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NATIONAL DAM SAFETY PROGRAM. POMPTON LAKE DAM (NJ00249), PASSAI--ETC(U)
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LEVEL II

PASSAIC RIVER BASIN

RAMAPO RIVER, PASSAIC COUNTY

NEW JERSEY

POMPTON LAKE DAM

PHASE I INSPECTION REPORT

NATIONAL DAM SAFETY PROGRAM

NJ 00249

DEPARTMENT OF THE ARMY
PHILADELPHIA DISTRICT, CORPS OF ENGINEERS
CUSTOM HOUSE - 2D & CHESTNUT STREETS
PHILADELPHIA, PENNSYLVANIA 19106
FEBRUARY 1978

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REPORT DOCUMENTATION PAGE

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17. DISTRIBUTION ST

18. SUPPLEMENTARY NOTES

Copies are obtainable from National Technical Information Service, Springfield, Virginia, 22151.

19. KEY WORDS (*Continue on reverse side if necessary and identify by block number*)

National Dam Safety Program
Dam Inspection Report Phase I
Pompton Lake Dam, N.J.
Dams - N.J.

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.

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

IN REPLY REFER TO
NAPEN-D

06 JUN 1978

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

Dear Governor Byrne:

Inclosed is the Phase I Inspection Report for Pompton Lakes Dam in Passaic 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 pages 1, 2, and 2a of the report. This assessment indicates that definite conclusions as to safety of the dam cannot be reached until a more detailed investigation is made. A complete visual inspection of the concrete structure was not made due to inability to stop the flow of water over the spillway and the lack of existing engineering data relative to the dam's design, construction and operation. Lack of existing data also prevented determination of the structure's stability and operational adequacy. In order to provide a more complete analysis, it is recommended that within one year from the approval date of this report, the owner obtain an engineering investigation to include the following as a minimun:

a. Detailed inspection of the concrete structure with the pool lowered. Drilling and sampling of the concrete dam if considered necessary as a result of this inspection.

b. Drilling and sampling at the abutments and rock ledge in the stream to provide for a more complete stability analysis.

c. More detailed structural and hydraulic analysis of the dam, including the safety of the abutment areas during high water stages which result in overtopping these areas.

d. Determine the extent of the reported deep scour hole 150 feet downstream of the dam, and its possible effects on the structure. Also periodic surveys and monitoring of this scour hole to insure that it poses no threat in the future.

e. Provisions for protecting the Route 202 highway embankment located at the east end of the dam. Further erosion of this embankment, especially during periods of high flows, could endanger the roadway and the public using it.

f. Improving the drawdown facilities so the pool can be lowered during emergencies, or for inspection or remedial work.

Two copies of the report are 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 Robert A. Roe of the Eighth 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.

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

1 Incl
As stated

HARRY V. DUTCHYSHYN
Colonel, Corps of Engineers
District Engineer

Cy Furn: w/incl (dupe)
Mr. Dirk C. Hofman
N.J. Dept. of Environmental Protection

PASSAIC RIVER BASIN **LEVEL II**

Name of Dam: Pompton Lakes Dam
County and State: Passaic County, State of New Jersey
Inventory Number: NJ00249

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

Prepared by: O'Brien and Gere Engineers, Inc.
Justin and Courtney Division

For: United States Army Engineer District, Philadelphia
United States Custom House
2nd and Chestnut Street
Philadelphia, Pennsylvania 19106

Date February 9, 1978

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PHASE 1 REPORT
NATIONAL DAM SAFETY PROGRAM

Name of Dam Pompton Lakes Dam

State Located New Jersey
County Located Passaic County
Stream Ramapo River
Date of Inspection January 6, 1978

ASSESSMENT OF
GENERAL CONDITIONS

The Pompton Lakes dam is a concrete overflow spillway for its entire length. A complete examination of the structure could not be made at the time of the inspection since four to six inches of water was flowing over the spillway.

The geometric shape of the spillway does not conform to present standards to insure proper hydraulic performance and structural stability. The shape of the spillway crest is narrow and the downstream slope is very steep. The resulting geometry is such that negative pressures could occur on the downstream face of the spillway when the reservoir water surface elevation is more than 1.3 ft. above the spillway crest. A stability analysis of the spillway reveals tension in the upstream face of the structure under normal loading (full reservoir). However, in plan view, the dam is curved in a modified arch shape between abutments and is supported at midstream by a massive basalt rock outcrop; both of these conditions improve the stability of the spillway. Factors of safety against overturning and sliding, including shear, are as follows (not including the effect of dam curvature or negative pressures on the downstream face):

Condition	Factors of Safety Overturning	Sliding
Normal Pool (Reservoir at spillway crest)	1.14	5.23
Normal Pool & .05 g earthquake load	1.06	4.66
Normal Pool & 5 KSF ice load	0.93	4.57
Probable Maximum Flood (PMF)	0.78	5.38

The present drawdown facilities are inadequate to provide a reliable means of lowering the reservoir level. Consideration should be given to improving drawdown facilities.

During the inspection, sloughing of the Route 202 highway embankment was observed at the east end of the spillway. Consideration should be given to the construction of a wall at this location to protect the highway and the spillway. It was also reported that an 18 foot deep scour hole exists in the stream channel downstream of the dam. This scour hole could not be examined during the field inspection due to the flow in the river. The location of this scour hole should be determined by a field survey and monitored periodically to determine if it affects the dam foundation.

A Phase I Investigation, in this case, cannot support conclusive findings due to:

1. Lack of complete data.
2. High water at the time of the inspection.
3. Evidence that observed defects and deficiencies may go deeper than a visual inspection reveals.
4. The need for more detailed structural and hydraulic analyses to determine the integrity of the dam during high flows.

In view of these factors, further investigations as outlined in the National Program of Inspection of Dams, Volume I, Appendix D, Chapter 4, paragraph 4.4, are recommended.

Definite conclusions regarding the safety of the Pompton Lakes Dam cannot be reached until a more detailed investigation is made.

John W. Williams

A complete visual inspection of the concrete structure could not be made due to inability to stop flow of water over the spillway and the lack of existing engineering data relative to the dam's design, construction and operation. Lack of existing data also prevented determination of the dam's stability and operational adequacy. In order to provide a more complete analysis, it is recommended that, within one year from the approval date of this report, the owner obtain an engineering investigation of the structure to include the following as a minimum:

a. Detailed inspection of the concrete structure with the pool lowered. Drilling and sampling of the concrete dam if considered necessary as a result of this inspection.

b. Drilling and sampling at the abutments and rock ledge in the stream to provide for a more complete stability analysis.

c. More detailed structural and hydraulic analysis of the dam including the safety of the abutment areas during high water stages which result in overtopping these areas.

d. Determine the extent of the reported deep scour hole 150 feet downstream of the dam and its possible effects on the structure. Also periodic surveys and monitoring of this scour hole to insure it poses no threat in the future.

e. Provisions for protecting the Route 202 highway embankment located at the east end of the dam. Further erosion of this embankment, especially during periods of high flows, could endanger the roadway and the public using it.

f. Improving the drawdown facilties so the dam's pool can be lowered during an emergency or for inspection or remedial work.

APPROVED: Harry V. Dutchyshyn
HARRY V. DUTCHYSHYN
Colonel, Corps of Engineers
District Engineer

DATE: 1 June 1978

OVERALL VIEW OF DAM

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM
NAME OF DAM POMPTON LAKE DAM ID # 00249

SECTION 1 - PROJECT INFORMATION

1.1 GENERAL

a. Authority. This report is authorized by the Dam Inspection Act, Public Law 92-367, and has been prepared in accordance with contract # DACW61-78-C-0052 between O'Brien and Gere Engineers, Justin and Courtney Division, and the United States Army Corps of Engineers, Philadelphia District.

b. Purpose of Inspection. The purpose of this inspection is to evaluate the structural and hydraulic condition of the Pompton Lakes Dam and appurtenant structures, and to determine if the dam consistutes a hazard to human life or property.

1.2 DESCRIPTION OF PROJECT

a. Description of Dam and Appurtenances. The dam at Pompton Lakes is principally a concrete gravity spillway structure. The spillway section is approximately 290 feet long, with a maximum height of 30 feet and is shaped by compound circular curves to create an ogee section. On the east end, the spillway is tied into the rock outcrop, and is now covered by the embankment of U.S. Route 202. The west end of the spillway is tied into a wing wall of a pump station intake.

The intake to the pump station consists of a concrete headwall with two 72" gated openings leading to a forebay, a screen chamber, and a 78" pump suction line. The pump station is located 200 feet downstream of the wing wall. A 30" diameter gated opening is located in the forebay, 9 feet below the crest of the dam. This opening is the only means of drawing down the reservoir without going through the pump station. Refer to Figure 1 through 3.

b. Location. Pompton Lakes Dam is located on the Ramapo River about 1.5 miles upstream of its confluence with the Pompton River. The drainage area of 160 square miles is about 35 miles long and an average of 4.5 miles wide, and is located in the States of New York and New Jersey (see Figure 4).

c. Size Classification. The maximum height of the dam is 30 feet. The reservoir volume to the spillway crest is about 2000 acre feet. Therefore, the dam is in the intermediate size category as defined by the Recommended Guidelines for Safety Inspection of Dams.

d. Hazard Classification. Due to population concentrations downstream, several thousand people's lives would be endangered in the event of failure of the dam. Therefore, the dam is in the high hazard category as defined by the Recommended Guidelines for Safety Inspection of Dams.

e. Ownership. The dam is owned by the North Jersey District Water Supply Commission of the State of New Jersey, Wanaque, New Jersey.

f. Purpose of Dam. The dam is used to store water for pumping into the Wanaque Reservoir, located 4 miles north of the dam and 100 feet higher. A secondary purpose of the reservoir is recreation.

g. Design and Construction History. The present structure was built in 1908 to replace a wooden crib structure which was washed out by a flood in October of 1903. Approximately 50 tons of steel rods are said to have been used to reinforce the spillway. The wing wall and intake to the pump station were rebuilt in the 1950's. New sluice gates and trash racks were installed at that time.

h. Normal Operational Procedures. Under normal conditions, the water surface elevation of the reservoir is at or above the spillway crest. A minimum discharge of 40 million gallons per day (mgd) is required by the State of New Jersey to be maintained in the downstream channel, when inflow to the reservoir equals or exceeds this amount.

The outlet facilities consist of a 30" pipe in the forebay and a 78" pipe leading to the pump station. The capacity of the 30" pipe, with the reservoir water surface at the spillway crest, is about 60 cubic feet per second (cfs). The capacity of the pump station is 150 cfs (100 mgd). The fifty-six year mean river discharge is 300 cfs. Therefore, the reservoir can be drawn down only during extended periods of low flow with the existing outlet facilities.

1.3 PERTINENT DATA

a. Drainage Area. The drainage area of the Pompton Lakes Reservoir is 160 square miles.

b. Discharge at Damsite. The maximum known flow at the damsite was 12,300 cfs. The spillway capacity is about 13,000 cfs with the reservoir water surface at the elevation of U.S. Route 202 at the dam.

c. <u>Elevation (ft. above MSL).</u>
Top of dam - 202.1
Maximum pool - design discharge - 212.0 (PMF)
Recreation pool - 202.1
Streambed at centerline of dam - 170.6
Maximum tailwater - 196.0 (PMF)

d. <u>Reservoir (miles).</u>
Length of Maximum Pool - 4.36
Length of Recreation Pool - 1.61

e. <u>Storage (acre-feet)</u>
Recreation pool - 2000
Design surcharge - 3650 (PMF)
Top of dam - 2000

f. <u>Reservoir Surface (acres)</u>
Top of dam - 210
Maximum pool - 400 (PMF)
Recreation pool - 210
Spillway crest - 210

g. <u>Dam</u>
Type - concrete gravity overflow
Length - 290 feet
Height - 30 feet (maximum)
Grout curtain - no information available

h. <u>Diversion and Regulating Tunnel</u>
None exists.

i. <u>Spillway</u>
Type - ogee
Length of weir - 290 feet
Crest elevation - 202.1 (MSL)
Gates - none

j. <u>Regulating Outlets</u>
30" pipe
78" pipe to pump station

SECTION 2 - ENGINEERING DATA

2.1 DESIGN

The available data relative to the dam and the appurtenant works consist of the following:

a. One drawing entitled PLANS FOR A MASONRY DAM AT POMPTON LAKES, N.J. dated March 18, 1918 (Figure 1 in appendix).

b. Plans & Sections of the Intake & Pump Station by NJWSD.

Ramapo Force Main Pumping Station Drawings:

1) Plot, Plan, Penstock & Screen Chambers, Sheet number P-1.

2) Pumps & Piping, Sheet number P-2.

3) Equipment Details, Sheet number P-3.

4) Engineering Structures - Penstock, Etc., Sheet number P-5.

5) Engineering Structures - Screen Chambers & After Bay, Sheet number P-6.

2.2 CONSTRUCTION

The available information on construction consisted of contract specifications and the letter dated March 20, 1918 by William L. Whitmore, Design Engineer (A copy of the letter is in the appendix).

2.3 OPERATION

The capacity of the pump station is small in comparison to normal river discharges and has little effect on reservoir flood levels. Operational procedures have little effect on the safety of the dam. (See Section 1.2h). Reservoir silt elevations were obtained from a survey by Phillips, O'Brien & Gere, 1977.

2.4 EVALUATION

Complete data regarding design and construction of the dam is not available. Additional information which would permit a more comprehensive Phase I Evaluation would include construction records, boring logs, and detailed construction drawings.

SECTION 3 - VISUAL INSPECTION

3.1 FINDINGS

a. General. The visual inspection of the Pompton Lakes Dam was made on January 6, 1978. At the time of the visit, about four to six inches of water was flowing over the full length of the spillway.

b. Dam. Some minor cracking and spalling of the concrete was noted. An additional inspection is recommended when overflow is not occurring.

c. Appurtenant Structures. Undermining of the embankment of U.S. Route 202 was noted at the east abutment. No defects or problems were noted in the outlet structure or the west side wing wall.

d. Reservoir Area. Considerable silting of the reservoir has occurred.

e. Downstream Channel. A deep scour hole is present about 150 feet downstream of the dam.

3.2 EVALUATION

No obvious deficiencies were observed during the Phase I Visual Inspection. However, in the interest of public safety, additional information should be obtained regarding the U.S. Route 202 embankment, the scour hole downstream of the dam and the conditions of the concrete in the spillway.

SECTION 4 - OPERATIONAL PROCEDURES

Operational procedures have been covered in Section 1.2.h. The North Jersey District Water Supply Commission informs Police, Civil Defense and residents when floods are imminent.

SECTION 5 - HYDRAULIC/HYDROLOGIC

5.1 EVALUATION OF FEATURES

a. Design Data. The following hydrologic conditions were determined:

1) The 500 year discharge - 30,000 cfs. This discharge was determined by the methods described in New Jersey Special Report #38, "Magnitude and Frequency of Floods in New Jersey with Effects of Urbanization", dated 1974, and the information from the United States Geological Survey gaging station at Mahwah, N.J.

2) The Probable Maximum Flood - 42,000 cfs. The Probable Maximum Flood (PMF) discharge was obtained from the United States Army Corps of Engineers, Philadelphia District.

b. Experience Data. A water stage recorder has been in service at Pompton Lakes since October of 1921. The maximum discharge of record was 12,300 cfs in March of 1936. On November 8th and 9th of 1977, an estimated discharge of 11,000 cfs was recorded. Under present conditions, a discharge of 13,000 cfs over the spillway would result in the flooding of U.S. Route 202 on the east end of the spillway. Notes included in past records indicate that the peak estimated flow during the period of record prior to construction of the present dam was 10,500 cfs in September of 1882.

SECTION 6 - STRUCTURAL STABILITY

6.1 EVALUATION OF STRUCTURAL STABILITY

a. Visual Observations. No structural inadequacies were noted during the visual inspection of the dam. However, a complete inspection was not possible due to the overflow conditions.

b. Design and Construction Data. The spillway shape is defined by a series of circular curves joined at tangent points. The curvature of the face from the crest to the point of inflection (approximately four feet below the crest) matches closely the Vicksburg, Mississippi Waterways Experiment Station crest shape for a design head of 1.32 feet (from EM 1110-2-1603). Therefore, for heads in excess of this value, the lower nappe of the overflow can be expected to separate from the spillway face, producing negative pressures on the downstream face of the dam. Since the heads of water for the 500 year flood (8.6 feet) and for the PMF (9.9 feet) are far greater than the design head, severe negative pressures and cavitation could result at these discharges. Analysis of the effect of these negative pressures is beyond the scope of this report and are not included in the calculations.

c. Operating Records. Operating Records are available only for the pumping facility. These records were not reviewed since they would not affect the safety of the dam.

d. Post Construction Changes. No alteration of the spillway is known to have been made since construction. Wing wall modifications were made on the west side wing wall in 1951.

e. Seismic Stability. The dam is located in the Triassic Highlands of northern New Jersey and is founded on a fine-grained basalt of the Triassic Newark Group as described on the Geologic Map of New Jersey, 1950 (see Figure 5 in the appendix). The formation appears to be weathered, slightly fractured and stained in outcrop becoming fresh and massive at very shallow depths. Rock outcrop is evident in both abutments and immediately downstream of the dam at mid-stream.

