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NATIONAL DAM SAFETY PROGRAM. WOODCLIFF LAKE DAM (NJ00247). HACK--ETC(U)  
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HACKENSACK RIVER BASIN  
PASCACK BROOK, BERGEN COUNTY  
NEW JERSEY

LEVEL #

# WOODCLIFF LAKE DAM

PHASE I INSPECTION REPORT  
NATIONAL DAM SAFETY PROGRAM

NJ00247

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DEPARTMENT OF THE ARMY  
PHILADELPHIA DISTRICT, CORPS OF ENGINEERS  
CUSTOM HOUSE - 2D & CHESTNUT STREETS  
PHILADELPHIA, PENNSYLVANIA 19106

JUNE 1978

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20. ABSTRACT (Continue on reverse side if necessary and identify by block number) This report cites results of a technical investigation as to the dam's adequacy. The inspection and evaluation of the dam is as prescribed by the National Dam Inspection Act, Public Law 92-367. The technical investigation includes visual inspection, review of available design and construction records, and preliminary structural and hydraulic and hydrologic calculations, as applicable. An assessment of the dam's general condition is included in the report.		





DEPARTMENT OF THE ARMY  
PHILADELPHIA DISTRICT, CORPS OF ENGINEERS  
CUSTOM HOUSE-2 D & CHESTNUT STREETS  
PHILADELPHIA, PENNSYLVANIA 19106

IN REPLY REFER TO

NAPEN-D

01 JUL 1978

Honorable Brendan T. Byrne  
Governor of New Jersey  
Trenton, New Jersey 08621

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Dear Governor Byrne:

Inclosed is the Phase I Inspection Report for Woodcliff Lake Dam, Bergen County, New Jersey which has been prepared under authorization of the Dam Inspection Act, Public Law 92-367. A brief assessment of the dam's condition is given on the first pages of the report.

Based on visual inspection, available records, calculations, and past operational performance, Woodcliff Lake Dam is judged to be in fair condition. The dam's spillway is considered seriously inadequate as 32 percent of the Probable Maximum Flood (PMF) would overtop the dam. To insure adequacy of the structure, the following actions, as a minimum are recommended:

a. Hydrologic and hydraulic investigations and engineering studies should be completed within twelve months of the date of approval of this report to determine corrective action required to increase the capacity of the spillway. Construction of an improved spillway should commence in calendar year 1979. Due to the potential for overtopping of the dam, a detailed emergency operation, warning and evacuation system should be developed and placed into operation by the owner within the next three months.

b. Engineering studies should be completed within six months of the date of approval of this report to determine the engineering properties of the embankment materials, the location of the phreatic line and determine the stability of the embankment. Necessary remedial measures should be initiated within 6 months of completion of these studies. The seepage areas should be monitored regularly until corrective measures are adopted. To facilitate seepage monitoring, the left downstream embankment area to 50 feet beyond the toe of the dam should be cleared, regraded and reseeded as required.

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NAPEN-D

Honorable Brendan T. Byrne

c. Piezometers or observation wells should be installed in the embankment near the left abutment as part of the studies recommended in paragraph b. above.

d. The following items should be accomplished within one year from the date of approval of this report:

- (1) The erosion at the end of the spillway chute and in the downstream channel should be repaired and protected from future erosion. The downstream sheet pile cut-off should be repaired and an energy dissipation sill installed.
- (2) The downstream embankment should be cleared of trees and brush and a suitable vegetative cover established.
- (3) Burrowing animals should be destroyed and their holes backfilled.
- (4) The water level recorder in the gaging station should be repaired.
- (5) The owner should upgrade his Operation and Maintenance procedures by issuing an Operation and Maintenance Manual.

e. The low level outlet valving should be relocated to the upstream side of the dam's core wall. A study should be made to determine the best method to accomplish this relocation within one year from the date of approval of this report. The valve relocation should be completed within calendar year 1979.

A copy of the report is being furnished to Mr. Dirk C. Hofman, New Jersey Department of Environmental Protection, the designated State Office contact for this program. Within five days of the date of this letter, a copy will also be sent to Congressman Andrew Maguire of the Seventh District. Under the provisions of the Freedom of Information Act, the inspection report will be subject to release by this office, upon request, thirty days after the date of this letter.

Additional copies of this report may be obtained from the National Technical Information Services (NTIS), Springfield, Virginia 22161 at a reasonable cost. Please allow four to six weeks from the date of this letter for NTIS to have copies of the report available.

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Honorable Brendan T. Byrne

An important aspect of the Dam Safety Program will be the implementation of the recommendations made as a result of the inspection. We accordingly request that we be advised of proposed actions taken by the State to implement our recommendations.

Sincerely yours,

*Harry V. Dutchyshyn*

HARRY V. DUTCHYSHYN  
Colonel, Corps of Engineers  
District Engineer

1 Incl  
As stated

Cy furn:  
Mr. Dirk C. Hofman, P.E.  
Department of Environmental Protection

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PHASE I REPORT  
NATIONAL DAM SAFETY PROGRAM

Name of Dam: Woodcliff Lake Dam, I.D. NJ 00247  
State Located: New Jersey  
County Located: Bergen  
Stream: Pascack Brook  
Date of Inspection: May 2 and 6, 1978

Assessment of General Condition of Dam with Respect to Safety and  
Recommended Action with Degree of Urgency

The general safety of Woodcliff Lake Dam is considered questionable in view of its lack of spillway capacity to pass the PMF, and is capable of only passing a flood equal to 31 percent of the PMF without overtopping the dam. The dam stability cannot be assessed on the basis of the available engineering data. A seepage source on the left abutment could affect the stability adversely and should be studied. The low level outlet is valved downstream of the dam's core wall, which is considered a safety hazard.

On the positive side, the dam has been in service for 73 years, and has performed adequately, although modifications in the spillway were and are required. The stream channel downstream of the spillway chute channel has been damaged and should be regraded to assure the continued safety of the installation. Suggested remedial actions are listed Section 7 together with a suggested timetable for their completion.



*Robert Gershowitz, P.E.*  
Robert Gershowitz, P.E.

(CONTINUED)

Based on visual inspection, available records, calculations, and past operational performance, Woodcliff Lake Dam is judged to be in fair condition. The dam's spillway is considered seriously inadequate as 32 percent of the Probable Maximum Flood (PMF) would overtop the dam. To insure adequacy of the structure, the following actions, as a minimum are recommended:

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c. Piezometers or observation wells should be installed in the embankment near the left abutment as part of the studies recommended in paragraph b. above.

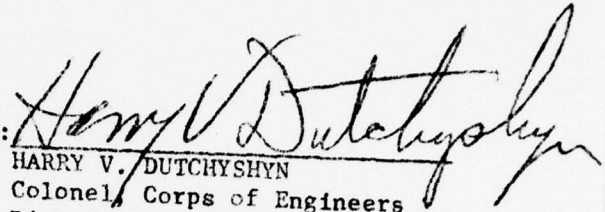
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APPROVED:

  
HARRY V. DUTCHYSHYN  
Colonel, Corps of Engineers  
District Engineer

DATE:

31 July 1978



WOODCLIFF LAKE DAM  
EMBANKMENT AND LOW LEVEL OUTLET-SPILLWAY  
IS OUT OF VIEW ON PICTURE'S LEFT SIDE

**HACKENSACK RIVER BASIN  
WOODCLIFF LAKE DAM  
BERGEN COUNTY, NEW JERSEY  
INVENTORY NUMBER: NJ00247**

**PHASE I INSPECTION REPORT  
NATIONAL DAM SAFETY PROGRAM**



**Prepared by  
HARRIS-ECI ASSOCIATES  
Woodbridge, New Jersey  
for  
DEPARTMENT OF THE ARMY  
PHILADELPHIA DISTRICT, CORPS OF ENGINEERS  
PHILADELPHIA, PENNSYLVANIA**

## TABLE OF CONTENTS

### ASSESSMENT OF GENERAL CONDITION OF DAM WITH RESPECT TO SAFETY AND RECOMMENDED ACTION WITH DEGREE OF URGENCY

		Page
SECTION 1	PROJECT INFORMATION	
	1.1 General	1
	1.2 Description of Project	1
	1.3 Pertinent Data	6
SECTION 2	ENGINEERING DATA	
	2.1 Design	9
	2.2 Construction	9
	2.3 Operation	9
	2.4 Evaluation	10
SECTION 3	VISUAL INSPECTION	
	3.1 Findings	11
	3.2 Evaluation	14
SECTION 4	OPERATION PROCEDURES	
	4.1 Procedures	16
	4.2 Maintenance of Dam	16
	4.3 Maintenance of Operating Facilities	17
	4.4 Description of any Warning System in Effect	17
	4.5 Evaluation	17
SECTION 5	HYDRAULIC/HYDROLOGIC	
	5.1 Evaluation of Features	18
SECTION 6	STRUCTURAL STABILITY	
	6.1 Evaluation of Structural Stability	21
SECTION 7	ASSESSMENT/REMEDIAL MEASURES	
	7.1 Dam Assessment	23
	7.2 Remedial Measures	26

TABLE OF CONTENTS

(Continued)

PLATES

REGIONAL VICINITY MAP	Drawing	1
PLANS AND DETAILS OF DAM	Drawings	2 to 8
GEOLOGIC MAP	Drawing	9

APPENDICES

APPENDIX A	CHECK LIST - VISUAL OBSERVATIONS	1
	CHECK LIST - ENGINEERING, CONSTRUCTION MAINTENANCE DATA	2-14
APPENDIX B	PHOTOGRAPHS	
APPENDIX C	SUMMARY OF ENGINEERING DATA	1
APPENDIX D	HYDROLOGIC COMPUTATIONS	1-39

PHASE I REPORT  
NATIONAL DAM SAFETY PROGRAM

WOODCLIFF LAKE DAM, I.D. NJ 00247

SECTION 1

1. PROJECT INFORMATION

1.1 General

a. Authority

The Dam Inspection Act, Public Law 92-367 of August 1972 authorizes the Secretary of the Army, through the Corps of Engineers to initiate a national program of dam inspections. Inspection for Woodcliff Lake Dam were carried out under Contract DACW61-78-C-0100 to the Department of the Army, Philadelphia District, Corps of Engineer by the engineering firm of Harris-ECI Associates of Woodbridge, New Jersey.

b. Purpose of Inspection

The purpose of the inspection and evaluation is to identify conditions which threaten the public safety and thus permit the correction of the conditions in a timely manner by the owners. The National Inventory of Dams will be updated by the data acquired during the inspection.

1.2 Description of Project

a. General Description of Dam and Appurtenances

Woodcliff Lake Dam is an earth embankment dam impounding the waters of Pascack Brook. The dam has an overall length of approximately 1,500 ft.

and a maximum depth above the stream bed of 38 ft. The top of the dam is 24 ft. wide and the upstream and downstream slopes are 2 horizontal on 1 vertical. The top of the dam is paved with a 2-lane bituminous roadway and is named Church Road. The upstream face is protected by cobble stone paving while the downstream face has a 12-inch surface layer of loam to support protective vegetation. In addition to the concrete core wall, headwater cut-off is achieved by interlocking sheet piling to an undetermined depth below the stripped ground surface. The sheet piling is embedded into a concrete core wall for a distance of approximately 4 feet, according to an available original plan prepared in 1903. The horizontal extent of the core wall and steel sheet piling is undetermined, and not shown on the plans uncovered in this phase. The dam abutments are formed of naturally higher ground. The spillway consists of an ungated low broadcrested concrete weir with a crest elevation at 94.33 feet east of the right abutment contact line. The spillway is surmounted by a four-span steel beam bridge resting on three intermediate concrete piers which carries Church Road across the spillway. The net spillway length is 79 feet and the spillway crest has a clear opening height between crest and bridge beam soffits of approximately 5 feet. A spillway chute connects the spillway crest to the Pascack Brook channel by turning through a 60-degree angle to the left, downstream of the crest. The total length of the spillway chute is approximately 650 feet measured from the center line of Church Road, and the chute drops a total of approximately 32 feet in elevation.

The spillway chute has been modified twice since its original construction. In 1937, a steel sheet pile cut-off was added to the bottom end of the chute and an earth access road was added alongside the right spillway Chute wall. In 1976, the spillway chute was repaved adding 5 inches of concrete dowelled into the existing slab. At that time,

the right stilling basin wall was heightened by 2.5 feet in the area where the chute turns through an angle of approximately 60 degrees, to prevent overtopping caused by supercritical flow standing waves. On November 8, 1977, during a severe rainstorm, the increased water velocity attained in the spillway chute by the smoother repaving caused considerable local damage on both sides of the channel downstream of the chute walls removing bank areas approximately 30-ft. wide by 100-ft. long, and redeposited the coarser fractions 75 ft. to 100 ft. downstream in the channel. The increased velocity also tore away the top edge of the steel sheet piling from the end of the chute slab concrete and bent it downstream.

The approach channel to the spillway crest consists of the upstream face of the dam embankment with its stone protection, flanked by the abutment walls of the spillway bridge.

The low level outlet consists of a 48-inch diameter cast iron line passing through the dam and controlled by two 36-inch diameter gate valves installed in series in a small gate house on the downstream side of the dam. The inlet end of the 48-inch line is located at the upstream toe of the slope of the embankment dam according to a 1903 original design drawing supplied by the owner. The outlet line has an invert at elevation 63.0 and the upstream embankment is protected in its vicinity by a small concrete apron slab with head and wingwalls. There are no trash rack provisions. On the discharge side, the low level outlet channel utilizes in part the original bed of Pascack Brook which has been protected by some stone armoring along its banks and invert.

The reservoir at Woodcliff Lake Dam impounds 2,750 acre feet of water from a tributary area of 19.4 square miles. The reservoir is traversed by a causeway that cuts the reservoir in half. The two parts of the causeway are connected by an arched culvert 25-foot wide by 16.8-foot high.

b. Location

Woodcliff Lake Dam is located on Pascack Brook, a tributary of the Hackensack River at Hillsdale, Bergen County, New Jersey. The nearest downstream community is Hillsdale itself and there are urbanized areas extending almost to the end of the spillway chute.

c. Size Classification

Woodcliff Lake Reservoir is classified as being "Intermediate" on the basis of its reservoir storage volume, which is less than 50,000 acre feet but more than 1,000 acre feet. It is classified as "Small" on the basis of its total height which is less than 40 feet. The larger of the two size determinations governs, and thus the dam is classified as "Intermediate" on size.

d. Hazard Classification

In the National Inventory of Dams, Woodcliff Lake Dam has been classified as having a High Hazard Potential on the basis that failure of the dam would cause excessive property damage to residences downstream, and could potentially cause more than a few deaths.

e. Ownership

Woodcliff Lake Dam is owned by the Hackensack Water Company, located in Weehawken, New Jersey.

f. Purpose of Dam

The purpose of the dam is to impound water for use in a water supply system operated by the Hackensack Water Company. The impounded water is released by means of the bottom outlet for subsequent use downstream by way of the natural channel of Pascack Brook, and is impounded again at Oradell Reservoir on the Hackensack River.

g. Design and Construction History

Drawings provided by the owner show that the dam was designed in 1903 by the engineering firm of Hering & Fuller, New York, New York. The dam was built in 1905 by the Fuller Construction Company using horsedrawn equipment according to the owner. The embankment shown on the plans is apparently unaltered, although the curved left abutment alignment shown on the 1903 plans was not carried out, and the dam was constructed on a straight centerline alignment throughout. The original spillway chute was modified twice since its construction, in 1937 and again in 1976, as has been described above. Plans for these modifications have been provided by the Owner and are appended as Drawings 5, 6 and 7. In 1976, plans were drawn up to install 12-inch high flash boards on the spillway crest. These flash boards were in use for a short time, but are not currently in use. No original computations are available for review and no inspection reports bearing on the construction have been uncovered.

The Hackensack Water Company in 1977 initiated an investigation to augment the spillway capacity of the dam and made a detailed topographic survey of the right abutment area as part of the study. Plans for providing auxiliary spillway capacity have not been finalized as of the date of this inspection report.

h. Normal Operational Procedures

The dam is used as an impounding reservoir designed to store the maximum possible amount of surface runoff from Pascack Brook. The reservoir levels are regulated in conjunction with the needs of Oradell Reservoir which impounds waters from Pascack Brook and the Hackensack River.

### 1.3 Pertinent Data

#### a. Drainage Areas

At dam site, the drainage areas are 19.4 square miles.

#### b. Discharge at Damsite

Maximum known flood at damsite:	Est. at 2,740 cfs at spillway on 11/8/77. Reservoir pool est. at Elev. 98.7
Warm water outlet at pool elevation:	NA
Diversion tunnel low pool outlet at pool elevation:	NA
Diversion tunnel outlet at pool elevation:	NA
Gated spillway capacity at pool elevation:	NA
Gated spillway capacity at maximum pool elevation:	NA
Ungated spillway capacity at maximum pool elevation:	1,650 cfs
Total spillway capacity at maximum pool elevation:	1,650 cfs

#### c. Elevation (feet above MSL)

Top of dam:	100
Maximum pool-design surcharge:	98.03
Full flood control pool:	NA
Normal pool:	94.33
Spillway crest (gated):	NA
Upstream portal invert diversion tunnel:	NA
Downstream portal invert diversion tunnel:	NA
Streambed at centerline of dam:	62
Maximum tailwater:	Unknown

d. Reservoir

Length of maximum pool:	6,000 feet
Length of normal pool:	5,000 feet
Length of flood control pool:	NA

e. Storage (acre-feet)

Normal pool:	2730 AF
Flood control pool:	NA
Design Surcharge:	3240 AF
Top of dam:	3640 AF

f. Reservoir Surface (acres)

Top of dam:	217 A
Maximum pool:	200 A
Flood-control pool:	NA
Recreation pool:	NA
Spillway crest:	169 A

g. Dam

Type:	Earth embankment with center concrete core wall
Length:	1,500 feet
Height:	38 feet
Top Width:	24 feet
Side Slopes:	2 horizontal on 1 vertical
Zoning:	None
Impervious core:	Concrete core wall
Cut-off	Steel sheet pile, depth unknown
Grout curtain:	None

**h. Diversion and Regulating Tunnel**

Type: NA  
Length: NA  
Closure: NA  
Access: NA  
Regulating Facilities: NA

**i. Spillway**

Type: Broad crested ogee sill  
Length of weir: 79 feet net  
Crest elevation: 94.33 MLS  
Gates: None  
U/S channel: None  
D/S channel: Spillway chute channel 630-foot long by 38-foot wide

**j. Regulating Outlets**

Type: Cast iron low level outlet, 48-in.dia.  
Length: Approximately 160 feet  
Closure: Two 36-inch diameter slide gate valves; one motor operated, the other hand operated  
Access: Gate accessible and housed in a gate house downstream of embankment  
Regulating Facilities: Regulation by partial opening of gate valve

## SECTION 2

### 2. ENGINEERING DATA

#### 2.1 Design

Available original design drawings have been provided by the owner (Drawings 2, 3 and 4). Drawings are also available for modifications made in 1937 and 1976. No design computations, soil borings, soil tests or other geotechnical data is available to assess the stability of the embankment properly. No information is available on the depth of the steel sheet pile cut-off under the concrete core wall, or its horizontal extent into the abutments. No information is given on the original plans as to the extent of the concrete core wall into the abutments. Information in the files of the New Jersey Department of Environmental Protection list the steel sheet pile cut-off as being 65-foot deep, but this could not be substantiated by any other source of information uncovered.

#### 2.2 Construction

No records have been uncovered as to the construction history of the dam, except for a statement from the owner's representative that it was constructed by horsedrawn equipment.

#### 2.3 Operation

No documents have been uncovered as to operating rules pertaining to the regulation of the reservoir. Letters in the files of the New Jersey Department of Environmental Protection pertain to complaints about high water levels in the reservoir. The reservoir is simply operated to capture the maximum amount of water from the Pascack Brook watershed. Hydrograph of several storm events were submitted by the Owner for use in preparing this report.

## 2.4 Evaluation

### a. Availability

The availability of data is fair considering the age of the dam. Data needed to fully assess the safety of the dam includes:

1. Subsurface information at the dam site, including engineering properties and parameters.
2. Soil properties of the embankment.
3. Data on the phreatic line within the dam section at several cross section lines including the maximum section and at the seepage area in the left abutment area.

A check list of engineering construction and maintenance data is included in Appendix A.

### b. Adequacy

At present the engineering data available is not sufficient to draw a conclusion on the stability of the earth embankment. The additional data listed above should be acquired for further assessment and analysis.

### c. Validity

Although the original design drawings dating back to 1903 differ in some respects from the dam as currently constructed, such as in the details of the dam axis, at the spillway bridge and at the low level outlet line, there is no reason to believe that the bulk of information shown on these drawing is not substantially correct and usable. The later drawings relating to modifications are considered accurate.

## SECTION 3

### 3. VISUAL INSPECTION

#### 3.1 Findings

##### a. General

The visual inspection made of Woodcliff Lake Dam revealed that the dam and appurtenances were in serviceable condition but a program of further investigations and remedial work is required to assure its continued safety.

##### b. Dam

Construction plans indicate an earth embankment with a concrete core wall and sheet pile cut-off. Observations indicate that the embankment, at least downstream, is a well-graded sand and gravel. It is reasonable to assume, based on visual observations, that the foundation consists of the same type of cohesionless material to bedrock. The crest of the dam is a two-lane asphalt paved road. Longitudinal cracking in the pavement is not believed related to embankment movement. No significant deviations in vertical or horizontal alignment were apparent.

The embankment slopes are two on one and there are no signs of past or present downstream slope instability. Upstream slopes are protected by riprap and no problems were apparent above the water line. Downstream slopes were overgrown with brush and small trees. Several holes dug by burrowing animals were observed.

Seepage was observed in four locations just downstream of the embankment toe near the left abutment. The seepage combines to form a 2-foot wide rivulet carrying 5 to 10 gpm of clear water. At the time of the inspection, vegetation in the area left of the gate control house was indigenous to swampy areas. One observation well was located at the toe of the downstream embankment approximately 75 feet left of the gate control house. The level of water in the well was 1 ft. below ground surface.

c. Appurtenant Structures

● Spillway Chute Channel

No seepage was visible in the area of the concrete spillway chute channel. Severe erosion, however, was observed at the end of the spillway chute channel which occurred as a result of the storm on November 8, 1977. According to the owner's engineering representative, repaving of the spillway chute slab in 1976 increased the discharge velocity. During the storm, the increased kinetic power of the discharging water, severely eroded the downstream channel banks for approximately 100 feet in length and 30 feet in width, cutting away the backfill behind the spillway chute wall for a distance of 15 feet upstream of the end of chute. The discharging water also tore away the steel sheet piling from the edge of the spillway chute concrete and displaced and bent it downstream. The spillway crest concrete and the spillway chute floor concrete is in acceptable to good condition. There is no cracking of the overlay concrete and all construction joints are in alignment. The chute walls are in acceptable condition, showing some minor surface deterioration. The jointing in walls is in good alignment.

● Spillway Bridge and Piers

The concrete piers are in acceptable condition with no significant cracking or deterioration observed. The spillway beams have been strengthened by the addition of cantilever support beams at the piers and brackets at the two abutment walls. The brackets and beams impede the clear spillway area somewhat. Additional soffit encasement concrete was observed on the upstream bridge basin of the left spillway channel opening. The bridge itself is in acceptable condition for light vehicular traffic.

● Low Level Outlet

The dam's low level outlet consists of a single 48-inch diameter cast iron pipe at invert elevation 63.1 MSL. The line is controlled by two

36-inch diameter gate valves and their associated 48-in. by 36-in. reducers. These two valves are installed back to back with the valve stems in the horizontal position. The valves are located in a small valve house at the toe of the embankment. The capacity of the low level outlet is estimated at 530 cfs at a normal pool elevation of 94.33.

The upstream valve is manually operated and is normally left in the open position. The downstream valve is motor operated and is used to control the flow through the line, providing water for use downstream and serving as an emergency drawdown for the reservoir. Operation of the valve could become a problem at higher tailwater elevations.

Both valves appeared to be in good condition. The packing glands were not leaking, and the 90 degree bevel gear reduction drives were well lubricated and appeared to be well maintained. The owner's representative stated that both valves are functional and are operated when necessary. At the time of the inspection, the motor operated valve had recently been opened to 25 percent (9 inches) opening, in order to provide downstream water for Pascack Brook, since the reservoir had fallen below the crest of the spillway. No records are available concerning the last complete inspection of the 48-inch line.

#### d. Reservoir Area

The major portion of the reservoir rim is gently sloped. No indications of instability were readily apparent in the remaining portions of the reservoir rim.

The reservoir was observed to feed into a small, shallow pond approximately 750 feet north of the dam axis from the left abutment. Total seepage was estimated to be about 5 to 10 gpm at the time of the inspection. Seepage water appeared to be clear and free of fine soil particles. Approximately 200 feet of soil lies between the pond and the reservoir.

Upon examining the surficial evidence, it is believed that the reservoir abutments and possibly the channel section are underlain by flacio-fluvial gravel and sands that mantle interbedded red sandstone and shale (Brunswick formation). Hummocky topography in gravel and sand deposits were observed downstream of the right abutment. A geologic map of the reservoir is appended to the end of this report as Plate 8.

e. Downstream Channel

The downstream channel has been severely eroded during the storm of November 8, 1977 as described above. The coarser scoured materials from the banks and from behind the spillway chute walls has been redeposited in Pascack Brook Channel some 75 feet downstream of the end of spillway channel. According to the Owner's engineering representative, the scour downstream of the end of the spillway chute channel is not deep.

3.2 Evaluation

The visual inspection showed that the following items could affect the safety of the dam.

1. Brush and trees covering the downstream slope could cause problems from the root system.
2. The immediate area of the downstream embankment to the left of the low level outlet gate house is overgrown with trees making convenient inspection of seepage difficult. Vegetation also exists on the right abutment, obscuring examination and surveillance efforts.

Seepage in this area is thought to be significant. Seepage may be occurring through cracks or joints in the corewall or through or under the sheet piling.

3. Burrowing animals have established habitations in the downstream embankment.
4. While the erosion at the end of the spillway chute channel does not pose an immediate threat to the stability of the dam at this time, due to its distance from the embankment, the situation should nevertheless be speedily corrected.
5. The low level outlet passes through the embankment dam before it is valved at the downstream side, This design feature is considered a safety hazard.

The visual inspection check list is included in Appendix A.

Photographs taken during the site inspection are included in Appendix B.

