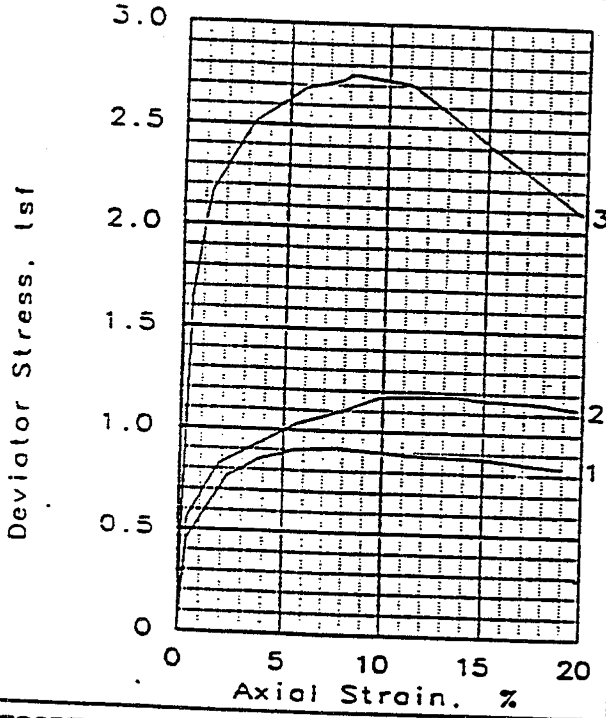
Date 08/06/97

GRADATION CURVES

ED-40



	TOTAL	EFFECTIVE
C, tsf	0	0
φ, deg	27.0	32.3
TAN φ	0.51	0.63



	SAMPLE NO.:	1	2	3
INITIAL	WATER CONTENT, %	30.4	25.9	13.8
	DRY DENSITY, pcf	94.0	97.9	99.5
	SATURATION, %	99.9	93.0	51.5
	VOID RATIO	0.846	0.774	0.744
	DIAMETER, in	1.38	1.38	1.38
	HEIGHT, in	3.08	3.08	3.08
AT TEST	WATER CONTENT, %	30.4	25.9	13.8
	DRY DENSITY, pcf	94.1	101.0	125.5
	SATURATION, %	100.0	100.0	100.0
	VOID RATIO	0.845	0.719	0.383
	DIAMETER, in	1.38	1.37	1.23
	HEIGHT, in	3.05	3.04	3.05
BACK PRESSURE, tsf		5.04	5.04	5.04
INIT. EFF. STR., tsf		0.50	1.01	1.51
FAILURE STRESS, tsf		0.92	1.18	2.75
PORE PRESSURE, tsf		-0.01	0.38	0.55
TIME TO FAILURE, min.		95	120	90
RATE, %/min.		0.08	0.11	0.09
ULTIMATE STRESS, tsf		0.87	1.16	2.42

DESCRIPTION GRAY, FAT CLAY (CH), WITH SOME SAND SIZES AND A TRACE OF MICA.

LL= 50	PL= 23	PI= 27	Gs= 2.78	CONTROLLED STRAIN TEST

TYPE OF SPECIMEN UNDISTURBED

TYPE OF TEST R w/pore pressures

REMARKS

PROJECT PORT MONMOUTH NJ

AREA

LAB. NO. 264/813

BORING. NO. PM-1A

SAMPLE. NO.

DEPTH/ELEV 5.0 - 7.0'

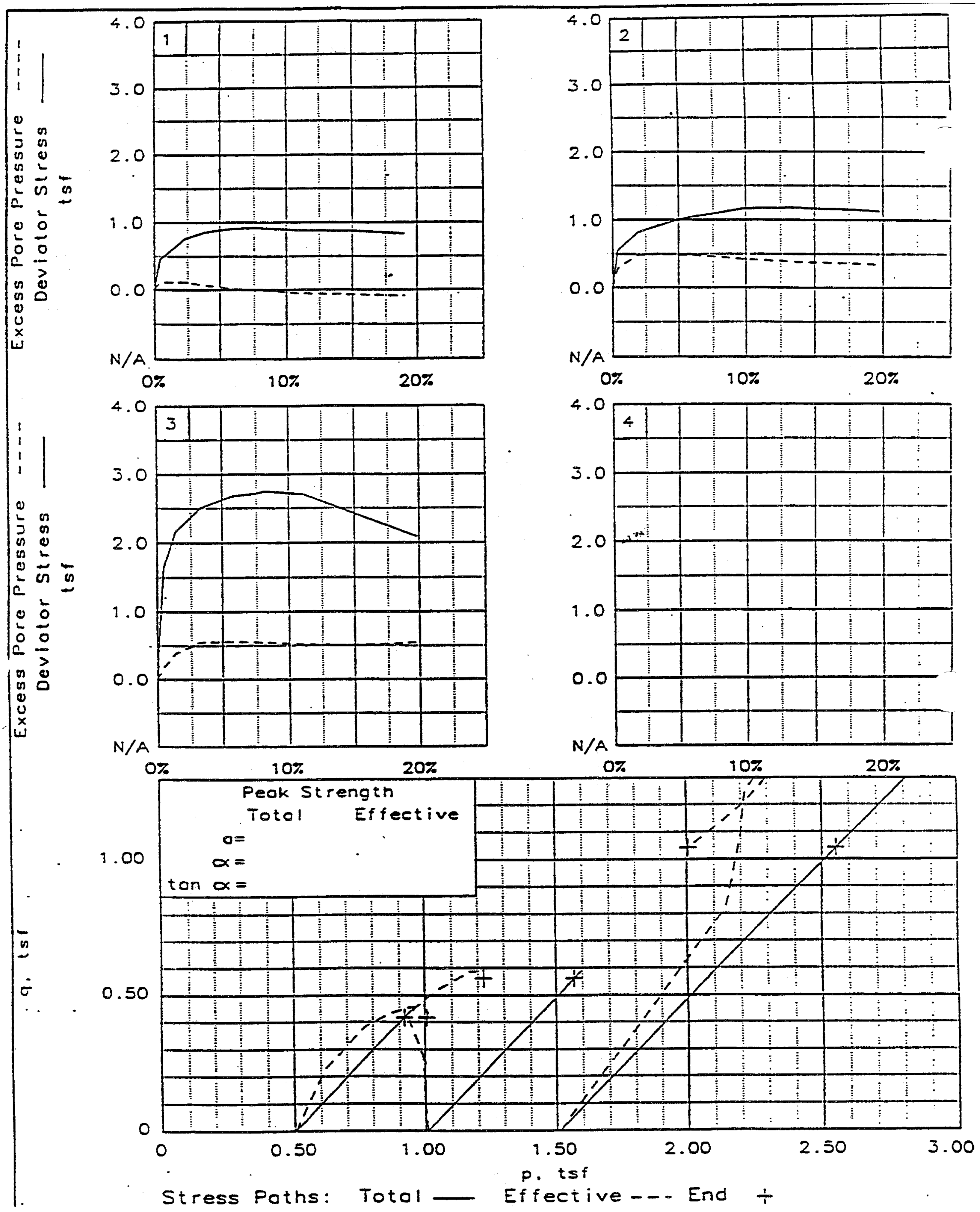
LABORATORY CESAD-EN-FL DATE 29 JULY 1997

Pg 1 of 2 TRIAXIAL SHEAR TEST REPORT

ED-41

Requisition No. W31XNJ71694138

rder No. 8275



Requisition No. W31XNJ71694138

W rder No. 8275

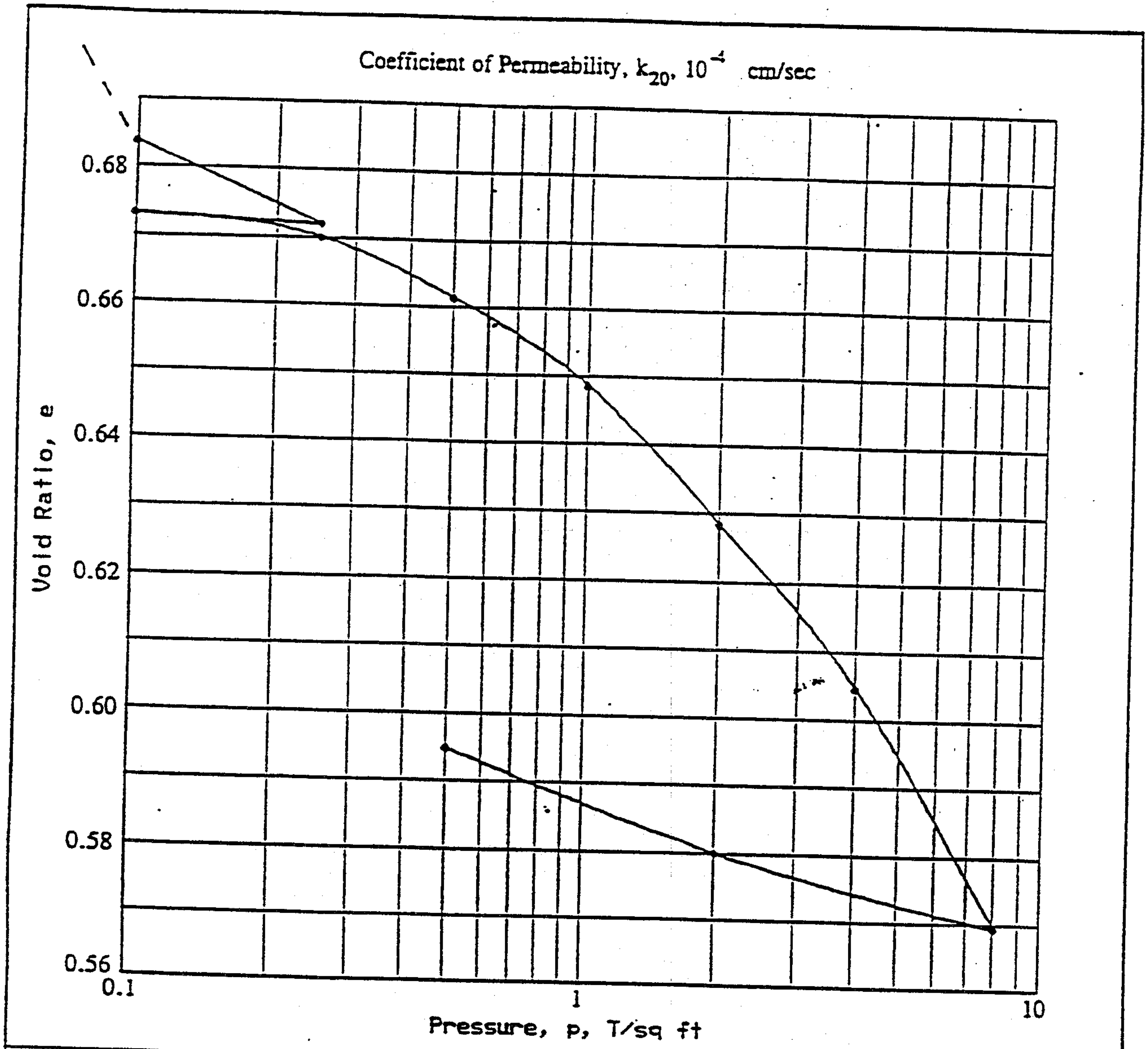
PROJECT PORT MONMOUTH NJ

BORING PM-1A SAMPLE DEPTH/ELEV 5.0 - 7.0'

TYPE OF TEST R w/pore pressures LAB NO. 264/813

LABORATORY CESAD-EN-FL Page No.: 2/2 DATE 29 JULY 1997

WORK ORDER: 0275
REQUISITION: W31XNJ71694138



Type of Specimen UNDISTURBED		Before Test			After Test	
Diam. 2.490 in.	Height 1.000 in.	Water Content	w_o	25.2 %	w_f	21.4 %
Overburden Pressure, p_o T/sq ft		Void Ratio	e_o	0.700	e_f	0.595
Preconsol. Pressure, p_c T/sq ft		Saturation	S_o	100.0 %	S_f	100.0 %
Compression Index, C_c		Dry Density	γ_d	102.1 lb/ft^3		

Classification GRAY, FAT CLAY (CH), WITH SOME SAND SIZES AND MICA.

LL	50	G_s	2.78	Project PORT MONMOUTH, NJ
PL	23	D_{10}		

Remarks 1. SPECIMEN FLOODED THROUGHOUT TEST. 2. TIME CURVES FURNISHED FOR LOADS OF 1.0, 2.0, 4.0 AND 8.0 TSF.

Lab No. 264/813

Boring No. PM-1A

Sample No.

Depth (ft) 5.0-7.0

Date 07/31/97

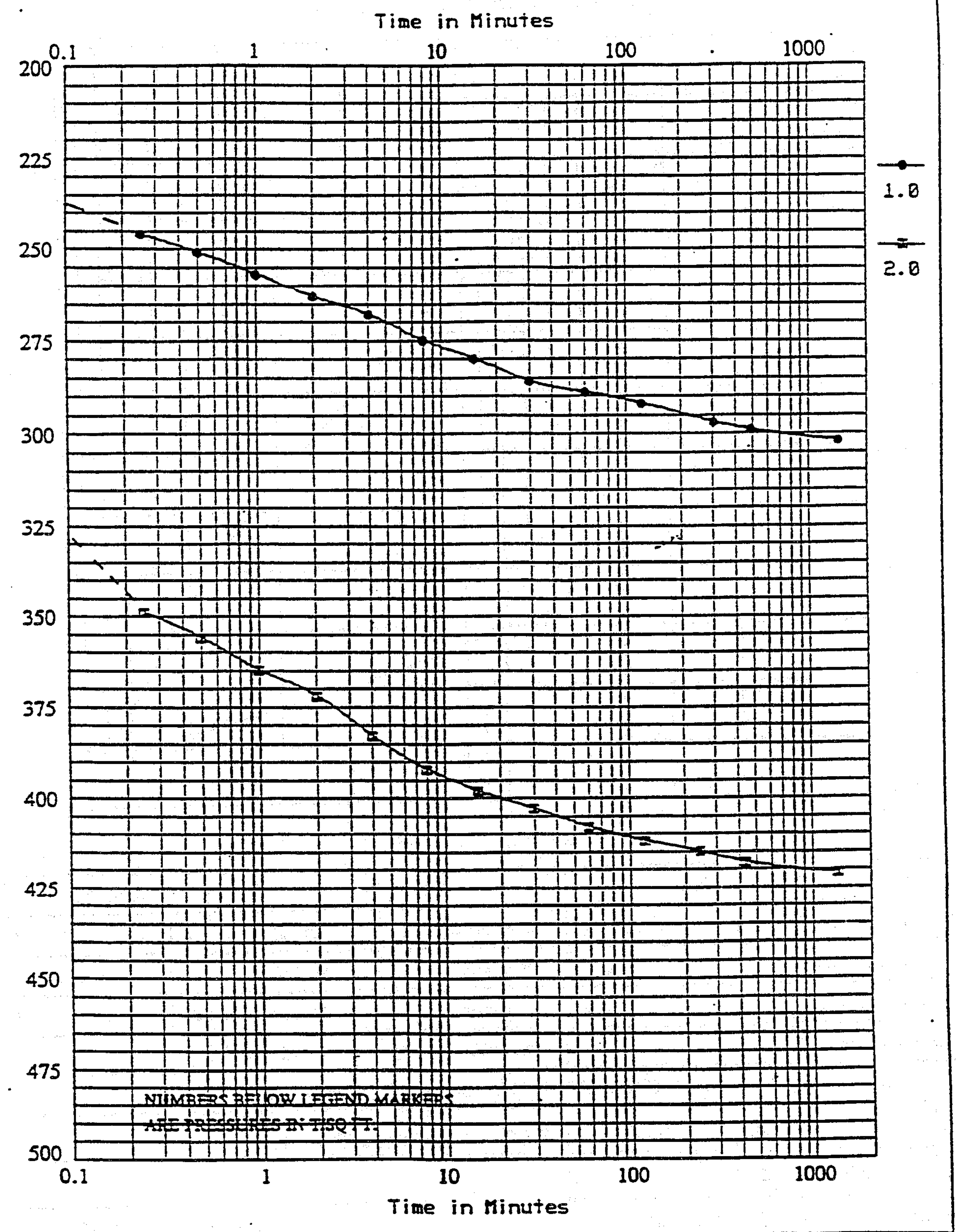
CONSOLIDATION TEST REPORT

DE ENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABOR
COR ENGINEERS, 611 SOUTH COBB DRIVE, MARIETTA, GA 30

ED-43

WORK ORDER: 8275
REQUSITION: W31XNJ71694138

DEP NT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORAT
COR ENGINEERS, 611 SOUTH COBB DRIVE, MARIETTA, GA 3



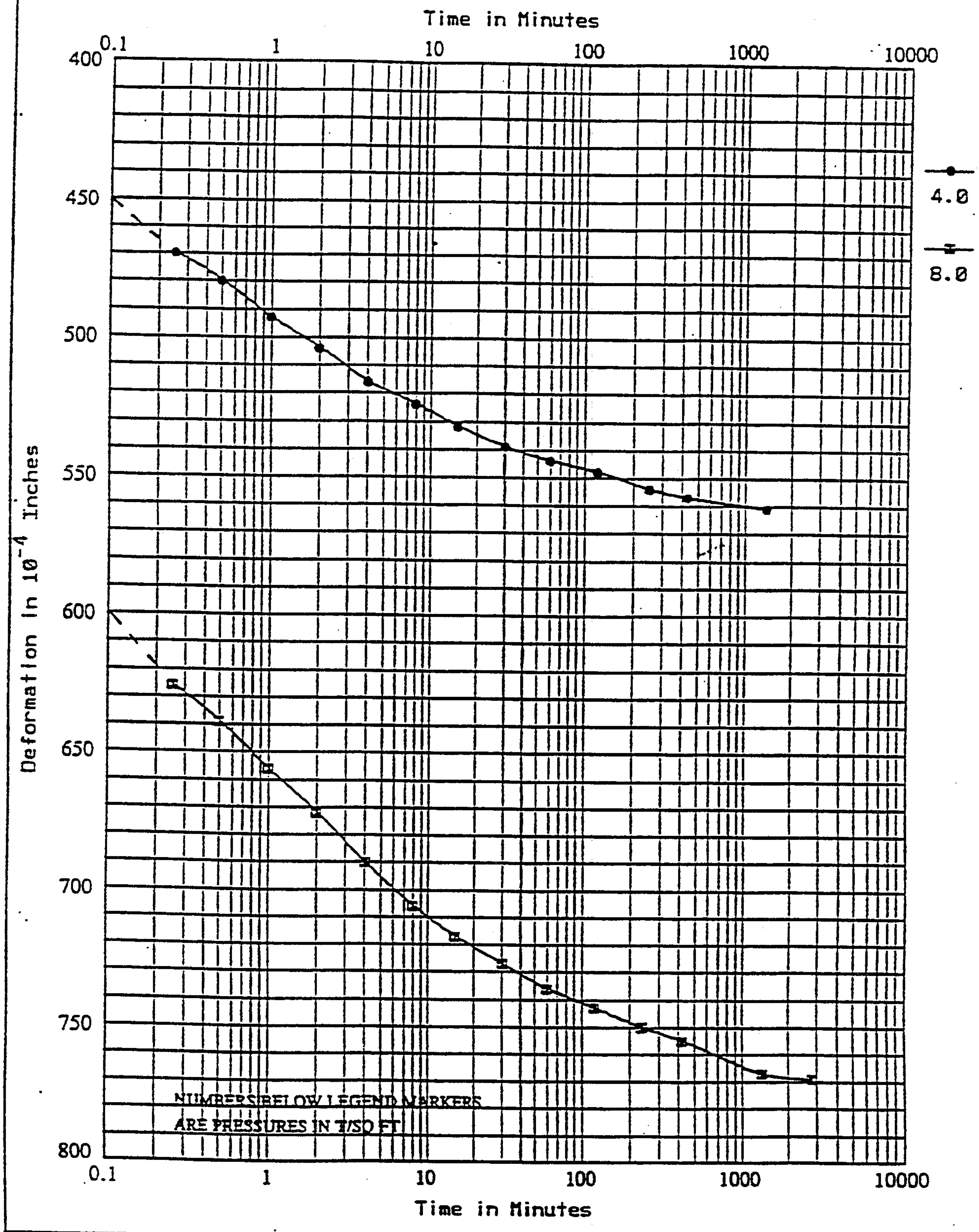
Project PORT MONMOUTH, NJ			
Area			Lab No. 264/813
Boring No. PM-1A	Sample No.	Depth (ft) 5.0-7.0	Date 07/31/97

CONSOLIDATION TEST - TIME CURVES

ED-44

WORK ORDER: 0275
REQUSITION: W31XNJ716V413b

DEP NT OF THE ARMY, SOUTH ATLANTIC DIVISION LABOR
COR ENGINEERS, 611 SOUTH COBB DRIVE, MARIETTA, GA

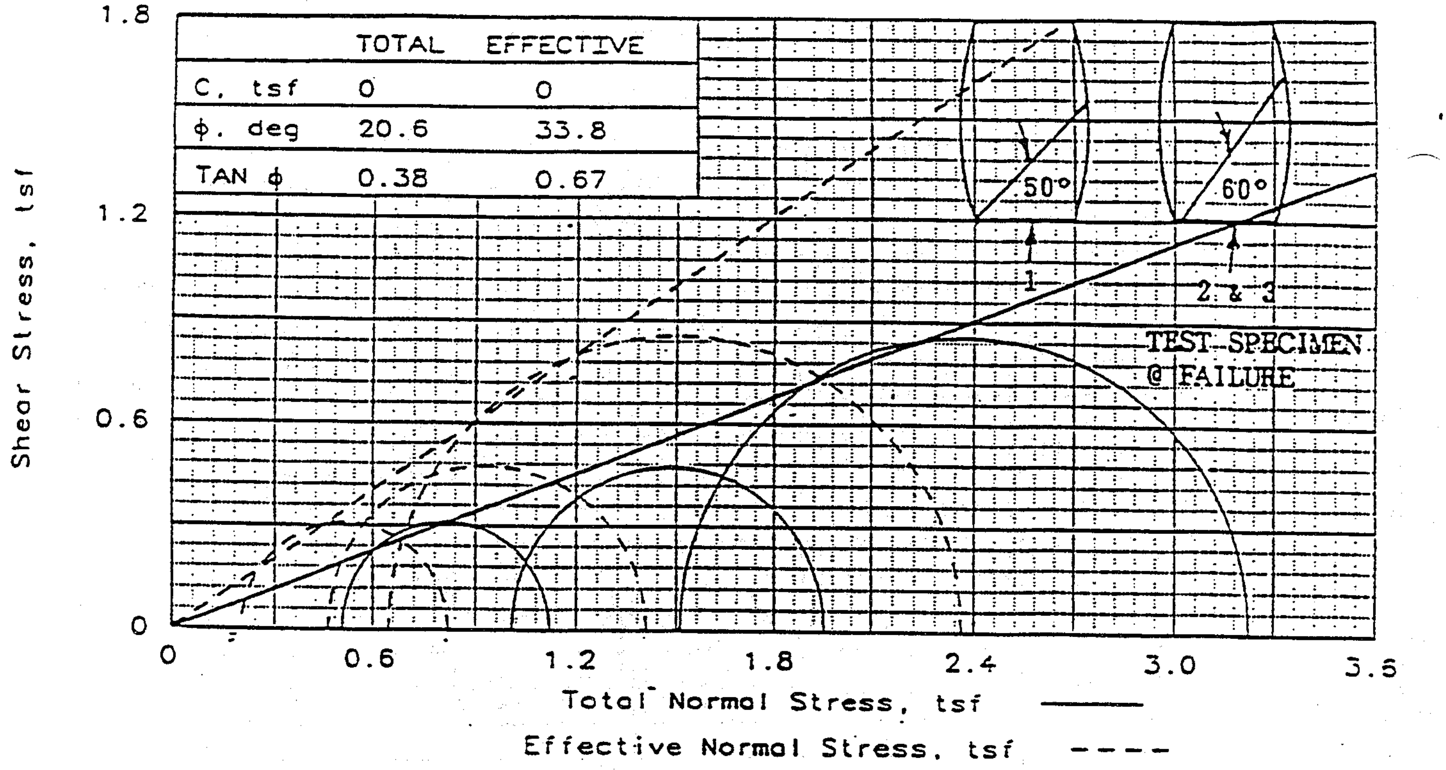


NUMBERS BELOW LEGEND MARKERS
ARE PRESSURES IN T/SQ FT

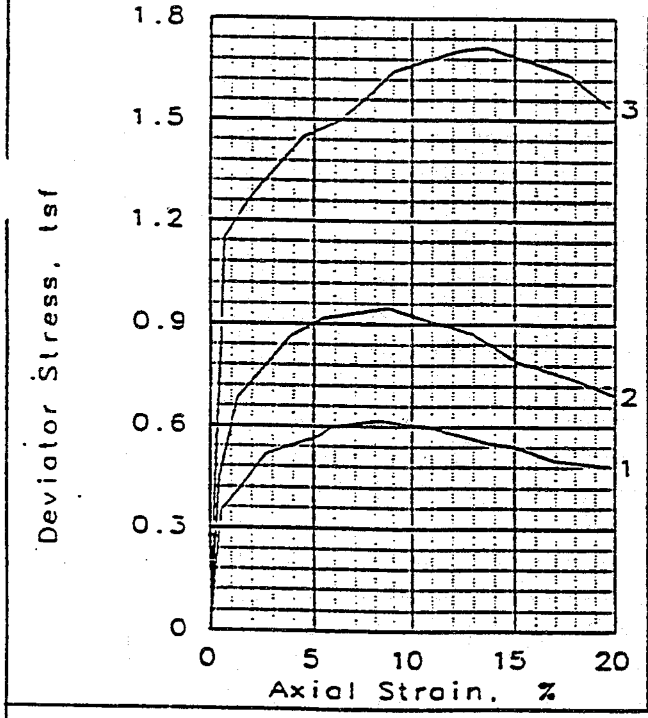
Project PORT MONMOUTH, NJ			
Area			Lab No. 264/813
Boring No. PM-1A	Sample No.	Depth (ft) 5.0-7.0	Date 07/31/97

CONSOLIDATION TEST - TIME CURVES

ED-45



	TOTAL	EFFECTIVE
C, tsf	0	0
ϕ, deg	20.6	33.8
TAN ϕ	0.38	0.67



SAMPLE NO.:		1	2	3
INITIAL	WATER CONTENT, %	72.9	84.4	25.7
	DRY DENSITY, pcf	55.8	50.2	95.8
	SATURATION, %	97.7	97.0	92.1
	VOID RATIO	2.000	2.332	0.747
	DIAMETER, in	1.38	1.38	1.38
	HEIGHT, in	3.08	3.08	3.08
AT TEST	WATER CONTENT, %	72.9	84.4	25.7
	DRY DENSITY, pcf	56.6	51.3	99.1
	SATURATION, %	100.0	100.0	100.0
	VOID RATIO	1.955	2.262	0.688
	DIAMETER, in	1.38	1.39	1.36
	HEIGHT, in	3.00	2.94	3.04
BACK PRESSURE, tsf		5.04	5.04	5.04
INIT. EFF. STR., tsf		0.50	1.01	1.51
FAILURE STRESS, tsf		0.62	0.95	1.71
PORE PRESSURE, tsf		0.30	0.55	0.86
TIME TO FAILURE, min.		95	90	120
RATE, %/min.		0.09	0.10	0.11
ULTIMATE STRESS, tsf			0.79	1.70

DESCRIPTION GRAY, FAT CLAY (CH), WITH A TRACE OF SAND SIZES AND MICA.

LL= 116	PL= 40	PI= 76	Gs= 2.68	CONTROLLED STRAIN TEST

TYPE OF SPECIMEN UNDISTURBED

TYPE OF TEST R w/pore pressures

REMARKS

PROJECT PORT MONMOUTH NJ

AREA

LAB. NO. 264/81

BORING. NO. PM-2A

SAMPLE. NO.

DEPTH/ELEV 9.0 - 11.5'

LABORATORY CESAD-EN-FL DATE 29 JULY 1997

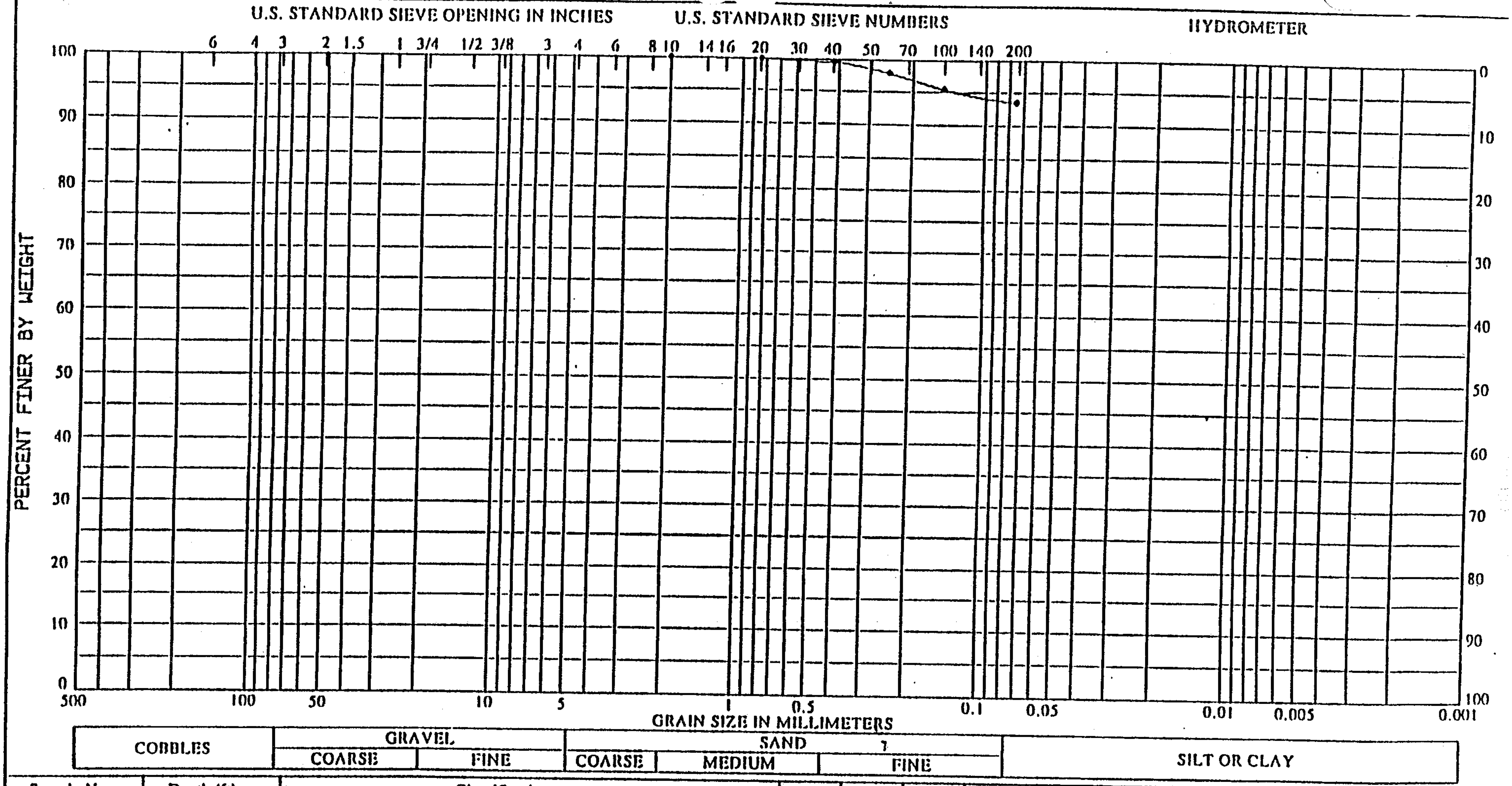
Pg 1 of 2 TRIAXIAL SHEAR TEST REPORT

Requisition No. W31XNJ71694138 W rder No. 8275

ED-46

DE TMENT OF THE ARMY, SOUTH ATLANTIC DIVIS LABORATORY
CO OF ENGINEERS, 611 SOUTH COBB DRIVE, MA , GA. 30060

WORK ORDER: 0275
REQUISITION: W31XNJ71



Sample No.	Depth (ft)	Classification	Nat w%	LL	PL	PI
	9.0 - 11.5	GRAY, FAT CLAY (CH), WITH A TRACE OF SAND AND MICA.		116	40	76

Project PORT MONMOUTH, NJ

Lab No. 264/814

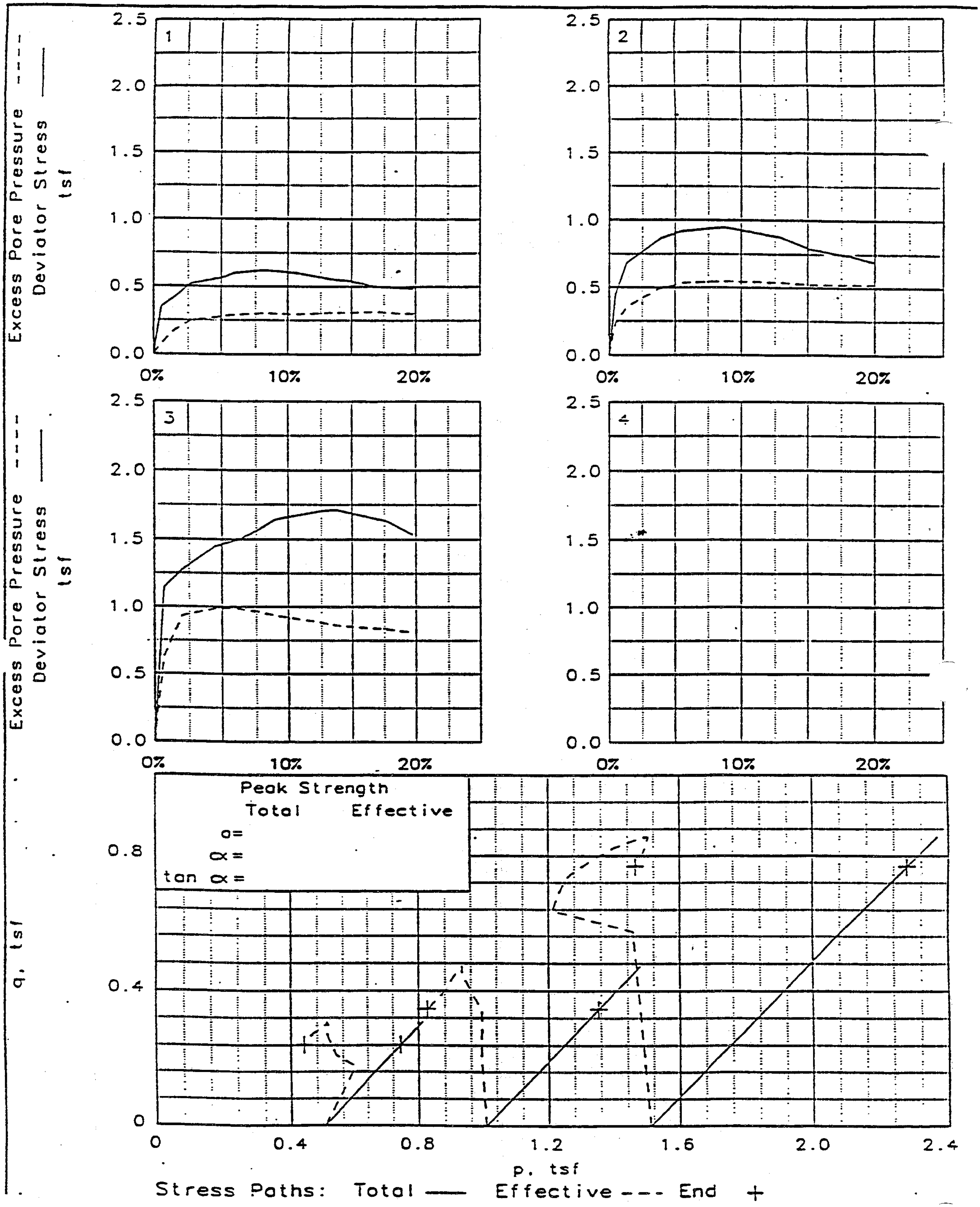
Boring No. PM-2A

Date 08/06/97

GRADATION CURVES

ED-47

Requisition No. W31XNJ71694138 W rder No. 8275



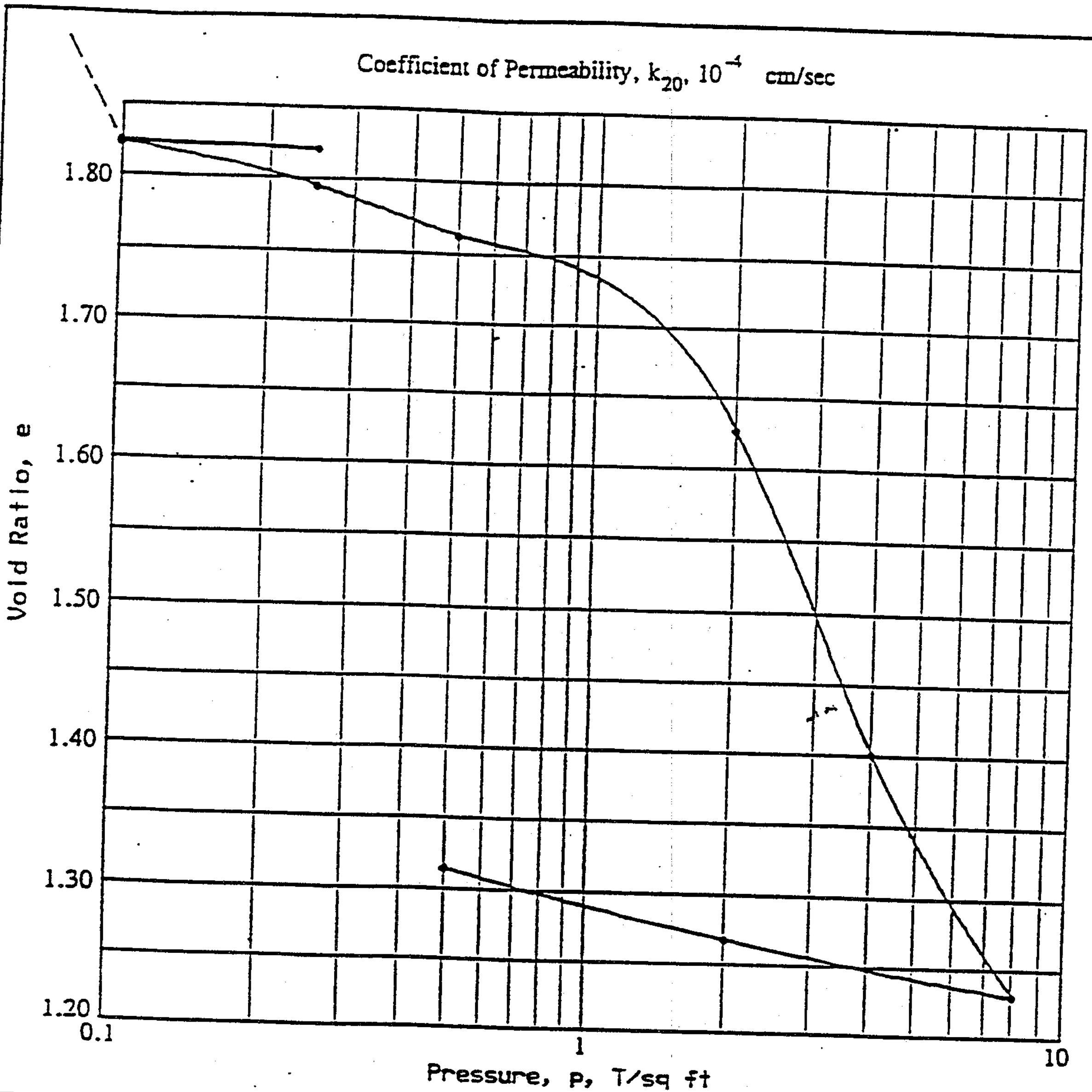
ROJECT PORT MONMOUTH NJ

BORING	PM-2A	SAMPLE		DEPTH/ELEV	9.0 - 11.5'
TYPE OF TEST	R w/pore pressures	LAB NO.	264/814		
LABORATORY	CESAD-EN-FL	Page No.:	2/2	DATE	29 JULY 1997

ED-4B

WORK ORDER: 0275
REQUISITION: W31XNJ71694138

NT OF THE ARMY, SOUTH ATLANTIC DIVISION LABOR
ENGINEERS, 611 SOUTH COBB DRIVE, MARIETTA, GA 30



Type of Specimen UNDISTURBED	Before Test			After Test	
Diam. 2.490 in. Height 1.000 in.	Water Content	w_o	74.8 %	w_f	49.0 %
Overburden Pressure, p_o T/sq ft	Void Ratio	e_o	2.035	e_f	1.314
Preconsol. Pressure, p_c T/sq ft	Saturation	S_o	98.4 %	S_f	100.0 %
Compression Index, C_c	Dry Density	γ_d	55.1 lb/ft^3		

Classification GRAY, FAT CLAY (CH), WITH A TRACE OF SAND AND MICA.

