US Army Corps
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New York District

DRAFT

General Design Memorandum

Passaic River
Flood Damage Reduction Project

Appendix C - Hydrology & Hydraulics



Great Falls, Paterson, April 1984

September 1995

Vol. I of II



US Army Corps
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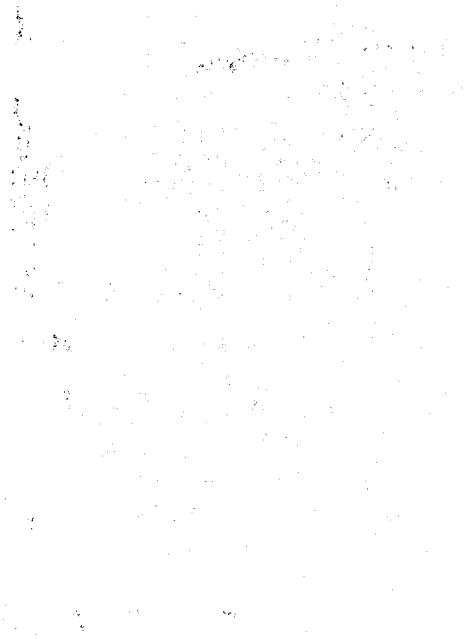

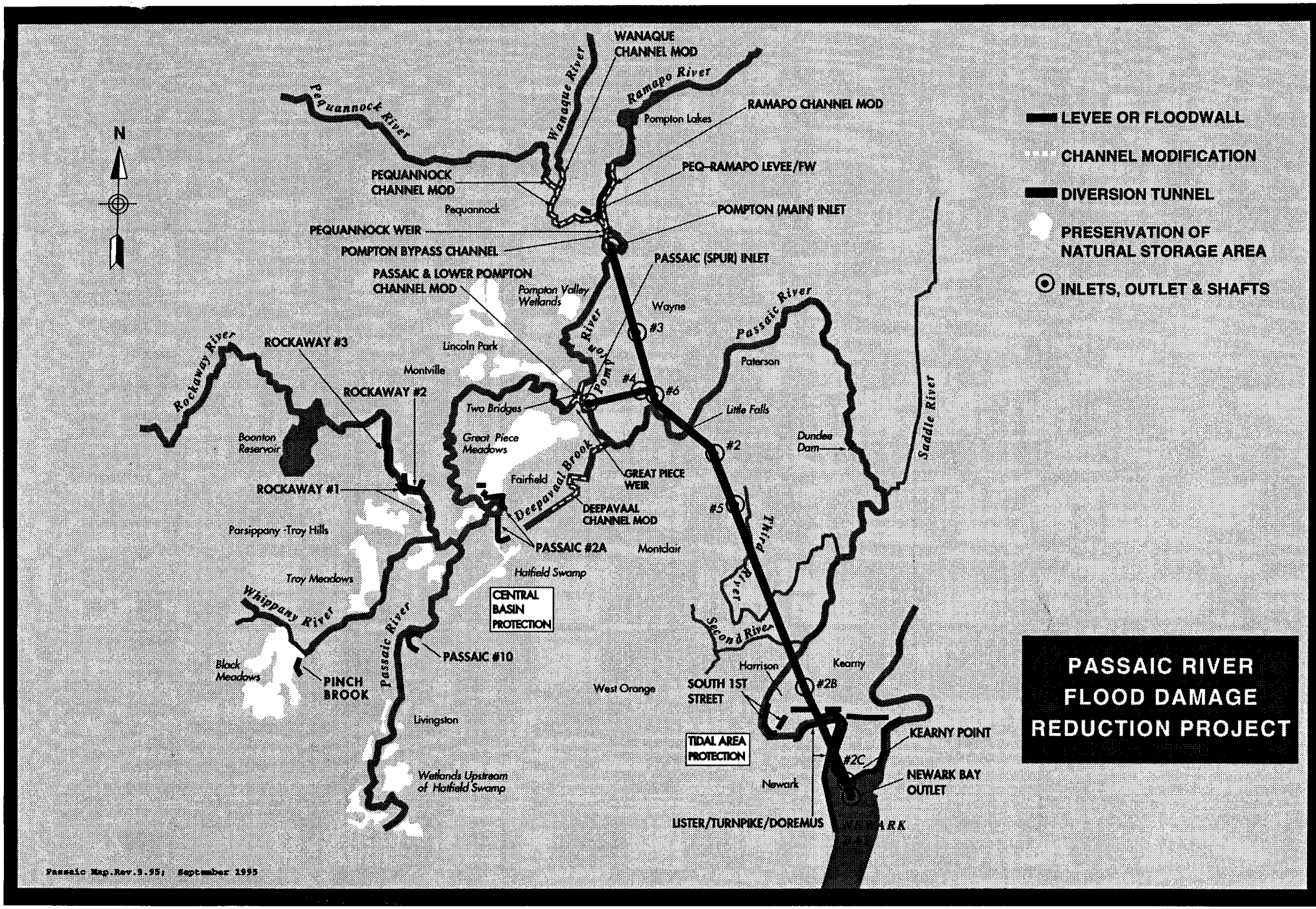
N
LEVEE OR FLOODWALL
CHANNEL MODIFICATION
DIVERSION TUNNEL
PRESERVATION OF NATURAL STORAGE AREA
INLETS, OUTLET & SHAFTS
PASSAIC RIVER FLOOD DAMAGE REDUCTION PROJECT
WANAQUE CHANNEL MOD
Wanaque River
Pequannock River
Ramapo River
Pompton Lakes
RAMAPO CHANNEL MOD
PEQUANNOCK CHANNEL MOD
PEQ-RAMAPO LEVEE/FW
Pequannock
POMPTON (MAIN) INLET
PEQUANNOCK WEIR
POMPTON BYPASS CHANNEL
PASSAIC (SPUR) INLET
PASSAIC & LOWER POMPTON CHANNEL MOD
Pompton Valley Wetlands
Pompton River
Wayne
#3
Passaic River
Paterson
Rockaway River
ROCKAWAY #3
Lincoln Park
Montville
#4
#6
ROCKAWAY #2
Two Bridges
Little Falls
Saddle River
Boonton Reservoir
Great Piece Meadows
#2
Dundee Dam
Fairfield
GREAT PIECE WEIR
ROCKAWAY #1
Deepavaal Brook
DEEPAVAAL CHANNEL MOD
#5
Third River
Parsippany-Troy Hills
PASSAIC #2A
Montclair
Hatfield Swamp
Troy Meadows
CENTRAL BASIN PROTECTION
Whippany River
Second River
Passaic River
PASSAIC #10
Harrison
Kearny
Black Meadows
PINCH BROOK
SOUTH 1ST STREET
#2B
West Orange
KEARNY POINT
TIDAL AREA PROTECTION
#2C
Livingston
NEWARK BAY OUTLET
Newark
Wetlands Upstream of Hatfield Swamp
LISTER/TURNPIKE/DOREMUS
Passaic Map.Rev.9.95; September 1995

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Hydrology & Hydraulics

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APPENDIX C

Hydrology and Hydraulics Appendix

September 1995

General Design Memorandum

A. General

1. Introduction

The Water Resources Development Act of 1990 (WRDA-1990) authorized for construction a modification of the plan recommended in the December 1987 Phase I GDM. The Congressionally authorized plan extended the tunnel approximately 6 miles downstream from the Third and Passaic River confluence to Newark Bay. This modified plan eliminated the need for the nine mitigation levees along the Lower Passaic River that were included in the Corps' 1987 recommended plan. WRDA-1990 also included a proposal for the Joseph G. Minish Passaic River Waterfront Park and Historic Area (in Newark, New Jersey) and language that would establish a Wetlands Bank and credits program. In addition, provisions of the act would eliminate structural improvements within certain locations of New Jersey.

As a result of the extension of the tunnel into Newark Bay, hydrologic and hydraulic studies have been performed on both the Passaic and Hackensack River Basins. Although no structural improvements are envisioned within the Hackensack River Basin (except those associated with the levees in Kearny, NJ), studies have been performed to determine the timing relationships between the two rivers, storm surge, sedimentation, circulation, salinity, water quality, and others. These studies, contained herein, document the impacts of placing the tunnel outlet within Newark Bay by focusing on the interrelation between the Passaic and Hackensack Rivers and Newark Bay.

The attached analysis summarizes the hydrologic and hydraulic studies performed for the General Design Memorandum (GDM). These studies have focused on the establishment of the existing, future unimproved and future improved conditions within the Passaic and Hackensack Basins. FDM studies will focus on the refinement of the future basin conditions for localized project areas. These future FDM studies have been scheduled, sequenced, costed and included in an overall PED Project Management Plan.

2. Objective and Purpose

The principle objective and purpose of this effort was to describe, detail and graphically illustrate how the entire Passaic River Flood Damage Reduction Project would work as an integrated system. This analysis was prepared to show basinwide

impacts for comparative conditions from the overall perspective of each of the features operating together as a complete system. The secondary objective, was directed at the development of accurate modeling tools capable of numerically reproducing the existing condition hydrologic and hydraulic basin response. This modeling effort was considered essential in determining the current baseline response now experienced within the basin. With the existing basin response firmly calibrated and verified, future unimproved and improved conditions studies were performed. This Appendix is a stand-alone document which will fully detail the existing, future unimproved and future improved conditions within the Passaic Basin.

3. Past Flood Problem

Prior flooding within the Passaic Basin has been chronic, frequent, resulted in significant property damage and loss of life. Since the early 1900's 26 lives have been lost along with many millions of dollars in damages to structures, contents, residential, commercial and industrial properties. Average annual damages within the basin now exceed $100 million.

The basin has been declared a federal disaster area 8 times in the recent past (1968, 1971, 1972, 1973, twice in 1975, 1984 and 1992) and experienced damages in April 1984 that would approximate $400 million (October 90 price level). Should the flood of record (October 1903) occur today, damages are estimated to be in excess of $2.1 billion. Floods are also more likely to lead to extensive polluting today, due to the highly urbanized (industrial and commercial) nature of the lower basin. With public awareness and concern increasingly focusing on Hazardous and Toxic Waste (HTRW), environmental and wetland issues, the basin as it exists today remains very vulnerable to uncontrolled flood waters.

For a brief description of individual past flood events, the reader is referred to the Section 13., Past Storms and Flood Events.

4. Existing Channel Conditions

The conditions of the Passaic River channel have physically changed since the occurrence of the October 1903 flood, due primarily to natural processes and man-made encroachments. The principal physical changes which have occurred in the Passaic channel are:

1) the completion of the Federal Navigation Channel from Newark Bay upstream to Eighth Street Bridge;

2) the reconstruction and addition of a large number of bridges across the Passaic River and its tributaries;

3) the placing of fill and walls along the banks of streams;

4) the removal of the Morris Canal and Pompton River crossing at Mountain View, New Jersey; and

5) the construction of reservoirs and dams within the basin.

Construction of the above features have tended to increase stages within the basin due to a reduction of channel area (i.e., bridges), while other improvements have increased the channel conveyance such as construction of the navigation channels in the lower Passaic River.

5. Prior Reports and Findings

Flooding in the Passaic River Basin has been studied at both the State and Federal level since before the turn of the century. From 1900 to 1940, the State of New Jersey produced reports containing a variety of recommendations, advancing flood control storage as the key to solving the problem. However, none of these recommendations were implemented.

In 1936 the Corps of Engineers (COE) became actively involved in the basin flood control planning effort as a result of the passage of Flood Control Acts. Since then, COE reports recommending plans of action were issued in 1939, 1948, 1962, 1969, 1972 and 1973. None of these plans were implemented because they did not receive widespread public support, with opposition based on the concerns of municipalities and various other interests throughout the Basin.

Historically, planning to solve the water related land resources problems and needs in the Passaic River Basin has been entangled in controversy and indecision. In the 50 years since the Corps of Engineers was first directed to plan solutions to the Passaic Basin's flood problems, public opposition for various reasons has prevented project implementation. As a result, the people of the Passaic River Basin continue to be plagued by economic losses, hazards to health and safety, and the threat of injury and loss of life from flooding. However, the Phase I GDM (December 1987), the forerunner of this analysis, proposed a comprehensive, basin-wide flood protection plan. As a result of the Phase I GDM report, Congress authorized for construction a modified version of the Corps Recommended Plan. The Corps has been involved in, along with other cooperating agencies, the detailed technical design of the 1990 Congressionally Authorized Plan. For a more detailed discussion on Prior Reports, the reader is referred to the Main Report, of the Phase II GDM.

6. Watershed Description

6.1. Passaic Basin:

The Passaic River which empties into Newark Bay, N.J. has a watershed area of 935 square miles. The basin lies in northeastern New Jersey and southeastern New York (Figure C-1). The roughly elliptical basin is bounded on the north and west by the Appalachian Highlands of New York and New Jersey, on the south by the First Watchung Mountains, and on the east by the Piedmont Plain. The watershed is divided into three distinct topographic and hydrologic regions, designated as the Highland Area, the Central Basin and the Lower Valley (Figure C-2).

6.2. Hackensack Basin:

The Hackensack River Basin which also empties into Newark Bay is situated in the northeasterly part of the State of New Jersey and the most southerly section of the State of New York, west of the Hudson River. The basin extends approximately 34 miles in a northerly direction from Newark Bay to Haverstraw, New York. The basin varies in width from 4 to 7 miles and drains an area of approximately 199 square miles, of which 132 square miles lie within the state of New Jersey and 65 square miles within the state of New York.

7. Topographic and Hydrologic Features

7.1. Passaic Basin:

The upper section, known as the Highland Area, consists of 500 square miles of the north-westerly portion of the Passaic River Basin. It is a mountainous and heavily wooded section of the Appalachian Mountain Province. This roughly rectangular area is about 13 miles wide and 38 miles long. It is characterized by a series of parallel ridges which are separated by steep-sided, narrow valleys in which flow the Ramapo, Wanaque, Pequannock, Rockaway and Whippany Rivers. This portion of the basin contains a large number of natural and artificial lake areas. The first three streams, the Ramapo, Wanaque and Pequannock join to form the Pompton River, the major flood producing river within the Passaic River Basin. The elevation of the Highland Area averages about 900 feet msl, and varies from about 1300 feet msl at the western rim to 300 feet msl in the valley at the eastern edge. The average stream slopes vary from 30 to 40 feet per mile (Table C-1 and Figure C-3).

The Central Basin, containing 262 square miles, is a flat, oval-shaped depression about 9 miles wide and 30 miles long. This area lies between the foot of the easterly slope of the Highland Area and the crescent-shaped Watchung Mountains to the south and east. Low-lying and marshy lands adjacent to the various streams form a floodplain that originally extended over 21,000 acres above Little Falls. The floodplain includes the

Great Piece Meadows, Hatfield Swamp, Troy Meadows, Black Meadow along the Passaic River, as well as the Bog and Vly Meadows adjacent to the Pompton River.

The Passaic River flows through this floodplain from the southwest and meanders gently to the north and east until it passes out of the area through the existing gorge at Little Falls. The five major tributaries from the Highland Area discharge into the Central Basin near Fairfield Township and at Two Bridges, the confluence of the Passaic and Pompton Rivers. The basin elevation averages 300 feet msl, varying from about 500 feet msl along the rims of the basin to 160 feet msl at the northeasterly end of the basin. The average stream slope varies from 19.5 feet per mile in the headwaters above Chatham to 1.4 feet per mile through the floodplain downstream of Chatham as given on Table C-1 and shown on Figures C-1 and C-3.

The Lower Valley, containing 173 square miles lies between Little Falls on the eastern edge of the Central Basin and Newark Bay. This roughly rectangular area, about 7 miles wide and 24 miles long, is heavily urbanized and densely populated. The valley has rolling sides and a comparatively flat, wide bottom which narrows down to about 0.75 mile below Dundee Dam. The average elevation of the area is about 250 feet msl, varying from 500 feet msl along the westerly edge of the basin to sea level at the mouth of the Passaic River. The main stream slope averages 2.9 feet per mile with concentrated falls of 33 feet at Beattie's Dam above Little Falls, 63 feet at the Great Falls above Paterson and 17 feet at Dundee Dam above the City of Passaic. The major tributary in this area is the Saddle River and it joins the Passaic River 15.5 miles above Newark Bay. The Saddle River has an average stream slope of 22 feet per mile. Smaller tributaries join the main stem of the Passaic at intervals below Two Bridges and are short and precipitous. These tributaries have stream slopes that vary from 45 to 194 feet per mile. The Passaic River upstream to Dundee Dam is subjected to tidal impacts from Newark Bay.

Each of the three topographic regions, have different flood producing characteristics. The basins of the Highland Area are the greatest flood producers in the Central Basin although they contain a large number of natural and artificial lakes and reservoirs. These impoundments, which tend to dampen the flood peaks to some extent, are used principally for water supply and recreational purposes. Although the flood peaks are somewhat reduced and slowed due to tributary lakes, reservoirs and valley storage between Pompton Lakes and the Passaic River at Two Bridges, the Pompton River peaks at Two Bridges from 24 to 48 hours earlier than the Passaic River peak during basin-wide floods. This phenomena has been recorded by the USGS stream flow gages for various historical flood events which have occurred, including the recent April 1984 event. The southerly upland tributaries, namely the Whippany and Rockaway Rivers, are as precipitous as the northerly tributaries, but they join the

Passaic River well upstream and hence are widely separated in time. This results in desynchronization of their peaks when compared to the northerly upland tributaries. In addition, the Rockaway and Whippany are greatly affected by the large valley storage in their lower reaches, and therefore, contribute less to the flood peaks in the Central Basin and Lower Valley.

Flooding upstream of Two Bridges is aggravated by the restricted river section and control above Little Falls which throttles the flow into the Lower Valley. This restriction can also create flow reversals of the Pompton River flood flows upstream into the Great Piece Meadows, (part of the Central Basin floodplain). Thus, the combined flow from the total watershed above Two Bridges (740.8 sq.mi.) raises the water level in the meadows until it becomes equal to or greater than that at Two Bridges. As a result, during periods of high runoff, the floodplain in the Central Basin acts as a natural detention reservoir. This action significantly retards the peak and reduces the flood intensities in the Lower Valley below Little Falls.

The Lower Valley is also subject to fluvial floods due to the short, flashy streams below Little Falls. These streams peak much earlier than the Upper Passaic River and the flood peaks are a result of high percents of runoff due to the extremely urbanized nature of the Lower Valley. Flood stages in the Passaic River downstream of Dundee Dam are also tidally influenced from Newark Bay. This area of the Passaic Basin is therefore vulnerable to both coastal Northeasterns and Hurricanes which can raise water levels within Newark Bay and upstream on the Passaic River. Pertinent watershed data for the Passaic River and its principal tributaries are given in Table C-1.

7.2. Hackensack Basin:

The Hackensack River Basin may be divided into two distinct areas. The lower third of the Basin, below Teaneck and Hackensack is known as the Meadowlands and the valley is covered by extensive tidal marsh and flat wetlands. North of the Meadowlands, the valley is composed of low rolling hills with marsh and flat lowlands bordering the stream at several locations. The drainage divide on the east and northeast separating it from the Hudson River Basin consists of the Hudson River Palisades. The Palisades range in elevation from 300 feet in the north to 100 feet in the south. The drainage divide separating the Hackensack River Basin from the Passaic River Basin to the west and south consists of rolling hills ranging in elevation from 600 feet in the north to 100 feet in the south.

The Hackensack River, a winding stream 45 miles long, rises in the vicinity of Haverstraw, N.Y., flows south and empties into Newark Bay at its junction with the Passaic River. The average fresh-water discharge at its mouth is approximately 292 cubic feet per second. The river has a comparatively flat slope of 1.7

feet per mile in the 22 river miles downstream of the Oradell Reservoir and 2.8 feet per mile in the 19 river miles between Oradell Reservoir and De Forest Reservoir. The river is tidal and navigable to New Milford N.J., approximately 20 miles from the mouth. At that point, navigation is cut off by low dams of the Hackensack Water Company, namely Oradell Reservoir.

Situated within the Piedmont physiographic province in northeastern New Jersey, the tidal portion of the Hackensack River drains an area of approximately 93 square miles. Nearly one third of this area falls within the Hackensack Meadowlands Development Commission (HMDC) land use district, which includes over 6,000 acres of wetlands.

The principal navigable tributaries of Hackensack River are Berry's Creek and Overpeck Creek, located 8.5 and 13.7 miles above the mouth of the river. Berry's Creek originates in Carlstadt, New Jersey and flows southwest and south to its confluence with the Hackensack River. Overpeck Creek originates in the eastern slopes of the valley at Tenafly, New Jersey, and flows south for a distance of 9 miles and joins the Hackensack River about 13.7 miles above the mouth at Little Ferry, New Jersey. Tributaries to Overpeck Creek include Teaneck Creek, Flat Rock Brook, and Metzler Brook.

8. Climate

The climate of the Passaic River Basin is characteristic of the entire Middle Atlantic Seaboard. Marked changes of weather are frequent, particularly during the spring and fall. The winters are moderate with average snowfall and the summers are moderate, hot and humid with frequent thunderstorms. The rainfall is well distributed throughout the year while the relative humidity is high. The average annual temperature varies from 48°F at Charlotteburg Reservoir to 57°F at Newark, with extremes from 26°F below zero (Canoe Brook) to 106°F above zero (Paterson). The growing season averages 171 days and the mean annual relative humidity varies from 67 percent to 73 percent. Prevailing winds are from the northwest with an annual average velocity of 9.7 miles per hour. Rainy days average about 121 per year. Climatological data are shown on Figures C-4, C-5, and C-6. Temperature data are given in Table C-2. Data for humidity and evaporation are given in Table C-3.

9. Precipitation Stations

The Passaic River Basin is presently served by a network of 41 official U.S. Weather bureau stations. Of these, 14 are equipped with automatic recording rainfall gages and 27 with standard, non-recording gages which are read one or more times daily. The location, period of record, and type of station are shown and summarized on Figure C-7. This figure also includes data on pertinent discontinued stations. Precipitation and mass curve station locations are shown on Figure C-8.

10. Annual and Monthly Precipitation

The average annual precipitation for the watershed is approximately 48 inches, as derived from a compilation of past record over a period of years at U.S. Weather bureau stations in and adjoining the basin. The observed extreme annual values were 85.99 inches at Paterson, N.J. (1882) and 25.26 inches at Morristown, N.J. (1930). Monthly extremes were 25.98 inches in September 1882 at Paterson and 0.02 inches at Plainfield and Jersey City in June 1949. The distribution if precipitation throughout the year is fairly uniform with higher amounts occurring during the summer months. Table C-4 provides a detailed summary of basin and adjacent rainfall gage data.

11. Snowfall

The average annual snowfall of 33.7 inches for the Passaic River Basin is equivalent to about four inches of rain. The snow season is longest in the Highland Area where it extends from the middle of October through the middle of April. The variation of snowfall over the basin is shown on Figure C-5. The average snowfall for the different areas and at individual stations is given in Table C-5. The depths of snow given are for freshly fallen snow with an approximate water content of one inch of water to 10 inches of snow.

12. Storm Types

Storms affecting the basin are characterized as tropical and extra-tropical events. The tropical storm arises in the tropics and the most severe, but not the most common occurrence of this type, is a hurricane. The season for tropical storms runs from June through November. Extratropical storms are developed from the interaction of a warm and a cold front. Thunderstorm activity is the frequent result of such interaction, but storms of much greater area extent also occur and are known as northeasters. Northeasters generally occur during November through April and are so named due to their predominate northeast winds. Thunderstorms are most frequent in the summer months and, due to rapid convective circulations, are generally limited in areal extent and can cause heavy rainfalls and localized flash floods.

13. Past Storms and Flood Events

A review of great storms which have occurred in the northeastern states (Table C-6) reveals that the Passaic River basin is located in the center of the North Atlantic storm belt. Those storms which have resulted in the worst floods within the Passaic River Basin are shown on Figures C-9 through C-16, and C-19, C-22 and C-25 (Isohyetal maps). The following paragraphs will briefly describe each event.

13.1. Storm of 20-24 September 1882.

This storm was the greatest recorded over the Passaic River Watershed prior to the storm of October 1903. This extra-tropical event resulted in a total maximum rainfall at Paterson, N.J., of 17.9 inches of which 11.4 inches occurred in 24 hours (Figure C-9). The average rainfall over the Passaic River Basin was about 9.2 inches. The storm was preceded by light rainfall during the months of July and August. The ground was reported to be quite dry at the start of the storm, resulting in high infiltration losses. This storm was less extensive than that of October 1903, and its destructive effects were generally confined to the Passaic River Basin.

13.2. Storms of February and March 1902.

This storm, a typical winter event of low intensity, extended over six days and consisted of heavy snow and sleet in February, followed in early March by moderate rains and a temperature rise. It was estimated that a total of 6.2 inches of rain and snow melt occurred over the Passaic River Basin. The maximum (water equivalent) precipitation recorded in 24 hours was 3.30 inches at Hanover, N.J., and 3.03 inches at Paterson, N.J.

This event was a major winter flood in the Passaic River Basin. Heavy rains falling on a deep snow cover combined with a rise in temperature caused a sudden increase in stream flow. The estimated peak discharge on the Passaic River at the S.U.M. (Society for Useful Manufactures) Dam located just upstream of the Great Falls at Paterson, N.J., was 22,500 cfs on 2 March 1902. A runoff volume of 5.20 inches was reported during this flood, equivalent to about 83 percent of the total rainfall and accumulated snow over the watershed. Maximum discharges for this flood at two other locations in the basin, as estimated by other agencies, are listed in Table C-7.

13.3. Storm of 7-12 October 1903.

This storm, one of the worst in the vicinity of the Passaic River Watershed, occurred following three months of excessive rainfall which had saturated the ground and raised groundwater levels throughout the Passaic River Basin. This extra-tropical storm was comparatively widespread but centered over Paterson, N.J., where a total of 15.5 inches of rain was recorded (Figure C-10). The greatest intensity occurred during the night of 8 October and morning of 9 October when over 75 percent of the rain fell. A maximum one day rainfall of 11.45 inches was recorded at Paterson. The total average rainfall over the Passaic River Basin was about 11.4 inches.

This flood is the maximum of record at many locations within the Passaic River watershed. The beginning of the sudden flood rise was almost simultaneous on all branches of the Passaic River at about 6:00 P.M. on Thursday, 8 October. The Pompton River

reached a maximum at 4:30 P.M. on Friday, 9 October, and continued at a high rate of discharge until noon of Saturday, 10 October. The peak on the Passaic River reached a maximum at about 12:00 P.M. at Two Bridges, 4:00 P.M. at Little Falls and 9:00 P.M. at Dundee Dam on Saturday, 10 October. The Whippany River at Whippany reached its peak at 5:00 P.M. on Friday, 9 October. On the same day the upper Passaic River at Chatham reached a maximum at midnight. Maximum discharges at various locations in the Passaic River watershed, as estimated or recorded by various agencies, are listed in Tables C-6 and C-7. The peak discharges on the Passaic River were 31,700 cfs at Little Falls and 35,800 cfs at Dundee Dam. The total volume of runoff at Dundee Dam was 6.50 inches, approximately 56 percent of the average rainfall over the basin.

13.4. Storm of 9-22 March 1936.

This extra-tropical type storm occurred over the northeastern United States, with separate centers over the Ohio River and Connecticut River watersheds. Intense rainfall on 11 and 12 March followed by heavy rain during 16 to 19 March falling on a heavy snow cover resulted in damaging floods throughout the northeastern states. The average precipitation over the Passaic River Watershed was 11.1 inches of which 6.0 inches was rainfall and 5.1 inches was the water content of the snow cover. A daily maximum rainfall of 2.58 inches was recorded at Paterson, N.J.

This was the worst winter flood that occurred between March 1902 and April 1984. The peak discharge recorded at the S.U.M Dam was 19,700 cfs on 13 March. The peak flow on the Pompton River occurred approximately 24 hours before the peak on the Passaic River. The total runoff volume was 5.99 inches, equivalent to about 67 percent of the total rainfall and accumulated snow cover (water equivalent).

13.5. Storm of 16-23 September 1938.

These storms comprised a complex series of events ending with a major hurricane. They resulted in heavy rainfall over a widespread area with centers of maximum precipitation over Connecticut and Massachusetts. The average total rainfall over the Passaic River Watershed was 7.0 inches. A maximum daily rainfall of 2.32 inches was recorded at Paterson, N.J. Isohyetals are shown on Figure C-11.

The storm of September 1938 caused severe flooding throughout the New England States. In the Passaic River Basin, the flood was moderate, with a peak flow of 8,350 cfs on the Passaic River at S.U.M Dam above Paterson on 23 September 1938. The runoff volume was 1.9 inches, equivalent to about 27 percent of the average rainfall over the basin.

13.6. Storm of 15-23 July 1945.

This extra-tropical storm consisted of six days of moderate rains followed by about 15 hours of heavy showers on 22 and 23 July. The storm centered over the eastern edge of the Passaic Basin, with maximum recorded values of 14.73 inches at Midland Park, N.J., and 14.64 inches at Suffern, N.Y. (Figure C-12). The heavy showers were localized and spotty, resulting in flash floods on many small streams in and near the Passaic River Basin. The maximum daily rainfall recorded in the vicinity was 7.60 inches at Little Falls, N.J.; 2.36 inches of rain fell at Wanaque, N.J. in the last three hours of the storm and 3.51 inches fell at Spring Valley, N.Y. in five hours after midnight of 22 July. An average of 8.5 inches of rain fell on the Passaic River Basin.

This was a major flood in the Lower Valley of the Passaic River. Flash flooding occurred on all small tributaries to the Passaic River below Two Bridges. The peak on the main stem was due to the simultaneous high flows from the short tributaries, as indicated by the shape of the flood hydrograph at the S.U.M. Dam. The first peak of 19,500 cfs, was caused primarily by the flow from the tributaries below Two Bridges while the second peak of 11,600 cfs occurred more than 24 hours later and was attributed to the flow from the Great Piece Meadows area above Two Bridges. The peak flow on the Pompton River at Pompton Plains, which has a drainage area equal to about one-half that just below Two Bridges, was estimated at 9,690 cfs and the peak time was estimated at one hour after the first peak in the lower Valley of the Passaic River at S.U.M. Dam, and 18 hours before the second peak of 11,600 cfs attributed to the Great Piece Meadows area. The runoff volume was 5.0 inches, equivalent to 59 percent of the average rainfall over the basin.

13.7. Storm of 11-16 August 1955.

Hot dry conditions over New Jersey and New York during the summer resulted in drought conditions in early August 1955. On 3 August a tropical low, which developed into Hurricane "Connie", was first detected in the Mid-Atlantic Ocean east of the Windward Islands. Moving slowly northwestward, it entered the mainland near Cape Lookout, N.C., on 12 August. The storm then moved up Chesapeake Bay, east of Washington, D.C., and northward through central Pennsylvania. The hurricane winds diminished and the storm passed over Central New York on 14 August. As a low pressure mass, "Connie" deposited 4 to 12 inches of rain over eastern Pennsylvania, southeastern New York and northern New Jersey. During this period, rainfall over the Passaic River Basin ranged from 11.48 inches to 5.64 inches with an average of over 8 inches. These rains filled many small, depleted reservoirs and saturated the soil, resulting in conditions conducive to the disastrous floods following the heavy rains of hurricane Diane which struck the area only five to six days later.

13.8. Storm of 17-20 August 1955.

Hurricane Diane, with greatly diminished winds, passed about 60 miles west of Washington, D.C. on the morning of 18 August and turned northeastward to pass between Harrisburg and Philadelphia that evening. The center moved eastward across New Jersey during the night, and on the morning of the 19th was centered a short distance south of central Long Island. From there it moved east-north-eastward between Martha's Vineyard and Nantucket. A continued inflow of tropical air and orographic lifting over the foothills of Pennsylvania and southern New England added to the storm rainfall to produce excessive rains over Pennsylvania, New York, New Jersey and New England on the 18th and 19th. The maximum daily rainfalls were 4.25 inches at Paterson, 4.92 inches at Wanaque Dam, 6.24 inches at Canistear Reservoir, and 4.84 inches at Suffern, New York. See Figure C-13. Average rainfall over the basin was 5 inches.

The storm of August 1955 was not as severe as some of the previous storms described. There was some flooding along the mainstream from Two Bridges to Paterson, but most of the flooding occurred on the tributaries, including the Ramapo, Pompton and Pequannock Rivers and Goffle and Molly Ann's Brooks. A peak flow of 9,850 cfs was recorded at S.U.M. Dam above Paterson on 20 August 1955. The runoff volume was 1.2 inches, equivalent to about 36 percent of the average rainfall over the basin.

13.9. Storm of 14-18 October 1955.

A cold front moved into eastern Pennsylvania and southern New York on the morning of 13 October and became stationary, with a coastal wave moving northward accompanied by moderate to heavy rains on the 14th and 15th of October. The center drifted slowly northward bringing abundant rains which continued in the northeast through the 16th. Concurrently, an extra-tropical storm from the Carolinas to New York brought high winds up to 57 mph on the 14th of October. These winds were accompanied by heavy rainfall extending through the 17th of October. The system finally drifted eastward and the rains ceased on the 19th. Average rainfall over the Passaic River Basin was about 5.5 inches (Figure C-14).

The storm of October 1955 caused flooding which was most severe on the Pompton, Ramapo and Wanaque Rivers, equalling or exceeding the March 1936 flood. The other tributaries were not as severely affected. Peak flow at Little Falls above Paterson was 11,600 cfs on 18 October 1955. The runoff volume was 3.01 inches, equivalent to about 55 percent of the average rainfall over the basin.

13.10. Storm of 11-12 September 1960.

Hurricane "Donna," travelling in a northeasterly direction across Long Island, lashed at the New Jersey coast on the 12th of September. As "Donna" moved northeastward off the Jersey coast, damaging winds, heavy rains and high tides struck the coastal areas of the state. Winds gusting up to 100 miles per hour swept the coast while rainfalls within the Passaic Basin was about 4.5 inches. Maximum daily rainfall totals were 4.04 inches at Plainfield, N.J. and 3.86 inches recorded at New Milford, N.J. See Figure C-15.

13.11 Storm of 27-31 May 1968, (Calibration event).

On 27 and 29 May 1968, two small, low-pressure disturbances formed in the Cape Hatteras area and moved into the Delaware Bay area. The forming of the second low pressure system extended heavy rainfall from north of New York City through the Hudson Valley and into Massachusetts. The maximum recorded precipitation of 7.96 inches in the Passaic River Basin occurred at Canoe Brook, with an average rainfall over the basin of 6.0 inches (Figure C-16). Thiessen polygons and mass curves of rainfall are shown on Figures C-17 and C-18, for this event.

The flooding of May 1968 caused widespread damage over the Passaic River Basin. Flooding occurred on the main stream and all major and most minor tributaries from the headwaters to the City of Passaic, about 12 miles upstream of the mouth. Flooding was most severe on the Pompton, Ramapo, Wanaque and Pequannock River equalling or exceeding the March 1936 flood. Peak flow at Little Falls was 13,500 cfs on 31 May 1968. The peak flow on the Pompton River at Pompton Plains occurred on 30 May 1968 and was estimated at 13,100 cfs.

13.12. Storm of 27-28 August 1971, (Calibration event).

The center of tropical storm "Doria" passed from Delaware Bay over the extreme southern portion of Cumberland County, thence northward through the central part of New Jersey in the early morning of the 28th. From New Brunswick, the storm passed over Newark, then across the Hudson River into southeastern New York State and Connecticut. Excessive rainfall occurred over the state for 24 hours just prior to "Doria". The communities hardest hit by "Doria" were Bound Brook, Manville, and Elizabeth, N.J. Three deaths by drowning were attributed to the storm. The average rainfall over the Passaic River Basin was approximately 8 inches. The total rainfall at Newark was 8.01 inches, at Mahwah, 9.25 inches and at Canoe Brook, 9.20 inches (Figure C-19). Thiessen polygons and mass curves of rainfall are shown on Figures C-20 and C-21, for this event.

The storm of August 1971 caused extensive flooding on the tributary system of the Passaic River. There was relatively little flood flow at Little Falls on the Passaic main stem in

relation to past floods; however, tributaries such as the Pompton River were flooded. The Pompton River at Pompton Plains had a peak discharge of 8,480 cfs. The Rockaway River above Boonton Reservoir had a peak flow of 3,086 cfs. On the Pompton River at Pompton Plains, the volume was 1.75 inches out of a basin average rainfall of 7.40 inches, or approximately 24%. This low percentage of runoff is accounted for by the very long dry period prior to the storm.

13.13. Storm of 6-8 November 1977, (Calibration event).

An extra-tropical storm travelled into the New Jersey coast on November 6. Strong east winds with speeds of 50 mph were experienced at the shore on November 7, then decreased on November 8. Rain began on the evening of November 6 and continued into the afternoon of November 8. Heaviest rainfall was in the northeastern part of the state, where 2 inches fell on the 7th followed by 8 inches or more on the 8th. The average rainfall over the Passaic River Basin was approximately 5.5 inches, with 9.25 inches at Newark, 6.04 inches at Mahwah and 4.10 inches at Bernardsville (Figure C-22). Thiessen polygons and mass curves of rainfall are shown on Figures C-23 and C-24, for this event.

The storm of November 1977 caused extensive flooding throughout northeastern New Jersey, particularly in Bergen County. In the Passaic River Basin, the main tributaries of the basin, which included the Saddle, Ramapo and Pompton Rivers, were hardest hit. The peak discharges for various locations include: Ramapo River at Pompton Lakes - 11,000 cfs; Pompton River at Pompton Plains - 11,000 cfs; Hohokus Brook at Hohokus - 6,600 cfs. The volume of runoff for the Third River during the peak rainfall period was equal to 1.94 inches versus a rainfall total of 3.39 inches, approximately 57% of runoff versus rainfall over this tributary's basin.

13.14. Storms of March and April 1984, (Calibration event).

The storm of 28-30 March 1984 was a northeaster of extra-tropical origin. It brought with it a major tidal surge (March 29th) and significant precipitation in the form of rain, sleet and snow. The maximum known accumulation of such precipitation (water equivalent basis) in the Passaic Basin was 3.87 inches (Little Falls). There was no snow on the ground at the onset of the storm.

The storm of 4-5 April 1984 was of extra-tropical nature arising out of a large, low-pressure system moving out of the Ohio Valley. It entered New Jersey on the 4th where it intensified towards dawn of the 5th with gale force winds and near-record rainfalls (there was no snowfall). Storm totals at the Little Falls, Charlotteburg, and Wanaque gages were, respectively, 5.34, 6.10 and 4.15 inches. Maximum 3-hour accumulations at the Little Falls and Charlotteburg gages were,

respectively 1.30 and 1.40 inches. Sub-24-hour data is unavailable for Wanaque due to the hourly gage malfunctioning. Isohyetals for this event are shown on Figure C-25 and Thiessen polygons and mass curves of rainfall are shown on Figures C-26 and C-27.

Total precipitation for the two storms, extending over a 10-day period and including rain and the water equivalent of sleet and snow, exceeded 9 inches at Little Falls, 8 inches at Mahwah and 7 inches at Charlotteburg, Oak Ridge and Canistear Reservoirs and at Greenwood Lake. For the Passaic Basin as a whole, the total was probably close to 7 inches. Snow on the ground at the start of the second storm was apparently minimal since at those few sites where observations were made, no more than traces were reported for the preceding day (3 April).

The flood of April 1984 resulted, in part, from high antecedent flows due to the precipitation in late March. At a number of gaging stations in the northern and western subareas of the Passaic Basin with 40 or more years of record, the peaks were the highest or second highest recorded (Table C-7). Estimated frequencies of the peaks ranged from approximately 25 years at Little Falls to approximately 50 years at Pompton Plains which exceeded the frequency of the rainfall event. Discharges at selected stations are given in Table C-6.

14. Mass Curves of Rainfall

Mass curves of rainfall for the storms used in the calibration process (May 1968, August 1971, November 1977 and April 1984) are shown on Figures C-18, C-21, C-24, and C-27. These data were used to calibrate and verify both the HEC-1 and UNET models for existing conditions analysis.

15. Runoff Records

There were, as of September 1995, 40 active gaging stations in the Passaic River Basin providing records of flood runoff. Fifteen of these were crest-stage gages which establish only the peak discharges of flood events. The remainder were continuous recording gages which establish not only peak discharges but also the discharge-time dependency of such events. Record lengths for the crest stage gages vary from 3 to 50 years; those for the recording gages vary from 3 to 65 years. The locations and periods of record of the various stations are given on Figure C-7. Peaks at selected stations are given in Tables C-6 and C-8. The U.S. Geological Survey publishes the data collected at these stations.

A number of gaging stations have been discontinued over the years since the program of systematic data record collection began. Where feasible, data from these gages (which are also included on Figure C-7) were used to supplement that from stations still in service.

During the course of this study additional stream gages were sought to assist in the calibration of a flood event, should a flood occur. If a flood event did not occur then these gages would collect critical data in establishing basin response and assist in project operation. In Water Year 1992, seven new gages were installed for both permanent and temporary deployment within the Passaic River Basin. Since 1992, these gages have been operated and maintained by the U.S. Geological Survey (USGS). Sites selected for long term recording include the following:

a. Passaic River at Point-No-Point, tidal area (stage only);
b. Pequannock River near the head of the project (flow and stage); and
c. Passaic River near Montville Pumping Station (in Great Piece Meadow Area) (stage only).

Also in Water Year 1992, short term gages were constructed and put in operation at:

a. Lower Pompton River near Mountain View near the Rt. 202 bridge (stage only);
b. Whippany River in Troy Meadows near I-280 (stage only);
c. Passaic River in Hatfield Swamp near Eagle Rock Avenue (stage only) and;
d. Deepavaal Brook near confluence with Green Brook (stage only).

15.1. Annual Runoff

Low flow in the Passaic River watershed is affected to a considerable extent by diversions for water supply from the Rockaway, Pequannock, Wanaque and Ramapo Rivers, Canoe Brook, and the Passaic River itself. However, the diversions do not affect flood flows. Average runoff from the 785 square-mile watershed above Paterson, N.J., is 1.56 cfs per square mile exclusive of average water supply diversions amounting to about 0.35 cfs per square mile, which brings the total corrected runoff to 1.91 cfs per square mile. This runoff is equivalent to 25.7 inches per year or 53 percent of the rainfall. Comparative runoff data for the principal gaging stations in the watershed are given in Table C-8. Seasonal variation in runoff occurs, with over 50 percent of the annual amount taking place in the months December through April.

15.2. Flood Runoff

Significant Passaic Basin floods were recorded in calendar years 1811, 1865, 1882, 1896, 1902, 1903, 1936, 1945, 1968, 1977, and 1984 (Table C-6). Little is known of the 1811 and 1865 events beyond the fact that they were respectively ranked second and fourth at Little Falls (discounting any changes that might arise from adjustments for watershed development). Of the more recent events, some were of basinwide consequence while others

were of interest only in specific localities. Descriptions of selected floods are given above (see Past Storms and Flood Events) for the major storm and flood events affecting the Passaic Basin.

Flood runoff from the Passaic Basin is affected by the existing topographic, hydraulic, man-made and natural features. The large existing natural storage areas residing in the Central Basin delay and reduce peak flows entering into the Lower Valley. Generally, the early peaking Pompton River and Upper Passaic Rivers join together at widely different times and thus do not, under normal conditions, peak together.

Considering the variable flood conditions within the basin, the unsteady flow characteristic experienced within the natural storage areas of the Central Basin and the tidally influenced reaches of the Lower Passaic River, previous modeling was especially difficult using the traditional tools available (HEC-1/HEC-2). A simulation program was therefore sought capable of developing one comprehensive basin wide model which could accommodate the noted phenomena.

B. Passaic River Basin Modeling

16. Basin Subdivision

The separating of the basin into individual subbasins was based on several factors. With the overall goal of simulating flood runoff from the complete 935 square mile watershed, the following factors were considered when identifying individual subbasin outlets:

(1) Confluences of major streams and/or rivers;

(2) Locations of existing or proposed hydraulic control features (ie - dams, falls, diversion structures, proposed tunnel inlets, etc.);

(3) Stream gage locations;

(4) Proposed locations for project elements;

(5) Damage area locations;

(6) Natural storage area locations; and

(7) Development of one comprehensive basin wide model.

To adequately address these items, the basin was divided into separable boundary condition basins and ultimately 189 smaller subbasins. The subbasins varied in size from 0.46 sq. mi to 50.9 sq. mi., as can be seen on Figure C-28. To develop the comprehensive basin wide model in a manageable way, a systematic

approach was used that would maintain the logic of the basin response. The major subdivisions and upstream boundary condition locations used in the UNET model were as follows:

Location	Drainage Area Sq.Mi.	Figure #
1. Passaic River at Chatham, NJ (@ USGS gage)	100.00	--- 29
2. Whippany River d/s of Morristown, NJ	38.42	--- 30
3. Rockaway River d/s of Boonton Reservoir	120.76	--- 31
4. Ramapo River at Pompton Lakes (@ USGS gage)	160.00	--- 32
5. Pequannock River at Mouth	85.05	--- 33
6. Wanaque River d/s of Wanaque Reservoir	98.05	--- 33
7. Pinch Brook above Black Brook	2.19	--- 34
8. Saddle River at Lodi, NJ (@ USGS gage)	54.60	--- 35
	659.07	

All of the above basins were developed with traditional HEC-1 modeling techniques. These basins would then produce upstream boundary condition hydrographs for input into the UNET unsteady flow hydraulic model by linking with HEC-DSS. Network diagrams for the various basins are noted above.

The final basin consisted of the common UNET/HEC-1 modeled area. This basin consisted of the remaining incremental subbasins below each of the boundary condition locations. Due to the large size of this area, it is shown on two network diagrams, Figures C-34 and C-36. It should be noted that the lower area also contains the downstream boundary location within Newark Bay.

Figure C-28 also shows the main basins and their respective subareas. The physiographic data for each of the subareas is shown in Table C-9.

In addition to physiographic characteristics of the basin, there are many man-made influences that impact the existing conditions hydrology. In general the three main types of man-made impacts are existing reservoirs, water supply diversions and urbanization. There are 10 reservoirs in the hydrologic models with a potential for having a downstream impact in reducing flood waves. The reservoirs are: Splitrock and Boonton Reservoirs in the Rockaway River Subbasin; Canistear, Oak Ridge, Clinton, Charlotteburg and Echo Lake Reservoirs in the Pequannock River Subbasin; and Greenwood Lake, Monksville and Wanaque Reservoir in the Wanaque River Subbasin. The analysis showed only the Boonton and Wanaque reservoirs have any significant downstream attenuation effects. The remaining reservoirs have either too little storage or are located on tributary headwaters well upstream of the main river system. Water supply diversions have little to no effect when calibrating major historic flood peaks (except diversion contributes cumulatively to reservoir draw-downs) or reproducing hypothetical storms (reservoirs assumed full). Urbanization impacts will be further discussed later in the report.

17. HYDPAR Analysis

The previously developed 1978 Data Bank, as constructed for the Phase I GDM, was reviewed, modified and edited for actual 1990 land use conditions (Field 7) and updated 2050 projections for future conditions land use classification. In addition, Field 12 was edited to account for simulated development within the 5,350 areas of Preservation of Natural Storage Areas. Actual 1990 land use values were developed by reviewing current aerial photography of the basin on a cell by cell (in excess of 58,900) basis and adjusting individual grid cells to account for the actual land use changes. The updating of the data bank was considered essential in developing a model capable of reproducing the Passaic basin's existing hydrologic and hydraulic response. The revised data bank was used in calculating the actual 1990 percent impervious and curve number for each of the individual subbasins.

Estimated land use projections for future conditions (2050) were also reviewed and modified based on existing development, 1980 and 1990 employment and population projections, and a review of municipal master plans. In some instances projected development was modified to reflect the current trends, since intensification or changes were no longer considered practical or desirable by local authorities.

To assist in the determination of the percent impervious and curve number within each subbasin, this study utilized the spatial data management and analytic programs available from the Hydrologic Engineering Center (HEC). The programs used could manipulate the spatial data base and were: BANK, RIA and HYDPAR. Each program performs a specialized function that links to the grid cell data base. All spatial data for the Passaic River Basin has been incorporated into one data base file as depicted on the sketch shown on the next page.

The processing interface element of the data base file is comprised of computer programs that compile and reformat geographic and resource data retrieved from the data bank into a format usable by the analytical computer programs. HYDPAR links the data base file to the flood hazard hydrologic soil group and land use to generate the modeling parameters required to simulate storm runoff. These parameters are then passed on to the HEC1 model for rainfall-runoff transformation.

The analytic element of the spatial programs is used to perform the detailed technical assessments that compute the requested parameters and can eventually be used to compare the existing condition to future projected conditions.

17.1. Data Bank Grid Cell Development

The data bank was modified from the Phase I GDM to include a new variable, the actual 1990 land use conditions and the replacement of the projected 2050 future conditions. Actual 1990 land use values are now stored in field seven (7) of the data bank with the actual 1978 land use data moved outward into field fifteen (also known as variable number 15). Field 11 now contains the most likely expected projections for future land use conditions as projected in 1992 and Field 12 includes the hypothetical development of the preservation lands. Field 12 is a simulated study which will be described in greater detail in subsequent paragraphs. As revised the new grid cell data base consists of 15 variables that have been stored in the multivariable data base as follows:

Variable #		Variable Description
1		Row (Grid cell location)
2		Column (Grid cell location)
3		Subbasin Number
4		Topographic Average Elevation
5		SCS Soils Type
6		Hydrologic Soil Group Number
7		Actual 1990 Land Use
8		Slope
9		Municipal Boundaries
10		1990 Land Use (as projected in 1978)
11		2050 Land Use (as projected in 1992)
12		2050 Land Use w/ Preservation Lands Developed (as projected in 1994) 13
13		Tributary
14		Index Flood Elevation
15		Actual 1978 Land Use

----------DATABANK FILENAME = PH2FUT94.DB----------

17.2. Modification to the Phase I GDM Subbasins

Review of the Phase I GDM delineated subbasins as developed on USGS quadrangle sheets indicated that modifications could be made to enhance HEC-1's ability for model response. This review resulted in the alteration, redefining, creating or subdividing 20 individual subbasins.

The majority of the subbasins were modified to improve the basin response within HEC-1 and were altered accordingly. However, three subbasins were modified in the HYDPAR analysis only, for the sole purpose of more closely agreeing with the HEC-1 subbasin delineated shapes and drainage areas. In all cases, subbasins were modified within HYDPAR by moving individual grid cells from one subbasin to a redefined subbasin. The HEC computer program BANK was used to edit the grid cell subbasin variable, such that computations performed for percent of impervious and curve number would accurately reflect the aerial coverage of that particular subbasin. In other words, the HEC-1 subbasin shape and size was as closely matched in the HYDPAR data bank as possible. It should be noted that each grid cell represents 10.33 acres.

The following table highlights the changes made to the individual subbasins that were altered:

	Subbasin #		Phase I GDM Drainage Area (sq. mi.)	Phase II GDM Drainage Area (sq. mi.)
1	220		0.41	1.65
2	230		6.42	5.18
3	370		4.88	2.19
4	373		0.58	3.27
5	520		10.40	5.72
6	526		----	2.49
7	528		----	2.19
8	600		0.62	1.17
9	610		2.27	1.72
10	825		0.18	----
11	870		12.84	13.02
12	1150		2.30	1.70
13	1155		0.34	0.94
14	1270		0.72	----
15	1330		4.31	5.03
16	1400	**		
17	1417	**		
18	1420	**		
19	1520		2.28	2.69
20	1525		0.41	----

** These subbasins were not altered in the HEC-1 analysis but were redefined in the HYDPAR analysis by transferring grid cells between subbasins, thereby more accurately representing the HEC-1 drainage areas.

17.3. Results - 1978, 1990 and Projected 2050 Land Use

The analysis of data in the multivariable file were performed with the HYDPAR program. The data bank grid cells were utilized to model the actual 1990 land use conditions for input into HYDPAR. The HYDPAR analysis could then be performed to both the percent of impervious and curve number within each subbasin.

The parameters generated were those required for the U.S. Soil Conservation Service (SCS) curve number technique and those needed for watershed modeling using subarea impervious and characteristics for the Snyder unit hydrograph procedure.

17.3.1. Percent Impervious Area

Grid cells were matched with one of thirteen land use classes identified prior to the construction of the data bank. Each of the Passaic Basin grid cells were coded with the closest land use category that would most accurately represent the 1978 and 1990 field conditions, as noted on the December 1989/January 1990 aerial photography. Future 2050 condition land use categories were based on actual development between 1978 to 1990 and review of individual municipal master plans for future development. HYDPAR was then used to calculate the subbasin percent imperviousness by using a weighted average of all the grid cells within each subbasin and dividing by the total number of grid cells within that subbasin. Results of this analysis are shown on Table C-10 for both the actual land use conditions in 1978 and 1990 as well as the projected 2050 values.

17.3.2. Curve Number Analysis

Each grid cell within the data bank had the hydrologic soil group identified for use in the determination of the subbasin weighted curve number. With the hydrologic soil group and land use condition known for each grid cell, standard SCS procedures were used to determine the curve number for each grid cell. HYDPAR was used to calculate the weighted average curve number based on all grid cells within the subbasin. Results of this analysis are given in Table C-11 that shows the actual land use conditions in 1978 and 1990 as well as the preliminary projected 2050 values.

18. HEC-1/UNET Modeling Procedure and Analysis

18.1. Introduction

Hydrologic and hydraulic models of the Passaic River Basin developed in the Phase I GDM, relied on the ability of HEC-1 and HEC-2 to adequately portray storage-outflow relationships. With a vast amount of the Central Basin in an undeveloped and predominantly wetland state, modeling the basin response with a storage-discharge relationship required the execution of hundreds

of HEC-2 runs over a range of flows for each storm duration. Internal diversions within the HEC1 model were also required, in order to depict the storage effects of this wetland area.

To account for the large overbank storage areas and slow moving Passaic channel waters of the Central Basin, the model must be capable of reproducing flows and stages over large geographical regions and time periods. Because of the complex nature of the Basin a new modeling tool was sought that would eliminate the need for repetitive HEC-1 and HEC-2 models. The new model should be capable of producing results in one model simulation, thereby replacing the multitude of steady-state computations previously required. Also with over 800 cross-sections and 90 river miles modeled in the Phase I study, a model that could use portions or all of this data was desirable.

After review of available steady state and unsteady flow numerical models, the UNET computer program was selected for use during the Phase II GDM effort. The UNET model was developed by Dr. Robert L. Barkau, a former employee of the St. Louis District and the Hydrologic Engineering Center. It has been in use, and in various stages of development, for approximately 10 years.

18.2. Basic Theory - HEC-1 and UNET

The HEC-1/UNET interface has combined the hydrologic and hydraulic functions that for the Phase I study were separate and quite distinct in their development and execution. The total runoff response to rainfall is captured and the result produces a discharge-stage-frequency relationship from one basin-wide model.

UNET is a one-dimensional unsteady flow model which computes flow through a full stream network. The system of open channels may be connected at their downstream or upstream ends. The flow distribution depends on the stage and conveyance characteristics of the downstream channel. UNET determines the flow distribution of two or more streams at their junction. Flow reversals are also determined in the model as a function of downstream stage. The solution to the continuity and momentum equations between each section is accomplished with the use of a four-point implicit, finite difference scheme. A new set of equations for the channel and overbanks are defined that are representative of the channel geometry.

HEC-1 is designed to simulate the surface runoff response of a river basin to rainfall by representing the basin as a interconnected system of hydrologic and hydraulic components. Each component models an aspect of the precipitation-runoff process within a portion of the basin, commonly referred to as a subbasin. A component may be a surface runoff entity, a stream channel, or a reservoir. The components are represented by a set of parameters which specify the particular characteristics of

that component and relations that describe the physical processes. The resulting model computes streamflow hydrographs at desired locations in the river basin.

18.3. HEC-1 Model Development

The HEC-1 model used in the Phase I study was restructured and updated to provide the necessary information to UNET. New subbasins were delineated where necessary and curve numbers were developed for actual 1990 land use. All the HEC-1 models were tied together to form one basin-wide model with ZW cards directing hydrographs to be written to HEC-DSS at various locations. HEC-1 then develops specified hydrographs for boundary conditions, point inflows and uniform lateral inflows. Hydrograph diversion schemes from the Phase I study were removed along with storage-outflow relationships that were in subbasins downstream of the UNET boundary condition locations. The HEC-1 model was also imported from a Harris version to the IBM PC version. For the Phase II GDM, the HEC-1 model consisted of 189 subbasins that range in size from 0.46 to 50.9 square miles. Precipitation is distributed throughout the basin so that the runoff generated by each subbasin tributary to the location of interest is consistent with the runoff contributed by the other subbasins. Consistency of successive downstream hydrographs is maintained in HEC-1 by generating each hydrograph from rainfall quantities that correspond to a specific subbasin size and a precipitation depth-drainage area relationship. Approximately 660 square miles or 70.5 percent of the Passaic Basin was modeled using traditional HEC-1 techniques. Flows from these areas were entered into UNET as an upstream boundary condition hydrograph.

18.4. UNET Model Development

The UNET model required the reconfiguration of some 800 cross sections from the HEC-2 backwater format to the forewater format for unsteady flow modeling and all cross section geometry was modified for the UNET code. The model was prepared to accept boundary condition hydrographs from HEC-DSS as well as point and lateral inflow hydrographs. The model as configured for without project conditions consists of over 1000 cross sections, 118 bridges, 95 miles of rivers and subdivided into 19 separate reaches. The improved conditions model increased to nearly 1300 cross-sections in 22 reaches. Sixty miles of the Passaic River main stem forms the basic model, with all significant tributaries modeled as individual reaches. The upstream boundary condition at each reach is defined as a stage-discharge hydrograph from HEC-DSS or internally determined from interactions with upstream reaches. UNET represents one comprehensive basin model for the Passaic River and all tributaries downstream of the boundary locations.

18.5. Interrelationship and Linking of Models

HEC-1 hydrograph data drives the UNET model. With the hydrograph data stored in HEC-DSS for the scenario and storm event under analysis, the HEC-1 hydrographs are retrieved into UNET as either a boundary condition, a point inflow or as a uniform lateral inflow. The data is used to energize UNET as it accesses the boundary conditions and the point and lateral inflows to predict downstream stages and discharges for the desired rainfall events. Figure C-37 depicts how the basin's subareas are defined between the two models.

18.6. Reach Definitions

The UNET model for existing without project condition was configured into 19 separate reaches. The upstream boundary condition at each reach is defined as either an imported stage or discharge hydrograph from HEC-DSS along the outer limits of the model or is internally determined from interactions with upstream model reaches. Boundary condition hydrographs are developed at the upstream limit of the UNET model for each major river under analysis. See the previous section entitled "Basin Subdivision", for a discussion on the upstream boundary locations.

Point inflow hydrographs were also developed using traditional HEC-1 techniques. These hydrographs were placed within the internal reaches of the UNET model and are specified at unique stations within the corresponding river reach. Point inflow can represent a single subbasin or it can be the result of runoff from multiple basins, such as a small tributary. UNET computations are altered by this incoming flow and the point inflow is accounted for within the numerical model for the reach.

Uniform lateral flows are developed from the HEC-1 subbasins that form the incremental area through which the UNET stream reaches flow. The HEC-1 inflow is displaced laterally between user defined cross sections. These sections define the upstream and downstream limits of the subbasin and UNET uniformly distributes the flow between these limits. Table C-12 outlines the 19 reaches and the upstream and downstream limits of each reach. Figure C-38 shows how the UNET river reaches and the off-channel storage areas (S1 through S4) are connected to form the complete UNET basin model, for existing and future unimproved conditions.

18.7. Model Time Step Selection

The HEC-1 model time step selected is one hour. This time step is adequate to define the exterior hydrograph peaks pertinent to the calibration and hypothetical floods of the modeled rivers. However, this time step would not be adequate to design the interior drainage facilities for the many components. For the basin wide development of exterior stage hydrographs and peaks, the one hour time step is considered reasonable and the

UNET model accepts the HEC-1 one hour flood hydrographs as input. UNET runs on a one hour time step until conditions exist that require a reach to be subdivided. When this occurs, the model selects an appropriate smaller time step in order to complete the computation.

18.8. Cross-Section Surveys

The Passaic Basin UNET model is configured with over 1000 cross sections and 118 bridges. Approximately 200 of these cross sections were recently surveyed. Table C-13 details the number of cross sections per reach. In general, the distances between cross sections average about 500 feet. However, in some river reaches such as the Great Piece Meadows, Black Brook (located in the lower Rockaway River) and Hatfield Swamp, the distance between cross-sections is greater than 1000 feet. The density of sections in these areas is considered adequate due to the lack of variation in the physical, topographical and hydraulic characteristics of the river and overbanks.

Near the mouth of the Passaic, a comparison of the sections from the previous HEC-2 models and the new surveys revealed that a some shoaling has occurred. The comparison also revealed that in portions of the channel downstream of Dundee Dam, some degradation has also occurred. Therefore, invert elevations were adjusted to the recent surveys, otherwise instabilities would result in the computation of stages and the translation of discharges.

A further review of cross sections both upstream and downstream of Dundee Dam was made and cross section comparisons are shown on Figures C-39a to C-39g. Based on these comparisons and investigations of cross sections at other locations, small changes in channel geometry are evident. However, the extent of this shoaling and erosion is insignificant in the range of floods investigated.

18.9. Internal Rating Curves For UNET

Internal rating areas where developed and coded into the UNET model at 3 hydraulic control structures. There rating curve locations where: Beatties Dam at Passaic River Station 1568+65 (Figure C-39h), SUM Dam at Passaic River Station 1338+85 (Figure C-39l) and Dundee Dam at Passaic River Station 912+50 Figure C-39j. These rating curves where used for all UNET model analysis including existing, future unimproved and future improved conditions.

18.10. Bridge Modeling

There are two procedures for modeling bridges with UNET and the Passaic model incorporates both approaches. Bridges are input in the cross section file and the model treats them as

interior boundary conditions. Flow is assumed to be the same at the headwater and tailwater sections and a change in water surface elevations between them is computed.

The normal bridge routine in UNET is similar to the HEC-2 normal bridge routine. This method is best suited for bridges without piers, bridges under high submergence and perched bridges. The cross section at the location of the bridge is handled just as any other cross section with the exception that the area of the bridge below the water surface is subtracted from the total flow area and the wetted perimeter is increased as needed to include the piers and low chord.

The special bridge routine in UNET differs slightly from the HEC-2 method. Special bridges should be used to consider the effects of pressure flow and weir flow over the top of the road. For low flow calculations UNET does not require the bridge opening to be approximated as a trapezoid but instead uses the actual ground profile and bridge geometry to determine the available flow area. The weir equation is used to calculate flow over the top of the roadway. Either weir lengths and elevations or the actual road profile can be used to model the weir flow. The CSECT program uses this data to create a family of free and submerged rating curves to be used by UNET at the bridge crossing.

There are 118 bridges coded in the Passaic model. Special bridges make up 42% of these and the remainder are normal bridges. New field surveys were taken on 45 of the bridges to be modeled. Field measurements including tape downs from the low chord to the channel bottom were taken on many of the other bridges. This field data, along with copies of bridge plans and inspection reports from the state highway departments and the old HEC-2 models provided the database from which the bridges were modeled in UNET. Five bridges downstream of Dundee Dam which were formerly not included in the model were again not modeled. These bridges were considered "high" bridges since the low chords of the bridge decks were sufficiently high enough to be above any anticipated flood crest. Thus, the bridge superstructure would have a negligible impact on the water surface profiles. Field surveys were not performed on these bridges. Table C-14 lists pertinent data for the all bridges modeled in this effort.

18.11. Bridge Review and Investigations

As part of the bridge modeling effort, an investigation was performed to determine the construction dates and obstruction potential of the bridges within project areas. Construction dates were considered important since over 95 miles have been modeled with 4 calibration events dating back to May 1968. Localized highwater mark matching for the historical events (at or near specific bridges) could be missed due to bridge modifications/ removal and/or replacements through time. In other words, bridges that were in place at the time of the flood

event may have been altered or removed and the new bridge would represent different hydraulic conditions. The following paragraphs will discuss the bridge investigations that were performed in more detail.

Twenty-eight bridges in the proposed project areas were investigated to determine the construction date and obstruction potential. Of these twenty-eight bridges, nineteen bridges were determined to be in existence in their present form for all calibrated flood events (May 1968, August 1971, November 1977, and April 1984). Fifteen of these nineteen bridges showed no unusual circumstances to further review either sediment buildup or debris potential. Of the four remaining pre-1968 bridges, two were closed to vehicular traffic during the June 1992 investigation (possible structural modifications may be considered), while two other bridges were noted to have debris potential, as existing debris was noted under the bridge. These four bridges will be further reviewed in subsequent studies to ascertain any changes made to the bridge decks or channel bottom.

Of the remaining nine bridges of the original twenty-eight investigated, eight bridges were found to be newly constructed and were measured for length and depth of their cross-sections including piers, if any. The Route 23 bridge over the Pompton River and the Vail Road bridge over the Rockaway River had only their decks reconstructed (their substructures were kept intact). The table below summarizes this information.

The final bridge of twenty-eight bridges investigated resulted in the determination that it had been totally removed approximately six years ago (June 1989). See summary below:

28 bridges studied:

- 19 bridges predate 1968 flood event
- 8 bridges have been reconstructed:

Reconstructed Bridge	Town	River	Reconstruction Date
1. Midland Ave.	Garfield	Saddle	1987
2. Saddle River Rd.	Garfield	Saddle	1977
3. Ackerman Ave.	Clifton	Passaic	1989
4. Route 23	Pequannock	Pompton	1988
5. Pompton Plains Rd.	Pequannock	Pompton	1976
6. Wanaque Ave.	Pompton Lakes	Wanaque	1986
7. Vail Road	Pine Brook	Rockaway	1989
. Maple Ave. (Wagraw Bridge)	Paterson	Passaic	1992

- 1 completely removed: (approximately June 1989)

1. Hillman St.	Paterson	Passaic

In addition to the review of bridges in project areas, it was noted that the Lake Hiawatha Dam (located on the Upper Rockaway River just downstream of the Boonton Reservoir) was found to have been completely removed prior to the April 1984 flood event. However, township representatives noted that the dam was in existence during the May 1968 flood event.

19. HEC-1/UNET Model Calibrations - Historical Events

19.1. Historical Rainfall Records

Rainfall gages used to determine isohyetal maps for the calibration events were both recording and non-recording gages. The gages used for each storm varied by event. The storms of May 1968, August 1971, November 1977 and April 1984 were used in the model calibration and verification process. The total event rainfall for each subbasin was determined by reviewing the isohyetal maps as shown in Figures C-16, C-19, C-22 and C-25. The recording gages and the Theissen polygons used to determine the spatial and temporal distribution patterns for the historical rainfall events are shown on Figures C-17, C-20, C-23, and C-26. Mass curves of rainfall for the above noted storms are displayed graphically on Figures C-18, C-21, C-24 and C-27.

19.2. Recession Parameters

The recession parameters, QRSCN and RTIOR, were determined from the observed hydrographs at the stream flow gages for the calibration storms. For each observed hydrograph, the ratio of QRCSN to the peak flow was computed for application to ungaged subareas. QRSCN was specified as a ratio of the peak flow when the falling limb discharge drops to 10 percent of the calculated peak discharge. The initial recession values of the ungaged subbasins were set equal to the values computed from the measured hydrographs at the gages. The recession parameter values for the ungaged subareas were revised during the calibration based on the characteristics of the area and the observed hydrographs. RTIOR, the ratio of a recession limb flow to the recession flow occurring one hour later, represents the decay rate of the hydrographs.

19.3. Flood Routing Procedures

The routing procedures selected for the HEC-1 model were a function of the availability of data. The Modified Puls method was used in routing reaches where storage-discharge relationships were available. Modified Puls was also used for all reservoir routings. The Muskingum method was used primarily in the upstream reaches where cross section data were not readily available and flood hydrograph translation and attenuation was sought. Within the UNET hydraulic model, the unsteady flow equation for the channel and floodplain was derived using the four-point implicit finite difference scheme. Under this scheme, space derivatives and function values are evaluated at an

interior point. For a reach of river, a system of simultaneous equations results. The simultaneous solution is an important aspect of this scheme since it allows information from the entire reach to influence the solution at any one point.

The translation and attenuation of the flood hydrographs was accomplished with three distinct methodologies each a function of the basin characteristics and data available to model the basin response. Without an understanding of flood wave routing in the Central Basin, one would expect peak discharges occurring at Beattie's Dam to be larger than those on the Pompton River at Pompton Plains since the difference in drainage area is so great (762 vs 355 sq. mi.). However, the 100-year statistical peak at Pompton Plains thru Water Year 1994 was 28,600 cfs while the corresponding statistical peak at Little Falls was 25,400 cfs. The peak for the larger Passaic watershed is significantly lower than that of the smaller Pompton basin. It should be noted that flood peaks on the Pompton River at its confluence with the Passaic River generally occur 24 to 40 hours prior to the Passaic River peak during basin-wide floods of substantial magnitude. A graphical display of this during the April 1984 event is shown in Figure C-40.

This phenomenon occurs because downstream of the Passaic and Pompton River confluence (known as Two Bridges), the Passaic River channel is very constricted, in part because of the presence of a narrow gorge and Beattie's Dam. This constraint tends to limit the flow out of the Central Basin area to the Lower Valley, and creates a backwater effect which influences flows upstream. This backwater causes an energy imbalance which forces a portion of the Pompton River flow to reverse and flow upstream along the Passaic River into the Central Basin floodplain, as well as downstream towards Beattie's Dam. The resulting combined flow from the total watershed above Two Bridges begins to fill the natural storage in Great Piece Meadow and Bog and Vly Meadow. In addition to the initial flow reversal, there is a second phase to this phenomenon when the peak stage along the Passaic River upstream of Two Bridges occurs 6-12 hours following the Pompton River peak discharge at Two Bridges. This peak stage results from the flow coming down the Passaic combining with the reversed flow of the Pompton River. This combination produces a higher stage along the Passaic and the lower Pompton Rivers than the individual peaks of the two streams. This is due to the decreasing availability of storage in the Great Piece Meadow and the high coincidental flows on the Pompton River. Thus, the discharge hydrograph in the vicinity of Two Bridges has two distinct peaks, while the stage hydrograph continues to rise in a single peak through the entire event. This flow reversal and its effect on stages and discharges in the Central Basin is one major reason for the use of an unsteady flow model to accurately simulate the behavior of flows in the area.

19.4. Calibration Hydrographs

To adequately calibrate the individual subbasin models to historical flood events, seven gaged locations have been modeled at the downstream limit of the respective subbasins. Their outlets are located at a USGS stream gaging station. The stations are:

1 - Passaic River at Chatham,
2 - Whippany River at Morristown,
3 - Rockaway River above Boonton Reservoir,
4 - Ramapo River at Pompton Lakes,
5 - Pompton River at Pompton Plains,
6 - Passaic River at Little Falls, and
7 - Saddle River at Lodi.

The computed hydrographs for the May 1968, November 1977 and April 1984 events were compared to the observed hydrographs at locations where gage data was available. The following paragraphs will summarize the calibration efforts.

Figures C-41a thru C-42p show the results of the calibration effort. The comparison hydrographs were evaluated in terms of peak flow, time to peak and the resultant runoff volume. Tables C-15a thru C-15c show the relative comparison of these criteria at all locations for which observed data were available. Peak flows were adjusted by the modification of curve numbers (CN). Time to peak and runoff volume were adjusted by adjusting the routing criteria and/or the Clark's "R" value to better approximate the observed event. All gage locations were not calibrated for each historical event because not all recording gages had complete streamflow data for each of the historical storms investigated. Some tributary areas were partially influenced by reservoirs and an attempt was made to start the simulation with the reservoirs at their historical stage at the beginning of a given flood event. There were some inconsistencies in the historic data which could not be explained and therefore, not all data could be used to accurately determine subbasin parameters.

The Passaic River Basin was calibrated with Clark unitgraph parameters determined from the Phase I study and curve numbers (CN) from the 1978 and 1990 land use data utilized by the HYDPAR program. The 1978 land use data was used for the 1968, 1971, and 1977 events, while the 1990 land use data was used for the 1984 event. As demonstrated in Table C-11, the curve numbers did not change greatly over that time period. The initial Tc and R values used in this study were based on the Phase I GDM study. See Figures C-43 and C-44. Based on the optimization results, the parameters were adjusted to reconstitute the historical events investigated. Adopted values of unitgraph parameters are shown in Table C-9. The results of the three (3) storm calibrations indicated that the model reproduced observed values within acceptable limits for the 1968, 1977 and 1984 historical

floods. The August 1971 flood was eliminated as a calibration flood event due to a lack of adequate basin-wide data. A more detailed description of the calibration and verification process is described in the following paragraphs.

19.4.1. November 1977

Saddle River at Lodi (November 1977). The parameters used in the reproduction of the November 1977 flood event on Saddle River at Lodi USGS stream gage were a combination of Clark unitgraph parameters and SCS loss rates. Adjustments to these parameters were necessary to reconstitute the historical events, and were made based on the hydrologic and physiographic characteristics of the basins affected. Tc ranged from 7.4 to 1.0 and R from 3.1 to 2.0. Adopted values of unitgraph parameters are shown in Table C-9. Table C-11 shows the runoff curve numbers used in the calibrations. Figures C-41a thru C-41e graphically illustrate the calibration effort for this event. The hydrograph at Lodi was input to UNET as a boundary condition. The computed peak discharge was 5230 cfs on November 9. The peak difference is 16 percent, the volume difference 4 percent and the timing difference is 2 hours. See Figure C-41e. Table C-15a shows the results of the calibration.

19.4.2. April 1984

Passaic River at Chatham (April 1984). The parameters used in the reproduction of the April 1984 flood event at the Passaic River at Chatham USGS stream gage were a combination of Clark unitgraph parameters and SCS loss rates. Adjustments to these parameters were necessary to reconstitute the historical events. Adjustments were made based on the hydrologic and physiographic characteristics of the basins affected. Tc ranged from 6.0 to 1.3 and R from 17.7 to 3.0. Adopted values of unitgraph parameters are shown in Table C-9. Table C-11 shows the runoff curve numbers used in the calibrations. The hydrograph at Chatham was input to UNET as a boundary condition. The computed peak discharge was 2000 cfs on April 5 . The peak difference of 3 percent and the volume difference of 2 percent are indicative of the model reliability. While the time of the precise peak appears to be in error, the broad crested hydrograph is reproduced accurately. Figure C-42b shows the reconstituted hydrograph. Table C-15b shows the results of the calibration.

Whippany at Morristown (April 1984). The same procedure as described above in the preceding sections were used to calibrate the Clark's Tc and R as well as the SCS Curve Number parameters. Tc varied from 5.0 to 1.5 and R from 25.0 to 5.4. These values are shown on Table C-9. Curve numbers used to simulate the April 1984 event ranged from 84 to 94. This reflects a Type III antecedent moisture condition. These curve numbers are shown in Table 11. Figure C-42c shows the reconstituted hydrograph. The

peak computed flow was 1710 cfs, 2% above the observed peak. Computed volume was 3% high with the timing of the computed hydrograph approximately 3 hours later than the observed.

Rockaway River Subbasin (April 1984). The parameters used in the reproduction of the April 1984 flood event at the Rockaway River below Boonton Reservoir USGS stream gage were a combination of Clark unitgraph parameters and SCS loss rates. The initial values of Tc and R were based on optimization results from the Phase I study. Adjustments to these parameters were necessary to reconstitute the historical events. Adjustments were made based on the hydrologic and physiographic characteristics of the basins affected curve numbers ranged from 81 to 92. Tc ranged from 4.0 to 1.0 and R from 22.5 to 7.0. Adopted values of unitgraph parameters are shown in Table C-9. Table C-11 shows the runoff curve numbers used in the calibrations. The hydrograph below Boonton was input to UNET as a boundary condition. The computed peak discharge was 4690 cfs on April 6. The peak difference of 10 percent and the timing difference of 6 hours are indicative of the model reliability. Table C-15b shows the results of the calibration. Figure C-42e shows the reconstituted hydrograph. For the Rockaway River above Boonton Reservoir, the peak computed flow was 5470 cfs, or the same as the observed peak. Computed volume was -2% from the observed while the HEC-1 derived peak was 3 hours late.

Ramapo River Basin Calibration (April 1984). The parameters used in the reproduction of the April 1984 flood event at the Ramapo River at Pompton Lakes USGS stream gage were a combination of Clark unitgraph parameters and SCS loss rates. The initial values of Tc and R were based on optimization results from the Phase I study. Adjustments to these parameters were necessary to reconstitute the historical events. Adjustments were made based on the hydrologic and physiographic characteristics of the basins affected. Tc ranged from 11.5 to 1.0 and R from 14.2 to 3.4. Adopted values of unitgraph parameters are shown in Table C-9. Table C-11 shows the runoff curve numbers used in the calibrations. The hydrograph at Pompton Lakes was input to UNET as a boundary condition. The computed peak discharge was 14,100 cfs on April 5. The peak difference of -8 percent and the timing difference of 2.5 hours are indicative of the model reliability. Table C-15b shows the results of the calibration. Figure C-42k shows the reconstituted hydrograph.

Pompton River at Pompton Plains (April 1984). Since the Ramapo River Basin is a subarea of the Pompton River Basin, and the Ramapo portion of the model was already calibrated, the Pequannock and Wanaque Rivers portion of the model parameters were adjusted to complete the calibration process at the Pompton Plains gage. Tc varied from 4.4 to 1.2 while R ranged from 27.8 to 4.5 for the Pequannock and Wanaque subareas of the Pompton Basin. SCS curve numbers, which are based on Antecedent Moisture

Condition III, ranged from 83 to 95. The SCS curve numbers and Clark unit hydrograph parameters are shown on Tables C-11 and C-9, respectively. Figure C-42l shows the reconstituted hydrograph.

Passaic River at Little Falls (April 1984). The drainage basin at Little Falls includes the previously calibrated areas of Passaic at Chatham, Rockaway above Boonton Reservoir, Whippany at Morristown and Pompton River at Pompton Plains. The remaining area to be calibrated was predominantly the Central Basin which has a unique physiographic configuration. The region contains large wetland areas, which made the use of an unsteady model an important component. Calibration of this area was achieved by adjusting both the hydrologic parameters of the HEC-1 rainfall-runoff model and the wetland storage area relationship of the UNET model. Figure C-42m shows the reconstituted hydrograph.

19.4.3. May 1968 - Calibration Verification

Figures C-45a to C-45j show the verification hydrographs as developed with HEC-1 and UNET for the May 1968 flood event.

The flood of May 29 through June 4, 1968 was used to verify the results of the calibration on a basin-wide storm event. Clark's Tc and R coefficients remained the same as for the other events. Adjustments to the curve numbers were made to account for the difference in land use cover between 1968 and 1984. Curve number adjustments were also made to account for differing antecedent moisture conditions. A comparison of these curve numbers is shown on Table C-11. Hydrographs peaks, time of peaks and runoff volumes are compared in Table C-15c. The May 1968 flood simulation produced excellent replication of the gaged hydrographs on the upper Passaic, Rockaway, upper Wanaque and Saddle Rivers, and adequate replications at the other gaged sites. These figures demonstrate that the HEC-1/UNET model simulation is a reasonable approximation of the actual Passaic Basin response to significant rainfall events, and can be used to predict changes in flood frequency due to the proposed improvements.

19.5. Profile Calibrations

In order to compute and predict the flood stages for future hypothetical events and the design storms for the project, the UNET model was calibrated to reproduce both flow and stage hydrographs recorded from significant historical events. The historical database used for calibration consists of gage records for the three gages within the UNET model. Those gages were the Pompton River at Pompton Plains, Passaic River at Pine Brook and Passaic River at Little Falls. Surveyed high water marks along the main stem of the Passaic River and modeled tributaries provided the basis for the calibration comparisons.

19.5.1. Channel Roughness "n" Values

The roughness coefficients (Manning's "n" values) used in the hydraulic calibration of the 19 reaches modeling the Passaic River system are based on engineering judgement of field conditions and an understanding of the values recommended for different channel and overbank areas as listed in hydraulic text such as Chow 's "Open Channel Hydraulics". The roughness coefficients used in the previously developed HEC-2 models served as a beginning point for the calibration process. These values were then adjusted to calibrate the water surface elevation to known high water marks. With the exception of the Saddle River, all of the tributaries were calibrated to the most recent largest floods of April 1984 and May 1968. Because of a lack of data on the eastern side of the basin for 1984, the Saddle River was then calibrated to the November 1977 flood event. Selected values for channel roughness ranged from 0.025 to 0.047 and from 0.05 to 0.19 for overbank areas. Table C-16 lists the roughness coefficients by tributary. Flow in portions of the overbank areas was further restricted thru the use of the effective flow options which allows for storage in the overbanks but limits conveyance.

The basin-wide calibration effort concentrated on the storm of April 1984. Being the most recent large flood in the basin (25 to 40 year event in many locations) it provided both the quantity and quality of data needed to produce a well calibrated model. Adjustments were first made in the model to match the computed hydrographs with the gage recordings. To facilitate this, actual gage records were input as boundary conditions to the UNET model where applicable. The model was then refined until the maximum water surface profile matched the high water marks. The tolerance for a match was considered +0.5 ft. and -1.0 ft. This would allow for variations in the elevations of high water marks due to velocity head, wave action, wave run-up, the effects of super elevation, jetting and drawdown through bridges, and the accuracy with which the floodmark elevations were measured. Noting that most of the variables associated the floodmarks tend to give results which are higher than the water surface elevation in the center of the stream, the acceptable tolerances were shifted in that direction.

Because of insufficient data on the Saddle River for the April 1984 flood, the Saddle River was calibrated using the November 1977 flood. With the HEC-1 flow hydrographs, the model was adjusted to successfully reproduce the recorded high water marks. The May 1968 and November 1977 floods were also used to verify the calibration of the remaining portion of the model by comparison of computed hydrographs to peak discharges and gage records.

Tables C-17a thru C-17c present the summary of the calibration efforts. A comparison of high water marks and computed water surface elevations are provided for these events.

19.6. Conclusion

Based on the calibration and verification process described above, the Passaic River Basin HEC-1 and UNET models were considered capable of synthezing both historical and hypothetical events. Therefore, the models were prepared for use with the hypothetical future with and without project conditions.

C. Hypothetical Storm Analysis

20. Discharge - Frequency Analysis

A frequency analysis was performed on six stream flow gages within the Passaic River Basin that had long-term records. Annual series frequency analyses were performed using computer program HECWRC, Flood Flow Frequency Analysis, dated April 1987. This program incorporates procedures from EC 1110-2-249, "Hydrologic Frequency Analysis," and Water Resources Council Bulletin 17B, "Guidelines for Determining Flood Flow Frequency." The log-Pearson Type III distribution method and Weibull plotting positions were used in the statistical analysis of the gage data. Expected probability adjustments were made to all gages to obtain the final adopted frequency curves. The frequency analysis for a seventh gage, the Saddle River at Lodi, was taken from the E2 milestone documentation provided to CENAD by CENAN-EN-H (formerly CENAN-PL). This report was dated December 1990. All statistical computations were performed on gage data through Water Year 1994 as adjusted for partial duration and urbanization, as discussed in the following paragraphs.

20.1. Record Homogeneity

Most of the USGS quadrangle maps covering the Passaic River Basin were originally prepared in the mid-1950's and photorevised in 1981. Inspection of the photorevisions, mapped in purple, revealed significant urban sprawl around the small outlying towns in the basin and along numerous transportation corridors. Some areas that have seen considerable urbanization are located in the Upper Passaic, Rockaway, and Whippany River basins where Interstates 78, 80 and 287 were constructed.

The gage record began in 1882 for Passaic River at Little Falls, the earliest of the six gage sites analyzed. The record for Pompton River at Pompton Plains, the most critical gage site for sizing the diversion tunnel, began in 1941. An adjustment to the gage data was computed in order to compensate for the effects of urbanization on the earlier peak discharges. The data from each of the six gages was divided into two sets, and a frequency curve computed for each set. The Water Year 1965 was used to separate the data sets for all of the gages except the Whippany River at Morristown, where Water Year 1970 was used. The logic for selecting these dates will be discussed in subsequent paragraphs. Frequency curve computations were performed for each

data set with the equivalent frequency points from each data set plotted against each other. A regression equation was then computed and a line fitted through the points. The data points in the older set were then adjusted by the regression equation. The regression equations were in the form of 2nd order polynomials, except for the Ramapo River at Pompton Lakes, which was a 3rd order polynomial.

Urbanization primarily effects the more frequent floods and has little effect upon the less frequent (larger) floods. This results from the ground becoming saturated before the peak of the less frequent events and most of the rainfall runs off. Likewise, the adjustment equations used to compensate for urbanization made a significant difference in the small floods but had little effect on larger floods. In fact, no adjustment was made to the October 1903 flood peak, the largest on record in the Passaic basin, at Pompton Lakes, Pompton Plains, or Little Falls. Only a small adjustment was made to the 1903 flood peak at Chatham, Morristown, and above Boonton Reservoir.

Table C-8 lists the observed and adjusted discharges for each gage. The equations used to adjust the observed data follows, where the adjusted peak flow (Y) was computed from the observed data (X) by use of the developed regression equation:

Passaic at Chatham (No. 13795) :

$$Y = 260+.988(X)-2.835(10)^{-6}(X)^2$$

Whippany at Morristown (No. 13815) :

$$Y = 555+0.823(X)+2.513(10)^{-5}(X)^2$$

Rockaway above Boonton Reservoir (No. 13805) :

$$Y = 81+1.554(X)-3.662(10)^{-5}(X)^2$$

Ramapo at Pompton Lakes (No. 13880) :

$$Y = -393+1.466(X)-4.244(10)^{-5}(X)^2+8.262(10)^{-10}(X)^3$$

Pompton at Pompton Plains (No. 13885) :

$$Y = 2814+1.135(X)-5.604(10)^{-6}(X)^2$$

Passaic at Little Falls (No. 13895) :

$$Y = 735+1.026(X)-4.4(10)^{-6}(X)^2$$

20.2. Justification of Urbanization Adjustment

The historically observed data from the six gages on which the urbanization adjustments were made, were tested for evidence of trends. Both a Run and Kendall Test were performed on the

historical flow data at the six gages. The Run Test was used to test the sample of data for randomness. It looks at the distribution of runs greater than, or less than, the mean. A data set with a significant trend will likely have fewer numbers of runs. The Kendall test is likewise designed to detect increasing or decreasing trends in the data. Both tests result in the computation of a z statistic, which is compared to the critical $z_{\alpha/2}$. If the absolute value of z is greater than $z_{\alpha/2}$, the null hypothesis of randomness should be rejected, meaning that the distribution is probably not random and there may be a trend, with a level of confidence of $(1-\alpha)$. These tests are more fully explained in "Evaluating the Effect of Watershed Changes on the Flood Frequency Curve", Richard H. McCuen and Wilbert O. Thomas, Jr. February 1991. This paper was prepared for the Bullentin 17B Work Group of the Hydrology Subcommittee.

The table below summarizes the results of the Run and Kendall Tests. The Run Test computations are provided as Attachment 1 while the Kendall Test computations are in Attachment 2.

Results of Run and Kendall Tests

Gage	Run z	Comments	Kendall z	Comments
Passaic at Chatham	-1.37	random	+1.75	trend
Whippany at Morristown	-0.24	random	+2.01	trend
Rockaway above Boonton	-1.65	marginal	+2.75	trend
Ramapo at Pompton Lakes	+0.00	random	+1.16	random
Pompton at Pomp. Plains	-0.86	random	+2.08	trend
Passaic at Little Falls	-0.83	random	-1.59	marginal

Note: for α=10%, $z_{\alpha/2} = \pm 1.64$

The results of the above tests were inconclusive. The Run Test identified one gage that was possibly non-random, while five tested as likely random data sets. The Kendall Test provided the opposite results; four possible trends and two likely random data sets. Only for the Rockaway above Boonton gage was the null hypothesis rejected in both tests, and only for the Ramapo at Pompton Lakes was the null hypothesis accepted in both tests. Despite the inconclusiveness of those tests, inspection of bar graphs of annual peak flows for each of the six gages, shown in Figures C-46a thru C-46f, clearly reveals an increasing trend in the flow, especially the lower and middle peaks. Only at the Little Falls gage was a trend not immediately evident. However, neglecting the data around the turn of the century, when other large scale changes in land use may have been affecting the data, a trend becomes apparent there as well. Land use changes that may have affected the Little Falls data around and before 1900 are farming of lowlands that later reverted to wetlands, and farming of lands that were later reforested.

Although a trend in the peak flow may not be statistically determined, it is visually apparent. To verify that it was not the result of some anomaly of wet and dry cycles in rainfall patterns, event rainfalls from 1949 through 1989 that coincided with the annual peak flow at the Pompton Plains streamflow gage were determined and tabulated at both the Wanaque Raymond Dam and Little Falls rain gages. Both rainfall gages produced similar results, as shown in Figure C-46g. From a visual inspection of Figure C-46g, there is minimal relationship apparent between total event rainfall and peak runoff. Antecedent moisture condition and seasonal variations of ground cover and temperature had a more significant impact. A Kendall Test for trends was run on both sets of rainfall data. The computed z statistic was 1.40 for Little Falls, and 0.60 for Wanaque Raymond Dam, both less than 1.64 and indicative of randomness in the rainfall data. These Kendall Test computations are also included in Attachment 2.

As discussed earlier, peak flow data before 1965 was adjusted for urbanization, while post 1965 data remained as recorded. Examination of the plotted peaks in Figures C-46a through C-46f reveals that 1965 is a dividing (low) point between cyclical periods of higher peak discharges. As presented in the Economics Appendix of the Phase I report, the population of the Passaic River Basin increased through the 1950's and early 1960's, but leveled off in the mid to late 1960's. That the population has stabilized since the mid- to late-1960's indicates that the decision not to adjust for urbanization since that time period is a reasonable assumption.

The Student T Test was used to examine the differences between the means of the pre-1965 and post-1965 data sets for the six gages. The peak flow values are divided into two data sets, with means and standard deviations computed for each set. The T statistic is similar in nature to the z statistic of the Kendall Test. T is compared to the critical $T_{\alpha/2}$ and if the absolute value of T is greater, then a significant difference in the means of the two data sets is indicated, and the two sets of data are presumed not from the same population. A summary of the results of the T Tests is presented in the following table.

Gage	T	Comments
Passaic at Chatham	-2.96	Signif. variance in means
Whippany at Morristown	-2.96	Signif. variance in means
Rockaway above Boonton	-3.25	Signif. variance in means
Ramapo at Pompton Lakes	-1.26	Possible null hypothesis
Pompton at Pomp. Plains	-1.90	Signif. variance in means
Passaic at Little Falls	-0.91	Possible null hypothesis

Note: for α=10%, $T_{\alpha/2} = \pm 1.64$

These results tend to verify what was observed in Figures C-46a through C-46f; that there is a difference in the data before and after 1965. The T Test computations are provided as Attachment 3.

A sensitivity analysis was performed to determine what effect choosing a different dividing date (other than 1965) would have. Divided data sets using 1959 and 1972 were selected for comparison to 1965. The Student T Test was again used to compare the differences in the means of the logarithms of the peak annual flow at the Pompton River at Pompton Plains gage. The Pompton Plains gage was selected for analysis because it is the most critical gage to the Passaic River project. Results of this analysis are presented in the following table.

Results of Sensitivity Analysis of Division Year
Pompton Plains Gage

Year of Division	Student T	Comment
1965	-2.32	Signif. variance in means
1959	-0.59	Possible null hypothesis
1972	-0.46	Possible null hypothesis

Note: The T statistic from this table differs from that presented in the previous table because it is based on logarithms of peak discharges while the previous table was based on the actual flow in cfs.

The T statistic is well above the 10% critical value of 1.64 for the 1965 division, but is well below the critical value for the other two division years tested.

The final statistical test that was performed was the Mann-Whitney U Test for distribution inequality. This test is frequently used where an episodic event divides the sub-sets of data. The method that was used to adjust for urbanization is equivalent to assuming such an episodic event. The computed value, U, of the Mann-Whitney Test is incorporated into a z statistic as in the Kendall Test. Once again the critical value for $z_{\alpha/2}$ is 1.64. The computed value of the z statistic for the Mann-Whitney Test at the Pompton Plains gage is -2.15, well above the critical value. That leads to a rejection of the null hypothesis that the two independent samples (pre- and post-1965) are from the same population. The Mann-Whitney computations are provided as Attachment 4.

The conclusion reached from these statistical and graphical analyses was that there is an increasing trend in the annual peak flow data, most likely due to urbanization, and that the methodology is a reasonable approximation of the effects of urbanization. While we recognize that urbanization is not generally an episodic event, neither is it a linear progression. It is admittedly an approximation to adjust the discharge for

urbanization uniformly over a given time period, however it appears to be no greater an approximation than other methodologies available. The adjustment that we performed had the greatest impact on lower peak discharges and little or no impact on larger peak discharges. This tracks well with the observations of the bar graphs of the annual peaks in which the major flood peaks appear to be random while the small and moderate flood peaks have a tendency to increase upward in cyclical fashion. No adjustments to discharges were made after 1965, which is justified by the fact that the growth of basin population slowed at about that time.

20.3. Gage Frequency Computations

Generalized skew coefficients, taken from Special Projects Memo 480 of the Hydrologic Engineering Center, "Generalized Skew Study for the State of New Jersey", December 1977, were input into the HECWRC model for each gaging station. The generalized skew which varied from 0.3 to 0.4, were weighted with the computed station skew for each gage to determine the adopted skew coefficients. The mean square errors of each skew were used to determine the relative weighting, and adopted skews were rounded to the nearest tenth.

The flood of 1903 was used as a historic event, denoting that all floods between that event and the start of the continuous gaging record were of a lesser magnitude. The only exceptions were the gages on the Passaic River at Little Falls and at Chatham, where the continuous record extended through 1903.

From a statistical viewpoint when reviewing all six gaged data, only one outlier was identified during the entire analysis. The 1989 flood on the Whippany River at Morristown was designated a low outlier.

The seven gage-frequency curves are shown on Figures C-47 to C-53. A summary of the frequency computation input data is given in Table C-8. Except for the Saddle River gage, all gages have been updated thru Water Year 1994 with minimum changes to the peak flow frequency curves. Tables C-18a to C-18f show the variation between 1992 and 1994 adjusted station statistics. The statistical parameters resulting from the gage systematic records, historical record extensions and with and without the urbanization adjustments are provided in Table C-19. Since the Little Falls gage had a continuous record prior to the 1903 flood, no historical extension was needed or performed.

Finally, due to the significance of the design flow at the Pompton Plains, N.J. gage, a special study was performed to review the historical variability of the 1% chance flood. The gage is located at the Pompton Inlet and the statistically computed peak flow directly influences the tunnel diameter size.

Since the tunnel is approximately 20.4 miles in length, the cost of this project element is highly influenced by the computed peak flow at this site.

To aid in the understanding of the possible variance of the design flow, station statistical data (mean, standard deviation and skew) were developed starting with Water Year 1970. Water Year 1970 was selected as the starting period since it would contain 30 years of systematic record. In development of station data for ungaged basins, 30 years is commonly used as the equivalent length of record. Having established the base year statistics, analysis then proceeded by determining yearly statistical values by incrementally adding each Water Year's peak flow. Station statistics were then tabulated along with various % chance storms that included expected probability. Results are shown on Table C-20 with graphical variation of the 1% chance and 10% chance year peak flows on Figure C-54. As can be seen in the statistically determined data, the 1% chance year peak flow has a slight upward trend since 1970. However, the variation narrows as the record length increases. Also this study has assumed a continuation of urbanization upstream of this site. As such, the current 1% chance peak flow of 28,600 cfs was increased for the year 2050 to 31,100 cfs. This increase would account for both future urbanization upstream of the gage and the likelihood of flood events occurring in the future which would raise the 100-year computed peak flow, as was observed to occur following the 1984 flood. Urbanization adjustments were made in the HYDPAR analysis which included review of development trends and municipal master plans. See Table C-20 and Figure C-54, which supports this effort.

21. Hypothetical Rainfall

Initial attempts to reproduce specific frequency flood peaks were based upon the use of like-frequency rainfalls. Since the model had been previously calibrated to historic events, and it is recognized that rainfall and flood frequencies are not necessarily coincidental, rainfall amounts were adjusted to achieve agreement with gage-frequency curves.

A 96-hour storm duration was selected due to the size and concentration time of the Passaic River drainage basin. Rainfall depths for the 1-, 2-, 5-, 10-, 25-, 50-, and 100- year storms were obtained from isopluvial maps presented in U. S. Department of Commerce Technical Paper (TP) Number 35 for the 1- hour duration rainfall, TP40 for the 2-, 3-, 6-, 12-, and 24- hour rainfall, and TP49 for the 48- and 96-hour duration rainfall depths. Partial duration rainfalls as determined from the above reports were used for all frequency storms analyzed.

All frequencies and depths were plotted on a log-log graph for each duration using the Microsoft Excel Spreadsheet Program. Individual values were adjusted slightly up or down to yield a

smooth curve through the data points. The 500- year rainfall depth was obtained by extrapolating the curves. Point rainfall for durations of 1 through 96 hours are shown in Table C-21.

The 96-hour rainfall distribution pattern was developed for each storm frequency by the HEC-1 hypothetical rainfall option using the duration depths specified in Table C-21. A triangular precipitation distribution is constructed automatically by the model such that the depth specified for any duration occurs during the central part of the storm. This results in a balanced 96-hour hyetograph, with the peak rainfall occurring in the 49th hour of the storm.

Aerial adjustments to point rainfall depths, obtained from TP40, are shown in Table C-22. The purpose of this adjustment is to account for the decreasing average depth of precipitation over a tributary area as the size of the contributing area increases. This adjustment allows the user to develop consistent depth-area relationships throughout the basin for a given % chance storm.

22. Statistical Data versus HEC-1/UNET Computed Data

The discharge-frequency curves at five of the calibration gages were computed with the use of the HEC-1 hydrologic model, while the discharge-frequency curves for the Pompton Plains and Little Falls gages were computed through the use of the UNET unsteady flow model. Three of the HEC-1 calibration gages, Ramapo River at Pompton Lakes, Passaic River at Chatham, and Saddle River at Lodi are also upstream boundary locations of the UNET model, and thus have significance for both models.

The models were able to reasonably reproduce specific flood frequency peaks at the calibration gages. This analysis was performed by maintaining the Clark unit hydrograph and Soil Conservation Service (SCS) runoff parameters calibrated for the historic flood events and using adjusted hypothetical rainfall amounts. The rainfall adjustments made to the theoretical specific frequency rainfall depths are shown in Table C-23. These adjustments were made to more accurately reproduce the statistically derived flows for the adopted peak flow frequency curves.

It is often assumed that the probability of a derived flood flow is the same as the probability of the rainfall event used in the simulation. This assumption is rarely right and often grossly in error (Linsley, Kohler and Paulhus, Hydrology for Engineers, 1975). Part of the variability of the rainfall-runoff relationship in the Passaic Basin may be explained by the large number of reservoirs located in all parts of the basin except the upper and lower reaches of the Passaic River mainstream. Linsley, et al, also stated that the relation based on storms of uniform areal distribution will yield runoff values which are too low when applied to storms with extremely uneven distributions. The size of the Passaic Basin and a sampling of historic events

indicate that unevenly distributed events are more normal than the basin wide uniform storm. A preliminary storm centering analysis further explored the use of non-uniform hypothetical rainfall spatial distributions and concluded that storm tracking, intensity, location and duration could all be expected to be important for tunnel operation.

Inspection of Table C-23 reveals that the Saddle River, for which an antecedent moisture condition (AMC) 1 was used, required an increase in rainfall amounts. AMC-1 conditions allowed the best reproduction of Saddle River gage data. This may result from the extensive impervious areas in the Saddle River Basin allowing it to dry more rapidly between rainfall events. This AMC-1 condition was extrapolated to the lower Passaic Basin which is also heavily urbanized.

In general, reductions in the theoretical rainfall values were required for the highland basins, many of which contain numerous small reservoirs, including the Whippany, Rockaway, Pequannock, Wanaque and Ramapo. Flatter basins, many containing wetland natural storage areas, generally required increases in rainfall values. These included the Upper Passaic, Central Valley and the Saddle River Basins.

Since the simulations yielded close approximations of like frequency events at the gages, it can be concluded that the models are a reliable tool for determining flood peaks at sites remote from the gaging stations. Furthermore, these models could also be used to predict future discharge-frequency relations caused by changes in land use and runoff characteristics as well as evaluating the impacts of various flood reduction measures on river discharges and stages.

A summary of the comparison of the modeled hypothetical storms to the gage discharge-frequency curves is presented in Table C-24. The following paragraphs discuss the results of the frequency analysis at each of the seven gages:

22.1. Passaic River at Chatham (HEC-1). Antecedent Moisture Condition 2 (AMC-2) SCS Curve Numbers were used in this model. The modeled storm flow values for each frequency hypothetical storm vary from 8.0% greater to 1.6% less than the statistically developed gage-frequency curve. All points are well within the confidence limits . No adjustments were made to rainfalls at any of the 8 frequencies considered.

22.2. Rockaway River above Boonton Reservoir (HEC-1). AMC-2 SCS Curve Numbers were used in this model. The modeled hypothetical flow values range from 7.1% greater to 5.0% less than the gage flow-frequency values, all within the confidence limits. The rainfall was adjusted downward by 7-15% for all floods.

22.3. Whippany River at Morristown (HEC-1). AMC-2 SCS Curve Numbers were used in this model. The modeled values range from 0.0% to 0.8% above the gage frequency curve, and are within the confidence limit bands. Rainfalls were adjusted by increasing totals to a maximum of 20% at the 1- year and varying downward to 15% at the 500- year.

22.4. Ramapo River at Pompton Lakes (HEC-1). AMC-2 SCS Curve Numbers were used in this model. The modeled values range from 3% above to 3.4% below the gage frequency curve and are well within the confidence limits . The 2- year through 50- year rainfalls were adjusted downward by 12 to 0%, but the 100- and 500- year rainfalls were factored upward by 2% and 10% respectively.

22.5. Pompton River at Pompton Plains (UNET). These hypothetical flood hydrographs were simulated through the use of the UNET model. Boundary hydrographs from HEC-1 were input to the UNET model for the Ramapo, Wanaque and Pequannock Rivers. The Ramapo boundary was previously calibrated to the Ramapo at Pompton Lakes gage, therefore adjustments to the computed discharge at Pompton Plains required adjustments to the Pequannock and Wanaque basins and the three subbasins located between the UNET boundary and the Pompton Plains gage. AMC-2 SCS Curve Numbers were used in all subbasins upstream of the Pompton Plains gage. The computed flows for the hypothetical events varied from 3.9% greater to 4.4% less than the statistically developed gage-frequency curve, all well within the confidence limits. All rainfall events greater than the two-year storm required adjustment, ranging from a 15% to 25% decrease. See Table C-23.

22.6. Passaic River at Little Falls (UNET). The UNET model was used to simulate the hypothetical flood hydrographs. Calibrated HEC-1 hydrographs were input as boundary conditions on the Passaic River at Chatham, the Whippany River above its confluence with the Black Brook and the Rockaway River downstream of Boonton Reservoir. Additionally, the UNET model had been verified at the Pompton Plains gage on the Pompton River upstream of this site. Adjustments were made to the rainfall depths on the Central Basin subareas. The Central Basin includes most of the natural storage areas of the Passaic Basin, including Great Piece Meadows, Hatfield Swamp, Troy Meadows, and Bog and Vly Meadows, all of which are represented in the UNET model.

AMC-2 SCS Curve numbers were used for all sub-basins tributary to the Little Falls gage point on the Passaic River. The computed flows for the hypothetical events varied from 1.1% below to 5.1% above the statistically developed gage frequency curve. This is well within the confidence limits.

An additional adjustment was made to the overbank distances in the Great Piece Meadows for the 50-, 100- and 500- year flood events. The Passaic River makes a sweeping turn from north to

east to southeast as it flows through the meadows. As flood heights increase, overbank flow spreads out farther, and the distance for the centroid of overbank flow is lessened on the right overbank. Flow in the left overbank ends at a bluff, therefore left overbank distances are unaffected and remain unchanged.

22.7. Saddle River at Lodi, NJ. (HEC-1). AMC-1 SCS curve numbers were used in this model. The modeled values range from 4.4% below to 4.3% above the gage frequency curve, and are well within the confidence limits. The 10- year through 500- year rainfall amounts were increased by 5%. The 1-, 2- and 5- year rainfalls were factored upward by 25%, 20% and 10% respectively.

23. Preservation of Natural Storage Analysis

23.1. General

The concept of preserving the existing 5,350 acres of natural storage areas residing in the Central Passaic River Basin was evaluated by looking at the possible changes in water surface stages that could occur if the lands were permitted to be developed. To simulate such development, these areas were subjected to the placement of fill and an overall change in land use was designated. The fill would be placed to reduce the flood storage volumes below the 100-year existing flood stages by up to a maximum of 20 percent. See Figure C-55 for the location of the preservations land areas.

However, New Jersey law requires compensatory cutting to offset the effects of filling within the Central Passaic River Basin flood plain areas. The State's attempt to retain flood storage, thru mandatory cutting, represents an alternative to keeping the existing undeveloped lands in their current state. Therefore, the impact on water surface stages throughout the Central Passaic Basin of filling and then filling and cutting were both modeled.

The modeling assumptions used in this analysis involved one of many possible fill-and-cut scenarios. The scenario chosen appears to be associated with impacts unlikely to be surpassed by any others. Therefore, the modeling analysis as performed may provide a reasonable basis for evaluating the value of maintaining the existing natural storage area lands in their present state.

23.1.1. Hydpar Analysis.

As discussed previously, a HYDPAR analysis was performed that revised the land use classification on 522 grid cells (10.33 acres each) or almost 5,400 acres. The revised land use changed SCS curve numbers that were used in the determination of the rainfall/runoff model hydrographs. Land use changes were based on the most likely type of development that would be allowed in

the preservation areas. In all, 30 HEC-1 subbasins were edited with the average curve number change equal to approximately +1.4. The number of subbasins that were altered in each River system were as follows:

River Basin	Number	Subbasins Revised *
Pequannock River	1	920
Pompton River	9	1100-1150,1160,1180-1190
Lower Rockaway	3	420-440
Lower Whippany	10	510-580
Central Passaic Basin	7	200,220-230,600-620,1220
	30	

* For Subbasin locations see Figures C-28 to C-37.

23.1.2. HEC-1 Analysis.

A revised HEC-1 model was developed with the curve number revisions as described above and new runoff hydrographs determined. The revised data was written to HEC-DSS and used as the driving hydrographs to the UNET model. Due to the small change in curve numbers, flood hydrographs by themselves would not be expected to significantly alter water levels within the UNET modeled area. To fully verify this theory, a UNET model was run that used the future-without-project conditions geometry and revised hydrographs only. As expected, impacts to the water surface elevations throughout the study area as a result of the increased runoff were virtually undetectable. Since increases in flow from these areas resulted in no water level changes, the study would now concentrate on the physical altering of the cross-sections, especially in the area of concern the Central Passaic River basin.

23.1.3. UNET - Analysis.

The impacts of the potential fills and cuts were incrementally modeled using the future-without-project UNET cross section file. Separate editions were used for modeling the future-without-filled and then the filled-and-cut scenarios.

In total, six separate UNET cross-section models were developed and analyzed as part of this study effort. Four UNET models were developed to investigate fill-only and two additional models were developed to include both cut-and fill-. The first of the UNET models was that associated with the untouched (no cut and no fill) scenario. This model would provide the base-line 100-year water surface stages used throughout this study effort. The second model more accurately computed cross-sectional areas since restrictions on hydraulic conveyances were removed. This model resulted in lower water surface stages. The third model incorporated conceptual filling at the critical cross-sections

(but no hydraulic value with respect to stages). The fourth model was an edited version of the first incorporating the filled cross-sections of the third and was directly comparable to the base-line condition.

23.1.3.1. Cross-Section Modifications.

Cross-sections critical to the preservation analysis were those which traversed extensive widths of preservation land. In addition, cross sections which were associated with long horizontal distances between sections were also considered critical. In either case, significant volumes of storage could be altered by developing the existing natural storage areas. These cross sections were subjected to area adjustment as the initial step to volume determination. Sections which did not meet both criteria were not considered as critical and were left untouched in the UNET models.

Cross-sectional areas computed by UNET are those associated with that part of a cross-section assumed subject to the longitudinal movement of water. Many cross-sections in the future conditions file assume partial-area longitudinal flow. This implies that the computed areas associated with the future conditions model would be less than the correct areas to which the 20 percent reduction (fill) rule could be applied. Any filling would then be substantially less than was actually permissible. Therefore, partial-area modeling to evaluate the preservation of natural storage areas would not provide a reliable answer.

In order to obtain more accurate estimates of the areas subject to the 20 percent fill rule, the partial-area files for the untouched and filled scenarios were edited. Conveyance restrictions were therefore removed from the UNET model. The conveyances at the affected sections were increased with the resultant water surface stages lowered. Hence, where stages are important, partial-area output must be consulted.

When computing the size of the fill, the initial area reduction for each cross-section where preservation lands were assumed to be developed was 20 percent. In view of the estimated area delineations used, the assumption was made that the reduction at a particular section was acceptable if it fell within 15 to 25 percent provided the average reduction for all sections in the same UNET reach was within 18 to 22 percent.

Area versus elevation data used in this analysis were based on output from the UNET model which had conveyance restrictions removed. Only special requirements which permanently modified 11 of the 537 cross-sections upstream of the Passaic and Pompton River confluence were retained. Elevations were then tabulated at fixed intervals to encompass the full range of cross-section heights. Minimum elevations tabulated were unique to each cross-section. Areas were separately developed for the left overbank,

channel and right overbank cross-section elements. This breakdown of cross sectional geometry would become important to the computation of storage volumes because of element length differences.

The areas to be filled were each blocked out on scaled cross-section drawings. The square-footage that could be reduced 20 percent by filling was known for each section from the area-elevation data as discussed above. A rectangular block containing 20 percent of the pertinent square-footage was added to each drawing. An area generally equal in size to this block was then delineated by approximation and the limits were then coded into the UNET model using an appropriate method.

23.1.3.2. Fills.

The siting of a conceptual fill within a cross-section was assumed to be flexible in order to simplify the area delineations. The delineated fills were not necessarily drawn within the lateral limits of the preservation lands and may have been placed at other cross sections that seemed appropriate when reviewing plan sheets or even on the opposite bank.

From the point of view of overbank storage where conveyance was not a factor, the loss of storage would be the same regardless of where the fill was placed and the area reduced. Therefore, in such cases, the fill was generally placed where the fewest X,Y data pairs would have to be changed or where the ground surface was least irregular. The first option minimized file editing. The second facilitated visually matching the sizes of delineated areas to the reference blocks (20 percent area) placed on the cross-section drawings.

A conveyence-altering X3 card would remove area below and to the left or right of a particular X,Y point. Thus, a left (or right) overbank fill to be simulated by this means would be located at the left (or right) edge of the cross-section involved. The actual X,Y pairs entered corresponded to the interior infringement location of the delineated fills.

Once the area to be filled at a cross-section had been determined, one of three approaches was used to perform the area reduction. The first approach was to insert an X3 card provided none existed. The X3 card would block area out the left or right below the given elevation. The second approach was to modify an existing control card which affected only one bank of the cross-section so that it would affect both banks. The third approach, and most time-consuming, used where an X3 card already affected both banks, was to change the data-points (X,Y pairs) defining the cross-section.

For those cross-sections not subject to existing conveyance restrictions or cross sections with single-bank area restrictions, the upper limit on fill placement was assumed to be

the 100-year flood stage associated with the baseline model. For cross sections subject to both-bank area restrictions, the criterion was the same unless the 100-year stage was higher than the area restriction stage in which case the latter was assumed to govern in order to preserve conveyance.

Where filling was simulated by changing the data pairs defining a cross-section, the simulation would have no effect on conveyance but would simply reduce flood storage. Where it was simulated through the editing or insertion of X3 cards, there was both a reduction in conveyance and a loss of storage.

Table C-25 shows the future and the filled cross-section scenarios. It contains a listing of the relevant cross-sectional areas, ratios of these areas and the differences between them. Finally, it contains a listing of the lengths and volumes of the fills. The volumes were computed from the lengths and the differential areas and converted to acre-feet. The volume of fill conceptually placed within the 100-year flood was 19,200 acre-feet (Table C-25) or approximately 0.47 inches of precipitation above Little Falls, N.J. Total runoff of the 100-year flood would be in excess of 5 inches. Therefore, the preservation lands could potentially store up to 10% of the runoff volume of the 100-year flood.

23.1.3.3. Cutting.

New Jersey statutes require the loss of flood storage volume to fill to be replaced by cutting within the Central Passaic Basin. Excavation within that geographical limit is required, but it does not appear that it must be done immediately adjacent to the fill placed. For that matter, replacement cutting may not necessary be on the same reach of stream. For the purpose of this analysis, a fifth and sixth UNET model was developed to compensate for the fill by cutting along and/or adjacent to the rivers. Fills and cuts were basically counterbalanced on a reach-by-reach basis, since this would minimize changes to hydrograph shape. It should be noted that an infinite number of cut solutions exist and this approach was emphasized to test the impacts of New Jersey's "no net fill" concept.

In view of the overbank and channel length differences between many cross sections and the likely siting of fills (on overbank locations) and cuts hypothetically near the river, a 100-percent area tradeoff (cut versus fill) at any given cross-section would not always result in a 100-percent volume offset. Therefore, a mathematical check of cut and fill volumes were made on a UNET reach-by-reach basis. Volumes were then calculated as the sum of the associated section elements (left overbank, channel, right overbank), where each element volume was calculated as the product of the cross-sectional area of the element multiplied by the respective distance to the next section. Table C-26 summarized these results.

23.1.4. Impacts and Analysis.

Volume restoration analysis efforts were considered completed when the reach-by-reach ratios fell within the 0.950-1.050 range. The volumes to be used to compute the ratios were those associated solely with the cross-sections under investigation, and these were not necessarily all the sections in a reach. The efforts were generally successful and the ratios of all reaches other than 1 and 10, which are located upstream of the most important storage elements, fell within the specified range as given in Table C-27.

On an overall basis, the final reach-by-reach volumes essentially equalled the initial ones, the ratio being 0.991. For Reach 4, which includes Troy Meadows, the corresponding ratio was 1.016. For Reach 6, which includes much of Hatfield Swamp, it was 1.012. For Reach 7, which includes Great Piece Meadows, it was 0.972. For Reaches 1 and 10, the ratios were, respectively, 0.923 and 1.150.

During the incremental analysis, stage variations were looked at with both filling alone as well as with compensatory cutting. Changes to stage at selected cross-sections are shown in Table C-28. This table is supported graphically by Figures C-56a thru C-59e.

Table C-28 also indicates that water level stages generally increased following placement of fill and decreased with the hypothetically permissible cutting. Maximum increase with fill alone was 0.5 feet, a level of change found at several sections including that at the Little Falls gaging station (17R153860).

Table C-26 deals with all cross-section scenarios. It contains a listing of gross volumes on a section-by-section basis. It also contains the areas upon which the volume determinations were based, assuming the river cut scenario was used.

With a modified (filled and cut) flood plain (6th model), rates of flow near the start of the runoff event would also be altered. The peak rate of 100-year runoff with the modified floodplain as hypothetically modeled would be 27,650 cfs, 850 cfs higher than that with an unmodified flood plain at Little Falls. This would also occur one hour earlier. Figure C-60 shows the 100-year pre-and post-scenario hydrographs at Little Falls, NJ.

As a more likely situation, compensating cutting for the filling of the preservation lands would occur in the elevated overbanks where no impact should occur to existing wetlands. As hypothetically modeled, no net fill would be maintained below the 100 year flood levels. Provided that the 19,200 acre-feet of cut would be made below the 100-year stage, no resultant water level increase would occur under the 100-year or rarer event. The rarer floods would completely submerge both the cut and fill

locations. However, stage increases would result at the lower (more common) flood events, since the cutting in this case would be assumed to occur at higher elevations in the overbank.

During the analysis of the preservation of natural storage, it became apparent that maintaining these lands in their current state would be highly beneficial. Not only would it prevent the development on these 5,350 acres but it would also avoid the disruption of thousands of acres to compensate for the placement of fill. For example, to compensate for the 19,200 acre-feet of fill, some 6,000 acres would need to be excavated an average depth of 3.2 feet. Therefore, the disruption to the entire Central Passaic basin area would involve a substantially larger land area than the 5,350 acres that could be potentially developed.

23.2. Invasion of Phragmites.

23.2.1. Introduction.

Currently, there are significant stands of the plant Phragmites within the flood plain of the Passaic River basin. In the future, an additional 3,500 acres within the Central Passaic basin could be subjected to this type of growth. This value was assumed since portions of the existing areas are too wet and generally have standing water for significant periods during the year. However, should the extent of coverage of these plants increase, there would be an associated reduction of low-lying flood storage for the reason discussed below. Thus, the study of the value of existing storage preservation would be incomplete without an analysis of the impact of this encroachment.

There is a long history of the spread of Phragmites within the Passaic Basin. Stands of this plant have been noted to have increased in areal extent over the period 1978-1990, and conditions appear favorable for the further expansion of these stands. In northern New Jersey, the Phragmites plant spreads largely by creeping rhizomes which develop above the level of the existing ground surface. Over time, silt and debris deposited from rainfall/runoff events would become trapped and accumulate within the rhizome net creating a new and elevated ground surface. Successive silt and debris trapping would be a process that could continue producing a net loss of storage in the future.

23.2.2. Methodology of Analysis. The methodology used to mimic the Phragmites encroachment was similar to the block-out technique used in the filling of preservation lands as discussed previously. However, since this encroachment would be a natural occurrence, there was no need to consider the preservation of conveyance and no attempt was made to replace the storage lost to encroachment.

The potential upper limit of root-mat build-up is at best indeterminate, and a function of many variables. Therefore, various thicknesses were assumed in order to permit a range for impact evaluation. This analysis considered root-mat buildup for three specific thicknesses, 1-foot, 2-foot and 3-foot.

The timing of the root-mat buildup is also highly subjective. This being the case, the assumption was made that, regardless of the thickness of the mat or the rate of growth, full development would occur by the year 2050. It is recognized that accumulation would undoubtedly vary from one part of the Passaic River basin to another; however, it was assumed that by the year 2050 the magnitude and distribution of the buildup would be relatively consistent. Hence, each of the three thicknesses was evaluated on an overall basis. Three separate UNET model cross section files were required and each file was independent of frequency. Impacts could occur over the entire range of frequency floods.

The cross-sections considered suitable for analysis were those associated with significant acreages having a potential for Phragmites infestation. Such sections were those which (1), cut across extensive widths of potentially threatened land, (2), cut across considerable lengths of such land or (3), met both of the preceding criteria.

Cross-sections were edited for area reduction in the same basic manner as were the sections used for the preservation analysis discussed above. Figure C-61 illustrates the use of X3 cards to block out the cross-sectional areas to be occupied by Phragmites root mats. This was a composite figure based on the use of three different X3 cards, one in each of the three independent UNET Phragmites models. Curves 3, 5, and 7 on this figure define, respectively, the upper limits of 1-foot, 2-foot and 3-foot root mats and reflect the corresponding progressive reduction in cross-sectional area below, for example, Elevation 180.

Figure C-62 illustrates the use of Y-component modifications to alter cross-sectional areas and is also a composite figure. Curve 1 passes through data-point pairs (unmodified) which define the existing cross-section. Curves 3, 5 and 7 pass through modified X,Y data point pairs which define root-mat upper limits.

Floods for all eight frequencies were used in the Phragmites analysis and these floods were the same used for the preservation and tunnel design. Runoff from the 100-year event only was applied to all three Phragmites models. Runoff from the other events was applied only to the model incorporating the greatest mat thickness since 100-year impacts were found to be relatively insignificant regardless of thickness.

As documented above, Phragmites infestation can be expected to result in the replacement of portions of the existing vegetation in threatened areas. As a result of this expected vegetation tradeoff, Manning's "n" values currently in the UNET models were felt to be representative of what might be expected to prevail following vegetation changeover. Therefore, all UNET model runs incorporate similar Manning's "n" for all conditions analyzed.

23.2.3. Results.

Impacts of the hypothetical Phragmites infestations were checked at the Little Falls gaging station on the Passaic River and at various confluences upstream. Table C-29 summarizes the changes at these checkpoints of the peak stages associated with the considered combinations of flood frequency and root-mat thickness. Figures C-56d and e thru C-59d and e show the stage hydrographs associated with the tabulated data and indicate changes in hydrograph shape or timing attributable to the various root mat thicknesses.

Tributary cross-sections identified in Table C-29 are the most downstream sections on Pinch Brook, Whippany River, Rockaway River and the Pompton River. Stages at these sections are the same as those at the nearest sections on the adjoining streams. Therefore, the stages reported are representative of those at the confluences involved. Stage changes associated with the Phragmites scenarios are given in Table C-29 and shown on Figures C-56d and e thru D-59d and e. For the 100-year event, the maximum differences between stages with no root mat and with 1-, 2- and 3-foot root mats were found to be 0.05 feet, 0.14 feet and 0.26 feet. The greatest change found was associated with the more common flood events and resulted in a 0.36 foot increase with a 3-foot root-mat buildup.

Discharge changes associated with continued Phragmites growth were also found to be relatively small. The greatest 100-year difference at any checkpoint cross-section was 7.2 percent at section 05R000210 on the Rockaway River just upstream of the confluence of the Passaic River. See Figure C-63 for a comparison of discharge hydrographs at this section.

24. Hypothetical Peak Flows and Stages - W/O Project

Hypothetical flood analysis as documented in this report consists of tabulated data, graphic displays of flood hydrographs and graphic plots of existing condition water surface profiles. This data has been included in the report to substantiate the analysis performed, as well provide the ability of developing rating curves (in appropriate areas) and maximum water surface elevations throughout the basin.

24.1. Existing Conditions

Hypothetical floods for existing conditions have been tabulated in Table C-24 for selected locations. In general, the selected locations were the gages of interest in which rainfalls were adjusted to match the peak flow frequency curves. With the rainfalls adjusted properly to match the adopted peak flow frequency curves, maximum stages and flows for a given percent chance storms were derived throughout the basin. Tables C-30a and C-30b provide the peak stages (elevations) and flows at numerous locations throughout the Passaic Basin. These tables provide stages and flows at various cross section locations by UNET river reach, for the 1-, 2-, 5-, 10-, 25-, 50-, 100- and 500- year hypothetical events.

Graphic display of the hypothetical events are shown on Figures C-64a thru C-64. These figures typically display the 1-, 10- and 100- year flood hydrographs at selected locations throughout the basin. Figure C-64a thru C-64c show flood hydrographs at the gaged locations. Flood hydrographs have been generated using both HEC-1 and UNET while HEC-DSS was used to plot the output. Provided in support of the flow reversal phenomenon at the confluence of the Pompton and Passaic Rivers are Figures C-64d thru C-64f. Flows and stages are plotted on these Figures for the 1, 10, and 100-year events. As demonstrated by UNET, Pompton River flows are entering the lower portion of the Great Piece Meadows and temporarily impounding Passaic flows. Review of stages in this area show that the Great Piece Meadows water surface elevation has a slight adverse slope as Pompton water moves upstream on the Passaic. Figures C-64g and h, show the flow and stage relationships at Pompton Plains and Little Falls for the 100-, 10- and 1- year events.

In the Lower Valley, Figures C-64i displays flow and stage conditions at Dundee Dam (a hydraulic control point) for 1-, 10- and 100-year events. Figure C-64j and C-64k demonstrate the ability of UNET to model the unsteady flow due to a variation in fluvial discharge and stage changes resulting from tidal cycles within this section of the Passaic River.

For comparative purposes, 10-year hydrographs are also presented. Changes in flows and stages over time are not as severe as those occurring during the 100-year event.

Also provided are selected loop rating curves for the existing 1992 conditions. The curves are shown on Figures C-65a to C-65f. As expected, loop rating curves in the Central Basin area show a large variation in elevation due to the high degree of tailwater effects on the relatively flat flowlines. This was evident in not only the 100- year but also the 10 -year loop rating curves.

Finally for the existing conditions analysis, water surface profiles and inundation maps have been provided and are attached as Plate C-1 to Plate C-91. Plate 49 provides an index sheet to the 42 inundation maps. The water surface profiles show the April 1984 calibration event as well as the 1-, 2-, 5-, 10-, 25-, 50-, 100- and 500- year hypothetical events. Table C-30a provides tabulated peak elevation data while Table C-30b gives the peak flows at the same locations. The inundation maps show the aerial extent and limits of flooding for both the 100- and 500- year events. Table C-17a provides the comparison of the high water marks for the April 1984 event and reflects the accuracy of the UNET simulation run.

24.2. Future Conditions

Hypothetical flood peaks for future conditions were determined and have been tabulated in Table C-31 for the seven selected gaged locations. Table C-31 provides a direct comparison of the existing to future unimproved condition runoffs at the these gages, with the percent increase in flow noted. As expected, future condition flows show a higher percent increase at the more frequent events (ie.- 2 year) than at the more severe events (ie. 100- year). The future condition hypothetical events were developed by use of the revised curve numbers for the year 2050. As noted in Table C-11, the future condition curve numbers did not change significantly and it was felt that such small revisions to the curve numbers would have no impact on the unit hydrograph parameters. Therefore, the only modification from the existing conditions analysis was the expected change in land use on a cell by cell basis, as performed in the HYDPAR evaluation.

No modifications were made to the existing bridges in the project areas or elsewhere in the UNET model. However, this office will monitor future bridge alterations planned or constructed as during the FDM phase of design. It should be noted that in general, bridge modifications have a very localized impact (except in the Central Basin along the Passaic and/or Pompton Rivers). Any bridge modifications/replacements/removals in the future in the vicinity of the Corps proposed project elements would be coordinated with the Corps since the Local Cooperation Agreement would require this arrangement.

At this time, no known significant channel modification work has been proposed to be performed within the basin, outside of the Corps tributary projects. Investigation of the possibility of channel modifications by others will continue during the FDM effort. Modifications to the future conditions computer files with and without the project would be made, should it become evident that such work will occur.

As was done with exiting conditions, future conditions peak elevations and flows were tabulated throughout the basin. Results of the simulations are given in Tables C-32a and C-32b. for the 1-, 2-, 5-, 10-, 25-, 50-, 100- and 500- year hypothetical events.

24.3. Stage Frequency Curves - Above Dundee Dam

Stage-frequency curves were developed from UNET's maximum water surface profiles throughout the entire basin. In all, 234 locations were selected for stage-frequency curve development. The number of curves created was highly dependent upon the data needs for economic evaluation and environmental impact assessment. These UNET derived stage-frequency curves would be appropriate for fluvial flow only and would be the complete solution for all 189 locations above Dundee Dam. However, for the 45 locations below Dundee Dam, the UNET developed stage-frequency curves would need to be combined with the impacts from storm surges. For these 45 stage-frequency curves the reader is referred to Section 25. Surge Modeling.

For all locations above Dundee Dam, water surface profiles from the eight hypothetical flood events were written to HEC-DSS for each of the conditions analyzed. These conditions were the Existing 1992 conditions without Corps project in-place; Existing 1992 conditions with Corps project in-place; Year 2050 without the Corps project in-place and Year 2050 with the Corps project in-place. Stage-frequency curves were calculated by the use of MATHPK, an HEC program capable of performing mathematical operations on HEC-DSS stored data. Water surface profiles were then sliced vertically at specific cross sections along all UNET modeled reaches with the data stored as water surface elevation versus frequency. The resultant values were then written to both a MATHPK HEC-DSS output file as well as ASCII output file in tabular format.

The MATHPK DSS output file served two important purposes. First the stage-frequency curve data generated was immediately transferable for economic evaluation. Comparisons for with and without project conditions could be made for the flood damage reduction category through the use of these curves. Secondly, graphical plots could easily be made at any location desired by use of HEC's DISPLAY program. Sample plots are shown on Figures C-66a to C-66c.

The MATHPK ASCII tabular output file was also extremely useful and was used in the development of water surface profile plots. In addition, the data was used to create tables describing water levels and flows throughout the basin for the various conditions analyzed.

D. Tidal Flooding and Storm-Surge Modeling

25. Surge Modeling

25.1. General

The lower 17.7 miles of the Passaic River from Dundee Dam to the mouth in Newark Bay are subject to occasional flooding, due mainly to storm surges. Consequently, the stages of the Passaic River from Newark Bay to Dundee Dam are influenced by a combination of fluvial and tidal conditions.

Likewise, the lower 22 miles of the Hackensack River from Oradell Reservoir to the mouth in Newark Bay are also influenced by both fluvial and tidal conditions.

The proposed tunnel outlet structure, along with several levees and floodwalls, would be constructed in the general vicinity of the Passaic and Hackensack confluence. The purpose of the levees and floodwalls would be to provide protection from tidal events. Discharges from the outlet structure could alter local flow patterns in the upper end of Newark Bay. Since the extent of tidal protection and the degree to which the project may influence the surrounding area must be determined, several extensive studies have been performed including a storm surge analysis and a hydrologic model for the Hackensack River Basin. The storm surge study which was completed in July 1992 by Ebasco Environmental in Bellevue, Washington for the New York District Corps of Engineers, through the Corps of Engineers Waterways Experiment Station (WES) is described below. A finalized version of the report was released by WES in September 1992 as Technical Report CERC 92-10, Passaic River Basin Flood Protection Project, Storm Surge Reduction.

25.2. Objectives

The primary goal of the Storm Surge Study was to develop stage versus frequency curves for tidal events within Newark Bay and the lower Passaic and Hackensack Rivers. The frequency curves are based on the combined effects of hurricanes and northeasters for conditions expected to exist in the year 2050. These curves were the basis for the design of the height and extent of the tidal levees.

The study also addressed whether a correlation between tidal surge levels and peak river flows exists. Previous studies which evaluated only coincidence of peak conditions have not fully addressed the relationship. Factors such as separation of residual storm surges from observed tidal heights (removal of astronomical tides), time lags and correlation factors were addressed for the first time, in order to fully evaluate overtopping and flanking.

Another objective of the study was to determine if the proposed tunnel would have an adverse affect on water levels in Newark Bay and the tidal zone areas of both rivers. The magnitude and extent of the effects were determined and are discussed both here as well as in the Improved Conditions modeling section.

25.3. Model History

For this analysis, a previous version of the storm surge model was updated and extended. A brief history of the development, set-up and use of the model is described below.

The current storm surge study uses results, methodologies and models which were adapted and expanded from a Flood Insurance Study of the City of New York developed for the New York State Department of Environmental Conservation. The flood insurance study was funded by the Flood Insurance Administration (FIA) of the Federal Emergency Management Agency (FEMA). The two computational hydraulic models used in the flood insurance study were calibrated and verified extensively.

An offshore surge generation model was developed using a finite-difference square grid with a five-nautical-mile resolution. The specific model used was the Tetra Tech SURGE Model (TT SURGE), which was previously developed for FEMA. Meteorological driving forces were employed to simulate the storm surges generated within Long Island Sound and along the New York Bight portion of the continental shelf.

The second hydrodynamic model was linked to the offshore model to provide a finer spatial resolution which was better suited to the complex inshore geometry. The nearshore surge propagation was computed using a link-node model of the embayments, waterways and rivers. The specific model used was the Dynamic Estuary Model (DEM).

In October 1983, a Tidal Flood Frequency Study of the Passaic River was completed by Camp, Dresser & McKee Inc. for the Corps of Engineers. This study included a sensitivity analysis of the models used in the Flood Insurance Study. The analysis showed that the offshore simulations were directly applicable to the Passaic River Study. However, the inland or nearshore model required extension and modification to accurately simulate the Passaic River.

Hydrographic considerations showed that simulation of storm surges along the Passaic River was greatly affected by the adjacent Hackensack River and the vast Meadowlands complex. Therefore, while the study was primarily focused on the Passaic River, the network was extended to include the effects of the Hackensack River. The Meadowlands portion of the Hackensack was only calibrated to accurately portray the effect on the Passaic River. Complete calibration of the Hackensack River was not conducted at that time.

In addition, the 1983 study used a large number of synthetic storms (252 hurricanes and 43 northeasters) which were simulated from a joint-probability analysis of meteorological inputs to derive the final frequency-elevation curves. The study also produced a set of hydrodynamically consistent frequency curves for the entire New York Bight.

In February 1989, a Reconnaissance Report of the Hackensack River Basin in New Jersey was completed. That report included further analysis and model modifications to address the storm surge problem along the Hackensack River. This study also started with the models used in the two studies mentioned above. The inland or nearshore surge propagation Dynamic Estuary Model (DEM) was improved and updated by adding: new floodplain topography data, additional Hackensack River cross-sections, accurate major highway crossing data, and Hackensack River discharges. The nearshore or DEM model was then recalibrated and validated in areas along the Hackensack which had not been accurately calibrated in the past. This study 1989 also modified the statistical approach with which the hurricane and northeaster frequencies were analyzed. The statistical method used was called the modified "bootstrap" method. The new method effectively reduced the number of storms analyzed from approximately 70,000 random storm combinations to 24 carefully selected events.

25.4. Model Geometry - 1992 DEM Model

The inland or nearshore link-node surge propagation model developed and used in previous studies was modified for this Phase II GDM effort in the following ways. The network or grid system resolution was increased in the area of the tunnel outlet and at other selected areas. The geometry was updated based on recent overbank surveys and pre-dredging soundings along the Passaic and Hackensack Rivers, Newark Bay, Kill Van Kull and Arthur Kill. The geometry was further modified to include the deepening of the navigation channels to a 40 foot minimum depth. Since the channel deepening construction was underway while the storm surge study was being conducted, several different geometric grids were maintained. The "pre-1980" grid was used to revalidate the DEM model to most of the historic conditions. The grid with the 40 foot channel depths was used to simulate future conditions in the year 2050. Figure C-67 depicts the complete grid system. Figure C-68 provides details of the grid network in Newark Bay and the surrounding areas.

25.5. Model Calibration

The refined grid with the "pre-1980" geometry was run with five events to ensure that the results still accurately reproduced the observed tide levels for those events. The five events consisted of one astronomic tide, two hurricanes, and two northeasters.

The first event simulated was a normal tide condition occurring in October of 1976. The October date was chosen because several tide cycles were recorded at about 20 locations throughout New York Harbor. Figures C-69a thru C-69h illustrate how the tidal marigrams have been reproduced at 8 of the 20 locations. The observed elevations, results of the old Hackensack model (February 1989), and the current simulation results are all shown on each figure. The current model more accurately simulates this event than any of the models used in previous studies.

The hurricanes of September 21, 1938 and September 12, 1960 were also simulated using the "pre-1980" grid. Figures C-70a and C-70b illustrate how well the computed elevations simulate the high and low tide observations for the 1938 event. Figures C-71a thru C-71d show the reproduction of the 1960 event.

Once again using the "pre-1980" grid the northeasters of November 1950 and 1953 were simulated. Figure C-72 shows how the computed results compare to the observed data for the 1950 event. Note that the actual water levels appear to have exceeded the maximum gage recording height. Figures C-73a thru C-73c compare the results of the 1953 event.

In addition, tide, wind and river flow data for the Northeaster of October and November 1991 (The "Halloween" Event) were gathered along the Passaic and Hackensack Rivers and at various locations throughout the Metropolitan Region. The event was of particular interest due to its recent occurrence and because it created unusually high tide levels without inland precipitation. Due to the lack of precipitation, flows in the Passaic and Hackensack Rivers were low resulting in observed water levels that were almost entirely due to storm induced ocean surge. However, the event occurred while the Newark Bay and Kill Van Kull channel deepening projects were under construction and represents neither "pre-" nor "post-" construction conditions. Comparisons between a set of 40-foot deep channels and the 35 foot deep channels indicate that there are minimal differences (less than 0.2 feet) in the resulting surge heights due to the limited 5 foot increase in depth. However, since it was easy to accomplish, a third geometric condition was used to model this event. The extent of the channel deepening at the time of the "Halloween" event was determined and the geometry of the model was adjusted to reflect this "in-between" condition. The results of this simulation are shown in Table C-33.

25.6. Coincidence Analysis

The water levels within the model are affected by the astronomic tides, river flows and meteorologically induced surges. The effect of each factor various greatly in various parts of the model, but their overall interaction can be very important in some areas. It is relatively easy to add the storm surge values onto the astronomic tide values and combine the

frequencies because they are independent events. However, river flows are not completely independent of the surge values. While storm surges can be directly linked to tropical events (hurricanes) and to extra-tropical events (northeasters), high river flows are also caused by both hurricanes and northeasters, and by localized high intensity thunderstorm events and lower intensity winter events with snowmelt. Consequently, in some cases the same meteorological event is creating both high river flows and storm surges, while in some cases one is occurring without the other. Therefore storm surges and high river flows are at times dependent and on other occasions they are completely independent. To further investigate this potential joint occurrence, a coincidence analysis was conducted.

Several approaches (graphical, tabular and analytical) were taken to investigate the relation between river flows and storm surges (observed tide minus astronomic tide). The approaches included cross-correlations and linear regressions of the data sets, examination of the frequency of joint occurrence, scatter plots of corresponding flow and surge events, and graphic overlays of tidal marigrams with corresponding flow hydrographs.

The tidal data consisted of hourly observed water levels at The Battery in New York and standard tidal prediction equations. The observed water level data was available from 1920 to 1991 with several gaps. The surge data was obtained by subtracting the predicted tide levels from the observed tide levels. The river flow data consisted of daily flow measurements from 1898 to 1990 at the Little Falls gage in New Jersey along the Passaic River.

Since observed water level data at the Battery was used to establish a surge/flow relationship at Kearny Point, the relationship between the Battery and Kearny Point was established. Figures C-74a thru C-74c show how the model translates and/or predicts the observed stages at both The Battery and Kearny Point. As can be seen from these figures, observed and predicted timing and magnitude (stage) between the two locations are in good agreement for each event. Observed heights at The Battery would therefore be very close to actual heights being observed at Kearny Point throughout the period of record.

The data time series was examined for periods when both the recorded tidal levels and the recorded river flows were available. This resulted in 21,171 daily observations or 58 years of record. A surge value of 1.85 feet and a flow of 3040 cubic feet per second (cfs) were used as threshold values to separate surge from non-surge events and flood from non-flood events respectively. The following tabulations were made: the number of times 1) both thresholds were exceeded, 2) neither threshold was exceeded, 3) the surge but not the flow was exceeded, and 4) flow but not surge was exceeded. The results are tabulated in Table C-34. Note that in the table, peak surge

heights are compared to the peak daily flow at the same time and then to peak daily flows 1, 2 and 3 days later and earlier. This results in an analysis of comparing the peak storm surge against a 6 day window of peak river flow. The tabular data indicates that the highest coincidence of surge events and flood events occur when the peak surge values are compared to peak flows which occurred 1 and 2 days later. However, also note that the number of times that both high flow and surge events occurred together is a very small percentage of the total number of events.

To visually display the historic relationship between storm surge and river runoff, several historic events were plotted. Figures C-75a thru C-75d provide a comparison of storm surges and peak runoffs occurring at the Battery and Little Falls gages respectively. Some of the events represent large surges and others represent large flows. However, for each event a distinct peak in both surge and flow can be observed. Note that some of the peaks are small in magnitude and they might have been lost in a purely numerical analysis. In addition, the size of one peak does not seem to affect the size of the other. Most importantly the figures indicate that a the peak storm surge occurs a number days prior to the peak river runoff.

Another method used to investigate the relationship between surge height and river flow visually, was to plot a scatter diagram. Figure C-76 displays these results. The diagram was created by plotting the surge height values on the horizontal axis and river flow values on the vertical axis. Therefore, each point shown on Figure C-76 represents an historical occurrence of peak surge and peak flow (within 72 hours). Since all flows above the average daily flow were used, the figure indicates a large number of occurrences where both surge and river flow are occurring. However, the proposed tunnel will not operate until the 1- year fluvial event is reached, which is approximately a flow of 6000 cfs at Little Falls. If only the flows greater than 6000 cfs are considered, then the number of occurrences is greatly reduced with resulting storm surges almost always less than 4 feet.

Using the results from Table C-34 and Figures C-75a thru C-75d a cross-correlation analysis and a lagged cross-correlation analysis between surge height and river flow was conducted. The analysis indicated that there was a peak correlation of +0.22 occurring when river flow was measured two days after the peak surge. The results of the analysis are shown in Figure C-77. A positive correlation value of 1.0 would imply complete dependence with both factors increasing or decreasing together. A correlation factor of 0.0 would indicate complete independence. The value of +0.22 indicates that the correlation between surge and flow is quite weak.

25.7. Tidal Boundary Conditions

The water surface elevations of the lower reaches of the UNET hydraulic model and the design head used to determine the tunnel size were dependent on the tidal boundary conditions. Therefore, it was important that the appropriate coincident tidal elevation be used when peak flow conditions are run. The coincidence analysis conducted in the storm surge study addressed this issue and resulted in three possible ways to determine an appropriate coincidental tide level.

The first approach reviewed the historical data with averaging of storm surges. While it quickly became evident that the magnitude of a storm surge was not a good means of predicting the magnitude of a river flow, the presence of one seems to indicate the possible presence of the other. If the probability of a specific river flow was statistically determined, then the probability of that flow was not altered by selecting the average downstream tidal elevation for that condition. Consequently, the maximum-daily surge elevation (within 72 hours of the observed river flow) were averaged for specified ranges of river flows. The results are summarized in Table C-35. Table C-35 indicates that the average storm surge occurring within 72 hours before the peak flow, ranges from 1.19 to 2.04 feet for the entire range of flows.

In the second approach, two stage-frequency curves were developed at The Battery, one for all events regardless of the Passaic River flow, and the other for stages when Passaic Rivers flows exceeded 6,000 cfs. Only flows larger than about 6,000 cfs are expected to be diverted into the tunnel. As expected, the conditional flow curve was significantly lower than the all condition flow curve as shown on Figure C-78.

Table C-35, Figure C-78 and the scatter diagram discussed in the previous section (Figure C-76) all present different means of determining the coincidental tide levels for use in the UNET model, the tunnel design and tunnel operation simulations.

Ultimately the average surge heights, summarized in Table C-35, were used to set the riverine UNET model tidal boundary. A conservatively high, but realistic, bay elevation was needed since the determination of water surface elevations is considered to be a capacity design condition.

Capacity design conditions usually involve the use of factors, such as high hydraulic losses, which increase water levels. Since the peak flow in most cases will last at least 12 hours, a normal astronomical high tide should occur during each event. Therefore an average storm surge can be superimposed over a normal high tide for any event without changing the probability of occurrence. Furthermore since peak flows can occur at any time during a month and since spring tide conditions exist for a few days every month the two could coincide for any event.

For existing conditions the UNET tidal boundary elevations were obtained by taking a mean sea level elevation of 0.7 feet NGVD, adding half the spring tide range of 6.0 feet and adding an average storm surge of 2.0 feet. This results in an existing condition tidal boundary elevation of 5.7 feet NGVD. For the future conditions analysis 0.5 foot was added for the 55 years of anticipated sea level rise to the year 2050. Therefore, the future conditions tidal boundary of 6.2 feet NGVD was obtained. These results are fairly close to the value of 5.5 feet NGVD which was used as a tidal boundary in the Phase I Passaic River GDM, as documented in December 1987.

The scatter diagram (Figure C-76) can also be used to determine a conservatively high tailwater for the design of the proposed tunnel capacity. Consideration was given to the creation of a boundary curve for the data on Figure C-76. A boundary curve which contains between 90 and 100 percent of the data points could be determined. If the tailwater for a tunnel design flow were based on a value along the boundary curve then there would be a certain amount of assurance (90 to 100 percent) that the tailwater would not be exceeded. The probability of a particular flow occurring while the boundary condition tailwater was present is not only significantly less likely than the probability of just the flow occurring but it is also hard to determine. However, the method does contain a high degree of confidence that the tailwater level will not be exceeded.

Finally the stage frequency curves developed in Figure C-78 could be used to determine the probability of a particular tailwater occurring while the tunnel is operating. Therefore, the tunnel design tailwater could be selected based on a certain probability of exceedence.

25.8. Flow Boundary Conditions

The water surface elevations in the upper end of the storm surge model were very dependent on river flow. Therefore an effort was made to determine a coincident flow condition to be used when simulating storm tide events. Similar to the analysis conducted for the UNET and tunnel design boundary condition, the average river flow that occurred within 72 hours of a surge of a specified height was determined. The result are presented in Table C-36 and they indicate that there is little variation in the average flows determined for different surge heights. The maximum average flows within a 72-hour period range from 1000 to 2000 cfs for different surge elevations which is fairly close to the average daily flow value of 1200 cfs at Little Falls, N.J. Table C-37 shows the maximum flow within a 72- hours period after several historic coastal events. The results in Table C-37 seem to indicate a slightly larger range. Since the effect of high flows on storm surge elevations is somewhat limited a conservatively high value of 5,000 cfs was selected which represents about the one year flow.

25.9. Statistical Procedure

The Joint Probability Method (JPM) was used to develop the frequency of coastal surge events (hurricanes and northeasters). The method involves the parameterization of each characteristic of a storm such that the probability of a certain magnitude or direction can be determined. Then the probability of a particular event is obtained by combining the probability of each characteristic.

In the 1983 Passaic River Storm Surge Study the JPM was used to develop over 70,000 combinations of storms and tides, which were used to develop the stage-frequency curves. Rather than rerun all the synthetic storms (with a modified grid) for the 1989 Hackensack Report an approach known as the modified "Bootstrap" method was used. The modified "Bootstrap" method uses a smaller subset of storms which can duplicate the original stage-frequency curves generated by the full data set. Generally the smaller subset is then run under different conditions to generate stage-frequency curves at other locations. The analysis completed for the 1989 Hackensack Report used a subset of 24 events. For this report the subset of 24 events was expanded to 33 events. The 33 events provided information for frequencies less than a 10% chance event and for frequencies greater than a 1% chance event. Table C-38 contains a list of the storms selected to reproduce the stage-frequency curves for this project.

25.10. Sensitivity Analysis

The purpose of the sensitivity analysis was two-fold. First the analysis was needed to determine if several future conditions alternatives had to be analyzed using the full set of 33 storms, or if the results would be so similar that only one or two future conditions plans would have to be analyzed using the full set of storms. The possible future conditions consists of the year 2050 conditions without a project, 2050 conditions with a tunnel but without surge levees, and 2050 conditions with both a tunnel and levees.

Secondly, the sensitivity of the storm surge elevations to the flow boundary condition was evaluated. In addition to the magnitude of the flow boundary condition, the coincidental timing and the use of a constant flow as opposed to a flow hydrograph also had to be considered.

Three events were used in the sensitivity analysis and are listed below:

a. The 11-13 October 1976 condition - represents a well documented non-surge event.

b. Synthetic hurricane H249 - represents a small hurricane/tide condition.

c. Synthetic hurricane H135 - represents a large hurricane/tide condition.

Each of the three events above was simulated using the following four conditions:

Future (2050) unimproved conditions with low river flows - Observed flows were used for the Oct 1976 event. The 1-year. flows of 5,285 cfs in the Passaic and 1,630 cfs in the Hackensack were used for the two hurricanes.

Future unimproved with high river flows - The 100- year. flows of 33,645 cfs in the Passaic and 7,135 cfs in the Hackensack were used for all events.

Future improved with a tunnel & without storm surge levees- The 100- year. flows of 20,000 cfs in the Passaic, 7135 cfs in the Hackensack and 30,000 cfs in the tunnel were used for all events.

Future improved with both a tunnel and storm surge levees - The 100- year. flows of 20,000 cfs in the Passaic, 7135 cfs in the Hackensack and 30,000 cfs in the tunnel were used for all events.

The results of the above simulations are summarized in Tables C-39, C-40 and C-41. Plotted results for various locations are shown on Figures C-79a thru C-79h for the 1976 event, Figures C-80a thru C-80h for hurricane 135, and Figure C-81a thru C-81h for hurricane 249.

The tables and figures indicate that in the lower reaches of both rivers and in Newark Bay there is no significant change in the water levels (less than 0.1 foot) due to the proposed tunnel. The most conspicuous change is a 0.4 foot change in the 10.0 foot surge levels experienced during Hurricane 135. The slight change in water levels was caused primarily by the proposed storm surge levees which prevent overbank flooding. While changes of less than 0.4 foot are beyond the accuracy of the model they due indicate a small relative difference between plans but the difference is considered minor.

Figures C-79c, C-80c and C-81c indicate that water levels in the Passaic less than 5 miles from the mouth will be lowered about 2 feet by the proposed tunnel. Other results indicate that improved water surfaces at the upper end (near the Saddle River) of this reach will be lowered 4 or 5 feet. This will be further discussed under later in this section under "Stage-Frequency Analysis".

The model results also indicate that there is no significant change in the water levels along any part of the Hackensack River due to the proposed tunnel or storm surge levees.

The sensitivity runs tabulated, plotted, and discussed above were based on continuous peak river flows and infinitely high levees for three very different events. In many cases unimproved conditions with low flows (average daily flows) were compared to improved conditions with 100- year flows. While the assumptions made should have maximized the adverse impacts of the proposed project, the results indicated minimal, insignificant and/or no increases in the water levels.

Further evaluation with two of the four future conditions was undertaken using the full set of 33 synthetic events. The results are discussed in the "Stage-Frequency Analysis" section below.

25.11. Stage Frequency Analysis

25.11.1. Storm Surge Conditions Only

Stage-frequency curves for various locations throughout the model were developed using 33 synthetic storms which were specifically selected to produce an accurate stage frequency curve. Knowing the frequency of each synthetic storm, the resulting elevation was used to create Figures C-82a to C-82h. Figures C-82a to C-82h represent future conditions with the proposed tunnel and storm surge levees in place and future conditions without a project. An increase in water levels of 0.89 foot was used to estimate sea level rise from 1950 to the year 2050 (future conditions).

The future "without project" scenario represents the year 2050 without a tunnel, without storm surge levees and with the 1 year flows of 5,285 cfs in the Passaic River and 1,630 cfs in the Hackensack River.

The "with project" scenario represents the year 2050 with a tunnel and storm surge levees in place, a flow of 1,000 cfs in the Passaic River, a flow of 6,000 cfs in the tunnel, and a flow of 1,630 cfs in the Hackensack River.

Note that the stage-frequency distributions do not include the joint probabilities of significant river flows. This will be further discussed in subsequent paragraphs below. The effect of the river flows in the lower reaches of the system were insignificant and the stage-frequency curves in these areas are accurate. However, the effect of river flows on the stage-frequency curves in the upper portions of the Passaic and Hackensack Rivers were significant and the stage-frequency curves as developed here represent a purely coastal condition. The main objective of the Storm Surge Study was to establish the design elevations and limits of the storm surge levees. All the proposed storm surge levees systems are within the lower reaches of the river where the effects of river flows are insignificant on establishing the finite levee heights.

25.11.2. Fluvial and Storm Surge Below Dundee Dam

As noted above, the storm surge model developed the purely coastal condition curves and ignored the river flooding from the Passaic. This approach was sufficient for the water levels in and around Newark Bay but not appropriate upstream on the Passaic River. At some point along the river depending on the event under analysis, the water levels would become equally influenced from both conditions and ultimately switch to fluvial dominance. The following paragraphs describe how the final combined stage frequency curves were derived.

Storm surge profiles were input into HEC-DSS for the Passaic River just as UNET writes maximum water profiles to HEC-DSS. Data was then developed with MATHPK that calculated coastal storm surges and hence storm surge only stage-frequency curves. These curves were retained and used in the next steps of the investigation.

As previously noted under the Section 24.0. Hypothetical Peak Discharges and Stages - Unimproved Conditions; MATHPK had already calculated the stage-frequency curves for the fluvial-only condition. These 45 curves were awaiting the storm surge stage-frequency curves for final combining using the joint probability approach. To aid in this analysis, an in-house software program was written to combine the separate fluvial-only and storm surge-only stage-frequency curves, thereby producing the Final Stage-Frequency curves for this area. This process was repeated for each condition:

a. Existing 1992 without project;
b. Existing 1992 with project in place;
c. Future 2050 without project; and
d. Future 2050 with project in place.

Sample stage-frequency curves (subset of the above 45 curves) for the area below Dundee Dam demonstrate indicate how the curves combine and where storm surge and fluvial conditions dominate. As can be seen on Figures C-83a, the lower reaches of the Passaic River (vicinity of the proposed Hurricane Levees) are completely dominated by the storm surge curve and the levees are not at all influenced by fluvial flooding. As one moves upstream, as seen on Figures C-83b, tidal and fluvial conditions become equally important and both play significant roles in determining the combined Final Stage-Frequency curve. However, at the upper reaches of the Passaic and Saddle Rivers, storm surge is relatively insignificant and the curves become fluvially dominated. See Figure C-83c. Depending on location, principal flooding could occur from one or multiple sources. All final curves, as developed as described above, were provided to economics for project benefit evaluation.

E. Hackensack River Basin Modeling and Analysis

26. Hackensack River Basin

Within the Hackensack Basin (198.76 square miles at the mouth) there are four existing water supply reservoirs (see below) which are operated, in general, to maintain constant pool elevation. This operation during the smaller observed hydrographs results with no change to the inflow and outflow hydrographs. For larger floods, this scenario holds true until spillway capacity has been reached and water is stored in the reservoir as surcharge storage. The storage-outflow relationships used in the current HEC-1 model reflect these operating constraint.

The reservoirs and their dates of impounding water are as follows:

a.	De Forest Lake	February 1956
b.	Lake Tappan	1966
c.	Woodcliff Lake	1905
d.	Oradell Reservoir	1922

26.1. HEC-1 Model Development

The computer program HEC-1 was selected for use in developing the hydrologic basin response within the entire Hackensack River Basin. HEC-1 was used to generate runoff hydrographs for all locations and subbasins.

The Hackensack River was subdivided in its entirety and the HEC-1 model as developed is considered operational to the confluence of the Hackensack River and Overpeck Creek (150.89 square miles). Due to unsteady flow regime existing within the lower Hackensack Basin (tidally influenced area, vast wetland storage areas with wide overbanks) a more sophisticated modeling technique (either 2 or 3 dimensional) will be required to accurately determine the peak flows and stages below Overpeck Creek within the river.

The descriptions of the Hackensack Basin that follow divide the basin into two sections, namely the upper and lower basins. Model development in the upper basin has been extensively performed with calibration and verification runs made with gaged data. The HEC-1 subbasins delineated within the lower portion were used to provide inflows to the hydrodynamic model of Newark Bay and the lower portions of the Passaic and Hackensack River Basins. The HEC-1 subbasin delineations for the entire watershed (both upper and lower portions) are shown on Figure C-84a.

The HEC-1 upper basin model was developed, calibrated and verified. Four (4) existing USGS gages are located within the

upper portion of the Hackensack Basin as shown on Figure C-84b, Network Diagram -Hackensack River -Upper Basin. The gages within this region of the basin are:

a. Hackensack River at West Nyack, NY (29.4 sq mi);
b. Hackensack River at Rivervale, NJ (58.0 sq mi);
c. Pascack Brook at Westwood, NJ (29.6 sq mi); and
d. Hackensack River at New Milford (113 sq mi).

Basin wide calibration to the gaged data has been completed with the unit hydrograph optimization performed. Basin wide calibration is documented in subsequent paragraphs. The lower HEC-1 portion of the Hackensack River Basin is shown on Figure C-84c.

The upper Hackensack River Basin has been subdivided into 18 subbasins, as shown on Figure C-84b. Pertinent data for the subbasins are given in Table C-42a. Subbasins vary in size from 27.50 square miles (above De Forest Lake) to 0.60 square mile. Subbasin areas have been determined, main channel slopes calculated and Tc and R values placed in the upper Hackensack HEC-1 model. To be consistent with the Passaic Basin's HEC-1/UNET models, a time step of 1 hour was selected. This time step appears to be adequate and can define the rising limb of flood hydrographs with sufficient ordinates.

26.2. HEC-1 Optimization Analysis - Upper Basin

Rainfall, streamflow and reservoir data were collected for seven recent storm events over the Hackensack River Basin. Storms for the optimization study were selected based on sufficient data to develop both individual optimized subbasin parameters as well as assist in the establishment of regional basin-wide parameters. The regional values would be used as a guide in the definition of the Tc and R for the ungaged subbasins. The optimization analysis performed consisted of analyzing 4 basin wide storm events over the Hackensack River Basin. The storms used in the optimization analysis were the following:

	Storm Event	Basin Average Rainfall
a.	April 1984	3.7 inches
b.	May 1984	6.4 inches
c.	April 1987	2.7 inches
d.	May 1989	4.4 inches

Mass curves of rainfall for the above storms are shown on Figures C-85a thru C-85d. These events caused both rises in stream and reservoir stages, and were considered representative of the existing conditions basin response for the Hackensack River.

Streamflow data were collected from the USGS, while reservoir inflow, outflow and stage hydrographs were collected from the Hackensack Water Company, owner and operator of the 4 existing water supply dams in the basin. Reservoir data were collected for De Forest Lake, Lake Tappan, Woodcliff Lake and Oradell Reservoir. All reservoirs except Woodcliff Lake are on the mainstem of the Hackensack River, with Woodcliff Lake located on Pascack Brook. See Figure C-84b.

During the optimization analysis, 24 computer simulations were performed at eight locations within the Hackensack Basin. The locations under analysis were the 4 existing dams/reservoirs and the 4 USGS gages. Results of the optimization analysis are given in Table C-42b and shown on Figures C-85e to C-85i.

Optimization runs were first simulated at Oradell Reservoir to determine if the Hackensack Basin above the reservoir could be modeled with one large subbasin. Oradell Reservoir inflows were back calculated from reservoir pool levels and spillway rating data. As such, optimizing to the inflow hydrograph proved to be quite difficult with optimized Tc's, see Table C-42b, seemingly low for a basin of 113 square miles that has known effects from upstream reservoir operation (refer to Figure C-84b for the Hackensack River Basin network diagram). After attempting to optimize the April 1984 and May 1989 events, it was concluded that the basin would need to be further subdivided into smaller subareas and optimization runs would concentrate at the existing USGS gages.

At the West Nyack, NY gage (downstream of De Forest Lake), optimization simulations were made for the April 1984, May 1984 (Figure C-85f) and May 1989 flood events. As can be seen on Figure C-85b, the mass curves of rainfall display three intense rainfall periods, while on Figure C-85f three subsequent rises in streamflow were recorded at this gage. HEC-1 simulated this event fairly well considering its complex rainfall distribution. The peak, volume and timing relationships that were optimized with HEC-1 agreed quite well with observed data and the results obtained were considered reasonable.

At the Rivervale, NJ gage (downstream of both De Forest Lake and Lake Tappan), optimization simulations were made for the April 1984, April 1987 and May 1989 (Figure C-85g) flood events. The gaged data is somewhat influenced by Lake Tappan just upstream, as a broad flat peak hydrograph can be observed at the gage. The results, as given in Table C-42b indicate a more reasonable and consistent relationship between Tc and R at the gage, than at Lake Tappan. In most cases the peak, volume and timing relationships optimized with HEC-1 agreed fairly well with observed historical data.

On the Pascack Brook at Westwood, NJ, optimization runs were performed for the April 1984 and April 1987 (Figure C-85i) flood events. Computed Tc and R values for the 2 storm events were close, and the simulations agreed quite well with observed data.

26.3. Reconstitution for Optimization Analysis

Flood hydrographs were reconstituted with the average of the optimized Tc and R values for 3 locations in the basin. The Tc and R values used at each of the gages, storm event and reconstituted results are provided below:

	Average		Storm	Reconstitution Results		
Location	Tc	R	Event	%Peak	%Volume	Timing
West Nyack	7.6	19.0	Apr 87	+13.8	+3.5	-2.0
Rivervale	25.6	21.4	May 84	+10.0	-10.1	+4.0
Westwood	16.0	7.0	May 89	+7.1	-1.6	0.0

Hydrograph plots as shown on Figures C-85j and C-85k show good matching results for the West Nyack and Rivervale gages. Volume calculations are reasonable when one compares the hydrographs and complex storm events.

26.4. Basin-wide Calibration and Verification

Basin-wide modeling efforts were focused on the 13 subbasin model above the New Milford gage. Once having completed the optimization study, regional unit graph values were established that would be used as a guide for Clark parameters at ungaged subbasins. Initial and modified Tc and R values were placed in the model, as well as Muskingum routing coefficients for downstream channel routing reaches. In areas where previous dam safety reports were prepared for the Corps, the storage outflow relationships were used for channel routing guidance. Two storms, April 1984 and May 1989, were selected to calibrate the basin wide model.

A composite unit hydrograph was constructed at an optimized gage location so that the upstream subbasins and channel routing parameters could be adjusted to match the optimized unit hydrograph. The HEC-1 model was run with no loss or base flow and one inch of rainfall was applied in one hour. Figure C-85l displays this technique for the unit hydrograph at the Westwood gage. As can be seen on this figure, curve 1 shows the inflow hydrograph from the 19.4 square mile subbasin into Woodcliff Lake. No storage at the reservoir occurred and curve 2, the reservoir routed outflow hydrograph, equals curve 1. Curve 3 represents the Woodcliff Lake outflow hydrograph channel routed downstream to the Westwood gage. Curve 4 represents the local 10.2 square mile subbasin between Woodcliff Lake and Westwood gage. Curve 5 represents the composite developed unitgraph and

would be directly compared to curve 6, the HEC-1 optimized unit hydrograph. As can be seen, curves 5 and 6 are quite close and the two subbasin model should reflect the basin response well.

The April 1984 flood event was first used to develop the basin wide model. Unit hydrograph parameters were entered into the model as noted in Table C-42a along with estimates of base flow and rainfall losses. For the Hackensack Basin, uniform and hourly losses were used since curve numbers were not determined. For this event initial losses were assumed to be 0.20 inches and hourly losses were 0.06 inches per hour. As can be seen on Figures C-85m to C-85o, the 13 subbasin model reproduced the April 1984 flood well and was within 10% of the estimated peak at New Milford.

The May 1989 flood was also selected to be used as a calibration event for the basin-wide model. For this event initial losses were 0.50 inches and hourly losses were 0.10 inches per hour. Results of the simulation for this event are shown on Figures C-85p to C-85r and results indicate that the model can reproduce the historical event.

As the final test of the 13 subbasin model, the May 1984 flood was selected to verify the basin-wide model. For this case the unitgraph parameters, routing coefficients and reservoir routings were held constant with only the base flow and rainfall losses altered. Results indicate that the basin wide model can reproduce other historical floods. As such, the model was considered acceptable for analysis of the hypothetical floods and associated volume-duration analysis downstream to the confluence of the Hackensack River and Overpeck Creek.

26.5. HEC-1 Model - New Milford to Overpeck Creek

The incremental watershed in this portion of the Hackensack River was divided into 5 subbasins. Three subbasins were Hirshfeld Brook, Coles Brook and Overpeck Creek. The remaining two subareas were incremental flow subbasins from which numerous storm sewers or minor tributaries are contained. Pertinent data for these subbasins are provided on Table C-42a.

These incremental areas are similar to those described below under "Upland Areas". The modeling approach used was the same, however the hydrographs were not treated as single-point outflows as was the case below Overpeck Creek. Instead, following generation of subarea runoff, each hydrograph was split into two segments with one half being input at the upstream end of the subarea and the remaining half entered at the downstream end. This approach was used to more accurately reflect the distribution of runoff within the subbasin. Pertinent data for all subbasins below Overpeck Creek are provided in Tables C-42c and C-42d.

For the tidal Hackensack River reaches between New Milford and Overpeck Creek, the analysis attempted to minimize the existing dynamic effects through judicious use of HEC1 capabilities. Above Overpeck Creek tidal movements are essentially linear due to the relative narrowness of the river channel and the lack of extensive lateral wetlands with well developed channels. The rise and fall of the tide implies that there is no unique, single-value storage outflow relation for this portion of the Hackensack River, but it was felt that a generalized relation could be developed which would adequately reproduce the basin response in this reach especially since Overpeck Creek is rather remote from Newark Bay. This area also has a very small tidal prism compared to the lower Hackensack Basin further downstream.

Flow line computations from which to estimate the generalized storage-outflow relation for the reach above Overpeck Creek do not exist. The decision was therefore made to take advantage of the relation inherent in Muskingum methodology, which does not require detailed physical information but must reflect travel times appropriate to the magnitudes of the upland runoff under consideration. These travel times were projected from an estimate of the average velocity just above Overpeck Creek during an ebb tide. This average velocity was computed from the following:

(a) Volume of tidal prism above Overpeck Creek (485 acre-feet);

(b) Duration of the falling tide (6 1/2 hrs);

(c) Approximate average cross-sectional flow area (3000 sq. ft.).

Initial estimates of travel times during flood runoff were determined by incrementing the mean ebb velocity (0.3 ft/sec) by the ratio of six-hour flood runoff volume to prism volume. This approach made no allowance for the likelihood that the average outflow cross-sectional area would change, that actual average flow rates would vary between New Milford and Overpeck Creek, or that the tidal prism volume is not evenly distributed over the routing reach. The assumption of an unchanging cross-section through which flood runoff must pass is one way of accounting for the higher velocities to be expected as flood magnitude increases and for the shorter flood wave travel times associated therewith.

The Muskingum "X" adopted for the channel routings was 0.35. This was adopted as being consistent with the relatively rapid passage of flood waves implied by the initial estimates of "K". Muskingum "X" was not changed following adjustments to the "K" value because it was felt that the outflow to the estuary below Overpeck Creek should be conservative in order to reflect a worst case situation at Newark Bay.

The initial reach travel times based on estimated velocities general resulted in Coles and Hirshfeld Brooks and incremental area runoff hydrographs appearing as spikes on the leading edge of main stem Hackensack hydrographs. This was not considered reasonable and travel time were adjusted upward until the spikes essentially disappeared. Final time of travel estimates used in this analysis are provided in Table C-42e.

Plotted on Figures C-85s through C-85u are the flood hydrographs in this area for the 2, 20 and 100 year runoff. These hydrographs represent the downstream limit of conventional HEC-1 modeling. Peak flows determined for the HEC-1 conventionally modeled area are provided in Table C-42f.

26.6. HEC-1 Model Development - Lower Basin

With the upper portion of the HEC-1 model simulating basin response to the confluence of the Hackensack River and Overpeck Creek (150.89 square miles of the 198.76 square mile basin), the lower portion of the HEC-1 model (47.87 sq. mi.) was developed similar to the Passaic HEC-1/UNET model interrelationships. Lateral or point flows (either upland, wetland or waterway hydrographs) were prepared for transfer with HEC-DSS to a 2-dimensional numerical model at WES. Final flows and stages below Overpeck Creek are documented in the Hydrodynamic Modeling effort for Newark Bay and lower Passaic and Hackensack Rivers. The following paragraphs describe the logic of subdividing the remaining portion of the Hackensack Basin.

Tidal movements downstream of Overpeck Creek are much more complex than upstream due to the lateral extent of numerous wetlands, within which there are well defined channels (both natural and artificial). Many of the artificial channels were constructed to drain the wetlands and were arranged in grids which frequently intersect more than one natural stream. These multiple intersections preclude the application of routing techniques since multiple outflow relationships would be required, each a function of downstream reaches. Figure C-84c shows the lower Hackensack Basin and the subbasins developed that were input as inflow points to the hydrodynamic model of the area.

26.7. HEC-1 Model -Lower Basin -Upland Areas

Upland areas are defined as areas lying above Elevation 10 feet NVGD. Eight (8) upland subbasins (including Wolf Creek) were delineated for this effort. These areas are generally found along the eastern and western edges of the meadowlands and are thin strips of land that are narrow in the direction of the steepest slope. This results in numerous storm sewer systems that are not heavily branched and drain directly to the Hackensack Meadowlands. Pertinent data on these subbasins are provided on Tables C-42c and C-42d. The subbasins are shown on Figure C-84c.

26.8. HEC-1 Model - Lower Basin - Wetland Areas

Wetland subareas are those areas in marshlands that are interconnected through a series of channels, both natural and artificial. For the purposes of this analysis, these areas were defined as lying between Elevation 0.0 and 10.0 feet NGVD. The 10- foot contour lies somewhat above the actual meadowland perimeter since most of the wetlands probably lie below Elevation 7.0 feet. However, Elevation 10 feet NGVD was a convenient dividing line since it appears readily on USGS Quadrangle maps at a scale of 1:24000. The inclusion of the additional area should be of little concern because of the general remoteness of the 7.0 to 10.0 foot elevations from Newark Bay. Ten (10) wetland area subbasins (including 3 contiguous water areas) were delineated with pertinent data provided on Tables C-42c and C-42d while Figure C-84c displays the locations of the subbasins.

26.9. Hackensack Basin-Volume-Duration-Frequency Curves (VDFC)

Peak discharges recorded at the various gaging stations may be slightly lower or higher than they would be without the four reservoirs with gate controlled spillways. For this reason, discharge-frequency relations based upon gage records were considered to be modified by the presence of the reservoirs and therefore not reliable for targeting HEC-1 peak flows. The volume-duration frequency curves developed at gaged locations would be considered more accurate due to the longer time base for their determination. It should be noted that Hackensack peak flow is significantly altered as the flood enters the wide meadowlands downstream of the New Milford gage. Thousands of acres of wetlands are available for use as storage and the peak is significantly reduced as the flood wave travels to the mouth of the Hackensack River at Newark Bay. Since no structural improvements are included in the proposed project on the Hackensack River (except the tidal levees near the mouth), runoff volumes were considered to be more critical item than peak flows studying the impacts from the Passaic project on the Hackensack River.

As part of the overall Hackensack Basin analysis, a detailed investigation was performed on the volume-duration data available at the four (4) gaged locations within the basin. Following is pertinent data for four Hackensack River gages, as shown on Figure C-84b:

	Gage Location	Drainage Area (Sq Mi)
a.	Hackensack River at West Nyack, NY	29.4
b.	Hackensack River at Rivervale, NJ	58.0
c.	Pascack Brook at Westwood, NJ	29.6
d.	Hackensack River at New Milford, NJ	113.0

A study was performed to assist in the calibration of the flood volumes that were generated with HEC-1 for hypothetical storms. This effort concentrates on the fluvially dominated areas above the confluence of the Hackensack River and Overpeck Creek, since discharges and stages downstream of this site are significantly influenced from tidal flows and storm surges. In the overall project design, runoff from the Hackensack Basin would combined with Passaic River runoff, along with storm surge. Although peak flow from the Hackensack would be important, runoff volume would be more critical to the stage determinations in and around proposed project features.

Due to construction of several reservoirs within the basin, a concurrent period of record was sought that included the effect of all reservoirs on stage and flow data. The concurrent period of record was determined to be water years 1967-1990, inclusive. Since the dams are operated for the purpose of water supply and have spillway gates, optimization of the individual basins would be complicated by varying reservoir storage (if the reservoirs were low at the start of the flood event) and the gate operations made during the flood event. The flood-volume duration curves would be used as a check on the hypothetical volumes that would be expected to flow past the individual sites under study.

26.9.1. Analysis

Mean daily flow data at each of the four (4) gages were categorized into the highest mean flow for durations of 1, 2, 3, 4, 5, 7, and 10 days for each water year. All processed duration data was checked and verified for consistency prior to the generation of the statistical parameters. Care was also exercised to ensure data compatibility due to upstream regulation at dams. Flow duration data used to generate the statistical parameters are given on Tables C-42g thru C-42j. Mean, standard deviation and skews were generated for each duration at each gage. Results of the statistical analyses are given in Table C-42k.

The basic approach used for the numerical analysis was taken from EC 1110-2-249, "Hydrologic Frequency Analysis", dated 5 June 1985. As discussed in paragraph 3-08c of Chapter 3, the probability distribution function recommended for flood volume duration frequency computations was the Log Pearson Type III distribution.

To ensure that volume-duration curves would not cross, the variations in standard deviation and skew were plotted against the mean for each gage, as shown on Figures C-86a thru C-86d. Statistical data used to plot the figures are given in Table C-42k. Paired data for the standard deviation and skew plotted against the mean at each gage were input into a least square regression analysis. With the new statistically developed mean, standard deviation and skew, for each duration at each gage, frequency curves could then be inspected for consistency.

26.9.2. Results

With the smoothed statistical parameters developed and given in Table C-421, frequency curves were developed for each duration at each gage. As shown on Figures C-87a thru C-87d the volume-duration frequency curves show no inconsistencies.

With the curves checked for consistency, the final step was to develop the cumulative volume-duration frequency curves. This was accomplished by multiplying the mean discharge values by the appropriate duration. These values were also adjusted to plot the results in 1000's of acre-feet for each specific frequency at each gage. Results of this effort are shown on Figures C-88a thru C-88d.

The hypothetical runoff modeling with the upper basin HEC1 model was performed and flood volumes were checked at the New Milford gage. Losses in the basin were adjusted to provide a smooth volume relationship throughout the range of frequencies. The 100- year HEC-1 derived hydrograph agrees closely with the historically developed volume duration data in a time distributed fashion, as shown on Figure C-89. This implies that the runoff volumes developed by the HEC-1 model are correctly adjusted and volumes entering at the upstream limit of the hydrodynamic model will be accurate.

F. Improved Conditions

27. Description of Improvements

27.1. General

The Passaic River Flood Damage Reduction Project would reduce flood levels at major damage areas along the Central and Lower Passaic River and the Pompton River. The proposed project would consists of 21.7 miles of large underground diversion tunnels, five gated hydraulic structures, preservation of 5,350 acres of natural flood storage areas, 7.1 miles of channel modifications, 6.8 miles of levees, 12.9 miles of floodwalls, and some minor floodproofing. In the Central Basin area of the Passaic, six levee/floodwall systems are proposed for construction along with one levee/floodwall system which is integral to the main tunnel inlet on the Pompton River. In the Newark Bay area, three levee/floodwall systems have been designed and included as part of the overall flood damage reduction plan. Figures C-90a to C-90kk illustrate the plan view of the project layout. The project features are described in more detail below.

27.2. Tunnels - General Tunnel Description

The principal feature of the proposed project is the dual inlet diversion tunnel. The main tunnel would be about 20.4 miles long, with a 42 foot inner diameter. Damaging floodwaters

would be diverted from an inlet at the upper Pompton River through the main tunnel to an outlet near the mouth of the Passaic River in Newark Bay.

A smaller spur tunnel would convey excess flood waters from the Central Passaic Basin area into the main tunnel through an underground connection. The spur tunnel would be approximately 1.3 miles long with a 23 foot inner diameter.

Both tunnels would be constructed entirely in bed rock and located between 110 and 450 feet below the ground surface. Excavation would be done primarily with Tunnel Boring Machines (TBMs), however some drill and blast construction would be used in a few unique areas. The tunnel is expected to be lined with 15 inches of cast-in-place concrete. A precast concrete liner could also be used but, the inner diameter of the tunnel would have to be increased by 1.5 to 2.0 feet (43' 9" inner diameter.) and the high velocity inlet sections would still have to be constructed with a cast-in-place liner. Profiles of the tunnels are shown on Figure C-91a and C-91b.

The project has a very high probability of eliminating all significant flood damages for storms up to a 100 year return period. In addition the project would greatly reduce flood elevations even when the tunnel capacity was exceeded. The largest areas benefiting from the tunnel and associated surface features would be the Passaic River from Dundee Dam in Clifton to the Pompton confluence in Fairfield, the entire Pompton River and the lower Ramapo, Pequannock and Wanaque Rivers in Wayne. Reductions in the 100 year flood would range from 4 to 7 feet on the Passaic River and would be as great as 10 feet on the Pompton, Pequannock, Ramapo and Wanaque Rivers. Further documentation of the flood reduction due to project implementation can be found in Section 29., Stage and Flow Reductions - Results of Project Improvements.

At the Pompton Inlet, the tunnel would skim between 0 and 30,000 cfs of excess flood waters out of the river while allowing between 4500 and 7000 cfs to bypass the inlet. The flow which would be allowed to bypass the Pompton Inlet (4500 to 7000 cfs) is between the existing 1 and 2 year frequencies. Generally, the bypass flow would increase as the size of the storm increases. Bypass flows of that magnitude would be out of bank but would not cause damages.

At the Two Bridges location, the spur tunnel would divert between 0 and 18,000 cfs of excess flood waters out of the river and would convey the flow into the main tunnel. Between 5500 to 6500 cfs would be allowed to bypass the Spur Tunnel Inlet. The flow which bypassed the Two Bridges Inlet (Spur Tunnel Inlet) once again would range between the existing 1 and 2 year frequency and would generally increase with the size of the storm. Bypass flows of that magnitude would be at about bank full capacity.

In plan view the tunnel would not follow an absolutely straight line from the Pompton Inlet to the outlet in Newark Bay. See Figure C-91c. Several extensive studies and cost analyses have been conducted to determine the optimal tunnel alignment. These studies concluded that the need for suitable workshaft locations and construction staging areas, far outweighs any savings associated with small reductions in the tunnel length due to straightening the alignment. Therefore, the tunnel would travel from one suitable workshaft location to the next in a generally direct path from the Pompton Inlet to the outlet. The workshafts are described further in the later paragraphs of this section.

27.2.1. Pompton Inlet

The Main Tunnel or Pompton Inlet would be connected directly to the 42 foot diameter tunnel and would be located at the head of the Pompton River near its confluence with the Ramapo and Pequannock Rivers. The proposed location is on the left bank of the Pompton River, upstream of the Pompton Plains Cross Road (Jackson Avenue) Bridge in Wayne Township. The location of the inlet is shown on Figure C-90a.

The surface structures would consist of a semi-circular gated diversion spillway, an access basin, an inner weir and a sloping tunnel inlet. There would be 11 vertical lift diversion gates, each 60 feet wide by 12 feet high with the crest of the gate sill at Elevation 164.0 NGVD. The gates would divert and regulate flow into a 216 foot radius semi-circular access basin which would be excavated to a depth of about 20 feet. The inner weir would be the highest point on the sloping drop into the main tunnel. The drop inlet would slope and converge from the 125 foot radius semi-circular inner weir to the 26 foot radius circular main tunnel chamber, some 155 feet below. Detailed drawings of the Pompton Inlet are shown on Figures C-92a thru C-92c.

The configuration of the inlet was modified from the "Morning Glory" type inlet which was proposed in the Phase GDM report to the presently proposed "Semi-Cone" shape. The profile of the proposed tunnel (shown on Figure C-91a) illustrates that much of the tunnel is below the outlet and that most of the tunnel slopes are very flat. Therefore, sections of the tunnel would always be full unless pumped out for maintenance and inspection purpose and air entrained in the flow that collects in the crown may remain trapped in the tunnel. Air trapped in the tunnel crown would reduce the tunnel capacity and may escape with explosive forces when it reaches an open vertical shaft.

The "Semi-Cone" inlet design limits inflow to only one side of the inlet while permitting air to escape upward along the opposite side. The new "Semi-Cone" type of inlet should entrain less air, better facilitate the removal of air and should display

superior performance over a range of flow conditions than the "Morning Glory" inlet. The new design will be model tested at the Waterways Experiment Station (WES) during the Feature Design Memorandum.

27.2.2. Spur Tunnel Inlet

The Spur Tunnel Inlet would connect to the 23 foot diameter spur tunnel and would be situated along the left bank of the Passaic River between the Two Bridges Road bridge and the Interstate Route 80 bridge. The area is just below the confluence with the Pompton River and is located in the township of Wayne. To utilize this site, a bridge for Fairfield Road would be built across the approach channel of the inlet structure. The location of the inlet is shown on Figure C-90f.

The surface structures, which are similar to the Pompton Inlet, would consist of a straight gated side channel spillway, an access basin, a semi-circular inner weir and a sloping tunnel inlet. There would be 5 vertical lift diversion gates, each 50 feet wide by 13 feet high with the crest of the gate sill at Elevation 156.0 NGVD. The gates would regulate diversion of flow into a 290 foot wide access basin which would be excavated to a depth of about 20 feet. The inner weir would be the highest point on the sloping drop into the spur tunnel. The drop inlet would slope and converge from the 75.5 foot radius semi-circular inner weir to the 15 foot radius circular spur tunnel chamber, some 160 feet below. Detailed drawing of the Spur Inlet are shown on Figures C-93a thru C-93c.

The Spur Tunnel Inlet also uses the new "Semi-Cone" type of design but it contains a straight approach access basin. Once again by limiting only one side of the inlet to inflow and by permitting air to escape up the other side, the "Semi-Cone" inlet should perform better than a "Morning Glory" type inlet. This inlet would also be model tested at WES during the Feature Design Memorandum (FDM).

27.2.3. Tunnel Outlet

The tunnel outlet would be located in the upper end of Newark Bay, where the Passaic and Hackensack Rivers meet. See Figure C-90h. The diverted excess floodwaters would flow from a depth of 409 feet vertically into the outlet structure. The outlet structure would be located about 1,850 feet offshore and would extended from a depth of about 20 feet below sea level to about 10 feet above sea level. The outlet would contain three 26 foot wide by 30 foot high, vertical lift gates that should distribute flow through an angle of about 700 and across the full channel depth of 30 feet. If one of the gates were to fail to open during a large event, flow would still be able to exit the tunnel through a 140 foot long semi-circular overflow section located at the back of the outlet. While the overflow section, located at an elevation of 10 feet NGVD (National Geodetic

Vertical Datum) is not as efficient as the gated sections, it does provide an emergency spillway outfall. Figures C-94a and C-94b illustrate the tunnel outlet.

Flow from the tunnel outlet would be discharging into a navigation channel. To determine if the outlet would have any impact on ship performance, a physical model study and a ship simulation model will also be conducted in the FDM.

27.2.4. De-aeration Measures

Several locations in the tunnel were selected to vent air out of the tunnel during diversion. The two primary locations are the tunnel inlets, however stations 783+00, 483+98 and 20+80, have also been selected along the main tunnel as good locations to purge air.

At each inlet a special de-aeration chamber has been designed. Air would be entrained at each inlet by hydraulic jumps when water levels in the tunnel are low and by plunging flow when water levels are higher. The diameter of the de-aeration chamber would be larger than the diameter of the tunnel to provide additional capacity when the flow is "bulked-up" with air. A vertical vent shaft would be placed at the optimum location in each de-aeration chamber.

At the Pompton Inlet the de-aeration chamber would be 500 feet long, 52 feet in diameter and would have a 15 foot diameter vent shaft about 200 feet downstream of the drop inlet. The Spur Tunnel Inlet de-aeration chamber would be 420 feet long, 30 feet in diameter and would have a 12 foot diameter vent shaft about 175 feet downstream of the drop inlet. Most of the inlet drawings also show the de-aeration chamber.

The remaining three 15 foot diameter vertical vent shafts would be at the following locations. One vent would be located at station 483+98 which would be a localized high point due to a sharp change in the tunnel slope at that point. There would be another vent about 200 feet downstream of the junction between the main and spur tunnels at station 783+00. There would be one more vent at Workshaft #2C (Station 20+80) which is just before the outlet.

27.2.5. Shafts

a) There would be several vertical shafts located along the tunnel alignments which would be constructed for a variety of reasons. Some of the uses would be the entrance and exit of personnel and construction materials, muck removal and air venting. There would be a total of 13 shafts including the 2 inlets and the outlet shaft.

b) Pompton River Inlet (Station 1077+47). Currently this area is a topsoil manufacturing operation with material stockpiled on and adjacent to the site. This site would be used as a major construction workshaft and it would be the Main Tunnel inlet once the project is completed.

c) Pompton Inlet Vent Shaft (1075+46). This 15 foot diameter shaft would be only 200 feet to the southeast of the main shaft of the Pompton Inlet. The shaft may be utilized during construction but its primary purpose would be to purge air from the de-aeration chamber during tunnel operation.

d) Workshaft #3 (Station 843+40). This 42 foot diameter shaft would be used to remove two tunnel boring machines (TBMs).

e) Ventshaft #6 (Station 785+75). This 15 foot diameter shaft would be located about 200 feet downstream of the junction of the Spur and Main tunnels. The purpose of this shaft is to vent air entrained by the highly turbulent flow at the junction.

f) Workshaft #2 (Station 623+76). Currently this site is a quarry which is located next to Montclair State College. This location would be used as a major construction site. Muck from two different tunnel borings would be removed through this 42 foot diameter shaft and tunnel muck could be stock piled at the site if necessary. Materials and equipment for construction of the tunnel would be moved in and out of this shaft. Upon completion of the tunnel system, permanent elevators and stairs would be installed to facilitate inspection and maintenance. In addition an operations/visitors center would be constructed at this site. Concept designs at this stage have been developed assuming this locations would house the main operation center.

g) Shaft #5 (Station 483+98). This 15 foot diameter shaft would be situated at a localized high point in the tunnel. The purpose of this shaft would be to purge air from the tunnel during the filling process. However, during construction this shaft would also be used as an access point to disengage the cutter heads from two TBMs. This hole could be referred to as a "vent and hook hole".

h) Workshaft #2B (Station 163+15). This 42 foot diameter shaft would serve as a major construction shaft. A TBM and tunnel muck would be removed from this hole and equipment and materials for construction would also be transported through this shaft.

i) Workshaft #2C (Station 20+80). Two shafts would be constructed at this sight which is located at the southern end of Kearny Point. One large shaft (42 feet in diameter) would be used to remove tunnel muck and to provide access for personnel, equipment and materials during construction. Once tunnel construction was completed, this shaft would be used to house dewatering pumps, pipes and other related equipment. Permanent

stairs and an elevator would also be provided for maintenance and inspection purposes. A permanent surface structure would also be constructed to house the pump controls and other maintenance equipment. The second shaft would only be 15 feet in diameter and would serve as the final vent to purge air from the tunnel during operation.

j) The Tunnel Outlet (Station 0+00). This shaft would be located about 1850 feet off shore and would taper from 42 to 45 feet in diameter. The gated outlet structure could be constructed at a nearby dock and then floated into position and sunk in place. The completed outlet structure would be used to discharge all flow entering both tunnels.

k) Spur Tunnel Inlet (Spur Station 69+75). This workshaft would be used to remove the spur tunnel TBM and it would be the Spur Tunnel Inlet once the project was completed.

l) Spur Tunnel Inlet Vent Shaft (Spur Station 68+00). This 12 foot diameter shaft would be about 175 feet to the east of the main shaft of the Spur Tunnel Inlet. The shaft may be utilized during construction but its primary purpose would be to purge air from the de-aeration chamber during tunnel operation.

m) Spur Tunnel Workshaft #4 (Spur Station 3+68). This workshaft would be used to remove tunnel muck from the spur tunnel and to move personnel, equipment and materials in and out of the spur tunnel.

27.2.6. Operations Centers

Two operations centers would be constructed as part of this project. The Main Operations Center would be located at the Montclair Workshaft (#2). All the rain and stream gages in the basin would be monitored from this location. Operation, monitoring and video surveillance of all the gated structures (Pompton Inlet, Spur Inlet, Tunnel Outlet, Pequannock Weir, and the Great Piece Weir) would be conducted from this center. Maintenance equipment, personnel, offices, a conference area and a visitors center would also be housed at this location. The elevators and stairs leading into the tunnel would also be part of this building. While operation of all the gated structures would normally be conducted from this location, secondary operational controls would be provided at each structure.

A Pump Station Operations Center would also be constructed as part of this project. While the pumps would be housed at the bottom of Workshaft #2C in Kearny Point, the Pump Station Operations Center would be located on the surface above the pumps. The dewatering pumps would be controlled from the Surface Facility. The pumps are designed to evacuate the water from the lower section of the tunnel in approximately 10 to 14 days, for maintenance and inspection purposes. Other maintenance equipment, personnel, offices, and a conference area would be

housed in the Pump Station operations center. Elevators and stairs into the tunnel would be part of this center. Figures C-95a and C-95b illustrate the pump station.

27.3. Associated Tunnel Surface Features

27.3.1. Pompton Inlet Channels and Levee

27.3.1.1. General. There are three major tributaries to the Pompton River which are located upstream of the Pompton Inlet. These tributaries are the Ramapo, Pequannock and Wanaque Rivers. Since the entire Pompton River would be located below the point of diversion, flood elevations along the Pompton would lowered significantly. However, an old hydraulic control system (the Morris Canal Feeder Dam) is presently located upstream of the proposed diversion inlet. The Morris Canal Feeder Dam System maintains elevated water levels in the lower portions of the Ramapo and Pequannock Rivers. Therefore, these upstream tributaries would not see a reduction in flood levels due to the proposed tunnel project unless additional modifications were made. Hence, several channels, a levee, and a gated structure would be constructed to aid in reducing flood levels along these three upstream tributaries and to maximize flow conveyance to the tunnel inlet. The project features associated with the Pompton Inlet are described below.

27.3.1.2. Pequannock Weir. The existing Morris Canal feeder dam system, which consists of an upper spillway (the Pequannock Weir), a lower spillway (the Ramapo Weir) and a 2450 foot long berm which connects them is considered to be a historic structure. As stated above the existing Feeder Dam System would both limit the amount of flow which gets to the inlet and would increase flood levels upstream. However, since the Morris Canal Feeder Dam is a historic structure, no modifications or alterations to the structure would be made. In addition, placing the tunnel inlet upstream of the feeder dam was considered to be unreliable since elements of the feeder dam system are between 50 and 150 years old. Therefore, the new Pequannock Weir was developed to supplement the existing Morris Canal Feeder Dam System and optimize the proposed tunnel system.

The new Pequannock Weir would be a gated control structure. The new structure would be placed on the right side of the Pequannock River within 200 feet of the existing weir. The structure consists of 4, 50 foot wide by 15.25 foot high, tainter gates. The tainter gates would normally be kept down, or in the closed position, and would only be opened during flooding events greater than the annual flood. See Figure C-96a for the layout and location of the weir and Figures C-96b thru C-96d for details of the structure.

The new Pequannock Weir has two primary purposes. During normal conditions (approximately 97% of the time), it would preserve the existing wetlands by maintaining the water levels

that exist today. During flood conditions the new Pequannock Weir would reduce damaging flood elevations upstream and it would permit flow to bypass the existing Morris Canal Feeder Dam. The structure would meet these goals by regulating the gates to maintain a one year flood level upstream and to bypass excess flow to the tunnel inlet via the Pequannock Bypass Channel.

27.3.1.3. Pequannock River Channel. The Pequannock River channel modifications would extend from the mouth of the Pequannock River at the main tunnel inlet, through the new Pequannock Weir, to a point some 1000 feet below the Paterson-Hamburg Turnpike bridge. The total length of improvements would be about 2.7 miles. The modification includes deepening the existing channel some 3 to 9 feet with the average increase in depth being about 6 feet. All the channels would be trapezoidal in shape with 1 on 2.5 side slopes unless constrained by real estate requirements. The channel excavation would be limited to the bottom and a single bank where possible. Limiting the construction work to one bank would preserve the existing vegetation which shades the river and keeps the water temperatures suitable for habitation. The proposed channel can be divided into three sections: above the Wanaque River Confluence, below the Wanaque River confluence and the Bypass channel which is below the new Pequannock Weir. Figures C-90c thru C-90e show the plan views of the channel work of the improved Pequannock channel and water surface elevations.

The section below the new Pequannock Weir (the Bypass Channel) would be the direct full capacity link between the upper tributaries and the tunnel inlet. This section would be about 1,600 feet in length with a channel slope of 0.08%. It generally would have a 130 foot bottom width, however it may be as wide as 230 feet at both the Pompton Inlet and the Pequannock Weir ends. At the connection with the new Pequannock Weir the channel would be excavated through a field in the right overbank. At the Pompton Inlet end, the bypass channel would connect with 400 feet of channel work along the Upper Pompton River and 600 feet of channel work along the lower Ramapo River. Although the bypass channel would rarely be used, it would generally be wet due to downstream tailwater levels. A small amount of riprap channel protection would be needed in the area of both structures.

The reach above the Pequannock Weir but below the Wanaque River confluence would be about 8900 feet long. The channel bottom width would be generally 100 feet. The channel slope would be 0.19% at the lower end and flattens to 0.04% for the remainder of the reach. The only riprap called for in this section would be a small section just upstream of the Pequannock weir.

The most upstream reach extends about 4,100 feet above the confluence with the Wanaque river. The channel bottom width tapers from 100 feet to 50 feet and the channel slope is 0.04% up

to the point where it starts to ties into the existing invert at which point the slope increases to 0.5%. Riprap and articulated concrete matting have been called for in the tie-in area to prevent head cutting at this location which would be subject to high velocities.

27.3.1.4. Wanaque River. The Wanaque River channel modification would extend from 1.1 miles upstream from its mouth to a point just beyond the Paterson-Hamburg Turnpike bridge. The proposed modification includes deepening the existing channel some 3 to 7 feet and increasing the channel bottom width from 50 to 74 feet. The channel slope would be generally 0.2% until it ties into the existing invert at which point the slope increases to 1.0%. About 650 feet of riprap would be needed upstream at the tie-in, to prevent head cutting. The lower 2,650 feet of channel would be protected with crushed stone or gravel since the sediment transport analysis indicated a trend of degradation in that area. The Wanaque River modifications and water surface elevations are show on Figures C-90c and C-90d.

27.3.1.5. Ramapo River. The major portion of the Ramapo River channel modifications would extend for 1.2 miles, from the newly proposed Pequannock Weir to just beyond the Paterson-Hamburg Turnpike bridge (downstream of Pompton Lakes Dam). The modification would include deepening the existing channel by 1 to 10 feet. The lower 1700 feet of the channel would have a bottom width of 100 feet and an invert slope of 0.09%. The remainder of the channel would have a bottom width of 60 feet and a slope of 0.03%. Almost the entire length (5400 feet) of the modified channel would be protected with riprap.

About 600 feet of the Ramapo River from the Pompton Inlet to the Ramapo Weir (part of the Morris Canal Feeder Dam) will be modified. The main purpose of these modifications is to convey flow efficiently into and around the Pompton Inlet. The modifications consists primarily of lowering the invert about 6 feet to elevation 160.0 feet NGVD. The channel bottom will be about 130 feet wide at the Pompton Inlet and 270 feet wide at the Ramapo Weir. The channel will be trapezoidal in shape with 1 on 2.5 side slopes. The Ramapo River modifications are shown on Figure C-90b.

27.3.1.6. Pequannock/Ramapo Levee System. This system is located on the right bank of the Ramapo River and would provide protection to existing structures located within Pompton Lakes. See Figure C-90b. To significantly reduce fluvial flooding 2,200 feet of levee and 2,910 feet of floodwall would be required. The levees would have an average height of 5.9 feet and base width of 46 feet. The floodwalls would have an average height of 5.7 feet. Runoff from the drainage area behind the levee/floodwall would be handled by a 3 cfs capacity pumping station and four ponding areas of 0.3 acres, 0.4 acres, 8.5 acres and 5.0 acres in size. The latter two ponding areas are part of the natural storage areas. Drainage through the entire levee/floodwall would

be handled by 7-24", 3-27" and 2-36" diameter pipes all of which will be controlled by both flap and sluice gates. Profile sheets for the levee system are shown on Figures C-97a and C-97b.

27.3.1.7. Upper Pompton Channel. Two small sections of the Upper Pompton River would be modified. The upper 400 feet of the Pompton River at the Main Tunnel Inlet would be deepened 4 feet to a depth of 161.0 feet NGVD. This portion of the channel work is designed to help convey flow into and around the Pompton Inlet.

A small 1700 foot long, 20 foot wide, 4 deep trapezoidal pilot channel would be excavated between the upper channel modification and the Pompton Plains Gaging Weir. The Pompton River modifications are shown on Figure C-90a.

27.3.2. Passaic and Lower Pompton Rivers Channels

27.3.2.1. General. Several channel modifications and a hydraulic control structure would be constructed to supplement the Spur Tunnel Inlet. These associated project features are described below. Figures C-90f to C-90g show the layout of the project features associated with the Spur Tunnel Inlet.

27.3.2.2. Passaic River Channel. Two sections of the Passaic River would be modified as part of the project. The first section would straddle the confluence with the Pompton River, it would be 0.5 miles long, and would extend from the Great Piece Weir (described below) to the Spur Tunnel Inlet. The purpose of this channel is to efficiently transition and convey flow through the Great Piece Weir to the Spur Tunnel Inlet. The channel would have a bottom width of 240 feet below the confluence with the Pompton and a 170 foot width above the confluence. The channel would be about 6 feet deep, trapezoidal with 1 on 2.5 side slopes and a 0.03% bottom slope. Two areas along the Passaic required riprap protection. The upper area is along a 550 foot section of existing bend in the river which is about 400 feet upstream of the proposed channel work. The other section extends from 25 feet upstream of the Great Piece Weir to 100 feet downstream of the weir.

The lower section would consist of a 3 foot deep, 20 foot wide, 1.25 mile long pilot channel. The pilot channel would extend from the Spur Inlet, downstream to a point above the Rt. 23 bridge where the existing and pilot channel inverts meet. The pilot channel would have "1 on 2.5" side slopes and a bottom slope of 0.03%. The pilot channel would be incised in the existing channel bottom and would not affect the existing banks. Without the pilot channel, the termination of the main channel, described above, would result in a low channel invert directly in front of the inlet. The pilot channel would eliminate the low

point and would convey low flow past the Spur Inlet when the gates are closed and the tunnel is not in operation. Eliminating the low point and allowing low flows to move downstream would help prevent sediment from accumulating in front of the inlet.

27.3.2.3. Lower Pompton Channel. The purpose of this channel would be to efficiently transition and convey flow to the Spur Tunnel Inlet. This channel would widen the lower 0.3 miles of the Pompton River to a 150 foot bottom width. The channel invert would slope at 0.025% from a 5 foot depth at the mouth, to the existing invert some 1500 feet upstream.

27.3.3. Great Piece Weir. The Great Piece Weir would be a hydraulic control structure in the Passaic River, located approximately 800 feet upstream of the confluence with the Pompton River. The weir would be situated within the Town of Fairfield and the Borough of Lincoln Park. The structure would contain five 30 foot wide by 10 foot high, torque tube bascule gates. The structure is illustrated in Figure C-98.

The purpose of the structure would be to periodically flood the wetlands of the Great Piece Meadows. Under most conditions the gates of the Great Piece Weir would be kept open (down). During normal or non-flood conditions the gates would remain open and water elevations in the area of the Great Piece Weir would be controlled by Beatties Dam, which is located 18,000 feet downstream. Since the crest of Beatties Dam is at elevation 158.2, the closed (lowered) gates of the Great Piece Weir would be covered with about two feet of water.

During rising and peak flood levels the gates would still remain open and flood waters would pass through the structure and proceed to the Pompton and Passaic River confluence were excess flood flows would be diverted into the Spur Tunnel and the remainder would continue down the Passaic. As the flood waters recede the gates would be operated to maintain a one to two year water level for a period of time that would satisfy fish and wildlife needs in the meadows. Therefore, the duration of flooding in the one to two year floodplain would not be adversely affected by the project.

27.4. Deepavaal Brook Channel Modifications

The proposed Deepavaal Channel Modifications are a departure from the levees originally proposed in the Phase I Study. The UNET analysis determined that adequate protection for the industrial and commercial buildings along Deepavaal Brook could be attained by modifying the existing Deepavaal Brook channel. It was also determined that the channel modifications would be more cost effective and would induced less flooding than the original levees. Figures C-90i and C-90j show the plan and profile views of the channel modifications. Figures C-99a thru C-99c show the profile and channel cross sections.

The Deepavaal Brook Channel modification would extend a distance of 7,660 feet from approximately Station 60+50 to approximately Station 137+10 of Deepavaal Brook. The modification would increase the existing channel to a bottom width of 30 feet and re-grade the existing channel bottom slope to 0.061%. The channel would be trapezoidal in shape with either "1 on 3" or "1 on 2.5" side slopes.

The proposed channel improvements would begin approximately 800 feet upstream of the New Dutch Lane bridge or 600' downstream of the Jersey City Aqueduct crossing. The improvements would follow the alignment of the existing channel as it skirts the Essex County Airport. The improvements would terminate about 1,200 feet upstream of the confluence with Green Brook in the area of Long Meadow Lane at approximately Station 137+10. The channel excavation would require that two, 72 inch diameter water mains of the Jersey City Water Supply system be lowered six to seven feet to pass under the new and lower channel.

Additional channel work would be required further downstream underneath the Fairfield Office Center to alleviate an unfavorable backwater condition upstream of the office building. Proposed channel improvements will begin at Station 23+50 of Deepavaal Brook and continue underneath the Fairfield Office Center to Station 29+10 of Deepavaal Brook. The building itself is positioned over Deepavaal Brook from Station 24+45 to 25+50. The proposed channel modifications include increasing the bottom width to 50' upstream and downstream of the building and lowering the channel invert elevation to 157' if it does not cause structural problems with the building foundation and footing.

27.5. Camp Lane - Raising of Existing Structures

About six houses along Camp Lane would be raised between two and three feet. See Figure C-90f. The structures are near the Passaic River just upstream of the Great Piece Weir and the confluence with the Pompton River. Since these building are relatively low (within the 2 year floodplain) they would incur frequent damages and/or limit the ability of the Great Piece Weir to maintain and inundate much of the wetlands in the Great Piece meadows. Therefore the structures will be raised to prevent frequent damage due to flooding and to facilitate inundation and maintenance of the wetlands. Also provided in this area would be the filling of low ground to elevation 166.2 NGVD or slightly above the top of the proposed gates at the Great Piece Weir. More information about these structures can be obtained in the Economic Appendix of this report.

27.6. Central Basin Levee/Floodwall Systems

Pertinent data for the individual levees have been summarized on Table C-43a. This data includes basic facts about the levee/floodwall physical components as well as the interior flood damage reduction facilities.

27.6.1. Passaic #2A Levee/Floodwall

The Passaic #2A line of protection would be located on the right bank of the Passaic River in West Caldwell and Fairfield Townships. It would consist of several levee/floodwall elements of new construction with an total length of about 9300 feet. At its upstream end, it would tie in to natural high ground while making use of existing high roadway embankments in the area including Interstate 80 running parallel to the Great Piece Meadows in this area . Figures C-90k to C-90m show the plan and Figures C-100a to C-100f the profile sheets of the proposed project element.

Levees would consist of approximately 6,216 feet, a top width of 10 feet and an average height of 7 feet with 1 on 3 side slopes. This levee system would not be exposed to erosive stream velocities and would not required to be armored.

Floodwalls would make up the remaining 3,082 linear feet of protection that comprises the rest of the line-of-protection. These walls would have an exposed height of about 5.5 feet. These walls have been designed as an "I" section, closely following standard Corps procedures.

The final new element would be a closure structure located at the crossing of I-80 over Horseneck Road. This location would require a 45 foot movable closure and approximately 110 liner feet of low levee tying into the I-80 overpass embankments. As intended, the closure would permit the sealing-off of the roadway opening under the Route 80 embankment and be essential to the overall line-of-protection. As proposed the closure and levee section would average 5.5 feet in height.

Along the existing highway embankments, existing culverts would need to fitted with both flap and sluice gates such that high exterior Passaic River flood waters would not flank the line-of-protection. These remote facilities are shown on Figures C-90n to C-90p. Interior flooding damage reduction facilities were investigated for 6 separate ponding areas and would include both gravity drains and small pumping stations. These facilities will be further discussed in Section 35. Levee/Floodwall Hydraulic Designs.

27.6.2. Passaic #10 Levee/Floodwall

The Passaic #10 levee/floodwall system would be located on the right bank of the Passaic River in Livingston Township and generally form a horseshoe shape line-of-protection around and an exposed industrial and commercial area. The levee/floodwall system would consist of 4,750 feet of levee and 79 feet of floodwall for a total length of approximately 4,829 feet. The top of protection varies from 178.1 to 178.6 which includes the necessary levee superiority. Figure C-90q show the plan and Figures C-101a to C-101f show the profile sheets of the proposed

project element.

Interior flooding facilities were evaluated for 2 separate subareas and in gravity drains with very small pump stations were included to complement the overall plan of protection. Two-24" and two-36" culverts would provide low exterior stage regulation (including both flap and sluice gates) while each side of the protected area would have a 3-cfs pumping station. Further design details are included in the interior flooding analysis (Section 35.4.) as well as Appendix J which details the FDM effort for this project element.

27.6.3. Pinch Brook Levee/Floodwall

This levee/floodwall would be located on the right bank of Pinch Brook, upstream of the confluence with Black Brook in the Township of East Hanover. The floodwall would be centrally located floodwall and have a length of 415 feet. The wing levees would combine to a length of 2,397 feet thereby providing an overall system length of 2,812 feet. Figure C-90r, and Figure C-102 show the plan and profile sheets of the proposed project element.

The Pinch Brook levees would have top widths of 10 feet and side slopes of 1 on 3. The most upstream levee section would have a length of about 1,275 feet and an average height of about 8 feet. It would be founded on compressible material and has been designed for settlement. The downstream levee would have a length of about 1,122 feet and an average height of about 9 feet. It would also be founded of compressible material, but the maximum potential for settlement would be somewhat smaller than the upstream section.

The Pinch Brook floodwall would be about 415 feet long and have an average exposed height of about 9.4 feet. It would be of "I" section and consist essentially of a deep reinforced concrete cap on steel sheet piling bottoming out an average of 23 feet below grade.

The interior flooding reduction facilities would be comprised of both gravity culverts, 2 - 36" CMP's and a small pump station of 3 cfs.

27.6.4. Rockaway #1 Levee/Floodwall

This line of protection would consist of 2 widely separated elements on the right bank of the Rockaway River in Parsippany-Troy Hills Township. New construction with an aggregate length of about 3,760 feet would be involved. The upstream portion of Rockaway #1 would consist solely of a levee and the downstream section of both levee and floodwall. Figures C-90s and Figures C-103a and C-103b show the plan profile sheets of the proposed project element.

The upstream levee section would have a length of 2,421 feet. The average height would be approximately 10.3 feet with the location of the structure set-back from the Rockaway River. The levee would have a top width of 10 feet and side slopes of 1 on 3. There would be an erosion control mat on the riverside slope.

This element, as presently configured, is dependent upon the raising of the existing Lake Hiawatha Flood Control levee and floodwall system. If the existing levee/floodwall were not raised, then the Rockaway #1 system would require another section of levee on the northern side to protect from flanking.

The downstream portion of the Rockaway #1 system would have a length of about 1,339 feet, of which 821 feet would be levee and 521 feet would be floodwall. The levee would have an average height of about 6 feet, a top width of 10 feet and side slopes of 1 on 3. The flood wall would have an average height of 3.3 feet. It would be of "I" section throughout and would consist essentially of a cast-in-place concrete cap on a proposed continuous line of steel sheet piling driven about 10.5 feet below existing grade.

The interior drainage reduction facilities would consist of numerous culverts both proposed and existing structures. In general the interior area protected by the levee system has as a western limit, Interstates I-80 and I-280. These high and wide roadways would as rear side protection and during high exterior stages. All existing culverts under the Interstate roadways would need both flap and sluice gate alterations to prevent flanking during flooding conditions. These remote facilities are shown on Figures C-90t to C-90v. As a final element on the upstream levee portion, a 1 cfs pump station would be provided for interior concerns.

27.6.5. Rockaway #2 Levee

This line of protection would be located in the Township of Montville on the left bank of the Rockaway River just opposite the upstream section of the Rockaway #1 system. Rockaway #2 would consist of a single levee 3,172 feet long having an average height of about 10 feet, top width of 10 feet and side slopes of 1 on 3. The river-side slope of the levee would be provided with an erosion control mat. No floodwall would be required as part of the line-of-protection. Figures C-90w and Figures C-104a and C-104b show the plan and profile sheets of the proposed project element.

Interior flood damage reduction facilities have been designed to match the conveyance of the existing 24' wide by 6' high box culvert under Change Bridge Road immediately downstream of the proposed levee unit. During high river flows that would block the proposed box culverts, a 10 cfs pump station would be provided to aid in flood damage reduction.

27.6.6. Rockaway #3 Levee/Floodwall

This line of protection would be located on the right bank of the Rockaway River in the Lake Hiawatha area of Parsippany-Troy Hills Township. It would include both new construction and the modification or replacement of existing levees and flood walls. The new line, which would provide a higher level of protection than the old, would have an aggregate length of approximately 7,530 feet versus the old length of about 6,130 feet. The levee would have the secondary purpose of preventing the flanking of the Rockaway #1 system. Figures C-90x and C-90y show the plan and Figures C-105a to C-105d show the profile sheets of the proposed project element.

New construction would include approximately 3 sections of flood wall with an aggregate length of about 1,400 feet. A new floodwall would be constructed at the most upstream end of the extended line-of-protection. It would be a stand alone structure serving an area not previously protected. It would be located about 500 feet upstream of any other component, and would have a height of about 6 feet. The other new floodwalls would extend the existing line-of-protection in both upstream and downstream directions. Their exposed heights would be about 8.5 feet. They would be of "I" section throughout and would consist essentially of deep, cast-in-place concrete caps on proposed continuous lines of steel sheet piling driven to a maximum depth of about 20.5 feet below existing grade.

The modifications would involve the raising of existing levees an average of about 5 feet. Where space permits, the existing levees will be incorporated into higher, wider ones. Where space is tight, flood walls with exposed heights averaging about 5.1 feet would be installed on top of the existing levees. These walls would be similar and would incorporate lighter sheet piling be driven to a depth of about 10.5 feet below grade.

A total of 3,835 feet of floodwall would be replaced. The replacement walls would have exposed heights of about 8.6 feet and would be of essentially the same construction as the walls used to lengthen the line of protection.

Under the proposed Rockaway #3 levee/floodwall design the existing 5 closures would be reduced to 4 structures. Interior flood damage reduction incorporate the existing pressure interceptors, culverts and 183 cfs pumping station sited at the centralized ponding area. All new floodwall sections would incorporate small drainage culverts to pass localized standing water to the Rockaway River.

27.7. Preservation of Natural Storage Areas

As part of the Federal project, this study includes the purchasing of 5,350 acres of Natural Storage Areas. Preserving these existing natural storage areas through the restriction of

both development and the placement of fill would be integral to the functioning of the basin-wide overall plan of improvement. This economically justified project feature would eliminate and alleviate the potential development to these areas and thus minimize the possible changes to stage and discharge hydrographs resulting from Central Basin urbanization in the future. Although future development and urbanization will continue to occur in the upland and remote portions of the basin, the purchasing of these Central Basin lands would prevent added development adjacent to the existing NJDEP floodways and overbank storage areas. It is also recognized that the purchasing of these lands would be in compliance with the State of New Jersey's "no net fill" concept for the Central Passaic Basin.

Removing these lands from the pressures of urban development would also help to ensure the integrity of the tunnel design. For the GDM the main tunnel has been designed to divert approximately 27,500 cfs with a Pompton River design head of 174.5 NGVD and the spur tunnel was designed to pass 13,000 cfs at 164.3 NGVD.

27.8. Tidal Area Protection Levees

27.8.1. Kearny Levee/Floodwall System

The line of protection for Kearny would incorporate both new single-purpose flood damage reduction facilities and the use of existing structures to serve as flood reduction elements. Modification to existing structures would include altering highway and railroad embankments as well as a petroleum containment berm. As proposed for construction, a total length of approximately 38,315 feet would be required consisting of 3,908 feet of levee (average height of 5.2' and 41.0' base width), 33,771 feet of floodwall (average height of 7.35') and 625 feet of closures. The top of protection would be constant at Elevation 14.9 NGVD. The proposed line of protection would also safeguard the PATH trains that cross Kearny from Newark, NJ to Jersey City, NJ. Figures C-90z to C-90ee show the plan, and Figure C-106a to C-106e show the profile and detail sheets of the proposed project element.

Included in the proposed plan are 19 closures that generally range in height from 1.5 to 8.5 feet and width from 15 to 60 feet. These facilities are intermittently spaced throughout the line of protection and would be required to be closed for effective flood damage reduction measures.

Interior flooding facilities were evaluated for 4 separate subareas and in general gravity drains were found to be sufficient. One ponding area would also contain a 75 cfs pump station.

27.8.2. Lister-NJ Turnpike-Doremus Levee/Floodwall System

The line of protection for the Lister Avenue - NJ Turnpike - Doremus Avenue section of the City of Newark would include a series of levees, floodwalls and closure structures. Existing embankments would also be used where both appropriate and feasible. The top of the line of protection would be at 14.9 feet NGVD. As proposed for construction, a total length of approximately 23,918 feet would be required consisting of 5,599 feet of levee (average height of 5.5' and 44.0' base width), 17,657 feet of floodwall (average height of 8.1') and 658 feet of closures. The proposed line of protection would require interior structures to prevent flanking during the most extreme events. Figures C-90ff to C-90jj show the plan, and Figures C-107a to C-107b (also C-106d and C-106e) profile and detail sheets of the proposed project element.

Included in the proposed plan are 28 closures that generally range in height from 5.4 to 11.0 feet and width from 4 to 30 feet. These facilities are intermittently spaced throughout the line of protection and would be required to be closed for effective flood damage reduction measures.

Interior flooding facilities were evaluated for 9 individual subareas and general gravity drains would be sufficient. However, within the Lister section of the levee, 2 pump stations would be required. These stations would be constructed for local interior runoff and would be 50 cfs and 100 cfs.

27.8.3. South 1st Street Levee/Floodwall System

The South First Street line of protection for the Town of Harrison would consist of a series of levees, flood walls and gated closure structures. It would begin and end at existing railroad embankments and tie in at intermediate points to three existing bridge approaches with the top of protection at Elevation 14.9 NGVD. The total length would be approximately 7,700 feet and would require 1,750 feet of levee (average height of 6.5' and 50.0' base width), 5,700 feet of floodwall (average height of 6.20') and 250 feet of closures. Figure C-90kk shows the plan and Figure C-108 the profile. Figures C-106d and C-106e show the detail sheets of the proposed project element.

Included in the proposed plan are 8 closures that generally range in height from 2.6 to 7.4 feet and width from 30 to 40 feet. These facilities are intermittently spaced throughout the line of protection and would be required to be closed for effective flood damage reduction measures.

Interior flooding facilities were evaluated for 3 separate subareas. Gravity drains would be required at all subareas and each subarea would also require a pump station. Proposed pump station sizes would be 30, 70 and 75 cfs.

28. Plan/Project Operation

28.1. Project Operation

Operating the proposed plan would require a central operational facility to manage this complex system of tunnel and local protection works. The sequence of gate operation at both tunnel inlets and Pequannock and Great Piece weirs is the key to the successful reduction of flood events. Successful operation would include the diversion of flow through the tunnel during the flood peak to reduce flood damages while ensuring sufficient base flow remains in the channel, and the retention of water upstream of the weirs once the flood peak has passed to satisfy environmental objectives.

Currently, the operation center has been sited at Workshaft #2 in Montclair, NJ. However, the convenience and safety of having the center located at one of the inlets will have to be weighed against the practicality of locating the center where the operators could perform other functions during non-flood times. Supporting facilities would be located at the inlets to allow for on-site control of the facility should communications with the control center be disrupted. Real-time hydrologic data such as streamflow, reservoir elevation and outflow, rainfall, depth of snowpack on the ground, tidal stage, temperature and wind speed and direction would all be transmitted to the center. The center would also be equipped to monitor weather radar reports, coupled to "ground truth" actual recording gages, to determine severity and movement of storms. The gates at each inlet would be equipped with sensors to track gate openings. Video cameras would be located at each inlet, enabling the operators to observe flow conditions, possible debris obstructions, or gate malfunctions, as well as to prevent and/or report vandalism. It is vital that the operators at the center have all information necessary to enable the best possible operation of the facilities.

Gages within the tunnel would inform the operator of water hammer or undesirable surges at vent shafts. Phase II GDM numerical studies have shown that surging and/or water hammer can be reduced by gradually filling and pressurizing the tunnel prior to any rapid gate opening. One or more gates at the main Pompton Inlet could be slightly opened when a significant flood event is projected in order to gradually pressurize the tunnel over at least six hours. It is expected that occasionally floods may be smaller than initially projected, and a "false alarm" would occur that may result in the filling of the tunnel, however the water bypassed in this manner should not be significant.

Some of the hydrologic data would be provided through the flood warning system that was designed, installed, and operated through a team effort involving the State of New Jersey, the U.S. Army Corps of Engineers, the National Oceanic and Atmospheric Administration and the U.S. Geological Survey. The Flood

Emergency Preparedness System for the Passaic River Basin currently includes radio reporting rain and stream gages, computers and a communication network. The operations center would be able to receive that data directly and use it, along with additional data, as input to a computer model that would make forecasts and simulate project operation. Not only would the center assure the proper operation of the tunnel, but it would also issue warnings and orders to make certain that project closure structures and interior drainage facilities are properly tended to and functional. The center would also make decisions as to when the Great Piece Meadows Weir would be raised or lowered, thereby governing the duration of flooding of wetlands. Finally, in the event of flood flows exceeding the tunnel capacity (greater than a 100- year event), warnings would be issued.

28.2. Normal Conditions - Inlets, Outlet and Weirs

During normal flow periods and minor floods up to a 1- year magnitude, the new Pequannock Weir would remain in the upright and closed position, maintaining flow in the Ramapo channel and over the old Morris Canal feeder dam. All tunnel entrance gates also remain closed. The Great Piece Weir would normally remain in a lowered, or open position, but should be exercised periodically, and could be raised during low- flow periods if it is determined to be environmentally beneficial.

28.3. Tunnel Operation - Inlets, Outlet and Weirs

During floods expected to exceed a 1- year magnitude, water surface elevation monitors in the river adjacent to each tunnel inlet would signal the appropriate time to open the tunnel inlet gates. When the Pompton Tunnel Inlet is opened, the tainter gates of the Pequannock Weir would also be opened to allow the flood water to approach the tunnel entrance through the enlarged Pequannock bypass channel in such a manner that it would enter the tunnel more efficiently. Approximately 4,500 cfs, or the 1-year flood, would continue to flow in the existing natural Pompton River channel while the remainder is passed through the tunnel, up to and including the 100- year flood. The tunnel inlet gates are to be continuously monitored and controlled to maintain a relatively constant flow in the channel downstream of the tunnel inlet. At the 500- year flood level, the tunnel would flow full and a flood approximately equal to a 10- year natural event would be experienced downstream of the main tunnel inlet. The Two Bridges tunnel inlet would close during the peak flow period at the Pompton Tunnel Inlet to maximize stage reduction within the Central Basin. During tunnel operating events of 100-year magnitude or smaller, a 1- year flood, or approximately bank-full flow, would remain in the channel. The Two Bridges tunnel inlet would remain partially open even during the peak flow at the Pompton Inlet to prevent damaging flood elevations

from local inflows in the Hoffman Grove vicinity. As flows decrease, the Great Piece Weir would be raised to increase the duration of higher water surface elevations in the wetlands, up to elevation 166.0 feet NGVD, the top of the bascule gates.

Controls for the Pompton Inlet gate are based on the water surface elevation of the Pompton River adjacent to the tunnel entrance. When the river elevation reaches 174.5 feet NGVD, or greater, the tunnel gates would open at a rate of one foot per minute up to a maximum gate opening of 12 feet. If the water level drops to elevation 174.0, the gates would close at one foot per minute. Between elevation 174.0 and 174.5, the gate opening remains static. Like the Pompton Inlet, the Spur, or Two Bridges Inlet, control is based on the adjacent Passaic River elevation. The opening and closing trigger elevations are 164.3 and 163.2 feet, respectively, at one foot per minute to a maximum gate opening of 13 feet. The Two Bridges Inlet has an additional control - if the water surface elevation near the Pompton Inlet exceeds 175.2 feet, then the Two Bridges Inlet would begin closing. This occurs when floods are greater than can be accommodated by the Pompton Inlet (exceeding the 100- year design event). Lesser floods would not cause the Pompton River elevation to continue increasing; the increasing gate opening would allow the flow to enter the tunnel without raising the water surface significantly.

28.4. Levees, Floodwalls and Closures

The levee/floodwalls would require proper operation of all flap and sluice gates (as backup). Pump stations would be operated to maintain the interior ponding to non-damaging levels and maintain wetlands. Closure facilities would also be required at 5 of the levee/floodwall systems. Details are provided on Table C-43b. Systems that require closures are:

- South 1st Street
- Lister/NJ Turnpike/Doremus
- Kearny
- Passaic #2A
- Rockaway #3

These facilities will be further addressed during the respective FDM for each of the levee/floodwall systems.

29. Stage and Flow Reductions - Project Improvements

The proposed plan would reduce the design flood, 1 % chance flood - 100 year, to non-significant damage levels in the Central and Lower basin. Areas upstream and downstream of the tunnel inlets would have water level reductions as a result of operating the tunnel. In the tidal areas where tunnel reductions have minimal effects, the plan includes levees and floodwalls to provide protection in the heavily urbanized areas adjacent to Newark Bay.

Data developed in this study reviewed four separate conditions for each of the 8 hypothetical floods as discussed previously in Section 24. Hypothetical Peak Discharges and Stages - Unimproved Conditions. The 4 conditions modeled were:

a) Existing conditions - Year 1992, no Corps project;
b) Present Improved - Year 1992, with Corps project;
c) Future unimproved - Year 2050, no Corps project; and
d) Future unimproved - Year 2050, with Corps project.

Present Improved conditions were only used to assist in the economic analysis of project benefits and generally will not be further discussed. Therefore, comparisons focused on conditions a), c) and d).

The following paragraphs will describe the impacts resulting from the proposed plan. Comparisons will include both hypothetical storm/runoff events as well as performance routings of major historical events (October 1903, May 1968 and April 1984) as adjusted to today's basin conditions.

29.1. Comparisons of Hypothetical Floods

Comparisons of future (Year 2050) with and without project conditions have been developed and are presented in a variety of ways. Tables, figures, plates and inundation maps compare the impacts of the plan of improvement. Generally, comparisons will be made in the Year 2050 or approximately 50 years into the future. Where appropriate, existing (Year 1992) condition data will be shown as well.

29.1.1. Water Surface Elevations

Peak water surface elevations were tabulated for the 8 flood events (1-, 2-, 5-, 10-, 25-, 50-, 100- and 500- year) at each cross section. Water surface elevations at selected locations are shown in Table C-44 for existing, future w/o and future with project. Figures 109a to 109jj show comparative plots for these three conditions for both the 1- and 100- year profiles.

Improved condition water surface profiles have also been developed and are shown on Plates C-92 to C-139. These plates show the Year 2050 for both with and without project conditions. Without project conditions are shown for the 10-, 100- and 500-year floods.

29.1.1.1. Upper Pompton River and Tributaries (UNET Reaches 8, 9, 10, 11, 12, & 13)

Within the upper tributary areas to the Pompton River, water surface elevations were lowered across the full range of frequencies. Reductions in stage on the Pequannock, Wanaque, and Ramapo Rivers were a direct result of the proposed channel improvements, the addition of the gated Pequannock Weir and the

operation of the tunnel. For the 1- year flood the channel improvements tend to lower the water levels in the upstream reaches above the existing Old Morris Canal Feeder Dam. However, as one approaches the existing structure, water levels at the 1-year flood would be equal for future with and without project conditions. See water surface comparative plots for the 1- and 100- year floods on Figures C-109a thru C-109jj.

29.1.1.2. Pompton River (UNET Reach 14)

The Pompton River, would receive substantial stage reductions with the proposed project in place. Water levels would be reduced up to a maximum of 11 feet in the vicinity of the main inlet. Flood level reductions along the Pompton River generally vary from 3 feet at the 2- year flood to 8 feet at the 100- year. Flood reductions are almost constant along the entire length of the Pompton River. For the 500- year flood, stage reductions would be about 6 feet. Water levels on the Pompton are influenced by the operation of the main inlet upstream and the spur inlet downstream. As can be seen on Table C-44, operation of the spur inlet gates would result in a flat stage-frequency curve from the 1- to 50- year floods (peak elevation at 164.3 to 164.4 NGVD). This operational mode would be extremely beneficial to the low lying residential area (Hoffman Grove), located at Section 99+40 along the Pompton River. With the project in place, the water surface levels at Hoffman Grove for the 100- year flood would be lowered from 177.0 NGVD, to approximately the start of damage at elevation 167.5 NGVD.

For the 1- year flood, stage reductions along the river would be minimal and closely agree with the future without project condition.

29.1.1.3. Upper Passaic River Tributaries (UNET Reaches 1, 2, 3, 4 and 5)

On the upper portions of the Rockaway and Whippany Rivers within the UNET modeled area, there would be no stage reductions resulting from the tunnel. For the Rockaway River, tunnel reductions would not be realized above Station 125+00. For the Whippany River, this occurs near Section 159+80. Below these stations tunnel reductions occur as can be seen on Table C-44.

Along the Rockaway River the proposed project includes three levee/floodwall sections that are in some locations on both banks. These units extended intermittently from Station 105+00 to Station 281+00. Due the proposed raising of the existing Lake Hiawatha levees and proposed construction of Rockaway #1 and #2, water surfaces are slightly higher with these features in place. Therefore, the upstream end of the Rockaway #3 system would require approximately 1100 feet of new floodwall construction.

29.1.1.4. Passaic River (UNET Reaches 6, 7, 15, 17 and 19)

Water surface reductions along the Passaic River would be predominantly downstream of the Passaic #10 levee/floodwall system proposed at Station 2618+80. For all flood events, stage reductions would be minor, less than 0.75 feet, in areas upstream of the Passaic and Rockaway confluence.

At the head of the Great Piece Meadows, stage reductions vary from 0.1 feet at the 2- year to 0.9 feet at the 500- year flood. At the proposed Great Piece Weir, Station 1745+00 stages would be reduced from 1.7 feet at the 2- year to 8.3 feet at the 100- year. It should be noted that under existing and future without project conditions, water surface elevations exceeding 171.1 NGVD at Passaic River Station 2355+75 would cross-over into Deepavaal Brook as a lateral inflow. This elevation is currently exceeded at approximately the 25- year event. The Passaic #2A levee/floodwall is designed to prevent the cross-over flow to elevation 175.9 NGVD, the top of levee/floodwall. This elevation was not reached for any of the hypothetical floods investigated.

Along the Passaic River between the confluence of the Pompton River (Station 1735+00) and Dundee Dam (Station 917+90), stage reductions would occur as a result of operating both tunnel inlets. Stage reductions vary going downstream from 2.1 to 0.7 feet for the 2- year flood; from 8.3 to 1.7 feet for the 100- year flood; and from 6.0 to 2.0 feet at the 500- year flood.

At the Passaic and Saddle River confluence (Station 822+00), stage reductions for fluvial flooding would be approximately 1 foot across the range of frequencies from 2- to 500- year. Between the Saddle River confluence and mouth of the Passaic River, stage reductions become minimal as the area becomes dominated from coastal storm surge events.

29.1.1.5. Deepavaal Brook (UNET Reach 16)

On Deepavaal Brook stage reductions would occur along its entirety as a result of tunnel operation, the construction of the proposed Passaic #2A levee and the 8,125 feet of channel improvement. The proposed levee located along right bank of the Passaic River between Stations 2403+00 and 2311+00 would prevent the Passaic River from crossing over into Deepavaal Brook during the rare and extreme flood events.

Stage reductions along Deepavaal Brook are as high as 7.8 feet during the 100- year event at the mouth. Typical reductions from the upstream end at Station 194+30 to its mouth vary from 1.2 to 1.7 feet for the 2- year; from 3.1 to 7.8 feet for the 100- year ; and 4.4 to 5.9 feet for the 500- year. Reductions increase in the downstream direction.

29.1.1.6. Saddle River (UNET Reach 18)

Reductions along the Saddle River are limited to lowering the Passaic River backwater within the Saddle River. The reductions to the Passaic River would be from 0.6 to 1.3 feet for various frequencies at Passaic River Station 822+00. Generally, due to the minor stage reductions, water levels return to without project conditions at approximately 3,000 feet upstream on the Saddle River.

29.1.2. Stage Frequency Curves

Stage-frequency curves were developed throughout the entire basin at 234 locations, of which 45 of these sites were in the tidal and fluvial area below Dundee Dam.

Above Dundee Dam, 189 stage-frequency curves were determined solely from the UNET derived maximum water surface profiles. Typical curves are shown for selected locations on Figures C-110a to C-110j which summarize the existing, future without project and future with project conditions. Stage reductions are visually shown where the project would lower water surface elevations.

Below Dundee Dam, the 45 stage frequency curves combined the effects of both storm surge and river fluvial flooding. Results for the improved condition are shown on Figures C-110k to C-110o.

29.1.3. Peak Discharges

Peak flows were also tabulated for each of the 8 hypothetical floods analyzed. Maximum flows at selected locations are shown in Table C-45. The table includes existing, future without and future with project.

While the maximum water surface and maximum flow profiles were developed by UNET, the reader is cautioned from assuming that the peaks would occur together. Where stage-discharge relationships are unique, elevation and flow would be consistent. However, this is not always true, as many areas are heavily influenced by unsteady flow and the rating curves have significant looping characteristics. Therefore, the reader should not assume that maximum water surface elevations and maximum flows occur within the same time period.

29.1.3.1. Upper Pompton River and Tributaries (UNET Reaches 8, 9, 10, 11, 12 and 13)

Within the upper tributary areas of the Pompton River, peak flows increased from existing to future without project conditions as a result of local and upstream urbanization. When comparing future without to future with project conditions, flows were slightly increased on the Wanaque, Pequannock and

Ramapo Rivers as a result of the proposed plan of improvement. However, even though the flows increased, water surface elevations were reduced by lowering the storage in the overbanks.

On the Ramapo River above the proposed Pequannock weir, flows increased about 1% across the range of frequencies from upstream urbanization. At the 2- year flood, improved flow increased from 4,660 cfs to 4,710 cfs and the 100- year flow increased from 19,970 cfs to 20,050 cfs.

On the Wanaque River, flows increased less than 2.0% across the range of frequencies and generally were less than 1%. At the 2- year flood, improved flows remained constant at 2,160 cfs and the 100- year flows increased from 7,420 cfs to 7,460 cfs.

On the Pequannock River above the proposed Pequannock Weir, flows decreased less than 1.0% across the range of frequencies, except at the 1- year where the flow was slightly increased. At the 2- year flood, improved flow decreased from 3,200 cfs to 3,190 cfs and at the 100- year flow decreased from 11,470 cfs to 11,400 cfs.

29.1.3.2. Pompton River (UNET Reach 14)

Along the Pompton River substantial reductions in flow occur due to the operating of the main inlet. At Station 340+90, the peak flow for the 100- year is reduced by 83%, while at the mouth the peak flow would be reduced by 74%. For the 2- year flood at Station 340+90, improved flows are decreased to 4,470 cfs from 7,780 cfs; at the 50- year, flows would decrease from 25,000 cfs to 4,840 cfs; and at the 500- year flows change from 43,580 cfs to 13,710 cfs.

The plan of operation at the main inlet has been revised from the Phase I GDM and now permits a continuous bypass flow for all flood events at approximately 4,300 cfs to 7,000 cfs. Inlet operation now diverts the peak flow and truncates the bypass hydrograph at Pompton Plains at around a 1-year flood. Figure C-111 illustrates this concept for the 1-, 10-, 50-, 100- and 500-year floods. The 500- year flood exceeds the design of the tunnel capacity and flows downstream on the Pompton River are significantly higher. The gate operation was designed to approximate bank full capacity for all floods between the 1-year and 100- year events.

29.1.3.3. Upper Passaic River Tributaries (UNET Reaches 1, 2, 3, 4 and 5)

On the upper portions of the Rockaway River, Station 201+80, changes in flows would occur for flood events greater than the 10- year event. No changes occurred to lesser flood events. Flows gradually increased from 1.2% at the 10- year to 2.7% at the 500- year. This would be a result of the 3 proposed

levee/floodwall systems. At the confluence with the Whippany River, the peak flow on the Rockaway River gradually increased from 1.7% at the 10- year to 5.7% at the 500- year flood.

On the Upper Whippany River at Station 255+10, changes in flow are less than 1%. However, at the confluence with the Rockaway River (Whippany River Station 2+15) the peak flows are approximately 1% lower with the improvements in place through the 10- year, and decrease to 3.9% at the 500- year flood.

29.1.3.4. Passaic River (UNET Reaches 6, 7, 15, 17 and 19)

In the upper areas above the confluence with the Rockaway River (Station 2403+80), flows would change less than 1% through the 25- year event. At the 50- year flood, flows would slightly decrease with the project in place -1.8%. For the 100-year, flow would slightly increase from by 2.6% and the 500- year flow would remain essentially the same.

Below the confluence with the Rockaway River, improved flows would significantly change from tunnel diversion and the Passaic #2A levee blocking overland flow into Deepavaal Brook. At Station 2199+97 (Horseneck Road Bridge in Great Piece Meadows), flows decrease approximately 2 to 3% for events to the 25 year flood. Above the 25- year event flow increases 2% at the 50- year, 8.8% at the 100- year and 15.3% at the 500- year.

At the confluence with the Pompton River, flows on the Passaic River decrease for all floods except the 500- year flood. For the lesser floods, peaks flows are reduced for the 2- year thru 25- year by 16% and the 100- year by 7%. At the 500- year event flow increases by 10.3%. However, stage reductions would be significant at this location across the entire range of frequencies.

Proceeding downstream on the Passaic River to the Spur Inlet (Station 1721+40), flow would be reduced across the entire range of floods. The 2- year flood would be lowered by 31%, the 25- year event by 70% and the 100- year by 74%.

Between Little Falls and Dundee Dam, are peak flows are lower for all flood events. At Little Falls, flows are reduced from 32% at the 2- year to 74% at the 100- year. The 500- year flows are lowered by 54%. At Dundee Dam, flows are lowered by 32% at the 2- year flood to 62% at the 100- year event, with the 500- year lowered by 53%.

Below Dundee Dam peak flow reductions would continue to the mouth of the river. At the Saddle river confluence, the 2- year flow would be lowered by 1%, the 25- year by 26% and the 100-year by 23%.

29.1.3.5. Deepavaal Brook (UNET Reach 16)

Flows along Deepavaal Brook would be significantly altered with the proposed project in place. At the upstream end (Station 189+65), peak flows are substantially reduced at the 25- year flood and above due to the proposed Passaic #2A levee preventing cross-over flow to Deepavaal Brook. No change would occur from the 1- year to 10- year, while the 25- year would change from 410 cfs to 60 cfs. The 50- year would change from 1210 cfs to 60 cfs and the 100-year from 1700 cfs to 70 cfs.

Near the mouth of Deepavaal Brook at Station 1+80, flow increases (substantial stage reductions occur) with the proposed project from the 1- year to 25- year event. See Table C-45. Substantial reductions occur at the 50-year peak from 1520 cfs to 1090 cfs and the 100-year peak from 2170 cfs to 1440 cfs.

29.1.3.6. Saddle River (UNET Reach 18)

Flows along the Saddle River would be unchanged for flood events up to the 25- year level, and would remain virtually identical, less than 0.5% increase from the 50- year to 500- year at Stations 28+95 and 2+50. This minor flow increase (stage reduction occurs) would result from lower Passaic River tailwaters.

29.1.4. Flow and Stage Hydrographs

Stage and flows would not be noticeably different above the Passaic and Rockaway River confluence, although minor reductions would occur during the more rare flood events. Below this confluence, stage reductions increase substantially moving downstream to the Passaic and Pompton River Confluence. Stage and flow hydrographs for the 10-, 100-, and 500- year with and without the project illustrate the change of the hydrograph shapes in this area. See Figures C-112a to C-112f.

Along the Pompton River, stage and flow hydrographs for improved conditions would be very similar for various hypothetical floods to the 50- year level. Figures C-112g to C-1121 show comparative plots for the 10-, 100- and 500- year events with and without the project. Today's 500- year flood event would be reduced to approximately a 10- year occurrence with the project. Stage reductions along the Pompton would significantly reduce potential damages.

Along the Passaic River from the confluence with the Pompton to Little Falls, maximum project effectiveness would occur. This region of the Central Basin would have both flows and stages significantly reduced due to operation of both the main and spur tunnel inlets. Stage and flow hydrographs along the Passaic River are shown on Figures C-112m to C-112r at the Spur Inlet, Willowbrook Mall (Station 1660+00) and Little Falls gage (Station

1538+60) for the 10-, 100- and 500- year with and without the project. Also included in this area are the hydrograph alterations along Deepavaal Brook. See Figure C-112s to C-112x for comparisons of the 10-, 100- and 500- year with and without project conditions along Deepavaal Brook.

29.1.5. Inundation With and Without Project

Inundation maps with and without the project have been developed and are shown on Plates C-50 to C-91 for existing conditions and Plates C-140 to C-181 for improved conditions. Areas flooded for the 100- and 500- year have been substantially reduced with the project in place. For an easier comparison of the 100- year, the main report has plan sheets showing with and without project conditions for the design event.

29.2. Performance Routings of Historical Events

29.2.1. October 1903, May 1968 and April 1984

Performance routings with the proposed project in place have been performed and are documented in the next few paragraphs. It should be noted that for comparison purposes, all UNET model runs were made for future basin conditions and would be reflective of the year 2050. Therefore, the peak flow and water surface elevations obtained by the UNET simulations for without-project conditions are slightly higher than the actual observed data.

Figures C-113a through C-113f show the discharge hydrographs that would occur with and without the proposed project in place. Discharge hydrographs have been plotted at the Pompton Plains and Little Falls gaging stations. Figures C-113a and C-113b apply to the flood of October 1903, Figures C-113c and C-113d apply to that of May-June 1968 and Figures C-113e and C-113f apply to that of April 1984.

Figures C-113g through C-113l show the stage hydrographs that were developed at the same gaging sites. Figures C-113g and C-113h apply to the flood of October 1903, Figures C-113i and C-113j apply to the flood of May-June 1968 and Figures C-113k and C-113l apply to that of April 1984.

Table C-46 identifies the highest value graphed on each of the various figures. It was noted that despite the significant differences between the 1968 and 1984 floods for the without-project condition, the corresponding with-project values would almost be equal. This primarily results from the scheme of operation proposed for gates controlling the entry of water into the diversion tunnels. For the October 1903 flood, no significant effort was made to attempt to adjust either the HEC-1 or UNET model to account for changes in urbanization, construction of bridges, reservoirs, etc. The effort as documented in the following paragraphs basically used the

historical rainfall to develop the runoff transformation. Checks against high water marks were made where available to adjust rainfall losses.

Also provided on Table C-46 is a comparison of the highest stages that would be associated with these events. Times associated with the peak stages are not necessarily the same as those associated with the highest discharge value.

29.2.1.1. Flood Profiles - Maximum Changes

Figures C-113m through C-113t show the changes in water surface profiles that would occur in a few of the affected reaches. The reaches shown are the Passaic River through Great Piece Meadows, the Pompton River, the Passaic River from its confluence with the Pompton River to Deepavaal Brook, and the entire length of Deepavaal Brook. Table C-47 summarizes the stage reductions by reach for each of the historical performance ratings.

Table C-46 also lists the reductions in stage that would occur along each stream for each of the simulated historical storm events. The average reduction was listed first, followed by the maximum and the minimum reductions. The average reduction was computed by summing the reduction at each cross-section and dividing by the number of cross-sections in a given reach . The maximum and minimum values generally apply at an individual cross-sections.

29.2.1.2. Great Piece Meadows

Figures C-113m and C-113n show the maximum water surface levels that would occur along the Passaic River within Great Piece Meadows (UNET reach 7). It should be noted that for all three floods, the project impacts would be greatest at the downstream end of the meadows near Two Bridges and taper off in the upstream direction. This would be expected due to the proximity of the meadows outlet to the Pompton River (with reduced flows) and the Spur Tunnel inlet. The average reduction for the 1903 event would be 5.2 feet, the maximum reduction 7.5 feet and the minimum reduction 1.1 feet. Corresponding reductions for the 1968 event would be 2.5, 4.7 and 0.2 feet and for the 1984 event, 3.9, 6.9, and 0.4 feet respectively.

29.2.1.3. Pompton River

Figures C-113o and C-113p show the maximum water surface levels that would occur along the Pompton River (UNET reach 14) for the various conditions analyzed. It should be noted that for all three floods, the project would significantly reduce stages over the entire length of the Pompton River. The average reduction for the 1903 event would be 6.9 feet, the maximum reduction 8.0 feet and the minimum reduction 5.4 feet.

Corresponding reductions for the 1968 event would be 5.7, 8.2 and 4.4 feet and for the 1984 event, 8.8, 10.9, and 6.7 feet respectively.

29.2.1.4. Pompton and Passaic to Deepavaal Brook

Figures C-113q and C-113r show the maximum water surface levels that would occur along the Passaic River between the Pompton River and Deepavaal Brook (UNET reach 15) under the criteria discussed above. Again, the project impacts would be significant throughout the reach. This would be expected since reduced flows enter the reach from the Pompton River and flow would also be diverted into the Spur Tunnel inlet. The average reduction for the 1903 event would be 7.1 feet, the maximum reduction 7.5 feet and the minimum reduction 6.9 feet. Corresponding reductions for the 1968 event would be 4.3, 4.7 and 4.2 feet and for the 1984 event, 6.4, 6.8, and 6.1 feet respectively .

29.2.1.5. Deepavaal Brook

Figures C-113s and C-113t show the maximum water surface levels that would occur along Deepavaal Brook (UNET reach 16) for the same conditions. The project impacts would be greatest at the mouth of Deepavaal Brook and taper off in the upstream direction. The large reductions at the mouth would be a direct result of lowering the Passaic River backwater into Deepavaal Brook. The smaller reductions in the upstream direction reflect the upward sloping stream gradient of Deepavaal Brook channel. The average reduction for the 1903 event would be 5.3 feet, the maximum reduction 7.0 feet and the minimum reduction 3.6 feet. Corresponding reductions for the 1968 event would be 1.6, 4.2 and -0.2 feet and for the 1984 event, 2.8, 6.1, and 1.0 feet respectively.

29.2.2. Other Areas of the Passaic River Basin

In the remaining areas of the Passaic River basin, stage reductions would also occur but of smaller magnitude. These reductions would be similar to those described for the hypothetical events described in the previous paragraphs.

G. Hydraulic Design Criteria

30. Hydrodynamic Modeling in Tidal Zone

30.1 Purpose

The purposes of the tidal hydrodynamic studies were to provide the technical data necessary to run the water quality and sediment transport models. The objective of the series of tidal area models were to evaluate the response of the estuary to the

proposed flood damage reduction project. The 3D hydrodynamic model overlapped with the water quality model since it provided an estimate of the effects on temperature and salinity.

30.2 General Approach

A series of studies were conducted by technical personnel at the U.S. Army Corps of Engineers Waterway Experiment Station (WES). The studies evaluated flow patterns, sediment transport and water quality with and without the project. Six reports describing the modeling efforts were written by WES employees. Each report dealt with the following information:

Report 1: Riverine Sediment Transport
Report 2: Estuarine Sediment Transport
Report 3: Field Data Collection (Estuarine)
Report 4: Three-Dimensional Estuarine Hydrodynamic Modeling for Water Quality
Report 5: Water Quality Modeling
Report 6: Executive Summary of the Model Study

This section of the GDM (Section 30.) will explain the 3D hydrodynamic model used in the water quality analysis. The 2D hydrodynamic model used in the estuarine sediment transport analysis will be explained in Section 32.5. Estuarine Sediment Transport Analysis.

30.3 Technical Approach

Evaluating the hydrodynamic response of water from the tunnel discharging into the estuary depended on several factors:

a. Relative temperatures: The temperature of water trapped in the tunnel may not be the same as the temperature of Newark Bay. The tunnel water is expected to reach a consistent year round temperature of 55 degree Fahrenheit (12.77 deg C) due to the depth of the tunnel.

b. Salinity Differences: The salinity of the estuarine waters is expected to be higher than the salinity of the fresher tunnel water.

c. Flood Magnitude: Water which was trapped in the tunnel, from a previous event, would be discharged into Newark Bay whenever the tunnel was used. During a small event (2- year), the least volume of additional water would follow the trapped water for mixing and dilution.

The mixing process would be very complex near the outlet structure. The changes in both the vertical and horizontal distribution of temperature and salinity are anticipated. Therefore, a three-dimensional model was required for the water quality interface.

The three dimensional model chosen to evaluate the water quality impact (ICM) was previously developed by the Corps of Engineers and the Environmental Protection Agency (EPA) for application to Chesapeake Bay. The ICM model was developed in conjunction with the CH3D hydrodynamic model to interface the hydrodynamic information with the water quality simulation requirements.

The hydrodynamic models available included a finite difference CH3D model and a finite element RMA-10 model. An appropriate coupling of RMA-10 finite element model to ICM has not been developed. Consequently, the CH3D hydrodynamic model was selected for development of the data for the water quality model.

30.4 Sampling and Data Collection

Field data was collected in May and June of 1994 in support of the hydrodynamic, water quality and sediment transport modeling efforts. The monitoring included continuous tidal elevation, salinity and temperature recording at either end of Newark Bay (Figure C-114). In addition, on three days, detailed velocity data was collected at six ranges (cross sections) (Figure C-114). The velocities were collected using acoustic Doppler current profilers (ADCP). The data was then used in the verification of the model. Details concerning the data collection and analysis can be found in WES Technical Report HL-95-XX.

30.5 CH3D Model Development

The CH3D hydrodynamic model is a finite difference model. Quantities computed by the model include three-dimensional velocities, surface elevation, vertical viscosity and diffusivity, water temperature, salinity, and density. Solutions to the hydrodynamics were obtained using one-minute intervals. Velocities were computed on the boundaries between cells. Temperature, salinity, and density were computed at the center of each cell.

A curvilinear grid system was used in the horizontal plane. Five fixed layers, which varied in thickness as the depth changed, were used in the vertical plane. This approach, called sigma-stretching, is sensitive to large changes in depth between adjacent horizontal cells. The inherent communication between the bottom layers of adjacent cells can cause unwanted exchanges of water (and salt or temperature) in areas with abrupt changes in depth.

The model grid included a portion of the Atlantic Ocean, Lower Bay, Jamaica Bay, Raritan Bay, Upper Bay, Arthur Kill, East River, Harlem River, a portion of Long Island Sound, Hudson River to Albany, Kill van Kull, Newark Bay, Hackensack River to Oradell Reservoir, and Passaic River to Dundee Dam. The grid contained

3032 cells in each of five layers for a total of 15,160 cells. The model computational grid is presented in Figures C-115 and C-116. The coverage was necessary to adequately simulate the complexity of the system and to facilitate easy model boundary conditions.

The large domain of the model required that the bathymetry and shoreline information be extracted from a variety of sources. The data sources ranged from navigation charts to topographic charts and the NOS digital bathymetric database. All portions of the system were transformed into a common coordinate system. For convenience and because a large portion of the system lies within that zone, the Long Island Sound (LIS) New York state plane projection was selected for use on the grid. A finite element mesh was developed using the FastTABS modeling interface. The finite element mesh was then converted to a cell-centered depth format by averaging of depths and cross-referencing the elements to an I-J cell reference system.

The computational requirements were determined by the number of cells in the grid (both horizontally and vertically) and the current velocity in the smaller cells. The model generally is restricted by a Courant-type stability limitation, which places an upper limit to the allowable time step for model stability. The grid developed for this study required a time step of 60 seconds. The controlling area was in the vicinity of Hell's Gate in East River where the velocities exceed 10 feet per second.

A large number of industrial and storm runoff inflow points were identified and incorporated into the model to facilitate the water quality analysis. A total of 29 inflows were incorporated into the model, as delineated in Table C-48. All of the major tributaries were included along with other significant loads. The inflow for each cell was uniformly distributed across the water depth. All river inflows were assigned a salinity of zero and temperature at equilibrium, with the exception of the tunnel outflow. The temperatures of the tunnel discharge will be discussed later.

30.6 Model Verification

The verification of the hydrodynamic model was performed in several steps. First the model was tuned to general tidal energy propagation, then adjustments for tidal currents were made. Finally, the model mixing was adjusted for simulation of temperature and salinity conditions.

While the 3D hydrodynamic model was able to simulate temperature and salinity information the Water Quality model is better suited to simulating that information. However, since temperature and salinity do provide an excellent means of checking the hydrodynamics of the model they were evaluated in both models.

Generally, the hydrodynamic model was first adjusted to performing well throughout the entire geographic domain. Following the large spatial domain, the May and June 1994 field data was used to verify the performance in and around Newark Bay. In addition, the hydrodynamic simulation was extended through September 1994 to assist in the verification of the water quality model.

30.6.1 Boundary Conditions

The boundary conditions specified for the verification period were taken from several sources. The tidal boundary conditions were developed as a combination of analyzed recorded tides and predicted tides. The recorded tides were filtered to remove the low frequency subtidal component and harmonically analyzed. The harmonically reconstituted tidal signal was in good agreement with the predicted tides for the area. Therefore, the predicted tides for Sandy Hook and Bridgeport, CT were used in conjunction with the subtidal forcings for the tidal boundary conditions.

The river inflows for the primary tributaries were obtained from the state geological survey offices. Secondary river runoffs were developed from rainfall data for Newark Airport and earlier rainfall runoff responses developed by Hydroqual.

The meteorological parameters required for the model are equilibrium temperatures and heat exchange coefficients along with wind speeds. These were developed from the NOAA local climatological data (LCD) records for Newark, NJ.

30.6.2 Mean Tidal Propagation

The verification of the model for tidal propagation was performed using a repetitive mean tidal forcing. Extensive long-term data exists throughout New York Harbor for mean tidal range and phasing (NOS, 1981). The verification was performed initially in two-dimensions by using a single vertical layer. Once the frictional coefficients were determined, the model was run with five vertical layers for final verification. Mannings coefficients were used in the 2D mode and for the 3D verification the Mannings coefficients were converted into roughness heights. The Manning coefficients used are summarized in Table C-49.

The tidal phasing of high and low waters and the mean tide ranges for the overall harbor are presented in Figures C-117 to C-119. The times of arrival of high and low water are referenced to high water at Sandy Hook, New Jersey. The model results are plotted against the long-term field observations (NOS, 1981). The horizontal axis of the plot shows the locations progressively through the harbor. Figure C-117 shows the profile from Sandy Hook, through the lower harbor and up the Hudson River. Figure C-118 shows the profile from the Battery through the East River

and Long Island Sound. Figure C-119 profile runs from Sandy Hook, through Raritan Bay, Arthur Kill, Newark Bay and up the Passaic River.

30.6.3 Tidal Elevation Verification

The numerical model was also verified to the tidal elevation data collected during the May-June 1994 field surveys. The agreement between model and field data was good with the model responding correctly to the subtidal forcings. The elevations are presented in Figures C-120 and C-121.

30.6.4 Current Velocities

The mean tidal currents for New York Harbor were summarized by NOS (1988). The summaries present speed in knots for cross-sections running through the Ambrose channel, the lower Hudson River and for the lower East River. Model velocities were verified to that data.

The model velocities in and around Newark Bay were also verified to the detailed current information collected in May and June of 1994. The velocity distribution verifications within each cross section are presented in WES Report HL-95-XX.

30.6.5 Initial Conditions

For low river discharges, the residence time of Newark Bay has been estimated to be about a month. Therefore, the short term response of the model to changes in temperature and salinity is strongly influenced by the initial conditions. The initial salinity and temperature conditions were developed from the field observations collected by WES and National Marine Fisheries Service (NMFS) personnel. The initial conditions were developed with a vertical salinity gradient of 0.02 ppt/ft and temperature gradient of a negative 0.10 deg C/ft.

30.6.6 Salinity Verification

The 3D model was verified for the transport and mixing processes by comparing the salinity time series information computed by the model to the data collected in the field. A typical salinity verification run is presented on Figure C-122.

30.6.7 Temperature Verification

Temperature verification was developed at the same stations that were used for the salinity verification. Typical results are shown on Figure C-123. The general trends in the model are correct, but detailed fluctuations in the temperatures are not matched. The detailed temperature fluctuations are beyond the ability of this model given the accuracy of the data available. Defining the surface heat exchange, based on local distributions

of wind and air temperatures and identification of industrial heat sources would be required to obtain a more accurate temperature result.

30.6.8 Simulation for Water Quality Model Calibration

After the hydrodynamic model was verified to the May-June 1994 monitoring period, the simulation was extended through September 1994. The hydrodynamic data was needed to calibrate the water quality model to water quality data collected by Stevens Institute of Technology during the period of July through September 1994. The meteorological conditions for the period were again taken from Newark Airport.

30.7 Production Testing

30.7.1 Project Conditions

The testing for the modeling was designed to test either "wet" or "dry" tunnel conditions. The "wet" tunnel condition implies that approximately 700,000,000 gallons of water, trapped in the tunnel from a previous flood, would be flushed into the estuary by flood waters. For that purpose, test were conducted for both winter and summer ambient estuary conditions. The tunnel water temperature was assumed to be a constant 55 degrees F (12.77 Deg C). The winter receiving water temperature was assumed to be 1 deg C, and the summer at 28 deg C. These ambient temperatures were confirmed as conservative extremes based on the NMFS data collection performed for this project. The "wet" and "dry" conditions were tested for the 2-, 25- and 100- year hypothetical flood events.

The representation of the outlet structure within the 3D model was made assuming that initial mixing would be completed within a short distance of the outlet structure. The outlet structure was represented as a single cell in the grid (Figure 2.23). It was also assumed that the tunnel discharge would be uniform over the vertical water column. The modeling effort, therefore, only addresses far field mixing issues.

30.7.2 Storm Hydrographs

The flood hydrographs for the major tributaries were developed from the HEC-1 and UNET models of the Passaic and Hackensack basins. The HEC-1 and UNET models included all tributaries downstream to the mouths of the Passaic and Hackensack rivers at the northern end of Newark Bay. Other tributaries to the harbor were derived from rainfall events associated with the 2-, 25- and 100- year events.

30.7.3 Boundary Condition Development

The boundary conditions needed for the simulation of the production events included tidal forcings, river inflows and meteorological forcings at the water surface (heat exchange). The tidal forcings used in all production testing was derived from the verification period of May 1994. The existing condition tests used the May 1994 tidal boundary conditions directly; the future with project testing used the May 1994 tides elevated by 0.5 ft reflecting potential sea level rise in the year 2050.

The river inflows were developed from the UNET and HEC-1 data. The 2- year flood used the UNET and HEC-1 data for the Passaic and Hackensack basins and mean annual flows for the remaining tributaries. Local runoff for point and non-point loadings were developed from the 2 year rainfall data.

The 25- year UNET and HEC-1 flows were used for the Passaic and Hackensack basins and the April 1984 actual storm flows, which approximated a 25- year event on the Passaic River Basin, were used in the remaining tributaries. Local loadings were derived from the 25- year rainfall data.

The 100- year events used the UNET and HEC-1 100- year flows for the Passaic and Hackensack in conjunction with the April 1984 flows in the remaining tributaries. This approach was considered to be appropriate, because of the unlikely chance of having 100-year events on all tributaries at the same time. The local loadings were developed from the 100- year rainfall data.

The ambient meteorological conditions were deliberately equalized in order to isolate the impacts of the tunnel. The air temperature and heat exchange coefficients were set in equilibrium with the ambient water temperature. No wind forcings were applied. As a consequence, the model response to the differential temperature of the tunnel water became easier to trace and visualize.

30.8 Hydrodynamic Results

The operation of the tunnel would result in localized increases in velocities as the discharge emerges from the outlet structure. The 2D finite element model used in the sediment transport analysis provided the most accurate estimate of the local currents. The peak tunnel discharge currents are presented in Figure C-124 for the 100- year flood. The maximum currents associated with the base (un-improved) conditions, which are determined by the flow at the mouth of the Passaic River are shown in Figure C-125. The 2- year and the 25- year current comparisons are shown in Figures C-126 and C-127. Peak currents in the navigation channel at the tunnel outlet are shown on Figure C-128. Note that the time of occurrence of the maximum

currents is different between the plan and base conditions for all hypothetical floods. The diversion through the tunnel would cause flows to peak earlier than they would without the project.

30.9 Conclusions

The effect of the tunnel diversion on circulation patterns is localized to a zone around the outlet structure. The relative effect would not be significant because of the normally high currents associated with the baseline flood events. There is a concern for navigation conditions in the immediate vicinity of the outlet structure where the currents are as large as 3 fps for the 100- year flood, 2.5 fps for the 25- year flood and 2 fps for the 2- year flood. The peak currents are generally in the direction of travel but these currents could still hinder coincident shipping with tunnel operation. These circulation patterns and navigational impacts will be more accurately accessed in the FDM with the physical model and the ship simulation studies.

31. Water Quality Model of Newark Bay

31.1. Purpose

The purpose of the Water Quality Analysis was to provide information required to evaluate the effect of the diversion tunnel on living resources, primarily shellfish and finfish, in the vicinity of the tunnel outlet.

31.2. Approach

31.2.1. General

Following consultation with the National Marine Fisheries Service (NMFS), three living-resource parameters were selected for examination: salinity, water temperature, and dissolved oxygen concentration. Two mathematical models were used to evaluate the water quality with and without the diversion tunnel. The CH3D-WES hydrodynamic model, which was discussed in the previous section of this report, calculated the movement of water throughout the system. Transport computed by CH3D-WES was transferred to the CE-QUAL-ICM water-quality model which computed salinity, water temperature, concentration of dissolved oxygen along with other substances.

31.2.2. Salinity

Salinity was included largely because ambient salinity and short-term deviations from the ambient conditions would be important to the viability of living resources. For impact on living resources alone, salinity as computed in the hydrodynamic model was sufficient. Salinity was independently computed in the water-quality model to verify that transport was correctly routed from the hydrodynamic model. When the two models were correctly

linked, computed salinity was identical in both of them. Salinity was also computed since it influences the dissolved oxygen saturation concentration.

31.2.3. Temperature

Temperature was included largely because ambient temperature and short-term deviations from ambient conditions would be important to the viability of living resources. For impact on living resources alone, temperature as computed in the hydrodynamic model was sufficient. Temperature was independently computed in the water-quality model since it is a primary determinant of the rate of biochemical reactions. Temperature also influences the dissolved oxygen saturation concentration.

31.2.4. Dissolved Oxygen

Dissolved oxygen is required for the existence of higher life forms. Oxygen availability determines the distribution of organisms and the flows of energy and nutrients in an ecosystem. Dissolved oxygen is a central component of the water-quality model.

31.2.5. Ultimate Biochemical Oxygen Demand

BODu is a quantity that indicates the amount of oxygen required to completely decay, by biological processes, the organic matter contained in a water sample. BODu, in itself, has no impact on living resources. Computation of the BODu concentration and rate of exertion are required for the subsequent computation of dissolved oxygen.

31.2.6. Chemical Oxygen Demand

In the context of this study, chemical oxygen demand (COD) is the concentration of reduced substances that are oxidizable by inorganic means. The concentration is specified in oxygen-equivalent units (that is, the amount of oxygen required to satisfy the chemical demand). COD was included largely to allow for potential discharge of reduced substances such as iron, sulfide, and methane from the flood diversion tunnel. A second potential source of COD is sulfide released from benthic sediments when dissolved oxygen in the water column is insufficient to satisfy sediment oxygen demand.

31.3. Water-Quality Model Development

31.3.1. Model Modification

The CE-QUAL-ICM model applied in the Passaic River study is a modified version of the model originally developed for Chesapeake Bay (Cerco and Cole 1994). Modifications consisted of reducing the number of variables to a suite appropriate for the

problem of interest. The suite consisted of five state variables: salinity, temperature, dissolved oxygen (DO), ultimate biochemical oxygen demand (BODu), and chemical oxygen demand (COD).

31.3.2. Model Grid

As with the hydrodynamic model, the extent of the water-quality model grid was determined by boundary condition requirements. The boundaries were located at a distance such that the boundary conditions did not influence predictions in the vicinity of the tunnel discharge.

The need to model water quality in the Passaic River, the Hackensack River, and Newark Bay was obvious. Lack of impact of the tunnel on the Atlantic Ocean was also obvious. The extent of the grid was determined by a set of tracer studies employing the model. The water-quality model was first run on the complete hydrodynamic grid. Conservative tracer was discharged at a number of potential boundaries in the Kills, the Hudson, and New York Bay. Locations where significant amounts of tracer did not travel to the tunnel were suitable boundary condition points. Boundaries were set in the Hudson River at Manhattan, at the entrance to the East River, at the lower end of Arthur Kill, and just outside the Narrows. The water-quality model grid (Figure C-115) contained 1363 cells in each of five layers for a total of 6815 cells. The model time step was 5 minutes.

31.3.3. Conservation of Mass

The CE-QUAL-ICM model solves a three-dimensional mass-conservation equation for a control volume. Control volumes in the model correspond to cells in x-y-z space on the CH3D grid. CE-QUAL-ICM solves a conservation of mass equation for each volume and state variable. Solution to the mass-conservation equation is via the finite-difference method using the QUICKEST algorithm in the horizontal directions and a Crank-Nicolson scheme in the vertical direction. The kinetics portion of the conservation of mass equation for four state variables is described below. Salinity is omitted since no internal sources or sinks of salinity exist. The actual equations used in the model are documented in WES Technical Report EL-95-XX.

31.3.4. Temperature

Formal computation of temperature requires formulation of a conservation of internal energy equation analogous to the conservation of mass equation. For practical purposes, the internal-energy equation was written as a conservation of temperature equation. The only source or sink of internal energy considered was the exchange between the atmosphere and the surface model layer.

31.3.5. Ultimate Biochemical Oxygen Demand

In the modelled system, BODu originates in external loads including industrial and municipal discharges, combined-sewer overflows, and tributary inflows. Sinks of BODu are limited to oxidation in the water column and settling into bottom sediments. Oxidation is represented as a first-order process. Oxidation is a function of ambient temperature and dissolved oxygen concentration. As temperature increases, oxidation rate increases. Oxidation is diminished as oxygen availability becomes limited in the water column.

31.3.6. Chemical Oxygen Demand

In this study, chemical oxygen demand is the concentration of reduced substances that are oxidizable through inorganic means. Potential sources of COD include discharge from the diversion tunnel and release from benthic sediments. COD may exist as reduced iron and manganese, as sulfide, or as methane. For model purposes, these were represented as a single state variable quantified in units of oxygen demand. An exponential function was used to describe the effect of temperature on chemical oxygen demand.

31.3.7. Dissolved Oxygen

The sole source of dissolved oxygen to the water column is atmospheric reaeration. Sinks include BOD within the water column and oxygen demand exerted by benthic sediments. The rate of reaeration used in the model segments that contained an air-water interface, was proportional to the dissolved oxygen deficit. Reaeration is influenced by a host of factors including temperature, wind, and salinity. No theory that unites all these factors into a single estuary reaeration formula was available. In the model, reaeration was treated as a constant which varied spatially.

The saturation dissolved oxygen concentration were varied with changes in temperature and salinity.

31.3.8. Sediment-Water Interactions

Conditions in the water column are influenced, in part, by materials exchanges with the benthic sediments. Sediment-water interactions occur only in the cells at the bottom layer of the three-dimensional grid. For these cells, the mass-conservation equation was modified to account for the benthic sediment flux.

The primary exchange in the model was sediment oxygen demand (SOD). Oxygen consumption depends upon water-column temperature and oxygen availability. As temperature increases, respiration in the sediment increases. Sediment oxygen consumption is reduced as oxygen concentration in the overlying water decreases.

If dissolved oxygen in the water column is insufficient to satisfy the sediment demand, reduced substances would be released to the water column. The released substances could be in the form of reduced iron, manganese, or sulfide, which were represented in the model as chemical oxygen demand.

31.4. Data Collection

31.4.1. Water Quality Data

To calibrate and verify the Water Quality Model, a data set including salinity, water temperature, dissolved oxygen and BOD was required. Spatial extent of the observations had to include the entire water-quality model grid. A contemporary data set was preferred. No data set was located that met the requirements. Consequently, a sample program was conducted.

Three surveys were conducted, at roughly one-month intervals, from July 1994 to September 1994. Thirty stations were sampled. Complete coverage of the system required sampling over a two-day period. Surveys were conducted during daylight, close to the period of highwater slack tide.

Water-quality sampling included:

a) salinity,
b) temperature,
c) dissolved oxygen,
d) BOD5, and
e) BODu (60-day).

Salinity, temperature, and dissolved oxygen were measured in-situ (in place) using a hydrolab meter. In-situ measures were collected at mid-depth for shallow stations and at one-meter intervals for deeper stations.

Water samples collected for laboratory analyses. At stations less than three meters depth, one sample was collected from mid-depth and analyzed for BODu. At stations greater than three but less than ten meters depth, samples were collected one meter below the surface and one meter off the bottom. The surface sample was analyzed for BODu, the bottom sample for BOD5. At stations greater than ten meters depth, samples were collected at surface, mid-depth, and bottom. The mid-depth sample was analyzed for BODu, the remaining samples for BOD5. A median BODu/BOD5 ratio was computed and used to scale up the BOD5 measures to appropriate BODu concentrations.

Supplementary data for model calibration was obtained from surveys conducted by the New York Department of Environmental Protection. Data collected by the National Marine Fisheries Service, May 1993-June 1994, was outside the period selected for

model calibration but was employed to characterize the system. The US Environmental Protection Agency's STORET data base was accessed to provide temperature, dissolved oxygen, and BOD data at the system fall lines.

31.4.2. BOD System Loads

Specification of BOD loading was required at the model limits (fall lines) on the Passaic, Hackensack, Elizabeth, Rahway, and Raritan Rivers. Observations at all fall lines were examined for relationship of BOD to flow. Only the Passaic indicated a relationship. Higher BOD concentrations were observed at lower flows, indicating a dilution effect at high flows (Figure C-129). Mean BOD5 concentrations were used to characterize the remaining fall lines. BODu was obtained by scaling up the BOD5 measurements.

Point sources of BOD include the discharge from municipal treatment plants and industries situated along the shores of the system. A data base of point sources was constructed from information obtained from the National Oceanic and Atmospheric Administration and from the New Jersey Department of Environmental Protection.

Nonpoint-source loads are distributed loads that enter the system at the edge of stream below the fall lines. Nonpoint-source loads to the system are characterized by the predominance of combined-sewer overflows (CSOs). Nonpoint-source flows and loads were determined from a rainfall-runoff model created for a Section 208 study of the New York-New Jersey harbor system.

31.4.3. Boundary Conditions

Dissolved oxygen and temperature boundary conditions were required at the system fall lines. These were derived from data in the STORET data base. Salinity, temperature, dissolved oxygen, and BOD boundary conditions were required at the open boundaries of the Hudson River, East River, Arthur Kill, and the Narrows. Boundary conditions at these locations were specified from observations collected during the water-quality sampling.

31.4.4. Tunnel Water Quality

The one concern regarding the impact of the tunnel diversion on water quality would be the water that may be trapped in the tunnel between flood events. The expected interval between events would be variable and could be two or more years. During that period the water would undergo deterioration. If the oxygen demand of biodegradable material greatly exceeds the available dissolved oxygen, anoxic degradation could occur. The anoxic degradation results in the production of reduced iron, sulfide, and methane. These materials may produce structural damage and could severely impact the dissolved oxygen concentration of the receiving waters when the tunnel eventually discharges.

Initial indications of the effect of storage on tunnel water were obtained from examination of data retrieved from the STORET data base. Observations were collected from the Passaic River at Little Falls, downstream of the proposed tunnel inlet. At flows sufficient to operate the diversion tunnel, dissolved oxygen concentration (Figure C-130) equaled or exceeded ultimate biochemical oxygen demand (Figure C-129). Mean DO for flow > 2000 cfs was 10.1 mg/L; mean BODu was 7.1 mg/L. The data indicated that the biodegradable material in the trapped tunnel water, would be consumed before the dissolved oxygen was depleted. Consequently anaerobic degradation should not occur and production of reduced end products such as sulfide or methane is not expected.

An incubation experiment was also conducted to test the effect of storage on tunnel water. Two experiments were planned. Design called for storm water to be collected in the vicinity of the tunnel inlet and incubated for six months. Samples for the first experiment were collected during July 29, 1994. Flow during this event peaked at 1000 cfs, substantially less than flow which would cause the diversion to operate. Incubation of this water continued until February 1995. Since that time, no storm events suitable for initiating the second experiment have occurred.

Storm water was incubated in three 25 liter reactors, sealed against introduction of atmospheric oxygen. One reactor was set up to measure the volume of gas production. The other two were set up to allow samples of water to be withdrawn. Samples were withdrawn monthly for five months and analyzed for:

a) dissolved oxygen,
b) immediate oxygen demand,
c) BOD5,
d) BODu,
e) COD,
f) total sulfide,
g) reduced iron, and
h) reduced manganese.

At the end of six months, a final measure of gas evolution was conducted.

At initiation of the experiment, the BOD level of 8.5 mg/L in the sample water was characteristic of river water at the tunnel diversion flows levels. BOD analyses (Figure C-131) indicated aerobic degradation of organic matter proceeded until DO (Figure C-132) diminished to about 1.5 mg/L, then ceased. (Note. No dissolved oxygen levels were collected at the beginning of the experiment. Fall-line dissolved oxygen characteristic of the flow and season was plotted at Day 0.) No gas evolution occurred nor were detectable quantities of immediate oxygen demand, COD, sulfide, iron, or manganese observed.

The experiment largely confirmed expectations based on the data. However, over the course of the experiment, not all of the organic matter was degraded. In the reactors, a community adapted to the low DO conditions did not develop. A community adapted to low DO would be expected to develop in the tunnel. Tunnel retention time would be longer than the experimental incubation, and the tunnel would be exposed to organisms in seepage from groundwater and in bay water leaking through the tunnel gates. A conservative assumption was employed in the model. It was assumed that all organic matter in the tunnel water would consume all the dissolved oxygen during degradation. It was also assumed that no production of anaerobic end products would occur.

1.5. Model Verification

Parameters in the model were calibrated to the July-September 1994 field observations. The calibration run simulated the period July 1 to September 15, 1994. Initial conditions were obtained by "looping" the model until equilibrium with conditions on July 1 was obtained. Parameters employed in the model are summarized in Table C-50.

Model and observations were examined in a variety of formats including time-series comparisons at eight locations, spatial comparisons along three axes (Figure C-133), and scatter plots of predictions versus observations. Only a sampling of results are presented in this summary: time series in the center of Newark Bay and spatial plots along the Passaic River-Kill van Kull axis and the Hackensack River-Arthur Kill axis. Observations indicated little vertical stratification of any parameter in the bay. Consequently, model results and data are vertically averaged.

The time series comparisons (Figure C-134) indicate that model performance in central Newark Bay, downstream of the tunnel is very good. The primary feature of the data is that minimum dissolved oxygen in this portion of the bay is about 4 mg/L at temperatures of (20° to 25° C or 68° to 77° F). Both of these key living-resource parameters are well-represented in the model as is salinity, the remaining parameter of interest. Salinity observations, collected at high slackwater are within the daily range of model predictions. The data is typical of the remaining portion of the bay as is the model performance in this region.

The longitudinal plots are dominated by the lengthy reaches of the Passaic and Hackensack Rivers. The Passaic (Figure C-135) extends from km 16 to 44 of the profile originating at the mouth of the Kill van Kull, nearly two-thirds of the axis. The Hackensack (Figures C-136) extends from km 32 to 68, more than half of the axis. Newark Bay occupies km 7 to 16 on the Passaic River axis, the same information is shown as km 19 to 32 on the

Hackensack axis. Kill van Kull occupies km 0 to 7 on the Passaic River axis. Arthur Kill stretches from km 0 to 19 on the Hackensack River profile.

The majority of the Passaic is tidal freshwater in both the observations and model (Figures C-135). Predicted salinity upstream of the mouth of the Passaic is generally underestimated but cannot be improved through tuning of the water-quality model. Examination of the limited available data indicates temperature and dissolved oxygen in the Passaic generally exceeds 4 mg/L while temperature ranges from (20° to 25° C or 68° to 77° F). The model reproduces these conditions with one exception, an observed dissolved oxygen sag to 4 mg/L near km 4 and 19. The origin of this sag is not known. It may be a transient condition due to a discharge or it may be a permanent feature due, for example, to benthal deposits.

Observed conditions and model performance in the Hackensack largely mirror the Passaic (Figure C-136). As with the Passaic, predicted salinity upstream of the mouth of the Hackensack is generally underestimated but cannot be improved through tuning of the water-quality model. Dissolved oxygen in the Hackensack generally exceeds 4 mg/L and is reproduced well by the model except for a sag to 3 mg/L. The sag is likely due to to a transient load not detailed in the point source data. Temperature in the Hackensack exceeds adjacent waters. A temperature peak at about km 40 appears due to a concentration of power plants in the vicinity. A heat source was included in the model to account for power plant discharge .

31.6. Production Runs

Base scenarios specified future conditions without the tunnel. Future conditions were obtained by increase existing conditions sea level by 15 cm (0.5 ft). Wet-tunnel scenarios examined future conditions with the tunnel in operation and with flood water remaining in the tunnel between flood events. Dry-tunnel scenarios examined future conditions with the tunnel in operation and with the tunnel pumped dry between flood events. Duration of each scenario was 27 days. The first thirteen days were a "spin-up" period which allowed model loads, temperature, and boundary conditions to reach equilibrium. The spin-up period was followed by a fourteen-day storm simulation.

Three flood conditions were simulated: 2-, 25- and 100- year storm. The 2- year storm would be about the minimum storm which would trigger operation of the diversion tunnel. Storm flows in the Passaic and Hackensack basins for all scenarios were obtained from HEC-1 and UNET data. The remaining River and tributary flows for the 2- year storm were set at long-term average values. Tributary flows for the 25- year and 100- year storms were based on records of the April 1984 flood event, which was generally a 25 to 40-year event in the Passaic Basin The April 1984 coincidental flows were used to approximate typical flows in

other rivers during flood conditions on the Passaic. Daily rainfall values used to generate the hypothetical storm flows in the Passaic and Hackensack basins were obtained from the Department of Commerce. Technical Papers 40 and 49 and National Weather Service Hydro-35. These were employed in relationships derived from the Section 208 study (Hazen and Sawyer 1978) to generate nonpoint-source flows in the remaining basins.

Simulations were run for February and July conditions. These are the months in which the coldest (1° C, 34° F) and warmest (28° C, 82° F) temperatures were observed in Newark Bay. The first 700,000,000 gallons of water discharges from the wet tunnel was at a temperature of 12.8° C (55° F). The conditions were selected to provide worst-case conditions for examination of temperature shock on living resources.

At initiation of each scenario, the temperature of the receiving water (Newark Bay) was specified as the equilibrium value. Temperature boundary conditions (tributaries and air) were also set at equilibrium values. These conditions were specified to isolate the impact of the trapped tunnel water under the "wet" scenario. Receiving water initially at equilibrium with the atmosphere and boundary waters should remain at equilibrium except for the trapped tunnel water, discharged at a different temperature.

31.7. Results

31.7.1. Dissolved Oxygen

Emphasis was placed on the results of the 2- year and 100-year storms. Results from the 25- year storm are intermediate between the two extremes. Output from the scenarios was forwarded to the NMFS for analysis of impact on living resources.

The first series of plots illustrates minimum dissolved oxygen concentration along an axis, shown on Figure C-137, from Kearny Point (km -1) to Staten Island (km 9). The origin (km 0) is at the tunnel outlet. Base conditions are compared to wet-tunnel condition. The upper plots (on Figures C-138 to C-141) indicate minimum dissolved oxygen that occurs at any time during the storm. The lower plots indicate the time at which the minimum occurred.

The profiles (Figures C-138 to C-141) indicate the impact of the water discharged from a wet tunnel on receiving water is limited to the immediate vicinity of the tunnel. The distance and extent to which the minimum dissolved oxygen values with the tunnel, were less than the minimum values without the tunnel were examined. During the July simulation the dissolved oxygen impact zone was less than half a kilometer (0.3 miles) long. During February the oxygen depression was more extensive but critical depletion (less than 5 to 2 gm/m3) did not occur.

The second series of plots (Figures C-142 to C-145) are time series of dissolved oxygen, salinity, temperature, and BOD immediately downstream of the tunnel discharge. Base conditions are compared to wet-tunnel conditions. Origin (Hour 0) is at commencement of the storm.

The plots indicate that minimum dissolved oxygen downstream of the tunnel is depressed by flood water from the wet tunnel. The July plot indicates that minimum oxygen levels with the tunnel in place are only slightly lower than the existing levels (Figures C-142 and C-144). The primary difference between the base and wet-tunnel conditions is that the dissolved-oxygen minimum arrives earlier with the tunnel in operation. During February (Figures C-143 and C-145), the magnitude of the depression is larger than in July but minimum dissolved oxygen remains well above critical levels (5 to 2 gm/m3) for living-resources.

Figures C-146 to C-151 indicate that discharge from a dry tunnel improves minimum dissolved oxygen levels. Three factors appear to be improving the oxygen levels with the tunnel. Headwaters, at the tunnel diversion points, with high oxygen levels would be further aerated as they fell into the tunnel. The water would then be rapidly routed to the bay were oxygen levels are lower due to sediment demand and exertion of local BOD loads. The introduction of highly oxygenated tunnel water into the bay would increase oxygen levels in the Bay. In addition the high velocities associated with the tunnel outlet would cause a large amount of mixing which would oxygenate the Bay through increased air/water surface contact.

31.7.2. Salinity Response to Flood Flows

The impacts of the tunnel system on salinity in the tidal areas were simulated by both the Hydrodynamic and the Water Quality models. The results of both studied are combined and summarized below.

The salinity response of the system is controlled by fresh water inflows from both rivers and the tunnel. For base (without tunnel) conditions, the water in Newark Bay turns completely fresh during any major flood event. During the early part of a flood event pockets of fresh water are moved up and down the estuaries with each tide cycle.

With the tunnel in place, parts of the Bay may experience a change in salinity at an earlier point in the storm. While is effect may occur earlier it is no more severe and is slightly more rapid than the conditions that currently exist. This effect is illustrated by the time history of salinity for several stations, Figures C-142 thru C-145 are from the Water Quality model and Figure C-152 is from the Hydrodynamic model. Station 4A is located in middle Newark Bay.

The rate of salinity change was further evaluated since it was considered critical to benthic life forms. The rate of salinity change was computed for the entire simulation and the maximum rate of change identified for base and plan conditions. The location of the maximum rate occurs in the northwestern portion of Newark Bay near the navigation channel. The 100- year salinity rate of change at Station 4A is plotted for base and plan conditions at that location in Figure C-153. The units of the rate of change are given in parts per thousand per hour. The plan shows about a 10 percent increase in the rate of change in salinity, but the effect is very short-lived. The National Marine Fisheries Service (NMFS) stated that this minor change would not harm the benthic community.

31.7.3. Temperature Impacts

The impacts of the tunnel system on temperature in the tidal areas were simulated by both the Hydrodynamic and the Water Quality models. The results of both studied are combined and summarized below.

As discussed previously the temperature impacts of the tunnel diversion can be seen by viewing the plan temperature results no comparison to base conditions is needed. The base temperature results were designed to be uniform by setting all the air, estuary and tributary temperatures equal. Therefore, any deviations from the ambient temperature for plan conditions, would be associated with the differential temperature of the tunnel discharge. Only the initial 700 million gallons of water discharged from the tunnel were given a differential temperature. The dry tunnel plan had no impact on temperature because there was no temperature difference associated with flow that was not stored in the tunnel for an extended period.

The greatest impact of the wet tunnel was expected to occur for the 2- year flood. During the 2- year flood the water trapped in the tunnel would be discharged and then flushed by a lower volume of additional fresh flood water. For larger floods, the additional flows would act as an additional mixing agent. The maximum temperature impact occurs when evacuation of the trapped tunnel water is complete. Figures C-154 and C-155 show the hydrodynamic results while Figures C-142 thru C-145 shown the water quality results. The impact could be greatest for summer conditions when cool water from the tunnel is negatively buoyant and enter the bottom of the navigation channel. The bottom velocities would be slower and would mix more slower than the upper layers would. For winter conditions, the warmer tunnel water would be buoyant and stay near the surface, where strong currents mix the tunnel water with the cold ambient winter waters.

The temperature effects would be extremely short-lived. The freshly diverted river waters at ambient temperature follow the trapped tunnel water and add to the mixing process. As seen, the

impact would be very transitory and the tunnel discharge (55° F) would be providing temporary relief from the extreme (1° F and 28° F) ambient temperatures. The National Marine Fisheries Service (NMFS) stated that these temporary changes would not pose a threat to the living resources of the estuary.

31.8. Conclusions

The dissolved-oxygen impact due to the discharge of oxygen-depleted water stored in the tunnel would be limited in extent and duration. The impact is buffered by dilution and reaeration at the outlet. The volume of stored tunnel water, 2.65×10^6 m^3, is small relative to the volume of Newark Bay, 1.16×10^8 m^3. Consequently, the oxygen-depleted water effectively disappears when mixed with the much larger bay volume. Dilution is further enhanced by the large volumes of additional water that would follow the discharge of trapped tunnel water and the additional flows discharging from the Hackensack and Passaic Rivers. Rapidly reaerated, at the outlet, of the trapped tunnel water further reduces adverse impacts in Newark Bay.

The change in salinity associated with the tunnel is almost nonexistent. Due to the presence of both the Passaic and Hackensack Rivers, Newark Bay is subject to large fluctuations in salinity during major flood events. Construction of the tunnel would introduce fresh water into the bay a little sooner during major flood events. The rate of salinity change at the outlet will increase slightly due to tunnel discharges. The salinity changes associated with the tunnel are insignificant when compared to the natural conditions which currently exist in the bay.

The water trapped in the tunnel would reach a temperature of about 55° F (12.8° C). While a water temperature of 55° F is moderate is could be discharged into extreme winter or summer conditions. The same mixing and dilution processes that buffered the salinity impacts also diminish the temperature impacts. Under extreme conditions the maximum change in temperature due to discharge of trapped tunnel water would be about 9° F (5° C) at the tunnel outlet. The temperature impacts are very limited in duration and spatial extent.

The water quality and hydrodynamic analyses have been fully co-ordinated with the National Marine Fisheries Service (NMFS) along with other environmental agencies. NMFS personnel stated that "it (is) unlikely that the tunnel will cause any substantial mortality of fish or ichthyoplankton and very little mortality to benthic infauna."

32. Sediment Transport

32.1. General

In accordance with EM 1110-2-4000, sediment transport studies have been completed as part of the GDM. The Basin wide analysis consisted of 1) an HEC-6 model which was used to analyze the riverine sediment transport process down to Dundee Dam; 2) a two dimensional TABS model which was used to analyze the sediment process in the tidal areas; and 3) several short analytical procedures which were used to analyze sediment transport in the tunnel. All three analyses were performed by various personnel at the U.S. Army Engineer Waterways Experiment Station (WES). The various analyzes are summarized in this section of the report.

32.2. Past Studies

The Phase I GDM study concluded that the channel configuration was not susceptible to sediment movement, the stable conditions which existed would not change and the few areas which may have been subject to erosion (the lower Ramapo) would be protected with riprap or other suitable materials.

In October 1990, a limited sediment assessment was completed by the Waterway Experiment Station WES. Since the amount of detailed sediment data available at that time was limited, the conclusions drawn were fairly conservative and general. The report stated that the Ramapo and Pequannock Rivers (the only rivers specifically documented in the report) could have a tendency for deposition in the lower flow range and scour during the higher flood events. The formation of shoals followed by large flow events might lead to bank erosion and transport of sediment downstream. The report sited the collection of additional data as critical to the success of a detailed sediment study.

32.3. Riverine Analysis

32.3.1. Purpose

The purpose of the upland or riverine sediment transport study was to determine how the river systems would respond to the proposed flood reduction project. Changes in the sediment transport patterns around the tunnel inlets, along the reaches downstream of the diversion points and in the Great Piece Meadows were of particular concern. The results of the analysis were used to assist in locating structures in the rivers, identifying areas which might require maintenance dredging and to identify areas which might require erosion protection. Finally, the information from the riverine study was used as input to the tunnel trap efficiency analysis and the tidal sediment transport study.

32.3.2. Approach

An approach was taken which addressed the following unsteady characteristics of the Passaic system. The upper reaches contain river networks whose peak flows occur at different times. The unsynchronized peaking of flow can cause flow reversals in some reaches. A flow reversal occurs when water from one river flows upstream on an adjoining stream at their confluence. Historically, there has been a notable reversal at the confluence of the Passaic and Pompton Rivers. Since the Pompton River generally peaks first, it has been observed flowing up the Passaic and into the Great Piece Meadows. Another notable characteristic was that the river bed materials varied significantly from reach to reach.

An HEC-6, one-dimensional, multi-grain, sediment transport model was selected for the analysis. Initially, HEC-6 had no provisions for closed loop or reverse flows. Revisions to the code were made during the course of the study so that reverse and branching flows could be modeled. To simulate the phasing of the flow peaks, hydrographs from the UNET model were used as hydraulic input to the HEC-6 model.

The personnel from WES took the following approach to the study. All existing and available data, maps, and model runs were reviewed. A field trip was conducted to view bed forms, general channel characteristics, vegetation and network orientation. Suspended sediment and channel bed samples were collected. District personnel and data collection crews from the USGS were contacted for useful information. All information and data was assessed. Geometric, hydrologic and frictional data ("n" values) from the UNET model were modified and used to create an HEC-6 model with 406 cross-sections. Based on the channel slope, velocity, width, depth, and D_{50} the Yang sediment transport function was selected for use in the model. Concentration verses flow graphs were developed based on field data. The channel bed field data was also incorporated into the model. The HEC-6 model network is shown on Figure C-156. The model was verified to hydraulic and sediment field data as well as the UNET computed water surface elevations. Generally, four conditions were run:

1. Existing Conditions (Base, 2000)
2. Future Unimproved (Base, 2050)
3. Present Improved (Plan, 2000)
4. Future Improved (Plan, 2050)

The 2-, 25-, and 100- year floods were selected as the storms to simulate. The 100- year event represents the design event, the 25- year approximates the April 1984 event (at Little Falls) and the 2- year represents the channel forming discharge. The two year also represents one of the smallest events for which the tunnel would be used. The differences between the four basic

conditions were compared and evaluated. A more detailed description of the study and findings follows. The final WES Report HL-95-XX is expected to be published in the later part of 1995.

32.3.3. Sampling and Data Collection

32.3.3.1. Suspended Sediment

Suspended sediment samples were collected by the USGS in May and June of 1992 and March and April of 1994. Both depth integrated and point samples were taken. Analysis of the samples was completed by USGS at their Harrisburg, Pa. laboratory. The flows for both events were approximately a 1- year flood. Total sediment concentrations and the fractions of sand, silt, and clay were provided. These data showed concentrations from 2 to 20 mg/l. The locations at which they were obtained were Pompton Plains, Preakness Brook, Pine Brook and New Milford. An analysis of the particle breakdown showed an average of 36 percent sand.

32.3.3.2. Bed Material

In April and May of 1992 and again in May and October of 1994 river bed samples were obtained at a total of approximately 55 sites. The samples were obtained manually using a punch sampler, clamshell bottom sampler, and hand shovel. These samples were analyzed by WES at their lab in Vicksburg, Mississippi. The gradation curves showed a D_{50} that varied from 130 mm in the Pequannock River to 0.2 mm in the Great Piece Meadows.

32.3.4. Field Observations of Existing Channels

32.3.4.1. Upper Tributaries

The upper reaches, namely the Pequannock, Wanaque and Ramapo Rivers have fairly coarse bed material. The D_{50} ranged from 130 mm to 6 mm. Generally the areas of greater channel slope contained coarser materials, and the milder slopes contained the finer material.

The Wanaque River seemed stable except in the lower section near the mouth (at Riverdale Road) and just downstream of the Paterson-Hamburg Turnpike Bridge. The slope of the river is steeper in this section than the overall channel slope. The water runs very clear, and no large areas of sand bars were noticed. The banks appeared stable in the observed areas of the reach, and no vegetation was noticed in the main channel.

The upper Pequannock River has a varying slope, but is generally stable throughout its length. This seems to be due to the nearness of bedrock to the surface and the coarseness of bed material. An area of relatively flat slope in the lower one third of the reach appears to have been an area of past deposition. Otherwise, the channel characteristics resemble the

Wanaque River. The lower Pequannock River south of Riverdale Road has a constant overall slope except around station 46+00, where the slope becomes very flat and deposition is clearly evident. In this flat section there are sand bars, deposited fines, and plant growth. Near station 29+65 the bed material becomes coarser within about 1/8 of a mile of the existing Pequannock Weir, where fines are again more evident. Regarding bank stability, bank erosion scars were evident for some distance downstream of Riverdale Road. The Bypass channel (Branch 10) has a D_{50} of 16 mm and is rich in both sand deposits and gravel. The banks seem stable and are highly vegetated, the channel has minimal vegetation, and the slope is flat.

The upper Ramapo River has a varying channel slope, being moderately steep in the upper section, and very flat, even adverse, in the lower portion. There is no vegetation in the channel, and only moderate vegetation on the banks. Observed deposition is near the confluence with the Pequannock River. Other channel and hydraulic features of this river resemble those of the lower Pequannock River. The lower Ramapo River is an area of deposition due to the existing feeder dam at this location. Banks are highly vegetated, and the channel has moderate vegetation in some places. Bed materials are sands and fines, and velocities are less than 1/2 foot per second at low flows.

32.3.4.2. Pompton River

The Pompton River is about 7 miles in length. The channel slope varies, with the steepest section being in the upper reaches . The bottom material in the upper reaches can be relatively coarse, D_{50} of 40 mm, with no vegetation in the channel, and generally stable banks. Some bedrock appears upstream of the Pompton Turnpike. In the lower two-fifths of the river there are areas of deposition and great deal of channel sinuosity. The banks are unstable, as indicated by local resident's attempts at stabilizing their property. Approaching Two Bridges, the bed gradations contain more fines, with a D_{50} of 0.75 mm. However, at Two Bridges, the bed gradation becomes very coarse again with a D_{50} of 35 mm. Apparently the coarser material is the result of eddies and reverse flows during large floods at the confluence of the two rivers. Taken as a whole, the Pompton River seems to be the most active reach of the entire network with regards to scour, deposition, and bank stability.

32.3.4.3. Passaic River

Along the Passaic River from Pine Brook to Two Bridges, the channel is very sinous and the bed gradations are fine. In this reach the river passes through the wetlands of the Great Piece Meadows. There is generally no vegetated in the channel, but the overbanks are heavy vegetation during the growing season. In the vicinity of Two Bridges, the bed gradations become very coarse, as was the case in the Pompton River at this location. Downstream of Two Bridges and upstream of Interstate 80, there is

a zone of deposition. This could be due to the initial delta development caused by the backwater influence of Beatties Dam. Further downstream, the portions of river that were observed seemed to be stable. There are rock outcrops in the areas of the dams, and the few bed gradation samples had D_{50}'s ranging from 4 mm to 65 mm.

32.3.4.4. Deepavaal Brook

This reach has a varying slope, which is steepest in the upper sections. At its upper end, it receives overflow from the Passaic River during very large and rare flood events. Bed samples taken at the lower end had a D_{50} of 16 mm. Deepavaal Brook passes through the wetlands and industrial areas of Long Meadow. There is generally no vegetation in the channel, the banks are heavily vegetated during the growing season and the overbanks are often heavily developed.

32.3.5. Base (unimproved) Conditions Model

As stated previously, geometric, flow and stage information were taken from the UNET model and used in the development of the HEC-6 model. The geometric information became the base of the HEC-6 models (unimproved and improved). The stage information was used as a comparison to verify the operation of the newly formed HEC-6 model. The flow data was used to run the different conditions. Both the spatial and the temporal distribution of flow, that existed in the UNET runs, were reproduced in the HEC-6 runs. Throughout the simulation, no correction was made to the flows for changes in bed geometry.

Another important aspect of the model was the channel forming discharge concept. This concept proposes that the major features of channel geometry throughout the life of a river or stream are brought about by the floods that run near bank full. Floods of this size are generally associated with a 2- year flood frequency. The 2- year flood was simulated with the intention of using the event to determine the "long term trends" which might occur to the channel. The model results of these changes were not meant to be definitive in terms of absolute magnitude, but only in the sense of defining trends of deposition or scour. The changes to the network over 50 years were important to assess. In order to simulate such changes using HEC-6, specific historic hydraulic data were required. Since such data were not available, the channel forming discharge concept was invoked. The average daily flow at Little Falls is 1148 cfs which translates to about 36 billion cubic feet of water per year. The peak flow of the 2- year flood at Little Falls after diversion would be approximately 5220 cfs. About 80 continuous days at the 2- year flow rate would move about the same volume of water through the channel that is moved in a years time by the average daily flow. In order to simulate 50 years of volume using the 2- year flood, it would be necessary to run a simulation for 80 days per year for 50 years; or 4000 days of simulation.

32.3.6. Plan (Improved) Conditions Model

The plan conditions model was created by modifying the base conditions model. The UNET geometry files for the plan conditions were obtained along with maps and drawings. All the information obtained was incorporated into the model. The fraction of sediment that might pass into the tunnel inlets was roughly estimated and input into the model. A listing of the sediment grain sizes, their transport rate, and their approximate vertical location in the water column was obtained from the HEC-6 model. Knowing the river invert at the tunnel entrance, and the tunnel inlet crest elevation, it was possible to estimate what percentage of the different grain sizes in the water column could pass over the crest of the inlet and enter the tunnel. These percentages were entered as coefficients in the input deck. If a "0" (zero) was entered, no sediment of that size was transported out of the model and into the flow diversion (tunnel inlet). If a "1" was entered, 100 percent of the sediment in the flow that went into the tunnel left the model with that flow. In reality, most grain sizes will have coefficients less than one, with the larger grain sizes having smaller coefficients. These coefficients were calculated for the sand and gravel transported in the model for the 2- year and 100- year flood for both the main and spur tunnel inlets. The values are shown in Table C-51.

The coefficient values are only very crude approximations, being obtained from a one-dimensional model, and should not be considered definitive. The actual transport phenomenon will be governed by flow streamlines and secondary currents, neither of which are accounted for by a one-dimensional model. At present, a physical model must be used in order to obtain more accurate information regarding sediment transported into the tunnel.

32.3.7. Verification of Base Conditions

When all aspects of the model were completed, verification runs were made to check the hydraulic performance of the HEC-6 model. The year 2000 was selected to represent the initial conditions. The initial condition runs were made with a single time step of six hours so that no significant bed changes were allowed to occur. Without bed changes the HEC-6 bed profile matched that of the UNET runs, and a valid comparison could be made between the HEC-6 and the UNET water surface elevations. The instantaneous 2- year peak flow in the area of the Ramapo and Pequannock confluence, occurring on day 3 hour 2400, was selected as the flow against which to verify. It is important to note that while this flow represents a peak flow condition at one location, it only represents coincidental conditions at other locations. With the proper hydrologic input, and some minor adjustments to Manning's "n" values, the water surface profiles of the UNET runs were matched very closely. Figures C-157a and C-157b show some typical comparisons of HEC-6 and UNET water surface profiles. The complete set of verification figures can be found in the WES report.

32.3.8. Verification of Plan Conditions

The plan condition model (improved) was verified using the same procedures as in the base condition. The water surface profiles of the improved conditions UNET model were generally matched by HEC-6 plan conditions model. There were two minor discrepancies due to a pilot channel in one case and the modeling location of the Pequannock Weir in the other case. Overall, the comparisons adequately assure the validity of the hydraulic portion of the plan conditions model.

32.3.9. 50- Year Simulation (Base)

Both the base (unimproved) and plan (improved) conditions were evaluated after 50 years of flow (year 2050). The methodology used to accomplish this task is discussed above under the paragraph "Base (unimproved) Conditions Model". The instantaneous 2- year flow, occurring on day 3 hour 2400, that was used in the model verification was also used to run the model through its 50 year simulation. Again, the model was viewed in terms of trends, not in a sense of absolute magnitudes of bed changes. Figures C-158a thru C-158h show the initial condition (year 2000) bed and water surface elevations compared to the future conditions (2050). Some trends were noted on branches 1, 3, 4, 6, 11 and 13.

Along Branch 1 (Passaic below Deepavaal Brook) there is a small trend of deposition behind Dundee and SUM dams. This is a normal and predictable occurrence.

Along Branch 3 (Passaic at Two Bridges) a trend showing deposition in the area where the tunnel spur inlet would be located (between sections 1720+00 and 1730+00) was observed. This also occurs along the bottom of Branch 4 in the vicinity of Two Bridges. Later runs and further investigation showed that these areas will tend towards deposition but, the magnitudes shown in these plots are exaggerated due to the channel forming discharges that were selected. The channel forming discharge was made as a steady state run, and the time chosen happened to have reverse flow at the Pompton confluence. The reverse flow had a lower velocity and thus allowed continual deposition at this location. When a full hydrograph is run through the system, the series of depositing reverse flows followed by forward flushing flows produces the more realistic bed changes.

Reach 6 (the Pompton) shows signs of being unstable with a weak trend of degradation in upstream areas and deposition at the downstream end. The lower Pequannock River, Branch 11, and the lower end of Branch 13, the Wanaque, also shows a trend towards degradation.

32.3.10. 50-Year Simulation (Plan)

As in the base conditions, the plan conditions were evaluated after 50 years of flow (year 2050). The 2- year flow, at hour 2400 of day 3, that was used in the model verification was also used to run the model through its 50 year total volume simulation. Again, the model results were viewed as trends, not as absolute differences in bed elevation. Figures C-159a to C-159e show some of the year 2000 results compared to the year 2050 results for plan conditions. Figures which compare the base and plan conditions to each other can be found in the WES report.

While some trends were noted along many of the branches, only those along Branches 7, 10, 11 and 13 shown a variation from the base conditions . Along Branch 7 (Ramapo below the Pequannock) the scouring trend seems to have changed towards deposition. The deposition at the bottom of Branch 10 (Bypass channel) may indicate a trend towards shoaling at the inlet. The scouring trends noticed along Branches 11 and 13 during base conditions seem to have been reduced by a small amount for plan conditions.

Since all the "long term trend" computations were based on a single steady state flow (2- year at hour 2400 on day 3) which did not capture the peak flows of several tributaries, they are not completely representative. In addition the "long term trends" did not attempt to capture the impact of solitary large events with rising and falling flows which could move and drop large amounts of sediment. Therefore, further work was done to better analyze the full sediment transport process.

32.3.11. Short Term Impacts

The full 14 day unsteady 2-, 25-, and 100- year hydrographs were superimposed on the 2050 base and plan conditions (future unimproved and future improved) runs. Thus, the dynamic impacts of "short term" floods were added to the "long term" results discussed above. In addition, the full unsteady 2-, 25-, and 100- year hydrographs were also run with the year 2000 plan conditions (initial improved conditions). In this case the initial condition runs (year 2000) were not just one time step, but a 14 day hydrograph of the given flood frequency.

For each condition, the maximum water surface elevations attained were plotted. These plots show the differences between base and plan conditions for the various flood frequencies in the year 2050. More importantly, they also show how the plan condition responded after 50 years of both "long term" and "short term" sediment movement. These simulations highlight areas of long term capacity loss and areas of possible maintenance problems.

All graphs for the 2-, 25-, and 100- year events are contained in the WES report. Since the information from all three simulations was remarkably similar, only the 100- year

event, which is the design event, has been presented in this report. Figures C-160a thru C-160m are the plots of the 100 year initial and future improved conditions. Detailed conclusions of the entire sediment transport analysis are discussed below. Generally, the analysis described in this paragraph showed that the proposed project would not loose flood conveyance due to sediment movement. At three locations flood levels may drop about 1 (one) foot due to 50 years of operation. Some sediment may accumulate at both inlets and in the section of the Ramapo that is behind the Old Morris Canal Feeder Dam. There are also three areas in the upper tributaries that could be subject to scour but, these locations have already been identified for some type of channel protection.

32.3.12. Conclusions and Recommendations

The HEC-6 model confirmed that with the plan in place, considerable reductions in water surfaces elevations were attained in most branches of the network. The greatest reductions were for the 100- year flood frequency. The differences between the proposed project condition in the years 2000 and 2050 are relatively small. This indicated that the plan itself would not be self defeating at any noticeable location. In other words, the plan would be generally stable, would remain stable through 50 years of operation and through the various flood hydrographs. Thus, there is no indication that excessive long term maintenance would be required.

The main places of concern for the sedimentation process were at Two Bridges, the Great Piece Weir, the tunnel inlets, and the Pompton River. At Two Bridges there was a special concern with the amount of deposition noticed during field observations and from the long term model runs. Especially, since the Great Piece Weir was proposed in this vicinity. It was concluded from field observations and the model runs, that while the project may reduce sedimentation in this area, deposition of up to 3 feet might occur after the 100 year flood. This could indicate a necessity for short term maintenance after extreme events. However, the proposed location of the Great Piece Weir, 700 feet upstream of Two Bridges, appears to put the weir at the edge of this deposition zone. Upstream of the Great Piece Weir there may be minimal deposition of fines from the wetlands.

Without exception every run showed the area of the spur inlet, Branch 3, as a zone of deposition. As previously mentioned, it could be due to the initial delta development caused by the backwater influence of Beatties Dam. Again the amount of deposition at this location seemed to be reduced by the plan, possibly as a result of the reduced degradation in the Pompton River. A sediment maintenance program would need to be developed for this area. The proposed physical model study will be used to further analyze this possible high maintenance area.

An investigation will be made to optimize the pilot channel as a sediment bypass. The amount of sediment entering the tunnel was analyzed and is documented in the next section of this report.

The Pompton River appears to be the most unstable reach in the network. It shows significant bed changes with a barely discernible trend on degradation in the upper sections, and possible deposition in the lower reaches. The rate of change of either seems to be reduced by the plan. Comparing the 100 year plan runs to the 100 year base runs shows that not only are the water surface elevations lower in the plan, but the slope of the water surface is less. This accounts for the reduced rates of degradation. However, it is recommended that photo-documentation of the stream banks be undertaken to show that this area is currently unstable. If not, future degradation could be mistakenly attributed to the project, when in fact the project seems to reduce the rate of degradation.

At the upper end of the Pompton River in the vicinity of the main inlet, a very slight indication toward scour was indicated. In Branch 10, the Bypass Channel, a tendency for deposition seemed evident. Due to the channel improvements at this location, some deposition was expected as the system tries to return to equilibrium. As with the spur inlet, defining the percentages and distribution of sediments that will actually enter the tunnel is beyond the capabilities of the HEC-6 model and will be addressed by the physical model study. The physical model would help to determine the types and amount of materials that would accumulate directly in front of the tunnel inlets, and thus help define the maintenance requirements to keep the sediment basins and gated structures operating effectively.

The section of the Ramapo River between the Pequannock and the Pompton rivers, Branch 7, displayed an increase in deposition due to the project. This location is behind the old Morris Canal Feeder Dam and would be subject to deposition. The project however seem to increase deposition by diverting flow away from this section of the river during large events. While the water surface elevations are largely unaffected by the sediment, this area would need to be included in a regular maintenance program.

Two areas of possible scour were also noted. In the vicinity of station 143+00 along Branch 12, a potential scour hole was identified. During the channel protection analysis (see Section 34.) riprap and articulated concrete matting was specified for this area. The lower section of Branch 13 was also identified as a potential scour area. While the riprap computations indicate that no riprap would be needed there, a layer of ungraded crushed stone was specified between the mouth and station 26+50 to ensure that bed material was not lost.

32.4. Tunnel Sediment Trapping Analysis

32.4.1. Purpose

The primary purpose of the tunnel trapping analysis was to determine if the capacity of the tunnel would be reduced by the accumulation of sediment in the tunnel. The secondary purpose was to determine the maintenance requirements needed to keep the tunnel free of sediment. The basic approach was to determine how much of the river sediment entering the tunnel would not exit. The amount of sediment leaving the tunnel was used as input for the tidal sediment transport study.

32.4.2. Approach

The tunnel sedimentation analysis was prepared by "Mobile Boundary Hydraulics" (a consulting firm) for WES, for the Passaic River Division. The final report will be available some time after the completion of this draft report. Estimates of water and sediment discharges entering the tunnel were obtained from the riverine sediment transport analysis described in the previous section.

Three methods were used to estimate the amount of sediment that might be trapped in the tunnel. The first method was to compare the settling velocity of (non-cohesive) sediment particles entering the tunnel to the exit velocities in the outlet shaft. The second approach was to analyze the erosion shear stress threshold for sediment in the tunnel and to compare that to the shear stresses induced by flow in the tunnel. The third and final method was to approximate flow in the tunnel with several open channel flow equations and to use an HEC-6T numeric model to estimate sediment transport.

32.4.3. Analysis

32.4.3.1. Settling Rates

For this method the velocity at the largest point in the vertical outlet shaft was computed. Since the widest section of the outlet shaft is 45 feet in diameter each hydrograph flow value was divided by 1590 sf, the area of a 45 foot diameter circle. A table of the velocities for the 2-, 25- and 100- year hydrographs was created. Using the data from the riverine sediment transport analysis (HEC-6), the size and amount of sediment entering the tunnel for each event was put in the table. Finally, a spreadsheet was used to compare the velocities in the outlet shaft, to the settling velocities for each size of non-cohesive sediment particles. The spreadsheet was used to keep track of any sediment that did not have sufficient velocity to escape the tunnel. Sample spreadsheets are shown in Table C-52. The full set of spreadsheets and computations will be in the final WES report. The analysis indicated that none of the non-cohesive sediment would be trapped in the tunnel. However, since

some water would remain in the tunnel after each event, some residue sediment might remain in the tunnel with the trapped water. The residue would be washed out of the tunnel during the next major event.

32.4.3.2. Shear Stress Method

The movement of cohesive sediments in the tunnel was analyzed by the following method. First, the shear stress of the water in the tunnel was computed. The computations were based on the Smooth Wall Velocity Law and the average flow velocity. Figure C-161 show the computed shear stresses for each event. Next, the critical shear stresses of the cohesive sediments were obtained from published sources. The critical shear stresses were based on the sediment plasticity and clay content. The calculated shear stresses in the tunnel were, at times during each event, sufficient to erode even dry sediment. Since, the tunnel invert would always be wet and since the critical shear stresses for wet materials is less than that for dry, the cohesive sediments in the tunnel would be suspended.

The shear stress method was also used to predict the movement of non-cohesive particles. The maximum grain size, estimated by the HEC-6 analysis, to enter the tunnel was 2 mm. The critical erosion shear stress for a 2 mm quartz particle ranges from 0.025 lb/sf to 0.06 lb/sf. Only the 25- and 100- year computed shear stresses were large enough to move quartz sediment. Therefore quartz sediment might deposit in the tunnel during small events but it would be moved by larger events.

32.4.3.3. HEC-6T Open Channel Approximation

An additional analysis of non-cohesive sediment transport was made by approximating open channel flow conditions. The open channel flow assumption allowed the using of an HEC-6T computer model to simulate the sediment transport. The open channel approximation involved taking the average tunnel velocity for a selected discharge and calculating an equivalent energy slope. The energy slope was then used for other flows to compute a range of Mannings roughness "n" values. The tunnel was then turned into an open channel, placed at the computed slope and analyzed with the computed "n" values. Four hydrographs were linked together to form a continuous hydrograph. The sequence of hydrographs was 2-, 25-, 100- and 2- year hydrographs. The sequence involved all three hydrographs and acknowledges that the smaller events occur more frequently. The results of the analysis indicated that residue would be left in the tunnel during the falling portion of the discharge hydrograph. The overall trapping efficiency is about 3 percent. The sediment would not expected to accumulate in the tunnel but would expected to be flushed out during the peak flow of the next flood.

32.4.4. Conclusions and Recommendations

All methods of analysis indicated that the sediment carried into the tunnel could be carried out of the tunnel. Since some water would remain in the tunnel after the flood has subsided, some residual sediment would settle in the tunnel. Any residual sediment in the tunnel would be resuspended and expelled during the next or larger flood events. The residual sediment in the tunnel should not require maintenance and it should not affect the hydraulic capacity of the tunnel system. The physical model study proposed for the FDM will re-evaluate the three dimensional movement of sediment into the tunnel. The results presented here are based on a one-dimensional analysis and may vary as the level of analysis improves.

32.5 Estuarine (Newark Bay) Analysis

32.5.1 Purpose

An estuarine sediment transport analysis was also conducted by personnel at WES. The purpose of the study was to determine the possible impacts of the Passaic River Flood Damage Reduction Project on sediment movement in the tidal portions of the Passaic and Hackensack Rivers, Newark Bay and adjacent waterways. The movement of sediment into the navigation channel was of particular concern as was the tracking of Passaic River sediment.

32.5.2 Approach

The modeling approach for the estuary was guided by two factors. First, the complexity of the estuarine geometry along with the design and placement of the outlet structure required a highly flexible discretization of the geometry. This requirement pointed to a generalized finite element modeling approach. Secondly, the characteristics of the estuary lend themselves to a two-dimensional sediment model. The Corps approach for similar projects has been the use of STUDH, the two-dimensional sediment transport component of the TABS-MD modeling system. In addition, when the study was initiated, there were no candidate finite difference sediment transport models available for immediate application to the project. Therefore, STUDH was selected as the sediment transport model..

32.5.3 Hydrodynamic Model

The STUDH approach required that a 2D hydrodynamic model be used for the development of the velocity data required by the transport model. The TABS-MD two-dimensional model RMA-2 was used the develop the hydrodynamic application.

32.5.4 Sediment Transport Model

The sediment transport model STUDH, a component of the TABS-MD modeling system, solves the depth-averaged convection-

diffusion equation with source-sink terms for exchange with the bottom. The model uses a finite element formulation with quadratic basis functions for interpolation of the concentration within each element. The model simulates both cohesive (clays) and noncohesive (sands) sediments. A source-sink term is defined by the shear stresses computed from the hydrodynamic model. With regard to erosion the sediments within Newark Bay behave as cohesive sediments since there are sufficient clay minerals present to trap fine sands and silts within the cohesive bonds.

The geographic limits of the sediment transport model were developed based on previous modeling experience. The tributaries were simulated to the heads of tide. The forcing of tides from Long Island Sound have previously been found to be critical for proper interaction through the East River. If the Long Island Sound boundary is located too close to the upper East River, minor inaccuracies are dramatically amplified and model instability arises. Therefore, the Long Island Sound boundary was placed near Montauk Point. The Atlantic tidal forcing was successfully applied just below the Narrows, using Ft. Hamilton tidal data. A secondary tidal forcing from the Atlantic was applied at the mouth of Arthur Kill using the Perth Amboy tidal records.

The sediment sources for deposition within the study area are believed to be from the upland areas. However, short term sedimentation is normally a result of redistribution of existing material by erosion from tidal currents and waves. The existing sediment concentrations are relatively low (less than 20 mg/l) during typical tidally dominated flows. Sediment inflow at Dundee dam and the tunnel outlet were obtained from the Riverine and tunnel trapping analyses.

32.5.5 Hydrodynamic Model Verification

The 2D hydrodynamic model was verified to the same field data that was used to verify the 3D hydrodynamic model used to drive the water quality model. The tidal propagations were established with the mean tidal boundary conditions. The tidal conditions for the May-June 1994 field data collection effort were also simulated.

The velocity verification was achieved by comparing the modeled velocities to the velocities observed during May-June 1994.

32.5.6 Sediment Transport Model Verification

The sediment transport model was also verified for the May-June 1994 period. The base assumption was that the cohesive bed must be brought into equilibrium with the hydrodynamic forces in the system before a verification test could be run. This concept was supported by the low suspended concentration levels measured in the field survey.

The sediment model was initialized with a cohesive bed structure representative of the sediments found in the harbor. The bed was defined as ten layers of material with an initial layer thickness and shear strength. The model was then run through a series of simulations using the tidal shear stresses experienced during the verification period. During this series the ambient currents were allowed to erode the softer layers of the movable bed.

This approach was used to obtain the low level of concentrations observed in the field. The intertidal processes involve the temporary deposition of suspended material near slack waters and the subsequent resuspension of the material during higher tidal currents. Figure C-162 shows the concentrations of suspended sediments computed by the model versus the measured concentrations.

The deposition patterns for the verification simulation are presented in Figure C-163. The patterns show deposition in the navigation channels at an annual rate of approximately 300,000 cubic yards, which is the same order of magnitude as field experience.

32.5.7 Production Testing

Testing for the sedimentation study involved simulation of the 2-, 25- and 100- year events for existing conditions and for future with the project conditions. The project conditions required the revision of the computational mesh (Figure C-164) to included detailed resolution in the area of the tunnel outlet structure.

The boundary conditions for the sediment modeling were derived from the same sources as for the water quality modeling with regard to the tributary flood flows. The suspended sediment concentrations at the inflow boundaries were derived from the upstream (riverine) sediment transport modeling.

The May 6, 1994 conditions were used to initialize the verification runs and the production runs. The initial sediment concentrations were also set to the values observed on May 6, 1994. The bed structure developed in the verification runs was used as the initial bed for the production runs.

32.5.8 Results

The greatest shear stress in Newark Bay occurred at the mouth of the Passaic River for the base 100- year flood, and at the outlet structure for the plan tunnel diversion. These shear stresses were taken directly from the sediment transport model.

The impact of the tunnel diversion on erosion and deposition patterns is presented in Figures C-165 to C-168. Figures C-165a and C-165b present the base conditions erosion patterns for the

2- and 100- year flood tests. The tunnel plan patterns are presented in Figures C-166a and C-166b. The primary effect of the tunnel diversion is to reduce the erosion that would occur in the mouth of the Passaic River for the base condition. The peak erosion in the mouth of the Passaic River was reduced from 10 cm for existing conditions to 3 cm with the plan for the 100- year flood.

The deposition patterns are presented for the base tests in Figures C-167a and C-167b. The plan test deposition patterns are shown in Figures C-168a and C-168b. Both base and plan show the same general deposition pattern with the plan conditions having a slightly smaller magnitude. The relative volumes of deposits in the channel reaches are summarized in Table C-53.

32.5.9 Conclusions

As a result of the tunnel diversion, flow during major flood events would be reduced in the lower section of the Passaic River. The reduction in flow would result in a slight reduction in scour.

The wider distribution of flow and the reduction in scour associated with the tunnel, would result in a slight reduction in deposition. In conclusion, the proposed project would produce a slight reduction of sediment transport in the Upper Newark Bay and the Lower Passaic River.

33. Maintenance

33.1. Maintenance Dredging

Several areas will require maintenance dredging or periodic inspection. Each area is described below. The recommendations made below should be adjusted to meet the conditions which would exist after construction.

33.1.1. Pompton Inlet

The area surrounding the Pompton Inlet would be subject to some deposition. Analysis indicates that this area could accumulate approximately 4 feet of sediment over a 50 year period. It is estimated that maintenance dredging would be required every 20 years.

33.1.2. Great Piece Weir to the Spur Tunnel Inlet

The channels leading to the Spur Tunnel Inlet would be subject to some deposition. Analysis indicates that this area could accumulate approximately 1 to 4 feet of sediment over a 50 year period. A maintenance dredging frequency of 20 years should be sufficient for this area.

33.1.3. Lower Ramapo River

The section of the Ramapo River between the Pequannock Weir and the Pompton Inlet would be subject to some deposition. Analysis indicates that the area behind the Ramapo Weir could accumulate about 4 feet of sediment over a 50 year period. Once again a 20 years interval between sediment removals should suffice.

33.1.4. Pompton River

The Pompton River is currently a very active river. Shoals seem to be moving along the river while erosion is a common occurrence. While the proposed project should act to somewhat stabilize the river, monitoring and periodic inspection is recommended.

33.1.5. Tunnel

Analysis indicates that the tunnel should be self cleaning. The physical model study, which will be conducted for the FDM, will produce more detailed results. If sediment removal is necessary the tunnel has been designed to collect sediment at Shaft #2C and at the outlet. Both locations are suitable for sediment removal. In addition specialized equipment, stored at the operations center, can be placed in the tunnel at Workshafts 2 & 2C. The tunnel equipment would move sediment to removal locations if necessary.

33.2. Mechanical Maintenance

There would be 18 major gates, 60 gated closure structures, 20 pumps, 200 flap gates, 200 sluice gates, 14 gages and a large amount of monitoring and operational equipment in the project. All of the devices enumerated would require annual inspection and exercise.

33.3. Levee Landscaping

There would be about 6.5 miles of levees in the project. Vegetation along the levee would have to be cut in the fall every year. In some locations, vegetation would be cut more often to facilitate recreational needs.

33.4. Tunnel

The tunnel should be inspected after every usage. On average the tunnel would be used once in a two year period. The pump station at Shaft #2C would be able to evacuate the lower tunnel in about 14 days. The upper tunnel will drain by gravity and can be inspected whenever the tunnel is not in use.

33.5. Personnel Requirements

It is estimated that at least one full time person with some part time assistance would be needed to properly supervise, monitor and maintain the project. Past experience indicates that proper maintenance would be obtained if Corps of Engineer's personnel were

used for the task. Using the operations center as a Corps of Engineers Operations/Construction Area Office would insure that experienced personnel was available when needed.

34. Channel Protection Analysis

The design for the protection of earthen channels against erosion is based on the criteria contained in: EM 1110-2-1601, "Hydraulic Design of Flood Control Channels" dated 1 July 1991, and Change #1 dated 30 June 1994.

Riprap requirements were analyzed for sections of the Passaic, Pompton, Pequannock, Ramapo, and Wanaque rivers, Deepavaal Brook and Newark Bay. All locations of proposed channel modifications were evaluated. In addition, any unimproved locations where channel velocities are expected to increase by 25% or more as a result of the project were also evaluated for erosion potential. Generally, the 100 year design flow was used to size the riprap. However, to design the riprap protection for the Great Piece Weir and Pequannock Weir gates, the critical condition involved the sudden opening (mis-operation) of one gate during dry or low tailwater conditions.

