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NATIONAL DAM INSPECTION REPORT. WILLIAMS BRIDGE DAM, PENNSYLVAN--ETC(U)
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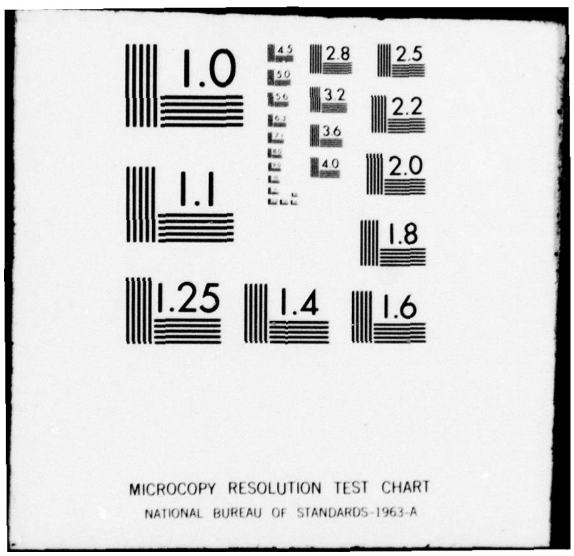

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LEVEL II

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SUSQUEHANNA RIVER BASIN

STAFFORD MEADOW BROOK, LACKAWANNA COUNTY

PENNSYLVANIA

# WILLIAMS BRIDGE DAM

## PENNSYLVANIA GAS AND WATER COMPANY

## PHASE I INSPECTION REPORT

## NATIONAL DAM INSPECTION PROGRAM

DISTRIBUTION STATEMENT A
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(6) National Dam Inspection Report. Williams Bridge Dam, Pennsylvania Gas and Water Company (NDS-ID-373), Susquehanna River Basin, Stafford Meadow Brook, Lackawanna County, Pennsylvania.
Phase I Inspection Report.

Prepared by
GANNETT FLEMING CORDDRY AND CARPENTER, INC.
Consulting Engineers
Harrisburg, Pennsylvania 17105

(15) DACW31-78-C-0046

For
DEPARTMENT OF THE ARMY
Baltimore District, Corps of Engineers
Baltimore, Maryland 21203

(11) MAY 1978

(12) 106p.

LEVEL II

SUSQUEHANNA RIVER BASIN

STAFFORD MEADOW BROOK, LACKAWANNA COUNTY

PENNSYLVANIA

# WILLIAMS BRIDGE DAM

PENNSYLVANIA GAS AND WATER COMPANY
(NDS ID No. 373)

# PHASE I INSPECTION REPORT

# NATIONAL DAM INSPECTION PROGRAM

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For

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Baltimore District, Corps of Engineers
Baltimore, Maryland 21203

MAY 1978

Contract DACW31-78-C-0046







411 004

Elmhurst Dam (File No. 35-18)

1. The spillway capacity is not considered to be seriously inadequate. However, because of the potential for overtopping of the dam, the Company should:

   a. Develop and implement an effective warning system for this dam that will be utilized during periods of unusual precipitation.

2. Complete the maintenance items as noted on Page a-2 of the Report.

It is requested that the Company develop an annual inspection program for each of these dams. This should include an annual report, regarding the condition of the dam, certified by a registered professional engineer, and submitted to this Department by no later than July 1 of each year.

It is requested that you advise this office by no later than July 15, 1978 of the Company's intention relative to complying with the recommendations as listed in the foregoing summary for each dam.

If you have any questions, please contact Joseph J. Ellam, Chief, Dam Safety Section, Division of Dams and Encroachments at 717-787-6826.

Sincerely yours,

Walter A. Lyon, Director
Bureau of Water Quality Management

WAL:VRB:JEE:jb

cc: File
30-day File
Alpha
Dr. Goddard
Col. G. K. Withers, District Engineer, U.S. Army Corps of Engineers
Chief, Facilities Section - Wilkes Barre Regional Office, BWQM
C. H. McConnell

SUSQUEHANNA RIVER BASIN

STAFFORD MEADOW BROOK, LACKAWANNA COUNTY

PENNSYLVANIA

# WILLIAMS BRIDGE DAM

PENNSYLVANIA GAS AND WATER COMPANY
(NDS ID No. 373)

PHASE I INSPECTION REPORT

NATIONAL DAM INSPECTION PROGRAM

MAY 1978

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PHASE I INSPECTION REPORT

NATIONAL DAM INSPECTION PROGRAM

BRIEF ASSESSMENT OF GENERAL CONDITION

AND

RECOMMENDED ACTION

| | |
|---|---|
| Name of Dam: | Williams Bridge Dam (NDS ID No. 373) |
| Owner: | Pennsylvania Gas and Water Company |
| State Located: | Pennsylvania |
| County Located: | Lackawanna |
| Stream: | Stafford Meadow Brook |
| Date of Inspection: | 13 April 1978 |
| Inspection Team: | Gannett Fleming Corddry and Carpenter, Inc.<br>Consulting Engineers<br>P.O. Box 1963<br>Harrisburg, Pennsylvania 17105 |

Based on the visual inspection, available records, calculations and past operational performance, Williams Bridge Dam is judged to be in fair condition. However, the spillway (main and auxiliary) will not pass the Probable Maximum Flood (PMF) or one-half the PMF without overtopping the dam. If Williams Bridge Dam should fail due to overtopping, the hazard to loss of life downstream from the dam would be significantly increased from that which would exist just prior to overtopping the dam. Based on criteria established for these studies, by the Department of the Army, Office of the Chief of Engineers (OCE), the spillway capacity is rated as seriously inadequate. The existing spillway can accommodate a flood with a peak inflow of 34 percent of the PMF peak flow.

For maximum pool condition, stability calculations for the auxiliary spillway show the resultant to be outside the downstream toe. The stability of this structure is, therefore, inadequate for the maximum pool condition.

Williams Bridge Dam is located upstream of Lake Scranton Dam and No. 5 Dam. If Williams Bridge Dam should fail because of overtopping, the overtopping potential and the potential for failure of Lake Scranton and No. 5 Dams is greatly increased.

In view of the concern for safety of Williams Bridge Dam, and because of the possible consequences of a failure, such as a domino-type failure of Lake Scranton and No. 5 Dams, the following measures are recommended to be undertaken by the Owner as soon as practical:

(1) Develop a detailed emergency operation and warning system for the Williams Bridge, Lake Scranton, and No. 5 Dam system.

(2) Perform additional studies to more accurately ascertain the spillway capacity required for Williams Bridge Dam as well as the nature and extent of mitigation measures required to make spillways hydraulically and structurally adequate.

In order to correct operational, maintenance and repair deficiencies and to more accurately determine the condition of the dam, the following measures are recommended to be undertaken by the Owner in a timely manner:

(1) Operate the gated outlets periodically to ensure they will be functional during emergency conditions. The usual practice is to open the blowoff discharge valve in late winter during the periods of high discharge to clean sediment from the bottom of the reservoir. Lubricate operating equipment. Provide access facilities to the gatehouse that would be usable during periods of high tailwater.

(2) Install three or more observation wells, or other instrumentation, in the downstream slope in order to detect possible leakage through the core wall. One of these observation wells, or other instrumentation, should be located in the vicinity of the wet area near the spillway and the others should be located at the Owner's discretion. Monitor and record visual and instrument observations.

(3) Visually monitor wet area of embankment near right abutment and record observations so any change is detectable.

(4) Repair damage to embankment with construction of French drains to remove water from the wet areas. Continue to monitor the main spillway right retaining wall for possible tilting.

(5) Fill low area at end of auxiliary spillway left approach wall.

(6) Perform maintenance on concrete caps over post-tensioning bars at main spillway right retaining wall, concrete of auxiliary spillway left approach wall, and concrete of downstream training wall.

(7) Remove trees behind auxiliary spillway downstream training wall.

(8) Fill holes in embankment caused by burrowing animals.

Before remedial work that corrects structural and hydraulic deficiencies in the spillways is complete, the following measures are recommended to be undertaken by the Owner:

(1) Provide round-the-clock surveillance of Williams Bridge Dam during periods of unusually heavy rains.

(2) When warnings of a storm of major proportions are given by the National Weather Service, the Owner should activate his emergency operation and warning system procedures.

GANNETT FLEMING CORDDRY AND CARPENTER, INC.

A. C. HOOKE
Head, Dam Section

COMMONWEALTH OF PENNSYLVANIA
REGISTERED PROFESSIONAL
ALBERT CHARLES HOOKE
ENGINEER
No. 2231-E

APPROVED BY:

JOHN H. KENWORTHY
LTC, Corps of Engineers
Acting District Engineer

DATE: 6 June 1978

WILLIAMS BRIDGE DAM

Williams Bridge Dam and Main Spillway
Looking Upstream

SUSQUEHANNA RIVER BASIN

STAFFORD MEADOW BROOK, LACKAWANNA COUNTY

PENNSYLVANIA

WILLIAMS BRIDGE DAM

PENNSYLVANIA GAS AND WATER COMPANY
(NDS ID No. 373)

PHASE I INSPECTION REPORT

NATIONAL DAM INSPECTION PROGRAM

# SECTION 1

## PROJECT INFORMATION

### 1.1 General.

a. Authority. The Dam Inspection Act, Public Law 92-367, authorized the Secretary of the Army, through the Corps of Engineers, to initiate a program of inspection of dams throughout the United States.

b. Purpose. The purpose of the inspection is to determine if the dam constitutes a hazard to human life or property.

### 1.2 Description of Project.

a. Dam and Appurtenances. Williams Bridge Dam is an earthfill structure with a central masonry core wall with an exposed concrete extension that projects 2.2 feet above the top of the embankment. There are two bends in the axis of the embankment at the right abutment. The dam is 650 feet long and 54 feet high at the original streambed. The main spillway is located immediately to the left of the embankment. It is a stepped, masonry gravity structure, 56.3 feet long, with earthfill against the upstream face. A masonry retaining wall is located at the right side of the outlet channel of the main spillway (referred to as main spillway right retaining wall). Adjacent to the main spillway at the left abutment is a masonry gravity auxiliary spillway, 103.3 feet long, with earthfill against the upstream face. The outlet works is located at the right side of the main spillway and consists of a masonry screen chamber and two cast-iron pipes that extend into a masonry valve chamber housing 24-inch gate valves. Various features of the dam are shown on the Plates at the end of the report and on the Photographs in Appendix D.

b. Location. The dam is located on Stafford Meadow Brook approximately 1/4 mile upstream of Lake Scranton. Williams Bridge Dam is shown on USGS Quadrangle, Olyphant, Pennsylvania, with coordinates N41°22'55" - E75°37'30" in Lackawanna County, Pennsylvania, and is 4 miles southeast of Scranton, Pennsylvania. The location map is shown on Plate 1.

c. Size Classification. Intermediate (54 feet high, 1,276 acre-feet).

d. Hazard Classification. High hazard.

e. Ownership. Pennsylvania Gas and Water Company, Wilkes-Barre, Pennsylvania.

f. Purpose of Dam. Water supply for Scranton, Pennsylvania.

g. Design and Construction History. The dam was designed for the Meadow Brook Water Company by Mr. E. Sherman Gould, consulting engineer of New York, and construction was completed in 1893. In 1902, in order to increase storage capacity, several modifications were made: the spillway crest was raised 5 feet, the core wall was raised 6 feet, and the height of embankment was raised 4 feet. In 1914, the Pennsylvania Water Supply Commission made a study that included stability, hydrology, and hydraulic analyses for Williams Bridge Dam. As a result of this study, modifications to improve the safety of the dam were recommended. In 1916 modifications were made and include: (1) strengthening the main spillway with vertical reinforcing bars, (2) adding concrete to auxiliary spillway to increase stability, (3) constructing a concrete training wall downstream of the auxiliary spillway to control floodwater discharge, (4) strengthening the masonry core wall by constructing concrete buttresses at the downstream side of the masonry core wall at one of the two bends in the axis of the embankment, (5) placing additional earthfill on downstream face of core wall, (6) constructing an exposed concrete extension to the masonry core wall, and (7) constructing a concrete approach wall at the left abutment of the auxiliary spillway.

In conjunction with the above modifications, piezometers were installed near the screen chamber, one on the upstream slope and one on the downstream slope of the embankment. Also, holes were drilled in the main spillway to check the hydrostatic pressure in the masonry joints. Holes were grouted after testing.

A 70-foot long crack in the main spillway right retaining wall, about 10 feet below the top of wall, was repaired in 1962. Eleven 5-5/8-inch diameter vertical holes were drilled through the wall, down 15 feet into bedrock. Three-inch diameter steel rods were grouted in the holes and were post-tensioned to 12,000 psi. The anchorages on the top of the wall were then covered with cast-in-place concrete blocks.

The exposed concrete extension to the masonry core wall was recently repaired with epoxy cement.

h. Normal Operating Procedure. Water is drawn from the reservoir 5.5 feet above the bottom of the screen chamber and flows by gravity through a 24-inch cast-iron pipe into the distribution lines of Scranton. Water can also be drawn from the bottom of the screen chamber through another lower 24-inch cast-iron pipe during periods of low flow and released into the stream. This lower pipe, referred to as the emergency or blowoff line, is also used for cleaning sediment from the intake tunnel and reservoir, and it is a means of drawing down the reservoir for repairs.

## 1.3 Pertinent Data.

a. Drainage Area. 5.0 square miles.

b. Discharge at Damsite. (cfs.)

Maximum known flood at damsite - 1,370 (estimated - May 1942).
Emergency drawdown line at maximum pool elevation - 130 (approximate).
Main spillway capacity with pool at auxiliary spillway crest - 190.
Total spillway capacity (main and auxiliary) at maximum pool elevation - 5,600 .

c. Elevation. (Feet above msl.)

Top of dam (top of core wall) - 1,366.2.
Top of embankment - 1,364.0.
Maximum pool (top of core wall) - 1,366.2.
Normal pool (main spillway crest) - 1,360.6.
Auxiliary spillway crest - 1,361.6.
Upstream portal invert outlet works - 1,316.5.
Downstream portal invert outlet works - 1,316.3.
Upstream invert water supply line - 1,322.0.
Streambed near outlet works - 1,315.0 (approximate).

d. Reservoir Length. (Miles.)

Normal pool - 0.50.
Maximum pool - 0.52.

e. Storage. (Acre-feet.)

Normal pool (main spillway crest) - 1,034.
Maximum pool (top of dam) - 1,276.

f. Reservoir Surface. (Acres.)

Normal pool (main spillway crest) - 43.
Maximum pool (top of dam) - 44.

g. Dam.

Type - Earthfill with central masonry core wall with an exposed concrete extension.

Length - Embankment only - 650 feet.
Embankment and main and auxiliary spillways - 810 feet.

Height - 54 feet.

Top width - Core wall - 2.5 feet.
Embankment - 10 feet.

Side slopes - Upstream - 1V on 3.5H.
Downstream, main section - 1V on 3H.
Downstream at right abutment - 1V on 2.5H.

Zoning - Homogeneous earthfill.
Central masonry core with exposed concrete extension.

Cutoff - Central masonry core wall to bedrock from main spillway to 110 feet toward right abutment. Core wall continues to right abutment founded 12 feet below natural surface (not on bedrock).

Grout curtain - None.

h. Diversion and Regulating Tunnel.

Type - Masonry intake tunnel, 6 feet wide by 8 feet high.
Masonry screen chamber, 6 feet wide by 49 feet high.
Cast-iron pipes - 24-inch diameter blowoff;
24-inch diameter water supply.

Length - Intake tunnel - 70 feet.
Screen chamber - 11 feet.
Cast-iron pipes - blowoff - 46 feet.
Water supply - connects to distribution system.

Access - Intake tunnel - none.
Screen chamber - screens could be replaced by using bulkheads and dewatering chamber.
Cast-iron pipes - access to upstream end by using bulkheads in screen chamber.

Regulating facilities - Two manually operated nonrising stem, enclosed, 24-inch gate valves, with exposed 2.5 to 1 gear reducers, for each 24-inch cast-iron pipe.

i. Spillway.

Type - Main spillway - broad crested weir (width 8 feet) with masonry steps.
Auxiliary spillway - broad crested weir (width 6 feet) with free overfall.

Length of weirs - Main spillway - 56.3 feet.
Auxiliary spillway - 103.3 feet.

Crest elevations - Main spillway - 1,360.6.
Auxiliary spillway - 1,361.6.

Upstream channel - 1V on 3.5H rock-faced embankment to Elevation 1360.0.

Downstream channel - Short protected length followed by natural channel. Small bridge about 500 feet downstream.

j. Regulating outlets - None other than outlet works.

## SECTION 2

## ENGINEERING DATA

### 2.1 Design.

a. Data Available. Very little engineering data was available for review for the original structures of for the 1901 modifications. In a study performed in 1914 by the Pennsylvania Water Supply Commission, an account of design concepts, geology, construction materials and methods, and design features was prepared for the original structures from interviews with the Owner, visual inspection, and other sources. The 1914 study also included analyses for hydrology, hydraulics, and stability of the principal features. Load assumptions and a summary of the results of the analyses are on file. That study and additional studies performed by the Commission in 1915 were the bases for the recommended improvements to the main and auxiliary spillways that were made in 1916. Design computations are on file for the 1962 repair of the main spillway right retaining wall using post-tensioned steel rods.

b. Design Features. The drawings indicate that the dam embankment is a homogeneous earthfill structure with a central masonry core wall, with an exposed concrete extension that projects 2.2 feet above the top of embankment. The dam is 650 feet long and 54 feet high at original streambed. The upstream embankment slope is 1V on 3.5H, and the downstream slope varies from 1V on 2.5H to 1V on 3.0H. The axis of the dam bends upstream towards the right abutment at two locations. A general plan of the dam is shown on Plate 1A. An overview photograph is shown on Page b. At the left end of the embankment, 130 feet of the masonry core wall is founded on bedrock. The remaining 520 feet of core wall at the right abutment is founded on a 2-foot thick concrete footing. A profile along the axis of the dam is shown on Plate 2. A discussion on geology is presented in Appendix E. No internal drainage facilities were provided for the embankment. Additional features include a main spillway, an auxiliary spillway, and an outlet works. The main spillway is located immediately to the left of the embankment. It is a stepped, masonry, gravity structure 56.3 feet long with earthfill against the upstream face. A section through the main spillway is shown on Plate 5. The main spillway is also shown on Photographs E, F and G. The auxiliary spillway is adjacent to the main spillway at the left abutment, and it is a masonry gravity structure 103.3 feet long with earthfill against the upstream face. Sections through the auxiliary spillway are shown on Plate 7. The auxiliary spillway is also shown on Photographs E and H. On the downstream side of the dam, a masonry retaining wall is located between the embankment and the outlet channel of the main spillway. The masonry retaining wall is shown on Photographs G and I.

The outlet works is located at the right side of the main spillway and consists of a 70-foot long masonry approach channel, a 70-foot long masonry tunnel that is 6 feet wide and 11 feet high, a masonry screen chamber located immediately upstream of the masonry core wall, and two 24-inch cast-iron pipes at different elevations that extend through the core wall into a masonry valve house located at the right side of the main spillway. A profile along the outlet works is shown on Plate 3. The screen chamber building is shown on Photograph A and the valve house is shown on Photograph G. Each 24-inch line has two totally enclosed 24-inch gate valves in tandem. The lower 24-inch line serves as a sediment blowoff and as the reservoir drawdown line, and it discharges into the outlet channel of the main spillway. The upper 24-inch line is 5.5 feet above the lower line and serves as the water supply line.

Modifications were made in 1916 and included (1) reinforcing the upper level of the main spillway by drilling 14 vertical holes, 18 inches from the upstream face, and grouting 1-1/2-inch diameter, 28-foot long, steel bars in these holes; (2) increasing the stability of the auxiliary spillway by doweling on the downstream face a concrete wedge, 13.7 feet wide at the base and 18 feet high; (3) placing a concrete training wall downstream of the auxiliary spillway to control floodwater discharge (Photograph F); (4) strengthening the masonry core wall at the two bends in the axis of the embankment by constructing, on the downstream side, concrete buttresses 3 feet thick, 26 feet wide at the base and 24.5 feet high; (5) placing earthfill on the downstream face of the core wall from the bends in the axis to the right abutment; (6) constructing an exposed concrete extension on the masonry core wall (Photograph D), 2.2 feet above the top of embankment; and (7) constructing a concrete approach wall at the left abutment of the auxiliary spillway (Photograph C). Details for strengthening the main spillway are shown on Plate 6 and concrete added to the auxiliary spillway is shown on Plate 7.

2.2 Construction.

a. Data Available. Almost no construction data for the original structures were available. Drawings, correspondence, and photographs relating to the 1916 modifications are available and seem to indicate full compliance with the requests of the Pennsylvania Water Supply Commission. Drawings and correspondence are also available for the 1962 repairs to the main spillway right retaining wall.

b. Construction Considerations. Correspondence in the files on the 1916 modifications indicate that the original masonry work was of good quality. Correspondence also indicated that the earthfill portions were compacted only to the extent that could be accomplished by men, horses, and hauling equipment moving over the filles. This would probably account for the irregularities observed in the embankments. Information on the 1962 repairs on the main spillway right retaining wall did not yield any concerns with respect to that work.

2.3 Operation. Few formal records of operation are available. The dam has been inspected at irregular intervals by Commonwealth authorities since 1921 and, in recent years, on an annual basis by the Owner.

Most of the observations made during this inspection are also indicated on previous inspections. The Owner indicated that the problems that were observed in this inspection have existed for several years.

2.4 Other Investigations. In 1972, the Owner had Justin & Courtney, Philadelphia, Pennsylvania, study the hydrology of Williams Bridge Dam and alternatives for increasing the spillway capacity.

2.5 Evaluation.

a. Availability. Engineering data was provided by the Division of Dams and Encroachments, Bureau of Water Quality Management, Department of Environmental Resources, Commonwealth of Pennsylvania, and by the Owner, Pennsylvania Gas and Water Company. The Owner made available an engineer, dam operators, and a valve crew for information and operating demonstrations during the visual inspection. The Owner also researched his files for additional information upon request of the inspection team.

b. Adequacy. The type and amount of design data and other engineering data are limited, and the assessment must be based on the combination of available data, visual inspection, performance history, hydrologic assumptions, and hydraulic assumptions.

c. Validity. There is no reason to question the validity of the available data.

## SECTION 3

## VISUAL INSPECTION

### 3.1 Findings.

a. General. The general appearance of this project indicated that some project features have deteriorated with age and are in need of repair, while other project features have been properly maintained and are in good condition.

b. Embankment Section.

(1) The dam embankment is maintained adequately. The portion of the upstream face that was visible appeared to be in good condition and had no signs of distress. Except at one area, the sod on the downstream surface is intact and maintained. No trees or shrubs were growing on either surface. The downstream surface is somewhat irregular in slope and has some local high and low spots. The irregularities are not considered abnormal due to the age of the dam and the nonmechanized methods of construction. Along the main spillway right retaining wall, the embankment slope is much flatter than the wall slope. To make the transition, the embankment slope is warped downward to the wall over a short distance (Photograph G).

(2) Two wet areas were observed on the downstream surface of the dam. The first is located along the toe of the embankment, 75 to 400 feet from the right abutment. Although clear, standing water about 2 inches deep was observed, there was no discernible flow, and areas adjacent to the wet area were dry. Based on a considerable growth of moss, it is judged that the wet area is a permanent condition. No erosion, sloughing or other adverse effects have resulted from the wet area.

(3) The second wet area (Photographs I and J) is a 4-foot by 15-foot area located at Elevation 1340 and 12 feet right of the main spillway right retaining wall. This area is soft, saturated, and no sod is covering it. There was some clear, standing water, but no flow was exiting from the area. There has been some sloughing at the uphill side. Areas above and below were not affected. It was probed to a depth of 2 feet with a 1/2-inch diameter stick before encountering significant resistance. The underlying embankment material was examined and was found to be clay with some sand and fine gravel. The source of the water could not be determined.

(4) Three holes, probably caused by groundhogs, were observed on the downstream surface about 80 feet to the right of the main spillway right retaining wall. There was no evidence of recent activity, and the holes are probably abandoned. The holes angled upstream at shallow depth and were dry. The holes are shown on Photograph K.

c. Appurtenant Structures.

(1) The downstream face of the main spillway could not be inspected in detail because of the amount of water flowing over it. However, it was observed that no blocks of stone were missing. The upstream face has earthfill against it and cannot be inspected.

(2) The auxiliary spillway was in overall poor condition. Within the first 25 feet left of the main spillway, an estimated 5 to 10 gallons per minute of leakage was coming through the deteriorated masonry joints on the downstream face. Mortar in this reach was cracked, weathered, and missing. The remainder of the downstream face was damp, but no areas of concentrated seepage were observed, and the masonry joints were intact and sound. The leakage is not resulting in any damage to other features. A mass of concrete that was in place against the downstream face is almost totally ineffective. The concrete is completely missing over about 50 percent of the exposed area and is very severely disintegrated over the remainder (Photograph H). This is the concrete that was added in 1916 in order to improve the stability of the auxiliary spillway.

(3) On the main spillway right retaining wall, 7 of the 11 concrete caps covering the tops of the post-tensioned steel bars that were installed in the 1962 repairs are disintegrating (Photograph I). The wall appeared to have a slight tilt toward the outlet channel. Survey points were marked on the wall, and the Owner said the wall is being monitored for movement.