LL	116	G_s	2.68	Project PORT MONMOUTH NJ
PL	40	D_{10}		

Remarks 1. SPECIMEN FLOODED THROUGHOUT TEST. 2. TIME CURVES FURNISHED FOR LOADS OF 0.5, 2.0 4.0 AND 8.0 TSF.

Lab No. 264/814

Boring No. PM-2A

Sample No.

Depth (ft) 9.0-11.5

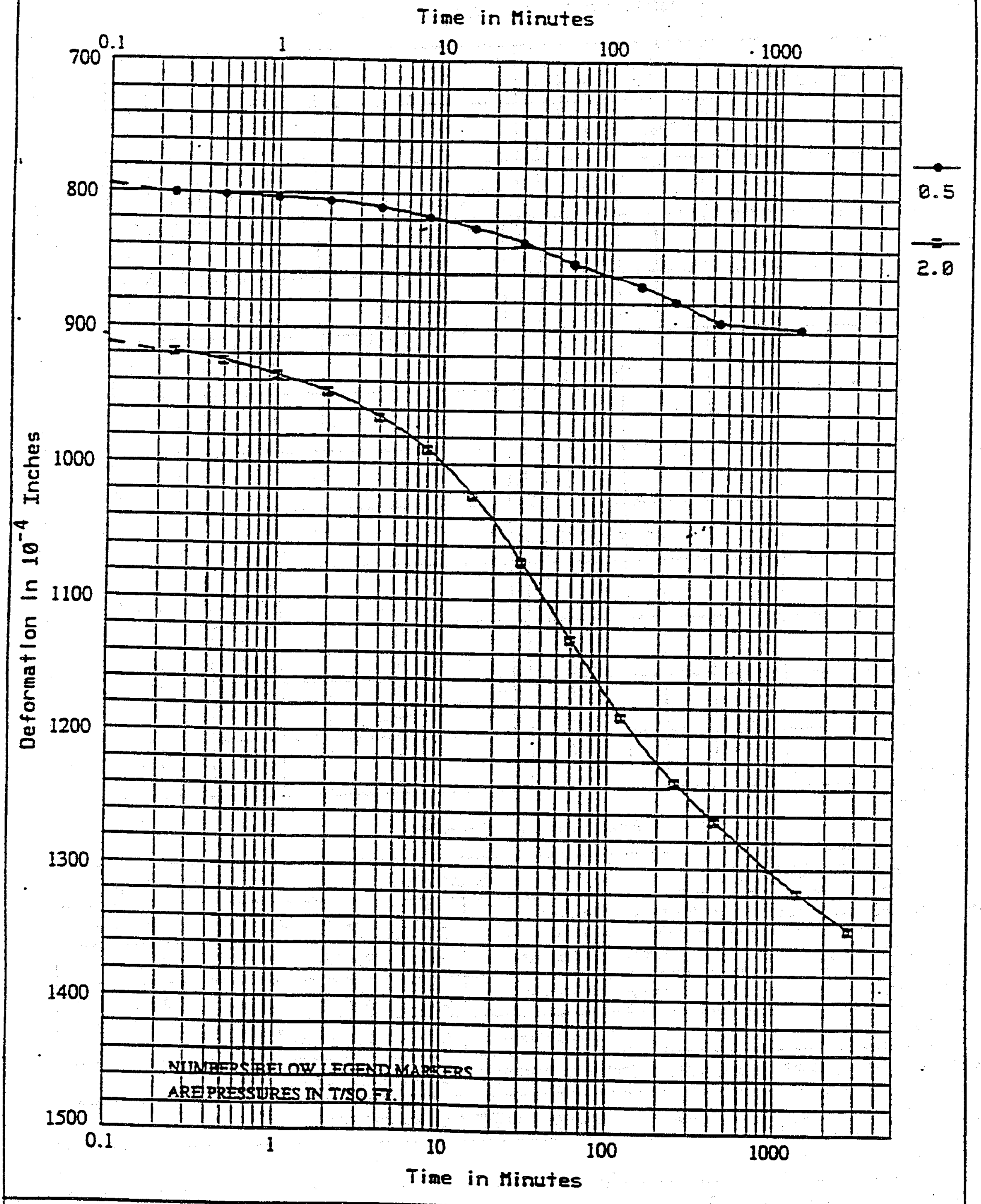
Date 08/01/97

CONSOLIDATION TEST REPORT

ED-49

WORK ORDER: 8275
REQUSITION: W31XNJ7169413b

DEPA' 'ENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORAT
CORI ENGINEERS, 611 SOUTH COBB DRIVE, MARIETTA, GA 3



Project PORT MONMOUTH, NJ

Area

Lab No. 264/814

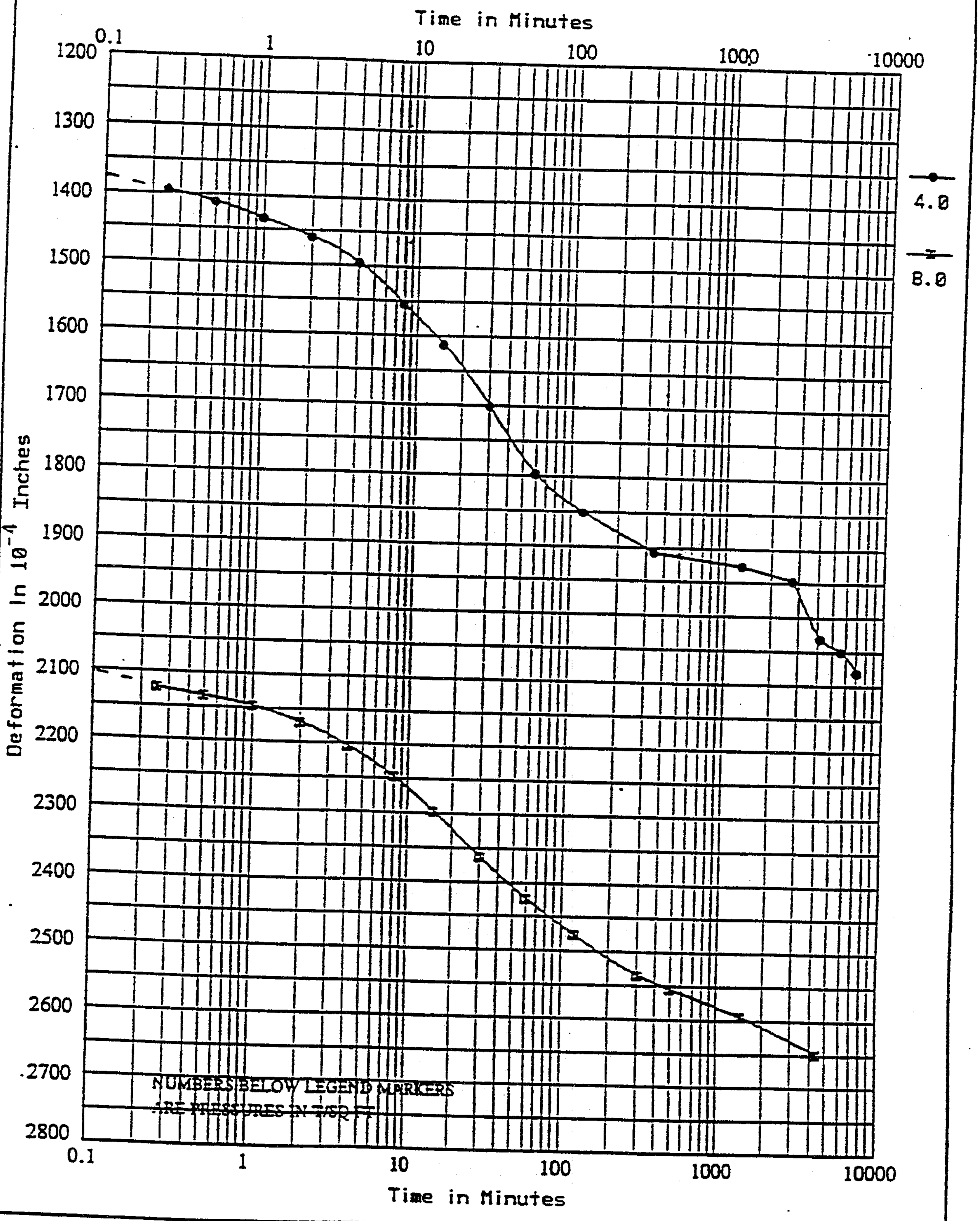
Boring No. PM-2A	Sample No.	Depth (ft) 9.0-11.5	Date 08/01/97

CONSOLIDATION TEST - TIME CURVES

ED-50

WORK ORDER: 0275
REQUSITION: W31XNJ71694138

DE NT OF THE ARMY, SOUTH ATLANTIC DIVISION LABO
CO ENGINEERS, 611 SOUTH COBB DRIVE, MARIETTA, GA 3

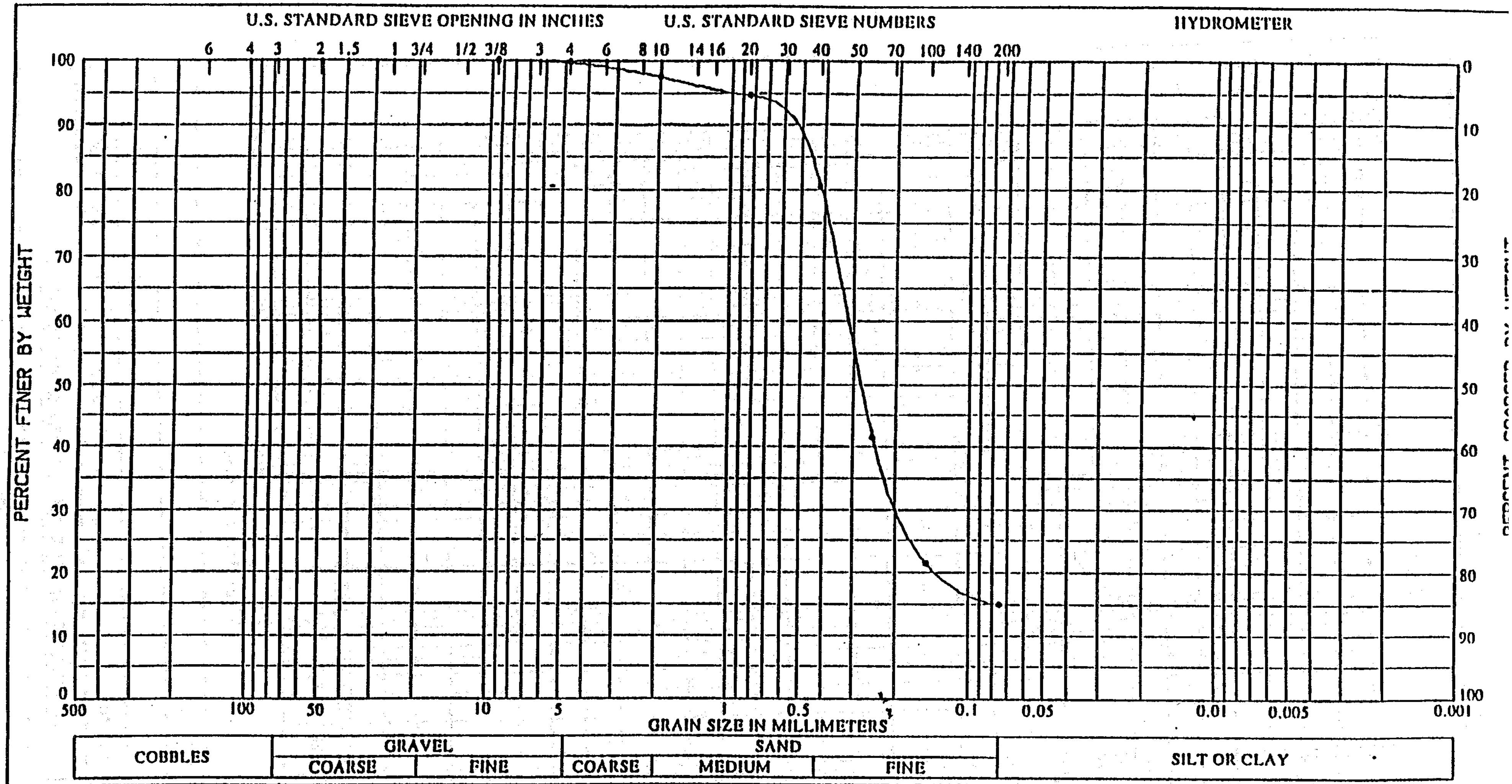


Project PORT MONMOUTH, NJ			
Area			Lab No. 264/814
Boring No. PM-2A	Sample No.	Depth (ft) 9.0-11.5	Date 08/01/97

CONSOLIDATION TEST - TIME CURVES

DEP MENT OF THE ARMY, SOUTH ATLANTIC DIVIS. ABORATORY
COR. JF ENGINEERS, 611 SOUTH COBB DRIVE, MARIETTA, GA. 30060

WORK ORDER: 8275
REQUISITION: W31XNJ71694138



Sample No.	Depth (ft)	Classification	Nat w%	LL	PL	PI
	13.0 - 15.5	(VISUAL) GRAY, SILTY SAND (SM), WITH A TRACE OF MICA.				

Project PORT MONMOUTH, NJ

Lab No. 264/815

Boring No. PM-12

Date 08/06/97

GRADATION CURVES

ED-52

June 2000

RARITAN BAY AND SANDY HOOK BAY, NEW JERSEY
FEASIBILITY REPORT
FOR
HURRICANE AND STORM DAMAGE REDUCTION
PORT MONMOUTH, NEW JERSEY

APPENDIX F

INTERIOR DRAINAGE

F

PORT MONMOUTH, NEW JERSEY
COMBINED FLOOD CONTROL AND SHORE PROTECTION
APPENDIX F - INTERIOR DRAINAGE

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PORT MONMOUTH, NEW JERSEY
COMBINED FLOOD CONTROL AND SHORE PROTECTION
INTERIOR DRAINAGE APPENDIX

INTRODUCTION

Scope

F1. Interior drainage facilities are required to safely store and discharge storm water runoff which collects on the protected side of levees and floodwalls associated with flood control projects. This Appendix describes interior drainage facilities for the Port Monmouth Combined Flood Control and Shore Protection Project, and documents how these facilities were developed to manage interior runoff consistent with the level of protection provided by the main protection works identified in this Feasibility Report.

PROPOSED PROTECTION PLAN

Description of Proposed Improvements

F2. General. The selected storm damage protection system comprises levees, floodwalls, seawalls, relocated dunes, storm gates and a pump station. The system will extend from State Highway 36 to Sandy Hook Bay then east along the shoreline to Port Monmouth Road and tie into the existing Keansburg levee by way of a storm gate. The following sections describe the facilities for each of the major plan components; the exposed bay shoreline, Pews Creek, Compton Creek, and drainage behind the levees. Figure F-1 is a general location map; Figures F-2 through F-8 show the proposed improvements.

F3. Bayshore. The selected plan for protection along the project's bay shoreline consists of a reconstructed dune fronted by a protection beach berm. The dune crest elevation is 16 ft. NGVD, with a crest width of 25 feet. Dune integrity will be ensured by periodic nourishment. .

F4. Pews Creek. The selected plan for protection in the Pews Creek area consists of levees, floodwall, a closure structure at the intersection of Port Monmouth Road and New Port Monmouth Road, and a storm gate and pump station system at Pews Creek. This portion of the protection system begins at the terminus of the new dune in the Bayshore area (see previous paragraph) and extends westerly to the existing Keansburg levee to the west of Pews Creek. The levee/floodwall system will have a crest elevation of 14 ft. NGVD.

F5. The storm gate at Pews Creek is a critical component in the line of protection system. The gate allows normal tidal exchange to inundate the marsh areas during non-storm/flood events, while using the same marsh areas for interior drainage storage during storm/flood conditions.

F6. Compton Creek. The selected plan for protection along Compton Creek consists of levees and floodwalls extending from State Highway 36 (near the intersection with Wilson Avenue) in a generally northerly direction to the new dune system proposed for the Bayshore area. Closure gates will be required at Campbell Avenue and at Broadway. The levee/floodwall system will have a crest elevation of 14 ft. NGVD.

Description of the Protected Areas

F7. **General.** Interior drainage facilities are required to safely store and discharge storm water runoff which collects on the protected side of dunes, berms, levees and floodwalls associated with flood control projects. For purposes of this project, an "interior drainage area" is a distinct land area which drains to one primary outlet location behind the proposed line-of-protection. The identification of such areas is complicated by the presence of anthropogenic features such as storm sewers which divert flow into or out of a drainage area. In some cases, otherwise distinct interior areas could become combined during rare storm events due to high ponding depths directly behind the line of protection works.

F8. **General Location.** The project area is located in Port Monmouth, Monmouth County, New Jersey, along an approximately 1.5-mile stretch of the Sandy Hook Bay shoreline, extending from Compton Creek at the east of the project area to Pews Creek at the west of the project area, and bounded at the south by NJ State Highway 36. For purposes of the interior drainage analysis, protected areas are divided into two main areas based on the receiving watershed: Pews Creek and Compton Creek. Figures F-2 through F-8 show the interior drainage basins.

F9. **Pews Creek.** As noted above, the storm gate across Pews Creek is considered integral to the line of protection. Thus, the entire Pews Creek watershed technically constitutes an interior drainage area. The basin was subdivided into four drainage sub-basins (PC1, PC2, PC3 and Lower). For practical purposes, only the Lower sub-basin is directly affected by the proposed line of protection and most of the analysis described in the remainder of this appendix will focus on this sub-basin. The Lower sub-basin consists of areas from just upstream of Route 36 to the storm gate, with about 1/3 of the area being low-lying wetlands regularly inundated by tides.

F10. **Compton Creek.** The proposed plan includes three interior drainage sub-basins (C1, C2, and C3) which parallel the line of protection works and are located along the west bank of Compton Creek. Most of the area in all three sub-basins is residential development within Port Monmouth.

DESIGN PROCEDURE

Interior Flood Control Simulation Models

F11. **General**. The interior flood control analysis was primarily conducted using two mathematical models to simulate the hydrologic response of interior drainage areas and the operation of interior drainage facilities. These models, both developed by the U.S. Army Corps of Engineers Hydrologic Engineering Center, are the Flood Hydrograph Package (HEC-1) and the Interior Flood Hydrology program (HEC-IFH).

F12. **HEC-IFH**. HEC-IFH is a computer program designed to route floods through interior drainage facilities to adjacent rivers or estuaries accounting for variable tailwater conditions. This program was utilized to simulate the surface runoff response of basins to precipitation while taking into account both the hydrologic and hydraulic components of these basins. The program was used exclusively for analysis of Compton Creek interior drainage areas. However, for Pews Creek interior drainage, HEC-1 was used to simulate surface runoff (see next section), while HEC-IFH was then used to route the runoff against the variable tidal tailwater conditions.

F13. **HEC-1**. HEC-1 is a computer program designed to simulate the surface runoff response of the basins to precipitation while taking into account both the hydrologic and hydraulic components of these basins. It also includes reservoir and pump components which can be used to route inflow through a reservoir, stream basin, etc., and to simulate action of pumping facilities to lift the stormwater. Because of its larger and more complex watershed characteristics, runoff in the Pews Creek watershed was simulated using HEC-1 rather than HEC-IFH. Subsequently, as noted above, the inflow hydrographs developed for Pews Creek with HEC-1 were routed through the impounded areas by HEC-IFH.

Basis of Design

F14. **Hydrologic Analysis of Interior Areas**. A single HEC-1 model was developed for the entire Pews Creek watershed and a site-specific HEC-IFH model was developed for each Compton Creek interior drainage watershed. Basic input parameters dveloped for the hydrologic models include: surface area; rainfall generated for a series of hypothetical storm events (2- to 500-year return period) and two historical (Hurricane Donna September 11-13, 1960) and the 6-8 March 1962 storms, runoff curve numbers developed per the methods described in Soil Conservation Service's Technical Release No. 55 "Urban Hydrology for Small Watersheds" (TR-55), and times of concentration. These input parameters are described in more detail in later sections.

F15. **Hydraulic Analysis of Facility Components**. Existing storm sewers and related appurtenances were evaluated for conveyance capacities through use of standard hydraulic analyses. Gravity flow within storm sewers and ditches were determined with Mannings' equation. The capacity of culverts having pooled water at their entrance was determined using inlet control analyses as described in Federal Highway Administration's Hydraulic Design Series No. 5 "Hydraulic Design of Highway Culverts" (HDS-5). Where conditions required investigating surcharged conditions, the Hazen-Williams equation was utilized. Proposed outlet structures such as culverts through floodwalls or levees integral to the project line of protection were analyzed using similar methods within HEC-IFH.

F16. **Selection of Analysis Methods**. The analysis presented herein is based on the concepts and guidelines contained in U.S. Army Corps of Engineers' Engineering Manual (EM) 1110-2-1413 "Hydrologic Analysis of Interior Areas."

Correlation Analysis

F17. **General**. Each of the preliminary structural alternatives will trap local drainage behind some form of a line of protection, such as a levee, floodwall or closure gate. Since the Port Monmouth area is extremely flat, runoff within the interior of the line of protection could spread out over a wide area, creating a potentially significant residual flood hazard. In order to release the interior runoff to Pews and Compton Creeks, outlet pipes with flap valves and sluice gates to control backflow will be provided at regular intervals. Since these gravity structures can not discharge runoff against high tailwater stages, it is important to develop an understanding of the relationship between the precipitation events creating significant interior runoff, and tidal events creating high exterior stages which block the gravity outlets.

F18. A review of historical precipitation and tides was performed in order to:

- Describe the meteorologic origins of storms as they relate to independence or dependence between interior and exterior flood events
- Identify the probability of coincident interior and exterior flooding
- Quantify any correlation between the amount of precipitation and peak tide level during storms
- Recommend tidal tailwater conditions for use in the HECIFH Analysis.

F19. Raritan Bay Tides. Tides at Port Monmouth are semi-diurnal and have a mean range of 4.6 feet and a spring range of 5.6 feet. Tide ranges for points in the project area are summarized in Table F-1. The maximum recorded storm water elevation at Sandy Hook was observed on December 11, 1992, when the reported water level was 8.69 ft. NGVD. The second highest water level at Sandy Hook was 8.56 ft. NGVD on September 12, 1960.

TABLE F-1

Tidal Datum Near Port Monmouth, New Jersey

	Elevations in feet					
Elevations of Datums	**Atlantic Highland**		**Way Cake Creek**		**Port Monmouth**	
	MLLW	**NGVD**	**MLLW**	**NGVD**	**MLLW**	**NGVD**
Mean Higher High Water (MHHW)	5.27	3.49	5.12	3.60	5.20	3.55
Mean High Water (MHW)	4.93	3.15	4.79	3.27	4.86	3.21
Mean Tide Level (MTL)	2.57	0.79	2.47	0.95	2.52	0.87
Mean Low Water (MLW)	0.20	-1.58	0.16	-1.36	0.18	-1.47
Mean Lower Low Water (MLLW)	0.00	-1.78	0.00	-1.52	0.00	-1.65
NGVD	1.78	0.00	1.52	0.00	1.65	0.00

Source: NOAA, 1987

F20. Currents in the project area are predominately tidal, with contribution from waves and creek discharges. Tidal currents 0.4 miles west of Sandy Hook have a measured average maximum velocity of 2.0 knots on the flood tide and 1.6 knots on the ebb tide. In the Raritan Bay Reach Channel north of Keansburg, the maximum average flood and ebb currents are 0.6 knots and 0.4 knots respectively

(NOAA, 1995). Bay currents in the vicinity of Port Monmouth are weaker, except under storm conditions.

F21. Flooding in the project area is typically caused by the combination of waves with storm-induced water levels and astronomical tide. The storm-induced water level is produced by several effects. Storm winds develop a shear force on the water surface causing a rise of the water surface along the coast. Decreasing barometric pressure raises the water surface drawing water from adjacent higher pressure areas. Finally, storm waves raise the water level along the shore as water piles up along the coast, an effect called wave setup.

F22. Two distinct classes of storms that affect the project area are northeasters (extra tropical) and hurricanes (tropical). Northeasters, named after the predominant direction of winds, are large-scale low pressure disturbances which usually occur from November through March. The severity of a northeaster is not as great as that of a hurricane. Although wind gusts can reach hurricane strength in a very severe northeaster, sustained wind speeds are rarely greater than 50 knots. The flood damage caused by the typical northeaster is often more a function of its duration rather than its intensity, as the longer storms have more opportunity to destroy both natural and engineered flood protection features. Also, as northeasters frequently last two to three days, it is possible for the storm to act during several periods of high astronomical tide. Hurricanes are rarer occurrences in the project area than northeasters. By the time hurricanes approach the latitudes of the north New Jersey coast, they are usually in a state of energy loss and are beginning to decay into the category of a tropical storm. The average period between hurricanes is about 5.7 years, or a probability of a hurricane in any given year, on the average of 0.175. Despite their infrequency and short duration, hurricanes have the potential to be devastating in the project area because of their high wind speed and high storm surge.

F23. A stage-frequency curve is a relation of flood water elevation to the probability of that elevation being equalled or exceeded. A storm surge curve was developed by the Corps of Engineers Coastal Engineering Research Center in January 1996. The combined hurricane and northeaster curve was used for design purposes in this investigation. The combined storm stages in the project area are presented in Table F-2.

F24. Sea level rise has been found to be a factor contributing to coastal erosion. Based on NOAA tide gauge readings between 1933 and 1986 at Sandy Hook, sea level has been increasing by an average of approximately 0.014 ft. per year. Tidal flooding is expected to increase in severity in direct relation to this increase which yields a rise of 0.74 feet over a 50-year period.

TABLE F-2
Tidal Stage Frequency (in Feet NGVD) for Port Monmouth, New Jersey
South Harbor Node (Lat 40.48N, Long 74.19W)

Return Period (Years)	Storm Type				
			Combined		
				95% Confidence Limit	
	Tropical	Extra Tropical	Water Level	Lower	Upper
2		6.3	6.5	6.5	6.5
5		7.4	7.6	7.5	7.8.
10		8.0	8.4	8.2	8.6
15		8.4	8.9	8.7	9.1
20	7.5	8.6	9.2	8.0	10.5
25	8.2	8.8	9.6	8.3	10.9
44	9.6	9.3	10.4	8.9	11.8
50	9.9	9.4	10.5	8.9	12.1
100	11.7	10.1	12.2	9.8	14.6
150	13.5	10.3	13.8	10.5	17.1
200	14.4	10.5	14.7	10.7	18.7
250	15.1	10.6	15.5	10.8	20.2

Source: Based on MTL Data from USACE (1996)

F25. Pews Creek. This creek divides Port Monmouth and Keansburg and is dredged periodically for navigation. The navigation project consists of a channel to the Monmouth Cove Marina with a 350-foot-long stone jetty to stabilize the east bank of the bay entrance and a recently constructed west bank jetty. Parallel bulkheads approximately 425 feet long stabilize the channel from the bay shoreline to the marina. The inlet channel was recently widened to approximately 60 feet through this reach.

The marina entrance is located approximately 825 feet from the tip of the jetty. The marina size is approximately 115,000 square feet and borders the creek for approximately 500 feet (Harrington, 1994). Dredge depths of the project are 8 feet below MLW for the channel and 6 feet below MLW for the basin (Sardonia, 1996), which is designed to maintain 6 ft. below MLW and 5 ft. below MLW navigation depths, respectively.

F26. The Middletown Township Master Drainage Plan shows an Intermediate Regional Flood (IRF) (approximately 100 yr.) Peak at Bray Avenue of 752 cfs for a drainage area of 713 acres. The Middletown Township Flood Insurance Study considered tidal flooding along Pews Creek. There are several bridges over Pews Creek including the reconstructed Port Monmouth Road Bridge. This new structure consists of five 35 ft. spans, three crossing the channel and one crossing each overbank. The bottom of the bridge deck was set above 14 ft. NGVD.

F27. The current in Pews Creek was measured on July 13, 1993, and was used to predict average current velocities in a study conducted by Stevens Institute of Technology (Harrington, 1994). The measurements were used to calibrate a model and calculate neap and spring tidal current speeds. Under spring tidal conditions the average peak flood and ebb currents were approximately 1.7 fps (1.0 knots) and 1.8 fps (1.1 knots) respectively at the critical cross section. Neap tide conditions produced flood and ebb current speeds of 0.5 fps and 0.4 fps. In addition, the investigation estimated the change in tidal elevations between the bay and the marina under neap and spring tidal conditions. Under both conditions, the range of tides were nearly identical at the bay and marina boundaries of the creek. The marina is located approximately 500 feet from the open bay.

F28. <u>Compton Creek</u>. This creek separates Belford and Port Monmouth and is the site of a Federal navigation project. A rock rubble jetty stabilizes the east side of the inlet. A concrete seawall protects the south side of the east channel bank. The project depth inside the mouth of the creek is 8 feet below mean low water (MLW) with a width of 75 feet. Outside of the mouth the project depth is 12 feet below MLW with a width of 150 feet.

F29. Although there are no plans to replace the Main Street Bridge over Compton Creek, reconstruction of Church Street and the Church Street bridge is currently underway. The new Church Street bridge will consist of 3 spans of 30 feet each. The Church Street roadway will be raised to approximately elevation 9 ft. NGVD and will improve emergency access.

F30. The Middletown Township Master Drainage Plan shows an IRF Peak of 1,210 cfs at Campbell Avenue for a drainage area of 3,631 acres (5.674 square miles). The IRF peak is available at a number of locations within the basin, including 10 nodes on Compton Creek itself and at the mouths and upstream intermediate points of six tributaries. In addition, the FEMA Flood Insurance Study includes estimated peak flows for four hypothetical events; the FEMA study used a basin area of 5.86 square miles. For the 100-yr event, the estimated peak flow is 1,326 cfs.

F31. Calculations for regional detention basins proposed for portions of upper Compton Creek were available from T&M Associates, who also prepared the original Master Drainage Plan. While the detention basins were apparently never built, the parameters provide an indication of hydrologic conditions in the basin. The calculations covered approximately 1,155 acres divided into 7 subareas in the upper basin. For the five subareas for which data was available, Runoff Curve Number. (RCN) values varied from 58 to 77, averaging 64, and times of concentration varied from 0.83 to 1.53 hours.

F32. The Flood Insurance Study indicates that flood elevations on Compton Creek without consideration of tidal backwater are far below the tidal flood stages. The 100-year flood stage at Campbell Avenue calculated without tidal effects is 7.0 ft. NGVD, just slightly above the 2-year tidal stage of 6.5 ft. NGVD. This strongly suggests that flooding in this area is almost exclusively due to tides on Raritan Bay.

F33. Review of Historic Storms. Precipitation and tidal data was obtained and reviewed for the period 1939 through 1992. Using these data, an assessment was made of the dependence between significant precipitation and tidal events. The following paragraphs describe the review and assessment in more detail.

F34. Data and Data Sources. Precipitation totals for historic storms were determined using databases and interfacing software developed by EarthInfo Inc. The databases utilized for this analysis are commercially distributed versions of National Climatic Data Center (NCDC) files containing daily precipitation totals as well as 60 and 15 minute interval precipitation amounts. A total of 57 years of precipitation data were considered in the analysis, starting in 1936 and continuing through the devastating storm of December 10-12, 1992.

F35. The search of precipitation databases followed two basic paths. The first analysis was performed in order to identify all precipitation events capable of creating interior flooding. Preliminary analyses conducted during the Reconnaissance Phase of this study indicated runoff from a 2-year 24-hour storm (3.4 inches of rain) would generate sufficient runoff volume to cause interior damage if gravity outlets were blocked by high exterior stages. The analysis to identify potentially damaging precipitations utilized a database filter to select all daily precipitation totals at Long Branch, New Jersey, exceeding 3 inches. A total of 39 storms were identified over the 56 year period, an average of 1 event per 1.4 years.

F36. The second investigation of the precipitation databases was performed to quantify the amount of precipitation associated with 25 known tidal flood events. For storms occurring prior to 1950, this analysis also used the Long Branch daily precipitation data. For storms in 1950 or later, the analysis used hourly precipitation data from Long Branch. Gaps in the hourly data file were selectively supplemented with either hourly precipitation from Rahway, New Jersey, or the daily totals at Long Branch.

F37. The tidal flood stages presented in Table F-3 were tabulated from a variety of published COE reports including the Raritan and Sandy Hook Bay NJ Cooperative Beach Erosion Control and Interim Hurricane Study (Survey), Nov., 1960; Report on Hurricane of 12 September 1960 (Donna), February 1961; Report on Storm of 6-8 March 1962, February 1963; Post Storm Evaluation for the March 29, 1984 Northeaster, March 1988; New Jersey Shore Protection Study Report of Limited Reconnaissance Study, September 1990. By far the most important of these resources was the 1960 Survey Report which provided a description and detailed listing of storm tides for every storm affecting the area prior to Hurricane Donna in 1960. Where possible this analysis sought to identify precipitation associated with all storms exceeding the zero damage threshold of 5.0 ft. NGVD.

F38. Evaluation of Peak Precipitation Events. The 39 potentially damaging Precipitation events identified from the daily precipitation database were tabulated and ranked in accordance with total 24 hour precipitation as seen in Table F-3. Weibull plotting positions were assigned and a precipitation frequency curve developed to determine whether the data reasonably reproduces accepted precipitation probability data. As seen in Figure F-9, the data selected for this analysis is similar to published precipitation values such as the 24-hour SCS 2-year, 25-year, and 100-year totals of 3.4, 6.0 and 7.5 inches, respectively, used in the Reconnaissance phase of this study. Accordingly, the data is considered suitable for use in evaluating the relationship between these precipitation events and tidal flood events.

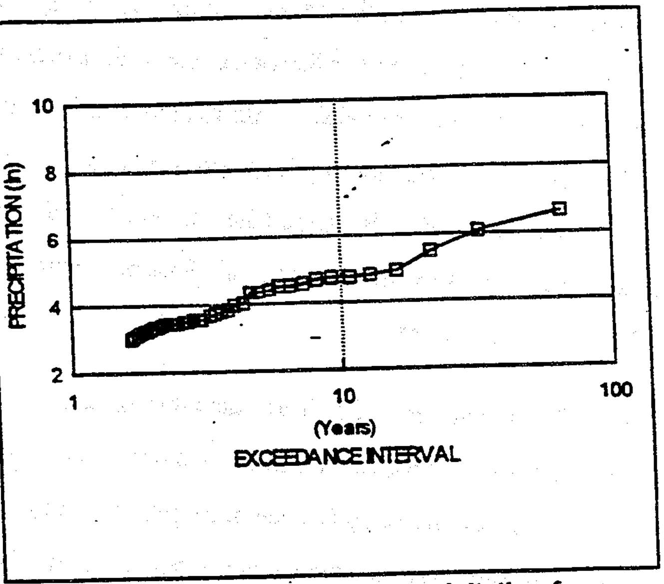

Figure F-9. Weibull plot of daily precipitation from 1936 to 1992 at Long Branch, NJ

F39. The record of significant precipitation events was compared to the description of tidal storms in the 1960 Hurricane Survey Report to identify the meterologic origins of the storms. This comparison considered only the 24 largest daily precipitation events which occurred during a period of 24 years coinciding with the detailed record of storms in the Survey Report. This analysis indicated that 12 of the daily precipitation peaks were associated with storms known to have caused tidal flooding. Of these 12 daily precipitation totals, nine were associated with storms of tropical origin and three were associated with storms of extatropical origin. Weibull plots of daily precipitation segregated by coincidence with tidal flooding (shown in Figure F-10) indicates that many of the largest

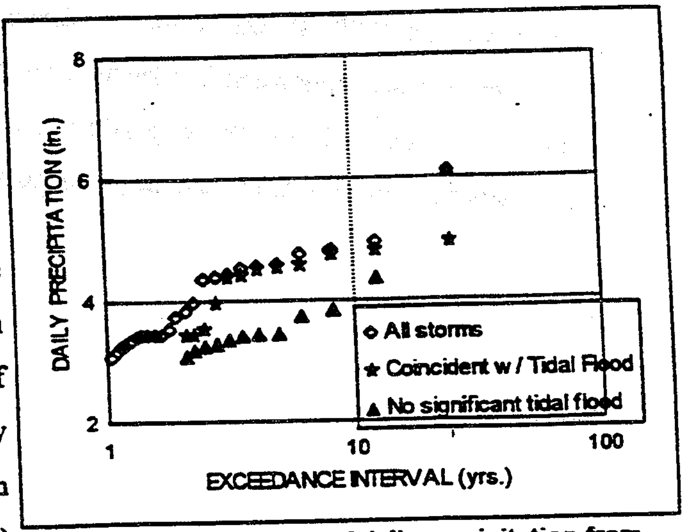

Figure F-10. Weibull plot of daily precipitation from 1936 to 1960 for Long Branch, NJ segregated by coincidence and tidal flooding

precipitation events are associated with the storms causing tidal flooding. The remaining 12 storms which occurred during the summer (2 in June, 4 in July, 3 in August, and 1 in Sept.), were reviewed and no tropical storms or hurricanes were identified.

TABLE F-3

Precipitation Data for Selected Tidal Floods

Storm Date	Precipitation Data				Tidal Data Peak @ Sandy Hook		Rainfall Data Reference
	24 Hour		1 Hour				
	Amount (inches)	Date	Amount (inches)	Time	Stage (ft NGVD)	Type	
09/21/38	4.54	09/21/38			5.85	H	LB - daily
03/03/42	1.07	03/03/42			5.0	ET	LB - daily
10/26/43	4.81	10/26/43			5.5	ET	LB - daily
09/14/44	4.94	09/14/44			7.7	H	LB - daily
11/30/44	0.88	11/30/44			5.5	ET	LB - daily
1/16/45	1.37	1/16/45			5.4	ET	LB - daily
11/25/45	1.39	11/22/45			5.1	ET	LB - daily
12/6/45	0.25	12/6/45			5.0	ET	LB - daily
11/9/47	2.13	11/8/47			5.1	ET	LB - daily
11/25/50	2.60	11/25/50	0.51	1700	7.2	ET	LB - hourly
12/8/50	0.82	12/7 1500 to 12/8 0800	0.23	0100	5.5	ET	LB - hourly
11/1/51	1.27	11/1/51	0.36	0900	5.3	ET	LB - hourly
10/23/53	0.18	10/25/53	0.30		5.8	ET	LB - hourly
11/6/53	1.55	11/6 1100 to - 11/7 0600	0.20	0500	7.9	ET	LB - hourly
5/3/54	0.60	5/3/54	0.30	1300	5.0	ET	LB - hourly
8/31/54	2.21	8/31/54	0.36	0500	6.4	H	LB - hourly
10/15/55	0.24	10/15/55	0.10	1600	6.2	ET	LB - hourly
1/10/56	0.43	1/10/56	0.21	0800	5.3	ET	LB - hourly
3/20/58	1.49	3/20/58	0.10	0200	6.2	ET	LB - hourly
9/12/60	4.41	9/12/60	1.39	1100	8.6	H	LB - hourly
3/7/62	1.15	3/6/62	0.15	0900	7.8	ET	Rahway - hourly
3/29/84	1.43	3/28 1100 to 3/29 1100	0.20	0300	6.4	ET	Rahway - hourly
9/28/85	1.67	9/28/85	.70	1100	7.0	H	LB daily (Freehold)
12/11/92	2.02	12/11/92	0.21	0600	8.68	ET	Rahway - hourly

NOTE: LB = Long Branch

F40. Evaluation of Significant Tidal Events. Of the 24 tidal events selected for review, five are of tropical origin and 19 are of extratropical origin. The average stage at Sandy Hook was slightly over 6 ft. NGVD and the average daily precipitation was approximately 1.8 inches. The peak daily precipitation amounts associated with these storms were highly variable with a standard deviation of approximately 1.4 inches and a 78% coefficient of variation (see Figure F-11).