Based on the original design drawings, subsequent inspection reports and the current field inspection, this rock appears stable and appears to have resisted hydraulic, mechanical and chemical action.

Although the dam is in proximity to the Ramapo Fault, which trends northeast-southwest, there does not appear to be any major stress-related joints or fractures at the site. However, recent seismic activity has been recorded along the Ramapo Fault and consideration is given to its possible effect on the stability of the structures in this area.

It is noted that on the Seismic Zone Map of the Contiguous United States, the dam is located in Zone 1, but due to the dam's proximity to the fault and its renewed activity, the Zone 2 seismic coefficient was used in the stability analysis.

f. Evaluation. A stability analysis was performed to investigate the structural stability of the spillway. The results of this analysis are included in the appendix. When analyzed as a gravity structure, under normal (full reservoir) conditions, the spillway is unstable. However, in plan view, the dam is curved in a modified arch shape between abutments and is supported at mid-stream by a massive basalt rock outcrop; both of these conditions improve the stability of the structure. Analysis of the effect of these factors is not within the scope of this report.

During the PMF, the abutments on both sides of the spillway are overtopped. The effect of this overtopping on the safety of the dam cannot be evaluated without borings to determine the elevation of the top of the rock and the condition of the rock.

SECTION 7 - ASSESSMENT/REMEDIAL MEASURES

7.1 DAM ASSESSMENT

a. Safety. The spillway is structurally inadequate to resist the forces produced by normal loading conditions (full reservoir), without inducing tension at the upstream face of the dam. The analysis assumed the dam to be a simple gravity structure. However, curvature of the spillway (in plan), and support from a massive basalt outcrop improve the spillway's stability considerably.

A deep scour hole is reported to exist in the channel about 150 feet downstream of the dam. This area should be surveyed and monitored annually to determine any erosion activity towards the dam.

b. Adequacy of Information. Adequate information is not available for a detailed analysis of the spillway. Additional information which would be required for a more comprehensive evaluation would include construction records, boring logs, and detailed construction drawings. Information is also lacking for determination of the negative pressures on the downstream face of the dam, for heads in excess of 1.8 feet (1.33 times the design head of 1.3 feet).

Survey data would be useful for determining the elevations on the drainage divide along the west side of the reservoir, approximately one mile north of the dam. The minimum elevation of the divide is between 200 feet and 220 feet above Mean Sea Level (MSL). Since the computed reservoir stage for the PMF is 212.0 feet MSL, a portion of the discharge could be diverted across this drainage divide and reduce the PMF discharge over the dam.

c. Urgency. Further evaluation of the dam is recommended as soon as possible.

d. Additional Investigations. Additional investigations of this dam should be made to evaluate the negative pressures which may be produced on the dam at high discharges, to estimate the effect of the dam curvature (in plan) on the stability of the structure, and to determine if strengthening of the dam is required. A range of discharges should be studied to obtain the maximum effect of headwater-tailwater differential in the stability analyses. Drilling and sampling of the concrete dam, abutments and rock outcrop should be included.

7.2 REMEDIAL MEASURES

a. Alternatives. Remedial measures that could be considered for improving the stability of the dam include the installation of post-tensioned rods or cables through the dam and into the rock foundation or the addition of concrete to the downstream face of the dam. Aeration of the lower nappe of the water flowing over the spillway is a means to prevent the formation of negative pressures on the downstream face of the spillway. High strength epoxy applied to the downstream face might protect the concrete from the effects of possible cavitation produced by negative pressures.

Sloughing of the embankment of U.S. Route 202 could be prevented by construction of a retaining wall. The wall could be built outside of the limits of the spillway at the east end of the dam to protect the highway and to improve flow conditions over the dam.

b. O&M Maintenance & Procedures. The present drawdown facilities are inadequate to provide a reliable means of lowering the reservoir level. Consideration should be given to improving drawdown facilities. Alternate means for the improvement of these facilities are:

1) Installation of a larger bypass line to the river from the pump station.

2) Installation of a large, gated opening in the spillway.

FIELD INSPECTION

REPORT

Check List
Visual Inspection
Phase 1

Name Dam Pompton Lakes Dam County Passaic State New Jersey Coordinators Mr. John Garofalo, Mr. Larry Woscyna, New Jersey DEP

Date(s) Inspection 1/6/78 Weather Clear Temperature 40°'s

Pool Elevation at Time of Inspection 202.5$\pm$ M.S.L. Tailwater at Time of Inspection 175$\pm$ M.S.L.

Inspection Personnel:

Mr. John J. Williams
Mr. Gurbaksh Sanghera
Mr. Lee DeHeer
Mr. Stefan Manea

Mr. Gurbaksh Sanghera Recorder

Accompanied by:

Mr. John Garofalo, New Jersey Department of Environmental Protection
Mr. Larry Woscyna, " " " "
Mr. Dean Noll, Chief Engineer, North Jersey District Water Supply Commission, State of N.J.

CONCRETE/MASONRY DAMS

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SEE PAGE ON LEAKAGE	No leakage observable since flow over the spillway was occurring at the time of inspection.	Should be inspected in Sept. or Oct. when no overflow is occurring
STRUCTURE TO ABUTMENT/EMBANKMENT JUNCTIONS	East abutment is beneath the road embankment for U.S. 202. Caving of the embankment is occurring due to erosion by spillway flow. West side abutment to wing wall shows no apparent damage.	A retaining wall should be built outside of the limits of the spillway to protect the highway at the east end of the dam and to improve flow conditions over the dam.
DRAINS	No drains observed on field inspection or shown on the plans.	None.
WATER PASSAGES	None.	None.
FOUNDATION	Rock outcrops near the dam consist of competent basalt which is highly resistant to weathering. Some sandstone is present in the river channel downstream as evidenced by reports of an 18 foot deep scour hole in the river bottom.	A topographic survey of the river bottom downstream of the dam should be made and updated annually to monitor the erosion activity in this area

CONCRETE/MASONRY DAMS

VISUAL EXAMINATION OF	OBERSVATIONS	REMARKS OR RECOMMENDATIONS
SURFACE CRACKS CONCRETE SURFACES	Minor cracking along the crest, and some spalling of concrete along the downstream face were observed at various locations, near the abutments.	The concrete surface should be investigated more closely when no spillage is occurring.
STRUCTURAL CRACKING	None observed since overflow was occurring over entire length of the dam at the time of inspection.	Same as above.
VERTICAL AND HORIZONTAL ALIGNMENT	No displacement observed.	Same as above.
MONOLITH JOINTS	Could not inspect due to overflow condition at time of inspection.	Same as above.
CONSTRUCTION JOINTS	Could not inspect due to overflow condition.	Same as above.

OUTLET WORKS

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CRACKING AND SPALLING OF CONCRETE SURFACES IN OUTLET CONDUIT	Outlet conduit consists of 72 inch & 78 inch steel pipes which lead to a screen chamber & pump station. Outlet conduits were not de-watered for inspection. These pipes are connected to 4 pump suction lines. Outlet to the river is through 4-12 inch pipes downstream of the pumps and one 30 inch pipe from screen chamber forebay.	None
INTAKE STRUCTURE	Intake structure consists of a screen chamber located adjacent to & downstream of the dam. No signs of structural inadequacies observed.	None
OUTLET STRUCTURE	Outlet from screen chamber forebay consists of 30 inch pipe located just below spillway crest. No problems with forebay or pipe were observed.	None
OUTLET CHANNEL	No problems observed.	None
EMERGENCY GATE	None	None

RESERVOIR

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SLOPES	Flat slopes - No signs of sliding	None
SEDIMENTATION	Reservoir appears to be heavily silted.	Reservoir siltation does not have a significant effect on the safety of the structure.

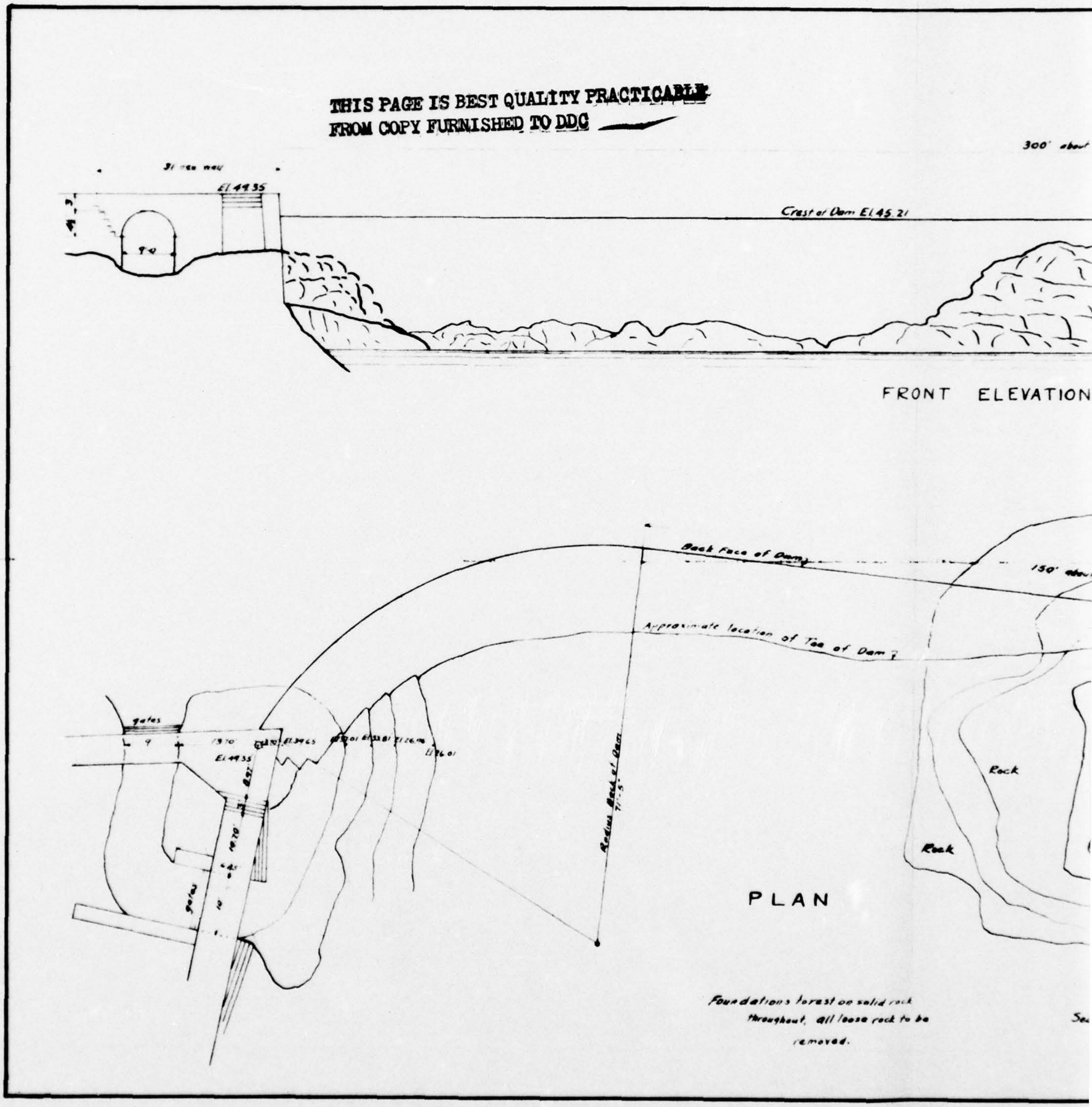
DOWNSTREAM CHANNEL

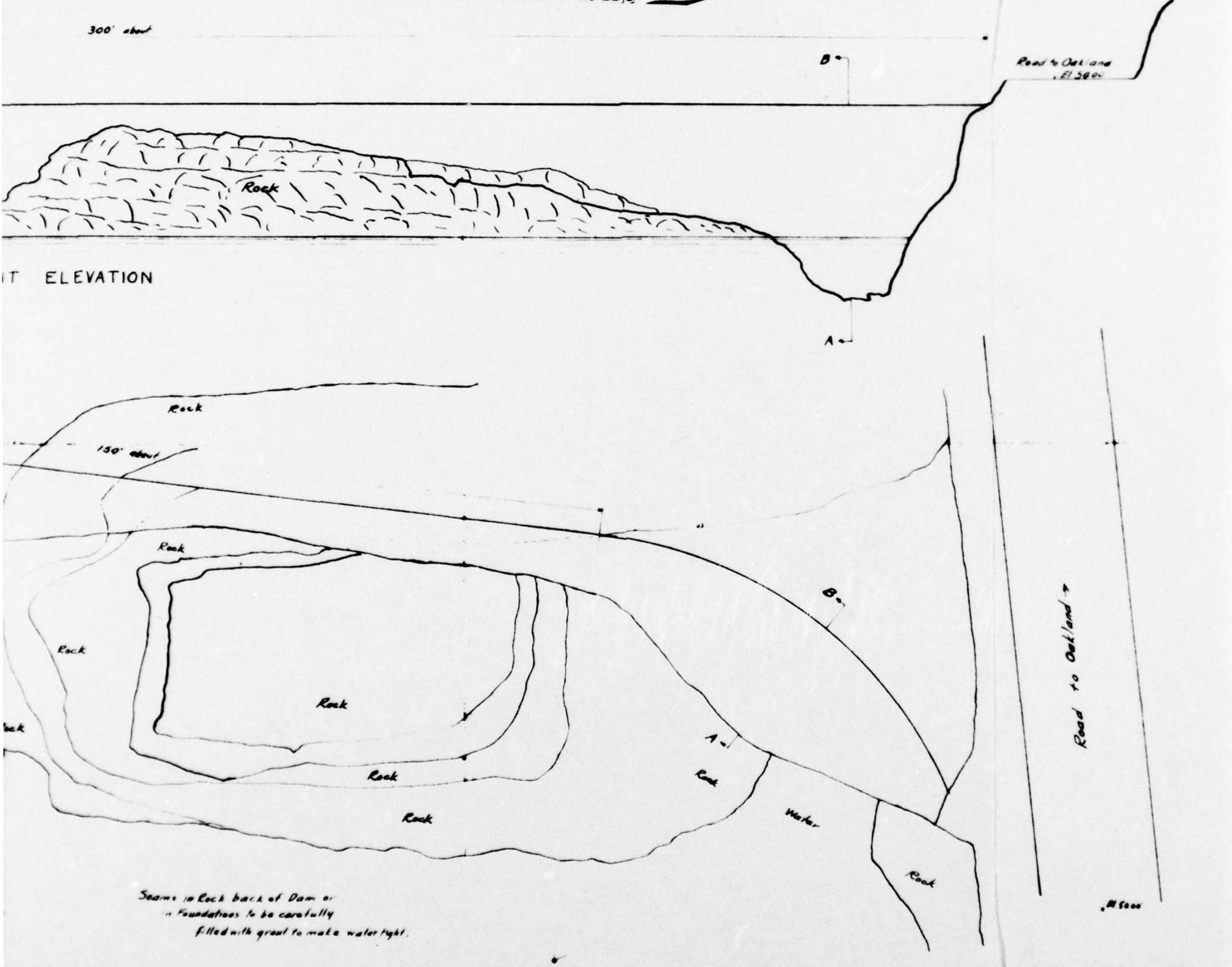
VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CONDITION (OBSTRUCTIONS, DEBRIS, ETC.)	Reports have been made of an 18 foot deep scour hole in channel downstream of the dam. The scour hole was not observed during inspection due to high flow in the river.	Topographic survey of river bottom should be made. See note under dam foundation in visual examination sheets.
SLOPES	Sloughing of route 202 highway embankment at east end of dam.	River bank on east side should be protected from erosion to prevent failure of highway embankment - See note under abutment section in visual examination sheets.
APPROXIMATE NO. OF HOMES AND POPULATION	Approximately 1,000 homes in flood plain area downstream of dam. (Approx. 4,000 people)	North Jersey District Water Supply Commission informs Police, Civil Defense and residents are informed when floods are imminent.
DOWNSTREAM STRUCTURE	A two span highway bridge approximately 450 feet downstream of dam forms constriction and can cause flooding of highway and business areas.	None.