## SECTION 4

### 4. OPERATIONAL PROCEDURES

#### 4.1 Procedures

Woodcliff Lake Dam is used to impound water on Pascack Brook for water supply uses. The strategy is to impound and store the maximum amount of water. As a result, an attempt is made to keep the reservoir as full as possible and to release reservoir waters in accordance with water supply needs at Oradell Dam on the Hackensack River, the main stream to which the Pascack Brook is tributary. The reservoir releases for water supply needs are routed downstream by way of the natural channel of Pascack Brook. During heavy rains, the low level outlet is opened to control the water level in the reservoir.

#### 4.2 Maintenance of Dam

The dam has been maintained by making periodic inspections of its facilities. The reservoir is visited every hour throughout the year by a security patrol to control unauthorized entry into the fenced-off reservoir and spillway areas at the dam. Water levels are normally read daily, and on an hourly basis when rain exceed one inch within 24 hours. A precipitation gage is maintained and read in the reservoir area. The security patrol makes note of any unusual occurrences at the dam site and reports in to the Manager of Operations. There is an alarm on the low level outlet gate house tied into the security patrol system to provide an alert in case of attempted tampering with the gate valve controls. The dam is visited once a year by the Hackensack Water Company's engineering staff for an on-the-ground assessment of its facilities. The Chief Engineer keeps a written memorandum of the inspection in his files.

#### 4.3 Maintenance of Operating Facilities

The low level outlet gate valve opening is adjusted in accordance with the water needs at Oradell Dam and the minimum discharges desired in Pascack Brook in the reach between the dam and the confluence with the Hackensack River. Maintenance of the gate valves is made on a periodic basis in conjunction with visits to adjust the valve opening.

#### 4.4 Description of Warning System in Effect

There is no fixed procedure for warning downstream residents in case of an emergency. The security patrol is in radio contact with the Hackensack Water Company headquarters and could respond to emergency situations in a reasonably short time.

#### 4.5 Evaluation

Surveillance and maintenance is in the hands of an experienced staff working for a major private water supply company. The Hackensack Water Company has its own engineering staff and has shown an active interest in maintaining its facilities in a serviceable condition as is evidenced by its recent repairs to the spillway chute channel slab. The company should formalize its maintenance and inspection program for proper documentation, in line with the increased public interest in dam safety. A program for the control of vegetation growing on the downstream slope of the dam and for 35 to 50 feet downstream of the toe of slope should be initiated. A program for the control and elimination of burrowing animals inhabiting the downstream embankment slope should also be initiated.

## SECTION 5

### 5. HYDRAULIC/HYDROLOGIC

#### 5.1 Evaluation of Features

##### a. Design Data

The Woodcliff Lake Dam impounds about 2,750 acre-feet of water in the reservoir at normal storage capacity. The watershed area above the Woodcliff Lake Dam is 19.4 square miles. The orientation of the watershed is from north to south with the lower one-third of the drainage area lying in the State of New Jersey and the upper two-thirds area in the State of New York. Length of Pascack Brook from the headwater of the Woodcliff Lake reservoir to the watershed divide is approximately 10.2 miles. A drainage map of the watershed of the Woodcliff Lake Dam is given in Plate 1, Appendix D.

The evaluation of the hydraulic and hydrologic features of the Woodcliff Lake Dam was based on criteria set forth in the Corps "Guidelines", Section 4.3 and additional guidance provided by the Philadelphia District Corps of Engineers. The Probable Maximum Flood (PMF) was calculated from the Probable Maximum Precipitation (PMP) using Hydrometeorological Report #33 with standard reduction factors. The Snyder Method was used for deriving the unit hydrograph with  $C_t = 4.3$  and  $640 C_p = 530$ .

Initial infiltration loss rates were applied using SCS procedures to the Probable Maximum Storm rainfall to obtain rainfall excess. The rainfall excess was then applied to the unit hydrograph to obtain the PMF hydrograph, utilizing computer program HEC-1. The computed peak discharge of PMF and one half of the PMF are 13,805 cfs and 6,902 cfs respectively.

These inflow hydrographs were routed through the reservoir by the modified Puls method utilizing computer program HEC-1. The peak outflow discharges for the PMF and one half of PMF are 13,762 cfs and 6,885 cfs respectively. Both the PMF and one half of the PMF result in overtopping of the dam.

The stage-outflow relation for the spillway and the reservoir stage-capacity data were based upon information provided by the Hackensack Water Company. The spillway rating curve and the reservoir capacity curves are presented in Plates 2 and 3 of Appendix D respectively.

b. Experience Data

Records of daily reservoir stage level are maintained since the reservoir was in operation. The reservoir water level usually maintains at lower than elevation 94.33 except during floods. There are no records indicating that the reservoir water surface elevation went over the dam crest at any time.

c. Visual Observations

It was noted the automatic water level recorder located on the left shore of the reservoir was not functioning and all the water level readings were taken manually. There is no evidence of excessive sedimentation due to recent developments in the drainage basin which could cause a sudden increase in sediment load which may pose danger to the dam. Detrimental scour and severe erosion were observed at the end of the spillway discharge channel which occurred as a result of the storm on November 8, 1977.

d. Overtopping Potential

As indicated in item a., both the PMF and the one half of the PMF, when routed through the Woodcliff Lake reservoir, result in overtopping the dam. The PMF and one half PMF overtopped the dam by 1.10 feet and 0.3 feet respectively.

The spillway is capable of passing a flood equal to about 31 percent of the PMF without overtopping the dam. Since PMF is the Spillway Design Flood (SDF) for this dam according to the recommended "Guidelines" for Inspection of Dams by the Corps, the spillway capacity of the Woodcliff Lake Dam is considered inadequate.

e. Reservoir Drawdown

The reservoir drawdown below the spillway crest elevation 94.33 is accomplished by permitting discharge through the 48-inch outlet pipe with invert elevation 63.08. Assuming drawdown to the top of the pipe, elevation 67.0 results in a maximum head differential of 27.33 feet. Assuming a constant inflow of 39 cfs (2 cfs/acre), the drawdown can be accomplished in 5 days and 14 hours. Assuming no inflow into the reservoir, the drawdown time is reduced to 4 days and 12 hours.

## SECTION 6

### 6.1 STRUCTURAL STABILITY

#### a. Visual Observations

There are no signs of embankment sloughing, local slides or slumps on the downstream side. The upstream side of the embankment was almost completely under water and was not available for visual inspection. The leakage and seepage in the left abutment, described in Section 3.1 - b. have not been monitored by the owner and no information was uncovered concerning their age or flow rate.

The spillway chute slab exhibits no visual evidence of cracking, slide failure, undermining or misalignment, in spite of the washout caused by the storm of November 8, 1977 and described in Section 1.2.

#### b. Design and Construction Data

No design computations were uncovered during the report preparation phase. No embankment or foundation soil parameters are available for carrying out a conventional stability analysis on the embankment. No construction data or specifications relating to the degree of embankment compaction are available for use in the stability analysis.

A stability analysis was made as part of this report on the downstream slope assuming that the embankment material had the following properties: angle of internal friction  $\phi = 30$  degrees, unit weight of embankment 130 pounds per cubic feet, and that the embankment downstream of the core wall was dry. The analysis showed that the downstream slope was stable under these conditions. The slope would be unstable if the downstream embankment slope were subject to high phreatic water levels, such as could exist in the seeping left abutment section. The stability of the section at the right abutment was not checked for the location of the phreatic line. It is recommended that embankment and

subsoil engineering parameters be acquired together with phreatic levels at sections exhibiting seepage and at another section not exhibiting seepage so that stability computation can be carried out to verify the preliminary assumptions made.

c. Operating Records

No operating records are available relating to the stability of the dam. Water levels in the one observation well discovered at the site have not been recorded. According to the owner's engineering officials, the embankment has served satisfactorily since 1905, its construction date.

d. Post Construction Changes

There have been no post construction changes affecting the stability of the embankment.

e. Seismic Stability

In general, projects located in Seismic Zone 0, 1 and 2 may be assumed to present no hazard from earthquake, provided the static stability conditions are satisfactory and conventional safety margins exist.

## SECTION 7

### 7. ASSESSMENT / REMEDIAL MEASURES

#### 7.1 Dam Assessment

##### a. Safety

The dam has been inspected visually and a review has been made of the available engineering data. This assessment is subject to the limitations inherent in the visual inspection procedures stipulated by the Corps of Engineers for Phase I Report.

The spillway capacity has been found to be inadequate. The routed PMF will top the dam crest by 1.0 foot and the routed one half PMF will top the dam by 0.3 foot. Overtopping of the embankment will carry with it a high risk of total failure of the embankment by erosive action of the overtopping water.

The spillway capacity has been determined by Corps of Engineers screening criteria and should be determined by the owner using more accurate and sophisticated methods and procedures.

The following actions are recommended:

1. All brush and trees should be removed from the downstream slope to avoid problems which may develop from their roots. The embankment should then be seeded to develop a growth of grass for surface erosion protection.
2. An area should be cleared in the immediate area of the downstream embankment on both sides of the low level outlet gate house to facilitate inspection. The cleared area should extend 35 to 50 feet

downstream of the toe line of the embankment, regraded and reseeded as required. This area should be maintained under a cut vegetative cover. The seepage in this area should be channelized, measured and monitored, and correlated to rainfall and reservoir levels.

3. Burrowing animals should be destroyed and their holes backfilled as best as possible, wherever they exist in the downstream embankment or at the area below the toe line.
4. Piezometers or observation wells should be installed in the embankment near the left abutment to determine the paths of the seepage observed. Seepage may be occurring through cracks or joints in the corewall or through or under the sheet piling. Once the location of the phreatic surface in the embankment is determined, then effective corrective measures can be considered and undertaken. Piezometers should also be installed in the vicinity of the gate house where there are no signs of leakage to provide a comparison with an apparently satisfactory section.

No assessment of the safety of the embankment can be made at this time pending acquisition of embankment and foundation material engineering properties and determination of phreatic levels in the downstream part of the abutment.

5. While the erosion at the end of the spillway chute does not pose an immediate threat to the stability of the dam at this time, due to its distance from the embankment, the situation should nevertheless be speedily corrected. The banks should be restored to their original contours and protected by stone. Studies should be made to determine the best way to dissipate the increased kinetic energy of the spillway chute water caused by lower friction losses on the repaved chute slab. The eroded materials should be removed from the stream channel and the channel regraded as required to produce a smooth water level surface during high stream stages.
6. The low level outlet valving should be moved to the upstream side of the dam's core wall. A study should be undertaken to determine the best way to accomplish this work and the approved plan implemented.
7. While the pond to the north of the left abutment did not appear to be a threat to the safety of the reservoir at the time of the inspection, periodic inspections of the area are recommended, to verify that the situation with respect to seepage is stable.
8. The timely repair of the automatic water level recorder in the gaging station is recommended.

b. Adequacy of Information

No data on the properties of the embankment material exists for assessment of the stability of the dam. A program for the acquisition of this data is recommended together with similar data pertaining to the underlying soil formation. A program to determine the location of the phreatic line along the downstream half of the dam is also recommended.

c. Urgency

It is recommended that the investigations of the engineering properties of the embankment materials and the location of the phreatic line be completed within six months. The maintenance items listed in Section 3.2 should be completed within one calendar year. A report on the alternatives for remedying the left abutment seepage should be completed within one calendar year. A report on the possibility of adding auxiliary spillway capacity should be completed within one calendar year. A report on shifting the low level outlet valving to the upstream side of the core wall should be completed within 12 months, and the modification completed within 18 months.

d. Necessity for Additional Investigations

Based on the findings listed in Section 7.1 - a., it is recommended that further investigations be authorized.

7.2 Remedial Measures

a. Alternatives

The remedial actions summarized in Section 7.1 - a., are recommended for implementation.

The course of action to be taken in the matter of the seepage emanating from the downstream toe of embankment slope depends upon the analysis of phreatic line geometry. Possible alternatives include:

1. Lowering the phreatic line by means of drains.
2. Regrading the area at downstream of the embankment toe.
3. Adding berm materials.
4. No action.

Possible alternatives for increasing the spillway capacity of the dam include:

1. Development of an auxiliary spillway on the right abutment to bring the total spillway capacity to up to at least one half PMF and possibly higher.
2. Development of a new and possibly gated service spillway which combined with the present spillway would pass the PMF. The present service spillway would serve as an auxiliary spillway.
3. Regulation of the reservoir water surface to some specified level below the present weir crest to allow for routing of a higher inflow than presently possible with the pool at elevation 94.33.
4. Removal of bridge piers and construction of a new bridge.
5. Consideration should be given to the use of energy dissipation at the bottom of the spillway for all spillway augmentation schemes.
6. A combination of any of the three alternatives.

b. O & M Procedures

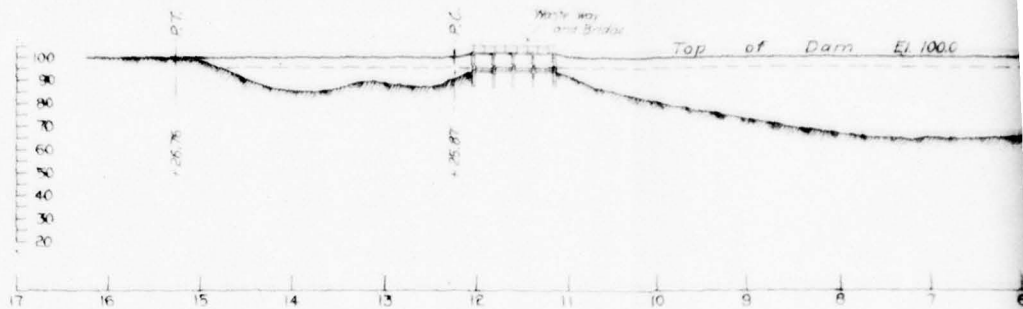
The owner should upgrade his O & M procedures by issuing a manual and check list for recommended procedures. The inspection and maintenance visits should be logged and documented. A communication channel should be maintained between the owner and civil authorities in the downstream community of Hillsdale in case of accident, high reservoir inflow conditions or a dam operating failure causing high water stages downstream.

PLATES

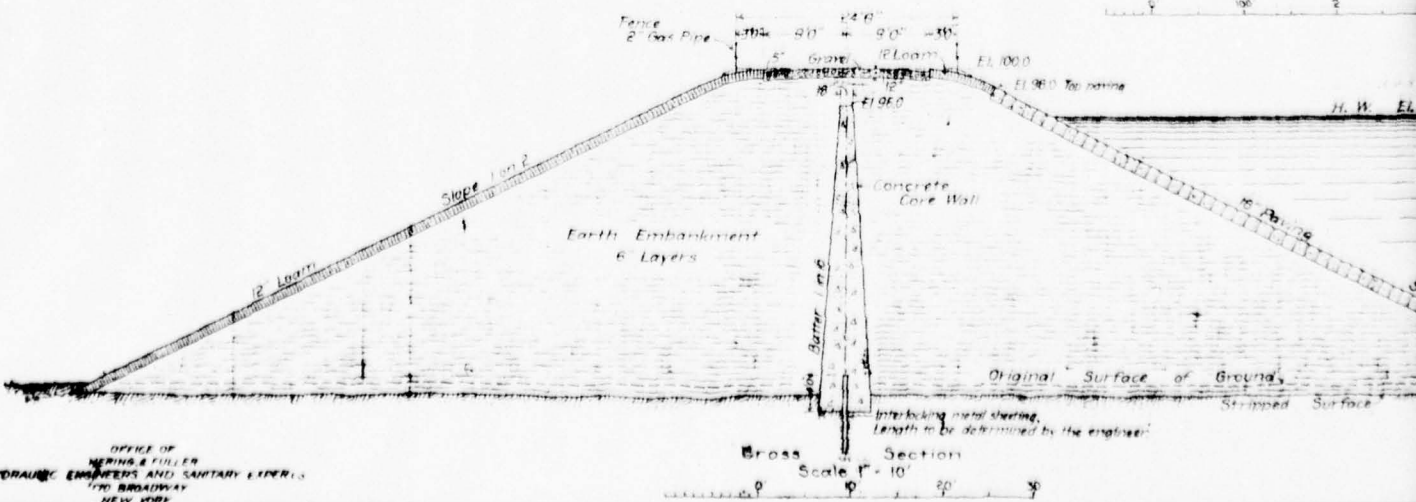
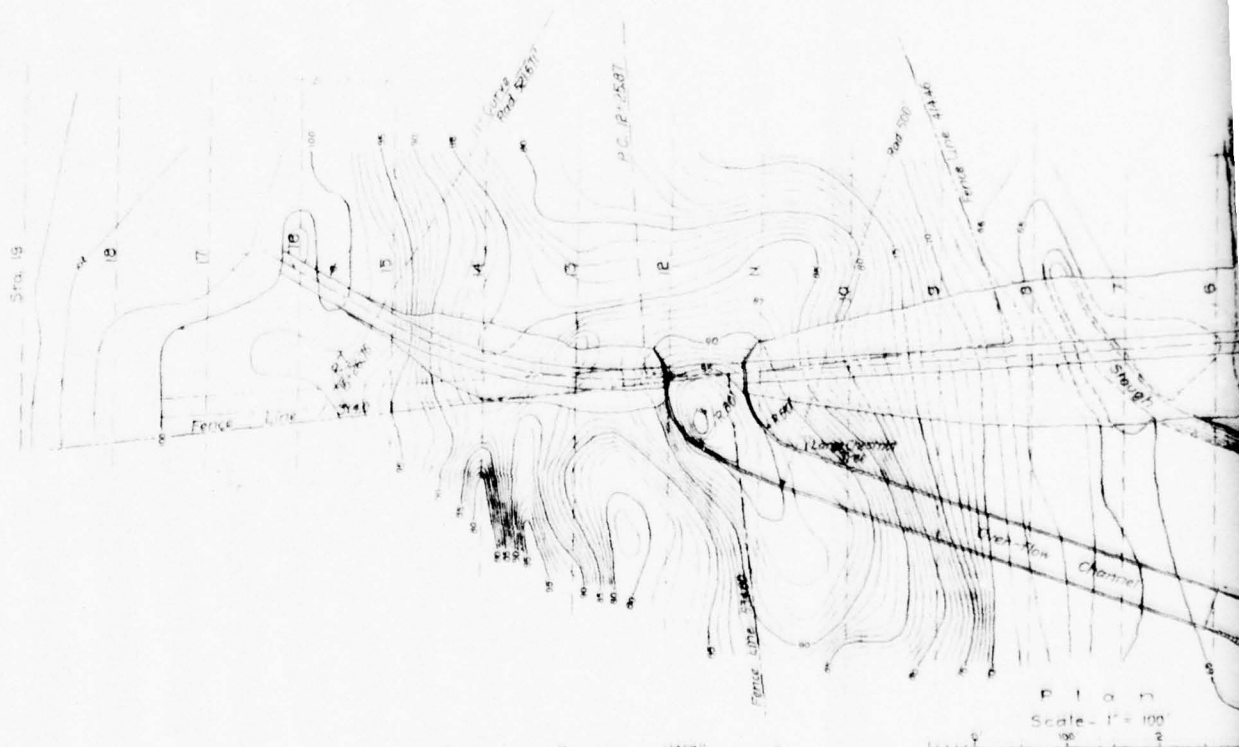


VICINITY MAP

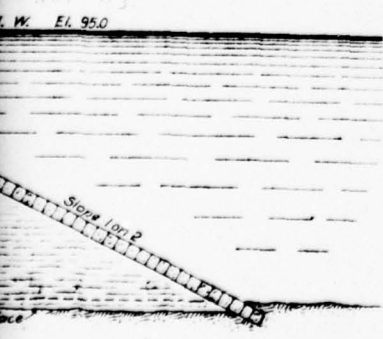
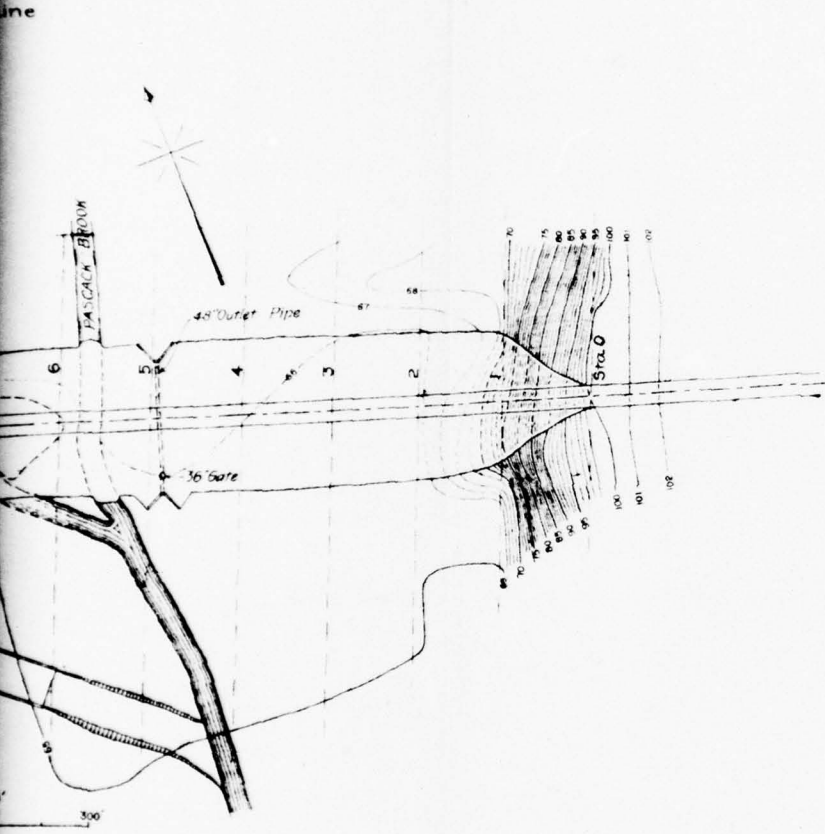
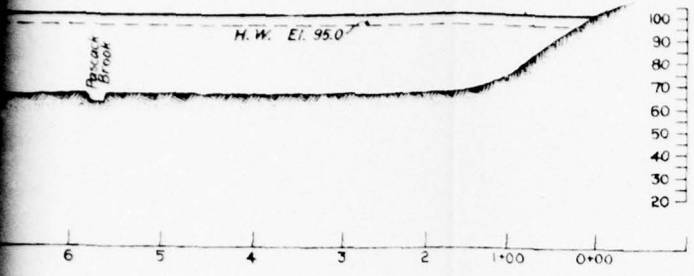
DWG. NO. 1



Profile on Center Line  
 Hor. Scale 1" = 100'  
 Vert. " 1" = 40'



OFFICE OF  
 MERRILL & FULLER  
 HYDRAULIC ENGINEERS AND SANITARY ENGINEERS  
 170 BROADWAY  
 NEW YORK



WOODCLIFF LAKE  
DWG. NO. 2

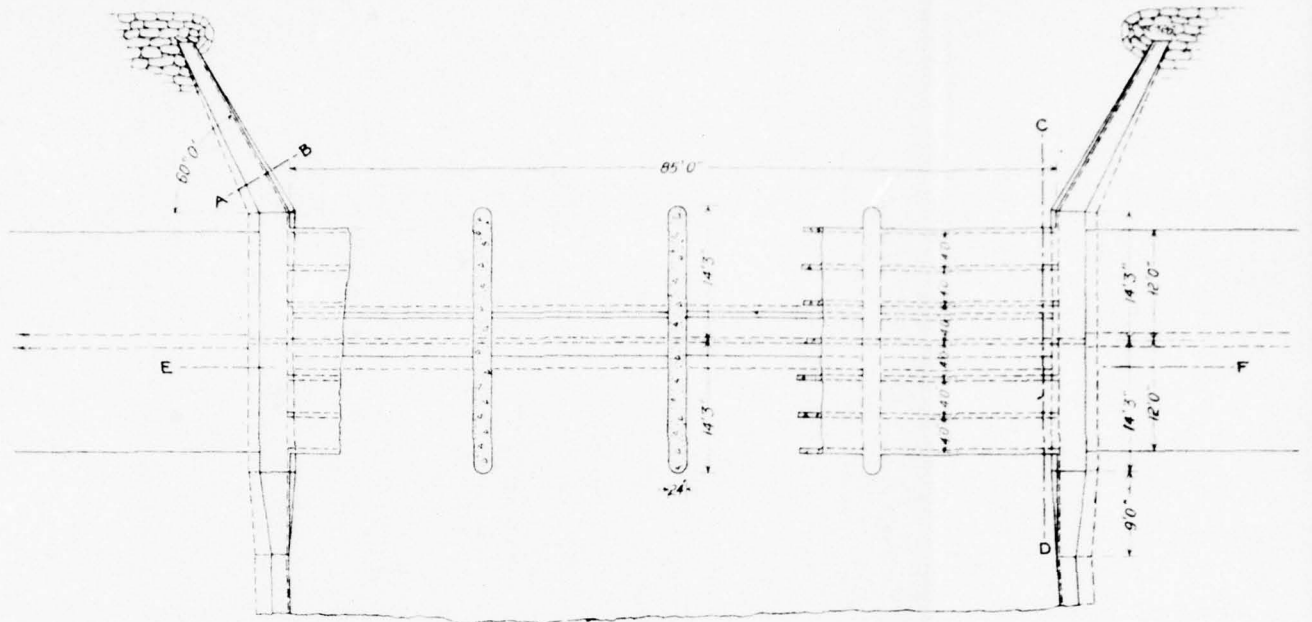
Dwg. # 113-1 (25)  
26

HACKENSACK WATER CO  
HILLSDALE RESERVOIR  
PLAN, PROFILE AND SECTION OF DAM

SCALES AS INDICATED  
AUG. 1903

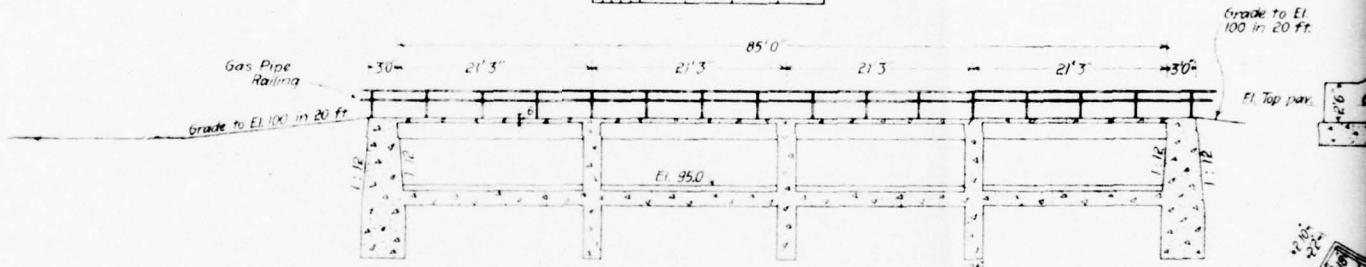
Approved *Henry J. Fuller*

Sheet No. 2



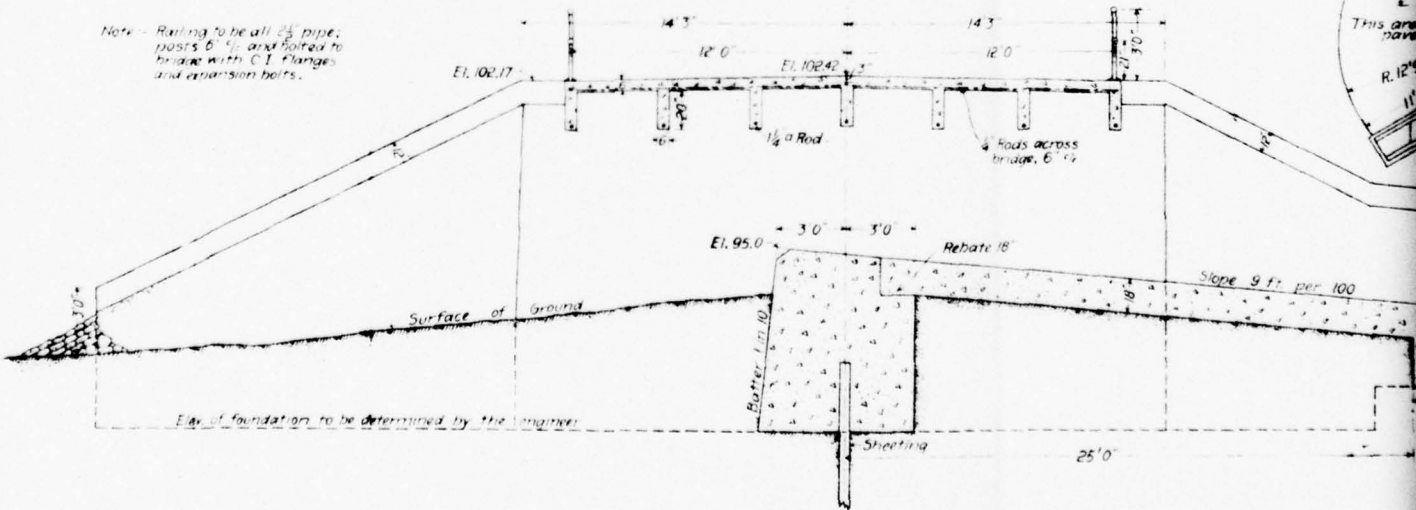
Sectional Plan  
of Waste-way and Bridge  
Scale 1" = 10'

0 10 20



Section E - F  
Scale 1" = 10'

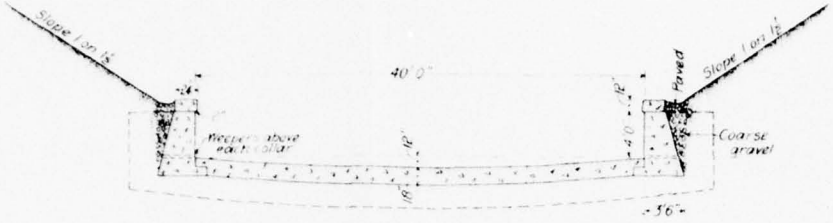
Note - Railing to be all 2 1/2" pipes, posts 6" dia. and bolted to bridge with C.I. flanges and expansion bolts.