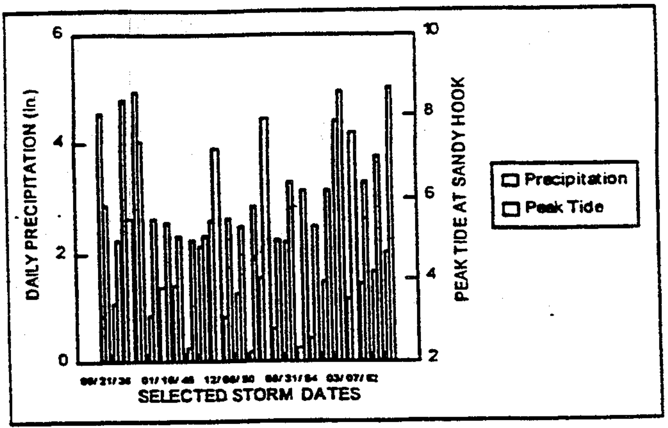

Figure F-11. Tides and precipitation for selected coastal storms at Sandy Hook, NJ

F41. Assessment of Dependence. The comparison of meteorologic causes of tidal events and significant precipitation events provides a critical component of the current analysis. The most important data in this regard are the peak daily precipitations and detailed storm descriptions for the period 1936 to 1960. These data indicate that there is an estimated 50% chance that a damaging precipitation event will occur due to the same meteorologic cause as a tidal flood event. The remaining significant precipitation events are assumed to be associated with thunderstorms or other events which do not generate significant tidal surge.

F42. Although the assessment of dependence indicated that only ½ of the precipitation events of concern are dependent on storms causing tidal flooding, many of the largest daily precipitation totals are associated with hurricanes or other storms of tropical origin. Since hurricanes also dominate the high end (>50 year) of the stage frequency curve, the potential for dependence between the most severe precipitation and tidal storms should be considered.

Data Analysis.

F43. Coincidence. Coincidence of precipitation and ocean surges may or may not be present along with dependence of same. The relative timing of precipitation and ocean surges must be considered to assess coincidence. That in part is determined by the times of concentration of the watersheds from which the precipitation causes flood runoff.

F44. For storms in which there is dependence between precipitation and tidal flooding, the hydrologic conditions of the basin determines whether there will be coincidence. Since both Compton and Pews Creeks, as well as the interior drainage areas associated with likely levee alternatives, have relatively short times of concentrations, runoff from these drainage areas will be coincident with the tidal surge. Review of mass precipitation curves and tidal hydrographs for the hurricanes of 1944 and 1960 suggests that the centroid of the precipitation mass curve typically precedes the peak of the storm surge. Such a condition would tend to put the peak runoff nearly coincident with the peak storm surge.

F45. Correlation. Correlation analyses were performed in an attempt to determine if there were any meaningful relationships between the elevation of tidal flooding and the depth of precipitation. This analysis considered both the peak tide and peak precipitation data files previously described. The initial analyses evaluated any direct relationships between precipitation and tides. The strength of the relationship is described by the coefficient of determination (R-squared), which represents the percentage of variation attributed to the relationship. Of all the data groupings considered, the strongest direct relationship between tides and daily precipitation was found in the analysis of peak precipitation events from 1936 to 1962. The coefficient of determination for this data, however, was only 0.184.

F46. Since direct comparisons between tidal stages and precipitation indicated little if any correlation, the analysis considered a series of data transforms based on the following two hypotheses.

1. Relationships could be linearized by using the log of either the precipitation or tidal stage.
2. Substitute, or dummy, values of 0 or 1 could be used to represent the absence or presence of a tidal flood.

F47. The strongest correlation was found between the tidal stage and the log of precipitation for peak daily precipitation events over the period 1936 to 1962. Although applying the data transform resulted in a stronger correlation, the coefficient of determination is still a relatively weak 0.23. This relationship is displayed in Figure F-12.

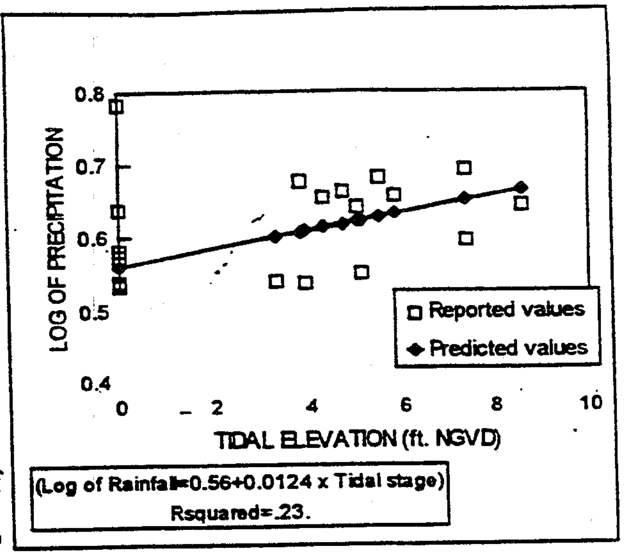

Figure F-12 Port Monmouth Feasibility Study sample correlation results

F48. Summary of Findings. The review of historical precipitation and tides was used to establish the meteorologic origins of storms and the impact on dependency between interior and exterior flood events. This analysis indicated that approximately 50% of significant precipitation events are dependant on the same conditions which result in tidal flooding. Most of the 10 largest daily precipitation values are associated with hurricanes or tropical storms. The average tidal stage of storms generating both significant tidal surge and significant precipitation is approximately 5.4 ft. NGVD at Sandy Hook. Available data also suggest that it would require an extremely rare flood on Pews or Compton Creeks to create flood depths similar to the 2-year tide.

F49. Since the project area consists of relatively small drainage basins with short times of concentration, it is anticipated that for tropical storms or Northeasters the timing of interior and exterior flooding will be coincident. Comparison of the precipitation mass curve to the tidal hydrograph for hurricane Donna indicated that the most intense precipitation occurred in the 2-3 hours before and 1 hour after the peak tide. Information available for the 1938 and 1944 Hurricanes also indicates that the most intense precipitation occurs as the eye of the storm approaches. Attempts to quantify any correlation between the amount of precipitation and peak tide level during storms did not provide any useful mathematical relationships.

F50. Since the review of historic storms indicates that approximately 50% of the significant precipitation events will be accompanied by coincident tidal flooding which could prevent gravity outflow and result in significant damage, the modeling of interior flood depths must consider tidal tailwater. The average peak tidal stage at Sandy Hook for storms with significant precipitation (>3 inches) was 5.4 ft. NGVD and the average peak tidal elevation for all tidal events examined was 6.0, both slightly below the current 2-year tidal stage of approximately 6.5 at that location. Accordingly it is considered reasonable to use the 2-year storm elevation as the hydrograph peak in modeling tailwater at drainage outlets along Compton Creek. The use of a 2-year stage is consistent with prior study results, such as the Passaic River correlation analysis (GDM, 1983) and reasonably reproduces typical historic conditions. It is also important to recognize that the range of possible tailwater conditions is extremely variable and can not be predicted with certainty.

F51. **Recommended Analysis Approach**. The interior drainage analysis for Compton Creek was based on a tailwater condition similar to a 2-year storm, with the peak surge elevation occurring shortly after the precipitation peak. A stage hydrograph for a 2-year tidal storm was synthesized as shown in Figure F-13 by combining astronomic tides and storm surge components. Scaling the tide range to the M_2 tide frequency approximates a typical astronomic tide. Similarly, applying a sine curve to a 24 hour storm with a maximum storm surge of 3.4 ft. results in the storm surge relationship shown in Figure F-13. Combining the storm surge and astronomic tide components yields a reasonable exterior storm hydrograph for use in modeling interior runoff. The peak hydrograph stage of 6.5 ft. NGVD occurs 13 hours after the start of the 24 hour storm, approximately coincident with runoff peaks.

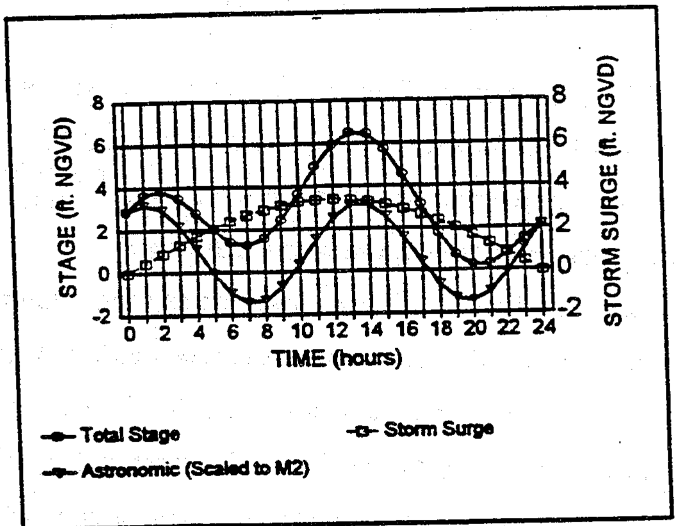

Figure F-13. Synthesized 2 year storm hydrograph

F52. Along Pews Creek it was determined that, due to the large interior area, there is a higher probability that the significant interior runoff events are dependent on the occurrence of a tropical storm. The tidal surge results for tropical storms (see Table F-2) were reviewed, and it was determined that a peak exterior stage of approximately 8.2 feet was more representative of a typical tropical event. This exterior condition was synthesized by combining the astronomic tides with a 5-foot storm surge. This exterior condition was considered appropriate given the high dependence between historic peak 24-hour rainfalls and the occurrence of tropical storms.

F53. The impact of uncertainty in the exterior stage hydrograph on interior flood stages was incorporated in the economic assessment of the interior alternatives. The procedures used to define uncertainty are constrained by the limitations of the HEC-FDA program which was used in the economic assessments. The HEC-FDA package is designed to conduct risk based formulation studies by integrating uncertainties in both economic and hydraulic/hydrologic parameters. Within the program, the only way to define uncertainty in the interior stage frequency relationship is by varying the equivalent record length in an order statistic analysis. Since confidence bands can not be directly input into the analysis, the HEC-FDA program will only use an approximate representation of any calculated uncertainty band.

F54. Given these limitations in the ultimate use of the results, the hydrologic analysis procedure used to estimate uncertainty did not need to be overly rigorous or complex. The procedure considered three tailwater conditions in the HEC-IFH analysis of Minimum Facility to define the magnitude of uncertainty in interior stages. The most likely interior stage for a given precipitation event was defined by routing against the synthesized 2-year tide. The low estimate of interior stages was obtained by routing against a commonly occurring condition such as mean high water. Since the historic data suggests that there is physical dependence between high storm surge and heavy precipitation during tropical events, the high estimate of interior stages was based on tailwater conditions during a significant hurricane, namely Hurricane Donna (September 1960).

F55. For each interior drainage area, HEC-IFH stage vs. frequency results were tabulated for each of the tailwater conditions. The most likely interior stage vs. frequency values, calculated using the 2-year exterior hydrograph, was input to the hydraulics module of HEC-FDA. The 'equivalent record length' variable was changed on a trial and error basis and the resulting program-generated confidence bands were tabulated. The minimum equivalant record length available in HEC-FDA of 10 years was selected as the best representation of the wide range of possible interior stages. As various interior drainage alternatives were formulated, HEC-IFH routings were performed against the 2-year exterior hydrograph. Using the record length selected as described above, HEC-FDA simulations of interior stages, damages and benefits were performed.

Hypothetical Rainfall Data

F56. Data for hypothetical storm events with storm durations of 2 hours through 24 hours were developed from Technical Paper No. 40, Rainfall Frequency Atlas of the United States. For shorter durations, 5 minutes through 60 minutes, the NWS Hydro-Meteorological Report No. 35 was utilized.

F57. Rainfall depths for the 48-hour duration were provided based on Technical Paper No. 49 ("Two- to Ten-Day Precipitation for Return Periods of 2 to 100 Years in the Contiguous United States"). The 500-year rainfall depths were determined by extrapolation on log-probability graph of the 2 to 100-year data.

F58. Hypothetical point rainfall depths for the events analyzed from the 2-year to 500-year return periods are shown in Table F-4.

TABLE F-4
Hypothetical Rainfall Data

Duration		Rainfall Depth (in.) for each Hypothetical Event					
		2 yrs	5 yrs	10 yrs	50 yrs	100 yrs	500 yrs
5	minutes	0.42	0.50	0.55	0.70	0.77	0.93
15	minutes	0.80	1.00	1.12	1.47	1.62	2.00
1	hour	1.40	1.80	2.06	2.75	3.05	3.80
2	hours	1.80	2.30	2.70	3.45	3.90	4.80
3	hours	2.00	2.60	2.95	3.85	4.30	5.40
6	hours	2.40	3.10	3.65	4.65	5.35	6.40
12	hours	2.90	3.70	4.35	5.50	6.35	7.70
24	hours	3.40	4.40	5.15	6.50	7.35	8.80
48	hours	3.90	5.40	6.00	8.00	9.20	11.00

Historical Rainfall Data

F59. Analysis of an historical event was performed using data from Hurricane Donna (September 11-12, 1960). Hourly rainfall data is from National Climate Data Center (NCDC) records as tabulated by on CD-ROM by Earthinfo, Inc.; data is from the NCDC's TD-3240 file. The rainfall data from the Long Branch Station in New Jersey was found to be most representative of Port Monmouth area, and was used. Table F-5 summarizes the hourly rainfall data of Hurricane Donna, as registered at Long Branch Station.

TABLE F-5

Hurricane Donna Rainfall Data

(September 1960)

Date: September 11, 1960

Hour	01	02	03	04	05	06	07	08	09	10	11	12
Rainfall (in.)	0	0	0	0	0	0	0	0	.02	.25	.03	.17
Cumulative Rainfall (in.)	0	0	0	0	0	0	0	0	.02	.27	.30	.47

Date: September 11, 1960

Hour	13	14	15	16	17	18	19	20	21	22	23	24
Rainfall (in.)	.27	.21	0	.01	0	.40	0	.07	.11	0	0	.01
Cumulative Rainfall (in.)	.74	.95	.95	.96	.96	1.36	1.36	1.43	1.54	1.54	1.54	1.55

Date: September 12, 1960

Hour	01	02	03	04	05	06	07	08	09	10	11	12
Rainfall (in.)	.10	.02	.13	.05	.07	.04	.04	.15	.57	.72	1.39	.85
Cumulative Rainfall (in.)	1.65	1.67	1.80	1.85	1.92	1.96	2.00	2.15	2.72	3.44	4.83	5.68

TABLE F-5

Hurricane Donna Rainfall Data

(September 1960)

Date: September 12, 1960

Hour	13	14	15	16	17	18	19	20	21	22	23	24
Rainfall (in.)	.15	.03	.09	.01	0	0	0	0	0	0	0	0
Cumulative Rainfall (in.)	5.83	5.86	5.95	5.96	5.96	5.96	5.96	5.96	5.96	5.96	5.96	5.96

F60. After preliminary results of the historical storm of March 1962, it was determined that this storm does not generate significant flooding (total rainfall of 1.15 inches), and no further evaluation with this storm was performed.

Delineation of Drainage Areas

F61. Mapping Sources. Interior drainage basins of Compton Creek were delineated on Plans surveyed and developed by Rogers Surveying at a scale of 1" = 200' (also available in a scale of 1" = 100' and 1" = 40') with 2-foot contour intervals, dated April 1995. Mapping information was further supplemented by field inspections.

F62. "Middletown Township Master Drainage Plan," Volume 5, prepared by T&M Associates was also utilized. This Plan shows the Pews Creek drainage basin delineation on a 1" =1300' scale drawing. Also a 1"=2000' scale USGS map with 10-foot contour intervals was utilized. The plan developed by Rogers Surveying includes the area north of Route 36 to Sandy Hook beach and a small section to the south of Route 36, and does not cover the complete drainage basin of Pews Creek.

F63. Delineation Methods. For purposes of this analysis, an "interior drainage area" is a distinct land area which drains to one primary outlet location behind the proposed line-of-protection. The identification of such areas is complicated by the presence of anthropogenic features such as storm sewers which divert flow into or out of a drainage area. In some cases, otherwise adjacent distinct interior areas become combined during rare storm events due to high ponding depths directly behind the line of protection works.

F64. The Middletown Township Master Drainage plan provides the drainage basin delineation for Pews Creek. It also includes tabulated data that describe the sub-basins locations and provides the creek length in each sub-basin with its average slope and tributary area in acres. The drainage sub-basin delineations were tranferred to the USGS map and the areas were planimetered for verification. For Pews Creek sub-basins PC2 and PC3 the planimetered areas were found to be in very close agreement with those of the Master Plan (a difference of 0.2% & less). However, for sub-basins PC1 and Lower, significant differences were observed (+86.7 acres and -146.1 acres for PC1 and Lower, respectively). The Lower Sub-basin is now limited by the proposed line of protection and does not extend to the shore, as shown in the Master Plan. Some of the reduction to this area is the result of this change to the boundary.

F65. The hydraulic paths for each sub-basin were developed using the available maps noted above. The line of protection with the location of the storm gate is shown in Figure F-2. The basin areas and hydraulic paths used to develop the hydrologic models are shown in Figure F-14A.

F66. The locations of interior drainage areas and basin divides for Port Monmouth were delineated and the interior drainage created behind the line of protection for Compton Creek area was divided into three basins. The basin areas and hydraulic paths used to develop the interior drainage facilities are shown in Figure F-14B.

F67. <u>Existing Drainage Systems</u>. Several existing drainage systems consisting of catch basins, manholes and storm sewer pipes, currently serve the Port Monmouth interior drainage area. These pipes range in size from 12" to 36" diameter, although a 19" x 30" ERCP is also present. Table F-6 provides a summary of the outlets by basins. Figures F-2 through F-8 show the location of these outlets.

TABLE F-6

Port Monmouth Interior Drainage

Existing Gravity Outlets

Basin	Outfall Designation	Outfall Location	Type	Size	Existing Inv. El. (ft. NGVD)
C1	CC-11	Warsha Ave.	RCP	15"	5.3
	CC-12	Wilson Ave (±250' S. of Campbell Ave.)	RCP	15"	5.3
	CC-13	Willow Street	RCP	15"	4.1
C2	CC-7	Main St./Broadway Intersection	RCP	18"	4.2
	CC-8	Creek Rd. (±900' E. of Wilson Ave. Int.)	RCP	19"x30"	3.1
	CC-10	Creek Rd. (±100' N. of Campbell Ave.)	RCP	24"	2.5
	CC-150	Wood Stock Ave.	CIP	12"	Not avail.
C3	CC-1	±200' S. of Park Ave.	CIP	15"	3.1
	CC-2	Main St. (±200' S. of Park Ave. Int.)	CIP	12"	2.2
	CC-6	Main St. (±500' S. of Renfrew Pl. Int.)	RCP	18"	Not avail.
	CC-128	Main St. (±400' S. of Renfrew Pl. Int.)	RCP	15"	2.5
	PC-6	Wilson Ave. (±350' N. of Renfrew Pl. Int.)	RCP	36"	Not avail.
		Redirected to Main St. (±350' N. of Renfrew Pl. Int.)			-.3 (Prop)

Development of Interior Runoff Hydrographs

F68. General Description. Hydrologic models were developed to estimate flood runoff hydrographs of interior drainage areas, using HEC-IFH for Compton Creek drainage and HEC-1 for Pews Creek drainage. (See following paragraphs for further discussion.) In both models, runoff hydrographs are computed based on precipitation data, Soil Conservation Service (SCS) runoff curve numbers, SCS dimensionless unit hydrographs and other hydrologic data. These are described in more detail in subsequent sections.

F69. SCS Runoff Curve Numbers. The Soil Conservation Service (SCS) Runoff Curve Number procedure as outlined in TR-55 (Urban Hydrology for Small Watersheds) was used to calculate precipitation losses for both the HEC-IFH and HEC-1 models. The Runoff Curve Numbers (RCN) are based on factors such as soil type, land use, ground cover, and antecedent moisture conditions.

F70. Soils. The Monmouth County Soil Survey Map, prepared by the Soil Conservation Service (SCS; now Natural Resources Conservation Service), was reviewed to determine the hydrologic soil group(s) (HSG) extant within the Pews Creek and Compton Creek interior drainage areas. Soil types within the project area are: SS and HV (in wetland areas), UD, KUA, UA, Fb, WnB, KIA, At, HLA and TUB (in developed areas), and HWA (shoreline areas).

F71. Most of these soils are assigned by SCS to an HSG, as well as being assigned an RCN. However, the SS, HV, UD and UA soils have not been assigned. Thus assignments were made based on a conversation with Port Monmouth Soil Survey personnel and the results of borings performed in the area (in 1990 and 1997). The SS soil consists of frequently flooded sulfaquents and sulfihemits. Thus, the soil has a very limited capability of absorbing more rainwater and therefore it was assigned to HSG D. A curve number of 89 was also assigned since type D has the lowest potential infiltration rate.

F72. Humaquepst (HV) soils have very similar characteristics as the SS soils. Therefore, HV soils were also assigned to Type D HSG.

F73. The UD and UA soils consist of udorthents, wet substratum and urban land complex to which HSG can be assigned only based on the result of borings. Borings taken in the project area in 1990 and 1997 show the soil to be sand or a combination of sand, silt and clayey sand. Depth to groundwater is generally only 2 to 3 feet, with a range of 1-13 feet. Based on the generally high groundwater elevations and noted soils components, infiltration is limited and these soils can be regarded as shallow sand which corresponds to HSG B.

F74. Figures F-15A and 15B show HSG distribution within the interior drainage areas.

F75. Land Use. Land use for all areas was determined based on field inspection and available maps. To estimate percent impervious area for residential areas within the Compton Creek basin, pilot areas containing 10 to 20 lots in each sub-basin were evaluated. Land use is also shown in Figure F-16.

F76. For the Pews Creek area, the interior drainage area is predominantly residential and estimated to consist of ⅓-acre lot sizes with an average of 30% impervious area. The commercial area is situated along Route 36. Open space generally surrounds Pews Creek, although there are a couple of small patches of open space within Pews Creek Sub-basins PC1 and PC2. (See Figure F-12.)

F77. Table F-7 summarizes the runoff curve number data for sub-areas within both the Compton Creek and Pews Creek interior drainage areas.

TABLE F-7

Runoff Curve Number Data

Sub-Basin	Land Use	Area (acre)	% Area	Soil Type	HSG	CN	% Impervious	Composite CN
Compton Creek Sub-basins								
C1	Residential	35.62	74.75	UD	B	69	25	76
	Open Space	3.55	7.45	UD	B	69	0	69
	Commercial	7.36	15.45	UD/KUA	B	92	85	97
	Wetland	1.12	2.35	SS	D	89	0	89
	Total	47.65	100.00					78
C2	Residential	48.81	96.00	UD	B	69	28	77
	Wetland	2.03	4.00	SS	D	89	0	89
	Total	50.84	100.00					78
C3	Residential	59.01	74.84	UD	B	69	35	79
	Open Space	3.40	4.33	HWB	A	49	0	49
	Open Space	1.66	2.12	SS	D	89	0	89
	Open Space	14.67	18.71	UD	B	69	0	69
	Total	78.41	100.00					76

TABLE F-7
Runoff Curve Number Data

Sub-Basin	Land Use	Area (acre)	% Area	Soil Type	HSG	CN	% Impervious	Composite CN
Pews Creek Sub-basins								
PC1	Open Space	16.2	8.6	TUB	A	49	0	49
	Residential	62.6	33.2	TUB	A	39	30	57
	Open Space	6.4	3.4	UD, HLA	B	69	0	69
	Residential	87.9	46.6	UD, HLA	B	61	30	72
	Open Space	11.7	6.2	At	D	89	0	89
	Residential	3.9	2.0	At	D	80	30	85
	Total	188.7	100.0					66
PC2	Residential	42.9	8.8	TUB	A	39	30	57
	Commercial	2.3	0.5	TUB	A	39	85	89
	Open Space	49.9	10.3	UD, HLA	B	69	0	69
	Residential	356.2	73.3	KUA	B	61	30	72
	Commercial	20.9	4.3	KUA, UD	B	61	85	92
	Residential	1.8	0.4	WnB	C	74	30	81
	Open Space	11.8	2.4	Hv	D	89	0	89
	Total	485.8	100.0					72
PC3	Residential	4.0	3.1	TUB	A	39	30	57
	Residential	86.1	67.8	UD, UA, KUA	B	61	30	72
	Commercial	25.2	19.9	UD, UA, KUA	B	61	85	92
	Open Space	7.4	5.8	SS	D	89	0	89
	Residential	4.3	3.4	SS	D	80	30	86
	Total	127.0	100.0					77
Lower	Residential	145.4	35.4	UA, UD	B	61	30	72
	Open Space	140.4	34.2	SS	D	89	0	89
	Residential	125.1	30.4	SS	D	80	30	86
	Total	410.9	100.0					82

F78. **T_C - Time of Concentration**. The hydraulic paths yielding the greatest times of concentration were identified for each sub-basin within the two interior drainage watersheds. The locations of these paths were determined through the use of available maps described previously. The lengths and slopes of the hydraulic paths were estimated and the results used to calculate the time of concentration for each basin according to the type of flow and surface conditions. Hydraulic paths are shown in Figure F-14. SCS TR55 was used to calculate the times of concentration.

F79. T_C's for some of the open channels within Pews Creek were initially estimated by assuming initial velocities of 2 fps to 4fps. These initial T_C's were used in HEC-1 with the 100-year storm and the resultant velocities from HEC1 output (@ KPC1, KPC2 & KPC3) were then used to re-estimate the time of concentration for open channel flow.

F80. **SCS Dimensionless Unit Hydrograph**. The SCS Dimensionless Unit Hydrograph employs a dimensionless table of flow rates versus time. The peak flow rate and time to peak of the unit hydrograph is computed from equations developed by the SCS. The unit hydrograph is interpolated in the aforementioned table based on the lag time, in hours, between the mass of the rainfall excess and the peak of the unit hydrograph. These basin lag times (T_{LAG}) were developed for each basin. Both HEC-IFH and HEC-1 use T_{LAG} to compute the time to peak of the unit graph.

F81. T_{LAG} was estimated based on:

$$T_{LAG} = 0.6 \times T_C$$

where T_C is the time of concentration

F82. Lag times and other related hydrologic data for Pews Creek and Compton Creek are summarized in Table F-8.

TABLE F-8
Summary of Hydrologic Data

Sub-Basin	Area (acres)	Area (sq. mi.)	SCS Runoff Curve No.	T_C (hours)	T_{LAG} (hours)
C1	47.7	0.0744	78	1.21	0.73
C2	50.8	0.0794	78	1.16	0.70
C3	78.4	0.1225	76	1.12	0.67
Total	176.9	0.2764			
PC1	188.7	0.2984	66	1.07	0.64
PC2	485.8	0.7591	72	2.06	1.24
PC3	127.0	0.1984	77	2.01	1.21
Lower	410.9	0.6420	82	1.45	0.87
Total	1,221.6	1.9087			

F83. **Routing**. On Pews Creek, the Muskingum-Cunge method was utilized for channel routing between the sub-basins, based on a trapezoidal channel configuration. Table F-9 summarizes the routing data. Reaches within the Compton Creek interior sub-basins were found to be too short to require routing.

TABLE F-9
Routing Data - Pews Creek

From	To	Method	Length (ft)	Slope (ft/ft)	Roughness Coeff. (n)	Shape	Bottom Width (ft)	Side Slope (H/V)
PC1	PC2	Muskingum-Cunge	4,400	0.002	0.035	Trap.	8	3/1
PC2	PC3	Muskingum-Cunge	1,100	0.0004	0.035	Trap.	10	3/1
PC3	Lower	Muskingum-Cunge	2,650	0.0004	0.035	Trap.	14	3/1

NOTE: Pews Creek was not surveyed. Length and slope are based on data from "Middletown Township Master Drainage Plan" and other available plans.

F84. **Inflow Hydrographs**. Based on the hydrologic and routing data described above and summarized in Tables F-8 and F-9, inflow hydrographs were developed for each basin for the different hypothetical storms, using the HECIFH program for the Compton Creek interior drainage areas and HEC-1 for the Pews Creek interior drainage areas. Summaries of peak inflows for all storm events analyzed are presented in Table F-10 and F-11.

TABLE F-10
Summary Of Peak Inflows - Compton Creek
(cfs)

Event	C1	C2	C3
2 Yr.	32	35	50
5 Yr.	51	55	82
10 Yr.	61	67	100
50 Yr.	91	99	150
100 Yr.	106	116	176
500 Yr.	138	150	229

TABLE F-11
Summary Of Peak Inflows - Pews Creek
(cfs)

Event	PC1	PC2	PC3	Lower	Pews Creek at Outlet (Total)
2 Yr.	69	174	60	294	466
5 Yr.	136	303	97	443	804
10 Yr.	173	377	118	531	999
50 Yr.	287	584	175	760	1,530
100 Yr.	348	700	206	877	1,828
500 Yr.	473	927	267	1,123	2,418
Sept. 1960	145	369	108	415	954

Interior Drainage Hydraulics.

F85. General. In addition to the development of hydrologic data, the analysis of interior drainage facilities requires additional input describing the physical and operational characteristics of the various alternatives. Input requirements consist of potential storage volumes and pumping rates. The HEC-IFH software package was utilized to evaluate the effectiveness of proposed pump stations by routing interior fluvial flood events through the available storage at the line of protection for various proposed alternatives. The criteria used to provide input to make these evaluations is described below.

F86. Elevation/Storage Relationships. In order to evaluate the storage capacity at the line of protection, elevation-storage relationships were developed. Using the 1" = 200' scale map, and commencing with the lowest elevation at the proposed ponding site and continuing up to elevation 8 ft. NGVD at Compton Creek and 10 feet for Pews Creek, the planimetric area enveloped by a particular elevation was estimated. For consecutive elevations evaluated, HEC-IFH program uses the average end-area method to compute the volume. The program then sums the volumes between elevations to generate an overall elevation-volume relationship for a particular ponding site.

F87. Extensive low-lying areas along Pews Creek yield significant storage capacity, bounded by Keansburg Levee on the west, the drainage divide with Compton Creek drainage basin to the east, the natural divide on the south, and the proposed levee and gate closure structure on the north. The divide between Pews Creek basin and the adjacent Compton Creek lies generally along Wilson Avenue, at approximate elevation 6. Although impounded flood waters can overtop this divide, for the analysis, available storage was assumed to be limited to the Pews Creek side of this divide. Tidal water inflow which occurs prior to closing the proposed storm gate across Pews Creek will reduce the available storage volume. Thus maximum storage capacity is available if the storm gate is closed at low tide.

F88. Table F-12 provides a summary of stage/storage relationships for the project interior drainage areas.

TABLE F-12
Interior Drainage Stage/Storage Data

Elevation (ft)	Area (acre)	Volume (ac./ft.)
Sub-area C-1		
2	0.04	0
4	0.41	0.5
6	1.8	2.6
8	5.6	10.0
10	15.4	30.9
Sub-area C-2		
2	0	0
3	0.1	0.1
4	1.9	1.1
6	3.3	6.2
8	21.7	31.2
Sub-area C-3		
3	0	0
4	0.3	0.2
5	8.3	4.5
6	26.6	21.9
8	53.4	101.9
Sub-area Lower (Pews Creek)		
1	6.4	0
3	141.7	148.1
4	157.4	297.7
6	212.6	667.7
8	270.0	1,150
10	305.6	1,726

F89. Pump Station Configuration. Table F-13 lists the criteria used to develop the configuration of pumping stations alternatives.

TABLE F-13
Pumping Station Criteria

Total Station Capacity	Number of Pumps	Design Capacity per Pump
200 cfs or less	2	½ total
Greater than 200 cfs but less than 450 cfs	3	⅓ total

F90. Pump-on/Pump-off Elevations. Pump-on elevations were determined in similar but slightly different manners for the Pews Creek area and the Compton Creek sub-areas. For Pews Creek, the pump-on elevation for the lead pump was selected based on the tide elevation that dictates the gate closure at Pews Creek, and the first significant damage elevation (Elevation 5.5). For Compton Creek, the pump-on elevation for the lead pump was selected based on the gravity outlet invert and the first significant damage elevation. In either case, it is anticipated that activation of the pump station will be done manually. (It is also anticipated that the Pews Creek closure gate will be activated manually.) Once pumps are activated, however, it is essential for all pumps to be operating before damage to structures within the interior drainage area occurs. The following general guidelines were used to select pump-on elevations:

- Minimum or lead pump-on elevation considered was at ½ foot below the tidal elevation at which the gate at Pews Creek is closed.
- Maximum pump-on elevation providing full station capacity (all pumps running) at least ½ foot below the first significant damage elevation.

F91. After determining the lead pump-on elevation, additional pumps were started at ½-foot intervals. Pump-off elevations are set to one foot below the pump-on elevation.

F92. Pump Cycle Time. Pumps are expected to have a minimum cycle time of six minutes, i.e., 10 starts/hour. To achieve this cycle time, an adequate volume of surface runoff from the interior drainage area must be stored and available whenever the pumping operation is initiated. The storage volume in cubic feet required between the lead pump-on and pump-off elevations is 90 times the pump capacity based on the following equation:

$$V = \frac{(6\ \text{min} \times 60\ \text{sec}) \times QPump}{4}$$

Where V is the volume in cubic feet and $QPump$ is the pump discharge rate in cubic feet per second.

F93. The Stage vs. Storage curves for interior ponds C1 and C2 were adjusted at lower elevations to ensure adequate storage was available to provide stable flood routings. Where adequate storage volumes are not provided by ditches or swales, additional excavation may be required or the cycle time reduced to four minutes, which is the minimum operating cycle time under the most critical flow conditions, per EM 1110-2-3102. Under this condition, the required storage volume could be reduced based on the following relationship:

$$V = \frac{(4\ \text{min} \times 60\ \text{sec}) \times QPump}{4}$$

Storage Volumes below this value could result in excessive pump starts which could result in overheating and damage to pump motors.

F94. Since Pews Creek is an existing waterway surrounded in the Project Area by extensive low-lying areas, the Stage vs. Storage curve provides adequate storage for stable flood routings. No additional excavation or adjustment is required.

ECONOMIC CRITERIA

Conditions

F95. Analysis of benefits and costs for formulation of interior drainage plans was conducted using the FY 1998 interest rate of 7-1/8 % over a 50-year period. Minor cahnges in interest rates will not substantially alter the recommendations.

Costs

F96. **General.** Interior drainage facility costs are based on incremental improvements and are additive to features integral to the line of protection. These costs consist of first construction costs, real estate costs, and annual operation and maintenance expenses. Each of these is described below.

F97. **First Construction Costs.** First construction costs assigned to interior drainage facilities include primary and secondary outlets, intake instructures and gates associated with the outlet, pond excavation, diversion pipes and pump stations. Interior drainage costs do not include major line of protection costs, but rather are limited to project features that may be altered by the interior drainage design. Costs for items were estimated based on information from several sources:

- Means Site Work & Landscape Cost Data
- Rodney Hunt (for sluice and flap gates)
- MCACES cost estimate of Green Brook Flood Control Project (GBFCP)

F98. The costs from GBFCP were updated using Engineering News- Record (ENR) Construction Cost Indices. The base costs from GBFCP included labor, equipment, and material, but not overhead. Thus, an additional 15% for overhead and another 10% for profit were added to the GBFCP base costs (a factor of 1.27).

F99. An additional 20% was added to the cost of gates to cover installation. Overhead (15%) and profit (10%) were also added to the costs provided by Rodney Hunt.

F100. For interior plan comparisons, an additional 15% for contingencies, 7% for engineering/ design, and 7% for supervision / administration are added to the first construction cost.

F101. **Real Estate Costs**. Real estate acquisitions associated with interior drainage facilities are based on the purchase of a permanent drainage easement where interior features are required (drainage ditch, diversion pipe easement, ponds, etc.). In the absence of a complete real estate assessment at this time, base land acquisition costs are assumed to be \$1/ft^3. Real estate acquisition costs include an adjustment to the base value of the land for survey and appraisal (5%) and contingencies (10%).

F102. **Operation and Maintenance**. Annual charges attributed to the operation and maintenance of interior drainage facilities consist of labor charges for the care and cleaning of pond areas, outlets and pump stations, as well as anticipated energy charges and annualized replacement costs. Operation and maintenance costs for interior drainage facilities are summarized for each alternative in subsequent sections of this appendix.

Benefits

F103. **General**. Benefits due to flood damage reduction are summarized for each interior drainage facility listed in subsequent sections. Flood damage reduction benefits for interior drainage facilities are calculated as the difference between minimum facility residual damages and residual damages associated with the interior drainage plan alternative being evaluated.

F104. **Residual Flood Damages**. As described in Appendix B - Benefits, the expected damage to each structure was calculated for various depths of flooding. These damages were then aggregated to determine composite stage vs. damage relationships for each basin. Table F-14 provides a summary of several important flood damage parameters.

TABLE F-14
Summary of Flood Damage Parameters

Sub-Basin	Total Number of Buildings	Total Depreciated Building Value	Elevation of Start of Street & Local Flooding	Elevation of Start of Significant Damage
C1	188	$7,690,020	6.5	7.0
C2	183	$13,410,170	4.7	7.0
C3	73	$17,966,190	4.4	5.0
Pews Creek	371	$34,215,620	4.5	6.0
TOTAL	815	$73,282,000		

F105. **Residual Annual Damages**. Residual damages were calculated using risk based simulation techniques. For this analysis, the stage vs. frequency curves calculated by HEC-IFH were input into HEC-FDA.

F106. This damage analysis assumed that there will be no significant coincidence between the residual interior flooding from rainfall and residual flooding from storms exceeding the line of protection. In accordance with EM 1110-2-1413, interior damage was calculated for a full range of interior flood events up to and including the 500-year storm.

Minimum Facility Plans

F107. General. The determination of interior facilities is being conducted using guidance from EM 1110-2-1413, dated January 15, 1987. The strategy outlined under this guidance follows the premise that interior facilities will be planned and evaluated separately from the line of protection, and should provide adequate drainage at least equal to that of the existing infrastructure. This initial plan represents the minimum interior facilities required to implement the line of protection plan. In order to minimize the environmental impact of these facilities, the outlet pipes discharge to existing drainage ditches where possible. Of the three primary and eleven secondary outlet pipes required for minimum facility in the Compton Creek interior area, only four will not discharge to existing drainage ditches. Pews Creek interior drainage will discharge directly back into the creek.

F108. For the Pews Creek interior area, once the storm gate is closed, there are basically two outlets: the pump discharge pipe and the adjacent gravity outlet, which allow gravity flow when tidal tailwater is below interior water levels.

F109. Table F-15 provides a summary of the minimum facility conditions for both the Compton Creek and Pews Creek interior drainage areas.

TABLE F-15
Summary of Minimum Facility Conditions

Sub-Basin	Construction Cost	Land Cost	Annual Cost	Expected Annual Residual Damages	Frequency of Significant Damage (Years)
C1	$159,400	$5,500	$14,800	$8,800	30
C2	$331,100	$2,600	$31,800	$15,000	>500
C3	$606,600	$16,500	$53,300	$165,500	2
Pews Creek	$168,400	$11,600	$16,300	$319,100	<2

Analysis of Alternate Plans

F110. **General.** The minimum facility plan was the starting point from which additional interior facilities planning commenced. The benefits accrued from alternative plans are attributable to the reduction in the residual flooding and damages which may have remained under the minimum facility condition. For an alternative plan to be justified, it must be implementable and reasonably maximize benefits vs. the additional cost required for its construction, operation and maintenance. Plan alternatives examined include the use of excavated ponds, pump stations and the use of pump stations in conjunction with ponds. Following is a general description of various plan alternatives considered during the development of interior drainage facilities.

F111. **Ponding Plan.** Ponding plans were analyzed as the first set of alternatives added to the minimum facility. The excavation of interior drainage ponds creates additional storage volume available for the containment of surface runoff. The additional storage volume captures and holds runoff from the contributing interior watershed, ultimately draining it to the exterior side of the line of protection when pond outlets are no longer blocked by exterior flooding. The storage volume made available by the excavation can effectively reduce high frequency residual interior flood heights and therefore reduce flood damages along the interior.

F112. The excavated pond alternatives were developed for interior drainage areas by using a procedure designed to create the largest possible pond for each location. The criteria used in developing the ponds consist of the following:

- No pond was to be excavated below elevation 3.5 ft. NGVD to avoid or minimize standing water conditions (The Geotechnical Draft Appendix shows groundwater at elevation 3 feet next to levee)
- Pond side slopes were set at 3:1 to allow easy maintenance and not pose a safety hazard

F113. Potentially viable excavated pond alternatives were evaluated to assess the residual damages as compared to minimum facility conditions to provide a determination of damage reduction benefits. As the existing wetlands areas along Pews Creek already provide extensive storage, no ponding alternatives were evaluated for Pews Creek interior drainage. The proposed pond sites for Compton Creek Sub-areas C1, C2 and C3 are shown in Figures F-2 through F-4.