The channel velocities and depths which were used to determine the scour potential, were obtained from the improved conditions UNET model. The Mannings "n" values which were used for the improved conditions model were the same as those determined in the calibration process described previously. The decision to keep the same friction factor was based on the fact that newly excavated channels generally return to the same frictional "n" value as time goes on and vegetation matures.

A LOTUS spreadsheet was used to analyzes the riprap requirements. Information about the channel side slopes, channel widths, average channel velocities, depths of flow, bend radii, bend angles and stone density were part of the LOTUS computations. The recommended riprap layer thicknesses were computed in the spreadsheets. Special evaluations were done for the areas just upstream and downstream of all the gated structures and at head sections where the improved channels meet the existing channels. The LOTUS results were then modified to incorporate special protection areas and to aid construction by minimizing the number of transitions from one riprap size to another.

At the head or tie-in section of the Pequannock channel articulated concrete matting was called for because the required riprap thickness was getting rather large (42"). There are several types of articulated concrete mats. Two specific systems were evaluated for application in this project. One type of matting (Armorflex) consists of square blocks that are tied together with cables. The other type (Tri-lock) consists of two different trilateral blocks that interlock with each other. Both systems have blocks that are designed to withstand the

anticipated velocities (about 15 fps). It may also be economical to place articulated mats at the head of the Wanaque channel and at the area just downstream of the outlet structure. Further analysis of the cost effectiveness and the exact size of blocks required can be done during the next level of design (FDM).

In additional some areas were identified in Section 32., Sediment Transport, as possible locations of significant scour, but they did not require riprap protection. In those areas a 6" layer of ungraded crushed stone was called for.

To determine the sensitivity of the riprap analysis, the conveyance of the channels was changed by 10% and the performance of the proposed riprap was evaluated with the resulting higher velocities. Generally there was almost no change in the velocities, however in a few areas changes of less than one foot per second were observed. In the areas where the increases in velocity were the largest, no riprap was required both before and after the conveyance was changed. At one location the changing in velocity of 0.5 feet per second resulted in a riprap layer thickness increases of only 3" inches but the layer had already been increased to simplify the construction process.

Attachment 5 contains a Sample Riprap Computation for one of the gated structures and a Sample LOTUS worksheet. The location and layer thicknesses of the required riprap are indicated on the profile and plan sheets. Tables C-54 and C-55 specify the riprap requirements. Note that the riprap specified for the outlet structure is actually a 36" layer which has been increased to 54" to compensate for the difficulty associated with deep underwater placement. The bedding required for each riprap layer is specified in Table C-56. The bedding material is an unspecified gradation of crushed stone.

35. Levee/Floodwall Hydraulic Designs

35.1. Establishment of Finite Levee Heights

Final levee heights for the Central Basin and Pequannock-Ramapo Levee were determined by adding an allowance for uncertainties to the water surface elevation for the 1% chance design profile. Uncertainties that were accounted for in the analysis were: flow, channel roughness, debris obstructions at bridges and blockage of tunnel inlet gates.

A sensitivity analysis of uncertainty for the 1% chance discharge value was conducted. Profiles with a 10% and a 20% flow increase above the best normal estimate were computed. The increase in stage from the 10% increase in flow ranged from 0.4 feet at Passaic #10 up to 0.9 foot on the Ramapo River. The 20% increase in flow resulted in a 0.8 foot increase on the Passaic River at Passaic #10 to 1.9 feet increase on the Ramapo River. The other proposed levee sites fell between that range, most at

1.2 feet, as shown in Table C-57. The 20% increase in discharge was selected as a component in determining the "design freeboard" for the final levee heights.

A sensitivity analysis of the effect of variance of channel roughness was also performed. Increases of 10% and 20% in the Manning's n values were simulated within the UNET model by decreasing the conveyance by 10% and 20% for the respective computations. Conveyance is inversely proportional to the Manning 's n roughness coefficient. The increase in water surface due to the 10% change in conveyance ranged from 0.6 feet on the Rockaway River to 0.8 foot on the Ramapo River. The water surface increase due to the 20% decrease in conveyance ranged from 1.2 feet on the Rockaway River to 1.6 feet on the Ramapo River.

Increases in water surface elevation at the levee sites due to debris blockage of bridges and tunnel inlet gates was minimal. The maximum increase due to this factor was 0.2 feet at the Passaic #10 levee site.

The combined effects of uncertainties in discharge, conveyance and debris blockage were determined and used to set the minimum designed freeboard water surface profile (Column (3) on Tables C-58 thru C-65). The minimum design freeboard profile value calculated was 2.1 feet on the Black Brook tributary to the Whippany River for the Pinch Brook levee/floodwall system. The maximum value determined was 3.5 feet on the Ramapo River.

35.2. Overtopping Analysis

To determine the pre-final levee height as shown on column (6) of Tables C-58 to C-65, the original normal flows were increased in the UNET boundary condition models until overtopping occurred at the preferred location. The preferred location for overtopping was generally either the downstream-most end or at the interior drainage ponding site. This computed water surface profile was compared with the minimum levee profile and the higher of the two elevations was used as the pre-final levee height, shown in column (6). Finally, to assure "superiority" of the levee in the area where overtopping would create the greatest potential hazard, one-half foot of additional "freeboard" was added at those sites, tapering to zero at the overtopping areas. The addition of this "levee superiority" value yielded the final levee height, shown in column (8).

Finally, column (9) on those same tables indicates the "design freeboard", or difference in elevation between the final levee height and the design water surface elevation. The Pequannock-Ramapo levee, shown in Tables C-64 and C-65, had the greatest increase over the design water surface, 3.5 to 3.9 feet. This reflects a greater sensitivity to uncertainties, primarily due to the steeper channel slope and at the Pequannock-Ramapo levee and the potential operational constraints at the proposed

Pequannock Weir. Rockaway #1, #2 and #3 levees, shown in Tables C-60 to C-63, have an increase of 2.5 to 2.9 feet over the design water surface elevation. The difference between final levee height and design water surface elevation at Passaic #2A levee (Table C-58) varies from 2.3 feet at the downstream end to 3.6 feet at the upstream end. The crest of the remaining levee, Pinch Brook (Table C-59) varies from 2.2 to 2.6 feet above the design water surface elevation.

35.3. Debris Evaluations

As part of the flowline sensitivity and levee overtopping analysis, an investigation of the potential development of debris jams at bridges was conducted for the Passaic River and its tributaries. The objective was to determine the potential change in water surface elevations due to the accumulation of debris. It was assumed that debris jams would occur at structural crossings of the river; ie. bridges, walkways, culverts, aqueducts, etc.

The accumulation of debris at any bridge near or just downstream of a proposed levees could adversely effect the finite levee heights provided at that location. Therefore all bridges near proposed levees, were selected for field inspection.

Photographs of other existing structural crossings in the Passaic River basin were then examined, and several river crossings were identified as having debris jam potential. A total of eighteen structures were isolated for further investigation. Ten structures were eliminated by conducting a field inspection. Eight structures were found to have a high debris jam potential.

At these eight structural crossings, the amount of debris accumulation was noted during the field inspection. Judgement was used to estimate the amount of debris that could accumulate during a flood event. The potential accumulated blockage area was compared to the overall flow area of the structure. Reduced bridge areas were then coded into the UNET model to obtain new water surface elevations. The following table describes the degree of blockage which was estimated for the 1% chance flood event.

STRUCTURE	DEBRIS
1. New Dutch Lane Bridge	10' Wide around the pier
2. Jersey City Aqueduct	3-4' Snagged on the dam
3. Route 46 Bridge	33% (Closed right bay of three)
4. Route 80 Bridge	16% (Closed half of right bay)
5. Borig Place Bridge	50% (Between bridge and hanging pipe)
6. Erie R.R. Bridge	70% (closed 7 on 8 bays)
7. Edwards Road Bridge	33% (debris in hanging girders)
8. Route 10 Bridge	15% (12' on pier and closed half of right bay)

35.4. Interior Flooding Analysis

35.4.1. Central and Pompton Basin Interior Flooding Analysis.

As part of the Central Basin studies, 6 potential sites were evaluated to determine with project interior flooding stage-frequency curves. These sites were: 1) Pinch Brook, 2) Passaic #10, 3) Passaic #2A (along the Passaic River), 4) Rockaway #1, 5) Rockaway #2, and 6) Rockaway #3. Levees that were formerly on Deepavaal Brook in the 1987 Phase I GDM were replaced in this phase of the design with a channel improvement plan. The Upper Pompton area, which previously included five levee systems, was also altered and now only contains a single levee/floodwall system at the Pequannock-Ramapo site. The removal of 4 levee/floodwall systems in the Upper Pompton area was due to channel redesign, improved hydraulic performance of the main tunnel inlet and further evaluations by other disciplines.

At the proposed Rockaway #3 levee site, an existing levee/floodwall system currently is in place. Known as the Lake Hiawatha Local Flood Protection Project, it consists of levees and floodwalls, pressure interceptors, storm drains, a ponding area and an 183 cfs pumping station. The Corps proposed plan at this site, the Rockaway #3 system, would raise the existing levees/floodwalls by approximately 4 to 5 feet and add approximately 1,750 feet to its overall length. Assessments made at the Rockaway #3 system were preliminary in nature, but study results can be used as a good indicator of the expected performance of existing facilities at this site. Further analysis would be necessary during the FDM to ensure existing project readiness for overall performance.

Minimum facilities that include gravity culverts, ponding areas and at some locations small pumping stations were evaluated as well as enhanced facilities in areas where residual flooding occurred. Small pump stations were included in some instances due to a combination of factors including the reducing of the overall project footprint requirements, New Jersey state ratio land rules, and the need to evacuate ponding areas in some reasonable time period less than 2 to 3 weeks. In certain cases optimized plans were evaluated for future improved conditions with the 42-foot main tunnel in place and larger interior flood loss reduction facilities. In this effort, all gravity outlets were assumed to include both flap and sluice gates.

Hypothetical floods for seven (7) frequency events (2-, 5-, 10-, 25-, 50-, 100- and 500- year) were developed for both exterior and interior conditions. Basin-wide UNET modeling produced hypothetical flood hydrographs for exterior conditions used in the analysis. Hypothetical events of the same frequency, for both interior and exterior, were assumed to occur simultaneously to analyze interior ponding elevations. For

sizing minimum facilities gravity outlets, a point rainfall 24-hour storm (applied to the interior area only) was used in combination with free outflow conditions (low exterior stages). For comparative plan conditions, a 96-hour storm duration was applied with the interior rainfall values adjusted to the total exterior drainage area. The drainage area adjustment was applied to the exterior area above the point where the minimum facility outlet would empty into the receiving stream. This concept was used since it was assumed that the same meteorological event was occurring throughout the entire Passaic River basin. This approach also determined the performance of the minimum facilities and selected plan design with exterior river conditions. Rainfall depths for the 1- through 96- hour durations for various frequencies are shown in Tables C-21 and C-22.

35.4.1.1. Ponding Areas. As part of the study, minimal grading and no excavation would be required in the proposed ponding areas. Ponding areas would be maintained in their current state, generally leaving the proposed areas in their natural habitat.

35.4.1.2. Pertinent Site Data. Many of the interior drainage areas were subdivided into smaller subareas by review of natural interior drainage boundaries. Those sites with just one interior area draining into one ponding site included the Pinch Brook, Passaic #10, Rockaway #2, and Rockaway #3 systems. Rockaway #1, Passaic #2A (on the Passaic River) and the Pequannock-Ramapo systems were further subdivided into numerous subareas. Table C-66 summarizes pertinent drainage area data for each site, the name of the receiving stream at the outlet, and the drainage area of the receiving stream.

35.4.1.3. Interior Runoff Parameters. In order to develop the runoff characteristics of the interior areas both the infiltration losses and the unit hydrographs were defined. For the Central Basin levee interior areas, the SCS curve number method was used for defining infiltration losses and both the Clark and SCS dimensionless method were used to define the unit hydrograph. The SCS dimensionless unit graph method was used unless several interior unitgraphs needed to be combined, then the Clark method was used due to the inability of the IFH program to directly input a combined unit hydrograph. Another parameter used in the runoff analysis was the percent imperviousness of the interior area. The percent imperviousness values of the interior area used in this analysis were taken from HEC-IFH, Interior Flood Hydrology Package-Users Manual, Table 4.3 on page 63, entitled "Values of SCS Curve Number for Urban and Suburban Areas". Curve numbers determined in this analysis have values that are higher than those shown in the basin wide UNET evaluations made for existing and unimproved conditions, see Table C-11. The higher values were considered appropriate because the smaller interior areas in this interior flooding

analysis were highly developed and a subset of the larger drainage areas identified in Table C-11. Table C-67 summarizes representative runoff parameters for each of the levee sites.

For the Pompton Basin interior areas, due to the very small size of the interior drainage area involved, the Clark unit hydrograph Tc and R values that were determined for various stream gages were not considered applicable. However, another approach was used in developing Tc and R using equations from EM 1110-2-1417 where: Tc = 8.29*(1+I)^-1.28 * (A/S)^.28; where I = impervious area in % and S = average watershed slope in feet per foot; R = C*(Tc) where C = 2.0 for suburban residential developed catchment. SCS curve numbers were developed to determine the amount of rainfall excess to be applied to the unit hydrograph. For subbasin 1090 (which includes all the subareas identified below), the basinwide value of 81 was used. This value represents the future year 2050 land use condition and an Antecedant Moisture Condition II, normal soil moisture. Table C-68 summarizes the runoff data for the Pequannock-Ramapo Levee.

35.4.1.5. Interior Flood Hydrographs. Interior flood hydrographs were developed from hypothetical storm rainfall and computed unit hydrographs. Base flow was added to interior storm runoff. An initial base flow of about 1 cfs per square mile was added at the beginning of storm runoff. When flows on the recession side of the inflow flood hydrograph fell to approximately 1 cfs, a groundwater discharge of 1 cfs was assumed to continue for a 24-hour period. This assumption was very important in light of the fact that pond outlets could be blocked by exterior river stages in the Central Basin for periods of up to 2 weeks. Under certain conditions, an outlet blocked by high river stages with continuous interior groundwater discharge after surface runoff ceases, could produce a secondary interior peak elevation higher than that produced by the inflow flood hydrograph.

35.4.1.6. Interior Seepage. Generally, no seepage through the levee, floodwall or roadway embankments were included in these evaluations. However, at the Passaic #10 site where the ring levee would be approximately 4,830 feet in length, seepage was evaluated and included in the analysis. Seepage was included there due to the availability of geotechnical information taken as part of the FDM for that project element. Seepage rates at the Passaic #10 levee increased the maximum ponded water levees by a few tenths of a foot. These pond level increases were not considered significant in the overall evaluation and seepage rates would be included during the FDM's when additional boring information at individual project element sites would be taken to further define the foundation conditions.

35.4.1.7. Interior Flood Peak Frequency. Table C-69 summarizes peak inflows for the 10 and 100- year floods produced by a 96-hour storm.

35.4.1.8. Minimum Facilities. Minimum facilities have been developed for each levee site (and each subarea). For levee sections of Rockaway #1 and Passaic #2A formed by I-280 and I-80, respectively, the existing outlets are the minimum facility. See Tables C-70 and C-71 which summarize the minimum facility outlet sizes for both the existing outlets along I-280 and I-80 and new outlets for added all levee segments. Within each subarea shown in the table, all ponding and drainage is assumed to be "hydraulically connected".

35.4.1.9. Interior Elevation Frequency Curves. Interior elevations were developed by routing hypothetical floods through interior ponding areas utilizing minimum facility outlets and coincident exterior frequency flood conditions. This was done for each of the seven (7) hypothetical storm events. Table C-72 summarizes peak ponding elevations at each levee site.

35.4.1.10. Analysis of Alternatives

The analysis of alternatives included ponding with minimum facility gravity outlets, ponding with enlarged gravity outlets, and ponding (including outlets) with various size pump stations. In the Central and Pompton Basin areas, pumps examined were generally small, less than 10 cfs, except for the analysis performed at Rockaway #2. At Rockaway #2, pump stations were analyzed up to 70 cfs in size.

Interior flood damage reduction facilities were enlarged at each ponding area site until the 100- year peak elevation was lowered to the start of damage elevation. This procedure was used since it was felt that if the 100- year elevation was below damaging level, then the ponding area could be optimized. Generally, this was true. If the minimum facility gravity outlets would lower the water levels to below the damaging level when exterior stages were considered, then no further analysis was necessary. This would be the selected facility.

Due to the long periods of time in which the exterior stages would be high, this analysis indicated that enlarging the gravity outlets had little impact in reducing interior flood levels when exterior flood stages were evaluated. This resulted in a review of various pumping station sizes to lower interior stage-frequency curves. Hence, pumping stations were reviewed as opposed to enlarging the gravity outlets.

After a series of rigorous examinations, multi-discipline review meetings were held which included cost, economic, real estate, layout, and environmental issues. Along with the prolonged elevated exterior stages, final facilities were selected at each ponding area site within each levee/floodwall system. The selected facilities have been tabulated and are presented in Table C-73.

Routings were performed at each of the ponding areas to develop the final interior stage-frequency curves. These routings were performed with the 96- hour storm duration applied and the interior rainfall adjusted to the exterior drainage area size. Final maximum water levels for the various return period events are provided in Table C-74.

35.4.2. Hurricane Levees

The interior drainage analysis for the Hurricane Levee system included three sites within Newark, Kearny and Harrison, New Jersey. The Lister Ave/NJ Turnpike/Doremus Avenue, Kearny Point, and South First Street levee systems were part of the recommended plan of improvement in the Passaic River Basin Phase 1 GDM, December 1987. Each site was comprised of from 1 to 4 individual drainage areas, with some drainage areas further divided into sub areas. Figures C-169a thru C-169d contain layouts of the three sites, and the associated drainage areas and subareas.

35.4.2.1. Analysis Conditions

Exterior conditions at the six sites are controlled by tidal stages. A coincidence analysis, performed for the Storm Surge Analysis for the Passaic River Basin indicated a low degree of coincidence between peak tidal stages and interior rainfall events. The study recommended using a normal tide plus 1.5 - 2.0 ft surge coincident with rainfall/runoff events in the Passaic River Basin. To be conservative, this interior drainage analysis superimposed tidal marigrams of historical storms, whose peak elevations (tide + surge) most closely matched the hypothetical stage frequency curve, over a constant elevation of +3.5. The marigram was then shifted so that the peak tidal stage was coincident with peak interior runoff. Tidal marigrams for the 2, 5, 10, 25, 50, 100 and 500 year events, representing both hurricanes and north easters, were provided by PRD at 2 locations, Kearny Point and East Newark. Figure C-170 represents the exterior stages for the range of events at Kearny Point.

Hypothetical precipitation was developed for the basin-wide UNET modeling study at the centriod of the basin (see Table C-21). This centroid developed rainfall was then used and adjusted for areal reduction when computing the basin-wide exterior stage hydrographs. However, for the Hurricane levee interior flooding analysis new rainfalls were developed specifically for Subbasin 1620. This precipitation applies to all the smaller subareas included in the interior area analysis. Precipitation depths are shown in Table C-75 for durations of 1 through 96 hours and frequencies of 2, 5, 10, 25, 50, 100 and 500-years. Hourly distributions of the precipitation were generated by the HEC-IFH computer program for further analysis. HEC-IFH reduced the one-hour precipitation into the needed 5-minute tabulation interval values during execution of the program.

35.4.2.2. Interior Runoff Parameters

The unit hydrographs used for the subareas in the interior flooding analysis are consistent with those of the Passaic River Basin study. Clark Tc and R parameters were optimized for a range of stream gages throughout the Passaic Basin. Plots were made of Tc vs drainage area and R vs drainage area These curves provide unitgraphs for ungaged subareas anywhere in the basin. For the interior analysis these curves were extrapolated to cover the range of drainage areas that make up the interior areas. Table C-76 shows the Clark unitgraph parameters for each interior subarea.

Precipitation losses in the interior area rainfall/runoff analysis use SCS curve numbers. The curve number used with each subarea was developed from a HYDPAR grid cell data bank prepared for the entire Passaic River Basin. Data was examined for each grid cell in Passaic subbasin 1620 that fell within the limits of the interior areas. For each grid a soil group and a land use for the year 2050 was determined. Information was available from prior work of the Passaic study to select a curve number for each cell based on the land use and the soil group. Weighted curve numbers were calculated for each interior subarea based on the number of cells within a subarea having the same curve number. The weighted curve number represents AMC condition II, or normal conditions. Using a SCS conversion table, all curve numbers were changed to represent AMC I, or dry basin conditions. Dry basin conditions are more compatible with assumptions made for the analysis of the rest of the Passaic basin. The initial abstraction used with the SCS curve number is defined as: IA = 0.2S, where S = (1000 -10CN)/CN. Table C-76 shows the adopted curve numbers for each subarea.

Seepage rates were developed for a typical levee section along the Passaic River. A seepage rate of .0000275 cfs per ft of head per linear ft of levee was applied to all subareas. In general, these rates produced negligible seepage rates/volumes throughout the study.

Interior pond inflow hydrographs were computed by the HECIFH program. HECIFH uses precipitation data, basin data (drainage area, SCS curve number), unit hydrograph data (Clark Tc & R), to compute interior runoff hydrographs for each sub area. Upper areas were routed into lower areas utilizing the time lag method. The lag time was computed based upon length, slope and channel characteristics. Overflows from adjacent drainage areas were imported as necessary to develop the total pond inflow hydrograph.

Interior Flood Peak Frequency Discharge The following Table C-77 summarizes peak inflows for the 2-, 5-, 10-, 25-, 50-, 100- and 500- year floods produced by a 96 hour storm.

35.4.2.3. Minimum Facility Analysis

An analysis of the Minimum Facilities, as defined in EM 1110-2-1413, was conducted to determine a starting point from which additional interior improvements will be evaluated. In general the minimum facility consists of natural ponding and gravity outlets consistent with the existing storm sewer system.

Interior ponds used in the minimum facility analysis consists of the natural storage available in existing ditches and low lying areas. The areas protected by the hurricane levees are generally flat, with minimum elevations, with the exceptions of drainage ditches, around +4 to +6 NGVD. Since the topography of the area is so flat, there are many structures within the lower 2-3 ft of the ponding areas.

In general, minimum facility gravity outlets consisted of extending the existing storm sewers through the levees/walls. In areas that are drained by established drainage ditches, 48" to 72" outlets were provided, depending upon the size of the ditches. In order to provide drainage for the overland runoff that would be blocked by the line of protection, 24" outlets were placed at a minimum spacing of 400 ft. In those areas where no utility information was provided, the minimum facility was assumed to be 1 36" primary outlet and 24" outlets spaced 400 ft apart. New minimum facility outlets were placed at an invert elevation of 0.0. All outlets were assumed to be "hydraulically connected", either by pipe or by ditch, so that all outlets would function together. Table C-78 contains data of the minimum facility outlets for all ponding areas.

35.4.2.4. Overflow Diversions. As previously stated, the topography of the drainage areas generally flat. As a result, there is typically a low elevation along the drainage divide between to adjacent areas where ponded runoff overflows from one area to another. Table C-79 contains the overflow elevations for the various drainage areas. The minimum facility analysis for these was a multi-step process. The minimum facility analysis was performed without overflow to determine which areas overflowed into adjacent areas. Diversion functions, Q vs EL, were developed for the upper areas and the upper areas reanalyzed. The overflow discharge was than imported into the lower area as auxiliary inflow, and the lower area was reanalyzed. If the peak stage of the lower area was higher than the diversion elevation, the two areas were combined and analyzed as one area for the higher frequencies. The final stage frequency curve for those areas would be a combination the two stage frequency curves (individual and combined areas).

Interior elevation frequency curves were developed by routing hypothetical interior frequency floods through the

interior ponding areas utilizing minimum facility outlets and the coincident exterior tidal conditions. Table C-80 summarizes peak ponding elevations for the 2-, 5-, 10-, 25-, 50-, 100- and 500-year events for the minimum facility analysis.

35.4.2.5. Analysis of Alternatives

Initially, the analysis of alternatives was to include both ponding and pumping as recommended in the Phase I GDM. However, ponding alternatives proved infeasible for several reasons. The drainage areas are essentially flat, highly developed industrial areas, with no natural storage for ponding. Existing minimum elevations in most areas are 4-6 ft above MSL. Excavated ponds were not possible because, in order to provide sufficient storage capacity, the invert of the pond would be within the tidal zone. As a result, the improvement alternatives analyzed in this analysis consisted primarily of pumping plans.

Pump Station Design. Each pump station consists of 3 pumps, each with a capacity of 1/3 of the capacity of the pump station. The pump on and pump off elevations for each pump are staggered by 0.5 ft. Pump #1 turns on at elevation 4.5 and off at elevation 3.0. Pump #2 turns on at elevation 5.0 and off at elevation 3.5. Pump #3 turns on at elevation 5.5 and off at elevation 4.0. A 0.25 acre pump sump, with an invert elevation of 0.5, was included in all pump stations to provide sufficient capacity for proper pump operation.

Initially, pumps were sized to provide a 100 year level of protection for the interior area. That is, the maximum interior pond stage for the 100 year event would be at or below the Zero damage level. Once that pump size was determined, several additional pump sizes were analyzed, usually one larger than the 100 year pump and 2 or 3 smaller. This provided a family of curves from which the optimum pump size could be determined.

For those areas in which overflow still occurred, alternative plans were paired to match a level of protection. For example, the pump in the upper area was sized for a 100 year level of protection. The overflow hydrographs were computed for the range of frequencies. The overflow hydrographs were then imported to the lower area, and the pump sized to provide an equal (100 year) level of protection. The process was then repeated other levels of protection to develop a range of alternatives.

As a result of the optimization process, additional pumping facilities were determined to be justified at six of the sixteen drainage areas. Table C-81 contains the optimum interior drainage facilities and their associated level of protection. Table C-82 summarizes peak ponding elevations for the 2-, 5-, 10-, 25-, 50-, 100- and 500- year events for the optimized plan.

35.5. Coincident Rainfall/Runoff for Wetlands

A coincident rainfall/runoff analysis was performed to determine from an historical perspective, the likelihood of localized rainfall occurring with high river stages at both the Passaic #2A and #10 levee/floodwall systems. The analysis was also performed to determine if the existing wetlands in the proposed ponding areas behind these systems could be maintained. These locations were selected due the advance engineering being performed at the Passaic #10 levee and the large ponding areas included as part of the Passaic #2A site.

The start of damage was determined to be critical at each of the project locations and was determined by economic evaluations for each of the subareas. Values at each of the subareas were as follows:

Ponding Area Site	Start of Damage Elevation (NGVD)
Passaic # 2A	
subarea ABC	168.5
subarea E-Fnew	168.5
subarea FF-K	168.0
subarea XY	169.5
subarea Z	168.0
subarea ZZ	168.5
Passaic #10	170.5

The analysis performed was predominantly an assembly and review of historical data as well as the development of in-house software programs to process the data. To initiate the investigation, historical streamflow data was collected for long term USGS records on the Passaic River at Chatham and Little Falls, N.J. Daily streamflow records at the Chatham gage were collected from 01 October 1937 through 30 September 1993 and at the Little Falls gage from 16 August 1897 to 29 July 1993.

Daily rainfall records in the vicinity of the proposed levee sites were also collected. Rainfall data was collected at three nearby rainfall gages. These gages were in Essex Fells, NJ, Springfield, NJ and Little Falls, NJ. Daily rainfall records were collected from 01 January 1948 through 29 July 1993 (limit of data on CD-ROM at that time), therefore the concurrent period-of-record between daily streamflow and rainfall was 01 January 1948 through 29 July 1993 or 16,647 days (approximately 45.5 years).

Streamflows were not readily available at either of the project sites and were generated by using the streamflow data at the gaged sites and rating curves as developed from the UNET

output. For the Passaic #10 levee, the gaged data at Chatham was used while at the Passaic #2A site the Little Falls streamflow gage was the basis for computations.

At the Passaic #10 site, the drainage area increases from 100 square miles at the Chatham gage to 128.51 square miles at the project site. However, the UNET model output indicated that although the drainage area increased in size, the peak flows were almost the same. This phenomena can be explained by the fact that the stream banks are low, the valley widens, and the overbank storage compensates for the larger drainage area. For these reasons the peak flows recorded at the Chatham gage were translated and used at the project site.

Converting to stage at the Passaic #10 site was performed by the use of a rating curve developed from the UNET output. Once the daily flow and rating curve were known, an in-house program converted daily discharges to daily river stage. Daily discharges were considered appropriate at this site due to the flat and broad peaks that pass at the Chatham gage during the passage of flood events. Results of the conversion produced a continuous river stage hydrograph at the project site. All data was stored in a sequential manner and river elevations ranged from the stream invert of 161.0 NGVD to 172.0 NGVD.

At the Passaic #2A site a similar procedure was used, except the Little Falls streamflow records were used. Due to the length of the Passaic #2A project area, three specific locations were studied. River elevations at the 3 locations ranged from a low of the Beatties Dam crest elevation of 158.2 to a maximum of NGVD. It should be noted, that Beatties Dam maintains an elevated water level under today's condition in the Great Piece Meadows due to backwater effects.

Data bases were then assembled at each project site that consisted of the daily flow, daily river elevation and concurrent rainfall at each of the recording sites. Multiple rainfall gages were used to ensure that at least one of the gages was operational in the concurrent record. The data was checked and verified. Following the verification, an additional in-house software program was used to extract information from the data base on how river elevation relates to rainfall.

During the coincident streamflow/rainfall analysis 4 conditions needed to be determined at the proposed project sites:

a. Low river stage, low rainfall.
b. Low river stage, high rainfall.
c. High river stage, low rainfall.
d. High river stage, high rainfall.

To differentiate between high and low river stage elevations, a threshold value was needed to be determined at each site. For the purposes of this study, elevation 165.0 was

selected at the Passaic #10 project location. This elevation corresponds to both the top of bank of the Passaic River and also to the exterior inverts for the proposed setback levee culverts. Proposed culverts are either 24" or 36" diameter conduits. At the Passaic #2A site, the Great Piece Meadows valley floor was predominantly above Elevation 164.0 NGVD and this value was used as the external threshold constraint at this location. High daily rainfall was deemed any daily rainfall total above 1" or more in a 24 hour daily period. This value was selected since it approximated an average monthly event at Passaic #10, occurring 688 occurrences over 45.5 yrs, and 795 occurrences at Passaic #2A.

Following the establishment of high/low river stages and high/low rainfall, the software program determined the coincident nature of stream stages and local rainfall. Tables C-83 thru C-86 show the results of queering of the data bases. In general, tables were developed by determining the number of days that any rainfall data base gages had 0", 0"-1", 1"-2", 2"-3", 3"-4", 4"-5" and 5"+ of rainfall for a given river stage.

35.5.1. Passaic #10

Of the 16,647 days in the period of record 11,484 days had no recorded rainfall. This represents 69% of the period-of-record. River stages would be above the critical value of 165.0 NGVD for only 1,252 days. This implies that river elevations are less than critical 92.4% of the time. Therefore, the outlets at the proposed levee would be partially to completely blocked from out of bank Passaic River stages approximately 7.6% of the time. Of the 1,252 days of high exterior stages, 758 days had no rainfall and 125 days had rainfalls in excess of 1". Therefore high river and high local rainfall occurred 0.8% of the time. The 1-year and 2- year existing stage frequencies at this site are 169.0 and 170.2 NGVD respectively. The following table summarizes the % of time the various conditions occurred:

Passaic #10 - Coincident Condition	**# of days occurred**	**% of time**
Low river Elev <165, low rainfall <1"	14832	89.1
Low river Elev <165, high rainfall >1"	563	3.3
High river Elev >165, low rainfall <1"	1127	6.8
High river Elev >165, high rainfall >1"	125	0.8
	16647	100.0

Having established how the historical record between river stage and rainfall would match-up over the 45+ year period-of-record, the next step was to examine if the existing wetlands in the proposed ponding areas could be maintained. It should be noted that the existing stage-frequencies at the project site for the 1-year and 2- year are 169.0 and 170.2 NGVD respectively.

Under today's condition with no levee/floodwall in place, the wetlands are maintained by a combination of high exterior river stages and local rainfall/runoff that discharge into these areas. High river stages occurred 1,252 days and would have flooded these lands (depending on the elevation reached during the river flood). As can be determined from Table C-83, local rainfall occurred on 5,163 days with the vast majority, 4,669 of these days 4,669 occurring with low river stages. During low river stages, the 4,669 local rainfall events would have drained through the existing wetlands as overland sheet flow and channel directly into the Passaic River. Local rainfall with high river flow occurred on 494 of the 1,252 days and would have fallen directly on the river flooded wetlands.

With the proposed Passaic #10 project in place, the high river stages would be removed and not permitted to flood the interior side of the proposed line-of-protection. See Figure C-171. However, even though 1,252 days of high exterior stages are removed, coincident rainfall/runoff occurred on 494 of those days and interior ponding would have resulted. The reduction of flooding to the interior wetlands would be complete for the 758 days when no local coincident rainfall occurred. Assuming the desire to maintain the interior wetland elevation at 169.0 NGVD, a runoff volume of approximately 8.0 acre-feet would be needed. This would require a total local coincident runoff of approximately 1.3 inches.

With the levee blocking overland flow to the Passaic River, all 5,163 days of local rainfall events would now have to pass through the proposed culverts and pumping station. These local rainfall events would be used to compensate for the loss of 758 high river days by operating the project to maintain the interior wetland. From the data available 688 days of rainfall would be available where rainfall totals exceeded 1". High local rainfall would almost completely compensate for the removal of the 758 days of the high river stages. In addition to the high local rainfall events, many of the smaller rainfall events (<1") from the 4,475 days of rain would accumulate in 2 or 3 day totals well above the 1.3 inches of rainfall needed. With operation of the gates (flap and sluice) and pump station, the interior levels could be regulated to achieve the desired results. Therefore, based on the above, the project when properly operated would be able to maintain the existing hydrology and hence the existing wetlands behind the proposed line-of-protection.

35.5.2. Passaic #2A

The construction features of the Passaic #2A levee site would be located upstream along the Passaic River above Horseneck Road. However, by incorporating the I-80 embankment into the line-of-protection, the effects of the system would extend downstream from the confluence of the Rockaway River with the Passaic River to the confluence of the Passaic and Pompton Rivers (at Two Bridges). This would be a river distance of almost

66,600 feet (Passaic River Stations 2403+80 to 1738+60). Due to the long distance and the fact that the Passaic #2A system has 6 ponding areas, 3 river sites were studied for wetland analysis. Analysis was performed on the Passaic River at Stations 1940+00, 2181+00 and 2250+00. It should be noted that since the vast majority of wetlands in the proposed plan of improvement are behind the existing I-80 embankment which already blocks overland flow from the Passaic River, impacts in this area should be minimal. The following paragraphs discuss the analysis performed to date.

Within the concurrent period of record of 16,647 days, rainfall was recorded on 6,127 days at this site. This result is slightly different from the Passaic #10 analysis due to the inclusion of the third rainfall gage at Little Falls, NJ which is very close to the Passaic #2A project site. Therefore, during 63.2% of the period-of-record or 10,520 days no rainfall was recorded.

After review of the Great Piece Meadows topography, it was concluded that for this purposes for the effort water elevations in the Meadows between 164.0 to 165.0 NGVD would be critical. Therefore the lower value of 164.0 was selected from a conservative standpoint. Depending on location, river stages would be above the critical value of 164.0 NGVD for only 461, 792 and 914 days. See Tables C-84 thru C-86. Therefore, river elevations are less than critical 97.2%, 95.2% and 94.5% of the time respectively. Therefore, the outlet conveyance channels would be partially to completely blocked from Passaic River stages approximately 2.8%, 4.8% and 5.5% of the time.

At Station 1940+00 (outlet to Ponding area "FF-K"), 461 days of high exterior stages occurred with 261 days having no rainfall and 38 days had rainfalls in excess of 1". Therefore high river and high local rainfall occurred 0.2% of the time. The following table summarizes the % of time the various conditions occurred:

Passaic #2A at Station 1940+00	# of days occurred	% of time
Low river Elev <164, low rainfall <1"	15429	92.7
Low river Elev <164, high rainfall >1"	757	4.5
High river Elev >164, low rainfall <1"	423	2.6
High river Elev >164, high rainfall >1"	38	0.2
	16647	100.0

At Station 2181+00 (outlet to Ponding area "E-Fnew"), 792 days of high exterior stages occurred with 458 days having no rainfall and 66 days had rainfalls in excess of 1". Therefore high river and high local rainfall occurred 0.4% of the time. The following table summarizes the % of time the various conditions occurred:

Passaic #2A at Station 2181+00	# of days occurred	% of time
Low river Elev <164, low rainfall <1"	15126	90.8
Low river Elev <164, high rainfall >1"	729	4.4
High river Elev >164, low rainfall <1"	726	4.4
High river Elev >164, high rainfall >1"	66	0.4
	16647	100.0

At Station 2250+00 (outlet to Ponding area "ABC"), 914 days of high exterior stages occurred with 529 days having no rainfall and 73 days had rainfalls in excess of 1". Therefore high river and high local rainfall occurred 0.4% of the time. The following table summarizes the % of time the various conditions occurred:

Passaic #2A at Station 2250+00	# of days occurred	% of time
Low river Elev <164, low rainfall <1"	15011	90.2
Low river Elev <164, high rainfall >1"	722	4.3
High river Elev >164, low rainfall <1"	841	5.1
High river Elev >164, high rainfall >1"	73	0.4
	16647	100.0

It should be noted that the existing stage-frequencies within Great Piece Meadows along the Passaic #2A and I-80 embankment vary for the 1- year from 165.0 to 167.5 and the 2- year from 166.0 and 168.2 NGVD as one proceeds upstream respectively.

With no levee/floodwall in place, the wetlands are maintained by a combination of high exterior river stages and local rainfall/runoff that discharge into these areas. At the most upstream site, high river stages occurred for 914 days and would have flooded these lands (depending on the elevation reached during the river flood). As can be determined from the above noted tables, local rainfall occurred on 6,127 days with the vast majority of these days (5,742) occurring with low river stages. During low river stages, the 5,742 local rainfall events would have passed through the existing wetlands as sheet flow and drain directly into the Passaic River. Local rainfall with high river flow occurred on 385 days and would have fallen directly on the river flooded wetlands.

With the proposed project in place, the high river stages would be removed from the interior side of the proposed line-of-protection. In fact, even though 914 days of high exterior stages are removed, coincident rainfall/runoff occurred on 385 of those days and interior ponding would have resulted. The reduction of flooding to the interior wetlands would be complete for the 529 days when no local coincident rainfall occurred. Assuming the desire to maintain the interior wetland elevation at 166.0 NGVD, the same elevation as the top of the proposed Great

Piece Meadows weir, flap and sluice gates could be opened to allow the external weir stages to back water into these interior areas during operation of the weir.

In addition, with the existing I-80 embankment and proposed levee/floodwalls blocking overland flow to the Passaic River, all 6,127 days of local rainfall events would now have to pass through the proposed culverts and pumping station. These local rainfall events would be used to compensate for the loss of 529 high river days by operating the interior project features to maintain the interior wetland as well as the operating the Great Piece Meadows weir from the exterior side. With operation of the gates (flap and sluice), pump station and exterior weir the interior levels could be regulated to achieve the desired results. Therefore, based on the above the project would be able to maintain the existing hydrology and hence the existing wetlands within the protected area.

36. Tunnel Design and Analysis

36.1. Tunnel Sizing Analysis at Pompton Inlet.

36.1.1. General. As part of the GDM a risk and uncertainty analysis was performed at the main tunnel inlet (Pompton Inlet). This study was undertaken in an attempt to design and document the most cost effective diversion tunnel that would handle the 1% chance flood. The Pompton Inlet site was selected for this analysis since this location is absolutely critical to the establishment of both design flow and diameter as the overall cost of the tunnel is significantly influenced by the inner diameter. Floodwaters diverted to the tunnel at this location would travel 20.4 miles to Newark Bay. This analysis is especially fortunate to have a long term USGS gage (Pompton River at Pompton Plains Gage # 1388500) located at the proposed main inlet site. With a systematic period of record dating back to 1940 (54 years of record) and an historical record extension to October 1903, ample data is available to adequately determine the design flow with a high degree of confidence. The analysis and procedures developed in the following paragraphs were performed in accordance with EC 1105-2-205, Risk Based Analysis for Evaluation of Hydrology/Hydraulic and Economics in Flood Damage Reduction Studies, 25 February 1994.

Due to the long lead time for design of the tunnel inlets, tunnel diameters, and other essential surface features, these studies were performed in the March/April 1994 time frame. At that time, the Pompton Plains gage had 52 years of systematic record and included data thru Water Year 1992. Since that time, additional statistics have been developed using the log-Pearson analysis that demonstrate little has changed to the peak flow frequency curve by adding in Water Year 1993 and 1994.

36.1.2. Peak Flow Frequency Curve Statistics. This portion of the study effort involved the determination of the peak flow frequency curve statistics. These statistics were developed in accordance with Bullentin #17B for Determining Flood Flow Frequency , United States Water Resources Council, as revised in September 1981 and EC 1110-2-249, Engineering and Design - Hydrologic Frequency Analysis, dated 5 January 1985. The Corps computer program HECWRC was used to calculate both the expected probability and computed curve data. The existing condition computations would be used to help determine the computed curve for future conditions (Yr 2050) with the proposed tunnel plan in place.

In standard Corps practice, peak flow frequency curves are developed and used that include the expected probability adjustment. However, for this analysis computations will be aimed at developing the future condition with project computed curve.

Table C-87 provides the basic data that was used to transfer from the computed curve of 1992 to the future conditions with project curve.

For the curve developed in Column B, the HECWRC program developed the following station statistics (52 years of record, expected probability and an historical adjustment of 89 years)

X = 3.8671 ; SD = 0.2397 ; g = 0.1000 (adopted).

Having established both existing 1992 conditions and future with project (Yr 2050), based on the expected probability, adjustments to the UNET peak flows were required. Therefore, peak flows were adjusted in Columns E and F based on the previously defined ratios.

As can be seen, two (2) adjustments were made to develop the future conditions computed curve. The first adjustment attempted to bring the UNET data more in line with the expected probability adjustment, while the second adjustment converted from the expected to computed curve. Peak flows at various frequencies (Column F above) were then simulated as closely as possible by inputing statistics into the @RISK program. After a trail and error solution, the statistics for the future w/project computed curve were as follows (See also Table C-88):

X = 3.8980 ; SD = 0.2378 ; g = 0.1000

The statistical curve developed is slightly higher at the rarer floods but is within 2% across all frequencies and considered well within reason. Therefore, the computed statistics were used in the tunnel sizing and reliability runs with LOTUS and the @RISK add-on module.

36.1.3. Development of Tunnel Diameter versus Flow Curves. Based on guidance received at technical design and review meetings, the District developed elevation-discharge curves for both a high and low tunnel roughness frictional value. These curves would establish the range of uncertainty associated with the hydraulic determination of the tunnel diameter for a given discharge.

As recommended by HQUSACE staff, the smooth pipe curve of the Moody Diagram was used to determine the lower limit of the friction factor for the tunnel. The approach resulted in a roughness height "k" = 0.0002 feet or a Mannings' "n" = 0.010.

The upper limit was obtained from Figure 14, page 28 of U.S. Department of the Interior, Bureau of Reclamation, Engineering Monograph No. 7, entitled "Friction Factors for Large Conduits Flowing Full". Assuming a highly jointed "pre-cast" tunnel or an usually rough and badly eroded "cast-in-place" tunnel, the selection resulted in a roughness height "k"=0.006 or a Mannings' "n"=0.016.

A Lotus spreadsheet was then developed that would calculate the maximum flow for a given tunnel diameter and K value, without exceeding the design head of 174.0 at the main tunnel inlet. The downstream tailwater was established as a constant 6.2 NGVD for future conditions sea level rise and storm surge in Newark Bay.

The resulting analysis revealed that for the 100 year flow of about 29,150 cfs, the size of a tunnel which would convey that flow without causing damages, could vary from 37 to 42.8 feet in diameter. The resulting range of 5.8 feet was assumed to be four times the standard deviation as noted by the technical staff of the Hydrologic Engineering Center. Therefore a standard deviation of 1.45 was used to establish a normal distribution of tunnel diameters around a curve which was set halfway between the upper and lower frictional curves. Figure C-172 displays this data.

During operation of the tunnel the peak flows approaching the Pompton Inlet will be separated. While operating, the tunnel will be designed to permit a continuous bypass flow of 4000 cfs downstream. Therefore, 4000 cfs was added to the tunnel discharge curves developed on Figure C-172. The total flow was entered into the HEC template for the combined system using the data in the first and last columns of Table C-89.

36.1.4. Tunnel Sizing Analysis. Inputing the future conditions with project computed peak flow frequency curve statistics and tunnel diameter versus flow (for the best fit curve) data into the @RISK program was accomplished with the use of HEC's template Lotus spreadsheet. For project sizing, each tunnel diameter was run with the standard deviation (SD)=1.45

feet for possible variation in diameter. Results of the project sizing analysis were tabulated at multiples of a thousand for each diameter and are summarized in Table C-90.

As can be seen in Table C-90, the true probability has minimal variation as the number of iterations increase from 5000 to 9000. Therefore, 9000 iterations was considered sufficiently large enough to obtain the true probability.

36.1.5. Results. From the above analysis, a 42 ft. diameter tunnel would be the most cost effective tunnel size that could effectively handle the 1% chance flood.

36.2. Inlets

36.2.1. Alternatives

The tunnel profile geometry (Figure C-91a) presents a variety of possible flow conditions and plays an important role in the configuration of the proposed inlet. The most notable feature of the tunnel profile was that the downstream portion of the tunnel would be completely below sea level. Due to the configuration the lower portion of the tunnel would either never empty or will fill with water long before the rest of the tunnel. Once the downstream portion of the tunnel was filled, air in the upper tunnel would collect in the crown of the tunnel. Air trapped in the tunnel crown would have to be vented through a separate system. There are two long sections of tunnel with almost no slope that may make collection and venting of air difficult, costly, and unpredictable. Therefore, consideration was given to designing and developing inlets that would reduce the amount of air entrained as flow entered the tunnel.

The first type of inlet investigated was a morning glory design as was proposed in the Phase I GDM. A morning glory inlet performs best when there is open channel flow in the conduit or when the capacity of the conduit submerges most of the inlet throat. While the morning glory inlet is considered simple and easy to construct, it does entrain large amounts of air. Since neither of the optimum morning glory conditions would be expected to occur often, other types of inlets were investigated. The design was however, further developed and compared to several other types.

The next inlet type investigated was a vortex inlet. This inlet has been used in Milwaukee, as part of their Sewage System. The University of Iowa's, Institute of Hydraulic Research conducted a series of physical model studies, in 1983, for the Milwaukee Metropolitan Sewerage District Inline Storage System. Using guidance from the model analysis, a preliminary sizing of the Pompton River inlet was developed. The approach channel for the inlet would have required a 25 foot wide by 60 to 90 foot deep rectangular section. A section that deep was considered

excessive, especially since the total drop of the inlet would be about 160 feet. Therefore, this type of inlet was eliminated from further consideration.

A sloping inlet design was investigated next. The University of Minnesota's, St. Anthony Falls Hydraulic Laboratory conducted a series of physical model tests of several different types of drop inlets for the Metropolitan Sanitary District of Greater Chicago in 1975. Several design features for sloping inlets were documented in the model study report. Some of the documented features were used to design a sloping inlet for the Pompton River. The sloping inlet design was later compared to several other types of inlet designs.

The Chicago Metropolitan Sanitary District ultimately selected a custom design for many of the drop inlets needed in their Tunnel and Reservoir Project (TARP). The custom design is often called the Chicago TARP Boot type inlet. Several features of the boot type of inlet were evaluated for use in the design of the Passaic inlets. Some features or concepts that were evaluated were splitter walls, a de-aeration chamber and a sloping ceiling.

A wet intake tower design was also developed. This type of design is often found at reservoir projects that have open channel flow downstream of the gate system. The tower design has some very simple geometry but it involved some very large underground gates. This design was also compared to several other types.

Ultimately, three preliminary inlet concept designs were presented at an interdisciplinary meeting. The sloping inlet, the morning glory and the wet tower were evaluated and compared. The basis of comparison was constructiblity, hydraulic performance, total cost, environmental impacts and operation and maintenance. Several other features of the inlet design were also evaluated. The general conclusions were that the sloping inlet had the best hydraulic performance and characteristics, the morning glory would be the easiest to construct and the wet tower was the least desirable of the three. In addition, the evaluating review team concluded that constructing the inlet away from the river with an access basin in the overbank was preferred to constructing an inlet in the river.

Using the information gathered at the meeting, the sloping inlet was redesigned to simplify construction. The semi-cone inlet was then developed and sited in an off channel access basin immediately adjacent to the river.

36.2.2. Design and Evaluation

Once a basic inlet shape was selected, the design and performance of the inlets were established. The inlets were designed for a 100- year event. The maximum flow that would

bypass an inlet during a 100- year event without causing damages was established. Rating curves at each inlet were used to determine the approximate elevation that corresponded to the maximum "safe" bypass. The maximum bypass flow was subtracted from the total flow at each inlet. The remaining flow and the maximum river elevation became the basic exterior target design conditions for the Pompton Inlet structure.

At the Spur Inlet, several factors led to the modification of the design flow. The spur tunnel would be constructed and operational before the Pompton Inlet was completed. Flow at the Spur Inlet would normally be partially regulated by the Pompton Inlet. Since the Pompton Inlet may not yet be completed or operational, then flow at the spur may be higher than anticipated. In addition, all flow estimates have been based on uniform rainfall throughout the entire Passaic River basin. If a storm event happens to center itself below the Pompton Inlet, flow at the Spur tunnel could be more intense than estimated. Therefore, a 30% increase in the flow was used to account for these uncertainties. Table C-91 illustrates the flows and elevation mentioned above.

Several other design considerations are described below. The approach velocity at the gates was kept below 3.0 feet per second and the gate entrance sills were placed at least 4.0 feet above the channel invert to limit the size and amount of sediment drawn into the access basin. The crest of the inner inlet weirs were placed at least 7.0 feet above the access basin floor to minimize the inflow of sediment into the tunnel and maximize safety of maintenance personnel and others when the basin would be dry. The slope of the drop shaft was set at 60°, which is the maximum slope recommended in the St. Anthony Falls, Chicago TARP model study report.

Key design procedures are described below. The crest of the weir at the top of the drop into the tunnel was assumed to be a hydraulic control and critical depth was computed at that point. A sub-critical flowline was computed through the access basin and into the river to ensure that the maximum water surface elevation in the river would not be exceeded if the inlet crest was not submerged and all the gates were fully open. In addition a super-critical flowline was computed down the inlet shaft and into the de-aeration chamber. The sequent (or conjugate) depths were computed for the super-critical elevations in the de-aeration chamber and the location of the hydraulic jump was determined. The computed jump and the model study design guidance, mentioned above, was used to design the de-aeration chamber. The vent shaft in the de-aeration chamber was sized and placed based on St. Anthony Falls, Chicago TARP system model study recommendations. A physical model of the inlets will be built for the FDM (next level of study). It is possible that the physical model study will show that the vent shaft in the chamber can be eliminated from the design.

Once the inlets were designed an operating procedure was developed to control the opening and closing of the gates and the water levels in the river at several locations. The operation of the gates is described in detail in Section 28. of this appendix. The LOTUS spread sheets developed to determine the pressure flow grade lines in the tunnel were only used to model peak steady flow conditions. Rather than attempting to link the LOTUS computations to the UNET model, the inlets and the remainder of the tunnel system were incorporated directly into the improved conditions UNET model. The UNET model was then used to refine the gate operation, bypass flows and maximum water surface elevations. A summary of the final 100- year conditions is shown in Table C-92. Note that since UNET is an unsteady flow model, the peak flows show in the table do not necessarily occur simultaneously and therefore are not additive.

36.3. Outlet

36.3.1. Design Considerations

The design of the outlet involved three major goals. The first goal was to minimize the affects on ship traffic in the nearby navigation channels. The second was to minimize the cost associated with the outlet. The third was to maintain the capacity of the tunnel system. Some of the design procedures used to meet each goal are described below. The final outlet layout and design is shown on Figures C-90h, C-94a and C-94b.

36.3.2. Navigation

To establish the conditions which might impact navigation, organizations and agencies were contacted. Several tug boat operators were questioned to establish the type, the maneuverability and the number of vessels using the channels. Several port operators were interviewed for usage and traffic information. The Coast Guard and the NY/NJ Port Authority were consulted for safety and other related information. Finally, ship simulation and ship response personnel at WES were asked to estimate the maximum cross current that would allow safe passage of the ships in the navigation channel.

In summary, the traffic in the Hackensack Navigation Channel is very light. Almost half the traffic is coal and lignite, the remainder is mainly petroleum products with some sand and gravel and a small amount of scrap metal. The vessels range in size from 300 to 600 feet in length and operate with one or two tugs. Vessels of those sizes and types should safely navigate the outlet area with cross currents of less than two feet per second. However, a ship simulation study will be conducted during the FDM to insure that conditions in the channel remain safe.

To ensure that cross current velocities in the navigation channel would be less than 2 feet per second, the flow from the outlet had to be distributed throughout an area of 15,500 square

feet. A section 30 feet deep by 520 feet wide, would provide the area necessary. To obtain a distribution of flow throughout the full 30 foot depth the tunnel outlet would have discharge across a depth of at least 20 feet. To distribute the flow across a 520 foot width it would appear that the discharge from the outlet would have to be dispersed in some manner or form. It was decided that the outlet gate sills should be between 20 to 30 feet below the water surface, and that the discharge should be dispersed in a multidirectional fashion.

36.3.3. Cost Considerations

Minimizing the cost of the outlet structure involved striking a balance between the excavation of additional tunnel length and the excavation of river and bay sediment. Generally, if the tunnel was placed on or near shore the total length of tunnel would be reduced but a long "outlet channel" would have to be excavated through the tidal flats. If the outlet were placed offshore, near a navigation channel, the length of tunnel would increase but the amount of excavation through potentially contaminated river or bay sediment would be reduced. The disposal of contaminated sediment is currently becoming more difficult and expensive depending on the level of contamination, the range varies $100 to $400 per cubic yards and is expected to change in the future. The cost of excavating rock using a tunnel boring machine has been established at about $8,000 per linear foot.

The siting of the tunnel outlet involved the careful balancing of tunnel and channel excavation costs. Three outlet sites were evaluated. One was just off Kearny Point, the second was next to the Hackensack Navigation Channel and the third was at the junction of the Passaic and the Hackensack Navigation Channels. The second location, which is adjacent to the Hackensack Channel was determined to be the most cost effective. Since this location is next to the nearest navigation channel, the amount of sediment excavation was minimized while the length of tunnel was increased as little as possible. The only disadvantage at this location is that the flow from the outlet would be entering the navigation channel perpendicular to the direction of vessel movement. However, if the velocities are sufficiently small, the angle of incidence will not be important.

36.3.4. Outlet Capacity

The capacity of the outlet structure can affect the capacity of the entire tunnel system. If the outlet is undersized it would affect the design flow of the entire system, however if the outlet structure is oversized it will have no significant effect on capacity.

One outlet design considered was an ungated circular overflow weir set at an elevation of 10.0 feet NGVD, which is just below the 100 year tide surge level. Critical depth for the 100 yr. design flow as it passes over a 200 foot long weir would

be about 8.5 feet deep. At a depth of 8.5 feet, the energy level would be about 12 feet. Adding 12 feet to the weir crest of 10.0 results in a total energy elevation of 22.0 feet NGVD. This increase in the downstream energy level from the design tide of 6.2 feet to 22 feet represents a head loss of 15.8 feet or about 9% of head available for the entire tunnel. To make up for the capacity lost by this type of outlet configuration, the tunnel diameter would have to be increased about 0.75 feet for the full 20.4 mile length. While this design is simple to construct and operate, it is obviously not cost effective and was eliminated from further development.

At this point the hydraulic design of the outlet centered on velocity and capacity considerations. The gate openings were designed such that the velocity though the gates would be about the same as the velocity in the shaft. The gates were oriented in diverging angles so the flow would spread and the velocities would diminish as they approached the navigation channel. The gate sills were placed at an elevation of -20 feet NGVD to help reduce the size of the gates while still distributing the flow across the full depth. The higher gate sill also facilitated the use of a "float-in" design. The "float-in" design involves constructing the outlet structure in a dry or graving dock at a near-by port facility (about a mile away). The completed outlet structure would then be floated over the outlet shaft and lowered into position. The "float-in" design was developed as an alternative to constructing the outlet with the use of coffer dams. A complete explanation of the "float-in" design is in the Structural Appendix. A description of the final outlet design can be found in Section 27., Description of Improvements, of this appendix.

36.4. Hydraulic Transient Analysis

36.4.1. Purpose

Discussions with technical personnel involved in the design, construction and operation of the Chicago TARP (tunnel and reservoir project) system indicated that there was a serious surging problem in the TARP system. In the Chicago system, waves of water traveling toward each other were producing large surges of water up the vertical shafts where the waves converged. The surges were in some cases unseating manhole covers and "geysering" at street level.

Since there are some similarities between the Chicago TARP system and the Passaic diversion tunnels, an analysis of the hydraulic transient dynamics in the proposed tunnel was undertaken. Doctors Song and He of the St. Anthony Falls hydraulics lab in Minnesota were contracted to perform the analysis. Dr. Song previously analyzed and solved the Chicago TARP hydraulic transient problem. Most of the information

presented below was extracted from the St. Anthony Falls Hydraulic Laboratory, Project Report #354 "Hydraulic Transient Study of the Passaic River Flood Protection Tunnel".

The following safety concerns needed to be evaluated using a hydraulic transient computer simulation program.

(1) Surge phenomena induced during the initial filling stage.

(2) Water hammer phenomena due to a sudden pressure change.

(3) The effects of the surge and water hammer on the inlets, workshafts and downstream outlet.

36.4.2. Model Description

36.4.2.1. Model Requirements

The hydraulic phenomena which needed to be modeled represented flow conditions which were very unsteady and highly dynamic. The model used had to be able to simultaneously calculate unsteady open channel flows, unsteady pressurized flows and including the abrupt change that occurred at the shock or the surge front.

The fully dynamic transient mixed flow mathematical model (MXTRANS) developed at the University of Minnesota, was used in the analysis. For more than twenty years, the above model has been widely applied to a number of large sewer tunnel systems, such as Chicago TARP Phase I [4,5] and TARP Phase II [6,7], Milwaukee Inline Storage System [8,9], and Rochester, New York, Genesee River Storage-Conveyance System [10].

36.4.2.2. Modeling Configuration

Figure C-173a shows the modeling schematic of the tunnel system. There are a total of 379 computation stations with the modeling configuration. The distance between stations is 300 ft except the junction stations (Stations 98, 121, and 122) which represent the same location. The stationing of the main tunnel, without the Spur branch, is displayed in Figure C-173b, which will be used to show the water elevation along the main tunnel.

36.4.2.3. Modeled Conditions

Twelve different simulations were run during the course of the study. Conditions for the final five cases are listed below.

Case: A1 -- 2- year storm and "dry" tunnel;
Case: A2 -- 100- year storm and "dry" tunnel;
Case: A3 -- 100- year storm and "wet" tunnel;
Case: A4 -- 500- year storm and "dry" tunnel;
Case: A5 -- 500- year storm and "wet" tunnel.

Design parameters which are common to all five cases were as follows:

Tide level	= 6.2 ft;
Gate-controlled outlet Manning friction coeff."n"	= 0.0133;
Pompton inlet elevation	= 173.0 ft for 2yr and 100yr storms
	= 179.0 ft for 500- yr storm
Spur inlet elevation	= 163.0 ft for 2yr and 100yr storms
	= 168.0 ft for 500yr storm

In addition, the proposed heights of Workshafts #2B and #2C were set at elevations 50 and 25 feet NGVD, respectively.

36.4.3. Model Results

36.4.3.1. General

The results of the final five cases were documented in detail in the above noted St. Anthony Falls report. Most of the cases generally showed similar results with Case A3 having the largest impacts. Since Case A3 represents the worst case, the results from that simulation have been presented in detail below. Results from other simulations are summarized afterwards.

36.4.3.2. Case - A3: 100- Year Storm with Wet Tunnel

For a "wet" tunnel condition the simulation was initiated with water already in the tunnel up to the downstream tide level of 6.2 feet. At this water level the downstream section of the tunnel would be pressurized, and the upstream section would still be a free water surface. As water enters the tunnel, the interface between the pressurized zone and the free surface zone would move very rapidly to the upstream ends. As the tunnel fills and the pressurized portion expands, a surge would develop at the interface between the pressurized zone and the free surface zone. The magnitude of the surge would increase as the inflow rate increases. When the surge migrates to the upstream ends, it would produce a sudden rise in water level.

Figures C-174a and C-174b show the time variation of water surface elevations at the two inlets and the outlet during the 100- year event with a "wet" tunnel. Figure C-174b, which is more detailed, shows the sudden rise in water surface elevations at hours 4 and 5. When the surge arrives at the Pompton inlet, the water surface elevation would rise from 30 feet to 172 feet in about 7 minutes, and would fall to 50 feet in about 6 minutes. This would be a severe surge but, it would not result in backflow (flow of water from the inlet into the river) at the inlet since the peak stage would still be lower than the river water surface elevations of 173.5 to 175.

As the surge travels in the tunnel, it would also affect the water surface elevations within each workshaft. Since the ground surface at most of the workshafts would be well above the free water surface elevation (hydraulic grade line), the shafts would not be adversely impacted by the surging. However, at the downstream end of the tunnel, near the outlet, the ground elevation for two of the shafts would be at or below the steady state free surface elevation in the tunnel. The time variation of water surface elevations at Workshafts #2B and #2C, are shown in detail on Figures C-175a and C-175b. Since workshaft #2C would be very close to the downstream end, the surge there would be weak. At workshaft #2B, the peak surge would be higher than the 50 foot elevation modeled as the top of the workshaft. Unless precautions are taken to prevent this occurrence a peak flow of 1050 cfs could escape the tunnel for less than 3 minutes. To avoid this condition, the top of the shaft was raised to elevation 55.

The maximum water surface elevations along the main tunnel are shown in Figure C-176. The maximum elevations shown are the peak elevations attained during the simulation, the peak elevation at one location probably does not occur at the same time as the maximum elevation at another location. The large pressure fluctuations which occur above and below shaft #2B, are a result of moving water hammer pressures. Since the shafts play a roll of releasing the high water hammer wave, the water surface elevations at the shaft locations are not very high. Nevertheless, the maximum water surface elevation at Workshaft #2C is higher than the elevation shown in Figure C-175b. The higher elevation is not identified in Figure C-175b because the plotting data in that figure is output every six seconds. The high water hammer wave at Workshaft #2C apparently lasts less than six seconds. In addition the water hammer may not be a serious problem since the areas in which it would occur are more than 400 ft underground.

36.4.3.3. Other Cases

Simulations for Case A1 indicate that the 2- year event and "dry" tunnel condition would pass through the tunnel without any unusual occurrences. The peak water surface elevation in the tunnel would have been about 15 feet NGVD. Since elevation 15 would not pressurize the upper tunnel and since it would be below the ground at all the shaft locations, there would be no problems associated with this case.

The 100- and 500- year, "wet" and "dry" conditions displayed very similar characteristics. The largest difference was between the "wet" and "dry" simulations. Water hammers in the lower end of the tunnel were observed during the "wet" conditions but were not present during the "dry" tunnel simulations. Since the drop

shafts would release the water hammer wave and since the high water hammer pressures would occur 400 feet underground, it is unlikely that water hammer would present a problem.

There was also a small difference between the 500- and 100-year events. The 500- year events seemed to have smaller hydraulic surges. The surge reduction was probably due to the more uniform and better timed inflow rates.

36.4.4. Conclusions

The following list of the important conclusions is based on the modeling results and analyses.

1. For the 2- year flood with an initially "dry" tunnel, the Passaic River Flood Damage Reduction Tunnel System would not have transient problems with the current design.

2. For the 100- year flood with an initially "dry" tunnel, a strong surge was identified throughout the tunnel system due to rapid increase of the inflow at both upstream inlets during the initial 5 hour period. As a result of the surge, Workshaft #2B was raised to elevation 55 feet NGVD to prevent a potential overflow problem.

3. For the 500- year flood with an initially "dry" tunnel, a weaker surge occurred in the tunnel system because inflow increased less rapidly than the 100- year event during the initial 5 hour period. The weaker surge would not result in any overflow at the vertical shafts.

4. Under the "wet" tunnel condition, the surge problem would still exists since the tunnel would only be partially filled at the start of the event. In addition, a minor water hammer problem would be induced in the downstream section.

5. Further modeling will be conducted during the FDM stage. A physical model of the tunnel system will be constructed and analyzed at WES. The results of the physical model will be used to determine if the tunnel system design requires modification and if a more detailed hydraulic transient model study would be necessary.

36.4.5. Recommendations

The following recommendations for improving the current design were made by the modelers at the St. Anthony Falls Hydraulics Laboratory:

1. There are two methods to improve the surge condition for the 100- year event. The first would be to control the inflow rate at an early stage. The second method would be to modify the design of workshaft #2B, such as the diameter, outlet geometry,

and location. The authors thought that a feasible method to avoid the strong surge under the current design would be to modify the inflow hydrograph during the first five hours. The rate of rise of the modified hydrograph should be slightly smaller than that of 500- year hydrograph. This work is recommended for future study.

2. The "wet" tunnel condition, with the tunnel initially filled to an elevation of 6.2 would further aggravate the hydraulic transient condition. It would be worthwhile to test other wet conditions with the initial water level higher than 6.2 ft. To avoid hydraulic transients, a substantial portion of the mild slope portion of the tunnel should be filled.

3. The gate outlet at the downstream end results in a significant head loss under the current design, and may have negative effects on the surging problems. Improvement can be accomplished by changing the size of the gates.

4. Because the inflow rate and the rate of increase of inflow is the most important factor affecting hydraulic transients before the tunnel is completely filled, it is recommended that a detailed modeling of the intake control structures and their operations under various river flow conditions be conducted.

37. Future Studies

37.1. Physical Model Study

During the FDM most of the tunnel system will be physically modeled. The model will be constructed and tested at WES. The analysis of a physical model is critical to further design. The model study should include the following features:

a) a portion of the main tunnel,
b) a portion of the spur tunnel,
c) both tunnel inlets,
d) a portion of the approach channels at both inlets,
e) the outlet,
f) a portion of Newark Bay around the outlet,
g) the junction of the spur and main tunnels,
h) all major vertical shafts, and
i) all major changes in tunnel slope.

All the features will be connected, forming one continuous model. The total model length may be reduced by eliminating portions of the main and spur tunnels which do not have a significant impact on the tunnel flow. It is estimated that the model will be about 1:40 scale.

The physical model study will address the following design issues:

a) the efficiency of the semi-cone drop inlets,
b) the efficiency of alternate inlet designs (morning glory),
c) the optimum slope of the drop inlet,
d) the optimum dimensions of the de-aeration chambers,
e) the optimum dimensions and locations of vent shafts (if needed),
f) the effect of entrained air on the tunnel capacity,
g) the movement of hydraulic transients and waterhammers,
h) the sediment entraining potential of the inlets,
i) the sediment trapping efficiency of the tunnel,
j) the efficiency of the outlet design, and
k) the discharge velocity distributions in Newark Bay,

The model study should take about 18-24 months to complete, results would be available sooner. The cost of the study is subject to a detailed Scope of Work, however the study is currently estimated at $750,000. Initiation of the study could begin upon approval of the scope of work and the availability of WES personnel.

37.2. Ship Simulation

The impact of tunnel discharges on vessels in the navigation channels (Upper Newark Bay, Lower Hackensack and possibly Lower Passaic) will be accessed in a Ship Simulation Study. The 2-D hydrodynamic model previously developed and described, can easily be adapted for use in this study. A visual model and a ship response model will need to be developed to complete the study. A ship simulation model would take less than a year to complete. The study would be dependant on results from the physical model. A rough estimate for the cost of this study is $300,000.

37.3 Hydraulic Transient Analysis

The hydraulic transient analysis discussed in Section 36.4 stated that three items warranted further investigation. The first item was to regulate inflow into the tunnel during the first five hours to reduce potential surges. The second was to increase the size of the outlet gates. The third was to pre-fill the tunnel to a higher elevation. All three items probably can be evaluated during the physical model study. However, if the need arises a more detailed numerical hydraulic transient analysis can be performed.

The time and cost of a future transient study, if needed, is based on the previous study. The cost is roughly estimated at $30,000 and it should take about six months to complete. The analysis would be most beneficial if initiated after the completion of the physical model study.

Tables

Table C-1

PERTINENT WATERSHED DATA

Stream	Locality	Distance Above Mouth Passaic River (Miles)	Tributary Length (Miles)	Elevation in Feet m.s.l. Mouth	Elevation in Feet m.s.l. Source	Vertical Fall (Feet)	Streambed Slope(a) (Ft./Mi.)	Drainage Area (Sq. Mi.)
Passaic River	Mouth, Newark	0	87.6	-40			7.9	935.0
	Above Saddle River	15.5	72.1	0			9.0	815.5
	Dundee Dam, Clifton	17.4	70.2	25.4		17.0	8.9	809.9
	S.U.M. Dam, Paterson	25.2	62.4	114.5		63.0	8.6	785.0
	Beatties Dam, Little Falls	29.7	57.9	157.6		33.0	8.5	762.2
	Two Bridges	33.0	54.6	157.6			9.0	740.8
	Chatham	64.5	23.1	200			19.5	100.0
	Millington	75.8	11.8	210			37.3	55.4
	Source	87.6	-	-	650		0	0
Saddle River	Mouth	15.5	23	0	505		22.0	60.6
	At Lodi (U.S.G.S. Gage)	18.3	-	17	505		24.2	54.6
	Above Hohokus Creek	25.8	-	48	505		36.0	23.3
Hohokus Creek	Mouth	25.8	11.5	48	570		45.4	19.5
Weasel Brook	Mouth	14.2	5.0	-5	590		119.0	7.1
	At Clifton (U.S.G.S. Gage)	16.0	-	65	590		194.0	4.5
Diamond Brook	Mouth	22.2	4.0	24	330		76.5	3.1
Goffle Brook	Mouth	23.3	7.2	24	560		74.4	8.9
Molly Ann's Brook	Mouth	25.8	6.7	108	560		67.5	8.6
Slippery Rock Brook	Mouth	26.1	2.5	115	550		174.0	0.9
Peckman River	Mouth	28.2	7.7	115	660		70.8	9.8
Singac Brook	Mouth	31.8	8.5	150	810		77.6	11.5
Pompton River	Mouth (Two Bridges	33.0	44.8(b)	157.6	1100.0		21.0	378.1
	At Mountain View	34.4	43.4	157.6	1100.0		21.6	377.3
	At Pompton Feeder Dam	40.0	37.7	175.5	1100.0	8.0	24.5	353.8
Pequannock River	Mouth	39.7	30.8	165.0	1250.0		35.2	192.6
	Macopin Intake	50.3	20.0	583.7	1250.0		33.3	63.7
Wanaque River	Mouth	41.8	25.0	175.0	1000.0		33.0	108.1
	Above Wanaque Dam	46.6	20.2	300.3	1000.0	90.0	34.6	90.4
Ramapo River	Pompton Lakes Dam	41.9	35.8	200.9	1100.0	25.0	25.1	160.0
	Mahwah (U.S.G.S. Gage)	53.9	24.3	255.0	1100.0		34.8	118.0
Rockaway River	Mouth	47.0	43.0	160.0	1314.0		26.8	205.7
	Above Boonton	55.2	34.8	305.4	1314.0		29.0	116.0
Whippany River	Mouth	48.2	21.0	160.0	1090.0		44.3	72.0
	Near Morristown (USGS Gage)	60.2	9.0	280.0	1090.0		90.0	29.4
Green Pond Brook	At Picatinny Lake Dam	76.0	6.3	710.0	1260.0		87.2	8.7
Meadow Brook	At Lake Denmark Dam	78.0	3.4	818.0	1222.0		118.9	4.2

FOOTNOTES: (a) Slope from location indicated to source.
(b) Pompton R. source assumed to be Ramapo R.

TABLE C-2
PASSAIC RIVER BASIN, NJ
TEMPERATURE DATA - EXTREME AND ANNUAL AVERAGES

Station	Elevation Ft (NGVD)	Period of Record		Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
BOONTON NJ	280.0	1926-1989	Max	71	73	83	92	96	97	102	100	104	88	80	77	104
			Min	-15	-20	-3	17	28	32	42	36	29	16	6	-10	-20
			Averg	28	26.5	40	54.5	62	64.5	72	68	66.5	52	43	33.5	51
CANOE BROOK NJ	180.0	1931-1989	Max	73	76	86	94	97	102	103	104	99	93	84	74	104
			Min	-25	-26	-6	12	25	31	41	6	26	13	-5	-16	-26
			Averg	24	25	40	53	61	66.5	72	55	62.5	53	39.5	29	48
CHARLOTTEBURG NJ	760.0	1926-1989	Max	71	74	85	92	96	99	105	100	100	92	81	70	105
			Min	-24	-25	-10	11	24	29	36	32	24	10	-1	-17	-25
			Averg	23.5	24.5	37.5	51.5	60	64	70.5	66	62	51	40	26.5	48
JERSEY CITY NJ	140.0	1948-1989	Max	70	72	80	87	97	100	102	100	99	85	81	70	102
			Min	0	-3	5	18	34	44	49	47	38	27	14	0	-3
			Averg	35	34.5	42.5	52.5	65.5	72	75.5	73.5	68.5	56	47.5	35	55
LITTLE FALLS NJ	150.0	1948-1989	Max	73	76	83	95	101	101	104	103	105	90	84	70	105
			Min	-11	-8	3	17	31	40	47	41	32	23	15	-6	-11
			Averg	31	34	43	56	66	70.5	75.5	72	68.5	56.5	49.5	32	55
NEWARK AP NJ	30.0	1948-1989	Max	74	76	84	93	98	102	105	103	105	92	85	72	105
			Min	-8	-2	7	16	35	46	54	45	39	28	15	-1	-8
			Averg	33	37	45.5	54.5	66.5	74	79.5	74	72	60	50	35.5	57
PATERSON NJ	100.0	1948-1974	Max	72	75	81	92	97	103	105	103	106	93	84	69	106
			Min	-6	-4	9	21	34	44	51	44	35	27	8	1	-6
			Averg	33	35.5	45	56.5	65.5	73.5	78	73.5	70.5	60	46	35	56
SOMERSVILLE NJ	160.0	1931-1989	Max	74	76	86	94	99	101	104	105	105	92	84	73	105
			Min	-16	-12	-1	16	26	34	44	38	29	12	5	-10	-16
			Averg	29	32	42.5	55	62.5	67.5	74	71.5	67	52	44.5	31.5	52
WEST POINT NY	320.0	1948-1989	Max	71	71	81	96	97	102	103	103	105	89	82	71	105
			Min	-9	-11	0	15	27	40	45	41	32	22	10	-7	-11
			Averg	31	30	40.5	55.5	62	71	74	72	68.5	55.5	46	32	53

TABLE C-3
EVAPORATION, RAIN, SNOW AND TEMPERATURE DATA
FOR THE PASSAIC RIVER BASIN, NJ

Station	Elevation Ft (NGVD)	Period of Record			Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
CANOE BROOK NJ	180.0	1948-1989	E	Avg M	----	----	----	----	4.56	5.22	5.79	4.82	3.61	2.08	----	----	----
			S	Avg M	7.83	8.07	3.67	0.42	0.00	0.00	0.00	0.00	0.00	0.00	0.58	4.07	27.16
			P	Avg M	3.21	2.91	3.68	3.99	4.24	3.96	4.44	4.62	4.37	3.46	4.12	3.47	46.58
			T	Averg	24	25	40	53	61	66.5	72	55	62.5	53	39.5	29	39
PLEASANTVILLE NJ	10.0	1948-1958	E	Avg M	----	----	----	3.09	5.13	5.06	6.53	5.18	3.45	2.40	1.35	----	----
			S	Avg M	1.71	4.81	3.39	0.30	0.00	0.00	0.00	0.00	0.00	0.00	0.03	1.70	14.00
			P	Avg M	3.65	3.35	4.06	3.62	3.41	3.12	3.87	5.47	3.36	3.49	4.04	3.66	44.80
			T	Averg	26.5	34.5	44.5	55	62	67	73.5	71.5	64.5	57	46	35	41.5
RUNYON NJ	20.0	1948-1957	E	Avg M	----	----	----	----	4.46	3.67	6.09	4.85	2.64	1.76	1.23	----	----
			S	Avg M	3.73	3.50	4.17	0.61	0.00	0.00	0.00	0.00	0.00	0.00	0.50	0.70	----
			P	Avg M	3.56	3.61	4.97	4.10	3.61	2.48	2.76	5.28	2.96	3.16	3.91	4.34	44.98
			T	Averg	28	36	36	51.5	59.5	71	74	69	63.5	53.5	48	27	45

Note: E - Evaporation in Inches (Average Monthly)
S - Snow in Inches (Average Monthly)
P - Precipatation in Inches (Average Monthly)
T - Temperature in Degrees F (Average Daily)

TABLE C-4

RAINFALL DATA, PASSAIC RIVER BASIN, NJ

MONTHLY AVERAGE, ANNUAL AVERAGE AND EXTREME RAINFALL (INCHES) BY STATIONS

Station	Elevation Ft (NGVD)	Period of Record		Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
BOONTON NJ	280.0	1926-1989	Max M	11.62	6.85	8.48	10.75	11.19	13.87	10.54	11.36	12.35	9.35	10.01	9.24	68.53
			Maxyr	1979	1981	1977	1983	1989	1972	1984	1955	1934	1955	1932	1983	1972
			Min M	0.41	0.83	1.22	0.97	0.74	0.46	0.87	0.47	0.26	0.26	0.30	0.21	32.51
			Minyr	1970	1987	1981	1985	1955	1949	1929	1964	1948	1963	1933	1955	1965
			Avg M	3.21	2.91	3.68	3.99	4.24	3.96	4.44	4.62	4.37	3.46	4.12	3.47	46.58
CANISTEAR NJ	1040.0	1948-1989	Max M	12.00	6.89	10.29	10.83	9.98	12.74	10.66	21.74	11.56	13.52	10.87	9.56	68.10
			Maxyr	1979	1981	1983	1983	1984	1972	1975	1955	1987	1955	1972	1957	1975
			Min M	0.49	0.71	0.79	1.13	0.81	0.61	0.77	0.98	0.92	0.12	0.84	0.54	29.11
			Minyr	1981	1968	1981	1963	1959	1949	1968	1980	1948	1963	1976	1980	1965
			Avg M	3.46	3.18	4.09	4.43	4.17	4.20	4.49	4.74	4.24	3.72	4.37	3.90	49.01
CANOE BROOK NJ	180.0	1931-1989	Max M	10.49	6.10	9.61	11.38	11.10	11.21	13.71	14.65	11.72	7.86	11.36	10.87	71.37
			Maxyr	1979	1981	1983	1983	1968	1972	1945	1942	1975	1943	1932	1983	1983
			Min M	0.50	0.84	1.47	0.72	1.14	0.05	0.44	0.19	0.27	0.61	0.39	0.14	32.92
			Minyr	1970	1968	1985	1963	1935	1949	1955	1964	1941	1963	1976	1955	1965
			Avg M	3.57	3.10	4.24	4.12	4.33	3.99	4.67	4.83	4.44	3.53	4.25	3.75	48.83
CHARLOTTEBURG NJ	760.0	1926-1989	Max M	11.97	7.39	10.28	10.86	12.41	12.57	11.57	16.94	12.58	12.66	11.47	10.02	73.26
			Maxyr	1979	1981	1983	1983	1989	1972	1975	1955	1934	1955	1972	1957	1952
			Min M	0.51	0.86	1.00	0.74	0.90	0.56	0.97	0.84	0.54	0.42	0.55	0.45	34.00
			Minyr	1981	1987	1981	1963	1964	1988	1968	1964	1941	1963	1933	1955	1965
			Avg M	3.61	3.32	4.18	4.30	4.29	4.27	4.53	4.78	4.37	3.95	4.65	3.92	50.26
CHATHAM NJ	190.0	1948-1963	Max M	6.58	4.83	8.60	7.38	5.66	5.71	6.68	12.66	6.92	6.68	7.15	8.54	56.80
			Maxyr	1953	1956	1953	1958	1952	1959	1960	1955	1960	1958	1951	1957	1952
			Min M	0.77	1.97	2.41	2.51	1.20	0.07	0.58	1.62	0.43	0.76	1.71	0.14	39.42
			Minyr	1955	1959	1949	1950	1959	1949	1955	1953	1948	1952	1953	1955	1957
			Avg M	3.26	3.42	4.53	4.64	3.43	2.91	4.06	4.81	3.49	3.02	4.05	4.34	46.35
ELIZABETH NJ	40.0	1931-1970	Max M	7.34	6.65	8.77	6.90	9.56	6.96	10.41	16.29	12.42	9.74	9.04	7.05	63.35
			Maxyr	1949	1956	1953	1958	1948	1948	1961	1955	1944	1943	1947	1948	1945
			Min M	0.53	1.01	1.25	0.15	1.01	0.04	0.71	0.55	0.07	0.41	0.66	0.12	29.47
			Minyr	1970	1968	1966	1963	1939	1949	1966	1964	1941	1963	1931	1955	1965
			Avg M	3.50	3.23	4.31	3.80	3.91	3.50	4.61	4.52	4.20	3.27	3.73	3.57	46.24

TABLE C-4 - Continued

RAINFALL DATA, PASSAIC RIVER BASIN, NJ

MONTHLY AVERAGE, ANNUAL AVERAGE AND EXTREME RAINFALL (INCHES) BY STATIONS

Station	Elevation Ft (NGVD)	Period of Record		Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
JERSEY CITY NJ	140.0	1948-1989	Max M	8.87	5.91	10.33	8.35	9.42	9.77	10.10	12.15	9.08	7.89	8.81	11.26	65.07
			Maxyr	1979	1988	1977	1983	1984	1972	1960	1955	1975	1955	1988	1973	1975
			Min M	0.02	0.38	0.67	0.84	0.47	0.02	0.60	0.31	0.51	0.18	0.38	0.10	27.88
			Minyr	1983	1982	1966	1963	1964	1949	1955	1981	1951	1963	1976	1955	1970
			Avg M	3.02	3.01	3.80	3.85	3.89	3.11	3.97	4.00	3.56	3.13	3.86	3.52	43.63
LITTLE FALLS NJ	150.0	1948-1989	Max M	10.62	6.97	10.47	11.54	10.84	13.92	12.38	14.55	11.41	9.78	11.08	10.77	70.60
			Maxyr	1979	1981	1983	1983	1984	1972	1975	1955	1975	1955	1977	1983	1983
			Min M	0.47	0.91	1.15	0.83	0.98	0.16	0.71	0.60	0.36	0.27	0.36	0.27	35.94
			Minyr	1981	1968	1981	1985	1962	1949	1968	1964	1948	1963	1976	1955	1964
			Avg M	3.49	3.31	4.34	4.39	4.29	3.82	4.51	4.47	4.48	3.72	4.36	4.19	49.61
MAHWAH NJ	250.0	1956-1988	Max M	8.81	8.06	10.41	11.08	10.45	12.22	9.72	9.72	8.94	8.61	9.77	9.71	65.13
			Maxyr	1979	1981	1983	1983	1984	1972	1960	1971	1975	1959	1972	1983	1972
			Min M	0.50	0.63	0.71	1.04	0.43	1.44	0.80	0.88	0.72	0.62	0.63	0.69	32.14
			Minyr	1981	1978	1981	1985	1964	1965	1968	1981	1980	1963	1976	1980	1964
			Avg M	3.28	3.17	3.77	4.52	4.09	4.15	4.27	4.54	4.12	3.64	4.38	3.91	47.60
NEWARK AP NJ	30.0	1948-1989	Max M	10.10	4.94	11.14	11.14	10.22	6.40	9.98	11.84	9.00	6.70	11.53	9.47	65.50
			Maxyr	1979	1979	1983	1983	1984	1975	1988	1955	1975	1955	1977	1983	1983
			Min M	0.45	1.22	1.10	0.90	0.52	0.07	0.89	0.50	0.95	0.21	0.51	0.27	26.09
			Minyr	1981	1968	1981	1963	1964	1949	1966	1964	1951	1963	1976	1955	1965
			Avg M	3.28	3.11	4.05	3.83	3.86	2.97	4.21	4.06	3.42	3.02	3.78	3.47	43.13
NEW MILFORD NJ	10.0	1948-1989	Max M	8.88	6.39	8.96	10.47	10.40	11.69	9.02	12.30	10.35	7.82	10.29	9.04	65.69
			Maxyr	1979	1981	1953	1983	1984	1972	1984	1955	1975	1955	1972	1973	1972
			Min M	0.38	0.46	1.00	0.41	0.73	0.14	1.07	0.72	0.84	0.20	0.58	0.26	26.01
			Minyr	1981	1987	1981	1985	1987	1949	1962	1964	1948	1963	1976	1955	1965
			Avg M	3.03	3.01	3.79	3.74	3.73	3.29	3.99	3.99	3.73	3.19	3.85	3.49	43.25
OAK RIDGE NJ	880.0	1948-1989	Max M	12.00	5.95	10.18	10.91	9.65	14.87	8.54	16.08	10.45	11.55	11.25	10.48	71.72
			Maxyr	1979	1981	1983	1983	1989	1972	1975	1955	1987	1955	1972	1957	1952
			Min M	0.97	0.81	1.03	0.90	1.19	0.34	1.18	1.06	0.64	0.26	0.85	0.56	33.43
			Minyr	1955	1987	1981	1963	1959	1988	1966	1984	1948	1963	1976	1955	1965
			Avg M	3.77	3.51	4.25	4.49	4.18	4.32	4.53	5.01	4.48	3.68	4.62	4.17	50.58

TABLE C-4 - Continued

RAINFALL DATA, PASSAIC RIVER BASIN, NJ

MONTHLY AVERAGE, ANNUAL AVERAGE AND EXTREME RAINFALL (INCHES) BY STATIONS

Station	Elevation Ft (NGVD)	Period of Record		Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
PATERSON NJ	100.0	1948-1974	Max M	7.55	6.44	10.35	8.24	10.03	12.95	9.28	14.69	11.25	9.60	11.25	10.08	65.42
			Maxyr	1949	1960	1953	1973	1972	1953	1960	1955	1966	1955	1972	1973	1953
			Min M	0.55	0.07	1.83	1.30	0.99	0.24	1.19	0.96	0.35	0.27	1.77	0.38	32.63
			Minyr	1969	1973	1963	1963	1964	1949	1966	1964	1948	1963	1949	1955	1965
			Avg M	3.28	3.74	4.51	4.35	4.11	3.85	4.31	4.85	4.30	3.60	4.77	4.52	48.28
PLAINFIELD NJ	90.0	1931-1989	Max M	8.48	5.63	8.77	9.13	9.27	9.72	14.27	15.64	11.46	7.66	9.51	9.89	72.99
			Maxyr	1979	1939	1980	1983	1984	1972	1938	1955	1975	1976	1972	1983	1975
			Min M	0.66	1.06	1.39	0.87	1.19	0.02	0.86	0.42	0.14	0.35	0.36	0.21	32.06
			Minyr	1981	1980	1981	1963	1965	1949	1955	1980	1941	1963	1976	1955	1965
			Avg M	3.56	3.11	4.18	3.98	4.31	4.03	4.90	4.87	4.33	3.50	4.04	3.72	48.36
RIDGEFIELD NJ	80.0	1948-1960	Max M	6.18	6.30	9.89	7.02	6.12	4.19	8.66	11.82	7.04	7.97	8.22	7.81	52.96
			Maxyr	1953	1960	1953	1953	1952	1956	1949	1955	1954	1955	1951	1948	1958
			Min M	0.79	1.94	3.14	2.23	1.73	0.04	0.77	2.05	0.84	1.17	1.34	0.32	39.66
			Minyr	1955	1959	1960	1950	1959	1949	1955	1951	1948	1952	1949	1955	1959
			Avg M	3.42	3.45	4.87	4.05	3.81	2.91	4.22	4.61	3.10	3.89	4.12	4.33	46.86
SOMERVILLE NJ	160.0	1931-1989	Max M	9.66	11.78	7.02	9.14	9.64	9.72	11.53	14.51	10.44	9.16	8.46	8.43	66.92
			Maxyr	1979	1936	1953	1983	1984	1938	1975	1955	1975	1955	1972	1973	1975
			Min M	0.53	1.01	1.34	0.48	0.77	0.08	1.14	0.43	0.18	0.43	0.27	0.29	29.87
			Minyr	1981	1980	1985	1963	1957	1949	1944	1964	1941	1963	1933	1989	1965
			Avg M	3.31	2.99	3.81	3.81	4.03	3.89	4.75	4.59	3.80	3.32	3.71	3.41	45.44
SUSSEX NJ	390.0	1948-1989	Max M	10.17	5.05	7.13	10.00	10.58	11.18	10.01	17.30	8.41	9.24	10.20	8.48	63.71
			Maxyr	1979	1950	1977	1983	1984	1972	1975	1955	1989	1955	1972	1972	1972
			Min M	0.71	0.70	0.77	0.85	0.78	0.21	0.94	0.85	0.35	0.82	1.18	0.70	29.56
			Minyr	1955	1987	1981	1963	1959	1949	1968	1981	1948	1964	1976	1955	1964
			Avg M	3.26	2.93	3.40	4.19	4.06	4.22	4.55	4.54	3.65	3.37	4.09	3.45	45.92
WOODCLIFF NJ	100.0	1948-1989	Max M	11.25	7.72	9.90	12.60	10.56	10.66	9.97	10.06	10.88	9.22	9.78	10.24	67.02
			Maxyr	1979	1981	1983	1983	1984	1972	1984	1955	1975	1955	1972	1983	1983
			Min M	0.49	0.45	0.95	0.82	0.88	0.24	1.35	0.77	0.98	0.20	0.54	0.20	31.25
			Minyr	1981	1987	1981	1985	1964	1949	1968	1964	1948	1963	1976	1955	1965
			Avg M	3.24	3.27	4.20	4.16	3.90	3.55	4.63	4.55	4.17	3.48	4.13	3.85	46.63

TABLE C-4 - Continued

RAINFALL DATA, PASSAIC RIVER BASIN, NJ

MONTHLY AVERAGE, ANNUAL AVERAGE AND EXTREME RAINFALL (INCHES) BY STATIONS

Station	Elevation Ft (NGVD)	Period of Record		Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
NEW YORK NY	20.0	1948-1989	Max M	8.68	5.76	8.73	11.51	9.27	8.15	12.33	16.05	9.63	7.32	9.92	7.70	60.84
LA GUARDIA AP			Maxyr	1979	1960	1953	1983	1984	1972	1975	1955	1975	1983	1972	1973	1983
			Min M	0.51	0.78	0.87	0.99	0.43	0.03	0.69	0.24	1.01	0.06	0.31	0.31	22.17
			Minyr	1981	1987	1966	1985	1964	1949	1954	1964	1951	1963	1976	1955	1965
			Avg M	3.11	3.09	3.88	3.82	3.75	3.31	4.06	4.04	3.33	3.17	3.86	3.53	42.85
WEST POINT NY	320.0	1948-1989	Max M	11.07	8.58	8.85	10.16	11.18	9.56	9.64	11.74	9.74	12.75	9.01	11.47	69.49
			Maxyr	1979	1981	1953	1983	1984	1972	1960	1955	1975	1955	1972	1983	1983
			Min M	0.45	0.41	0.59	1.20	0.94	0.53	1.16	1.17	1.13	0.12	0.57	0.72	34.68
			Minyr	1970	1987	1981	1963	1959	1988	1966	1966	1948	1963	1976	1988	1964
			Avg M	3.47	3.41	4.09	4.18	4.63	3.92	4.00	4.09	4.06	3.97	4.46	4.13	48.53

TABLE C-5

SNOWFALL DATA, PASSAIC RIVER BASIN, NJ

Station	Elevation Ft (NGVD)	Period of Record		Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
BOONTON NJ	280.0	1926-1989	Max M	23.00	26.00	23.00	9.50	0.00	0.00	0.00	0.00	0.00	0.00	5.00	24.00	66.80
			Maxyr	1978	1978	1967	1982	1963	1954	1989	1989	1989	1952	1989	1966	1967
			Min M	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
			Minyr	1956	1956	1956	1987	1989	1989	1989	1989	1989	1989	1988	1988	1956
			Avg M	7.83	8.07	3.67	0.42	0.00	0.00	0.00	0.00	0.00	0.00	0.58	4.07	27.16
CHARLOTTEBURG NJ	760.0	1926-1989	Max M	31.50	34.50	34.30	13.50	0.00	0.00	0.00	0.00	0.00	2.50	17.00	26.70	88.80
			Maxyr	1945	1926	1958	1956	1977	1989	1954	1943	1952	1979	1938	1966	1961
			Min M	0.30	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	16.90
			Minyr	1980	1938	1966	1989	1989	1989	1989	1989	1989	1989	1988	1974	1953
			Avg M	9.98	10.98	7.15	1.66	0.00	0.00	0.00	0.00	0.00	0.09	1.71	7.19	38.89
ELIZABETH NJ	40.0	1931-1970	Max M	18.80	22.50	25.00	7.00	0.00	0.00	0.00	0.00	0.00	0.30	15.50	28.90	60.40
			Maxyr	1961	1961	1956	1938	1952	1951	1969	1950	1969	1940	1938	1947	1947
			Min M	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	8.00
			Minyr	1969	1938	1966	1969	1969	1969	1969	1969	1969	1969	1969	1966	1953
			Avg M	6.56	7.51	4.63	0.44	0.00	0.00	0.00	0.00	0.00	0.01	0.99	5.92	26.56
LITTLE FALLS NJ	150.0	1948-1989	Max M	22.80	19.70	23.10	5.20	0.00	0.10	0.00	0.00	0.00	1.00	5.70	20.40	60.70
			Maxyr	1961	1967	1960	1956	1954	1951	1954	1951	1989	1952	1951	1960	1961
			Min M	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	8.90
			Minyr	1989	1984	1989	1989	1989	1989	1989	1989	1989	1989	1988	1987	1953
			Avg M	7.25	7.09	5.06	0.35	0.00	0.00	0.00	0.00	0.00	0.03	0.63	5.87	29.93
NEWARK AP NJ	30.0	1948-1989	Max M	27.40	26.10	26.00	13.80	0.00	0.00	0.00	0.00	0.00	0.30	5.70	24.00	65.80
			Maxyr	1978	1979	1956	1982	1977	1952	1989	1951	1989	1952	1989	1960	1978
			Min M	0.50	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	4.00
			Minyr	1950	1981	1979	1989	1989	1989	1989	1989	1989	1989	1988	1965	1973
			Avg M	7.61	8.16	4.86	0.75	0.00	0.00	0.00	0.00	0.00	0.01	0.58	5.04	26.93
NEWTON NJ	600.0	1948-1989	Max M	32.00	30.90	27.30	12.00	0.00	0.00	0.00	0.00	0.00	2.00	10.50	21.10	72.50
ST PAUL'S ABBY			Maxyr	1961	1967	1967	1982	1977	1989	1952	1989	1989	1979	1968	1969	1967
			Min M	0.00	1.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	2.00
			Minyr	1962	1968	1959	1989	1989	1989	1989	1989	1989	1989	1973	1974	1959
			Avg M	9.54	10.21	6.86	1.53	0.00	0.00	0.00	0.00	0.00	0.09	1.74	6.25	36.55

TABLE C-5 Continued

SNOWFALL DATA, PASSAIC RIVER BASIN, NJ

Station	Elevation Ft (NGVD)	Period of Record		Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
PATERSON NJ	100.0	1948-1974	Max M	32.50	23.70	29.10	6.00	0.00	0.00	0.00	0.00	0.00	0.00	3.00	22.50	75.40
			Maxyr	1961	1961	1956	1957	1955	1951	1951	1974	1974	1952	1955	1960	1961
			Min M	0.10	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	5.50
			Minyr	1962	1968	1966	1969	1973	1974	1974	1974	1974	1973	1973	1953	1973
			Avg M	7.60	7.99	6.50	0.68	0.00	0.00	0.00	0.00	0.00	0.00	0.47	7.50	30.86
PLAINFIELD NJ	90.0	1931-1989	Max M	28.00	29.70	22.50	10.60	0.00	0.00	0.00	0.00	0.00	0.60	14.60	25.90	64.20
			Maxyr	1978	1934	1956	1982	1977	1955	1954	1989	1952	1940	1938	1947	1978
			Min M	0.10	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	7.00
			Minyr	1934	1981	1986	1989	1989	1989	1989	1989	1989	1989	1988	1972	1973
			Avg M	8.23	8.81	4.86	0.70	0.00	0.00	0.00	0.00	0.00	0.03	1.08	6.01	29.96
RIDGEFIELD NJ	80.0	1948-1960	Max M	16.60	15.50	24.20	5.00	0.00	0.00	0.00	0.00	0.00	0.30	3.50	25.00	54.80
			Maxyr	1954	1958	1956	1957	1955	1955	1951	1959	1959	1952	1953	1948	1958
			Min M	0.30	0.50	0.10	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	9.80
			Minyr	1950	1959	1954	1951	1959	1959	1959	1959	1959	1959	1954	1953	1953
			Avg M	5.65	5.58	9.27	1.29	0.00	0.00	0.00	0.00	0.00	0.03	0.86	7.79	28.33
SOMERVILLE NJ	160.0	1931-1989	Max M	22.50	25.00	27.40	10.30	0.00	0.00	0.00	0.00	0.00	1.00	8.00	28.00	54.90
			Maxyr	1978	1967	1941	1982	1989	1955	1989	1989	1989	1962	1938	1933	1978
			Min M	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	5.00
			Minyr	1933	1981	1983	1989	1989	1989	1989	1989	1989	1989	1988	1965	1931
			Avg M	7.24	7.40	4.88	0.78	0.00	0.00	0.00	0.00	0.00	0.05	0.77	5.24	26.27
SUSSEX NJ	390.0	1948-1989	Max M	49.50	32.00	36.00	12.00	0,00	0.00	0.00	0.00	0.00	0.80	16.50	22.00	82.70
			Maxyr	1987	1961	1967	1982	1967	1955	1955	1989	1989	1972	1968	1969	1967
			Min M	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	4.00
			Minyr	1960	1965	1965	1987	1989	1989	1989	1989	1989	1989	1988	1966	1958
			Avg M	9.64	8.85	5.39	1.73	0.00	0.00	0.00	0.00	0.00	0.03	1.75	7.28	36.73
NEW YORK NY LA GUARDIA AP	20.0	1948-1989	Max M	20.30	26.40	18.90	8.20	0.00	0.00	0.00	0.00	0.00	1.20	6.10	22.30	46.80
			Maxyr	1982	1983	1958	1982	1977	1989	1951	1989	1989	1962	1989	1948	1961
			Min M	0.40	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	4.10
			Minyr	1950	1981	1986	1989	1989	1989	1989	1989	1989	1989	1988	1953	1973
			Avg M	6.83	7.45	4.16	0.61	0.00	0.00	0.00	0.00	0.00	0.03	0.47	3.92	23.04

TABLE C-5 Continued
SNOWFALL DATA, PASSAIC RIVER BASIN, NJ

Station	Elevation Ft (NGVD)	Period of Record		Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
WARWICK NY	540.0	1948-1974	Max M	45.20	32.10	35.30	14.00	0.00	0.00	0.00	0.00	0.00	3.50	14.50	31.00	104.80
			Maxyr	1961	1967	1956	1956	1974	1974	1974	1974	1974	1952	1962	1969	1961
			Min M	1.80	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	25.60
			Minyr	1962	1968	1973	1974	1974	1974	1974	1974	1974	1974	1973	1965	1965
			Avg M	11.47	11.01	9.93	1.97	0.00	0.00	0.00	0.00	0.00	0.20	2.51	11.57	48.33

TABLE C-6

GREAT STORMS OVER THE NORTHEASTERN STATES

Storm No.	Date	Location of Storm Center	Distance (Miles) and Direction of Storm Center Thru Paterson	Type of Storm	Maximum Storm Rainfall at Center (inches)	Average Rainfall Over Passaic Area (inches)	Precent of Total Rain Falling in Period Indicated		Area of 9-inch Isohyetal (sq. mi.)
							Maximum 1-day	Maximum 2-day	
1	July 17-18, 1942	Port Allegheny, Pa.	219 N.W.	Extra-tropical	35.0	1.3	100	100	1,250
2	Aug. 31-Sept. 1, 1940	Ewan, N.J.	100 S.W.	Extra-tropical	24.0	1.5	100	100	1,450
3	August 19,1939	Manahawkin, N.J.	85 S.	Extra-tropical	18.0	1.9	100	100	2,800
4	Sept. 20-24,1882	Paterson, N.J.	0	Extra-tropical	17.90	9.2	-	82	2,756
5	Sept. 2-6, 1935	Easton, Md.	180 S.W.	Hurricane	16.68	2.5	50-63	89-95	8,663
6	Aug. 20-26, 1933	Peekamoose, N.Y.	71 N.W.	Hurricane	15.98	5.8	49-69	66-83	5,959
7	July 26-29, 1897	Jewel, Md.	192 S.W.	Hurricane	15.80	4.5	-	-	256
8	Sept. 16-23, 1938	Hubbardston, Mass.	151 N.E	Hurricane	15.60	7.0	43-65	65-79	14,226
9	Oct. 7-12, 1903	Paterson, N.J.	0	Extra-tropical	15.51	11.4	50-81	90-98	3,565
10	July 15-23, 1945	Midland Park, N.J.	6 N.N.E.	Extra-tropical	14.73	8.5	28-59	35-70	440
11	July 6-13, 1935	Hector, N.Y.	179 N.W.	Extra-tropical	14.23	2.5	44-73	78-90	1,946
12	Sept. 16-23, 1938	Storrs, Conn.	115 N.E.	Hurricane	14.16	7.0	31-51	56-71	14,226
13	Aug. 20-26, 1933	York, Pa.	149 S.W.	Hurricane	14.01	5.8	43-70	79-83	5,959
14	Aug. 9-13, 1928	Cheltenham, Md.	206 S.W.	Hurricane	13.29	0.8	52-83	88-100	2,050
15	Sept. 14-17, 1933	Provincetown, Mass.	221 N.E.	Hurricane	13.27	3.3	43-75	67-95	1,585
16	July 19-23, 1919	Boonton, N.J.	13 S.W.	Hurricane	12.97	7.8	25-65	50-78	1,223
17	Oct. 3-4, 1869	Canton, Conn.	91 N.E.	Extra-tropical	12.35	4.0	-	100	900
18	May 17-25, 1894	Bridgeton, N.J.	117 S.W.	Extra-tropical	12.07	2.6	-	-	--
19	Mar. 9-22, 1936	Plymouth, N.H.	226 N.E.	Transcontinental	11.18	6.0	28	30	--
20	May 17-25, 1894	Moorestown, N.J.	81 S.W.	Extra-tropical	10.41	2.6	-	-	230
21	Sept. 26-28, 1942	Elka Park, N.Y.	86 N.	Extra-tropical	10.40	3.0	58	100	20
22	July 11-15, 1897	Southington, Conn.	81 N.E.	Hurricane	10.30	4.3	80-90	100	--
23	Mar. 9-22, 1936	Fitchburg, Mass.	166 N.E.	Transcontinental	9.70	6.0	33	37	154
24	Nov. 2-6, 1927	Somerset, Vt.	151 N.E.	Hurricane	9.68	3.0	72-100	95-100	400
25	Mar. 9-22, 1936	Southbridge, Mass.	135 N.E.	Transcontinental	9.29	6.0	36	48	141
26	Aug. 17-20, 1955	Westfield, Mass.	138 N.E.	Hurricane	19.75	5.0	62-92	95-98	--
27	Oct. 14-18, 1955	West Schokan, N.Y.	115 N.	Extra-tropical	17.80	5.5	25-87	26-87	--
28	Sept. 12, 1960	Cold Brook, N.Y.	80 N.	Hurricane	8.39	4.5	47-70	81-99	--
29	May 29, 30, 1968	Newark, N.J.	5 S.E.	Extra-tropical	7.3	4.5	60	100	--
30	Aug. 27, 28, 1971	Bound Brook, N.J.	20 S.W.	Hurricane	9.2	8.0	55-85	100	--
31	June 17-24, 1972	Harrisburg, Pa.	125 W.	Hurricane	18.3	3.5	10-30	50-75	--
32	Aug. 23, 1973	Watchung, N.J.	10 S.W.	Local	8.0	2.2	95	100	--
33	Sept. 23-27, 1975	Westminster, Md.	200 S.W.	Extra-tropical	14.3	7.7	25-35	45-65	--
34	Nov. 7, 8, 1977	Sandy Hook, N.J.	55 S.E.	Extra-tropical	8.2	5.5	80	100	--

TABLE C-7

FLOODS OF RECORD AT SELECTED GAGES

Stream	Gaging Station	Drainage Area (sq. mi.)	Operating Agency	Years of Record	Peak Discharge C.F.S.	Date	Average Discharge (C.F.S. per Sq. Mi) Average for Period[b]
Passaic	Millington	55.4	U.S.G.S.	Nov.1903-July 1906	2,000	Oct. 10, 1903	1.81
Passaic	Millington	55.4	U.S.G.S.	Nov.1921-Sept.1990	2,000	Oct. 10, 1903	1.59
Passaic	Chatham	100.0	U.S.G.S.	Feb.1903-Dec.1911	5,150	Oct. 9, 1903	1.73
Passaic	Chatham	100.0	U.S.G.S.	Jan.1938-Sept.1990	5,150	Oct. 9, 1903	1.72
Passaic	Paterson (S.U.M. DAM)	785.0	U.S.G.S.	1877-1893	31,700	Oct. 10, 1903	1.91
Passaic	Little Falls[f]	762.0	U.S.G.S.	Sept.1897-Sept.1990	31,700	Oct. 10, 1903	1.52
Passaic	Paterson (S.U.M. DAM)	785.0	U.S.G.S.	Sept.1897-Sept.1953[a]	31,700	Oct. 10, 1903	1.90
Whippany	Morristown	29.4	U.S.G.S.	Sept.1921-Sept.1990	3.200	Oct. 10, 1903	1.82
Rockaway	Boonton)	119.0	J.C.W.D.-U.S.G.S.	Jan.1906-Sept.1990	7,560	Oct. 10, 1903	1.18
	) Below Reservoir						
Rockaway	Boonton)	119.0	J.C.W.D.-U.S.G.S.	Jan.1906-Sept.1953[a]	7,560[e]	Oct. 10, 1903	1.89
Pequannock	Macopin Dam	63.7	N.W.D.-U.S.G.S.	Jan.1892-Sept.1922	6,100	Oct. 10, 1903	2.03
Pequannock	Macopin Dam	63.7	N.W.D.-U.S.G.S	Oct.1922-Sept.1990	6,100	Oct. 10, 1903	0.71
Pequannock	Macopin Dam	63.7	N.W.D.-U.S.G.S	Oct.1922-Sept.1953[a]	6,100	Oct. 10, 1903	1.93
Wanaque	Awosting (Greenwood Lake)	27.1	N.J.D.W.S.C.-U.S.G.S.	May 1919-Sept.1990	1,300	Oct. 16, 1955	2.03
Wanaque	Awosting	27.1	N.J.D.W.S.C.-U.S.G.S.	Oct.1934-Sept.1953[a]	1,190	Mar. 31, 1951	1.98
Wanaque	Wanaque	90.4	N.J.D.W.S.C.-U.S.G.S.	May 1912-May 1915	11,100	Oct. 9, 1903	2.23
Wanaque	Wanaque	90.4	N.J.D.W.S.C.-U.S.G.S.	Sept.1929-Sept.1990	11,100	Oct. 9, 1903	.66
Wanaque	Wanaque	90.4	N.J.D.W.S.C.-U.S.G.S.	May 1919-Sept.1953[a]	11,100	Oct. 9, 1903	1.86)
Wanaque	Monks	40.4	U.S.G.S.	Jan.1935-Sept.1984	3,640	Aug. 19, 1955	2.06)
Wanaque	Monks	40.4	U.S.G.S.	Jan.1935-Sept.1953[a]	2,660	Mar. 31, 1951	2.07
Ramapo	Mahwah	118.0	U.S.G.S.	Feb.1903-Dec.1906[c]	12,400	Oct. 9, 1903	2.59
Ramapo	Mahwah	118.0	U.S.G.S.	Sept.1922-Sept.1990	12,400	Oct. 9, 1903	1.92
Ramapo	Pompton Lakes	160.0	U.S.G.S.	Sept.1921-Sept.1990	15,800	Oct. 9, 1903	1.80
Ramapo	Pompton Lakes	160.0	U.S.G.S.	Dec.1953-Sept.1966[a]	15,800	Oct. 9, 1903	1.92
Saddle	Lodi	54.6	U.S.G.S.	Sept.1923-Sept.1990	7,000	Oct. 9, 1903	1.83
Pompton	Pompton Feeder Dam	355.0	U.S.G.S.	June 1940-Sept.1990	28,340	Oct. 9, 1903	1.37
Pompton	Pompton Feeder Dam	355.0	U.S.G.S.	June 1940-Sept.1953[a,d]	28,340	Oct. 9, 1903	2.02
Ringwood cr.	Wanaque	19.1	U.S.G.S.	Jan.1935-Sept.1990	1,150	Mar. 30, 1951	1.75
Cupsaw Br.	Wanaque	4.4	U.S.G.S.	Jan.1935-Sept.1958	536	Mar. 11, 1936	1.83
Blue Mine Br.	Wanaque	1.7	U.S.G.S.	Jan.1935-Sept.1958	458	Mar. 30, 1951	1.36
Weasel Br.	Clifton	4.4	U.S.G.S.	Mar.1937-Sept.1962	556	Mar. 12, 1962	1.34
West Br.	Wanaque	11.8	U.S.G.S.	Jan.1935-Sept.1978	1,900	Mar. 30, 1951	2.04
Second River	Belleville	11.6	U.S.G.S.	May 1937-Sept.1964	3,300	Jul. 23, 1938	2.03

a. Corrected for water supply diversions and storage.

b. Averages were based on the water year extending from October through September, calendar year.

c. Records from 1907-1914 consist of gage heights only, published by U.S. Weather Bureau.

d. Records available from March 1903 to December 1904.

e. Average daily value.

f. Record at Paterson (drainage area 785.0 square miles) prior to October 1955.

Operating Agencies:

J.C.W.D. - Jersey City Water Department

N.W.D. - Newark Water Department

N.J.D.W.S.C. - North Jersey District Water Supply Commission

Table C-8

Annual Peak Discharges --- Selected Gages
Passaic River at Chatham, NJ
Gage # 1379500

Water Year	Adjusted Value (cfs)	Orginal Value (cfs)	Water Year	Orginal Value (cfs)
1904	3490	3300	1965	980
1905	3198	3000	1966	1010
1906	1432	1190	1967	1320
1907	3062	2860	1968	2560
1908	1834	1600	1969	1640
1909	1598	1360	1970	1420
1910	2478	2260	1971	2540
1911	778	525	1972	1520
1938	2137	1910	1973	3200
1939	1218	972	1974	1400
1940	1334	1090	1975	2520
1941	1005	756	1976	1240
1942	2244	2020	1977	1390
1943	1657	1420	1978	1520
1944	1210	964	1979	1710
1945	1284	1040	1980	1370
1946	1284	1040	1981	899
1947	1116	869	1982	1240
1948	1647	1410	1983	1380
1949	1941	1710	1984	1930
1950	921	670	1985	960
1951	1255	1010	1986	1550
1952	1530	1290	1987	1210
1953	1392	1150	1988	1220
1954	902	651	1989	1330
1955	1628	1390	1990	1190
1956	1392	1150	1991	1250
1957	1343	1100	1992	1700
1958	1383	1140	1993	1260
1959	1019	770	1994	1790
1960	1353	1110		
1961	1530	1290		
1962	1687	1450		
1963	1226	980		
1964	1119	872		

** No urbanization adjustment made beyond 1964 - use observed peaks.

Revised: 8 May 1995

Table C-8 (continued)

Annual Peak Discharges --- Selected Gages
Whippany River at Morristown, NJ
Gage # 01381500

Water Year	Adjusted Value (cfs)	Orginal Value (cfs)	Water Year	Adjusted Value (cfs)	Orginal Value (cfs)
1904	2751	2480	1960	1130	684
1922	1421	1020	1961	952	475
1923	1243	815	1962	1544	1160
1924	1874	1530	1963	861	367
1925	1382	975	1964	890	401
1926	1215	783	1965	1158	716
1927	931	450	1966	1093	640
1928	2302	2000	1967	973	500
1929	1659	1290	1968	1365	956
1930	1033	570	1969	1079	624
1931	1017	552	1970 **		1310
1932	1062	604	1971		2280
1933	2137	1820	1972		946
1934	1271	847	1973		1360
1935	945	467	1974		1450
1936	1847	1500	1975		1020
1937	966	491	1976		752
1938	1553	1170	1977		1480
1939	998	529	1978		1150
1940	1847	1500	1979		2080
1941	1126	679	1980		1100
1942	1597	1220	1981		1130
1943	1007	540	1982		871
1944	1093	640	1983		1250
1945	939	460	1984		1680
1946	1421	1020	1985		1120
1947	1051	591	1986		1060
1948	1255	829	1987		1150
1949	1712	1350	1988		1080
1950	868	375	1989		1380
1951	1350	938	1990		1350
1952	1721	1360	1991		1340
1953	1233	804	1992		1510
1954	790	282	1993		690
1955	1544	1160	1994		969
1956	1317	900			
1957	1069	613			
1958	1338	925			
1959	981	509			

** No urbanization adjustment made beyond 1969- use observed peaks.

Revised: 8 May 1995

Table C-8 (continued)

Annual Peak Discharges --- Selected Gages
Rockaway River above Boonton Reservoir, NJ
Gage # 01380500

Water Year	Adjusted Value (cfs)	Orginal Value (cfs)	Water Year	Orginal Value (cfs)
1904	10218	8050	1965**	1490
1938	3406	2260	1966	1070
1939	2054	1310	1967	1200
1940	3586	2390	1968	3400
1941	1462	908	1969	1550
1942	1893	1200	1970	2870
1943	2257	1450	1971	3550
1944	2502	1620	1972	2200
1945	2387	1540	1973	4100
1946	2243	1440	1974	4300
1947	2286	1470	1975	2230
1948	2745	1790	1976	2550
1949	5188	3590	1977	3970
1950	1585	991	1978	2590
1951	3682	2460	1979	5430
1952	6160	4360	1980	2670
1953	3113	2050	1981	2040
1954	1158	705	1982	1780
1955	3309	2190	1983	3030
1956	2958	1940	1984	5600
1957	2098	1340	1985	1460
1958	3155	2080	1986	1830
1959	1849	1170	1987	2530
1960	2603	1690	1988	1430
1961	2660	1730	1989	2350
1962	2845	1860	1990	3350
1963	1393	862	1991	1830
1964	1598	1000	1992	2310
			1993	1690
			1994	1380

** No urbanization adjustment made beyond 1964- use observed peaks.

Revised: 8 May 1995

Table C-8 (continued)

Annual Peak Discharges --- Selected Gages
Ramapo River at Pompton Lakes, NJ Gage # 01388000

Water Year	Adjusted Value (cfs)	Orginal Value (cfs)	Water Year	Orginal Value (cfs)
1904	16466	16466	1965**	2160
1922	6956	5900	1966	1460
1923	2727	2270	1967	1380
1924	7882	6800	1968	9530
1925	3893	3200	1969	3160
1926	2403	2020	1970	4540
1927	8300	7220	1971	6420
1928	4766	3930	1972	3190
1929	2429	2040	1973	5580
1930	2008	1720	1974	7040
1931	1606	1420	1975	3380
1932	2206	1870	1976	3090
1933	6722	5680	1977	5290
1934	3211	2650	1978	10300
1935	2166	1840	1979	5130
1936	12786	12300	1980	6700
1937	4268	3510	1981	3630
1938	7700	6620	1982	2590
1939	3966	3260	1983	5160
1940	3573	2940	1984	15400
1941	2193	1860	1985	2300
1942	2324	1960	1986	2760
1943	3324	2740	1987	6490
1944	2636	2200	1988	2030
1945	9579	8560	1989	5510
1946	3046	2520	1990	4070
1947	3072	2540	1991	2560
1948	2842	2360	1992	2400
1949	6551	5520	1993	2810
1950	2087	1780	1994	2280
1951	9542	8520		
1952	5739	4780		
1953	3880	3190		
1954	1525	1360		
1955	9588	8570		
1956	12544	12000		
1957	3072	2540		
1958	5706	4750		
1959	1901	1640		
1960	4316	3550		
1961	4789	3950		
1962	3046	2520		
1963	1457	1310		
1964	1525	1360		

** No urbanization adjustment made beyond 1964 - use observed peaks.

Revised: 8 May 1995

Table C-8 (continued)

Annual Peak Discharges --- Selected Gages
Pompton River at Pompton Plains, NJ
Gage # 1388500

Water Year	Adjusted Value (cfs)	Orginal Value (cfs)	Water** Year	Orginal Value (cfs)
1904	29275	29275	1965**	2600
1941	5550	2440	1966	1980
1942	5992	2840	1967	2880
1943	7602	4310	1968	13100
1944	6279	3100	1969	4220
1945	13286	9690	1970	8520
1946	6817	3590	1971	8610
1947	6992	3750	1972	7240
1948	6279	3100	1973	11500
1949	10252	6780	1974	9730
1950	5595	2480	1975	6150
1951	16721	13100	1976	6670
1952	13389	9790	1977	9000
1953	9401	5980	1978	11800
1954	4828	1790	1979	9520
1955	16523	12900	1980	12000
1956	19348	15800	1981	4600
1957	7319	4050	1982	3880
1958	10114	6650	1983	10100
1959	5351	2260	1984	25400
1960	9145	5740	1985	2710
1961	8608	5240	1986	4270
1962	6323	3140	1987	12600
1963	4504	1500	1988	3260
1964	4727	1700	1989	12500
			1990	5730
			1991	4370
			1992	6640
			1993	7170
			1994	4200

** No urbanization adjustment made beyond 1964 - use observed peaks.

Revised: 8 May 1995

Table C-8 (continued)

Annual Peak Discharges --- Selected Gages
Passaic River at Little Falls, NJ (pg 1/2)
Gage # 1389500

Water Year	Adjusted Value (cfs)	Orginal Value (cfs)	Water Year	Adjusted Value (cfs)	Orginal Value (cfs)
1882	19000	19000	1936	19200	19200
1896	16700	16700	1937	6148	5400
1898	10085	9500	1938	9944	9350
1899	8499	7830	1939	6411	5670
1900	10019	9430	1940	6363	5620
1901	9868	9270	1941	5824	5070
1902	23400	23400	1942	7750	7050
1903	11489	11000	1943	8173	7490
1904	32300	32300	1944	5834	5080
1905	9386	8760	1945	19500	19500
1906	6840	6110	1946	7789	7090
1907	8757	8100	1947	6236	5490
1908	9793	9190	1948	5971	5220
1909	7209	6490	1949	9575	8960
1910	9594	8980	1950	5086	4320
1911	4143	3370	1951	12781	12400
1912	10930	10400	1952	12414	12000
1913	8135	7450	1953	8116	7430
1914	8643	7980	1954	3744	2970
1915	8471	7800	1955	10415	9850
1916	6859	6130	1956	12045	11600
1917	5293	4530	1957	6020	5270
1918	7209	6490	1958	7054	6330
1919	9262	8630	1959	4372	3600
1920	12045	11600	1960	7837	7140
1921	6970	6244	1961	8289	7610
1922	7280	6563	1962	5716	4960
1923	8351	7675	1963	4023	3250
1924	11291	10787	1964	4193	3420
1925	6768	6036			
1926	6264	5519			
1927	9335	8707			
1928	6699	5965			
1929	5869	5116			
1930	4386	3614			
1931	4722	3953			
1932	5411	4650			
1933	9120	8480			
1934	6402	5660			
1935	4720	3950			

NOTE: Values 19,000 and above yielded a lower number after being adjusted. This was overridden and the older value was retained.

Table C-8 (continued)

Annual Peak Discharges --- Selected Gages
Passaic River at Little Falls, NJ (pg 2/2)
Gage # 1389500

Water Year	Orginal Value (cfs)
1965 **	4630
1966	3630
1967	5720
1968	13500
1969	5210
1970	9110
1971	8010
1972	10300
1973	10600
1974	8910
1975	9640
1976	6390
1977	8440
1978	9180
1979	10200
1980	9870
1981	4370
1982	4230
1983	9740
1984	18400
1985	3090
1986	6150
1987	10300
1988	3670
1989	10700
1990	6020
1991	4160
1992	6410
1993	8700
1994	5960

** No urbanization adjustment made beyond 1964 - use observed peaks.

Revised: 8 May 1995

Table C-9

Unit Hydrograph Parameters

Subbasin Number	Drainage Area (sq mi)	Clark Unitgraph Parameter TC	R
630	6.47	2.0	13.0
640	5.51	2.0	12.0
650	1.37	1.6	7.4
660	5.40	2.2	11.8
670	2.81	1.5	9.5
680	1.33	1.6	7.4
690	3.59	3.0	10.0
700	5.42	2.0	12.0
710	10.30	3.0	14.7
720	2.35	1.6	8.9
730	6.66	2.5	12.5
740	5.11	2.0	11.5
750	4.54	2.5	11.0
760	0.90	1.5	7.0
770	1.91	3.0	7.0
780	10.70	4.0	15.0
790	7.08	3.0	13.0
800	3.60	3.5	10.5
810	27.10	2.5	10.0
820	13.30	3.5	4.5
830	19.10	2.2	27.8
840	4.38	1.2	9.3
850	11.80	1.2	16.0
860	1.71	1.2	8.3
870	13.02	4.4	3.0
880	5.84	2.3	7.5
890	1.80	1.7	5.0
900	8.09	3.0	15.0
910	0.65	2.5	4.5
920	2.05	1.5	4.5
930	8.58	5.6	7.0
935	50.90	11.5	14.2
940	18.10	7.4	9.2

Table C-9 (continued)

Unit Hydrograph Parameters

Subbasin Number	Drainage Area (sq mi)	Clark Unitgraph Parameter TC	R
960	1.84	3.3	4.2
950	3.55	4.1	5.0
970	1.05	3.0	4.2
980	8.66	5.6	7.0
985	12.30	3.0	7.6
990	7.38	5.3	7.6
1000	0.63	2.5	4.2
1010	3.59	4.1	5.0
1015	0.76	2.7	4.4
1020	0.65	2.6	4.2
1030	20.10	5.6	10.5
1050	2.13	2.0	3.4
1060	0.71	1.0	3.4
1040	7.82	3.8	5.1
1070	4.59	3.1	4.3
1080	6.66	3.8	5.1
1090	1.11	2.0	4.5
1100	2.60	1.0	5.0
1110	1.58	1.0	4.0
1120	3.54	1.2	5.5
1130	2.25	1.1	4.5
1140	4.37	1.2	5.8
1150	1.70	1.1	4.3
1155	0.94	1.1	3.8
1160	2.38	1.1	4.7
1170	1.96	1.0	4.4
1180	1.90	1.0	4.4
1190	0.89	1.0	4.0
240	23.10	4.0	22.5
250	14.90	3.0	22.1
245	7.70	3.0	15.0
270	4.59	2.0	13.1
260	0.91	1.0	7.0
280	1.30	1.5	8.5
290	2.90	2.5	10.1
300	4.90	2.5	13.6

Table C-9 (continued)

Unit Hydrograph Parameters

Subbasin Number	Drainage Area (sq mi)	Clark Unitgraph Parameter TC	R
310	3.55	2.0	12.1
315	5.00	2.0	15.0
320	3.38	2.0	12.0
322	1.02	1.5	8.5
325	8.10	2.6	17.6
330	5.35	2.2	15.3
340	7.36	2.7	16.9
342	1.18	1.5	8.6
345	1.20	1.5	8.7
350	3.07	2.0	12.5
370	2.19	1.5	11.0
375	2.95	2.0	11.5
373	3.27	2.0	12.0
377	3.67	2.4	13.1
360	1.04	1.0	9.2
380	0.68	1.0	8.1
385	2.69	2.0	11.5
390	3.00	1.0	5.3
400	1.76	1.0	4.3
410	7.71	1.5	7.3
420	3.36	2.0	7.2
430	1.59	1.0	4.2
440	0.85	1.0	5.0
450	14.00	5.0	25.0
455	2.31	2.0	8.9
460	3.55	2.7	10.2
470	0.63	1.5	5.4
480	1.94	2.1	8.4
485	1.53	1.7	7.8
490	3.31	2.3	10.0
500	2.13	2.1	7.0
510	9.02	2.0	9.7
520	5.72	2.1	8.2

Table C-9 (continued)

Unit Hydrograph Parameters

Subbasin Number	Drainage Area (sq mi)	Clark Unitgraph Parameter TC	R
526	2.49	1.8	5.9
528	2.10	1.4	5.2
530	1.38	2.0	5.0
570	4.04	2.3	8.7
540	11.60	2.0	8.0
550	1.31	1.0	4.0
560	1.89	1.5	9.0
580	0.58	1.1	6.0
10	3.27	1.5	3.0
15	5.56	1.5	3.0
20	3.97	1.5	5.5
30	25.80	6.0	18.0
40	12.60	6.0	14.6
50	1.33	1.3	9.0
60	2.87	1.5	13.6
70	2.50	1.4	8.7
80	20.50	6.0	17.7
90	11.80	5.0	14.6
100	3.40	1.5	5.3
110	4.60	2.0	5.4
120	1.80	1.4	3.9
130	3.76	1.1	5.6
140	2.80	1.0	5.0
150	1.50	1.0	4.2
160	2.38	1.1	4.7
170	3.37	1.1	5.5
180	1.78	1.5	5.0
190	2.72	1.0	5.0
200	10.20	2.5	12.0
210	3.85	1.2	5.6
220	1.65	1.6	7.5
230	5.18	2.0	9.5
590	3.09	1.0	5.4
600	1.17	1.0	4.1
610	1.72	1.5	7.5
620	11.30	3.0	16.0

Table C-9 (continued)

Unit Hydrograph Parameters

Subbasin Number	Drainage Area (sq mi)	Clark Unitgraph Parameter TC	R
1200	1.89	1.0	4.4
1210	2.16	1.0	4.5
1220	1.52	2.0	6.0
1230	1.71	1.0	4.0
1240	6.90	1.4	7.0
1250	4.35	1.0	6.0
1260	4.03	1.0	6.0
1280	2.02	1.1	2.4
1290	4.36	1.4	3.0
1300	3.40	1.3	2.7
1330	5.03	1.5	3.0
1310	1.45	1.0	2.1
1315	3.66	1.3	2.8
1317	0.54	1.0	2.0
1320	2.29	1.2	2.6
1350	3.73	1.5	2.7
1340	8.71	1.9	3.7
1360	2.97	1.2	2.6
1380	6.64	1.5	3.4
1370	1.31	1.0	2.0
1385	1.37	1.0	2.0
1390	3.57	1.5	2.6
1400	21.60	7.4	5.0
1410	1.40	1.0	2.1
1413	2.63	1.5	3.5
1415	9.11	3.5	4.4
1417	3.10	1.6	3.7
1420	2.02	1.1	2.5
1430	3.12	1.2	2.7
1440	2.02	1.1	2.3
1445	1.28	1.0	2.1
1450	1.12	1.0	2.0
1460	4.97	1.4	3.1

Table C-9 (continued)

Unit Hydrograph Parameters

Subbasin Number	Drainage Area (sq mi)	Clark Unitgraph Parameter TC	R
1470	0.59	1.0	2.0
1475	0.30	1.0	2.0
1480	0.46	1.0	2.0
1490	0.88	1.0	2.0
1500	4.54	1.4	3.0
1505	4.45	2.6	5.5
1510	2.04	2.8	4.4
1570	3.57	1.5	2.6
1520	2.69	1.3	2.8
1530	6.55	1.7	3.5
1540	2.74	1.2	2.7
1550	2.51	1.2	2.6
1560	0.97	1.0	2.0
1580	4.24	1.4	2.8
1590	10.81	1.5	2.8
1600	0.79	1.0	1.5
1610	7.88	2.0	3.5
1620	13.00	2.5	4.5

Note: Subbasin numbers are presented in this table in the same sequence that they follow in the HEC-1 model.

Table C-10

% IMPERVIOUS FOR PASSAIC SUBBASINS - HYDPAR ANALYSIS

SUBBASIN #	DRAINAGE AREA SQ.MILES	ACTUAL 1978 LAND USE CONDITIONS IMPERVIOUS	ACTUAL 1990 LAND USE CONDITIONS IMPERVIOUS	PROJECTED 2050 LAND USE CONDITIONS IMPERVIOUS
10.00	3.21	11.23	12.25	13.61
15.00	5.81	10.00	12.15	17.34
20.00	3.87	24.39	25.90	25.44
30.00	25.39	18.43	19.22	21.29
40.00	12.28	21.40	21.55	22.17
50.00	1.21	22.57	24.69	24.69
60.00	2.79	21.18	22.91	25.40
70.00	2.49	16.20	16.90	18.11
80.00	20.85	13.96	16.30	23.54
90.00	11.85	21.38	23.31	28.66
100.00	3.36	29.10	29.38	31.47
110.00	4.66	28.85	29.60	29.56
120.00	1.94	16.60	18.37	22.55
130.00	3.79	34.32	35.94	35.94
140.00	2.73	13.75	13.75	28.32
150.00	1.57	20.70	20.70	28.22
160.00	2.40	13.95	13.95	31.11
170.00	3.42	17.41	17.69	25.06
180.00	1.86	31.63	32.15	33.72
190.00	2.61	25.69	25.69	26.11
200.00	10.36	28.17	28.17	34.51
210.00	3.84	22.17	22.17	29.39
220.00	1.58	22.04	22.04	24.74
230.00	5.20	23.66	23.66	29.05
240.00	23.23	10.23	10.91	17.54
245.00	7.86	16.97	18.33	28.46
250.00	15.12	30.36	30.52	34.60
260.00	0.89	35.95	35.95	36.73
270.00	4.70	16.64	18.19	22.26
280.00	1.32	22.10	25.89	32.90
290.00	2.81	35.83	37.27	39.25
300.00	4.92	16.69	17.79	21.15
310.00	3.62	24.51	26.01	31.08
315.00	5.12	20.24	20.76	20.76
320.00	3.29	7.44	8.55	14.69
322.00	1.03	6.66	7.64	16.72
325.00	8.47	9.83	10.25	15.49
330.00	5.37	28.77	30.95	31.54
340.00	7.63	18.15	18.98	27.50
342.00	1.21	27.55	31.56	36.09
345.00	1.18	27.92	27.62	32.25

Table C-10 (continued)

% IMPERVIOUS FOR PASSAIC SUBBASINS - HYDPAR ANALYSIS

SUBBASIN #	DRAINAGE AREA SQ.MILES	ACTUAL 1978 LAND USE CONDITIONS IMPERVIOUS	ACTUAL 1990 LAND USE CONDITIONS IMPERVIOUS	PROJECTED 2050 LAND USE CONDITIONS IMPERVIOUS
350.00	3.08	22.57	22.98	28.33
360.00	1.13	13.87	13.87	17.51
370.00	2.23	21.92	23.51	28.70
373.00	3.26	8.19	10.14	20.50
375.00	2.87	6.95	9.51	16.70
377.00	3.73	11.93	14.06	15.63
380.00	0.71	13.93	14.32	17.02
385.00	2.70	15.92	16.45	21.99
390.00	3.20	54.74	55.28	60.72
* 400.00	1.74	23.74	14.77	27.13
410.00	7.68	16.05	17.99	21.91
420.00	3.45	33.31	33.19	34.20
430.00	1.60	17.82	19.54	20.05
440.00	0.79	34.94	34.94	40.57
450.00	13.88	11.33	12.33	16.83
455.00	2.34	15.88	16.80	20.71
460.00	3.50	13.56	13.56	33.28
470.00	0.60	26.41	26.41	33.38
480.00	2.03	14.71	14.81	26.94
485.00	1.57	19.98	20.40	24.27
490.00	3.34	25.76	26.23	29.06
500.00	2.10	43.63	43.63	44.72
510.00	9.09	29.95	29.95	37.31
520.00	5.91	29.34	29.34	30.93
526.00	2.40	24.52	24.52	30.03
528.00	2.15	27.20	27.20	30.09
530.00	1.36	34.50	34.50	36.70
540.00	11.57	30.70	31.21	37.15
550.00	1.47	18.31	18.31	23.49
560.00	1.92	32.56	32.56	32.56
570.00	4.23	24.17	24.17	24.18
580.00	0.50	30.19	30.19	30.19
590.00	3.13	24.30	24.30	30.41
600.00	1.18	35.11	35.11	38.01
610.00	1.61	23.83	23.83	25.82
620.00	11.33	28.06	29.46	29.60
630.00	6.57	5.98	6.01	17.86
640.00	5.75	14.63	14.63	16.74
650.00	1.34	9.71	9.71	12.98
660.00	5.37	9.53	9.22	21.42
670.00	2.70	5.84	6.04	6.04
680.00	1.29	3.64	4.70	4.70

* Lake Hiawatha was removed between 1978 and 1990.

Table C-10 (continued)

% IMPERVIOUS FOR PASSAIC SUBBASINS - HYDPAR ANALYSIS

SUBBASIN #	DRAINAGE AREA SQ.MILES	ACTUAL 1978 LAND USE CONDITIONS IMPERVIOUS	ACTUAL 1990 LAND USE CONDITIONS IMPERVIOUS	PROJECTED 2050 LAND USE CONDITIONS IMPERVIOUS
690.00	3.52	25.21	25.73	26.31
700.00	5.34	14.69	14.99	22.15
710.00	10.17	14.93	14.93	15.01
720.00	2.36	8.97	10.00	15.15
730.00	6.44	8.38	8.73	9.97
740.00	5.00	17.58	18.00	18.07
750.00	4.44	19.20	19.39	20.04
760.00	0.86	3.74	3.74	4.11
770.00	1.92	11.41	12.24	15.34
780.00	11.09	18.00	18.83	21.94
790.00	7.46	20.22	21.20	24.13
800.00	3.74	18.91	22.14	25.84
810.00	29.09	19.06	19.11	23.66
820.00	13.35	6.19	8.86	9.36
825.00	Subbasin not in data.....			
830.00	16.69	10.51	10.62	23.28
840.00	4.23	12.43	13.45	13.48
850.00	11.72	9.29	9.79	12.46
860.00	0.98	5.00	5.00	7.23
870.00	14.43	30.89	31.25	31.51
880.00	6.02	12.61	12.86	14.34
890.00	1.81	22.23	22.70	24.82
900.00	8.41	15.93	16.29	21.25
910.00	0.73	30.78	30.56	30.78
920.00	2.11	27.64	33.50	35.35
930.00	8.76	17.56	18.94	29.42
935.00	52.04	12.89	13.53	19.54
940.00	18.64	9.18	10.26	10.26
950.00	3.57	15.34	15.57	21.35
960.00	1.89	6.47	6.97	13.35
970.00	1.07	14.79	15.56	35.80
980.00	9.01	11.10	11.80	22.19
985.00	12.56	12.09	12.30	18.03
990.00	7.94	18.71	20.02	26.12
1000.00	0.66	43.66	43.66	44.95
1010.00	3.57	31.22	35.59	35.59
1015.00	0.79	23.51	23.86	25.02
1020.00	0.65	43.47	43.47	48.35
1030.00	17.88	10.52	11.95	15.58
1040.00	7.67	22.39	24.30	26.00
1050.00	2.08	9.50	10.41	10.41
1060.00	0.69	9.28	11.93	13.44
1070.00	4.52	18.93	21.83	26.30
1080.00	6.49	34.25	36.07	41.14

Table C-10 (continued)

% IMPERVIOUS FOR PASSAIC SUBBASINS - HYDPAR ANALYSIS

SUBBASIN #	DRAINAGE AREA SQ.MILES	ACTUAL 1978 LAND USE CONDITIONS IMPERVIOUS	ACTUAL 1990 LAND USE CONDITIONS IMPERVIOUS	PROJECTED 2050 LAND USE CONDITIONS IMPERVIOUS
1090.00	1.16	31.65	31.65	46.46
1100.00	2.58	32.36	34.53	41.04
1110.00	1.57	24.04	36.33	43.30
1120.00	3.57	10.91	14.10	16.18
1130.00	2.23	10.35	11.01	13.84
1140.00	4.29	18.80	20.11	24.38
1150.00	1.68	20.90	24.29	24.29
1155.00	0.92	21.37	24.09	29.46
1160.00	2.31	29.36	31.71	33.24
1170.00	1.97	30.10	31.78	33.99
1180.00	1.99	33.46	35.41	37.11
1190.00	0.90	34.89	34.93	39.52
1200.00	1.99	17.86	18.00	23.60
1210.00	2.15	27.74	27.74	31.59
1220.00	1.52	42.87	42.93	46.17
1230.00	1.69	49.96	52.57	54.75
1240.00	6.68	24.27	26.40	36.01
1250.00	4.37	33.06	37.02	42.61
1260.00	4.26	42.95	46.78	49.80
1270.00	Subbasin not in data.....			
1280.00	2.03	18.25	19.55	36.46
1290.00	4.44	34.18	35.03	41.94
1300.00	3.52	31.70	32.86	43.29
1310.00	1.48	21.27	21.29	24.35
1315.00	3.70	21.09	21.42	30.29
1317.00	0.53	42.03	42.79	47.36
1320.00	2.29	38.58	38.58	46.39
1330.00	5.10	38.53	40.14	46.12
1340.00	8.85	27.80	27.84	30.24
1350.00	3.76	57.26	57.33	59.73
1360.00	2.92	35.88	35.88	36.29
1370.00	1.28	52.05	52.96	52.96
1380.00	8.01	51.46	51.46	51.73
1385.00	1.24	50.91	50.97	54.52
1390.00	3.63	52.24	52.50	52.50
1400.00	21.98	21.01	22.49	24.38
1410.00	1.26	28.44	28.58	30.19
1413.00	2.73	22.38	26.69	26.69
1415.00	9.01	24.55	25.25	25.97
1417.00	2.99	28.52	29.87	29.87
1420.00	2.15	29.59	30.11	30.16
1430.00	2.91	32.54	32.54	32.94
1440.00	1.82	37.18	37.48	37.50
1445.00	1.82	31.35	31.35	32.81

Table C-10 (continued)

% IMPERVIOUS FOR PASSAIC SUBBASINS - HYDPAR ANALYSIS

SUBBASIN #	DRAINAGE AREA SQ.MILES	ACTUAL 1978 LAND USE CONDITIONS IMPERVIOUS	ACTUAL 1990 LAND USE CONDITIONS IMPERVIOUS	PROJECTED 2050 LAND USE CONDITIONS IMPERVIOUS
1450.00	1.65	44.64	47.45	48.26
1460.00	5.16	40.35	41.53	42.62
1470.00	0.61	68.29	68.29	72.61
1475.00	0.45	39.61	39.61	39.61
1480.00	0.68	65.36	65.36	65.36
1490.00	1.26	43.28	43.28	45.86
1500.00	4.79	48.91	48.67	49.44
1505.00	3.16	47.96	48.16	49.21
1510.00	2.18	50.51	50.51	50.51
1520.00	2.79	42.71	42.94	43.69
1525.00	Subbasin not in data.....			
1530.00	7.25	35.65	36.35	38.39
1540.00	3.03	35.37	35.37	35.37
1550.00	2.76	43.46	43.46	43.46
1560.00	1.00	62.08	62.08	63.92
1570.00	3.58	50.44	50.44	50.44
1580.00	4.36	49.65	49.65	49.65
1590.00	9.62	41.25	41.25	42.45
1600.00	0.71	48.25	48.25	48.25
1610.00	8.07	45.09	45.29	47.73
1620.00	12.85	59.01	60.36	64.43

Table C-11

CURVE NUMBERS

Sub Basin Number	HYDPAR Theoretical Values			Hypothetical Floods		Historical Floods Calibrated		
	1978	1990	2050	1990	2050	1984	1977	1968
630	74.3	74.4	78.2	65	69	82	45	65
640	75.6	75.6	77.0	67	68	83	47	67
650	74.9	74.9	76.7	66	68	82	46	66
660	73.3	73.2	77.2	64	68	81	44	64
670	72.4	72.5	72.5	64	64	81	44	64
680	71.7	72.3	72.3	63	63	80	43	63
690	75.0	75.4	75.7	66	67	82	46	66
700	69.0	69.2	72.4	60	63	78	40	60
710	75.2	75.2	75.3	66	66	82	46	66
720	72.8	73.2	74.5	64	66	81	44	64
730	70.1	70.3	70.8	61	62	78	41	61
740	75.8	76.0	76.4	67	67	83	47	67
750	76.1	76.2	76.3	67	67	83	47	67
760	69.9	69.9	70.2	61	61	78	41	61
770	75.7	75.9	77.2	67	68	83	47	67
780	78.2	78.5	80.2	70	71	85	52	51
790	78.6	79.1	80.4	70	71	85	51	51
800	77.5	78.4	81.5	69	73	84	50	51
810	77.6	77.7	79.5	78	80	90	60	60
820	72.4	73.5	73.5	67	67	83	47	66
830	75.8	75.8	80.3	85	89	94	71	71
840	75.0	75.3	75.3	83	83	93	68	68
850	73.7	73.9	74.5	83	84	93	68	68
860	73.5	73.5	74.4	83	83	93	68	68
870	79.5	79.7	79.8	87	87	95	74	87
880	75.2	75.4	75.8	75	76	88	56	56
890	78.1	78.2	79.1	73	74	87	54	54
900	76.7	76.8	78.5	68	70	84	48	48
910	85.8	85.9	85.9	81	81	92	64	64
920	80.6	82.1	82.5	73	74	87	54	54
930	77.6	78.2	81.2	78	81	90	78	78
935	76.7	77.0	79.0	77	79	89	77	77
940	74.3	74.6	74.6	75	75	88	74	74
960	74.5	74.8	77.3	75	77	88	74	74
950	73.1	73.3	74.9	73	75	87	73	73
970	69.3	69.6	78.1	70	78	85	69	69
980	73.3	73.4	76.7	73	77	87	73	73
985	74.1	74.2	76.6	74	77	88	74	74
990	73.1	74.0	75.9	74	76	88	73	73

Table C-11 (continued)

CURVE NUMBERS

Sub Basin Number	HYDPAR Theoretical Values			Hypothetical Floods		Historical Floods Calibrated		
	1978	1990	2050	1990	2050	1984	1977	1968
1000	81.5	80.6	81.6	82	82	92	82	82
1010	79.7	81.6	81.6	80	80	91	80	79
1015	77.8	78.1	78.4	78	78	90	78	78
1020	83.9	84.0	85.4	84	85	93	84	84
1030	72.0	72.6	73.7	68	69	87	47	67
1050	72.5	73.0	73.0	68	68	87	48	68
1060	72.6	73.3	74.3	68	69	84	48	68
1040	77.4	78.1	79.0	73	74	86	53	72
1070	76.9	77.6	79.5	73	75	86	53	72
1080	82.6	83.6	85.6	79	81	90	60	78
1090	82.4	82.6	86.3	78	81	90	60	60
1100	86.9	87.5	88.8	83	84	92	66	82
1110	80.8	85.5	87.3	81	82	89	58	76
1120	73.1	74.9	75.5	70	71	84	48	68
1130	72.8	73.6	74.4	69	69	84	48	68
1140	78.0	78.9	81.0	74	76	87	54	73
1150	76.5	77.0	77.9	72	73	86	53	72
1155	80.8	81.6	83.4	77	78	89	58	76
1160	82.0	83.2	83.2	78	78	89	59	77
1170	85.3	86.0	87.4	81	82	91	63	80
1180	85.1	86.0	86.4	81	81	91	63	80
1190	85.9	86.1	87.3	81	82	92	64	81
240	72.5	73.0	76.3	68	71	84	68	47
250	80.0	80.2	81.5	75	77	88	75	57
245	75.1	75.4	79.8	70	75	85	70	50
270	71.6	72.4	74.6	67	70	83	67	46
260	85.7	85.7	85.9	81	81	92	81	64
280	78.1	79.6	81.7	74	76	88	73	54
290	80.3	81.1	82.3	76	77	89	75	57
300	68.8	69.4	71.3	64	66	81	64	44
310	77.6	78.2	80.6	73	76	87	73	54
315	76.4	76.6	76.6	72	72	86	71	52
320	70.9	71.5	73.8	67	69	83	66	46
322	71.7	72.4	77.9	67	73	83	67	67
325	72.9	73.0	75.3	68	70	84	68	67
330	80.4	81.0	81.8	76	77	89	75	75
340	70.7	71.3	75.7	66	71	82	66	66
342	78.3	79.5	81.6	75	77	88	73	73
345	79.8	80.0	82.3	75	77	88	75	75

Table C-11 (continued)

CURVE NUMBERS

Sub Basin Number	HYDPAR Theoretical Values			Hypothetical Floods		Historical Floods Calibrated		
	1978	1990	2050	1990	2050	1984	1977	1968
350	77.1	77.3	79.8	73	76	87	72	72
370	78.8	79.9	82.0	75	77	89	74	74
375	73.1	74.5	77.3	70	72	86	69	69
373	73.1	73.8	78.1	70	74	85	69	69
377	73.4	74.2	75.5	70	72	85	69	69
360	74.5	74.5	76.9	71	73	86	71	71
380	74.4	74.6	76.3	71	72	86	71	71
385	77.3	77.5	80.3	74	76	88	73	73
390	85.1	85.3	88.7	81	85	92	80	80
400	75.6	73.0	76.7	73	77	87	76	76
410	75.3	76.1	77.7	76	78	89	75	75
420	85.8	85.8	86.2	86	86	94	86	86
430	79.7	80.4	80.5	80	81	91	80	80
440	83.7	83.7	84.7	84	85	93	84	84
450	71.2	71.9	74.2	71	73	86	71	52
455	73.3	74.0	75.6	74	76	88	73	54
460	69.6	69.6	79.1	70	79	85	70	51
470	82.8	82.8	84.1	83	84	93	83	67
480	67.9	67.9	75.2	68	75	84	68	48
485	75.7	75.9	79.1	76	79	89	76	58
490	78.9	79.0	80.9	79	81	91	79	62
500	86.1	86.1	86.4	86	86	94	86	86
510	82.1	82.1	84.5	82	85	92	82	82
520	81.6	81.6	82.2	82	82	92	82	82
526	81.9	81.9	83.3	82	83	92	82	82
528	82.0	82.0	82.9	82	83	92	82	82
530	83.3	83.3	83.3	83	83	93	83	83
570	80.2	80.2	80.2	80	80	91	82	80
540	82.3	82.6	85.0	83	85	93	82	82
550	79.9	79.9	82.5	79	82	91	80	79
560	81.8	81.8	81.8	82	82	92	82	82
580	82.1	82.1	82.1	82	82	92	82	82
10	76.7	77.1	77.7	77	78	89	77	59
15	67.6	69.0	72.7	69	73	84	68	48
20	79.4	80.1	80.1	80	80	91	79	42
30	75.4	75.8	76.8	76	77	89	75	75
40	78.9	79.1	79.4	79	79	91	79	79
50	79.8	81.0	81.0	81	81	92	80	80
60	74.7	75.9	77.6	76	78	89	75	75

Table C-11 (continued)

CURVE NUMBERS

Sub Basin Number	HYDPAR Theoretical Values			Hypothetical Floods		Historical Floods Calibrated		
	1978	1990	2050	1990	2050	1984	1977	1968
70	78.5	78.9	79.5	79	80	91	79	79
80	73.3	74.3	77.9	74	78	88	73	73
90	76.4	77.2	79.5	77	80	89	76	76
100	75.8	76.1	76.5	76	77	89	76	76
110	76.4	76.7	76.7	77	77	89	76	76
120	63.0	63.8	67.1	64	67	81	63	63
130	71.4	72.2	72.6	72	73	86	71	71
140	75.0	75.0	78.2	75	78	88	75	75
150	75.5	75.5	78.1	76	78	89	76	76
160	75.5	75.5	80.3	76	80	89	76	76
170	75.3	75.4	77.8	75	78	88	75	75
180	67.1	67.2	68.6	67	69	83	67	67
190	84.2	84.2	84.5	84	85	93	84	84
200	76.3	76.3	78.9	76	79	89	76	76
210	70.6	70.6	73.5	71	74	86	71	71
220	71.1	71.1	71.5	71	72	86	71	71
230	73.2	73.2	75.2	73	75	87	73	73
590	75.9	75.9	77.7	76	78	89	76	76
600	80.0	80.0	80.8	80	81	91	80	80
610	73.7	73.7	74.1	74	74	88	74	74
620	77.3	78.1	78.2	78	78	90	77	77
1200	72.9	73.1	76.0	73	76	87	54	73
1210	79.5	79.5	80.7	80	81	91	63	80
1220	73.9	73.9	76.3	74	76	88	55	74
1230	79.6	80.8	82.2	81	82	91	63	80
1240	79.4	80.2	83.6	80	84	91	62	79
1250	82.0	82.9	85.2	83	85	93	66	82
1260	82.8	84.7	85.3	85	85	94	67	83
1280	77.2	77.6	81.4	51	54	85	50	50
1290	77.2	77.7	80.3	51	53	85	50	50
1300	80.8	81.4	84.5	55	60	74	55	55
1330	87.5	88.0	89.2	63	64	80	63	63
1310	78.7	78.8	80.4	51	52	85	51	51
1315	80.7	81.0	84.3	55	59	88	55	55
1317	82.0	82.5	84.1	56	58	88	55	55
1320	88.2	88.0	90.1	64	67	81	64	64
1350	92.4	92.4	92.8	70	72	85	70	70
1340	80.6	80.6	81.3	54	54	73	54	54
1360	84.0	84.0	84.4	57	57	75	57	57

Table C-11 (continued)

CURVE NUMBERS

Sub Basin Number	HYDPAR Theoretical Values			Hypothetical Floods		Historical Floods Calibrated		
	1978	1990	2050	1990	2050	1984	1977	1968
1380	90.7	90.7	90.8	67	67	83	67	67
1370	92.9	93.1	93.1	70	70	85	70	70
1385	87.2	87.2	88.8	61	63	79	61	61
1390	91.0	91.0	91.0	67	67	83	67	67
1400	78.7	79.1	79.4	56	56	74	56	56
1410	81.5	81.4	81.9	64	66	66	64	64
1413	78.2	79.3	79.3	62	62	79	60	60
1415	77.0	77.6	77.7	60	60	78	59	59
1417	78.8	79.4	79.4	62	62	79	62	62
1420	83.6	84.1	84.1	68	68	84	68	68
1430	82.1	82.1	82.3	58	58	76	58	58
1440	85.7	85.8	86.0	60	60	78	60	60
1445	83.2	83.2	83.6	56	57	74	56	56
1450	85.8	87.7	87.7	63	63	80	60	60
1460	82.4	82.7	82.9	56	56	74	56	56
1470	90.9	90.9	91.6	67	68	83	67	67
1475	84.8	84.8	84.8	59	59	77	59	59
1480	90.3	90.3	90.3	65	65	81	65	65
1490	86.1	86.1	86.6	60	62	78	60	60
1500	90.4	90.3	90.5	70	72	85	70	70
1505	90.9	90.9	91.1	72	72	86	72	72
1510	93.0	93.0	93.0	75	75	88	75	75
1570	91.1	91.1	91.1	72	72	86	72	72
1520	90.7	90.7	90.8	72	72	86	72	72
1530	86.5	86.5	87.1	66	66	82	66	66
1540	89.5	89.4	89.5	68	68	84	68	68
1550	91.4	91.3	91.4	72	72	86	72	72
1560	91.4	91.4	91.6	72	73	86	72	72
1580	91.6	91.6	91.6	73	73	87	73	72
1590	89.5	89.5	89.9	70	70	85	70	70
1600	92.5	92.5	92.5	75	75	88	75	75
1610	91.5	91.4	91.8	72	73	86	72	72
1620	92.3	92.3	92.5	73	74	87	73	73

Note: Subbasin numbers are presented in this table in the same sequence that they follow in the HEC-1 model.

Table C-12

UNET MODEL
PASSAIC RIVER MODEL

River Name	Reach Number	Upstream Boundary	Downstream Boundary
Rockaway	1		5
Whippany	2		4
Black Brook	3		4
Whippany	4	2 & 3	5
Rockaway	5	1 & 4	7
Passaic	6		7
Passaic	7	5 & 6	15
Wanaque	8		10
Pequannock	9		10
Pequannock	10	8 & 9	Storage Area 8
Ramapo	11		Storage Area 8
Ramapo	12	Storage Area 8	14
Bypass Cut	13	Storage Area 8	14
Pompton	14	12 & 13	15
Passaic	15	7 & 14	17
Deepavaal	16		17
Passaic	17	15 & 16	19
Saddle	18		19
Passaic	19	17 & 18	
Upper Main Tunnel	20	Storage Area 11	22
Spur Tunnel	21	Storage Area 12	22
Lower Main Tunnel	22	21 & 22	

TABLE C-13
Number of Cross-Sections per Reach

Reach #	River	Length (miles)	Number of X-Sections	Average Distance Between X-Sections (ft.)
1	Rockaway R.	4.88	62	415
2	Whippany R.	1.86	20	492
3	Black Brook	1.38	6	1212
4	Whippany R.	4.76	57	441
5	Rockaway R.	0.97	5	1020
6	Passaic R.	14.28	109	692
7	Passaic R.	12.67	40	1672
8	Wanaque R.	1.75	43	215
9	Pequannock R.	1.27	29	231
10	Pequannock R.	1.70	22	408
11	Rammapo R.	1.20	22	288
12	Ramaopo R.	0.70	11	332
13	Pequan. Bypass	0.30	5	300
14	Pompton R.	6.77	107	334
15	Passaic R.	0.80	15	280
16	Deepavaal Brook	3.68	58	335
17	Passaic R.	16.50	212	411
18	Saddle R.	3.24	67	256
19	Passaic R.	15.57	115	715
20	Upper Tunnel	5.59	61	500
21	Spur Tunnel	1.20	15	500
22	Lower Tunnel	14.58	160	500
		115.65	1241	

Note: See Figure C-38 for locations of these reaches.

TABLE C-14 Bridge Data

No.	Station	Community	Locality or Name	Type	# of Spans	Single Opening (ft.)	Horizontal Opening (ft.)	Bridge Opening (sq.ft.)
PASSAIC RIVER								
1	40+20	Newark	C.R.R. of N.J.	R.R Swing Thru Truss and 5 Deck Girders	6	100	567	> 10000
2	75+20	Newark	Lincoln Highway	Hwy, Vert Lift Truss and 2 Deck Girders	3	300	515	> 10000
3	84+80	Newark	Pulaski Skyway	Hwy, Thru Steel Arch and 2 Wing Spans	1	520	520	> 10000
4	114+00	Newark	Conrail R.R.	R.R Swing Thru Truss and 5 Deck Girders	6	111	648	> 10000
5	117+90	Newark	N. J. Turnpike	Hwy, Deck Plate Girder	3	390	660	> 10000
6	222+20	Newark	Jackson Ave	Hwy, Swing Thru Truss 1 Fixed Truss, 1 Deck Girder	3	145	295	> 10000
7	242+40	Newark	Amtrak R.R.	R.R. Vert Lift Thru Truss and 2 Plate Girders	3	200	386	> 10000
8	276+37	Newark	Bridge Street	Hwy, Swing Thru Truss	3	80	273	6840
9	285+52	Newark	Conrail R.R.	R.R Swing Deck Truss and 2 Deck Trusses	3	75	327	> 10000
10	287+20	Newark	Route Int. 280	Hwy, Vert. Lift	3	200	545	> 10000

TABLE C-14 (con't) Bridge Data

No.	Station	Community	Locality or Name	Type	# of Spans	Single Opening (ft.)	Horizontal Opening (ft.)	Bridge Opening (sq.ft.)
11	299+08	Newark	Central Ave	Hwy Swing Thru Truss and 2 Plate Girders	3	75	211	8600
12	312+43	Newark	Erie R.R.	R.R., Base Thru Truss and 3 Deck Girders	4	134	322	7820
13	407+09	Newark	Conrail R.R.	R.R. Swing Deck Girder and 4 Deck Girders	5	70	396	7530
14	443+77	Belleville	Rutgers Street Hwy	Base Thru Truss and 3 Thru Trusses	4	100	279	7560
15	541+53	Lyndhurst	Kingsland Ave	Hwy, Swing Thru Truss and 2 Thru Trusses	3	99	333	8290
16	597+28	Clifton	D.L. and W. R.R.	R.R. Swing Deck Girder	4	99	295	> 10000
17	609+30	Clifton	Route 3	Hwy, Double Bascule	4	125	418	> 10000
18	680+46	Passaic	Union Ave	Hwy, Swing Thru Truss and 2 Deck Trusses	3	65	218	5870
19	728+35	Passaic	Gregory Ave	Hwy, Swing Thru Truss	2	91	166	4142
20	760+60	Passaic	Second Street	Hwy, Deck Girder Double Base 2 Conc.	3	99	267	5515

TABLE C-14 (con't) Bridge Data

No.	Station	Community	Locality or Name	Type	# of Spans	Single Opening (ft.)	Horizontal Opening (ft.)	Bridge Opening (sq.ft.)
21	790+84	Passaic	West 8th Street	Hwy, Thru Truss Base and 2 Thru Trusses	3	108	244	3945
22	830+83	Passaic	Wall Street	Hwy, Thru Steel Girder	3	73	219	3220
23	846+81	Passaic	Erie R.R. Spur	Trestle, Wood	23	11	231	3737
24	847+73	Passaic	Monroe Street	Hwy, Deck Conc. Arch	3	91	247	3584
25	894+50	Clifton	Akerman Ave	Hwy, Deck Conc. Arch	7	55	379	7550
26	945+25	Paterson	Garden State Parkway	Hwy, Deck Steel Girder	7	90	630	8943
27	960+14	Paterson	Route 46	Hwy, Deck Conc. Arch	5	108	498	7257
28	997+43	Paterson	Route Int 80	Hwy, Deck Steel Girder	4	90	360	12827
29	998+11	Paterson	Market Street	Hwy, Conc. Covered Girder	6	55	323	4700
30	1004+80	Paterson	N.Y.S. and W.R.R.	R.R. Deck Plate Girder	4	95	370	10826
31	1059+77	Paterson	Broadway	Hwy, Open Spandrel Conc Arch	3	118	301	5268

TABLE C-14 (con't) Bridge Data

No.	Station	Community	Locality or Name	Type	# of Spans	Single Opening (ft.)	Horizontal Opening (ft.)	Bridge Opening (sq.ft.)
32	1103+40	Paterson	East 33rd Street	Hwy, Thru Plate Girder	3	80	240	4100
33	1143+40	Paterson	Fifth Ave. Fairlawn	Hwy, Thru Truss	2	128	256	5050
34	1170+20	Paterson	Wagraw Bridge	Hwy, Conc. Arch	3	106	279	3461
35	1183+20	Paterson	19th Street Bridge	Hwy, Conc. Encased Deck Girder	6	57	315	4884
36	1203+74	Hawthorne	N.Y.S. and W.R.R.	R.R. 3 Plate Girders	3	117	283	10444
37	1213+50	Hawthorne	Erie R.R.	R.R. Truss and 2 Thru Plate Girders	3	125	286	5368
38	1242+34	Paterson	Sixth Avenue	Hwy, Deck, Steel Arch	4	72	207	4254
39	1274+35	Paterson	Straight Street	Hwy, Steel Thru Truss	1	248	248	4286
40	1290+45	Paterson	Arch Street	Hwy, Steel Thru Truss	1	174	174	2677
41	1298+36	Paterson	Main Street	Hwy, Deck Steel Arch	3	71	190	2355
42	1304+74	Paterson	West Broadway	Hwy, Deck Conc.Arch	3	93	263	3233

TABLE C-14 (con't) Bridge Data

No.	Station	Community	Locality or Name	Type	# of Spans	Single Opening (ft.)	Horizontal Opening (ft.)	Bridge Opening (sq.ft.)
43	1308+05	Paterson	Mulberry Street	Hwy, Thru Steel Truss	1	100	100	919
44	1339+35	Paterson	Spruce Street	Hwy, Deck, Steel and Conc.	5	55	238	4982
45	1357+61	Paterson	Rockland Street	Footbridge, Thru Steel Arch	3	124	252	4378
46	1398+80	West Paterson	Glover Street	Hwy, 2 Span Conc. Plate Girde	r 2	138	275	4280
47	1417+23	West Paterson	Hillery Street	Hwy, Thru Steel Truss	4	81	323	9570
48	1444+90	Totowa	Route Int. 80	Hwy, Deck Steel Girder	6	120	522	13500
49	1505+76	West Paterson	Lackawanna Ave.	Hwy, Thru Steel Truss	3	97	278	4238
50	1510+65	Little Falls	Newark Aqueduct	Aqueduct, 3 Thru Steel Trusses	3	98	326	5136
51	1512+70	Little Falls	Route 46	Hwy, Conc., Rib Arch	5	91	442	9840
52	1558+55	Little Falls	N.J.D.W.S.C.	Aqueduct Deck Steel Truss	1	162	162	5945

TABLE C-14 (con't) Bridge Data

No.	Station	Community	Locality or Name	Type	# of Spans	Single Opening (ft.)	Horizontal Opening (ft.)	Bridge Opening (sq.ft.)
53	1627+74	Little Falls	Erie R.R.	R.R. Thru Plate Girder	3	83	249	4208
54	1630+97	Wayne	Route 23	Hwy, Deck Conc. Arch	3	99	257	3436
55	1710+50	Caldwell	Route 46	Hwy, Deck Conc. Encased Steel Girder	3	79	229	3218
56	1717+30	Two Bridges	Route Int. 80	Hwy, Deck Steel Girder	3	130	380	8573
57	1739+40	Two Bridges	Two Bridges Road	Hwy, Deck Conc. Encased Steel Girder	3	83	247	2483
58	2198+60	Caldwell	Horseneck Road	Hwy,Thru Steel Girder	5	45	215	2141
59	2309+50	Caldwell	Route Int. 80	Hwy, Deck Steel Girder	3	127	381	6042
60	2346+50	Pine Brook	Route 46	Hwy, Deck Conc. Encased I Beam	4	60	240	3318
61	2364+20	Pine Brook	Bloomfield Ave	Hwy, Conc. Encased I Beam	4	46	178	2530
62	2506+85	Pine Brook	Route Int. 280	Hwy, Deck Steel Girder	3	150	250	2950
63	2525+90	East Hanover	Eagle Rock Road	Hwy, Deck Conc. Encased I Beam	2	70	140	1812

TABLE C-14 (con't) Bridge Data

No.	Station	Community	Locality or Name	Type	# of Spans	Single Opening (ft.)	Horizontal Opening (ft.)	Bridge Opening (sq.ft.)
64	2563+05	East Hanover	Morristown & Erie R.R.	R.R. Trestle	12	14	168	3631
65	2673+30	East Hanover	Route 10	Hwy, Deck Conc.	3	36	108	114
66	2676+65	East Hanover	Mt Pleasant Ave	Conc. Encased Girder	3	32	90	1157
67	2770+50	Florham Park	Columbia-S. Orange Ave.	Hwy, Deck Conc.	3	33	99	5790
68	2941+20	Lower Chatham	Passaic Ave	Hwy, Deck Steel Girder	2	50	98	890
69	3032+10	Chatham	Route 24	Hwy, Deck Conc. Arch	1	80	80	1310
70	3048+40	Chatham	Main Street Conn.	Hwy, Deck Steel and Conc. Arch	1	80	80	764
71	3049+00	Chatham	Main Street	Hwy, Deck Steel and Conc. Arch	1	40	40	700
72	3085+50	Chatham	Chatham Rd	Hwy, Deck Conc. Arch	1	74	74	547
73	3115+60	Stanley	Watchung Ave	Hwy, Deck Steel Girder	2	37	73	1819
74	3147+25	Stanley	D.L. and W.R.R.	R.R. Stone Masonry Arch	1	46	46	825

TABLE C-14 (con't) Bridge Data

No.	Station	Community	Locality or Name	Type	# of Spans	Single Opening (ft.)	Horizontal Opening (ft.)	Bridge Opening (sq.ft.)
ROCKAWAY RIVER								
1	94+70	Rockaway Neck	Footbridge	Steel Truss	1	81	81	807
2	121+05	Pine Brook	Route 46	Hwy, Conc. Encased I Beam	3	38	115	1139
3	125+20	Pine Brook	Old Bloomfield Rd.	Hwy, Steel Girder	1	70	70	975
4	136+50	Pine Brook	Route 80	Hwy, Deck Steel Girder	3	78	220	3196
5	200+65	Pine Brook	Vail Road	Hwy, Thru Steel Truss	2	43	86	817
6	307+52	Mountain Lakes	Knoll Road	Hwy, Thru Steel Truss	1	90	90	676
WHIPPANY RIVER								
1	18+25	Troy Hill	Edward's Road	Hwy, Steel Thru Truss	1	35	35	207
2	21+00	Troy Hill	Route 280	Hwy, Deck Steel Girder	2	50	101	2220
3	23+90	Troy Hill	Ridgedale Ave.	Hwy, Deck Steel Girder	2	96	192	1982

TABLE C-14 (con't) Bridge Data

No.	Station	Community	Locality or Name	Type	# of Spans	Single Opening (ft.)	Horizontal Opening (ft.)	Bridge Opening (sq.ft.)
4	151+60	Troy Hill	Troy Hill Road	Hwy, Steel Truss	1	60	60	412
5	192+15	E. Hanover	Morristown and Erie R.R.	R.R. Steel Beam and Timber	11	11	106	1040
6	198+75	E. Hanover	Railroad Spur	R.R. Steel Beam and Timber	1	55	55	564
7	223+50	E. Hanover	Melanie Lane	Hwy, Steel Girder	2	46	91	595
8	243+50	Hanover	Route 10	Hwy, Steel Beam	2	42	84	964
9	331+40	Whippany	Whippany Road	Hwy, Steel Truss	1	60	60	372
POMPTON RIVER								
1	3+22	Lincoln Park	Two Bridges	Hwy, 2 Steel Thru Trusses	2	80	159	1830
2	74+70	Mountain View	Boonton Turnpike	Hwy, Deck Steel Girder	2	85	173	2486
3	138+90	Wayne	Lackwanna R.R.	Trestle, 2 Thru Steel Trusses 2 Steel Deck Plate Girder	5	105	287	3448
4	221+95	Pequannock	Erie Lackwanna R.R.	Trestle, 3 Thru Plate girder	3	82	171	3010

TABLE C-14 (con't) Bridge Data

No.	Station	Community	Locality or Name	Type	# of Spans	Single Opening (ft.)	Horizontal Opening (ft.)	Bridge Opening (sq.ft.)
5	246+05	Pequannock	Newark and Pompton Turnpike	Hwy, Thru Girder Conc. Encase	d 2	86	152.5	2730
6	250+60	Pequannock	Newark Supply	Aqueduct, Steel Thru Truss	4	77	223	3419
7	274+35	Pequannock	Route 23	Hwy, Deck Conc. Encased Girde	5	74	224	3463
8	348+40	Pequannock	Pompton Plains Rd	Hwy, Thru Steel Plate Girder	2	50	79	1320
RAMAPO RIVER								
1	72+90	Pompton	Dawes Hwy	Hwy, Deck , Conc. Arch	1	91	91	937
2	98+65	Pompton	Paterson Hamburg Turnpike	Hwy, Conc. Beam	3	39	114	1172
PEQUANNOCK RIVER								
1	47+28	Pompton Plains	Footbridge	Footbridge, Steel Suspension	1	108	108	1040
2	103+85	Riverdale	Riverdale Road	Hwy, Steel Truss	2	57	113	2320
3	150+40	Pompton Lakes	Paterson Hamburg Turnpike	Hwy, Conc. Beam	3	40	116	1100

TABLE C-14 (con't) Bridge Data

No.	Station	Community	Locality or Name	Type	# of Spans	Single Opening (ft.)	Horizontal Opening (ft.)	Bridge Opening (sq.ft.)
4	170+40	Pompton Lakes	Erie Lackawanna R.R.	R.R. Wood Pile Bents	15	12	177	1242
WANAQUE RIVER								
1	26+45	Pompton Lakes	Hershfield Park Footbridge	Footbridge, Concrete Girder	3	60	105	815
2	52+15	Pompton Lakes	Paterson Hamburg Turnpike	Hwy, Conc. Deck and Conc. Arch	2	50	107	1400
3	67+80	Pompton Lakes	Lenox Ave. Footbridge	Footbridge, Concrete Girder	3	60	105	748
4	80+20	Pompton Lakes	Susquehanna Western R.R.	R.R. Steel Truss	1	100	100	1640
5	88+00	Pompton Lakes	Wanaque Ave	Hwy, Conc. Deck Steel Girders	1	95	95	1900
DEEPAVAAL BROOK								
1	4+25	Fairfield	Little Falls Rd.	Hwy, Conc. Beam	1	47	47	450

TABLE C-14 (con't) Bridge Data

No.	Station	Community	Locality or Name	Type	# of Spans	Single Opening (ft.)	Horizontal Opening (ft.)	Bridge Opening (sq.ft.)
2	22+00	Fairfield	Passaic Ave.	Hwy, Conc. Deck	4	16	54	577
3	23+50	Fairfield	Culvert thru Building	Culvert	1	37	37	492
4	52+95	Fairfield	New Dutch Lane	Hwy, Conc. Beam	2	20	40	400

Table C-15a
Calibration Analysis
Comparison of Computed and Observed
Peaks, Volumes and Lag Times
November 1977

Number	Name	Peak			Volume	Lag
		Comp (CFS)	Obsd (CFS)	Diff (PCT)	Diff (A) (PCT)	Time (B) (HRS)
13875	Ramapo at Mahwah	10690	11600	-8	+11	+3.5
13880	Ramapo at Pompton Lakes	10991	10500	+5	-1	+4
13905	Saddle at Ridgewood	4372	4620	-5	+25	0
13915	Saddle at Lodi	5225	4500	+16	+4	+2

(A) A positive volume difference indicates that the computed volume exceeded the observed volume for the corresponding time period (period length varied between gages and/or floods).

(B) A positive time difference indicates that the computed peak occurred later than the observed peak. A negative time difference indicates that the computed peak occurred earlier than the observed peak.

Table C-15b
Calibration Analysis
Comparison of Computed and Observed
Peaks, Volumes and Lag Times
April 1984

Number	Name	Peak			Volume	Lag
		Comp (CFS)	Obsd (CFS)	Diff (PCT)	Diff (A) (PCT)	Time (B) (HRS)
13790	Passaic at Millington	1340	1370	-2	+6	-5
13795	Passaic at Chatham	1994	1930	+3	+2	-24(C)
13805	Rockaway above Boonton Res	5467	5470	0	-2	+5
13810	Rockaway below Boonton Res	4690	4260	+10	-3	+6
13815	Whippany at Morristown	1712	1680	+2	+13	+3
13825	Pequannock at Macopin	4288	4880	-12	+2	+2.25
13835	Wanaque at Awosting	2758	2800	-2	-17	-5
13840	Wanaque at Monksville	3754	3100	+21	+26	+4
13870	Wanaque at Wanaque	8922	9500	-6	-7	-4.25
13875	Ramapo at Mahwah	12790	15300	-16	+4	+2.25
13880	Ramapo at Pompton Lakes	14071	15400	-9	-8	+2.5
13885	Pompton at Pompton Plains	26215	25400	+3	-1	+0.5

Table C-15b (continued)
Calibration Analysis
Comparison of Computed and Observed
Peaks, Volumes and Lag Times
April 1984

Number	Name	Peak Comp (CFS)	Peak Obsd (CFS)	Peak Diff (PCT)	Volume Diff (A) (PCT)	Lag Time (B) (HRS)
13895	Passaic at Little Falls	19079	18400	+4	+6	+1
13905	Saddle at Ridgewood	1905	1560	+22	-15	0
13910	Hohokus at Hohokus	2124	2570	-17	-8	+1
13915	Saddle at Lodi	4050	3350	+21	+4	+1

(A) A positive volume difference indicates that the computed volume exceeded the observed volume for the corresponding time period (period length varied between gages and/or floods).

(B) A positive time difference indicates that the computed peak occurred later than the observed peak. A negative time difference indicates that the computed peak occurred earlier than the observed peak.

(C) Not an indicator of a bad flood reproduction given the breadths of the hydrographs involved, which result from extensive upstream storage.

Table C-15c
Calibration Analysis
Comparison of Computed and Observed
Peaks, Volumes and Lag Times
May 1968

Number	Name	Peak			Volume Diff (A)	Lag Time (B)
		Comp (CFS)	Obsd (CFS)	Diff (PCT)	(PCT)	(HRS)
13790	Passaic at Millington	1077	1070	1	-2	+9
13795	Passaic at Chatham	2641	2560	+3	+12	+5
13805	Rockaway above Boonton Reservoir	3003	2980	+1	-10	+1
13835	Wanaque at Awosting	919	870	+6	+3	0
13840	Wanaque at Monksville	3114	3330	-6	-1	+3
13870	Wanaque at Wanaque	5067	5920	-14	-9	+1
13880	Ramapo at Pompton Lakes	8490	9530	-11	-5	+3
13885	Pompton at Pompton Plains	14395	14700	-2	+11	+7
13895	Passaic at Little Falls	13495	12600	+7	0	+4
13915	Saddle at Lodi	3303	3330	-1	-31	0

(A) A positive volume difference indicates that the computed volume exceeded the observed volume for the corresponding time period (period length varied between gages and/or floods).

(B) A positive time difference indicates that the computed peak occurred later than the observed peak. A negative time difference indicates that the computed peak occurred earlier than the observed peak.

Table C-16

ROUGHNESS COEFFICIENTS

From	To	Left Overbank	Right Overbank	Channel
PASSAIC RIVER				
REACH 19				
Mouth	Rt. 3	0.05	0.05	0.025
Rt. 3	Saddle River	0.08	0.08	0.035
REACH 17				
Saddle River	Route 46 (Clifton)	0.08	0.08	0.028
Route 46	Broadway	0.08	0.08	0.035
Broadway	Fifth Ave. (Fairlawn)	0.08	0.08	0.032
Fifth Ave.	Straight St.	0.08	0.08	0.035
Straight St.	Great Falls	0.10	0.10	0.040
Great Falls	Rt. 23	0.08	0.08	0.040
Rt. 23	Deepavaal Brook	0.08	0.08	0.035
REACH 15				
Deepavaal Brook	Two Bridges	0.10	0.10	0.033
REACH 7				
Two Bridges	Rockaway River	0.10	0.10	0.033
REACH 6				
Rockaway River	Route 10	0.10	0.10	0.028
Route 10	Study Limit	0.10	0.10	0.030
SADDLE RIVER				
REACH 18				
Mouth	Study Limit	0.16	0.16	0.045
DEEPAVAAL BROOK				
REACH 16				
Mouth	Study Limit	0.10	0.10	0.047
POMPTON RIVER				
REACH 14				
Mouth	Boonton Turnpike	0.10	0.10	0.030
Boonton Turnpike	Ramapo River	0.13	0.13	0.037
PEQUANNOCK RIVER				
REACH 13 (Bypass)				
Mouth	Diversion Dam	0.10	0.10	0.035
REACH 10				
Mouth	Wanaque River	0.18	0.18	0.045
REACH 9				
Wanaque River	Paterson-Hamburge Tpk.	0.18	0.18	0.048
Pat.-Ham. Tpk.	Study Limit	0.18	0.18	0.045

Table C-16 (cont)

ROUGHNESS COEFFICIENTS

From	To	Left Overbank	Right Overbank	Channel
RAMAPO RIVER				
REACH 12				
Mouth	Diversion Dam	0.15	0.15	0.038
REACH 11				
Diversion Dam	Study Limit	0.15	0.15	0.038
WANAQUE RIVER				
REACH 8				
Mouth	Study Limit	0.17	0.17	0.043
ROCKAWAY RIVER				
REACH 5				
Mouth	Whippany River	0.12	0.12	0.035
REACH 1				
Mouth	Study Limit	0.15	0.15	0.039
WHIPPANY RIVER				
REACH 4				
Mouth	Pinch Brook	0.12	0.12	0.035
REACH 2				
Pinch Brook	R.R. Spur	0.12	0.12	0.035
PINCH BROOK				
REACH 3				
Mouth	Study Limit	0.12	0.12	0.035

Table C-17a FLOODMARK CALIBRATION for the 1984 EVENT

Stream	Location	Station	Agency	Recorded Elevation (ft.)	Computed Elevation (ft.)
Passaic River					
	Chatham Gage	3153+95	U.S.G.S.	200.7	200.8
	upstream Passaic Ave.	2941+75	U.S.G.S.	174.3	173.3
	downstream Passaic Ave.	2941+05	U.S.G.S.	174.3	173.3
	u/s Columbia Rd.	2770+80	U.S.G.S.	173.4	173.2
	d/s Columbia Rd.	2769+90	U.S.G.S.	173.4	173.2
	d/s N.J. Route 10	2671+30	U.S.G.S.	173.2	173.1
	u/s Eagle Rock Ave.	2526+65	U.S.G.S.	172.5	172.8
	d/s Eagle Rock Ave.	2525+90	U.S.G.S.	172.5	172.8
	Pine Brook Gage	2346+50	U.S.G.S.	172.2	171.7
	u/s Two Bridges	1739+90	U.S.G.S.	171.8	171.2
	d/s Two Bridges	1739+40	U.S.G.S.	171.6	171.1
	u/s U.S. Route 46	1712+50	U.S.G.S.	170.9	170.4
	u/s Pompton Turnpike	1631+27	U.S.G.S.	168.2	168.4
	d/s Pompton Turnpike	1629+68	U.S.G.S.	168.2	168.2
	u/s Beattie's Dam	1569+65	U.S.G.S.	164.0	164.2
	Little Falls Gage	1538+60	U.S.G.S.	133.3	133.0
	Little Falls	1537+60	U.S.G.S.	133.1	132.8
	u/s Lackawanna Ave.	1506+36	U.S.G.S.	130.9	130.6
	d/s Lackawanna Ave.	1505+46	U.S.G.S.	130.9	130.6
	u/s Lincoln Street	1399+40	U.S.G.S.	124.7	124.8
	d/s Lincoln Street	1397+30	U.S.G.S.	124.7	124.8
	Spruce Street	1340+00	U.S.G.S.	121.2	121.2
	u/s West Broadway	1306+06	U.S.G.S.	50.7	40.7

Table C-17a (con't) FLOODMARK CALIBRATION

1984 EVENT

Stream	Location	Station	Agency	Recorded Elevation (ft.)	Computed Elevation (ft.)
	d/s West Broadway	1304+49	U.S.G.S.	49.8	49.6
	u/s Straight Street	1274+85	U.S.G.S.	47.7	46.8
	d/s Straight Street	1274+35	U.S.G.S.	47.6	46.8
	u/s Sixth Street	1243+06	U.S.G.S.	43.4	43.5
	d/s Sixth Street	1240+84	U.S.G.S.	43.2	43.3
	u/s Lincoln Ave.	1183+65	U.S.G.S.	40.8	40.3
	d/s Lincoln Ave.	1183+20	U.S.G.S.	40.8	40.3
	u/s East 33rd Street	1103+88	U.S.G.S.	35.9	35.9
	d/s East 33rd Street	1103+30	U.S.G.S.	35.9	35.9
	u/s Erie R.R.	1005+42	U.S.G.S.	33.8	32.8
	d/s Erie R.R.	1002+40	U.S.G.S.	33.3	32.6
	u/s Route 46	961+06	U.S.G.S.	30.8	31.0
	d/s Route 46	959+59	U.S.G.S.	30.7	31.0
	u/s Dundee Dam	917+90	U.S.G.S.	30.5	30.8
	u/s Ackerman Ave.	895+10	U.S.G.S.	16.3	15.7
	d/s Ackerman Ave.	894+35	U.S.G.S.	16.3	15.7
	u/s Monroe Street	848+30	U.S.G.S.	12.8	13.0
	d/s Monroe Street	847+40	U.S.G.S.	12.8	12.9
Rockaway River					
	u/s Vail Road	201+60	U.S.G.S.	176.4	176.0
	d/s Vail Road	200+55	U.S.G.S.	176.2	175.7

Table C-17a (con't) FLOODMARK CALIBRATION

1984 EVENT

Stream Location	Station	Agency	Recorded Elevation (ft.)	Computed Elevation (ft.)
u/s Bloomfield Ave.	126+60	U.S.G.S.	174.5	174.1
d/s Bloomfield Ave.	125+10	U.S.G.S.	174.3	173.9
Pequannock River				
u/s Paterson-Hamburg Tpk.	151+00	U.S.G.S.	201.5	200.5
d/s Paterson-Hamburg Tpk.	150+40	U.S.G.S.	199.4	199.8
u/s Riverdale Road	105+95	U.S.G.S.	192.2	191.9
d/s Riverdale Road	102+65	U.S.G.S.	190.7	190.9
Foot Bridge	15+84	U.S.G.S.	186.4	186.1
Wanaque River				
Lake Inez Dam	92+65	U.S.G.S.	205.6	205.3
u/s Wanaque Ave.	90+10	U.S.G.S.	204.5	204.8
u/s R.R. Bridge	80+80	U.S.G.S.	203.0	202.7
d/s R.R. Bridge	79+05	U.S.G.S.	202.7	202.5
u/s Paterson-Hamburg Tpk.	53+10	U.S.G.S.	197.9	197.2
d/s Paterson-Hamburg Tpk.	52+00	U.S.G.S.	197.0	196.6
Ramapo River				
u/s Dawes Highway	73+45	U.S.G.S.	188.4	188.2
d/s Dawes Highway	72+70	U.S.G.S.	188.4	188.1
u/s Feeder Dam	12+30	U.S.G.S.	185.4	185.5
d/s Feeder Dam	10+30	U.S.G.S.	184.6	184.9

Table C-17a (con't) FLOODMARK CALIBRATION

1984 EVENT

Stream Location	Station	Agency	Recorded Elevation (ft.)	Computed Elevation (ft.)
Pompton River				
Pompton Plains Gage	350+50	U.S.G.S.	184.5	184.4
u/s Jackson Ave.	349+20	U.S.G.S.	184.4	184.3
d/s Jackson Ave.	348+40	U.S.G.S.	184.3	184.3
u/s N.J. Route 23	275+55	U.S.G.S.	182.0	182.0
d/s N.J. Route 23	274+35	U.S.G.S.	182.0	181.9
u/s Conrail R.R.	222+15	U.S.G.S.	177.9	178.2
d/s Conrail R.R.	221+95	U.S.G.S.	177.8	178.2
u/s Lackawanna R.R.	139+20	U.S.G.S.	176.7	176.6
d/s Lackawanna R.R.	138+90	U.S.G.S.	176.2	176.3
u/s Boonton Road	75+14	U.S.G.S.	173.7	173.6
d/s Boonton Road	74+70	U.S.G.S.	173.1	172.8
u/s Two Bridges	3+40	U.S.G.S.	171.9	171.2
d/s Two Bridges	3+22	U.S.G.S.	171.8	171.1
Deepavaal Brook				
Fairfield Place	127+22	U.S.G.S.	171.1	170.8
New Dutch Lane	53+90	U.S.G.S.	170.3	170.3
Little Falls Road	3+75	U.S.G.S.	170.0	170.2
Saddle River				
Lodi Gage	170+55	U.S.G.S.	34.5	34.7

Table C-17b FLOODMARK CALIBRATION

1968 EVENT

Stream Location	Station	Agency	Recorded Elevation (ft.)	Computed Elevation (ft.)
PASSAIC RIVER				
Chatham Gage	3153+95	U.S.G.S.	201.4	201.8
Bloomfield Ave.	2363+30	Passaic County	170.6	170.3
Pine Brook Gage	2346+50	U.S.G.S.	170.4	170.1
I-80 Pine Brook	2309+50	Passaic County	170.3	169.9
Horseneck Road	2198+60	Passaic County	170.1	169.4
Two Bridges	1739+90	Passaic County	168.8	169.0
I - 80 in Wayne	1717+90	Passaic County	168.5	168.6
Little Falls Gage	1538+60	U.S.G.S.	130.7	130.6
Lackawanna Ave.	1505+96	Passaic County	127.9	128.4
West Broadway	1305+26	Passaic County	45.3	47.6
5th Avenue Bridge	1143+40	Passaic County	35.0	35.9
Pequannock River				
Riverdale Road	104+45	Passaic County	188.7	189.0
Wanaque River				
Wanaque Ave.	88+00	Passaic County	201.5	201.0
Paterson-Hamburg	52+15	Passaic County	194.9	193.9
Ramapo River				
Dawes Highway	73+31	Passaic County	185.7	185.6

Table C-17b (con't) FLOODMARK CALIBRATION

1968 EVENT

Stream Location	Station	Agency	Recorded Elevation (ft.)	Computed Elevation (ft.)
Pompton River				
Pompton Plains Gage	350+50	U.S.G.S.	182.2	182.4
Rt. 23	275+55	Town of Fairfield	178.6	177.9
Two Bridges	3+40	Passaic County	168.8	169.0

Table C-17c FLOODMARK CALIBRATION

1977 EVENT

Stream	Location	Station	Agency	Recorded Elevation (ft.)	Computed Elevation (ft.)
Saddle River					
	Lodi Gage	171+30	U.S.G.S.	37.4	37.5
	Outwater Lane	167+25	U.S.G.S.	37.1	37.1
	Borig Place	150+00	U.S.G.S.	32.4	32.2
	U.S. Highway 46	140+45	U.S.G.S.	31.8	31.4
	u/s Arnot Street	119+80	U.S.G.S.	30.6	29.7
	d/s Arnot Street	119+00	U.S.G.S.	29.6	28.6
	u/s Passaic Ave.	98+60	U.S.G.S.	27.5	27.2
	d/s Passaic Ave.	98+00	U.S.G.S.	26.9	26.8
	Terrace Ave.	81+90	U.S.G.S.	25.8	25.8
	Midland Ave.	01+00	U.S.G.S.	13.0	12.8

Table C–18a 18–Aug–95

PASSAIC RIVER @ CHATHAM (ANNUAL SERIES)

STATISTICS INCLUDES
EXPECTED PROBABILITY

	THRU WATER YEAR 92	THRU WATER YEAR 93	THRU WATER YEAR 94
0.002	5130	5070	5050
0.010	3760	3730	3720
0.020	3270	3240	3240
0.050	2690	2670	2670
0.100	2290	2280	2280
0.200	1920	1910	1920
0.500	1430	1420	1430
0.990	788	790	795

MEAN	3.1685	3.1675	3.1688
STANDARD DEVIATION	0.1412	0.1403	0.1397
ADOPTED SKEW	0.6	0.6	0.6
SYSTEMATIC EVENTS	62	63	64
HISTORIC PERIOD	89	90	91

WY93 FLOW = 1260 CFS
WY94 FLOW = 1790 CFS

Table C–18b 18–Aug–95

ROCKAWAY RIVER ABOVE BOONTOWN
(ANNUAL SERIES)

STATISTICS INCLUDES
EXPECTED PROBABILITY

	THRU WATER YEAR 92	THRU WATER YEAR 93	THRU WATER YEAR 94
0.002	11700	11600	11600
0.010	8140	8060	8050
0.020	6890	6830	6810
0.050	5440	5390	5370
0.100	4470	4440	4410
0.200	3580	3550	3530
0.500	2430	2410	2390
0.990	1010	1010	992

MEAN	3.3977	3.3948	3.3904
STANDARD DEVIATION	0.1872	0.1870	0.1884
ADOPTED SKEW	0.4	0.4	0.4
SYSTEMATIC EVENTS	55	56	57
HISTORIC PERIOD	89	90	91

WY93 FLOW = 1690 CFS
WY94 FLOW = 1380 CFS

WHIPPANY RIVER @ MORRISTOWN (ANNUAL SERIES)

STATISTICS INCLUDES
EXPECTED PROBABILITY

	THRU WATER YEAR 92	THRU WATER YEAR 93	THRU WATER YEAR 94
0.002	3450	3390	3370
0.010	2690	2660	2650
0.020	2400	2380	2370
0.050	2050	2040	2030
0.100	1800	1800	1790
0.200	1550	1550	1550
0.500	1220	1210	1210
0.990	750	718	716

MEAN	3.0973	3.0937	3.0923
STANDARD DEVIATION	0.1159	0.1190	0.1189
ADOPTED SKEW	0.6	0.5	0.5
SYSTEMATIC EVENTS	71	72	73
HISTORIC PERIOD	89	90	91

WY93 FLOW = 690 CFS
WY94 FLOW = 969 CFS

Table C-18d 18-Aug-95

RAMAPO RIVER @ POMPTON LAKES

(ANNUAL SERIES)

STATISTICS INCLUDES
EXPECTED PROBABILITY

	THRU WATER YEAR 92	THRU WATER YEAR 93	THRU WATER YEAR 94
0.002	33800	33200	32800
0.010	20500	20200	20000
0.020	16200	16100	15900
0.050	11700	11600	11500
0.100	8910	8830	8760
0.200	6520	6480	6420
0.500	3780	3770	3740
0.990	1110	1120	1110

MEAN	3.5955	3.5935	3.5904
STANDARD DEVIATION	0.2640	0.2627	0.2623
ADOPTED SKEW	0.4	0.4	0.4
SYSTEMATIC EVENTS	71	72	73
HISTORIC PERIOD	89	90	91

WY93 FLOW = 2810 CFS
WY94 FLOW = 2280 CFS

Table C–18e 18–Aug–95

POMPTON RIVER @ POMPTON PLAINS
(ANNUAL SERIES)

STATISTICS INCLUDES
EXPECTED PROBABILITY

	THRU WATER YEAR 92	THRU WATER YEAR 93	THRU WATER YEAR 94
0.002	42600	41800	41400
0.010	29400	28900	28600
0.020	24600	24300	24100
0.050	19000	18900	18700
0.100	15300	15200	15000
0.200	11800	11700	11600
0.500	7300	7290	7220
0.990	2020	2050	2030

MEAN	3.8671	3.8669	3.8625
STANDARD DEVIATION	0.2397	0.2375	0.2376
ADOPTED SKEW	0.1	0.1	0.1
SYSTEMATIC EVENTS	52	53	54
HISTORIC PERIOD	89	90	91

WY93 FLOW = 7170 CFS
WY94 FLOW = 4200 CFS

PASSAIC RIVER @ LITTLE FALLS

(ANNUAL SERIES)

STATISTICS INCLUDES
EXPECTED PROBABILITY

	THRU WATER YEAR 92	THRU WATER YEAR 93	THRU WATER YEAR 94
0.002	37200	37000	36700
0.010	25700	25500	25400
0.020	21600	21600	21400
0.050	17000	17000	16900
0.100	13900	13900	13800
0.200	11100	11100	11000
0.500	7520	7530	7510
0.990	3300	3320	3320

MEAN	3.8916	3.8920	3.8909
STANDARD DEVIATION	0.1877	0.1868	0.1862
ADOPTED SKEW	0.5	0.5	0.5
SYSTEMATIC EVENTS	97	98	99
HISTORIC PERIOD	97	98	99

WY93 FLOW = 8700 CFS
WY94 FLOW = 5960 CFS

Table C-19

Statistical Parameters
Discharge-Frequency Relations
Principal Gages
Passaic River Basin, N.J.

Gaging Station	Station #	Source	# of Years	Mean	Std. Dev.	Cmptd. Skew	Gnlzd. Skew	Adptd. Skew
Passaic River at Chatham, NJ	3795	(A)	64	3.1198	0.1587	0.2757	0.4000	0.3000
		(B)	91	3.1242	0.1629	0.3326	0.4000	0.4000
		(C)	64	3.1646	0.1350	0.6940	0.4000	0.6000
		(D)	91	3.1688	0.1395	0.7451	0.4000	0.6000
Whippany River at Morristown,NJ	3815	(A)	73	2.9517	0.2001	-0.2806	0.4000	-0.1000
		(B)	91	2.9565	0.2041	-0.2165	0.4000	-0.1000
		(C)	73	3.0884	0.1138	0.3996	0.4000	0.4000
		(D)	91	3.0923	0.1189	0.5372	0.4000	0.5000
Rockaway River Above Boonton Reservoir, NJ	3805	(A)	57	3.2942	0.2023	0.2106	0.3000	0.2000
		(B)	91	3.3010	0.2105	0.3616	0.3000	0.3000
		(C)	57	3.3835	0.1782	0.1882	0.3000	0.2000
		(D)	91	3.3904	0.1884	0.4589	0.3000	0.4000
Ramapo River at Pompton Lakes, NJ	3880	(A)	73	3.5423	0.2620	0.4183	0.4000	0.4000
		(B)	91	3.5497	0.2696	0.4698	0.4000	0.4000
		(C)	73	3.5834	0.2556	0.3428	0.4000	0.4000
		(D)	91	3.5904	0.2623	0.3892	0.4000	0.4000
Pompton River at Pompton Plains, NJ	3885	(A)	54	3.7395	0.2855	0.0154	0.4000	0.1000
		(B)	91	3.7474	0.2929	0.1037	0.4000	0.2000
		(C)	54	3.8558	0.2310	-0.0946	0.4000	0.1000
		(D)	91	3.8625	0.2376	0.0224	0.4000	0.1000
Passaic River at Little Falls, NJ	3895	(A)	99	3.8613	0.2012	0.4770	0.4000	0.5000
		(C)	99	3.8909	0.1862	0.4984	0.4000	0.5000

(A) Systematic Record with No Urbanization Adjustment.
(B) Historical Adjustment with No Urbanization Adjustment.
(C) Systematic Record with Urbanization Adjustment Included.
(D) Historical Adjustment with Urbanization Adjustment Included.

Revised: 5 May 1995

Table C-20

Annual Peak Flow Frequency Curve Calculation With Urbanization Adjustment 1940 - 1964

Pompton River at Pompton Plains, NJ

Water Year	Peak Flow (cfs)	Statistical Parameters			Hypothetical Floods (cfs)				
		Mean X	Standard Deviation	Skew	2yr	10yr	50yr	100yr	500yr
1970	8520	3.8604	0.2446	0.1000	7180	15500	25700	31200	47300
1971	8610	3.8629	0.2412	0.0000	7290	15300	24500	29200	42400
1972	7240	3.8630	0.2376	0.0000	7290	15100	24000	28500	41000
1973	11500	3.8690	0.2366	0.0000	7400	15300	24200	28600	41100
1974	9730	3.8726	0.2341	0.0000	7460	15200	24000	28400	40500
1975	6150	3.8704	0.2313	0.0000	7420	15000	23500	27700	39300
1976	6670	3.8692	0.2284	0.0000	7400	14800	23100	27100	38200
1977	9000	3.8716	0.2258	0.0000	7440	14800	22900	26800	37600
1978	11800	3.8769	0.2253	-0.1000	7600	14800	22300	25900	35100
1979	9520	3.8887	0.2051	0.2000	7620	14600	22700	26800	38400
1980	12000	3.8937	0.2045	0.1000	7770	14700	22200	25900	36000
1981	4600	3.8793	0.2226	-0.1000	7640	14800	22100	25500	34400
1982	3880	3.8726	0.2245	0.0000	7460	14700	22600	26500	36800
1983	10100	3.8757	0.2228	-0.1000	7580	14700	21900	25200	33900
1984	25400	3.8876	0.2341	0.0000	7720	15700	24500	28800	40500
1985	2710	3.8777	0.2411	0.0000	7550	15700	24700	29300	41500
1986	4270	3.8725	0.2412	0.1000	7390	15600	25300	30200	44300
1987	12600	3.8773	0.2410	0.0000	7540	15600	24700	29100	41200
1988	3260	3.8699	0.2441	0.1000	7340	15600	25400	30500	44700
1989	10100	3.8727	0.2425	0.0000	7460	15500	24500	29000	41000
1990	5730	3.8705	0.2406	0.0000	7420	15400	24200	28500	40200
1991	4370	3.8661	0.2405	0.1000	7280	15300	24700	29500	42900
1992	6640	3.8671	0.2397	0.1000	7300	15300	24600	29400	42600
1993	7170	3.8669	0.2375	0.1000	7290	15200	24300	28900	41800
1994	4200	3.8625	0.2376	0.1000	7220	15000	24100	28600	41400

Table C-22

Hypothetical Rainfall Depth-Area Relationships 96-Hour Duration

Area (sq mi)	Point	0.2	20.0	50.0	100.0	1000.0
Reduction Factor	100.0%	99.9%	97.2%	95.0%	93.3%	89.0%
Frequency %	**Rainfall Depth (inches)**					
1.000	4.00	3.99	3.89	3.80	3.73	3.56
0.500	4.80	4.79	4.66	4.56	4.48	4.27
0.200	6.10	6.09	5.93	5.79	5.69	5.43
0.100	7.20	7.19	7.00	6.84	6.72	6.41
0.040	8.50	8.49	8.26	8.07	7.93	7.57
0.020	9.40	9.39	9.14	8.93	8.77	8.37
0.010	10.70	10.69	10.40	10.17	9.98	9.52
0.002	12.70	12.69	12.34	12.07	11.85	11.30

**Revised 22 October 1992

Table C-21

Hypothetical Rainfall
Point Rainfall Values

Duration (Hours)	Recurrance Interval 2 YR	5 YR	10 YR	25 YR	50 YR	100 YR	500 YR*
1.00	1.15	1.75	2.10	2.40	2.70	3.00	3.65
2.00	1.45	2.20	2.60	3.00	3.35	3.75	4.55
3.00	1.65	2.50	2.95	3.40	3.75	4.25	5.15
6.00	2.00	3.05	3.60	4.15	4.55	5.20	6.30
12.00	2.40	3.60	4.30	5.00	5.45	6.25	7.60
18.00	2.70	4.00	4.75	5.50	6.05	6.95	8.40
24.00	2.90	4.30	5.10	5.95	6.50	7.45	9.00
48.00	3.50	5.10	6.00	7.10	7.80	9.00	10.70
72.00	3.90	5.65	6.65	7.90	8.70	9.95	11.90
96.00	4.20	6.10	7.20	8.50	9.40	10.70	12.70

* 500-year storm determined by extrapolation according to procedures in Technical Paper 40 for return periods longer than 100 years.

Table C-23

- Hypothetical Rainfall - Adjustments to Rainfall

Frequency	Passaic above Chatham	Rockaway	Whippany	Ramapo	Saddle & Lower Passaic	Pequannock and Wanaque	Passaic Chatham to Dundee Dam
1.000	0%	-7%	20%	-18%	25%	-30%	-30%
0.500	0%	-10%	10%	-12%	20%	-20%	-25%
0.200	0%	-10%	0%	-12%	10%	-15%	-20%
0.100	0%	-15%	-5%	-12%	5%	-20%	-15%
0.040	0%	-15%	-10%	-3%	5%	-25%	-10%
0.020	0%	-15%	-10%	0%	5%	-25%	0%
0.010	0%	-15%	-15%	2%	5%	-25%	-5%
0.002	0%	-10%	-15%	10%	5%	-20%	10%

Revised : 15 November 1993

Table C-24

SELECTED GAGE LOCATIONS - EXISTING CONDITIONS - DISCHARGES
PASSAIC at CHATHAM, #13795 (HEC1 NODE PT = 2110)

FREQUENCY	PEAK FLOW, cfs COMPUTED	GAGE	DIFFERENCE	CONFIDENCE LIMITS .05 LIM	.95 LIM
1.000	1150	1140	0.8%		
0.500	1480	1450	2.0%	1530	1340
0.200	1980	1920	3.1%	2070	1780
0.100	2450	2280	7.5%	2500	2080
0.040	3020	2860	5.6%	3210	2530
0.020	3420	3240	5.5%	3650	2810
0.010	4020	3720	8.0%	4230	3140
0.002	4970	5050	-1.6%	5820	4010

ROCKAWAY ABOVE BOONTON, #13805 (HEC1 NODE PT = 329)

FREQUENCY	PEAK FLOW, cfs COMPUTED	GAGE	DIFFERENCE	CONFIDENCE LIMITS .05 LIM	.95 LIM
1.000	1810	1770	2.3%		
0.500	2420	2550	-5.0%	2630	2170
0.200	3780	3530	7.1%	3950	3160
0.100	4590	4410	4.0%	5030	3870
0.040	5970	5850	2.1%	6860	4930
0.020	7060	6810	3.7%	8060	5590
0.010	8670	8050	7.7%	9640	6400
0.002	12130	11600	4.5%	14100	8530

GAGE DATA THRU WY94; 1 & 2 YR ADJUSTED FOR 90 DAY SEPARATION (PARTIAL DURATION).
REVISED: 5 MAY 1995

Table C-24 (cont)

SELECTED GAGE LOCATIONS - EXISTING CONDITIONS - DISCHARGES
WHIPPANY at MORRISTOWN, #13815 (HEC1 NODE PT = 3109)

FREQUENCY	PEAK FLOW, cfs COMPUTED	GAGE	DIFFERENCE	CONFIDENCE LIMITS .05 LIM	.95 LIM
1.000	1030	1020	0.9%		
0.500	1220	1210	0.8%	1270	1150
0.200	1560	1550	0.6%	1650	1460
0.100	1880	1790	5.0%	1930	1660
0.040	2200	2180	0.9%	2340	1950
0.020	2530	2370	6.7%	2610	2130
0.010	2820	2650	6.4%	2940	2330
0.002	3640	3370	8.0%	3780	2830

RAMAPO at POMPTON LAKES, #13880 (HEC1 NODE PT = 4217R)

FREQUENCY	PEAK FLOW, cfs COMPUTED	GAGE	DIFFERENCE	CONFIDENCE LIMITS .05 LIM	.95 LIM
1.000	2610	2600	0.4%		
0.500	4300	4300	0.0%	4340	3320
0.200	6610	6420	3.0%	7380	5620
0.100	8780	8760	0.2%	10300	7450
0.040	12950	12900	0.4%	15900	10400
0.020	15450	15900	-2.8%	19600	12500
0.010	19350	20000	-3.2%	25000	15200
0.002	31700	32800	-3.4%	42000	22700

GAGE DATA THRU WY94; 1 & 2 YR ADJUSTED FOR 90 DAY SEPARATION (PARTIAL DURATION).
REVISED: 5 MAY 1995

Table C-24 (cont)

SELECTED GAGE LOCATIONS - EXISTING CONDITIONS - DISCHARGES
SADDLE @ LODI, #13915 (HEC1 NODE PT = 5176)

FREQUENCY	PEAK FLOW, cfs COMPUTED	GAGE	DIFFERENCE	CONFIDENCE LIMITS .05 LIM	.95 LIM
1.000	1510	1580	-4.4%		
0.500	1850	1800	2.8%	2000	1640
0.200	2700	2700	0.0%	3150	2460
0.100	3650	3500	4.3%	4020	3020
0.040	4670	4500	3.8%	5400	3800
0.020	5270	5370	-1.9%	6350	4410
0.010	6430	6280	2.4%	7550	5020
0.002	8760	8730	0.3%	10600	6600

POMPTON @ POMPTON PLAINS, #13885 (WITHIN UNET)

FREQUENCY	PEAK FLOW, cfs COMPUTED	GAGE	DIFFERENCE	CONFIDENCE LIMITS .05 LIM	.95 LIM
1.000	4200	4300	-2.3%		
0.500	7260	7600	-4.4%	8170	6380
0.200	11650	11600	0.4%	13500	10100
0.100	14720	15000	-1.8%	17800	12700
0.040	20160	20600	-2.1%	25100	16600
0.020	24050	24100	-0.2%	29800	19000
0.010	29730	28600	3.9%	35900	21900
0.002	42040	41400	1.6%	52800	29200

GAGE DATA THRU WY94; 1 & 2 YR ADJUSTED FOR 90 DAY SEPARATION (PARTIAL DURATION).
REVISED: 5 MAY 1995

Table C-24 (cont)

SELECTED GAGE LOCATIONS - EXISTING CONDITIONS - DISCHARGES
PASSAIC @ LITTLE FALLS, #13895 (WITHIN UNET)

	PEAK FLOW, cfs			CONFIDENCE LIMITS	
FREQUENCY	COMPUTED	GAGE	DIFFERENCE	.05 LIM	.95 LIM
1.000	5360	5100	5.1%		
0.500	7790	7800	-0.2%	8060	6980
0.200	11560	11000	5.1%	12000	10200
0.100	14350	13800	3.9%	15300	12500
0.040	18450	18400	0.2%	20800	16200
0.020	21990	21400	2.8%	24400	18500
0.010	26050	25400	2.6%	29200	21400
0.002	36280	36700	-1.1%	42900	29200

GAGE DATA THRU WY94; 1 & 2 YR ADJUSTED FOR 90 DAY SEPARATION (PARTIAL DURATION).
REVISED: 5 MAY 1995

TABLE C-25

AREAS, AREA RATIOS AND
AREA AND VOLUME CHANGES
WITH 100-YEAR FLOOD STAGE (C.O.E.) FILL
FUTURE-W/O VERSUS FILLED CONDITIONS

REACH AND X-SECT NUMBER	LENGTH OF FILL (A) (FT)	FUTURE W/O AREA (SQ FT)	FILLED AREA (SQ FT)	RATIO FILLED VERSUS FUTURE W/O	AREA CHANGE (FUTURE W/O LESS FILLED) (SQ FT)	CORRESP CHANGE IN VOL (B) (AC-FT)
01R029060	450	5981	4924	0.8233	1057	11
01R028647	697	7617	5917	0.7768	1700	27
01R027950	950	10917	9429	0.8637	1488	32
01R027000	150	12675	10205	0.8051	2470	9
01R026850	610	11783	9811	0.8326	1972	28
01R026280	140	24983	20139	0.8061	4844	16
01R024070	250	18311	15185	0.8293	3126	18
01R023680	100	21224	17694	0.8337	3530	8
01R019135	550	16623	14398	0.8661	2225	28
01R018100	100	52705	40564	0.7696	12141	28
01R018000	225	29262	25520	0.8721	3742	19
01R017215	700	60752	45811	0.7541	14941	240
01R016280	550	56813	43775	0.7705	13038	165
01R013530	130	15760	12627	0.8012	3133	9
01R013400	200	12977	11167	0.8605	1810	8
01R013165	530	10958	8087	0.7380	2871	35
TOTALS		369341	295253		74088	681
AVERAGES				0.8127		

TABLE C-25 (CONTD)

REACH AND X-SECT NUMBER	LENGTH OF FILL (A) (FT)	FUTURE W/O AREA (SQ FT)	FILLED AREA (SQ FT)	RATIO FILLED VERSUS FUTURE W/O	AREA CHANGE (FUTURE W/O LESS FILLED) (SQ FT)	CORRESP CHANGE IN VOL (B) (AC-FT)
03R007275	2700	8067	6667	0.8265	1400	87
03R004640	520	6615	5493	0.8304	1122	13
03R004080	1350	37913	33038	0.8714	4875	151
03R002750	1275	34819	28549	0.8199	6270	184
03R001535	1350	44599	37535	0.8416	7064	219
03R000150	150	1871	1524	0.8145	347	1
TOTALS		133884	112806		21078	655
AVERAGES				0.8341		
04R024185	130	31459	26802	0.8520	4657	14
04R024130	610	14266	12279	0.8607	1987	28
04R023555	510	22234	19589	0.8810	2645	31
04R023010	460	20938	17874	0.8537	3064	32
04R022090	600	12925	11289	0.8734	1636	23
04R021290	110	23188	19288	0.8318	3900	10
04R020945	600	22208	19442	0.8755	2766	38
04R020000	125	20072	16781	0.8360	3291	9
04R019215	525	7303	6015	0.8236	1288	16
04R018935	160	33498	28148	0.8403	5350	20
04R018795	825	40644	32351	0.7960	8293	157
04R017970	590	51641	42180	0.8168	9461	128
04R017375	1075	47556	37552	0.7896	10004	247
04R015980	635	62604	53751	0.8586	8853	129
04R015020	740	52128	44621	0.8560	7507	128
04R014230	170	60677	50148	0.8265	10529	41
04R014060	925	66603	54033	0.8113	12570	267
04R012980	900	71756	58838	0.8200	12918	267
04R011555	815	68097	55758	0.8188	12339	231
04R010640	530	50880	43102	0.8471	7778	95
04R009690	550	74680	60413	0.8090	14267	180
04R008765	1025	64852	54928	0.8470	9924	234
04R007080	1000	68689	54946	0.7999	13743	315
04R006090	775	73494	61147	0.8320	12347	220

TABLE C-25 (CONTD)

REACH AND X-SECT NUMBER	LENGTH OF FILL (A) (FT)	FUTURE W/O AREA (SQ FT)	FILLED AREA (SQ FT)	RATIO FILLED VERSUS FUTURE W/O	AREA CHANGE (FUTURE W/O LESS FILLED) (SQ FT)	CORRESP CHANGE IN VOL (B) (AC-FT)
04R005135	1175	75620	61693	0.8158	13927	376
04R002840	250	48400	38159	0.7884	10241	59
TOTALS		1186412	981127		205285	3293
AVERAGES				0.8331		
**********						
06R301720	760	17294	15872	0.9178	1422	25
06R300750	450	21340	16089	0.7539	5251	54
06R300020	750	20902	17796	0.8514	3106	53
06R298720	400	19824	16630	0.8389	3194	29
06R297845	500	35764	29291	0.8190	6473	74
06R296650	800	42195	36375	0.8621	5820	107
06R295200	650	38498	32723	0.8500	5775	86
06R293740	300	28482	23321	0.8188	5161	36
06R293290	280	52249	40141	0.7683	12108	78
06R292040	1700	57821	46331	0.8013	11490	448
06R286470	490	45982	38110	0.8288	7872	89
06R285190	1110	35253	28613	0.8116	6640	169
06R283580	825	42724	37521	0.8782	5203	99
06R282180	1160	33447	28109	0.8404	5338	142
06R281025	900	25931	21772	0.8396	4159	86
06R280040	990	34909	29517	0.8455	5392	123
06R278390	500	20252	16153	0.7976	4099	47
06R276320	680	11206	9088	0.8110	2118	33
06R271405	1080	22469	17616	0.7840	4853	120
06R270395	940	17226	13449	0.7807	3777	82
06R269410	950	17581	14571	0.8288	3010	66
06R265590	600	11682	10069	0.8619	1613	22
06R265075	195	13097	11085	0.8464	2012	9
06R264880	470	12992	10700	0.8236	2292	25
06R264410	925	38469	32804	0.8527	5665	120
06R263170	360	33784	28113	0.8321	5671	47
06R262740	810	27944	23652	0.8464	4292	80
06R261880	325	20975	16416	0.7826	4559	34
06R261250	430	16874	13634	0.8080	3240	32
06R260870	800	18299	15420	0.8427	2879	53

TABLE C-25 (CONTD)

REACH AND X-SECT NUMBER	LENGTH OF FILL (A) (FT)	FUTURE W/O AREA (SQ FT)	FILLED AREA (SQ FT)	RATIO FILLED VERSUS FUTURE W/O	AREA CHANGE (FUTURE W/O LESS FILLED) (SQ FT)	CORRESP CHANGE IN VOL (B) (AC-FT)
06R260190	600	18638	15523	0.8329	3115	43
06R259185	720	16380	13320	0.8132	3060	51
06R254485	490	14794	12376	0.8366	2418	27
06R253815	1400	16658	12704	0.7626	3954	127
06R252535	110	38590	31114	0.8063	7476	19
06R252360	430	28397	23604	0.8312	4793	47
06R251930	620	47050	38893	0.8266	8157	116
06R250650	590	102310	86384	0.8443	15926	216
06R249995	410	32552	26379	0.8104	6173	58
06R249565	3100	48665	41637	0.8556	7028	500
06R242000	1620	114269	94915	0.8306	19354	720
TOTALS		1313768	1087830		225938	4391
AVERAGES				0.8262		
**********						
07R208400	6000	69163	54394	0.7865	14769	2034
07R199400	5400	82132	68386	0.8326	13746	1704
07R194000	1060	89194	72907	0.8174	16287	396
07R190700	3100	82002	64567	0.7874	17435	1241
07R187560	140	73341	62314	0.8496	11027	35
07R187420	600	78888	65759	0.8336	13129	181
07R184500	100	69369	58822	0.8480	10547	24
07R184170	3400	71608	59750	0.8344	11858	926
TOTALS		615697	506899		108798	6542
AVERAGES				0.8237		
**********						
10R007134	220	14570	12279	0.8428	2291	12
10R006240	95	13396	11270	0.8413	2126	5
10R005765	565	18085	14973	0.8279	3112	40

TABLE C-25 (CONTD)

REACH AND X-SECT NUMBER	LENGTH OF FILL (A) (FT)	FUTURE W/O AREA (SQ FT)	FILLED AREA (SQ FT)	RATIO FILLED VERSUS FUTURE W/O	AREA CHANGE (FUTURE W/O LESS FILLED) (SQ FT)	CORRESP CHANGE IN VOL (B) (AC-FT)
TOTALS		46051	38522		7529	57
AVERAGES				0.8373		
**********						
11R007230	300	19812	18315	0.9244	1497	10
11R007010	290	24035	21729	0.9041	2306	15
11R006630	445	22212	17208	0.7747	5004	51
11R006240	520	21366	16827	0.7876	4539	54
11R005730	305	23151	19177	0.8283	3974	28
11R005465	305	20416	17245	0.8447	3171	22
TOTALS		130992	110501		20491	181
AVERAGES				0.8440		
**********						
14R026700	170	23695	18828	0.7946	4867	19
14R026530	200	32727	27059	0.8268	5668	26
14R026280	815	30729	23359	0.7602	7370	138
14R024110	435	59404	48716	0.8201	10688	107
14R023675	560	77273	59928	0.7755	17345	223
14R023035	580	82873	64506	0.7784	18367	245
14R022445	165	97291	77186	0.7934	20105	76
14R021810	350	93335	71376	0.7647	21959	176
14R021500	790	52812	41667	0.7890	11145	202
14R019190	415	101356	86832	0.8567	14524	138
14R018185	650	76731	59553	0.7761	17178	256
14R016480	350	71741	58309	0.8128	13432	108
14R016220	600	68266	56510	0.8278	11756	162
14R015710	250	82884	67874	0.8189	15010	86
14R014880	250	87596	71142	0.8122	16454	94
14R009660	110	38378	32749	0.8533	5629	14
14R009320	515	37396	28557	0.7636	8839	105
14R008805	150	30488	24431	0.8013	6057	21
14R008650	160	23432	18633	0.7952	4799	18
14R008510	660	37988	30924	0.8140	7064	107

TABLE C-25 (CONTD)

REACH AND X-SECT NUMBER	LENGTH OF FILL (A) (FT)	FUTURE W/O AREA (SQ FT)	FILLED AREA (SQ FT)	RATIO FILLED VERSUS FUTURE W/O	AREA CHANGE (FUTURE W/O LESS FILLED) (SQ FT)	CORRESP CHANGE IN VOL (B) (AC-FT)
14R007940	300	65375	54747	0.8374	10628	73
14R005540	240	45593	37691	0.8267	7902	44
14R005180	160	32520	24837	0.7637	7683	28
14R004935	300	32720	24208	0.7399	8512	59
14R004590	180	42041	32988	0.7847	9053	37
14R004130	130	39666	30195	0.7612	9471	28
14R003945	110	39666	29714	0.7491	9952	25
14R003840	670	48966	38394	0.7841	10572	163
14R002950	170	131617	104075	0.7907	27542	107
14R002810	355	51400	37308	0.7258	14092	115
14R002420	115	45833	37863	0.8261	7970	21
TOTALS		1781792	1420159		361633	3022
AVERAGES				0.7943		
**********						
15R172140	140	25332	20934	0.8264	4398	14
TOTALS		25332	20934		4398	14
AVERAGES				0.8264		
**********						
16R017780	150	9963	8764	0.8797	1199	4
16R017620	780	13087	11069	0.8458	2018	36
16R016833	680	15833	13495	0.8523	2338	36
16R016170	580	12274	9970	0.8123	2304	31
16R015595	640	18470	15371	0.8322	3099	46
16R014955	605	14025	9938	0.7086	4087	57
16R014345	915	7217	5581	0.7733	1636	34
16R013425	620	15138	12966	0.8565	2172	31
16R012330	580	13901	12279	0.8833	1622	22
16R011705	810	10431	9121	0.8744	1310	24
16R010900	470	8298	6955	0.8382	1343	14
16R010430	560	10749	9184	0.8544	1565	20

TABLE C-25 (CONTD)

REACH AND X-SECT NUMBER	LENGTH OF FILL (A) (FT)	FUTURE W/O AREA (SQ FT)	FILLED AREA (SQ FT)	RATIO FILLED VERSUS FUTURE W/O	AREA CHANGE (FUTURE W/O LESS FILLED) (SQ FT)	CORRESP CHANGE IN VOL (B) (AC-FT)
16R006050	430	6934	5617	0.8101	1317	13
TOTALS AVERAGES		156320	130310	0.8324	26010	369

****

OVERALL RATIO
NET VS GROSS AREAS 0.8264

TOTAL VOLUME CHANGE (FILL)
ACRE-FT 19203

****
****

(A) ALL FILL WAS CONCEPTUALLY PLACED ON EITHER LEFT OR RIGHT OVERBANK OF CROSS-SECTION.
(B) INDIVIDUAL SECTION ENTRIES COMPUTED FROM ELEMENT LENGTH AND AREA DIFFERENCE.

TABLE C-26

COMPARISON OF AREAS AND VOLUMES
100-YEAR FLOOD STAGE (C.O.E.)
FUTURE W/O, FILLED AND FILLED-AND-CUT CONDITIONS

REACH AND X-SECT NUMBER	AREA			VOLUME (A)			VOL LOSS BY INDIV SECT (B)	
	FUTURE W/O (SQ FT)	FILL ONLY (SQ FT)	FILL AND CUT (SQ FT)	FUTURE W/O (AC-FT)	FILL ONLY (AC-FT)	FILL AND CUT (AC-FT)	WITH FILL ALONE (C) (AC-FT)	WITH FILL AND CUT (AC-FT)
01R029060	5981	4924	7428	61	50	76	11	-15
01R028647	7617	5917	8082	122	94	128	28	-6
01R027950	10917	9429	12919	239	207	285	32	-46
01R027000	12675	10205	12450	44	35	43	9	1
01R026850	11783	9811	12188	156	129	159	27	-3
01R026280	24983	20139	24571	80	65	79	15	1
01R024070	18311	15185	18527	134	116	142	18	-8
01R023680	21224	17694	21935	70	62	78	8	-8
01R019135	16623	14398	17940	251	223	279	28	-28
01R018100	52705	40564	49126	121	93	113	28	8
01R018000	29262	25520	31368	193	174	214	19	-21
01R017215	60752	45811	55536	464	224	277	240	187
01R016280	56813	43775	54066	332	168	200	164	132
01R013530	15760	12627	15708	47	38	47	9	0
01R013400	12977	11167	14585	64	56	74	8	-10
01R013165	10958	8087	10257	132	97	123	35	9
SUBTOTALS	369341	295253	366686	2510	1831	2317	679	193
03R007275	8067	6667	8091	495	408	494	87	1
03R004640	6615	5493	6601	80	66	80	14	0
03R004080	37913	33038	35945	1171	1020	1109	151	62
03R002750	34819	28549	31688	1013	830	920	183	93
03R001535	44599	37535	43920	1385	1166	1366	219	19
03R000150	1871	1524	2106	6	5	7	1	-1
SUBTOTALS	133884	112806	128351	4150	3495	3976	655	174
04R024185	31459	26802	31409	92	78	92	14	0
04R024130	14266	12279	14912	199	171	208	28	-9

TABLE C-26 (CONTD)

REACH AND X-SECT NUMBER	AREA			VOLUME (A)			VOL LOSS BY INDIV SECT (B)	
	FUTURE W/O (SQ FT)	FILL ONLY (SQ FT)	FILL AND CUT (SQ FT)	FUTURE W/O (AC-FT)	FILL ONLY (AC-FT)	FILL AND CUT (AC-FT)	WITH FILL ALONE (C) (AC-FT)	WITH FILL AND CUT (AC-FT)
04R023555	22234	19589	22976	261	230	270	31	-9
04R023010	20938	17874	20900	225	192	225	33	0
04R022090	12925	11289	13440	184	161	188	23	-4
04R021290	23188	19288	21551	64	54	58	10	6
04R020945	22208	19442	22908	320	282	332	38	-12
04R020000	20072	18781	19561	58	48	56	10	2
04R019215	7303	6015	8067	77	61	74	16	3
04R018935	33498	28148	32082	121	102	115	19	6
04R018795	40644	32351	39882	769	612	755	157	14
04R017970	51641	42180	49707	697	569	671	128	26
04R017375	47556	37552	43120	1196	949	1086	247	110
04R015980	62604	53751	58299	918	789	864	129	54
04R015020	52128	44621	55968	882	754	947	128	-65
04R014230	60677	50148	60248	236	194	233	42	3
04R014060	66603	54033	68898	1420	1153	1469	267	-49
04R012980	71756	58838	70438	1490	1223	1461	267	29
04R011555	68097	55758	64711	1261	1030	1192	231	69
04R010640	50880	43102	55770	629	535	690	94	-61
04R009690	74680	60413	73574	1003	823	1003	180	0
04R008765	64852	54928	72249	1503	1270	1654	233	-151
04R007080	68689	54946	70861	1520	1204	1527	316	-7
04R006090	73494	61147	78416	1311	1092	1402	219	-91
04R005135	75620	61693	80823	2095	1720	2270	375	-175
04R002840	48400	38159	45284	304	245	291	59	13
SUBTOTALS	1186412	983127	1196054	18835	15541	19133	3294	-299
06R301720	17294	15872	19505	369	344	419	25	-50
06R300750	21340	16089	20358	223	168	209	55	14
06R300020	20902	17796	22491	282	228	285	54	-3
06R298720	19824	16630	20473	191	162	190	29	1
06R297845	35764	29291	37393	456	382	483	74	-27
06R296650	42195	36375	43287	879	772	971	107	-92
06R295200	38498	32723	38553	577	490	569	87	8
06R293740	28482	23321	31629	193	157	192	36	1
06R293290	52249	40141	49376	437	359	445	78	-8
06R292040	57821	46331	60570	2023	1575	1954	448	69
06R286470	45982	38110	43696	658	570	651	88	7
06R285190	35253	28613	34842	871	701	848	170	23

TABLE C-26 (CONTD)

REACH AND X-SECT NUMBER	AREA			VOLUME (A)			VOL LOSS BY INDIV SECT (B)	
	FUTURE W/O (SQ FT)	FILL ONLY (SQ FT)	FILL AND CUT (SQ FT)	FUTURE W/O (AC-FT)	FILL ONLY (AC-FT)	FILL AND CUT (AC-FT)	WITH FILL ALONE (C) (AC-FT)	WITH FILL AND CUT (AC-FT)
06R283580	42724	37521	45606	866	767	922	99	-56
06R282180	33447	28109	39379	866	723	1003	143	-137
06R281025	25931	21772	25426	544	458	534	86	10
06R280040	34909	29517	35437	828	706	838	122	-10
06R278390	20252	16153	20197	272	225	282	47	-10
06R276320	11206	9088	11443	190	157	197	33	-7
06R271405	22469	17616	21907	510	390	484	120	26
06R270395	17226	13449	17127	367	285	361	82	6
06R269410	17581	14571	16941	329	263	303	66	26
06R265590	11682	10069	12243	154	132	161	22	-7
06R265075	13097	11085	13627	78	69	91	9	-13
06R264880	12992	10700	13163	143	118	155	25	-12
06R264410	38469	32804	39156	937	817	972	120	-35
06R263170	33784	28113	35087	302	253	317	49	-15
06R262740	27944	23652	28626	528	448	544	80	-16
06R261880	20975	16416	20776	183	149	196	34	-13
06R261250	16874	13634	15940	167	135	158	32	9
06R260870	18299	15420	19787	328	275	357	53	-29
06R260190	18638	15523	19037	275	232	283	43	-8
06R259185	16380	13320	16296	345	295	359	50	-14
06R254485	14794	12376	15956	199	172	223	27	-24
06R253815	16658	12704	15392	416	289	347	127	69
06R252535	38590	31114	36859	111	92	106	19	5
06R252360	28397	23604	27813	280	233	275	47	5
06R251930	47050	38893	42619	967	851	918	116	49
06R250650	102310	86384	105802	1476	1260	1565	216	-89
06R249995	32552	26379	30372	329	271	312	58	17
06R249565	48665	41637	49206	3088	2588	3058	500	30
06R242000	114269	94915	115116	4250	3530	4281	720	-31
SUBTOTALS	1313768	1087830	1328509	26487	22091	26818	4396	-331
07R208400	69163	54394	66989	9667	7633	9347	2034	320
07R199400	82132	68386	82747	10182	8478	10258	1704	-76
07R194000	89194	72907	87796	2665	2269	2684	396	-19
07R190700	82002	64567	78740	5165	3925	4795	1240	370
07R187560	73341	62314	72041	236	200	232	36	4
07R187420	78888	65759	75330	2425	2244	2492	181	-67
07R184500	69369	58822	68910	284	260	316	24	-32

TABLE C-26 (CONTD)

REACH AND X-SECT NUMBER	AREA			VOLUME (A)			VOL LOSS BY INDIV SECT (B)	
	FUTURE W/O (SQ FT)	FILL ONLY (SQ FT)	FILL AND CUT (SQ FT)	FUTURE W/O (AC-FT)	FILL ONLY (AC-FT)	FILL AND CUT (AC-FT)	WITH FILL ALONE (C) (AC-FT)	WITH FILL AND CUT (AC-FT)
07R184170	71608	59750	69637	4301	3376	3832	925	469
SUBTOTALS	615697	506899	602190	34925	28385	33956	6540	969
10R007134	14570	12279	15206	106	95	141	11	-35
10R006240	13396	11270	15505	48	43	65	5	-17
10R005765	18085	14973	18729	259	219	269	40	-10
SUBTOTALS	46051	38522	49440	413	357	475	56	-62
11R007230	19812	18315	23138	73	63	80	10	-7
11R007010	24035	21729	26781	180	164	200	16	-20
11R006630	22212	17208	21982	206	155	197	51	9
11R006240	21366	16827	22081	248	194	254	54	-6
11R005730	23151	19177	23632	139	111	137	28	2
11R005465	20416	17245	22795	146	124	163	22	-17
SUBTOTALS	130992	110501	140409	992	811	1031	181	-39
14R026700	23695	18828	25031	92	73	98	19	-6
14R026530	32727	27059	34742	155	129	164	26	-9
14R026280	30729	23359	29549	561	424	539	137	22
14R024110	59404	48716	58526	593	486	584	107	9
14R023675	77273	59928	67603	997	774	874	223	123
14R023035	82873	64506	75808	1107	863	1019	244	88
14R022445	97291	77186	97129	369	292	368	77	1
14R021810	93335	71376	89900	742	566	706	176	36
14R021500	52812	41667	52305	952	750	940	202	12
14R019190	101356	86832	106477	1924	1786	2211	138	-287
14R018185	76731	59553	74212	1275	1018	1266	257	9
14R016480	71741	58309	72023	433	325	377	108	56
14R016220	68266	56510	71493	840	678	850	162	-10
14R015710	82884	67874	81763	747	661	778	86	-31
14R014880	87596	71142	86517	499	404	486	95	13
14R009660	38378	32749	39862	99	85	103	14	-4

TABLE C-26 (CONTD)

REACH AND X-SECT NUMBER	AREA			VOLUME (A)			VOL LOSS BY INDIV SECT (B)	
	FUTURE W/O (SQ FT)	FILL ONLY (SQ FT)	FILL AND CUT (SQ FT)	FUTURE W/O (AC-FT)	FILL ONLY (AC-FT)	FILL AND CUT (AC-FT)	WITH FILL ALONE (C) (AC-FT)	WITH FILL AND CUT (AC-FT)
14R009320	37396	28557	34068	442	338	403	104	39
14R008805	30488	24431	30754	104	83	105	21	-1
14R008650	23432	18633	22419	85	68	81	17	4
14R008510	37988	30924	34324	568	461	512	107	56
14R007940	65375	54747	63643	437	364	412	73	25
14R005540	45593	37691	43708	330	286	331	44	-1
14R005180	32520	24837	29419	153	125	153	28	0
14R004935	32720	24208	30949	234	176	226	58	8
14R004590	42041	32988	41941	234	196	284	38	-50
14R004130	39666	30195	37445	122	94	116	28	6
14R003945	39666	29714	36293	100	75	91	25	9
14R003840	48966	38394	43899	772	610	694	162	78
14R002950	131617	104075	125744	511	403	488	108	23
14R002810	51400	37308	44661	412	297	338	115	74
14R002420	45833	37863	43811	123	102	115	21	8
SUBTOTALS	1781792	1420159	1726018	16012	12992	15712	3020	300
15R172140	25332	20934	25475	81	67	82	14	-1
SUBTOTALS	25332	20934	25475	81	67	82	14	-1
16R017780	9963	8764	10365	38	34	40	4	-2
16R017620	13087	11069	12994	237	201	235	36	2
16R016833	15833	13495	16074	246	209	249	37	-3
16R016170	12274	9970	12197	160	129	158	31	2
16R015595	18470	15371	17436	271	226	256	45	15
16R014955	14025	9938	11481	197	140	162	57	35
16R014345	7217	5581	6782	151	116	141	35	10
16R013425	15138	12966	15729	216	185	225	31	-9
16R012330	13901	12279	14587	197	176	209	21	-12
16R011705	10431	9121	10754	182	158	187	24	-5
16R010900	8298	6955	8614	88	74	91	14	-3
16R010430	10749	9184	10551	113	92	105	21	8
16R006050	6934	5617	7454	69	56	73	13	-4
SUBTOTALS	156320	130310	155018	2165	1796	2131	369	34

TABLE C-26 (CONTD)

REACH AND X-SECT NUMBER	AREA			VOLUME (A)			VOL LOSS BY INDIV SECT (B)	
	FUTURE W/O (SQ FT)	FILL ONLY (SQ FT)	FILL AND CUT (SQ FT)	FUTURE W/O (AC-FT)	FILL ONLY (AC-FT)	FILL AND CUT (AC-FT)	WITH FILL ALONE (C) (AC-FT)	WITH FILL AND CUT (AC-FT)
TOTALS	5759589	4706341	5718150	106569	87366	105631	19203	938
LOSS (D)		1053248	41439		19203	938		
RATIO (E)		0.817	0.993		0.820	0.991		

(A) THE VOLUMES FOR EACH CONDITION ARE THE SUMS OF VOLUMES COMPUTED SEPARATELY FOR LEFT OVERBANK, CHANNEL AND RIGHT OVERBANK, THE LENGTHS OF THESE ELEMENTS FREQUENTLY BEING DIFFERENT.

(B) IF ENTRY IS NEGATIVE, CUTTING OVERCOMPENSATED FOR FILLING.

(C) ENTRIES FOR INDIVIDUAL SECTIONS MAY DIFFER FROM CORRESPONDING ONES IN TABLE QQQ BY 1 OR 2 ACRE FEET.

(D) FUTURE-W/O TOTAL MINUS COLUMN TOTAL. IF FILL-AND-CUT COLUMN ENTRY IS POSITIVE, CUTTING DID NOT TOTALLY OFFSET FILLING.

(E) COLUMN TOTAL DIVIDED BY FUTURE-W/O TOTAL. IF FILL-AND-CUT COLUMN ENTRY IS LESS THAN 1.000, CUTTING DID NOT TOTALLY OFFSET FILLING.

TABLE C-27

RESIDUAL STORAGE LOSSES
FINAL "UNET" MODEL
(REACH-BY-REACH AND OVERALL)

REACH	NUMBER OF SECTIONS INVEST	PRE-CUT AND FILL VOLUME (AC-FT)	POST-CUT AND FILL VOLUME (AC-FT)	NET VOLUME LOSS (A) (AC-FT)	NET VOLUME LOSS (A) (PCT)	RATIO POST/PRE VOLUMES
01	16	2510	2317	193	7.69	0.923
03	6	4150	3976	174	4.19	0.958
04 (B)	26	18835	19133	-299	- 1.58	1.016
06 (C)	41	26487	26818	-331	- 1.25	1.012
07 (D)	8	34925	33956	969	2.77	0.972
10	3	413	475	- 62	-15.01	1.150
11	6	992	1031	- 39	- 3.93	1.039
14	31	16012	15712	300	1.87	0.981
15	1	81	82	-1	1.23	1.012
16	13	2165	2131	34	1.57	0.984
ALL		106569	105631	938	0.88	0.991

(A) A NEGATIVE ENTRY INDICATES THAT CUTTING OVER-COMPENSATED FOR FILLING.
(B) INCLUDES TROY MEADOWS.
(C) INCLUDES MUCH OF HATFIELD SWAMP.
(D) INCLUDES GREAT PIECE MEADOW.

TABLE C-28

STAGE IMPACTS
FILL ONLY AND FILL PLUS CUT
ALL FREQUENCIES

X-sect and/or Location	Frequency	Scenario		
		No Work	Fill Only	Fill /Cut
	(yrs)	(NGVD)	(NGVD)	(NGVD)
03R000150 Pinch-Whip Confl	500	184.38	184.70	184.20
	100	182.43	182.76	182.27
	50	181.82	182.18	181.59
	25	181.03	181.27	180.69
	10	180.25	180.48	179.83
	5	179.35	179.60	178.77
	2	178.30	178.44	177.59
	1	177.71	177.82	176.95
04R000215 Whip-Rock Confl	500	178.02	178.43	178.22
	100	175.43	175.89	175.64
	50	174.39	174.83	174.58
	25	173.34	173.70	173.47
	10	172.05	172.34	172.09
	5	171.07	171.29	171.04
	2	169.76	169.88	169.68
	1	169.00	169.07	168.87
05R000210 Rock-Pass Confl	500	178.00	178.41	178.20
	100	175.40	175.86	175.61
	50	174.36	174.80	174.54
	25	173.30	173.66	173.42
	10	172.00	172.28	172.03
	5	170.99	171.21	170.96
	2	169.65	169.77	169.55
	1	168.85	168.92	168.69
14R016710 Beav-Pomp Confl	500	181.22	181.00	180.99
	100	178.84	179.11	178.70
	50	177.39	177.71	177.20
	25	176.26	176.54	176.01
	10	174.36	174.60	173.70
	5	172.94	173.05	172.24
	2	170.61	170.72	169.87
	1	168.35	168.43	167.58

TABLE C-28 (CONTD)

STAGE IMPACTS
FILL ONLY AND FILL PLUS CUT
ALL FREQUENCIES

X-sect and/or Location	Frequency	Scenario		
		No Work	Fill Only	Fill /Cut
	(yrs)	(NGVD)	(NGVD)	(NGVD)
14R000222 (A)	500	176.07	176.47	176.28
Pomp-Pass Confl	100	173.51	173.97	173.72
	50	172.33	172.76	172.54
	25	171.19	171.56	171.35
	10	169.62	169.94	169.68
	5	168.41	168.68	168.40
	2	166.37	166.56	166.27
	1	164.73	164.85	164.55
17R153860	500	138.02	138.55	138.27
Pass @ Lttl Flls	100	135.37	135.85	135.61
	50	134.13	134.57	134.35
	25	133.01	133.39	133.18
	10	131.43	131.78	131.50
	5	130.19	130.47	130.18
	2	128.15	128.34	128.04
	1	126.53	126.65	126.35

(A) Stages are identical to those at cross-section 07R173860 in Great Piece Meadows.

TABLE C-29

PHRAGMITES STAGE IMPACTS
VARIOUS MAT THICKNESSES
ALL FREQUENCIES

X-sect and/or Location	Frequency	Root-mat Thickness			
		No Mat	1-foot	2-feet	3-feet
	(yrs)	(NGVD)	(NGVD)	(NGVD)	(NGVD)
03R000150	500	184.38			184.63
Pinch-Whip Confl	100	182.43	182.48	182.57	182.69
	50	181.82			182.11
	25	181.03			181.26
	10	180.25			180.50
	5	179.35			179.69
	2	178.30			178.59
	1	177.71			177.99
04R000215	500	178.02			178.21
Whip-Rock Confl	100	175.43	175.48	175.53	175.65
	50	174.39			174.63
	25	173.34			173.60
	10	172.05			172.34
	5	171.07			171.40
	2	169.76			170.11
	1	169.00			169.36
05R000210	500	178.00			178.18
Rock-Pass Confl	100	175.40	175.45	175.50	175.62
	50	174.36			174.59
	25	173.30			173.55
	10	172.00			172.28
	5	170.99			171.32
	2	169.65			169.99
	1	168.85			169.19
14R016710	500	181.22			181.24
Beav-Pomp Confl	100	178.84	178.85	178.85	178.86
	50	177.39			177.42
	25	176.26			176.29
	10	174.37			174.45
	5	172.94			172.98
	2	170.61			170.64
	1	168.35			168.39

TABLE C-29 (CONTD)

PHRAGMITES STAGE IMPACTS
VARIOUS MAT THICKNESSES
ALL FREQUENCIES

X-sect and/or Location	Frequency	Root-mat Thickness			
		No Mat	1-foot	2-feet	3-feet
	(yrs)	(NGVD)	(NGVD)	(NGVD)	(NGVD)
14R000222 (A)	500	176.07			176.27
Pomp-Pass Confl	100	173.51	173.55	173.62	173.73
	50	172.33			172.56
	25	171.19			171.43
	10	169.62			169.88
	5	168.41			168.70
	2	166.37			166.69
	1	164.73			164.96
17R153860	500	138.02			138.25
Pass @ Lttl Flls	100	135.37	135.41	135.49	135.60
	50	134.13			134.36
	25	133.01			133.25
	10	131.43			131.71
	5	130.19			130.48
	2	128.15			128.47
	1	126.53			126.76

(A) Stages are identical to those at cross-section 07R173860 in Great Piece Meadows.

Table C-30a
Existing Conditions - Water Surface Elevations

X-Sect	Distance in 1000's of Feet	1 Year ELEV FEET	2 Year ELEV FEET	5 Year ELEV FEET	10 Year ELEV FEET	25 Year ELEV FEET	50 Year ELEV FEET	100 Yr ELEV FEET	500 Yr ELEV FEET
PASSAIC RIVER - Unet Reach 6									
1	315.800	199.9	200.5	201.3	201.9	202.6	203.0	203.6	204.5
9	314.400	199.0	199.5	200.1	200.6	201.1	201.4	201.8	202.5
85	262.740	169.4	170.3	171.7	172.8	174.1	175.0	175.9	178.3
87	261.250	169.4	170.3	171.6	172.8	174.1	175.0	175.9	178.3
PASSAIC2 RIVER - Unet Reach 7									
2	240.300	168.5	169.4	170.7	171.7	173.0	174.0	175.1	177.7
6	235.585	167.7	168.4	169.7	170.6	171.9	172.9	174.0	176.6
11	234.650	167.7	168.4	169.6	170.5	171.8	172.8	173.8	176.3
58	175.810	164.5	166.1	168.1	169.4	171.0	172.1	173.3	175.9
60	174.630	164.5	166.1	168.1	169.4	171.0	172.1	173.3	175.9
61	174.370	164.5	166.1	168.1	169.4	170.9	172.1	173.3	175.9
67	173.860	164.5	166.1	168.1	169.3	170.9	172.1	173.2	175.8
PASSAIC3 RIVER - Unet Reach 15									
1	173.500	164.5	166.1	168.1	169.3	170.9	172.1	173.2	175.8
4	172.000	164.2	165.8	167.8	169.0	170.6	171.7	172.9	175.5
15	170.090	163.9	165.5	167.4	168.5	170.0	171.2	172.4	175.0
PASSAIC4 RIVER - Unet Reach 17									
5	167.000	163.5	165.0	166.8	167.9	169.3	170.4	171.6	174.1
21	162.290	163.0	164.2	165.8	166.8	168.0	169.0	169.9	172.1
31	157.600	161.2	162.0	163.1	163.9	165.0	166.0	166.9	169.1
35	156.870	161.0	161.6	162.5	163.1	164.0	164.9	165.6	167.6
36	155.850	126.8	127.8	127.8	127.9	128.0	128.1	128.2	129.2
42	153.860	126.3	127.9	129.9	131.1	132.8	133.9	135.1	137.8
47	152.000	123.8	125.6	127.8	129.2	130.9	132.1	133.3	136.1
74	143.830	119.7	121.3	123.4	124.8	126.7	128.1	129.5	132.3
93	133.870	117.9	118.8	120.1	120.9	121.8	122.7	123.6	125.6
94	133.520	50.6	52.9	56.0	58.2	61.1	63.4	65.3	69.4
96	133.350	50.5	52.9	56.0	58.2	61.3	63.6	65.6	69.9
117	128.000	38.2	40.1	42.4	43.8	45.5	46.8	48.9	53.3
136	119.700	34.3	35.7	37.7	39.0	40.6	41.9	43.3	47.7
143	117.300	33.5	34.8	36.7	37.9	39.5	40.8	42.1	45.9
153	111.860	30.4	31.8	33.7	34.9	36.5	37.9	39.3	41.8
158	108.900	29.7	31.1	32.8	34.0	35.4	36.5	37.6	40.1
163	105.950	29.3	30.5	32.1	33.2	34.5	35.6	36.6	39.0
174	99.760	28.3	29.2	30.5	31.2	32.2	33.0	33.7	35.4
184	95.300	27.8	28.6	29.5	30.1	30.9	31.6	32.3	33.9

Table C-30a (Continued)
Existing Conditions - Water Surface Elevations

X-Sect	Distance in 1000's of Feet	1 Year ELEV FEET	2 Year ELEV FEET	5 Year ELEV FEET	10 Year ELEV FEET	25 Year ELEV FEET	50 Year ELEV FEET	100 Yr ELEV FEET	500 Yr ELEV FEET
PASSAIC4 RIVER (Continued)									
191	91.790	27.7	28.5	29.4	30.0	30.7	31.3	31.9	33.4
192	91.790	10.1	11.5	13.5	14.6	16.2	17.6	20.2	24.5
202	89.100	8.9	10.5	12.7	13.7	15.3	16.6	19.4	22.9
215	83.800	7.1	8.0	9.6	10.8	12.7	14.2	15.7	18.8
PASSAIC5 RIVER - Unet Reach 19									
1	82.200	6.9	7.9	9.3	10.5	12.4	13.8	15.2	18.3
7	78.200	6.5	7.2	8.4	9.4	11.1	12.6	14.1	17.4
32	61.030	6.0	6.2	6.7	7.3	8.5	9.6	10.7	13.3
102	15.200	5.7	5.7	5.7	5.8	5.8	5.8	5.9	6.1
118	3.200	5.7	5.7	5.7	5.7	5.7	5.7	5.7	5.7
SADDLE RIVER - Unet Reach 18									
1	17.130	30.4	31.9	33.8	35.3	36.9	37.5	38.6	40.0
26	11.940	20.6	22.5	24.6	26.0	27.7	28.7	30.2	32.4
76	2.895	10.6	12.4	14.3	15.6	17.4	18.4	20.0	22.7
DEEPAVAAL BROOK - Unet Reach 16									
4	18.625	168.6	169.1	169.6	170.2	170.7	172.1	173.1	175.3
13	13.425	168.4	169.0	169.6	170.1	170.6	172.0	173.0	175.2
17	10.900	167.9	168.6	169.3	169.9	170.4	171.7	172.8	175.2
30	6.050	164.3	165.6	167.4	168.6	170.1	171.3	172.5	175.1
42	3.220	164.0	165.5	167.4	168.5	170.0	171.2	172.4	175.1
58	0.720	163.9	165.5	167.4	168.5	170.0	171.2	172.4	175.0
64	0.340	163.9	165.5	167.4	168.5	170.0	171.2	172.4	175.0
65	0.180	163.9	165.5	167.4	168.5	170.0	171.2	172.4	175.0
POMPTON RIVER - Unet Reach 14									
2	35.775	174.7	177.6	180.7	182.0	183.5	184.3	185.4	187.2
3	35.090	174.3	177.3	180.4	181.7	183.2	184.1	185.1	186.9
8	34.840	174.3	177.2	180.3	181.6	183.1	183.9	185.0	186.7
18	30.350	171.6	174.4	177.3	179.3	181.5	182.5	183.5	185.2
29	27.030	170.1	172.9	175.8	177.9	180.0	181.1	182.2	183.9
50	21.810	169.0	171.7	174.4	175.8	177.3	178.1	179.3	181.6
62	14.880	167.2	169.4	171.5	172.9	175.5	176.9	178.4	180.8
76	9.940	165.9	168.0	170.2	171.9	174.1	175.5	176.7	179.5
86	7.514	165.2	167.0	169.1	170.8	172.8	173.9	174.9	177.1
97	2.950	164.7	166.3	168.4	169.6	171.3	172.6	173.8	176.3

Table C-30a (Continued)
Existing Conditions - Water Surface Elevations

X-Sect	Distance in 1000's of Feet	1 Year ELEV FEET	2 Year ELEV FEET	5 Year ELEV FEET	10 Year ELEV FEET	25 Year ELEV FEET	50 Year ELEV FEET	100 Yr ELEV FEET	500 Yr ELEV FEET
POMPTON RIVER (Continued)									
101	1.835	164.6	166.2	168.3	169.5	171.2	172.4	173.6	176.1
106	0.480	164.4	166.1	168.1	169.3	170.9	172.1	173.3	175.9
BYPASS - Unet Reach 13									
1	1.500	176.0	178.3	181.0	182.2	183.8	184.7	185.8	187.8
4	0.300	175.1	177.9	180.8	182.1	183.7	184.6	185.6	187.6
5	0.050	174.9	177.7	180.8	182.1	183.7	184.5	185.6	187.5
RAMAPO RIVER - Unet Reach 11									
5	9.410	180.8	182.9	185.2	186.8	189.2	190.3	191.4	192.9
9	7.800	179.9	181.7	184.1	185.7	188.0	188.9	189.8	191.1
16	7.010	179.6	181.3	183.7	185.2	187.3	188.4	189.3	190.7
21	5.140	178.5	179.6	182.3	183.8	185.4	186.1	186.7	189.0
RAMAPO2 RIVER - Unet Reach 12									
1	3.655	178.0	179.0	182.2	183.7	185.4	186.1	186.8	188.8
6	1.430	177.6	178.3	181.8	183.2	184.9	185.6	186.4	188.4
7	1.070	177.6	178.3	181.8	183.2	184.9	185.6	186.4	188.4
8	1.050	174.9	177.8	180.9	182.2	183.7	184.6	185.6	187.5
9	0.770	174.9	177.7	180.8	182.1	183.7	184.5	185.6	187.5
PEQUANNOCK RIVER - Unet Reach 9									
2	17.105	207.4	209.2	210.2	210.3	210.5	210.6	210.9	212.1
6	16.843	203.7	204.6	205.7	206.2	206.9	207.5	208.4	209.8
9	15.200	195.1	195.7	196.6	197.1	197.7	198.3	199.3	201.3
15	14.450	190.2	191.6	193.4	194.2	195.0	195.8	197.0	199.1
24	12.500	184.5	186.4	188.3	189.2	190.0	190.7	191.6	193.3
30	10.630	183.7	185.6	187.4	188.5	189.4	190.1	191.0	192.9
PEQUANNOCK2 RIVER - Unet Reach 10									
5	9.510	183.0	184.8	186.4	187.1	187.9	188.5	189.3	191.2
10	7.130	181.4	182.7	184.2	185.1	186.3	187.0	187.7	189.8
13	5.040	179.9	181.2	183.2	184.4	185.9	186.6	187.4	189.4
18	3.750	179.0	180.1	182.6	184.0	185.6	186.3	187.0	189.1
19	2.960	178.4	179.5	182.4	183.8	185.5	186.2	186.9	188.9

Table C-30b (Continued)
Existing Conditions - Peak Flows

X-Sect	Distance in 1000's of feet	1 Year FLOW CFS	2 Year FLOW CFS	5 Year FLOW CFS	10 Year FLOW CFS	25 Year FLOW CFS	50 Year FLOW CFS	100 Yr FLOW CFS	500 Yr FLOW CFS
PASSAIC4 RIVER (Continued)									
191	91.790	5422.	7861.	11663.	14481.	18637.	22266.	26374.	36840.
192	91.785	5422.	7861.	11663.	14481.	18637.	22266.	26374.	36840.
202	89.100	5457.	7873.	11680.	14494.	18656.	22289.	26567.	36859.
215	83.800	5645.	8004.	11793.	14576.	18743.	22378.	27574.	36924.
PASSAIC5 RIVER - Unet Reach 19									
1	82.200	6089.	8518.	12185.	14998.	19254.	22961.	27796.	37876.
7	78.200	6327.	8708.	12316.	15155.	19386.	23135.	27462.	37809.
32	61.030	7786.	9948.	13168.	15868.	20036.	23781.	27807.	38271.
102	15.200	16545.	18425.	21115.	23104.	26527.	29947.	33510.	44841.
118	3.200	20960.	22839.	25511.	27493.	30902.	34190.	37336.	48108.
SADDLE RIVER - Unet Reach 18									
1	17.130	1510.	2141.	2980.	3651.	4674.	5273.	6429.	8760.
26	11.940	1539.	2171.	3007.	3679.	4717.	5320.	6474.	8747.
76	2.895	1587.	2224.	3072.	3743.	4796.	5397.	6555.	8839.
DEEPAVAAL BROOK - Unet Reach 16									
4	18.625	56.	55.	57.	61.	291.	1007.	1587.	2748.
13	13.425	70.	85.	105.	130.	305.	985.	1606.	2947.
17	10.900	145.	194.	293.	376.	540.	1091.	1748.	3171.
30	6.050	160.	219.	328.	419.	582.	1122.	1805.	3309.
42	3.220	177.	250.	372.	479.	650.	1153.	1860.	3418.
58	0.720	195.	275.	407.	524.	703.	1207.	1935.	3542.
64	0.340	201.	284.	420.	540.	716.	1226.	1961.	3587.
65	0.180	203.	287.	424.	545.	720.	1236.	1975.	3609.
POMPTON RIVER - Unet Reach 14									
2	35.775	4197.	7258.	11657.	14734.	20173.	24057.	29739.	42029.
3	35.090	4199.	7255.	11650.	14721.	20161.	24045.	29729.	42034.
8	34.840	4200.	7254.	11649.	14717.	20157.	24042.	29727.	42034.
18	30.350	4195.	7193.	11516.	14427.	20029.	23952.	29709.	42048.
29	27.030	4185.	7148.	11423.	14232.	19921.	23909.	29665.	41962.
50	21.810	4087.	7011.	11207.	13914.	19632.	23651.	28809.	40964.
62	14.880	4133.	7014.	11105.	13791.	19548.	23186.	27943.	40304.
76	9.940	4118.	6957.	10948.	13579.	18881.	22816.	27777.	40126.
86	7.514	4146.	6984.	10974.	13598.	18939.	22907.	28023.	40512.
97	2.950	4056.	6842.	10761.	13275.	18494.	22606.	27505.	39906.

Table C-30b (Continued)
Existing Conditions - Peak Flows

X-Sect	Distance in 1000's of feet	1 Year FLOW CFS	2 Year FLOW CFS	5 Year FLOW CFS	10 Year FLOW CFS	25 Year FLOW CFS	50 Year FLOW CFS	100 Yr FLOW CFS	500 Yr FLOW CFS
POMPTON RIVER (Continued)									
101	1.835	4032.	6803.	10696.	13182.	18367.	22441.	27322.	39765.
106	0.480	4019.	6770.	10629.	13084.	18161.	22179.	26913.	39443.
BYPASS - Unet Reach 13									
1	1.500	50.	50.	50.	50.	50.	50.	50.	50.
4	0.300	1898.	3544.	5117.	5931.	8297.	11935.	18141.	27174.
5	0.050	1898.	3541.	5091.	5914.	8291.	11928.	18138.	27163.
RAMAPO RIVER - Unet Reach 11									
5	9.410	2614.	4298.	6610.	8763.	12949.	15448.	19346.	27086.
9	7.805	2616.	4302.	6606.	8751.	12934.	15448.	19349.	27096.
16	7.010	2617.	4304.	6598.	8735.	12904.	15435.	19340.	27090.
21	5.140	2620.	4306.	6531.	8645.	12850.	15395.	19322.	27051.
RAMAPO2 RIVER - Unet Reach 12									
1	3.655	2303.	3723.	6695.	9003.	13643.	18478.	24885.	33453.
6	1.430	2302.	3723.	6692.	8994.	11899.	12278.	12670.	14993.
7	1.070	2302.	3723.	6691.	8993.	11895.	12262.	12643.	14992.
8	1.050	2302.	3723.	6691.	8993.	11895.	12262.	12643.	14992.
9	0.770	2302.	3722.	6691.	8993.	11893.	12257.	12635.	14992.
PEQUANNOCK RIVER - Unet Reach 9									
2	17.105	443.	968.	1857.	2275.	2753.	3288.	4202.	6271.
6	16.843	443.	968.	1857.	2275.	2753.	3288.	4201.	6270.
9	15.200	443.	968.	1857.	2274.	2753.	3288.	4201.	6269.
15	14.450	443.	968.	1857.	2274.	2753.	3288.	4200.	6268.
24	12.500	443.	968.	1857.	2275.	2754.	3289.	4202.	6274.
30	10.630	444.	970.	1863.	2286.	2770.	3309.	4223.	6311.
PEQUANNOCK2 RIVER - Unet Reach 10									
5	9.510	1555.	2954.	5247.	6320.	7486.	8774.	10692.	15570.
10	7.134	1559.	2961.	5257.	6325.	7502.	8798.	10738.	15631.
13	5.040	1561.	2966.	5258.	6331.	7518.	8816.	10776.	15682.
18	3.750	1565.	2973.	5270.	6349.	7551.	8846.	10824.	15765.
19	2.965	1570.	2980.	5283.	6362.	7571.	8864.	10847.	15808.

Table C- 30a (Continued)
Existing Conditions - Water Surface Elevations

X-Sect	Distance in 1000's of Feet	1 Year ELEV FEET	2 Year ELEV FEET	5 Year ELEV FEET	10 Year ELEV FEET	25 Year ELEV FEET	50 Year ELEV FEET	100 Yr ELEV FEET	500 Yr ELEV FEET
WANAQUE RIVER - Unet Reach 8									
1	9.250	196.1	197.6	199.6	200.4	201.3	202.1	202.8	205.0
12	7.595	194.0	195.6	197.5	198.2	199.2	199.8	200.4	202.0
36	2.615	186.5	188.0	189.6	190.4	191.1	191.8	192.6	194.5
41	0.860	184.2	186.0	187.9	189.0	189.9	190.6	191.6	193.5
42	0.255	183.7	185.6	187.4	188.5	189.4	190.1	191.0	192.9
ROCKAWAY RIVER - Unet Reach 1									
5	29.060	176.0	176.7	178.0	178.6	179.8	180.4	181.2	183.1
13	24.750	173.1	173.9	175.3	176.1	177.7	178.7	179.7	182.1
18	23.680	172.7	173.5	175.1	175.8	177.5	178.5	179.6	182.0
32	18.100	171.4	172.2	173.7	174.5	176.3	177.4	178.4	180.9
36	15.350	170.6	171.6	173.1	174.1	176.0	177.0	178.0	180.5
53	11.900	169.9	170.7	172.0	172.7	173.8	174.8	175.8	178.1
54	11.550	169.8	170.6	171.9	172.6	173.6	174.6	175.5	178.0
62	6.720	168.7	169.5	170.8	171.8	173.0	174.1	175.1	177.7
ROCKAWAY2 RIVER - Unet Reach 5									
1	5.100	168.7	169.5	170.8	171.8	173.0	174.1	175.1	177.7
2	3.160	168.6	169.5	170.8	171.8	173.0	174.1	175.1	177.7
5	0.210	168.6	169.4	170.7	171.7	173.0	174.1	175.1	177.7
WHIPPANY RIVER - Unet Reach 2									
1	34.975	201.0	201.5	202.1	202.7	203.4	204.1	204.6	205.4
12	32.590	188.8	189.3	190.0	190.6	191.2	191.6	192.1	193.0
20	25.510	177.4	178.0	179.0	180.0	180.8	181.6	182.1	184.1
WHIPPANY2 RIVER - Unet Reach 4									
6	24.130	177.1	177.7	178.8	179.8	180.6	181.4	182.1	183.7
31	14.230	173.0	173.5	174.3	175.0	175.8	176.7	177.4	179.6
52	1.060	168.8	169.6	170.8	171.8	173.1	174.1	175.1	177.7
PINCH BROOK - Unet Reach 3									
1	7.275	177.4	178.0	179.0	180.0	180.8	181.6	182.2	184.1
3	4.080	177.4	178.0	179.0	180.0	180.8	181.6	182.2	184.1
6	0.150	177.4	178.0	179.0	180.0	180.8	181.6	182.1	184.1

Table C-30b
Existing Conditions - Peak Flows

X-Sect	Distance in 1000's of feet	1 Year FLOW CFS	2 Year FLOW CFS	5 Year FLOW CFS	10 Year FLOW CFS	25 Year FLOW CFS	50 Year FLOW CFS	100 Yr FLOW CFS	500 Yr FLOW CFS
PASSAIC RIVER - Unet Reach 6									
1	315.800	1151.	1476.	1981.	2449.	3020.	3417.	4025.	4973.
9	314.400	1152.	1477.	1983.	2451.	3023.	3421.	4030.	4980.
85	262.740	1133.	1479.	2060.	2653.	3367.	4028.	4549.	5984.
87	261.250	1128.	1467.	2040.	2629.	3313.	3929.	4483.	5984.
PASSAIC2 RIVER - Unet Reach 7									
2	240.300	2701.	3488.	4887.	6073.	7780.	9341.	10906.	15310.
6	235.585	2701.	3486.	4905.	6120.	7882.	9766.	11074.	15615.
11	234.650	2704.	3492.	4914.	6133.	7775.	9199.	10160.	13043.
58	175.810	2875.	4190.	6334.	7912.	9961.	11298.	12411.	16269.
60	174.630	2884.	4212.	6365.	7951.	10010.	11351.	12470.	16343.
61	174.370	2886.	4216.	6373.	7959.	10020.	11362.	12482.	16359.
67	173.860	2887.	4220.	6378.	7965.	10027.	11369.	12490.	16369.
PASSAIC3 RIVER - Unet Reach 15									
1	173.500	5143.	7527.	11220.	13947.	17857.	20841.	24234.	33345.
4	172.000	5150.	7536.	11231.	13955.	17873.	20853.	24234.	33360.
15	170.090	5235.	7643.	11379.	14135.	18144.	21241.	24680.	34084.
PASSAIC4 RIVER - Unet Reach 17									
5	167.000	5346.	7770.	11542.	14329.	18416.	21949.	26007.	36200.
21	162.290	5350.	7776.	11548.	14338.	18427.	21959.	26018.	36223.
31	157.600	5355.	7782.	11556.	14345.	18440.	21977.	26040.	36256.
35	156.865	5356.	7783.	11558.	14347.	18442.	21979.	26044.	36261.
36	155.855	5356.	7783.	11558.	14347.	18442.	21979.	26044.	36261.
42	153.860	5359.	7786.	11562.	14352.	18449.	21989.	26054.	36278.
47	152.000	5360.	7787.	11563.	14353.	18452.	21993.	26059.	36286.
74	143.830	5372.	7798.	11584.	14379.	18494.	22055.	26133.	36426.
93	133.865	5381.	7810.	11599.	14400.	18522.	22099.	26183.	36511.
94	133.520	5381.	7810.	11599.	14400.	18522.	22099.	26183.	36511.
96	133.350	5381.	7810.	11599.	14400.	18522.	22099.	26183.	36512.
117	128.000	5390.	7819.	11612.	14416.	18545.	22132.	26221.	36578.
136	119.700	5403.	7834.	11632.	14442.	18582.	22188.	26285.	36685.
143	117.300	5404.	7835.	11634.	14444.	18586.	22192.	26291.	36695.
153	111.860	5410.	7844.	11644.	14456.	18603.	22218.	26320.	36745.
158	108.900	5411.	7845.	11646.	14459.	18606.	22223.	26324.	36752.
163	105.950	5413.	7848.	11649.	14463.	18611.	22230.	26331.	36768.
174	99.763	5415.	7852.	11654.	14469.	18620.	22243.	26347.	36794.
184	95.300	5417.	7855.	11657.	14474.	18627.	22251.	26357.	36812.

Table C-30b (Continued)
Existing Conditions - Peak Flows

X-Sect	Distance in 1000's of feet	1 Year FLOW CFS	2 Year FLOW CFS	5 Year FLOW CFS	10 Year FLOW CFS	25 Year FLOW CFS	50 Year FLOW CFS	100 Yr FLOW CFS	500 Yr FLOW CFS
WANAQUE RIVER - Unet Reach 8									
1	9.250	1065.	1905.	3296.	3967.	4665.	5434.	6261.	9139.
12	7.595	1065.	1906.	3298.	3969.	4667.	5436.	6264.	9140.
36	2.615	1122.	2034.	3542.	4274.	5030.	5866.	6770.	9886.
41	0.860	1123.	2035.	3543.	4267.	5014.	5851.	6751.	9865.
42	0.255	1123.	2035.	3543.	4266.	5011.	5849.	6750.	9865.
ROCKAWAY RIVER - Unet Reach 1									
5	29.060	1104.	1617.	2862.	3576.	5559.	6636.	8340.	12092.
13	24.750	1116.	1635.	2879.	3595.	5472.	6532.	8203.	11903.
18	23.680	1117.	1638.	2879.	3596.	5432.	6492.	8158.	11838.
32	18.100	1127.	1649.	2884.	3585.	5234.	6368.	7992.	11464.
36	15.350	1142.	1675.	2923.	3626.	5241.	6461.	8075.	11540.
53	11.900	1142.	1676.	2921.	3605.	5181.	6436.	8028.	11372.
54	11.550	1143.	1677.	2921.	3606.	5182.	6433.	8024.	11364.
62	6.715	1140.	1670.	2905.	3582.	5130.	6339.	7879.	11083.
ROCKAWAY2 RIVER - Unet Reach 5									
1	5.100	1716.	2274.	3349.	4044.	5252.	6295.	7229.	9657.
2	3.160	1709.	2259.	3322.	4016.	5193.	6220.	7126.	9611.
5	0.210	1692.	2231.	3264.	3958.	5067.	6050.	6975.	9516.
WHIPPANY RIVER - Unet Reach 2									
1	34.975	1200.	1397.	1759.	2096.	2446.	2795.	3107.	4059.
12	32.590	1200.	1396.	1759.	2095.	2445.	2793.	3106.	4059.
20	25.510	1167.	1333.	1677.	1936.	2140.	2357.	2543.	3161.
WHIPPANY2 RIVER - Unet Reach 4									
6	24.130	1055.	1268.	1644.	1916.	2284.	2659.	2944.	3791.
31	14.230	810.	971.	1216.	1449.	1720.	1976.	2181.	2804.
52	1.060	781.	913.	1165.	1492.	1882.	2370.	2736.	3937.
PINCH BROOK - Unet Reach 3									
1	7.275	122.	188.	307.	425.	560.	719.	793.	1171.
3	4.080	240.	356.	625.	927.	1305.	1717.	1864.	2834.
6	0.150	233.	284.	344.	414.	492.	579.	622.	1257.

Table C-31

SELECTED GAGE LOCATIONS - FUTURE CONDITIONS - DISCHARGES

2-YEAR (50% CHANCE) FLOOD EVENT

GAGE LOCATION	COMPUTED PEAK FLOW, cfs EXISTING (1990)	FUTURE (2050)	INCREASE
Passaic at Chatham	1480	1560	5.4%
Rockaway above Boonton Res.	2420	2730	12.8%
Whippany at Morristown	1220	1380	13.1%
Ramapo at Pompton Lakes	4300	4660	8.4%
Saddle at Lodi	1850	1860	0.5%
Pompton at Pompton Plains	7260	7790	7.3%
Passaic at Little Falls	7790	8250	5.9%

5-YEAR (20% CHANCE) FLOOD EVENT

GAGE LOCATION	COMPUTED PEAK FLOW, cfs EXISTING (1990)	FUTURE (2050)	INCREASE
Passaic at Chatham	1980	2090	5.6%
Rockaway above Boonton Res.	3780	4170	10.3%
Whippany at Morristown	1560	1740	11.5%
Ramapo at Pompton Lakes	6610	7070	7.0%
Saddle at Lodi	2700	2710	0.4%
Pompton at Pompton Plains	11650	12370	6.2%
Passaic at Little Falls	11560	12170	5.3%

Table C-31 (cont)

SELECTED GAGE LOCATIONS - FUTURE CONDITIONS - DISCHARGES

10-YEAR (10% CHANCE) FLOOD EVENT

GAGE LOCATION	COMPUTED PEAK FLOW, cfs EXISTING (1990)	FUTURE (2050)	INCREASE
Passaic at Chatham	2450	2570	4.9%
Rockaway above Boonton Res.	4590	5030	9.6%
Whippany at Morristown	1880	2080	10.6%
Ramapo at Pompton Lakes	8780	9270	5.6%
Saddle at Lodi	3650	3660	0.3%
Pompton at Pompton Plains	14720	15570	5.8%
Passaic at Little Falls	14350	15030	4.7%

25-YEAR (4% CHANCE) FLOOD EVENT

GAGE LOCATION	COMPUTED PEAK FLOW, cfs EXISTING (1990)	FUTURE (2050)	INCREASE
Passaic at Chatham	3020	3150	4.3%
Rockaway above Boonton Res.	5970	6510	9.0%
Whippany at Morristown	2200	2410	9.5%
Ramapo at Pompton Lakes	12950	13500	4.3%
Saddle at Lodi	4670	4680	0.2%
Pompton at Pompton Plains	20160	21090	4.6%
Passaic at Little Falls	18450	19180	3.9%

Table C-31 (cont)

SELECTED GAGE LOCATIONS - FUTURE CONDITIONS - DISCHARGES

50-YEAR (2% CHANCE) FLOOD EVENT

GAGE LOCATION	COMPUTED PEAK FLOW, cfs EXISTING (1990)	FUTURE (2050)	INCREASE
Passaic at Chatham	3420	3550	3.8%
Rockaway above Boonton Res.	7060	7570	7.2%
Whippany at Morristown	2530	2750	8.7%
Ramapo at Pompton Lakes	15450	16010	3.6%
Saddle at Lodi	5270	5280	0.2%
Pompton at Pompton Plains	24050	25030	4.1%
Passaic at Little Falls	21990	22650	3.0%

100-YEAR (1% CHANCE) FLOOD EVENT

GAGE LOCATION	COMPUTED PEAK FLOW, cfs EXISTING (1990)	FUTURE (2050)	INCREASE
Passaic at Chatham	4020	4170	3.7%
Rockaway above Boonton Res.	8670	9190	6.0%
Whippany at Morristown	2820	3050	8.1%
Ramapo at Pompton Lakes	19350	20000	3.4%
Saddle at Lodi	6430	6440	0.2%
Pompton at Pompton Plains	29730	31070	4.5%
Passaic at Little Falls	26050	26800	2.9%

Table C-31 (cont)

SELECTED GAGE LOCATIONS - FUTURE CONDITIONS - DISCHARGES

500-YEAR (.2% CHANCE) FLOOD EVENT

GAGE LOCATION	COMPUTED PEAK FLOW, cfs EXISTING (1990)	FUTURE (2050)	INCREASE
Passaic at Chatham	4970	5120	3.0%
Rockaway above Boonton Res.	12130	12740	5.0%
Whippany at Morristown	3640	3890	6.9%
Ramapo at Pompton Lakes	31700	32700	3.1%
Saddle at Lodi	8760	8770	0.1%
Pompton at Pompton Plains	42040	43630	3.7%
Passaic at Little Falls	36280	37110	2.3%

Table C-32a
Future Conditions w/o Project - Water Surface Elevations

X-Sect	Distance in 1000's of Feet	1 Year ELEV FEET	2 Year ELEV FEET	5 Year ELEV FEET	10 Year ELEV FEET	25 Year ELEV FEET	50 Year ELEV FEET	100 Yr ELEV FEET	500 Yr ELEV FEET
PASSAIC RIVER - UNET REACH 6									
1	315.800	200.1	200.7	201.5	202.1	202.7	203.2	203.7	204.6
9	314.400	199.2	199.6	200.2	200.7	201.2	201.5	201.9	202.6
85	262.740	169.7	170.5	171.9	173.1	174.4	175.2	176.2	178.6
87	261.250	169.7	170.5	171.9	173.1	174.4	175.2	176.2	178.6
PASSAIC2 RIVER - UNET REACH 7									
2	240.300	168.8	169.6	171.0	172.0	173.3	174.3	175.4	178.0
6	235.585	168.0	168.7	169.9	170.9	172.1	173.2	174.3	176.9
11	234.650	167.9	168.6	169.8	170.8	172.0	173.0	174.0	176.6
58	175.810	164.8	166.4	168.4	169.7	171.2	172.4	173.6	176.1
60	174.630	164.7	166.4	168.4	169.7	171.2	172.4	173.6	176.1
61	174.370	164.7	166.4	168.4	169.6	171.2	172.4	173.5	176.1
67	173.860	164.5	166.1	168.1	169.3	170.9	172.1	173.2	175.8
PASSAIC3 RIVER - UNET REACH 15									
1	173.500	164.7	166.4	168.4	169.6	171.2	172.3	173.5	176.1
4	172.000	164.5	166.1	168.1	169.3	170.8	172.0	173.2	175.8
15	170.090	164.2	165.7	167.7	168.8	170.3	171.4	172.7	175.3
PASSAIC4 RIVER - UNET REACH 17									
5	167.000	163.8	165.3	167.1	168.2	169.6	170.7	171.9	174.4
21	162.290	163.2	164.5	166.1	167.0	168.2	169.1	170.1	172.2
31	157.600	161.4	162.1	163.3	164.1	165.2	166.2	167.1	169.3
35	156.865	161.1	161.7	162.6	163.2	164.2	165.0	165.8	167.7
36	155.855	127.0	127.8	127.8	127.8	127.9	128.1	128.2	129.3
42	153.860	126.5	128.2	130.2	131.4	133.0	134.1	135.4	138.0
47	152.000	124.1	125.9	128.1	129.5	131.1	132.3	133.6	136.3
74	143.830	120.0	121.6	123.7	125.1	127.0	128.4	129.7	132.5
93	133.865	118.0	119.0	120.3	121.1	122.0	122.9	123.8	125.7
94	133.520	51.0	53.3	56.5	58.7	61.7	63.7	65.6	69.8
96	133.350	50.9	53.3	56.5	58.8	61.8	64.0	66.0	70.2
117	128.000	38.5	40.4	42.7	44.1	45.8	47.1	49.2	53.5
136	119.700	34.5	36.0	38.0	39.2	40.9	42.1	43.5	48.0
143	117.300	33.8	35.1	37.0	38.2	39.8	41.0	42.3	46.1
153	111.860	30.6	32.1	34.0	35.2	36.8	38.1	39.5	42.0
158	108.900	30.0	31.3	33.1	34.2	35.6	36.7	37.8	40.3
163	105.950	29.5	30.7	32.4	33.4	34.7	35.7	36.8	39.2
174	99.763	28.5	29.4	30.6	31.4	32.4	33.1	33.9	35.5
184	95.300	27.9	28.7	29.7	30.3	31.0	31.7	32.4	34.0

Table C-32a (Continued)
Future Conditions w/o Project - Water Surface Elevations

X-Sect	Distance in 1000's of Feet	1 Year ELEV FEET	2 Year ELEV FEET	5 Year ELEV FEET	10 Year ELEV FEET	25 Year ELEV FEET	50 Year ELEV FEET	100 Yr ELEV FEET	500 Yr ELEV FEET
PASSAIC4 RIVER (Continued)									
191	91.790	27.9	28.6	29.5	30.1	30.8	31.4	32.0	33.5
192	91.785	10.4	11.9	13.8	14.9	16.6	17.9	20.6	24.8
202	89.100	9.3	11.0	12.9	14.0	15.6	16.9	19.8	23.2
215	83.800	7.6	8.5	10.0	11.2	13.1	14.5	16.0	19.1
PASSAIC5 RIVER - UNET REACH 19									
1	82.200	7.5	8.4	9.8	11.0	12.8	14.1	15.6	18.6
7	78.200	7.1	7.7	8.9	10.0	11.6	13.0	14.5	17.6
32	61.030	6.5	6.7	7.3	7.8	8.9	10.0	11.0	13.5
102	15.200	6.2	6.2	6.2	6.2	6.3	6.3	6.4	6.5
118	3.200	6.2	6.2	6.2	6.2	6.2	6.2	6.2	6.2
SADDLE RIVER - UNET REACH 18									
1	17.130	30.4	31.9	33.8	35.3	36.9	37.6	38.6	40.0
26	11.940	20.6	22.5	24.6	26.1	27.8	28.7	30.2	32.4
76	2.895	10.7	12.4	14.3	15.6	17.5	18.4	20.0	22.7
DEEPAVAAL BROOK - UNET REACH 16									
4	18.625	168.7	169.2	169.7	170.2	171.0	172.5	173.3	175.5
13	13.425	168.6	169.1	169.7	170.2	170.9	172.3	173.2	175.5
17	10.900	168.1	168.7	169.4	170.0	170.7	172.1	173.1	175.4
30	6.050	164.5	165.9	167.7	168.8	170.3	171.6	172.8	175.4
42	3.220	164.3	165.8	167.7	168.8	170.3	171.5	172.7	175.3
58	0.720	164.2	165.7	167.7	168.8	170.3	171.5	172.7	175.3
64	0.340	164.2	165.7	167.7	168.8	170.3	171.4	172.7	175.3
65	0.180	164.2	165.7	167.7	168.8	170.3	171.4	172.7	175.3
POMPTON RIVER - UNET REACH 14									
2	35.775	175.2	178.0	181.0	182.2	183.7	184.5	185.6	187.5
3	35.090	174.8	177.7	180.7	182.0	183.4	184.2	185.3	187.1
8	34.840	174.7	177.6	180.6	181.9	183.3	184.1	185.2	187.0
18	30.350	172.0	174.8	177.8	179.8	181.7	182.7	183.7	185.4
29	27.030	170.6	173.3	176.3	178.3	180.3	181.3	182.4	184.1
50	21.810	169.5	172.1	174.7	176.1	177.5	178.3	179.6	181.9
62	14.880	167.6	169.7	171.8	173.3	175.9	177.2	178.7	181.1
76	9.940	166.2	168.3	170.5	172.2	174.5	175.7	177.0	179.7
86	7.514	165.5	167.3	169.4	171.2	173.1	174.1	175.1	177.3
97	2.950	165.0	166.6	168.7	169.9	171.6	172.8	174.0	176.6

Table C-32a (Continued)
Future Conditions w/o Project - Water Surface Elevations

X-Sect	Distance in 1000's of Feet	1 Year ELEV FEET	2 Year ELEV FEET	5 Year ELEV FEET	10 Year ELEV FEET	25 Year ELEV FEET	50 Year ELEV FEET	100 Yr ELEV FEET	500 Yr ELEV FEET
POMPTON RIVER (Continued)									
101	1.835	164.8	166.5	168.6	169.8	171.5	172.7	173.8	176.3
106	0.480	164.7	166.3	168.4	169.6	171.2	172.4	173.6	176.2
BYPASS - UNET REACH 13									
1	1.500	176.5	178.6	181.3	182.5	184.0	184.9	186.1	188.0
4	0.300	175.6	178.3	181.2	182.4	183.9	184.8	185.9	187.8
5	0.050	175.3	178.2	181.1	182.4	183.9	184.7	185.9	187.7
RAMAPO RIVER - UNET REACH 11									
5	9.410	181.2	183.2	185.5	187.1	189.4	190.5	191.6	193.1
9	7.805	180.2	182.1	184.5	186.1	188.2	189.1	189.9	191.3
16	7.010	180.0	181.6	184.1	185.6	187.6	188.6	189.5	190.9
21	5.140	178.7	179.8	182.7	184.1	185.6	186.2	186.9	189.2
RAMAPO2 RIVER - UNET REACH 12									
1	3.655	178.1	179.3	182.6	184.0	185.6	186.2	186.9	189.0
6	1.430	177.7	178.6	182.1	183.6	185.1	185.8	186.6	188.7
7	1.070	177.7	178.5	182.1	183.5	185.1	185.8	186.6	188.7
8	1.050	175.4	178.2	181.2	182.4	183.9	184.8	185.9	187.7
9	0.770	175.3	178.2	181.1	182.4	183.9	184.7	185.9	187.7
PEQUANNOCK RIVER - UNET REACH 9									
2	17.105	208.5	209.5	210.2	210.4	210.5	210.7	211.0	212.2
6	16.843	203.8	204.8	205.8	206.4	207.1	207.7	208.6	209.9
9	15.200	195.3	195.7	196.7	197.3	197.8	198.5	199.5	201.6
15	14.450	190.4	191.8	193.6	194.4	195.2	196.0	197.2	199.3
24	12.500	184.7	186.6	188.7	189.4	190.2	190.9	191.8	193.5
30	10.630	183.9	185.8	188.0	188.8	189.6	190.3	191.3	193.1
PEQUANNOCK2 RIVER - UNET REACH 10									
5	9.510	183.2	185.0	186.6	187.3	188.1	188.7	189.5	191.4
10	7.134	181.6	182.9	184.4	185.3	186.5	187.1	187.9	190.0
13	5.040	179.9	181.2	183.2	184.4	185.9	186.6	187.4	189.4
18	3.750	179.1	180.3	182.9	184.3	185.8	186.4	187.2	189.3
19	2.965	178.5	179.8	182.5	184.0	185.6	186.3	187.0	189.1

Table C-32a (Continued)
Future Conditions w/o Project - Water Surface Elevations

X-Sect	Distance in 1000's of Feet	1 Year ELEV FEET	2 Year ELEV FEET	5 Year ELEV FEET	10 Year ELEV FEET	25 Year ELEV FEET	50 Year ELEV FEET	100 Yr ELEV FEET	500 Yr ELEV FEET
WANAQUE RIVER - UNET REACH 8									
1	9.250	196.3	197.8	199.8	200.7	201.5	202.3	203.3	207.0
12	7.595	194.2	195.8	197.6	198.6	199.4	199.9	200.8	202.5
36	2.615	186.7	188.1	189.8	190.5	191.3	191.9	192.9	194.8
41	0.860	184.4	186.2	188.4	189.2	190.1	190.8	191.9	193.8
42	0.255	183.9	185.8	188.0	188.8	189.6	190.3	191.3	193.1
ROCKAWAY RIVER - UNET REACH 1									
5	29.060	176.3	177.1	178.3	179.0	180.1	180.8	181.7	184.0
13	24.750	173.5	174.2	175.8	176.6	178.3	179.4	180.7	183.4
18	23.680	173.1	173.9	175.6	176.4	178.2	179.3	180.6	183.4
32	18.100	171.8	172.7	174.3	175.3	177.2	178.3	179.6	182.4
36	15.350	171.3	172.2	173.9	174.9	176.9	178.1	179.4	182.2
53	10.890	170.1	170.8	172.1	172.7	173.8	174.8	175.8	178.2
54	9.915	170.1	170.8	172.0	172.6	173.7	174.7	175.7	178.2
60	6.715	169.0	169.8	171.1	172.1	173.3	174.4	175.4	178.0
ROCKAWAY2 RIVER - UNET REACH 5									
1	5.100	169.0	169.8	171.1	172.1	173.3	174.4	175.4	178.0
2	3.160	168.9	169.7	171.0	172.0	173.3	174.4	175.4	178.0
5	0.210	168.8	169.6	171.0	172.0	173.3	174.4	175.4	178.0
WHIPPANY RIVER - UNET REACH 2									
1	34.975	201.4	201.8	202.4	203.1	203.8	204.4	205.0	205.4
12	32.590	189.2	189.7	190.4	191.0	191.4	191.9	192.3	193.3
20	25.510	177.7	178.3	179.4	180.2	181.0	181.8	182.4	184.4
WHIPPANY2 - UNET REACH 4									
6	24.130	177.4	178.0	179.2	180.1	180.9	181.7	182.3	183.9
31	14.230	173.2	173.7	174.5	175.2	176.1	177.0	177.7	179.8
52	1.060	169.1	169.8	171.1	172.1	173.4	174.4	175.4	178.0
PINCH BROOK - UNET REACH 3									
1	7.275	177.7	178.3	179.4	180.3	181.0	181.8	182.4	184.4
3	4.080	177.7	178.3	179.4	180.3	181.0	181.8	182.4	184.4
6	0.150	177.7	178.3	179.4	180.2	181.0	181.8	182.4	184.4

Table C-32b
Future Conditions w/o Project - Peak Flows

X-Sect	LOCATION 1000FT UNT	1 Year FLOW CFS UNT	2 Year FLOW CFS UNT	5 Year FLOW CFS UNT	10 Year FLOW CFS UNT	25 Year FLOW CFS UNT	50 Year FLOW CFS UNT	100 Yr FLOW CFS UNT	500 Yr FLOW CFS UNT
PASSAIC RIVER - UNET REACH 6									
1	315.800	1245.	1564.	2091.	2570.	3149.	3551.	4166.	5119.
9	314.400	1246.	1565.	2093.	2573.	3152.	3555.	4171.	5126.
85	262.740	1224.	1582.	2202.	2811.	3547.	4215.	4728.	6197.
87	261.250	1217.	1568.	2179.	2784.	3482.	4112.	4667.	6190.
PASSAIC2 RIVER - UNET REACH 7									
2	240.300	2976.	3764.	5220.	6470.	8314.	10005.	11452.	15950.
6	235.585	2974.	3758.	5244.	6528.	8535.	11181.	11744.	16249.
11	234.650	2978.	3764.	5253.	6542.	8166.	10768.	11222.	13525.
58	175.810	3177.	4546.	6745.	8368.	10356.	11688.	12814.	16689.
60	174.630	3188.	4569.	6779.	8411.	10405.	11743.	12872.	16762.
61	174.370	3190.	4573.	6787.	8420.	10416.	11754.	12884.	16778.
67	173.860	3191.	4576.	6792.	8426.	10422.	11761.	12892.	16788.
PASSAIC3 RIVER - UNET REACH 15									
1	173.500	5497.	7982.	11819.	14592.	18506.	21398.	24777.	34229.
4	172.000	5504.	7990.	11831.	14606.	18521.	21407.	24781.	34245.
15	170.090	5600.	8107.	11990.	14812.	18810.	21820.	25244.	34993.
PASSAIC4 RIVER - UNET REACH 17									
5	167.000	5714.	8238.	12154.	15009.	19149.	22603.	26751.	37056.
21	162.290	5719.	8243.	12161.	15017.	19159.	22619.	26764.	37080.
31	157.600	5724.	8249.	12168.	15026.	19173.	22638.	26787.	37114.
35	156.865	5724.	8250.	12170.	15028.	19175.	22641.	26790.	37119.
36	155.855	5724.	8250.	12170.	15028.	19175.	22641.	26790.	37119.
42	153.860	5727.	8253.	12174.	15033.	19182.	22650.	26801.	37137.
47	152.000	5728.	8254.	12176.	15035.	19185.	22654.	26805.	37145.
74	143.830	5742.	8268.	12198.	15066.	19230.	22722.	26885.	37294.
93	133.865	5754.	8280.	12214.	15087.	19261.	22768.	26937.	37386.
94	133.520	5754.	8280.	12214.	15087.	19261.	22768.	26937.	37386.
96	133.350	5754.	8281.	12215.	15087.	19261.	22769.	26937.	37387.
117	128.000	5764.	8291.	12228.	15104.	19285.	22804.	26976.	37455.
136	119.700	5776.	8304.	12246.	15129.	19323.	22863.	27040.	37564.
143	117.300	5777.	8306.	12248.	15131.	19326.	22868.	27046.	37575.
153	111.860	5783.	8313.	12258.	15144.	19344.	22893.	27075.	37628.
158	108.900	5784.	8315.	12260.	15146.	19348.	22898.	27078.	37637.
163	105.950	5786.	8317.	12263.	15150.	19353.	22906.	27087.	37652.
174	99.763	5788.	8321.	12268.	15156.	19362.	22919.	27102.	37677.
184	95.300	5790.	8324.	12271.	15161.	19368.	22929.	27113.	37694.

Table C-32b (Continued)
Future Conditions w/o Project - Peak Flows

X-Sect	LOCATION 1000FT UNT	1 Year FLOW CFS UNT	2 Year FLOW CFS UNT	5 Year FLOW CFS UNT	10 Year FLOW CFS UNT	25 Year FLOW CFS UNT	50 Year FLOW CFS UNT	100 Yr FLOW CFS UNT	500 Yr FLOW CFS UNT
PASSAIC4 RIVER (Continued)									
191	91.790	5795.	8329.	12277.	15168.	19378.	22944.	27130.	37723.
192	91.785	5795.	8329.	12277.	15168.	19378.	22944.	27130.	37723.
202	89.100	5833.	8360.	12295.	15181.	19400.	22967.	27152.	37743.
215	83.800	6041.	8513.	12407.	15256.	19497.	23058.	27248.	37802.
PASSAIC5 RIVER - UNET REACH 19									
1	82.200	6489.	8975.	12803.	15697.	20021.	23655.	27895.	38790.
7	78.200	6744.	9178.	12935.	15872.	20166.	23823.	28029.	38714.
32	61.030	8231.	10446.	13788.	16615.	20886.	24492.	28664.	39215.
102	15.200	17365.	19342.	22010.	24125.	27643.	30804.	34210.	45618.
118	3.200	21772.	23736.	26395.	28467.	31975.	35156.	38393.	48923.
SADDLE RIVER - UNET REACH 18									
1	17.130	1514.	2146.	2985.	3656.	4679.	5279.	6436.	8771.
26	11.940	1545.	2177.	3015.	3687.	4723.	5327.	6483.	8760.
76	2.895	1597.	2237.	3086.	3756.	4807.	5408.	6566.	8853.
DEEPAVAAL BROOK - UNET REACH 16									
4	18.625	54.	54.	56.	59.	410.	1203.	1695.	2851.
13	13.425	71.	88.	110.	133.	427.	1228.	1770.	3067.
17	10.900	154.	207.	308.	399.	567.	1342.	1919.	3292.
30	6.050	170.	231.	342.	442.	611.	1380.	1981.	3440.
42	3.220	188.	263.	387.	502.	674.	1419.	2042.	3551.
58	0.720	206.	289.	424.	546.	727.	1481.	2122.	3675.
64	0.340	210.	297.	431.	557.	735.	1498.	2145.	3710.
65	0.180	214.	302.	438.	567.	744.	1515.	2167.	3744.
POMPTON RIVER - UNET REACH 14									
2	35.775	4636.	7788.	12383.	15585.	21087.	25031.	31108.	43547.
3	35.090	4637.	7784.	12373.	15567.	21082.	25023.	31110.	43563.
8	34.840	4637.	7784.	12371.	15563.	21080.	25020.	31110.	43566.
18	30.350	4629.	7724.	12221.	15257.	20978.	24971.	31103.	43583.
29	27.030	4608.	7680.	12118.	15057.	20852.	24962.	31030.	43461.
50	21.810	4499.	7537.	11876.	14732.	20512.	24681.	30248.	42344.
62	14.880	4545.	7528.	11675.	14511.	20376.	24023.	29268.	41721.
76	9.940	4523.	7456.	11516.	14109.	19887.	23696.	29103.	41554.
86	7.514	4552.	7478.	11540.	14061.	19961.	23955.	29346.	41964.
97	2.950	4449.	7330.	11312.	13670.	19483.	23592.	28750.	41332.

Table C-32b (Continued)
Future Conditions w/o Project - Peak Flows

X-Sect	LOCATION 1000FT UNT	1 Year FLOW CFS UNT	2 Year FLOW CFS UNT	5 Year FLOW CFS UNT	10 Year FLOW CFS UNT	25 Year FLOW CFS UNT	50 Year FLOW CFS UNT	100 Yr FLOW CFS UNT	500 Yr FLOW CFS UNT
POMPTON RIVER (Continued)									
101	1.835	4423.	7290.	11243.	13564.	19344.	23444.	28570.	41199.
106	0.480	4405.	7252.	11171.	13457.	19116.	23079.	28143.	40867.
BYPASS - UNET REACH 13									
1	1.500	50.	50.	50.	50.	50.	50.	50.	50.
4	0.300	2132.	3760.	5239.	6132.	9081.	12939.	19727.	28100.
5	0.050	2131.	3749.	5219.	6118.	9076.	12932.	19722.	28092.
RAMAPO RIVER - UNET REACH 11									
5	9.410	2911.	4661.	7072.	9275.	13495.	16017.	19997.	27750.
9	7.805	2913.	4666.	7069.	9259.	13487.	16027.	20005.	27759.
16	7.010	2915.	4668.	7058.	9229.	13465.	16027.	20003.	27747.
21	5.140	2919.	4664.	6995.	9141.	13413.	15997.	19980.	27679.
RAMAPO2 RIVER - UNET REACH 12									
1	3.655	2509.	4143.	7243.	9579.	14766.	19644.	26269.	34683.
6	1.430	2509.	4143.	7233	9555.	12059.	12406.	12707.	15519.
7	1.070	2508.	4142.	7233.	9555.	12059.	12406.	12707.	15519.
8	1.050	2508.	4142.	7233.	9555.	12059.	12406.	12707.	15519.
9	0.770	2507.	4141.	7232.	9554.	12056.	12400.	12697.	15518.
PEQUANNOCK RIVER - UNET REACH 9									
2	17.105	495.	1051.	1977.	2409.	2901.	3463.	4398.	6502.
6	16.843	495.	1051.	1977.	2409.	2901.	3463.	4398.	6502.
9	15.200	495.	1051.	1977.	2409.	2901.	3463.	4397.	6501.
15	14.450	495.	1051.	1977.	2409.	2901.	3463.	4397.	6500.
24	12.500	495.	1051.	1977.	2409.	2902.	3465.	4400.	6505.
30	10.630	495.	1053.	1985.	2420.	2920.	3486.	4430.	6537.
PEQUANNOCK2 RIVER - UNET REACH 10									
5	9.510	1705.	3162.	5536.	6657.	7824.	9144.	11313.	16378.
10	7.134	1709.	3171.	5545.	6656.	7844.	9173.	11360.	16385.
13	5.040	1711.	3176.	5551.	6664.	7862.	9199.	11393.	16407.
18	3.750	1716.	3187.	5570.	6682.	7894.	9233.	11443.	16495.
19	2.965	1720.	3199.	5584.	6696.	7910.	9252.	11468.	16532.

Table C-32b (Continued)
Future Conditions w/o Project - Peak Flows

X-Sect	LOCATION 1000FT UNT	1 Year FLOW CFS UNT	2 Year FLOW CFS UNT	5 Year FLOW CFS UNT	10 Year FLOW CFS UNT	25 Year FLOW CFS UNT	50 Year FLOW CFS UNT	100 Yr FLOW CFS UNT	500 Yr FLOW CFS UNT
WANAQUE RIVER - UNET REACH 8									
1	9.250	1157.	2021.	3459.	4138.	4845.	5621.	6874.	9846.
12	7.595	1158.	2022.	3461.	4154.	4846.	5623.	6876.	9847.
36	2.615	1222.	2161.	3725.	4462.	5234.	6078.	7447.	10627.
41	0.860	1223.	2162.	3756.	4455.	5215.	6062.	7421.	10611.
42	0.255	1223.	2162.	3807.	4454.	5213.	6061.	7420.	10611.
ROCKAWAY RIVER - UNET REACH 1									
5	29.060	1321.	1863.	3225.	4100.	6095.	7179.	8926.	12701.
13	24.750	1331.	1879.	3234.	4081.	5972.	6993.	8675.	12380.
18	23.680	1333.	1881.	3231.	4061.	5925.	6934.	8601.	12285.
32	18.100	1338.	1888.	3226.	3977.	5670.	6674.	8230.	11663.
36	15.350	1353.	1912.	3263.	3998.	5664.	6736.	8268.	11707.
53	10.890	1354.	1914.	3263.	3975.	5595.	6687.	8192.	11496.
54	9.915	1354.	1914.	3263.	3971.	5584.	6671.	8161.	11428.
62	6.715	1348.	1906.	3243.	3943.	5533.	6595.	8055.	11222.
ROCKAWAY2 RIVER - UNET REACH 5									
1	5.100	1928.	2485.	3639.	4315.	5563.	6525.	7439.	10099.
2	3.160	1919.	2470.	3611.	4285.	5504.	6458.	7380.	10062.
5	0.210	1899.	2438.	3551.	4221.	5380.	6325.	7247.	9997.
WHIPPANY RIVER - UNET REACH 2									
1	34.975	1348.	1555.	1934.	2284.	2644.	3000.	3319.	4324.
12	32.590	1348.	1554.	1933.	2283.	2642.	2998.	3319.	4320.
20	25.510	1299.	1494.	1829.	2079.	2280.	2495.	2705.	3329.
WHIPPANY2 - UNET REACH 4									
6	24.130	1157.	1389.	1746.	2038.	2413.	2800.	3074.	3923.
31	14.230	880.	1043.	1288.	1527.	1795.	2077.	2272.	2902.
52	1.060	833.	986.	1262.	1591.	1981.	2502.	2839.	4071.
PINCH BROOK - UNET REACH 3									
1	7.275	129.	197.	317.	436.	571.	730.	803.	1181.
3	4.080	234.	350.	635.	935.	1306.	1709.	1851.	2824.
6	0.150	251.	298.	359.	411.	491.	582.	629.	1234.

Table C-33

Comparison of Observed and Simulated Tidal Elevations
For the "Halloween" Storm of October 1991

Location	Node	Observed Height[1]	Observed Time[2]	Predicted Height[1]	Predicted Time
New York Harbor					
Caven Point, New York Harbor	60	6.2	W (2pm)	6.02	W (2 pm)
Passaic River					
BASF Plant, Kearny	262	7.62-7.94	T (debris)	7.23	T (3:45 am)
4th St., Newark	171	6.83 7.43	W (3 pm) T (debris)	6.66 7.29	W (3 pm) T (4 am)
Fire Pier, Newark	172	7.75	T (debris)	7.31	T (4 am)
Page St., Lyndhurst	216	5.72-6.22 6.25-7.6	W (3:30 pm) T (debris)	6.78 7.37	W (3:30 pm) T (3:45 am)
Old Boat House, East Rutherford	219	6.34	T (debris)	7.39	T (3:45 am)
Hackensack River					
Daily News, Kearny	264	> 7.1	T (??)	7.22	T (3:30 am)
Fence Outlet, Ridgefield Park	207	5.34 5.94	W (3 pm) T (7-8 am)	5.74 5.79	W (3 pm) T (7:30 am)
Gas Station, South Hackensack	206	4.95	T (7 am)	6.06	T (7 am)
Marina, South Hackensack	210	5.79	T (debris)	6.91	T (5:15 am)
Hess, Bogota	211	7.2	T (debris)	6.94	T (5:15 am)

[1] Feet NGVD
[2] W = Wednesday afternoon, 30 October 1991
T = Thursday morning, 31 October 1991

Table C-34

Tallies of coincidence between surge height and river flow events

********** COINCIDENCE ANALYSIS **********

THE TOTAL NUMBER OF POINTS USED WAS 21171
TIME SERIES SURGE.DLY : MEAN = .95 STD. DEV. = .72
TIME SERIES RIVER.DLY : MEAN = 1096.90 STD. DEV. = 1330.02

SURGE HEIGHT THRESHOLD VALUE IS 2.35 FEET
RIVER FLOW THRESHOLD VALUE IS 3040.00 CFS

THRESHOLD VALUES WERE SPECIFIED BY USER

WHEN COMPARING SURGE HEIGHT WITH RIVER FLOW 3 DAYS LATER:	
BOTH SERIES EXCEED THRESHOLDS	179 TIMES
SURGE EXCEEDS THRESHOLD AND FLOW IS LESS THAN THRESHOLD	640 TIMES
SURGE IS LESS THAN THRESHOLD AND FLOW EXCEEDS THRESHOLD	1432 TIMES
BOTH SERIES ARE LESS THAN THRESHOLDS	18917 TIMES
WHEN COMPARING SURGE HEIGHT WITH RIVER FLOW 2 DAYS LATER:	
BOTH SERIES EXCEED THRESHOLDS	192 TIMES
SURGE EXCEEDS THRESHOLD AND FLOW IS LESS THAN THRESHOLD	627 TIMES
SURGE IS LESS THAN THRESHOLD AND FLOW EXCEEDS THRESHOLD	1419 TIMES
BOTH SERIES ARE LESS THAN THRESHOLDS	18930 TIMES
WHEN COMPARING SURGE HEIGHT WITH RIVER FLOW 1 DAY LATER:	
BOTH SERIES EXCEED THRESHOLDS	191 TIMES
SURGE EXCEEDS THRESHOLD AND FLOW IS LESS THAN THRESHOLD	628 TIMES
SURGE IS LESS THAN THRESHOLD AND FLOW EXCEEDS THRESHOLD	1420 TIMES
BOTH SERIES ARE LESS THAN THRESHOLDS	18929 TIMES
WHEN COMPARING SURGE HEIGHT WITH RIVER FLOW ON THE SAME DAY:	
BOTH SERIES EXCEED THRESHOLDS	137 TIMES
SURGE EXCEEDS THRESHOLD AND FLOW IS LESS THAN THRESHOLD	682 TIMES
SURGE IS LESS THAN THRESHOLD AND FLOW EXCEEDS THRESHOLD	1474 TIMES
BOTH SERIES ARE LESS THAN THRESHOLDS	18875 TIMES
WHEN COMPARING RIVER FLOW WITH SURGE HEIGHT 1 DAY LATER:	
BOTH SERIES EXCEED THRESHOLDS	91 TIMES
SURGE EXCEEDS THRESHOLD AND FLOW IS LESS THAN THRESHOLD	728 TIMES
SURGE IS LESS THAN THRESHOLD AND FLOW EXCEEDS THRESHOLD	1520 TIMES
BOTH SERIES ARE LESS THAN THRESHOLDS	18829 TIMES
WHEN COMPARING RIVER FLOW WITH SURGE HEIGHT 2 DAYS LATER:	
BOTH SERIES EXCEED THRESHOLDS	94 TIMES
SURGE EXCEEDS THRESHOLD AND FLOW IS LESS THAN THRESHOLD	725 TIMES
SURGE IS LESS THAN THRESHOLD AND FLOW EXCEEDS THRESHOLD	1517 TIMES
BOTH SERIES ARE LESS THAN THRESHOLDS	18832 TIMES
WHEN COMPARING RIVER FLOW WITH SURGE HEIGHT 3 DAYS LATER:	
BOTH SERIES EXCEED THRESHOLDS	88 TIMES
SURGE EXCEEDS THRESHOLD AND FLOW IS LESS THAN THRESHOLD	731 TIMES
SURGE IS LESS THAN THRESHOLD AND FLOW EXCEEDS THRESHOLD	1523 TIMES
BOTH SERIES ARE LESS THAN THRESHOLDS	18826 TIMES

Table C-35

Average Surges[1] Coincident with Given Passaic River Flows

Flow Range (cfs)	Surge (feet) 2 days Prior	1 day Prior	Same Day	Maximum 72 Hours
0.- 1000.	.90	.90	.88	1.19
1000.- 2000.	.99	1.00	1.02	1.38
2000.- 3000.	1.05	1.04	1.06	1.46
3000.- 4000.	1.04	1.02	1.12	1.46
4000.- 5000.	1.01	.98	1.16	1.47
5000.- 6000.	1.07	1.16	1.21	1.55
6000.- 7000.	1.28	1.21	1.25	1.61
7000.- 8000.	1.15	1.03	1.16	1.44
8000.- 9000.	1.13	1.27	1.40	1.64
9000.- 10000.	1.18	1.33	1.42	1.65
10000.- 11000.	1.65	1.61	1.51	1.85
11000.- 12000.	1.36	1.63	1.64	1.91
12000.- 13000.	1.42	1.32	1.27	1.42
13000.- 14000.	1.60	1.40	1.64	1.75
14000.- 15000.	1.91	1.32	1.49	2.04
15000.- 16000.	--- no flows in record ---			
16000.- 17000.	2.16	2.23	1.91	2.23
17000.- 18000.	--- no flows in record ---			
18000.- 19000.	1.32	1.11	1.05	1.32
> 19000.	--- no flows in record ---			

Note: [1]Surge in storm tide minus astronomic tide.

Table C-36

Average Passaic River Flows for Given Surge Heights

Surge[1] height (ft)	Passaic River Flow (cfs) 2 days Later	1 day Later	Same Day	Maximum 72 Hours
-2.50	101.	116.	108.	116.
-2.00	1522.	1751.	1836.	1858.
-1.50	1335.	1351.	990.	1459.
-1.00	1213.	1247.	1095.	1400.
-.50	1126.	1207.	1174.	1339.
.00	888.	942.	938.	1053.
.50	889.	931.	946.	1053.
1.00	1193.	1195.	1194.	1373.
1.50	1145.	1125.	1123.	1307.
2.00	1356.	1293.	1276.	1527.
2.50	1673.	1507.	1507.	1855.
3.00	1876.	1518.	1449.	2013.
3.50	1717.	1324.	1286.	1837.
4.00	1715.	1187.	1150.	1805.
4.50	901.	919.	910.	1002.
5.00	1782.	1561.	1534.	1818.
5.50	2602.	1002.	1018.	2629.
6.00	1356.	363.	387.	1356.
6.50	943.	148.	161.	943.
7.00	--- no surges in record ---			
7.50	--- no surges in record ---			
8.00	--- no surges in record ---			
8.50	1000.	296.	348.	1000.

Note: [1] Surge in storm tide minus astronomic tide.

Table C-37

Passaic River Flows for Selected Storm Events

Storm Event	Passaic River Flow (cfs) 2 days Later	1 day Later	Same Day	Maximum 72 Hours
21 Sept 1938	9,140	7,020	4,590	9,140
25 Nov 1950	3,920	4,130	1,000	4,130
12 Sept 1960	5,040	4,910	2,190	5,040
10 Nov 1977	5,310	7,130	8,960	8,960
7 Apr 1984	13,200	16,000	18,000	18,000

Table C-38

Selected Storm Events

Storm[1]	Lag (Hours)	Tide No.	Peak Surge (Ft. NVGD)	Incremental Probability of Occurence	Cumulative Probablity
N017	4.5	3	12.25	0.0013	0.0013
H242*	11.5	2	11.75	0.0007	0.0020
H135	15.0	1	11.25	0.0006	0.0026
H203*	8.5	3	11.00	0.0006	0.0032
H144	16.0	5	10.75	0.0007	0.0039
H196*	14.5	1	10.5	0.0008	0.0047
H117	14.5	3	10.25	0.0009	0.0056
N014*	4.5	4	10.0	0.0010	0.0066
H192	20.5	4	9.75	0.0017	0.0083
H200	10.5	4	9.50	0.0021	0.0104
H090	18.5	4	9.25	0.0024	0.0128
N015	10.5	4	9.08	0.0027	0.0155
N001	4.0	3	8.92	0.0027	0.0182
H184	20.5	5	8.75	0.0041	0.0223
H225	2.0	2	8.58	0.0053	0.0276
H151	13.0	1	8.42	0.0053	0.0329
N020	3.5	3	8.25	0.0075	0.0404
H027	13.0	2	8.08	0.0093	0.0497
H191	10.5	5	7.92	0.0093	0.0590
H235	15.0	5	7.75	0.0120	0.0710
H025	5.0	5	7.58	0.0140	0.0850
H041	10.5	5	7.42	0.0140	0.0990
H250	7.0	3	7.25	0.0210	0.1200
N041*	17.5	1	7.125	0.0200	0.1400
H078	21.5	2	7.00	0.0300	0.1700
H008*	22.5	4	6.875	0.0200	0.1900
N028	2.5	3	6.75	0.0600	0.2500
H066*	21.5	3	6.25	0.0700	0.3200
N009	6.5	1	6.25	0.1400	0.4600
H166*	8.5	2	6	0.1300	0.5900
H249	1.5	4	5.75	0.2700	0.8600
N018*	16.0	2	5.5	0.1310	0.9910
H001	1.5	5	5.25	0.0090	1.0000

Note: Hxxx = refers to a hurricane
Nxxx = refers to a notheaster
"*" refers to new storm event for this study

Table C-39

Maximum Predicted Water Elevations (feet) at Selected Nodes
October 1976 Tides and Four Project Scenarios

Node	Year 2050 Conditions			
	Low Flows 1976 Unimproved	100-Year Flows Unimproved	100-Year Flows With Tunnel	100-Year Flows with Tunnel and Levees
Newark Bay				
49 (Newark Airport)	3.75	3.80	3.81	3.81
262 (Kearny Point)	3.78	3.83	3.89	3.89
Passaic River				
173 (East Newark)	3.86	7.06	4.99	4.99
217 (Third River)	3.95	12.60	8.05	8.05
226 (Garfield)	4.06	18.61	13.93	13.93
Hackensack River				
182 (Snake Hill)	3.82	3.92	3.95	3.95
190 (Berry's Creek)	3.82	4.02	4.04	4.04
213 (New Milford)	4.03	10.20	10.21	10.21

Table C-40

Maximum Predicted Water Elevations (feet) at Selected Nodes for Synthetic Hurricanes 135 and Four Project Scenarios

Node	Year 2050 Conditions			
	Low Flows 1976 Unimproved	100-Year Flows Unimproved	100-Year Flows With Tunnel	100-Year Flows with Tunnel and Levees
Newark Bay				
49 (Newark Airport)	13.14	13.41	13.44	13.82
262 (Kearny Point)	13.41	13.86	13.89	14.09
Passaic River				
173 (East Newark)	13.58	14.86	14.89	14.99
217 (Third River)	13.97	16.34	15.80	15.99
226 (Garfield)	14.89	19.75	17.52	17.69
Hackensack River				
182 (Belleville)	12.87	13.23	13.24	13.29
190 (Berry's Creek)	11.36	11.72	11.74	11.78
213 (New Milford)	8.43	11.63	11.66	11.65

Table C-41

Maximum Predicted Water Elevations (feet) at Selected Nodes for Synthetic Hurricanes 249 and Four Project Scenarios

Node	Year 2050 Conditions			
	Low Flows 1976 Unimproved	100-Year Flows Unimproved	100-Year Flows With Tunnel	100-Year Flows with Tunnel and Levees
Newark Bay				
49 (Newark Airport)	5.65	5.70	5.73	5.73
262 (Kearny Point)	5.69	5.75	5.80	5.80
Passaic River				
173 (East Newark)	5.90	8.46	6.67	6.67
217 (Third River)	6.19	13.14	9.17	9.17
226 (Garfield)	7.08	18.74	14.39	14.39
Hackensack River				
182 (Belleville)	5.66	5.75	5.78	5.78
190 (Berry's Creek)	5.54	5.68	5.71	5.71
213 (New Milford)	5.89	10.62	10.63	10.63

Table C-42a

Subarea Pertinent Data
Hackensack River Basin

SUBAREA (A)	PASCACK BROOK (B) INCREM AREA (SQ MI)	PASCACK BROOK (B) ACCUM AREA (SQ MI)	HACKENSACK RIVER INCREM AREA (SQ MI)	HACKENSACK RIVER ACCUM AREA (SQ MI)	CLARK'S UNITGRAPH PARAMETERS TC (HRS)	CLARK'S UNITGRAPH PARAMETERS R (HRS)	BASIN LENGTH (MILES)	BASIN SLOPE (FT/MI)	IMPERVIOSITY (PCT)
Above De Forest Lake Dam			27.50	27.50	2.00	12.00			11
Local inflow			1.90		2.00	12.00			11
Above West Nyack Gage				29.40	7.57	19.00			
Local inflow (incl. Lake Tappan)			19.60		20.00	10.00			11
Above Lake Tappan Dam				49.00					
Local inflow			9.00		2.00	12.00			11
Above Rivervale Gage				58.00	25.60	21.40		4.40	
Local inflow			0.86		2.00	12.00			11
Above Pascack Brook				58.86					
Pascack Brook at Woodcliff Lake Dam	19.40	19.40			12.00	4.00			16
Local inflow	10.20				10.00	4.00			16
Pascack Brook at Westwood Gage		29.60			16.00	7.00		36.20	
Local inflow	1.35				2.00	12.00			16
Pascack Brook at mouth		30.95	30.95						
Including Pascack Brook				89.81					
Dorotockey's Run			4.61		2.00	12.00			11
Dwaar Kill			4.17		2.00	12.00			11
Tenakill Brook			8.75		2.00	12.00			11
Local inflow (incl. Oradell Res.)			6.02		2.00	12.00			11
Above Oradell Reservoir Dam				113.36				4.00	
Local inflow			0.60		2.00	12.00			11
Above New Milford Gage				113.96				4.10	
Hirshfeld Brook			4.48		3.50	6.00			25
Below Hirshfeld Brook				118.44					
Local Inflow (C)			2.84		2.50	3.05			25

Continued

Table C-42a(cont)

Subarea Pertinent Data
Hackensack River Basin

SUBAREA (A)	PASCACK BROOK (B) INCREM AREA (SQ MI)	PASCACK BROOK (B) ACCUM AREA (SQ MI)	HACKENSACK RIVER INCREM AREA (SQ MI)	HACKENSACK RIVER ACCUM AREA (SQ MI)	CLARK'S UNITGRAPH PARAMETERS TC (HRS)	CLARK'S UNITGRAPH PARAMETERS R (HRS)	BASIN LENGTH (MILES)	BASIN SLOPE (FT/MI)	IMPERVIOSITY (PCT)
Above Coles Brook				121.28					
Coles Brook			6.52		3.50	6.00			25
Below Coles Brook				127.80					
Local Inflow (C)			6.49		2.50	3.00			25
Above Overpeck				134.29					
Overpeck Creek Above Tide Dam			16.60		4.00	12.00			25
Including Overpeck Creek Above Tide Dam				150.89					

(A) Main stem of Hackensack River unless noted.

(B) Only tributary basin which has been subdivided and for which recorded hydrographs are available.

(C) Essentially lateral inflow as areas are steepest in their shortest dimension with drainage perpendicular to the main stem.

TABLE C-42b

Optimized Clark Unitgraph Parameters
Hackensack River Basin
New York and New Jersey

HYDROGRAPH (A)	DRAINAGE AREA (B) (sq. mi)	STORM EVENT	OPTIMIZED CLARK PARAMETERS TC (Hrs)	OPTIMIZED CLARK PARAMETERS R (Hrs)	DIFFERENCES BETWEEN COMPUTED AND INPUT HYDROGRAPHS (C) PEAK (D) (Pct)	DIFFERENCES BETWEEN COMPUTED AND INPUT HYDROGRAPHS (C) VOLUME (E) (Pct)	DIFFERENCES BETWEEN COMPUTED AND INPUT HYDROGRAPHS (C) TIMING (F) (Hrs)
Inflow to De Forest Lake (G)	27.5	Apr 84	1.61	11.79	-3.0	.0	3.0
		May 84	1.03	15.34	-24.5	.2	10.7
At U.S.G.S. Gaging Station, West Nyack, N.Y.	29.4	Apr 84	8.75	16.95	15.1	.2	2.0
		May 84	5.69	26.32	8.5	9.8	-2.8
		May 89	8.27	13.95	2.3	.0	-1.4
Inflow to Lake Tappan (G)	49.0	Apr 84	28.39	13.09	27.3	-.5	7.7
		Apr 87	25.35	21.52	16.8	.0	-1.9
		May 89	36.28	6.67	-8.0	-.1	7.9
At U.S.G.S. Gaging Station, Rivervale, N.J.	58.0	Apr 84	22.00	30.00	19.5	2.5	-5.1
		Apr 87	25.52	20.04	13.6	.0	-2.2
		May 89	29.40	14.05	-12.1	-.1	4.5
Inflow to Woodcliff Lake (Pascack Brook) (G)	19.4	Apr 87	8.91	1.91	-8.4	-8.4	-15.1
		May 89	1.06	8.40	-1.3	.0	5.6
At U.S.G.S. Gaging Station, Westwood, N.J. (Pascack Brook)	29.6	Apr 84	15.41	7.64	8.1	-.3	-.2
		Apr 87	15.13	4.57	1.1	-.2	-.8
Inflow to Oradell Reservoir (G)	113.0	Apr 84	4.86	26.51	3.2	.2	-1.8
		May 89	8.28	22.11	-7.2	.0	-.2
At U.S.G.S. Gaging Station, New Milford, N.J.	113.0	Apr 87	18.76	27.86	11.6	.0	5.3
		May 89	12.80	16.56	12.8	.0	5.4

(A) Main Stem of Hackensack River unless otherwise noted.
(B) As published by U.S.G.S.
(C) Computed hydrograph used unitgraph based on optimized Clark parameters.
(D) If negative, computed hydrograph peak was lower.
(E) If negative, computed hydrograph volume was lower.
(F) If negative, computed hydrograph volume accumulated faster than did that of input hydrograph.
(G) Supplied by Hackensack Water Company.

Table C-42c

Drainage Areas Tributary to Hydrodynamic Model Nodes (Square Miles)

Input Node	HEC-1 Point Runoff	Non-Point Runoff Upland	Non-Point Runoff Wetland	Water	Total
206	150.89 (A)				150.89
246		1.58			1.58
245			2.86		2.86
240			5.19		5.19
238		1.25			1.25
205			4.02		4.02
203				0.55	0.55
202		0.95			0.95
200	2.04 (B)	3.00	4.60		9.64
197			3.38		3.38
193				0.63	0.63
190			4.85		4.85
188		2.50	3.12		5.62
184		1.49			1.49
183				1.47	1.47
179		4.39			4.39
Total	152.93	15.16	28.02	2.65	198.76

(A) Hackensack above Overpeck Creek plus Overpeck Creek.
(B) Wolf Creek.

TABLE C-42d

Watershed Parameters
HEC-1 Modeling
Lower Hackensack Basin

Hydrodynamic Model Input Node	Hydrograph Originating From (A)	Area (Sq.Mi.)	Clark Parameters TC (Hrs)	Clark Parameters R (Hrs)	Imperviosity (Pct)
246	Upland	1.58	1.50	2.00	40
245	Wetland	2.86	3.00	3.00	50
240	Wetland	5.19	3.00	3.00	50
238	Upland	1.25	2.00	3.00	30
205	Wetland	4.02	3.00	3.00	50
203	Water	0.55	1.00	0.50	100
202	Upland	0.95	1.00	1.00	40
200	Wolf Creek	2.04	1.50	1.50	40
"	Upland	3.00	2.00	3.00	35
"	Wetland	4.60	3.00	3.00	50
197	Wetland	3.38	3.00	3.00	50
193	Water	0.63	1.00	0.50	100
190	Wetland	4.85	3.00	3.00	50
188	Upland	2.50	1.50	2.50	35
"	Wetland	3.12	3.00	3.00	50
184	Upland	1.49	1.50	2.00	30
183	Water	1.47	1.00	0.50	100
179	Upland	4.39	2.00	3.00	30
		47.87 (B)			

(A) Upland and wetland areas have multiple outlets but were modeled as having single outlets using composite Clark parameters.

(B) Total drainage area of Hackensack River is (47.87+150.89) = 198.76 square miles.

Table C-42e

Time of Travel Estimates
HEC-1 Modeling Reaches
Hackensack River Estuary (A)

Frequency (Unless otherwise noted) (Years)	6-Hour Volumes		Ratio Of Volumes	Initial			Final	
	At New Milford (cfs)	With Tidal Prism (cfs)		Velocity Through Constant X-Sect (B) (fps)	Reach Travel Times (Sec)	Reach Travel Times (Hrs)	Reach Travel Times (C) (Hrs)	Velocity Through Constant X-Sect (fps)
100	7821	8724	9.7	2.91	14500	4.0	6.0	1.94
20	5898	6901	7.5	2.25	18800	5.2	8.0	1.46
10	4717	5620	6.2	1.86	22700	6.3	10.0	1.17
2	2304	3207	3.6	1.08	39100	10.9	12.0	0.99
Prism	-	903 (D)	1.0	0.30	-	-		

(A) New Milford to Overpeck Creek, approximately 8.0 miles.
(B) Directly proportional to ratio of volumes assumed to pass through X-sections.
(C) Apply half above Coles Brook and half below.
(D) Actually for 6.5 hours.

Table C-42f

Instantaneous Peaks (A) and Average Rates of Runoff by Duration

Event Frequency (Years)	Location	1 Hour (cfs)	6 Hours (cfs)	12 Hours (cfs)	24 Hours (cfs)	72 Hours (cfs)
100	New Milford	7905	7821	7601	6902	4259
	US Overpeck Ck	8880	8827	8704	8319	5324
	DS Overpeck Ck	10344	10204	10113	9613	6054
20	New Milford	5962	5898	5733	5207	3214
	US Overpeck Ck	6557	6517	6410	6043	3953
	DS Overpeck Ck	7239	7206	7128	6830	4469
10	New Milford	4767	4717	4587	4169	2578
	US Overpeck Ck	5226	5195	5112	4814	3218
	DS Overpeck Ck	5746	5721	5656	5421	3668
2	New Milford	2327	2304	2242	2042	1265
	US Overpeck Ck	2501	2486	2446	2299	1563
	DS Overpeck Ck	2728	2715	2680	2552	1782

(A) One-hour average flow considered to be instantaneous peak for HEC-1 model time step selected (one hour).

TABLE C-42g
FLOW-DURATION DATA
HACKENSACK RIVER AT WEST NYACK, N.Y.