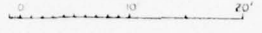
Section C - D  
Scale 1/4" = 1'

0 10

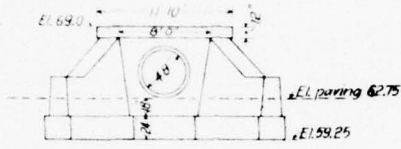
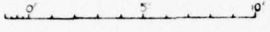
OFFICE OF  
HERN & FOLEY  
HYDRAULIC ENGINEERS AND SANITARY EXPERTS  
170 BROADWAY  
NEW YORK



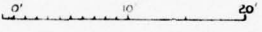
Section of Waste Channel  
Scale  $\frac{1}{8}'' = 1'$



Section A - B  
Scale  $\frac{1}{4}'' = 1'$

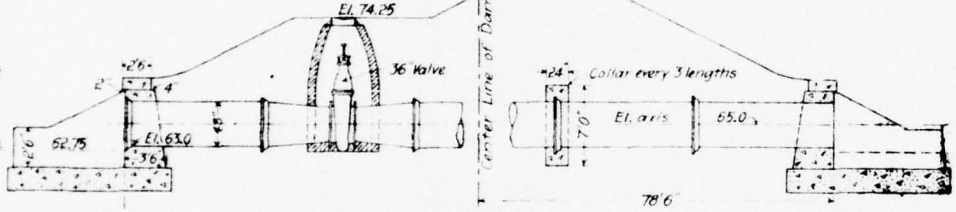


End View of Outlet  
Scale  $\frac{1}{8}'' = 1'$



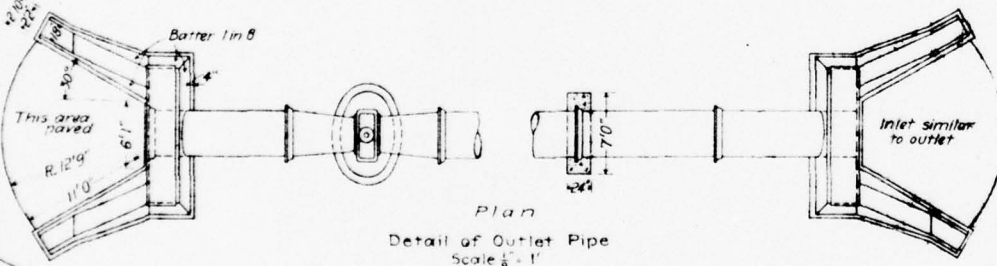
grade to El. 60 in 20 ft.

El. Top par.

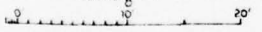


Use 13 lengths of straight pipe

Sectional Elevation



Plan  
Detail of Outlet Pipe  
Scale  $\frac{1}{8}'' = 1'$



WOODCLIFF LAKE  
DWG. NO. 3

Dwg No. 113-1 (25) 47

HACKENSACK WATER CO  
HILLSDALE RESERVOIR  
WASTE-WAY AND OUTLET PIPE

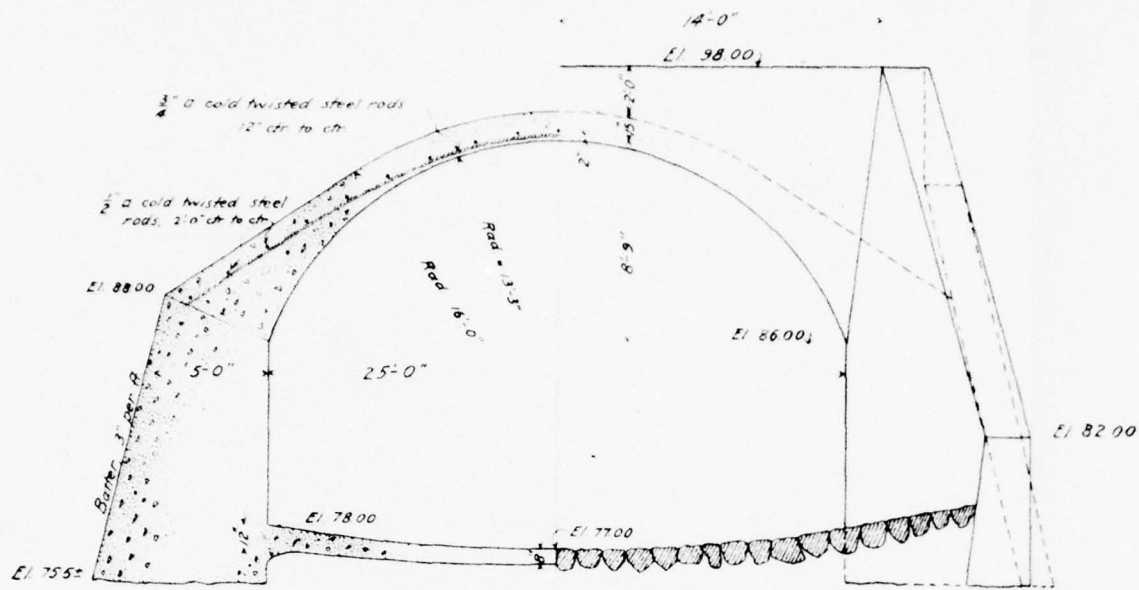
SCALES AS INDICATED 80 77

AUG. 1907

Approved by *[Signature]*

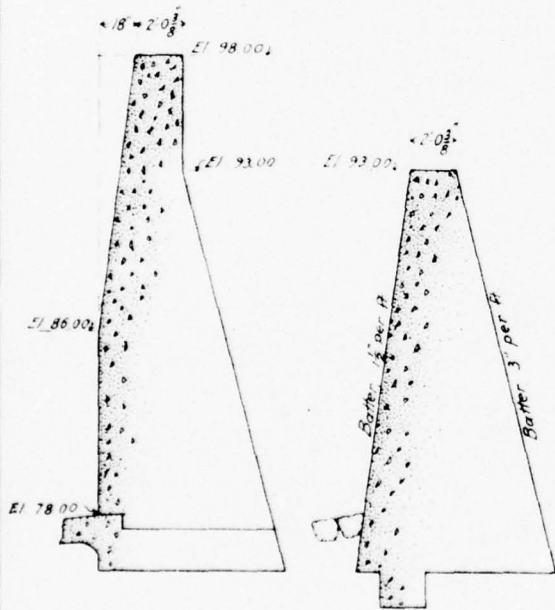
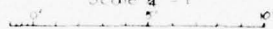
Sheet No. 3

*J*



SECTION AB-CD

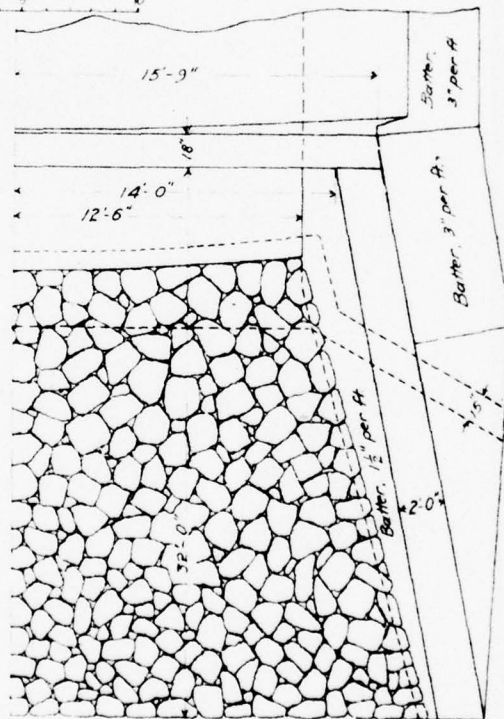
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SECTION GH

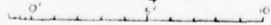
SECTION KL

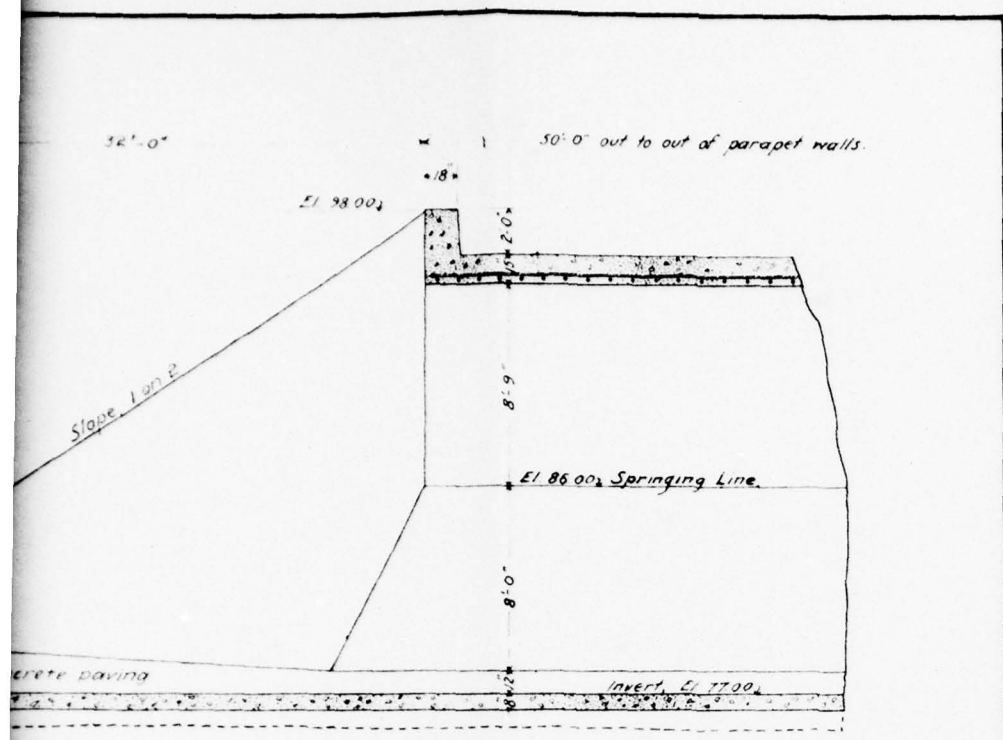
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PLAN UP STREAM END

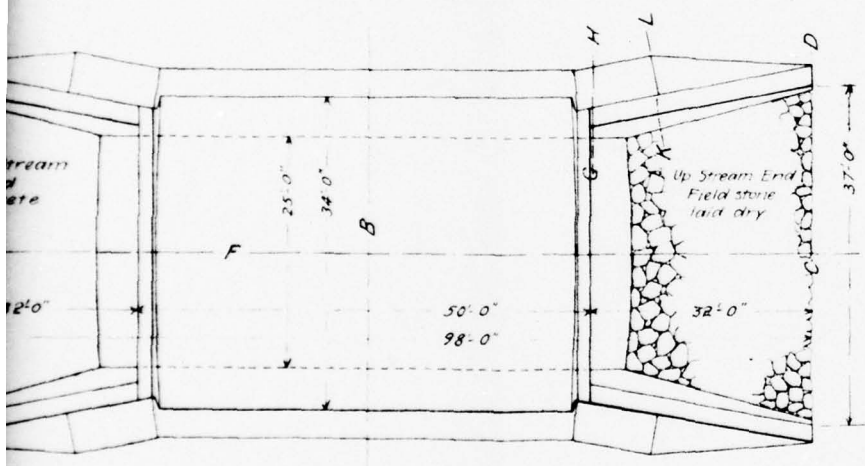
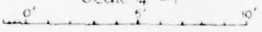
Scale  $\frac{1}{4}'' = 1'$





SECTION EF

Scale  $\frac{1}{4}'' = 1'$



GENERAL PLAN

Scale  $1'' = 10'$



WOODCLIFF LAKE

Dwg. 113-A (25)

DWG. NO. 4

HACKENSACK WATER CO  
HILLSDALE RESERVOIR  
CONCRETE ARCH BRIDGE FOR ROAD CROSSING

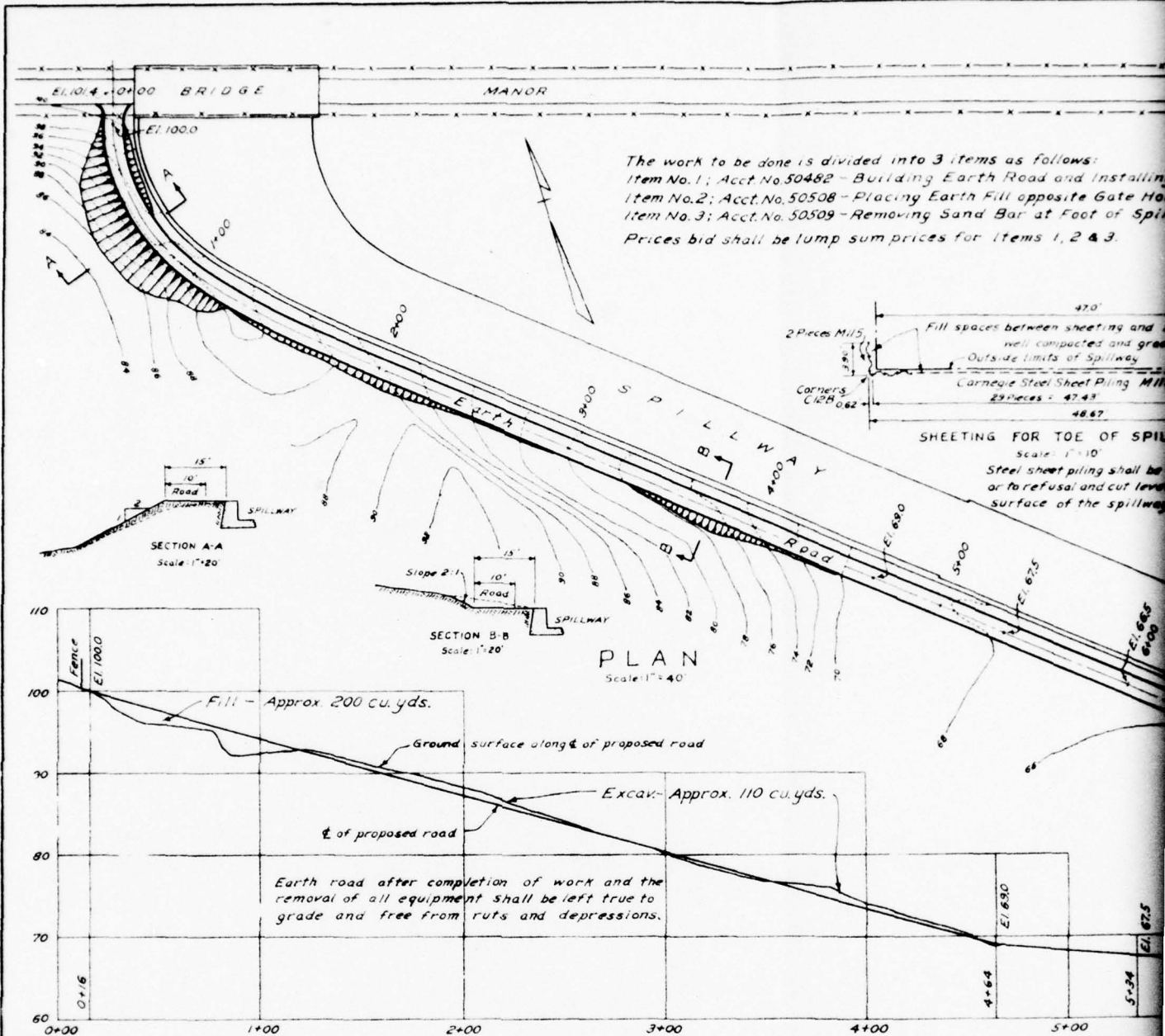
SCALES AS INDICATED

MARCH 1904

Approved *Henry S. Fuller*

Sheet No. 4

2



**SPECIFICATIONS**

The extent of the work as shown is approximate and may be varied within reasonable limits without affecting prices bid to suit actual requirements.

The use of existing structures for any purpose in construction will not be permitted unless full consent and approval have been obtained from the Hydraulic Engineer of the Macksack Water Co. Vehicles using the bridge across the spillway will be restricted to a maximum total load of 5 tons, including weight of vehicle.

Sanitary conveniences must be provided and strict regulations shall be issued and enforced for their use.

Excavated material shall be spread on adjacent low lands, in a manner which will permit water to run off and not form pools.

Fill can be obtained from the Company's property just southeast of the dam.

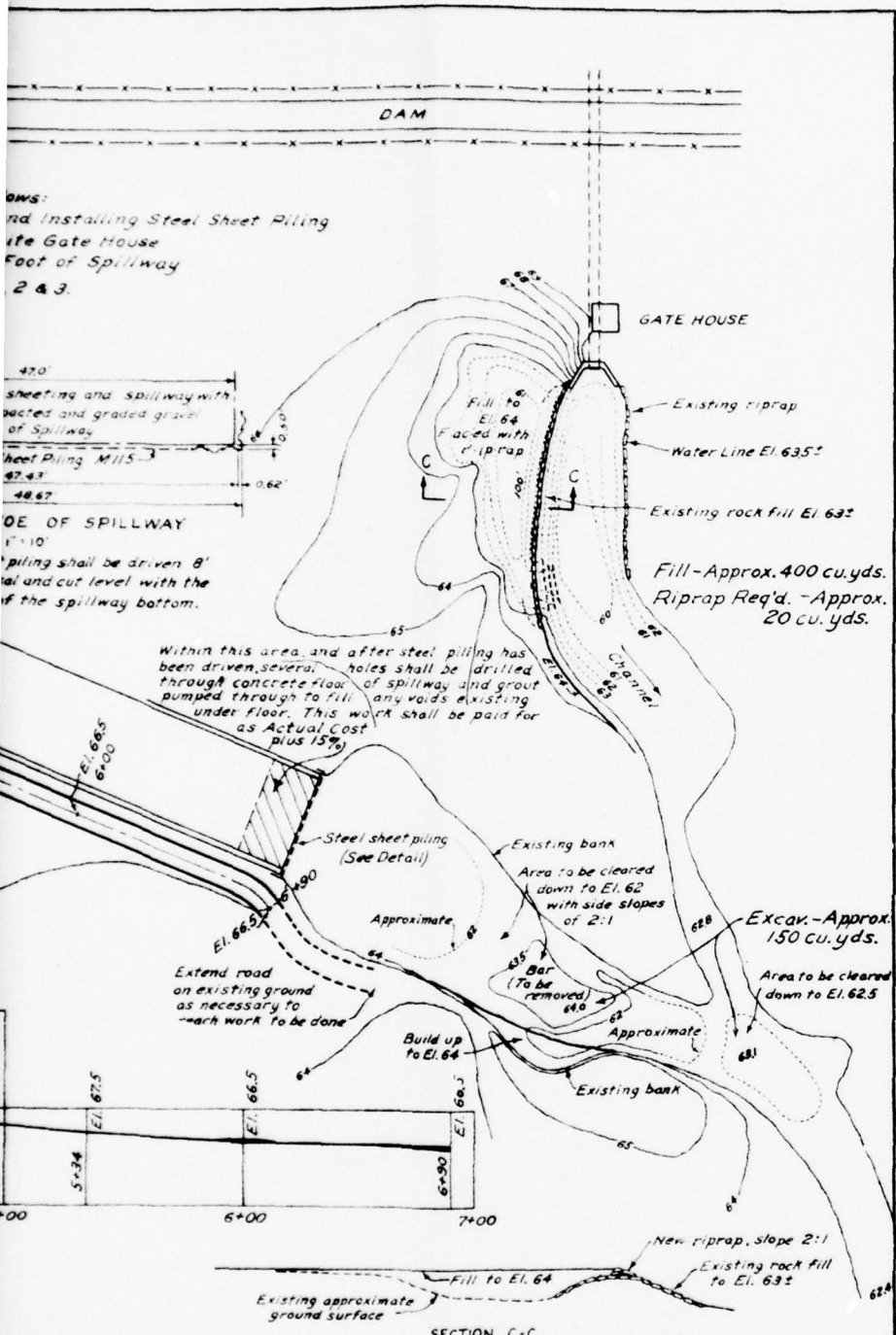
The site of work including borrow pit, shall be left in a

good condition upon completion of the work.

All conditions, procedure and terms relating to the work to its completion shall be subject to the approval of the Hydraulic Engineer.

The quantities of earth excavation and fill are approximate only, but will not vary materially from the amount of work to be done.

The prices bid shall be lump sum prices including all materials, plant equipment and tools for the work hereon shown to be done.



OWS:  
and Installing Steel Sheet Piling  
the Gate House  
Foot of Spillway  
2 & 3.

470'  
sheeting and spillway with  
packed and graded gravel  
of Spillway  
Sheet Piling Mills  
47.43'  
48.67'

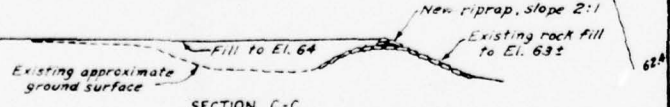
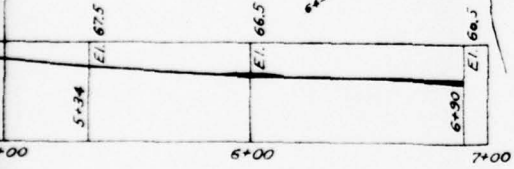
DE OF SPILLWAY  
1" = 10'  
Piling shall be driven 8'  
vertical and cut level with the  
bottom of the spillway bottom.

Within this area, and after steel piling has  
been driven, several holes shall be drilled  
through concrete floor of spillway and grout  
pumped through to fill any voids existing  
under floor. This work shall be paid for  
as Actual Cost plus 15%.