F114. The costs of each ponding alternative were added to the minimum facility cost to arrive at the total cost of the alternative. The excavation of a pond is reflected as additional capital or first project cost and also results in increased annual operation and maintenance expenses. Capital expenditures affected by the development of a pond include temporary and permanent easements, excavation, disposal, grading and revegetation. The addition of a storage pond also requires additional maintenance costs for such activities as seeding, fertilizing and mowing as well as general cleanup subsequent to a storm event.

F115. Table F-16 summarizes the results of the ponding alternatives analysis.

TABLE F-16

Summary of Ponding Alternatives

	Sub-Basin & Alternative				
	C1 - Pond Alt 1	C2 - Pond Alt 1	C2 -Pond Alt 2	C2 - Pond Alt 3	C3 - Pond Alt 1
Pond Size	1.40 acre	1.30 acre	0.46 acre	1.3 acre & 0.46 acre	1.9 acre
Construction Costs*	$410,310	$513,540	$422,360	$604,800	$830,940
Land Costs**	$78,800	$70,600	$27,700	$95,700	$115,900
Annual Costs***	$40,600	$52,000	$41,400	$61,200	$83,700
Incremental Annual Costs***	$25,800	$20,200	$9,300	$29,400	$30,400
Annual Residual Damage***	$2,630	$6,386	$10,383	$4,292	$129,933
Annual Damage Reduction****	$6,170	$8,595	$4,597	$10,688	$35,610
Net Excess Benefit****	($19,630)	($11,605)	($4,703)	($18,712)	$5,210
Frequency of Street Flooding (yrs)	9	<2	<2	<2	<2
Frequency of Significant Damage (yrs)	98	>500	>500	>500	<2

* Construction Costs include 7% Engineering & Design, 7% Administration, plus 15% contingencies

** Land Costs include 5% Survey & Appraisal plus 10% contingencies

*** Annual Costs and Residual Damages are based on 7-1/8% with 50-year life; Annual Costs include Operation & Maintenance

**** Compared to Minimum Facility conditions

TABLE F-17

Summary of Pumping Alternatives - Compton Creek Interior Areas

	Sub-Basin & Alternative						
	C1 - Pump Alt 1	C1 - Pump Alt 2	C1 - Pump Alt 3	C3 - Pump Alt 1	C3 - Pump Alt 2	C3 - Pump Alt 3	C3 - Pump Alt 4
Total Pump Capacity	40 cfs	60 cfs	80 cfs	40 cfs	60 cfs	80 cfs	120 cfs
Construction Costs*	$1,532,010	$1,637,010	$1,742,010	$1,979,210	$2,084,210	$2,189,210	$2,399,210
Land Costs**	$5,500	$5,500	$5,500	$16,500	$16,500	$16,500	$16,500
Annual Costs***	$131,640	$140,510	$149,890	$170,610	$179,860	$189,460	$208,020
Incremental Annual Costs****	$116,840	$125,710	$135,090	$117,310	$126,560	$136,160	$154,720
Annual Residual Damage	$630	$180	$70	$35,579	$20,209	$10,951	$5,355
Annual Damage Reduction****	$8,170	$8,620	$8,730	$129,964	$145,334	$154,591	$160,188
Net Excess Benefit****	($108,670)	($117,090)	($126,360)	$12,654	$18,774	$18,431	$5,468
Frequency of Street Flooding (yrs)	47	210	>500	3	5	14	43
Frequency of Significant Damage (yrs)	200	450	>500	14	31	73	275

* Construction Costs include 7% Engineering & Design, 7% Administration, plus 15% contingencies

** Land Costs include 5% Survey & Appraisal plus 10% contingencies

*** Annual Costs and Residual Damages are based on 7-1/8% with 50-year life; Annual Costs include Operation & Maintenance

**** Compared to Minimum Facility conditions

TABLE F-18

Summary of Pumping Alternatives - Pews Creek Interior Areas

	Alternative				
	Pump Alt 1	Pump Alt 2	Pump Alt 3	Pump Alt 4	Pump Alt 5
Total Pump Capacity	60 cfs	100 cfs	120 cfs	150 cfs	180 cfs
Construction Costs*	$1,646,010	$1,856,010	$1,961,010	$2,118,510	$2,276,010
Land Costs**	$11,600	$11,600	$11,600	$11,600	$11,600
Annual Costs***	$151,500	$172,720	$182,650	$200,960	$220,230
Incremental Annual Costs****	$135,200	$156,420	$166,350	$184,660	$203,930
Annual Residual Damage	$108,125	$58,565	$47,750	$40,060	$32,202
Annual Damage Reduction****	$211,020	$260,580	$271,395	$279,085	$286,944
Net Excess Benefit****	$75,821	$104,160	$105,045	$94,425	$83,014
Frequency of Street Flooding (yrs)	<1	<1	<1	<1	<1
Frequency of Widespread Damage (yrs)	22	46	59	77	125

* Construction Costs include 7% Engineering & Design, 7% Administration, plus 15% contingencies

** Land Costs include 5% Survey & Appraisal plus 10% contingencies

*** Annual Costs and Residual Damages are based on 7-1/8% with 50-year life; Annual Costs include Operation & Maintenance

**** Compared to Minimum Facility conditions

TABLE F-19
Summary of Ponding/Pumping Alternatives

	Sub-Basin & Alternative
	C1 - Pond/Pump Alt 1
Pond Size	1.4 acre
Total Pump Capacity	40 cfs
Construction Costs*	$1,782,920
Land Costs**	$78,800
Annual Costs***	$157,440
Incremental Annual Costs****	$142,640
Annual Residual Damage	$220
Annual Damage Reduction****	$8,580
Net Excess Benefit****	($134,060)
Frequency of Street Flooding (yrs)	190
Frequency of Significant Damage (yrs)	>500

* Construction Costs include 7% Engineering & Design, 7% Administration, plus 15% contingencies

** Land Costs include 5% Survey & Appraisal plus 10% contingencies

*** Annual Costs and Residual Damages are based on 7-1/8% with 50-year life; Annual Costs include Operation & Maintenance

**** Compared to Minimum Facility conditions

F116. **Pumping Plan**. Pumping plans incorporate the use of pump stations in conjunction with the Minimum Facility features developed for each interior area. Pump stations were considered as a means of reducing residual flood heights within interior ponds through the mechanical displacement of accumulated surface runoff from the interior watershed. To determine how efficiently a pump station reduces flooding, the resultant reduction in residual flood damages is compared to the initial and annual costs of developing and operating the pump station.

F117. As with the ponding alternatives, the costs of pumping alternatives are additive to the minimum facility cost. The construction of a pump station creates additional capital or first project costs and also increases annual maintenance and operation costs. Capital expenditures affected by the addition of pump stations include mechanical equipment and associated housing. In addition, the establishment of pump stations also increases the cost of project operation and maintenance, specifically in the area of power consumption and equipment operation, maintenance and replacement.

F118. Only pump station alternatives were considered for the Pews Creek interior area since sufficient storage is available within the low-lying areas along the creek to preclude the need for additional excavation. Pump capacities from 60 cfs to to 180 cfs were analyzed as alternates to the minimum facility.

F119. Pumping alternatives are summarized for Compton Creek interior areas in Table F-17 and for Pews Creek interior area in Table F-18.

F120. **Ponding/Pumping Plan**. Previously developed ponding and pumping plans were used as a basis to develop ponding/pumping plan alternatives. The ponding/pumping alternatives were analyzed using the storage areas developed for the ponding alternatives and a pump station that would be reasonably cost-effective. Based on an initial assessment of the results of the separate ponding and pumping plans, only one combined alternative was assumed feasible. Performance routing and economic analysis as described previously was performed for this single alternative, in Sub-Area C1. Table F-19 summarizes the results of the analysis.

F121. As noted previously, the large storage available within the Pews Creek interior area precluded the need to evaluate additional excavation for ponding.

Optimum Plan

F122. The optimum Plan is defined as the plan that maximizes the net excess benefits over cost. As outlined within the description of minimum facility, the planning and development of interior drainage facilities is performed independently from the line of protection. Each interior drainage area is analyzed individually to determine the optimum alternative. Within each interior drainage area, the economics for a series of alternate facilities were evaluated and compared to determine which contributes the highest level of net excess benefits to the project. The optimum interior drainage alternatives for each Basin is presented in Table F-20.

TABLE F-20
Optimum Interior Drainage Plan Summary

Basin	Facility	Construction Cost*	Land Cost**	Annual Cost***	Annual Residual Damages**	Level of Protection (Years)****
C1	Min Fac	$159,400	$5,500	$14,800	$8,800	30
C2	Min Fac	$331,100	$2,600	$31,800	$14,980	>500
C3	60 cfs Pump	$2,084,210	$16,500	$179,860	$20,209	31
Pews Creek	120 cfs Pump	$1,961,010	$11,600	$182,650	$47,750	59

* Construction Costs include 7% Engineering & Design, 7% Administration, plus 15% contingencies

** Land Costs include 5% Survey & Appraisal plus 10% contingencies

*** Annual Costs and Residual Damages are based on 7-1/8% with 50-year life; Annual Costs include Operation & Maintenance

**** Level of protection with reference to significant damage due to interior flooding.

Selected Interior Plans

F123. As described above, alternative interior plans were formulated to provide safe and reliable protection from interior flooding. Due consideration was given to evaluating only feasible alternatives, that is alternatives that are implementable and provide equitable protection to properties within the line of protection. Selection of a recommended plan thus focused on economics, that is providing the optimum reduction in damages for the cost of protection.

F124. Using these criteria, minimum facilities were selected for recommendation for Compton Creek Sub-basins C1 and C2, while pumping stations were selected for recommendation for Compton Creek Sub-basin C3 and the Pews Creek Lower Sub-basin. Table F-22 summarizes the selected plan costs, annual damages and levels of protection. "Level of protection" refers to the elevation of first significant damage. Along Pews Creek there is a small area along Plymouth Avenue subject to damage at approximately elevation 5.5 NGVD, which will be subject to frequent flooding. More severe widespread flooding does not occur until elevations exceed 6.0 NGVD, approximately a 59-year event. Figure F-17 presents stage (elevation) - frequency curves for each of the selected interior plans. Typical outlet structures for the Selected Plan are presented in Figure F-18, and a typical pump station is presented in Figure F-19.

With- and Without Project Stage/Frequency Analysis

F125. To analyze the effects of the selected NED plan for Port Monmouth on the existing drainage facilities of the adjacent Keansburg project, a with- and without- project stage/frequency analysis was conducted. The results of this analysis are summarized in Table F-21 below:

Table F-21

Summary of Interior Flood Stages (feet NGVD)

Return Period (years)	Without Project WSE	Without Project WSE + Wave Setup	With-Project WSE			
			C1	C2	C3	Pews
2	6.50	7.10	6.55	5.58	3.79	5.50
5	7.60	8.40	6.74	6.29	4.38	5.50
10	8.40	9.30	6.81	6.47	4.65	5.50
50	10.50	11.50	7.09	6.69	5.15	5.92
100	12.20	13.20	7.25*	6.75*	5.35*	6.26*
500	N/A	N/A	7.60*	6.91*	5.64*	6.84*

F126. Based upon the results of this analysis, there are no adverse impacts of the Port Monmouth project on the existing drainage facilities of the adjacent Keansburg project.

TABLE F-22
Selected Interior Drainage Plan Summary

Basin	Facility	Gravity Outlets Size & Location	Pond Location	Total Pump Station Size	Constr. Cost*	Land Cost**	O&M Cost	Annual Cost***	Annual Residual Damages ***	Significant Damage Elevation (ft NGVD)	Street Flooding Elevation (ft NGVD)	Level of Protection (Years)
C1	Minimum Facility	48" dia. main outlet south of Campbell Ave 18" dia. outlet (1) south of Willow Ave	NA	NA	$159,400	$5,500	$2,700	$14,800	$8,800	7.0	6.5	30
C2	Minimum Facility	48" dia main outlet near Creek Rd 18" dia. outlets (4) near Broadway, at Woodstock Ave, at Creek Rd, and near Creek Rd & Collins Ave 24" dia. outlet (1) near Creek Rd & Collins Ave	NA	NA	$331,100	$2,600	$7,200	$31,800	$14,980	7.0	4.7	>500
C3	60 cfs Pump Station	48" dia. main outlet along Main St north of Renfrew Place 18" dia. outlets (5)	NA	60 cfs	$2,084,210	$16,500	$25,160	$179,860	$20,210	5.0	4.4	31
Pews Creek	120 cfs Pump Station	2 @ 48" dia., integral to pump station, adjacent to Pews Creek storm gate	NA	120 cfs	$1,961,010	$11,600	$37,450	$182,650	$47,750	6.0	4.5	59****
TOTAL					$4,535,720	$36,200	$72,510	$409,110	$91,740			

* Construction Costs include 7% Engineering & Design, 7% Administration, plus 15% contingencies

** Land Costs include 5% Survey & Appraisal plus 10% contingencies

*** Annual Costs and Residual Damages are based on 7-1/8% with 50-year life; Annual Costs include Operation & Maintenance

**** Local flooding at Plymouth Avenue will continue to occur on a more frequent basis. The level of protection at Pews Creek was defined based on the occurrence of widespread flooding.

SUB-APPENDIX F1 - DETAILED FORMULATION PLANS

Compton Creek Sub-Basin C1

F127. **General.** Interior drainage area C1 is located along the left (west) bank of Compton Creek from south of Route 36 area near Chestnut Street to the north between Campbell and Collins Avenues. The area extends west beyond Wilson Avenue to Main Street in the New Street area. The interior drainage area of C1 comprises 47.65 acres of developed urban land, with minimal wetlands. The lowest buildings are located at elevation 7 ft. NGVD while Willow Street starts to flood at about elevation 6.5 ft. NGVD.

Land Use	Area (ac)	% of Area
Residential	24.05	50.47
Open Space	13.83	29.03
Bldg/Rdwy	7.43	15.59
Wetland	2.34	4.91
Total	**47.65**	**100.00**

F128. Review of the maps and field investigation revealed that drainage systems consisting of catch basins, manholes and storm drainage pipes, serve the Port Monmouth interior drainage area. There are three existing storm outfalls within sub-basin C1. The following is a summary of the existing storm drainage outlets.

EXISTING STORM DRAINAGE OUTLETS

Outfall Designation	Outfall Location	Type	Size	Existing Inv. El. (ft.)
CC-11	Warsha Ave.	RCP	15"	5.3
CC-12	Wilson Ave (±250' S. of Campbell Ave.)	RCP	15"	5.3
CC-13	Willow Street	RCP	15"	4.1

F129. In addition to the minimum facility, one ponding alternative, three pumping alternatives and one combined ponding/pumping alternative were analyzed. These are described in subsequent paragraphs.

F130. **Minimum Facility.** Minimum facility for sub-basin C1 has a primary outlet and secondary outlet as noted below. Both the primary and secondary outlets are being provided with a sluice gate and trash rack. The outlets will also be provided with flap gates to prevent tidal surges from entering the protected area. Ditches will be constructed along the landward side of the levee to direct runoff toward either the primary or secondary outlet. The starting pond elevation was set to 3 feet, which agrees with the groundwater elevation.

MINIMUM FACILITY OUTLET STRUCTURES

Outlet	Location	Size
Primary	300' N. of Willow Street	48" RCP
Secondary	100' S. of Willow Street	18" RCP

F131. **Ponding Alternatives.** One ponding alternative was analyzed. The ponding site considered is located between a residential area and the levee, near Willow Street and an existing ditch. The pond alternative evaluated is trapezoidal in shape, 3 feet deep (surrounding area has a maximum elevation of 7 feet), and has a surface area of approximately 1.4 acres. The HECIFH stage/area relationship was revised to include additional storage provided by the pond.

F132. **Pumping Alternatives.** HEC-IFH models were developed using the Minimum facility works in combination with three pump station alternatives. The pump station alternatives were based on a total station capacity of 40, 60 and 80 cfs, each consisting of two pumps of equal capacity. The first pump-on was set at 4.0 feet and the second one at 4.3 feet with pump-off at one foot lower, at 3 and 3.3 feet, respectively.

F133. **Ponding/Pumping Alternative.** The smallest pump station alternative of 40 (2 x 20 cfs) was evaluated in combination with the 1.4-acre pond described above.

Compton Creek Sub-Basin C2

F134. **General.** Interior drainage area C2 is located along the left (west) bank of Compton Creek from sub-basin area C1 extending north just beyond Broadway. A segment of proposed levee and Wilson Avenue form the east and west boundaries of the interior drainage area. The interior drainage area of C2 totals 50.84 acres of predominantly residential development with limited wetlands areas. The lowest buildings are located at elevation 7 ft. NGVD while flooding of Creek Road and Main Street start at 4.7 and 5.7 ft. NGVD, respectively.

Land Use	Area (ac)	% of Area
Residential	48.81	96.00
Wetland	2.03	4.00
Total	**50.84**	**100.00**

F135. There are four existing storm outfalls from sub-basin C2. The following is a summary of the storm drainage outlets.

EXISTING STORM DRAINAGE OUTLETS

Outfall Designation	Outfall Location	Type	Size	Existing Inv. El. (ft.)
CC-7	Main St./Broadway Intersection	RCP	18"	4.2
CC-8	Creek Rd. (±900' E. of Wilson Ave. Int.)	HRCP	19"x30"	3.1
CC-10	Creek Rd. (± 150' S.E. of Collins Ave)	RCP	24"	2.5
CC-150	Wood Stock Ave. near Broad St.	CIP	12"	NA

F136. In addition to the minimum facility, three ponding alternatives, but no pumping or ponding/pumping alternatives were analyzed. These are described in the paragraphs below.

F137. **Minimum Facility.** Minimum facility for sub-basin C2 will have a primary outlet and five secondary outlets as noted below. Both the primary and secondary outlets are being provided with a flap gate, sluice gate, and trash rack. Drainage ditches along the toe of the levee will direct runoff of the protected side of the levee to a nearby outfall. Starting pond elevation was set to 3 feet, which is the same as groundwater elevation.

MINIMUM FACILITY OUTLET STRUCTURES

Outlet	Location	Size
Primary	At the extension of Creek Rd.	48" RCP (Extension exist. CC-8 &CC-150)
Secondary #1	Broadway near Main St. Int.	18" RCP (Extension of exist. CC-7)
Secondary #2	150' S. of Pinehurst Ave.	18" RCP
Secondary #3	200' S. of Creek Road	18" RCP
Secondary #4	Near Creek Rd. Collins Ave Int.	24" RCP (Extension of exist. pipe CC-10)
Secondary #5	100' N. of Collins Ave.	18" RCP

F138. **Ponding Alternatives.** As described below, three ponding alternatives were analyzed.

F139. Pond Alternative 1. The ponding site considered is located near the intersection of Broadway and Main Street. The pond alternative evaluated is triangular in shape, 4 feet deep, and has a surface area of approximately 1.3 acres. The HEC-IFH stage/area relationship was revised to include additional storage provided by the pond. The pond bottom slopes towards the levee, and is set to elevation of 3.5 ft. NGVD at the middle of the pond, next to the levee. The toe ditch from this point slopes towards north and south. The secondary outlets one and two, located at the north and south limits of the proposed pond, are below the pond invert at elevation of 2.5 ft. NGVD in order to discharge the flow from Pond 1 to outside the levee.

F140. Pond Alternative 2. The ponding site is located adjacent to Creek Road and the levee. It is irregular in shape, and has a surface area of approximately 0.5 acre. Similar to Pond Alt. 1, HEC-IFH stage/area relationship was revised to include additional storage. Flow from this pond discharges through the main outlet, located approximately 250 feet to its south.

F141. Pond Alternative 3. Pond alternative 3 consists of the two excavated ponds described under Pond Alt. 1 and Pond Alt. 2 combined.

F142. **Pumping Alternative.** Due to the low resultant interior damages in C2 in the Minimum facility analysis ($15,000 Total equivalent Annual), no pump alternatives were considered.

F143. **Ponding/Pumping Alternative.** As with the pumping alternatives, due to the low interior damages in C2 in the Minimum facility analysis ($15,000 Total equivalent Annual), no ponding/pumping alternatives were considered.

Compton Creek Sub-Basin C3

F144. **General.** The C3 interior drainage area is also located on the left (west) bank of Compton Creek. Main Street and Wilson Avenue form the east and west boundaries of the area with the dune/berm forming the north boundary and C2 (just south of Lydia Place) forming the south boundary. The interior drainage area C3 is comprised of 78.74 acres, the majority of which is residential. The area near Monmouth Avenue is subject to some of the most frequent flooding in the area. Street elevations in this area are as low as 4.4 ft. NGVD. The lowest buildings are located at elevation 5 ft. NGVD.

Land Use	Area (ac)	% of Area
Residential	59.01	74.94
Open Space	19.73	25.06
Total	**78.74**	**100.00**

F145. There are several existing storm drainage systems within sub-basin C3. Currently a significant portion of the area is drained to Pews Creek via a 36-inch diameter pipe which outlets at PC6. Since the pipe drains a very low area, the plan is to redirect drainage to Compton Creek via a 36" diversion pipe. This will increase the zero damage elevation at Pews Creek, reduce the required frequency of storm gate closure, and increase the volume of storm water storage available on Pews Creek. The following is a summary of the storm drainage outlets.

EXISTING STORM DRAINAGE OUTLETS

Outfall Designation	Outfall Location	Type	Size	Existing Inv. El. (ft.)
CC-1	±200' S. of Park Ave., ±400' E. of Park Ave. & Main St. Int.	CIP	15"	3.1
CC-2	Main St. (±250' S. of Park Ave. Int.)	CIP	12"	2.2
CC-6	Main St. & Lydia Pl. Int.	RCP	18"	NA
CC-128	Main St. (±400' S. of Renfrew Pl. Int.)	RCP	15"	2.5
PC-6	Wilson Ave. (±350' N. of Renfrew Pl. Int.)	RCP	36"	NA

F146. In addition to the minimum facility, one ponding alternative and four pumping alternatives, but no ponding/pumping alternatives, were analyzed. These are described in the paragraphs below.

F147. **Minimum Facility.** Minimum facility for sub-basin C3 has a primary outlet and five secondary outlets as noted below. Both the primary and secondary outlets are being provided with a flap gate, sluice gate, and trash rack. The 36-inch storm water diversion pipe will be approximately 750 feet long. Ditches are included on the protected side for the levee to direct runoff to primary or secondary outlets. The starting pond elevation was set at 3 feet, which agrees with the groundwater elevation.

MINIMUM FACILITY OUTLET STRUCTURES

Outlet	Location	Size
Primary	± 300' N of Renfrew St.	2 x 48" RCP
Secondary #1	Lydia Pl.	18" RCP (New extension of exist. CC-6)
Secondary #2	Main St. (±500' S. of Renfrew Pl. Int.)	18" RCP (New extension of exist. CC-128)
Secondary #3	Main St. (±250' S. of Park Ave. Int)	18" RCP (New extension of exist. CC-2)
Secondary #4	400' E. of Main St./Park Ave. Int.	18" RCP (Extend existing CC-1)
Secondary #5	Pt. Monmouth Rd., ±100' E. of Park Ave. & Pt. Monmouth Rd. Int.	18" RCP

F148. **Ponding Alternative.** The ponding site considered is located north of Renfrew Place between Main Street and Brainard Avenue. The pond alternative evaluated is rectangular in shape, 1.5 feet deep on average, and has a surface area of approximately 1.9 acres. The HEC-IFH stage/area relationship was revised to include additional storage provided by the pond. The bottom of pond elevation is limited to 3.5 feet, due to the groundwater level.

F149. **Pumping Alternatives.** HEC-IFH models were developed using the Minimum Facility in combination with four pump station alternatives. Pump stations of 40, 60, 80 and 120 cfs total capacity, each consisting of two pumps of half of the total capacity, were utilized for pump alternatives 1, 2 ,3 and 4, respectively. First pump-on elevation was set at elevation 3.5 ft. NGVD, F150. and the second was set at elevation 3.7 ft. NGVD. Pump-offs were set to one foot below pump-on elevations, to 2.5 and 2.7 ft. NGVD, respectively.

F151. **Ponding/Pumping Alternative.** No ponding/pumping alternatives were considered.

Pews Creek

F152. **General.** The Line of Protection works include the construction of a storm gate across Pews Creek, about 300 feet upstream of the recently completed New Port Monmouth Road bridge.

F153. **Minimum Facility.** Minimum facility consists of two 48-inch diameter pipes through the floodwall located between the storm gate and existing Keansburg Levee just west of Pews Creek. The diversion channel constructed during installation of the storm gate will be utilized as the inlet and outlet channel for the pipes. Each pipe will be equipped with a flap gate. No ditch is provided along the levee toe as it directly abuts the marshes of Pews Creek. Thus, there are no secondary outlets. The minimum facilities are described below:

MINIMUM FACILITY OUTLET STRUCTURES

Outlet	Location	Size
Primary	100' W. of Pews Creek & 300' S. of New Port Monmouth Road bridge	2 x 48" DIP

F154. **Ponding Alternatives.** No ponding alternatives were considered. The extensive low-lying wetlands areas along Pews Creek behind the line of protection offer significant storage capacity without resorting to additional excavation.

F155. **Pumping Alternatives.** Pump stations were considered as a means of displacing accumulated surface runoff from the interior watershed.

F156. Several pump stations, varying in size from 60 cfs to 180 cfs, were evaluated Use of a pump during the historical event of September 1960 (Hurricane Donna) was also evaluated. However, since the latter event did not result in high water surface elevations, only the use of the smallest pump alternative was incorporated in its analysis. Under all alternatives, the first pump is set to start at elevation of 4 ft. NGVD and to stop at elevation of 3 ft. NGVD (one foot lower). Additional pumps are set to start at 4.5 and 5.0 ft. NGVD and to stop at 3.5 feet and 4.0 ft. NGVD, respectively.

F157. Table F-23 summarizes the different pump sizes analyzed. It also includes the interior water surface elevation for closed tidal gates and no pump activity for comparison. Starting elevation for interior water surface for all pumping alternatives was set at 5.5 feet.

TABLE F-23

Summary of Pump Alternatives - Pews Creek

HEC-1 Plan	Pump Size (cfs)	Resultant Interior W.S. Elev. (ft)
100 Yr - NP	None	6.96
100 Yr - P1	2 @ 30 = 60	6.64
100 Yr - P2	2 @ 50 = 100	6.40
100 Yr - P3	2 @ 60 = 120	6.26
100 Yr - P4	2 @ 75 = 150	6.14
100 Yr - P5	2 @ 90 = 180	5.93

F158. **Ponding/Pumping Alternatives.** No ponding/pumping alternatives were considered. The extensive low-lying wetlands areas along Pews Creek behind the line of protection offer significant storage capacity without resorting to additional excavation. Any excavation of this wetland area is likely to be very expensive due to extensive restoration requirements.

F159. **Operation and Maintenance of Flood Control Systems.** Flood control system requirements are described in Paragraphs 257-263 in the main report. These paragraphs reflect the latest operation and maintenance project support features and may be subject to refinement during the Preconstruction, Engineering and Design Phase.

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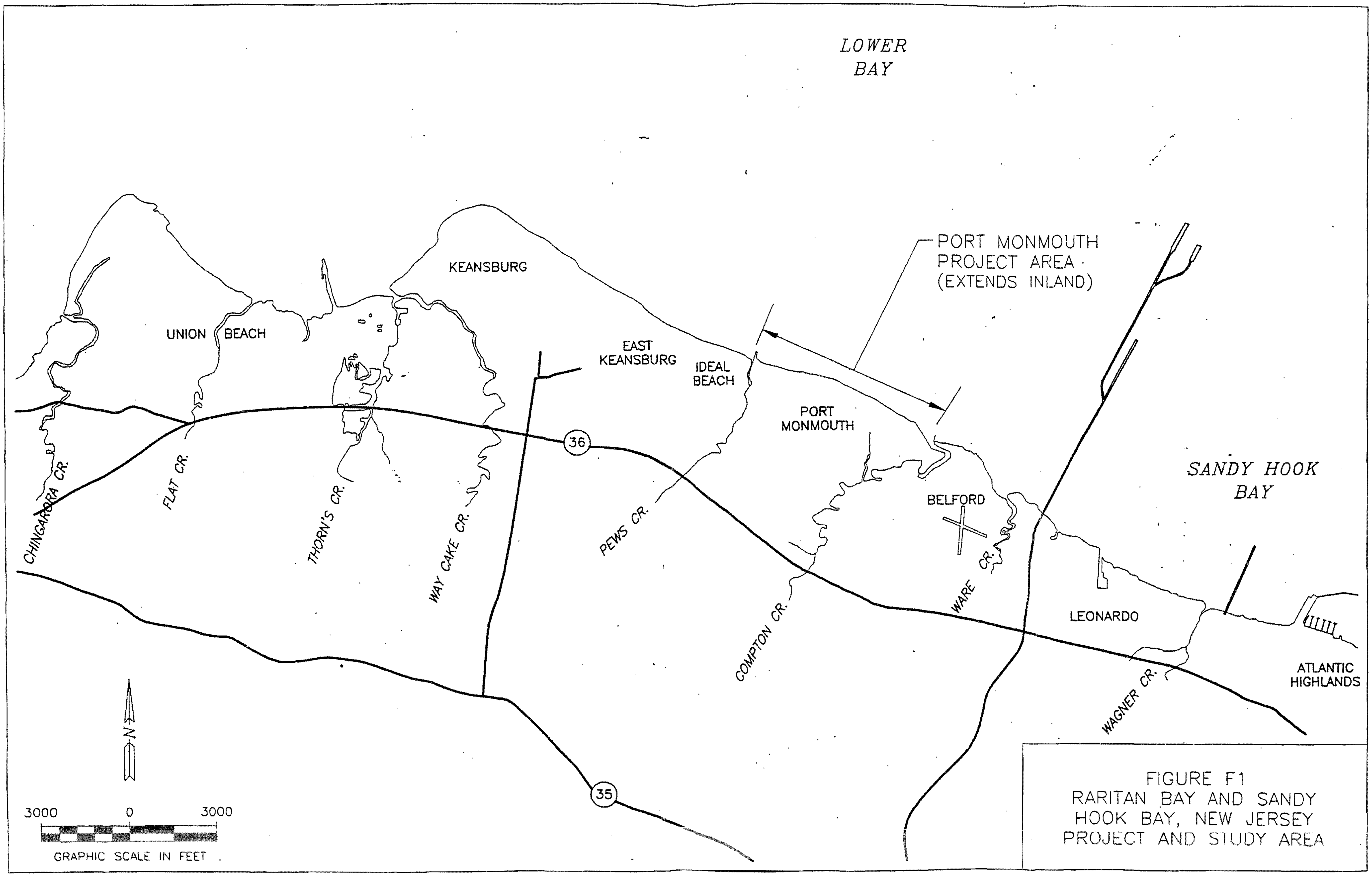
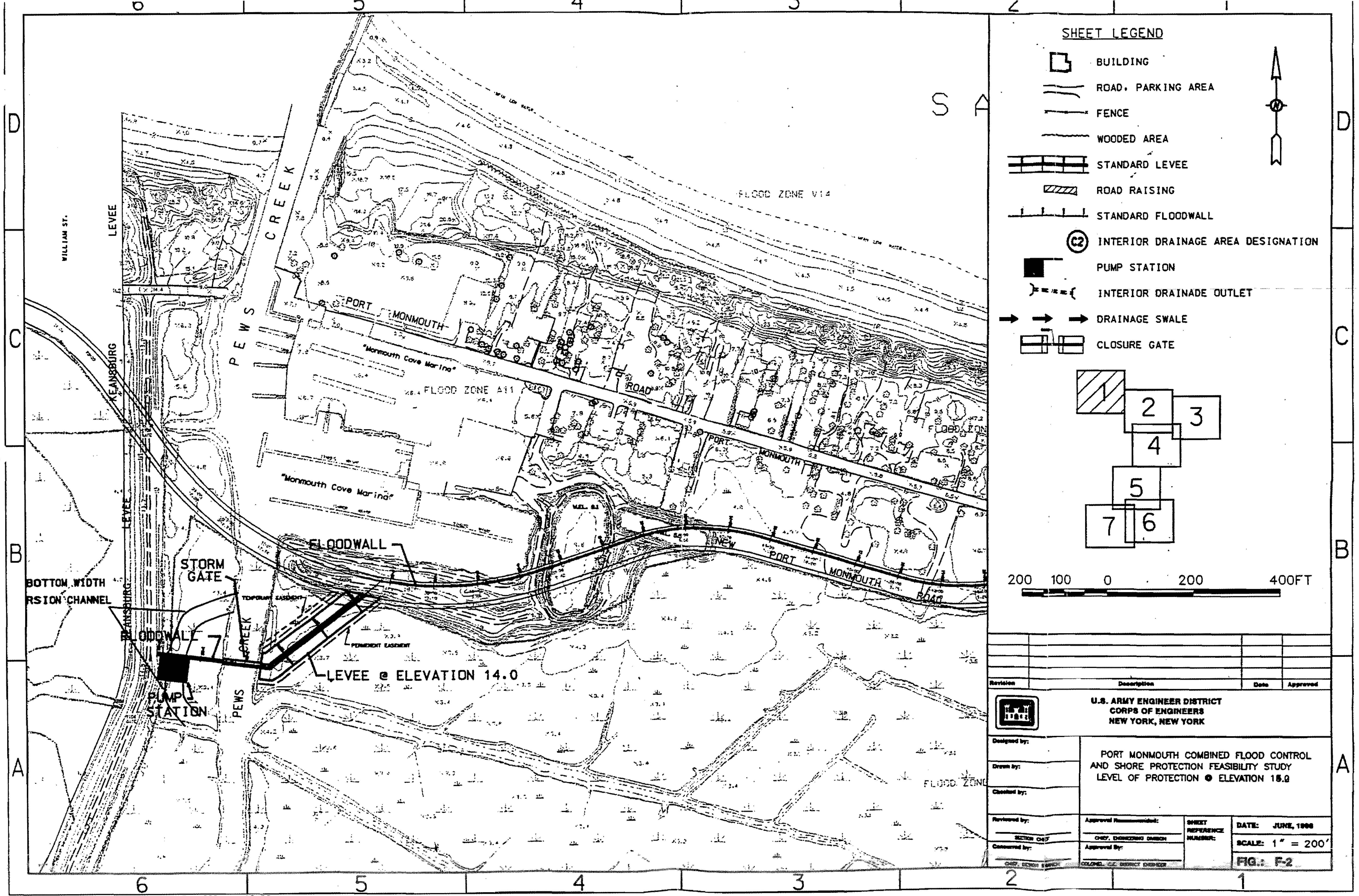


FIGURE F1
RARITAN BAY AND SANDY HOOK BAY, NEW JERSEY
PROJECT AND STUDY AREA

(follows pg F1-11)



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U.S. ARMY ENGINEER DISTRICT
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NEW YORK, NEW YORK

PORT MONMOUTH COMBINED FLOOD CONTROL
AND SHORE PROTECTION FEASIBILITY STUDY
LEVEL OF PROTECTION @ ELEVATION 15.0

Designed by:

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Reviewed by:
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Concurred by:
CHIEF, DESIGN BRANCH

Approval Recommended:
CHIEF, ENGINEERING DIVISION

Approval By:
COLONEL, C.E. DISTRICT ENGINEER

SHEET REFERENCE NUMBER:

DATE: JUNE, 1998

SCALE: 1" = 200'

FIG.: F-2

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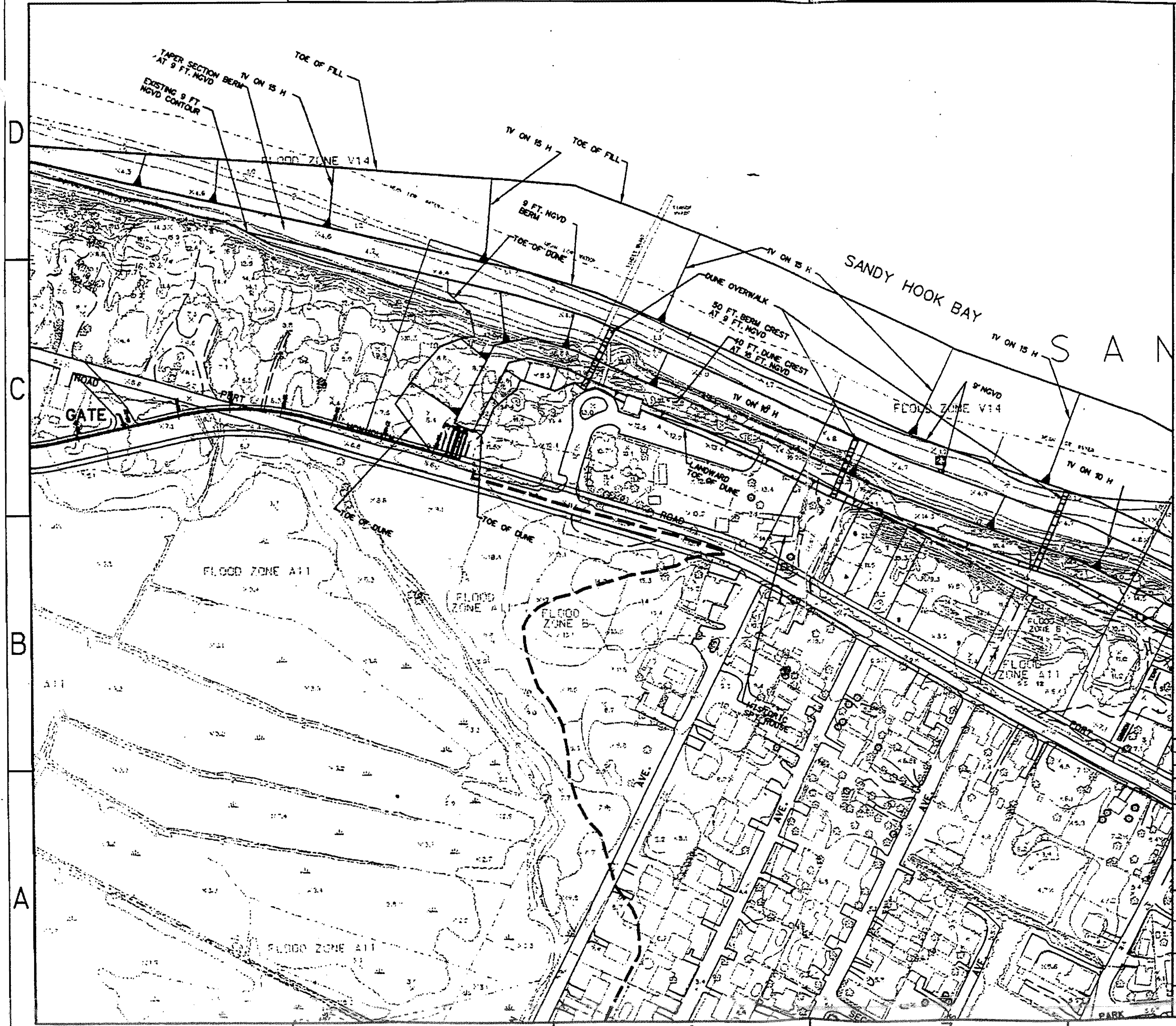
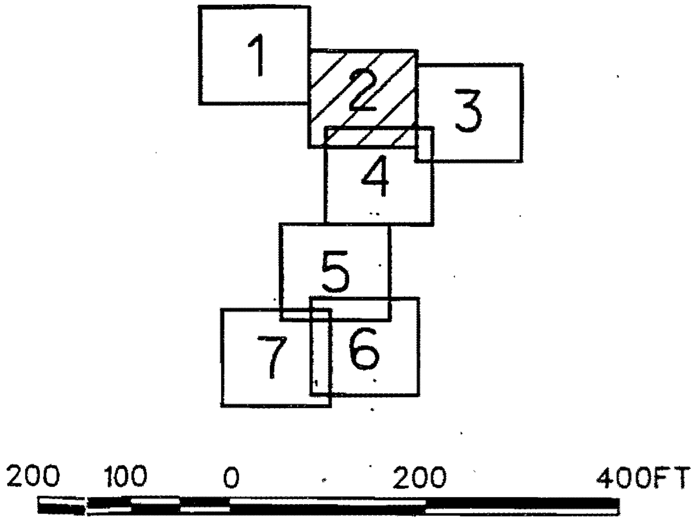


SHEET LEGEND

- BUILDING
- ROAD, PARKING AREA
- FENCE
- WOODED AREA
- STANDARD LEVEE
- ROAD RAISING
- STANDARD FLOODWALL
- Ⓒ2 INTERIOR DRAINAGE AREA DESIGNATION
- PUMP STATION
- INTERIOR DRAINADE OUTLET
- DRAINAGE SWALE
- CLOSURE GATE