	HIGHEST MEAN FLOW FOR DURATION INDICATED (A) (CUBIC FEET PER SECOND)						
WATER YEAR (B)	1 DAY	2 DAY	3 DAY	4 DAY	5 DAY	7 DAY	10 DAY
1967	295	285	259	241	233	220	204
1968	643	487	431	363	321	275	217
1969	290	239	182	157	145	125	111
1970	752	601	517	450	383	295	232
1971	533	424	385	354	306	242	184
1972	555	535	413	386	400	379	342
1973	1320	1023	784	659	565	431	344
1974	299	288	258	229	213	207	189
1975	773	675	587	532	468	355	266
1976	494	464	358	287	246	244	212
1977	512	421	360	304	258	215	203
1978	549	508	392	323	278	229	177
1979	808	726	674	634	598	511	402
1980	842	591	490	405	365	294	242
1981	86	56	42	35	35	28	22
1982	209	188	155	132	116	98	98
1983	554	471	377	337	331	273	280
1984	1300	1042	858	727	615	472	354
1985	90	59	45	40	40	40	40
1986	304	224	197	192	175	152	127
1987	595	586	502	448	391	363	311
1988	209	186	154	135	122	111	98
1989	897	607	501	410	350	296	263

(A) Processed from mean daily flow records.
(B) Water Year - 1 October to 30 September of the following year.

Notes: - Drainage Area = 29.4 square miles.
- DeForest Lake upstream of this site.

TABLE C-42h
FLOW-DURATION DATA
HACKENSACK RIVER AT RIVERVALE, N.J.

	HIGHEST MEAN FLOW FOR DURATION INDICATED (A) (CUBIC FEET PER SECOND)						
WATER YEAR (B)	1 DAY	2 DAY	3 DAY	4 DAY	5 DAY	7 DAY	10 DAY
1967	381	378	338	333	328	306	292
1968	772	707	665	594	539	457	360
1969	231	201	173	158	146	138	137
1970	546	520	442	373	316	257	218
1971	630	543	455	425	396	323	254
1972	1030	995	872	784	775	754	675
1973	662	483	415	359	353	306	301
1974	361	327	301	273	265	259	243
1975	1448	1144	920	778	658	506	389
1976	845	689	598	493	418	382	345
1977	892	720	585	510	436	353	285
1978	891	825	704	579	489	399	332
1979	961	798	679	572	497	431	382
1980	1510	1340	1075	907	803	641	510
1981	133	98	98	98	97	97	96
1982	472	347	278	266	257	215	174
1983	1338	1052	890	805	757	634	628
1984	2185	2105	1822	1524	1332	1019	753
1985	372	264	205	170	158	144	120
1986	552	485	389	359	332	282	235
1987	1600	1239	1077	973	856	788	658
1988	349	333	294	259	232	223	207
1989	2030	1580	1159	947	796	629	540
1990	881	841	642	545	480	388	325

(A) Processed from mean daily flow records.
(B) Water Year - 1 October to 30 September of the following year.

Notes: - Drainage Area = 58.0 square miles.
- DeForest Lake and Lake Tappan are upstream of this site.

TABLE C-42i
FLOW-DURATION DATA
PASCACK BROOK AT WESTWOOD, N.J.

WATER YEAR (B)	HIGHEST MEAN FLOW FOR DURATION INDICATED (A) (CUBIC FEET PER SECOND) 1 DAY	2 DAY	3 DAY	4 DAY	5 DAY	7 DAY	10 DAY
1967	545	416	313	262	226	181	157
1968	874	708	515	410	366	300	231
1969	688	436	320	258	218	176	145
1970	702	650	495	402	342	269	209
1971	1770	1226	979	779	648	492	369
1972	914	583	454	450	431	393	327
1973	1100	848	631	508	429	338	277
1974	307	233	220	186	164	154	152
1975	760	599	563	469	410	325	248
1976	645	481	359	290	245	236	190
1977	1080	835	606	487	406	312	238
1978	616	453	389	338	292	225	174
1979	683	548	453	436	455	403	309
1980	506	406	368	345	304	256	203
1981	652	439	327	260	227	184	142
1982	265	200	146	138	135	111	91
1983	442	389	354	339	330	263	266
1984	739	616	530	474	435	382	317
1985	385	267	198	162	138	112	107
1986	335	244	183	214	186	154	124
1987	671	521	440	382	330	316	262
1988	318	273	216	188	181	148	131
1989	1040	819	611	491	415	326	282
1990	568	397	305	250	224	208	177

(A) Processed from mean daily flow records.
(B) Water Year - 1 October to 30 September of the following year.

Notes: - Drainage Area = 29.6 square miles.
- Woodcliff Lake upstream of this site.

TABLE C-42j
FLOW-DURATION DATA
HACKENSACK RIVER AT NEW MILFORD, N.J.

WATER YEAR (B)	HIGHEST MEAN FLOW FOR DURATION INDICATED (A) (CUBIC FEET PER SECOND) 1 DAY	2 DAY	3 DAY	4 DAY	5 DAY	7 DAY	10 DAY
1967	1000	881	763	683	595	486	469
1968	2590	2070	1621	1289	1151	918	673
1969	738	429	297	234	210	233	184
1970	1670	1068	830	786	702	537	396
1971	2420	2000	1787	1520	1291	965	696
1972	2880	2790	2223	2040	1848	1646	1430
1973	2390	1830	1353	1236	1053	829	629
1974	1060	745	616	533	530	529	524
1975	4010	2820	2317	1910	1639	1211	871
1976	1770	1765	1382	1077	878	848	694
1977	3110	2508	1804	1425	1148	831	588
1978	1690	1605	1376	1220	1018	766	591
1979	1910	1755	1441	1283	1295	1277	960
1980	2660	2025	1760	1531	1326	1070	831
1981	71	59	44	34	27	19	13
1982	979	560	427	426	368	286	210
1983	2196	1895	1616	1472	1390	1114	1112
1984	4230	3945	3330	2878	2489	1914	1405
1985	503	393	368	362	331	253	182
1986	698	636	599	554	509	439	346
1987	2870	2315	2007	1755	1499	1327	1067
1988	636	395	344	311	283	274	221
1989	4200	3200	2607	2075	1708	1268	1084
1990	2590	1619	1239	978	874	815	673

(A) Processed from mean daily flow records.
(B) Water Year - 1 October to 30 September of the following year.

Notes: - Drainage Area = 113.0 square miles.
- DeForest Lake, Lake Tappan, Woodcliff and Oradell Reservoir are upstream of this site.

TABLE C-42k
STATISTICAL PARAMETERS (A)
FLOW DURATION DATA - STATION DATA
HACKENSACK RIVER BASIN GAGING STATIONS
NEW JERSEY AND NEW YORK

STREAM AND LOCATION	USGS STATION ID #	FLOW DURATION IN DAYS	STATISTICAL PARAMETER		
			MEAN	STANDARD DEVIATION	SKEW
Hackensack River at West Nyack, New York (DA=29.4 sq mi)	01376800	1	2.658	0.3158	-0.8679
		2	2.591	0.3059	-1.3017
		3	2.548	0.2305	-0.4252
		4	2.490	0.2276	-0.4260
		5	2.408	0.3000	-1.3947
		7	2.344	0.2793	-1.4144
		10	2.279	0.2551	-1.4102
Hackensack River at Rivervale, New Jersey (DA=58.0 sq mi)	01377000	1	2.853	0.3014	-0.4672
		2	2.804	0.2671	-0.0690
		3	2.711	0.2962	-0.4991
		4	2.656	0.2829	-0.4633
		5	2.614	0.2712	-0.4274
		7	2.551	0.2521	-0.3291
		10	2.488	0.2378	-0.2696
Pascack Brook at Westwood, New Jersey (DA=29.6 sq mi)	01377500	1	2.796	0.1992	-0.0074
		2	2.677	0.1977	-0.0931
		3	2.577	0.1973	-0.2472
		4	2.514	0.1841	-0.2566
		5	2.462	0.1807	-0.2385
		7	2.384	0.1774	-0.3309
		10	2.301	0.1647	-0.3217
Hackensack River at New Milford, New Jersey (DA=113.0 sq mi)	01378500	1	3.238	0.2830	-0.5688
		2	3.128	0.3136	-0.6483
		3	3.041	0.3113	-0.6415
		4	2.981	0.3011	-0.6798
		5	2.927	0.2936	-0.6887
		7	2.852	0.2729	-0.5773
		10	2.752	0.2786	-0.6304

(A) Above parameters define the station data lines through the scattered data points on Figures C-86a through C-86d. The figures generally follow the example in EC 1110-2-249 (Figure 3-05), "Engineering and Design, Hydrologic Frequency Analysis ", dated 5 June 1985.

TABLE C-421
STATISTICAL PARAMETERS (A)
FLOW DURATION DATA - SMOOTH DATA
HACKENSACK RIVER BASIN GAGING STATIONS
NEW JERSEY AND NEW YORK

STREAM AND LOCATION	USGS STATION ID #	FLOW DURATION IN DAYS	STATISTICAL PARAMETER		
			MEAN	STANDARD DEVIATION	SKEW
Hackensack River at West Nyack, New York	01376800	1	2.658	0.288	-0.706
		2	2.591	0.283	-0.826
		3	2.548	0.279	-0.903
		4	2.490	0.275	-1.006
(DA=29.4 sq mi)		5	2.408	0.268	-1.151
		7	2.344	0.263	-1.266
		10	2.279	0.258	-1.382
Hackensack River at Rivervale, New Jersey	01377000	1	2.853	0.298	-0.363
		2	2.804	0.291	-0.363
		3	2.711	0.279	-0.361
		4	2.656	0.271	-0.361
(DA=58.0 sq mi)		5	2.614	0.265	-0.360
		7	2.551	0.257	-0.359
		10	2.488	0.248	-0.358
Pascack Brook at Westwood, New Jersey	01377500	1	2.796	0.205	-0.037
		2	2.677	0.196	-0.116
		3	2.577	0.189	-0.182
		4	2.514	0.185	-0.224
(DA=29.6 sq mi)		5	2.462	0.181	-0.259
		7	2.384	0.175	-0.311
		10	2.301	0.170	-0.366
Hackensack River at New Milford, New Jersey	01378500	1	3.238	0.305	-0.620
		2	3.128	0.300	-0.626
		3	3.041	0.296	-0.631
		4	2.981	0.293	-0.634
(DA=113.0 sq mi)		5	2.927	0.291	-0.637
		7	2.852	0.287	-0.641
		10	2.752	0.283	-0.647

(A) Above parameters define the regression lines through the scattered data points on Figures C-86a through C-86d. The figures generally follow the example in EC 1110-2-249 (Figure 3-05), "Engineering and Design, Hydrologic Frequency Analysis ", dated 5 June 1985.

Table C - 43a

Pertinent Data
Levees, Floodwalls and Closure Structures
Interior Flood Control Provisions

Group	Facility	Drain Area Name (A)	Pond Area (acres)	Pond Volume (ac-ft)	Total Length of Prot Pond Damage St El (ft NGVD)	Levee Length Elev to Buy to (ft NGVD)	Flood-wall Length	# and Aggr Length Clo Str	Avg Hgt Levee #/Size Culv (C) (inches)	Avg Hgt Flood-wall # Sluice Gates	Total Length Drainage Ditches # Flap Gates	Avg Levee Base Width	Number and Size of Pump Sta (cfs)
Central Basin Levees	Passaic 810		10.0	25.8	4630 ft 176.5	4750 ft 176.0	80 ft	none	8.0 ft 2-24 2-36	none 2 2	none 2 2	34 ft	2 @ 3
	Pinch Bk		10.5	19.1	2812 ft 177.0	2397 ft 176.0	415 ft	none	(D) 2-36	9.4 ft 2	none 2	(E)	1 @ 3
	Rockaway #1 (DS)	AF-HI	38.1	38.3	2421 ft 173.5	2421 ft 173.0	none	none	10.3 ft [2-120X72] [1- 38X54]	none oult 1	none oult 1	72 ft	none
		G	7.4	13.7	175.0	173.0			1-36	1	1		1 @ 1
	Rockaway #1 (DS)	ABC	7.5	9.2	1339 ft 173.0	818 ft 173.0	521 ft	none	5.9 ft 2-48	3.3 ft 2	none 2	45 ft	none
		DE	7.5	9.2	172.0	173.0			2-48	2	2		none
		F	15.0	21.1	176.0	172.0			1-24 2-36	1 2	1 2		none
		GH	16.6	23.8	174.0	174.0			2-36	2	2		none
		I	none	N/A	N/A	N/A			2-24	2	2		none

Table C - 43a (Cont'd)

Pertinent Data
Levees, Floodwalls and Closure Structures
Interior Flood Control Provisions

Group	Facility	Drain Area Name (A)	Pond Area (acres)	Pond Volume (ac-ft)	Total Length of Prot Pond Damage St El (ft NGVD)	Levee Length Elev to Buy to (ft NGVD)	Flood-wall Length	# and Aggr Length Clo Str	Avg Hgt Levee #/Size Culv (C) (inches)	Avg Hgt Flood-wall # Sluice Gates	Total Length Drainage Ditches # Flap Gates	Avg Levee Base Width	Number and Size of Pump Sta (cfs)
Central Basin Levees (Cont'd)	Rockaway @2		41.8	107.0	3172 ft 173.5	3172 ft 173.0	none	none	10.0 ft (1-288X72)	none mult	none mult	70 ft	1 @ 10
	Rockaway @3				7527 ft	825 ft	6638 ft	4/64 ft	7.0 ft	8.5 ft @	none	45 ft	
	Passaic @2A				9343 ft	6216 ft	3082 ft	1/45 ft	7.0 ft	5.5 ft	none	52 ft	
		ABC	42.5	57.2	168.5	168.0			[1- 42] [1- 84] 1- 24 3- 42 1- 48 1-102	1 1 1 3 1 1	1 1 1 3 1 1		1 @ 5
		FF-K	117.0	187.5	168.0	168.0			[5- 36] [3- 60] 2- 84	5 3 2	5 3 2		none
		E-Fnew	53.3	57.5	168.5	168.0			[1- 42] [1- 84]	1 1	1 1		none
		XY	26.0	34.5	169.5	169.0			1- 36 1- 60	1 1	1 1		1 @ 3
		ZZ	none	N\A	N/A	N/A			2- 24 1- 36	2 1	2 1		1 @ 1
		Z	3.9	5.7	168.0	168.0			[1- 60]	1	1		1 @ 2

Table C - 43a (Cont'd)

Pertinent Data
Levees, Floodwalls and Closure Structures
Interior Flood Control Provisions

Group	Facility	Drain Area Name (A)	Pond Area (acres)	Pond Volume (ac-ft)	Total Length of Prot Pond Damage St El (ft NGVD)	Levee Length Elev to Buy to (ft NGVD)	Flood-wall Length	# and Aggr Length Clo Str	Avg Hgt Levee #/Size Culv (C) (inches)	Avg Hgt Flood-wall # Sluice Gates	Total Length Drainage Ditches # Flap Gates	Avg Levee Base Width	Number and Size of Pump Sta (cfs)
Peapton Levees	Basapo - Pequann				5110 ft	2200 ft	2910 ft	none	5.9 ft	5.7 ft	none	45 ft	
		RP1	8.5	23.0	186.0	184.5			(2- 24)	2	2		1 @ 3
		RP1A	0.3	0.5	182.0	182.0			3- 27	3	3		none
		RP2	0.4	0.3	182.0	181.5			(1- 24)	1	1		none
		RP3	5.0	4.0	181.0	181.0			2- 24	2	2	@	none
									2- 36	2	2		
Hurricane Levees	South 1st Street				7700 ft	1750 ft	5700 ft	8/250 ft	6.5 ft	6.2 ft	5635 ft	50 ft	
		S1	none	N/A	N/A	N/A			5- 24	5	5		1 @ 75
									1- 48	1	1		
		S2	none	N/A	N/A	N/A			3- 24	3	3		1 @ 70
									1- 36	1	1		
		S3	none	N/A	N/A	N/A			3- 24	3	3		1 @ 30
									1- 36	1	1		
	Kearny				38304 ft	3900 ft	33771 ft	19/625 ft	5.2 ft	7.4 ft	135129 ft	41 ft	
		K1	none	N/A	N/A	N/A			13- 24	13	13		1 @ 75
									2- 36	2	2		
		K2	none	N/A	N/A	N/A			1- 48	1	1		
									2- 24	2	2		
		K3	none	N/A	N/A	N/A			(1- 66)	1	1		
									10- 24	10	10		
		K4	none	N/A	N/A	N/A			4- 36	4	4		
									35- 24	35	35		

Table C - 43a (Cont'd)

Pertinent Data
Levees, Floodwalls and Closure Structures
Interior Flood Control Provisions

Group	Facility	Drain Area Name (A)	Pond Area (acres)	Pond Volume (ac-ft)	Total Length of Prot Pond Damage St El (ft NGVD)	Levee Length Elev to Buy to (ft NGVD)	Flood-wall Length	# and Aggr Length Cle Str	Avg Hgt Levee #/Size Culv (C) (inches)	Avg Hgt Flood-wall # Sluice Gates	Total Length Drainage Ditches # Flap Gates	Avg Levee Base Width	Nuaber and Size of Pump Sta (cfs)
Hurricane Levees (Cont'd)	Lister-Turnpike-Doreaus				23914 ft	5599 ft	17657 ft	28/658 ft	5.5 ft	8.1 ft	(B)	44 ft	
		List 1	none	N/A	N/A	N/A			5- 24	5	5		none
									1- 36	1	1		
		List 2	none	N/A	N/A	N/A			[1- 72]	1	1		1 @ 100
		List 3	none	N/A	N/A	N/A			7- 24	7	7		
									[1- 24]	1	1		1 @ 50
									6- 24	6	6		
									1- 48	1 @ 1	1		
		Tpke	none	N/A	N/A	N/A			7- 24	7	7		none
									1- 48	1	1		
		Dor 1	none	N/A	N/A	N/A			[1- 60]	1	1		none
									4- 24	4	4		
		Dor 2	none	N/A	N/A	N/A			6- 24	6	6		none
									1- 48	1	1		
		Dor 3A	none	N/A	N/A	N/A			[1- 60]	1	1		none
		Dor 4	none	N/A	N/A	N/A			7- 24	7	7		none
									2- 36	2	2		
		Dor 5	none	N/A	N/A	N/A			9- 24	9	9		none
									1- 36	1	1		
	Total Len				106472 ft	34856 ft	70774 ft	1642 ft					

(A) If more than one.
(B) A=14597 ft; B=8152 ft.
(C) Brackets denote existing culvert(s) to be incorported into plan of improvement. Where a single dimension is given, the culvert is of circular cross-section.
(D) Southerly section, about 8 ft; northerly section, about 9 ft.
(E) Southerly section, about 60 ft; northerly section, about 25 ft.

Table C - 43b

07-Aug-95

Proposed Closure Facilities

Hurricane Levees

South 1st Street (Passaic River at Station 261+53) Sheet 1 of 2

Levee Location (Station)	Gate Width (Feet)	Top of Protection (NGVD)	Gate Height (Feet)	Gate Sill Elevation (NVGD)	W/Project in Place Water Level Frequency (%)	W/Project in Place Return Period (Yes)
0+40	40	14.9	6.4	8.5	6	16.7
10+00	30	14.9	6.9	8.0	10	10.0
11+40	30	14.9	7.4	7.5	15	6.7
33+30	30	14.9	6.4	8.5	6	16.7
54+80	30	14.9	6.4	8.5	6	16.7
56+40	30	14.9	6.4	8.5	6	16.7
3+40	30	14.9	2.6	12.3	0.4	250.0
12+70	30	14.9	6.7	8.2	8	12.5
Total	250 FT					

Kearny Point (Passaic River at Station 70+00)

Levee Location (Station)	Gate Width (Feet)	Top of Protection (NGVD)	Gate Height (Feet)	Gate Sill Elevation (NVGD)	W/Project in Place Water Level Frequency (%)	W/Project in Place Return Period (Yes)
Section FK-1						
4+20	15	14.9	7.0	7.9	9	11.1
13+30	15	14.9	8.5	6.4	40	2.5
16+10	15	14.9	8.5	6.4	40	2.5
18+80	35	14.9	8.5	6.4	40	2.5
30+00	25	14.9	7.0	7.9	9	11.1
33+30	50	14.9	8.5	6.4	40	2.5
34+30	50	14.9	8.5	6.4	40	2.5
35+30	50	14.9	8.5	6.4	40	2.5
88+60	30	14.9	6.9	8.0	8	12.5
91+80	30	14.9	7.2	7.7	10	10.0
176+20	30	14.9	8.4	6.5	38	2.6
178+30	30	14.9	8.4	6.5	38	2.6
195+00	30	14.9	8.9	6.0	100	1.0
201+20	35	14.9	8.9	6.0	100	1.0
208+10	35	14.9	13.0	1.0	100	1.0
237+75	60	14.9	5.0	9.9	2	50.0
238+50	30	14.9	5.0	9.9	2	50.0
Section FP-1						
14+92	30	14.9	1.5	13.4	0.2	500.0
Section FP-1						
25+25	30	14.9	6.7	8.2	7	14.3
Total	625 FT					

Table C - 43b (Continued)

08-Aug-95

Proposed Closure Facilities

Lister/Turnpike/Doremus Floodwall/Levee System
(Passaic River at Station 70+00)

Levee Location (Station)	Gate Width (Feet)	Top of Protection (NGVD)	Gate Height (Feet)	Gate Sill Elevation (NVGD)	W/Project in Place Water Level Frequency (%)	W/Project in Place Return Period (Yes)
Section A						
23+00	30	14.9	7.5	7.4	16	6.3
65+00	30	14.9	6.9	8.0	8	12.5
66+50	30	14.9	8.1	6.8	30	3.3
91+60	30	14.9	6.4	8.5	5	20.0
94+00	30	14.9	6.4	8.5	5	20.0
131+40	30	14.9	9.9	5.0	100	1.0
138+00	30	14.9	7.9	7.0	22	4.5
145+00	30	14.9	8.9	6.0	100	1.0
Subtotal	240	FT				
Section B						
163+00	30	14.9	8.4	6.5	35	2.9
164+10	30	14.9	6.9	8.0	8	12.5
169+50	30	14.9	11.4	3.5	100	1.0
177+80	30	14.9	8.9	6.0	100	1.0
179+50	30	14.9	8.9	6.0	100	1.0
182+00	30	14.9	9.2	5.7	100	1.0
185+10	30	14.9	8.2	6.7	32	3.1
193+00	4	14.9	7.4	7.5	12	8.3
193+40	10	14.9	7.6	7.3	18	5.6
194+25	4	14.9	8.3	6.6	35	2.9
197+20	10	14.9	5.9	9.0	3.5	28.6
200+30	30	14.9	5.9	9.0	3.5	28.6
212+00	10	14.9	8.6	6.3	100	1.0
214+00	10	14.9	7.9	7.0	22	4.5
214+50	20	14.9	7.4	7.5	12	8.3
215+50	10	14.9	7.4	7.5	12	8.3
217+50	10	14.9	7.9	7.0	22	4.5
219+80	30	14.9	5.4	9.5	3	33.3
221+00	30	14.9	5.4	9.5	3	33.3
226+25	30	14.9	5.9	9.0	3.5	28.6
Subtotal	418	FT				
Total	658	FT				

Central Basin Levees

Passaic #2A

Levee Location (Station)	Gate Width (Feet)	Top of Protection (NGVD)	Gate Height (Feet)	Gate Sill Elevation (NVGD)	W/Project in Place Water Level Frequency (%)	W/Project in Place Return Period (Yes)
Horseneck Road	45	174.4	5.4	169.0	10.7	9.3
Total	45	FT				

Rockaway #3

Levee Location (Station)	Gate Width (Feet)	Top of Protection (NGVD)	Gate Height (Feet)	Gate Sill Elevation (NVGD)	W/Project in Place Water Level Frequency (%)	W/Project in Place Return Period (Yes)
30+00	12	184.1	8.6	175.5	33	3.0
47+30	20	183.8	9.2	174.6	40	2.5
50+10	20	183.7	9.7	174.0	50	2.0
55+70	12	183.6	8.1	175.5	20	5.0
Total	64	FT				

Table C – 44

30 August 95

Comparisons of Water Surface Elevations for Recommended Plan
Existing, Future w/o Project, Future w/ Project

Location	Station #		Hypothetical Events 1 Year	2 Year	5 Year	10 Year	25 Year	50 Year	100 Year	500 Year	Observed April 1984
UPPER TRIBUTARIES											
Pequannock River at Wanaque River	9R10630	Existing	183.5	185.6	187.4	188.5	189.4	190.1	191.0	192.9	191.9
		Future w/o	183.9	185.8	188.0	188.8	189.6	190.3	191.3	193.1	
		Future w/	178.7	179.3	181.6	182.6	183.5	184.4	185.6	188.7	
		Difference	−5.2	−6.5	−6.4	−6.2	−6.1	−5.9	−5.7	−4.4	
Wanaque River at Mouth	8R255	Existing	183.7	185.6	187.4	188.5	189.4	190.1	191.0	192.9	191.9
		Future w/o	183.9	185.8	188.0	188.8	189.6	190.3	191.3	193.1	
		Future w/	178.7	179.3	181.6	182.6	183.5	184.4	185.6	188.7	
		Difference	−5.2	−6.5	−6.4	−6.2	−6.1	−5.9	−5.7	−4.4	
Ramapo River	11R7800	Existing	179.9	181.7	184.1	185.7	188.0	188.9	189.8	191.1	188.6
		Future w/o	180.2	182.1	184.5	186.1	188.2	189.1	189.9	191.3	
		Future w/	176.4	176.3	176.8	180.0	182.1	183.2	185.0	187.7	
		Difference	−1.8	−3.8	−5.7	−6.1	−6.1	−5.9	−4.9	−3.6	
Rockaway River	1R23680	Existing	172.7	173.5	175.1	175.8	177.5	178.5	179.6	182.0	176.5
		Future w/o	173.1	173.9	175.6	176.4	178.2	179.3	180.6	183.4	
		Future w/	173.1	174.0	175.6	176.5	178.4	179.5	180.8	183.6	
		Difference	0.0	0.1	0.0	0.1	0.2	0.2	0.2	0.2	
Rockaway River	1R11900	Existing	169.9	170.7	172.0	172.7	173.8	174.8	175.8	178.1	173.2
		Future w/o	170.2	171.0	172.3	173.0	174.1	175.1	176.0	178.4	
		Future w/	170.2	171.0	172.3	173.0	174.1	175.0	175.8	177.8	
		Difference	0.0	0.0	0.0	0.0	0.0	−0.1	−0.2	−0.6	
Whippany River	3R4060	Existing	177.4	178.0	179.0	180.0	180.8	181.6	182.2	184.1	180.1
		Future w/o	177.7	178.3	179.4	180.3	181.0	181.8	182.4	184.4	
		Future w/	177.7	178.3	179.5	180.3	181.1	181.8	182.4	184.4	
		Difference	0.0	0.0	0.1	0.0	0.1	0.0	0.0	0.0	
TRIBUTARIES											
Pompton River above Main Inlet	14R35775	Existing	174.7	177.6	180.7	182.0	183.5	184.3	185.4	187.2	184.7
		Future w/o	175.2	178.0	181.0	182.2	183.7	184.5	185.6	187.5	
		Future w/	174.6	174.6	174.7	174.5	174.8	174.8	174.9	180.9	
		Difference	−0.6	−3.4	−6.3	−7.7	−8.9	−9.7	−10.7	−6.6	
Pompton River at Pompton Plains	14R35090	Existing	174.3	177.3	180.4	181.7	183.2	184.1	185.1	186.9	184.4
		Future w/o	174.8	177.7	180.7	182.0	183.4	184.2	185.3	187.1	
		Future w/	174.4	174.5	174.5	174.6	175.0	175.0	175.5	181.2	
		Difference	−0.4	−3.2	−6.2	−7.4	−8.4	−9.2	−9.8	−5.9	
Pompton River at Mouth	14R222	Existing	164.5	166.1	168.1	169.3	170.9	172.1	173.2	175.6	171.1
		Future w/o	164.7	166.4	168.4	169.6	171.2	172.3	173.5	176.1	
		Future w/	164.3	164.3	164.3	164.3	164.3	164.4	165.2	170.1	
		Difference	−0.4	−2.1	−4.1	−5.3	−6.9	−7.9	−8.3	−6.0	
Deepavaal at Mouth	16R180	Existing	163.9	165.5	167.4	168.5	170.0	171.2	172.4	175.0	170.2
		Future w/o	164.2	165.7	167.7	168.8	170.3	171.4	172.7	175.3	
		Future w/	163.9	164.0	164.0	164.0	164.1	164.1	164.9	169.4	
		Difference	−0.3	−1.7	−3.7	−4.8	−6.2	−7.3	−7.8	−5.9	
Saddle	16R2695	Existing	10.6	12.4	14.3	15.6	17.4	18.4	20.0	22.7	16.1
		Future w/o	10.7	12.4	14.3	15.6	17.5	18.4	20.0	22.7	
		Future w/	10.8	12.5	14.4	15.7	17.5	18.4	20.0	22.4	
		Difference	0.1	0.1	0.1	0.1	0.0	0.0	0.0	−0.3	

30 August 95

Comparisons of Water Surface Elevations for Recommended Plan
Existing, Future w/o Project, Future w/ Project

Location	Station #		Hypothetical Events 1 Year	2 Year	5 Year	10 Year	25 Year	50 Year	100 Year	500 Year	Observed April 1984
PASSAIC RIVER											
at Chatham	6R315800	Existing	199.9	200.5	201.3	201.9	202.6	203.0	203.6	204.5	201.2
		Future w/o	200.1	200.7	201.5	202.1	202.7	203.2	203.7	204.6	
		Future w/	200.1	200.7	201.5	202.1	202.7	203.2	203.7	204.6	
		Difference	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
at Pine Brook	7R234650	Existing	167.7	168.4	169.6	170.5	171.6	172.6	173.6	176.3	171.7
		Future w/o	167.9	168.6	169.8	170.8	172.0	173.0	174.0	176.6	
		Future w/	168.0	168.7	169.8	170.5	171.4	172.1	172.8	174.5	
		Difference	0.1	0.1	0.0	−0.3	−0.6	−0.9	−1.2	−2.1	
Above Pompton River	7R173630	Existing	164.5	166.1	168.1	169.3	170.9	172.1	173.2	175.6	171.1
		Future w/o	164.7	166.4	168.4	169.6	171.2	172.3	173.5	176.1	
		Future w/	164.3	164.3	164.3	164.3	164.3	164.4	165.2	170.1	
		Difference	−0.4	−2.1	−4.1	−5.3	−6.9	−7.9	−8.3	−6.0	
Below Pompton River	15R173500	Existing	164.5	166.1	168.1	169.3	170.9	172.1	173.2	175.6	171.1
		Future w/o	164.7	166.4	168.4	169.6	171.2	172.3	173.5	176.1	
		Future w/	164.3	164.3	164.3	164.3	164.3	164.4	165.2	170.1	
		Difference	−0.4	−2.1	−4.1	−5.3	−6.9	−7.9	−8.3	−6.0	
Below Two Bridges Inlet	15R172140	Existing	164.2	165.8	167.8	169.0	170.6	171.8	172.9	175.6	170.8
		Future w/o	164.5	166.1	168.1	169.3	170.9	172.0	173.2	175.8	
		Future w/	164.2	164.3	164.3	164.3	164.3	164.4	165.2	170.0	
		Difference	−0.3	−1.8	−3.8	−5.0	−6.6	−7.6	−8.0	−5.8	
at Little Falls	17R153860	Existing	126.3	127.9	129.9	131.1	132.6	133.9	135.1	137.6	133.0
		Future w/o	126.5	128.2	130.2	131.4	133.0	134.1	135.4	138.0	
		Future w/	126.4	126.4	126.5	126.6	126.8	127.1	127.3	132.2	
		Difference	−0.1	−1.8	−3.7	−4.8	−6.2	−7.0	−8.1	−5.8	
at Great Falls	17R133865	Existing	117.9	118.8	120.1	120.9	121.8	122.7	123.6	125.6	122.0
		Future w/o	118.0	119.0	120.3	121.1	122.0	122.9	123.8	125.7	
		Future w/	117.9	118.0	118.1	118.2	118.7	119.4	119.6	121.6	
		Difference	−0.1	−1.0	−2.2	−2.9	−3.3	−3.5	−4.2	−4.1	
at Dundee Dam	17R91790	Existing	27.7	28.5	29.4	30.0	30.7	31.3	31.9	33.4	30.8
		Future w/o	27.9	28.6	29.5	30.1	30.8	31.4	32.0	33.5	
		Future w/	27.8	27.9	28.0	28.5	29.2	29.9	30.3	31.5	
		Difference	−0.1	−0.7	−1.5	−1.6	−1.6	−1.5	−1.7	−2.0	
at Saddle River	19R82200	Existing	6.9	7.9	9.3	10.5	12.4	13.8	15.2	16.3	12.1
		Future w/o	7.5	8.4	9.8	11.0	12.8	14.1	15.6	16.6	
		Future w/	7.4	7.8	8.5	9.8	11.6	13.3	14.5	17.3	
		Difference	−0.1	−0.6	−1.3	−1.2	−1.2	−0.8	−1.1	−1.3	
at Mouth	19R3200	Existing	5.7	5.7	5.7	5.7	5.7	5.7	5.7	5.7	4.8
		Future w/o	6.2	6.2	6.2	6.2	6.2	6.2	6.2	6.2	
		Future w/	6.2	6.2	6.2	6.2	6.2	6.2	6.2	6.2	
		Difference	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	

Table C-45

30 August 95

Comparisons of Peak Flows for Recommended Plan
Existing, Future w/o Project, Future w/ Project

Location	Station #		Hypothetical Events 1 Year	2 Year	5 Year	10 Year	25 Year	50 Year	100 Year	500 Year	Observed April 1984
UPPER TRIBUTARIES											
Pequannock River	9R10630	Existing	440.0	970.0	1660.0	2290.0	2770.0	3310.0	4220.0	6310.0	4640.0
at Wanaque River		Future w/o	500.0	1050.0	1990.0	2420.0	2920.0	3490.0	4430.0	6540.0	
		Future w/	490.0	1050.0	1960.0	2410.0	2910.0	3470.0	4410.0	6530.0	
		Difference	-10.0	0.0	-10.0	-10.0	-10.0	-20.0	-20.0	-10.0	
Wanaque River	8R255	Existing	1120.0	2040.0	3540.0	4270.0	5010.0	5850.0	6750.0	9670.0	9930.0
at Mouth		Future w/o	1220.0	2160.0	3610.0	4450.0	5210.0	6060.0	7420.0	10610.0	
		Future w/	1220.0	2160.0	3730.0	4480.0	5250.0	6090.0	7460.0	10850.0	
		Difference	0.0	0.0	120.0	30.0	40.0	30.0	40.0	240.0	
Ramapo River	11R7800	Existing	2620.0	4300.0	6610.0	8750.0	12930.0	15450.0	19350.0	27100.0	14070.0
		Future w/o	2910.0	4670.0	7070.0	9260.0	13490.0	16030.0	20010.0	27760.0	
		Future w/	2910.0	4670.0	7080.0	9290.0	13510.0	16030.0	20020.0	27770.0	
		Difference	0.0	0.0	10.0	30.0	20.0	0.0	10.0	10.0	
Rockaway River	1R23680	Existing	1120.0	1640.0	2680.0	3600.0	5430.0	6460.0	8160.0	11840.0	4340.0
		Future w/o	1330.0	1880.0	3230.0	4060.0	5930.0	6930.0	8600.0	12290.0	
		Future w/	1330.0	1880.0	3240.0	4090.0	5990.0	7030.0	8740.0	12440.0	
		Difference	0.0	0.0	10.0	30.0	60.0	100.0	140.0	150.0	
Rockaway River	1R11900	Existing	1140.0	1660.0	2920.0	3610.0	5180.0	6440.0	8030.0	11370.0	4230.0
		Future w/o	1350.0	1910.0	3260.0	3960.0	5600.0	6700.0	8210.0	11530.0	
		Future w/	1360.0	1920.0	3290.0	4040.0	5760.0	6910.0	8540.0	12080.0	
		Difference	10.0	10.0	30.0	80.0	160.0	210.0	330.0	530.0	
Whippany River	3R4060	Existing	240.0	360.0	630.0	930.0	1310.0	1720.0	1860.0	2830.0	690.0
		Future w/o	230.0	350.0	640.0	940.0	1310.0	1710.0	1850.0	2820.0	
		Future w/	240.0	350.0	640.0	940.0	1310.0	1710.0	1860.0	2830.0	
		Difference	10.0	0.0	0.0	0.0	0.0	0.0	10.0	10.0	
TRIBUTARIES											
Pompton River	14R35775	Existing	4200.0	7260.0	11660.0	14730.0	20170.0	24060.0	29740.0	42030.0	26260.0
above Main Inlet		Future w/o	4640.0	7790.0	12380.0	15590.0	21090.0	25030.0	31110.0	43550.0	
		Future w/	4800.0	9910.0	12700.0	15960.0	21370.0	25250.0	31440.0	43280.0	
		Difference	160.0	2120.0	320.0	370.0	280.0	220.0	330.0	-270.0	
Pompton River	14R35090	Existing	4200.0	7260.0	11650.0	14720.0	20060.0	24050.0	29730.0	42030.0	26220.0
at Pompton Plains		Future w/o	4670.0	7760.0	12370.0	15570.0	21060.0	25020.0	31110.0	43560.0	
		Future w/	4440.0	4740.0	5030.0	4420.0	6620.0	6330.0	6940.0	13740.0	
		Difference	-230.0	-3040.0	-7340.0	-11150.0	-14460.0	-18690.0	-24170.0	-29820.0	
Pompton River	14R222	Existing	4020.0	6760.0	10620.0	13070.0	18150.0	22160.0	26890.0	39410.0	16300.0
at Mouth		Future w/o	4400.0	7250.0	11160.0	13450.0	19100.0	23060.0	28120.0	40640.0	
		Future w/	4090.0	4530.0	5590.0	5220.0	6170.0	6340.0	9010.0	12330.0	
		Difference	-310.0	-2720.0	-5570.0	-8230.0	-12930.0	-16720.0	-19110.0	-28510.0	
Deepavaal	16R180	Existing	200.0	290.0	420.0	550.0	720.0	1240.0	1980.0	3610.0	750.0
at Mouth		Future w/o	210.0	300.0	440.0	570.0	710.0	1520.0	2170.0	3740.0	
		Future w/	350.0	500.0	710.0	890.0	1090.0	1340.0	1440.0	2010.0	
		Difference	140.0	200.0	270.0	320.0	380.0	-180.0	-730.0	-1730.0	
Saddle	16R2695	Existing	1590.0	2220.0	3070.0	3740.0	4800.0	5400.0	6560.0	8840.0	3810.0
		Future w/o	1600.0	2240.0	3090.0	3760.0	4810.0	5410.0	6570.0	8850.0	
		Future w/	1600.0	2240.0	3090.0	3760.0	4810.0	5420.0	6580.0	8880.0	
		Difference	0.0	0.0	0.0	0.0	0.0	10.0	10.0	30.0	

Table C-45

30 August 95

Comparisons of Peak Flows for Recommended Plan
Existing, Future w/o Project, Future w/ Project

Location	Station #		Hypothetical Events 1 Year	2 Year	5 Year	10 Year	25 Year	50 Year	100 Year	500 Year	Observed April 1984
PASSAIC RIVER											
at Chatham	6R315800	Existing	1150.0	1460.0	1960.0	2450.0	3020.0	3420.0	4030.0	4970.0	1930.0
		Future w/o	1250.0	1560.0	2090.0	2570.0	3150.0	3550.0	4170.0	5120.0	
		Future w/	1250.0	1560.0	2090.0	2570.0	3150.0	3550.0	4170.0	5120.0	
		Difference	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
at Pine Brook	7R234650	Existing	2700.0	3490.0	4910.0	6130.0	7780.0	9200.0	10160.0	13040.0	6180.0
		Future w/o	2960.0	3760.0	5250.0	6540.0	8170.0	10770.0	11220.0	13530.0	
		Future w/	2930.0	3700.0	5200.0	6500.0	8300.0	9660.0	12450.0	16010.0	
		Difference	-50.0	-60.0	-50.0	-40.0	130.0	-910.0	1230.0	2480.0	
Above Pompton River	7R173660	Existing	2890.0	4220.0	6360.0	7970.0	10030.0	11370.0	12490.0	16370.0	7960.0
		Future w/o	3190.0	4580.0	6790.0	8430.0	10420.0	11760.0	12890.0	16790.0	
		Future w/	3070.0	6940.0	5750.0	6720.0	8740.0	10090.0	11980.0	16520.0	
		Difference	-120.0	2360.0	-1040.0	-1710.0	-1680.0	-1670.0	-910.0	1730.0	
Below Pompton River	15R173500	Existing	5140.0	7530.0	11220.0	13950.0	17660.0	20640.0	24230.0	33350.0	16660.0
		Future w/o	5500.0	7960.0	11820.0	14590.0	18510.0	21400.0	24780.0	34230.0	
		Future w/	5270.0	6800.0	10330.0	10910.0	13130.0	14390.0	17860.0	27060.0	
		Difference	-230.0	-1160.0	-1490.0	-3660.0	-5380.0	-7010.0	-6920.0	-7170.0	
Below Two Bridges Inlet	15R172140	Existing	5150.0	7540.0	11230.0	13950.0	17670.0	20650.0	24230.0	33360.0	16670.0
		Future w/o	5500.0	7990.0	11630.0	14600.0	18520.0	21410.0	24780.0	34240.0	
		Future w/	5280.0	5360.0	5450.0	5530.0	5570.0	5600.0	6560.0	15560.0	
		Difference	-220.0	-2630.0	-6360.0	-9070.0	-12950.0	-15810.0	-18220.0	-18660.0	
at Little Falls	17R153660	Existing	5400.0	7790.0	11560.0	14350.0	18450.0	21990.0	26050.0	36260.0	19060.0
		Future w/o	5730.0	8250.0	12170.0	15030.0	19180.0	22650.0	26800.0	37140.0	
		Future w/	5500.0	5600.0	5660.0	5720.0	5660.0	6010.0	6790.0	16930.0	
		Difference	-230.0	-2650.0	-6510.0	-9310.0	-13320.0	-16640.0	-20010.0	-20210.0	
at Great Falls	17R133665	Existing	5360.0	7810.0	11600.0	14400.0	18520.0	22100.0	26180.0	26510.0	19120.0
		Future w/o	5750.0	8260.0	12210.0	15090.0	19260.0	22770.0	26940.0	37360.0	
		Future w/	5530.0	5660.0	5830.0	6190.0	7440.0	9260.0	10140.0	17360.0	
		Difference	-220.0	-2620.0	-6380.0	-8900.0	-11820.0	-13510.0	-16800.0	-20000.0	
at Dundee Dam	17R91790	Existing	5420.0	7860.0	11660.0	14460.0	18640.0	22270.0	36370.0	36540.0	19190.0
		Future w/o	5800.0	8330.0	12280.0	15170.0	19380.0	22940.0	27130.0	37720.0	
		Future w/	5570.0	5760.0	6130.0	7770.0	10690.0	14440.0	16290.0	23960.0	
		Difference	-230.0	-2570.0	-6150.0	-7400.0	-8490.0	-8500.0	-10640.0	-13740.0	
at Saddle River	19R82200	Existing	6090.0	8520.0	12190.0	15000.0	19250.0	22960.0	27800.0	37680.0	19670.0
		Future w/o	6490.0	6960.0	12800.0	15800.0	20020.0	23660.0	27900.0	38790.0	
		Future w/	6270.0	6910.0	9050.0	10670.0	14720.0	19060.0	21540.0	30500.0	
		Difference	-220.0	-70.0	-3750.0	-4730.0	-5300.0	-4580.0	-6360.0	-8290.0	
at Mouth	19R3200	Existing	20960.0	22840.0	25510.0	27500.0	30900.0	34190.0	37340.0	48110.0	33260.0
		Future w/o	21770.0	23740.0	26400.0	28470.0	31960.0	35160.0	38390.0	48920.0	
		Future w/	21650.0	22870.0	24410.0	25970.0	30370.0	34340.0	37590.0	48140.0	
		Difference	-120.0	-870.0	-1990.0	-2500.0	-1610.0	-820.0	-800.0	-780.0	

TABLE C-46

STAGES AND FLOWS
OBSERVED, FUTURE W/O AND IMPROVED CONDITIONS (A)
1903, 1968 AND 1984 FLOODS

FLOOD	COND	POMPTON PLAINS FLOW (CFS)	POMPTON PLAINS STAGE (NGVD)	LITTLE FALLS FLOW (CFS)	LITTLE FALLS STAGE (NGVD)
OCTOBER 1903	W/O PROJ	37838 ( )	186.3 (186.2)	30789 (31675)	136.5 ( )
	W/ PROJ	13043	180.8	10399	129.4
	Change	-24795	-5.5	-20390	-7.1
MAY-JUNE 1968	W/O PROJ	14400 (13100)	182.4 (182.2)	13500 (13500)	130.8 (130.7)
	W/ PROJ	4610	174.6	5810	126.7
	Change	-9790	-7.8	-7690	-4.1
APRIL 1984	W/O PROJ	26220 (25400)	184.4 (184.5)	19100 (18400)	133.0 (132.9)
	W/ PROJ	4740	174.9	5900	127.1
	Change	-21480	-9.5	-13200	-5.9

(A) PARENTHESES DENOTE OBSERVED DATA. WHERE ENTRIES ARE MISSING, DATA IS UNAVAILABLE OR UNRELIABLE.

TABLE C-47

REDUCTIONS IN MAXIMUM WATER SURFACE ELEVATIONS
EXISTING AND IMPROVED CONDITIONS
SELECTED REACHES
PASSAIC RIVER STREAMS
1903, 1968 AND 1984 FLOODS

UNET REACH / STREAM LENGTH			1903 FLOOD (FT)	1968 FLOOD (FT)	1984 FLOOD (FT)
07	PASSAIC - THROUGH GREAT PIECE MEADOWS	AVG	5.2	2.5	3.9
		MAX	7.5	4.7	6.9
		MIN	1.1	0.2	0.4
14	POMPTON - ENTIRE LENGTH	AVG	6.9	5.7	8.8
		MAX	8.0	8.2	10.9
		MIN	5.4	4.4	6.7
15	PASSAIC - POMPTON TO DEEPAVAAL	AVG	7.1	4.3	6.4
		MAX	7.5	4.7	6.8
		MIN	6.9	4.2	6.1
16	DEEPAVAAL - ENTIRE LENGTH	AVG	5.3	1.6	2.8
		MAX	7.0	4.2	6.1
		MIN	3.6	-0.2 *	1.0

* SLIGHT INCREASE RESULTS FROM ADOPTED SCHEME OF TUNNEL GATE OPERATION. REFLECTED IN AVERAGE REDUCTION.

Table C - 48

CH3D River Inflows

Description of Location	Average Discharge (cfs)
Elizabeth River at Northern End of Authur Kill	294
Rahway River	624
Loadings below Rahway River on Arthur Kill	105
Raritan River	810
Loadings at mouth of Raritan River	409
Lower Newark Bay - West Shore	43
Western shore - middle Newark Bay - off Port Elizabeth	74
Newark Bay - Northeast Shore	15
Kill van Kull Loads - South side at Western End	108
Loading at Mouth of Jamaica Bay	0
Norrows Loadings - East Shore	249
Kill van Kull Loads - North shore at Eastern End	0
South of Port Jersey Berth on West Shore of Upper Bay	13
Liberty Park loads on West Shore of Upper Bay	446
Hoboken - West Shore of Hudson River	35
Lower East River - East Shore	0
Above Hells gate - west shore of East River	0
East Shore Hudson near GW Bridge	0
East Shore Hudson River near Dobbs Ferry	0
West Shore Hudson River near Tappanzee Bridge	0
Lower Hackensack River Lateral Inflows	0
Lower Passaic River Lateral Inflows	59
Lower Hackensack River Confluence	70
Upper Hackensack River Confluence	147
Oradell Reservoir - Hackensack Basin	55
Middle Passaic River loadings and Tributaries	324
Passaic River at Dundee Dam	1583
Hackensack River at New Milford	135
Hudson River at Albany	7000

Table C - 49

Estuary Mannings "n" Values

Zone of Model	Shallow Water values*
Default Value for Entire Grid	0.035
Lower Bay - Coney Island Reach	0.011
Jamaica Bay	0.017
Raritan River	0.060
Arthur Kill	0.400
Upper Bay	0.011
Kill van Kull	0.014
Lower Newark Bay	0.200
Upper Newark Bay	0.300
Passaic River	0.500
Hackensack River	0.015
Lower East River	0.050
Hells Gate area of East River	0.050
Upper East River	0.050
Western Long Island Sound	0.010
Harlem River	0.017
Hudson River Manhattan to Haverstraw	0.200
Hudson River - Haverstraw to West Point	0.015
Hudson River - West Point to Kingston	0.125
Hudson River - Kingston to Saugerties	0.250
Hudson River - Saugerties to Catskill	0.800
Hudson River - Catskill to Schodack	0.400
Upper Hudson River - Schodack to Albany	0.070

* n_o as used in Equation 1

Table C-50
Water Quality Model Initial Conditions

Parameter	Value	Units
BENDOb	1.0	gm m^{-2} day^{-1}
KBOD	0.05	day^{-1}
KCOD	20.0	day^{-1}
KHDOB	0.5	gm DO m^{-3}
KHDOC	1.5	gm DO m^{-3}
KHSO	2.0	gm DO m^{-3}
KR	1.0	m day^{-1}
KSO	0.069	$^oC^{-1}$
KTBOD	0.069	$^oC^{-1}$
TRBOD	20.0	^oC
TRSO	20.0	^oC
WBOD	1.0	m day^{-1}

Table C-51

Tunnel Sediment Inlet Coefficients

	Very Fine Sand	Fine Sand	Medium Sand	Coarse Sand
Pompton Inlet	0.83	0.79	0.53	0.09
Spur Inlet	0.73	0.42	0.07	0.00

Table C-52 100 Year Inflow Data

Passaic River Tunnel Trap Efficiency Study

(100 Year Flood)

Time Step (hr)	Discharge Rate (CFS)	Velocity (ft/s)	Sediment Inflow Rate (Tons/Day) VFS * 0.018	FS 0.057	MS 0.131	CS 0.278	VCS 0.481	VFG 0.72	FG 1.04	MG 1.48	CG 2.09	VCG 2.95	SC 4.18	LC 5.91	SB 8.36	Total ----	Sediment Mass (Tons)	Sediment Volume (cy)
6	2	0.001	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
12	2	0.001	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
18	2	0.001	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
24	2	0.001	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
30	2	0.001	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
36	2	0.001	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
42	2	0.001	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
48	2	0.001	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
54	12917	8.12	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
60	22283	14.01	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
66	26453	16.63	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
72	26177	16.46	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
78	23310	14.66	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
84	29007	18.24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
90	26773	16.83	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
96	22295	14.02	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
102	19957	12.55	0.02	0.01	0	0	0	0	0	0	0	0	0	0	0	0.03	0.0075	0.00555
108	18085	11.37	0.4	0.31	0.08	0.01	0	0	0	0	0	0	0	0	0	0.8	0.2	0.148
114	16619	10.45	3.68	2.94	0.65	0.08	0	0	0	0	0	0	0	0	0	7.35	1.8375	1.35975
120	15561	9.78	15.62	13.05	2.27	0.27	0.02	0	0	0	0	0	0	0	0	31.23	7.8075	5.77755
126	14722	9.26	0.05	0.04	0.01	0	0	0	0	0	0	0	0	0	0	0.1	0.025	0.0185
132	13865	8.72	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
138	13180	8.29	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
144	12720	8.00	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
150	12237	7.69	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
156	11758	7.39	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
162	11188	7.03	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
168	11100	6.98	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

* Settling Velocity by Particle Class

Table C-52 (cont.) 100 Year Inflow Data

Passaic River Tunnel Trap Efficiency Study

(100 Year Flood)

Time Step (hr)	Discharge Rate (CFS)	Velocity (ft/s)	Sediment Inflow Rate (Tons/Day) VFS	FS	MS	CS	VCS	VFG	FG	MG	CG	VCG	SC	LC	SB	Total	Sediment Mass (Tons)	Sediment Volume (cy)
			* 0.018	0.057	0.131	0.278	0.481	0.72	1.04	1.48	2.09	2.95	4.18	5.91	8.36	-----		
174	10484	6.59	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
180	9814	6.17	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
186	9121	5.73	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
192	9056	5.69	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
198	8416	5.29	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
204	7757	4.88	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
210	7055	4.44	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
216	7049	4.43	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
222	6392	4.02	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
228	6347	3.99	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
234	5699	3.58	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
240	5007	3.15	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
246	5007	3.15	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
252	4336	2.73	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
258	4320	2.72	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
264	3654	2.30	0.02	0.02	0.01	0	0	0	0	0	0	0	0	0	0	0.05	0.0125	0.00925
270	3640	2.29	0.02	0.02	0.01	0	0	0	0	0	0	0	0	0	0	0.05	0.0125	0.00925
276	2968	1.87	0.2	0.15	0.05	0	0	0	0	0	0	0	0	0	0	0.4	0.1	0.074
282	2965	1.86	0.18	0.14	0.04	0	0	0	0	0	0	0	0	0	0	0.36	0.09	0.0666
288	2944	1.85	0.18	0.14	0.04	0	0	0	0	0	0	0	0	0	0	0.36	0.09	0.0666
294	2286	1.44	1.51	1.13	0.37	0.01	0	0	0	0	0	0	0	0	0	3.02	0.755	0.5587
300	2277	1.43	1.39	1.03	0.35	0.01	0	0	0	0	0	0	0	0	0	2.78	0.695	0.5143
306	1588	1.00	9.76	6.91	2.79	0.07	0	0	0	0	0	0	0	0	0	19.53	4.8825	3.61305
312	1602	1.01	9.39	6.65	2.68	0.07	0	0	0	0	0	0	0	0	0	18.79	4.6975	3.47615
318	1597	1.00	8.31	5.7	2.55	0.06	0	0	0	0	0	0	0	0	0	16.62	4.155	3.0747
324	1588	1.00	7.24	4.72	2.46	0.06	0	0	0	0	0	0	0	0	0	14.48	3.62	2.6788
330	917	0.58	2.25	1.26	0.96	0.02	0	0	0	0	0	0	0	0	0	4.49	1.1225	0.83065

* Settling Velocity by Particle Class

Table C - 53

Model Verification - Shoaling Distribution

Segment Number	Channel Segment	Shoaling (Cu Yd/ Year)	Percentage
1	Kill van Kull	0	0
2	Elizabeth Port Channel	50,200	17
3	South of Shooters Island	39,000	13
4	Newark Bay Channel	17,100	6
5	Bergen Pont Turning Basin	22,000	8
6	Arthur Kill Gulfport Channel	0	0
7	Port Elizabeth Turning Basin	800	0
8	South Elizabeth Channel	19,300	7
9	Ports Elizabeth and Newark	145,000	49
10	North Newark Bay Channel	0	0
11	Passaic River	0	0
12	Hackensack River	0	0
13	Upper Newark Bay	0	0
	Total	293,400	

Table C-54

Riprap Requirements

STATIONING in FEET	RIPRAP LAYER THICKNESS in INCHES		
PASSAIC RIVER	LEFT	BOTTOM	RIGHT
1763+00 to 1757+50	12	12	12
1746+75 to 1746+50	12	12	12
1746+50 to 1745+90	Great Piece Weir		
1745+90 to 1745+50	36	36	36
1745+50 to 1745+00	24	24	24
WANAQUE RIVER			
55+00 to 52+75	12	12	12
52+75 to 51+50	30	36	30
51+50 to 48+50	12	12	12
0+00 to 26+50	(6)	(6)	(6)
PEQUANNOCK RIVER			
146+00 to 145+00	24	24	24
145+00 to 142+50	Articu. Concrete Mat		
142+50 to 138+00	24	24	24
17+00 to 16+50	12	12	12
16+50 to 16+00	Pequannock Weir		
16+00 to 15+50	36	36	36
15+50 to 15+00	24	24	24
15+00 to 14+50	12	12	12
14+50 to 13+00	(6)	(6)	(6)
13+00 to 4+00	18	18	18
0+25 to 0+00	NA	12	NA
RAMAPO RIVER			
100+15 to 73+50	12	12	12
73+50 to 70+00	24	24	24
70+00 to 66+00	18	18	18
66+00 to 46+00	12	12	12
NEWARK BAY OUTLET			
0+00 to 0+50	12	54	12
0+50 to 1+50	12	12	12

( ) indicates ungraded crushed stone or gravel

Table C-55

Riprap Gradation Specifications

Layer Thickness	W_{100} (lbs) max	min	W_{50} (lbs) max	min	W_{15} (lbs) max	min
54"	2331	933	691	467	346	146
36"	2331	933	691	467	346	146
30"	1350	,540	400	270	200	84
24"	691	276	205	138	102	43
18"	292	117	86	58	43	18
12"	86	35	26	17	13	5
6" crushed stone	Not applicable					

Table C-56

Riprap Bedding & Filter Requirements

Riprap Layer Thickness	Filter Thickness [1]	Bedding Layer Thickness [2]
54"	18"	9"
30" thru 36"	12"	6"
24"	None	12"
18"	None	9"
12"	None	6"
Articulated Concrete Mat	None	Non-woven Filter Fabric

[1] Filter layers have the same gradations as riprap.
[2] Both gravel and crushed stone are suitable for bedding.

Table C-57

DETERMINATION OF MINIMUM DESIGNED FREEBOARD — REVISED 19 AUGUST 1994

LEVEE SYSTEM	CONVEY. +10 %	CONVEY. +20%	Q +10%	Q +20%	Q & CONVEY. +10%+10%	DEBRIS	DEBRIS & CONVEY.	DESIGN FREE-BOARD	NORMAL CONDITION 100YR. BASE	FREE-BOARD + BASE	LEVEE STATIONS	SELECTED OVERTOP. LOCATION
PASSAIC 2A	0.6	0.9	0.7	1.3	1.1	-	1.1	2.3	172.1	174.4	2311+00 to 2403+00	2313+00
PASSAIC/ DEEP. 2A	0.2	0.4	0.3	0.5	0.4	-	0.4	1.5	171.5	173.0	81+00 to 179.00	85+00
PASSAIC/ DEEP. 2C	0.1	0.3	0.2	0.4	0.4	0	0.4	1.5	172.5	174.0	126+00 to 142+00	128+00
PINCH BROOK	0.4	0.8	0.6	1.2	1.0	-	1.0	2.1	182.4	184.5	30+00 to 68+00	57+00
ROCKAWAY 1	0.5	1.2	0.6	1.2	1.2	0.1	1.2	2.5	180.1 (US)	182.6 (US)	181+00 to 192+00	190+00
									175.5 (DS)	178.0 (DS)	105+00 to 121+00	115+00
ROCKAWAY 2	0.5	1.2	0.6	1.2	1.2	0.1	1.2	2.5	179.7	182.2	147+00 to 172+00	157+00
ROCKAWAY 3	0.5	1.2	0.6	1.2	1.2	0.1	1.2	2.5	181.0	183.5	218+00 to 281+00	236+00
PEQ./ RAMAPO	0.9	1.9	0.8	1.6	1.7	0	1.6	3.5	178.5	182.0	20+00 to 45+00	35+00 (REACH 10)
											50+00 to 98+00	(REACH 11)
PASSAIC #10	0.4	0.8	0.6	1.2	1.0	0.2	1.2	2.2	175.7	177.9	2608+00 to 2627+00	(REACH 6)

- (LESS THAN A TENTH)

Table C-58

DETERMINATION OF FINAL LEVEE HEIGHTS

14 OCTOBER 1994

LEVEE UNIT: PASSAIC #2A
RIVER: PASSAIC RIVER
UNET REACH #: 7

LEVEE @ 2311+00 to 2403+00

%DESIGN FLOW: 151%

(1) STATION (IN FEET)	(2) DESIGN WATER SURFACE ELEVATION (NGVD)	(3) MINIMUM DESIGNED FREEBOARD (FEET)	(4) MINIMUM LEVEE PROFILE (NGVD)	(5) WATER SURFACE PROFILE WITH % DESIGN FLOW OVERTOPPING AT LEAST HAZARDOUS LOCATION (NGVD)	(6) PRE-FINAL LEVEE HEIGHT (FEET)	(7) LEVEE SUPERIORITY (FEET)	(8) FINAL LEVEE HEIGHTS (NGVD)	(9) DIFFERENCE BETWEEN DESIGN WATER SURFACE AND FINAL LEVEE HEIGHT (NGVD)
2403+00	174.64	2.30	176.94	177.74	177.74	0.50	178.20	3.56
2385+90	173.62	2.30	175.92	176.48	176.48	0.50	177.00	3.38
2364+20	173.20	2.30	175.50	176.10	176.10	0.40	176.50	3.30
2363+30	173.13	2.30	175.43	175.65	175.65	0.40	176.10	2.97
2355+85	173.10	2.30	175.40	175.73	175.73	0.40	176.10	3.00
2347+55	172.98	2.30	175.28	175.59	175.59	0.30	175.90	2.92
2346+95	172.70	2.30	175.00	174.98	175.00	0.30	175.40	2.70
2346+50	172.75	2.30	175.05	175.07	175.07	0.30	175.40	2.65
2332+55	172.63	2.30	174.93	174.94	174.93	0.30	175.20	2.57
2318+60	172.36	2.30	174.66	174.66	174.66	0.20	174.90	2.54
2311+50	172.07	2.30	174.37	174.28	174.37	0.10	174.50	2.43
2313+00	172.10	2.30	174.40	174.40	174.40	0.00	174.40	2.30
2309+50	172.05	2.30	174.35	174.24	174.35	0.10	174.40	2.35

* DETERMINED TO BE THE LEAST HAZARDOUS LOCATION FOR INITIAL OVERTOPPING

Table C-59

DETERMINATION OF FINAL LEVEE HEIGHTS

14 OCTOBER 1994

LEVEE UNIT: PINCH BROOK
RIVER: PINCH BROOK on WHIPPANY RIVER
UNET REACH #: 3

LEVEE @ 30+00 to 68+00
%DESIGN FLOW: 135%

(1) STATION (IN FEET)	(2) DESIGN WATER SURFACE ELEVATION (NGVD)	(3) MINIMUM DESIGNED FREEBOARD (FEET)	(4) MINIMUM LEVEE PROFILE (NGVD)	(5) WATER SURFACE PROFILE WITH % DESIGN FLOW OVERTOPPING AT LEAST HAZARDOUS LOCATION (NGVD)	(6) PRE-FINAL LEVEE HEIGHT (FEET)	(7) LEVEE SUPERIORITY (FEET)	(8) FINAL LEVEE HEIGHTS (NGVD)	(9) DIFFERENCE BETWEEN DESIGN WATER SURFACE AND FINAL LEVEE HEIGHT (NGVD)
72+75	182.44	2.10	184.54	184.57	184.57	0.40	185.00	2.56
57+00	182.44	2.10	184.54	184.57	184.57	0.00	184.60	2.16
46+40	182.44	2.10	184.54	184.57	184.57	0.20	184.80	2.36
40+80	182.44	2.10	184.54	184.57	184.57	0.20	184.80	2.36
27+50	182.44	2.10	184.54	184.57	184.57	0.20	184.80	2.36
15+35	182.44	-------	182.44	184.57	-------	-------	-------	-------
1+50	182.44	-------	182.44	184.57	-------	-------	-------	-------

* DETERMINED TO BE THE LEAST HAZARDOUS LOCATION FOR INITIAL OVERTOPPING

Table C-60

DETERMINATION OF FINAL LEVEE HEIGHTS

14 OCTOBER 1994

LEVEE UNIT: ROCKAWAY #1 (D/S SECTION)

LEVEE @ 105+00 to 121+00

RIVER: ROCKAWAY RIVER

UNET REACH #: 1

%DESIGN FLOW: 150%

(1) STATION (IN FEET)	(2) DESIGN WATER SURFACE ELEVATION (NGVD)	(3) MINIMUM DESIGNED FREEBOARD (FEET)	(4) MINIMUM LEVEE PROFILE (NGVD)	(5) WATER SURFACE PROFILE WITH % DESIGN FLOW OVERTOPPING AT LEAST HAZARDOUS LOCATION (NGVD)	(6) PRE-FINAL LEVEE HEIGHT (FEET)	(7) LEVEE SUPERIORITY (FEET)	(8) FINAL LEVEE HEIGHTS (NGVD)	(9) DIFFERENCE BETWEEN DESIGN WATER SURFACE AND FINAL LEVEE HEIGHT (NGVD)
122+00	176.50	2.50	179.00	178.68	179.00	0.20	179.20	2.70
121+00	175.98	2.50	178.48	178.41	178.48	0.40	178.90	2.92
119+00	175.86	2.50	178.36	178.32	178.36	0.30	178.70	2.84
115+50	175.51	2.50	178.01	178.07	178.07	0.20	178.30	2.79
115+00	175.50	2.50	178.00	178.05	178.06	0.00	178.00	2.50
108+90	175.42	2.50	177.92	178.02	178.02	0.20	178.20	2.78
99+10	175.28	2.50	177.78	177.96	177.96	0.20	178.20	2.92

* DETERMINED TO BE THE LEAST HAZARDOUS LOCATION FOR INITIAL OVERTOPPING

Table C-61

DETERMINATION OF FINAL LEVEE HEIGHTS

14 OCTOBER 1994

LEVEE UNIT: ROCKAWAY #1 (U/S SECTION)
RIVER: ROCKAWAY RIVER
UNET REACH #: 1

LEVEE @ 181+00 to 192+00

%DESIGN FLOW: 142%

(1) STATION (IN FEET)	(2) DESIGN WATER SURFACE ELEVATION (NGVD)	(3) MINIMUM DESIGNED FREEBOARD (FEET)	(4) MINIMUM LEVEE PROFILE (NGVD)	(5) WATER SURFACE PROFILE WITH % DESIGN FLOW OVERTOPPING AT LEAST HAZARDOUS LOCATION (NGVD)	(6) PRE-FINAL LEVEE HEIGHT (FEET)	(7) LEVEE SUPERIORITY (FEET)	(8) FINAL LEVEE HEIGHTS (NGVD)	(9) DIFFERENCE BETWEEN DESIGN WATER SURFACE AND FINAL LEVEE HEIGHT (NGVD)
199+30	180.32	2.50	182.82	182.70	182.82	0.40	183.20	2.88
191+40	180.10	2.50	182.60	182.58	182.60	0.30	182.90	2.80
190+00	180.10	2.50	182.60	182.58	182.60	0.00	182.60	2.50
181+00	179.93	2.50	182.43	182.53	182.53	0.20	182.70	2.77
180+00	179.90	2.50	182.40	182.53	182.53	0.20	182.70	2.80

* DETERMINED TO BE THE LEAST HAZARDOUS LOCATION FOR INITIAL OVERTOPPING

Table C-62

DETERMINATION OF FINAL LEVEE HEIGHTS

14 OCTOBER 1994

LEVEE UNIT: ROCKAWAY #2
RIVER: ROCKAWAY RIVER
UNET REACH #: 1

LEVEE @ 147+00 to 172+00

%DESIGN FLOW: 137%

(1) STATION (IN FEET)	(2) DESIGN WATER SURFACE ELEVATION (NGVD)	(3) MINIMUM DESIGNED FREEBOARD (FEET)	(4) MINIMUM LEVEE PROFILE (NGVD)	(5) WATER SURFACE PROFILE WITH % DESIGN FLOW OVERTOPPING AT LEAST HAZARDOUS LOCATION (NGVD)	(6) PRE-FINAL LEVEE HEIGHT (FEET)	(7) LEVEE SUPERIORITY (FEET)	(8) FINAL LEVEE HEIGHTS (NGVD)	(9) DIFFERENCE BETWEEN DESIGN WATER SURFACE AND FINAL LEVEE HEIGHT (NGVD)
172+20	179.82	2.50	182.32	182.23	182.32	0.40	182.70	2.88
162+80	179.71	2.50	182.21	182.22	182.22	0.30	182.50	2.79
157+00	179.70	2.50	182.20	182.20	182.20	0.00	182.20	2.50
153+50	179.65	2.50	182.15	182.18	182.18	0.20	182.40	2.75
151+30	179.56	2.50	182.06	182.09	182.09	0.20	182.30	2.74
143+00	179.40	2.50	181.90	181.98	181.98	0.20	182.20	2.80

* DETERMINED TO BE THE LEAST HAZARDOUS LOCATION FOR INITIAL OVERTOPPING

Table C-63

DETERMINATION OF FINAL LEVEE HEIGHTS

14 OCTOBER 1994

LEVEE UNIT: ROCKAWAY #3
RIVER: ROCKAWAY RIVER
UNET REACH #: 1

LEVEE @ 218+00 to 281+00

%DESIGN FLOW: 142%

(1) STATION (IN FEET)	(2) DESIGN WATER SURFACE ELEVATION (NGVD)	(3) MINIMUM DESIGNED FREEBOARD (FEET)	(4) MINIMUM LEVEE PROFILE (NGVD)	(5) WATER SURFACE PROFILE WITH % DESIGN FLOW OVERTOPPING AT LEAST HAZARDOUS LOCATION (NGVD)	(6) PRE-FINAL LEVEE HEIGHT (FEET)	(7) LEVEE SUPERIORITY (FEET)	(8) FINAL LEVEE HEIGHTS (NGVD)	(9) DIFFERENCE BETWEEN DESIGN WATER SURFACE AND FINAL LEVEE HEIGHT (NGVD)
286+50	181.85	2.50	184.35	184.10	184.35	0.30	184.60	2.75
270+00	181.35	2.50	183.85	183.72	183.85	0.30	184.20	2.85
262+80	181.26	2.50	183.76	183.64	183.76	0.20	184.00	2.74
252+60	181.17	2.50	183.67	183.58	183.67	0.20	183.90	2.73
243+80	181.09	2.50	183.59	183.50	183.59	0.20	183.80	2.71
243+38	181.09	2.50	183.59	183.50	183.59	0.10	183.70	2.61
240+70	181.07	2.50	183.57	183.49	183.57	0.10	183.70	2.63
236+80	181.02	2.50	183.52	183.46	183.52	0.10	183.70	2.68
236+00	181.00	2.50	183.50	183.46	183.50	0.00	183.50	2.50
234+30	181.00	2.50	183.50	183.44	183.50	0.00	183.50	2.50
229+20	180.98	2.50	183.48	183.42	183.48	0.10	183.60	2.62
224+00	180.94	2.50	183.44	183.38	183.44	0.20	183.60	2.66
222+90	180.94	2.50	183.44	183.38	183.44	0.20	183.60	2.66
212+40	180.85	2.50	183.35	183.32	183.35	0.20	183.50	2.65

* DETERMINED TO BE THE LEAST HAZARDOUS LOCATION FOR INITIAL OVERTOPPING

Table C-64

DETERMINATION OF FINAL LEVEE HEIGHTS

14 OCTOBER 1994

LEVEE UNIT: PEQUANNOCK-RAMAPO
RIVER: PEQUANNOCK RIVER
UNET REACH #: 10

LEVEE @ 20+00 to 45+00
on PEQUANNOCK RIVER
%DESIGN FLOW: 140%

(1) STATION (IN FEET)	(2) DESIGN WATER SURFACE ELEVATION (NGVD)	(3) MINIMUM DESIGNED FREEBOARD (FEET)	(4) MINIMUM LEVEE PROFILE (NGVD)	(5) WATER SURFACE PROFILE WITH % DESIGN FLOW OVERTOPPING AT LEAST HAZARDOUS LOCATION (NGVD)	(6) PRE-FINAL LEVEE HEIGHT (FEET)	(7) LEVEE SUPERIORITY (FEET)	(8) FINAL LEVEE HEIGHTS (NGVD)	(9) DIFFERENCE BETWEEN DESIGN WATER SURFACE AND FINAL LEVEE HEIGHT (NGVD)
46+30	180.05	3.50	183.55	182.91	183.55	0.20	183.70	3.65
37+50	178.77	3.50	182.27	182.13	182.27	0.20	182.50	3.73
35+00 *	178.50	3.50	182.00	182.00	182.00	0.00	182.00	3.50
29+60	178.06	3.50	181.56	181.80	181.80	0.20	182.00	3.94

* DETERMINED TO BE THE LEAST HAZARDOUS LOCATION FOR INITIAL OVERTOPPING

Table C-65

DETERMINATION OF FINAL LEVEE HEIGHTS

14 OCTOBER 1994

LEVEE UNIT: PEQUANNOCK-RAMAPO
RIVER: RAMAPO RIVER
UNET REACH #: 11

LEVEE @ 50+00 to 98+00
%DESIGN FLOW: 140%

(1) STATION (IN FEET)	(2) DESIGN WATER SURFACE ELEVATION (NGVD)	(3) MINIMUM DESIGNED FREEBOARD (FEET)	(4) MINIMUM LEVEE PROFILE (NGVD)	(5) WATER SURFACE PROFILE WITH % DESIGN FLOW OVERTOPPING AT LEAST HAZARDOUS LOCATION (NGVD)	(6) PRE-FINAL LEVEE HEIGHT (FEET)	(7) LEVEE SUPERIORITY (FEET)	(8) FINAL LEVEE HEIGHTS (NGVD)	(9) DIFFERENCE BETWEEN DESIGN WATER SURFACE AND FINAL LEVEE HEIGHT (NGVD)
98+60	188.21	3.50	191.71	191.44	191.71	0.30	192.00	3.79
97+70	188.10	3.50	191.60	191.26	191.60	0.30	191.80	3.70
94+10	187.65	3.50	191.15	190.86	191.15	0.30	191.40	3.75
91+30	187.18	3.50	190.68	190.32	190.68	0.30	190.80	3.62
86+00	186.71	3.50	190.21	189.86	190.21	0.30	190.40	3.69
82+50	185.94	3.50	189.44	188.89	189.44	0.10	189.60	3.66
78+00	185.02	3.50	188.52	187.81	188.52	0.00	188.50	3.48
74+20	184.02	3.50	187.52	186.62	187.52	0.00	187.50	3.48
73+40	183.85	3.50	187.35	186.72	187.35	0.00	187.40	3.55
73+30	183.27	3.50	186.77	185.51	186.77	0.00	186.80	3.53
72+90	183.03	3.50	186.53	185.01	186.53	0.00	186.50	3.47
72+80	183.56	3.50	187.06	186.20	187.06	0.00	187.10	3.54
72+30	183.29	3.50	186.79	185.90	186.79	0.00	186.80	3.51
70+10	182.58	3.50	186.08	185.26	186.08	0.00	186.10	3.52
66+30	181.25	3.50	184.75	184.44	184.75	0.20	185.00	3.75
62+40	181.15	3.50	184.65	184.31	184.65	0.20	184.80	3.65
57+30	180.36	3.50	183.86	183.71	183.86	0.30	184.20	3.84
54+70	179.75	3.50	183.25	182.93	183.25	0.20	183.40	3.65
51+40	179.04	3.50	182.54	182.22	182.54	0.20	182.70	3.66
46+30	178.06	3.50	81.56	181.80	181.80	0.20	183.00	3.92
* N/A								

* DETERMINED TO BE THE LEAST HAZARDOUS LOCATION FOR INITIAL OVERTOPPING

Table C-66

Central and Pompton Basin Levees Pertinent Drainage Data			
Levee Site	Interior Drainage Area (sq.mi.)	Outlet Works Receiving Stream	Receiving Stream Drainage Area (sq.mi.)
Pinch Brook	0.17	Pinch Brook	7.9
Passaic #10	0.12	Passaic River	133.0
Passaic #2A (Passaic)		Passaic River	355.0
subarea FF-K	1.26	" "	"
subarea E-FN	0.38	" "	"
subarea ABC	0.84	" "	"
subarea ZZ	0.02	" "	"
subarea Z	0.13	" "	"
subarea XY	0.21	" "	"
Rockaway #1 (North of I-80)			
subarea AF,HI	0.86	Whippany River	70.0
subarea G	0.05	Rockaway River	133.0
(South of I-80)			
subarea ABC	0.06	Rockaway River	133.0
subarea DE	0.08	" "	133.0
subarea F	0.09	Whippany River	70.0
subarea GH	0.15	" "	70.0
subarea I	0.04	" "	70.0
Rockaway #2	1.60	Rockaway River	133.0
Rockaway #3	1.66	Rockaway River	130.0
Pequ.- Ramapo			
subarea RP1	0.12	Ramapo River	160.0
subarea RP1A	0.03	Ramapo River	160.0
subarea RP2	0.01	Ramapo River	160.0
subarea RP3	0.20	Ramapo River	160.0
		Pequ. River	192.0

Table C-67

Central Basin Levees Representative Runoff Drainage Data				
Levee Site	Basin Slope (%)	Curve Number	SCS Lag Time (Hours)	Imperviousness (%)
Pinch Brook	3.10	81	0.27	30
Passaic #10	2.20	89	0.20	64
Passaic #2A (Passaic)				
subarea FF-K	0.50	78	2.04	18
subarea E-FN	0.70	77	1.33	14
subarea ABC	0.70	86	1.54	50
subarea ZZ	1.30	89	0.19	64
subarea Z	0.90	85	0.72	45
subarea XY	1.50	83	0.82	38
Rockaway #1 (North of I-80)				
subarea AF,HI	1.60	90	0.33	70
subarea G	0.80	89	0.21	64
(South of I-80)				
subarea ABC	1.10	82	0.18	35
subarea DE	0.70	90	0.53	70
subarea F	0.50	84	0.44	43
subarea GH	1.10	86	0.24	45
subarea I	0.70	94	0.55	80
Rockaway #2	4.70	80	1.18	20
Rockaway #3	2.40	90	0.76	70

Note: Values shown in the table are "average or weighted" and represent a subarea or the total levee site.

Table C-68

Pompton Basin : Pequannock-Ramapo Levee Basin Characteristics and Unitgraph Parameters				
Subarea	Basin Slope(%)	Imperviousness(%)	Clark R	Clark Tc
RP1	0.011	25	0.50	0.25
RP1A	0.001	25	0.32	0.16
RP2	0.007	25	0.28	0.14
RP3	0.004	25	0.77	0.38

Table C-69

Central Basin and Pompton Levees Interior Flood Peak Inflows			
Levee Site	Drainage Area (sq.mi.)	10-yr Peak Inflow (c.f.s.)	100-yr Peak Inflow (c.f.s.)
Pinch Brook	0.17	288	441
Passaic #10	0.12	168	243
Passaic #2A (Passaic)			
subarea FF-K	1.26	441	661
subarea E-FN	0.38	180	273
subarea ABC	0.84	429	597
subarea ZZ	0.02	19	25
subarea Z	0.13	97	135
subarea XY	0.21	136	206
Rockaway #1 (North of I-80)			
subarea AF,HI	0.86	929	1357
subarea G	0.05	69	100
(South of I-80)			
subarea ABC	0.06	78	118
subarea DE	0.08	81	118
subarea F	0.09	99	148
subarea GH	0.15	195	289
subarea I	0.04	38	55
Rockaway #2	1.60	838	1323
Rockaway #3 *	1.66	242 *	350 *
Pequn - Ramapo			
RP1	0.12	83	128
RP1A	0.03	23	35
RP2	0.01	11	17
RP3	0.20	171	265

* The Rockaway #3 levee site is supposed to have a pressure interceptor system that bypasses the upland 15-year peak inflow past the lowlands and through the existing Lake Hiawatha line of protection. Inflows at Rockaway #3 in the table account for this system being in place and operational.

Table C-70

Central and Pompton Basin Levees Minimum Facilities - Gravity Outlet			
Levee Site	Subarea	Status	Minimum Facilities
Pinch Brook	One Area	New	2-36" CMP
Passaic #10	One Area	New	2-36" CMP, 2-24" CMP
Rockaway #1 (North of I-80)	A-F, H-I	Exists	2-10' W x 6'H, Conc. Box 1-2.5' W x 4.5' H, Conc. Pipe
	G	New	1-36" CMP
Rockaway #1 (South of I-80)	ABC	N/A	Interior location Line of Protection
	DE	New	2-48" CMP
	F	Exists	1-24" CMP, 2-36" CMP
	GH	Exists	2-36" CMP 2-10' W x 6'4" H Conc. Box
	I	Exists	1-24" CMP (No Data, Assumed Pipe Size)
Rockaway #2	One Area	Exists	1-24'W x 6' H. Conc. Box
Rockaway #3	One Area	Exists	Existing Facilities
Pequa - Ramapo			
	RP1	New	2-24" CMP
	RP1A	New	4-27" CMP
	RP2	New	1-24" CMP
	RP3	New	2-36" CMP
		Exists	2-24" RCP
		New	2-24" RCP

Table C-71

Central Basin Levees Minimum Facilities Passaic #2A Only			
Levee Site	Subarea	Status	Minimum Facilities
Passaic #2A (Passaic)	FF-K	Exists	5-36" RCP, 3-60" RCP 1-12' W x 7' H, Conc. Box
	E-FN	Exists	1-84" RCP, 1-42" RCP
	ABC	Exists	2-24" RCP, 1-36" RCP 1-48" RCP
	ABC	New	1-24" RCP, 1-102" RCP 1-48" RCP
	ZZ	New	1-36" RCP, 2-24" RCP
	Z	Exists	1-60" RCP
	XY	New	1-60" CMP, 1-36" RCP

Table C-72

Central and Pompton Basin Levees Sizing w/ Minimum Facilities - Gravity Drains Only								
Levee Site	Start of Damage	2 Year	5 Year	10 Year	25 Year	50 Year	100 Year	500 Year
Pinch Brook	177.0	175.5	176.4	176.9	177.6	178.2	178.7	179.8
Passaic #10	170.5	168.3	168.9	169.2	169.8	170.2	170.5	171.3
Passaic #2A (Passaic)								
subarea FF-K	168.0	165.1	165.6	165.9	166.3	166.9	167.1	167.7
subarea E-FN	168.5	166.0	166.4	166.6	167.0	167.2	167.4	167.8
subarea ABC	168.5	167.2	167.8	168.2	168.7	169.1	169.3	170.0
subarea ZZ	168.5	166.8	167.5	168.1	168.6	169.2	169.5	170.4
subarea Z	168.0	167.7	168.3	168.7	169.0	169.2	169.4	169.7
subarea XY	169.5	167.7	168.1	168.4	168.8	169.1	169.3	169.8
Rockaway #1 (North of I-80)								
subarea AF,HI	173.5	171.7	172.4	172.7	173.0	173.2	173.5	173.9
subarea G (South of I-80)	175.0	171.7	172.3	172.8	173.2	173.5	173.8	174.0
subarea ABCDE	173.0	170.0	171.6	171.9	172.6	173.2	173.5	174.3
subarea F	176.0	169.5	170.2	170.6	171.0	171.3	171.4	171.8
subarea GH	174.0	171.2	171.5	171.7	171.9	172.0	172.3	173.2
subarea I	174.0	172.3	172.8	173.1	173.3	173.5	173.7	174.0
Rockaway #2	173.5	171.0	171.9	172.7	173.6	174.4	175.1	176.9
Rockaway #3		172.7	173.2	173.5	173.9	174.1	174.3	174.8
Pequa - Ramapo								
RP1	186.0	181.1	181.8	182.3	182.8	183.5	184.6	186.0
RP1A	182.0	181.1	181.4	181.6	181.8	181.9	182.1	182.4
RP2	182.0	180.5	180.8	180.9	181.1	181.6	182.0	182.3
RP3	181.0	179.5	180.2	180.4	180.6	180.7	180.9	181.8

Table C-73

Selected Plan of Improvement

Levee / Pond	Selected Facilities
Pinch Brook	Min-Fac + 3 cfs Pump Sta
Passaic #10	Min-Fac + 6 cfs Pump Sta
Passaic #2A (Passaic)	
subarea FF-K	Minimum Facility
subarea E-FN	Minimum Facility
subarea ABC	Min-Fac + 5 cfs Pump Sta
subarea ZZ	Min-Fac + 1 cfs Pump Sta
subarea Z	Min-Fac + 2 cfs Pump Sta
subarea XY	Min-Fac + 3 cfs Pump Sta
Rockaway #1 (North of I-80)	
subarea AF,HI	Minimum Facility
subarea G	Min-Fac + 1 cfs Pump Sta
South of I-80)	
subarea ABCDE	Minimum Facility
subarea F	Minimum Facility
subarea GH	Minimum Facility
subarea I	Minimum Facility
Rockaway #2	Min-Fac + 10 cfs Pump Sta
Rockaway #3	Minimum Facility
Pequa - Ramapo	
RP1	Min-Fac + 3 cfs Pump Sta
RP1A	Minimum Facility
RP2	Minimum Facility
RP3	Minimum Facility

Table C-74

Central and Pompton Basin Ponding Elevations with Selected Facilities for Interior Drainage								
Levee Site	Start of Damage	2 Year	5 Year	10 Year	25 Year	50 Year	100 Year	500 Year
Pinch Brook	177,0	174.2	175.0	175.6	176.4	177.1	177.7	178.9
Passaic #10	170.5	168.2	168.8	169.0	169.2	169.4	169.6	169.9
Passaic #2A (Passaic)								
subarea FF-K	168.0	165.1	165.6	165.9	166.3	166.9	167.1	167.7
subarea E-FN	168.5	166.0	166.4	166.6	167.0	167.2	167.4	167.8
subarea ABC	168.5	166.8	167.5	167.8	168.4	168.8	169.0	169.7
subarea ZZ	168.5	166.8	167.5	168.0	168.0	168.0	168.0	168.5
subarea Z	168.0	167.7	167.7	167.8	168.3	168.6	168.8	169.1
subarea XY	169.5	167.7	168.1	168.4	168.8	169.1	169.3	169.8
Rockaway #1 (North of I-80)								
subarea AF,HI	173.5	171.7	172.4	172.7	173.0	173.2	173.5	173.9
subarea G	175.0	171.7	172.3	172.8	173.2	173.5	173.8	174.0
South of I-80)								
subarea ABCDE	173.0	170.0	171.6	171.9	172.6	173.2	173.5	174.3
subarea F	176.0	169.5	170.2	170.6	171.0	171.3	171.4	171.8
subarea GH	174.0	171.2	171.5	171.7	171.9	172.0	172.3	173.2
subarea I	174.0	172.3	172.8	173.1	173.3	173.5	173.7	174.0
Rockaway #2	173.5	171.0	171.7	172.4	173.3	174.1	174.8	176.6
Rockaway #3		172.7	173.2	173.5	173.9	174.1	174.3	174.8
Pequa - Ramapo								
RP1	186.0	181.1	181.8	182.2	182.6	183.0	183.7	185.2
RP1A	182.0	181.1	181.4	181.6	181.8	181.9	182.1	182.4
RP2	182.0	180.5	180.8	180.9	181.1	181.6	182.0	182.3
RP3	181.0	179.5	180.2	180.4	180.6	180.7	180.9	181.8

Table C-75

Hypothetical Precipitation Depths

DURATION	RETURN PERIOD IN YEARS						
(Hours)	2	5	10	25	50	100	500
1	1.74	1.93	2.21	2.52	2.84	3.15	3.83
2	1.75	2.20	2.60	3.00	3.35	3.75	4.55
3	1.95	2.50	2.95	3.40	3.75	4.25	5.15
6	2.35	3.05	3.60	4.15	4.55	5.20	6.30
12	2.80	3.60	4.30	5.00	5.45	6.25	7.60
24	3.35	4.30	5.10	5.85	6.50	7.45	9.00
48	4.00	5.10	6.00	7.10	7.80	9.00	10.70
96	5.76	6.71	7.56	8.93	9.87	11.24	13.24

Table C-76

Drainage Area Data

Subarea	Drainage Area (Sq. Mi)	SCS Curve Number	Clark Unitgraph	
			Tc	R
Lister Ave				
L1	.046	83	.64	2.72
L2	.279	83	.76	3.02
L2A	.042	83	.64	2.71
L3	.157	85	.72	2.90
Turnpike				
T	.133	78	.71	2.88
Doremus Ave				
D1	.245	83	.75	3.00
D1A	.119	83	.70	2.86
D1B	.034	85	.62	2.66
D1C	.055	80	.66	2.75
D2	.096	80	.69	2.84
D2A	.029	81	.62	2.64
D2B	.022	78	.60	2.61
Doremus Exten				
D3A	.403	87	.79	3.06
D3B	.227	78	.75	2.97
D4	.118	80	.70	2.86
D5	.550	81	.81	3.12
Kearny Point				
K1	.276	81	.76	3.00
K2	.142	80	.72	2.90
K3	.495	85	.80	3.10
K4	.625	80	.82	3.15
South First St.				
S1	.234	83	.74	2.97
S2	.157	83	.72	2.90
S3	.060	83	.66	2.75

Table C-77

Peak Flows of Pond Inflow Hydrographs

Pond	2 Year	5 Year	10 Year	25 Year	50 Year	100 Year	500 Year
Lister Ave							
L1	12	16	20	25	28	32	40
L2	62	91	118	146	168	196	251
L3	43	64	89	104	123	149	193
Turnpike							
T	33	48	61	73	82	94	114
Doremus Ave							
D1	79	118	152	186	213	250	316
D2	28	39	49	59	68	81	114
Doremus Exten							
D3A	135	186	231	277	312	360	446
D4	24	34	44	53	61	70	88
D5	110	155	195	338	432	536	695
Kearny Point							
K1	58	81	102	124	141	164	204
K2	29	40	51	62	71	83	104
K3	114	156	193	231	261	300	371
K4	121	171	217	264	302	351	440
South First St							
S1	51	71	88	106	120	139	173
S2	36	50	62	75	85	98	122
S3	14	19	24	29	33	38	48

Table C-78

Minimum Facility Gravity Outlets

Levee	Status	Type	Length	Number	Size
Lister Ave					
L1	New	Primary	100 ft	1	36 in
	New	Secondary	100 ft	4	24 in
	New	Secondary	10 ft	1	24 in
L2	Existing	Primary	100 ft	1	72 in
	New	Secondary	100 ft	2	24 in
	New	Secondary	10 ft	5	24 in
L3	New	Primary	100 ft	1	48 in
	New	Secondary	100 ft	5	24 in
	New	Secondary	10 ft	1	24 in
	Existing	Secondary	100 ft	1	24 in
Turnpike					
T	New	Primary	100 ft	1	48 in
	New	Secondary	100 ft	7	24 in
Doremus Ave					
D1	Existing	Primary	100 ft	1	60 in
	New	Secondary	10 ft	4	24 in
D2	New	Primary	100 ft	1	48 in
	New	Secondary	100 ft	3	24 in
	New	Secondary	10 ft	3	24 in
Doremus Exten					
D3A	New	Primary	100 ft	2	60 in
	New	Secondary	100 ft	1	36 in
	New	Secondary	100 ft	1	24 in
D4	New	Primary	100 ft	2	36 in
	New	Secondary	100 ft	7	24 in
D5	New	Primary	100 ft	1	36 in
	New	Secondary	100 ft	9	24 in

Table C-78 con't

Minimum Facility Gravity Outlets

Levee	Status	Type	Length	Number	Size
Kearny Point					
K1	New	Primary	100 ft	2	36 in
	New	Secondary	100 ft	6	24 in
	New	Secondary	10 ft	7	24 in
K2	New	Primary	100 ft	1	48 in
	New	Secondary	10 ft	2	24 in
K3	Existing	Primary	100 ft	1	66 in
	New	Secondary	100 ft	4	24 in
	New	Secondary	10 ft	6	24 in
K4	New	Primary	100 ft	4	36 in
	New	Secondary	100 ft	18	24 in
	New	Secondary	10 ft	17	24 in
South First St.					
S1	New	Primary	100 ft	1	48 in
	New	Secondary	100 ft	3	24 in
	New	Secondary	10 ft	2	24 in
S2	New	Primary	100 ft	1	36 in
	New	Secondary	100 ft	1	24 in
	New	Secondary	10 ft	2	24 in
S3	New	Primary	100 ft	1	36 in
	New	Secondary	100 ft	3	24 in

Table C-79

Overflow Data

Levee	Overflow Elevation	Into Area	Return Period	0 Damage Elevation
Lister Ave				
L1	8.7	L2	15	7.5
L2	6.0	L3	<2	7.0
L3	6.0	L2	5	6.5
Turnpike				
T	6.8	D2	30	8.0
Doremus Ave				
D1	6.4	D2	2	6.5
D2	6.4	D1	7	6.0
Doremus Exten				
D3B	7.0	D5	8	5.5
D4	9.0	D3B	>500	7.0
D5	7.0	D3B	30	6.0
Kearny Point				
K1	8.6	K3	>500	7.0
K2	7.6	K3	>500	7.0
K3	7.6	K2	400	6.0
K4	8.9	K3	>500	6.3
South First St.				
S1	10.6	S2	>500	6.5
S2	8.7	S3	15	6.5
S3	8.7	S2	25	8.0

Table C-80

Peak Pond Elevations - Minimum Facility

Pond	2 Year	5 Year	10 Year	25 Year	50 Year	100 Year	500 Year
Lister Ave							
L1	6.23	6.97	7.50	7.96	8.56	8.86	9.44
L2	5.82	6.28	6.62	6.93	7.16	7.44	8.05
L3	5.51	5.90	6.42	6.78	7.16	7.44	8.05
Turnpike							
T	5.48	5.97	6.48	6.68	7.24	7.48	8.02
Doremus Ave							
D1	6.18	6.61	6.92	7.25	7.40	7.58	7.94
D2	4.92	5.12	5.40	5.73	6.04	6.39	7.94
Doremus Exten							
D3B	6.07	6.49	7.29	7.91	8.11	8.23	8.39
D4	6.18	6.84	7.13	7.42	7.64	7.91	8.56
D5	5.75	5.98	6.32	6.88	7.30	7.77	8.98
Kearny Point							
K1	6.30	6.81	7.01	7.39	7.53	7.74	8.34
K2	5.01	5.33	5.76	6.22	6.46	6.69	7.29
K3	5.48	5.81	6.16	6.55	6.79	7.04	7.71
K4	5.50	5.84	6.17	6.49	6.67	6.87	7.42
South First St							
S1	6.28	7.17	7.59	7.86	9.32	8.67	9.46
S2	6.35	8.23	8.42	8.69	8.95	9.17	9.65
S3	6.29	7.16	7.57	8.64	8.98	9.20	9.62

Table C-81

Optimized Plan of Improvement

Pond	Optimized Plan	Level of Protection
Lister Ave		
L1	Minimum Facility	10
L2	Min - Fac + 100 cfs Pump Sta	100
L3	Min - Fac + 50 cfs Pump Sta	100
Turnpike		
T	Minimum Facility	500+
Doremus Ave		
D1	Minimum Facility	8
D2	Minimum Facility	2
Doremus Exten		
D3B	Minimum Facility	
D4	Minimum Facility	
D5	Minimum Facility	
Kearny Point		
K1	Min - Fac + 75 cfs Pump Sta	75
K2	Minimum Facility	300
K3	Minimum Facility	8
K4	Minimum Facility	15
South First St		
S1	Min - Fac + 75 cfs Pump Sta	
S2	Min - Fac + 70 cfs Pump Sta	
S3	Min - Fac + 30 cfs Pump Sta	500

Table C-82

Peak Pond Elevations - Optimized Plan

Pond	2 Year	5 Year	10 Year	25 Year	50 Year	100 Year	500 Year
Lister Ave							
L1	6.23	6.97	7.50	7.96	8.56	8.86	9.44
L2	5.82	6.28	6.62	6.93	7.16	7.44	8.05
L3	5.51	5.90	6.42	6.78	7.16	7.44	8.05
Turnpike							
T	5.48	5.97	6.48	6.68	7.24	7.48	8.02
Doremus Ave							
D1	6.18	6.61	6.92	7.25	7.40	7.58	7.94
D2	4.92	5.12	5.40	5.73	6.04	6.39	7.94
Doremus Exten							
D3B	6.07	6.49	7.29	7.91	8.11	8.23	8.39
D4	6.18	6.84	7.13	7.42	7.64	7.91	8.56
D5	5.75	5.98	6.32	6.88	7.30	7.77	8.98
Kearny Point							
K1	6.30	6.81	7.01	7.39	7.53	7.74	8.34
K2	5.01	5.33	5.76	6.22	6.46	6.69	7.29
K3	5.48	5.81	6.16	6.55	6.79	7.04	7.71
K4	5.50	5.84	6.17	6.49	6.67	6.87	7.42
South First St							
S1	6.28	7.17	7.59	7.86	9.32	8.67	9.46
S2	6.35	8.23	8.42	8.69	8.95	9.17	9.65
S3	6.29	7.16	7.57	8.64	8.98	9.20	9.62

Table C - 83

Coincident Flooding Analysis: Passaic #10 Levee

06-Sep-95

Passaic River @ Station 2618+80

Passaic River Level	Total Days Above River Level	Coincidental Rainfall Number of Days Equal to or Greater Than							Total Days of Rain
		0"	0-1"	1-2"	2-3"	3-4"	4-5"	5+"	
161.00	16647	11484	4475	526	118	33	2	9	5163
162.00	6121	3840	1849	303	93	28	1	7	2281
163.00	3265	1995	973	200	68	22	1	6	1270
164.00	2031	1248	601	117	47	13	0	5	783
165.00	1252	758	369	71	38	12	0	4	494
166.00	757	454	217	47	25	11	0	3	303
167.00	482	291	133	31	16	9	0	2	191
168.00	276	166	74	18	8	8	0	2	110
169.00	163	98	44	10	5	4	0	2	65
170.00	50	29	13	3	2	1	0	2	21
171.00	8	6	1	0	1	0	0	0	2
172.00	0	0	0	0	0	0	0	0	0

* Period of Record

- Stream Flows @ Chatam, NJ — 01 Oct 1937 - 30 Sept 1993 : 20,454 days
- Rainfall @ Essex Fells, NJ — 01 Jan 1948 - 29 July 1993 : 16,647 days
- Rainfall @ Springfield, NJ — 01 Jan 1948 - 29 July 1993 : 16,647 days

* Concurrent Stream Flow and Rainfall are 01 Jan 1948 through 29 July 1993.

Table C - 84
Passaic #2A Coincident Rainfall Analysis

06-Sep-95

Passaic River @ Station 1940+00

Passaic River Level	Total Days Above River Level	Coincidental Rainfall Number of Days Equal to or Greater Than							Total Days of Rain
		0"	0-1"	1-2"	2-3"	3-4"	4-5"	5+"	
158.2	16647	10520	5332	597	142	36	7	13	6127
160.0	4116	2431	1394	195	68	14	5	9	1685
162.0	1249	728	418	60	30	9	2	2	521
164.0	461	261	162	24	7	5	1	1	200
166.0	124	75	41	6	1	1	0	0	49
167.0	51	29	19	3	0	0	0	0	22
168.0	12	9	3	0	0	0	0	0	3
169.0	5	4	1	0	0	0	0	0	1
170.0	2	2	0	0	0	0	0	0	0
171.0	1	1	0	0	0	0	0	0	0
172.0	0	0	0	0	0	0	0	0	0

Concurrent Period of Record 01 Jan 1948 - 29 July 1993

Gages Used:

Streamflow :	Passaic River at Little Falls, NJ	16 Aug 1897 - 29 July 1993
Rainfall :	Springfield, NJ	01 Jan 1948 - 29 July 1993
	Essex Falls, NJ	01 Jan 1948 - 29 July 1993
	Little Falls, NJ	01 Jan 1948 - 29 July 1993

Critical Elevation at 166.0

Table C - 85
Passaic #2A Coincident Rainfall Analysis 06-Sep-95

Passaic River @ Station 2181+00

Passaic River Level	Total Days Above River Level	Coincidental Rainfall Number of Days Equal to or Greater Than							Total Days of Rain
		0"	0-1"	1-2"	2-3"	3-4"	4-5"	5+"	
158.2	16647	10520	5332	597	142	36	7	13	6127
160.0	5064	2967	1746	241	77	19	5	9	2097
162.0	1917	1127	643	88	41	10	4	4	790
164.0	792	458	268	37	19	7	1	2	334
166.0	357	206	122	19	4	5	0	1	151
167.0	102	63	35	4	0	0	0	0	39
168.0	25	14	11	0	0	0	0	0	11
169.0	5	4	1	0	0	0	0	0	1
170.0	2	2	0	0	0	0	0	0	0
171.0	1	1	0	0	0	0	0	0	0
172.0	0	0	0	0	0	0	0	0	0

Concurrent Period of Record 01 Jan 1948 - 29 July 1993

Gages Used:

Streamflow :	Passaic River at Little Falls, NJ	16 Aug 1897 - 29 July 1993
Rainfall :	Springfield, NJ	01 Jan 1948 - 29 July 1993
	Essex Falls, NJ	01 Jan 1948 - 29 July 1993
	Little Falls, NJ	01 Jan 1948 - 29 July 1993

Critical Elevation at 166.0

Table C - 86
Passaic #2A Coincident Rainfall Analysis

06-Sep-95

Passaic River @ Station 2250+00

Passaic River Level	Total Days Above River Level	Coincidental Rainfall: Number of Days Equal to or Greater Than 0″	0-1″	1-2″	2-3″	3-4″	4-5″	5+″	Total Days of Rain
158.2	16647	10520	5332	597	142	36	7	13	6127
160.0	5360	3140	1850	254	81	21	5	9	2097
162.0	2195	1300	734	99	44	10	4	4	790
164.0	914	529	312	41	21	8	1	2	334
166.0	442	248	158	23	7	5	0	1	151
167.0	246	150	75	15	4	1	0	1	39
168.0	62	37	22	3	0	0	0	0	11
169.0	9	8	1	0	0	0	0	0	1
170.0	3	3	0	0	0	0	0	0	0
171.0	1	1	0	0	0	0	0	0	0
172.0	0	0	0	0	0	0	0	0	0

Concurrent Period of Record 01 Jan 1948 - 29 July 1993

Gages Used:

Streamflow :	Passaic River at Little Falls, NJ	16 Aug 1897 - 29 July 1993
Rainfall :	Springfield, NJ	01 Jan 1948 - 29 July 1993
	Essex Falls, NJ	01 Jan 1948 - 29 July 1993
	Little Falls, NJ	01 Jan 1948 - 29 July 1993

Critical Elevation at 166.0

Table C-87

Pompton Plains, NJ Thru Water Year 1992 w/Urbanization Adjustment						
	Existing Conditions			Future w/Project		
% Chance Exceed-ance	(A)* Computed from HECWRC	(B)* With Expected Probability Adjustment	(C)** UNET	(F)* Computed	(E)* With Expected Probability Adjustment	(D)** UNET
0.002	38,500	42,600	42,040	40,080	44,350	43,770
0.010	27,700	29,400	29,730	29,150	30,940	31,290
0.020	23,600	24,600	24,050	24,740	25,790	25,210
0.040		20,400	20,160		21,610	21,350
0.100	15,000	15,300	14,720	16,270	16,600	15,970
0.200	11,700	11,800	11,650	12,690	12,800	12,640
0.500	7,300	7,300	7,260	7,860	7,860	7,820
0.990	2,120	2,020	(4200)	2,270	2,270	(4600)

* Annual Series Data.
** All UNET Runs Include Expected Probability Adjustment.

Column A - Computed peak flow frequency for 1992, via HECWRC.
Column B - Peak flow curve, with expected probability, via HECWRC.
Column C - UNET model runs for existing conditions, 1992.
Column D - UNET model runs for future with project conditions, 2050.
Column E - ((Column B/Column C) x Column D)
Column F - ((Column A/Column B) x Column E)

Table C-88

Tunnel Sizing Analysis
Targeted Peak Flows versus Computed Flows

% Chance Exceedence	Target Peak Flows	@ Risk Statistical Flows	% Error
0.002	40080	40873	+1.9
0.010	29150	29421	+0.9
0.020	24740	25064	+1.3
0.040	---	---	---
0.100	16270	16039	-1.4
0.200	12690	12499	-1.5
0.500	7860	7835	-0.3
0.990	2270	2303	+1.4

Table C-89

Tunnel Sizing Analysis
Tunnel Diameter versus Flow Capacity

Tunnel Dia (Feet)	Tunnel Flow (CFS)	Bypass Flow (CFS)	Total Flow (CFS)
10.0	695	4000	4695
30.0	11750	4000	15750
35.0	17425	4000	21425
40.0	24375	4000	28375
41.0	26025	4000	30025
42.0	27500	4000	31500
43.0	29255	4000	33255
45.0	32825	4000	36825
48.0	38775	4000	42775
55.0	54750	4000	58750

Table C - 90

PASSAIC RIVER FLOOD PROTECTION — 8 APRIL 1994

SIMULA TUNNEL SIZING

TUNNEL HEAD AT ELEVATION 170.0 NGVD, SD = 1.45 FEET

FUTURE CONDITIONS W/TUNNEL FEATURES

N = 52 ; I = No of Iterations

		True Probability								
Computed Probability	Tunnel Diameter	I = 1000	I = 2000	I = 3000	I = 4000	I = 5000	I = 6000	I = 7000	I = 8000	I = 9000
0.1058	30.0	0.1130	0.1195	0.1163	0.1120	0.1090	0.1062	0.1074	0.1078	0.1094
0.0372	35.0	0.0450	0.0455	0.0430	0.0423	0.0414	0.0398	0.0406	0.0408	0.0411
0.0118	40.0	0.0200	0.0190	0.0170	0.0150	0.0148	0.0142	0.0147	0.0145	0.0147
0.0091	41.0	0.0180	0.0165	0.0142	0.0125	0.0120	0.0115	0.0121	0.0120	0.0121
0.0073	42.0	0.0170	0.0135	0.0117	0.0105	0.0102	0.0095	0.0101	0.0100	0.0100
0.0057	43.0	0.0150	0.0120	0.0100	0.0088	0.0086	0.0081	0.0086	0.0085	0.0086
0.0044	44.0	0.0130	0.0105	0.0087	0.0070	0.0068	0.0067	0.0067	0.0066	0.0068
0.0034	45.0	0.0070	0.0065	0.0053	0.0045	0.0048	0.0047	0.0046	0.0046	0.0049

Table C-91

Initial Inlet Design Conditions

Inlet	100 yr. Flow cfs	Bypass Flow cfs	Inlet Flow cfs	Max. Elev. ft. NGVD
Pompton	31,000	4,000	27,000	173.5
Spur	15,000	5,000	13,000*	165.0

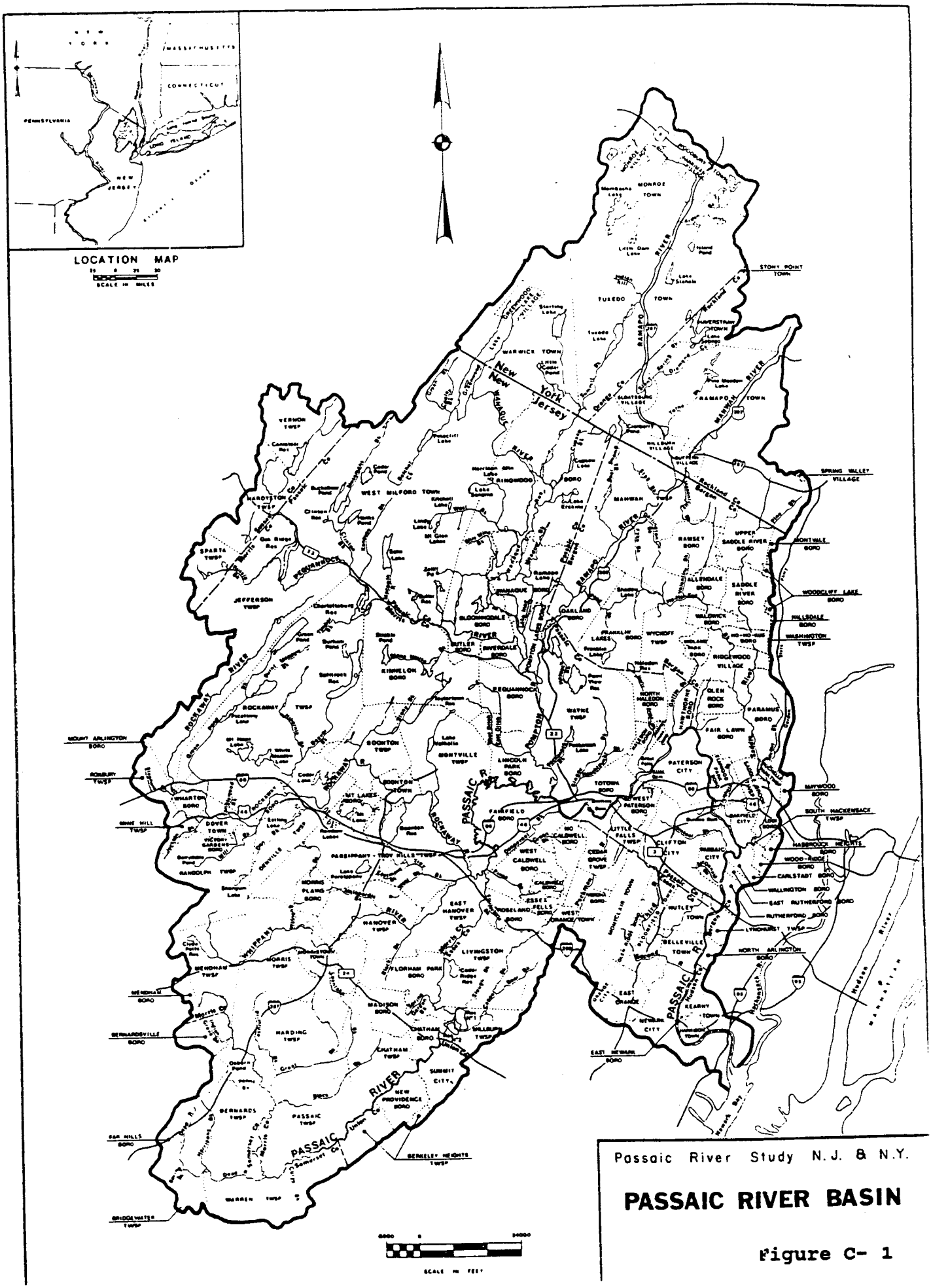
* Increased 30% for construction sequencing and storm centering.

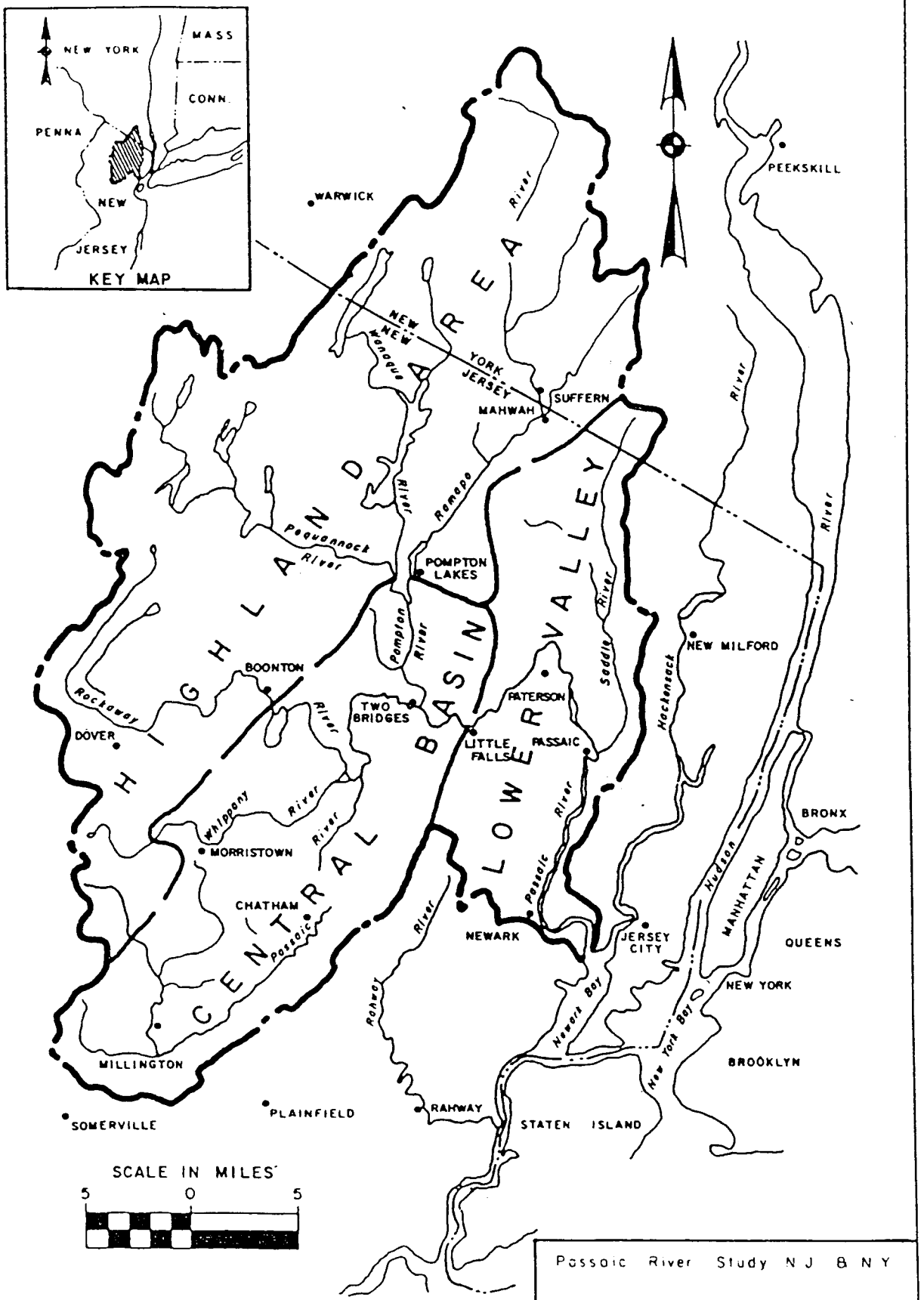
Table C-92

Final UNET Inlet Stages and Flows

	Pompton Inlet	Spur Inlet
100 yr. Flow	31,400	17,900
Bypass Flow	5,000	6,500
Inlet Flow	29,000	13,400
Gate Opening Elev.	174.5	164.3
Gate Closing Elev.	174.0	163.2
Max. River Elev.	175.0	165.0

Figures

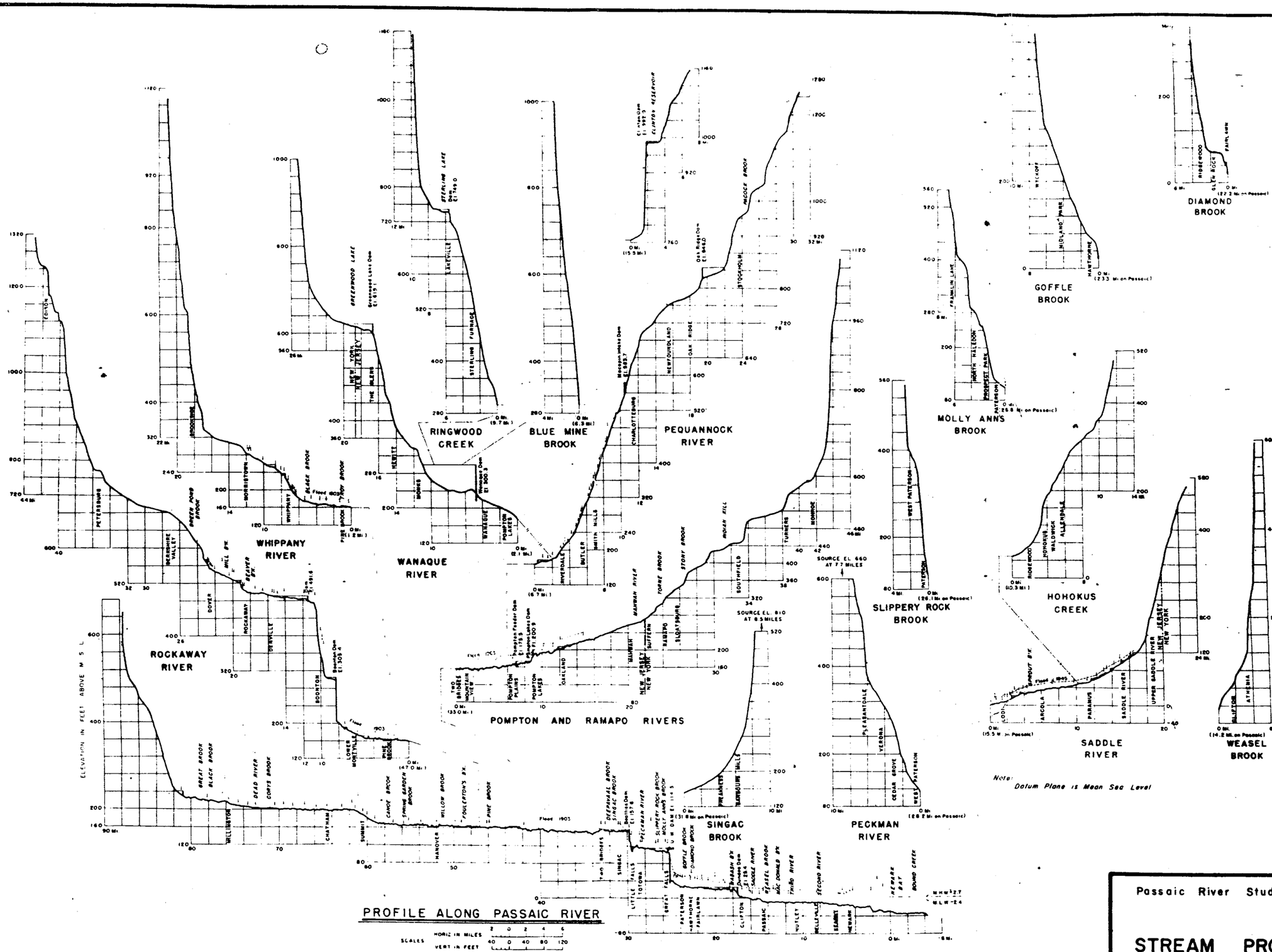

LOCATION MAP
SCALE IN MILES
New York
New Jersey
Passaic River Study N.J. & N.Y.
PASSAIC RIVER BASIN
Figure C- 1



Passaic River Study N J & N Y

TOPOGRAPHIC REGIONS

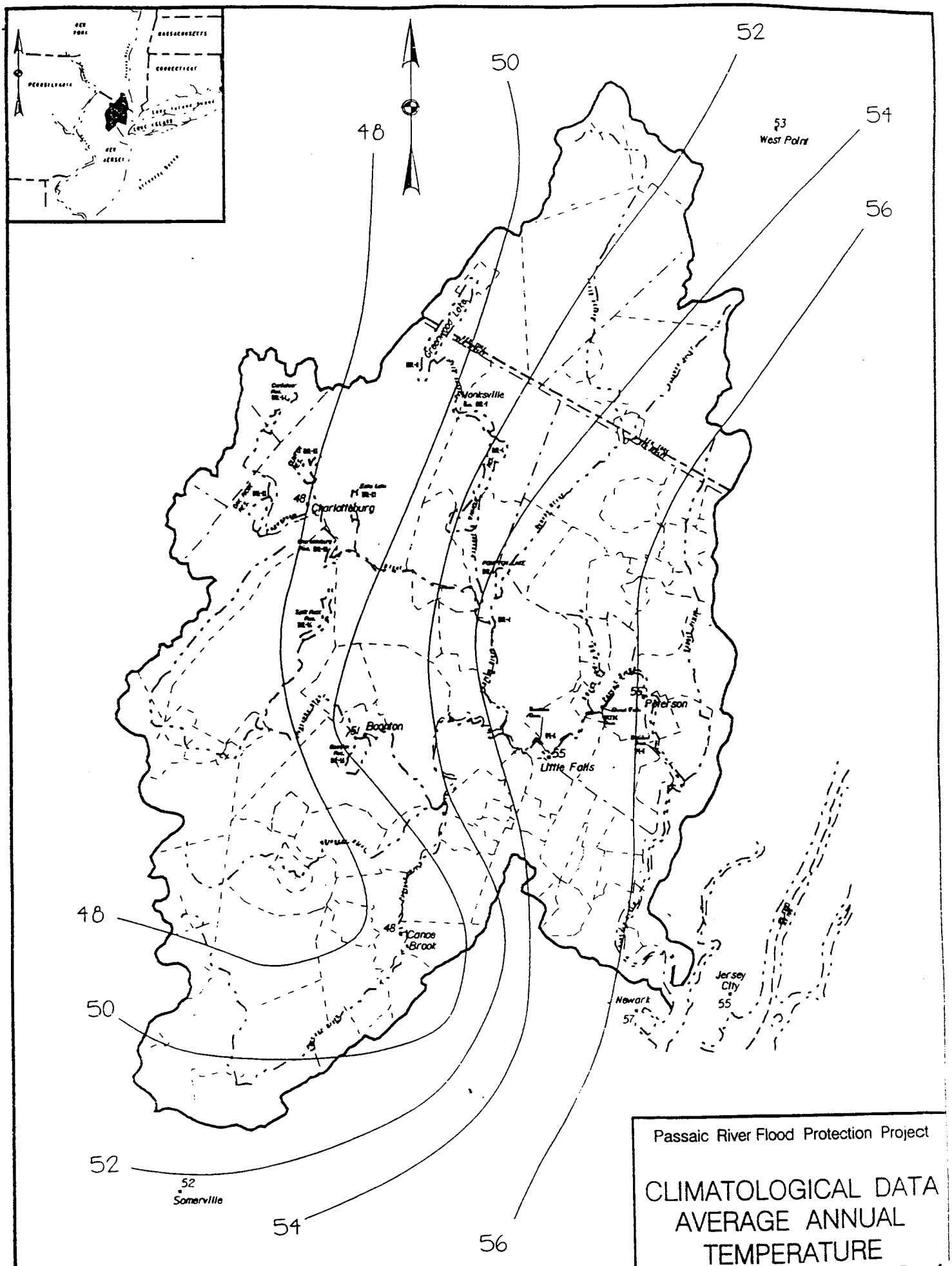
Figure C- 2



Passaic River Study, N.J. & N.Y.

STREAM PROFILES

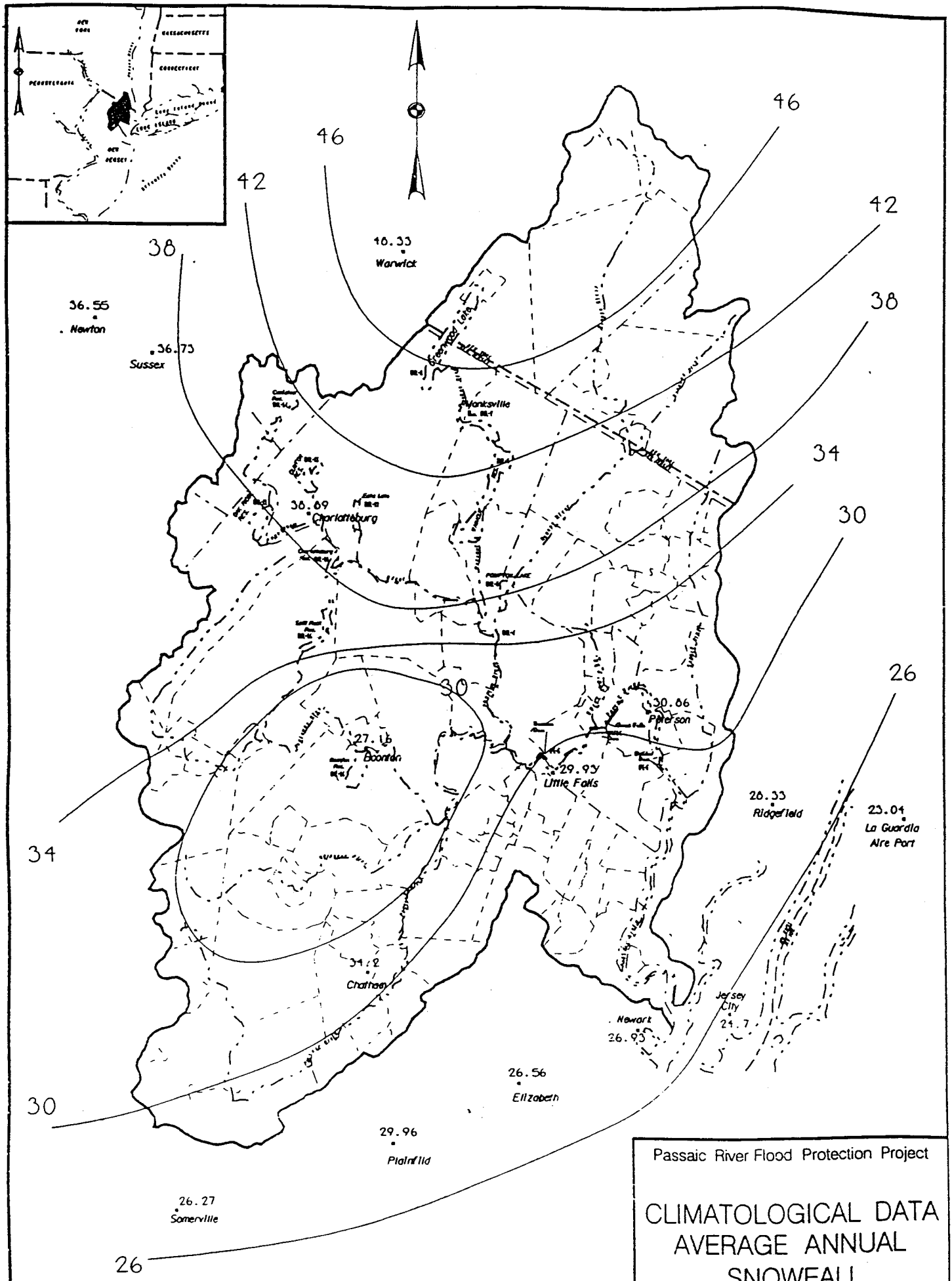
Figure C-3



Passaic River Flood Protection Project

CLIMATOLOGICAL DATA AVERAGE ANNUAL TEMPERATURE

Figure C- 4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42
38
34
30
26
46.33
Warwick
36.55
Newton
36.73
Sussex
36.69
Charlotteburg
Hanksville
30.66
Paterson
27.16
Boonton
29.93
Little Falls
26.33
Ridgefield
25.04
La Guardia Aire Port
34.2
Chatham
Jersey City
24.7
Newark
26.93
26.56
Elizabeth
29.96
Plainfield
26.27
Somerville
Passaic River Flood Protection Project
CLIMATOLOGICAL DATA
AVERAGE ANNUAL
SNOWFALL

Figure C-5

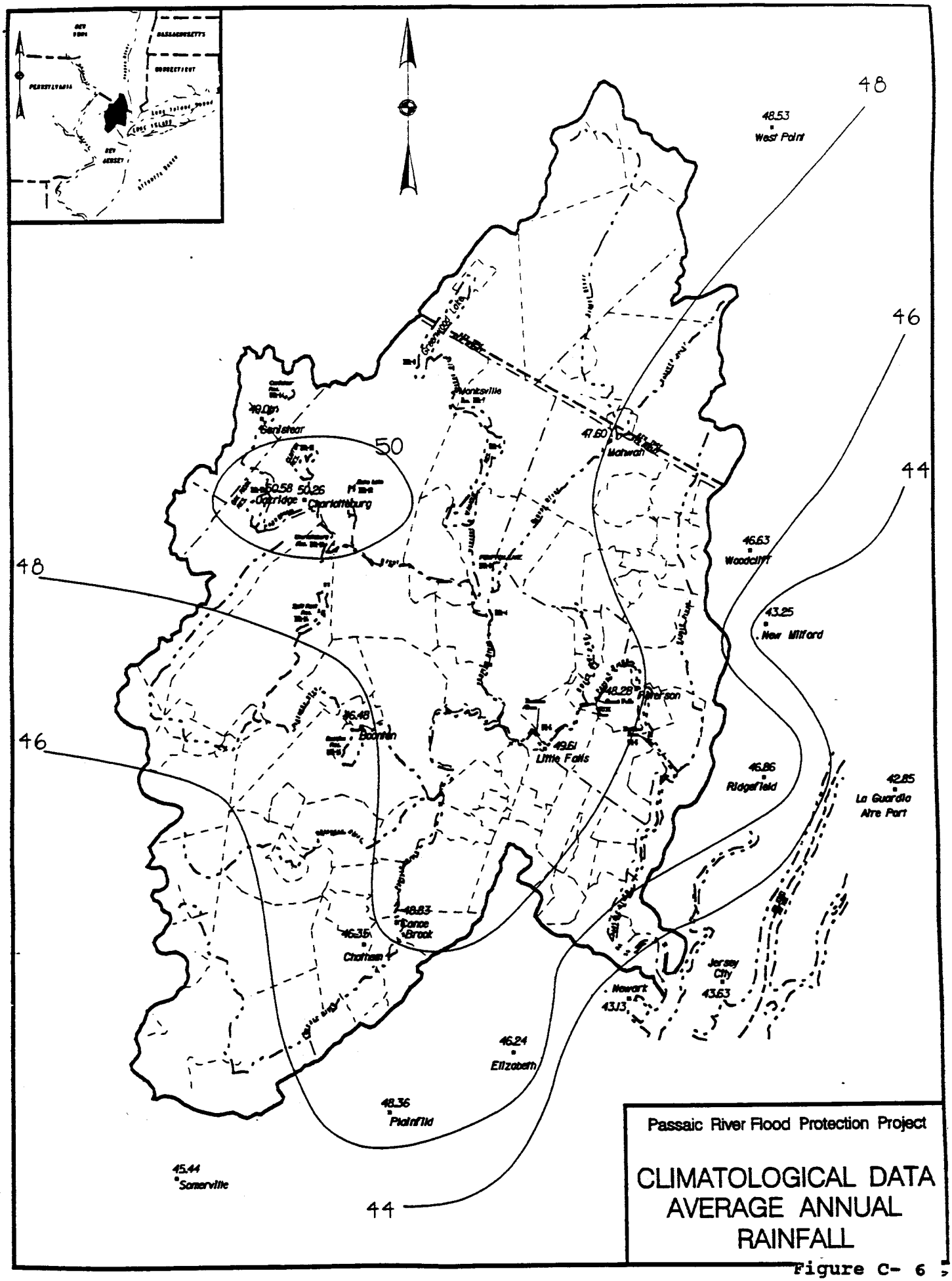


Figure C- 6

climate3.dgn

RAIN GAGES

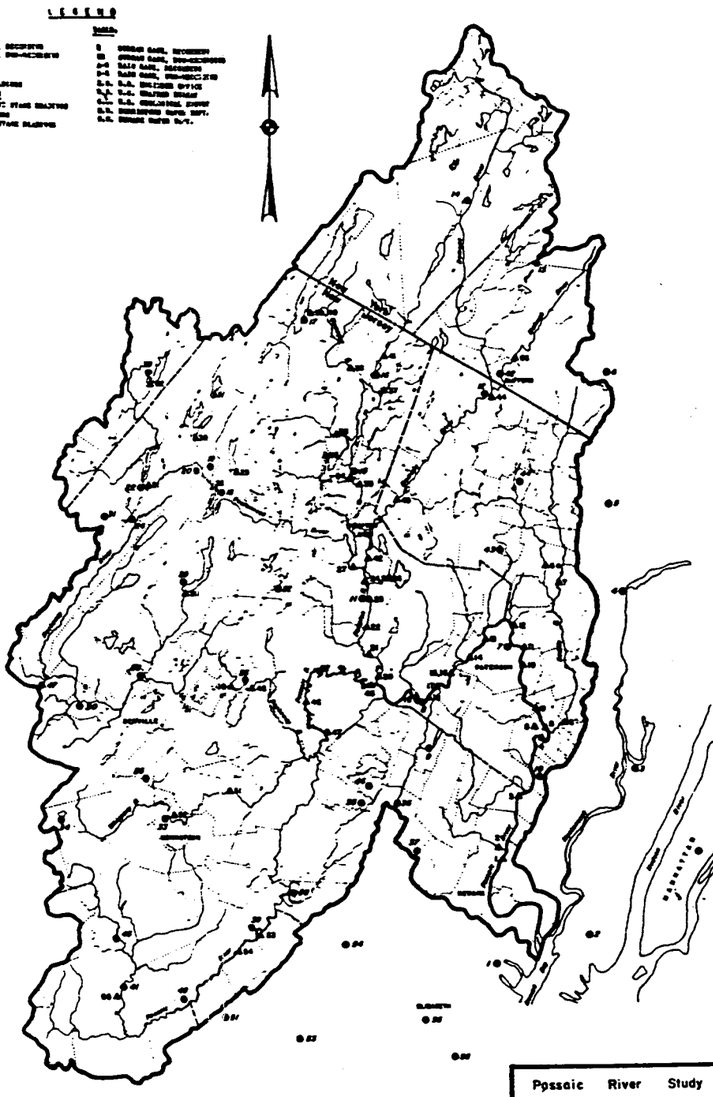
STREAM GAGES

NO	STREAM	LOCATION	GAGE EL. FT. MSL	TYPE GAGE	AGENCY
			DRAINAGE SQ.MI.		

PERIOD OF RECORD

1920 1930 1940 1950 1960 1970 1980 1990

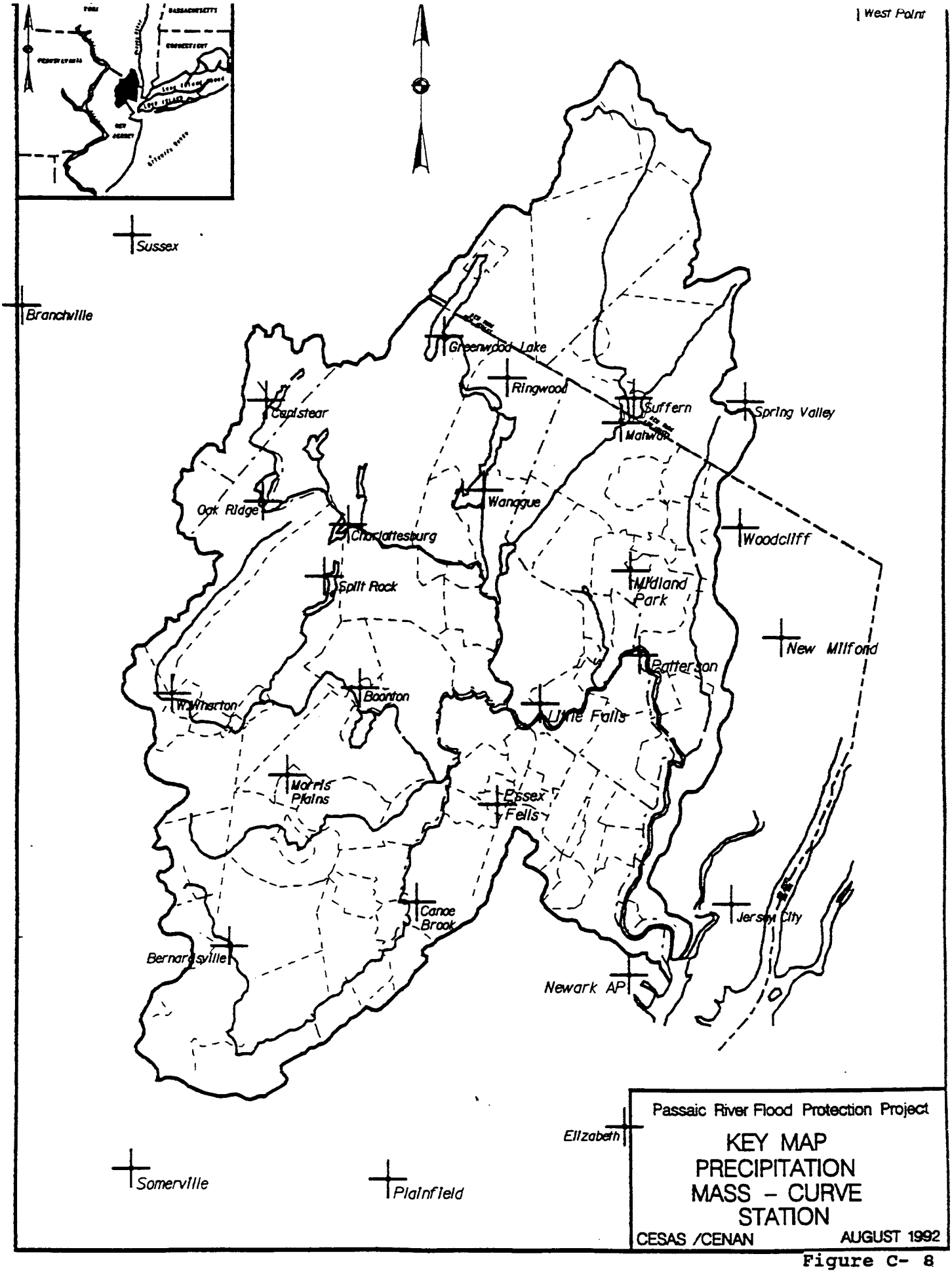
LEGEND



Passaic River Study N.J. & N.Y.

RAIN AND STREAM GAGES

Figure C-7



Passaic River Flood Protection Project

KEY MAP
PRECIPITATION
MASS - CURVE
STATION

CESAS /CENAN AUGUST 1992

Figure C- 8

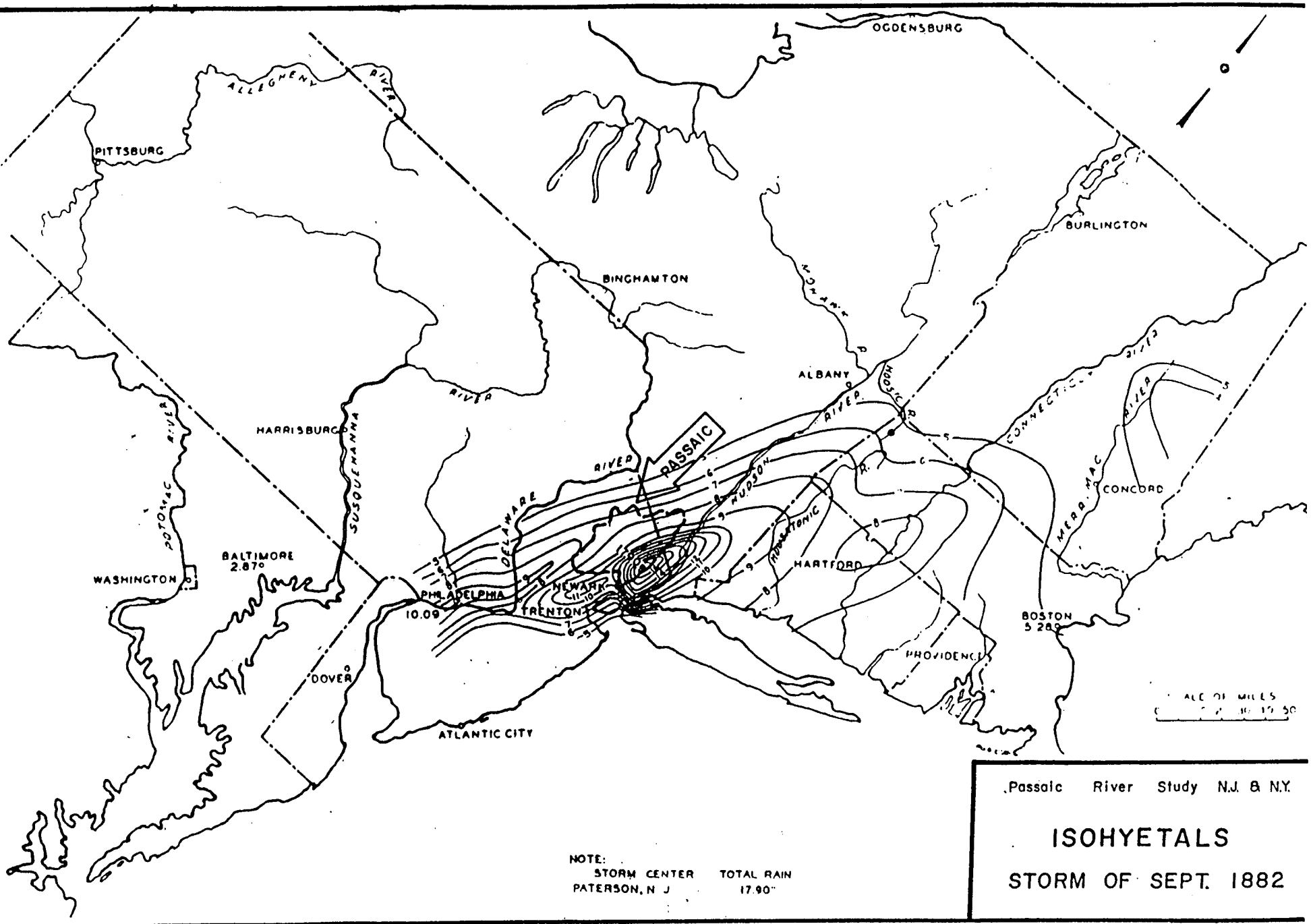
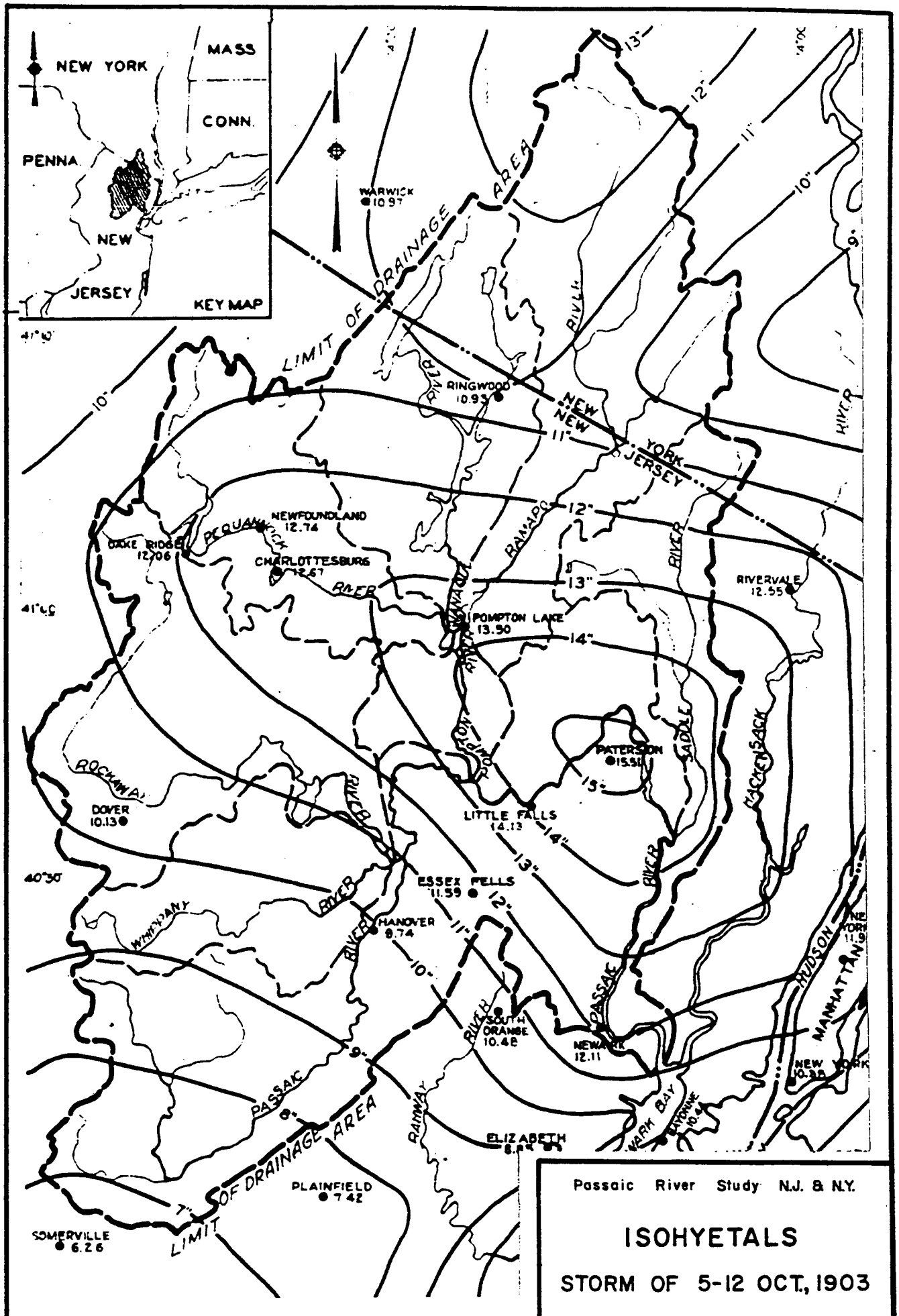

OGDENSBURG
ALLEGHENY RIVER
PITTSBURG
BINGHAMTON
BURLINGTON
ALBANY
HOOSIC R.
CONNECTICUT RIVER
MERR. MAC
CONCORD
HARRISBURG
SUSQUEHANNA RIVER
POTOMAC RIVER
PASSAIC
DELAWARE RIVER
HUDSON
HOUSATONIC
HARTFORD
BALTIMORE
2.87"
WASHINGTON
PHILADELPHIA
10.09
NEWARK
TRENTON
BOSTON
5.28
PROVIDENCE
DOVER
ATLANTIC CITY
SCALE OF MILES
NOTE:
STORM CENTER TOTAL RAIN
PATERSON, N J 17.90"
Passaic River Study N.J. & N.Y.
ISOHYETALS
STORM OF SEPT. 1882

Figure C-6



Passaic River Study N.J. & N.Y.

ISOHYETALS

STORM OF 5-12 OCT., 1903

Figure C-10

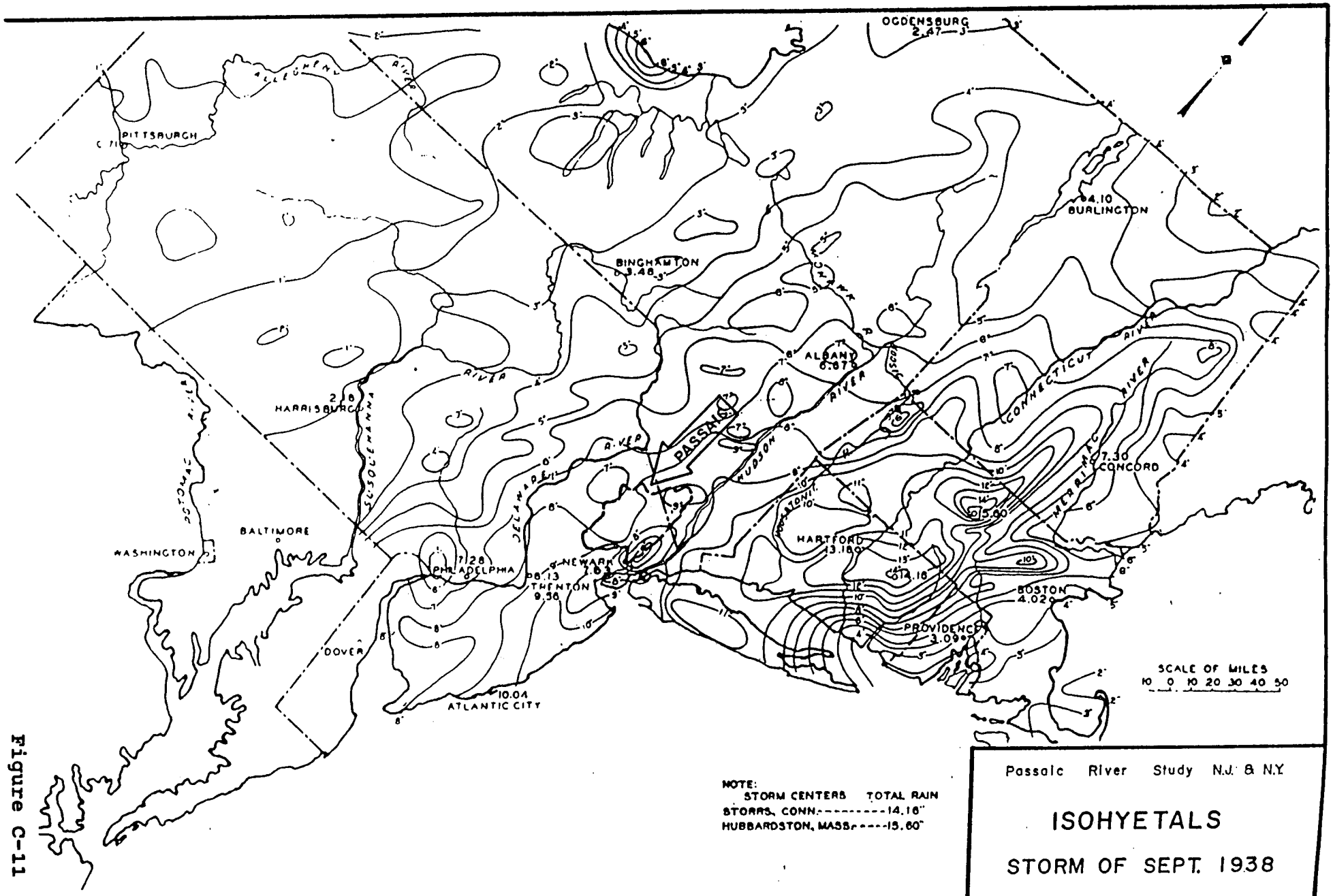
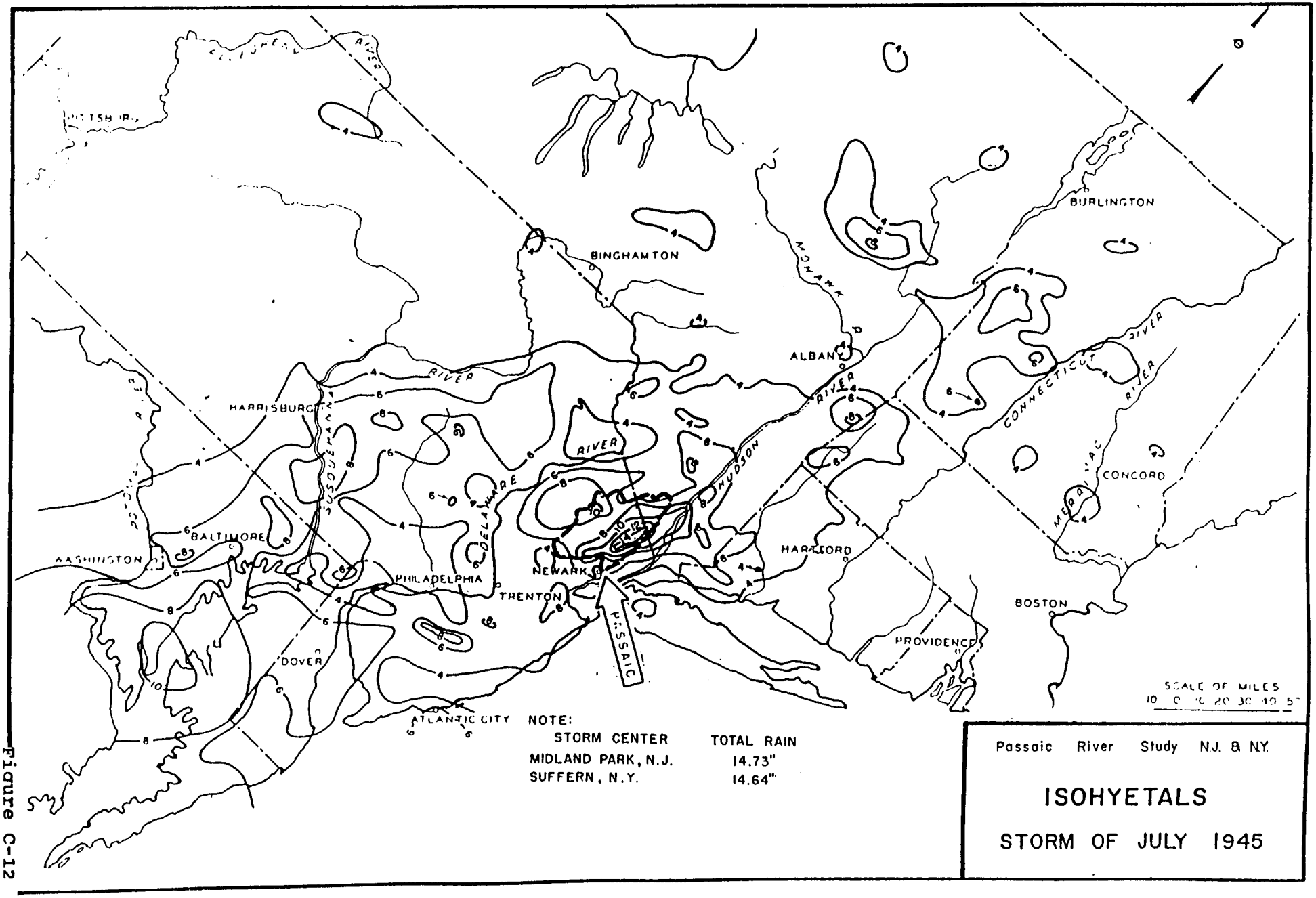

OGDENSBURG
2.47
PITTSBURGH
BINGHAMTON
3.48
BURLINGTON
4.10
ALBANY
6.67
HARRISBURG
2.18
PASSAIC
HUDSON
RIVER
CONNECTICUT
RIVER
MERRIMAC
RIVER
7.30
CONCORD
BALTIMORE
WASHINGTON
PHILADELPHIA
7.28
NEWARK
TRENTON
9.56
HARTFORD
13.18
14.16
15.60
BOSTON
4.02
PROVIDENCE
3.09
DOVER
10.04
ATLANTIC CITY
SCALE OF MILES
10 0 10 20 30 40 50
NOTE:
STORM CENTERS TOTAL RAIN
STORRS, CONN.----------14.16"
HUBBARDSTON, MASS.-----15.60"
Passaic River Study N.J. & N.Y.
ISOHYETALS
STORM OF SEPT. 1938

Figure C-11



NOTE:

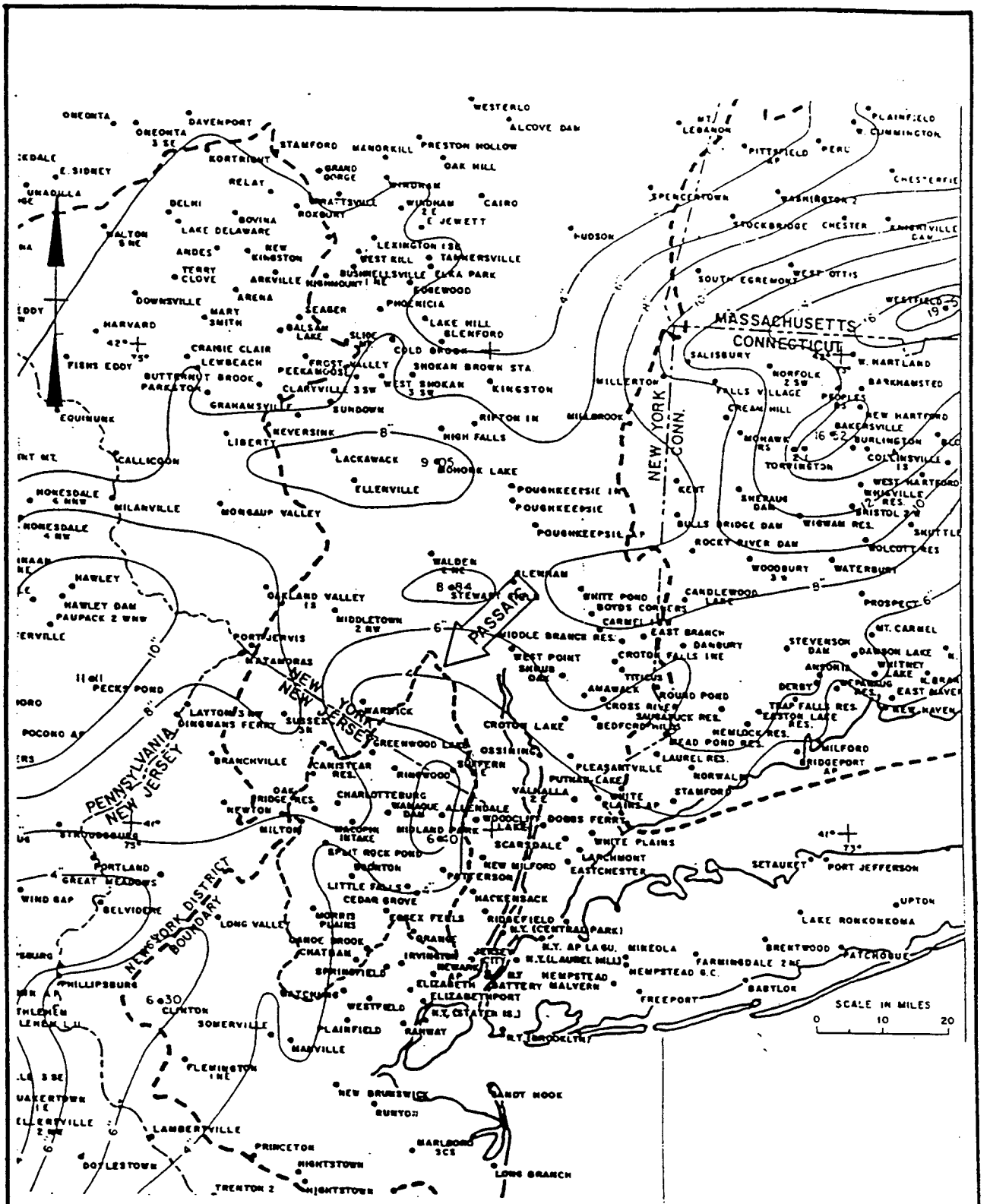
STORM CENTER	TOTAL RAIN
MIDLAND PARK, N.J.	14.73"
SUFFERN, N.Y.	14.64"

Passaic River Study N.J. & N.Y.

ISOHYETALS

STORM OF JULY 1945

Figure C-12

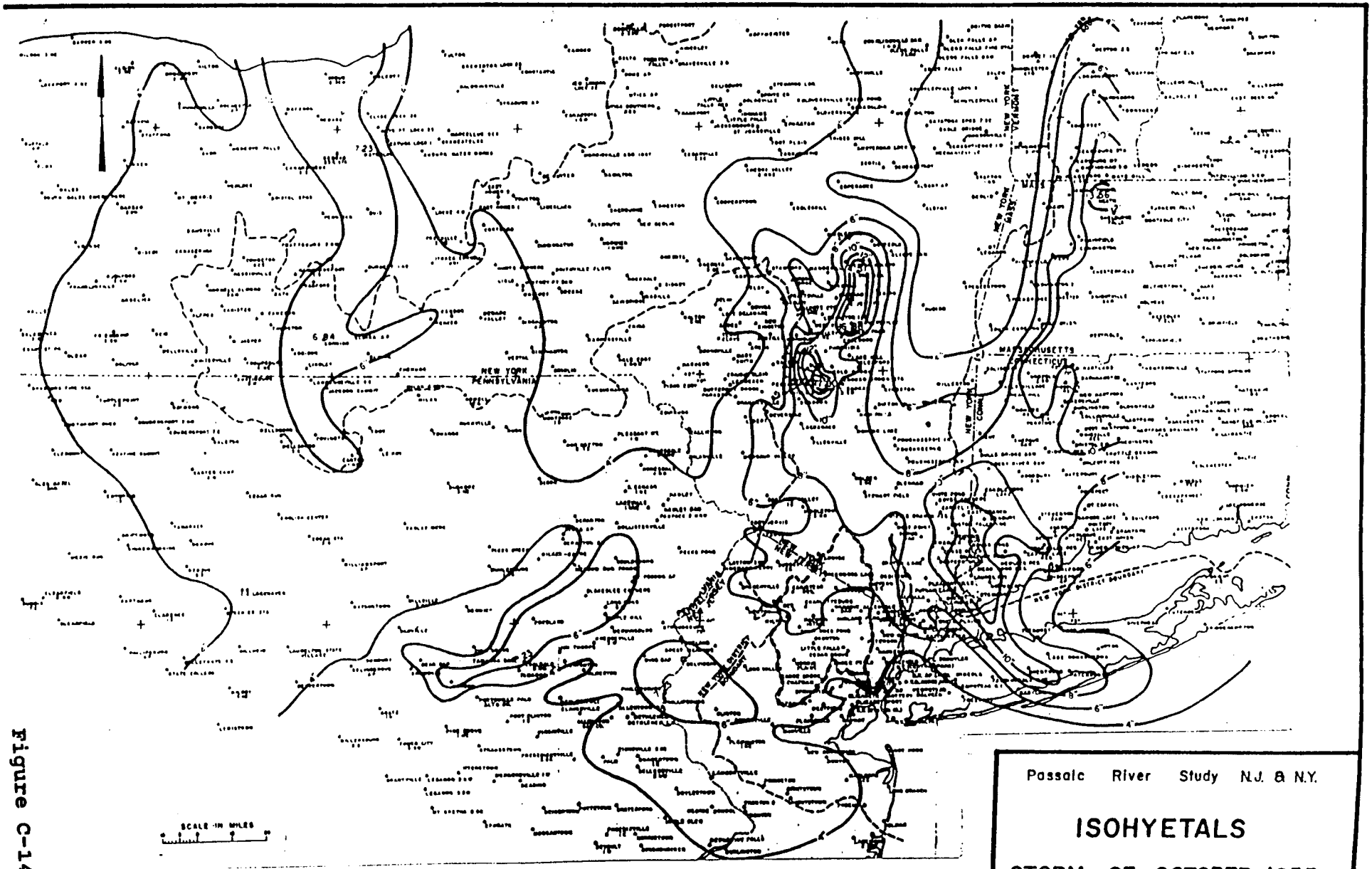


Passaic River Study N.J. & N.Y.

ISOHYETALS

STORM OF AUGUST 1955

Figure C-13

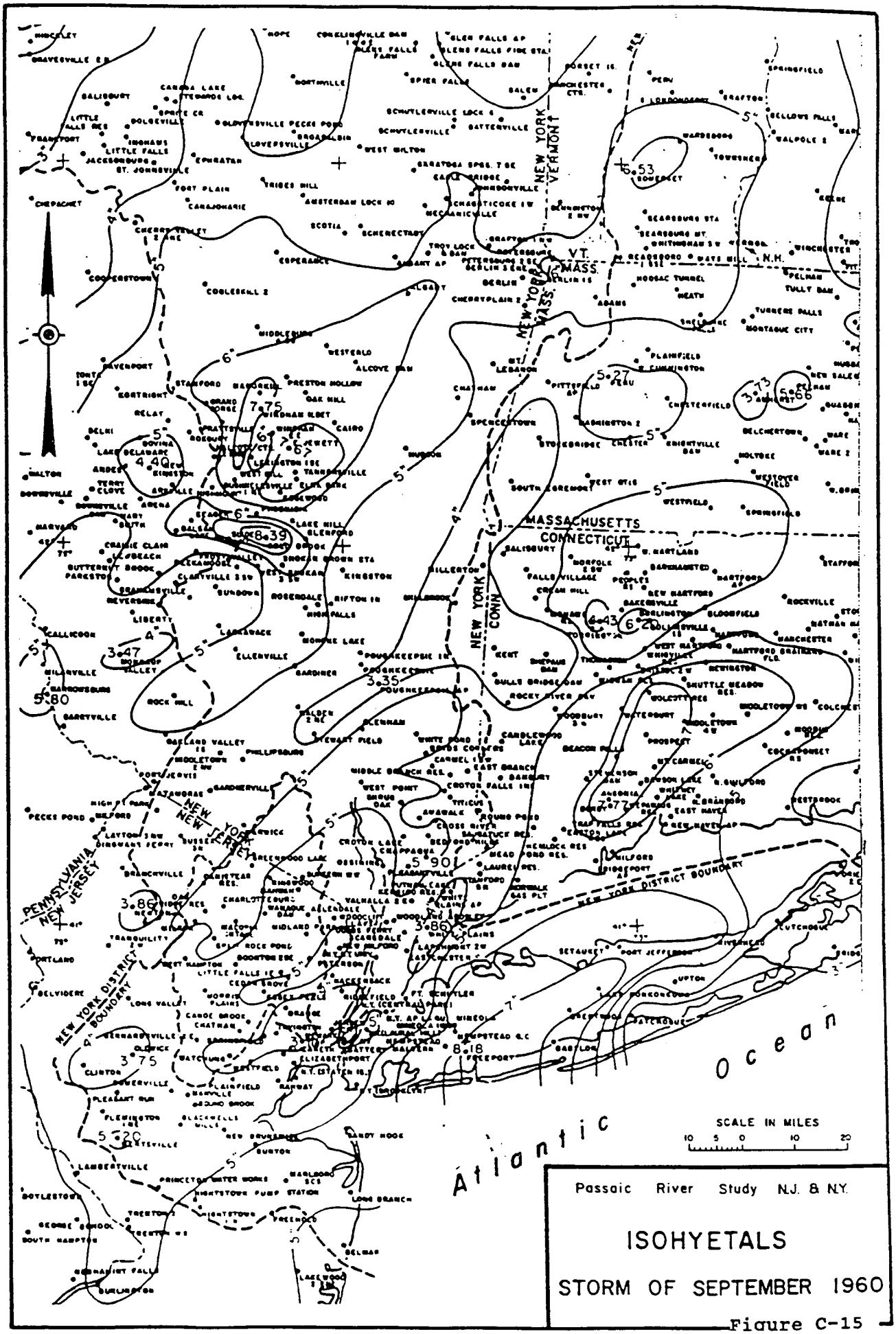


Passaic River Study N.J. & N.Y.

ISOHYETALS

STORM OF OCTOBER 1955

Figure C-14

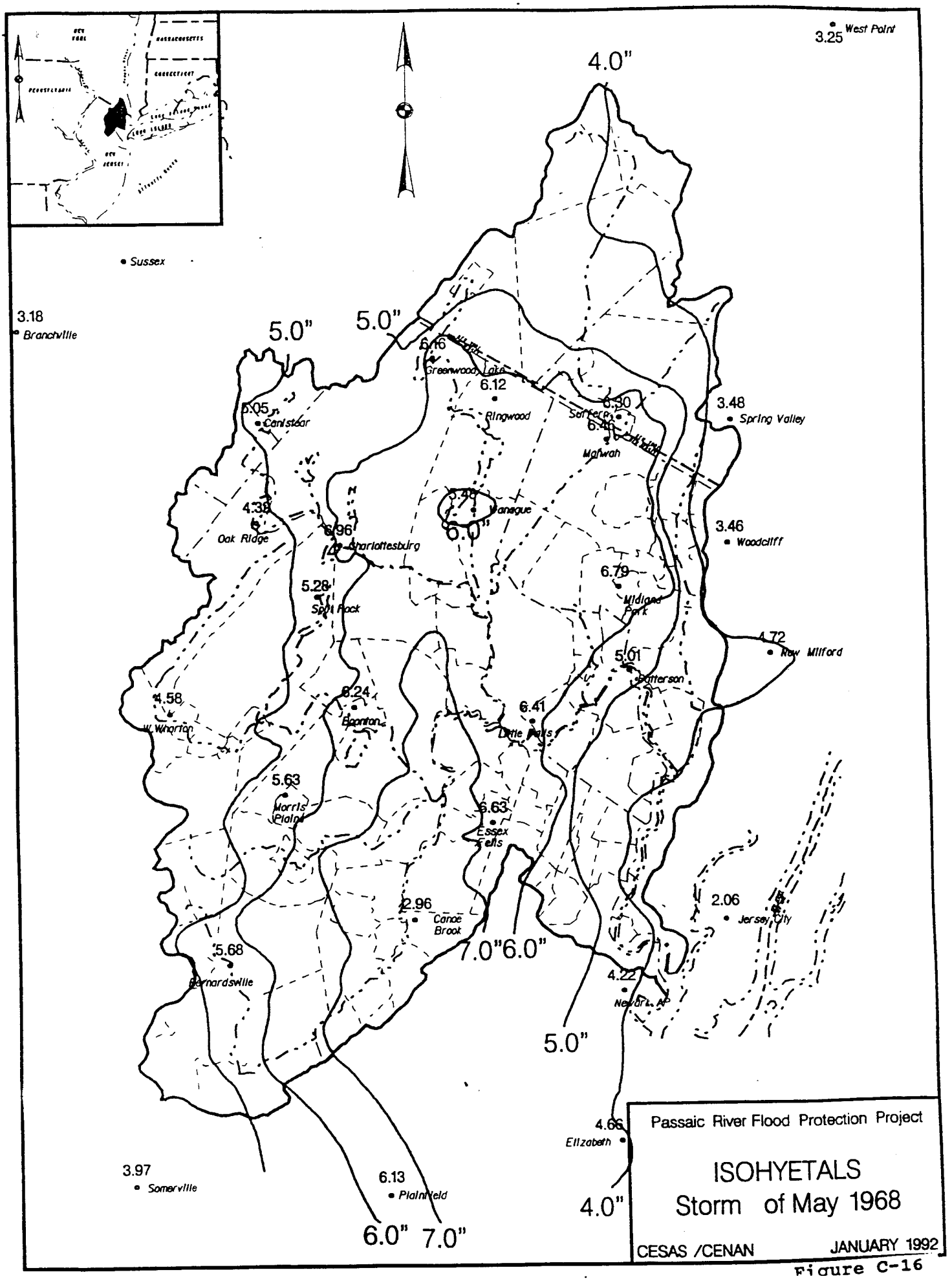


Passaic River Study N.J. & N.Y.

ISOHYETALS

STORM OF SEPTEMBER 1960

Figure C-15



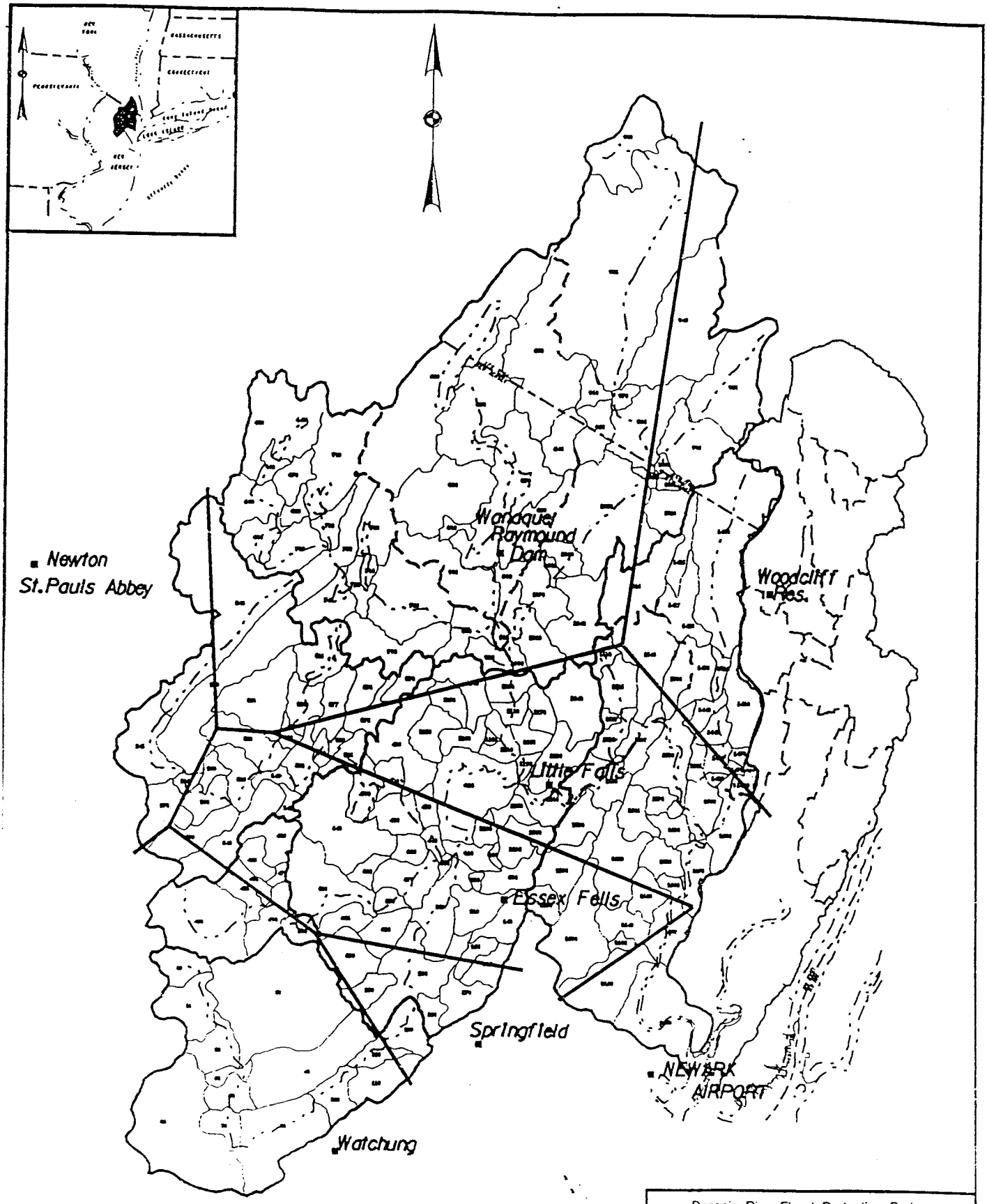
Passaic River Flood Protection Project

ISOHYETALS
Storm of May 1968

CESAS /CENAN JANUARY 1992

Figure C-16

revised on 05/94



Passaic River Flood Protection Project

THIESSEN POLYGONS
STORM OF MAY 1968

CESAS /CENAN JANUARY 1992

Figure C-17

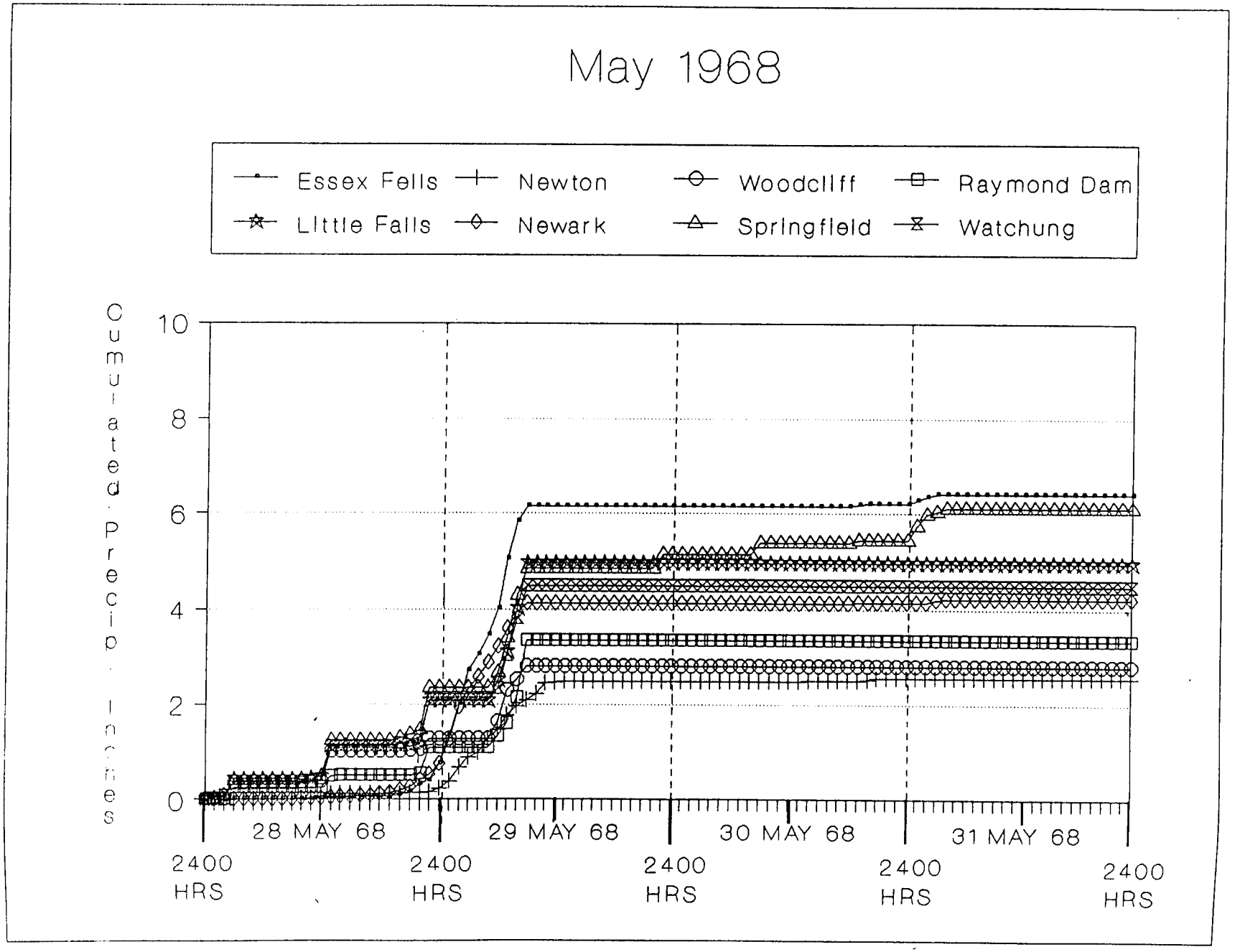
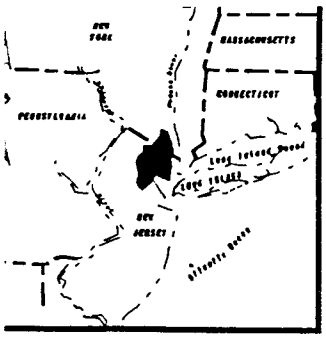

May 1968
Essex Falls
Newton
Woodcliff
Raymond Dam
Little Falls
Newark
Springfield
Watchung
Cumulated Precip - Inches
10
8
6
4
2
0
28 MAY 68
29 MAY 68
30 MAY 68
31 MAY 68
2400 HRS
2400 HRS
2400 HRS
2400 HRS
2400 HRS

Figure C-18

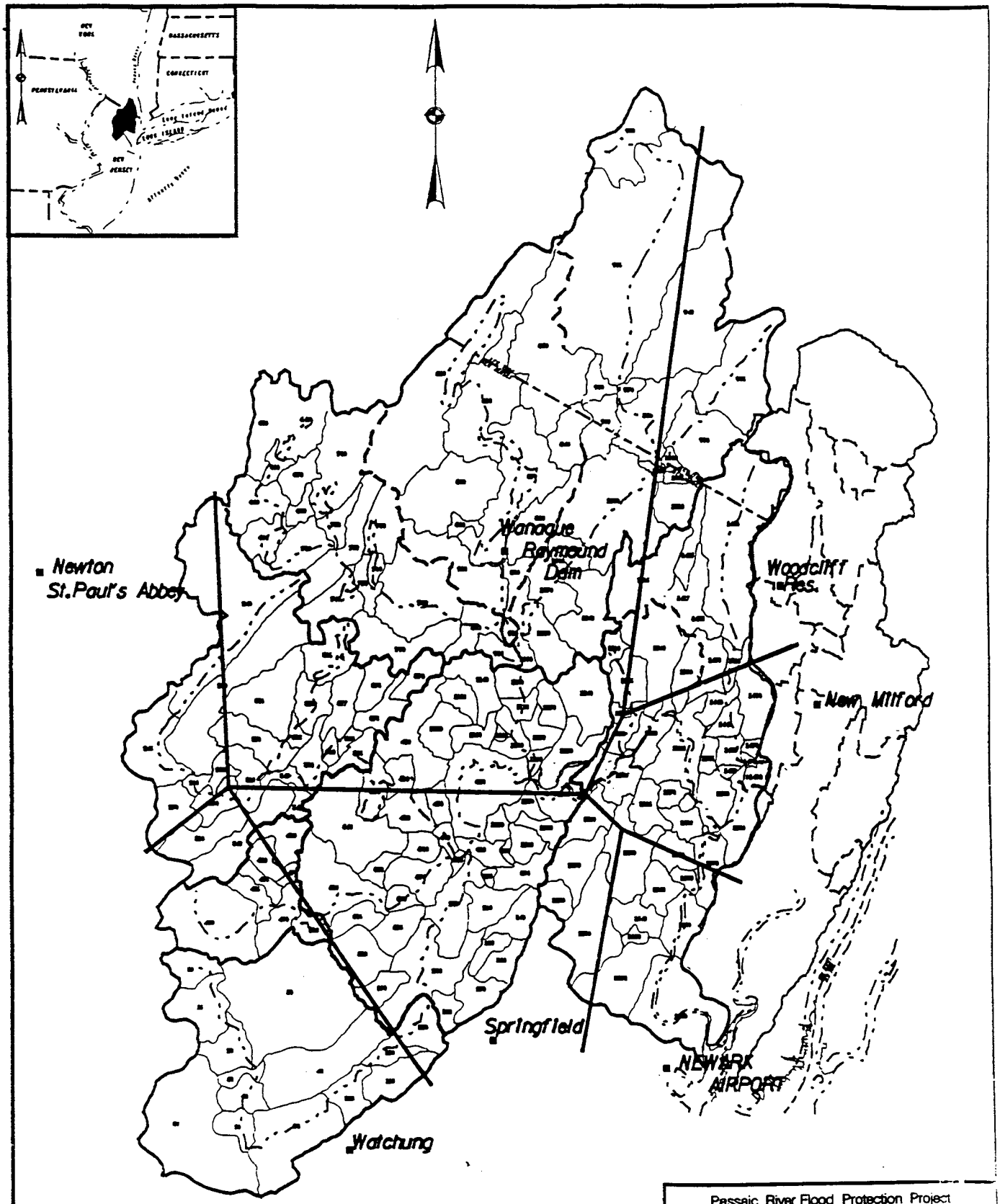


Passaic River Flood Protection Project

ISOHYETALS
Storm of 27–29 August 1971

CESAS /CENAN JANUARY 1992

Figure C-19



Passaic River Flood Protection Project

THIESSEN POLYGONS
STORM OF AUGUST 1971

CESAS /CENAN JANUARY 1992

Figure C-20

tpsoa4.dgn

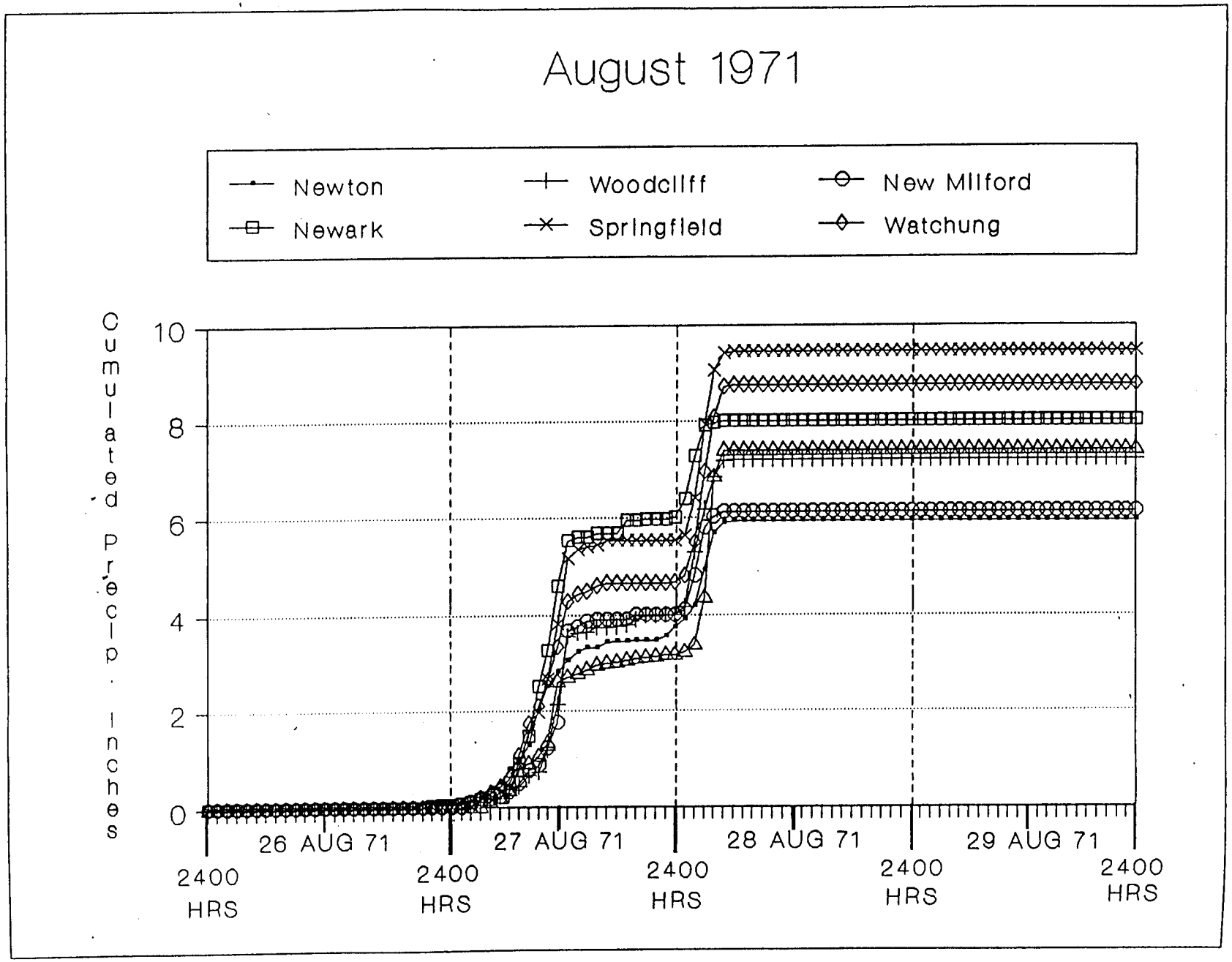
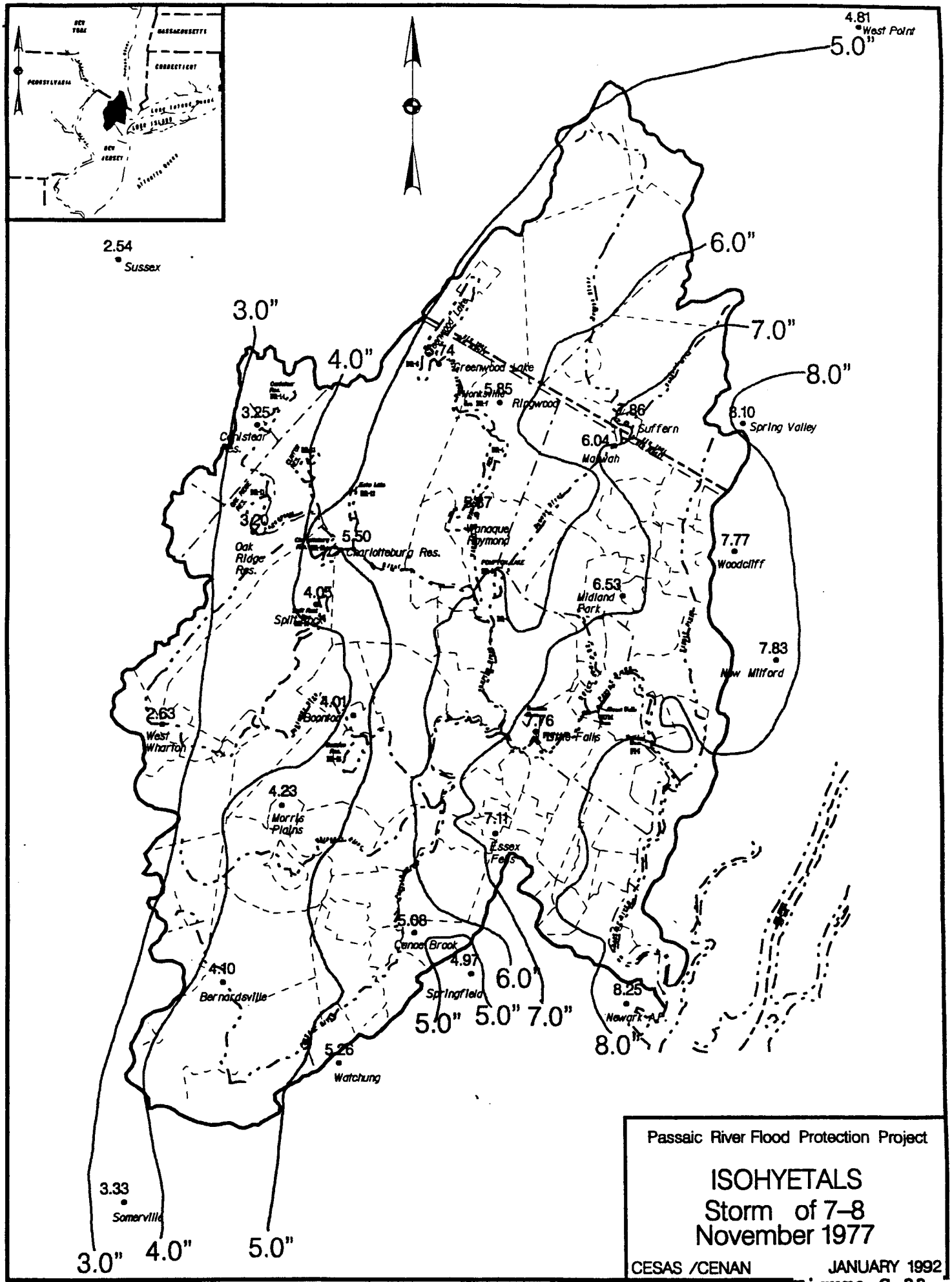

August 1971
Newton
Newark
Woodcliff
Springfield
New Milford
Watchung
Cumulated Precip - Inches
0
2
4
6
8
10
26 AUG 71
27 AUG 71
28 AUG 71
29 AUG 71
2400 HRS
2400 HRS
2400 HRS
2400 HRS
2400 HRS

Figure C-21



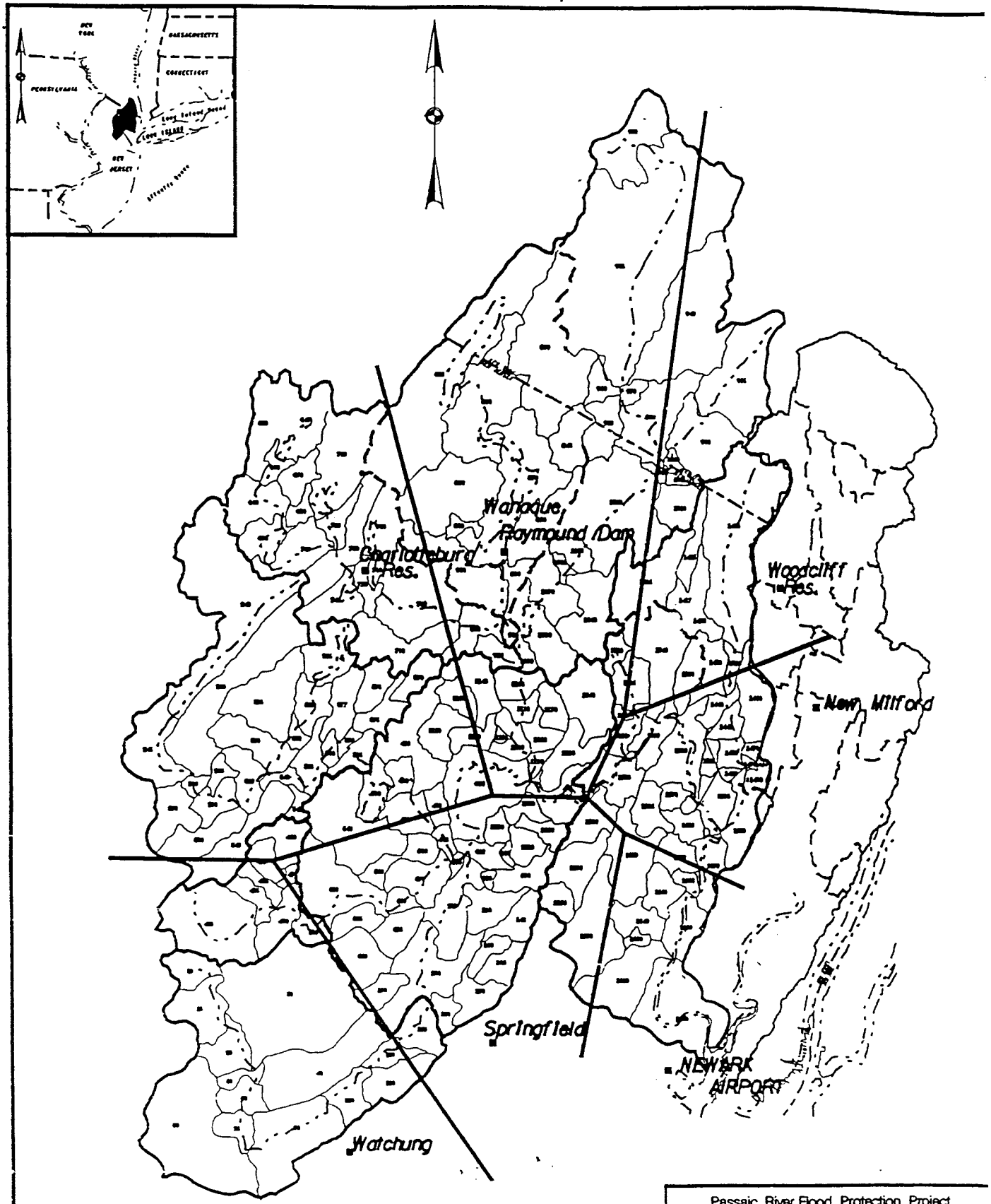
Passaic River Flood Protection Project

ISOHYETALS
Storm of 7–8 November 1977

CESAS /CENAN JANUARY 1992

Figure C-22

Revised on 05/31/



Passaic River Flood Protection Project

THIESSEN POLYGONS
STORM OF NOVEMBER
1977

CESAS /CENAN JANUARY 1992

Figure C-23

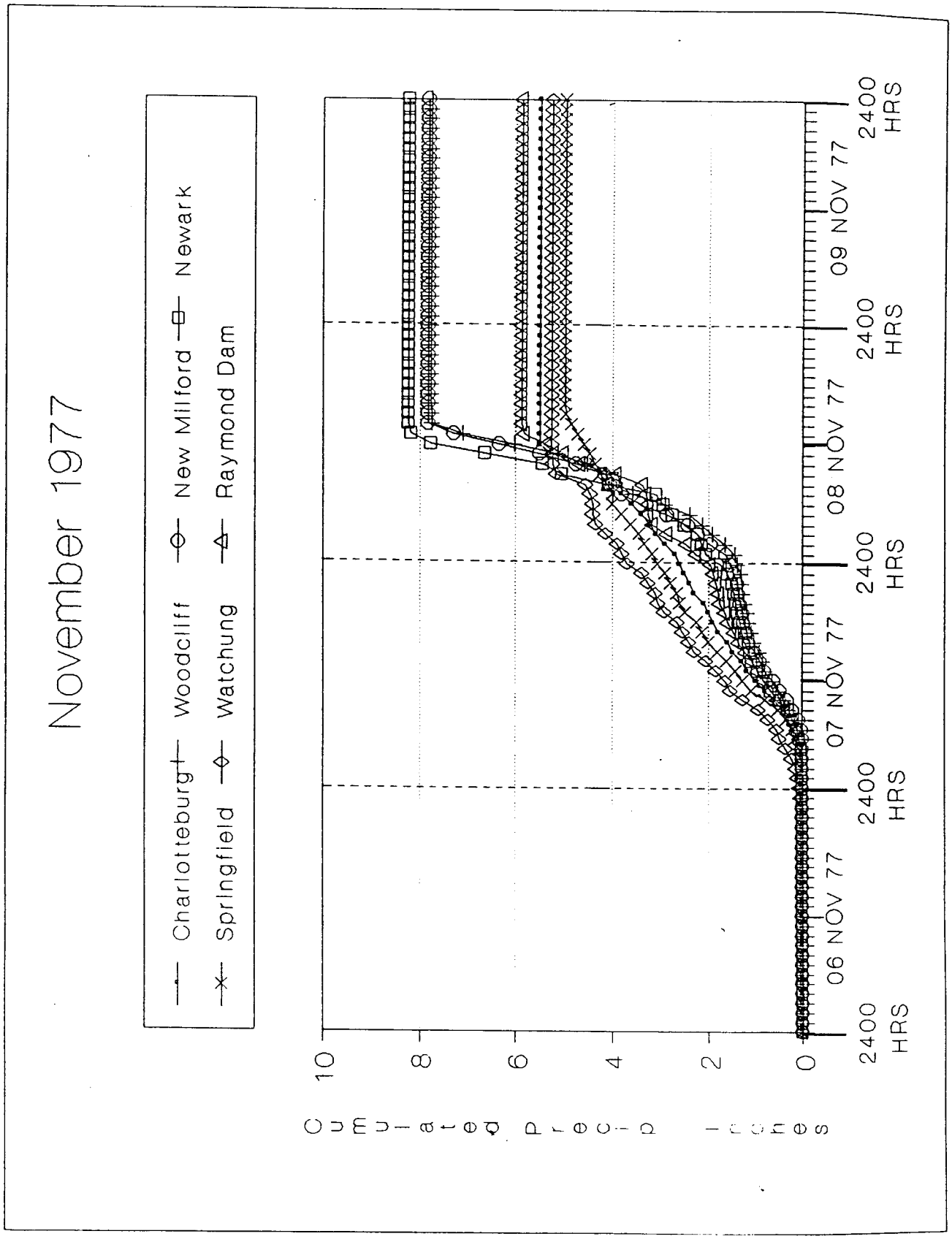
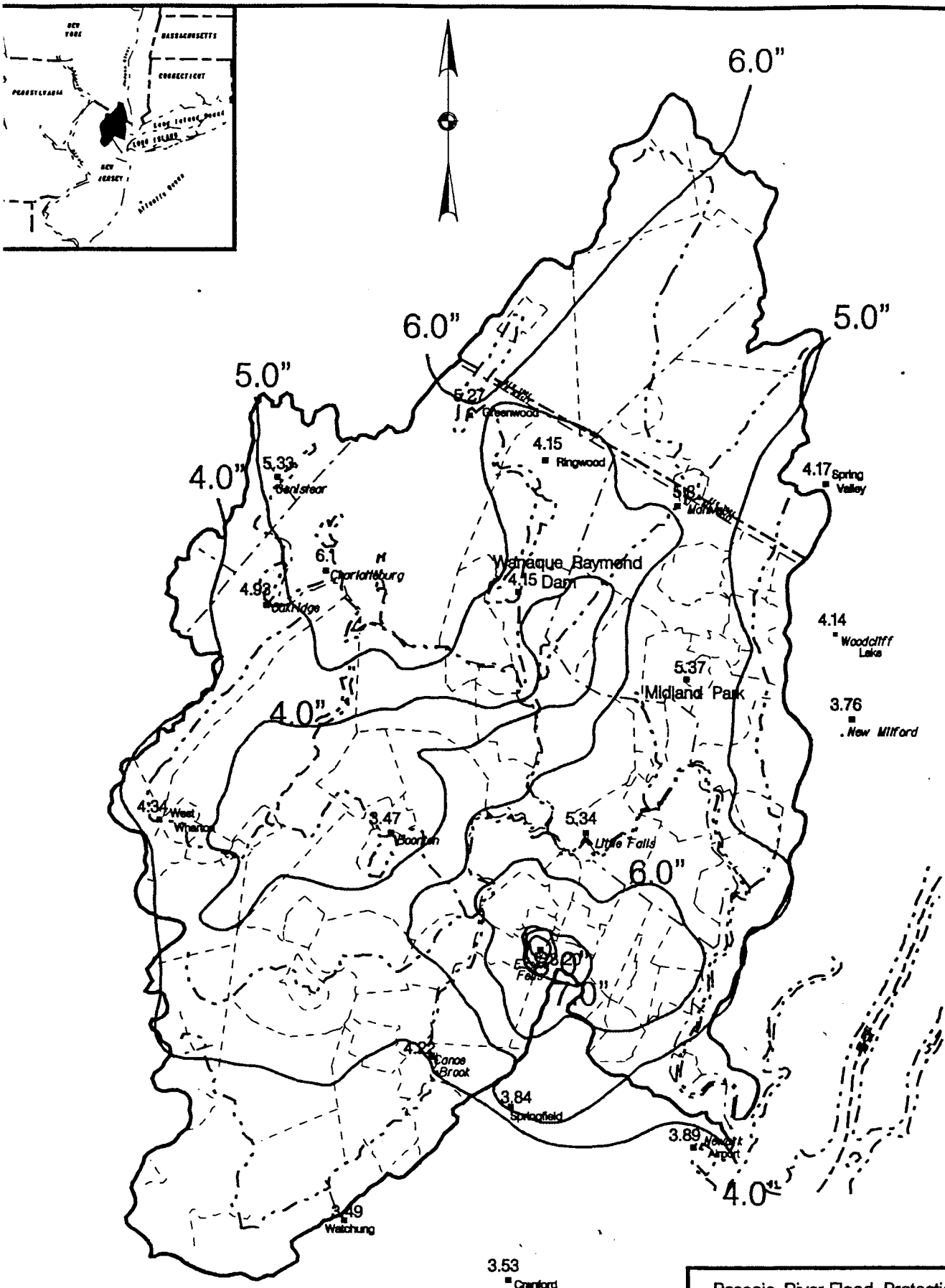

November 1977
Charlotteburg
Woodcliff
New Milford
Newark
Springfield
Watchung
Raymond Dam
Cumulated Precip - Inches
0
2
4
6
8
10
2400 HRS
06 NOV 77
2400 HRS
07 NOV 77
2400 HRS
08 NOV 77
2400 HRS
09 NOV 77
2400 HRS

Figure C-24

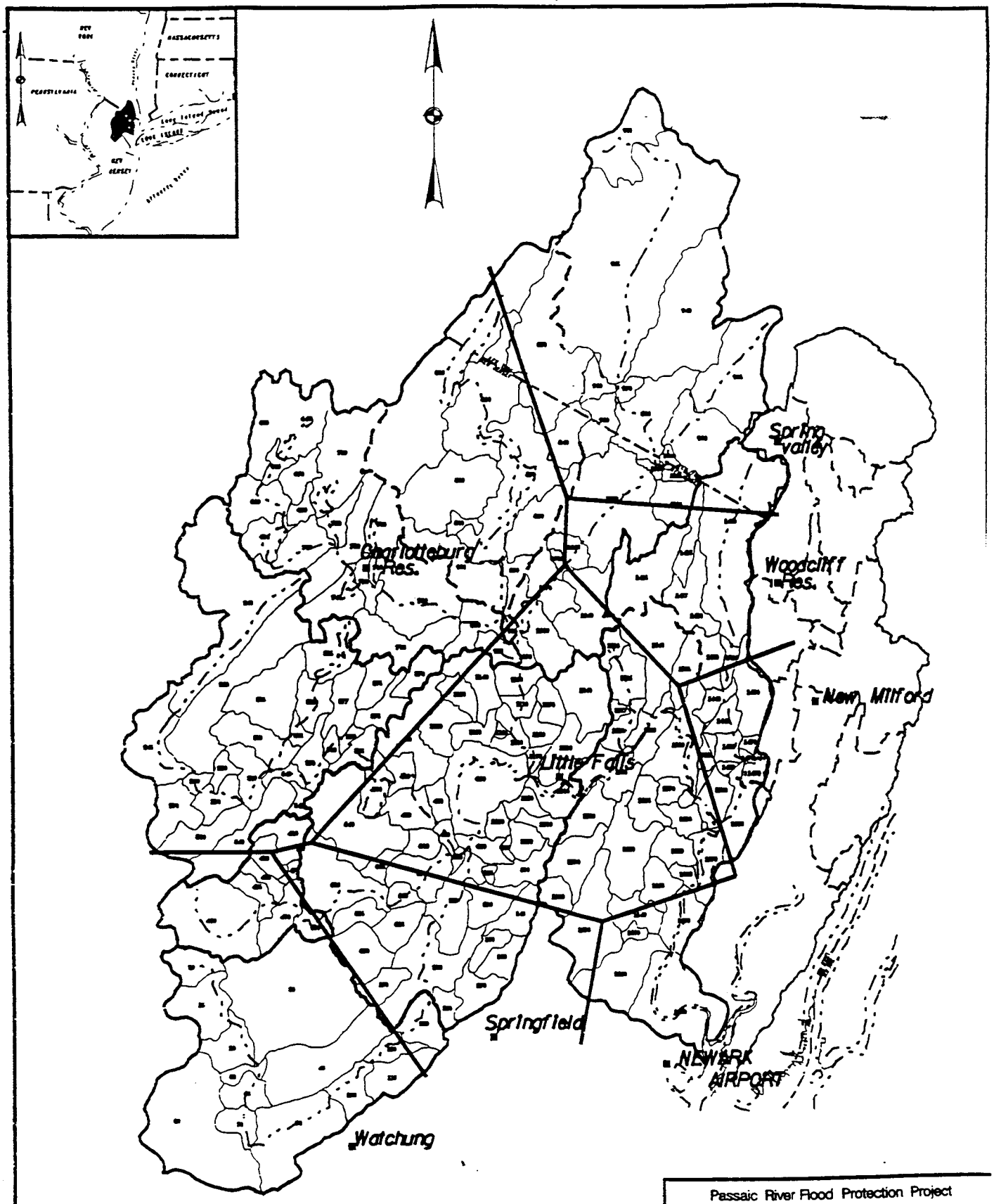


Passaic River Flood Protection Project

ISOHYETALS
Storm of April 1984

CESAS /CENAN JANUARY 1992

Figure C-25



Passaic River Flood Protection Project

THIESSEN POLYGONS
STORM OF APRIL 1984

CESAS /CENAN JANUARY 199

Figure C-26

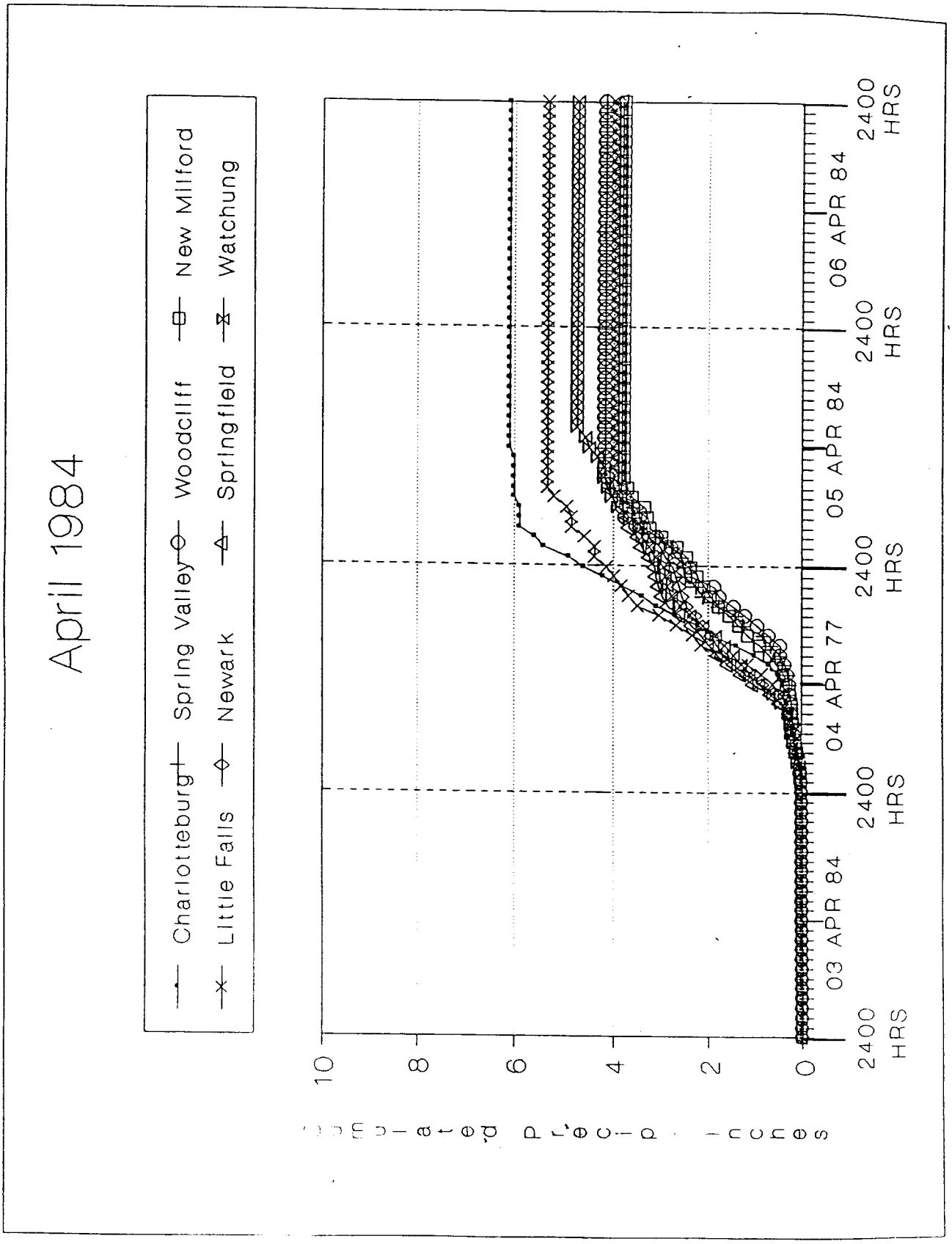
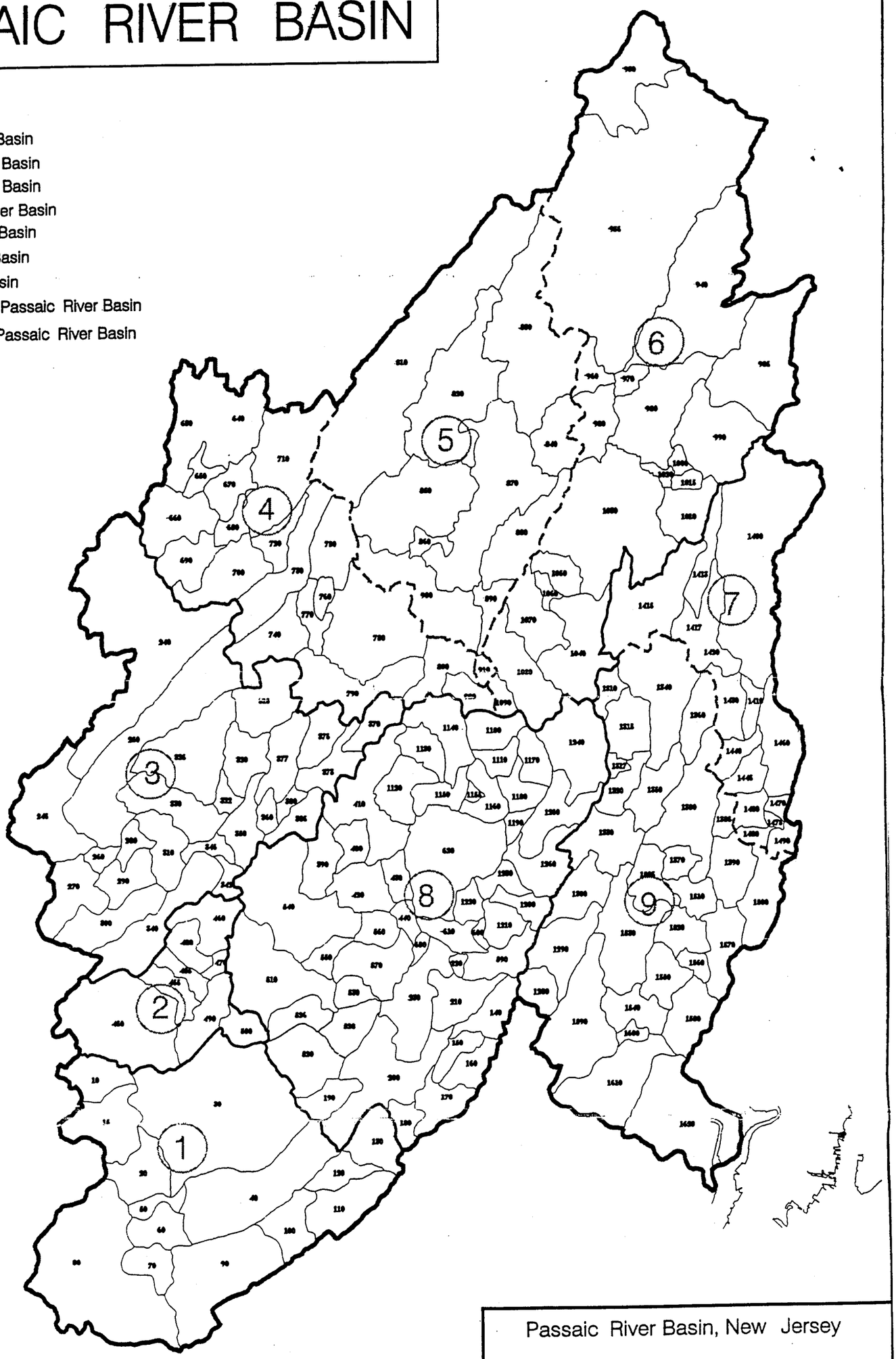

April 1984
Charlotteburg
Spring Valley
Woodcliff
New Milford
Little Falls
Newark
Springfield
Watchung
10
8
6
4
2
0
2400 HRS
03 APR 84
2400 HRS
04 APR 77
2400 HRS
05 APR 84
2400 HRS
06 APR 84
2400 HRS

Figure C-27

PASSAIC RIVER BASIN

LEGEND:

1=Upper Passaic Basin
2=Whippany River Basin
3=Rockaway River Basin
4=Pequannock River Basin
5=Wanaque River Basin
6=Ramapo River Basin
7=Saddle River Basin
8=Central Basin of Passaic River Basin
9=Lower Valley of Passaic River Basin

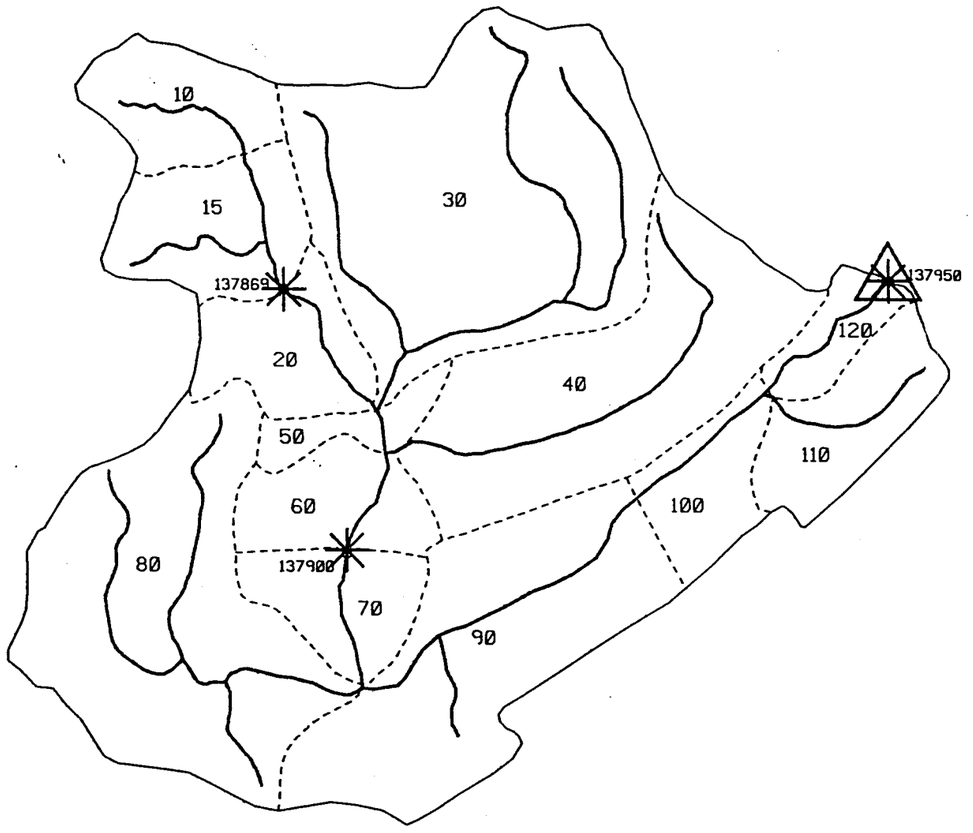


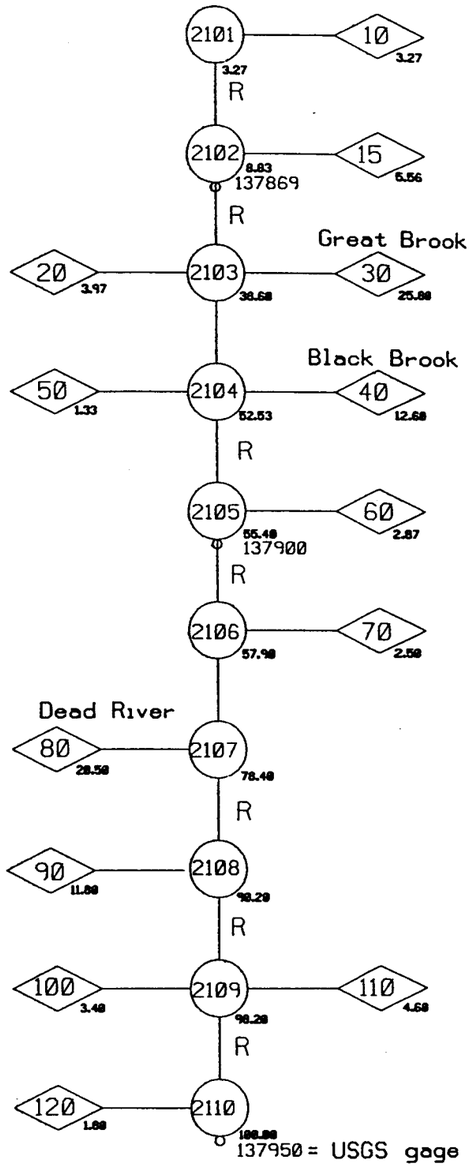
Passaic River Basin, New Jersey

HEC1 /UNET
Subarea Delineations

Figure C-28

PASSAIC RIVER BASIN ABOVE CHATHAM, NJ





KEY

- Subbasin Number and Drainage Area (Sq. Mi.)
- Node Point and Number
- U.S.G.S. Gage
- R — Route Reach
- UNET Boundary Condition Location

Note: Traditional HEC1 modeling is used for the portion of the Passaic River, as shown on this figure.

Passaic River Study
Passaic River above Chatham,NJ

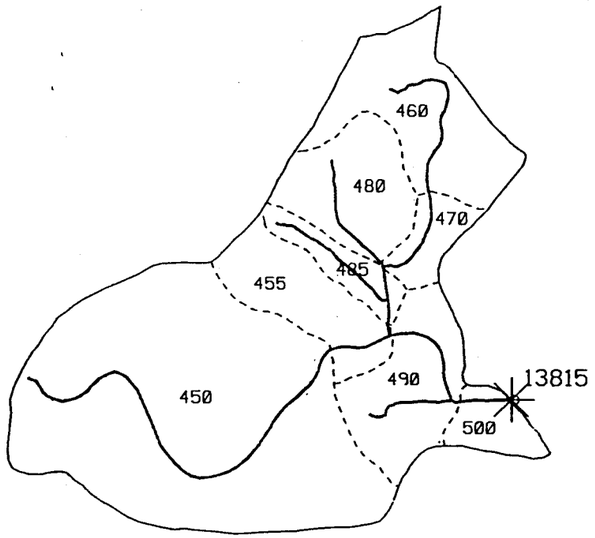
HEC1/UNET SUBAREA NETWORK DIAGRAM

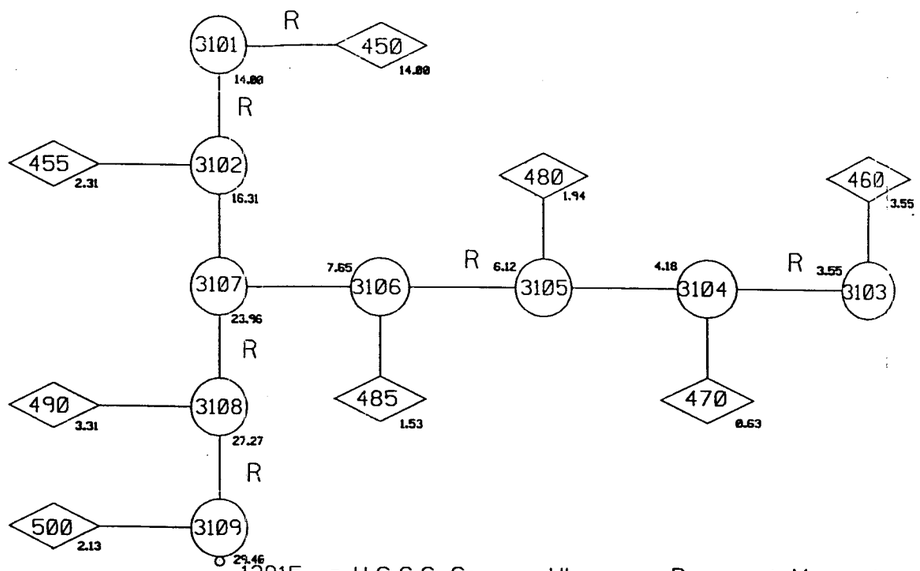
Figure C-29

KEY

- Subbasin Number and Drainage Area (Sq. Mi.)
- Node Point and Number
- U.S.G.S. Gage
- R — Route Reach

WHIPPANY RIVER AT MORRISTOWN, NJ





13815 = U.S.G.S. Gage - Whippany River at Morristown, N.J.

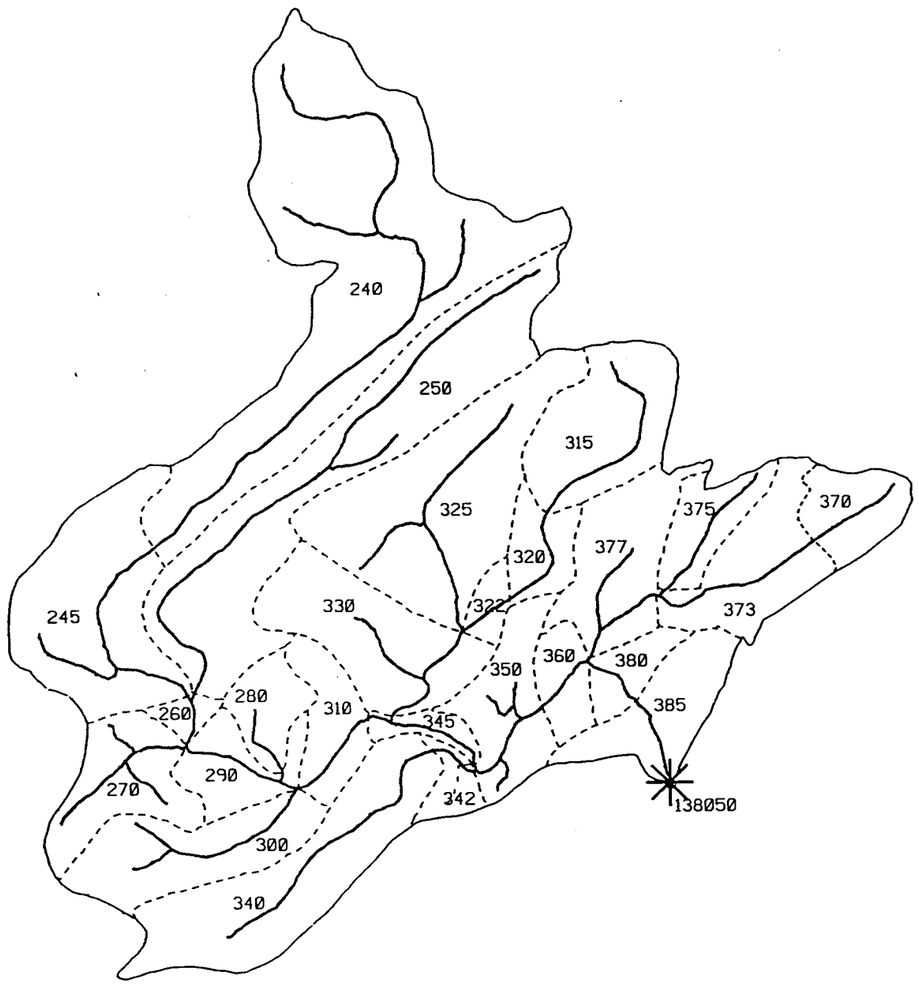
Note: Traditional HEC1 modeling was used for the portion of the Whippany River, as shown on this figure.

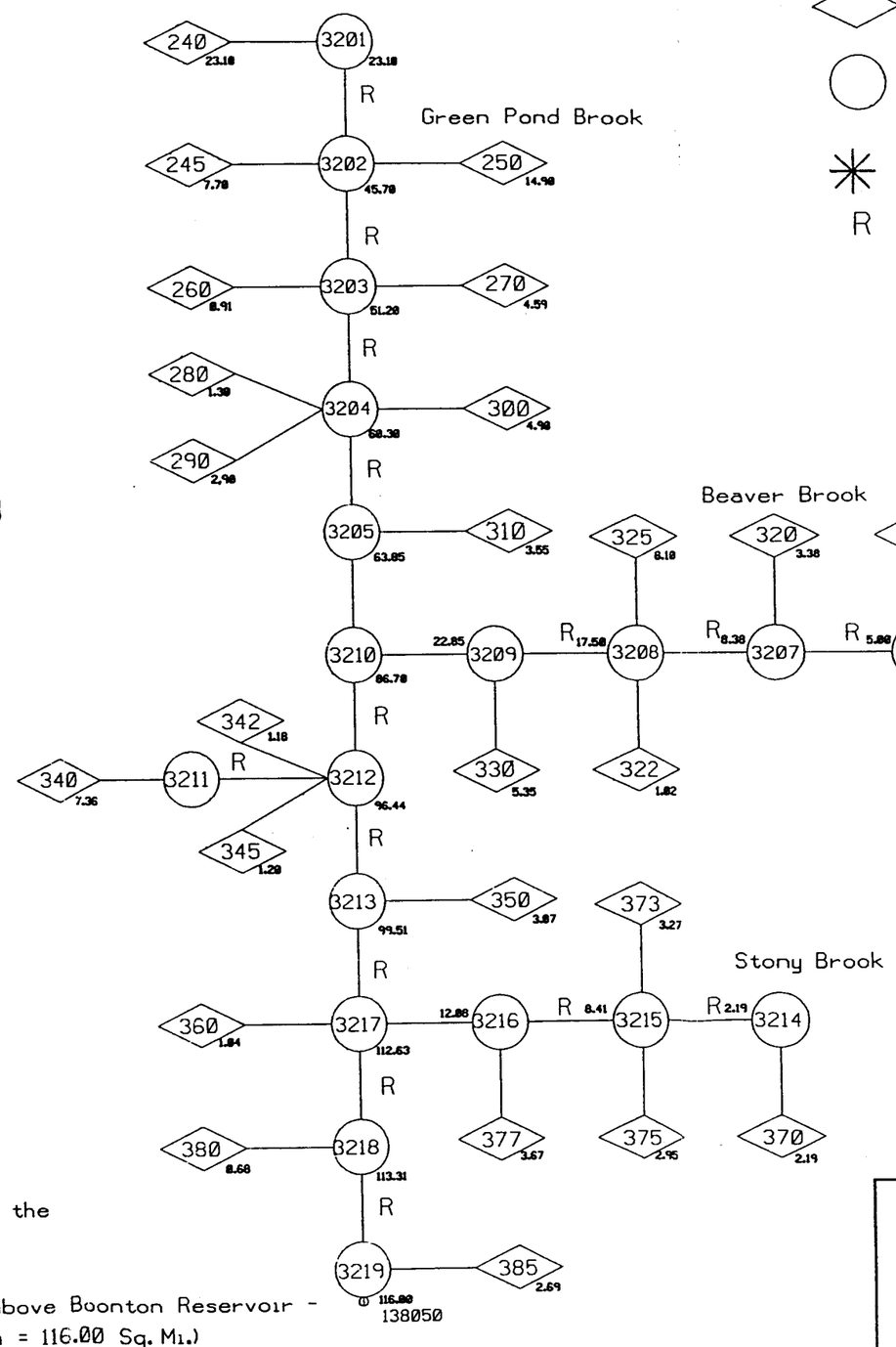
Passaic River Study
WHIPPANY RIVER

HEC1/UNET SUBAREA NETWORK DIAGRAM

Figure C-30

ROCKAWAY RIVER ABOVE BOONTON RESERVOIR





KEY

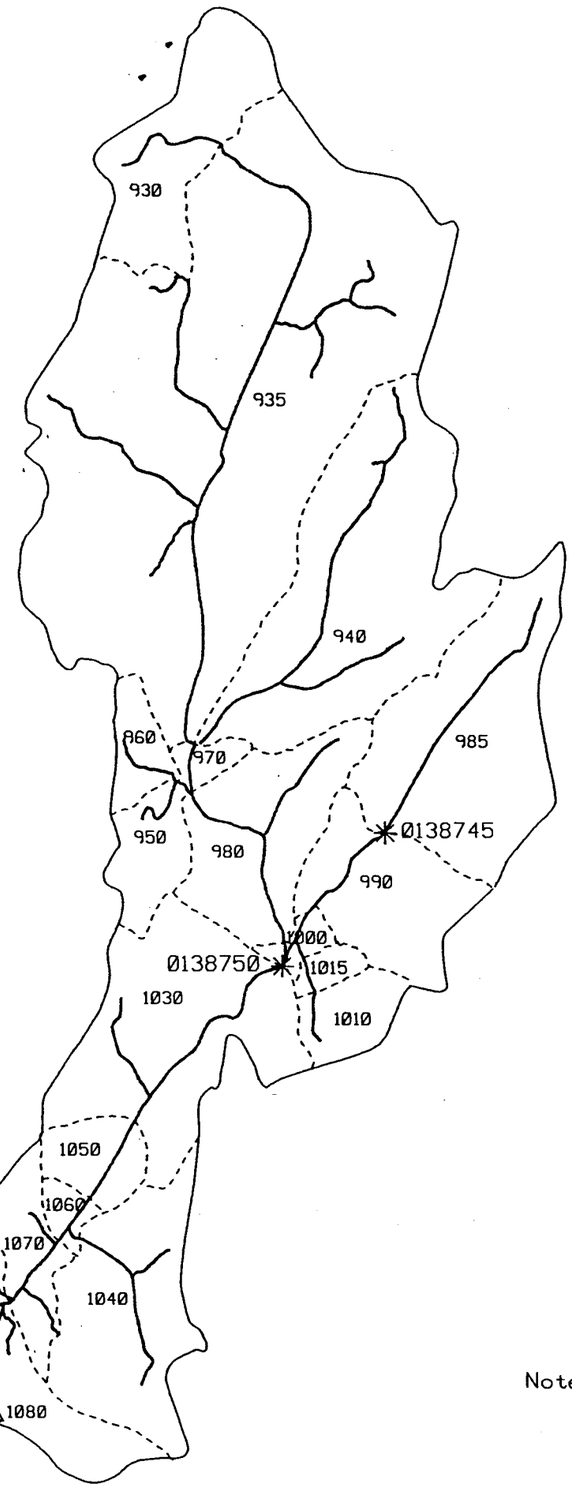
- Subbasin Number and Drainage Area (Sq. Mi.)
- Node Point and Number
- U.S.G.S. Gage
- R — Route Reach

Note: Traditional HEC1 modeling was used for the portion of the Rockaway River, as shown on this figure.

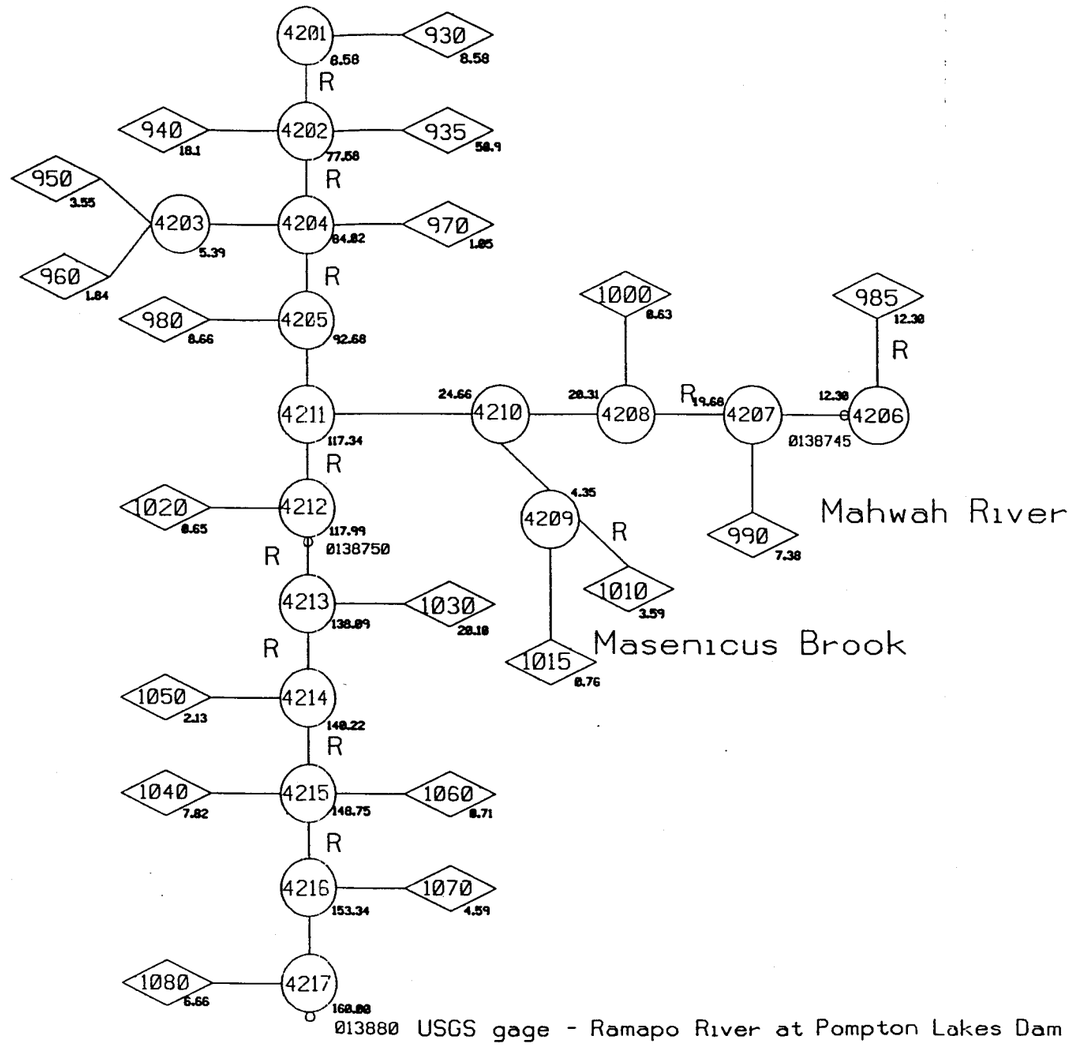
Rockaway River above Boonton Reservoir -
(DA = 116.00 Sq. Mi.)

Passaic River Study
ROCKAWAY RIVER ABOVE BOONTON RESERVOIR
HEC1 / UNET SUBAREA NETWORK DIAGRAM

Figure C-31



RAMAPO RIVER BASIN



013880 USGS gage - Ramapo River at Pompton Lakes Dam
(DA = 160 Sq.Mi.)

KEY

- Subbasin Number and Drainage Area (Sq. Mi.)
- Node Point and Number
- U.S.G.S. Gage
- UNET Boundary Condition Location
- R — Route Reach

Note: Traditional HEC1 modeling is used for the portion of the Ramapo River, as shown on this figure.

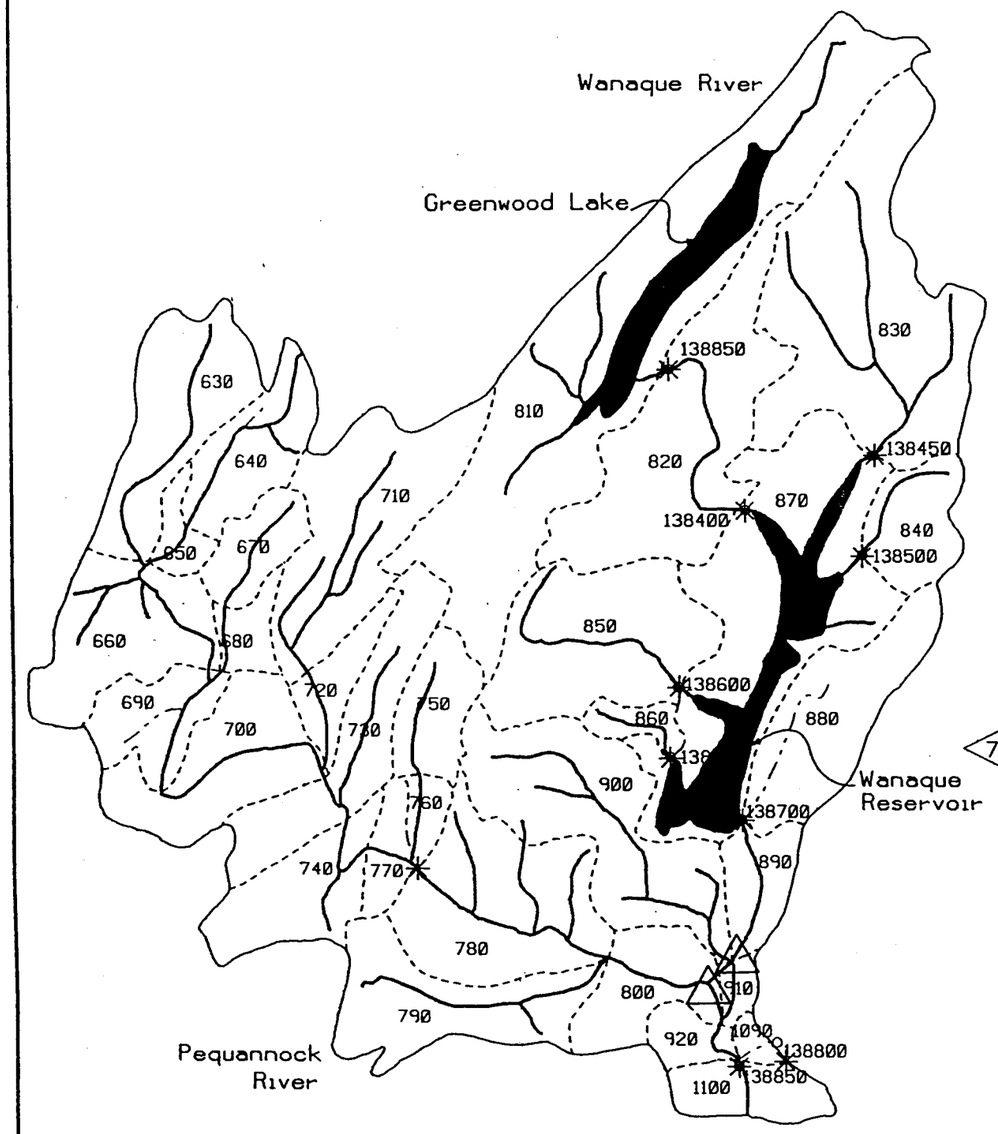
Passaic River Study
RAMAPO RIVER BASIN
HEC1 / UNET SUBAREA
NETWORK DIAGRAM

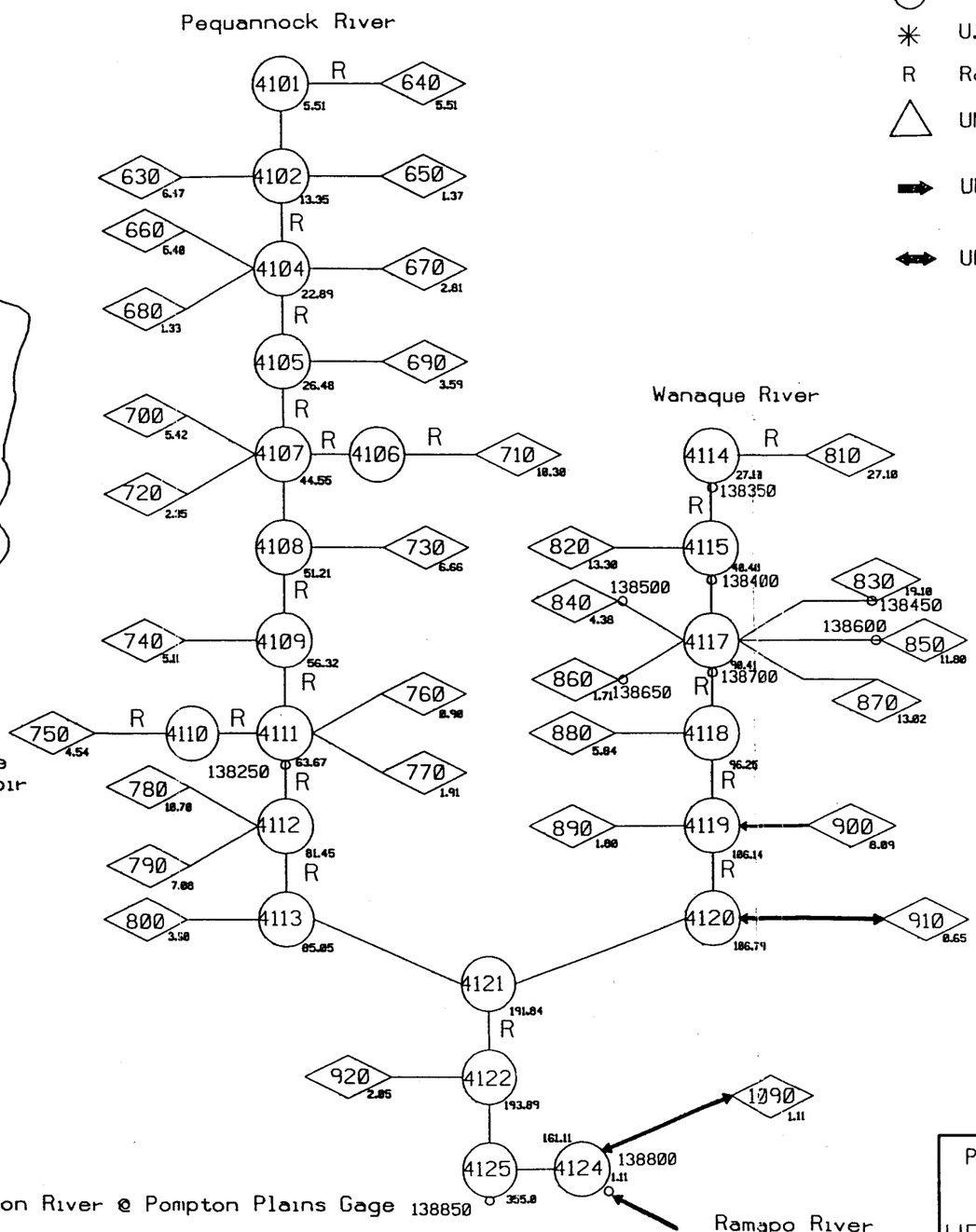
Figure C-32

POMPTON RIVER BASIN

KEY

- Subbasin Number and Drainage Area (Sq. Mi.)
- Note Point Number
- U.S.G.S. Gage
- R Route Reach
- UNET Boundary Condition Location
- UNET Point Inflow Subbasin
- UNET Lateral Inflow Subbasin





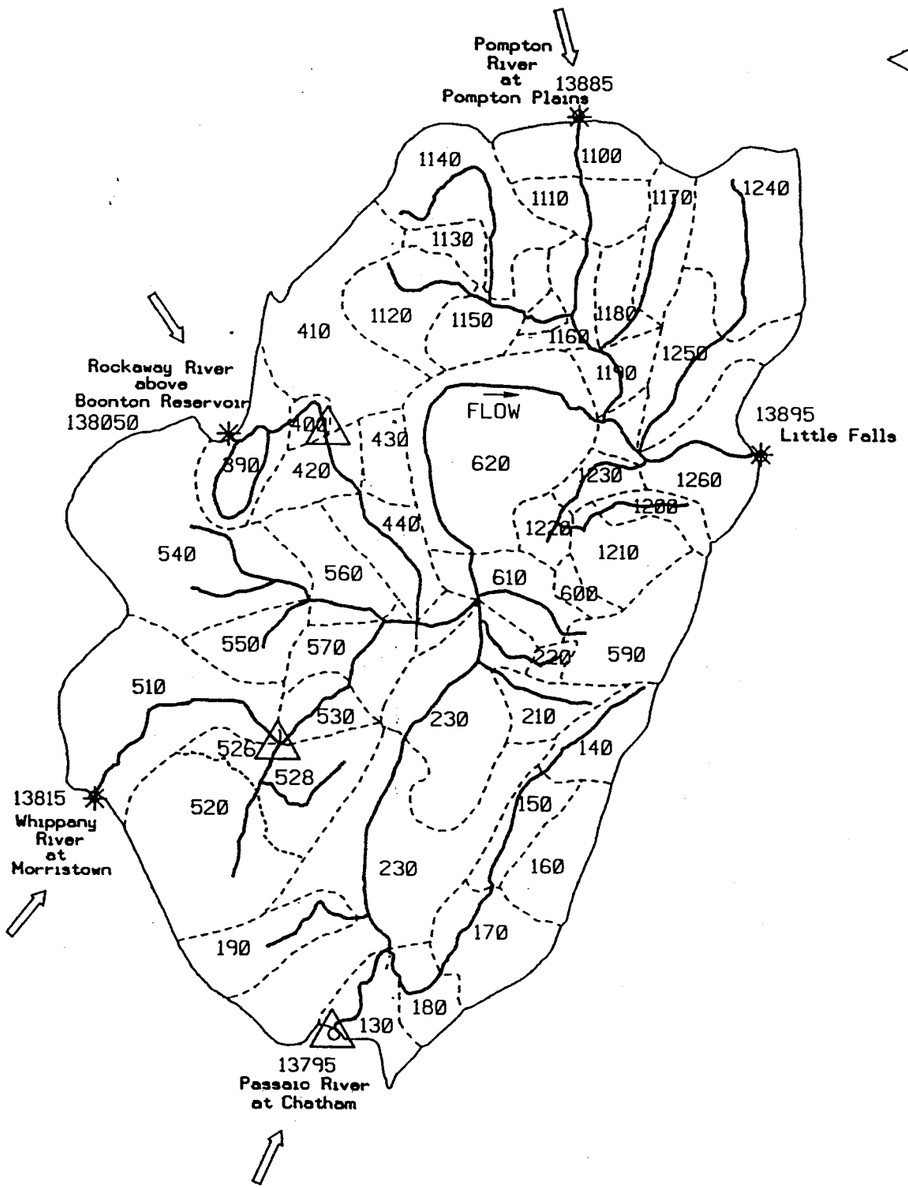
Passaic River Study

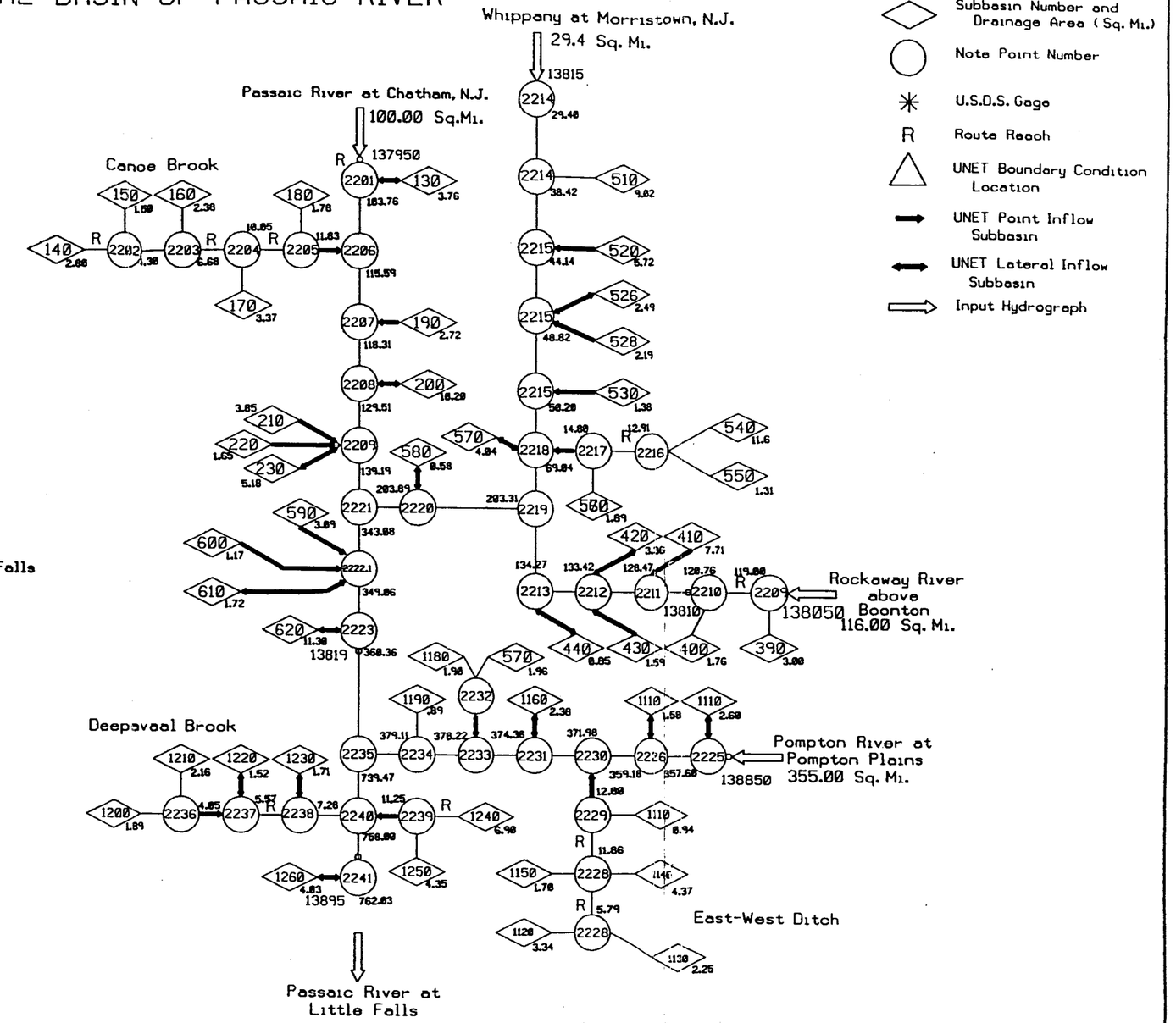
POMPTON RIVER BASIN

HEC1 / UNET SUBAREA NETWORK DIAGRAM

Figure C-33

CENTRAL BASIN OF PASSAIC RIVER

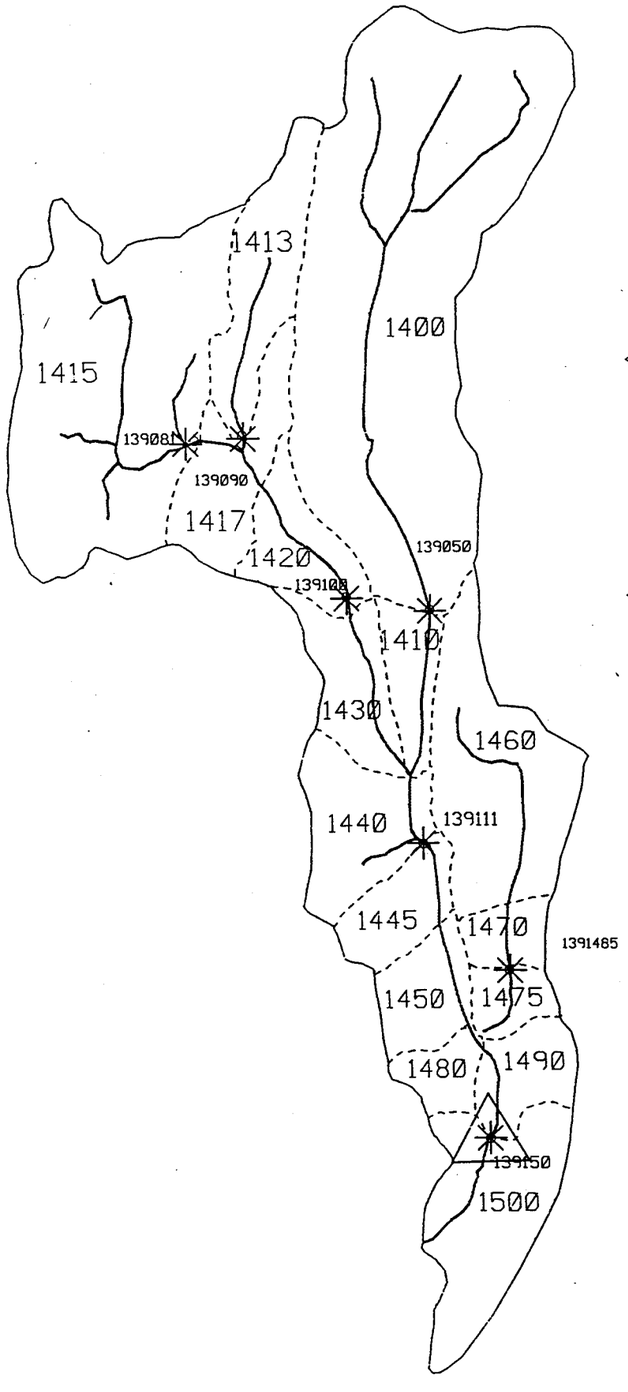




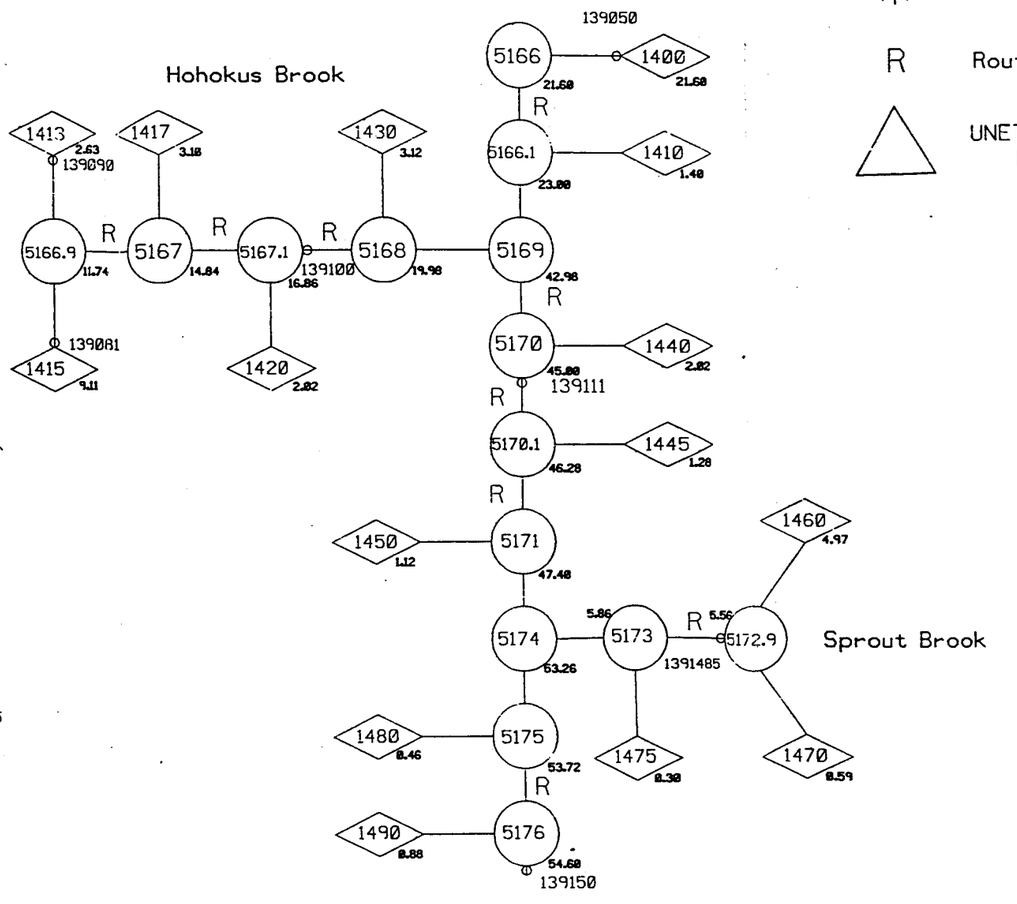
Passaic River Study
CENTRAL BASIN

HEC1/UNET SUBAREA NETWORK DIAGRAM

Figure C-34



SADDLE RIVER BASIN



KEY

- Subbasin Number and Drainage Area (Sq. Mi.)
- Node Point and Number
- U.S.G.S. Gage
- R — Route Reach
- UNET Boundary Condition Location

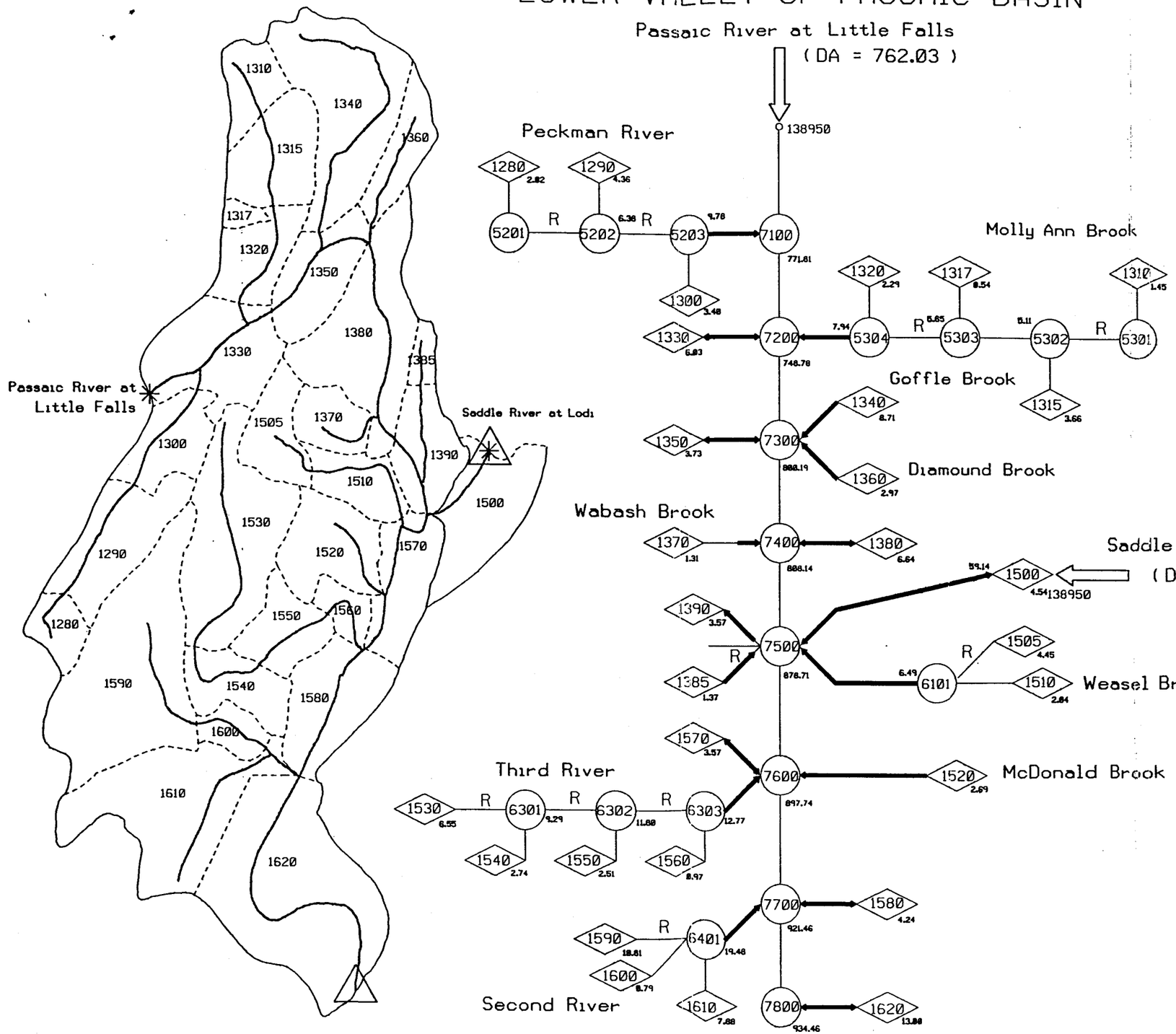
Note: Traditonal HEC1 modeling is used for the portion of the Saddle River downstream through Subbasin 1490, as shown on this figure.

Passaic River Study
SADDLE RIVER BASIN

HEC1 / UNET SUBAREA NETWORK DIAGRAM

Figure C-35

LOWER VALLEY OF PASSAIC BASIN



KEY

- Subbasin Number and Drainage Area (Sq. Mi.)
- Note Point Number
- U.S.G.S Gage
- R — Route Reach
- UNET Boundary Condition Location
- UNET Point Inflow Subbasin
- UNET Lateral Inflow Subbasin
- Input Hydrograph

lwrvally.dgn

Passaic River Study
LOWER VALLEY

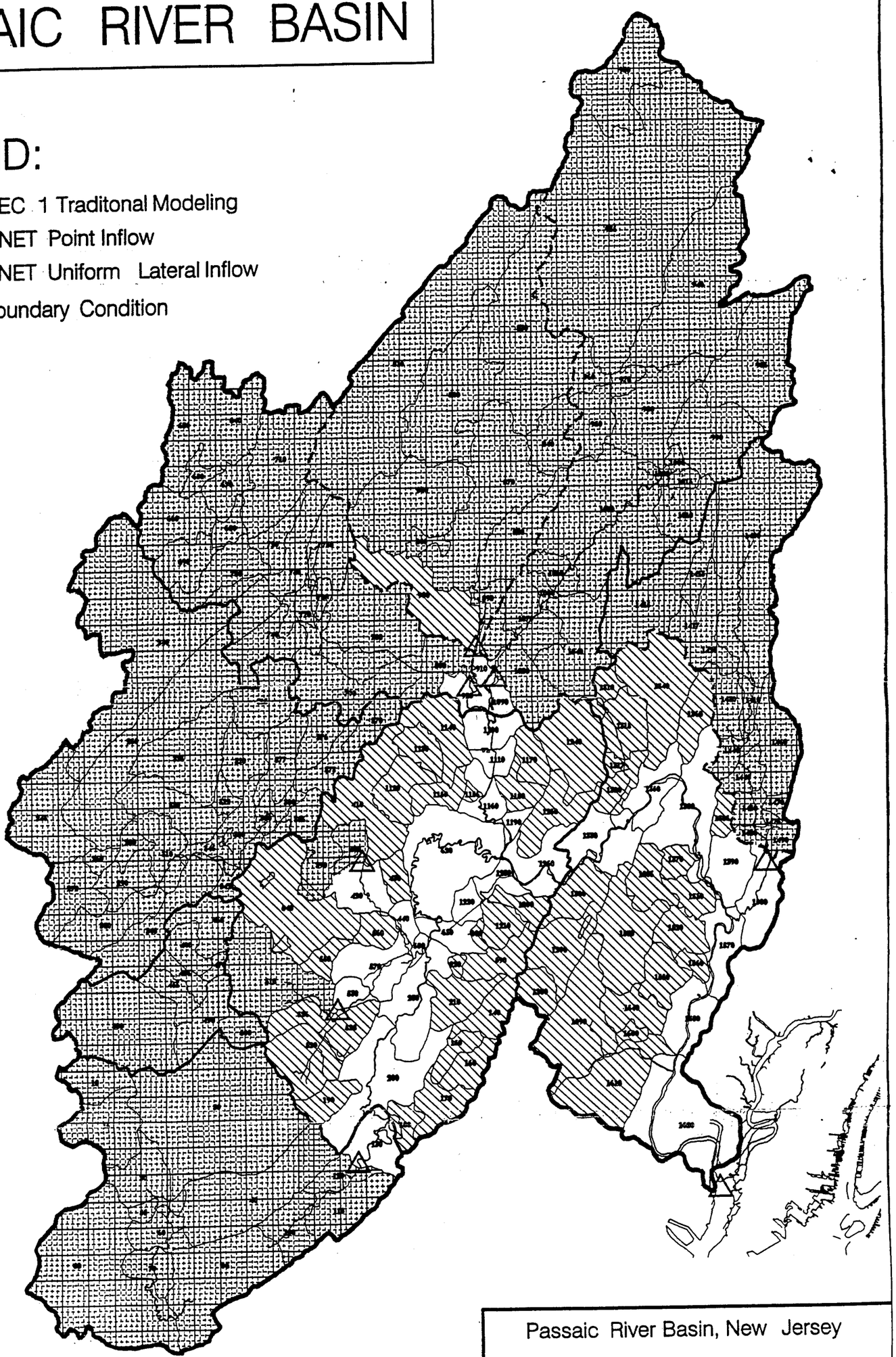
HEC1/UNET SUBAREA NETWORK DIAGRAM

Figure C-36

PASSAIC RIVER BASIN

LEGEND:

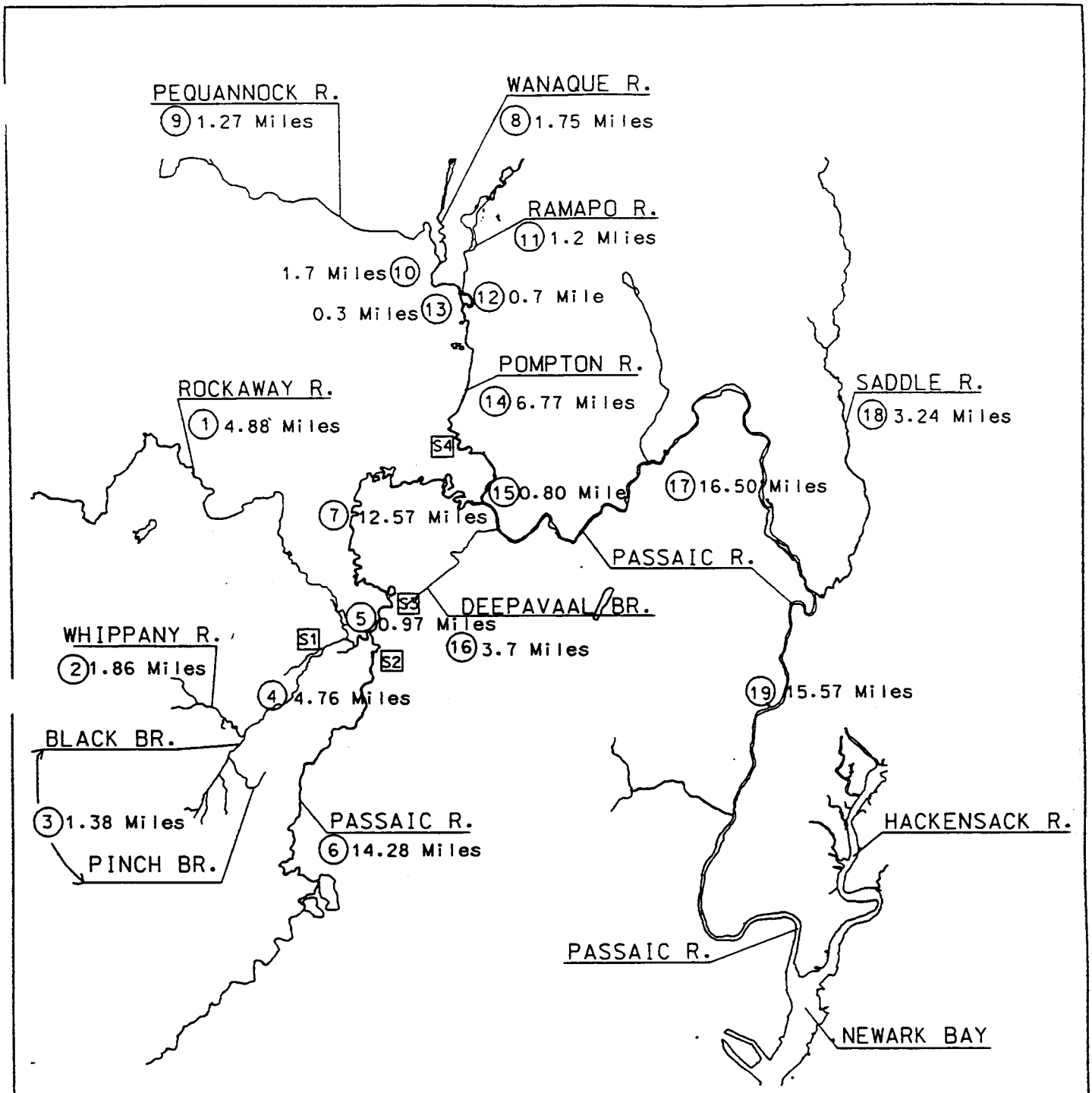
- HEC 1 Traditonal Modeling
- UNET Point Inflow
- UNET Uniform Lateral Inflow
- △ Boundary Condition



Passaic River Basin, New Jersey

HEC1 /UNET
Boundary, Point and Lateral
Inflows

Figure C-37



UNET Storage Areas

No.	Location	Area (Acres)	UNET Reach No.
S1	Troy Meadow	832	4
S2	Hatfield	224	6
S3	Hatfield 2	179	7
S4	Bog and Vly	536	14

Passaic River Flood Control Project

PHASE II GDM

UNET REACH LOCATIONS

CENAN-PR-C MARCH 1992

Figure C-38

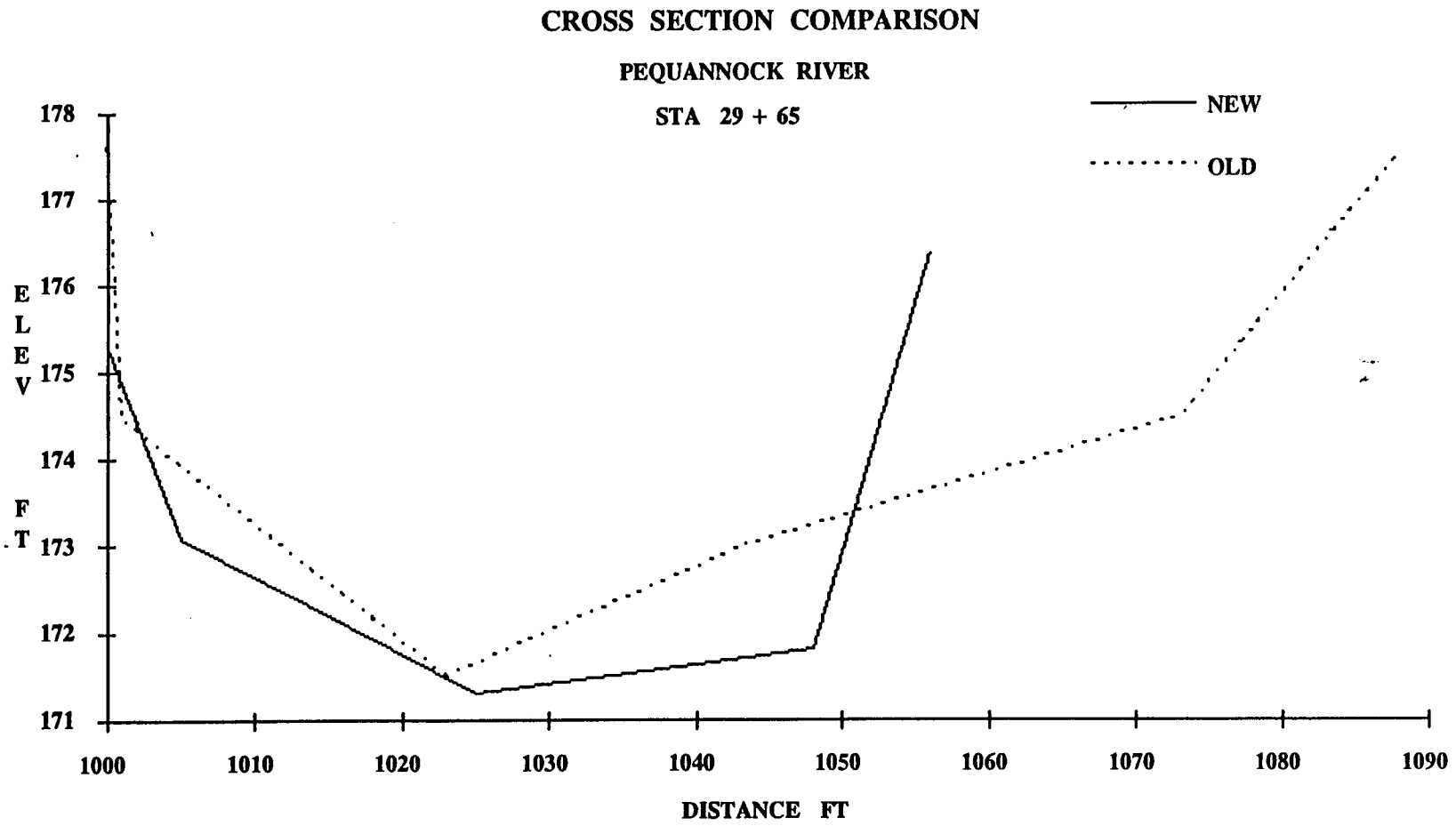

CROSS SECTION COMPARISON
PEQUANNOCK RIVER
STA 29 + 65
NEW
OLD
178
177
176
175
174
173
172
171
ELEV FT
1000
1010
1020
1030
1040
1050
1060
1070
1080
1090
DISTANCE FT

Figure C-39a

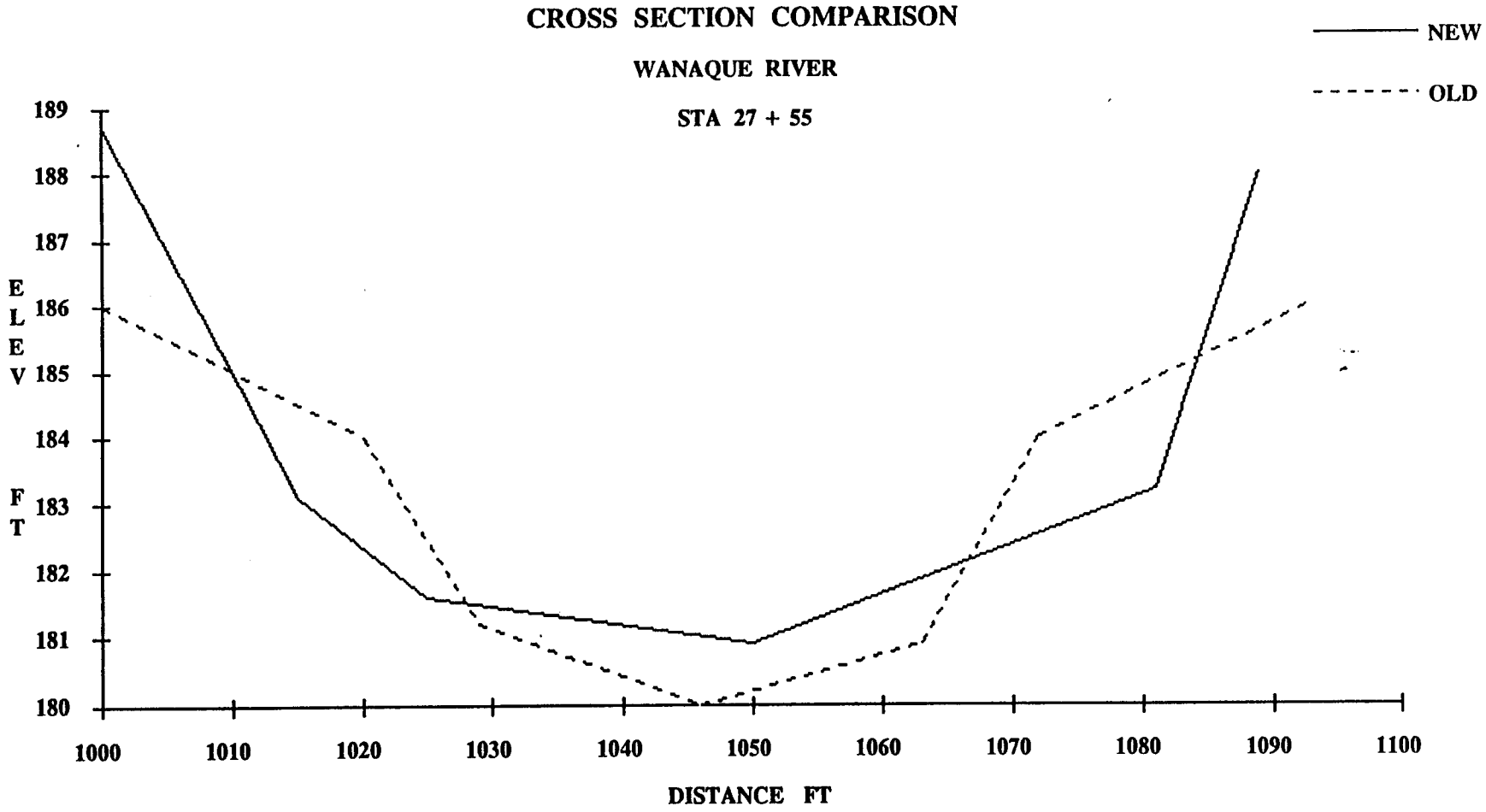

CROSS SECTION COMPARISON
WANAQUE RIVER
STA 27 + 55
NEW
OLD
ELEV FT
189
188
187
186
185
184
183
182
181
180
1000
1010
1020
1030
1040
1050
1060
1070
1080
1090
1100
DISTANCE FT

Figure C-39b

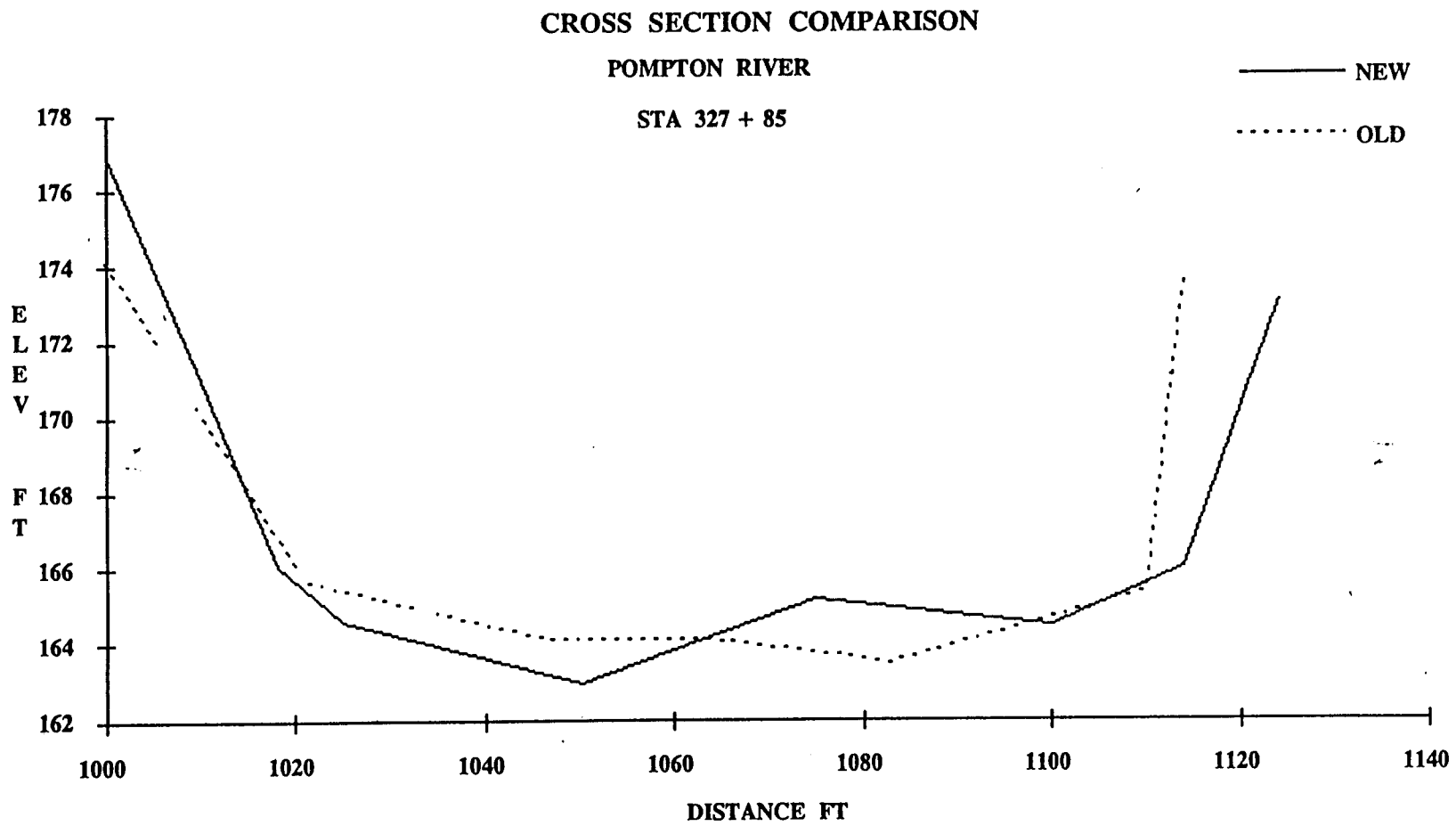

CROSS SECTION COMPARISON
POMPTON RIVER
STA 327 + 85
NEW
OLD
178
176
174
172
170
168
166
164
162
E
L
E
V
F
T
1000
1020
1040
1060
1080
1100
1120
1140
DISTANCE FT

Figure C-39c

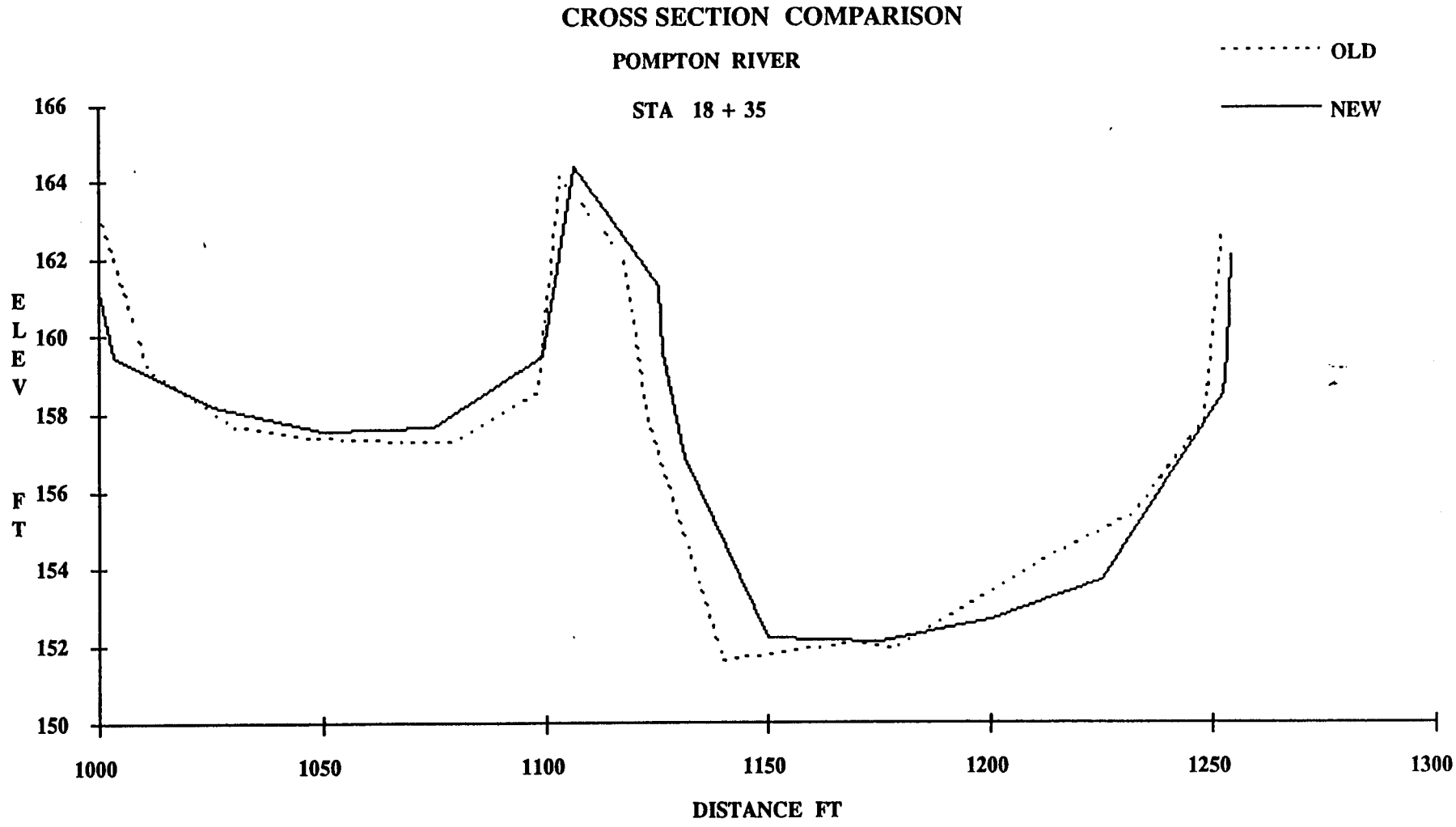

CROSS SECTION COMPARISON
POMPTON RIVER
STA 18 + 35
OLD
NEW
ELEV FT
166
164
162
160
158
156
154
152
150
1000
1050
1100
1150
1200
1250
1300
DISTANCE FT

Figure C-39d

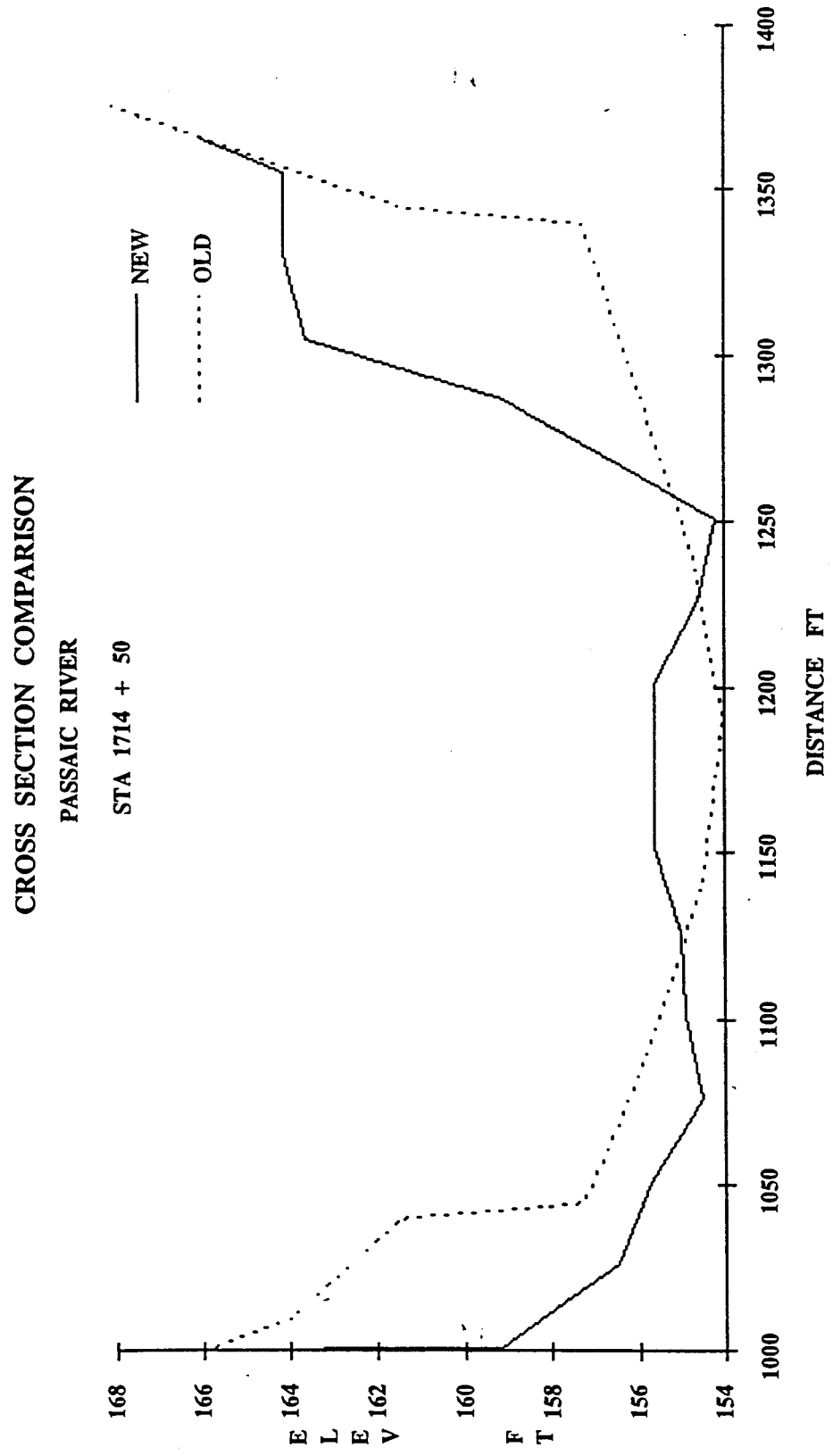

CROSS SECTION COMPARISON
PASSAIC RIVER
STA 1714 + 50
NEW
OLD
ELEV FT
168
166
164
162
160
158
156
154
1000
1050
1100
1150
1200
1250
1300
1350
1400
DISTANCE FT

Figure C-39e

CROSS SECTION COMPARISON

PASSAIC RIVER

STA 1307 + 00

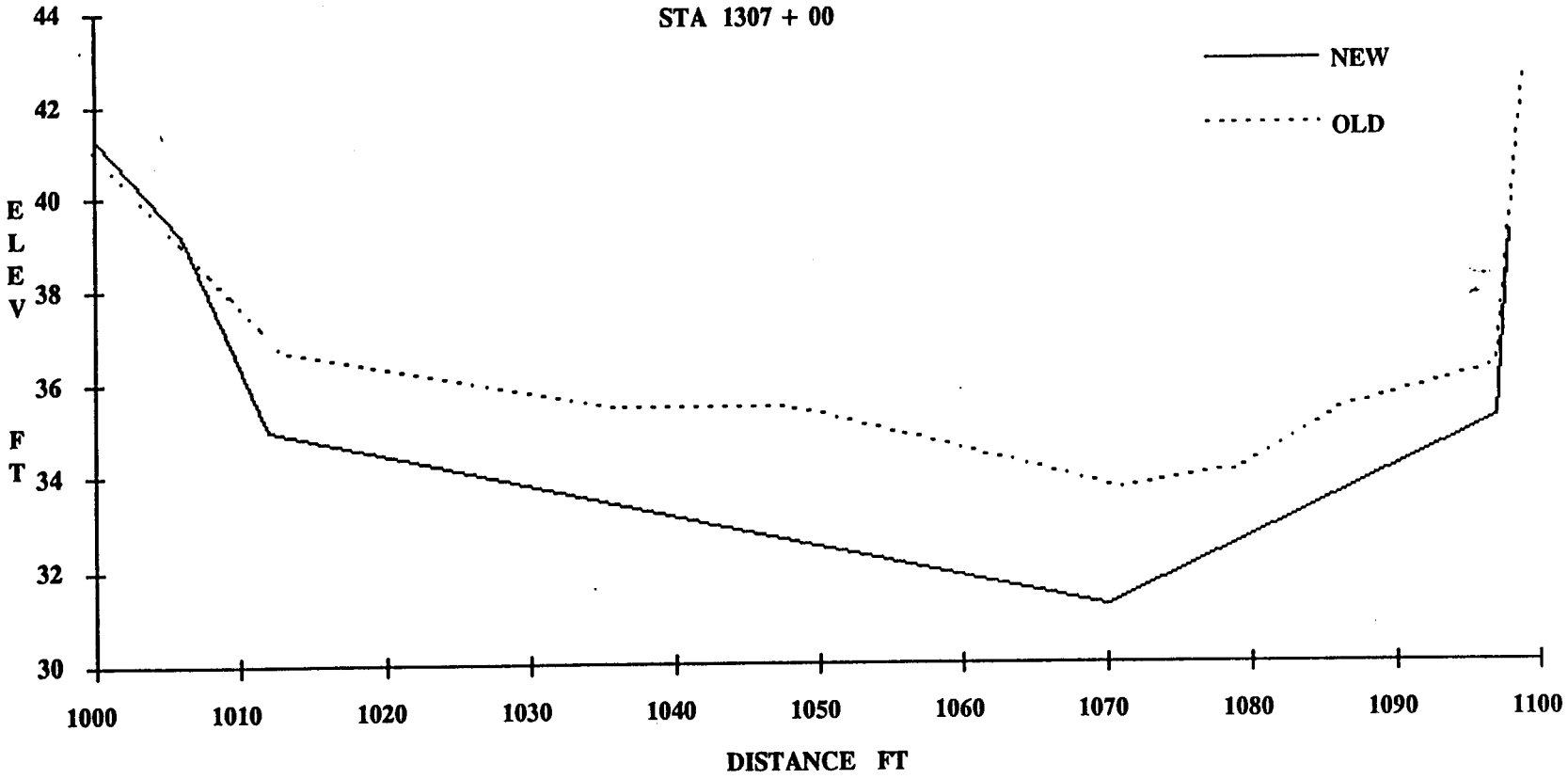


Figure C-39f

CROSS SECTION COMPARISON

PASSAIC RIVER

STA 782 + 00

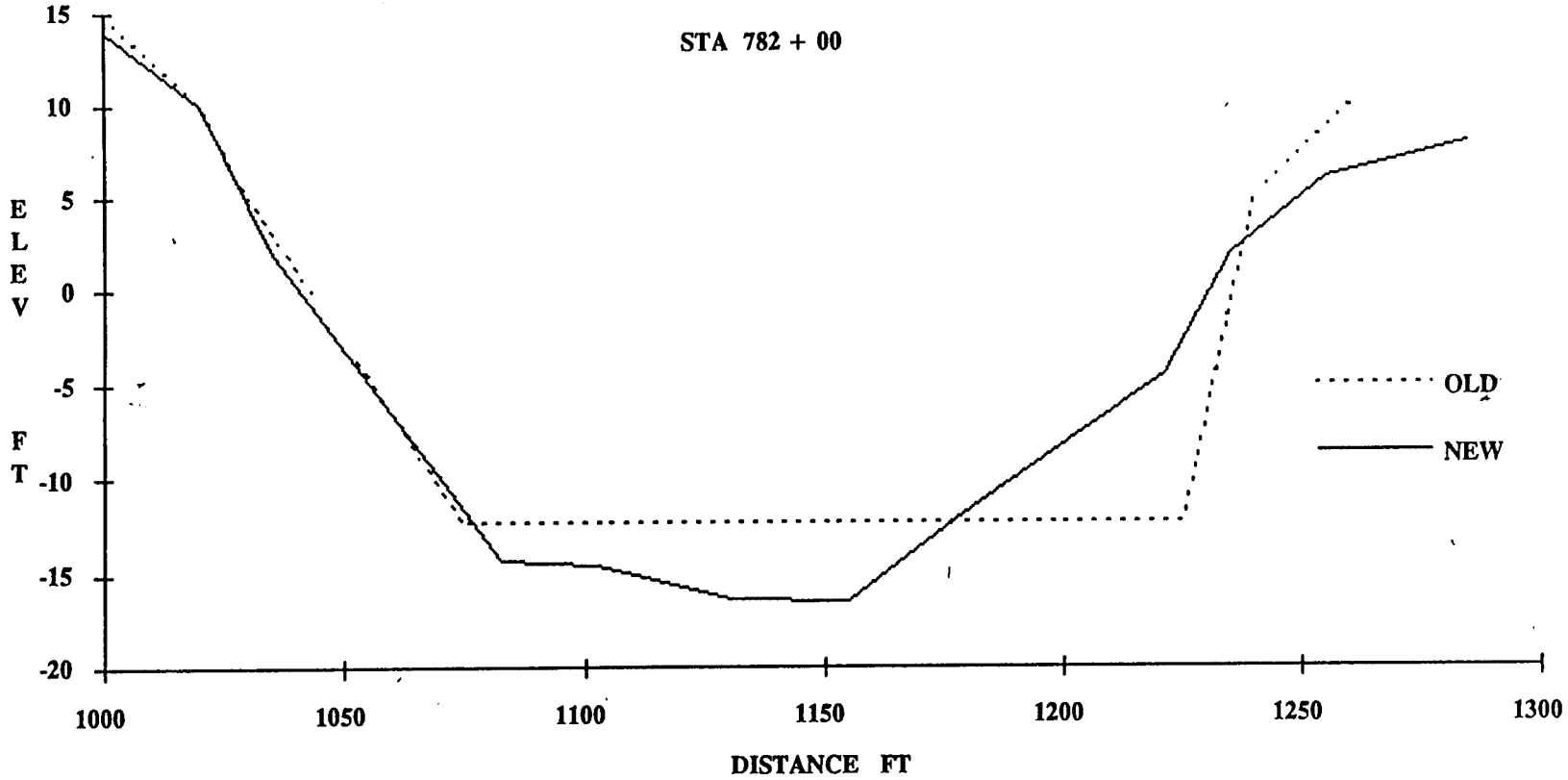


Figure C-39g

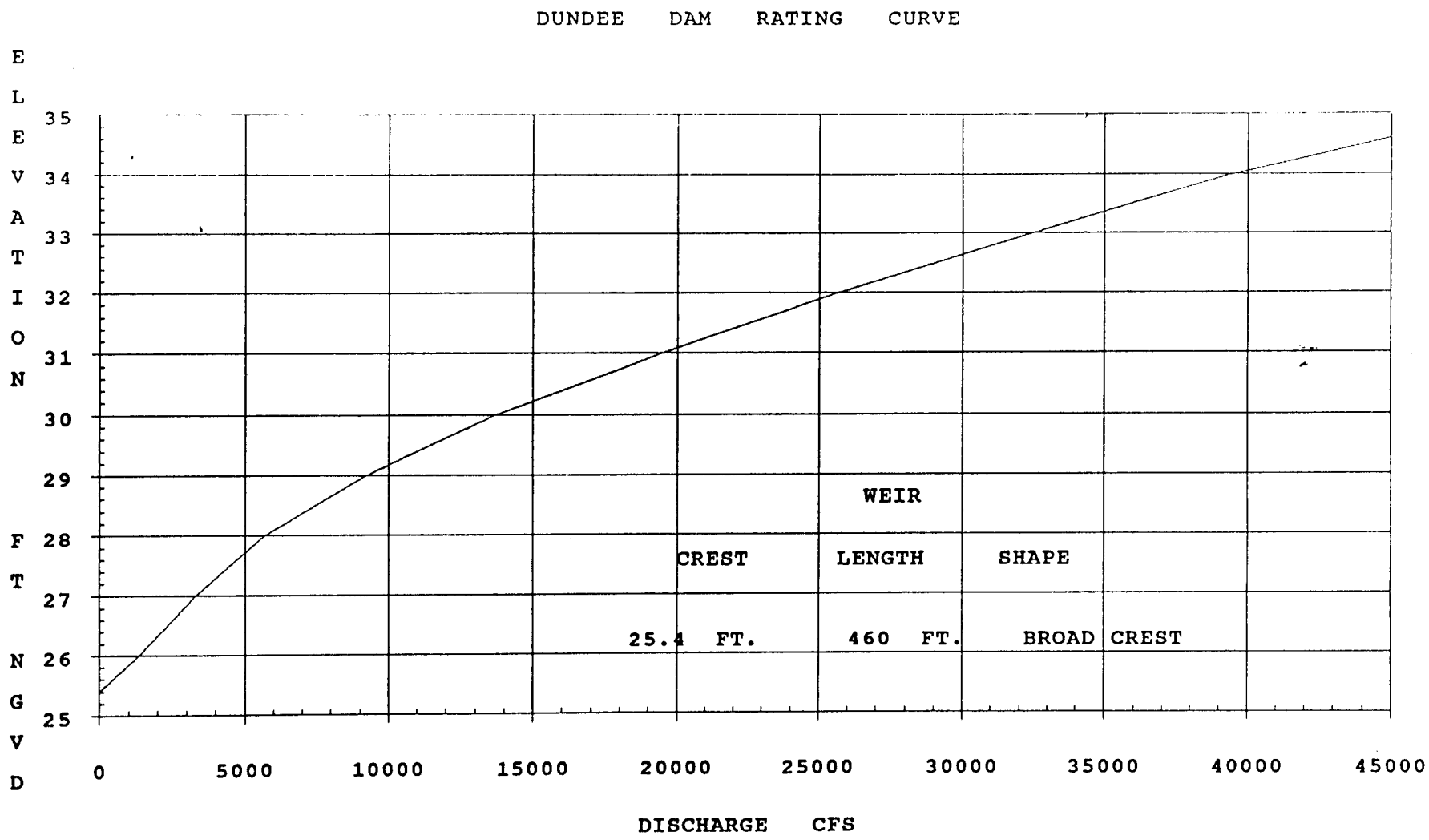

DUNDEE DAM RATING CURVE
ELEVATION FT NGVD
35
34
33
32
31
30
29
28
27
26
25
0
5000
10000
15000
20000
25000
30000
35000
40000
45000
DISCHARGE CFS
WEIR
CREST
LENGTH
SHAPE
25.4 FT.
460 FT.
BROAD CREST

Figure C-39h

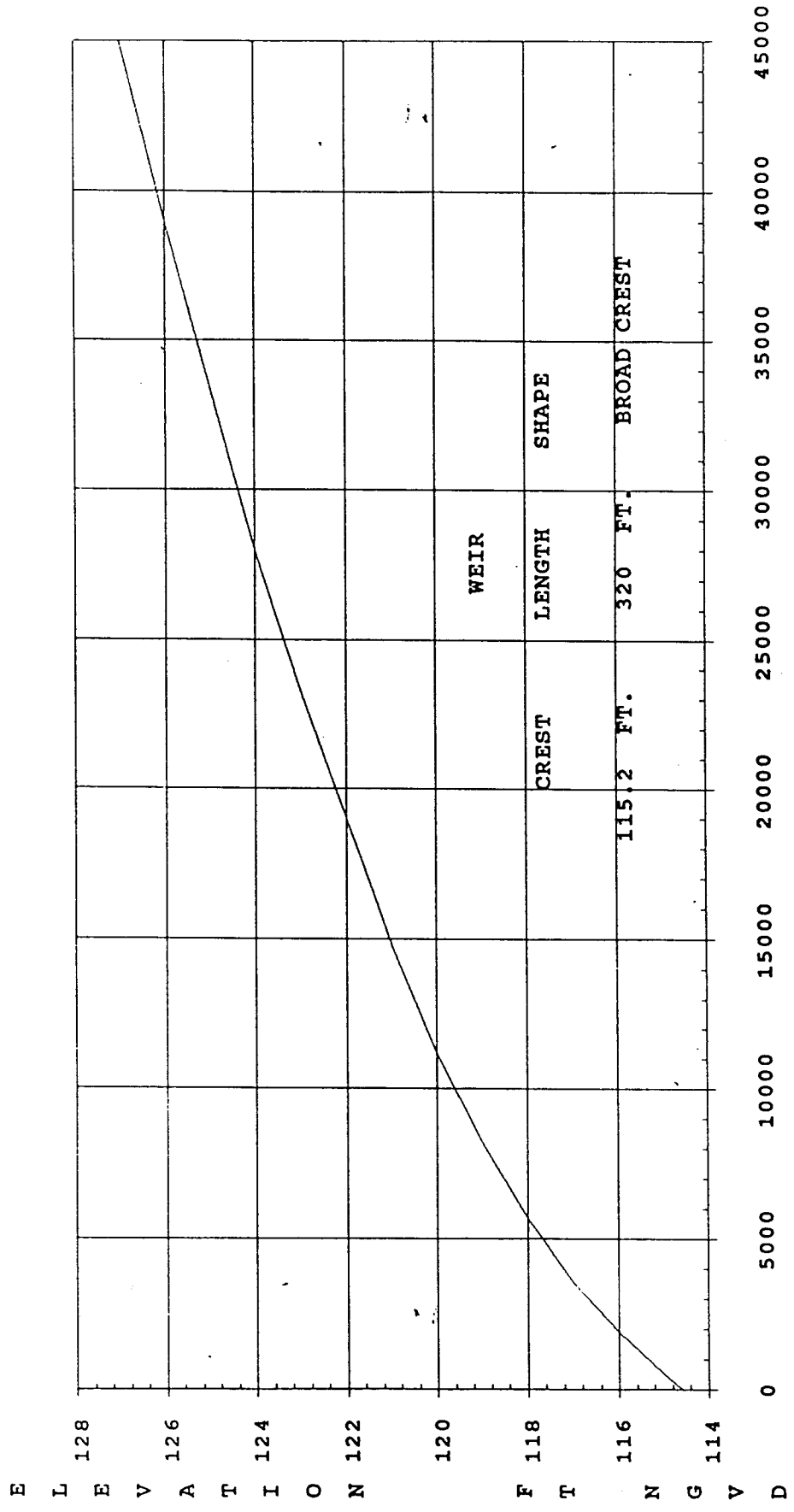

SUM DAM RATING CURVE
ELEVATION FT NGVD
128
126
124
122
120
118
116
114
0
5000
10000
15000
20000
25000
30000
35000
40000
45000
DISCHARGE CFS
WEIR
CREST LENGTH 115.2 FT.
SHAPE 320 FT. BROAD CREST

Figure C-39i

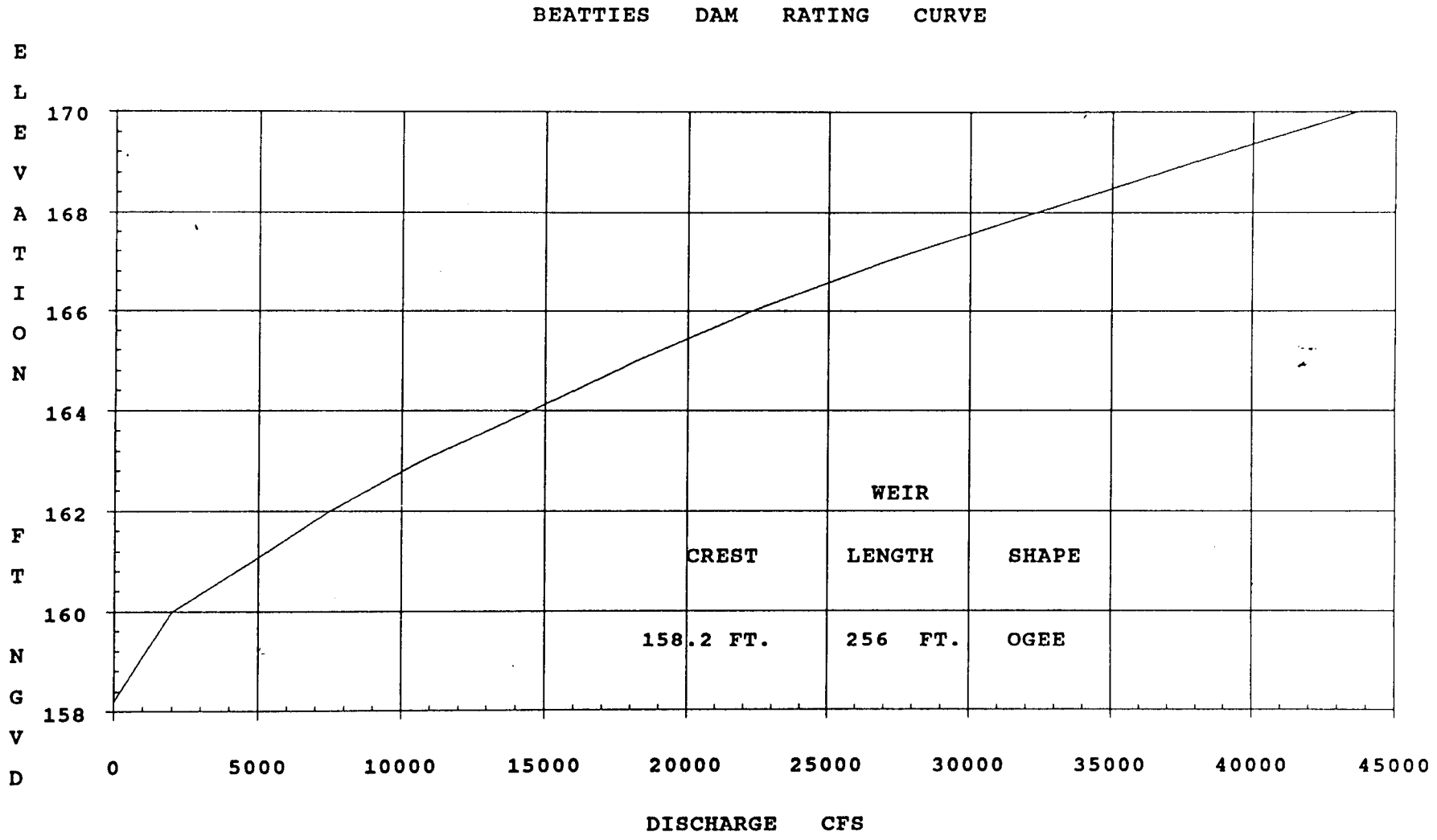

BEATTIES DAM RATING CURVE
ELEVATION FT NGVD
170
168
166
164
162
160
158
0
5000
10000
15000
20000
25000
30000
35000
40000
45000
DISCHARGE CFS
WEIR
CREST
LENGTH
SHAPE
158.2 FT.
256 FT.
OGEE

Figure C-39j

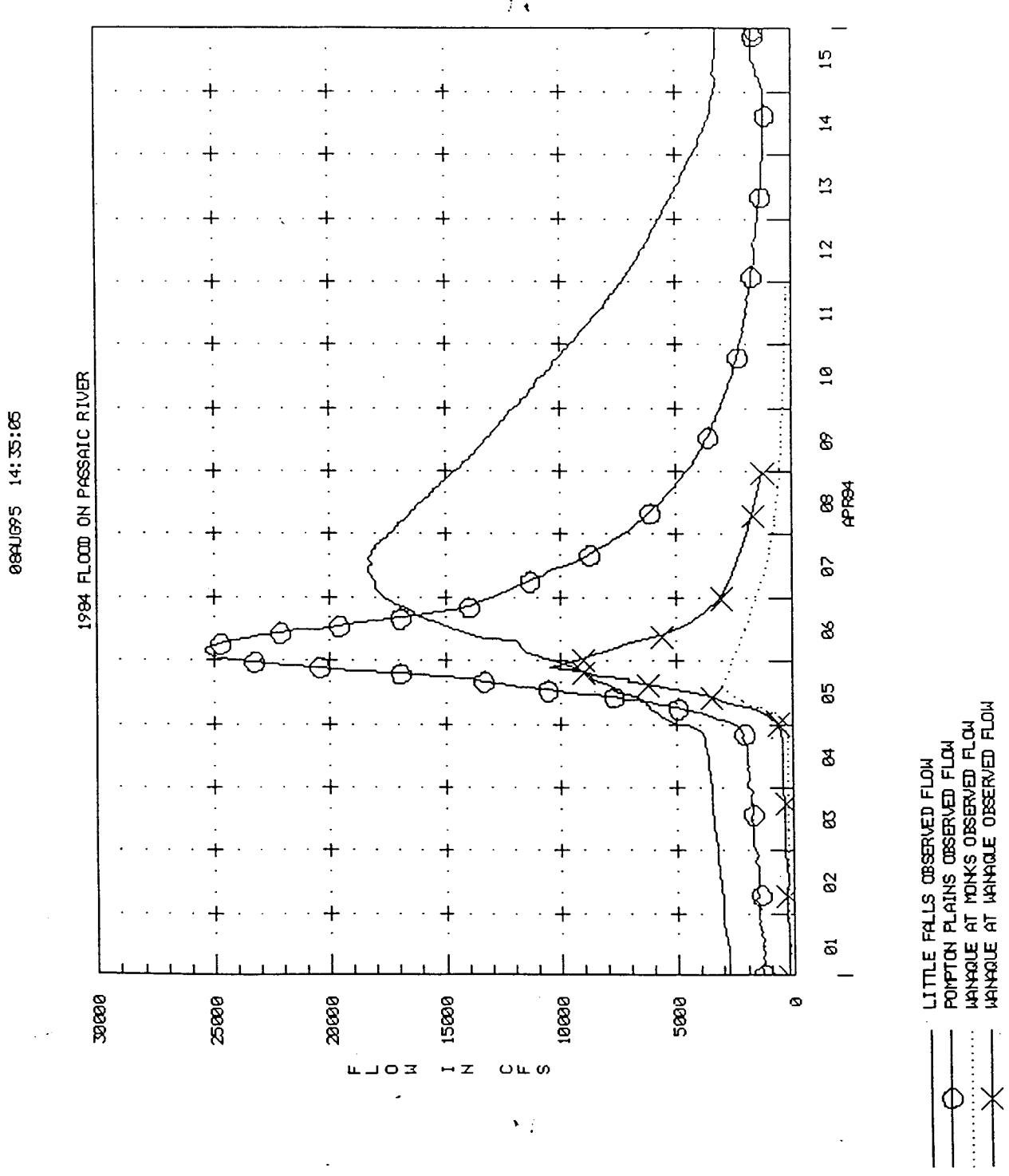

08AUG95 14:35:05
1984 FLOOD ON PASSAIC RIVER
FLOW IN CFS
30000
25000
20000
15000
10000
5000
0
01
02
03
04
05
06
07
08
09
10
11
12
13
14
15
APR84
LITTLE FALLS OBSERVED FLOW
POMPTON PLAINS OBSERVED FLOW
WANAQUE AT MONKS OBSERVED FLOW
WANAQUE AT WANAQUE OBSERVED FLOW

Figure C-40

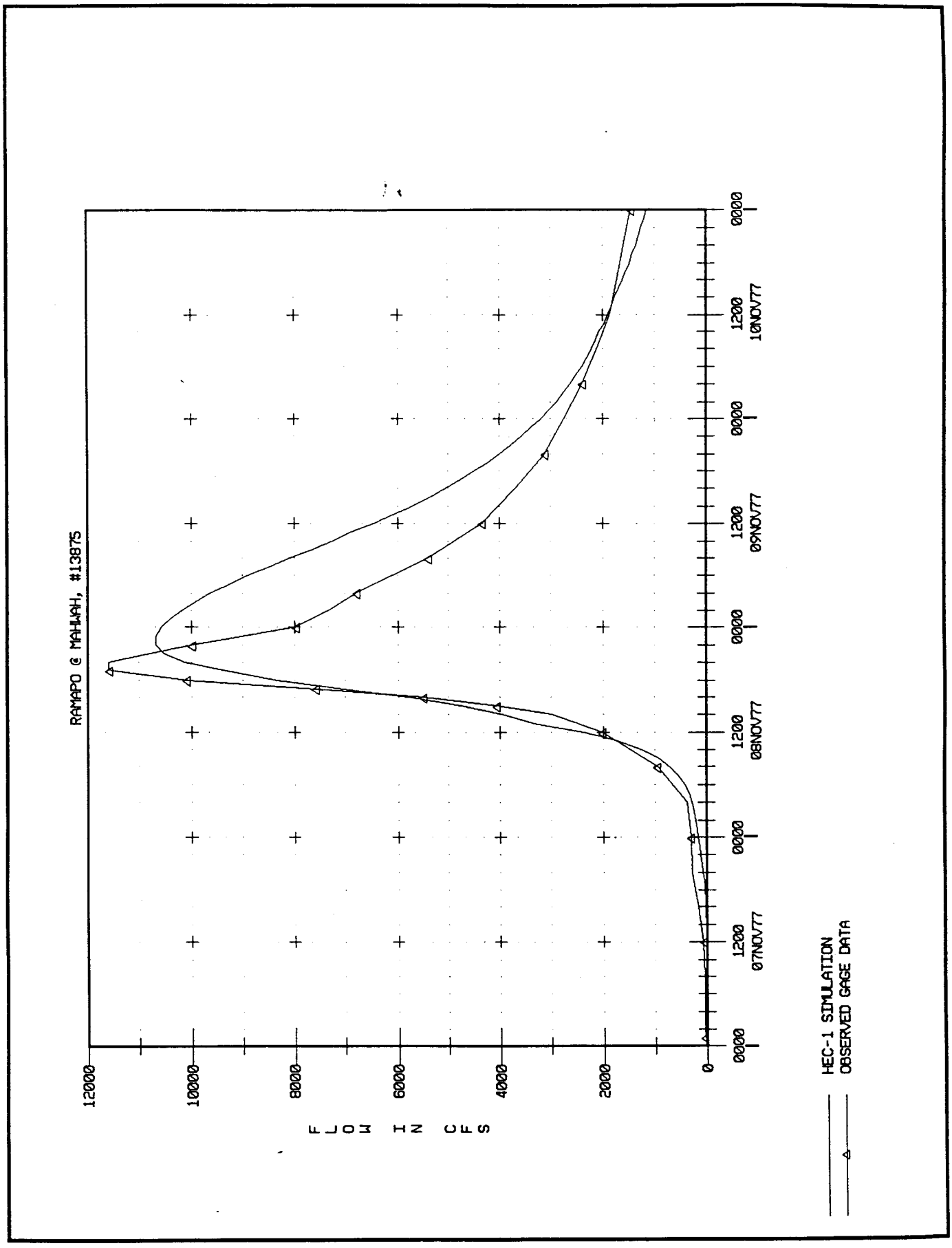

RAMAPO @ MAHWAH, #13875
FLOW IN CFS
12000
10000
8000
6000
4000
2000
0
0000
1200
07NOV77
0000
1200
08NOV77
0000
1200
09NOV77
0000
1200
10NOV77
0000
HEC-1 SIMULATION
OBSERVED GAGE DATA