ITEM	REMARKS
DESIGN REPORTS	The present dam at Pompton Lakes was built in 1908. No design reports were found in the present file. Two reports on the dam were made in 1918 when the dam was acquired by the Borough of Pompton Lakes. One report, made by William L. Whitmore, the Engineer in charge of the design and construction of the dam states that the concrete structure was one of the best structures in the state at that time. Another report acknowledges that the resultant force on the dam falls outside the middle third of the base under maximum flood conditions and states that the circular shape and additional reinforcement will "take care of this departure from standard design for a gravity type structure."
GEOLOGY REPORTS	Report of 1918 only mentions that the foundation is solid trap rock.
DESIGN COMPUTATIONS HYDROLOGY & HYDRAULICS DAM STABILITY SEEPAGE STUDIES	Maximum recorded discharge over dam = 12,300 cfs (3.56 feet over crest) Highest head over crest =4.4 feet (12,000 cfs) PMF = 113,000 cfs (17.3 ft. over crest) Dam is unstable as simple gravity section for full pool (reservoir at dam crest) condition and higher. Curvature and rock outcrop in the center of the dam improve the stability to an unknown degree. No seepage studies made since dam is concrete founded on rock throughout.
MATERIALS INVESTIGATIONS BORING RECORDS LABORATORY FIELD	None
POST-CONSTRUCTION SURVEYS OF DAM	None
BORROW SOURCES.	N/A

ITEM	REMARKS
MONITORING SYSTEMS	
	None
MODIFICATIONS	
	West side wing wall was reconstructed with new sluice gates and improved forebay.
HIGH POOL RECORDS	
	March 12, 1936 - 205.66 feet above MSL (12,300 cfs) October 16, 1955 - 206.50 feet above MSL (12,000 cfs) November 8, 1977 - 206.14 feet above MSL (11,000 cfs)
POST CONSTRUCTION ENGINEERING STUDIES AND REPORTS	
	Inspections were made of the dam in 1918 and in 1977. Conclusions and recommendations from those reports are included in the appendix.
PRIOR ACCIDENTS OR FAILURE OF DAM DESCRIPTION REPORTS	
	The original structure at Pompton Lakes was a wooden crib dam that failed during a storm in 1903.
MAINTENANCE OPERATION RECORDS	Records available only for pumping station.

FIGURES


El 49.35
Crest of Dam El 45.21
300' about
FRONT ELEVATION
Back Face of Dam
150' about
Approximate location of Toe of Dam
gates
El 49.35
Rock
Rock
Radius Back of Dam 71'-5"
PLAN
Foundations to rest on solid rock throughout, all loose rock to be removed.


300' about
B
Road to Oakland
Rock
IT ELEVATION
A
Rock
150' about
Rock
Rock
Rock
B
A
Rock
Rock
Rock
Rock
Water
Rock
Road to Oakland
Seams in Rock back of Dam or
in Foundations to be carefully
filled with grout to make water tight.

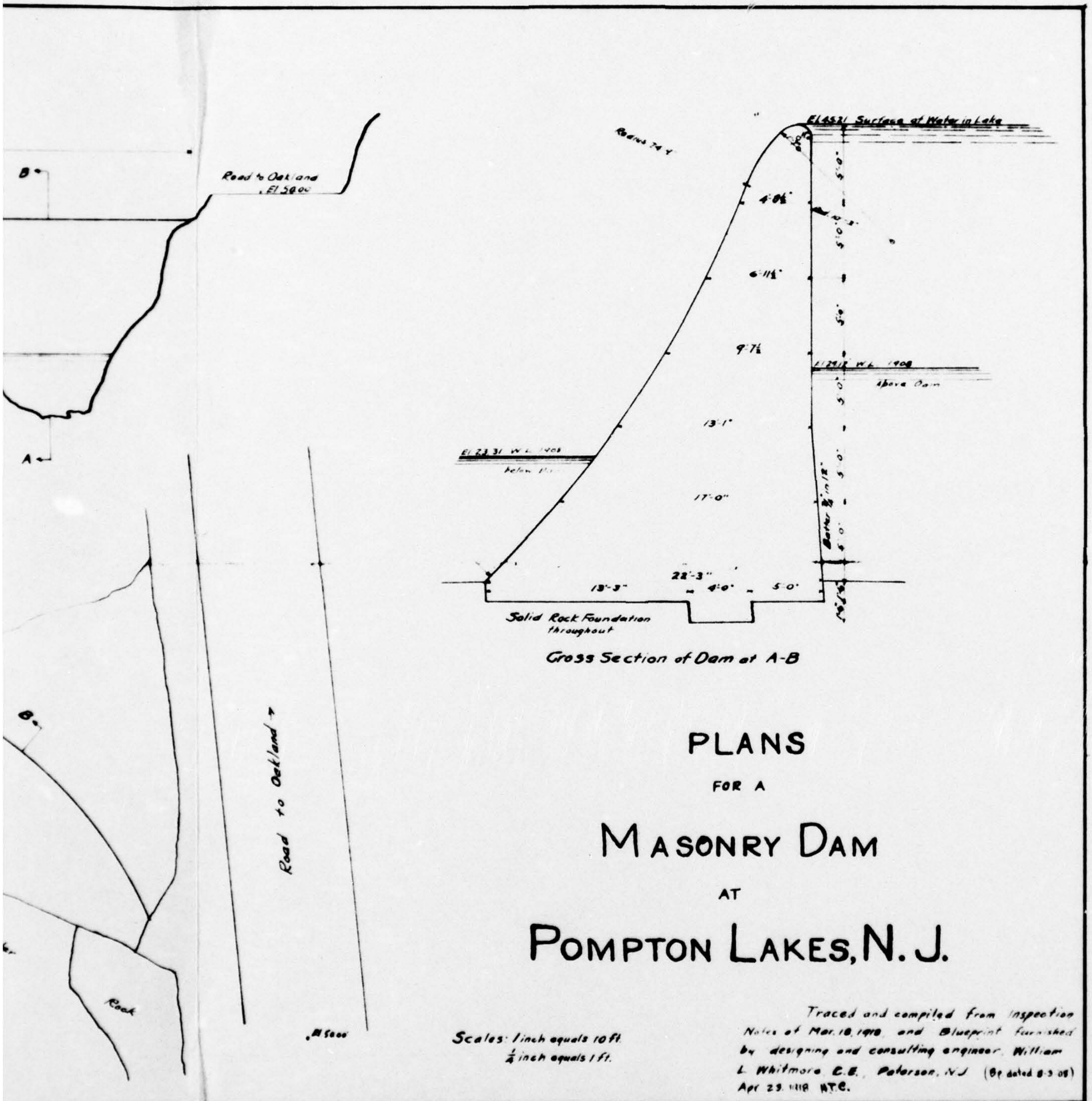
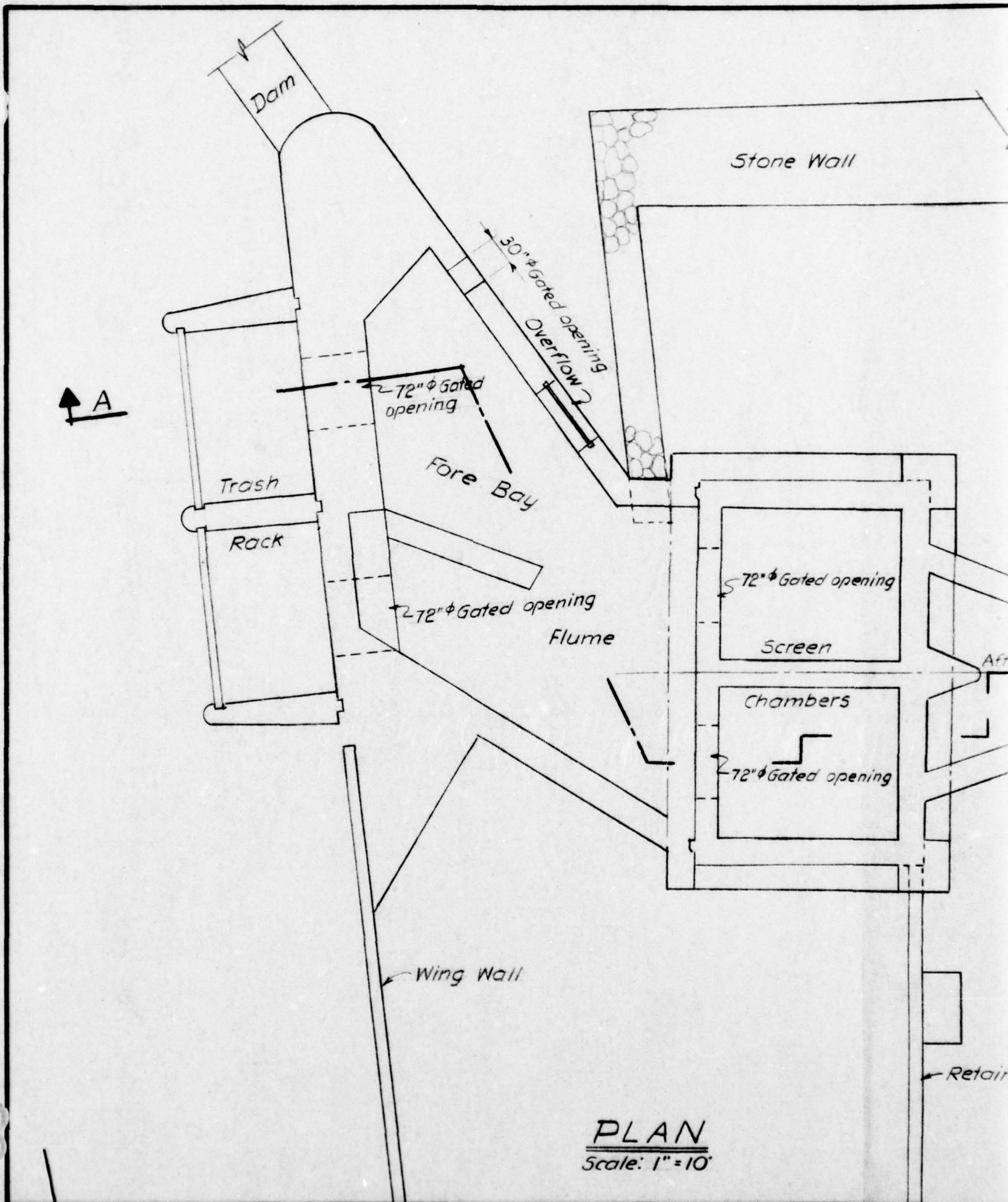
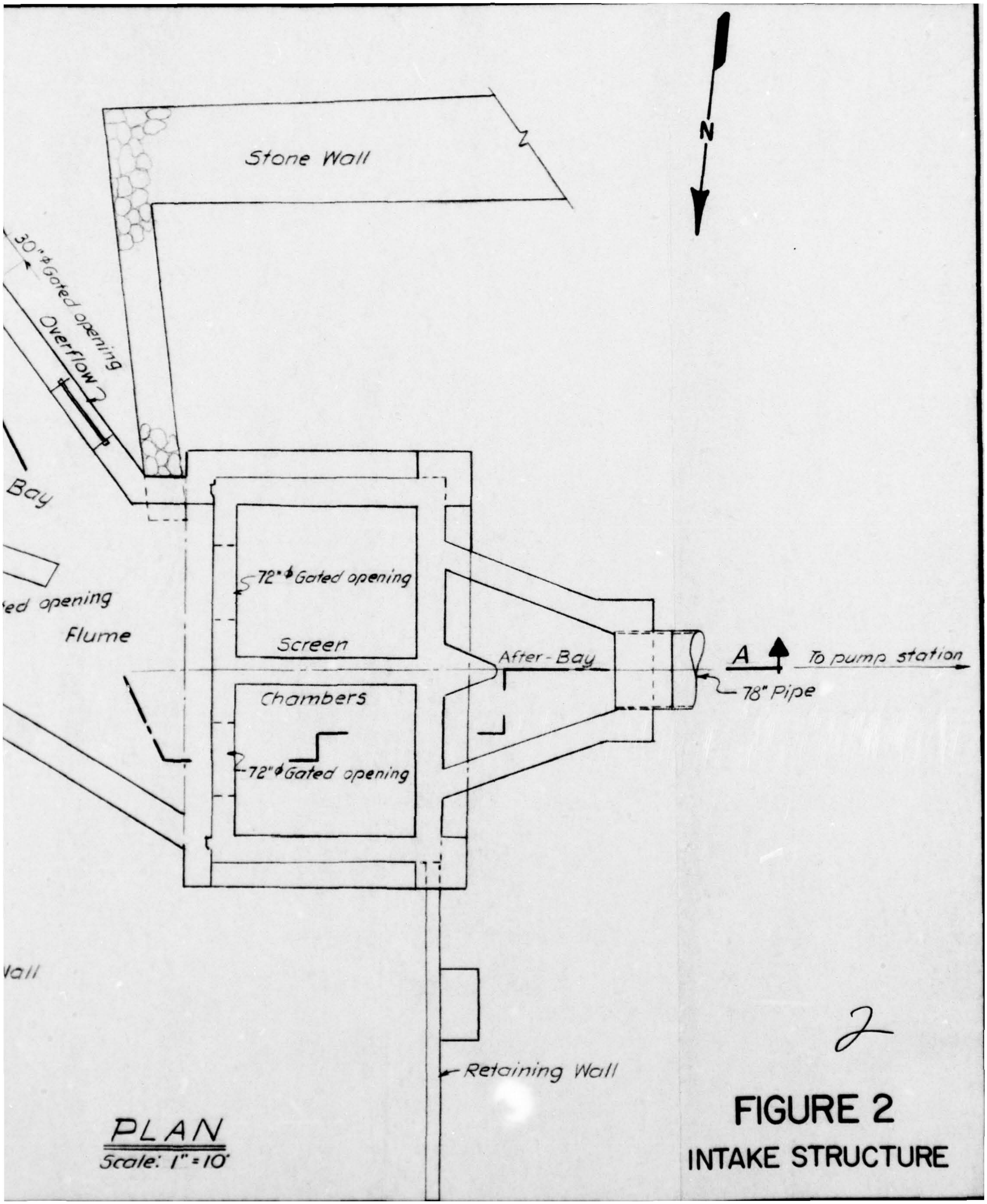

Road to Oakland
El 56.00
B
A
Road to Oakland
Rock
El 56.00
Radius 74.4
El 45.21 Surface of Water in Lake
5'-0"
4'-0½"
6'-11½"
9'-7½"
El 29.12 W.L. 1908
Above Dam
13'-1"
El 23.31 W.L. 1908
Below Dam
17'-0"
Batter ¾ in 12"
22'-3"
13'-3"
4'-0"
5'-0"
Solid Rock Foundation
throughout
Cross Section of Dam at A-B
PLANS
FOR A
MASONRY DAM
AT
POMPTON LAKES, N. J.
Scales: 1 inch equals 10 ft.
¼ inch equals 1 ft.
Traced and compiled from Inspection
Notes of Mar. 10, 1918, and Blueprint furnished
by designing and consulting engineer, William
L. Whitmore C.E., Paterson, N.J. (Bp dated 8-3-08)
Apr 23, 1918 H.T.C.

FIGURE 1

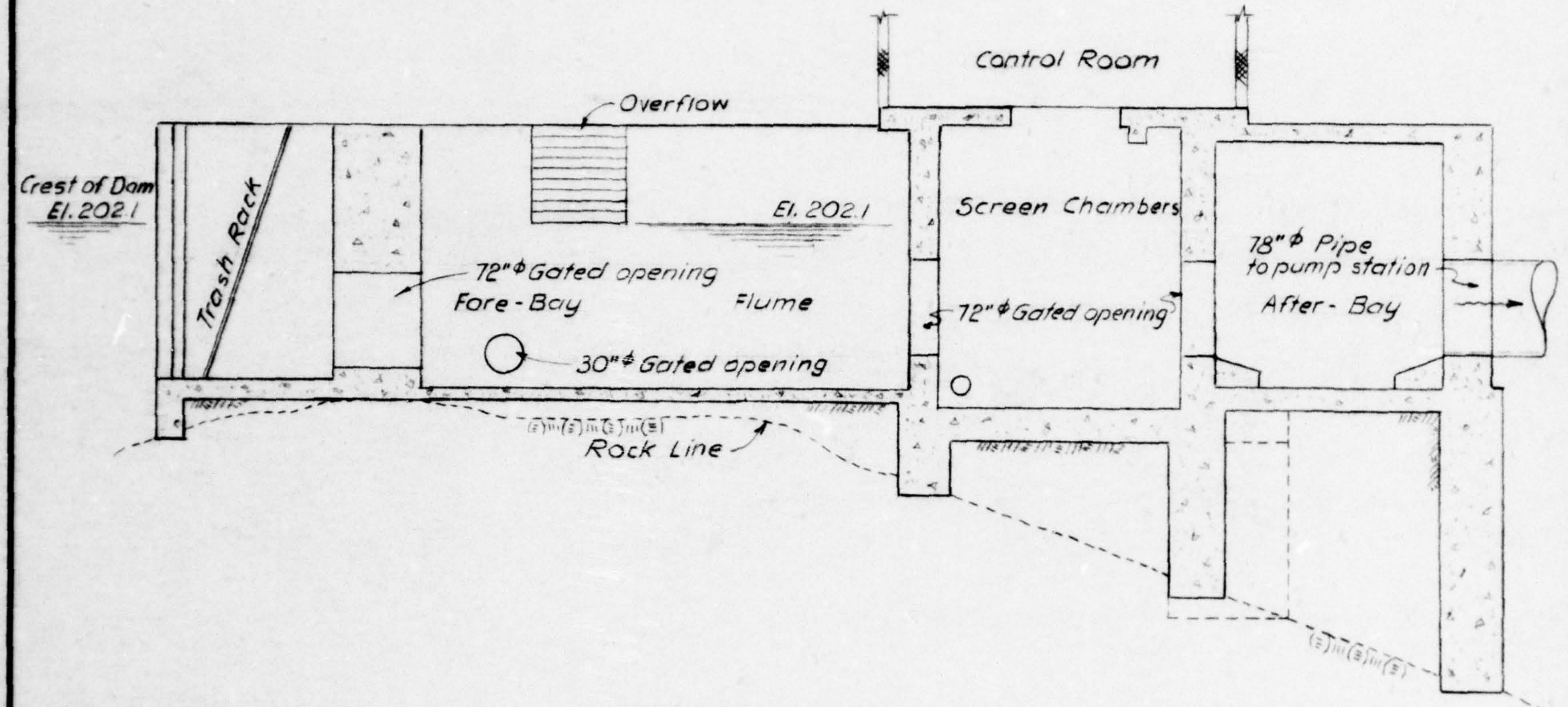


PLAN
Scale: 1" = 10'