Steel sheet piling  
(See Detail)  
Approximate  
Extend road  
on existing ground  
as necessary to  
mark work to be done

Existing bank  
Area to be cleared  
down to El. 62  
with side slopes  
of 2:1

Excav. - Approx.  
150 cu. yds.  
Area to be cleared  
down to El. 62.5



SECTION C-C  
Scale: 1" = 10'

DWG. NO. 5

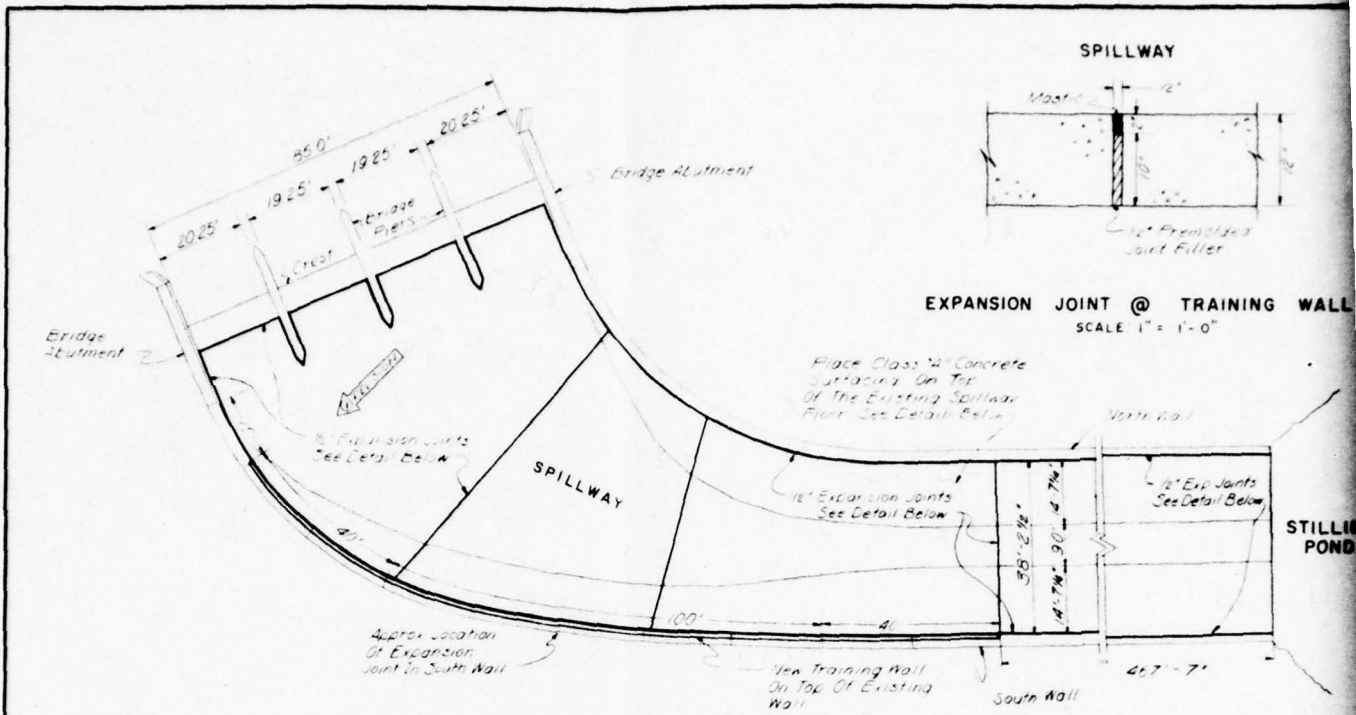
DRAWING NO. 113-1  
49 P

work.  
Terms relative to the prosecution  
be subjected to the satisfaction  
engineer.  
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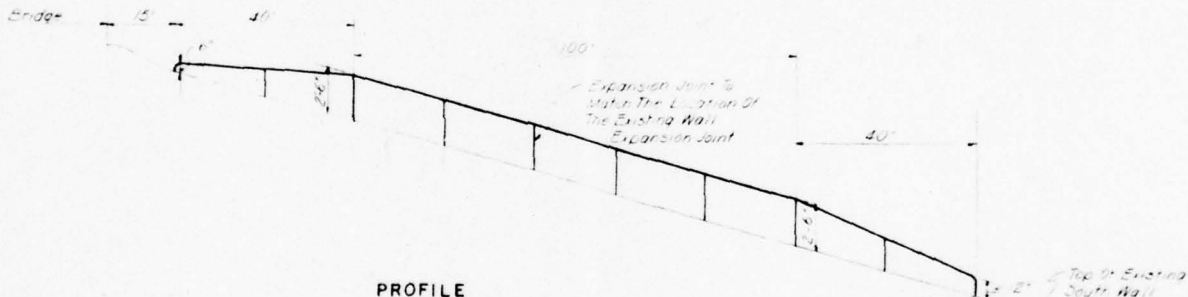
um prices complete for furnish-  
t and tools for performing the

HACKENSACK WATER CO. WEEHAWKEN, N. J.		
WOODCLIFF LAKE RESERVOIR IMPROVEMENTS TO SPILLWAY		
SCALE As Shown	DATE 5-18-37.	
ACCOUNT NO. 50482	INVESTIGATION NO. —	EXTENSION NO. —
DRAWN BY Peter	CHECKED BY [Signature]	APPROVED [Signature]

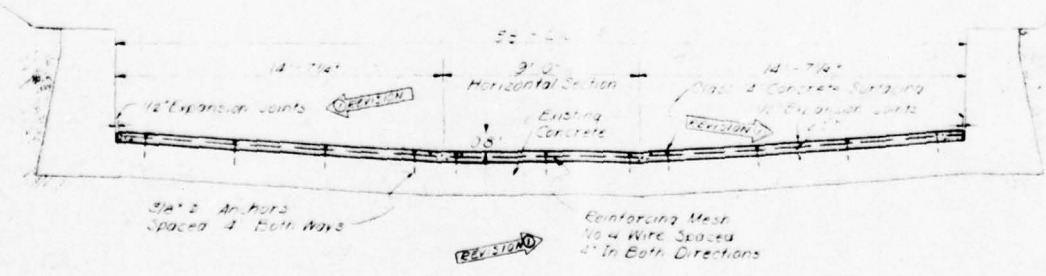
2



**PLAN**  
SCALE 1" = 20'



**PROFILE**  
**NEW TRAINING WALL**  
SCALE HOR 1" = 20'  
VERT 1" = 5'



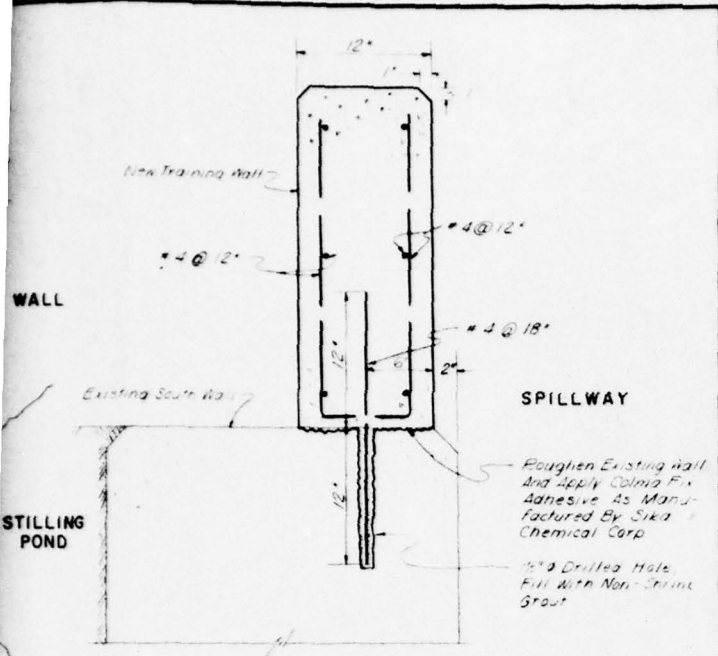
**TYPICAL SPILLWAY SECTION**  
SCALE 1/4" = 1'-0"

**END**  
SCALE 1" = 1'-0"

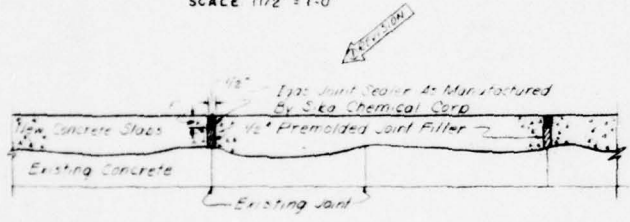
Note 1: All Loose or Disturbed Concrete To Be Removed To Entire Depth. To Be Replaced With Class #4 Concrete. Rebar To Be Installed In Accordance With Plans. Surfacting The Existing Concrete With Thorbond 2. Manure Drywall Product Inc. Be.

Note 2: Max. Sealing Between In Transverse Direction 60' Except Otherwise Engineer In The Field.

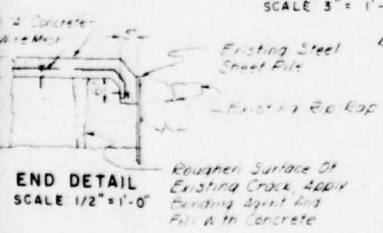
Revision 2 10/7/72



TYPICAL SECTION  
NEW TRAINING WALL  
SCALE 1 1/2" = 1'-0"



EXPANSION JOINT DETAIL  
@ SPILLWAY FLOOR  
SCALE 3" = 1'-0"



END DETAIL  
SCALE 1/2" = 1'-0"

DWG. NO. 6

113-1-2  
76

DRAWING NO.

HACKENSACK WATER CO.  
WEEHAWKEN, N. J.

IMPROVEMENTS TO  
WOODCLIFF LAKE SPILLWAY  
HILLSDALE, N. J.

NEW TRAINING WALL  
AND SPILLWAY FLOOR REPAIR

SCALE AS SHOWN DATE: AUG. 1976

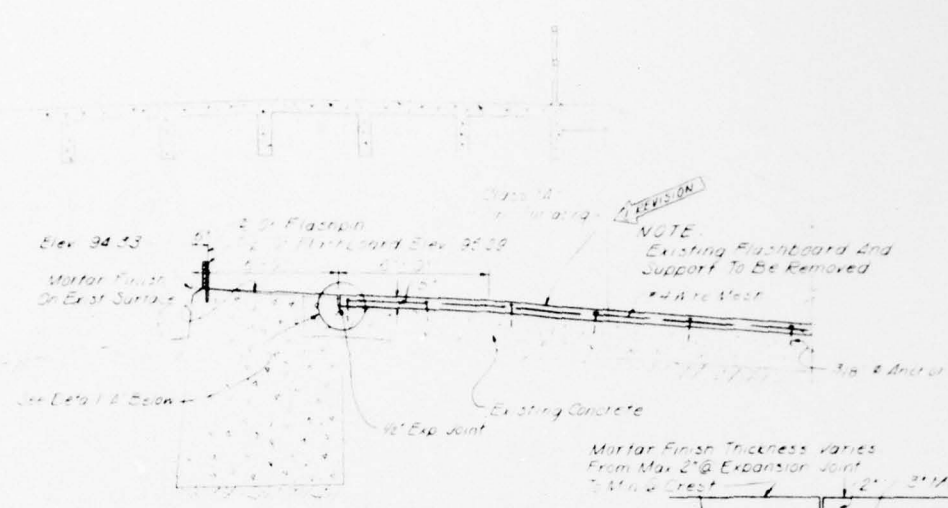
DESIGN ITEM NO.	INVESTIGATION NO.	ESTIMATION NO.
DRAWN BY	CHECKED BY	APPROVED BY

Consolidated Concrete To  
The Entire Surface Of Existing  
To The Quality of Existing  
and 20 Placing Class 4 Concrete  
The Bonding Agent Shall Be  
2. Making Tests By Standard  
and In The Approved Equal

Between The Expansion Joints  
In Direction, It's Not Exceed  
Otherwise Determined By The  
The Field

10/7/76

2



SECTION THRU SPILLWAY CREST  
SCALE 1/4" = 1'-0"

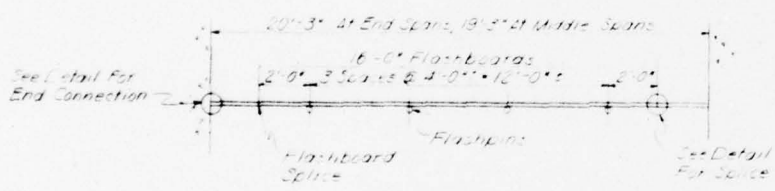
NOTE:  
Existing Flashboard And  
Support To Be Removed  
#4 Wire Mesh

Mortar Finish Thickness Varies  
From Max 2" @ Expansion Joint  
To Min 1/2" Crest

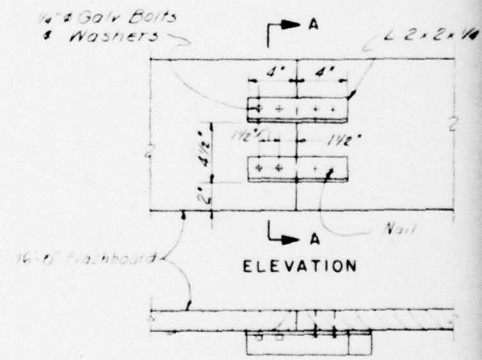
Apply Bonding Agent  
Before Finishing Mortar  
Or Cross 4" Concrete

5" Clear  
Concrete  
Exist C  
Removes  
Varies f  
@ Expans  
0" Min @  
Stream &  
Joint.

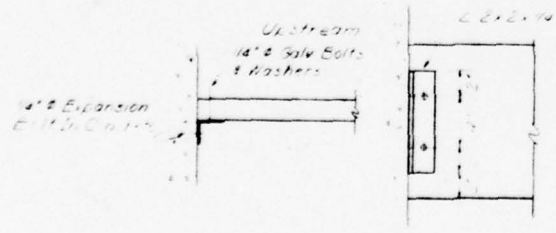
DETAIL "A"



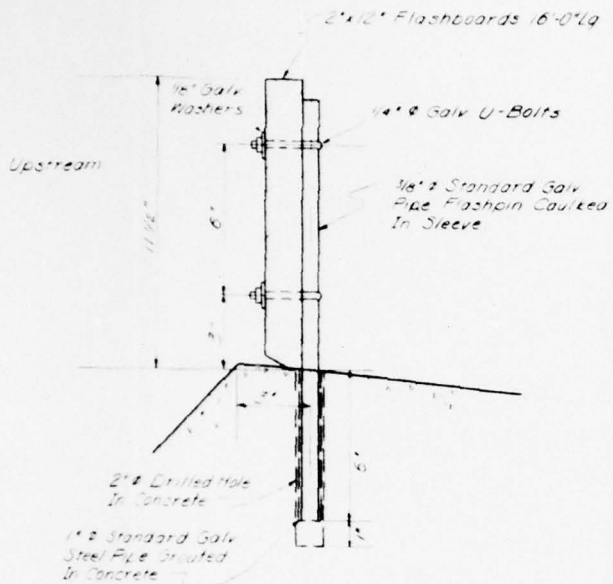
PLAN OF FLASHBOARD CENTERING  
SCALE 1/4" = 1'-0"



DETAIL OF FLASHBOARD  
SCALE 1/4" = 1'-0"

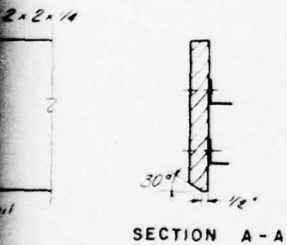


DETAIL OF END CONNECTIONS  
SCALE 1/2" = 1'-0"



DETAIL OF FLASHPIN  
SCALE 3" = 1'-0"

5" Class 11  
Concrete  
Exst Conc To Be  
Removed Thickness  
Varies From 3" Max  
@ Expansion Joint To  
0" Min @ 6' 0" Down  
Stream Of Expansion  
Joint



SECTION A-A

BOARD SPLICE  
= 1'-0"

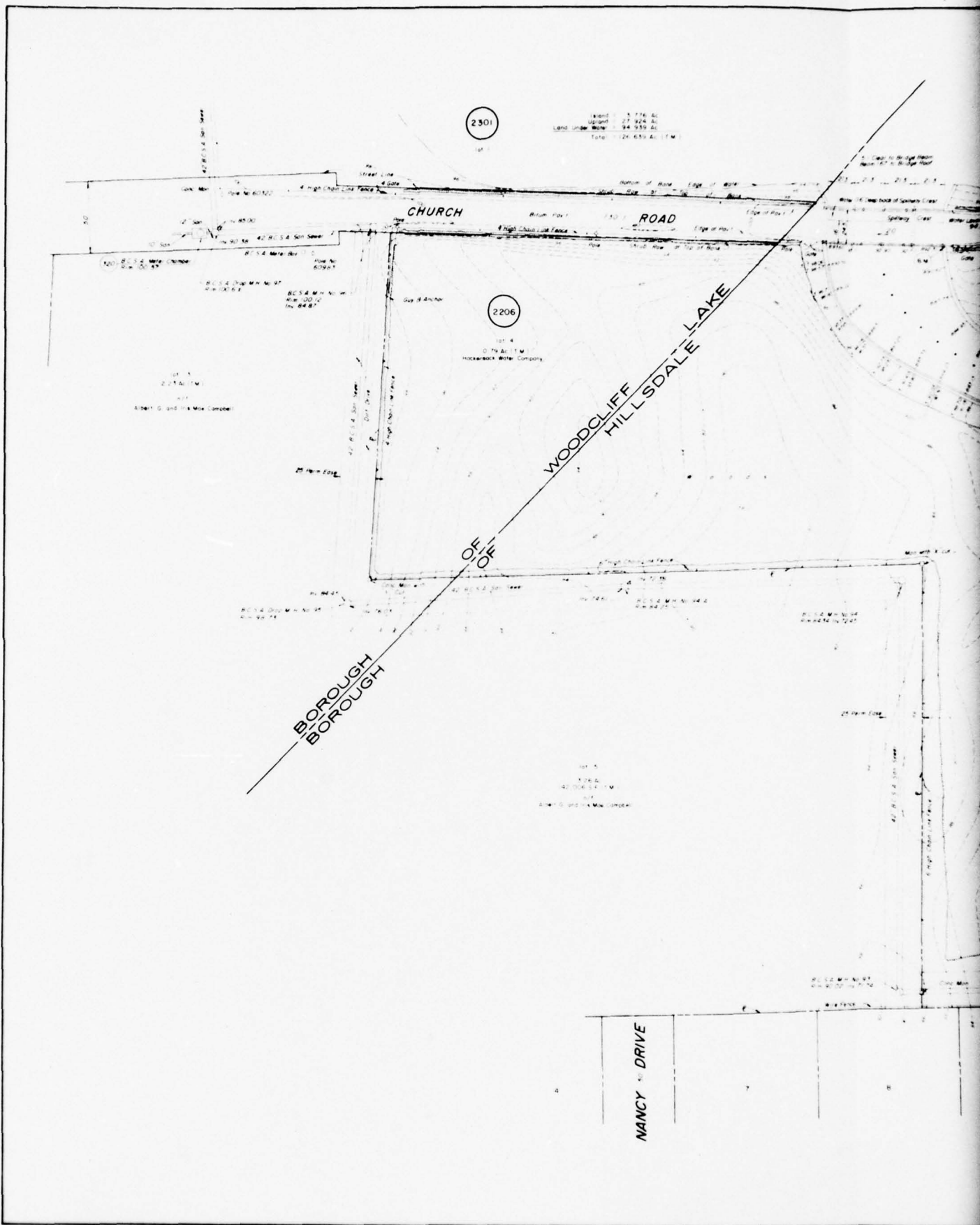
DWG. NO. 7

DRAWING NO. 113-1-1  
76

HACKENSACK WATER CO. WEEHAWKEN, N. J.		
IMPROVEMENTS TO WOODCLIFF LAKE SPILLWAY HILLSDALE, N. J.		
FLASHBOARD INSTALLATION & DETAILS		
SCALE AS SHOWN		DATE AUG 1976
BUDGET ITEM NO	INVESTIGATION NO	EXTENSION NO
DRAWN BY EJM	CHECKED BY JH	APPROVED BY JH

Revision ①, 10/7/76

5



Island	1.176 Ac
Upland	17.924 Ac
Land Under Water	94.939 Ac
Total	114.039 Ac (1.1 M)

2301  
1st 1

2206  
1st 4  
0.79 Ac (1.1 M)  
Hockaback Water Company

CHURCH ROAD

WOODCLIFF HILLSDALE LAKE

BOROUGH BOROUGH

NANCY DRIVE

1st 1  
2.71 Ac (1.1 M)  
Albert G. and Mrs. Mabel

1st 5  
1.76 Ac  
42,006 S. 4.1 M  
Albert G. and Mrs. Mabel

1st 2  
1.02 Ac (1.1 M)  
42,006 S. 4.1 M

42,006 S. 4.1 M  
1st 2  
1.02 Ac (1.1 M)  
42,006 S. 4.1 M

1st 2  
1.02 Ac (1.1 M)  
42,006 S. 4.1 M

1st 2  
1.02 Ac (1.1 M)  
42,006 S. 4.1 M

1st 2  
1.02 Ac (1.1 M)  
42,006 S. 4.1 M

42,006 S. 4.1 M  
1st 2  
1.02 Ac (1.1 M)

1st 2  
1.02 Ac (1.1 M)  
42,006 S. 4.1 M



WOODCLIFF LAKE  
(RESERVOIR)

1006

22.36 A.C.  
S. 643.55 S.E.  
WOODCLIFF LAKE RESERVOIR  
HOCKESSACK WATER COMPANY, I.M.

CHURCH

ROAD

1005

22.36 A.C.  
S. 643.55 S.E.  
WOODCLIFF LAKE RESERVOIR  
HOCKESSACK WATER COMPANY, I.M.

1008

22.36 A.C.  
S. 643.55 S.E.  
WOODCLIFF LAKE RESERVOIR  
HOCKESSACK WATER COMPANY, I.M.

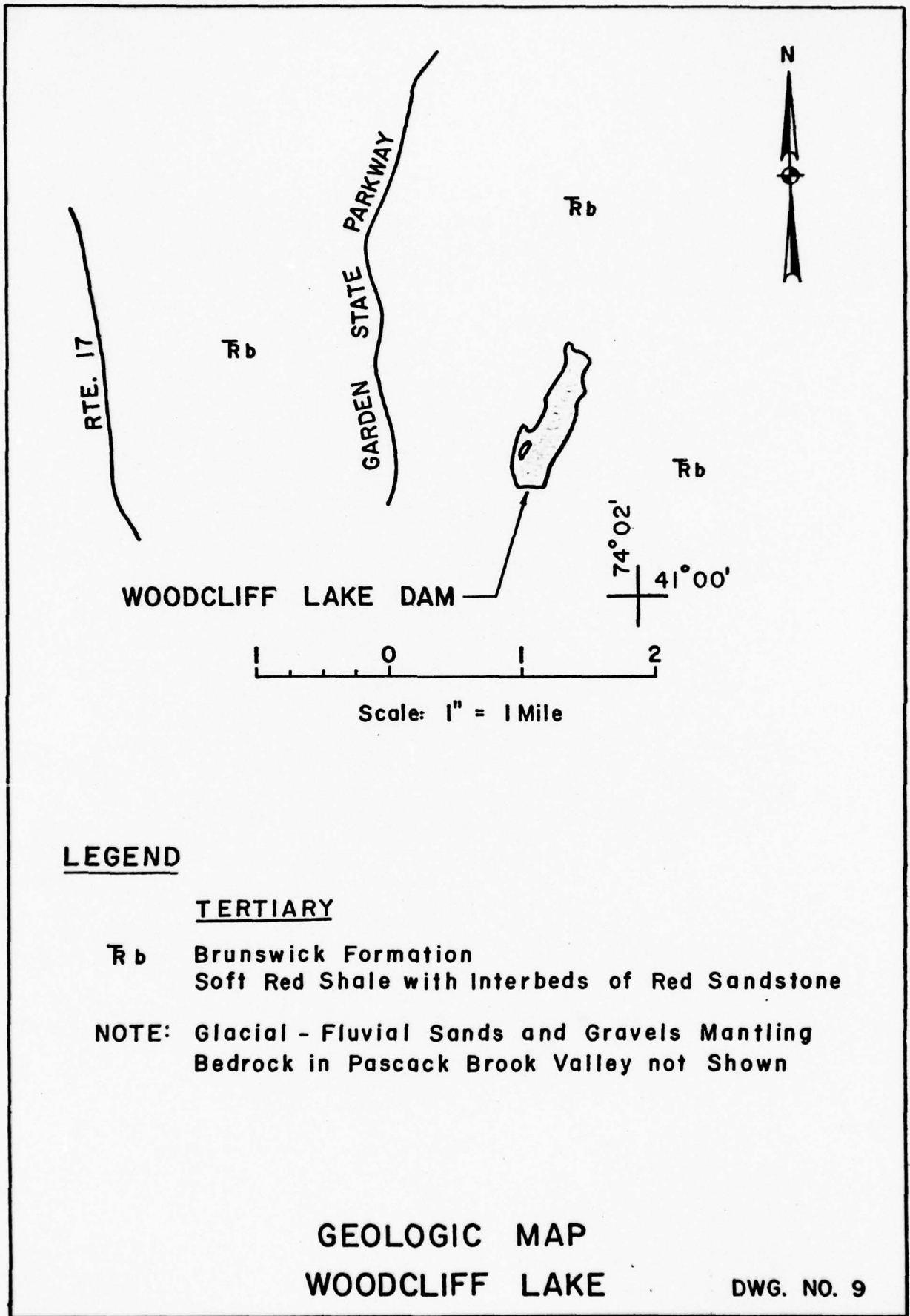
1008

Subdivision Plot  
York Map No. 5547 - Section One  
York Map No. 5547 - 1143 617160



2





**LEGEND**

**TERTIARY**

**Rb** Brunswick Formation  
Soft Red Shale with Interbeds of Red Sandstone

**NOTE:** Glacial - Fluvial Sands and Gravels Mantling  
Bedrock in Pascack Brook Valley not Shown

**GEOLOGIC MAP  
WOODCLIFF LAKE**

**DWG. NO. 9**

APPENDIX A

CHECK LIST - VISUAL OBSERVATIONS

CHECK LIST - ENGINEERING, CONSTRUCTION  
MAINTENANCE DATA

CHECK LIST  
VISUAL INSPECTION  
PHASE 1

Name Dam WOODCLIFF LAKE DAM County Bergen State New Jersey Coordinators \_\_\_\_\_

Date(s) Inspection May 2, 1978 Weather Partly Cloudy Temperature 50°F  
May 6, 1978 Raining 55°F

Pool Elevation at Time of Inspection 93.27 M.S.L. Tailwater at Time of Inspection 62.5 M.S.L. at low level outlet  
Approx.