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PORT MONMOUTH COMBINED FLOOD CONTROL
AND SHORE PROTECTION FEASIBILITY STUDY
LEVEL OF PROTECTION @ ELEVATION 14.0

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SCALE: 1" = 200'

FIG.: F-3

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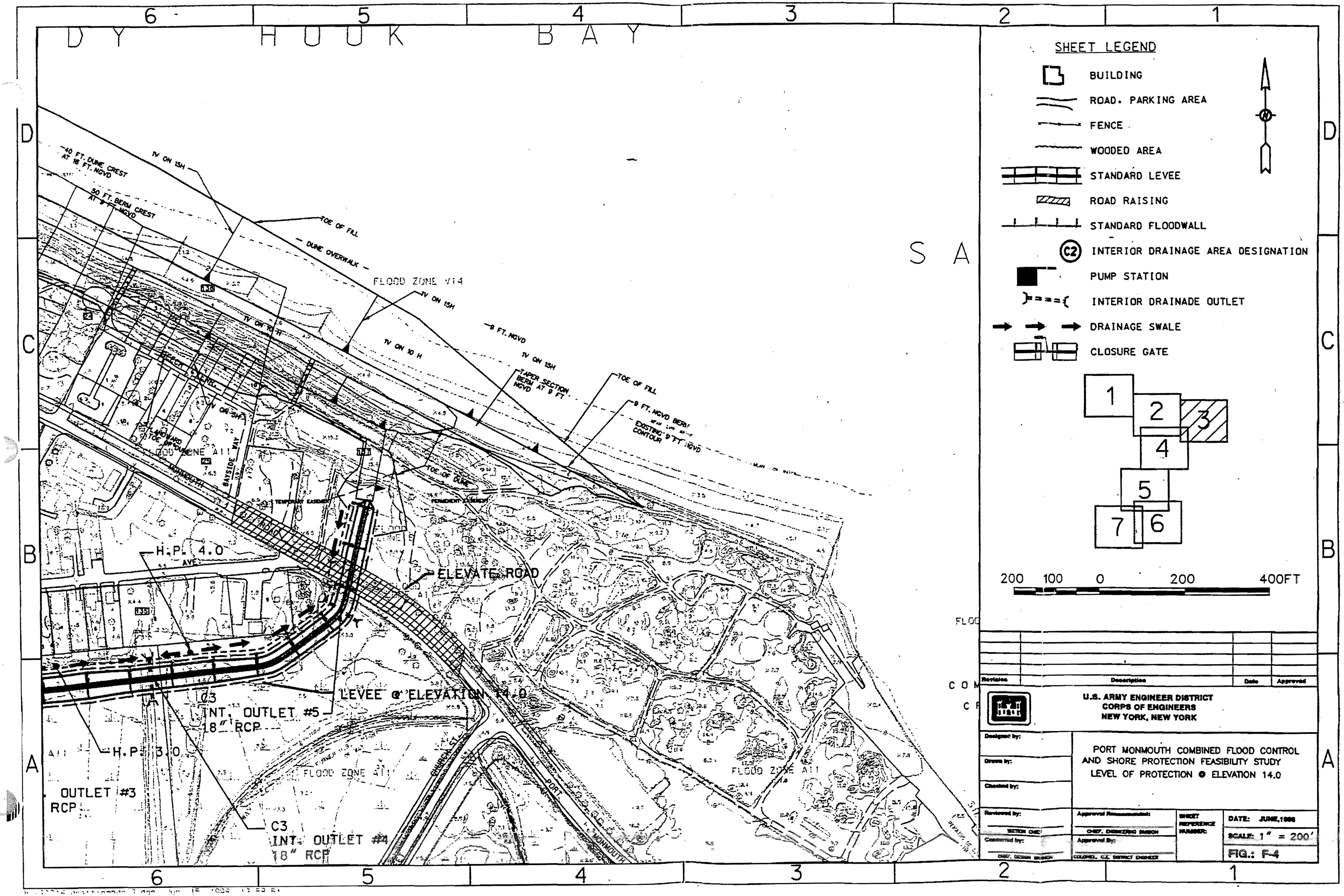

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PORT MONMOUTH COMBINED FLOOD CONTROL
AND SHORE PROTECTION FEASIBILITY STUDY
LEVEL OF PROTECTION @ ELEVATION 14.0

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Approved By:
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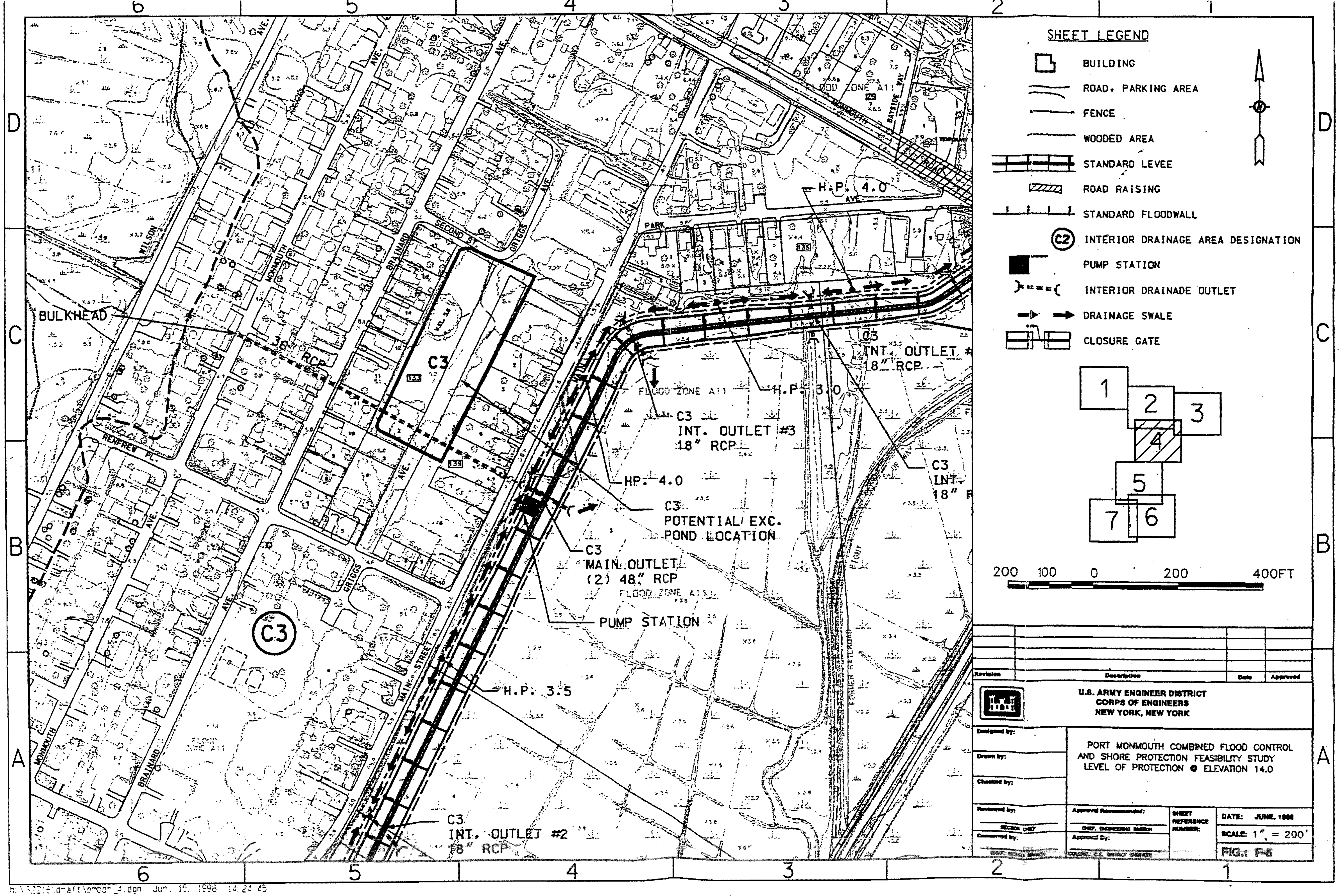
SHEET REFERENCE NUMBER:

DATE: JUNE,1998

SCALE: 1" = 200'

FIG.: F-4



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PORT MONMOUTH COMBINED FLOOD CONTROL
AND SHORE PROTECTION FEASIBILITY STUDY
LEVEL OF PROTECTION @ ELEVATION 14.0

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DATE: JUNE, 1996

SCALE: 1" = 200'

FIG.: F-5

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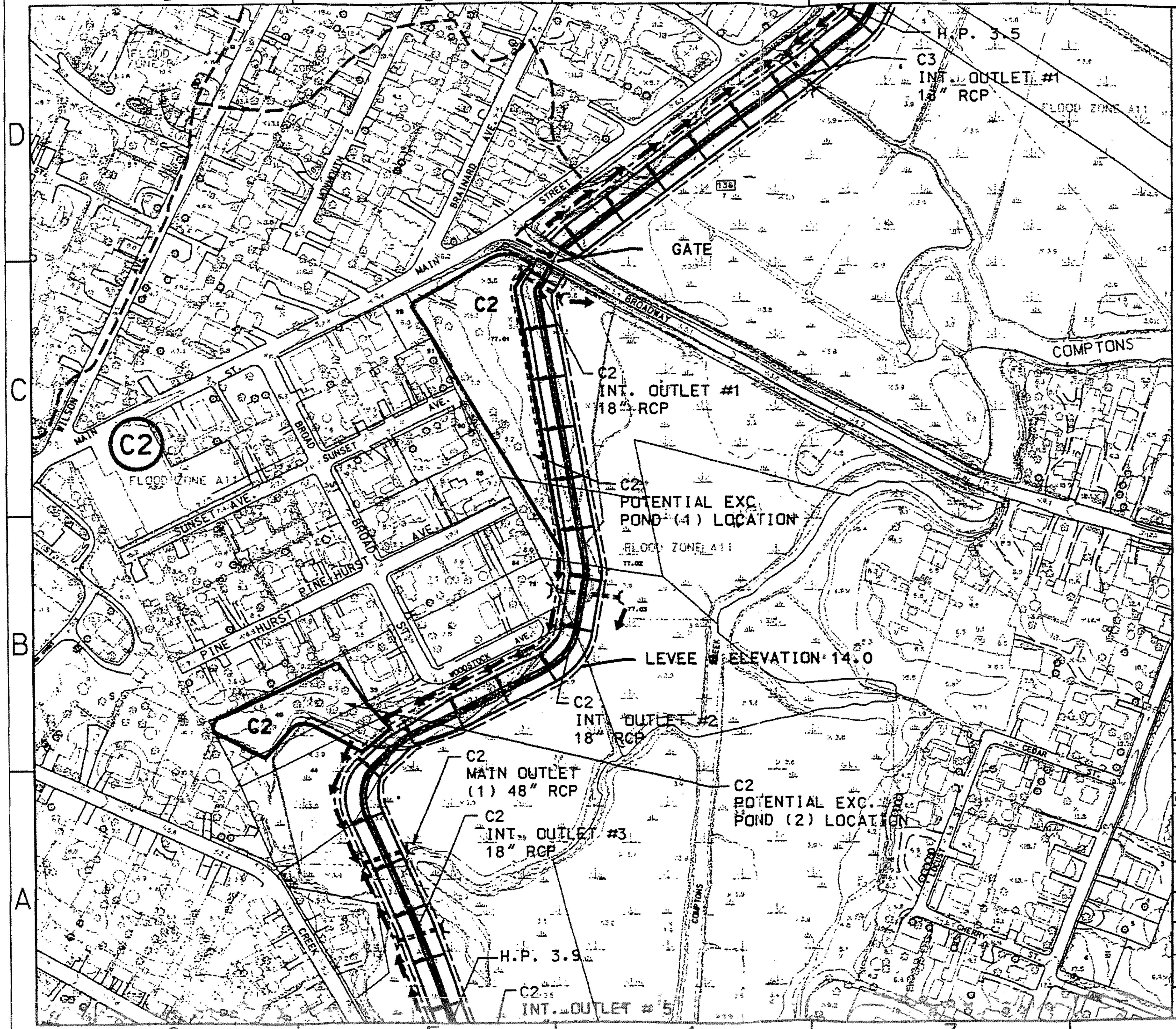
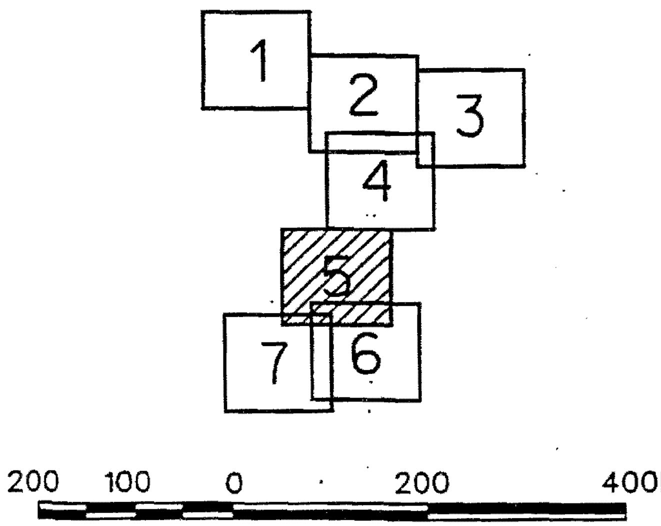


SHEET LEGEND

- BUILDING
- ROAD, PARKING AREA
- FENCE
- WOODED AREA
- STANDARD LEVEE
- ROAD RAISING
- STANDARD FLOODWALL
- Ⓒ2 INTERIOR DRAINAGE AREA DESIGNATION
- ■ PUMP STATION
- }- - - -{ INTERIOR DRAINADE OUTLET
- → → → DRAINAGE SWALE
- CLOSURE GATE



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PORT MONMOUTH COMBINED FLOOD CONTROL
AND SHORE PROTECTION FEASIBILITY STUDY
level of protection @ elevation 14.0

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Approval By:
COLONEL, C.E. DISTRICT ENGINEER

SHEET REFERENCE NUMBER:

DATE: JUNE, 1996

SCALE: 1" = 200'

FIG.: F-6

1 FIG.:F-6

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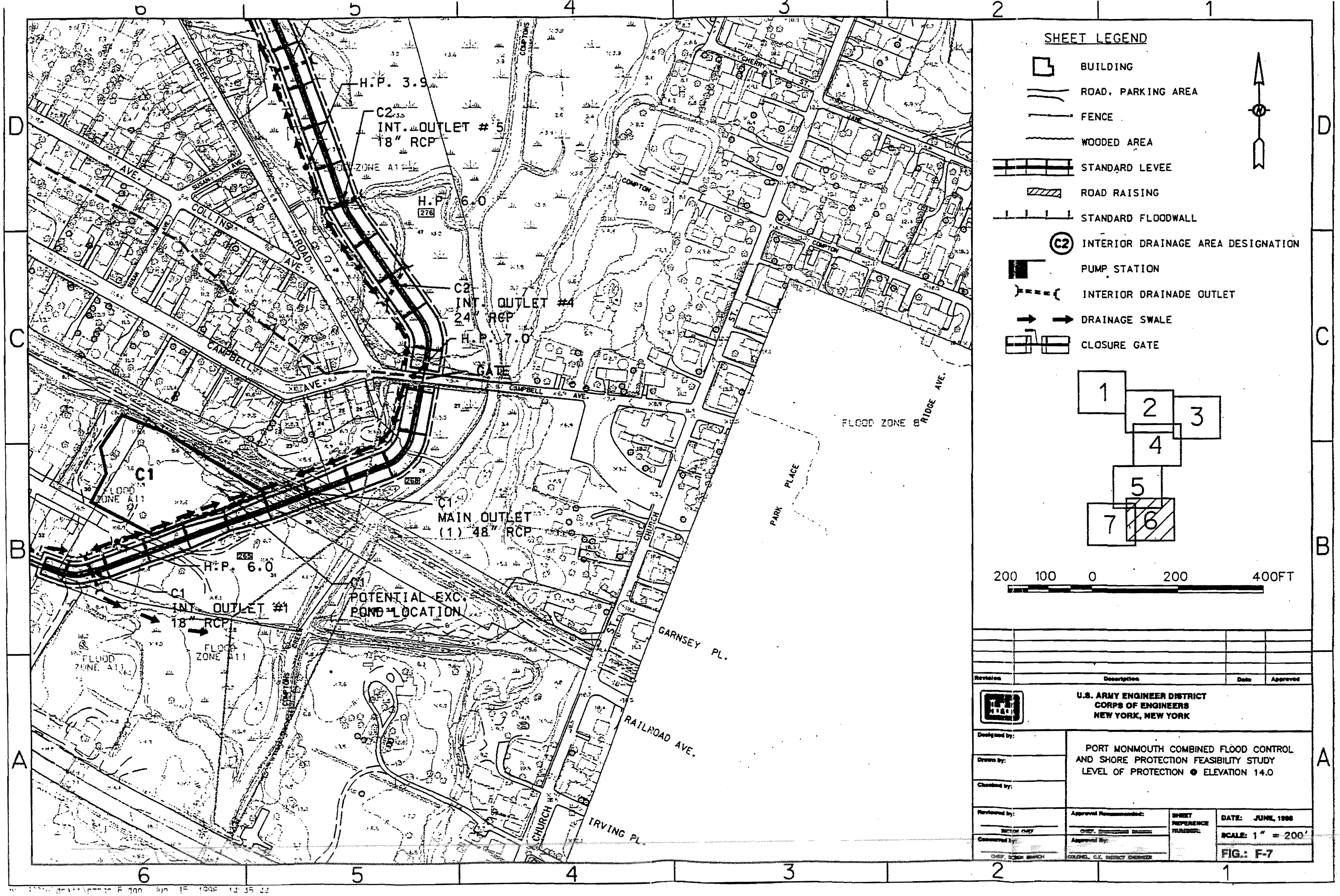

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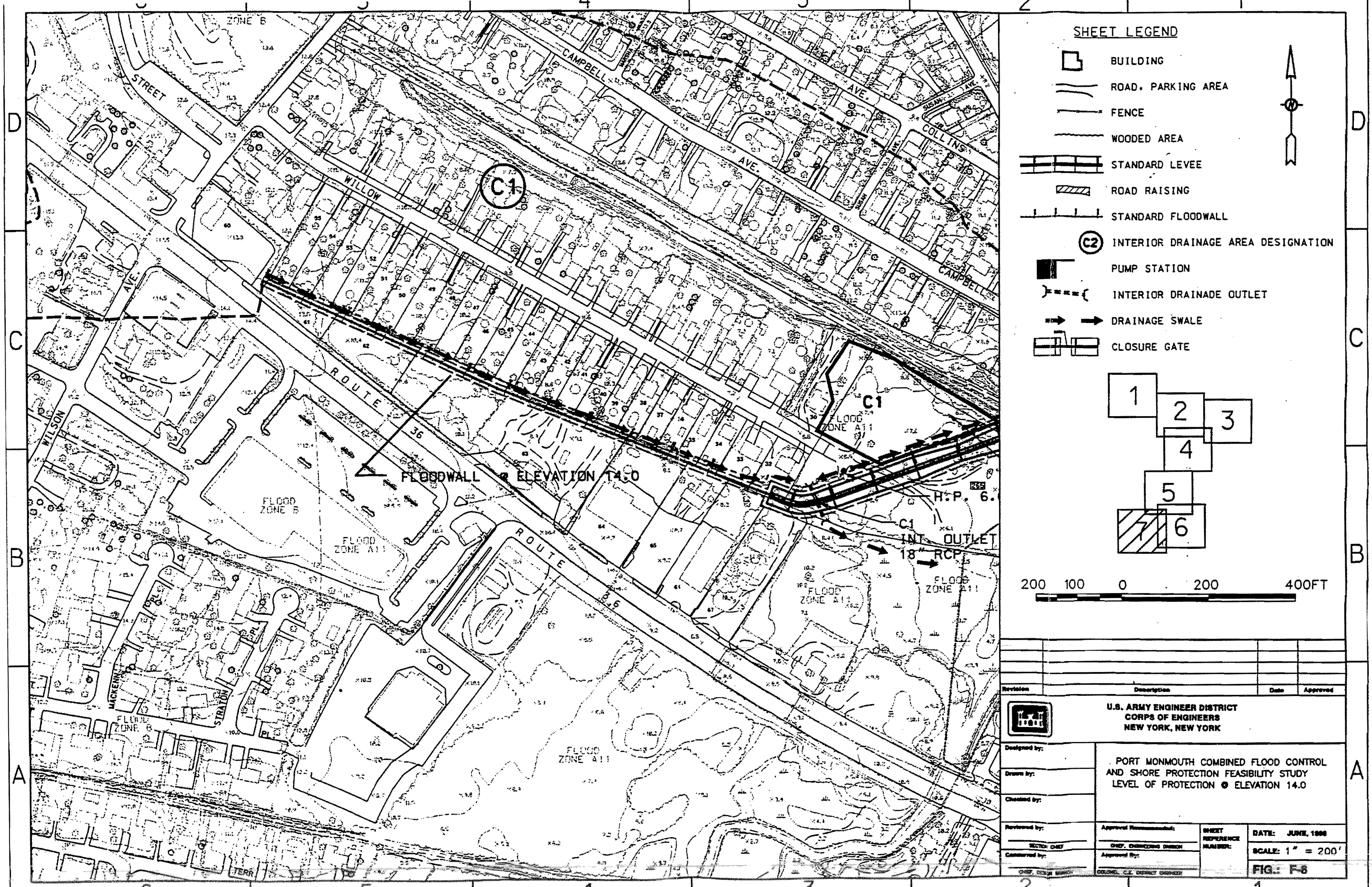
PORT MONMOUTH COMBINED FLOOD CONTROL
AND SHORE PROTECTION FEASIBILITY STUDY
LEVEL OF PROTECTION @ ELEVATION 14.0

DATE: JUNE, 1998

SCALE: 1" = 200'

FIG.: F-7



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AND SHORE PROTECTION FEASIBILITY STUDY
LEVEL OF PROTECTION @ ELEVATION 14.0

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DATE: JUNE, 1998

SCALE: 1" = 200'

FIG.: F-8

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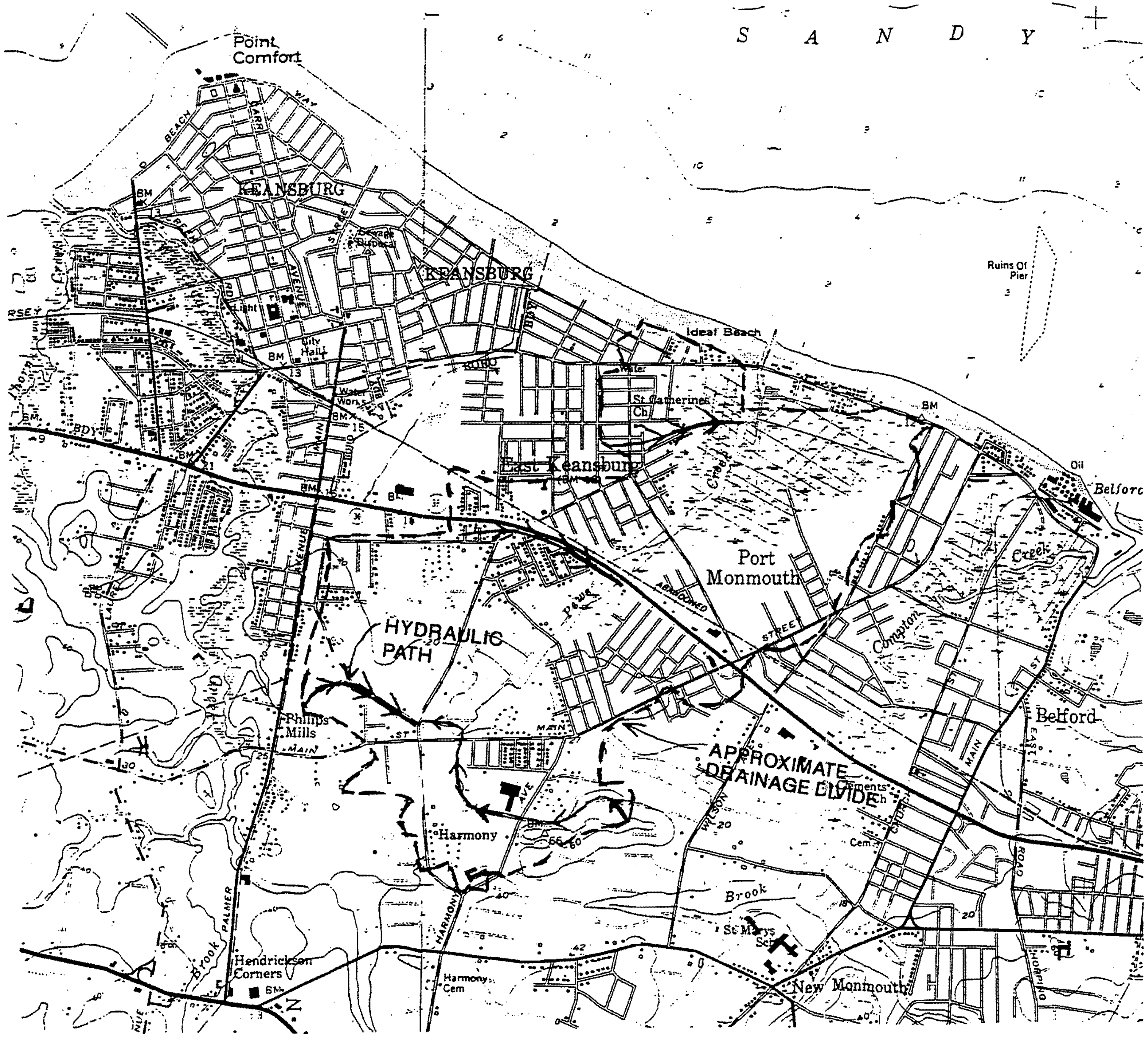
Figures F-9 through F-13 inclusive are in the body of the text



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PORT MONMOUTH COMBINED FLOOD CONTROL
AND SHORE PROTECTION FEASIBILITY STUDY

PEWS CREEK BASIN AREA
(WITH HYDRAULIC PATH)

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Concurred by:
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Approval Recommended:
CHIEF, ENGINEERING DIVISION

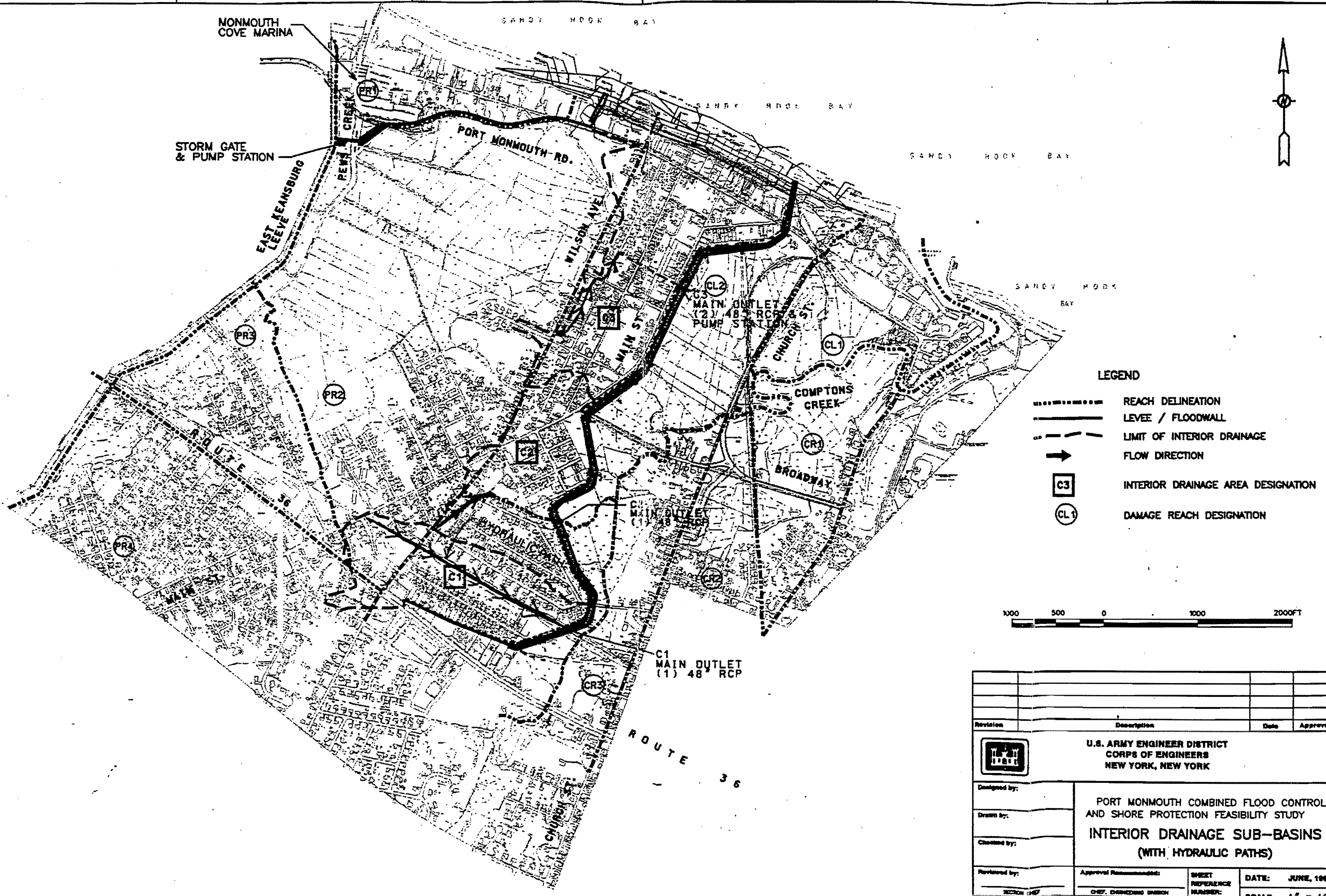
Approval By:
COLONEL, C.E. DISTRICT ENGINEER

SHEET REFERENCE NUMBER:

DATE: JUNE, 1998

SCALE: 1" = 2000'

FIG.: F-14A



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PORT MONMOUTH COMBINED FLOOD CONTROL
AND SHORE PROTECTION FEASIBILITY STUDY

INTERIOR DRAINAGE SUB-BASINS
(WITH HYDRAULIC PATHS)

Approval Recommended:
CHIEF, ENGINEERING DIVISION

Approved By:
COLONEL, C.E. DISTRICT ENGINEER

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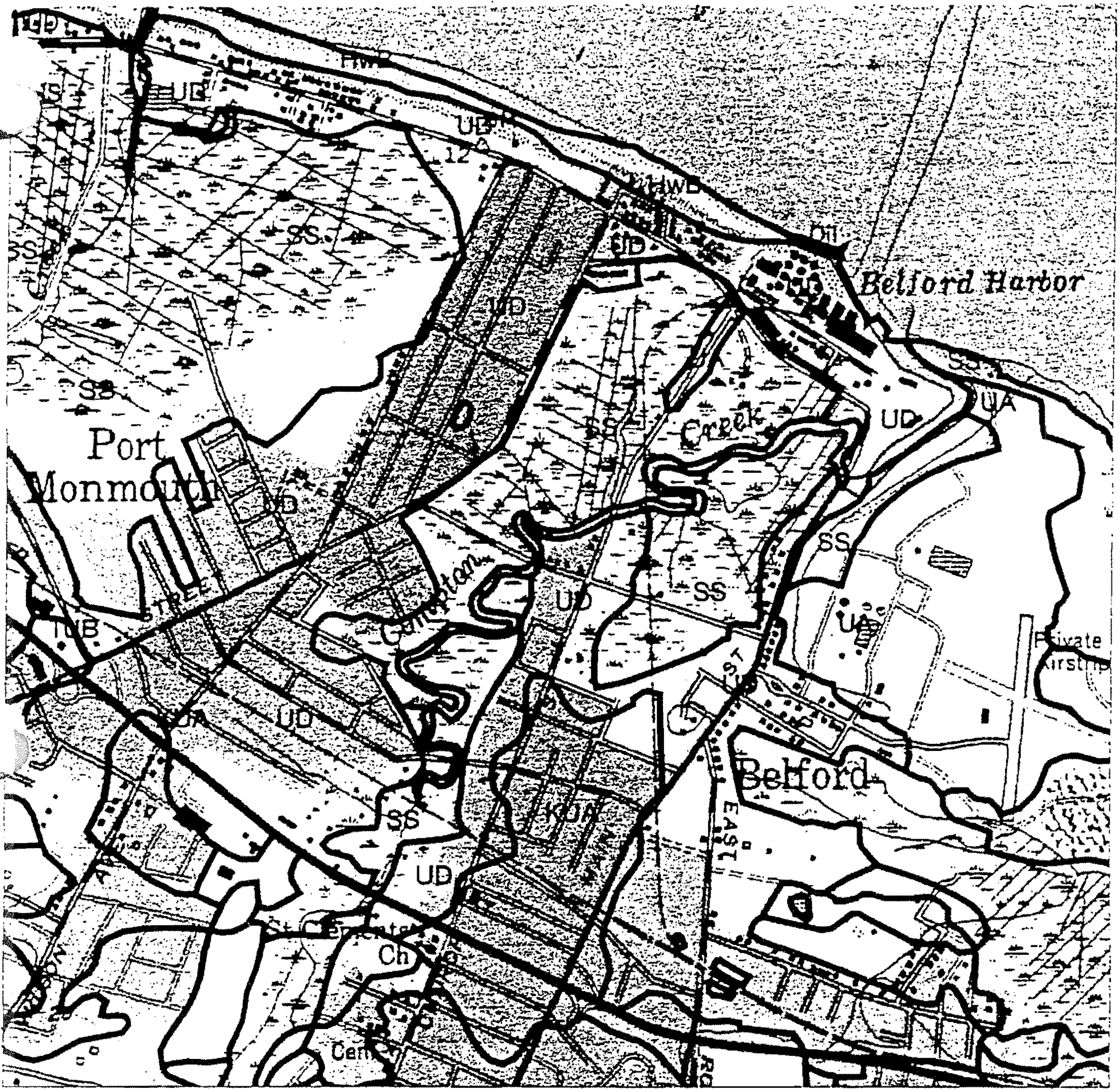
DATE: JUNE, 1998

SCALE: 1" = 1000'

FIG.: F-14B



LEGEND:
SS: Sulfaquents and Sulfihemists
UA: Udorthents
UD: Udorthents-Urban Land Complex
KUA: Klej Loamy Sand

N

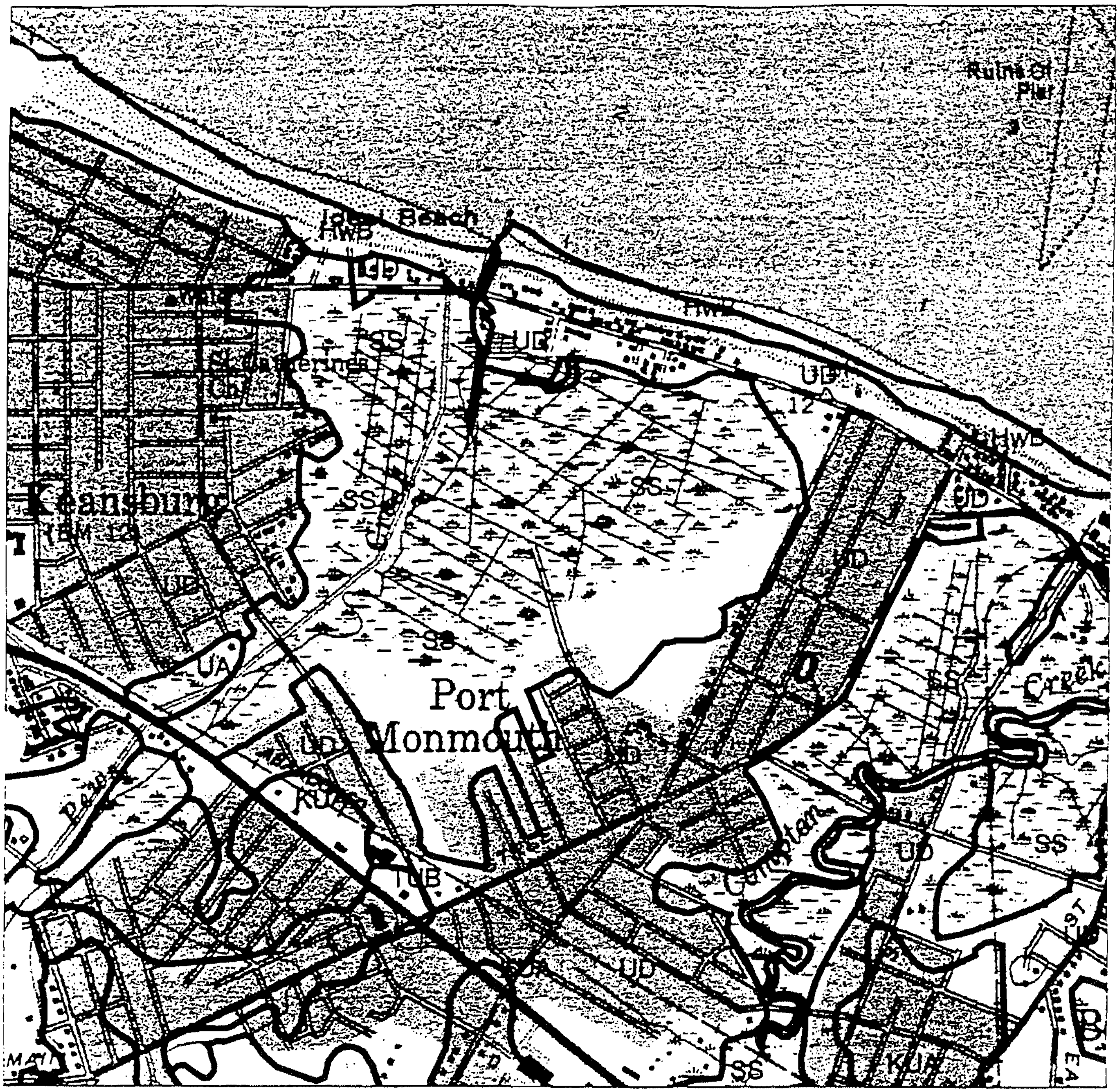
SOURCE:
Soil Survey of Monmouth County, New Jersey
Sheet Numbers 2 and 3

SCALE:
1" = 1,250'

FIGURE F-15A

PROJECT AREA SOILS (COMPTON CREEK)

Port Monmouth Combined Flood Control And Shore Protection Feasibility Study



LEGEND:
SS: Sulfaquents and Sulfihemists
UA: Udorthents
UD: Udorthents-Urban Land Complex
KUA: Klej Loamy Sand
TUB: Tinton Loamy Sand