Figure C-41a

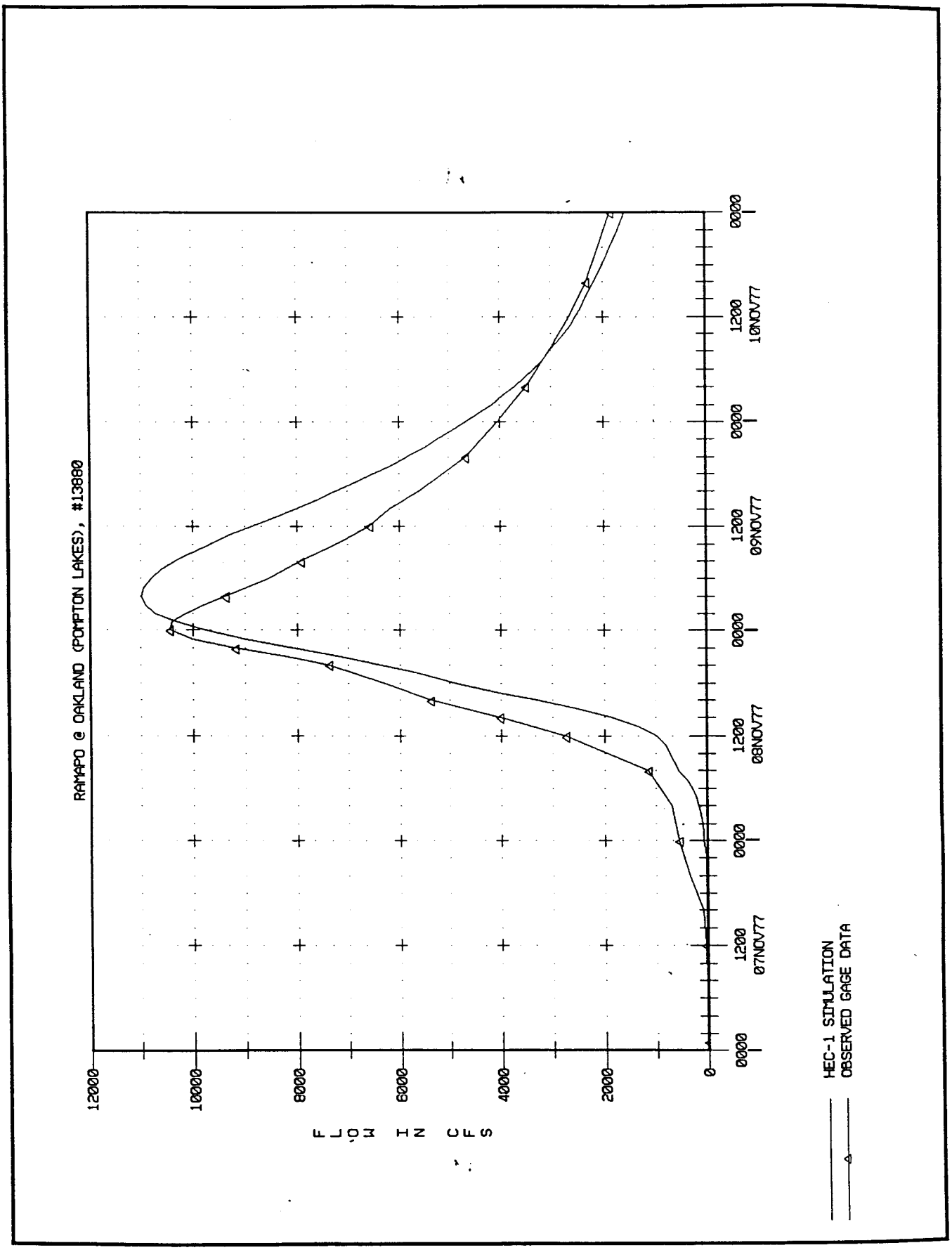

RAMAPO @ OAKLAND (POMPTON LAKES), #13880
FLOW IN CFS
12000
10000
8000
6000
4000
2000
0
0000
1200
07NOV77
0000
1200
08NOV77
0000
1200
09NOV77
0000
1200
10NOV77
0000
HEC-1 SIMULATION
OBSERVED GAGE DATA

Figure C-41b

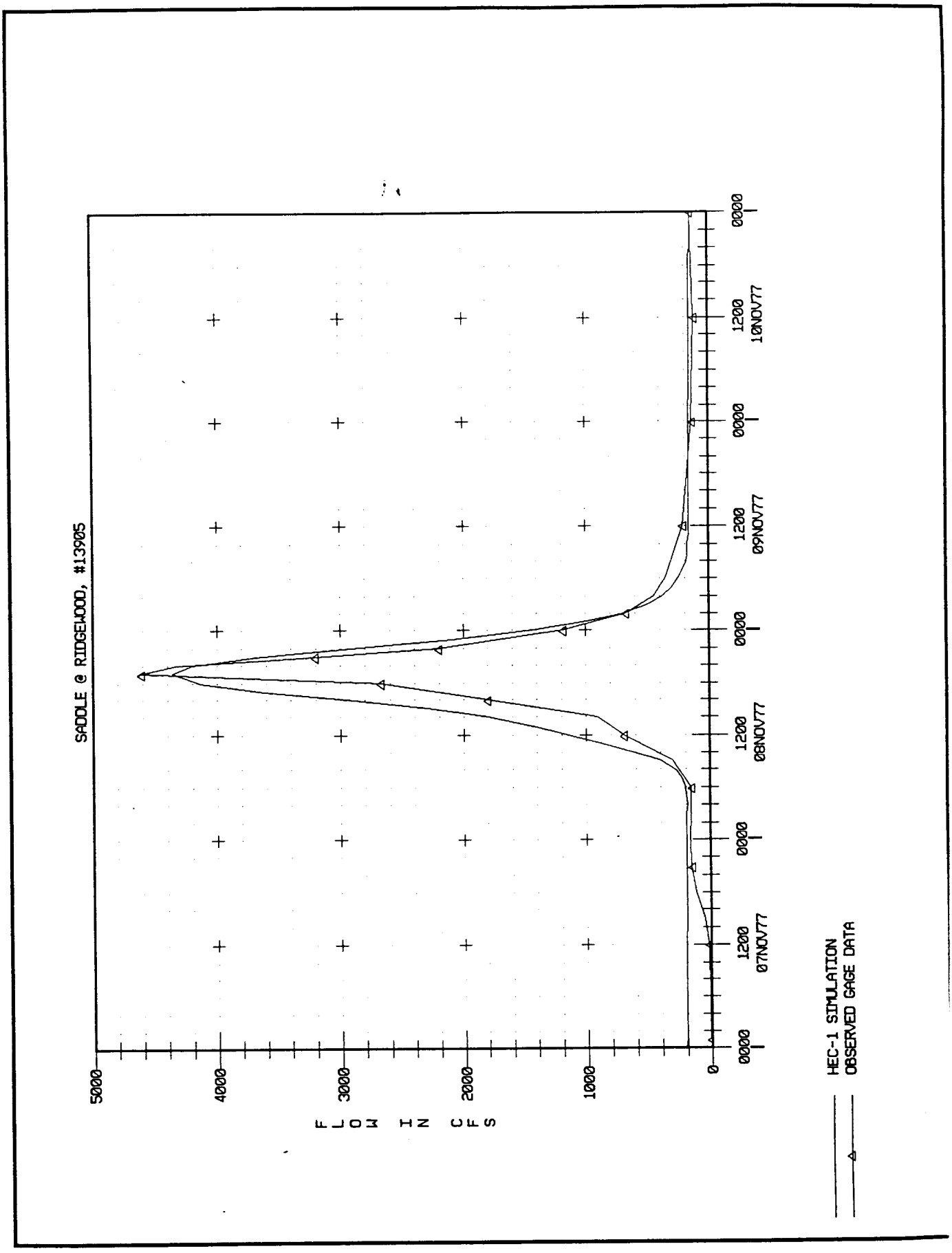

SADDLE @ RIDGEWOOD, #13905
FLOW IN CFS
5000
4000
3000
2000
1000
0
0000
1200
07NOV77
0000
1200
08NOV77
0000
1200
09NOV77
0000
1200
10NOV77
0000
HEC-1 SIMULATION
OBSERVED GAGE DATA

Figure C-41c

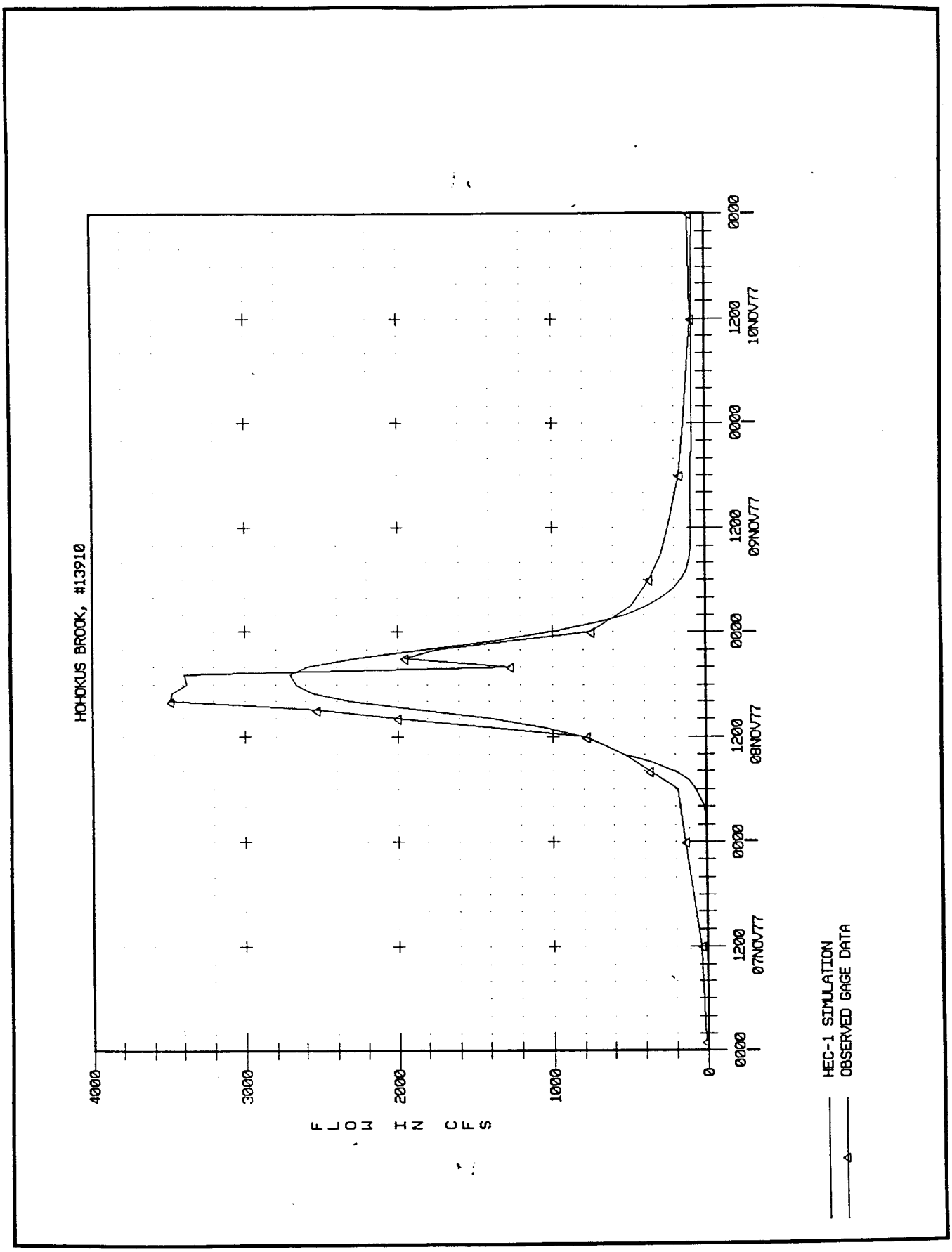

HOHOKUS BROOK, #13910
FLOW IN CFS
4000
3000
2000
1000
0
0000
1200
07NOV77
0000
1200
08NOV77
0000
1200
09NOV77
0000
1200
10NOV77
0000
HEC-1 SIMULATION
OBSERVED GAGE DATA

Figure C-41d

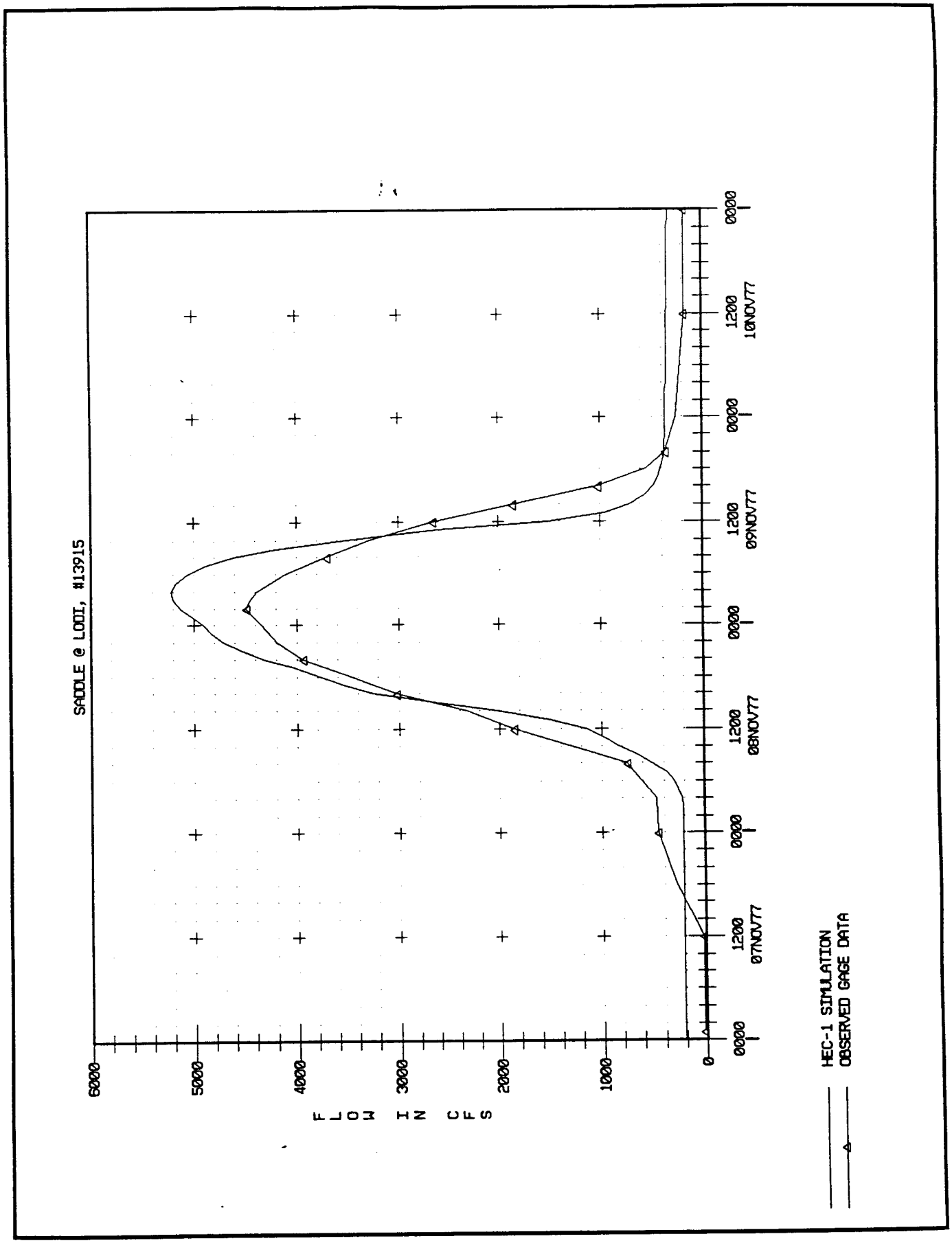

SADDLE @ LODI, #13915
FLOW IN CFS
6000
5000
4000
3000
2000
1000
0
0000
1200
07NOV77
0000
1200
08NOV77
0000
1200
09NOV77
0000
1200
10NOV77
0000
HEC-1 SIMULATION
OBSERVED GAGE DATA

Figure C-41e

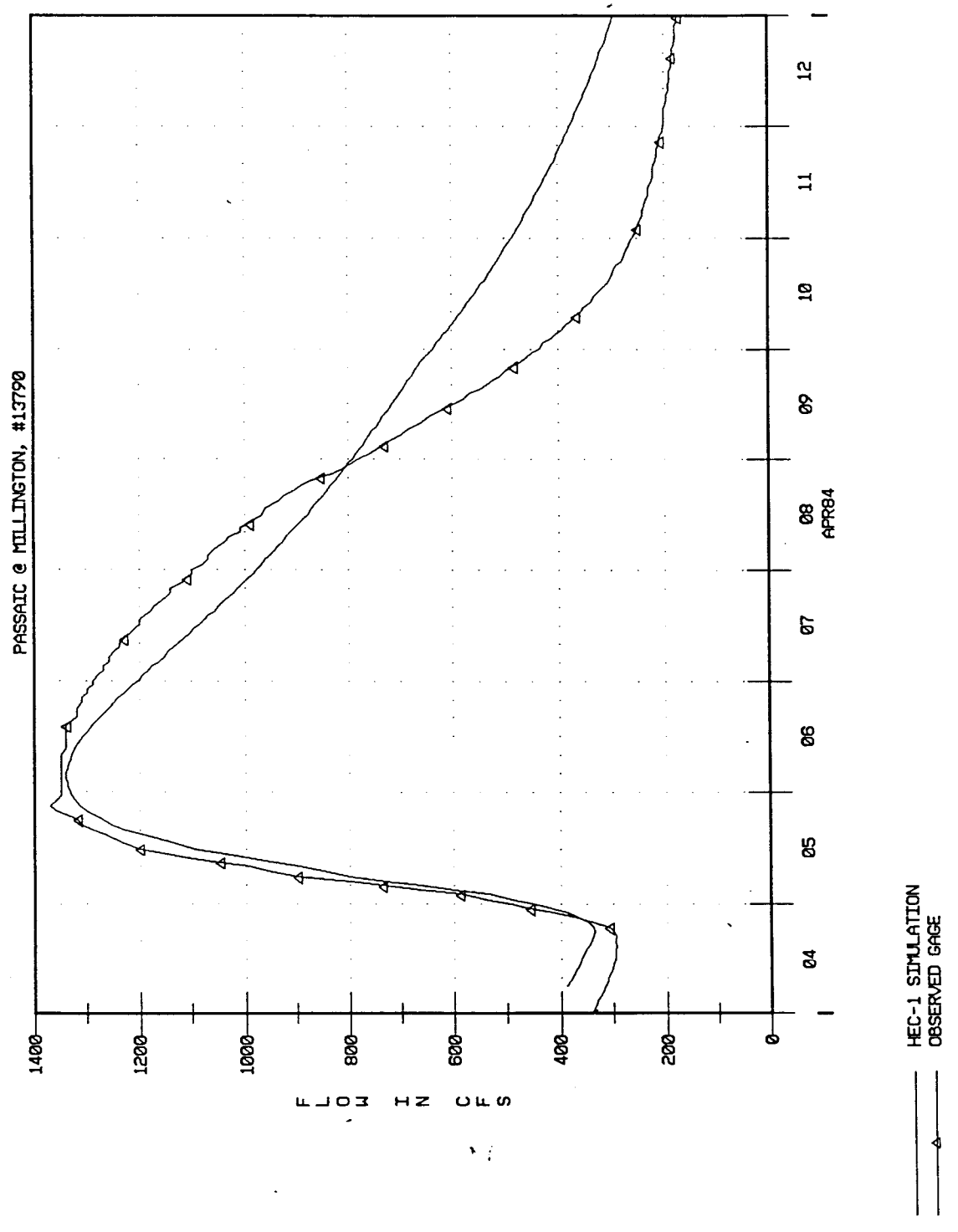

PASSAIC @ MILLINGTON, #13790
FLOW IN CFS
1400
1200
1000
800
600
400
200
0
04
05
06
07
08
09
10
11
12
APR84
HEC-1 SIMULATION
OBSERVED GAGE

Figure C-42a

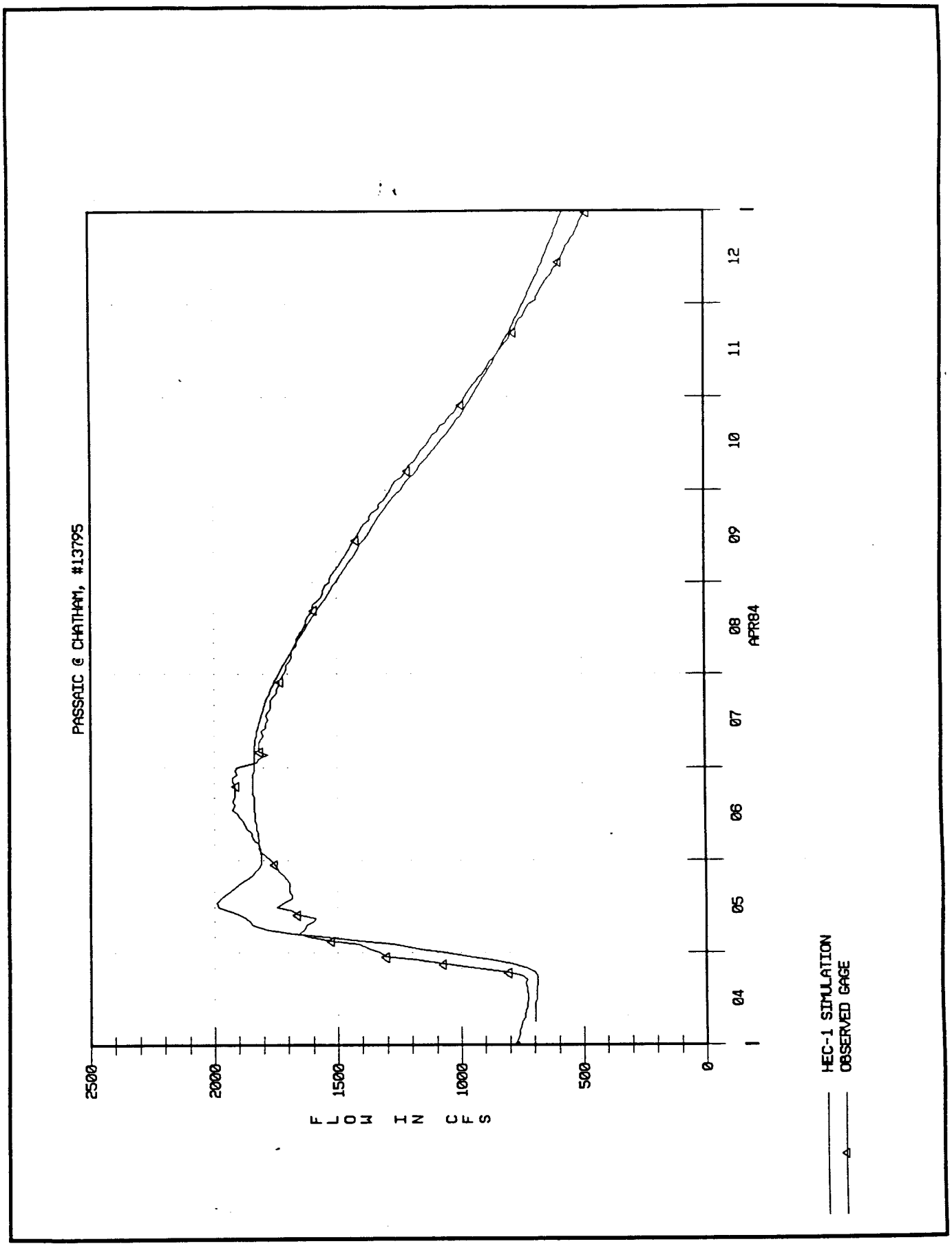

PASSAIC @ CHATHAM, #13795
FLOW IN CFS
2500
2000
1500
1000
500
0
04
05
06
07
08
09
10
11
12
APR84
HEC-1 SIMULATION
OBSERVED GAGE

Figure C-42b

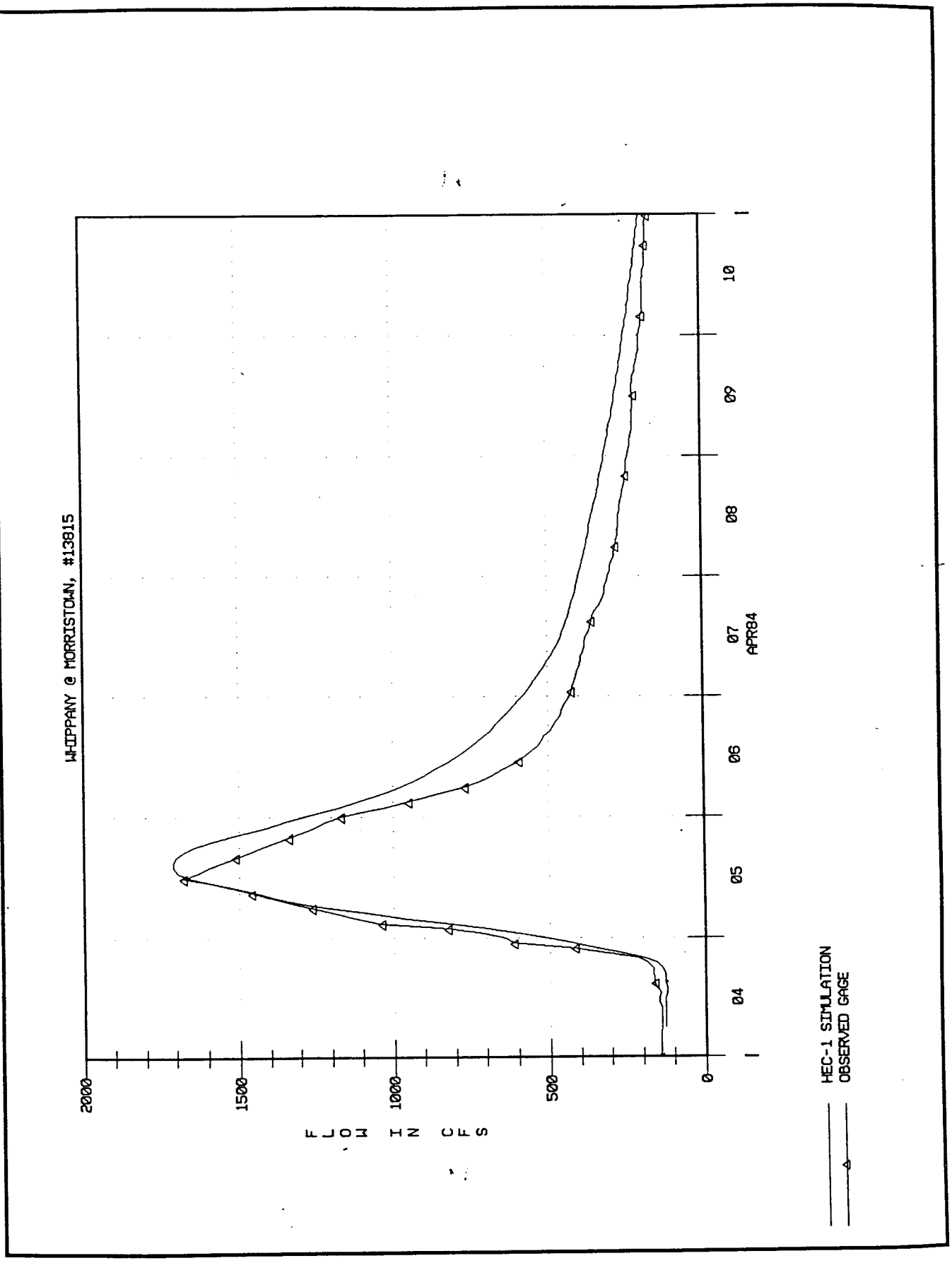

WHIPPANY @ MORRISTOWN, #13815
FLOW IN CFS
2000
1500
1000
500
0
04
05
06
07
APR84
08
09
10
HEC-1 SIMULATION
OBSERVED GAGE

Figure C-42c

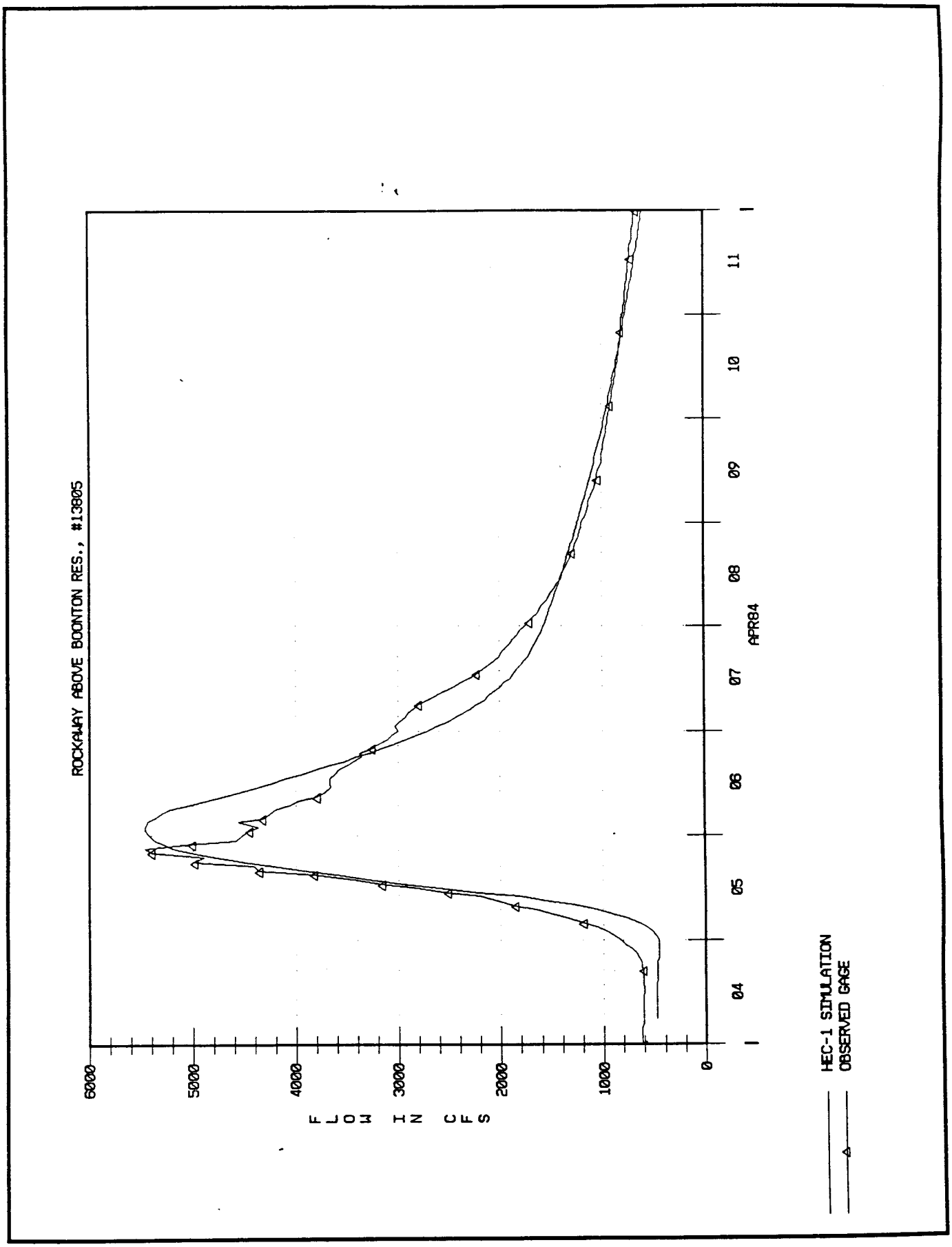

ROCKAWAY ABOVE BOONTON RES., #13005
FLOW IN CFS
6000
5000
4000
3000
2000
1000
0
04
05
06
07
08
09
10
11
APR84
HEC-1 SIMULATION
OBSERVED GAGE

Figure C-42d

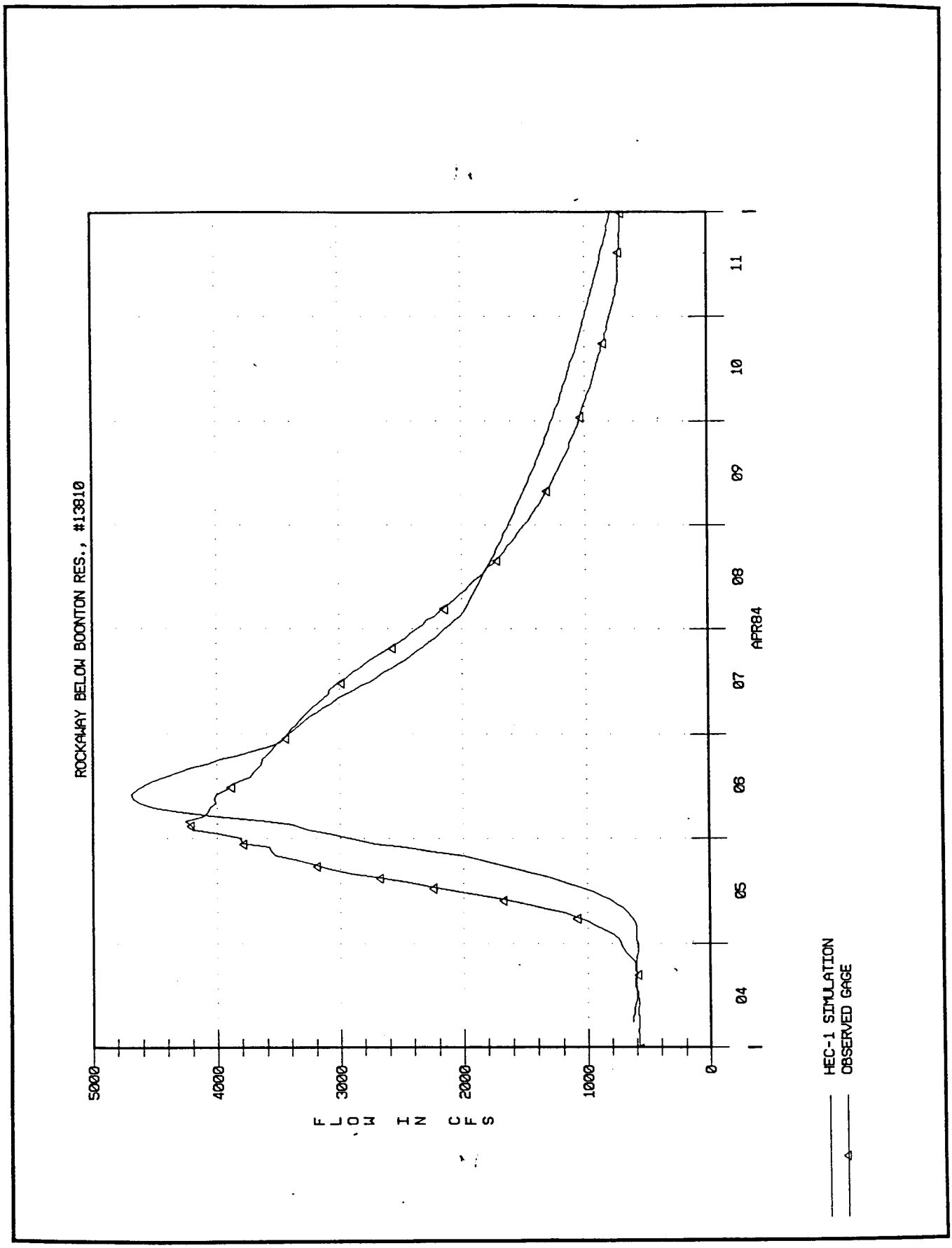

ROCKAWAY BELOW BOONTON RES., #13810
FLOW IN CFS
5000
4000
3000
2000
1000
0
04
05
06
07
08
09
10
11
APR84
HEC-1 SIMULATION
OBSERVED GAGE

Figure C-42e

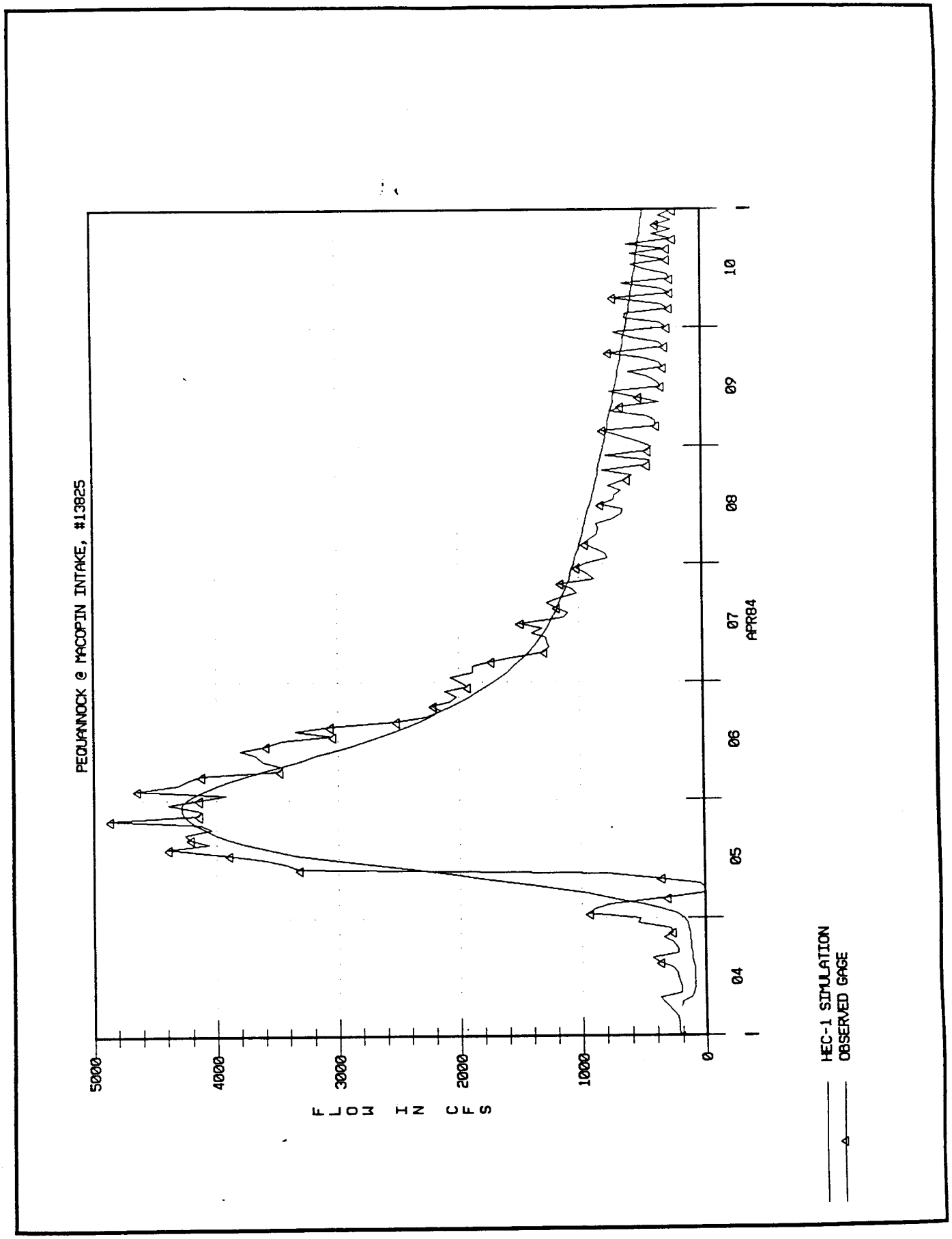

PEQUANNOCK @ MACOPIN INTAKE, #13825
FLOW IN CFS
5000
4000
3000
2000
1000
0
04
05
06
07
APR84
08
09
10
HEC-1 SIMULATION
OBSERVED GAGE

Figure C-42f

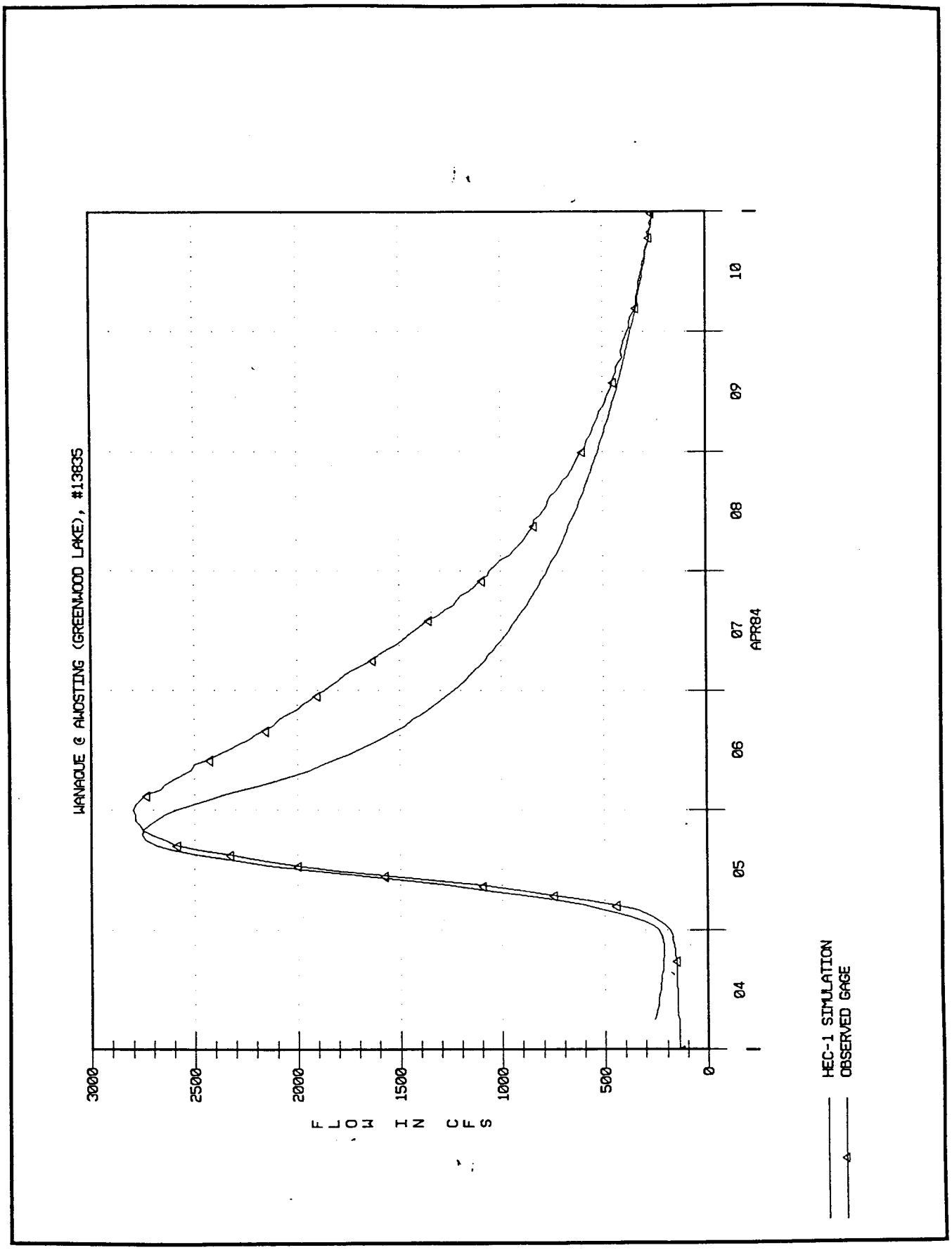

WANAQUE @ AWOSTING (GREENWOOD LAKE), #13835
FLOW IN CFS
3000
2500
2000
1500
1000
500
0
04
05
06
07
APR84
08
09
10
HEC-1 SIMULATION
OBSERVED GAGE

Figure C-42g

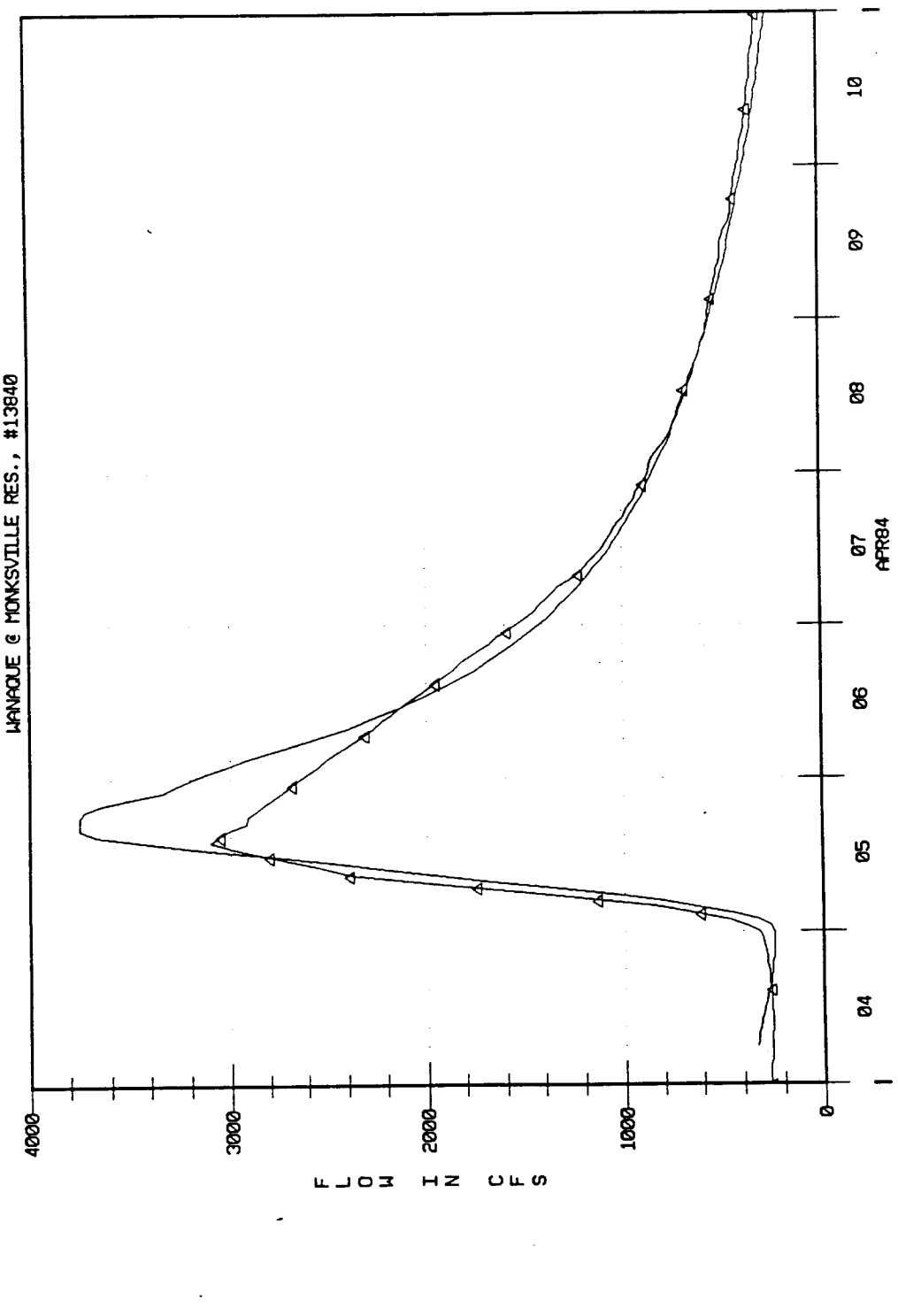

WANAQUE @ MONKSVILLE RES., #13840
FLOW IN CFS
4000
3000
2000
1000
0
04
05
06
07
APR84
08
09
10
HEC-1 SIMULATION
OBSERVED GAGE

Figure C-42h

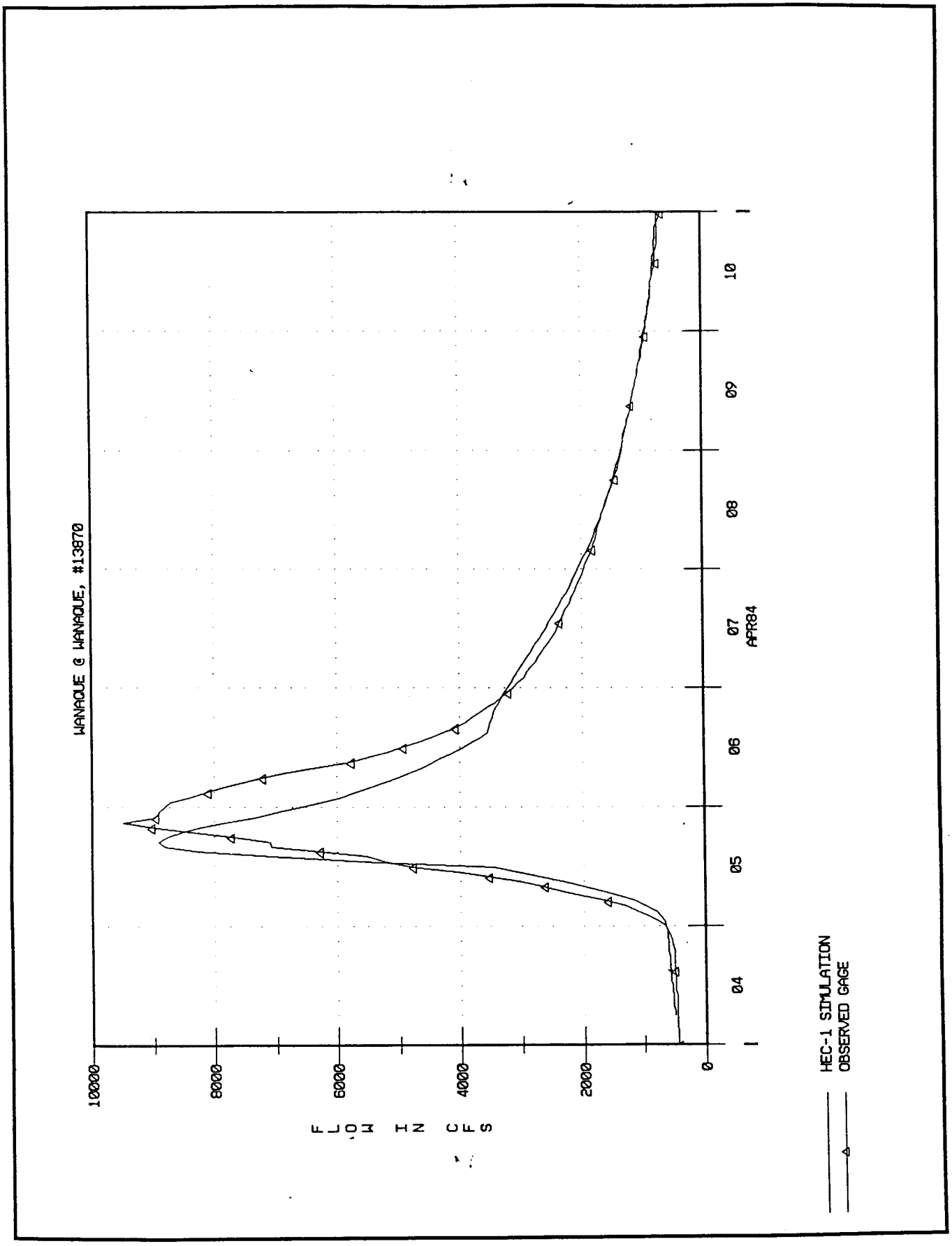

WANAQUE @ WANAQUE, #13870
FLOW IN CFS
10000
8000
6000
4000
2000
0
04
05
06
07
APR84
08
09
10
HEC-1 SIMULATION
OBSERVED GAGE

Figure C-42i

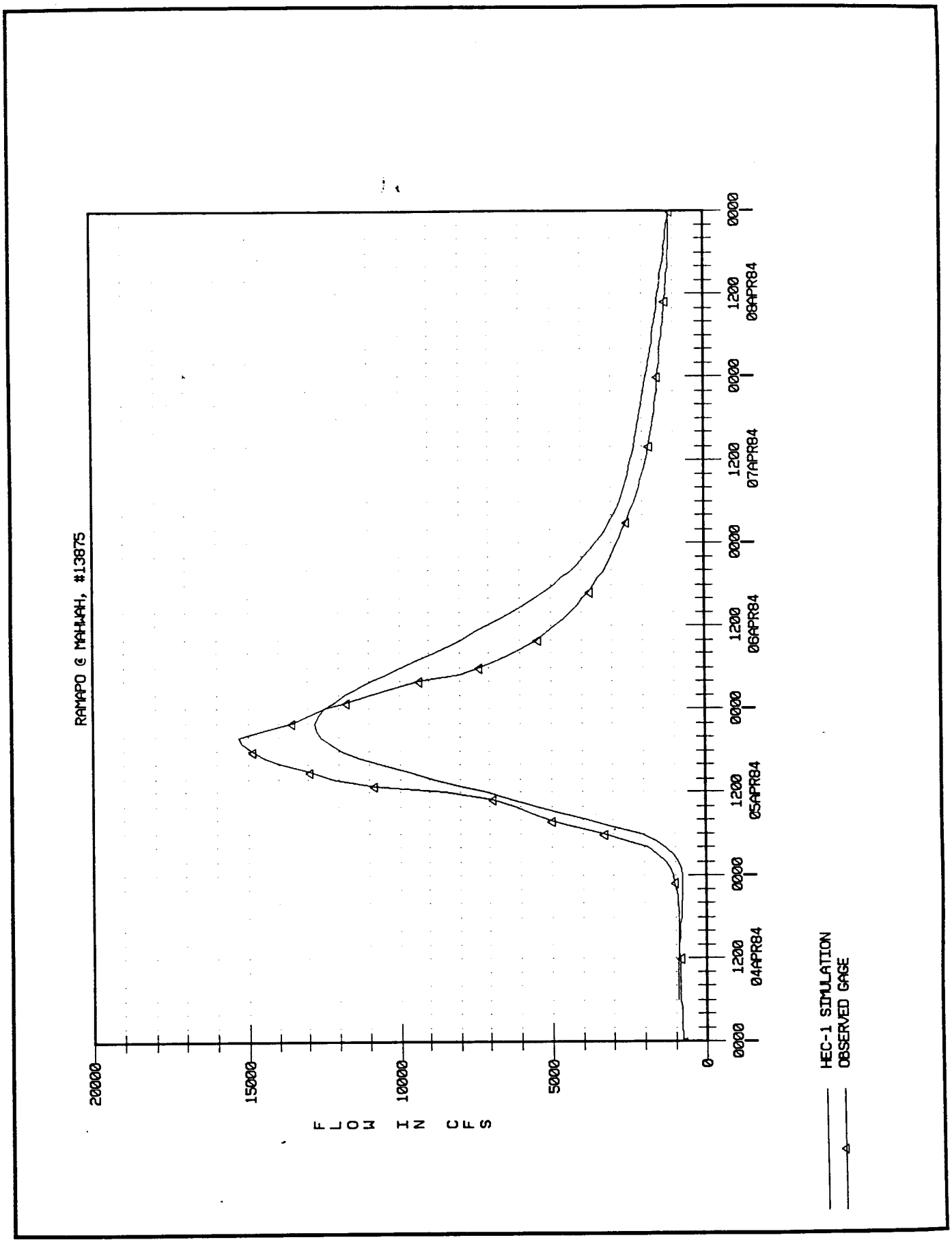

RAMAPO @ MAHWAH, #13875
FLOW IN CFS
20000
15000
10000
5000
0
0000
1200
04APR84
0000
1200
05APR84
0000
1200
06APR84
0000
1200
07APR84
0000
1200
08APR84
HEC-1 SIMULATION
OBSERVED GAGE

Figure C-42j

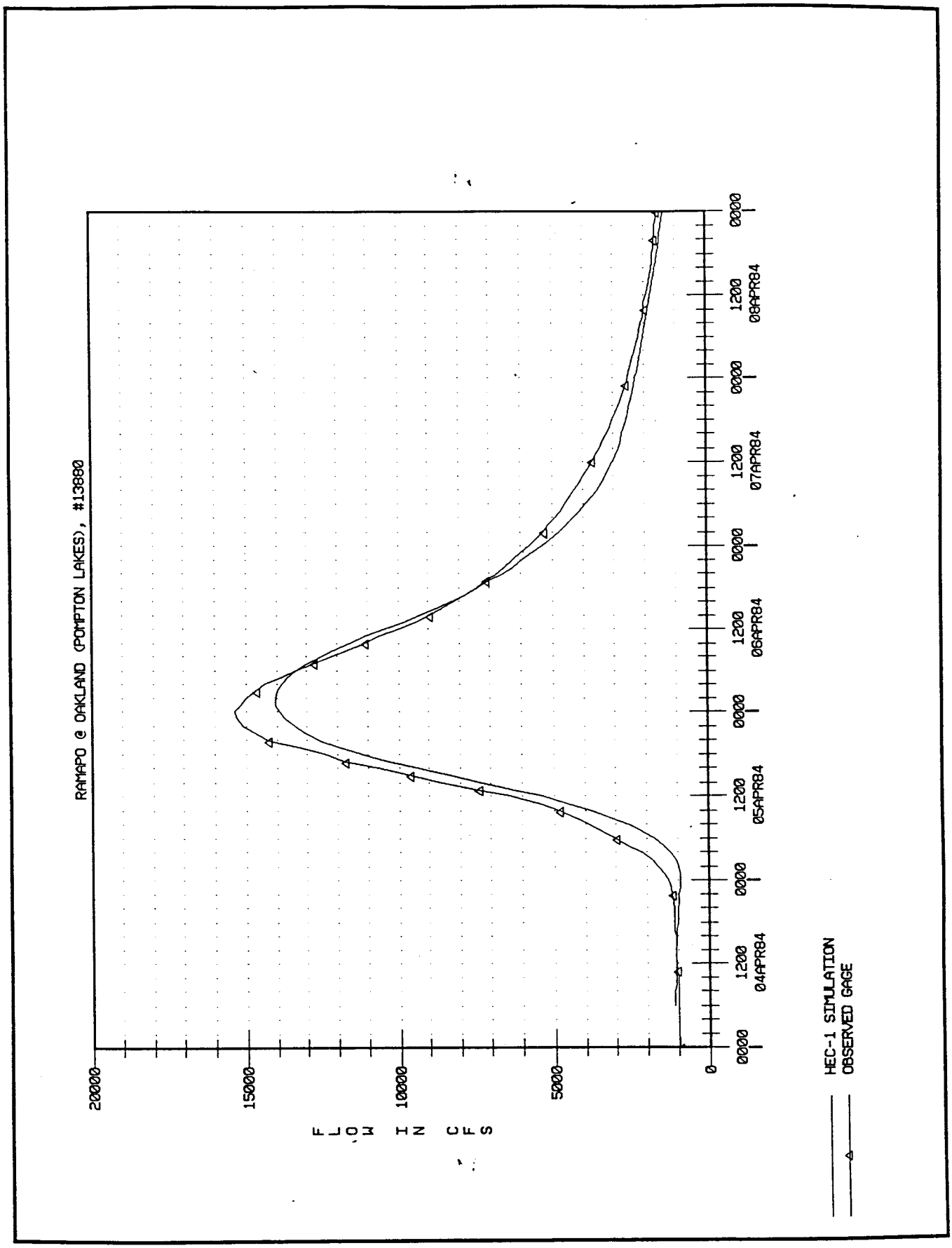

RAMAPO @ OAKLAND (POMPTON LAKES), #13880
FLOW IN CFS
20000
15000
10000
5000
0
0000
1200
04APR84
0000
1200
05APR84
0000
1200
06APR84
0000
1200
07APR84
0000
1200
08APR84
HEC-1 SIMULATION
OBSERVED GAGE

Figure C-42k

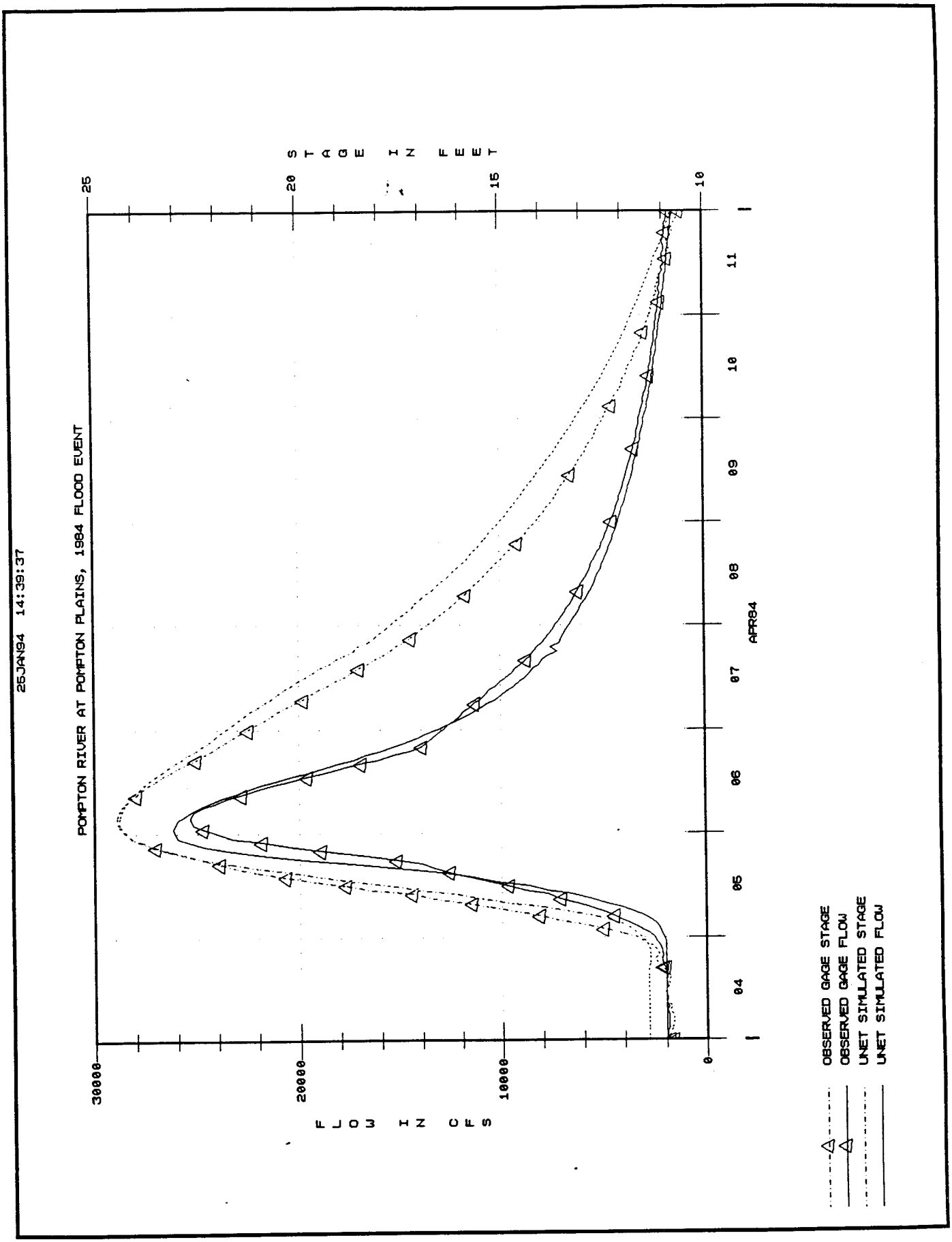

25JAN94 14:39:37
POMPTON RIVER AT POMPTON PLAINS, 1984 FLOOD EVENT
STAGE IN FEET
FLOW IN CFS
APR84
OBSERVED GAGE STAGE
OBSERVED GAGE FLOW
UNET SIMULATED STAGE
UNET SIMULATED FLOW

Figure C-421

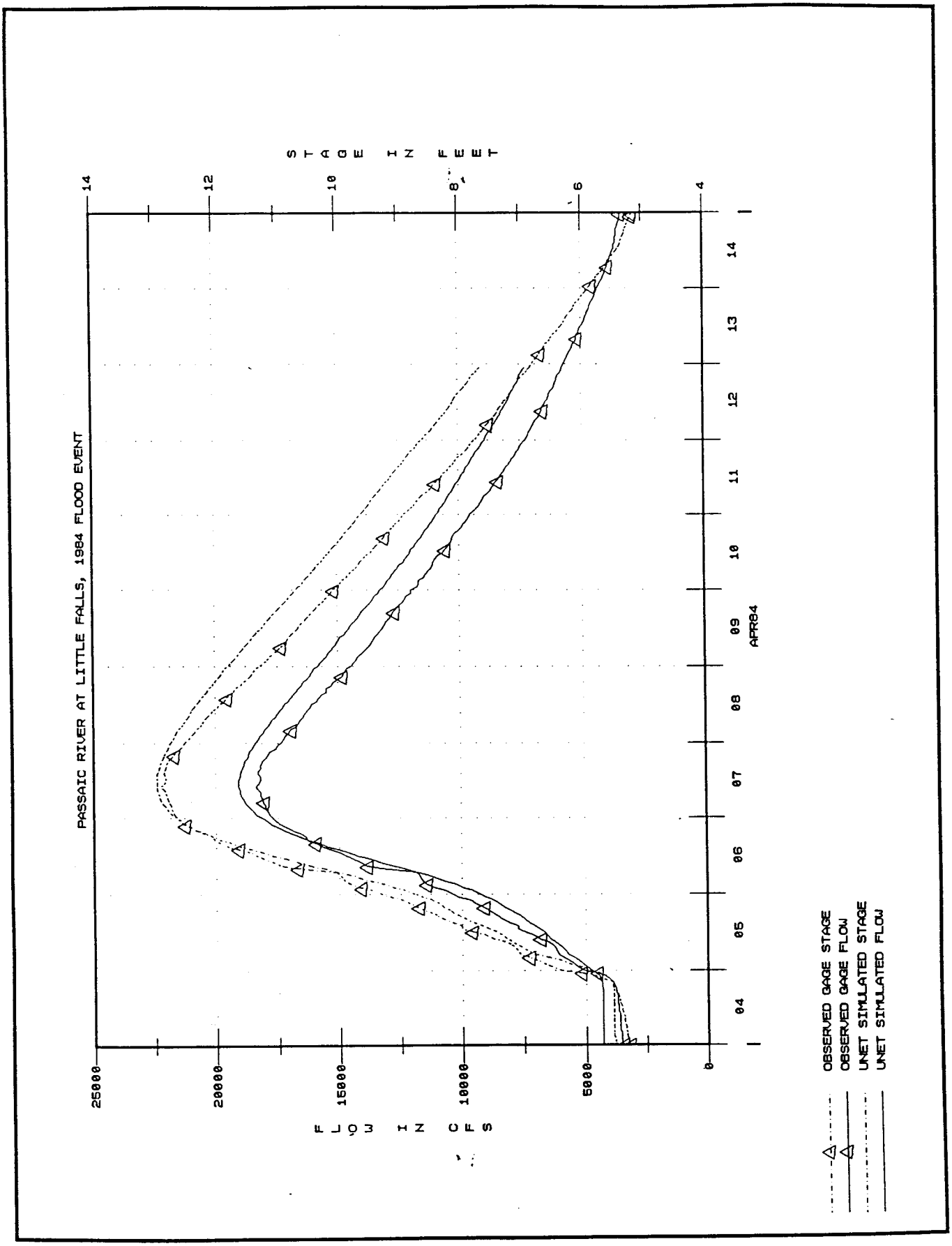

PASSAIC RIVER AT LITTLE FALLS, 1984 FLOOD EVENT
STAGE IN FEET
FLOW IN CFS
APR84
OBSERVED GAGE STAGE
OBSERVED GAGE FLOW
UNET SIMULATED STAGE
UNET SIMULATED FLOW

Figure C-42m

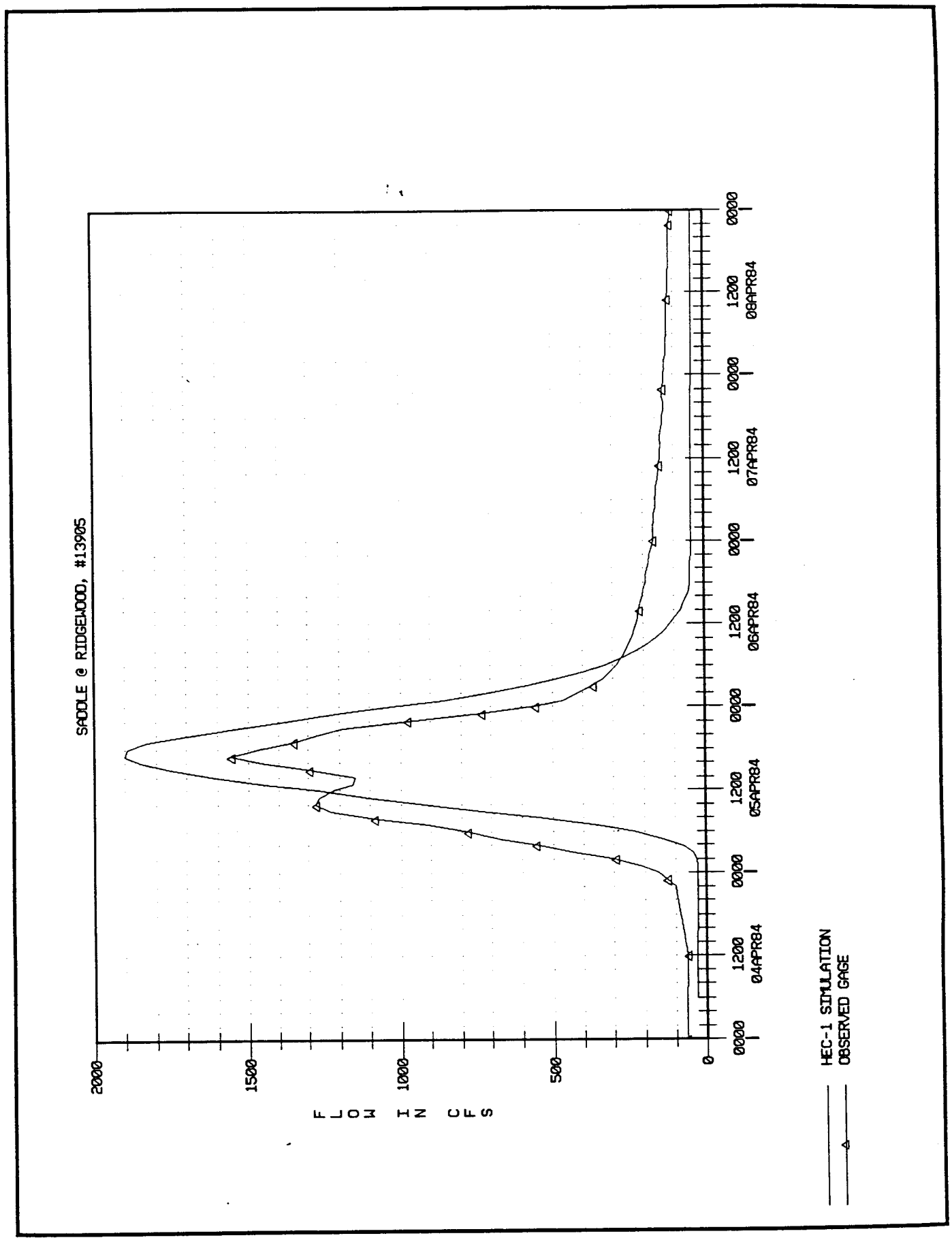

SADDLE @ RIDGEWOOD, #13905
FLOW IN CFS
2000
1500
1000
500
0
0000
1200
04APR84
0000
1200
05APR84
0000
1200
06APR84
0000
1200
07APR84
0000
1200
08APR84
0000
HEC-1 SIMULATION
OBSERVED GAGE

Figure C-42n

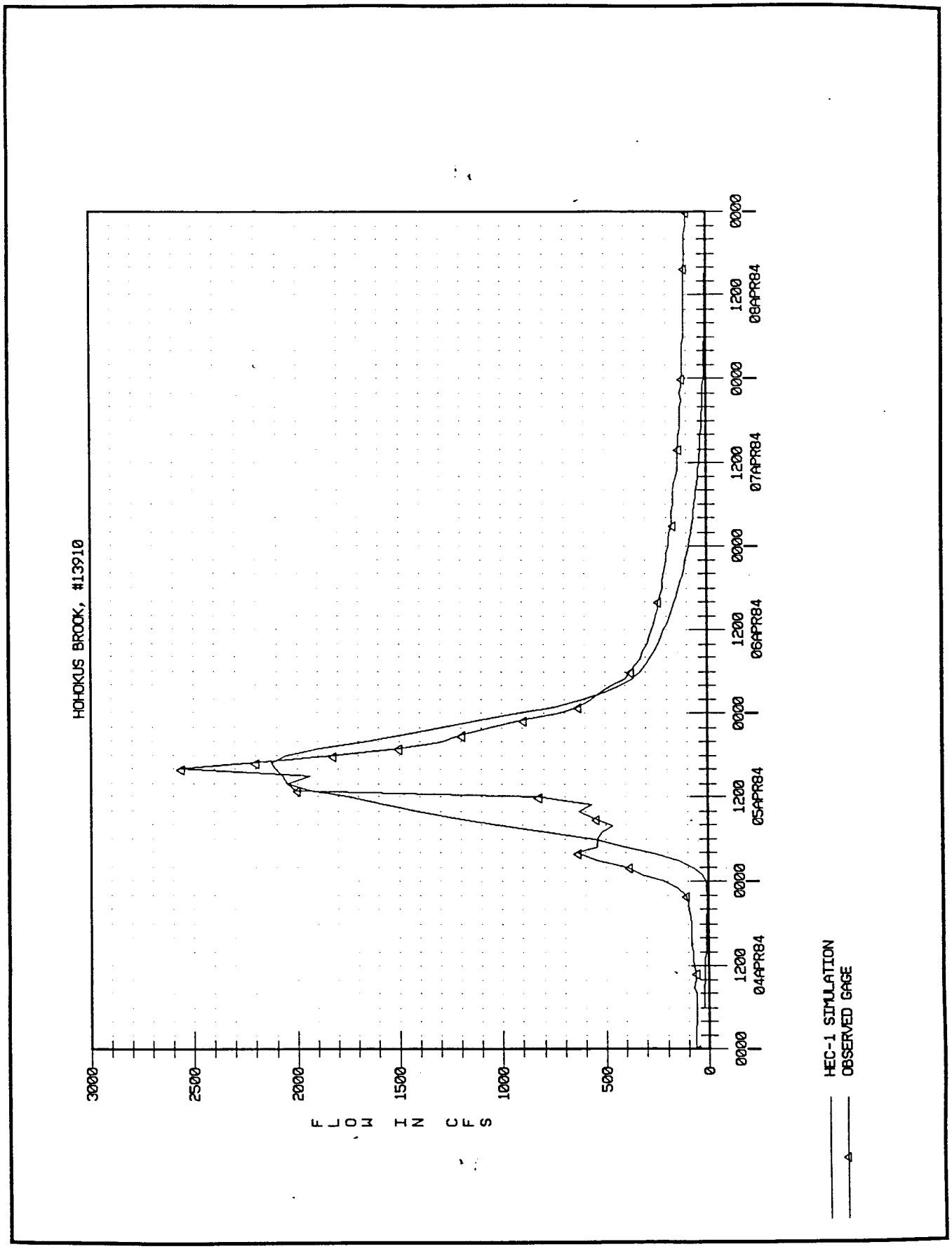

HOHOKUS BROOK, #13910
FLOW IN CFS
3000
2500
2000
1500
1000
500
0
0000
1200
04APR84
05APR84
06APR84
07APR84
08APR84
HEC-1 SIMULATION
OBSERVED GAGE

Figure C-420

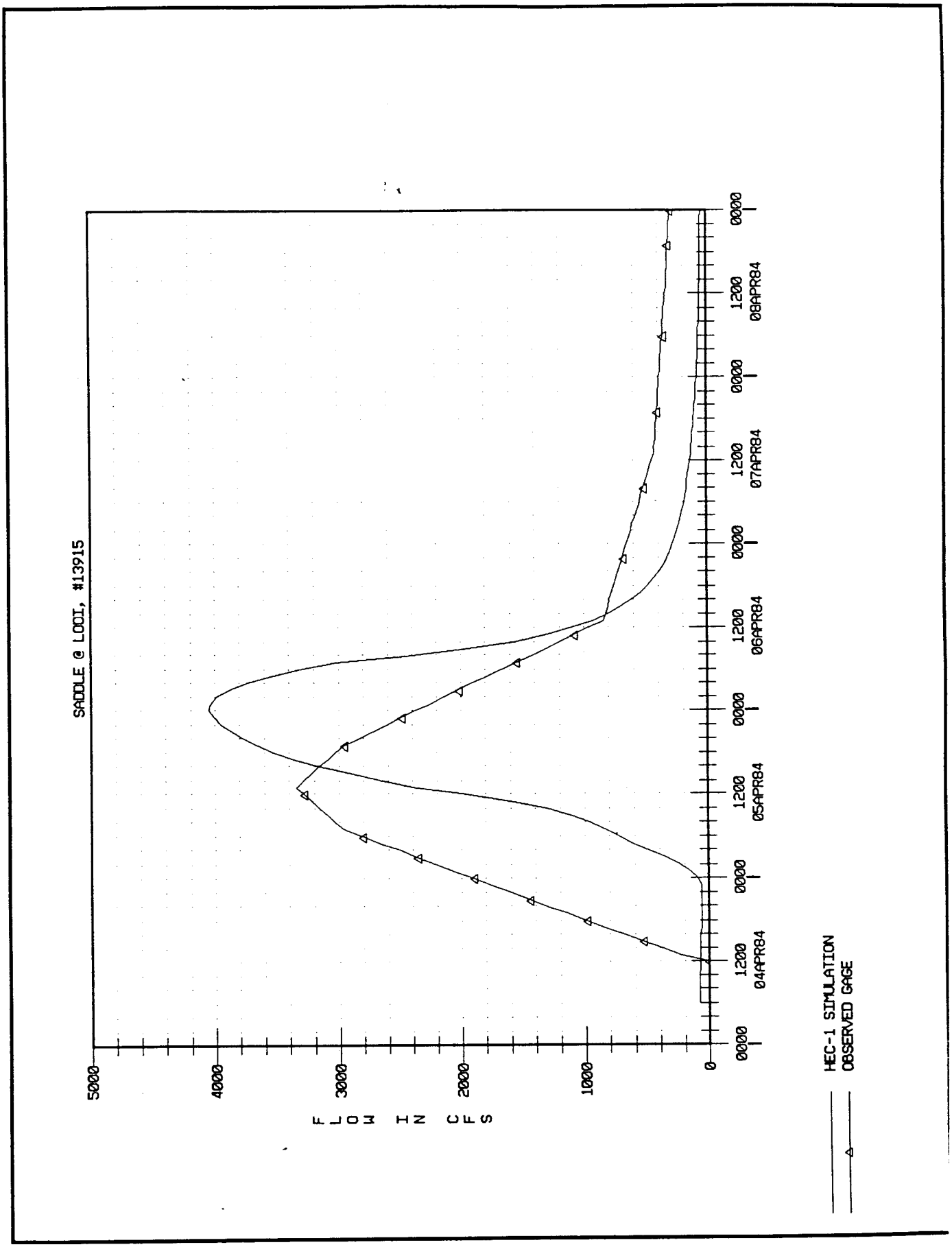
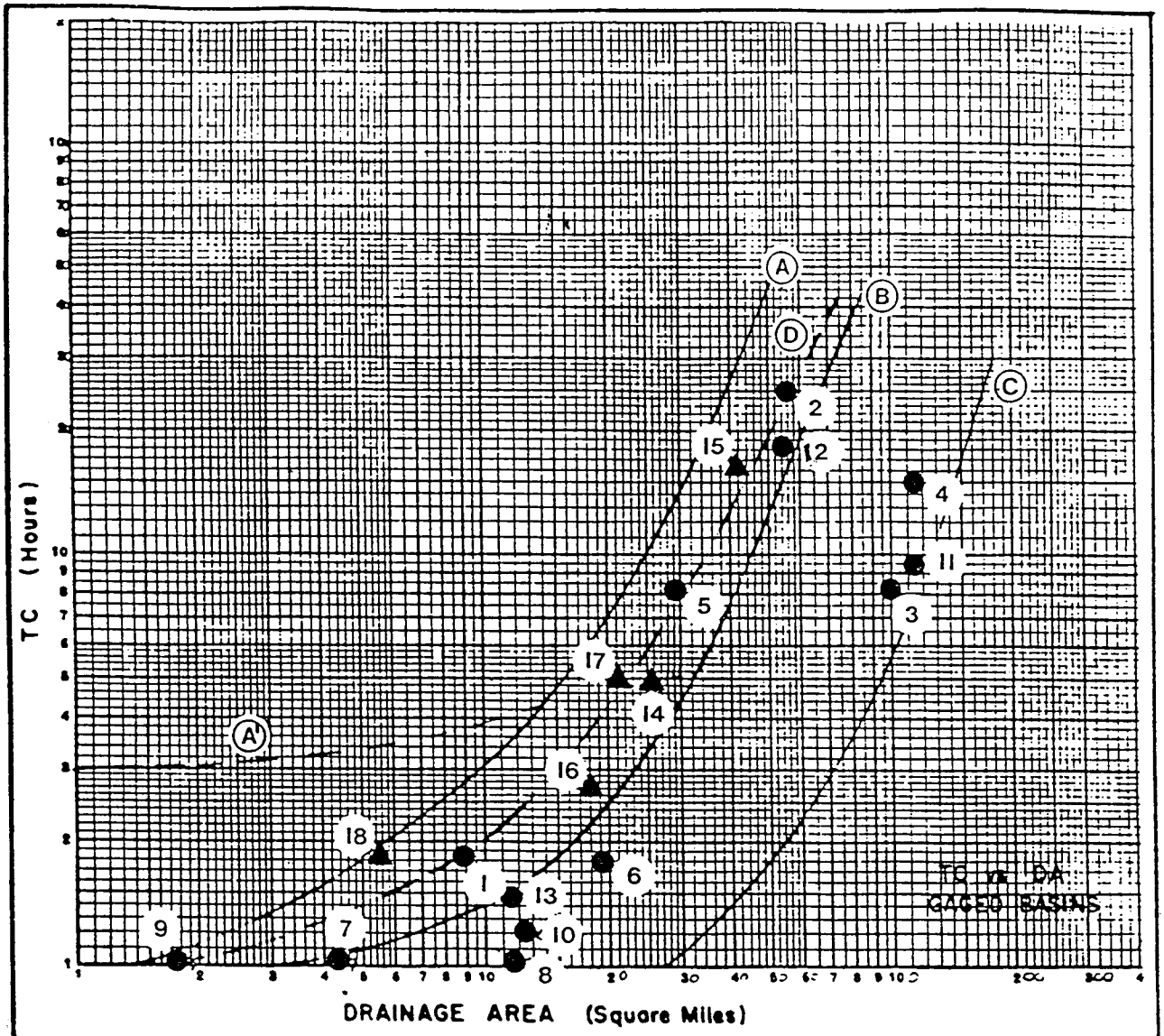

SADDLE @ LOOI, #13915
FLOW IN CFS
5000
4000
3000
2000
1000
0
0000
1200
04APR84
0000
1200
05APR84
0000
1200
06APR84
0000
1200
07APR84
0000
1200
08APR84
0000
HEC-1 SIMULATION
OBSERVED GAGE

Figure C-42p



Plot N°	U.S.G.S. Gages	
1	137869	Passaic R, Bernardsville
2	137900	Passaic R, Millington
3	137050	Passaic R, Chatham
4	138050	Rockaway R, above Boonton
5	138150	Whippany R, Morristown
6	138450	Ringwood Br, Wanaque
7	138500	Cupsaw Br, Wanaque
8	138600	West Br, Wanaque
9	138650	Blue Mine Br, Wanaque
10	138745	Mahwah R, Suffern
11	138750	Ramapo R, Mahwah
12	139150	Saddle R, Lodi
13	139221	Third R, at Passaic R.
14	139450	Rahway R., Springfield
15	139500	Rahway R., Rahway
16	139350	Elizabeth R., Elizabeth
17	139600	Robinson's Br., Rahway
18	140354	Stony Br., Watchung

LEGEND

A = UPPER LIMIT OF ENVELOPE CURVE.

A' = MAXIMUM ENVELOPE OF FINAL TC'S IN HIGHLAND AREAS.

B = BEST FIT BY EYE CURVE.

C = LOWER LIMIT OF ENVELOPE CURVE.

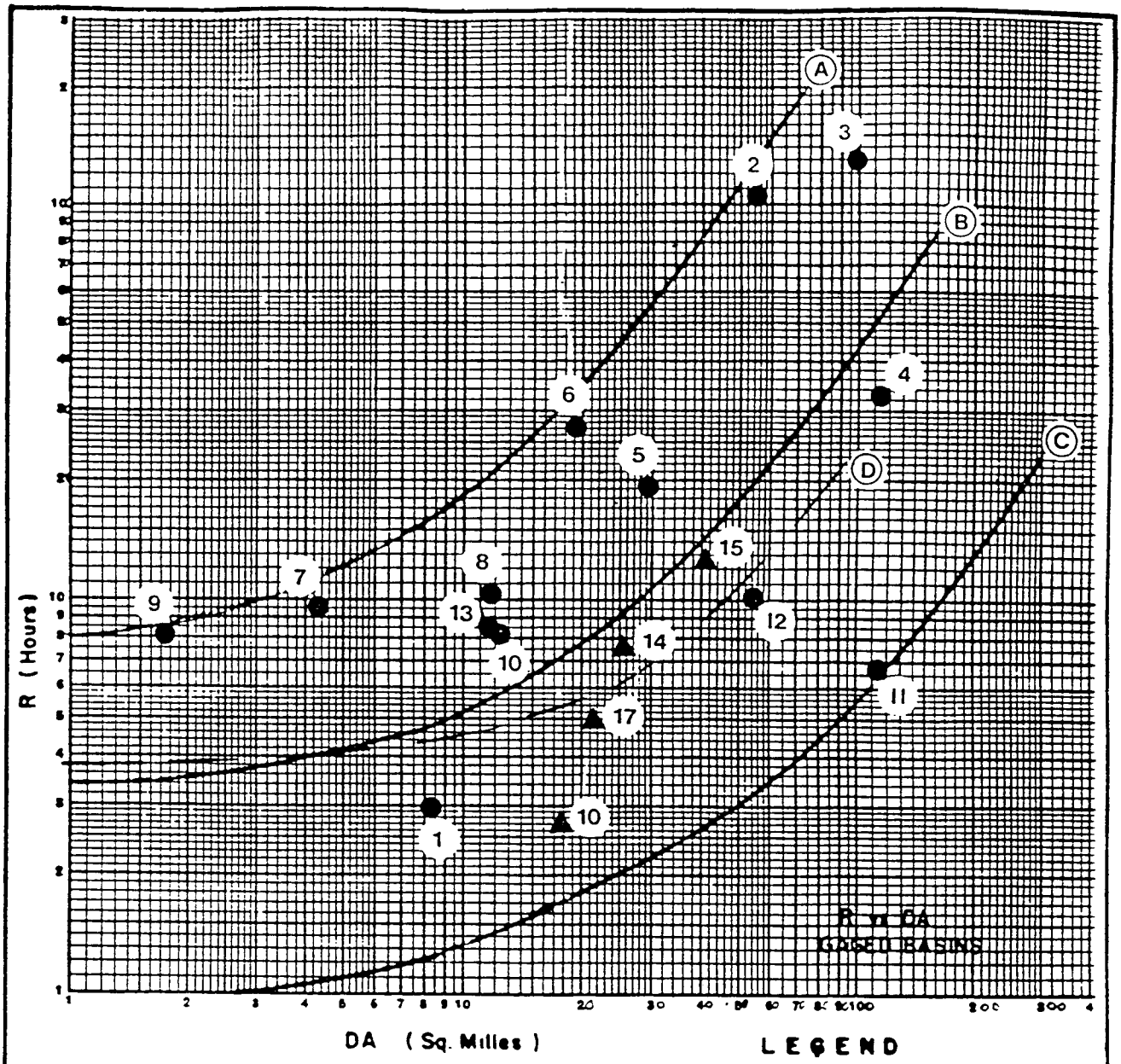
D = ORIGINAL ESTIMATES OF TC AND R FOR THE LOWER VALLEY ONLY.

⊙ = HEC 1 OPTIMIZATION RUNS.

△ = POINTS OBTAINED FROM C.O.E. RAHWAY RIVER STUDY.

Passaic River Flood Protection Project
Phase II GDM
CLARK'S TIME of CONCENTRATION
"TC"
VS
DRAINAGE AREA
CENAN-PR-C MARCH 1992

Figure C-43



LEGEND

A = UPPER LIMIT OF ENVELOPE CURVE.

A = MAXIMUM ENVELOPE OF FINAL TC'S IN HIGHLAND AREAS.

B = BEST FIT BY EYE CURVE.

C = LOWER LIMIT OF ENVELOPE CURVE.

D = ORIGINAL ESTIMATES OF TC AND R FOR THE LOWER VALLEY ONLY.

⊙ = HEC 1 OPTIMIZATION RUNS.

△ = POINTS OBTAINED FROM C.O.E. RAHWAY RIVER STUDY.

NOTE: See Figure 20 for Plot N° Identification.

Passaic River Flood Protection Project

Phase II GDM

CLARK'S "R"
VS
DRAINAGE AREA

CENAN-PR-C MARCH 1992

Figure C-44

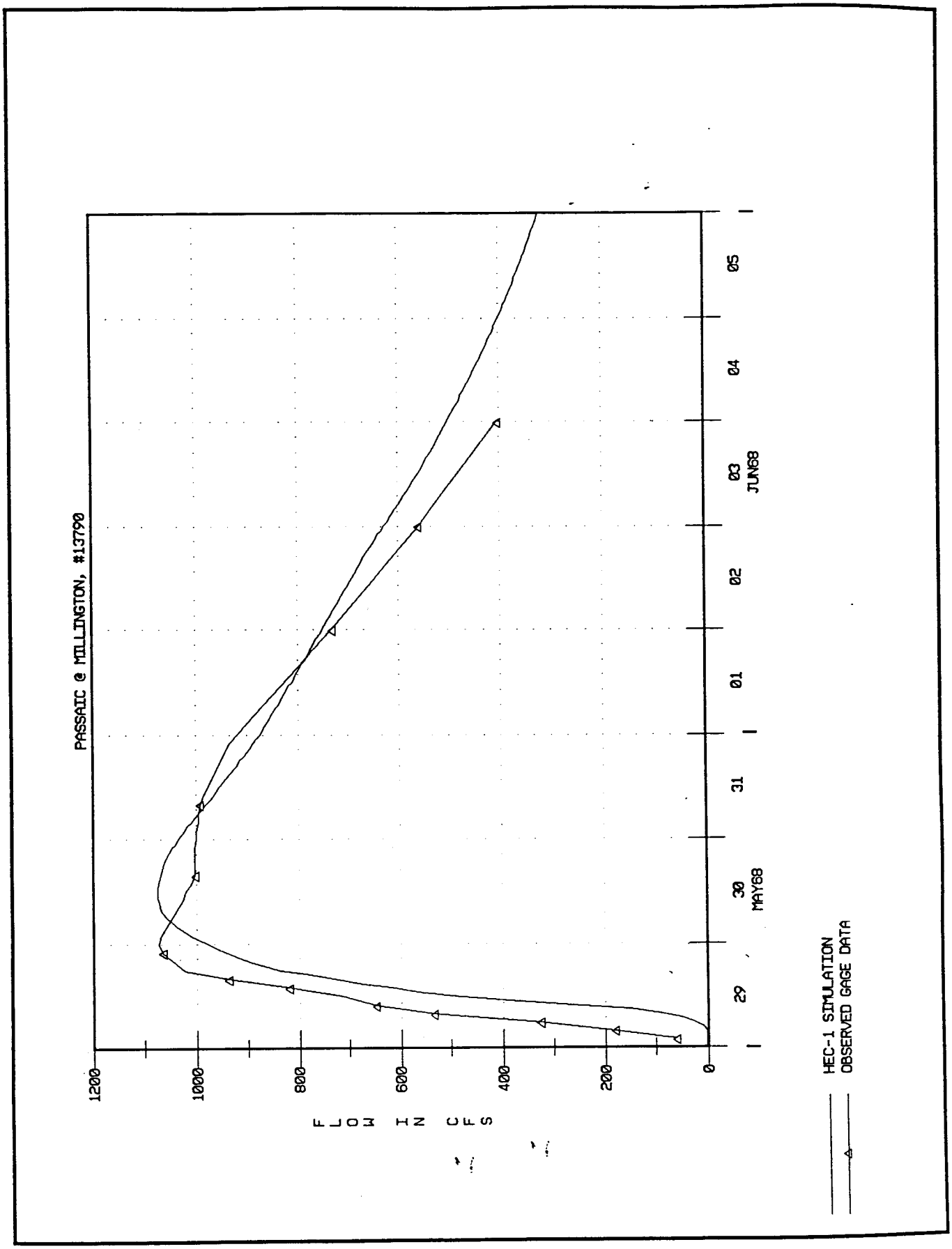

PASSAIC @ MILLINGTON, #13790
FLOW IN CFS
0
200
400
600
800
1000
1200
29
31
01
02
03
04
05
30
MAY68
JUN68
HEC-1 SIMULATION
OBSERVED GAGE DATA

Figure C-45a

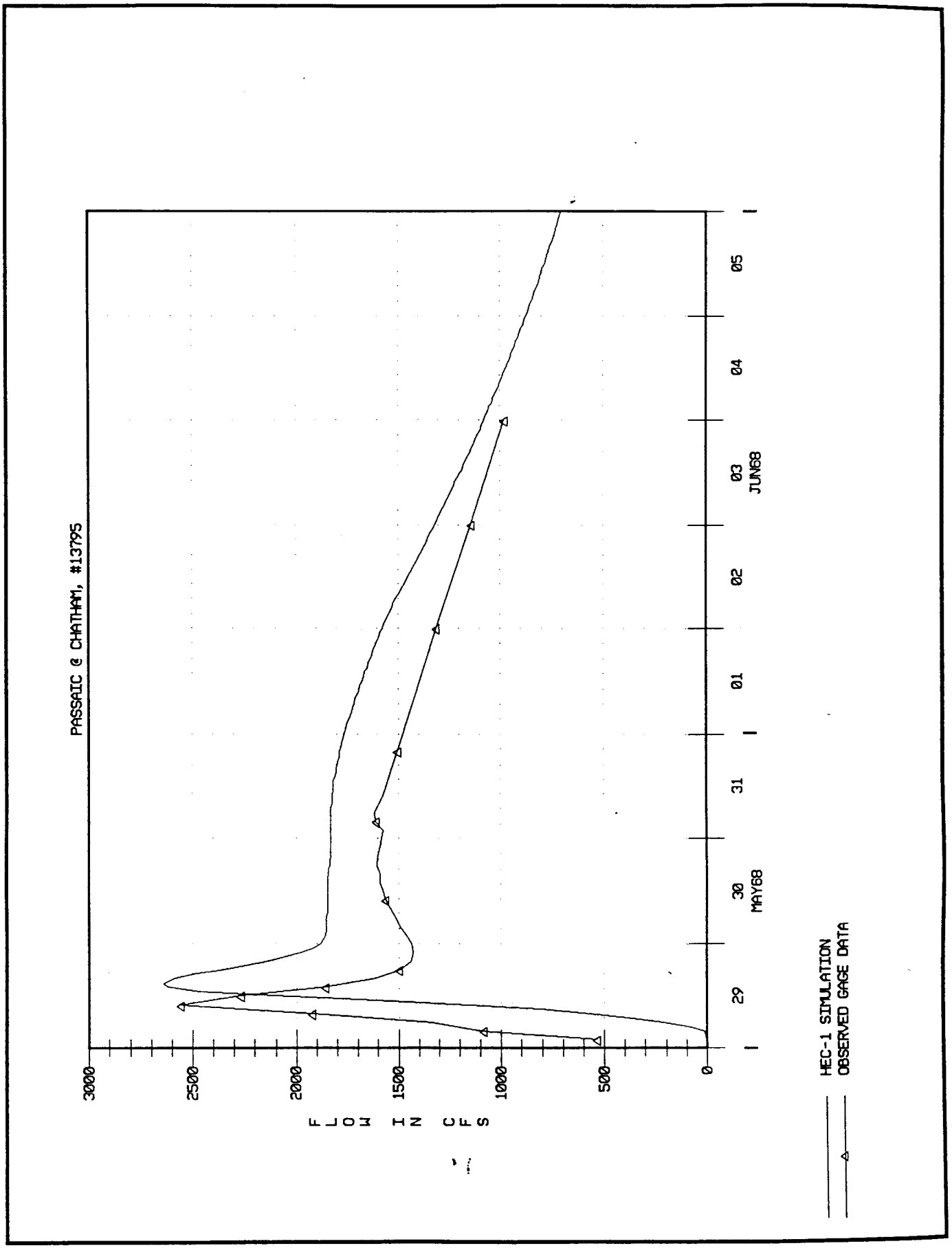

PASSAIC @ CHATHAM, #13795
FLOW IN CFS
3000
2500
2000
1500
1000
500
0
29
30
MAY68
31
01
02
03
JUN68
04
05
HEC-1 SIMULATION
OBSERVED GAGE DATA

Figure C-45b

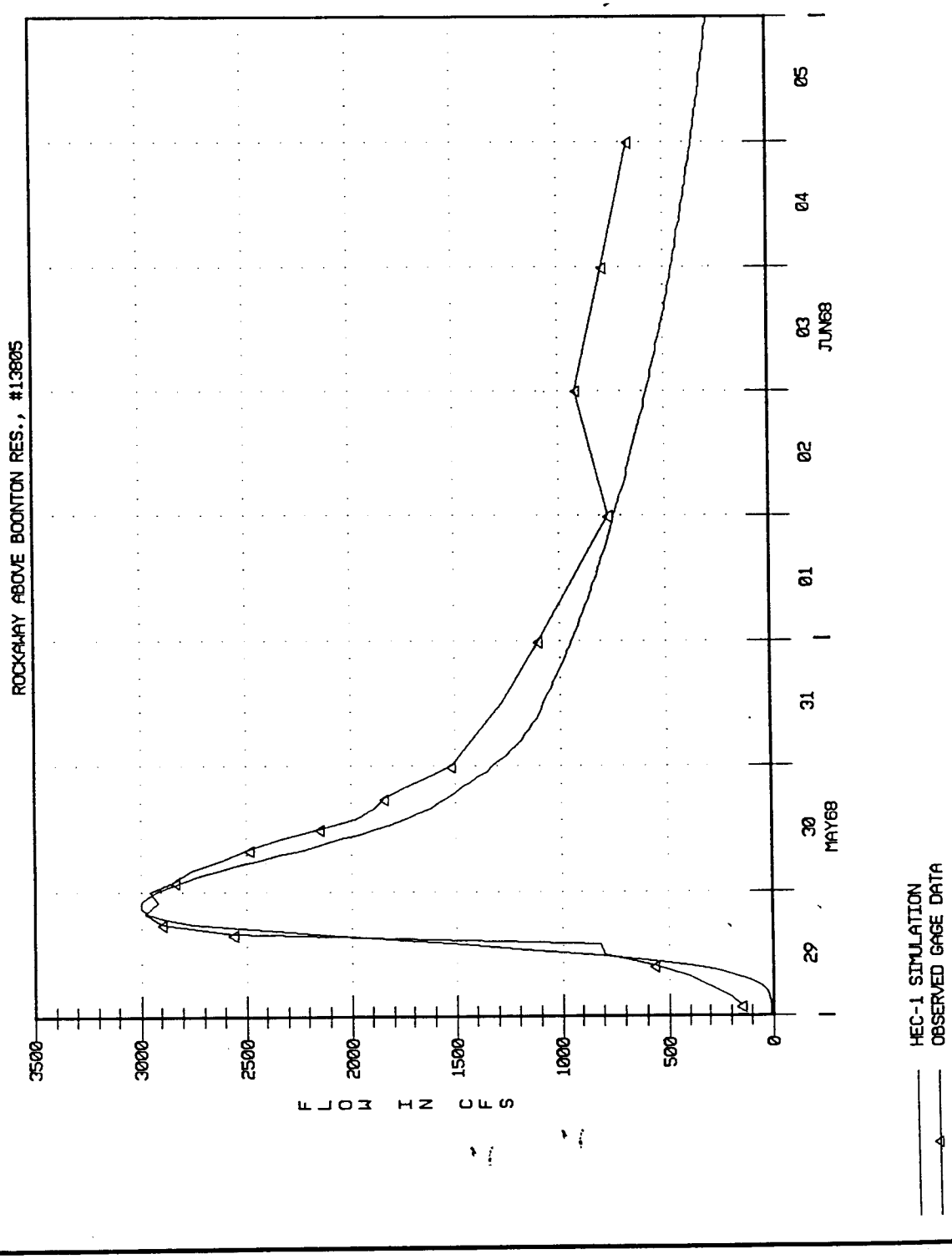

ROCKAWAY ABOVE BOONTON RES., #13805
FLOW IN CFS
3500
3000
2500
2000
1500
1000
500
0
29
30
MAY68
31
01
02
03
JUN68
04
05
HEC-1 SIMULATION
OBSERVED GAGE DATA

Figure C-45c

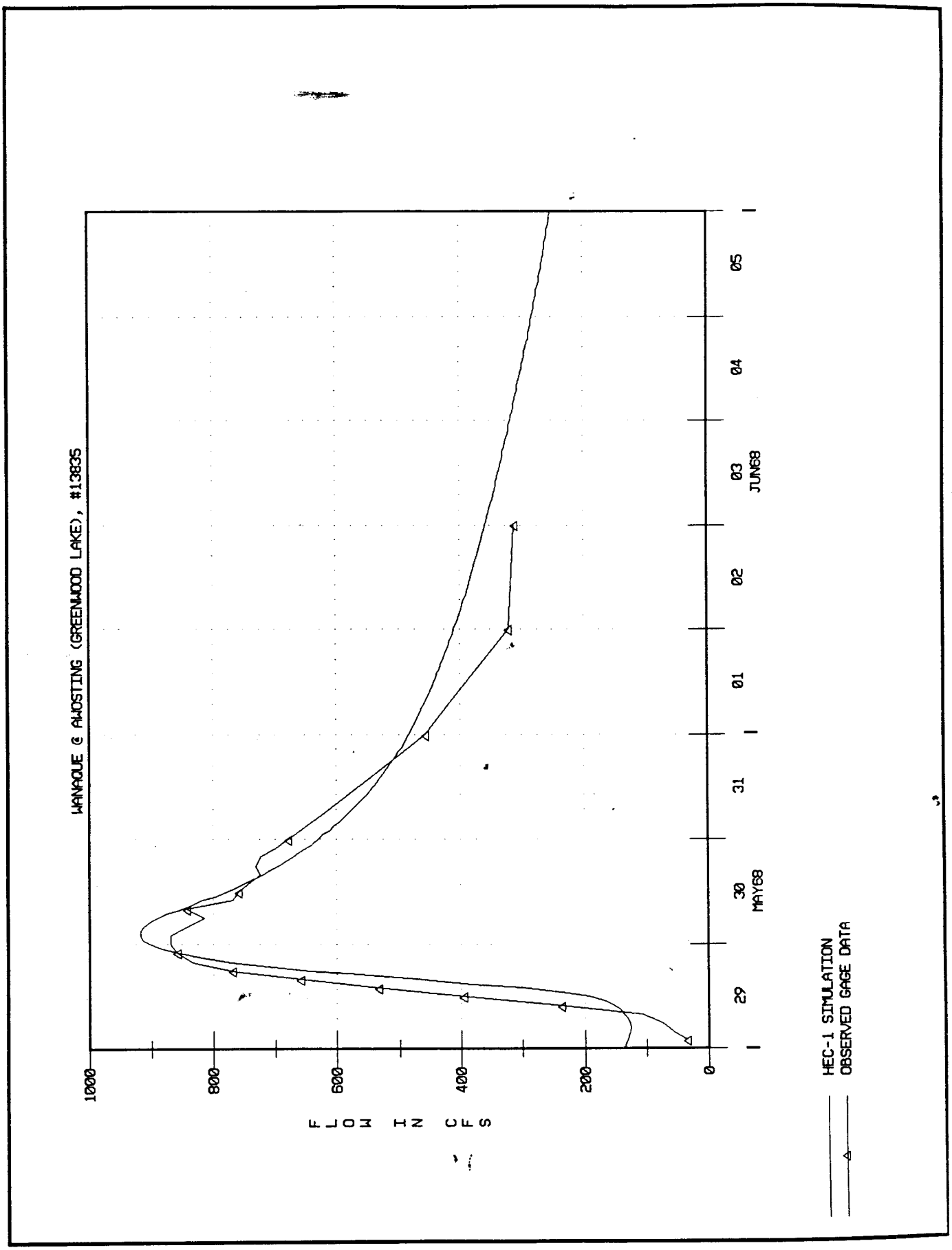

WANAQUE @ AWOSTING (GREENWOOD LAKE), #13835
FLOW IN CFS
1000
800
600
400
200
0
29
30
MAY68
31
01
02
03
JUN68
04
05
HEC-1 SIMULATION
OBSERVED GAGE DATA

Figure C-45d

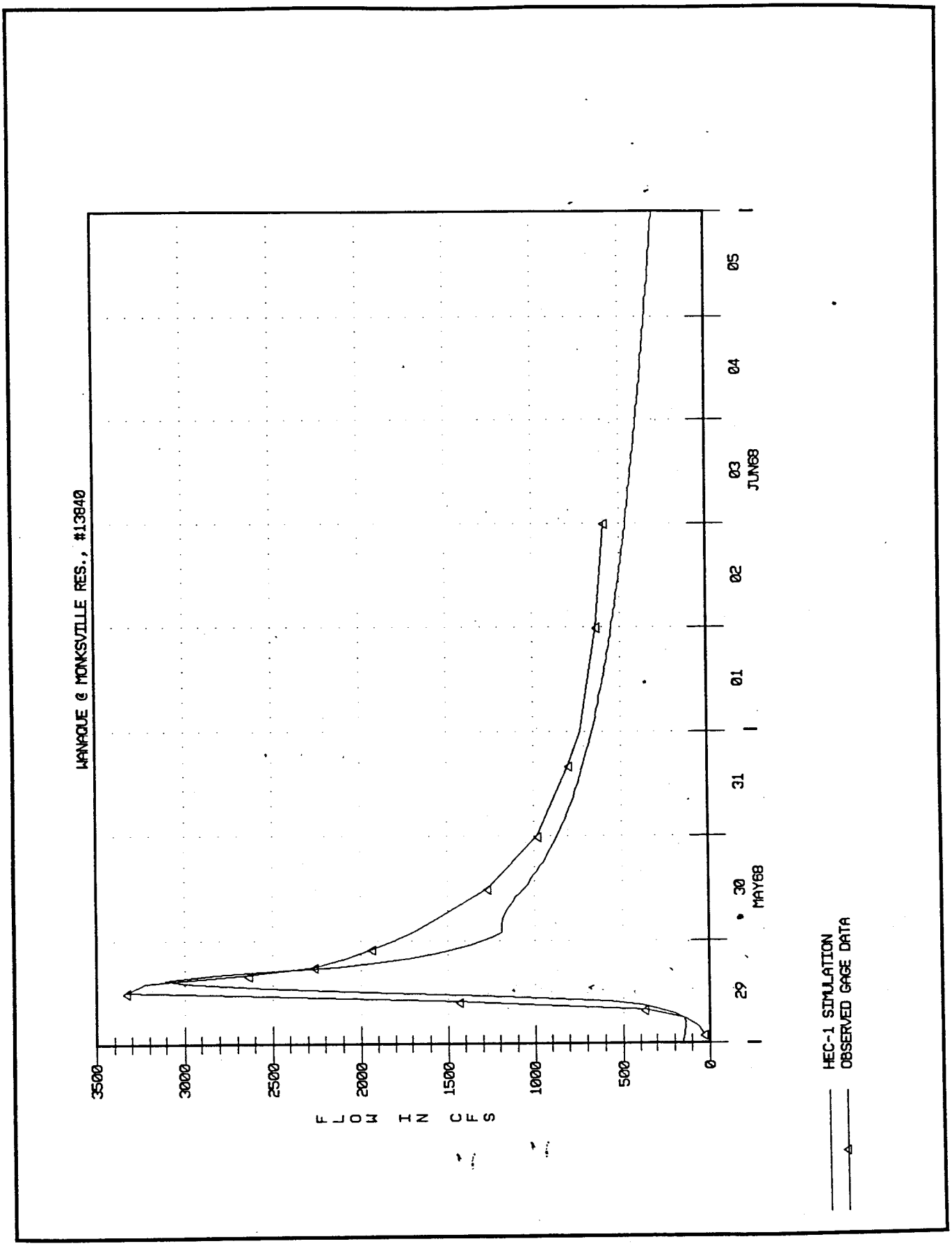

WANAQUE @ MONKSVILLE RES., #13840
FLOW IN CFS
3500
3000
2500
2000
1500
1000
500
0
29
30
31
01
02
03
04
05
MAY68
JUN68
HEC-1 SIMULATION
OBSERVED GAGE DATA

Figure C-45e

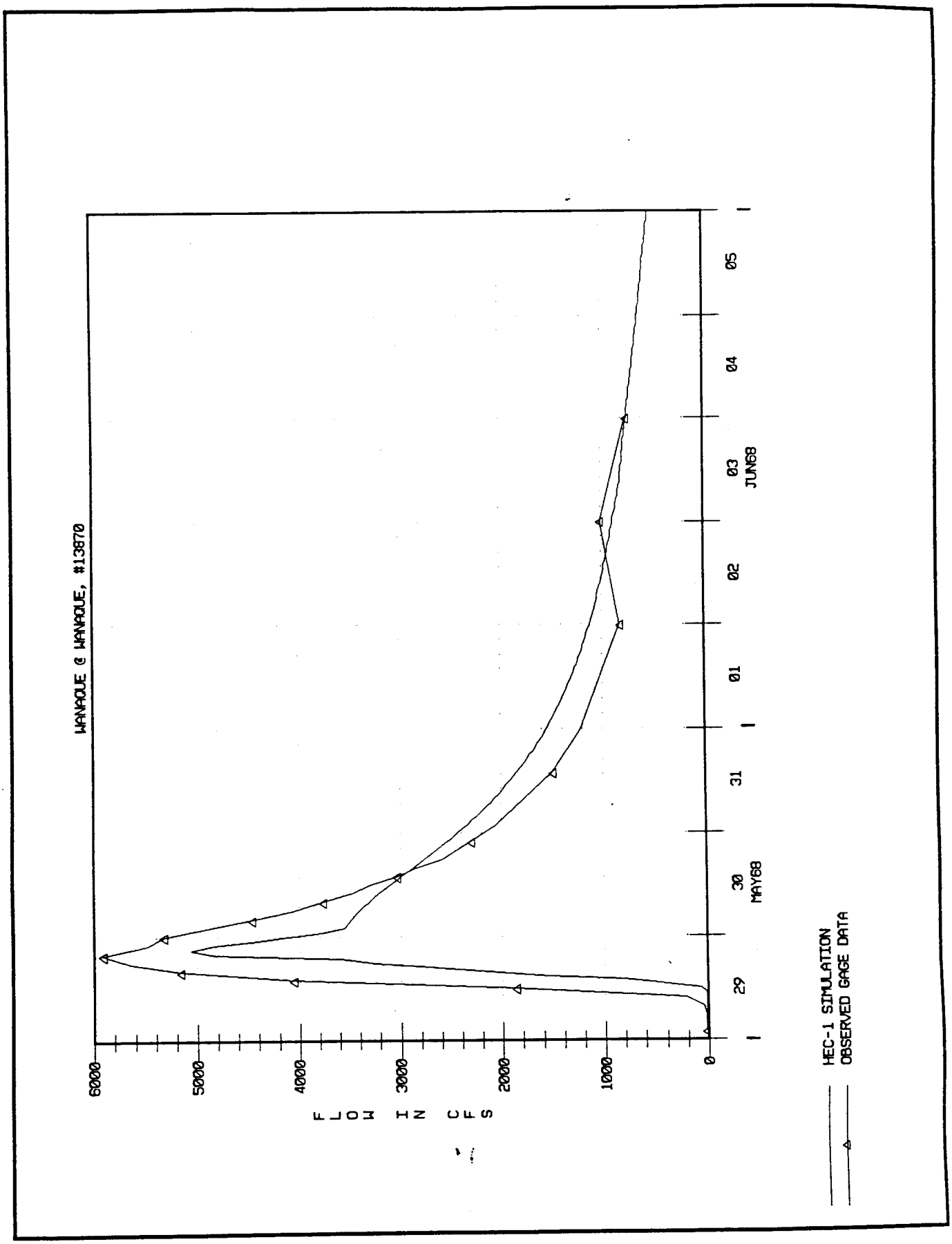

WANAQUE @ WANAQUE, #13870
FLOW IN CFS
6000
5000
4000
3000
2000
1000
0
29
30
31
01
02
03
04
05
MAY68
JUN68
HEC-1 SIMULATION
OBSERVED GAGE DATA

Figure C-45f

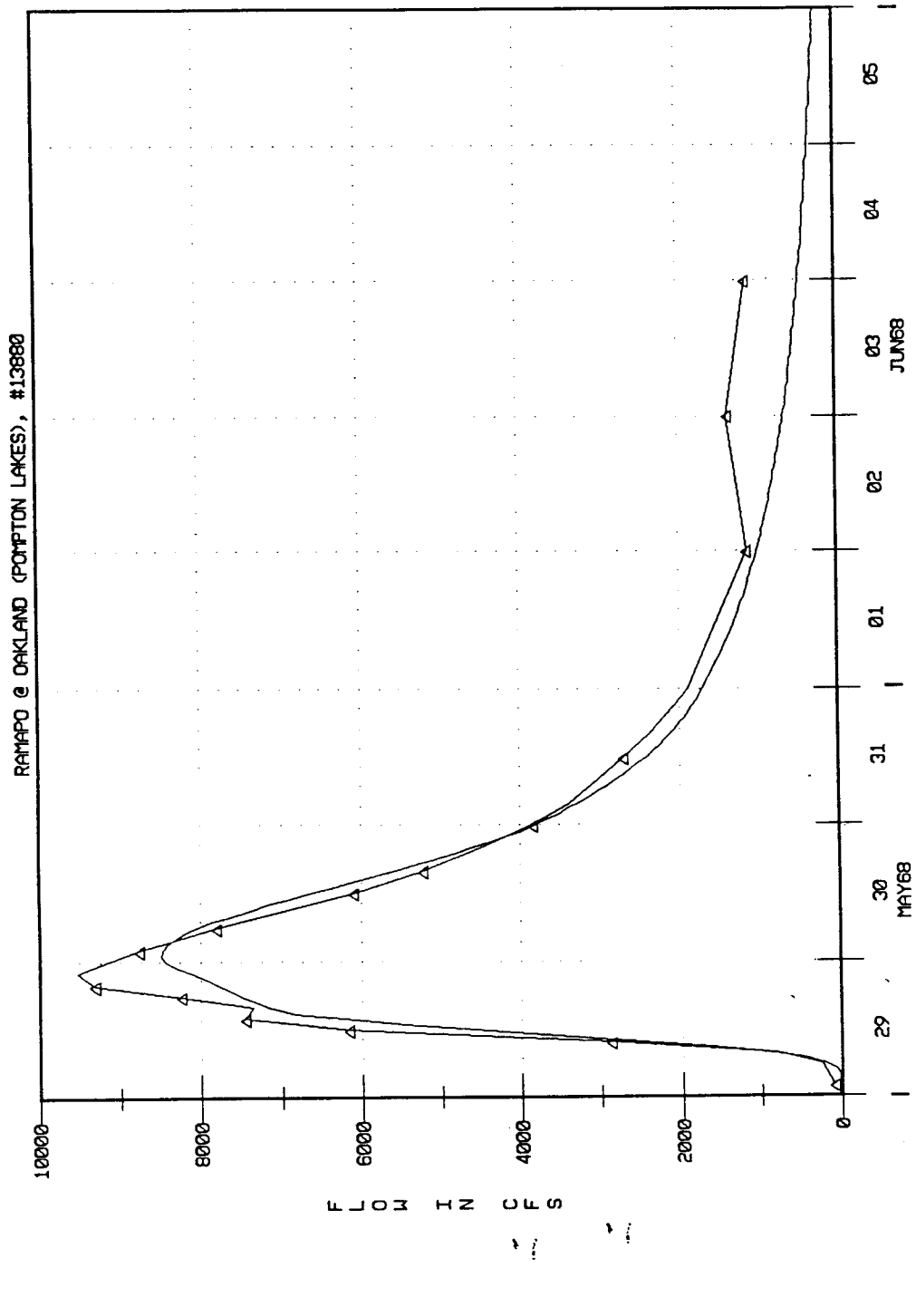

RAMAPO @ OAKLAND (POMPTON LAKES), #13880
10000
8000
6000
4000
2000
0
FLOW IN CFS
29
30
MAY68
31
01
02
03
JUN68
04
05
HEC-1 SIMULATION
OBSERVED GAGE DATA

Figure C-45g

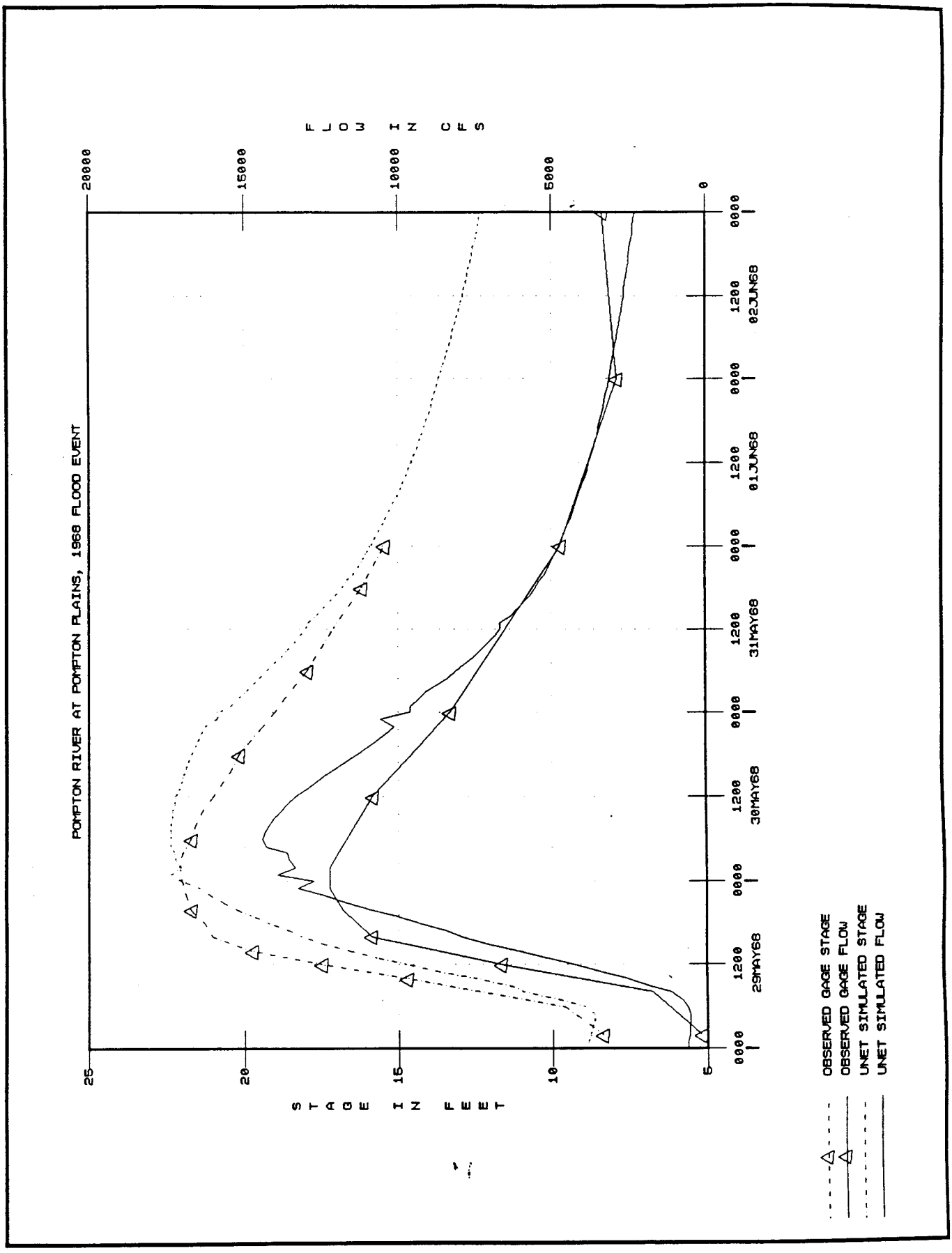

POMPTON RIVER AT POMPTON PLAINS, 1968 FLOOD EVENT
STAGE IN FEET
FLOW IN CFS
25
20
15
10
5
20000
15000
10000
5000
0
0000
1200
29MAY68
30MAY68
31MAY68
01JUN68
02JUN68
OBSERVED GAGE STAGE
OBSERVED GAGE FLOW
UNET SIMULATED STAGE
UNET SIMULATED FLOW

Figure C-45h

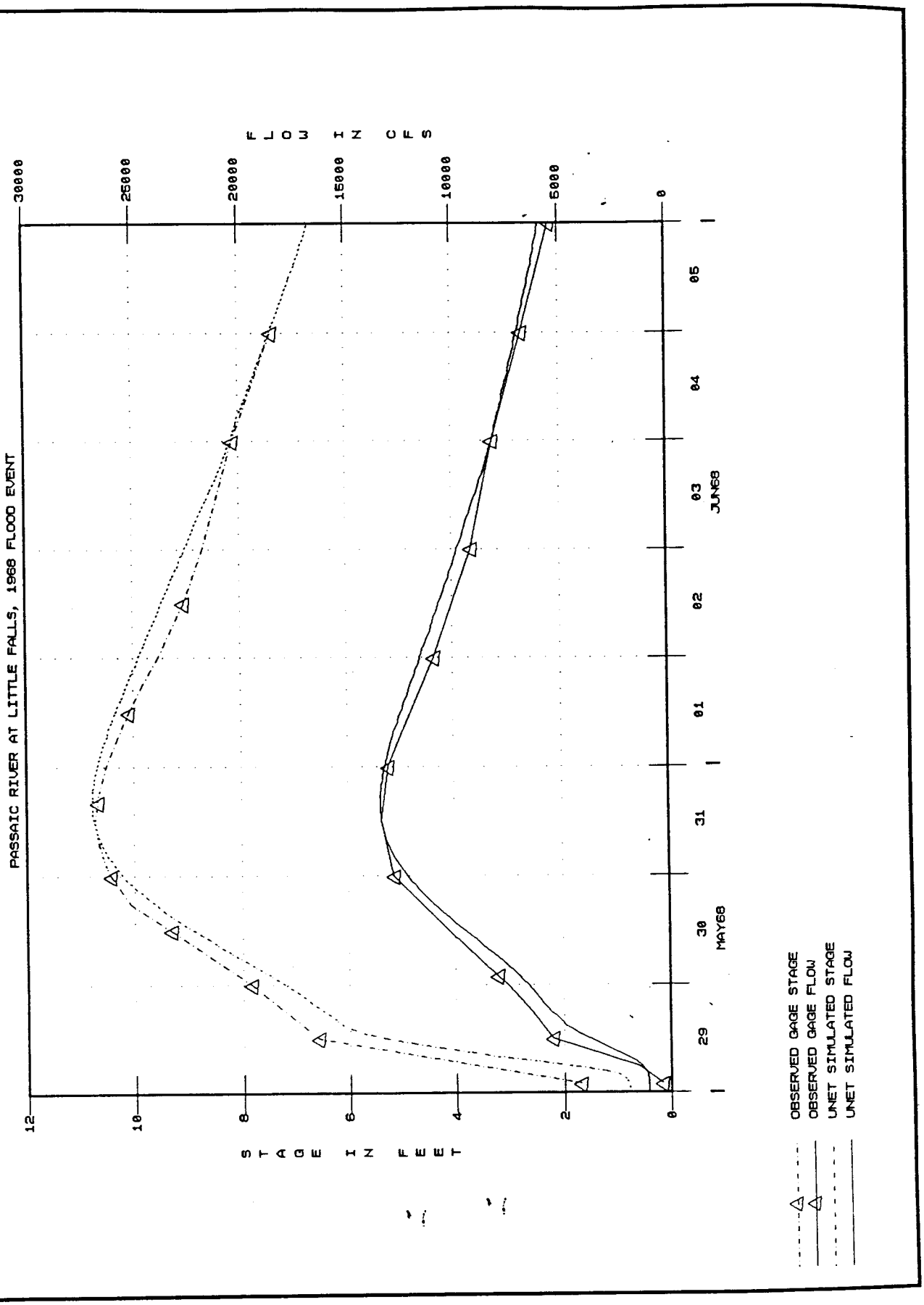

PASSAIC RIVER AT LITTLE FALLS, 1968 FLOOD EVENT
STAGE IN FEET
FLOW IN CFS
12
10
8
6
4
2
0
30000
25000
20000
15000
10000
5000
0
29
30
31
01
02
03
04
05
MAY68
JUN68
OBSERVED GAGE STAGE
OBSERVED GAGE FLOW
UNET SIMULATED STAGE
UNET SIMULATED FLOW

Figure C-45i

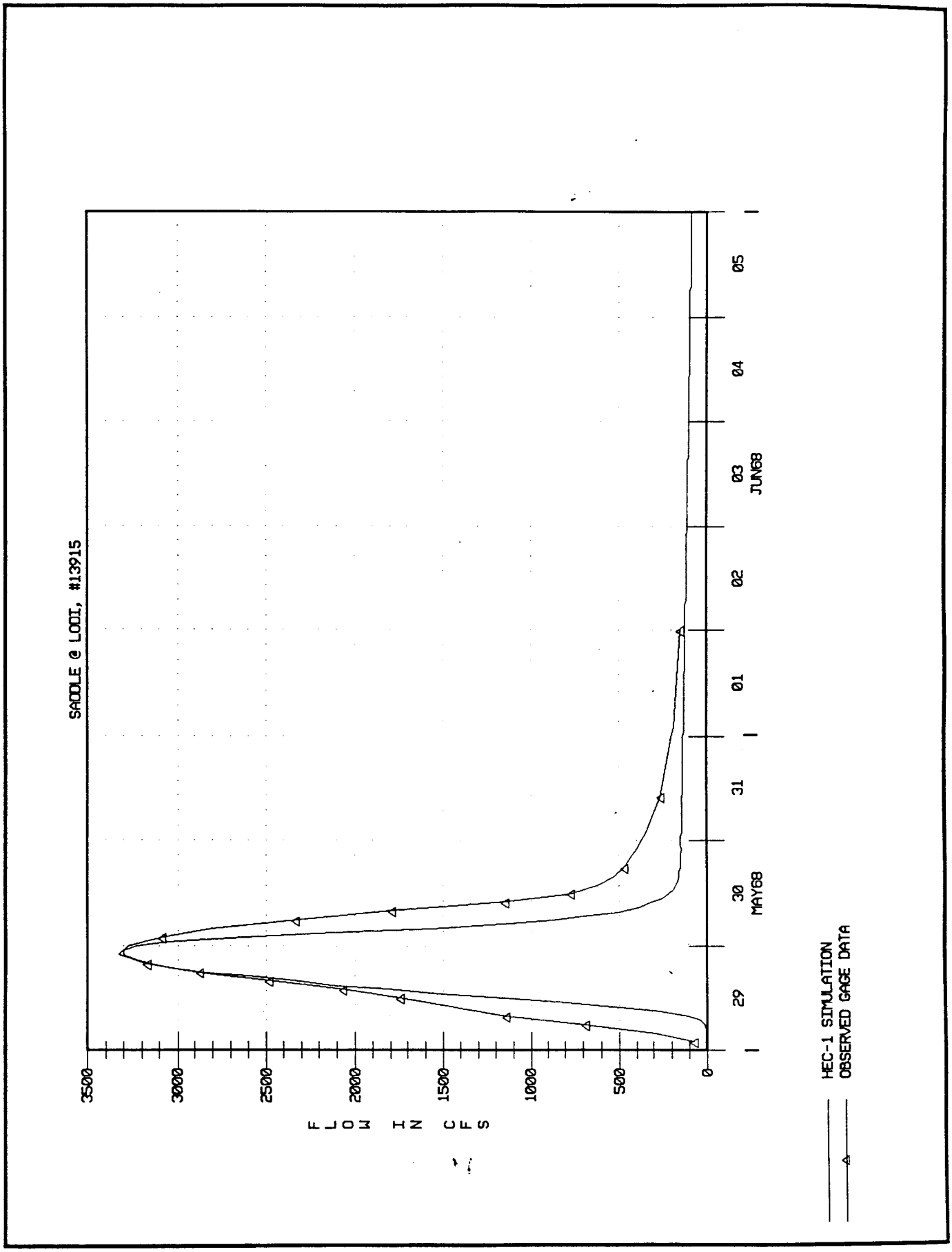

SADDLE @ LOVI, #13915
FLOW IN CFS
3500
3000
2500
2000
1500
1000
500
0
29
30
MAY68
31
01
02
03
JUN68
04
05
HEC-1 SIMULATION
OBSERVED GAGE DATA

Figure C-45j

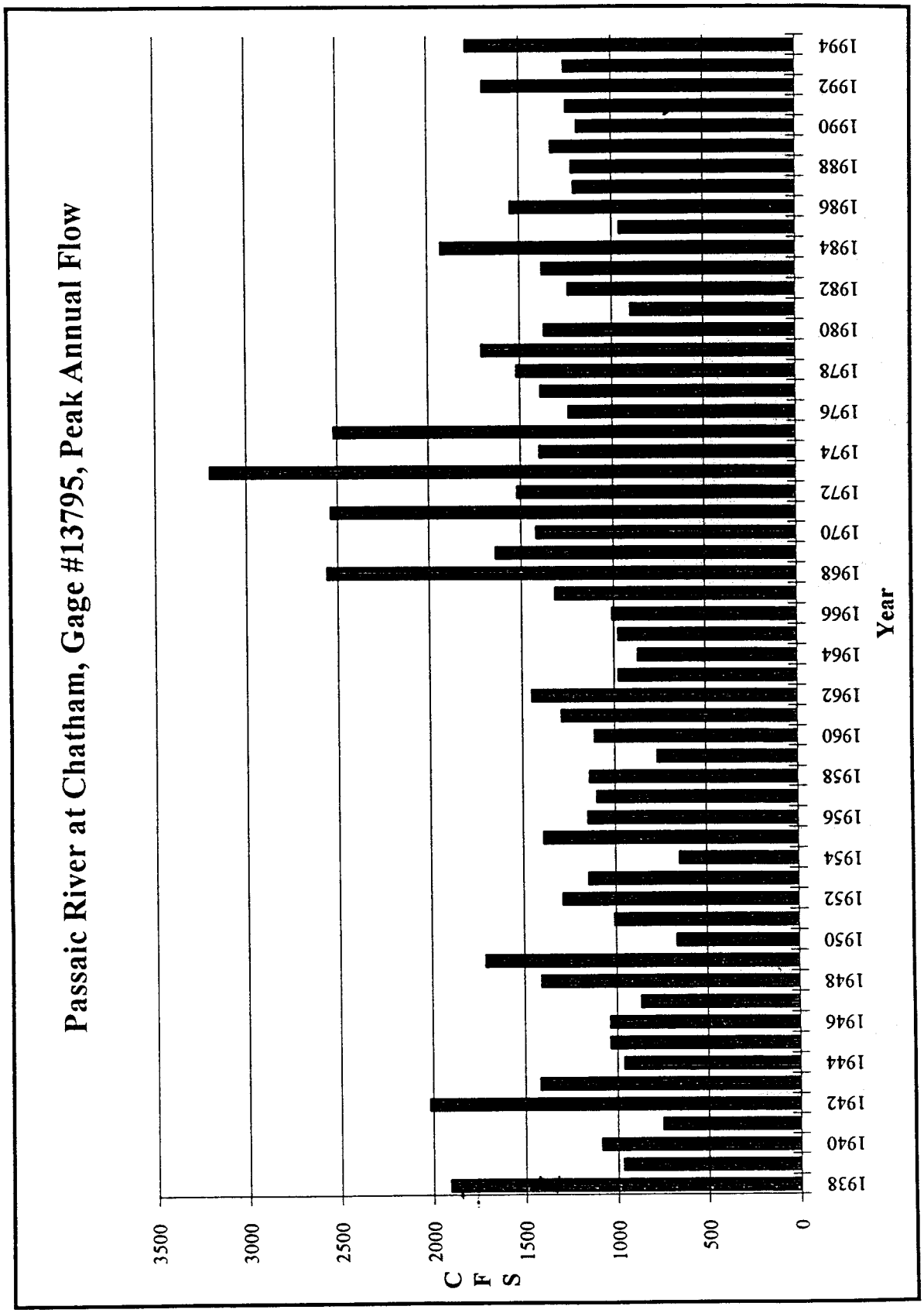

Passaic River at Chatham, Gage #13795, Peak Annual Flow
CFS
3500
3000
2500
2000
1500
1000
500
0
Year
1938
1940
1942
1944
1946
1948
1950
1952
1954
1956
1958
1960
1962
1964
1966
1968
1970
1972
1974
1976
1978
1980
1982
1984
1986
1988
1990
1992
1994

Figure C-46a

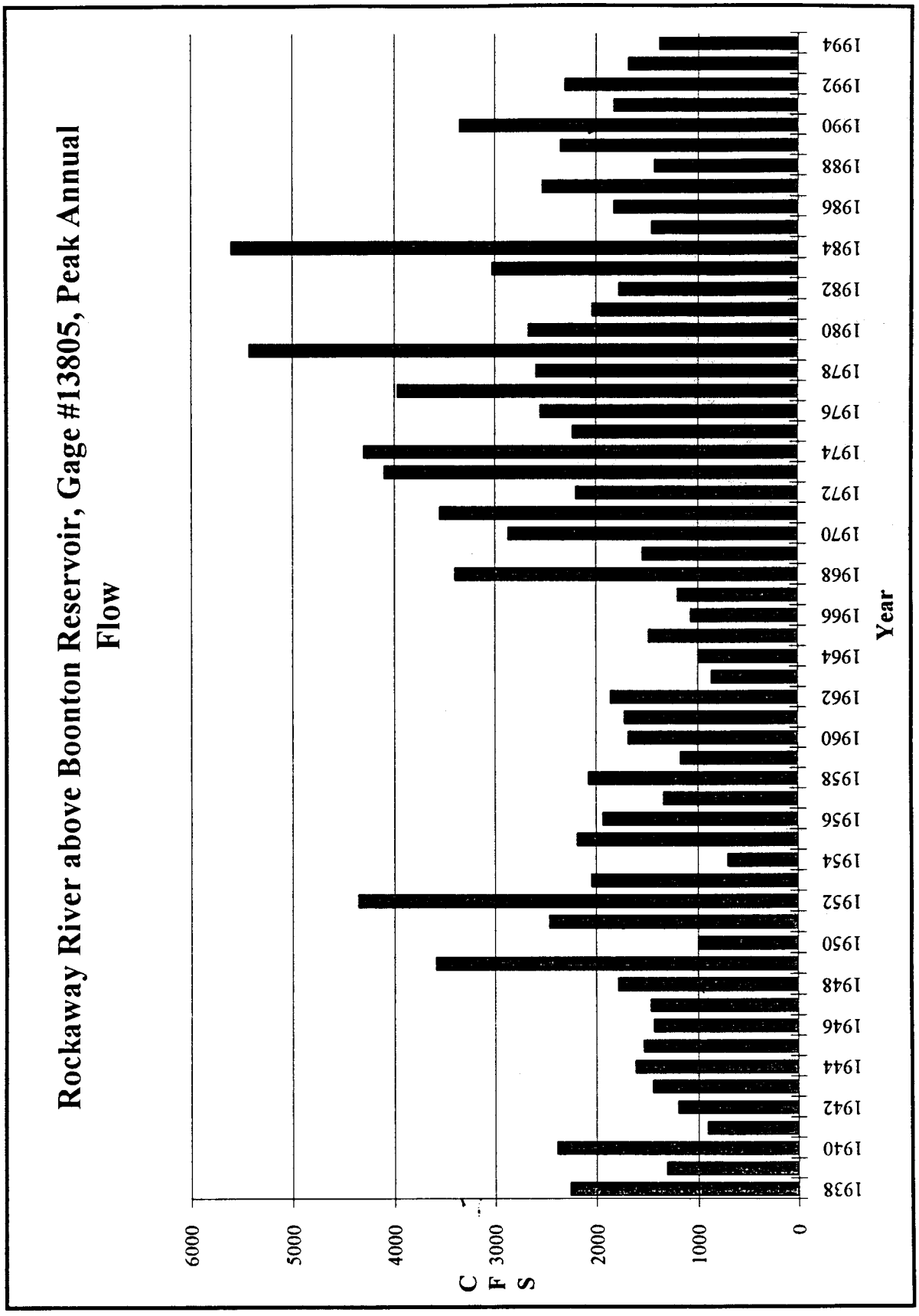

Rockaway River above Boonton Reservoir, Gage #13805, Peak Annual Flow
CFS
6000
5000
4000
3000
2000
1000
0
1938
1940
1942
1944
1946
1948
1950
1952
1954
1956
1958
1960
1962
1964
1966
1968
1970
1972
1974
1976
1978
1980
1982
1984
1986
1988
1990
1992
1994
Year

Figure C-46b

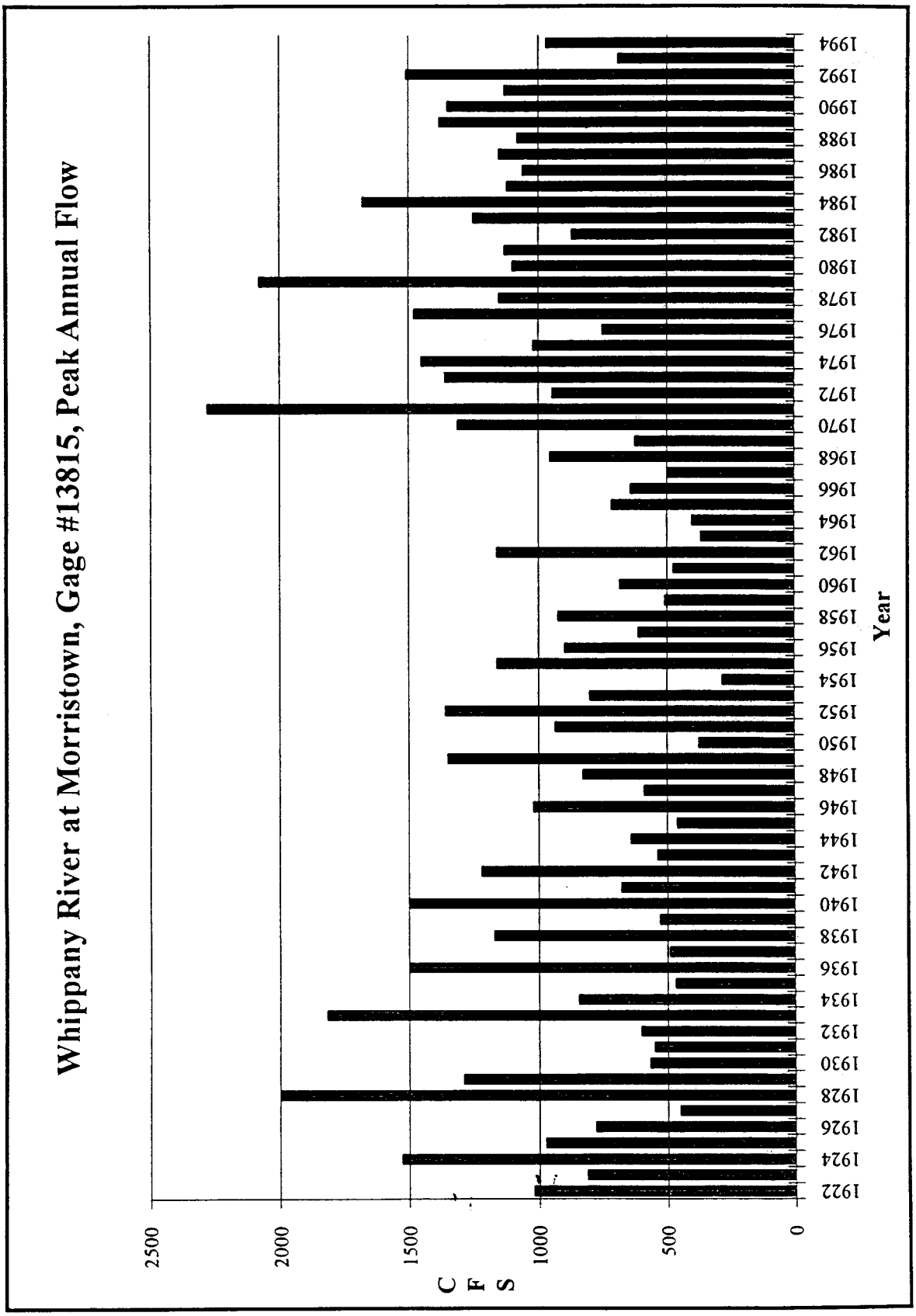

Whippany River at Morristown, Gage #13815, Peak Annual Flow
CFS
2500
2000
1500
1000
500
0
Year
1994
1992
1990
1988
1986
1984
1982
1980
1978
1976
1974
1972
1970
1968
1966
1964
1962
1960
1958
1956
1954
1952
1950
1948
1946
1944
1942
1940
1938
1936
1934
1932
1930
1928
1926
1924
1922

Figure C-46c

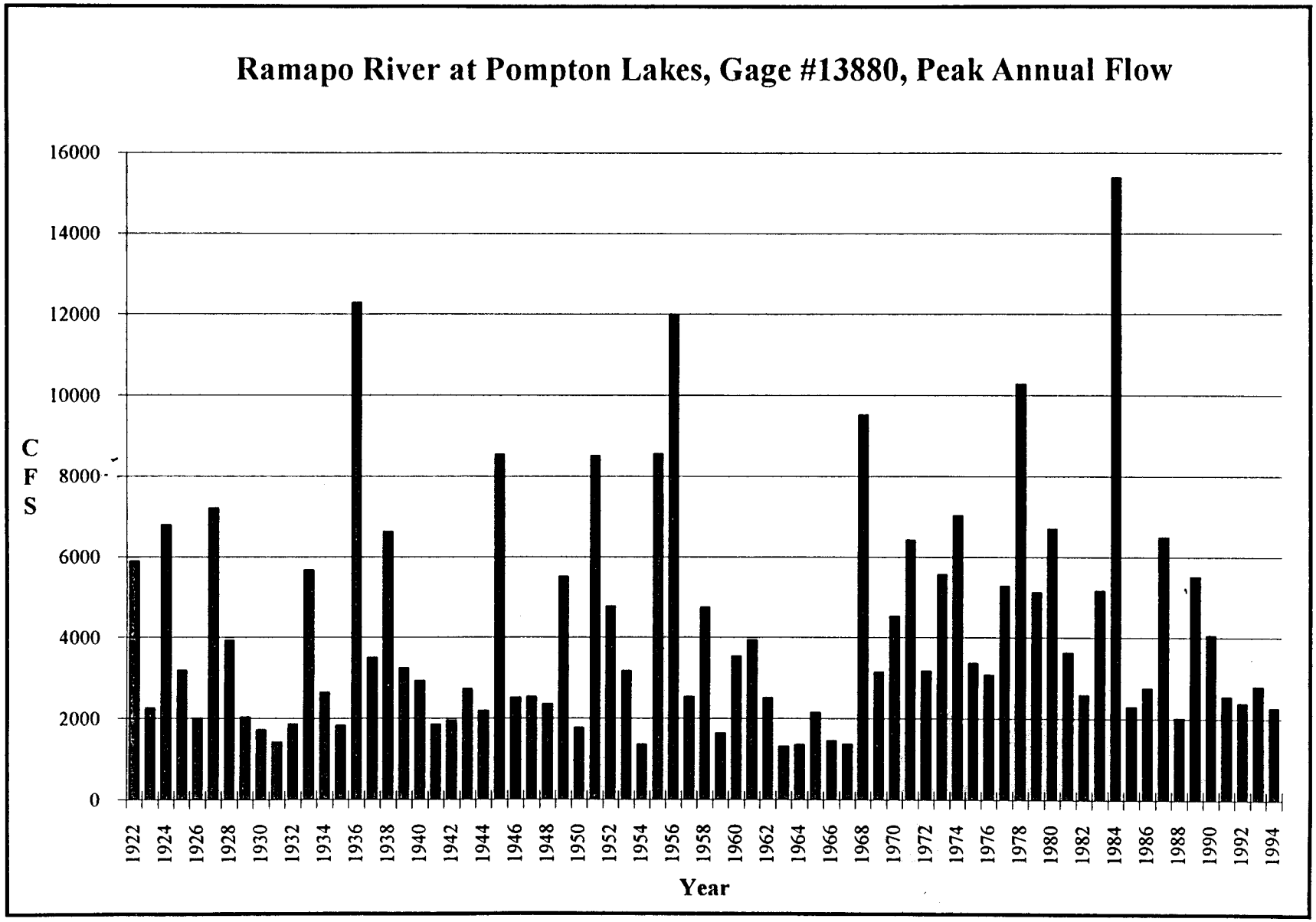

Ramapo River at Pompton Lakes, Gage #13880, Peak Annual Flow
CFS
16000
14000
12000
10000
8000
6000
4000
2000
0
1922
1924
1926
1928
1930
1932
1934
1936
1938
1940
1942
1944
1946
1948
1950
1952
1954
1956
1958
1960
1962
1964
1966
1968
1970
1972
1974
1976
1978
1980
1982
1984
1986
1988
1990
1992
1994
Year

Figure C-46d

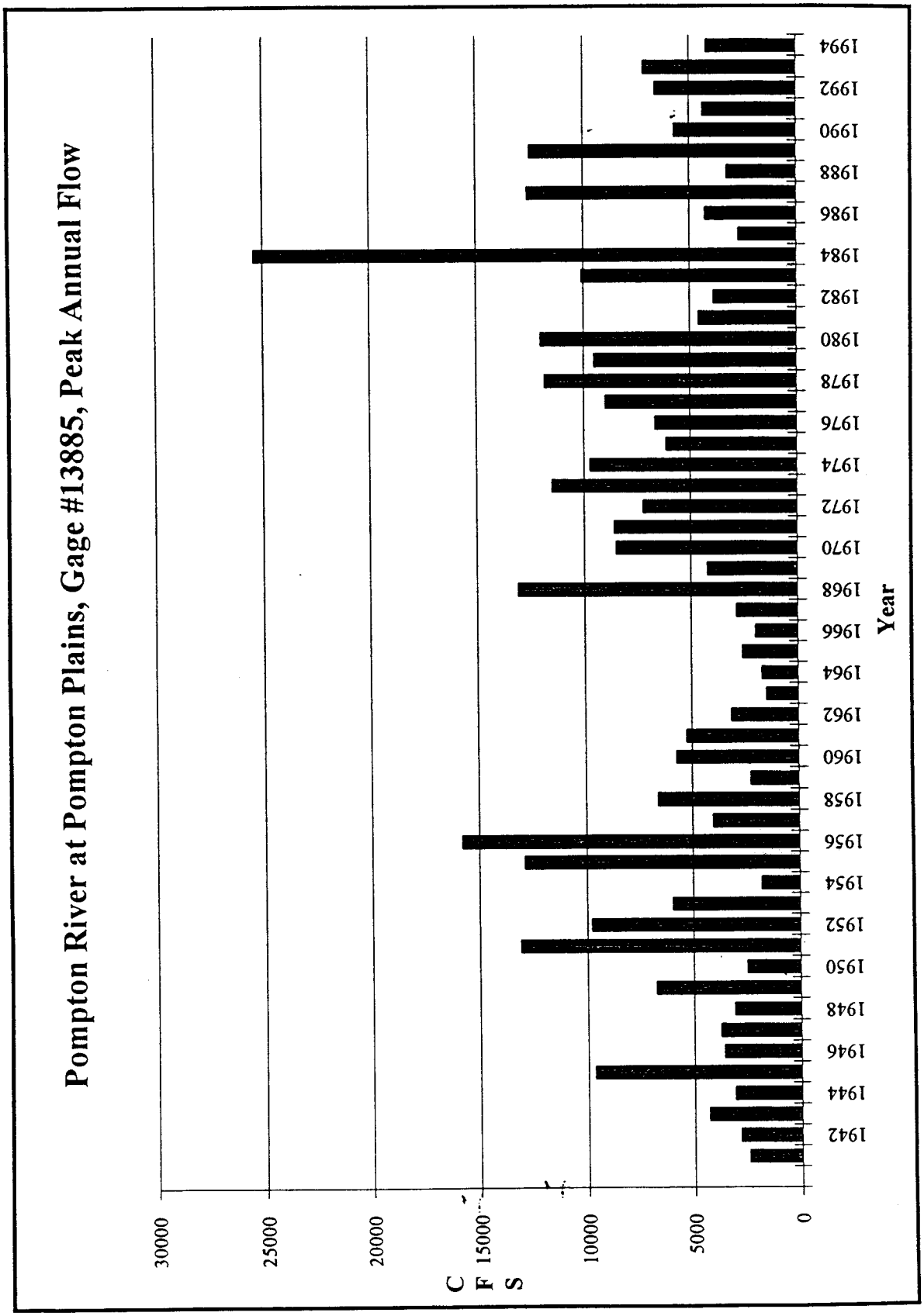

Pompton River at Pompton Plains, Gage #13885, Peak Annual Flow
CFS
30000
25000
20000
15000
10000
5000
0
Year
1994
1992
1990
1988
1986
1984
1982
1980
1978
1976
1974
1972
1970
1968
1966
1964
1962
1960
1958
1956
1954
1952
1950
1948
1946
1944
1942

Figure C-46e

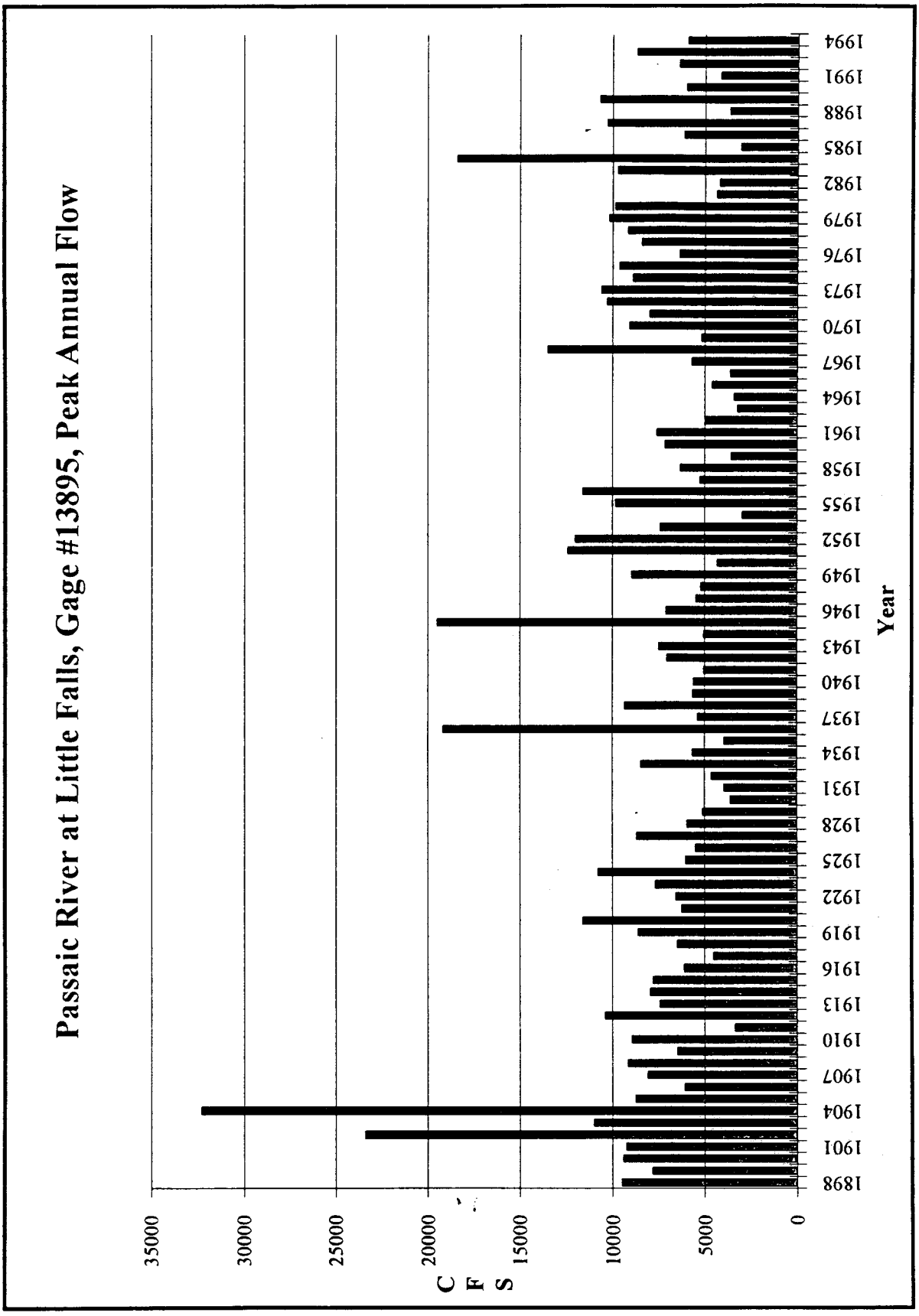

Passaic River at Little Falls, Gage #13895, Peak Annual Flow
CFS
35000
30000
25000
20000
15000
10000
5000
0
Year
1898
1901
1904
1907
1910
1913
1916
1919
1922
1925
1928
1931
1934
1937
1940
1943
1946
1949
1952
1955
1958
1961
1964
1967
1970
1973
1976
1979
1982
1985
1988
1991
1994

Figure C-46f

Comparison of Rainfall at Little Falls and Wanaque with Peak Flow at Pompton Plains

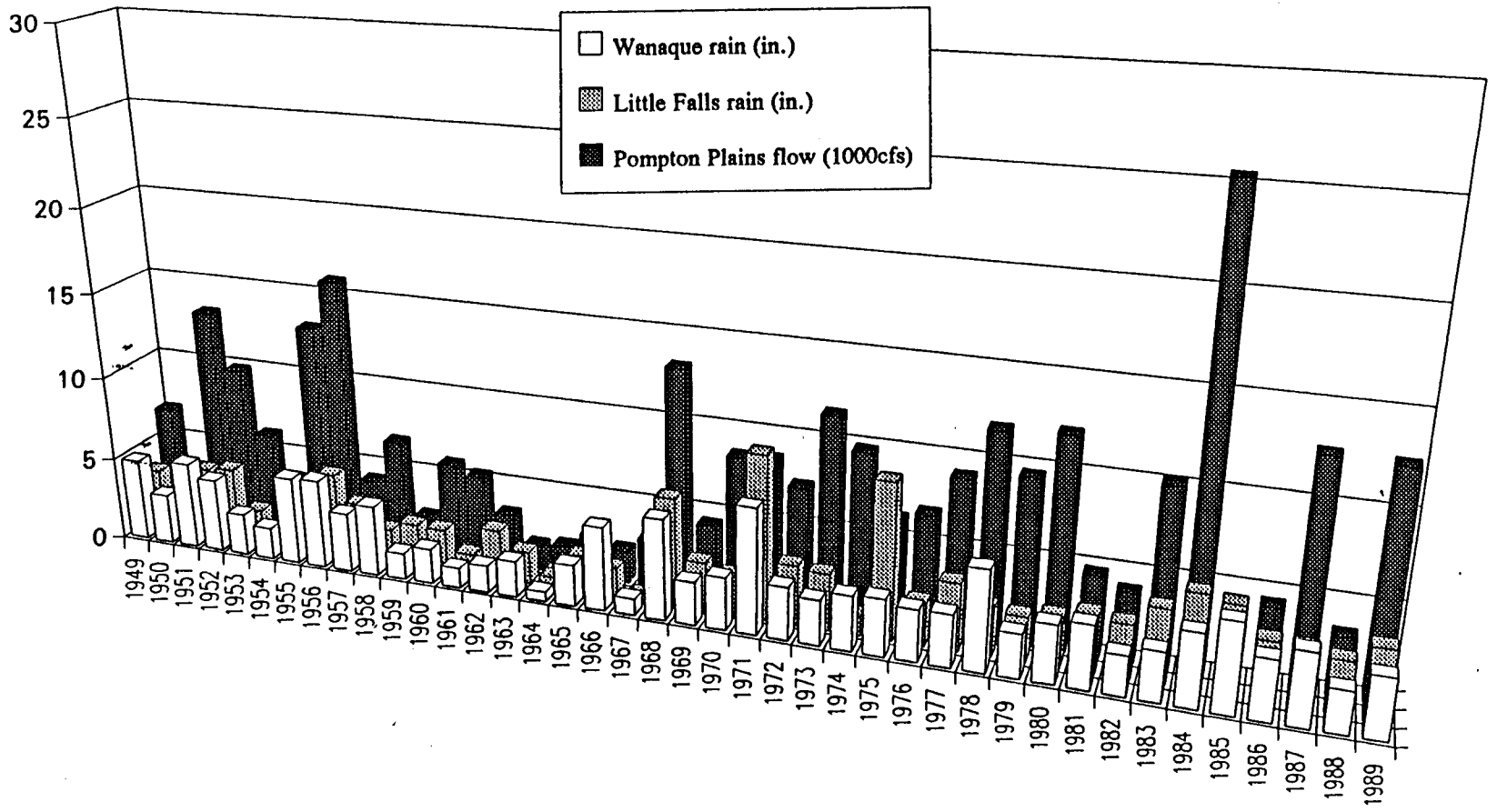


Figure C-46g

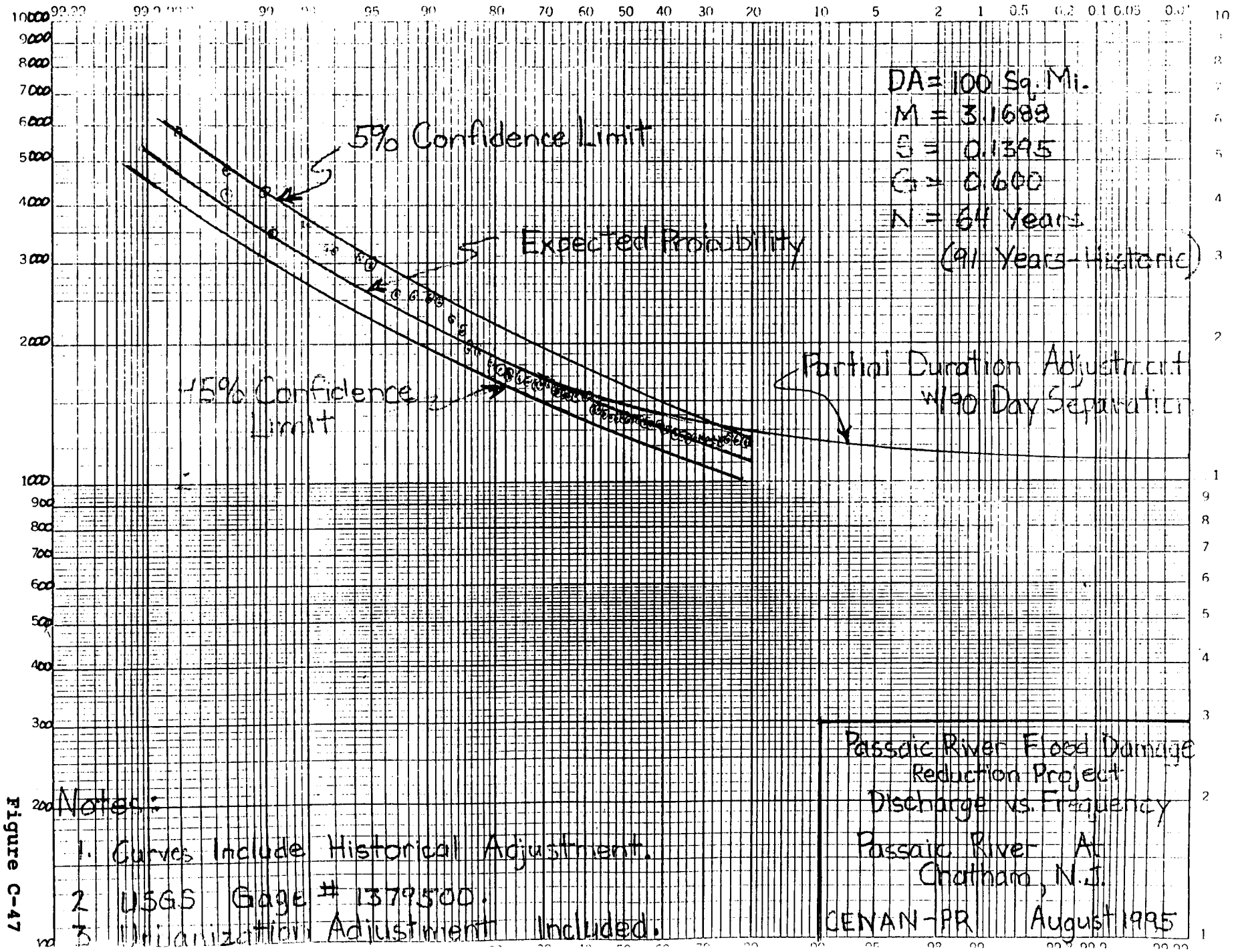

DA = 100 Sq. Mi.
M = 3.1689
S = 0.1395
G = 0.600
N = 64 Years
(91 Years-Historic)
5% Confidence Limit
Expected Probability
95% Confidence Limit
Partial Duration Adjustment w/90 Day Separation
Notes:
1. Curves Include Historical Adjustment.
2. USGS Gage # 1379500.
3. Urbanization Adjustment Included.
Passaic River Flood Damage Reduction Project
Discharge vs. Frequency
Passaic River At Chatham, N.J.
CENAN-PR August 1995

Figure C-47

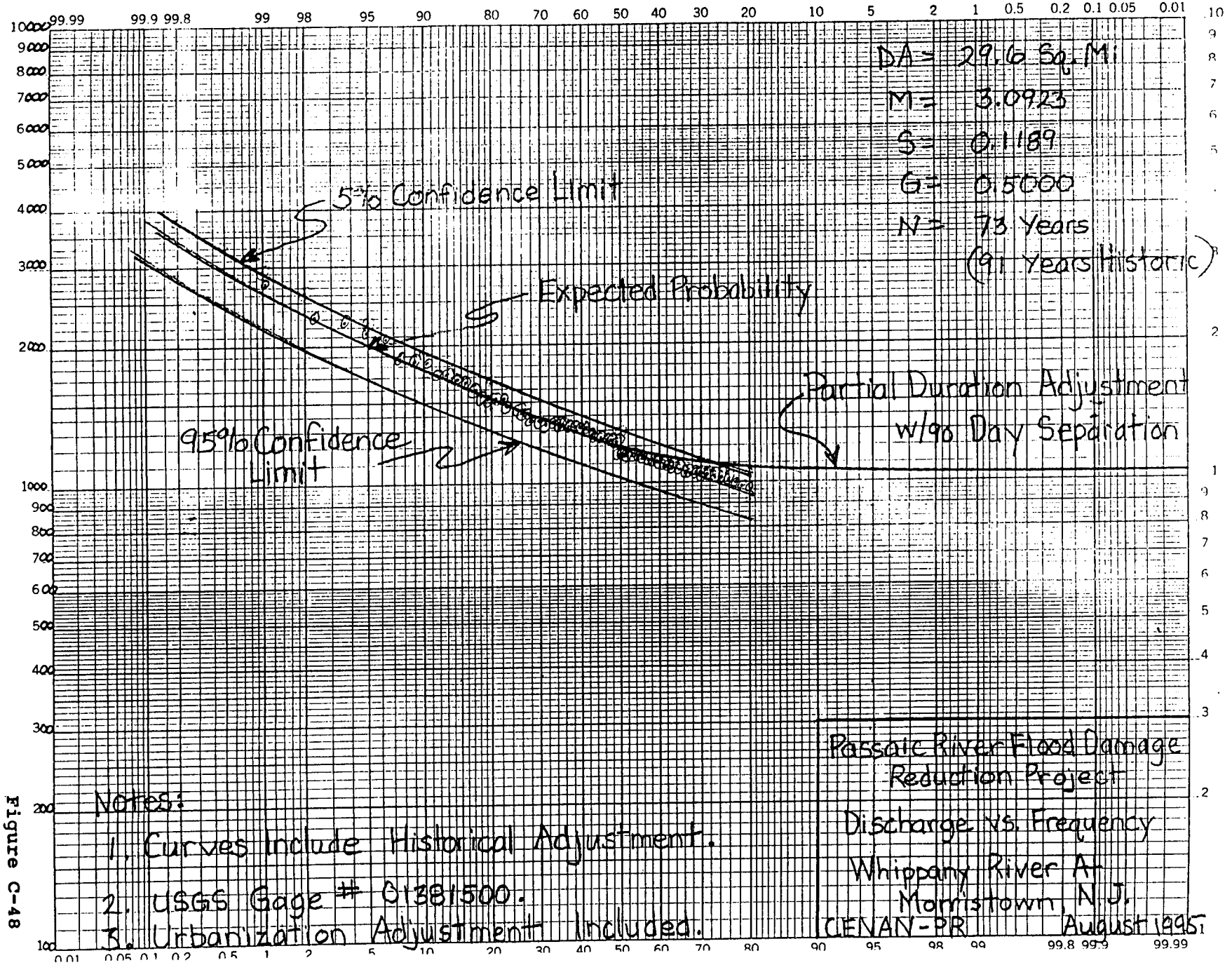

K•Σ PROBABILITY X 2 LOG CYCLES
KEUFFEL & ESSER CO. MADE IN U.S.A.
99.99 99.9 99.8 99 98 95 90 80 70 60 50 40 30 20 10 5 2 1 0.5 0.2 0.1 0.05 0.01
10000 9000 8000 7000 6000 5000 4000 3000 2000 1000 900 800 700 600 500 400 300 200 100
DA = 29.6 Sq. Mi
M = 3.0923
S = 0.1189
G = 0.5000
N = 73 Years
(91 Years Historic)
5% Confidence Limit
Expected Probability
95% Confidence Limit
Partial Duration Adjustment w/96 Day Separation
Notes:
1. Curves Include Historical Adjustment.
2. USGS Gage # 01381500.
3. Urbanization Adjustment Included.
Passaic River Flood Damage Reduction Project
Discharge vs. Frequency
Whippany River At Morristown, N.J.
CENAN-PR August 1995

Figure C-48

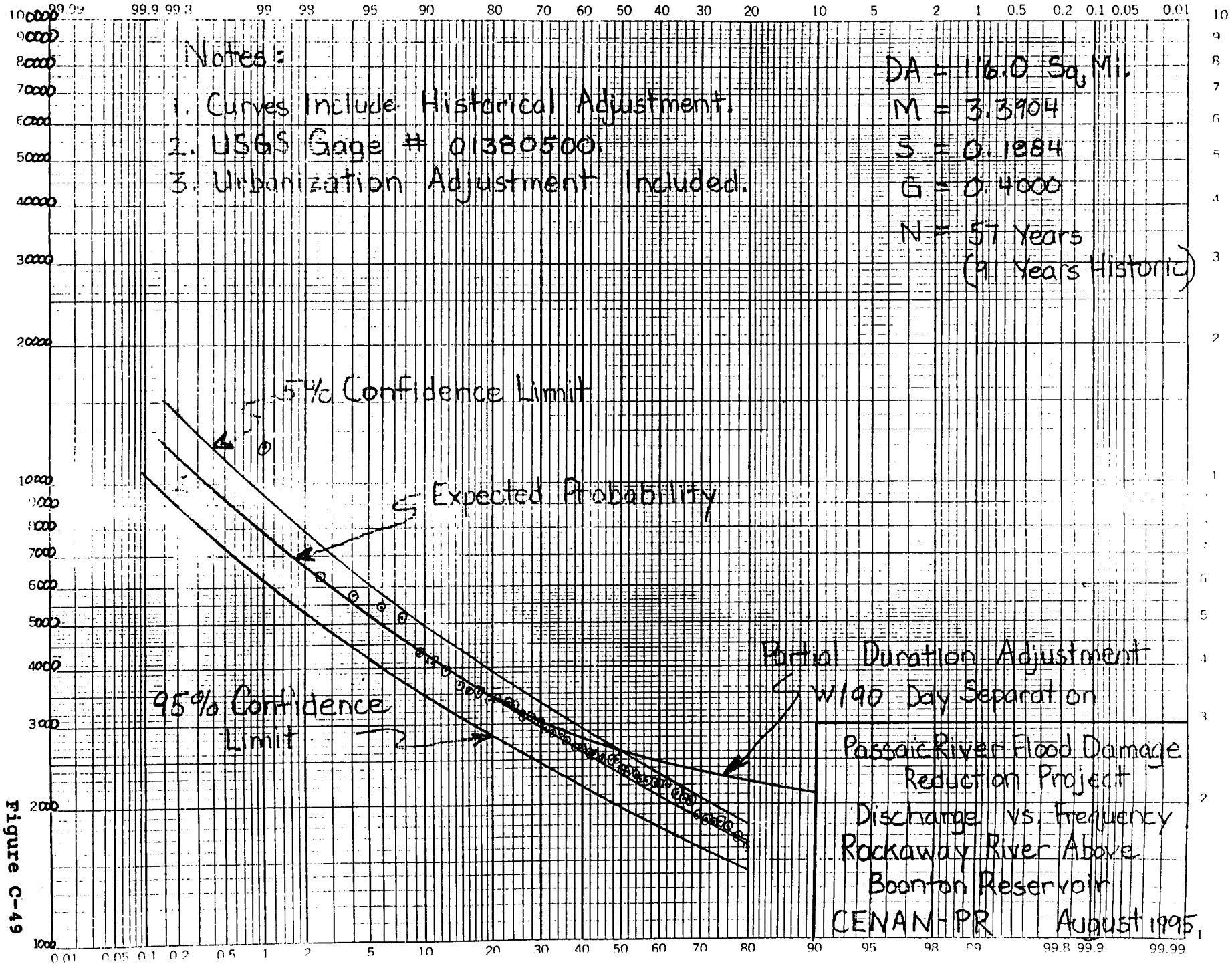

Notes:
1. Curves Include Historical Adjustment.
2. USGS Gage # 01380500.
3. Urbanization Adjustment Included.
DA = 116.0 Sq. Mi.
M = 3.3904
S = 0.1884
G = 0.4000
N = 57 Years
(9 Years Historic)
95% Confidence Limit
Expected Probability
95% Confidence Limit
Partial Duration Adjustment w/90 Day Separation
Passaic River Flood Damage Reduction Project
Discharge vs. Frequency
Rockaway River Above Boonton Reservoir
CENAN-PR August 1995

Figure C-49

K&E PROBABILITY X 2 LOG CYCLES
KEUFFEL & ESSER CO. MADE IN U.S.A.

46 8043

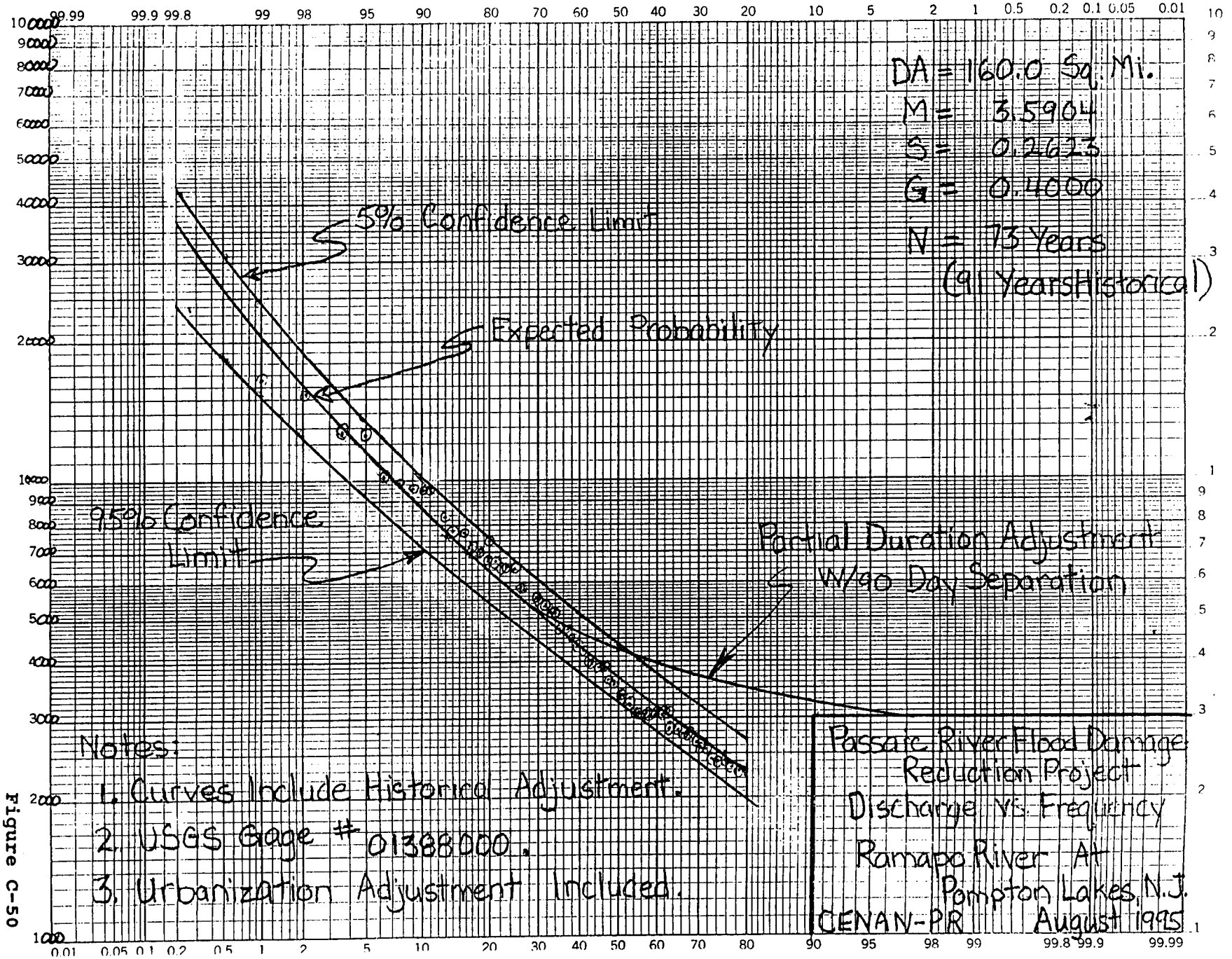


Figure C-50

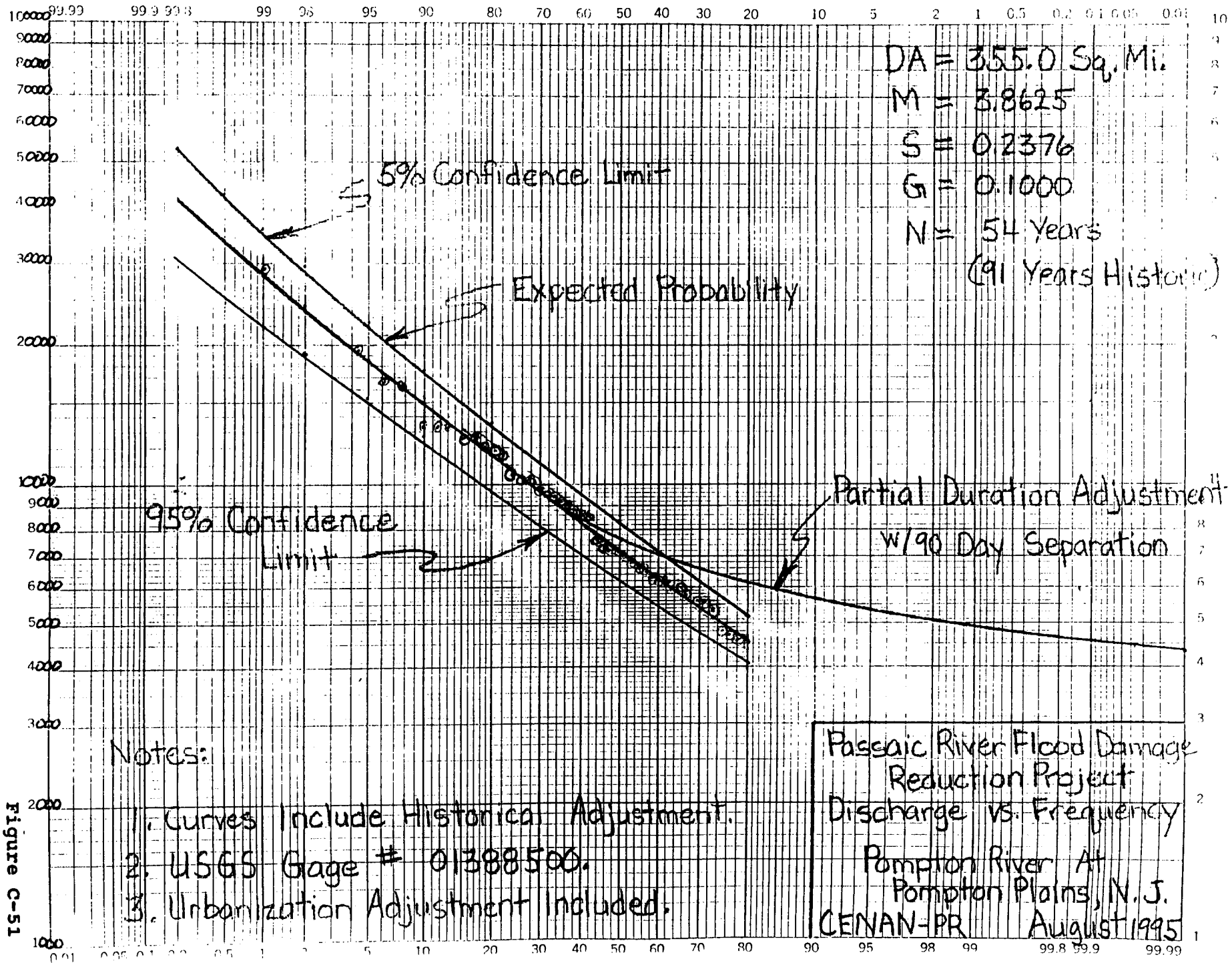

DA = 355.0 Sq. Mi.
M = 3.8625
S = 0.2376
G = 0.1000
N = 54 Years
(91 Years Historic)
5% Confidence Limit
Expected Probability
95% Confidence Limit
Partial Duration Adjustment w/90 Day Separation
Notes:
1. Curves Include Historical Adjustment.
2. USGS Gage # 01388500.
3. Urbanization Adjustment Included.
Passaic River Flood Damage Reduction Project
Discharge vs. Frequency
Pompton River At Pompton Plains, N.J.
CENAN-PR August 1995

Figure C-51

K•E PROBABILITY X 2 LOG CYCLES
KEUFFEL & ESSER CO. MADE IN U.S.A.

46 8043

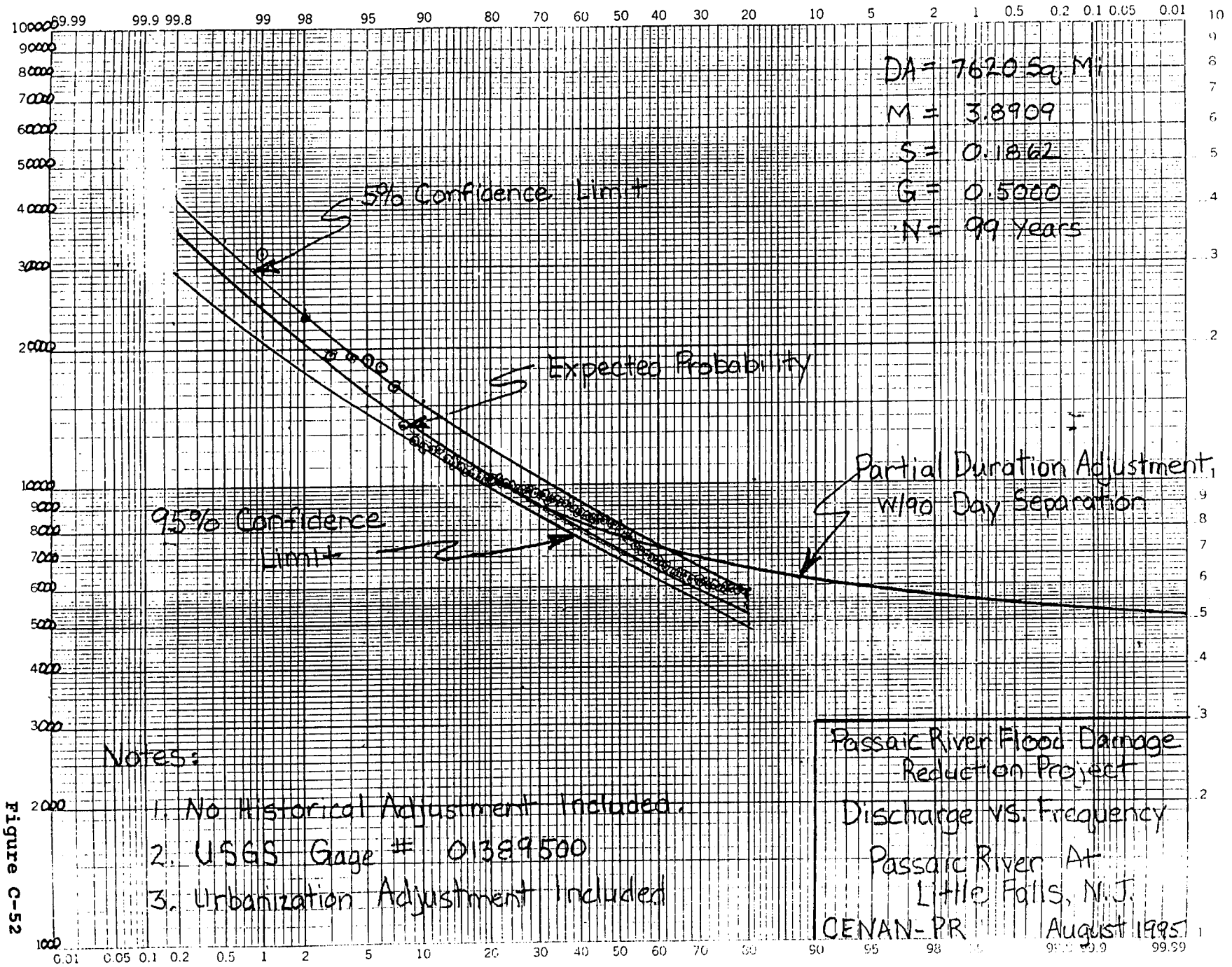


Figure C-52

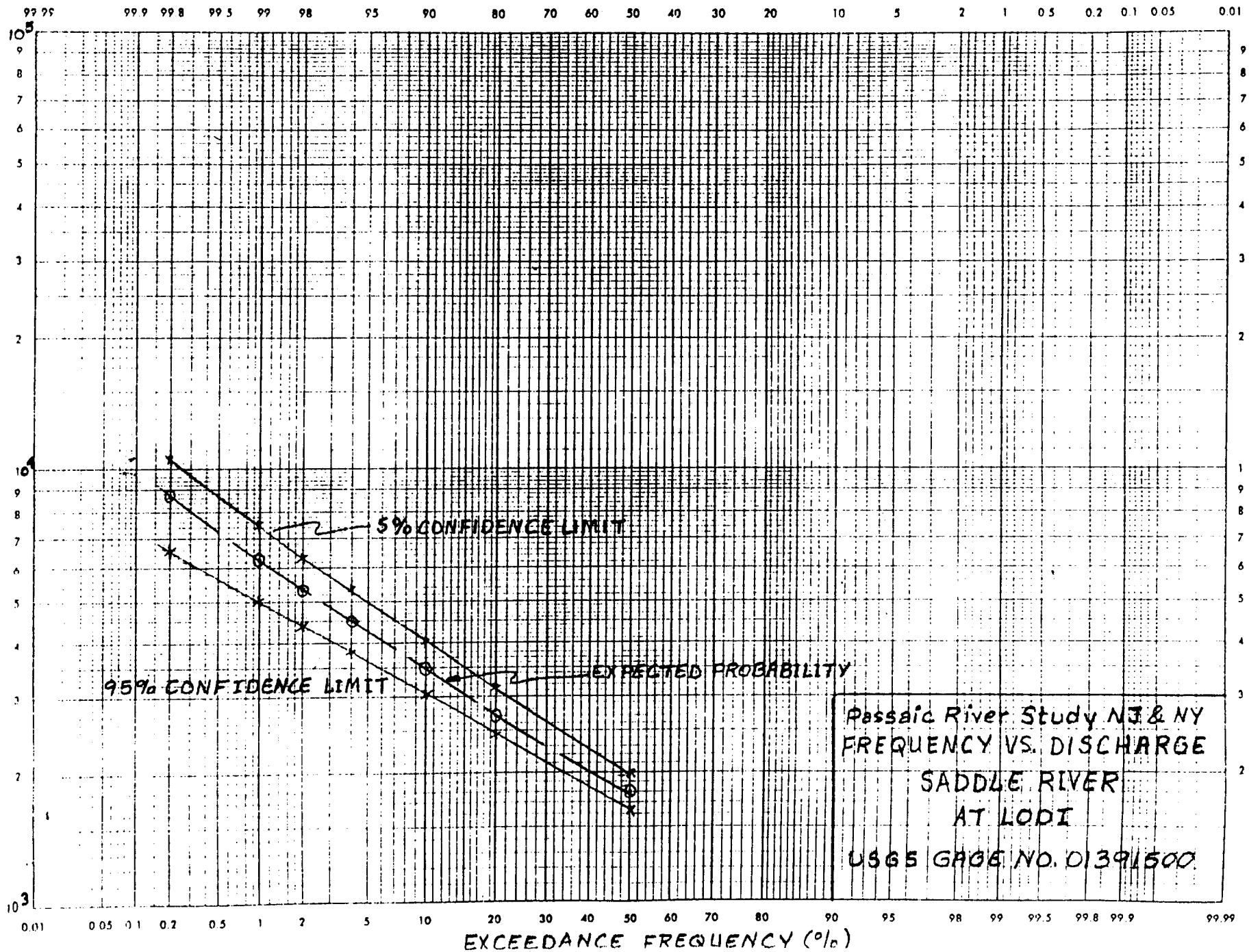
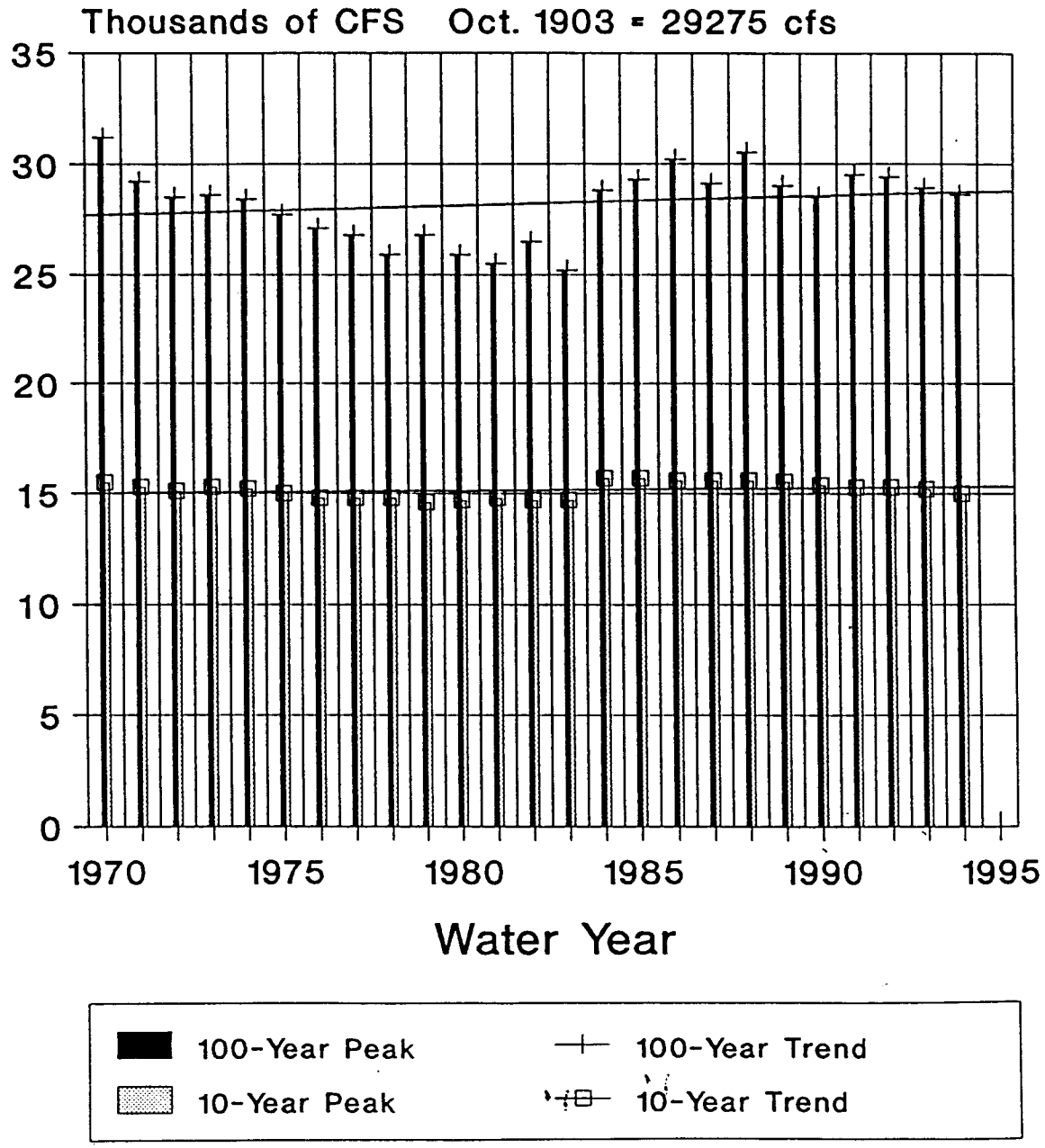

99.95 99.9 99.8 99.5 99 98 95 90 80 70 60 50 40 30 20 10 5 2 1 0.5 0.2 0.1 0.05 0.01
PEAK DISCHARGE (CFS)
5% CONFIDENCE LIMIT
95% CONFIDENCE LIMIT
EXPECTED PROBABILITY
Passaic River Study NJ & NY
FREQUENCY VS. DISCHARGE
SADDLE RIVER
AT LODI
USGS GAGE NO. 01391500
0.01 0.05 0.1 0.2 0.5 1 2 5 10 20 30 40 50 60 70 80 90 95 98 99 99.5 99.8 99.9 99.99
EXCEEDANCE FREQUENCY (%)

Figure C-53

100 Year at Pompton Plains
1940-1994 data - 8 May 1995



With Urbanization Adjustment 1940-1964

Figure C-54

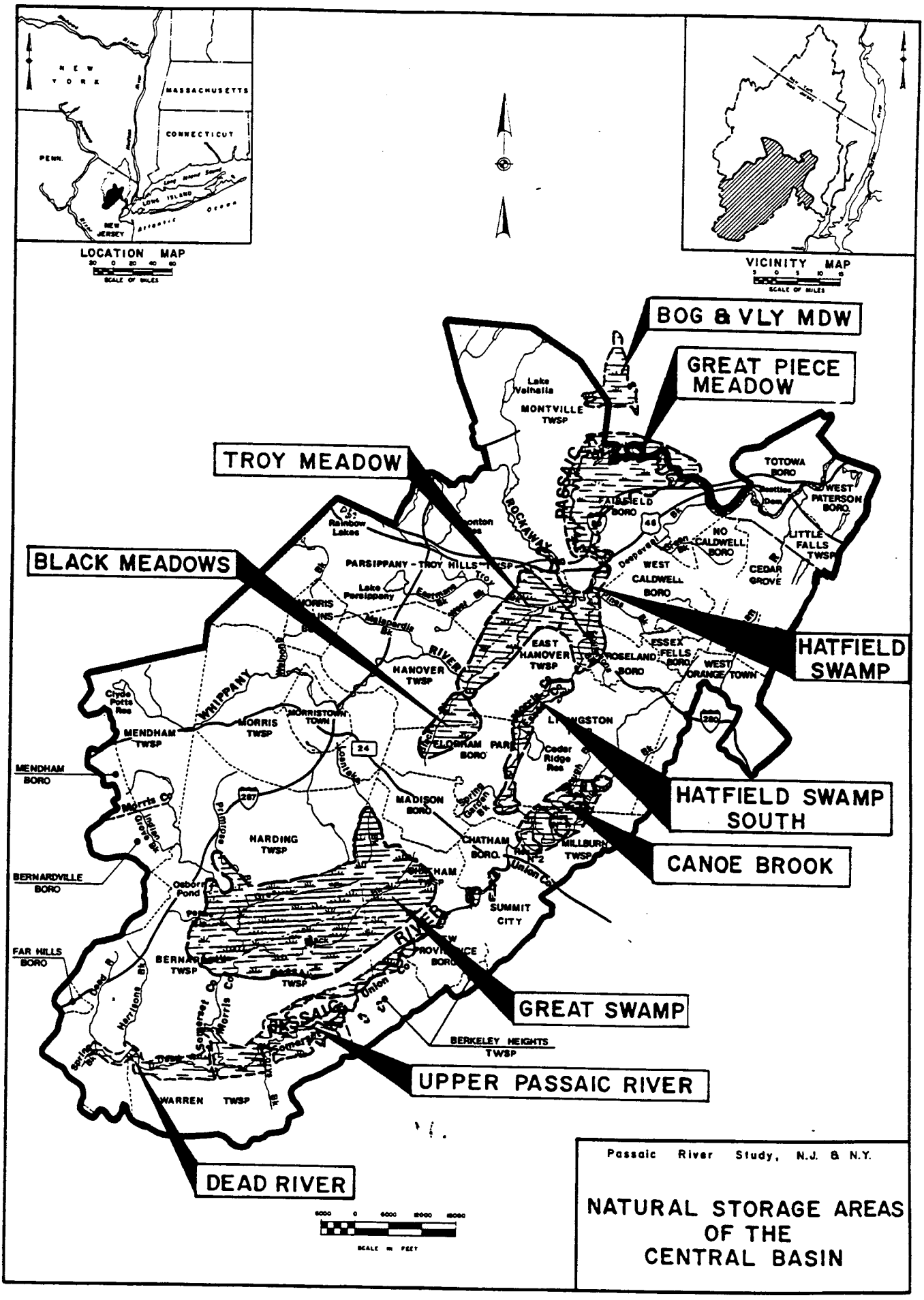

LOCATION MAP
VICINITY MAP
BOG & VLY MDW
GREAT PIECE MEADOW
TROY MEADOW
BLACK MEADOWS
HATFIELD SWAMP
HATFIELD SWAMP SOUTH
CANOE BROOK
GREAT SWAMP
UPPER PASSAIC RIVER
DEAD RIVER
NEW YORK
MASSACHUSETTS
CONNECTICUT
PENN.
NEW JERSEY
LONG ISLAND
Lake Valhalla
MONTVILLE TWSP
TOTOWA BORO
WEST PATERSON BORO
FAIRFIELD BORO
NO CALDWELL BORO
LITTLE FALLS TWSP
WEST CALDWELL BORO
CEDAR GROVE
PARSIPPANY - TROY HILLS TWSP
Lake Parsippany
MORRIS PLAINS
HANOVER TWSP
EAST HANOVER TWSP
ESSEX FELLS BORO
ROSELAND BORO
WEST ORANGE TOWN
LIVINGSTON
FLORHAM PARK
MENDHAM TWSP
MORRISTOWN TOWN
MORRIS TWSP
MENDHAM BORO
BERNARDSVILLE BORO
HARDING TWSP
MADISON BORO
CHATHAM BORO
CHATHAM TWSP
MILLBURN TWSP
SUMMIT CITY
FAR HILLS BORO
BERNARDS TWSP
BERKELEY HEIGHTS TWSP
WARREN TWSP
Passaic River Study, N.J. & N.Y.
NATURAL STORAGE AREAS OF THE CENTRAL BASIN

Figure C-55

08MAY95 08:36:20

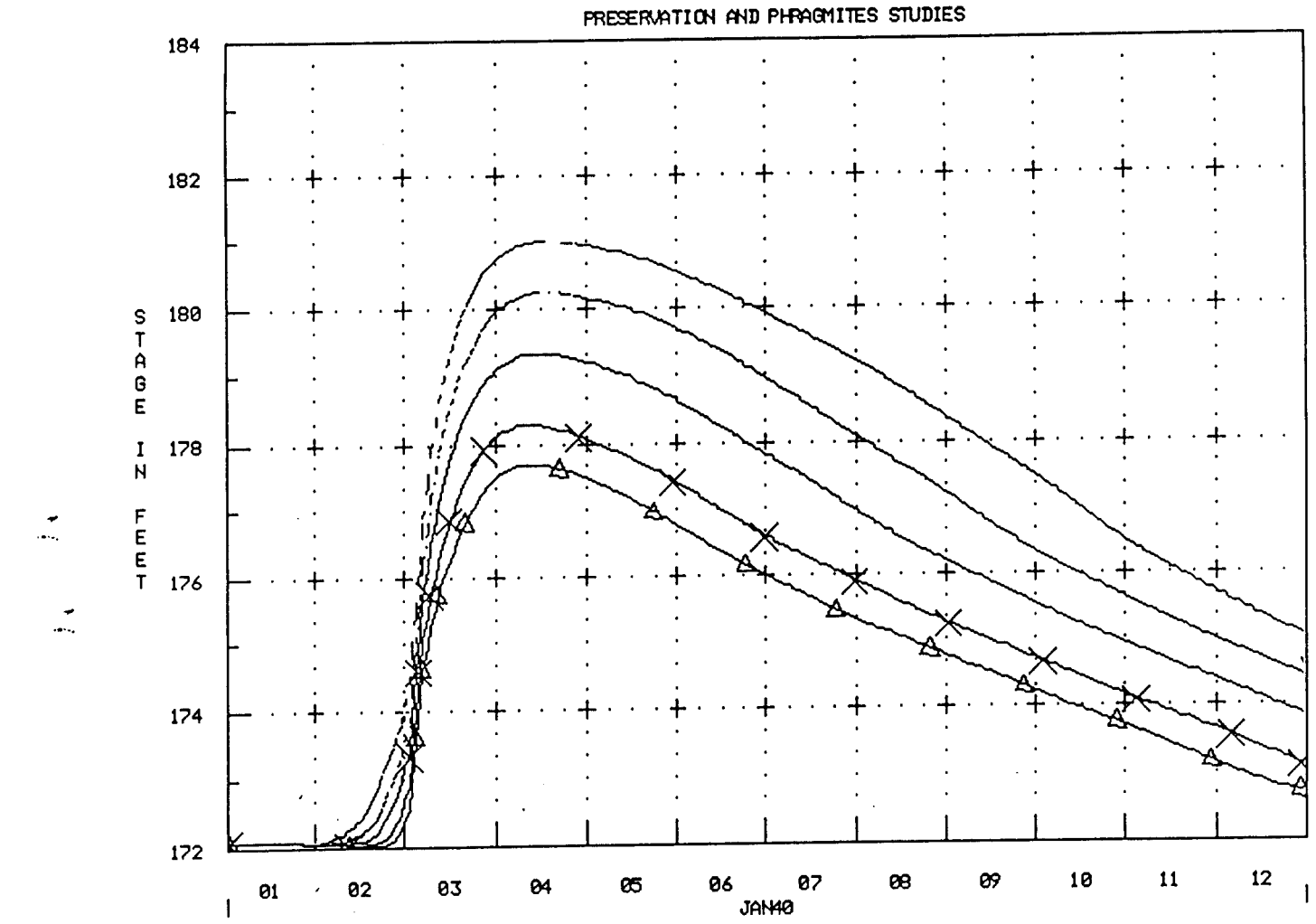


Figure C-56a

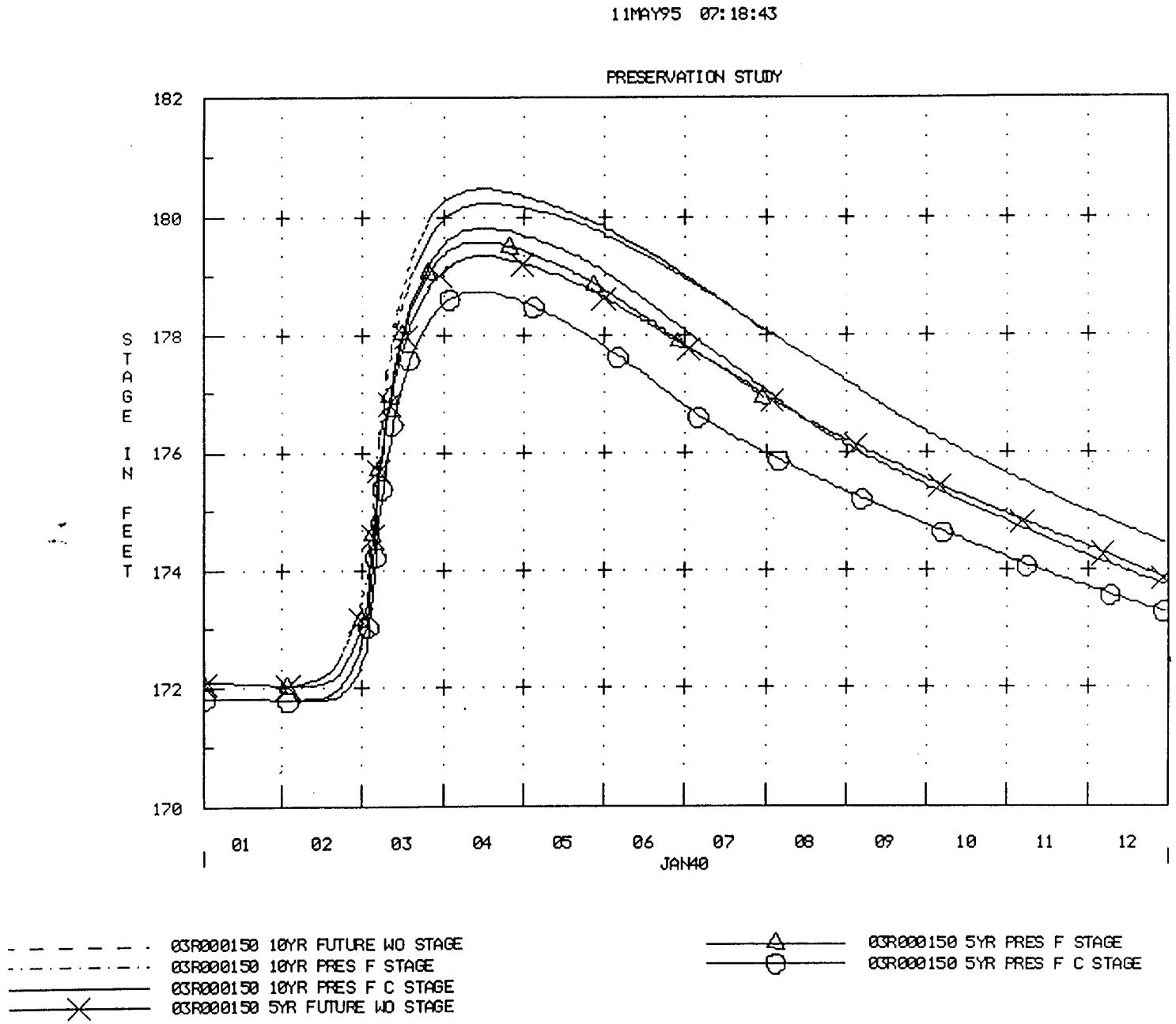

11MAY95 07:18:43
PRESERVATION STUDY
182
180
178
176
174
172
170
STAGE IN FEET
01
02
03
04
05
06
07
08
09
10
11
12
JAN40
03R000150 10YR FUTURE WO STAGE
03R000150 10YR PRES F STAGE
03R000150 10YR PRES F C STAGE
03R000150 5YR FUTURE WO STAGE
03R000150 5YR PRES F STAGE
03R000150 5YR PRES F C STAGE

Figure C-56b

11MAY95 07:30:22

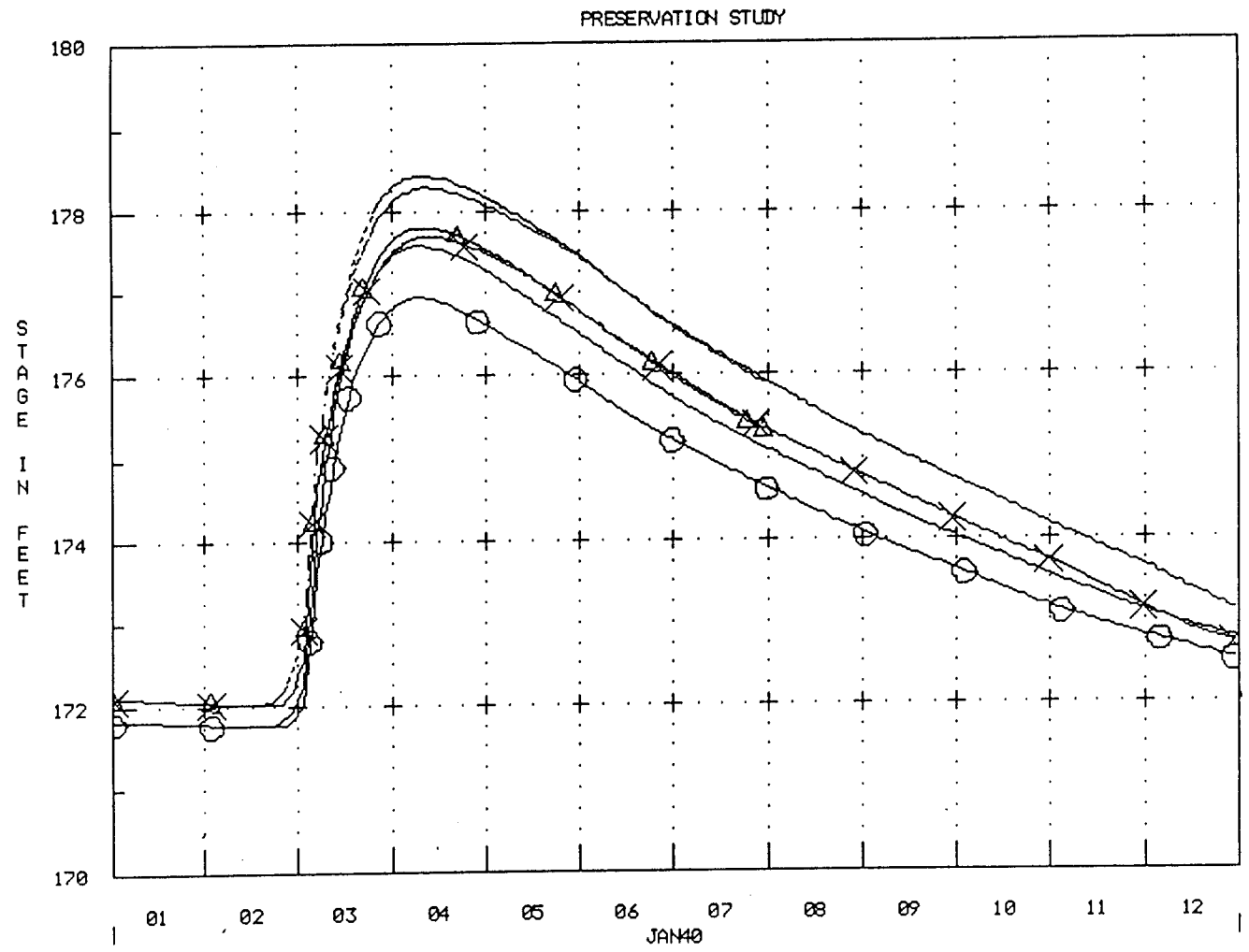


Figure C-56c

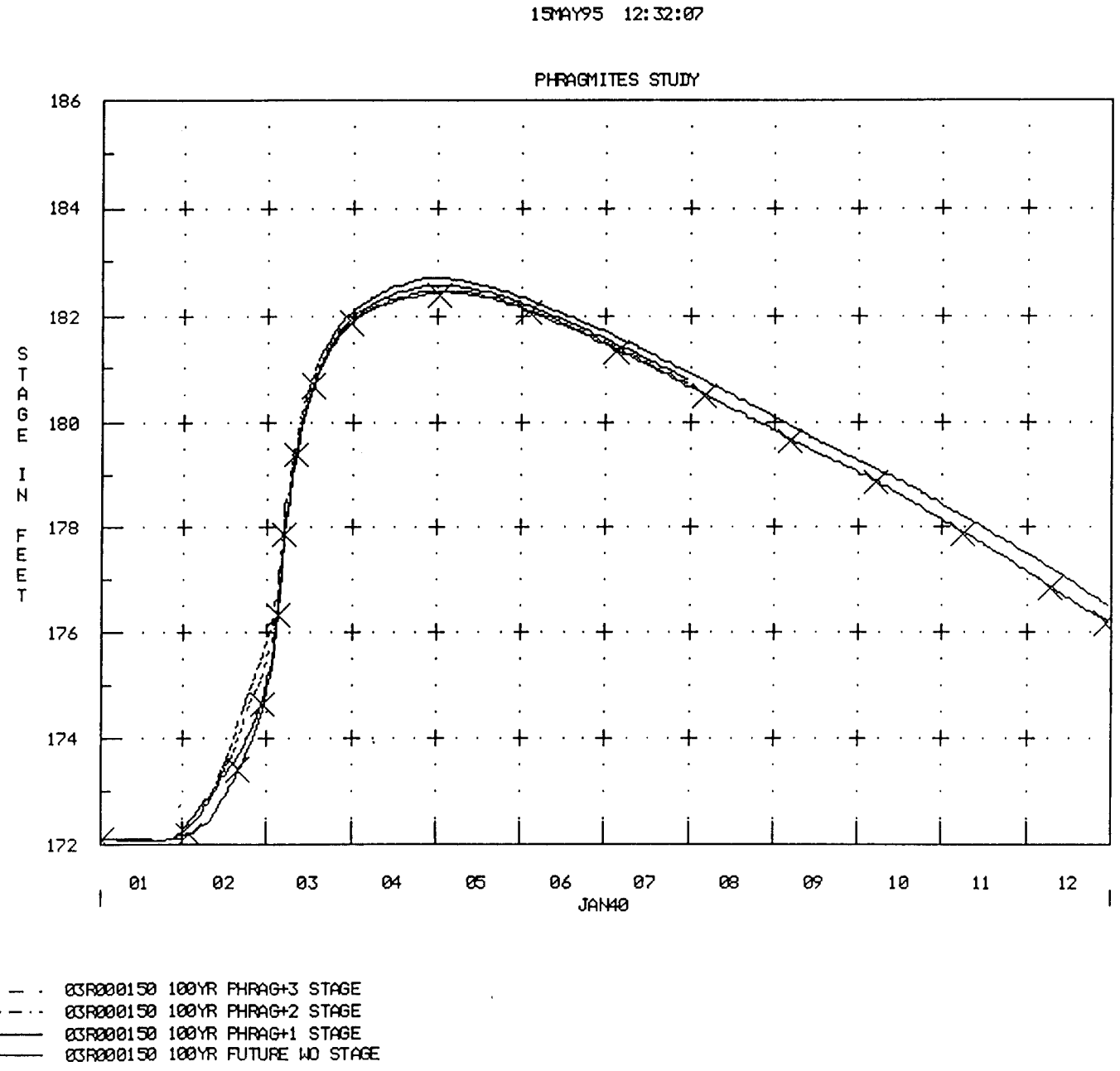

15MAY95 12:32:07
PHRAGMITES STUDY
STAGE IN FEET
186
184
182
180
178
176
174
172
01
02
03
04
05
06
07
08
09
10
11
12
JAN40
03R000150 100YR PHRAG+3 STAGE
03R000150 100YR PHRAG+2 STAGE
03R000150 100YR PHRAG+1 STAGE
03R000150 100YR FUTURE WO STAGE

Figure C-56d

15MAY95 12:48:56

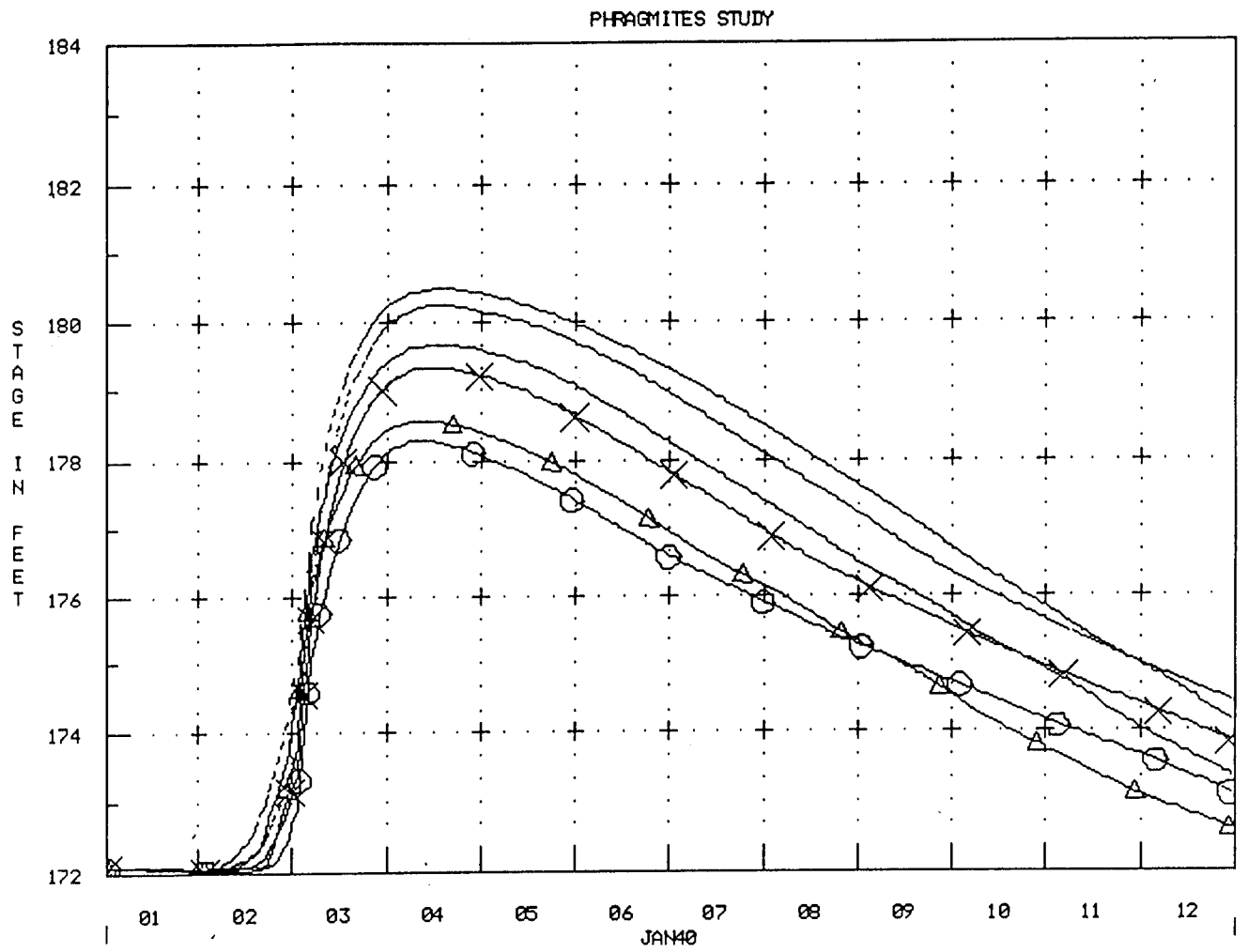


Figure C-56e

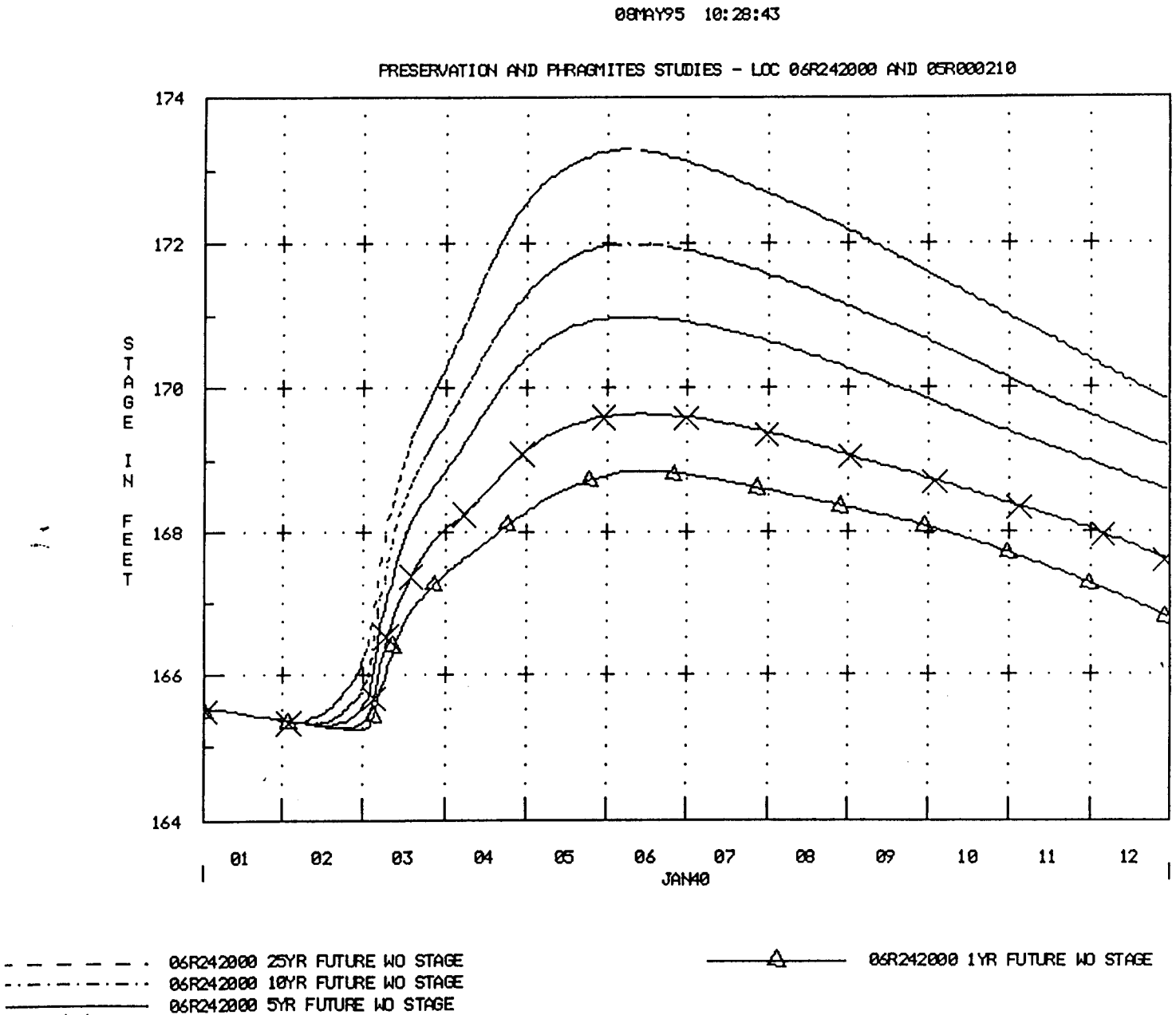

08MAY95 10:28:43
PRESERVATION AND PHRAGMITES STUDIES - LOC 06R242000 AND 05R000210
STAGE IN FEET
174
172
170
168
166
164
01
02
03
04
05
06
07
08
09
10
11
12
JAN90
06R242000 25YR FUTURE WO STAGE
06R242000 10YR FUTURE WO STAGE
06R242000 5YR FUTURE WO STAGE
06R242000 2YR FUTURE WO STAGE
06R242000 1YR FUTURE WO STAGE

Figure C-57a

16MAY95 13:58:00

PRESERVATION STUDY - THESE CURVES ALSO APPLY TO LOC 05R000210

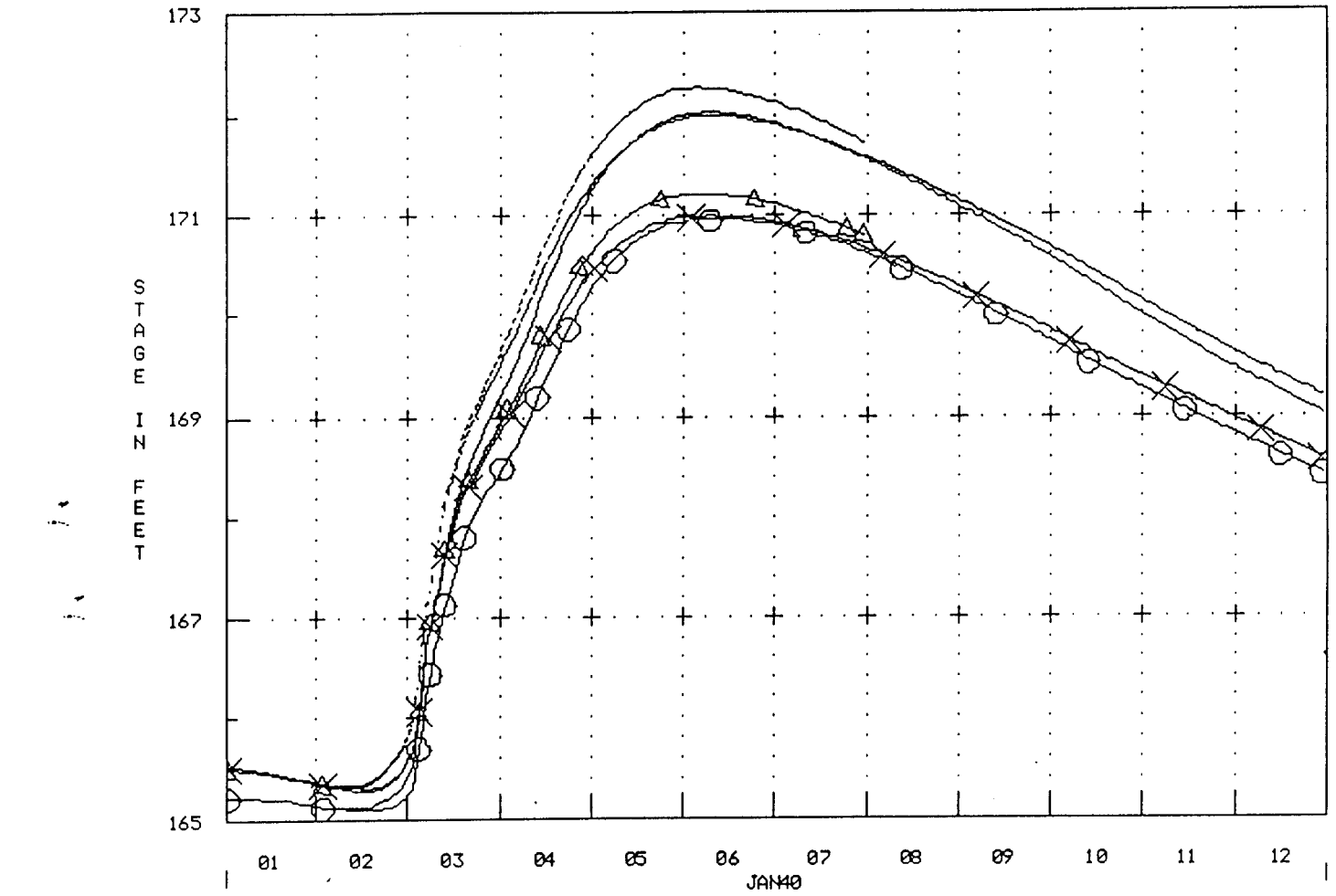


Figure C-57b

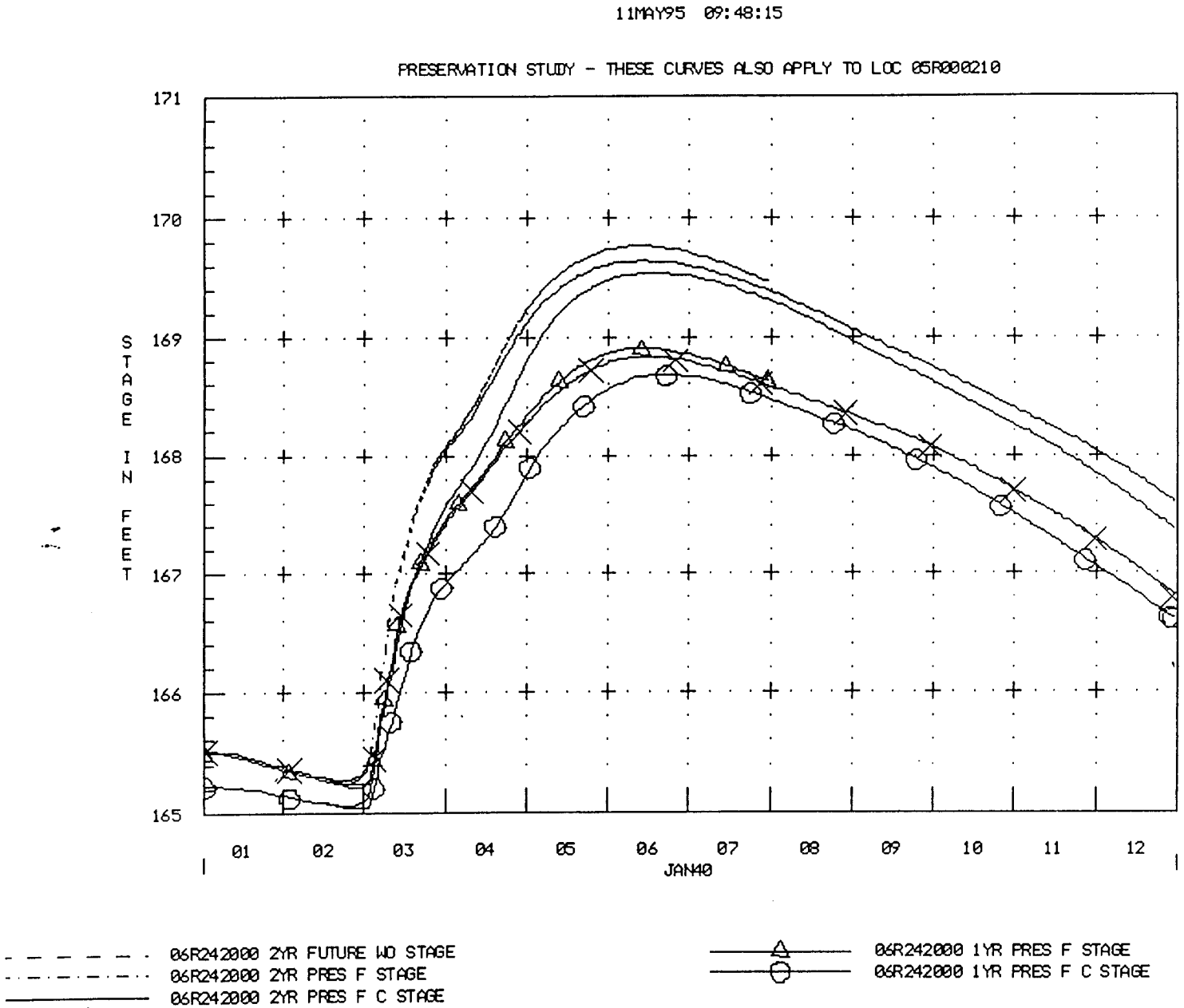

11MAY95 09:48:15
PRESERVATION STUDY - THESE CURVES ALSO APPLY TO LOC 05R000210
STAGE IN FEET
171
170
169
168
167
166
165
01
02
03
04
05
06
07
08
09
10
11
12
JAN40
06R242000 2YR FUTURE WO STAGE
06R242000 2YR PRES F STAGE
06R242000 2YR PRES F C STAGE
06R242000 1YR FUTURE WO STAGE
06R242000 1YR PRES F STAGE
06R242000 1YR PRES F C STAGE

Figure C-57c

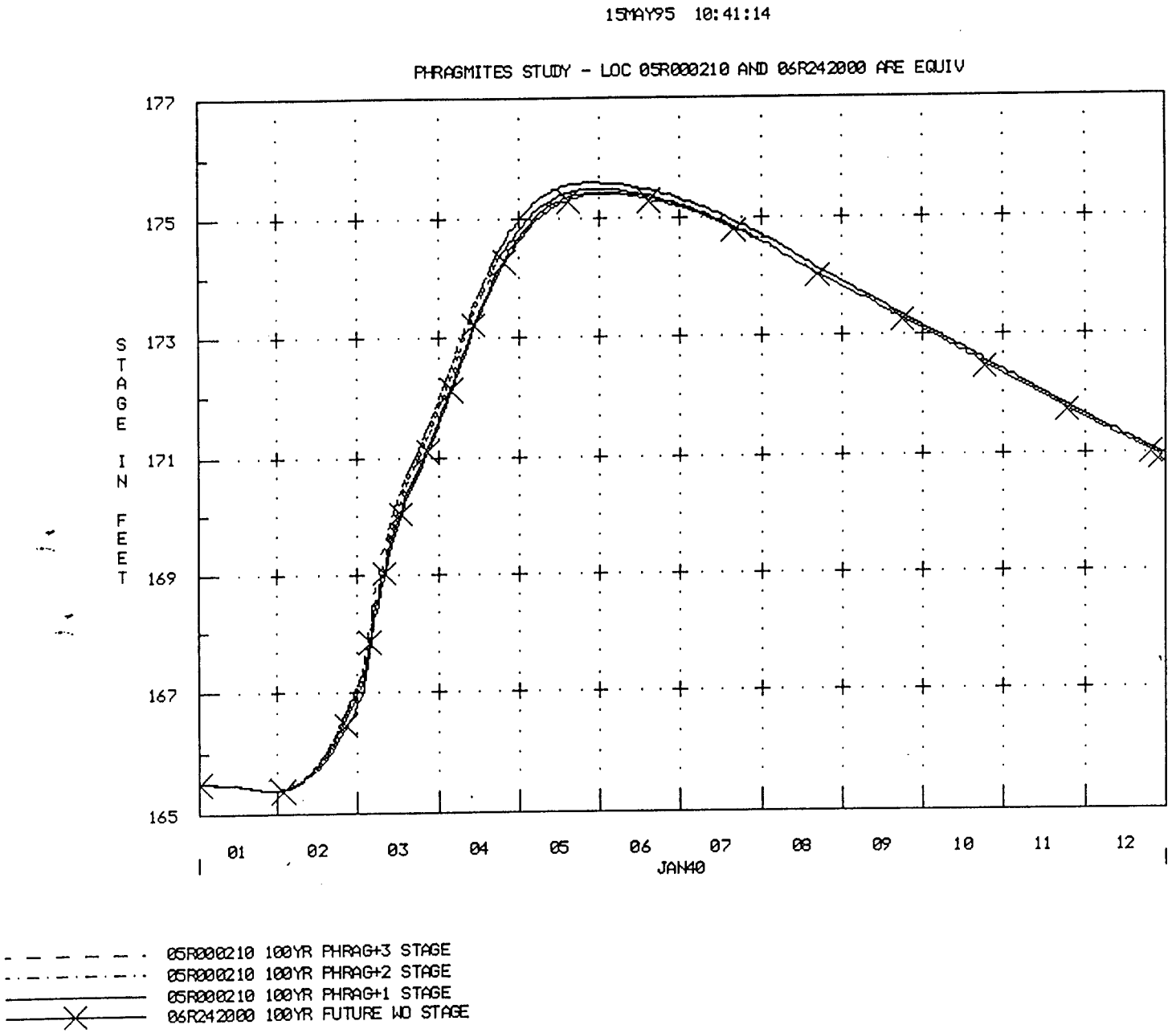

15MAY95 10:41:14
PHRAGMITES STUDY - LOC 05R000210 AND 06R242000 ARE EQUIV
177
175
173
171
169
167
165
STAGE IN FEET
01
02
03
04
05
06
07
08
09
10
11
12
JAN40
05R000210 100YR PHRAG+3 STAGE
05R000210 100YR PHRAG+2 STAGE
05R000210 100YR PHRAG+1 STAGE
06R242000 100YR FUTURE WO STAGE

Figure C-57d

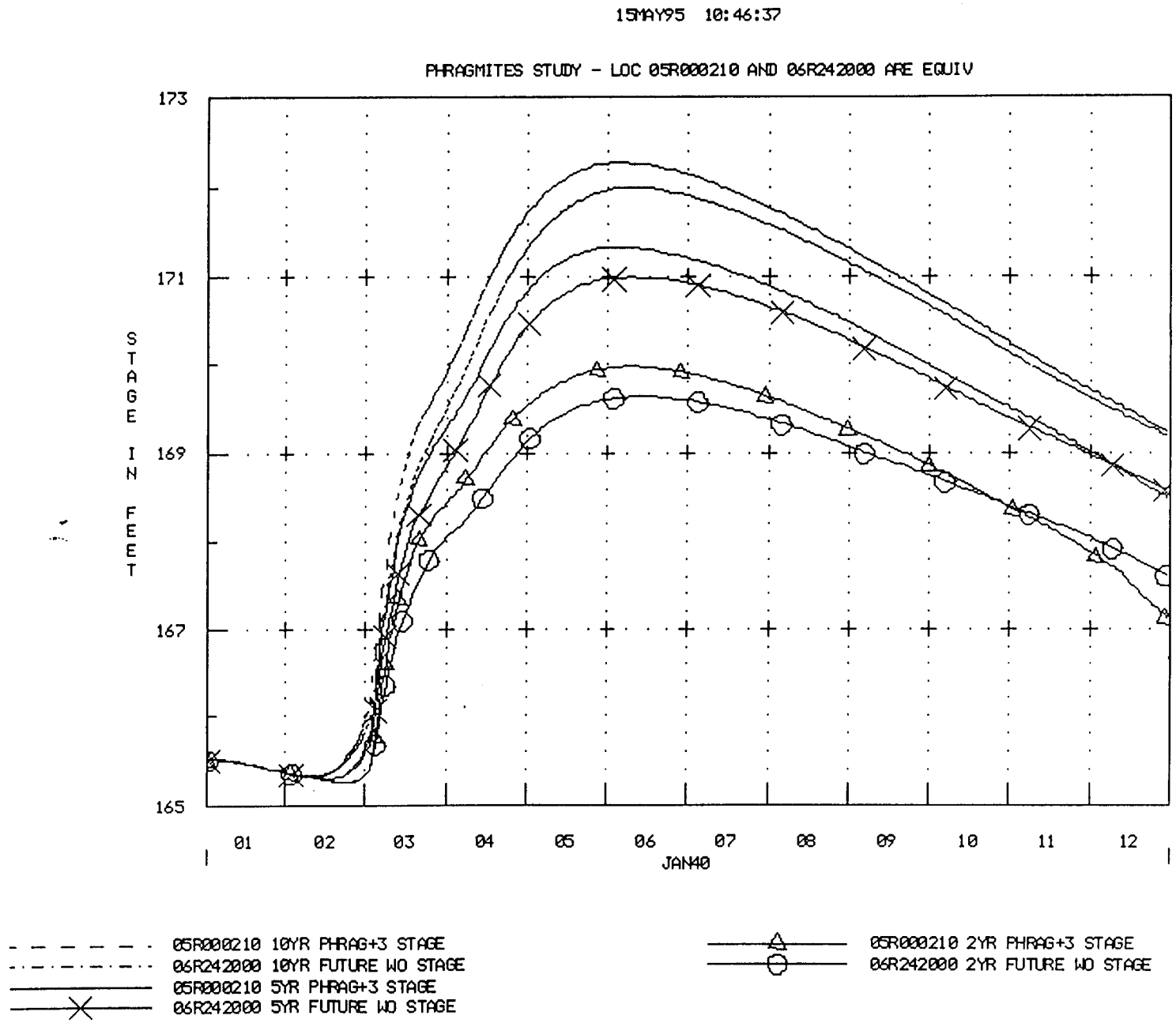
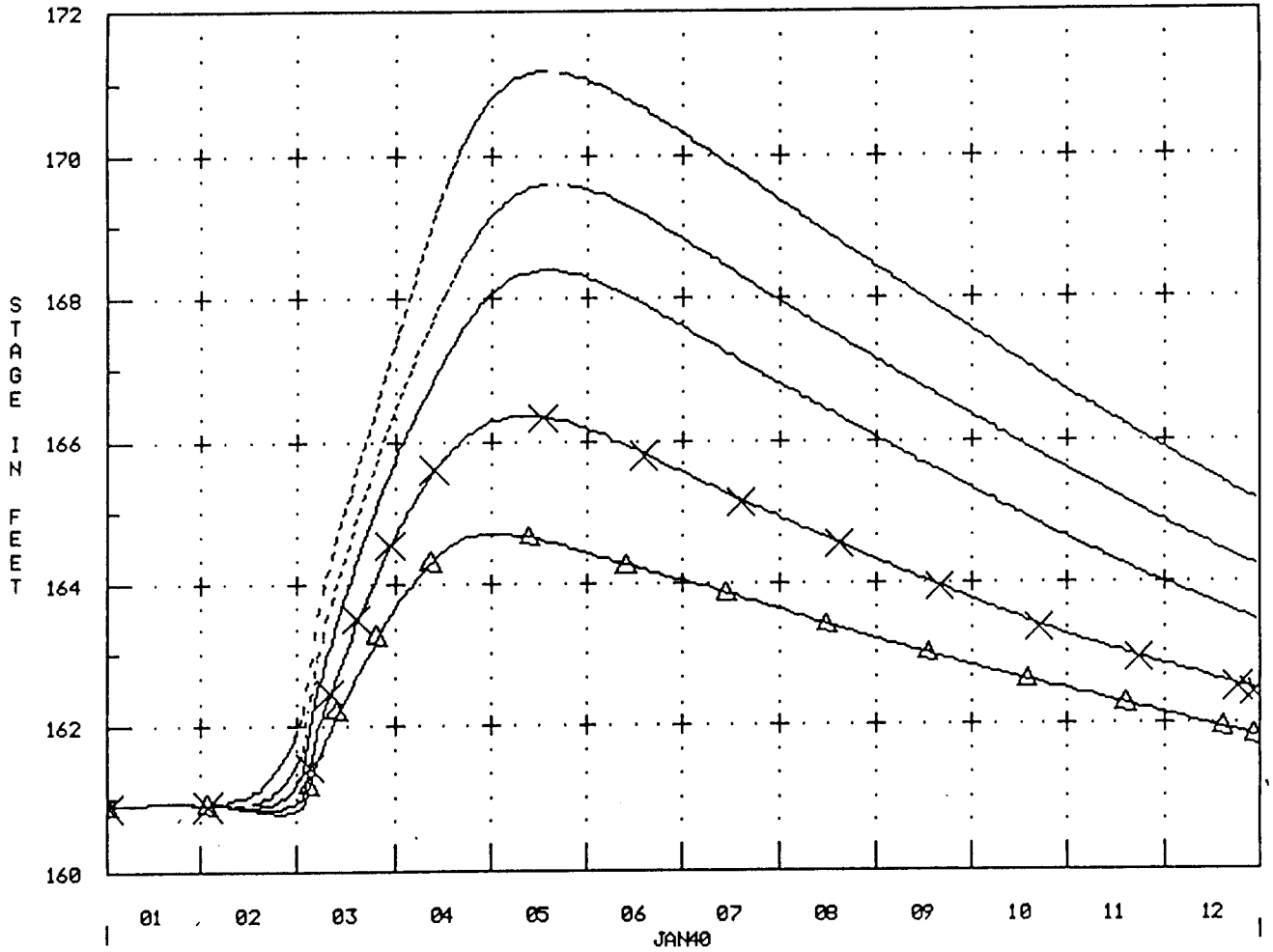

15MAY95 10:46:37
PHRAGMITES STUDY - LOC 05R000210 AND 06R242000 ARE EQUIV
173
171
169
167
165
STAGE IN FEET
01
02
03
04
05
06
07
08
09
10
11
12
JAN40
05R000210 10YR PHRAG+3 STAGE
06R242000 10YR FUTURE WO STAGE
05R000210 5YR PHRAG+3 STAGE
06R242000 5YR FUTURE WO STAGE
05R000210 2YR PHRAG+3 STAGE
06R242000 2YR FUTURE WO STAGE

Figure C-57e

08MAY95 10:31:22

PRESERVATION AND PHRAGMITES STUDIES - LOC 15R173500 AND 14R000222



15R173500 25YR FUTURE WO STAGE
15R173500 10YR FUTURE WO STAGE
15R173500 5YR FUTURE WO STAGE
15R173500 2YR FUTURE WO STAGE
15R173500 1YR FUTURE WO STAGE

Figure C-58a

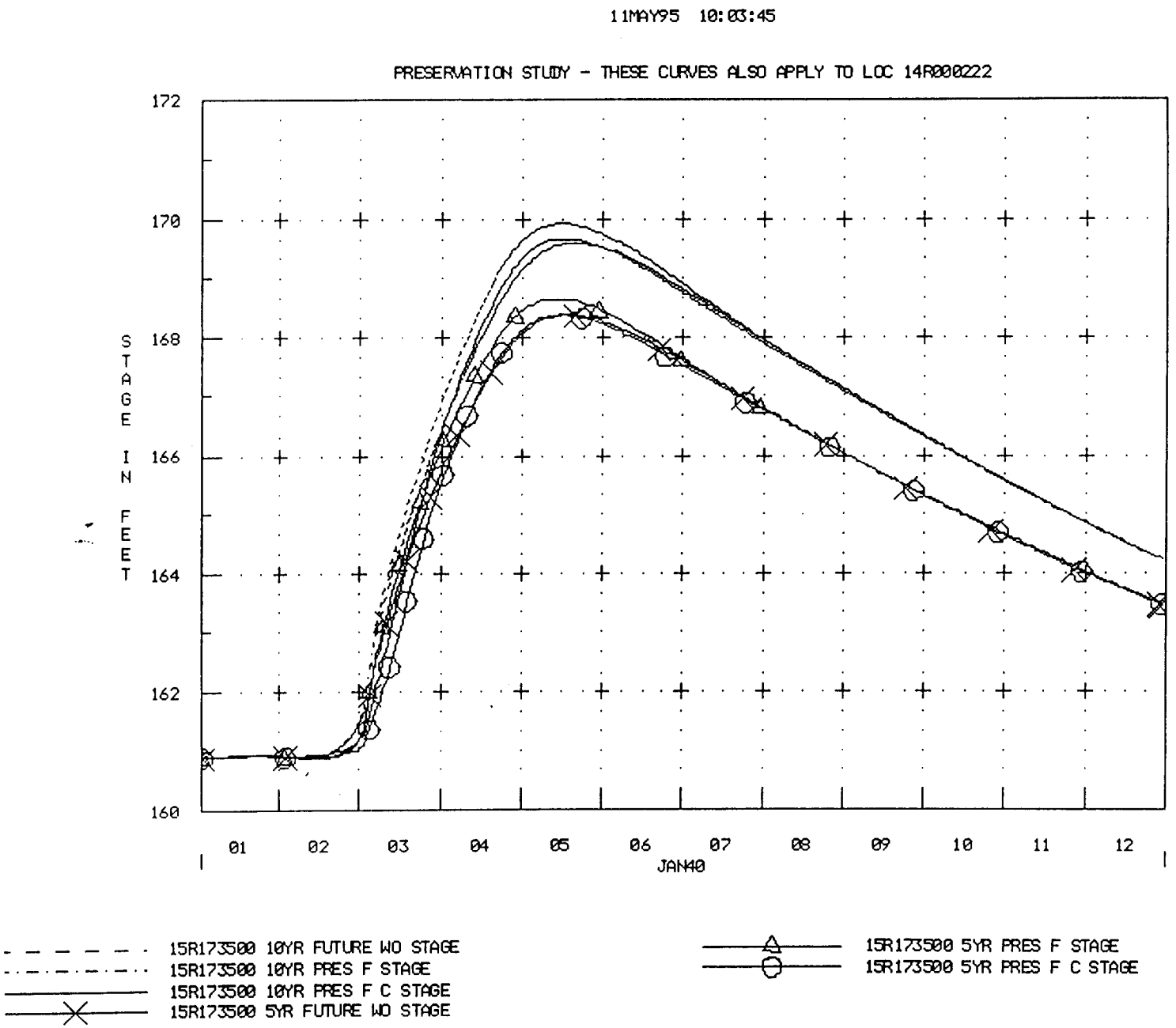
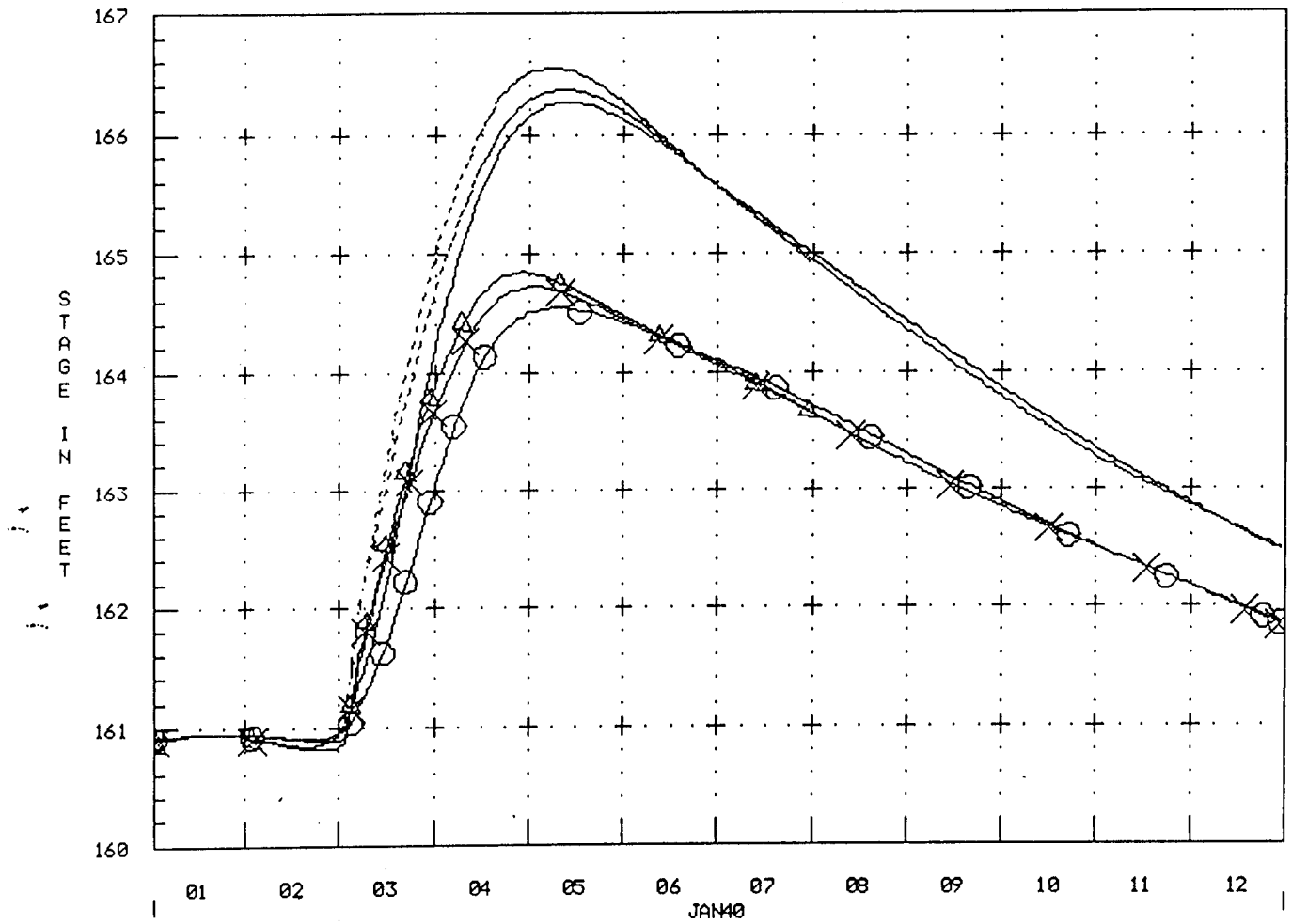

11MAY95 10:03:45
PRESERVATION STUDY - THESE CURVES ALSO APPLY TO LOC 14R000222
STAGE IN FEET
172
170
168
166
164
162
160
01
02
03
04
05
06
07
08
09
10
11
12
JAN40
15R173500 10YR FUTURE WO STAGE
15R173500 10YR PRES F STAGE
15R173500 10YR PRES F C STAGE
15R173500 5YR FUTURE WO STAGE
15R173500 5YR PRES F STAGE
15R173500 5YR PRES F C STAGE

Figure C-58b

11MAY95 10:15:57

PRESERVATION STUDY - THESE CURVES ALSO APPLY TO LOC 14R000222



15R173500 2YR FUTURE WO STAGE
15R173500 2YR PRES F STAGE
15R173500 2YR PRES F C STAGE
15R173500 1YR FUTURE WO STAGE
15R173500 1YR PRES F STAGE
15R173500 1YR PRES F C STAGE

Figure C-58c

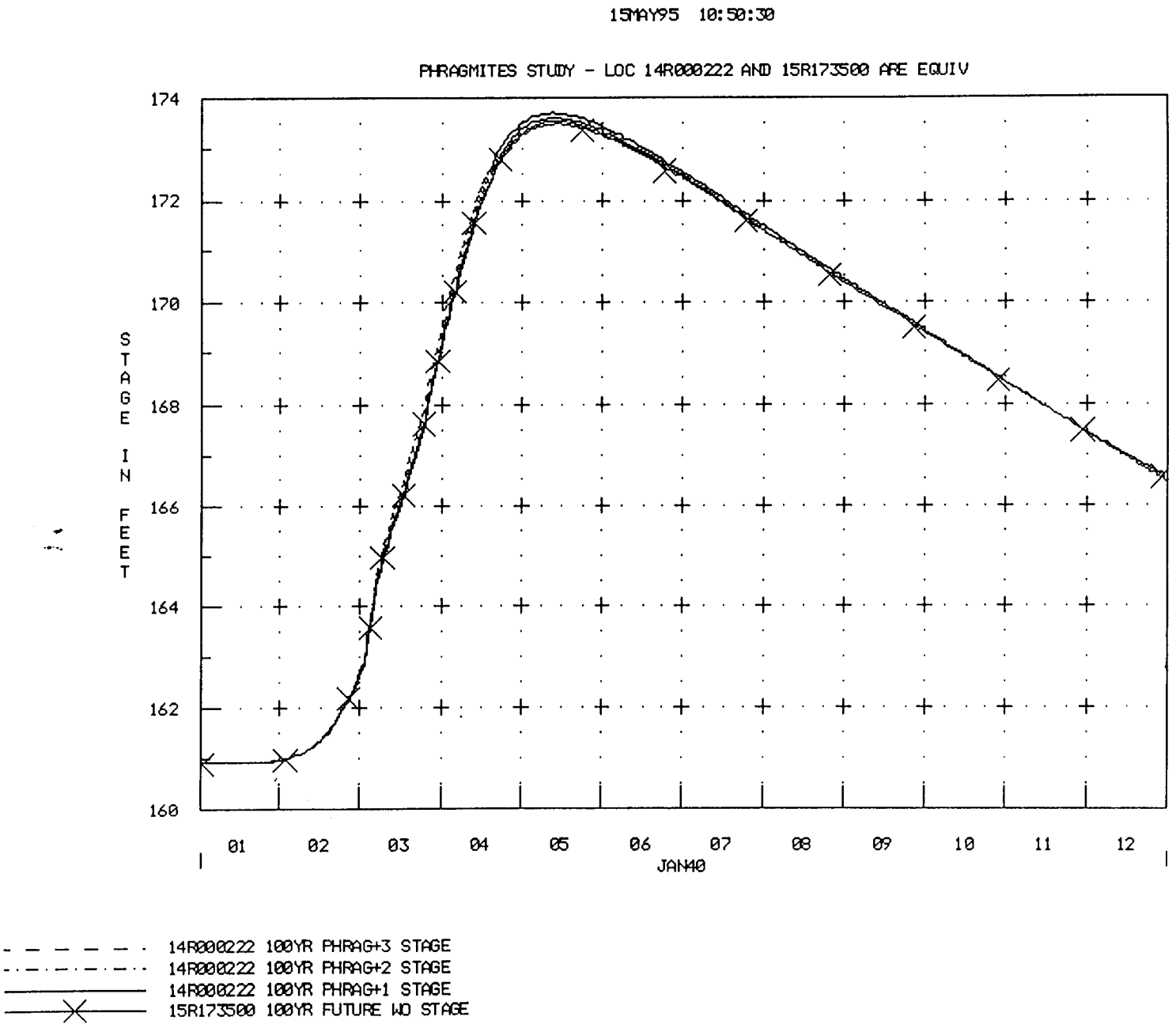

15MAY95 10:50:30
PHRAGMITES STUDY - LOC 14R000222 AND 15R173500 ARE EQUIV
174
172
170
168
166
164
162
160
STAGE IN FEET
01
02
03
04
05
06
07
08
09
10
11
12
JAN40
14R000222 100YR PHRAG+3 STAGE
14R000222 100YR PHRAG+2 STAGE
14R000222 100YR PHRAG+1 STAGE
15R173500 100YR FUTURE WO STAGE

Figure C-58d

15MAY95 10:55:43

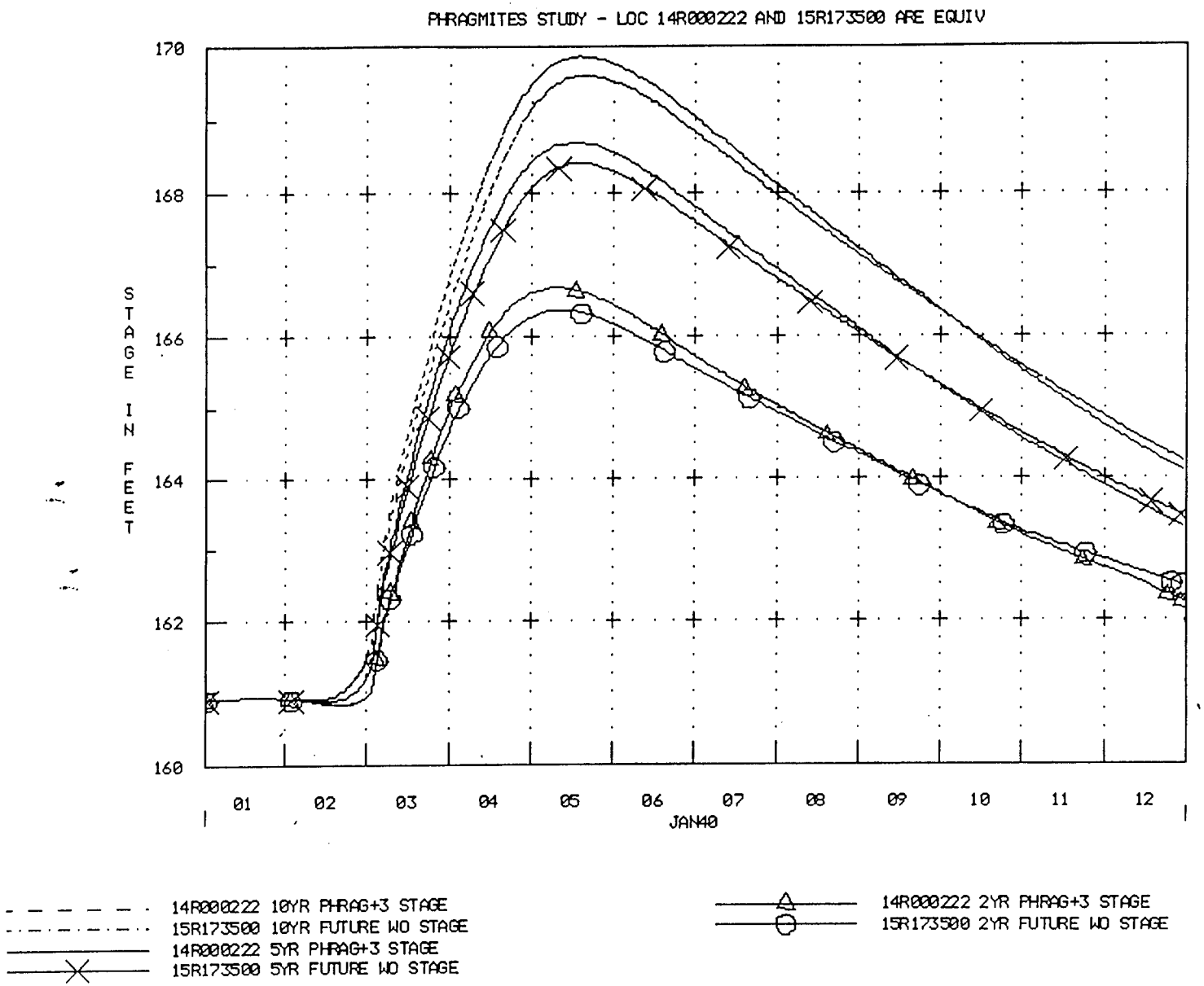


Figure C-58e

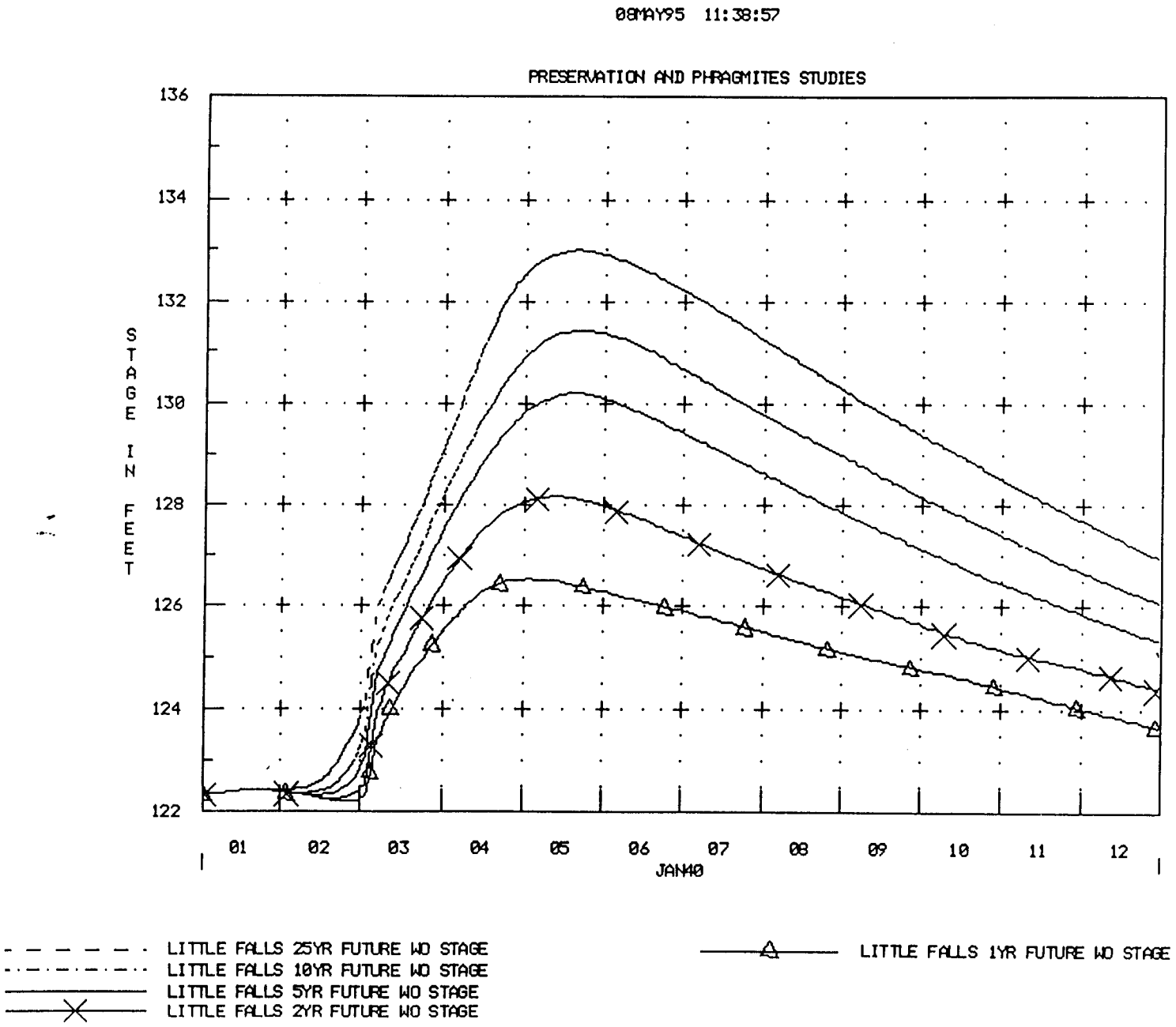

08MAY95 11:38:57
PRESERVATION AND PHRAGMITES STUDIES
136
134
132
130
128
126
124
122
STAGE IN FEET
01
02
03
04
05
06
07
08
09
10
11
12
JAN40
LITTLE FALLS 25YR FUTURE WO STAGE
LITTLE FALLS 10YR FUTURE WO STAGE
LITTLE FALLS 5YR FUTURE WO STAGE
LITTLE FALLS 2YR FUTURE WO STAGE
LITTLE FALLS 1YR FUTURE WO STAGE

Figure C-59a

15MAY95 09:38:15

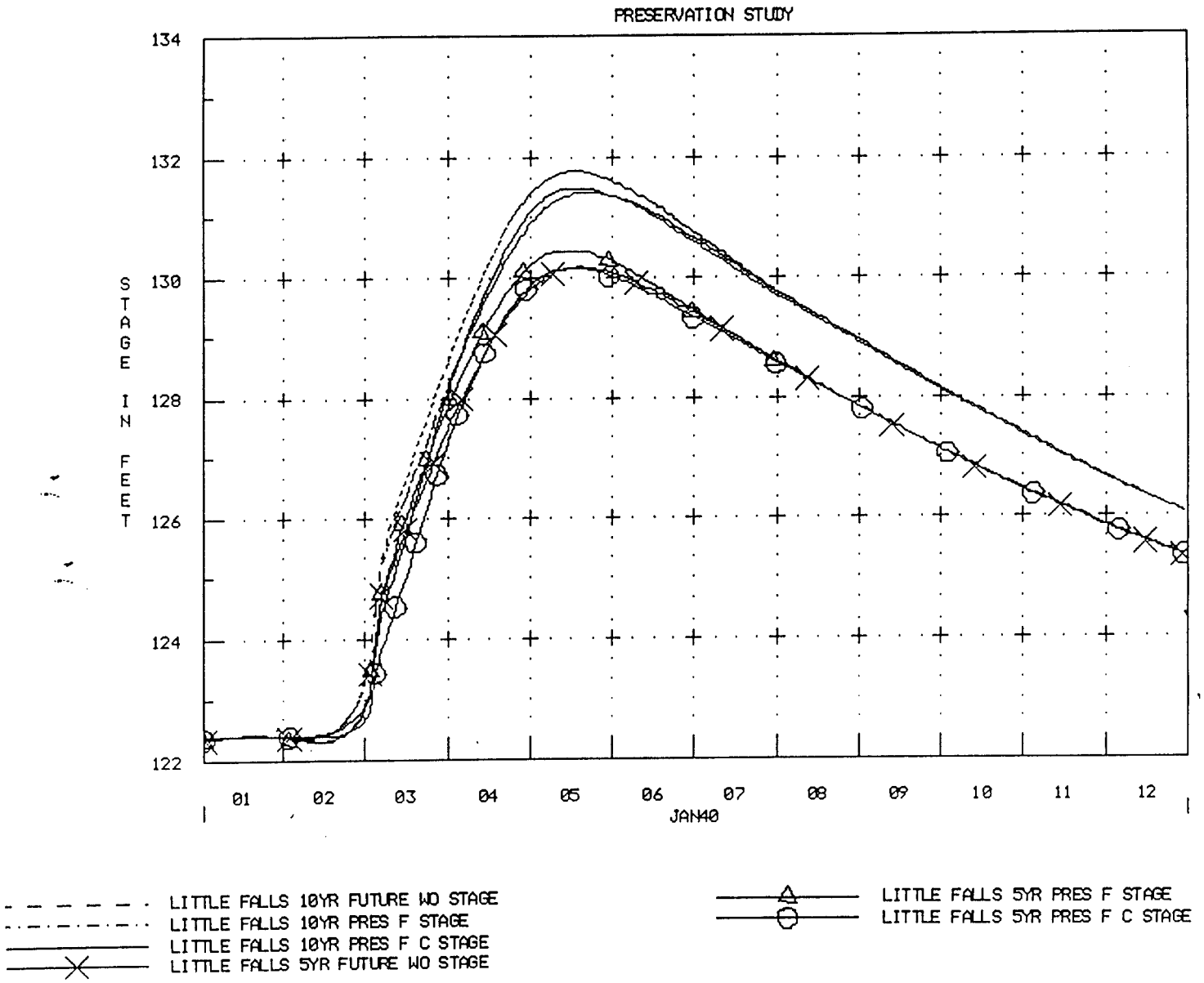


Figure C-59b

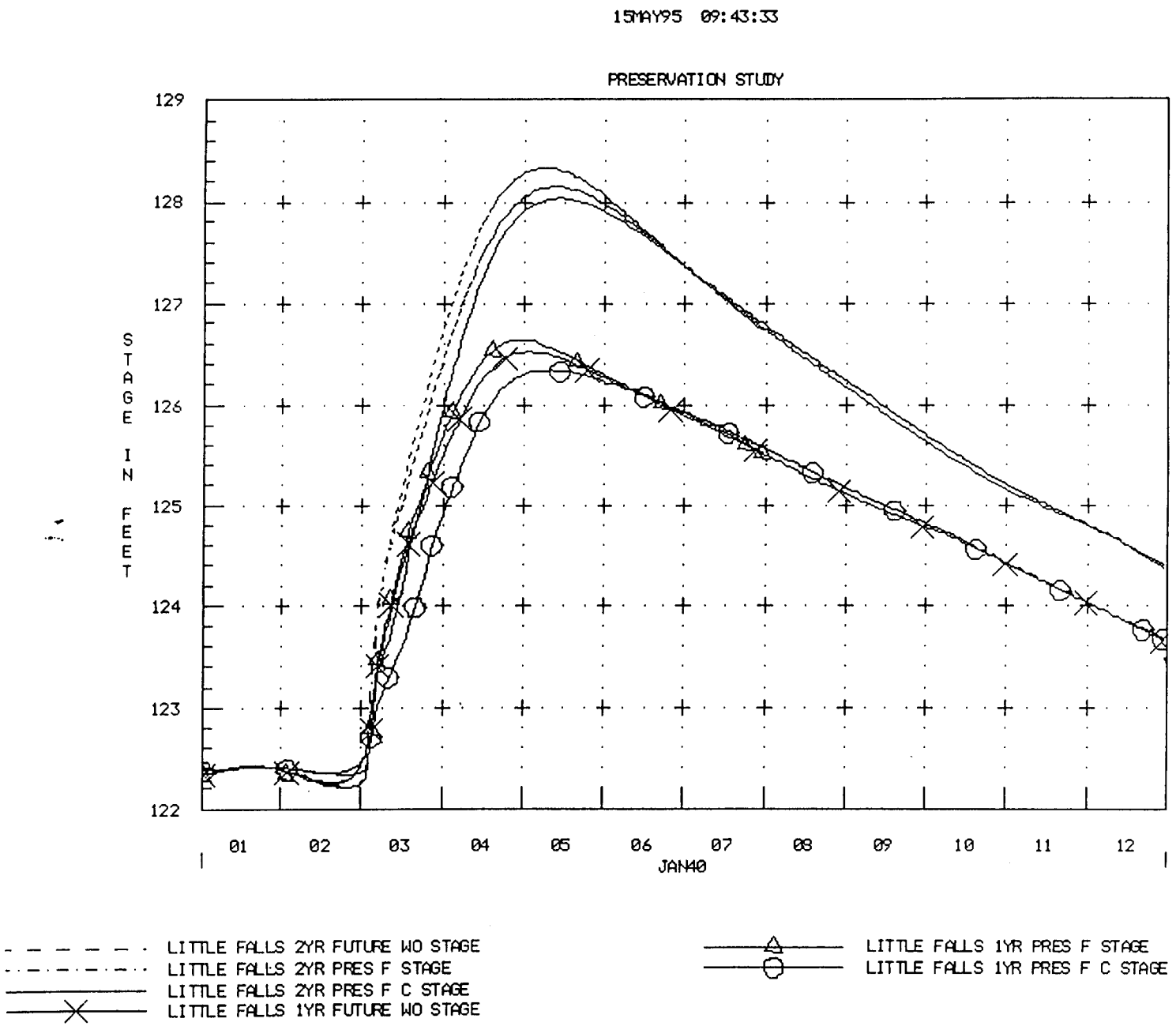

15MAY95 09:43:33
PRESERVATION STUDY
STAGE IN FEET
129
128
127
126
125
124
123
122
01
02
03
04
05
06
07
08
09
10
11
12
JAN40
LITTLE FALLS 2YR FUTURE WO STAGE
LITTLE FALLS 2YR PRES F STAGE
LITTLE FALLS 2YR PRES F C STAGE
LITTLE FALLS 1YR FUTURE WO STAGE
LITTLE FALLS 1YR PRES F STAGE
LITTLE FALLS 1YR PRES F C STAGE

Figure C-59c

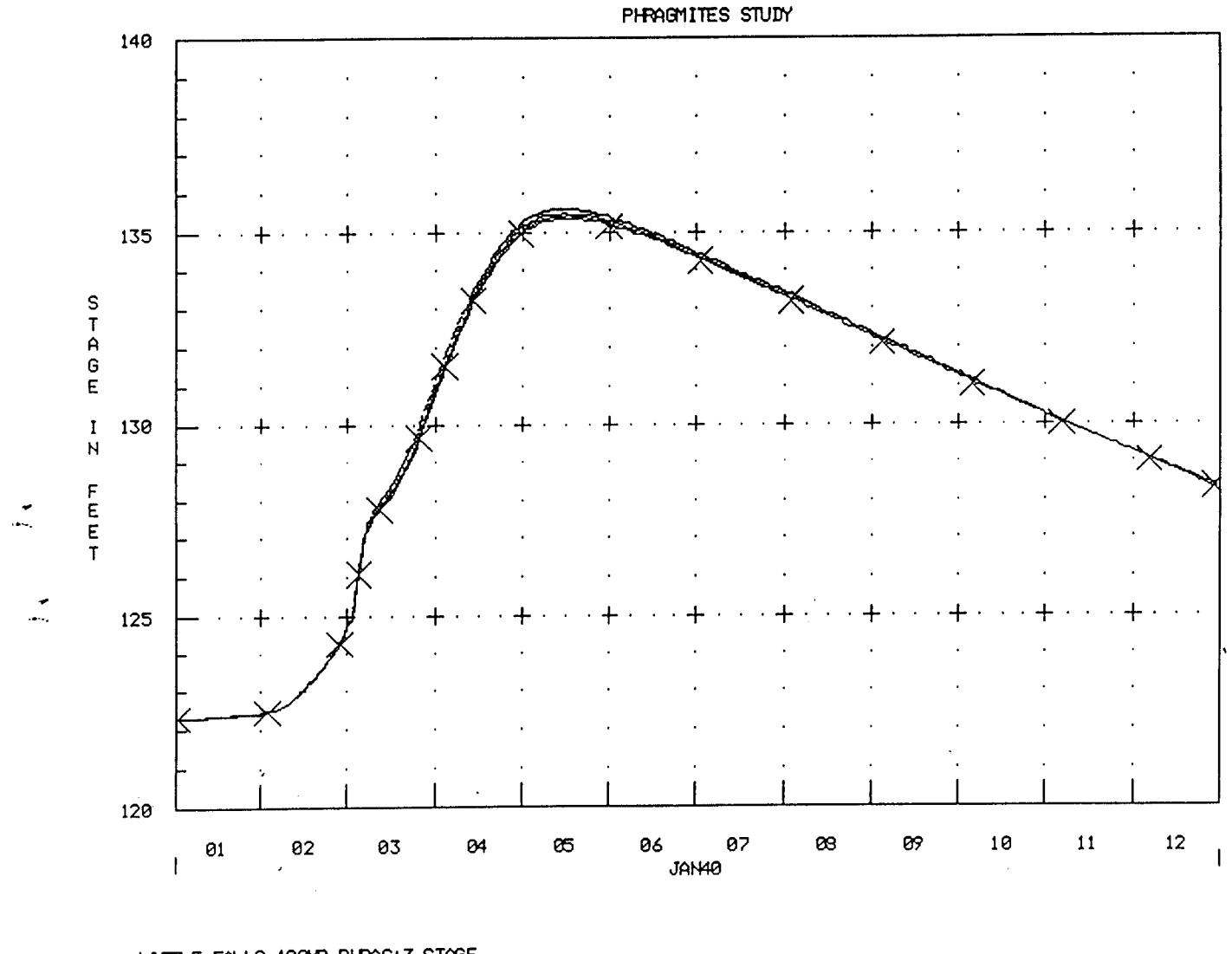

15MAY95 12:34:58
PHRAGMITES STUDY
140
135
130
125
120
STAGE IN FEET
01
02
03
04
05
06
07
08
09
10
11
12
JAN40
LITTLE FALLS 100YR PHRAG+3 STAGE
LITTLE FALLS 100YR PHRAG+2 STAGE
LITTLE FALLS 100YR PHRAG+1 STAGE
LITTLE FALLS 100YR FUTURE WO STAGE

Figure C-59d

15MAY95 10:33:26

PHRAGMITES STUDY

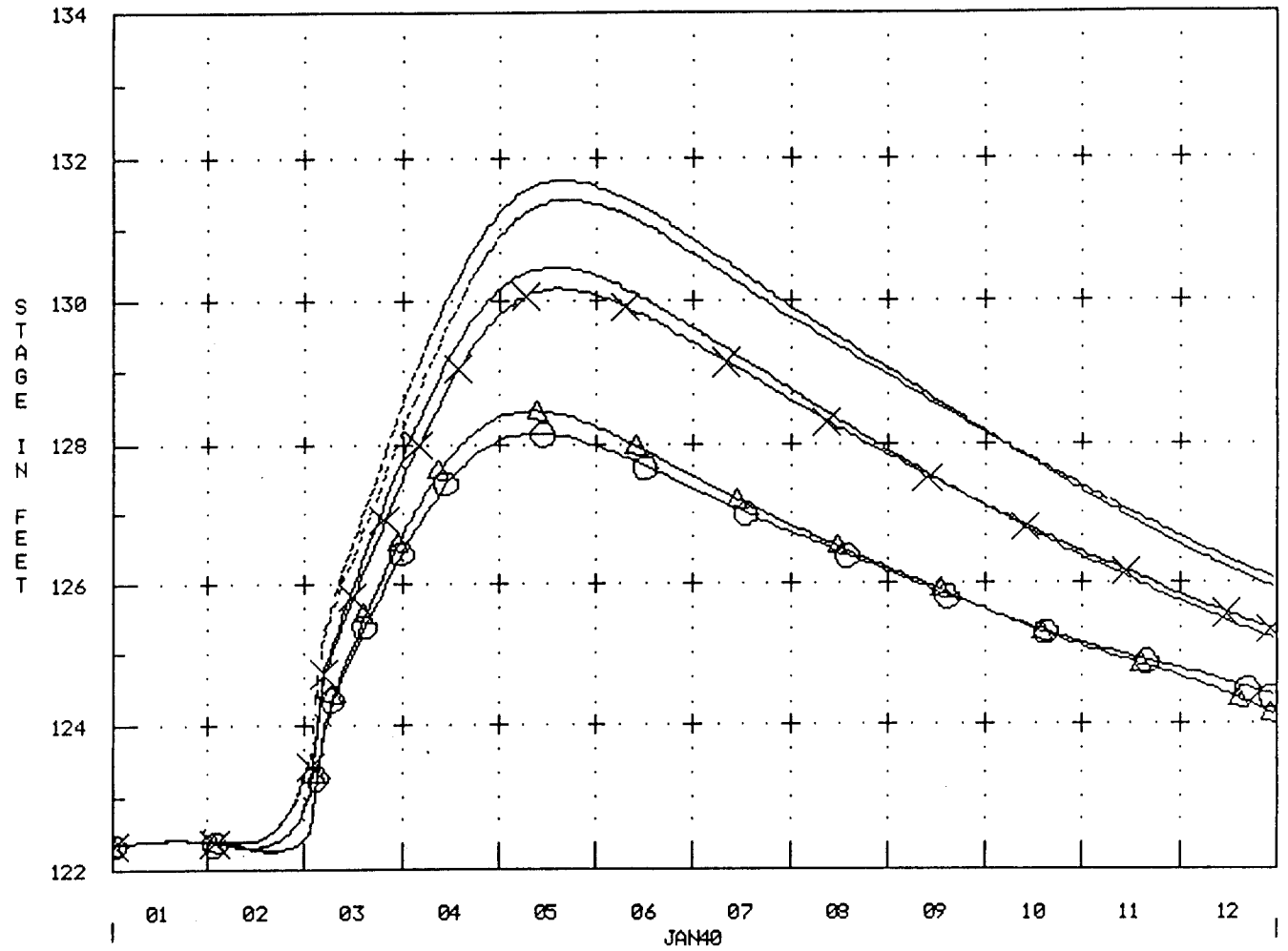


Figure C-59e

08AUG95 15:43:49

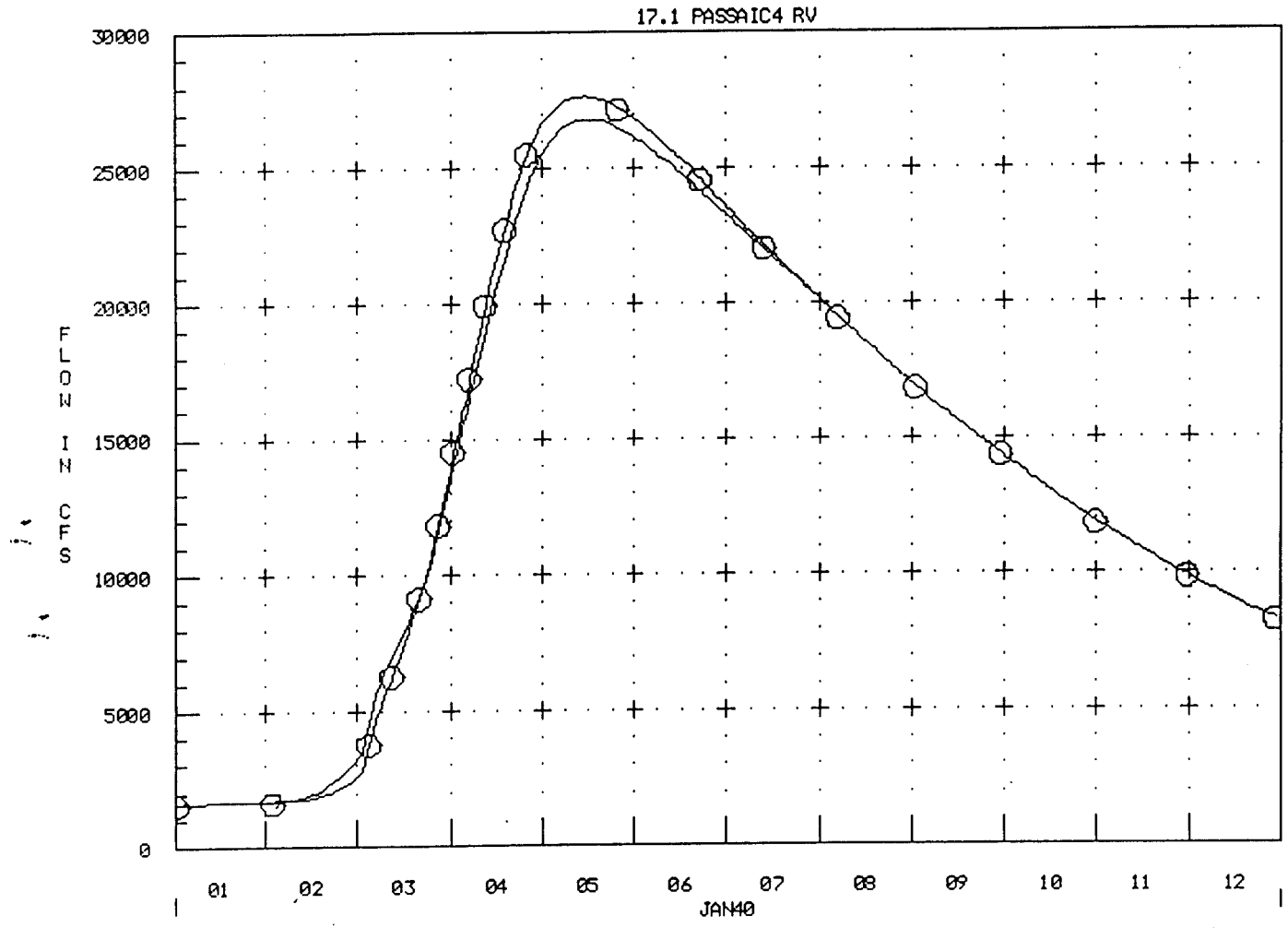


Figure C-60

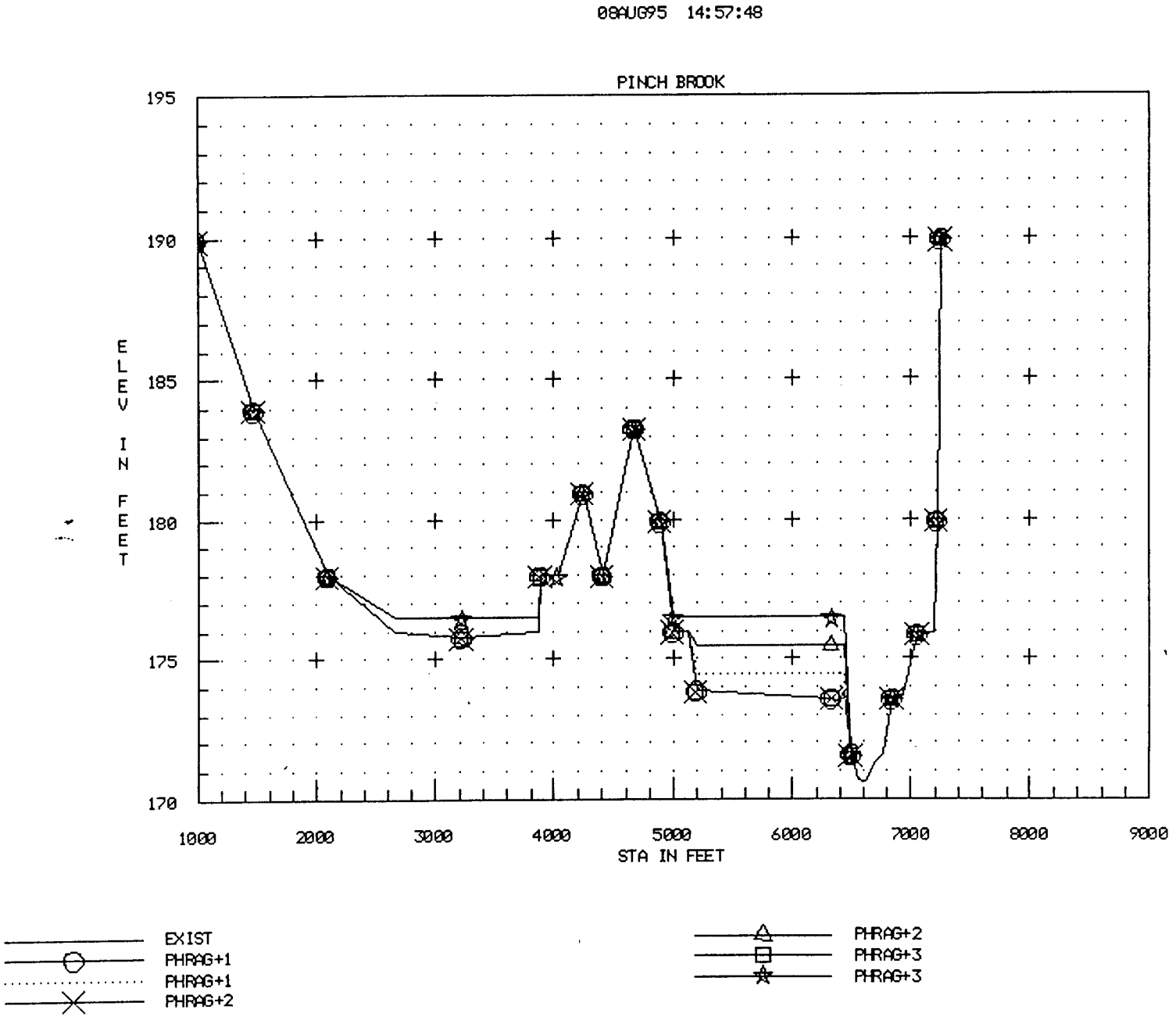

08AUG95 14:57:48
PINCH BROOK
195
190
185
180
175
170
ELEV IN FEET
1000
2000
3000
4000
5000
6000
7000
8000
9000
STA IN FEET
EXIST
PHRAG+1
PHRAG+1
PHRAG+2
PHRAG+2
PHRAG+3
PHRAG+3

Figure C-61

08AUG95 15:04:33

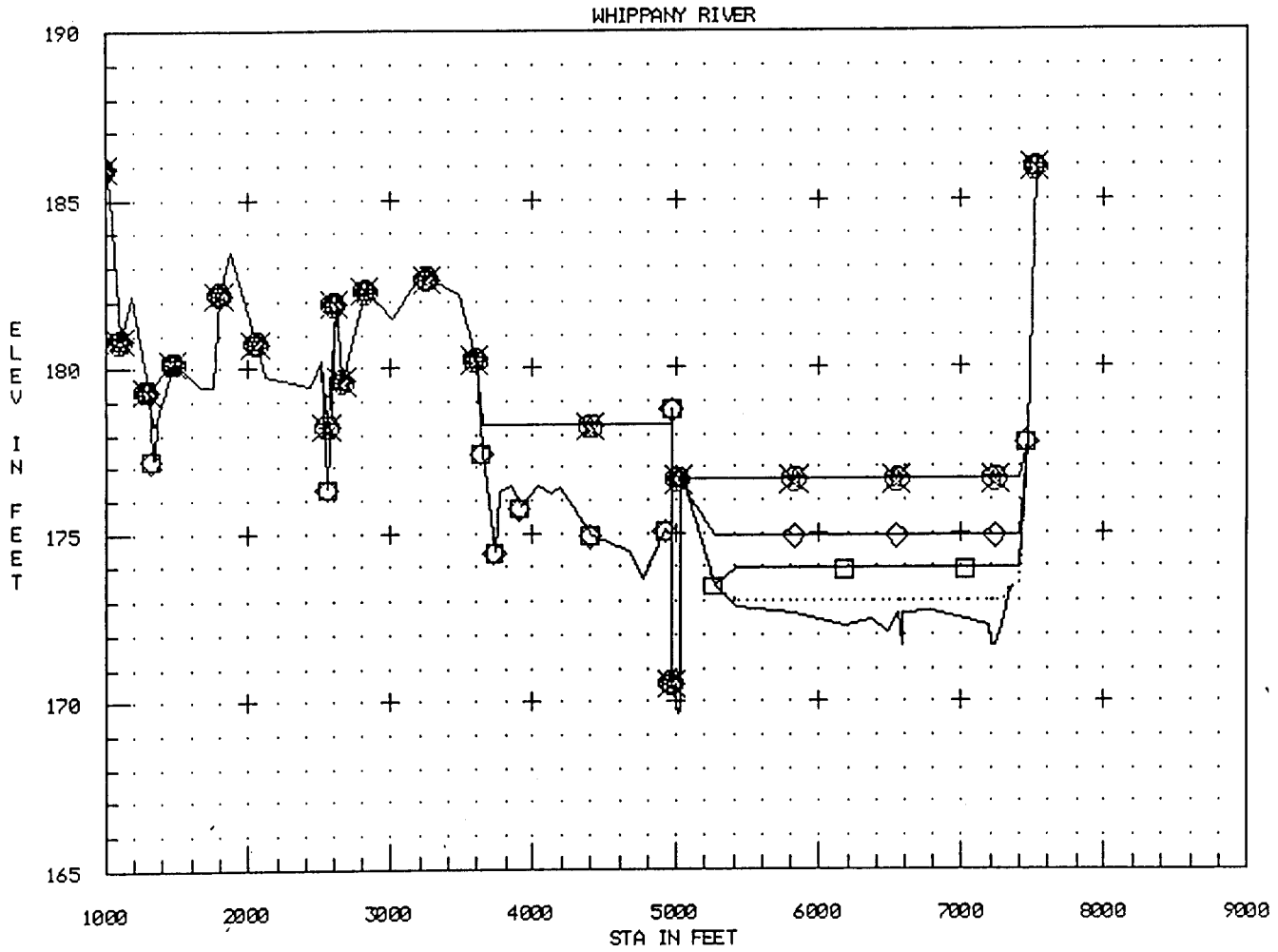


Figure C-62

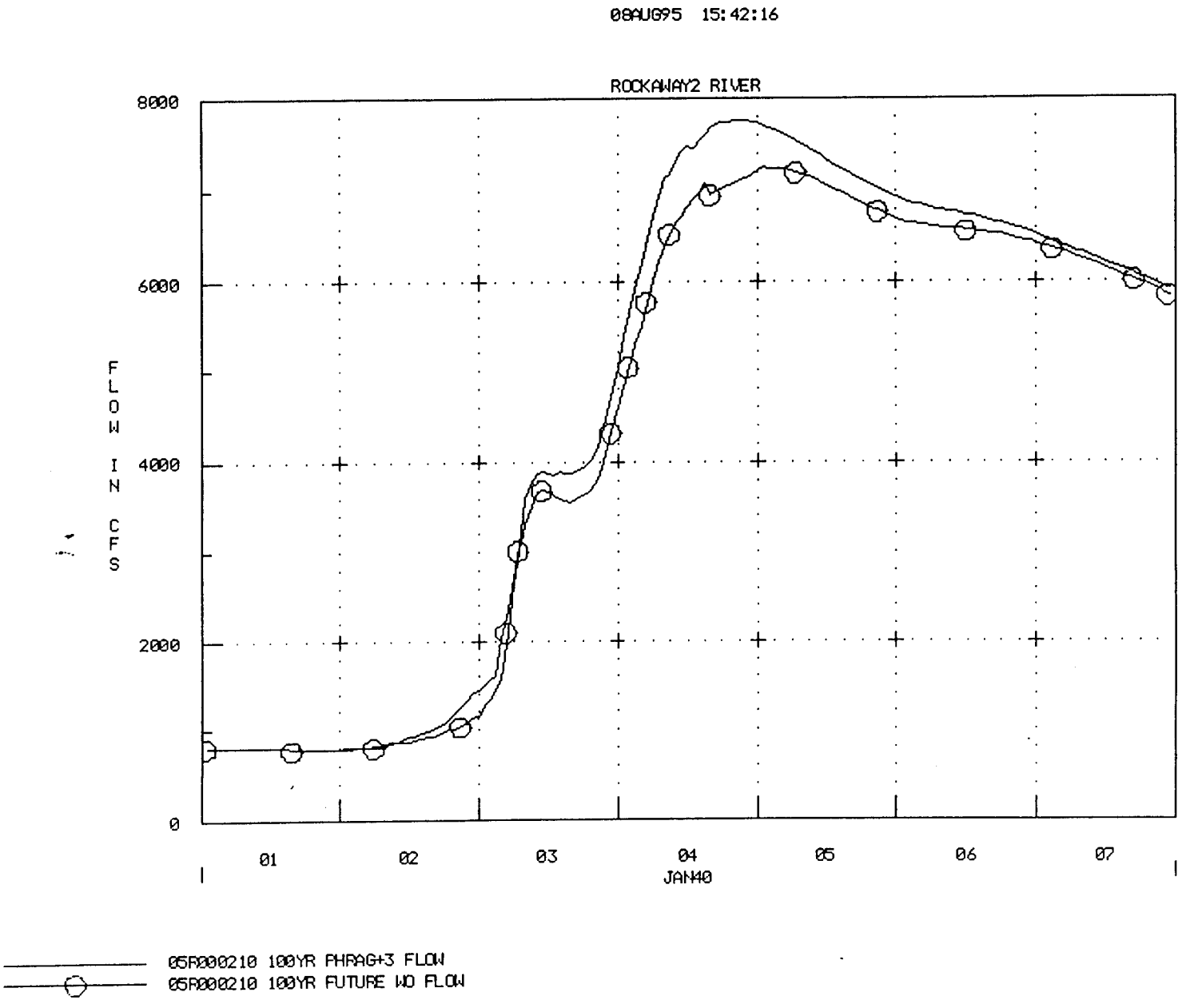

08AUG95 15:42:16
ROCKAWAY2 RIVER
8000
6000
4000
2000
0
FLOW IN CFS
01
02
03
04
05
06
07
JAN40
05R000210 100YR PHRAG+3 FLOW
05R000210 100YR FUTURE WO FLOW

Figure C-63

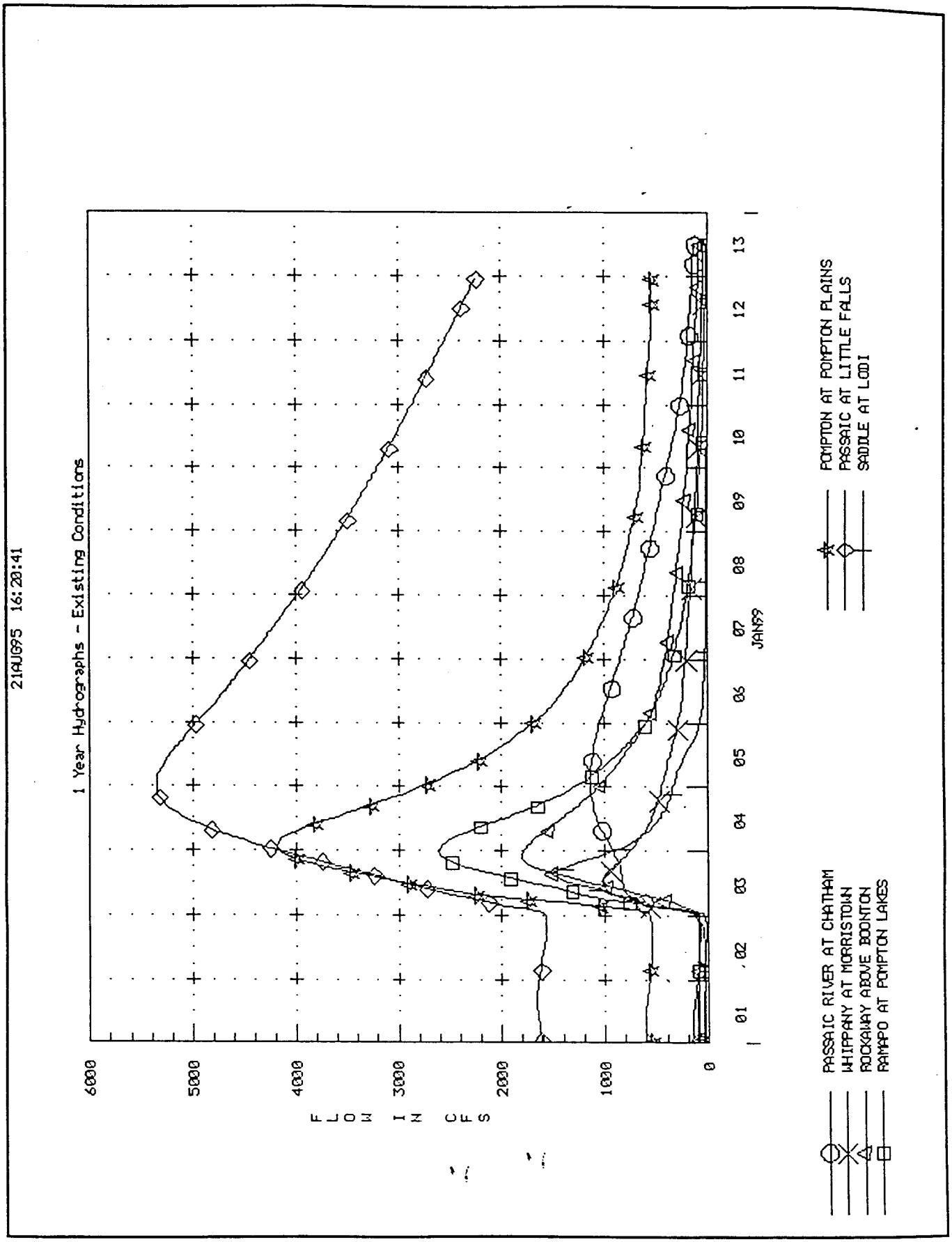

21AUG95 16:20:41
1 Year Hydrographs - Existing Conditions
FLOW IN CFS
6000
5000
4000
3000
2000
1000
0
01
02
03
04
05
06
07
08
09
10
11
12
13
JAN99
PASSAIC RIVER AT CHATHAM
WHIPPANY AT MORRISTOWN
ROCKAWAY ABOVE BOONTON
RAMAPO AT POMPTON LAKES
POMPTON AT POMPTON PLAINS
PASSAIC AT LITTLE FALLS
SADDLE AT LODI

Figure C-64a

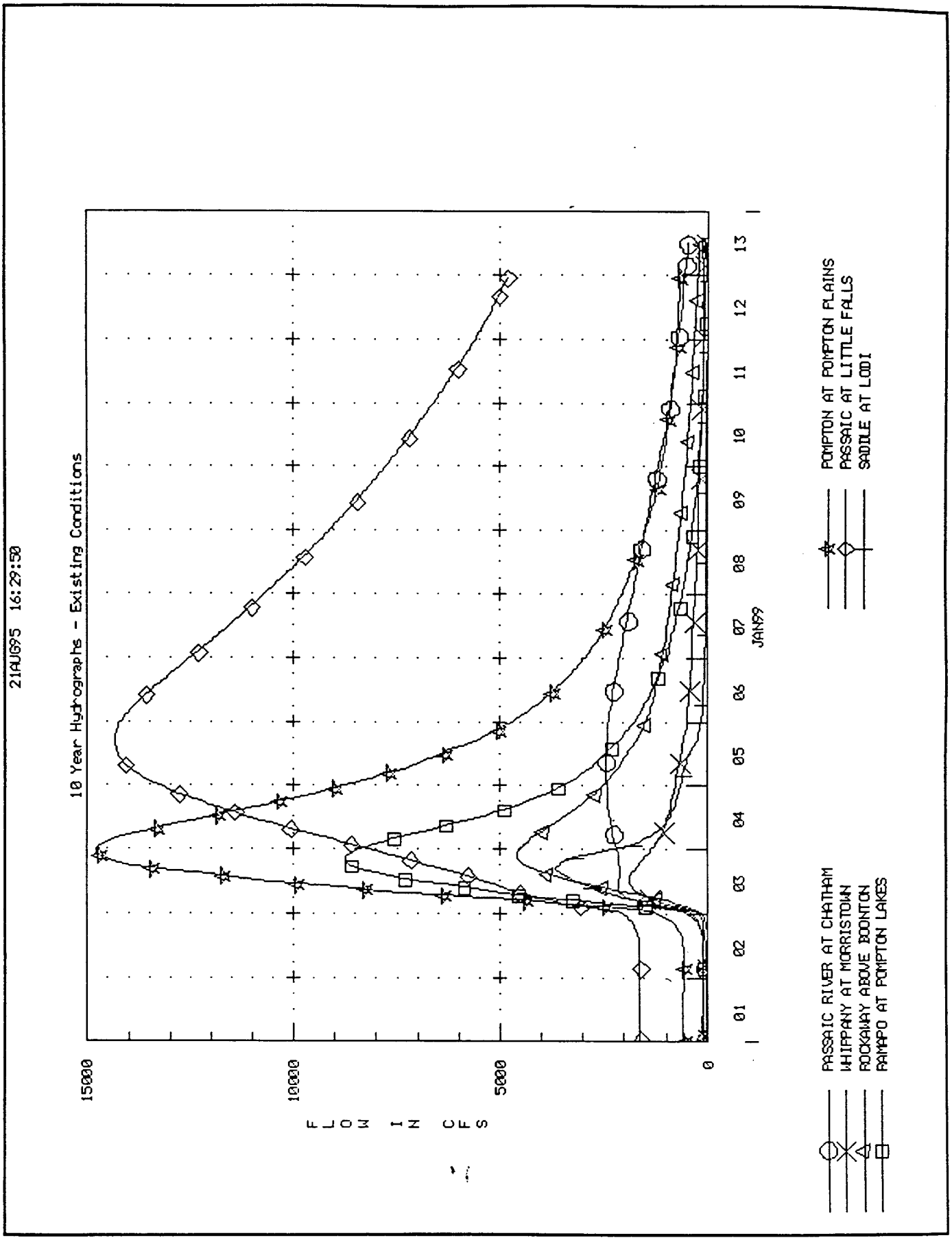

21AUG95 16:29:50
10 Year Hydrographs - Existing Conditions
FLOW IN CFS
15000
10000
5000
0
01
02
03
04
05
06
07
08
09
10
11
12
13
JAN99
PASSAIC RIVER AT CHATHAM
WHIPPANY AT MORRISTOWN
ROCKAWAY ABOVE BOONTON
RAMAPO AT POMPTON LAKES
POMPTON AT POMPTON PLAINS
PASSAIC AT LITTLE FALLS
SADDLE AT LODI

Figure C-64b

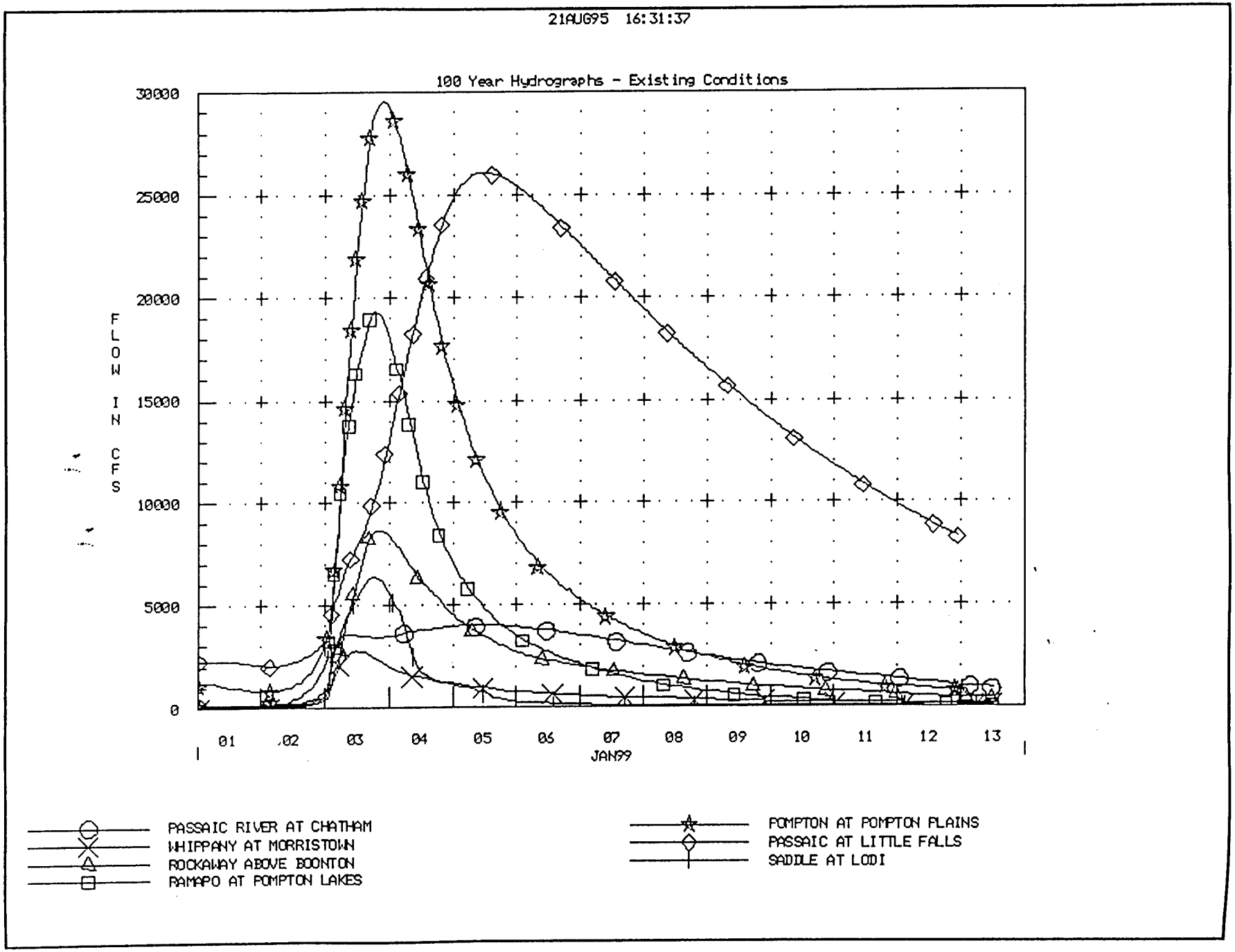

21AUG95 16:31:37
100 Year Hydrographs - Existing Conditions
30000
25000
20000
15000
10000
5000
0
FLOW IN CFS
01
02
03
04
05
06
07
08
09
10
11
12
13
JAN99
PASSAIC RIVER AT CHATHAM
WHIPPANY AT MORRISTOWN
ROCKAWAY ABOVE BOONTON
RAMAPO AT POMPTON LAKES
POMPTON AT POMPTON PLAINS
PASSAIC AT LITTLE FALLS
SADDLE AT LODI

Figure C-64c

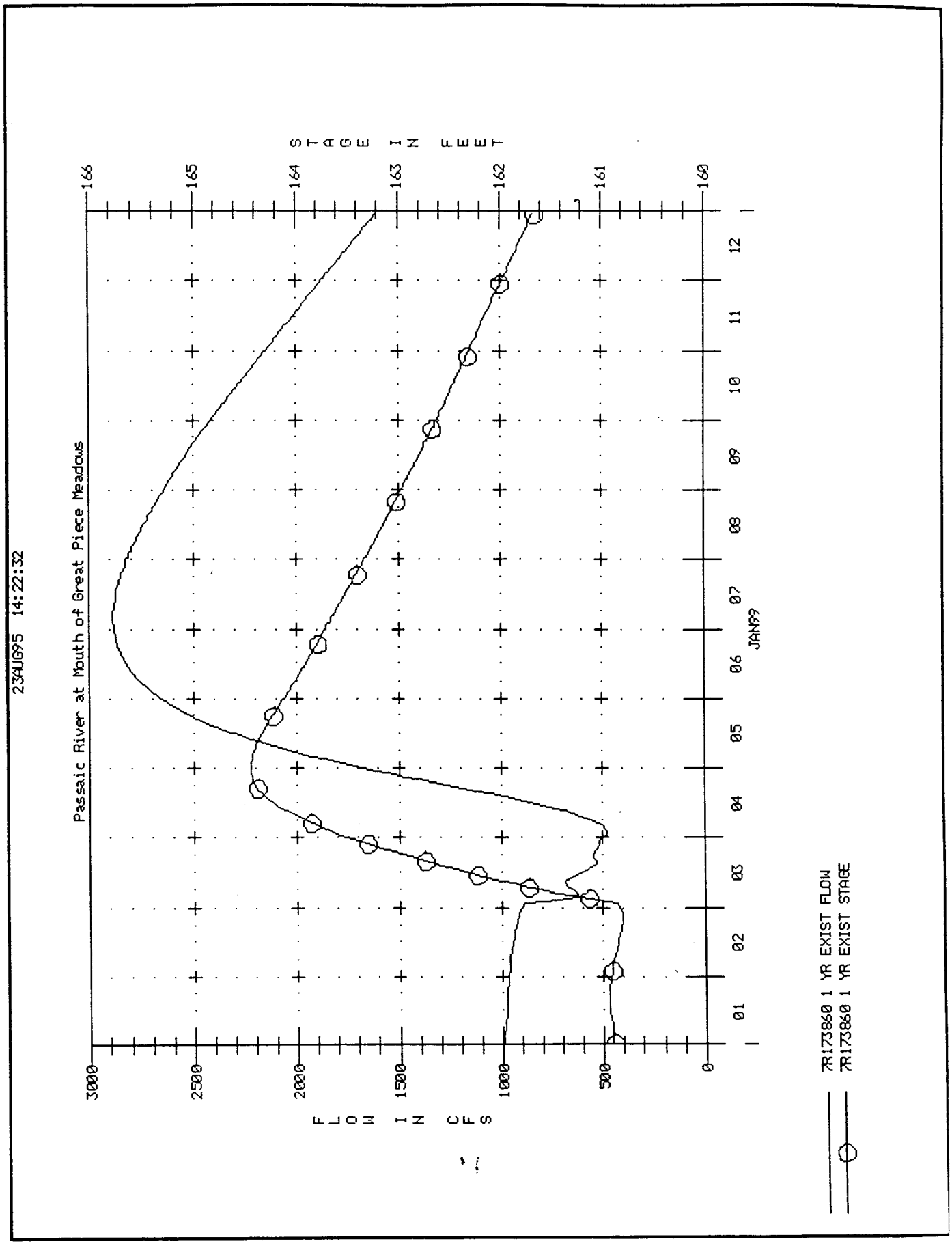

23AUG95 14:22:32
Passaic River at Mouth of Great Piece Meadows
FLOW IN CFS
STAGE IN FEET
JAN99
01
02
03
04
05
06
07
08
09
10
11
12
0
500
1000
1500
2000
2500
3000
160
161
162
163
164
165
166
JR173860 1 YR EXIST FLOW
JR173860 1 YR EXIST STAGE

Figure C-64d

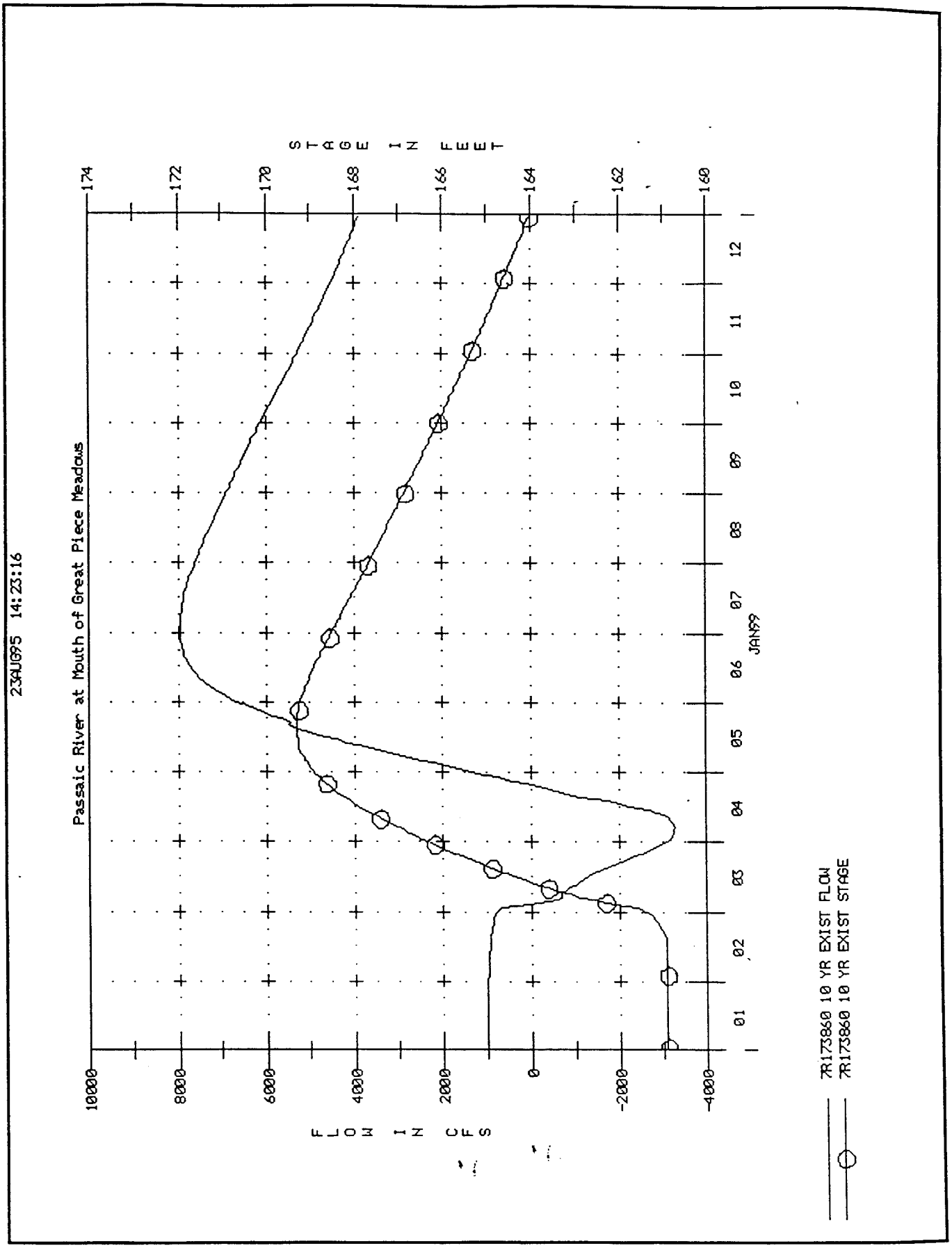

23AUG95 14:23:16
Passaic River at Mouth of Great Piece Meadows
STAGE IN FEET
FLOW IN CFS
JAN99
7R173860 10 YR EXIST FLOW
7R173860 10 YR EXIST STAGE

Figure C-64e

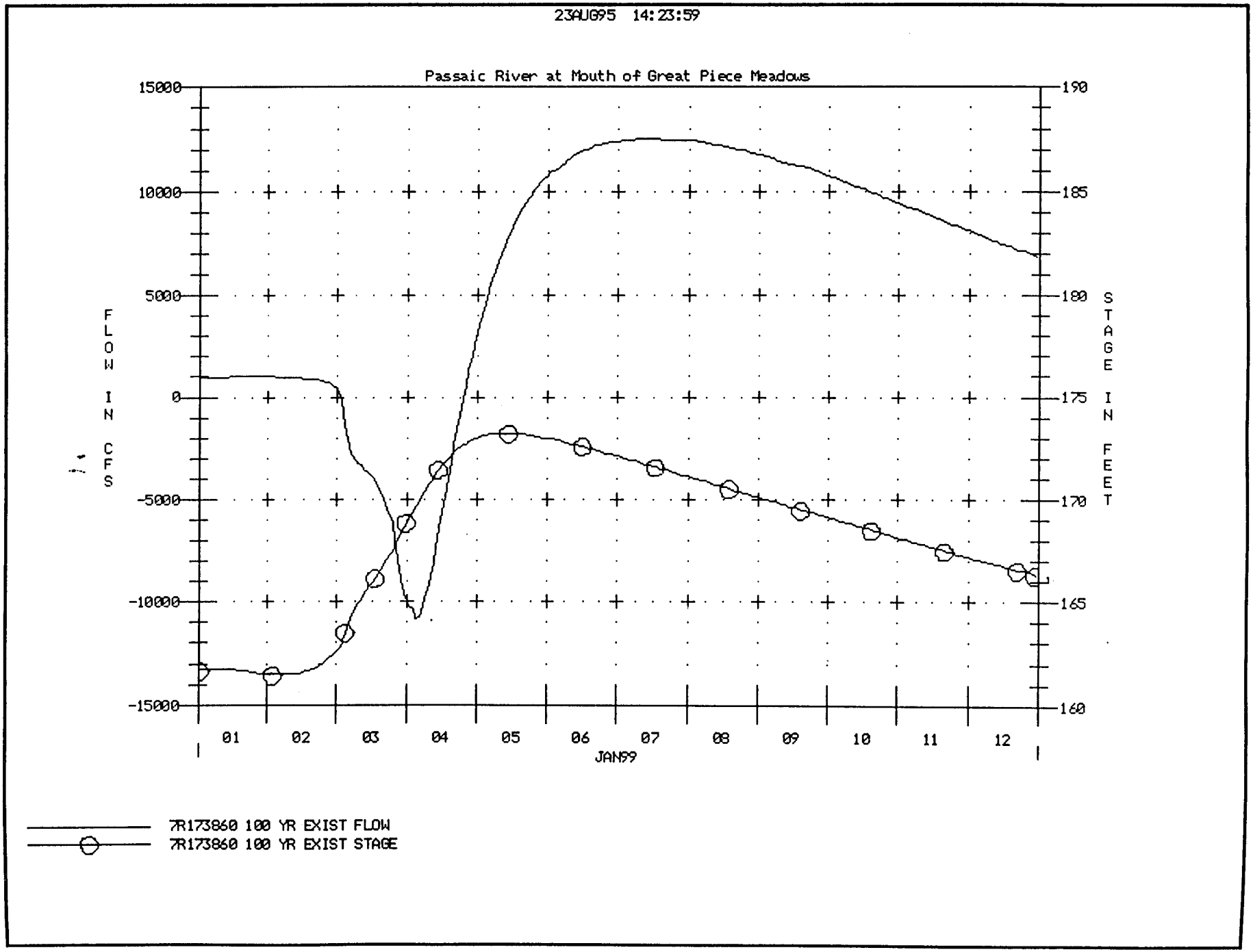

23AUG95 14:23:59
Passaic River at Mouth of Great Piece Meadows
15000
10000
5000
0
-5000
-10000
-15000
FLOW IN CFS
190
185
180
175
170
165
160
STAGE IN FEET
01
02
03
04
05
06
07
08
09
10
11
12
JAN99
7R173860 100 YR EXIST FLOW
7R173860 100 YR EXIST STAGE