Inspection Personnel:

Seymour Roth, May 2  
David Kerkes, May 2 and 5  
Yin Au-Yeung, May 2  
Recorder: Seymour M. Roth

William Flynn, May 2  
Lynn Brown, May 6  
Larry Woscyna, NJ-DEP, May 2

Owner: Hackensack Water Company - Representatives: - John Cannizo, Director, Engineering Design  
(on May 2, 1978) and Construction  
- George M. Haskew, Jr., Senior Vice President,  
and Chief Engineer  
- James Butler, Director of System Operation

CONCRETE/MASONRY DAMS

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS AND RECOMMENDATIONS
SEE PAGE ON LEAKAGE	NA	
STRUCTURE TO ABUTMENT/EMBANKMENT JUNCTIONS	NA	
DRAINS	NA	
WATER PASSAGES	NA	
FOUNDATIONS	NA	

CONCRETE/MASONRY DAMS

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SURFACE CRACKS CONCRETE SURFACES	NA	
STRUCTURAL CRACKING	NA	
VERTICAL & HORIZONTAL ALIGNMENT	NA	
MONOLITH JOINTS	NA	
CONSTRUCTION JOINTS	NA	

EMBANKMENT

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SURFACE CRACKS	None apparent; the top of the embankment is paved with a two-lane asphalt road. The cracks in the pavement are not considered to be related embankment cracking, movement or safety.	
UNUSUAL MOVEMENT OR CRACKING AT OR BEYOND THE TOE	None apparent or visible.	
SLOUGHING OR EROSION OF EMBANKMENT AND ABUTMENT SLOPES	None apparent. Upstream face of embankment is rip-rapped and in good condition. The downstream slope is covered in places with medium to heavy brush growth. The downstream embankment face does not appear to be firm or highly compacted. This could be because of the construction methods employed in 1903 utilized horse drawn compaction equipment or the foot deep loam layer. There are some active animal burrows visible on downstream face.	Clean D/S slope of brush and trees, and plant with grass or other suitable vegetative cover. Extend brush-free zone 35 ft. beyond toe of embankment line.
VERTICAL & HORIZONTAL ALIGNMENT OF THE CREST	No apparent deviations from line and grade are visible. The dam has been constructed on a straight line axis, whereas the original dam drawings available for inspection show that the right abutment part of the embankment curves upstream.	
RIPRAP FAILURES	No rip-rap failures were observed above the waterline. Rip-rap also extends for a good part of the reservoir rim between the embankment and the causeway crossing the reservoir at North of the causeway, there is no riprap shore protection.	

EMBANKMENT

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS AND RECOMMENDATIONS
JUNCTION OR EMBANKMENT AND ABUTMENT, SPILLWAY AND DAM	No unusual problems or conditions were noticed at the junction of the embankment with the abutments or the spillway.	
ANY NOTICEABLE SEEPAGE	Noticeable seepage exists in the left embankment section in the vicinity of the left abutment. Four separate seepage areas were observed, where the phreatic line intersects the ground at the toe or a short distance downstream of the toe. The natural ground, at these places, slopes away from the toe of the embankment. The seepage combines into a small rivulet 2-ft. wide, flowing at the rate of 5-10 gpm. The seepage water appears clear. One observation well 75 to left of the gate house shows the phreatic line at that location to be one foot below ground level. Large areas of skunk cabbage growth at the downstream toe of slope is a further indication of a high water table.	Suggest installation of a toe drain system that would lower the phreatic line 4 to 6 ft, to include inspection manholes that would allow monitoring of seepage rate and clarity of seepage water.
STAFF GAGE AND RECORDER	An automatic water level recorder on the left shore of the reservoir was not operating on day of inspection. The water levels are read out twice a day currently, and on an hourly basis when rainfall exceeds one inch per 24 hours. A rainfall recorder also is installed in the same vicinity. The rainfall recorder is surrounded by trees, perhaps affecting its accuracy.	Repair water level recorder to its intended function.
DRAINS		

OUTLET WORKS

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS AND RECOMMENDATIONS
CRACKING & SPALLING OF CONCRETE SURFACES IN STILLING BASIN	The outlet conduit was not available for inspection due to its use in discharging water. The headwall of the outlet conduit is spalled but in serviceable condition.	
INTAKE STRUCTURE	There is no intake structure as such. The outlet pipe extends into the reservoir bottom through the embankment. There is no trash rack structure provided at the pipe inlet.	
OUTLET CHAMBER IN DAM	The outlet structure consists of a small gate house housing two 48-in. diameter valves in series, laid on their sides. The U/S emergency valve is manually operated, the D/S service valve is motor operated. The gate valve installation varies from the arrangement shown on dam drawings available.	Provide a correct plan showing outlet structure and valving configuration.
OUTLET FACILITIES	The 48-inch line discharges through a headwall directly into an outlet channel approximately 8-ft. wide, with rip-rapped invert and sides in acceptable condition. There is no stilling basin. The outlet channel joins the Pascaack Brook at the end of the spillway chute channel.	
EMERGENCY GATE	One 36-inch diameter gate valve upstream of the motor operated service valve.	

UNGATED SPILLWAY

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS AND RECOMMENDATIONS
CONCRETE WEIR	Low concrete crest sill in acceptable condition. No major cracks, areas of deterioration or dislocations.	
APPROACH CHANNEL	A rip-rapped short approach channel upstream of the crest sill between wingwalls containing the embankment was in acceptable condition.	
DISCHARGE CHANNEL	Curved discharge channel leading into Pasack Brook. Walls and slabs have no cracks or dislocations. The floor has been repaved in 1976. The walls date from 1903. The right chute wall has been raised, 2'6", in 1976 to contain chute flow standing waves at supercritical flow. Repaving of chute floor has allegedly increased chute flow velocity, so that rain of Nov. 8, 1977 has caused severe downstream channel bank damage and tearing away steel pile cut-off at end of discharge chute.	Repair downstream sheet pile cut-off. Provide energy dissipation sill at end of channel chute slabs.
BRIDGE AND PIER	A four-span bridge setting on low, 5-ft. high pier carries Church Street over the spillway crest. The concrete in the three piers is in acceptable condition. The bridge beams have cantilever bracket supports which cut down the net spillway area. Abutment beam brackets appear to have been added after original construction. Some soffit repairs have been made on the upstream beam in the left bridge span opening consisting of 8 x 8 inch bottom flange encasement extending 1 or 2 in. below the original beam soffit line.	
DOWNSTREAM STREAM CHANNEL OF PASCACK BROOK	Rains of Nov. 8, 1977 have caused severe lateral bank erosion at end of discharge channel. The eroded material has been deposited in brook bed approx. 75 ft. downstream. According to Hackensack Water Co. officials, no significant depth of brook channel erosion exists, only lateral bank damage.	Regrade channel; remove eroded material downstream. Add heavy stone protection to bottom and bank slopes.

GATED SPILLWAY

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS AND RECOMMENDATIONS
CONCRETE SILL	NA	
APPROACH CHANNEL	NA	
DISCHARGE CHANNEL	NA	
BRIDGE AND PIERS	NA	
GATES & OPERATION EQUIPMENT	NA	

INSTRUMENTATION

VISUAL EXAMINATION OF MONUMENTATION/ SURVEYS	OBSERVATIONS	REMARKS AND RECOMMENDATIONS
	None observed.	
OBSERVATION WELLS	One piezometer well approximately 75 ft. to left of gate house was observed. The water level was observed approximately one foot below ground level at this location.	
WEIRS	None.	
PIEZOMETERS	None. The owner is currently considering installation of piezometer.	Install piezometers in the embankment at seepage areas to trace phreatic line level. Add piezometers in non-leakage areas to calibrate normal non-leakage phreatic line.
OTHER		

RESERVOIR

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS AND RECOMMENTATIONS
SLOPES	Generally gentle to flat. No signs of rim sloughing. Reservoir between causeway and dam axis has rip-rap protection consisting of cobble sized stone armoring at the water line extending 2 feet above it.	
SEDIMENTATION	Below the causeway, there seems to be no evidence of sedimentation. Above the causeway, some sedimentation exists due to new construction, not seriously affecting the reservoir capacity.	

DOWNSTREAM CHANNEL

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS AND RECOMMENDATIONS
CONDITION (OBSTRUCTIONS, DEBRIS, ETC.)	The channel banks downstream of the spillway chute have been seriously eroded in the rainstorm discharges of Nov. 8, 1977, removing a bank area of 100-foot long by 15-foot wide on each side of the spillway chute. The coarser materials have been redeposited in the channel some 50 to 75 ft. downstream of the end of the spillway chute.	Regrade downstream channel; restore banks and rip-rap bank slopes and stream bottom.
SLOPES	The slopes are well defined below the damaged areas described above.	
APPROXIMATE NUMBER OF HOMES AND POPULATION	Approximately 10 homes exist immediately downstream of the spillway chute. The downstream area is urbanized.	

CHECK LIST  
ENGINEERING DATA  
DESIGN, CONSTRUCTION, OPERATION

ITEM	REMARKS
PLAN OF DAM	Existing plans of original construction available. Plans for reconstruction of spillway chute available. New topographic survey 1978 for right abutment area available.
REGIONAL VICINITY MAP	Available.
CONSTRUCTION HISTORY	Available orally from Hackensack Water Co. officials. Some records plans of post construction repairs and alterations available.
TYPICAL SECTIONS OF DAM	Available.
HYDROLOGIC/HYDRAULIC DATA	Rating curve for outlet and spillway is available. Area capacity curve available. Hydrographs for major storms available.
OUTLETS - PLAN	} Available, but not accurate nor detailed, apparently not } built according to plans.
- DETAILS	
- CONSTRAINTS	
- DISCHARGE RATINGS	
RAINFALL / RESERVOIR RECORDS	Available These are available but have not been inspected.

CHECK LIST  
ENGINEERING DATA  
DESIGN, CONSTRUCTION, OPERATION  
(continued)

ITEM	REMARKS
DESIGN REPORTS	None available.
GEOLOGY REPORTS	None available.
DESIGN COMPUTATIONS HYDROLOGY & HYDRAULICS DAM STABILITY SEEPAGE STUDIES	No original computation available. Rating curves for discharge of spillway and outlet works. None. None.
MATERIALS INVESTIGATIONS BORING RECORDS LABORATORY FIELD	Very crude shallow depth investigation record available, but not considered useful. None. None. None.
POST-CONSTRUCTION SURVEYS OF DAM	1978 topographic survey of right abutment has been made and is available. Purpose of survey is to locate a possible auxiliary spillway.
BORROW SOURCES	No records uncovered. According to Hackensack Water Co. officials, the dam was built of local materials.
SPILLWAY PLAN - SECTIONS	) Available. )
- DETAILS	

CHECK LIST  
ENGINEERING DATA  
DESIGN, CONSTRUCTION, OPERATION  
(continued)

ITEM	REMARKS
OPERATING EQUIPMENT PLANS AND DETAILS	} None.
MONITORING SYSTEMS	A reservoir security patrol on 24-hour duty. Dam and reservoir are visited every hour.
MODIFICATIONS	A row of sheet piling was added in 1937 to the end of the spillway chute channel. Downstream rip-rap in Pascack Brook was also added then. The spillway chute channel was repaved in 1976 with 5 inches of concrete. The right chute wall was raised 2'-6" in the upper reaches. Flash boards were added to the spillway weir areas in 1976 but have been abandoned.
HIGH POOL RECORDS	Orally given by Hackensack Water Co. officials as one foot below dam crest in 1975. Water level in reservoir recorded twice a day, and on an hourly basis if rainfall exceeds one inch.
POST CONSTRUCTION ENGINEERING STUDIES AND REPORTS	Currently a study is underway to add spillway capacity in the form of an auxiliary spillway on the right abutment area.
PRIOR ACCIDENTS OR FAILURE OF DAM - DESCRIPTION - REPORTS	The downstream brook channel was washed out in November 1977, eroding channel banks and causing damage to the steel sheet pile cut-off at the end of the spillway chute channel.
MAINTENANCE, OPERATION RECORDS	None.

APPENDIX B

PHOTOGRAPHS

ALL PHOTOGRAPHS TAKEN DURING MAY 1978

WOODCLIFF LAKE DAM

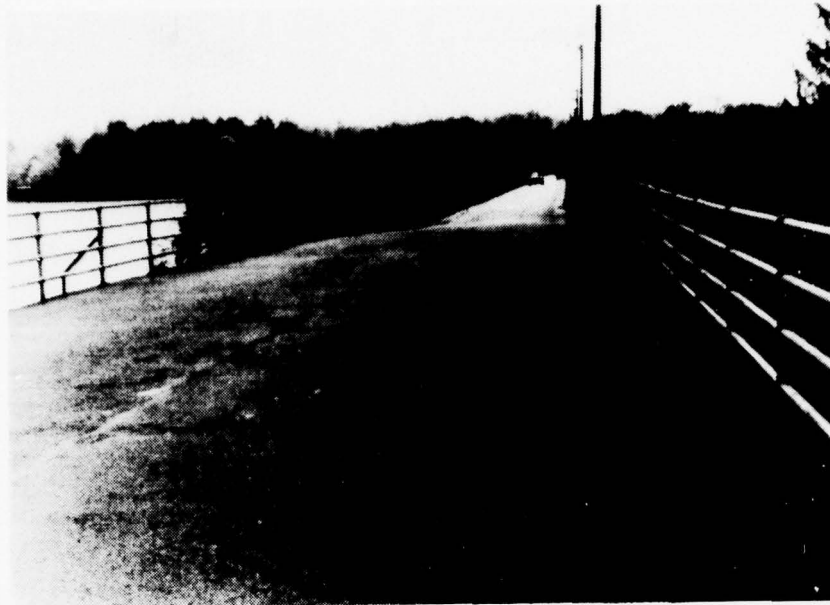


Photo 1 - Looking at dam crest along Church Road toward left abutment from spillway bridge on right abutment



Photo 2 - Looking at dam embankment and low level outlet Gate House from downstream

WOODCLIFF LAKE DAM



Photo 3 - Low level outlet Gate House

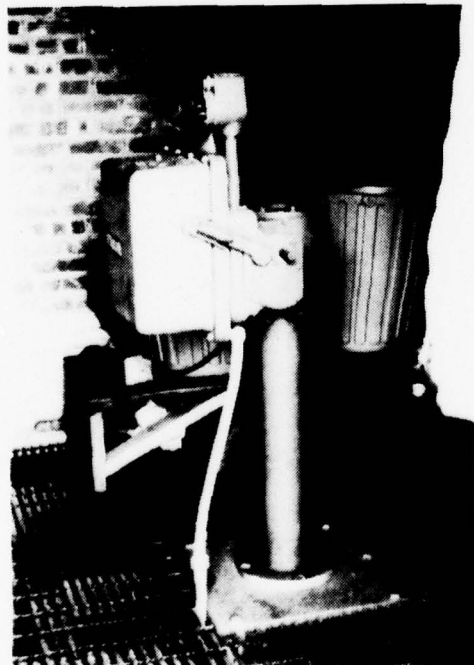


Photo 4 - Motor operated 36-inch service gate valve on 48-inch diameter low level line; hand operated 36-inch diameter emergency gate is upstream of service gate

WOODCLIFF LAKE DAM



Photo 5 - View of downstream face of embankment standing in left abutment area looking toward right abutment



Photo 6 - View of downstream face of embankment standing in left abutment area looking toward left abutment; note large areas of skunk cabbage growth at toe of embankment indicating high water table

WOODCLIFF LAKE DAM



Photo 7 - Detail of seepage area  
on left abutment-rivulet  
downstream of embankment  
toe



Photo 8 - Upstream face of embankment showing spillway  
opening, stone protection on upstream slope and  
approach wing walls

WOODCLIFF LAKE DAM



Photo 9 - Upstream face of embankment showing stone protection at the waterline; looking toward left abutment



Photo 10 - Upstream face of spillway weir under spillway bridge showing approach stone protection, and abandoned and bent flashboard pins

WOODCLIFF LAKE DAM

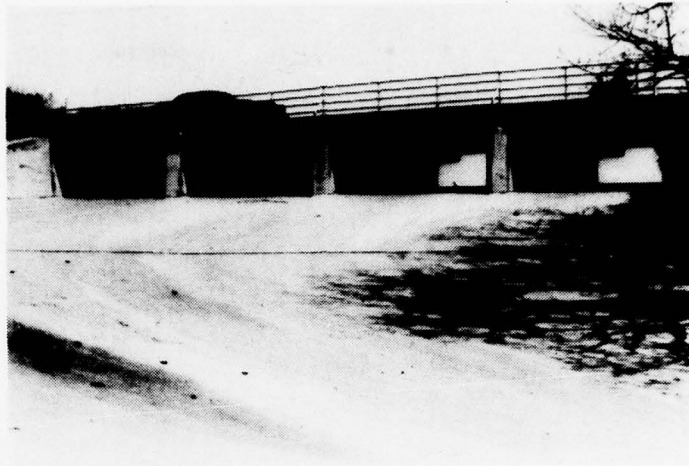


Photo 11 - View of spillway chute channel looking upstream at spillway bridge and Church Road; spillway chute turns through an angle of approximately 60 degrees at upper end



Photo 12 - View of spillway chute channel looking downstream; chute slab has been repaved in 1976

WOODCLIFF LAKE DAM

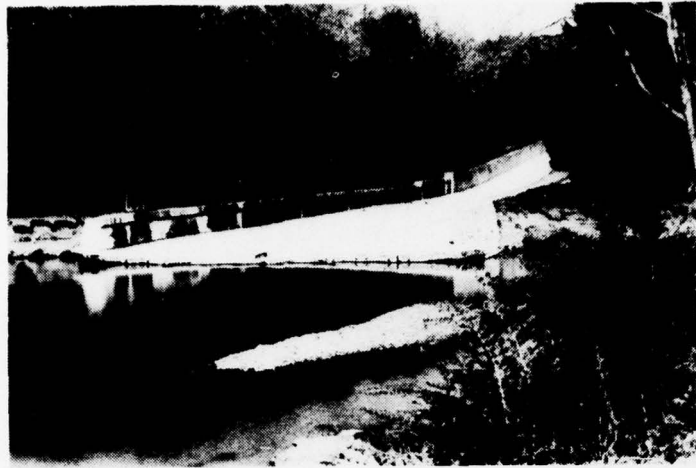


Photo 13 - View of spillway chute channel from downstream; Channel area downstream of chute wall has been severely eroded by spillway discharges during storm of November 8, 1977

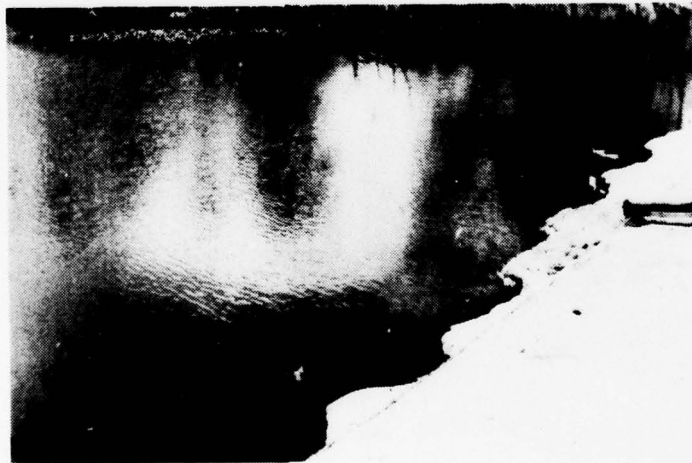


Photo 14 - Detail of damage to steel sheet pile cut-off installed at end of spillway chute channel caused during storm of November 8, 1977. Sheet piling has been torn away from end of chute slab and bent downstream

WOODCLIFF LAKE DAM



Photo 15 - View of upstream face of causeway dividing Woodcliff Lake into two parts; connected by a single barrel culvert 25-foot wide by 18'-9" high

APPENDIX C

SUMMARY OF ENGINEERING DATA

1

CHECK LIST  
HYDROLOGIC AND HYDRAULIC DATA  
ENGINEERING DATA

Name of Dam: WOODCLIFF LAKE DAM

Drainage Area Characteristics: 194 sq.mi. on the Pascack Brook, Hackensack River Basin

Elevation Top Normal Pool (Storage Capacity): 94.33

Elevation Top Flood Control Pool (Storage Capacity): NA

Elevation Maximum Design Pool: 98.03

Elevation Top Dam: 100<sup>±</sup> (Length 1,500 ft.)

SPILLWAY CREST:

- a. Elevation 94.33
- b. Type Uncontrolled concrete overflow, broad crested weir under bridge
- c. Width Broad crest
- d. Length 79 ft. net opening
- e. Location Spillover Near right abutment at Church Road bridge
- f. No. and Type of Gates None

OUTLET WORK:

- a. Type 1-36"Ø service gate valve, a 36"Ø emergency gate valve on a 48"Ø line
- b. Location Center of embankment
- c. Entrance Inverts 63.00<sup>±</sup>
- d. Exit Inverts 61.86<sup>±</sup>
- e. Emergency Draindown Facilities None

HYDROMETEOROLOGICAL GAGES: Two

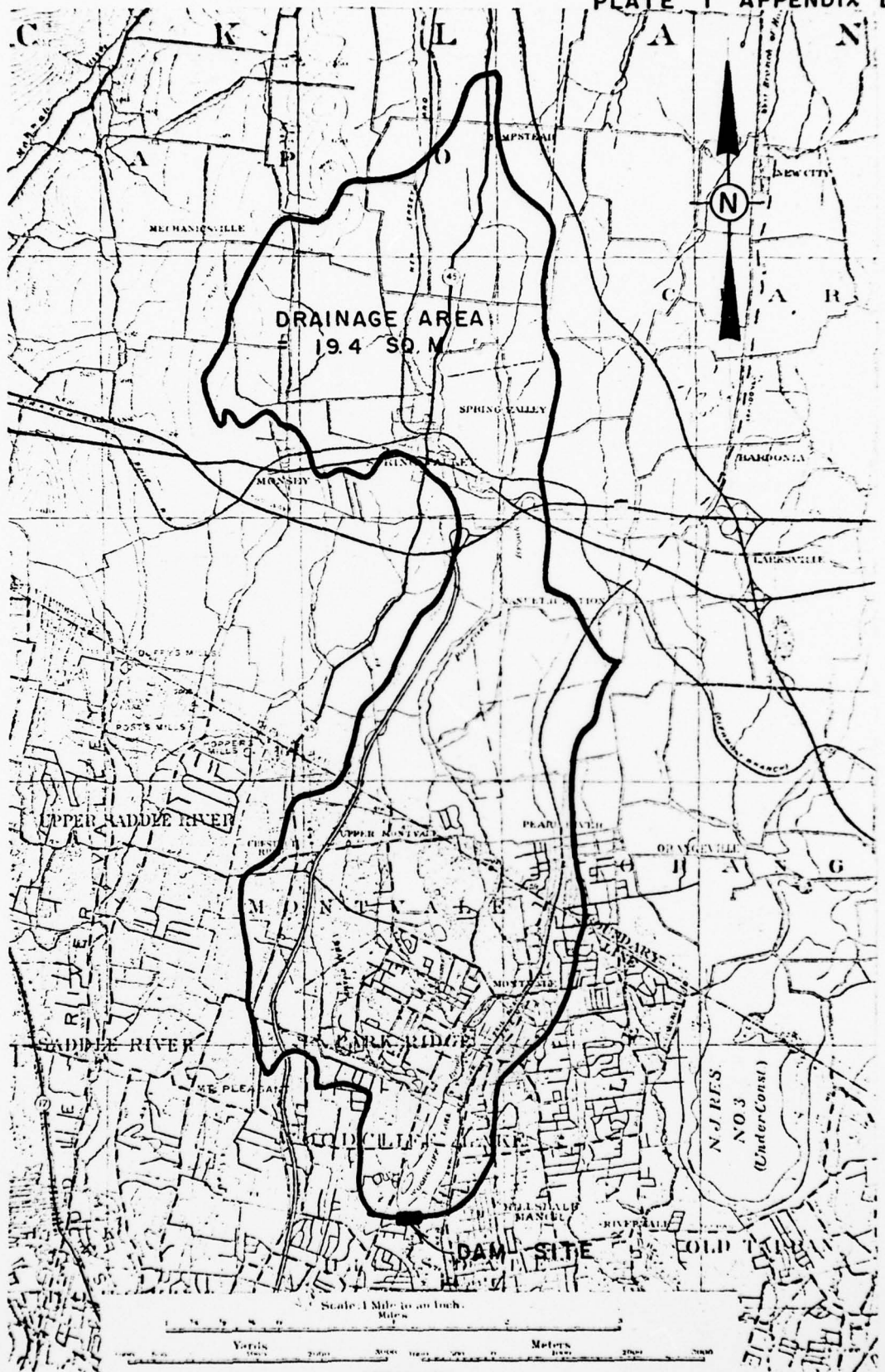
- a. Type One reservoir water level recorder and a rain gage
- b. Location On left shore near dam
- c. Records 1932 to current