SOURCE:
Soil Survey of Monmouth County, New Jersey
Sheet Numbers 2 and 3

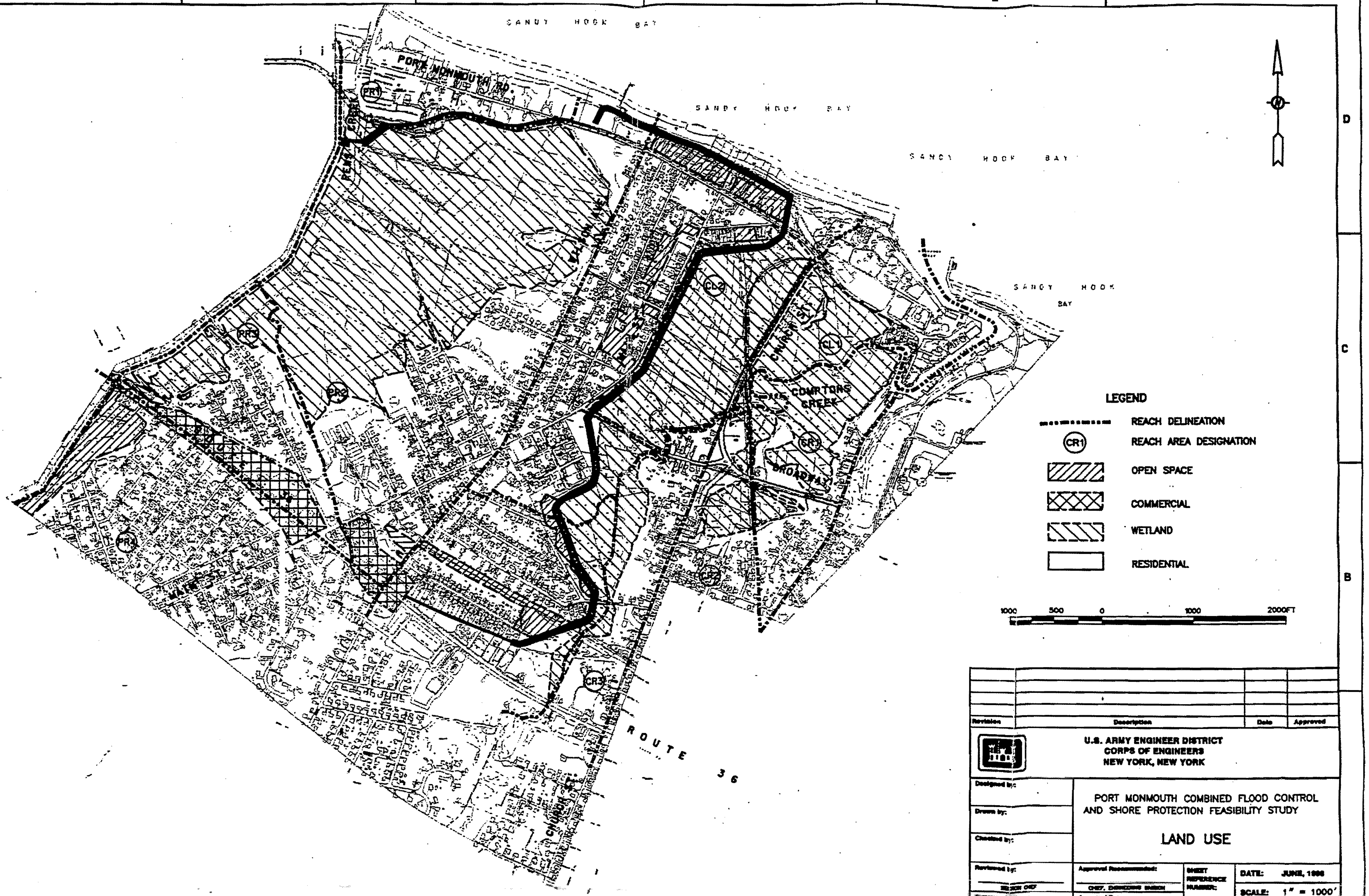
CALE:
1" = 1,250'

N

FIGURE F-15B

PROJECT AREA SOILS (PEWS CREEK)

Port Monmouth Combined Flood Control And Shore Protection Feasibility Study



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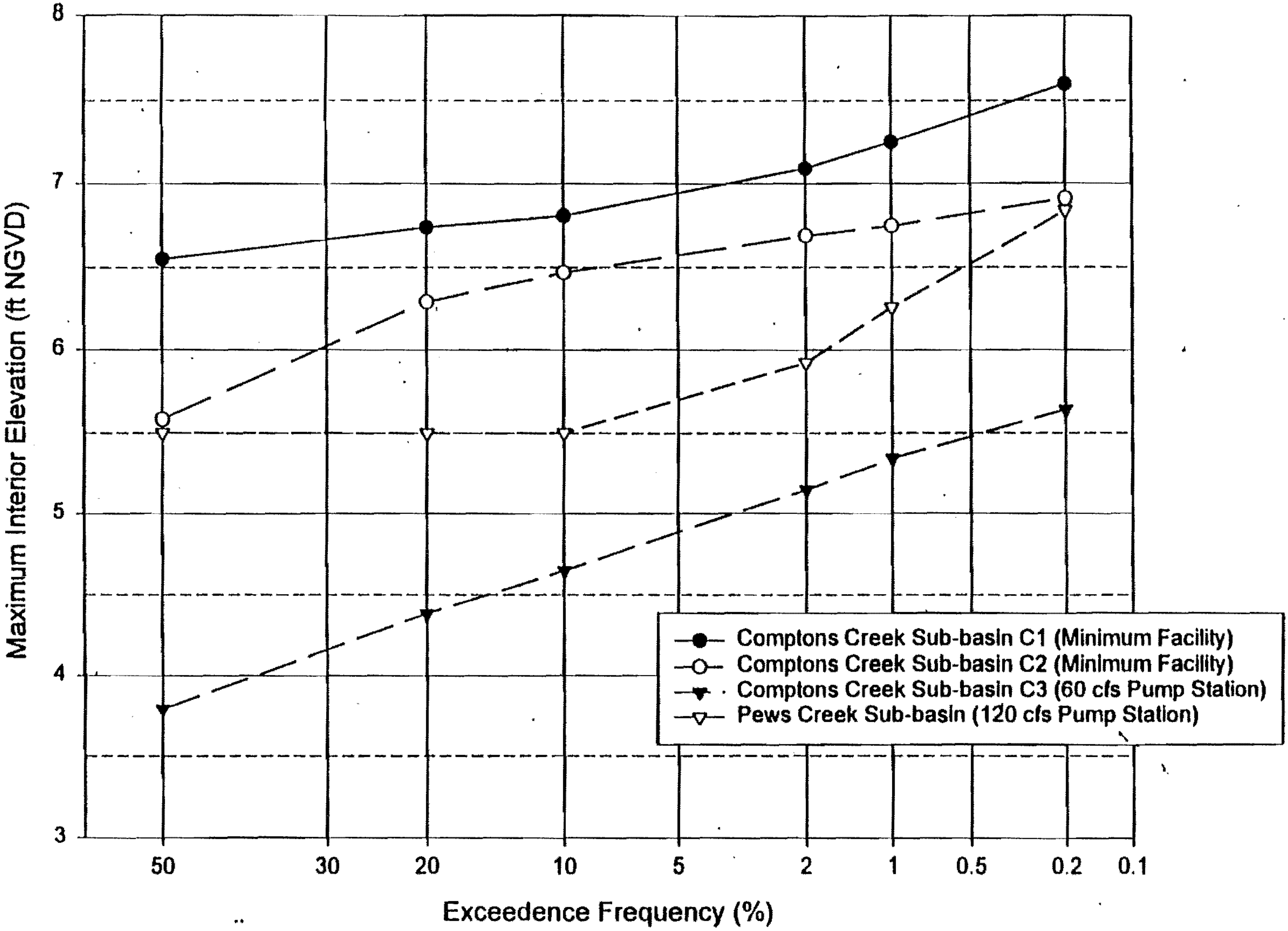
PORT MONMOUTH COMBINED FLOOD CONTROL
AND SHORE PROTECTION FEASIBILITY STUDY

LAND USE

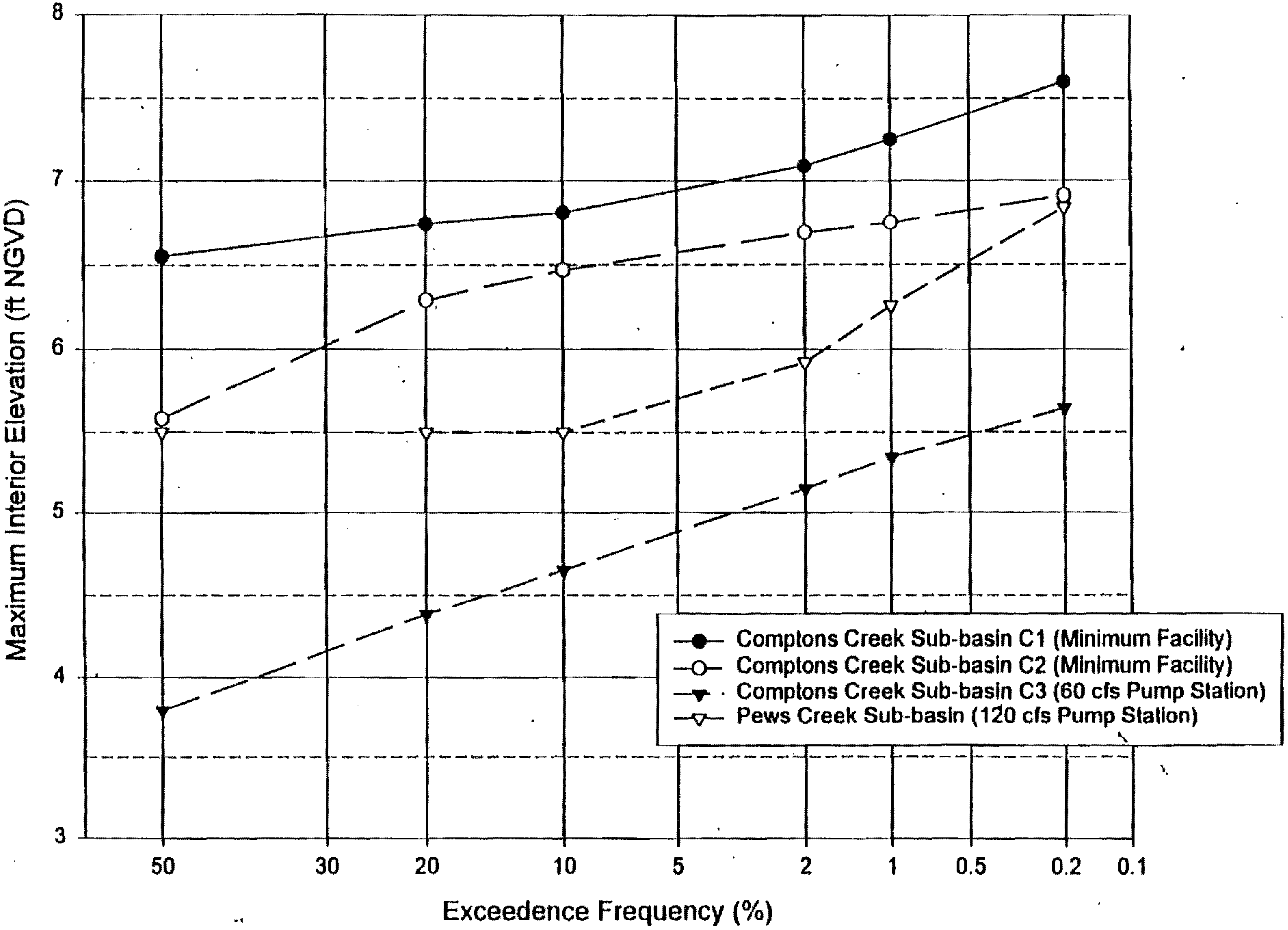
DATE: JUNE, 1998
SCALE: 1" = 1000'
FIG.: F-16

1 FIG; F-16



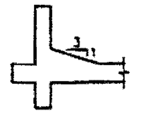
Port Monmouth Combined Flood Control and Shore Protection Feasibility Study

FIGURE F-17
Interior Elevation - Frequency Curves
for Selected Interior Drainage Plans

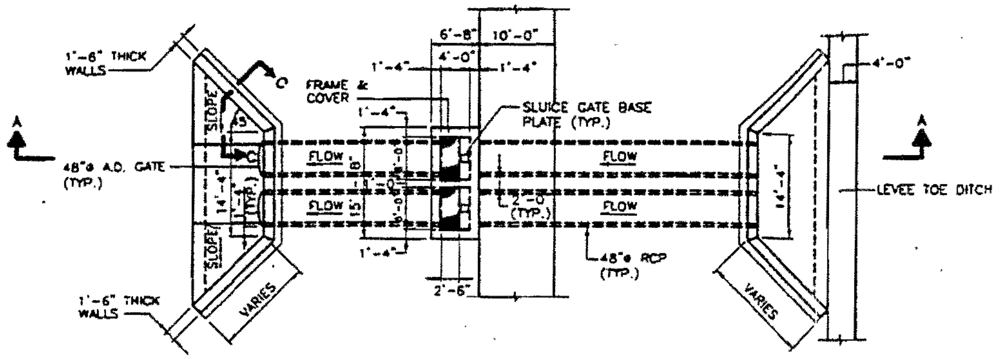


Port Monmouth Combined Flood Control and Shore Protection Feasibility Study

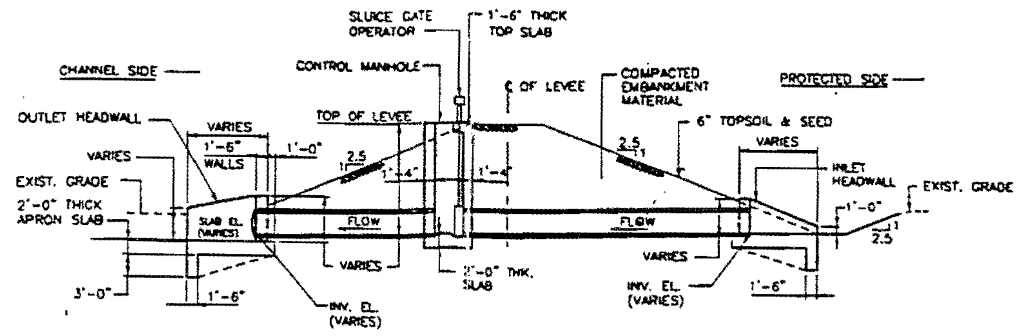
FIGURE F-17
Interior Elevation - Frequency Curves for Selected Interior Drainage Plans



SECTION C–C



PLAN TYPICAL 2 PIPE OUTLET



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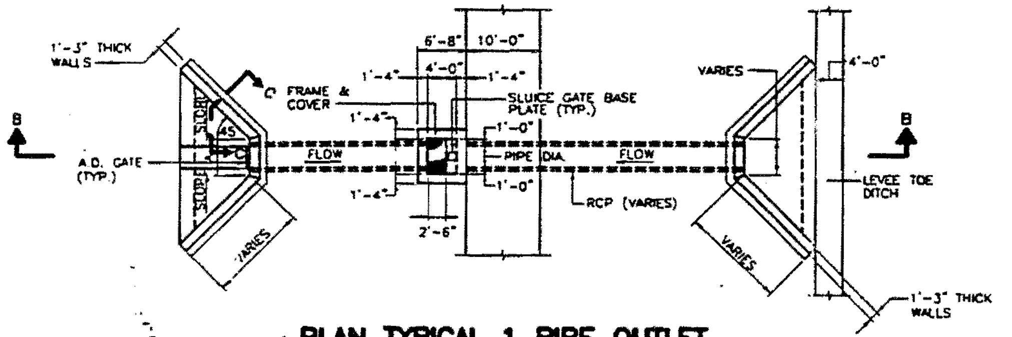
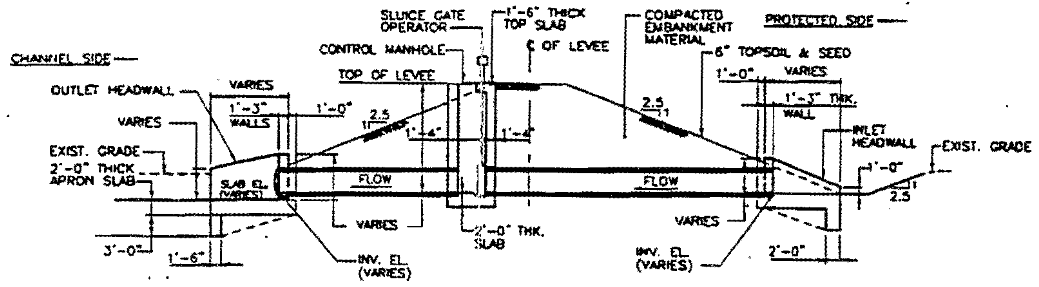


PLAN TYPICAL 1 PIPE OUTLET



SECTION B–B

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PORT MONMOUTH COMBINED FLOOD CONTROL
AND SHORE PROTECTION FEASIBILITY STUDY

TYPICAL MAIN OUTLET DETAILS
INTERIOR DRAINAGE

Approval Recommended:
CHIEF, ENGINEERING DIVISION

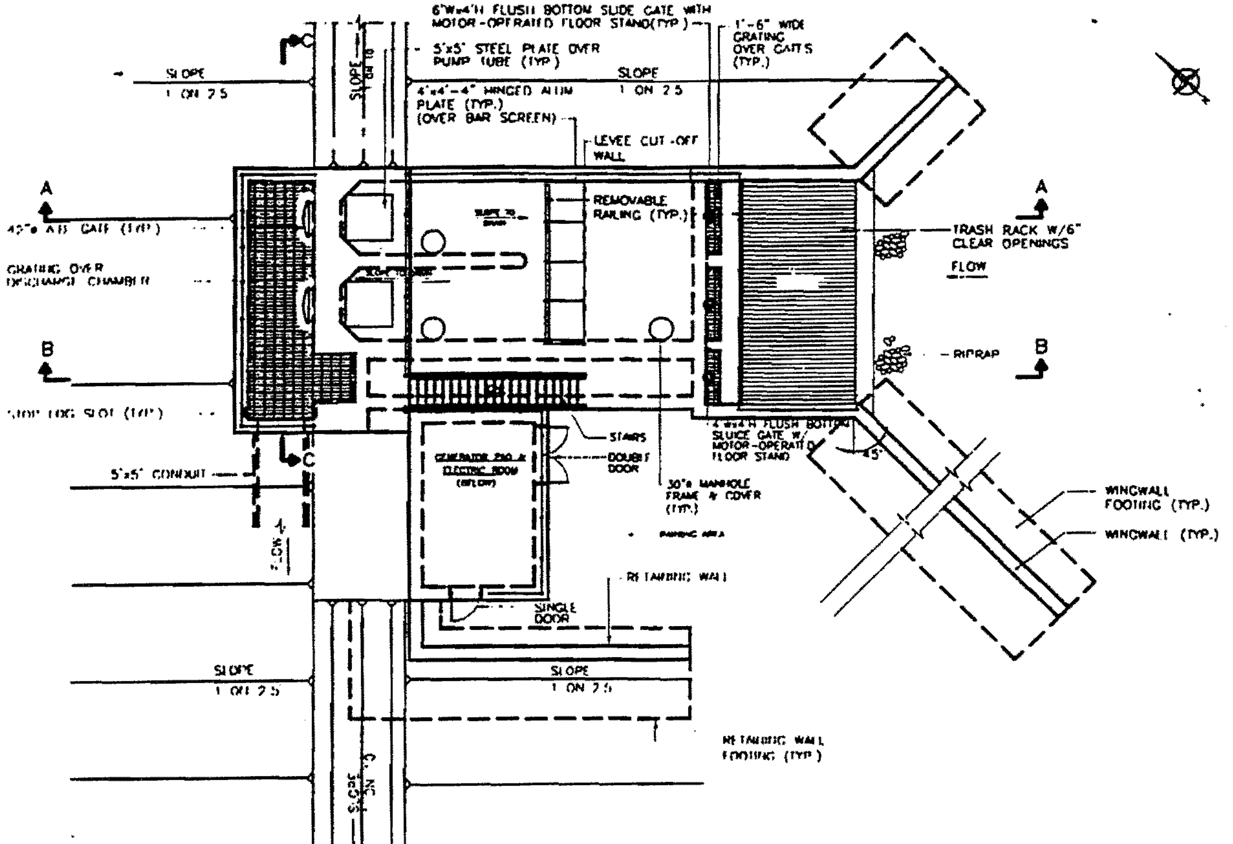
Approval By:
COLONEL, C.E. DISTRICT ENGINEER

SHEET REFERENCE NUMBER:

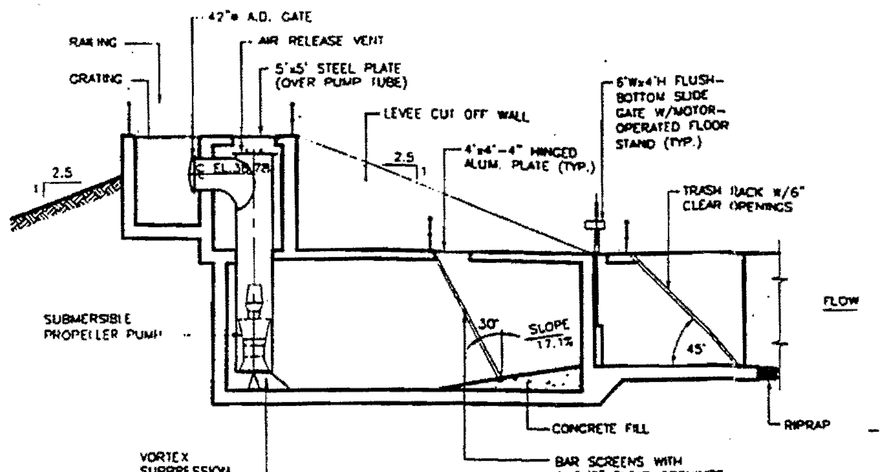
DATE: JUNE 1998

SCALE: AS SHOWN

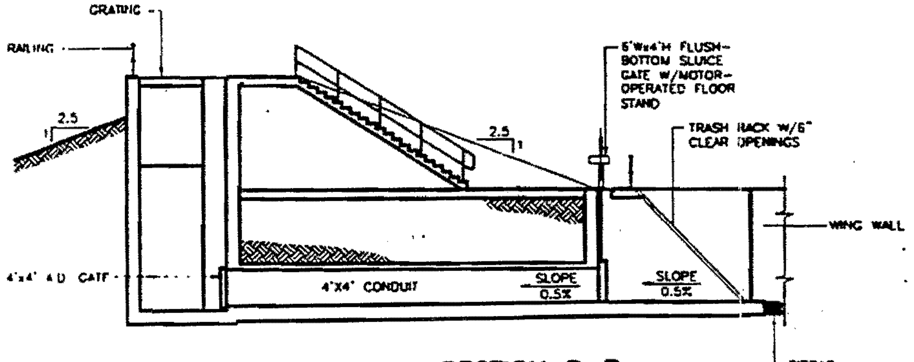
FIG.: F-18



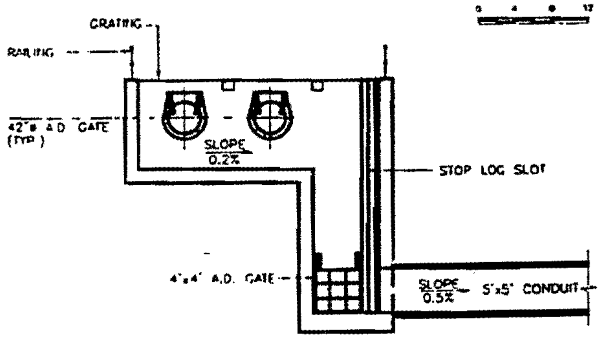
UPPER PLAN



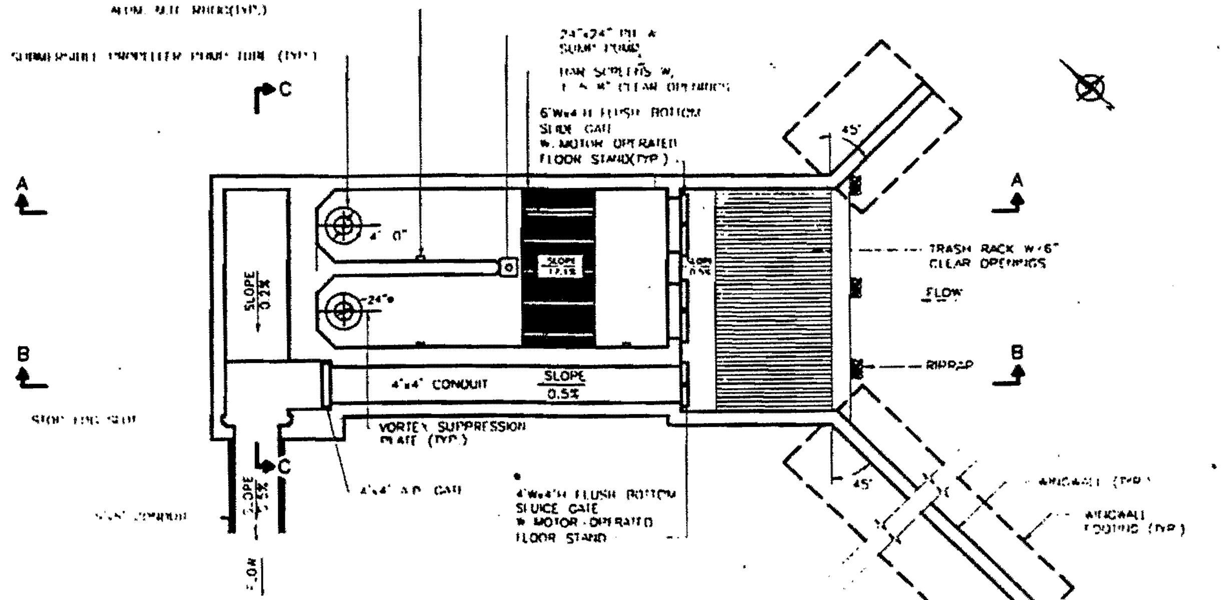
SECTION A–A



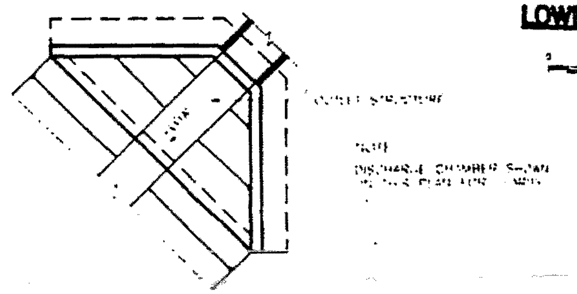
SECTION B–B



SECTION C–C



LOWER PLAN



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NEW YORK, NEW YORK

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PORT MONMOUTH COMBINED FLOOD CONTROL AND SHORE PROTECTION FEASIBILITY STUDY

TYPICAL PUMP STATION

Approval Recommended:
CHIEF, ENGINEERING DIVISION
Approved By
COLONEL, C.E. DISTRICT ENGINEER

SHEET REFERENCE NUMBER:

DATE: JUNE 1998
SCALE: AS SHOWN
FIG.: F-19

JUNE 2000

RARITAN BAY AND SANDY HOOK BAY, NEW JERSEY
FEASIBILITY REPORT
PORT MONMOUTH COMBINED HURRICANE AND STORM DAMAGE REDUCTION FEASIBILITY STUDY

APPENDIX H

MONITORING

H

RARITAN BAY AND SANDY HOOK BAY, NEW JERSEY
FEASIBILITY REPORT
PORT MONMOUTH COMBINED HURRICANE AND STORM DAMAGE REDUCTION FEASIBILITY STUDY

Appendix H - Monitoring

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FEASIBILITY REPORT
PORT MONMOUTH COMBINED HURRICANE AND STORM DAMAGE REDUCTION FEASIBILITY STUDY

Appendix H - Monitoring

Introduction

H-1. Shore protection projects are periodically monitored in order to 1) improve the understanding of the physical processes acting on a beach fill project and their effect on the beach fill performance, and 2) plan the timing and volumetric requirements of renourishment or other maintenance. Because of the project's relatively small coastline and bay conditions with relatively small wave spectra compared to the open ocean coastlines and no significant inlet pressures, the collection of physical data (waves, currents, etc.) at Port Monmouth and the correlation with the beach fill performance may not yield any significant insights into general beach fill or dune performance. Therefore, the project monitoring will neglect the collection of physical data. Rather, the monitoring program will focus on monitoring the project response and appropriately modifying (if necessary) the renourishment schedule described in this study.

H-2. The proposed monitoring program for Port Monmouth consists of the monitoring, of the project response. There are two tasks within the monitoring of the shoreline protection project, fill monitoring and shoreline change monitoring. A schedule of monitoring activities is presented in Table H-1.

Fill Monitoring

H-3. **Beach Profiles.** Beach and dune profiles will be surveyed every other year throughout the project life. Special profile surveys will be conducted subsequent to all major storm events. A total of 12 profiles will be surveyed, PL-210 through PL-221 as identified in Figure H-1. Repetitive survey of these monitoring profiles will track the movement of fill alongshore and offshore, provide estimates of erosion and/or accretion, and determine dune stability. Most important, the profiles will indicate when renourishment is necessary, and if the integrity of the design dune profile is violated. The profile will extend from the landward edge of the design dune out to approximately 300 feet offshore.

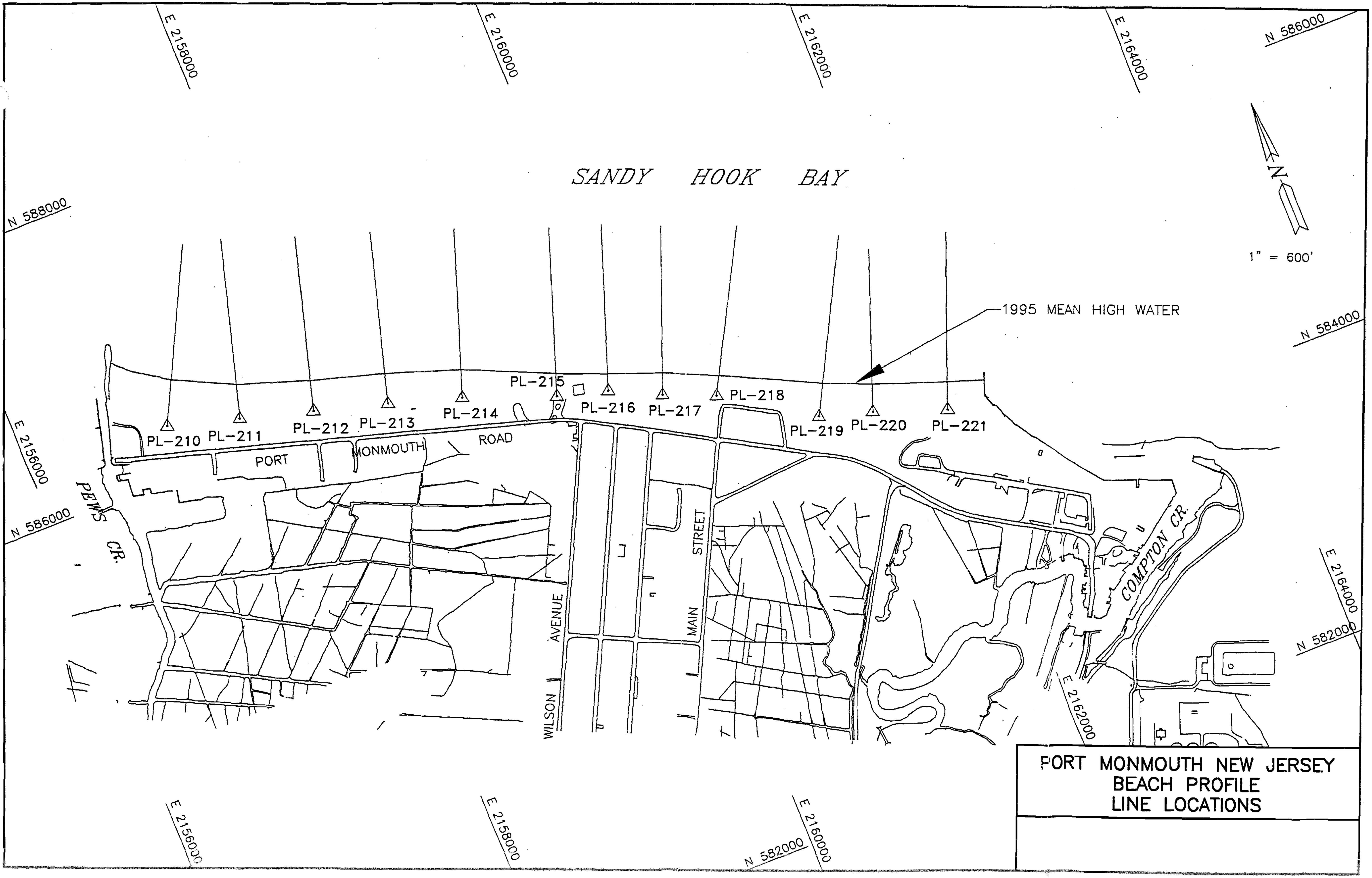
H-4. **Beach Sediment Samples.** In addition to profiles, sediment redistribution across the entire profile will be monitored every other year at the same time of the beach profile surveying. This will be accomplished by taking sediment grab samples along 3 profile lines (PL-213, PL-217, PL-219). These samples will be taken at a minimum of five subaerial and nearshore sample locations (dune crest, berm crest, mean high water, mid-tide level, and mean low water) per profile line, and two locations offshore. Offshore samples will be taken at -4.0, and -8.0 NGVD. The northing, easting, and elevation of each sample will be

TABLE H-1

SCHEDULE OF COASTAL MONITORING ACTIVITIES
PORT MONMOUTH, NEW JERSEY

	10 Year Renourishment Cycle									
	YEAR 1 (1)	YEAR 2	YEAR 3	YEAR 4	YEAR 5	YEAR 6	YEAR 7	YEAR 8	YEAR 9	YEAR 10 (2)
Fill Monitoring										
Beach Profiles	X		X		X		X		X	
Creek Bathymetry	X (3)		X (3)		X (3)		X (3)		X (3)	
Sediment Samples	X		X		X		X		X	
Shoreline Change										
Aerial Photography	X (4)	X (4)	X (4)	X (4)	X (4)	X (4)	X (4)	X (4)	X (4)	X (4)
Environmental Monitoring										
Pews Creek Tide Gate Monitoring	X	X								
Dune Monitoring	X	X	X							
Levee Monitoring	X	X	X							
Benthos Finfish Monitoring	X	X								
Piping Plover Monitoring	X	X								
Monitoring Reports		X		X		X		X		X

(1) First Year Following Initial or Maintenance Fill Operation
(2) Maintenance Fill (or Expiration of Project)
(3) First Two Renourishment Cycles Only
(4) Twice a Year for the First Renourshment Cycle, Then Once a Year Thereafter



PORT MONMOUTH NEW JERSEY
BEACH PROFILE
LINE LOCATIONS

FIGURE H-1

recorded. Sediment grab sampling will be conducted during the profile survey to optimize fieldwork.

H-5. **Beach and Dune Response Data Analysis.** Data analysis will include profile volume change and shape readjustment, area of loss or gain on profile, volume of fill remaining in the project, assessment of alongshore and cross-shore fill movement from beach and near-shore fill placement area, and storm response. Sediment analysis will include grain size statistics of native and fill material as well as their readjustment over the monitoring period, to be used in the assessment of fill and renourishment factors for future fill requirements.

H-6. **Sediment Sample Analysis.** Sieve analysis will be performed in accordance with the American Society for Testing and Materials (ASTM) standard methods designation D-422-63 for particles size analysis of soils (ASTM, 1987), and in accordance with the Shore Protection Manual (SPM, 1994 ed.) and TP77-6 "Review of Design Elements for Beach Fill Evaluation." These methods cover the quantitative determination of the distribution of sand size particles. The results from the sieve analysis should be used to calculate mean grain size, sorting, and silt percentage for each sediment sample and composite samples.

H-7. **Creek Channel Monitoring.** Pews Creek and Compton Creek are in close proximity to the coastal shoreline project area. Beach fill projects in Port Monmouth could potentially contribute to shoaling of one (or both) of the creeks. Although the initial beach fill volume is low, subsequent renourishment activity may influence the creek bathymetry. Monitoring of the creeks will be required every other year during the first two renourishment cycles and will include bathymetric surveys at the mouth of each creek.

Shoreline Change Monitoring

H-8. **Aerial Photography.** Aerial photography overflights of the project area and adjacent areas will be conducted throughout the project life. The photographs will be used to create a comprehensive controlled coverage. Flights will be performed twice a year for the first renourishment cycle and then yearly throughout the project life. Additionally, aerial photographs will be taken subsequent to major storm events throughout the project life.

H-9. **Shoreline Change Data Analysis.** Data analysis will result in maps of successive shorelines. In conjunction with beach profile analysis, the data will result in estimates of volume changes in beach fill within the project reaches and downdrift areas. Cross-shore and alongshore sediment movement will be compared. Shoreline data will be digitized, and compiled into a database of shorelines.

Environmental Monitoring

H-10. **Project Monitoring.** Monitoring of the dune and levee planting and wetland mitigation area will be conducted to determine the effectiveness of mitigation and to ensure a high percentage of vegetation success. The following sections provide a brief description of the District's proposed monitoring programs.

H-11. **Dune Monitoring.** The purpose of the post-construction dune monitoring is to document the stability of the constructed dune and to record annual changes in vegetation. The program is intended to identify changes in the structure and composition of vegetation over time, and to provide mitigation criteria in case of dune failure due to extrinsic factors such as blowouts and overwash.

H-12. **Levee Monitoring.** The District has developed a post-construction levee monitoring plan to assess the immediate and long-term success of the revegetation effort. Specifically, the plan will provide quantitative and qualitative measurements of the vegetative communities along the newly constructed levee. The plan is intended to identify changes in the structure and composition of vegetation over time, and to identify areas where supplemental planting may be required.

H-13. **Mitigation Monitoring.** The District has developed a post-construction monitoring plan for the selected wetland mitigation area. In particular, the plan is intended to ensure that the District's mitigation goals and objectives are fulfilled through documentation of the success/failure of the planting effort. The intent of the plan is to quantify the change in habitat conditions through the sampling of vegetation and hydrology over time. In addition, the post-construction monitoring program will identify potential problem areas and ensure that corrective actions are implemented in a timely manner (USACE 2000b).

H-14. **Intertidal and Sub-tidal Monitoring.** Monitoring of intertidal and sub-tidal habitats will be performed to provide information on impacts to shallow water faunal assemblages resulting from the selected plan. Accordingly, the results of the intertidal and sub-tidal monitoring effort would provide data to quantify impacts and recovery of benthic resources, as well as characterize the re-colonized benthic community. Currently, there is a lack of specific knowledge about the effects of beach nourishment activities on intertidal and sub-tidal resources in the region, therefore, data obtained through intertidal and sub-tidal monitoring for the selected plan would help to provide a firm technical base upon which to plan future nourishment projects in the region.

H-15. **Pews Creek Tidal Marsh Monitoring.** The purpose for post-construction monitoring of the tidal marsh associated with Pews Creek is to substantiate the District's position that the placement of a storm gate has minimal effect on the daily tidal cycle. Tide gages will be placed throughout the tidal marsh to ascertain tidal levels before and after placement of the storm gate. In addition, other water quality parameters may be measured and sampling of vegetation conducted.

H-16. **Piping Plover Monitoring.** The construction of the selected plan will expand the existing beach potentially creating more suitable piping plover nesting habitat. The monitoring plan will utilize the existing protocols as established along the Atlantic coast of New Jersey's Piping Plover Monitoring Plan. In addition, this is a recommendation of the USFWS pursuant to their Fish and Wildlife Coordination Act 2(b) Report (see Appendix E).

Analysis and Reporting

H-17. Bi-annual reports will be compiled from monitoring data during each nourishment cycle, as well as a final report at the end of the tenth year summarizing conclusions drawn over the nourishment period. In general, reports should provide description of observed changes in the beach fill area and concurrent observed coastal processes and sediment data. Cause and effect relations between coastal and geomorphic processes and observed fill behavior will be presented. Suggestions for improved renourishment design based on analysis of observations over the 10-year period will be made.

Costs

H-18. The cost estimates associated with monitoring are included in Table H-2. Total coastal monitoring costs would be approximately $51,000 annually over the 50-year project life (Table H-3). Estimated costs subject to change due to conditions at the time of implementation of the monitoring program. Environmental costs would be incurred over the first three years with an annualized cost of $52,100 (Table H-4).