(4) The auxiliary spillway left approach wall has very severe scaling along its top over about 30 linear feet to a maximum depth of 3 inches. The upstream end of the wall stops about 30 feet short of tying into high ground, and a low spot, having a maximum depth of 1.5 feet, exists where water would flow around the left end of the dam during large floods (Photograph C). However, examination of the probable flow path indicates that it probably would not have any effect on the dam. The water would flow across rock surfaces and would enter the outlet channel well downstream from the dam.

(5) Most of the face of the auxiliary spillway downstream training wall has fine to medium horizontal cracks, and several wide ones were observed. About 20 percent of the face has very severe scaling to a maximum depth of about 4 inches. Two 4-inch evergreens are growing immediately behind the wall, and other trees and shrubs are growing just slightly back from it (Photograph F). Although the leakage from the auxiliary spillway was flowing along the base of the wall, there was no erosion or undermining. No evidence of tilting was observed.

(6) The intake tunnel and screen chamber could not be examined because of the pool elevation. The valves were coated heavily

with rust, and there were several inches of water in the valve chamber. Exposed gears had not been recently lubricated. The downstream valve was opened about 6 inches by two caretakers in 30 minutes. With partial opening, water was surging out of the discharge pipe. Pennsylvania Gas and Water Company employees did not want the valve opened further because it could draw sediment into the supply line. Access to the valve house is by ladder from the top of the main spillway right retaining wall to the bottom of the channel. The ladder is rusted and loose.

d. Reservoir Area. The slopes adjacent to the reservoir are covered with hardwoods and evergreens. No evidence was noted of creep, rock slides or land slides. The Owner indicated that sedimentation is not a problem from the standpoint of reduced reservoir capacity. The Williams Bridge watershed is primarily owned by the Pennsylvania Gas and Water Company and is undeveloped.

e. Downstream Channel. The channel downstream of the main spillway turns slightly to the right immediately downstream of the main spillway, but it is straight beyond. The portion immediately below the main spillway is bounded by the right retaining wall and rock outcrops on the left. The bottom is lined with derrick stones. A small, low bridge about 500 feet downstream from the dam might act as an obstruction for large discharges, but it would probably be lost before seriously affecting the dam. The channel immediately downstream of the dam is shown on Photograph L. Lake Scranton is 0.3 mile downstream of Williams Bridge Dam. Except for the caretaker's house, there are no dwellings between the dam and Lake Scranton.

3.2 Evaluation.

a. Embankment Section.

(1) The wet area at the toe of the embankment has been noted since 1921, and it appears to be stable at present. No damage has resulted from it, and unless it changes significantly in the future, no problems are anticipated.

(2) It is unclear whether the wet area on the embankment adjacent to the main spillway was observed on previous inspections. The Owner indicated that it is not a recent event. However, the combination of local saturation, sloughing, and the proximity of this wet area to adjacent critical features, particularly to the main spillway right retaining wall that has a history of unsatisfactory performance, is such that the wet area is of general concern.

(3) No evidence of recent activity was observed in the holes that were caused by a burrowing animal. The holes were dry. Consequently, the holes are not considered to endanger the dam. However, they hinder mowing operations and could be re-inhabited.

b. Appurtenant Structures.

(1) As noted previously, the downstream face of the main spillway was not inspected.

(2) Leakage at the auxiliary spillway has been observed since 1921. The Owner indicated that the leakage rate has been stable in recent years. The concrete that was placed along the downstream face of the auxiliary spillway in 1916 had already deteriorated slightly by 1921. At present, it should be considered totally ineffective. Research of available information shows that this concrete was added in 1916 because a 1914 study concluded it was necessary for stability against overturning. It is not possible to assign a date as to when the concrete became ineffective nor, therefore, can it be determined whether the section has withstood a major flood since the concrete became ineffective. The combined effect of the masonry joint deterioration with its accompanying leakage and the loss of the concrete along the face that was added for stability results in the present stability of the auxiliary spillway being inadequate. Failure of the auxiliary spillway could result in progressive failure of the main spillway and dam.

(3) The continued disintegration of the concrete caps over the post-tensioned steel bars on the main spillway right retaining wall could cause corrosion and potential ineffectiveness of the bars that could result in failure of this critical feature.

(4) As described previously, during a large flood, water would discharge around the upstream end of the auxiliary spillway left approach wall. Although effect on the dam is minimal, it is undesirable to have this type of uncontrolled flow potential. Although the concrete is suffering from early stages of deterioration, it is not presently of great concern.

(5) Although the condition of the auxilary spillway downstream training wall does not present a significant hazard to the dam at the present time, long-term neglect will result in advanced deterioration and potential failure. Failure of this feature could result in damage to both the main and auxiliary spillways.

(6) At the valve house, with the existing facilities, the valve pit is inaccessible during periods of high tailwater and cannot be used as an emergency discharge during floods. The gears on the valves were not lubricated recently, and the Owner stated that the gates were not recently opened prior to the inspection. This places the functioning of the gates in question during emergency conditions.

c. Reservoir Area. No conditions were noted in the reservoir area that might present significant hazard to the dam.

d. <u>Downstream Channel</u>. Loss of the bridge across the outlet channel would not affect the safety of the dam. Alternate routes would allow access to either side of the dam.

## SECTION 4

## OPERATIONAL PROCEDURES

4.1 Procedure. The reservoir is maintained at main spillway crest Elevation 1360.6 with excess reservoir inflow cascading over the stepped masonry spillway. A 24-inch diameter cast-iron pipe water supply line draws water from the reservoir at Elevation 1322.0 to gravity feed the distribution lines in Scranton. The two gate valves on the water supply line are normally open. A 24-inch diameter cast-iron pipe emergency line or blowoff line at Elevation 1316.5 can discharge water into the outlet channel of the main spillway. The upstream valve on the blowoff line is normally open and the downstream valve is normally closed. The blowoff line is very seldom used because sediment might be stirred up in the screen chamber and would get into the water supply line, which is located about 5.5 feet above the blowoff line. Another reason for not using the blowoff line more frequently is that sediment from the bottom of the reservoir makes the stream turbid.

4.2 Maintenance of Dam. The dam is visited daily by two caretakers who check the chlorination equipment, and if water is not flowing over the main spillway, they check the reservoir elevation. When the reservoir is below the main spillway crest, the caretakers report the reservoir elevation to the Owner's Engineering Department. This information is used by the Engineering Department for regulating flows in the distribution system. The caretakers are also responsible for observing the general condition of the dam and appurtenant structures and for reporting any changes or deficiencies to the Owner's Engineering Department. A Pennsylvania Gas and Water Company engineer makes a formal inspection of the dam each year, and the records are kept on file and are used for determining priority of repairs. Informal inspections are also made when the engineer is on the site for other reasons. Special problems, such as the condition of the main spillway right retaining wall, are monitored at intervals as determined by the Owner's Engineering Department. The embankment is mowed at regular intervals and brush is cut annually.

4.3 Maintenance of Operating Facilities. The two rows of screens in the masonry screen chamber are cleaned in the fall when leaves tend to clog them or whenever there is indication of a pressure drop. For other facilities there is no regular maintenance program.

4.4 Warning Systems in Effect. The Owner furnished the inspection team with a chain of command diagram for Williams Bridge Dam and a generalized emergency notification list that is applicable for all the Pennsylvania Gas and Water Company dams. The Owner said that during periods of heavy rainfall, available personnel are dispatched to the dams to observe conditions. All company vehicles are equipped with radios, and the personnel can communicate with each other and with a central control facility. Evaluation of risk is made by the Owner's Engineering Department. The Owner's Engineering Department is also responsible for notification of emergency conditions to the local authorities. Detailed emergency operational procedures have not been formally established for Williams Bridge Dam but are as directed by the Owner's Engineering Department. The caretaker for the Williams Bridge Dam lives immediately below the dam.

4.5 Evaluation. Except for not opening the blowoff line on a regular basis, the operational procedure appears to be satisfactory. Infrequent operation of this blowoff could affect its functioning satisfactorily during emergency conditions. The procedures used by the Owner for inspecting the dam are adequate, but the repairs have not been timely. Maintenance of the operating facilities is not on a regular basis. In general, the warning system is adequate, but it is not in sufficient detail for Williams Bridge Dam when its overall condition and importance is considered.

SECTION 5

HYDROLOGY AND HYDRAULICS

5.1 Evaluation of Features.

a. Design Data.

(1) No hydrologic or hydraulic analyses for the original Williams Bridge Dam design were available for review. The spillway capacity has been estimated several times for the various construction modifications that have evolved. Spillway capacity, as used in this Section, represents the combined capacity of the main and auxiliary spillways.

(2) In the recommended guidelines for safety inspection of dams, the Department of the Army, Office of the Chief of Engineers (OCE), established criteria for rating the capacity of spillways. The recommended spillway design flood for the size (intermediate) and hazard potential (high) classification of Williams Bridge Dam is the PMF. If the dam and spillway are not capable of passing the PMF without overtopping failure, the spillway capacity is rated as inadequate. If the dam and spillway are capable of passing one-half of the PMF without overtopping failure, the spillway capacity is not rated as seriously inadequate. A spillway capacity is rated as seriously inadequate if all of the following conditions exist:

(a) There is a high hazard to loss of life from large flows downstream of the dam.

(b) Dam failure resulting from overtopping would significantly increase the hazard to loss of life downstream from the dam from that which would exist just before overtopping failure.

(c) The dam and spillway are not capable of passing one-half of the PMF without overtopping failure.

(3) The Williams Bridge watershed is primarily owned by the Pennsylvania Gas and Water Company and is undeveloped. Hydrologic analysis for this study was based on existing conditions and the effects of future development of the watershed were not considered.

b. Experience Data. In 1972, Justin & Courtney prepared a hydrology report and an improvement study that showed the 6-hour Probable Maximum Flood (PMF) peak discharge at 17,000 cfs and the present spillway capacity at 5,600 cfs at maximum pool. The PMF was determined by several independent methods and the 17,000 cfs was selected. These computations are included in Appendix C. Although another PMF determination might be desirable before design of remedial work is undertaken, the PMF determined by Justin & Courtney is considered to be satisfactory for this study. Calculations were performed to check the accuracy of the Justin & Courtney report spillway capacity estimate of 5,600 cfs. The spillway capacity was calculated to be 5,560 cfs. The 5,600 cfs spillway capacity was accepted for this study.

c. Visual Observations. On the date of the inspection, no conditions were observed that would indicate that the spillway capacity would be significantly reduced during a flood occurrence.

d. Overtopping Potential. For an occurrence of the PMF, the peak inflow of 17,000 cfs is greater than the spillway capacity of Williams Bridge Dam. A check of the surcharge storage effect of Williams Bridge Reservoir shows that the storage available is insufficient to contain an inflow with a peak flow of 17,000 cfs without overtopping the dam (Appendix C). If Williams Bridge Dam should fail because of overtopping, the overtopping potential and the potential for failure of Lake Scranton and No. 5 Dams is greatly increased.

e. Downstream Conditions. Lake Scranton is located 0.3 mile below Williams Bridge Dam and the dam for Lake Scranton is located 1.3 miles downstream from Williams Bridge Dam. No. 5 Dam is located 3.0 miles downstream from Lake Scranton Dam. The locations of these dams are shown on Plate 1. The only dwelling between Williams Bridge Dam and Lake Scranton is the caretaker's house. Tracks of the Erie-Lackawanna Railroad and the Lackawanna and Wyoming Valley Railroad cross Stafford Meadow Brook about 0.5 mile below No. 5 Dam. Starting at about 1.0 mile below No. 5 Dam, Stafford Meadow Brook runs parallel to Interstate Route 81 for a reach of about 1.0 mile. At the end of this reach, Stafford Meadow Brook runs under Interstate Route 81 and into the City of Scranton, which is the first populated area below Williams Bridge Dam. The City of Scranton is about 6.3 stream miles below Williams Bridge Dam and about 2.0 stream miles below No. 5 Dam. The downstream conditions indicate that a high hazard classification is warranted for Williams Bridge Dam.

f. Spillway Adequacy.

(1) The spillway will not pass the PMF without overtopping the dam. One-half of the PMF inflow is 8,500 cfs and is greater than the spillway capacity. A check of the surcharge storage effect of Williams Bridge Reservoir shows that the surcharge storage available is insufficient to contain an inflow with a peak flow of 8,500 cfs without overtopping the dam (Appendix C).

(2) The maximum tailwater is estimated to be Elevation 1322.3 at the spillway capacity of 5,600 cfs. At maximum pool elevation, there is a difference of about 40 feet between headwater and tailwater. If Williams Bridge Dam should fail due to overtopping, the hazard to loss

of life downstream from the dam will be significantly increased from that which would exist just prior to overtopping.

(3) Based on established OCE criteria as outlined in Paragraph 5.1.a.(2), the spillway capacity of Williams Bridge Dam is rated as seriously inadequate. Considering the effects of the surcharge storage of 242 acre-feet, the spillway discharge capacity of 5,600 cfs can accommodate a flood with a peak inflow of 5,800 cfs for a storm of the same duration as the PMF. This is 34 percent of the PMF peak inflow.

## SECTION 6

## STRUCTURAL STABILITY

6.1 Evaluation of Structural Stability.

a. Visual Observations.

(1) General. The visual inspection of the dam resulted in a number of observations relevant to structural stability. These observations are listed herein for the various features.

(2) Embankment. Wet areas were observed at the toe of the embankment near the right abutment and on the downstream surface adjacent to the main spillway. Irregularity of embankment slopes was observed. The detailed descriptions of the conditions are in Paragraphs 3.1.b.(2) and 3.1.b.(3). The detailed evaluation of the conditions are in Paragraphs 3.2.a.(1) and 3.2.a.(2).

(3) Appurtenant Structures. Possible tilting of the main spillway right retaining wall was noted. Masonry joint deterioration, leakage, and disintegration of concrete on the downstream face of the auxiliary spillway were observed. The detailed descriptions and evaluations of the conditions are in Paragraphs 3.1.c and 3.2.b, respectively.

b. Design and Construction Data. There were no records of design data or stability computations for the original structures built in 1893 or for modifications performed in 1902. However, stability analysis computations, dated 1914, are on file and were used as the basis for modifications to the dam that were made in 1916. Included in these computations are stability analyses of the proposed improvements for the main and auxiliary spillways. The improvements consisted of installing steel reinforcement near the upstream face of the main spillway to strengthen the upper section and adding mass concrete on the downstream face of the auxiliary spillway to ensure stability against overturning at maximum pool level. As far as is known, the main spillway improvements are still effective. However, the concrete added to the downstream face of the auxiliary spillway is almost totally disintegrated and, therefore, is not effective.

Stability analyses were made for both the main and auxiliary spillways as part of this study using water at the maximum pool level. Only the bottom of each section was considered. The stability analysis performed for the main spillway showed that the toe pressure and sliding factor are within acceptable limits and the resultant is outside the middle third but within the base, about 6 feet from the downstream toe. OCE guidelines on overturning recommended that the resultant be within the middle third. Although the resultant is outside the middle third, it is within the base and considering that the spillway is on a rock foundation and the toe pressure is within acceptable limits, the resultant being outside the middle third is not considered to be a significant deviation from the recommended guidelines.

The stability analysis performed for the auxiliary spillway, for maximum pool condition, considered the downstream concrete as ineffective, thus duplicating the physical conditions of 1914. As in the 1914 analysis, the sliding factor was found to be within acceptable limits, but the resultant was located outside the downstream toe. The auxiliary spillway, therefore, is structurally inadequate for maximum loading condition.

Computations are also on file for the design of post-tensioned anchors used to repair the main spillway right retaining wall in 1962. Other than perhaps some possible slight inward movement at an unknown time, performance of the retaining wall has been satisfactory to date.

c. Operating Records. Based on the operating records, there is no evidence that any stability problems have occurred for the main spillway, auxiliary spillway or embankment during the operational history of the dam. It is known that the modifications performed in 1916 were made under direction of the Pennsylvania Water Supply Commission to correct design deficiencies. However, the maximum pool level attained after the concrete on the downstream face of the auxiliary spillway became ineffective is not known, so there is no conclusive operational history for that feature for its present condition. The cracking of the main spillway right retaining wall, which ultimately resulted in about a 4-inch lateral displacement that was repaired in 1962, was first noted on previous inspections in 1941. The Owner said that the wall is presently being monitored at unspecified intervals.

d. Post-Construction Changes. As noted herein, there is adequate information on file concerning modifications made to Williams Bridge Dam after 1916.

e. Seismic Stability. Williams Bridge Dam is located in Seismic Zone 1. Normally, it can be considered that if a dam in this zone is stable under static loading conditions, it can be assumed safe for any expected earthquake loading. However, since there is the potential of earthquake forces moving or cracking the masonry core wall, the theoretical seismic stability of this dam cannot be assessed.

SECTION 7

ASSESSMENT, RECOMMENDATIONS, AND REMEDIAL MEASURES

7.1 Dam Assessment.

a. Safety.

(1) Based on the visual inspection, available records, calculations and past operational performance, Williams Bridge Dam is judged to be in fair condition. However, deficiencies of varying degree of importance were noted. A summary of the features and observed deficiencies is listed below:

| Feature and Location | Observed Deficiencies |
|---|---|
| Embankment: | |
| Right abutment | Wet area at toe. |
| Near spillway | Wet area and sloughing. |
| Right of spillway | Holes by burrowing animals. |
| Auxiliary Spillway: | |
| Downstream face | Leakage and masonry joint deterioration; concrete disintegration. |
| Left approach wall | Low area at end; concrete deterioration. |
| Downstream training wall | Trees too close; concrete deterioration. |
| Outlet Works: | Lack of regular maintenance and operation. |

(2) The overtopping potential analysis shows that the dam will be overtopped by the PMF and by one-half the PMF. Based on OCE criteria, as outlined in Paragraph 5.1.a.(2), the spillway capacity is rated as seriously inadequate. The existing spillway can accommodate a flood with a peak inflow of 34 percent of the PMF peak inflow.

(3) Review of the 1914 stability computations for the auxiliary spillway and computations made for the purpose of this study indicate that the auxiliary spillway, in its present condition, might possibly fail by overturning before the maximum pool elevation is reached. The computed location of the resultant is outside the base. Concrete was added to the downstream face in 1916 after the 1914 analysis indicated that failure by overturning during floods was probable.

That concrete is now completely ineffective for the purpose of ensuring stability. Failure of the auxiliary spillway might result in progressive failure of Williams Bridge Dam. The risk is compounded by masonry joint deterioration and leakage that might affect the ability of the structure to act monolithically.

(4) Review of the 1914 stability computations for the main spillway and computations made for the purpose of this study indicate that the resultant is within the base, about 6 feet from the downstream toe. The computed factor of safety for sliding and toe pressure were within acceptable limits.

b. Adequacy of Information. The information available is such that an assessment of the condition of the dam can be inferred from the combination of visual inspection, past performance, and computations performed prior to and as part of this study.

c. Urgency. Williams Bridge Dam is located upstream of Lake Scranton Dam and No. 5 Dam. If Williams Bridge Dam should fail because of overtopping, the overtopping potential and the potential for failure of Lake Scranton and No. 5 Dams is greatly increased. The recommendations in Paragraph 7.2 should be implemented as soon as practical or in a timely manner as noted.

d. Necessity for Further Investigations. In order to accomplish some of the remedial measures outlined in Paragraph 7.2, further investigations will be required.

7.2 Recommendations and Remedial Measures.

a. In view of the concern for safety of Williams Bridge Dam and because of the possible consequences of a failure, such as a domino-type failure of Lake Scranton and No. 5 Dams, the following measures are recommended to be undertaken by the Owner as soon as practical:

(1) Develop a detailed emergency operation and warning system for the Williams Bridge, Lake Scranton, and No. 5 Dam system.

(2) Perform additional studies to more accurately ascertain the spillway capacity required for Williams Bridge Dam, as well as the nature and extend of mitigation measures required to make spillways hydraulically and structurally adequate.

b. In order to correct operational, maintenance, and repair deficiencies and to more accurately determine the condition of the dam, the following measures are recommended to be undertaken by the Owner in a timely manner:

(1) Operate the gated outlets periodically to ensure they will be functional during emergency conditions. The usual practice is to open the blowoff discharge valve in late winter during periods of high discharge to clean sediment from the bottom of the reservoir. Lubricate operating equipment. Provide access facilities to the gatehouse that would be useable during periods of high tailwater.

(2) Install three or more observation wells or other instrumentation in the downstream slope in order to detect possible leakage through the core wall. One of these observation wells or other instrumentation should be located in the vicinity of the wet area near the spillway, and the others should be located at the Owner's discretion. Monitor and record visual and instrument observations.

(3) Visually monitor wet area of embankment near right abutment and record observations so any change is detectable.

(4) Repair damage to embankment with construction of French drains to remove water from the wet areas. Continue to monitor the main spillway right retaining wall for possible tilting.

(5) Fill low area at end of auxiliary spillway left approach wall.

(6) Perform maintenance on concrete caps over post-tensioning bars at main spillway right retaining wall, concrete of auxiliary spillway left approach wall, and concrete of downstream training wall.

(7) Remove trees behind auxiliary spillway downstream training wall.

(8) Fill holes in embankment caused by burrowing animals.

c. Before remedial work, that corrects structural and hydraulic deficiencies in the spillways, is complete, the following measures are recommended to be undertaken by the Owner:

(1) Provide round-the-clock surveillance of Williams Bridge Dam during periods of unusually heavy rains.

(2) When warnings of a storm of major proportions are given by the National Weather Service, the Owner should activate his emergency and warning system procedures.