Figure C-64f

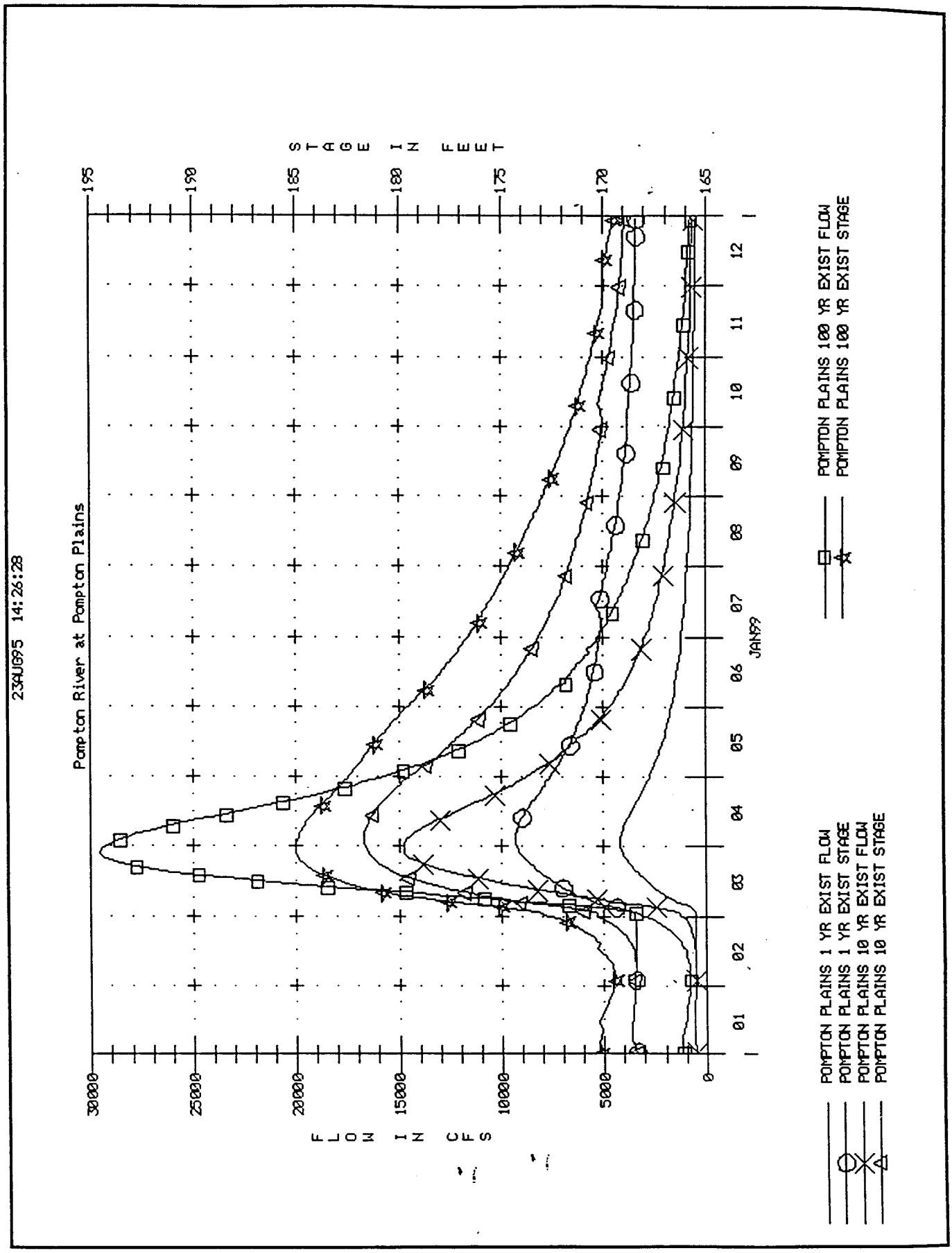

23AUG95 14:26:28
Pompton River at Pompton Plains
FLOW IN CFS
STAGE IN FEET
JAN99
POMPTON PLAINS 1 YR EXIST FLOW
POMPTON PLAINS 1 YR EXIST STAGE
POMPTON PLAINS 10 YR EXIST FLOW
POMPTON PLAINS 10 YR EXIST STAGE
POMPTON PLAINS 100 YR EXIST FLOW
POMPTON PLAINS 100 YR EXIST STAGE

Figure C-64g

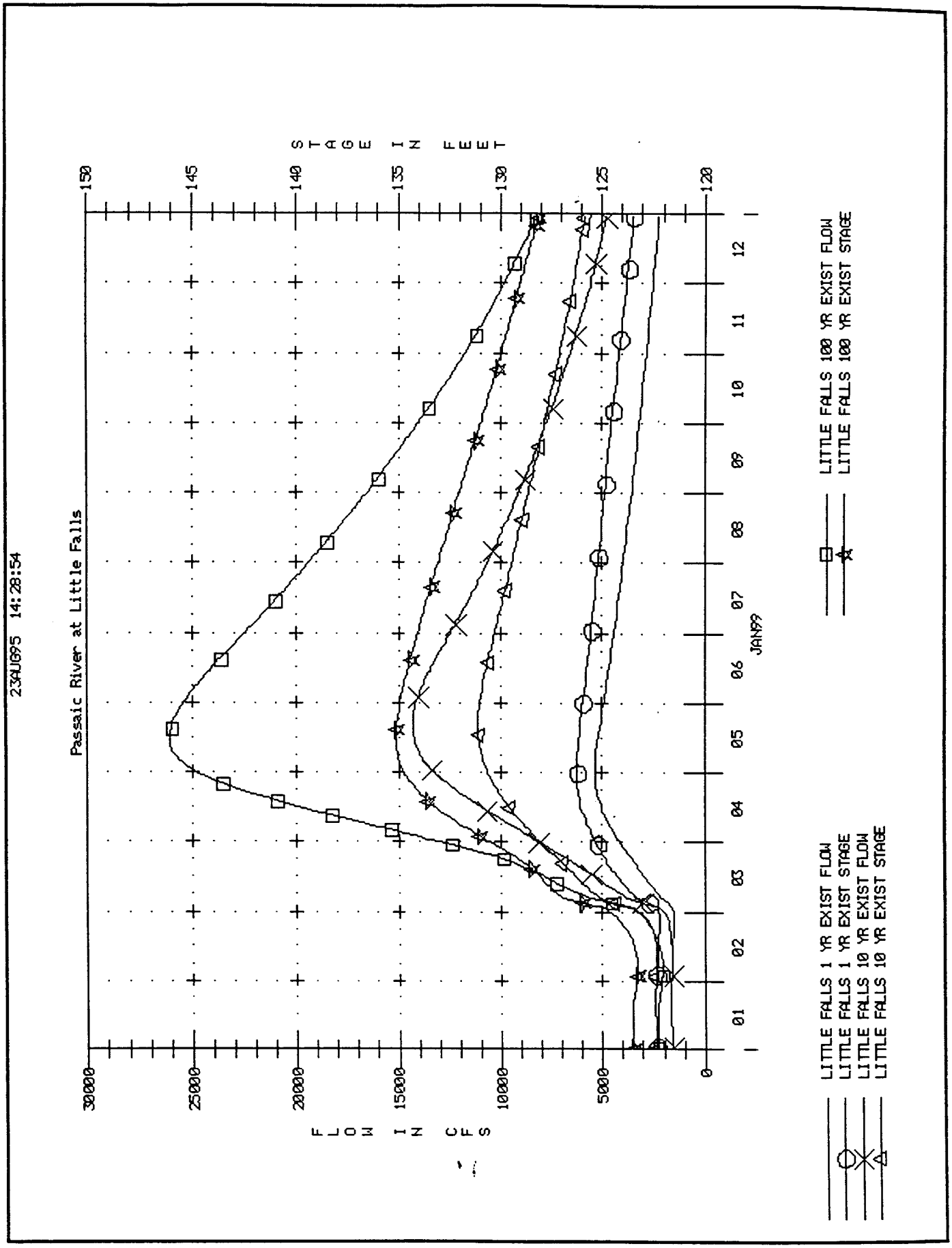

23AUG95 14:28:54
Passaic River at Little Falls
FLOW IN CFS
STAGE IN FEET
JAN99
LITTLE FALLS 1 YR EXIST FLOW
LITTLE FALLS 1 YR EXIST STAGE
LITTLE FALLS 10 YR EXIST FLOW
LITTLE FALLS 10 YR EXIST STAGE
LITTLE FALLS 100 YR EXIST FLOW
LITTLE FALLS 100 YR EXIST STAGE

Figure C-64h

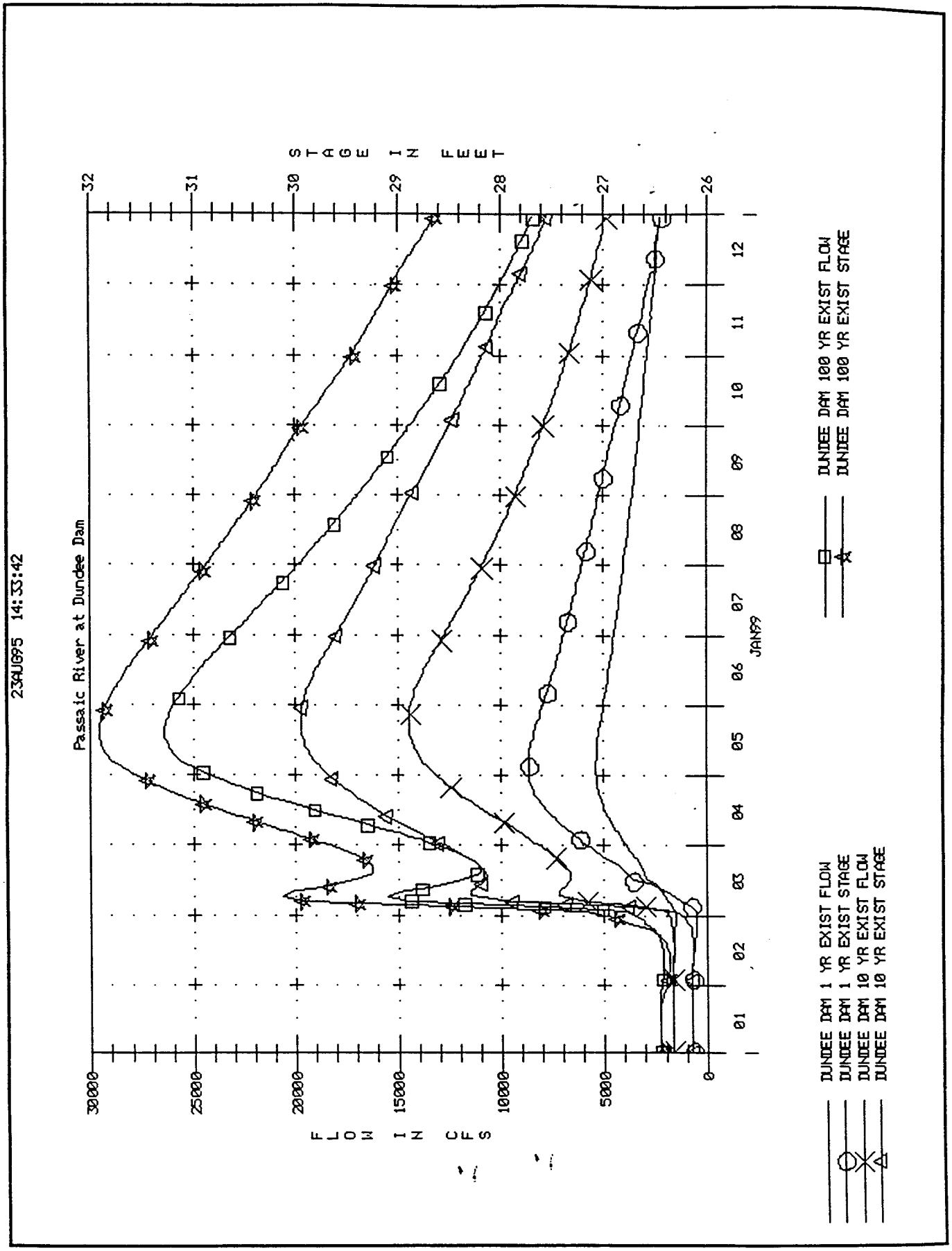

23AUG95 14:33:42
Passaic River at Dundee Dam
STAGE IN FEET
FLOW IN CFS
JAN99
DUNDEE DAM 1 YR EXIST FLOW
DUNDEE DAM 1 YR EXIST STAGE
DUNDEE DAM 10 YR EXIST FLOW
DUNDEE DAM 10 YR EXIST STAGE
DUNDEE DAM 100 YR EXIST FLOW
DUNDEE DAM 100 YR EXIST STAGE

Figure C-64i

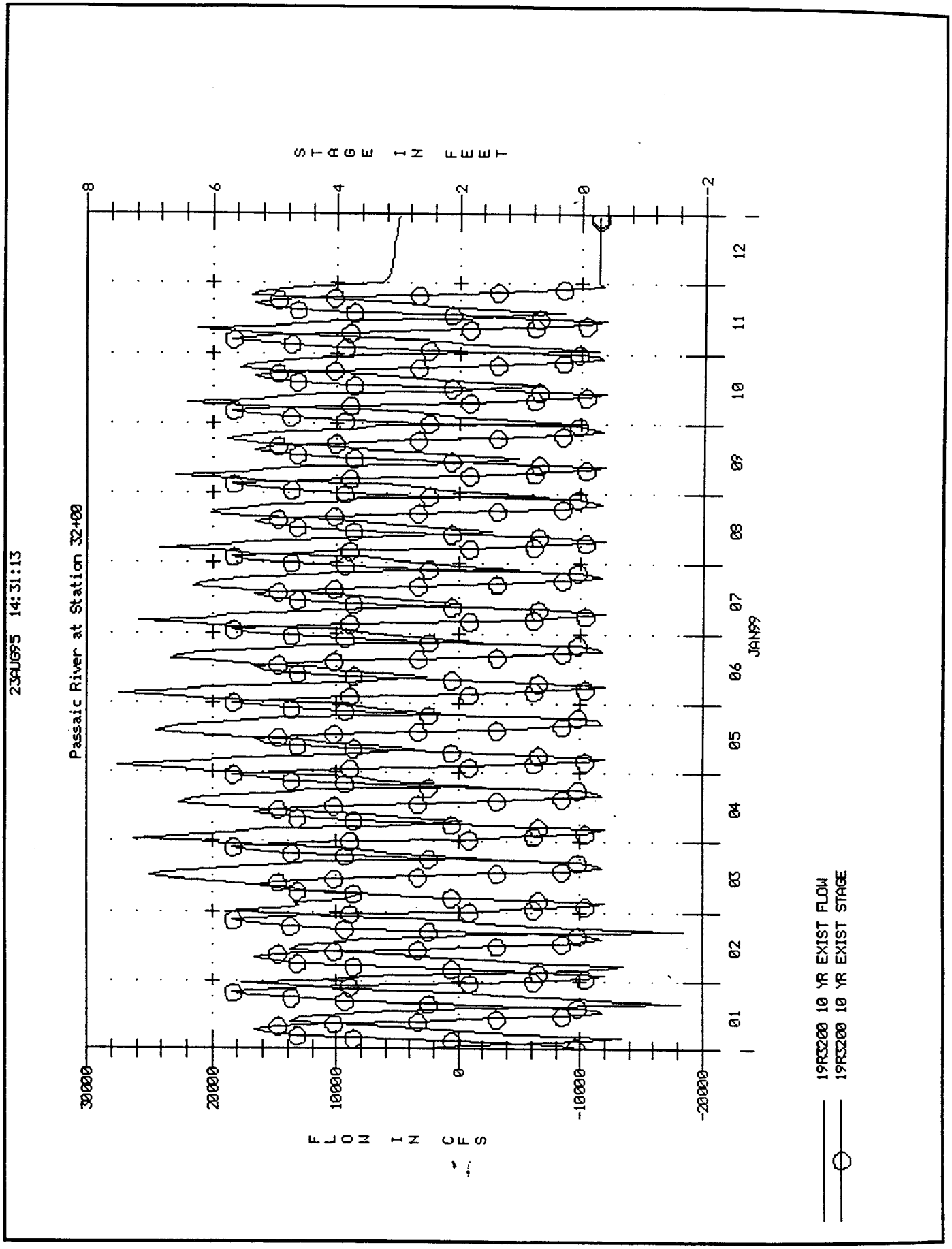

23AUG95 14:31:13
Passaic River at Station 32+00
STAGE IN FEET
FLOW IN CFS
JAN99
19R32B0 10 YR EXIST FLOW
19R32B0 10 YR EXIST STAGE

Figure C-64j

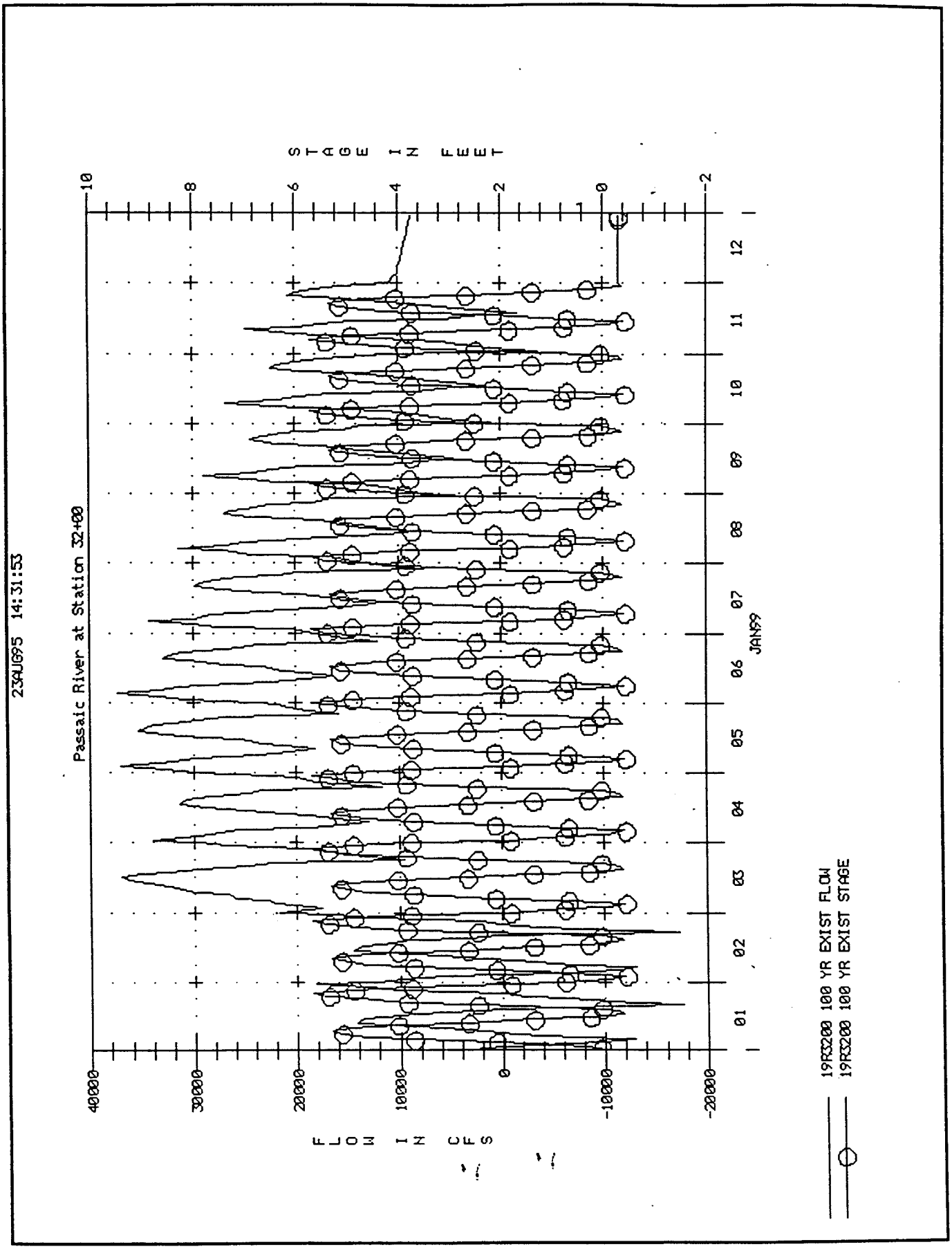

23AUG95 14:31:53
Passaic River at Station 32+00
STAGE IN FEET
10
8
6
4
2
0
-2
FLOW IN CFS
40000
30000
20000
10000
0
-10000
-20000
01
02
03
04
05
06
07
08
09
10
11
12
JAN99
19RS200 100 YR EXIST FLOW
19RS200 100 YR EXIST STAGE

Figure C-64k

Passaic River, Station 2403+00, Rating Curve

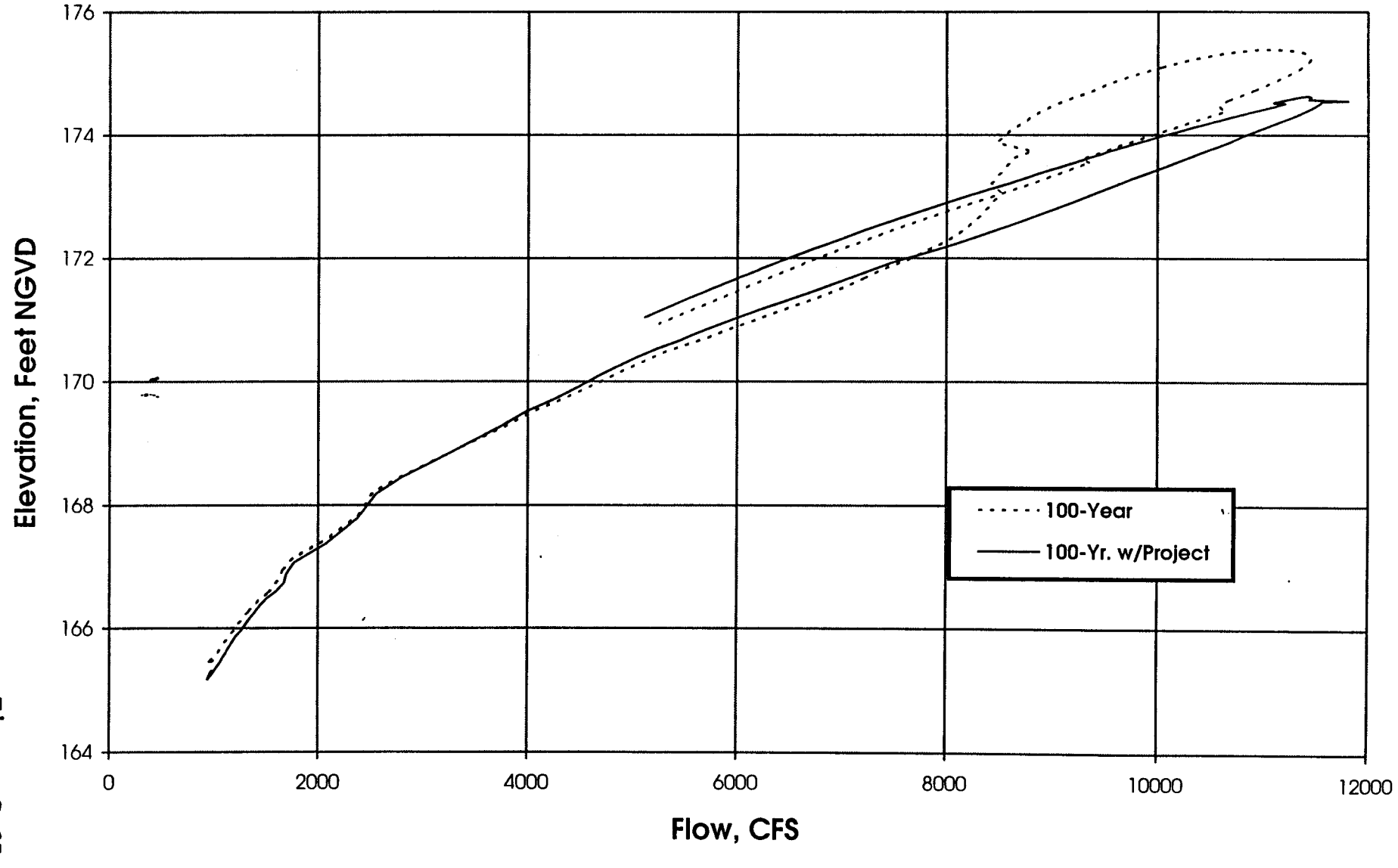


Figure C-65a

Pequannock River, Sta 71+34, Rating Curve

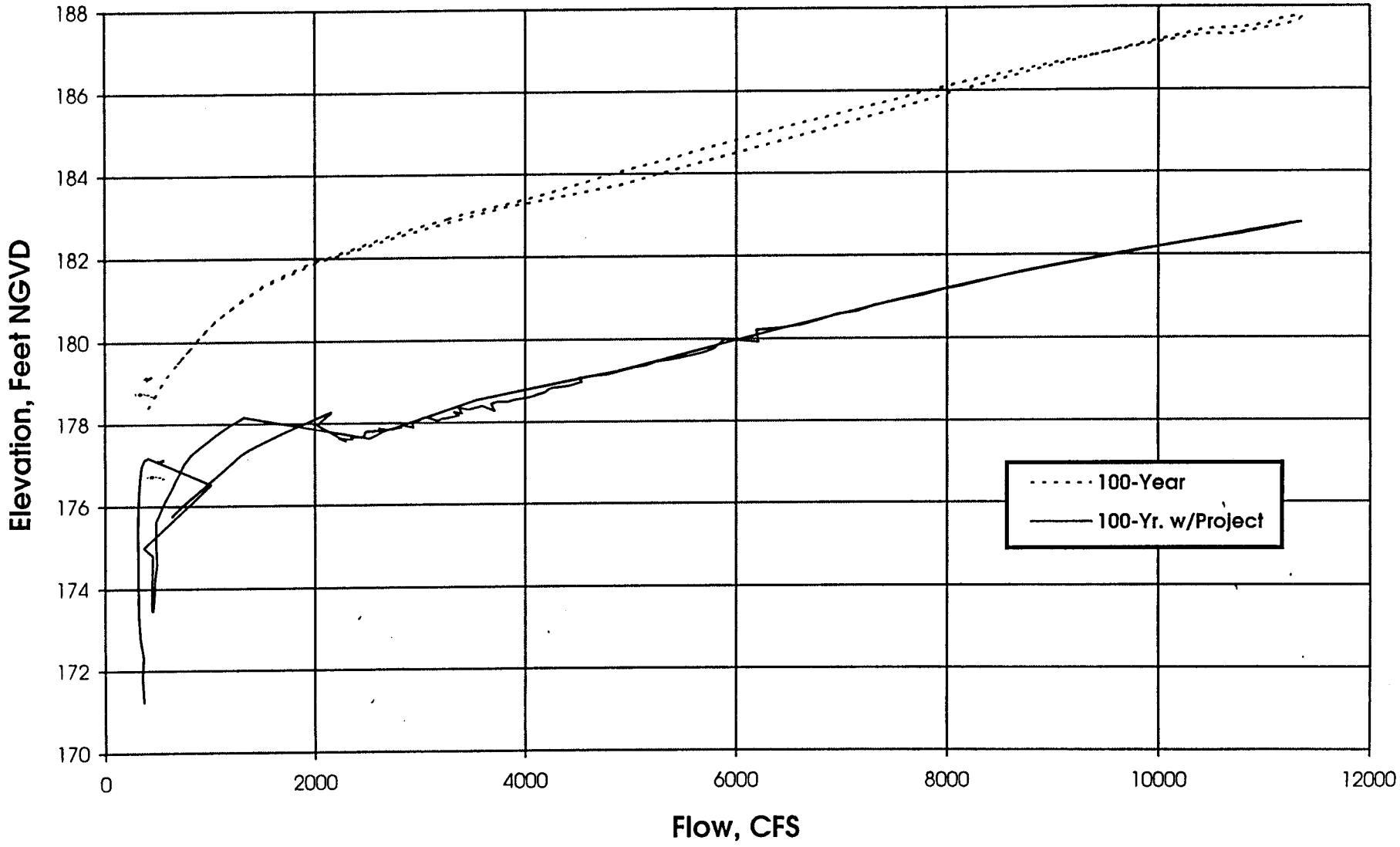


Figure C-65b

Ramapo River, Station 941+10, Rating Curve

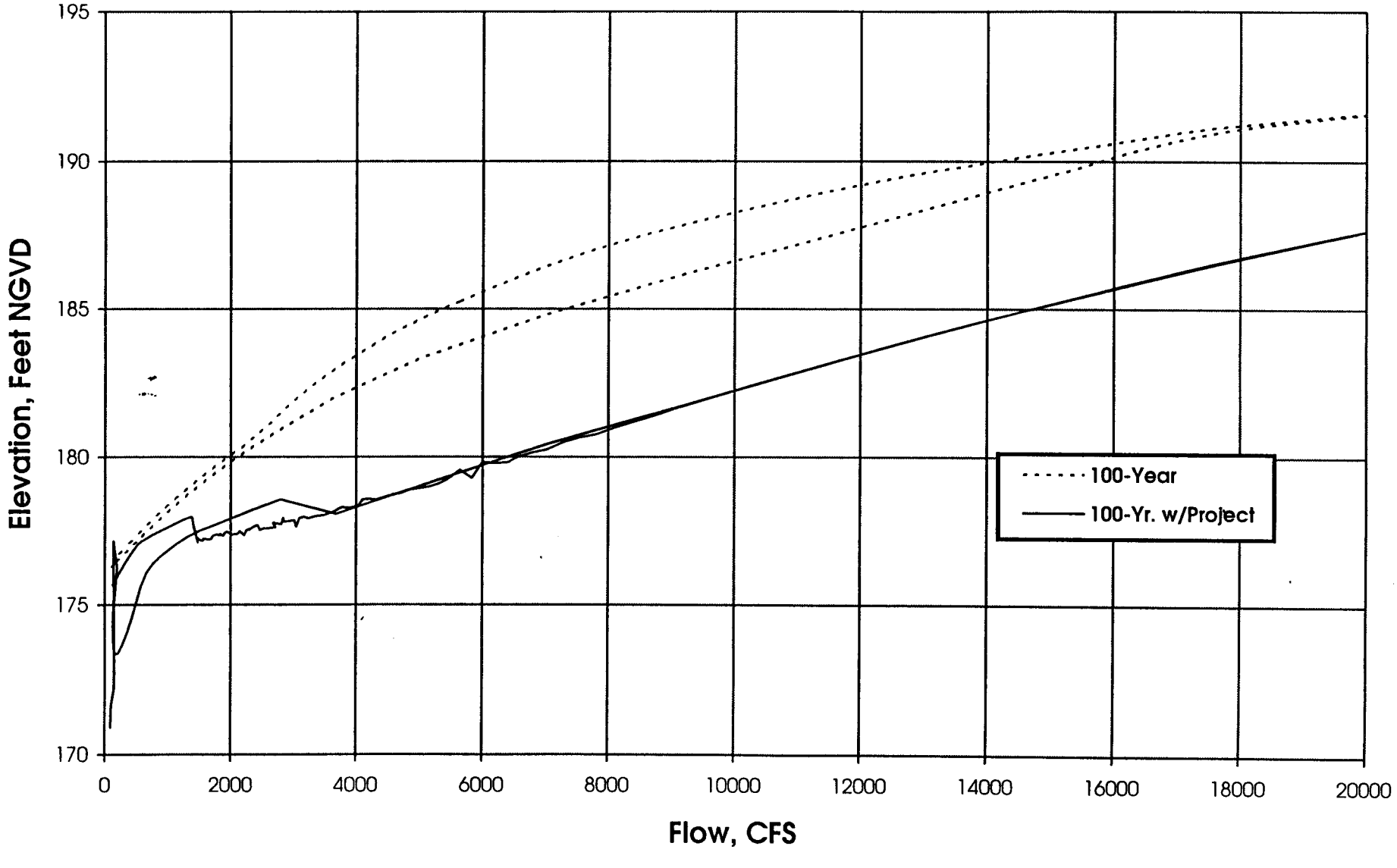


Figure C-65c

Pompton River at Hoffman Grove, Rating Curve

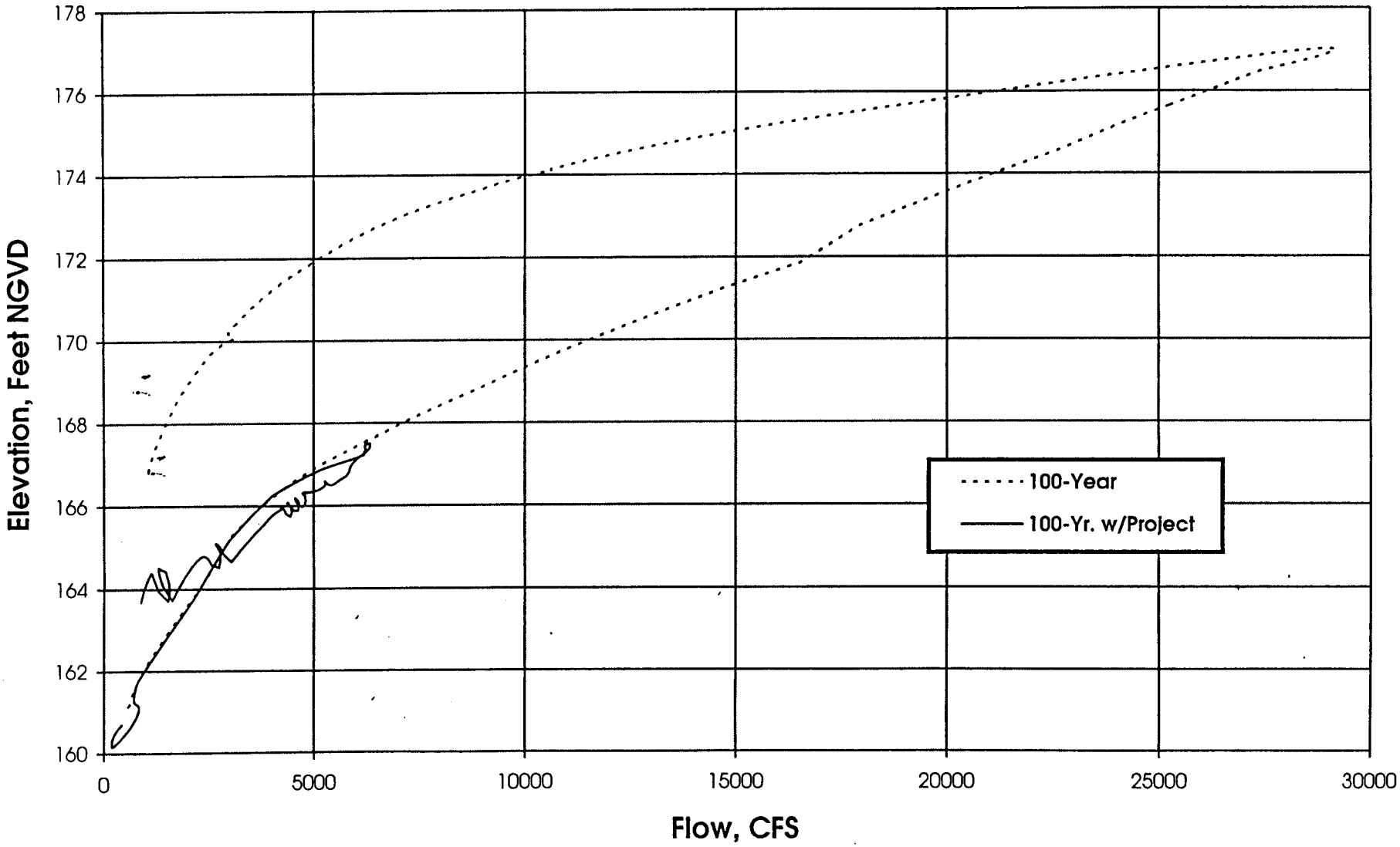


Figure C-65d

Deepavaal Brook, Sta 109+00, Rating Curve

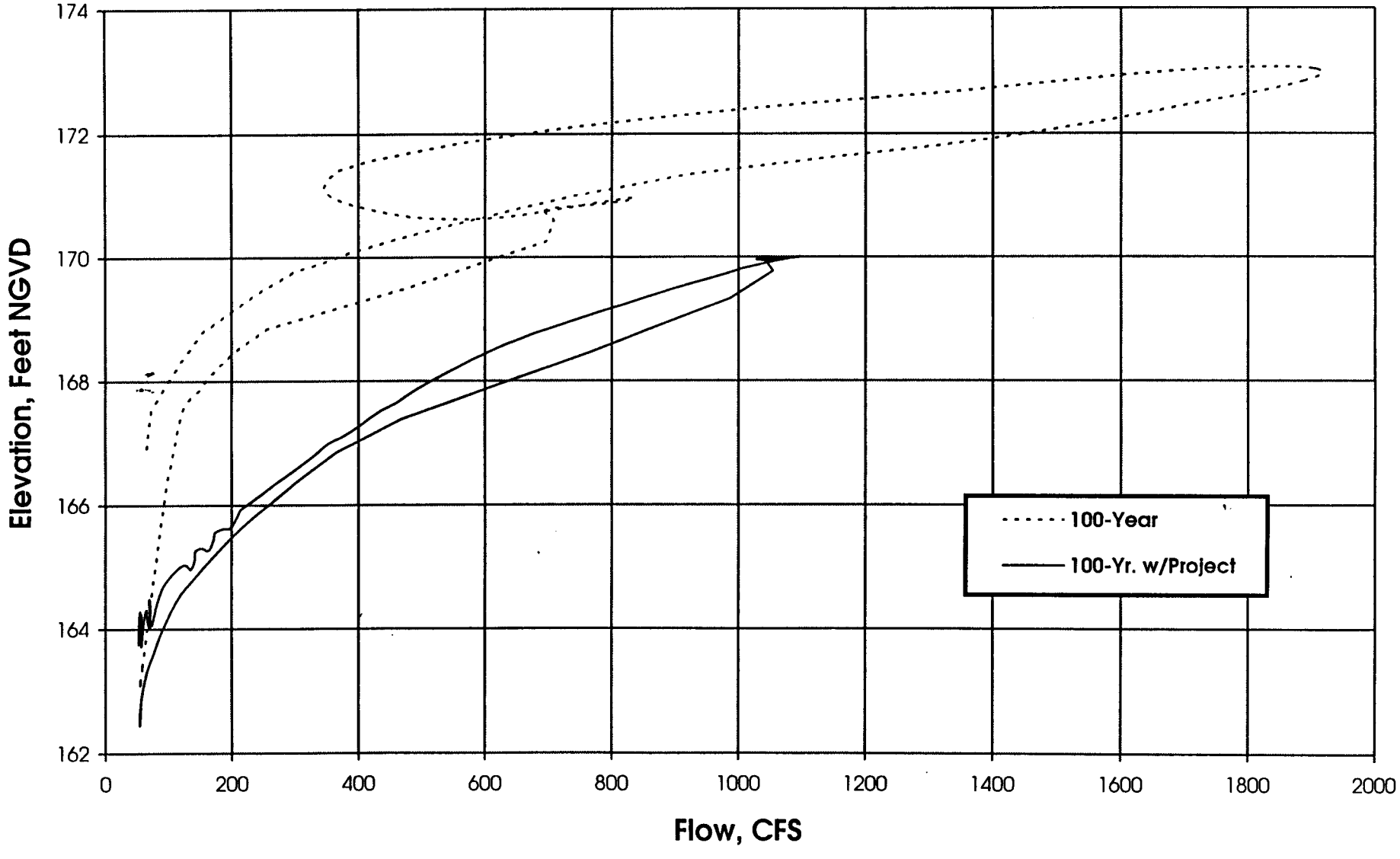


Figure C-65e

Passaic River at Little Falls, Rating Curve

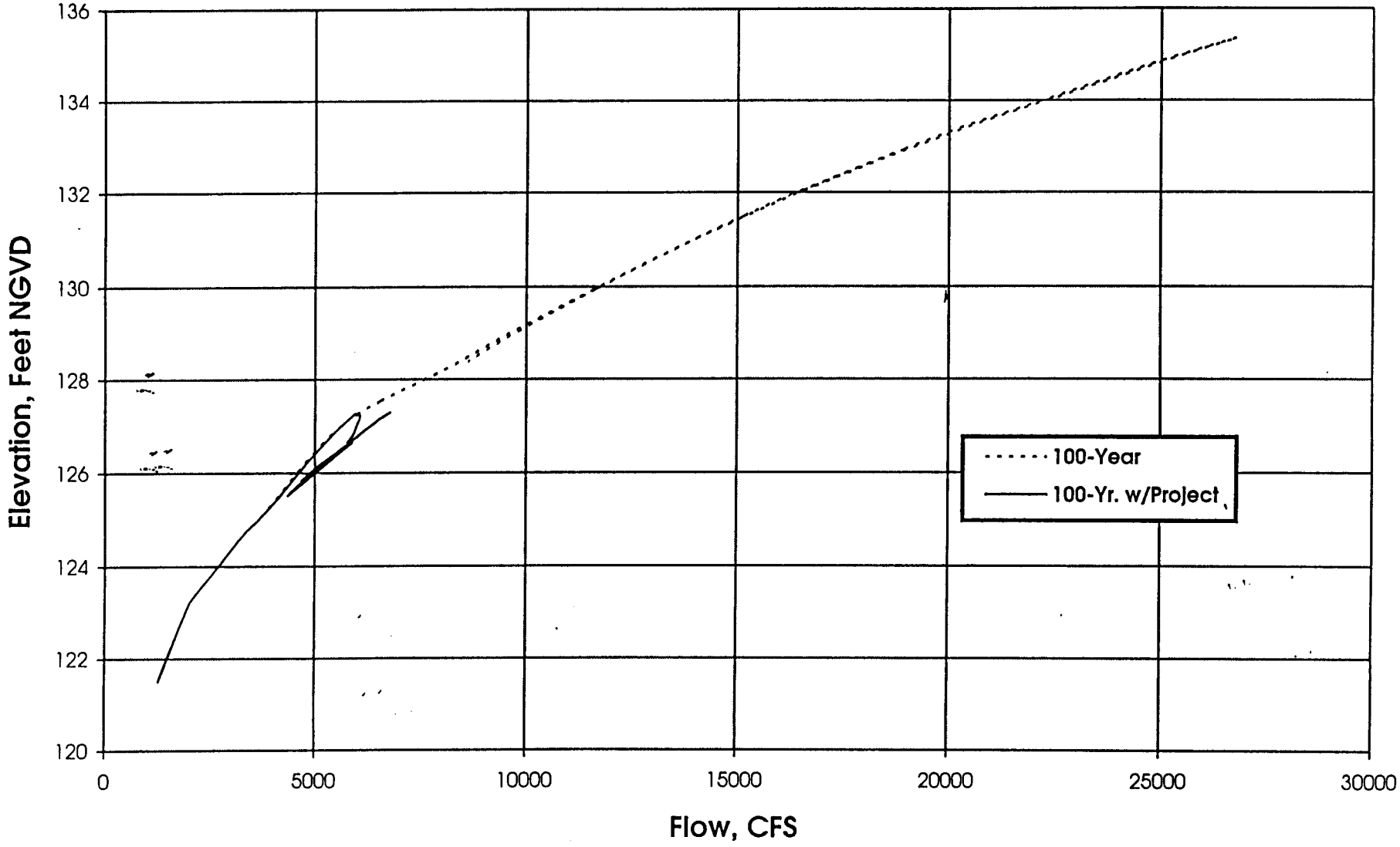


Figure C-65f

06SEP95 13:40:00

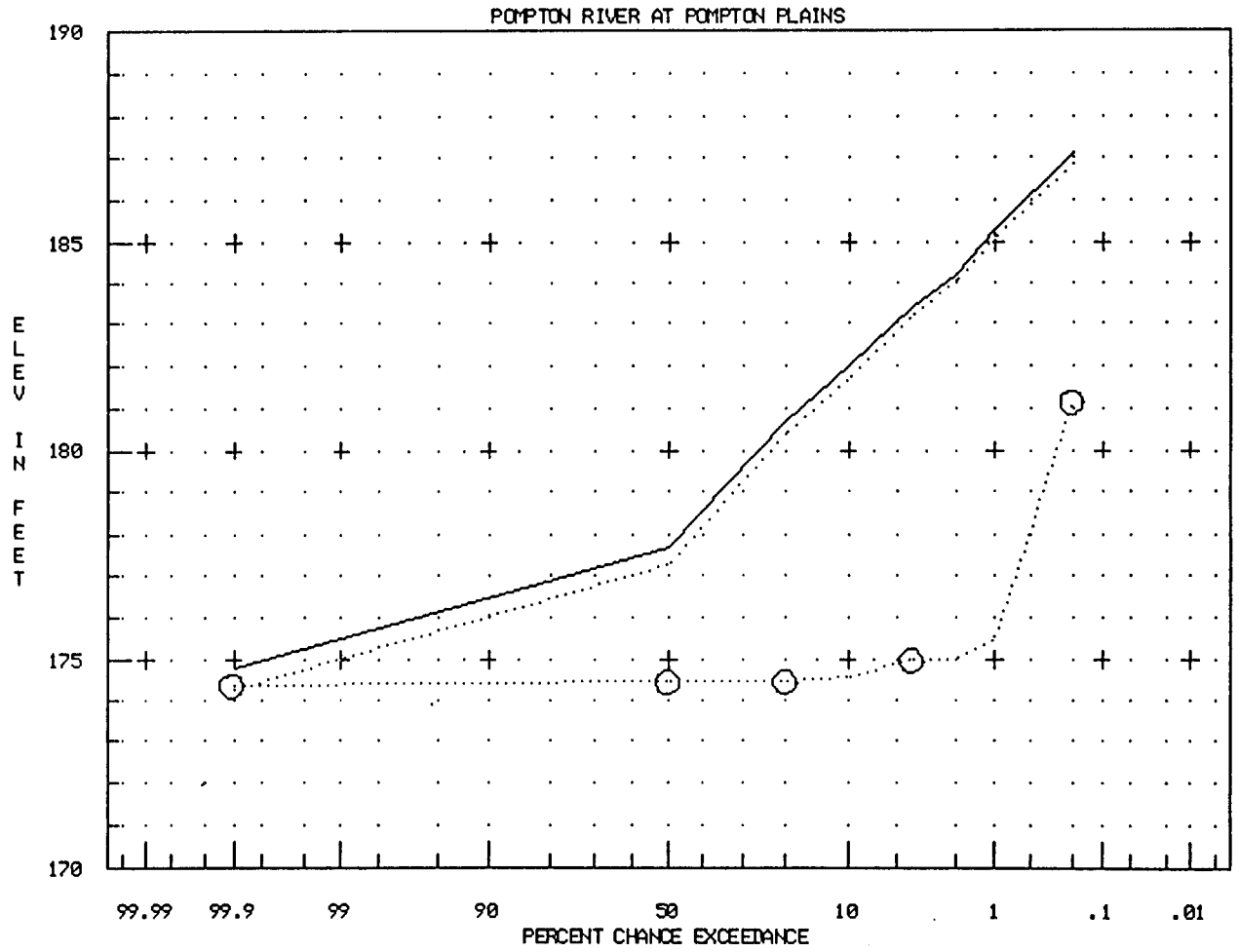


Figure C-66a

06SEP95 13:39:38

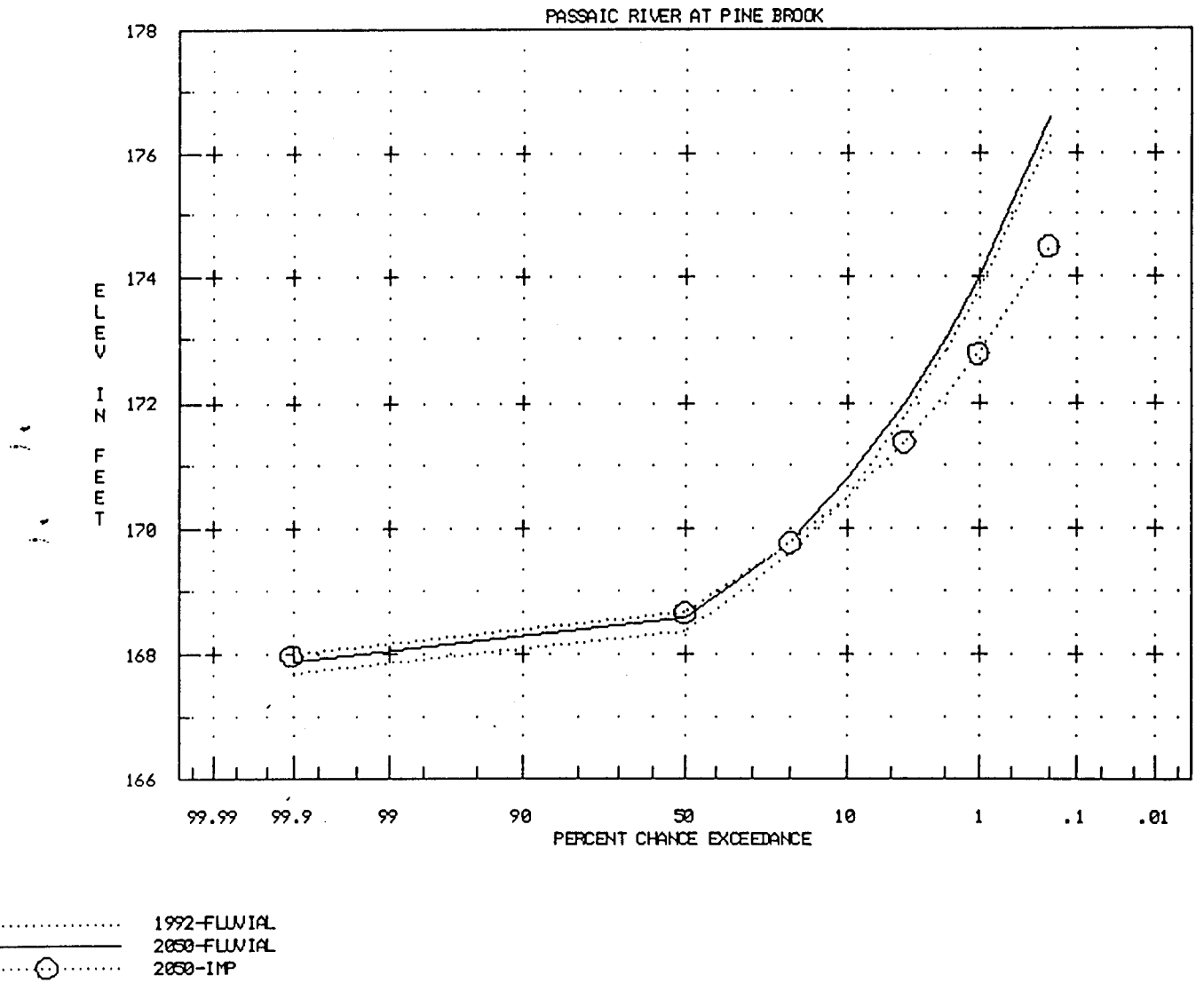


Figure C-66b

06SEP95 13:39:17

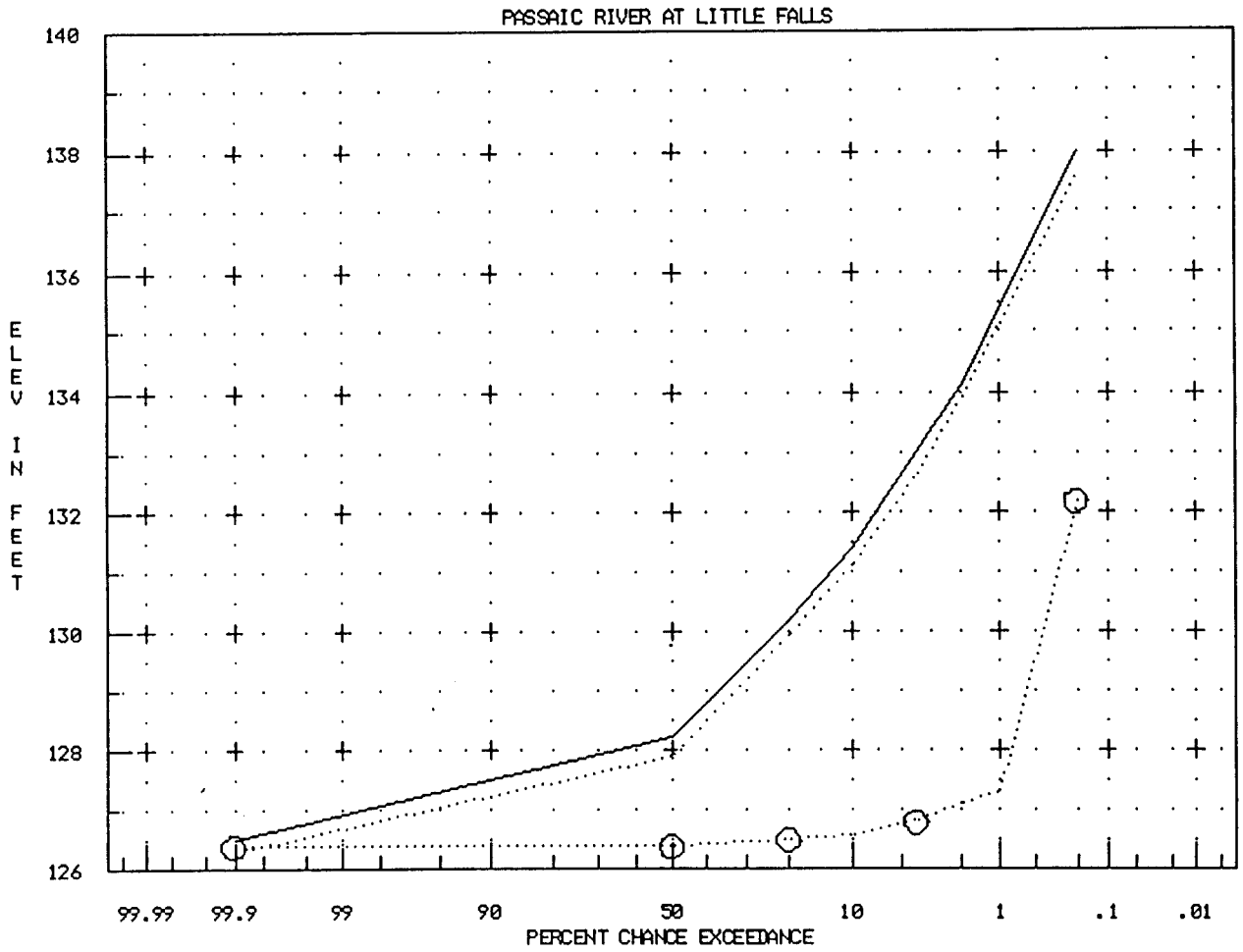


Figure C-66a

Link-Node Grid of Study Area

(Note: Numbers Identify Nodal Points)

Cartography by EBASCO Environmental
May 1992

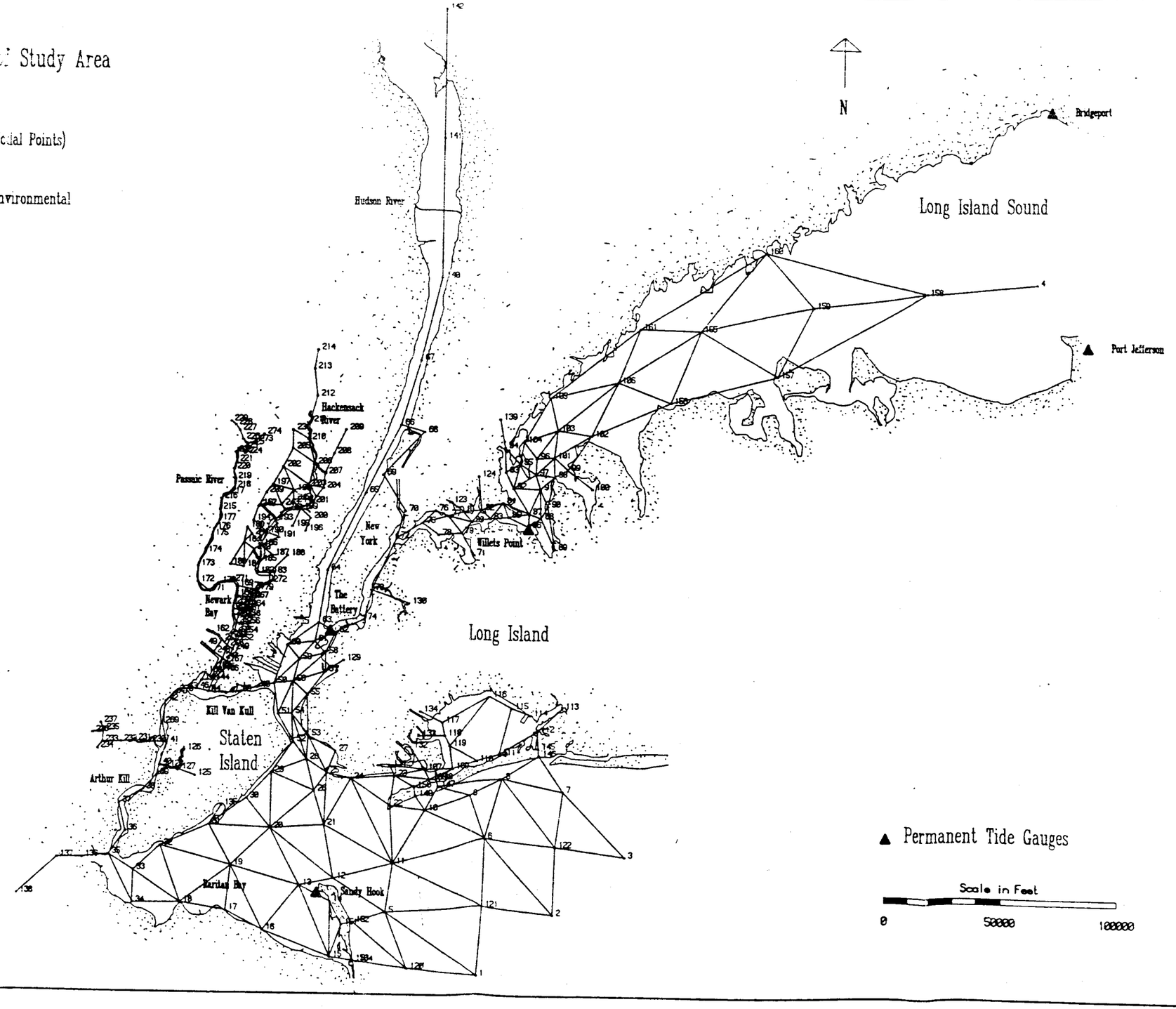


Figure C-67

Detail of Link-Node Grid
Newark Bay Area

(Note: Numbers Identify Nodal Points)

Cartography by
EBASCO Environmental
May 1992

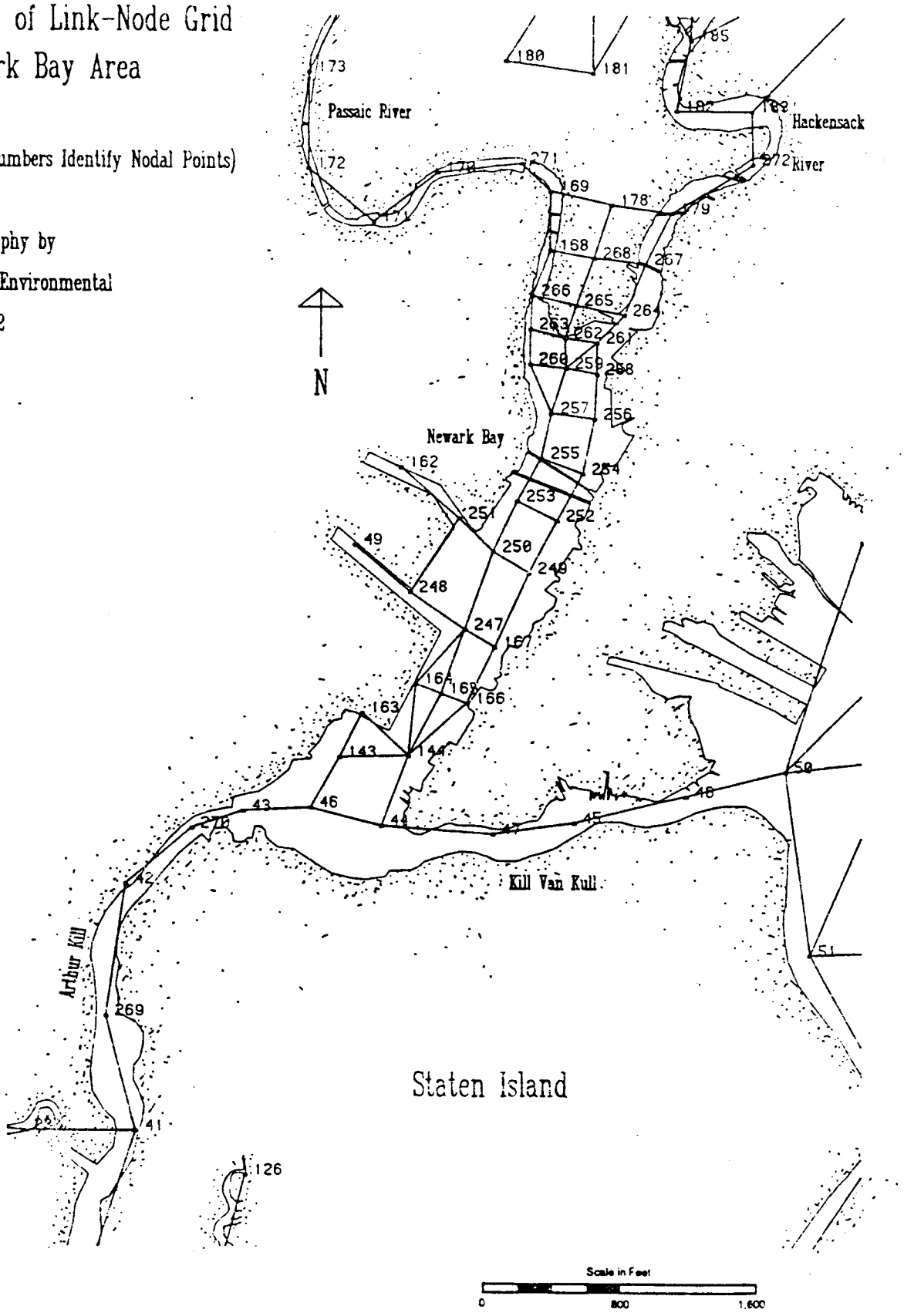


Figure C-68

Tidal heights at New
Brunswick for tide of 11-13 October 1976

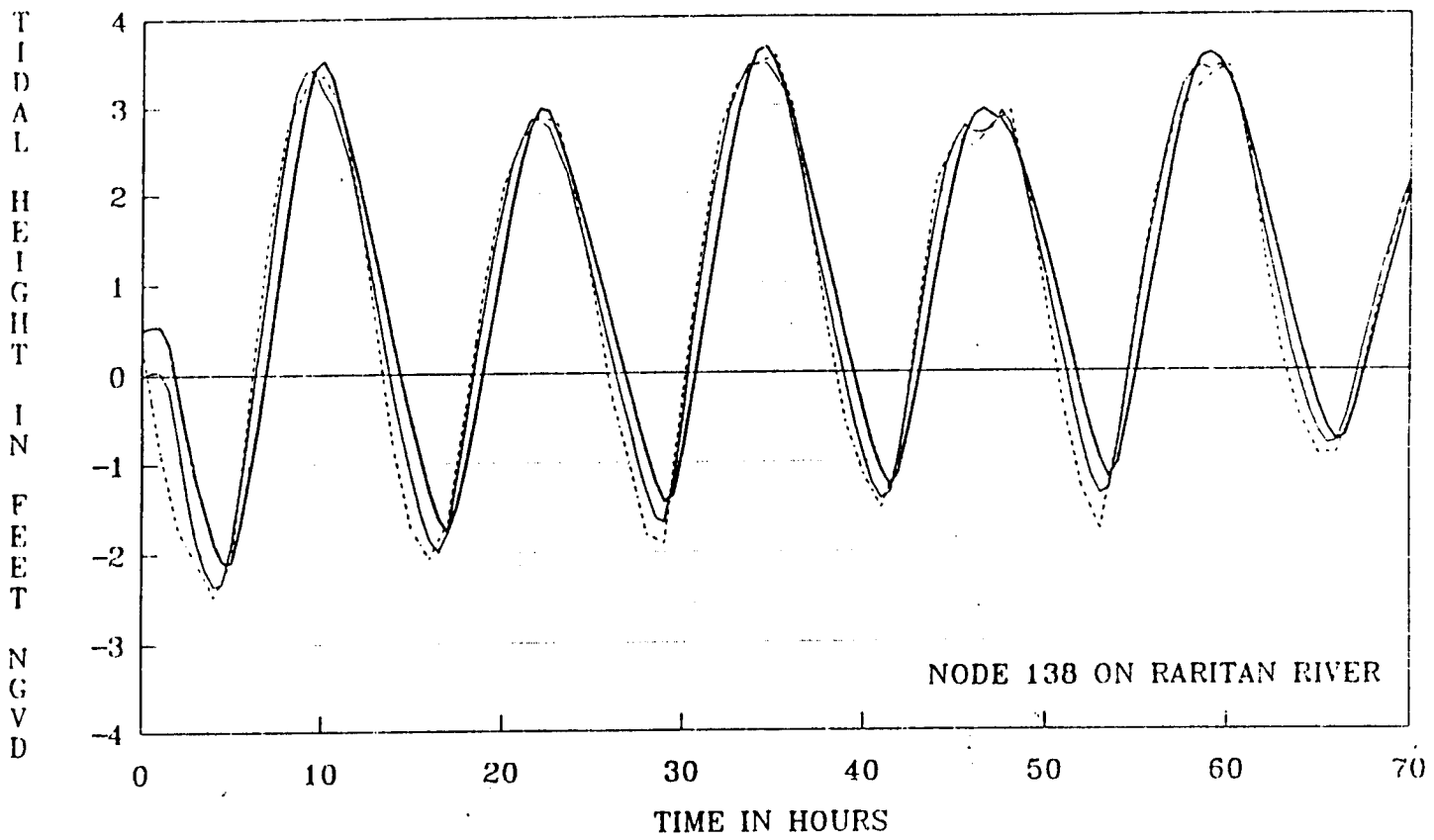


Figure C-69a

Tidal heights at Port Ivory
for tide of 11-13 October 1976

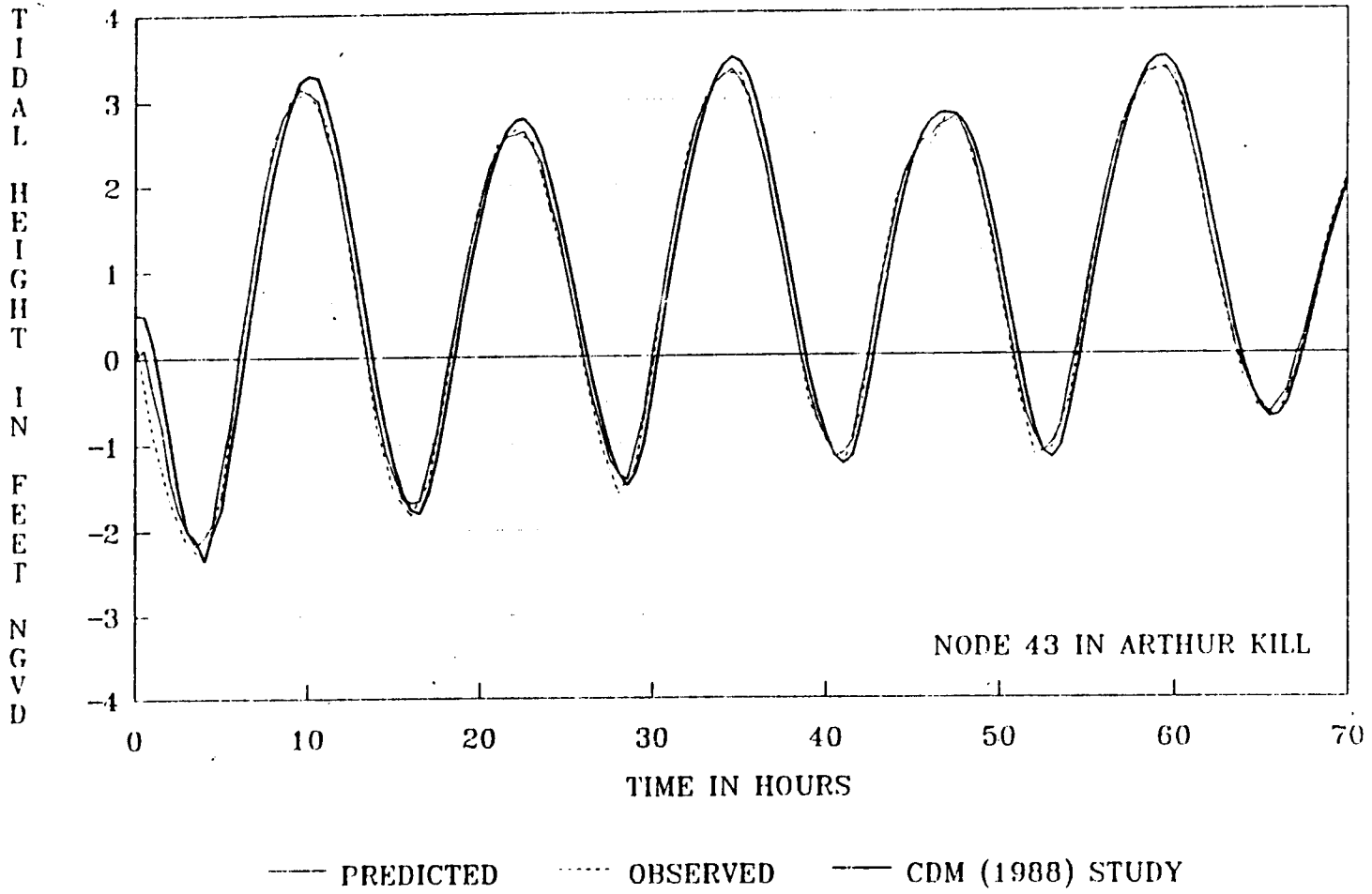


Figure C-69b

Tidal heights at Point No
Point for tide of 11-13 October 1976

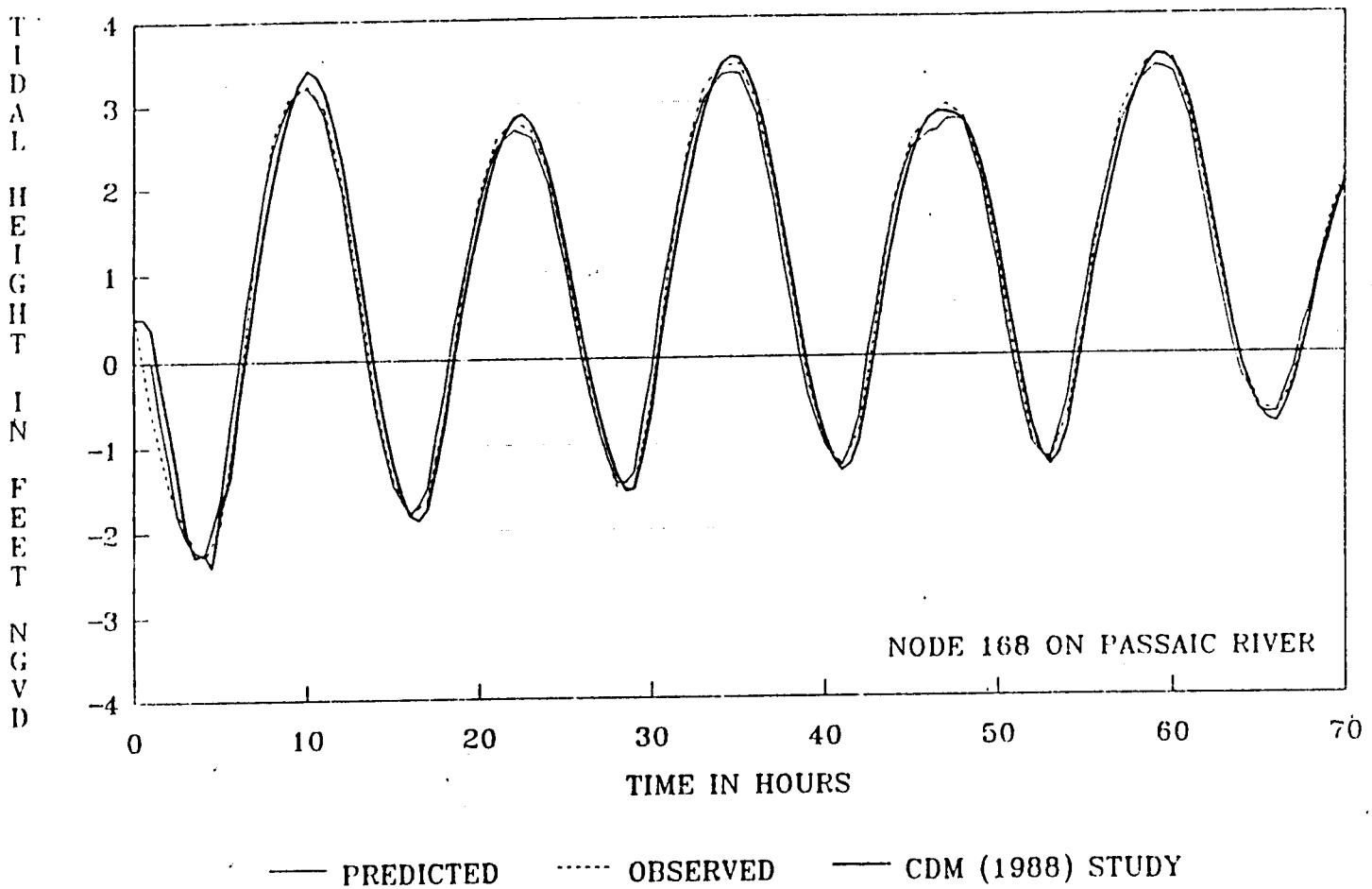


Figure C-69c

Tidal heights at Belleville
for tide of 11-13 October 1976

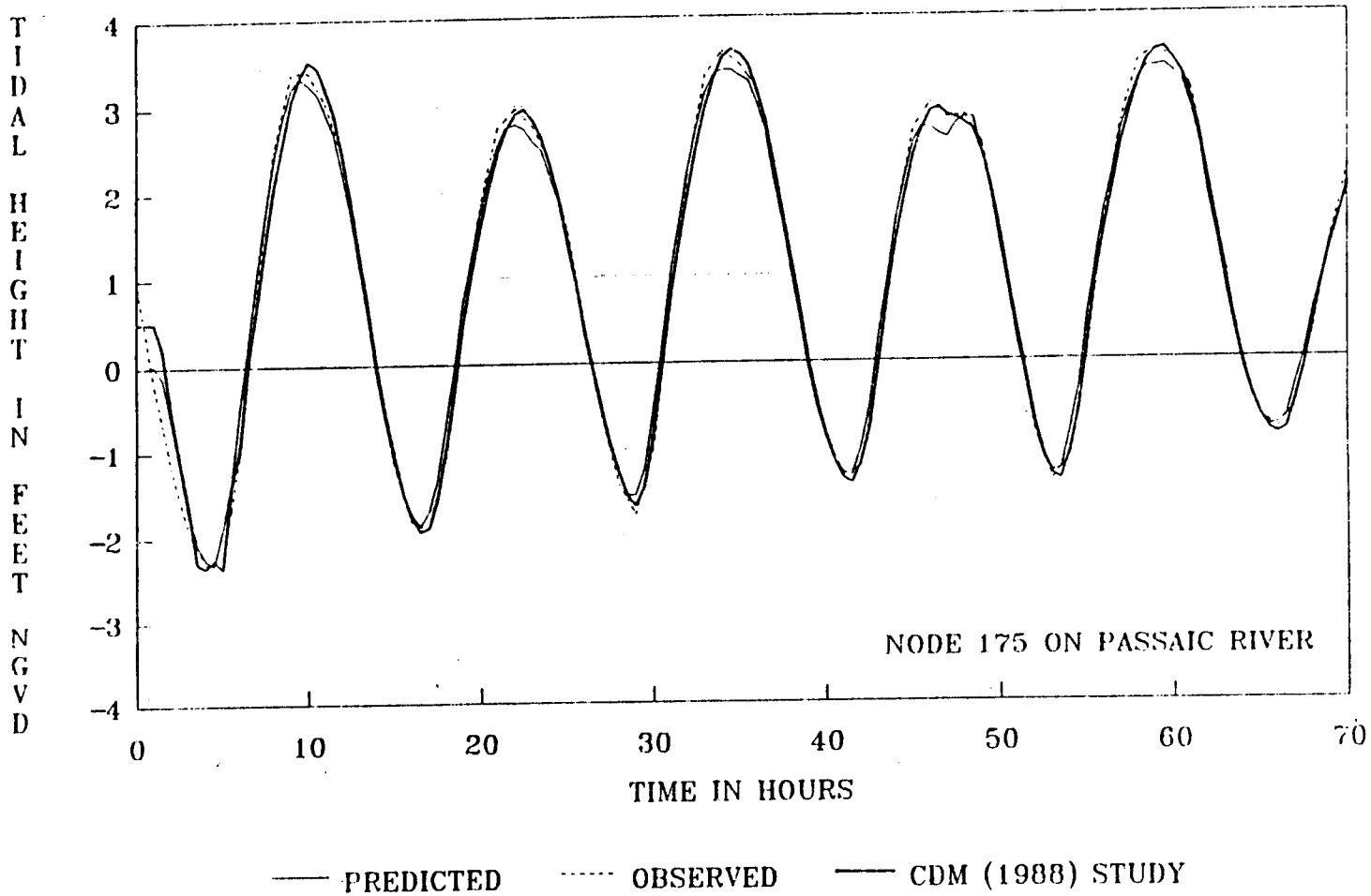


Figure C-69d

Tidal heights at Garfield
for tide of 11-13 October 1976

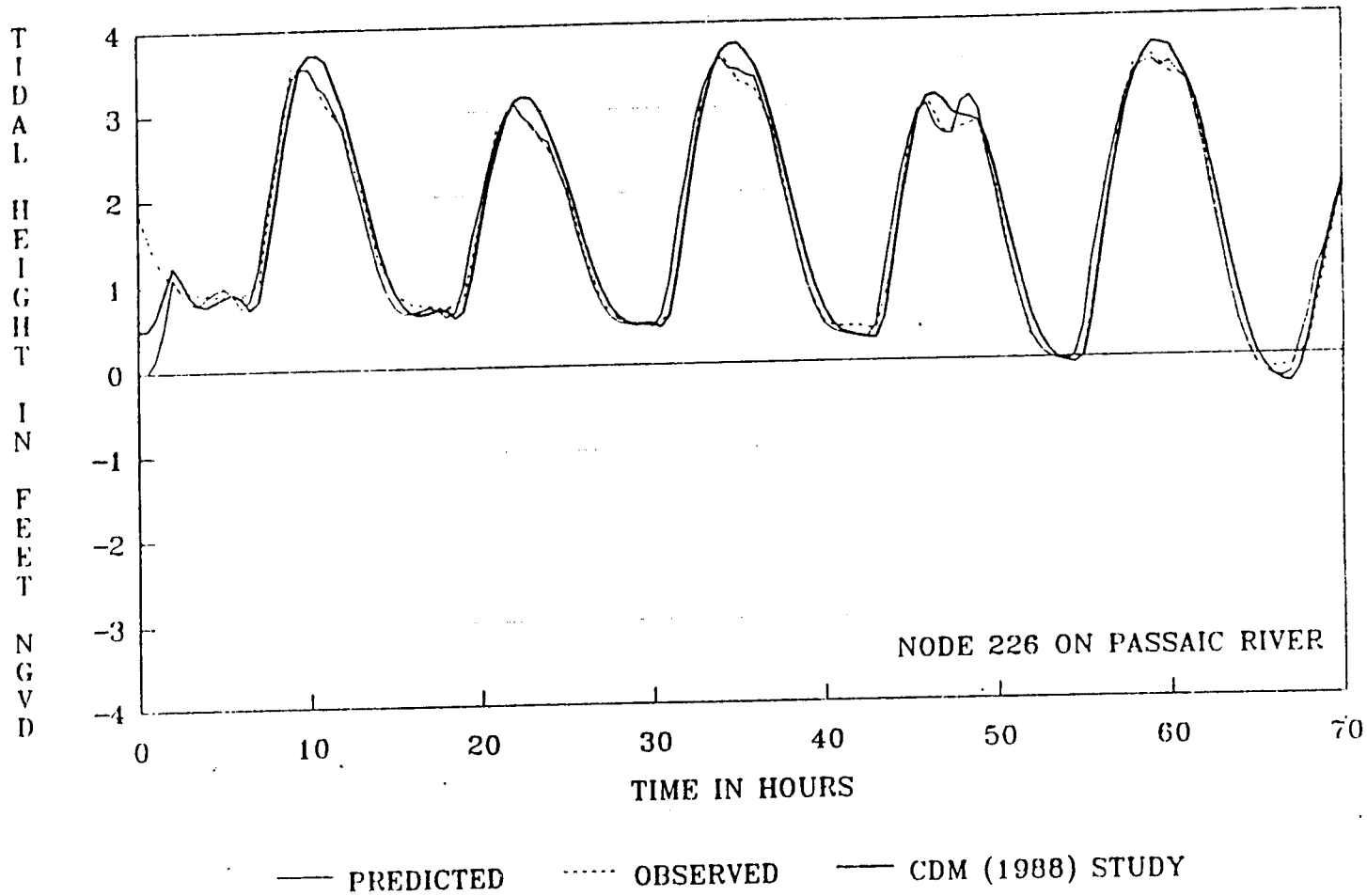


Figure C-66e

Tidal heights at Kearny
Point for tide of 11-13 October 1976

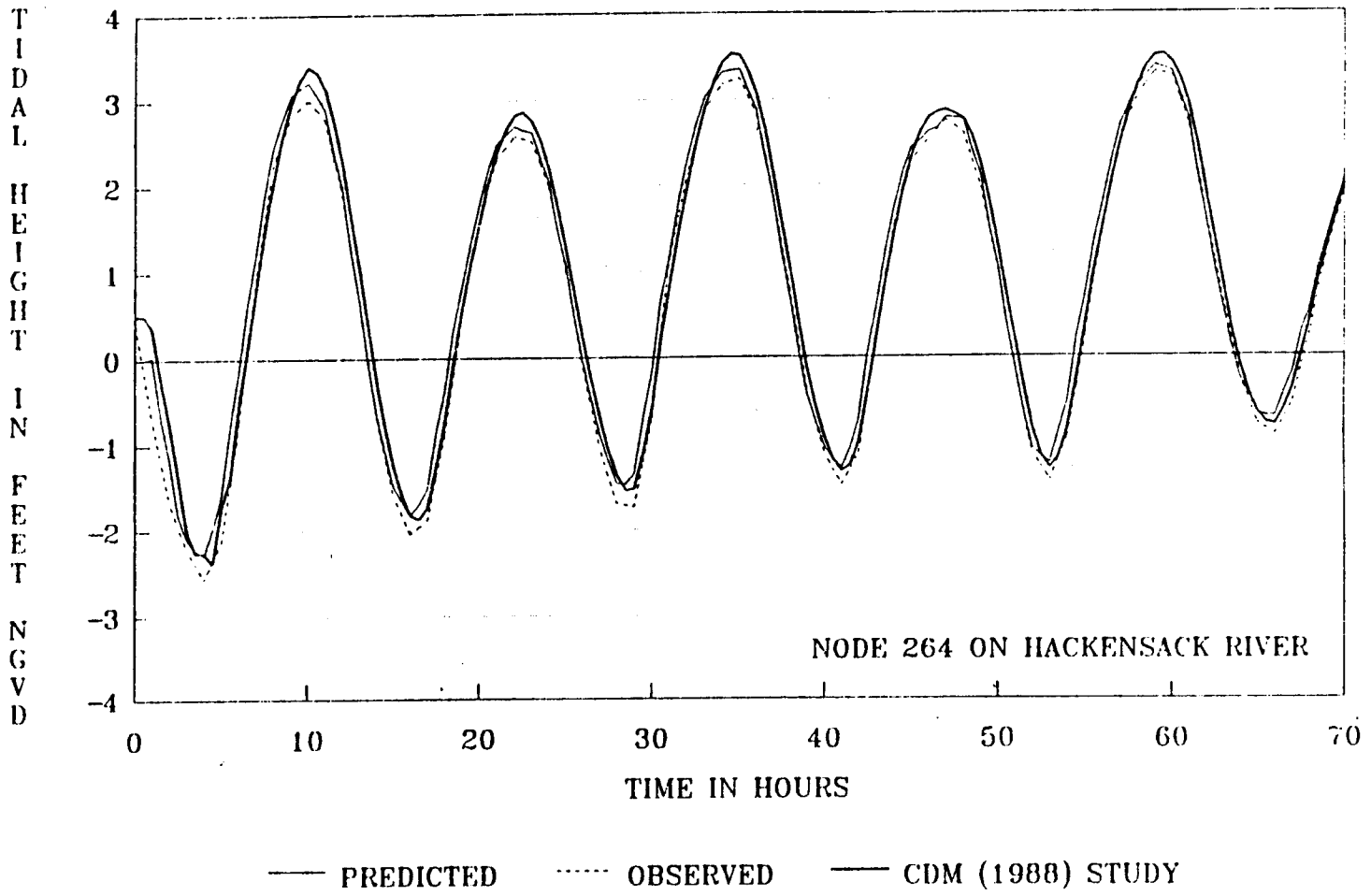


Figure C-69f

Tidal heights at Belleville
Tpke for tide of 11-13 October 1976

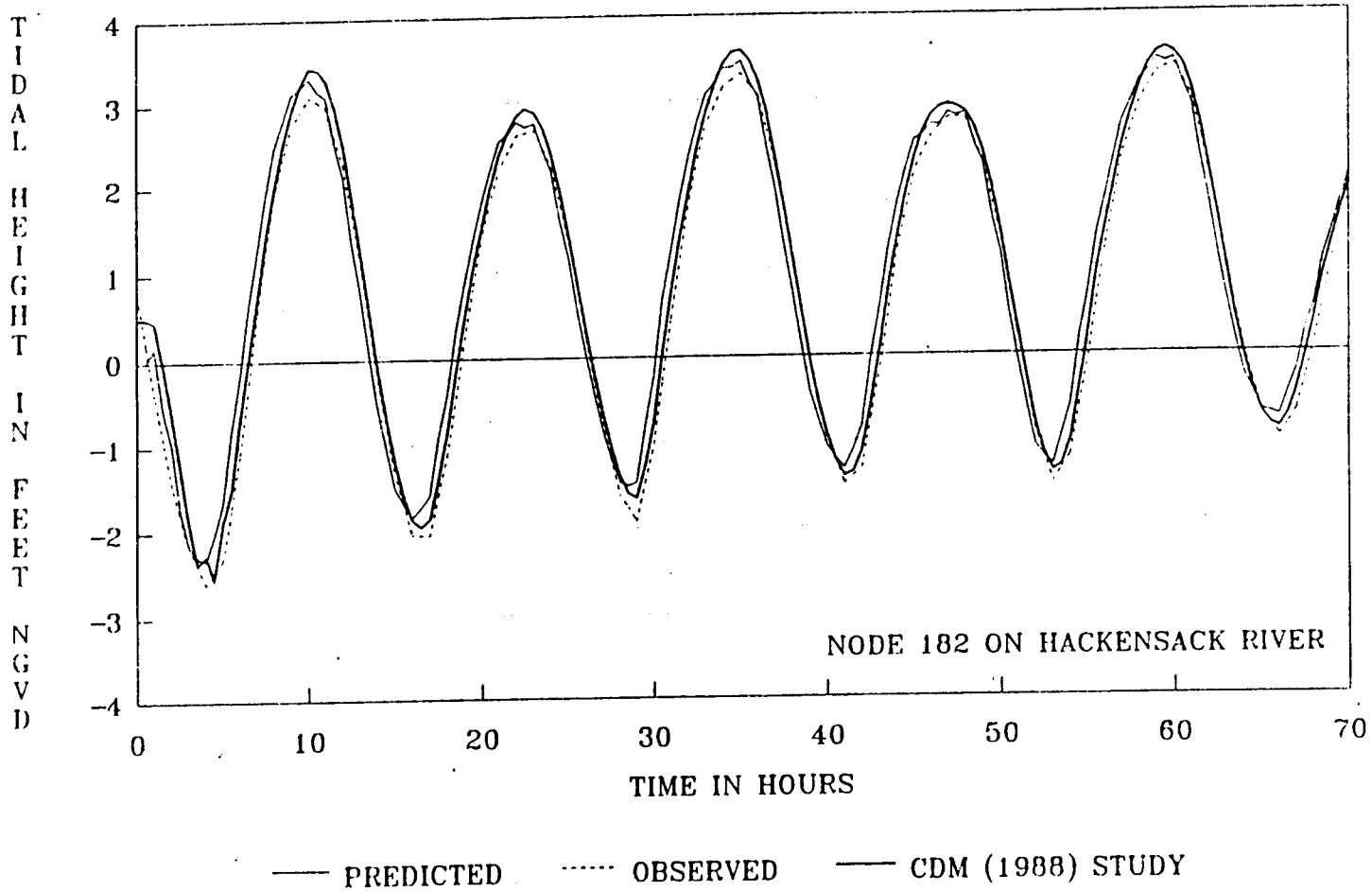


Figure C-699

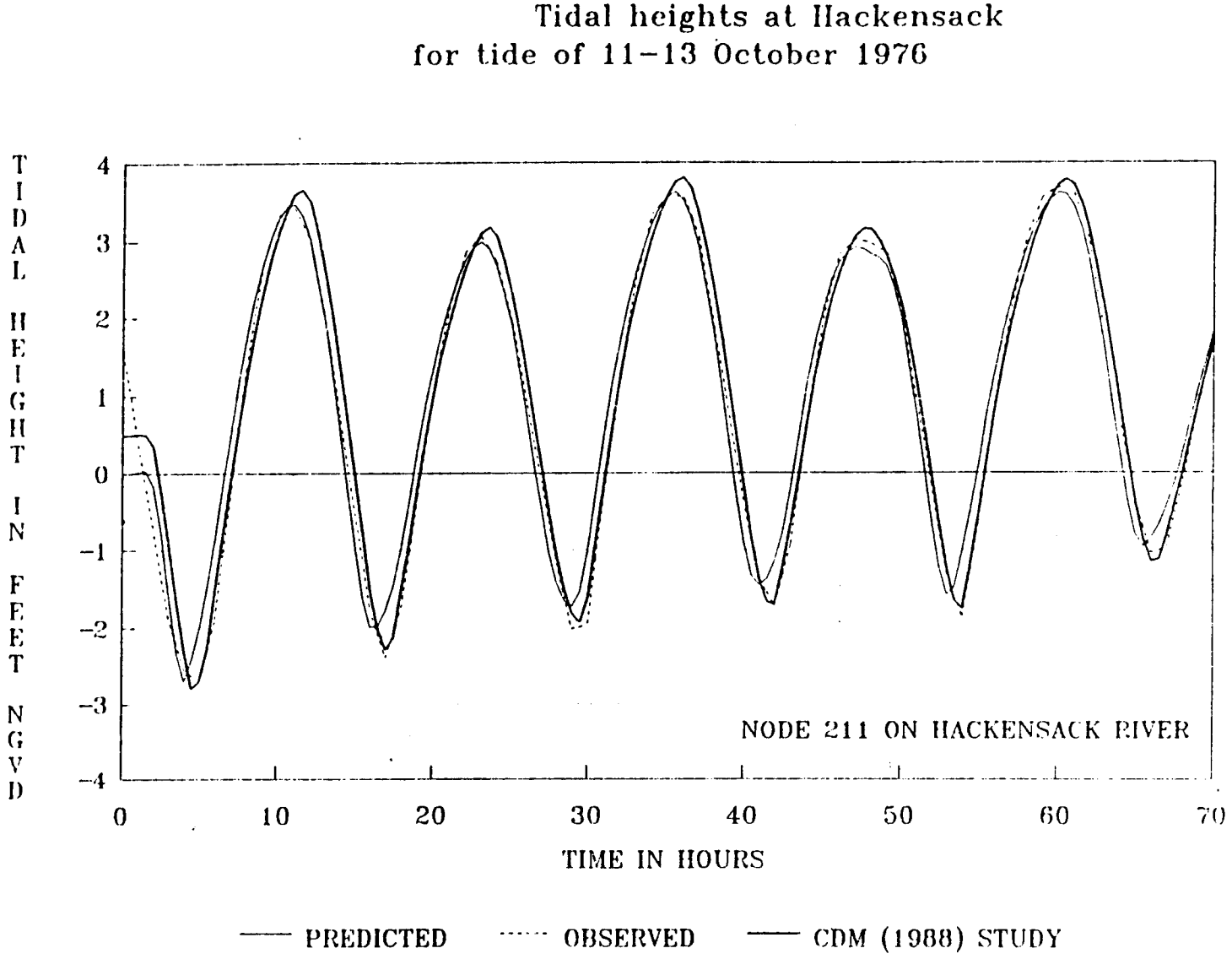

Tidal heights at Hackensack
for tide of 11-13 October 1976
TIDAL HEIGHT IN FEET NGVD
4
3
2
1
0
-1
-2
-3
-4
0
10
20
30
40
50
60
70
TIME IN HOURS
NODE 211 ON HACKENSACK RIVER
PREDICTED
OBSERVED
CDM (1988) STUDY

Figure C-69h

Tidal heights at Elm Park
for hurricane of 21 September 1938

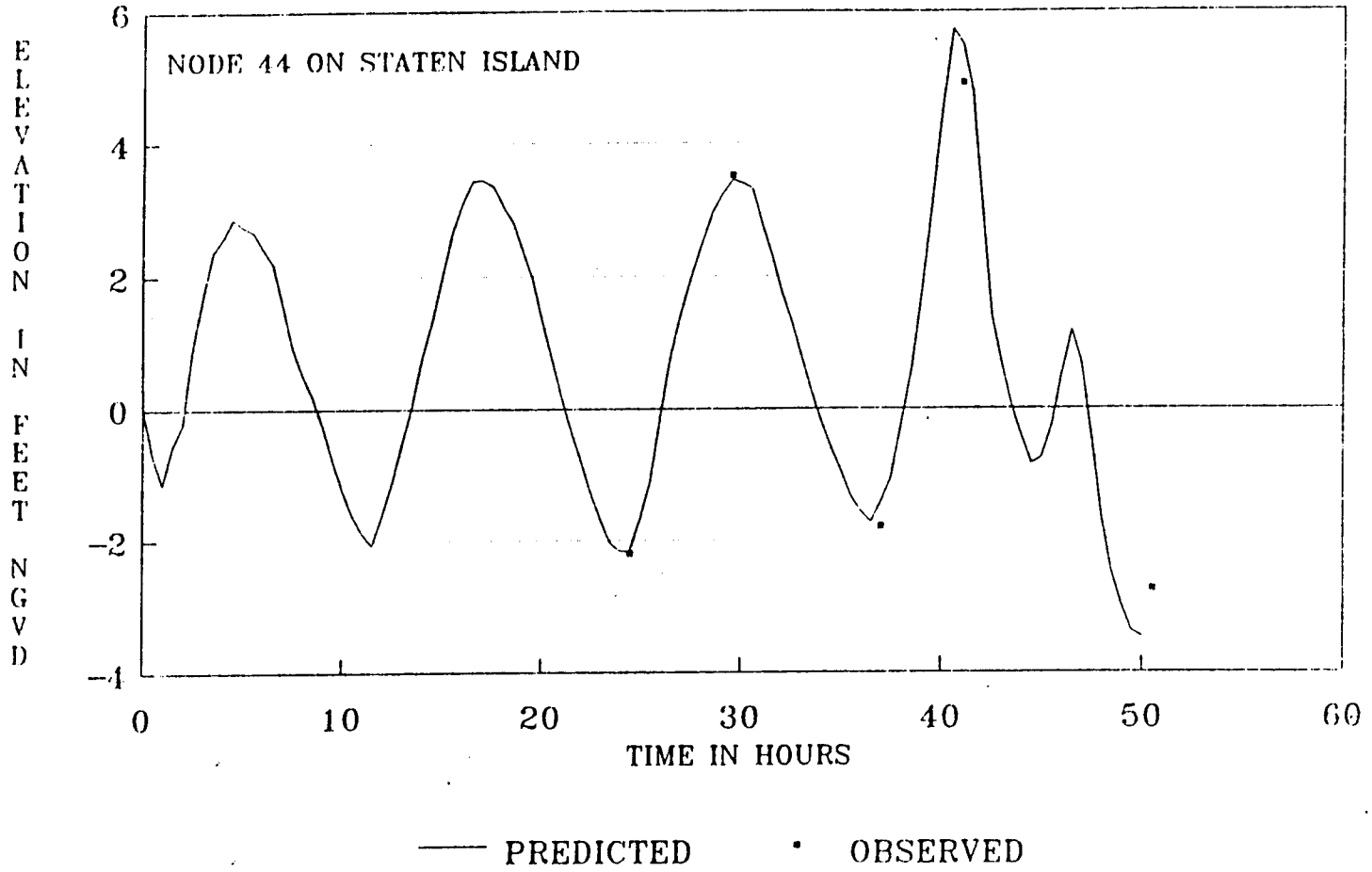


Figure C-70a

Tidal heights at East Newark
for hurricane of 21 September 1938

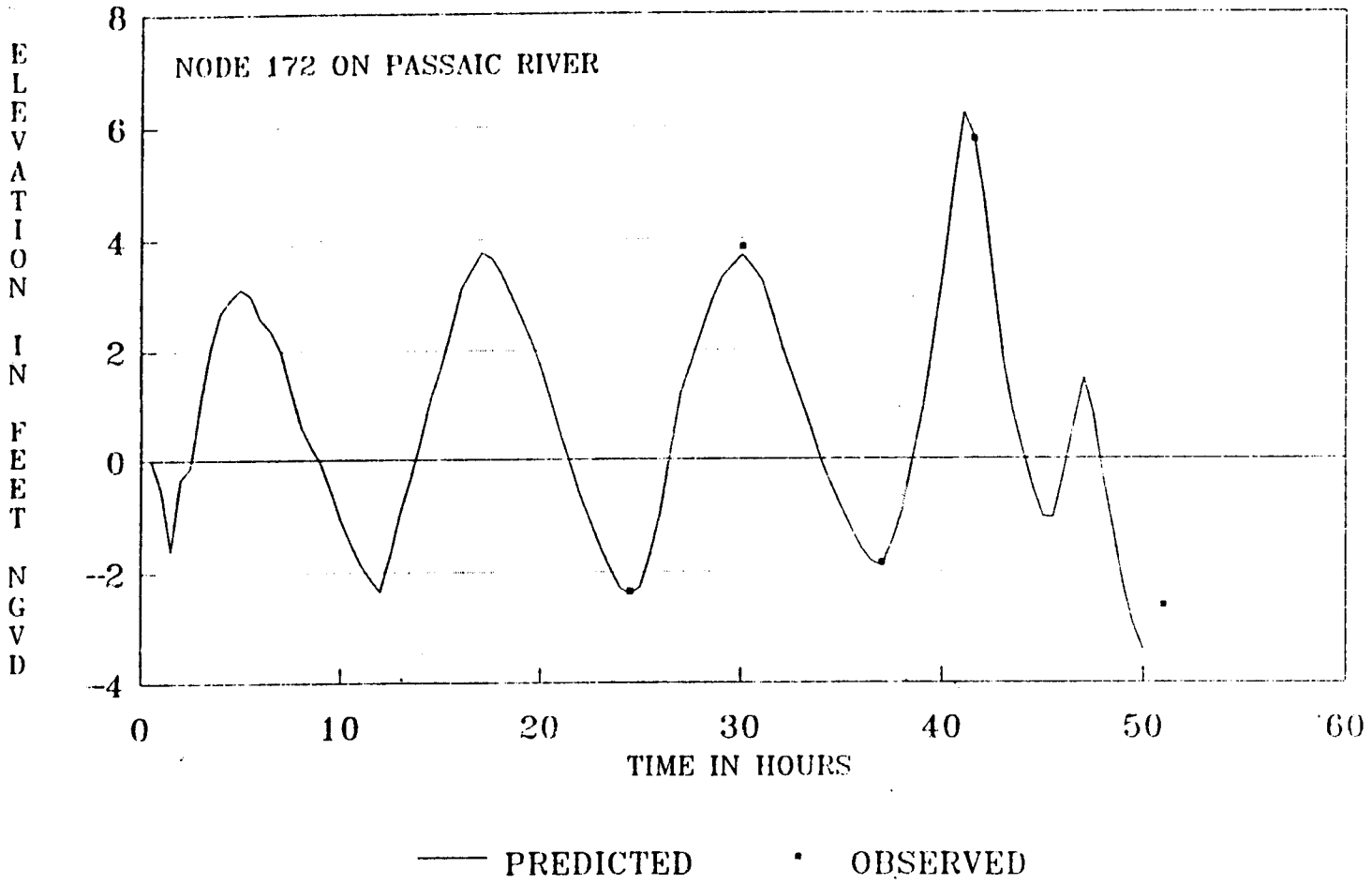


Figure C-70b

Tidal heights at Fort Hamilton for hurricane of 12 September 1960

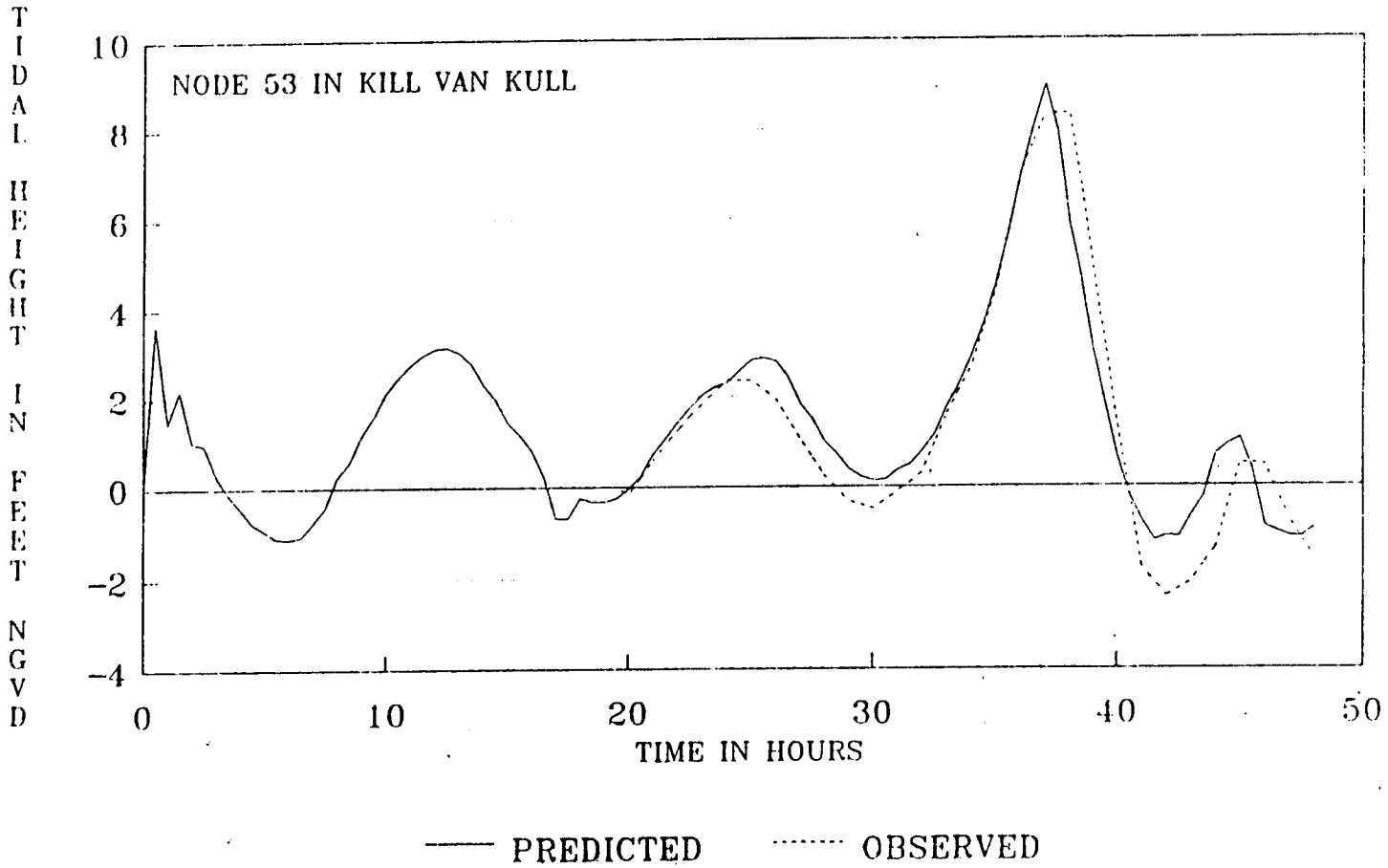


Figure C-71a

Tidal heights at Elm Park
for hurricane of 12 September 1960

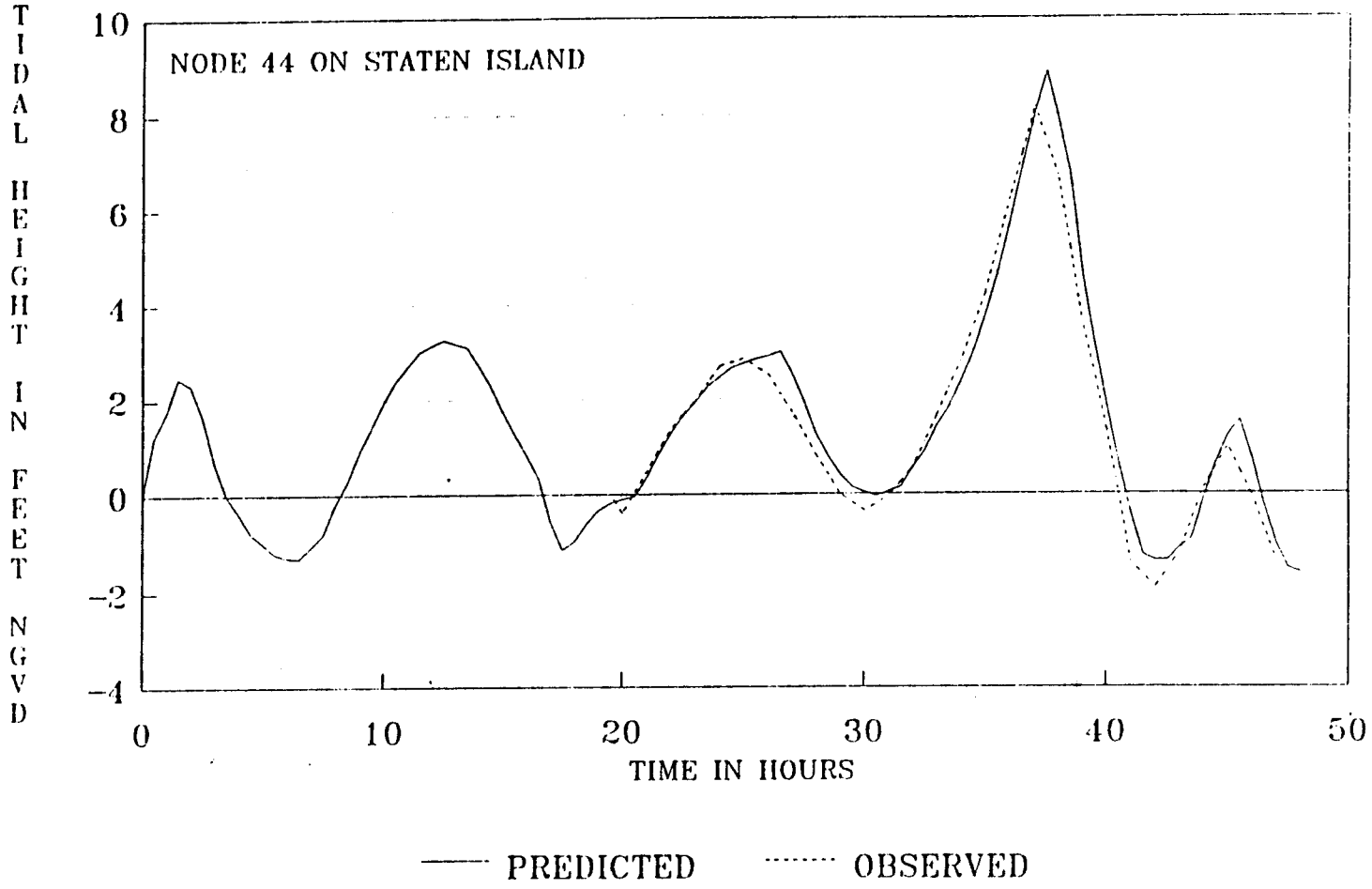


Figure C-71b

Tidal heights at Carteret
for hurricane of 12 September 1960

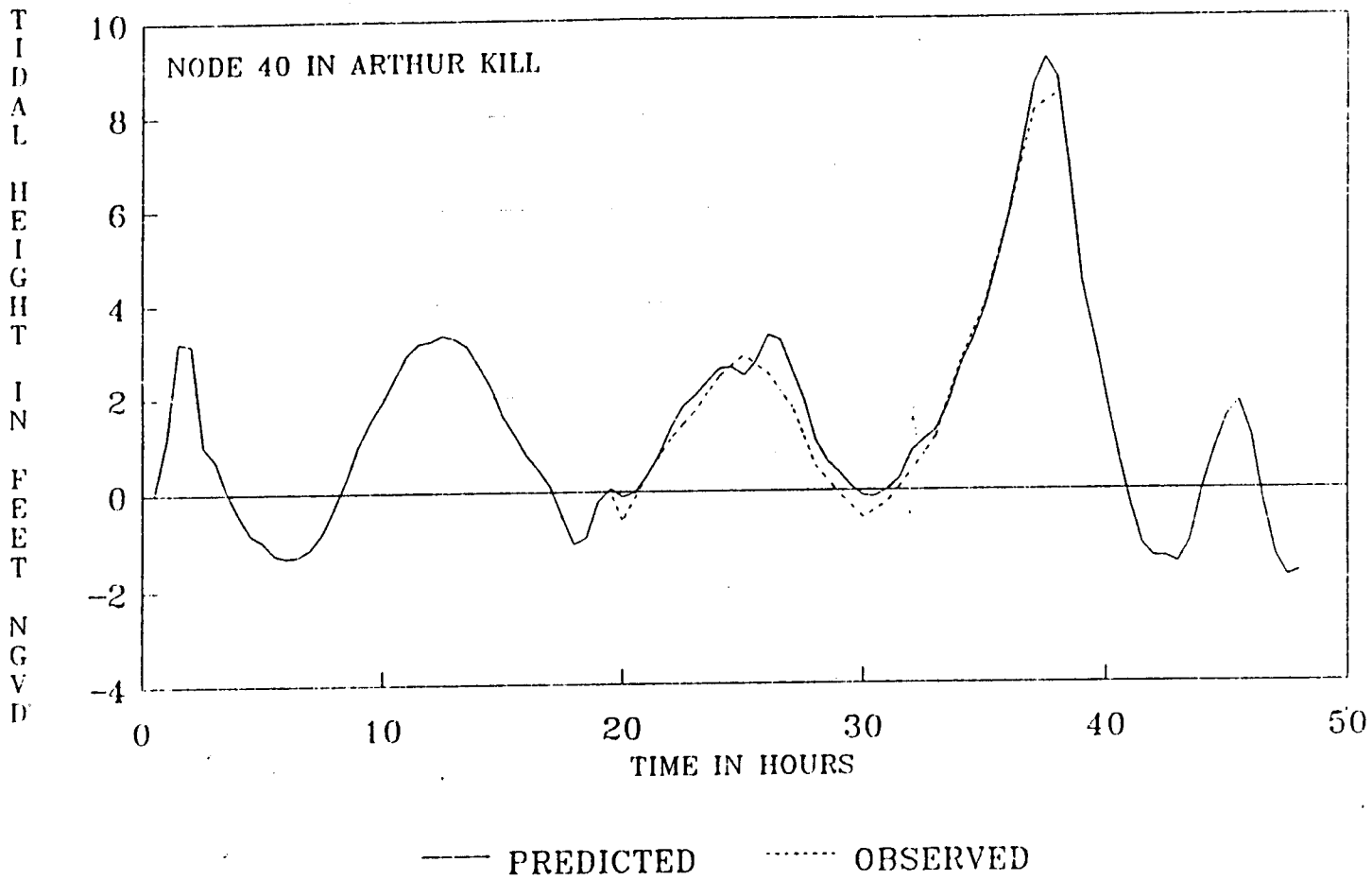
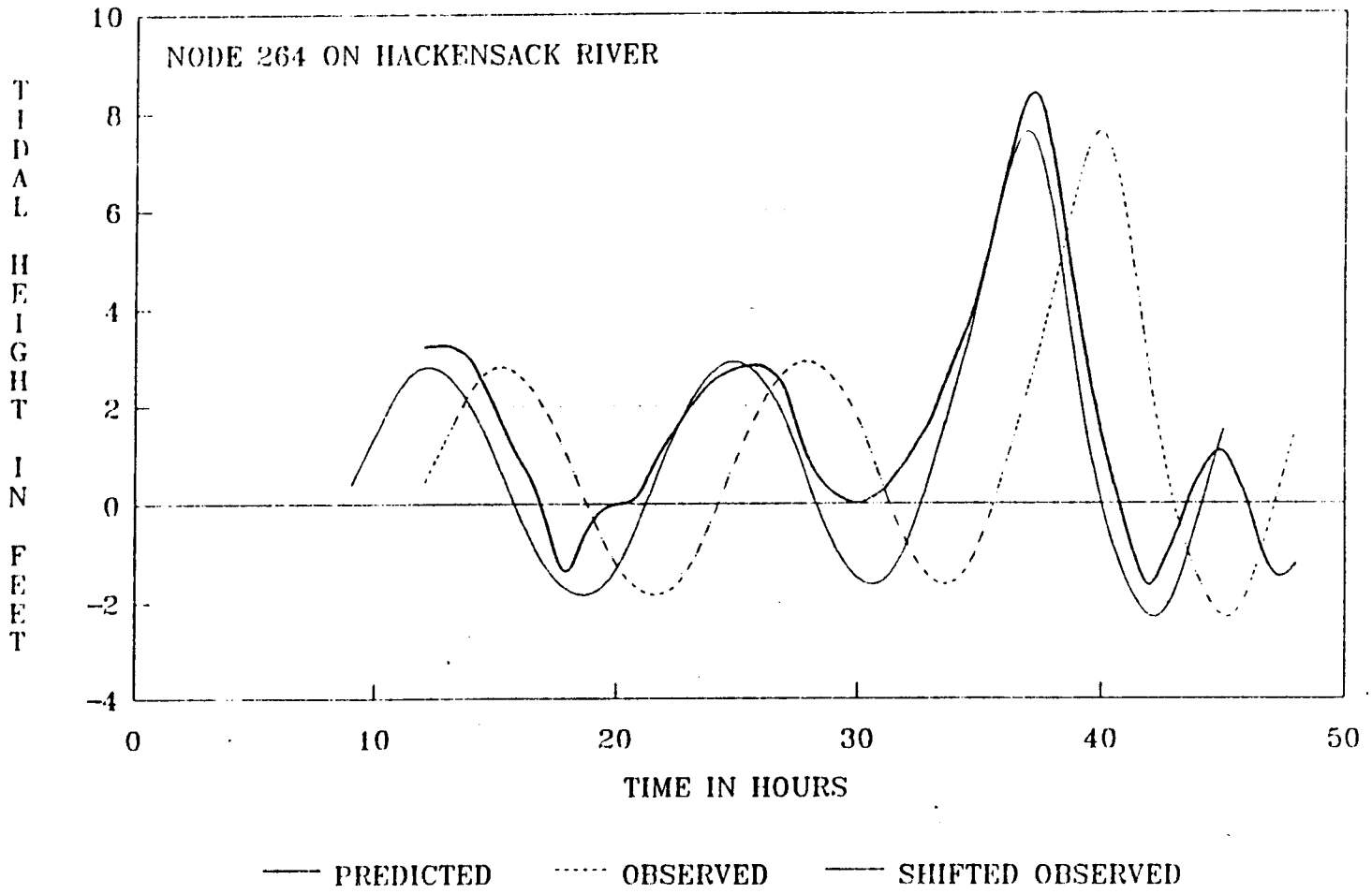


Figure C-71c

Tidal heights at Kearny Point for hurricane of September 1960



Note: Observational values were obtained from a tidal height curve measured at the mouth of the Hackensack River. The time origin for the observations was not specified in terms of time-of-day. The SHIFTED OBSERVED line is the curve obtained by shifting the time index of the observations by 3 hours so that the peaks at approximately hour 37 coincide.

Figure C-7.1d

Tidal heights at Elm Park
for northeaster of 24–26 November 1950

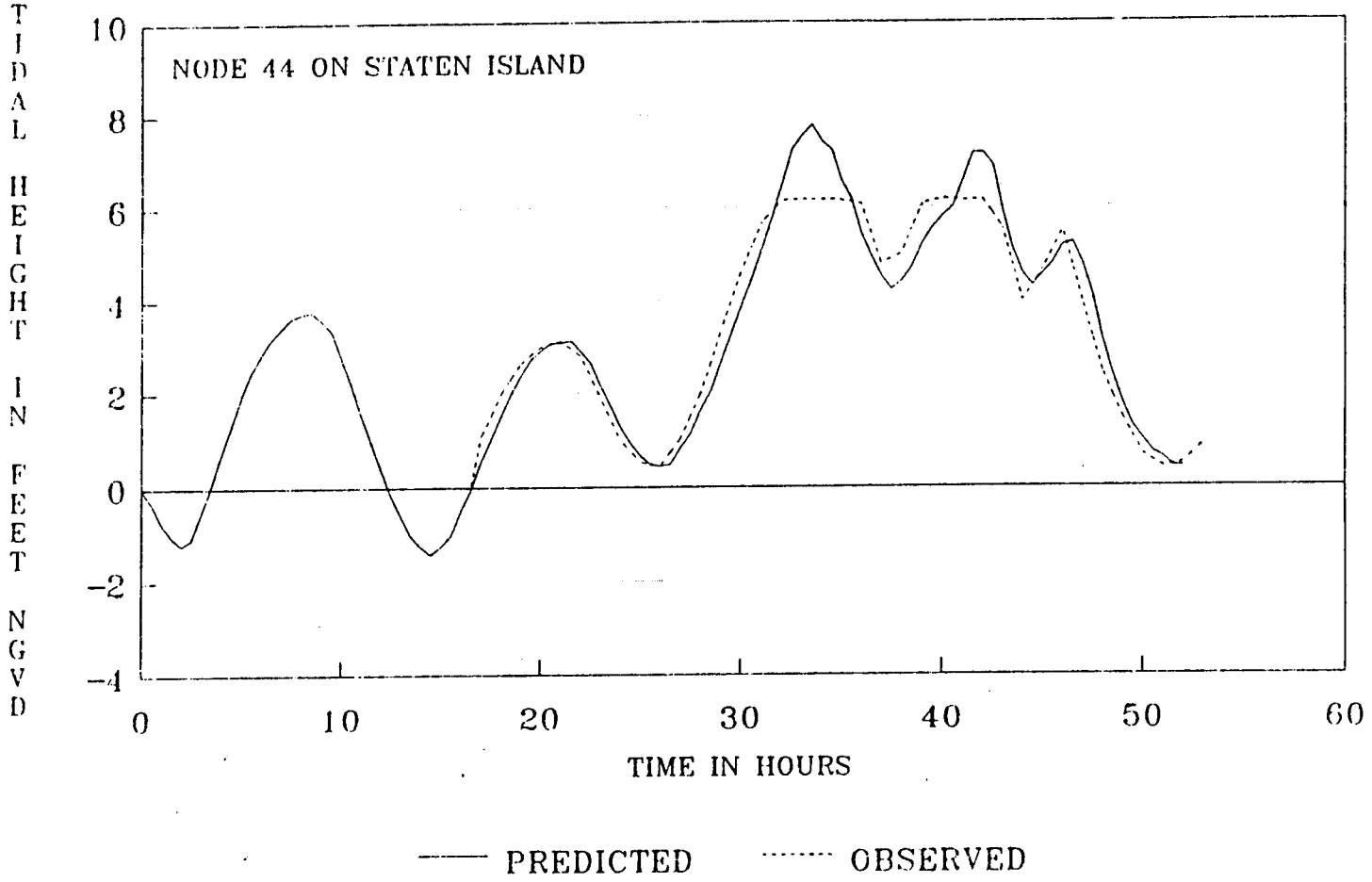


Figure C-72

Tidal heights at Fort
Hamilton for northeaster of 6–8
November 1953

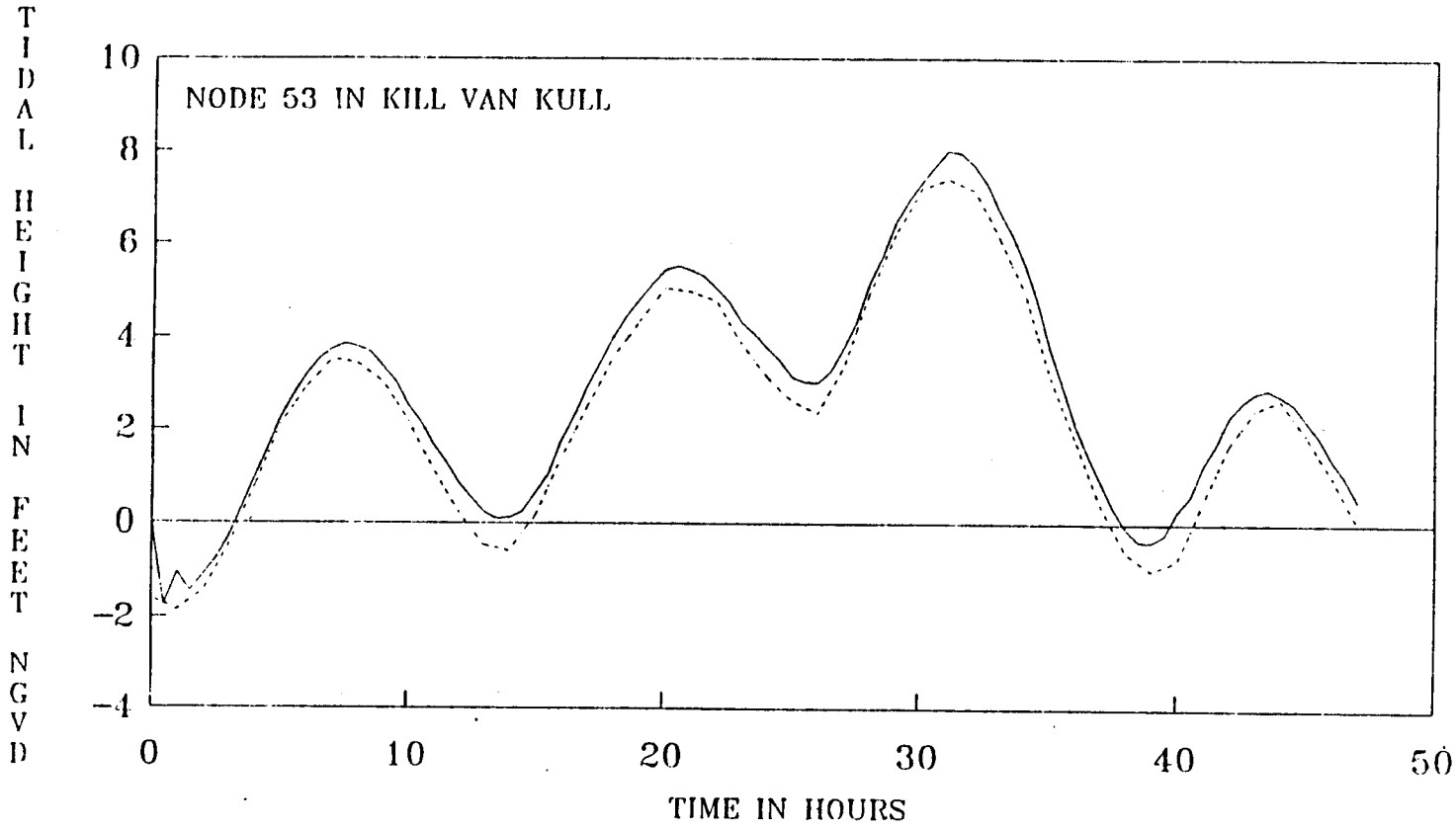


Figure C-73a

Tidal heights at Elm Park
for northeaster of 6–8 November 1953

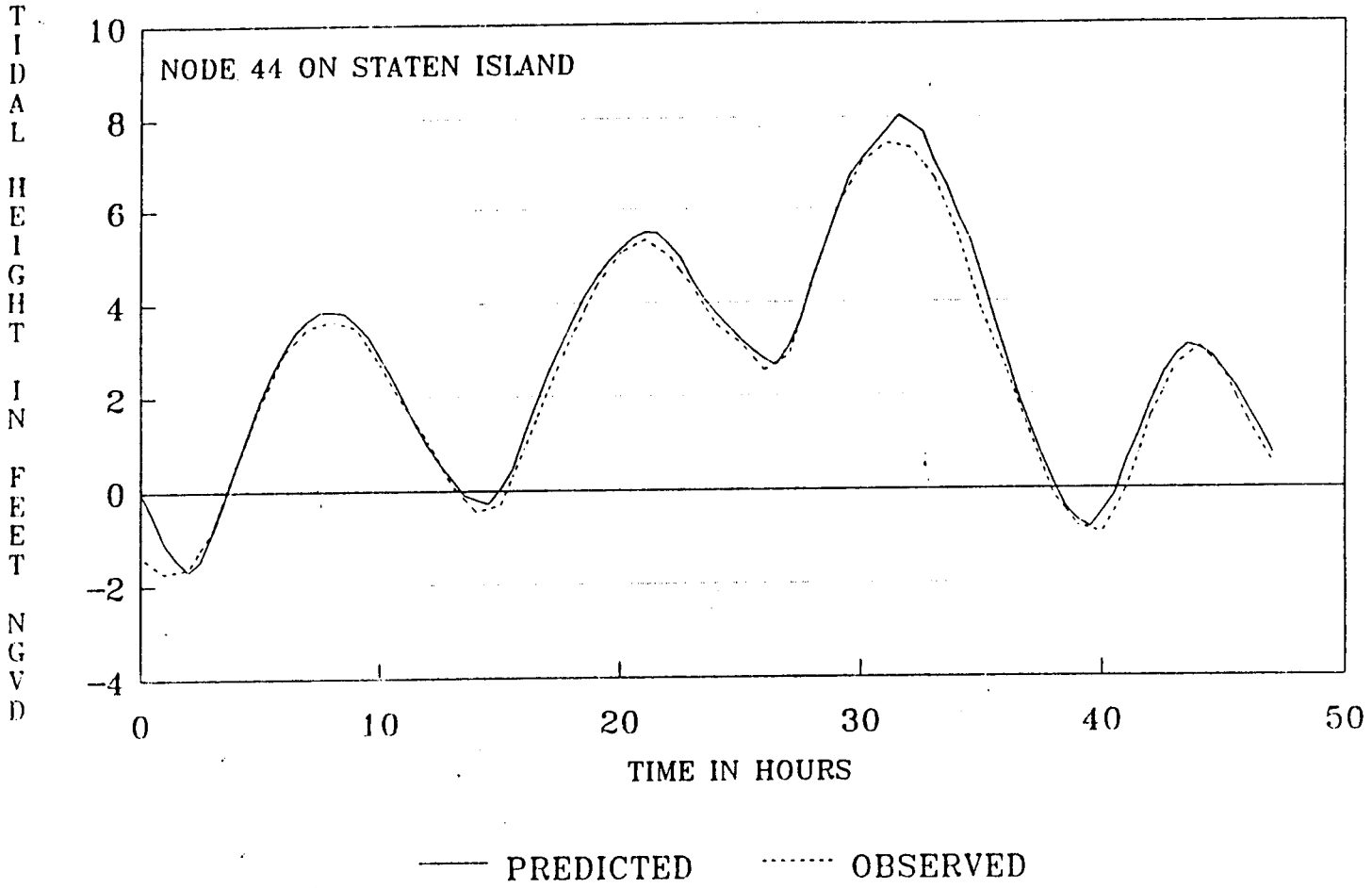


Figure C-73b

Tidal heights at East Newark
for northeaster of 6-8 November 1953

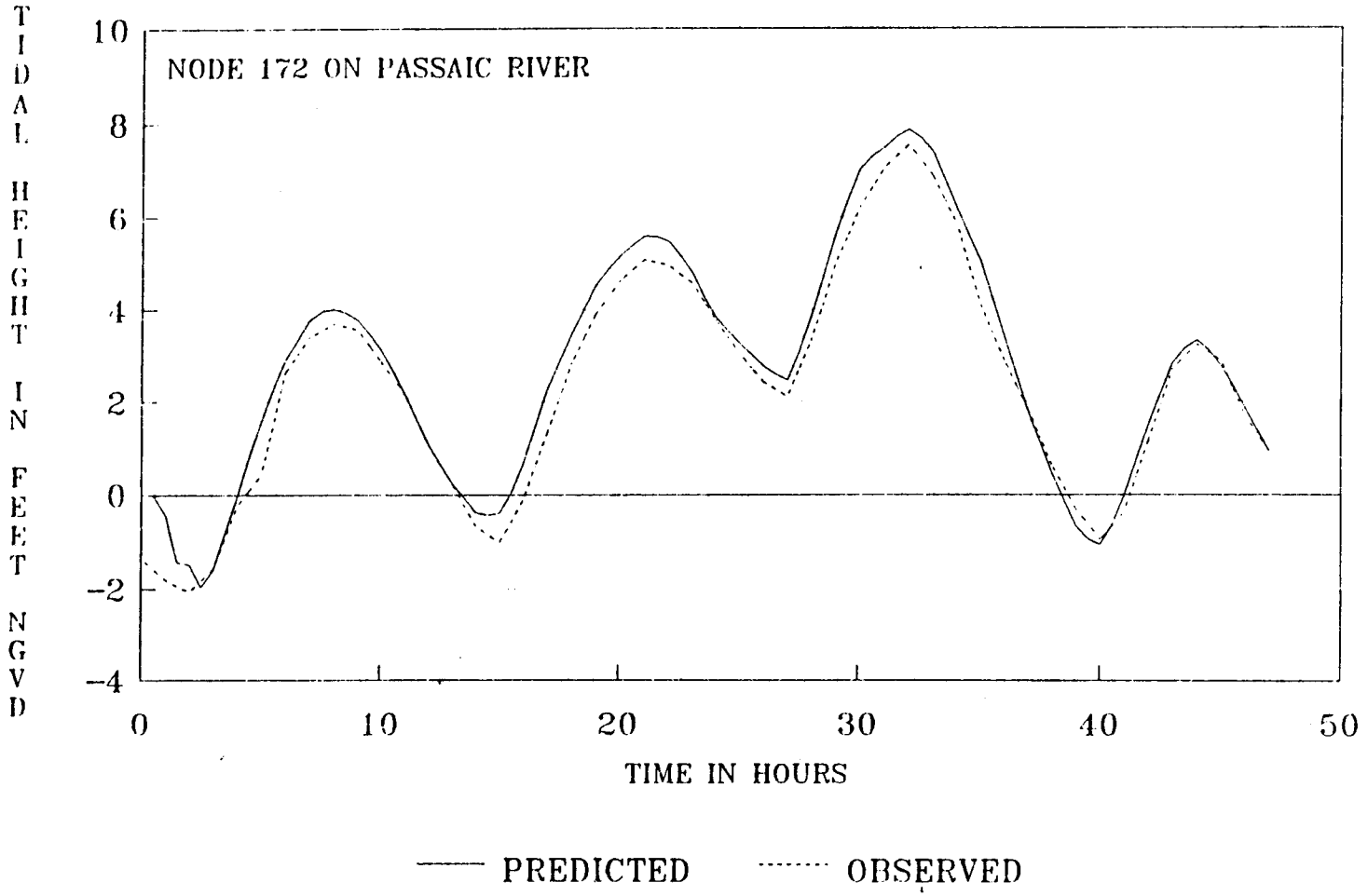


Figure C-73c

Tidal heights at Kearny Point and The Battery for northeaster of 24–26 November 1950

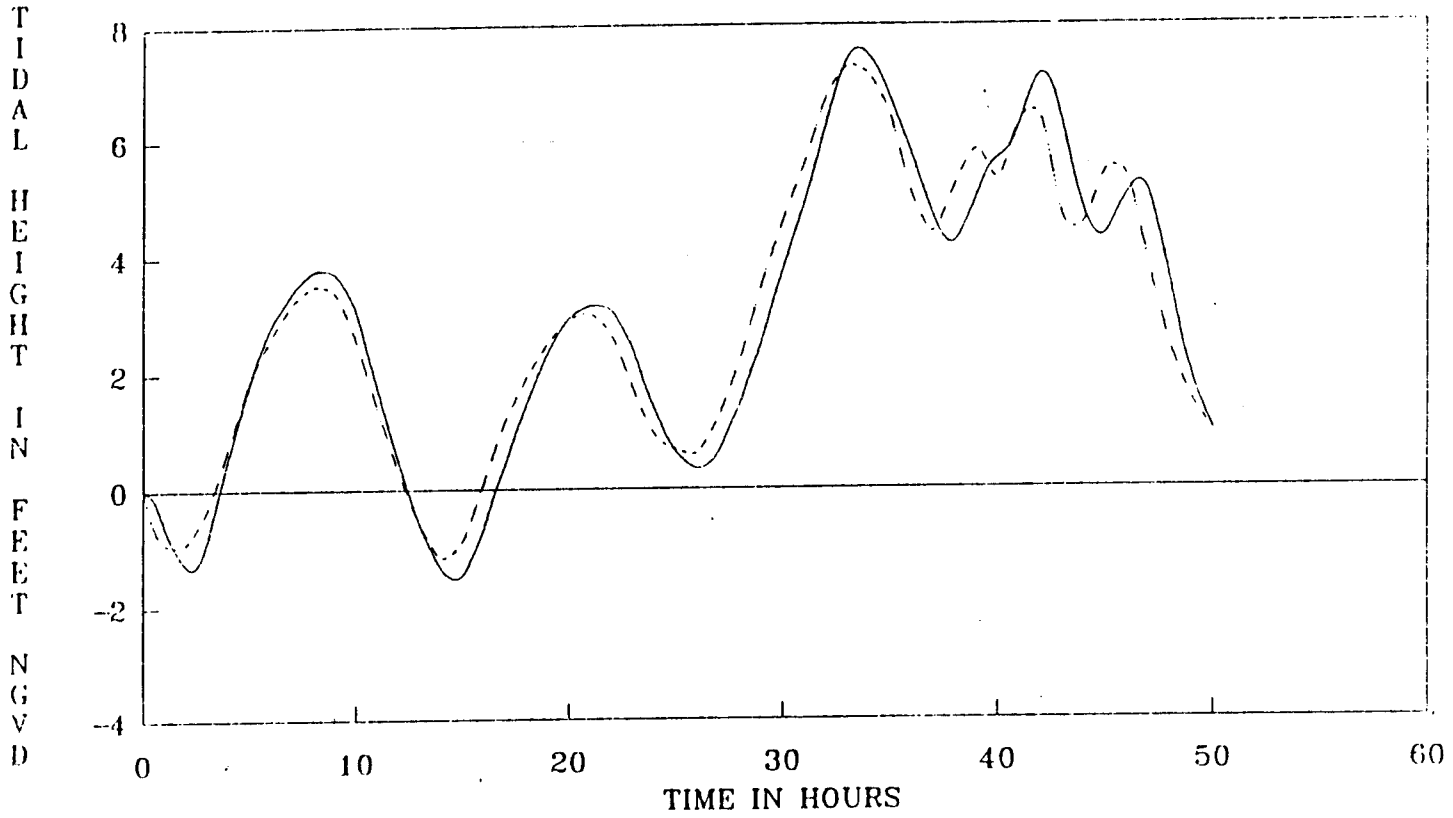


Figure C-74a

Tidal heights at Kearny Point and The Battery for hurricane of 12 September 1960

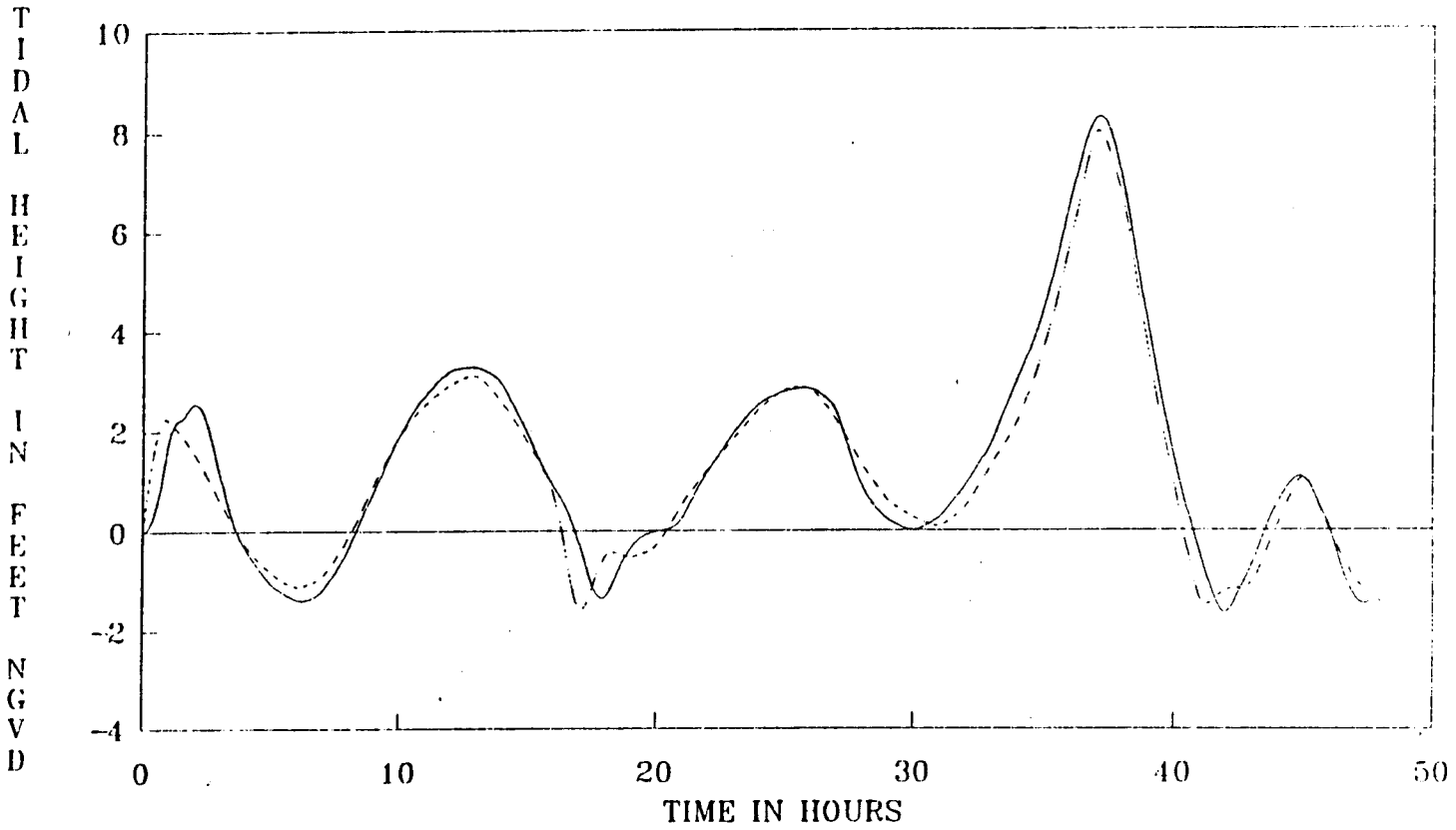


Figure C-74b

Tidal heights at Kearny Point and The Battery for tide of 11–13 October 1976

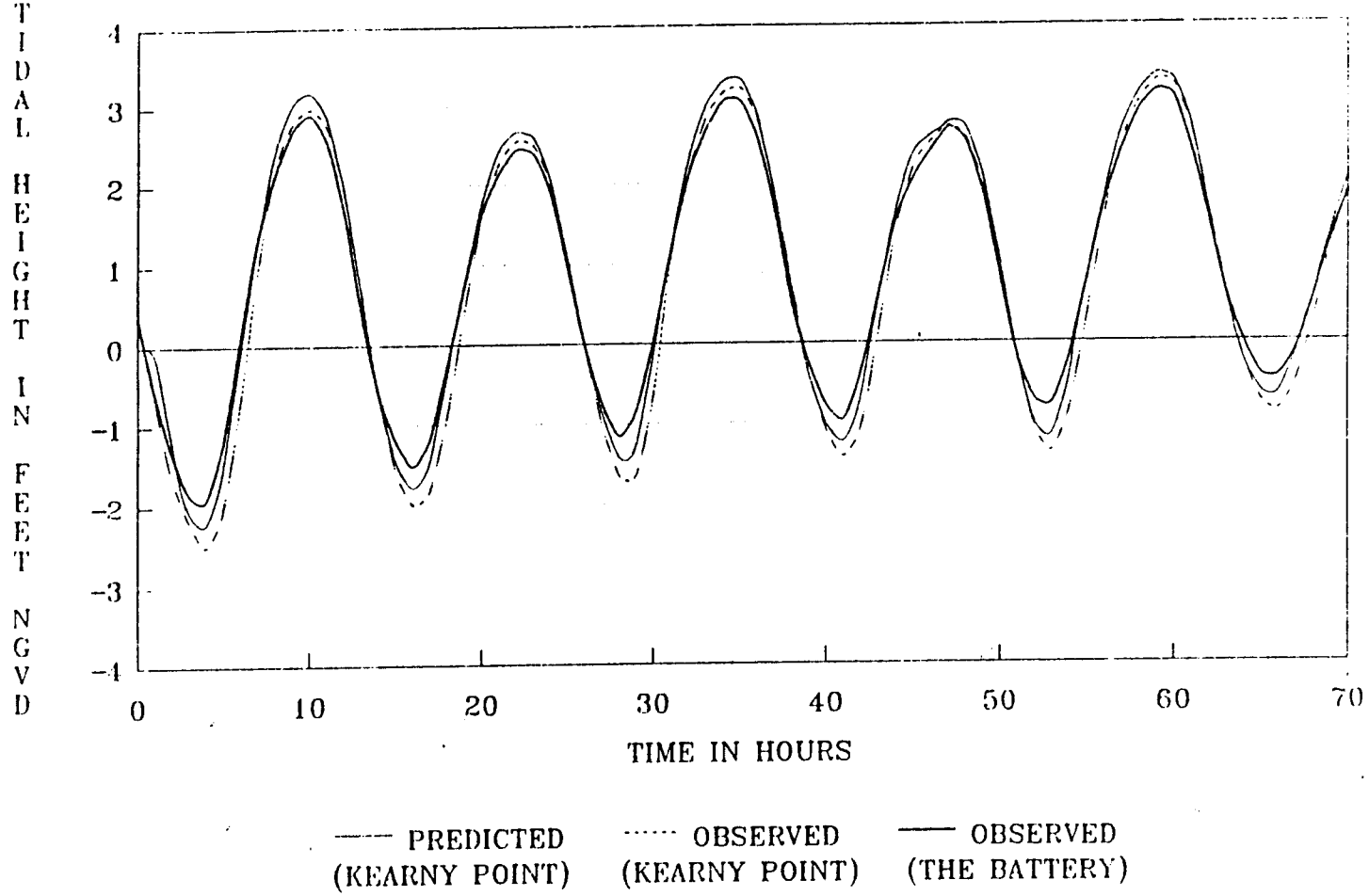


Figure C-74c

Predicted tide height and observed water level at The Battery and flow of the Passaic River measured at the Little Falls, New Jersey gage during the 21 September 1938 storm event

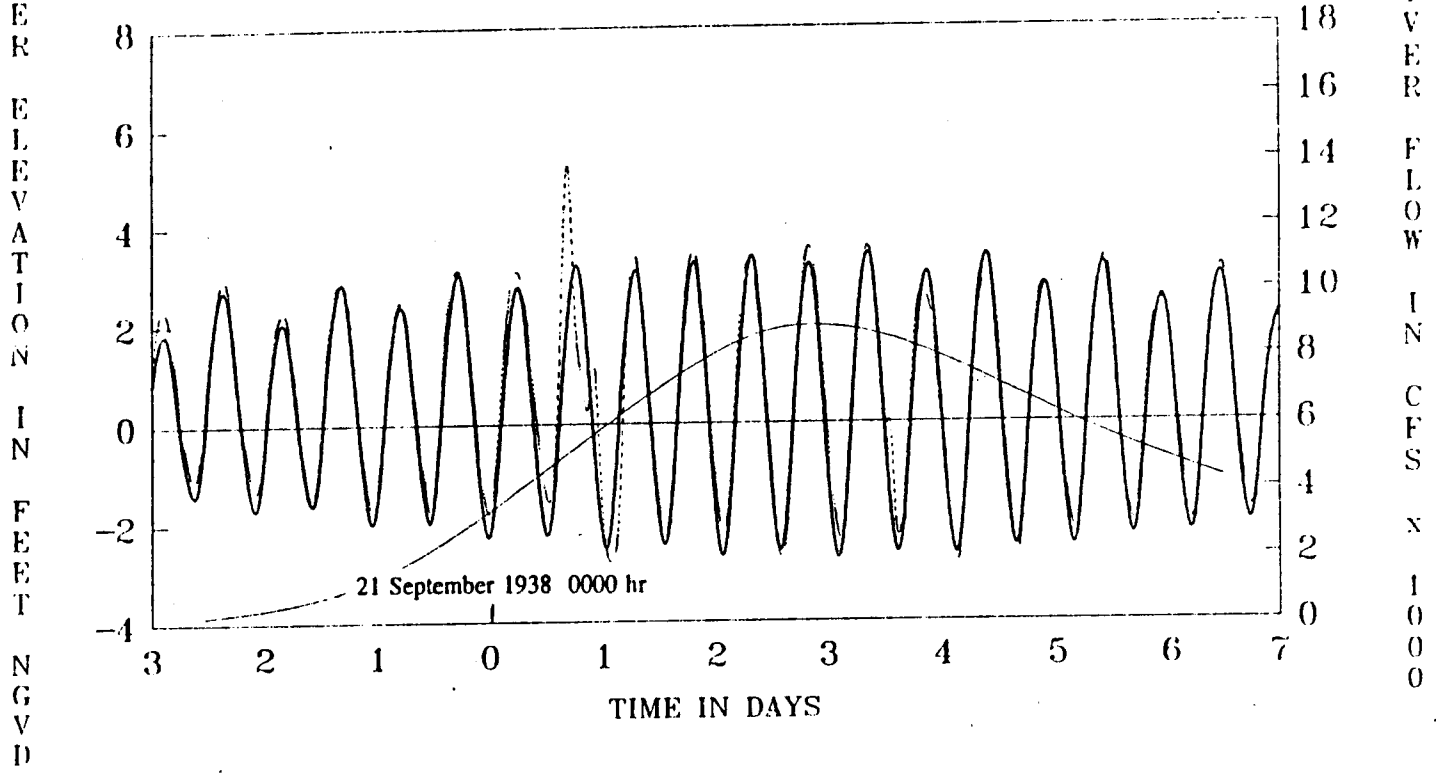


Figure C-75a

Predicted tide height and observed water level at The Battery and flow of the Passaic River measured at the Little Falls, New Jersey gage during the 25 November 1950 storm event

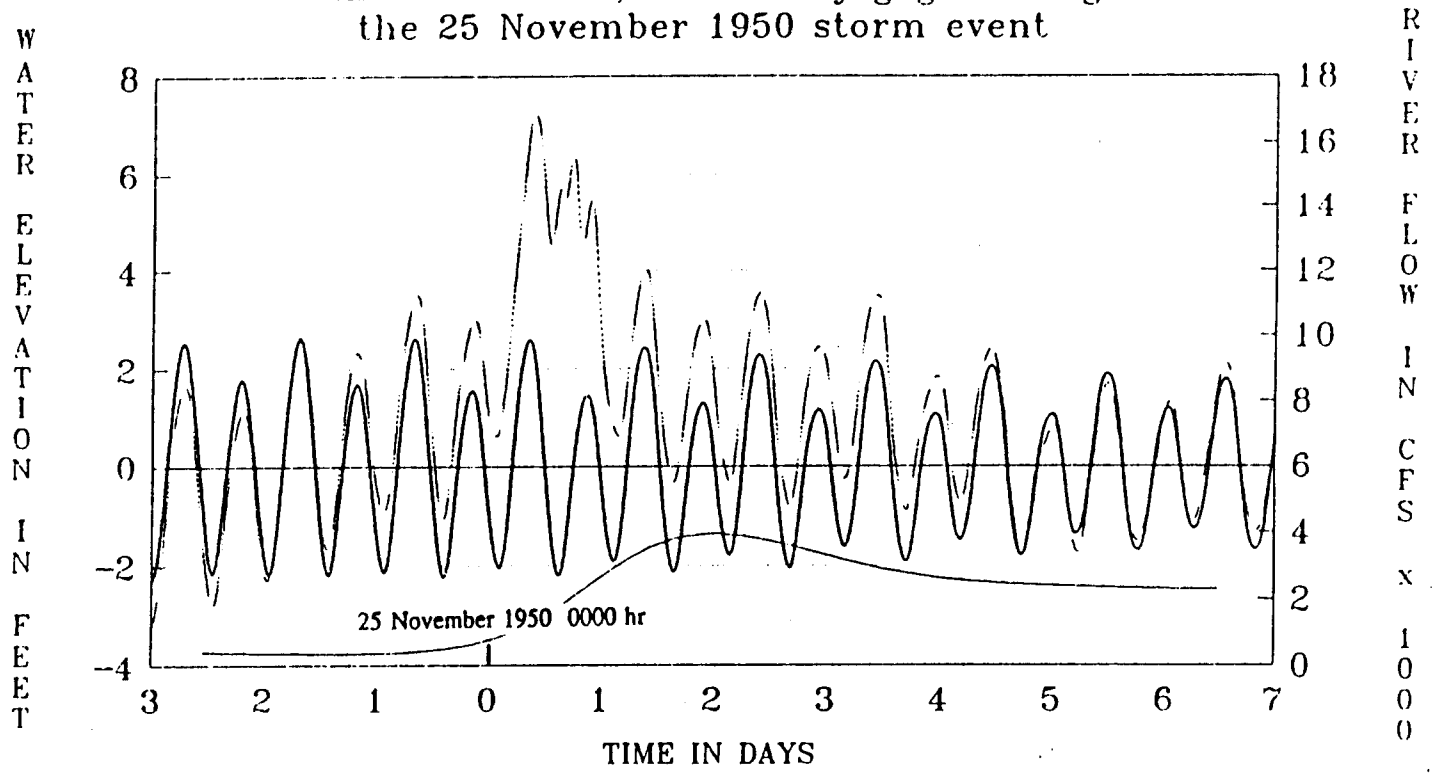


Figure C-75b

Predicted tide height and observed water level at The Battery and flow of the Passaic River measured at the Little Falls, New Jersey gage during the 12 September 1960 storm event

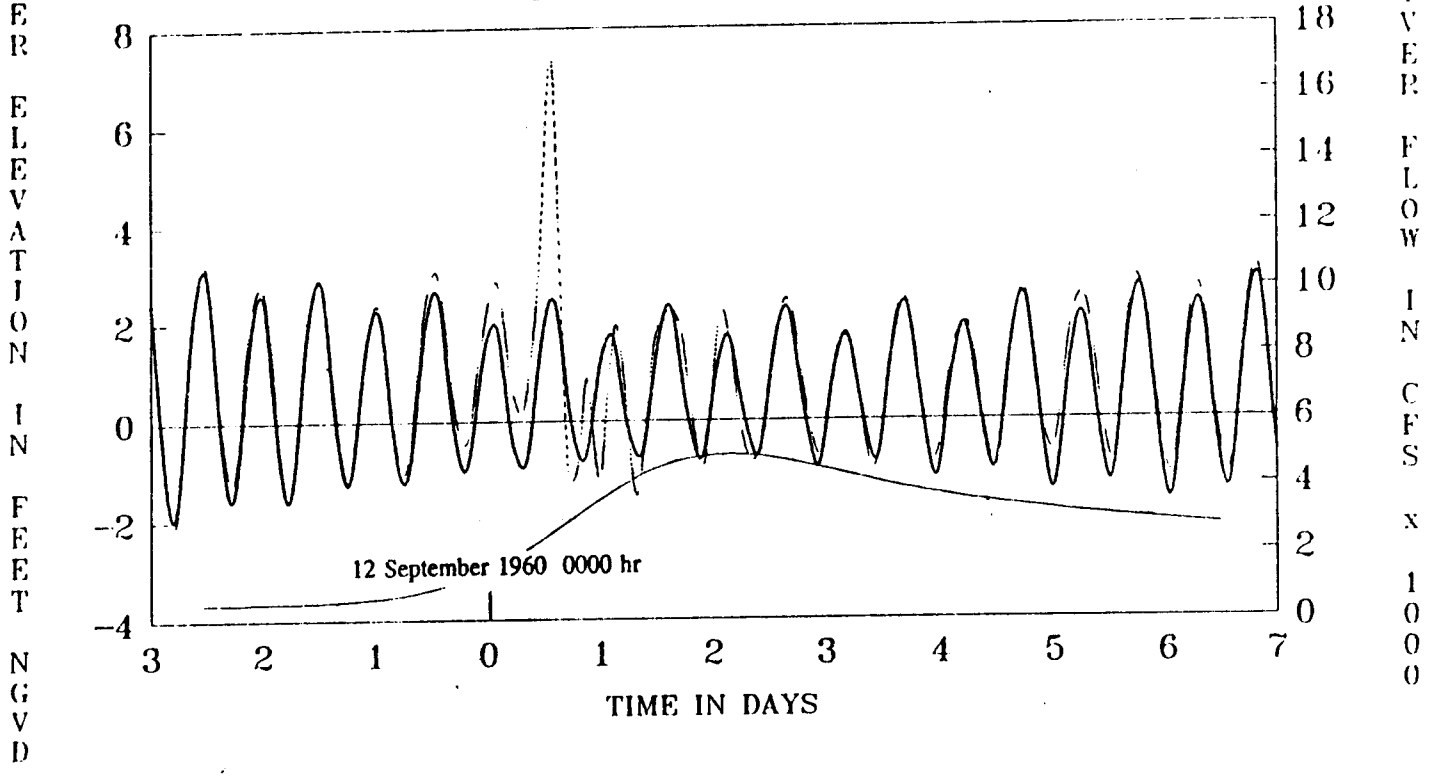


Figure C-75c

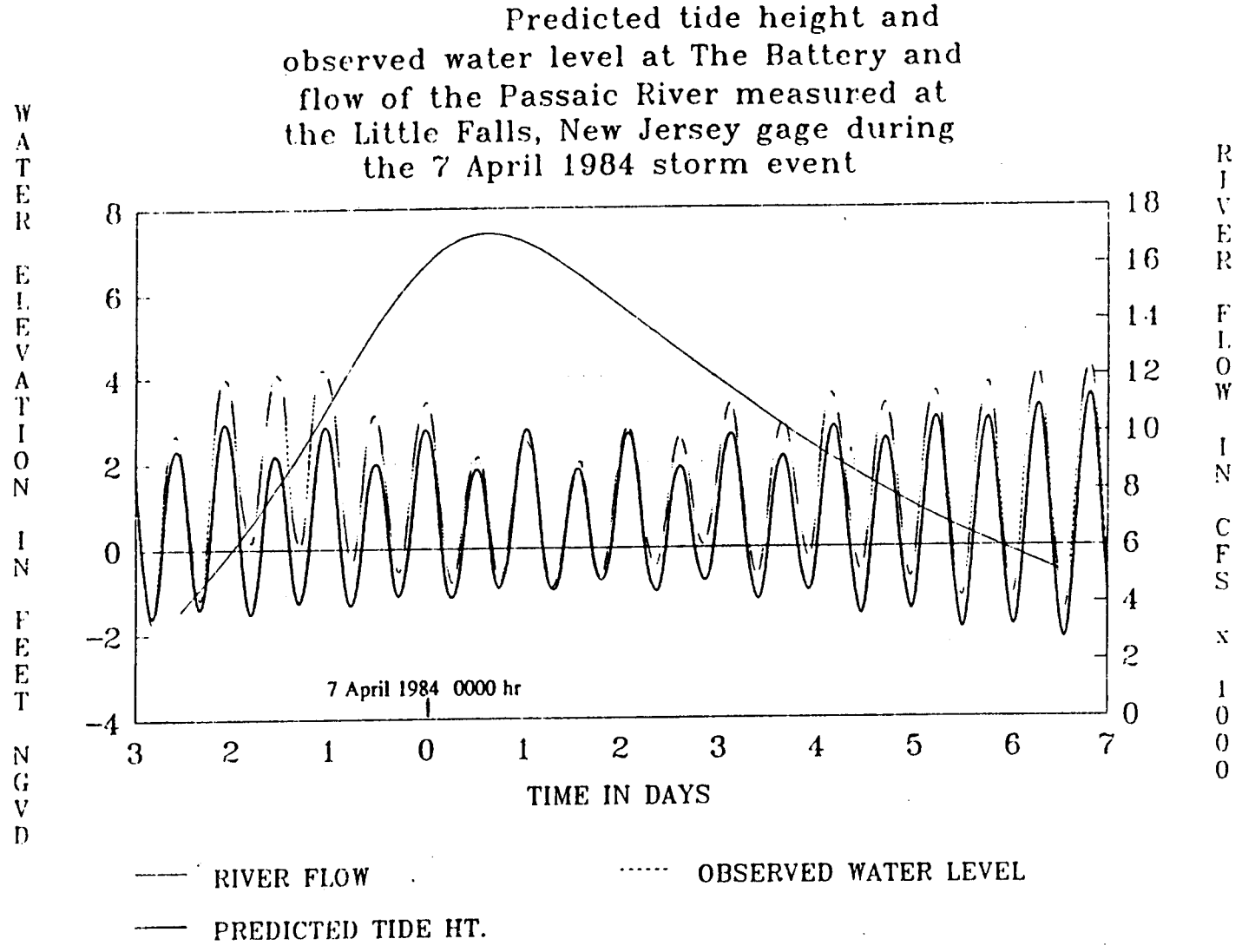
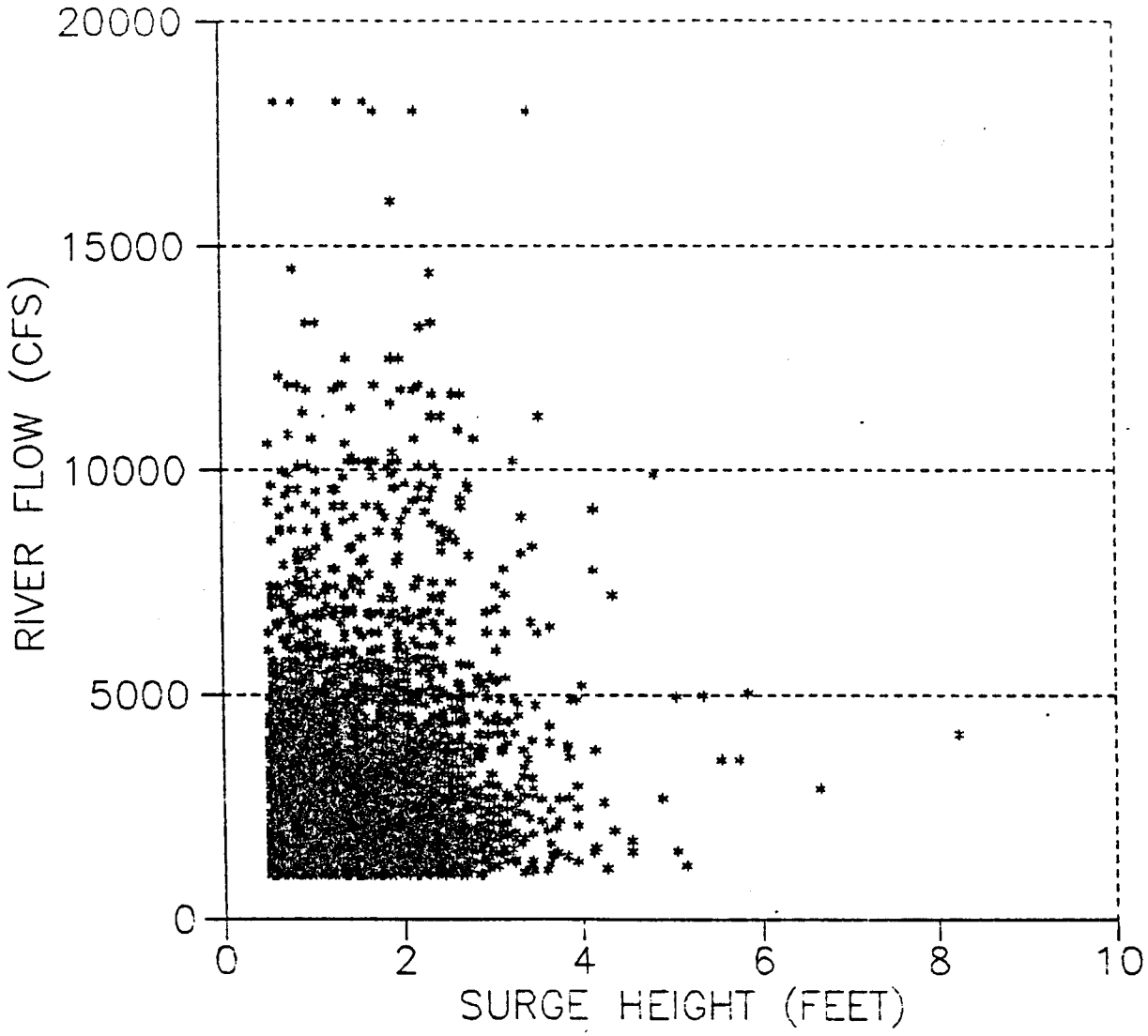
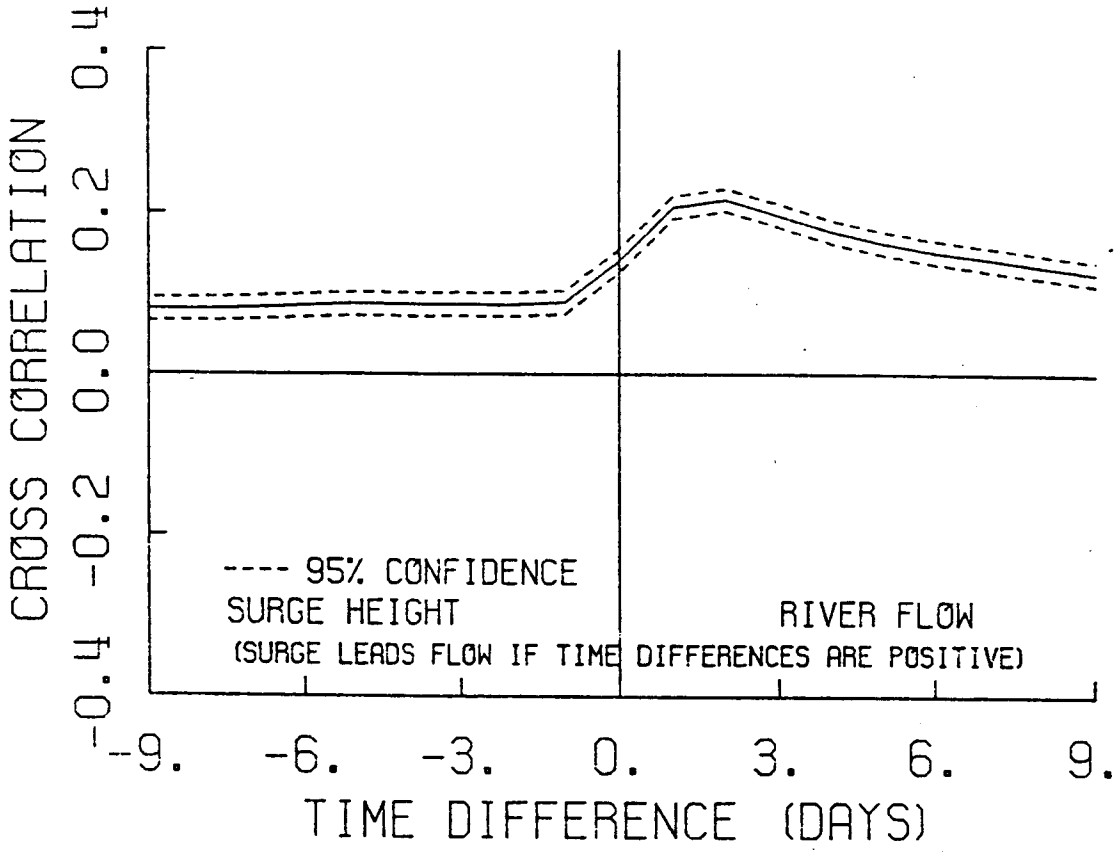

Predicted tide height and observed water level at The Battery and flow of the Passaic River measured at the Little Falls, New Jersey gage during the 7 April 1984 storm event
WATER ELEVATION IN FEET NGVD
RIVER FLOW IN CFS x 1000
8
6
4
2
0
−2
−4
18
16
14
12
10
8
6
4
2
0
7 April 1984 0000 hr
3 2 1 0 1 2 3 4 5 6 7
TIME IN DAYS
RIVER FLOW
OBSERVED WATER LEVEL
PREDICTED TIDE HT.

Figure C-75d



Values of surge height greater than 0.5 feet and maximum river flow greater than 1,000 cfs within the following 72 hours.

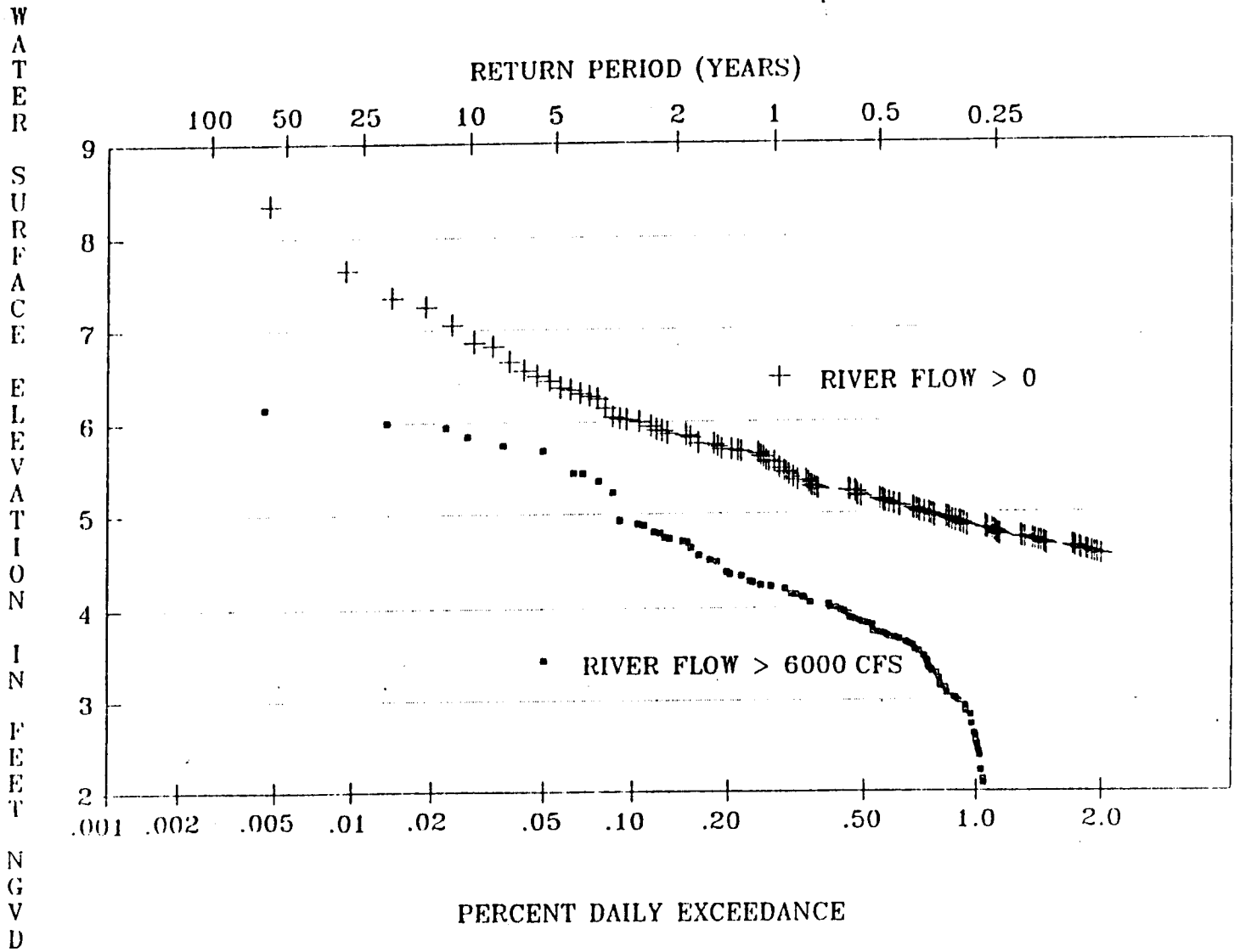
Figure C-76



Cross correlation between surge height at The Battery and Passaic River flow measured at the Little Falls, New Jersey gage.

Figure C-77

Stage-frequency curves and return periods for water surface elevation at The Battery and Passaic River flows at the Little Falls gage



Note: Tabulated river flow values exceeded 6,000 CFS on 230 out of 21,174 days.

Figure C-78

Figure 61. Predicted water level elevations at Newark Airport for year 2050 conditions and October 1976 tides

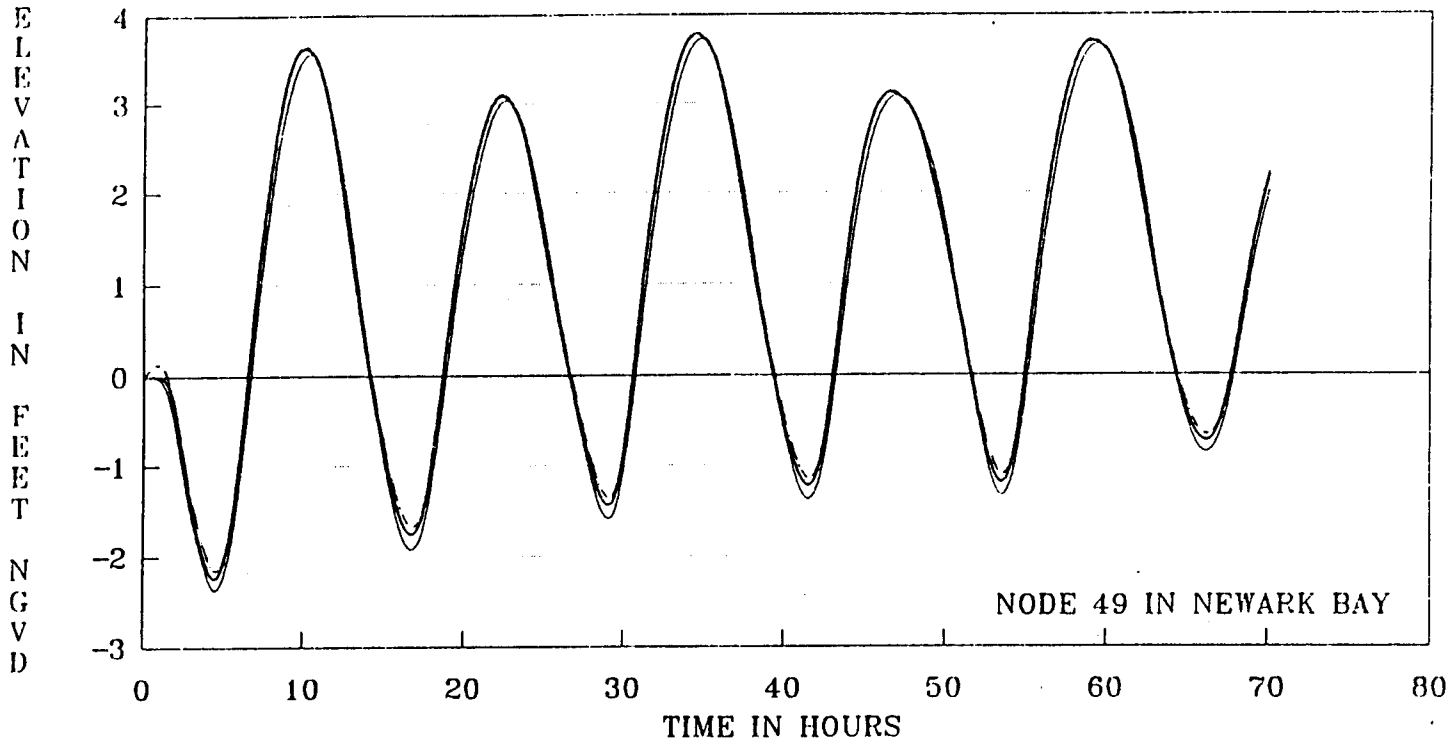


Figure C-79a

Figure 62. Predicted water level elevations at Kearny Point for year 2050 conditions and October 1976 tides

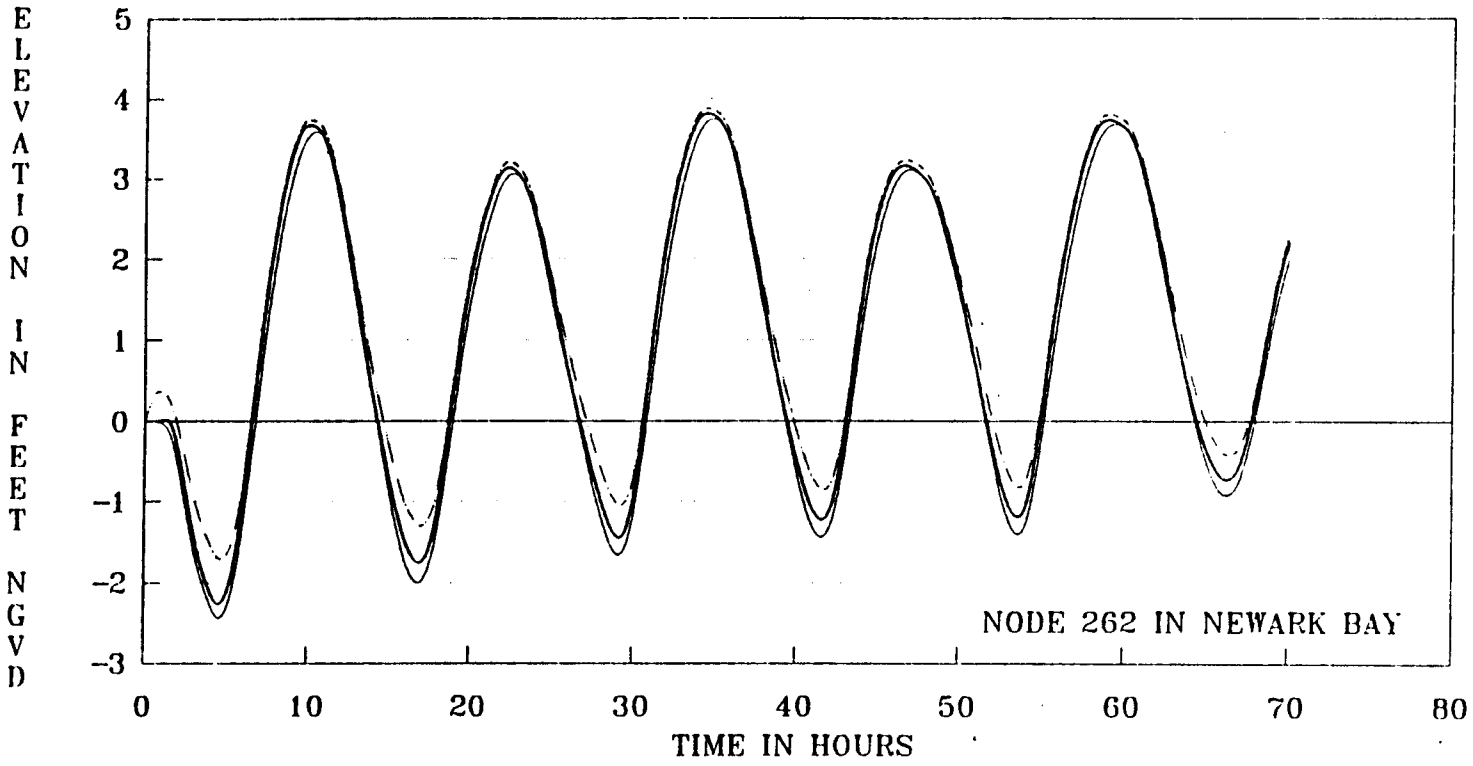


Figure 63. Predicted water level elevations at East Newark for year 2050 conditions and October 1976 tides

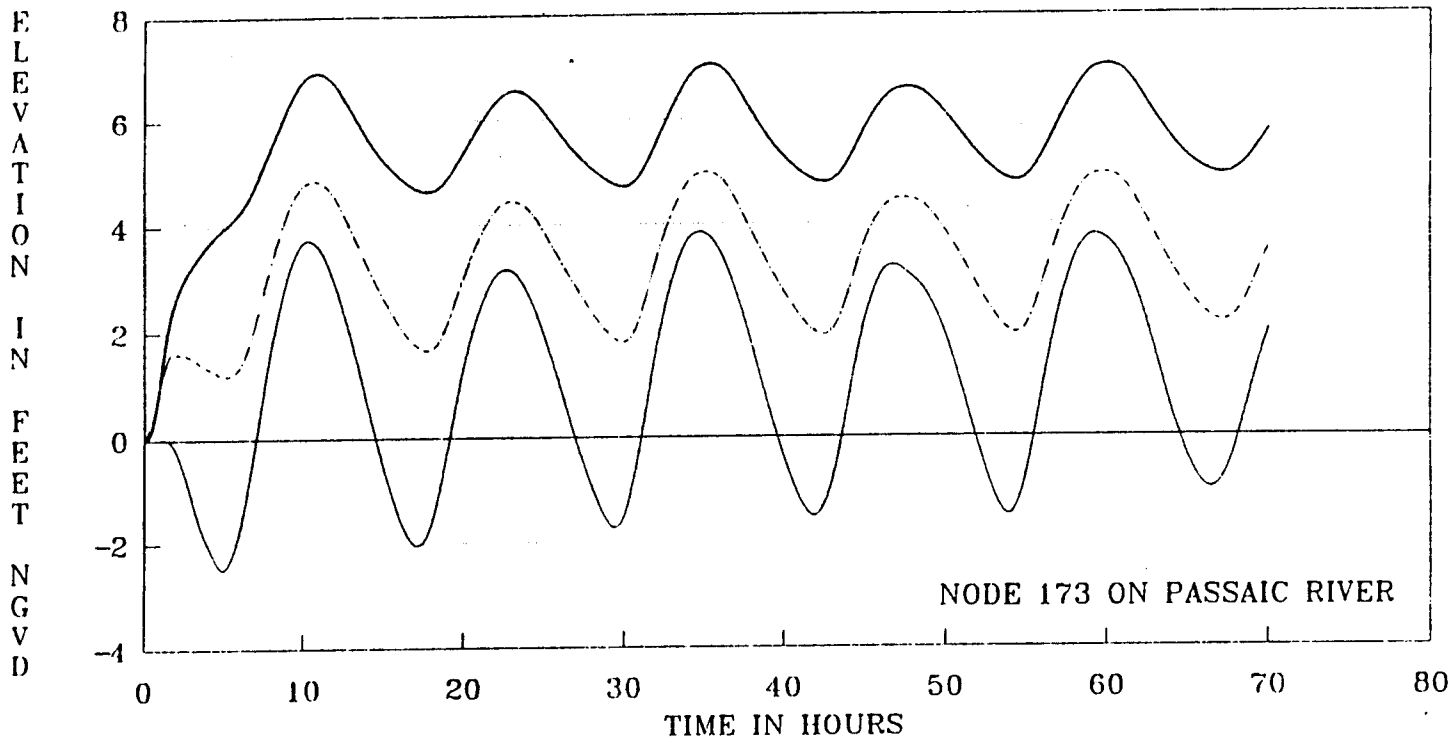


Figure 64. Predicted water level elevations at Third River for year 2050 conditions and October 1976 tides

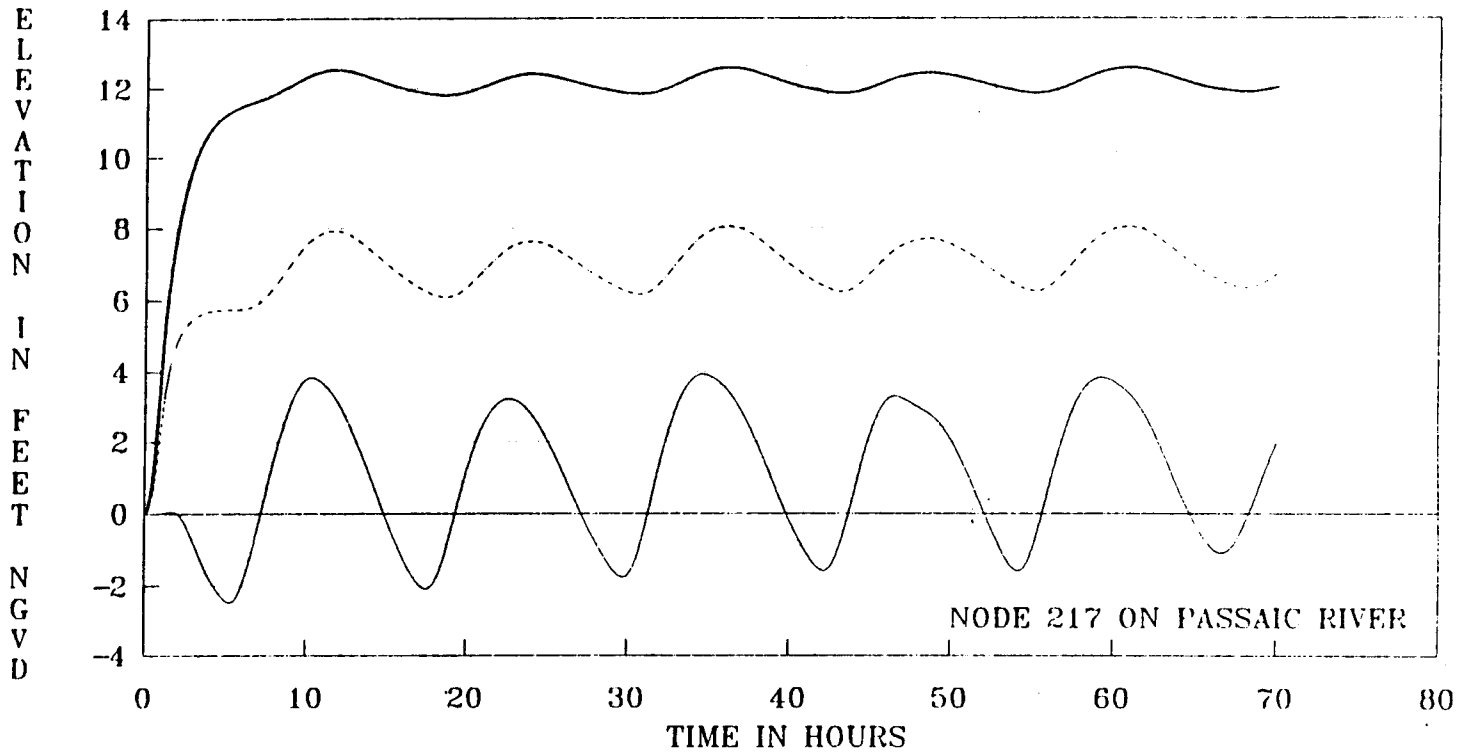


Figure 65. Predicted water level elevations at Garfield for year 2050 conditions and October 1976 tides

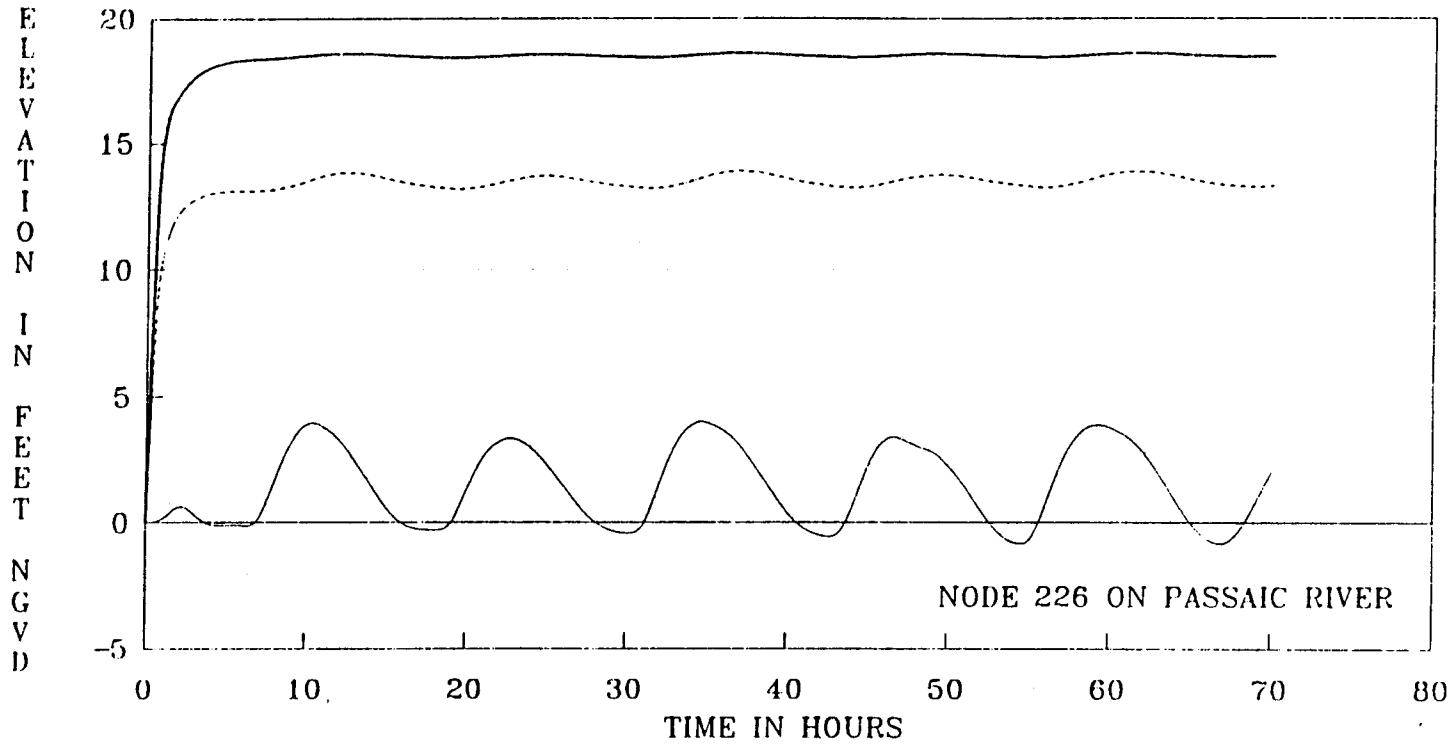


Figure 66. Predicted water level elevations at Snake Hill for year 2050 conditions and October 1976 tides

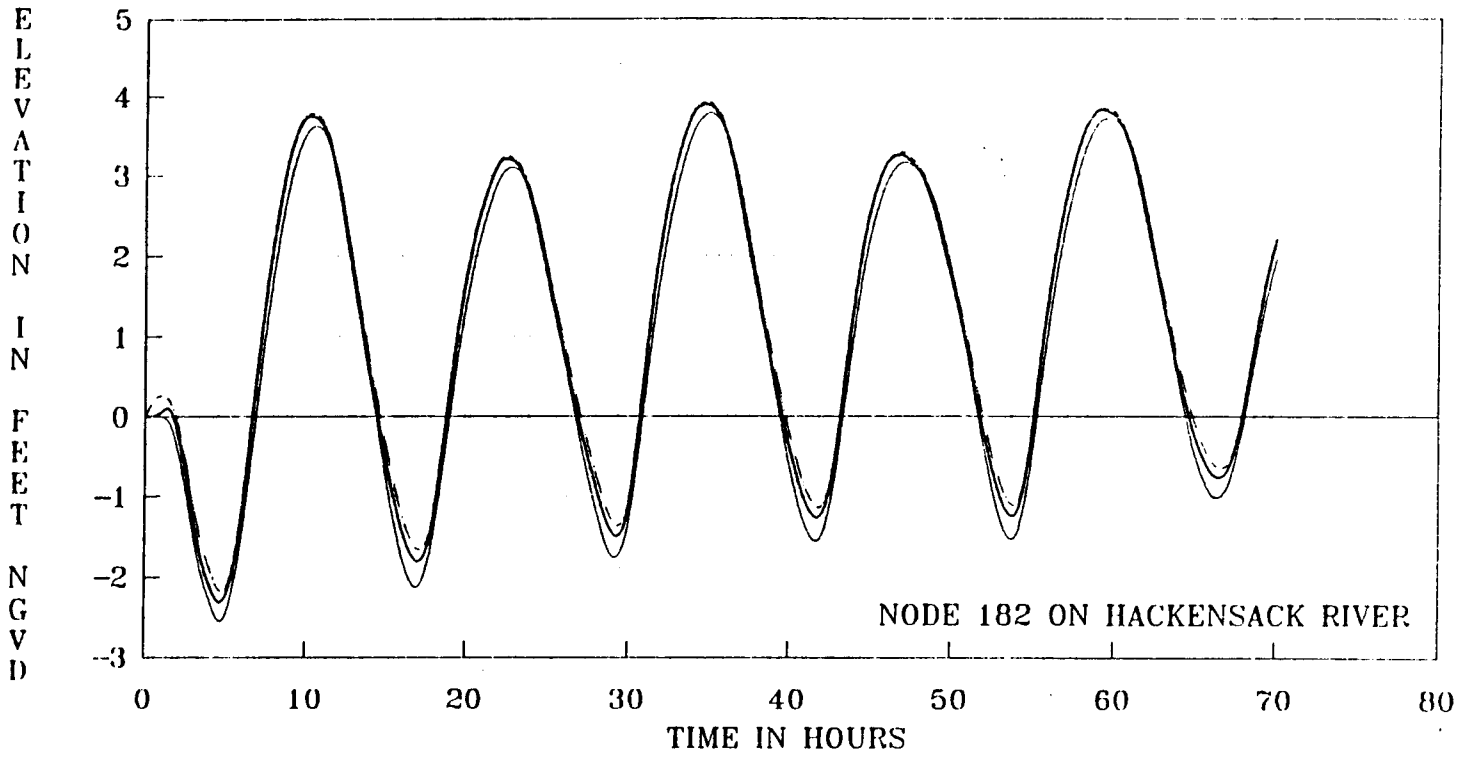


Figure 67. Predicted water level elevations at Berry's Creek for year 2050 conditions and October 1976 tides

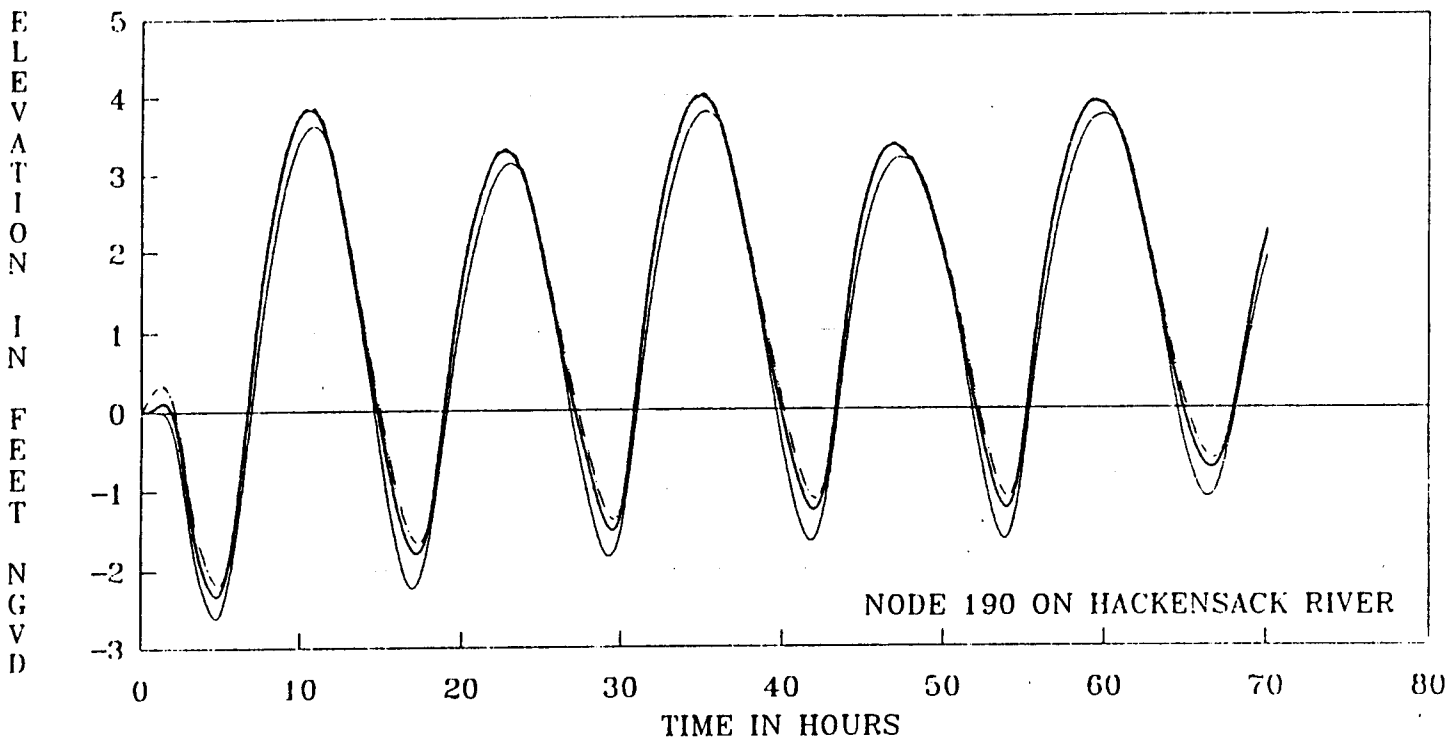


Figure 68. Predicted water level elevations at New Milford for year 2050 conditions and October 1976 tides

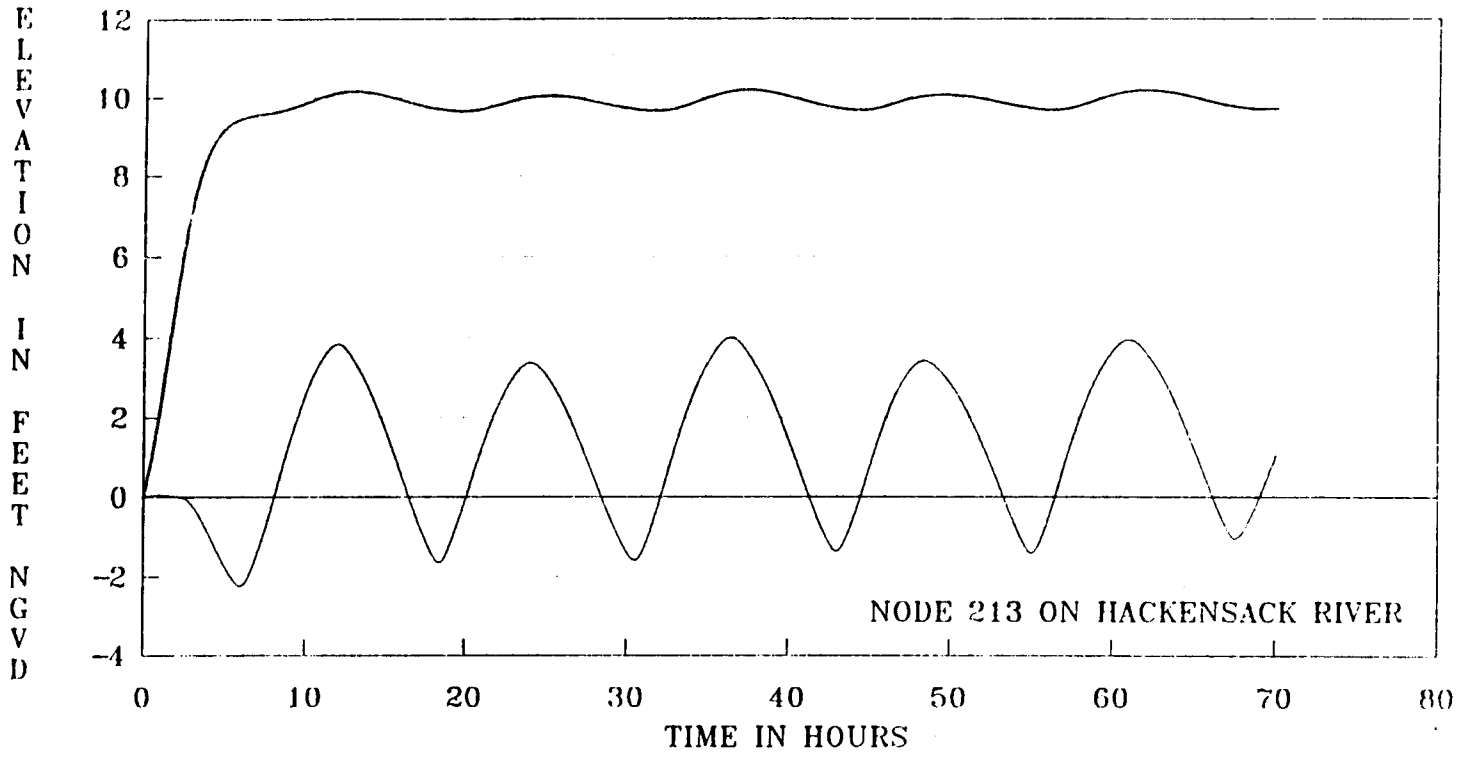


Figure C-79h

Figure 69. Predicted water level elevations at Newark Airport for year 2050 conditions and synthetic hurricane 135

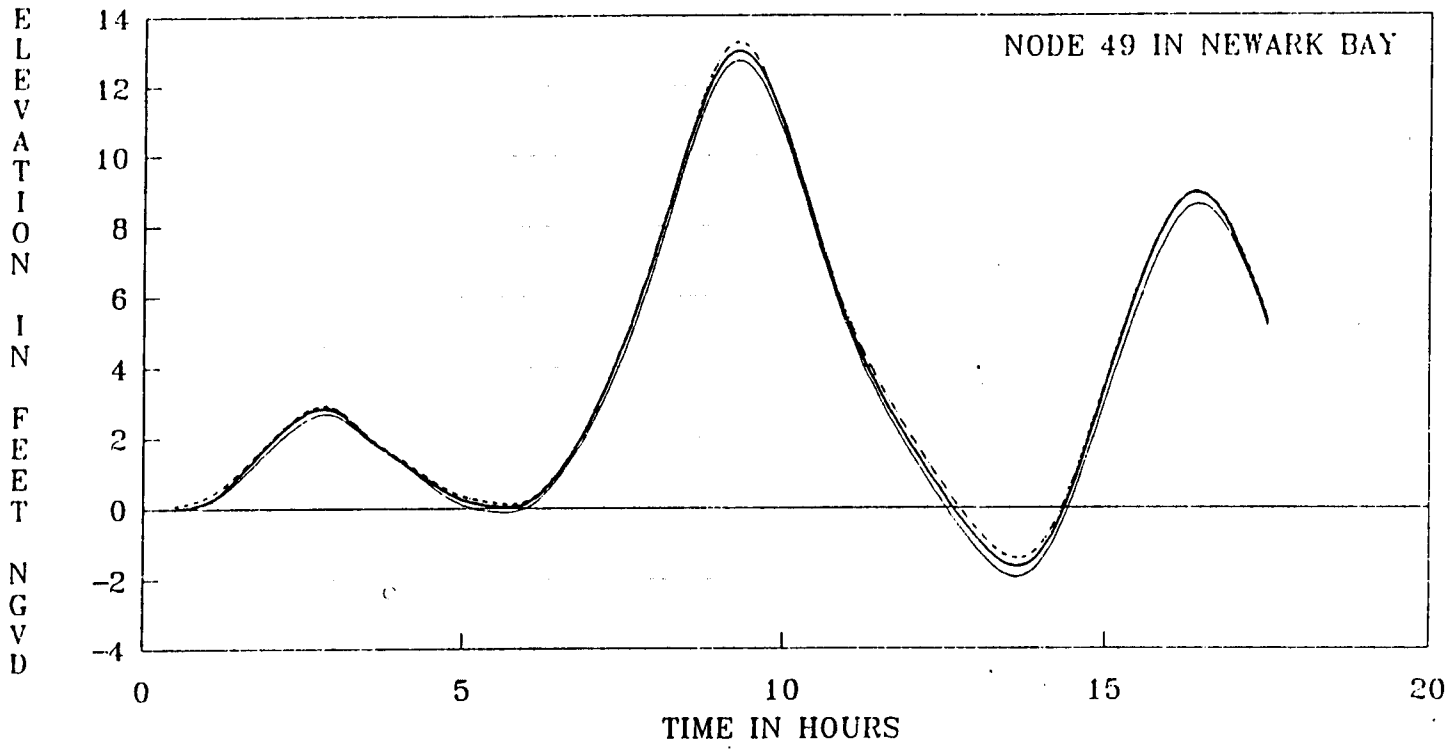


Figure C-80a

Figure 70. Predicted water level elevations at Kearny Point for year 2050 conditions and synthetic hurricane 135

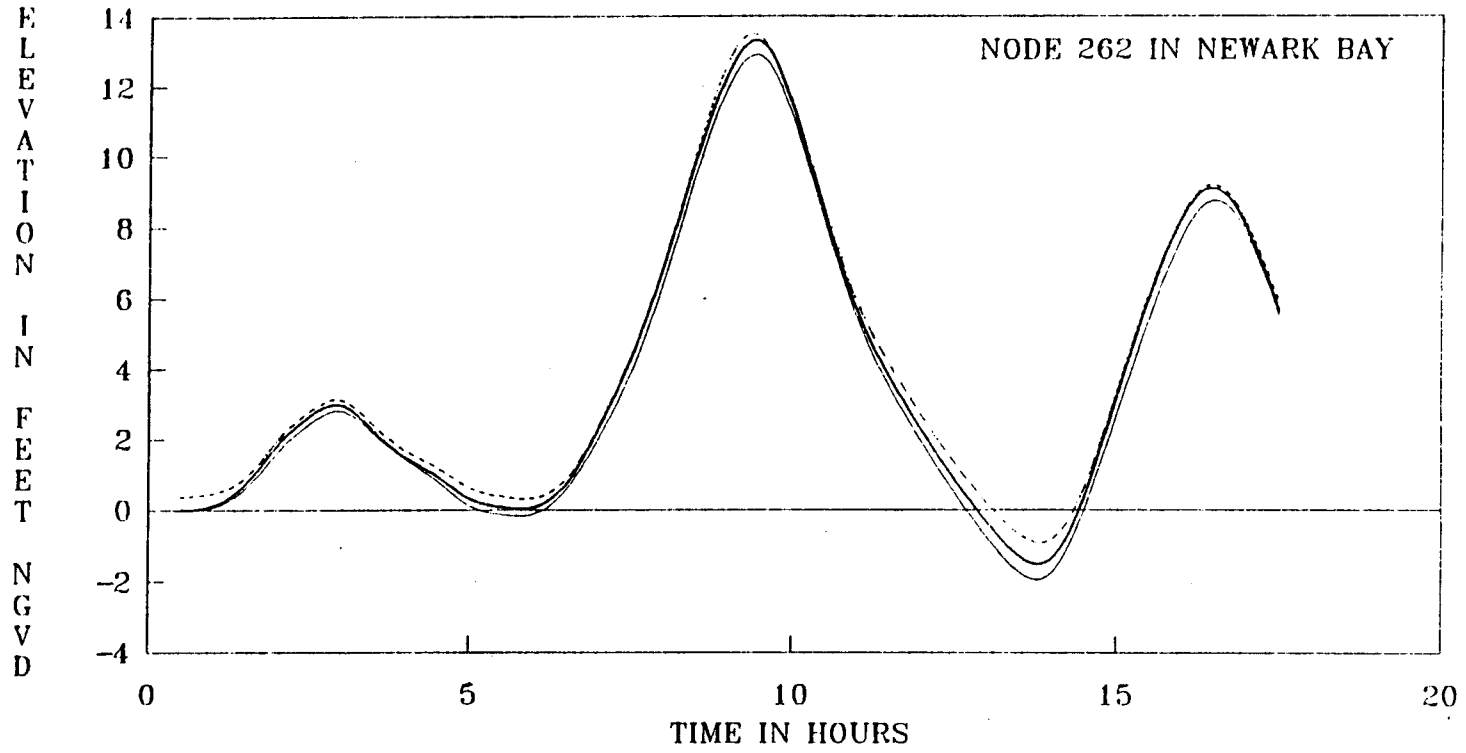


Figure C-80b

Figure 71. Predicted water level elevations at East Newark for year 2050 conditions and synthetic hurricane 135

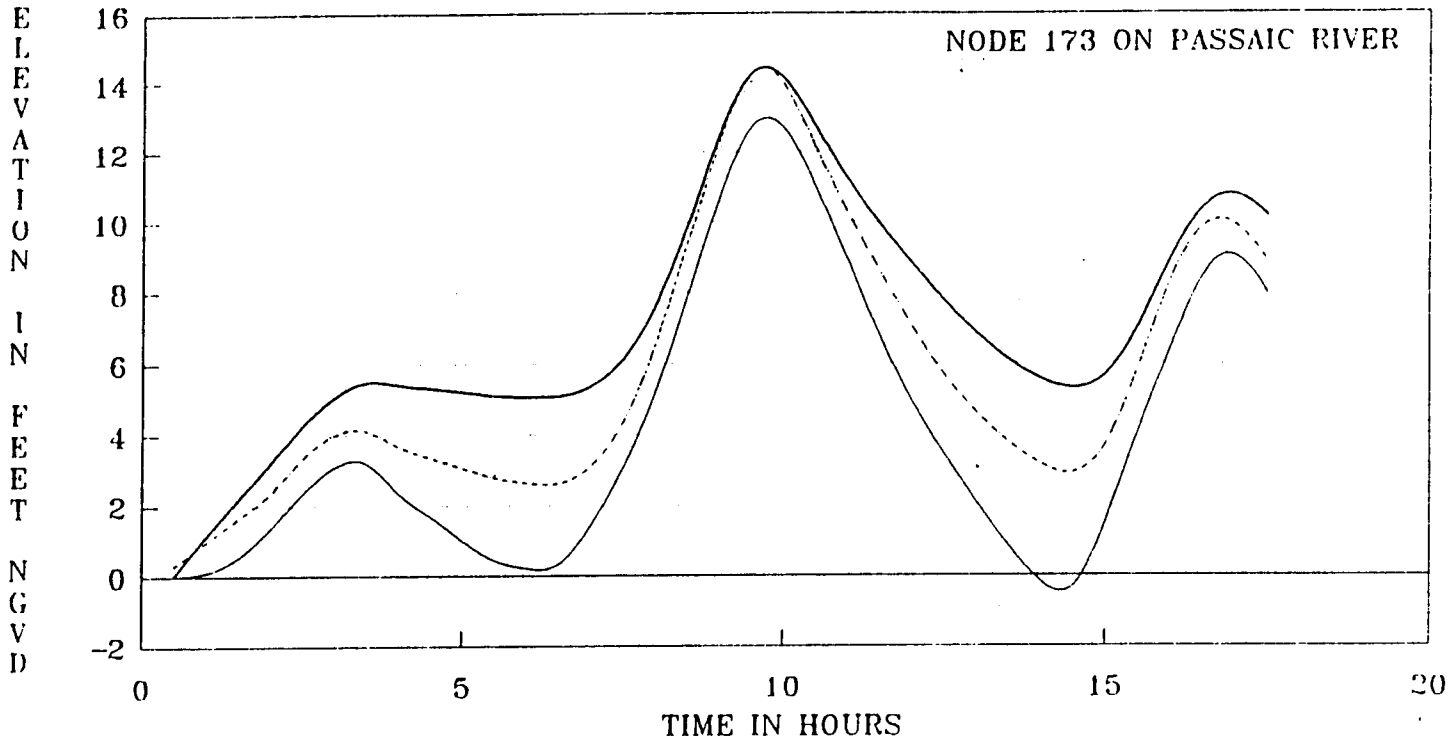


Figure 72. Predicted water level elevations at Third River for year 2050 conditions and synthetic hurricane 135

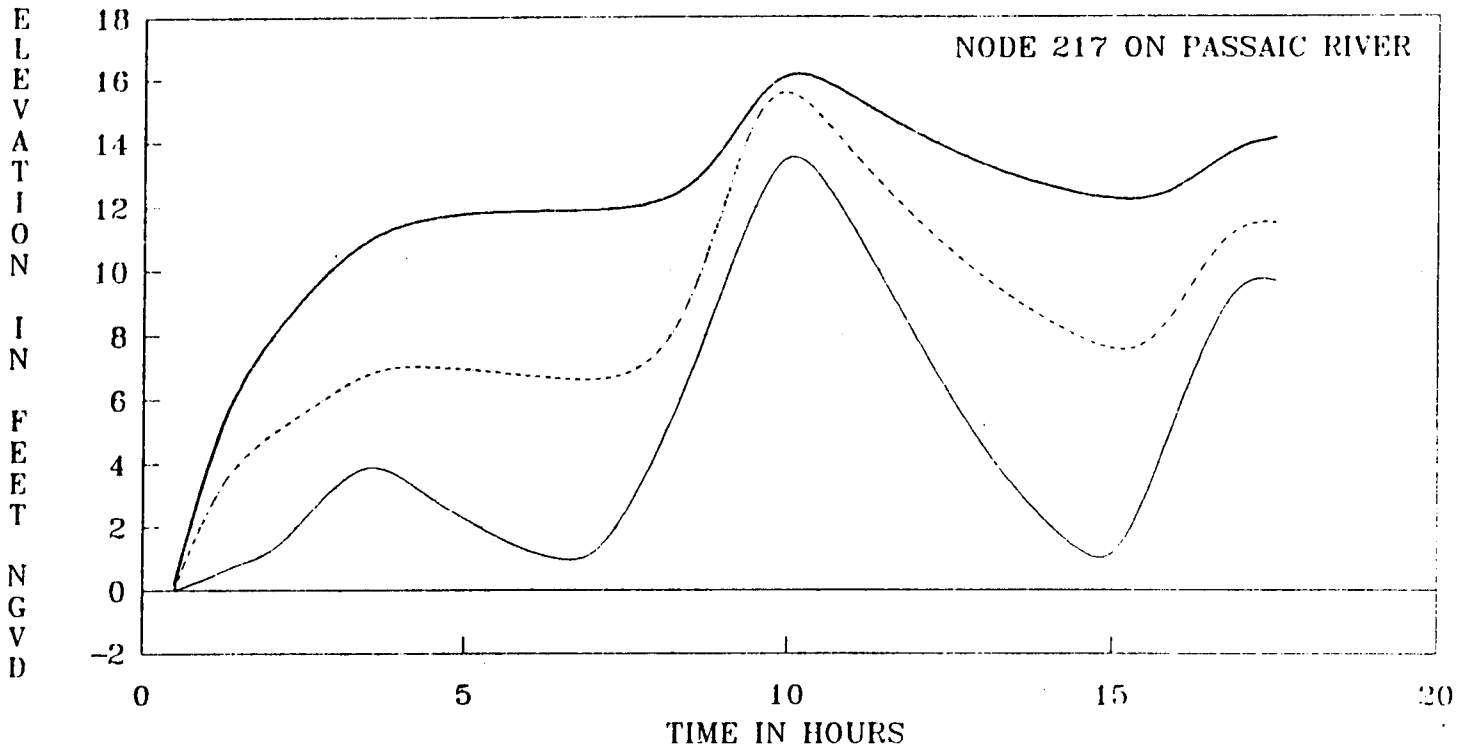


Figure C-80d

Figure 73. Predicted water level elevations at Garfield for year 2050 conditions and synthetic hurricane 135

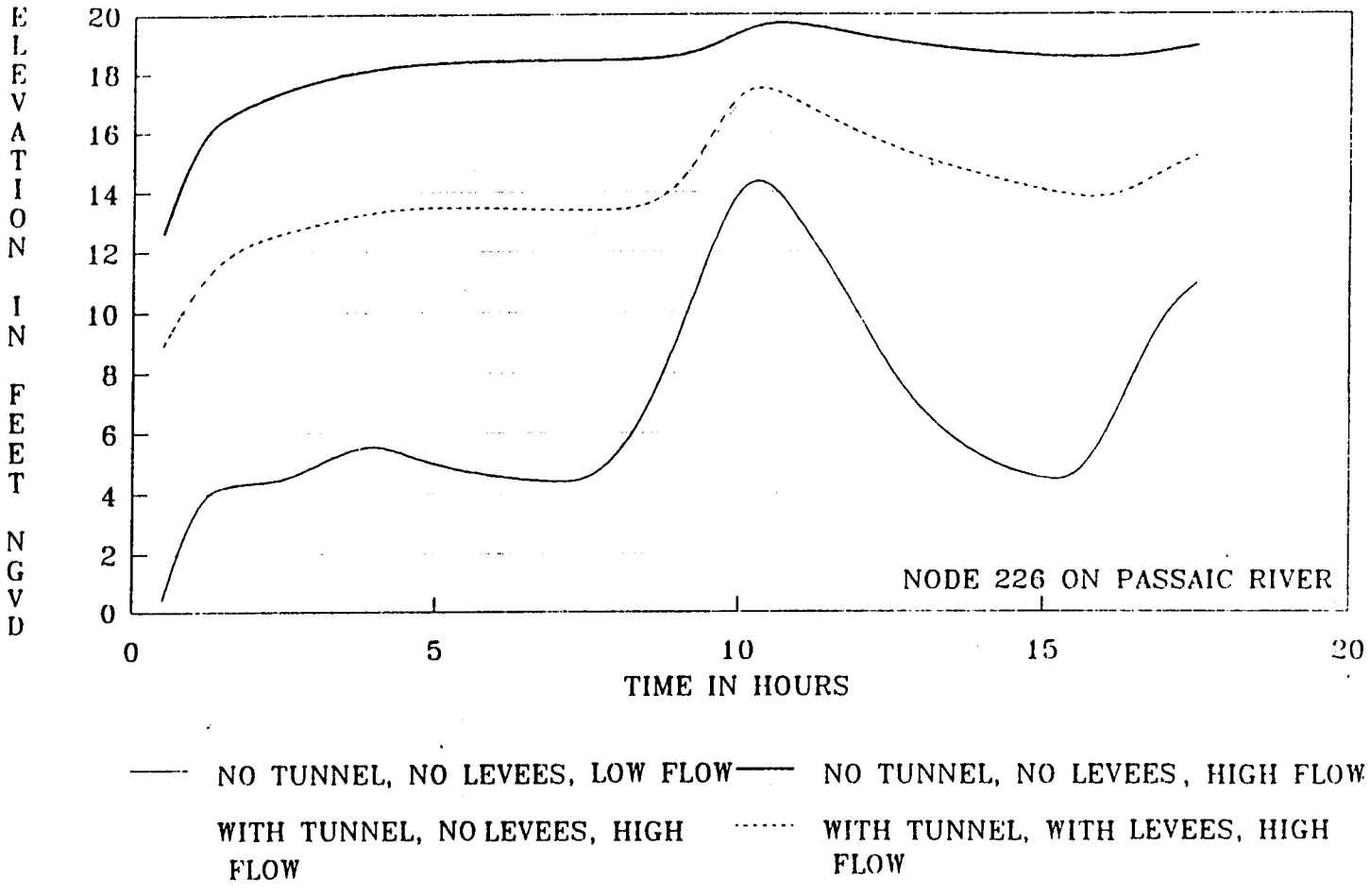


Figure 74. Predicted water level elevations at Snake Hill for year 2050 conditions and synthetic hurricane 135

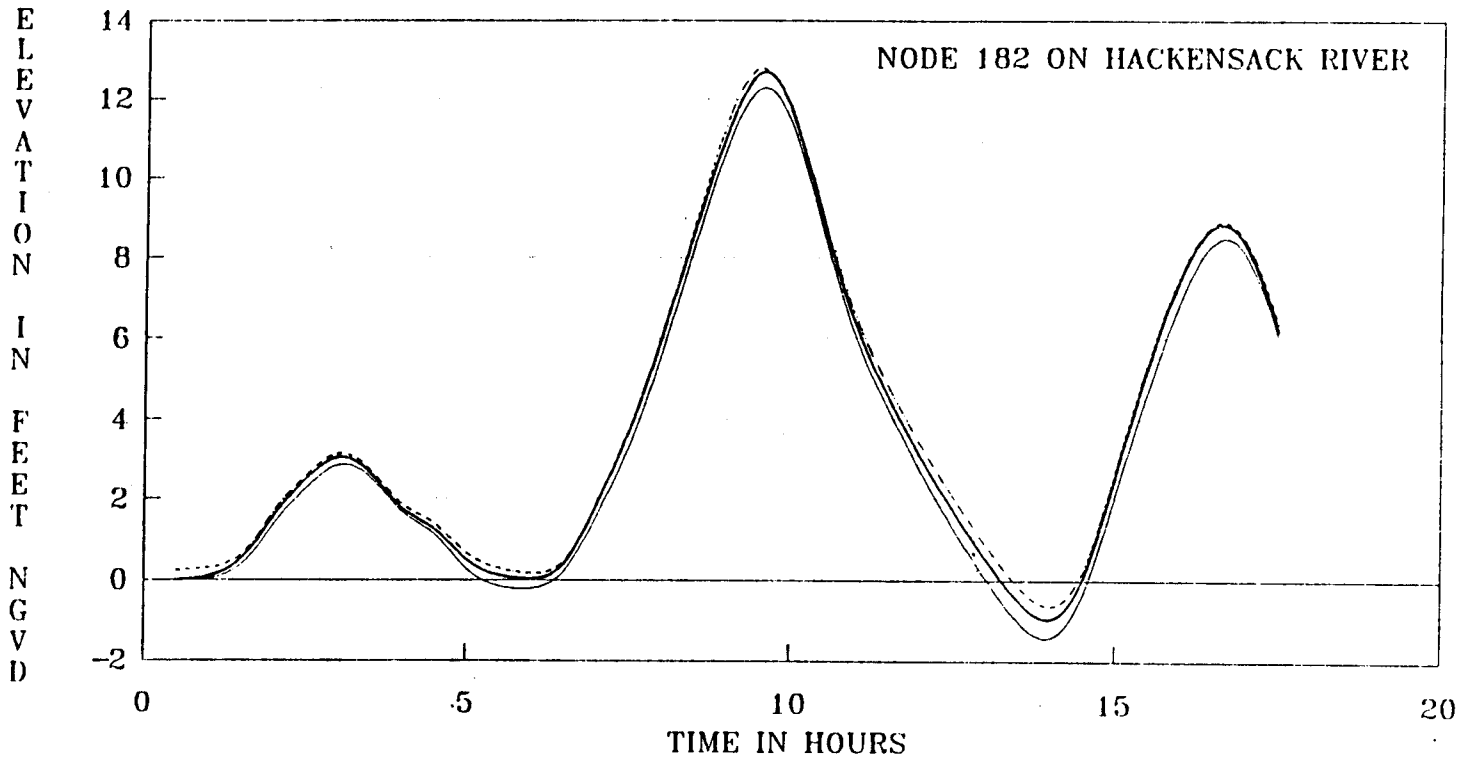


Figure C-80f

Figure 75. Predicted water level elevations at Berry's Creek for year 2050 conditions and synthetic hurricane 135

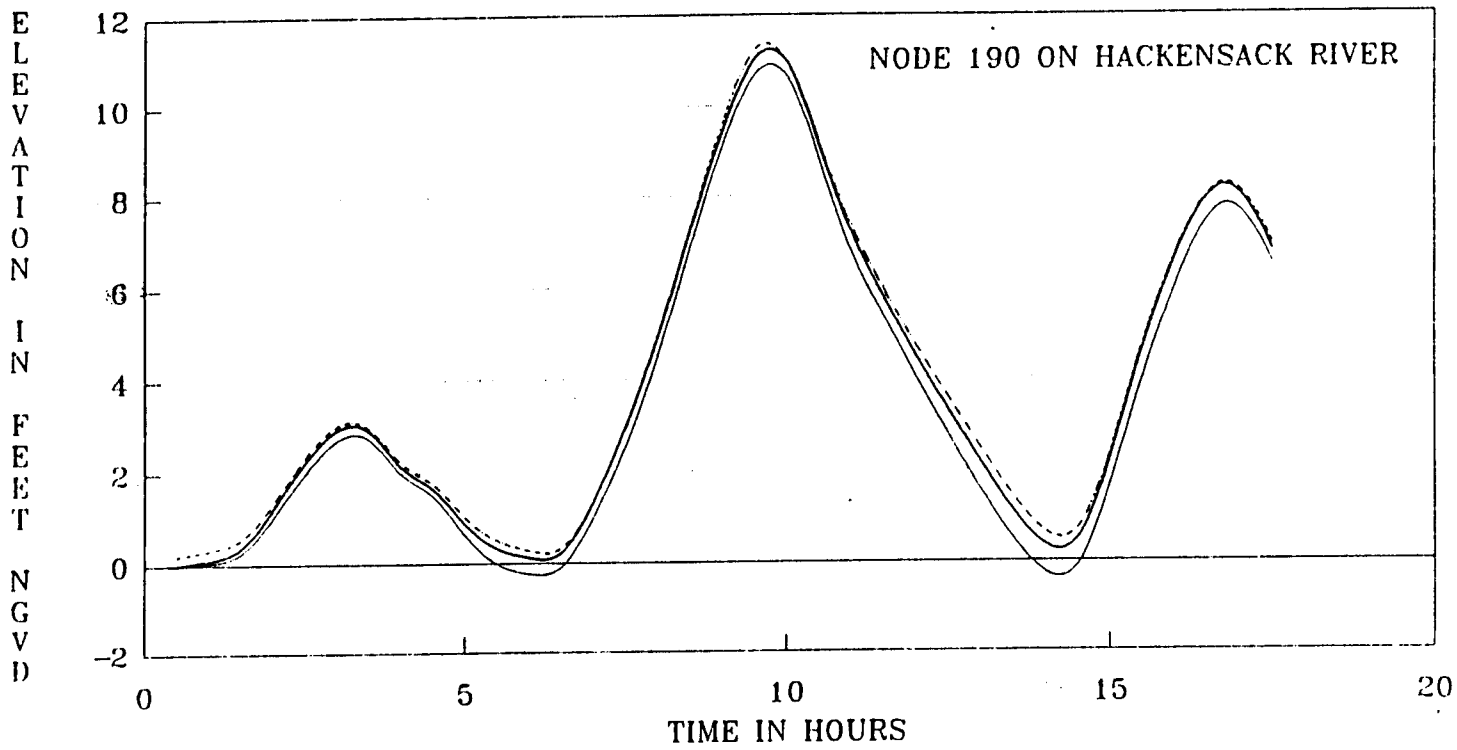


Figure C-809

Figure 76. Predicted water level elevations at New Milford for year 2050 conditions and synthetic hurricane 135

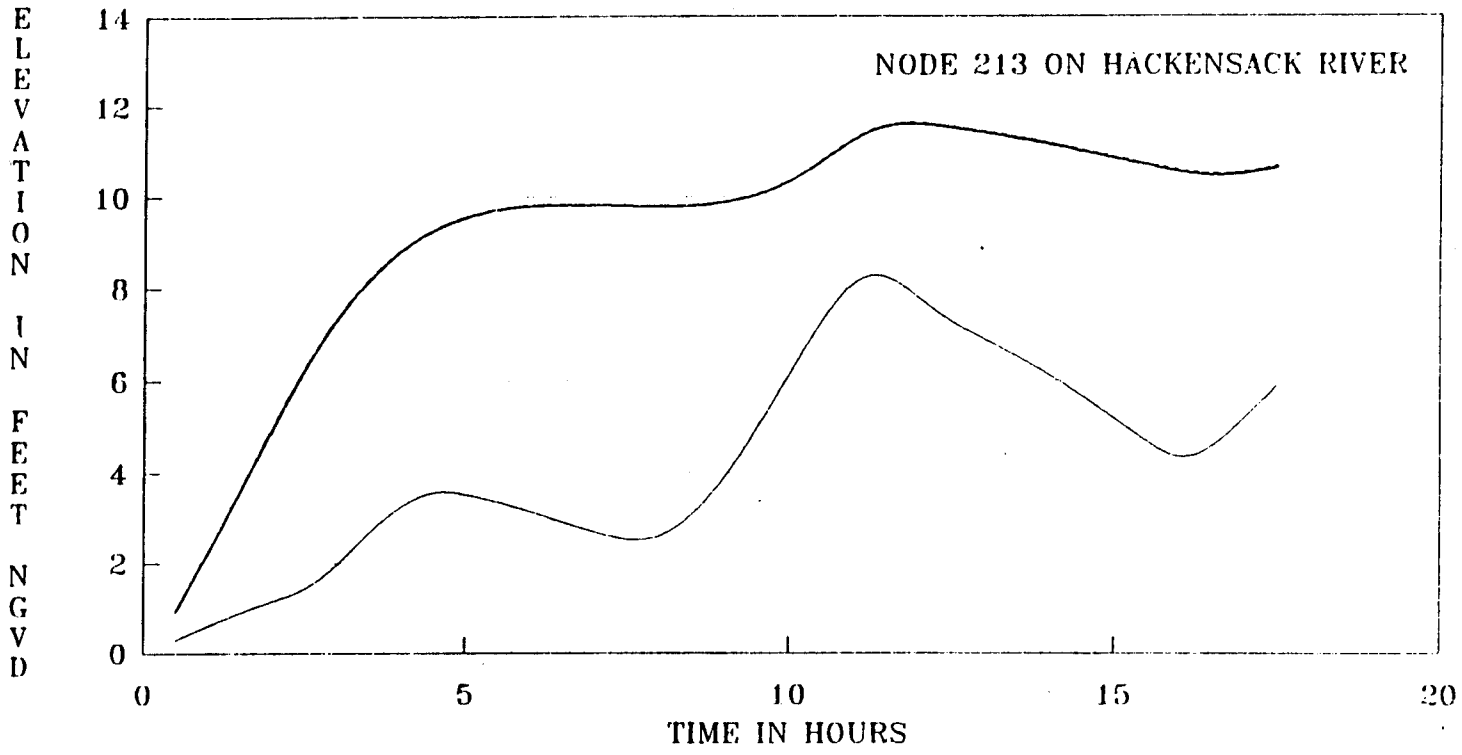


Figure 77. Predicted water level elevations at Newark Airport for year 2050 conditions and synthetic hurricane 249

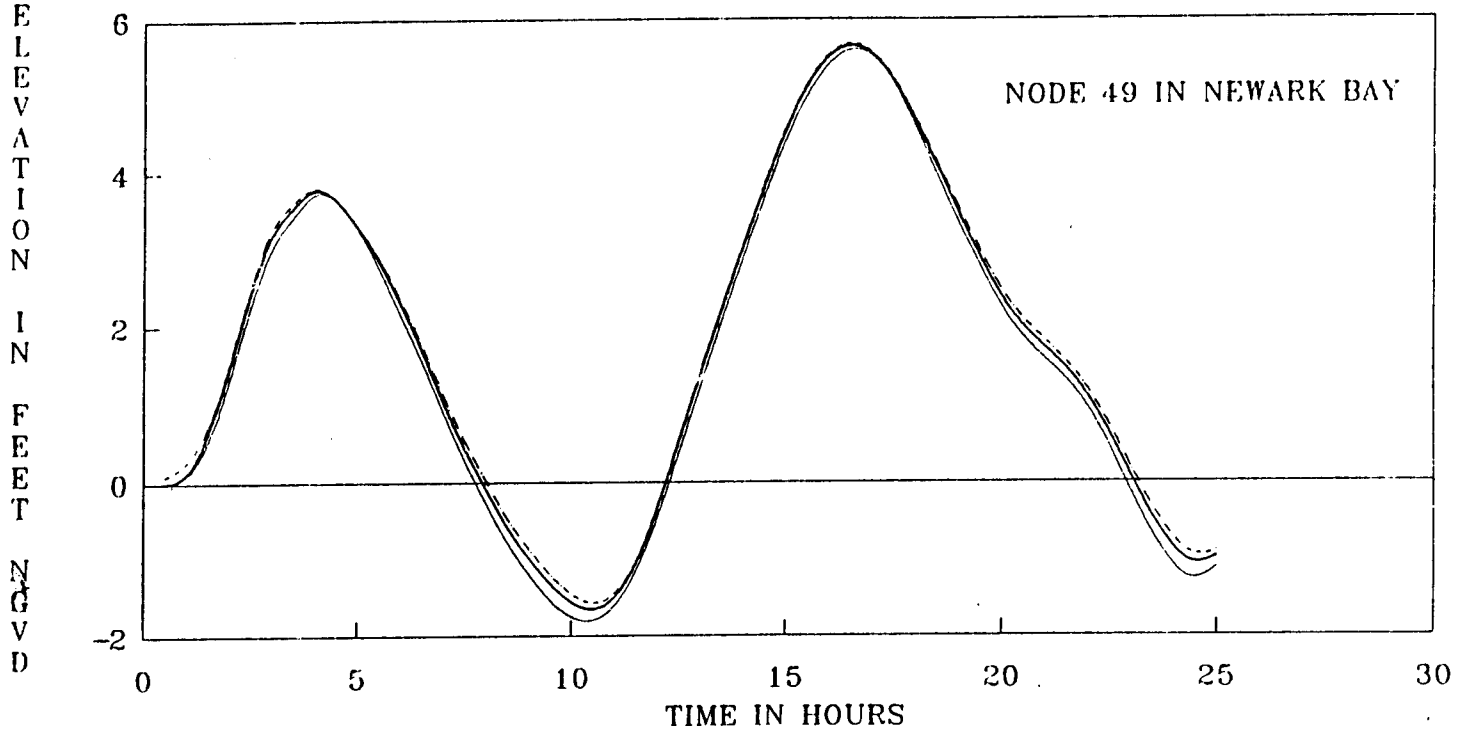


Figure C-81a

Figure 78. Predicted water level elevations at Kearny Point for year 2050 conditions and synthetic hurricane 249

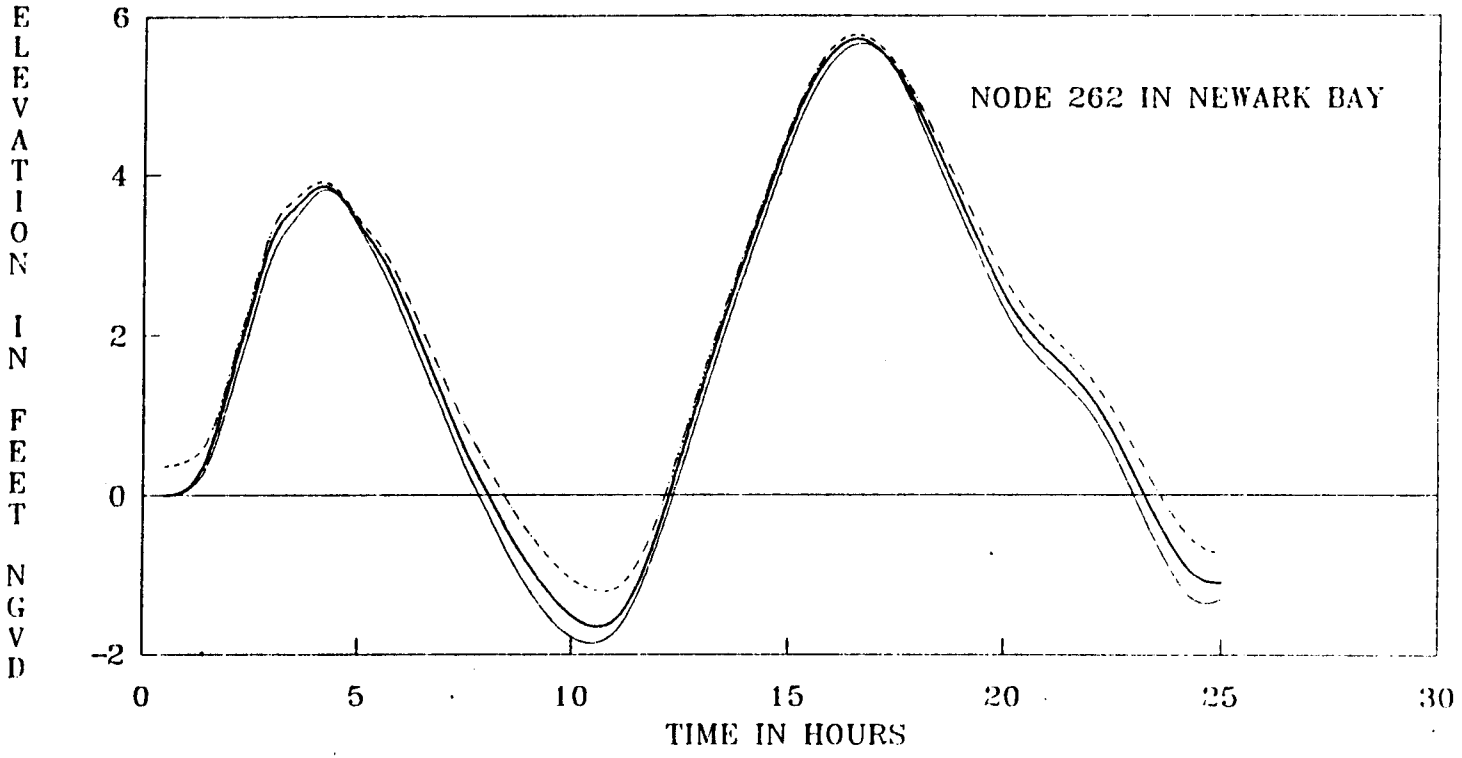


Figure C-81b

Figure 79. Predicted water level elevations at East Newark for year 2050 conditions and synthetic hurricane 249

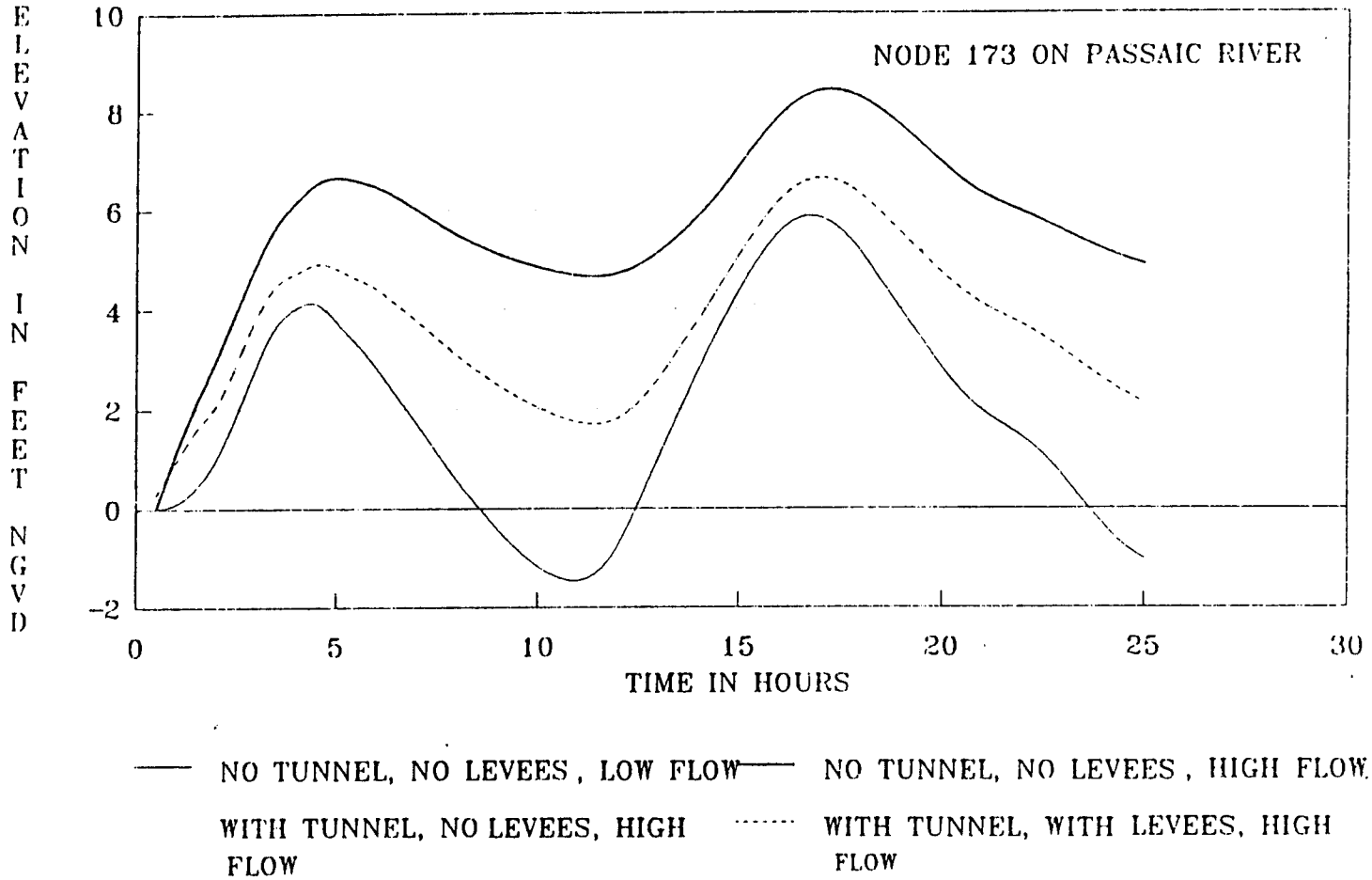


Figure 80. Predicted water level elevations at Third River for year 2050 conditions and synthetic hurricane 249

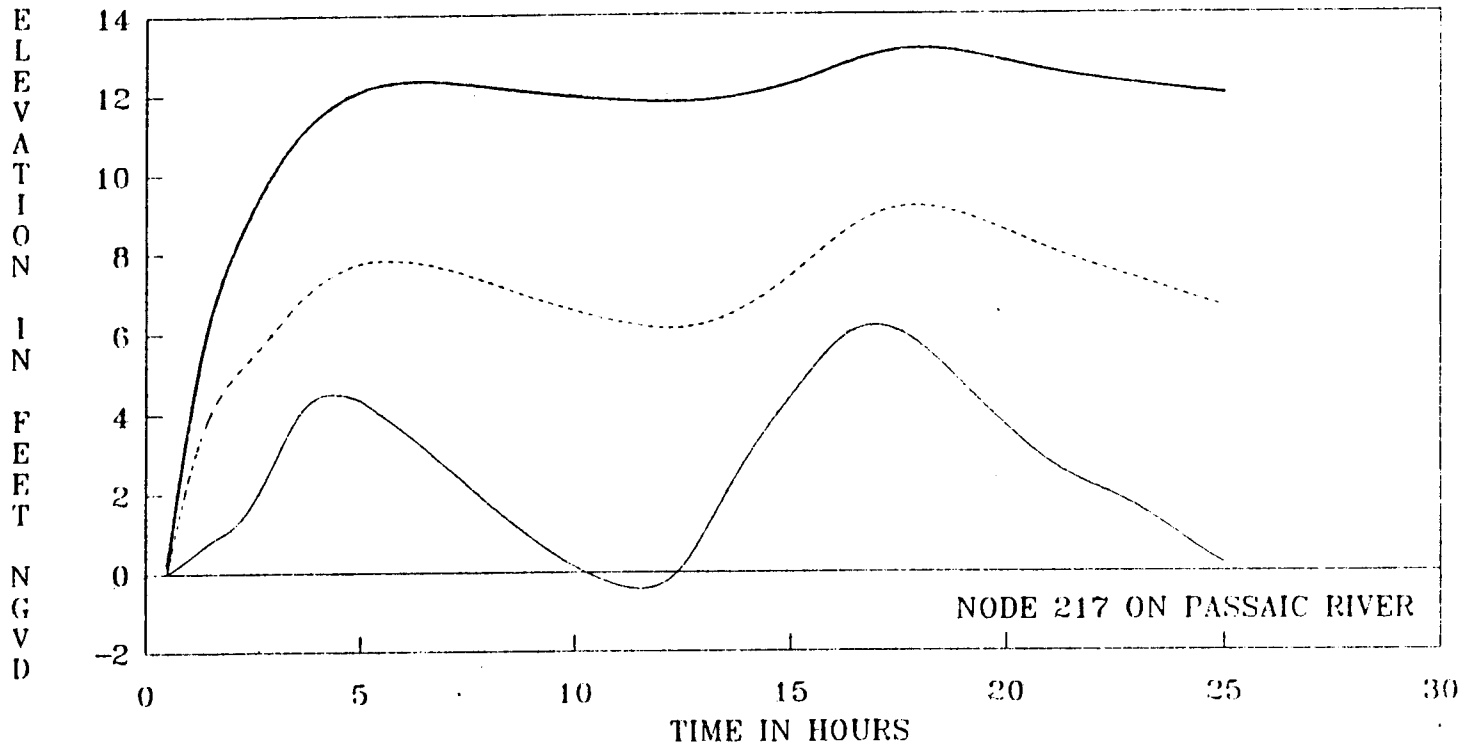


Figure C-81d

Figure 81. Predicted water level elevations at Garfield for year 2050 conditions and synthetic hurricane 249

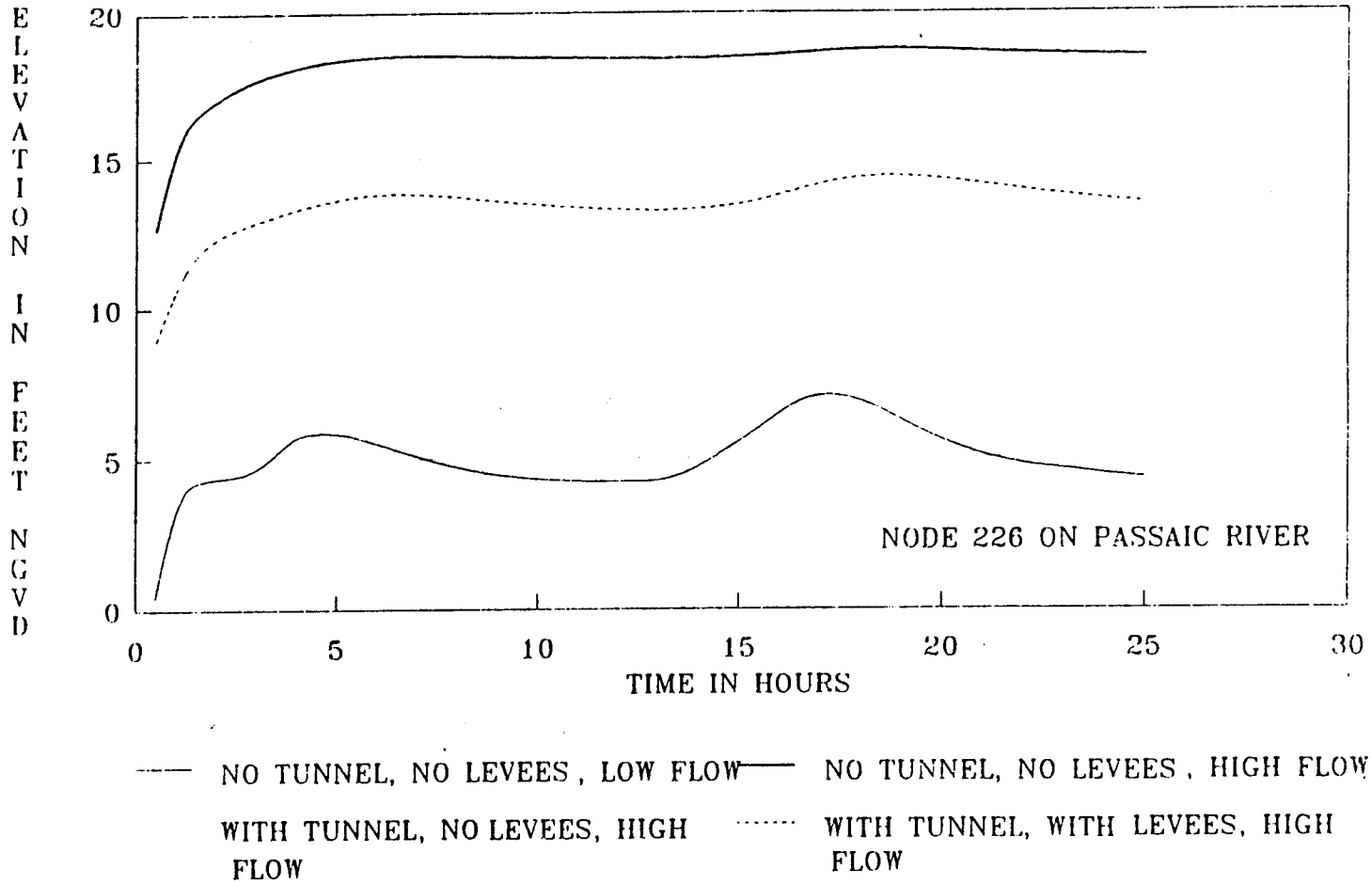


Figure C-81e

Figure 82. Predicted water level elevations at Snake Hill for year 2050 conditions and synthetic hurricane 249

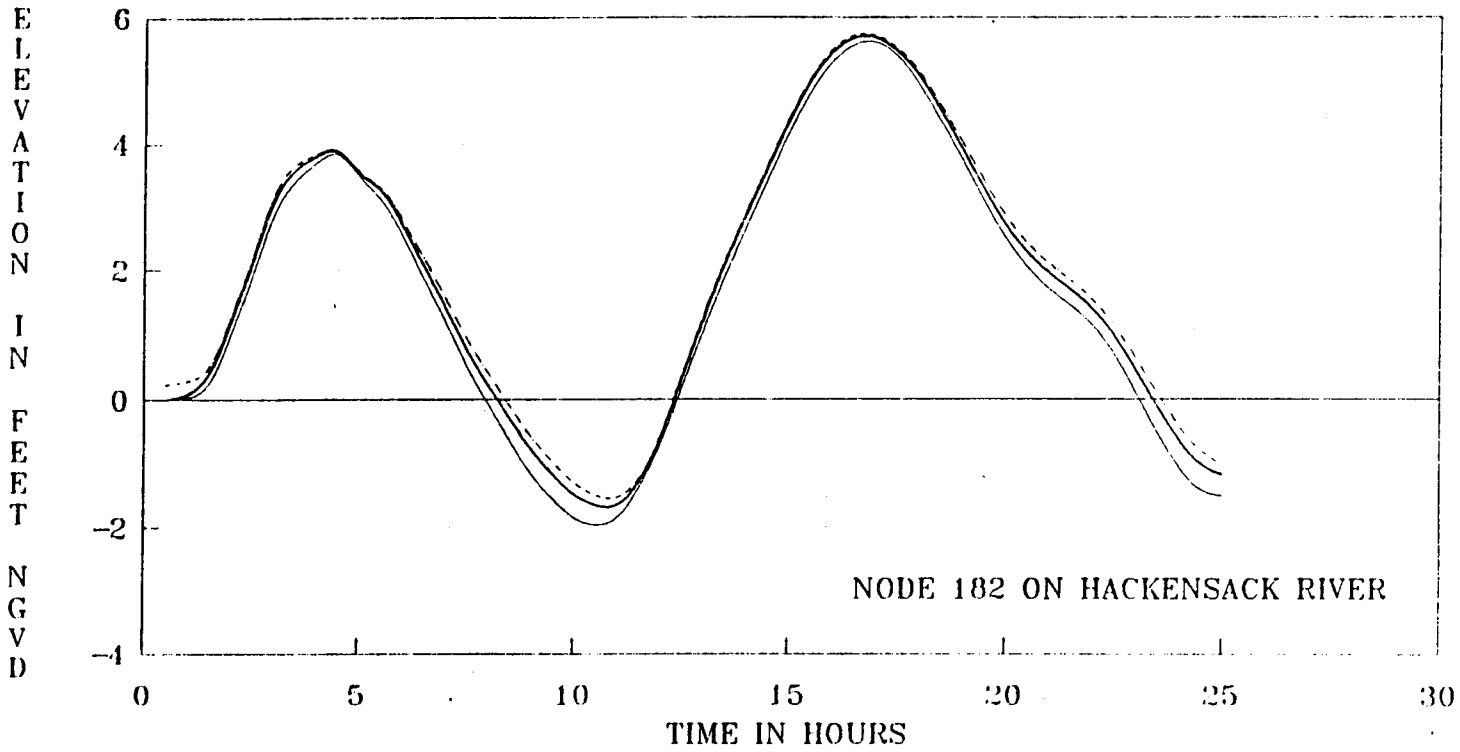


Figure 83. Predicted water level elevations at Berry's Creek for year 2050 conditions and synthetic hurricane 249

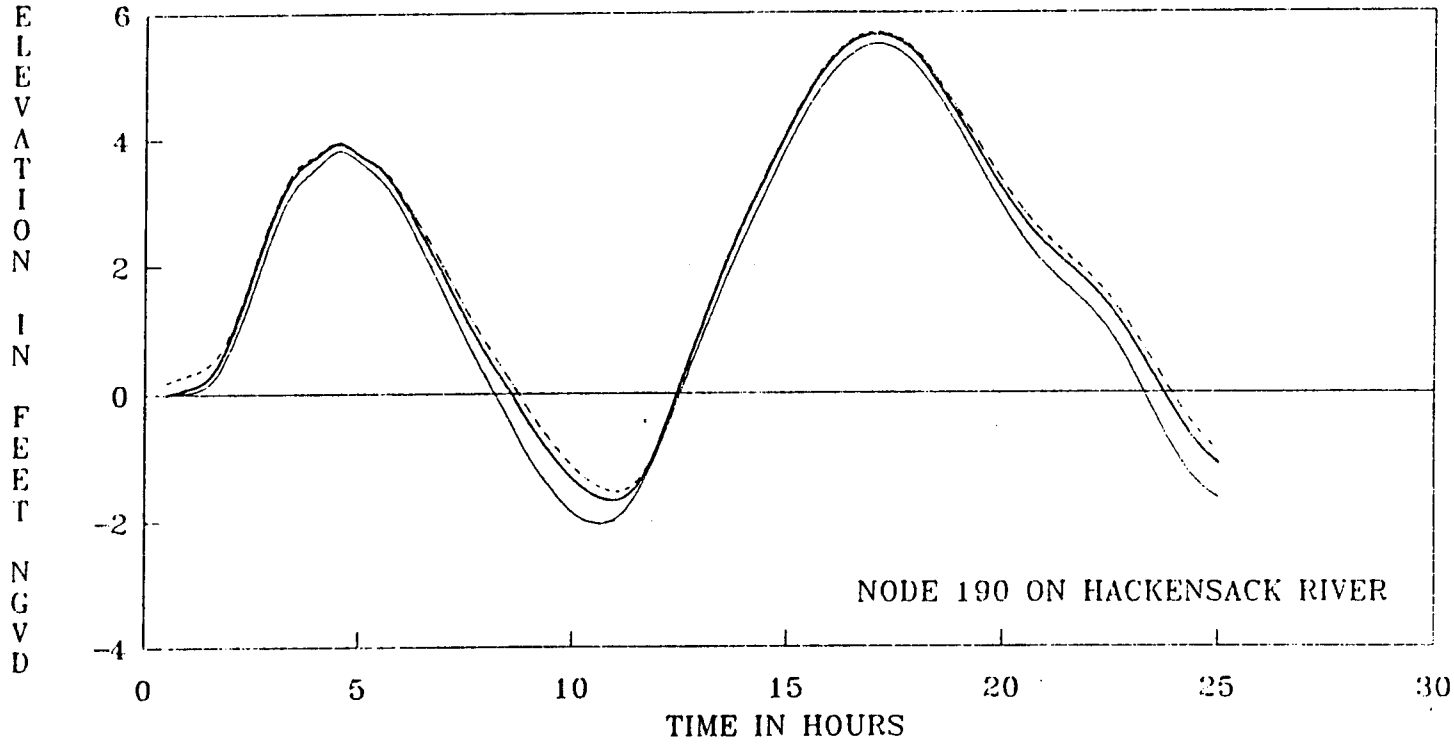
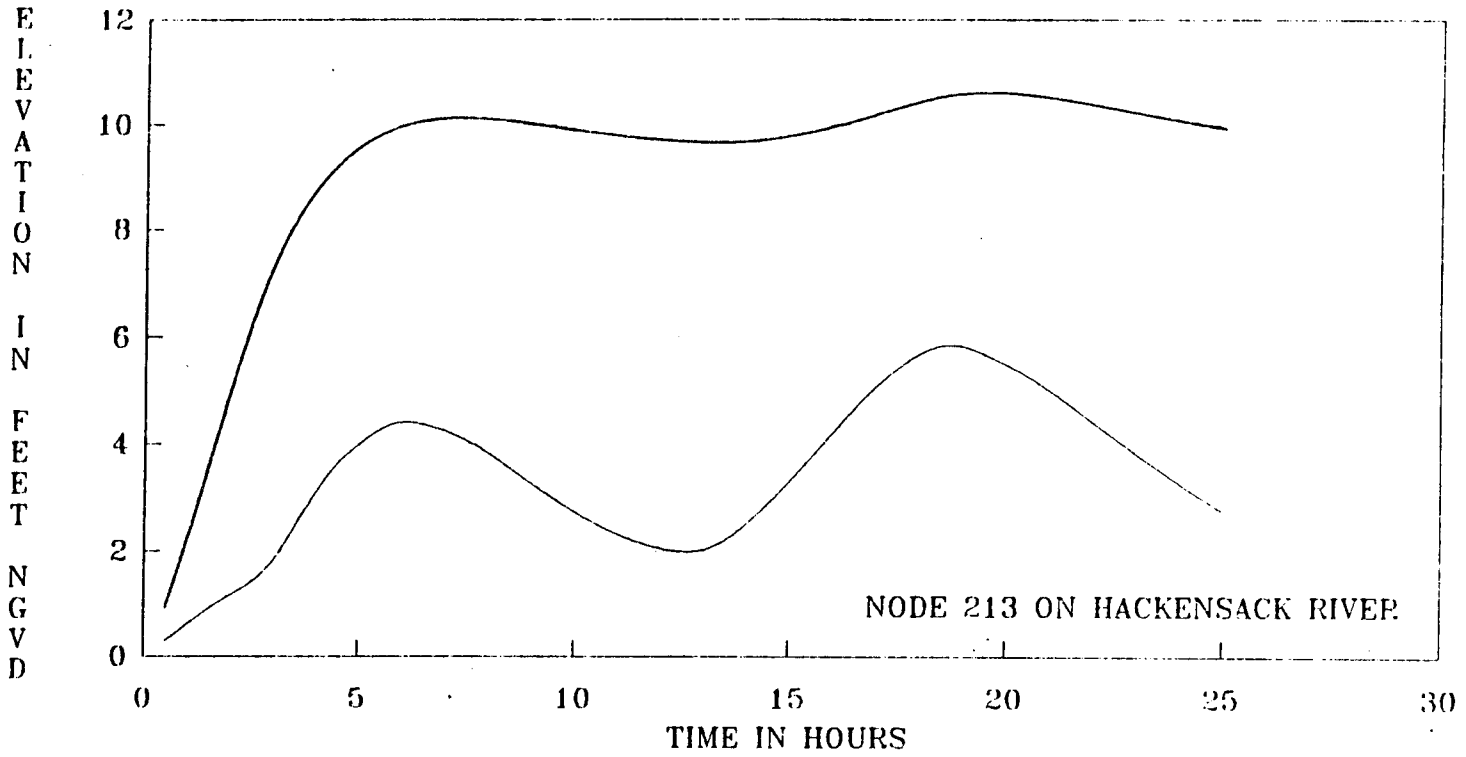


Figure 84. Predicted water level elevations at New Milford for year 2050 conditions and synthetic hurricane 249



Stage-frequency curves for
node 49 (Newark Airport) in Newark Bay
for 33 synthetic storms

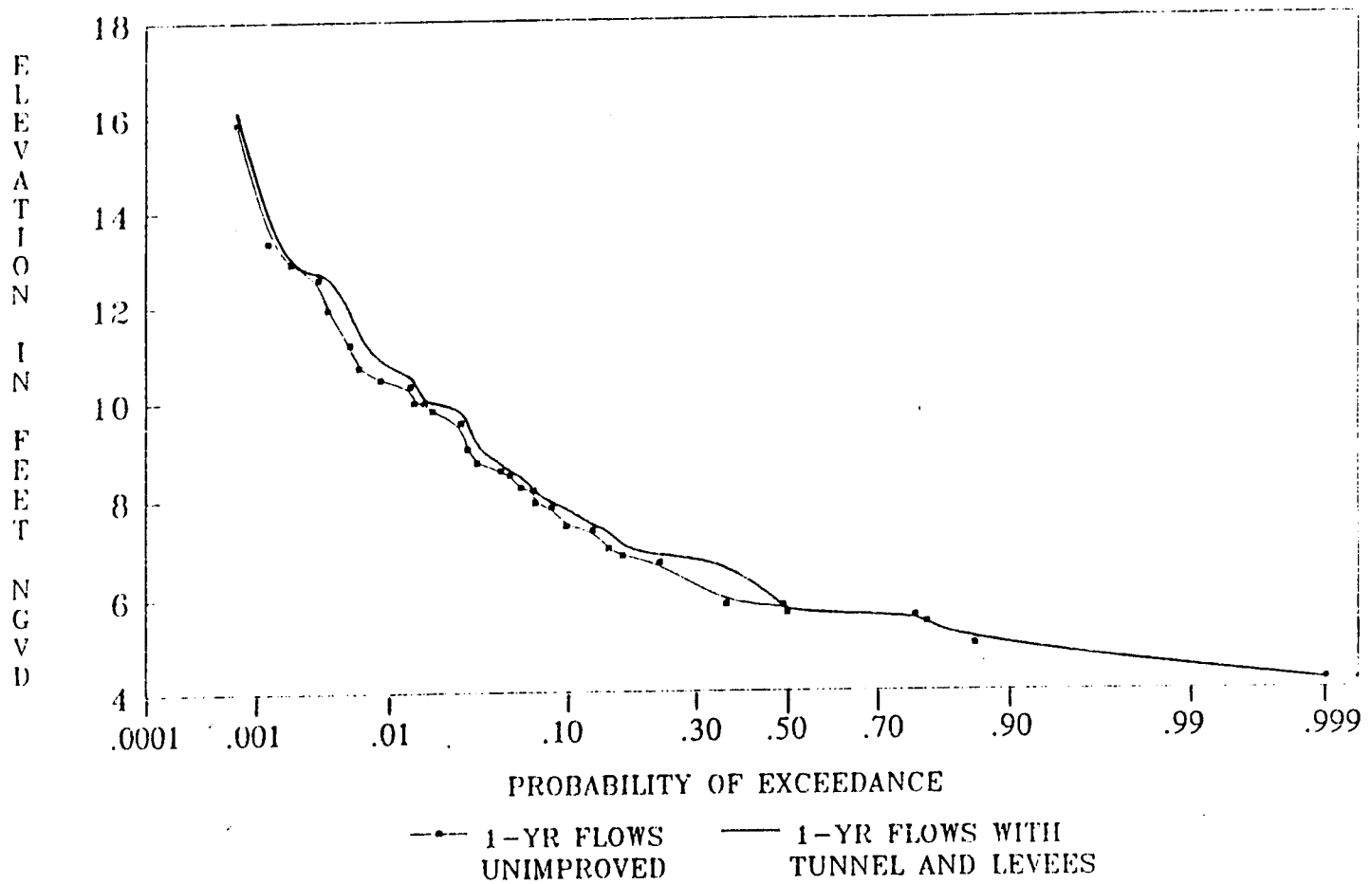


Figure C-82a

Stage-frequency curves for
node 262 (Kearny Point) in Newark Bay
for 33 synthetic storms

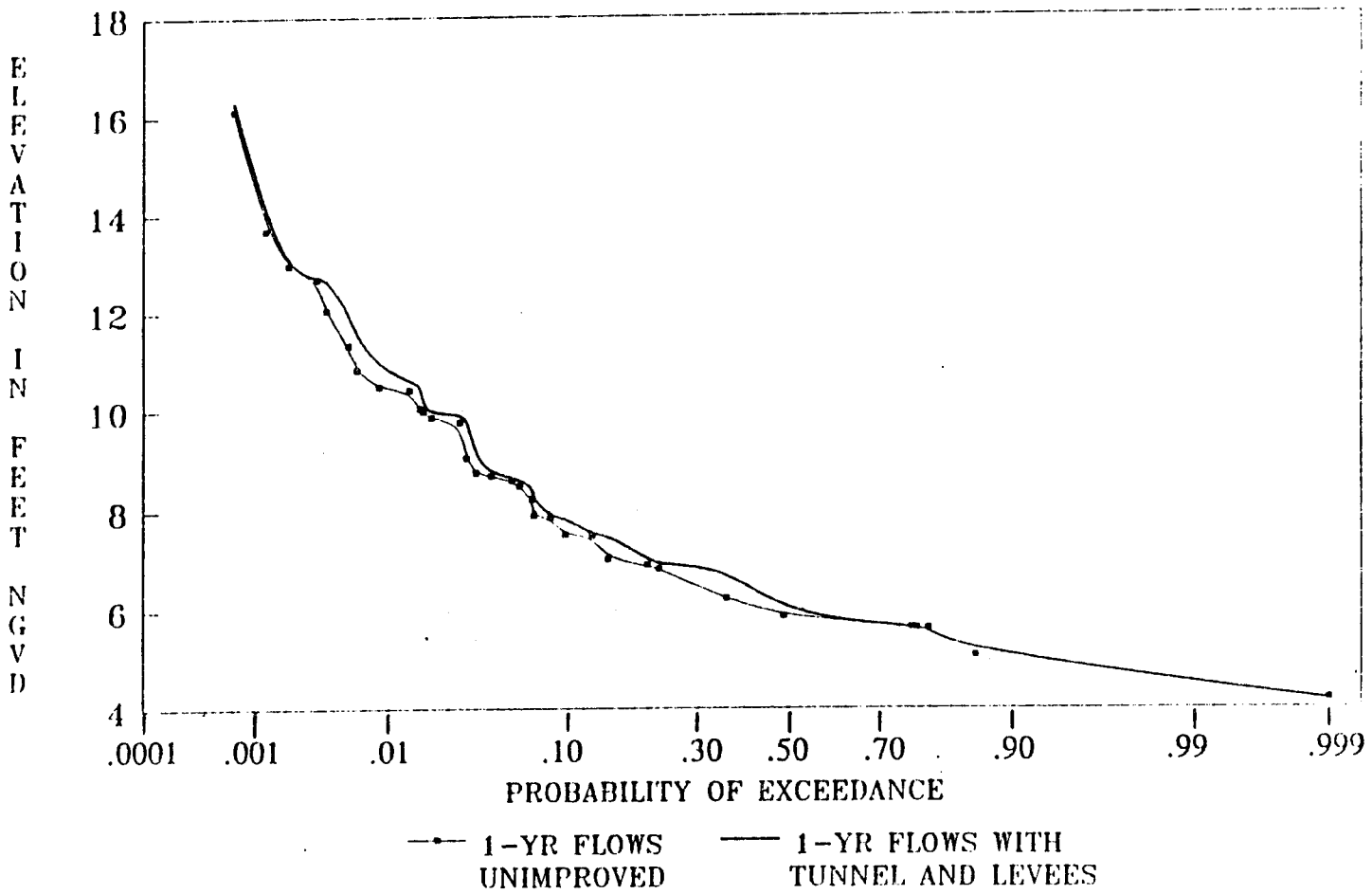


Figure C-82b

Stage-frequency curves for
node 173 (East Newark) on the Passaic
River for 33 synthetic storms

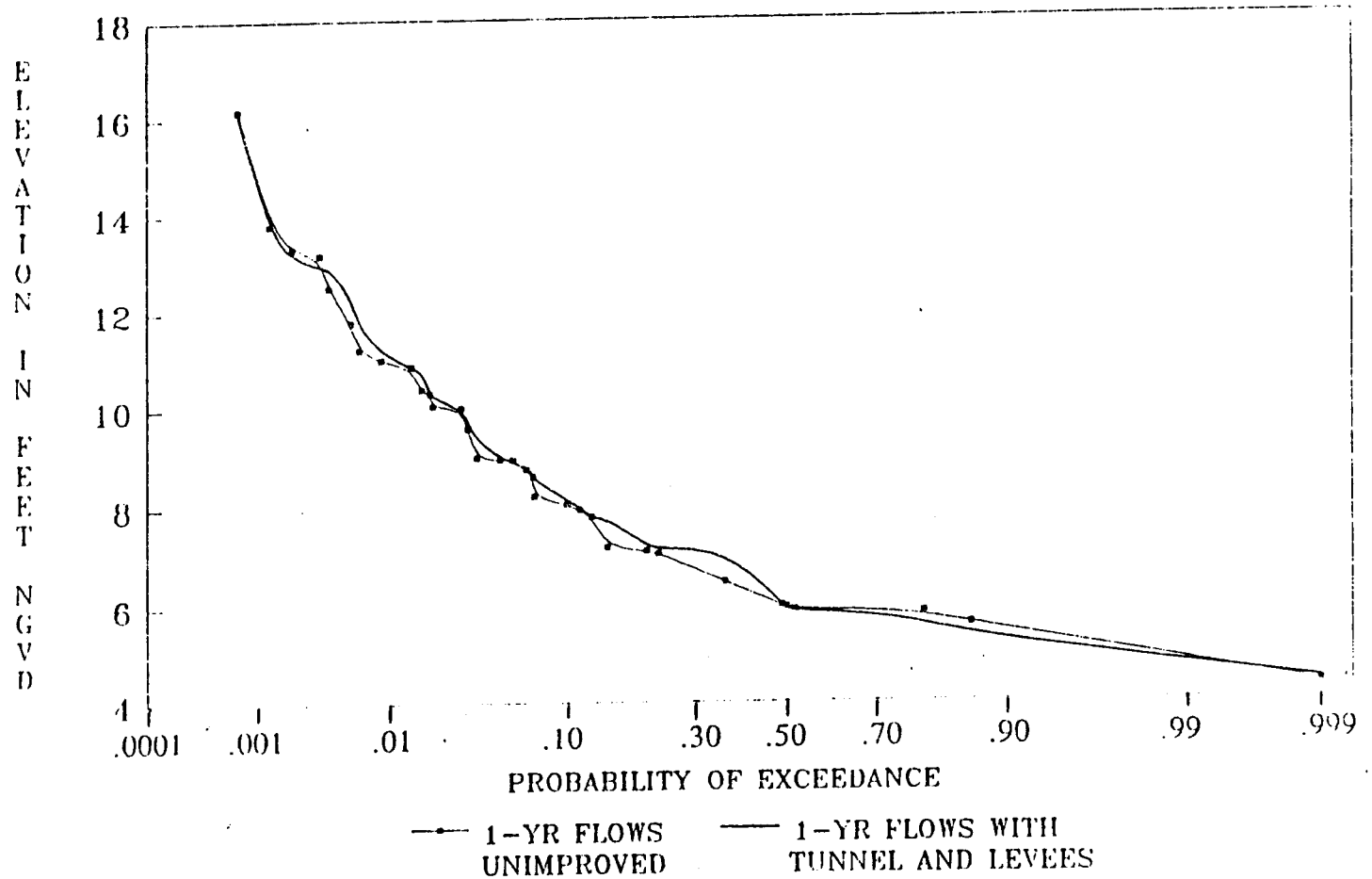


Figure C-82c

Stage-frequency curves for
node 217 (Third River) on the Passaic
River for 33 synthetic storms

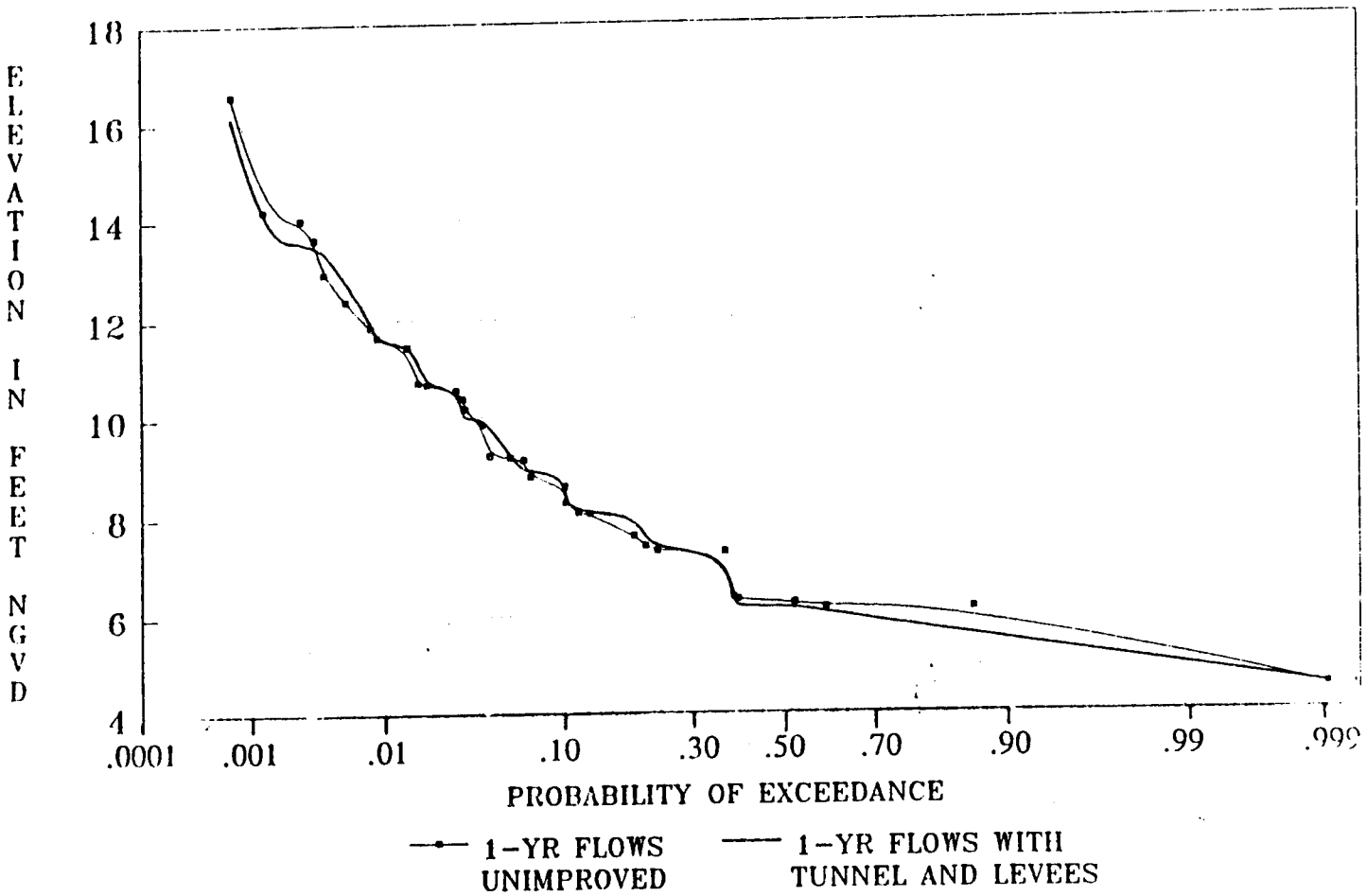


Figure C-82d

Stage-frequency curves for
node 226 (Garfield) on the Passaic
River for 33 synthetic storms

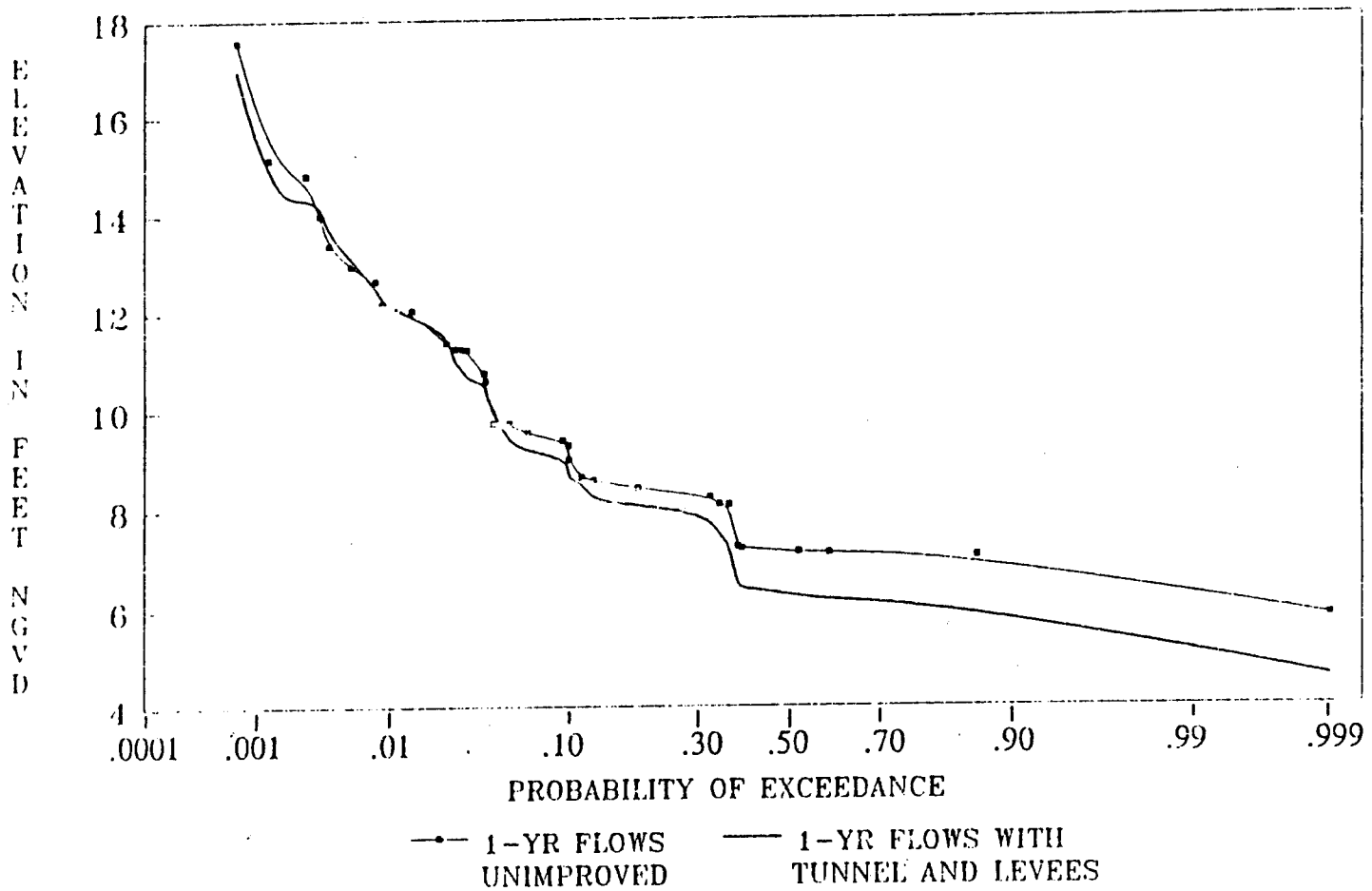


Figure C-82e

Stage-frequency curves for
node 182 (Snake Hill) on the Hackensack
River for 33 synthetic storms

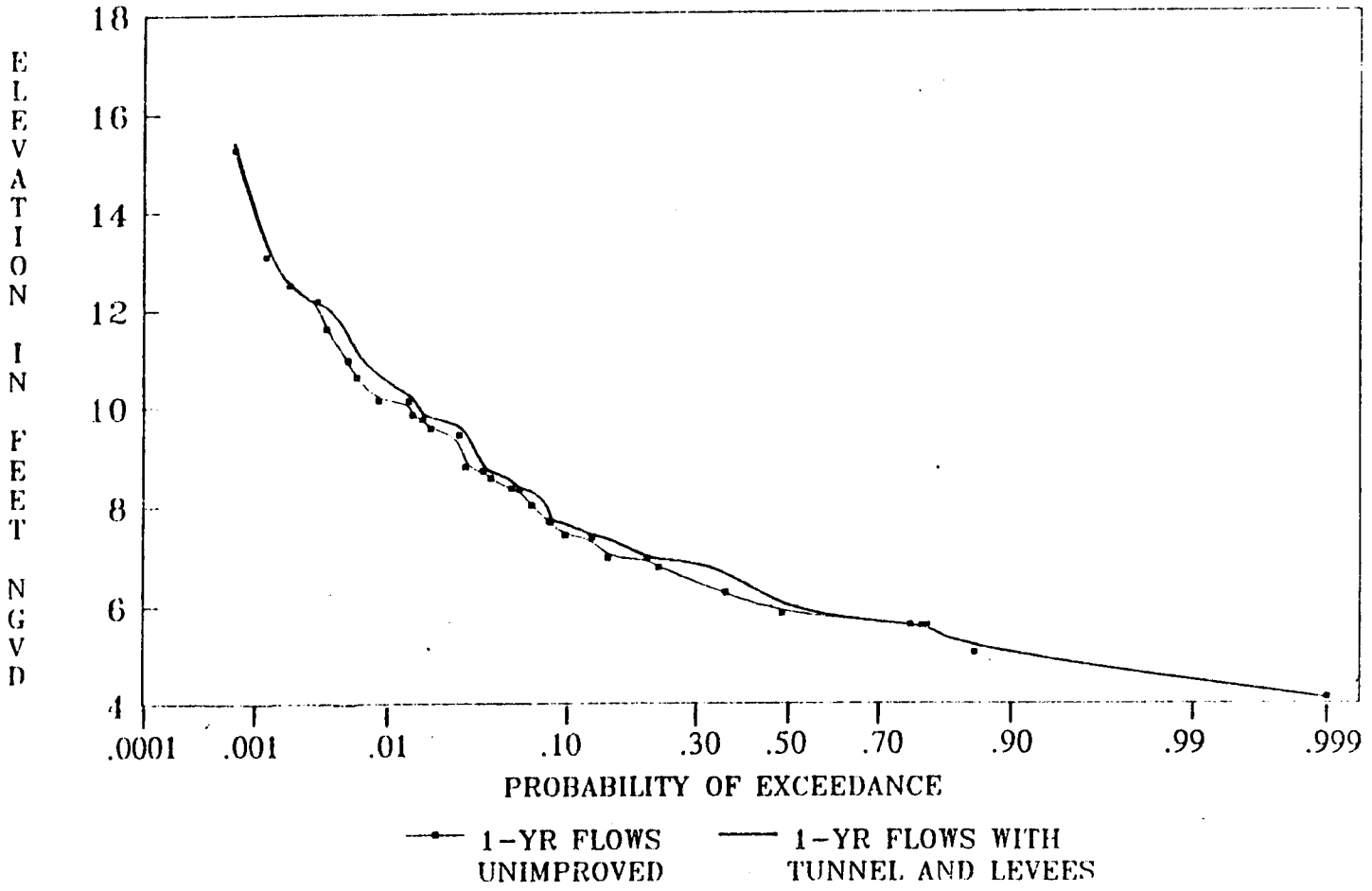


Figure C-82f

Stage-frequency curves for
node 190 (Berry's Creek) on the
Hackensack River for 33 synthetic storms

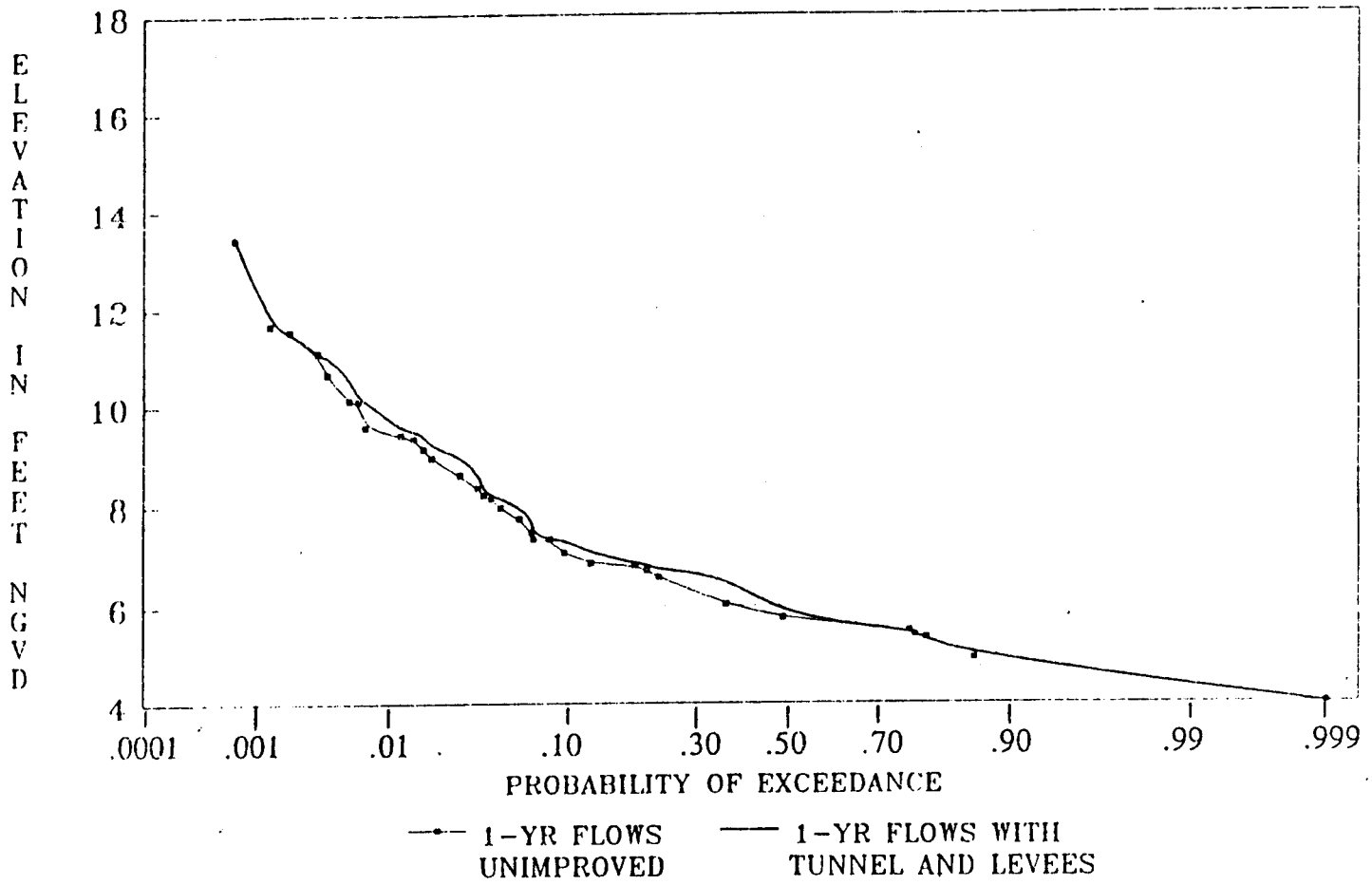


Figure C-82g

Stage-frequency curves for
node 213 (New Milford) on the Hackensack
River for 33 synthetic storms

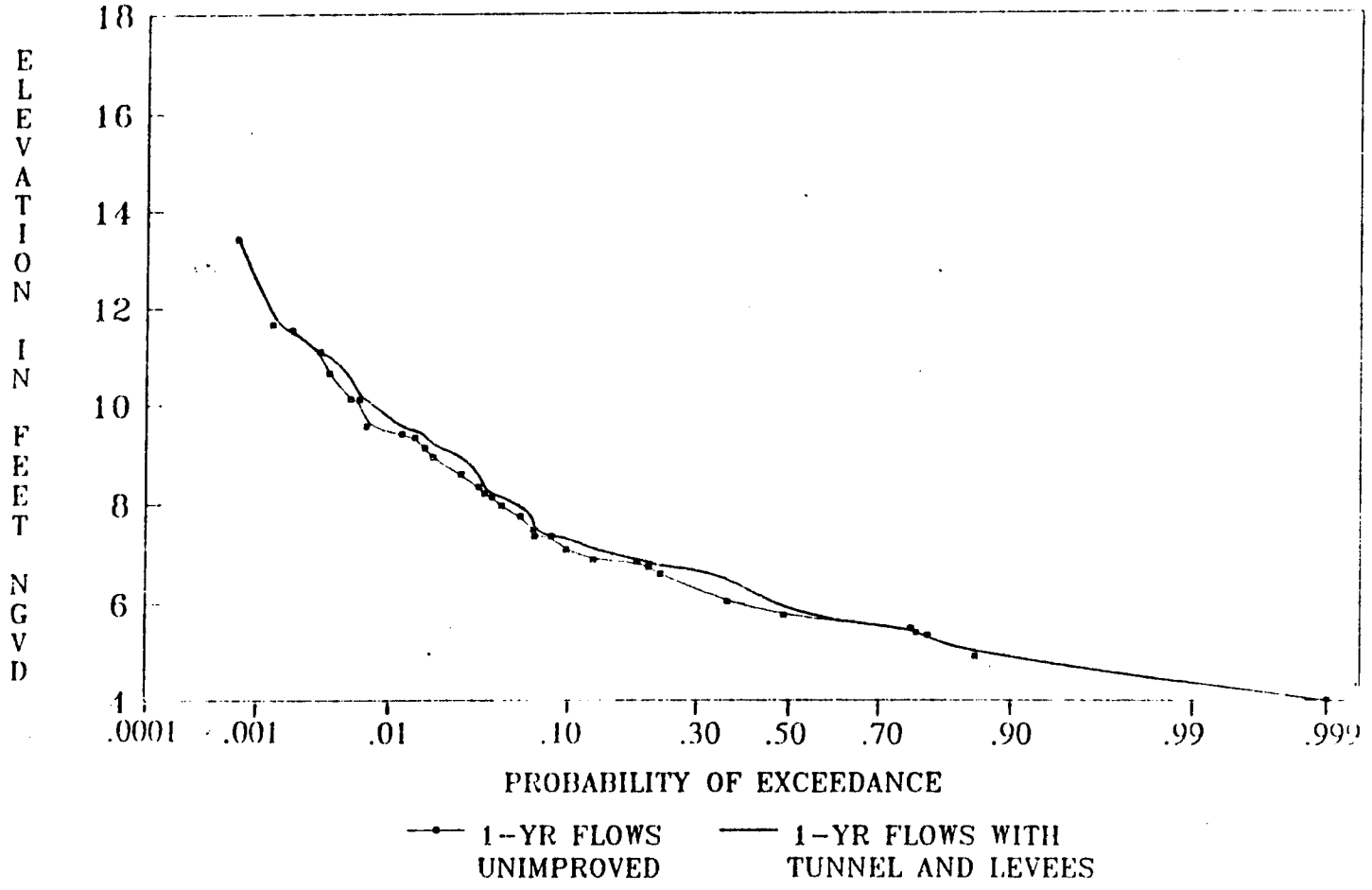
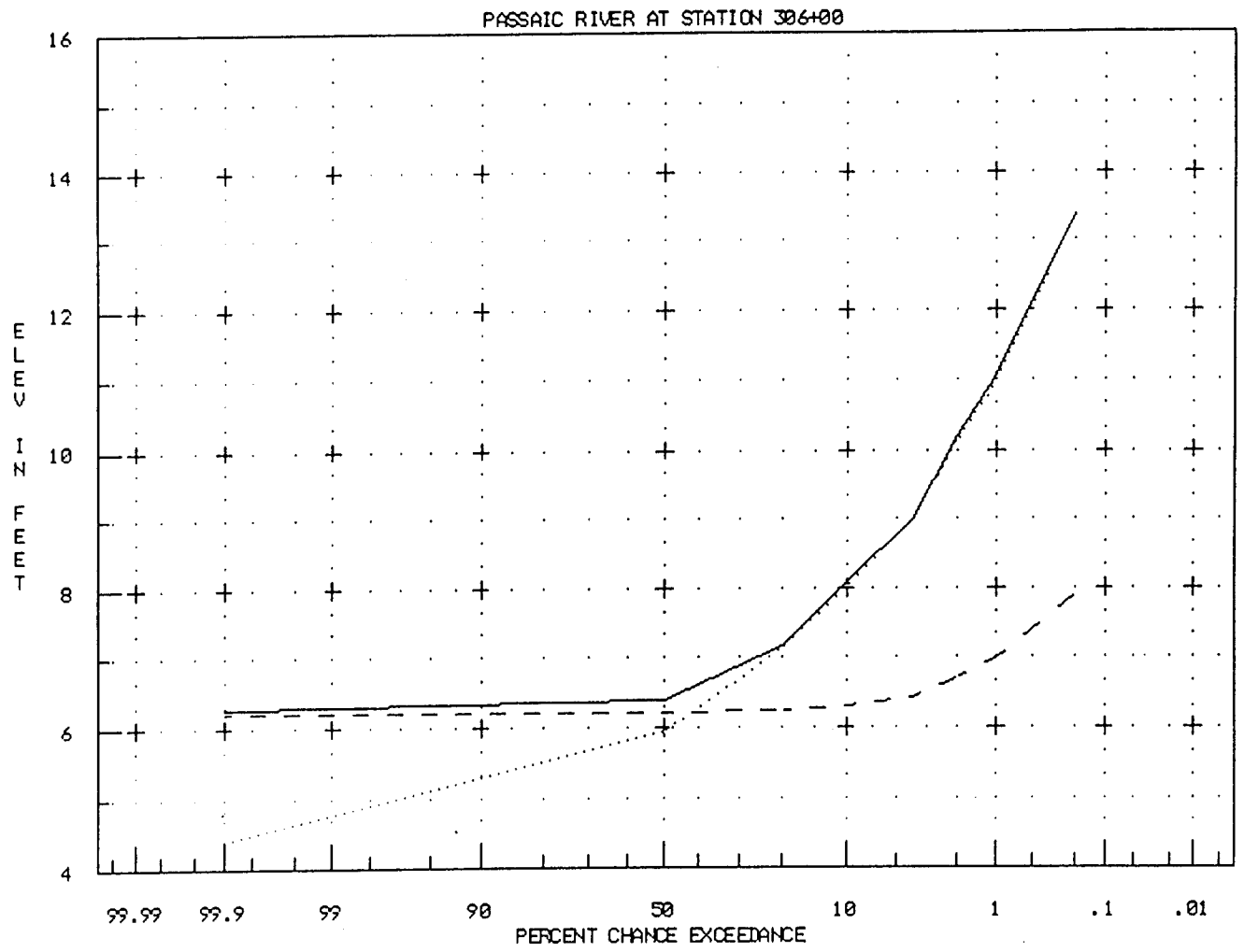


Figure C-82h

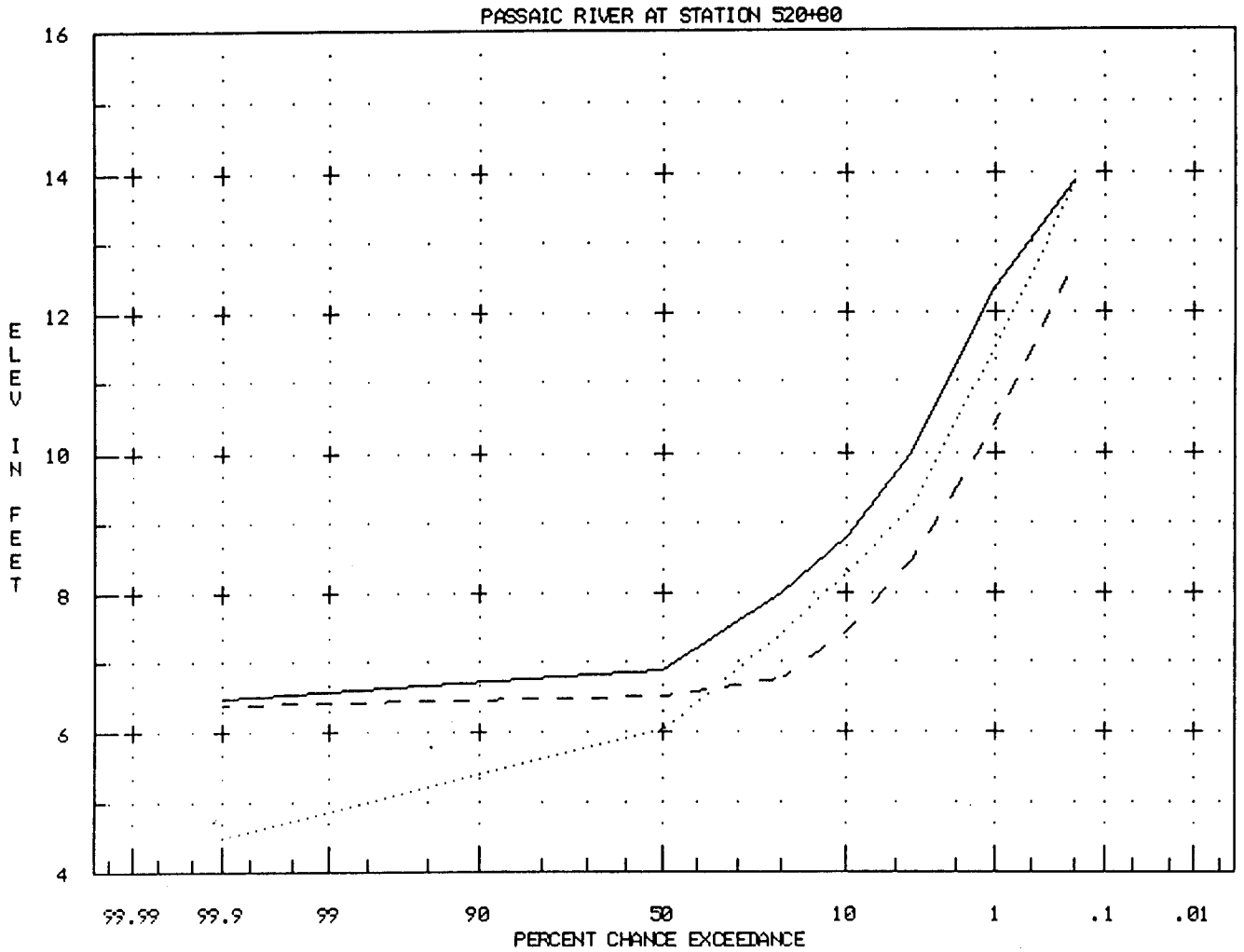
09MAY95 09:03:37



010021 2050-STORM SURGE
010021 2050-FLUVIAL-IMP
010021 2050-COMBINED-IMP

Figure C-83a

09MAY95 09:13:54



010052 2050-STORM SURGE
010052 2050-FLUVIAL-IMP
010052 2050-COMBINED-IMP

Figure C-83b

09MAY95 10:34:14

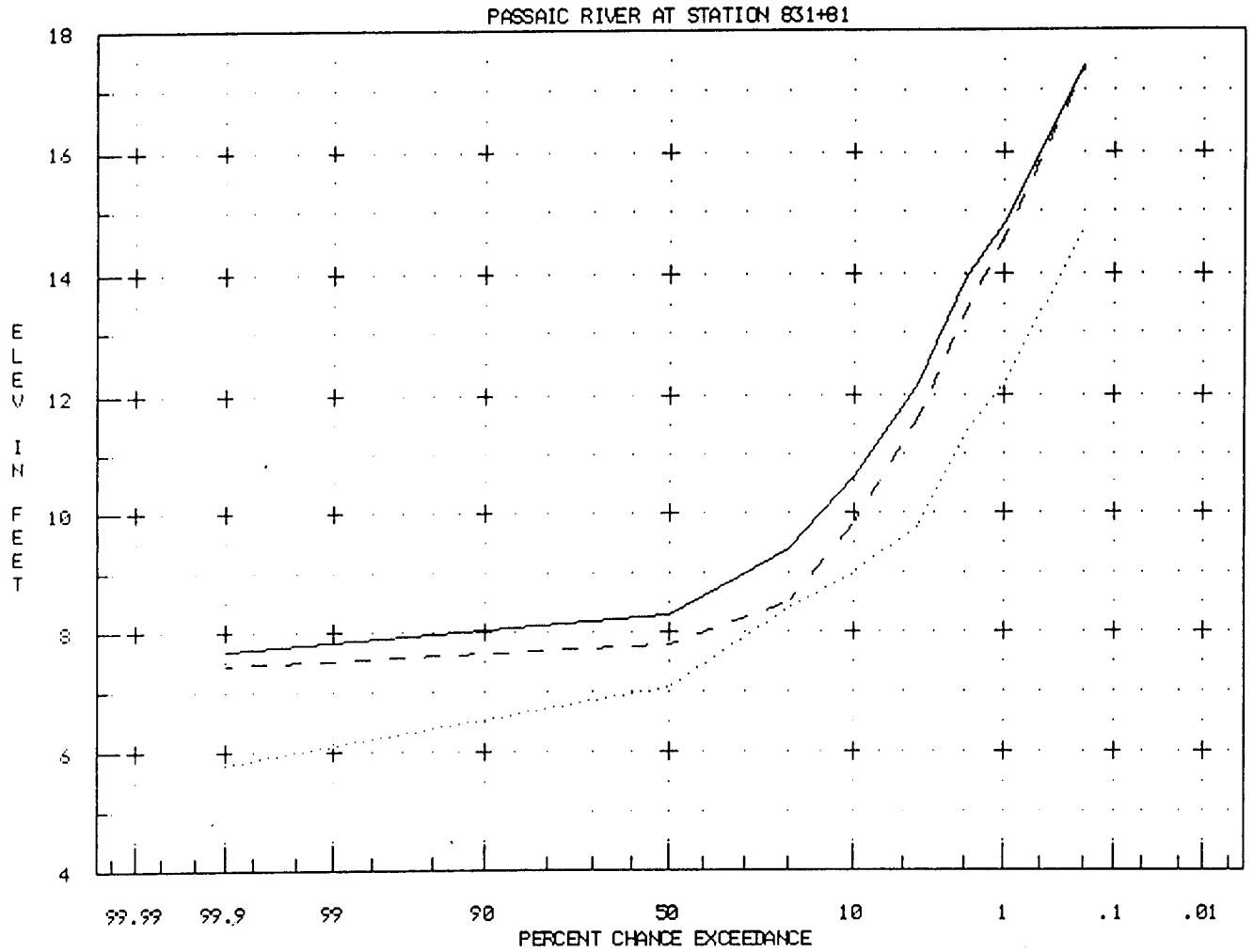
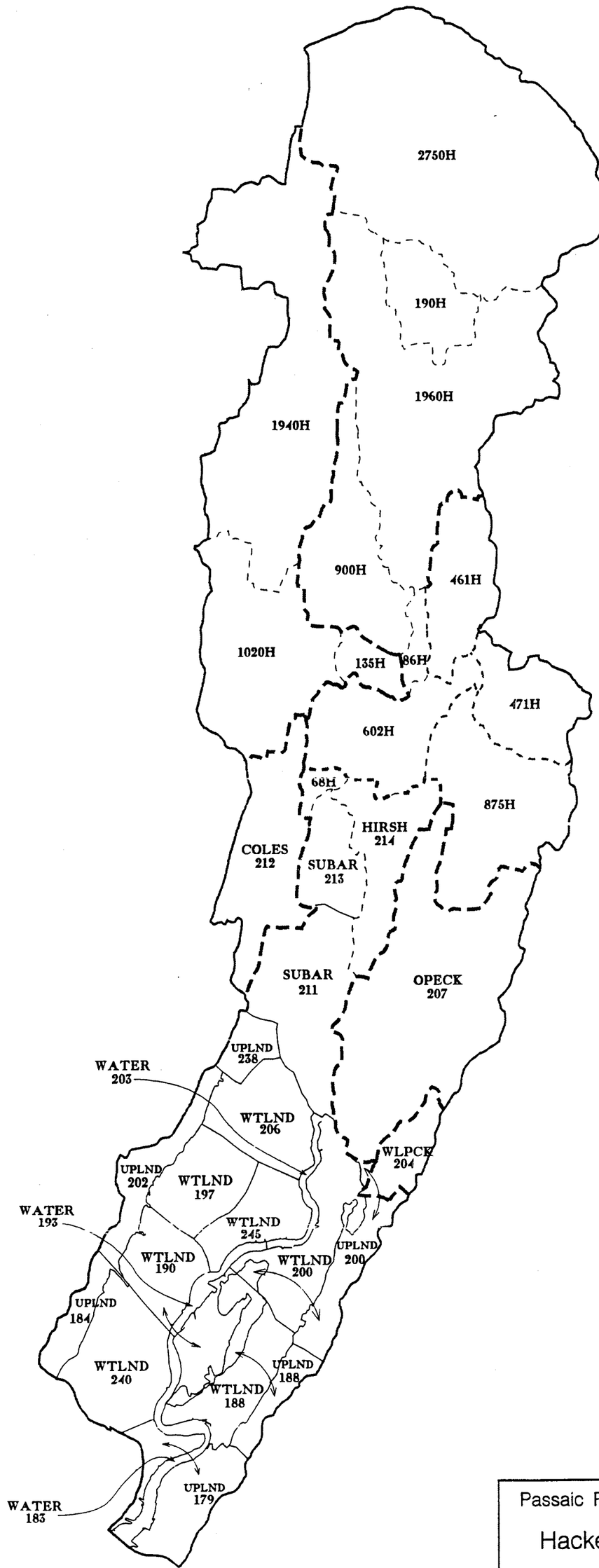


Figure C-83c



Passaic River Flood Control Project

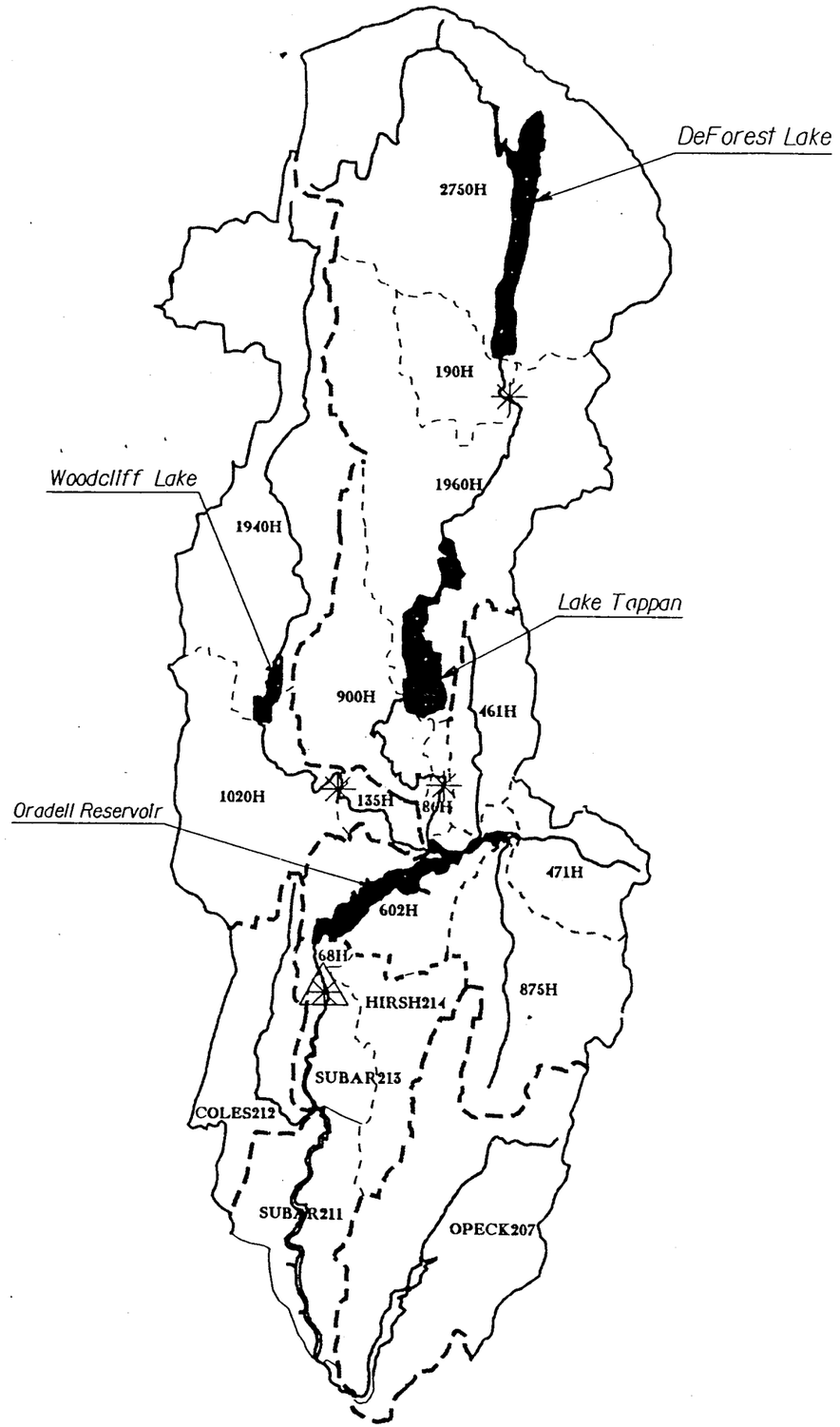
Hackensack River Basin

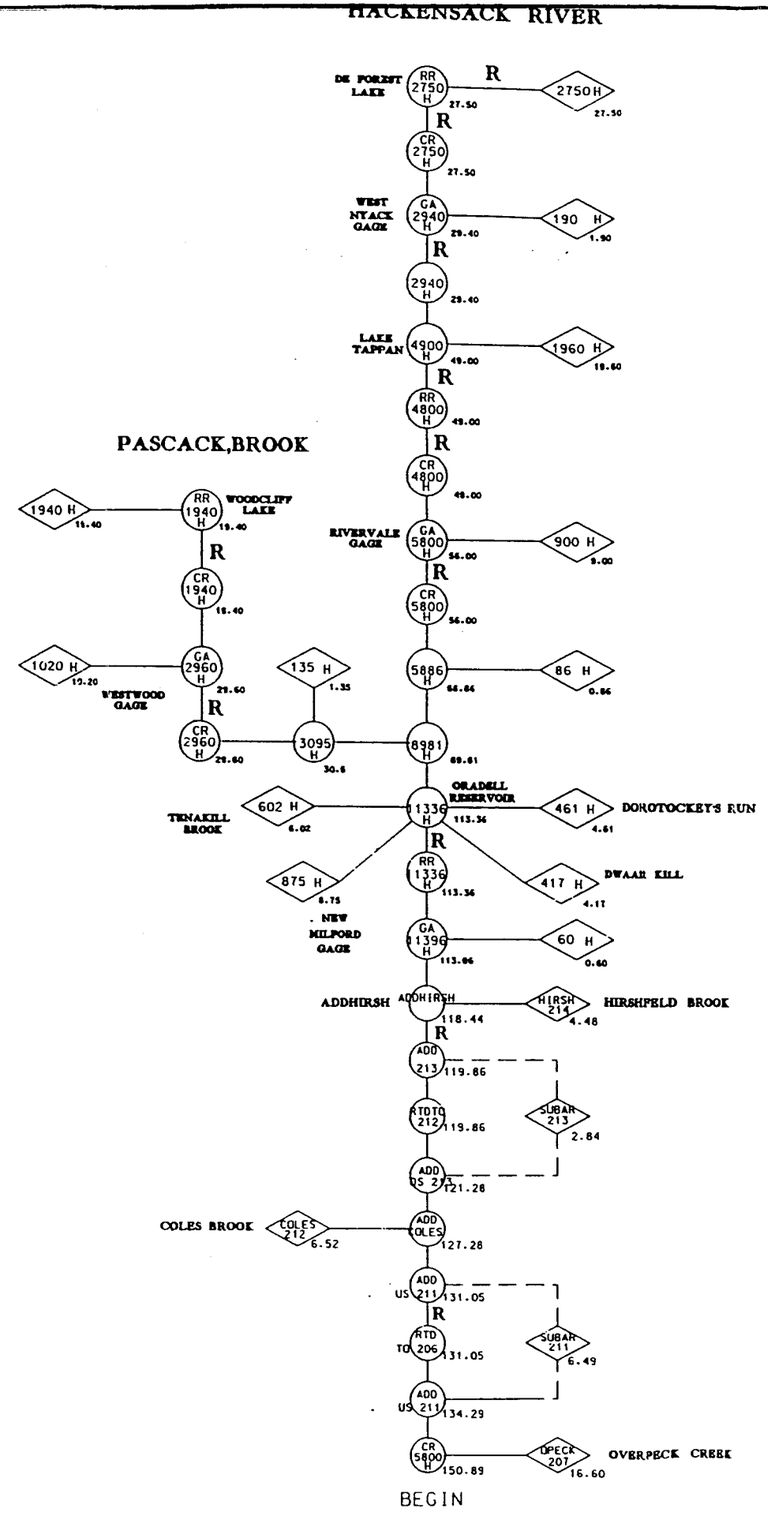
HEC1

Subarea Delineation

CENAN-PR-C AUGUST 1992

Figure C-84a





KEY

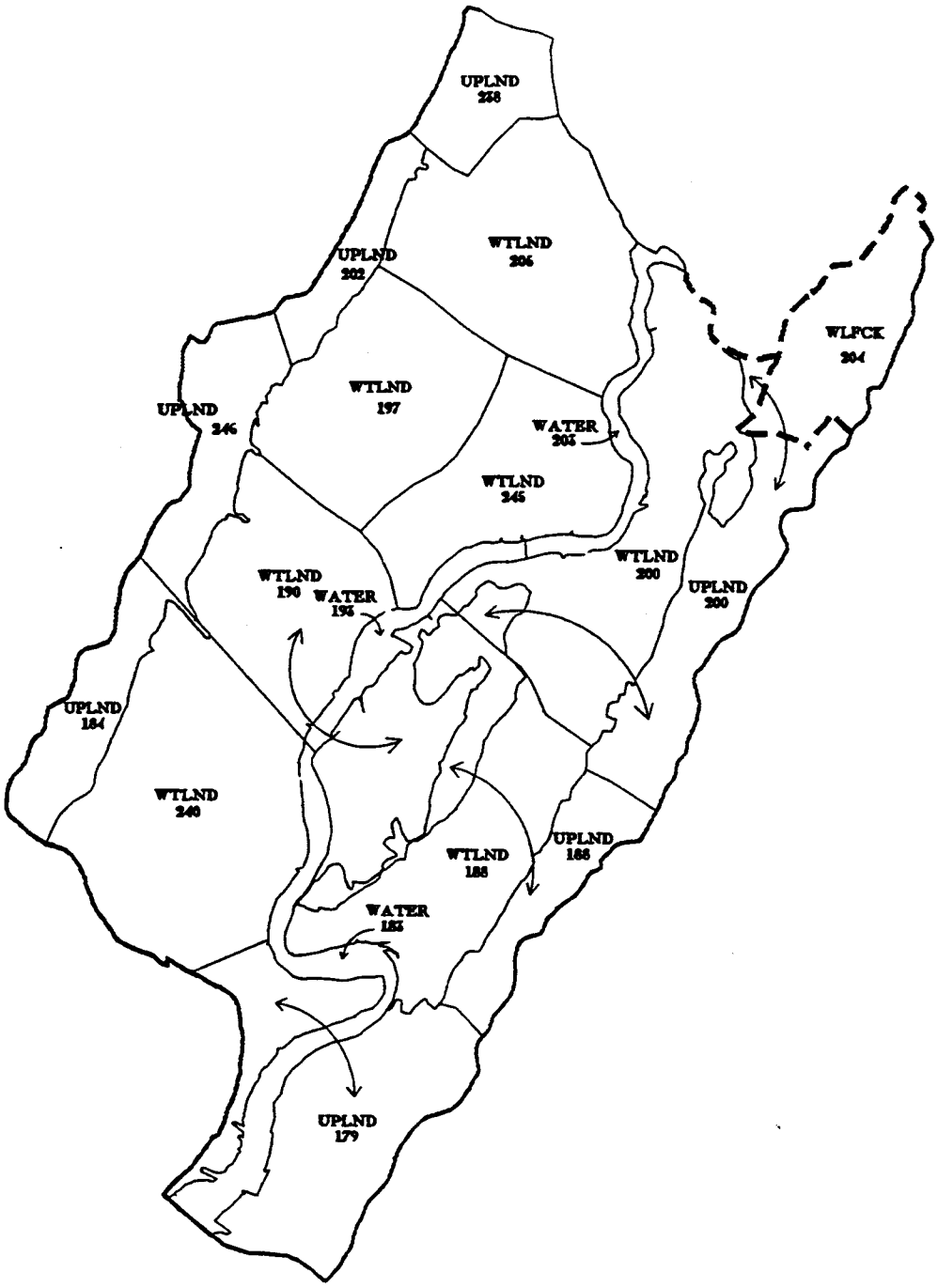
- Subbasin Number and Drainage Area
- Node Point Number
- USGS Gage Location
- R — Route Reach
- HEC1 Distrubuted Flow (see Text)