2

FIGURE 2
INTAKE STRUCTURE



SECTION A-A

Scale: 1"=10'

FIGURE 3

INTAKE STRUCTURE

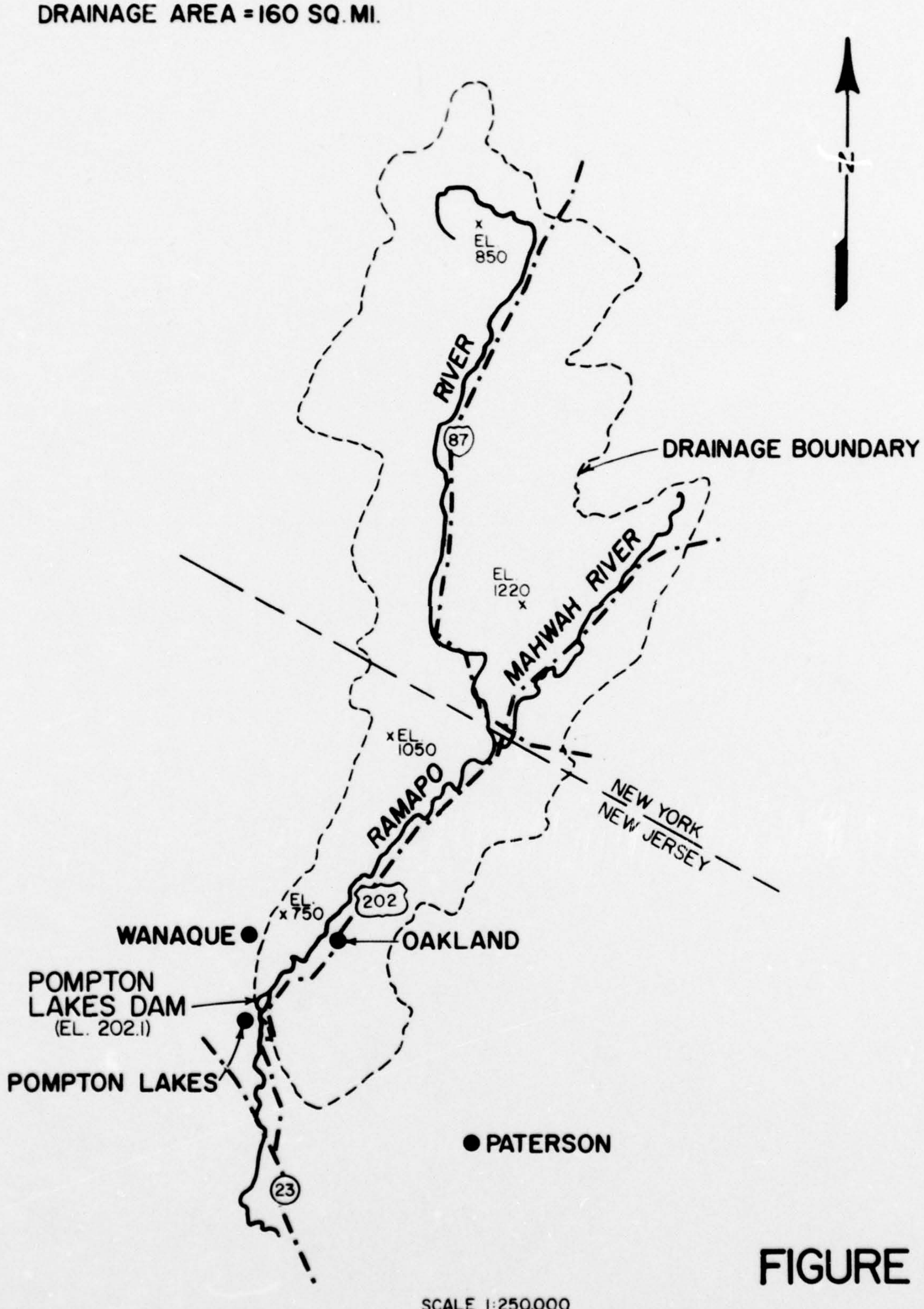
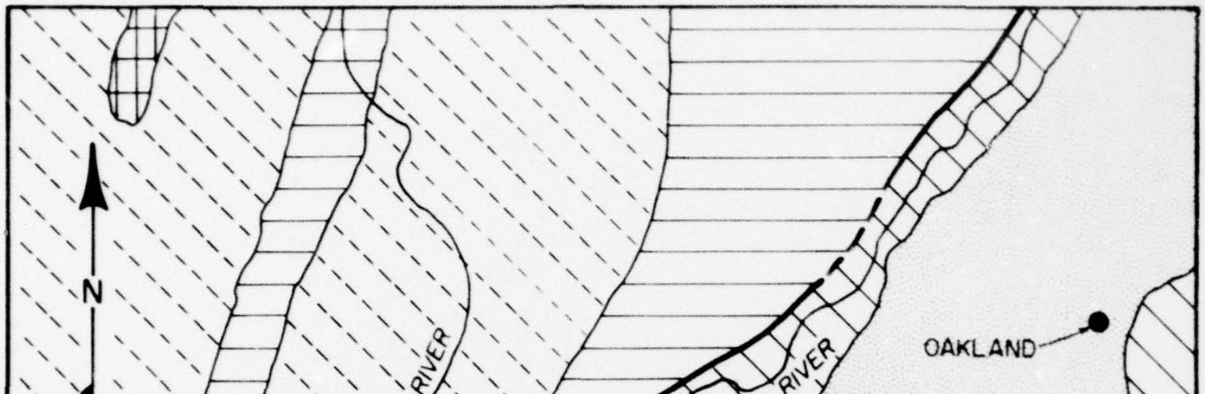


FIGURE 4

DRAINAGE BASIN MAP



SCALE 1:17,300

LEGEND:

BASALT FLOWS	FINE-GRAINED TRAP ROCK IN EXTENSIVE FLOWS.	
BRUNSWICK FORMATION	SOFT RED SHALE WITH SANDSTONE BEDS.	
BYRAM GNEISS	GRAY GRANITOID GNEISS	
FRANKLIN LIMESTONE	COARSE WHITE MARBLE, MAGNESIAN IN PART.	
LOSEE GNEISS	WHITE GRANITOID GNEISS	
	RAMAPO FAULT	

FIGURE 5

GEOLOGIC MAP

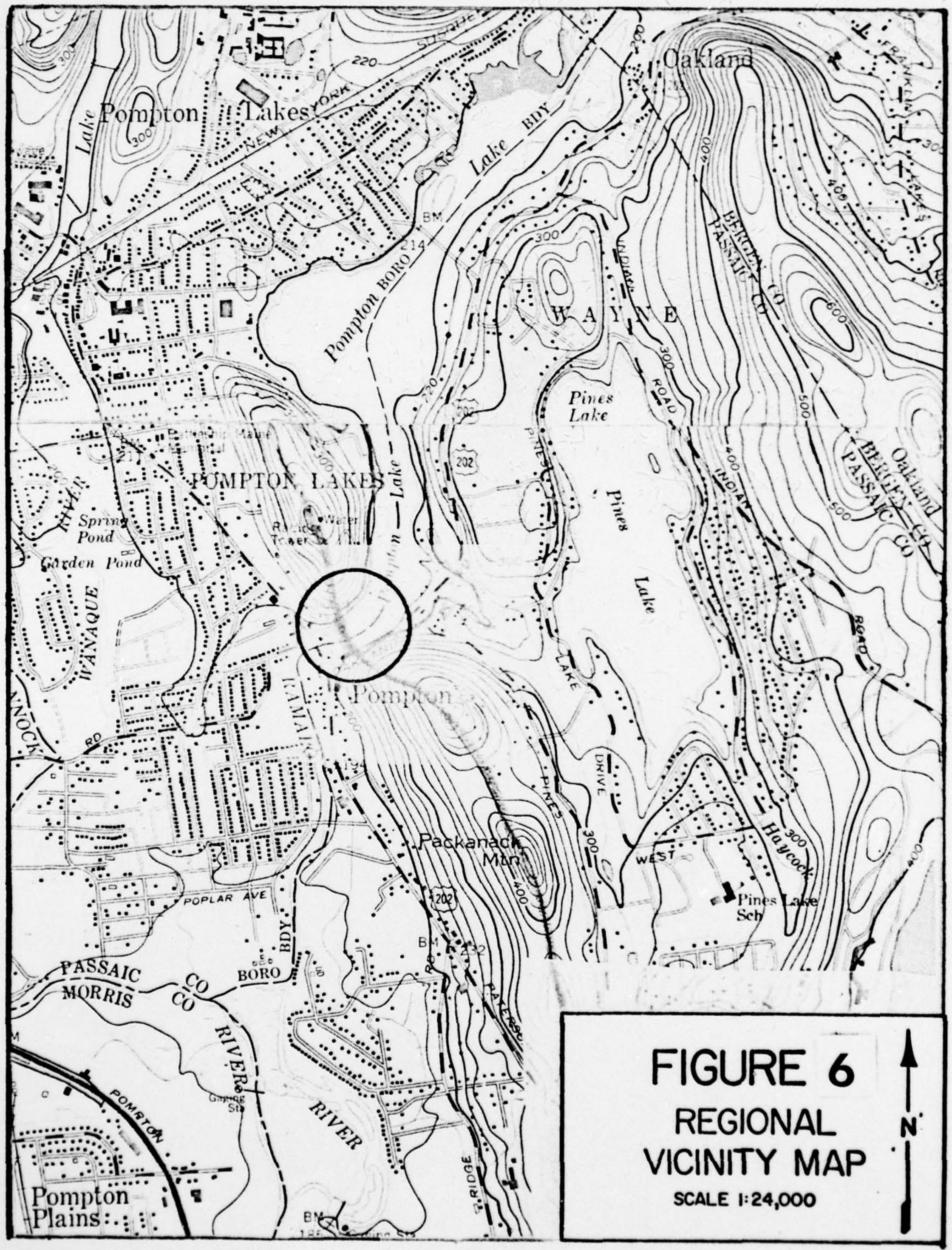
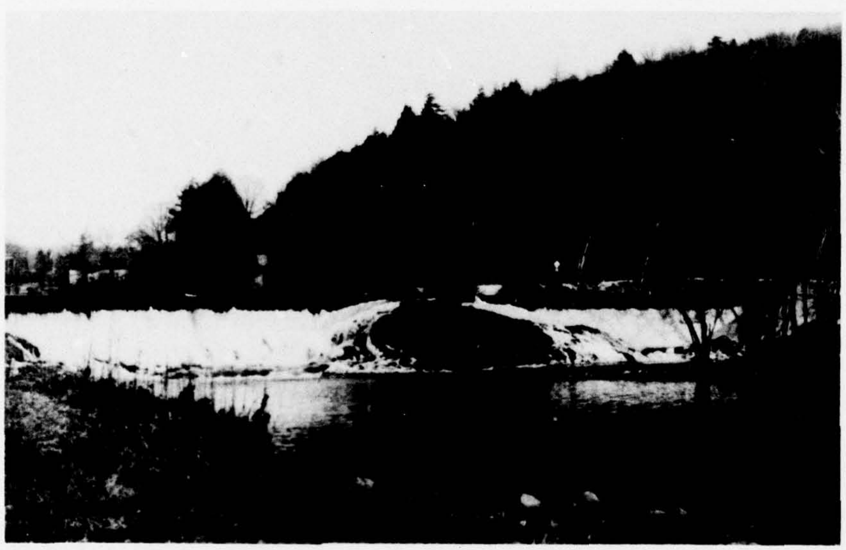


FIGURE 6
REGIONAL VICINITY MAP
SCALE 1:24,000

APPENDIX

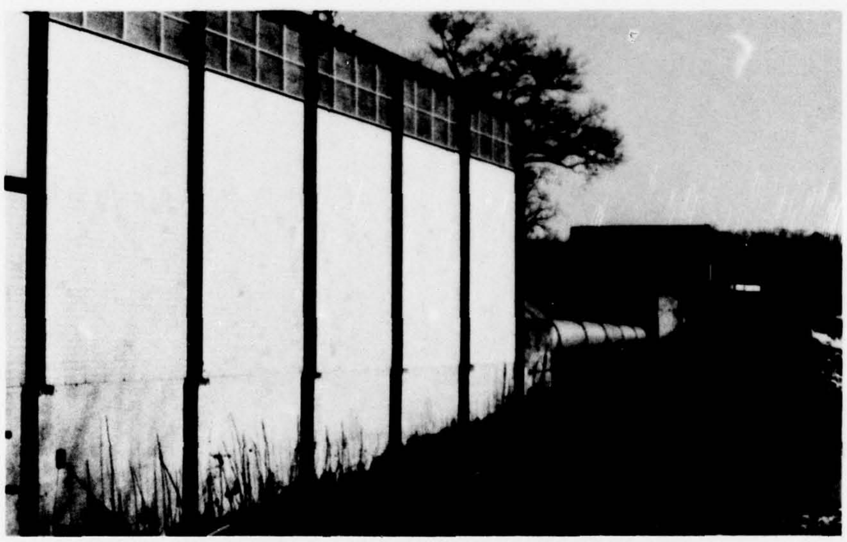
PHOTOGRAPHS

VIEW LOOKING TOWARD RIGHT BANK



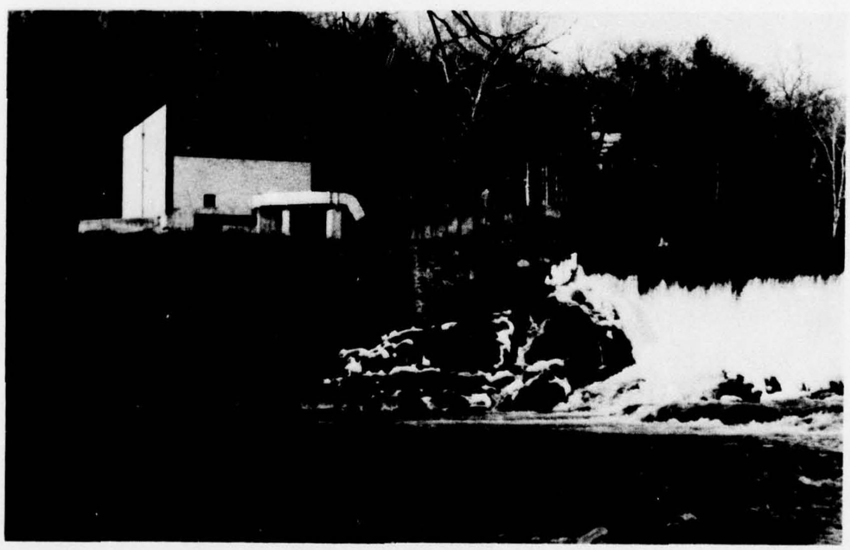
VIEW LOOKING UPSTREAM SHOWING ROCK OUTCROP AT CENTER AND EROSION OF HIGHWAY EMBANKMENT

VIEW SHOWING LEFT BANK DOWNSTREAM OF THE DAM



VIEW OF RIGHT BANK DOWNSTREAM OF THE DAM SHOWING PUMPING STATION IN FOREGROUND AND SCREEN CHAMBER IN BACKGROUND

CLOSEUP OF EMBANKMENT EROSION ON LEFT (EAST) BANK



RIGHT BANK SHOWING INTAKE AND SCREEN CHAMBER

PREVIOUS INSPECTION REPORTS

Trenton, N. J., Mar. 15, 1918.

Department of Conservation and Development,

Trenton, N. J.

Gentlemen:

As representing Edward Corning, Esq., of Watervliet, the owner of a certain lake and water-power at Pompton Lakes, Passaic County, N. J., I respectfully request that your Department inspect the existing concrete dam at said point with the idea of reporting to the Mayor and Council of the Borough of Pompton Lakes as to its strength.

The reason for this request is as follows: The Borough of Pompton Lakes is acquiring this water-power on a 21-years' lease with the privilege of purchasing. The cost, together with the necessary improvements to be made before the water-power can be used, will be from $150,000 to $200,000. Before the Borough takes this property over, they wish to know that this dam is safe and within your requirements.

This dam was built about ten years ago by the F. R. Long Co., of Hackensack, and the engineer in charge was William L. Whittemore. The dam is of concrete construction, and is heavily reinforced, and while everybody believes it to be of superior construction, they want some official statement regarding its strength. It is likewise made one of the conditions of the proposed agreement between the Borough and Mr. Corning that no changes are to be made in this dam until after they are approved as to strength, etc., by your Department. If you could give this matter your early attention, it would greatly oblige.