MAXIMUM NON-DAMAGING DISCHARGE: 1,650 cfs

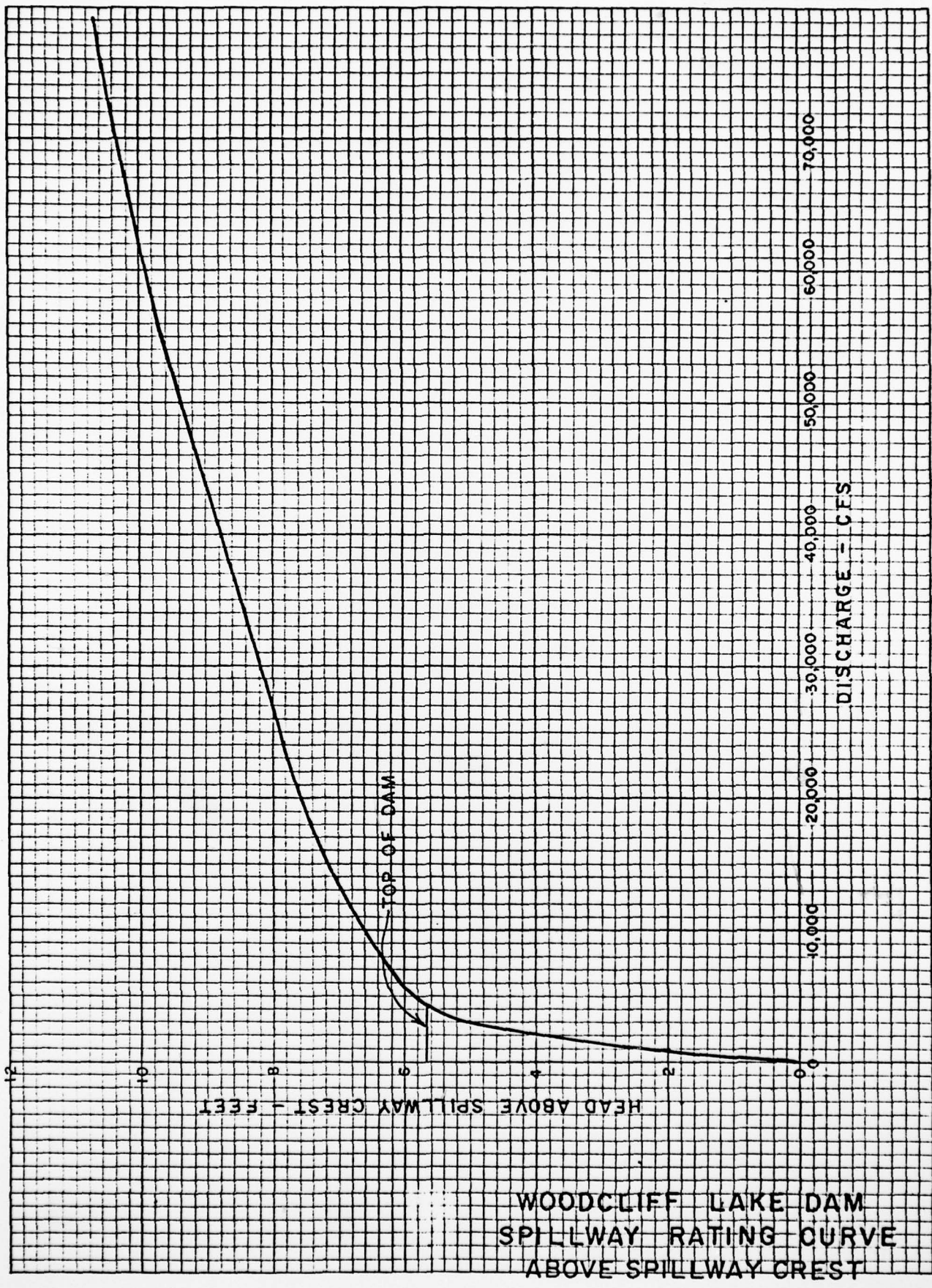
APPENDIX D

HYDROLOGIC COMPUTATIONS

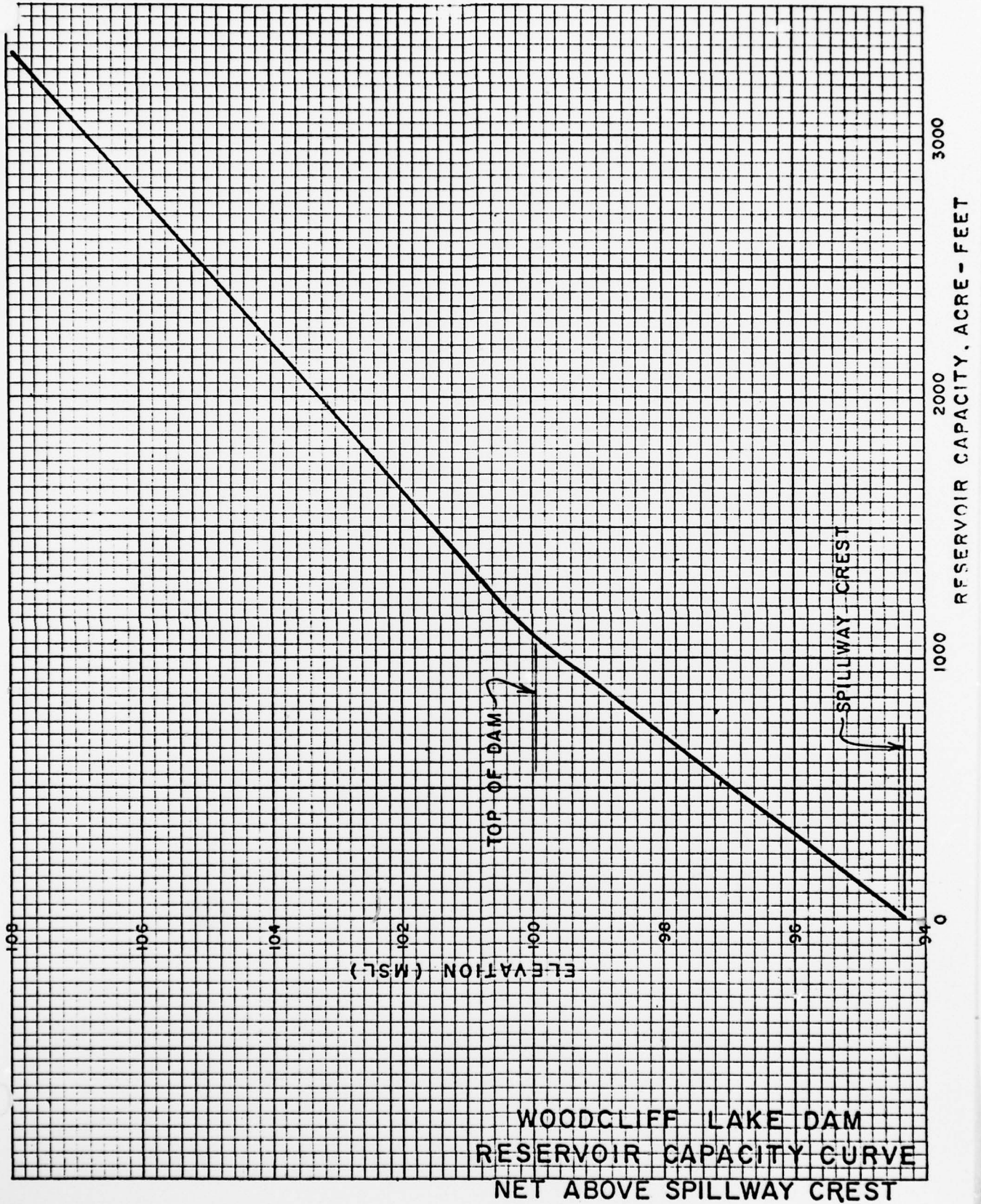
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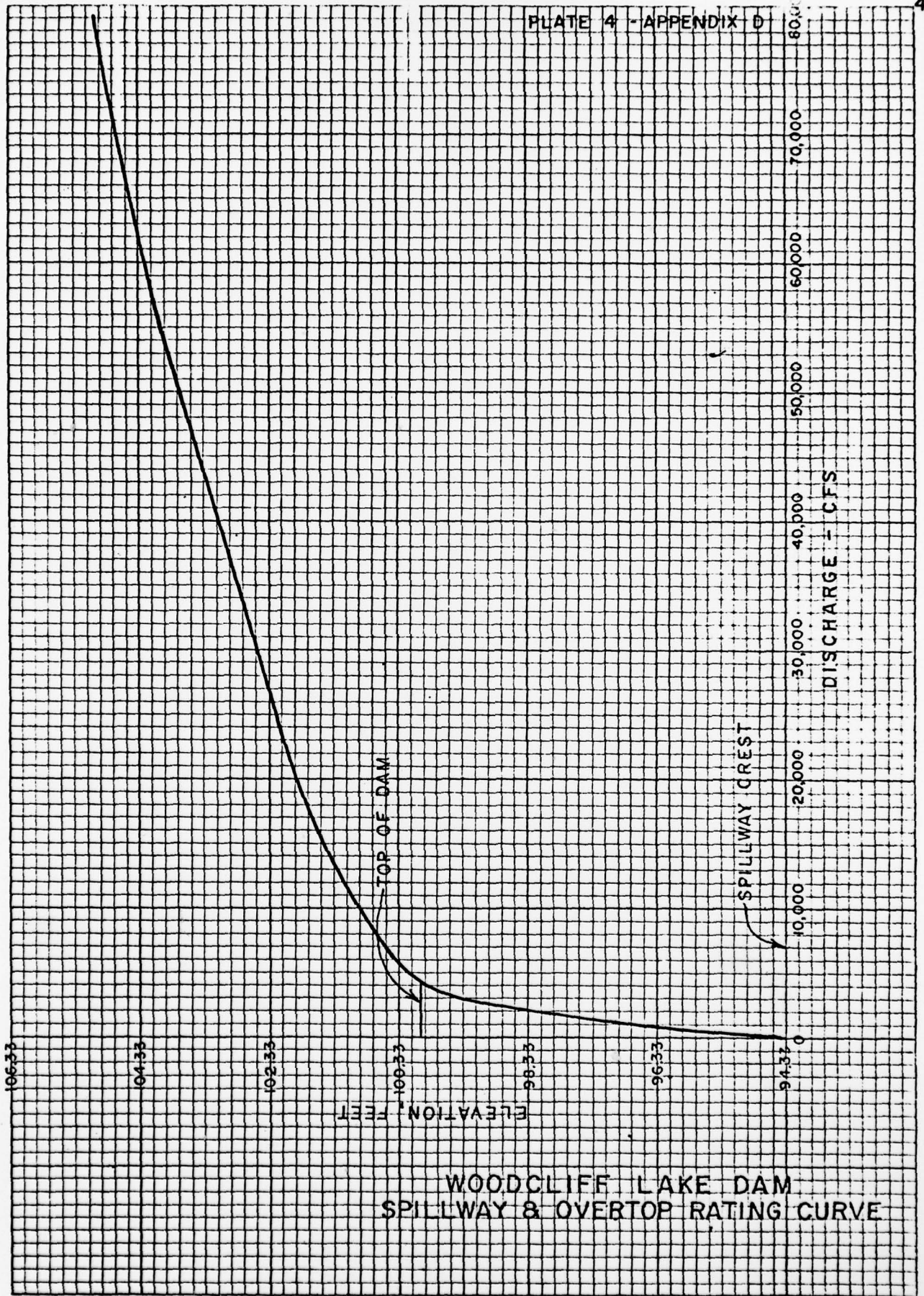
WOODCLIFF LAKE DAM - DRAINAGE BASIN



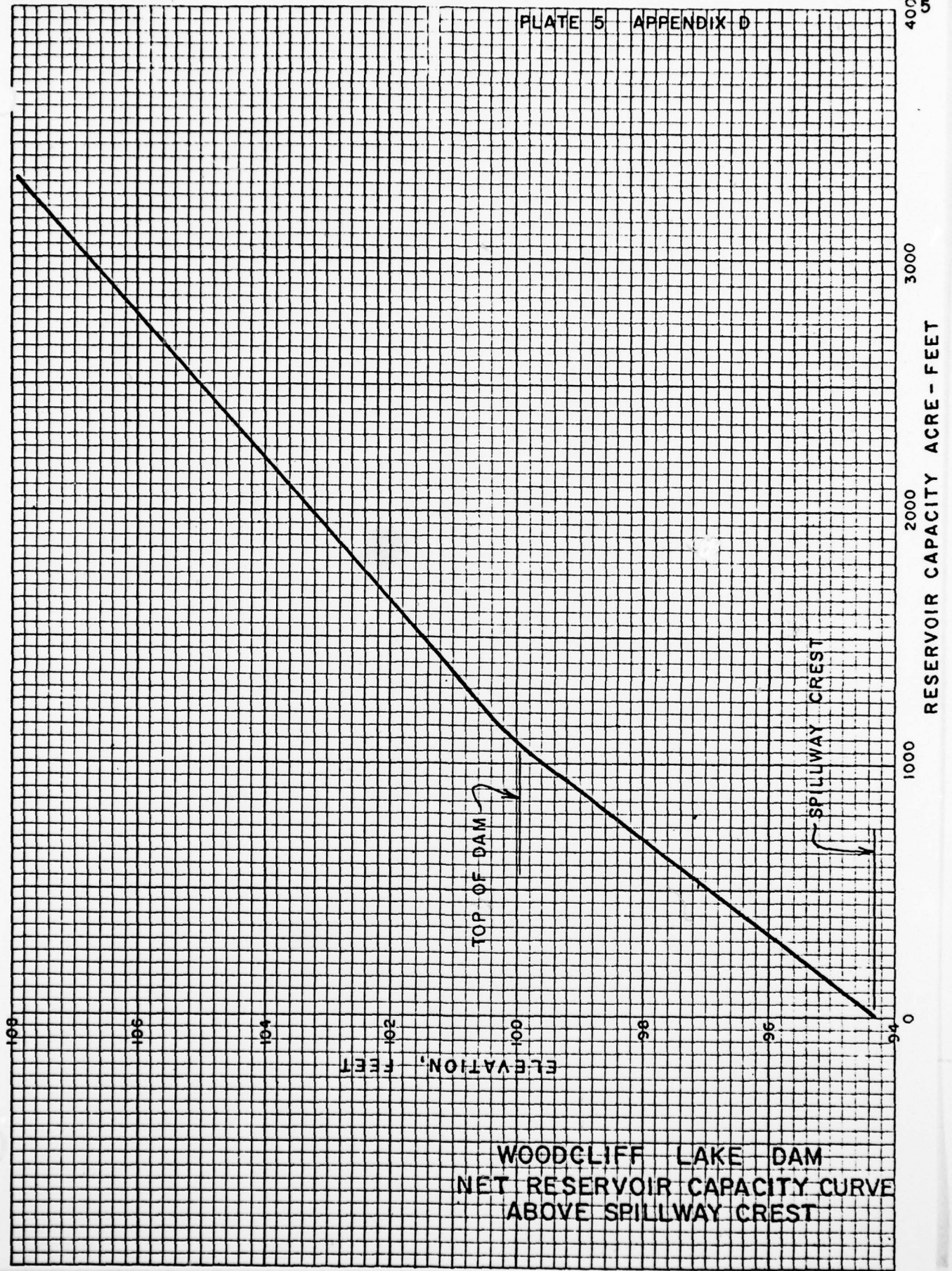
WOODCLIFF LAKE DAM  
SPILLWAY RATING CURVE  
ABOVE SPILLWAY CREST



WOODCLIFF LAKE DAM  
RESERVOIR CAPACITY CURVE  
NET ABOVE SPILLWAY CREST

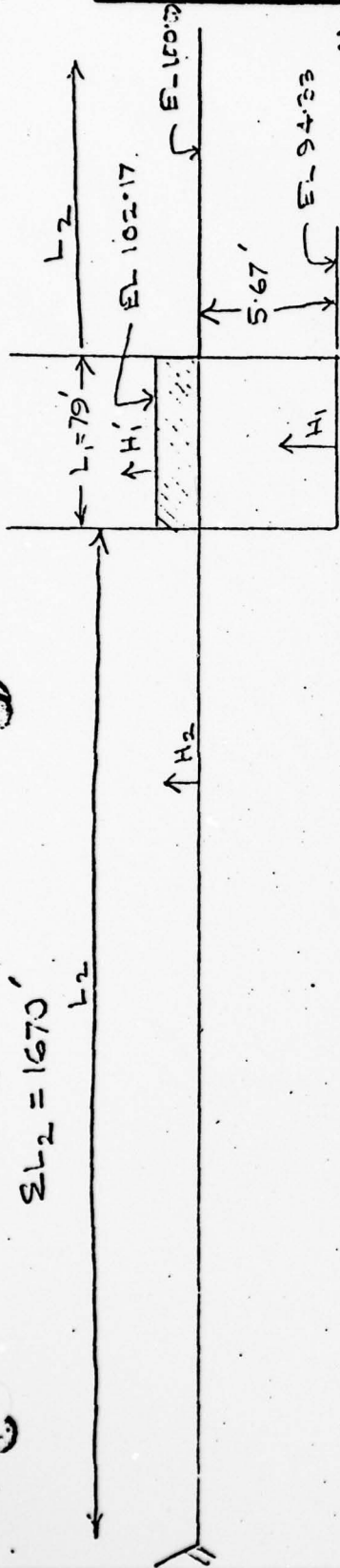


WOODCLIFF LAKE DAM  
SPILLWAY & OVERTOP RATING CURVE



WOODCLIFF LAKE DAM  
NET RESERVOIR CAPACITY CURVE  
ABOVE SPILLWAY CREST

WOODCREEK LAKE  
 SHEET NO. 1 OF  
 JOB NO. 100-0  
 BY SAAS DATE



$EL_2 = 1670'$

$Q = C_1 L_1 H_1^{1.5}$  with  $EL_{100}$   
 $Q = C_1 \sqrt{AV} H_1 + C_2 L_1 H_1^{1.5}$   
 $+ C_2 L_2 H_2^{1.5}$  at  $EL_{100}$

Elev.	H <sub>1</sub>	H <sub>1</sub> '	H <sub>2</sub>	L <sub>1</sub>	L <sub>1</sub> '	L <sub>2</sub>	C <sub>1</sub>	C <sub>1</sub> '	C <sub>2</sub>	Q
94.33	0									782000 = 782
96.33	2			79			3.5			2307000 = 2307
98.33	4			79			3.65			
100.00	5.67		0	79		0	3.80		0	4053000 = 4053
101.00	6.67		1	79		1670	*0.60		3.3	557000 + 5511 = 11,081
102.17	7.94	0	2.17	79	0	1670	*0.60	0	3.4	603900 + 18150 = 24,189
104	9.67	1.93	4	79	79	1670	*0.60	3.3	3.6	6707 + 645 + 48096 = 55,448
105	10.67	2.93	5	79	79	1670	*0.60	3.4	3.8	7045 + 1279 + 70950 = 79,274

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NEW JERSEY DAM SAFETY INSPECTION

SHEET NO. 1 OF 1

WOODCLIFF LAKE DAM

JOB NO. 1209-001

RESERVOIR AREA CAPACITY DATA

BY MAS DATE July 77

Lia

WOODCLIFF LAKE DAM

RESERVOIR AREA CAPACITY DATA

SUMMARY

Elevation (Feet)	Reservoir Surface Area (Acres)	Incremental Volume of Reservoir (Ac-Ft)	Net Vol. of Reservoir Above El. 94.33	Remarks
94.33	169	0	0	Area is obtained from existing Reservoir Wall Surface Elevation Vs Air Curves
95.00	172	114	114	22
95.50	176	87	201	23
100	217	884	1085	Area is obtained from USGS topo maps
110	344.8	2809	3894	23

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WOOD CLIFF DAM

SHEET NO. 1 OF

NEW COMPUTATION OF POINTS

JOB NO. 1209-001-1

ON Y2 & Y3 CARDS FOR HEC-1

BY KLB DATE 7-5-78

Ljm

#	TABULATION OF ELEVATION, HEAD ABOVE SPILLWAY (CREST ETC.)		Y2	Y3
	ELEVATION	HEIGHT ABOVE CREST	STORAGE (AC-FY)	DISCHARGE (CFS)
1	94.33	0'	0.	0.
2	97.0	2.67	509.	1200.
3	99.0	4.67	890.	3000.
4	99.5	5.17	985.	3600.
5	100.0	5.67	1080.	4900.
6	100.5	6.17	1221.	7000.
7	101.0	6.67	1361.	10000.
8	102.0	7.67	1643.	18200.
9	103.5	9.17	2064.	36000.
10	105.0	10.67	2486.	61800.

DOWN THE  
FRONT FACE OF  
CURVES

NOTE: CAPACITY CURVE IS TWO STRAIGHT LINES DEFINED BY THE FOLLOWING 3 POINTS

	ELEV. (FT)	CAP. (AC-FY)
SPILLWAY CREST	94.33	0
TOP OF DAM	100.00	1080
Limit of DATA	108	3330

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PROBABLE MAXIMUM FLOOD CALCULATION (PMF)

Drainable Area = 19.4 square miles

From Hydrometeorological Report #225

"Seasonal Variation of the Probable Maximum Precipitation East of the 105th Meridian for Areas from 10 to 1,000 square miles and Duration of 6, 12, 24 and 48 hours" 1956.

For Drainage Area 10 square miles

the 6 hour duration PMF is 25 inches for Zone "C" at Woodcliff Lake watershed.

Since the drainage area is larger than 10 square miles, an area reduction factor of 0.94 is applied.

The reduced 6 hour PMF is  $0.94 \times 25 = 23.75$  inches.

PMF values for rainfall durations of 6, 12, 24, 48 hours are:

Duration (Hrs)	PMF (inches)
6 hr	$1 \times 23.75 = 23.75$
12 hr	$1.09 \times 23.75 = 25.89$
24 hr	$1.17 \times 23.75 = 27.79$
48 hr	$1.27 \times 23.75 = 30.16$

PMF values shown above are reduced by 8.8% to account for misalignment of basin and rainfall isohyets.

the PMF for deriving PMF are therefore as following:

Duration (Hrs)	PMF (inches)
6	19.29
12	21.02
24	22.57
48	24.49

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NEW JERSEY DAM SAFETY INSPECTION

SHEET NO. 1 OF 2

WOODCLIFF LAKE DAM

JOB NO. 1209-001

PMP - PMF

BY MAS DATE 5/22

WOODCLIFF LAKE DAM.PMP - PMF DERIVATION

- 1) SOIL GROUP "C" & AMC-II  
 2) CN = 80

SOLUTION

1) SOIL GROUP "C"  $\Rightarrow$  0.12"/hr minim loss rate

2) CN = 80  $\Rightarrow$  S = 2.50 in

$$\text{Eq } Q = \frac{(P - 0.25)^2}{P + 0.85}$$

Thus

$$Q = \frac{(P - 0.50)^2}{P + 2.0}$$

See page 2 for the remainder  
 of the solution.

AD-A058 153

HARRIS ECI ASSOCIATES WOODBRIDGE NJ  
NATIONAL DAM SAFETY PROGRAM. WOODCLIFF LAKE DAM (NJ00247). HACK--ETC(U)  
JUN 78 R GERSHOWITZ

F/6 13/2

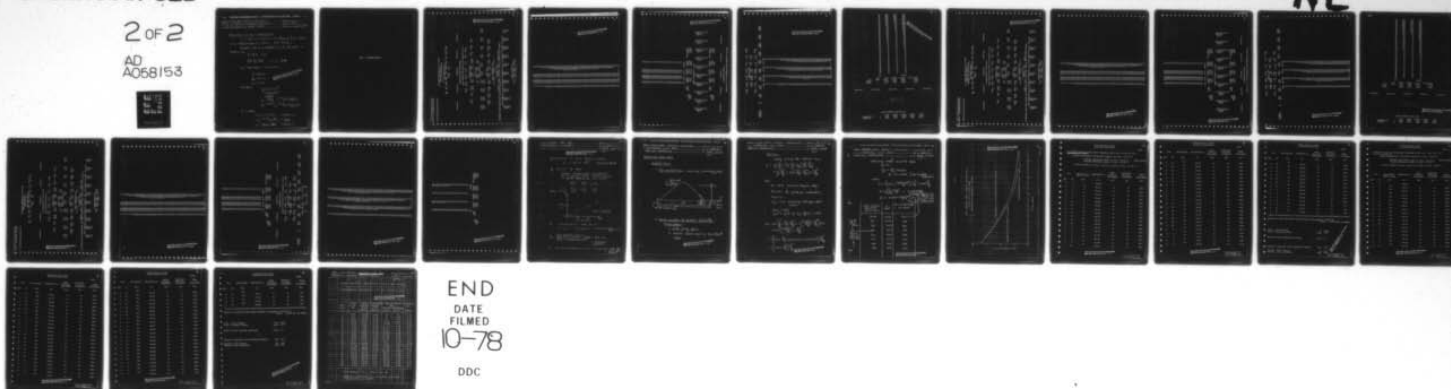
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UNCLASSIFIED

2 OF 2

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A058153



END  
DATE  
FILMED  
10-78

DDC

NEW JERSEY DAM SAFETY INSPECTION  
 PUF DERIVATION - Woodcliff Lake Dam  
 PROBABLE MAXIMUM FLOOD - UHG

SHEET NO. 2 OF \_\_\_\_\_  
 JOB NO. 1209  
 BY Y.N. DATE May 16 1952

### DERIVATION OF UNIT HYDROGRAPH

This dam is located on the Passaic River portion of the Hackensack River Basin. D.A. = 19.4 sq. mi.

Snyder method is adopted for the derivation of UHG with

$$C_k = 4.3 \text{ and}$$

$$640 C_p = 530, \text{ or } C_p = 0.828$$

From topographic map we have

$$L = 10.2 \text{ mi.}$$

$$L_c = 4.3 \text{ mi}$$

$$S = 0.0112$$

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Therefore

$$k_p = (C_k (L L_c))^{0.3}$$

$$= 4.3 (10.2 \cdot 4.3)^{0.3}$$

$$= \underline{13.37}$$

$$k_r = k_p / 5.5 = 13.37 / 5.5 = \underline{2.43 \text{ hr}}$$

$$q_p = 640 C_p / k_p = 530 / 13.37 = \underline{39.6 \text{ cfs}}$$

$$T_{in} = 2H_r$$

$$T_{pe} = T_p + 2.25 (k_p - k_r) = 13.37 \text{ hrs.}$$

$$Q_{pe} = 640 C_p / k_{pe} = 530 / 13.37 = 39.97$$

$$Q_p = 39.97 \cdot 19.4 = \underline{775.4 \text{ cfs}}$$

HEC 1 - COMPUTATIONS

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\*\*\*\*\*  
MEL-1 VISION DATED JAN 1973  
\*\*\*\*\*

DAM SAFETY INSPECTION - NEW JERSEY  
WOODCLIFF LAKE DAM  
PMF FLOOD ROUTING

0.21  
JOB SPECIFICATION  
NO NHR NMIN IDAY IHR IMIN METRC IPLT IPRT NSTAN  
90 1 0 0 0 0 0 0 0 0 0  
JOPEK 3  
0

\*\*\*\*\*  
\*\*\*\*\*  
\*\*\*\*\*

SUB-AREA RUNOFF COMPUTATION

INPUT SNYDER COEFFICIENTS THEN MULTIPLY BY 0.5

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME	ISNOW	ISAME	LOCAL
5	0	0	0	0	0	1	0	0	0

HYDROGRAPH DATA

IHYDG	IUHG	TAREA	SNAP	IRSDA	TKSPC	RATIO	ISNOW	ISAME	LOCAL
0	1	19.40	0.00	19.40	0.00	0.310	0	0	0

PRECIP DATA

NP	STORM	DAJ	DAK
24	0.00	0.00	0.00

LOSS DATA

STRKR	DLTKR	RTIOL	ERAIN	STIRKS	RTIOK	STRIL	CNSTL	ALSMX	RTIMP
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

UNIT HYDROGRAPH DATA  
TP= 13.37 CP=0.82 NTA= 0

RECESSION DATA

STRTOF	0.00	GRCSNF	0.00	RTIORE	1.00
--------	------	--------	------	--------	------

APPROXIMATE CLARK COEFFICIENTS FROM GIVEN SNYDER CP AND TP ARE TC=17.56 AND RE= 5.88 INTERVALS

UNIT HYDROGRAPH	42	END-OF-PERIOD	ORDINATES,	LAGE=	13.35	HOURS,	CP=	0.82	VOL=	0.99
18.	69.	137.	292.	372.	452.	530.	607.	674.		
723.	755.	769.	755.	726.	602.	612.	524.	442.		
373.	314.	265.	188.	159.	134.	113.	80.	80.		
68.	57.	48.	34.	29.	24.	20.	17.	14.		
12.	10.									