Passaic River Study
HACKENSACK RIVER
UPPER BASIN

HEC1/UNET SUBAREA NETWORK DIAGRAM

CENAN-PR-C AUGUST 1992

Figure C-84b



Passaic River Flood Control Project

Hackensack River Basin

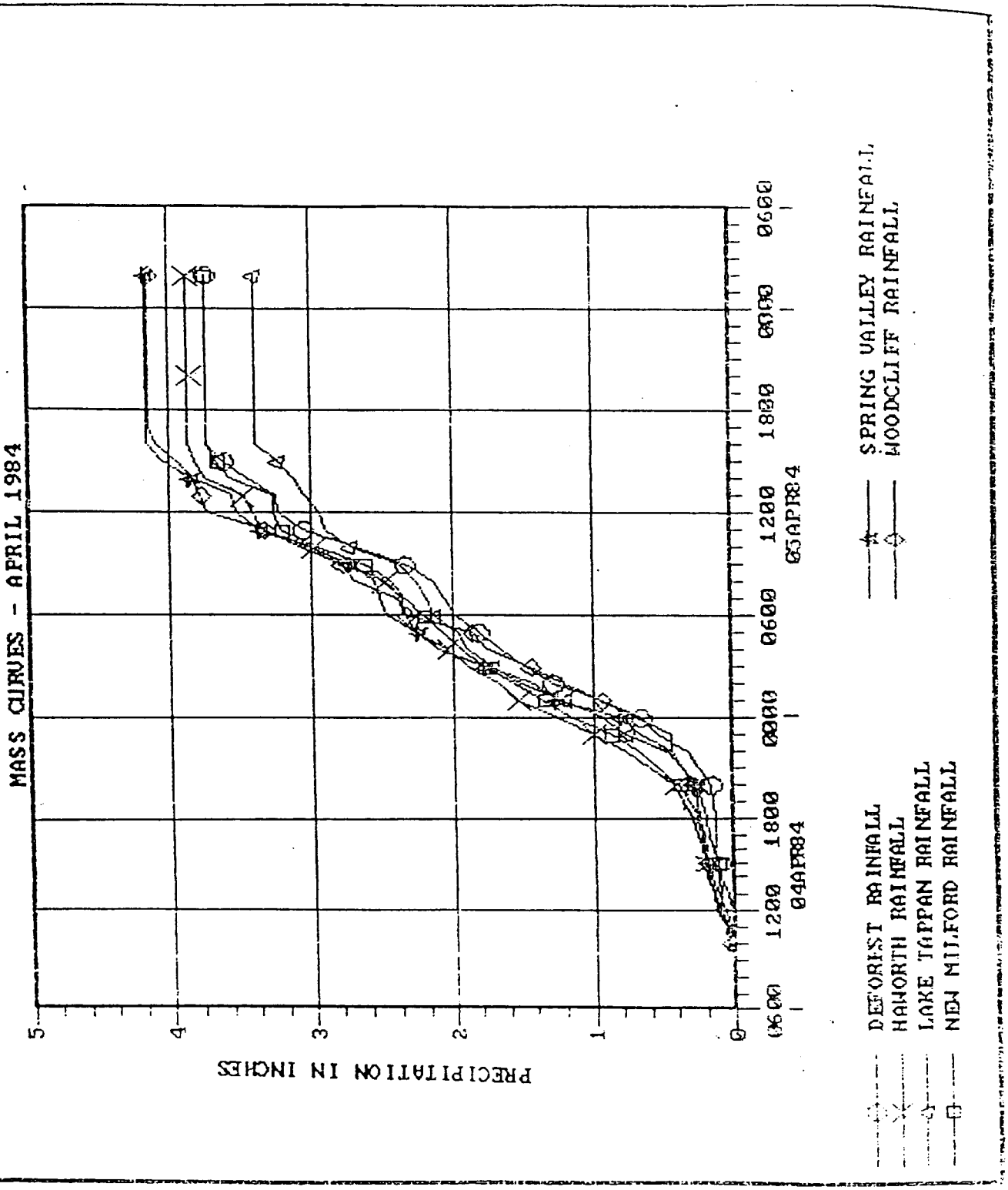
LOWER BASIN

HEC1

Subarea Delineation

CENAN-PR-C AUGUST 1992

Figure C-84c

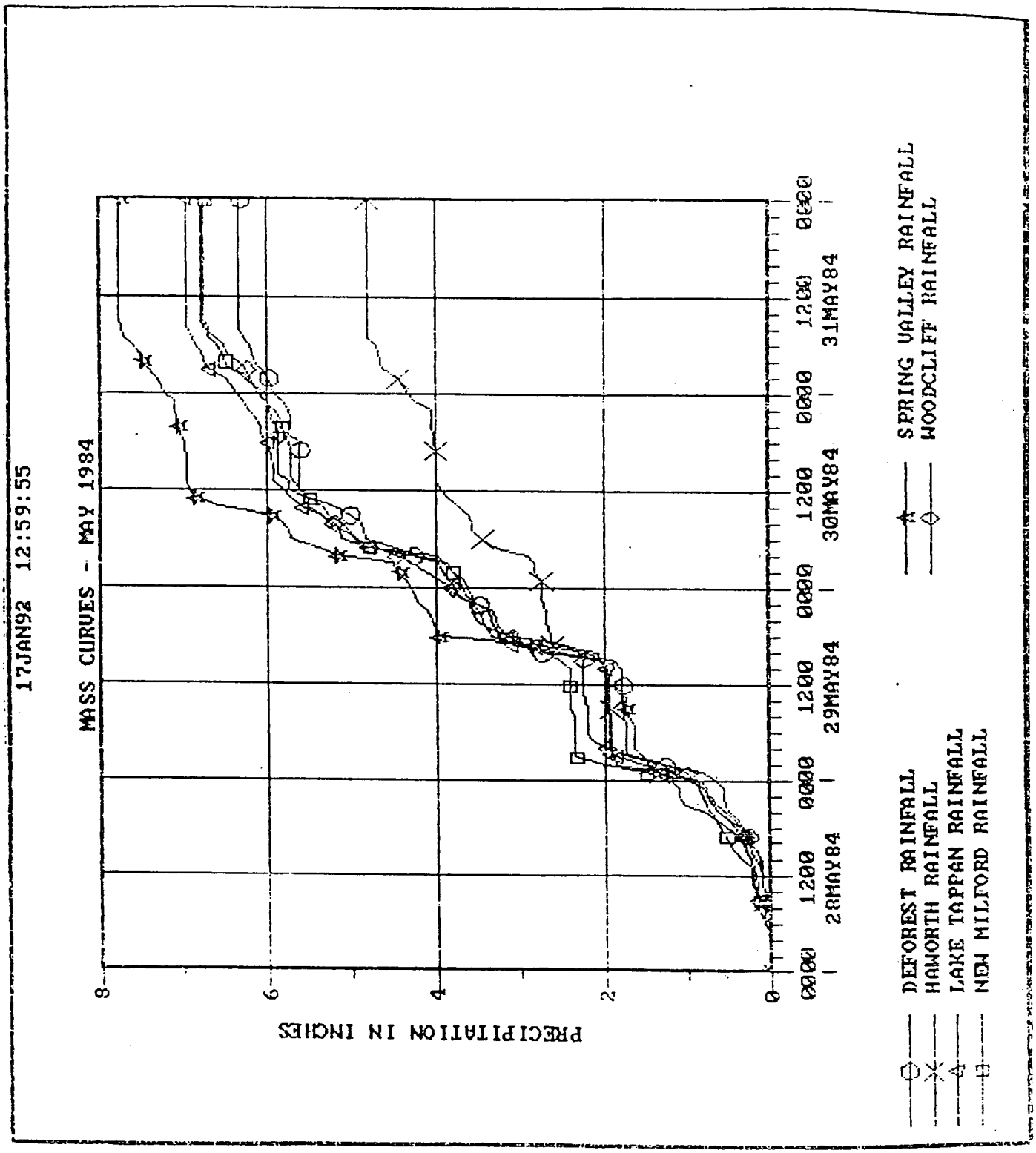


Passaic River Flood Protection Project
Hackensack Basin
Mass Curve Of Rainfall
April 1984

CENAN-PR-C

MARCH 1992

Figure C-85a

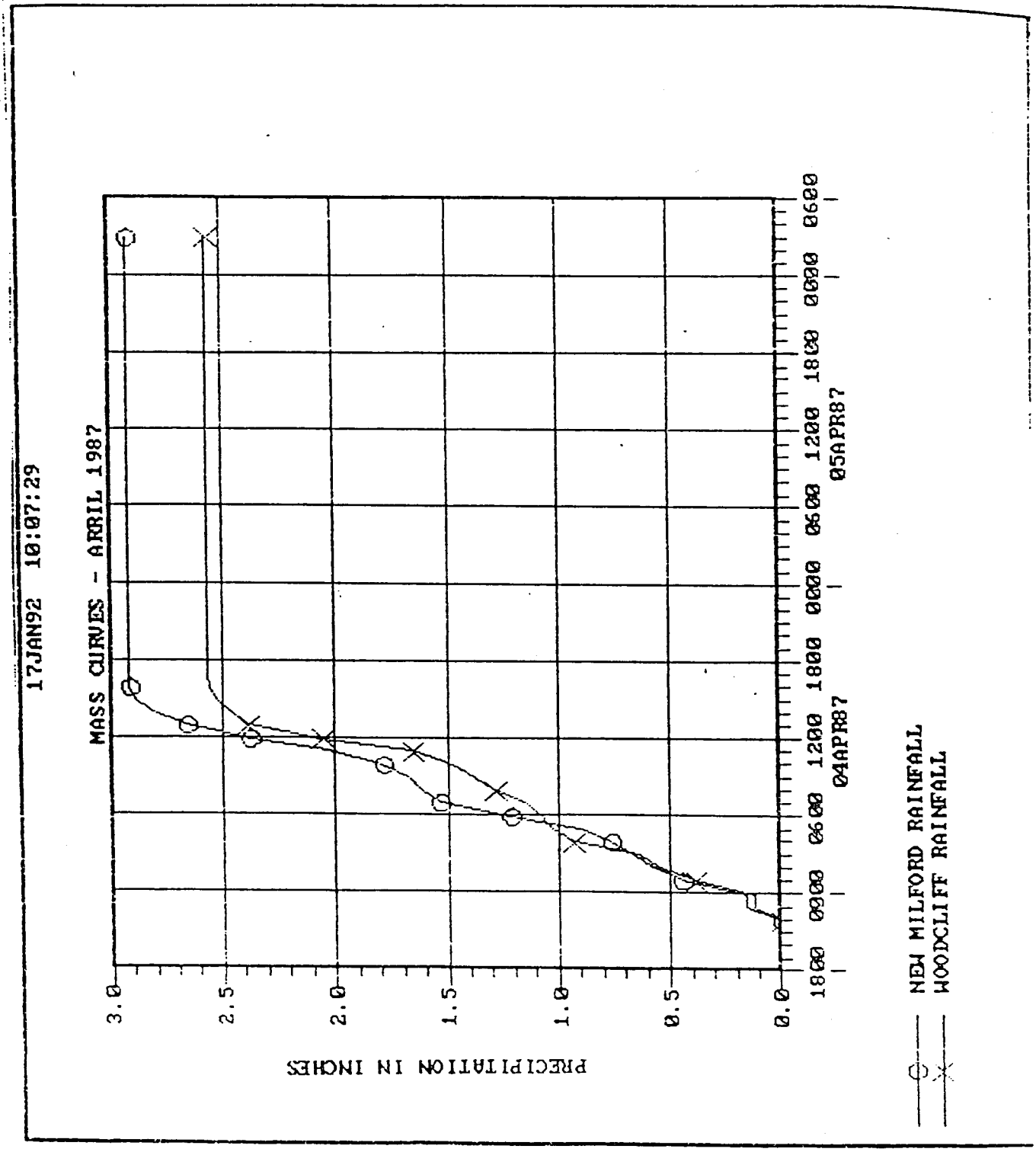


Passaic River Flood Protection Project
Hackensack Basin
Mass Curve of Rainfall
May 1984

CENAN-PR-C

MARCH 1992

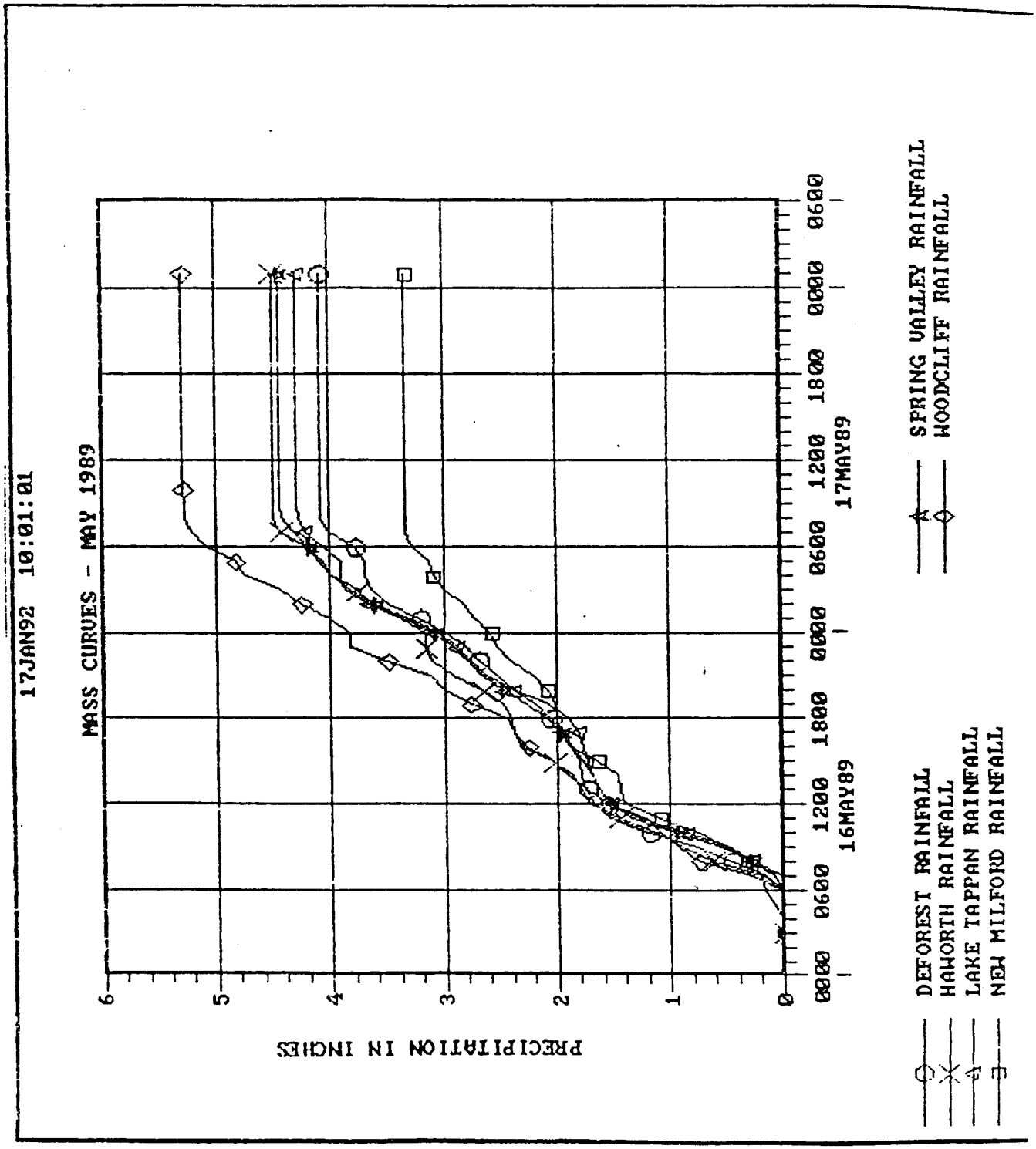
Figure C-85b



Passaic River Flood Protection Project
Hackensack Basin
Mass Curve of Rainfall
April 1987

CENAN-PR-C MARCH 199

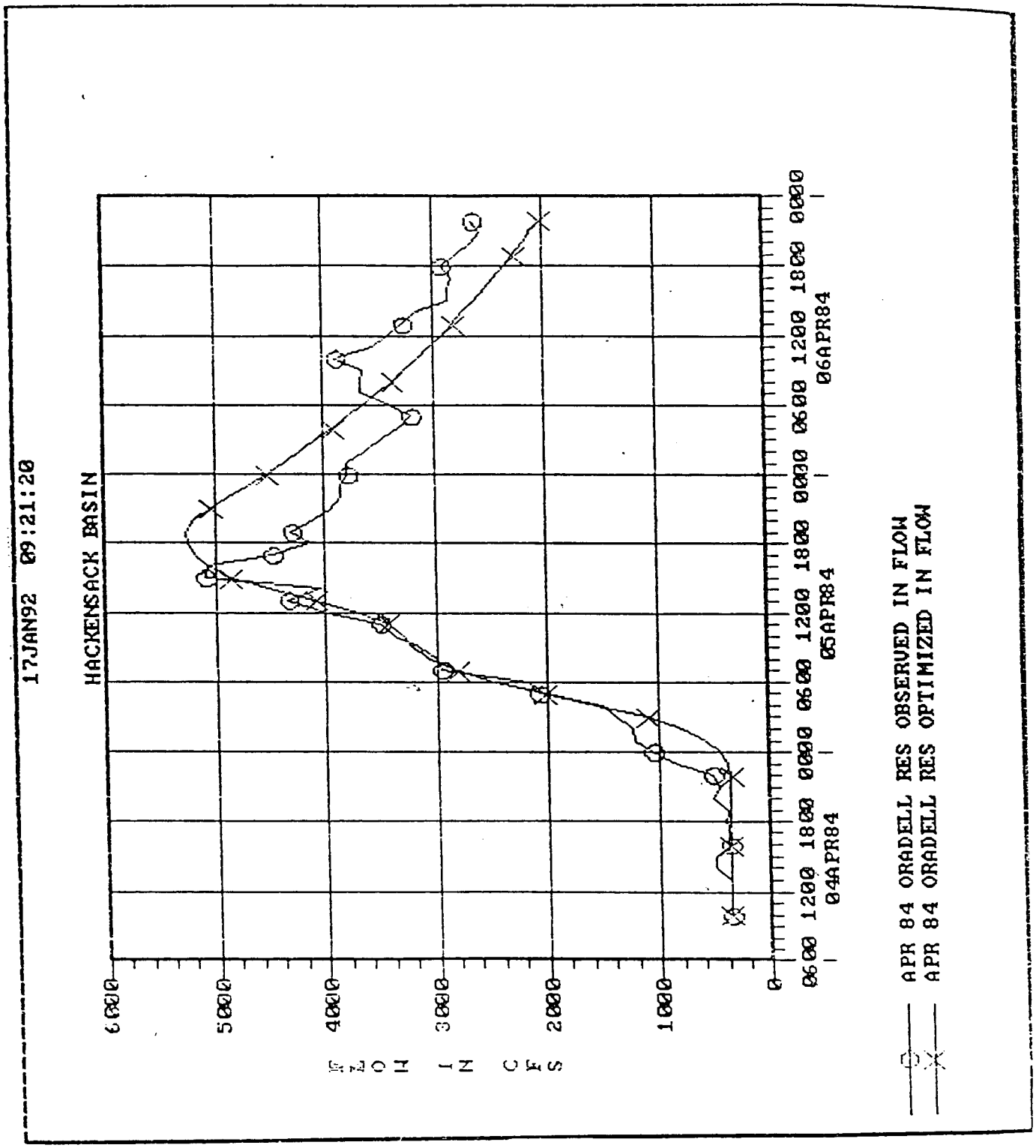
Figure C-85c



Passaic River Flood Protection Project
Hackensack Basin
Mass Curve of Rainfall
May 1989

CENAN-PR-C MARCH 199

Figure C-85d

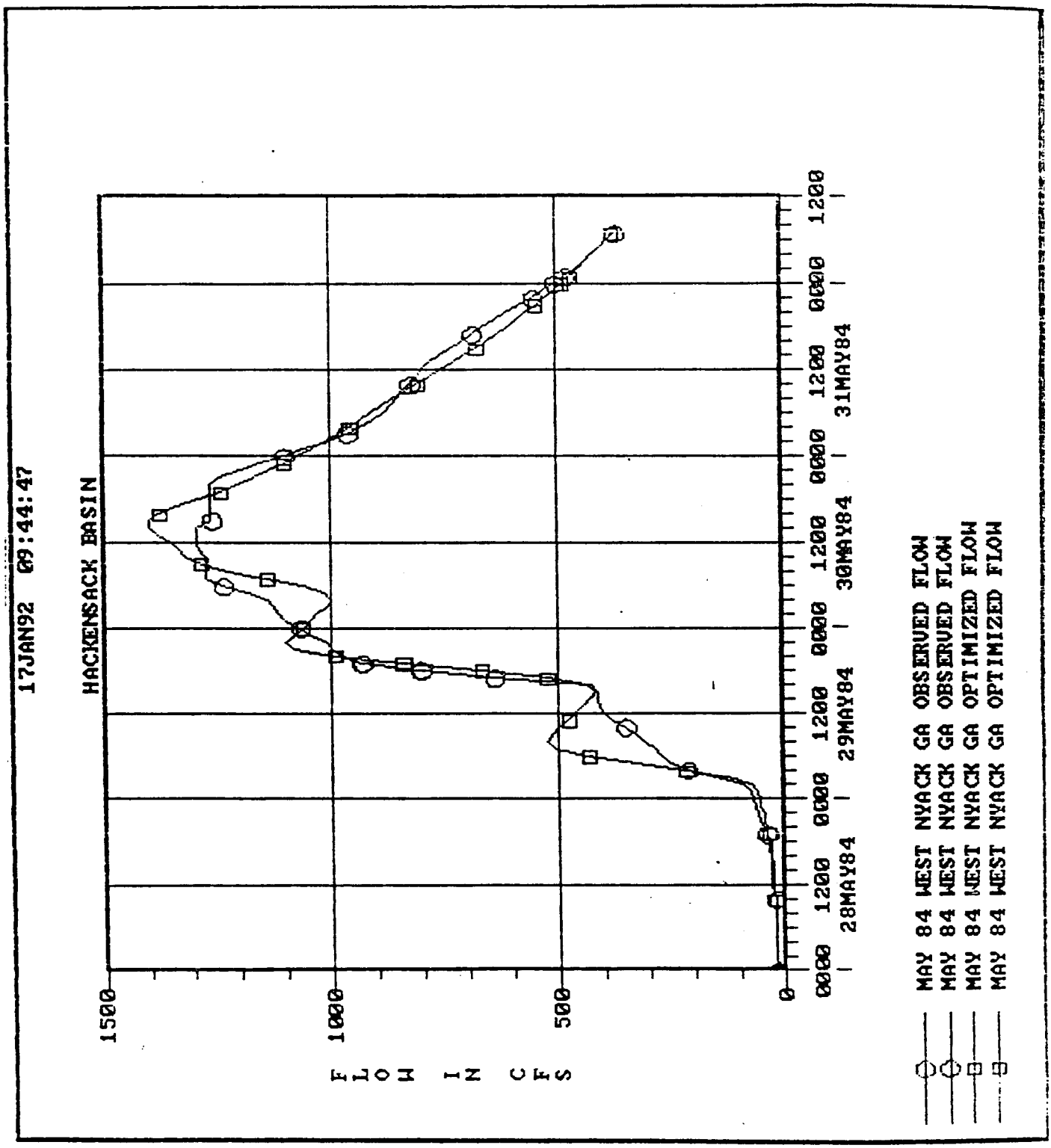


Passaic River Flood Protection Project
Hackensanck Basin
Optim. Analysis - Oradell
April 1984

CENAN-PR-C

MARCH 1992

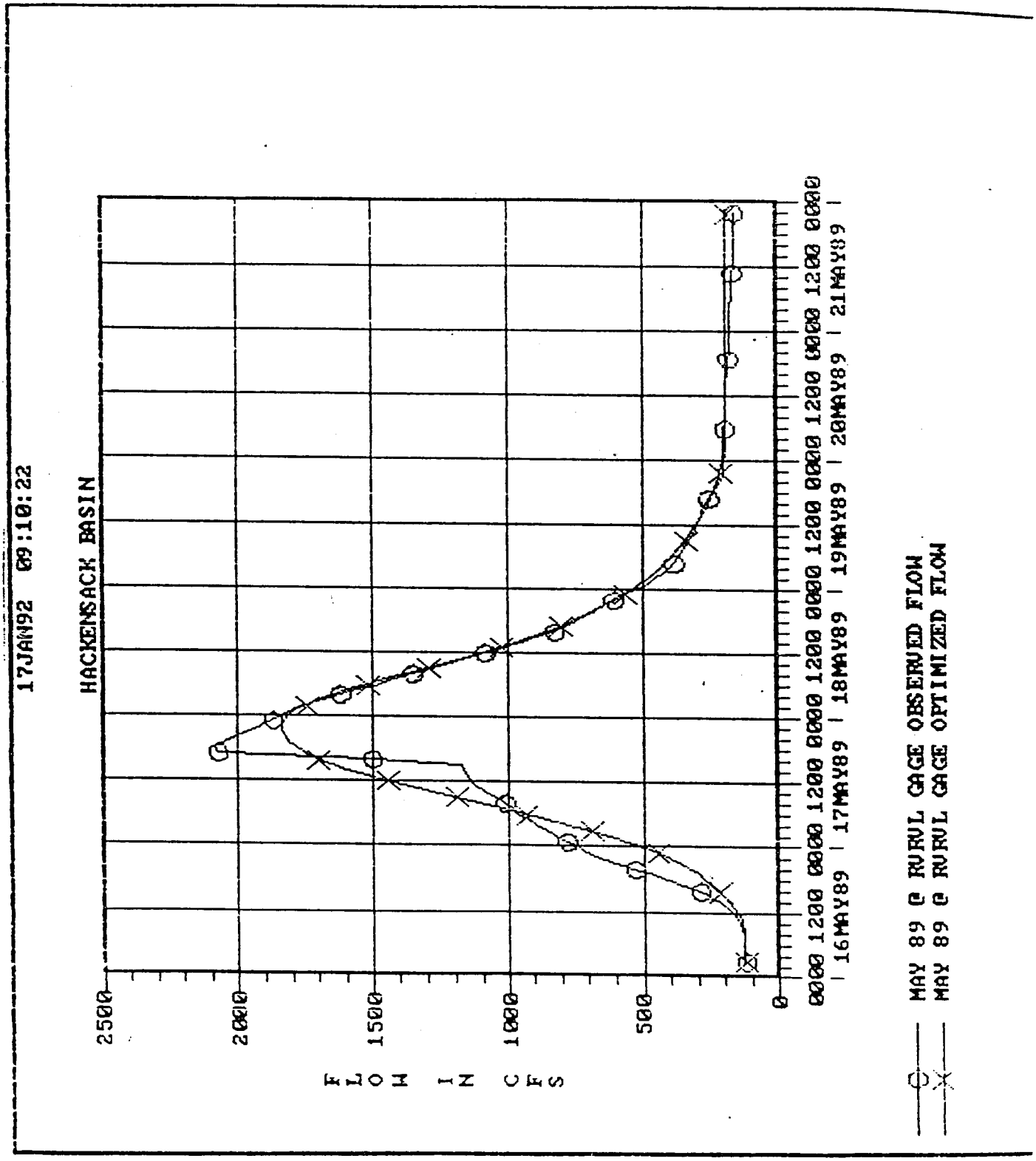
Figure C-85e



Passaic River Flood Protection Project
Hackensack Basin
Optim. Analysis - West Nyack
May 1984

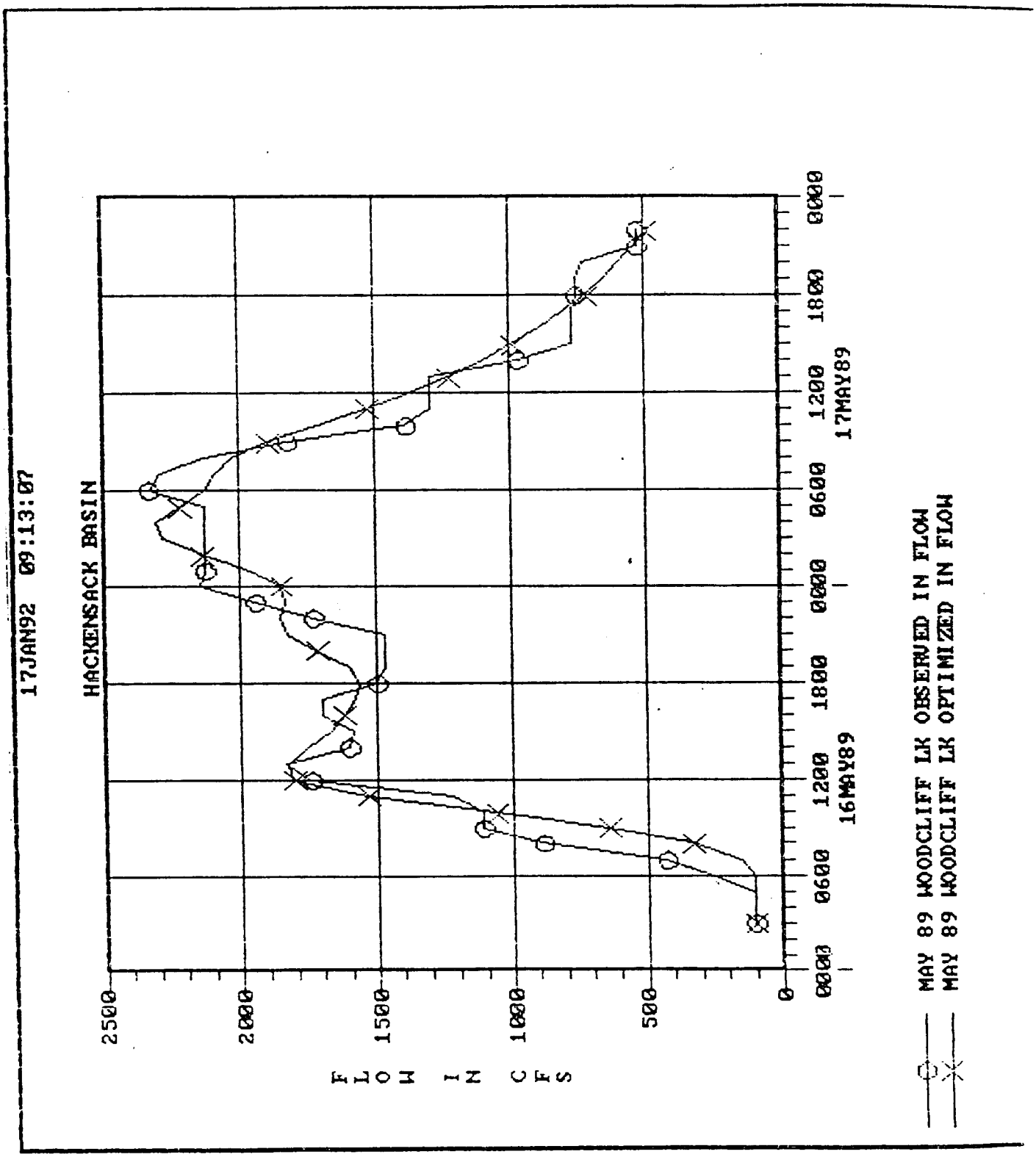
CENAN-PR-C MARCH 1992

Figure C-85f



Passaic River Flood Protection Project
Hackensack Basin
Optim. Analysis - Rivervale
May 1989

CENAN-PR-C **Figure C-85g** MARCH 1992

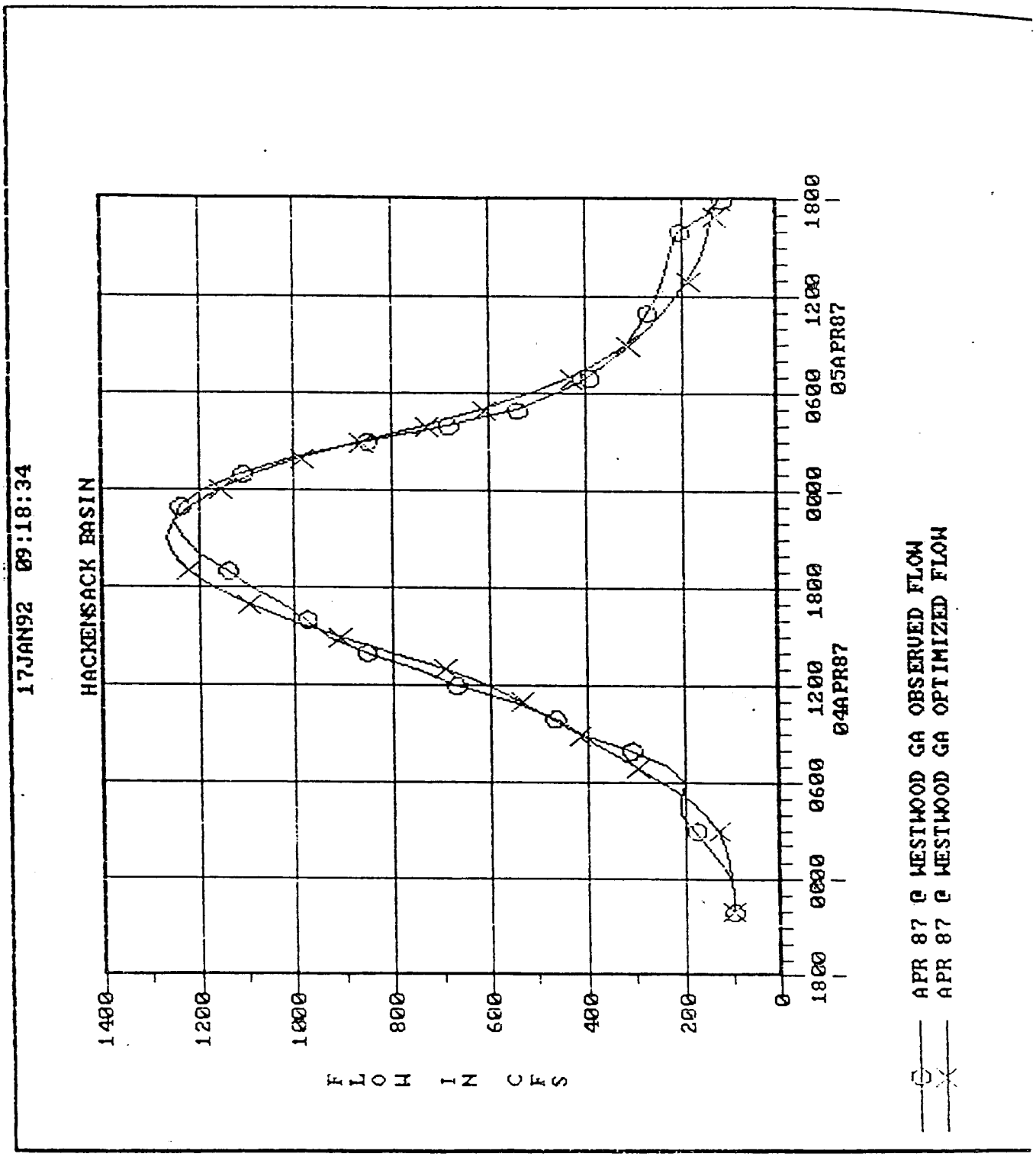


Passaic River Flood Protection Project
Hackensack Basin
Optim. Analysis - Woodcliff
May 1989

CENAN-PR-C

MARCH 1992

Figure C-85h

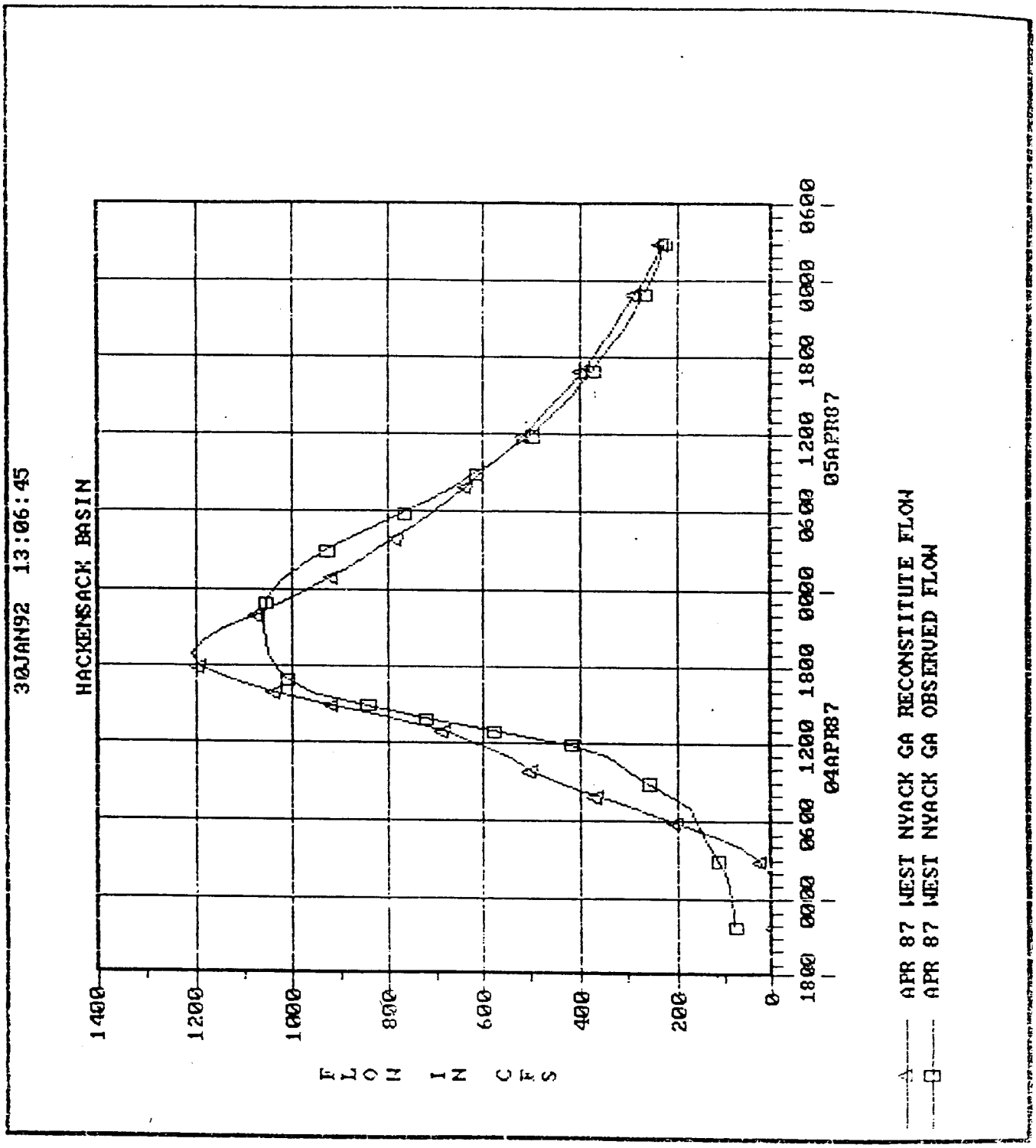


Passaic River Flood Protection Project
Hackensack Basin
Optim. Analysis - Westwood
April 1987

CENAN-PR-C

MARCH 199

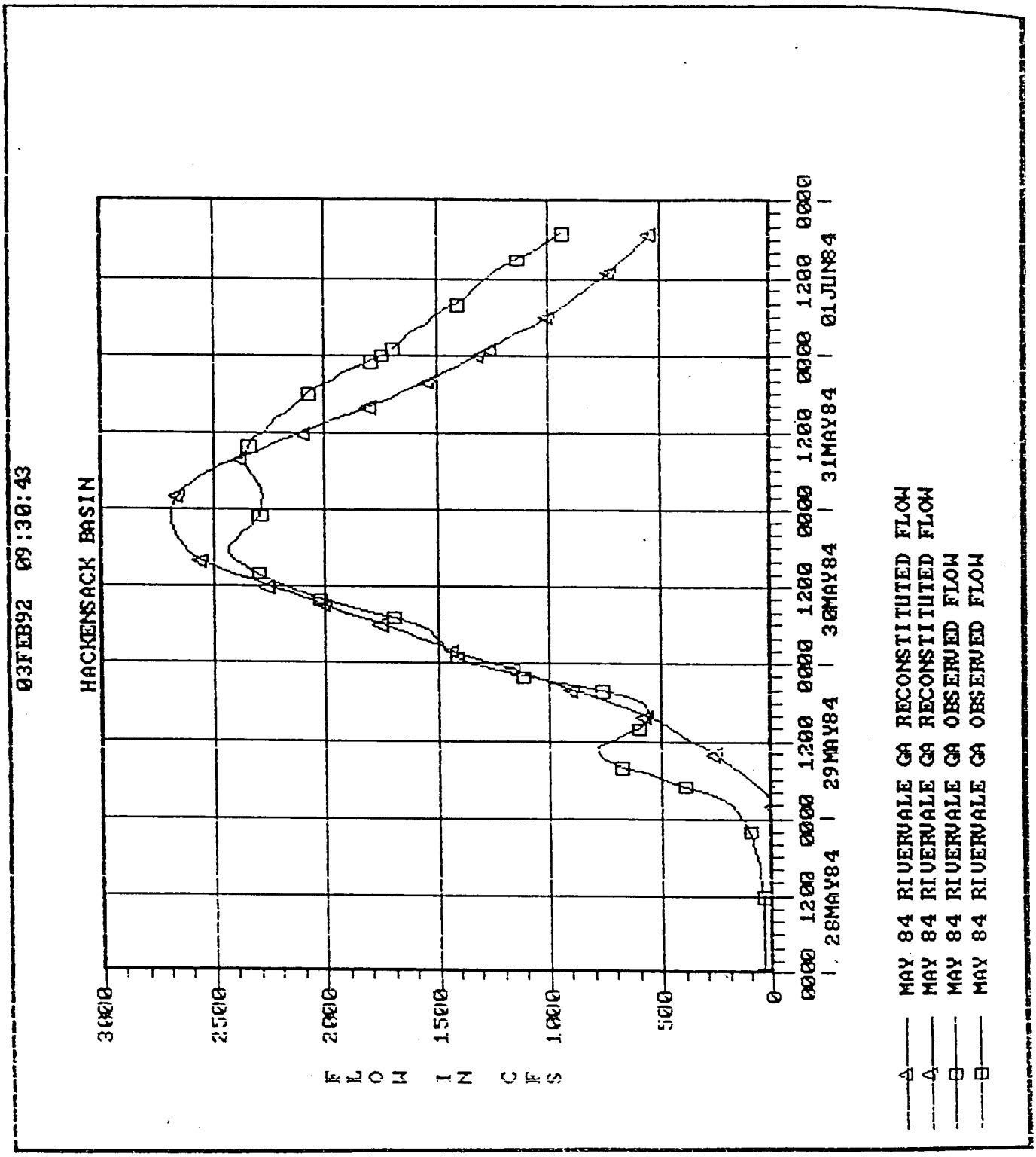
Figure C-85i



Passaic River Flood Protection Project
Hackensack Basin
Recon. Analysis - West Nyack
April 1987

CENAN-PR-C MARCH 1992

Figure C-85j

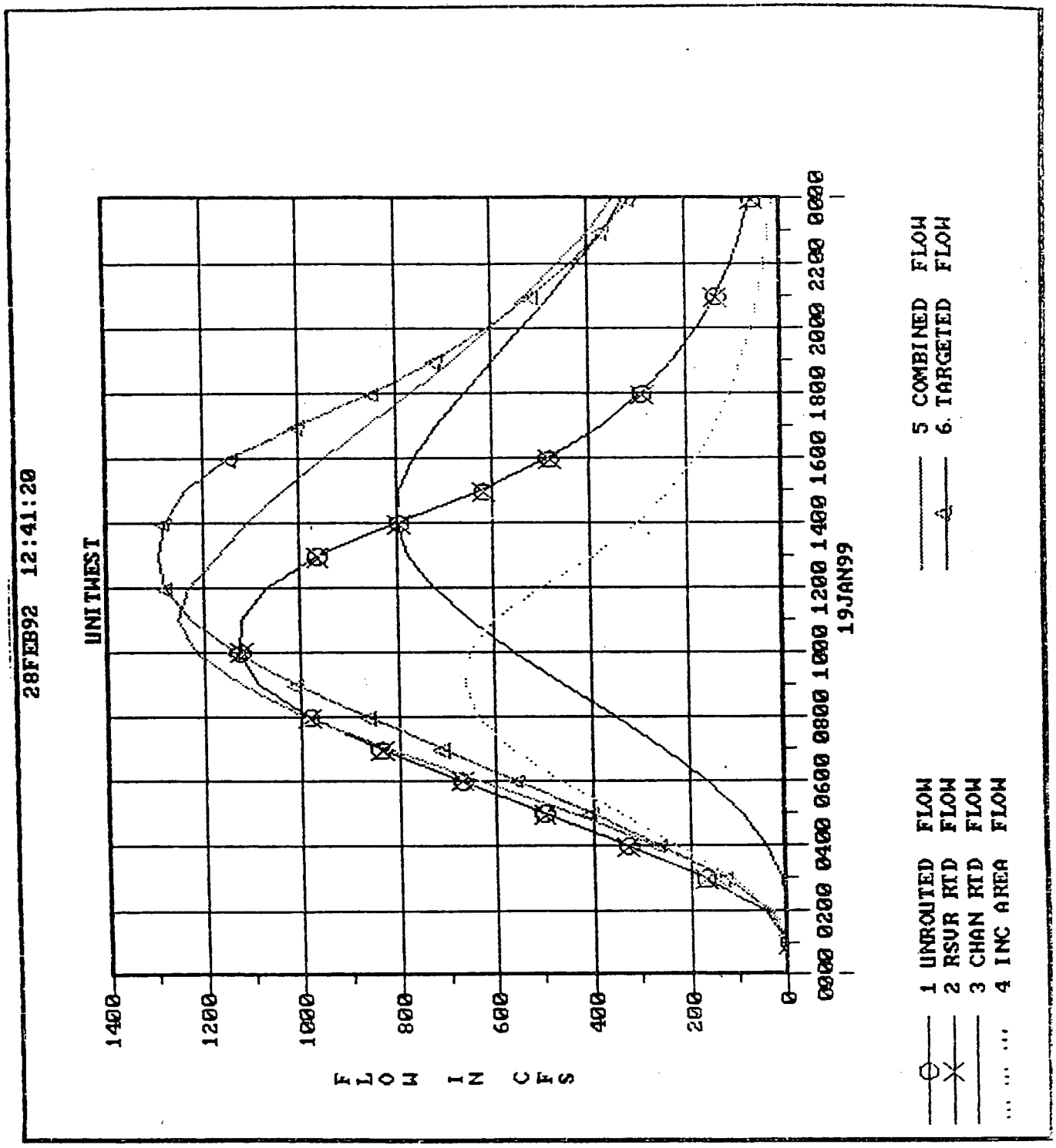


Passaic River Flood Protection Project
Hackensack Basin
Recon. Analysis - Rivervale
May 1984

CENAN-PR-C

MARCH 1992

Figure C-85k

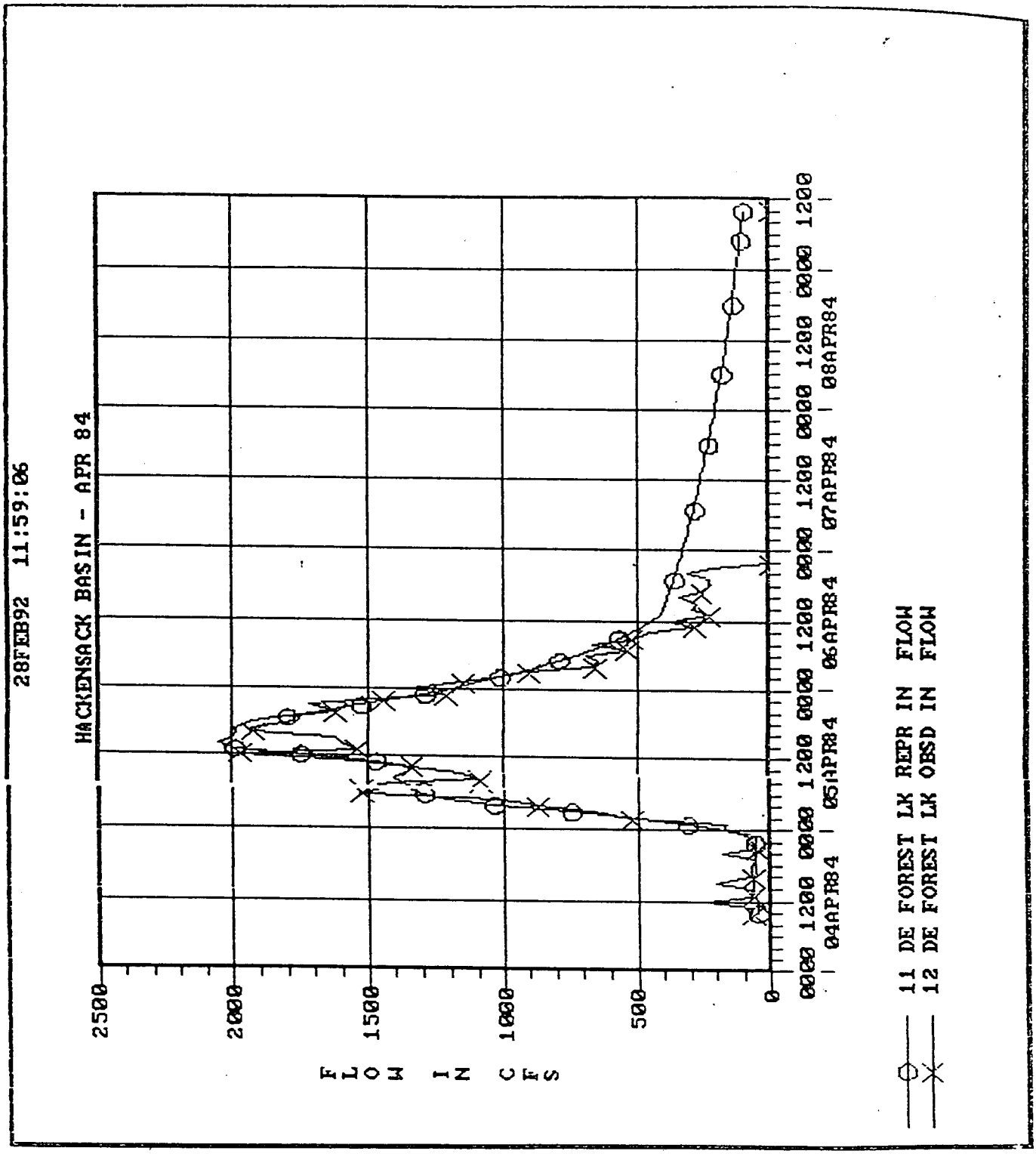


Passaic River Flood Protection Project
Hackensack Basin
Composite Unitgraph at Westwood

CENAN-PR-C

MARCH 1992

Figure C-851

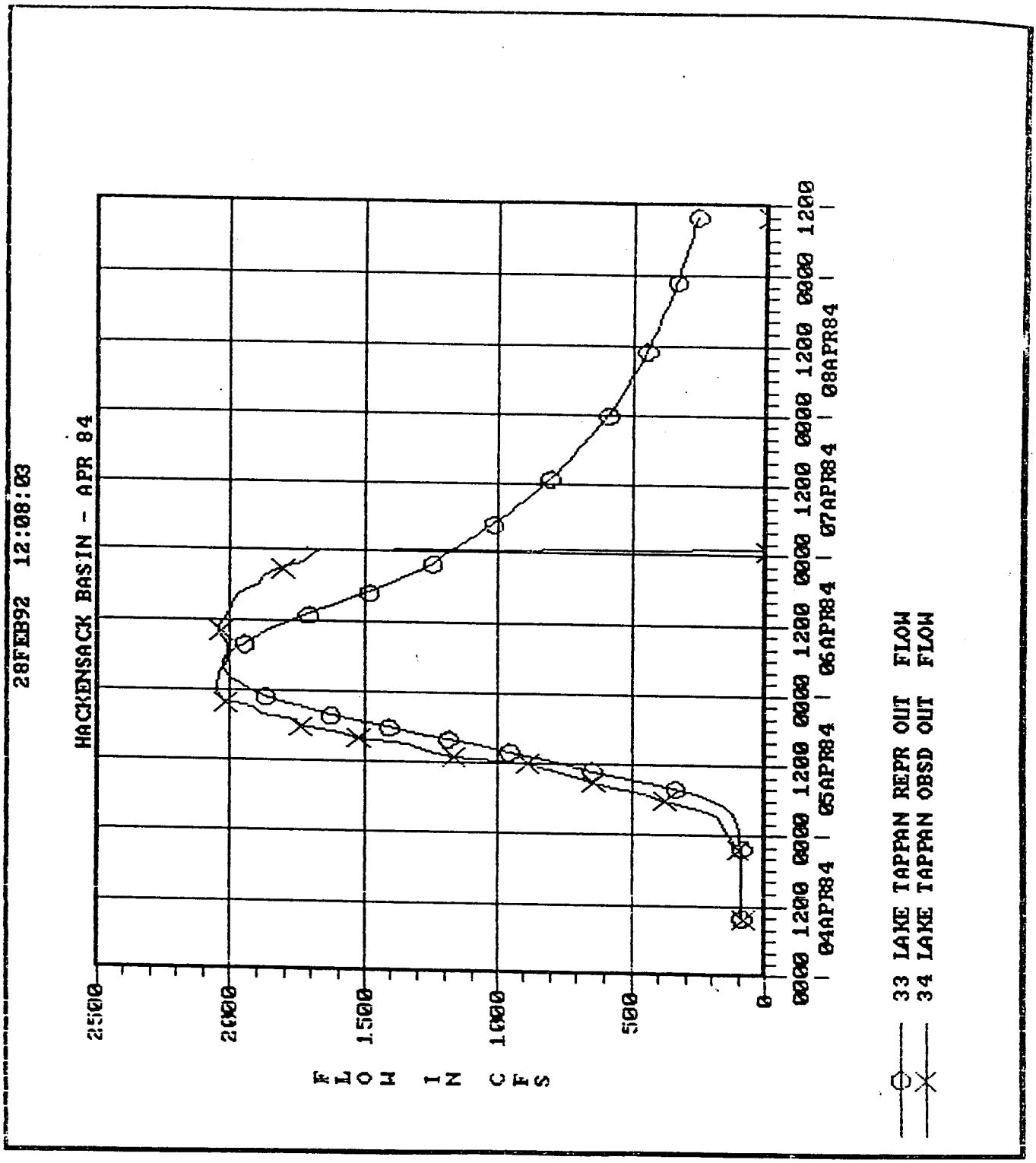


Passaic River Flood Protection Project
Hackensack Basin
Basin-Wide Cali.-DeForest In
April 1984

CENAN-PR-C

MARCH 1992

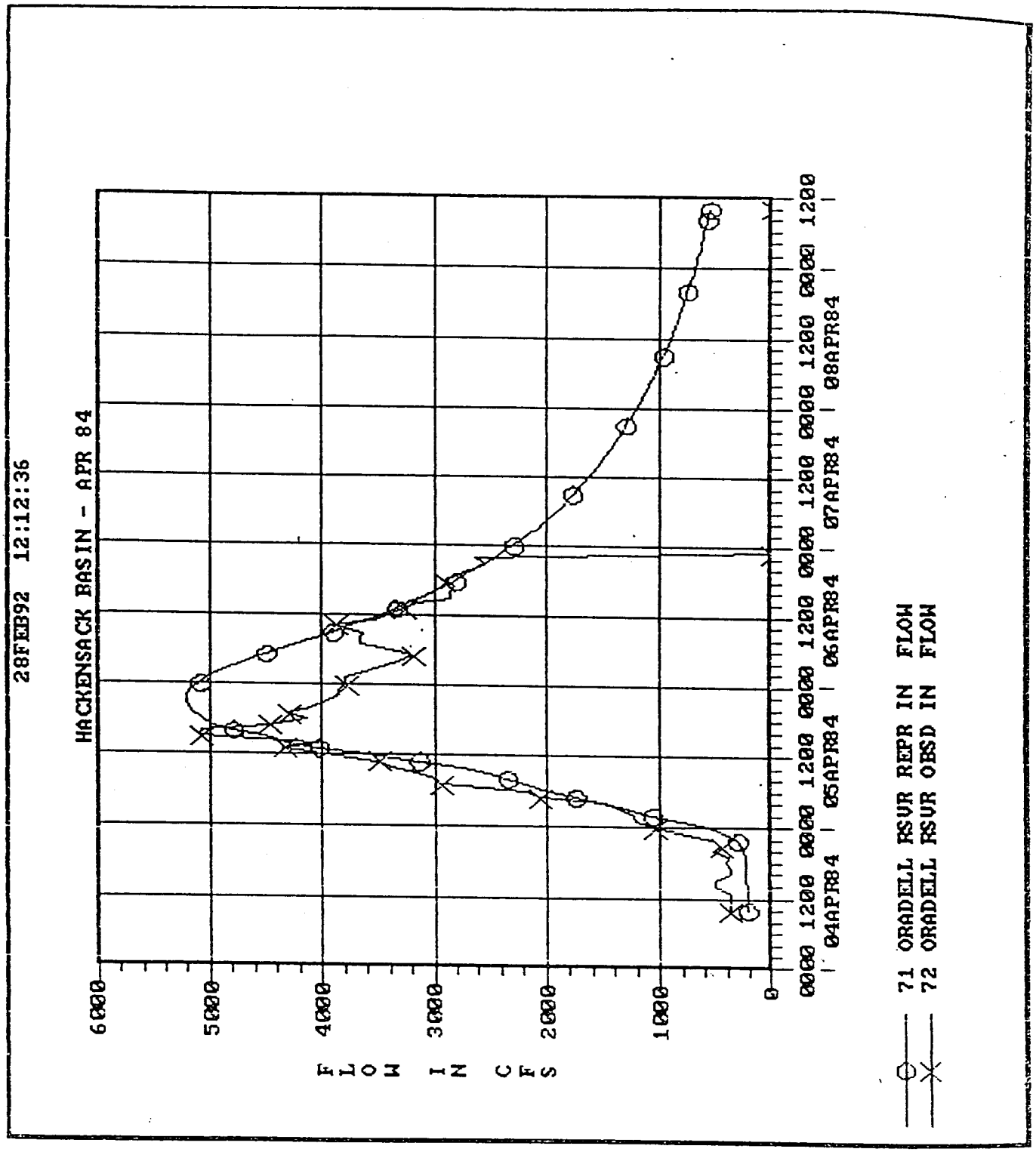
Figure C-85m



Passaic River Flood Protection Project
Hackensack Basin
Basin-Wide Cali.-LakeTappan Out
April 1984

CENAN-PR-C MARCH 1992

Figure C-85n

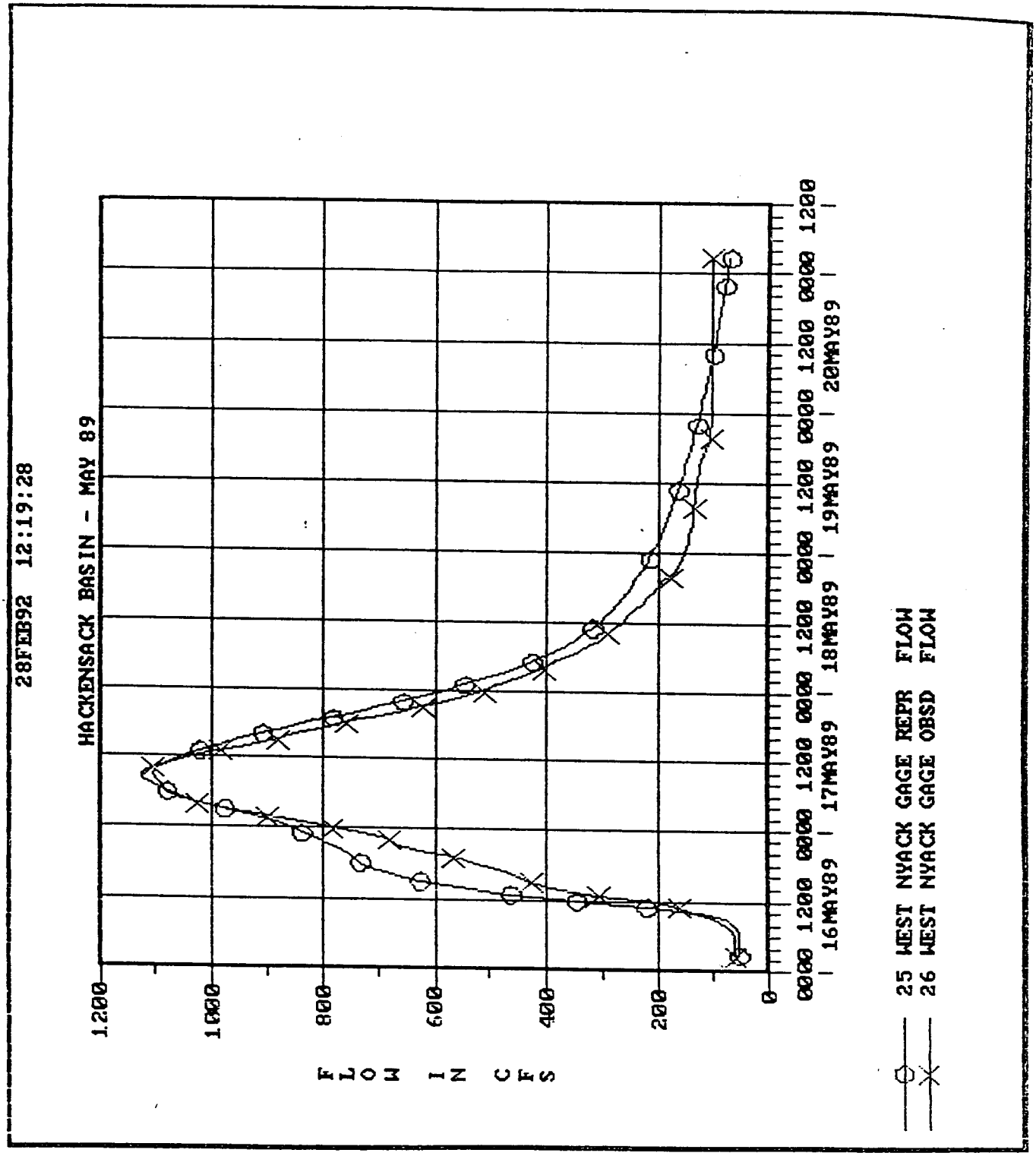


Passaic River Flood Protection Project
Hackensack Basin
Basin-wide Cali.-Oradell In
April 1984

CENAN-PR-C

MARCH 1992

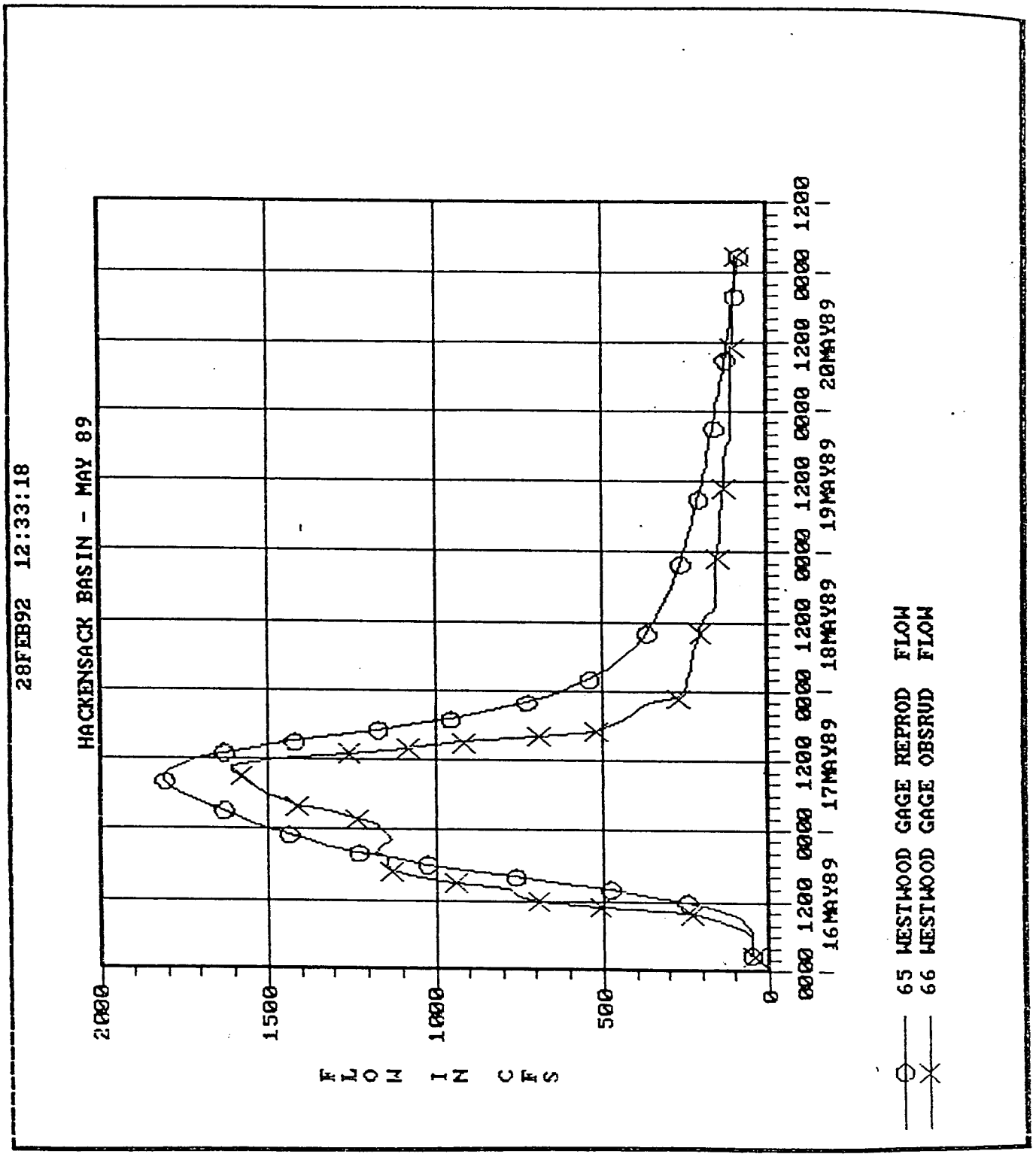
Figure C-850



Passaic River Flood Protection Project
Hackensack Basin
Basin-wide Cali.-West Nyack
May 1989

CENAN-PR-C MARCH 1992

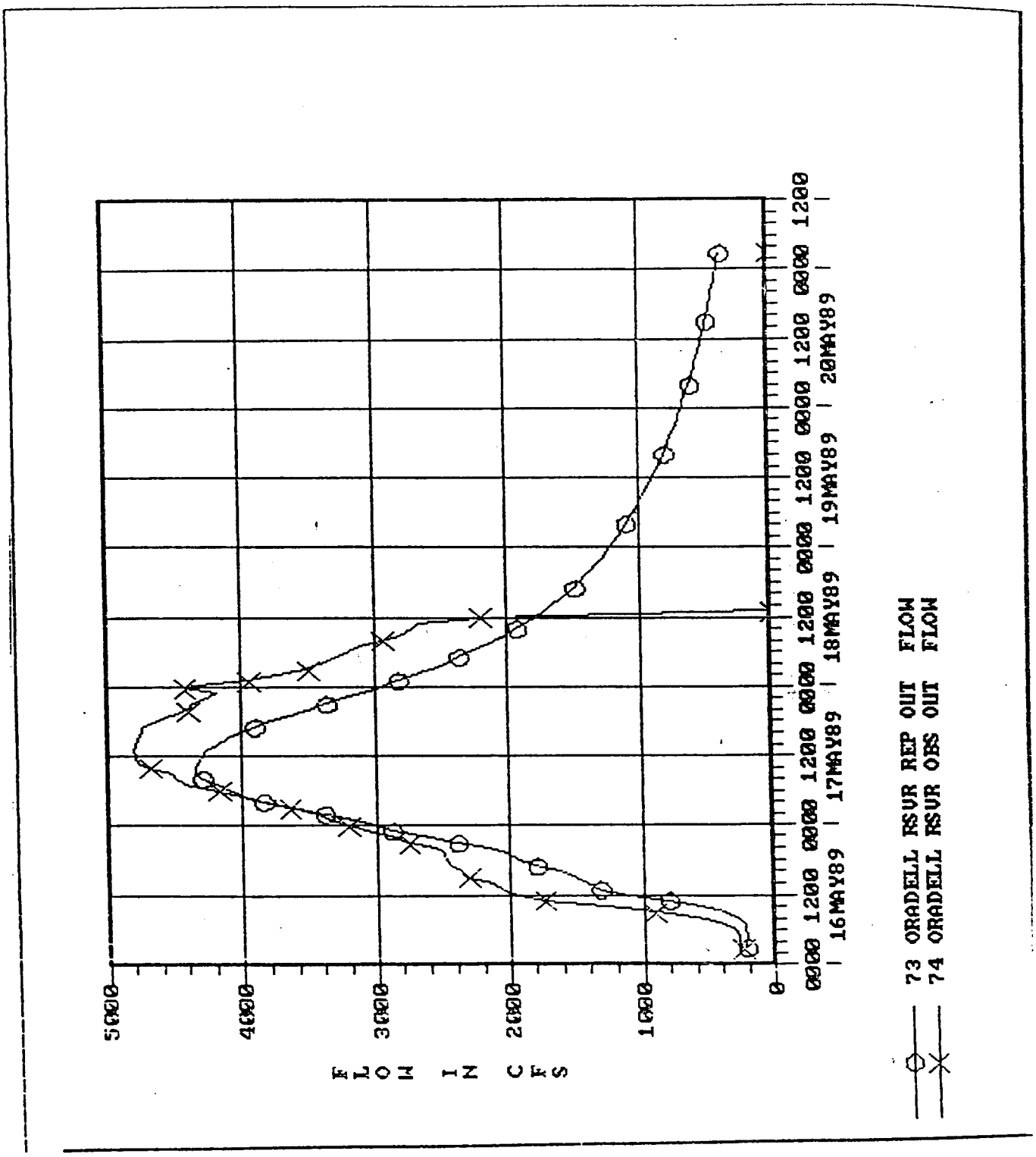
Figure C-85p



Passaic River Flood Protection Project
Hackensack Basin
Basin-wide Cali.-Westwood
May 1989

CENAN-PR-C MARCH 1992

Figure C-85q

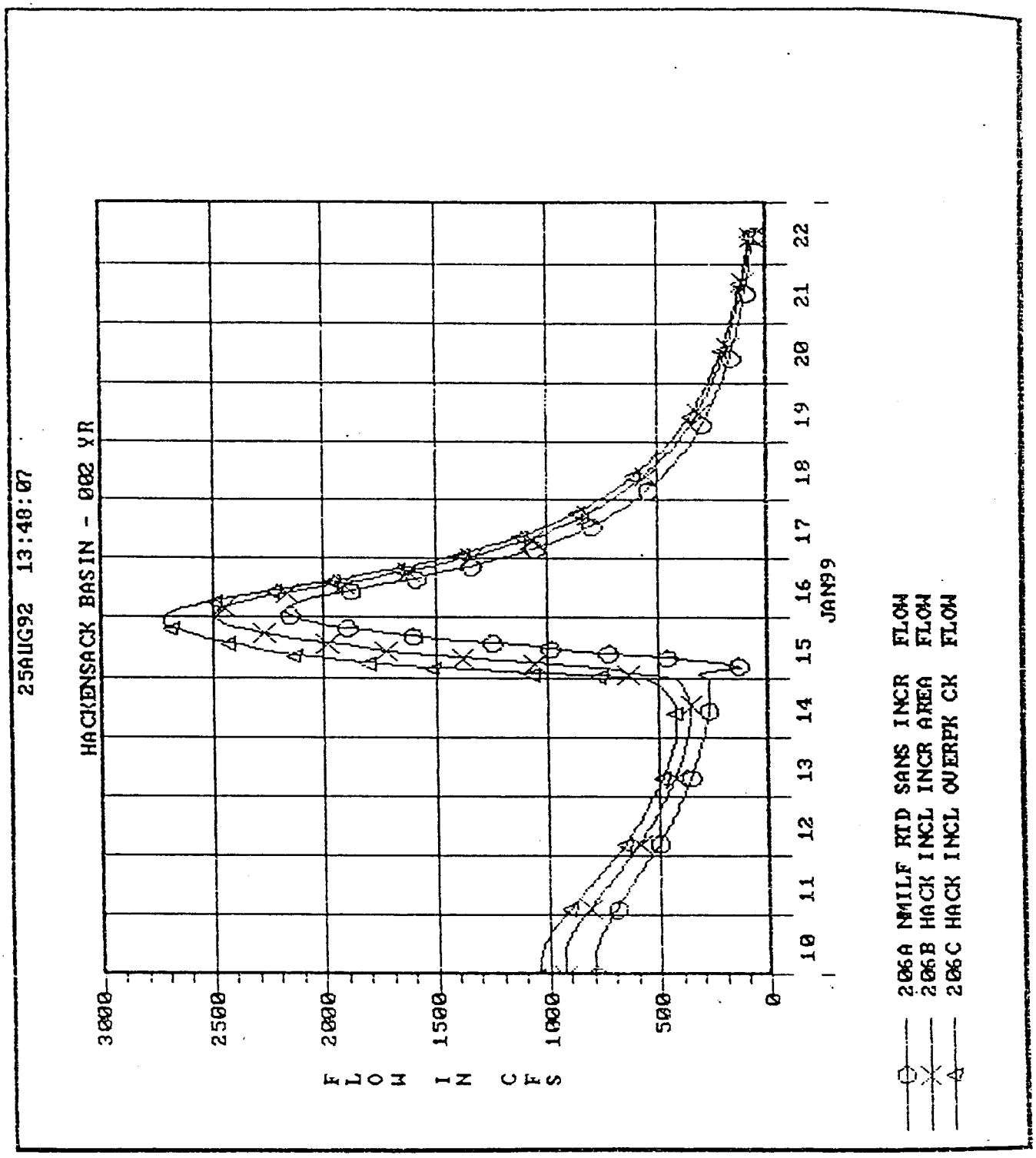


Passaic River Flood Protection Project
Hackensack Basin
Basin-wide Cali.-Oradell Out
May 1989

CENAN-PR-C

MARCH 1992

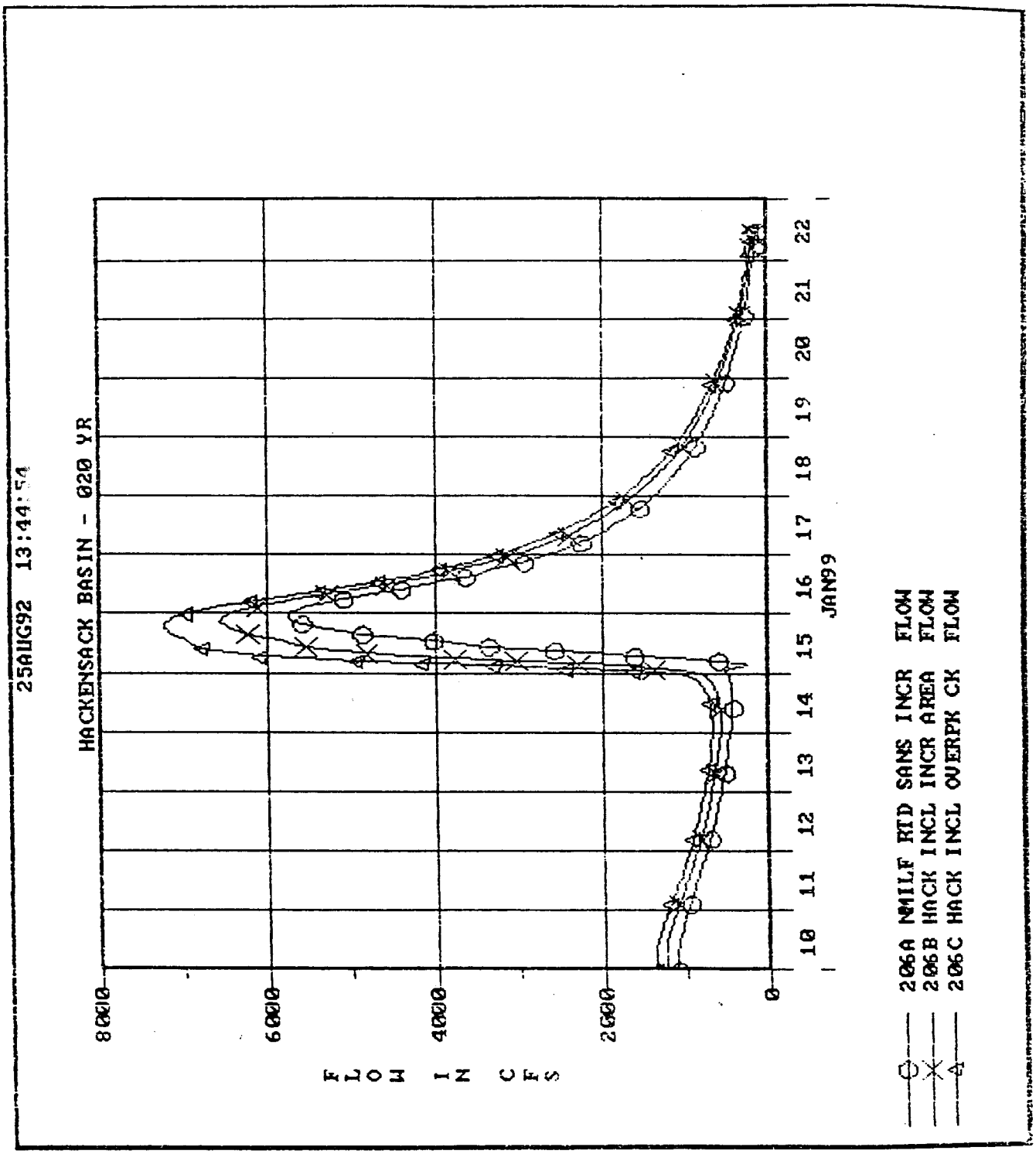
Figure C-85r



Passaic River Flood Protection Project
Hackensack Basin
2 Year Flood Hydrograph
HEC1 Model Limit

CENAN-PR-C

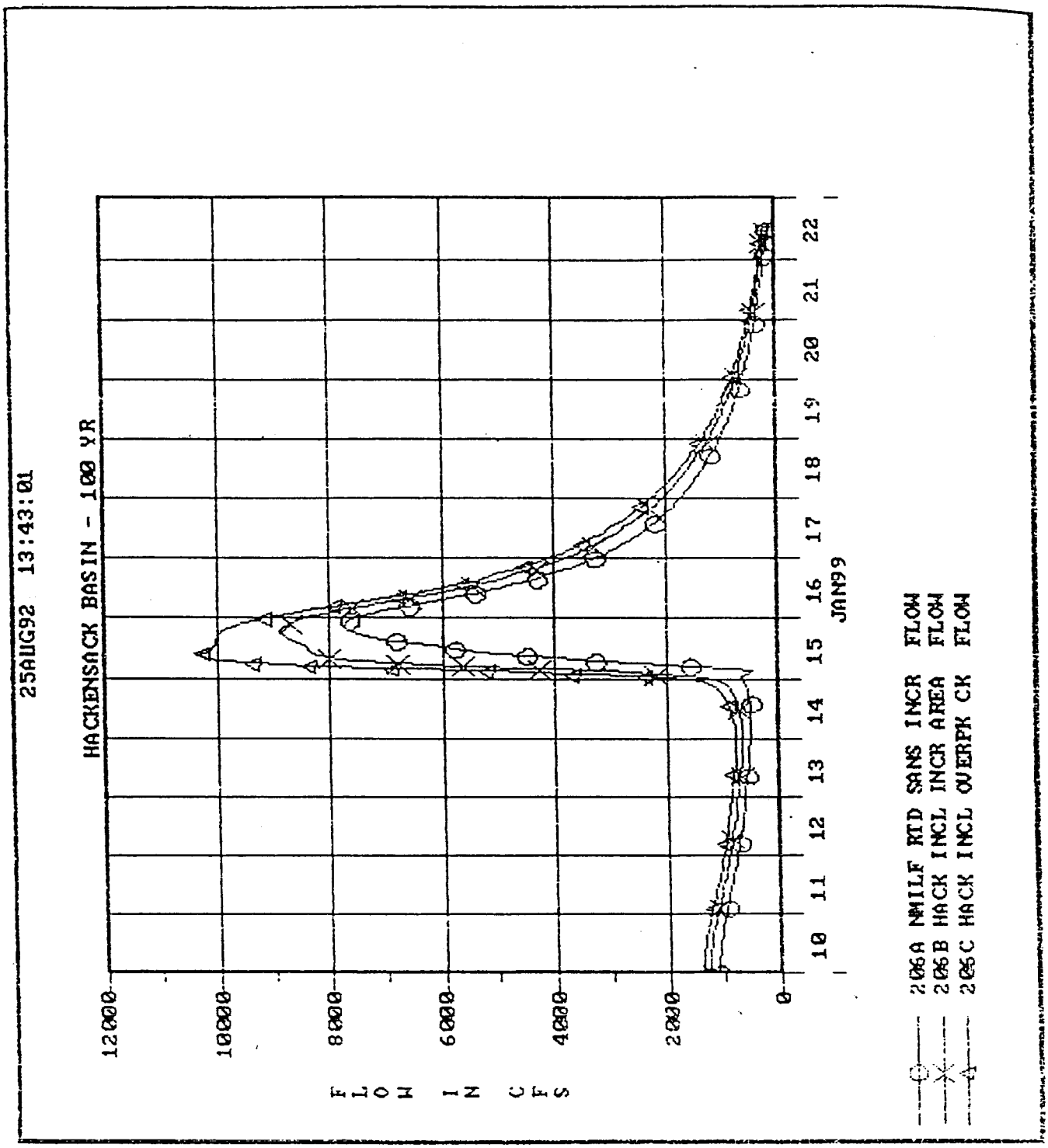
Figure C-85s



Passaic River Flood Protection Project
Hackensack Basin
20 Year Flood Hydrograph
HEC1 Model Limit

CENAN-PR-C August 1992

Figure C-85t

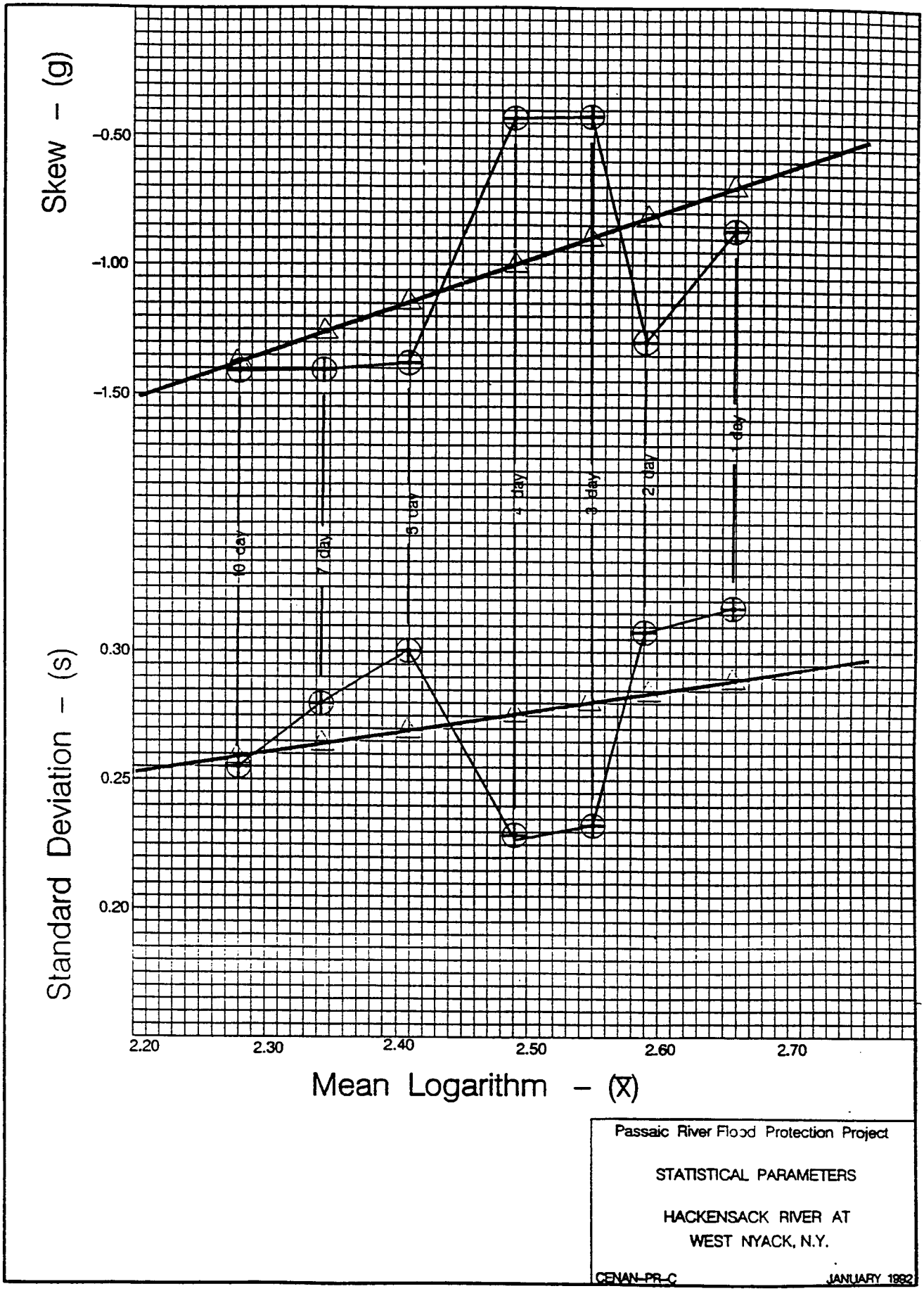


Passaic River Flood Protection Project
Hackensack Basin
100 Year Flood Hydrograph
HEC1 Model Limit

CENAN-PR-C

August 1992

Figure C-85u



stapar2.dgn

Figure C-86a

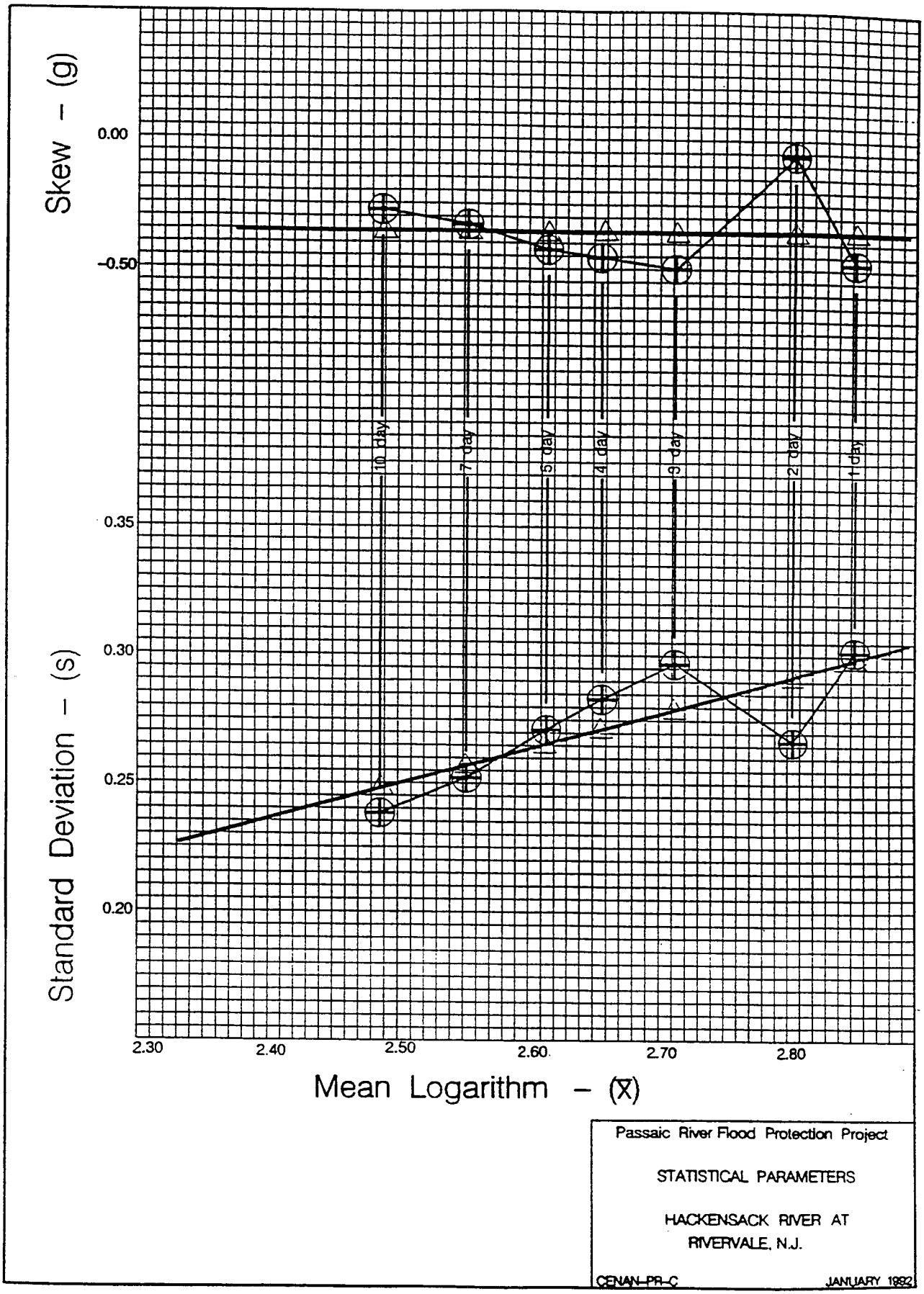
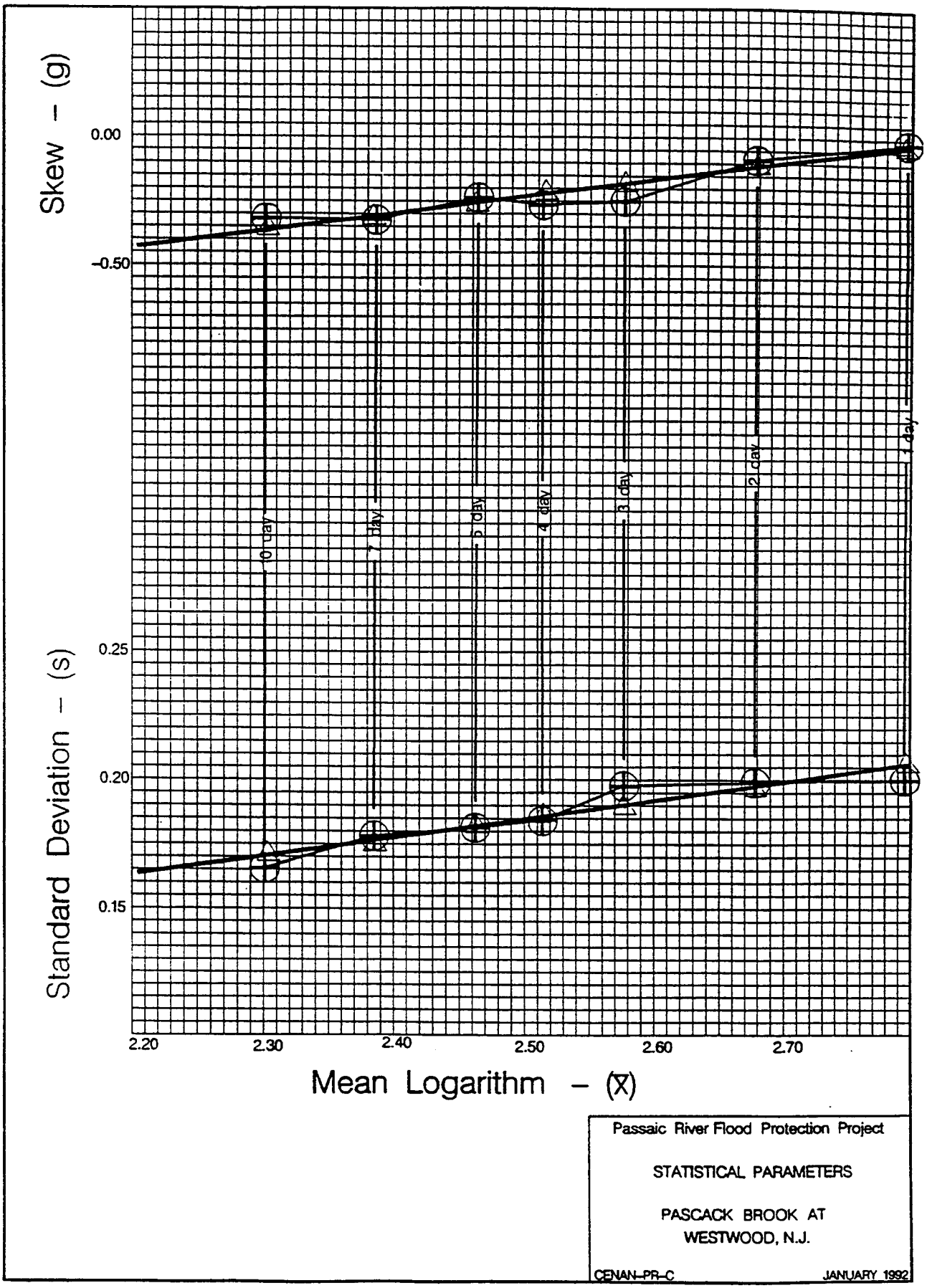

Skew - (g)
0.00
-0.50
Standard Deviation - (s)
0.35
0.30
0.25
0.20
10 day
7 day
5 day
4 day
3 day
2 day
1 day
2.30
2.40
2.50
2.60
2.70
2.80
Mean Logarithm - (X̄)
Passaic River Flood Protection Project
STATISTICAL PARAMETERS
HACKENSACK RIVER AT
RIVERVALE, N.J.
CENAN-PR-C
JANUARY 1982

Figure C-86b



stapar4.dgn

Figure C-86c

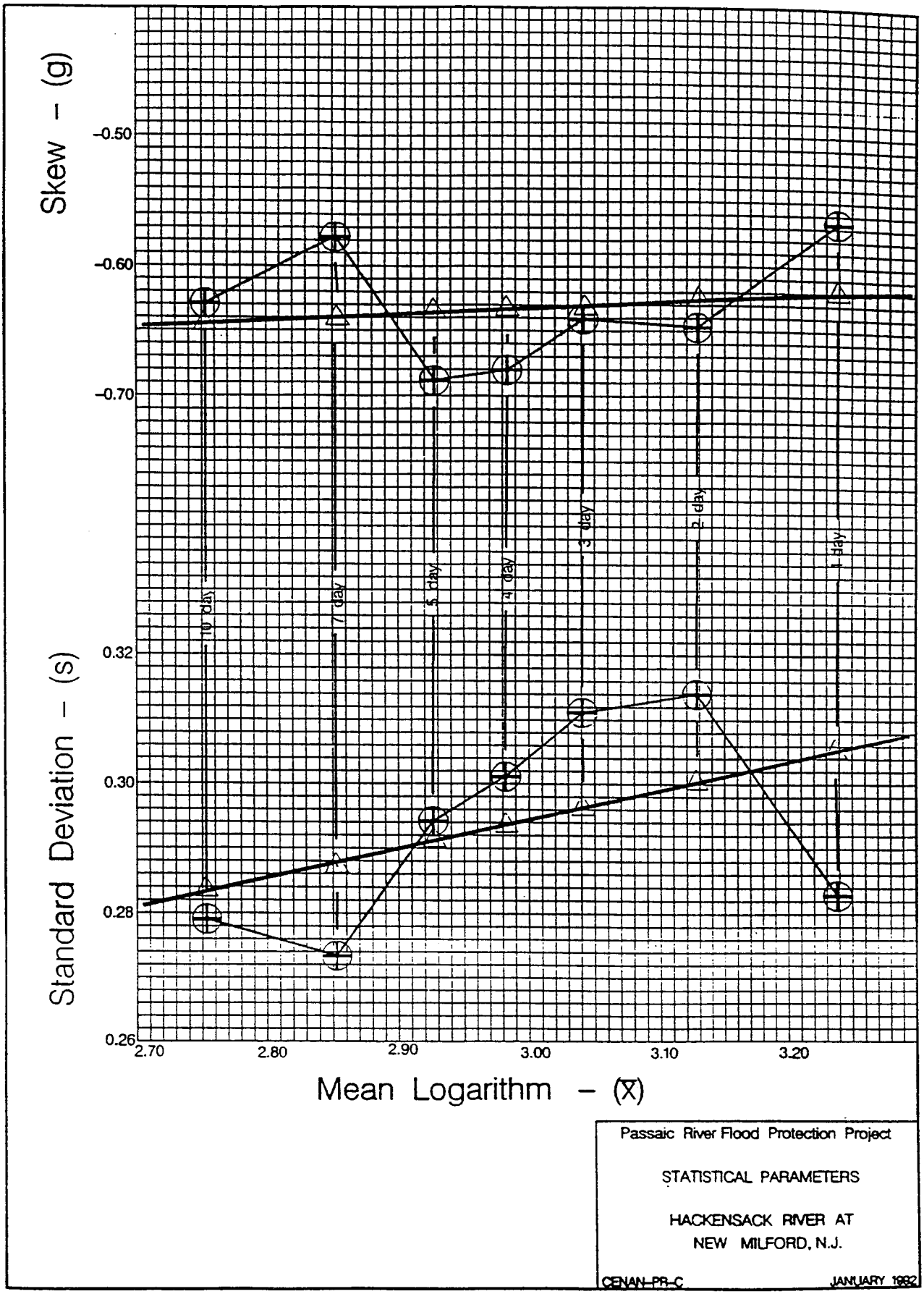
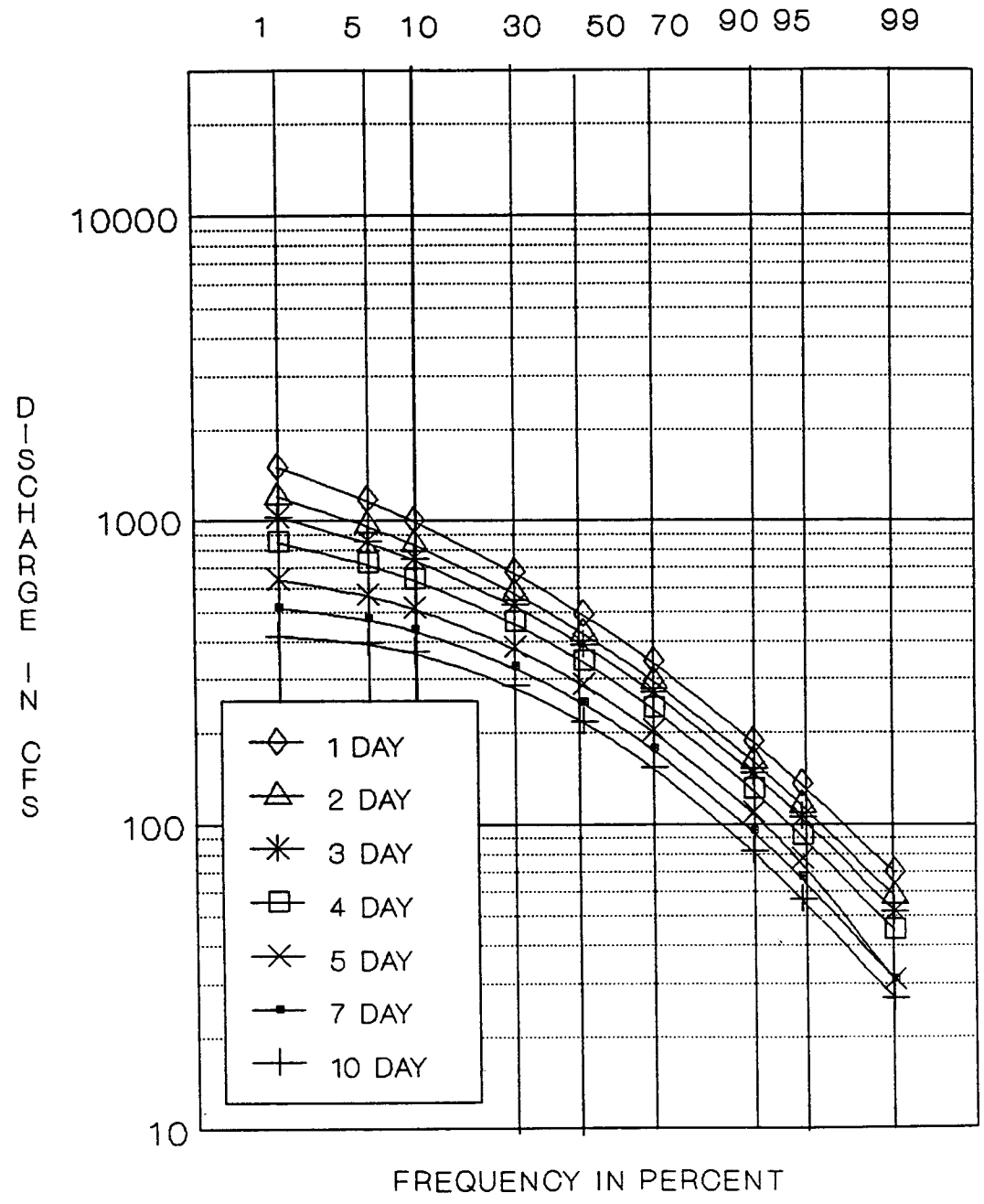

Skew - (g)
-0.50
-0.60
-0.70
Standard Deviation - (s)
0.32
0.30
0.28
0.26
10 day
7 day
5 day
4 day
3 day
2 day
1 day
2.70
2.80
2.90
3.00
3.10
3.20
Mean Logarithm - (X̄)
Passaic River Flood Protection Project
STATISTICAL PARAMETERS
HACKENSACK RIVER AT
NEW MILFORD, N.J.
CENAN-PR-C
JANUARY 1982

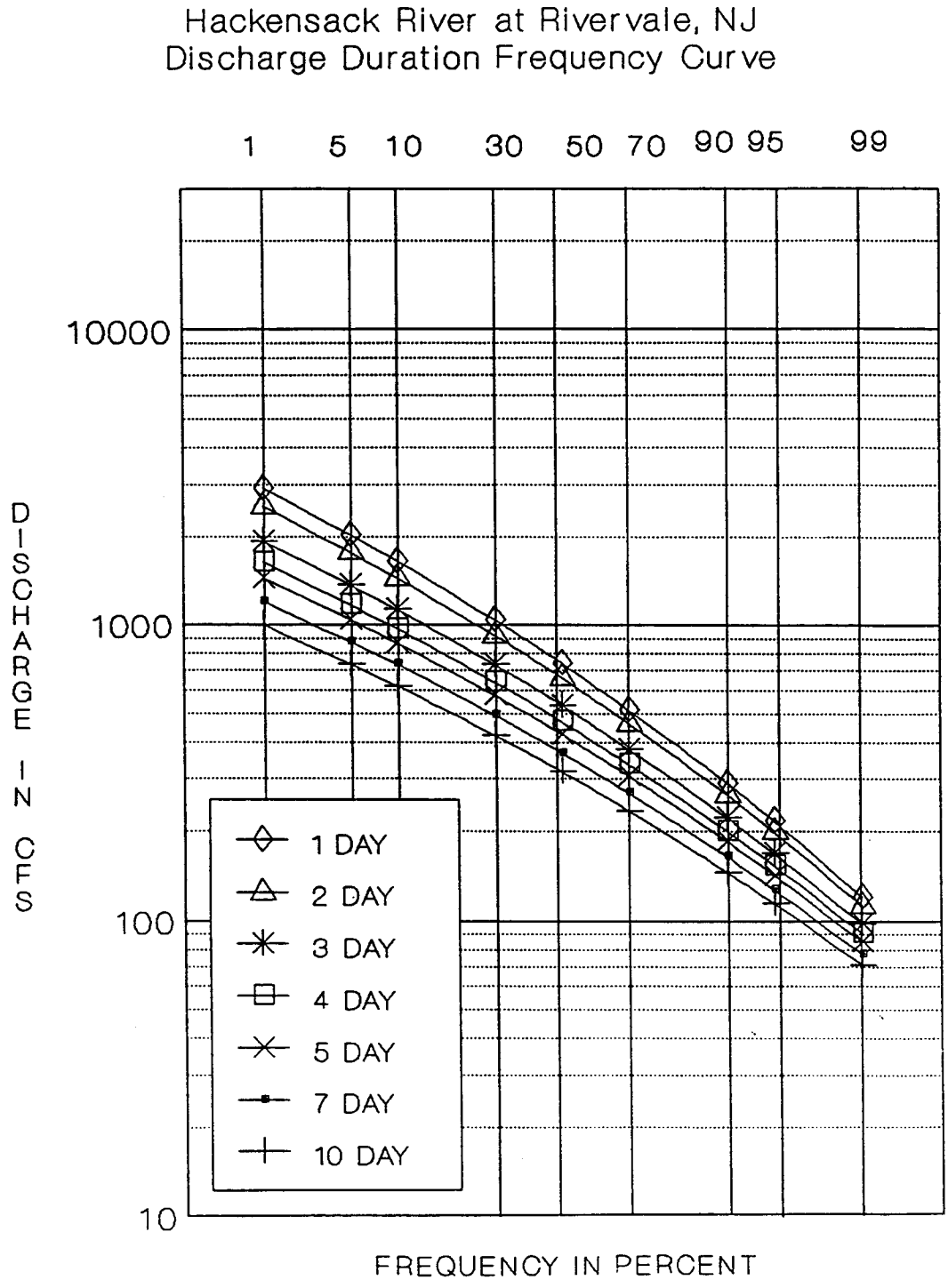
Figure C-86d

Hackensack River at West Nyack, NY
Discharge Duration Frequency Curve



July 1992

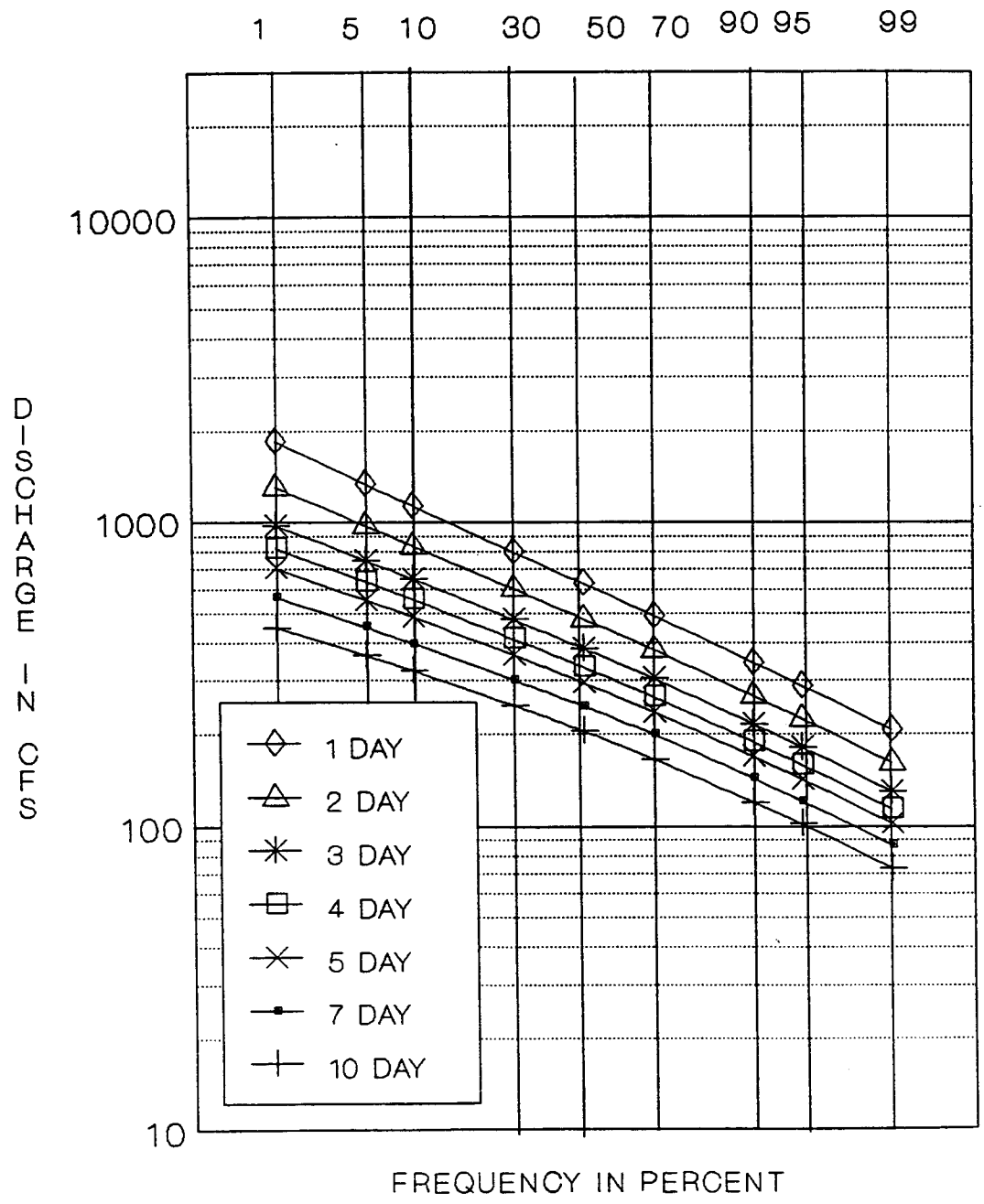
Figure C-87a



July 1992

Figure C-87b

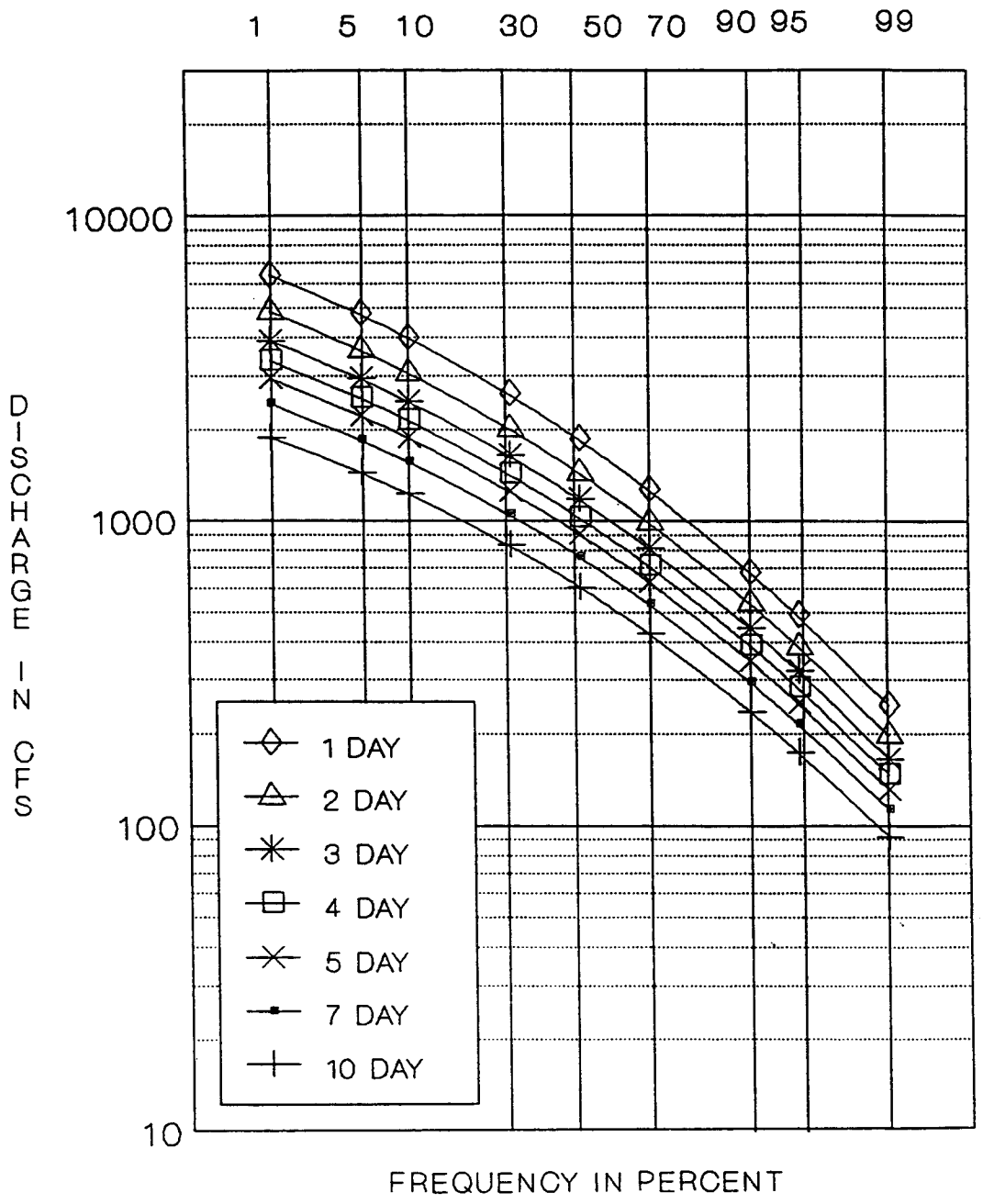
Pascack Brook at Westwood, NJ
Discharge Duration Frequency Curve



July 1992

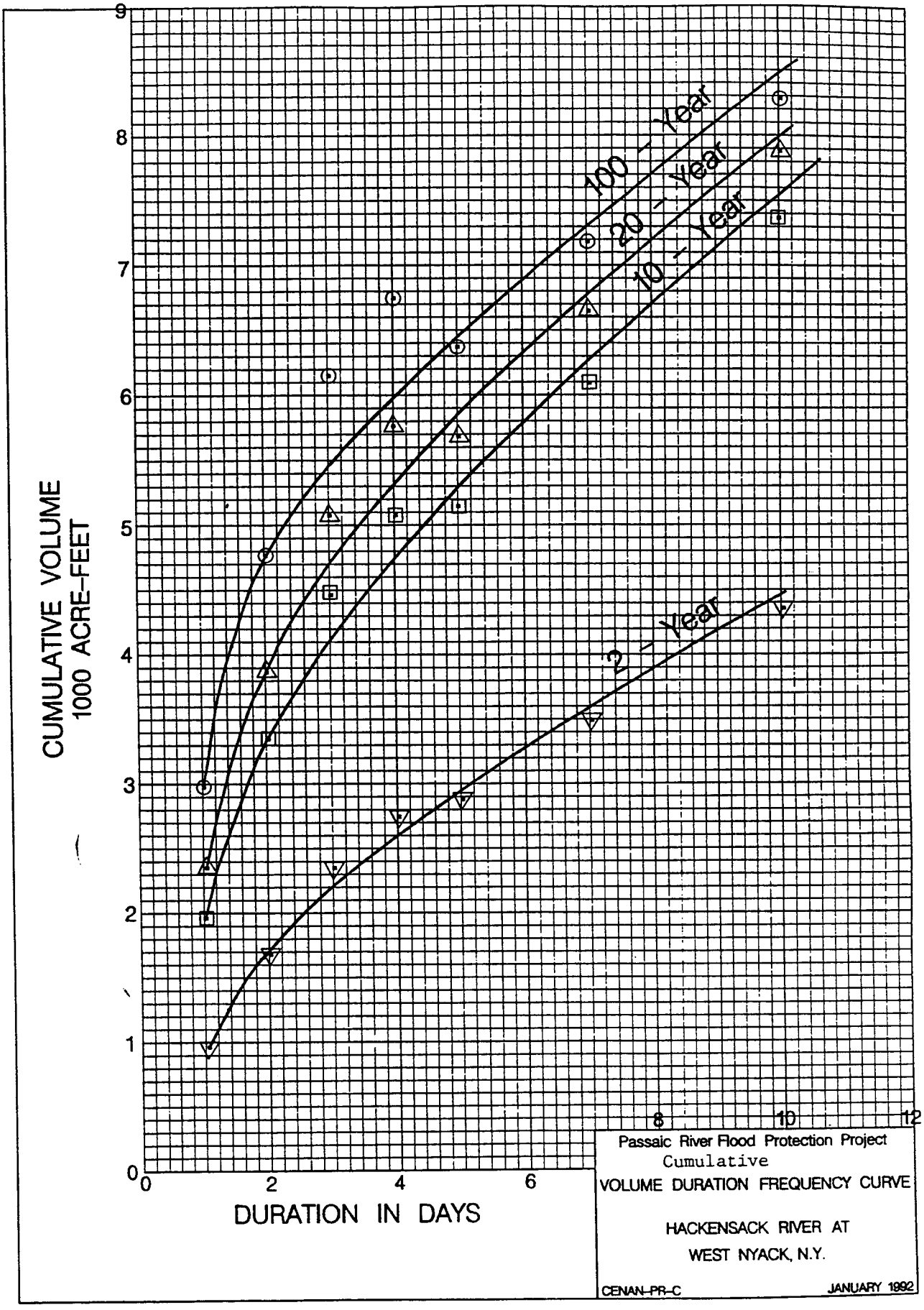
Figure C-87c

Hackensack River at New Milford
Discharge Duration Frequency Curve



July 1992

Figure C-87d



/dc2.dgn

Figure C-88a

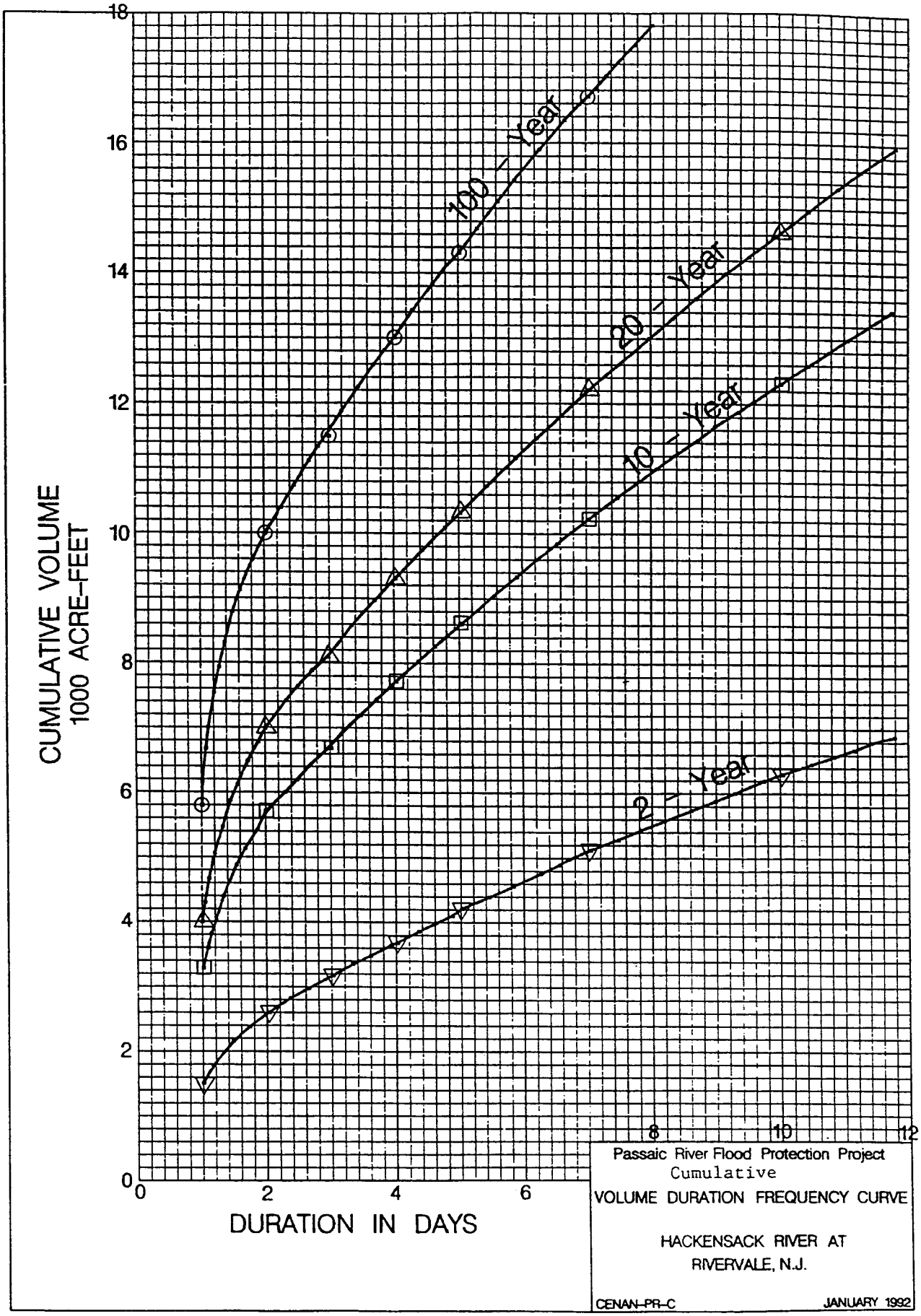
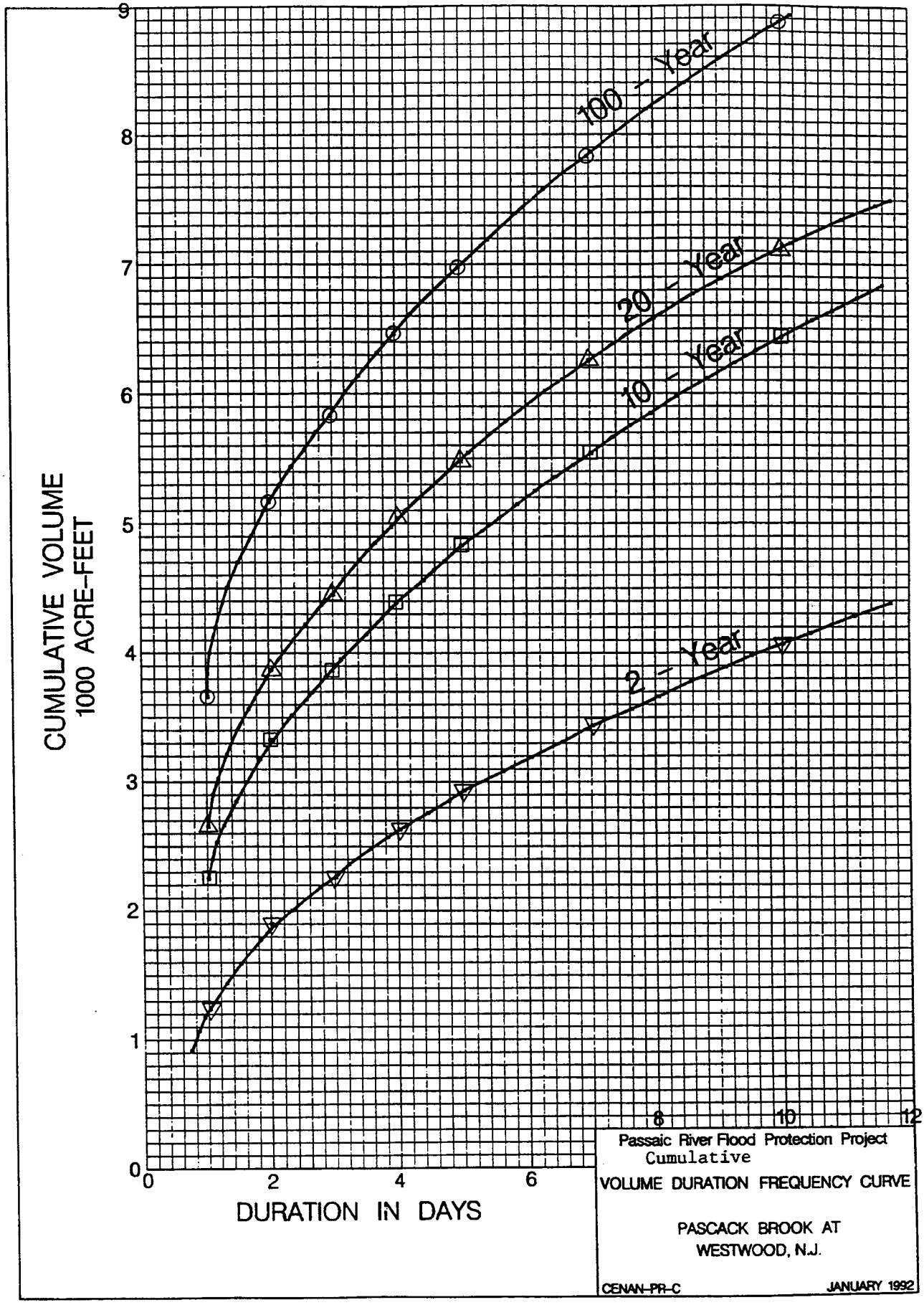

CUMULATIVE VOLUME
1000 ACRE-FEET
DURATION IN DAYS
100 - Year
20 - Year
10 - Year
2 - Year
0
2
4
6
8
10
12
14
16
18
Passaic River Flood Protection Project
Cumulative
VOLUME DURATION FREQUENCY CURVE
HACKENSACK RIVER AT
RIVERVALE, N.J.
CENAN-PR-C
JANUARY 1992

Figure C-88b



vdc3.dgn

Figure C-88c

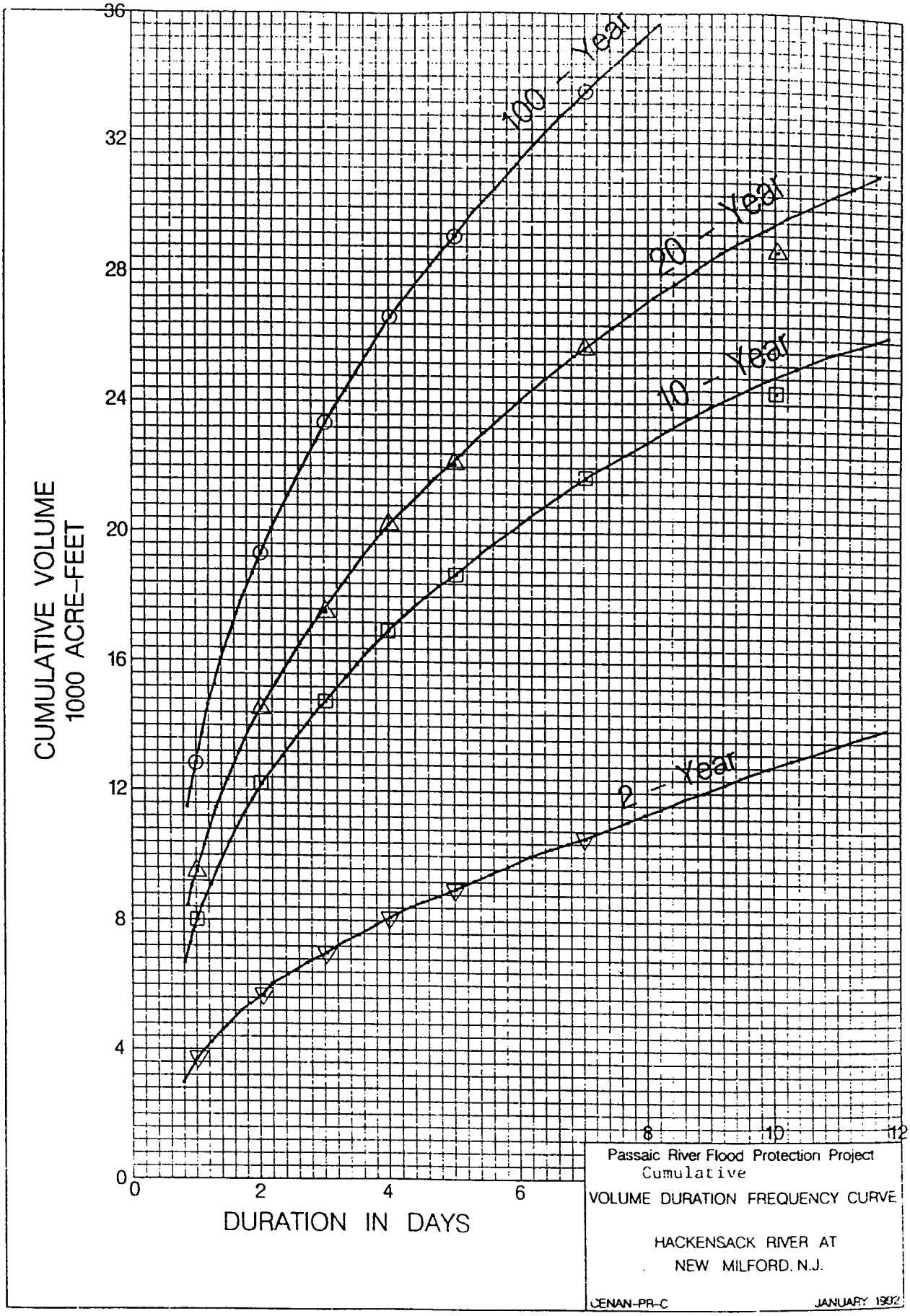
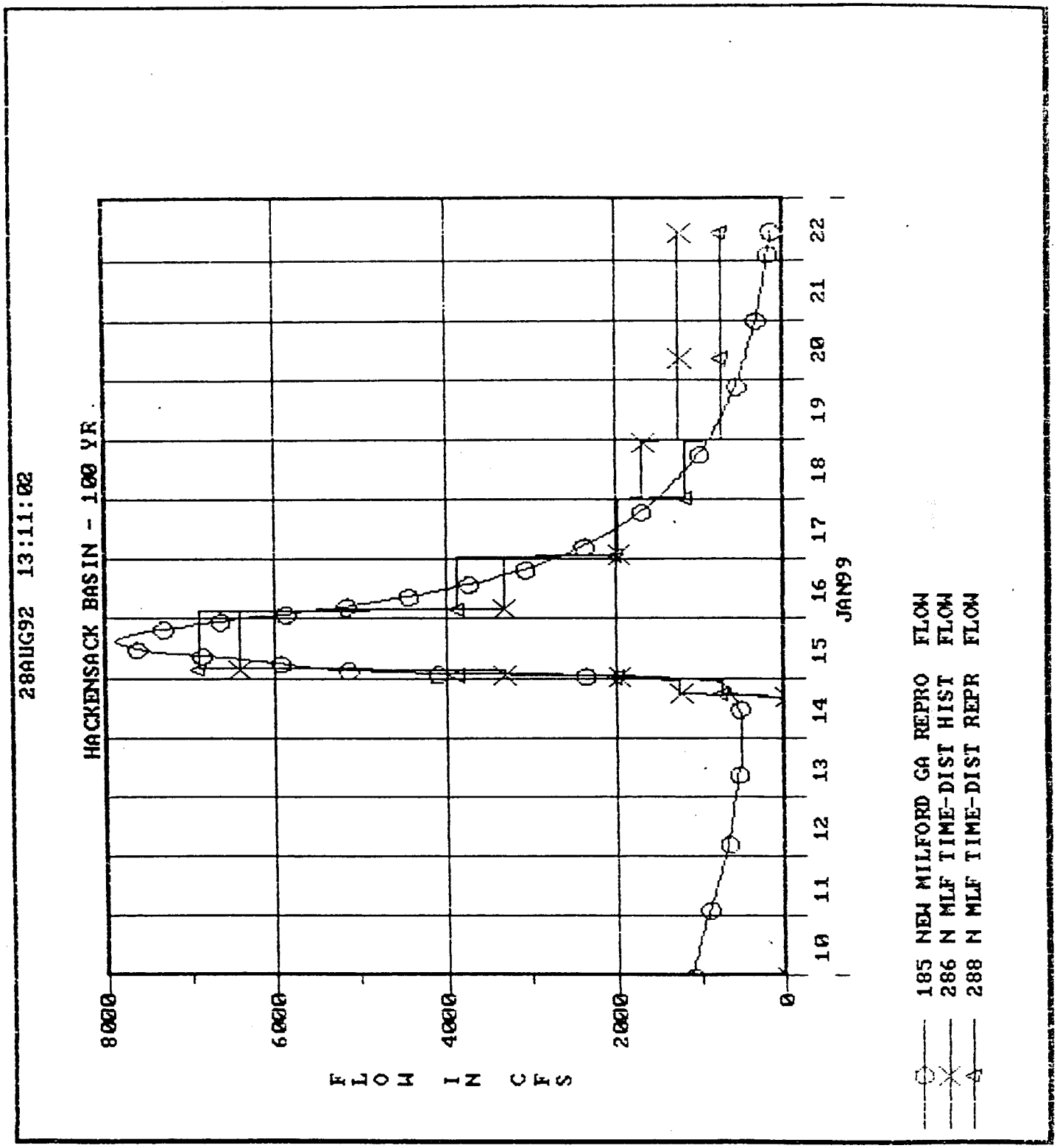

CUMULATIVE VOLUME
1000 ACRE-FEET
36
32
28
24
20
16
12
8
4
0
0
2
4
6
8
10
12
DURATION IN DAYS
100 - Year
20 - Year
10 - Year
2 - Year
Passaic River Flood Protection Project
Cumulative
VOLUME DURATION FREQUENCY CURVE
HACKENSACK RIVER AT
NEW MILFORD, N.J.
CENAN-PR-C
JANUARY 1992

Figure C-88d



Passaic River Flood Protection Project
Hackensack Basin
100 Year Time Distributed Volume
New Milford, NJ

CENAN-PR-C August 1992

Figure C-89

Please refer to Volume II of II of Appendix C for Figures C-90a through C-108, which are oversized (11" x 17").

27APR95 10:15:56

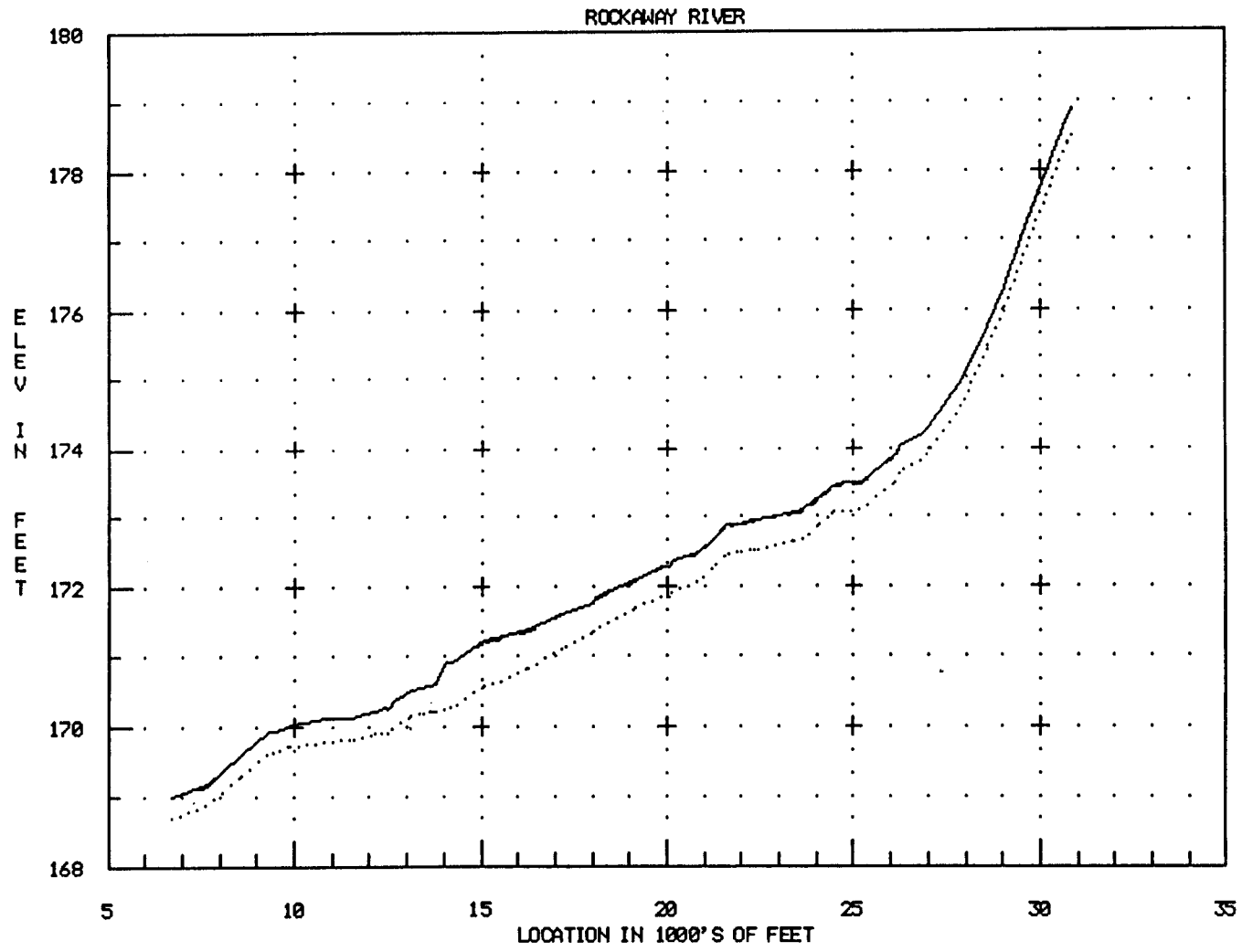


Figure C-109a

27APR95 10:31:52

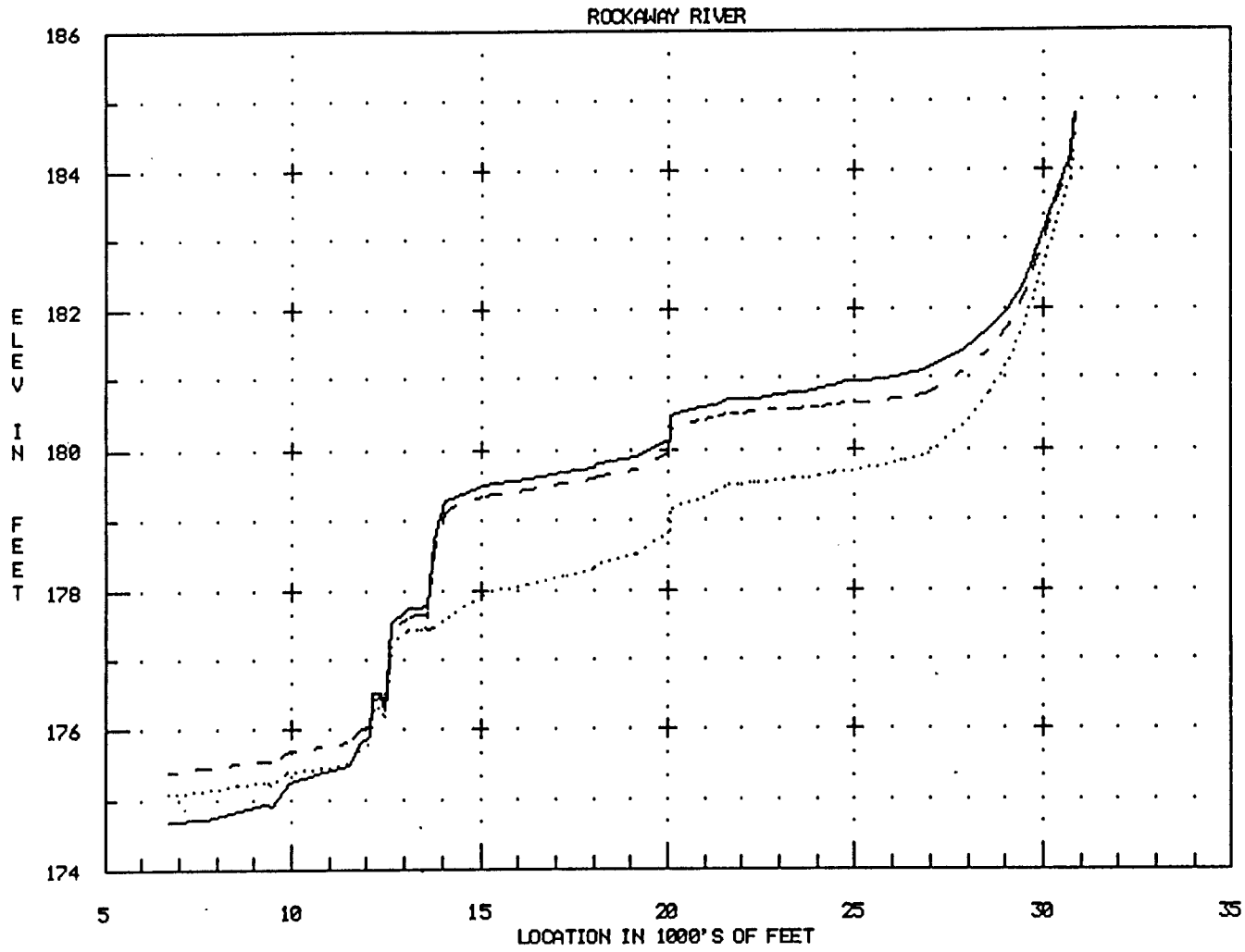


Figure C-109b

27APR95 10:16:21

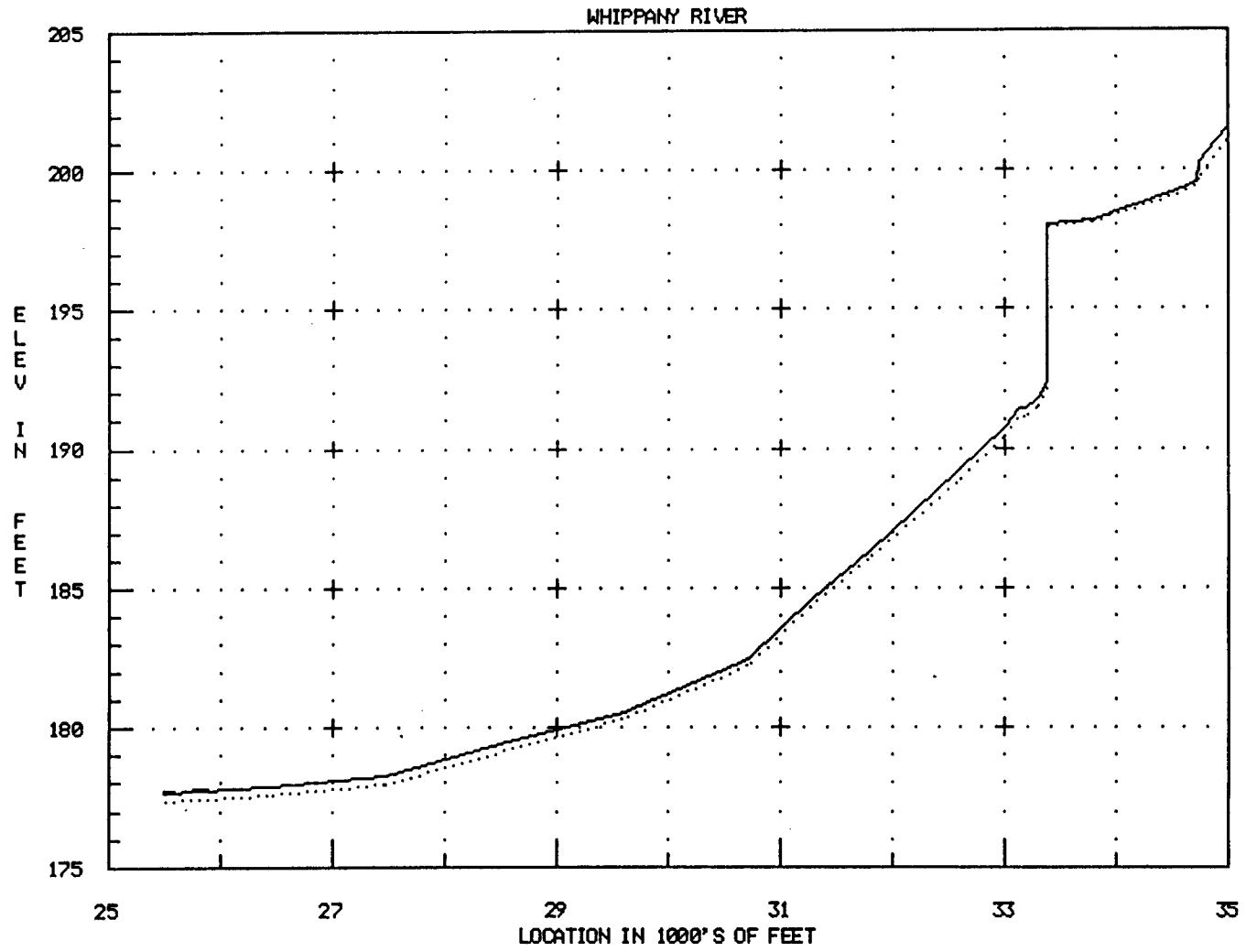


Figure C-109c

27APR95 10:32:19

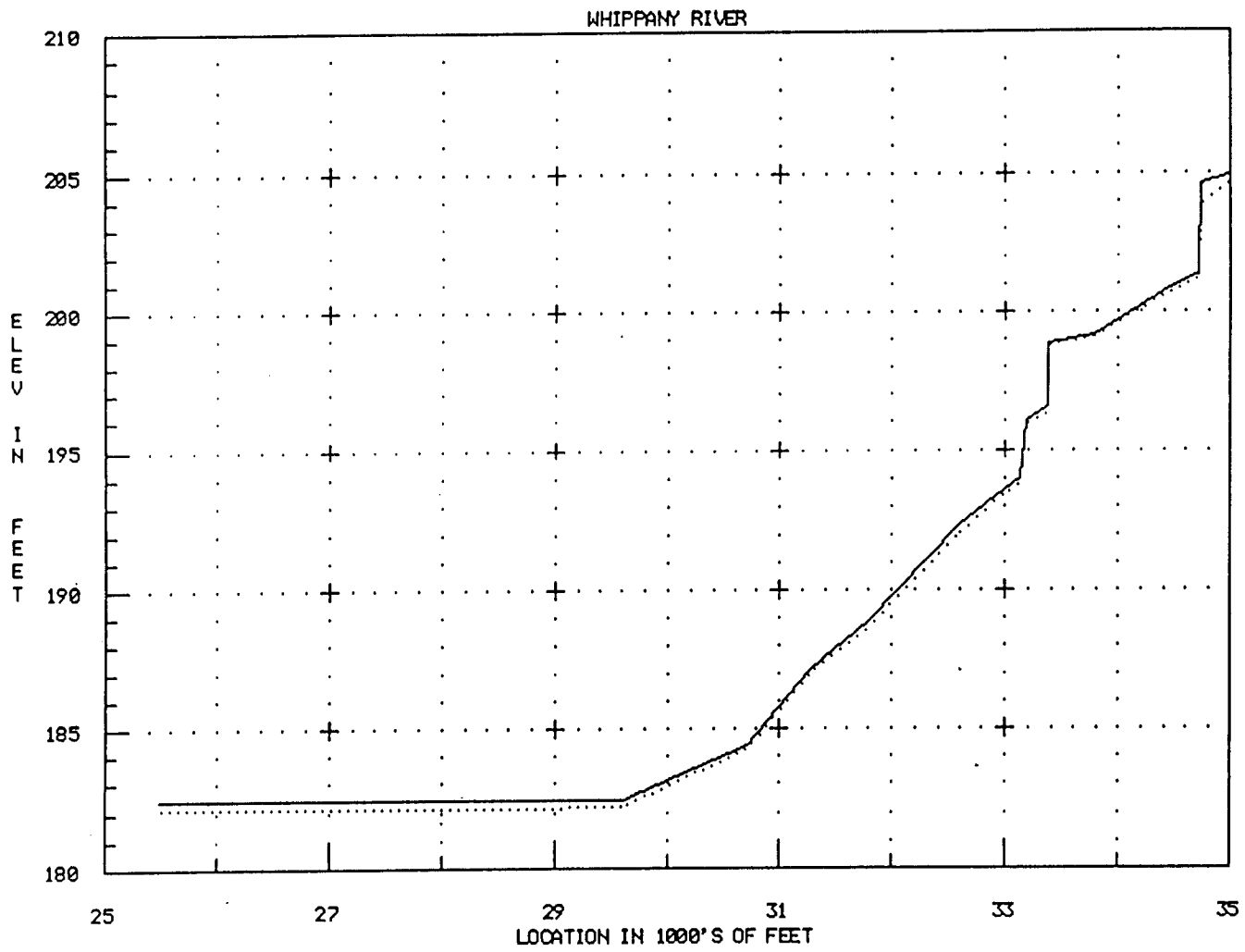


Figure C-109d

27APR95 10:16:46

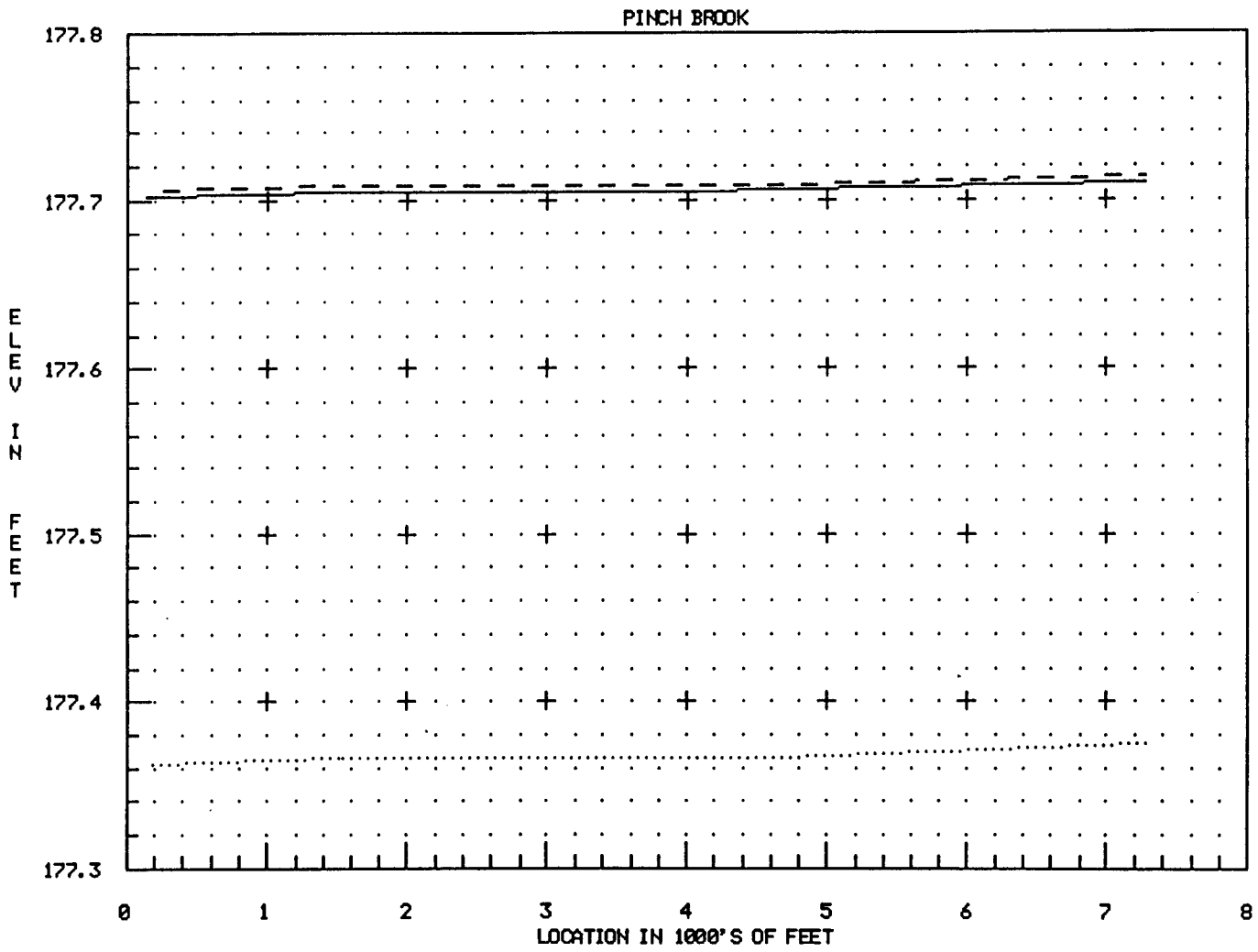


Figure C-109e

27APR95 10:32:46

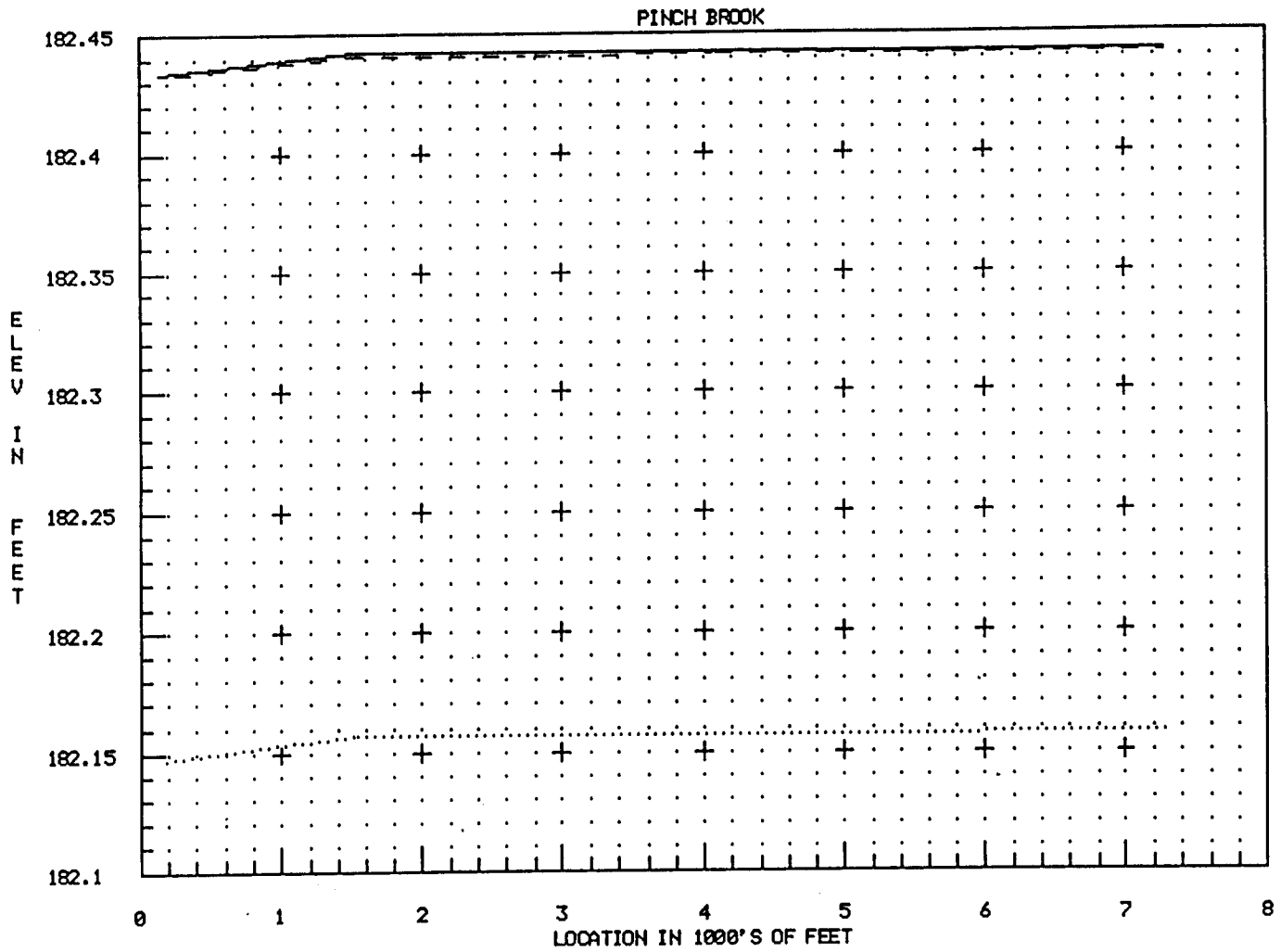


Figure C-109f

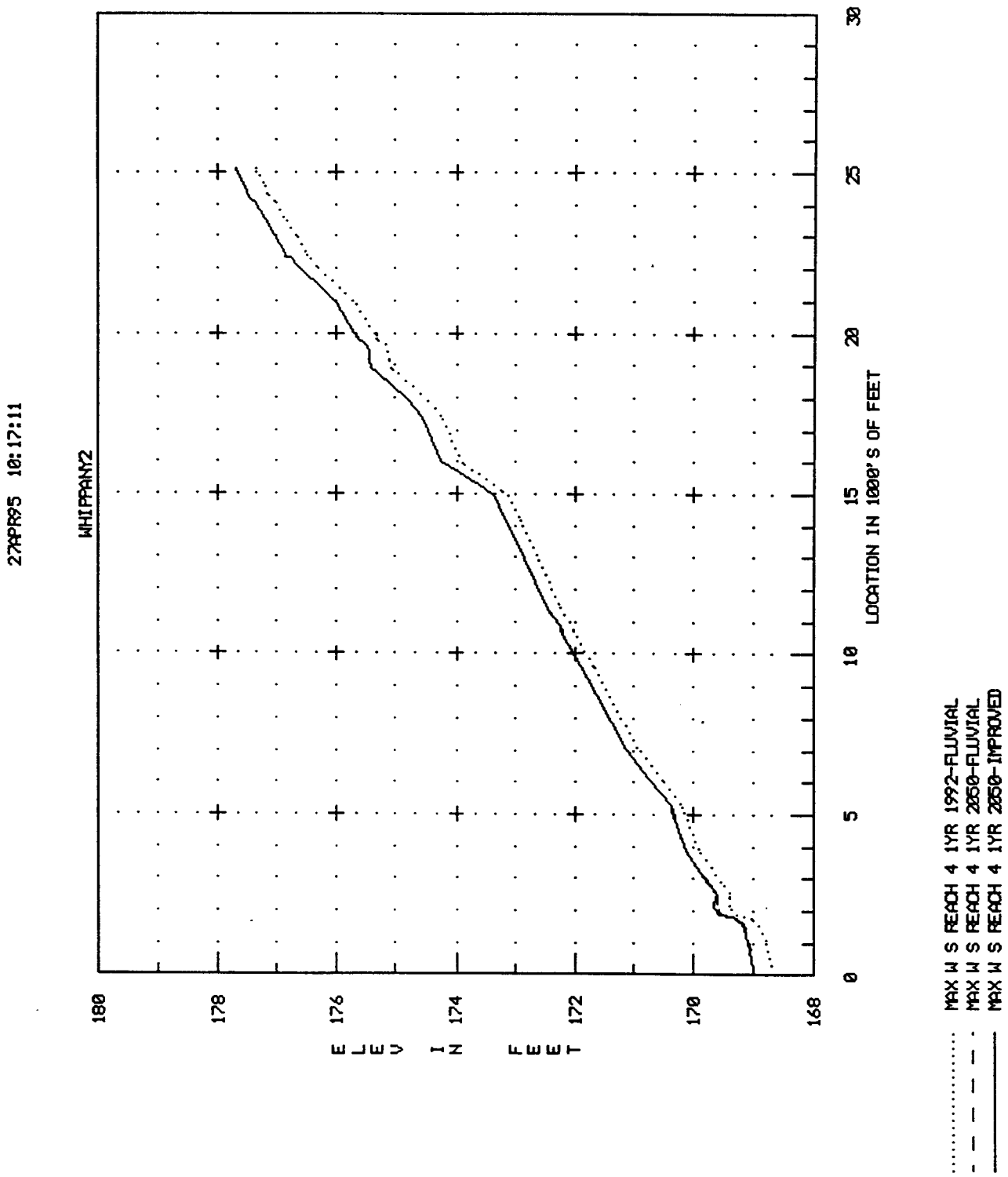


Figure C-109g

27APR95 10:33:13

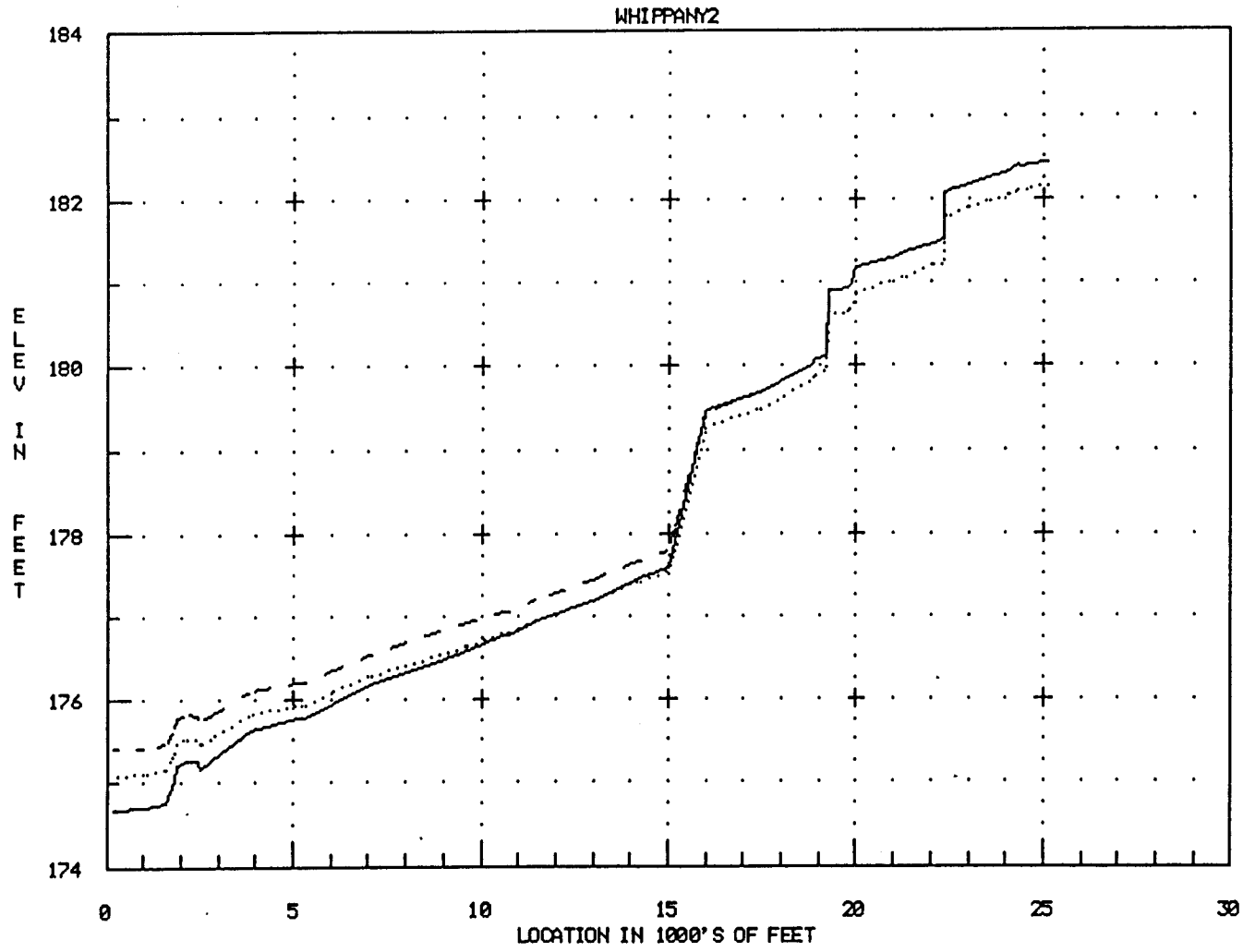


Figure C-109h

27APR95 10:18:02

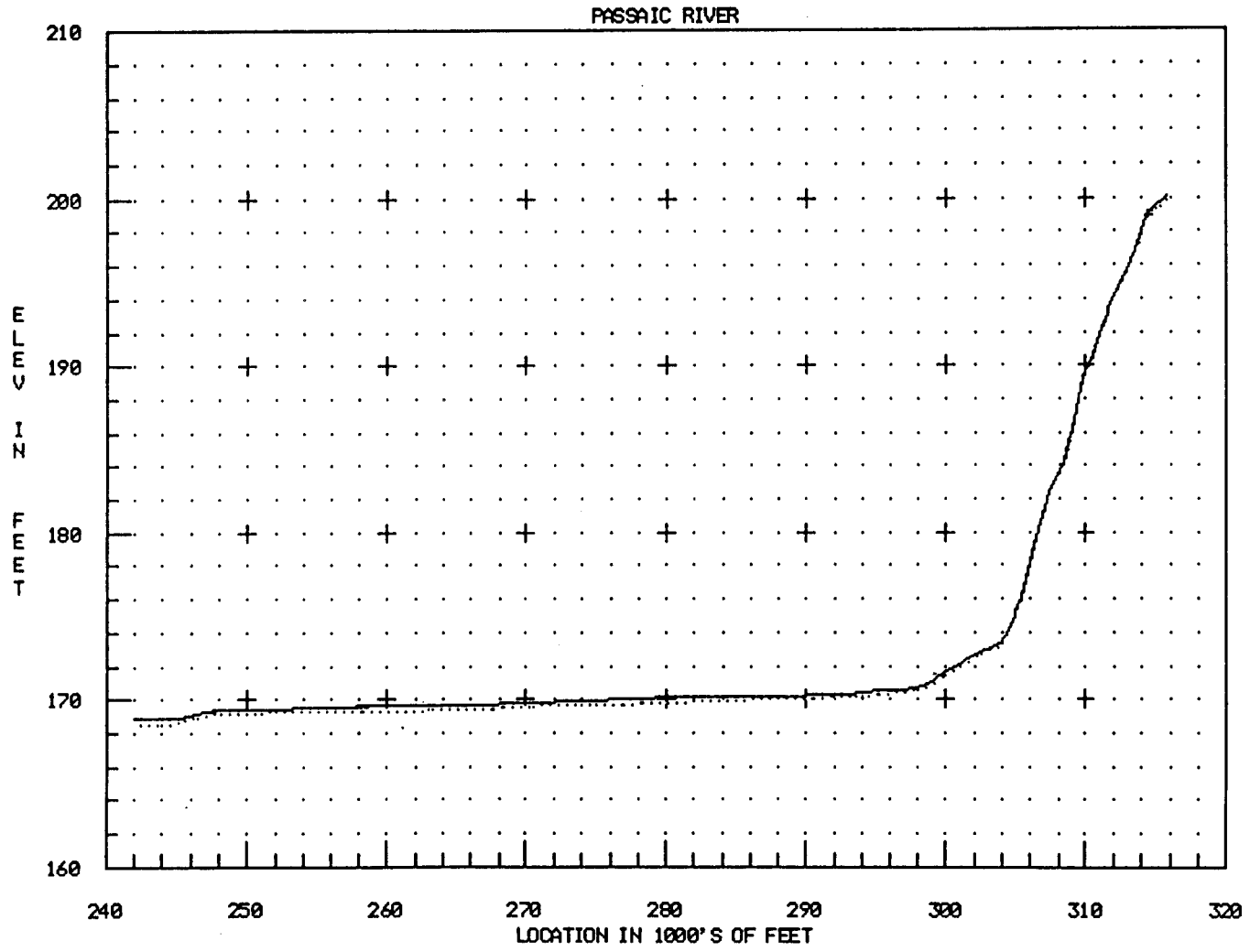


Figure C-109i

27APR95 10:34:06

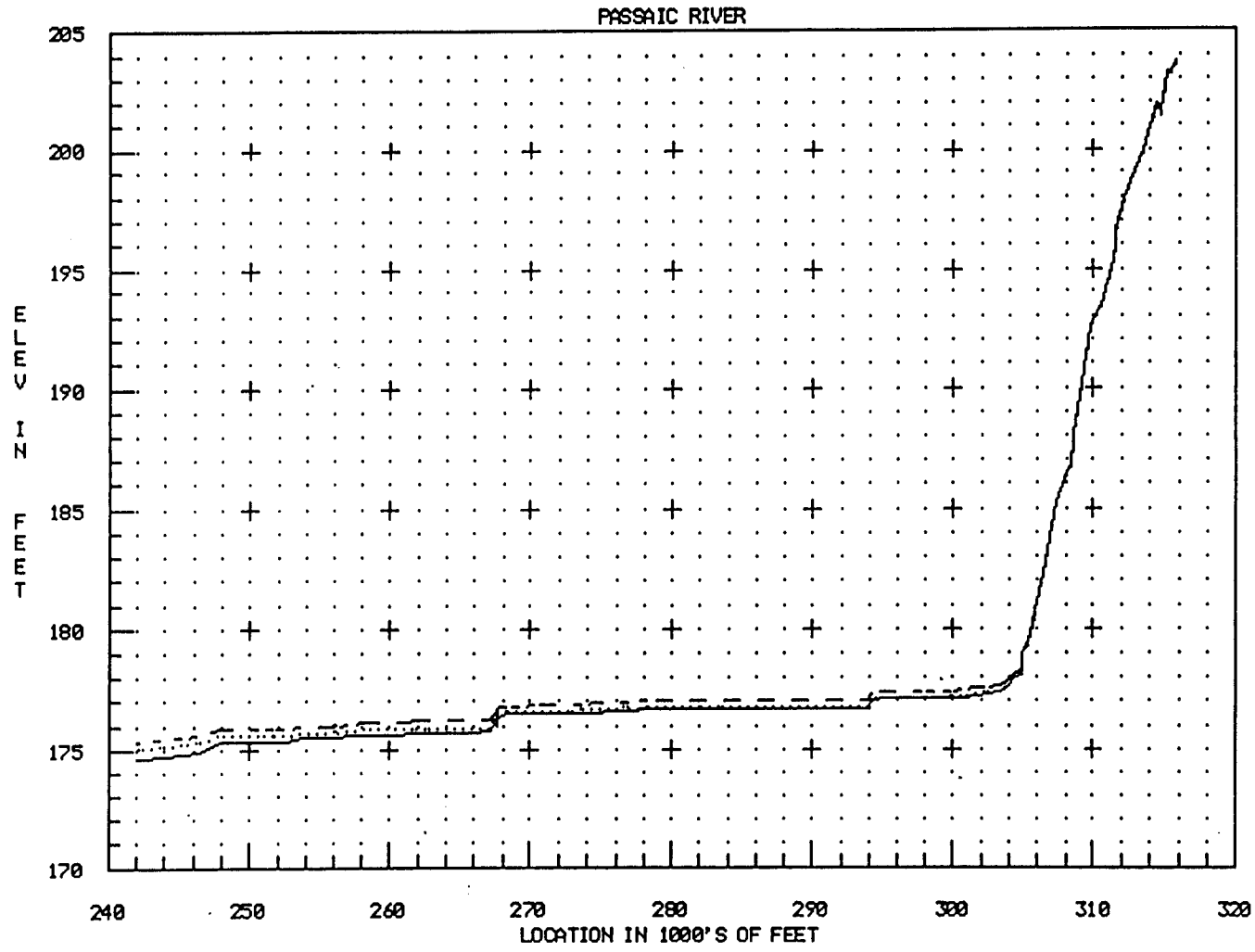


Figure C-109j

27APR95 10:18:28

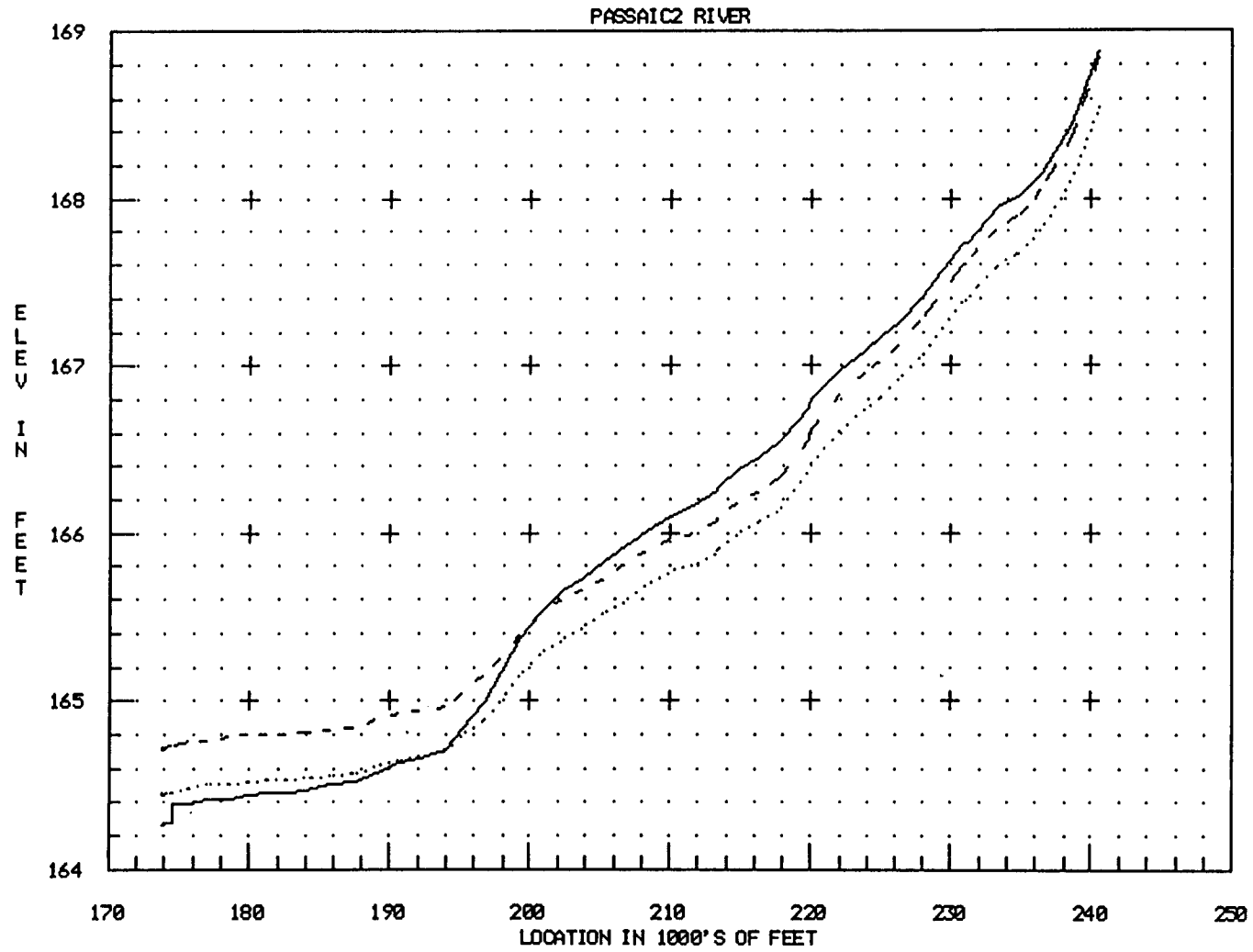


Figure C-109k

27APR95 10:34:34

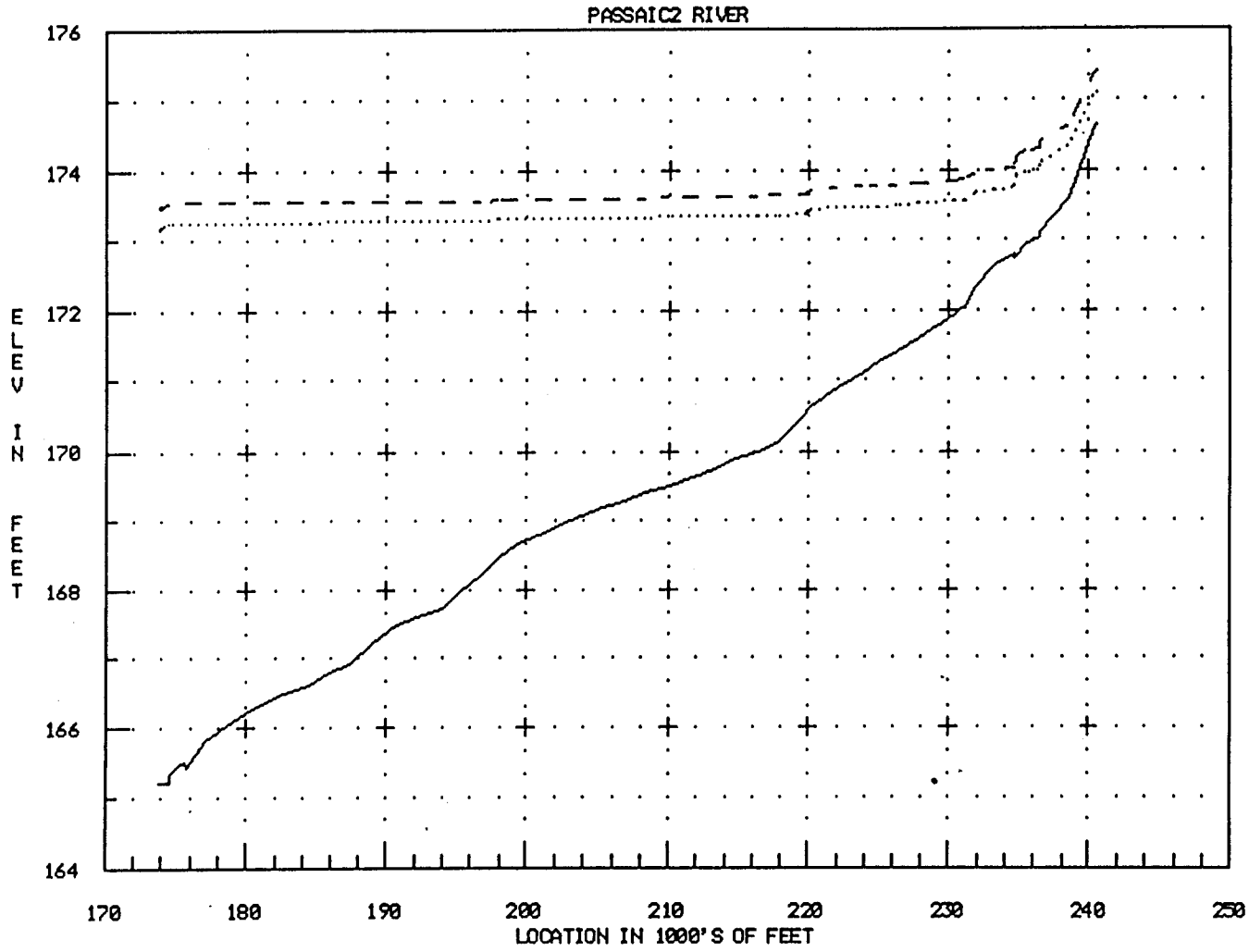


Figure C-1091

27APR95 10:18:54

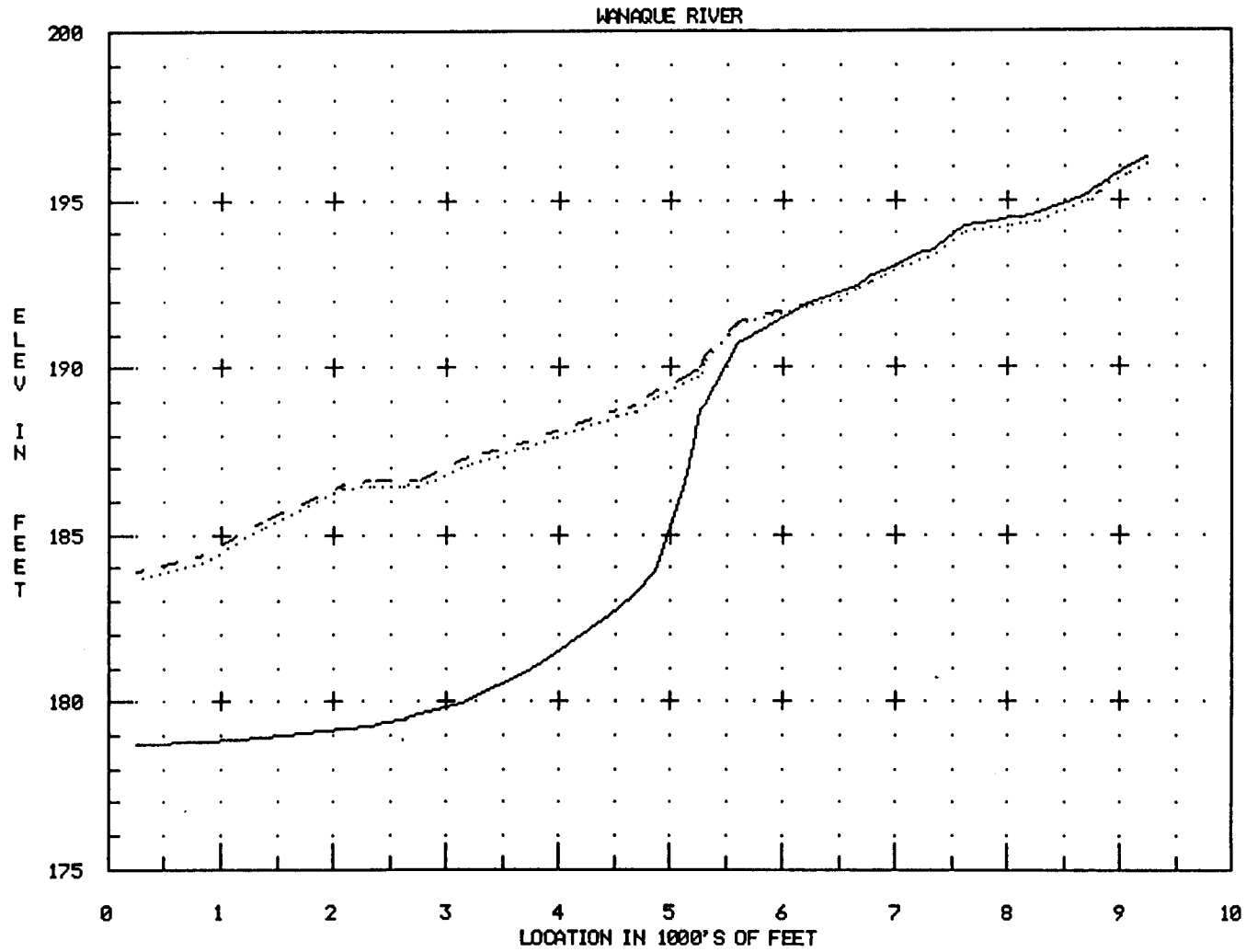


Figure C-109m

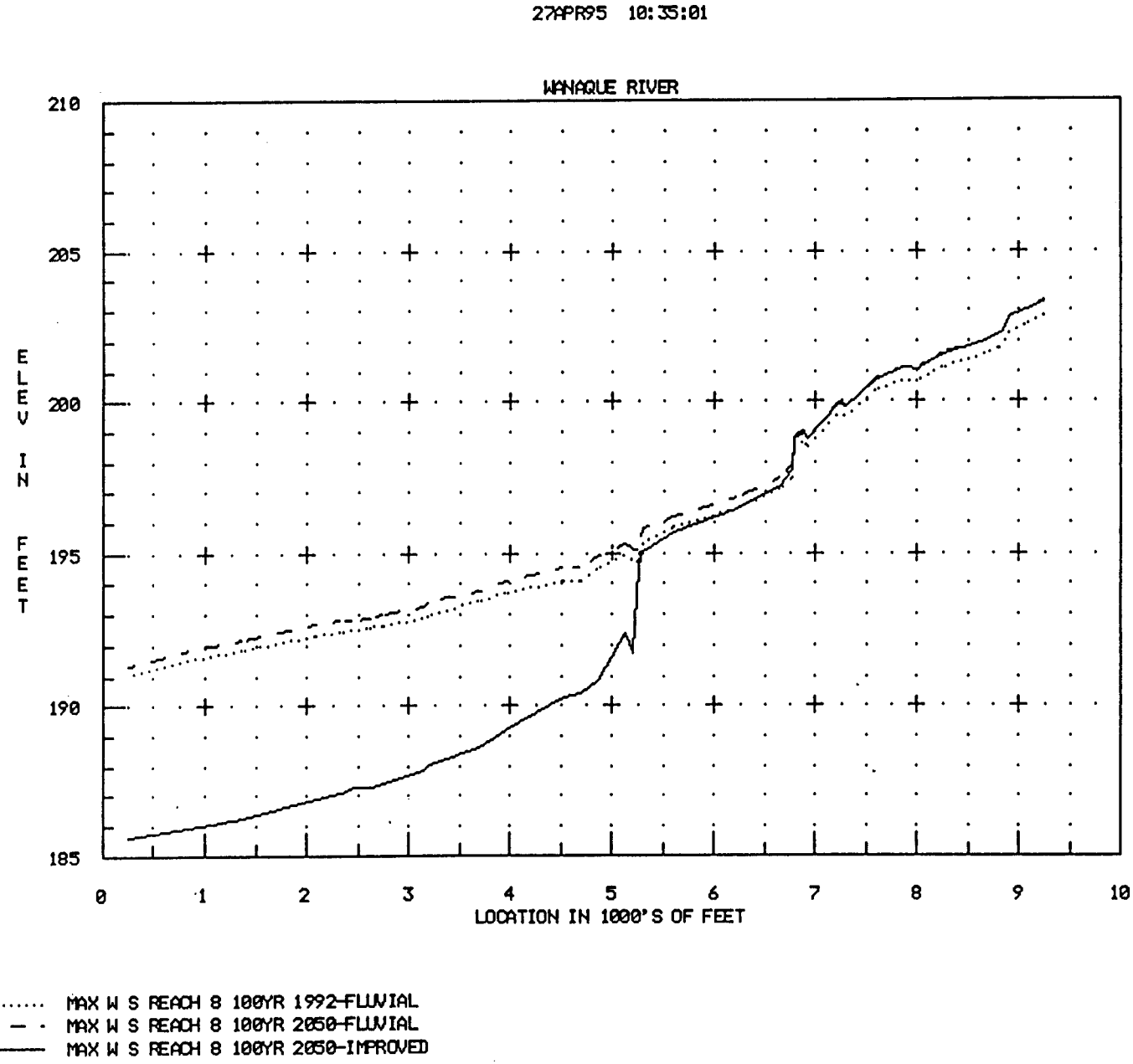


Figure C-109n

27APR95 10:19:19

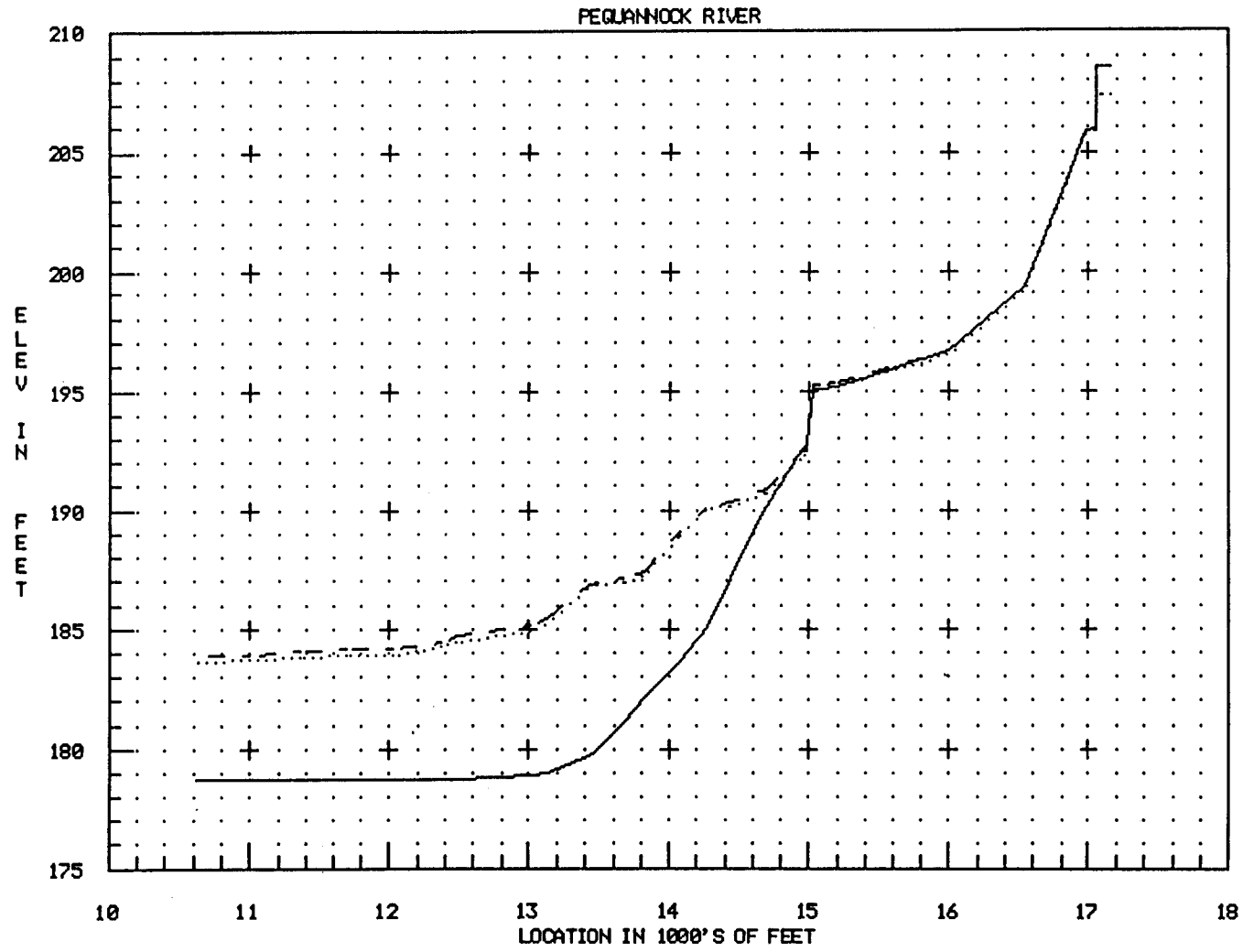


Figure C-1090

27APR95 10:35:28

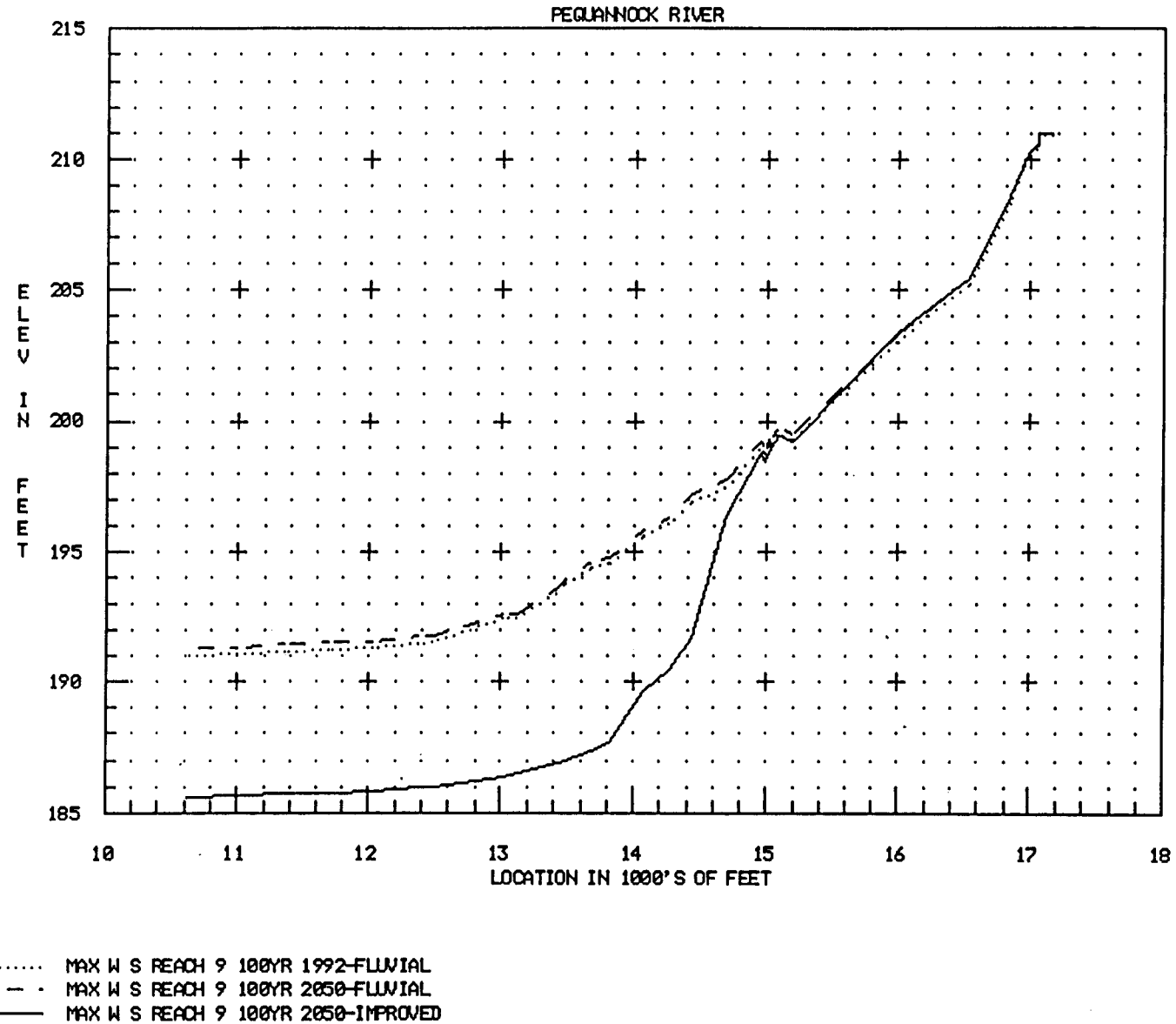


Figure C-109p

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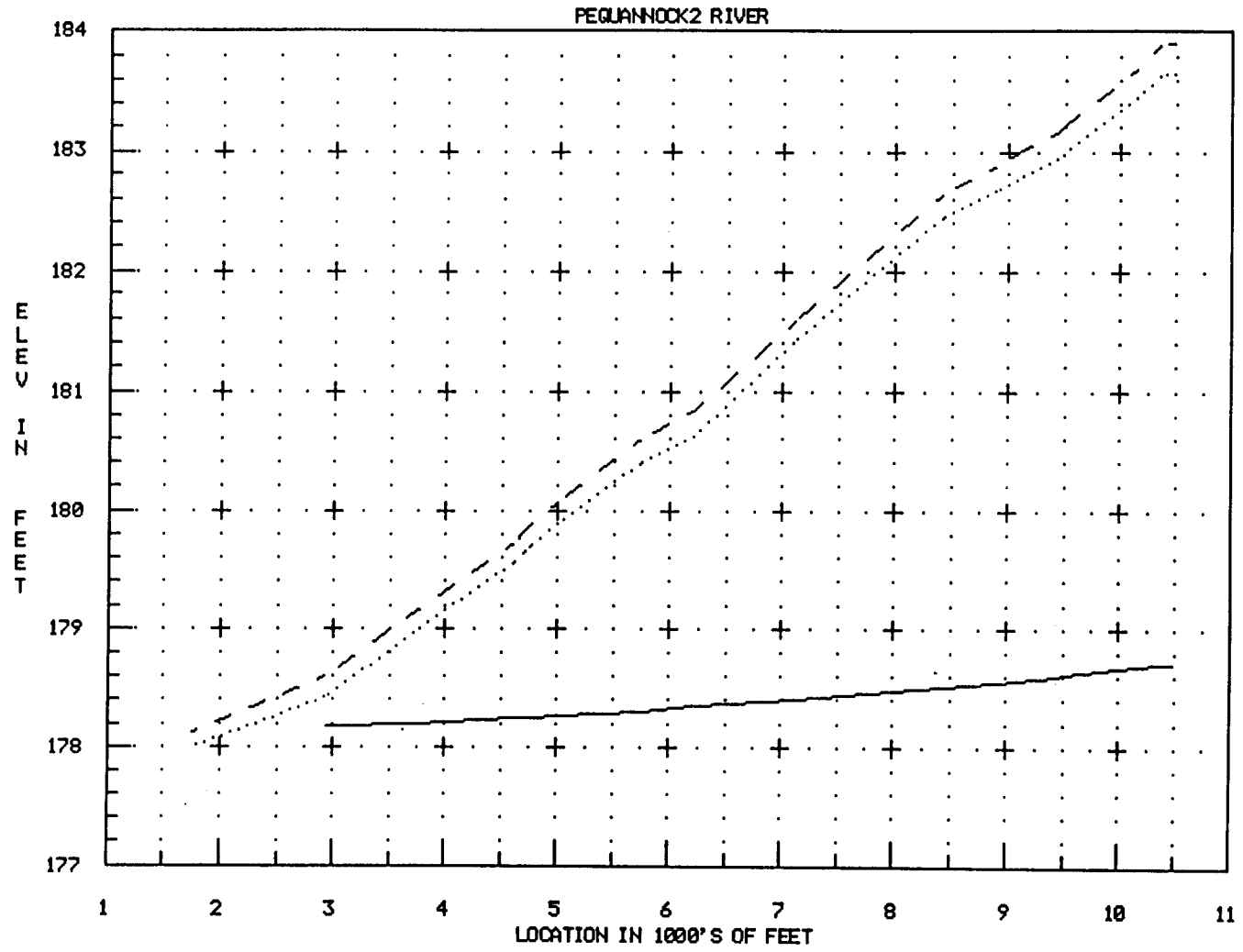

Figure C-109q

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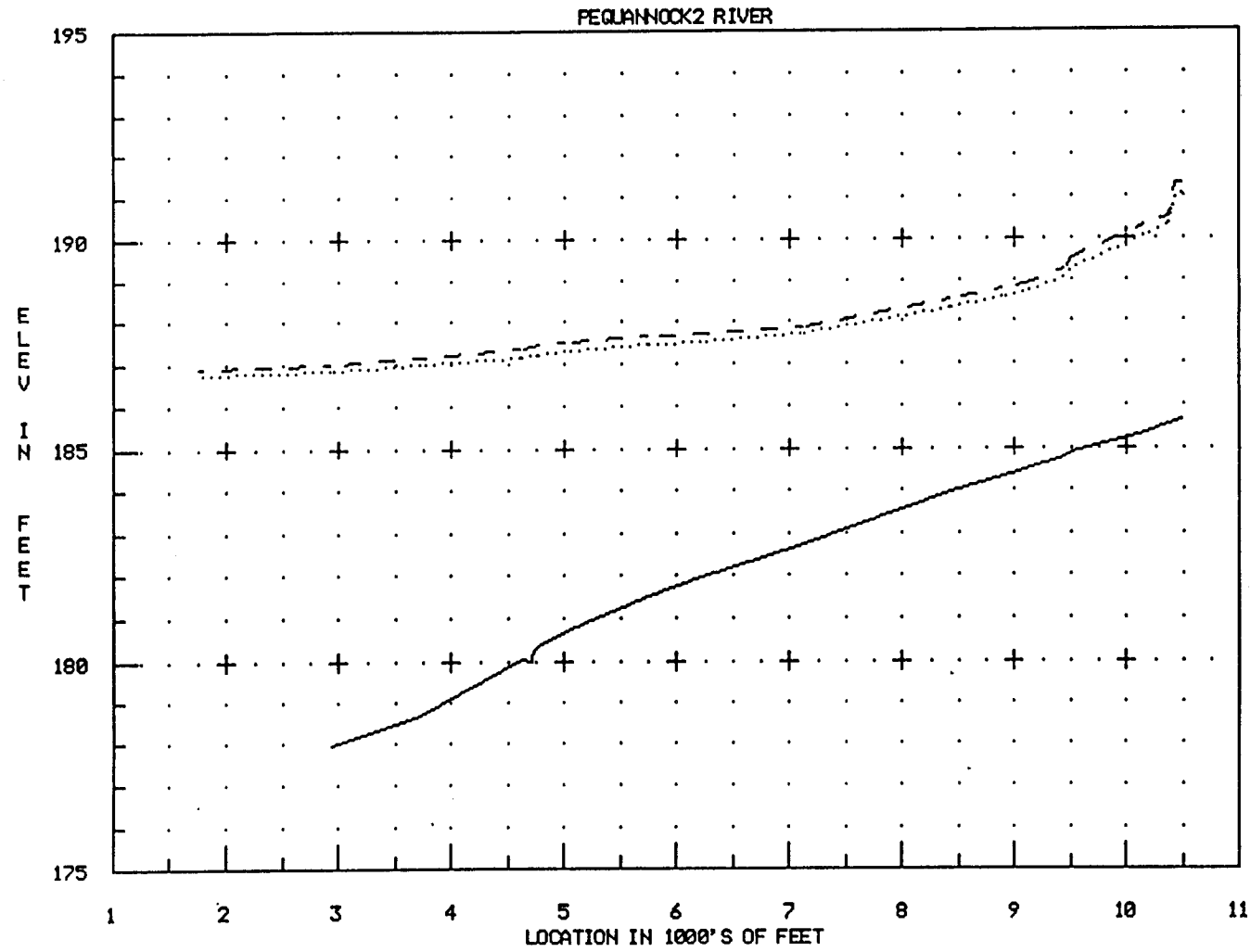

Figure C-109r

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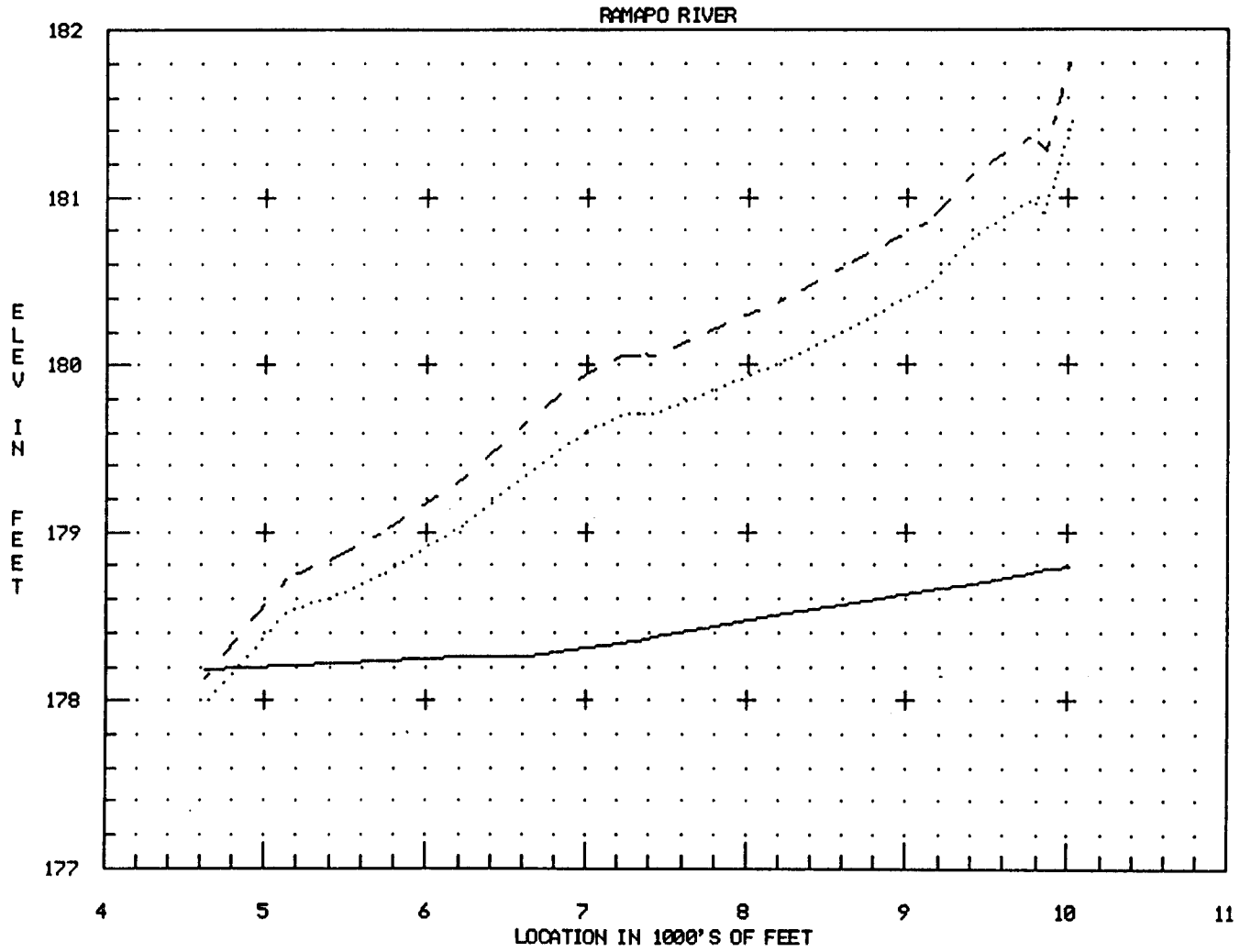
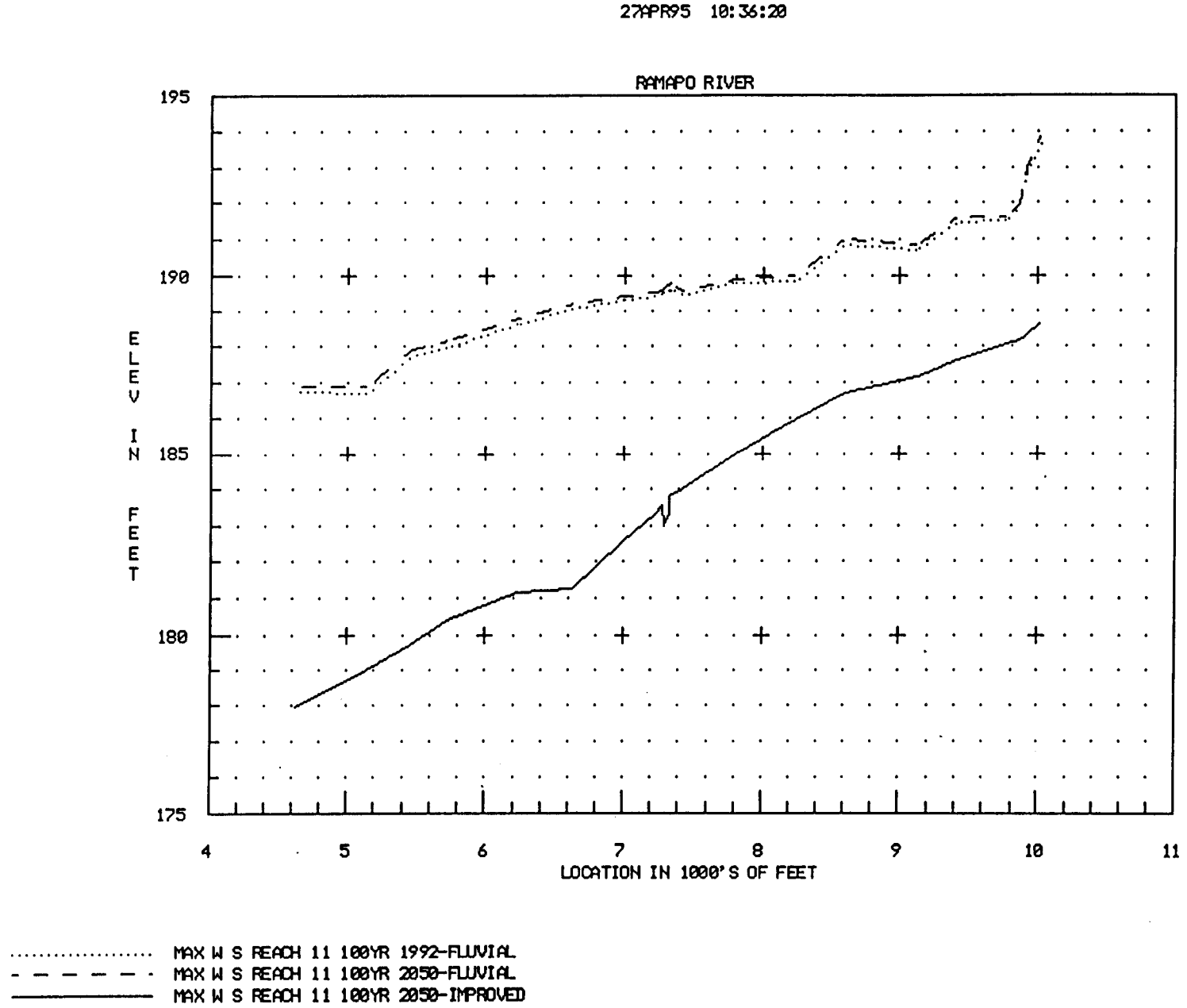


Figure C-109s



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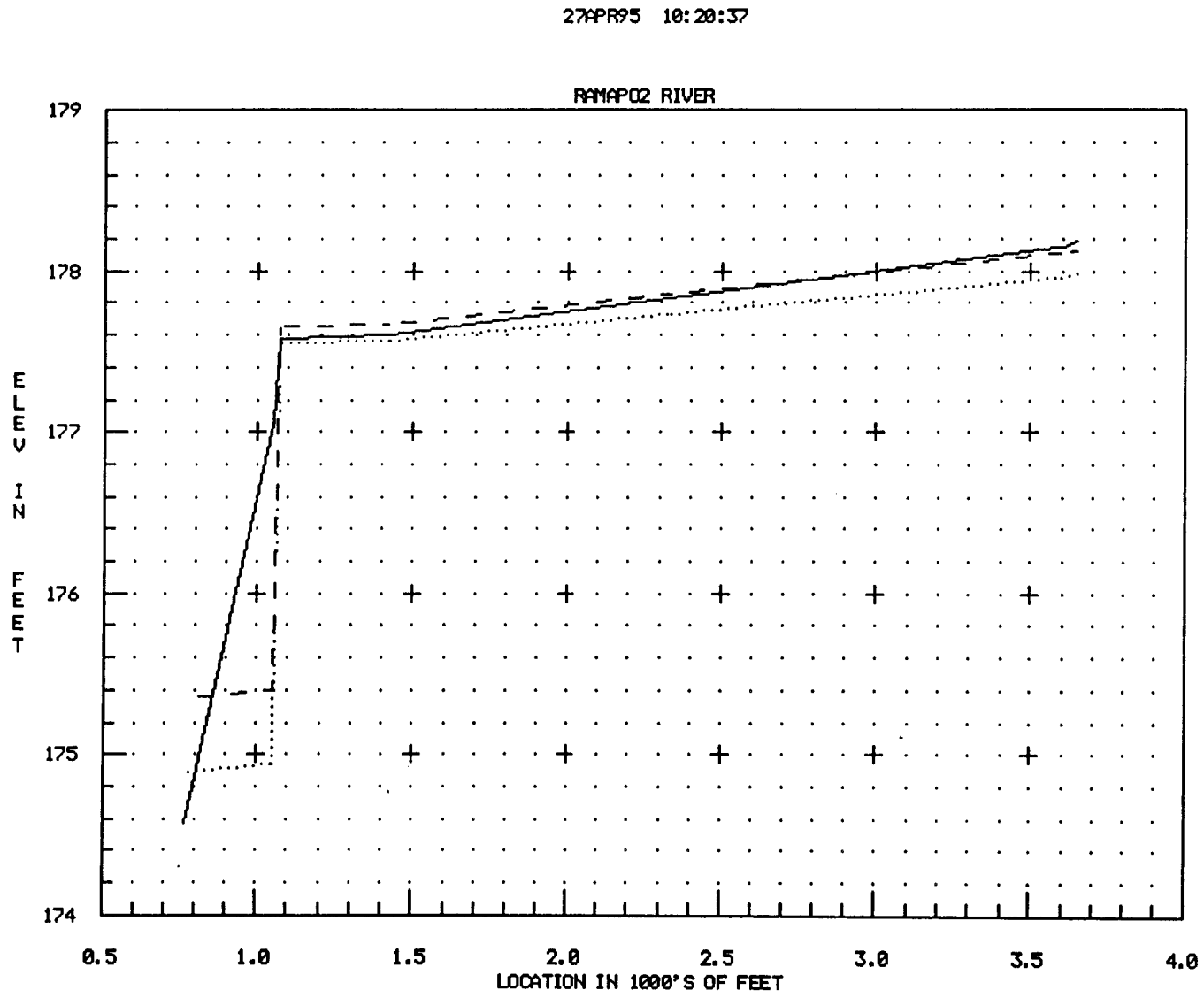

Figure C-109u

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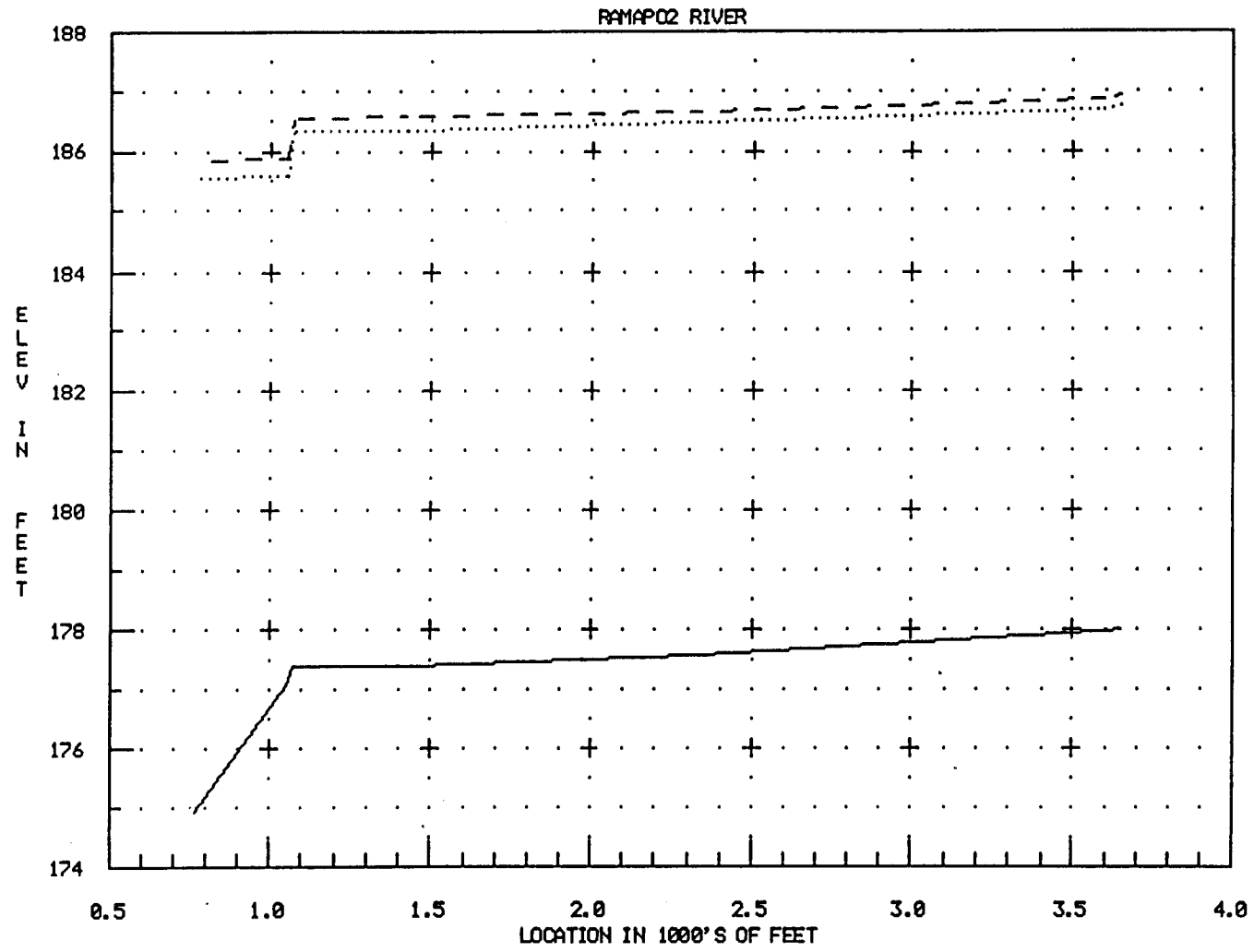

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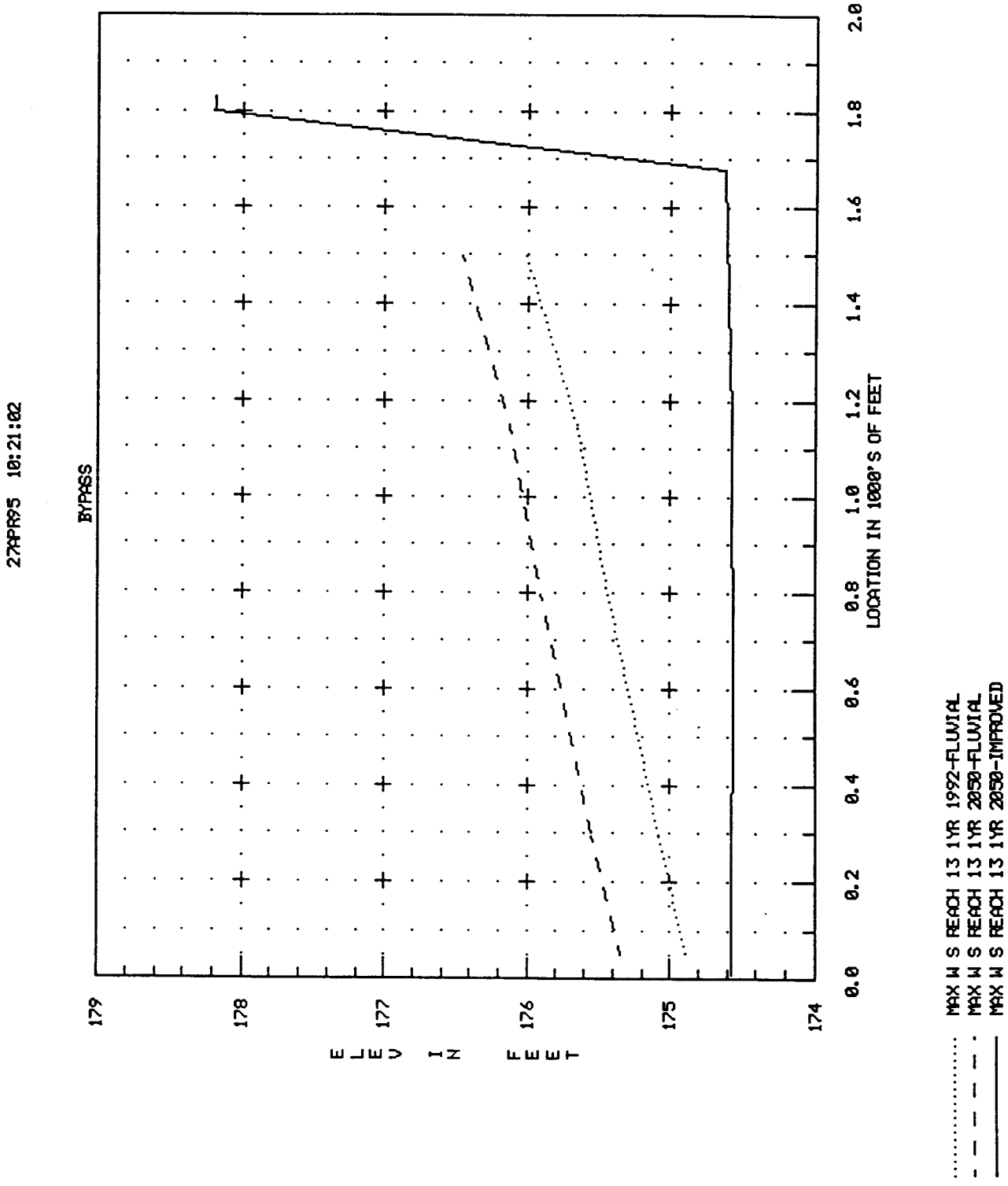

Figure C-109w

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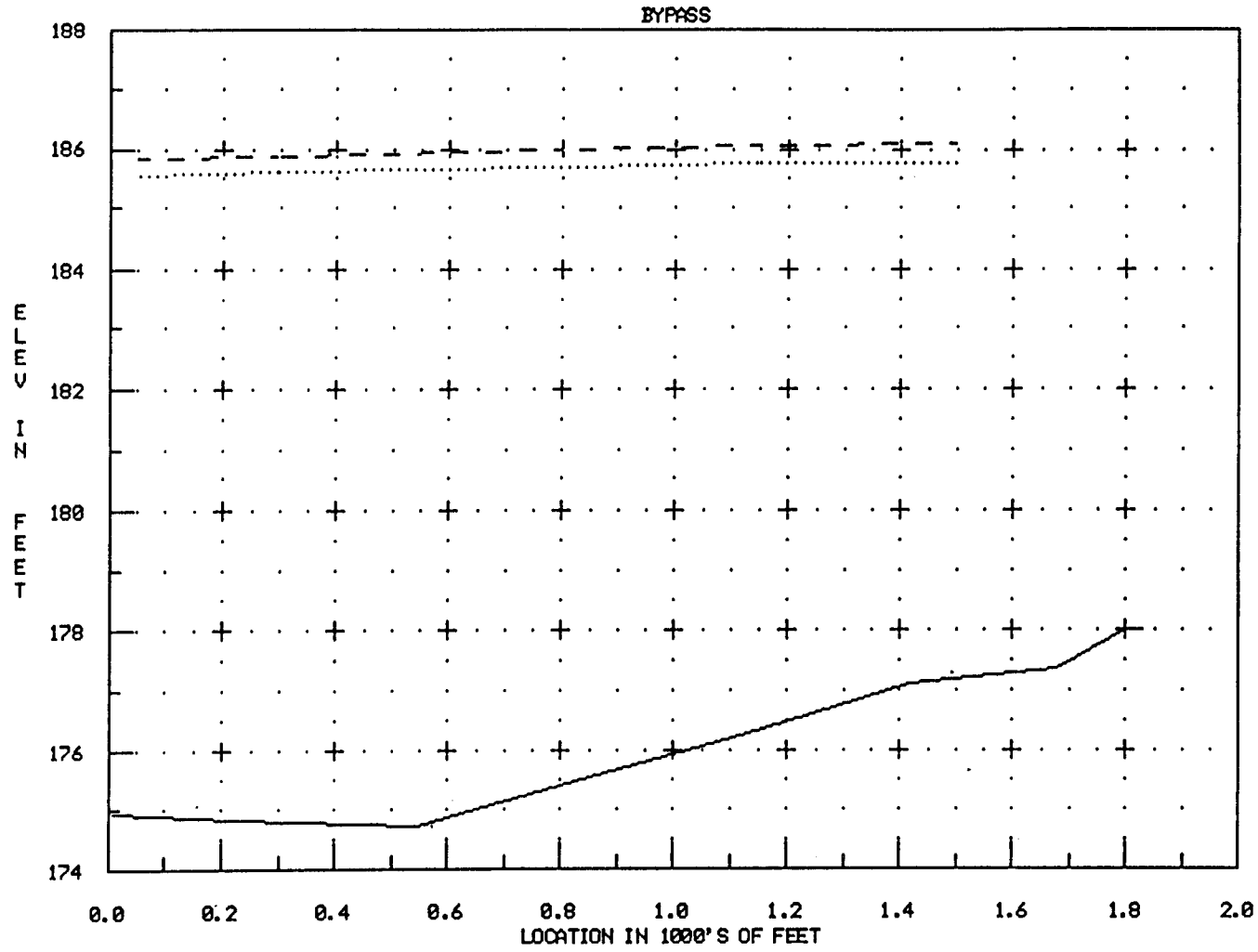

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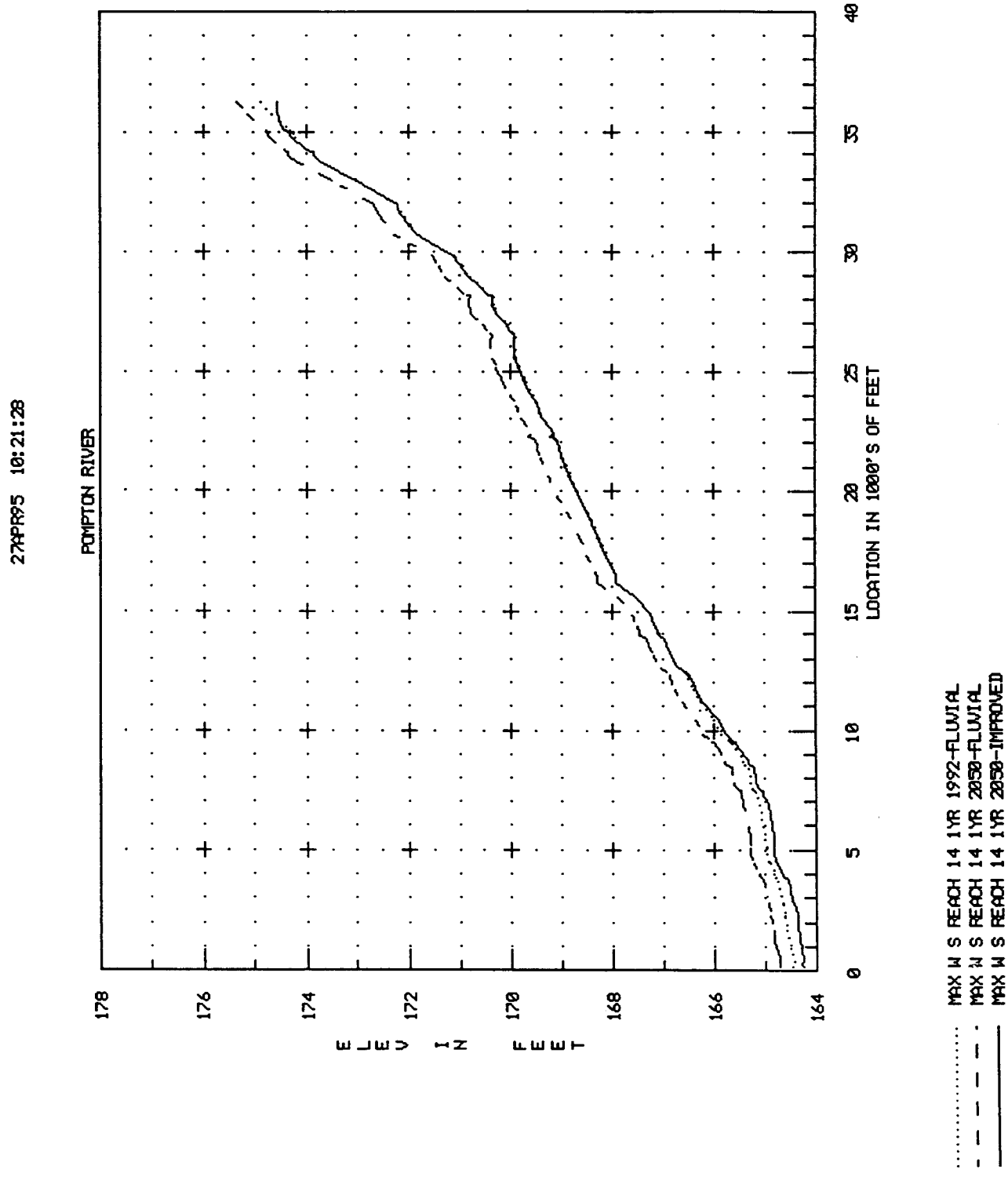

Figure C-109y

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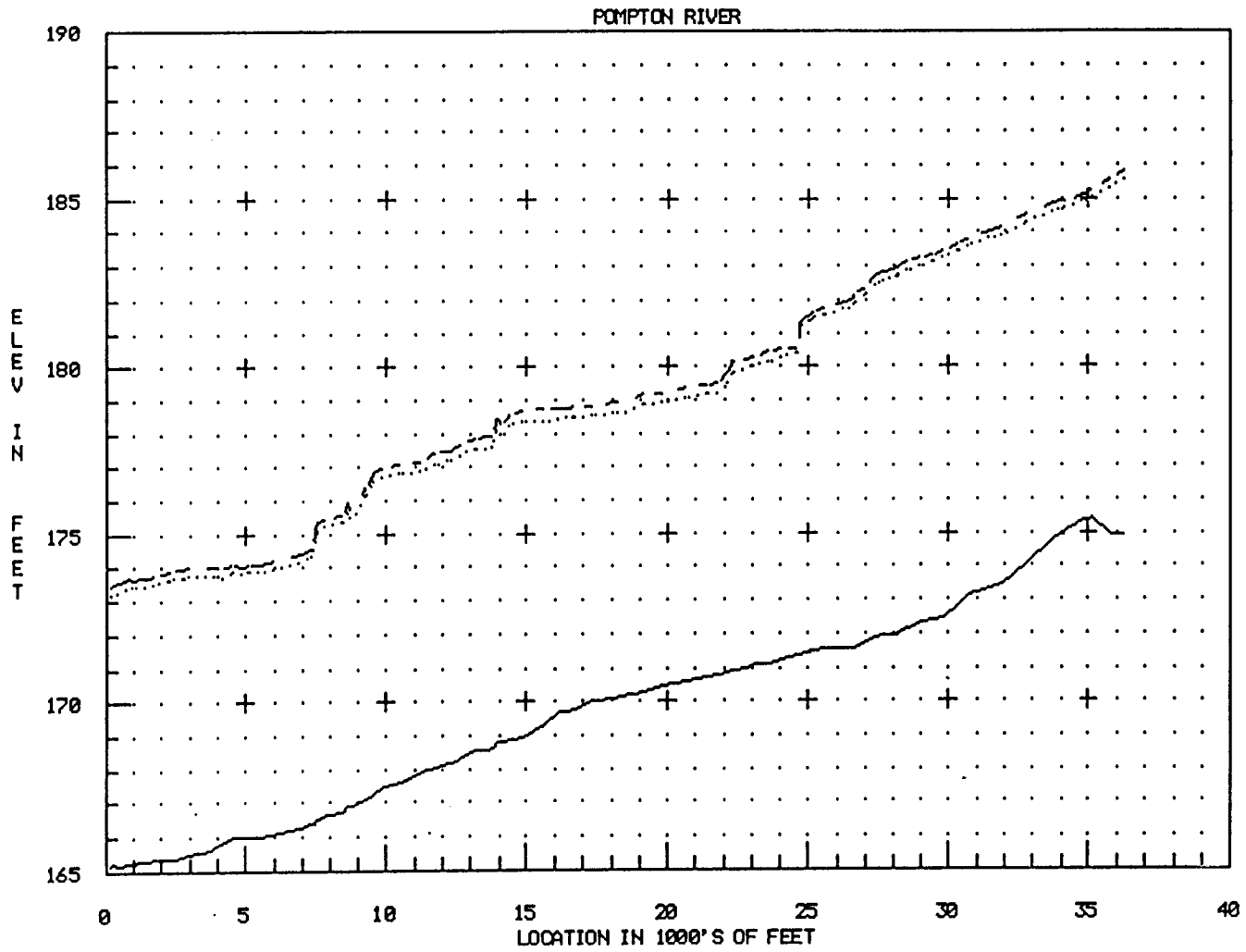

Figure C-109z

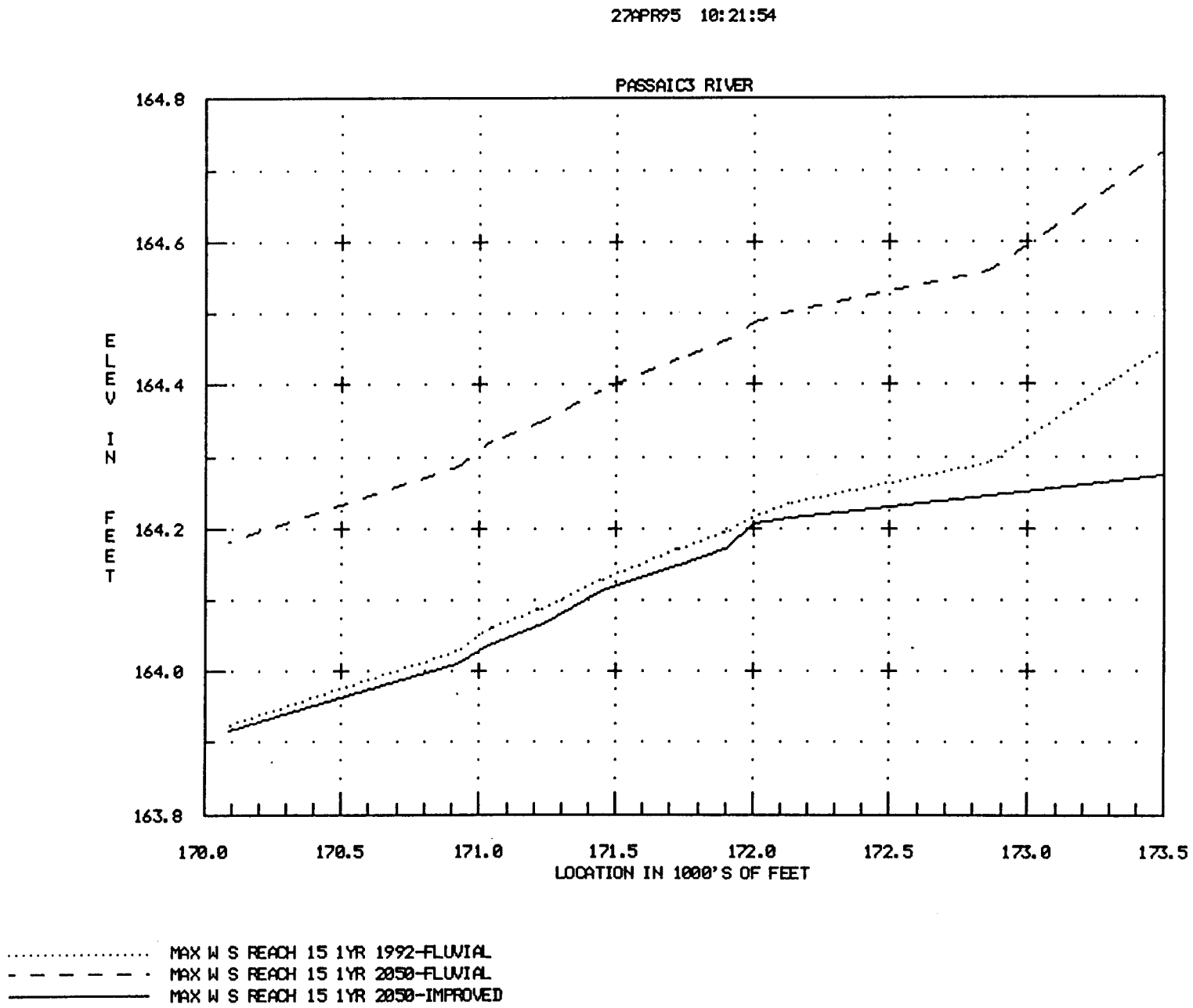

Figure C-109aa

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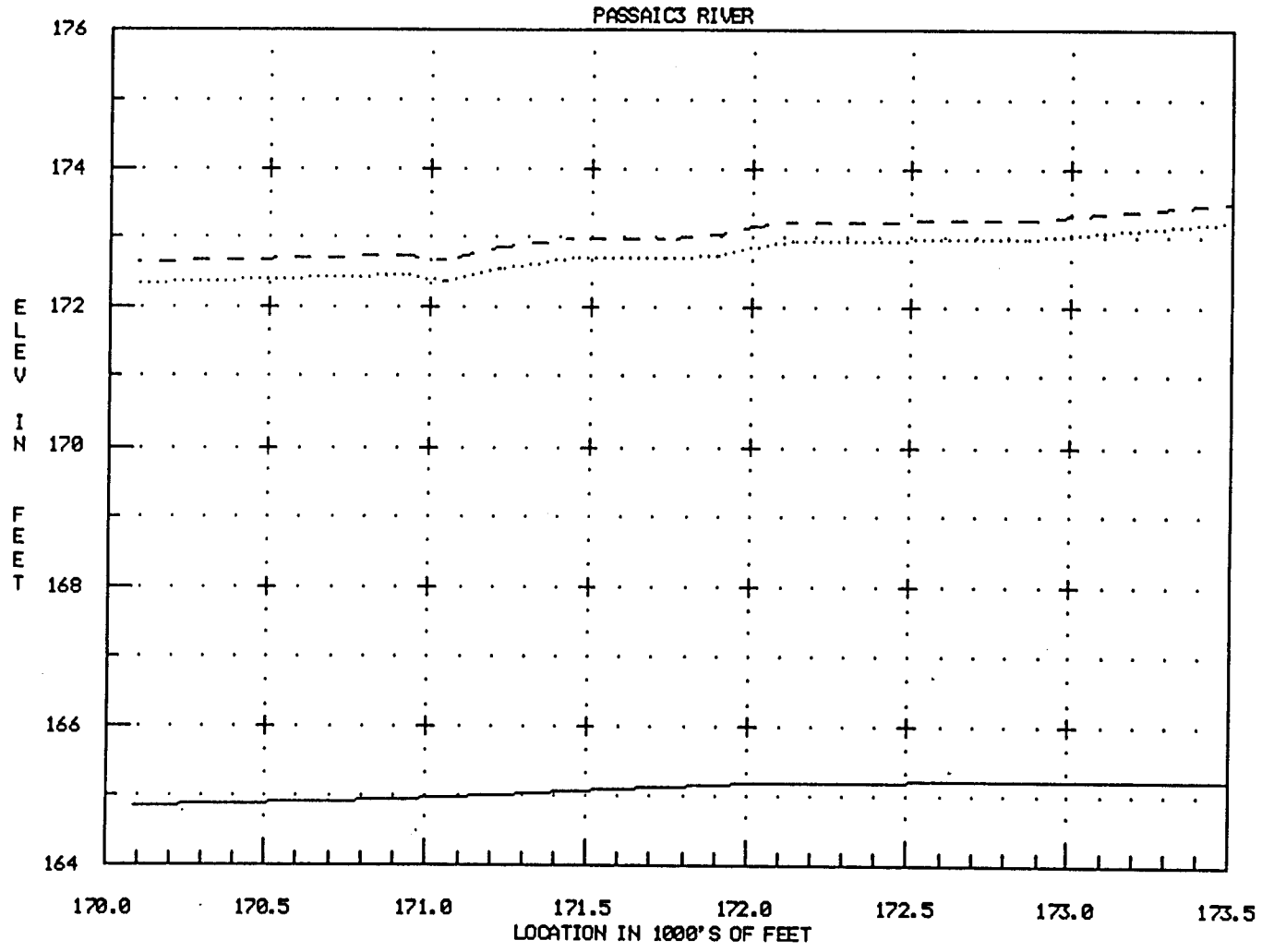


Figure C-109bb

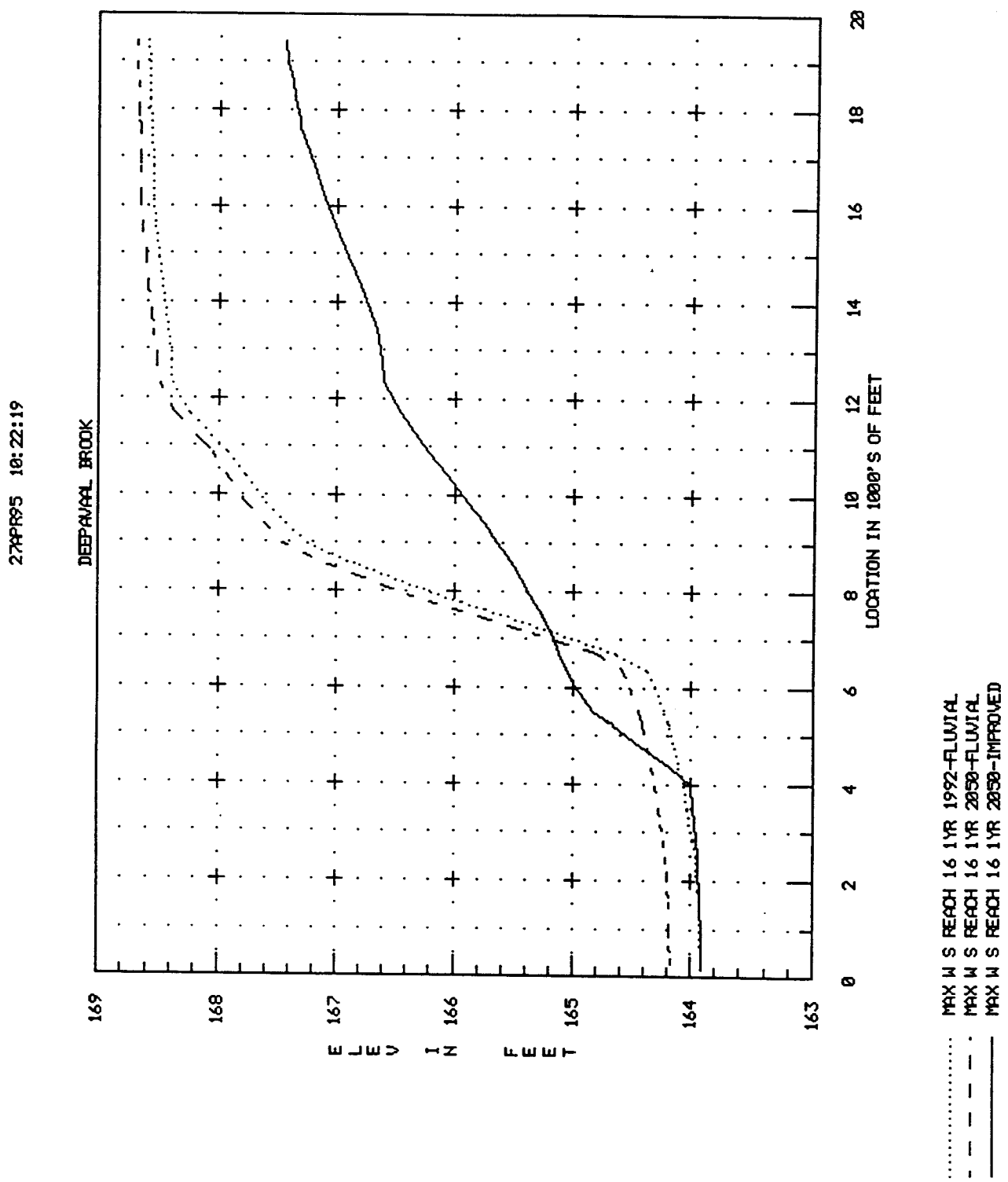


Figure C-109cc

27APR95 10:38:33

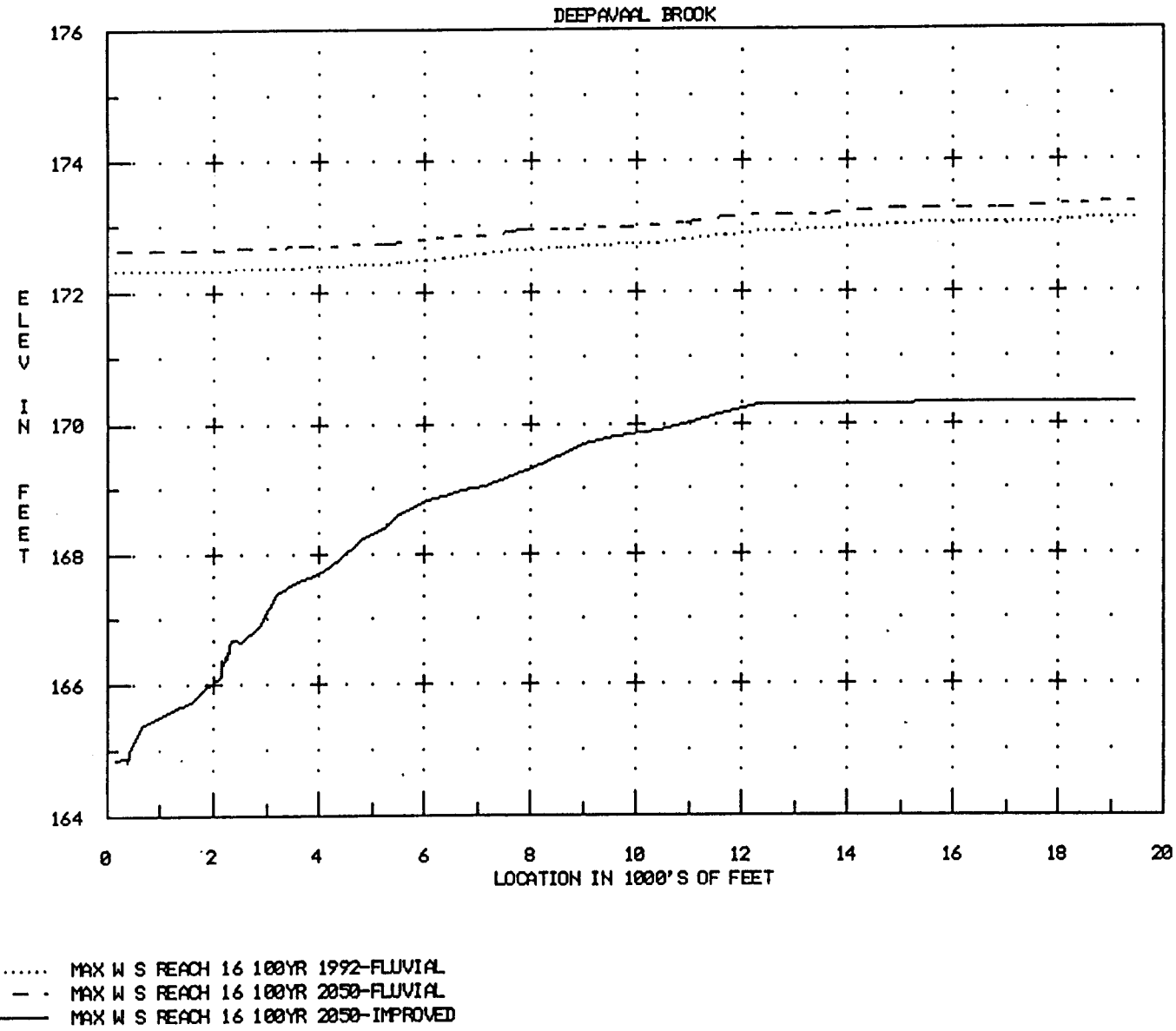


Figure C-109dd

27APR95 10:22:46

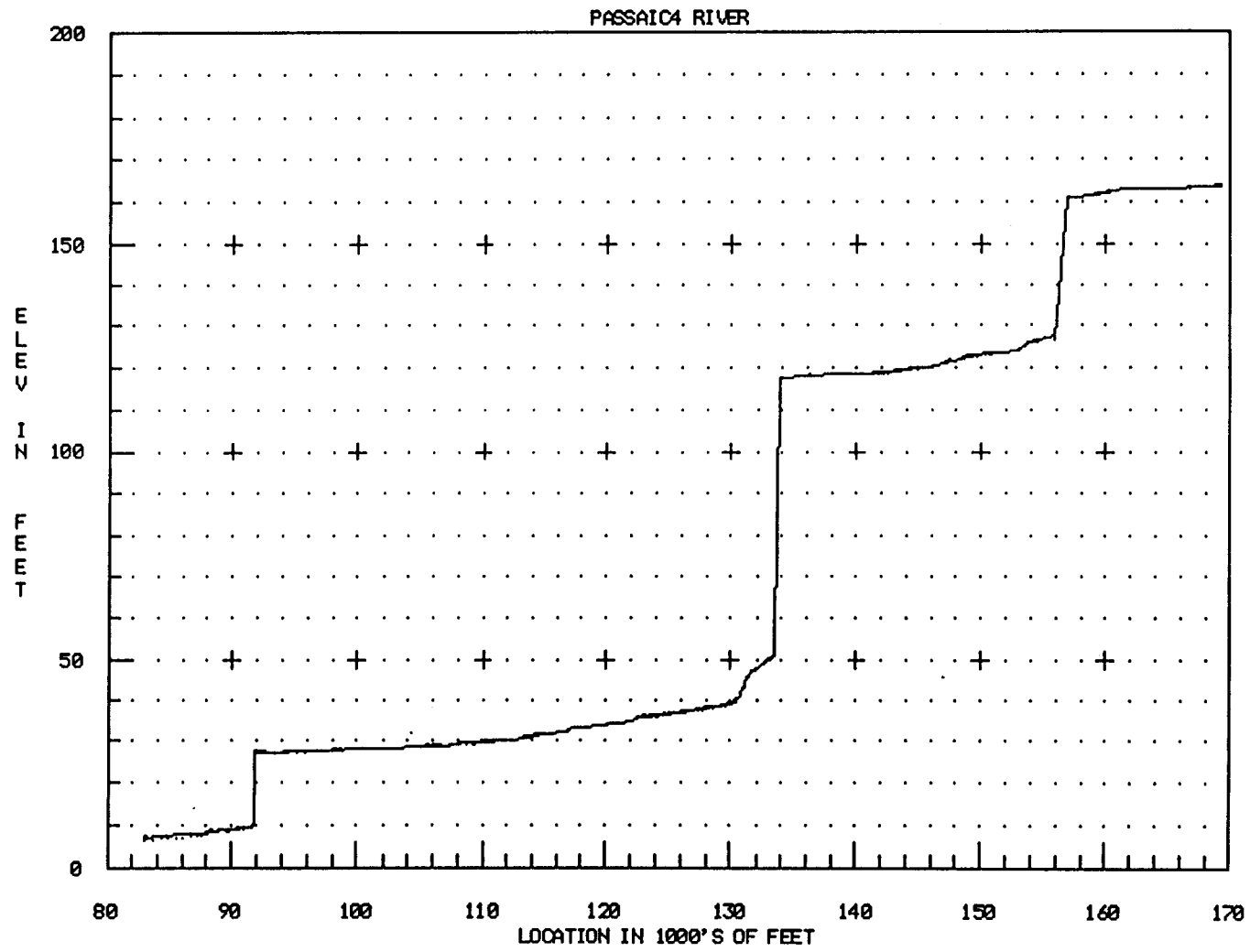


Figure C-109ee

27APR95 10:39:01

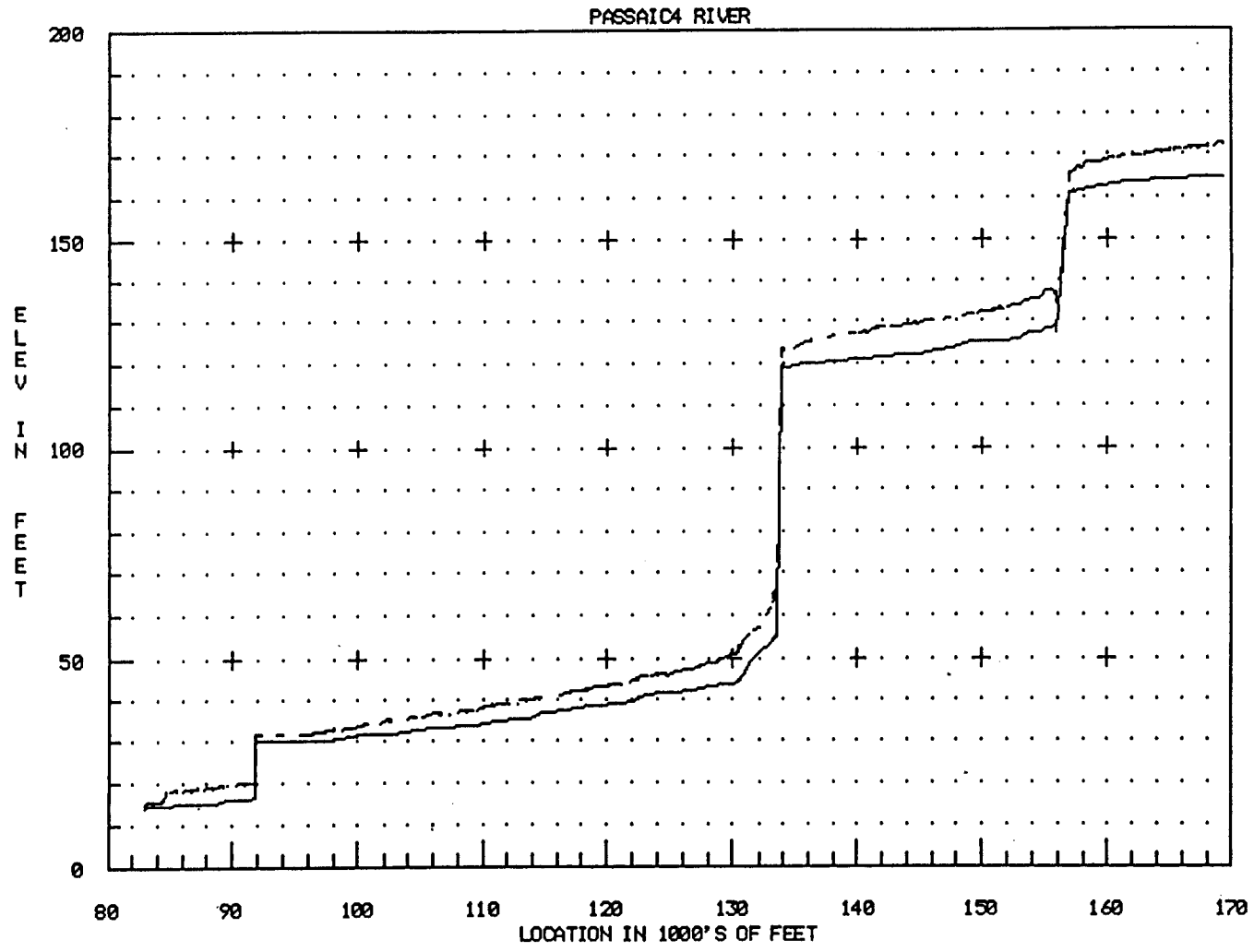


Figure C-109ff

27APR95 10:23:12

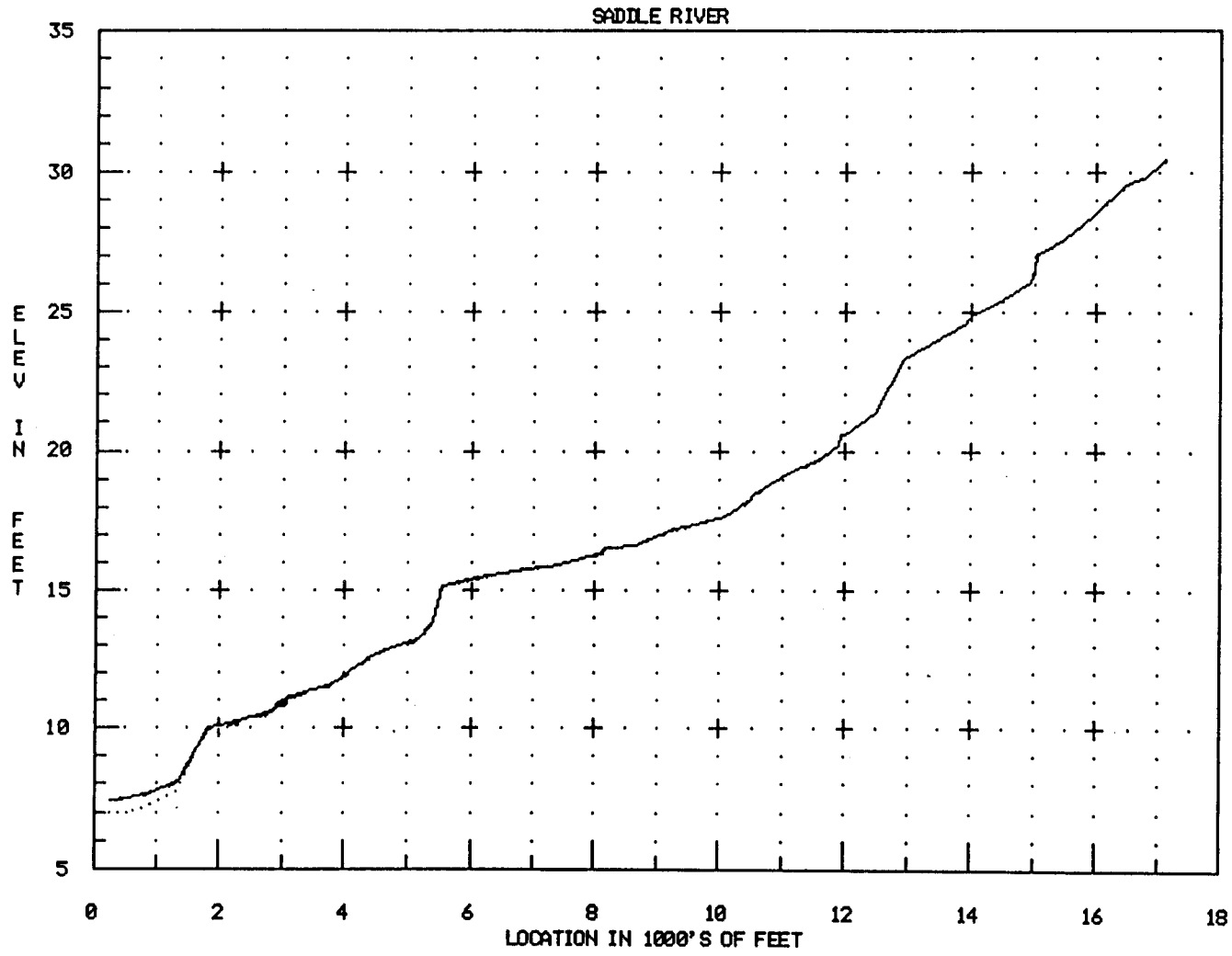


Figure C-109gg

27APR95 10:39:28

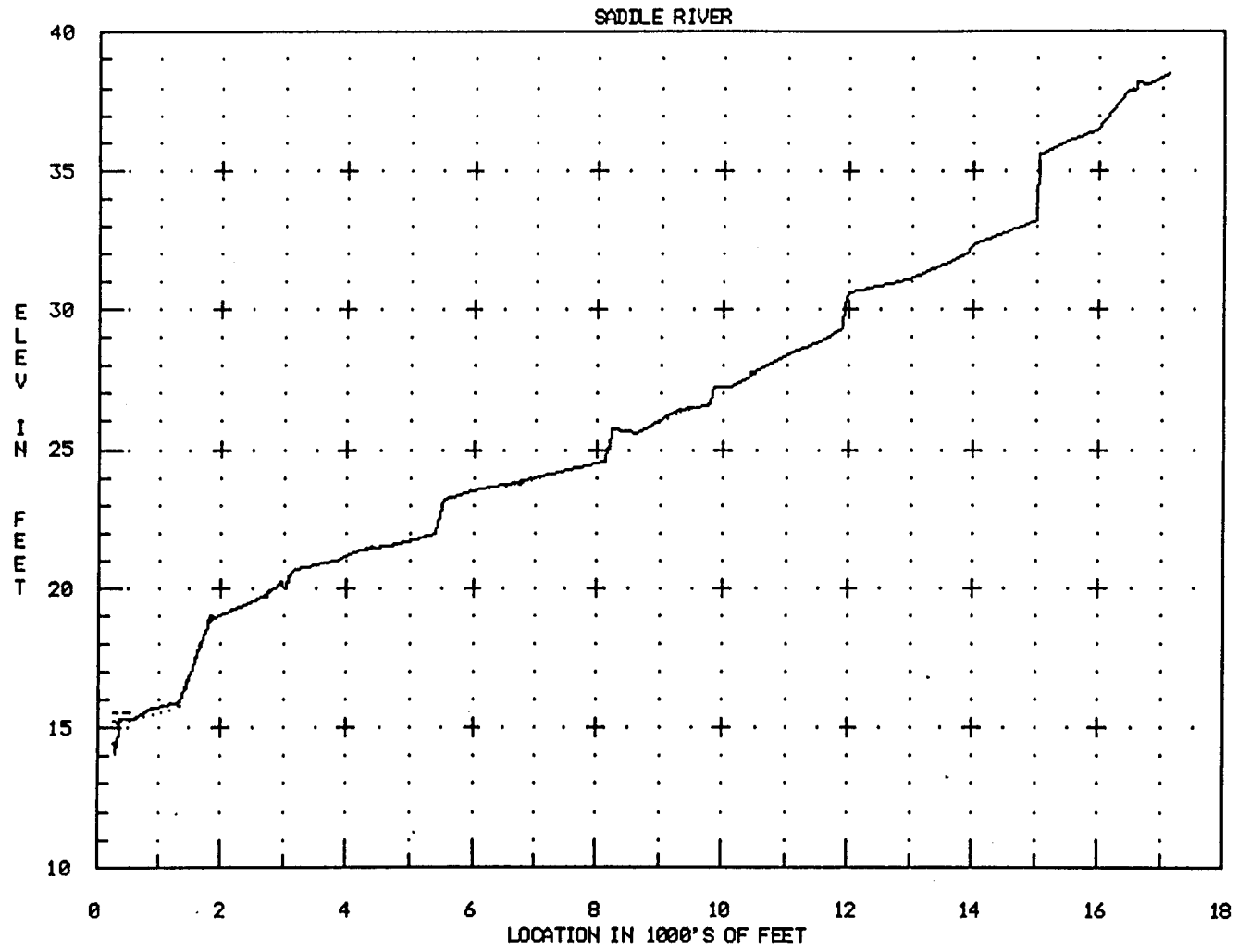


Figure C-109h

27APR95 10:23:38

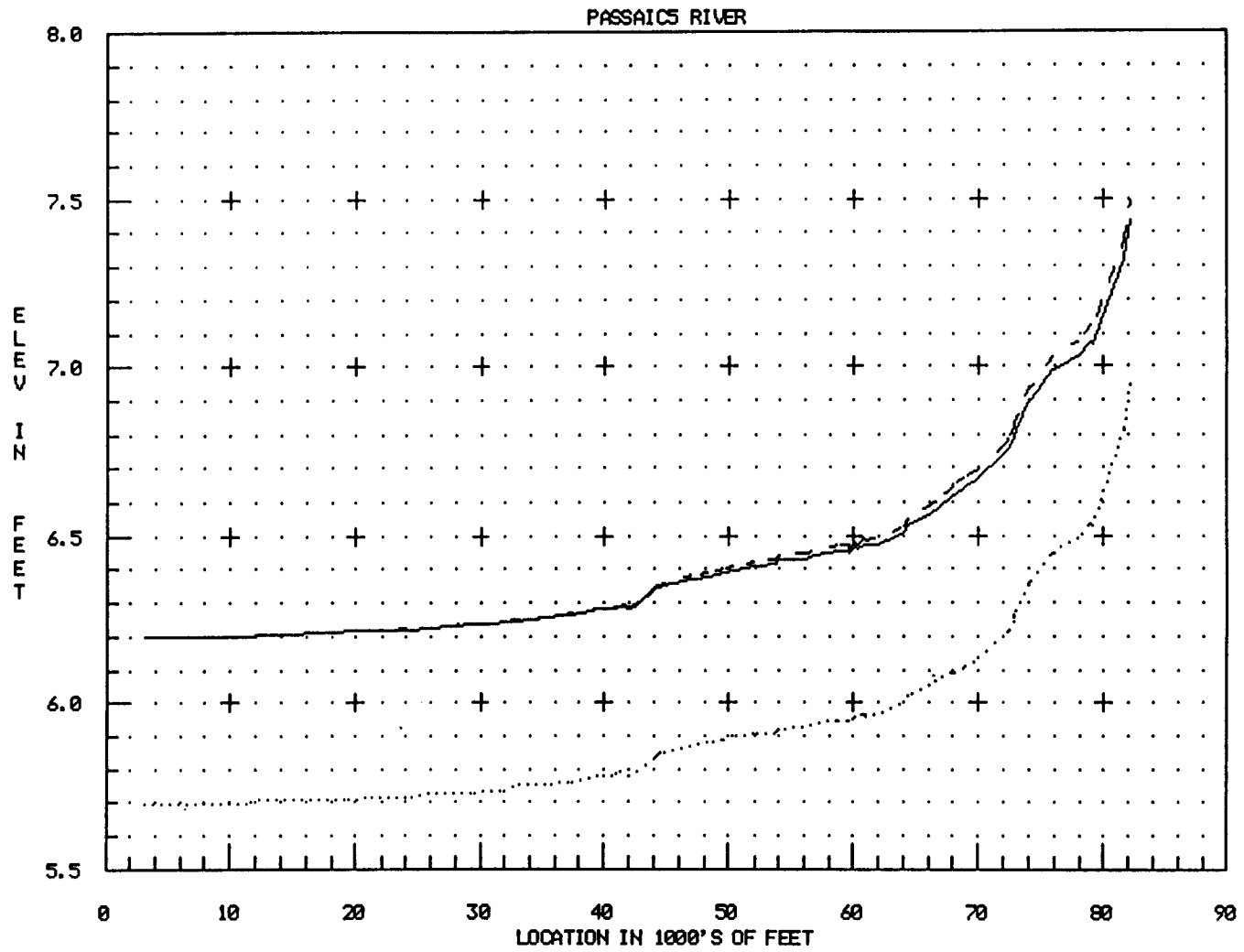


Figure C-109ii

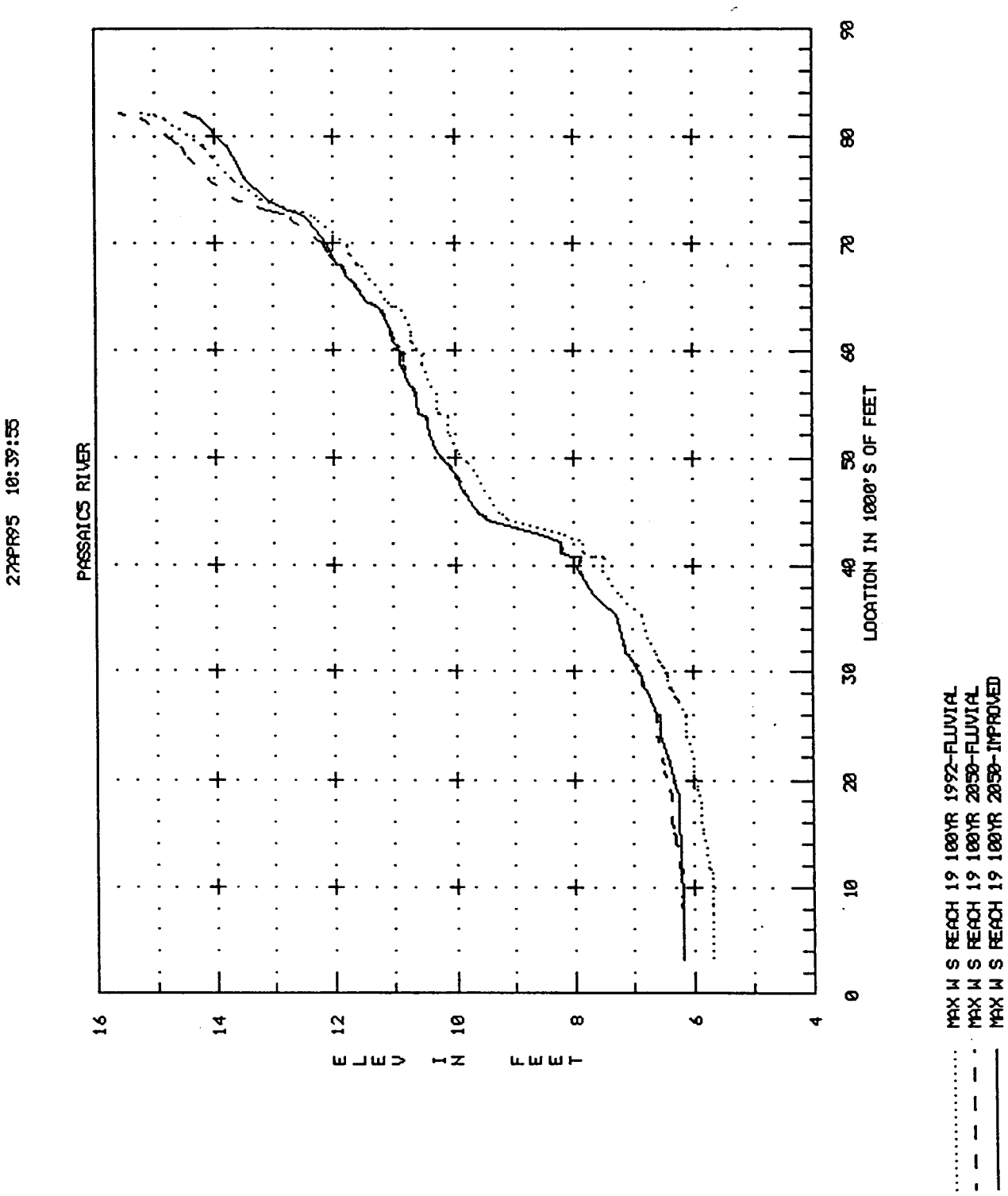
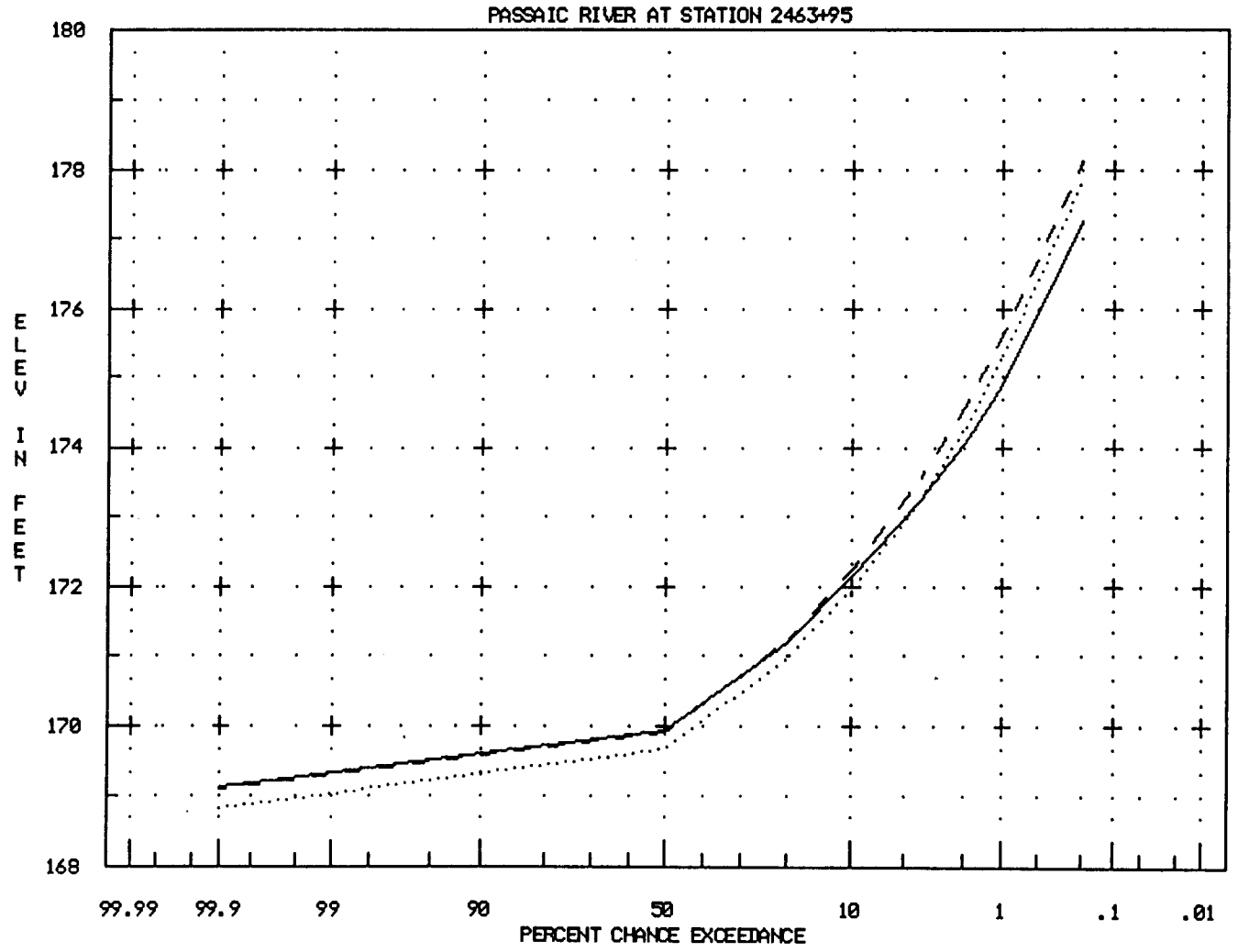


Figure C-109jj

08MAY95 07:02:30



020072 1992-FLUVIAL
020072 2050-FLUVIAL
020072 2050-IMPROVED

Figure C-110a

08MAY95 06:59:49

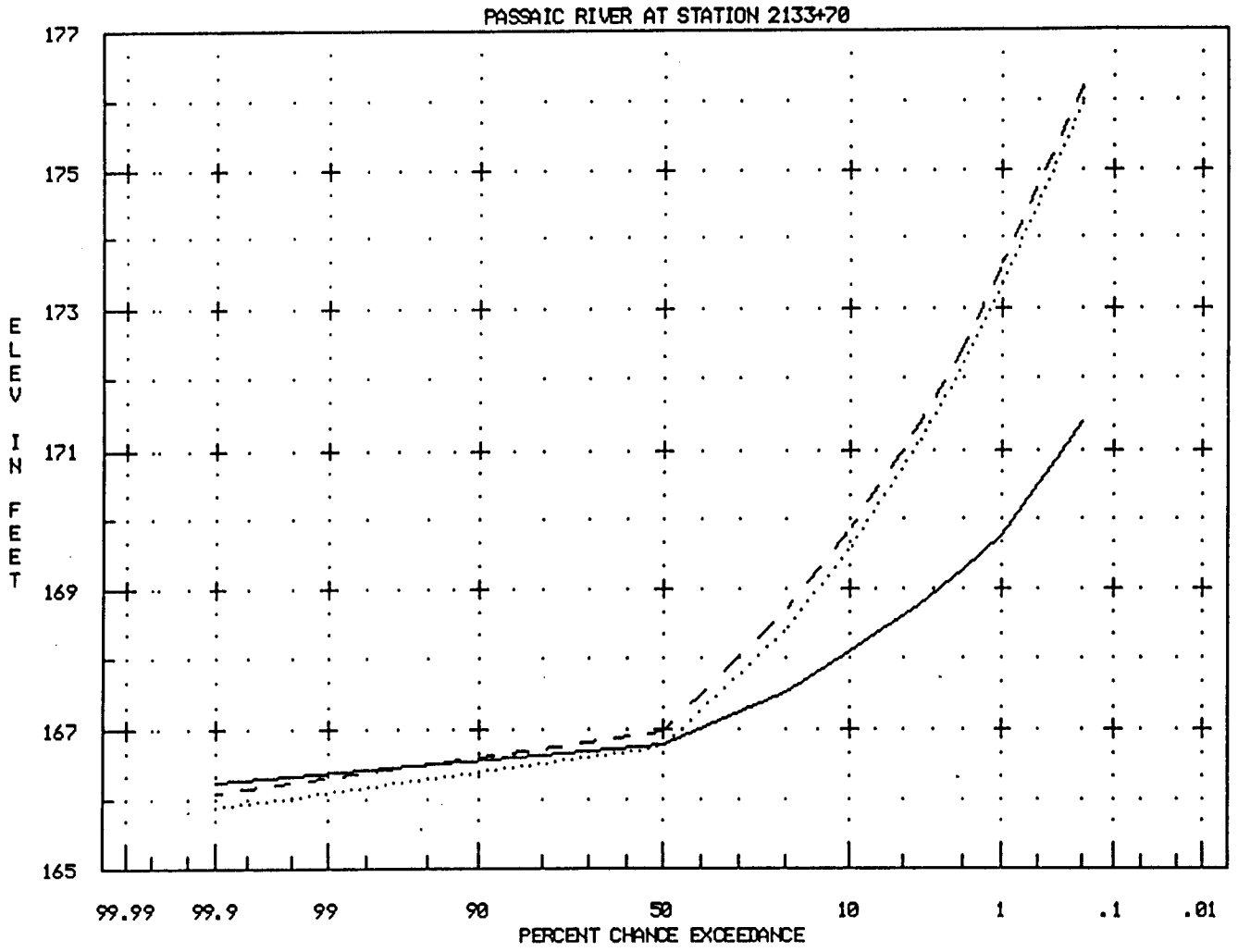
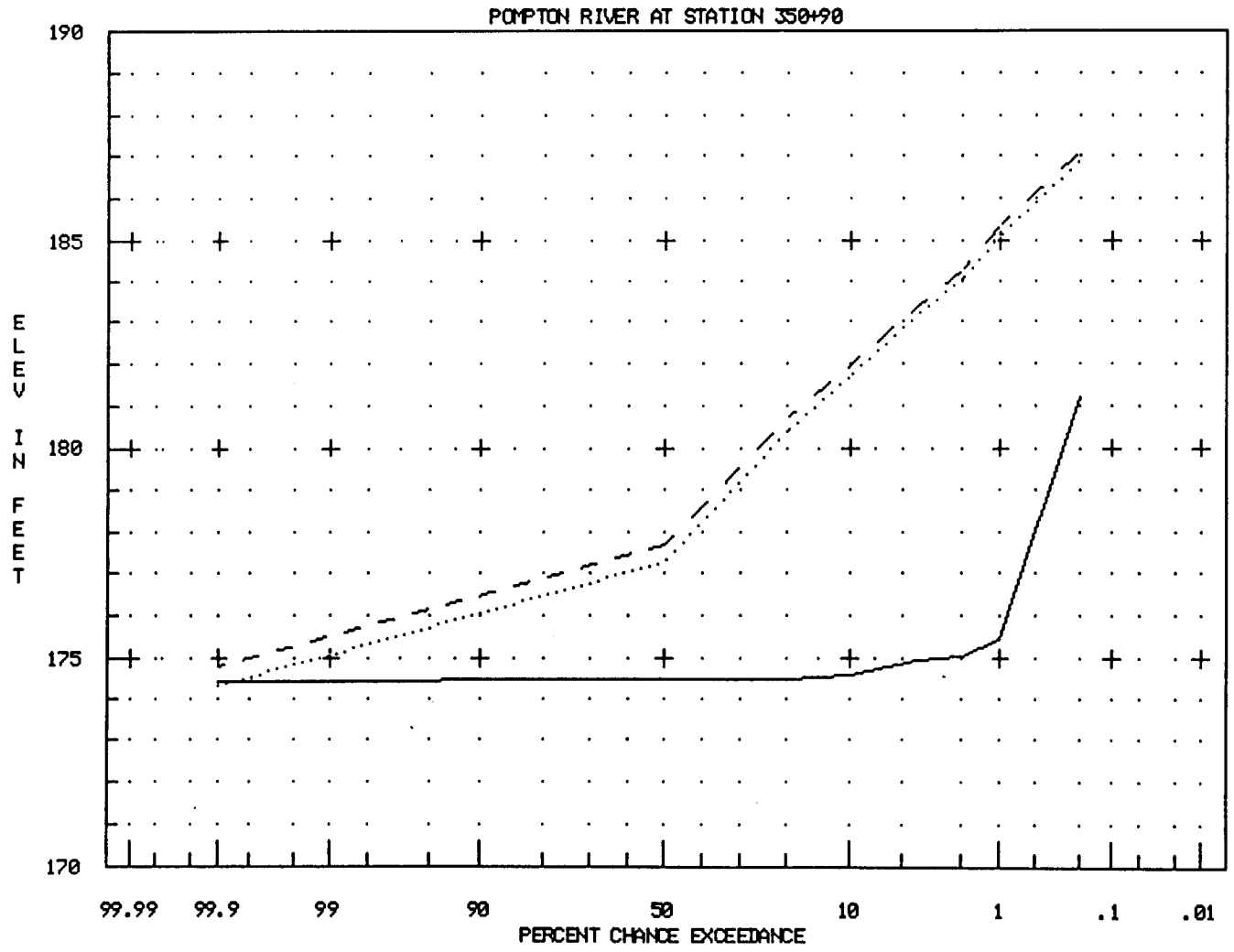


Figure C-110b

08MAY95 06:36:41



600061 1992-FLUVIAL
600061 2050-FLUVIAL
600061 2050-IMPROVED

Figure C-110c

08MAY95 07:59:30

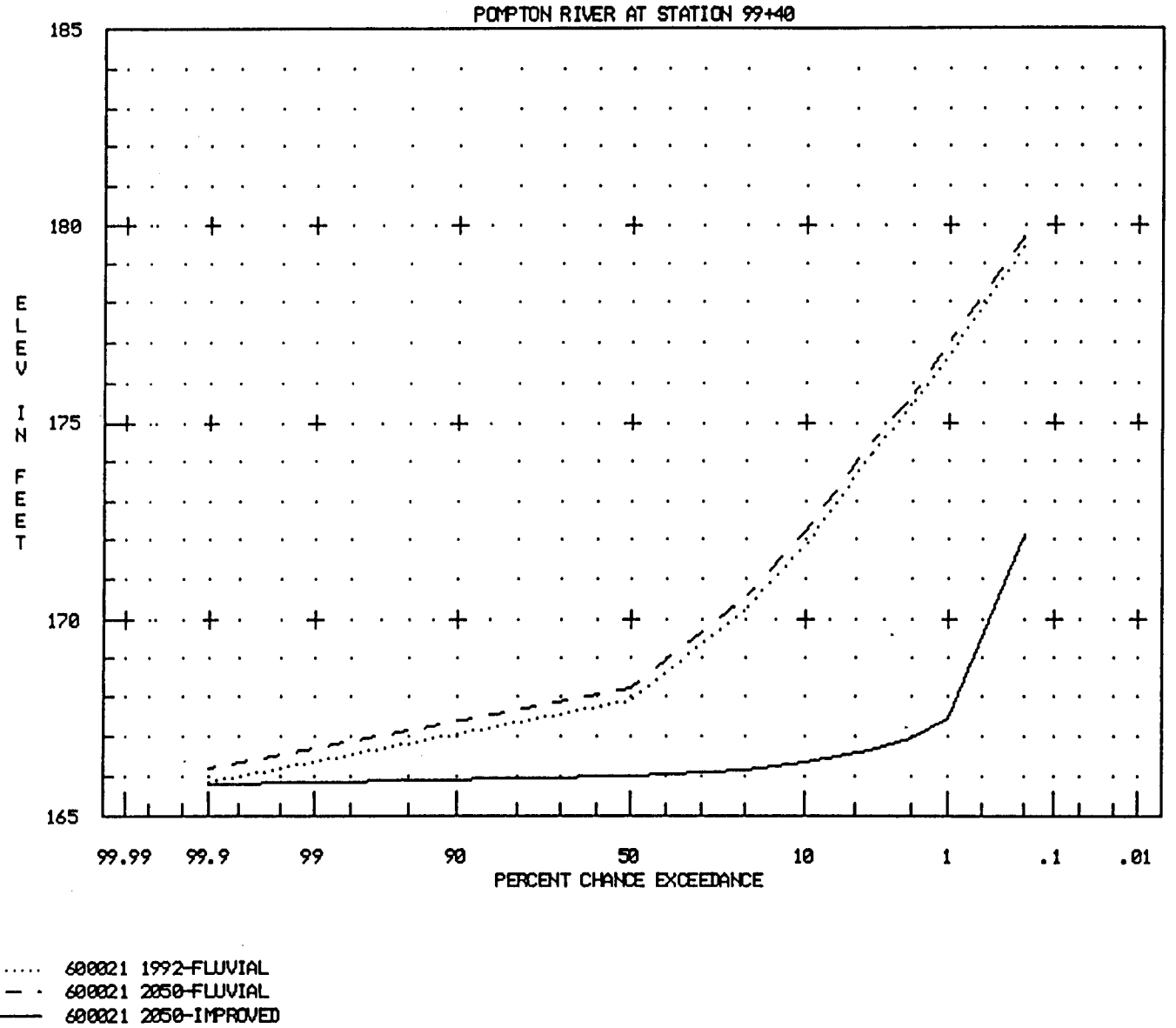


Figure C-110d

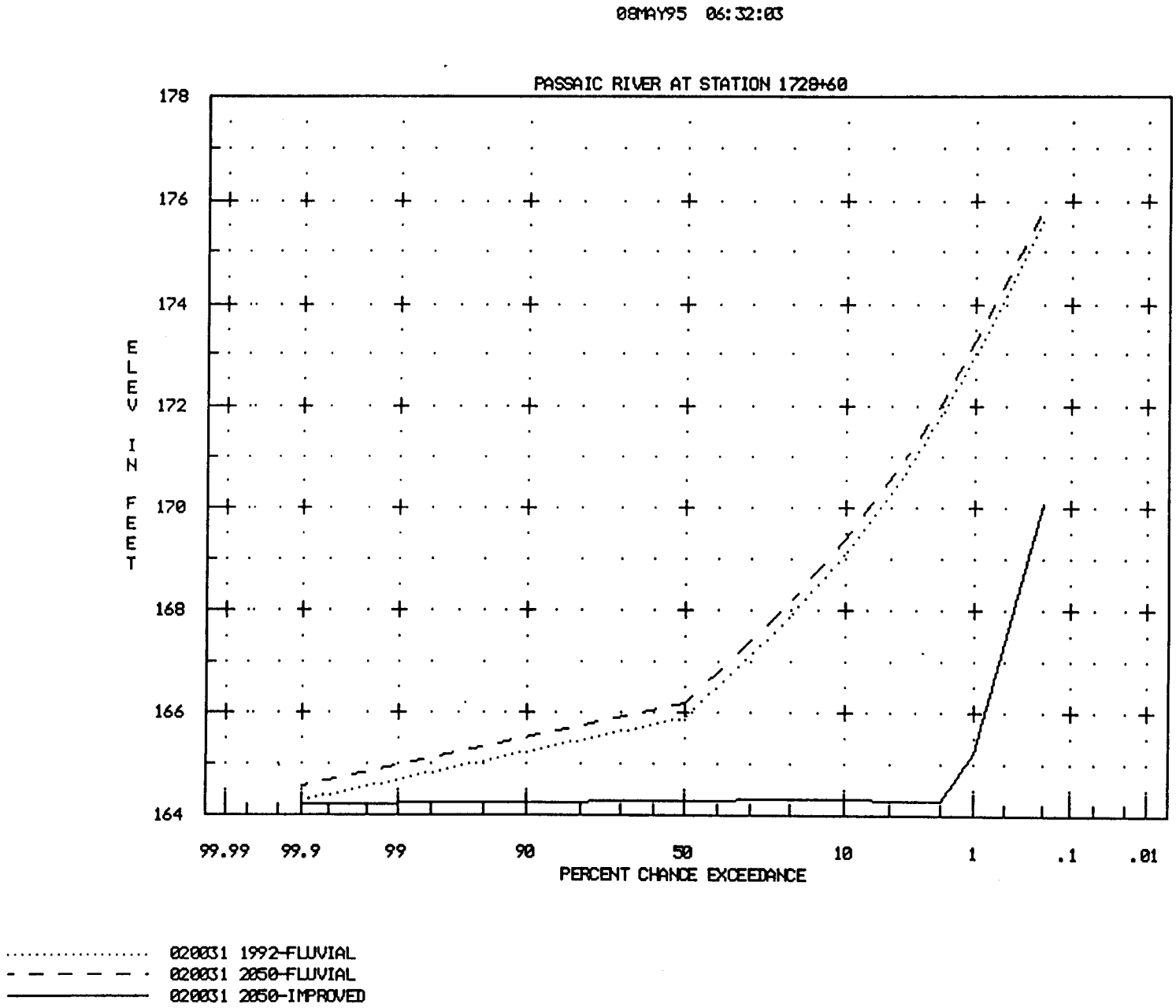


Figure C-110e

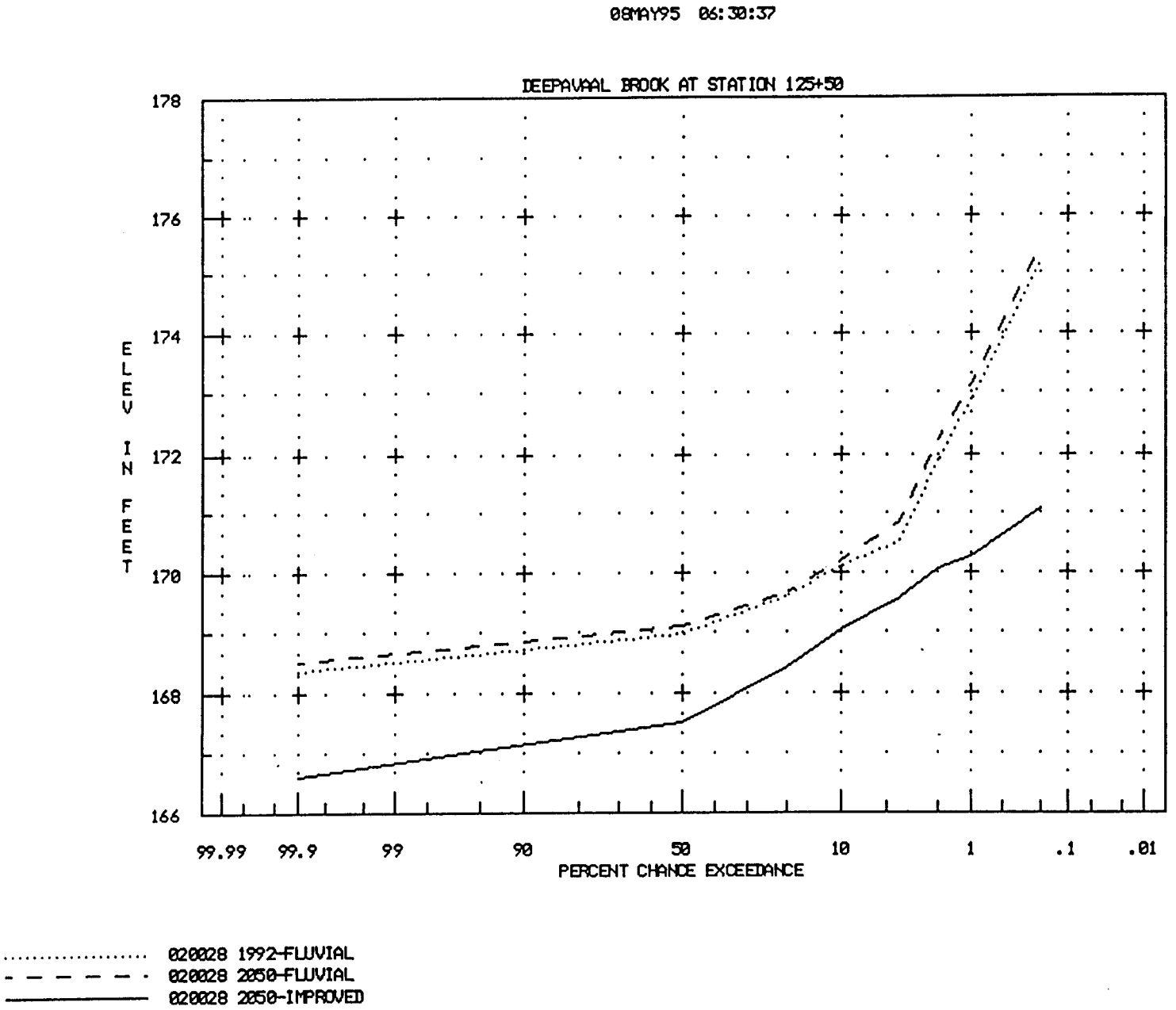


Figure C-110f

08MAY95 06:29:49

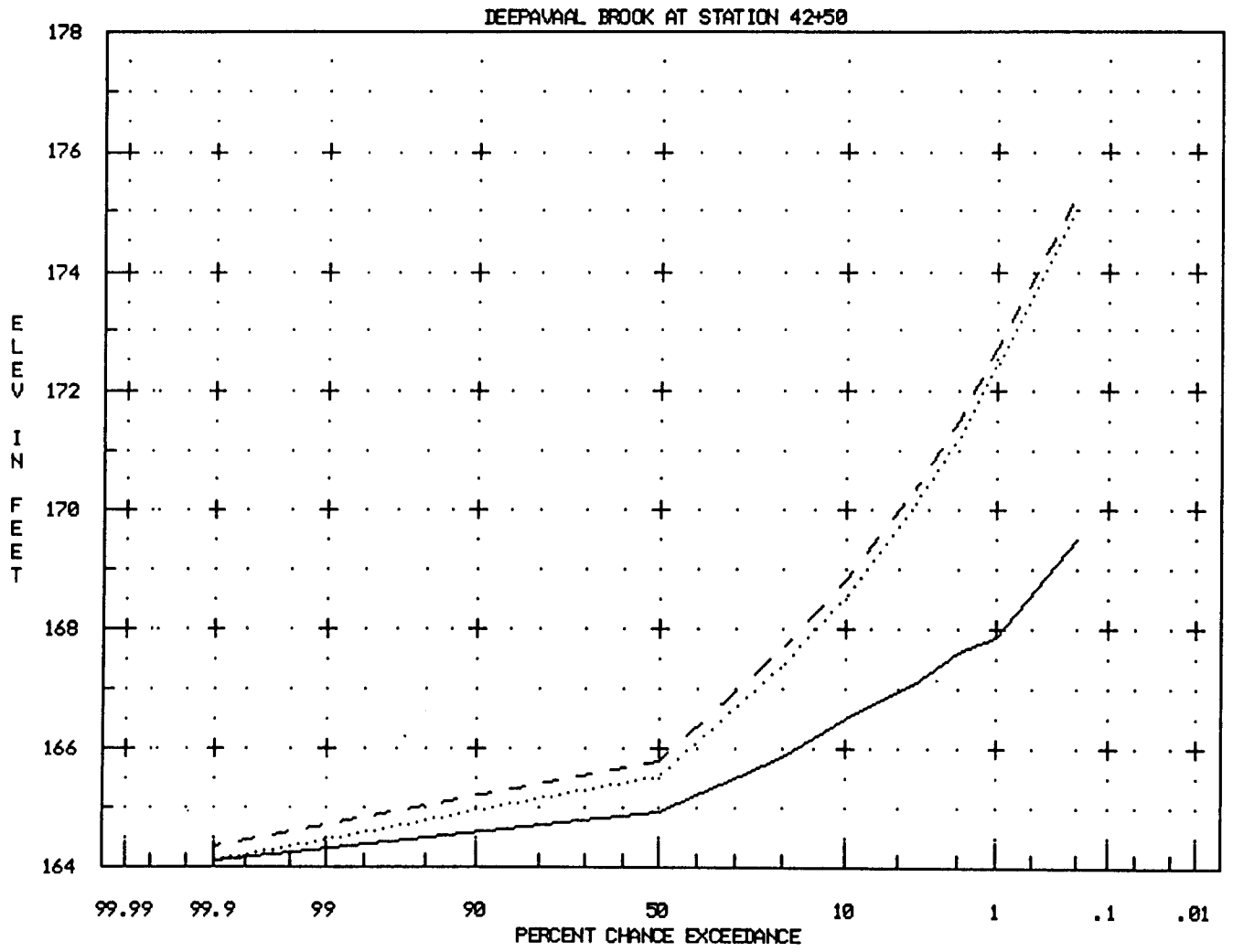


Figure C-110g

08MAY95 06:29:16

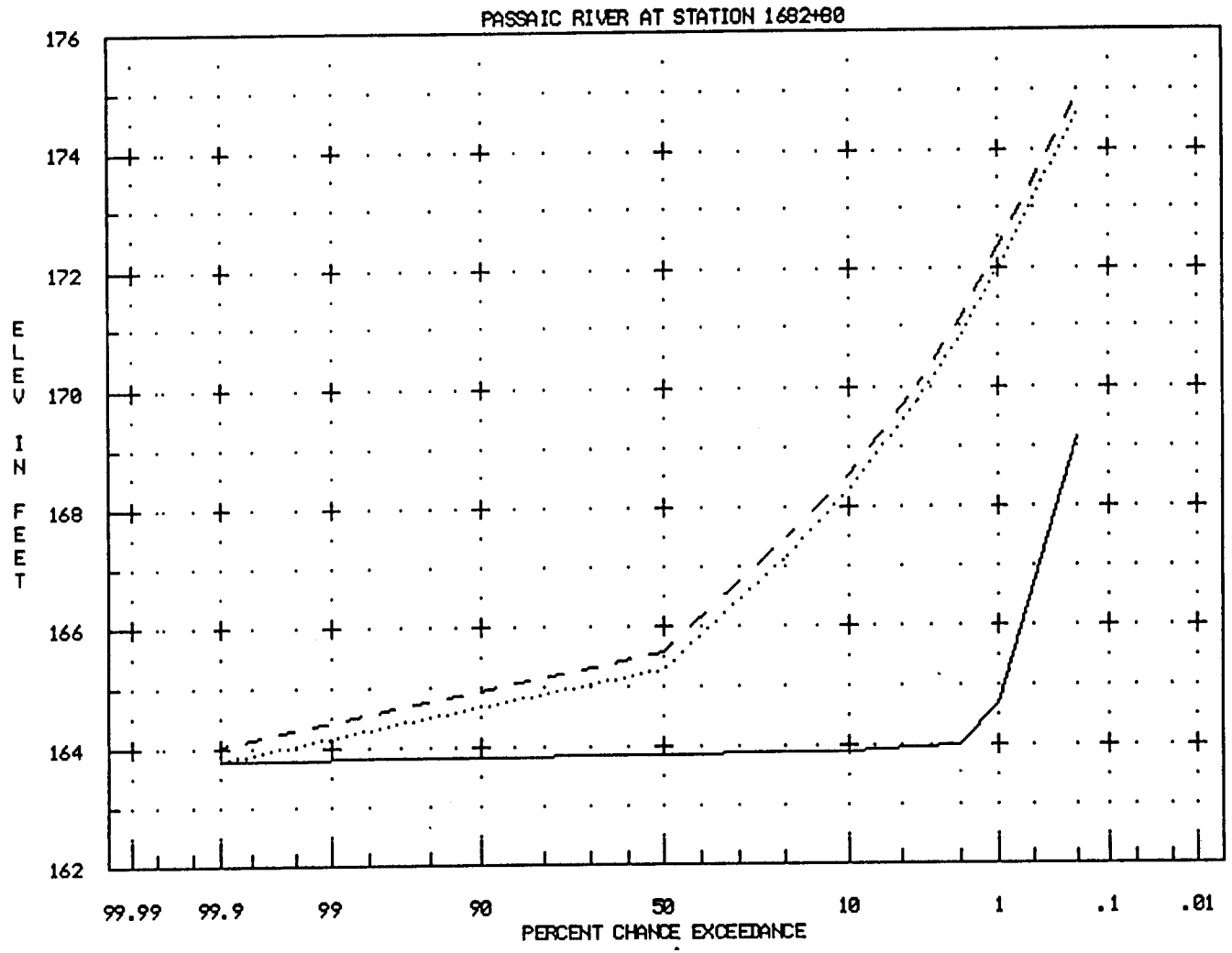


Figure C-110h

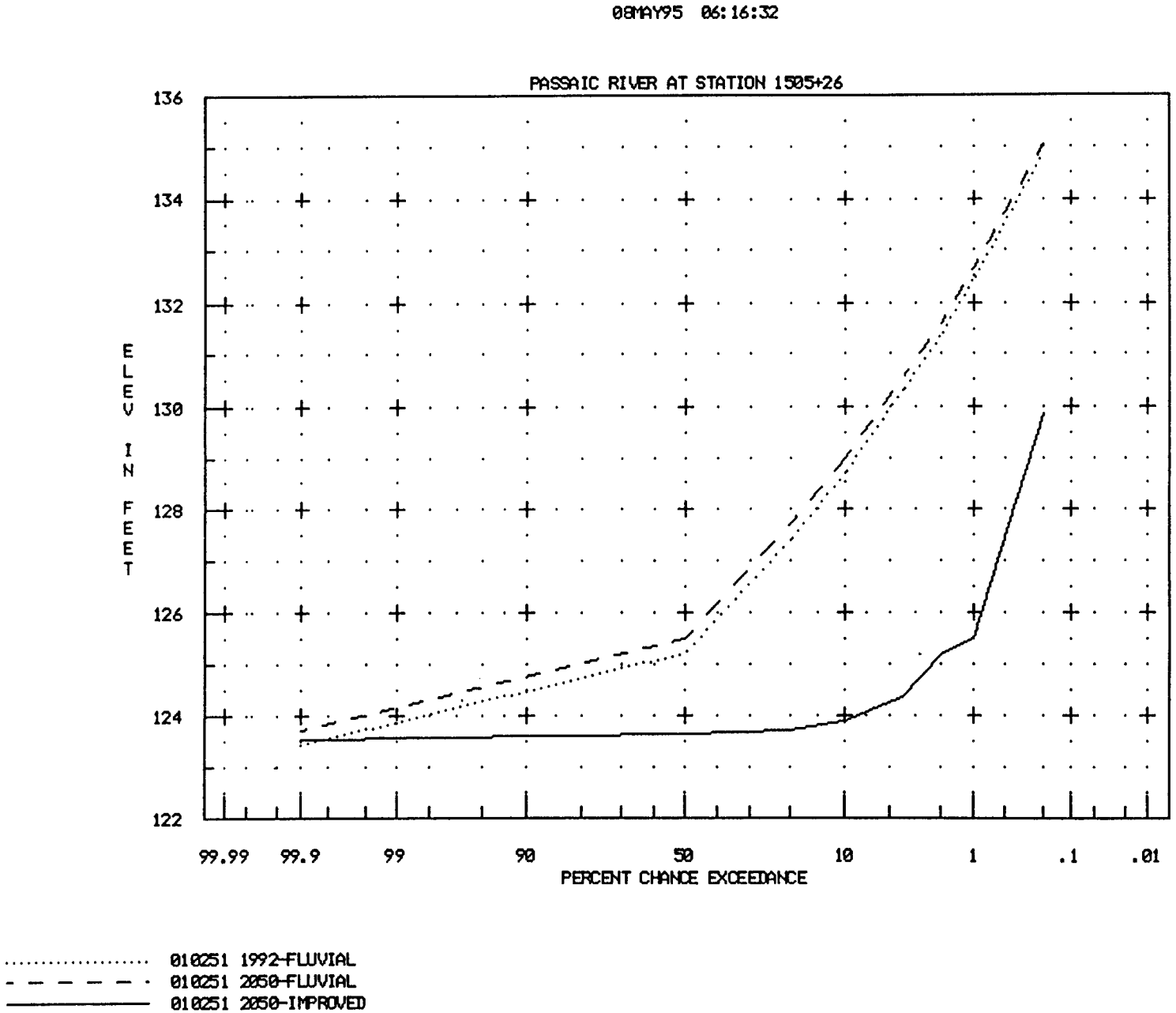


Figure C-1101

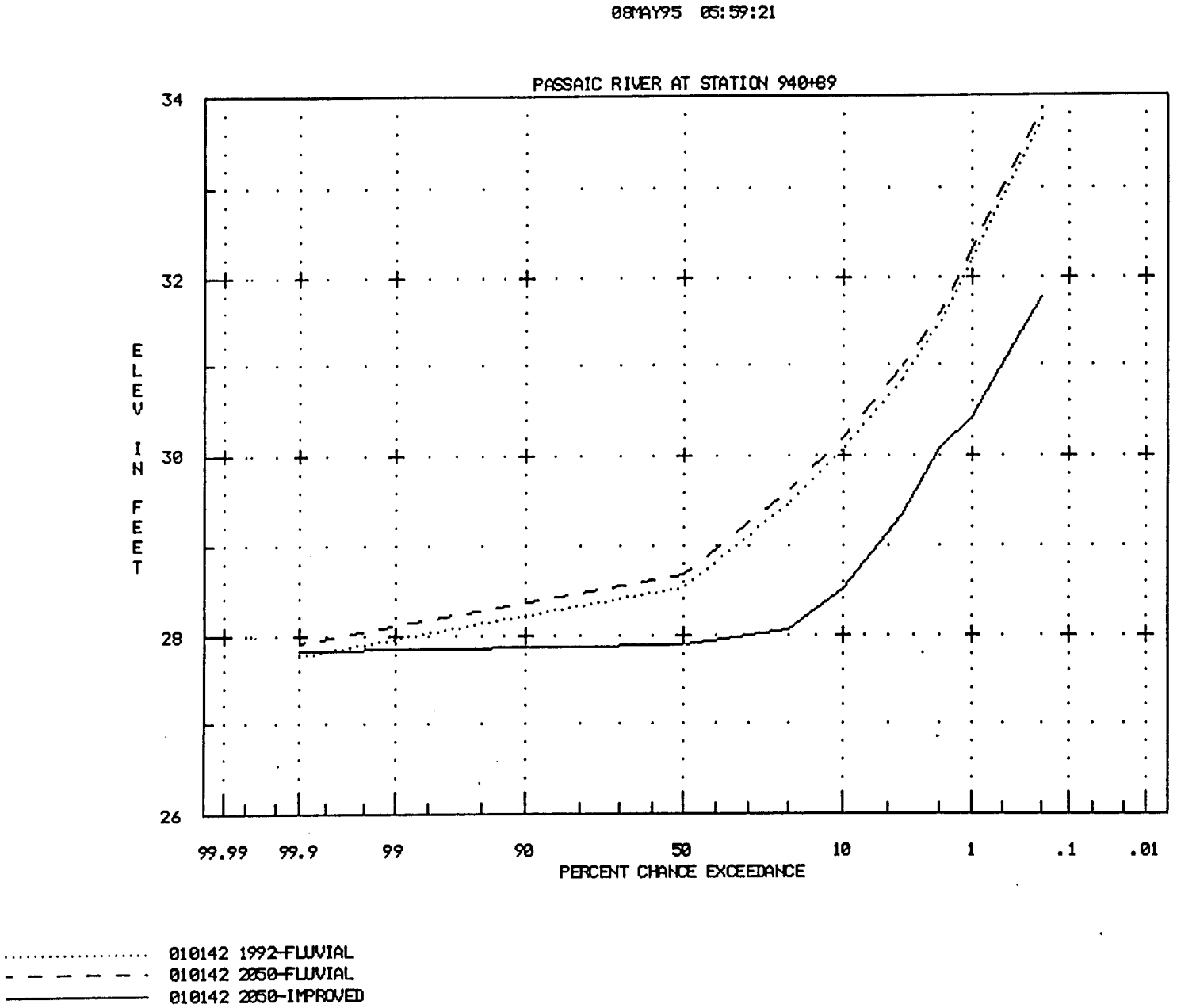
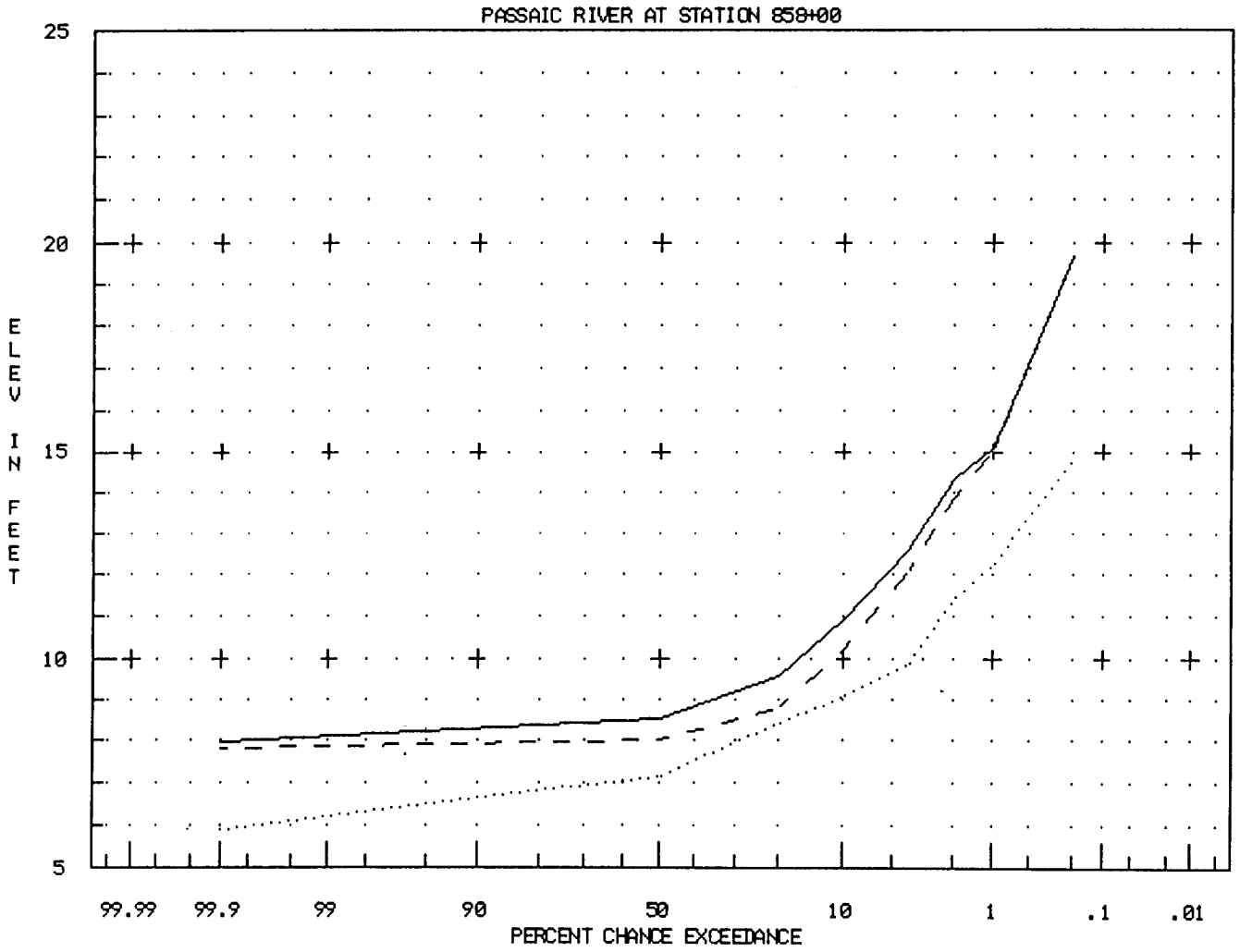