TABLE H-2

PROJECT MONITORING COSTS

	COST PER EVENT	MAINTENANCE CYCLE (First 10 Years)
BEACH PROFILES	$30,000	$150,000
SEDIMENT SAMPLING	$5,000	$25,000
AERIALS (2)	$15,000	$300,000
CHANNEL CREEK BATHYMETRY (3)	$15,000	$75,000
Pews Creek Tide Gate Monitoring (4)	$25,000	$50,000
Dune Monitoring (5)	$3,300	$10,000
Levee Monitoring (5)	$3,300	$10,000
Benthos Finfish Monitoring (4)	$350,000	$700,000
Piping Plover Monitoring (4)	$30,000	$60,000
REPORT	$10,000	$50,000
TOTAL (1)	**$486,600**	**$1,430,000**

(1) 1998 PRICE LEVEL
(2) TWICE A YEAR FOR FIRST RENOURSIHMENT CYCLE THEN ONCE A YEAR THEREAFTER
(3) FIRST TWO RENOURISHMENT CYCLES ONLY
(4) ONCE A YEAR FOR THE FIRST TWO YEARS
(5) ONCE A YEAR FOR THE FIRST THREE YEARS ONLY

TABLE H-3

ANNUAL COASTAL MONITORING COST ESTIMATE
PORT MONMOUTH - COASTAL MONITORING

YEAR	FUTURE WORTH	PRESENT WORTH FACTOR	PRESENT WORTH
1	$80,000	0.93787	$75,029
2	$40,000	0.87959	$35,184
3	$80,000	0.82494	$65,995
4	$40,000	0.77368	$30,947
5	$80,000	0.72561	$58,049
6	$40,000	0.68053	$27,221
7	$80,000	0.63824	$51,060
8	$40,000	0.59859	$23,944
9	$80,000	0.56140	$44,912
10	$40,000	0.52651	$21,061
11	$65,000	0.49380	$32,097
12	$25,000	0.46312	$11,578
13	$65,000	0.43434	$28,232
14	$25,000	0.40736	$10,184
15	$65,000	0.38204	$24,833
16	$25,000	0.35831	$8,958
17	$65,000	0.33604	$21,843
18	$25,000	0.31516	$7,879
19	$65,000	0.29558	$19,213
20	$25,000	0.27722	$6,930
21	$50,000	0.25999	$13,000
22	$25,000	0.24384	$6,096
23	$50,000	0.22869	$11,434
24	$25,000	0.21448	$5,362
25	$50,000	0.20115	$10,058
26	$25,000	0.18865	$4,716
27	$50,000	0.17693	$8,847
28	$25,000	0.16594	$4,148
29	$50,000	0.15563	$7,781
30	$25,000	0.14596	$3,649
31	$50,000	0.13689	$6,844
32	$25,000	0.12838	$3,210
33	$50,000	0.12041	$6,020
34	$25,000	0.11293	$2,823
35	$50,000	0.10591	$5,295
36	$25,000	0.09933	$2,483
37	$50,000	0.09316	$4,658
38	$25,000	0.08737	$2,184
39	$50,000	0.08194	$4,097
40	$25,000	0.07685	$1,921
41	$50,000	0.07207	$3,604
42	$25,000	0.06760	$1,690
43	$50,000	0.06340	$3,170
44	$25,000	0.05946	$1,486
45	$50,000	0.05576	$2,788
46	$25,000	0.05230	$1,307
47	$50,000	0.04905	$2,452
48	$25,000	0.04600	$1,150
49	$50,000	0.04314	$2,157
50	$25,000	0.04046	$1,012
SUM OF PRESENT WORTHS			$740,592
TOTAL ANNUAL COST			$51,133

INTEREST = 6.625%
PROJ.LIFE = 50
CAPITAL RECOVERY FACTOR = 0.06904

TABLE H-4

ANNUAL ENVIRONMENTAL MONITORING COST ESTIMATE

Year	Tide	Dune	Levee	Benthos/Finfish	Plover	Total	Present Worth Factor	Present Worth
1	$25,000	$3,300	$3,300	$350,000	$30,000	$411,600	0.94	$386,904
2	$25,000	$3,300	$3,300	$350,000	$30,000	$411,600	0.88	$362,208
3	-----	$3,400	$3,400	-----	-----	$6,800	0.82	$5,576
					Total	**$830,000**		**$754,688**
							Annual Cost (50-year period)	**$52,100**

Discount Rate = **6.625%**

June 2000

RARITAN BAY AND SANDY HOOK BAY, NEW JERSEY
FEASIBILITY REPORT
FOR
HURRICANE AND STORM DAMAGE REDUCTION
PORT MONMOUTH, NEW JERSEY

APPENDIX J

REAL ESTATE

J

RARITAN BAY and SANDY HOOK BAY, NJ

REAL ESTATE PLAN

FOR

HURRICANE and STORM DAMAGE REDUCTION

PORT MONMOUTH, NJ

US ARMY CORPS OF ENGINEERS
NEW YORK DISTRICT
REAL ESTATE DIVISION

JUNE 2000

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Preamble

A. Introduction: This Real Estate Plan is prepared in support of the Draft Feasibility Report dated June 2000 for Hurricane and Storm Damage Reduction for Raritan Bay, Sandy Hook Bay, Port Monmouth, New Jersey.

B. Authorization: The present study of the Raritan Bay and Sandy Hook shorefront areas was authorized by a resolution of the Committee of Public Works and Transportation of the U.S. House of Representatives adopted 1 August 1990.

C. Designation: Raritan and Sandy Hook Bay, New Jersey Hurricane and Storm Damage Reduction Project, Port Monmouth, New Jersey.

D. Location: Community of Port Monmouth, Middletown Township, Monmouth County, New Jersey. Port Monmouth is situated in northern Monmouth County, approximately 30 miles southwest of the City of New York and approximately 40 miles northeast of the Trenton, New Jersey, the state capital. The Project encompasses approximately 1.5 miles of shoreline along Raritan Bay (northern limit), extending from Compton Creek on the east to Pews Creek on the west, and the existing inland 15 foot NGVD contour line, which lies a short distance south of New Jersey Route 36, approximately 6,000 feet from the bayshore.

E. Non-Federal Sponsor: The non-Federal sponsor for this Project is the State of New Jersey (Department of Environmental Protection) ("NJDEP" or the "Sponsor"). In accordance with the provisions of the Water Resources Development Act of 1986 (WRDA 86), sixty-five (65%) percent of the construction costs of the Project will be borne by the Government and the remaining thirty-five (35%) percent will be borne by the Sponsor. The cost of scheduled beach renourishment will be shared equally ("50/50") between the Government and the Sponsor.

1. **Statement of Purpose**: The purpose of this Real Estate Plan is to present the overall plan describing the minimum real estate requirements for the Raritan Bay, Sandy Hook Bay, Port Monmouth (NJ), Flood Control and Shore Protection Project.

This Plan supersedes all Plans previously submitted for this Project. This Real Estate Plan is tentative in nature and both the final real property acquisition lines and costs are subject to change after approval of the Decision Document to which this Plan is appended.

2. **Project Purpose and Features**:

A. The Port Monmouth area experiences damage from tidal inundation from the waters of Pews Creek and Compton Creek. During even moderate storms, tidal floodwaters enter the creeks and quickly spread over the broad, low-lying flood plain from both the east and west. A tidal stage of ten feet Mean Sea Level ("MSL") results in flooding so severe that most residents north of Route 36 are stranded. Extensive damage to hundreds of structures has been recorded in the Port Monmouth area during such storms.

B. The Plan of Improvement for the Port Monmouth Project calls for a beach berm and dune system along the Sandy Hook Bayshore, with a system of levees and floodwalls provided along both Pews and Compton Creeks. This protection is to extend continuously from the adjacent East Keansburg, NJ levee, across Pews Creek, along the bayshore, and thence along undeveloped lands adjoining Compton Creek to higher existing elevation. The plan details levees (7,070 ft.) and floodwalls (3,585 ft.) featuring a peak elevation of +14 feet NGVD, with a beach fill featuring a berm of width 50 feet at an elevation of +9 feet NGVD backed by a dune of crest width 25 feet at an elevation of +16 feet NGVD. In order to accommodate this design, the selected plan includes a storm gate across Pews Creek, three local road closure gates, one raising of Port Monmouth Road, and pedestrian dune walkovers. The bay shore protection requires 378,500 cubic yards of initial fill to be placed from a designated offshore borrow site including 125,000 cubic yards of advance fill and 127,300 cubic yards of fill every 10 years thereafter for 50 years. The construction of the levees requires 107,800 cubic yards of fill.

C. Required Lands, Easements, Rights-of-Way, Relocations and Disposal Areas (LERRD) – The estimated total acreage required for the Project is approximately **61.67 acres**, consisting of approximately **12.80 acres** in fee; approximately **48.87 acres** of perpetual and temporary easements: consisting of a Flood Protection and Levee Easement (**16.22 acres**), a Perpetual Beach Nourishment Easement (**13.14 acres**), a Perpetual Restrictive Dune Easement (**12.50 acres**)); a pipeline easement (**0.21 acres** and a temporary work area easement (**6.80 acres**). All of the foregoing is situated in Middletown Township. Access to the Project LER will be via existing public roads.

Approximately 93 tracts are to be established for the various estates, with approximately the same number of individual affected ownerships, (91 private and 2 public (Middletown Township and Monmouth County). In many instances, differing estates of varying acreages will be acquired from the same owners.

The land to be acquired in fee is required for environmental mitigation purposes and consists of approximately 20 tracts in Middletown Township situated along Port Monmouth Road, Plymouth Avenue and Creek Road in the vicinity of the Project. As many as 20 ownerships may be affected, all private. The average acreage to be acquired is less than one acre per ownership.

The land to be acquired for the Flood Protection Levee Easement consists of approximately 36 tracts. As many as 29 ownerships may be affected, 27 private and two public (Middletown Township Monmouth County). The average acreage to be acquired is less than one acre per ownership.

The land to be acquired for the Perpetual Beach Nourishment Easement consists of approximately 6 tracts. As many as 3 ownerships may be affected, one private and two public (Middletown Township and Monmouth County). The average acreage to be acquired is approximately four acres per ownership.

The land to be acquired for the Perpetual Restrictive Dune Easement consists of approximately 16 tracts. As many as 5 ownerships may be affected, 3 private and 2 public (Middletown Township and Monmouth County). The average acreage to be acquired is approximately 2 acres per ownership.

The land to be acquired for the Temporary Work Area Easement consists of approximately 52 tracts. The average acreage to be acquired is approximately 0.15 acre per ownership. As many as 42 ownerships may be affected, 40 private and 2 public (Middletown Township and Monmouth County). The proposed temporary work areas are typically adjacent to land to be acquired for other purposes (fee or easement and involve similar ownerships. As many as 15 private ownerships, however, may be used solely as temporary work areas. The average acreage to be acquired for this purpose is approximately 0.05 acre per ownership.

The land to be acquired for the Pipeline Easement consists of approximately 6 tracts. As many as six ownerships may be affected, five private and one public (Middletown Township). The total acreage to be acquired is approximately 0.21 acre.

(Note: Due to the "overlapping" of real estate interests, the arithmetic sum total of tracts does not equal 93 tracts.)

The Sponsor will be responsible for obtaining the real estate interests and performing any necessary facility/utility relocations.

The Project does not require acquisition of real property interests for borrow or disposal areas. No disposal areas will be required for any purpose. Approximately 380,000 cubic yards of sand will be required for beach fill. An additional 635,000 cubic yards of sand will be required for periodic renourishment (5 authorized renourishment cycles at 10-year intervals during the Project's 50-year life, at approximately 127,300 cubic yards per cycle). The required sand will be obtained from the offshore "Sea Bright" undersea borrow area, which has been permitted to the Corps of Engineers for this purpose by the non-Federal sponsor. The Sea Bright borrow area has an authorized capacity of approximately 55 million cubic yards of sand.

In addition, construction of the proposed levee system will require approximately 110,000 cubic yards of earthen material. The Contractor from existing, Project-approved, commercial sources in a 50-mile radius of the Project area will obtain this material.

There is one "Facility" Relocation required for this Project. This relocation is described and discussed in Paragraph 16 hereof, "Facility, Utility Relocations."

A summary of the acreage needed for the Project and the uses thereof is as follows:

Fee	**Acres**
Sub-total:	12.80
Permanent (Perpetual) Easements	
Pipeline Easement	0.21
Flood Protection Levee Easement	16.22
Perpetual Beach Nourishment Easement	13.14
Perpetual Restrictive Dune Easement	12.50
Sub-total:	42.07
Temporary Easements	
Temporary work area easement	6.80
Grand Total:	61.67 **acres**

D. Appraisal Information - The highest and best use of the land is as follows: Wetlands (public use/parklands), Residential Land, Residential Improved, and Commercial Land.

A summary of real estate costs, using a May 1998 valuation (Gross Appraisal) is as follows:

Real Estate Payments

a. Permanent Easements:	42.07acres		$ 488,361
b. Temporary Easements:	06.80 acres		$ 40,188
c. Severance Damages:			$ 48,116
		Sub total	$ 576,665
Mitigation Lands (fee acquisitions):			
13 Parcels and 13 owners	12.80 acres		$ 65,000.
Severance Damages			$ 9,922.
		Sub total	$ 74,922
REAL ESTATE PAYMENTS		TOTAL	$ 651,587

Administrative Costs:		
Federal		$ 35,000
Non-Federal		$ 71,000
	Subtotal	$ 106,000
Contingency (25%)		$ 189,250
Total Real Estate Costs, Lands and Damages		$ 946,837
	Say:	**$ 946,000**

3. **Non-Federal Sponsor Owned Lands**: The non-Federal Sponsor does not own any lands required for the construction, operation or maintenance of the Project. As discussed in Paragraph 5 below, "Existing Federal Projects," the Sponsor has, however, provided easements to support the adjacent Hurricane and Storm Damage Reduction project at Keansburg, New Jersey.

4. **Estates**: There are six "standard estates" to be obtained by the non-Federal Sponsor: Fee ("standard estate" No. 1); Flood Protection Levee Easement ("standard estate" No. 9); Pipeline Easement "standard estate" No. 13, Temporary Work Area Easement (4 years' duration) ("standard estate: No. 15); Perpetual Beach Nourishment Easement (un-numbered) and Perpetual Restrictive Dune Easement (un-numbered). The complete text of these estates is included in **Exhibits "A-1 and A-2."**

5. **Existing Federal Projects**: The Port Monmouth Project, if authorized, will become a component of the Raritan Bay and Sandy Hook Bay, New Jersey, Hurricane and Storm Damage Reduction Project, which includes authorized projects at nearby Keansburg, NJ and Old Bridge, NJ. Studies are also ongoing for other potential projects proposed for inclusion as components of the Raritan Bay and Sandy Hook Bay, New Jersey, Hurricane and Storm Damage Reduction Project at nearby Highlands, Leonardo, Union Beach, Keyport, and Cliffwood Beach (also situated along the shores of Raritan Bay in Monmouth County, NJ).

There is a shoreline protection project at Sea Bright to Manasquan Inlet, New Jersey on the Atlantic coast from whose offshore borrow area, excess sand will be taken for the Port Monmouth Project.

There is also an existing Federal Hurricane and Storm Damage Reduction Project at Keansburg, New Jersey situated directly west of the proposed Port Monmouth project. It is proposed that the levees to be constructed in connection with the Port Monmouth project will be "tied-in" to the existing levee system for the Keansburg project.

The State of New Jersey is also the non-Federal Sponsor for the above Projects. Except for a limited amount of land incidental to the "tie-in" of the proposed Port Monmouth Project levee system to the existing Keansburg project levee system, no LER for the Projects discussed above are required for the Port Monmouth Project. The existing easements provided by the non-Federal sponsor to support the Keansburg levee system are adequate to support its said "tie-in" to the proposed Port Monmouth levee system.

6. **Federally-Owned Lands**: There are no known Federal Government owned lands in the Project area.

7. **Navigational Servitude**: All lands required for the Port Monmouth Project is landward of the Mean High Water Mark. The Government will not exercise its rights under the doctrine of Navigational Servitude for the Port Monmouth Project, which in addition is not a Navigation project.

8. **Project Maps**: Attached as **Exhibits "F1 - F8"**. Exhibit F-1 depicts the general area of the Project. Exhibits F-2 - F-8 depict the project features and the minimum LER necessary to support these features.

9. **Induced Flooding**: No induced flooding is currently anticipated as a result of this Project.

10. **Baseline Cost Estimate**: A baseline cost estimate, in M/CASES format, is attached hereto as **Exhibits "B1 - B4**."

11. **Compliance with Public Law 91-646:** No persons, farms or businesses will be relocated for this project. Therefore, relocation assistance pursuant to Title II of Public Law 91-646, as amended, will not be required.

12. **Mineral and Timber Activities**: There are no present or anticipated mineral extraction or timber harvesting activities in the Project area and vicinity.

13. **Assessment of the Non-Federal Sponsor's Land Acquisition Experience and Ability**: An Assessment of the non-Federal Sponsor's Real Estate Acquisition Capability is attached hereto as **Exhibits "C-1 and C-2."** The New Jersey Department of Environmental Protection (NJDEP) is the non-Federal sponsor and has the legal and professional capability and experience to acquire and provide the LER for the construction, operation and maintenance of the project. NJDEP has condemnation authority and quick-take capability but it is not anticipated that these actions will be required for this project. NJDEP has successfully acquired the LER for the Keansburg, Sea Bright to Manasquan Inlet, the Long Branch, and Asbury Park Projects.

14. **Zoning**: Application or enactment of zoning ordinances are not anticipated for the Port Monmouth Project.

15. **Acquisition Schedules**: A schedule of acquisition by the non-Federal Sponsor is attached hereto as **Exhibits "D-1**(beach nourishment) **and D-2** (levee construction)**."**

16. **Facility/Utility Relocations**: There is one facility relocation anticipated for the Port Monmouth Project. Specifically, a 550-foot long section of Port Monmouth Road that is owned by Middletown Township will be raised by several feet along its existing 66-foot wide right-of-way as a component of the proposed levee system. The proposed road raising (elevation in grade) is not expected to adversely affect ingress and egress of any abutting properties.

17. **Hazardous, Toxic or Radiological Waste ("HTRW")**: There are no known contaminants or HTRW problems associated with the LER required for construction, operation and maintenance of the Port Monmouth Project.

18. **Project Support**: Local officials, landowners and other residents in the Project area are supportive of this Project.

19. **Notification to Non-Federal Sponsor**: Based on its past sponsorship of other Corps water resource (Civil Works) projects and ongoing discussions during the Project's Feasibility phase, the non-Federal Sponsor is aware of the risks of acquiring LER required for the Project prior to the signing of the Project Cooperation Agreement ("PCA"). Formal written notification of the risks of such acquisition, in accordance with paragraph 12-31, of Chapter 12, of the Corps of Engineers Real Estate Handbook, ER 405-1-12, will be forwarded to the non-Federal Sponsor during the Project's Preliminary Engineering and Design ("PED") phase.

21. **Other Issues:**

a. There are no known historical artifacts in the project area. However, there is one historical structure within the project area, a two-story wood building facing Sandy Hook Bay that is known as the Seabrook-Wilson House (a/k/a the "Spy House"). This building is listed on the National Register of Historical Places. The project will have no impact upon the structural integrity or be detrimental to the historical significance of this building.

b. The Non Federal Sponsor will provide public access to the beaches to be improved in accordance with the Public Access Plan attached hereto as **Exhibit "E".**

22. **Recommendations**: This report has been prepared in accordance with the Corps of Engineers Regulation ER 405-1-12. It is recommended that this report be approved.

Robert W. Hyatt
Chief, Real Estate Division

RARITAN BAY AND SANDY HOOK BAY, NEW JERSEY
FEASIBILITY REPORT
FOR
HURRICANE AND
STORM DAMAGE REDUCTION
PORT MONMOUTH, NJ

Real Estate Plan

Exhibit A - Estates

page JA

STANDARD ESTATE # 1

FEE: the fee simple title to (the land described in Schedule A) (Tract Nos. _________, ____________and __________), subject, however, to existing easements for public roads and highways, public utilities, railroads, and pipelines.

STANDARD ESTATE #9

FLOOD PROTECTION LEVEE EASEMENT: a perpetual and assignable right and easement in (the land described in Schedule A) Tracts Nos. ________, _______ and _______) to construct, maintain, repair, operate, patrol and replace a flood protection levee, including all appurtenances thereto; reserving, however, to the owners, their heirs and assigns, all such rights and privileges in the land as may be used without interfering with or abridging the rights and easement hereby acquired; subject, however, to existing easements for public roads and highways, public utilities, railroads and pipelines.

STANDARD ESTATE #13

UTILITY AND/OR PIPELINE EASEMENT

A perpetual and assignable easement and right-of-way in, on, over and across (the land described in Schedule A) (Tracts Nos.________, _________ and______), for the location, construction, operation, maintenance, alteration; repair and patrol of (overhead) (underground) specifically name type of utility; together with the right to trim, cut, fell and remove therefrom all trees, underbrush, obstructions and other vegetation, structures, or obstacles within the limits of the right-of-way; reserving, however, to the landowners, theirs heirs and assigns, all such rights and privileges as may be used without interfering with or abridging the rights and easement hereby acquired; subject, however, to existing easements for public roads and highways, public utilities, railroads and pipelines

STANDARD ESTATE #15

TEMPORARY WORK AREA EASEMENT: a temporary easement and right-of-way in, over and across the land described in Schedule A (Tract No. ___) for a period not to exceed forty-eight (48) months, beginning with the date of possession of the land is granted to the United States, for use by the United States, its representatives, agents and contractors as a work area including the right to move, store, and remove equipment and supplies and also to erect and remove temporary structures.

UN-NUMBERED ESTATE

PERPETUAL BEACH NOURISHMENT EASEMENT: A perpetual and assignable easement and right-of-way in, on, over and across the land described in Schedule A (Tract No. ___) to construct, operate, maintain, patrol, repair, renourish, and replace the beach berm and appurtenances thereto, including the right to borrow and/or deposit fill, together with the right to trim, cut, fell and remove therefrom all trees, underbrush, obstructions, and any other vegetation, structures, or obstacles within the limits of the easement;

EXHIBIT "A-1"

reserving, however, to the grantor(s), (his) (her) (its) (their) (heirs,) successors and assigns, all such rights and privileges as may be used without interfering with or abridging the rights and easement hereby acquired; subject, however, to existing easements for public roads and highways, public utilities, railroads and pipelines.

UN-NUMBERED ESTATE

PERPETUAL RESTRICTIVE DUNE EASEMENT: A perpetual and assignable easement and right-of-way in, on, over and across the land described in Schedule A (Tract No.___) to construct, operate, maintain, patrol, repair, rehabilitate, and replace a dune system and appurtenances thereto, together with the right to post signs, plant vegetation and prohibit the grantor(s), (his) (her) (its) (their) (heirs,) successors, assigns and all others from entering upon or crossing over said dune easement; reserving, however, to the grantor(s), (his) (her) (its) (their) (heirs,) successors and assigns, the right to construct dune walkover structures in accordance with any applicable Federal, State or local laws or regulations, provided that such structures shall not violate the integrity of the dune in shape or dimension and prior approval of the plans and specifications for such structures shall have been obtained from the District Engineer, U.S. Army Engineer District, and all other rights and privileges as may be used without interfering with or abridging the rights and easement hereby acquired; subject, however, to existing easements for public roads and highways, public utilities, railroads and pipelines.

EXHIBIT "A-2'

RARITAN BAY AND SANDY HOOK BAY, NEW JERSEY
FEASIBILITY REPORT
FOR
HURRICANE AND
STORM DAMAGE REDUCTION
PORT MONMOUTH, NJ

Real Estate Plan
Exhibit B – Baseline Cost Estimate

J B

	TOTAL PROJECT COSTS	non-Federal	Federal	Contingency	Project Cost
01	**LANDS AND DAMAGES**	$23,250	$11,000	$8,563	42,813
	(rounded)				**43,000**
01A	**PROJECT PLANNING**	**0**	**7,000**		
01A10	REAL ESTATE SUPPLEMENT/PLAN		5,000		
01A20	PRELIMINARY RE ACQUISITION MAPS		1,000		
01A30	PHYSICAL TAKINGS ANALYSIS		0		
01A40	PRELIMINARY ATTORNEY'S OPINION OF COMPENSABILITY		0		
01A50	ALL OTHER RE ANALYSES/DOCUMENTS		1,000		
01B	**ACQUISITIONS**	**19,750**	**2,000**		
01B10	BY GOVERNMENT	19,750			
01B20	BY LOCAL SPONSOR (LS)				
01B30	BY GOVT ON BEHALF OF LS				
01B40	REVIEW OF LS		2,000		
01C	**CONDEMNATIONS**	**0**	**0**		
01C10	BY GOVERNMENT				
01C20	BY LS				
01C30	BY GOVT ON BEHALF OF LS				
01C40	REVIEW OF LS				
01D	**INLEASING**	**0**	**0**		
01D10	BY GOVERNMENT				
01D20	BY LS				
01D30	BY GOVT ON BEHALF OF LS				
01D40	REVIEW OF LS				
01E	**APPRAISAL**	**3,500**	**2,000**		
01E10	BY GOVT (IN HOUSE)				
01E20	BY GOVT (CONTRACT)				
01E30	BY LS	3,500			
01E40	BY GOVT ON BEHALF OF LS				
01E50	REVIEW OF LS		2,000		
01F	**PL 91-646 ASSISTANCE**	**0**	**0**		
01F10	BY GOVERNMENT				
01F20	BY LS				
01F30	BY GOVT ON BEHALF OF LS				
01F40	REVIEW OF LS				
01G	**TEMPORARY PERMITS/LICENSES/RIGHTS-OF-ENTRY**	**0**	**0**		
01G10	BY GOVERNMENT				
01G20	BY LS	0			
01G30	BY GOVT ON BEHALF OF LS				
01G40	REVIEW OF LS		0		
01G50	OTHER				
01G60	DAMAGE CLAIMS				
01H	**AUDITS**	**0**	**0**		
01H10	BY GOVERNMENT				
01H20	BY LS				
01H30	BY GOVT ON BEHALF OF LS				
01H40	REVIEW OF LS				
01J	**ENCROACHMENTS AND TRESPASS**	**0**	**0**		

EXHIBIT "B-1"

01J10	BY GOVERNMENT		
01J20	BY LS		
01J30	BY GOVT ON BEHALF OF LS		
01J40	REVIEW OF LS		
01K	**DISPOSALS**	**0**	**0**
01K10	BY GOVERNMENT		
01K20	BY LS		
01K30	BY GOVT ON BEHALF OF LS		
01K40	REVIEW OF LS		
01N00	**FACILITY/UTILITY RELOCATIONS**	**0**	**0**
01Q00	**RESERVED FOR FUTURE HQUSACE USE**	**0**	**0**
01R	**REAL ESTATE PAYMENTS**	**0**	**0**
01R1	**LAND PAYMENTS**	0	0
01R1A	BY GOVERNMENT		
01R1B	BY LS	0	0
01R1C	BY GOVT ON BEHALF OF LS		
01R1D	REVIEW OF LS		
01R2	**PL 91-646 ASSISTANCE PAYMENTS**	0	0
01R2A	BY GOVERNMENT		
01R2B	BY LS		
01R2C	BY GOVT ON BEHALF OF LS		
01R2D	REVIEW OF LS		
01R3	**DAMAGE PAYMENTS**	0	0
01R3A	BY GOVERNMENT		
01R3B	BY LS	0	
01R3C	BY GOVT ON BEHALF OF LS		
01R3D	REVIEW OF LS		0
01R9	**OTHER**		
01T	**LERRD CREDITING**	**0**	**0**
01T10	LAND PAYMENTS	0	0
01T20	ADMINISTRATIVE COSTS		0
01T30	PL 91-646 ASSISTANCE		
01T40	ALL OTHER		

EXHIBIT - "B-2"

	TOTAL PROJECT COSTS	non-Federal	Federal	Contingency	Project Cost
01	**LANDS AND DAMAGES**	$698,705	$24,000	$180,676	903,381
	(rounded)				**903,000**
01A	**PROJECT PLANNING**	**0**	**15,000**		
01A10	REAL ESTATE SUPPLEMENT/PLAN		8,000		
01A20	PRELIMINARY RE ACQUISITION MAPS		2,000		
01A30	PHYSICAL TAKINGS ANALYSIS		0		
01A40	PRELIMINARY ATTORNEY'S OPINION OF COMPENSABILITY		3,000		
01A50	ALL OTHER RE ANALYSES/DOCUMENTS		2,000		
01B	**ACQUISITIONS**	**40,000**	**2,000**		
01B10	BY GOVERNMENT				
01B20	BY LOCAL SPONSOR (LS)	40,000			
01B30	BY GOVT ON BEHALF OF LS				
01B40	REVIEW OF LS		2,000		
01C	**CONDEMNATIONS**	**0**	**0**		
01C10	BY GOVERNMENT				
01C20	BY LS				
01C30	BY GOVT ON BEHALF OF LS				
01C40	REVIEW OF LS				
01D	**INLEASING**	**0**	**0**		
01D10	BY GOVERNMENT				
01D20	BY LS				
01D30	BY GOVT ON BEHALF OF LS				
01D40	REVIEW OF LS				
01E	**APPRAISAL**	**8,000**	**2,000**		
01E10	BY GOVT (IN HOUSE)				
01E20	BY GOVT (CONTRACT)				
01E30	BY LS	8,000			
01E40	BY GOVT ON BEHALF OF LS				
01E50	REVIEW OF LS		2,000		
01F	**PL 91-646 ASSISTANCE**	**0**	**0**		
01F10	BY GOVERNMENT				
01F20	BY LS				
01F30	BY GOVT ON BEHALF OF LS				
01F40	REVIEW OF LS				
01G	**TEMPORARY PERMITS/LICENSES/RIGHTS-OF-ENTRY**	**0**	**0**		
01G10	BY GOVERNMENT				
01G20	BY LS	0			
01G30	BY GOVT ON BEHALF OF LS				
01G40	REVIEW OF LS		0		
01G50	OTHER				
01G60	DAMAGE CLAIMS				
01H	**AUDITS**	**0**	**0**		
01H10	BY GOVERNMENT				
01H20	BY LS				
01H30	BY GOVT ON BEHALF OF LS				
01H40	REVIEW OF LS				
01J	**ENCROACHMENTS AND TRESPASS**	**0**	**0**		
01J10	BY GOVERNMENT				
01J20	BY LS				
01J30	BY GOVT ON BEHALF OF LS				
01J40	REVIEW OF LS				

EXHIBIT "B-3"

01K	**DISPOSALS**	**0**	**0**
01K10	BY GOVERNMENT		
01K20	BY LS		
01K30	BY GOVT ON BEHALF OF LS		
01K40	REVIEW OF LS		
01N00	**FACILITY/UTILITY RELOCATIONS**	**0**	**0**
01Q00	**RESERVED FOR FUTURE HQUSACE USE**	**0**	**0**
01R	**REAL ESTATE PAYMENTS**	**650,705**	**3,000**
01R1	**LAND PAYMENTS**	587,324	0
01R1A	BY GOVERNMENT		
01R1B	BY LS	587,324	0
01R1C	BY GOVT ON BEHALF OF LS		
01R1D	REVIEW OF LS		
01R2	**PL 91-646 ASSISTANCE PAYMENTS**	0	0
01R2A	BY GOVERNMENT		
01R2B	BY LS		
01R2C	BY GOVT ON BEHALF OF LS		
01R2D	REVIEW OF LS		
01R3	**DAMAGE PAYMENTS**	63,381	1,000
01R3A	BY GOVERNMENT		
01R3B	BY LS	63,381	
01R3C	BY GOVT ON BEHALF OF LS		
01R3D	REVIEW OF LS		1,000
01R9	**OTHER**		
01T	**LERRD CREDITING**	**0**	**2,000**
01T10	LAND PAYMENTS	0	
01T20	ADMINISTRATIVE COSTS		2,000
01T30	PL 91-646 ASSISTANCE		
01T40	ALL OTHER		

EXHIBIT "B-4

RARITAN BAY AND SANDY HOOK BAY, NEW JERSEY
FEASIBILITY REPORT
FOR
HURRICANE AND
STORM DAMAGE REDUCTION
PORT MONMOUTH, NJ

Real Estate Plan
Exhibit C – Assessment of Non-Federal Sponsor's R.E. Acquisition Capability

JC

ASSESSMENT OF NON-FEDERAL SPONSOR'S REAL ESTATE ACQUISITION CAPABILITY

PROJECT:

NON-FEDERAL SPONSOR:
New Jersey Department of Environmental Conservation

I. Legal Authority:

a. Does the sponsor have legal authority to acquire and hold title to real property for project purposes ? YES

b. Does the sponsor have the power of eminent domain for this project? YES

c. Does the sponsor have the "quick-take" authority for this project? YES

d. Are any of the lands/interests in land required for the project located outside the sponsor's political boundary? NO

e. Are any of the lands/interests in land required for the project owned by an entity whose property the sponsor cannot condemn? NO

II. Human Resources Requirements:

a. Will the sponsor's in-house staff require training to become familiar with the real estate requirements of Federal projects including P.L. 91-646, as amended? NO

b. If the answer to II.a. is yes, has a reasonable plan been developed to provide such training? N/A

c. Does the sponsor's in-house staff have sufficient real estate acquisition experience to meet its responsibilities for the project? YES

d. Is the sponsor's projected in-house staffing level sufficient considering its other work load, if any, and the project schedule? YES

e. Can the sponsor obtain contractor support, if required , in a timely fashion? YES

EXHIBIT C-1

RARITAN BAY AND SANDY HOOK BAY, NEW JERSEY
FEASIBILITY REPORT
FOR
HURRICANE AND
STORM DAMAGE REDUCTION
PORT MONMOUTH, NJ

Real Estate Plan
Exhibit D – Acquisition Schedule

JD

EXHIBIT "D-1"

EXHIBIT "D-1" - SCHEDULE OF ACQUISITIONS BY LS
FOR PORT MONMOUTH FOR TEMPORARY WORK AREA EASEMENT, PERPERUAL BEACH NOURISHMENT EASEMENT, AND PERPETUAL RESTRICTIVE DUNE EASMENT

ID	Task Name	Duration	Start	Finish	Predecessors
1	START REAL ESTATE ACQUISITION	388d	1/3/01	6/28/02	
2	SIGNED PCA OBTAINED BY NAN	0d	1/3/01	1/3/01	
3	OBTAIN LERRD	388d	1/3/01	6/28/02	2
4	(NAN-RE) FORMAL TRANSMITTAL OF FINAL DRAWINGS TO ACQUIRE LER	1d	1/3/01	1/3/01	2
5	(LS) PREPARE MAPPING AND GET LEGAL DESCRIPTION	60d	1/4/01	3/28/01	4
6	(NAN-RE) REVIEW MAPPING AND LEGAL DESCRIPTION	10d	3/30/01	4/12/01	5
7	(LS) OBTAIN TITLE EVIDENCE	45d	4/13/01	6/14/01	6
8	(NAN-RE) REVIEW TITLE EVIDENCE	15d	6/15/01	7/5/01	7
9	(LS) OBTAIN TRACT APPRAISAL	25d	7/6/01	8/9/01	8
10	(NAN-RE) REVIEW TRACT APPRAISAL	45d	8/10/01	10/11/01	9
11	(LS) CONDUCT NEGOTIATIONS	45d	10/12/01	12/13/01	10
12	(NAN-RE) REVIEW COUNTER OFFERS	10d	12/14/01	12/27/01	11
13	(LS) PERFORM CLOSINGS	45d	12/28/01	2/28/02	12
14	(NAN-RE) REVIEW CLOSINGS	10d	3/1/02	3/14/02	13
15	(LS) SUBMIT AUTHORIZATION FOR ENTRY TO CONSTRUCT	10d	3/15/02	3/28/02	14
16	(NAN-RE) REVIEW AUTHORIZATION FOR ENTRY TO CONSTRUCT	20d	3/29/02	4/25/02	15
17	(NAN-OC) REVIEW AUTHORIZATION FOR ENTRY TO CONSTRUCT BY OFFICE OF COUN	20d	4/26/02	5/23/02	16
18	(NAN-RE) CERTIFY LER FOR CONSTRUCTION	20d	5/24/02	6/20/02	17
19	(NAN-RE) DELIVERY OF CERTIFICATION PRIOR TO CONSTRUCTION AWARD	45d	6/21/02	8/22/02	18
20	CONSTRUCTION AWARD DATE	0d	8/28/02	8/28/02	19

EXHIBIT "D-2" - SCHEDULE OF ACQUISITIONS
FOR PORT MONMOUTH BY LOCAL SPONSOR FOR FEE ACQUISITIONS, LEVEE EASEMENTS, AND TEMPORARY WORK AREA EASEMENTS

EXHIBIT D-2

ID	Task Name	Duration	Start	Finish	Predecessors
23	(LS) PREPARE AND SUBMIT CREDIT REQUESTS	180d	8/26/04	5/4/05	21
24	(NAN-RE) REVIEW AND APPROVE CREDIT TO LS	180d	8/26/04	5/4/05	21
25	(NAN-RE) ESTABLISH VALUE FOR CREDITABLE LERRD TO PM	180d	8/26/04	5/4/05	21

EXHIBIT "D-2" - SCHEDULE OF ACQUISITIONS
FOR PORT MONMOUTH BY LOCAL SPONSOR FOR FEE ACQUISITIONS, LEVEE EASEMENTS, AND TEMPORARY WORK AREA EASEMENTS

EXHIBIT D-2

ID	Task Name	Duration	Start	Finish	Predecessors
23	(LS) PREPARE AND SUBMIT CREDIT REQUESTS	180d	8/26/04	5/4/05	21
24	(NAN-RE) REVIEW AND APPROVE CREDIT TO LS	180d	8/26/04	5/4/05	21
25	(NAN-RE) ESTABLISH VALUE FOR CREDITABLE LERRD TO PM	180d	8/26/04	5/4/05	21

Accessways – Three wooden dune overwalk structures will be provided as part of the project to ensure integrity of the protective dune.

Beach – The zone of unconsolidated material that extends landward from the low water lie to the place where there is marked change in material or physiographic form, or to the line of permanent vegetation.

Public Benefits – Benefits resulting from public recreational use and the prevention of damage to publicly-owned facilities.

Public Use – Available for use by any and all of the general public on equal terms.

Hurricane and Storm Damage Reduction Benefits – Benefits from the prevention of damages to Federal and Public property and facilities (i.e. lands and/or structures, except non-Federal public lands dedicated to park and conservation use) and developed private property and facilities due to shore erosion and/or tidal inundation.

4. The Proposed Project

The primary placement area includes beach fill at +9.0 feet NGVD and 50 foot berm width with taper sections to the east and west. The beach berm is backed by a beach dune at +16.0 feet NGVD with a 25 foot dune crest. However, advance fill is proposed in this area to offset long term erosion rates. Both the primary and two secondary areas will receive beach nourishment.

5. Public Access Plan

The location of the bay shore project (including the primary and secondary areas) is within the boundaries of State of New Jersey, Monmouth County and Middletown Township property. The Green Acres park, and the planned park infrastructure will provide sufficient public access to meet the Federal requirements for public access.

EXHIBIT "E - 2"

RARITAN BAY AND SANDY HOOK BAY, NEW JERSEY
FEASIBILITY REPORT
FOR
HURRICANE AND
STORM DAMAGE REDUCTION
PORT MONMOUTH, NJ

Real Estate Plan
Exhibit E – Public Access Plan

JE

RARITAN BAY AND SANDY HOOK BAY, NEW JERSEY
HURRICANE AND STORM DAMAGE REDUCTION
PORT MONMOUTH, NJ

PUBLIC ACCESS PLAN

1. Background

a. Purpose

The purpose of the public access plan is to describe public accessibility to the proposed dune and beach area that will be created as a result of the proposed Raritan Bay and Sandy Hook Bay, Port Monmouth, New Jersey Hurricane and Storm Damage Reduction Feasibility Study. In order for the project to conform with Federal and State regulations, public access is required.

b. Scope

The geographic scope of this public access plan includes the beachfront areas, which shall be provided beach fill in accordance with the recommended hurricane and storm damage protection plan for the area of Port Monmouth. The project is divided into three separate bayshore sections. The primary, central section spans 2,640 feet between beach profile lines (PL) 215 and 219. The western taper section extends 1,450 feet and the eastern taper section extends 550 feet. A system of levees and floodwalls will tie in to the primary bayshore beach and dune section to provide continuous, comprehensive protection.

2. Shoreline Ownership Category and Project Benefits

In accordance with ER 1165-2-130, all of the shores within the geographic scope of this project are considered to be under the general category of "Publicly Owned and/or Privately Owned with Public Benefits" for the purpose of Hurricane and Storm Damage Reduction. Recreational benefits are considered to be incidental for the hurricane and storm damage reduction purpose of this project. The project in its entirety is located within the boundaries of State, County and Township property. The bayshore land is predominantly under the jurisdiction of the Green Acres Program, which guarantees public access.

3. Definitions

Conservation Areas – Locations where human uses are generally excluded because of resource sensitivity. These locations include the areas subject to a dune conservation easement, which will be appropriately fenced and vegetated to ensure the integrity of the protective dune. These locations also include Green Acres Program lands and appurtenances which will be maintained for recreation and conservation.

EXHIBIT "E - 1"

f. Will the sponsor likely request USACE assistance in acquiring real estate? NO.

III. Other Project Variables:

a. Will the sponsor's staff be located within reasonable proximity to the project site? YES

b. Has the sponsor approved the project/real estate schedule/milestone? YES

IV. Overall Assessment:

a. Has the sponsor performed satisfactorily on other USACE projects? YES

b. With regard to this project, the sponsor is anticipated to be: HIGHLY CAPABLE

V. Coordination:

a. Has this assessment been coordinated with the sponsor? YES

b. Does the sponsor concur with this assessment? YES

Prepared by:

Robert A. Hass
Realty Specialist

Reviewed and approved by:

Robert W. Hyatt
Chief, Real Estate Division

EXHIBIT C-2

State of New Jersey

Christine Todd Whitman
Governor

Department of Environmental Protection

Natural and Historic Resources
Division of Engineering and Construction

Robert C. Shinn, Jr.
Commissioner

June 5, 2000

Mr. Paul Sabalis
Project Manager
New York Dist. Corps of Engineers
26 Federal Plaza
New York, NY 10278

Subject: Project 4015-Federal Shore Protection Project
Port Monmouth Section

Dear Mr. Sabalis:

I have reviewed the public access plan for the above subject project. I find the access plan complete and acceptable to this office. As this project moves forward, please be assured of our continued support for this vital project.