END-OF-PERIOD FLOW

TIME	1	0.00	0.00	0.00	0.00
------	---	------	------	------	------

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2	0.00	0.00	0.00
3	0.00	0.00	0.00
4	0.02	0.02	0.02
5	0.04	0.04	0.04
6	0.06	0.06	0.06
7	0.07	0.07	0.07
8	0.06	0.06	0.06
9	0.06	0.06	0.06
10	0.06	0.05	0.05
11	0.06	0.06	0.06
12	0.07	0.07	0.07
13	1.46	1.46	1.46
14	2.03	2.03	2.03
15	2.71	2.71	2.71
16	7.11	7.11	7.11
17	2.58	2.58	2.58
18	2.00	2.00	2.00
19	0.06	0.06	0.06
20	0.06	0.06	0.06
21	0.07	0.07	0.07
22	0.06	0.06	0.06
23	0.06	0.06	0.06
24	0.06	0.06	0.06
25	0.00	0.00	0.00
26	0.00	0.00	0.00
27	0.00	0.00	0.00
28	0.00	0.00	0.00
29	0.00	0.00	0.00
30	0.00	0.00	0.00
31	0.00	0.00	0.00
32	0.00	0.00	0.00
33	0.00	0.00	0.00
34	0.00	0.00	0.00
35	0.00	0.00	0.00
36	0.00	0.00	0.00
37	0.00	0.00	0.00
38	0.00	0.00	0.00
39	0.00	0.00	0.00
40	0.00	0.00	0.00
41	0.00	0.00	0.00
42	0.00	0.00	0.00
43	0.00	0.00	0.00
44	0.00	0.00	0.00
45	0.00	0.00	0.00
46	0.00	0.00	0.00
47	0.00	0.00	0.00
48	0.00	0.00	0.00
49	0.00	0.00	0.00
50	0.00	0.00	0.00
51	0.00	0.00	0.00
52	0.00	0.00	0.00
53	0.00	0.00	0.00
54	0.00	0.00	0.00
55	0.00	0.00	0.00
56	0.00	0.00	0.00
57	0.00	0.00	0.00
58	0.00	0.00	0.00
59	0.00	0.00	0.00
60	0.00	0.00	0.00
61	0.00	0.00	0.00
62	0.00	0.00	0.00



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ISTAQ 5  
 ICOMP 1  
 IECUN 0  
 IIAPE 0  
 JPLT 0  
 JPRT 0  
 INAME 1  
 NSTPS 0  
 NSTUL 0  
 LAG 0  
 AMSKK 0  
 X 0  
 TSK 0  
 STORA -1

STORAGE= 0. 509. 890. 905. 1080. 1221. 1361. 1643. 2064. 2486.  
 OUTFLOW= 0. 1200. 3000. 3600. 4900. 7000. 10000. 18200. 36000. 61800.

TIME	EOP	STOR	AVG IN	EOP	OUT
1	0.	0.	0.	0.	0.
2	0.	0.	0.	0.	0.
3	0.	0.	0.	0.	0.
4	0.	0.	0.	0.	0.
5	0.	0.	0.	0.	0.
6	0.	0.	1.	0.	0.
7	0.	0.	3.	0.	0.
8	0.	0.	6.	1.	1.
9	1.	1.	11.	3.	3.
10	2.	2.	18.	6.	6.
11	4.	4.	26.	9.	9.
12	6.	6.	35.	14.	14.
13	8.	8.	50.	20.	20.
14	13.	13.	83.	31.	31.
15	22.	22.	151.	53.	53.
16	39.	39.	281.	93.	93.
17	70.	70.	505.	166.	166.
18	120.	120.	823.	283.	283.
19	189.	189.	1209.	447.	447.
20	279.	279.	1633.	658.	658.
21	385.	385.	2072.	909.	909.
22	506.	506.	2511.	1193.	1193.
23	627.	627.	2938.	1758.	1758.
24	736.	736.	3338.	2274.	2274.
25	834.	834.	3693.	2757.	2757.
26	918.	918.	3979.	3181.	3181.
27	984.	984.	4180.	3594.	3594.
28	1021.	1021.	4290.	4093.	4093.
29	1032.	1032.	4312.	4252.	4252.
30	1032.	1032.	4248.	4249.	4249.
31	1024.	1024.	4096.	4139.	4139.
32	1009.	1009.	3856.	3934.	3934.
33	988.	988.	3531.	3643.	3643.
34	955.	955.	3137.	3415.	3415.
35	910.	910.	2719.	3127.	3127.
36	855.	855.	2322.	2837.	2837.
37	795.	795.	1970.	2554.	2554.
38	734.	734.	1669.	2265.	2265.
39	675.	675.	1413.	1986.	1986.
40	620.	620.	1194.	1728.	1728.
41	571.	571.	1009.	1493.	1493.
42	526.	526.	851.	1283.	1283.
43	485.	485.	718.	1145.	1145.
44	445.	445.	606.	1049.	1049.
45	404.	404.	511.	953.	953.
46	365.	365.	431.	861.	861.
47	327.	327.	363.	772.	772.
48	292.	292.	302.	691.	691.

49	260.	258.	613.
50	230.	217.	543.
51	203.	183.	479.
52	178.	154.	421.
53	156.	130.	369.
54	137.	109.	323.
55	119.	90.	282.
56	103.	71.	244.
57	89.	53.	210.
58	76.	32.	179.
59	63.	13.	149.
60	52.	5.	124.
61	43.	1.	102.
62	35.	1.	84.
63	29.	0.	69.
64	24.	0.	57.
65	20.	0.	47.
66	16.	0.	38.
67	13.	0.	31.
68	11.	0.	26.
69	9.	0.	21.
70	7.	0.	17.
71	6.	0.	14.
72	5.	0.	12.
73	4.	0.	9.
74	3.	0.	8.
75	2.	0.	6.
76	2.	0.	5.
77	1.	0.	4.
78	1.	0.	3.
79	1.	0.	3.
80	1.	0.	2.
81	0.	0.	2.
82	0.	0.	1.
83	0.	0.	1.
84	0.	0.	1.
85	0.	0.	0.
86	0.	0.	0.
87	0.	0.	0.
88	0.	0.	0.
89	0.	0.	0.
90	0.	0.	0.

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SUM 72440.

CFS	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
INCHES	4252.	4052.	2620.	1005.	72440.
AC-FT		1.94	5.02	5.78	5.78
		2010.	5200.	5988.	5989.

\*\*\*\*\*



1801 SOUTH MAYMO, DENVER, COLORADO 80202

RUNOFF SUMMARY, AVERAGE FLOW

HYDROGRAPH AT	PEAK	6-HOUR	24-HOUR	72-HOUR	AREA
ROUTED TO	4323.	4196.	2762.	1006.	19.40
	4252.	4052.	2620.	1005.	19.40

\*\*\*\*\*  
 REC-1 SIMON DATED JAN 1973  
 \*\*\*\*\*

DAM SAFETY INSPECTION - NEW JERSEY  
 WOODCLIFF LAKE DAM  
 ONE HALF PMF FLOOD ROUTING

JOB SPECIFICATION  
 NO NHR NMIN IDAY IHR IMIN METRC IPRT INSTAN  
 90 1 0 0 0 0 0 0 0 0  
 JOPEK 3 NWT 0

\*\*\*\*\*  
 \*\*\*\*\*  
 \*\*\*\*\*

SUB-AREA RUNOFF COMPUTATION

INPUT SNYDER COEFFICIENTS THEN MULTIPLY BY 0.5

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME	ISAME	LOCAL
5	0	0	0	0	0	1	0	0

HYDROGRAPH DATA		PRECIP DATA	
IHYDG	IAREA	NP	STORM
0	19.40	24	0.00
0.00	0.00	0.00	0.00
0.06	1.46	0.04	2.71
0.07	0.06	0.06	7.11

LOSS DATA		RECESSION DATA	
STRKH	DLTKR	STRKS	RTIOL
0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.00
0.06	0.07	0.07	2.58
0.07	0.06	0.06	2.00

UNIT HYDROGRAPH DATA  
 TP= 13.57 CP=0.82 NTA= 0

APPROXIMATE CLARK COEFFICIENTS FROM GIVEN SNYDER CP AND IP AKE TC=17.56 AND RE 5.88 INTERVALS

UNIT HYDROGRAPH	42	END-OF-PERIOD	ORDINATES,	LAG=	13.35	HOURS,	CP=	0.82	VOL=	0.99
187	69.	137.	292.	572.	452.	530.	607.	674.		
723.	755.	769.	755.	726.	682.	612.	524.	442.		
373.	314.	265.	188.	159.	134.	113.	95.	80.		
68.	57.	48.	54.	29.	24.	20.	17.	14.		
12.	10.									

END-OF-PERIOD FLOW  
 TIME RAIN EXCS COMP 0  
 1 0.00 0.00 0.00

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2	0.00	0.00	0.00	0.
3	0.00	0.00	0.00	0.
4	0.02	0.02	0.02	0.
5	0.04	0.04	0.04	2.
6	0.06	0.06	0.06	6.
7	0.07	0.07	0.07	15.
8	0.06	0.06	0.06	28.
9	0.06	0.06	0.06	46.
10	0.06	0.06	0.06	69.
11	0.06	0.06	0.06	97.
12	0.07	0.07	0.07	130.
13	1.46	1.46	1.46	194.
14	2.03	2.03	2.03	343.
15	2.71	2.71	2.71	632.
16	7.11	7.11	7.11	1185.
17	2.58	2.58	2.58	2075.
18	2.00	2.00	2.00	3235.
19	0.06	0.06	0.06	4565.
20	0.06	0.06	0.06	5970.
21	0.07	0.07	0.07	7397.
22	0.06	0.06	0.06	8805.
23	0.06	0.06	0.06	10151.
24	0.06	0.06	0.06	11387.
25	0.00	0.00	0.00	12440.
26	0.00	0.00	0.00	13233.
27	0.00	0.00	0.00	13734.
28	0.00	0.00	0.00	13945.
29	0.00	0.00	0.00	13879.
30	0.00	0.00	0.00	13532.
31	0.00	0.00	0.00	12896.
32	0.00	0.00	0.00	11983.
33	0.00	0.00	0.00	10798.
34	0.00	0.00	0.00	9444.
35	0.00	0.00	0.00	8100.
36	0.00	0.00	0.00	6883.
37	0.00	0.00	0.00	5832.
38	0.00	0.00	0.00	4938.
39	0.00	0.00	0.00	4177.
40	0.00	0.00	0.00	3530.
41	0.00	0.00	0.00	2980.
42	0.00	0.00	0.00	2514.
43	0.00	0.00	0.00	2121.
44	0.00	0.00	0.00	1789.
45	0.00	0.00	0.00	1509.
46	0.00	0.00	0.00	1272.
47	0.00	0.00	0.00	1073.
48	0.00	0.00	0.00	904.
49	0.00	0.00	0.00	762.
50	0.00	0.00	0.00	642.
51	0.00	0.00	0.00	541.
52	0.00	0.00	0.00	456.
53	0.00	0.00	0.00	384.
54	0.00	0.00	0.00	323.
55	0.00	0.00	0.00	260.
56	0.00	0.00	0.00	201.
57	0.00	0.00	0.00	146.
58	0.00	0.00	0.00	60.
59	0.00	0.00	0.00	28.
60	0.00	0.00	0.00	6.
61	0.00	0.00	0.00	4.

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63	0.00	0.00	0.00	0.00	2.
64	0.00	0.00	0.00	0.00	1.
65	0.00	0.00	0.00	0.00	0.
66	0.00	0.00	0.00	0.00	0.
67	0.00	0.00	0.00	0.00	0.
68	0.00	0.00	0.00	0.00	0.
69	0.00	0.00	0.00	0.00	0.
70	0.00	0.00	0.00	0.00	0.
71	0.00	0.00	0.00	0.00	0.
72	0.00	0.00	0.00	0.00	0.
73	0.00	0.00	0.00	0.00	0.
74	0.00	0.00	0.00	0.00	0.
75	0.00	0.00	0.00	0.00	0.
76	0.00	0.00	0.00	0.00	0.
77	0.00	0.00	0.00	0.00	0.
78	0.00	0.00	0.00	0.00	0.
79	0.00	0.00	0.00	0.00	0.
80	0.00	0.00	0.00	0.00	0.
81	0.00	0.00	0.00	0.00	0.
82	0.00	0.00	0.00	0.00	0.
83	0.00	0.00	0.00	0.00	0.
84	0.00	0.00	0.00	0.00	0.
85	0.00	0.00	0.00	0.00	0.
87	0.00	0.00	0.00	0.00	0.
88	0.00	0.00	0.00	0.00	0.
89	0.00	0.00	0.00	0.00	0.
90	0.00	0.00	0.00	0.00	0.

SUM 18.76 18.76 233683.

PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
13945.	13536.	8910.	3245.	233683.
CFS	6.49	17.08	18.67	18.67
INCHES	6715.	17682.	19322.	19322.
AC-FT				

RUNOFF MULTIPLIED BY 0.50

0.	0.	1.	3.	7.	14.	23.	34.
40.	65.	171.	316.	1037.	1617.	2282.	2985.
3698.	4402.	5693.	6220.	6867.	6972.	6939.	6766.
6448.	5991.	4722.	4050.	2916.	2469.	2088.	1765.
1490.	1257.	894.	754.	536.	452.	381.	321.
270.	220.	161.	130.	73.	30.	14.	3.
2.	1.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.

PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
6972.	6768.	4455.	1622.	116842.
CFS	3.24	8.54	9.33	9.33
INCHES	3357.	8841.	9661.	9661.
AC-FT				

\*\*\*\*\*

HYDROGRAPH ROUTING

ROUTE HALF OF PMF-HYDROGRAPH THRU WOODCLIFF LAKE DAM

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# NL

STORAGE= 0. 509. 890. 985. 1080. 1221. 1362. 1643. 2064. 2486.  
 OUTFLOW= 0. 1200. 3000. 3600. 4900. 7000. 10000. 18200. 36000. 61800.

ISTAG	ICOMP	IECUN	ITAPE	JPLT	JPRT	INAME
5	1	0	0	0	0	1
ROUTING DATA						
	GLOSS	AVG	IRES	ISAME		
	0.0	0.000	1	0		
NSTPS NSTDL LAG AMSKK X TSK STORA						
0	0	0	0.000	0.000	0.000	-1.
TIME EOP STOR AVG IN EOP OUT						
1	0	0	0	0	0	0
2	0	0	0	0	0	0
3	0	0	0	0	0	0
4	0	0	0	0	0	0
5	0	0	0	0	0	0
6	0	0	2	0	0	0
7	0	0	5	1	0	0
8	1	10	10	3	0	0
9	2	18	18	5	0	0
10	4	29	29	10	0	0
11	6	41	41	15	0	0
12	9	57	57	23	0	0
13	14	81	81	33	0	0
14	21	134	134	51	0	0
15	36	243	243	85	0	0
16	64	454	454	151	0	0
17	114	815	815	268	0	0
18	193	1327	1327	456	0	0
19	306	1950	1950	722	0	0
20	450	2653	2653	1061	0	0
21	612	3342	3342	1690	0	0
22	775	4050	4050	2461	0	0
23	931	4759	4759	3260	0	0
24	1053	5384	5384	4541	0	0
25	1127	5956	5956	5600	0	0
26	1168	6418	6418	6223	0	0
27	1195	6741	6741	6618	0	0
28	1210	6920	6920	6848	0	0
29	1216	6956	6956	6930	0	0
30	1212	6852	6852	6871	0	0
31	1198	6607	6607	6670	0	0
32	1175	6219	6219	6327	0	0
33	1143	5695	5695	5845	0	0
34	1103	5060	5060	5247	0	0
35	1058	4386	4386	4607	0	0
36	1013	3745	3745	3985	0	0
37	967	3178	3178	3486	0	0
38	915	2692	2692	3158	0	0
39	855	2279	2279	2837	0	0
40	792	1927	1927	2540	0	0
41	729	1627	1627	2242	0	0
42	669	1373	1373	1958	0	0
43	614	1159	1159	1697	0	0
44	564	977	977	1462	0	0
45	520	824	824	1253	0	0
46	479	695	695	1130	0	0
47	438	586	586	1033	0	0
48	397	494	494	937	0	0

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49	358.	416.	845.
50	321.	351.	757.
51	286.	296.	675.
52	254.	249.	600.
53	225.	210.	530.
54	198.	176.	468.
55	174.	145.	410.
56	152.	115.	358.
57	131.	86.	310.
58	112.	51.	264.
59	93.	22.	221.
60	77.	8.	183.
61	64.	2.	151.
62	52.	2.	124.
63	43.	1.	102.
64	35.	0.	84.
65	29.	0.	69.
66	24.	0.	57.
67	20.	0.	47.
68	16.	0.	36.
69	13.	0.	31.
70	11.	0.	26.
71	9.	0.	21.
72	7.	0.	17.
73	6.	0.	14.
74	5.	0.	12.
75	4.	0.	9.
76	3.	0.	8.
77	2.	0.	6.
78	2.	0.	5.
79	1.	0.	4.
80	1.	0.	3.
81	1.	0.	3.
82	1.	0.	2.
83	0.	0.	2.
84	0.	0.	1.
85	0.	0.	1.
86	0.	0.	1.
87	0.	0.	0.
88	0.	0.	0.
89	0.	0.	0.
90	0.	0.	0.

SUM 116839.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	6930.	6711.	4296.	1622.	116839.
INCHES		3.21	8.24	9.33	9.33
AC-FT		3329.	8526.	9659.	9661.

\*\*\*\*\*



FEDERAL BUREAU OF INVESTIGATION

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RUNOFF SUMMARY, AVERAGE FLOW

	PEAK	6-HOUR	24-HOUR	72-HOUR	AREA
HYDROGRAPH AT	5	6972.	6768.	4455.	1622.
ROUTED TO	5	6930.	6711.	4296.	1622.
					19.40
					19.40

\*\*\*\*\*  
REC-1 REVISION DATED JAN 1973  
\*\*\*\*\*

DAM SAFETY INSPECTION - NEW JERSEY  
WOODCLIFF LAKE DAM  
PMF ROUTING

JOB SPECIFICATION  
NQ NHR NMIN IDAY IMR IMIN METRC IPLT IPRT NSTAN  
90 1 0 0 0 0 0 0 0 0 0  
JUPEK 3 NWT 0  
3 0

\*\*\*\*\*  
\*\*\*\*\*  
\*\*\*\*\*  
\*\*\*\*\*

SUB-AREA RUNOFF COMPUTATION

INPUT SNYDER COEFFICIENTS

ISTAG ICOMP IECUN IIAPE JPLT JPRT INAME  
5 0 0 0 0 0 1  
IHYDG IUHG TAREA SNAP IRSUA TRSPC RATIO ISNOW ISAME LOCAL  
0 1 19.40 0.00 19.40 0.00 0.000 0 0 0  
HYDROGRAPH DATA

PRECIP DATA

NP STORM DAK  
24 0.00 0.00 0.00  
PRECIP PATIEKN  
0.00 0.04 0.06 0.07 0.06 0.06  
0.06 0.07 1.46 2.03 2.71 7.11 2.58 2.00 0.06 0.06  
0.07 0.06 0.06

LOSS DATA

STRKR DLTKR RTIOL ERAIN SIRKS RTIOK STRTL CNSTL ALSMX RTIMP  
0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00 0.00

UNIT HYDROGRAPH DATA  
TP= 13.37 CP=0.82 NTA= 0

RECESSION DATA

STRTQE 0.00 WRCSN= 0.00 KTIOR= 1.00  
APPROXIMATE CLARK COEFFICIENTS FROM GIVEN SNYDER CP AND TP ARE TC=17.56 AND R= 5.88 INTERVALS

UNIT HYDROGRAPH 42 END-OF-PERIOD ORDINATES, LAG= 13.35 HOURS, CP= 0.82 VOL= 0.99  
18. 69. 137. 213. 292. 372. 452. 530. 607. 674.  
723. 759. 769. 759. 726. 682. 612. 524. 442.  
373. 314. 265. 188. 159. 134. 113. 95. 80.  
60. 57. 48. 34. 29. 24. 17. 14.  
12. 10.

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END-OF-PERIOD FLOW  
TIME 1 RAIN EXCS COMP 0  
1 0.00 0.00 0.00

1	0.00	0.00	0.00	0.00
2	0.00	0.00	0.00	0.00
3	0.02	0.02	0.02	0.02
4	0.04	0.04	0.04	0.04
5	0.06	0.06	0.06	0.06
6	0.07	0.07	0.07	0.07
7	0.06	0.06	0.06	0.06
8	0.06	0.06	0.06	0.06
9	0.06	0.06	0.06	0.06
10	0.06	0.06	0.06	0.06
11	0.07	0.07	0.07	0.07
12	0.07	0.07	0.07	0.07
13	1.46	1.46	1.46	1.46
14	2.03	2.03	2.03	2.03
15	2.71	2.71	2.71	2.71
16	7.11	7.11	7.11	7.11
17	2.58	2.58	2.58	2.58
18	2.00	2.00	2.00	2.00
19	0.06	0.06	0.06	0.06
20	0.06	0.06	0.06	0.06
21	0.07	0.07	0.07	0.07
22	0.06	0.06	0.06	0.06
23	0.06	0.06	0.06	0.06
24	0.06	0.06	0.06	0.06
25	0.00	0.00	0.00	0.00
26	0.00	0.00	0.00	0.00
27	0.00	0.00	0.00	0.00
28	0.00	0.00	0.00	0.00
29	0.00	0.00	0.00	0.00
30	0.00	0.00	0.00	0.00
31	0.00	0.00	0.00	0.00
32	0.00	0.00	0.00	0.00
33	0.00	0.00	0.00	0.00
34	0.00	0.00	0.00	0.00
35	0.00	0.00	0.00	0.00
36	0.00	0.00	0.00	0.00
37	0.00	0.00	0.00	0.00
38	0.00	0.00	0.00	0.00
39	0.00	0.00	0.00	0.00
40	0.00	0.00	0.00	0.00
41	0.00	0.00	0.00	0.00
42	0.00	0.00	0.00	0.00
43	0.00	0.00	0.00	0.00
44	0.00	0.00	0.00	0.00
45	0.00	0.00	0.00	0.00
46	0.00	0.00	0.00	0.00
47	0.00	0.00	0.00	0.00
48	0.00	0.00	0.00	0.00
49	0.00	0.00	0.00	0.00
50	0.00	0.00	0.00	0.00
51	0.00	0.00	0.00	0.00
52	0.00	0.00	0.00	0.00
53	0.00	0.00	0.00	0.00
54	0.00	0.00	0.00	0.00
55	0.00	0.00	0.00	0.00
56	0.00	0.00	0.00	0.00
57	0.00	0.00	0.00	0.00
58	0.00	0.00	0.00	0.00
59	0.00	0.00	0.00	0.00
60	0.00	0.00	0.00	0.00
61	0.00	0.00	0.00	0.00
62	0.00	0.00	0.00	0.00

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63	0.00	0.00	0.00	2.
64	0.00	0.00	0.00	1.
65	0.00	0.00	0.00	0.
66	0.00	0.00	0.00	0.
67	0.00	0.00	0.00	0.
68	0.00	0.00	0.00	0.
69	0.00	0.00	0.00	0.
70	0.00	0.00	0.00	0.
71	0.00	0.00	0.00	0.
72	0.00	0.00	0.00	0.
73	0.00	0.00	0.00	0.
74	0.00	0.00	0.00	0.
75	0.00	0.00	0.00	0.
76	0.00	0.00	0.00	0.
77	0.00	0.00	0.00	0.
78	0.00	0.00	0.00	0.
79	0.00	0.00	0.00	0.
80	0.00	0.00	0.00	0.
81	0.00	0.00	0.00	0.
82	0.00	0.00	0.00	0.
83	0.00	0.00	0.00	0.
84	0.00	0.00	0.00	0.
85	0.00	0.00	0.00	0.
86	0.00	0.00	0.00	0.
87	0.00	0.00	0.00	0.
88	0.00	0.00	0.00	0.
89	0.00	0.00	0.00	0.
90	0.00	0.00	0.00	0.

SUM 18.76 18.76 233683.

PEAK 6-HOUR 24-HOUR 72-HOUR TOTAL VOLUME  
 13945. 13536. 8910. 3245. 255685.  
 INCHES 6.49 17.08 18.67 18.67  
 AC-FT 6715. 17682. 19322. 19322.

\*\*\*\*\*  
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 \*\*\*\*\*  
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HYDROGRAPH ROUTING

ROUTE PMF HYDROGRAPH THRU WOODCLIFF LAKE DAM

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRI	INAME
5	1	0	0	2	0	1
ROUTING DATA						
GLOSS	CLOSS	AVG	IRES	ISAME		
0.0	0.000	0.00	1	0		

NSTPS	NSTD L	LAG	AMSKK	X	TSK	STOKA
0	0	0	0.000	0.000	0.000	-1.

STORAGE= 0. 509. 890. 985. 1080. 1221. 1361. 1643. 2064. 2486.  
 CUTFLOW= 0. 1200. 3000. 3600. 4900. 7000. 10000. 10200. 18200. 36000. 61800.

TIME	EOP	STOR	AVG	IN	EOP	OUT
1	0.	0.	0.	0.	0.	0.
2	0.	0.	0.	0.	0.	0.
3	0.	0.	0.	0.	0.	0.