SUSQUEHANNA RIVER BASIN

STAFFORD MEADOW BROOK, LACKAWANNA COUNTY

PENNSYLVANIA

WILLIAMS BRIDGE DAM

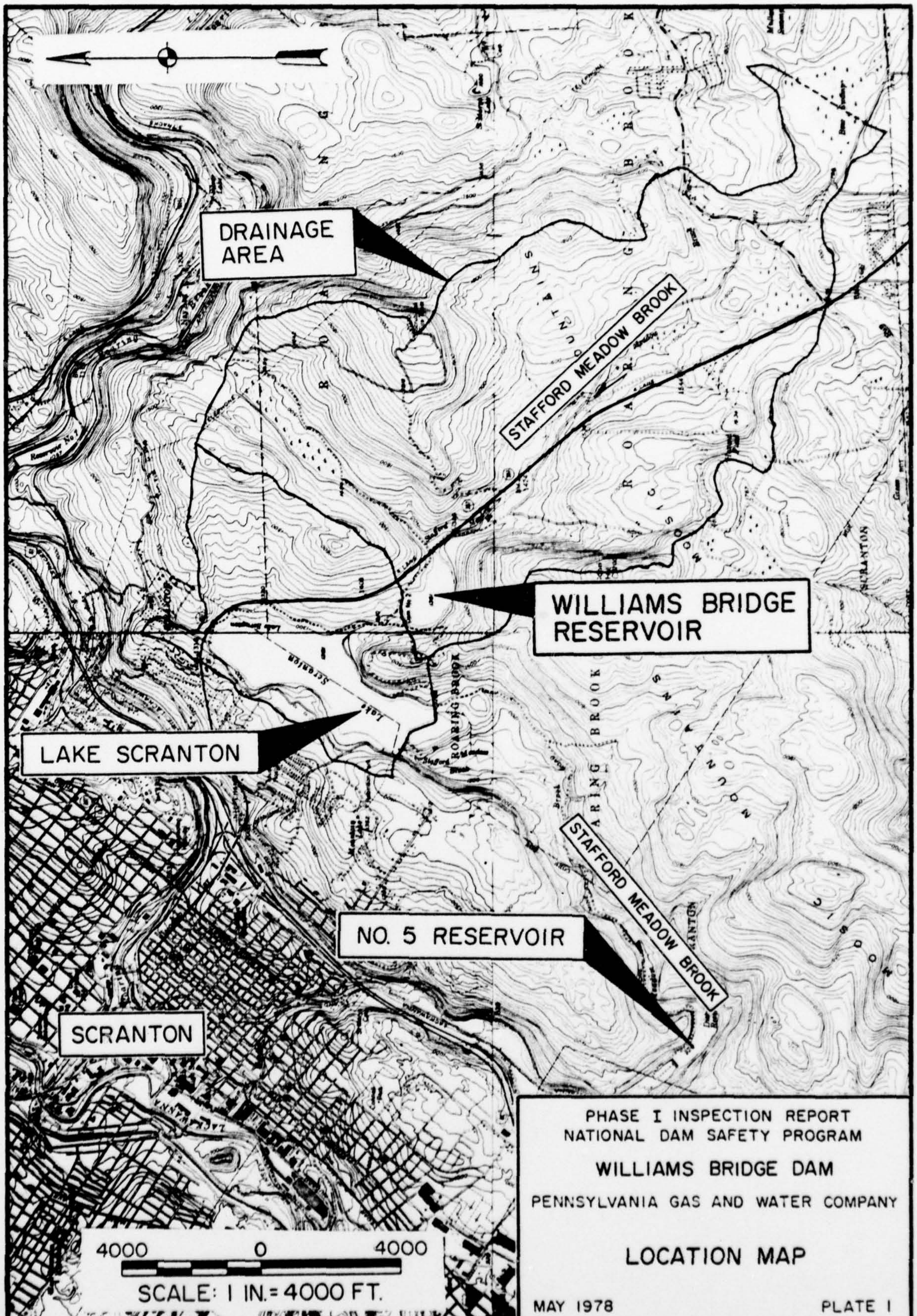
PENNSYLVANIA GAS AND WATER COMPANY

PHASE I INSPECTION REPORT

NATIONAL DAM INSPECTION PROGRAM

MAY 1978

# PLATES

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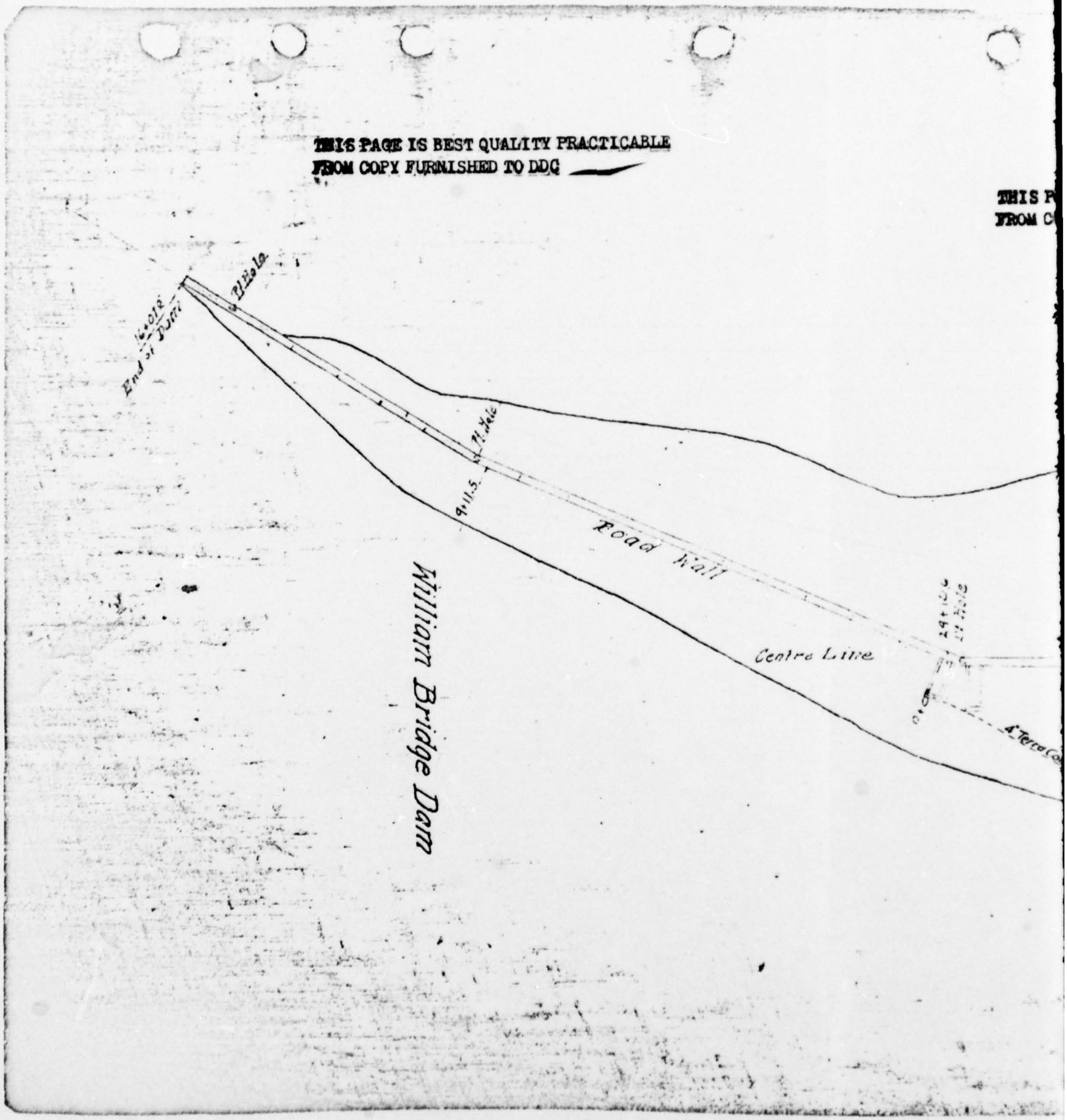
WILLIAMS BRIDGE DAM
PENNSYLVANIA GAS AND WATER COMPANY

LOCATION MAP

MAY 1978 PLATE 1


Road Wall
Centre Line
William Bridge Dam

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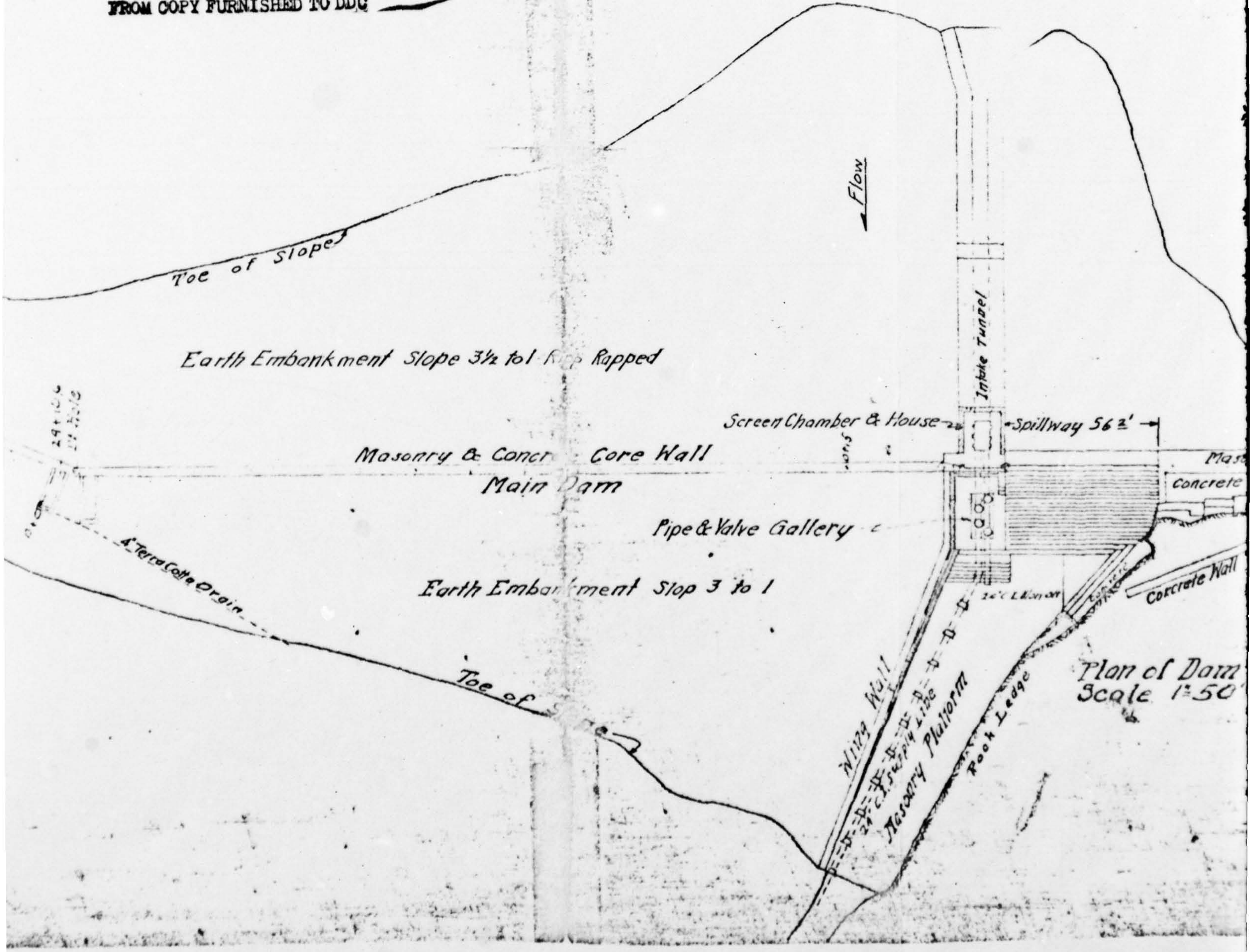

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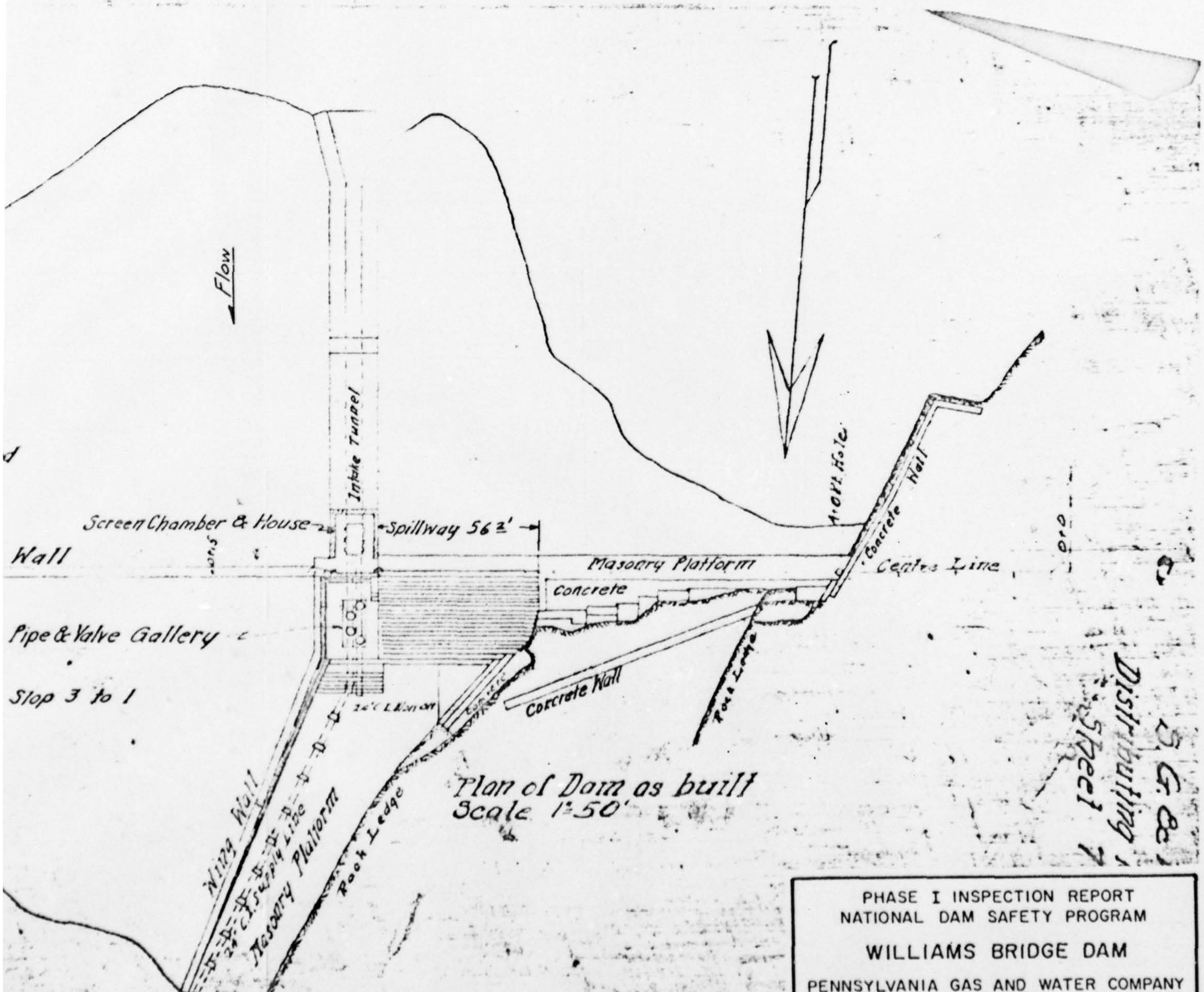


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WILLIAMS BRIDGE DAM

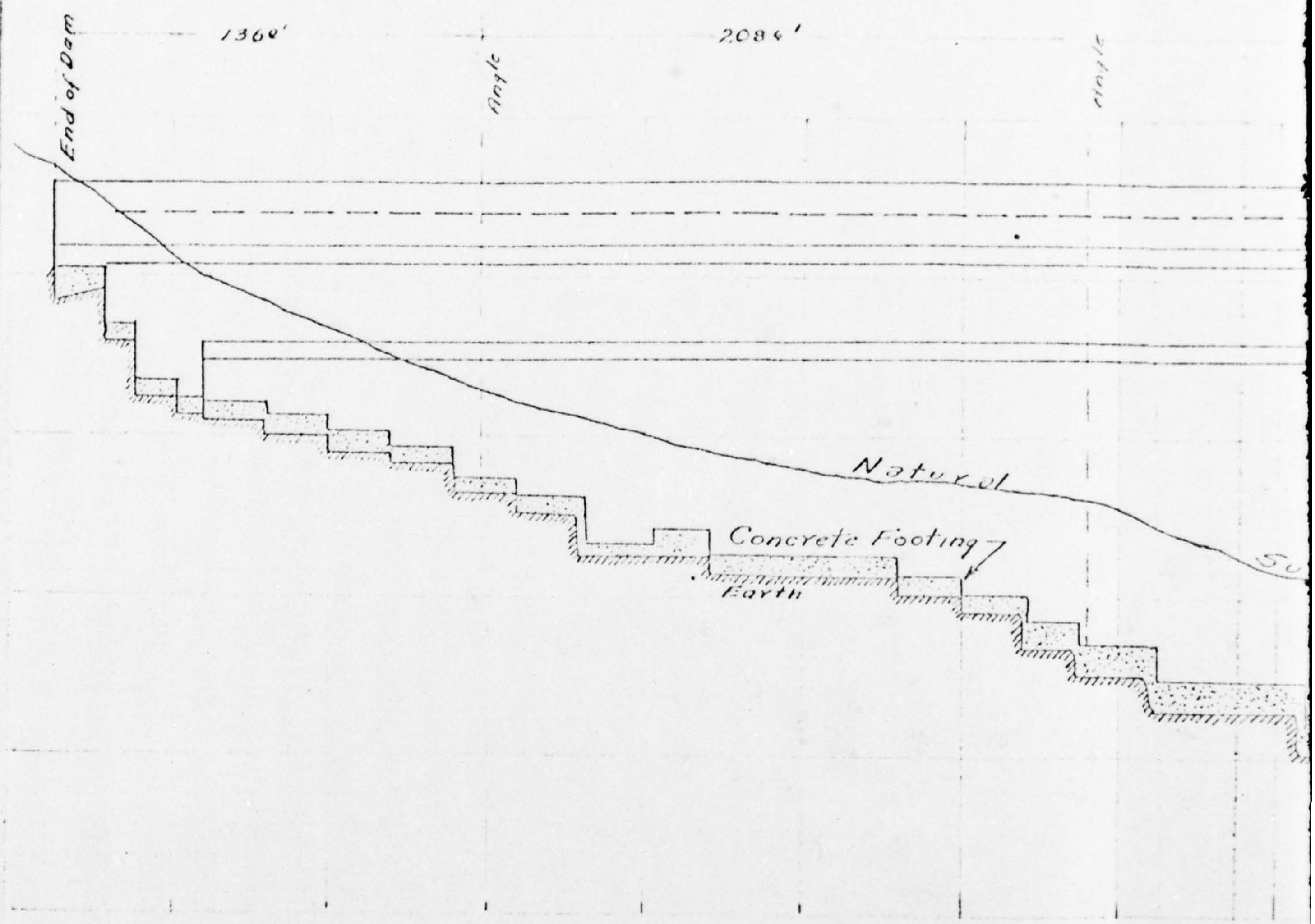
PENNSYLVANIA GAS AND WATER COMPANY

GENERAL PLAN

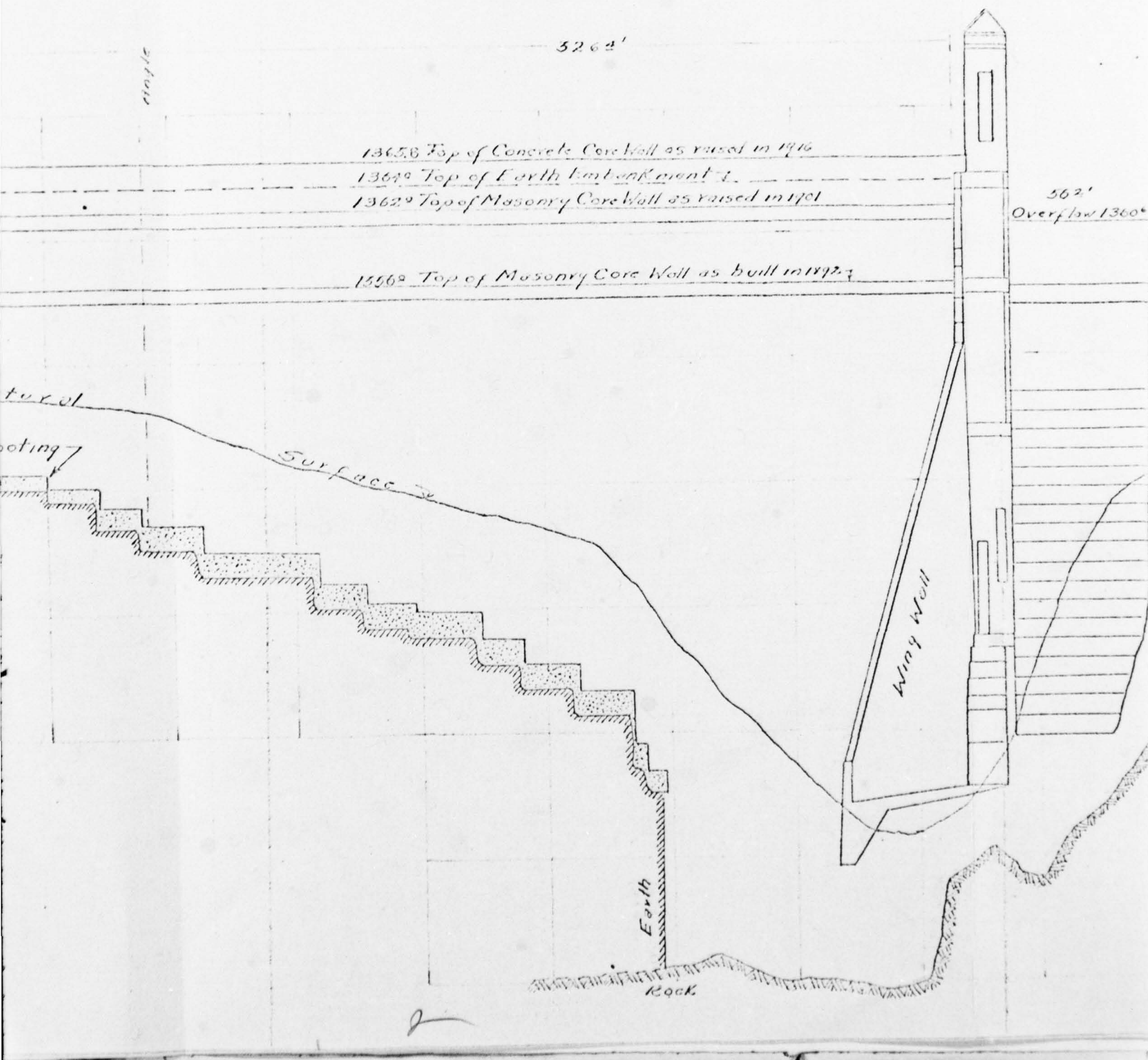
MAY 1978

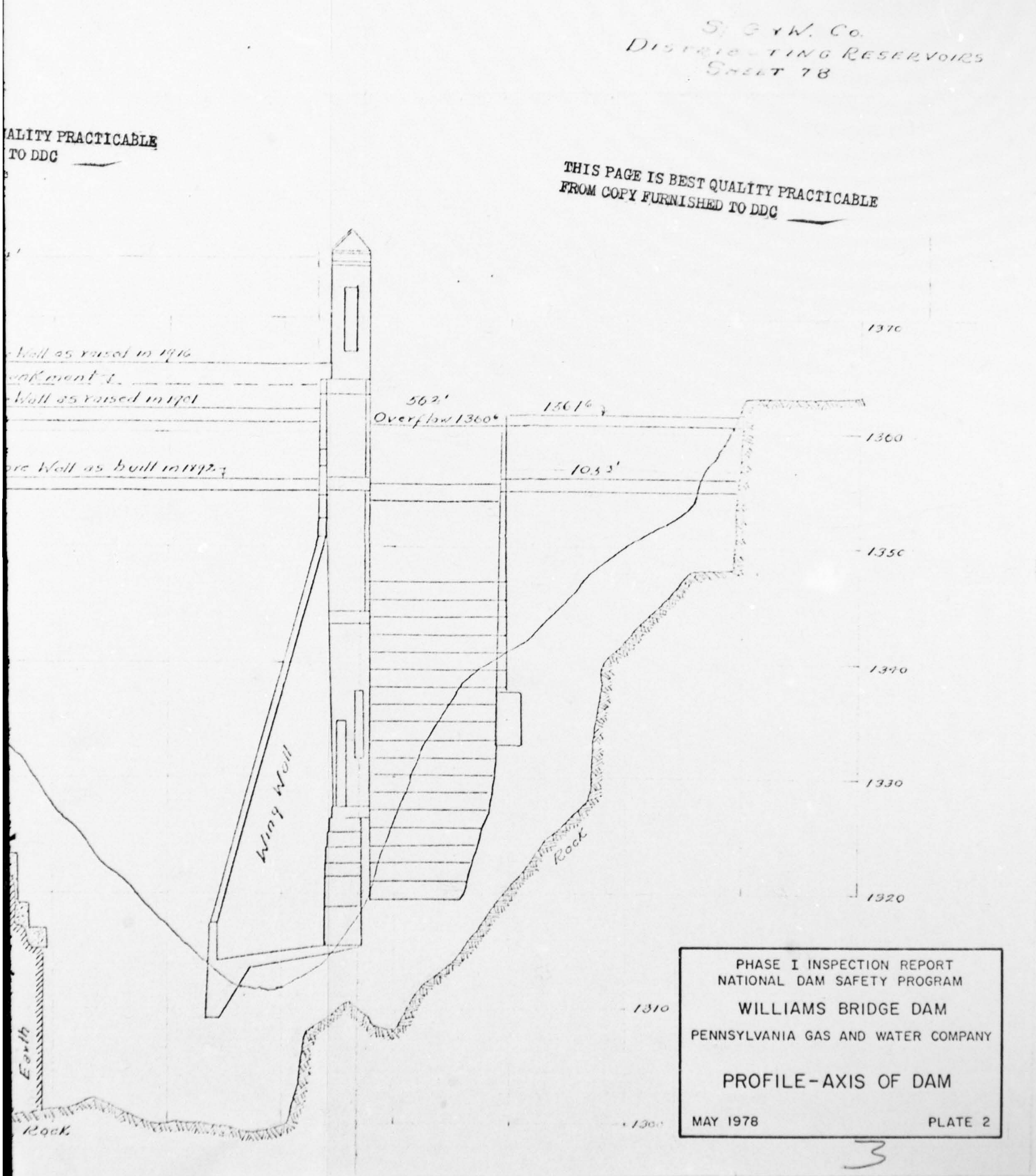
PLATE 1A

3




WILLIAMS BRIDGE DAM
Scales: Hor. 1" = 50'; Ver 1" = 10'


326½'
Angle
1365.6 Top of Concrete Core Wall as raised in 1916
1364.0 Top of Earth Embankment
1362.0 Top of Masonry Core Wall as raised in 1901
56½'
Overflow 1360.0
1356.0 Top of Masonry Core Wall as built in 1892
...tural
Surface
...ooting
Wing Wall
Earth
Rock





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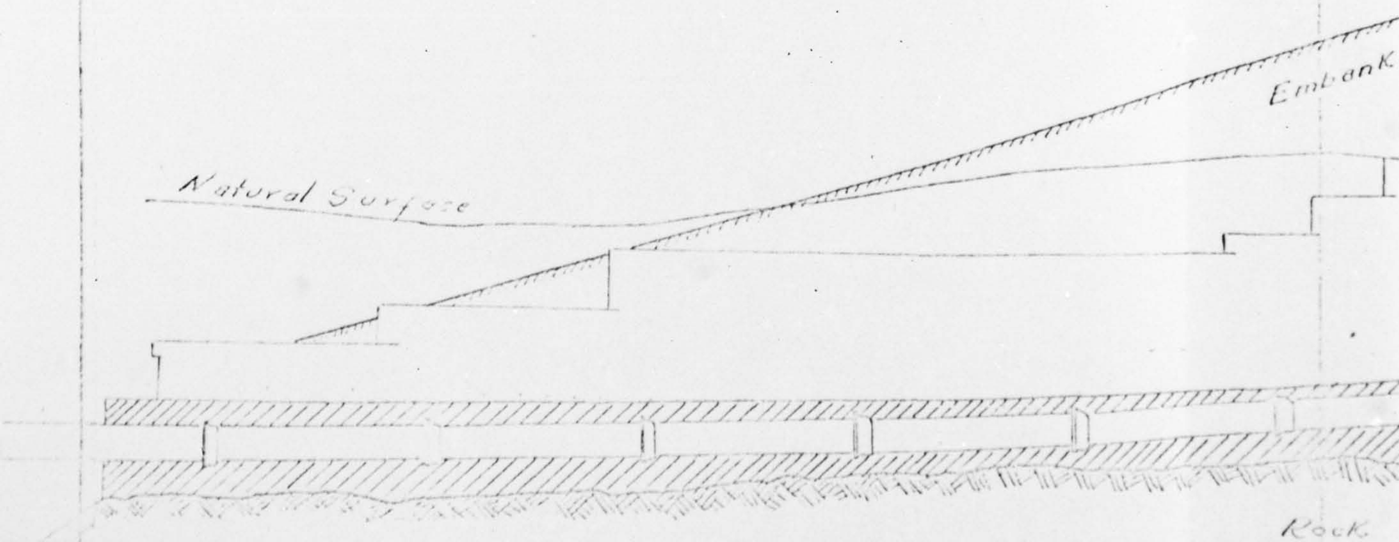
WILLIAMS BRIDGE DAM