Very truly yours,

[signature]

Member of Assembly for Passaic County

WM. L. WHITMORE
CIVIL ENGINEER AND SURVEYOR
ROOMS 21 & 22
PATERSON SAVINGS INSTITUTION
TELEPHONE 577

PATERSON, N. J. March 20, 1918

Department of Conservation & Development,
State House,
Trenton, N. J.

Attention of Mr. H. T. Critchlow, C. E.

Dear Sir:

Referring to the Pompton dam located in the Borough of Pompton Lakes, Passaic County, N. J.--

The old wooden dam was washed out by the flood of 1903. The present dam was built during the fall of 1908. The location of the dam was slightly changed from the plan to secure a better footing. It was built exactly according to the plans and specifications submitted to you as to measurements and mixture of cement, sand and stone. About fifty tons of steel rods were distributed through the concrete which was not included in the specifications. The dam is built on a solid rock botton from end to end, and keys into rock at each end. It was constructed in forty six days from the day it was started.

The work was done under the supervision of Mr. Ludlum and myself and the Superintendent was an experienced man in concrete work. I think I can safely say there is no better structure in concrete work in the State of New Jersey. The contractor was the F. R. Long Co. who sublet the contract to Schweirs & Sutton of 90 West Street, New York.

Yours very truly,

WLW/JS

William L. Whitmore

A5

Report on Inspection of Pompton Lakes Dam on Ramapo River at Pompton Lakes, N. J.

Trenton, N.J. Mar. 28,1918.

In response to a request from Honorable H.G. Hershfield, member of Assembly for Passaic County, under date of March 15, 1918, for an inspection of the Pompton Lakes Dam on the Ramapo River at Pompton Lakes, N. J. for the purpose of reporting to the Mayor and Council of the borough of Pompton Lakes as to the condition and safety:

The borough of Pompton Lakes propose acquiring the water power rights to this dam, now owned by Edward Corning, Esq. of Watervliet, N.Y., on a 21 year lease with the privilege of purchasing, and wishes to know whether the structure is safe and within the requirements of this department. An inspection of the structure was made on March 18, 1918 by the writer, accompanied by Mr. Hershfield and Mayor Stephen H.B. Jacobs of Pompton Lakes. On the same day a conference was held with the Engineer in charge of the construction work, Wm. L. Whitmore of Paterson, N.J., who also furnished a set of plans and specifications followed in the building of the dam. Mr. Whitmore has also filed a written statement under date of March 20, 1918 giving certain information regarding the construction work.

History.- The present masonry dam was built during the fall of 1908 (previous to the passage of Act 1912 regulating the construction of dams in this State) by the Ludlum Steel & Iron Co. to replace the wooden structure that was washed out by the flood of October, 1903. The contractor was F.R. Long Company of Hackensack N.J., who sublet the contract to Schweirs & Sutton of No.90 West Street, New York City; the Superintendent of Construction was Geo. W. Cisco of Hawthorne, N.J.

Description of Dam.- The main dam is of concrete construction reinforced with about 50 tons of steel rods, not included in the specifications but made use of on account of their being readily obtained from the old steel works nearby. The foundation is solid trap rock throughout and the structure is keyed into this foundation from end to end. The plan of the dam is on a straight line crest for approximately the middle third, where a rock ledge rises up dividing the gorge into two channels; while across these channels the dam has a curved crest to either bank. The total length of the crest is about 300 ft. and the maximum height above bed rock is about 30 ft. in the south channel, making the normal lake level about 22 ft. above the stream immediately below the dam. The cross section of the dam is of the "Ogee" type having a width of 27 inches one foot below the crest and a maximum width of 22.25 ft. at the base. At the south end the concrete is keyed into the rock all the way up to the crest, while on the north end the crest is tied into an old masonry wing wall which extends across the head of the raceway into the rock slope.

Wing Wall.- This wing wall, which is part of the original structure, contains the gates used to regulate the flow of water into the raceway leading down to the old steel mill now in ruins. These gates are of timber construction, leak badly, are in very poor condition and would be hard to operate. The wing wall has a total length of about 90 ft. to the north bank, a uniform width of 5 ft. and an average height of about 10 ft. above the rock foundati with the top elevation about 4 ft. above the crest of the main dam. At the location of the gates a channel about 5 ft. deep and 9 ft. wide has been cut into the rock. This is the only outlet from the lake and permits of lowering the water about 9 ft. below normal leve in the dry season. At the corner where the main dam abuts on the ol masonry there is a gap about 3 ft. long and 1-1/2 ft. high caused by the removal of two or three large blocks of stone by the ice going out this past spring.

Raceway.- The raceway channel is carried along the north bank of the river, being supported on the side toward the stream by a retaining wall and embankment about 250 ft. long extending from th corner where the dam proper joins the wing walldown to the old spring shop which fronts on the highway. This retaining wall has a height of about 21 ft. above the water level below the dam, a top elevation about 3 ft. below the top of the wing wall and 3 ft. above the normal water level in the raceway. The control gates are locat near the up-stream end of the wall and permit the unwatering of the raceway. These gates are of timber construction, leak considerably, are in very poor condition and would be difficult to operate. A section of the old retaining embankment about 135 ft. long was washed out during the flood of 1903 and was replaced about two years later by a concrete wall 21 ft. high above the footing of large boulders at the water level below the dam. The top width is 2.8 ft. with the outside face vertical while the inside steps down until at a poi 9 ft. below the top it is 5 ft. wide and is reported to be that width to the bottom. The normal water level in the raceway is about 3 ft. below the top of the wall, while the bottom of the channel is about 9 to 10 ft. below the top, making the water about 6 to 7 ft. deep. Th concrete portion of the retaining wall permits the water in the raceway to leak through and issue from the outside face in small streams or jets. Any water which may percolate down behind the wal must certainly find free outlet beneath the footing into the stream bed, otherwise the pressure from the water against the entire inside face of the wall would have caused it to go out long ago.

Floods.- In order to arrive at the maximum water pressure against the dam and the wing wall a study of the flood flows on the Ramapo has been made with the following findings:

The drainage area of the Ramapo at the confluence with the Pompton River is 160.7 square miles, while at the Pompton Lakes dam

1.5 miles above the mouth, the drainage area is 159.5 square miles. In Volume III of the New Jersey Geological Survey (1894), page 155, it is stated that during the flood of September , 1882, the Ramapo had a maximum discharge of 10,540 cubic feet per second, which is equivalent to a water level in the lake of about 4.5 ft. above the elevation of the crest of the dam. In the U.S.Geological Survey Water Supply Paper 85, page 55, it is stated that during the flood of February - March, 1902 "on the Ramapo River at Pompton Furnace the highest water was observed at 6 P.M. on March 1, when it reached 4.1 ft. above the dam crest". In the annual report of the New Jersey Geological Survey for 1903, pages 27 and 28, it is stated that during the flood of October 1903 "the Ramapo at about 5.40 A.M. of Oct. 9, when the Pompton Steel Works dam broke, had reached a discharge of 7123 cubic feet per second. --- Undoubtedly the maximum discharge was much in excess of this." This same report in table V on page 27 states that the maximum discharge of the Ramapo during the flood of 1896 was 8,731 cubic feet per second. Thus it is seen that during a period of less than 25 years there were 4 floods that caused a lake level of over 4 ft. above the crest of the dam and a possibility that during the flood of 1903 the level would have been 5 ft. above the crest if the dam had not broke. (The crest of the old dam was of the same elevation as the existing structure) Therefore, in analyzing the strength of the dam and wing wall a flood level 5 ft. above the crest of the dam has been considered.

Conclusions.- A careful analysis of the cross-section of the main dam under maximum flood conditions shows it to be of safe design, although the line of resultant pressures passes without the line of the "middle third". However, the reinforcement and the circular arch shape across the deep channels will take care of this departure from standard design for a gravity-type section. However, in order to permit the arch action to be effective for the entire height of the dam, the damage to the corner of the old masonry wall where the north end of the dam joins it should be repaired so that a solid abutment will be obtained. An analysis of the section of the wing wall extending from the end of the dam proper to the north bank shows that it is unsafe under the possible conditions of the water level in the lake being up to the top of the wall (4.2 ft. above crest dam) and raceway drained due to unwatering through gates, damage to retaining wall, or other causes. This wall should be made water tight, and raised 1 ft. to prevent over-topping during floods. A buttress wall should be built on the down-stream side just north of the gateway opening rising to within 3 ft. of the top and extending down stream on a slope of 6 inches per foot of height. This wall should be 3 ft. wide and of concrete construction securely keyed into the ledge rock and bonded to the masonry wall by means of steel rods. The section of the concrete retaining wall for the raceway was also analyzed, using the measurements given above. This wall is probably of safe design if the water in the raceway is not allowed to rise higher than an elevation of 3 ft. below the top of the wall. Provision should be made for ample overflow at the control gates to regulate this level. The raising of th wing wall 1 ft. , as suggested above, was with the idea of preventing flooding of the raceway during high water. The raceway should be unwatered and an examination made to determine whether there is free passage under the wall for any water that may percolate down behind it from the raceway. This condition has evidently obtained in the past, but any gradual or sudden clogging up of these outlets for seepage water would undoubtedly prove disastrous.

Recommendations.- In order to put the structure in safe condition to withstand the flood conditions as set forth above and to protect the lives and property in the valley below, as well as guard against any interruption in the operation of a water power development connected with the dam, the following recommendations are made:

1 - Repair damage in masonry corner where north end of dam proper joins old masonry.

2 - Construct buttress wall on down-stream side of wing wall.

3 - Raise wing wall 1 ft. and stop leakage by water-proofing up-stream face.

4 - Stop leaks in concrete retaining wall and examine the footing and provide weep passages for seepage water from the raceway, if found necessary.

5 - Replace present wooden sluice gates in wing wall and control gates in raceway with metal gates for facilitating for rapid one-man operation.

6 - Provide overflow at control gates in raceway to prevent water level rising within 3 ft. of the top of the retaining wall.

Respectively submitted

APR -1 '18

Water Engineer.

INSPECTION REPORT
POMPTON DAM ON RAMAPO RIVER
DAM #22

Upon the request of Mr. William Yurasek, Oakland Borough Engineer, inspection was made on August 4, 1977 at 1:00 P.M. of the existing Pompton Lake Dam on Ramapo River in the Borough of Pompton Lakes, Passaic County.

Present during the inspection were:

Mr. Sidney H. Stone . . . . Administrator, Boro. of Oakland
Mr. William Yurasek, P.E. . . . . . . Oakland Boro. Engineer
Mr. Jack Harmen . . . . Administrator, Wayne Township
Mr. Ed Tesch. . . . . Engineer, Wayne Township
Mr. J. Wiland . . . . N.J. District Water Supply Comm.
Mr. Boutros Habek . . . . Dam Section, Bureau of Flood Plain Management

<u>Observations</u>

It was observed that the existing concrete dam appears to be in a good, solid condition.

The existing dam is founded on a rock bed and buttressed with an existing huge natural rock. There is a deep pool downstream of the dam, but it is formed in the rocks and it does not appear to have any effect on the safety of the dam foundation.

Mr. Yurasek requested this inspection after his visit to the site on such a day where the flow was high in the river, and the bed was not visible. He was aware that the above mentioned pool might have some effect on the dam safety.

On the day when inspection took place, the flow was very low, and the bed of the stream was very easy to see.

All people at the meeting agreed that the dam and its foundation appears to be very solid. The pool in question is at least 150 feet downstream of the dam and it is formed in natural solid rocks. Also they agreed that any danger of failure of the dam is not visible. See attached dam pictures for the record.

~~Upon Mr. Yurasek's request another inspection was made to the existing~~ upstream weir (almost one mile U.S. Dam 22) named Hemlock Street weir. The weir is in a very poor condition and the wall along the left bank is falling down (See Photos #1,2). The left side of the weir where a low flow is supposed to be very obvious to be washed out by floods (See Photos 2,3,4,5). On the right bank up and downstream of the weir there are few existing homes which are affected by this weir. Thirty feet downstream of the weir there are big stones placed in the bed to act as energ ~~dissipators (See Photo #6).~~

Recommendations

It is receommended that the Hemlock weir be repaired or replaced by another adequate structure, however, any failure to this weir might not affect the downstream Pompton Lake Dam #22.

Boutros Habek

DAM STABILITY

JUSTIN & COURTNEY, INC.
Division of O'Brien & Gere Engineers, Inc.
PHILADELPHIA, PA

SHEET NO. A18 OF ______

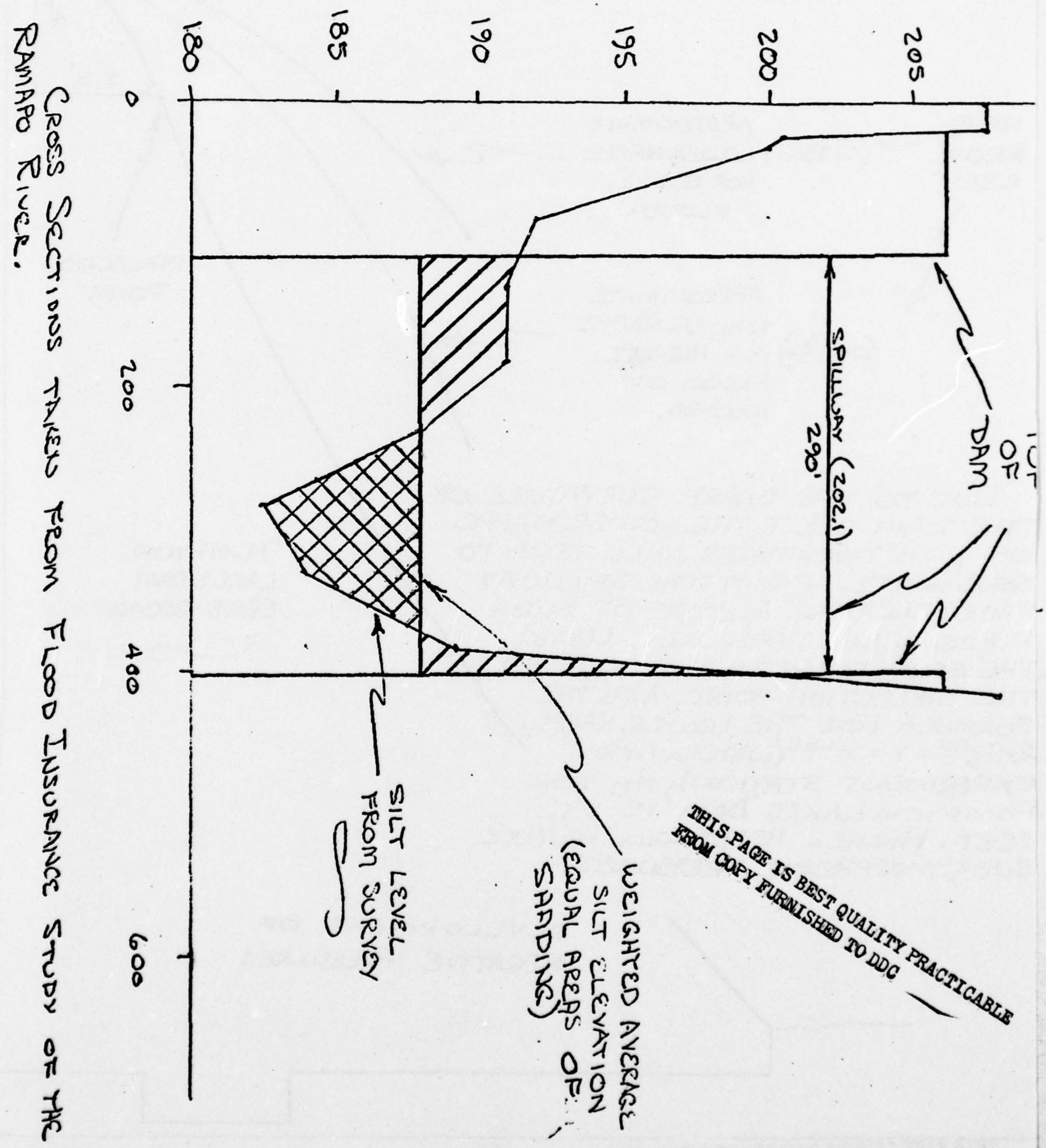
DATE 1/16/78

NAME OF CLIENT Corps of Engineers

COMP. BY ______

PROJECT Pompton Lakes Dam

CHECKED BY ______



JUSTIN & COURTNEY, INC.
Division of O'Brien & Gere Engineers, Inc.
PHILADELPHIA, PA