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5	0.	1.	0.	0.
6	0.	4.	4.	0.
7	1.	10.	10.	2.
8	2.	21.	21.	6.
9	4.	37.	37.	11.
10	8.	58.	58.	20.
11	13.	83.	83.	31.
12	19.	114.	114.	46.
13	28.	162.	162.	66.
14	43.	269.	269.	102.
15	72.	487.	487.	171.
16	128.	908.	908.	302.
17	228.	1630.	1630.	537.
18	387.	2855.	2855.	913.
19	604.	3900.	3900.	1648.
20	854.	5267.	5267.	2831.
21	1084.	6684.	6684.	4966.
22	1241.	8101.	8101.	7437.
23	1350.	9478.	9478.	9354.
24	1388.	10769.	10769.	10794.
25	1430.	11913.	11913.	12016.
26	1461.	12836.	12836.	12911.
27	1482.	13483.	13483.	13536.
28	1494.	13940.	13940.	13867.
29	1495.	14312.	14312.	13916.
30	1487.	14705.	14705.	13686.
31	1470.	15214.	15214.	13171.
32	1442.	15439.	15439.	12372.
33	1405.	15991.	15991.	11301.
34	1361.	16121.	16121.	10013.
35	1307.	16772.	16772.	8846.
36	1247.	17491.	17491.	7574.
37	1190.	18357.	18357.	6538.
38	1131.	19385.	19385.	5659.
39	1074.	20556.	20556.	4824.
40	1023.	21854.	21854.	4123.
41	975.	23255.	23255.	3540.
42	923.	24747.	24747.	3212.
43	863.	2618.	2618.	2875.
44	800.	1955.	1955.	2574.
45	736.	1649.	1649.	2272.
46	675.	1391.	1391.	1984.
47	618.	1175.	1175.	1719.
48	568.	989.	989.	1480.
49	523.	853.	853.	1269.
50	482.	702.	702.	1137.
51	441.	592.	592.	1040.
52	400.	498.	498.	944.
53	361.	420.	420.	851.
54	323.	353.	353.	762.
55	288.	291.	291.	679.
56	254.	230.	230.	594.
57	222.	173.	173.	524.
58	190.	103.	103.	449.
59	160.	44.	44.	377.
60	132.	17.	17.	313.
61	109.	5.	5.	258.
62	90.	4.	4.	213.
63	74.	2.	2.	176.
64	61.	1.	1.	145.

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65	50.	0.	119.
66	41.	0.	98.
67	34.	0.	00.
68	20.	0.	66.
69	23.	0.	54.
70	19.	0.	43.
71	15.	0.	37.
72	12.	0.	30.
73	10.	0.	25.
74	8.	0.	20.
75	7.	0.	16.
76	5.	0.	13.
77	4.	0.	11.
78	3.	0.	9.
79	3.	0.	7.
80	2.	0.	6.
81	2.	0.	5.
82	1.	0.	4.
83	1.	0.	3.
84	1.	0.	2.
85	1.	0.	2.
86	0.	0.	1.
87	0.	0.	1.
88	0.	0.	1.
89	0.	0.	1.
90	0.	0.	0.
SUM		233680.	

PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
13916.	13515.	8723.	3245.	233680.
CFS	6.48	16.73	18.67	18.67
INCHES	6705.	17312.	19319.	19322.
AC-FT				

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WOOD CLIFF LAKE DAM  
RESERVOIR EVACUATION

SHEET NO. 1 OF 3<sup>27</sup>  
JOB NO. 1209-001-1  
BY HLB DATE 7.14.79

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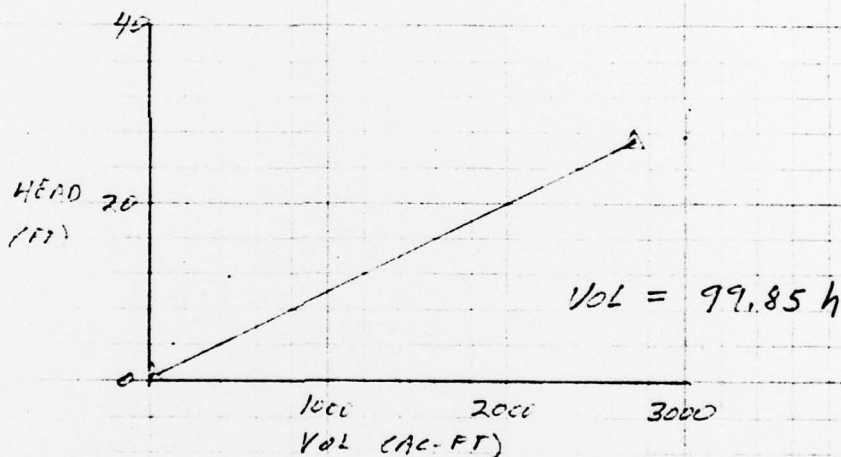
a) DISCHARGE VS. HEAD (FROM PREVIOUS WORK)

$$Q = 100.79 \sqrt{H} \quad 67 \leq H \leq 94.33$$

b) STORAGE VS. HEAD

ASSUME STRAIGHT LINE RELATIONSHIP  
FROM NORMAL WATER SURFACE VOLUME  
TO ZERO VOLUME AT ZERO HEAD.

	ELEV (FT)	HEAD (FT)	VOL. (AC-FT)
NWS.	94.33	27.33	2729
TOP OF ORIFICE	67.00	0	0



c) CATCHMENT AREA = 19.4 SQ. MI

$$I_{INFLOW} = 2 \text{ CFS/SQ MI} = 2 \times 19.4 = \underline{38.8 \text{ CFS}}$$

d) WITH CONSTANT INFLOW  
RESERVOIR EVACUATION TIME = 134 HR  
(FROM COMPUTER PRINTOUT)

$$= \underline{\underline{5.58 \text{ DAYS}}}$$

e) RESERVOIR EVACUATION TIME (0 INFLOW) = 108 HR  
= 4.50 DAYS

NEW JERSEY DAM SAFETY COMMISSION

SHEET NO. 1 OF 3

WOODCLIFF LAKE DAM

JOB NO. 1259-051

OUTLET CAPACITY

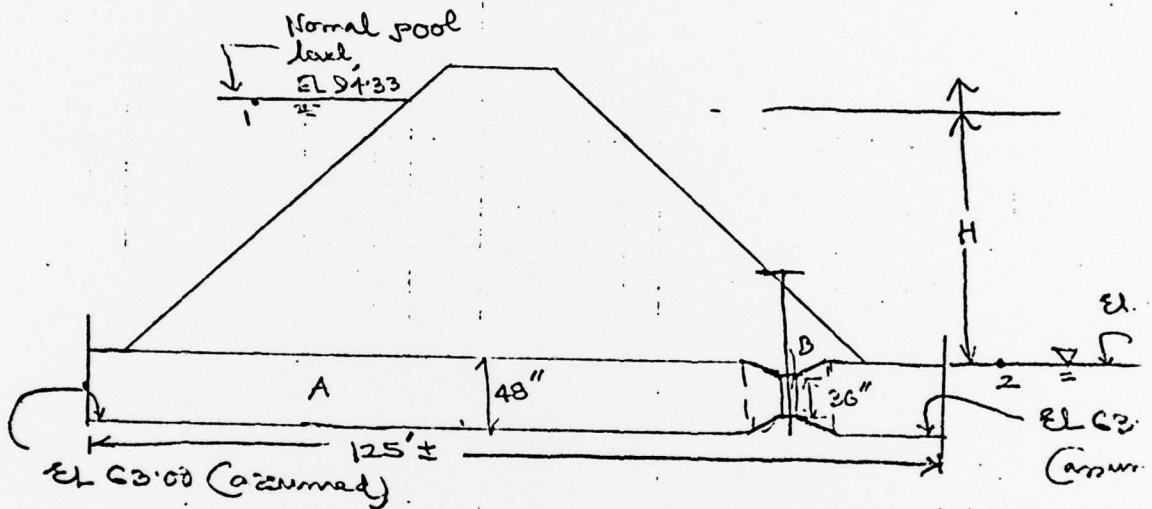
BY SAAS DATE 65

LR

WOODCLIFF LAKE DAM

Outlet Works

1. Dimensions (from drawings, dated Aug. 1953)



2. Determination of Outlet Capacity

Assumptions:

1. Gate fully open
2. Outlet submerged to the top of pipe

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U.S.

Solution:

Apply energy eqn. between 1 &amp; 2.

$$H = K_c \frac{V_A^2}{2g} + f \frac{L}{D} \cdot \frac{V_A^2}{2g} + K_c \left( \frac{V_B^2}{2g} - \frac{V_A^2}{2g} \right) \\ + K_g \frac{V_B^2}{2g} + K_{ex} \left( \frac{V_B^2}{2g} - \frac{V_A^2}{2g} \right) + \frac{V_A^2}{2g}$$

Now

$$K_c = 0.5 \text{ assuming square edge}$$

$$K_c = 0.1 \text{ for gradual contraction}$$

$$K_{ex} = 0.1$$

$$K_g = 0.19 \text{ assuming wide open gate value.}$$

$$V_B A_B = V_A A_A$$

$$V_B 3^2 = V_A 4^2 \Rightarrow V_B = \frac{16}{9} V_A = 1.78 V_A$$

$$\therefore H = 0.5 \frac{V_A^2}{2g} + \frac{fL}{D} \cdot \frac{V_A^2}{2g} + 0.1 \left( \frac{1.78^2 V_A^2}{2g} - \frac{V_A^2}{2g} \right) \times 2 \\ + 0.19 \left( \frac{1.78^2 V_A^2}{2g} \right) + \frac{V_A^2}{2g}$$

$$= \left[ 0.5 + \frac{fL}{D} + 0.2 \times 2.16 + 0.19 \times 3.16 + 1 \right] \frac{V_A^2}{2g}$$

$$= \left( 2.53 + \frac{fL}{D} \right) \frac{V_A^2}{2g}$$

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NEW JERSEY DAM SAFETY PROGRAM

SHEET NO. 3 OF 3

WINDLIFE LAKE DAM

JOB NO. 1269-001

OUTLET CAPACITY

BY MAS DATE 6-67

Assuming rough concrete pipe, L/m

$$\epsilon = .01$$

$$\frac{\epsilon}{D} = \frac{.01}{4} = 0.0025$$

$$\Rightarrow f = .0145 \quad (\text{for Complete turbulence})$$

\(\therefore\) thus

$$H = \left( 2.53 + \frac{.0145 \times 125}{4} \right) \frac{V_A^2}{2g} = 2.98 \frac{V_A^2}{2g}$$

$$V_A = \sqrt{\frac{1}{2.98} \sqrt{2gH}} = 0.58 \sqrt{2gH}$$

$$Q = 0.58 A \sqrt{2gH}$$

$$V = \frac{Q}{A} = \frac{527}{185 \times 4} = 42'$$

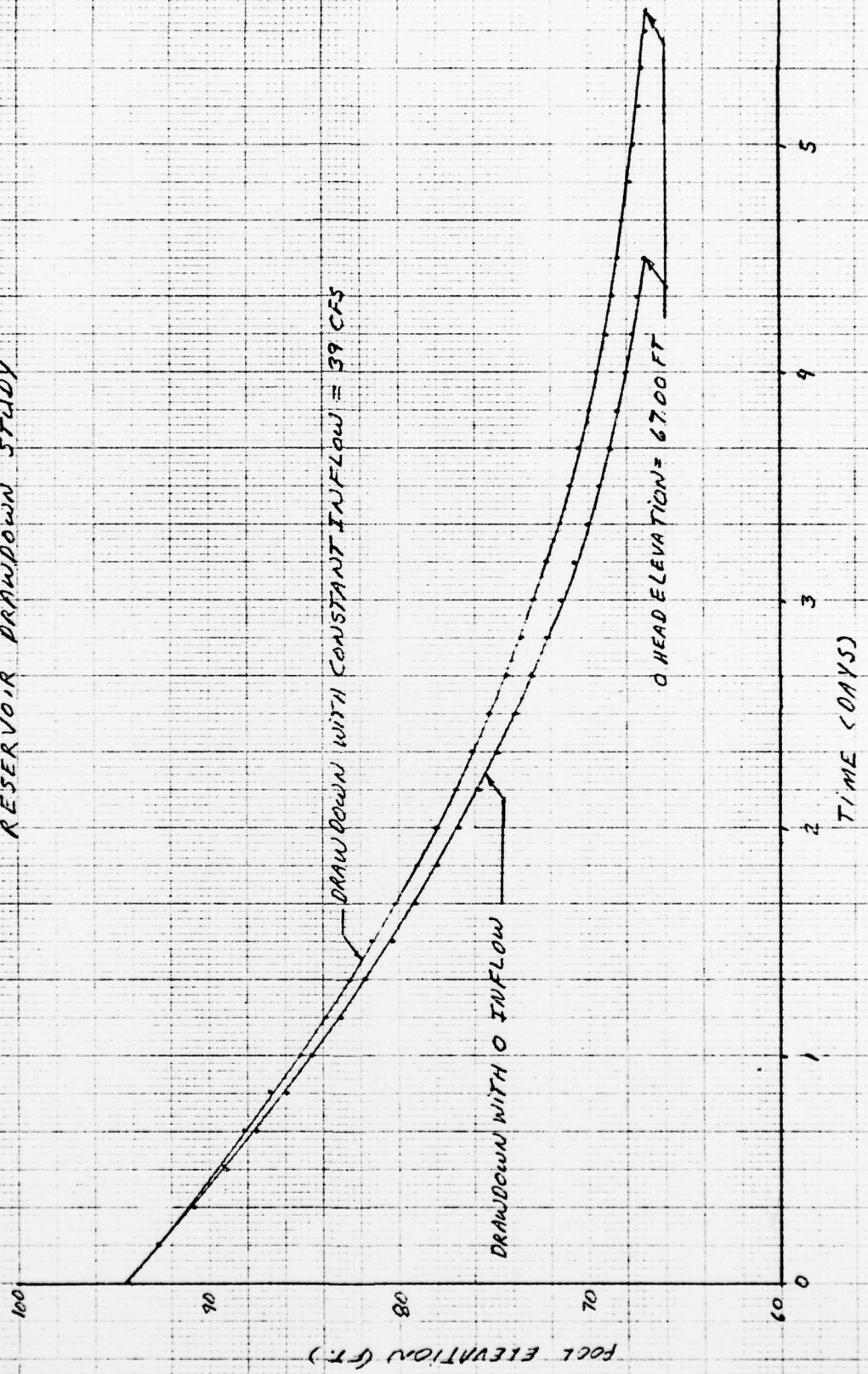
$$R = 42 \times 4 \times 1.5 = 1.68 \times 10^5$$

0.15 Complete turbulence

	Water Surface Elevation behind dam	H feet	$Q = 0.58 A \sqrt{2gH}$ $= 100.79 \sqrt{H}$
	94.33	27.33	527
	96	29.00	543
	98	31.00	561
	100	33.00	579
Top of dam ↓ El. 100	102	35.00	596
Assume dam will be raised ↓	104	37.00	613
	106	39.00	629
	108	41.00	645

NEW JERSEY DAM SAFETY INSPECTION  
WOOD CLIFF LAKE DAM  
RESERVOIR DRAWDOWN STUDY

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FLOOD ROUTING STUDY  
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WOOLCLIFF LAKE DAM RESERVOIR DRAWDOWN STUDY (DA = 19.4 SQ. MI.)

1.0000 UNREGULATED DIVERSION CONDUIT AT ELEV 67.00 FT

MAXIMUM OPERATION LEVEL AT ELEV 94.33 FT (FROM OPERAT

MINIMUM OPERATION LEVEL AT ELEV 67.00 FT

ROUTING STARTS AT ELEV 94.33 FT, ENDS AT ELEV 67.00 FT

TIME		AVG. INFLOW	RESERVOIR EL	MAIN SPILLWAY DISCHARGE	OVERFLOW SPILLWAY DISCHARGE	Outlet DISCHARGE
DAY	HR	CFS	FT	CFS	CFS	CFS
0	0	0.	94.33			
0	2	0.	93.43	0.	0.	318.
0	4	0.	92.54	0.	0.	510.
0	6	0.	91.66	0.	0.	501.
0	8	0.	90.80	0.	0.	492.
0	10	0.	89.96	0.	0.	483.
0	12	0.	89.13	0.	0.	474.
0	14	0.	88.32	0.	0.	466.
0	16	0.	87.52	0.	0.	457.
0	18	0.	86.74	0.	0.	448.
0	20	0.	85.97	0.	0.	439.
0	22	0.	85.22	0.	0.	430.
1	0	0.	84.48	0.	0.	421.
1	2	0.	83.76	0.	0.	413.
1	4	0.	83.05	0.	0.	404.
1	6	0.	82.36	0.	0.	396.
1	8	0.	81.68	0.	0.	387.
1	10	0.	81.02	0.	0.	378.

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FLOOD ROUTING STUDY  
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TIME		AVG. INFLOW	RESERVOIR EL	MAIN SPILLWAY DISCHARGE	OVERFLOW SPILLWAY DISCHARGE	Outlet DISCHARGE
DAY	HR	CFS	FT	CFS	CFS	CFS
1	12	0.	80.37	0.	0.	369.
1	14	0.	79.74	0.	0.	360.
1	16	0.	79.13	0.	0.	351.
1	18	0.	78.53	0.	0.	344.
1	20	0.	77.94	0.	0.	335.
1	22	0.	77.37	0.	0.	327.
2	0	0.	76.81	0.	0.	318.
2	2	0.	76.27	0.	0.	309.
2	4	0.	75.74	0.	0.	300.
2	6	0.	75.23	0.	0.	290.
2	8	0.	74.74	0.	0.	281.
2	10	0.	74.26	0.	0.	271.
2	12	0.	73.80	0.	0.	262.
2	14	0.	73.35	0.	0.	252.
2	16	0.	72.92	0.	0.	243.
2	18	0.	72.51	0.	0.	234.
2	20	0.	72.11	0.	0.	224.
2	22	0.	71.73	0.	0.	215.
3	0	0.	71.37	0.	0.	207.
3	2	0.	71.02	0.	0.	198.
3	4	0.	70.68	0.	0.	190.
3	6	0.	70.36	0.	0.	181.
3	8	0.	70.05	0.	0.	173.

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FLOOD ROUTING STUDY  
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TIME		AVG. INFLOW	RESERVOIR EL	MAIN SPILLWAY DISCHARGE	OVERFLOW SPILLWAY DISCHARGE	Outlet DISCHARGE
DAY	HR	CFS	FT	CFS	CFS	CFS
3	10	0.	69.76	0.	0.	166.
3	12	0.	69.48	0.	0.	158.
3	14	0.	69.21	0.	0.	151.
3	16	0.	68.96	0.	0.	143.
3	18	0.	68.72	0.	0.	137.
3	20	0.	68.49	0.	0.	130.
3	22	0.	68.27	0.	0.	124.
4	0	0.	68.06	0.	0.	118.
4	2	0.	67.86	0.	0.	113.
4	4	0.	67.67	0.	0.	107.
4	6	0.	67.49	0.	0.	102.
4	8	0.	67.31	0.	0.	97.
4	10	0.	67.15	0.	0.	93.
4	12	0.	67.00	0.	0.	88.

\*\*\*\*\*

RESERVOIR ELEVATION WENT UNDER MINIMUM WATERSURFACE ELEVATION  
AFTER 4 DAYS AND 12 HOURS.

TOTAL INFLOW VOLUME	0.	ACFT
TOTAL DISCHARGE VOLUME	2730.	ACFT
MAXIMUM WATER SURFACE ELEVATION	94.33	FT
MAXIMUM DISCHARGE THRU DIVERSION CONDUIT	518.	CFS
MAXIMUM TOTAL INFLOW	0.	CFS
MAXIMUM TOTAL DISCHARGE	527.	CFS

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WOODCLIFF LAKE DAM RESERVOIR DRAWDOWN STUDY (DA = 19.4 SQ. MI.)

1.0000 UNREGULATED DIVERSION CONDUIT AT ELEV 67.00 FT

MAXIMUM OPERATION LEVEL AT ELEV 94.33 FT (FROM OPERAT:  
 MINIMUM OPERATION LEVEL AT ELEV 67.00 FT

ROUTING STARTS AT ELEV 94.33 FT, ENDS AT ELEV 67.00 FT

TIME		AVG. INFLOW	RESERVOIR EL	MAIN SPILLWAY DISCHARGE	OVERFLOW SPILLWAY DISCHARGE	Outlet DISCHARGE
DAY	HR	CFS	FT	CFS	CFS	CFS
0	0		94.33			
0	2	39.	93.49	0.	0.	519.
0	4	39.	92.67	0.	0.	511.
0	6	39.	91.86	0.	0.	503.
0	8	39.	91.06	0.	0.	495.
0	10	39.	90.28	0.	0.	486.
0	12	39.	89.51	0.	0.	478.
0	14	39.	88.76	0.	0.	470.
0	16	39.	88.02	0.	0.	462.
0	18	39.	87.29	0.	0.	454.
0	20	39.	86.58	0.	0.	446.
0	22	39.	85.88	0.	0.	438.
1	0	39.	85.20	0.	0.	430.
1	2	39.	84.53	0.	0.	422.
1	4	39.	83.87	0.	0.	414.
1	6	39.	83.23	0.	0.	407.
1	8	39.	82.60	0.	0.	399.
1	10	39.	81.98	0.	0.	391.

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FLOOD ROUTING STUDY

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TIME		AVG. INFLOW	RESERVOIR EL	MAIN SPILLWAY DISCHARGE	OVERFLOW SPILLWAY DISCHARGE	Outlet DISCHARGE
DAY	HR	CFS	FT	CFS	CFS	CFS
1	12	39.	81.38	0.	0.	383.
1	14	39.	80.79	0.	0.	375.
1	16	39.	80.22	0.	0.	367.
1	18	39.	79.66	0.	0.	359.
1	20	39.	79.11	0.	0.	351.
1	22	39.	78.58	0.	0.	344.
2	0	39.	78.05	0.	0.	337.
2	2	39.	77.55	0.	0.	329.
2	4	39.	77.05	0.	0.	322.
2	6	39.	76.57	0.	0.	314.
2	8	39.	76.10	0.	0.	306.
2	10	39.	75.64	0.	0.	298.
2	12	39.	75.20	0.	0.	290.
2	14	39.	74.77	0.	0.	282.
2	16	39.	74.36	0.	0.	274.
2	18	39.	73.96	0.	0.	265.
2	20	39.	73.58	0.	0.	257.
2	22	39.	73.21	0.	0.	249.
3	0	39.	72.85	0.	0.	241.
3	2	39.	72.50	0.	0.	233.
3	4	39.	72.17	0.	0.	226.
3	6	39.	71.86	0.	0.	218.
3	8	39.	71.55	0.	0.	211.

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FLOOD ROUTING STUDY  
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TIME		AVG. INFLOW	RESERVOIR EL	MAIN SPILLWAY DISCHARGE	OVERFLOW SPILLWAY DISCHARGE	Outlet DISCHARGE
DAY	HR	CFS	FT	CFS	CFS	CFS
		39.				
3	10	39.	71.26	0.	0.	204.
3	12	39.	70.98	0.	0.	197.
3	14	39.	70.71	0.	0.	190.
3	16	39.	70.46	0.	0.	184.
3	18	39.	70.21	0.	0.	177.
3	20	39.	69.98	0.	0.	171.
3	22	39.	69.75	0.	0.	165.
4	0	39.	69.54	0.	0.	159.
4	2	39.	69.33	0.	0.	154.
4	4	39.	69.14	0.	0.	148.
4	6	39.	68.95	0.	0.	143.
4	8	39.	68.78	0.	0.	138.
4	10	39.	68.61	0.	0.	134.
4	12	39.	68.45	0.	0.	129.
4	14	39.	68.30	0.	0.	125.
4	16	39.	68.15	0.	0.	121.
4	18	39.	68.01	0.	0.	117.
4	20	39.	67.88	0.	0.	113.
4	22	39.	67.75	0.	0.	110.
5	0	39.	67.63	0.	0.	106.
5	2	39.	67.52	0.	0.	103.
5	4	39.	67.41	0.	0.	100.
5	6	39.	67.31	0.	0.	97.

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FLOOD ROUTING STUDY  
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TIME		AVG. INFLOW	RESERVOIR EL	MAIN SPILLWAY DISCHARGE	OVERFLOW SPILLWAY DISCHARGE	Outlet DISCHARGE
DAY	HR	CFS	FT	CFS	CFS	CFS
		39.				
5	8		67.21	0.	0.	94.
5	10	39.	67.11	0.	0.	92.
5	12	39.	67.02	0.	0.	89.
5	14	39.	67.00	0.	0.	87.

\*\*\*\*\*

RESERVOIR ELEVATION WENT UNDER MINIMUM WATERSURFACE ELEVATION  
AFTER 5 DAYS AND 14 HOURS.

TOTAL INFLOW VOLUME 449. ACFT  
TOTAL DISCHARGE VOLUME 3184. ACFT

MAXIMUM WATER SURFACE ELEVATION 94.33 FT

MAXIMUM DISCHARGE THRU DIVERSION CONDUIT 519. CFS

MAXIMUM TOTAL INFLOW 39. CFS  
MAXIMUM TOTAL DISCHARGE 527. CFS

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TRCOT

Lim

PMP RAINFALL DERIVATION (MAX 24 HOURS)

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TIME (HR.)	TOTAL 24 HR %	TOTAL RAINFALL DEPTH (INCH)	INCREMENTAL RAINFALL DEPTH (INCH)	DIRECT RUNOFF (PMF)		INCREMENTAL LOSS (INCH)
				ACCUM. (INCH)	INCREMENTAL (INCH)	
1	0.81	0.18	0.18	0	0	0.18
2	1.61	0.36	0.18	0	0.00	0.18
3	2.42	0.55	0.19	0	0.00	0.19
4	3.23	0.73	0.18	0.02	0.02	0.16
5	4.04	0.91	0.18	0.06	0.04	0.14
6	4.84	1.09	0.18	0.11	0.06	0.12
7	5.65	1.28	0.19	0.19	0.07	0.12
8	6.46	1.46	0.18	0.27	0.06	0.12
9	7.27	1.64	0.18	0.36	0.06	0.12
10	8.07	1.82	0.18	0.46	0.06	0.12
11	8.88	2.00	0.18	0.56	0.06	0.12
12	9.69	2.19	0.19	0.68	0.07	0.12
13	18.24	4.12	1.93	2.14	1.46	0.47
14	28.50	6.43	2.31	4.17	2.03	0.28
15	41.32	9.33	2.90	6.88	2.71	0.19
16	73.80	16.66	7.33	13.99	7.11	0.22
17	85.76	19.36	2.70	16.65	2.58	0.12
18	95.16	21.48	2.12	18.75	2.00	0.12
19	95.97	21.66	0.18	18.92	0.06	0.12
20	96.78	21.84	0.18	19.10	0.06	0.12
21	97.59	22.03	0.19	19.29	0.07	0.12
22	98.39	22.21	0.18	19.47	0.06	0.12
23	99.20	22.39	0.18	19.65	0.06	0.12
24	100.00	22.57	0.18	19.82	0.06	0.12

SOIL GROUP "C" , USING CN 80

\* MINIMUM LOSS RATE OF 0.12"/HR USED FOR REMAINDER OF STORM