PENNSYLVANIA GAS AND WATER COMPANY

PROFILE-AXIS OF DAM

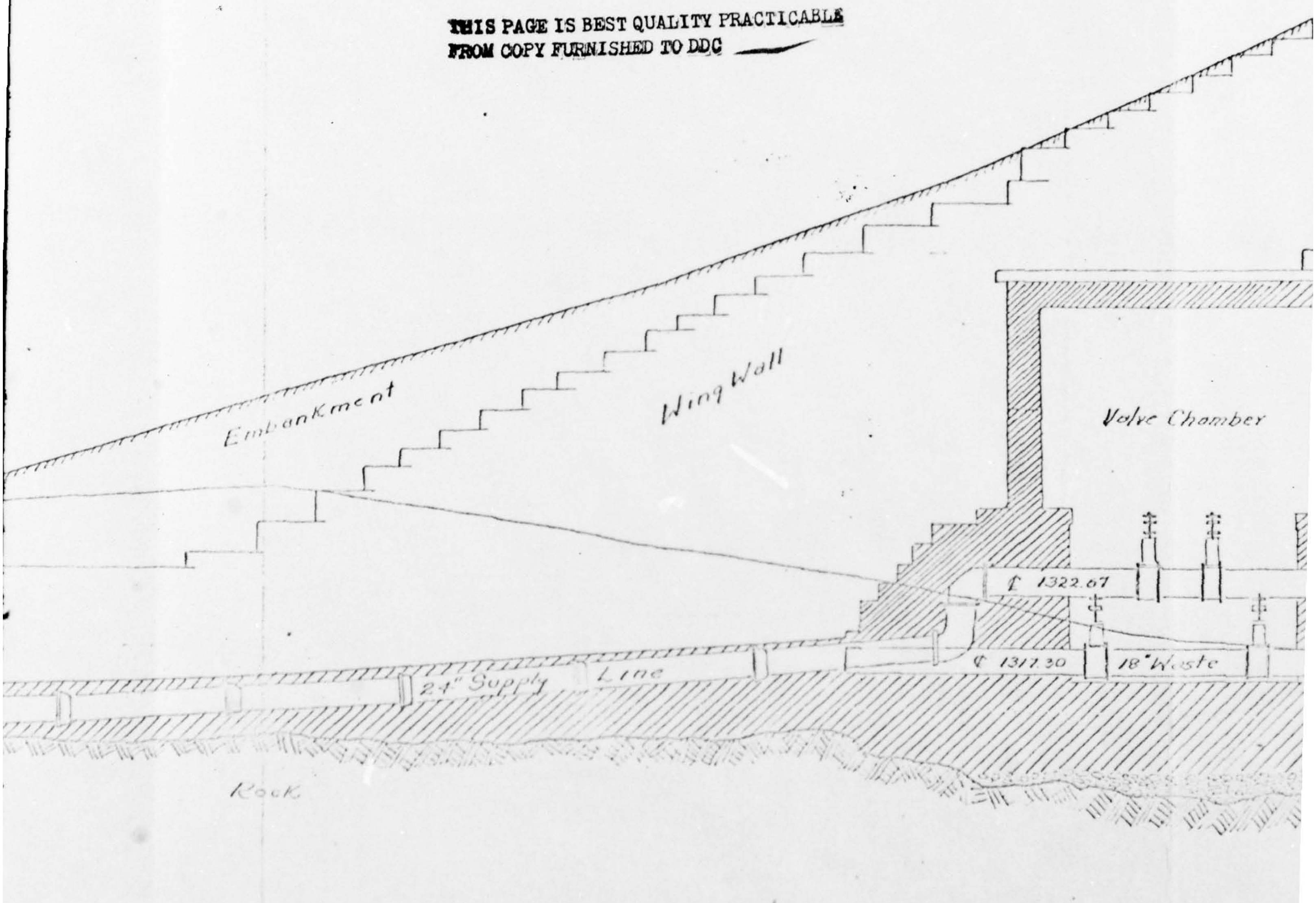
MAY 1978

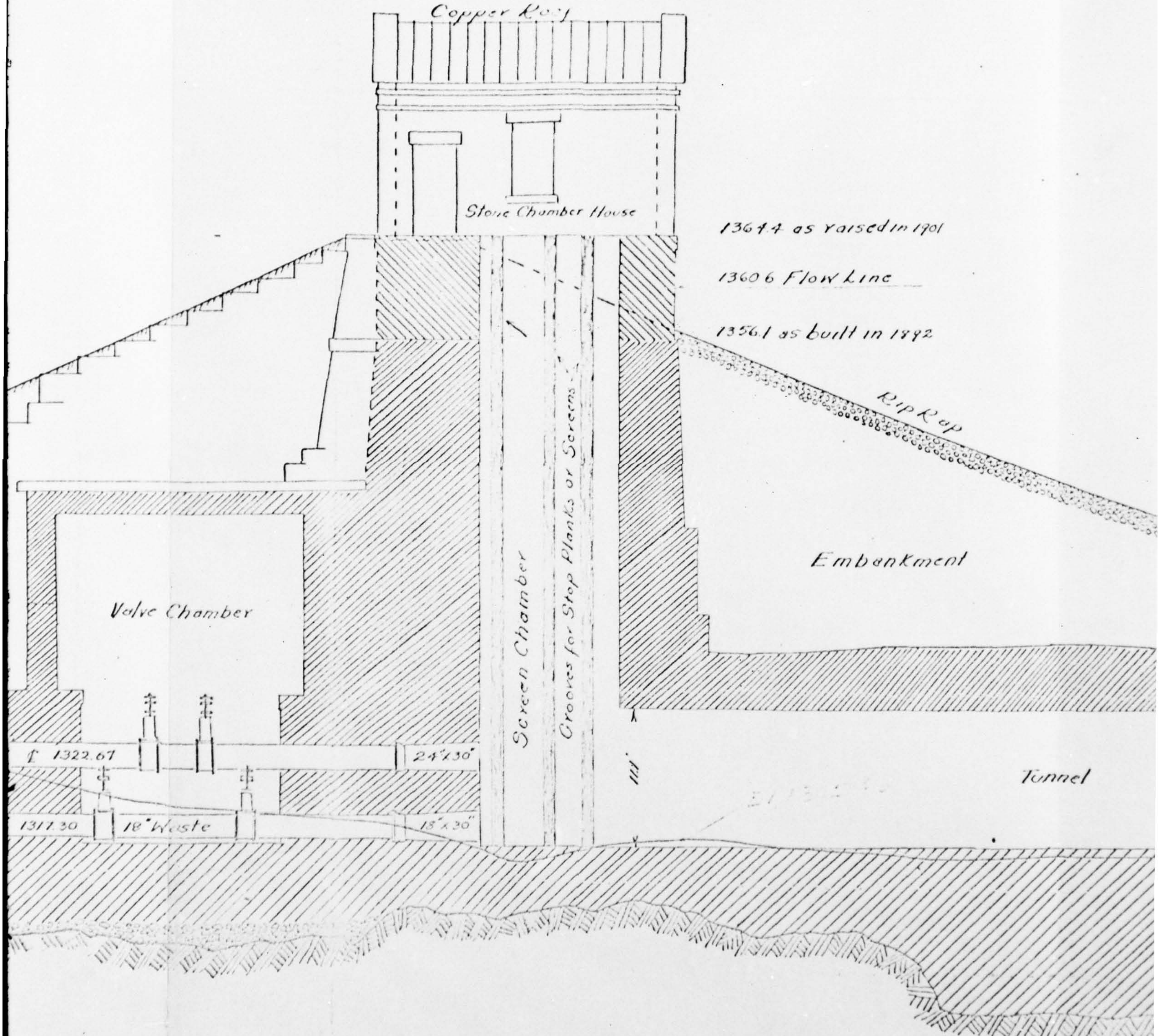
PLATE 2


Natural Surface
Embank
Rock

# WILLIAMS BRIDGE DAM

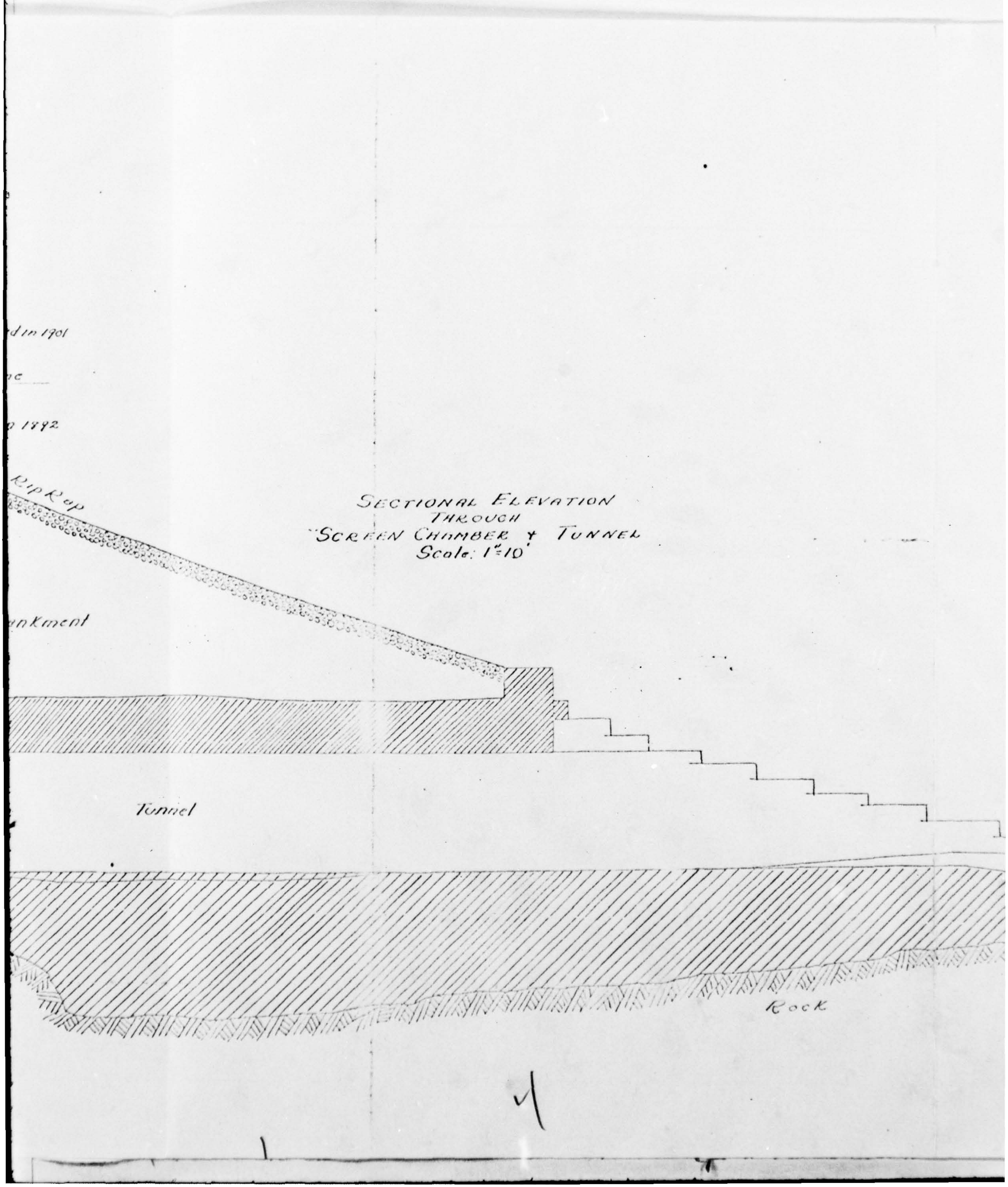






Copper Roof
Stone Chamber House
1364.4 as raised in 1901
1360.6 Flow Line
1356.1 as built in 1892
Rip Rap
Embankment
Valve Chamber
Screen Chamber
Grooves for Stop Planks or Screens
1322.67
24"x30"
Tunnel
1317.30
18" Waste
18"x30"

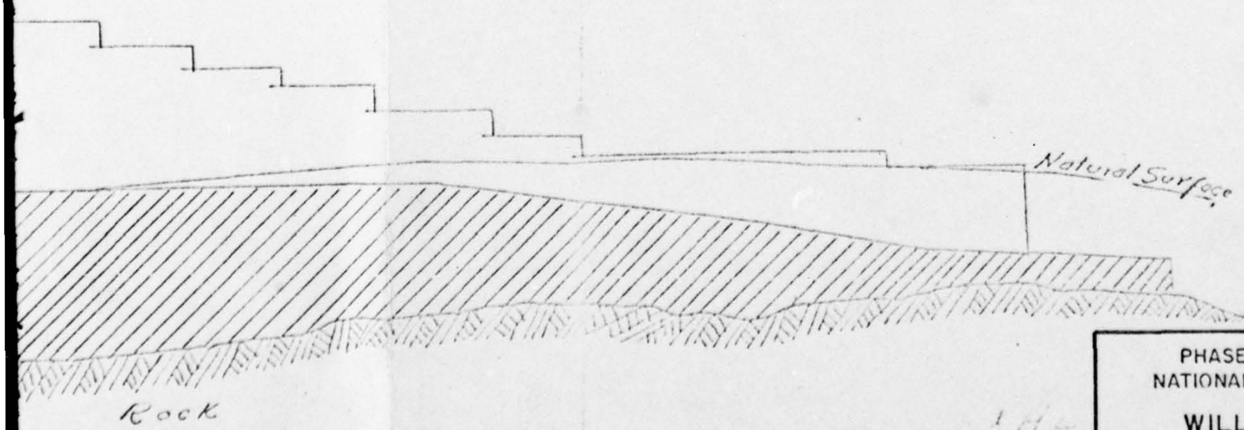

3.




SECTIONAL ELEVATION
THROUGH
"SCREEN CHAMBER & TUNNEL
Scale: 1"=10'

S. G & W. Co.
Distributing Reservoirs
Sheet 79



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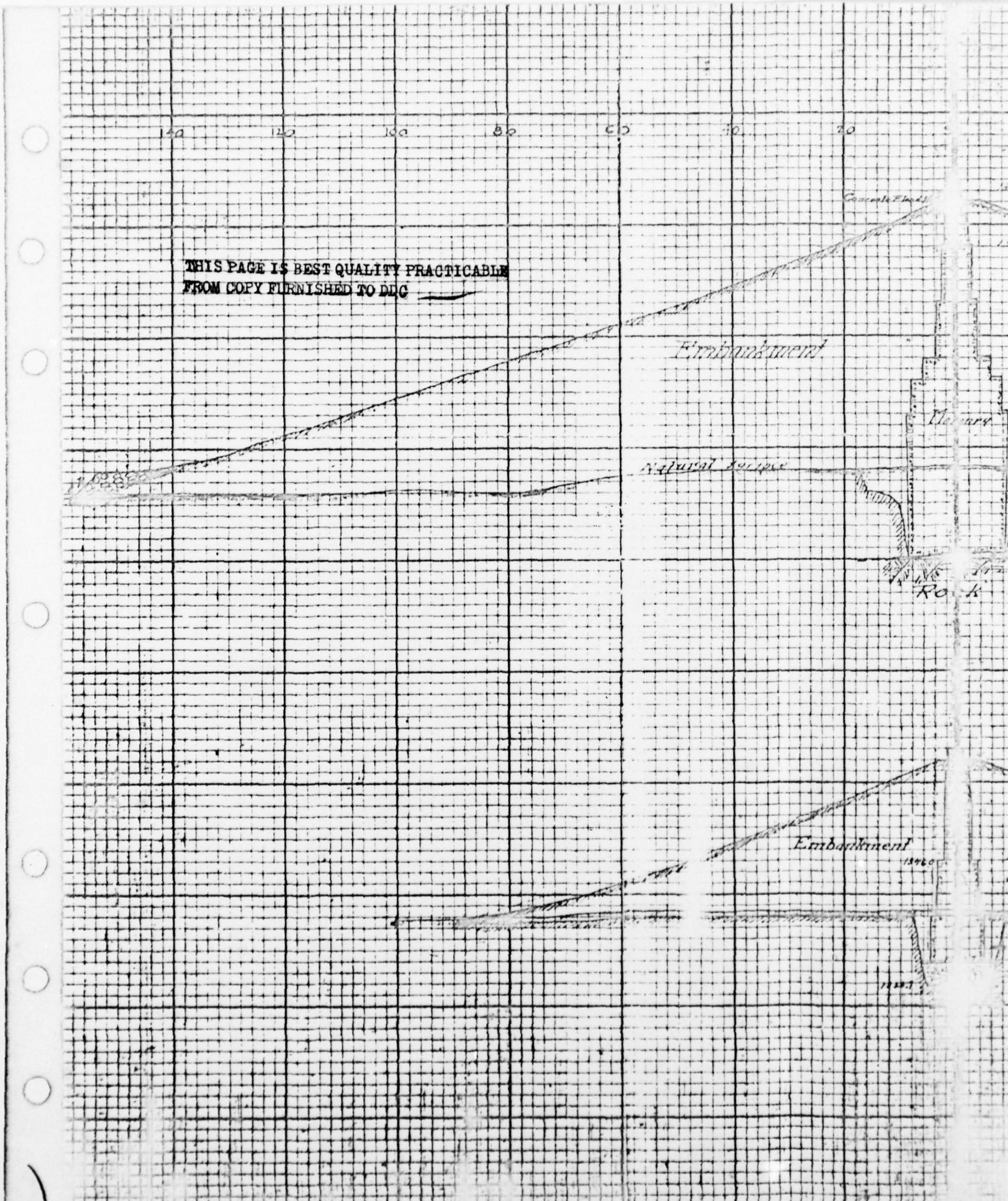
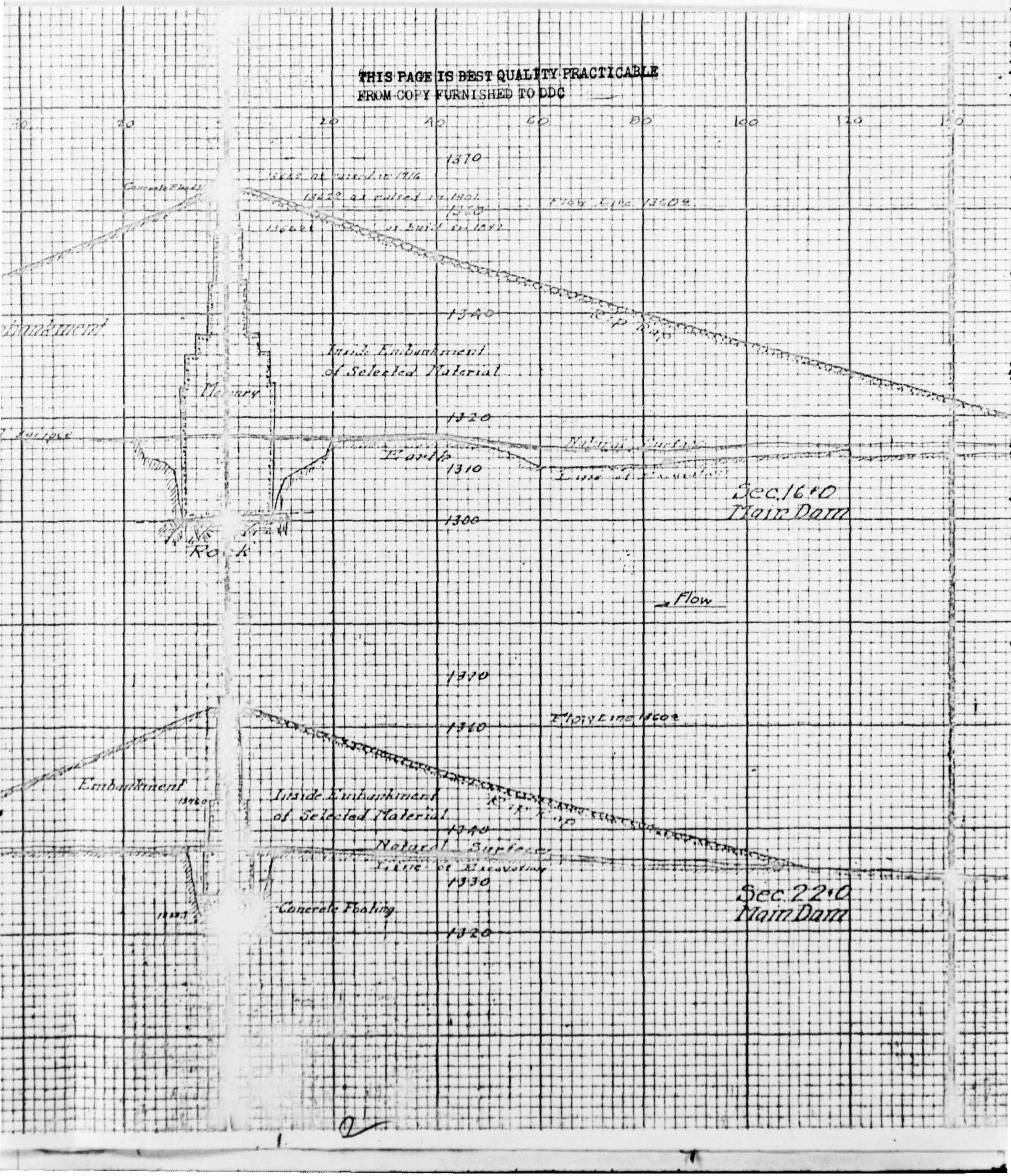
WILLIAMS BRIDGE DAM

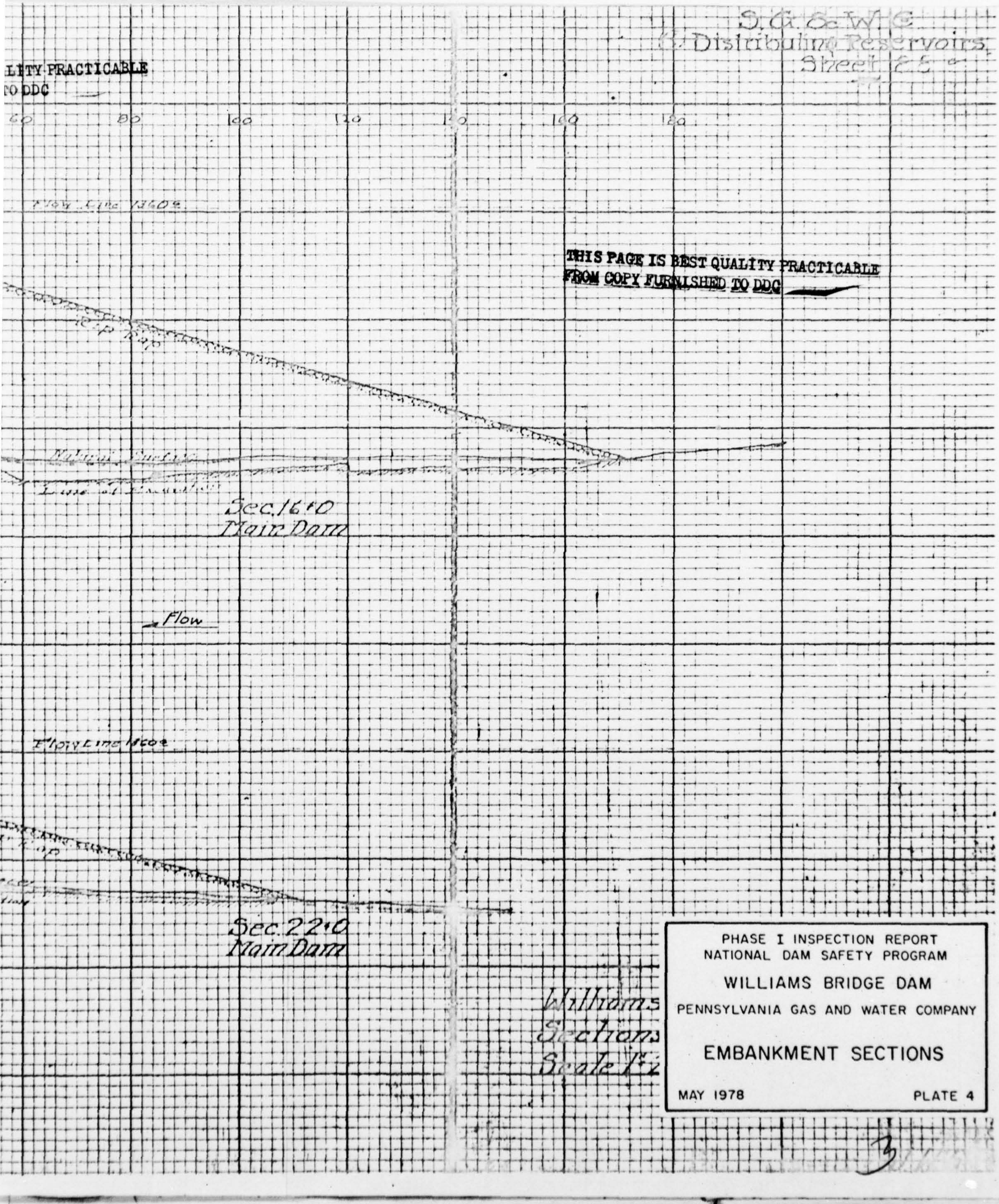
PENNSYLVANIA GAS AND WATER COMPANY

PROFILE - ℄ OUTLET WORKS

MAY 1978 PLATE 3


140
120
100
80
60
40
20
Embankment
Natural Surface
Rock
Embankment


20
40
60
80
100
120
1370
1360
1340
Rip Rap
Inside Embankment
of Selected Material
Masonry
1320
Natural Surface
Earth
1310
Line of Excavation
Sec. 16+0
Main Dam
1300
Rock
Flow
1370
Flow Line 1360.0
1360
Embankment
Inside Embankment
of Selected Material
1340
Natural Surface
Line of Excavation
1330
Sec. 22+0
Main Dam
Concrete Footing
1320