SHEET NO.____ OF____

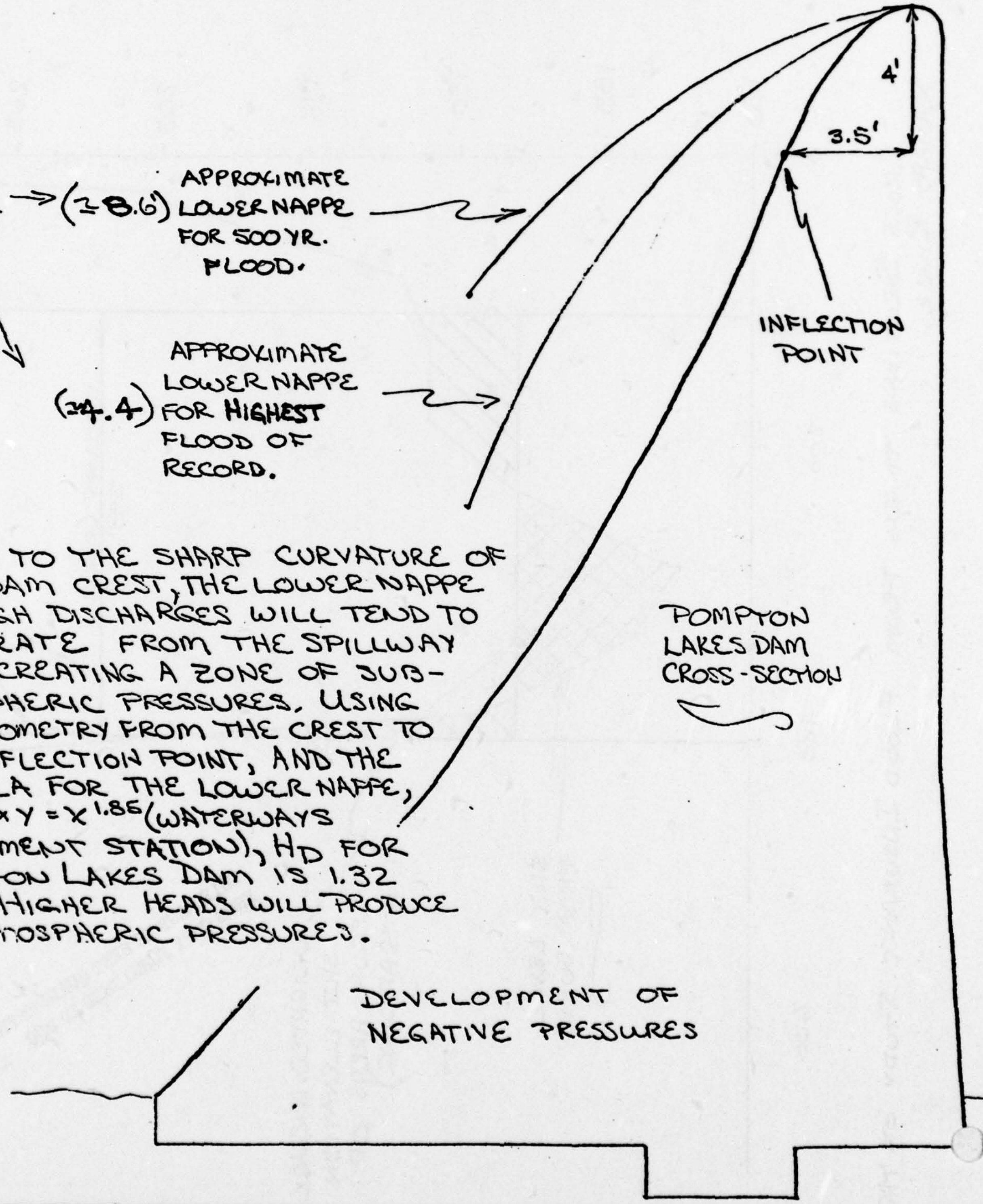
DATE 1/25/78

NAME OF CLIENT Corps of Engineers

COMP. BY DBC

PROJECT Pompton Lakes Dam

CHECKED BY____



DUE TO THE SHARP CURVATURE OF THE DAM CREST, THE LOWER NAPPE OF HIGH DISCHARGES WILL TEND TO SEPARATE FROM THE SPILLWAY FACE, CREATING A ZONE OF SUB-ATMOSPHERIC PRESSURES. USING THE GEOMETRY FROM THE CREST TO THE INFLECTION POINT, AND THE FORMULA FOR THE LOWER NAPPE, $2 \times H_D^{.85} \times Y = X^{1.85}$ (WATERWAYS EXPERIMENT STATION), H_D FOR POMPTON LAKES DAM IS 1.32 FEET. HIGHER HEADS WILL PRODUCE SUBATMOSPHERIC PRESSURES.

DEVELOPMENT OF NEGATIVE PRESSURES

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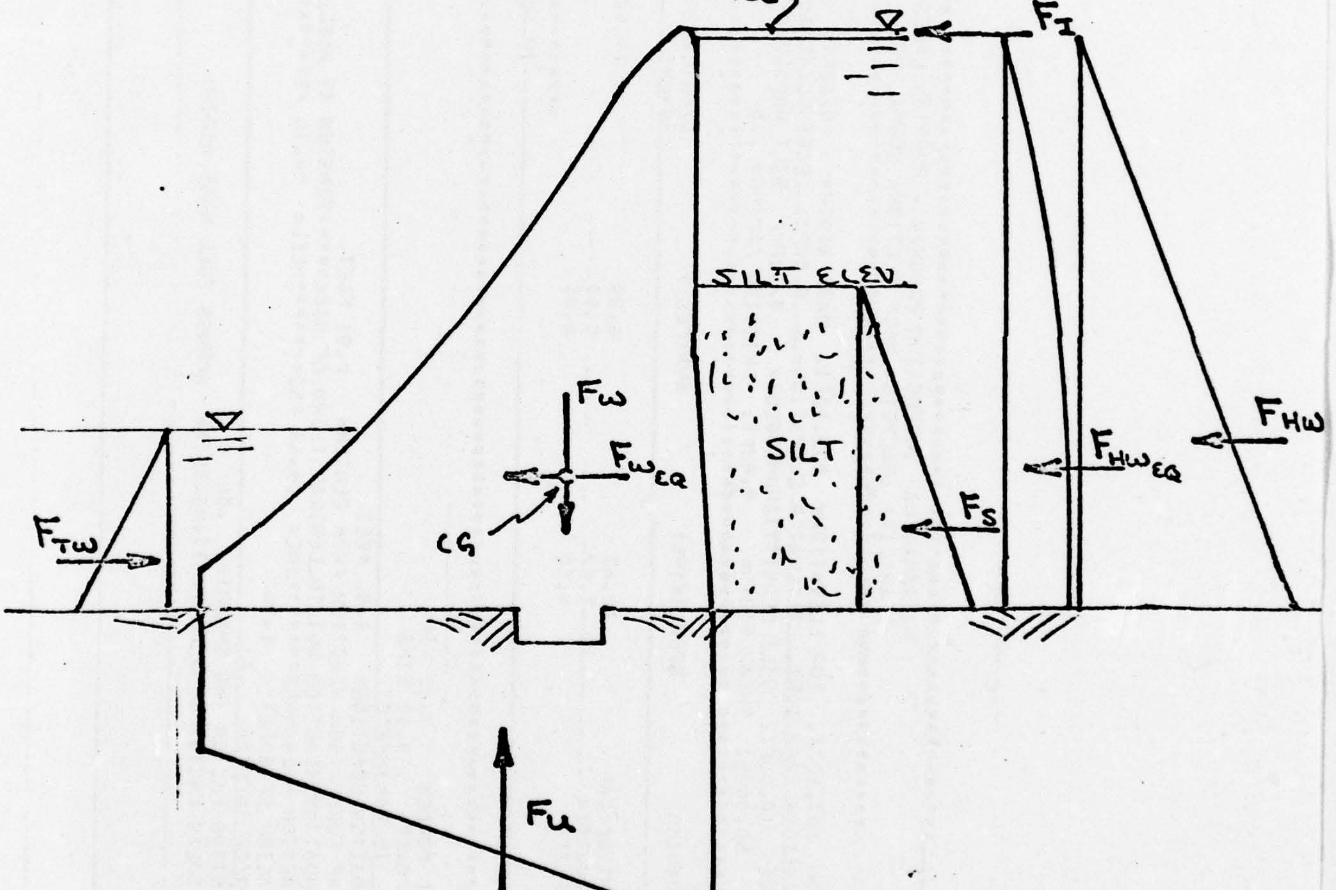
JUSTIN & COURTNEY, INC.
Division of O'Brien & Gere Engineers, Inc.
PHILADELPHIA, PA

SHEET NO._____ OF _____
DATE 3/31/78
COMP. BY DBC
CHECKED BY _____

NAME OF CLIENT Corps of Engineers
PROJECT Pompton Lakes Dam

The forces shown below were used as necessary and with magnitudes varying for each analysis.



F_{HW} - headwater force
$F_{HW_{EQ}}$ - headwater inertia (earthquake)
F_I - ice force
F_S - silt force
F_{TW} - tailwater force
F_u - uplift
F_W - weight of dam
$F_{W_{EQ}}$ - dam inertia (earthquake)

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NATIONAL DAM INSPECTION PROGRAM - POMPTON LAKES DAM
STABILITY ANALYSIS, HALF SECTION, NORMAL POOL

BASE ELEVATION= 187.10FT, TOP ELEVATION= 202.10FT, BASE WIDTH= 9.62FT, DENSITY= 145.00PCF
HEADWATER ELEVATION= 202.10FT, TAILWATER ELEVATION= 0.00FT, EARTHQUAKE ACCELERATION= .000G (HORIZ), .000G (VERT)
SILT ELEVATION= 0.00FT, SILT DENSITY(SUBMERGED)= 0.00PCF SILT PRESSURE COEFFICIENT(K)= .33
SHEAR STRESS= 50.00PSI SHEAR WIDTH= 9.62FT, FRICTION FACTOR= .70

LOADING	FORCE(KIPS)	ARM(FEET)	STABILIZING MOMENT	OVERTURNING MOMENT
WEIGHT OF DAM	12.62	6.29	79.40	
HEADWATER	7.02	5.00		35.06
UPLIFT	4.50	6.41		28.87
			79.40	63.94

NET HORIZONTAL FORCE= 7.02 KIPS
NET VERTICAL FORCE= 8.12 KIPS
NET MOMENT= 15.46KIP-FEET
X-BAR OF FOUNDATION REACTION= 1.90 FEET
ECCENTRICITY OF FOUNDATION REACTION FROM CENTER= 2.91 FEET
*********FOUNDATION REACTION NOT IN CENTRAL THIRD OF BASE*****TENSION AT HEEL OF DAM*****
FOUNDATION REACTION PRESSURES*******TOE= 16.49 PSI*******HEEL= -4.76 PSI*******
OVERTURNING FACTOR OF SAFETY= 1.24
SLIDING FACTOR OF SAFETY= .81
DEVELOPED FRICTION FACTOR (NO SHEAR)= .86
SLIDING WITH SHEAR FACTOR OF SAFETY= 10.68(SHEAR ACROSS FULL BASE WIDTH)

NATIONAL DAM INSPECTION PROGRAM - POMPTON LAKES DAM
STABILITY ANALYSIS, HALF SECTION, EARTHQUAKE

BASE ELEVATION= 187.10FT. TOP ELEVATION= 202.10FT. BASE WIDTH= 9.62FT. DENSITY= 145.00PCF
HEADWATER ELEVATION= 202.10FT. TAILWATER ELEVATION= 0.00FT. EARTHQUAKE ACCELERATION= .050G (HORIZ), .000G (VERT)
SILT ELEVATION= 0.00FT. SILT DENSITY(SUBMERGED)= 0.00PCF SILT PRESSURE COEFFICIENT(K)= .33
SHEAR STRESS= 50.00PSI SHEAR WIDTH= 9.62FT. FRICTION FACTOR= .70

LOADING	FORCE(KIPS)	ARM(FEET)	STABILIZING MOMENT	OVERTURNING MOMENT
WEIGHT OF DAM	12.62	6.29	79.40	
HEADWATER	7.02	5.00		35.06
UPLIFT	4.50	6.41		28.87
EARTHQUAKE INDUCED LOADINGS				
INERTIA-WATER	.38	6.00		2.30
HORIZONTAL INERTIA-DAM	.63	5.86		3.70
			79.40	69.93

NET HORIZONTAL FORCE= 8.03 KIPS
NET VERTICAL FORCE= 8.12 KIPS
NET MOMENT= 9.47KIP-FEET
X-BAR OF FOUNDATION REACTION= 1.17 FEET
ECCENTRICITY OF FOUNDATION REACTION FROM CENTER= 3.64 FEET
*********FOUNDATION REACTION NOT IN CENTRAL THIRD OF BASE*****TENSION AT HEEL OF DAM*****
FOUNDATION REACTION PRESSURES*******TOE= 19.19 PSI*******HEEL= -7.46 PSI*******
OVERTURNING FACTOR OF SAFETY= 1.14
SLIDING FACTOR OF SAFETY= .71
DEVELOPED FRICTION FACTOR (NO SHEAR)= .99
SLIDING WITH SHEAR FACTOR OF SAFETY= 9.33(SHEAR ACROSS FULL BASE WIDTH)

NATIONAL DAM INSPECTION PROGRAM - POMPTON LAKES DAM
STABILITY ANALYSIS, HALF SECTION, ICE LOADING

BASE ELEVATION= 187.10FT. TOP ELEVATION= 202.10FT. BASE WIDTH= 9.62FT. DENSITY= 145.00PCF
HEADWATER ELEVATION= 202.10FT. TAILWATER ELEVATION= 0.00FT. EARTHQUAKE ACCELERATION= .000G (HORIZ), .000G (VERT)
SILT ELEVATION= 0.00FT. SILT DENSITY(SUBMERGED)= 0.00PCF SILT PRESSURE COEFFICIENT(K)= .33
SHEAR STRESS= 50.00PSI SHEAR WIDTH= 9.62FT. FRICTION FACTOR= .70

LOADING	FORCE(KIPS)	ARM(FEET)	STABILIZING MOMENT	OVERTURNING MOMENT
WEIGHT OF DAM	12.62	6.29	79.40	
HEADWATER	7.02	5.00		35.06
UPLIFT	4.50	6.41		28.87
ICE LOAD	5.00	14.50		72.50
			79.40	136.44

NET HORIZONTAL FORCE= 12.02 KIPS
NET VERTICAL FORCE= 8.12 KIPS
NET MOMENT= -57.04KIP-FEET
X-BAR OF FOUNDATION REACTION= -7.02 FEET
ECCENTRICITY OF FOUNDATION REACTION FROM CENTER= 11.83 FEET
*********FOUNDATION REACTION NOT IN CENTRAL THIRD OF BASE****TENSION AT HEEL OF DAM****
FOUNDATION REACTION PRESSURES*******TOE= 49.13 PSI*******HEEL= -37.41 PSI*******
OVERTURNING FACTOR OF SAFETY= .58
SLIDING FACTOR OF SAFETY= .47
DEVELOPED FRICTION FACTOR (NO SHEAR)= 1.48
SLIDING WITH SHEAR FACTOR OF SAFETY= 6.24(SHEAR ACROSS FULL BASE WIDTH)

NATIONAL DAM INSPECTION PROGRAM - POMPTON LAKES DAM
STABILITY ANALYSIS, HALF SECTION, 500 YR. FLD.

BASE ELEVATION= 187.10FT. TOP ELEVATION= 202.10FT. BASE WIDTH= 9.62FT. DENSITY= 145.00PCF
HEADWATER ELEVATION= 210.70FT. TAILWATER ELEVATION= 194.40FT. EARTHQUAKE ACCELERATION= .000G (HORIZ), .000G (VERT).
SILT ELEVATION= 0.00FT. SILT DENSITY(SUBMERGED)= 0.00PCF SILT PRESSURE COEFFICIENT(K)= .33
SHEAR STRESS= 50.00PSI SHEAR WIDTH= 9.62FT. FRICTION FACTOR= .70

LOADING	FORCE(KIPS)	ARM(FEET)	STABILIZING MOMENT	OVERTURNING MOMENT
WEIGHT OF DAM	12.62	6.29	79.40	
HEADWATER	15.67	6.34		95.47
TAILWATER	1.66	2.43	4.04	
UPLIFT	9.27	5.66		52.45
			83.44	147.93

NET HORIZONTAL FORCE= 13.41 KIPS
NET VERTICAL FORCE= 3.35 KIPS
NET MOMENT= -64.48KIP-FEET
X-BAR OF FOUNDATION REACTION= -19.25 FEET
ECCENTRICITY OF FOUNDATION REACTION FROM CENTER= 24.06 FEET
**********FOUNDATION REACTION NOT IN CENTRAL THIRD OF BASE*****TENSION AT HEEL OF DAM*****
FOUNDATION REACTION PRESSURES*******TOE= 38.70 PSI*******HEEL= -33.87 PSI*******
OVERTURNING FACTOR OF SAFETY= .56
SLIDING FACTOR OF SAFETY= .17
DEVELOPED FRICTION FACTOR (NO SHEAR)= 4.00
SLIDING WITH SHEAR FACTOR OF SAFETY= 5.34(SHEAR ACROSS FULL BASE WIDTH).