Figure C-110j

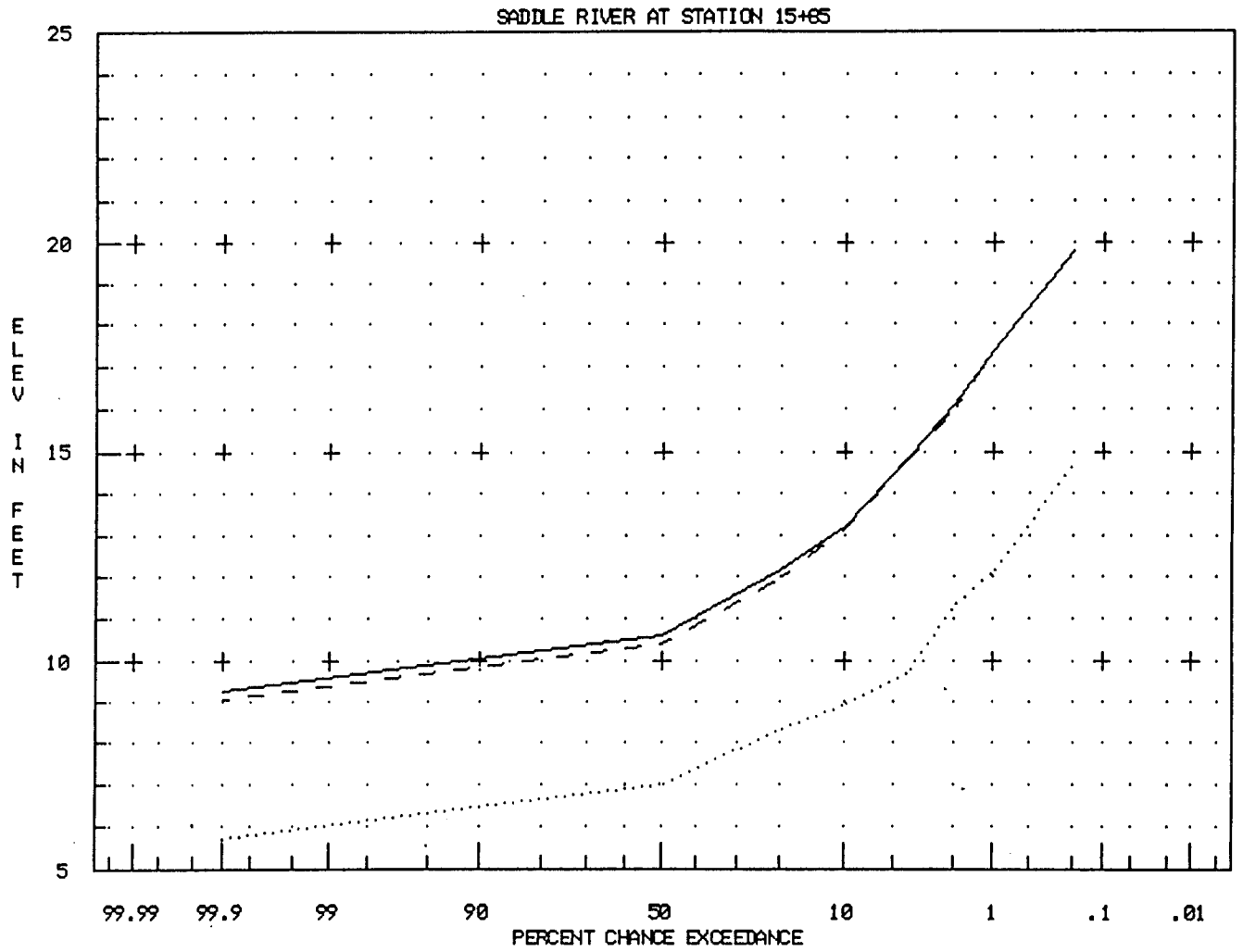
09MAY95 10:42:37



010121 2050-STORM SURGE
010121 2050-FLUVIAL-IMP
010121 2050-COMBINED-IMP

Figure C-110k

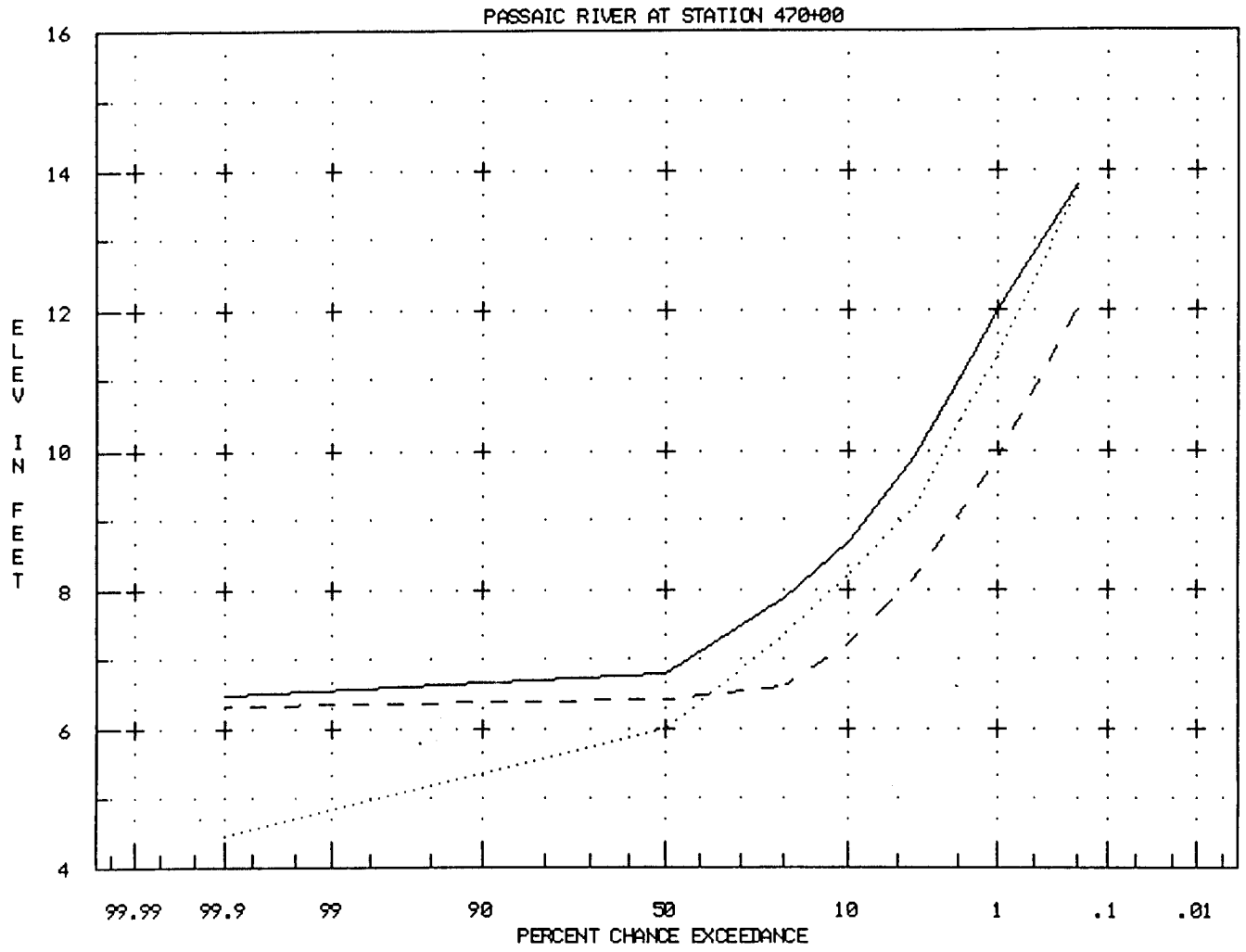
09MAY95 09:42:50



010105 2050-STORM SURGE
010105 2050-FLUVIAL-IMP
010105 2050-COMBINED-IMP

Figure C-1101

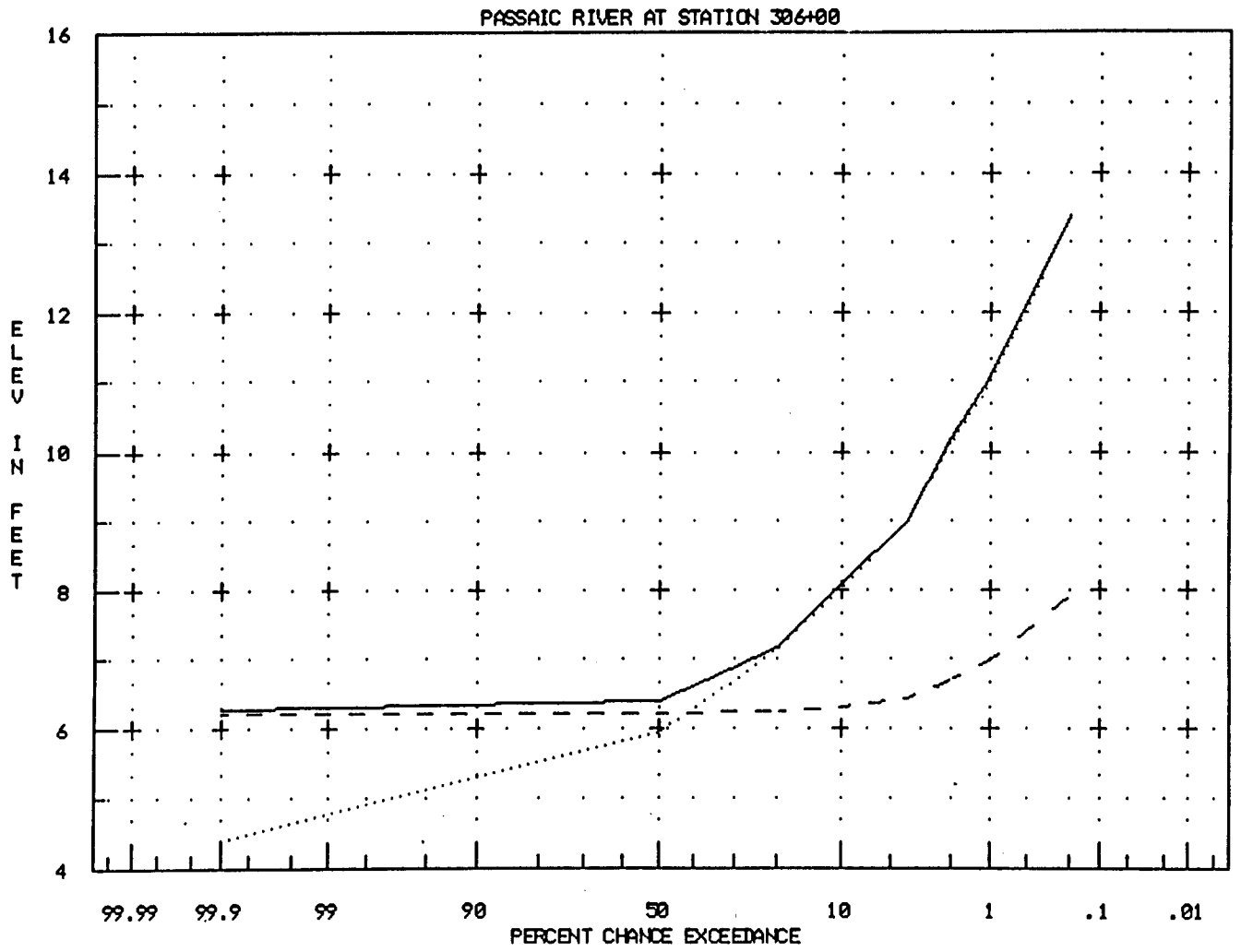
09MAY95 09:10:58



010042 2050-STORM SURGE
010042 2050-FLUVIAL-IMP
010042 2050-COMBINED-IMP

Figure C-110m

09MAY95 09:03:37



010021 2050-STORM SURGE
010021 2050-FLUVIAL-IMP
010021 2050-COMBINED-IMP

Figure C-110n

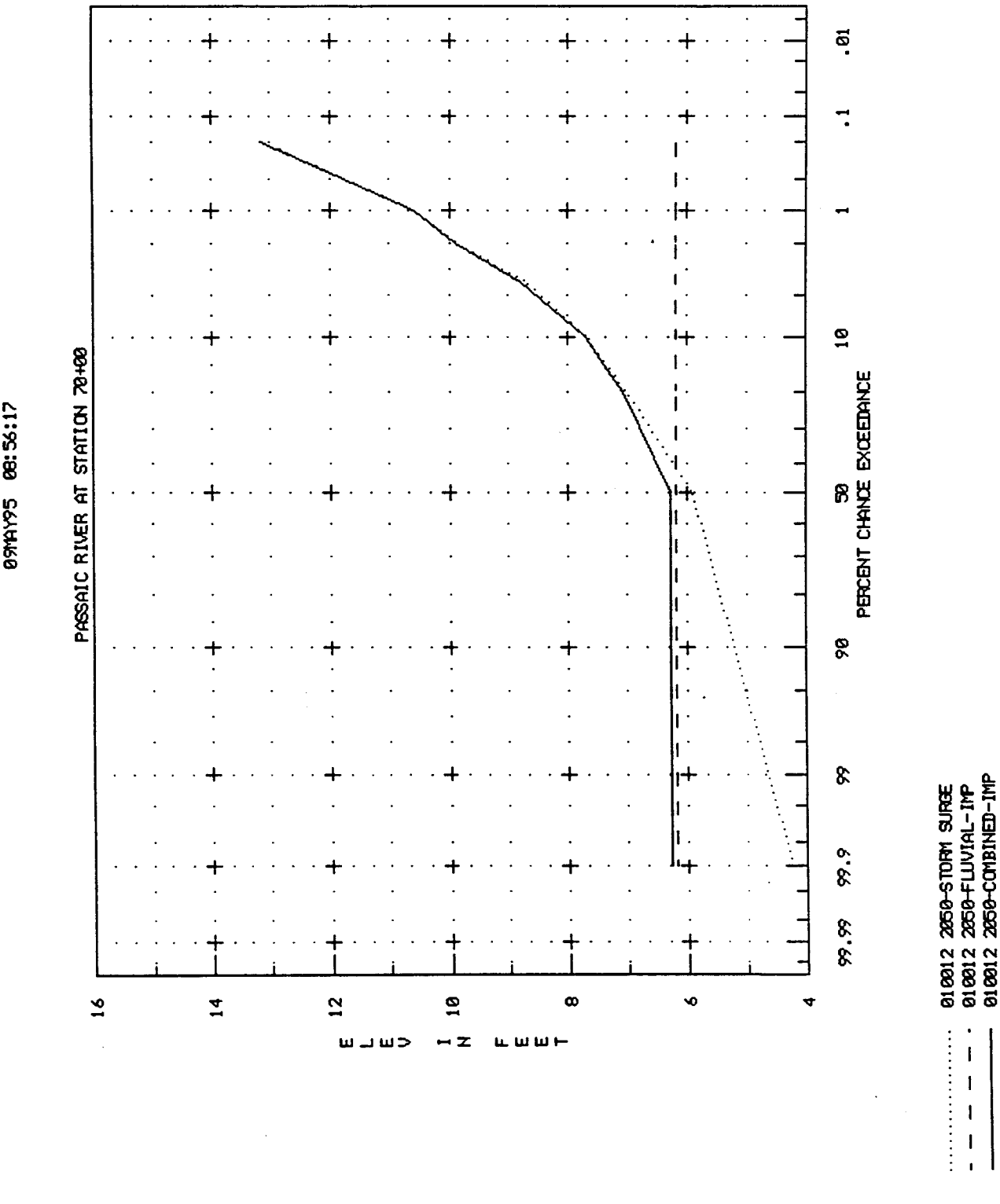


Figure C-110o

30AUG95 10:22:17

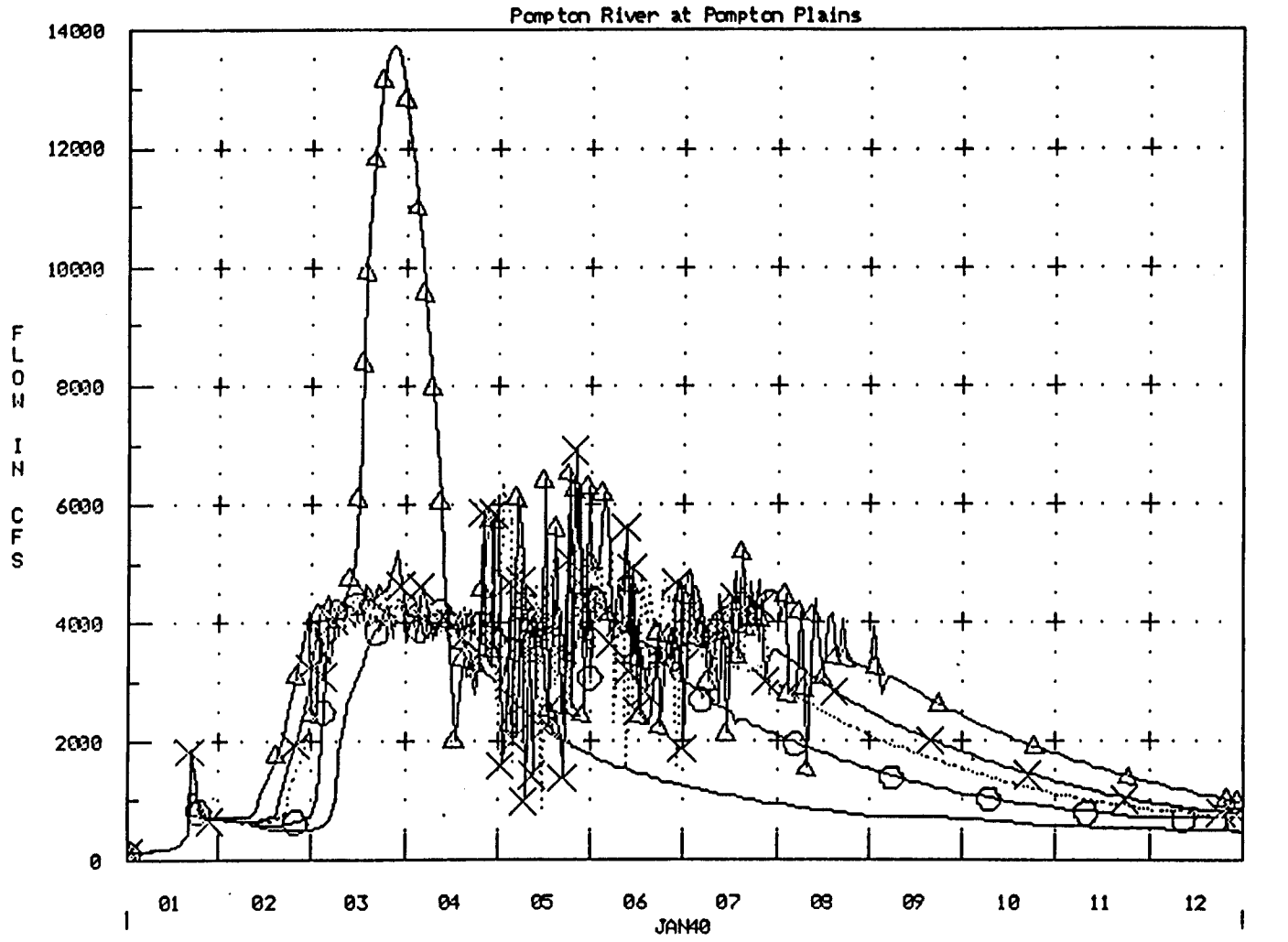


Figure C-111

28AUG95 10:13:06

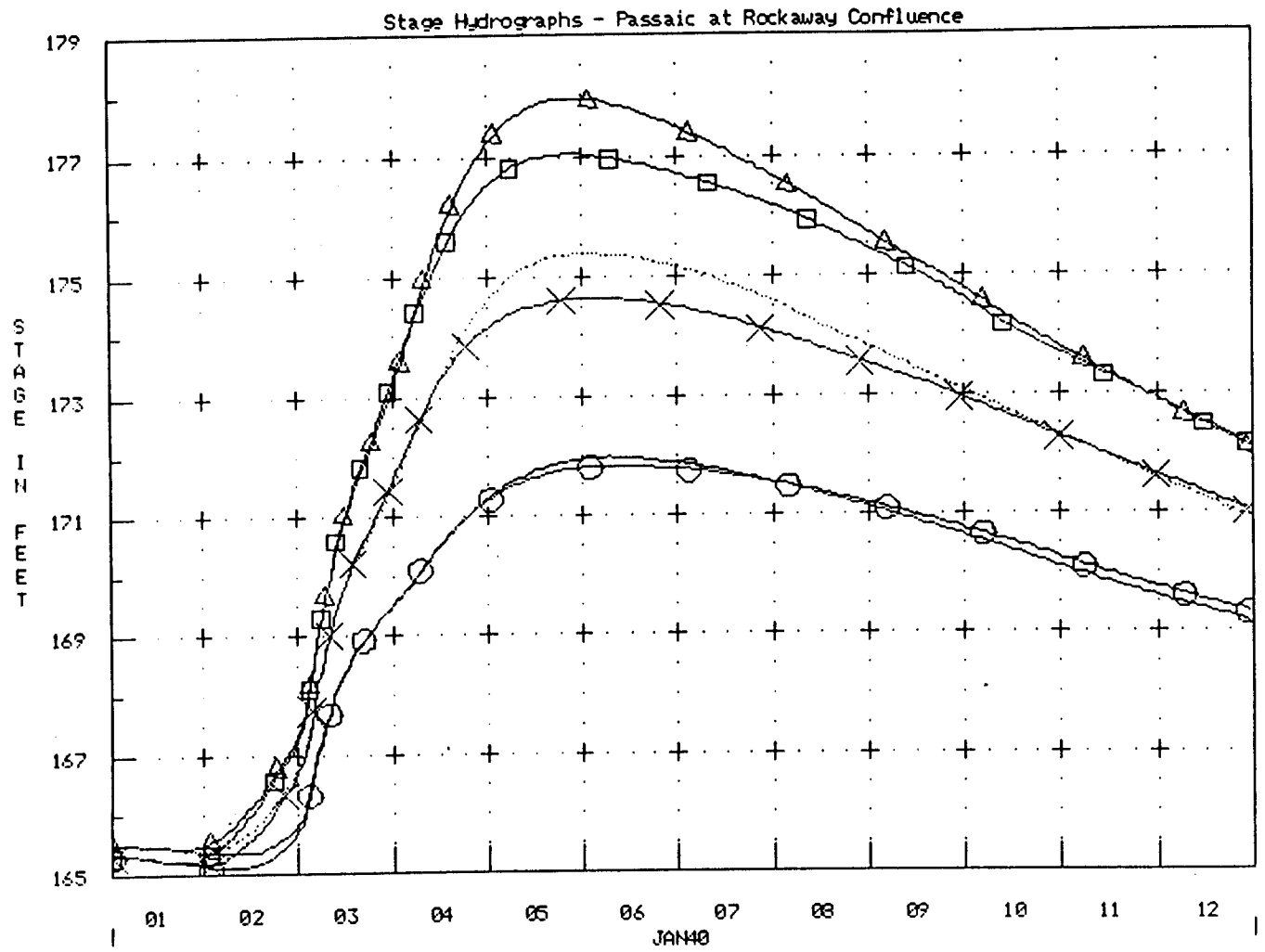


Figure C-112a

28AUG95 15:40

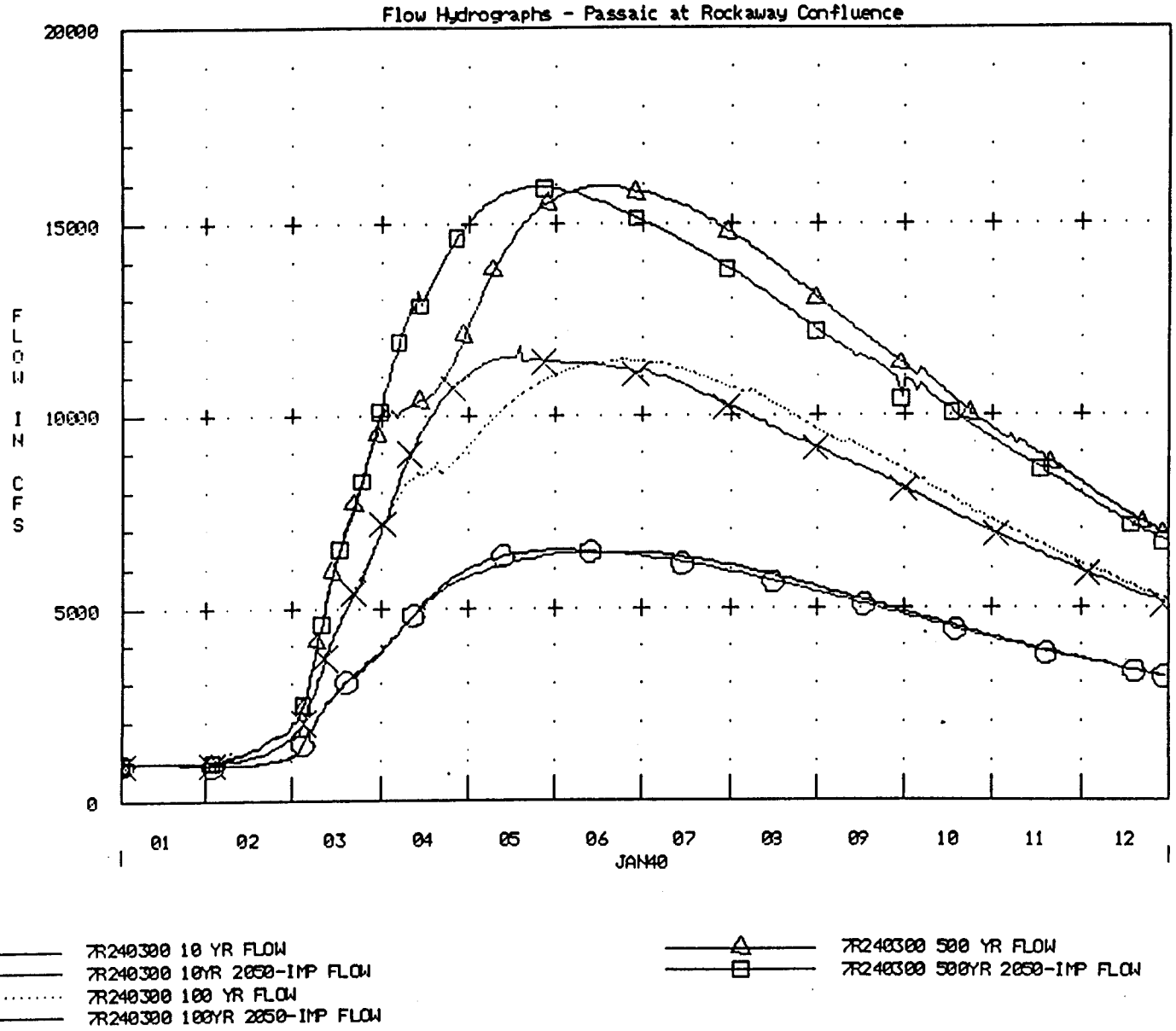


Figure C-112b

28AUG95 10:17:58

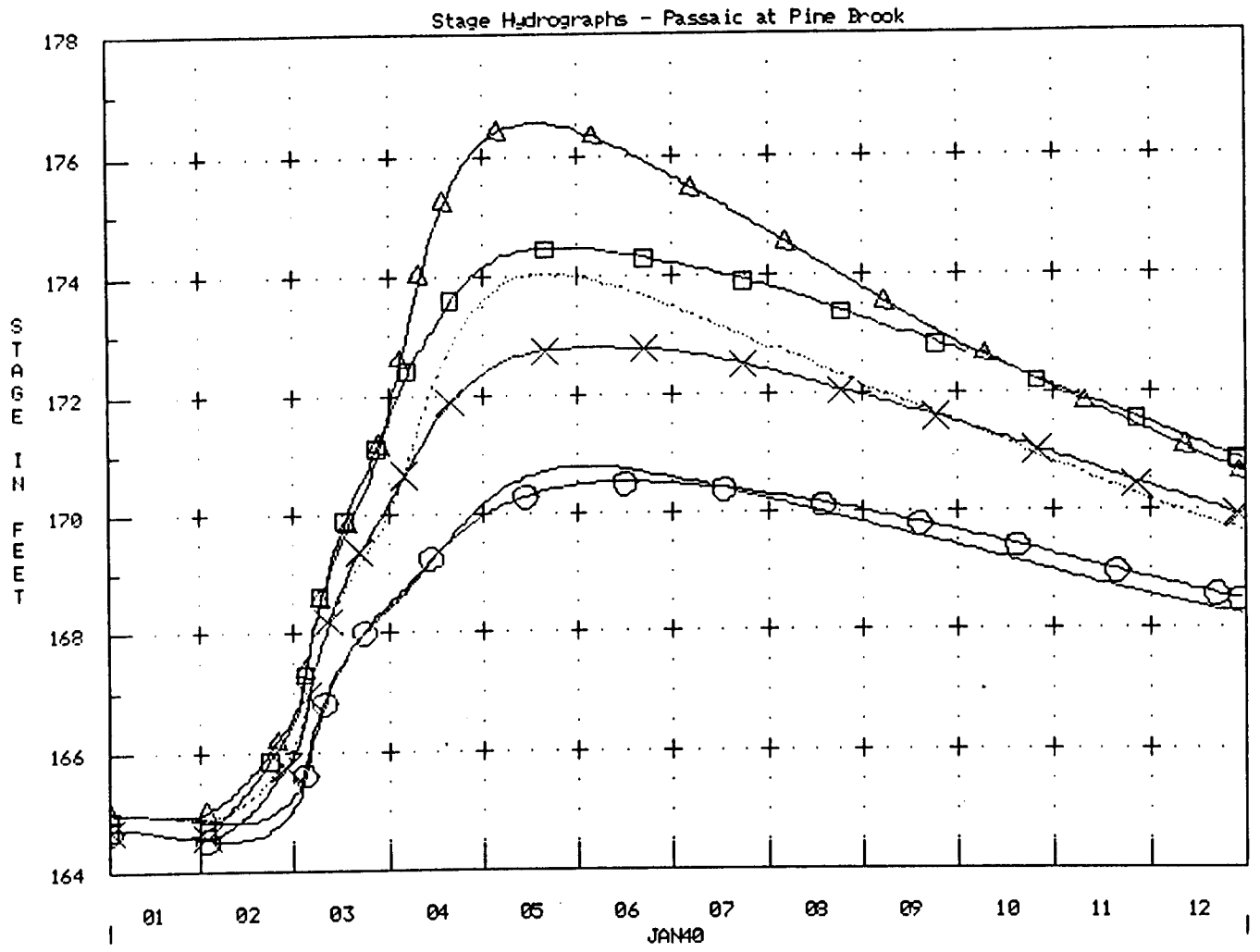


Figure C-112c

28AUG95 10:19:10

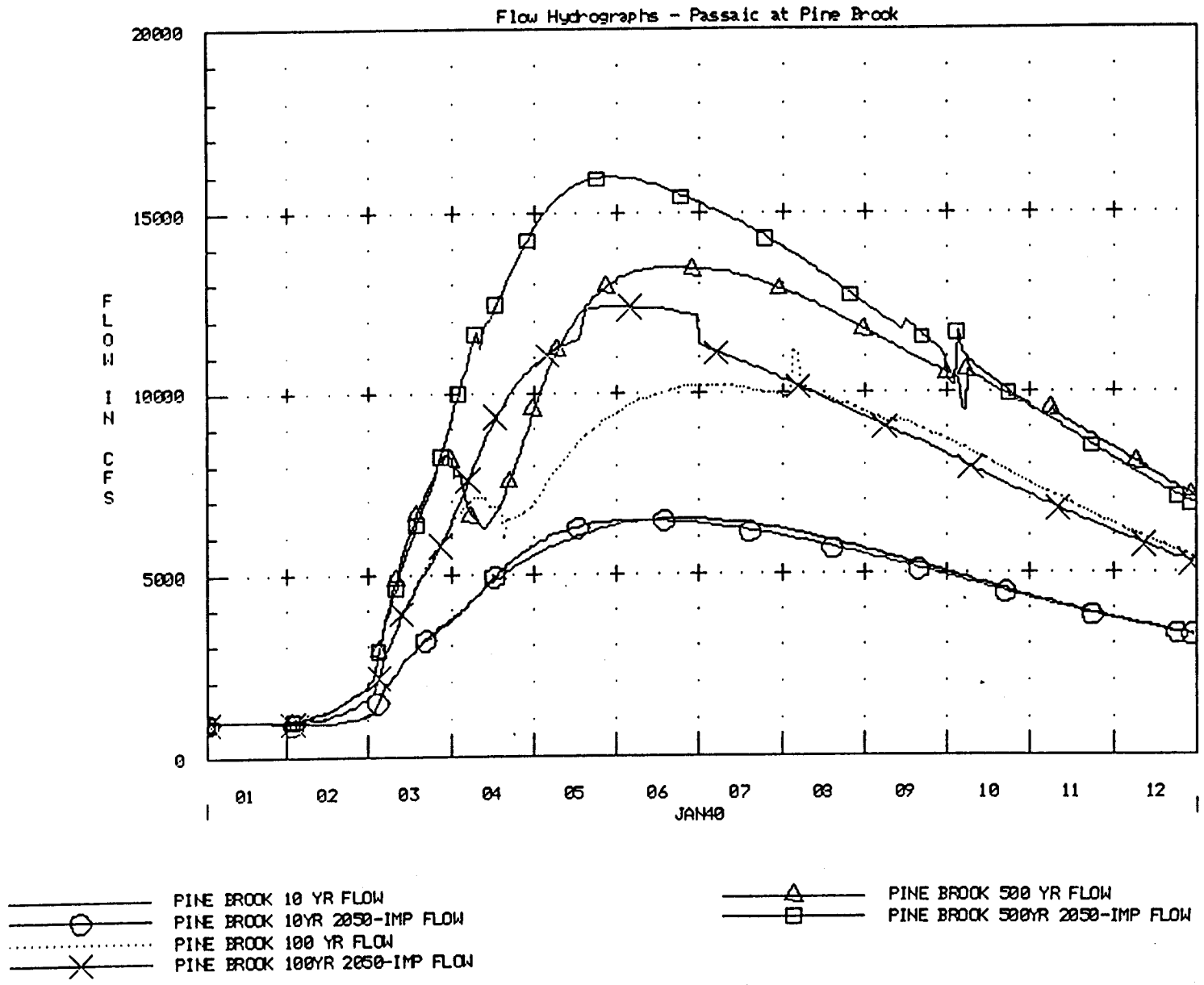


Figure C-112d

28AUG95 10:22:22

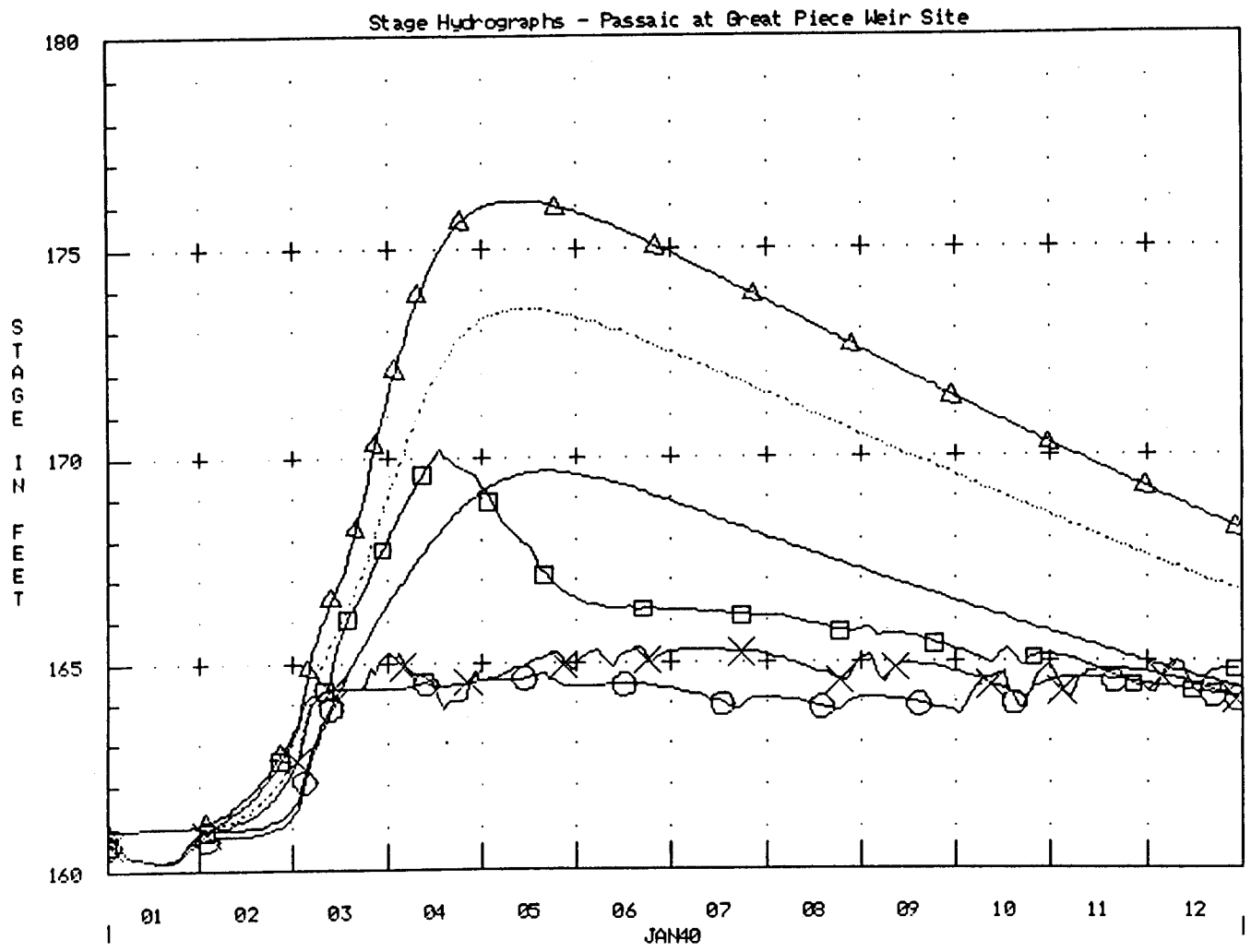


Figure C-112e

28AUG95 10:24:12

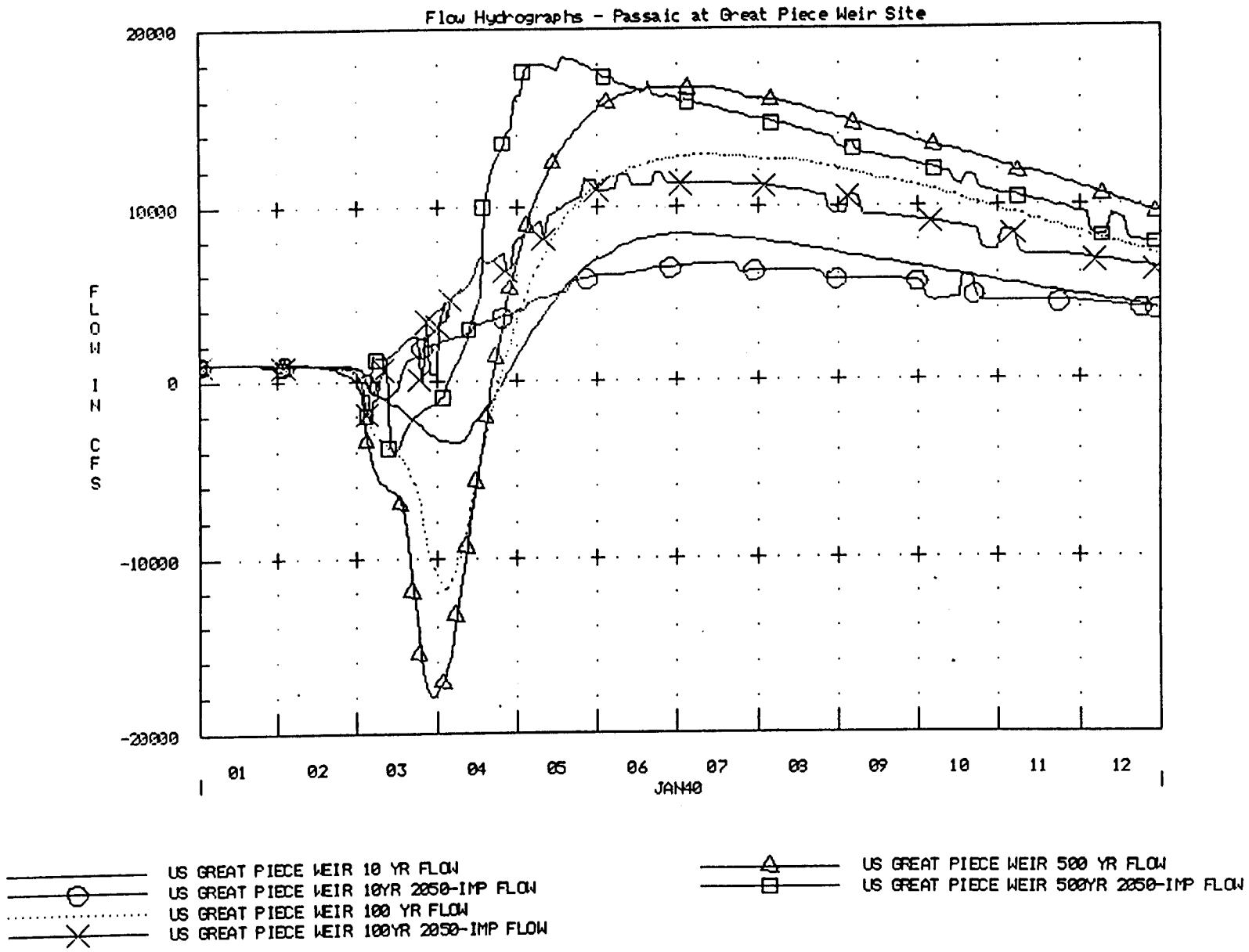


Figure C-112f

28AUG95 13:28:43

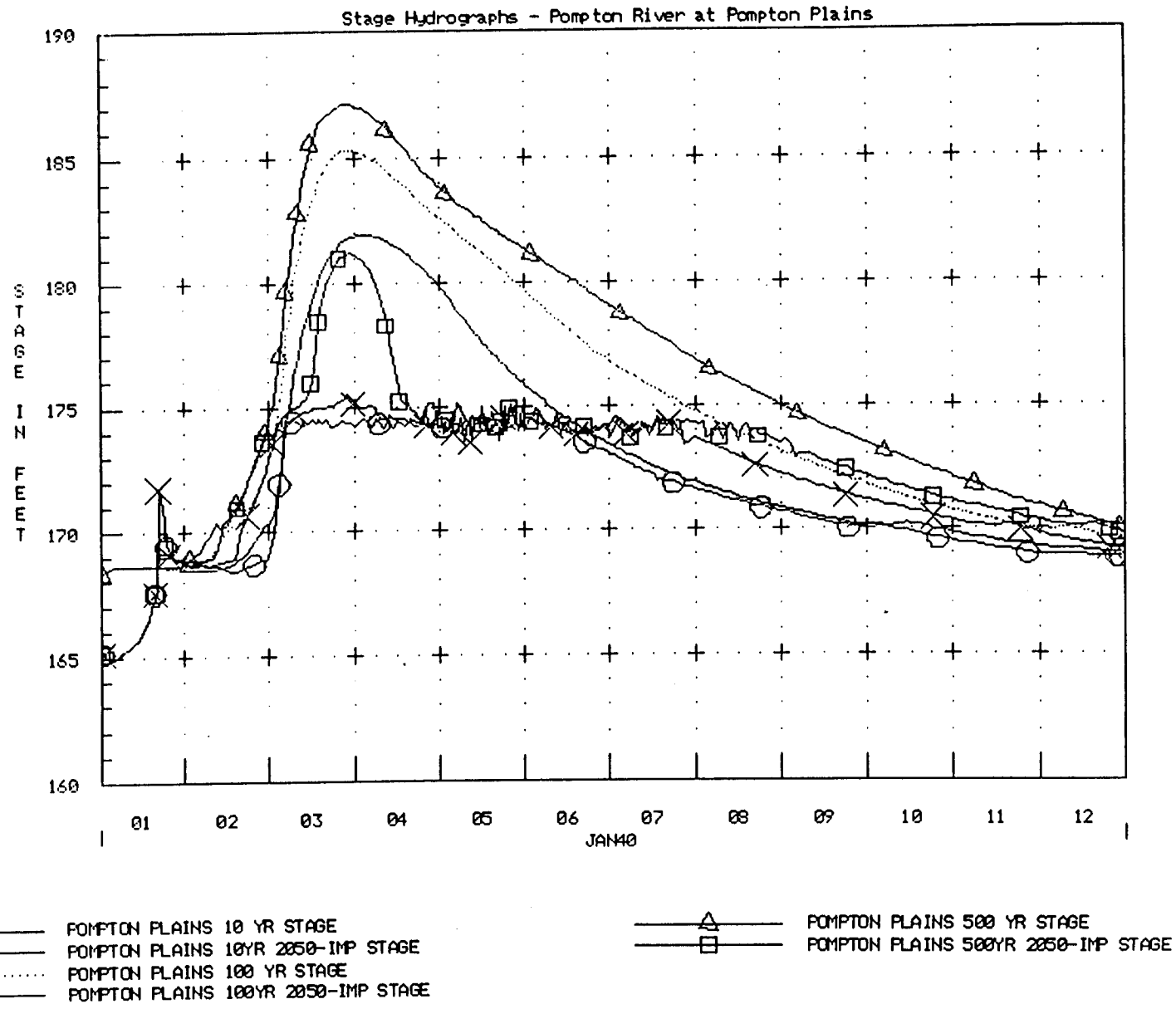


Figure C-112g

28AUG95 13:30:22

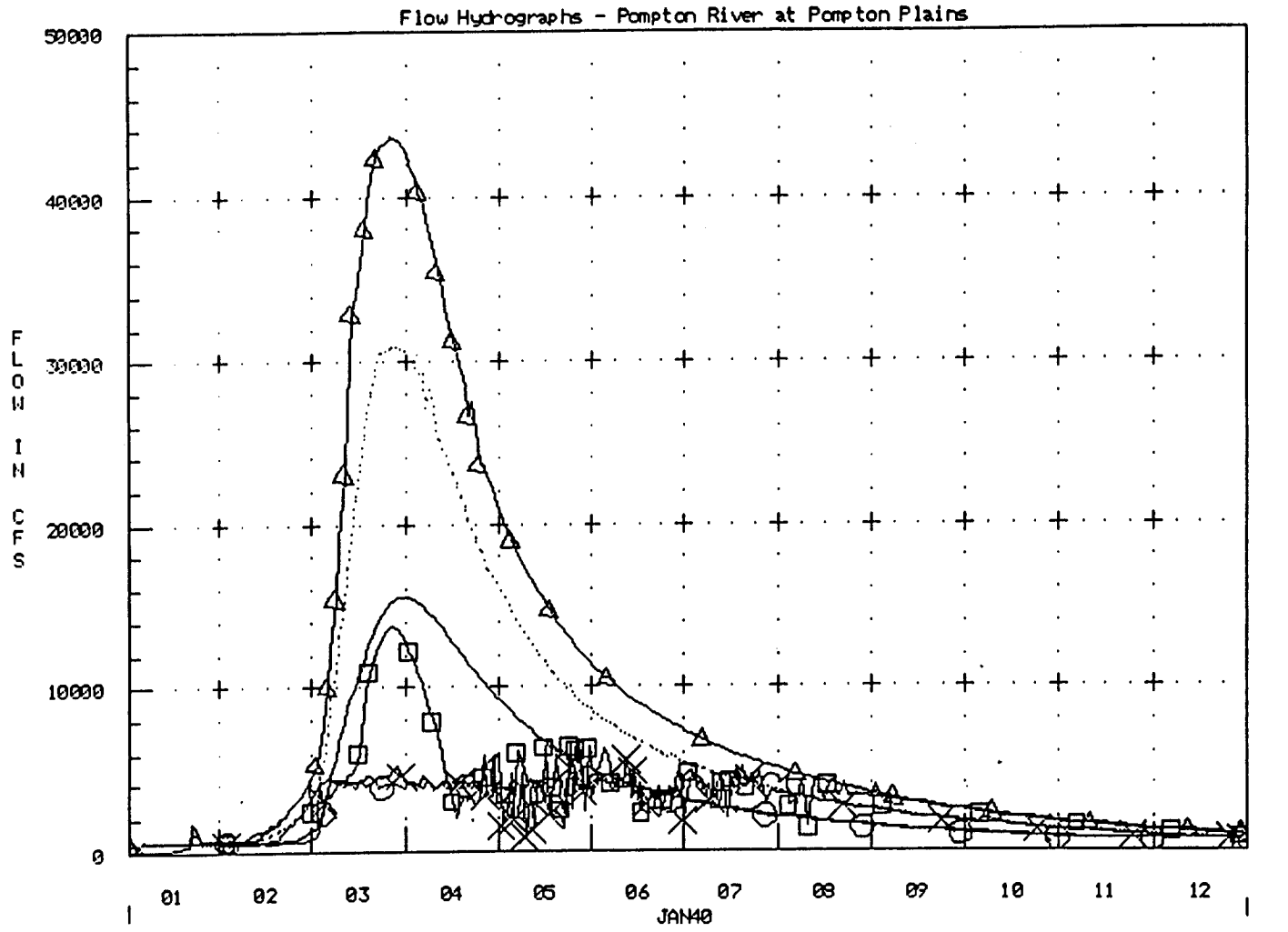


Figure C-112b

28AUG95 13:32:42

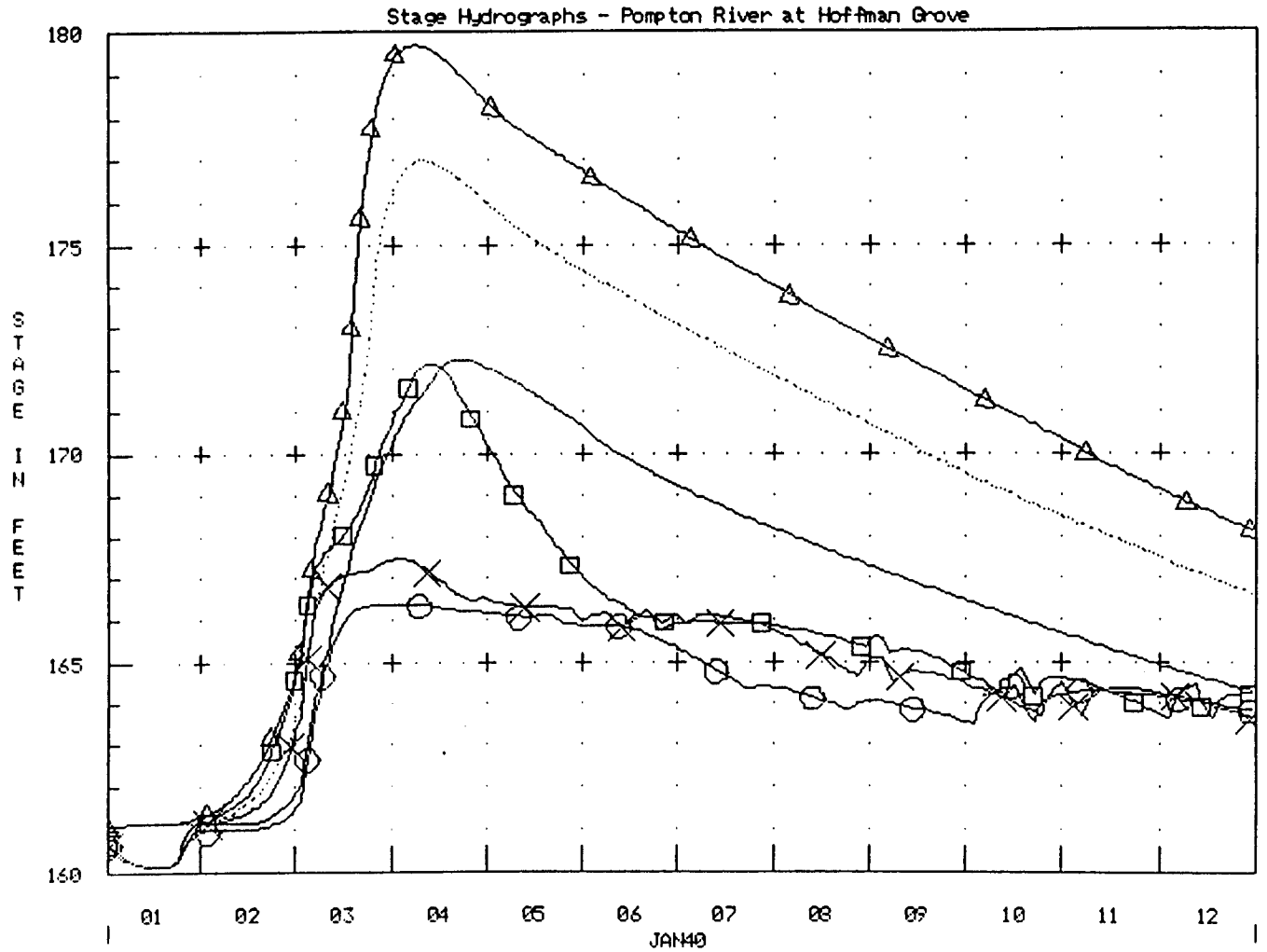


Figure C-1121

28AUG95 13:34:42

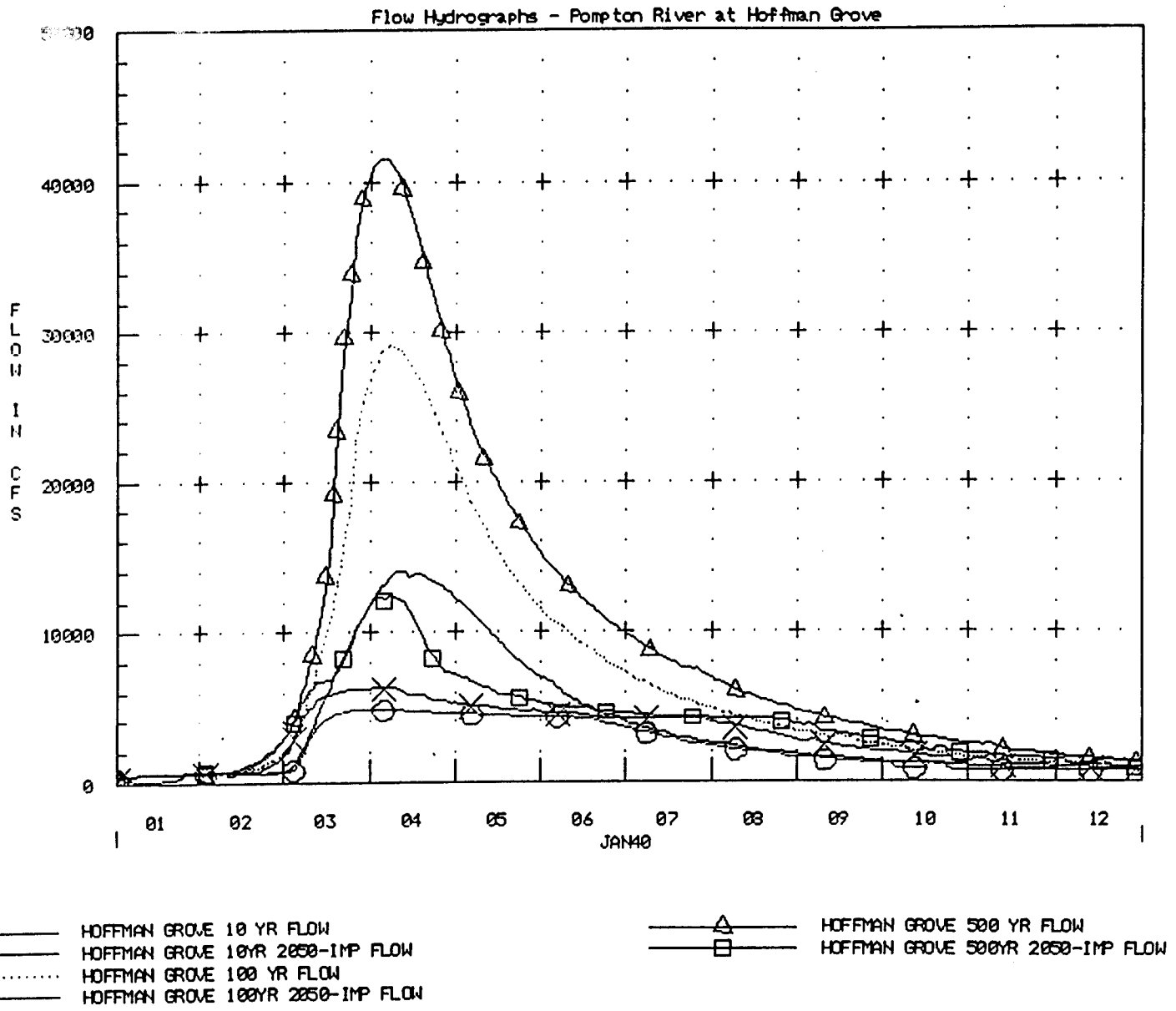


Figure C-112j

28AUG95 13:39:17

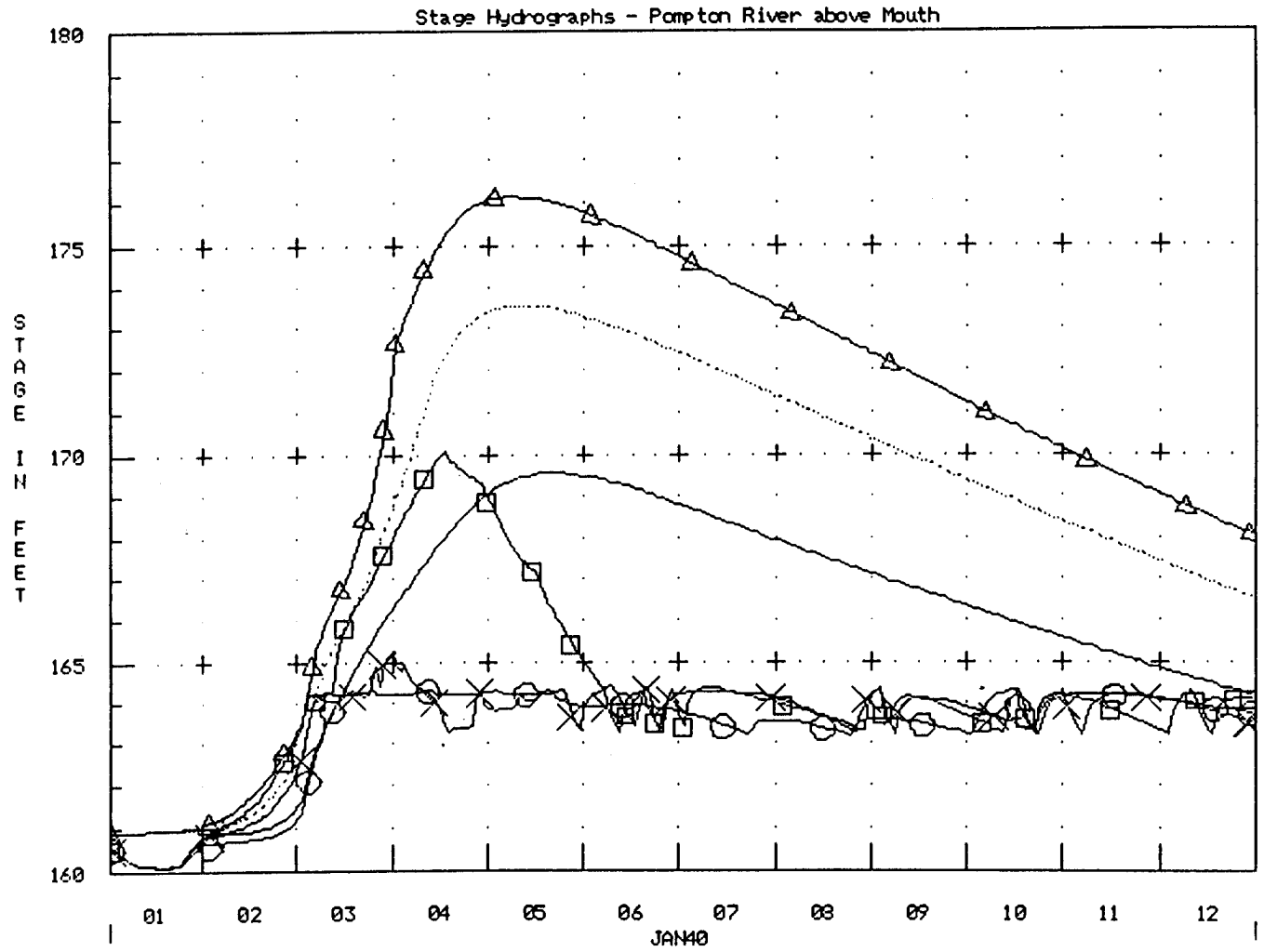


Figure C-112k

28AUG95 13:40:55

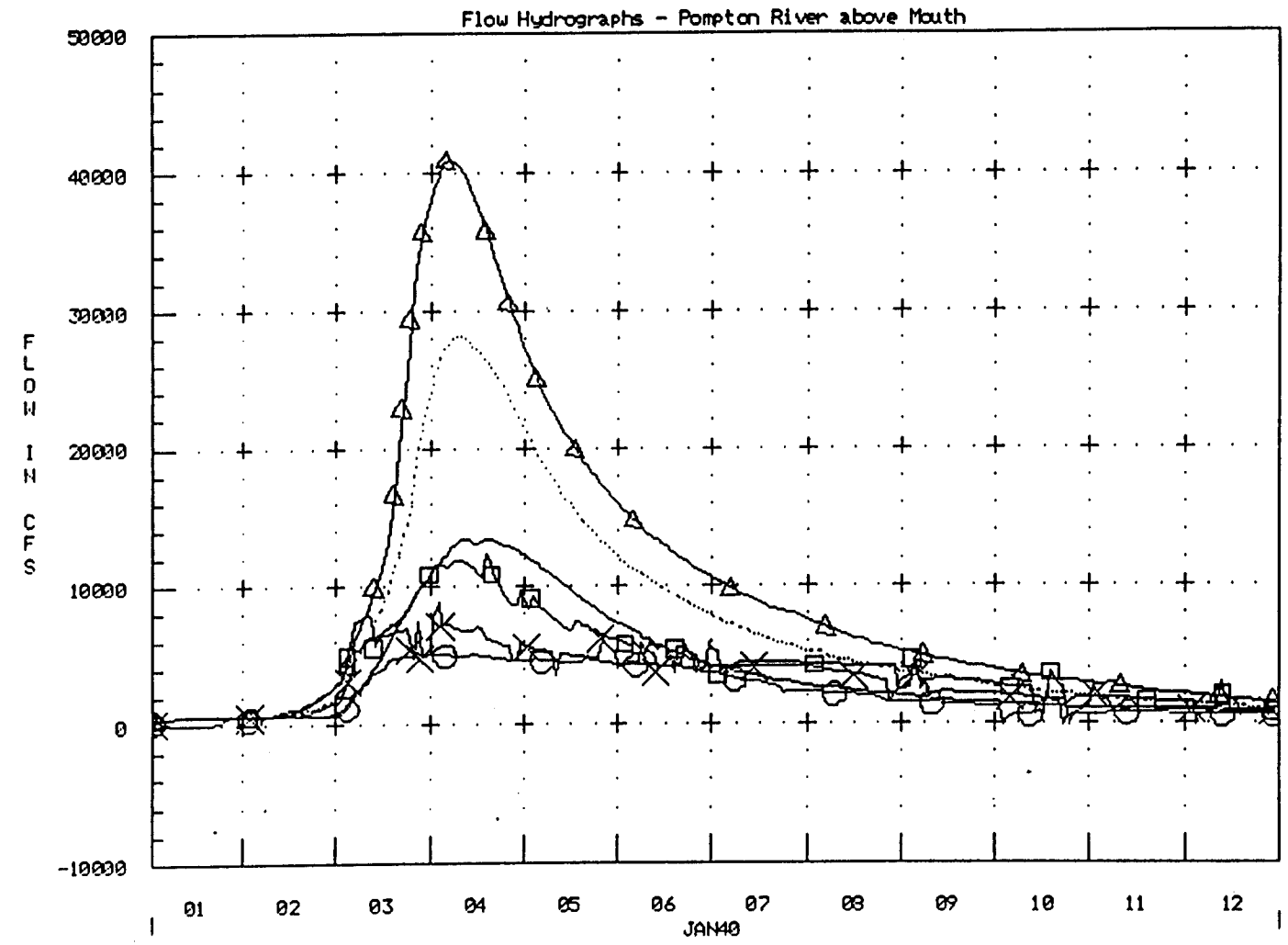


Figure C-1121

28AUG95 13:47:17

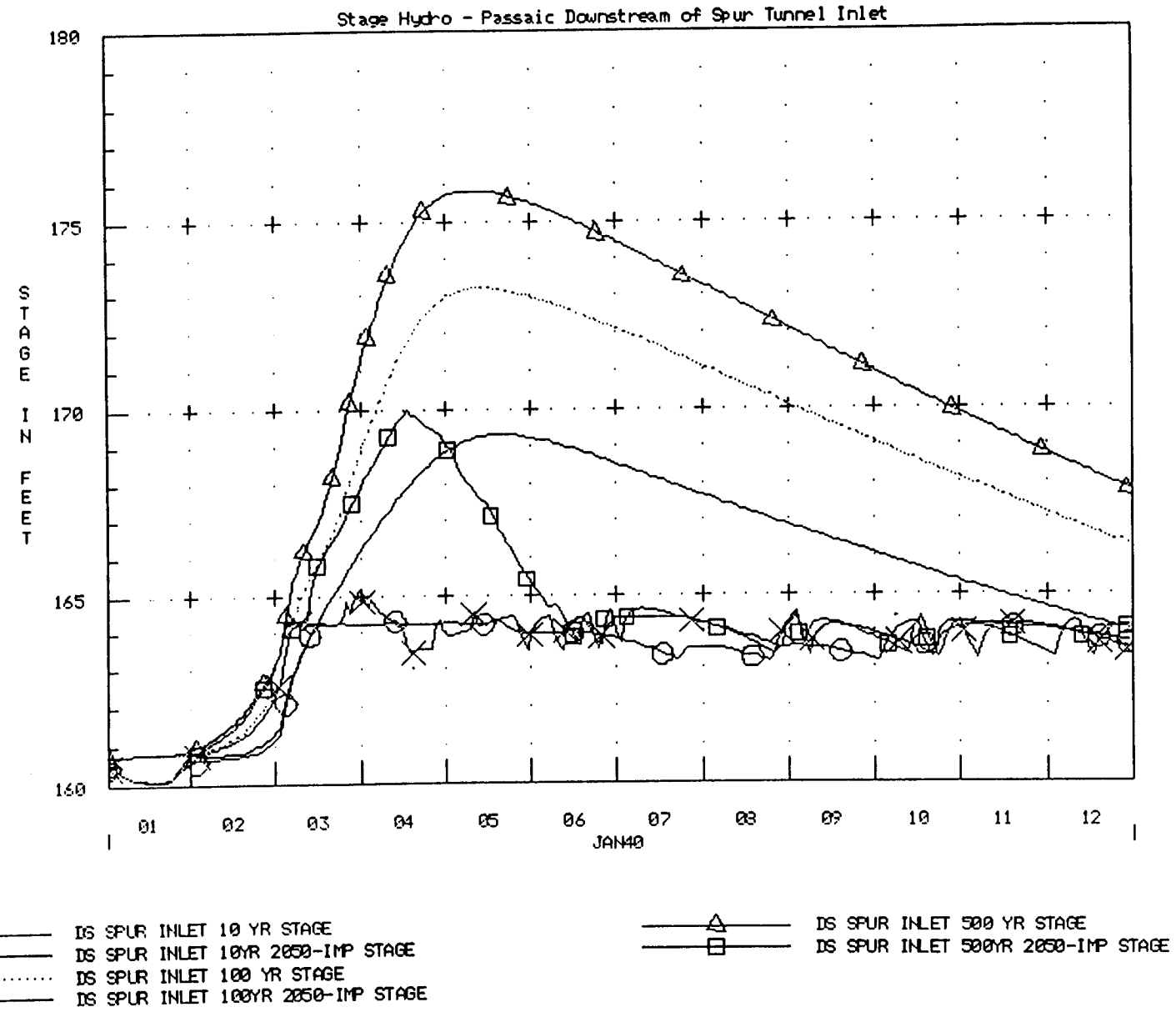


Figure C-112m

28AUG95 13:55:30

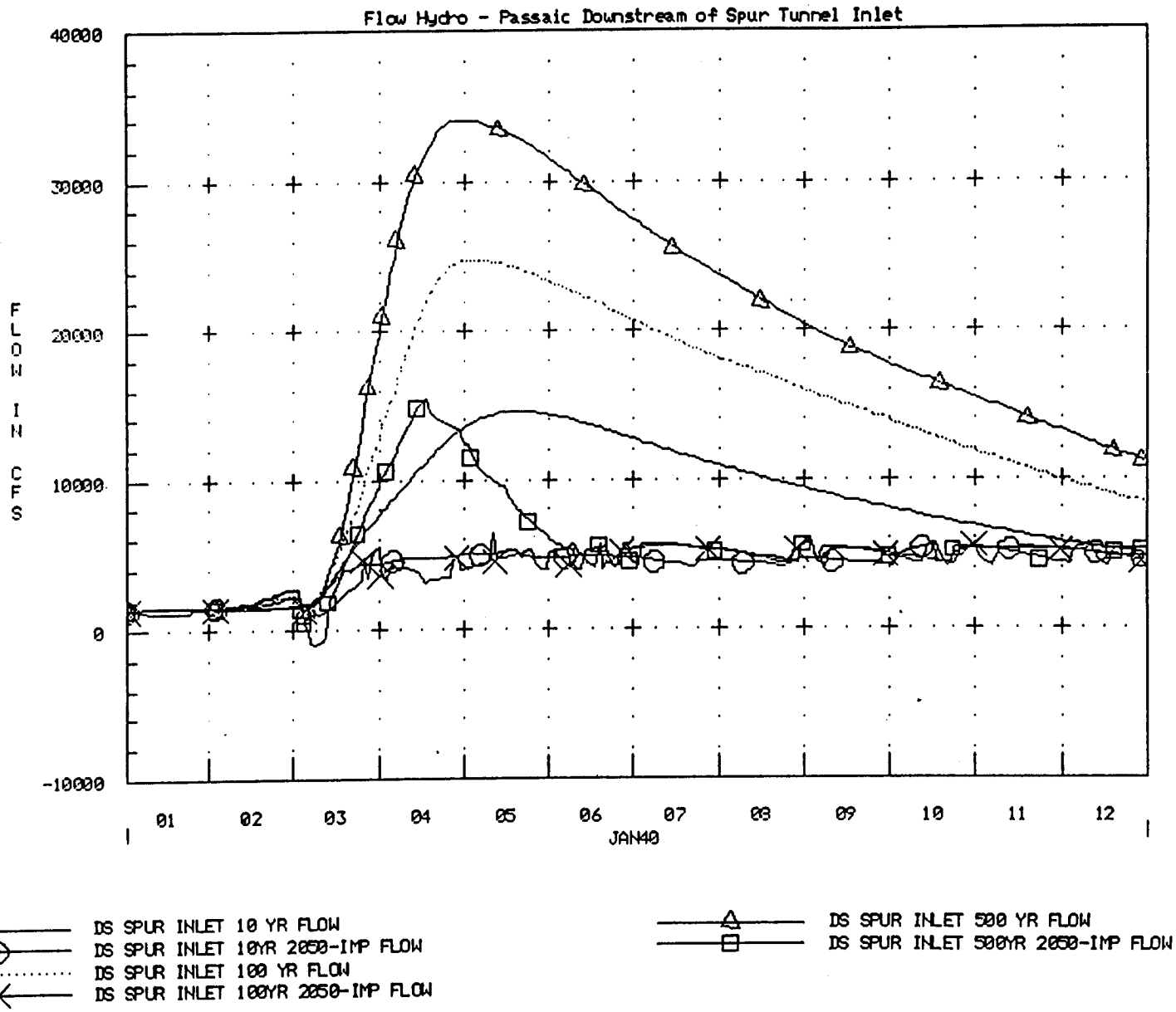


Figure C-112n

28AUG95 14:00:06

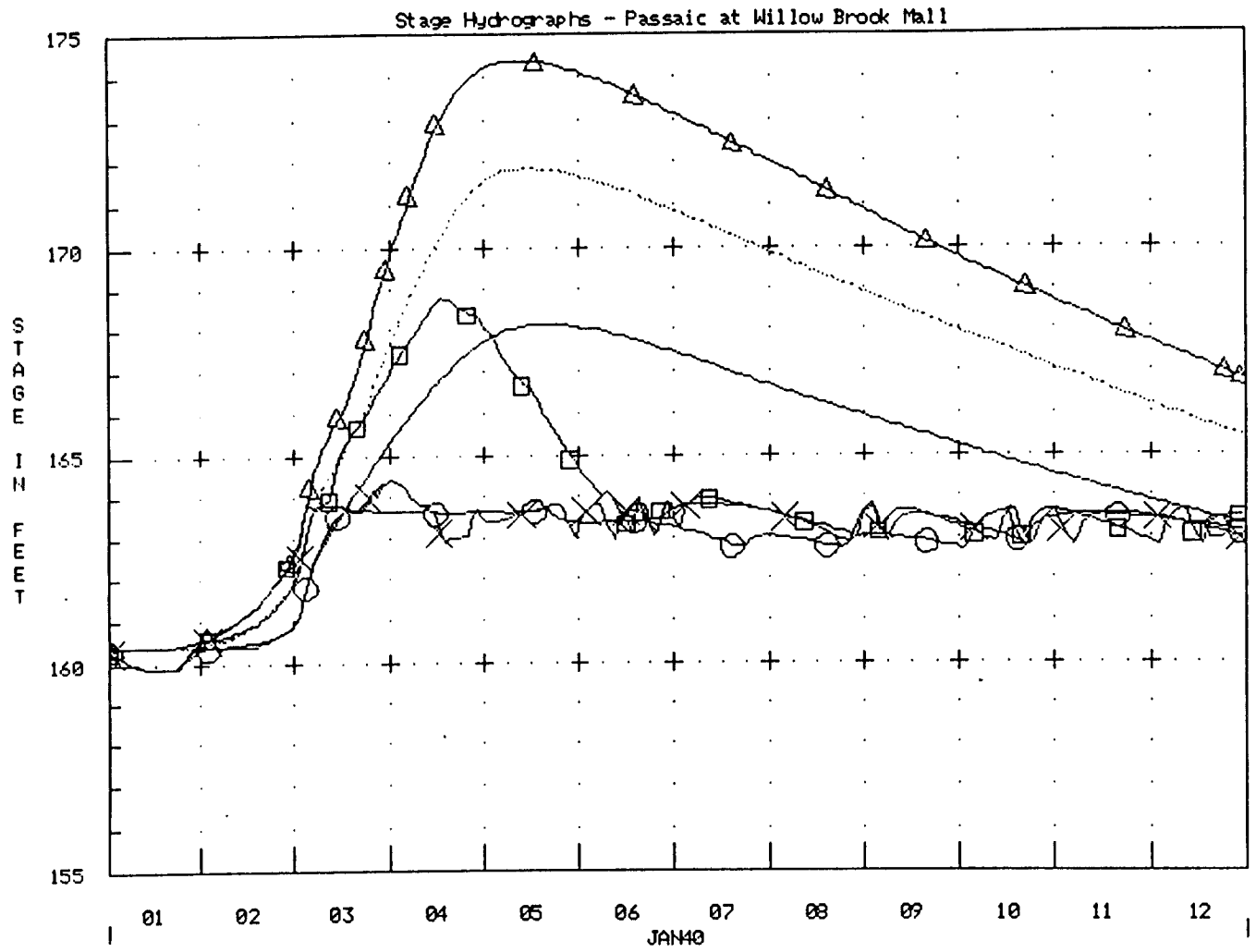


Figure C-1120

28AUG95 14:02:53

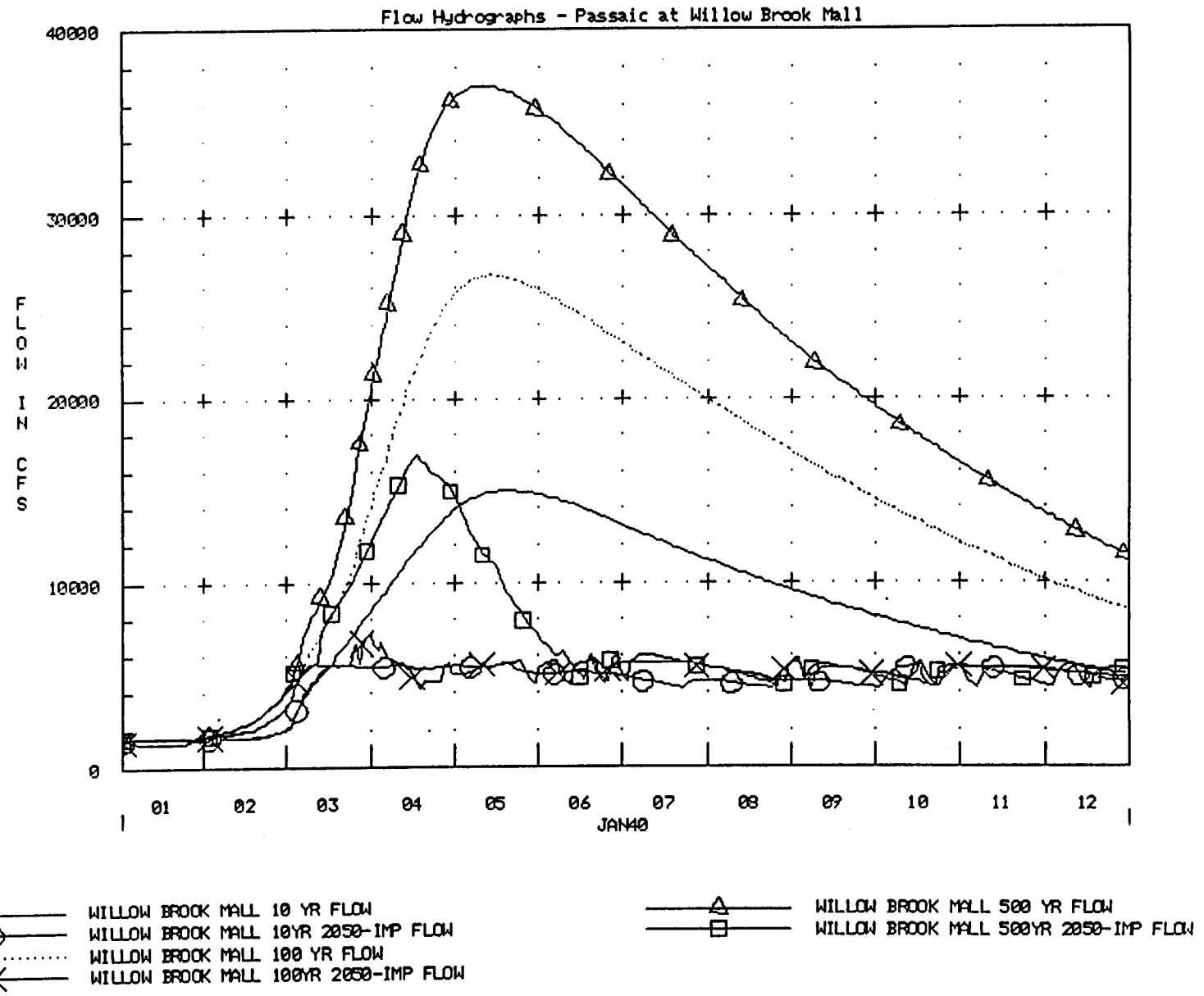


Figure C-112p

28AUG95 14:04:40

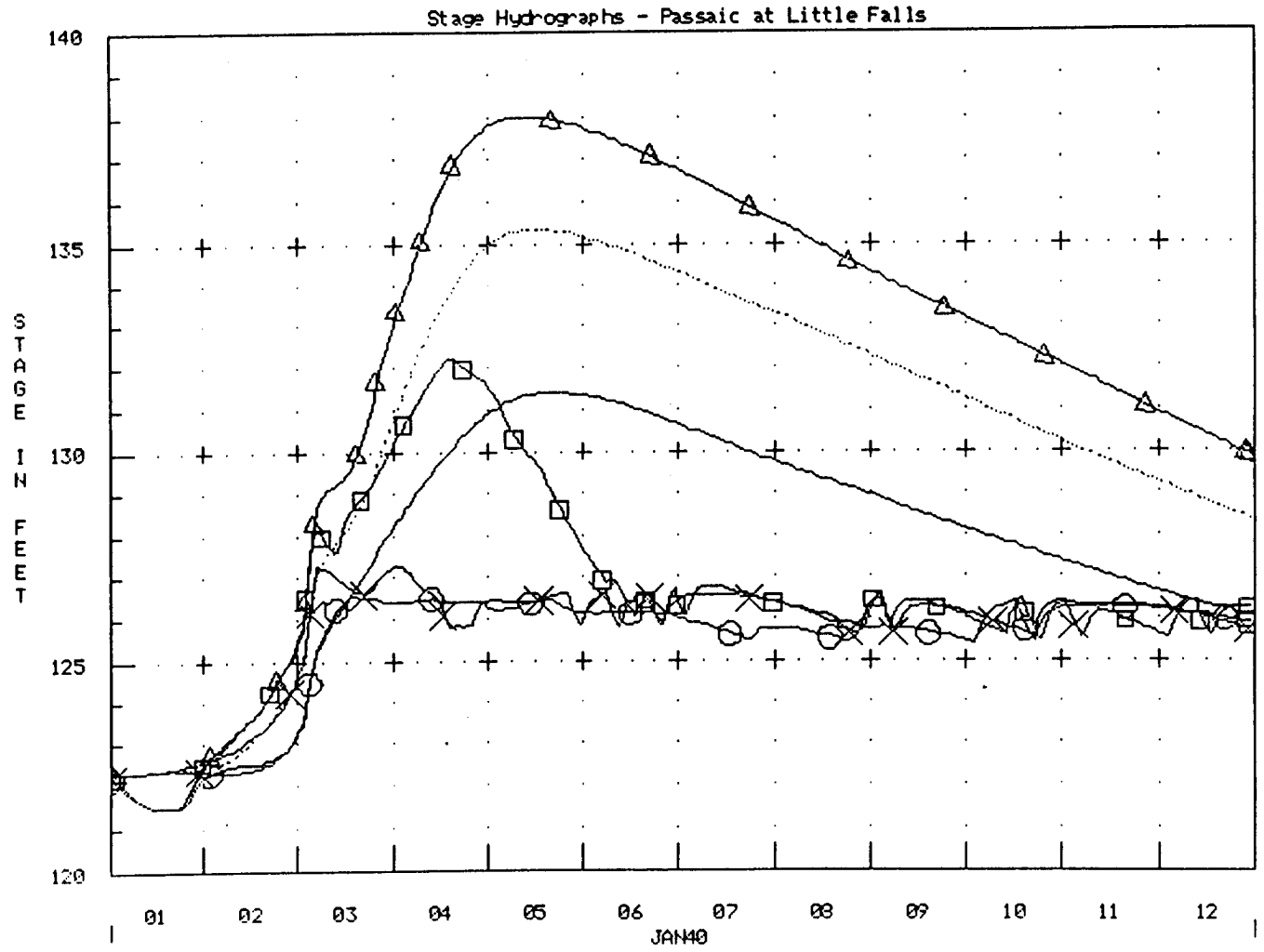


Figure C-112g

28AUG95 14:05:53

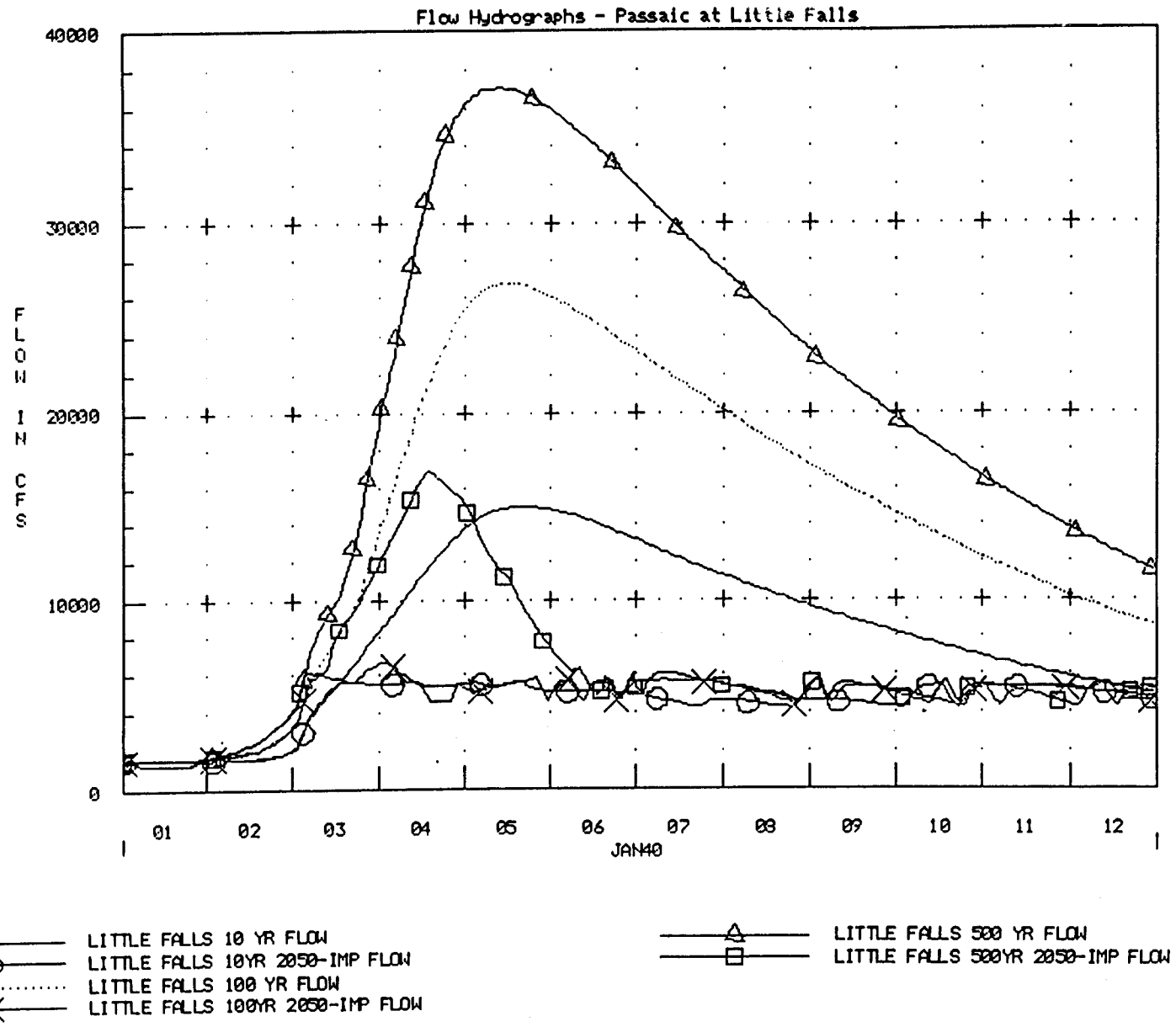


Figure C-112r

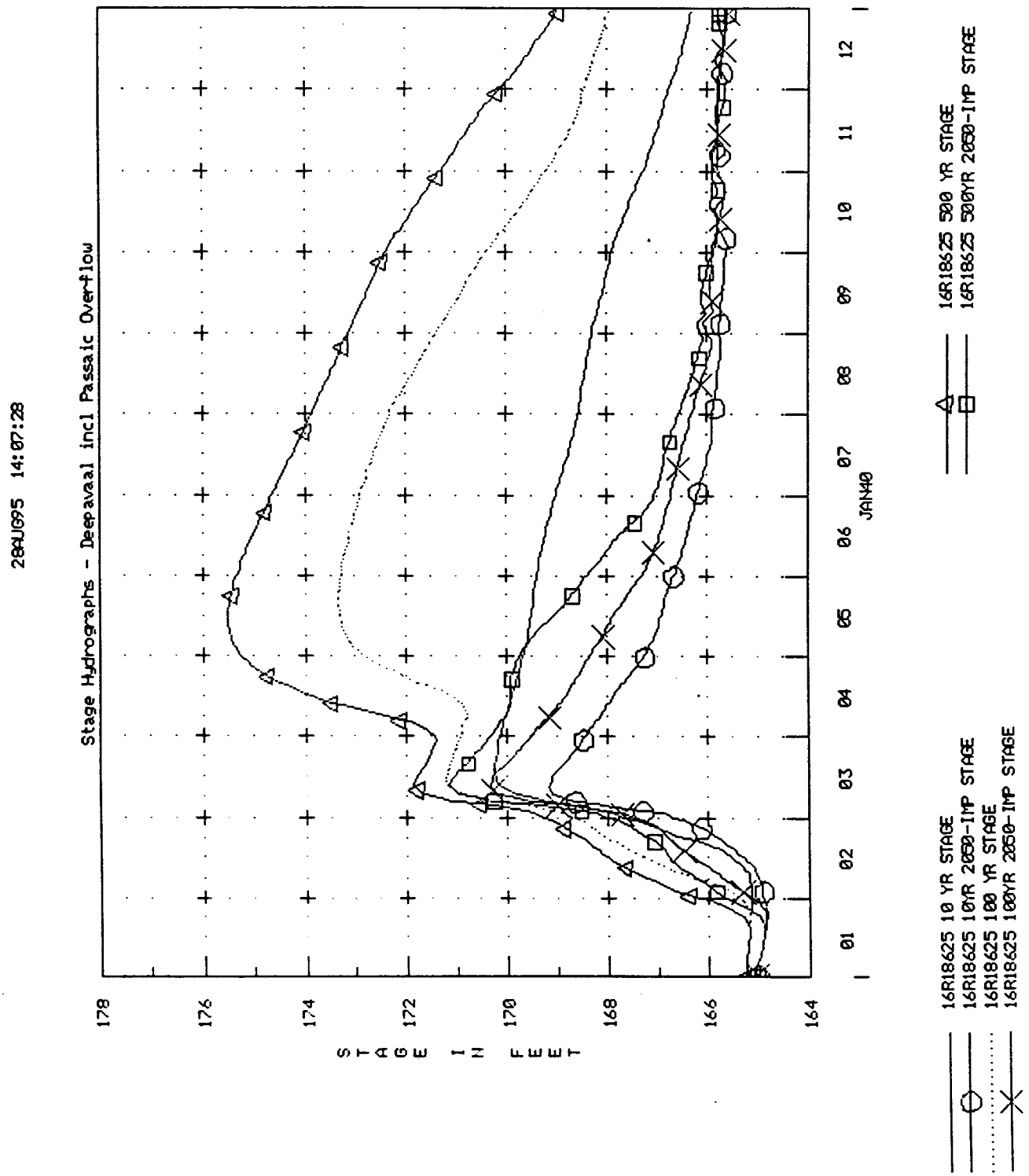


Figure C-112s

28AUG95 14:08:51

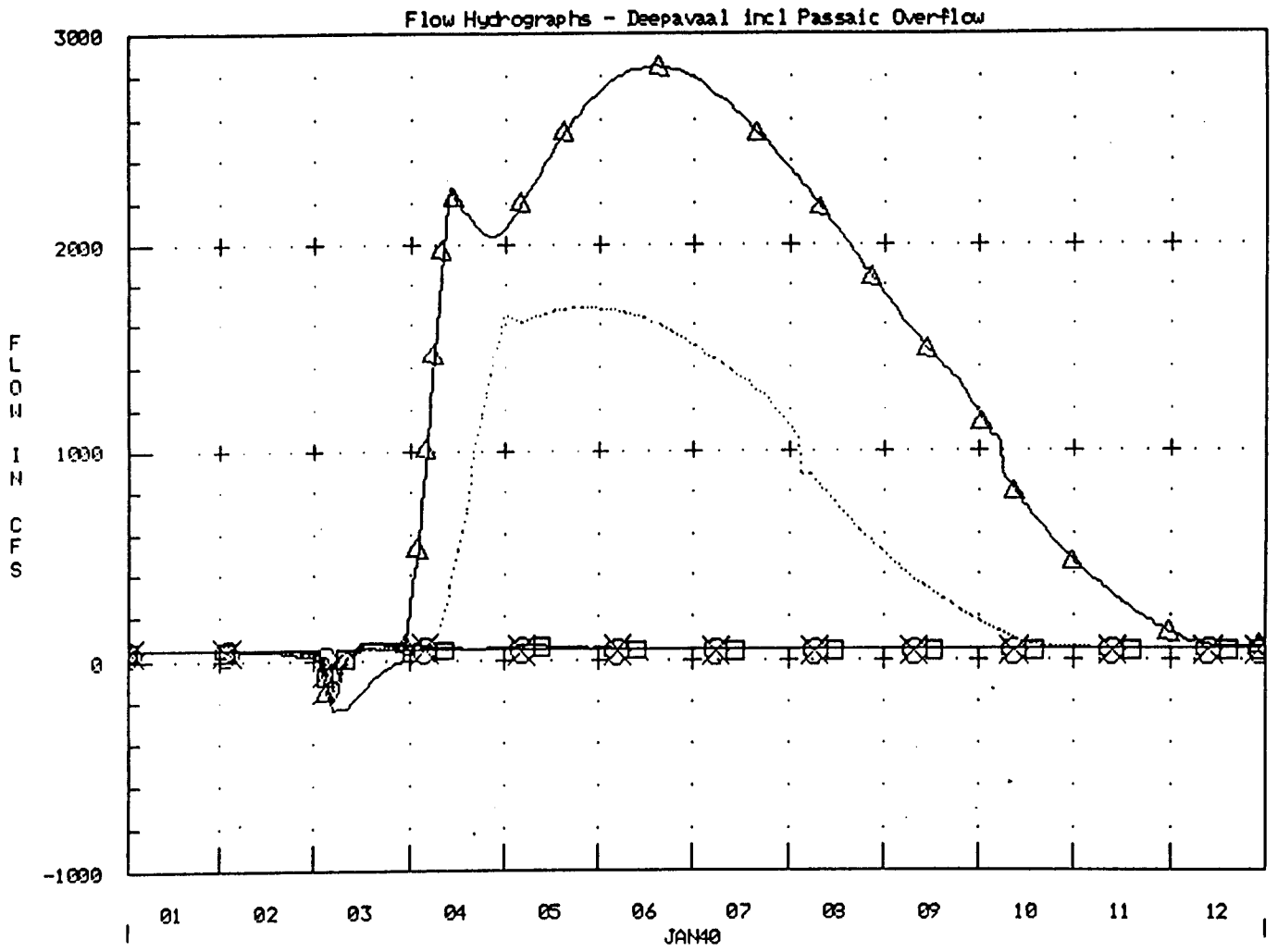


Figure C-112t

28AUG95 14:12:36

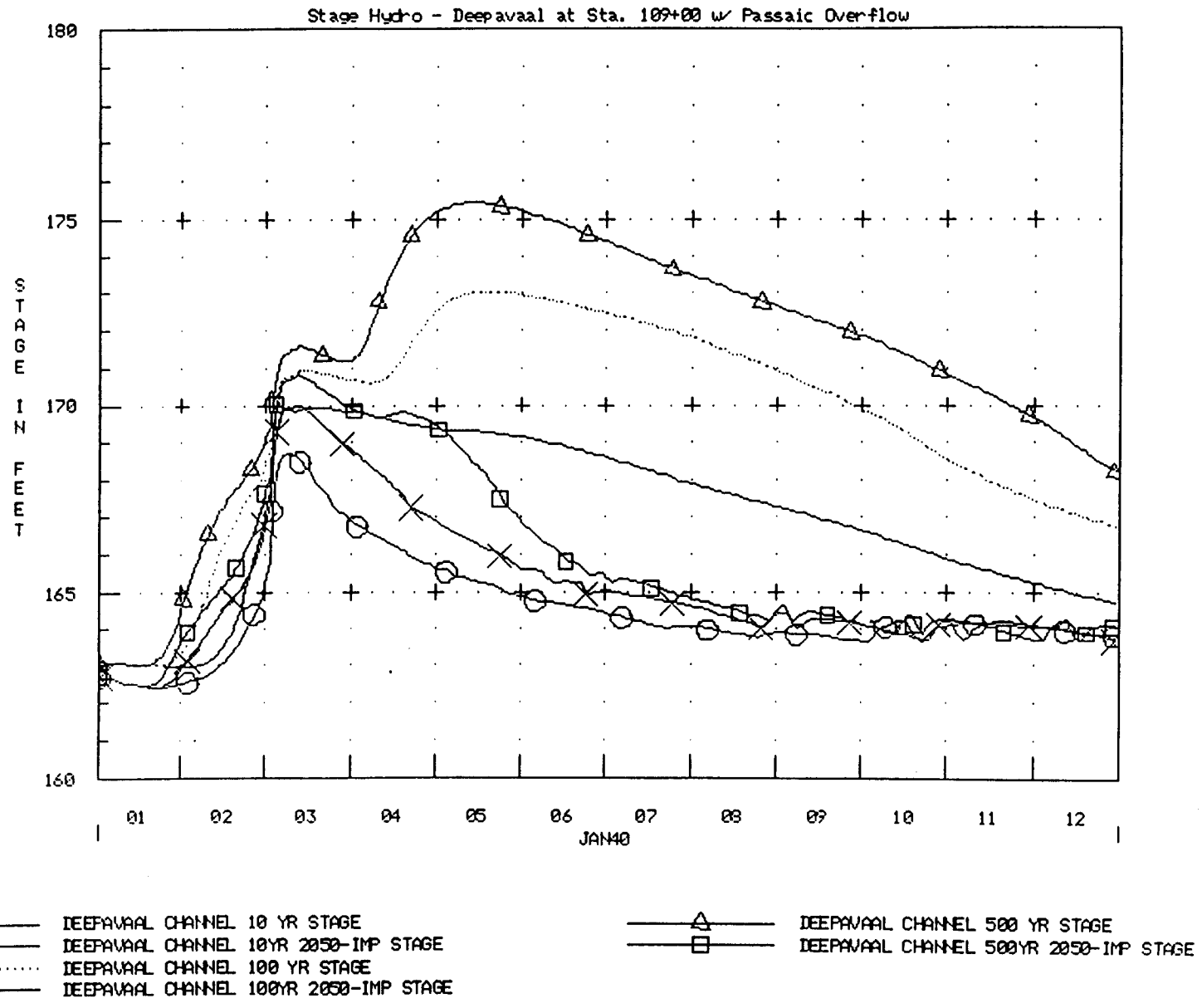


Figure C-112u

28AUG95 14:14:52

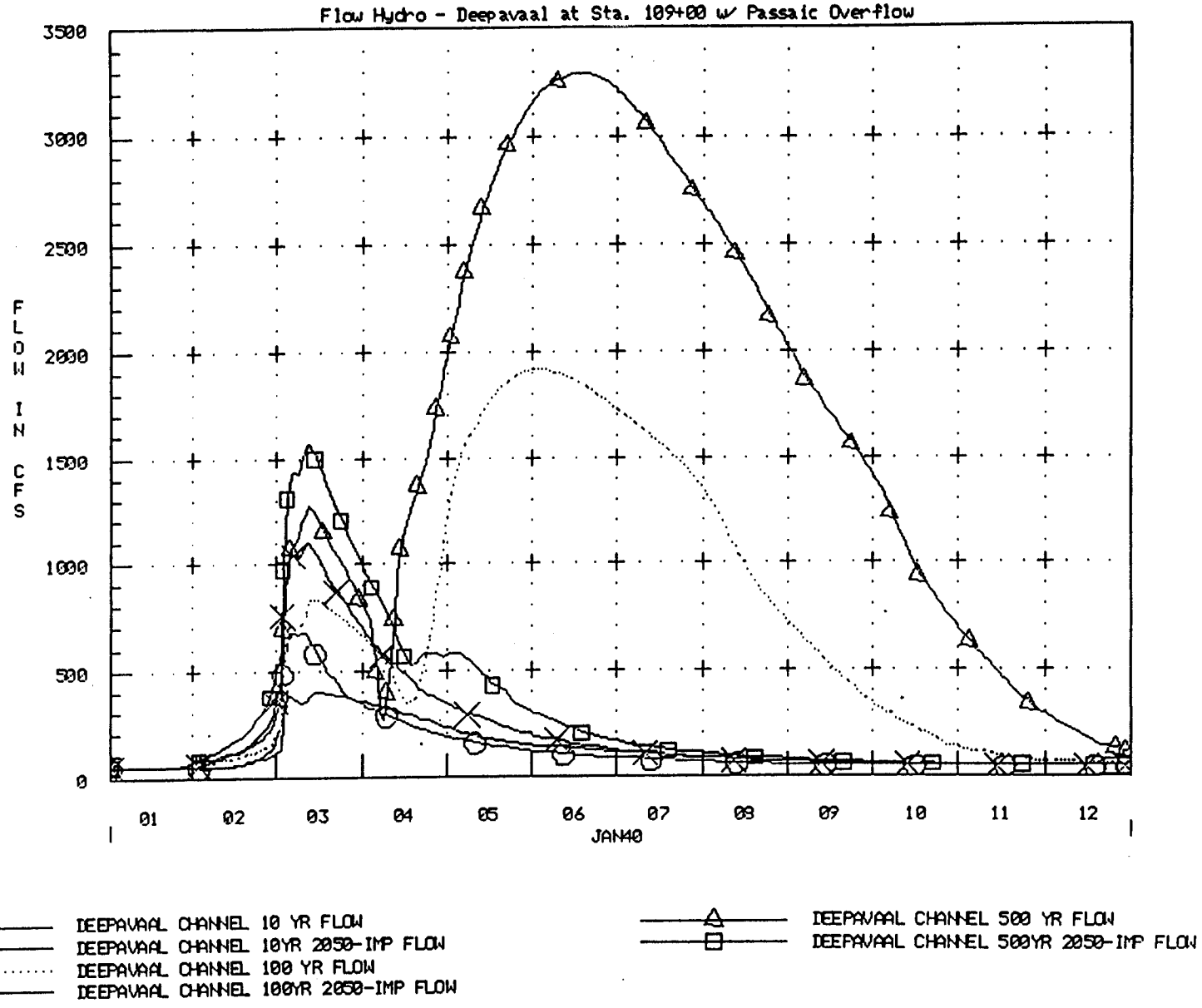


Figure C-112v

28AUG95 14:16:26

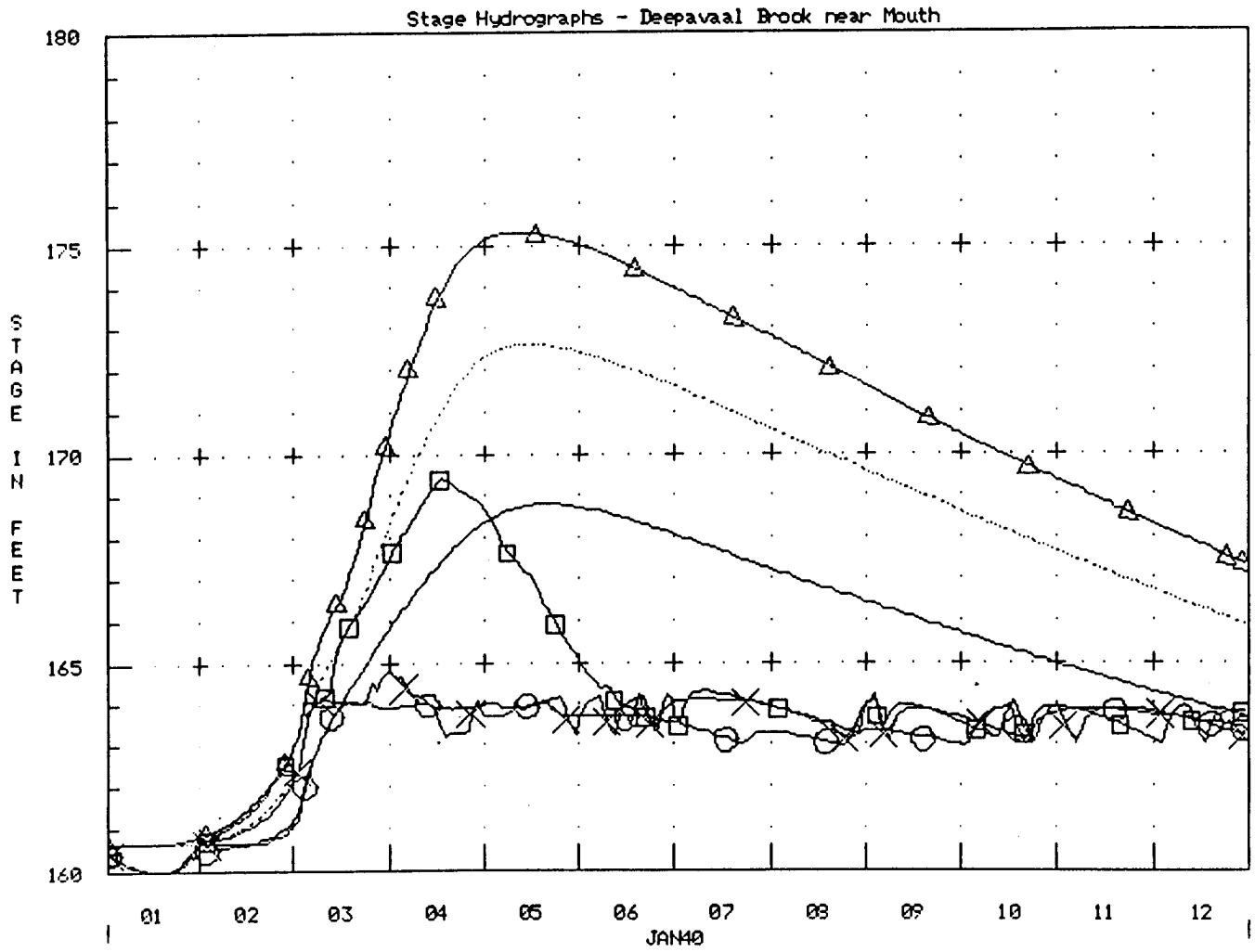


Figure C-112W

28AUG95 14:17:25

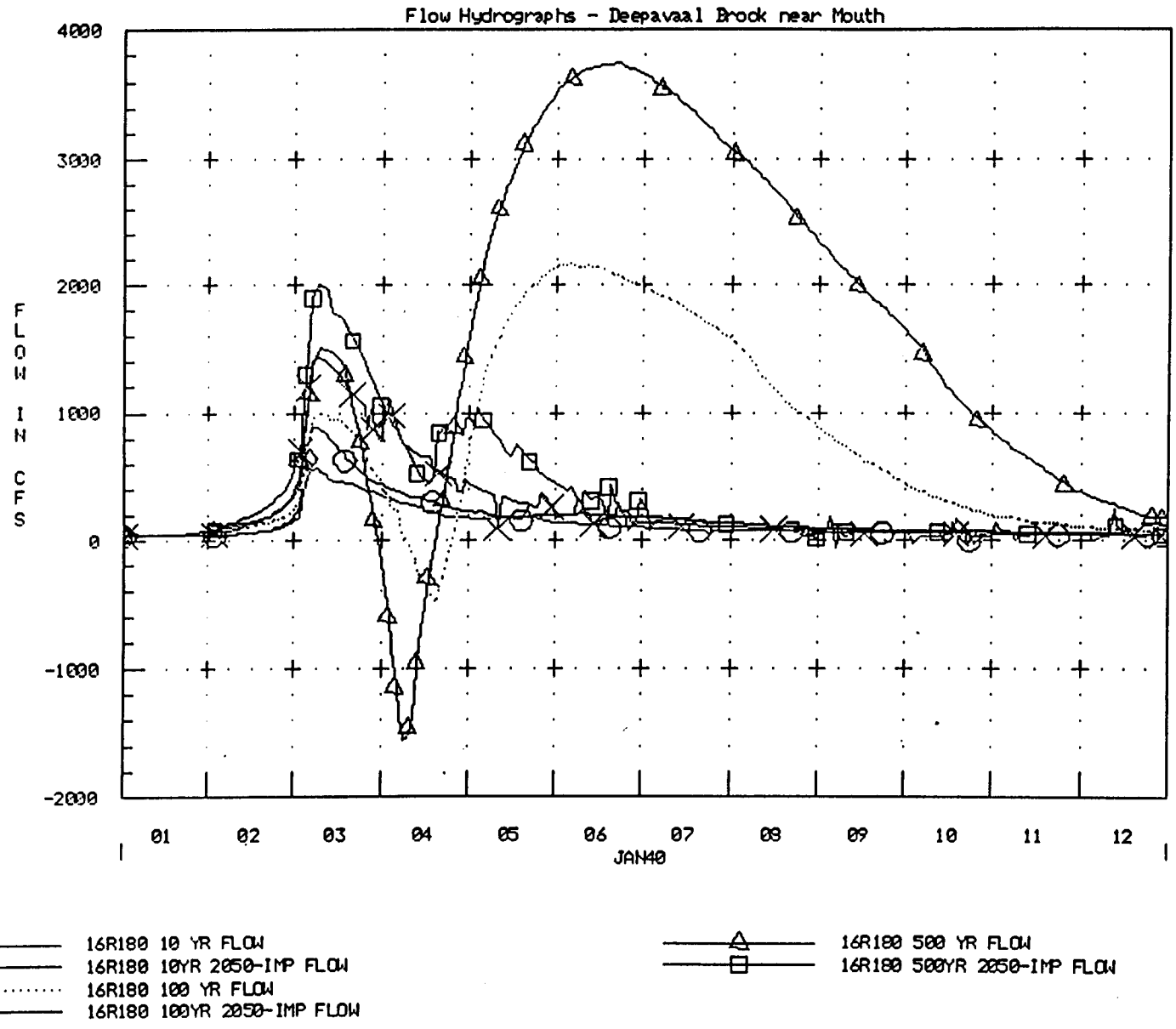


Figure C-112x

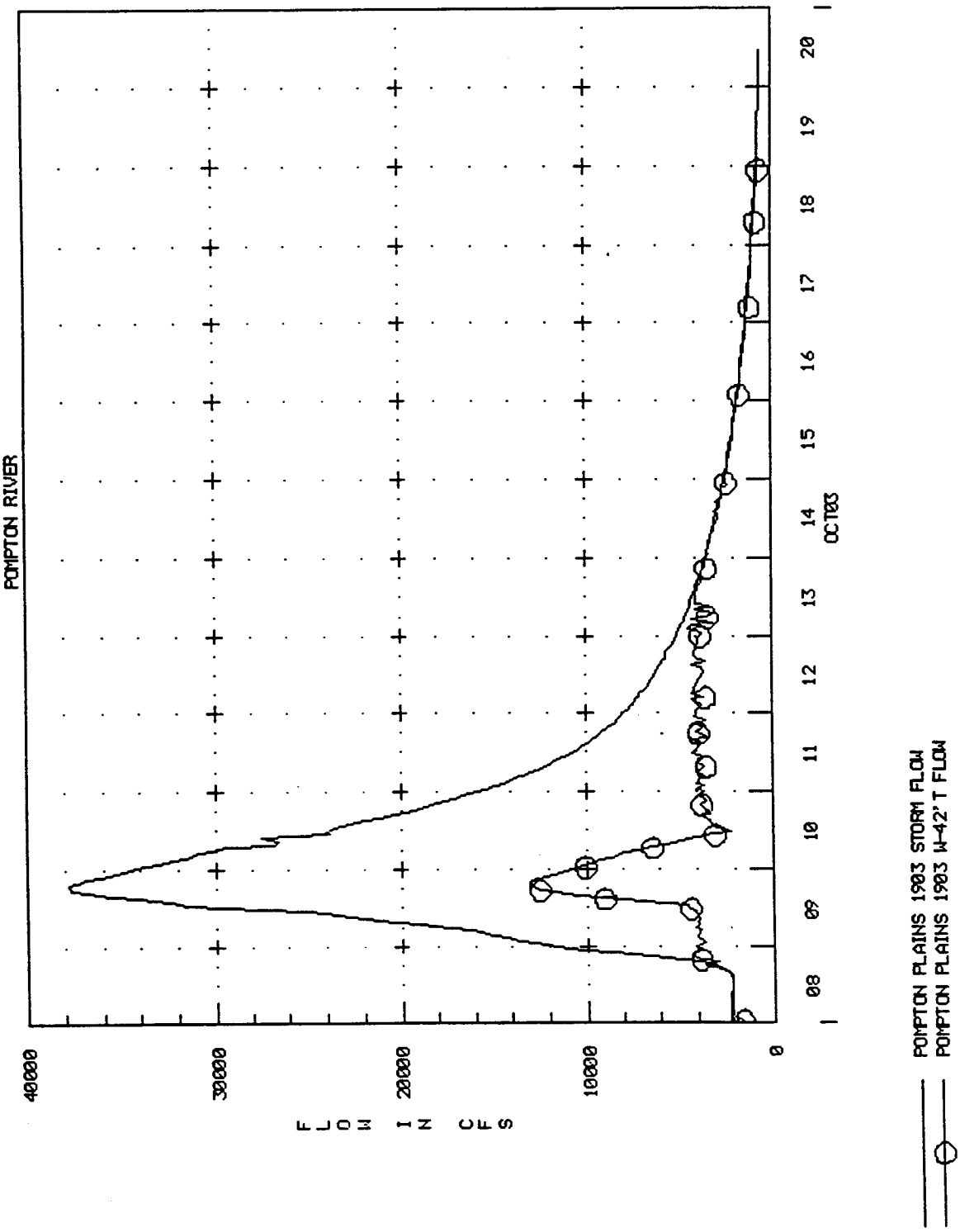


Figure C-113a

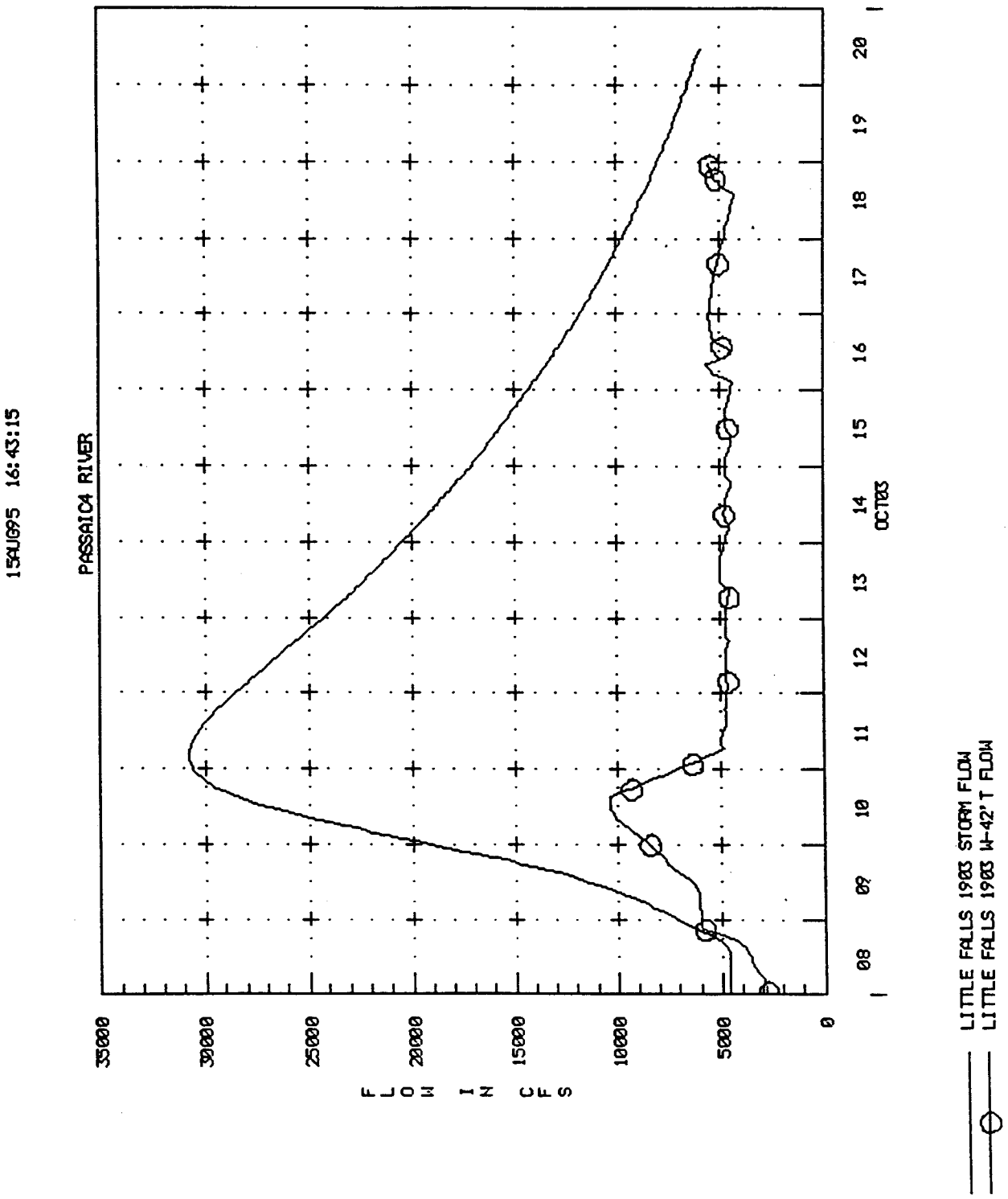
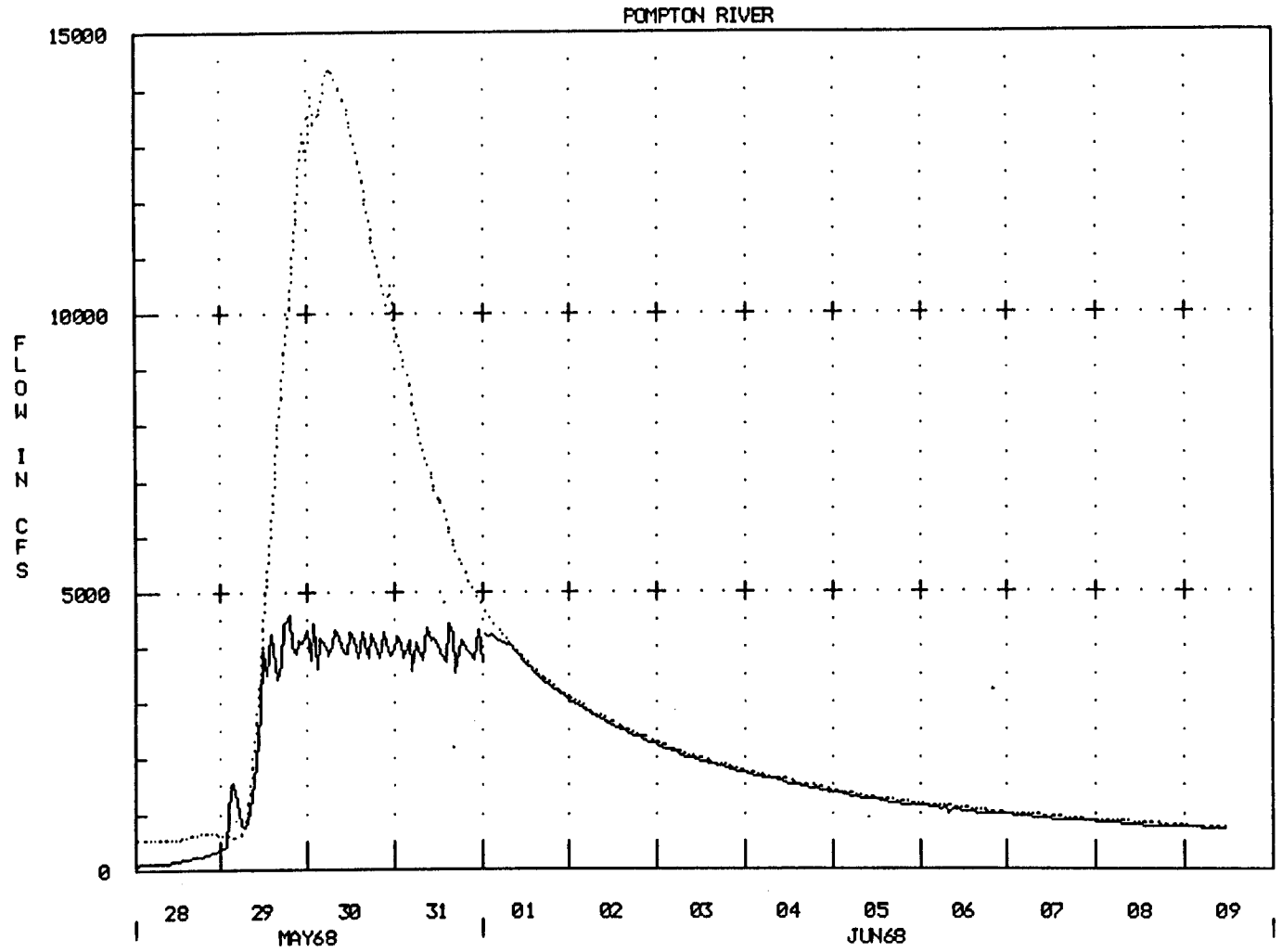


Figure C-113b

07AUG95 10:05:38



POMPTON PLAINS 1968 STORM FLOW
POMPTON PLAINS 1968 STORM FLOW
POMPTON PLAINS MAY 1968 W-42'T FLOW
POMPTON PLAINS MAY 1968 W-42'T FLOW

Figure C-113c

07AUG95 10:03:50

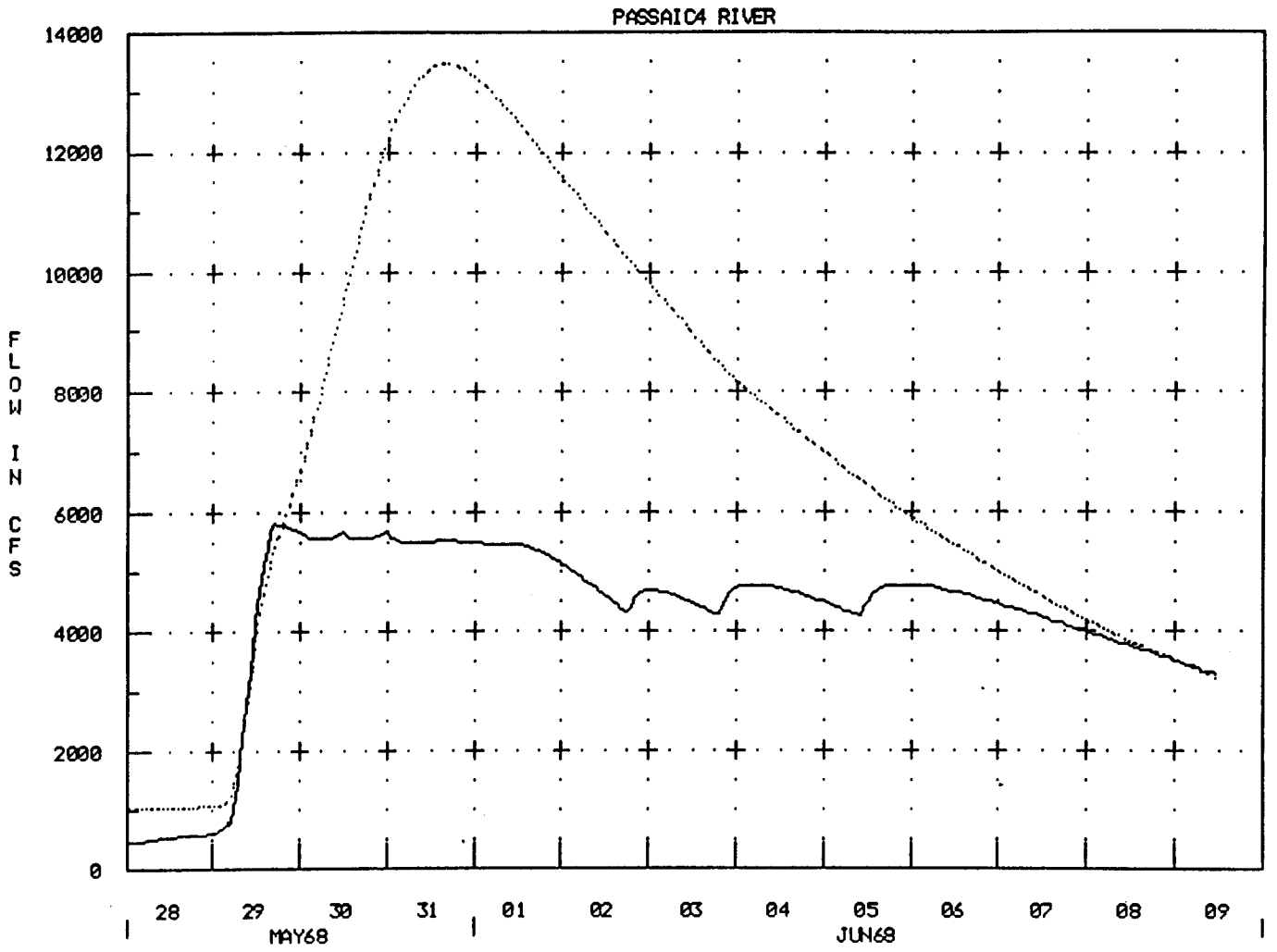
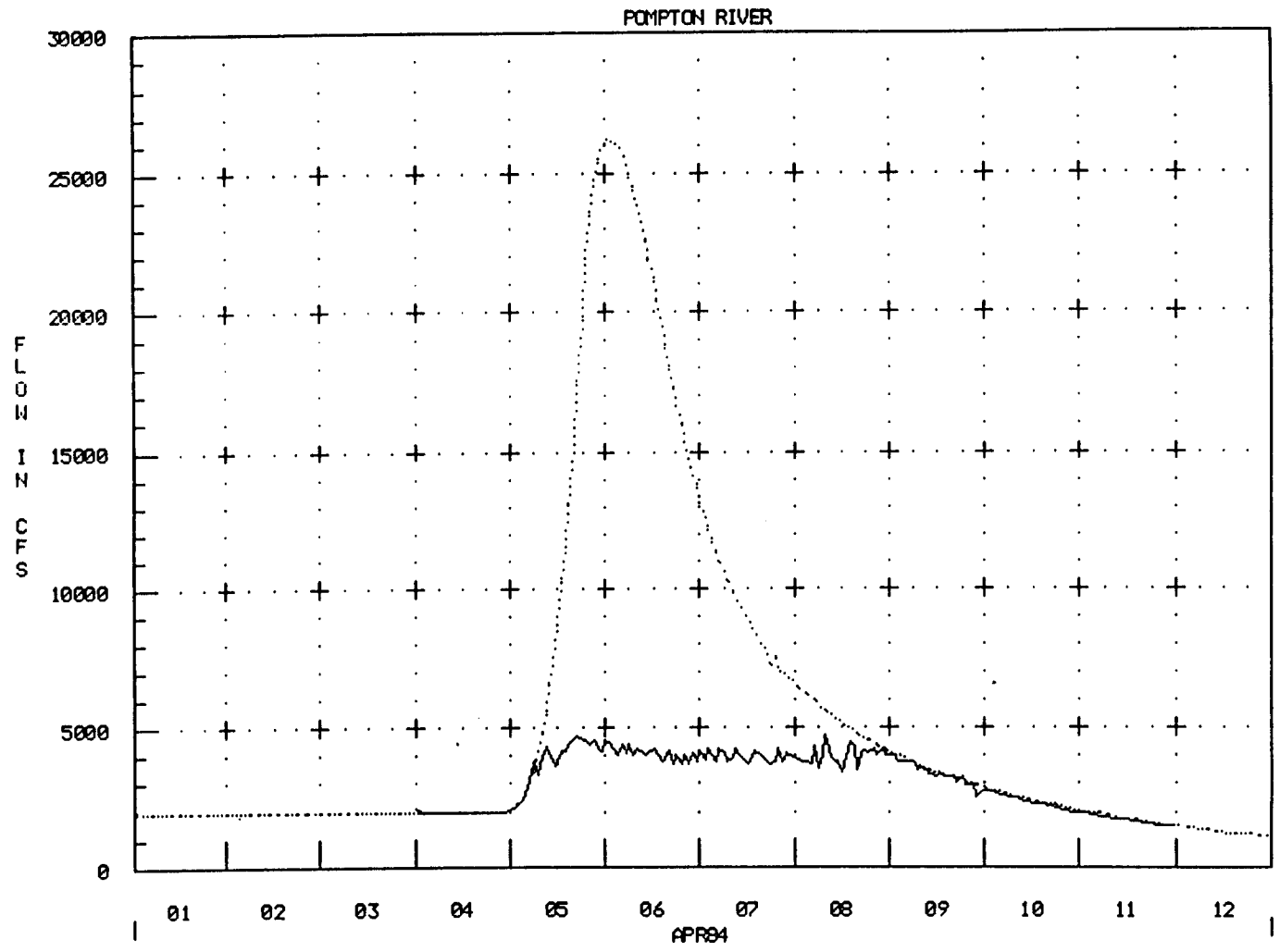


Figure C-113d

04AUG95 16:19:03



..... POMPTON PLAINS 1984 EVENT FLOW
——— POMPTON PLAINS APR84 W-42'T FLOW

Figure C-113e

04AUG95 16:17:35

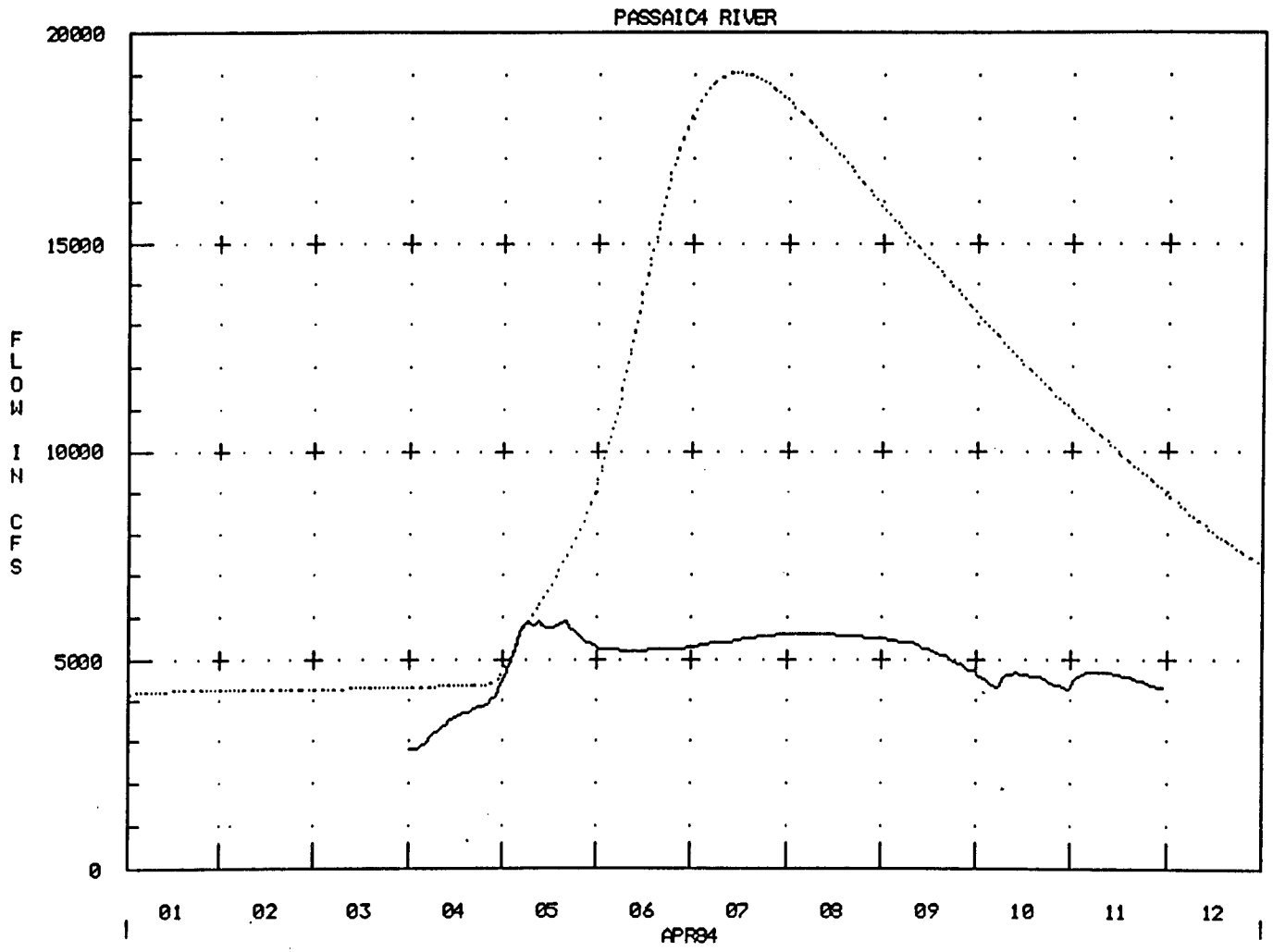


Figure C-113f

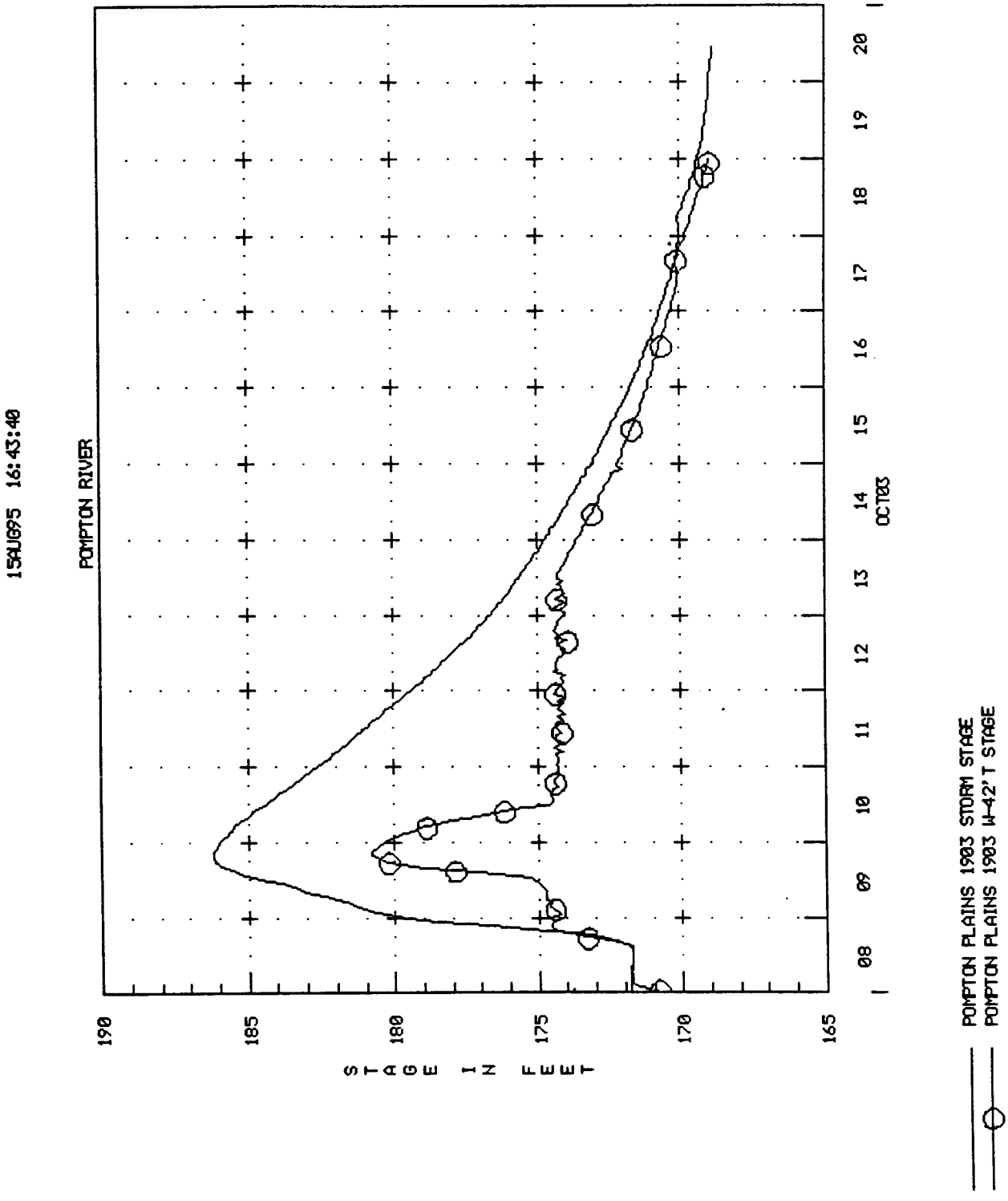


Figure C-113g

15AUG95 16:44:02

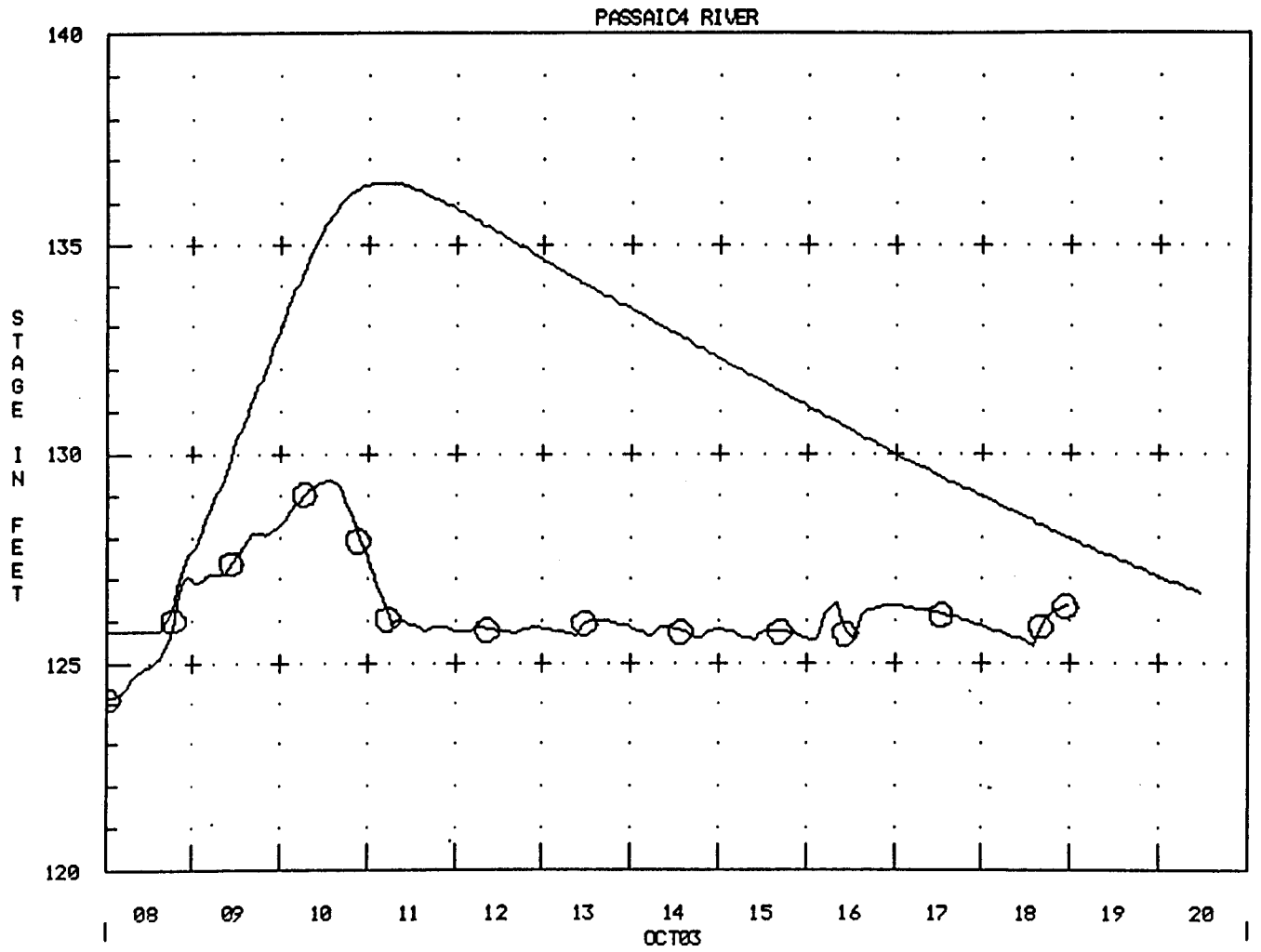


Figure C-113h

07AUG95 10:06:08

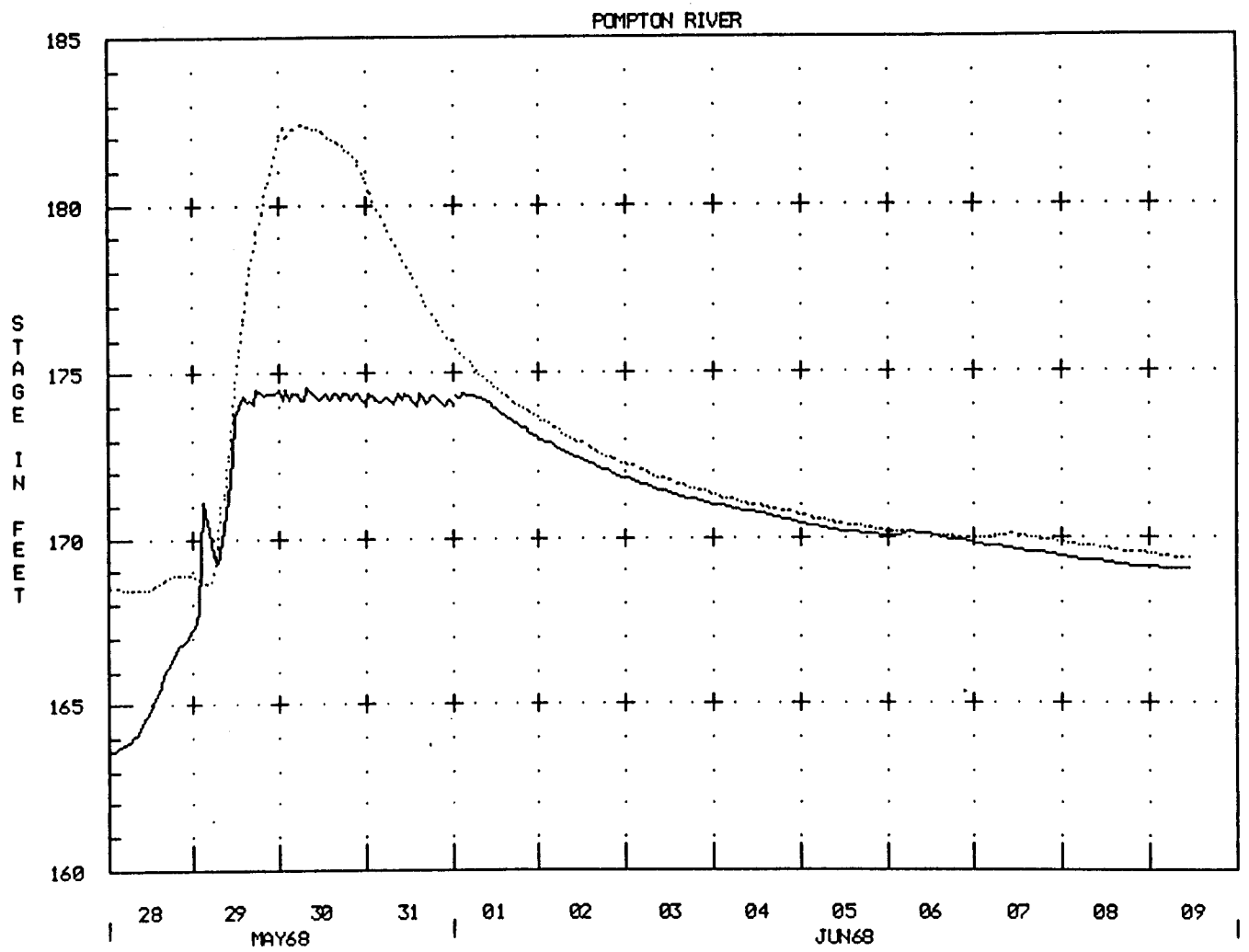


Figure C-113i

07AUG95 10:05:13

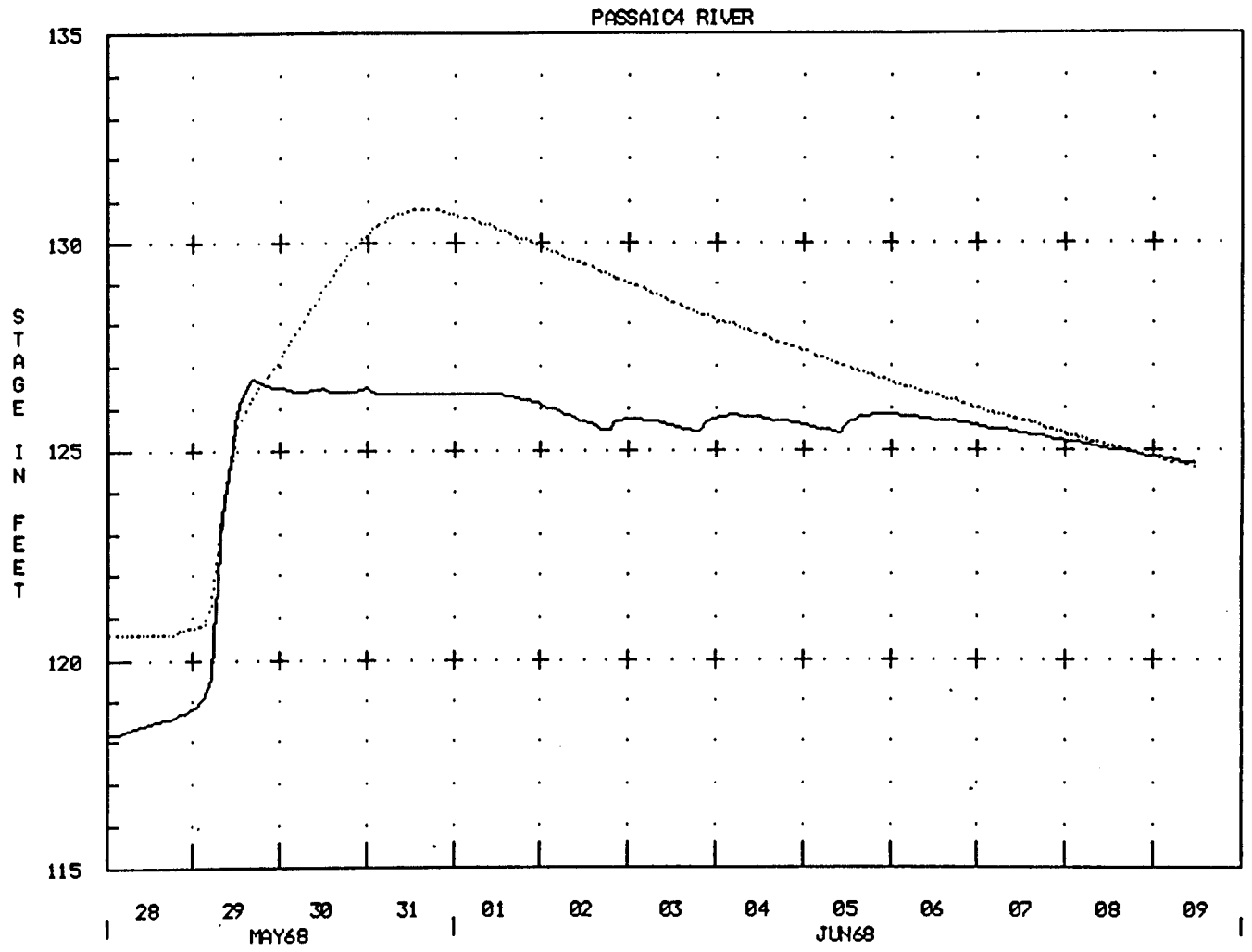


Figure C-113j

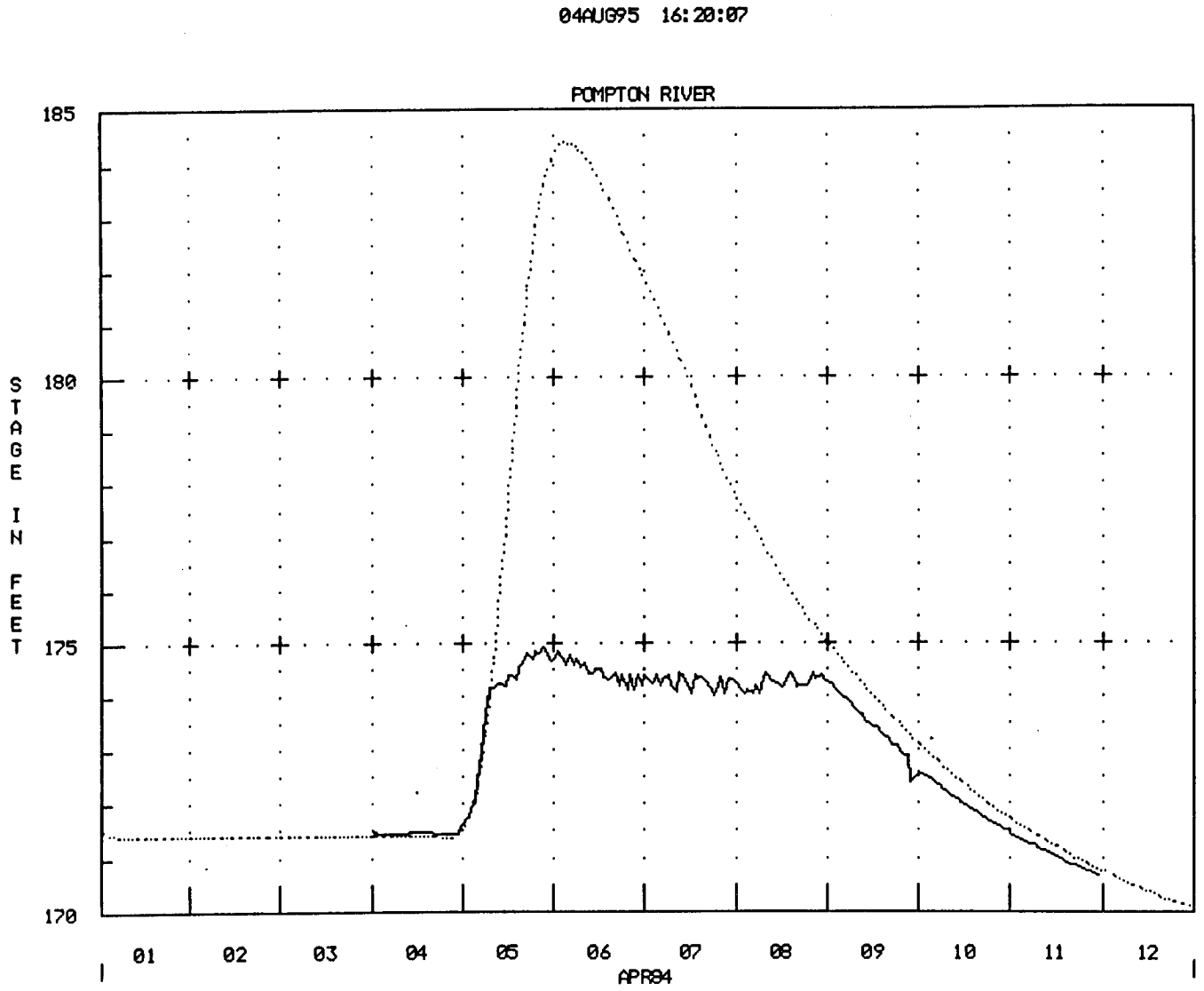


Figure C-113k

04AUG95 16:18:18

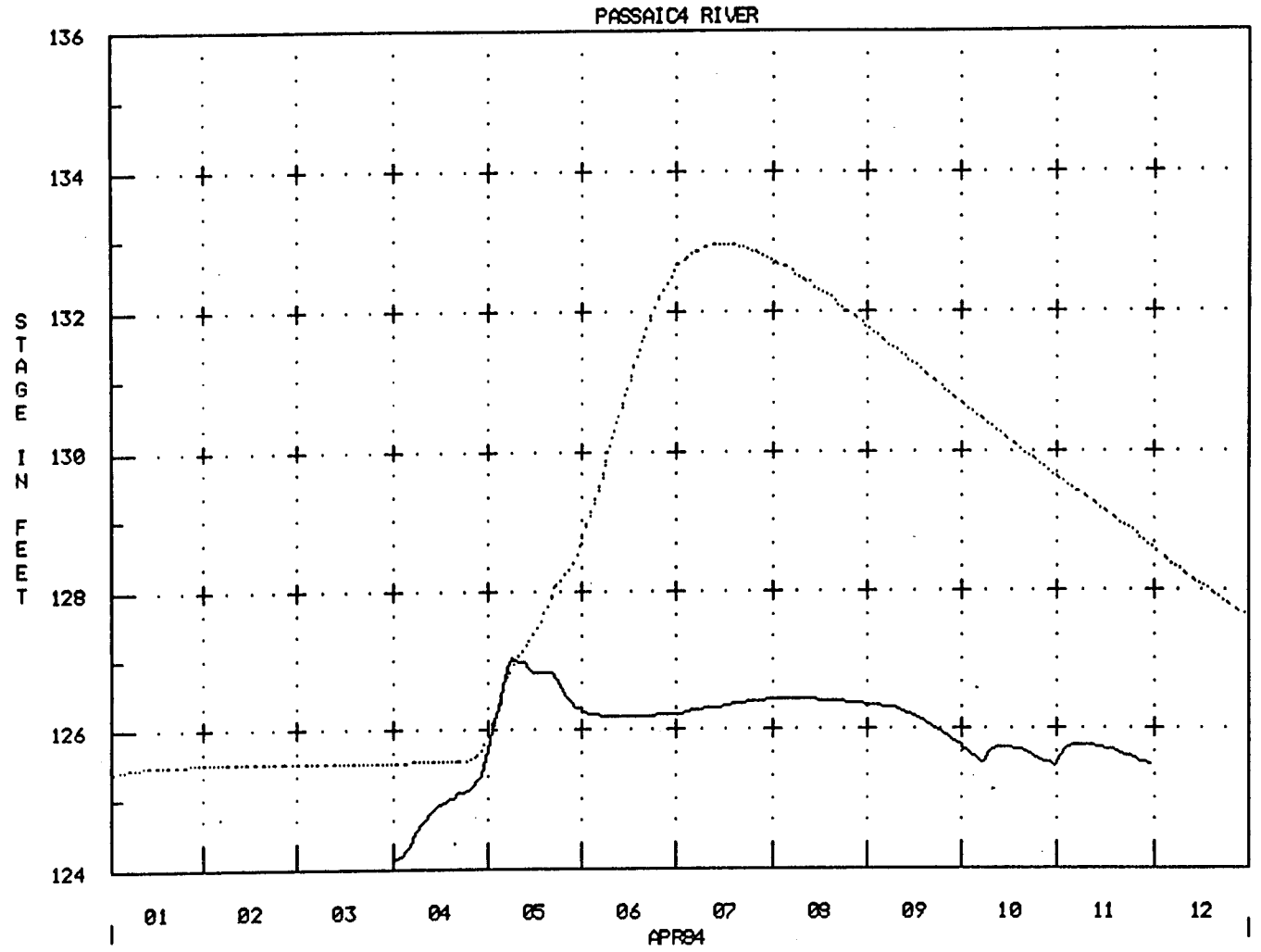


Figure C-1131

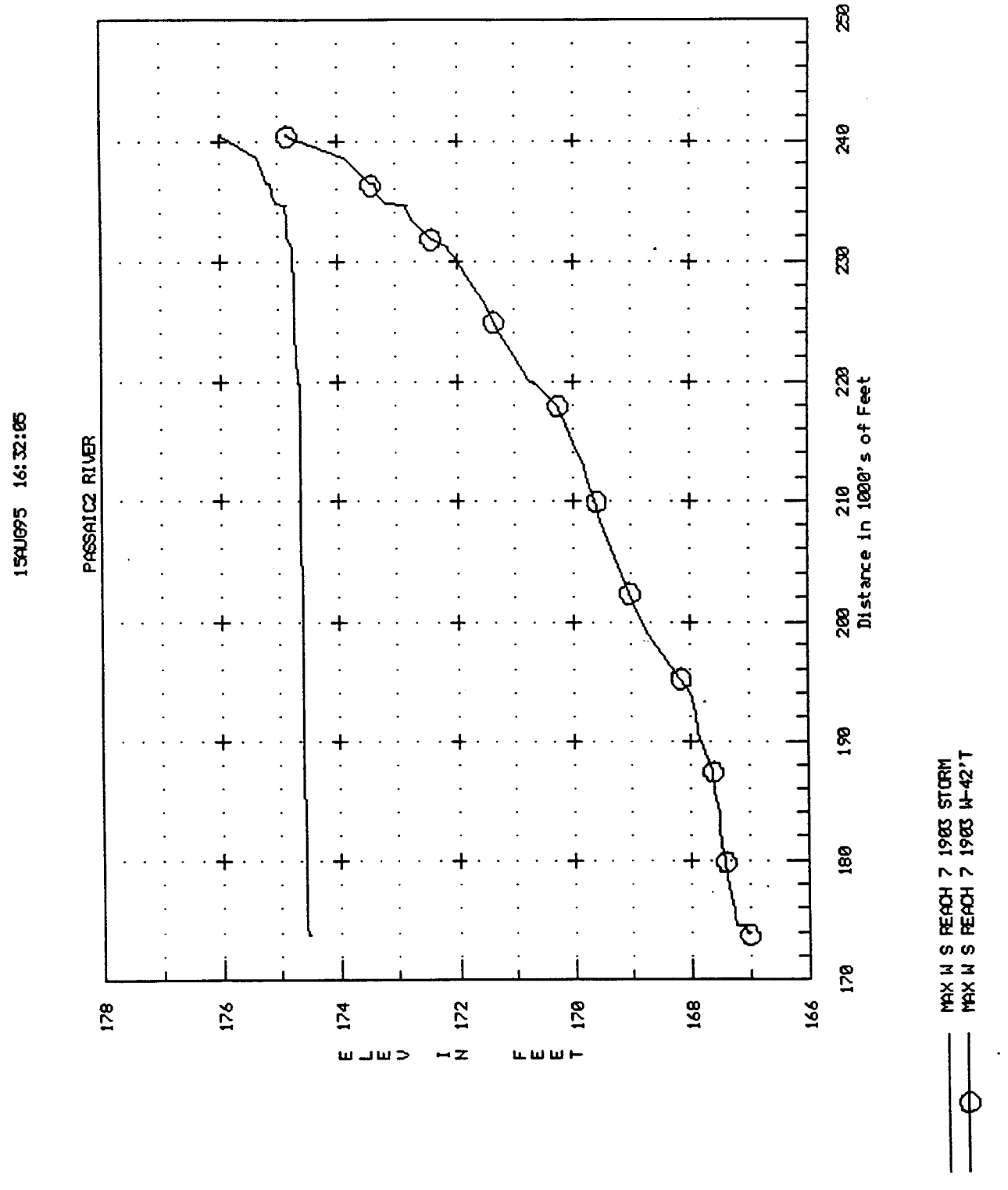


Figure C-113m

07AUG95 10:11:46

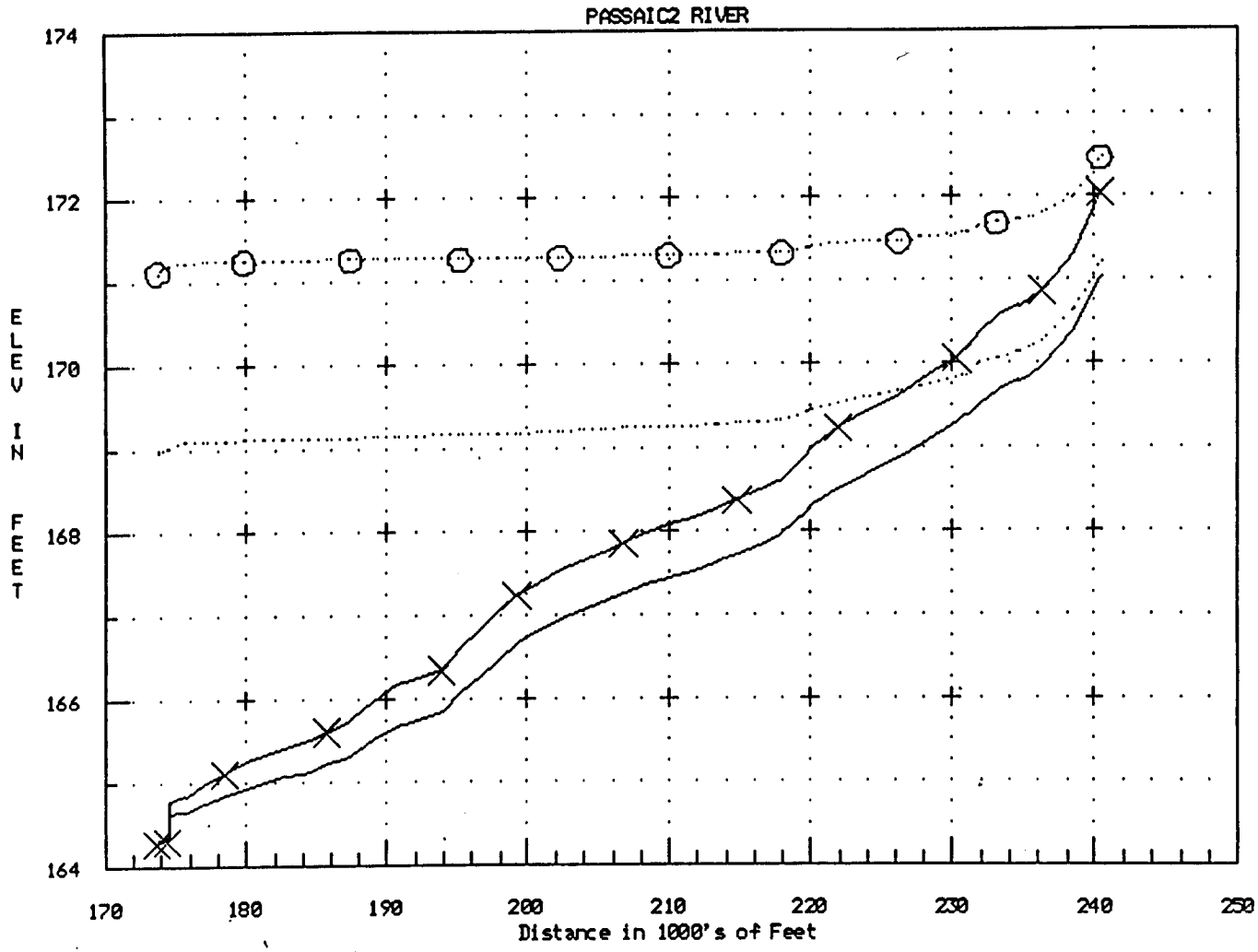


Figure C-113n

15AUG95 16:32:26

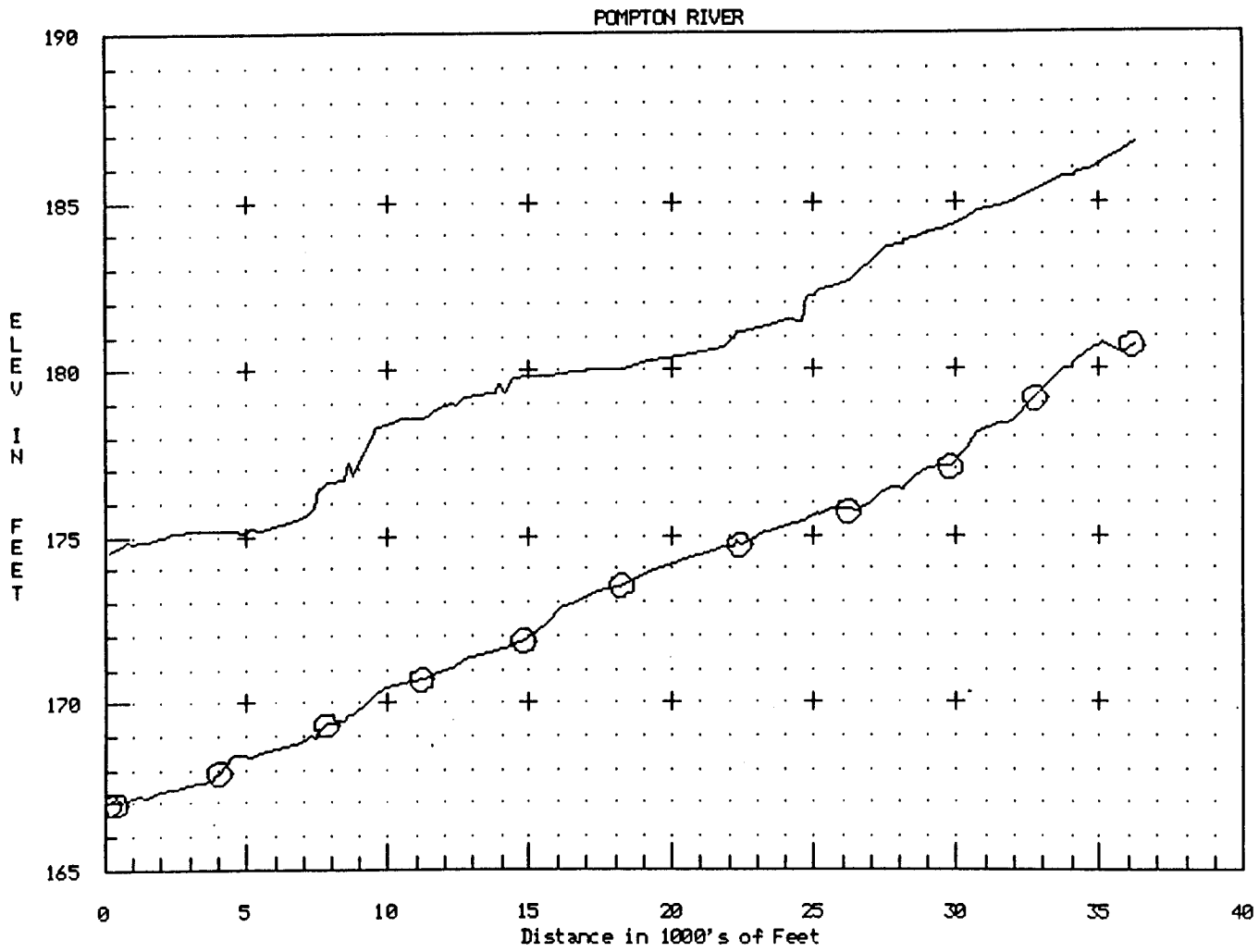


Figure C-113o

07AUG95 10:12:52

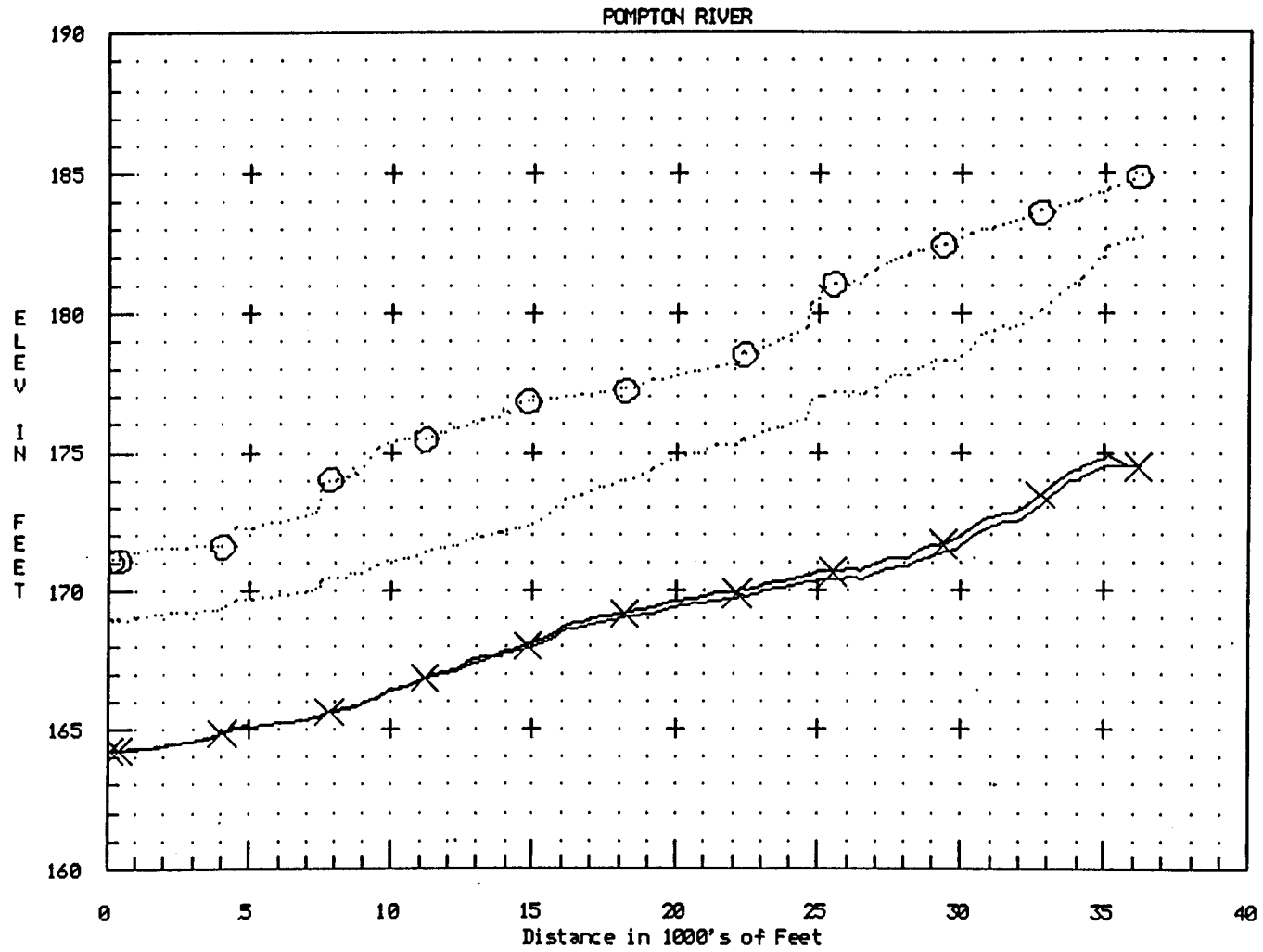


Figure C-113p

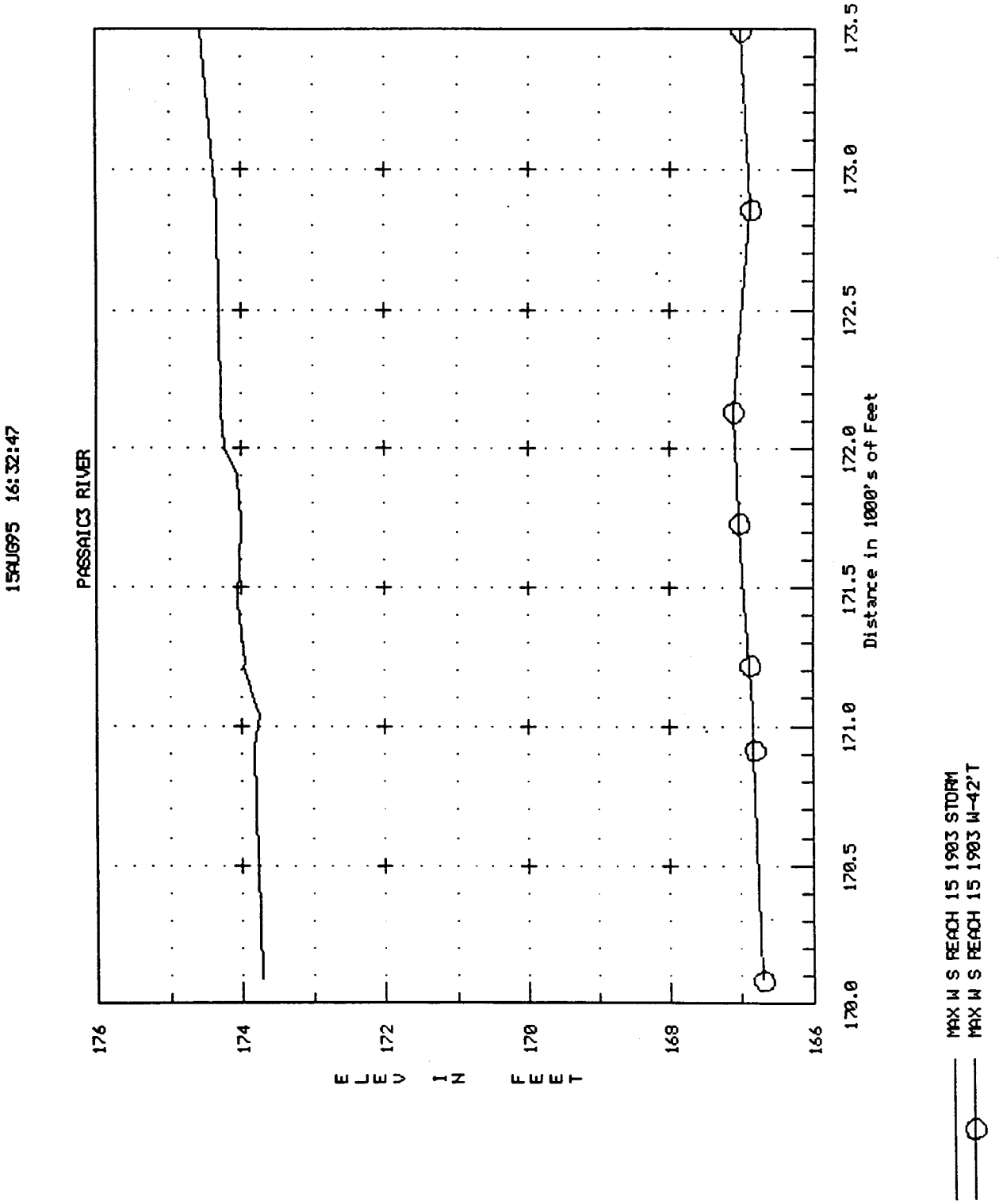


Figure C-113q

07AUG95 10:12:24

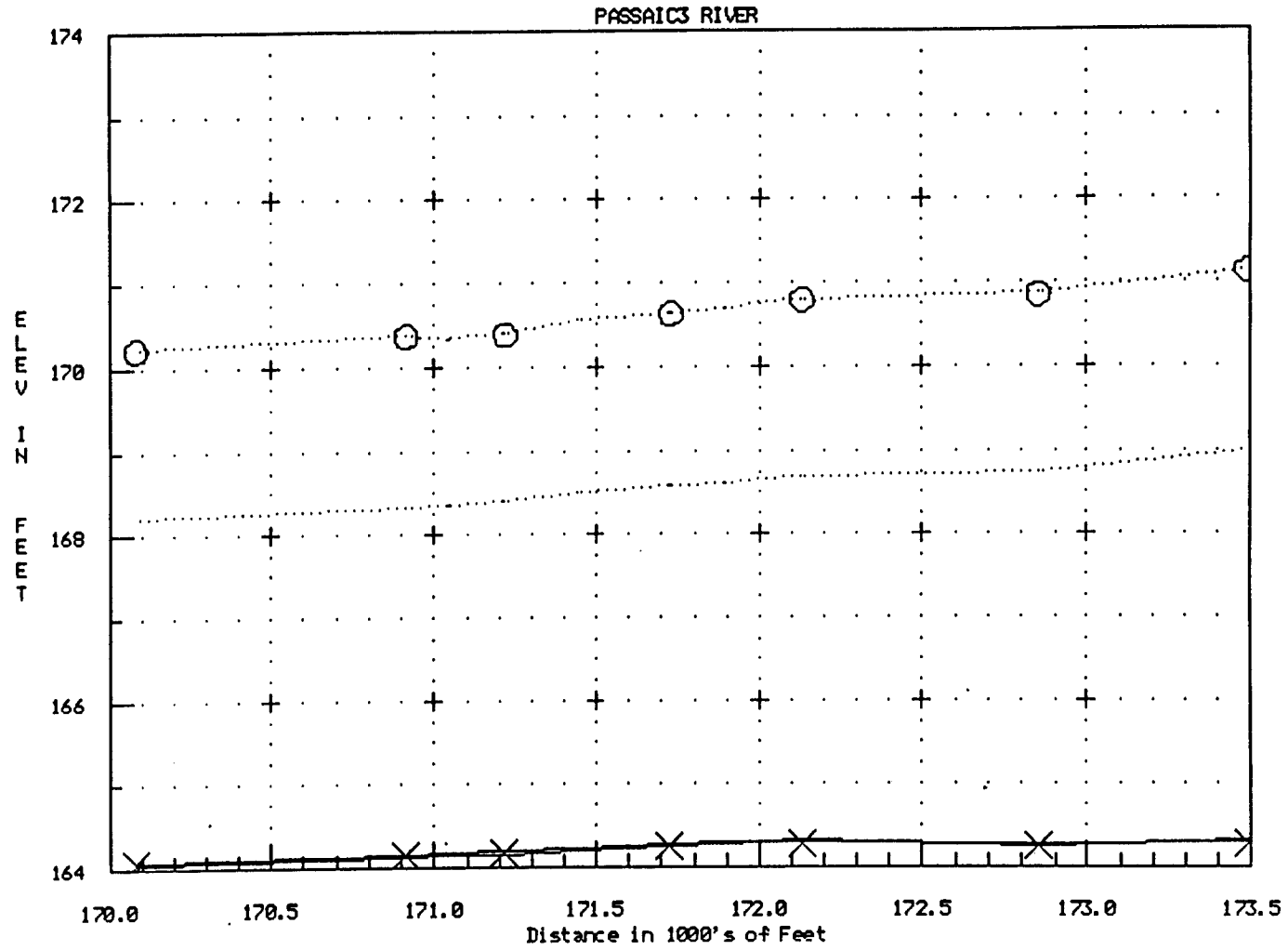


Figure C-113r

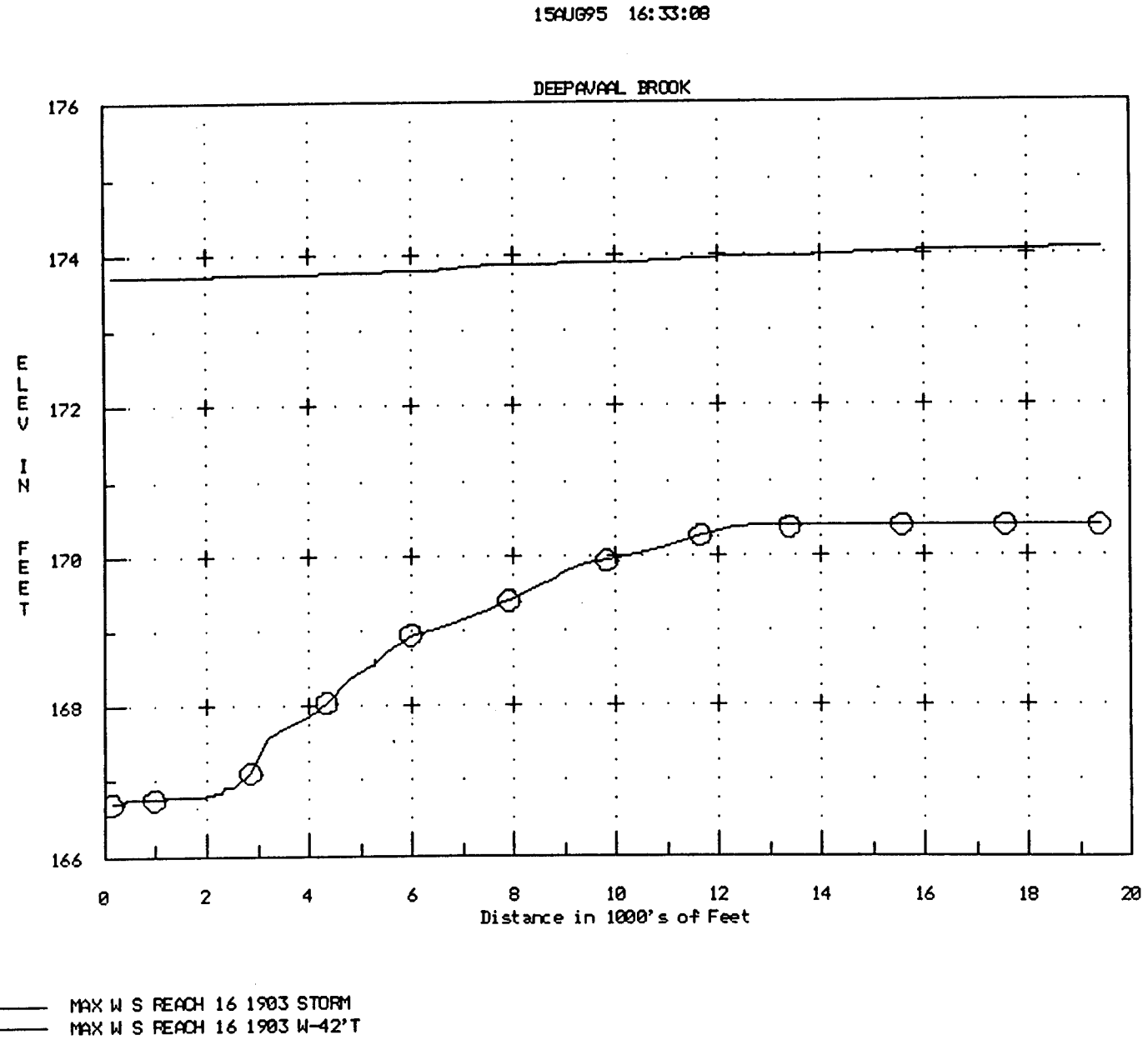


Figure C-113s

07AUG95 10:11:18

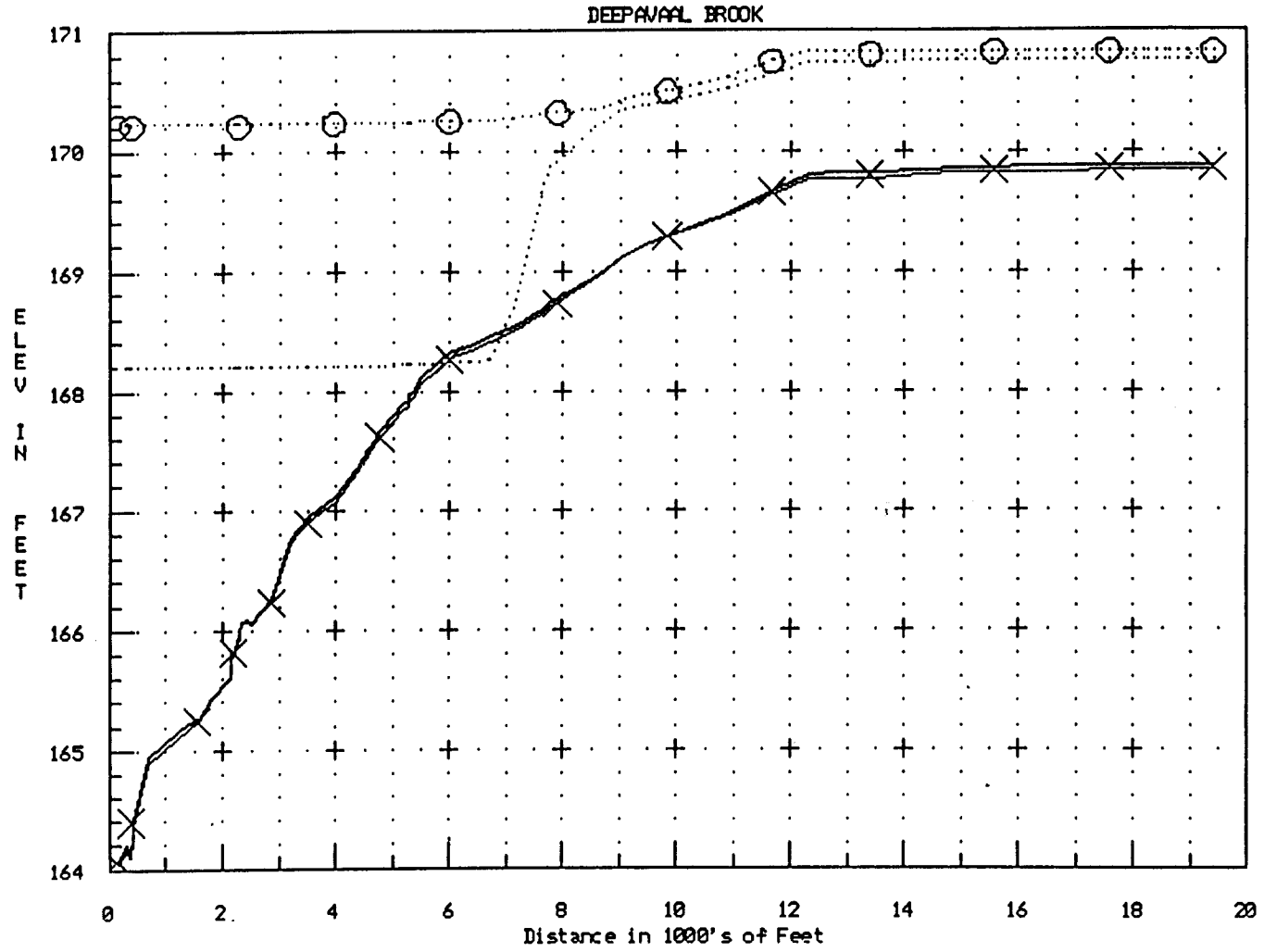


Figure C-113t

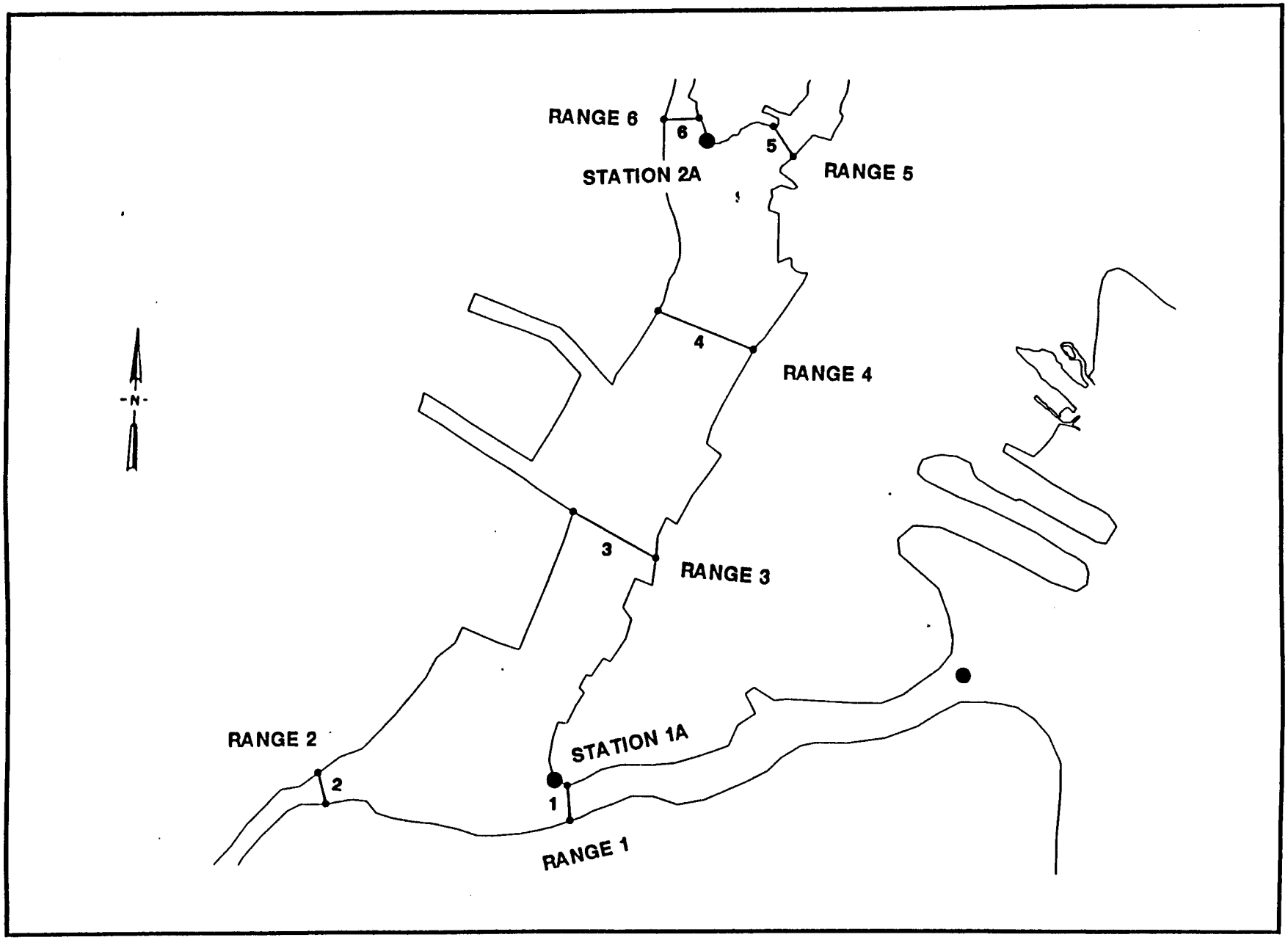


Figure C-114

Station Locations for 1994 Data Collection

CH3D Model - Overall Grid

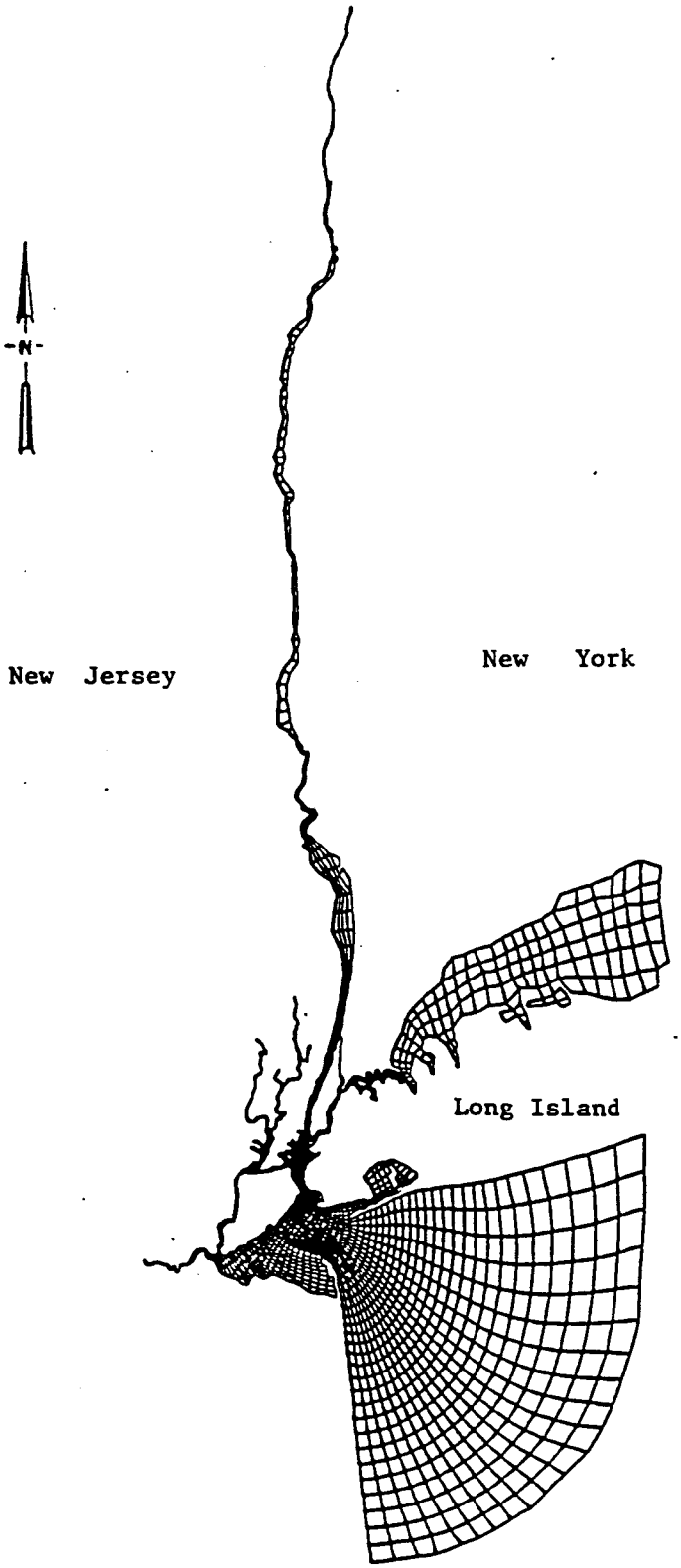
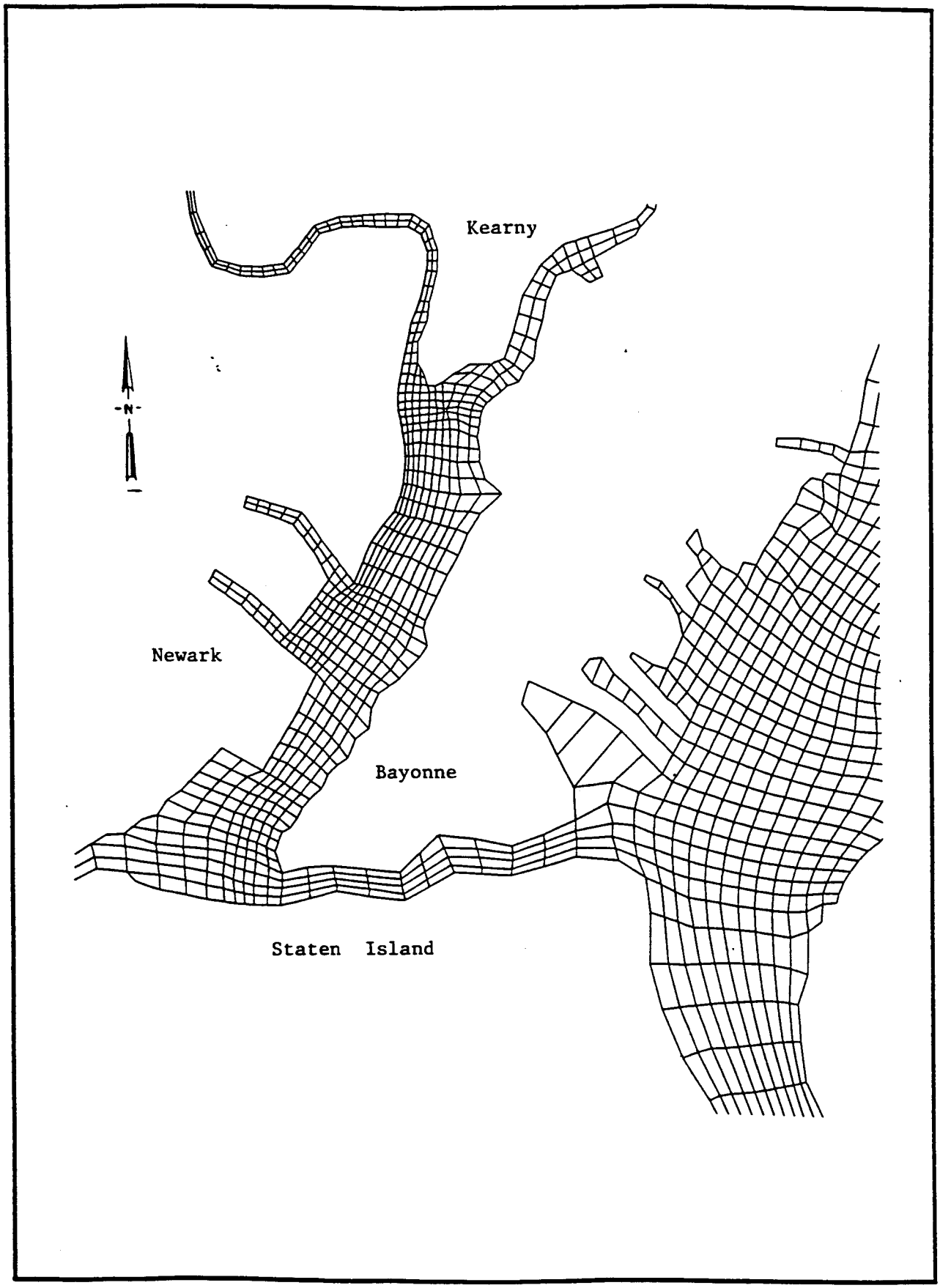
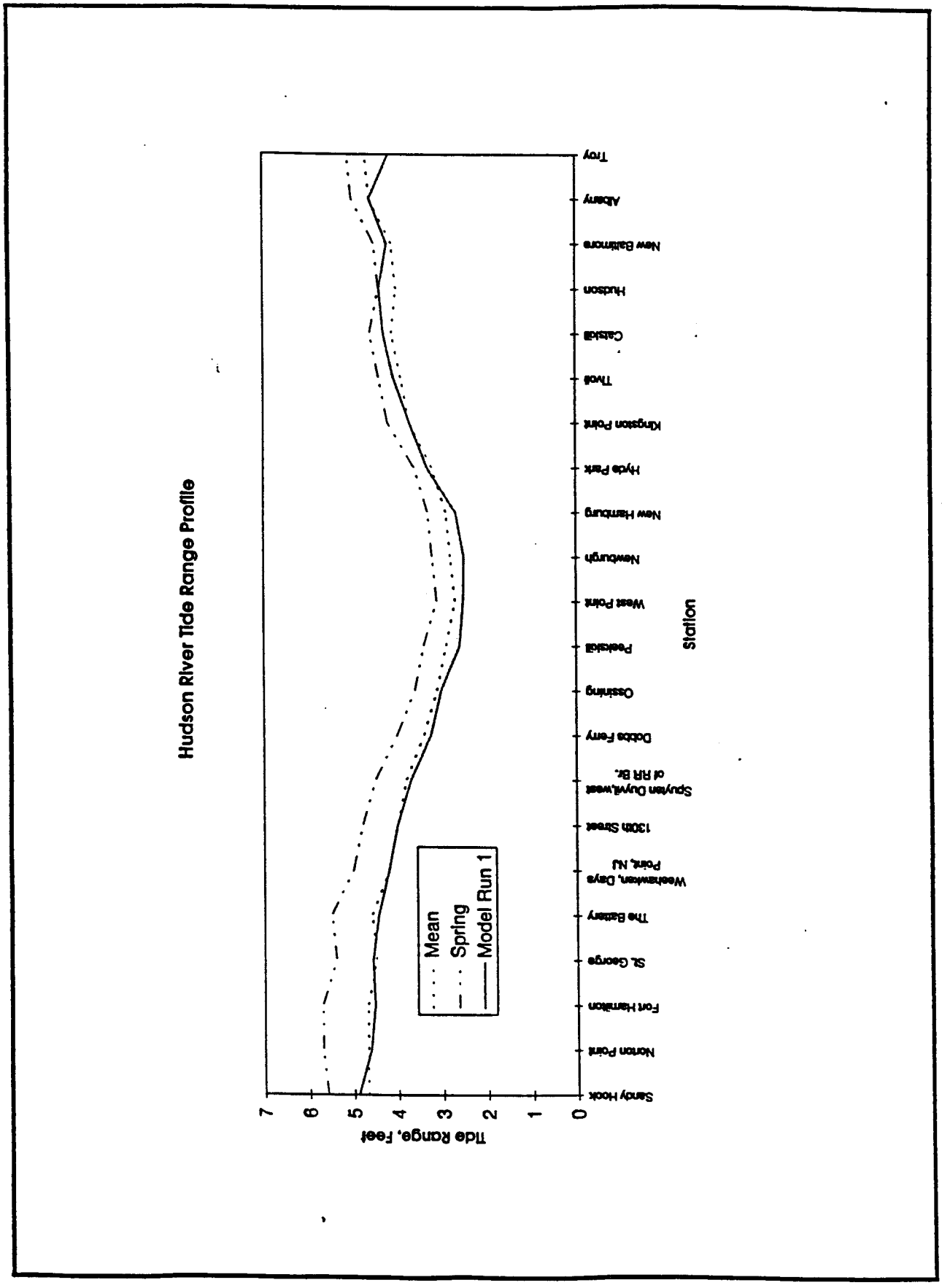


Figure C-115



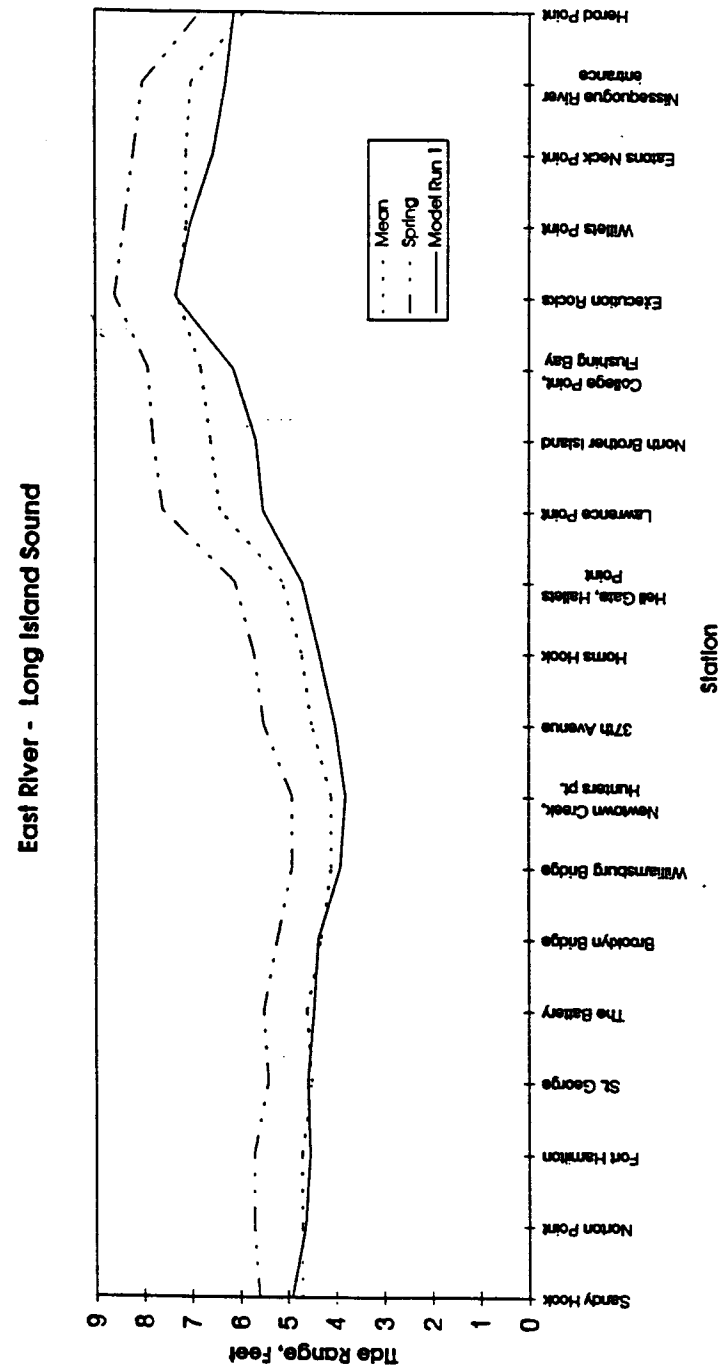
CH3D Grid Detail in Newark Bay

Figure C-116



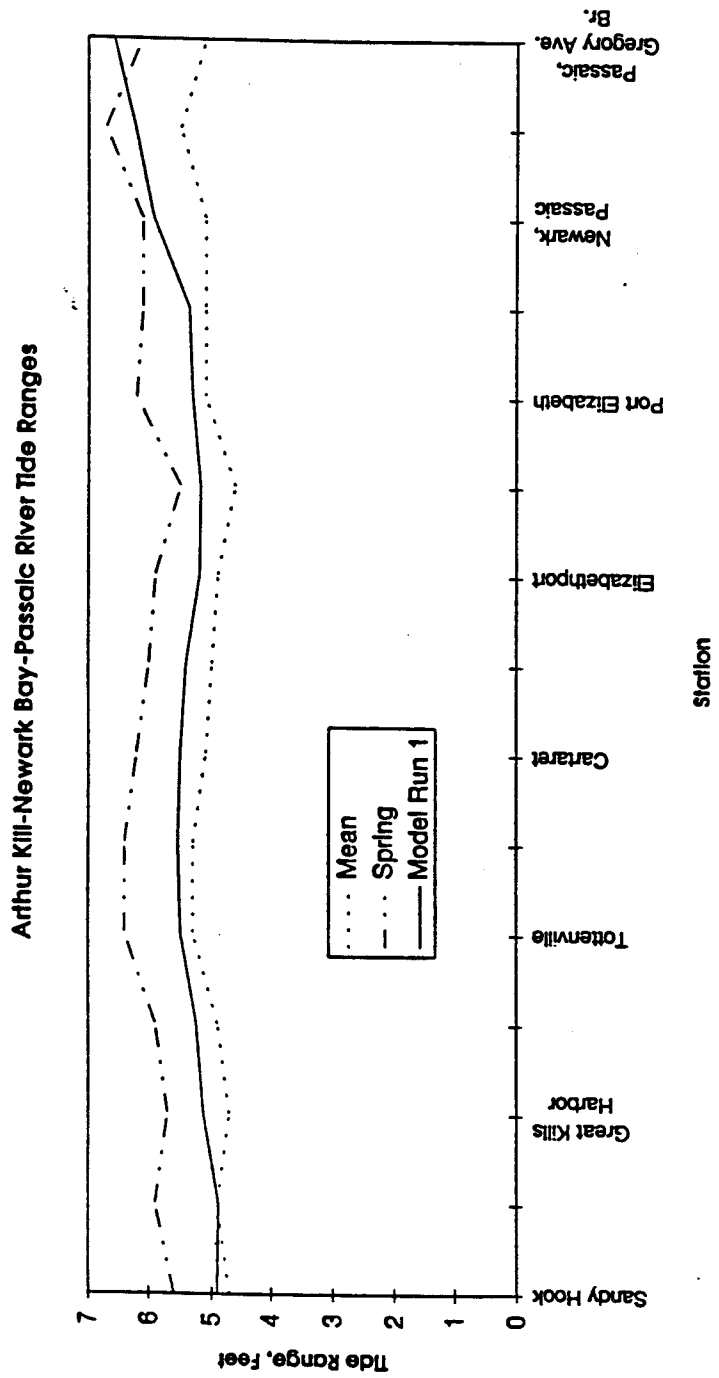
Tidal Amplitude Verification for Hudson River

Figure C-117



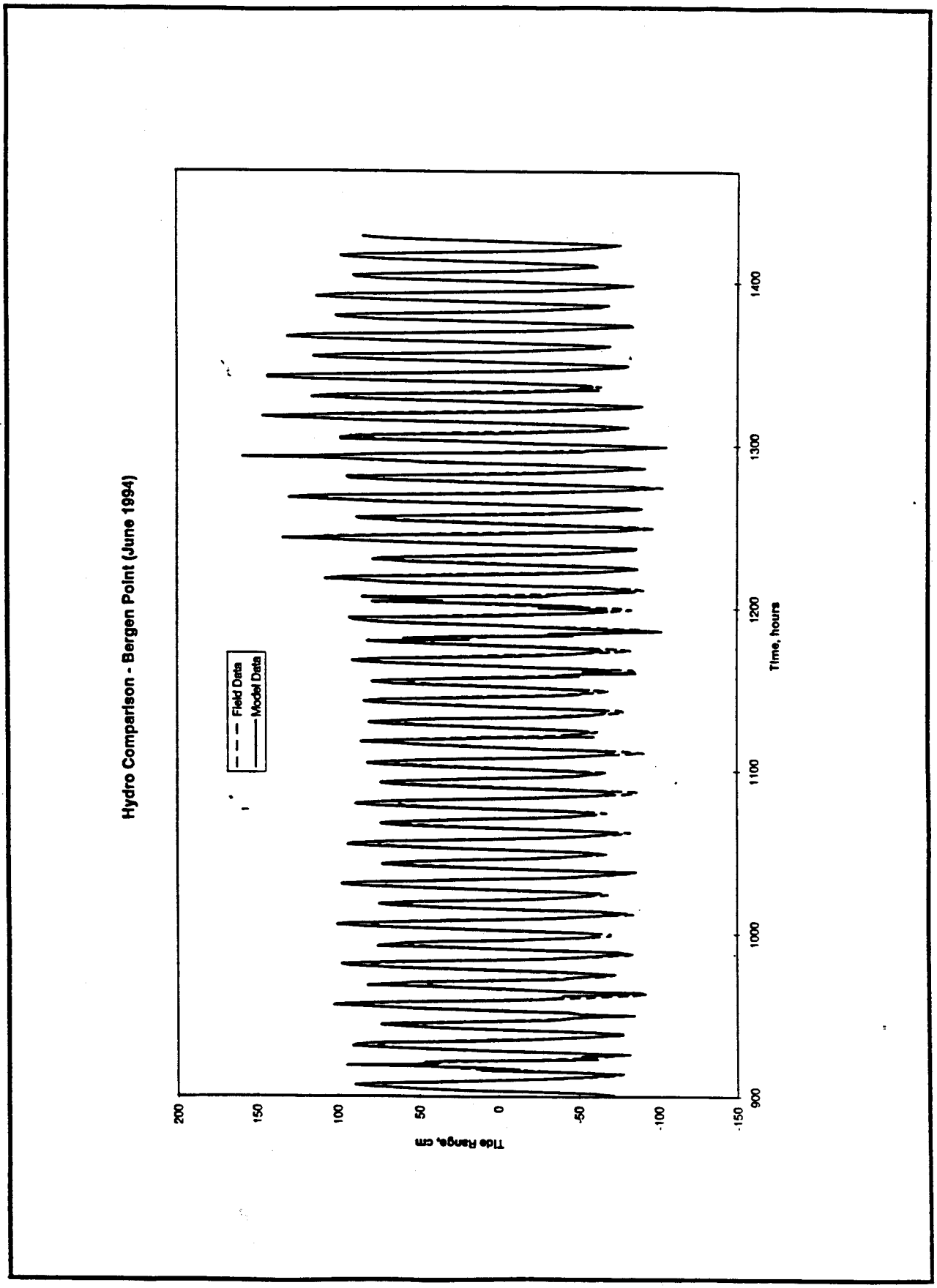
Tidal Amplitude Verification for East River

Figure C-118



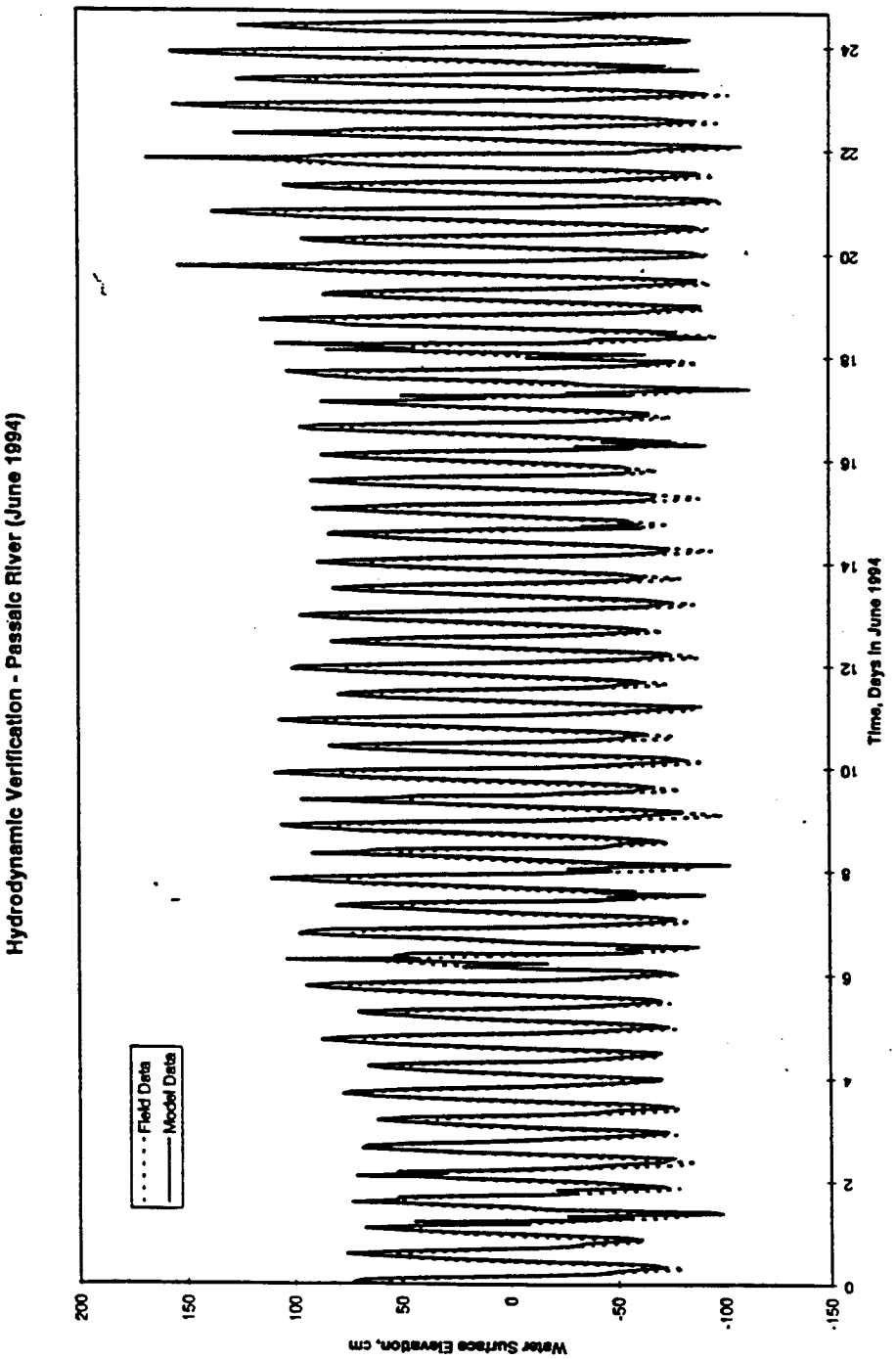
Tidal Amplitude Verification for Arthur Kill and Passaic River

Figure C-119



CH3D tidal Verification at Bergen Point

Figure C-120



CH3D tidal Verification at Kearny Point

Figure C-121

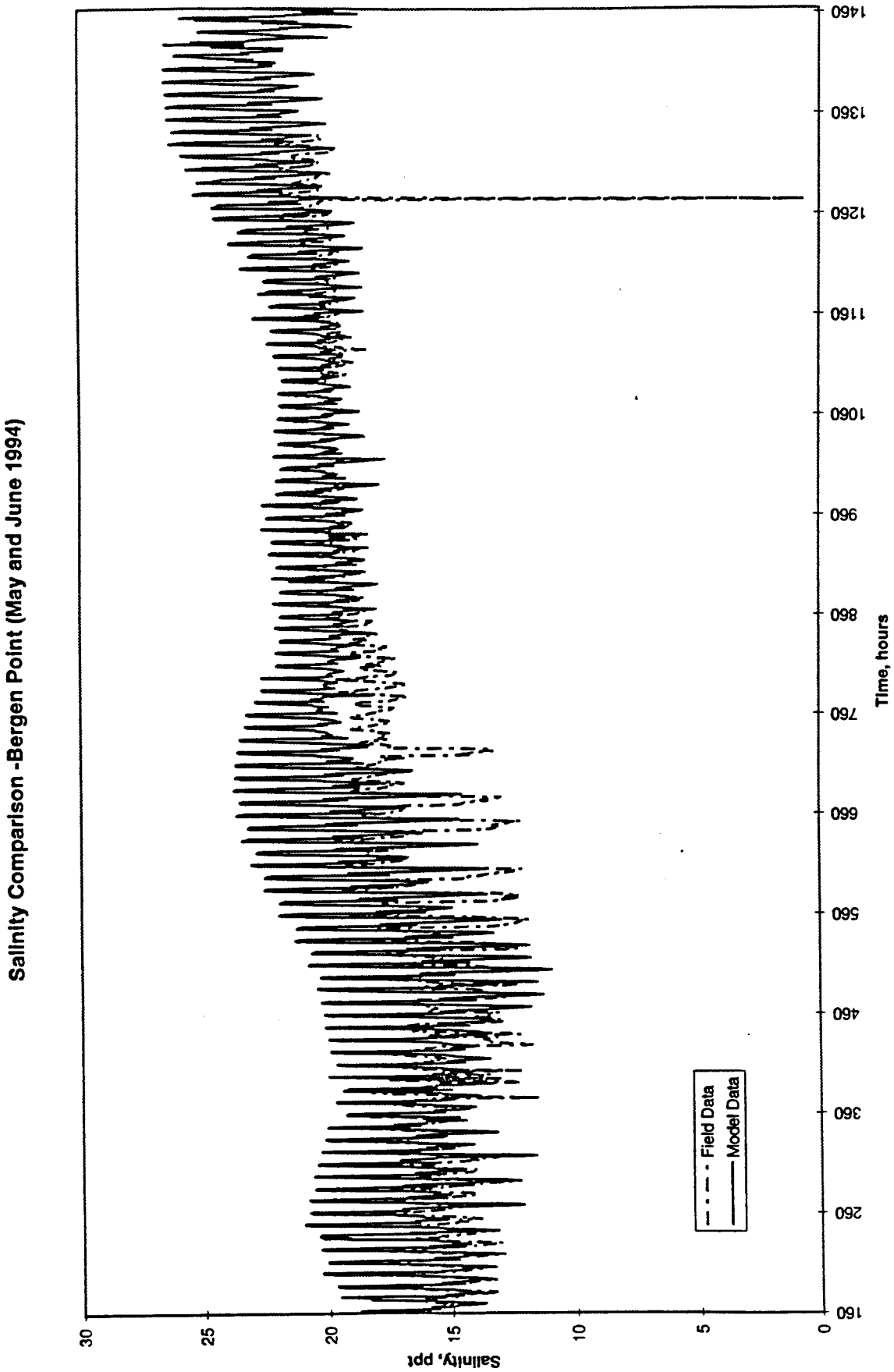
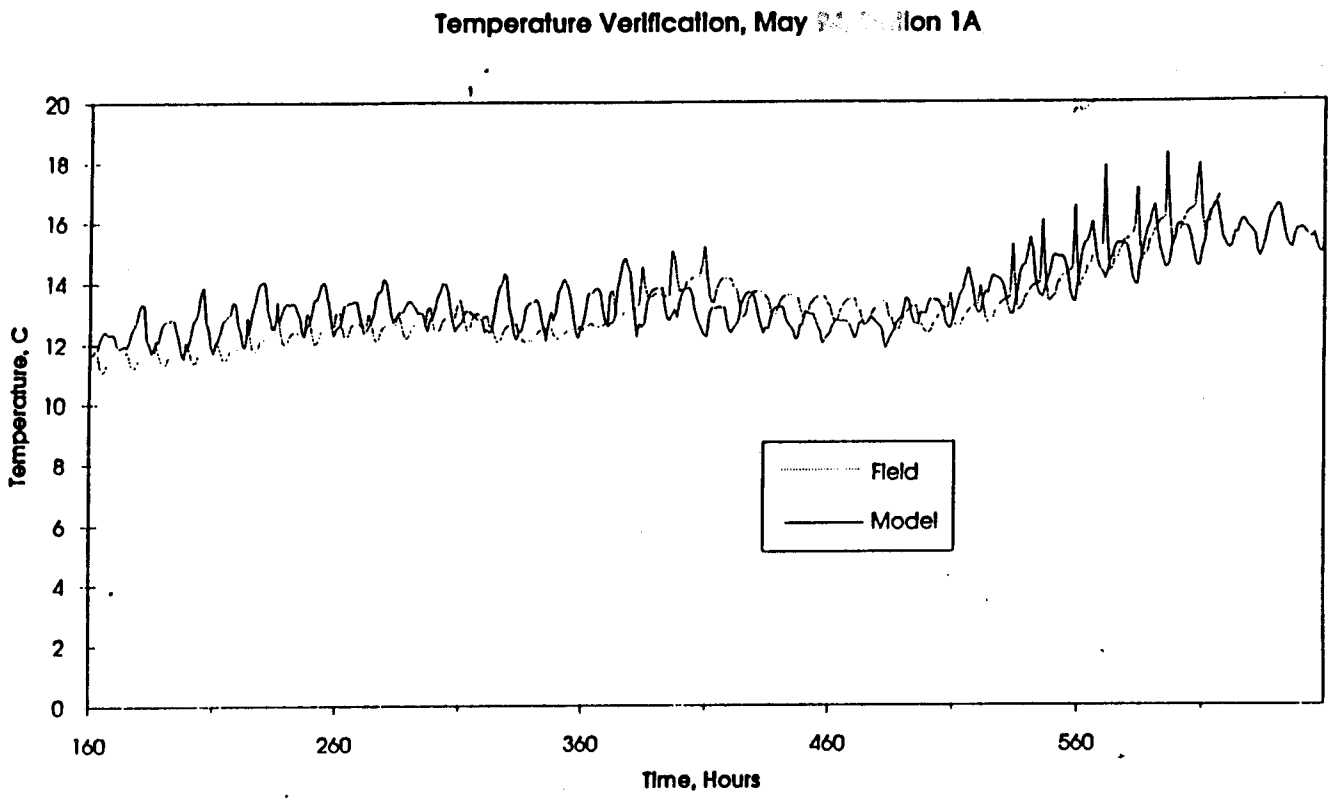
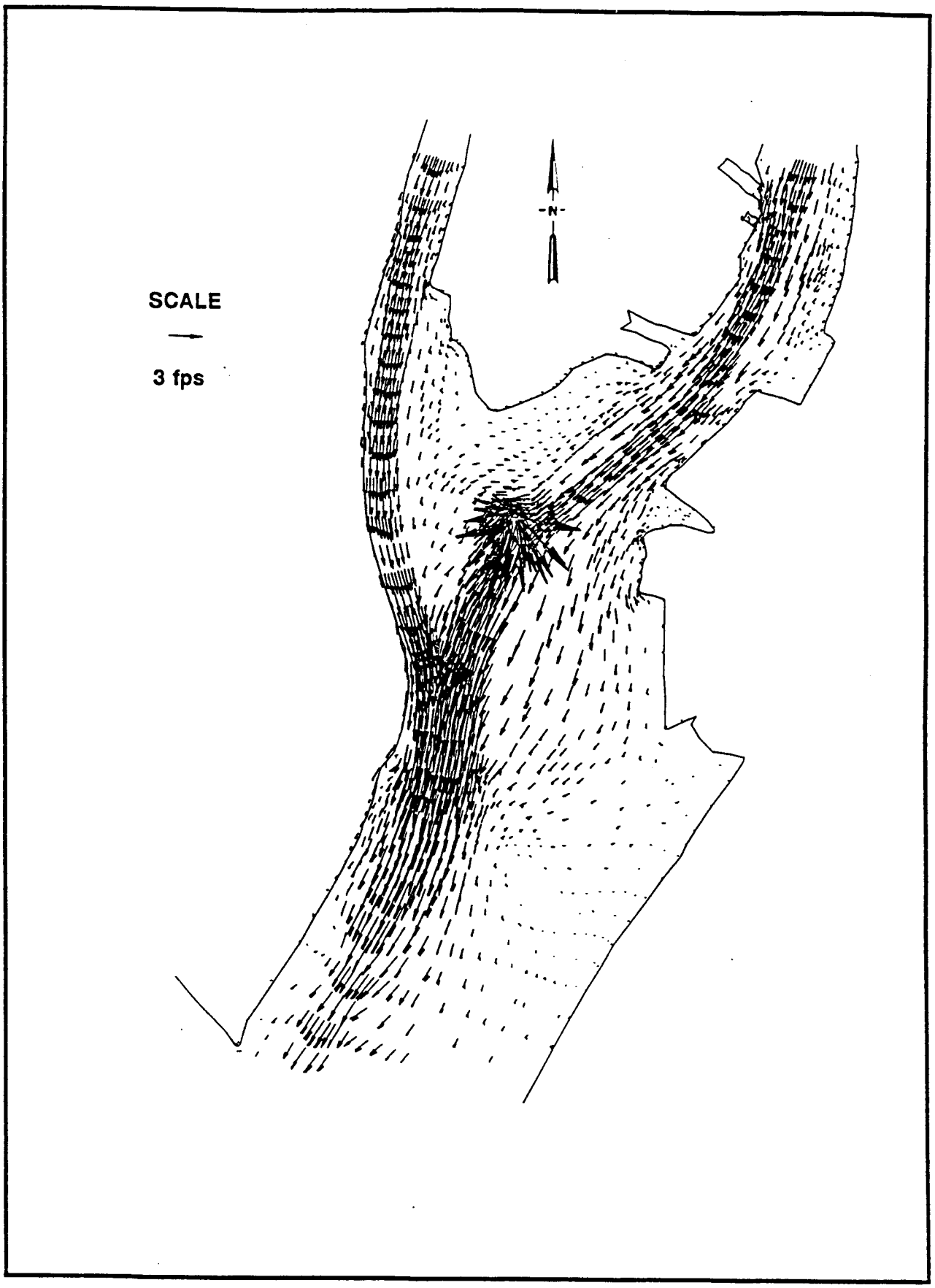

Salinity Comparison -Bergen Point (May and June 1994)
Salinity, ppt
30
25
20
15
10
5
0
Time, hours
160
260
360
460
560
660
760
860
960
1060
1160
1260
1360
1460
Field Data
Model Data

Figure C-122



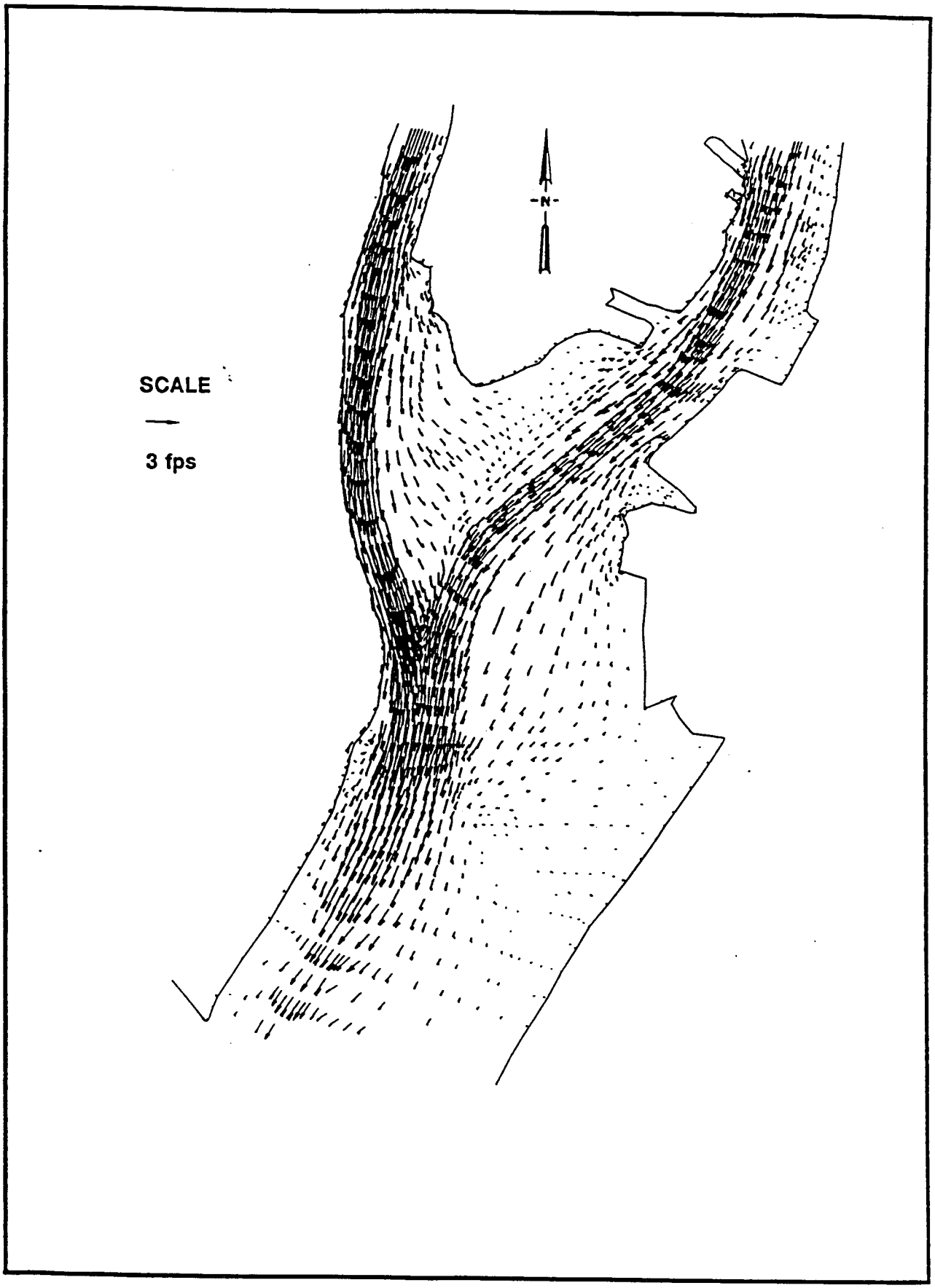
Temperature Verification at Bergen Pt

Figure C-123

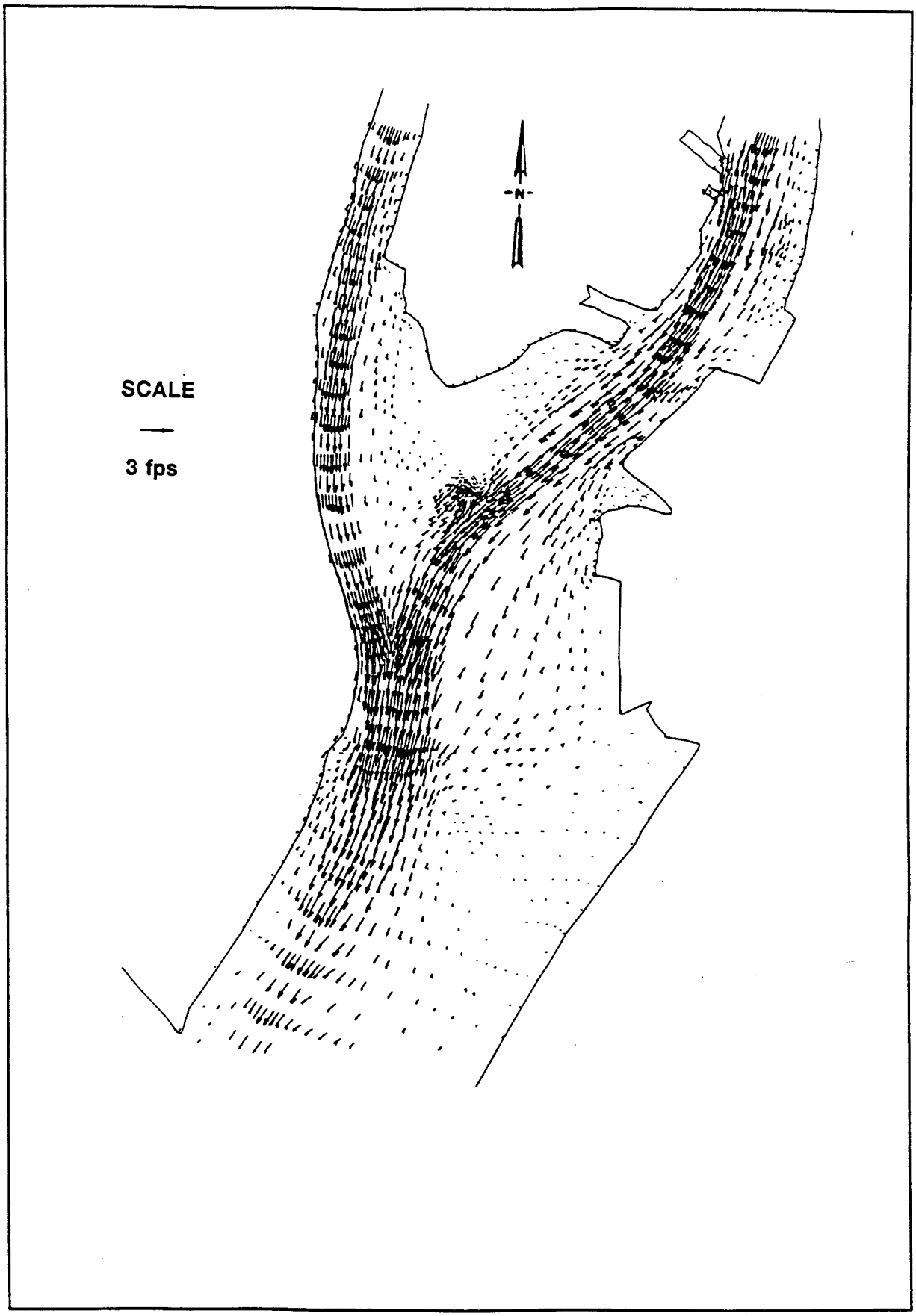


RMA2 100-yr Peak Currents with Tunnel

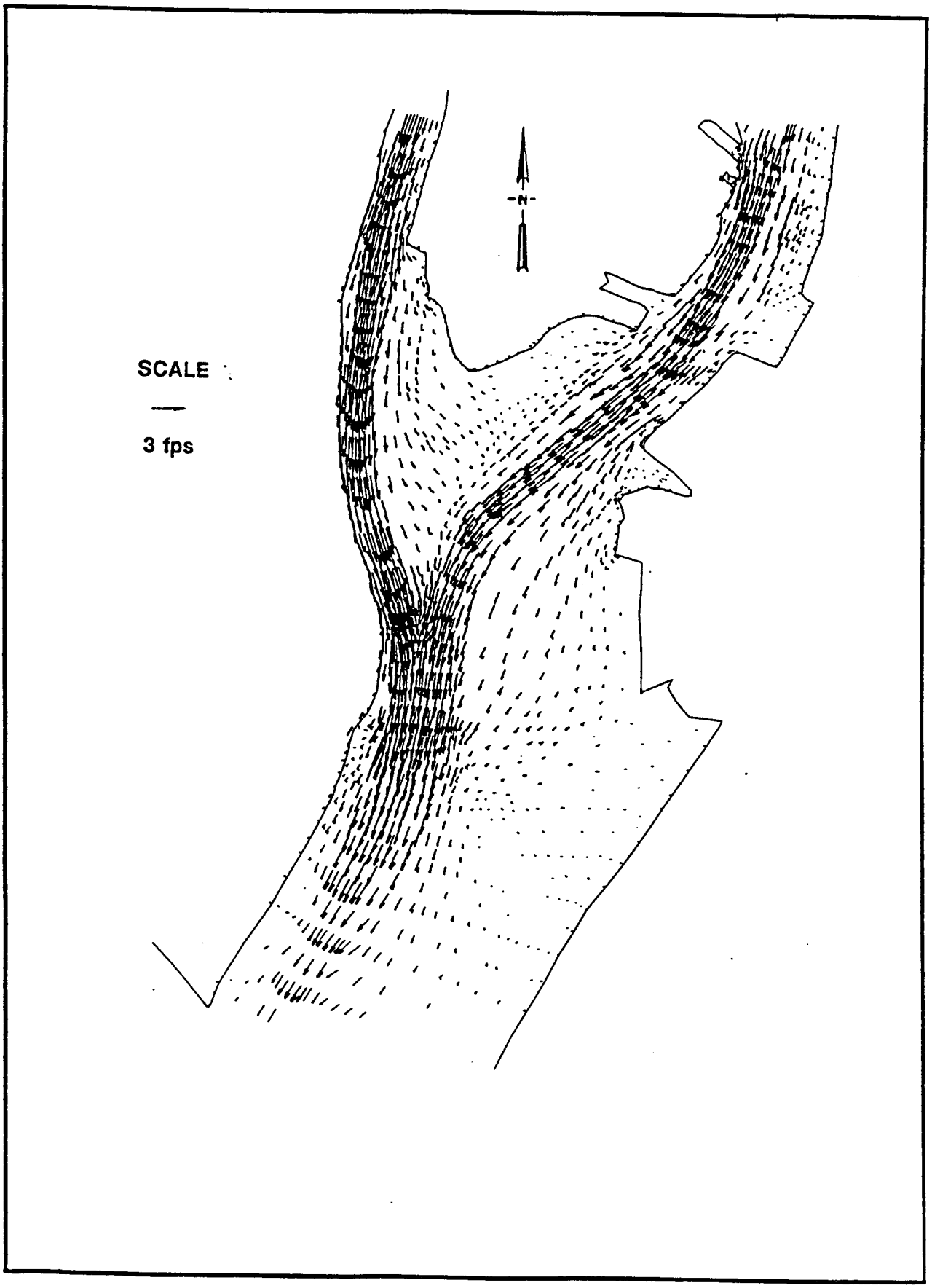
Figure C-124



RMA2 100-yr Peak Base Currents **Figure C-125**

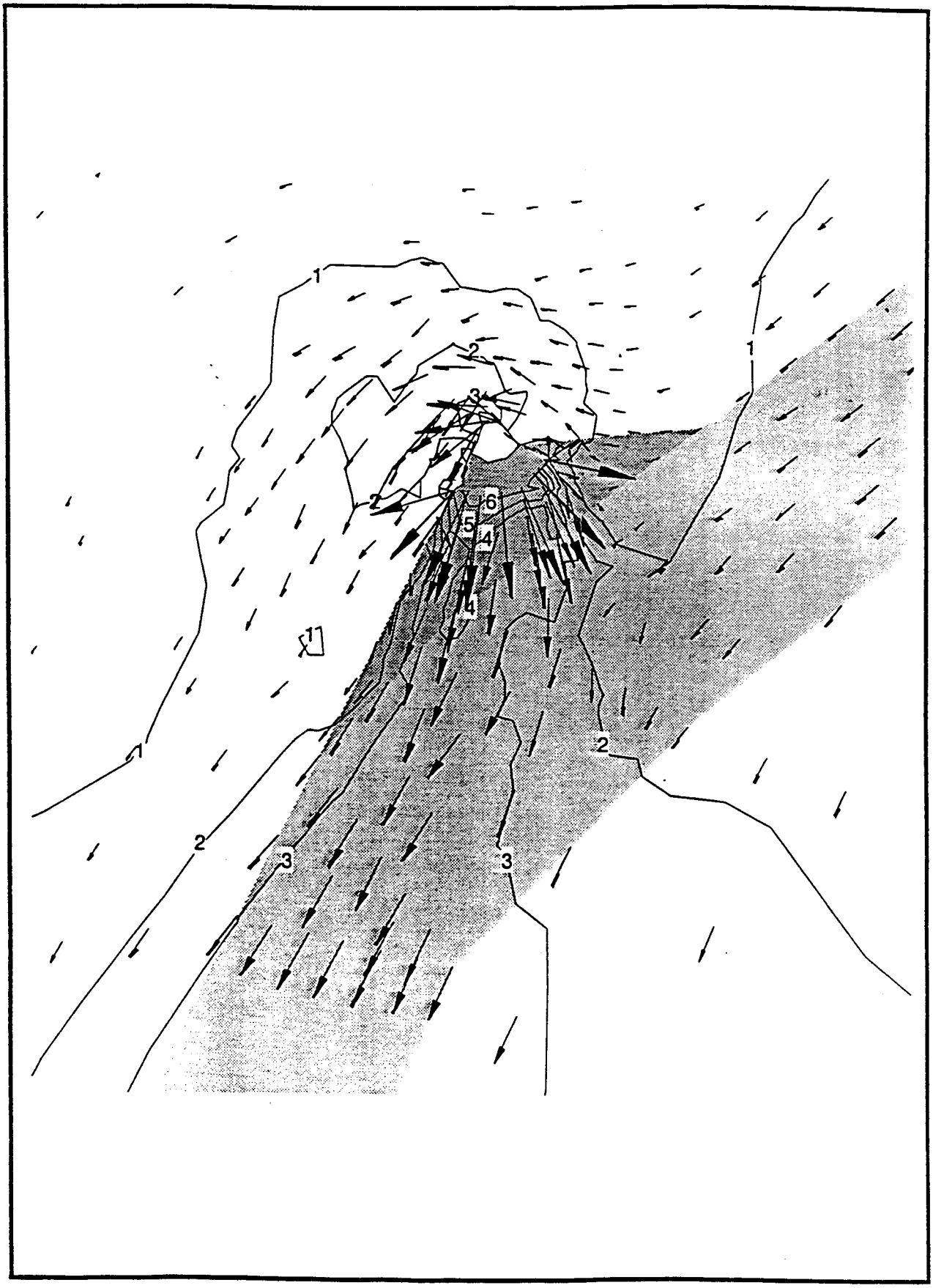


RMA2 2-yr Peak Currents with Tunnel **Figure C-126**



RMA2 2-yr Peak Base Currents

Figure C-127



Plan Peak Currents at Outlet Structure for 100-yr Flood

Passaic River at Little Falls

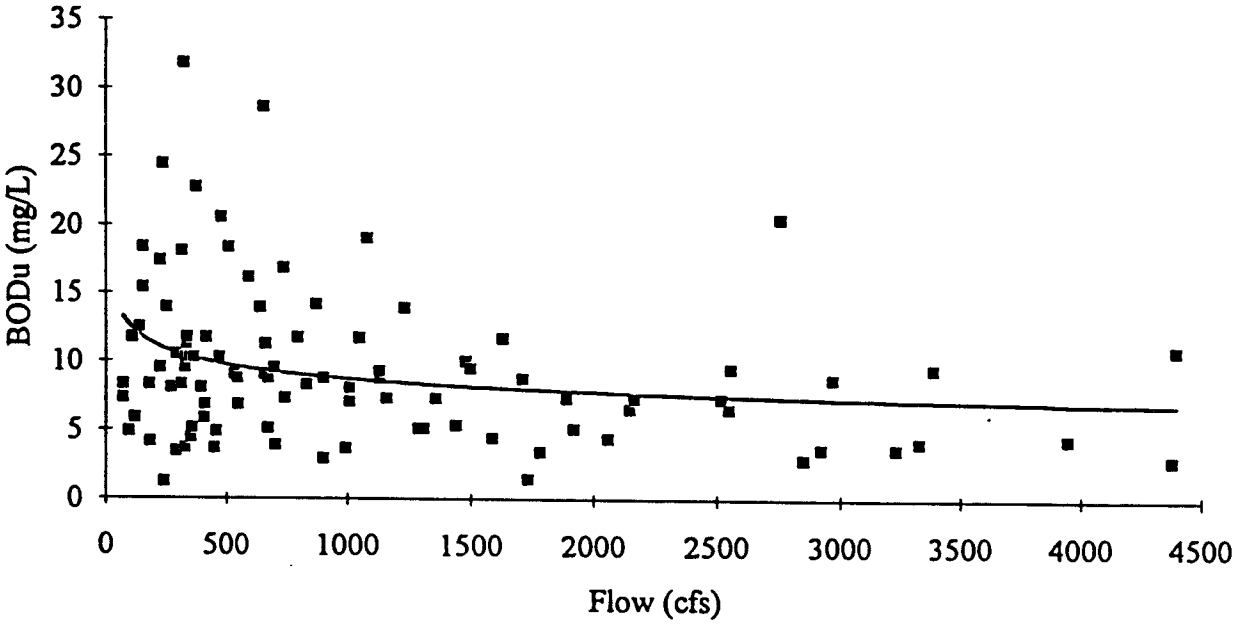


Figure C-129

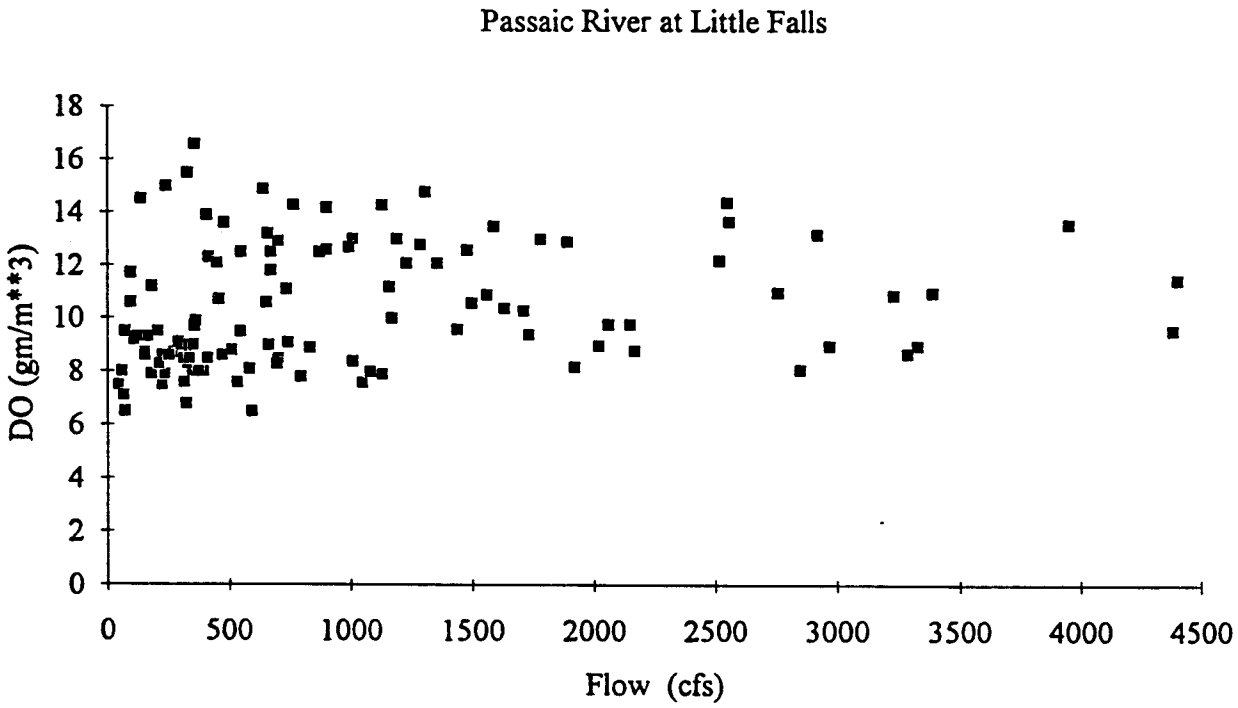
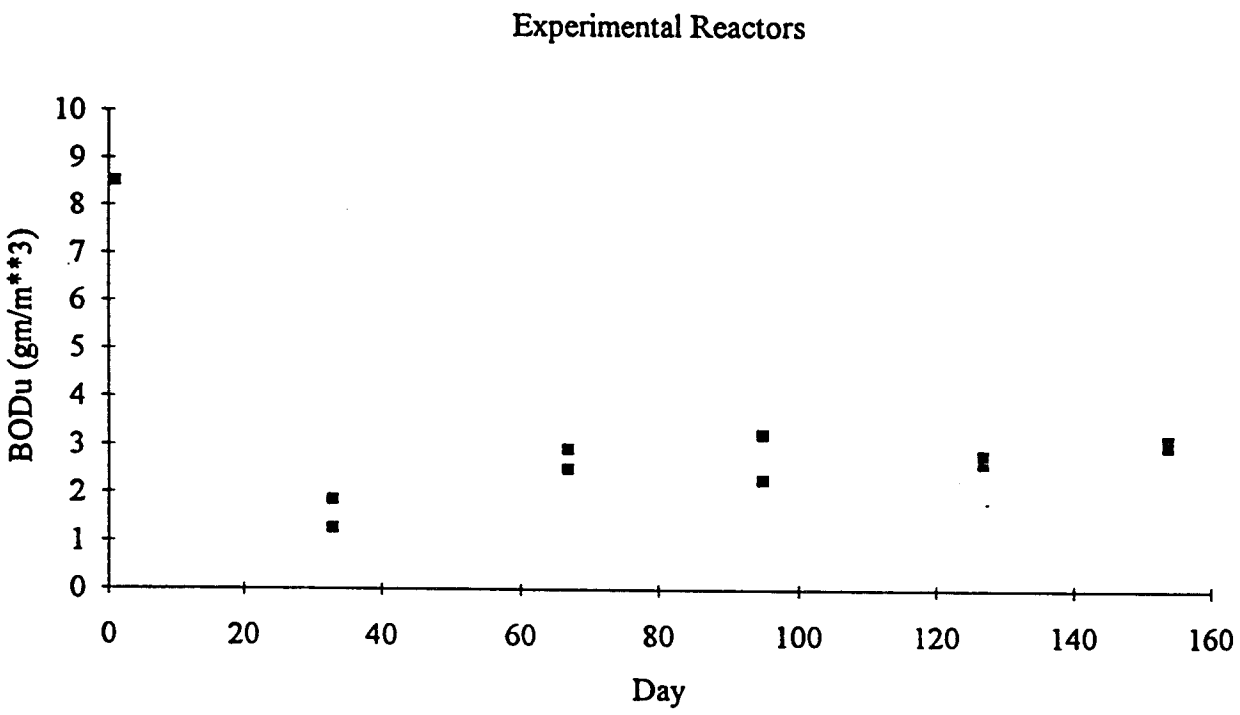

Passaic River at Little Falls
DO (gm/m**3)
18
16
14
12
10
8
6
4
2
0
0
500
1000
1500
2000
2500
3000
3500
4000
4500
Flow (cfs)

Figure C-130


Experimental Reactors
BODu (gm/m**3)
10
9
8
7
6
5
4
3
2
1
0
0
20
40
60
80
100
120
140
160
Day

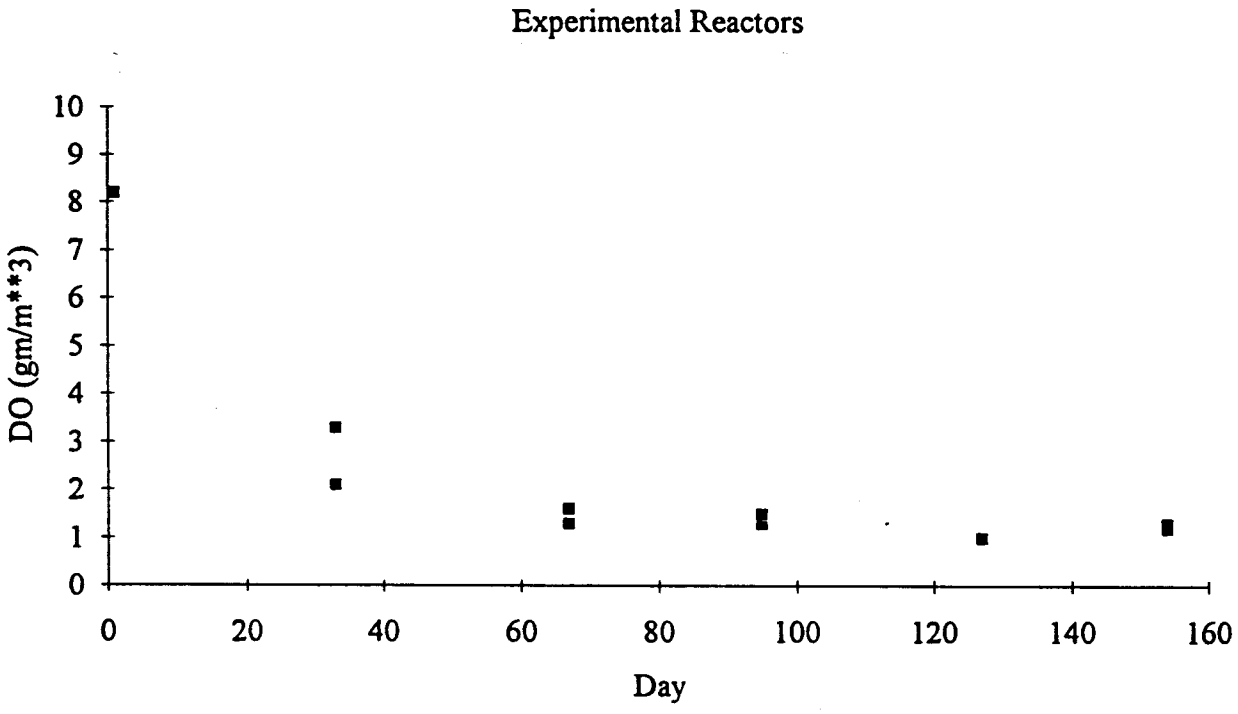

Experimental Reactors
DO (gm/m**3)
10
9
8
7
6
5
4
3
2
1
0
0
20
40
60
80
100
120
140
160
Day

Figure C-132

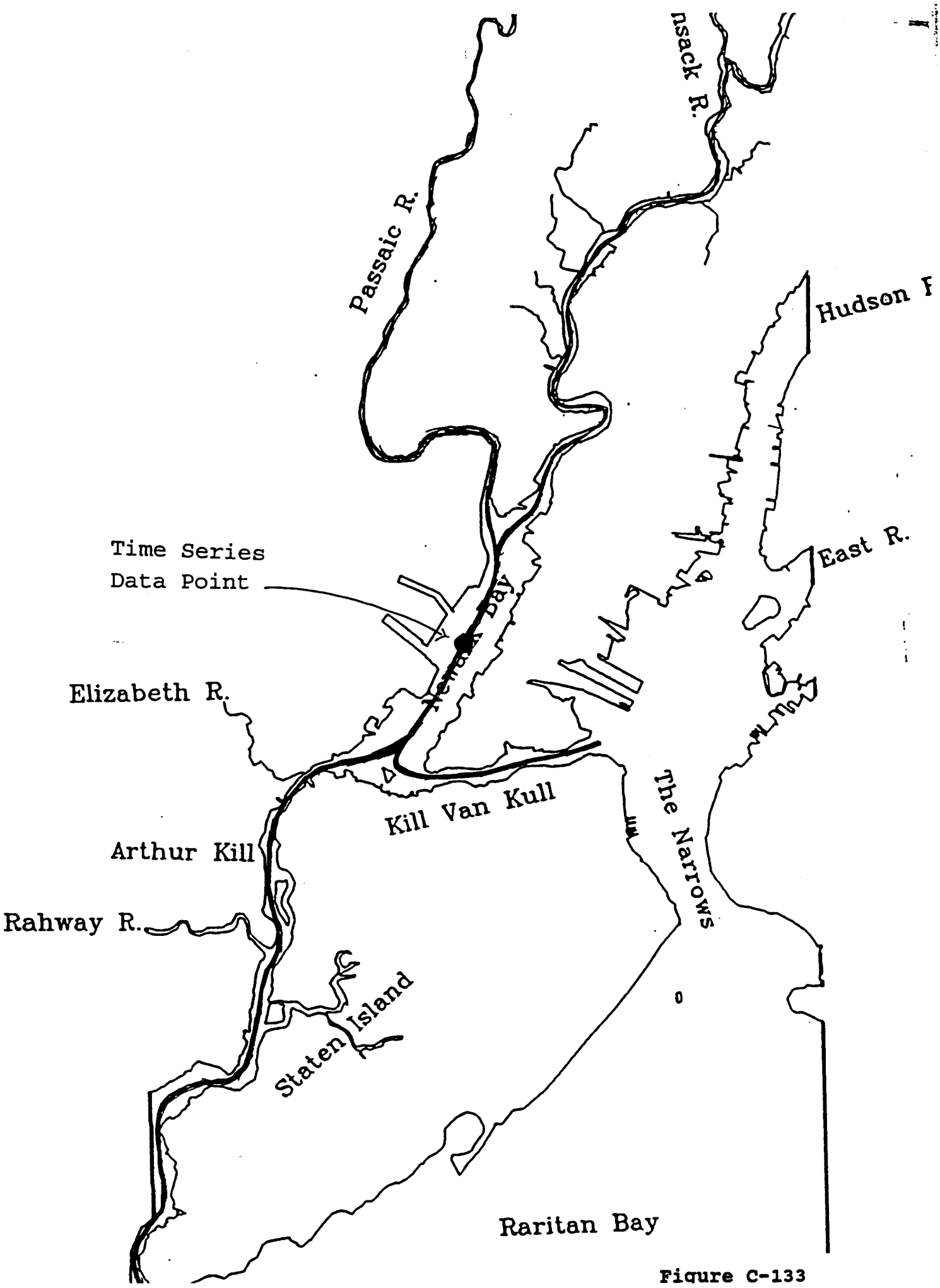

nsack R.
Passaic R.
Hudson R
East R.
Time Series
Data Point
Newark Bay
Elizabeth R.
Kill Van Kull
The Narrows
Arthur Kill
Rahway R.
Staten Island
Raritan Bay

Figure C-133

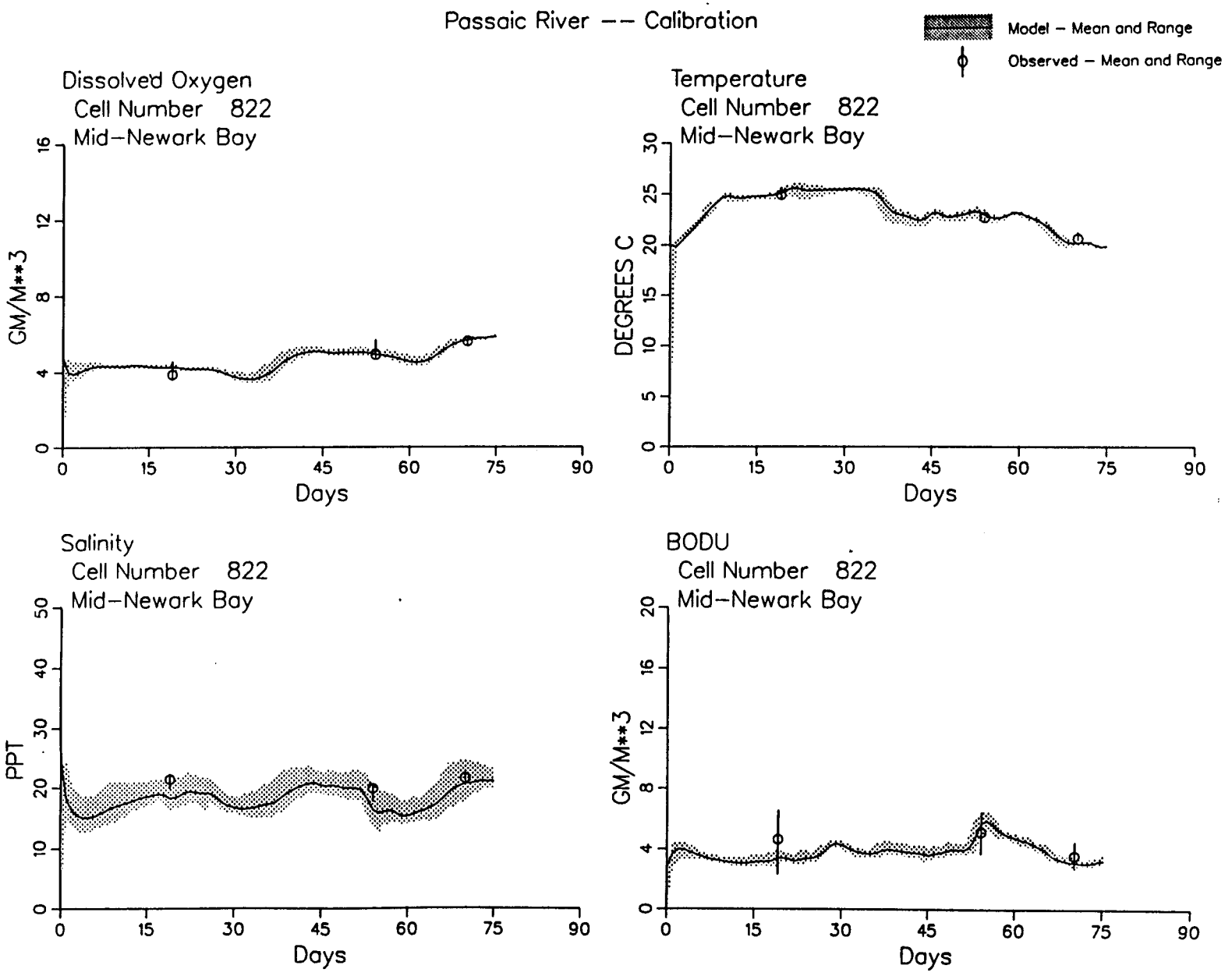

Passaic River -- Calibration
Model – Mean and Range
Observed – Mean and Range
Dissolved Oxygen
Cell Number 822
Mid–Newark Bay
GM/M**3
Days
Temperature
Cell Number 822
Mid–Newark Bay
DEGREES C
Days
Salinity
Cell Number 822
Mid–Newark Bay
PPT
Days
BODU
Cell Number 822
Mid–Newark Bay
GM/M**3
Days

Figure C-134

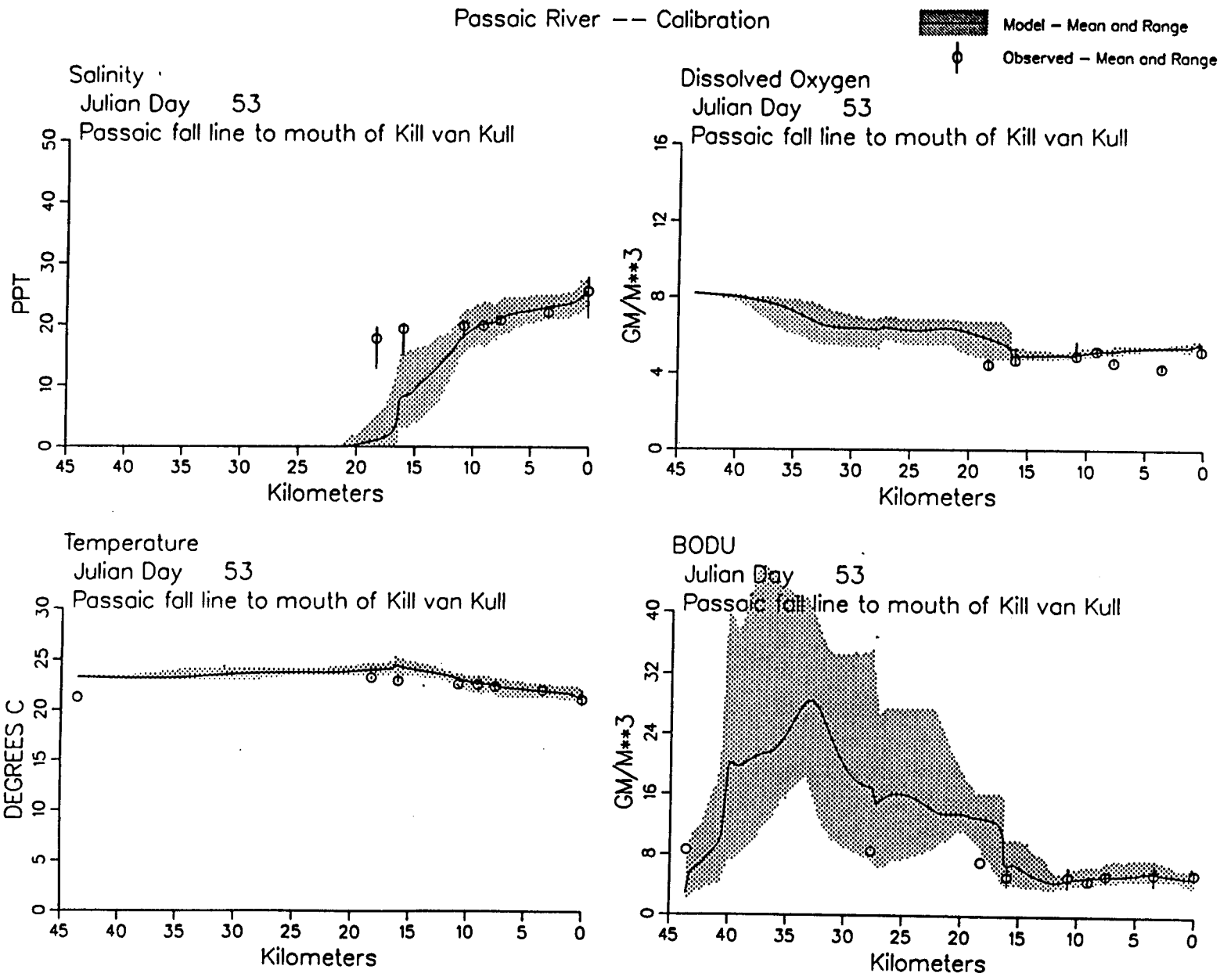

Passaic River -- Calibration
Model – Mean and Range
Observed – Mean and Range
Salinity
Julian Day 53
Passaic fall line to mouth of Kill van Kull
PPT
Kilometers
Dissolved Oxygen
Julian Day 53
Passaic fall line to mouth of Kill van Kull
GM/M**3
Kilometers
Temperature
Julian Day 53
Passaic fall line to mouth of Kill van Kull
DEGREES C
Kilometers
BODU
Julian Day 53
Passaic fall line to mouth of Kill van Kull
GM/M**3
Kilometers

Figure C-135

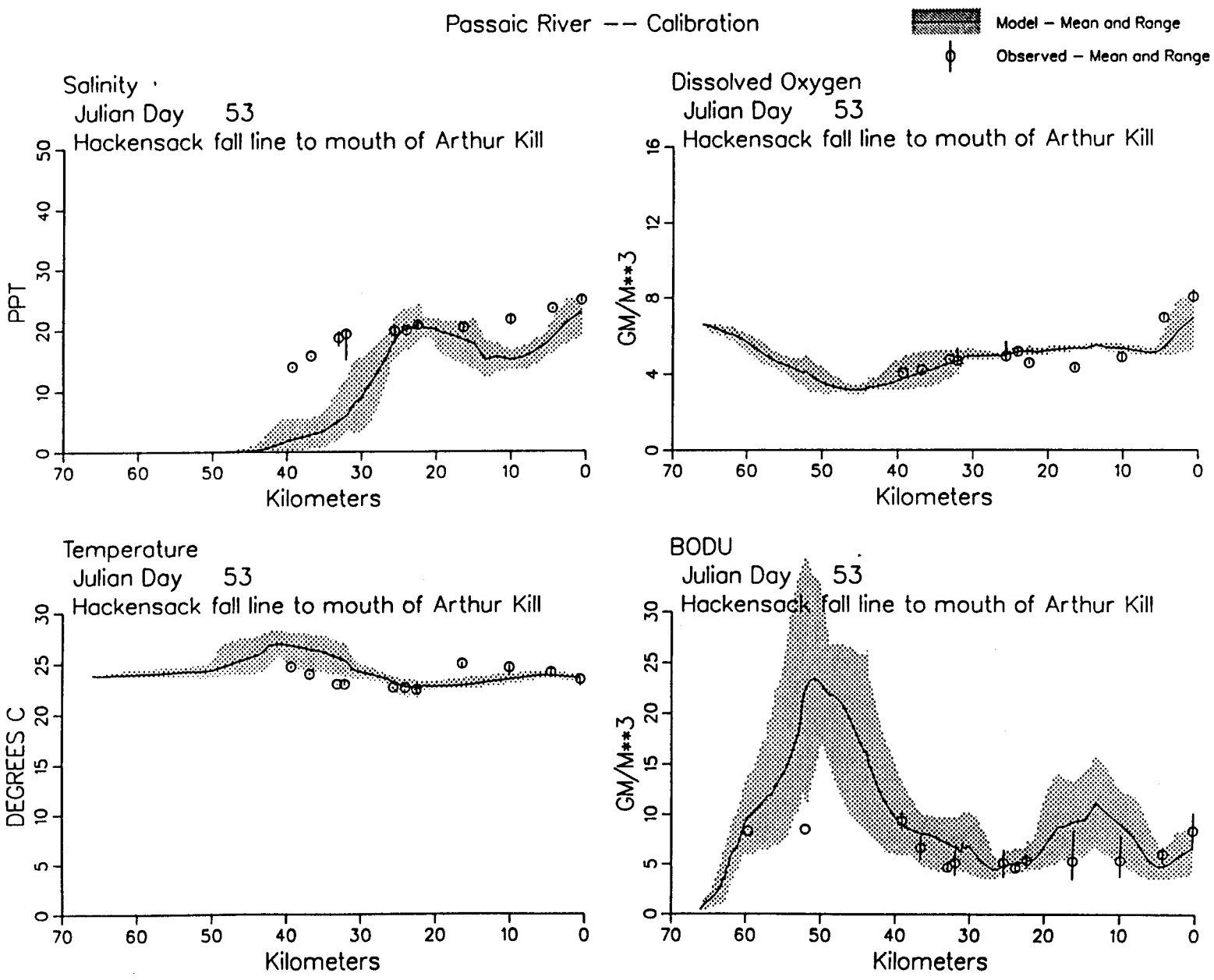

Passaic River -- Calibration
Model – Mean and Range
Observed – Mean and Range
Salinity
Julian Day 53
Hackensack fall line to mouth of Arthur Kill
PPT
Kilometers
Dissolved Oxygen
Julian Day 53
Hackensack fall line to mouth of Arthur Kill
GM/M**3
Kilometers
Temperature
Julian Day 53
Hackensack fall line to mouth of Arthur Kill
DEGREES C
Kilometers
BODU
Julian Day 53
Hackensack fall line to mouth of Arthur Kill
GM/M**3
Kilometers

Figure C-136

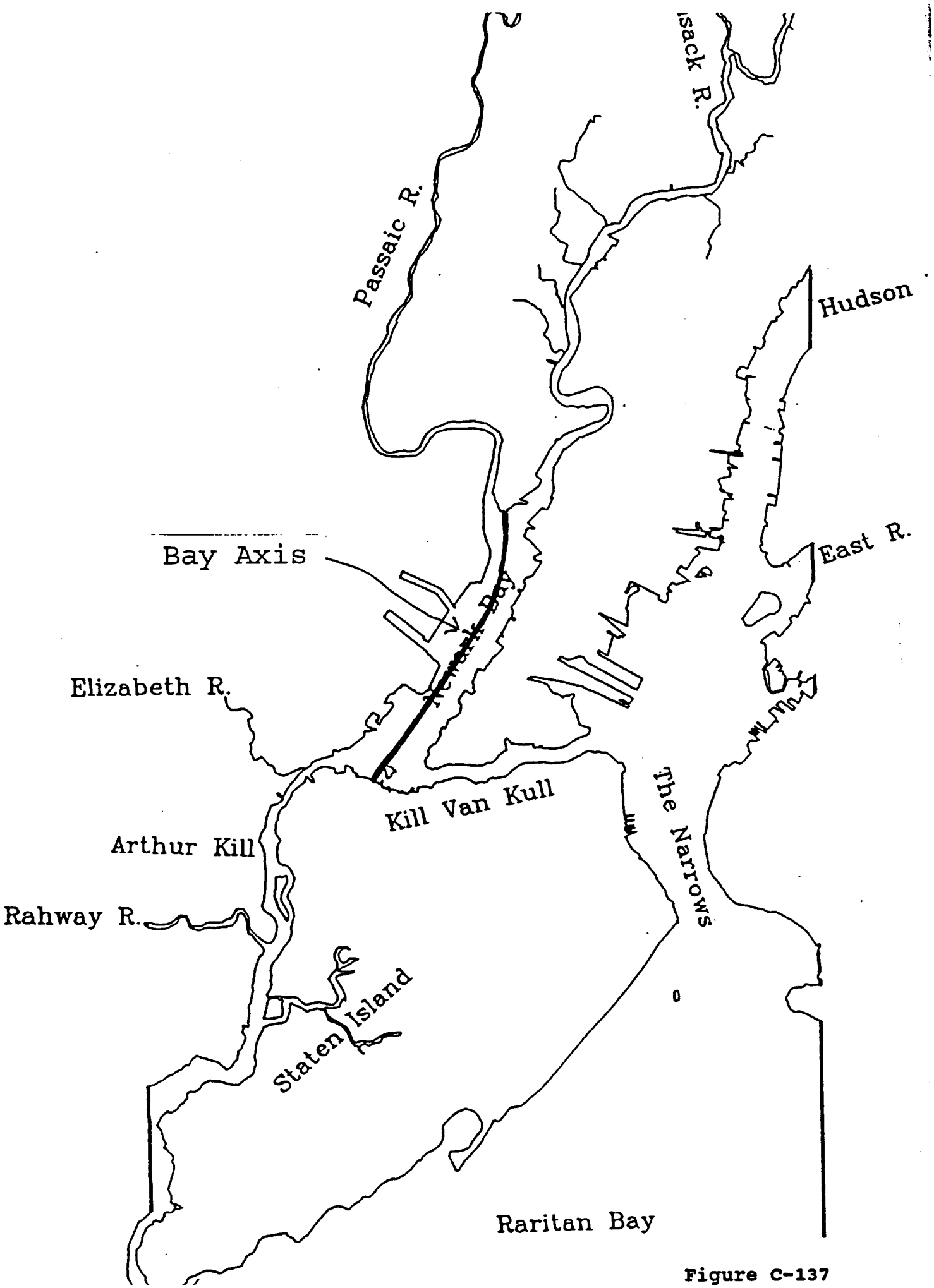

Passaic R.
Hudson
Bay Axis
East R.
Newark Bay
Elizabeth R.
Kill Van Kull
Arthur Kill
The Narrows
Rahway R.
Staten Island
Raritan Bay

Figure C-137

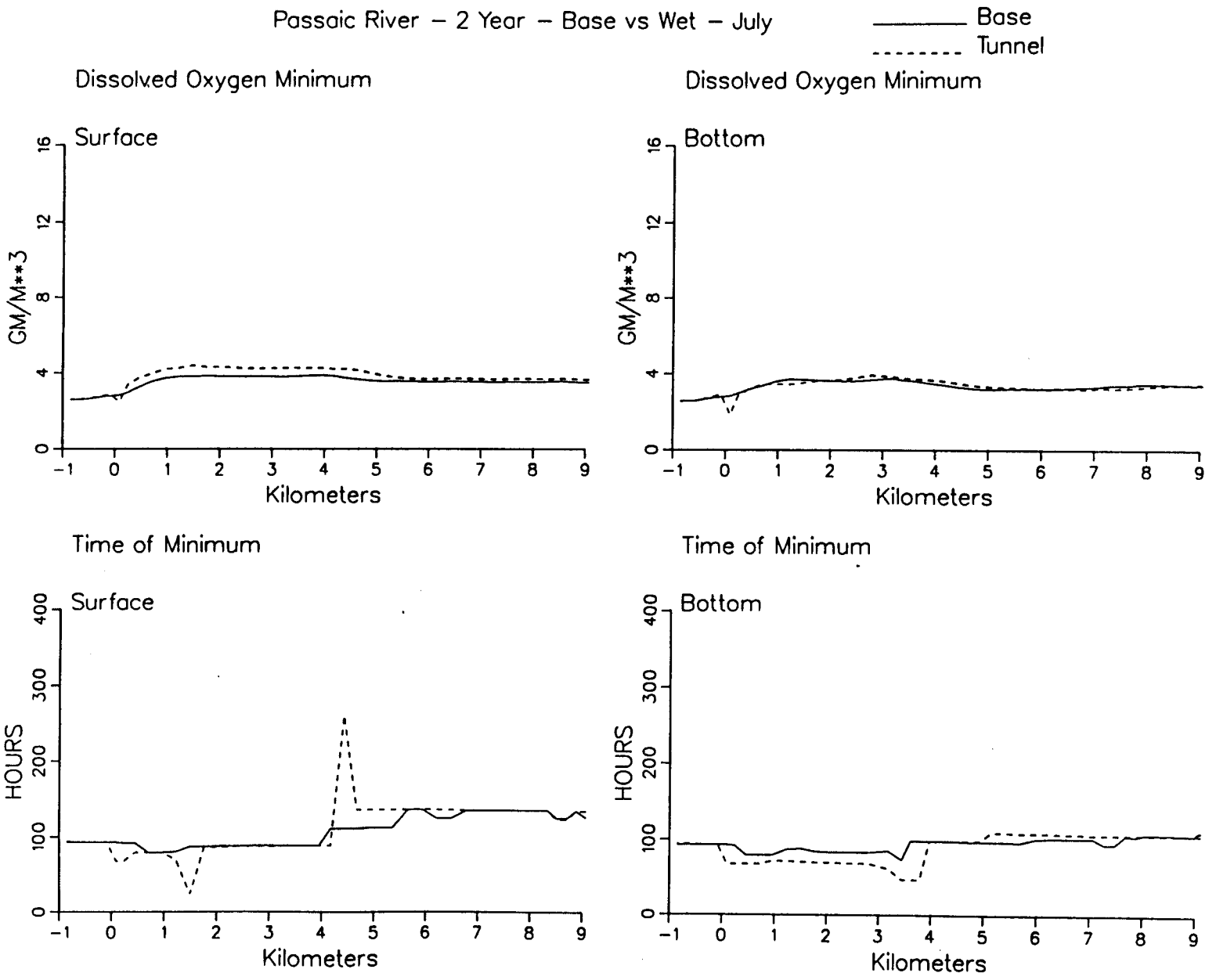

Passaic River – 2 Year – Base vs Wet – July
Base
Tunnel
Dissolved Oxygen Minimum
Surface
GM/M**3
Kilometers
Dissolved Oxygen Minimum
Bottom
GM/M**3
Kilometers
Time of Minimum
Surface
HOURS
Kilometers
Time of Minimum
Bottom
HOURS
Kilometers

Figure C-138

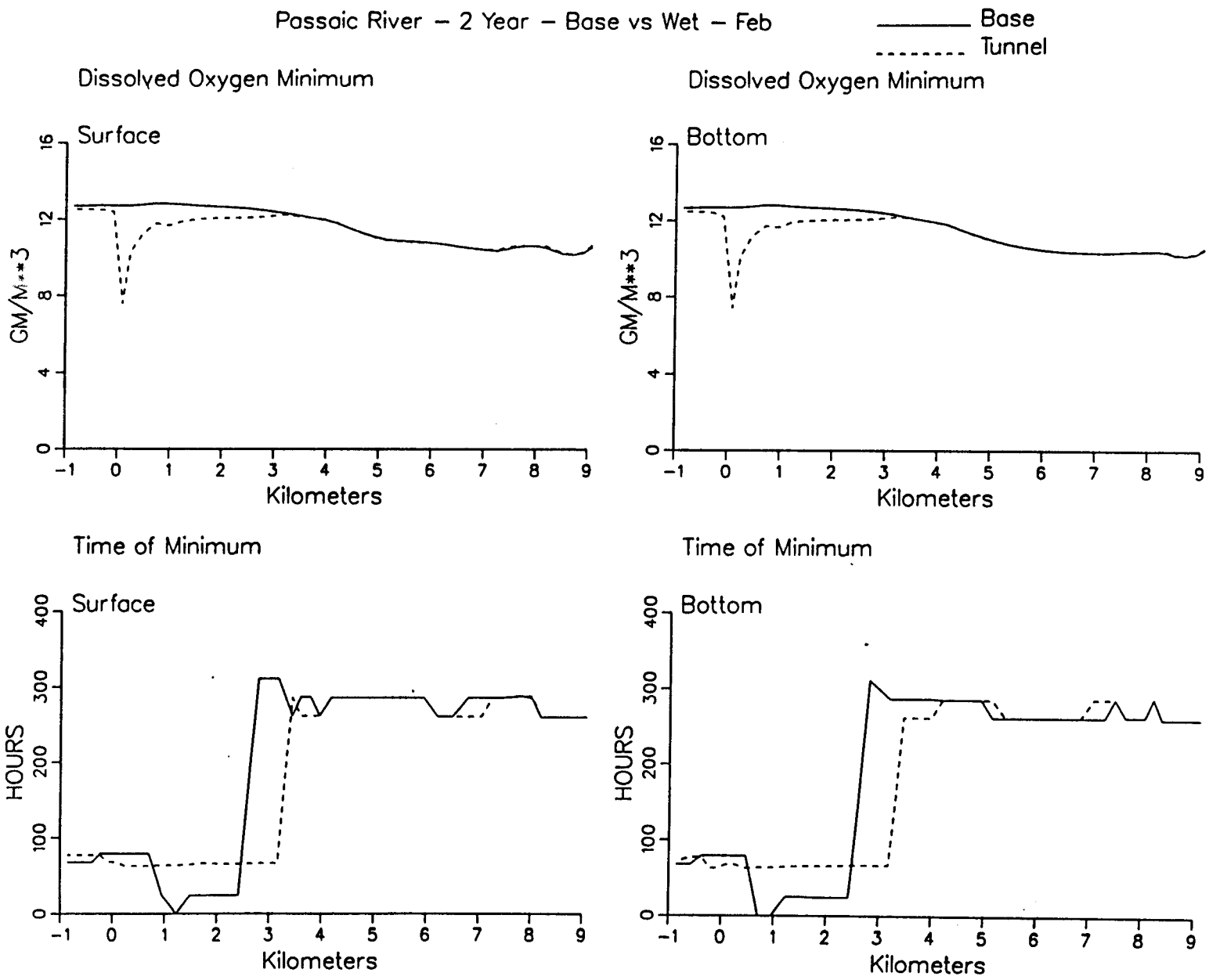

Passaic River – 2 Year – Base vs Wet – Feb
Base
Tunnel
Dissolved Oxygen Minimum
Surface
GM/M**3
Kilometers
Dissolved Oxygen Minimum
Bottom
GM/M**3
Kilometers
Time of Minimum
Surface
HOURS
Kilometers
Time of Minimum
Bottom
HOURS
Kilometers

Figure C-139

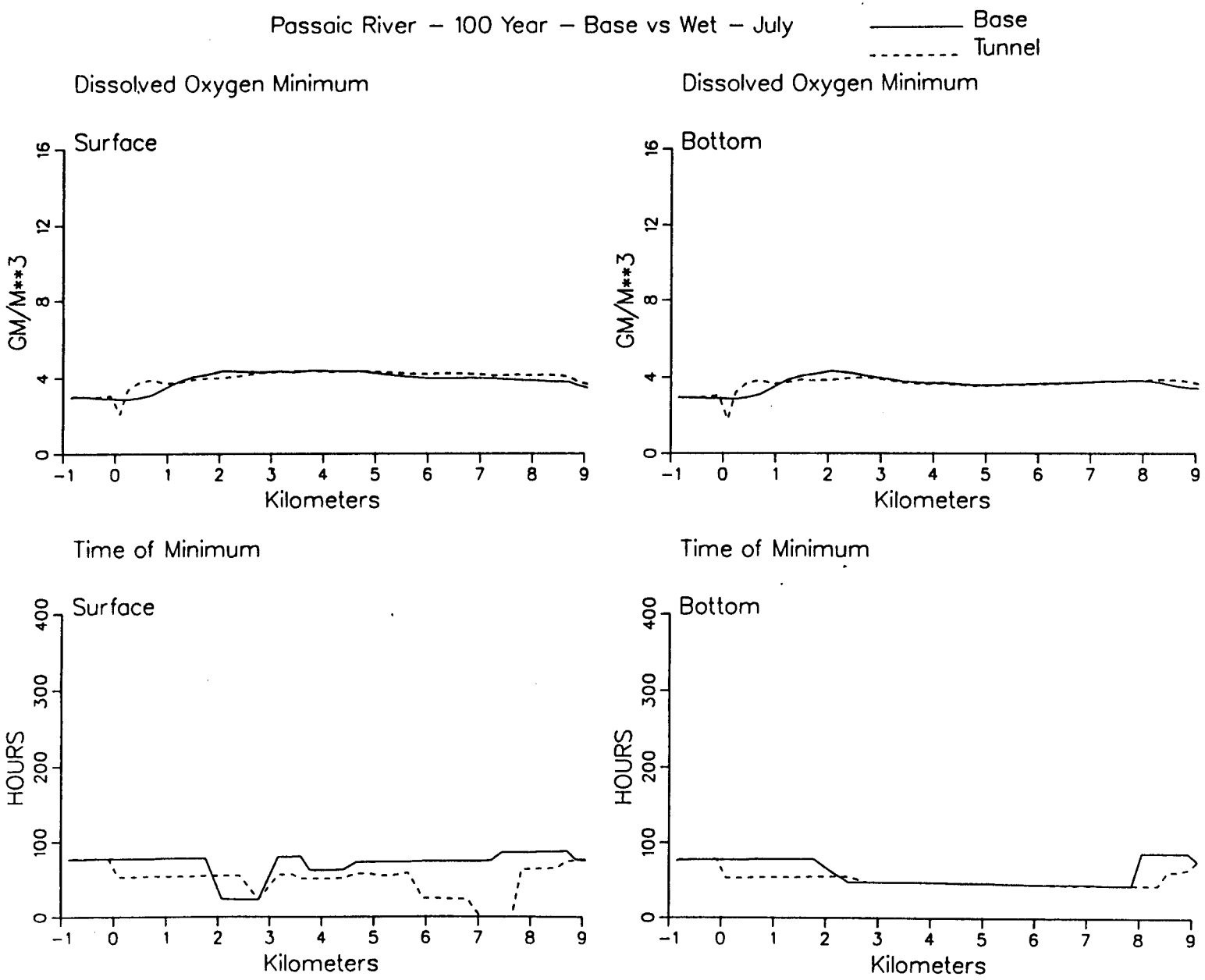

Passaic River – 100 Year – Base vs Wet – July
Base
Tunnel
Dissolved Oxygen Minimum
Surface
GM/M**3
Kilometers
Dissolved Oxygen Minimum
Bottom
GM/M**3
Kilometers
Time of Minimum
Surface
HOURS
Kilometers
Time of Minimum
Bottom
HOURS
Kilometers

Figure C-140

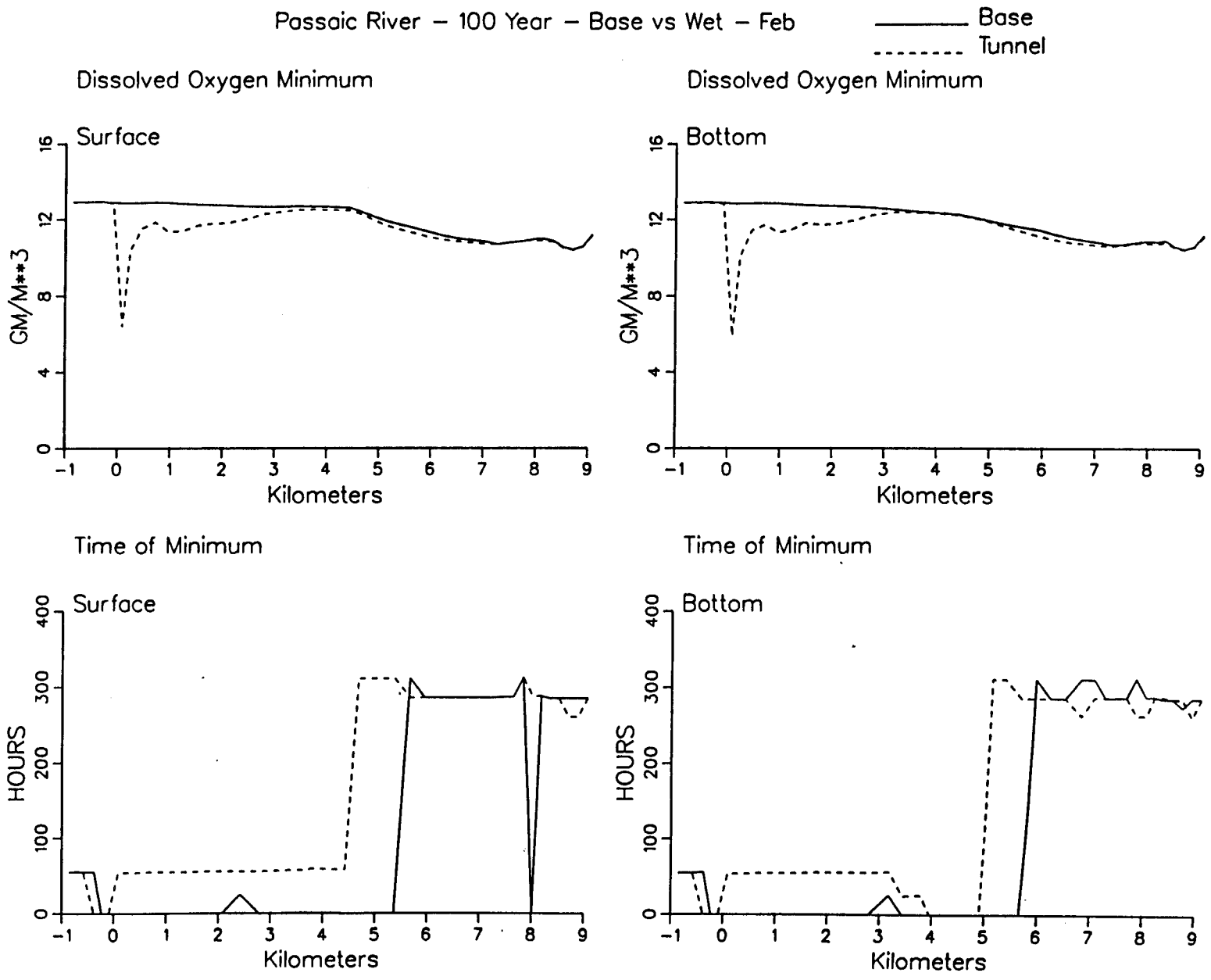

Passaic River – 100 Year – Base vs Wet – Feb
Base
Tunnel
Dissolved Oxygen Minimum
Surface
GM/M**3
Kilometers
Dissolved Oxygen Minimum
Bottom
GM/M**3
Kilometers
Time of Minimum
Surface
HOURS
Kilometers
Time of Minimum
Bottom
HOURS
Kilometers

Figure C-141

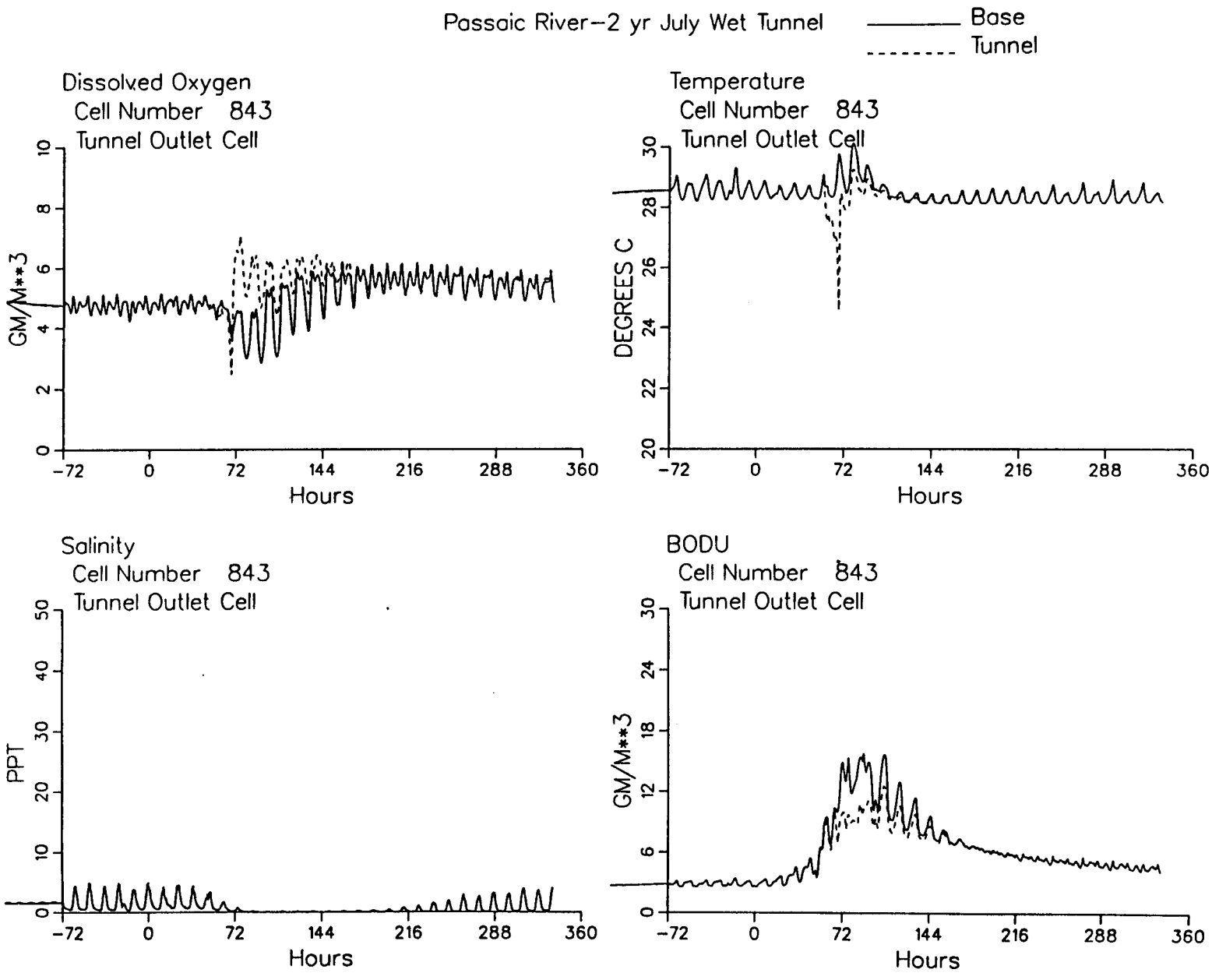

Passaic River–2 yr July Wet Tunnel
Base
Tunnel
Dissolved Oxygen
Cell Number 843
Tunnel Outlet Cell
GM/M**3
Hours
Temperature
Cell Number 843
Tunnel Outlet Cell
DEGREES C
Hours
Salinity
Cell Number 843
Tunnel Outlet Cell
PPT
Hours
BODU
Cell Number 843
Tunnel Outlet Cell
GM/M**3
Hours

Figure C-142

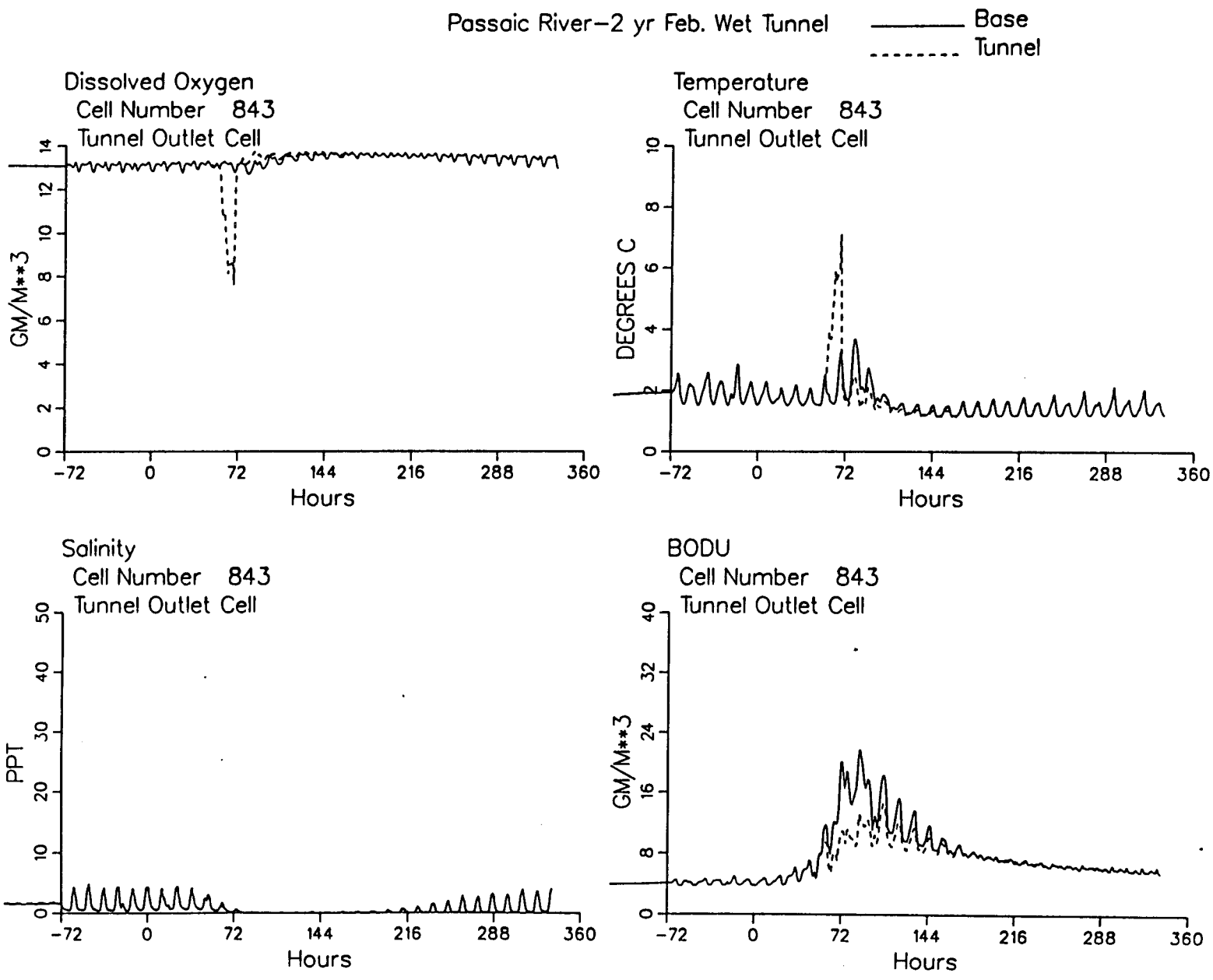

Passaic River–2 yr Feb. Wet Tunnel
Base
Tunnel
Dissolved Oxygen
Cell Number 843
Tunnel Outlet Cell
GM/M**3
Hours
Temperature
Cell Number 843
Tunnel Outlet Cell
DEGREES C
Hours
Salinity
Cell Number 843
Tunnel Outlet Cell
PPT
Hours
BODU
Cell Number 843
Tunnel Outlet Cell
GM/M**3
Hours

Figure C-143

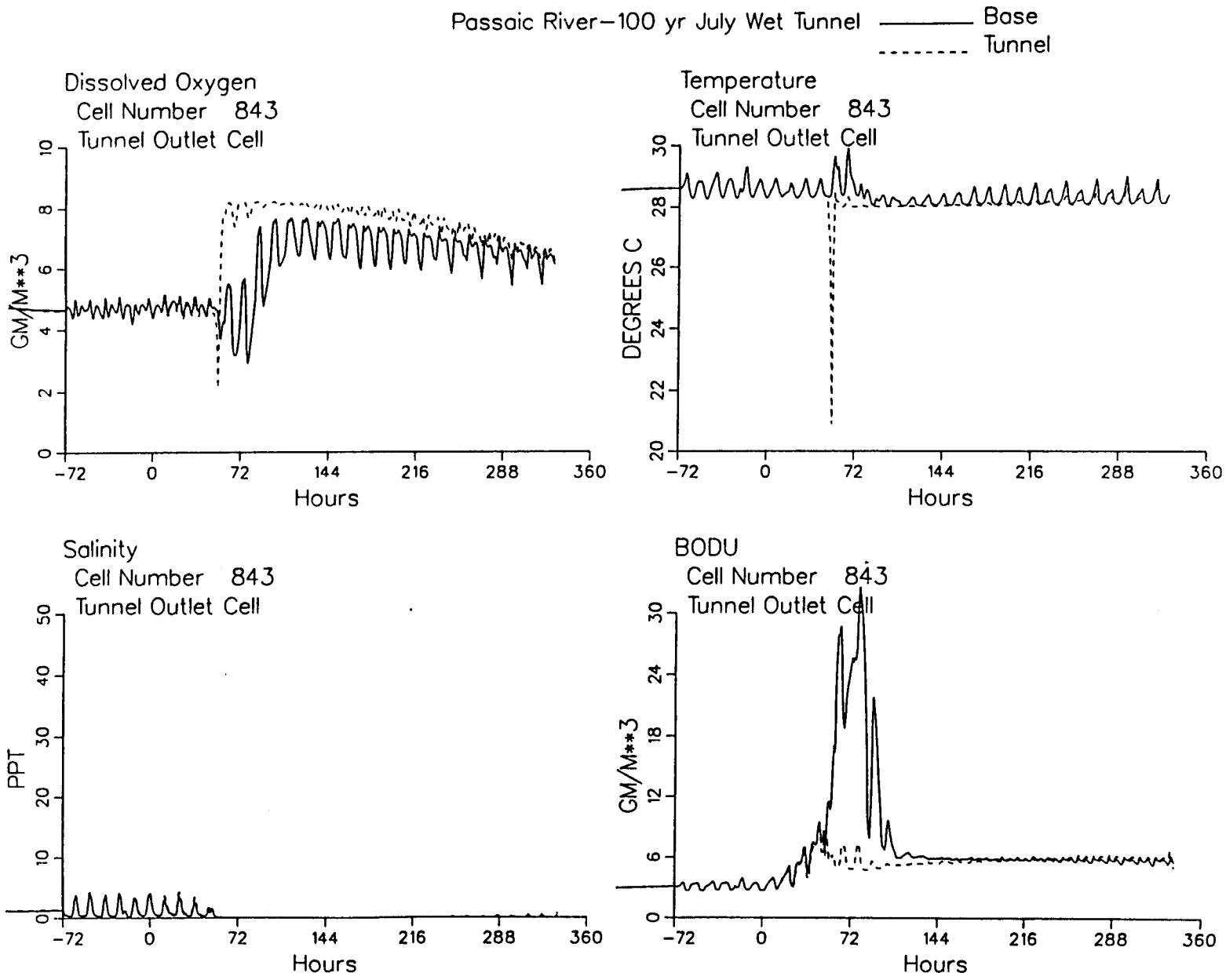

Passaic River–100 yr July Wet Tunnel
Base
Tunnel
Dissolved Oxygen
Cell Number 843
Tunnel Outlet Cell
GM/M**3
Hours
Temperature
Cell Number 843
Tunnel Outlet Cell
DEGREES C
Hours
Salinity
Cell Number 843
Tunnel Outlet Cell
PPT
Hours
BODU
Cell Number 843
Tunnel Outlet Cell
GM/M**3
Hours

Figure C-144

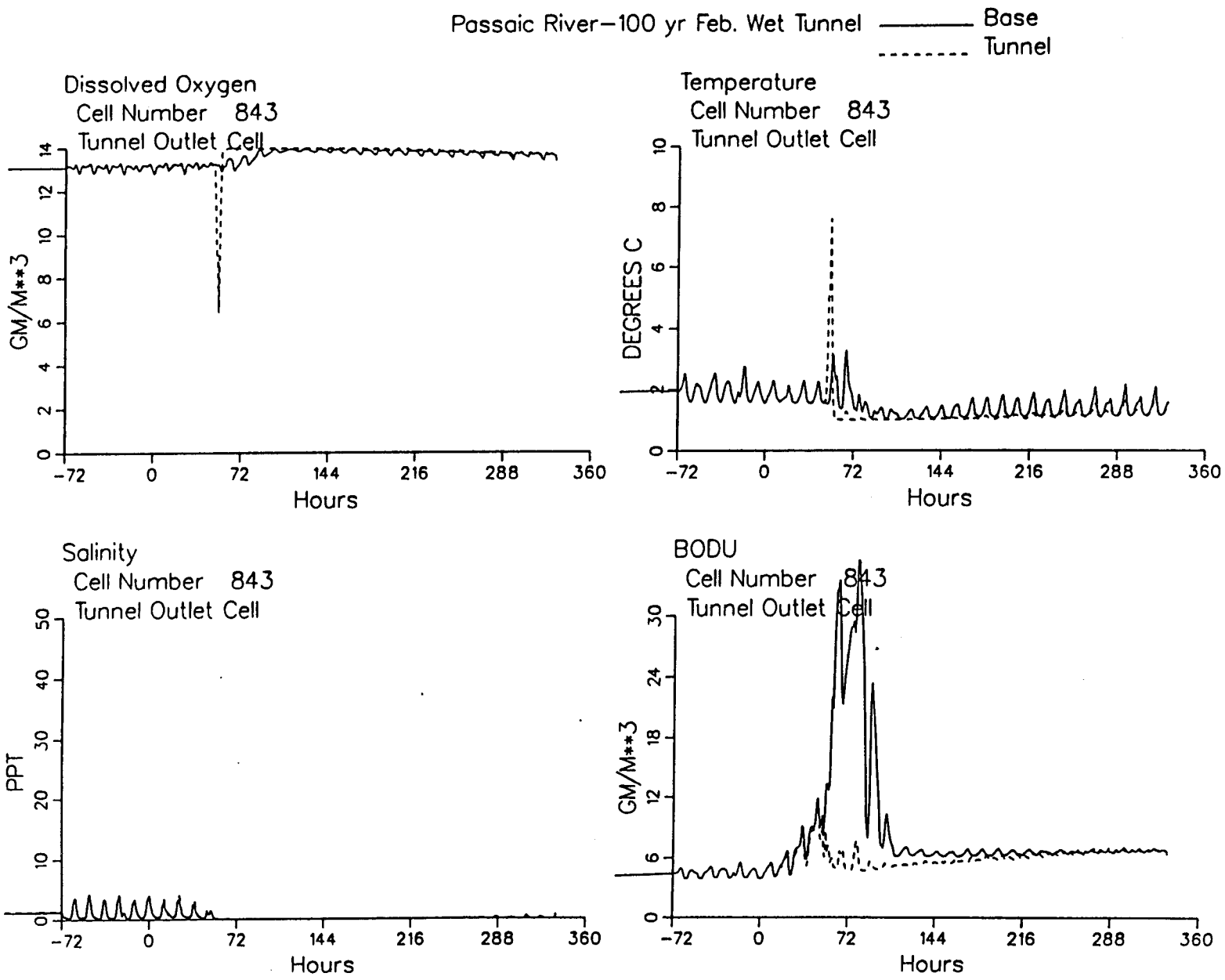

Passaic River—100 yr Feb. Wet Tunnel
Base
Tunnel
Dissolved Oxygen
Cell Number 843
Tunnel Outlet Cell
GM/M**3
Hours
Temperature
Cell Number 843
Tunnel Outlet Cell
DEGREES C
Hours
Salinity
Cell Number 843
Tunnel Outlet Cell
PPT
Hours
BODU
Cell Number 843
Tunnel Outlet Cell
GM/M**3
Hours

Figure C-145

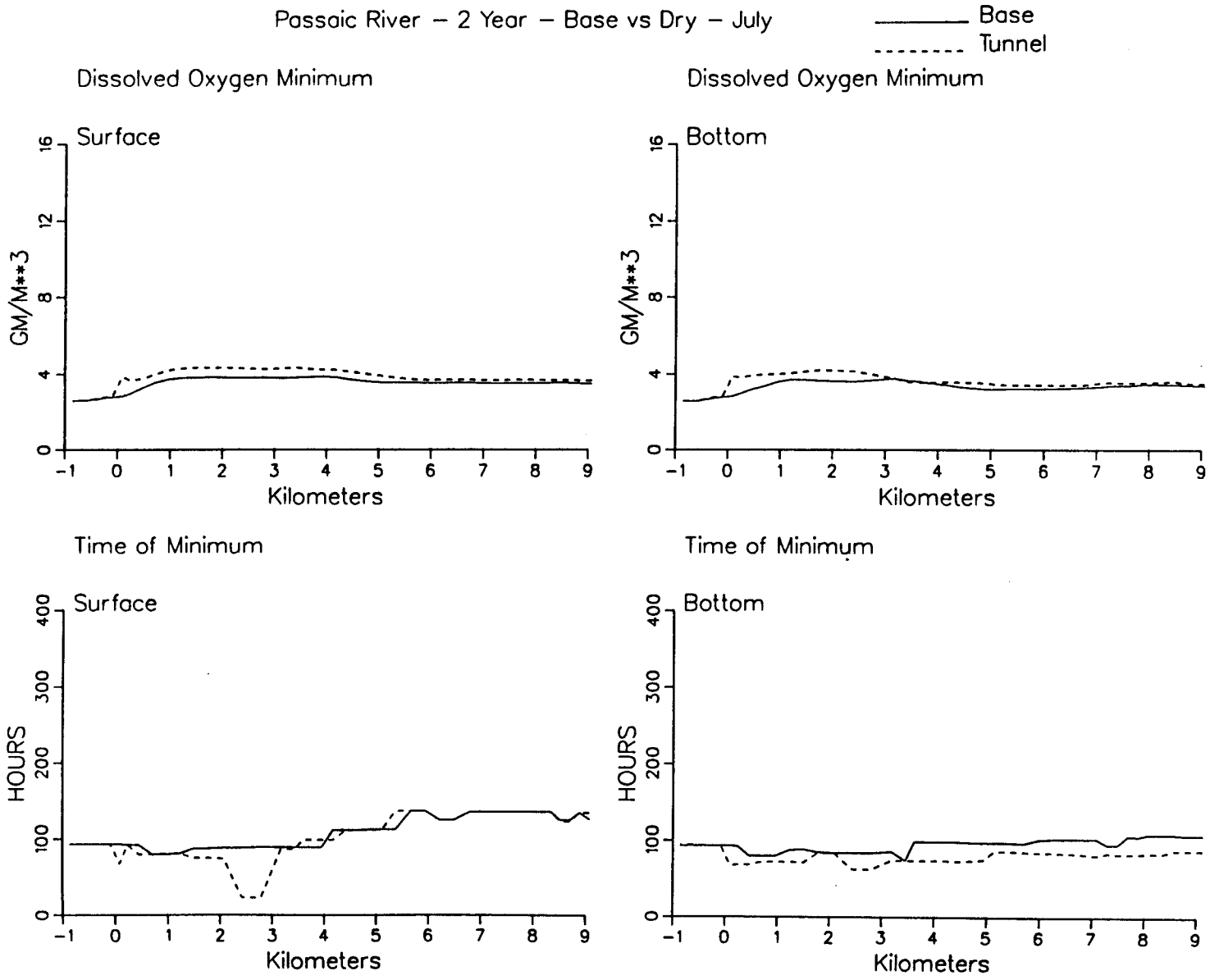

Passaic River – 2 Year – Base vs Dry – July
Base
Tunnel
Dissolved Oxygen Minimum
Surface
GM/M**3
Kilometers
Dissolved Oxygen Minimum
Bottom
GM/M**3
Kilometers
Time of Minimum
Surface
HOURS
Kilometers
Time of Minimum
Bottom
HOURS
Kilometers

Figure C-146

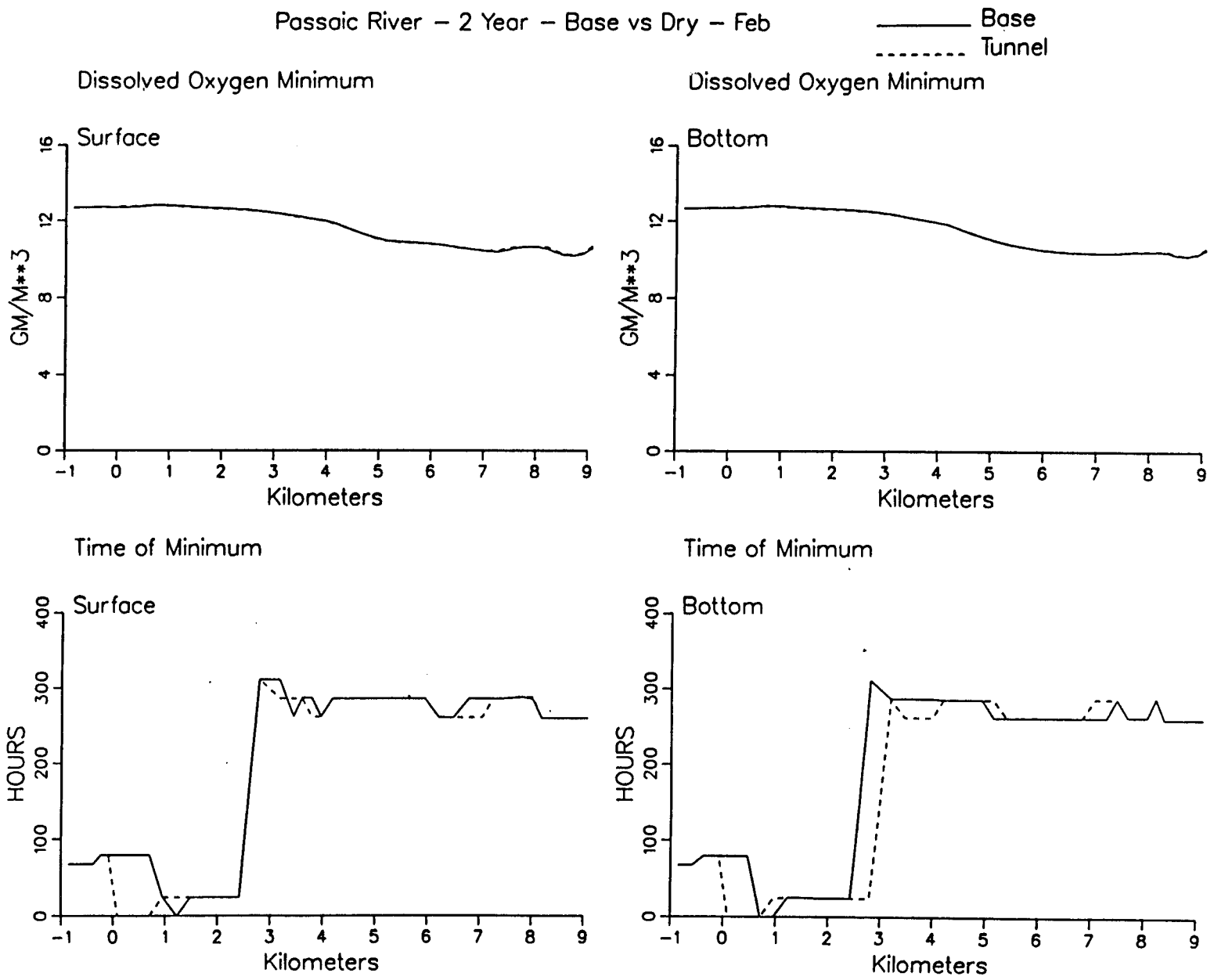

Passaic River – 2 Year – Base vs Dry – Feb
Base
Tunnel
Dissolved Oxygen Minimum
Surface
GM/M**3
Kilometers
Dissolved Oxygen Minimum
Bottom
GM/M**3
Kilometers
Time of Minimum
Surface
HOURS
Kilometers
Time of Minimum
Bottom
HOURS
Kilometers

Figure C-147

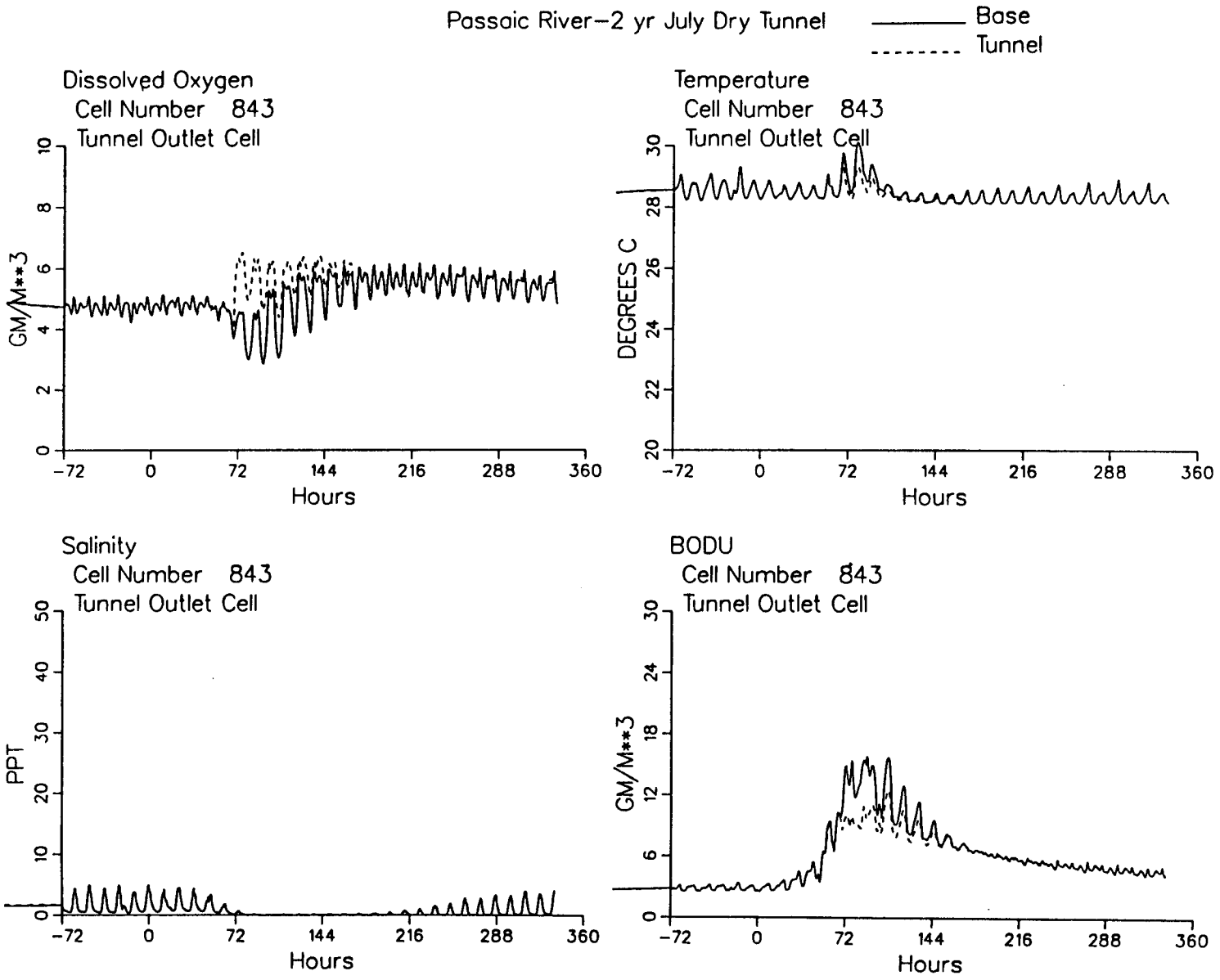

Passaic River–2 yr July Dry Tunnel
Base
Tunnel
Dissolved Oxygen
Cell Number 843
Tunnel Outlet Cell
GM/M**3
0
2
4
6
8
10
-72
0
72
144
216
288
360
Hours
Temperature
Cell Number 843
Tunnel Outlet Cell
DEGREES C
20
22
24
26
28
30
-72
0
72
144
216
288
360
Hours
Salinity
Cell Number 843
Tunnel Outlet Cell
PPT
0
10
20
30
40
50
-72
0
72
144
216
288
360
Hours
BODU
Cell Number 843
Tunnel Outlet Cell
GM/M**3
0
6
12
18
24
30
-72
0
72
144
216
288
360
Hours

Figure C-148

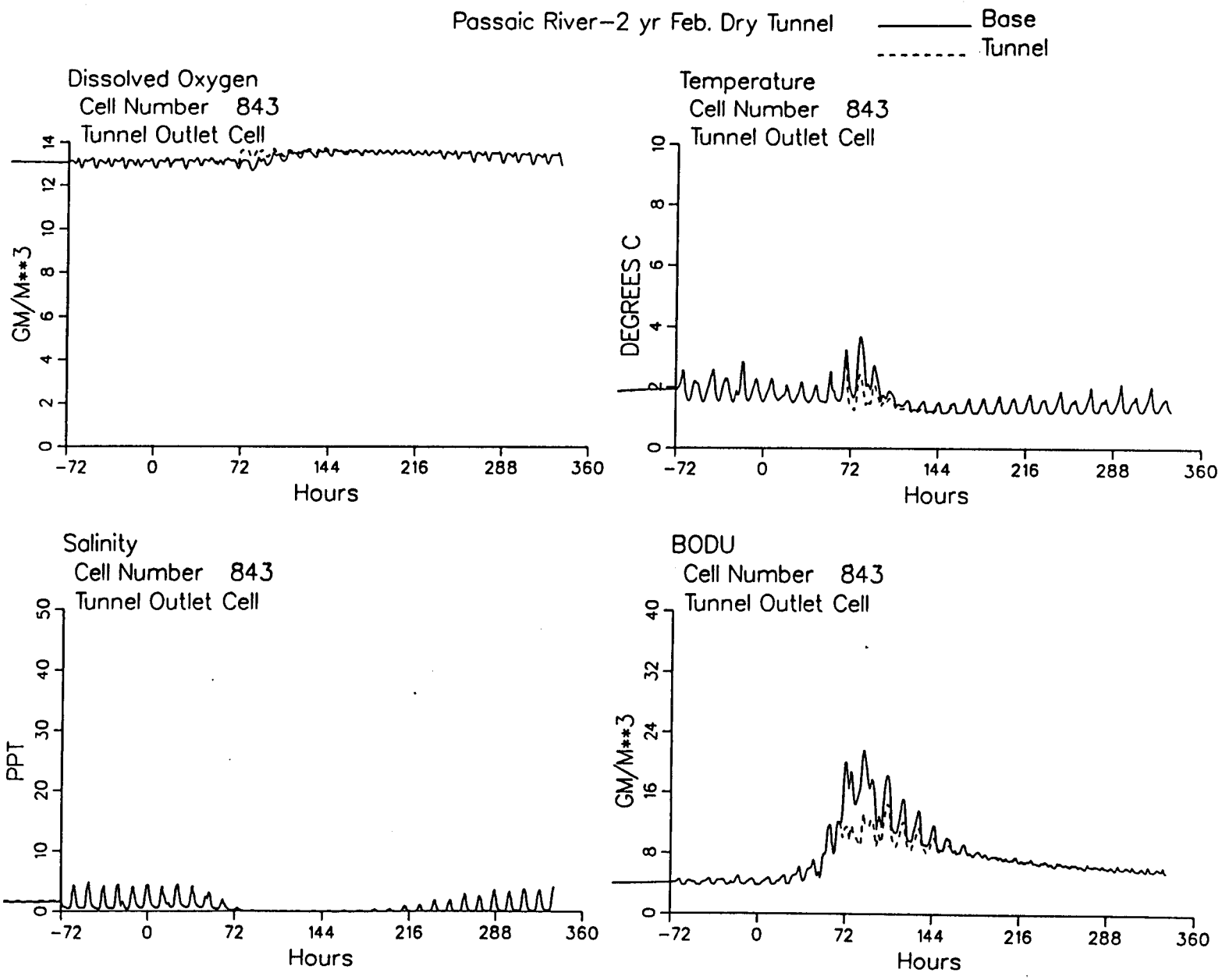

Passaic River–2 yr Feb. Dry Tunnel
Base
Tunnel
Dissolved Oxygen
Cell Number 843
Tunnel Outlet Cell
GM/M**3
Hours
Temperature
Cell Number 843
Tunnel Outlet Cell
DEGREES C
Hours
Salinity
Cell Number 843
Tunnel Outlet Cell
PPT
Hours
BODU
Cell Number 843
Tunnel Outlet Cell
GM/M**3
Hours

Figure C-149

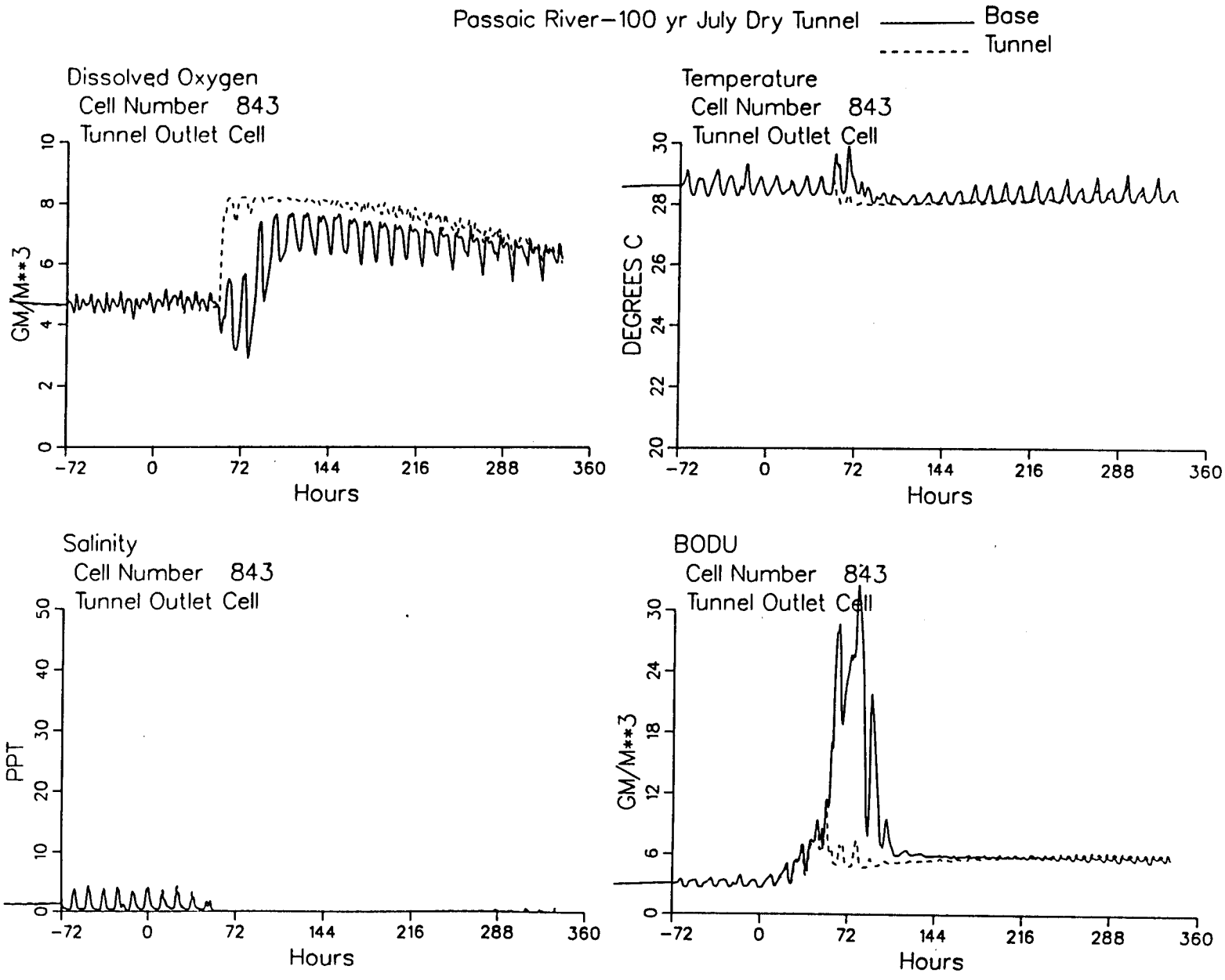

Passaic River–100 yr July Dry Tunnel
Base
Tunnel
Dissolved Oxygen
Cell Number 843
Tunnel Outlet Cell
GM/M**3
Hours
Temperature
Cell Number 843
Tunnel Outlet Cell
DEGREES C
Hours
Salinity
Cell Number 843
Tunnel Outlet Cell
PPT
Hours
BODU
Cell Number 843
Tunnel Outlet Cell
GM/M**3
Hours