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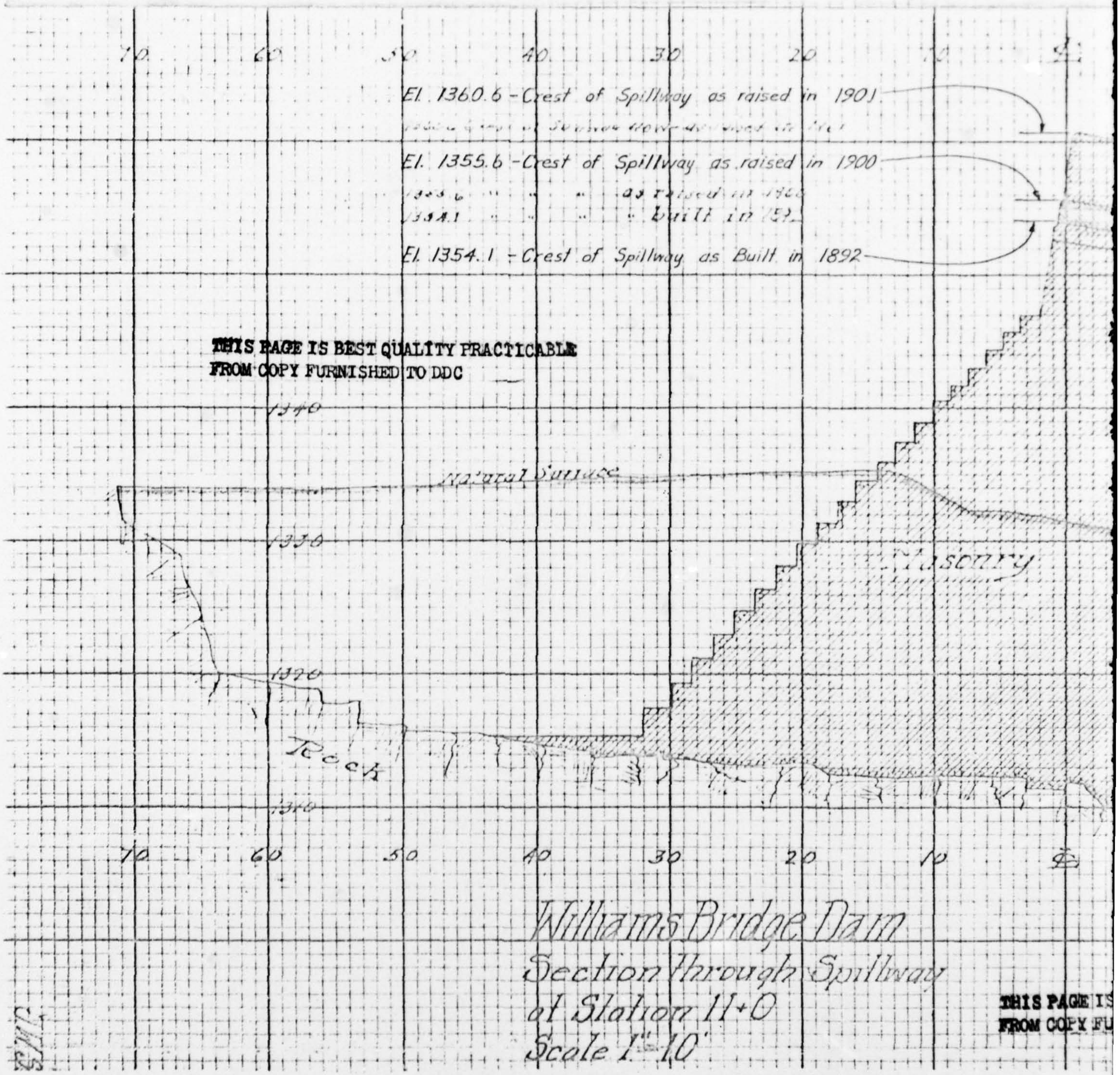
NATIONAL DAM SAFETY PROGRAM

WILLIAMS BRIDGE DAM

PENNSYLVANIA GAS AND WATER COMPANY

EMBANKMENT SECTIONS

MAY 1978 PLATE 4



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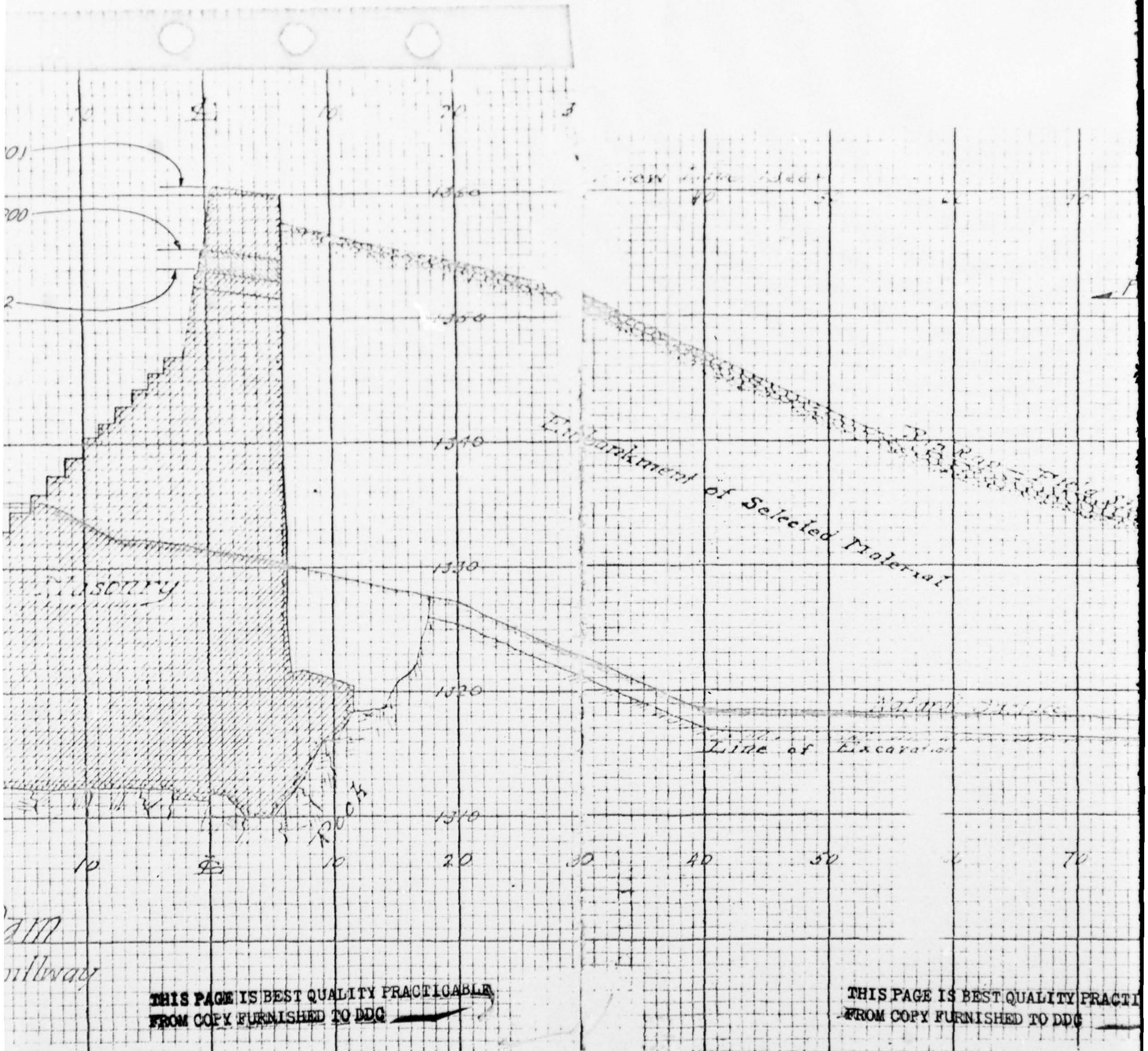




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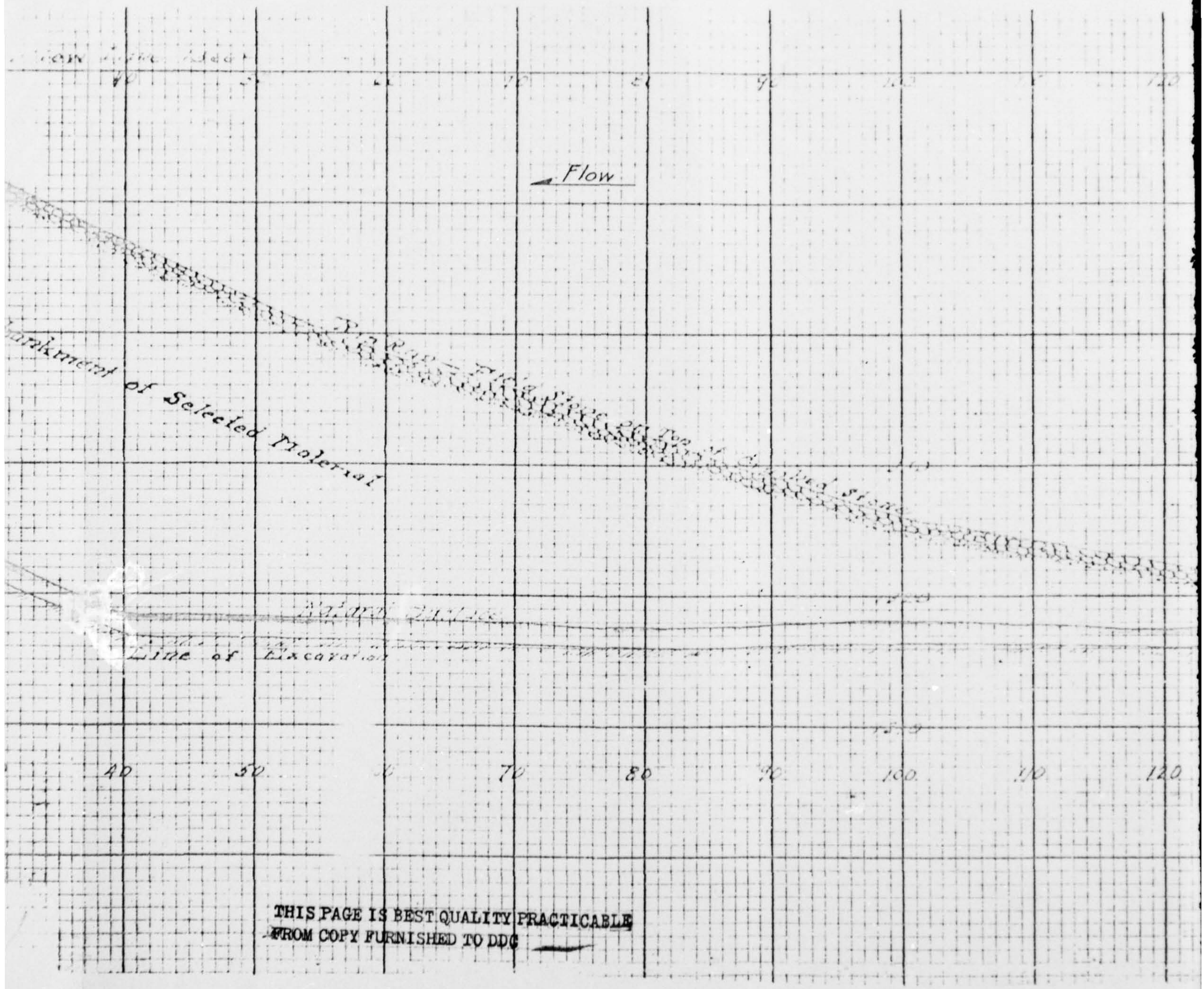


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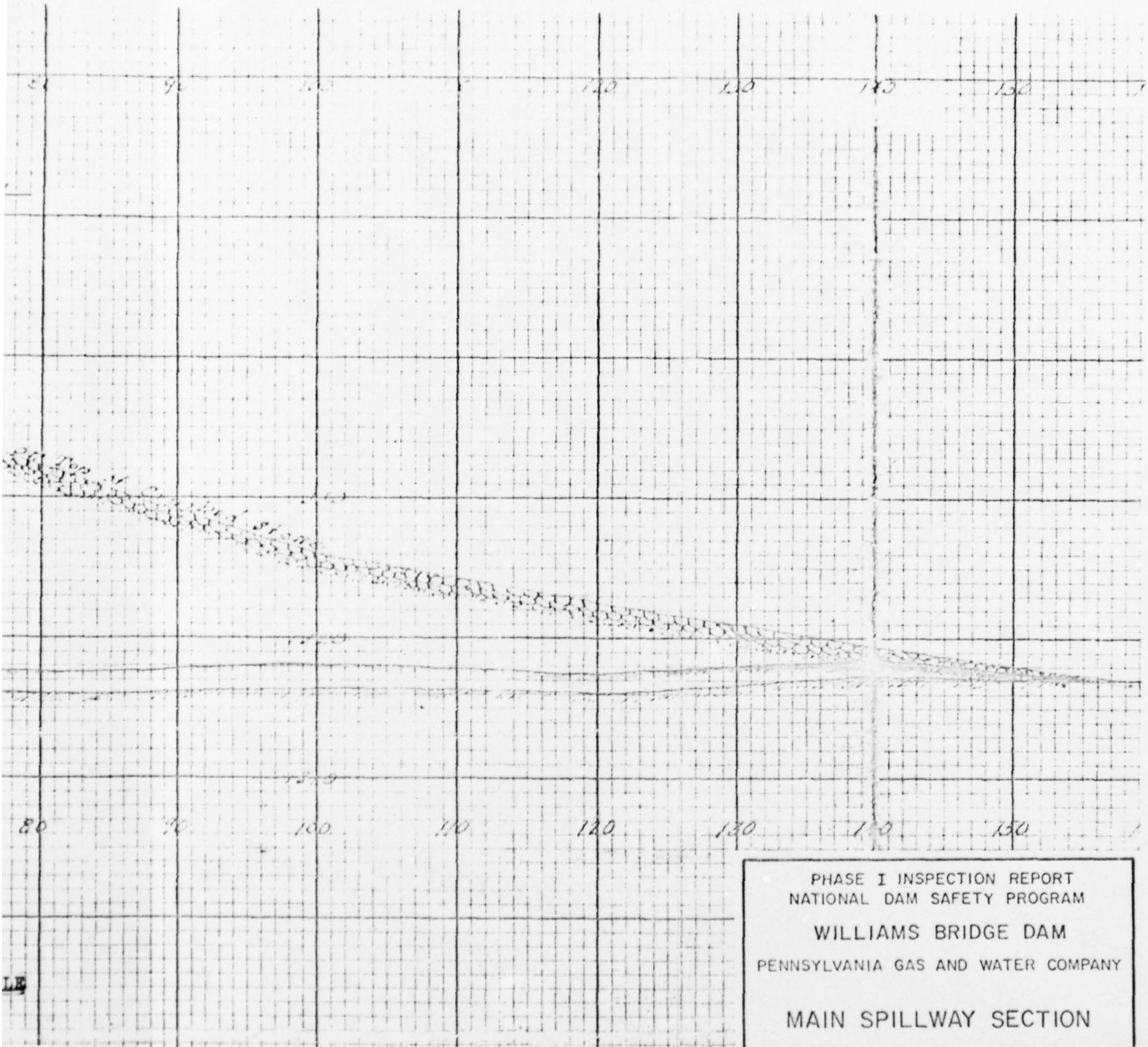



PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

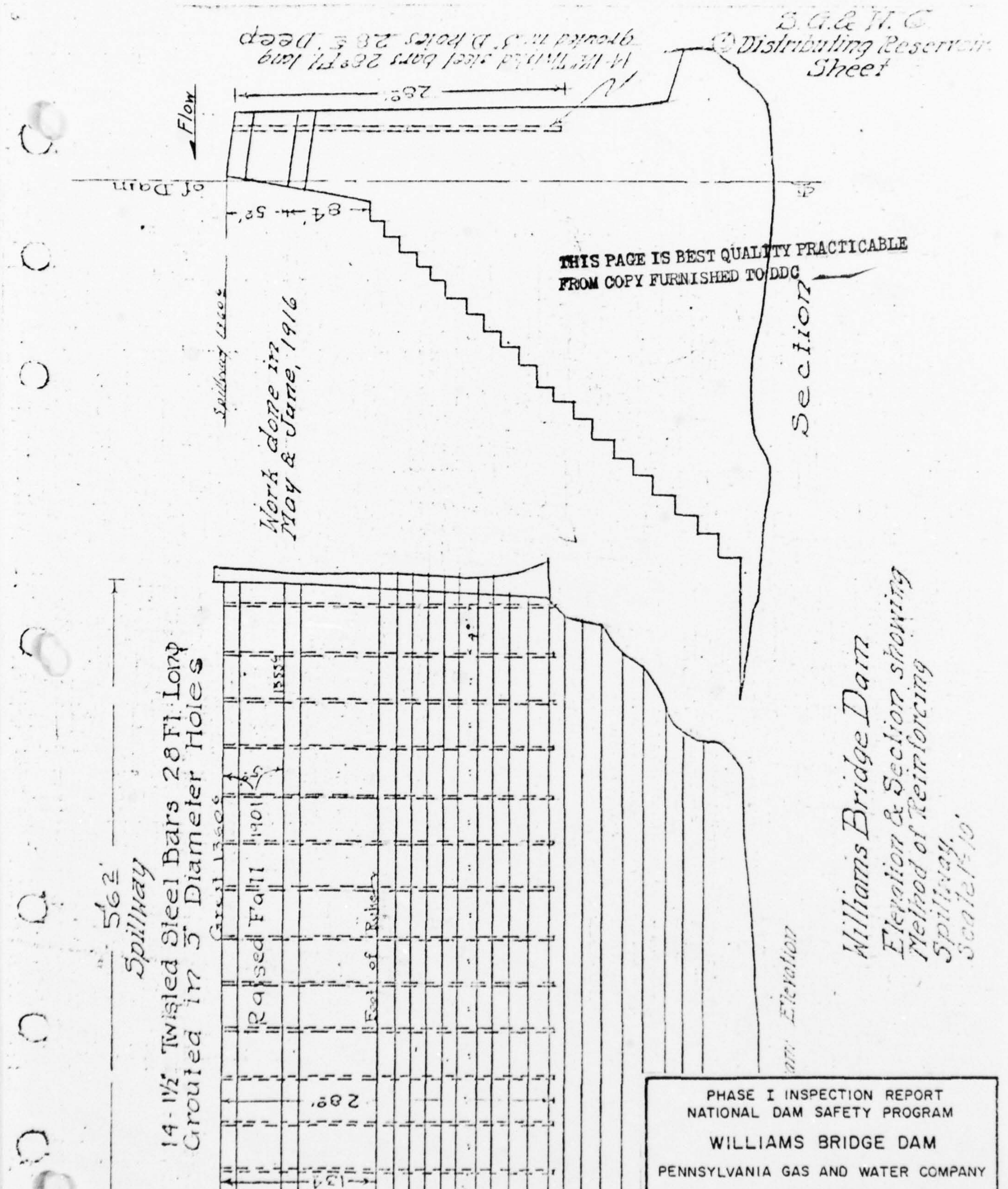
WILLIAMS BRIDGE DAM

PENNSYLVANIA GAS AND WATER COMPANY

MAIN SPILLWAY SECTION

MAY 1978 PLATE 5

4





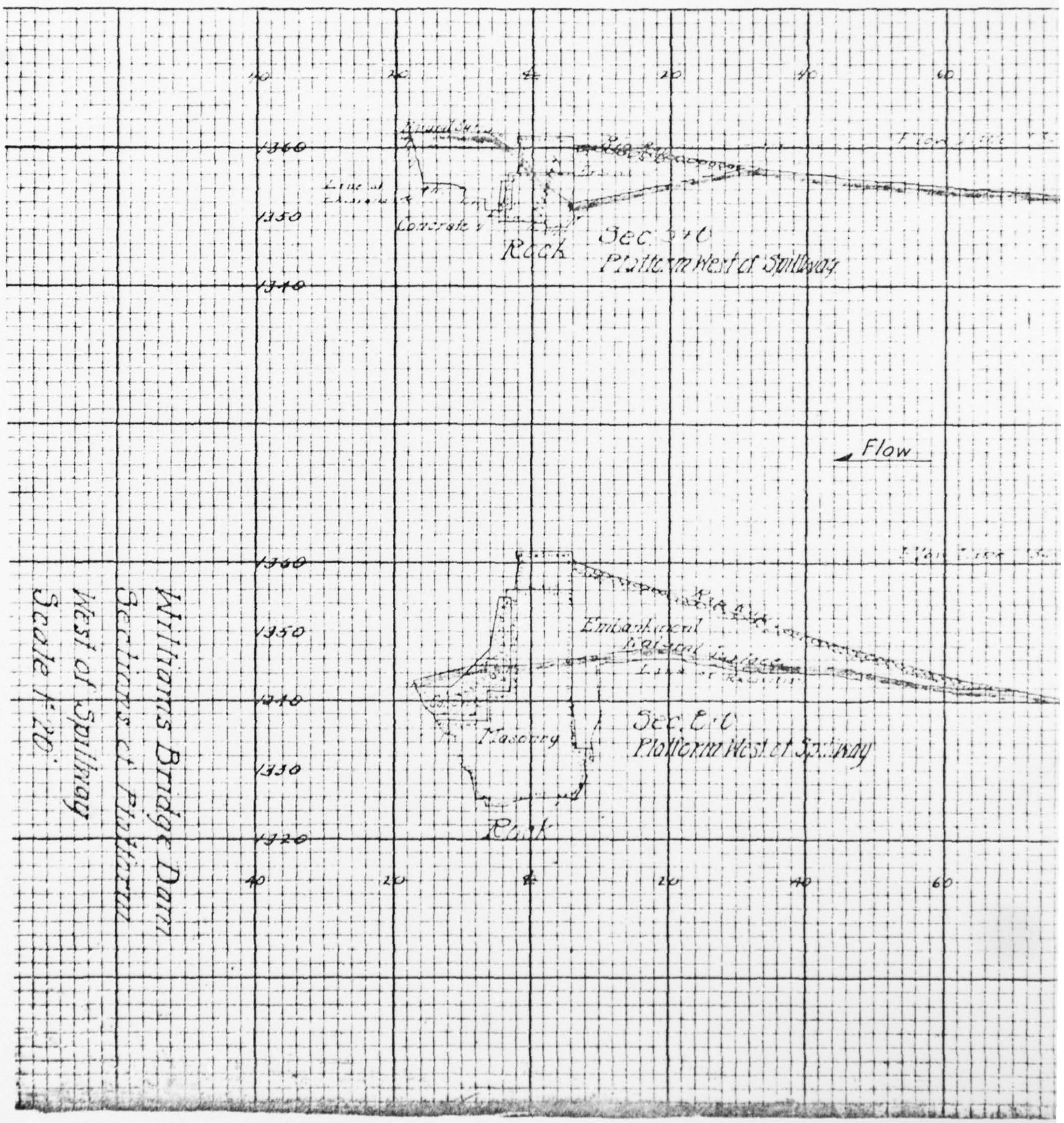

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

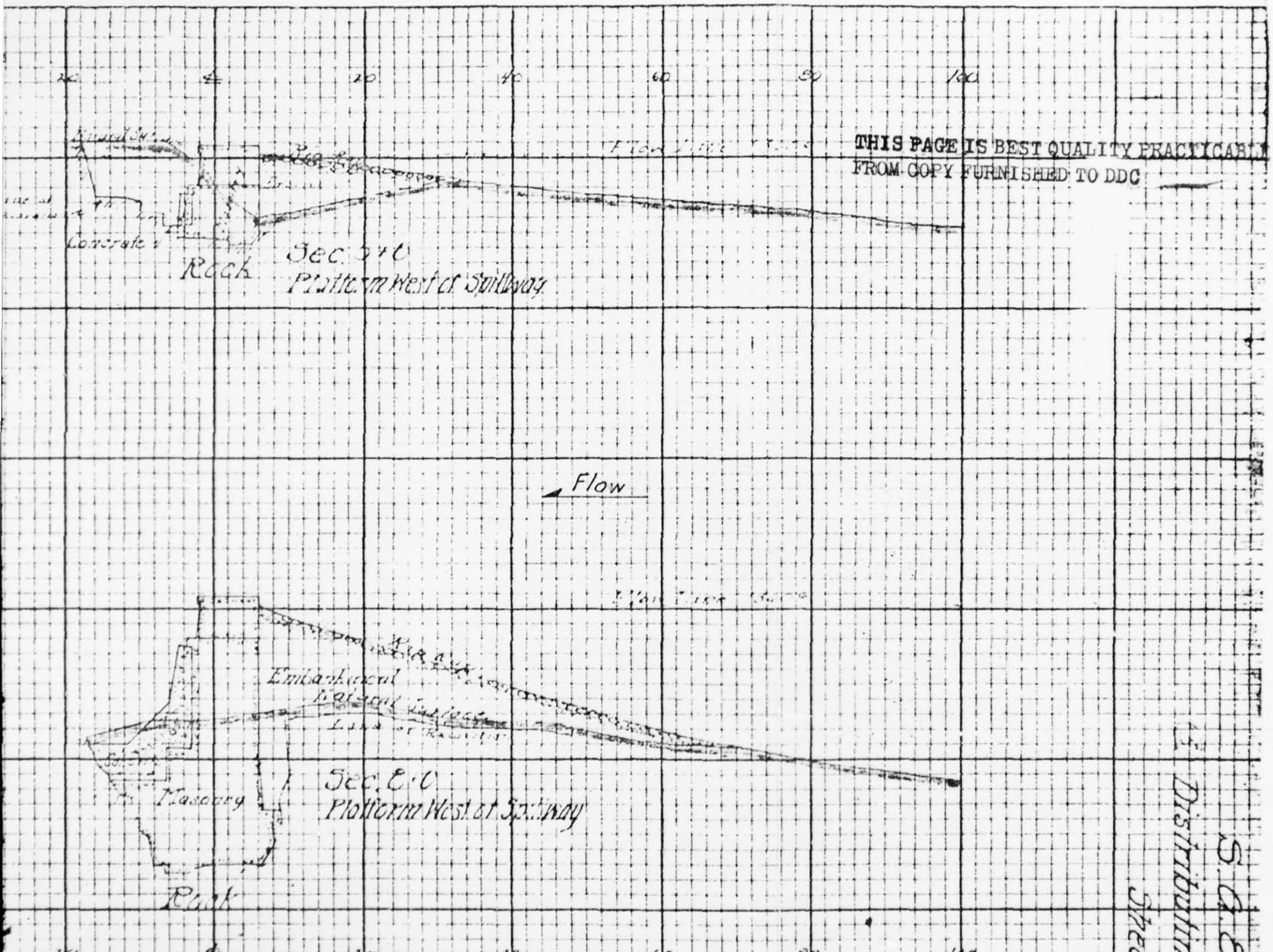
WILLIAMS BRIDGE DAM

PENNSYLVANIA GAS AND WATER COMPANY

DETAILS FOR STRENGTHENING
MAIN SPILLWAY (1916)