NATIONAL DAM SAFETY PROGRAM-POMPTON LAKES DAM
STABILITY ANALYSIS, HALF SECTION, PROBABLE MAXIMUM FLOOD

BASE ELEVATION= 197.10FT. TOP ELEVATION= 202.10FT. BASE WIDTH= 9.62FT. DENSITY= 145.00PCF
HEADWATER ELEVATION= 212.10FT. TAILWATER ELEVATION= 195.00FT. EARTHQUAKE ACCELERATION***.000G (HORIZ),.000G (VERT)
SILT ELEVATION= 0.00FT. SILT DENSITY(SUBMERGED)= 0.00PCF SILT PRESSURE COEFFICIENT(K)= .33
SHEAR STRESS= 50.00PSI SHEAR WIDTH= 9.62FT. FRICTION FACTOR= .70

LOADING	FORCE(KIPS)	ARM(FEET)	STABILIZING MOMENT	OVERTURNING MOMENT
WEIGHT OF DAM	12.20	6.28	76.59	
HEADWATER	16.29	6.42		104.60
TAILWATER	2.47	2.95	7.32	
UPLIFT	10.14	5.57		56.50
			*********	*********
			83.91	161.09

NET HORIZONTAL FORCE= 13.82 KIPS
NET VERTICAL FORCE= 2.06 KIPS
NET MOMENT= -77.18KIP-FEET
X-BAR OF FOUNDATION REACTION= -37.48 FEET
ECCENTRICITY OF FOUNDATION REACTION FROM CENTER= 42.29 FEET
**********FOUNDATION REACTION NOT IN CENTRAL THIRD OF BASE*****TENSION AT HEEL OF DAM*****
FOUNDATION REACTION PRESSURES*******TOE= 40.70 PSI*******HEEL= -37.72 PSI*******
OVERTURNING FACTOR OF SAFETY= .52
SLIDING FACTOR OF SAFETY= .10
DEVELOPED FRICTION FACTOR (NO SHEAR)= 6.71
SLIDING WITH SHEAR FACTOR OF SAFETY= 5.12(SHEAR ACROSS FULL BASE WIDTH)

NATIONAL DAM INSPECTION PROGRAM - POMPTON LAKES DAM
STABILITY ANALYSIS, FULL SECTION, NORMAL POOL

BASE ELEVATION= 170.60FT, TOP ELEVATION= 202.10FT, BASE WIDTH= 22.25FT, DENSITY= 145.00PCF
HEADWATER ELEVATION= 202.10FT, TAILWATER ELEVATION= 0.00FT, EARTHQUAKE ACCELERATION= .000G (HORIZ), .000G (VERT)
SILT ELEVATION= 188.00FT, SILT DENSITY(SUBMERGED)= 70.00PCF SILT PRESSURE COEFFICIENT(K)= .33
SHEAR STRESS= 50.00PSI SHEAR WIDTH= 22.25FT, FRICTION FACTOR= .70

LOADING	FORCE(KIPS)	ARM(FEET)	STABILIZING MOMENT	OVERTURNING MOMENT
WEIGHT OF DAM	50.82	15.05	764.99	
HEADWATER	30.96	10.49		324.74
UPLIFT	21.87	14.83		324.36
SILT	3.53	5.80		20.47
			764.99	669.57

NET HORIZONTAL FORCE= 34.49 KIPS
NET VERTICAL FORCE= 28.96 KIPS
NET MOMENT= 95.43KIP-FEET
X-BAR OF FOUNDATION REACTION= 3.30 FEET
ECCENTRICITY OF FOUNDATION REACTION FROM CENTER= 7.83 FEET
**********FOUNDATION REACTION NOT IN CENTRAL THIRD OF BASE*****TENSION AT HEEL OF DAM*****
FOUNDATION REACTION PRESSURES*******TOE= 28.12 PSI*******HEEL= -10.04 PSI*******
OVERTURNING FACTOR OF SAFETY= 1.14
SLIDING FACTOR OF SAFETY= .59
DEVELOPED FRICTION FACTOR (NO SHEAR)= 1.19
SLIDING WITH SHEAR FACTOR OF SAFETY= 5.23(SHEAR ACROSS FULL BASE WIDTH)

NATIONAL DAM INSPECTION PROGRAM - POMPTON LAKES DAM
STABILITY ANALYSIS, FULL SECTION, EARTHQUAKE

BASE ELEVATION= 170.60FT. TOP ELEVATION= 202.10FT. BASE WIDTH= 22.25FT. DENSITY= 145.00PCF
HEADWATER ELEVATION= 202.10FT. TAILWATER ELEVATION= 0.00FT. EARTHQUAKE ACCELERATION= .050G (HORIZ), .000G (VERT)
SILT ELEVATION= 188.00FT. SILT DENSITY(SUBMERGED)= 70.00PCF SILT PRESSURE COEFFICIENT(K)= .33
SHEAR STRESS= 50.00PSI SHEAR WIDTH= 22.25FT. FRICTION FACTOR= .70

LOADING	FORCE(KIPS)	ARM(FEET)	STABILIZING MOMENT	OVERTURNING MOMENT
WEIGHT OF DAM	50.82	15.05	764.99	
HEADWATER	30.96	10.49		324.74
UPLIFT	21.87	14.83		324.36
EARTHQUAKE INDUCED LOADINGS				
INERTIA-WATER	1.69	12.60		21.25
HORIZONTAL INERTIA-DAM	2.54	10.87		27.62
SILT	3.53	5.80		20.47
			764.99	718.44

NET HORIZONTAL FORCE= 38.71 KIPS
NET VERTICAL FORCE= 28.96 KIPS
NET MOMENT= 46.56KIP-FEET
X-BAR OF FOUNDATION REACTION= 1.61 FEET
ECCENTRICITY OF FOUNDATION REACTION FROM CENTER= 9.52 FEET
**********FOUNDATION REACTION NOT IN CENTRAL THIRD OF BASE*****TENSION AT HEEL OF DAM*****
FOUNDATION REACTION PRESSURES*******TOE= 32.23 PSI*******HEEL= -14.16 PSI*******
OVERTURNING FACTOR OF SAFETY= 1.06
SLIDING FACTOR OF SAFETY= .52
DEVELOPED FRICTION FACTOR (NO SHEAR)= 1.34
SLIDING WITH SHEAR FACTOR OF SAFETY= 4.66(SHEAR ACROSS FULL BASE WIDTH)

NATIONAL DAM INSPECTION PROGRAM - POMPTON LAKES DAM
STABILITY ANALYSIS, FULL SECTION, ICE LOADING

BASE ELEVATION= 170.60FT. TOP ELEVATION= 202.10FT. BASE WIDTH= 22.25FT. DENSITY= 145.00PCF
HEADWATER ELEVATION= 202.10FT. TAILWATER ELEVATION= 0.00FT. EARTHQUAKE ACCELERATION= .000G (HORIZ), .000G (VERT)
SILT ELEVATION= 188.00FT. SILT DENSITY(SUBMERGED)= 70.00PCF SILT PRESSURE COEFFICIENT(K)= .33
SHEAR STRESS= 50.00PSI SHEAR WIDTH= 22.25FT. FRICTION FACTOR= .70

LOADING	FORCE (KIPS)	ARM(FEET)	STABILIZING MOMENT	OVERTURNING MOMENT
WEIGHT OF DAM	50.82	15.05	764.99	
HEADWATER	-30.96	10.49		324.74
UPLIFT	21.87	14.83		324.36
SILT	3.53	5.80		20.47
ICE LOAD	5.00	31.00		155.00
			764.99	824.57

NET HORIZONTAL FORCE= 39.49 KIPS
NET VERTICAL FORCE= 28.96 KIPS
NET MOMENT= -59.57KIP-FEET
X-BAR OF FOUNDATION REACTION= -2.06 FEET
ECCENTRICITY OF FOUNDATION REACTION FROM CENTER= 13.18 FEET
**********FOUNDATION REACTION NOT IN CENTRAL THIRD OF BASE*****TENSION AT HEEL OF DAM*****
FOUNDATION REACTION PRESSURES*******TOE= 41.17 PSI*******HEEL= -23.09 PSI*******
OVERTURNING FACTOR OF SAFETY= .93
SLIDING FACTOR OF SAFETY= .51
DEVELOPED FRICTION FACTOR (NO SHEAR)= 1.36
SLIDING WITH SHEAR FACTOR OF SAFETY= 4.57(SHEAR ACROSS FULL BASE WIDTH)

NATIONAL DAM INSPECTION PROGRAM - POMPTON LAKES DAM
STABILITY ANALYSIS, FULL SECTION, 500 YR. FLD.

BASE ELEVATION= 170.60FT. TOP ELEVATION= 202.10FT. BASE WIDTH= 22.25FT. DENSITY= 145.00PCF
HEADWATER ELEVATION= 210.70FT. TAILWATER ELEVATION= 194.40FT. EARTHQUAKE ACCELERATION= .000G (HORIZ) .000G (VERT)
SILT ELEVATION= 188.00FT. SILT DENSITY(SUBMERGED)= 70.00PCF SILT PRESSURE COEFFICIENT(K)= .33
SHEAR STRESS= 50.00PSI SHEAR WIDTH= 22.25FT. FRICTION FACTOR= .70

LOADING	FORCE (KIPS)	ARM (FEET)	STABILIZING MOMENT	OVERTURNING MOMENT
WEIGHT OF DAM	50.82	15.05	764.99	
HEADWATER	47.86	12.35		591.30
TAILWATER	17.67	7.93	140.07	
UPLIFT	44.36	12.07		535.46
SILT	3.53	5.80		20.47
			905.06	1147.23

NET HORIZONTAL FORCE= 33.72 KIPS
NET VERTICAL FORCE= 6.46 KIPS
NET MOMENT= -242.17KIP-FEET
X-BAR OF FOUNDATION REACTION= -37.46 FEET
ECCENTRICITY OF FOUNDATION REACTION FROM CENTER= 48.58 FEET
**********FOUNDATION REACTION NOT IN CENTRAL THIRD OF BASE*****TENSION AT HEEL OF DAM*****
FOUNDATION REACTION PRESSURES*******TOE= 28.45 PSI*******HEEL= -24.42 PSI*******
OVERTURNING FACTOR OF SAFETY= .79
SLIDING FACTOR OF SAFETY= .13
DEVELOPED FRICTION FACTOR (NO SHEAR)= 5.22
SLIDING WITH SHEAR FACTOR OF SAFETY= 4.89(SHEAR ACROSS FULL BASE WIDTH)

NATIONAL DAM SAFETY PROGRAM-POMPTON LAKES DAM
STABILITY ANALYSIS, FULL SECTION, PROBABLE MAXIMUM FLOOD

BASE ELEVATION= 170.50FT. TOP ELEVATION= 202.10FT. BASE WIDTH= 22.25FT. DENSITY= 145.00PCF
HEADWATER ELEVATION= 212.00FT. TAILWATER ELEVATION= 196.00FT. EARTHQUAKE ACCELERATION***.000G (HORIZ),.000G (VERT)
SILT ELEVATION= 0.00FT. SILT DENSITY(SUBMERGED)= 0.00PCF SILT PRESSURE COEFFICIENT(K)= .33
SHEAR STRESS= 50.00PSI SHEAR WIDTH= 22.25FT. FRICTION FACTOR= .70

LOADING	FORCE(KIPS)	ARM(FEET)	STABILIZING MOMENT	OVERTURNING MOMENT
WEIGHT OF DAM	50.43	15.02	757.26	
HEADWATER	50.42	12.53		631.55
TAILWATER	20.13	8.46	170.26	
UPLIFT	46.37	12.01		557.08
			*********	*********
			927.51	1188.63

NET HORIZONTAL FORCE= 30.29 KIPS
NET VERTICAL FORCE= 4.06 KIPS
NET MOMENT= -261.12KIP-FEET
X-BAR OF FOUNDATION REACTION= -64.37 FEET
ECCENTRICITY OF FOUNDATION REACTION FROM CENTER= 75.50 FEET
**********FOUNDATION REACTION NOT IN CENTRAL THIRD OF BASE*****TENSION AT HEEL OF DAM*****
FOUNDATION REACTION PRESSURES*******TOE= 27.04 PSI*******HEEL= -24.51 PSI*******
OVERTURNING FACTOR OF SAFETY= .78
SLIDING FACTOR OF SAFETY= .09
DEVELOPED FRICTION FACTOR (NO SHEAR)= 7.47
SLIDING WITH SHEAR FACTOR OF SAFETY= 5.38(SHEAR ACROSS FULL BASE WIDTH)

RECOMMENDED GUIDELINES

FOR SAFETY INSPECTION OF DAMS

CHAPTER 4

CHAPTER 4 - PHASE II INVESTIGATION

4.1. Purpose. The Phase II investigation will be supplementary to Phase I and should be conducted when the results of the Phase I investigation indicate the need for additional in-depth studies, investigations or analyses.

4.2. Scope. The Phase II investigation should include all additional studies, investigations and analyses necessary to evaluate the safety of the dam. Included, as required, will be additional visual inspections, measurements, foundation exploration and testing, materials testing, hydraulic and hydrologic analysis and structural stability analyses.

4.3. Hydraulic and Hydrologic Analysis. Hydraulic and hydrologic capabilities should be determined using the following criteria and procedures. Depending on the project characteristics, either the spillway design flood peak inflow or the spillway design flood hydrograph should be the basis for determining the maximum water surface elevation and maximum outflow. If the operation or failure of upstream water control projects would have significant impact on peak flow or hydrograph analyses, the impact should be assessed.

4.3.1. Maximum Water Surface Based on SDF Peak Inflow. When the total project discharge capability at maximum pool exceeds the peak inflow of the recommended SDF, and operational constraints would not prevent such a release at controlled projects, a reservoir routing is not required. The maximum discharge should be assumed equal to the peak inflow of the spillway design flood. Flood volume is not controlling in this situation and surcharge storage is either absent or is significant only to the extent that it provides the head necessary to develop the release capability required.

4.3.1.1. Peak for 100-Year Flood. When the 100-year flood is applicable under the provisions of Table 3 and data are available, the spillway design flood peak inflow may be determined by use of "A Uniform Technique for Determining Flood Frequencies," Water Resources Council (WRC), Hydrology Committee, Bulletin 15, December 1967. Flow frequency information from regional analysis is generally preferred over single station results when available and appropriate. Rainfall-runoff techniques may be necessary when there are inadequate runoff data available to make a reasonable estimate of flow frequency.

4.3.1.2. Peak for PMF or Fraction Thereof. When either the Probable Maximum Flood peak or a fraction thereof is applicable under the provisions of Table 3, the unit hydrograph - infiltration loss technique is generally the most expeditious method of computing the spillway design flood peak for most projects. This technique is discussed in the following paragraph.

4.3.2. Maximum Water Surface Based on SDF Hydrograph. Both peak and volume are required in this analysis. Where surcharge storage is significant, or where there is insufficient discharge capability at maximum pool to pass the peak inflow of the SDF, considering all possible operational constraints, a flood hydrograph is required. When there are upstream hazard areas that would be imperiled by fast rising reservoirs levels, SDF hydrographs should be routed to ascertain available time for warning and escape. Determination of probable maximum precipitation or 100-year precipitation, whichever is applicable, and unit hydrographs or runoff models will be required, followed by the determination of the PMF or 100-year flood. Conservative loss rates (significantly reduced by antecedent rainfall conditions where appropriate) should be estimated for computing the rainfall excess to be utilized with unit hydrographs. Rainfall values are usually arranged with gradually ascending and descending rates with the maximum rate late in the storm. When applicable, conservatively high snowmelt runoff rates and appropriate releases from upstream projects should be assumed. The PMP may be obtained from National Weather Service (NWS) publications such as Hydrometeorological Report (HMR) 33. Special NWS publications for particular areas should be used when available. Rainfall for the 100-year frequency flood can be obtained from the NWS publication "Rainfall Frequency Atlas of the United States," Technical Paper No. 40; Atlas 2, "Precipitation Frequency Atlas of Western United States;" or other NWS publications. The maximum water surface elevation and spillway design flood outflow are then determined by routing the inflow hydrograph through the reservoir surcharge storage, assuming a starting water surface at the bottom of surcharge storage, or lower when appropriate. For projects where the bottom of surcharge space is not distinct, or the flood control storage space (exclusive of surcharge) is appreciable, it may be appropriate to select starting water surface elevations below the top of the flood control storage for routings. Conservatively high starting levels should be estimated on the basis of hydrometeorological conditions reasonably characteristic for the region and flood release capability of the project. Necessary adjustment of reservoir storage capacity due to existing or future sediment or other encroachment may be approximated when accurate determination of deposition is not practicable.

4.3.3. Acceptable Procedures. Techniques for performing hydraulic and hydrologic analyses are generally available from publications prepared by Federal agencies involved in water resources development or textbooks written by the academic community. Some of these procedures are rather sophisticated and require expensive computational equipment and large data banks. While results of such procedures are generally more reliable than simplified methods, their use is generally not warranted in studies connected with this program unless they can be performed quickly and inexpensively. There may be situations where the more complex techniques have to be employed to obtain reliable results; however, these cases will be exceptions rather than the rule. Whenever the acceptability of procedures is in question, the advice of competent experts should be sought. Such expertise is generally available in the Corps of Engineers, Bureau of

Reclamation and Soil Conservation Service. Many other agencies, educational facilities and private consultants can also provide expert advice. Regardless of where such expertise is based, the qualification of those individuals offering to provide it should be carefully examined and evaluated.