If you have any questions or concerns, please feel free to give me a call.

Sincerely,

Bernard J. Moore
Administrator

mm
Encl.

Phone
(732) 255-0770

1510 Hooper Avenue, Toms River, NJ 08753

Fax
(732) 255-0774

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EXHIBIT "E - 3"

EXHIBIT "D-2"

EXHIBIT "D-2" - SCHEDULE OF ACQUISITIONS
FOR PORT MONMOUTH BY LOCAL SPONSOR FOR FEE ACQUISITIONS, LEVEE EASEMENTS, AND TEMPORARY WORK AREA EASEMENTS

ID	Task Name	Duration	Start	Finish	Predecessors
1	START REAL ESTATE ACQUISITION	965d	8/27/01	5/6/05	
2	SIGNED PCA OBTAINED BY NAN	0d	1/3/01	1/3/01	
3	OBTAIN LEERD	965d	8/27/01	5/6/05	2
4	(NAN-RE) FORMAL TRANSMITTAL OF FINAL DRAWINGS	2d	8/14/01	8/15/01	2
5	(LS) PREPARE MAPPING AND GET LEGAL DESCRIPTION	90d	8/16/01	12/19/01	4
6	(NAN-RE) REVIEW MAPPING AND LEGAL DESCRIPTION	15d	12/20/01	1/9/02	5
7	(LS) OBTAIN TITLE EVIDENCE	60d	1/10/02	4/3/02	6
8	(NAN-RE) REVIEW TITLE EVIDENCE	30d	4/4/02	5/15/02	7
9	(LS) OBTAIN TRACT APPRAISALS	45d	5/16/02	7/17/02	8
10	(NAN-RE) REVIEW TRACT APPRAISALS	70d	7/18/02	10/23/02	9
11	(LS) CONDUCT NEGOTIATIONS	60d	10/24/02	1/15/03	10
12	(NAN-RE) REVIEW COUNTEROFFERS	15d	1/16/03	2/5/03	11
13	(LS) PERFORM CLOSINGS	50d	2/6/03	4/16/03	12
14	(NAN-RE) REVIEW CLOSINGS	20d	4/17/03	5/14/03	13
15	(LS) ACCOMPLISH PL-91-646 TITLE II RELOCATIONS	120d	5/15/03	10/29/03	14
16	(NAN-RE) REVIEW PL91-646 PAYMENTS	20d	10/30/03	11/26/03	15
17	(LS) SUBMIT AUTHORIZATION FOR ENTRY TO CONSTRUCT	20d	11/27/03	12/24/03	16
18	(NAN) REVIEW AUTHORIZATION FOR ENTRY TO CONSTRUCT	30d	12/25/03	2/4/04	17
19	(NAN-OC) REVIEW AUTHORIZATION FOR ENTRY TO CONSTRUCT BY OFFICE OF COUNSEL	20d	2/5/04	3/3/04	18
20	(NAN-RE) CERTIFY LER FOR CONSTRUCTION	20d	3/4/04	3/31/04	19
21	(NAN-RE) DELIVERY OF CERTIFICATION PRIOR TO CONSTRUCTION AWARD	60d	4/1/04	6/23/04	20
22	CONSTRUCTION AWARD DATE	0d	8/25/04	8/25/04	21

RARITAN BAY AND SANDY HOOK BAY, NEW JERSEY
FEASIBILITY REPORT
FOR
HURRICANE AND
STORM DAMAGE REDUCTION
PORT MONMOUTH, NJ

Real Estate Plan
Exhibit F – Real Estate Project Maps

JF

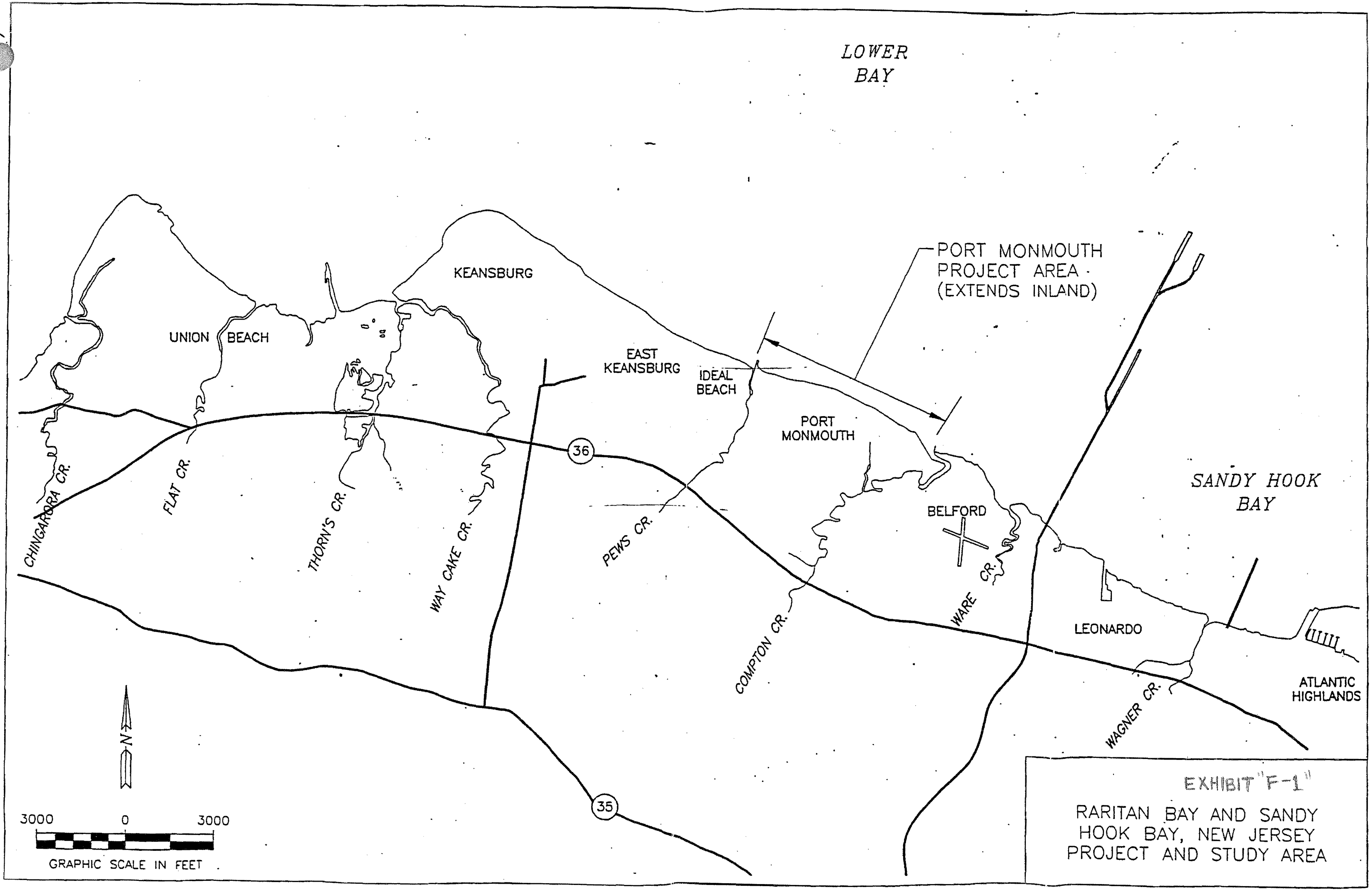
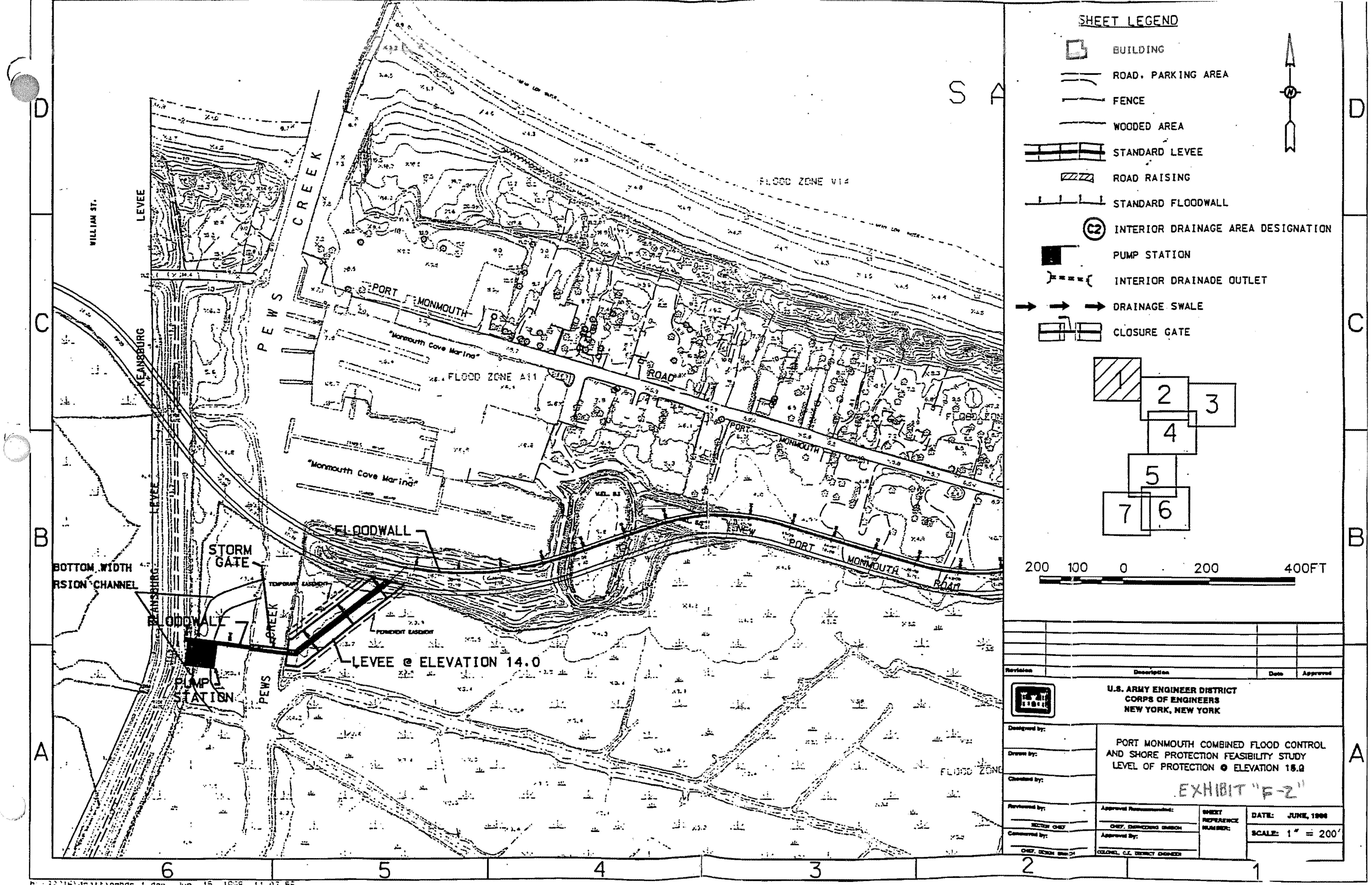


EXHIBIT "F-1"

RARITAN BAY AND SANDY
HOOK BAY, NEW JERSEY
PROJECT AND STUDY AREA

EXHIBIT "F - 1'



Revision	Description	Date	Approved

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
NEW YORK, NEW YORK

Designed by:

Drawn by:

Checked by:

Reviewed by:
SECTION CHIEF

Concurred by:
CHIEF, DESIGN BRANCH

Approval Recommended:
CHIEF, ENGINEERING DIVISION

Approved By:
COLONEL, C.E. DISTRICT ENGINEER

PORT MONMOUTH COMBINED FLOOD CONTROL
AND SHORE PROTECTION FEASIBILITY STUDY
LEVEL OF PROTECTION @ ELEVATION 15.0

EXHIBIT "F-2"

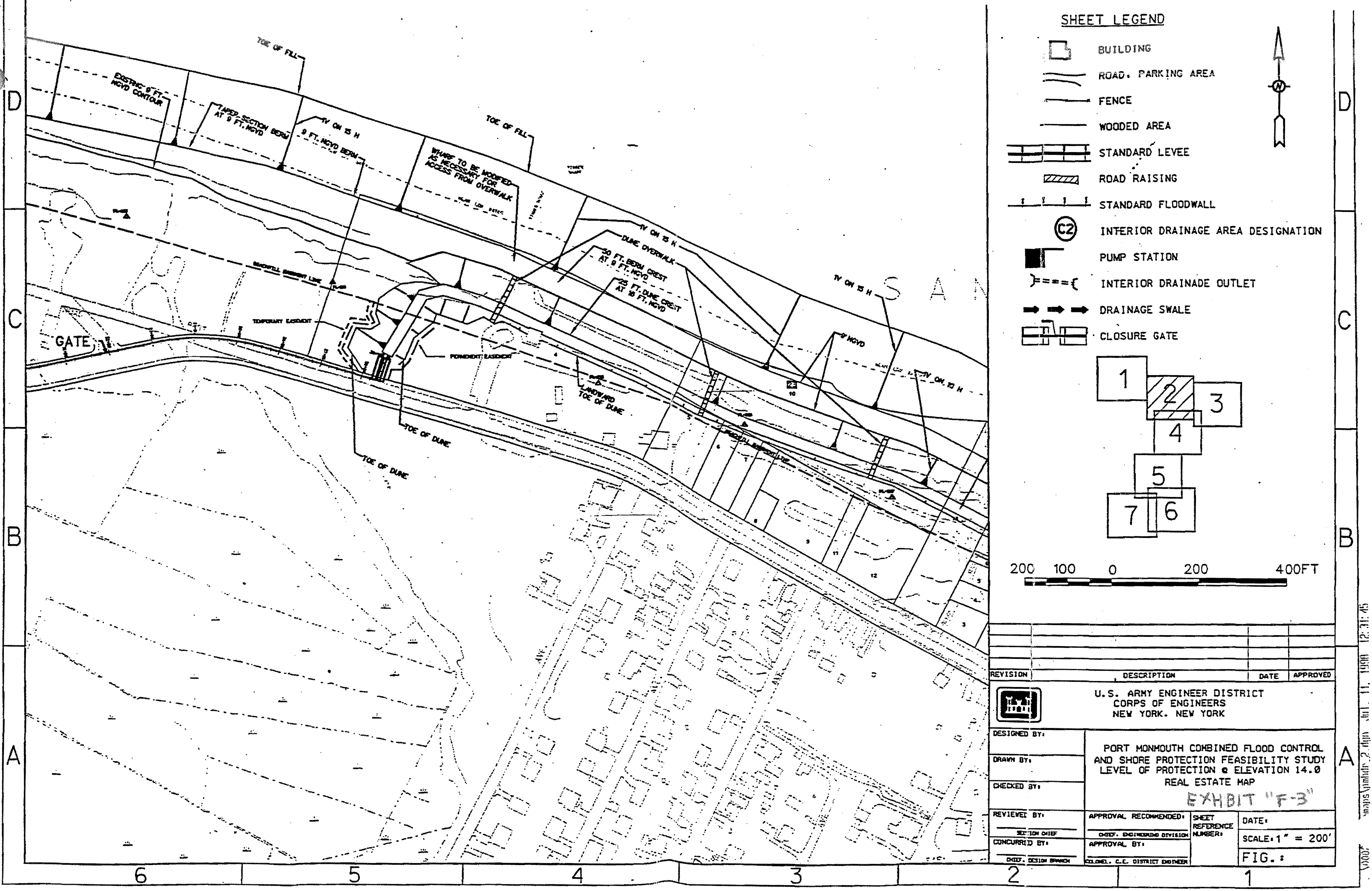
SHEET REFERENCE NUMBER:

DATE: JUNE, 1998

SCALE: 1" = 200'

h:\32216\draft\pmbdr_1.dgn Jun. 15, 1998 11:07:55

EXHIBIT "F - 2"



REVISION	DESCRIPTION	DATE	APPROVED

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
NEW YORK. NEW YORK

DESIGNED BY:

DRAWN BY:

CHECKED BY:

REVIEWED BY:
SECTION CHIEF

CONCURRED BY:
CHIEF, DESIGN BRANCH

PORT MONMOUTH COMBINED FLOOD CONTROL
AND SHORE PROTECTION FEASIBILITY STUDY
LEVEL OF PROTECTION @ ELEVATION 14.0
REAL ESTATE MAP

EXHIBIT "F-3"

APPROVAL RECOMMENDED:
CHIEF, ENGINEERING DIVISION

APPROVAL BY:
COLONEL, C.E. DISTRICT ENGINEER

SHEET REFERENCE NUMBER:

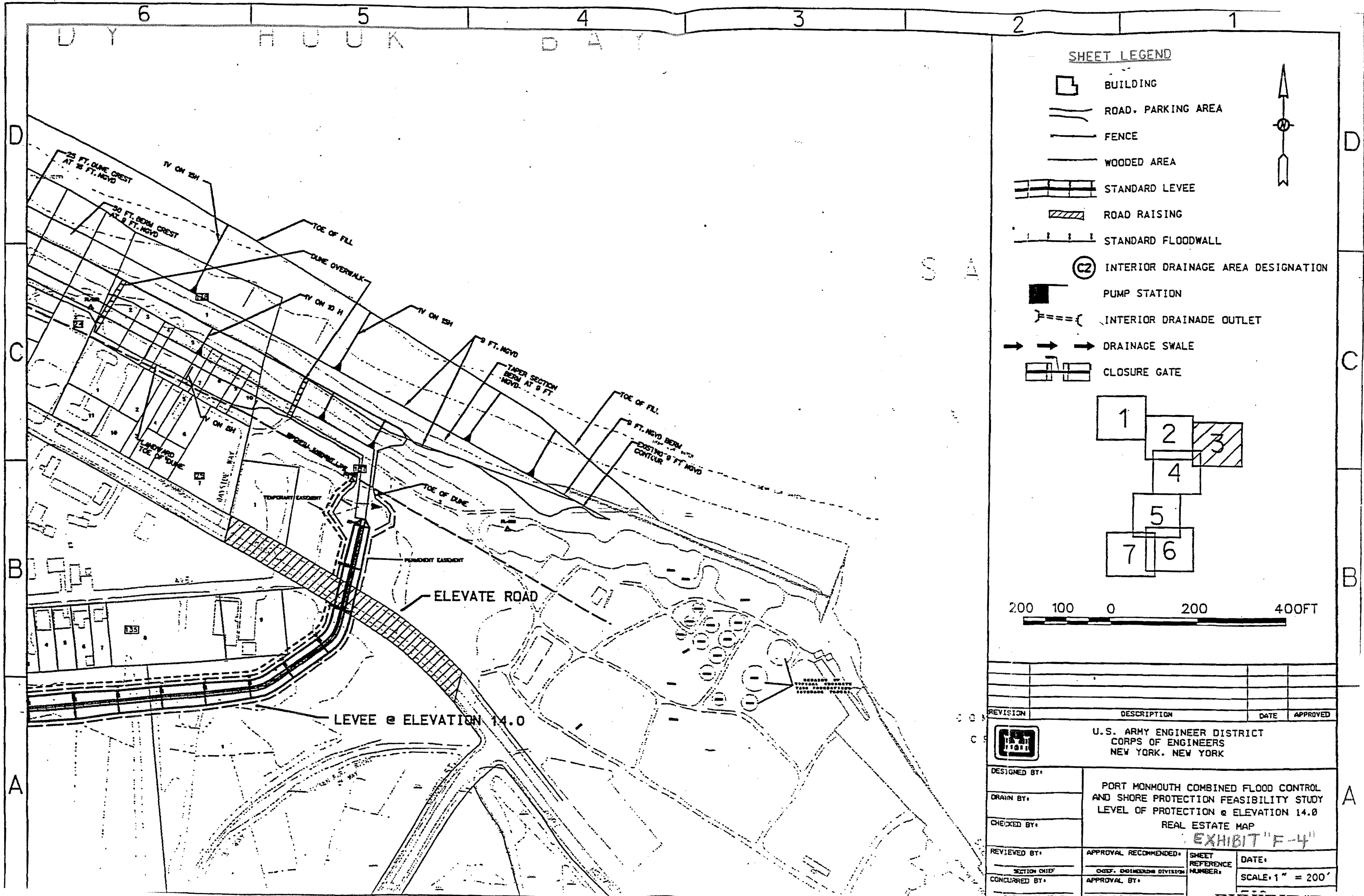
DATE:

SCALE: 1" = 200'

FIG.:

Jul. 11, 1990 12:31:45

EXHIBIT "F -



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U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
NEW YORK, NEW YORK

DESIGNED BY:

DRAWN BY:

CHECKED BY:

PORT MONMOUTH COMBINED FLOOD CONTROL
AND SHORE PROTECTION FEASIBILITY STUDY
LEVEL OF PROTECTION @ ELEVATION 14.0
REAL ESTATE MAP

EXHIBIT "F-4"

REVIEWED BY:
SECTION CHIEF

APPROVAL RECOMMENDED:
CHIEF, ENGINEERING DIVISION

SHEET REFERENCE NUMBER:

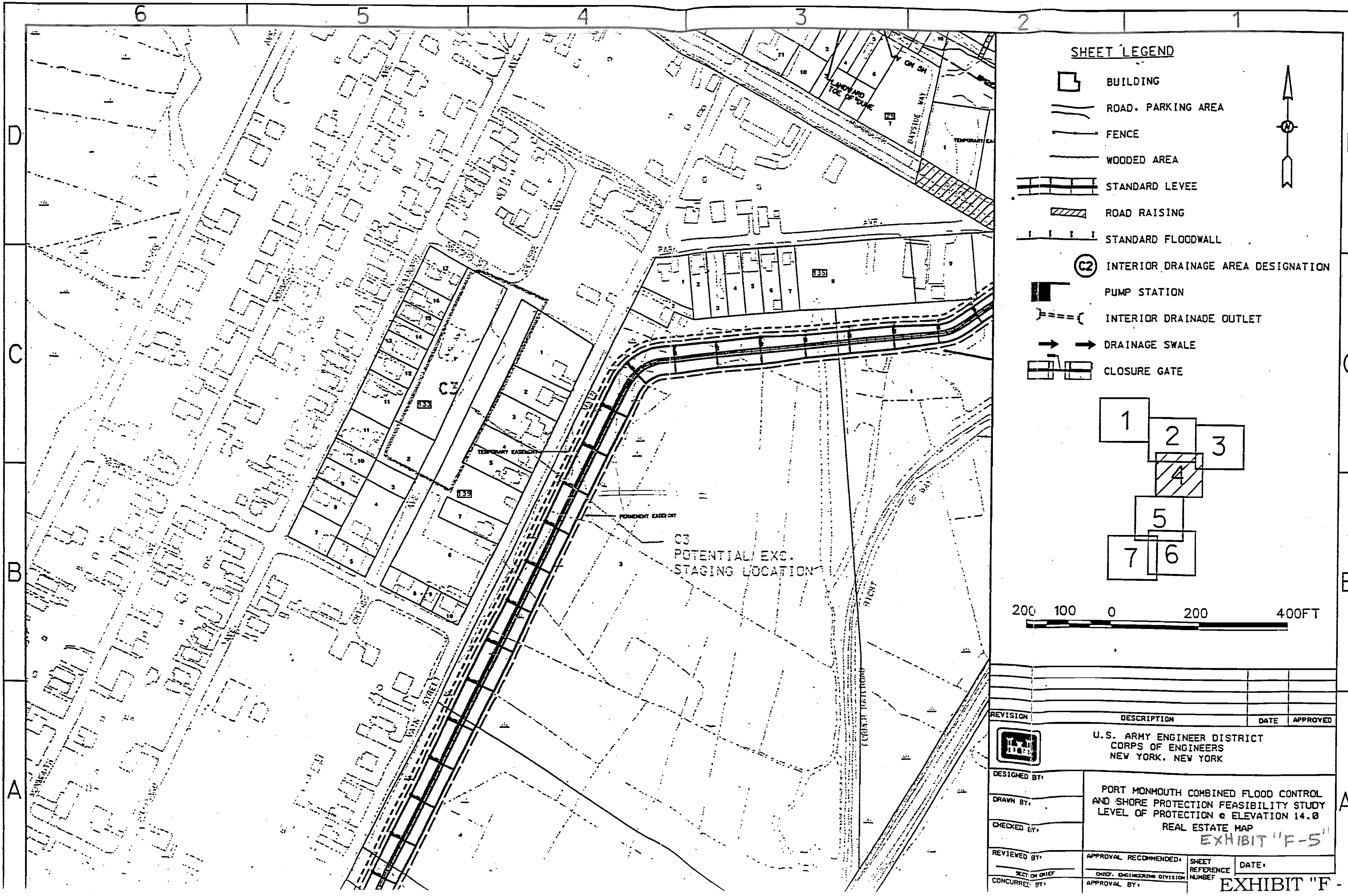
DATE:

SCALE: 1" = 200'

CONCURRED BY:
CHIEF, DESIGN BRANCH

APPROVAL BY:
COLONEL, C.E. DISTRICT ENGINEER

EXHIBIT "F - 4



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U.S. ARMY ENGINEER DISTRICT
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NEW YORK, NEW YORK

DESIGNED BY:

DRAWN BY:

CHECKED BY:

REVIEWED BY:

SECTION CHIEF

CONCURRED BY:

PORT MONMOUTH COMBINED FLOOD CONTROL
AND SHORE PROTECTION FEASIBILITY STUDY
LEVEL OF PROTECTION @ ELEVATION 14.0
REAL ESTATE MAP

EXHIBIT "F-5"

APPROVAL RECOMMENDED:

CHIEF, ENGINEERING DIVISION

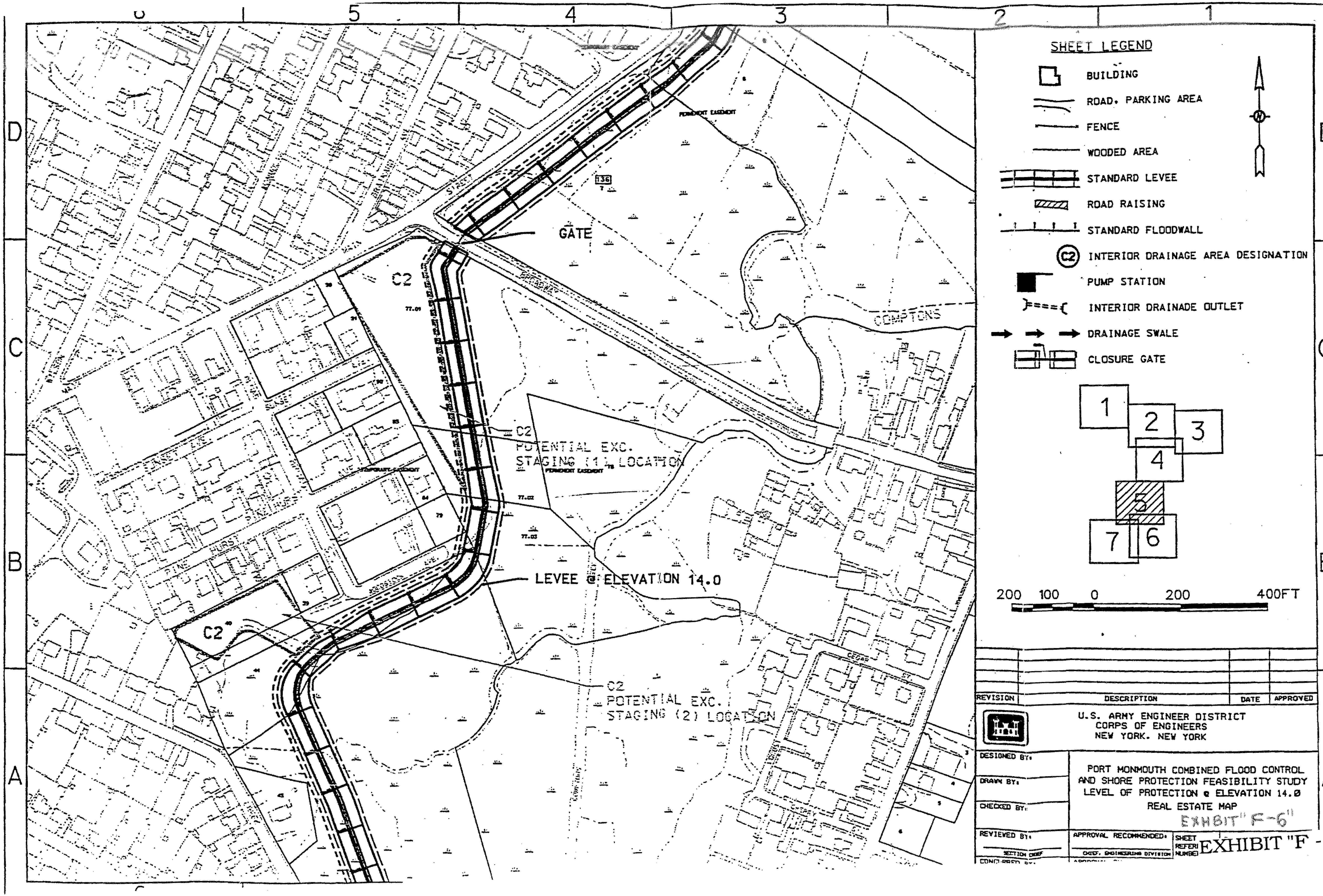
APPROVAL BY:

SHEET REFERENCE NUMBER

DATE:

EXHIBIT "F - 5"

3semaps\pmbdr_4.dgn Jul. 11, 1990 12:43:27



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NEW YORK, NEW YORK

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DRAWN BY:
CHECKED BY:
REVIEWED BY:
SECTION CHIEF
CONCURRED BY:

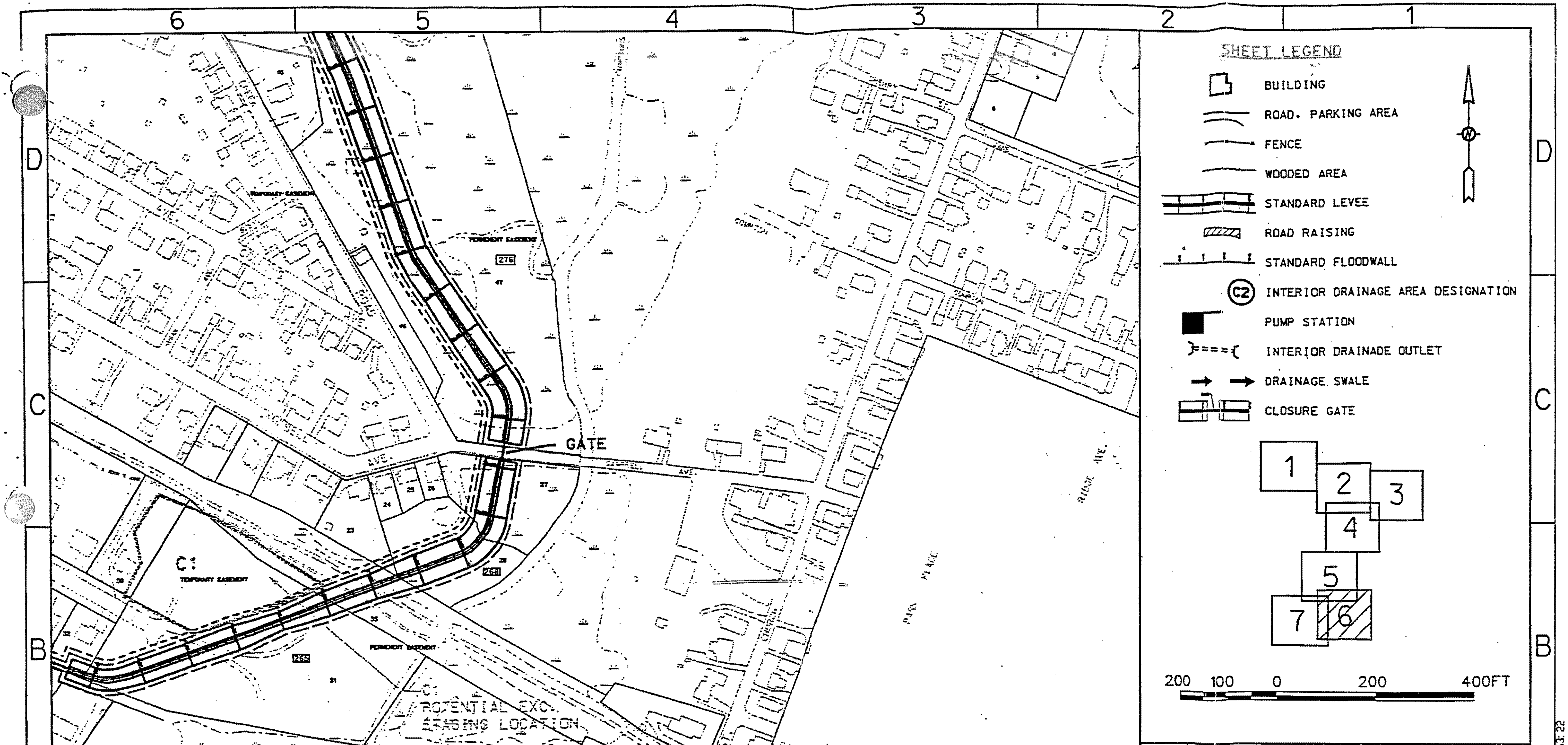
PORT MONMOUTH COMBINED FLOOD CONTROL
AND SHORE PROTECTION FEASIBILITY STUDY
LEVEL OF PROTECTION @ ELEVATION 14.0
REAL ESTATE MAP

EXHIBIT "F-6"

APPROVAL RECOMMENDED:
CHIEF, ENGINEERING DIVISION
SHEET REFERENCE NUMBER

EXHIBIT "F - 6"

apslexhibit 5.dgn Jul. 11, 1990 12:40:54



REVISION	DESCRIPTION	DATE	APPROVED

U.S. ARMY ENGINEER DISTRICT
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NEW YORK, NEW YORK

DESIGNED BY:
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CHECKED BY:
REVIEWED BY:
SECTION CHIEF
CONCURRED BY:
CHIEF, DESIGN BRANCH

PORT MONMOUTH COMBINED FLOOD CONTROL
AND SHORE PROTECTION FEASIBILITY STUDY
LEVEL OF PROTECTION @ ELEVATION 14.0
REAL ESTATE PLAN

EXHIBIT "F-7"

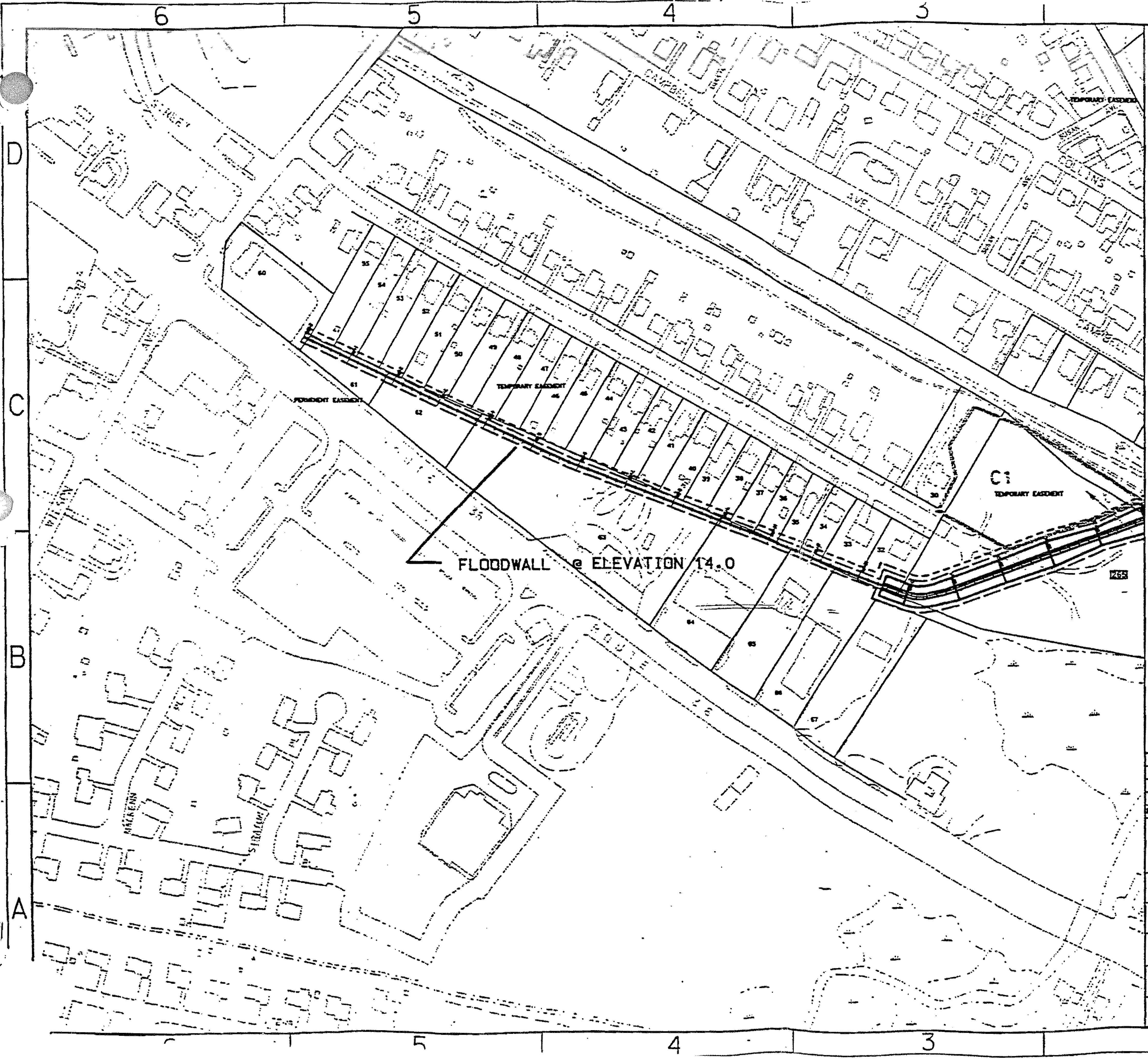
APPROVAL RECOMMENDED:
CHIEF, ENGINEERING DIVISION
APPROVAL BY:
COLONEL, C.E. DISTRICT ENGINEER

SHEET REFERENCE NUMBER:

DATE:
SCALE: 1" = 200'
FIG.:

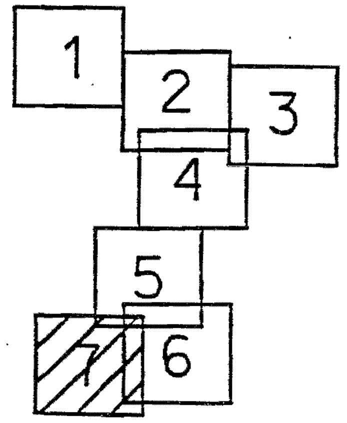
...\basemaps\pmbdr_6.dgn Jul. 11, 1998 12:53:22

EXHIBIT "F - 7"



SHEET LEGEND

☐	BUILDING
═	ROAD, PARKING AREA
—×—	FENCE
——	WOODED AREA
☰	STANDARD LEVEE
▨	ROAD RAISING
┴┴┴	STANDARD FLOODWALL
(C2)	INTERIOR DRAINAGE AREA DESIGNATION
■	PUMP STATION
}====(	INTERIOR DRAINADE OUTLET
→ →	DRAINAGE SWALE
⊟	CLOSURE GATE



200 100 0 200 400FT

F8

REVISION	DESCRIPTION	DATE	APPROVED

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
NEW YORK, NEW YORK

DESIGNED BY:

DRAWN BY:

CHECKED BY:

PORT MONMOUTH COMBINED FLOOD CONTROL
AND SHORE PROTECTION FEASIBILITY STUDY
LEVEL OF PROTECTION @ ELEVATION 14.0
REAL ESTATE PLAN

EXHIBIT "F-8"

REVIEWED BY:
SECTION CHIEF

CONCURRED BY:
CHIEF, DESIGN BRANCH

APPROVAL RECOMMENDED:
CHIEF, ENGINEERING DIVISION

APPROVAL BY:
COLONEL, C.E. DISTRICT ENGINEER

SHEET REFERENCE NUMBER:

DATE:

SCALE: 1" = 200'

FIG.:

00020\basemaps\pmbfr_7.dgn Jul. 11, 1990 12:57:27

EXHIBIT "F - 8"