Figure C-150

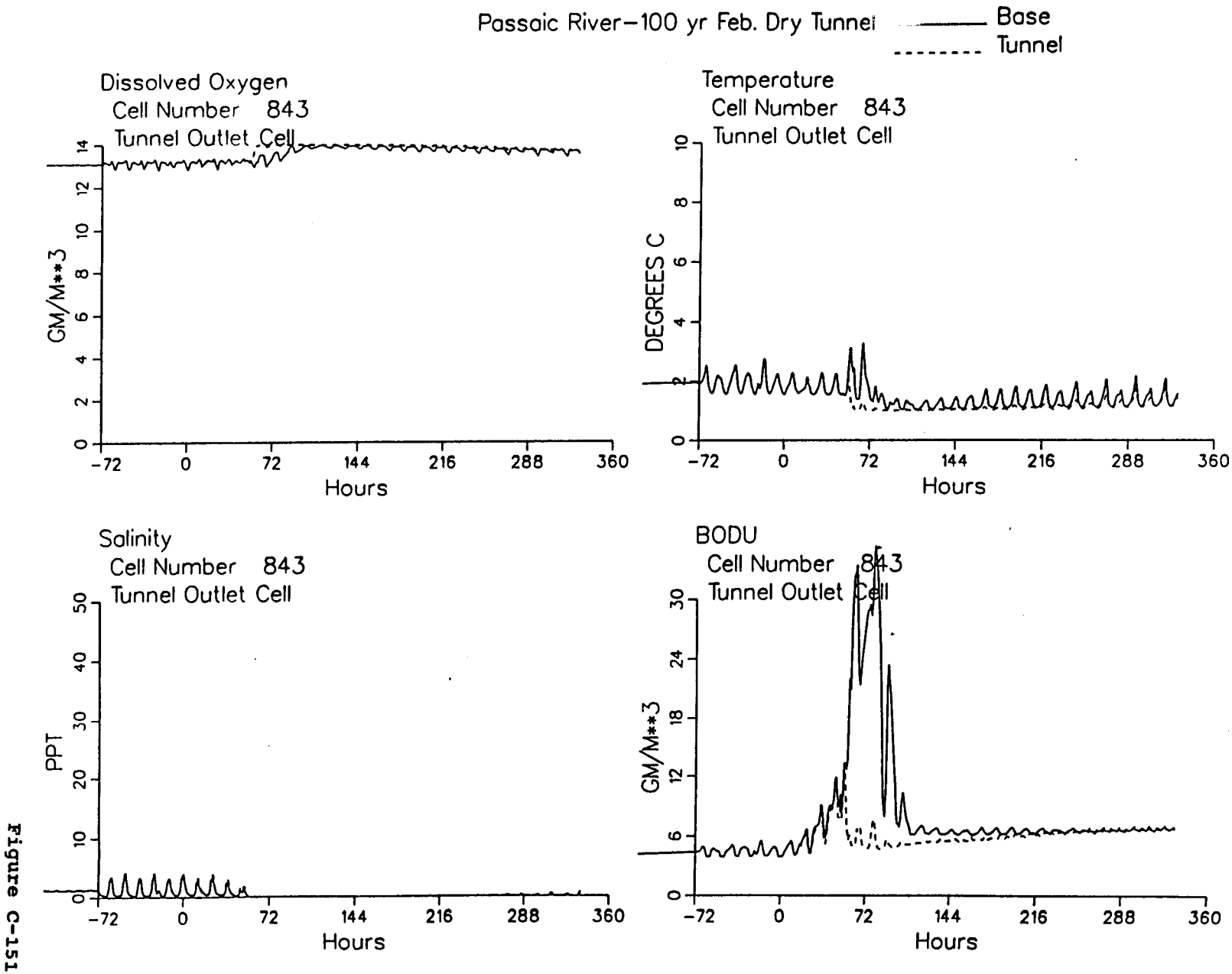
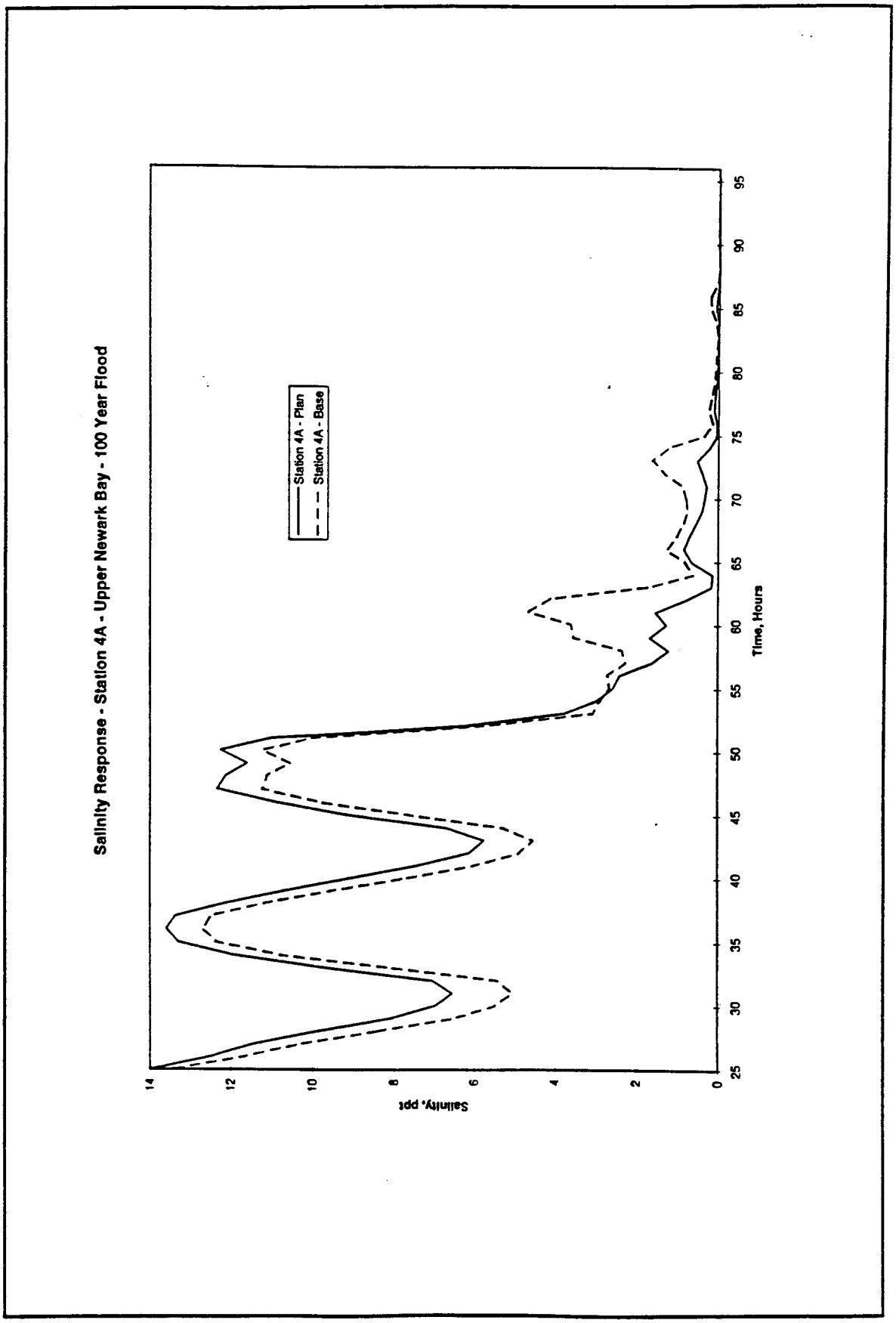
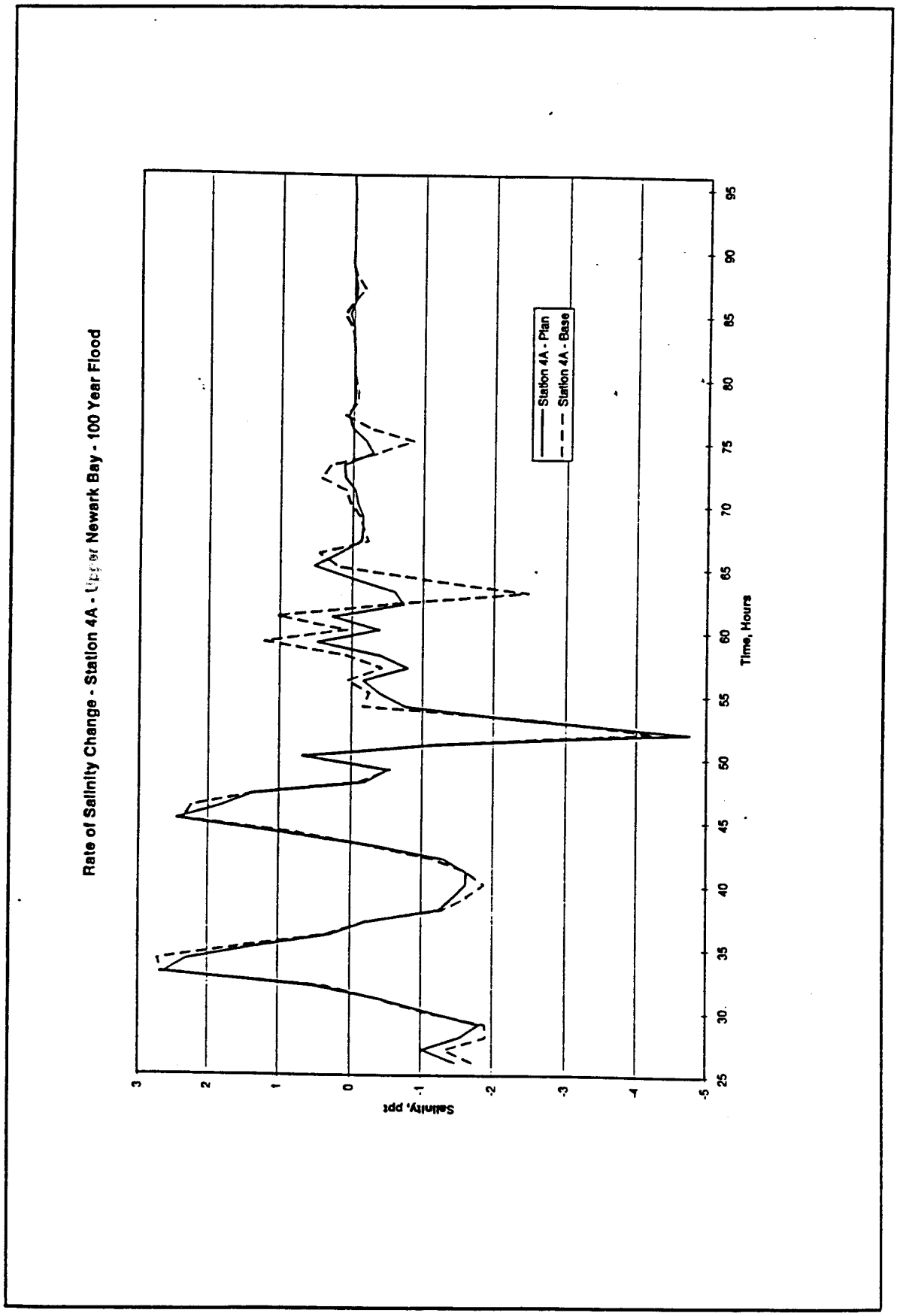

Passaic River–100 yr Feb. Dry Tunnel
Base
Tunnel
Dissolved Oxygen
Cell Number 843
Tunnel Outlet Cell
GM/M**3
Hours
Temperature
Cell Number 843
Tunnel Outlet Cell
DEGREES C
Hours
Salinity
Cell Number 843
Tunnel Outlet Cell
PPT
Hours
BODU
Cell Number 843
Tunnel Outlet Cell
GM/M**3
Hours

Figure C-151



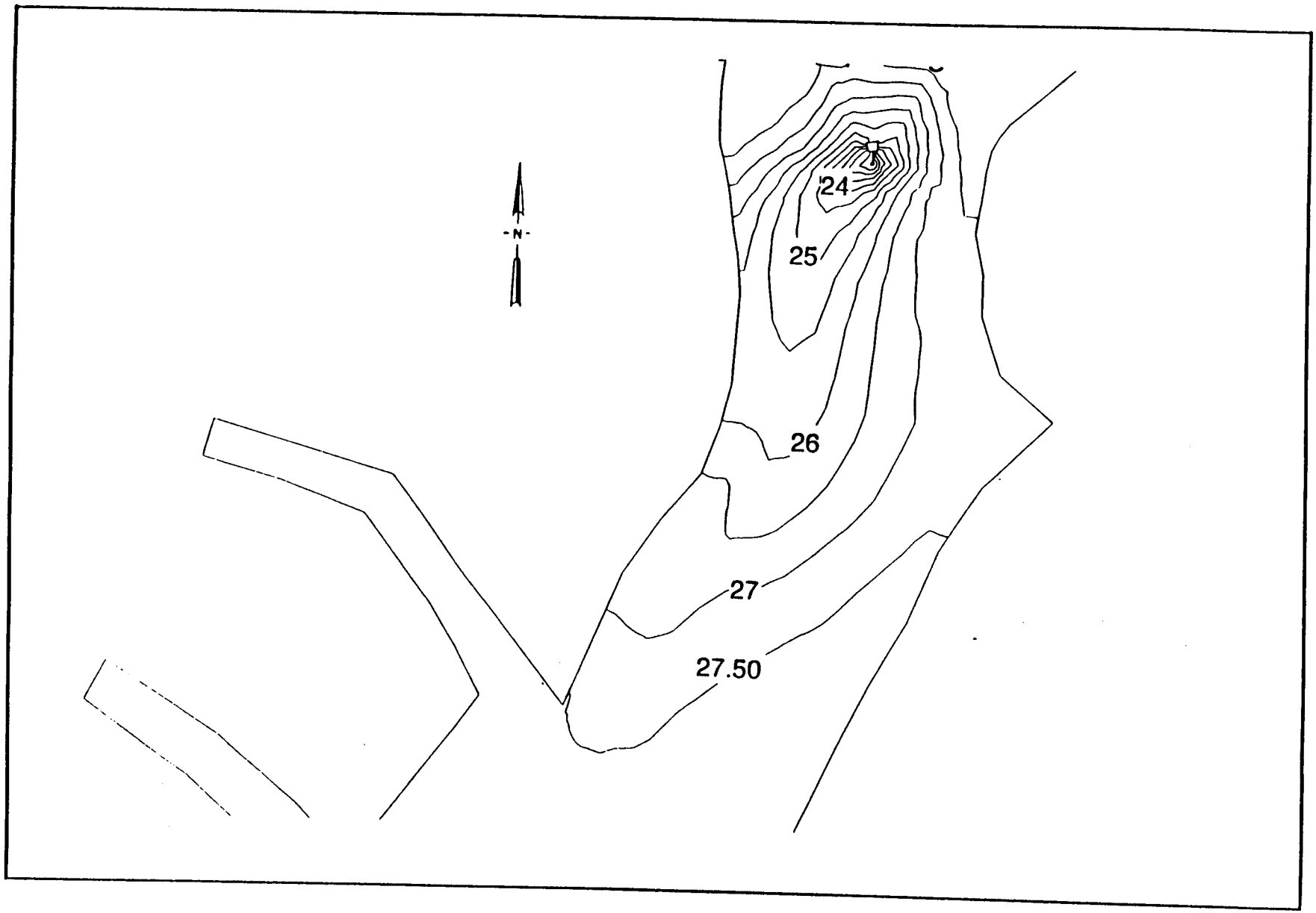
Salinity Timesseries at Station 4A for 100-yr Flood

Figure C-152



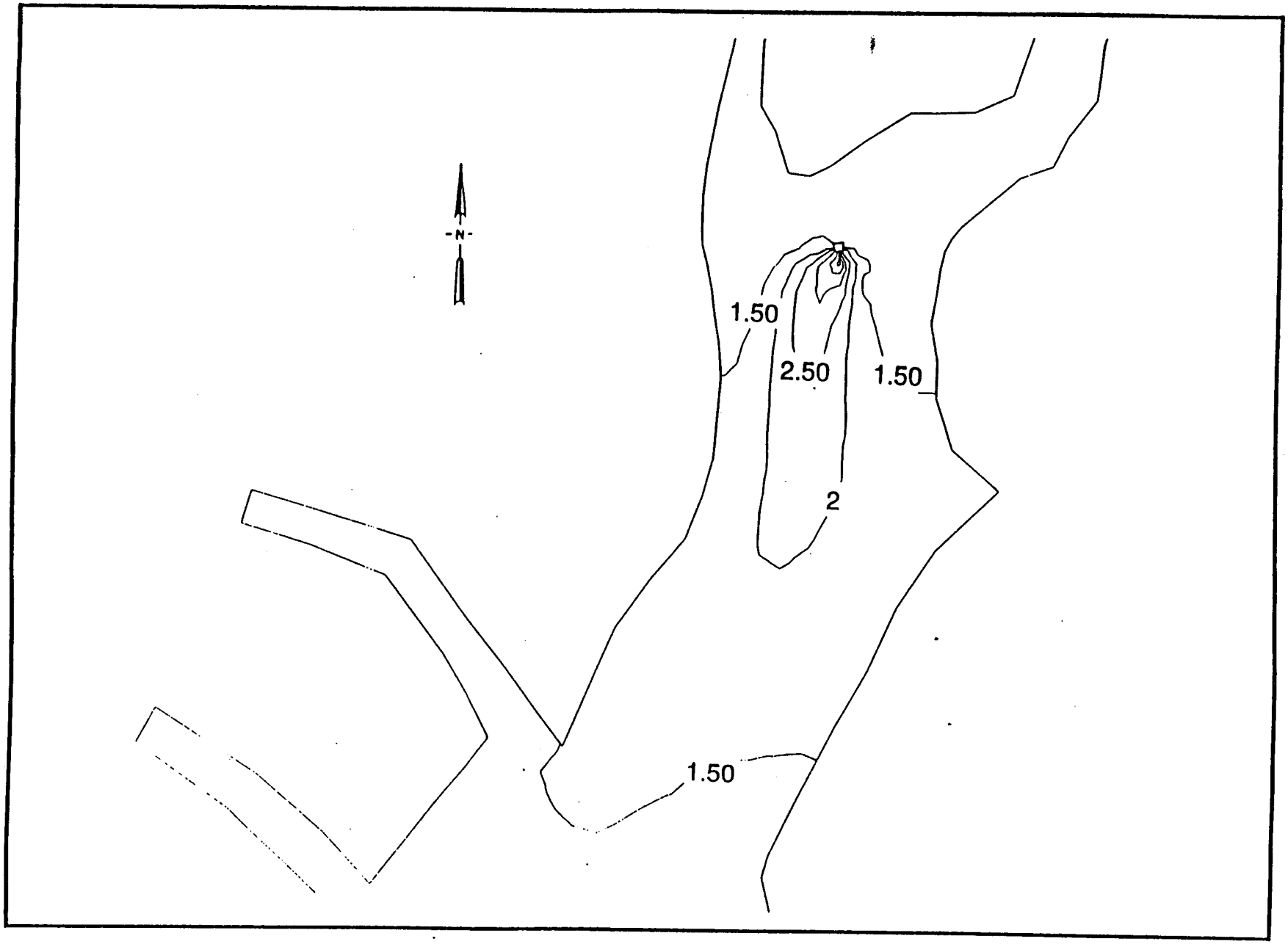
Rate of Salinity Change at Station 4A for 100-yr Flood

Figure C-153



Maximum Temperature Impact Contours; 2-yr Flood Summer Bottom

Figure C-154



Maximum Temperature Impact Contours; 2-yr Flood Winter Surface

Figure C-155

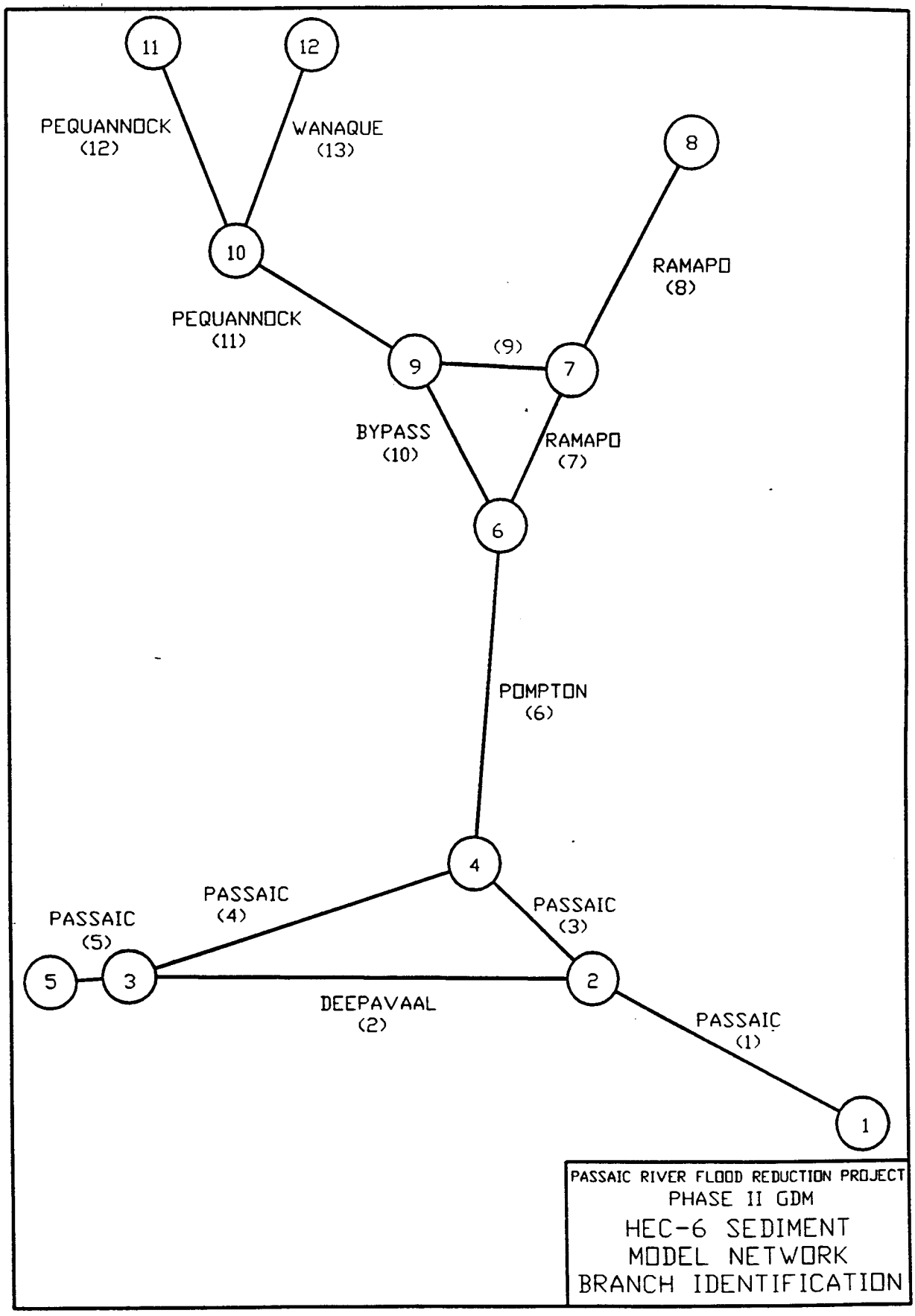

11
12
PEQUANNOCK
(12)
WANAQUE
(13)
8
10
RAMAPO
(8)
PEQUANNOCK
(11)
(9)
9
7
BYPASS
(10)
RAMAPO
(7)
6
POMPTON
(6)
4
PASSAIC
(4)
PASSAIC
(5)
PASSAIC
(3)
5
3
2
DEEPAVAAL
(2)
PASSAIC
(1)
1
PASSAIC RIVER FLOOD REDUCTION PROJECT
PHASE II GDM
HEC-6 SEDIMENT
MODEL NETWORK
BRANCH IDENTIFICATION

Figure C-156

BED AND WATER SURFACE PROFILES
BRANCH 1, PASSAIC RIVER

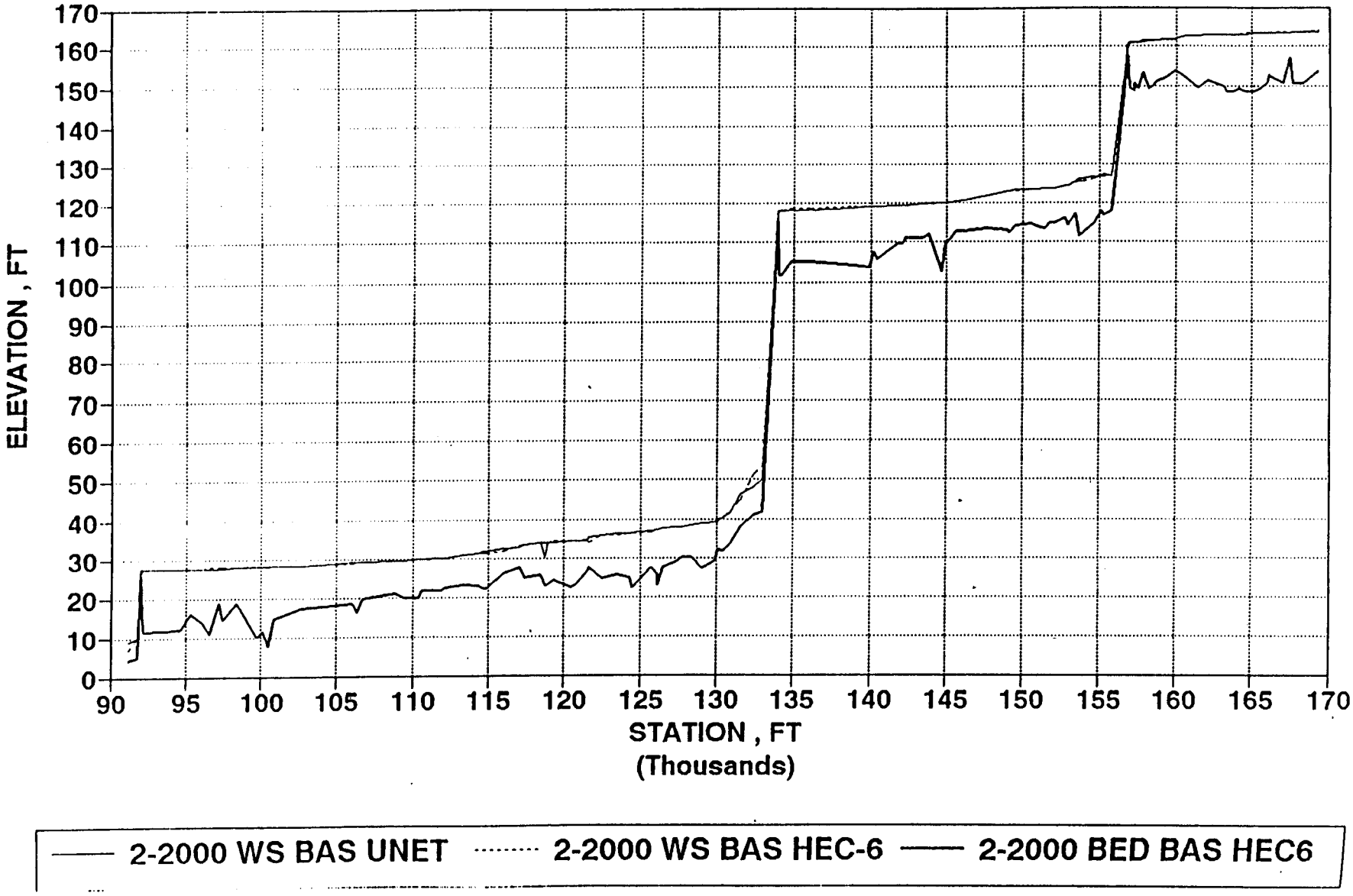


Figure C-157a

BED AND WATER SURFACE PROFILES
BRANCH 6, POMPTON RIVER

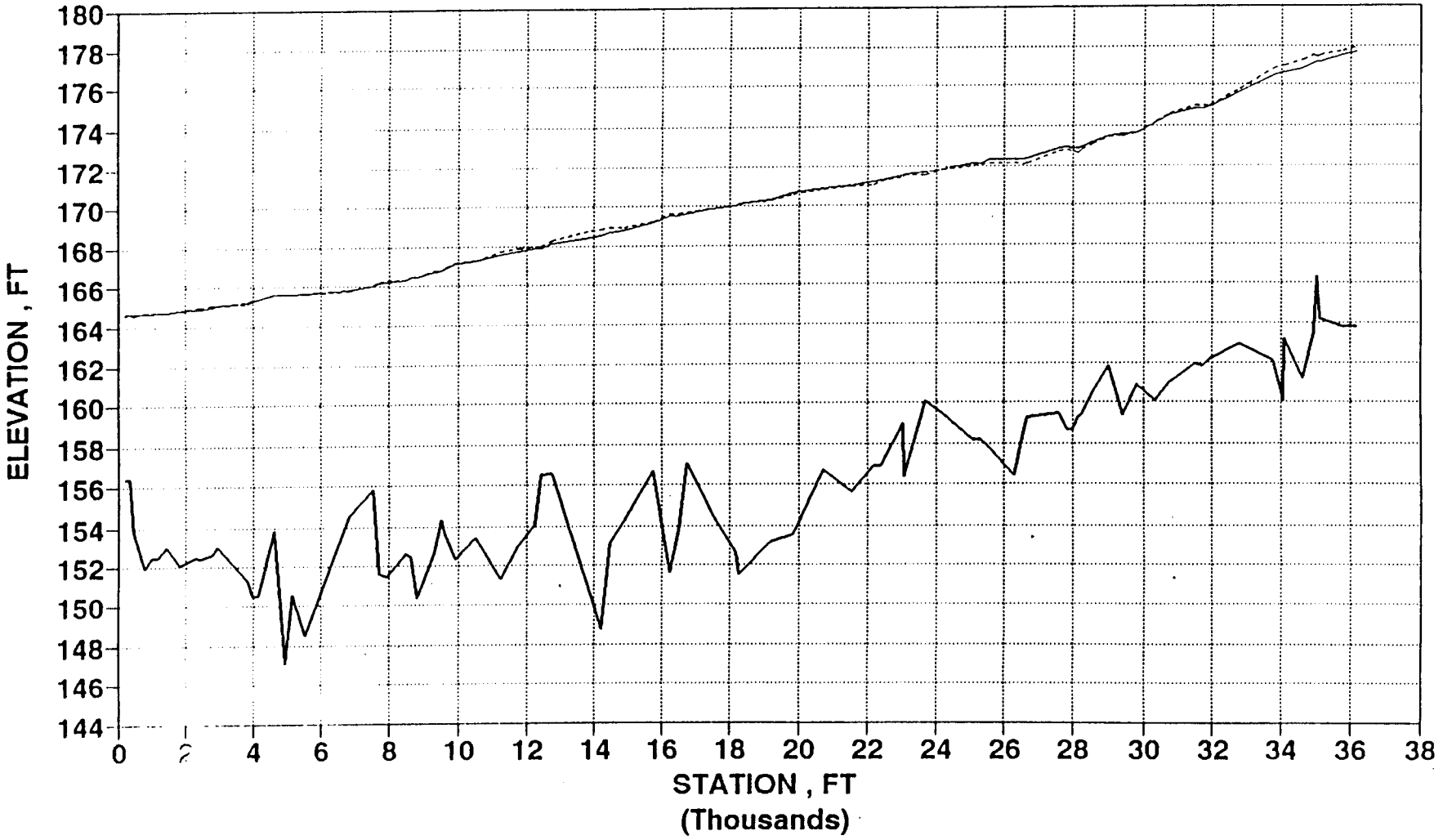


Figure C-157b

BED AND WATER SURFACE PROFILES
BRANCH 1, PASSAIC RIVER

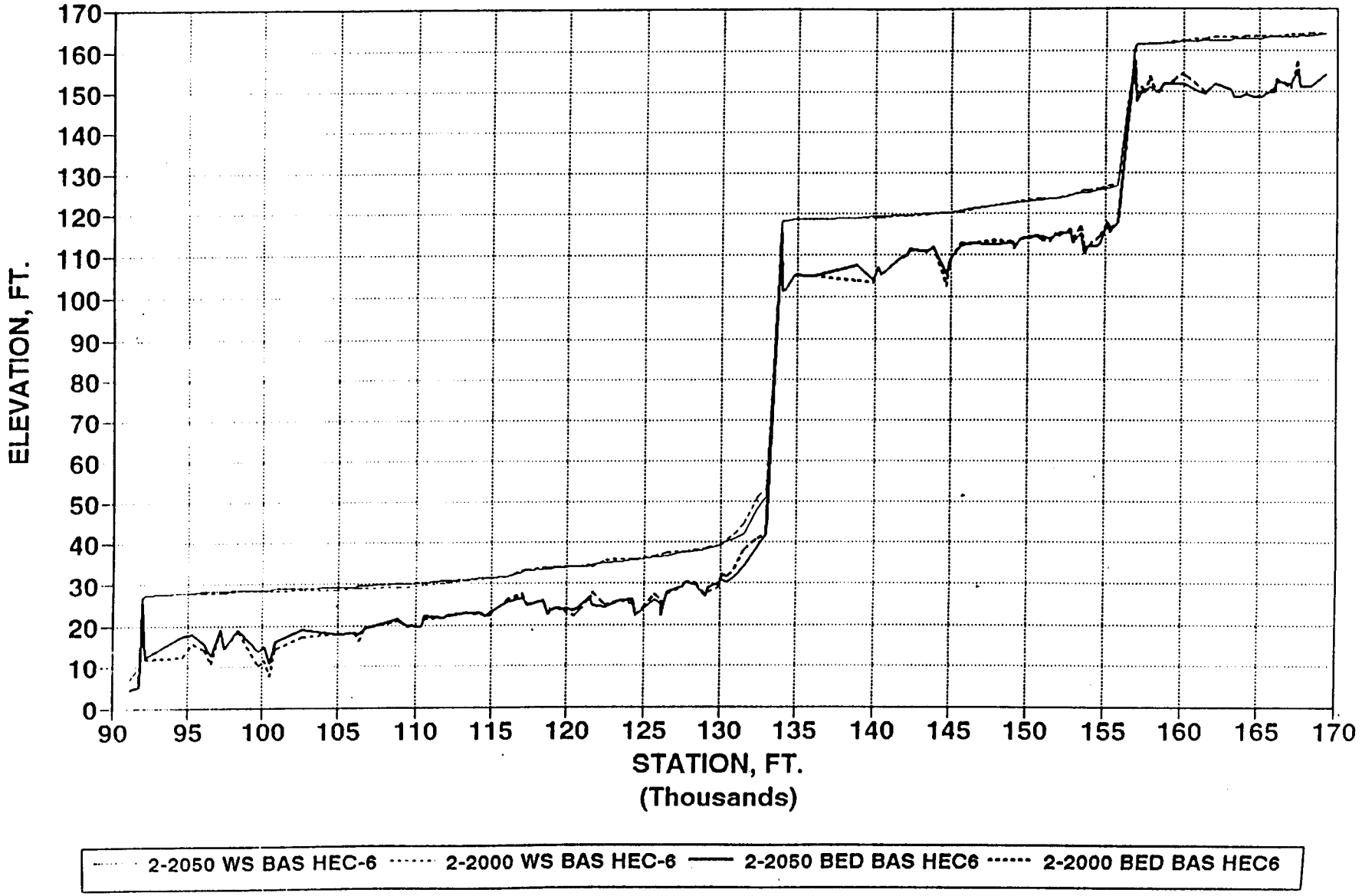


Figure C-158a

BED AND WATER SURFACE PROFILES
BRANCH 3, PASSAIC RIVER

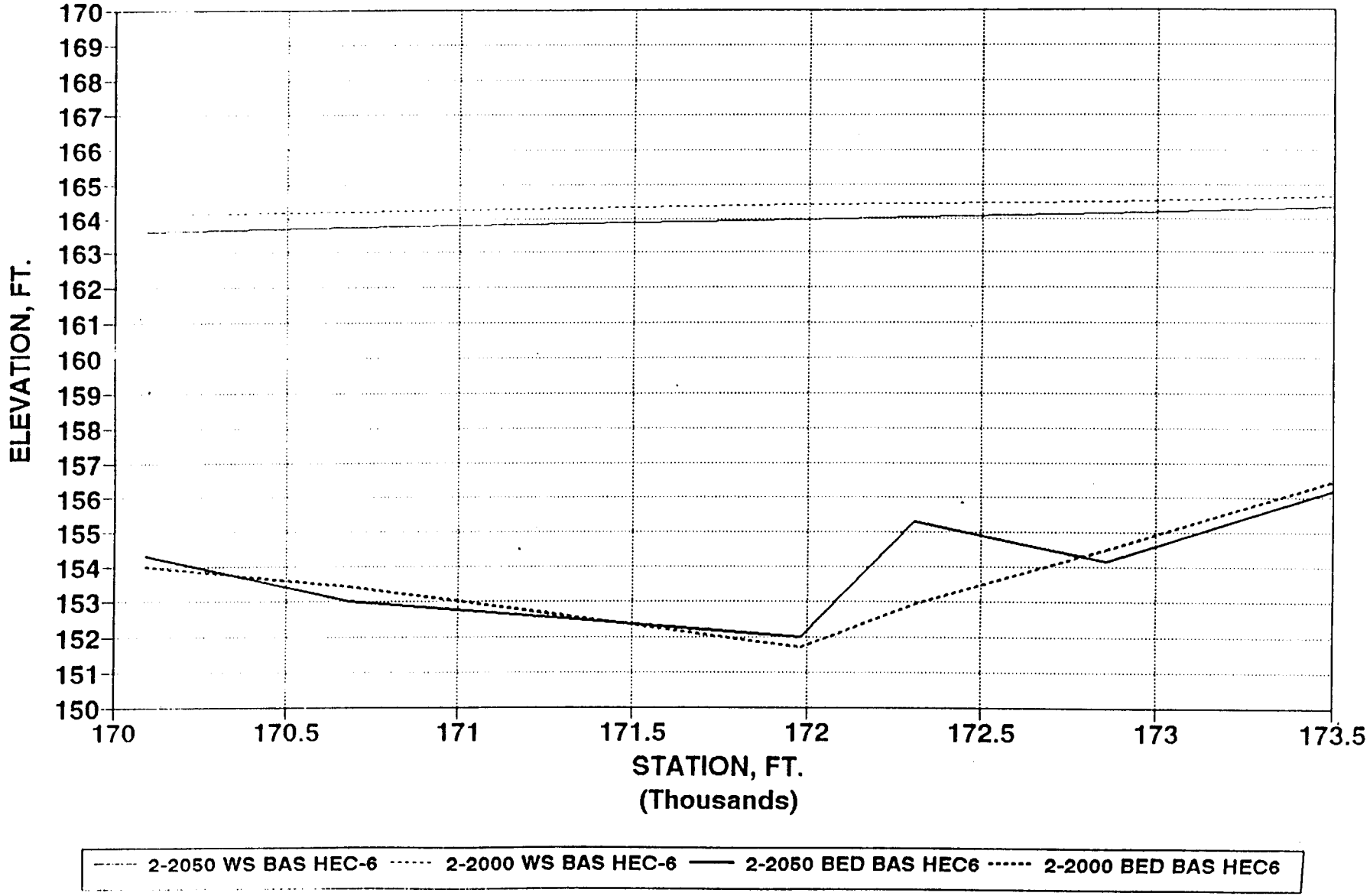


Figure C-158b

BED AND WATER SURFACE PROFILES
BRANCH 4, PASSAIC RIVER

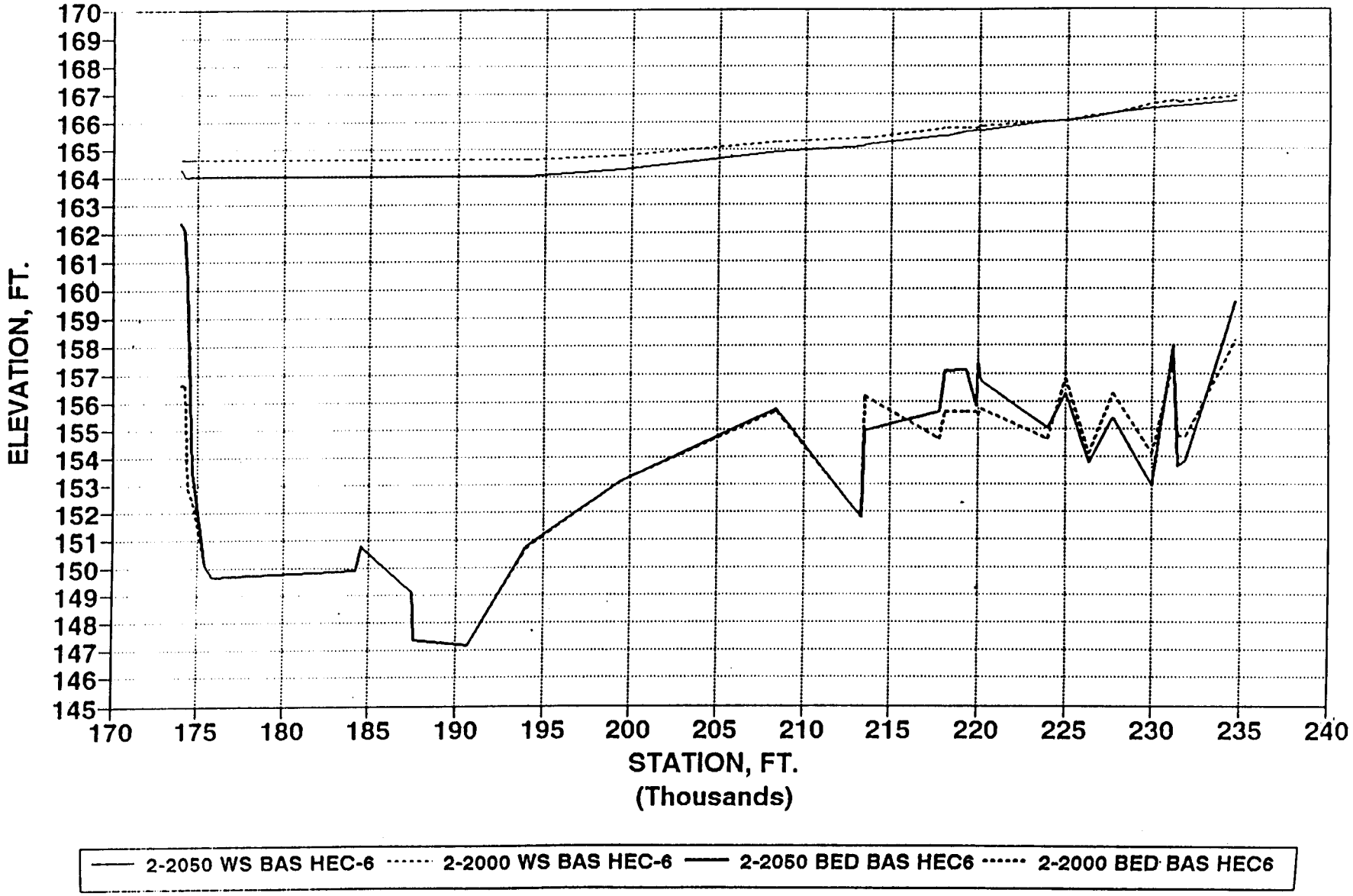


Figure C-158c

BED AND WATER SURFACE PROFILES
BRANCH 6, POMPTON RIVER

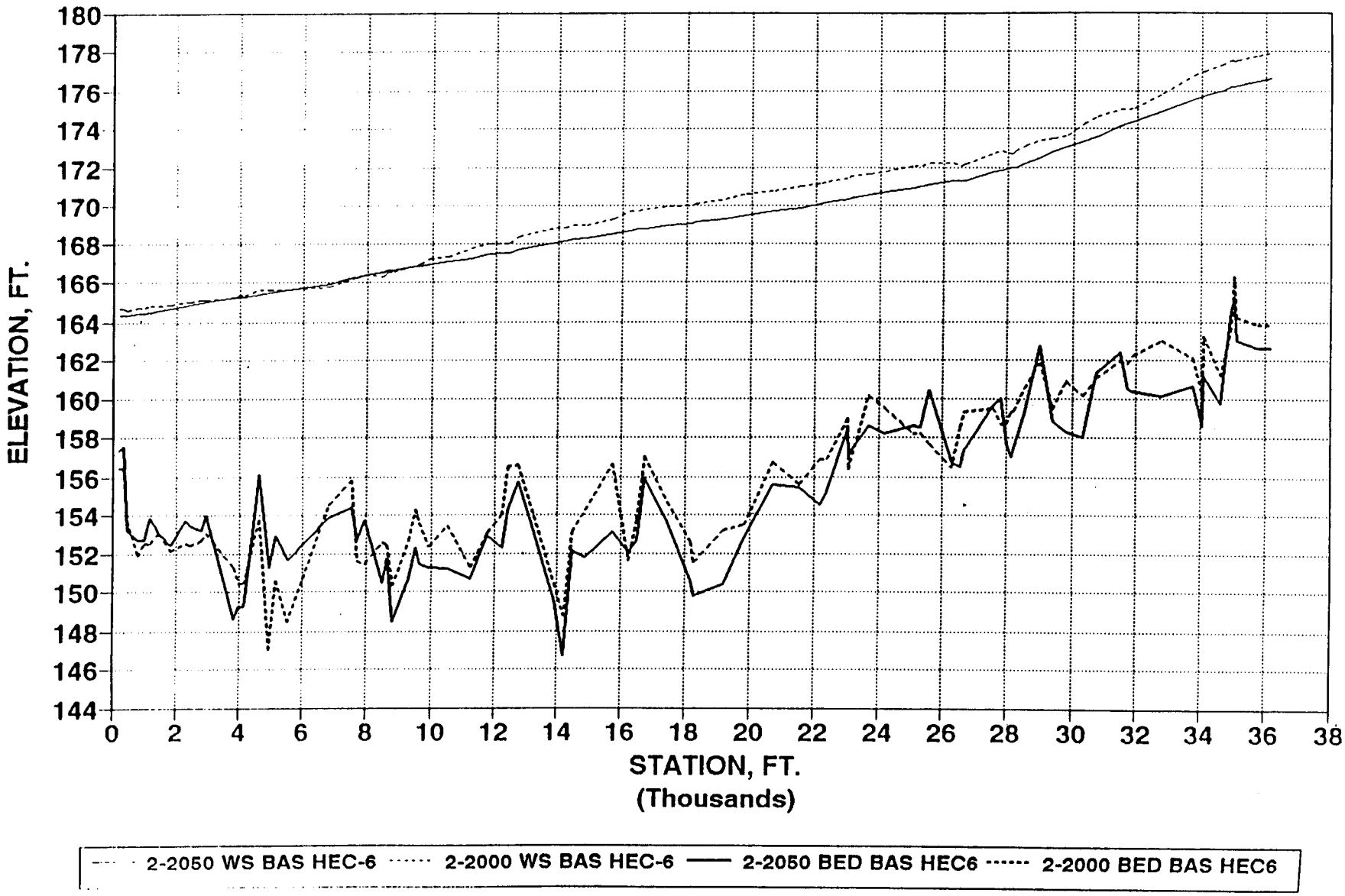


Figure C-15d

BED AND WATER SURFACE PROFILES
BRANCH 7, RAMAPO RIVER

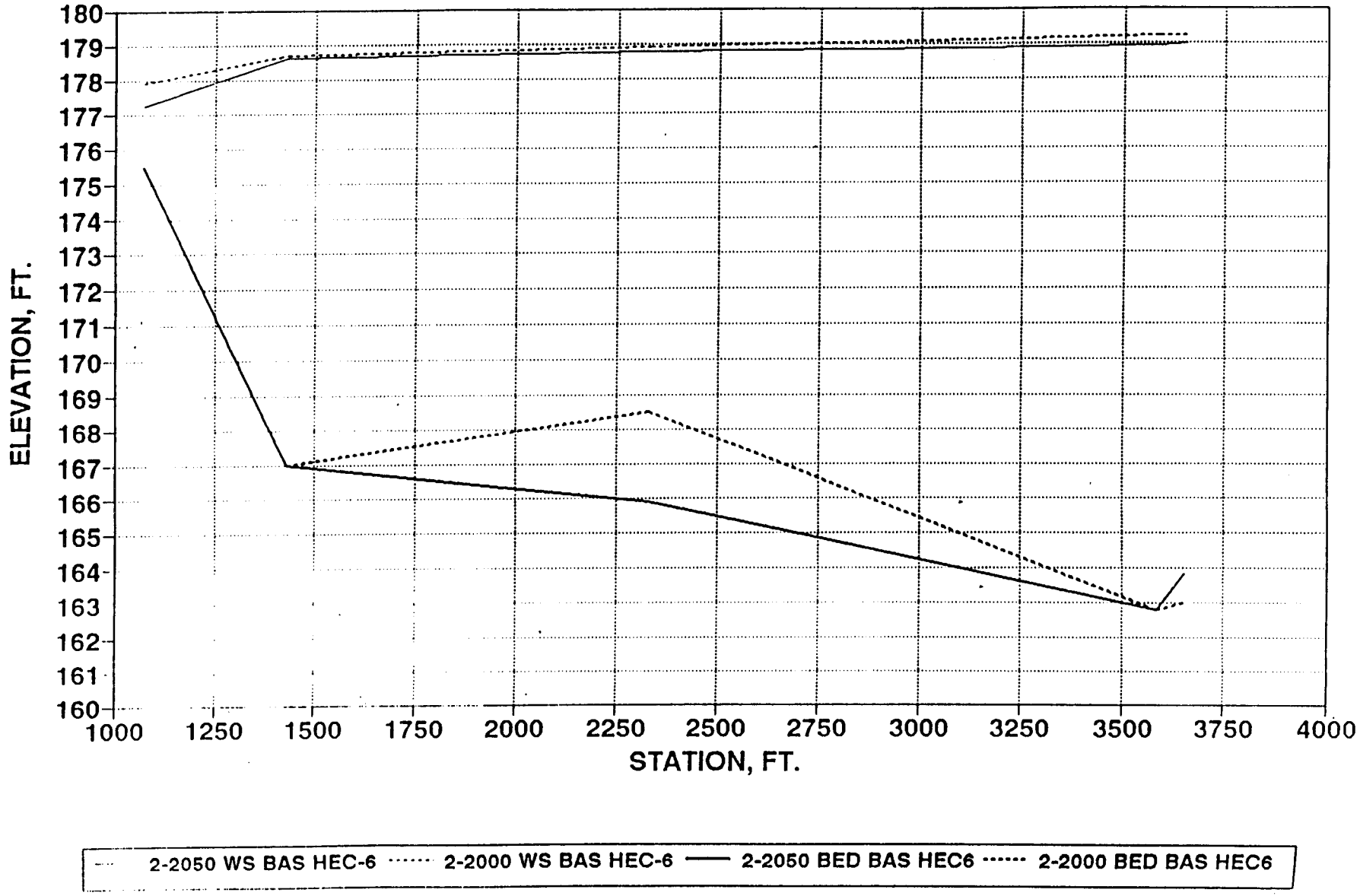


Figure C-158e

BED AND WATER SURFACE PROFILES
BRANCH 10, BYPASS

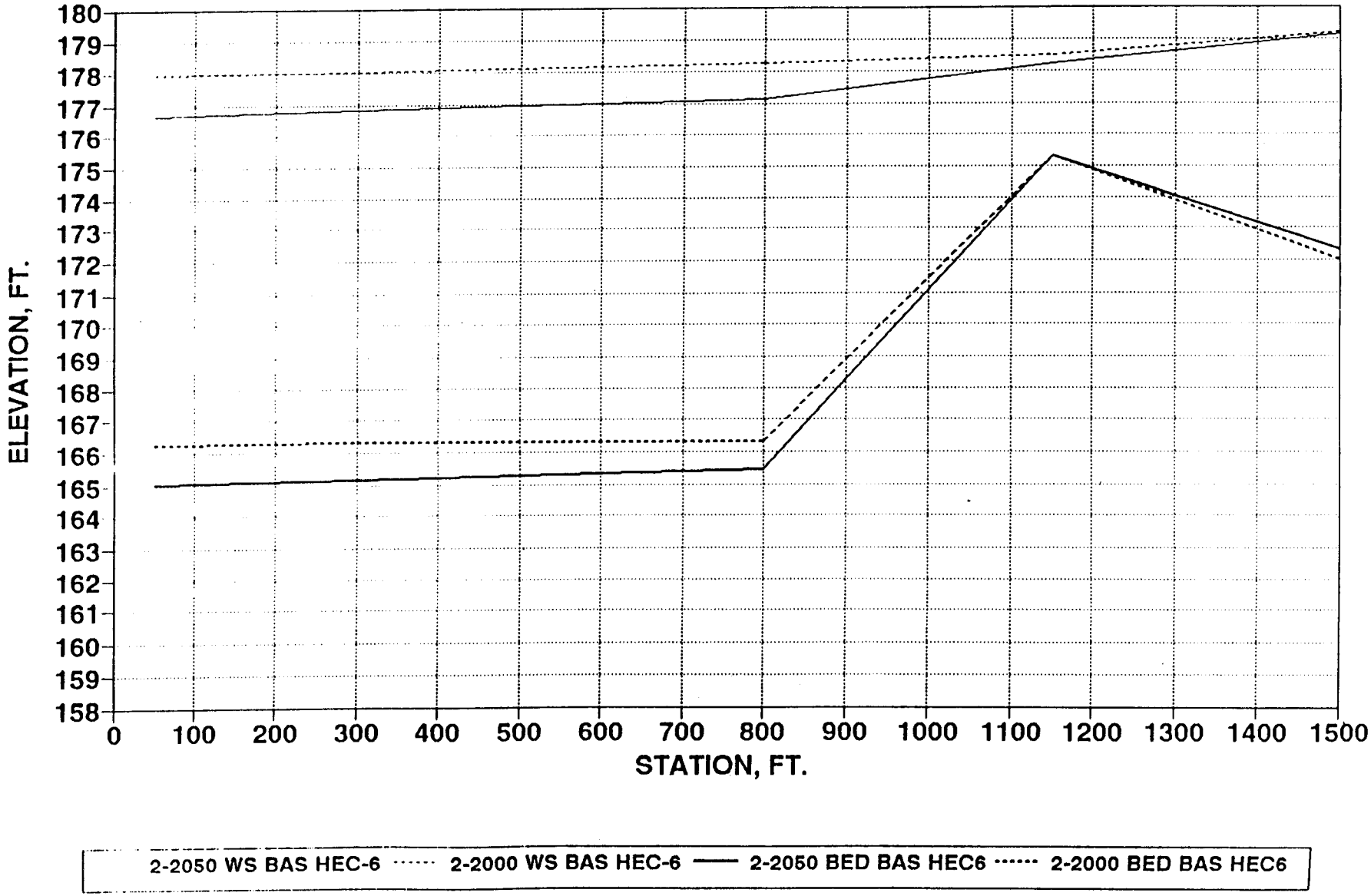


Figure C-158f

BED AND WATER SURFACE PROFILES
BRANCH 11, PEQUANNOCK RIVER

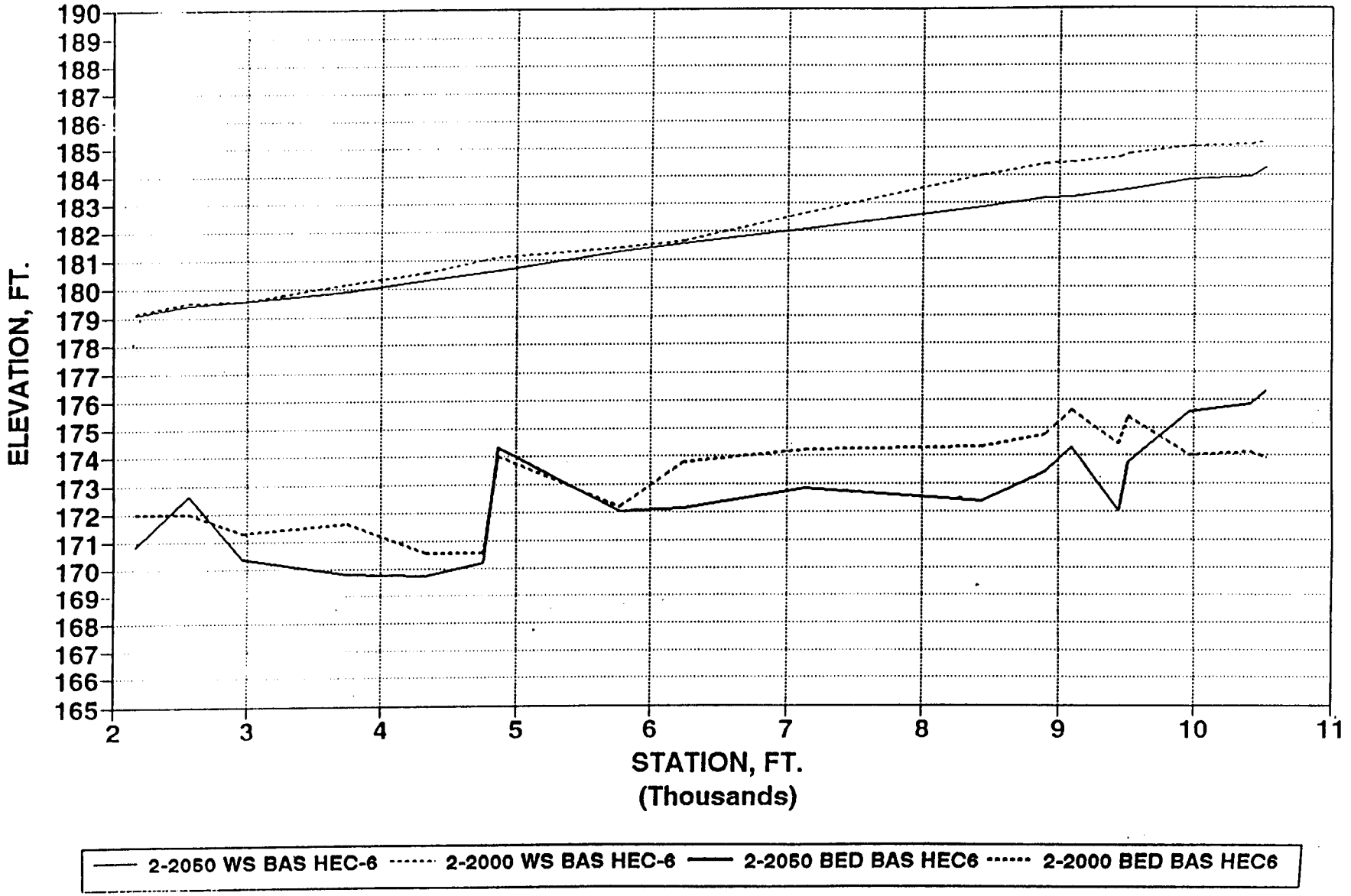


Figure C-158g

BED AND WATER SURFACE PROFILES
BRANCH 13, WANAQUE RIVER

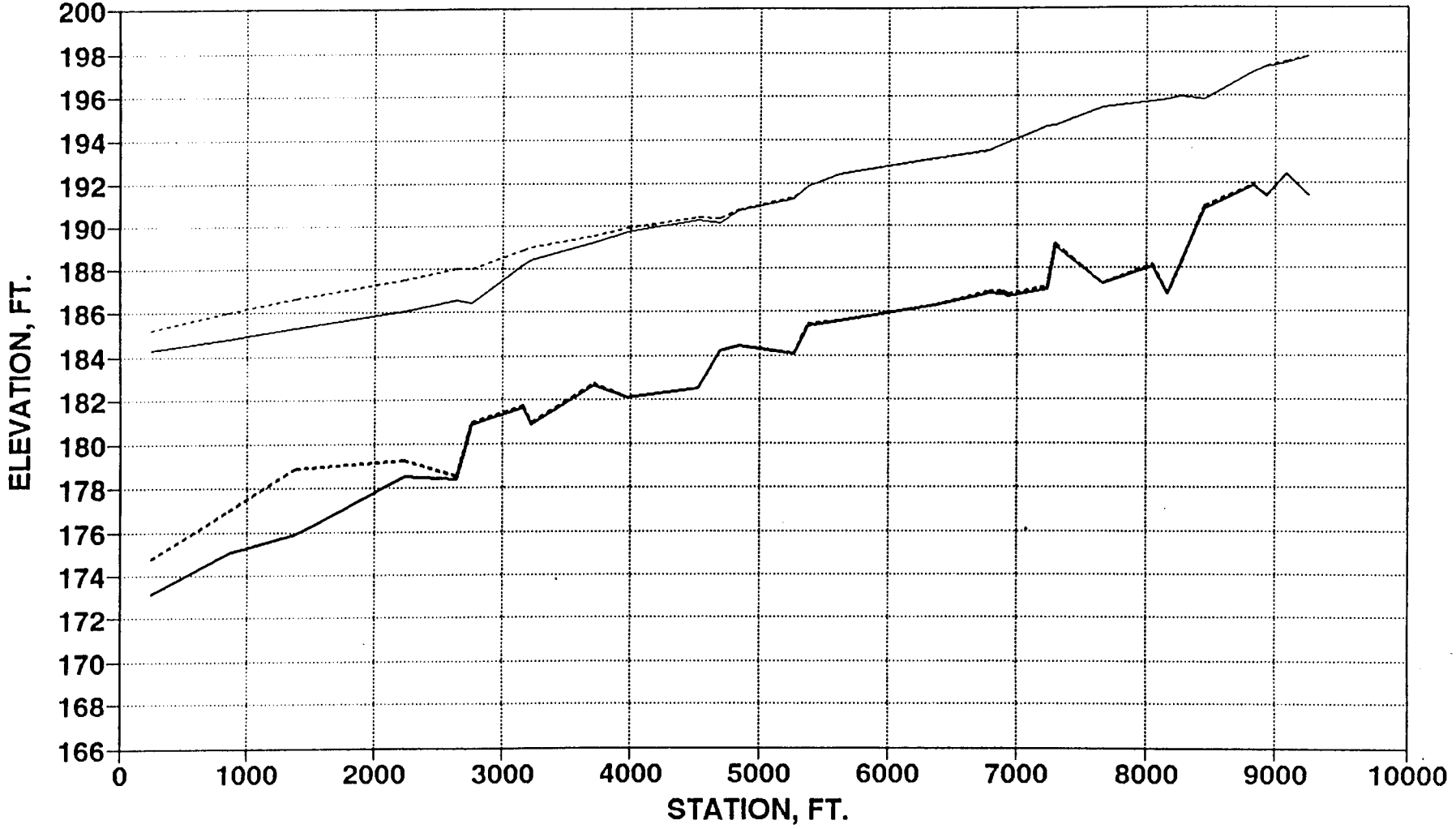


Figure C-158h

BED AND WATER SURFACE PROFILES
BRANCH 6, POMPTON RIVER

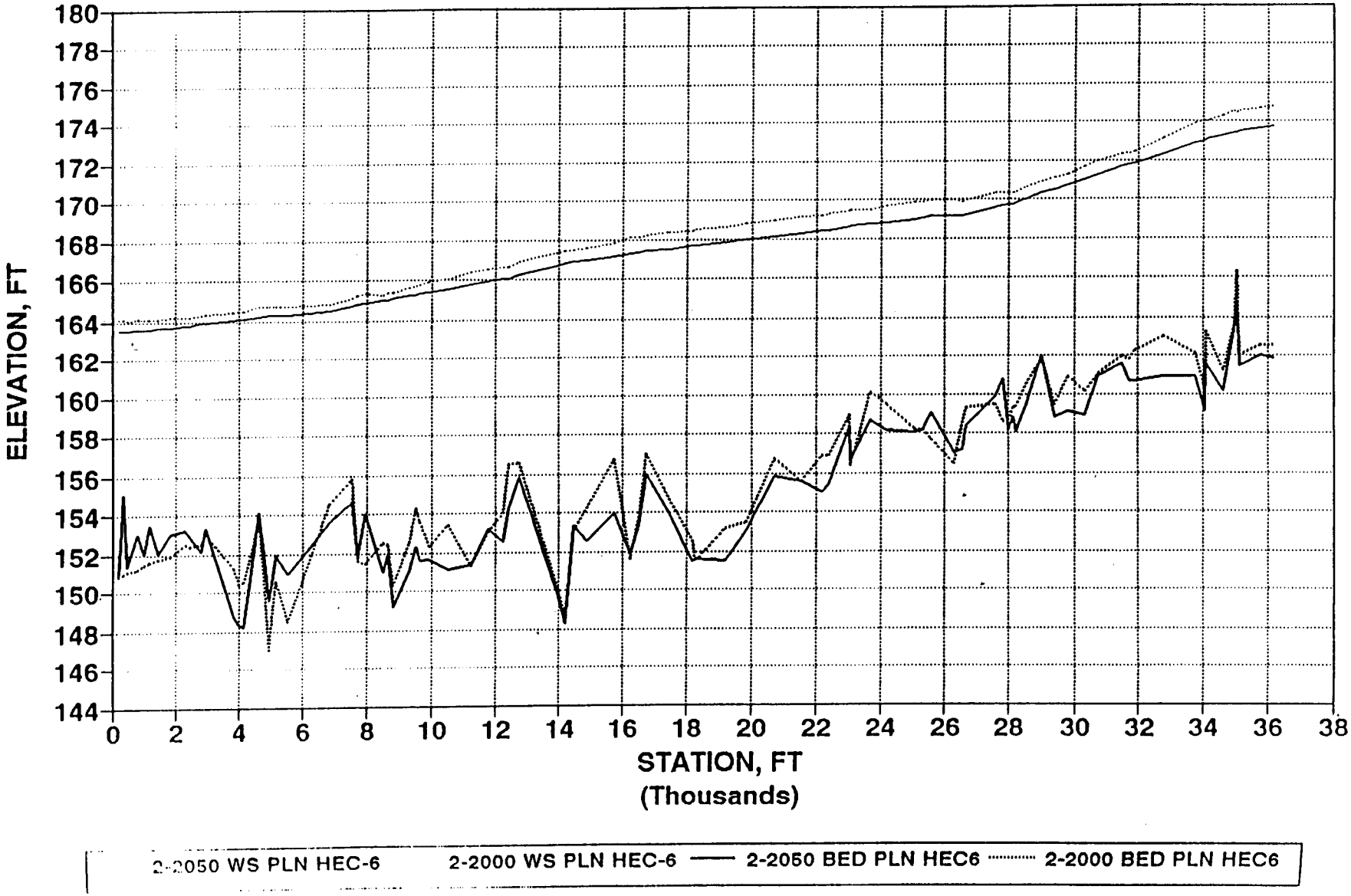


Figure C-159a

BED AND WATER SURFACE PROFILES
BRANCH 7, RAMAPO RIVER

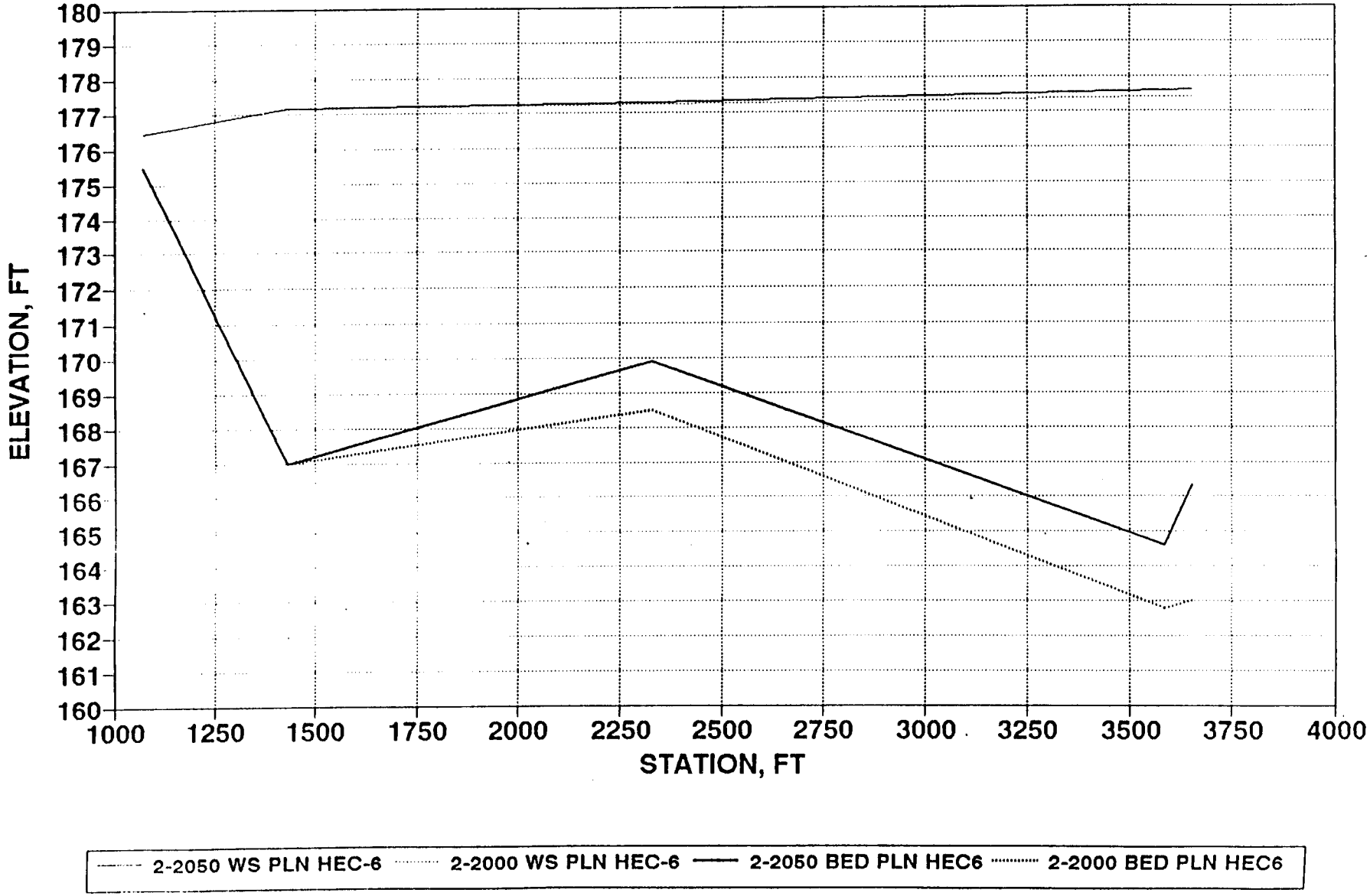


Figure C-159b

BED AND WATER SURFACE PROFILES
BRANCH 10, BYPASS

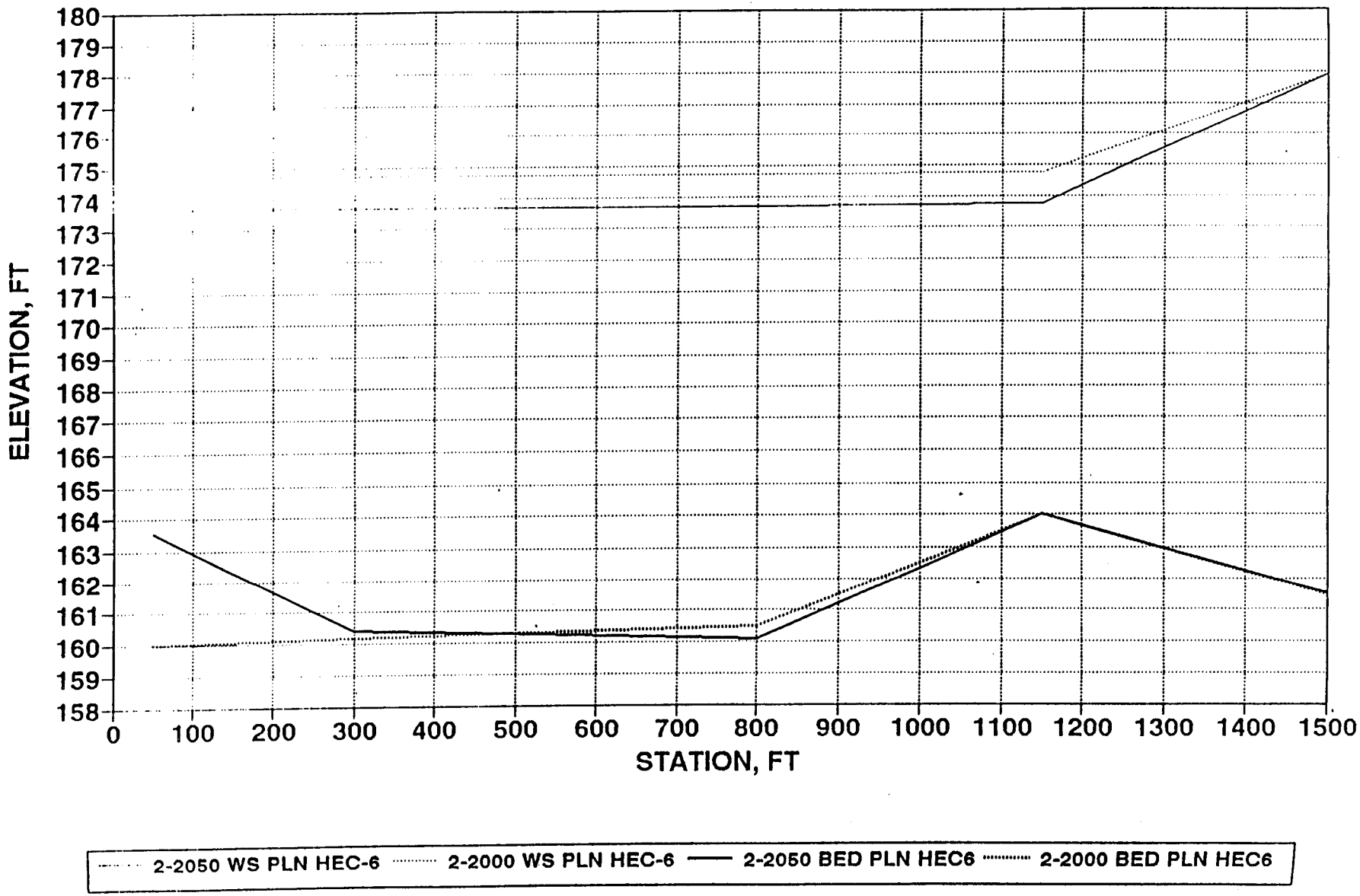


Figure C-159c

BED AND WATER SURFACE PROFILES
BRANCH 11, PEQUANNOCK RIVER

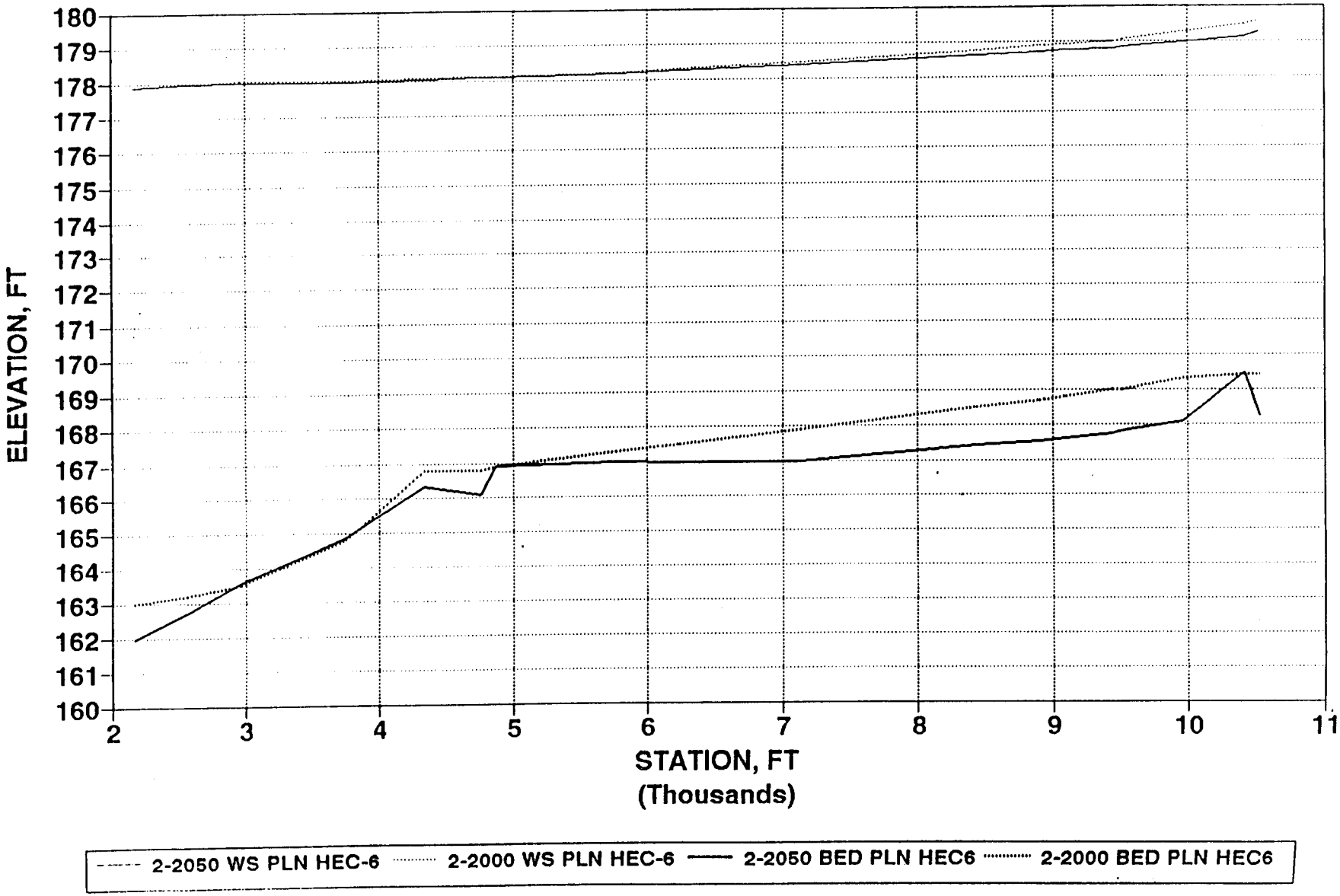


Figure C-159d

BED AND WATER SURFACE PROFILES
BRANCH 13, WANAQUE RIVER

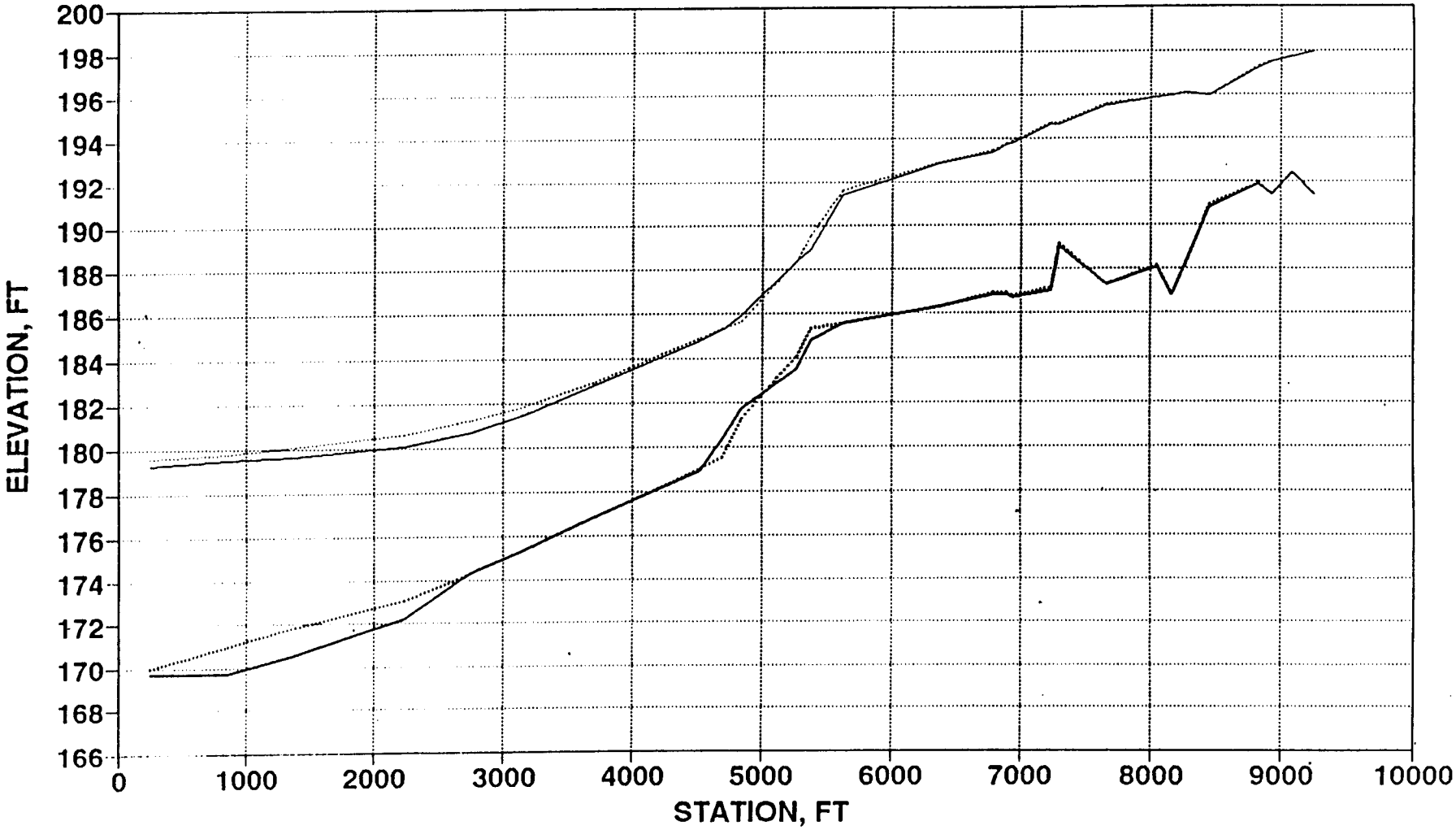


Figure C-159e

BED AND WATER SURFACE PROFILES
BRANCH 1, PASSAIC RIVER

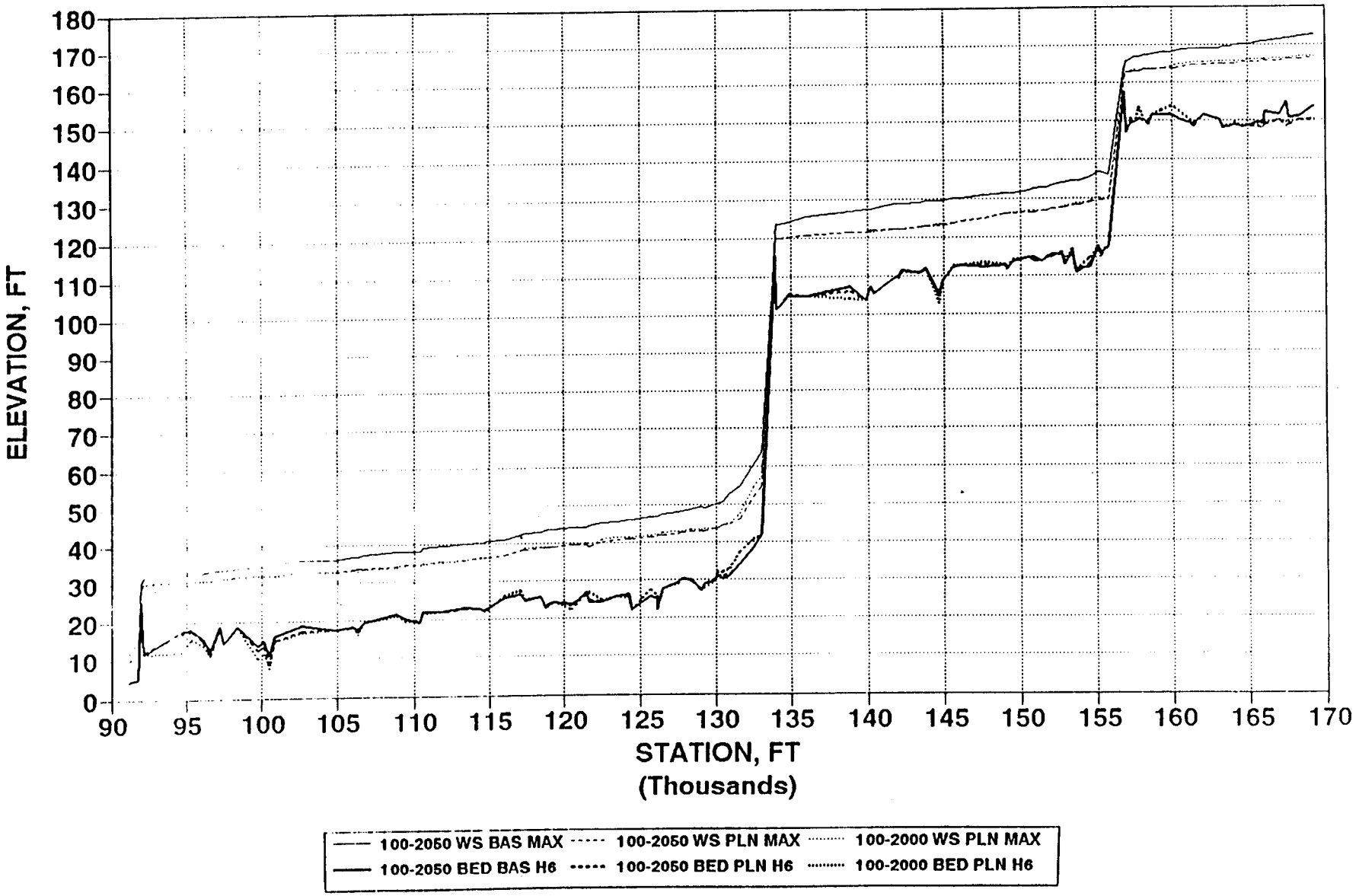


Figure C-16a

BED AND WATER SURFACE PROFILES
BRANCH 2, DEEPAVAAL BROOK

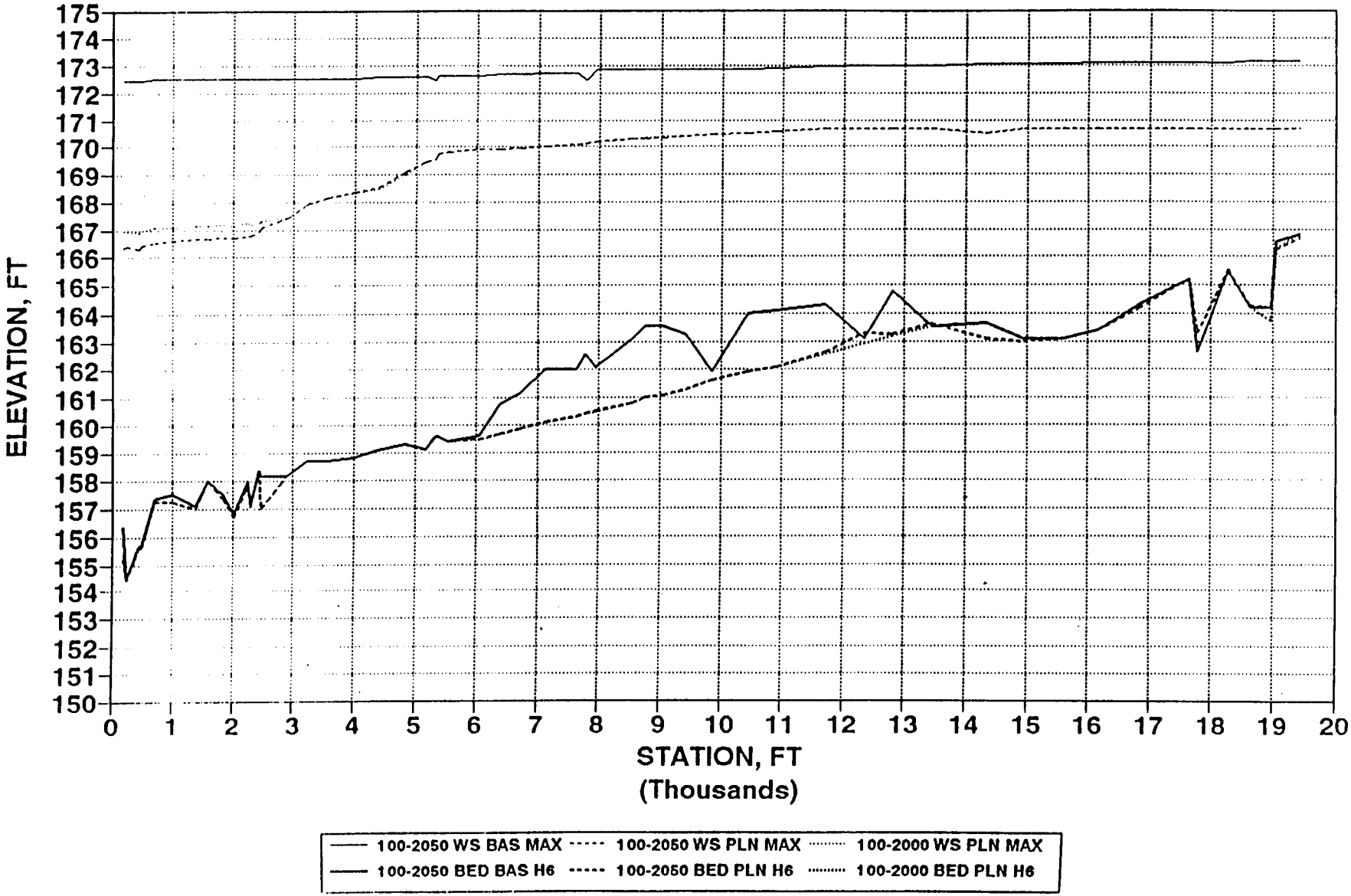


Figure C-160b

BED AND WATER SURFACE PROFILES
BRANCH 3, PASSAIC RIVER

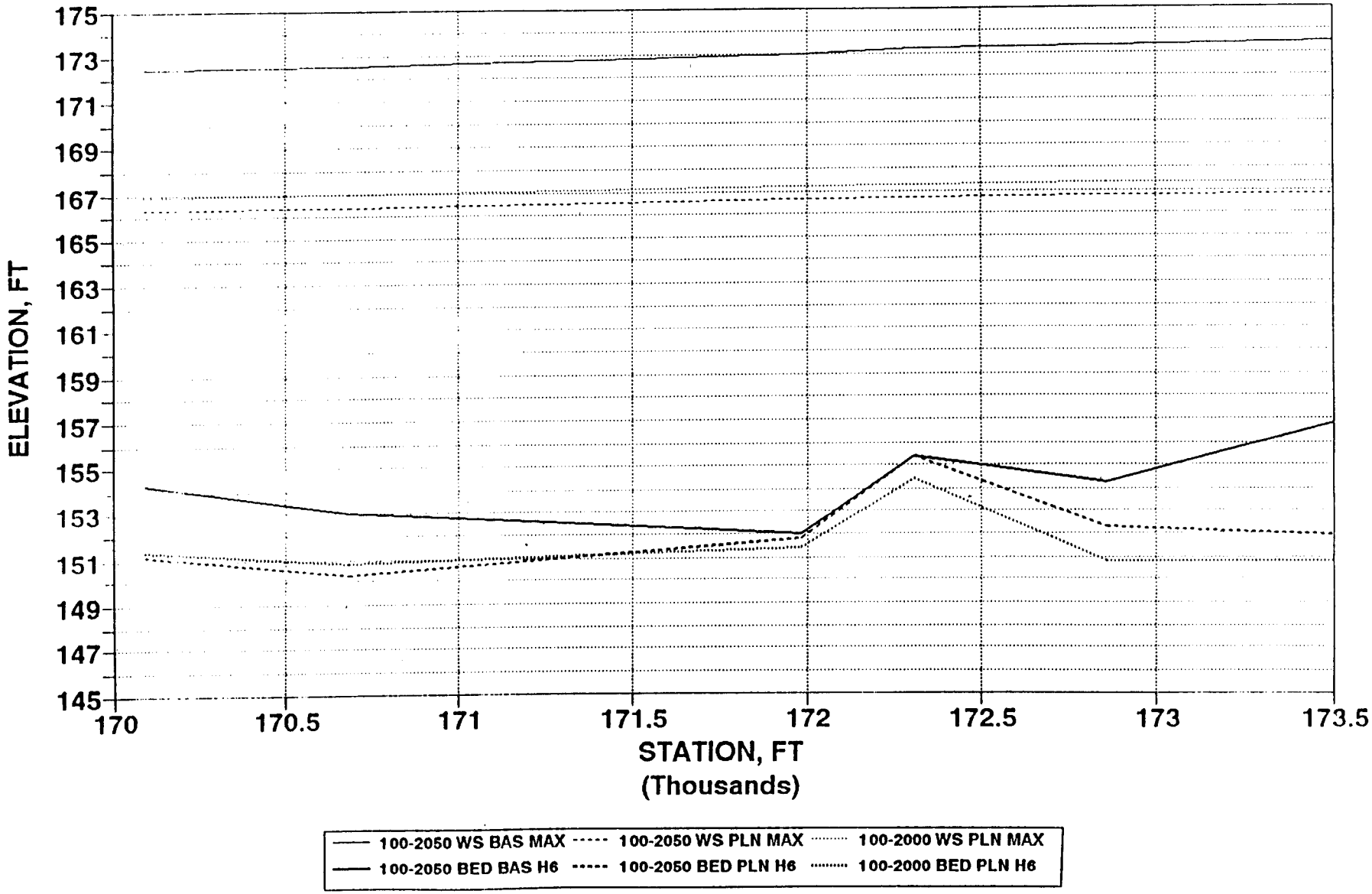


Figure C-160c

BED AND WATER SURFACE PROFILES
BRANCH 4, PASSAIC RIVER

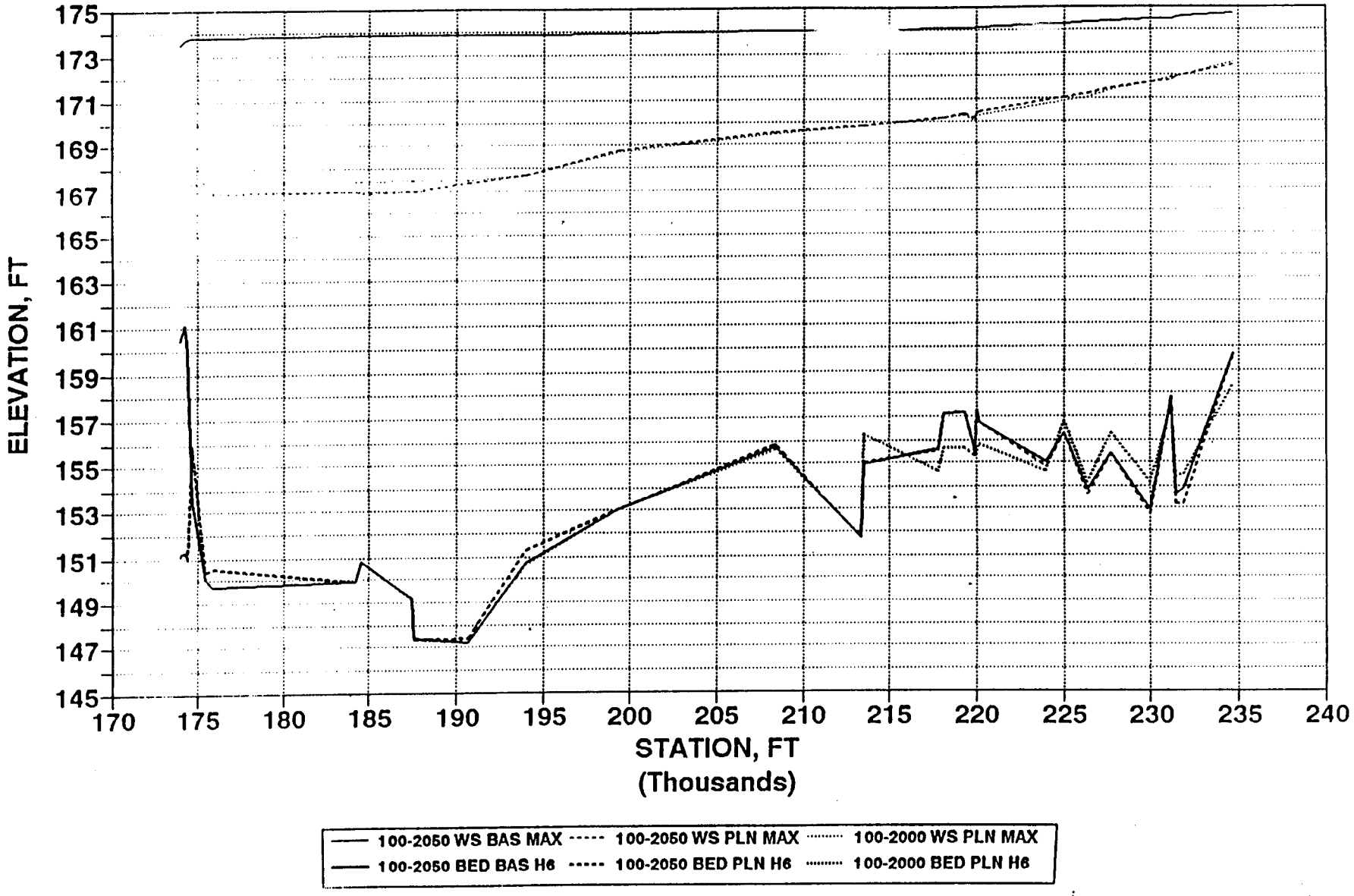


Figure C-160d

BED AND WATER SURFACE PROFILES
BRANCH 5, PASSAIC RIVER

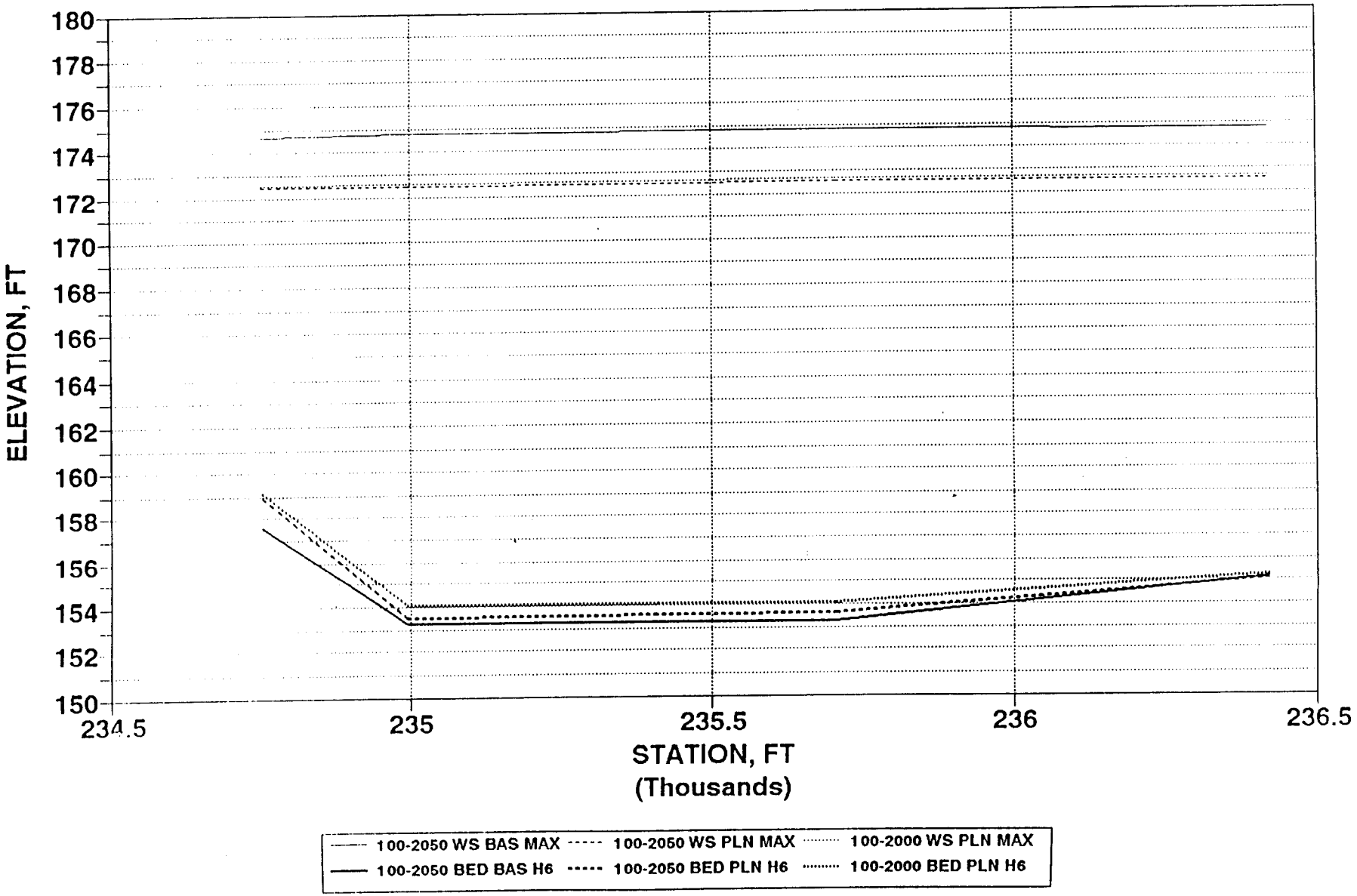


Figure C-160e

BED AND WATER SURFACE PROFILES
BRANCH 6, POMPTON RIVER

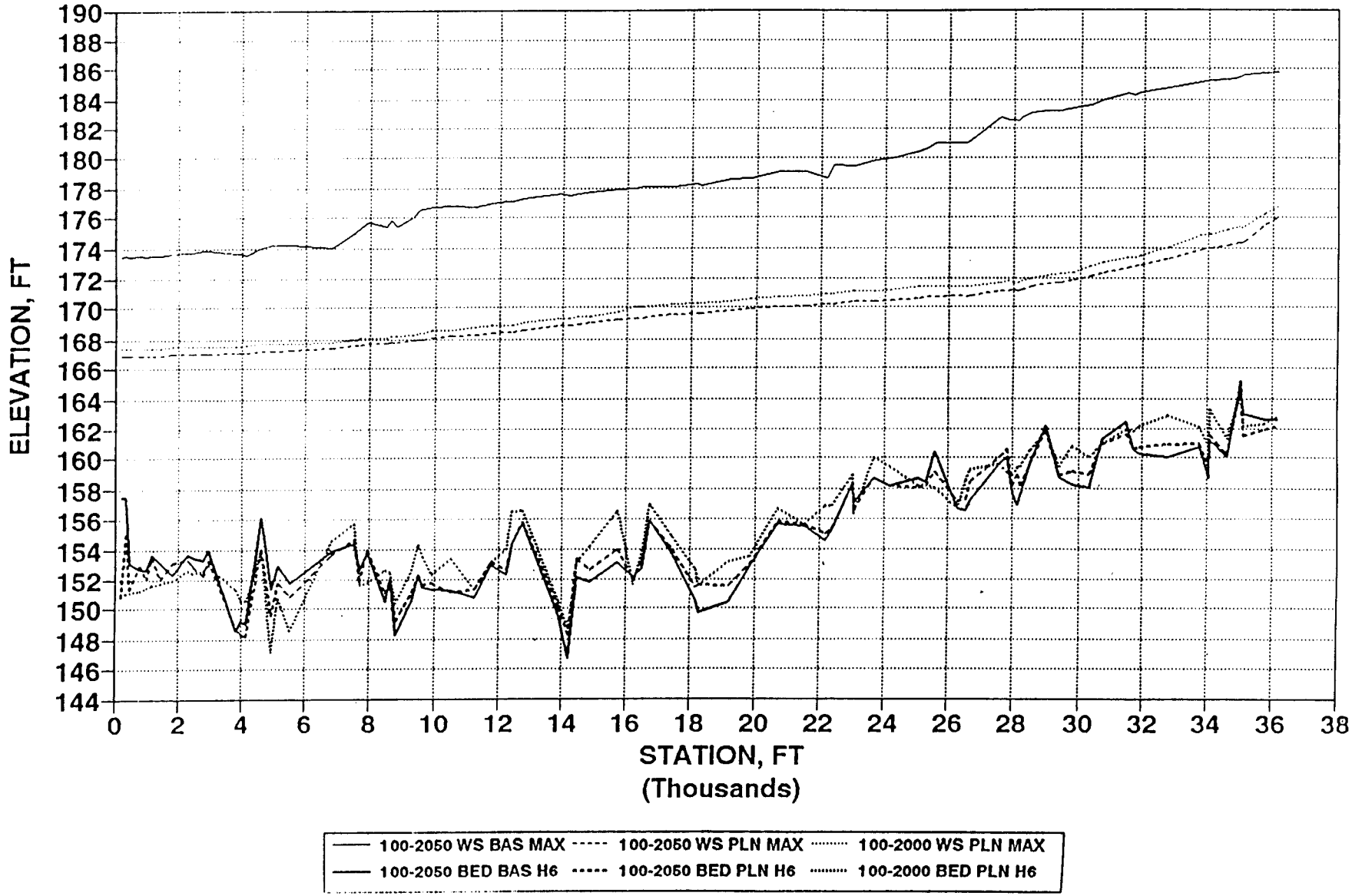


Figure C-160f

BED AND WATER SURFACE PROFILES
BRANCH 7, RAMAPO RIVER

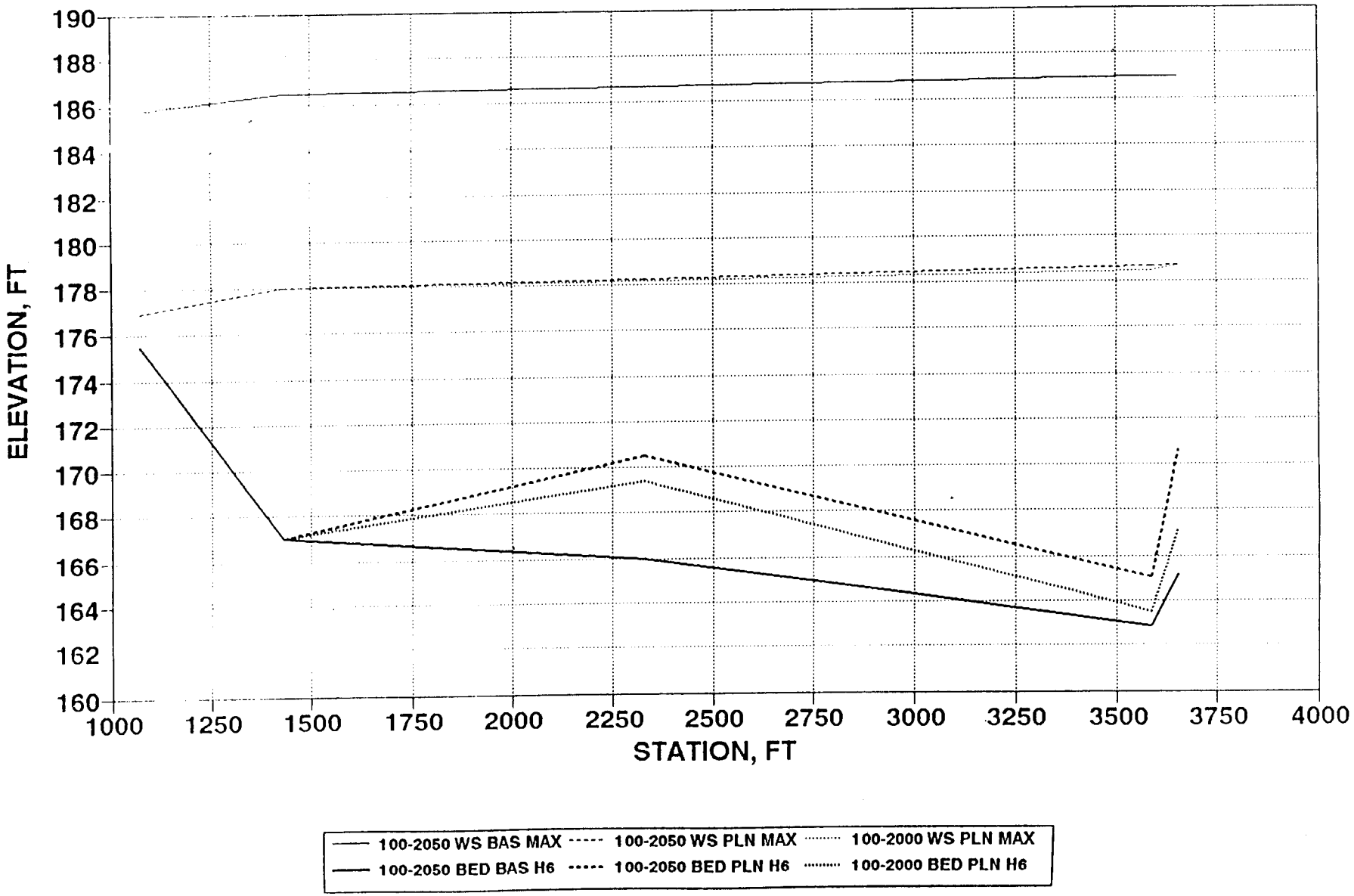


Figure C-160g

BED AND WATER SURFACE PROFILES
BRANCH 8, RAMAPO RIVER

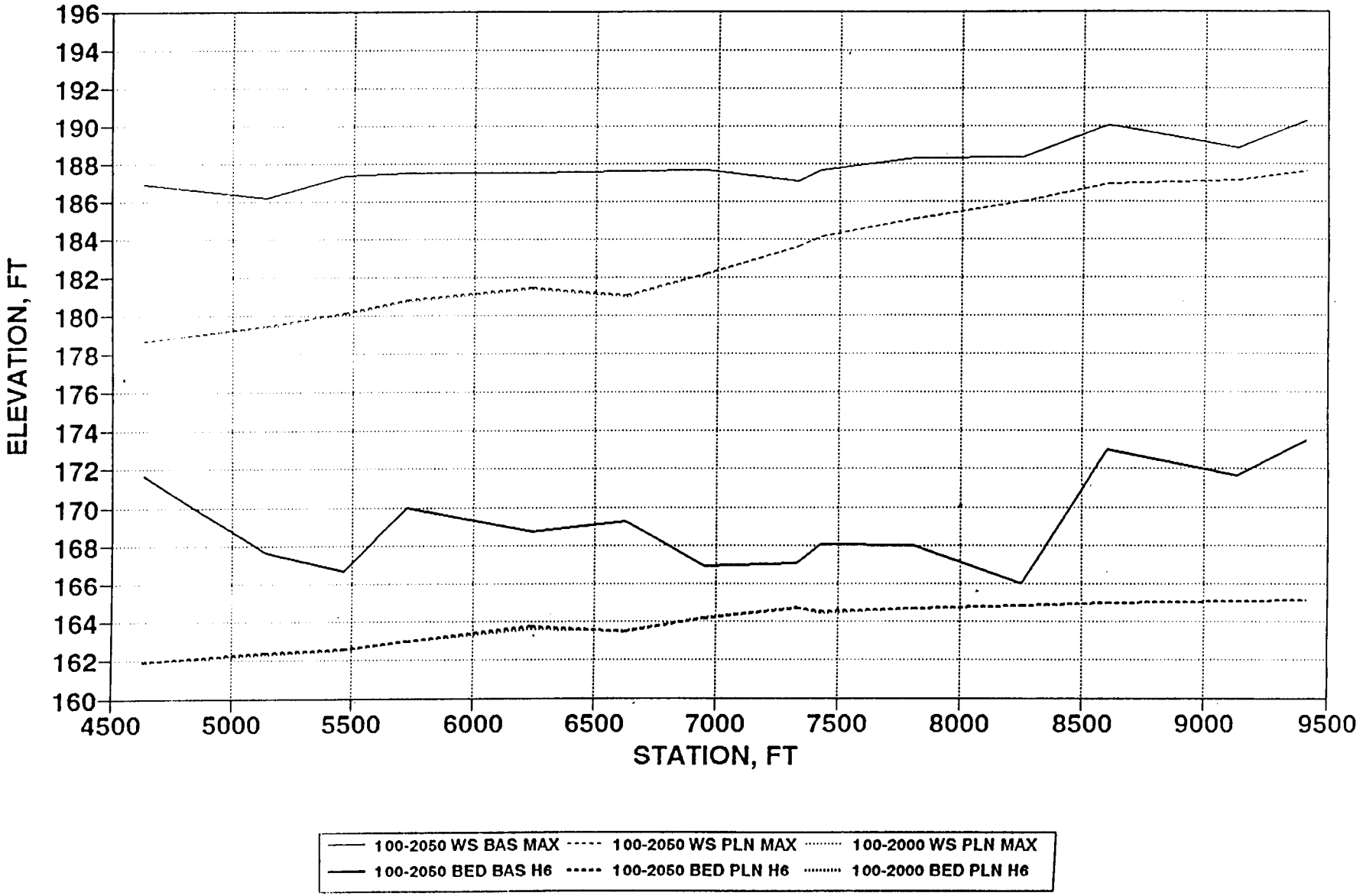


Figure C-160h

BED AND WATER SURFACE PROFILES
BRANCH 9,

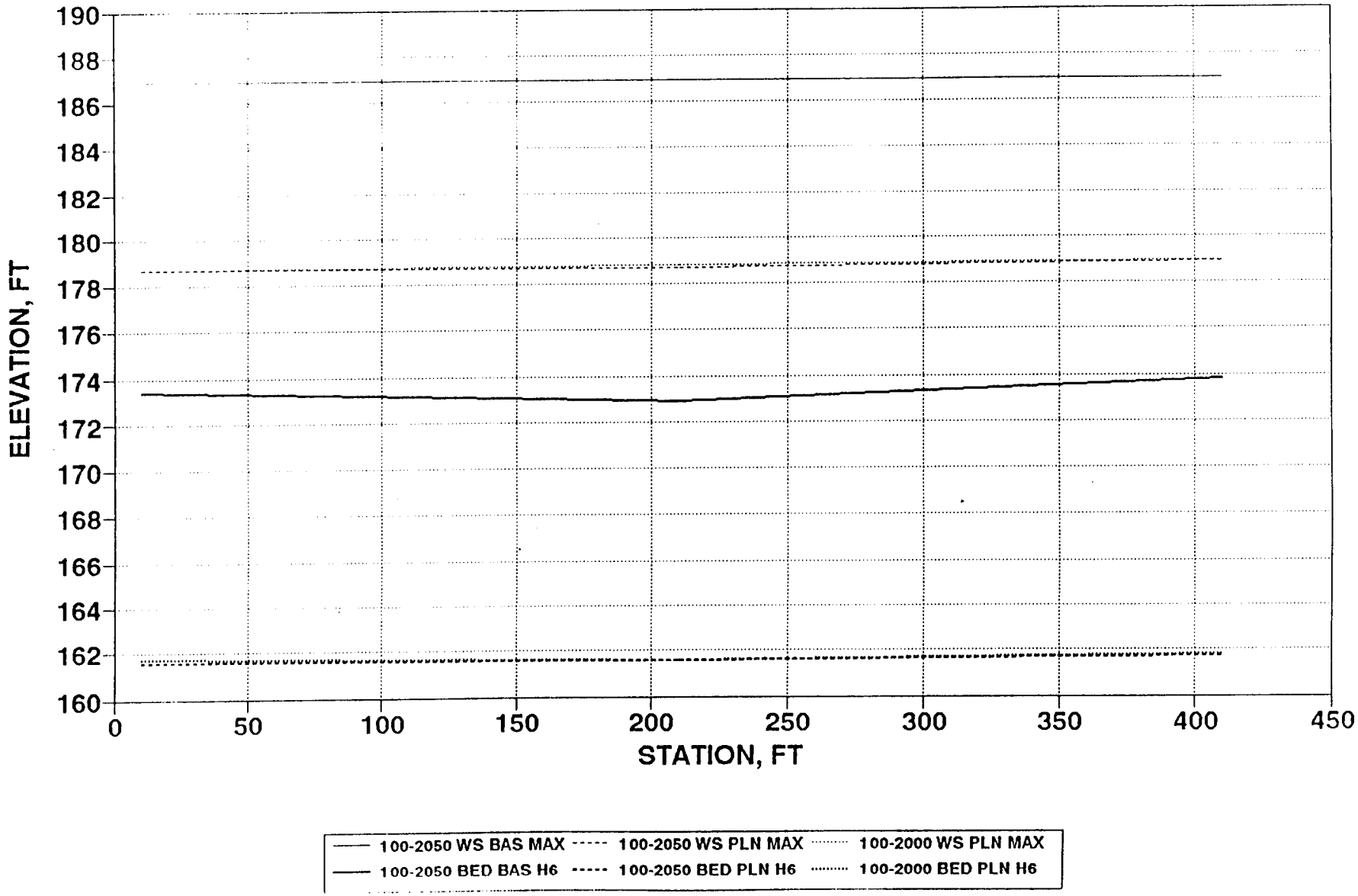


Figure C-1601

BED AND WATER SURFACE PROFILES
BRANCH 10, BYPASS

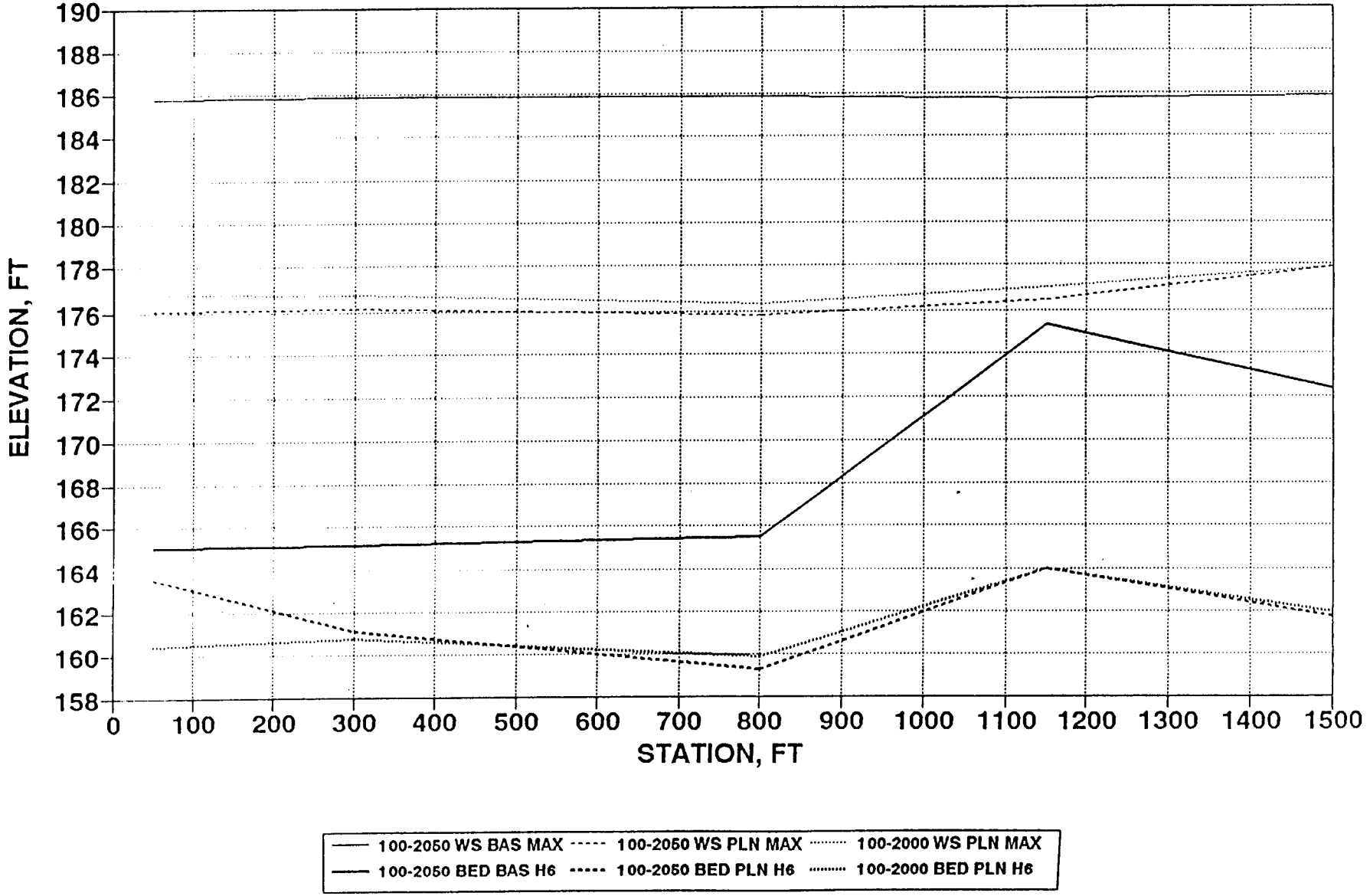


Figure C-160j

BED AND WATER SURFACE PROFILES
BRANCH 11, PEQUANNOCK RIVER

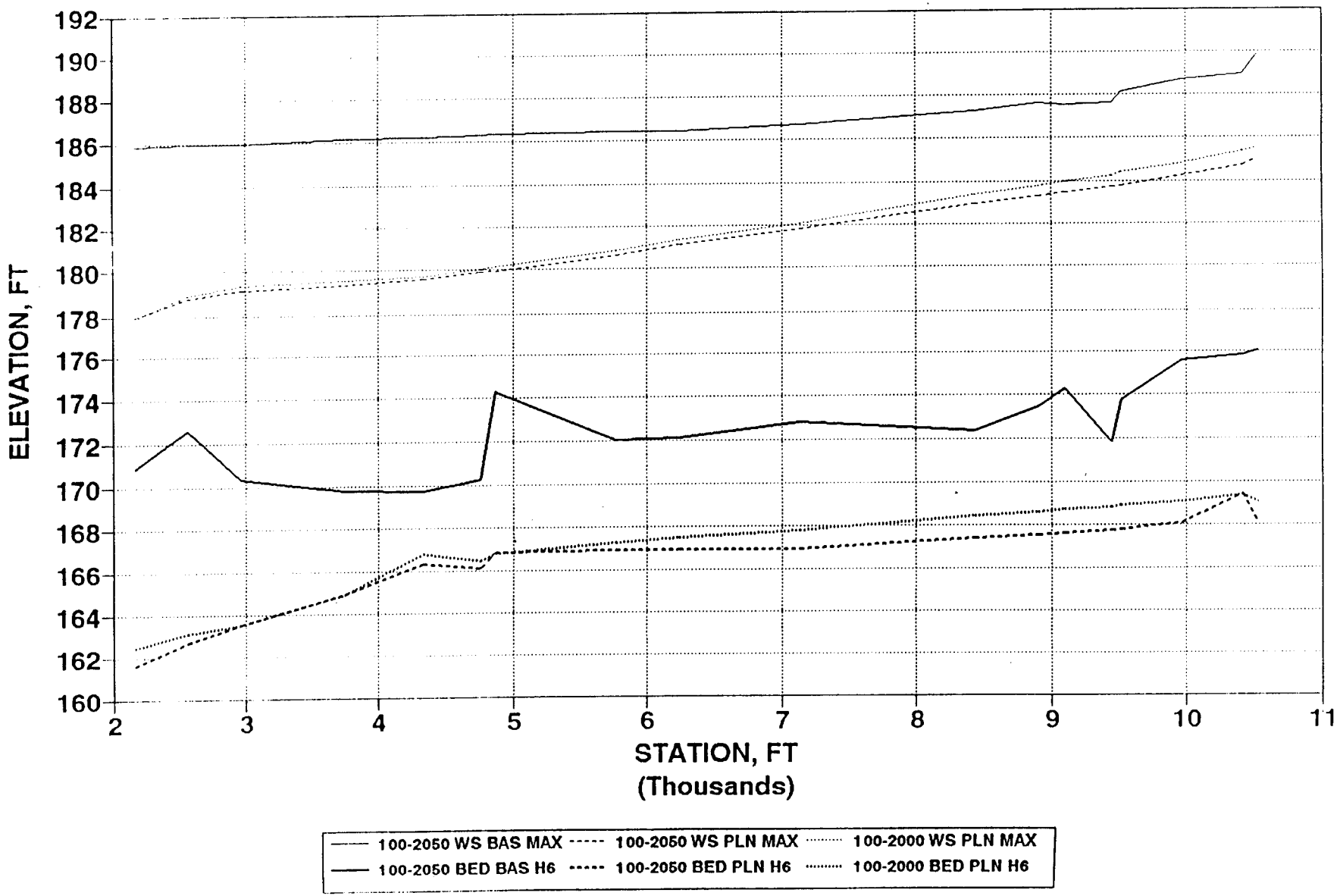


Figure C-160k

BED AND WATER SURFACE PROFILES
BRANCH 12, PEQUANNOCK RIVER

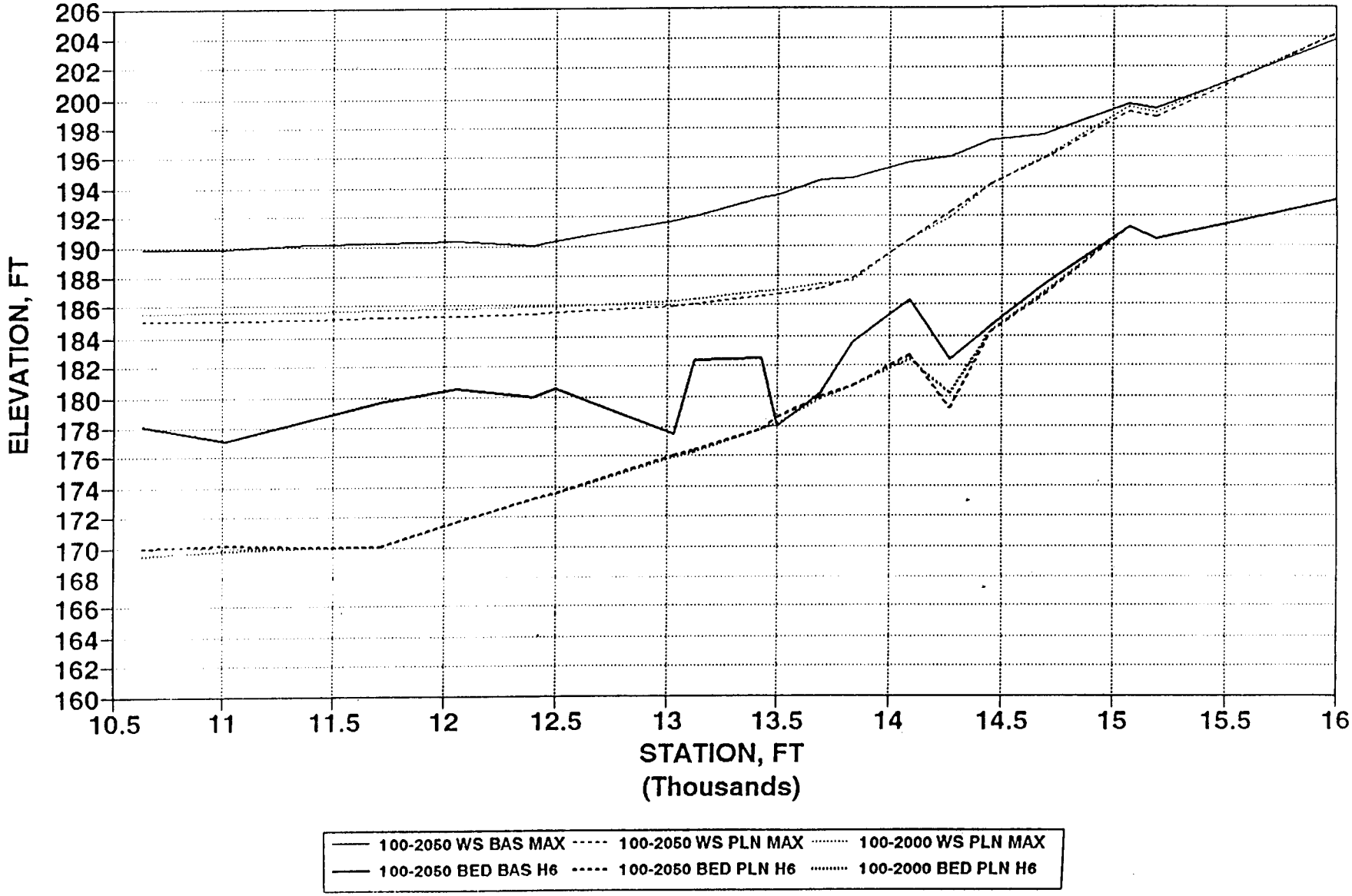


Figure C-1601

BED AND WATER SURFACE PROFILES
BRANCH 13, WANAQUE RIVER

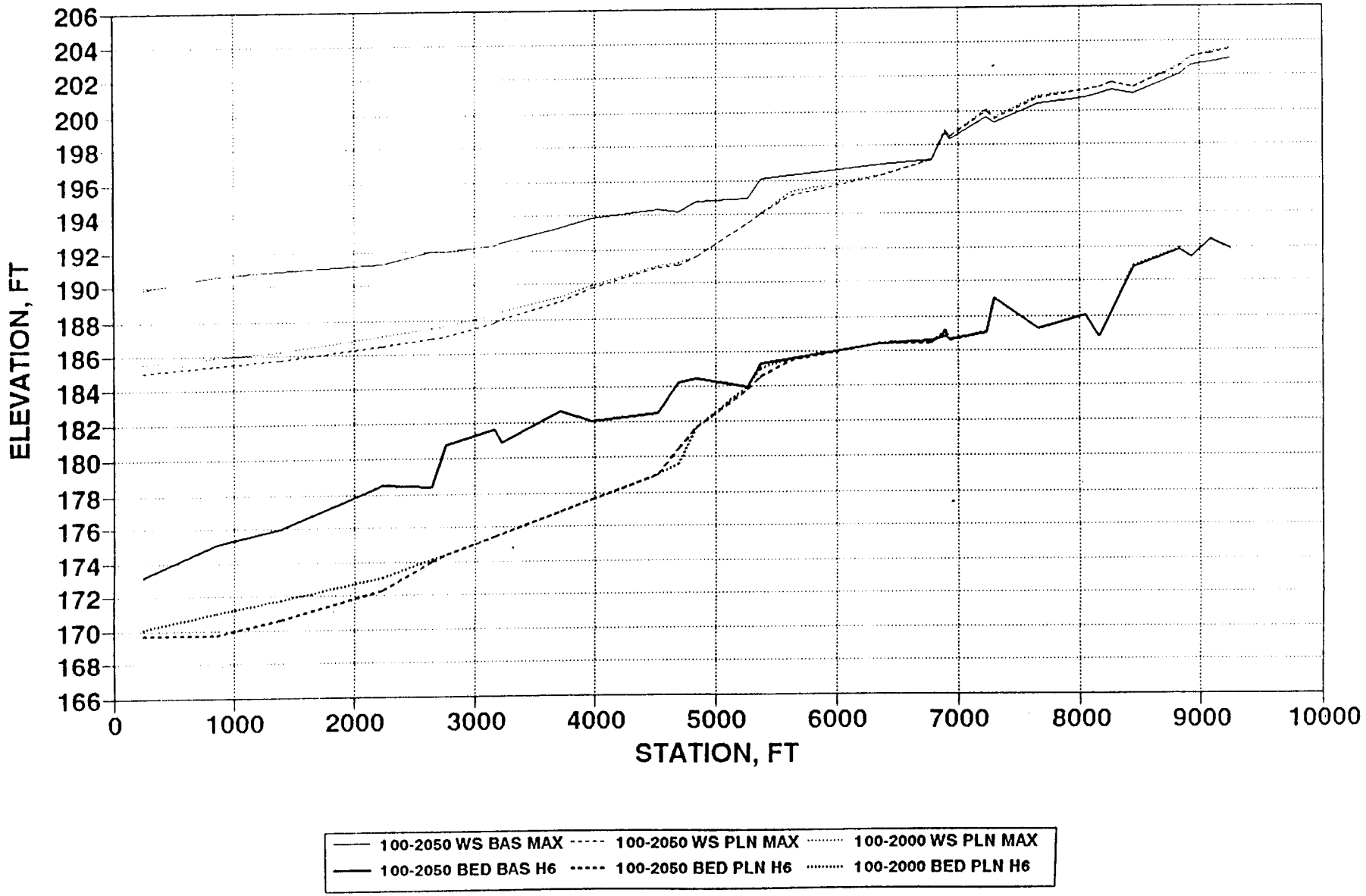


Figure C-160m

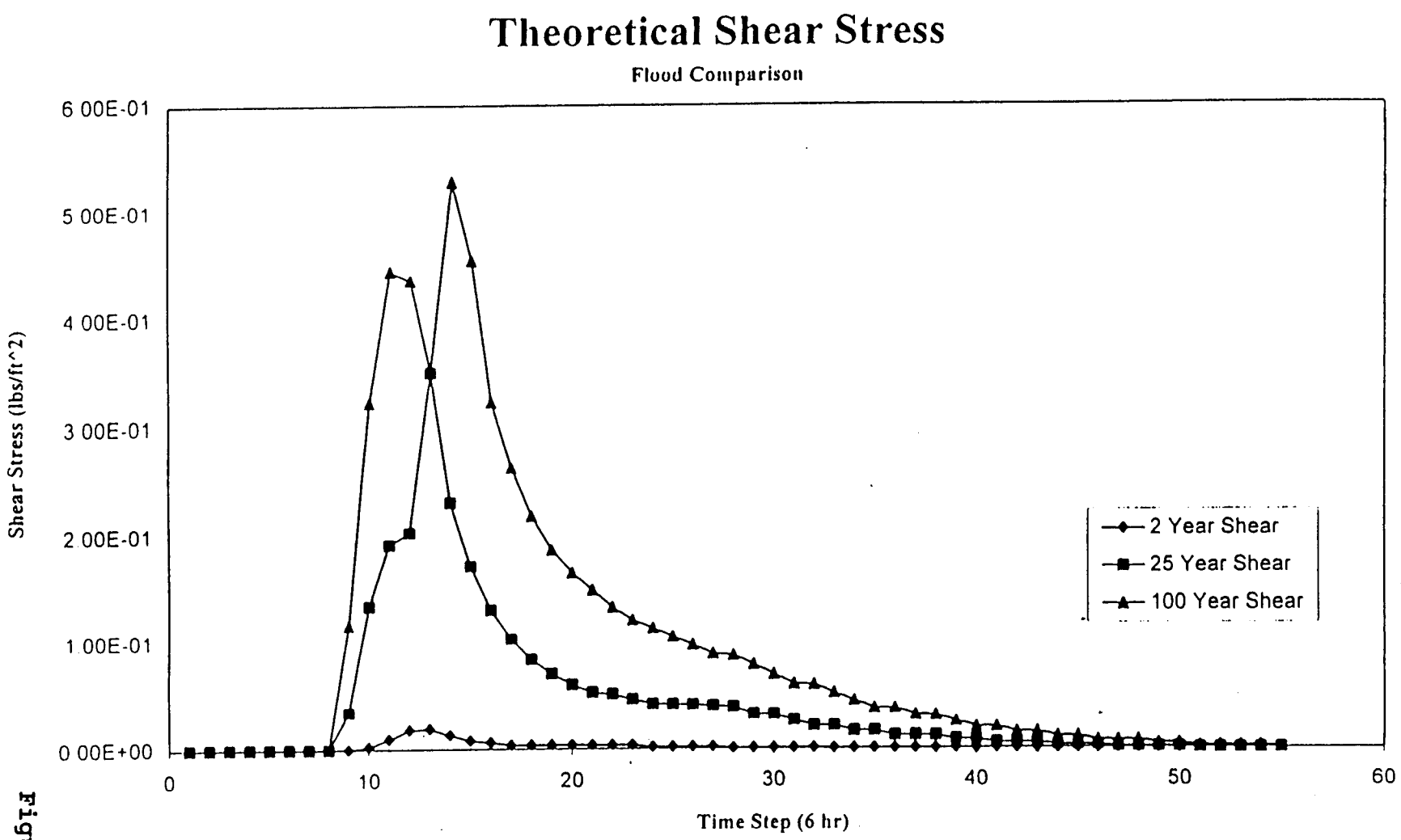
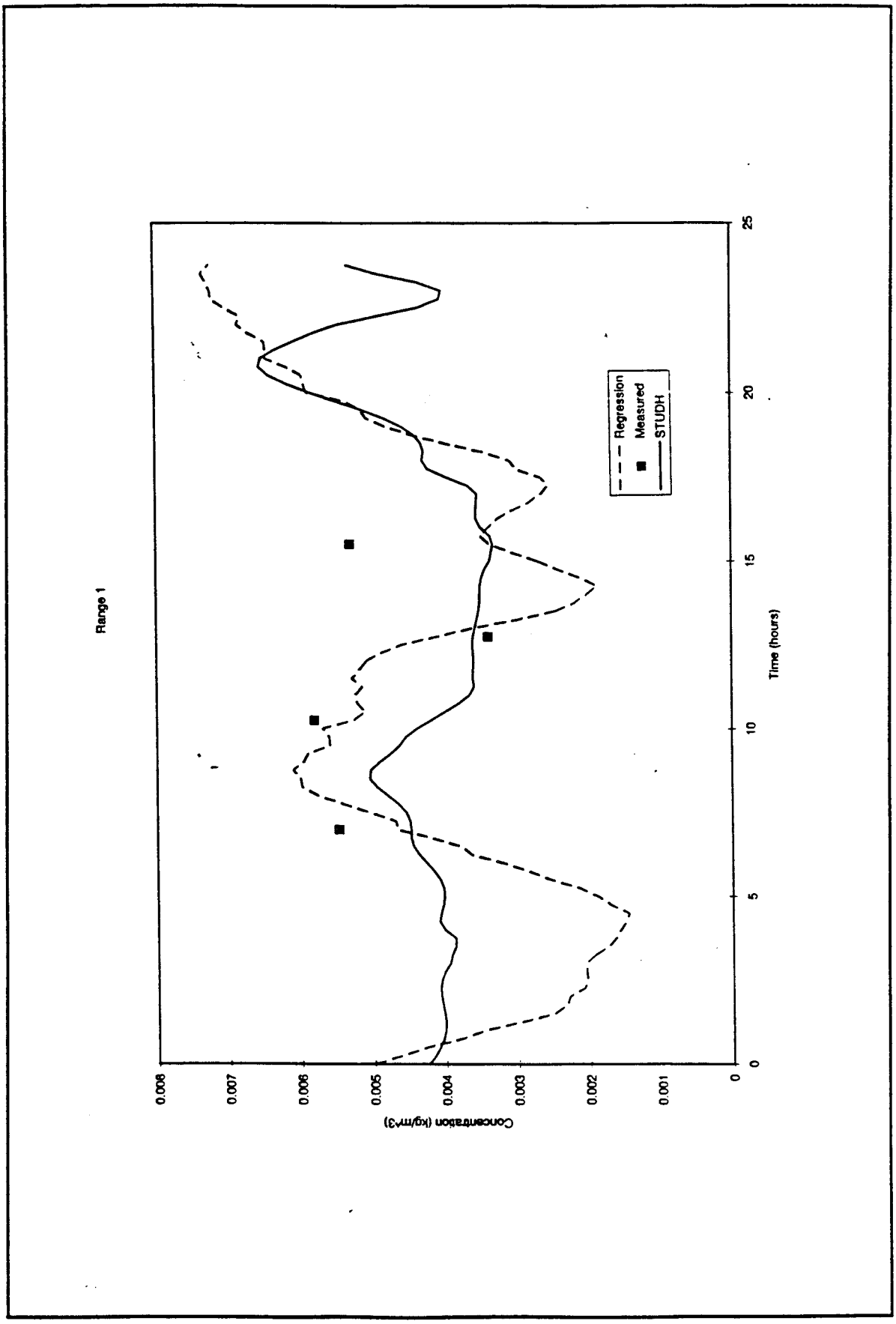


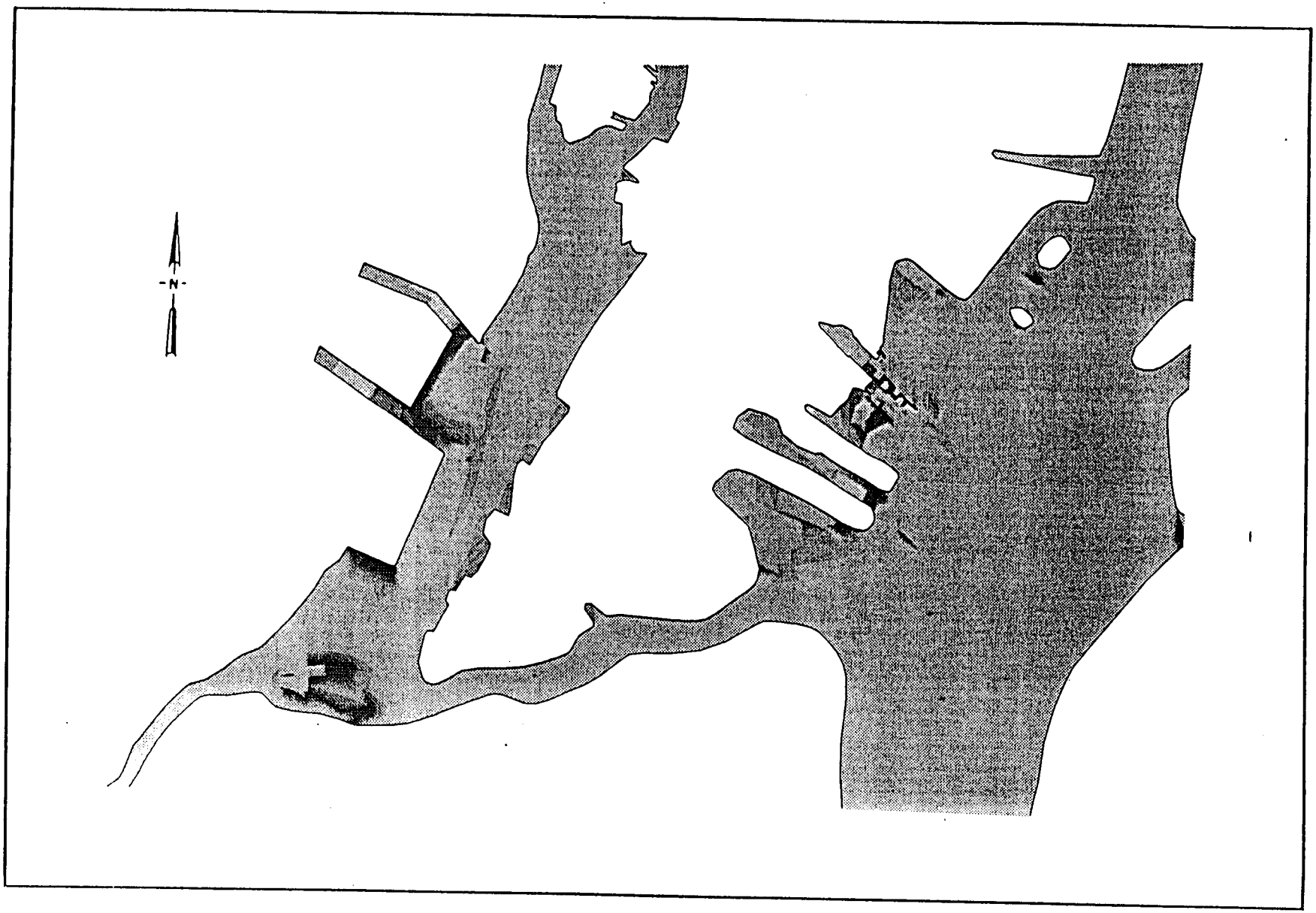
Figure C-161

: Comparison of Shear Stresses



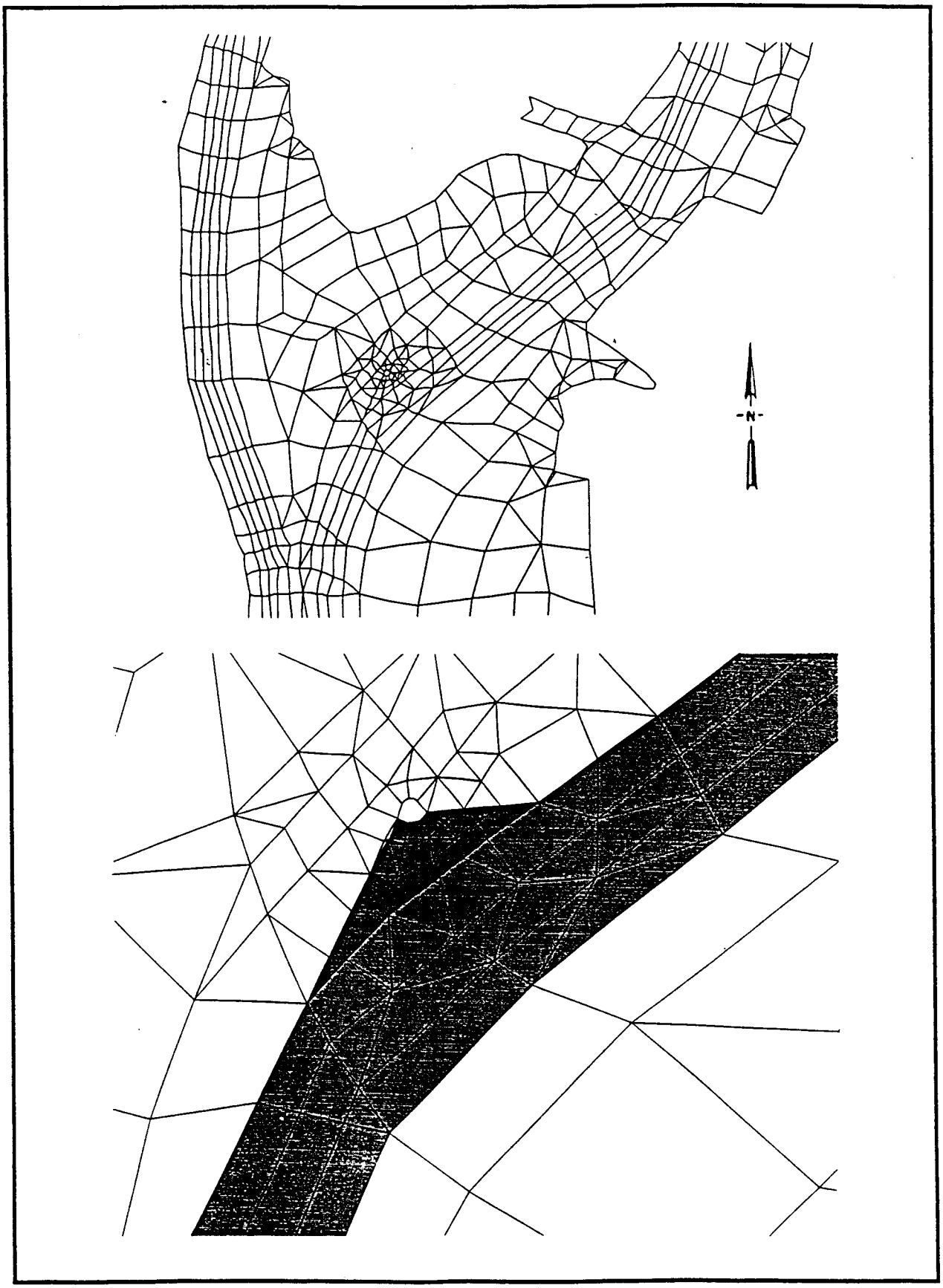
STUDH Concentration Verification

Figure C-162



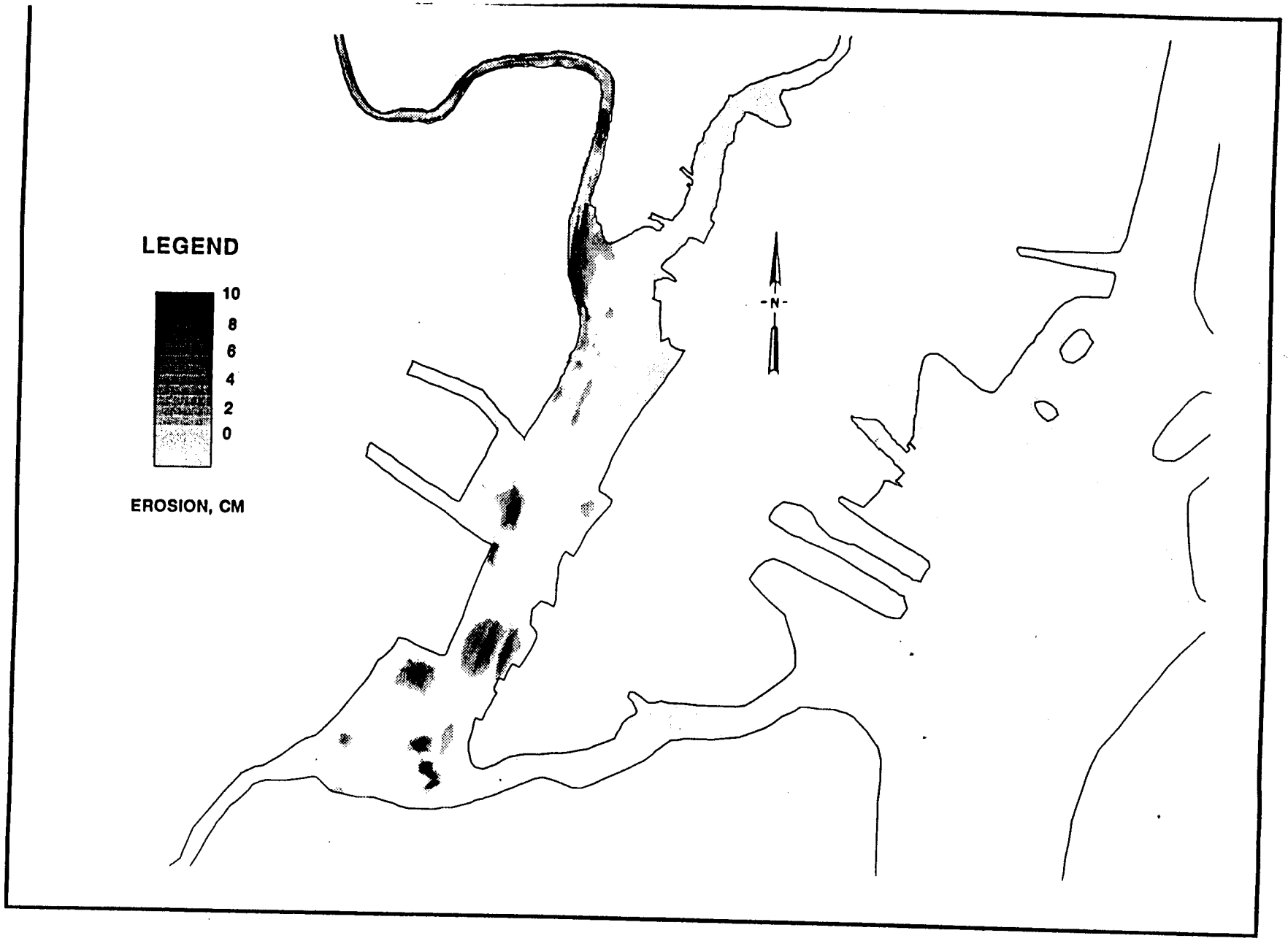
STUDH Deposition Patterns for Verification

Figure C-163



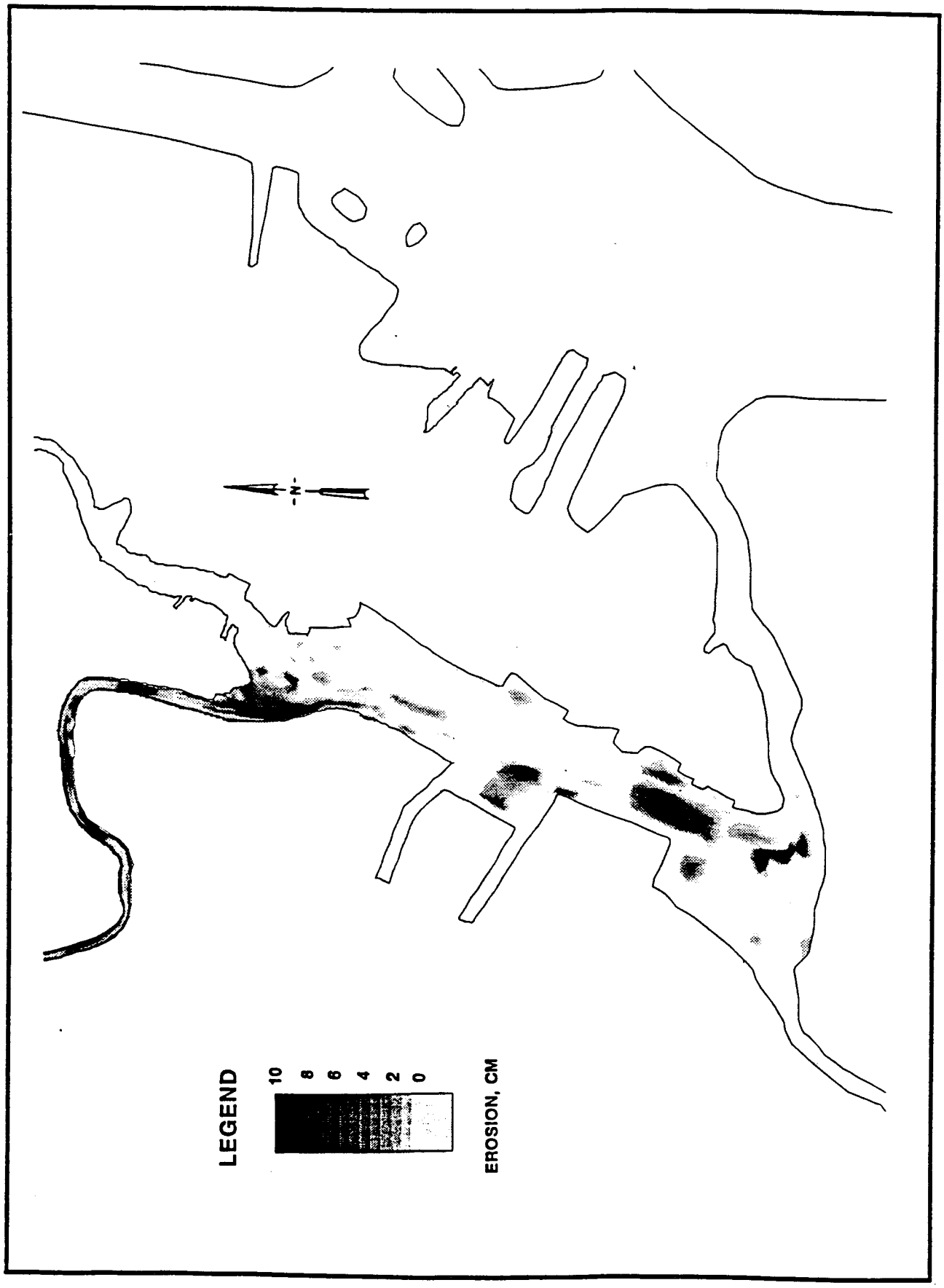
RMA2 Tunnel Outlet Structure Schematization

Figure C-164



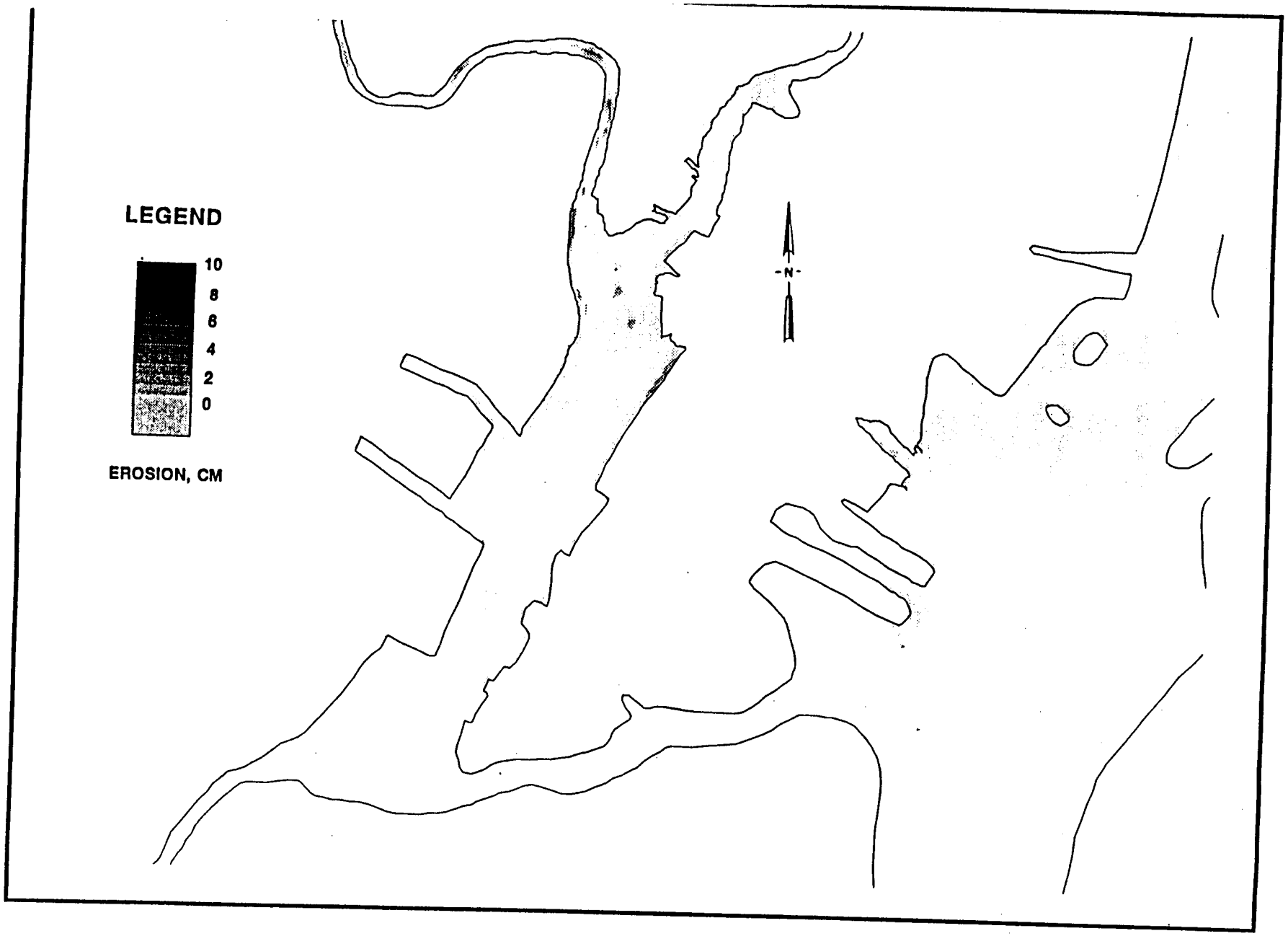
Base Erosion Pattern for 2-Year Flood

Figure C-165a



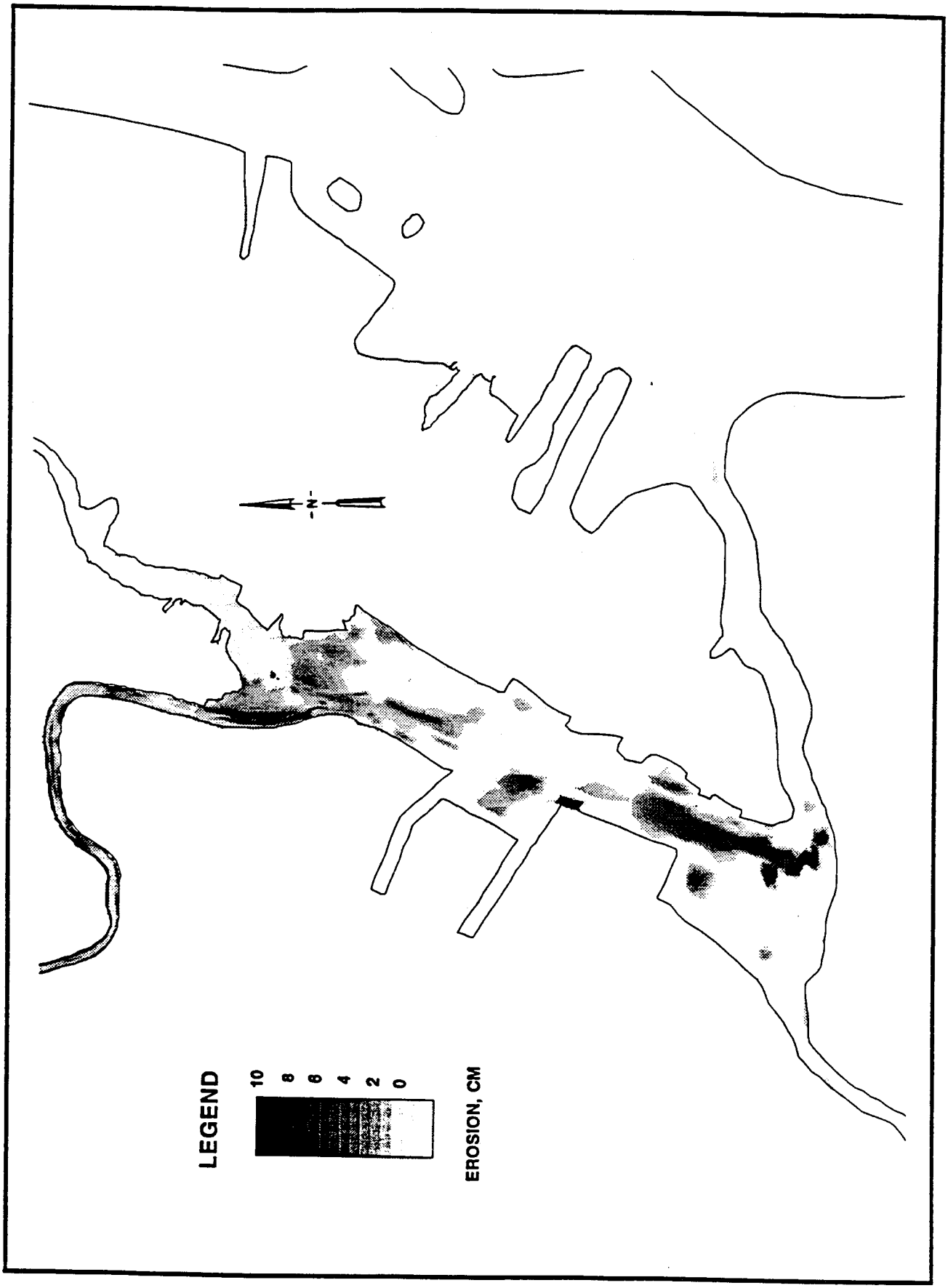
Base Erosion Pattern for 100-Year Flood

Figure C-165b



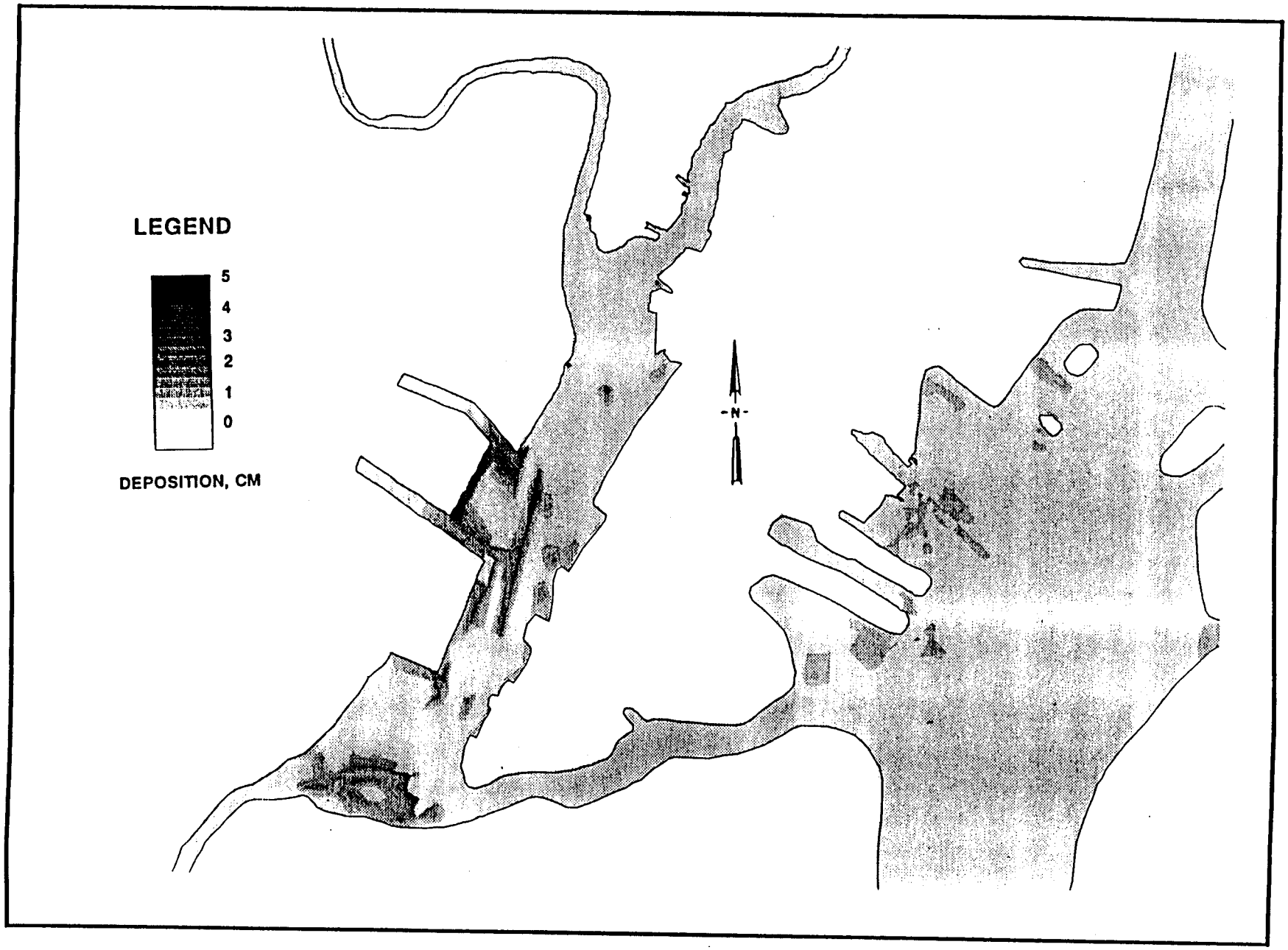
Plan Erosion Pattern for 2-Year Flood

Figure C-166a



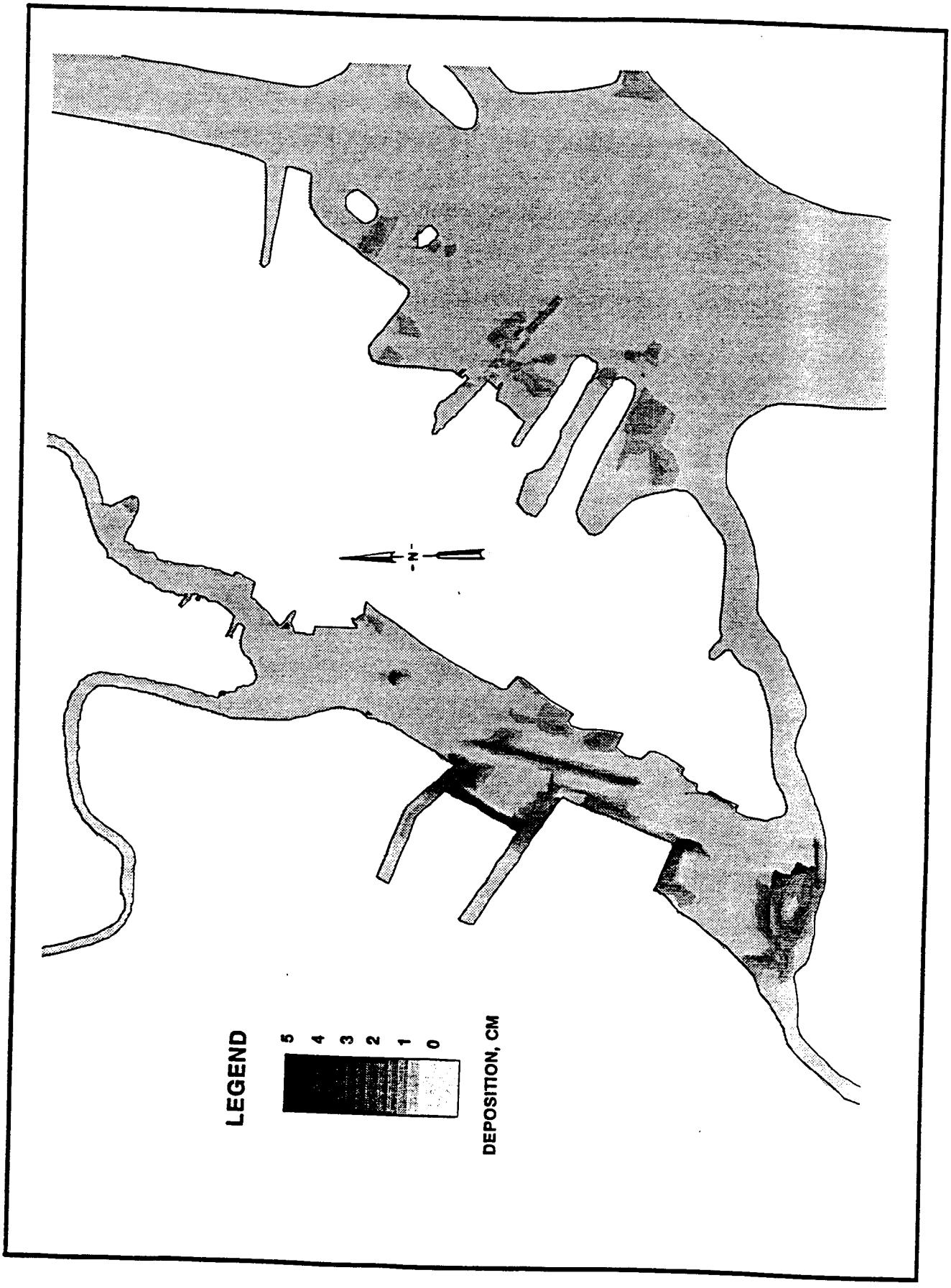
Plan Erosion Pattern for 100-Year Flood

Figure C-166b



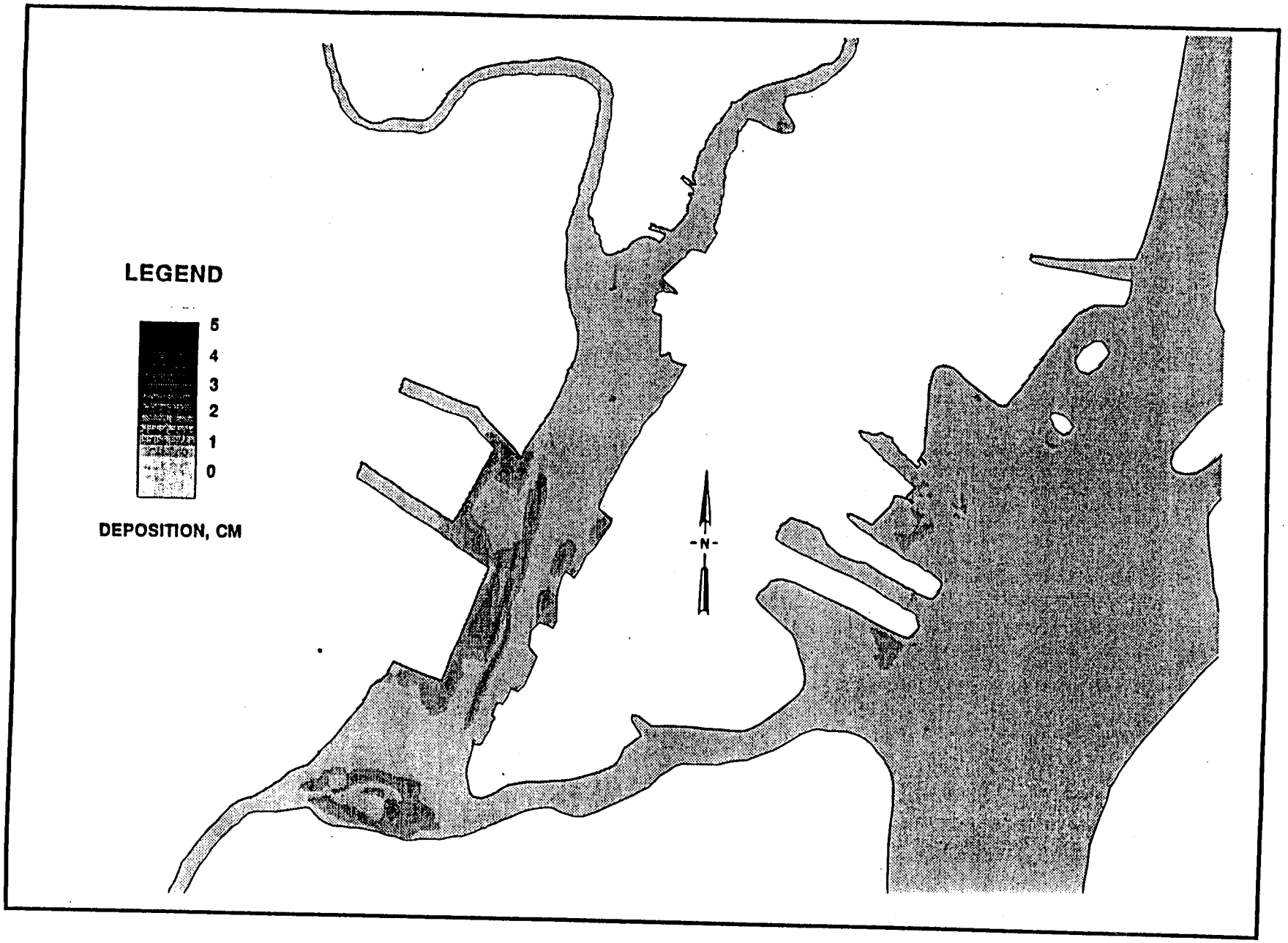
Base Deposition Pattern for 2-Year Flood

Figure C-167a



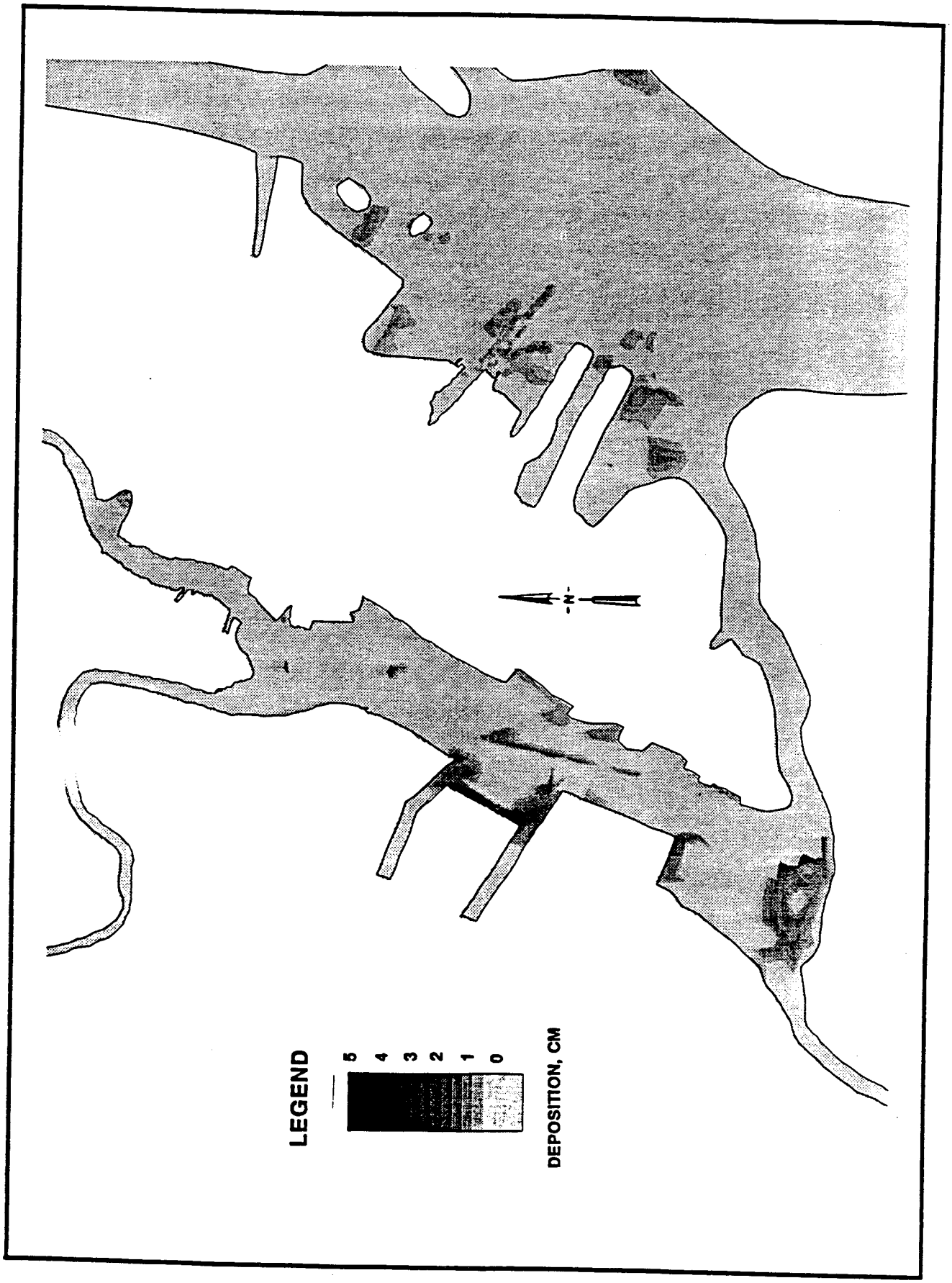
Base Deposition Pattern for 100-Year Flood

Figure C-167b



Plan Deposition Pattern for 2-Year Flood

Figure C-168a



Plan Deposition Pattern for 100-Year Flood

Figure C-168b

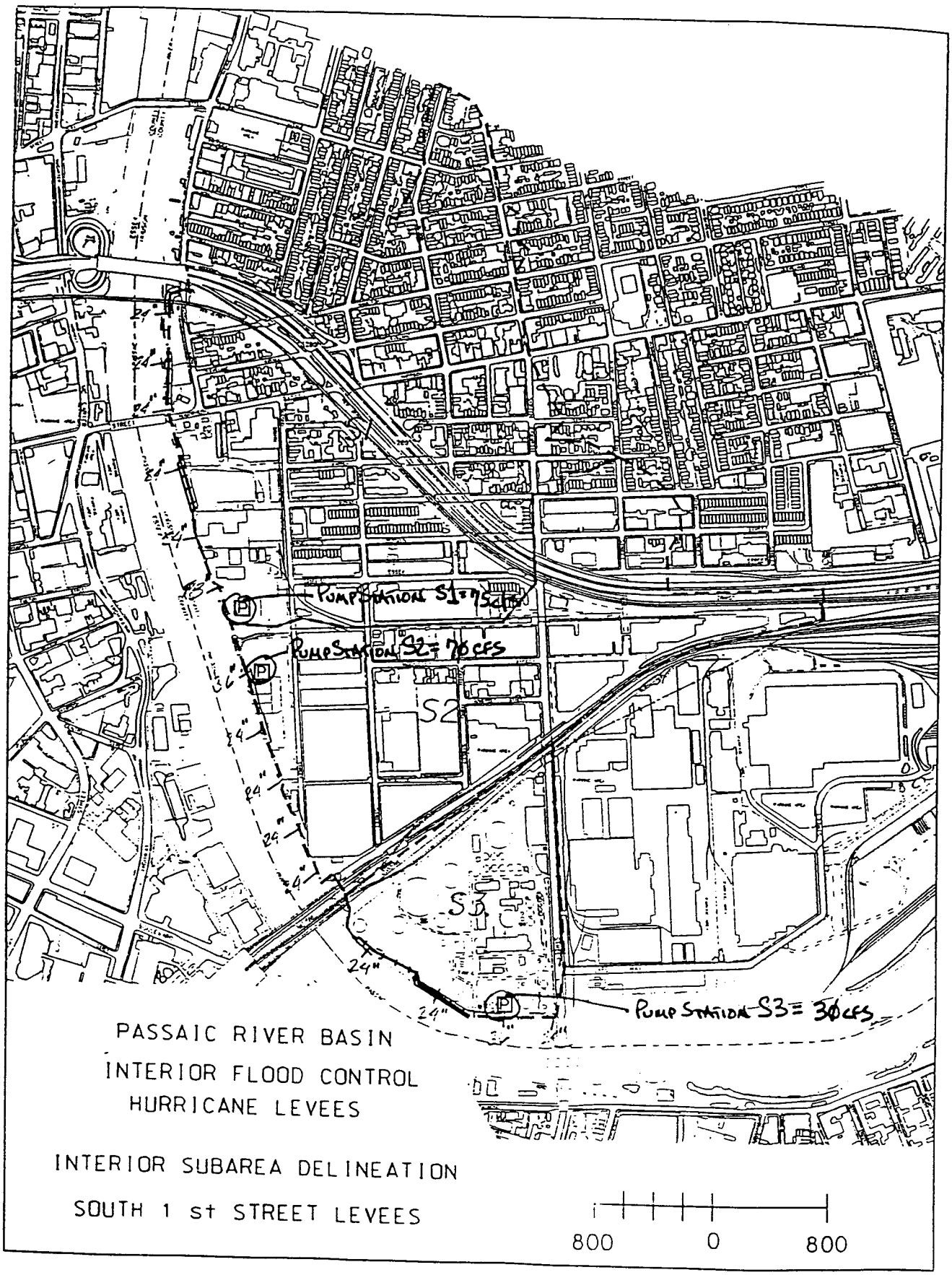


Figure C-169a

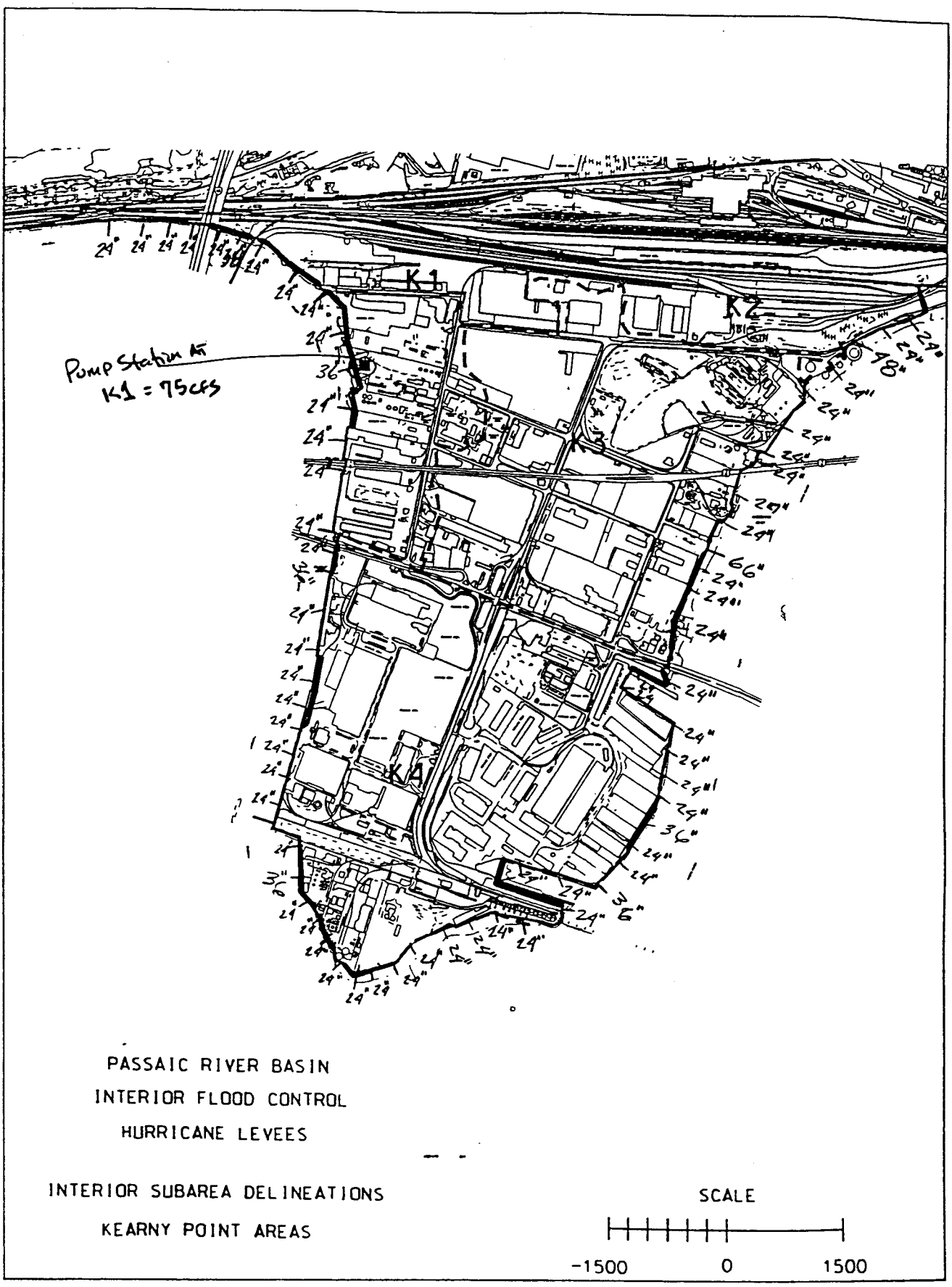
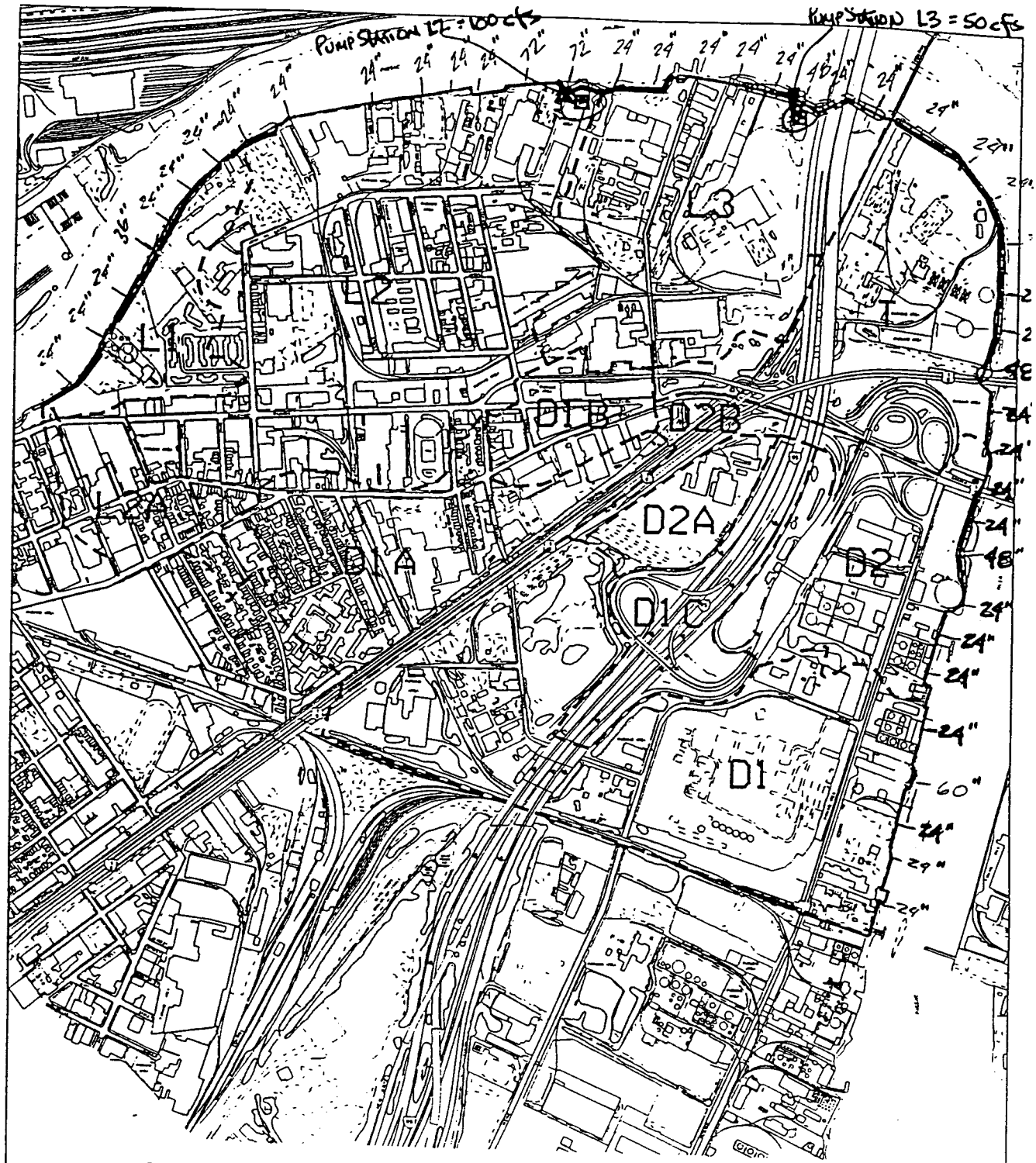


Figure C-169b



PASSAIC RIVER BASIN
INTERIOR FLOOD CONTROL
HURRICANE LEVEES
INTERIOR SUBAREA DELINEATIONS
LISTER AVE. TURNPIKE AND
DOREMUS AVE AREAS

SCALE

-1000 0 1000

Figure C-169c

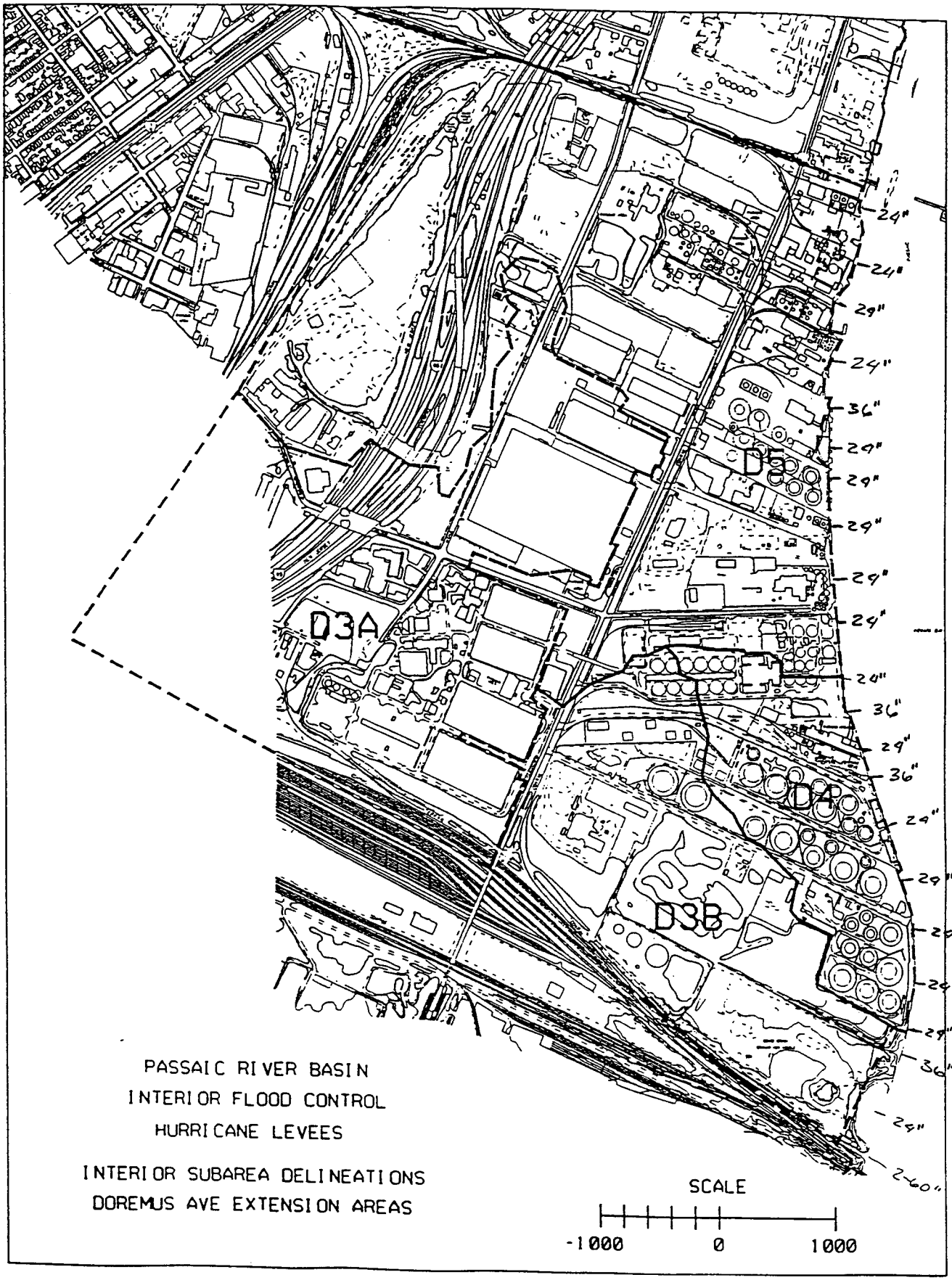


Figure C-169d

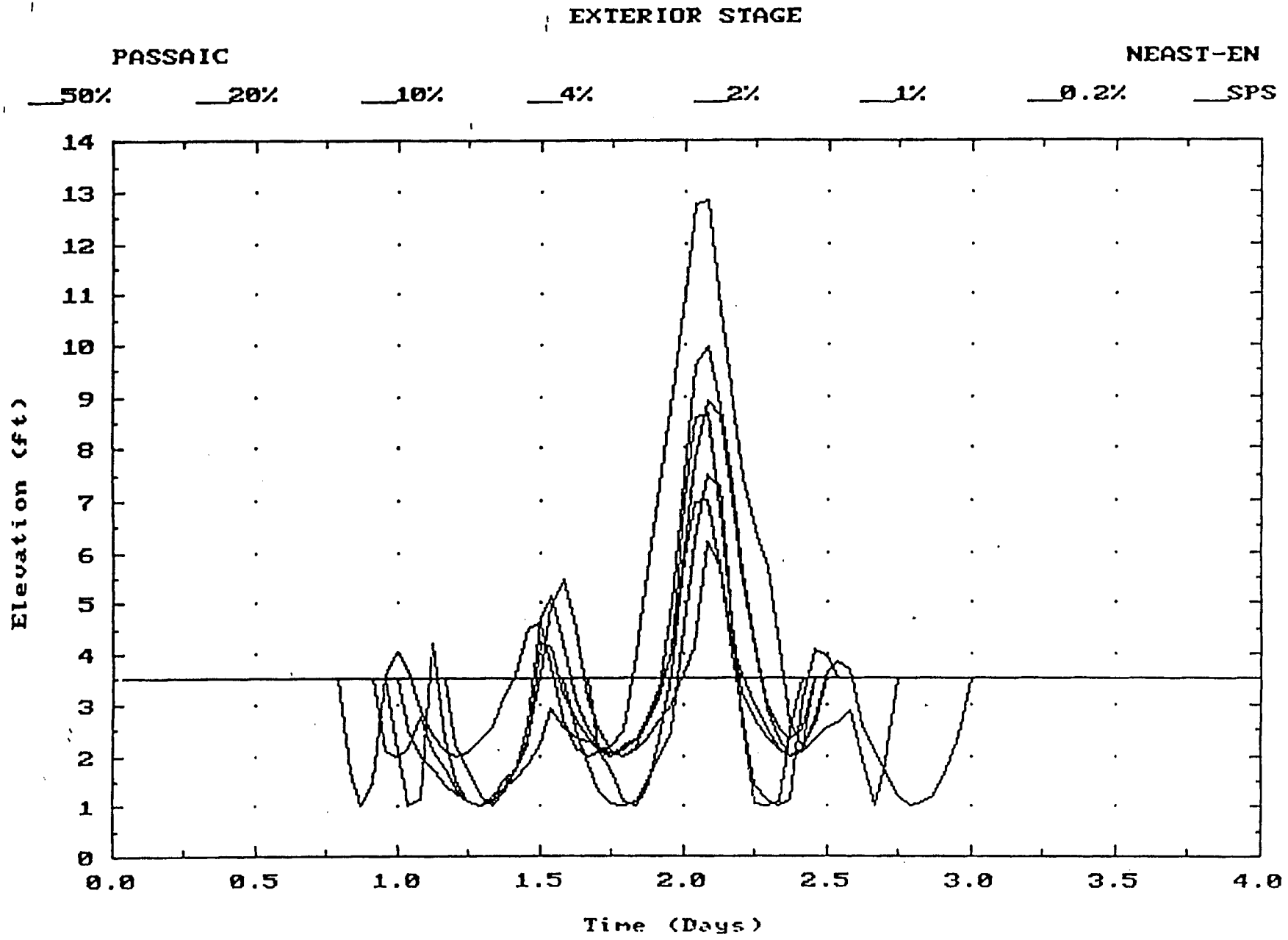

EXTERIOR STAGE
PASSAIC
NEAST-EN
50%
20%
10%
4%
2%
1%
0.2%
SPS
Elevation (ft)
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
Time (Days)

Figure C-170

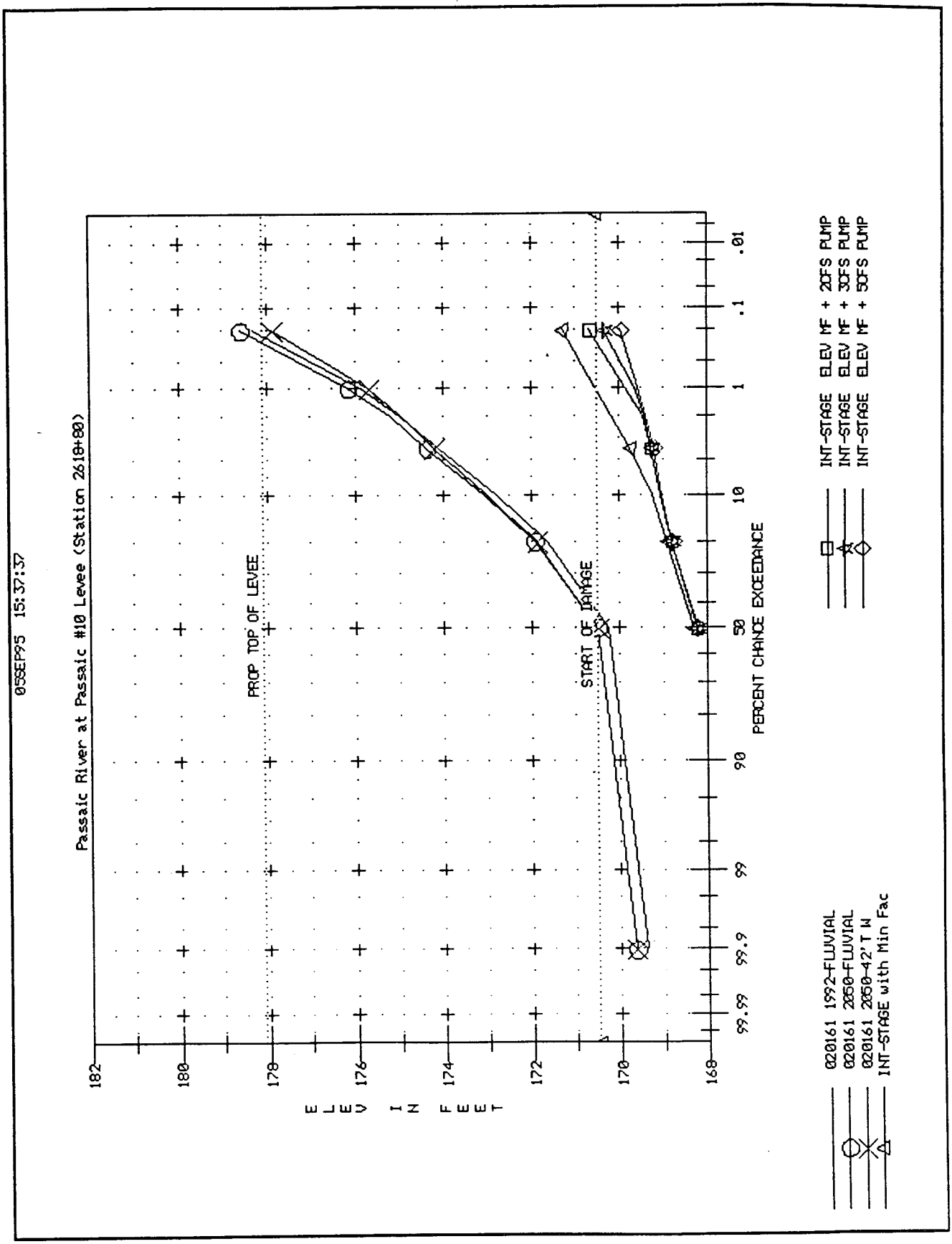
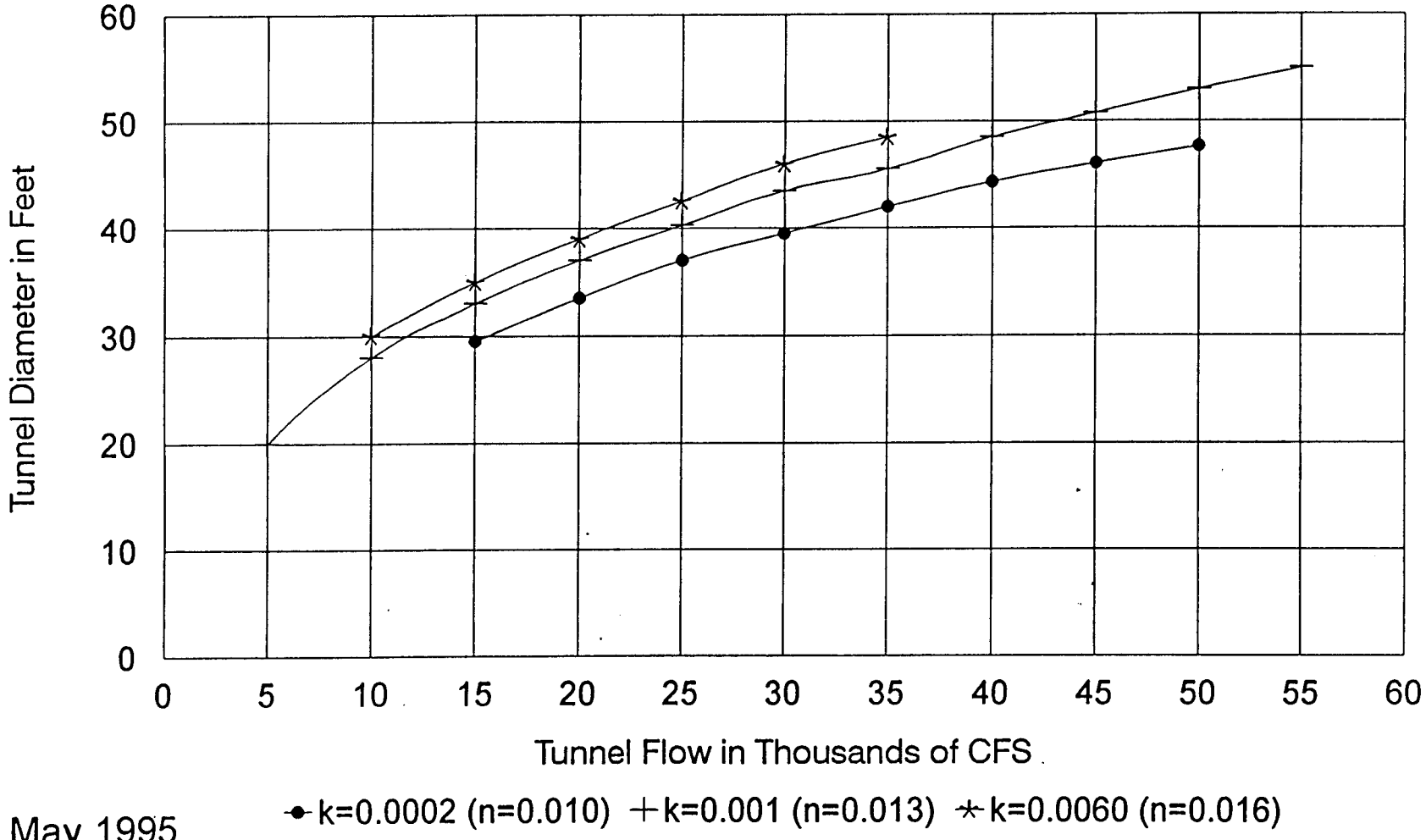

05SEP95 15:37:37
Passaic River at Passaic #10 Levee (Station 2618+80)
PROP TOP OF LEVEE
START OF DAMAGE
ELEV IN FEET
182
180
178
176
174
172
170
168
99.99
99.9
99
90
50
10
1
.1
.01
PERCENT CHANCE EXCEEDANCE
020161 1992-FLUVIAL
020161 2050-FLUVIAL
020161 2050-42'T W
INT-STAGE with Min Fac
INT-STAGE ELEV MF + 20FS PUMP
INT-STAGE ELEV MF + 30FS PUMP
INT-STAGE ELEV MF + 50FS PUMP

Figure C-171

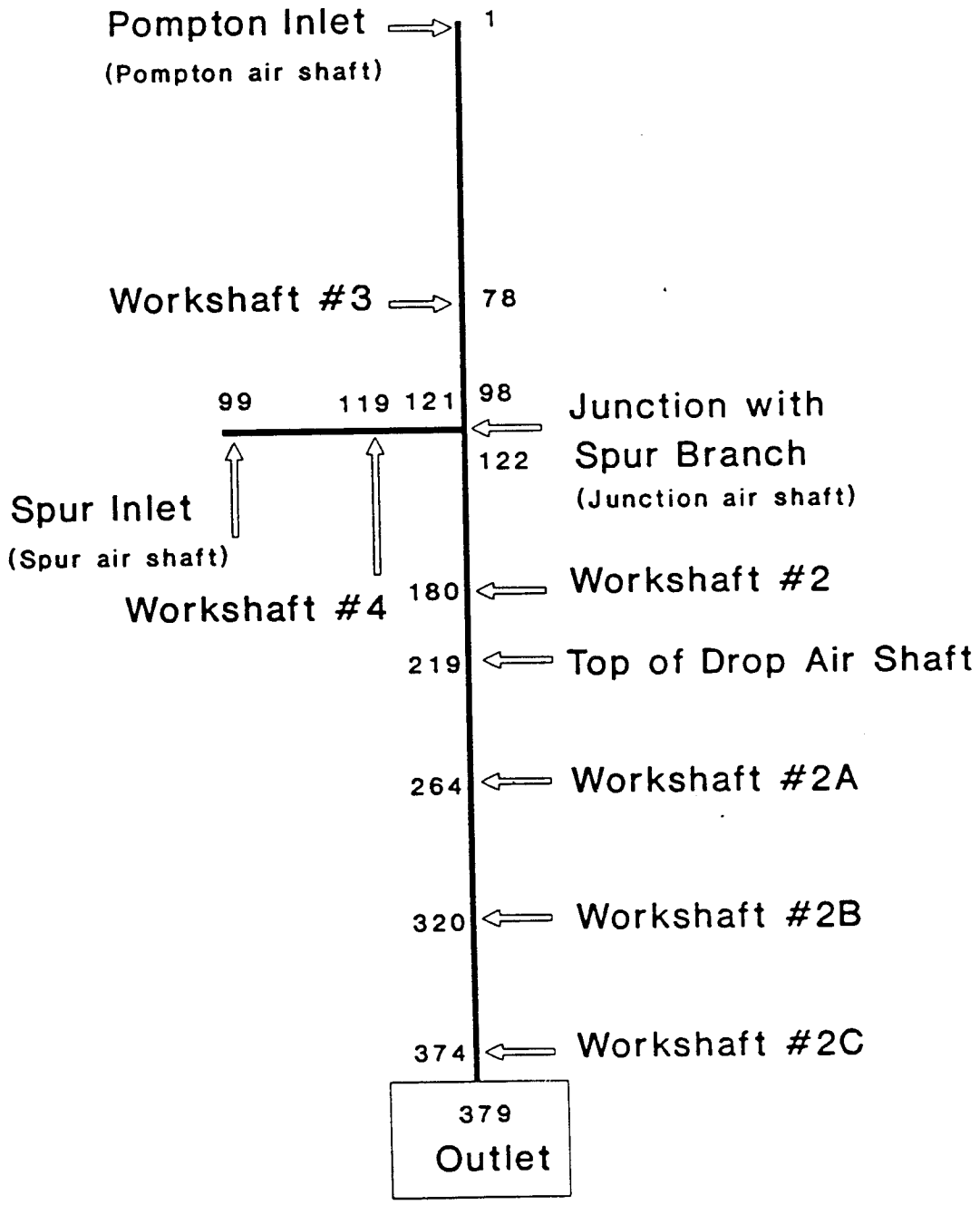
Tunnel Diameter versus Flow

(H = 164 feet)



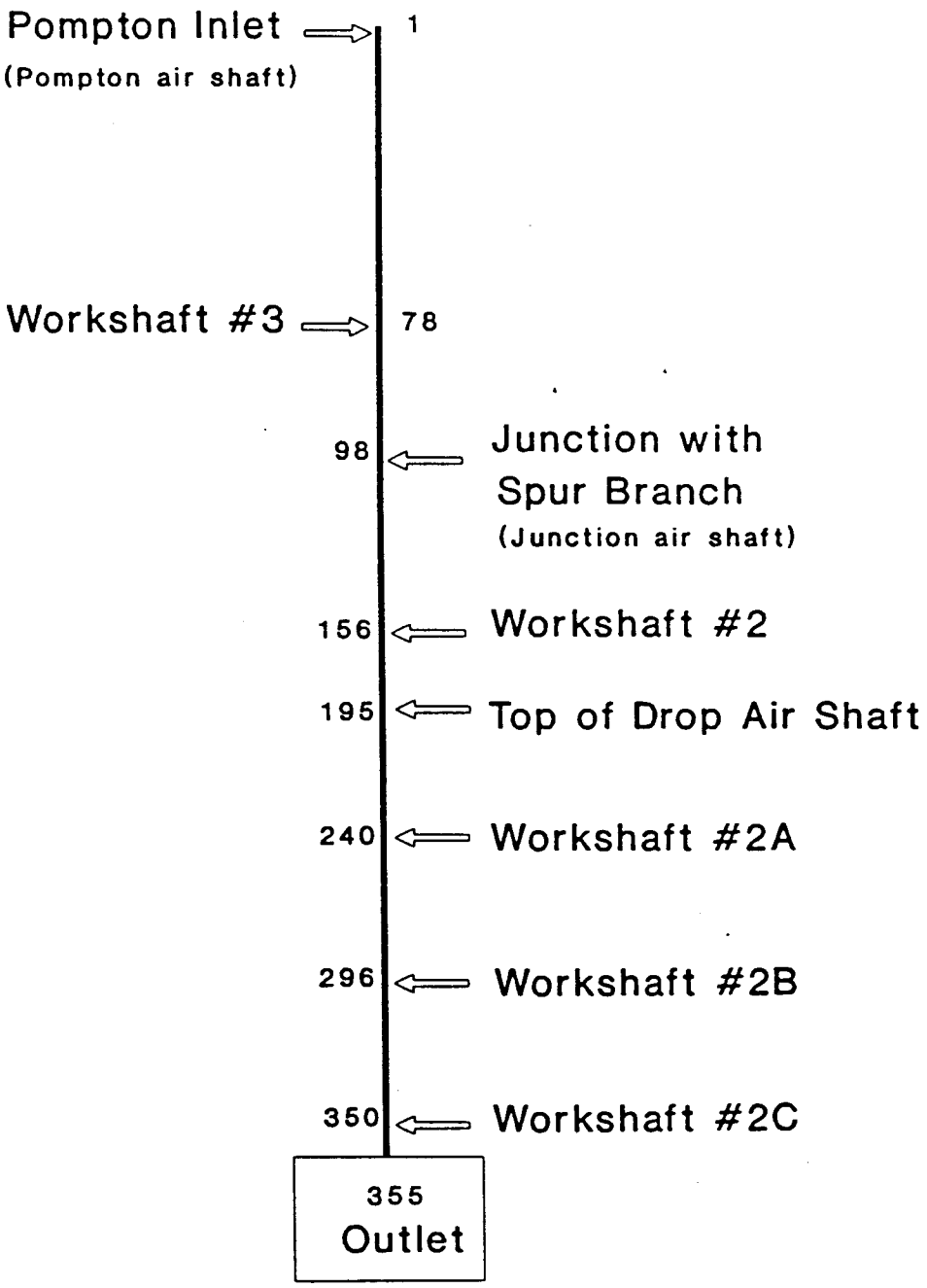
May 1995

Figure C-172



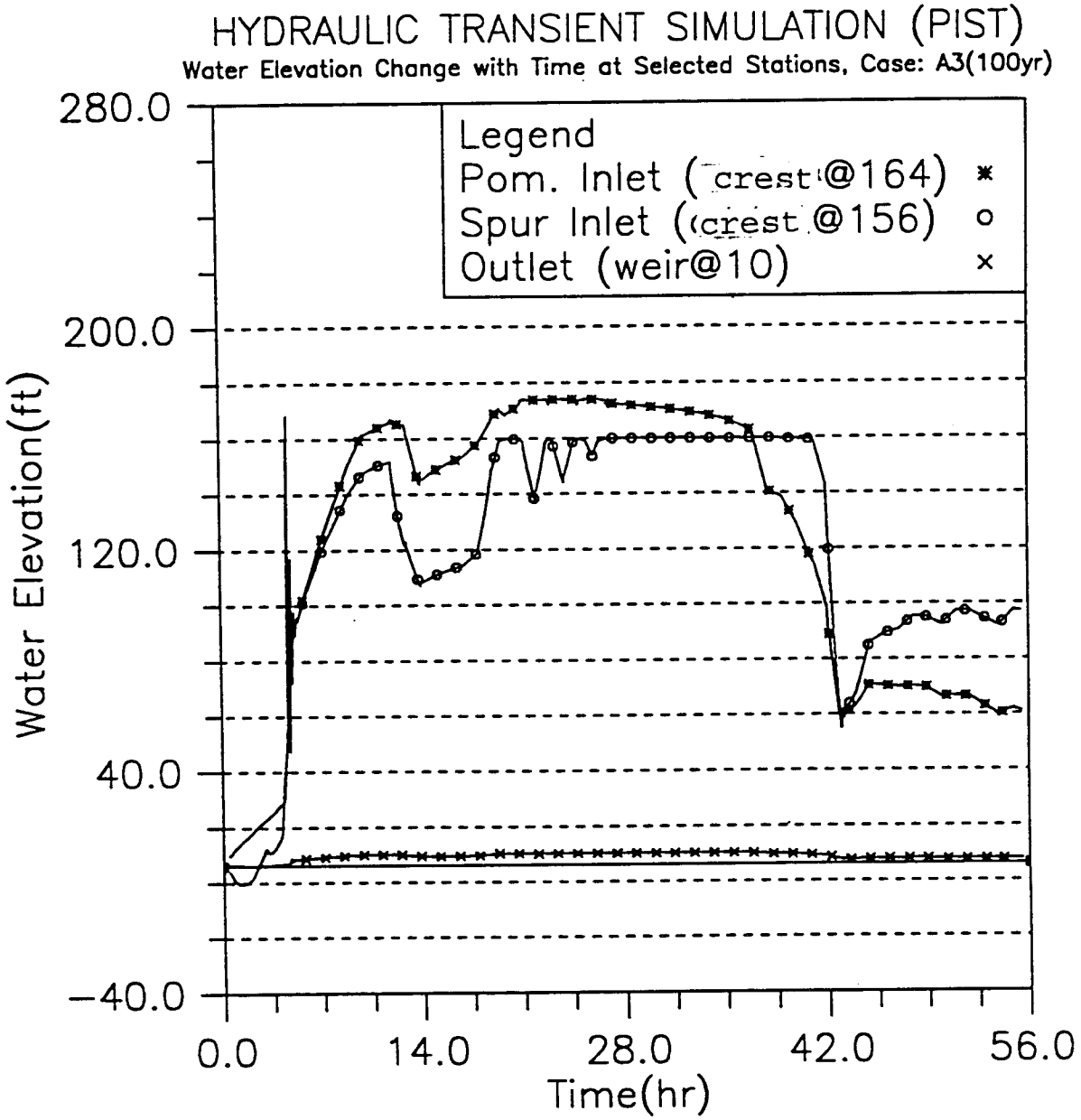
Schematic of the Passaic River Flood Protection Tunnel System and the computation stations

Figure C-173a



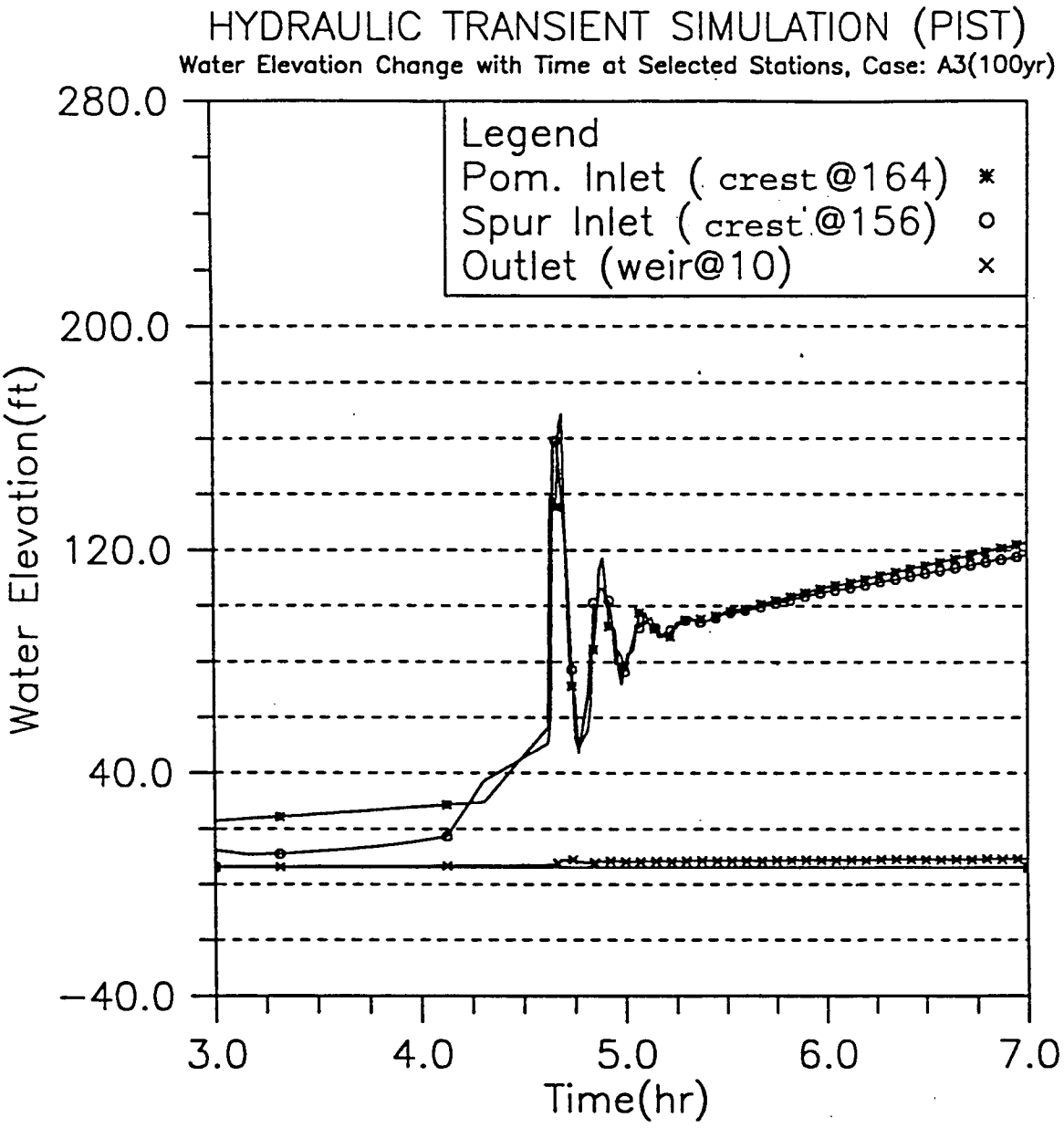
Schematic of the main tunnel, excluding the Spur Branch, and the station numbers for showing the hydraulic grade line along the main tunnel

Figure C-173b



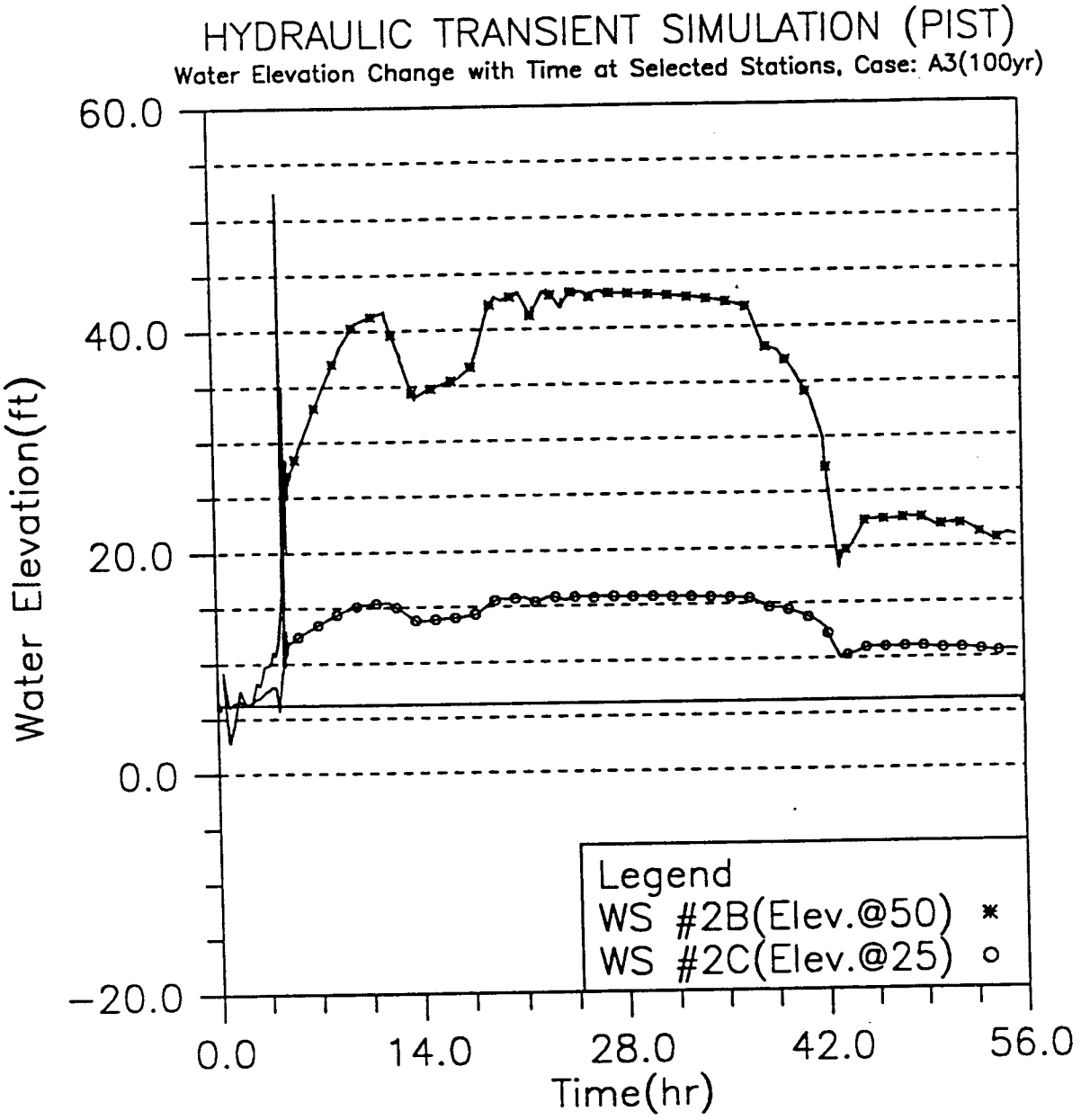
Time variation of water surface elevations at Pompton inlet, Spur inlet and downstream end during the simulated time period; modeling case: 100yr storm and wet tunnel (Case A3)

Figure C-174a



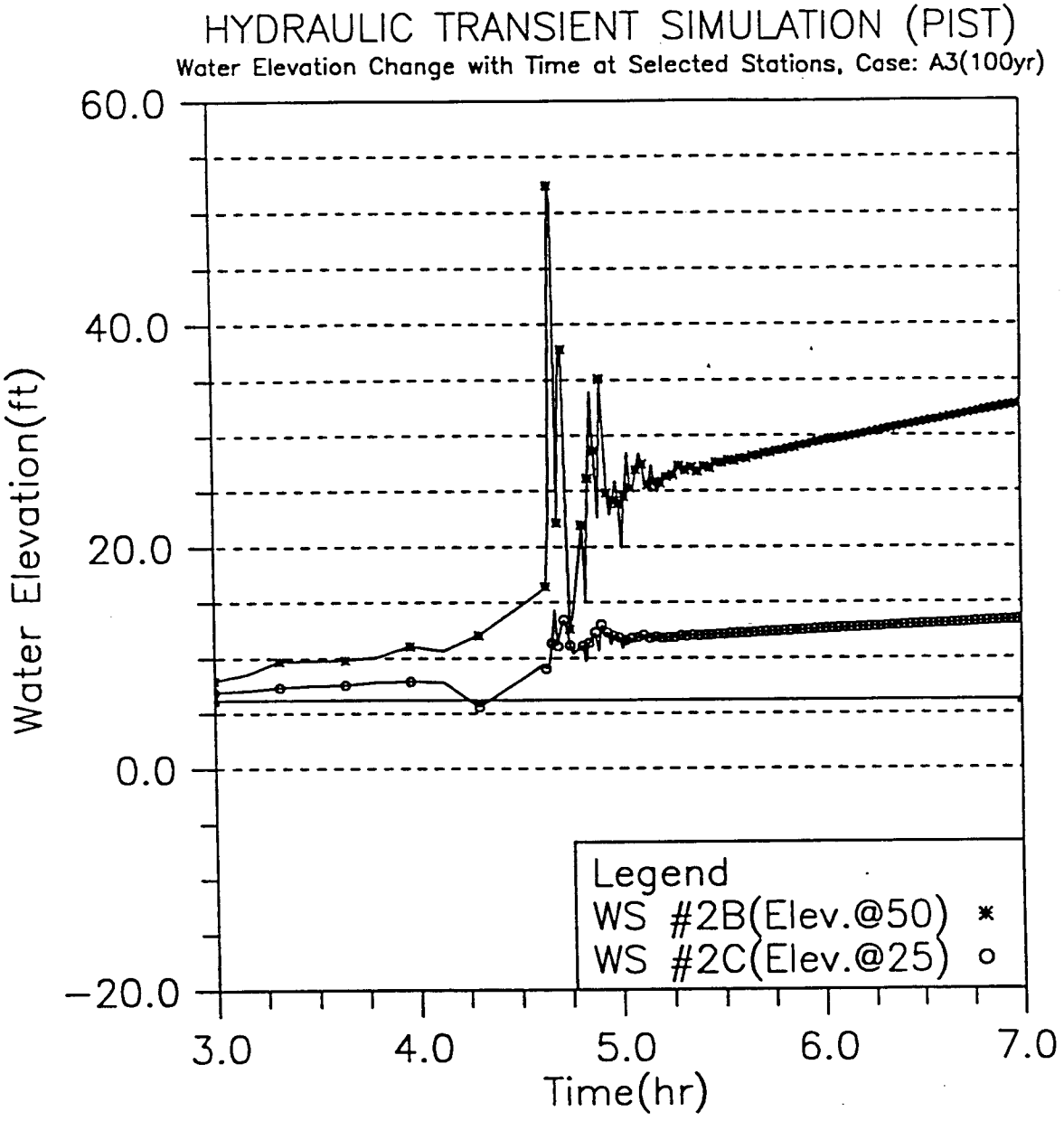
Detailed time variation of water surface elevations at Pompton inlet, Spur inlet and downstream end during the early surging period; modeling case: 100yr storm and wet tunnel (Case A3)

Figure C-174b



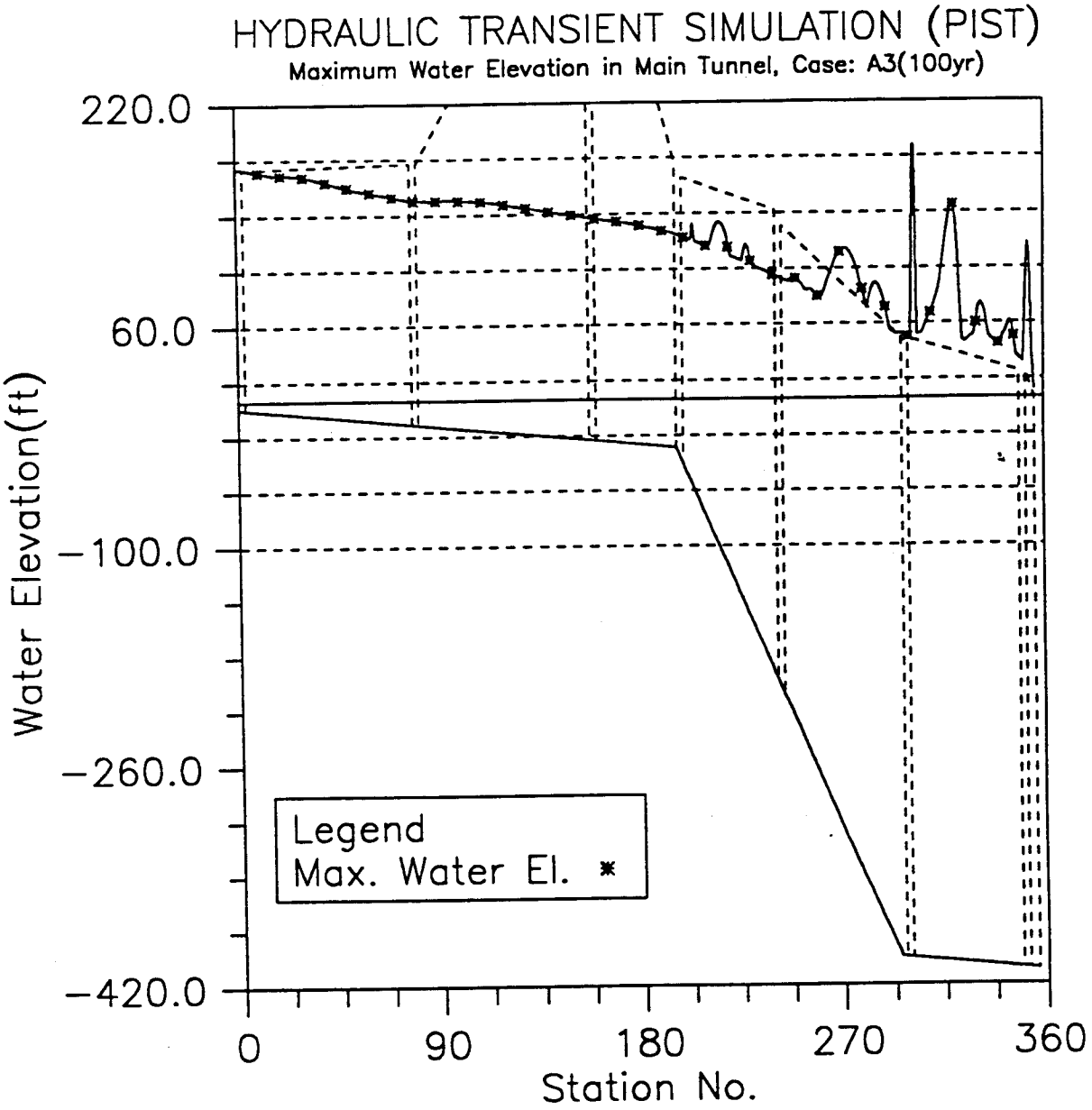
Time variation of water surface elevations at Workshafts #2B and #2C during the simulated time period; modeling case: 100yr storm and wet tunnel (Case A3)

Figure C-175a



Detailed time variation of water surface elevations at Workshafts #2B and #2C during the early surging period; modeling case: 100yr storm and wet tunnel (Case A3)

Figure C-175b



The maximum water surface elevations along the main tunnel; modeling case: 100yr storm and wet tunnel (Case A3)

Figure C-176

Attachments

Attachment 1

RUN TEST

8/18/92

RUN TEST

Passaic River at Chatham

Rank	Year	Flow
1	1973	3200
2	1968	2560
3	1971	2540
4	1975	2520
5	1942	2020
6	1984	1930
7	1938	1910
8	1949	1710
9	1979	1710
10	1969	1640
11	1986	1550
12	1978	1520
13	1972	1520
14	1962	1450
15	1943	1420
16	1970	1420
17	1948	1410
18	1974	1400
19	1977	1390
20	1955	1390
21	1983	1380
22	1980	1370
23	1989	1330
24	1967	1320
25	1952	1290
26	1961	1290
27	1991	1250

Rank	Year	Flow
28	1982	1240
29	1976	1240
30	1988	1220
31	1987	1210
32	1990	1190
33	1956	1150
34	1953	1150
35	1958	1140
36	1960	1110
37	1957	1100
38	1940	1090
39	1946	1040
40	1945	1040
41	1951	1010
42	1966	1010
43	1965	980
44	1963	980
45	1939	972
46	1944	964
47	1985	960
48	1981	899
49	1964	872
50	1947	869
51	1959	770
52	1941	756
53	1950	670
54	1954	651

Median = 1245 ✓

8/18/92

```
TT
TT   *****  NO  URBANIZATION ADJUSTMENT 7/14/92 *****
TT
TT ADDED WATER YEARS 1990 & 1991          18 FEBRUARY 1992
TT FLOOD FLOW FREQUENCY TEST
TT PASSAIC RIVER BASIN, NEW JERSEY
TT PASSAIC RIVER AT CHATHAM  --  GAGE #379500
TT DA=100 SQ MI
TT WY1989 VALUE CORRECTED FROM 734 TO 1330
TT 1970, 1973, 1978, & 1984 VALUES TAKEN FROM NEW YORK FILE
TT COMPUTED SKEW OF .56 TOO HIGH, USE GEN. SKEW OF 0.4
TT NOTE: ACTUAL GENERALIZED SKEW FROM CHART IS 0.3.
J1      1        0        0        0        0        0        1
ID 379500  PASSAIC RIVER AT CHATHAM
GS  7950   0.207      .4      # OF RUNS
QR  7950   1904   3300
QR  7950   1905   2000+   1
QR  7950   1906   1190-   2
QR  7950   1907   2860+
QR  7950   1908   1600+
QR  7950   1909   1360+
QR  7950   1910   2260+   3
QR  7950   1911    525-
QR  7950   1938   1910+   1
QR  7950   1939    972-
QR  7950   1940   1090-
QR  7950   1941    756-   2
QR  7950   1942   2020+
QR  7950   1943   1420+   3
QR  7950   1944    964-
QR  7950   1945   1040-
QR  7950   1946   1040-
QR  7950   1947    869-   4
QR  7950   1948   1410+
QR  7950   1949   1710+   5
QR  7950   1950    670-   6
QR  7950   1951   1010-
QR  7950   1952   1290+   7
QR  7950   1953   1150-
QR  7950   1954    651-   8
QR  7950   1955   1390+   9
QR  7950   1956   1150-
QR  7950   1957   1100-
QR  7950   1958   1140-
QR  7950   1959    770-
QR  7950   1960   1110-   10
QR  7950   1961   1290+
QR  7950   1962   1450+   11
QR  7950   1963    980-
QR  7950   1964    872-
QR  7950   1965    980-
QR  7950   1966   1010-   12
QR  7950   1967   1320+
QR  7950   1968   2560+
QR  7950   1969   1640+
QR  7950   1970   1420+
QR  7950   1971   2540+   13
QR  7950   1972   1520+
QR  7950   1973   3200+
QR  7950   1974   1400+
```

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QR	7950	1975	2520 + 13	
QR	7950	1976	1240 − 0	14
QR	7950	1977	1390 +	
QR	7950	1978	1520 +	
QR	7950	1979	1710 +	
QR	7950	1980	1370 + 5	15
QR	7950	1981	899 −	
QR	7950	1982	1240 − 16	16
QR	7950	1983	1380 +	
QR	7950	1984	1930 + 17	17
QR	7950	1985	960 − 18	18
QR	7950	1986	1550 + 19	19
QR	7950	1987	1210 −	20
QR	7950	1988	1220 − 20	
QR	7950	1989	1330 + 21	21
QR	7950	1990	1190 − 22	22
QR	7950	1991	1250 + 23	23
ED				

$n_1 = 27$ ✓

$n_2 = 27$ ✓

OF RUNS = 23 ✓ = U

$$\bar{U} = \frac{2n_1n_2}{n_1+n_2} + 1 = \frac{2(27)(27)}{27+27} + 1 = \boxed{28 = \bar{U}}$$ ✓

$$S_U^2 = \frac{2n_1n_2(2n_1n_2 - n_1 - n_2)}{(n_1+n_2)^2(n_1+n_2-1)} = \frac{2(27)(27)(2(27)(27)-27-2}{(27+27)^2(27+27-1}$$

$$\boxed{S_U^2 = 13.25}$$ ✓

$$S_U = \sqrt{13.25} = \boxed{3.639 = S_U}$$ ✓

$$Z = \frac{U-\bar{U}}{S_U} = \frac{23-28}{3.639} = \boxed{-1.374 = Z}$$ ✓

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RUN TEST
58

Whippany River at Morristown

Rank	Year	Flow
1	1971	2280
2	1979	2080
3	1928	2000
4	1933	1820
5	1984	1680
6	1924	1530
7	1940	1500
8	1936	1500
9	1977	1480
10	1974	1450
11	1989	1380
12	1952	1360
13	1973	1360
14	1990	1350
15	1949	1350
16	1970	1310
17	1929	1290
18	1983	1250
19	1942	1220
20	1938	1170
21	1962	1160
22	1955	1160
23	1987	1150
24	1978	1150
25	1981	1130
26	1991	1130
27	1985	1120
28	1980	1100
29	1988	1080
30	1986	1060
31	1975	1020
32	1922	1020
33	1946	1020
34	1925	975
35	1968	956

Rank	Year	Flow
36	1972	946
37	1951	938
38	1958	925
39	1956	900
40	1982	871
41	1934	847
42	1948	829
43	1923	815
44	1953	804
45	1926	783
46	1976	752
47	1965	716
48	1960	684
49	1941	679
50	1966	640
51	1944	640
52	1969	624
53	1957	613
54	1932	604
55	1947	591
56	1930	570
57	1931	552
58	1943	540
59	1939	529
60	1959	509
61	1967	500
62	1937	491
63	1961	475
64	1935	467
65	1945	460
66	1927	450
67	1964	401
68	1950	375
69	1963	367
70	1954	282

Median = 951 ✓

$M_1 = M_2 = 35$

8/18/92

```
TT
TT      ***** NO URBANIZATION ADDJUSTMENT     7/15/92      *****
TT
TT ADDED WATER YEARS 1990 & 1991          18 FEBRUARY 1992
TT FLOOD FLOW FREQUENCY TEST
TT PASSAIC RIVER BASIN - WHIPPANY RIVER AT MORRISTOWN
TT DA=29.4 SQ MI
TT 1968, 1971, & 1978 VALUES ENTERED FROM NEW YORK FILE
TT computed skew of .68 too high, use generalized skew of 0.4.
TT NOTE: original generalized skew was 0.3
TT 1984 VALUE OF 1200 NOT TAKEN FROM NY FILE (USGS VALUE OF 1680 USED)
TT WATER YEAR 1989 FLOW CORRECTED FROM  237 cfs to 1380 cfs
J1      1        0        0        0        0        0        1
ID 381500  WHIPPANY RIVER AT MORRISTOWN
GS  8150   0.207      .4
QH  8150    1904    2480
QR  8150    1922    1020   +   1
QR  8150    1923     815   -   2
QR  8150    1924    1530   +   3
QR  8150    1925     975   +
QR  8150    1926     783   -   4
QR  8150    1927     450   -
QR  8150    1928    2000   +   5
QR  8150    1929    1290   +
QR  8150    1930     570   -
QR  8150    1931     552   -   6
QR  8150    1932     604   -
QR  8150    1933    1820   +   7
QR  8150    1934     847   -
QR  8150    1935     467   -   8
QR  8150    1936    1500   +   9
QR  8150    1937     491   -   10
QR  8150    1938    1170   +   11
QR  8150    1939     529   -   12
QR  8150    1940    1500   +   13
QR  8150    1941     679   -   14
QR  8150    1942    1220   +   15
QR  8150    1943     540   -
QR  8150    1944     640   -   16
QR  8150    1945     460   -
QR  8150    1946    1020   +   17
QR  8150    1947     591   -
QR  8150    1948     829   -   18
    8150    1949    1350   +   19
    8150    1950     375   -   20
    8150    1951     938   -
    8150    1952    1360   +   21
    8150    1953     804   -   22
    8150    1954     282   -
    8150    1955    1160   +   23
    8150    1956     900   -
QR  8150    1957     613   -
QR  8150    1958     925   -   24
QR  8150    1959     509   -
QR  8150    1960     684   -
QR  8150    1961     475   -
QR  8150    1962    1160   +   25
QR  8150    1963     367   -
QR  8150    1964     401   -   26
QR  8150    1965     716   -
```

QR	8150	1966	640	−	
QR	8150	1967	500	−	
QR	8150	1968	956	+	27
QR	8150	1969	624	−	28
QR	8150	1970	1310	+	29
QR	8150	1971	2280	+	
QR	8150	1972	946	−	30
QR	8150	1973	1360	+	
QR	8150	1974	1450	+	31
QR	8150	1975	1020	+	
QR	8150	1976	752	−	32
QR	8150	1977	1480	+	
QR	8150	1978	1150	+	
QR	8150	1979	2080	+	33
QR	8150	1980	1100	+	
QR	8150	1981	1130	+	
QR	8150	1982	871	−	34
QR	8150	1983	1250	+	
QR	8150	1984	1680	+	
QR	8150	1985	1120	+	
QR	8150	1986	1060	+	
QR	8150	1987	1150	+	
QR	8150	1988	1080	+	35
QR	8150	1989	1380	+	
QR	8150	1990	1350	+	
QR	8150	1991	1130	+	
ED					

8/18/92

$n_1 = n_2 = 35$ ✓

$\therefore$ # of runs = 35

$U = 35$ ✓

$$36 = \bar{U} = \frac{2(n_1)(n_2)}{(n_1+n_2)} + 1 = \frac{2(35)(35)}{(35+35)} + 1 = 36$$

$$17.246 = S_U^2 = \frac{2(n_1)(n_2)(2n_1n_2 - n_1 - n_2)}{(n_1+n_2)^2(n_1+n_2-1)} = \frac{2(35)(35)(2(35)(35)-35-35)}{(35+35)^2(35+35-1)}$$

$$= \frac{(2450)(2380)}{(4900)(69)} = 17.246$$

$$S_U = 4.1528$$

$$-0.2408 = Z = \frac{U - \bar{U}}{S_U} = \frac{35-36}{4.1528} = -0.2408$$

8/18/92
SB3

RUN TEST

Rockaway River Above Boonton Reservoir

Rank	Year	Flow
1	1984	5600
2	1979	5430
3	1952	4360
4	1974	4300
5	1973	4100
6	1977	3970
7	1949	3590
8	1971	3550
9	1968	3400
10	1990	3350
11	1983	3030
12	1970	2870
13	1980	2670
14	1978	2590
15	1976	2550
16	1987	2530
17	1951	2460
18	1940	2390
19	1989	2350
20	1938	2260
21	1975	2230
22	1972	2200
23	1955	2190
24	1958	2080
25	1953	2050
26	1981	2040
27	1956	1940

Median = 1900 ✓

Rank	Year	Flow
28	1962	1860
29	1991	1830
30	1986	1830
31	1948	1790
32	1982	1780
33	1961	1730
34	1960	1690
35	1944	1620
36	1969	1550
37	1945	1540
38	1965	1490
39	1947	1470
40	1985	1460
41	1943	1450
42	1946	1440
43	1988	1430
44	1957	1340
45	1939	1310
46	1942	1200
47	1967	1200
48	1959	1170
49	1966	1070
50	1964	1000
51	1950	991
52	1941	908
53	1963	862
54	1954	705

$M_1 = M_2 = 27$

```
TT
TT      *****    NO URBANIZATION ADJUSTMENTS    7/15/92    *****
TT
TT ADDED WATER YEARS 1990 & 1991        18 FEBRUARY 1992
TT FLOOD FLOW FREQUENCY TEST
TT PASSAIC RIVER BASIN - ROCKAWAY RIVER ABOVE BOONTOWN RESERVOIR
TT DA=116 SQ MI
TT WATER YEAR 1989 CORRECTED FROM  979 cfs to 2350 cfs
TT 1943 & 1984 VALUES TAKEN FROM NEW YORK FILE
J1     1       0       0       0       0       0       1
ID 380500  ROCKAWAY RIVER ABOVE BOONTON RESERVOIR
GS  8050   0.207      .3
QH  8050    1904    8050
QR  8050    1938    2260  +   1
QR  8050    1939    1310  -   2
QR  8050    1940    2390  +   3
QR  8050    1941     908  -
QR  8050    1942    1200  -
QR  8050    1943    1450  -
QR  8050    1944    1620  -   4
QR  8050    1945    1540  -
QR  8050    1946    1440  -
QR  8050    1947    1470  -
QR  8050    1948    1790  -
QR  8050    1949    3590  +   5
QR  8050    1950     991  -   6
QR  8050    1951    2460  +
QR  8050    1952    4360  +   7
QR  8050    1953    2050  +
QR  8050    1954     705  -   8
QR  8050    1955    2190  +
QR  8050    1956    1940  +   9
QR  8050    1957    1340  -   10
QR  8050    1958    2080  +   11
QR  8050    1959    1170  -
QR  8050    1960    1690  -
QR  8050    1961    1730  -
QR  8050    1962    1860  -   12
QR  8050    1963     862  -
QR  8050    1964    1000  -
QR  8050    1965    1490  -
QR  8050    1966    1070  -
QR  8050    1967    1200  -
QR  8050    1968    3400  +   13
QR  8050    1969    1550  -   14
QR  8050    1970    2870  +
QR  8050    1971    3550  +
QR  8050    1972    2200  +
QR  8050    1973    4100  +
QR  8050    1974    4300  +   15
QR  8050    1975    2230  +
QR  8050    1976    2550  +
QR  8050    1977    3970  +
QR  8050    1978    2590  +
QR  8050    1979    5430  +
QR  8050    1980    2670  +
QR  8050    1981    2040  +
QR  8050    1982    1780  -   16
QR  8050    1983    3030  +
QR  8050    1984    5600  +   17
```

Run TEST

8/18/92

SPR

Median = 1900 cfs

8/18/92

QR	8050	1985	1460	−	18
QR	8050	1986	1830	−	
QR	8050	1987	2530	+	19
QR	8050	1988	1430	−	20
QR	8050	1989	2350	+	21
QR	8050	1990	3350	+	
QR	8050	1991	1830	−	22
ED					

$n_1 = n_2 = 27$ ✓

# of runs = 22 ∴ $U = 22$ ✓

$$\boxed{28 = \bar{U}} = \frac{2(n_1)(n_2)}{(n_1+n_2)} + 1 = \frac{2(27)(27)}{(27+27)} + 1$$

$$\boxed{13.25 = S_u^2} = \frac{2(n_1)(n_2)(2n_1n_2 - n_1 - n_2)}{(n_1+n_2)^2(n_1+n_2-1)} = \frac{2(27)(27)(2(27)(27)-27-27)}{(27+27)^2(27+27-1)}$$

$$= \frac{(1458)(1404)}{(2916)(53)} = 13.2458$$

$S_u = 3.6394$ ✓

$$\boxed{-1.6488 = Z} = \frac{U - \bar{U}}{S_u} = \frac{22-28}{3.6394} = -1.6488$$

Run Test 8/18/92

Ramapo River at Pompton Lakes

Rank	Year	Flow
1	1984	15400
2	1936	12300
3	1956	12000
4	1978	10300
5	1968	9530
6	1955	8570
7	1945	8560
8	1951	8520
9	1927	7220
10	1974	7040
11	1924	6800
12	1980	6700
13	1938	6620
14	1987	6490
15	1971	6420
16	1922	5900
17	1933	5680
18	1973	5580
19	1949	5520
20	1989	5510
21	1977	5290
22	1983	5160
23	1979	5130
24	1952	4780
25	1958	4750
26	1970	4540
27	1990	4070
28	1961	3950
29	1928	3930
30	1981	3630
31	1960	3550
32	1937	3510
33	1975	3380
34	1939	3260
35	1925	3200

Rank	Year	Flow
36	1953	3190
37	1972	3190
38	1969	3160
39	1976	3090
40	1940	2940
41	1986	2760
42	1943	2740
43	1934	2650
44	1982	2590
45	1991	2560
46	1947	2540
47	1957	2540
48	1946	2520
49	1962	2520
50	1948	2360
51	1985	2300
52	1923	2270
53	1944	2200
54	1965	2160
55	1929	2040
56	1988	2030
57	1926	2020
58	1942	1960
59	1932	1870
60	1941	1860
61	1935	1840
62	1950	1780
63	1930	1720
64	1959	1640
65	1966	1460
66	1931	1420
67	1967	1380
68	1954	1360
69	1964	1360
70	1963	1310

Median = 3195 ✓

$M_1 = M_2 = 35$

8/18/92

```
TT
TT *********   NO  URBANIZATION   ADJUSTMENT     7/14/92     ***********
TT
TT ADDED WATER YEARS 1990 & 1991  ** 1904 value edited to 16466 2/27/92 ***
TT FLOOD FLOW FREQUENCY TEST
TT PASSAIC RIVER BASIN - RAMAPO RIVER AT POMPTON LAKES
TT DA = 160 SQ MI
TT 1989 REVISED FROM 1195 TO 5510 ON 7/14/92
TT 1904 USED AS HISTORIC EVENT
TT 1968 VALUE USED FROM GAGE
TT 1984 VALUE NOT TAKEN FROM NEW YORK FILE (USED USGS VALUE OF 15400)
J1      1       0       0       0       0       0       1
ID 388000  RAMAPO    RIVER AT POMPTON LAKES
GS  8950    0.207      .4
QH  8800    1904    16466
QR  8800    1922     5900  +  1
QR  8800    1923     2270  -  2
QR  8800    1924     6800  +
QR  8800    1925     3200  +  3
QR  8800    1926     2020  -  4
QR  8800    1927     7220  +
QR  8800    1928     3930  +  5
QR  8800    1929     2040  -
QR  8800    1930     1720  -
QR  8800    1931     1420  -  6
QR  8800    1932     1870  -
QR  8800    1933     5680  +  7
QR  8800    1934     2650  -
QR  8800    1935     1840  -  8
QR  8800    1936    12300  +
QR  8800    1937     3510  +  9
QR  8800    1938     6620  +
QR  8800    1939     3260  +
QR  8800    1940     2940  -
QR  8800    1941     1860  -
QR  8800    1942     1960  -  10
QR  8800    1943     2740  -
QR  8800    1944     2200  -
QR  8800    1945     8560  +  11
QR  8800    1946     2520  -
QR  8800    1947     2540  -
QR  8800    1948     2360  -  12
QR  8800    1949     5520  +  13
QR  8800    1950     1780  -  14
QR  8800    1951     8520  +  15
QR  8800    1952     4780  +
QR  8800    1953     3190  -
QR  8800    1954     1360  -  16
QR  8800    1955     8570  +
QR  8800    1956    12000  +  17
QR  8800    1957     2540  -  18
QR  8800    1958     4750  +  19
QR  8800    1959     1640  -  20
QR  8800    1960     3550  +
QR  8800    1961     3950  +  21
QR  8800    1962     2520  -
QR  8800    1963     1310  -
QR  8800    1964     1360  -
QR  8800    1965     2160  -
QR  8800    1966     1460  -
```

RUN TEST 8/18/92 SAE

QR	8800	1967	1380	−	22
QR	8800	1968	9530	+	23
QR	8800	1969	3160	−	24
QR	8800	1970	4540	+	
QR	8800	1971	6420	+	25
QR	8800	1972	3190	−	26
QR	8800	1973	5580	+	
QR	8800	1974	7040	+	27
QR	8800	1975	3380	+	
QR	8800	1976	3090	−	28
QR	8800	1977	5290	+	
QR	8800	1978	10300	+	
QR	8800	1979	5130	+	29
QR	8800	1980	6700	+	
QR	8800	1981	3630	+	
QR	8800	1982	2590	−	30
QR	8800	1983	5160	+	
QR	8800	1984	15400	+	31
QR	8800	1985	2300	−	
QR	8800	1986	2760	−	32
QR	8800	1987	6490	+	33
QR	8800	1988	2030	−	34
QR	8800	1989	5510	+	
QR	8800	1990	4070	+	35
QR	8800	1991	2560	−	36
ED					

$n_1 = n_2 = 35$

$U = 36$

# of runs = 36

$$\boxed{36 = \bar{U}} = \frac{2(n_1 n_2)}{(n_1 + n_2)} + 1 = \frac{2(35)(35)}{(35+35)} + 1$$

$$\boxed{17.246 = S_U^2} = \frac{2(35)(35)(2(35)(35) - 35 - 35)}{(35+35)^2 (35+35-1)}$$

$$\frac{(2450)(2380)}{(4900)(69)} = 17.246$$

$$S_U = 4.1529$$

$$\boxed{\emptyset = Z} = \frac{U - \bar{U}}{S_U} = \frac{36-36}{4.1529} = \emptyset$$

8/18/17

Pompton River at Pompton Plains

Rank	Year	Flow
1	1984	25400
2	1956	15800
3	1968	13100
4	1951	13100
5	1955	12900
6	1987	12600
7	1989	12500
8	1980	12000
9	1978	11800
10	1973	11500
11	1983	10100
12	1952	9790
13	1974	9730
14	1945	9690
15	1979	9520
16	1977	9000
17	1971	8610
18	1970	8520
19	1972	7240
20	1949	6780
21	1976	6670
22	1958	6650
23	1975	6150
24	1953	5980
25	1960	5740

Rank	Year	Flow
26	1990	5730
27	1961	5240
28	1981	4600
29	1991	4370
30	1943	4310
31	1986	4270
32	1969	4220
33	1957	4050
34	1982	3880
35	1947	3750
36	1946	3590
37	1988	3260
38	1962	3140
39	1944	3100
40	1948	3100
41	1967	2880
42	1942	2840
43	1985	2710
44	1965	2600
45	1950	2480
46	1941	2440
47	1959	2260
48	1966	1980
49	1954	1790
50	1964	1700
51	1963	1500

Median = 5730 ✓

8/18/4

```
TT
TT      *****   NO URBANIZATION ADJUSTMENT  ******  7/14/92
TT
TT PASSAIC RIVER BASIN - POMPTON RIVER AT POMPTON PLAINS
TT DA=355 SQ MI    *** All gage DATA - no urbanization adjustment ***
TT VALUES FOR 1968 & 1978 TAKEN FROM NEW YORK FILE
TT WATER YEAR 1989 CORRECTED FROM 10100 TO 12500  ---  7/14/92
J1      1       0       0       0       0       0       1
ID 388500  POMPTON   RIVER AT POMPTON PLAINS
GS  8850   0.207      .4
QH  8850   1904   29273
QR  8850   1941   2440
QR  8850   1942   2840
QR  8850   1943   4310
QR  8850   1944   3100
QR  8850   1945   9690
QR  8850   1946   3590
QR  8850   1947   3750
QR  8850   1948   3100
QR  8850   1949   6780
QR  8850   1950   2480
QR  8850   1951   13100
QR  8850   1952   9790
QR  8850   1953   5980
QR  8850   1954   1790
QR  8850   1955   12900
QR  8850   1956   15800
QR  8850   1957   4050
QR  8850   1958   6650
QR  8850   1959   2260
QR  8850   1960   5740
QR  8850   1961   5240
QR  8850   1962   3140
QR  8850   1963   1500
QR  8850   1964   1700
QR  8850   1965   2600
QR  8850   1966   1980
QR  8850   1967   2880
QR  8850   1968   13100
QR  8850   1969   4220
QR  8850   1970   8520
QR  8850   1971   8610
QR  8850   1972   7240
QR  8850   1973   11500
QR  8850   1974   9730
QR  8850   1975   6150
QR  8850   1976   6670
QR  8850   1977   9000
QR  8850   1978   11800
QR  8850   1979   9520
QR  8850   1980   12000
QR  8850   1981   4600
QR  8850   1982   3880
QR  8850   1983   10100
QR  8850   1984   25400
QR  8850   1985   2710
QR  8850   1986   4270
QR  8850   1987   12600
QR  8850   1988   3260
```

QR	8850	1989	12500 +	
QR	8850	1990	5730 +	22
QR	8850	1991	4370 −	23
ED				

$n_1 = 25$ ✓

$n_2 = 25$ ✓

OF RUNS = $\boxed{23 = U}$ ✓

$$\bar{U} = \frac{2(25)(25)}{25+25} + 1 = \boxed{26 = \bar{U}}$$ ✓

$$S_U^2 = \frac{2(25)(25)(2(25)(25) - 25 - 25)}{(25+25)^2(25+25-1)} = \frac{(1250)(1200)}{(2500)(49)}$$

$$\boxed{S_U^2 = 12.25}$$ ✓

$$\boxed{S_U = 3.5}$$ ✓

$$Z = \frac{U - \bar{U}}{S_U} = \frac{23-26}{3.5} = \boxed{-.86 = Z}$$ ✓

Passaic River at Little Falls

Rank	Year	Flow
1	1904	32300
2	1902	23400
3	1945	19500
4	1936	19200
5	1984	18400
6	1968	13500
7	1951	12400
8	1952	12000
9	1956	11600
10	1920	11600
11	1903	11000
12	1924	10787
13	1989	10700
14	1973	10600
15	1912	10400
16	1987	10300
17	1972	10300
18	1979	10200
19	1980	9870
20	1955	9850
21	1983	9740
22	1975	9640
23	1898	9500
24	1900	9430
25	1938	9350
26	1901	9270
27	1908	9190
28	1978	9180
29	1970	9110
30	1910	8980
31	1949	8960
32	1974	8910
33	1905	8760
34	1927	8707
35	1919	8630
36	1933	8480
37	1977	8440
38	1907	8100
39	1971	8010
40	1914	7980
41	1899	7830
42	1915	7800
43	1923	7675
44	1961	7610
45	1943	7490
46	1913	7450
47	1953	7430

Rank	Year	Flow
48	1960	7140
49	1946	7090
50	1942	7050
51	1922	6563
52	1918	6490
53	1909	6490
54	1976	6390
55	1958	6330
56	1921	6244
57	1986	6150
58	1916	6130
59	1906	6110
60	1925	6036
61	1990	6020
62	1928	5965
63	1967	5720
64	1939	5670
65	1934	5660
66	1940	5620
67	1926	5519
68	1947	5490
69	1937	5400
70	1957	5270
71	1948	5220
72	1969	5210
73	1929	5116
74	1944	5080
75	1941	5070
76	1962	4960
77	1932	4650
78	1965	4630
79	1917	4530
80	1981	4370
81	1950	4320
82	1982	4230
83	1991	4160
84	1931	3953
85	1935	3950
86	1988	3670
87	1966	3630
88	1930	3614
89	1959	3600
90	1964	3420
91	1911	3370
92	1963	3250
93	1985	3090
94	1954	2970

Median = ~~7420~~ 7285

$$\frac{7430 + 7140}{2} = 7285$$

```
TT
TT     *****  NO  Urbanization Adjustment     7/14/92    ******
TT
TT ADDED WATER YEARS 1990 & 1991
TT FLOOD FLOW FREQUENCY TEST
TT PASSAIC RIVER - LITTLE FALLS
TT DA=762 SQ MI
TT WATER YEAR 1989 VALUE CHANGED FROM  10200 TO 10700
TT 1811, 1865, & 1882 NOT INCLUDED AS HISTORIC DATA FOR N=88 PER NY
TT 1882 IS INCLUDED AS A QR AS PER NEW YORK FILE
TT 1904, 1978 VALUES TAKEN FROM NEW YORK FILE
J1     1        0        0        0        0        0        1
ID  389500  PASSAIC RIVER AT LITTLE FALLS
GS  8950    0.207       .4
QR  8950     1882    19000
QR  8950     1898    18700
QR  8950     1898     9500 +
QR  8950     1899     7830 +
QR  8950     1900     9430 +
QR  8950     1901     9270 +
QR  8950     1902    23400 +
QR  8950     1903    11000 +
QR  8950     1904    32300 +
QR  8950     1905     8760 +(1)
QR  8950     1906     6110 -2
QR  8950     1907     8100 +
QR  8950     1908     9190 +3
QR  8950     1909     6490 -4
QR  8950     1910     8980 +5
QR  8950     1911     3370 -6
QR  8950     1912    10400 +
QR  8950     1913     7450 +
QR  8950     1914     7980 +
QR  8950     1915     7800 +7
QR  8950     1916     6130 -
QR  8950     1917     4530 -
QR  8950     1918     6490 -8
QR  8950     1919     8630 +
QR  8950     1920    11600 +9
QR  8950     1921     6244 -
QR  8950     1922     6563 -10
QR  8950     1923     7675 +
QR  8950     1924    10787 +11
QR  8950     1925     6036 -
QR  8950     1926     5519 -12
QR  8950     1927     8707 +13
QR  8950     1928     5965 -
QR  8950     1929     5116 -
QR  8950     1930     3614 -
QR  8950     1931     3953 -
QR  8950     1932     4650 -14
QR  8950     1933     8480 +15
QR  8950     1934     5660 -
QR  8950     1935     3950 -16
QR  8950     1936    19200 +17
QR  8950     1937     5400 -17
QR  8950     1938     9350 +18
QR  8950     1939     5670 -19
QR  8950     1940     5620 -
QR  8950     1941     5070 -20
```

8/18/9

QR	8950	1942	7050		
QR	8950	1943	7490	+	21
QR	8950	1944	5080	−	22
QR	8950	1945	19500	+	23
QR	8950	1946	7090	−	
QR	8950	1947	5490	−	24
QR	8950	1948	5220	−	
QR	8950	1949	8960	+	25
QR	8950	1950	4320	−	26
QR	8950	1951	12400	+	
QR	8950	1952	12000	+	
QR	8950	1953	7430	+	27
QR	8950	1954	2970	−	28
QR	8950	1955	9850	+	
QR	8950	1956	11600	+	29
QR	8950	1957	5270	−	
QR	8950	1958	6330	+	
QR	8950	1959	3600	−	30
QR	8950	1960	7140	−	
QR	8950	1961	7610	+	31
QR	8950	1962	4960	−	
QR	8950	1963	3250	−	
QR	8950	1964	3420	−	
QR	8950	1965	4630	−	
QR	8950	1966	3630	−	32
QR	8950	1967	5720	−	
QR	8950	1968	13500	+	33
QR	8950	1969	5210	−	34
QR	8950	1970	9110	+	
QR	8950	1971	8010	+	
QR	8950	1972	10300	+	
QR	8950	1973	10600	+	
QR	8950	1974	8910	+	35
QR	8950	1975	9640	+	
QR	8950	1976	6390	−	36
QR	8950	1977	8440	+	
QR	8950	1978	9180	+	
QR	8950	1979	10200	+	37
QR	8950	1980	9870	+	
QR	8950	1981	4370	−	
QR	8950	1982	4230	−	38
QR	8950	1983	9740	+	
QR	8950	1984	18400	+	39
QR	8950	1985	3090	−	
QR	8950	1986	6150	−	40
QR	8950	1987	10300	+	41
QR	8950	1988	3670	−	42
QR	8950	1989	10700	+	43
QR	8950	1990	6020	−	
QR	8950	1991	4160	−	44
ED					

$n_1 = 47$ ✓

$n_2 = 47$ ✓

OF RUNS $(U) = \boxed{44 = U}$

$$\bar{U} = \frac{2(47)(47)}{(47+47)} + 1 = \boxed{48} = \bar{U}$$

$$S_U^2 = \frac{2(47)(47)(2(47)(47)-47-47)}{(47+47)^2(47+47-1)} = \boxed{23.25}$$

$$\boxed{S_U = 4.822}$$

$$\frac{(4418)(4324)}{(8836)(93)}$$

$$Z = \frac{44-48}{4.822} = \boxed{\cancel{-1.658 = Z}}$$

$$Z = -0.8295$$

Attachment 2

KENDALL TESTS

PASSAIC RIVER AT CHATHAM				
DA=100 SQ MI 18 AUGUST 1992 JPB				
HISTORICAL VALUES REMOVED				
Year	Q (cfs)	# >	# <	Σ=(#>)-(#<)
1938	1910	6	47	-41
1939	972	43	9	34
1940	1090	36	15	21
1941	756	48	2	46
1942	2020	4	45	-41
1943	1420	12	36	-24
1944	964	40	7	33
1945	1040	34	12	22
1946	1040	34	11	23
1947	869	41	3	38
1948	1410	13	30	-17
1949	1710	5	37	-32
1950	670	40	1	39
1951	1010	32	8	24
1952	1290	19	20	-1
1953	1150	26	12	14
1954	651	37	0	37
1955	1390	13	23	-10
1956	1150	25	10	15
1957	1100	27	7	20
1958	1140	25	8	17
1959	770	32	0	32
1960	1110	25	6	19
1961	1290	18	12	6
1962	1450	10	19	-9
1963	980	24	4	20
1964	872	27	0	27
1965	980	24	2	22
1966	1010	23	2	21
1967	1320	16	8	8
1968	2560	1	22	-21
1969	1640	5	17	-12
1970	1420	8	13	-5
1971	2540	1	19	-18
1972	1520	5	14	-9
1973	3200	0	18	-18
1974	1400	5	12	-7
1975	2520	0	16	-16
1976	1240	9	6	3
1977	1390	4	10	-6
1978	1520	3	10	-7
1979	1710	1	11	-10
1980	1370	3	8	-5
1981	899	10	0	10
1982	1240	5	4	1
1983	1380	2	6	-4

WHIPPANY RIVER AT MORRISTOWN				
DA=29.4 SQ MI 18 AUGUST 1992 JPB				
HISTORICAL VALUES REMOVED				
Year	Q (cfs)	# >	# <	Σ=(#>)-(#<)
1922	1020	30	39	-9
1923	815	41	27	14
1924	1530	5	62	-57
1925	975	31	35	-4
1926	783	40	25	15
1927	450	60	4	56
1928	2000	2	61	-59
1929	1290	14	48	-34
1930	570	48	13	35
1931	552	48	12	36
1932	604	46	13	33
1933	1820	2	56	-54
1934	847	34	23	11
1935	467	51	5	46
1936	1500	3	52	-49
1937	491	48	6	42
1938	1170	14	39	-25
1939	529	44	8	36
1940	1500	3	48	-45
1941	679	36	14	22
1942	1220	12	37	-25
1943	540	40	8	32
1944	640	35	12	23
1945	460	42	4	38
1946	1020	22	23	-1
1947	591	37	7	30
1948	829	29	14	15
1949	1350	8	34	-26
1950	375	39	2	37
1951	938	24	16	8
1952	1360	6	33	-27
1953	804	26	12	14
1954	282	37	0	37
1955	1160	10	26	-16
1956	900	23	12	11
1957	613	29	5	24
1958	925	22	11	11
1959	509	28	4	24
1960	684	25	6	19
1961	475	28	2	26
1962	1160	10	19	-9
1963	367	28	0	28
1964	401	27	0	27
1965	716	23	3	20
1966	640	23	2	21
1967	500	24	0	24

Kendall Tests

PASSAIC RIVER AT CHATHAM – GAGE #379500				
Year	Q (cfs)	# >	# <	Σ=(#>)–(#<)
1984	1930	0	7	-7
1985	960	6	0	6
1986	1550	0	5	-5
1987	1210	3	1	2
1988	1220	2	1	1
1989	1330	0	2	-2
1990	1190	1	0	1
1991	1250	0	0	0
S =	235			
VAR(S) =	17967			
Z =	1.75			

PASSAIC RIVER BASIN - WHIPPANY RIVER AT MORRISTOW				
Year	Q (cfs)	# >	# <	Σ=(#>)–(#<)
1968	956	19	4	15
1969	624	22	0	22
1970	1310	8	13	-5
1971	2280	0	20	-20
1972	946	17	2	15
1973	1360	5	13	-8
1974	1450	3	14	-11
1975	1020	14	2	12
1976	752	15	0	15
1977	1480	2	12	-10
1978	1150	5	8	-3
1979	2080	0	12	-12
1980	1100	8	3	5
1981	1130	5	5	0
1982	871	9	0	9
1983	1250	3	5	-2
1984	1680	0	7	-7
1985	1120	4	2	2
1986	1060	5	0	5
1987	1150	2	2	0
1988	1080	3	0	3
1989	1380	0	2	-2
1990	1350	0	1	-1
1991	1130	0	0	0
S =	397			
VAR(S) =	38908.3			
Z =	2.01			

ROCKAWAY RIVER ABOVE BOONTON RESERVOIR				
DA=116 SQ MI 18 AUGUST 1992 JPB				
REMOVED HISTORICAL FLOWS				
Year	Q (cfs)	# >	# <	Σ=(#>)-(#<)
1938	2260	19	34	-15
1939	1310	43	9	34
1940	2390	17	34	-17
1941	908	48	2	46
1942	1200	42	7	35
1943	1450	38	10	28
1944	1620	32	15	17
1945	1540	33	13	20
1946	1440	36	9	27
1947	1470	34	10	24
1948	1790	28	15	13
1949	3590	6	36	-30
1950	991	39	2	37
1951	2460	15	25	-10
1952	4360	2	37	-35
1953	2050	19	19	0
1954	705	37	0	37
1955	2190	17	19	-2
1956	1940	19	16	3
1957	1340	29	5	24
1958	2080	17	16	1
1959	1170	29	3	26
1960	1690	23	8	15
1961	1730	22	8	14
1962	1860	18	11	7
1963	862	28	0	28
1964	1000	27	0	27
1965	1490	22	4	18
1966	1070	25	0	25
1967	1200	24	0	24
1968	3400	6	17	-11
1969	1550	20	2	18
1970	2870	8	13	-5
1971	3550	5	15	-10
1972	2200	13	6	7
1973	4100	3	15	-12
1974	4300	2	15	-13
1975	2230	10	6	4
1976	2550	7	8	-1
1977	3970	2	12	-10
1978	2590	5	8	-3
1979	5430	1	11	-10
1980	2670	3	8	-5
1981	2040	5	5	0
1982	1780	7	2	5
1983	3030	2	6	-4

RAMAPO RIVER AT POMPTON LAKES				
DA = 160 SQ MI 18 AUGUST 1992 JPB				
REMOVED HISTORICAL VALUES				
Year	Q (cfs)	# >	# <	Σ=(#>)-(#<)
1922	5900	15	54	-39
1923	2270	50	18	32
1924	6800	10	57	-47
1925	3200	32	34	-2
1926	2020	52	13	39
1927	7220	8	56	-48
1928	3930	25	38	-13
1929	2040	48	14	34
1930	1720	54	7	47
1931	1420	56	4	52
1932	1870	50	9	41
1933	5680	13	45	-32
1934	2650	36	21	15
1935	1840	49	7	42
1936	12300	1	54	-53
1937	3510	25	29	-4
1938	6620	9	44	-35
1939	3260	25	27	-2
1940	2940	29	22	7
1941	1860	43	7	36
1942	1960	42	7	35
1943	2740	30	18	12
1944	2200	38	9	29
1945	8560	5	41	-36
1946	2520	33	12	21
1947	2540	31	13	18
1948	2360	33	10	23
1949	5520	11	31	-20
1950	1780	35	6	29
1951	8520	5	35	-30
1952	4780	14	25	-11
1953	3190	21	17	4
1954	1360	35	2	33
1955	8570	4	32	-28
1956	12000	1	34	-33
1957	2540	25	9	16
1958	4750	12	21	-9
1959	1640	28	4	24
1960	3550	16	15	1
1961	3950	14	16	-2
1962	2520	22	7	15
1963	1310	28	0	28
1964	1360	27	0	27
1965	2160	23	3	20
1966	1460	24	1	23
1967	1380	24	0	24

Kendall Tests

ROCKAWAY RIVER ABOVE BOONTON RESERVOIR					RAMAPO RIVER AT POMPTON LAKES				
Year	Q (cfs)	# >	# <	Σ=(#>)−(#<)	Year	Q (cfs)	# >	# <	Σ=(#>)−(#<)
1984	5600	0	7	-7	1968	9530	2	21	-19
1985	1460	5	1	4	1969	3160	16	6	10
1986	1830	3	2	1	1970	4540	11	10	1
1987	2530	1	3	-2	1971	6420	5	15	-10
1988	1430	3	0	3	1972	3190	13	6	7
1989	2350	1	1	0	1973	5580	5	13	-8
1990	3350	0	1	-1	1974	7040	2	15	-13
1991	1830	0	0	0	1975	3380	10	6	4
					1976	3090	10	5	5
					1977	5290	5	9	-4
S =	369				1978	10300	1	12	-11
VAR(S) =	17967				1979	5130	5	7	-2
					1980	6700	1	10	-9
Z =	2.75				1981	3630	5	5	0
					1982	2590	6	3	3
					1983	5160	3	5	-2
					1984	15400	0	7	-7
					1985	2300	5	1	4
					1986	2760	3	2	1
					1987	6490	0	4	-4
					1988	2030	3	0	3
					1989	5510	0	2	-2
					1990	4070	0	1	-1
					1991	2560	0	0	0
					S =	229			
					VAR(S) =	38908.3			
					Z =	1.16			

POMPTON RIVER AT POMPTON PLAINS					PASSAIC RIVER - LITTLE FALLS				
TT DA=355 SQ MI no urbanization adjustment					DA=762 SQ MI				
WATER YEAR PEAK FLOW DATA					REMOVED 1882 AND 1896 VALUES				
Year	Q (cfs)	# >	# <	Σ=(#>)-(#<)	Year	Q (cfs)	# >	# <	Σ=(#>)-(#<)
1941	2440	45	5	40	1899	7830	39	54	-15
1942	2840	41	8	33	1900	9430	22	70	-48
1943	4310	29	19	10	1901	9270	23	68	-45
1944	3100	37	10	27	1902	23400	1	89	-88
1945	9690	13	33	-20	1903	11000	9	80	-71
1946	3590	33	12	21	1904	32300	0	88	-88
1947	3750	32	12	20	1905	8760	26	61	-35
1948	3100	34	9	25	1906	6110	50	36	14
1949	6780	18	24	-6	1907	8100	30	55	-25
1950	2480	36	5	31	1908	9190	20	64	-44
1951	13100	2	38	-36	1909	6490	41	42	-1
1952	9790	10	29	-19	1910	8980	22	60	-38
1953	5980	19	19	0	1911	3370	77	4	73
1954	1790	35	2	33	1912	10400	11	69	-58
1955	12900	3	33	-30	1913	7450	33	46	-13
1956	15800	1	34	-33	1914	7980	28	50	-22
1957	4050	24	10	14	1915	7800	28	49	-21
1958	6650	15	18	-3	1916	6130	41	35	6
1959	2260	29	3	26	1917	4530	60	15	45
1960	5740	16	15	1	1918	6490	36	38	-2
1961	5240	17	13	4	1919	8630	24	49	-25
1962	3140	23	6	17	1920	11600	6	66	-60
1963	1500	28	0	28	1921	6244	36	35	1
1964	1700	27	0	27	1922	6563	33	37	-4
1965	2600	25	1	24	1923	7675	26	43	-17
1966	1980	25	0	25	1924	10787	7	61	-54
1967	2880	23	1	22	1925	6036	34	33	1
1968	13100	1	22	-21	1926	5519	40	26	14
1969	4220	19	3	16	1927	8707	21	44	-23
1970	8520	11	10	1	1928	5965	34	30	4
1971	8610	10	10	0	1929	5116	43	20	23
1972	7240	10	9	1	1930	3614	56	6	50
1973	11500	5	13	-8	1931	3953	52	9	43
1974	9730	6	11	-5	1932	4650	46	14	32
1975	6150	9	7	2	1933	8480	21	38	-17
1976	6670	8	7	1	1934	5660	35	23	12
1977	9000	7	7	0	1935	3950	49	8	41
1978	11800	4	9	-5	1936	19200	1	55	-54
1979	9520	5	7	-2	1937	5400	36	19	17
1980	12000	3	8	-5	1938	9350	15	39	-24
1981	4600	5	5	0	1939	5670	32	21	11
1982	3880	7	2	5	1940	5620	32	20	12
1983	10100	3	5	-2	1941	5070	37	14	23
1984	25400	0	7	-7	1942	7050	26	24	2
1985	2710	6	0	6	1943	7490	22	27	-5
1986	4270	4	1	3	1944	5080	34	14	20
1987	12600	0	4	-4	1945	19500	0	47	-47
1988	3260	3	0	3	1946	7090	23	23	0

Kendall Tests

POMPTON RIVER AT POMPTON PLAINS					PASSAIC RIVER - LITTLE FALLS				
Year	Q (cfs)	# >	# <	Σ=(#>)-(#<)	Year	Q (cfs)	# >	# <	Σ=(#>)-(#<)
1989	12500	0	2	-2	1947	5490	28	17	11
1990	5730	0	1	-1	1948	5220	29	15	14
1991	4370	0	0	0	1949	8960	16	27	-11
					1950	4320	32	10	22
					1951	12400	2	39	-37
S =	257				1952	12000	2	38	-36
VAR(S) =	15158.3				1953	7430	18	21	-3
					1954	2970	37	1	36
Z =	2.08				1955	9850	9	28	-19
					1956	11600	2	34	-32
					1957	5270	22	13	9
					1958	6330	18	16	2
					1959	3600	29	4	25
					1960	7140	16	16	0
					1961	7610	15	16	-1
					1962	4960	20	10	10
					1963	3250	27	2	25
					1964	3420	26	2	24
					1965	4630	20	7	13
					1966	3630	24	2	22
					1967	5720	18	7	11
					1968	13500	1	23	-22
					1969	5210	17	6	11
					1970	9110	10	12	-2
					1971	8010	12	9	3
					1972	10300	3	17	-14
					1973	10600	2	17	-15
					1974	8910	8	10	-2
					1975	9640	6	11	-5
					1976	6390	8	8	0
					1977	8440	7	8	-1
					1978	9180	6	8	-2
					1979	10200	3	10	-7
					1980	9870	3	9	-6
					1981	4370	6	5	1
					1982	4230	6	4	2
					1983	9740	3	6	-3
					1984	18400	0	8	-8
					1985	3090	6	1	5
					1986	6150	2	4	-2
					1987	10300	1	4	-3
					1988	3670	3	1	2
					1989	10700	0	3	-3
					1990	6020	0	2	-2
					1991	4160	0	1	-1
					S =	-489			
					VAR(S) =	93733.7			
					Z =	-1.59			

TT WANAQUE RAINFALL
TT CHARLOTTEBURG used FOR WY75 & 76
TT KENDALL TEST 17 JULY 1992

1949.00	4.90	8.00	32.00	-24.00
1950.00	2.97	23.00	16.00	7.00
1951.00	5.16	5.00	33.00	-28.00
1952.00	4.38	7.00	30.00	-23.00
1953.00	2.44	26.00	10.00	16.00
1954.00	1.84	30.00	5.00	25.00
1955.00	5.22	4.00	30.00	-26.00
1956.00	5.32	3.00	30.00	-27.00
1957.00	3.50	9.00	23.00	-14.00
1958.00	4.19	5.00	26.00	-21.00
1959.00	1.60	27.00	3.00	24.00
1960.00	2.11	25.00	4.00	21.00
1961.00	1.22	26.00	2.00	24.00
1962.00	1.65	25.00	2.00	23.00
1963.00	2.20	24.00	2.00	22.00
1964.00	0.58	25.00	0.00	25.00
1965.00	2.51	21.00	3.00	18.00
1966.00	5.00	4.00	19.00	-15.00
1967.00	0.99	22.00	0.00	22.00
1968.00	5.98	1.00	20.00	-19.00
1969.00	2.58	17.00	3.00	14.00
1970.00	3.12	11.00	8.00	3.00
1971.00	7.40	0.00	18.00	-18.00
1972.00	3.12	10.00	7.00	3.00
1973.00	2.65	13.00	3.00	10.00
1974.00	3.19	9.00	6.00	3.00
1975.00	3.30	8.00	6.00	2.00
1976.00	3.00	9.00	4.00	5.00
1977.00	3.01	8.00	4.00	4.00
1978.00	5.87	0.00	11.00	-11.00
1979.00	2.54	8.00	2.00	6.00
1980.00	3.38	5.00	4.00	1.00
1981.00	3.66	3.00	5.00	-2.00
1982.00	2.29	7.00	0.00	7.00
1983.00	2.83	5.00	1.00	4.00
1984.00	4.15	1.00	4.00	-3.00
1985.00	5.06	0.00	4.00	-4.00
1986.00	3.37	2.00	1.00	1.00
1987.00	4.08	0.00	2.00	-2.00
1988.00	2.36	1.00	0.00	1.00
1989.00	3.40	0.00	0.00	0.00

S = 54.00
VAR(S) = 7926.67
Z = 0.60

TT LITTLE FALLS RAINFALL
TT
TT KENDALL TEST 17 JULY 1992

1949.00	3.55	12.00	28.00	-16.00
1950.00	2.60	24.00	15.00	9.00
1951.00	4.11	9.00	29.00	-20.00
1952.00	4.31	6.00	31.00	-25.00
1953.00	1.94	29.00	7.00	22.00
1954.00	0.82	33.00	2.00	31.00
1955.00	4.29	6.00	28.00	-22.00
1956.00	4.92	5.00	28.00	-23.00
1957.00	3.33	10.00	22.00	-12.00
1958.00	1.99	25.00	6.00	19.00
1959.00	2.53	20.00	10.00	10.00
1960.00	2.46	21.00	8.00	13.00
1961.00	1.25	25.00	3.00	22.00
1962.00	2.95	14.00	13.00	1.00
1963.00	1.93	22.00	4.00	18.00
1964.00	0.61	24.00	1.00	23.00
1965.00	2.21	21.00	3.00	18.00
1966.00	1.76	21.00	2.00	19.00
1967.00	0.33	22.00	0.00	22.00
1968.00	6.41	2.00	19.00	-17.00
1969.00	2.90	14.00	6.00	8.00
1970.00	2.41	18.00	1.00	17.00
1971.00	9.65	0.00	18.00	-18.00
1972.00	3.54	6.00	11.00	-5.00
1973.00	3.33	7.00	9.00	-2.00
1974.00	1.15	15.00	0.00	15.00
1975.00	9.11	0.00	14.00	-14.00
1976.00	2.49	13.00	0.00	13.00
1977.00	3.91	4.00	8.00	-4.00
1978.00	2.81	10.00	1.00	9.00
1979.00	2.59	10.00	0.00	10.00
1980.00	2.87	9.00	0.00	9.00
1981.00	3.31	5.00	3.00	2.00
1982.00	3.08	6.00	1.00	5.00
1983.00	4.23	2.00	4.00	-2.00
1984.00	5.34	0.00	5.00	-5.00
1985.00	5.04	0.00	4.00	-4.00
1986.00	3.37	1.00	2.00	-1.00
1987.00	3.28	1.00	1.00	0.00
1988.00	2.95	1.00	0.00	1.00
1989.00	3.92	0.00	0.00	0.00

S = 126.00
VAR(S) = 7926.67
Z = 1.40

Attachment 3

STUDENT T TESTS

1938	1910	1965	980
1939	972	1966	1010
1940	1090	1967	1320
1941	756	1968	2560
1942	2020	1969	1640
1943	1420	1970	1420
1944	964	1971	2540
1945	1040	1972	1520
1946	1040	1973	3200
1947	869	1974	1400
1948	1410	1975	2520
1949	1710	1976	1240
1950	670	1977	1390
1951	1010	1978	1520
1952	1290	1979	1710
1953	1150	1980	1370
1954	651	1981	899
1955	1390	1982	1240
1956	1150	1983	1380
1957	1100	1984	1930
1958	1140	1985	960
1959	770	1986	1550
1960	1110	1987	1210
1961	1290	1988	1220
1962	1450	1989	1330
1963	980	1990	1190
1964	872	1991	1250

PASSAIC AT CHATHAM

sum	**31224**	**sum**	**41499**
M1	**1156**	**M2**	**1537**
n1	**27**	**n2**	**27**
SD1	**342.86**	**SD2**	**557.85**
T=			
-2.96347			

T Test

1922	1020
1923	815
1924	1530
1925	975
1926	783
1927	450
1928	2000
1929	1290
1930	570
1931	552
1932	604
1933	1820
1934	847
1935	467
1936	1500
1937	491
1938	1170
1939	529
1940	1500
1941	679
1942	1220
1943	540
1944	640
1945	460
1946	1020
1947	591
1948	829
1949	1350
1950	375
1951	938
1952	1360
1953	804
1954	282
1955	1160
1956	900
1957	613
1958	925
1959	509
1960	684
1961	475
1962	1160
1963	367
1964	401

1965	716
1966	640
1967	500
1968	956
1969	624
1970	1310
1971	2280
1972	946
1973	1360
1974	1450
1975	1020
1976	752
1977	1480
1978	1150
1979	2080
1980	1100
1981	1130
1982	871
1983	1250
1984	1680
1985	1120
1986	1060
1987	1150
1988	1080
1989	1380
1990	1350
1991	1130
sum	31565
M2	1169
n2	27
SD2	405.80

WHIPPANY @ MORRISTOWN

sum	**37195**
M1	**865**
n1	**43**
SD1	**416.12**

sum	**31565**
M2	**1169**
n2	**27**
SD2	**405.80**
T=	**-2.96123**

1938	2260	1965	1490
1939	1310	1966	1070
1940	2390	1967	1200
1941	908	1968	3400
1942	1200	1969	1550
1943	1450	1970	2870
1944	1620	1971	3550
1945	1540	1972	2200
1946	1440	1973	4100
1947	1470	1974	4300
1948	1790	1975	2230
1949	3590	1976	2550
1950	991	1977	3970
1951	2460	1978	2590
1952	4360	1979	5430
1953	2050	1980	2670
1954	705	1981	2040
1955	2190	1982	1780
1956	1940	1983	3030
1957	1340	1984	5600
1958	2080	1985	1460
1959	1170	1986	1830
1960	1690	1987	2530
1961	1730	1988	1430
1962	1860	1989	2350
1963	862	1990	3350
1964	1000	1991	1830

SUM	**47396**	**72400**
M1	**1755**	**2681**
n1	**27**	**27**
SD1	**805.54**	**1209.85**

ROCKAWAY RIVER ABOVE BOONTON RESERVOIR

T=(M1-M2)/{[(n1S1^2+n2S2^2)/(n1+n2-2)](n1+n2)/(n1n2)}^0.5

T= -3.24879

T Test

1922	5900	1965	2160
1923	2270	1966	1460
1924	6800	1967	1380
1925	3200	1968	9530
1926	2020	1969	3160
1927	7220	1970	4540
1928	3930	1971	6420
1929	2040	1972	3190
1930	1720	1973	5580
1931	1420	1974	7040
1932	1870	1975	3380
1933	5680	1976	3090
1934	2650	1977	5290
1935	1840	1978	10300
1936	12300	1979	5130
1937	3510	1980	6700
1938	6620	1981	3630
1939	3260	1982	2590
1940	2940	1983	5160
1941	1860	1984	15400
1942	1960	1985	2300
1943	2740	1986	2760
1944	2200	1987	6490
1945	8560	1988	2030
1946	2520	1989	5510
1947	2540	1990	4070
1948	2360	1991	2560
1949	5520		
1950	1780		
1951	8520		
1952	4780		
1953	3190		
1954	1360		
1955	8570		
1956	12000		
1957	2540		
1958	4750		
1959	1640		
1960	3550		
1961	3950		
1962	2520		
1963	1310		
1964	1360		

RAMAPO AT POMPTON LAKES

SUM	**169270**	**130850**
M1	**3937**	**4846**
n1	**43**	**27**
SD1	**2775.06**	**3104.73**

T=(M1-M2)/{[(n1S1^2+n2S2^2)/(n1+n2-2)](n1+n2)/(n1n2)}^0.5

-1.25637

1941	2440	1965	2600
1942	2840	1966	1980
1943	4310	1967	2880
1944	3100	1968	13100
1945	9690	1969	4220
1946	3590	1970	8520
1947	3750	1971	8610
1948	3100	1972	7240
1949	6780	1973	11500
1950	2480	1974	9730
1951	13100	1975	6150
1952	9790	1976	6670
1953	5980	1977	9000
1954	1790	1978	11800
1955	12900	1979	9520
1956	15800	1980	12000
1957	4050	1981	4600
1958	6650	1982	3880
1959	2260	1983	10100
1960	5740	1984	25400
1961	5240	1985	2710
1962	3140	1986	4270
1963	1500	1987	12600
1964	1700	1988	3260
		1989	12500
		1990	5730
		1991	4370

POMPTON @ POMPTON PLAINS

SUM	**131720**	
M1	**5488**	**214940**
n1	**24**	**7961**
SD1	**3986.19**	**27**
		4988.78

T=(M1-M2)/{[(n1S1^2+n2S2^2)/(n1+n2-2)](n1+n2)/(n1n2)}^0.5
T=
-1.9008114

T Test

		1931	3953	1965	4630
		1932	4650	1966	3630
		1933	8480	1967	5720
		1934	5660	1968	13500
		1935	3950	1969	5210
		1936	19200	1970	9110
		1937	5400	1971	8010
		1938	9350	1972	10300
		1939	5670	1973	10600
1905	8760	1940	5620	1974	8910
1906	6110	1941	5070	1975	9640
1907	8100	1942	7050	1976	6390
1908	9190	1943	7490	1977	8440
1909	6490	1944	5080	1978	9180
1910	8980	1945	19500	1979	10200
1911	3370	1946	7090	1980	9870
1912	10400	1947	5490	1981	4370
1913	7450	1948	5220	1982	4230
1914	7980	1949	8960	1983	9740
1915	7800	1950	4320	1984	18400
1916	6130	1951	12400	1985	3090
1917	4530	1952	12000	1986	6150
1918	6490	1953	7430	1987	10300
1919	8630	1954	2970	1988	3670
1920	11600	1955	9850	1989	10700
1921	6244	1956	11600	1990	6020
1922	6563	1957	5270	1991	4160
1923	7675	1958	6330		
1924	10787	1959	3600		
1925	6036	1960	7140		
1926	5519	1961	7610		
1927	8707	1962	4960		
1928	5965	1963	3250	**Passaic at Little Falls**	
1929	5116	1964	3420		
1930	3614				
		SUM	**433269**	**SUM**	**214170**
		M1	**7221.15**	**M2**	**7932**
		n1	**60**	**n2**	**27**
		SD1	**3246.0562**	**SD2**	**3502.64**

T= **-0.91139**

age 13869500, Pompton at Pompton Plains

Pre-65	log Q	Post-65	log Q	Pre-59	log Q	Post-59	log Q	Pre-72	log Q	Post-72	log Q
1941	3.387	1965	3.415	1941	3.387	1959	3.354	1941	3.387	1972	3.860
1942	3.453	1966	3.297	1942	3.453	1960	3.759	1942	3.453	1973	4.061
1943	3.634	1967	3.459	1943	3.634	1961	3.719	1943	3.634	1974	3.988
1944	3.491	1968	4.117	1944	3.491	1962	3.497	1944	3.491	1975	3.789
1945	3.986	1969	3.625	1945	3.986	1963	3.176	1945	3.986	1976	3.824
1946	3.555	1970	3.930	1946	3.555	1964	3.230	1946	3.555	1977	3.954
1947	3.575	1971	3.935	1947	3.575	1965	3.415	1947	3.575	1978	4.072
1948	3.491	1972	3.860	1948	3.491	1966	3.297	1948	3.491	1979	3.979
1949	3.831	1973	4.061	1949	3.831	1967	3.459	1949	3.831	1980	4.079
1950	3.394	1974	3.988	1950	3.394	1968	4.117	1950	3.394	1981	3.663
1951	4.117	1975	3.789	1951	4.117	1969	3.625	1951	4.117	1982	3.589
1952	3.991	1976	3.824	1952	3.991	1970	3.930	1952	3.991	1983	4.004
1953	3.777	1977	3.954	1953	3.777	1971	3.935	1953	3.777	1984	4.405
1954	3.253	1978	4.072	1954	3.253	1972	3.860	1954	3.253	1985	3.433
1955	4.111	1979	3.979	1955	4.111	1973	4.061	1955	4.111	1986	3.630
1956	4.199	1980	4.079	1956	4.199	1974	3.988	1956	4.199	1987	4.079
1957	3.607	1981	3.663	1957	3.607	1975	3.789	1957	3.607	1988	3.513
1958	3.823	1982	3.589	1958	3.823	1976	3.824	1958	3.823	1989	4.097
1959	3.354	1983	4.004			1977	3.954	1959	3.354	1990	3.758
1960	3.759	1984	4.405			1978	4.072	1960	3.759	1991	3.640
1961	3.719	1985	3.433			1979	3.979	1961	3.719		
1962	3.497	1986	3.630			1980	4.079	1962	3.497		
1963	3.176	1987	4.079			1981	3.663	1963	3.176		
1964	3.230	1988	3.513			1982	3.589	1964	3.230		
		1989	4.097			1983	4.004	1965	3.415		
		1990	3.758			1984	4.405	1966	3.297		
		1991	3.640			1985	3.433	1967	3.459		
						1986	3.630	1968	4.117		
						1987	4.079	1969	3.625		
						1988	3.513	1970	3.930		
						1989	4.097	1971	3.935		
						1990	3.758				
						1991	3.640				
	3.642		***3.822***		***3.704***		***3.756***		***3.651***		***3.690***
	0.085		***0.068***		***0.081***		***0.089***		***0.085***		***0.079***
		1965	***-2.320***			*1959*	***-0.594***			*1972*	***-0.462***

student t statistic for split record

Attachment 4

MANN - WHITNEY TESTS

Mann-Whitney test

year	flow	ordered	flow	group #	rank	group 1	group -1
1941	2440	1	1500	1	1	1	0
1942	2840	1	1700	-1	2	0	2
1943	4310	1	1790	1	3	3	0
1944	3100	1	1980	-1	4	0	4
1945	9690	1	2260	1	5	5	0
1946	3590	1	2440	1	6	6	0
1947	3760	1	2480	1	7	7	0
1948	3100	1	2600	-1	8	0	8
1949	6780	1	2710	-1	9	0	9
1950	2480	1	2840	1	10	10	0
1951	13100	1	2880	-1	11	0	11
1952	9790	1	3100	1	12	12	0
1953	5980	1	3100	1	13	13	0
1954	1790	1	3140	1	14	14	0
1955	12900	1	3260	-1	15	0	15
1956	15800	1	3590	1	16	16	0
1957	4050	1	3760	1	17	17	0
1958	6650	1	3880	-1	18	0	18
1959	2260	1	4050	1	19	19	0
1960	5740	1	4220	-1	20	0	20
1961	5240	1	4270	-1	21	0	21
1962	3140	1	4310	1	22	22	0
1963	1500	1	4370	-1	23	0	23
1964	1700	-1	4600	-1	24	0	24
1965	2600	-1	5240	1	25	25	0
1966	1980	-1	5730	-1	26	0	26
1967	2880	-1	5740	1	27	27	0
1968	13100	-1	5980	1	28	28	0
1969	4220	-1	6150	-1	29	0	29
1970	8520	-1	6650	1	30	30	0
1971	8610	-1	6670	-1	31	0	31
1972	7240	-1	6780	1	32	32	0
1973	11500	-1	7240	-1	33	0	33
1974	9730	-1	8520	-1	34	0	34
1975	6150	-1	8610	-1	35	0	35
1976	6670	-1	9000	-1	36	0	36
1977	9000	-1	9520	-1	37	0	37
1978	11800	-1	9690	1	38	38	0
1979	9520	-1	9730	-1	39	0	39
1980	12000	-1	9790	1	40	40	0
1981	4600	-1	10100	-1	41	0	41

year	flow	ordered	flow	group #	rank	group 1	group -1
1982	3880	-1	11500	-1	42	0	42
1983	10100	-1	11800	-1	43	0	43
1984	25400	-1	12000	-1	44	0	44
1985	2710	-1	12000	-1	45	0	45
1986	4270	-1	12500	-1	46	0	46
1987	12000	-1	12900	1	47	47	0
1988	3260	-1	13100	1	48	48	0
1989	12500	-1	13100	-1	49	0	49
1990	5730	-1	15800	1	50	50	0
1991	4370	-1	25400	-1	51	0	51
						sum ranks	
						510	816

Mann-whitney U statistic

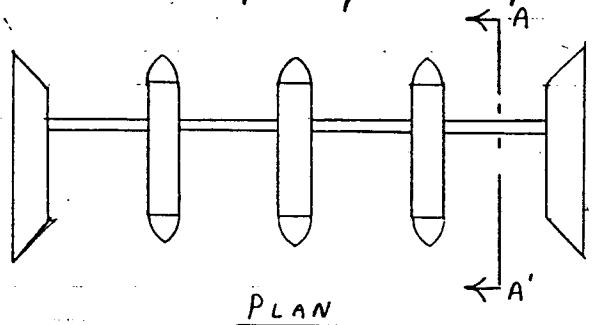
Ua	**210**
Ub	**438**
z	**-2.1513**

Attachment 5

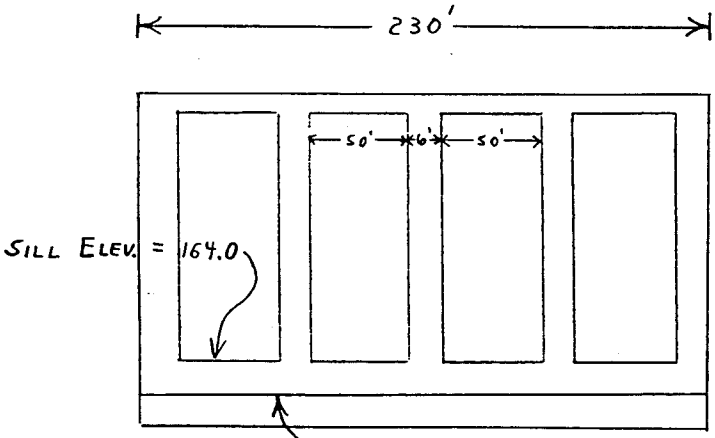
Sample Riprap Calculations

COMPUTATION SHEET			Page 1 of 4
Subject Pequannock Weir RipRap		Project Passaic River FRP Phase II GDM	
Computed by R.A.	Date 3/15/95	Checked by	Date

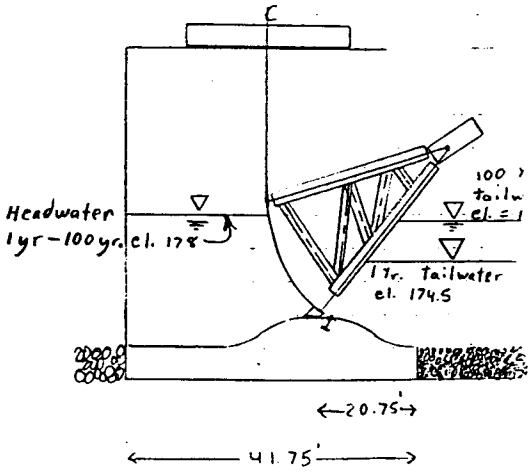
Pequannock Weir
Rip Rap Computations



PLAN



ELEVATION



SECTION AA'

NANY FORM 229 Apr 79

CORPS OF ENGINEERS, U.S. ARMY PASSAIC RIVER DIVISION	COMPUTATION SHEET	PAGE 2 OF 4 PAGES
		DATES 3/23
CENAN-PR-T NEW YORK DISTRICT	SUBJECT: Pequannock Weir Rip Rap	
COMPUTED BY: AP	COMPUTATION:	NUMBER
CHECKED BY: RLA		

Approach Channel

Q_{100} = 28,815 cfs

Approach Depth = 178' - $Apcon_{161}$ = d = 17'

Approach Area = width x d = 230' x 17' = 3,910 sf

Approach Velocity = Q/A = $\frac{28,815}{3,910}$ = 7.37 fps

Radius of Bend in Channel = 900' Top Width ≈ 275' ∠ ≈ 30° R/W = 3.27

Bottom Width ≈ 230' BW/d = 14

EM 1110-2-1601 Plate B-33 ⇒ $\frac{V_{ss}}{V_{avg}}$ = 1.08 ⇒ V_{ss} = 7.37 x 1.08

V_{ss} = 7.96 fps

d = 17' V_{ss} = 7.96 fps Plate B-37 ⇒ D_{30} ≈ 0.2'

Table 3-1 page 3-3 with γ_{stone} = 165 #/ft³ ⇒ Riprap Layer Thickness < 9"

Use 50' of 12" Riprap layer thickness for Approach

Exit Channel

Q_{100} = 28,815

Exit Depth = 177.5' - 161' = 16.5'

Exit Width = 4 openings at 50' wide = 200'

Exit Area = 200' x 16.5' = 3,300 ft²

Exit Velocity = $\frac{28,815}{3,300}$ = 8.73 fps

Radius of Bend = 500' Top Width ≈ 350' R/W = 1.4 ∠ ≈ 60'

Bottom Width = 230' d = 16.5' BW/d = 14

CORPS OF ENGINEERS, U.S. ARMY
PASSAIC RIVER DIVISION

CENAN-PR-T NEW YORK DISTRICT

COMPUTATION SHEET

PAGE 3 OF 4 PAGES

DATES 5/23

SUBJECT: Pequannock Weir Riprap

COMPUTED BY: AP

CHECKED BY: RLS

COMPUTATION:

NUMBER

Method #1

Plate B-33 $\Rightarrow \frac{V_{ss}}{V_{avg}} \approx 1.2$ $\therefore V_{ss} = 8.73 \times 1.2 = 10.5$

Plate B-37 $\Rightarrow D_{ss} \approx 0.42$ Table 3-1 $\Rightarrow$ Layer Thickness $\leq 12"$

Method #2 Riprap downstream of a hydraulic structure

Plate B-29 with $V = 8.73$ fps & $\gamma_{stone} = 165$ #/ft^3

$\therefore W_{50} \approx 77$ lbs. Table 3-1 $\Rightarrow$ Layer thickness = 21"

Worst Case One gate is accidently opened completely

Say upstream water surface elevation = 178.0 (1 Yr. Max)

downstream water surface elev. = 174.5 (1 Yr)

Gate Sill = 164.0 $H = 178.0 - 164.0 = 14'$ say $Q = CLH^{3/2} = 3.3(50)(14)^{3/2}$

$= 8,643$ cfs

Assume Flow spreads from 50' width to 55'

Area = width $\times$ (tailwater - invert) = $55' \times (174.5' - 161) = 742.5$ ft^2

Velocity = $8643/742.5 = 11.6$ fps $\gamma_{Stone} = 165$ #/ft^3

CORPS OF ENGINEERS, U.S. ARMY PASSAIC RIVER DIVISION CENAN-PR-T NEW YORK DISTRICT	COMPUTATION SHEET	PAGE 4 OF 4 PAGES DATES 3/23
	SUBJECT: Pequannock Weir Rip Rap	
COMPUTED BY: CP CHECKED BY: RZA	COMPUTATION:	NUMBER

Plate B-29 $\Rightarrow$ $W_{50_{max}} = 450$ #

Table 3-1 $\Rightarrow$ Layer Thickness = 36"

EM 1110-2-1605 page 5-30

Layer #1 Length = $3 \times d_2 = 3 \times 13.5 = 40.5'$

Layer #2 = .8 x Layer #1 = 29" length = 40.5'

Layer #3 = .6 x Layer #1 = 22" length = $2 \times d_2 = 27'$

Layer #4 = .4 x Layer #1 = 14" length = 27'

Total length = 135'

Use : 36" thick layer for 50' and a 24" layer for 50'

and a 12" layer for 50'

This should be sufficient for an unlikely event.

PROJECT: WANAQUE RIVER Reach # 8 DATE: May-95

Riprap computations are based upon the new methodology presented in EM 1110-2-1601, 1 July 1991, revised 30 June, 1994

Governing Equation: $d30(min) = Sf * Cs * Cv * Ct * D * [(\gamma w/(\gamma s - \gamma w))]^{0.5} * V/(K1 * g * D)^{0.5}]^{2.5}$

where:
- d30= stone size of which 30% is finer by weight
- Sf = Factor of Safety, Usually = 1.1
- Cs = Stability Coefficient, = .30 for angular stone, .375 for rounded stone
- Cv = Vertical Velocity Distribution Coefficient
  - = 1.0 for Straight Sections, or 1.283 - .2*LOG(R/W) For Outside of Bends
- Ct = Thickness Coefficient = 1.44-.56*TI+.14*TI^2
- D = Local Depth of Flow (ft). For sideslope riprap, uses depth 20% up from toe
- V = Local Average Channel Velocity, Usually = Vss (ft/s)
- γw = Specific Weight of Water = 62.4 lbs/ft3
- γs = Specific Weight of Stone, Usually = 165 lbs/ft3
- g = Gravitational Constant = 32.2 ft/s2
- K1 = (1.25-1.811/SS+3.343/SS^2)^-.5, more conservative than Carter, Carlson and Lane Equation

Procedure for Spreadsheet Computations:
- Input Sf, γs, Cs and TI applicable to all sections
- Input Station, Vavg, D, R, WS, B, A, SS and Channel Type for each section

where:
- Station = Cross-section location label
- Vavg = Average Channel Velocity = VCH from HEC2
- R = Centerline Radius of Channel Bend
- WS = Water Surface Width, Channel Only
- B = Bottom Width of Channel
- A = Angle of Bend, in Degrees. Use 0 for Straight Sections
- SS = SideSlope of Channel
- Channel Type = T for Trapezoidal or N for Natural
- TI = Thickness Factor = Layer Thickness / d100(max)
- Sf, γs, Cs and D defined above

Copy equations, including interpolation tables, down as needed for each section

Note: Layer thickness assumes the Standard Gradation, where d100(max) = 2.054 * d30(min)

Sf = 1.1 γs = 165 Cs = 0.3 TI = 1 => Ct = 1

Station	Vavg (fps)	D (ft)	R (ft)	WS (ft)	B (ft)	A (deg)	SS (ft/ft)	Cha/R/WI Type	B/D	Vss/Vavg	Vss (fps)	Sf*Cv*Cs*Ct*D	K1	V/(K1*g*D)^.5 ($\gamma w/(\gamma s-\gamma w)$)^0.5	OUTSIDE BEND d30(MIN) (ft)	LAYER THICK COMP (in)	LAYER THICK USE (in)	INSIDE BEND d30(MIN) (ft)	LAYER THICK COMP. (in)	LAYER THICK USE (in)	BOTTOM d30(MIN) (ft)	LAYER THICK COMP (in)	LAYER THICK USE (in)
92+80	8.05	11.9	200	100	40	120	2	N 2	8.36	1.863	7.99	1.22	3.65	0.79	0.88	0.467	0.842	8.4	9	0.260	8.9	9	0.274	8.8	9
90+80	4.88	10.7	200	100	40	120	2	N 2	8.75	1.863	7.72	1.22	3.44	0.79	0.88	0.466	0.324	8.0	9	0.255	8.8	9	0.269	8.4	Not Req'd
89+20	8.28	11.4	200	100	40	120	2	N 2	8.50	1.863	8.57	1.22	3.69	0.79	0.88	0.521	0.368	9.6	12	0.316	7.8	9	0.311	7.7	9
89+30	7.34	10.4	200	100	40	120	2	N 2	8.66	1.863	11.62	1.22	3.94	0.79	0.88	0.760	0.905	22.5	24	0.740	16.2	18	0.725	17.9	18
89+00	7.34	10.4	200	100	40	120	2	N 2	8.66	1.863	11.62	1.22	3.94	0.79	0.88	0.761	0.906	22.5	24	0.741	16.3	18	0.726	17.9	18
89+90	7.23	11.1	999	100	40	0	3	T 9.9	8.60	1.000	7.23	1.08	3.18	0.79	0.99	0.431	0.207	5.1	Not Req'd	0.192	4.7	Not Req'd	0.273	6.7	9
82+70	5.49	13.4	999	150	80	0	3	T 8.6	8.73	1.000	5.49	1.12	3.96	0.79	0.99	0.297	0.102	2.5	Not Req'd	0.092	2.3	Not Req'd	0.136	3.3	Not Req'd
81+40	6.10	14.5	999	120	70	0	3	T 8.3	4.82	1.000	6.10	1.10	4.21	0.79	0.99	0.318	0.129	3.2	Not Req'd	0.117	2.9	Not Req'd	0.169	4.2	Not Req'd
80+80	6.07	13.1	360	175	70	70	3	T 2.1	5.34	1.281	7.65	1.22	4.21	0.79	0.99	0.419	0.257	6.5	Not Req'd	0.212	5.2	Not Req'd	0.239	5.9	Not Req'd
80+20	6.17	12.9	360	160	70	70	3	T 2.3	5.42	1.252	7.72	1.21	4.12	0.79	0.99	0.426	0.262	6.6	Not Req'd	0.217	5.4	Not Req'd	0.244	6.0	Not Req'd
78+75	4.80	13.7	200	100	40	120	3	T 2	2.92	1.446	6.93	1.22	4.42	0.79	0.99	0.372	0.200	4.9	Not Req'd	0.164	4.0	Not Req'd	0.186	4.6	Not Req'd
78+95	6.43	13.5	999	200	60	0	3	T 8.8	8.94	1.000	6.43	1.17	4.17	0.79	0.99	0.294	0.105	2.6	Not Req'd	0.089	2.2	Not Req'd	0.139	3.4	Not Req'd
72+90	6.67	10.7	999	130	60	0	3	T 7.6	8.60	1.000	6.67	1.11	3.13	0.79	0.99	0.525	0.356	8.5	9	0.304	7.5	9	0.443	10.9	12
72+30	6.89	12.8	999	140	65	0	3	T 7.1	8.07	1.000	6.89	1.11	3.79	0.79	0.99	0.365	0.163	4.0	Not Req'd	0.146	3.6	Not Req'd	0.215	5.3	Not Req'd
69+20	9.12	11.9	999	120	70	0	3	T 8.5	8.67	1.000	9.12	1.10	3.46	0.79	0.99	0.824	0.569	9.1	9	0.336	8.3	9	0.466	12.0	12
68+85	7.44	12.0	999	100	70	0	3	T 9.9	8.62	1.000	7.44	1.08	3.44	0.79	0.99	0.426	0.219	5.4	Not Req'd	0.202	5.0	Not Req'd	0.266	7.1	9
67+90	6.89	11.9	999	110	70	0	3	T 9.0	8.69	1.000	6.89	1.09	3.42	0.79	0.99	0.366	0.162	4.6	Not Req'd	0.167	4.1	Not Req'd	0.240	6.9	Not Req'd
67+80	8.20	10.8	999	110	70	0	3	T 9.0	8.60	1.000	8.20	1.09	3.10	0.79	0.99	0.466	0.256	7.1	9	0.264	6.5	9	0.360	8.4	9
66+75	8.98	10.0	999	120	70	0	3	T 8.3	8.67	1.000	8.98	1.10	2.91	0.79	0.99	0.562	0.371	9.1	9	0.336	8.3	9	0.469	12.1	12
62+30	8.73	10.1	90	300	60	87	3	T 2	5.96	1.358	7.68	1.22	3.25	0.79	0.99	0.460	0.279	6.9	9	0.228	5.6	Not Req'd	0.259	6.4	Not Req'd
56+20	4.84	9.9	100	120	100	110	3	T 2	10.07	1.294	6.67	1.22	5.20	0.79	0.99	0.370	0.143	3.5	Not Req'd	0.117	2.9	Not Req'd	0.133	3.3	Not Req'd
53°15	8.43	9.4	999	100	80	0,	3	T 9.9	8.54	1.000	8.43	1.08	2.69	0.79	0.99	0.546	0.317	7.8	9	0.293	7.2	9	0.416	10.3	12
52+70	8.37	10.2	999	100	80	0	3	T 9.9	4.90	1.000	8.37	1.08	2.92	0.79	0.99	0.520	0.306	7.5	9	0.282	7.0	9	0.403	9.9	12
52°15	13.63	7.7	999	100	40	0	3	T 9.9	8.16	1.000	13.63	1.08	2.22	0.79	0.99	0.972	1.108	27.3	30	1.023	25.2	30	1.460	36.0	36
51°35	8.95	8.5	999	85	80	0	3	T 11.	8.66	1.000	8.95	1.07	2.41	0.79	0.99	0.808	0.372	9.2	9	0.348	8.6	9	0.491	12.1	12
48°80	7.97	9.5	300	180	85	47	3	T 2	8.90	1.132	9.02	1.22	3.08	0.79	0.99	0.660	0.423	10.4	12	0.346	8.5	9	0.409	10.1	12
47+00	7.12	10.8	300	140	80	47	3	T 2.1	7.35	1.146	8.16	1.22	3.50	0.79	0.99	0.490	0.316	7.8	9	0.260	6.4	Not Req'd	0.296	7.3	9
45°25	8.62	11.3	999	255	70	0	3	T 8.9	8.20	1.000	8.62	1.16	3.47	0.79	0.99	0.344	0.129	3.2	Not Req'd	0.111	2.7	Not Req'd	0.170	4.2	Not Req'd
39°75	6.41	11.6	999	140	74	0	3	T 7.1	8.36	1.000	6.41	1.11	3.41	0.79	0.99	0.373	0.156	3.8	Not Req'd	0.140	3.4	Not Req'd	0.205	5.1	Not Req'd
37+05	6.54	11.8	190	145	80	70	3	T 2	6.80	1.241	8.12	1.22	3.60	0.79	0.99	0.469	0.308	7.6	9	0.232	5.8	Not Req'd	0.286	7.1	9
32+20	8.65	12.8	155	140	75	36	3	T 2	6.01	1.135	8.41	1.22	4.03	0.79	0.99	0.360	0.169	4.2	Not Req'd	0.139	3.4	Not Req'd	0.162	4.0	Not Req'd
31+85	8.16	12.5	999	135	74	0	3	T 7.4	8.93	1.000	8.16	1.11	3.65	0.79	0.99	0.348	0.136	3.4	Not Req'd	0.125	3.1	Not Req'd	0.162	4.0	Not Req'd
27°85	8.78	13.0	999	160	74	0	3	T 8.2	8.66	1.000	8.78	1.12	3.66	0.79	0.99	0.317	0.117	2.9	Not Req'd	0.104	2.6	Not Req'd	0.156	3.8	Not Req'd
26°85	8.36	13.2	999	150	75	0	3	T 8.6	8.67	1.000	8.36	1.12	3.91	0.79	0.99	0.292	0.097	2.4	Not Req'd	0.087	2.1	Not Req'd	0.128	3.1	Not Req'd
26°45	8.37	13.2	999	150	75	0	3	T 8.6	8.66	1.000	8.37	1.12	3.90	0.79	0.99	0.293	0.097	2.4	Not Req'd	0.087	2.1	Not Req'd	0.128	3.2	Not Req'd
26°15	8.36	13.2	999	150	75	0	3	T 8.2	8.66	1.000	8.36	1.12	3.91	0.79	0.99	0.294	0.099	2.4	Not Req'd	0.088	2.2	Not Req'd	0.130	3.2	Not Req'd
24°80	4.16	13.6	200	100	40	120	3	T 2	2.95	1.446	6.02	1.22	4.36	0.79	0.99	0.324	0.141	3.5	Not Req'd	0.116	2.8	Not Req'd	0.151	3.7	Not Req'd
23+60	4.87	13.6	200	100	40	120	3	T 2	2.94	1.446	7.04	1.22	4.39	0.79	0.99	0.379	0.206	5.1	Not Req'd	0.170	4.2	Not Req'd	0.193	4.8	Not Req'd
21+10	4.62	13.8	200	100	40	120	3	T 2	2.90	1.446	6.96	1.22	4.45	0.79	0.99	0.372	0.202	5.0	Not Req'd	0.166	4.1	Not Req'd	0.189	4.6	Not Req'd
13+80	5.11	14.4	200	100	40	120	3	T 2	2.78	1.446	7.38	1.22	4.64	0.79	0.99	0.386	0.231	5.7	Not Req'd	0.189	4.7	Not Req'd	0.215	5.3	Not Req'd
8+50	4.74	15.0	200	100	40	120	3	T 2	2.67	1.446	6.85	1.22	4.83	0.79	0.99	0.351	0.190	4.7	Not Req'd	0.155	3.8	Not Req'd	0.176	4.3	Not Req'd
2+55	4.66	15.6	200	100	40	120	3	T 2	2.56	1.446	6.62	1.22	5.05	0.79	0.99	0.332	0.172	4.2	Not Req'd	0.141	3.5	Not Req'd	0.160	3.9	Not Req'd