4.3.4. Freeboard Allowances. Guidelines on specific minimum freeboard allowances are not considered appropriate because of the many factors involved in such determinations. The investigator will have to assess the critical parameters for each project and develop its minimum requirement. Many projects are reasonably safe without freeboard allowance because they are designed for overtopping, or other factors minimize possible overtopping. Conversely, freeboard allowances of several feet may be necessary to provide a safe condition. Parameters that should be considered include the duration of high water levels in the reservoir during the design flood; the effective wind fetch and reservoir depth available to support wave generation; the probability of high wind speed occurring from a critical direction; the potential wave runup on the dam based on roughness and slope; and the ability of the dam to resist erosion from overtopping waves.

4.4. Stability Investigations. The Phase II stability investigations should be compatible with the guidelines of this paragraph.

4.4.1. Foundation and Material Investigations. The scope of the foundation and materials investigation should be limited to obtaining the information required to analyze the structural stability and to investigate any suspected condition which would adversely affect the safety of the dam. Such investigations may include borings to obtain concrete, embankment, soil foundation, and bedrock samples; testing specimens from these samples to determine the strength and elastic parameters of the materials, including the soft seams, joints, fault gouge and expansive clays or other critical materials in the foundation; determining the character of the bedrock including joints, bedding planes, fractures, faults, voids and caverns, and other geological irregularities; and installing instruments for determining movements, strains, suspected excessive internal seepage pressures, seepage gradients and uplift forces. Special investigations may be necessary where suspect rock types such as limestone, gypsum, salt, basalt, claystone, shales or others are involved in foundations or abutments in order to determine the extent of cavities, piping or other deficiencies in the rock foundation. A concrete core drilling program should be undertaken only when the existence of significant structural cracks is suspected or the general qualitative condition of the concrete is in doubt. The tests of materials will be necessary only where such data are lacking or are outdated.

4.4.2. Stability Assessment. Stability assessments should utilize in situ properties of the structure and its foundation and pertinent geologic information. Geologic information that should be considered includes groundwater and seepage conditions; lithology, stratigraphy, and geologic details disclosed by borings, "as-built" records, and geologic interpretation; maximum past overburden at site as deduced from geologic evidence; bedding, folding and faulting; joints and joint systems; weathering; slickensides, and field evidence relating to slides, faults, movements and earthquake activity. Foundations may present problems where they contain adversely oriented joints, slickensides or fissured material, faults, seams of soft materials, or weak layers. Such defects and excess pore water pressures may contribute to instability. Special tests may be necessary to determine physical properties of particular materials. The results of stability analyses afford a means of evaluating the structure's existing resistance to failure and also the effects of any proposed modifications. Results of stability analyses should be reviewed for compatibility with performance experience when possible.

4.4.2.1. Seismic Stability. The inertial forces for use in the conventional equivalent static force method of analysis should be obtained by multiplying the weight by the seismic coefficient and should be applied as a horizontal force at the center of gravity of the section or element. The seismic coefficients suggested for use with such analyses are listed in Figures 1 through 4. Seismic stability investigations for all high hazard category dams located in Seismic Zone 4 and high hazard dams of the hydraulic fill type in Zone 3 should include suitable dynamic procedures and analyses. Dynamic analyses for other dams and higher seismic coefficients are appropriate if in the judgment of the investigating engineer they are warranted because of proximity to active faults or other reasons. Seismic stability investigations should utilize "state-of-the-art" procedures involving seismological and geological studies to establish earthquake parameters for use in dynamic stability analyses and, where appropriate, the dynamic testing of materials. Stability analyses may be based upon either time-history or response spectra techniques. The results of dynamic analyses should be assessed on the basis of whether or not the dam would have sufficient residual integrity to retain the reservoir during and after the greatest or most adverse earthquake which might occur near the project location.

4.4.2.2. Clay Shale Foundation. Clay shale is a highly overconsolidated sedimentary rock comprised predominantly of clay minerals, with little or no cementation. Foundations of clay shales require special measures in stability investigations. Clay shales, particularly those containing montmorillonite, may be highly susceptible to expansion and consequent loss of strength upon unloading. The shear strength and the resistance to deformation of clay shales may be quite low and high pore water pressures may develop under increase in load. The presence of slickensides in clay shales is usually an indication of low shear stength. Prediction of field behavior of clay shales should not be based solely on results of conventional laboratory tests since they may be misleading. The use of peak shear strengths for clay shales in stability analyses may be unconservative because of nonuniform stress distribution and possible progressive failures. Thus the available shear resistance may be less than if the peak shear strength were mobilized simultaneously along the entire failure surface. In such cases, either greater safety factors or residual shear strength should be used.

4.4.3. Embankment Dams.

4.4.3.1. Liquefaction. The phenomenon of liquefaction of loose, saturated sands and silts may occur when such materials are subjected to shear deformation or earthquake shocks. The possibility of liquefaction must presently be evaluated on the basis of empirical knowledge supplemented by special laboratory tests and engineering judgment. The possibility of liquefaction in sands diminishes as the relative density increases above approximately 70 percent. Hydraulic fill dams in Seismic Zones 3 and 4 should receive particular attention since such dams are susceptible to liquefaction under earthquake shocks.

4.4.3.2. Shear Failure. Shear failure is one in which a portion of an embankment or of an embankment and foundation moves by sliding or rotating relative to the remainder of the mass. It is conventionally represented as occurring along a surface and is so assumed in stability analyses, although shearing may occur in a zone of substantial thickness. The circular arc or the sliding wedge method of analyzing stability, as pertinent, should be used. The circular arc method is generally applicable to essentially homogeneous embankments and to soil foundations consisting of thick deposits of fine-grained soil containing no layers significantly weaker than other strata in the foundation. The wedge method is generally applicable to rockfill dams and to earth dams on foundations containing weak layers. Other methods of analysis such as those employing complex shear surfaces may be appropriate depending on the soil and rock in the dam and foundation. Such methods should be in reputable usage in the engineering profession.

4.4.3.3. Loading Conditions. The loading conditions for which the embankment structures should be investigated are (I) Sudden drawdown from spillway crest elevation or top of gates, (II) Partial pool, (III) Steady state seepage from spillway crest elevation or top of gate elevation, and (IV) Earthquake. Cases I and II apply to upstream slopes only; Case III applies to downstream slopes; and Case IV applies to both upstream and downstream slopes. A summary of suggested strengths and safety factors are shown in Table 4.

TABLE 4

FACTORS OF SAFETY /

Case	Loading Condition	Factor of Safety	Shear // Strength	Remarks
I	Sudden drawdown from spillway crest or top of gates to minimum drawdown elevation.	1.2*	Minimum composite of R and S shear strengths See Figure 5.	Within the drawdown zone submerged unit weights of materials are used for computing forces resisting sliding and saturated unit weights are used for computing forces contributing to sliding.
II	Partial pool with assumed horizontal steady seepage saturation.	1.5	$\frac{R+S}{2}$ for $R < S$ S for $R > S$	Composite intermediate envelope of R and S shear strengths. See Figure 6.
III	Steady seepage from spillway crest or top of gates with $K_h/K_v = 9$ assumed**	1.5	Same as Case II	
IV	Earthquake (Cases II and III with seismic loading)	1.0	***	See Figures 1 through 4 for Seismic Coefficients.

/ Not applicable to embankments on clay shale foundation. Experience has indicated special problems in determination of design shear strengths for clay shale foundations and acceptable safety factors should be compatible with the confidence level in shear strength assumptions.

// Other strength assumptions may be used if in common usage in the engineering profession.

* The safety factor should not be less than 1.5 when drawdown rate and pore water pressure developed from flow nets are used in stability analyses.

** K_h/K_v is the ratio of horizontal to vertical permeability. A minimum of 9 is suggested for use in compacted embankments and alluvial sediments.

*** Use shear strength for case analyzed without earthquake. It is not necessary to analyze sudden drawdown for earthquake loading. Shear strength tests are classified according to the controlled drainage conditions maintained during the test. R tests are those in which specimen drainage is allowed during consolidation (or swelling) under initial stress conditions, but specimen drainage is not allowed during application of shearing stresses. S tests allow full drainage during initial stress application and shearing is at a slow rate so that complete specimen drainage is permitted during the complete test.

4.4.3.4. <u>Safety Factors</u>. Safety factors for embankment dam stability studies should be based on the ratio of available shear strength to developed shear strength, S_D:

$$S_D = \frac{C}{F.S.} + \sigma \frac{\tan \phi}{F.S.} \tag{1}$$

C = cohesion

ϕ = angle of internal friction

σ = normal stress

The factors of safety listed in Table 4 are recommended as minimum acceptable. Final accepted factors of safety should depend upon the degree of confidence the investigating engineer has in the engineering data available to him. The consequences of a failure with respect to human life and property damage are important considerations in establishing factors of safety for specific investigations.

4.4.3.5. <u>Seepage Failure</u>. A critical uncontrolled underseepage or through seepage condition that develops during a rising pool can quickly reduce a structure which was stable under previous conditions, to a total structural failure. The visually confirmed seepage conditions to be avoided are (1) the exit of the phreatic surface on the downstream slope of the dam and (2) development of hydrostatic heads sufficient to create in the area downstream of the dam sand boils that erode materials by the phenomenon known as "piping" and (3) localized concentrations of seepage along conduits or through pervious zones. The dams most susceptible to seepage problems are those built of or on pervious materials of uniform fine particle size, with no provisions for an internal drainage zone and/or no underseepage controls.

4.4.3.6. Seepage Analyses. Review and modifications to original seepage design analyses should consider conditions observed in the field inspection and piezometer instrumentation. A seepage analysis should consider the permeability ratios resulting from natural deposition and from compaction placement of materials with appropriate variation between horizontal and vertical permeability. An underseepage analysis of the embankment should provide a critical gradient factor of safety for the maximum head condition of not less than 1.5 in the area downstream of the embankment.

$$F.S = i_c/i = \frac{H_c/D_b}{H/D_b} = D_b \frac{(\gamma_m - \gamma_w)}{H\gamma_w} \qquad (2)$$

i_c = Critical gradient

i = Design gradient

H = Uplift head at downstream toe of dam measured above tailwater

H_c = The critical uplift

D_b = The thickness of the top impervious blanket at the downstream toe of the dam

γ_m = The estimated saturated unit weight of the material in the top impervious blanket

γ_w = The unit weight of water

Where a factor of safety less than 1.5 is obtained the provision of an underseepage control system is indicated. The factor of safety of 1.5 is a recommended minimum and may be adjusted by the responsible engineer based on the competence of the engineering data.

4.4.4. Concrete Dams and Appurtenant Structures.

4.4.4.1. Requirements for Stability. Concrete dams and structures appurtenant to embankment dams should be capable of resisting overturning, sliding and overstressing with adequate factors of safety for normal and maximum loading conditions.

4.4.4.2. Loads. Loadings to be considered in stability analyses include the water load on the upstream face of the dam; the weight of the structure, internal hydrostatic pressures (uplift) within the body of the dam, at the base of the dam and within the foundation; earth and silt loads; ice pressure, seismic and thermal loads, and other loads as applicable. Where tailwater or backwater exists on the downstream side of the structure it should be considered, and assumed uplift pressures should be compatible with drainage provisions and uplift measurements if available. Where applicable, ice pressure should be applied to the contact surface of the structure at normal pool elevation. A unit pressure of not more than 5,000 pounds per square foot should be used. Normally, ice thickness should not be assumed greater than two feet. Earthquake forces should consist of the inertial forces due to the horizontal acceleration of the dam itself and hydrodynamic forces resulting from the reaction of the reservoir water against the structure. Dynamic water pressures for use in conventional methods of analysis may be computed by means of the "Westergaard Formula" using the parabolic approximation (H.M. Westergaard, "Water Pressures on Dams During Earthquakes," Trans., ASCE, Vol 98, 1933, pages 418-433), or similar method.

4.4.4.3. Stresses. The analysis of concrete stresses should be based on in situ properties of the concrete and foundation. Computed maximum compressive stresses for normal operating conditions in the order of 1/3 or less of in situ strengths should be satisfactory. Tensile stresses in unreinforced concrete should be acceptable only in locations where cracks will not adversely affect the overall performance and stability of the structure. Foundation stresses should be such as to provide adequate safety against failure of the foundation material under all loading conditions.

4.4.4.4. Overturning. A gravity structure should be capable of resisting all overturning forces. It can be considered safe against overturning if the resultant of all combinations of horizontal and vertical forces, excluding earthquake forces, acting above any horizontal plane through the structure or at its base is located within the middle third of the section. When earthquake is included the resultant should fall within the limits of the plane or base, and foundation pressures must be acceptable. When these requirements for location of the resultant are not satisfied the investigating engineer should assess the importance to stability of the deviations.

4.4.4.5. Sliding. Sliding of concrete gravity structures and of abutment and foundation rock masses for all types of concrete dams should be evaluated. by the shear-friction resistance concept. The available sliding resistance is compared with the driving force which tends to induce sliding to arrive at a sliding stability safety factor. The investigation should be made along all potential sliding paths. The critical path is that plane or combination of planes which offers the least resistance.

4.4.4.5.1. Sliding Resistance. Sliding resistance is a function of the unit shearing strength at no normal load (cohesion) and the angle of friction on a potential failure surface. It is determined by computing the maximum horizontal driving force which could be resisted along the sliding path under investigation. The following general formula is obtained from the principles of statics and may be derived by resolving forces parallel and perpendicular to the sliding plane:

$$R_R = V \tan(\phi + \alpha) + \frac{cA}{\cos\alpha(1 - \tan\phi \tan\alpha)} \qquad (3)$$

where

R_R = Sliding Resistance (maximum horizontal driving force which can be resisted by the critical path)

ϕ = Angle of internal friction of foundation material or, where applicable, angle of sliding friction

V = Summation of vertical forces (including uplift)

c = Unit shearing strength at zero normal loading along potential failure plane

A = Area of potential failure plane developing unit shear strength "c"

α = Angle between inclined plane and horizontal (positive for uphill sliding)

For sliding downhill the angle α is negative and Equation (1) becomes:

$$R_R = V \tan(\phi - \alpha) + \frac{cA}{\cos\alpha\ (1 + \tan\phi \tan\alpha)} \qquad (4)$$

When the plane of investigation is horizontal, and the angle α is zero and Equation (1) reduced to the following:

$$R_R = V \tan\phi + cA \qquad (5)$$

4.4.4.5.2. Downstream Resistance. When the base of a concrete structure is embedded in rock or the potential failure plane lies below the base, the passive resistance of the downstream layer of rock may sometimes be utilized for sliding resistance. Rock that may be subjected to high velocity water scouring should not be used. The magnitude of the downstream resistance is the lesser of (a) the shearing resistance along the continuation of the potential sliding plane until it daylights or (b) the resistance available from the downstream rock wedge along an inclined plane. The theoretical resistance offered by the passive wedge can be computed by a formula equivalent to formula (3):

$$P_p = W \tan(\phi + \alpha) + \frac{cA}{\cos\alpha\,(1 - \tan\phi \tan\alpha)} \qquad (6)$$

P_p = passive resistance of rock wedge

W = weight (buoyant weight if applicable) of downstream rock wedge above inclined plane of resistance, plus any superimposed loads

ϕ = angle of internal friction or, if applicable, angle of sliding friction

α = angle between inclined failure plane and horizontal

c = unit shearing strength at zero normal load along failure plane

A = area of inclined plane of resistance

When considering cross-bed shear through a relatively shallow, competent rock strut, without adverse jointing or faulting, W and α may be taken at zero and 45°, respectively, and an estimate of passive wedge resistance per unit width obtained by the following equation:

$$P_p = 2\,cD \qquad (7)$$

where

D = Thickness of the rock strut

4.4.4.5.3. Safety Factor. The shear-friction safety factor is obtained by dividing the resistance R_R by H, the summation of horizontal service loads to be applied to the structure:

$$S_{s-f} = \frac{R_R}{H} \tag{8}$$

When the downstream passive wedge contributes to the sliding resistance, the shear fruction safety factor formula becomes:

$$S_{s-f} = \frac{R_R + P_p}{H} \tag{9}$$

The above direct superimposition of passive wedge resistance is valid only if shearing rigidities of the foundation components are similar. Also, the compressive strength and buckling resistance of the downstream rock layer must be sufficient to develop the wedge resistance. For example, a foundation with closely spaced, near horizontal, relatively weak seams might not contain sufficient buckling strength to develop the magnitude of wedge resistance computed from the cross-bed shear strength. In this case wedge resistance should not be assumed without resorting to special treatment (such as installing foundation anchors). Computed sliding safety factors approximating 3 or more for all loading conditions without earthquake, and 1.5 including earthquake, should indicate satisfactory stability, depending upon the reliability of the strength parameters used in the analyses. In some cases when the results of comprehensive foundation studies are available, smaller safety factors may be acceptable. The selection of shear strength parameters should be fully substantiated. The bases for any assumptions; the results of applicable testing, studies and investigations; and all pre-existing, pertinent data should be reported and evaluated.