MAY 1978 PLATE 6


Williams Bridge Dam
Sections of Platform
West of Spillway
Scale 1:20
Platform West of Spillway
Rock
Concrete
Flow
Embankment
Masonry
Platform West of Spillway
Rock
1360
1350
1340
1330
1320
40
20
20
40
60






PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

WILLIAMS BRIDGE DAM

PENNSYLVANIA GAS AND WATER COMPANY

AUXILIARY SPILLWAY SECTIONS

MAY 1978 PLATE 7

SUSQUEHANNA RIVER BASIN

STAFFORD MEADOW BROOK, LACKAWANNA COUNTY

PENNSYLVANIA

WILLIAMS BRIDGE DAM

PENNSYLVANIA GAS AND WATER COMPANY

PHASE I INSPECTION REPORT

NATIONAL DAM INSPECTION PROGRAM

MAY 1978

# APPENDIX A

## CHECKLIST - ENGINEERING DATA

CHECKLIST

ENGINEERING DATA

DESIGN, CONSTRUCTION, AND OPERATION
PHASE I

NAME OF DAM: Williams Bridge

NDS ID NO.: 373 DER ID NO.: 35-20



| ITEM | REMARKS |
|---|---|
| AS-BUILT DRAWINGS | Construction drawings of original structure and subsequent modifications available. |
| REGIONAL VICINITY MAP | Project is shown on Olyphant, PA-Quadrangle Sheet N 4122.5 - W 7530/7.5, 1946 Photorevised 1969. |
| CONSTRUCTION HISTORY | Constructed 1892-1893 by Meadow Brook Water Company. Modified 1902, 1916, and 1962. |
| TYPICAL SECTIONS OF DAM | Design sections available of original structure and modifications. |
| OUTLETS:<br>Plan<br>Details<br>Constraints<br>Discharge Ratings | Plan and details available. No discharge ratings. |

| ITEM | REMARKS |
|---|---|
| RAINFALL/RESERVOIR RECORDS | None |
| DESIGN REPORTS | 1916 Permit Application Report for proposed spillway strengthening. |
| GEOLOGY REPORTS | 1914 Report on history and design has general geologic description. |
| DESIGN COMPUTATIONS:<br>Hydrology and Hydraulics<br>Dam Stability<br>Seepage Studies | 1914 hydraulic and stability analysis of spillway.<br>1915 design computations for proposed spillway strenghtening.<br>1961 design computations to repair spillway ret. wall.<br>1972 computations of hydrology and alternate plans to increase spillway capacity. |
| MATERIALS INVESTIGATIONS:<br>Boring Records<br>Laboratory<br>Field | None |
| POSTCONSTRUCTION SURVEYS OF DAM | None |

ENGINEERING DATA



| ITEM | REMARKS |
|---|---|
| BORROW SOURCES | Materials obtained from onsite. Embankment is clay with some sand. Stone masonry is sandstone and conglomerate. |
| MONITORING SYSTEMS | Caretakers visit dam daily to check chlorine equipment and observe water level. |
| MODIFICATIONS | 1902 spillway crest raised 5 feet; core wall raised 6 feet; dam raised 4 feet.<br>1916 spillway strengthened with steel bars; concrete added to downstream face auxiliary spillway for stability; concrete training wall built below auxiliary spillway; two concrete buttresses added to core wall at (cont'd. Sheet 4a) |
| HIGH POOL RECORDS | None |
| POSTCONSTRUCTION ENGINEERING STUDIES AND REPORTS | 1914 evaluation of hydraulics and stability.<br>1972 Justin & Courtney hydrology report and improvement study. |
| PRIOR ACCIDENTS OR FAILURE OF DAM:<br>Description<br>Reports | Spillway right retaining wall cracked about 10 feet from top of wall for length of 70 feet (approximately 1941). |

ENGINEERING DATA



| ITEM | REMARKS |
| --- | --- |
| MAINTENANCE AND OPERATION RECORDS | Repair records available; no detailed operation records. |
| SPILLWAY:<br>Plan<br>Sections<br>Details | Construction drawings of original structure and subsequent modifications. |
| OPERATING EQUIPMENT:<br>Plans<br>Details | Plans and cross sections available; no equipment details. |
| PREVIOUS INSPECTIONS<br>Dates<br>Deficiencies | 1921 seepage at left side of spillway; concrete deterioration at auxiliary spillway; seepage at right end of dam.<br>1925 seepage and concrete disintegration.<br>1928 same as 1925<br>1930 seepage at junction spillway and left hillside; concrete disintegration.<br>1933 same as 1930 plus disintegration of concrete on crest.<br>1941 disintegration of concrete on crest; natural ground 1'-6" low at left end of dam; auxiliary spillway concrete facing disintegration; seepage left side of spillway and along gate house; crack in wasteway wall.<br>1953 concrete disintegration on crest wall, auxiliary spillway facing, and auxiliary spillway training wall.<br>1957 same as 1953.<br>1965 same as 1957.<br><br>Note: Terminology and description from original inspection reports. |

ENGINEERING DATA



| ITEM | REMARKS |
| --- | --- |
| MODIFICATIONS (Cont'd. from Sheet 3) | Alignment change; embankment added along downstream face of previously exposed core wall; 2-foot concrete extension added to masonry core wall.<br>1962 spillway right retaining wall repaired using post-tensioned steel rods; repair of deteriorating concrete extension on core wall. |
| | |
| | |
| | |
| | |

CHECKLIST

ENGINEERING DATA

HYDROLOGY AND HYDRAULICS

NAME OF DAM: Williams Bridge NDS ID NO.: 373 DER ID NO.: 35-20

ELEVATION TOP NORMAL POOL (STORAGE CAPACITY): El. 1360.6

ELEVATION TOP FLOOD CONTROL POOL (STORAGE CAPACITY): El. 1366.2

ELEVATION MAXIMUM DESIGN POOL: El. 1366.2 (overtopping elevation)

ELEVATION TOP DAM: El. 1366.2

SPILLWAY CREST:
a. Elevation Main - El. 1360.6; Auxiliary - El. 1361.6
b. Type Main - broad crested with steps; Auxiliary - broad crested; free overfall
c. Width Main - 6 feet; Auxiliary - 8 feet
d. Length Main - 56.3 feet; Auxiliary - 103.3 feet
e. Location Spillover Auxiliary - left abutment; Main - right of auxiliary
f. Number and Type of Gates Ungated

OUTLET WORKS:
a. Type 24-inch blowoff and 24-inch supply line
b. Location Adjacent to right side of main spillway
c. Entrance Inverts Blowoff - El. 1316.5; Supply - El. 1322.0
d. Exit Inverts Blowoff - El. 1316.3; Supply - not applicable
e. Emergency Draindown Facilities 24-inch blowoff

HYDROMETEOROLOGICAL GAGES:
a. Type None
b. Location None
c. Records None

MAXIMUM NONDAMAGING DISCHARGE: Unknown

SUSQUEHANNA RIVER BASIN

STAFFORD MEADOW BROOK, LACKAWANNA COUNTY

PENNSYLVANIA

WILLIAMS BRIDGE DAM

PENNSYLVANIA GAS AND WATER COMPANY

PHASE I INSPECTION REPORT

NATIONAL DAM INSPECTION PROGRAM

MAY 1978

# APPENDIX B

## CHECKLIST - VISUAL INSPECTION

CHECKLIST

VISUAL INSPECTION

PHASE I

Name of Dam: Williams Bridge County: Lackawanna State: Pennsylvania

NDS ID No.: 373 DER ID No.: 35-20

Type of Dam: Earth/masonry core wall; gravity spillway Hazard Category: High

Date(s) Inspection: April 13, 1978 Weather: Clear Temperature: 68° F.

General Soil Condition: Moist

Pool Elevation at Time of Inspection: 1360.8 msl/Tailwater at Time of Inspection: 1312.5 msl

Note: Water surface elevation at typical stream section (300 feet downstream) = El. 1289.0

Inspection Personnel:

| | | |
|---|---|---|
| W. Seip (GFCC) | L. Insalaco (DER) | I. Skoritowski (PG&W) |
| I. Crouse (GFCC) | W. McDonnell (DER) | B. Glocker (PG&W) |
| D. Ebersole (GFCC) | D. Kaufman (PG&W) | D. Pensick (PG&W) |
| | | I. Tunis (PG&W) |

D. Wilson (GFCC) Recorder

EMBANKMENT

Sheet 1 of 2

| VISUAL EXAMINATION OF | OBSERVATIONS | REMARKS OR RECOMMENDATIONS |
|---|---|---|
| SURFACE CRACKS | Upstream surface submerged. No cracks on downstream surface. | Three groundhog holes on downstream surface right of spillway. No signs of recent activity and holes were dry. |
| UNUSUAL MOVEMENT OR CRACKING AT OR BEYOND THE TOE | None | |
| SLOUGHING OR EROSION: Embankment Slopes Abutment Slopes | Sloughing of 4-foot wide by 15-foot long portion of embankment about 12 feet to right of spillway retaining wall at wet area. See additional notes about wet area. | |
| CREST ALIGNMENT: Vertical Horizontal | No irregularities. | |
| RIPRAP FAILURES | None | |

EMBANKMENT

Sheet 2 of 2

| VISUAL EXAMINATION OF | OBSERVATIONS | REMARKS OR RECOMMENDATIONS |
|---|---|---|
| JUNCTION OF EMBANKMENT WITH:<br>Abutment<br>Spillway<br>Other Features | Jct. embankment - spillway retain. wall: embankment slope is flatter than wall slope; embankment slope is warped over short distance to meet wall slope. | |
| ANY NOTICEABLE SEEPAGE OR WET AREAS | 1. Wet area at toe embankment beginning about 75 feet from right abutment to about 400 feet from right abutment; and<br>2. Wet area on downstream face just right of spillway. | 1. Water standing 2 inches deep in swale along toe; not flowing; water is clear; and<br>2. 4' x 15' area saturated and soft but no flow from it. |
| STAFF GAGE AND RECORDER | None | |
| DRAINS | Owner's drawings show 4-inch drain from buttress point to downstream toe (1916); outlet not found; no wet spots in vicinity. | |
| | | |

CONCRETE/MASONRY DAMS (AUXILIARY SPILLWAY)



| VISUAL EXAMINATION OF | OBSERVATIONS | REMARKS OR RECOMMENDATIONS |
|---|---|---|
| ANY NOTICEABLE SEEPAGE | Estimated total leakage through downstream face about 5-10 gpm. Almost all is within 25 feet of left end of spillway. | No erosion resulting from leakage. |
| JUNCTION OF STRUCTURE WITH:<br>Abutment<br>Embankment<br>Other Features | Leakage and loss of mortar at spillway junction.<br>Low area between left end of approach wall and high ground. | Water would flow around left abutment during high flood but would be routed to channel well below spillway. Erosion unlikely because of rock outcrops. |
| DRAINS | Four 1-1/2-inch diameter pipe drains remaining in disintegrated downstream concrete facing. | Functioning of drains unimportant in view of massive structure disintegration. |
| WATER PASSAGES | None | |
| FOUNDATION | Massive sandstone outcrops on left abutment. No evidence of foundation problems. | |

CONCRETE/MASONRY DAMS (AUXILIARY SPILLWAY)

Sheet 2 of 2

| VISUAL EXAMINATION OF | OBSERVATIONS | REMARKS OR RECOMMENDATIONS |
|---|---|---|
| CONCRETE SURFACES:<br>Surface Cracks<br>Spalling | Concrete on downstream face: total disintegration over 50% of area; severe over remainder. | Concrete is ineffective. |
| STRUCTURAL CRACKING | None | |
| ALIGNMENT:<br>Vertical<br>Horizontal | Low area at left abutment tie-in as noted above. | |
| MONOLITH JOINTS | Mortar joints on downstream face are poor where leakage exists; sound in other areas. | |
| CONSTRUCTION JOINTS | Not applicable. | |
| STAFF GAGE OR RECORDER | None | |

## OUTLET WORKS

Sheet 1 of 1

| VISUAL EXAMINATION OF | OBSERVATIONS | REMARKS OR RECOMMENDATIONS |
|---|---|---|
| CRACKING AND SPALLING OF CONCRETE SURFACES IN INTAKE TUNNEL | Masonry tunnel inaccessible. | Bulkhead could be made to fit upstream portal of tunnel. |
| INTAKE STRUCTURE | Masonry in good condition. Chain hoist and trolley beam coated with rust. | |
| OUTLET STRUCTURE | Efflorescence on wall separating valve chamber from screen chamber. 6 inches of water in bottom of chamber. | |
| OUTLET CHANNEL | Tailwater could make valve chamber inaccessible. | Provide access ladder from right wall to roof of valve chamber and another ladder from roof to steps at downstream portal. |
| EMERGENCY GATE | Two men opened 24-inch discharge valve 6 inches in 30 minutes. Valve had not been opened for several years. | PG&W concerned that further opening would draw sediment into 24-inch water supply line. |

UNGATED SPILLWAY (MAIN SPILLWAY)

Sheet 1 of 1

| VISUAL EXAMINATION OF | OBSERVATIONS | REMARKS OR RECOMMENDATIONS |
|---|---|---|
| MASONRY WEIR | Downstream face indeterminate but no blocks missing. Masonry joints sound where visible. | |
| APPROACH CHANNEL | Clear; no operating constraints. | |
| DISCHARGE CHANNEL | Lined with hand-placed stone to end of spillway right retaining wall; all in place. | |
| BRIDGE AND PIERS | None | |
| RIGHT SPILLWAY WINGWALL | 7 of the 11 concrete caps covering post-tensioned bars are disintegrating. Wall alignment being monitored by owner. | |

## INSTRUMENTATION

Sheet 1 of 1

| VISUAL EXAMINATION OF | OBSERVATIONS | REMARKS OR RECOMMENDATIONS |
|---|---|---|
| MONUMENTATION/SURVEYS | Owner is monitoring spillway right retaining wall by survey points. | |
| OBSERVATION WELLS | None | |
| WEIRS | None | |
| PIEZOMETERS | Piezometer on downstream surface installed 1916. | Inspected and judged to be inoperative (has soil 3.7 feet from top of casing). |
| OTHER | None | |

RESERVOIR AND WATERSHED

Sheet 1 of 1

| VISUAL EXAMINATION OF | OBSERVATIONS | REMARKS OR RECOMMENDATIONS |
|---|---|---|
| SLOPES | Mild to steep slopes with rock outcrops; no evidence of creep, rock slides, or land slides. | |
| SEDIMENTATION | No sediment problem reported by owner. | |
| WATERSHED DESCRIPTION | Controlled, forested, no development. | |
| | | |
| | | |

DOWNSTREAM CHANNEL

Sheet 1 of 1

| VISUAL EXAMINATION OF | OBSERVATIONS | REMARKS OR RECOMMENDATIONS |
|---|---|---|
| CONDITION:<br>Obstructions<br>Debris<br>Other | Small bridge about 500 feet downstream. | Bridge would probably be lost in large flood. |
| SLOPES | No evidence of erosion or instability. | |
| APPROXIMATE NUMBER OF HOMES AND POPULATION | Damtender's house only house immediately downstream. Lake Scranton is about 1/4 mile downstream. | Effect of dam failure on Lake Scranton is primary consideration. |
| | | |
| | | |

| VISUAL EXAMINATION OF | OBSERVATIONS | REMARKS OR RECOMMENDATIONS |
|---|---|---|
| AUXILIARY SPILLWAY DOWNSTREAM TRAINING WALL | Very severe scaling and disintegration of top. About 20% of face has severe scaling to maximum depth of 4 inches. Trees at back of wall. | Failure of wall probably would not result in failure of dam. |
| EMBANKMENT CREST WALL | Excellent condition. | Overtopping would probably result in embankment erosion. |
| | | |
| | | |
| | | |

SUSQUEHANNA RIVER BASIN

STAFFORD MEADOW BROOK, LACKAWANNA COUNTY

PENNSYLVANIA

WILLIAMS BRIDGE DAM

PENNSYLVANIA GAS AND WATER COMPANY

PHASE I INSPECTION REPORT

NATIONAL DAM INSPECTION PROGRAM

MAY 1978

# APPENDIX C

# HYDROLOGY AND HYDRAULICS

| | | | |
|---|---|---|---|
| **GANNETT FLEMING CORDDRY AND CARPENTER, INC.** HARRISBURG, PA. | SUBJECT: WILLIAMS BRIDGE (35-20) | FILE NO. 7613.1C | |
| | HYDROLOGY AND HYDRAULICS ANALYSIS | SHEET NO. 1 OF 5 SHEETS | |
| | FOR: USCE - BALTIMORE DISTRICT | | |
| | COMPUTED BY: JMC | DATE: 5/3/78 | CHECKED BY: WS, DATE: 5/4/78 |

CLASSIFICATION

HIGH HAZARD, SINCE DOWNSTREAM POPULATION IS 103,000

INTERMEDIATE SIZE, SINCE HEIGHT = 54 FEET AND CAPACITY = 1,023 AC-FT

REFERENCE: "RECOMMENDED GUIDELINES FOR SAFETY INSPECTION OF DAMS," p. D-3

SPILLWAY DESIGN FLOOD (SDF)

THE SDF SHOULD BE THE PMF (FROM p. D-12 OF "REC. GUIDELINES...")

HYDROLOGY AND HYDRAULICS ANALYSIS

REFERENCE: PHASE I PROCEDURE PACKAGE

II. A. 1. THE PMF INFLOW HYDROGRAPH PEAK IS AVAILABLE FROM THE OWNER (PG&W) OF THE DAM. FROM A 1972 STUDY BY JUSTIN & COURTNEY OF PHILADELPHIA, THE PMF PEAK IS 17,000 CFS.

a. DESIGN CALCULATIONS ARE NOT AVAILABLE

c. THE HYDROGRAPH IS INADEQUATE, SINCE ONLY PEAK IS AVAILABLE

2. PMF INFLOW HYDROGRAPH IS INADEQUATE

a. ACH CHECKED WITH MIKE KANOWITZ, THE BALTIMORE DISTRICT CONTACT. USE THE 1972 STUDY PEAK OF 17,000 CFS AND OBTAIN TOTAL TIME FROM THE PHASE I PROCEDURE PACKAGE GRAPH OF T[illegible] VS. D.A. FOR THE SUSQUEHANNA RIVER BASIN

B. ABILITY OF SPILLWAY TO PASS PMF

1. CAPACITY OF SPILLWAY

| | | | |
|---|---|---|---|
| IN 1914, | TOP OF DAM ELEVATION | = | 1364.0' |
| | MAIN SPILLWAY ELEVATION | = | 1360.6' |
| | AUXILIARY SPILLWAY ELEVATION | = | 1361.6' |
| | MAXIMUM H ON MAIN SPILLWAY | = | 3.4' |
| | MAXIMUM H ON AUXILIARY SPILLWAY | = | 2.4' |

CAPACITY OF BOTH SPILLWAYS FROM 1915 WATER SUPPLY COMMISSION REPORT = 2,320 CFS

SINCE THEN, DAM WAS RAISED TO TOP ELEV. = 1366.2'

SPILLWAY ELEVS. REMAINED CONSTANT

6313

MAXIMUM H ON MAIN SPILLWAY = 5.6'

MAXIMUM H ON AUXILIARY SPILLWAY = 4.6'

NOTE: ASSUME LOW GROUND AT LEFT U/S APPROACH CHANNEL DOES NOT CAUSE CONSIDERABLE EROSION DAMAGE D/S AND CARRIES INSIGNIFICANT AMOUNT OF DISCHARGE.

BROAD-CRESTED WEIR EQUATION (p. 372 VENNARD, ELEMENTARY FLUID MECHANICS)

$$q = \sqrt{g(2H/3)^3}$$

$$\therefore Q = L\sqrt{g(2H/3)^3}$$

CHECKING WITH FORMER CAPACITY OF 2,320 CFS,

WITH L OF MAIN SPILLWAY = 56.3'

AND L OF AUXILIARY SPILLWAY = 103.3'

$$Q_{TOTAL} \stackrel{?}{=} L_M\sqrt{g(2H_M/3)^3} + L_A\sqrt{g(2H_A/3)^3}$$

$$2{,}320 \stackrel{?}{=} 56.3\sqrt{32.2(2\times3.4/3)^3} + 103.3\sqrt{32.2(2\times2.4/3)^3}$$

$$2{,}320 \stackrel{?}{=} 1{,}090 + 1{,}186$$

$$2{,}320 \approx 2{,}276$$

GENERALIZED WEIR EQUATION

$$Q = CLH^{3/2}$$

CALIBRATING C FROM FORMER CAPACITY OF 2,320 CFS (ASSUME C = CONSTANT)

$$Q_{TOTAL} = C\,L_M\,H_M^{3/2} + C\,L_A\,H_A^{3/2}$$

$$2{,}320 = C(56.3)(3.4)^{3/2} + C(103.3)(2.4)^{3/2}$$

$$2{,}320 = 353C + 384C$$

$$C = 3.15$$

DETERMINE PRESENT SPILLWAY CAPACITY

$$Q_{TOTAL} = 3.15(56.3)(5.6)^{3/2} + 3.15(103.3)(4.6)^{3/2}$$

$$Q_{TOTAL} = 2{,}350 + 3{,}210$$

$Q_{TOTAL}$ = 5,560* CFS = CAPACITY OF MAIN AND AUXILIARY SPILLWAYS

FROM 1972 JUSTIN & COURTNEY STUDY, SPILLWAY CAPACITY = 5,600 CFS (√)

* 5,560 CFS WAS USED FOR COMPUTATIONS, BUT THE ROUNDED VALUE OF 5,600 CFS WAS USED IN THE TEXT

3. THE PMF PEAK FLOW IS GREATER THAN THE SPILLWAY CAPACITY (17,000 > 5,560)
   b. ROUTING OF THE PMF IS NOT AVAILABLE
      (1.) THE SPILLWAY WILL PASS (5,560 / 17,000) = 0.327 = p = 32.7% OF THE PMF PEAK
      (2.) INCLUSIVE 3 METHOD TO ESTIMATE STORAGE EFFECT OF RESERVOIR
         (a) TRIANGULAR SHAPE FOR PMF HYDROGRAPH

(b) FROM GRAPH OF TOTAL TIME VS. D.A. FOR SUSQUEHANNA RIVER BASIN:

TOTAL TIME = 24.5 HOURS

$1 - p = 1 - 0.327 = 0.673 = \frac{\Delta ADC}{\Delta ADB}$

$\Delta ADB = \frac{1}{2} bh = \frac{1}{2}(24.5 \text{ HOURS})(17,000 \text{ CFS}) = 208,250$ CFS-HOURS

SUBSTITUTING, $\Delta ADC = (0.673)\,\Delta ADB = (0.673)\,208,250 = 140,150$ CFS-HOURS

∴ 140,150 CFS-HOURS OF STORAGE IS REQUIRED TO PASS PMF W/O OVERTOPPING

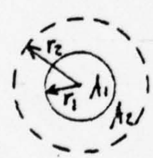
$140,150 \frac{FT^3}{SEC} \text{ HOURS} \times \frac{60 \text{ MIN}}{1 \text{ HOUR}} \times \frac{60 \text{ SEC}}{1 \text{ MIN}} \times \frac{1 \text{ ACRE}}{43,560 \text{ FT}^2} = 11,580$ AC-FT REQUIRED

(C) INCREMENTAL STORAGE AVAILABLE BETWEEN NORMAL POOL ELEVATION AND MAXIMUM POOL ELEVATION

| | | |
|---|---|---|
| NORMAL POOL ELEVATION = SPILLWAY CREST ELEVATION | = | 1360.6' |
| MAXIMUM POOL ELEVATION = TOP OF DAM ELEVATION | = | 1366.2' |
| AREA OF RESERVOIR WITH W.S. @ SPILLWAY CREST | = | 42.64 ACRES |
| AREA OF RESERVOIR WITH W.S. @ TOP OF DAM | = | ? |

ASSUME RESERVOIR SIDE SLOPES OF 2H ON 1V AND ASSUME CIRCULAR SHAPE

∴ WITH 2H ON 1V SLOPE AND $\Delta V = 5.6'$, $\Delta H = 2(5.6') = 11.2'$




$A = \pi r^2$, WHERE $r$ = EQUIVALENT RADIUS OF ASSUMED CIRCULAR SHAPE

$42.64 \text{ ACRES} \times \frac{43,560 \text{ FT}^2}{1 \text{ ACRE}} = \pi r_1^2$

$591,220 = r_1^2$

$r_1 = 768.9$ FT

$r_2 = r_1 + 11.2' = 768.9' + 11.2' = 780.1$ FEET

$A_2 = \pi r_2^2 = \pi (780.1')^2 = 1,912,000 \text{ FT}^2$

$A_2 = 43.89$ ACRES

INCREMENTAL STORAGE $= \left(\frac{A_1 + A_2}{2}\right)\Delta V$

$= \left(\frac{42.64 + 43.89}{2}\right) 5.6' = 242.3$ AC-FT

INCREMENTAL STORAGE = 242 AC-FT

STORAGE REQUIRED = 11,580 AC-FT >> STORAGE AVAILABLE = 242 AC-FT

C. PROCEDURES FOR DETERMINATION OF ADEQUATE / INADEQUATE SPILLWAY CAPACITY

2. STORAGE REQUIRED IS GREATER THAN STORAGE AVAILABLE

a. ETL 1110-2- STATES THREE CONDITIONS THAT MUST EXIST BEFORE THE SPILLWAY CAPACITY IS CONSIDERED TO BE SERIOUSLY INADEQUATE. (CHECK CONDITION "C." IS THE SPILLWAY ABLE TO PASS ½ PMF W/O OVERTOPPING FAILURE?)

b. REPEAT CALCULATIONS FOR $\frac{1}{2}$ PMF PEAK

$\frac{1}{2}$ PMF PEAK = $\frac{1}{2}$ (17,000) = 8,500 CFS



II. B. ABILITY OF SPILLWAY TO PASS $\frac{1}{2}$ PMF

1. CAPACITY OF SPILLWAY = 5,560 CFS

3. $\frac{1}{2}$ PMF PEAK FLOW IS GREATER THAN THE SPILLWAY CAPACITY (8,500 > 5,560)

b. ROUTING OF $\frac{1}{2}$ PMF IS NOT AVAILABLE

(1.) THE SPILLWAY WILL PASS (5,560 / 8,500) = 0.654 = p = 65.4% OF $\frac{1}{2}$ PMF PEAK

(2.) INCLOSURE 3 METHOD TO ESTIMATE STORAGE EFFECT OF RESERVOIR

(a) TRIANGULAR SHAPE FOR $\frac{1}{2}$ PMF HYDROGRAPH

(b) SAME AS BEFORE, EXCEPT THAT PEAK IS NOW 8,500 CFS

$1-p = 1 - 0.654 = 0.346 = \frac{\Delta AOC}{\Delta AOB}$

$\Delta AOB = \frac{1}{2} b h = \frac{1}{2}$ (24.5 HOURS)(8,500 CFS) = 104,130 CFS-HOURS

SUBSTITUTING, $\Delta AOC = (0.346)\ \Delta AOB = (0.346)(104,130)$ = 36,030 CFS-HOURS

$\therefore$ 36,030 CFS-HOURS IS REQUIRED TO PASS $\frac{1}{2}$ PMF W/O OVERTOPPING

$$36,030\ \frac{FT^3}{SEC}\ HOURS \times \frac{60\ MIN}{1\ HOUR} \times \frac{60\ SEC}{1\ MIN} \times \frac{1\ ACRE}{43,560\ FT^2} = 2,978\ AC\text{-}FT$$

(c) INCREMENTAL STORAGE AVAILABLE BETWEEN NORMAL POOL ELEVATION AND MAXIMUM POOL ELEVATION - SEE SHEET 3 - = 242 AC-FT

STORAGE REQUIRED = 2,978 AC-FT >> STORAGE AVAILABLE = 242 AC-FT

C. PROCEDURES FOR DETERMINATION OF ADEQUATE/ INADEQUATE SPILLWAY CAPACITY

2. STORAGE REQUIRED IS GREATER THAN STORAGE AVAILABLE

a. ETL 1110-2-

① THERE IS A HIGH HAZARD OF LOSS OF LIFE FROM LARGE FLOWS DOWNSTREAM OF DAM

② CHECK TAILWATER AT INSTANT BEFORE OVERTOPPING OCCURS

③ THE DAM AND SPILLWAYS ARE NOT CAPABLE OF PASSING $\frac{1}{2}$ PMF WITHOUT OVERTOPPING FAILURE.

c. TAILWATER AT INSTANT BEFORE OVERTOPPING OCCURS

SPILLWAY CAPACITY DISCHARGE = 5,560 CFS ; FROM HEC-2 COMPUTER RUN,☆

TAILWATER DEPTH @ Q = 5,560 CFS IS 10.1 FEET

| | |
|---|---|
| TOP OF DAM ELEVATION | = 1366.2' |
| HEIGHT OF DAM | = 54' |
| BOTTOM OF DAM ELEVATION | = 1312.2' |

☆ COMPUTER MODELED FROM A SURVEYED CROSS-SECTION D/S OF DAM

GANNETT FLEMING CORDDRY AND CARPENTER, INC.
HARRISBURG, PA.

SUBJECT: WILLIAMS BRIDGE (35-20) — FILE NO. 7513.1C
HYDROLOGY AND HYDRAULICS ANALYSIS — SHEET NO. 5 OF 5 SHEETS
FOR: USCE - BALTIMORE DISTRICT
COMPUTED BY: JMC — DATE: 5/3/78 — CHECKED BY: WS — DATE: 5/4/78

TAILWATER DEPTH = 10.1'
TAILWATER ELEVATION = 1322.3'
TOP OF DAM ELEV. − TAILWATER ELEV. = 1366.2' − 1322.3' = 43.9'

PERCENT OF PMF THAT SPILLWAY CAN PASS
GENERAL FORMULA

$$\%\text{ OF PMF THAT SPILLWAY CAN PASS} = \frac{Q_T}{Q_{PMF}} \times 100\%,$$

WHERE $Q_T = Q_{SPILLWAY} + \frac{2S}{\Delta t}$,

$S = \sum_{i=1}^{n} S_i$ FOR UPSTREAM RESERVOIR CASES,

AND T = TOTAL TIME OF PMF HYDROGRAPH FROM CURVE FOR SUSQUEHANNA RIVER BASIN

$$\%\text{ OF PMF} = \frac{5{,}560 + \left(\frac{2 \times (242)\text{ AC-FT}}{24.5\text{ HOURS}} \times \frac{43{,}560\text{ FT}^2\text{ HRS}}{3{,}600\text{ AC-SECS}}\right)}{17{,}000} \times 100\%$$

$$= \frac{5{,}560 + 239}{17{,}000} \times 100\%$$

$$= 34\%$$

Williams Bridge Watershed
Hydrology

JC



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Triangular hydrograph analysis described in Small Dams.

A = 5.4 sq. mi.

Duration = 1 hr. (T.R. used 0.5 hr)

time of concentration = 2 hours (assume) (T.R. used 1.64 hrs)

$T_p = D/2 + 0.6 t_c$

$T_p$ .5 + 1.2 = 1.7 hrs.

$q_p = \frac{484 A Q}{T_p} = \frac{484 \times 5.4 \times 1}{1.7} = 1540$ cfs / 1 inch runoff.

6 hour storm PMP = 24"
Distribution - Small Dams Zone C - Figure 4 p32

| | | |
|---|---|---|
| 1st hr | = 12" | 50% |
| 2nd | - 4" | 15% |
| 3rd | - 3" | 10% |
| 4th | - 2" | 10% |
| 5th | - 2" | 10% |
| 6th | 1" | 5% |
| | 24" | |

$T_b = 2.67 \times t_p =$
$= 2.67 \times 1.7 = 4.5$ hours.

| Time hours | Δ Rain = (runoff) ΔQ | Δ Peak. = 1540 × ΔQ | Begin time | Time of Peak | Time @ end. |
|---|---|---|---|---|---|
| 0 | | | | | |
| | 12 | 18,500 | 0 | 1.7 | 4.5 |
| 1 | | | | | |
| | 4 | 6,150 | 1 | 2.7 | 5.5 |
| 2 | | | | | |
| | 3 | 4,620 | 2 | 3.7 | 6.5 |
| 3 | | | | | |
| | 2 | 3,080 | 3 | 4.7 | 7.5 |
| 4 | | | | | |
| | 2 | 3,080 | 4 | 5.7 | 8.5 |
| 5 | | | | | |
| | 1 | 1,540 | 5 | 6.7 | 9.5 |
| 6 | | | | | |

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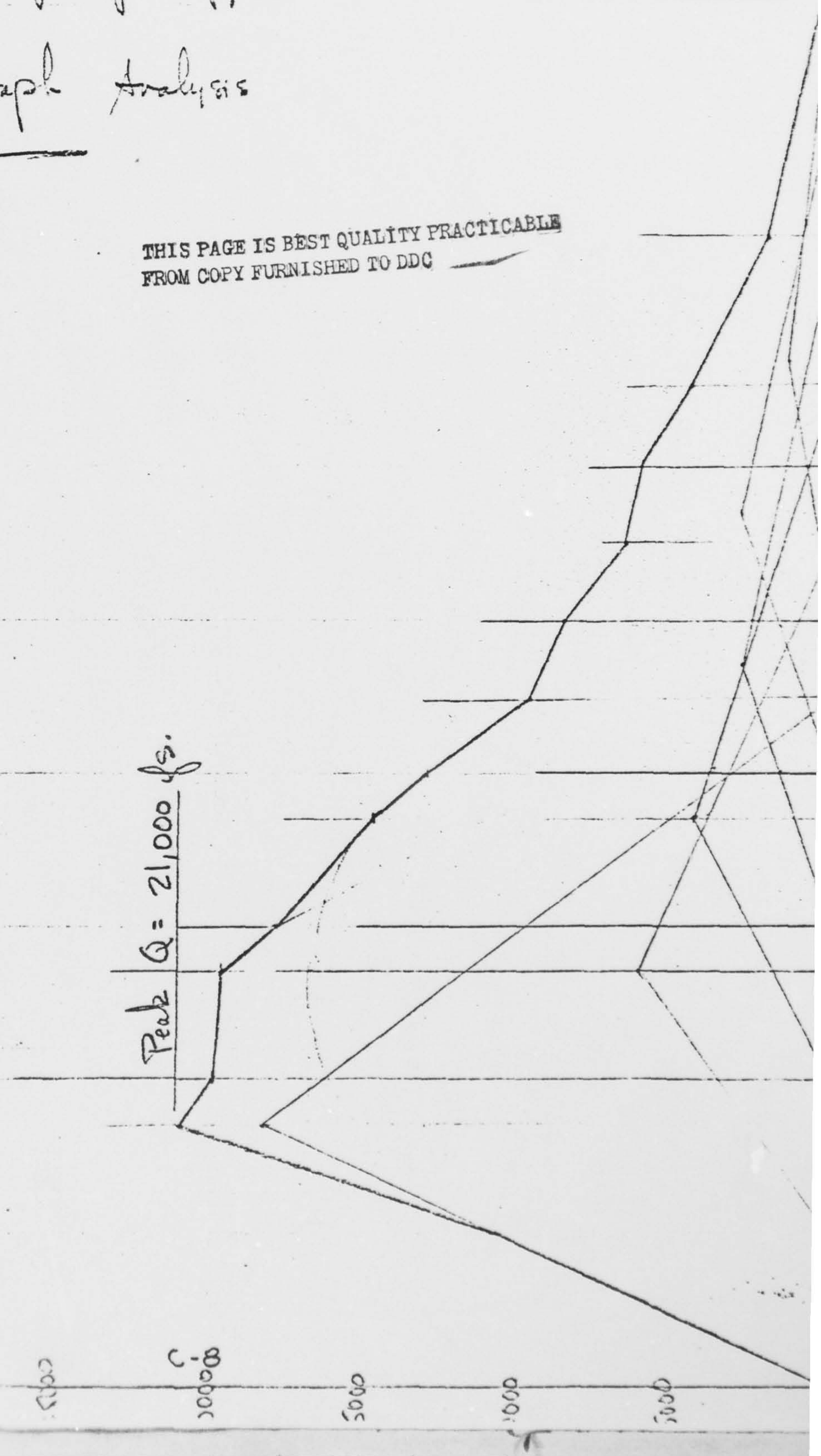
Triangular Hydrograph Analysis





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PROJECT Williams Bridge Hydrology



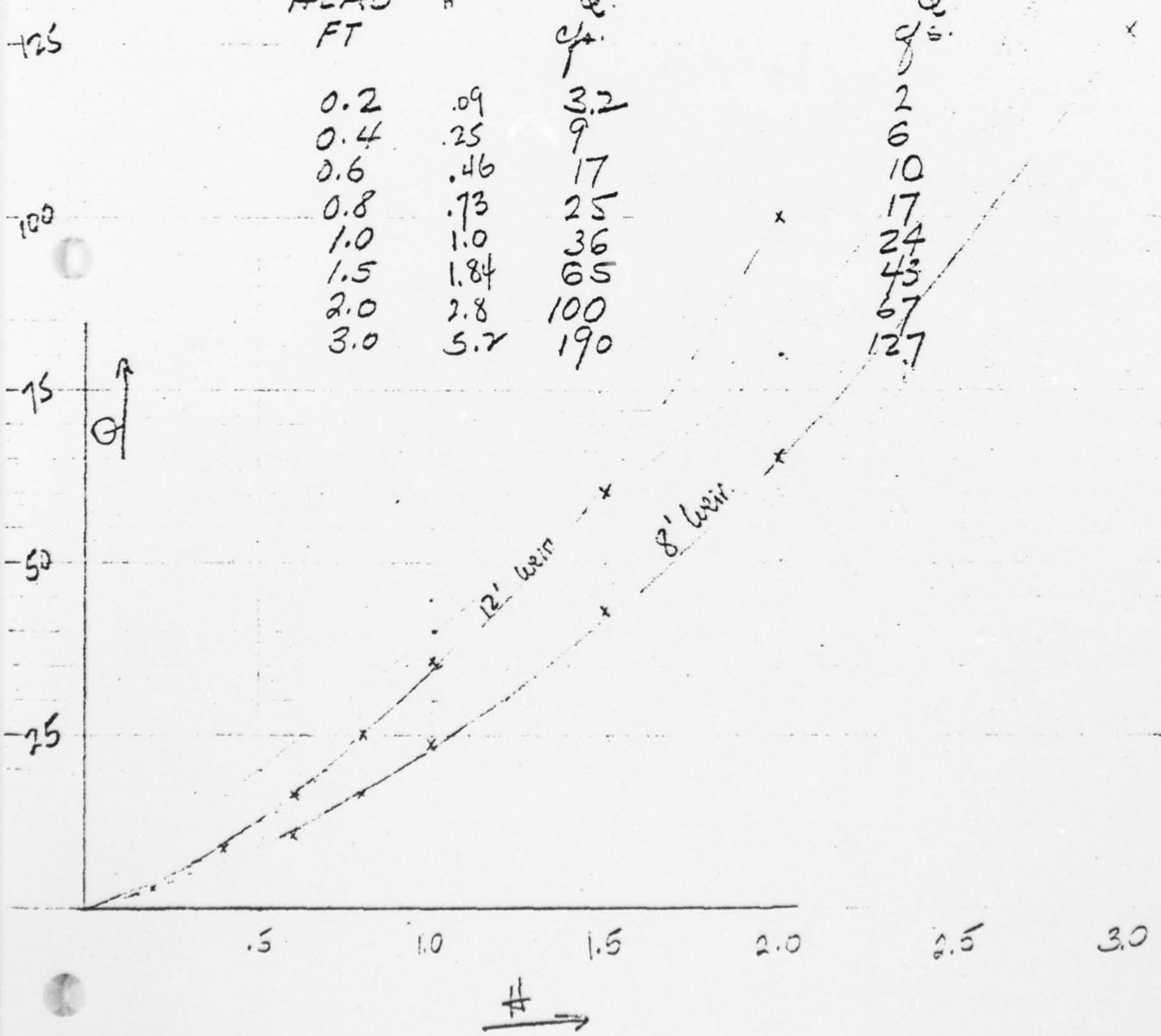
Weir Calibration.

$$Q = 3.0\ L\ H^{3/2}$$

Stafford Meadowbrook L = 12'

Long Swamp L = 8'

| HEAD FT | $H^{3/2}$ | 12' weir Q cfs | 8' weir Q cfs |
|---|---|---|---|
| 0.2 | .09 | 3.2 | 2 |
| 0.4 | .25 | 9 | 6 |
| 0.6 | .46 | 17 | 10 |
| 0.8 | .73 | 25 | 17 |
| 1.0 | 1.0 | 36 | 24 |
| 1.5 | 1.84 | 65 | 43 |
| 2.0 | 2.8 | 100 | 67 |
| 3.0 | 5.2 | 190 | 127 |



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ACOWA Manual #13 Unit hydrograph method.

Lag time = $1.05 A^{0.6}$ A = 5.4 t = 2.85 (3 hrs)

Max. 6 hr precip. = 24"

cfs intervals = $\frac{A \times N_d}{.03719}$ $N_d$ = 24 (1 hr intervals)

Duration = D = 1 hr. = 1/3 t (lag time)

cfs intervals = $\frac{5.4 \times 24}{.03719}$ = 3500

| Time lags. | Distribution % | U.H. ordinates | Time in hours |
|---|---|---|---|
| − 0.17 | 0 | 0 | −0.5 |
| + 0.16 | 7.5 | 270 | 0.5 |
| .5 | 31.5 ← Peak | 1100 | 1.5 |
| .83 | 20 | 700 | 2.5 |
| 1.16 | 12 | 420 | 3.5 |
| 1.5 | 6 | 210 | 4.5 |
| 1.83 | 4 | 140 | 5.5 |
| | 81.0 | | |

Rainfall distribution 12", 4", 3", 2", 2", 1" @ 1 hr intervals
Total = 24"

| Time hrs. | U.H. Ord. | 1st hr. | 2nd | 3rd | 4th | 5th | 6th | Total |
|---|---|---|---|---|---|---|---|---|
| −0.5 | 0 | 0 | - | - | - | - | - | 0 |
| .5 | 270 | 3240 | 0 | - | - | - | - | 3510 |
| 1.5 | 1100 | 13200 | 1100 | 0 | - | - | - | 14,300 ← |
| 2.5 | 700 | 8400 | 4400 | 800 | 0 | - | - | 13,600 |
| 3.5 | 420 | 5000 | 2800 | 3300 | 540 | 0 | - | 11,640 |
| 4.5 | 210 | 2500 | 1680 | 2100 | 2200 | 540 | 0 | 9020 |
| 5.5 | 140 | 1700 | 840 | 1260 | 1400 | 2200 | 270 | 7600 |

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PROJECT Williams Bridge Hydrology — COMP. BY [illegible]

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Storm of 8/27/71

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Hydrographs @ Williams Bridge & Long Swamp.

| ① Time | ② Head on Williams Bridge weir | ③ Q Williams Bridge | ④ Head on Long Swamp weir | ⑤ Q @ Long Swamp | ⑥ Total Q W.B. & L.S. | % | ⑦ U.H. ⑥/.074 |
|---|---|---|---|---|---|---|---|
| Midnight | .22 | 4 | .12 | 1 | 5 | 2% | 67.5 |
| 1 | .22 | 4 | .13 | 1 | 5 | 2% | 67.5 |
| 2 | .24 | 5 | .14 | 1 | 6 | 2% | 81 |
| 3 | .26 | 5 | .14 | 1 | 6 | 2% | 81 |
| 4 | .30 | 6 | .15 | 1 | 7 | 3% | 86 |
| 5 | .50 | 12 | .20 | 2 | 14 | 5% | 190 |
| 6 | .54 | 14.5 | .22 | 2.5 | 17 | 7% | 230 |
| 7 | .48 | 12 | .22 | 2 | 14 | 6% | 190 |
| 8 | .44 | 10 | .22 | 2 | 12 | [illegible] | 160 |
| 9 | .43 | 10 | .22 | 2 | 12 | 4.5 | 160 |
| hours 10 | .43 | 10 | .22 | 2 | 12 | 4.5 | 160 |
| 11 | .43 | 10 | .22 | 2 | 12 | 4.5 | 160 |
| 17 12 | .43 | 10 | .22 | 2 | 12 | 4.5 | 160 |
| 1 | .44 | 10 | .20 | 2 | 12 | 4.5 | 160 |
| 2 | .44 | 10 | .19 | 2 | 12 | 4.5 | 160 |
| 3 | .44 | 10 | .19 | 2 | 12 | 4.5 | 160 |
| 4 | .42 | 10 | .19 | 2 | 12 | 4.5 | 160 |
| 5 | .42 | 10 | .19 | 2 | 12 | 4.5 | 160 |
| 18 6pm. | .40 | 9 | .18 | 2 | 11 | | 150 |
| | | | | | 204 | | |
| 24 6am. | .28 | 6 | .15 | 1 | 7 | | 100 |

Total cfs hrs = 204 + 6 hrs × $\frac{11+7}{2}$ (6PM – 6am) = 204 + 54 = 258

Total drainage area = 5.4 sq mi

Inches of runoff = $\frac{258}{5.4} \times \frac{15.5}{\times 10^{-4}}$ = .074"

Total Rainfall = 0.95

Beginning of rise @ 1:15 Peak @ 5:30

$t_p = 5.5 - 1.2 = 4.3$ D = 1.0

$t_p = D/2 + 0.6\ T_c$ $0.6T_c = -0.5 + 4.3 = 3.8$

$T_c = 3.8/0.6 = 6.35$ hours.

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PROJECT Williams Bridge Hydrology

Storm of 8/27

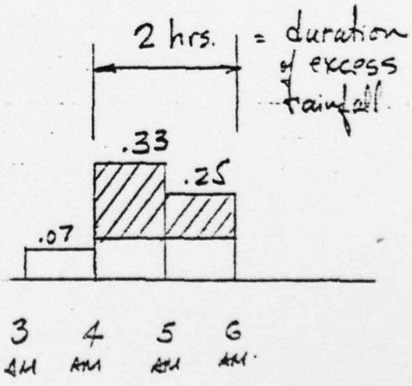
| Time | Rain for period. | Total Rainfall. | Head on Weir | Q |
|---|---|---|---|---|
| | Williams Bridge gage. | | Williams Bridge Weir | |
| Noon | | | | |
| | 0 | | | |
| 1 | | | | |
| | 0 | | | |
| 2 | | | | |
| | 0 | | | |
| 3 | | | | |
| | 0 | | | |
| 4 | | | | |
| | 0 | | | |
| 5 | | | | |
| | 0.03 | | | |
| 6 | | .03 | | |
| | .05 | | | |
| 7 | | .08 | | |
| | .02 | | | |
| 8 | | .10 | | |
| | 0 | | | |
| 9 | | .10 | | |
| | .03 | | | |
| 10 | | .13 | | |
| | .02 | | | |
| 11 | | .15 | | |
| | .05 | | | |
| 12 Midnight | | .20 | | |
| | .02 | | | |
| 1 | | .22 | | |
| | .05 | | | |
| 2 | | .27 | | |
| | .03 | | | |
| 3 | | .30 | | |
| | .07 ↙ | | | |
| 4 | | .37 | | |
| | .33 ↙ | | | |
| 5 | | .70 | | |
| | .25 ← | | | |
| 6 | | .95 | | |

Duration of excess rainfall.

$0.35 = y + (y - .08)$ ← excess precip

$2y = .43$

$y = 0.21$

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PROJECT Williams Bridge Hydrology

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Storm of 6/27/71

6 hr. PMP - 6 hr storm - 24"

| distribution | Hr. | % | inches. |
|---|---|---|---|
| | 1 | 50 | 12 |
| | 2 | 15 | 4 |
| | 3 | 10 | 3 |
| | 4 | 10 | 2 |
| | 5 | 10 | 2 |
| | 6 | 5 | 1 |

Note: Base flow not subtracted

| Hr. | U.H. ord. | 1st hr. | 2nd hr | 3rd hr | 4th hr. | 5th hr. | 6th hr. | Total cfs |
|---|---|---|---|---|---|---|---|---|
| 0 | 68 | 800 | — | — | — | — | — | 800 |
| 1 | 68 | 800 | 270 | — | — | — | — | 1070 |
| 2 | 81 | 970 | 270 | 200 | — | — | — | 1440 |
| 3 | 81 | 970 | 320 | 200 | 140 | — | — | 1630 |
| 4 | 86 | 1000 | 320 | 240 | 140 | 140 | — | 1840 |
| 5 | 190 | 2300 | 340 | 240 | 160 | 140 | 70 | 3250 |
| 6 | 230 | 2780 | 760 | 260 | 160 | 160 | 70 | 4190 |
| 7 | 190 | 2300 | 920 | 570 | 170 | 160 | 80 | 4200 |
| 8 | 160 | 1900 | 760 | 690 | 380 | 170 | 80 | 3980 |
| 9 | 160 | 1900 | 640 | 570 | 460 | 380 | 85 | 3985 |
| 10 | 160 | 1900 | 640 | 480 | 380 | 460 | 190 | 4050 |
| 11 | 160 | 1900 | 640 | 480 | 320 | 380 | 230 | 3950 |
| 12 | 160 | 1900 | 640 | 480 | 320 | 320 | 320 | 3980 |
| 18 | 150 | 1800 | 640 | 480 | 320 | 320 | 320 | 3880 |
| 24 | 100 | 1200 | 640 | 480 | 320 | 320 | 320 | 3280 |

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PROJECT Williams Bridge Hydrology

Determination of $T_c$

1. SCS Hydrology Handbook - Section 4 p 3.15-2 & 3.15-3

$$T_c = \frac{0.71}{b} \qquad b = \frac{\log q_1 - \log q_2}{t_2 - t_1}$$

b = slope of recession curve on semilog paper.

| Storm date: | 7/29/71 | 8/27/71 |
|---|---|---|
| 1. $q_1$ | 165 | 60 |
| 2. $q_2$ | 115 | 45 |
| 3. $\log q_1$ | 2.2175 | 0.77815 |
| 4. $\log q_2$ | 2.0607 | 0.65321 |
| 5 (3)-(4) | .1568 | .12494 |
| 6 $t_1$ (hours) | 4 | 6 |
| 7. $t_2$ | 6 | 8 |
| 8. $(t_2 - t_1)$ | 2 | 2 |
| 9. (5)/(8) = b | .0784 | .06247 |
| 10 0.71/b = $T_c$ in hours. | .71/.0784 = 9 hrs. | 11.4 hrs |

11. Average $T_c$ in hours $\frac{9+11.4}{2} = \frac{20.4}{2} = 10.2$ hrs. $= T_c$

N.G.

2. SCS handbook Figure 3.15-3 - also small Dams p 47

L = 20,000 ft.
H = 400 ft.
$T_c$ = 1.2 hrs.

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PROJECT Lake Scranton

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Peak Discharge using method described in Chow's Hydrology Handbook p. 21-38 & 39 (H.L. Cook's method)

Table 21-22

| Watershed Characteristic. | W. |
|---|---|
| Rolling - avg slope ≈ 6% | 25 |
| Infiltration - clay | 15 |
| Vegetal cover 90% - good cover | 5 |
| Surface Storage. more than 3% swamps | 5 |
| Total W = | 50 |

Assume Rainfall factor = 0.9

50 x 0.9 = 45 ← W

Drainage Area = 5.4 Sq. mi = 3500 Acres.

50 year frequency peak = 2,900 cfs. (see sheet 14a following)

From Weather Bureau Tech. Paper 40 (Rainfall Frequency Atlas)
10 Sq. mi 6 hr PMP = 24"
50 yr freq. (10 Sq. mi 6 hr) = 4" $\frac{24}{4} \times 2900 =$ 17,400 cfs. for PMP.

Rational Formula

Q = CIA  Use C = 0.40

D = Tc = 1 hr.  I = 12.5 in/hr PMP

A = 5

Q = 0.40 x 12.5 x 5 x 640 = 16000 c/s

Sht. No. 10



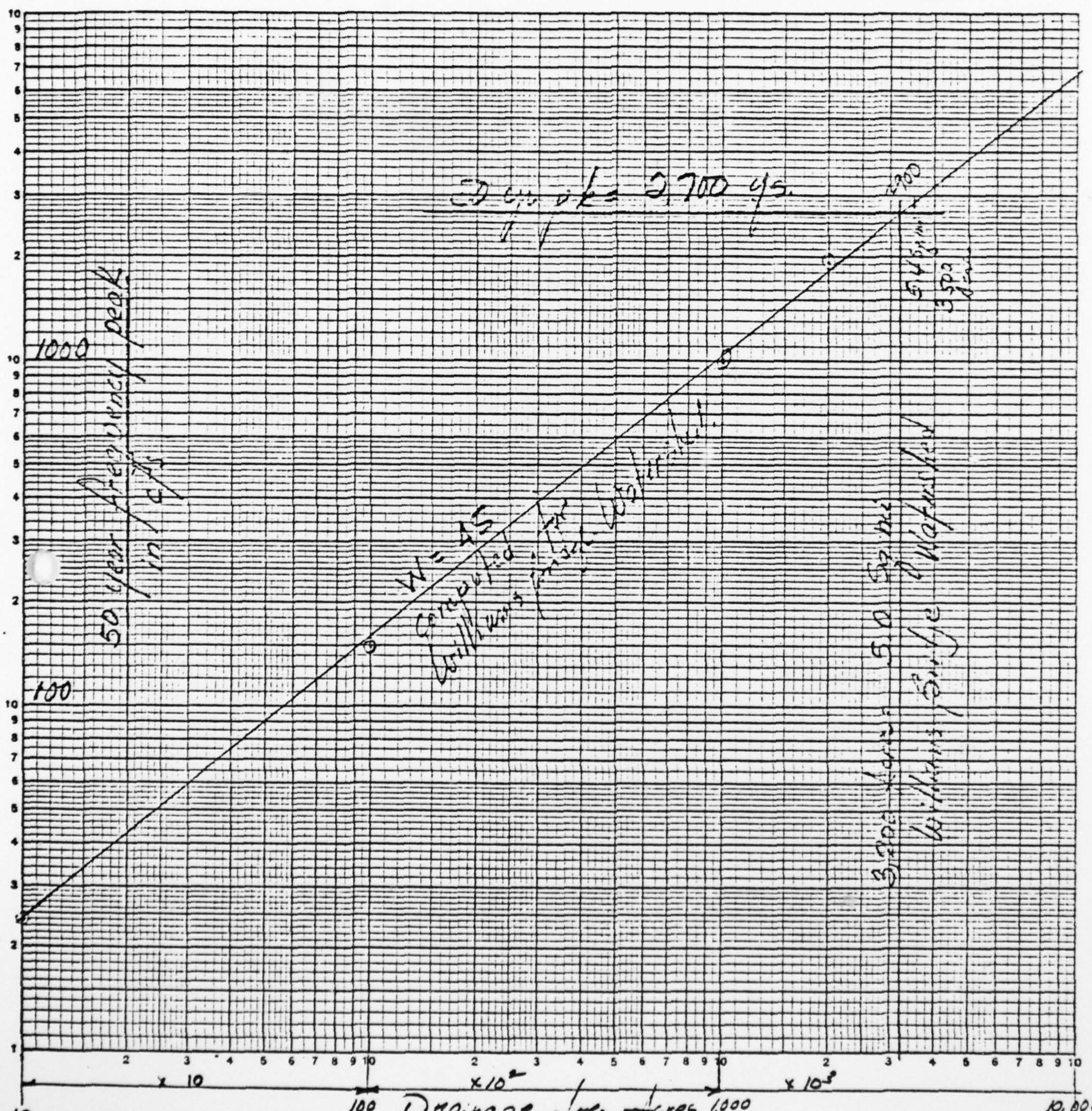


Figure 21-8 - Chow Hydrology Handbook

Williams Bridge Hydrology
12/71

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PROJECT Wms Bridge & Lake Scranton Hydrology

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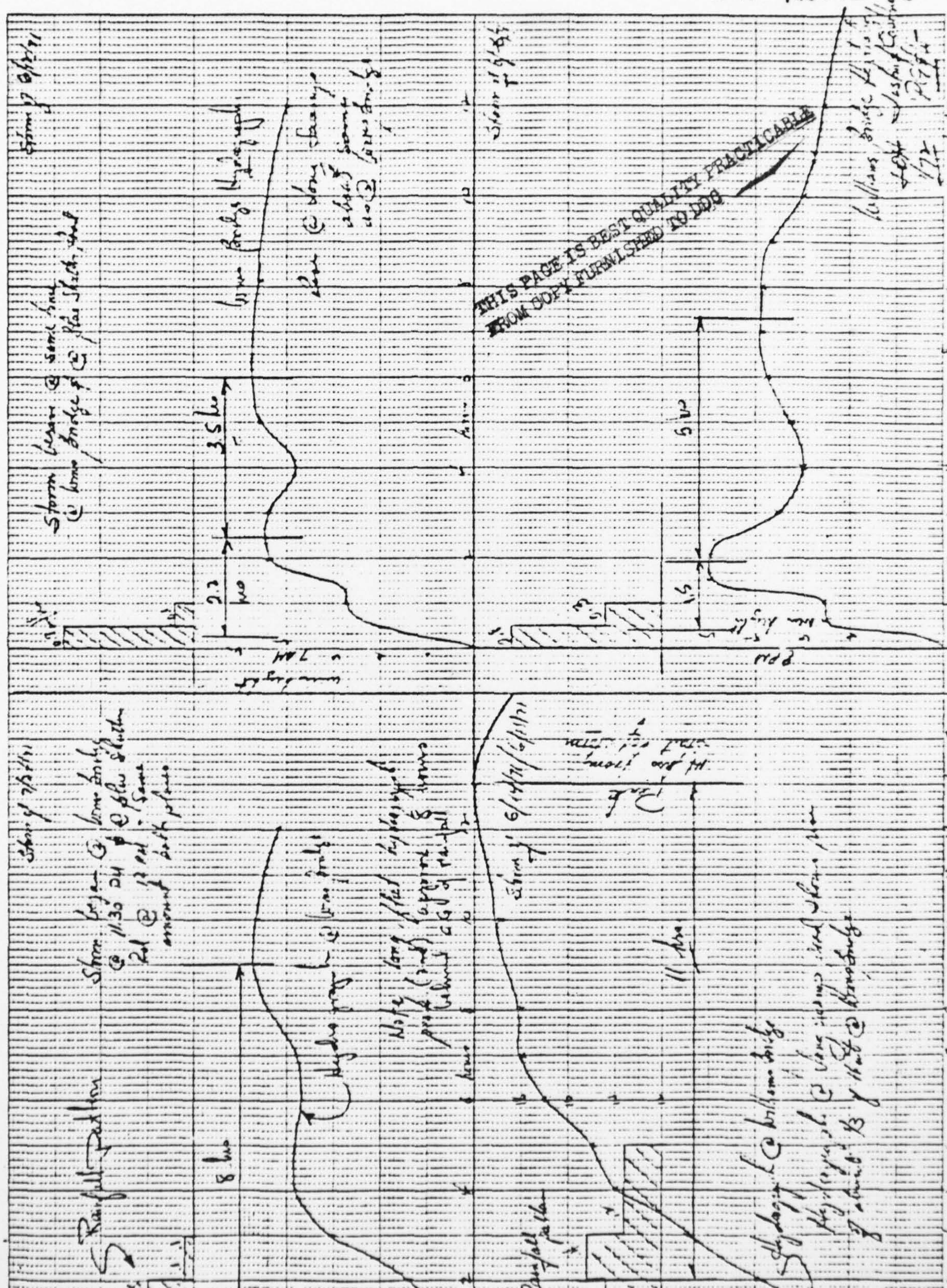
The following two pages are graphs showing the rainfall pattern & streamflow hydrographs for various storms on the Williams Bridge watershed. Storms of 7/29/71 & 8/27/71 are shown on previous pages of this report.

These hydrographs show the flat recession curves, & therefore the long period of runoff from storms on the watershed.

Sht. No. 12





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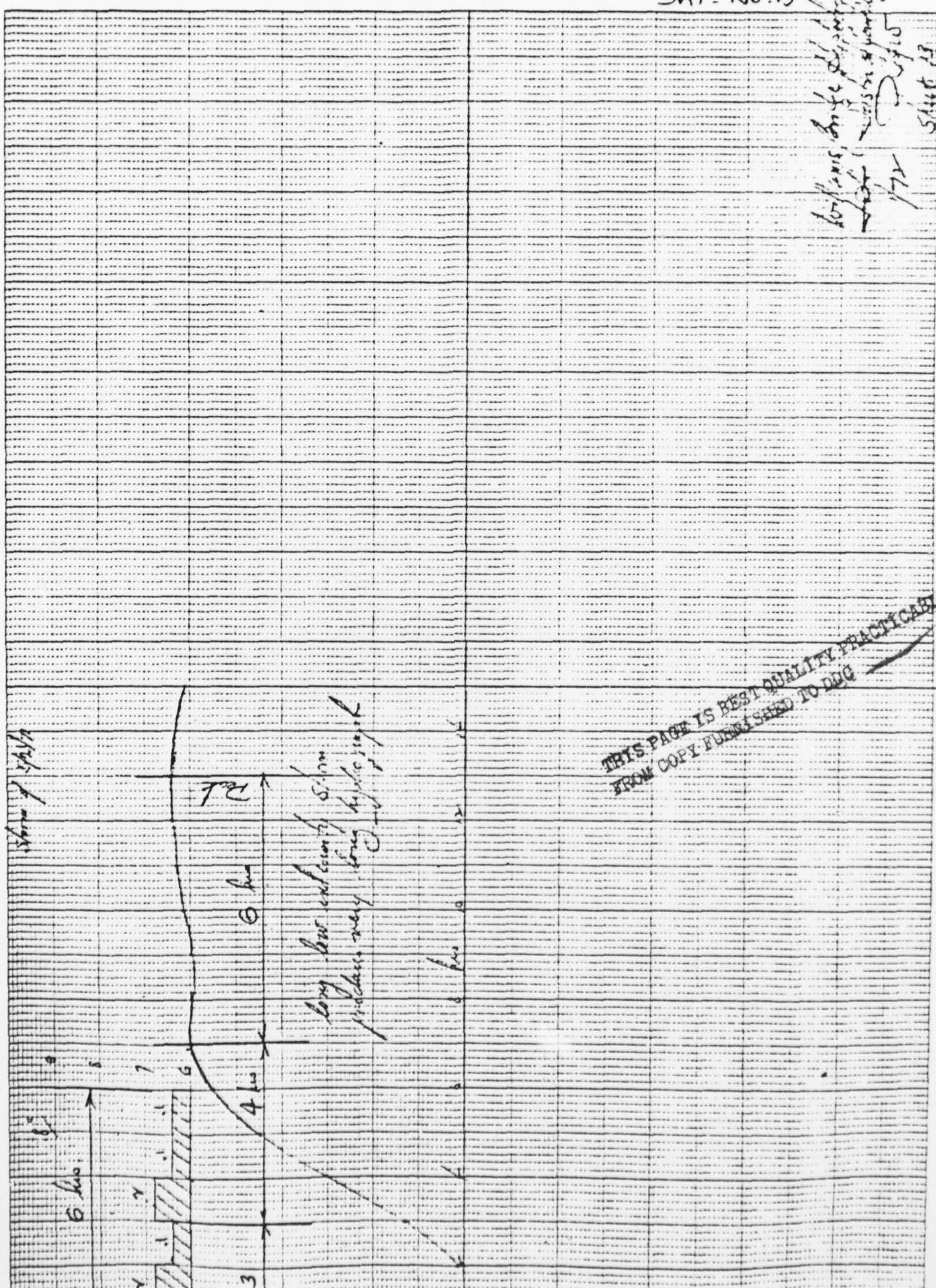

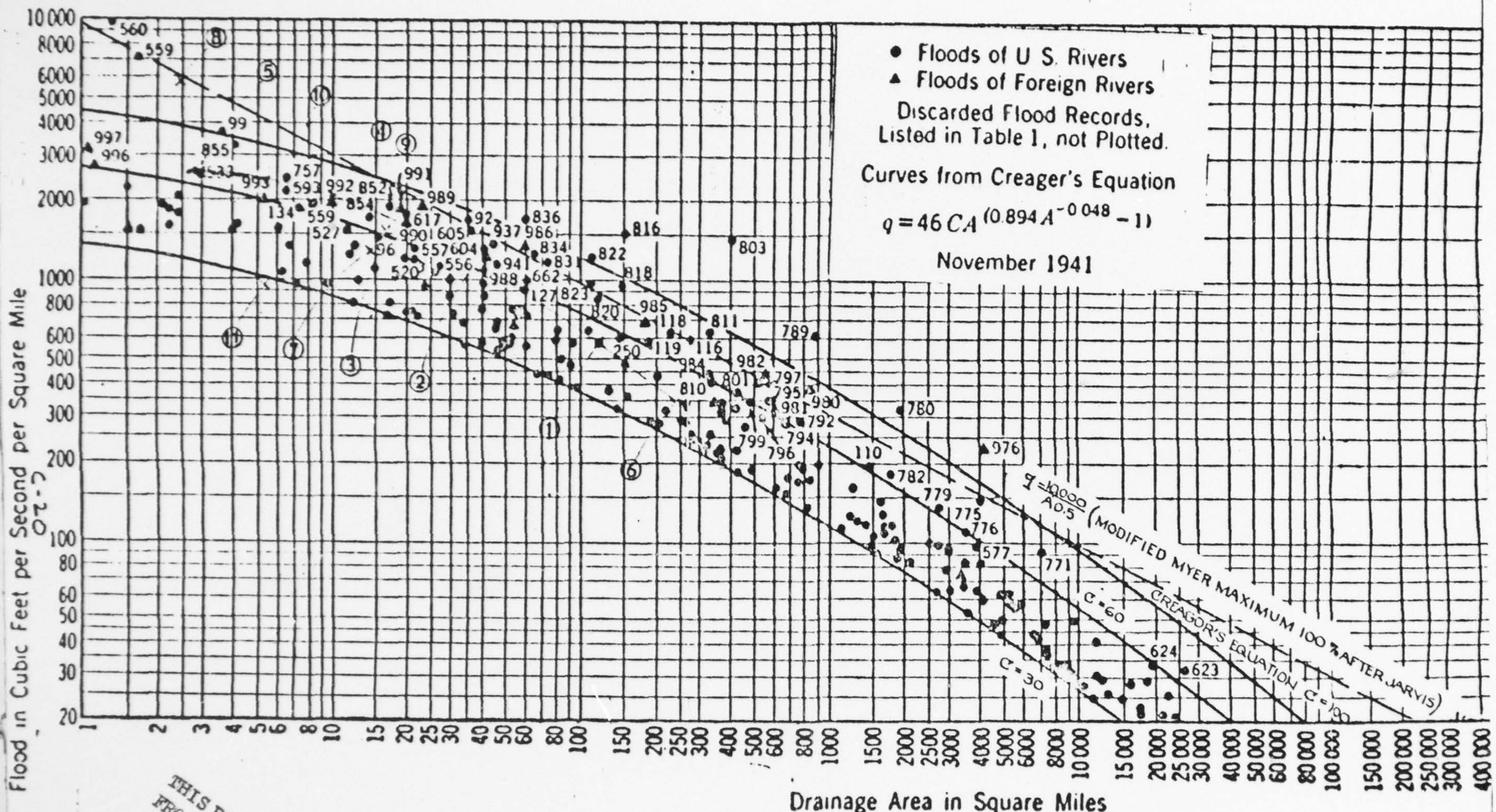


FIG. 1. Record of unusual flood discharges.

Sht. No. 14

Sht 16.15

Tabulation of Storms within an Approximate 100 mile Radius

of Freytown which have occurred in last 35 years

| | Date | Location | Drainage Area Sq. Miles | Peak Flow C.F.S. | C.F.S. per Sq. Mi. | Meyers Coeff. | Distance from Freytown |
|---|---|---|---|---|---|---|---|
| (1.) | 8/X/55 | Broadhead Cr. Analomink, Pa. | 124.0 | 72,200 | 582 | 6,400 | 21.0 |
| (2.) | 8/X/55 | E. Branch Wollenpaupak, Greentown, Pa. | 33.9 | 33,000 | 975 | 5,780 | 17.5 |
| (3.) | 9/1/40 | Salem Creek Woodstown, N.J. | 17.5 | 26,100 | 1,480 | 6,200 | 112.0 |
| (4.) | 7/X/35 | Trumansburg Cr. Trumansburg, N.Y. | 11.5 | 17,800 | 1,550 | 5,300 | 107.0 |
| (5.) | 7/X/35 | Glen Creek Townsend, N.Y. | 2.91 | 7,330 | 2,520 | 4,300 | 103.0 |
| (6.) | 7/18/42 | First Fork Sinnemahoming, Pa. | 243.0 | 80,000 | 324 | 5,130 | 135.0 |
| (7.) | 7/18/42 | Freeman Run | 14.9 | 19,000 | 1,275 | 4,920 | 135.0 |
| (8.) | 10/X/64 | P. G. & W. Crystal Lake | 2.5 | 11,500 | 5,950 | 8,000 | 21.0 |
| (9.) | 10/X/66 | P. G. & W. Hollister | 12.0 | 23,340 | 1,945 | 6,720 | 3.0 |
| (10.) | | Hazen & Sawyer Freytown | 5.5 | 13,000 | 2,360 | 5,580 | 0.0 |
| (11.) | | Chester Engineers Freytown | 5.5 | 5.230 | 951 | 2,230 | 0.0 |

AD-A063 130 GANNETT FLEMING CORDDRY AND CARPENTER INC HARRISBURG PA F/G 13/2
NATIONAL DAM INSPECTION REPORT. WILLIAMS BRIDGE DAM, PENNSYLVAN--ETC(U)
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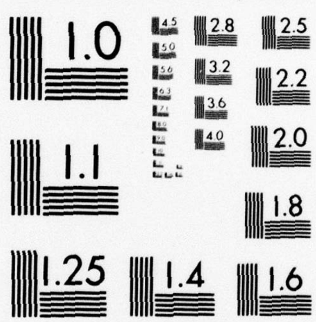

1.0
4.5
5.0
5.6
6.3
2.8
3.2
3.6
4.0
2.5
2.2
2.0
1.1
1.8
1.25
1.4
1.6

GANNETT FLEMING CORDDRY AND CARPENTER, INC.
HARRISBURG, PA.

SUBJECT ______ FILE NO. ______
______ SHEET NO. ______ OF ______ SHEETS
FOR ______
COMPUTED BY ______ DATE ______ CHECKED BY ______ DATE ______

Note:

BASED ON THE JUSTIN & COURTNEY REPORT, A DESIGN DISCHARGE OF 17,000 CFS WAS CHOSEN FOR ALTERNATIVE SPILLWAY DESIGNS STUDIED BY JUSTIN & COURTNEY IN 1972.



8319

SUSQUEHANNA RIVER BASIN

STAFFORD MEADOW BROOK, LACKAWANNA COUNTY

PENNSYLVANIA

WILLIAMS BRIDGE DAM

PENNSYLVANIA GAS AND WATER COMPANY

PHASE I INSPECTION REPORT

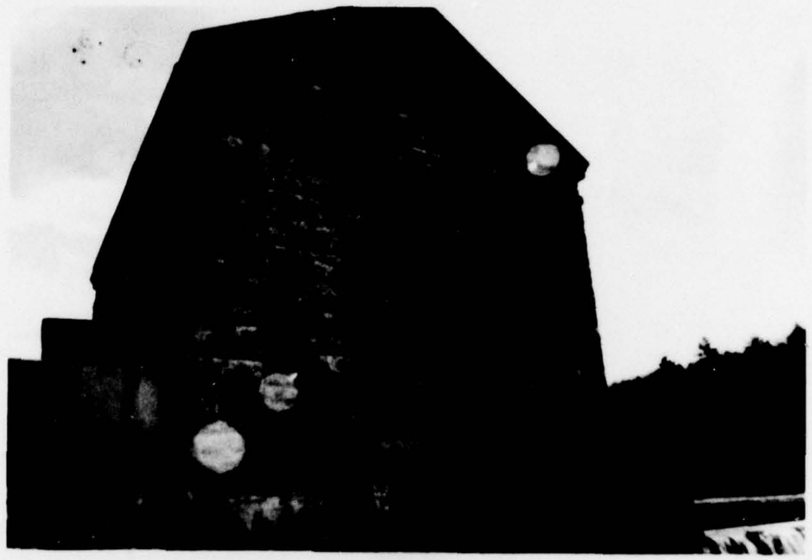
NATIONAL DAM INSPECTION PROGRAM

MAY 1978

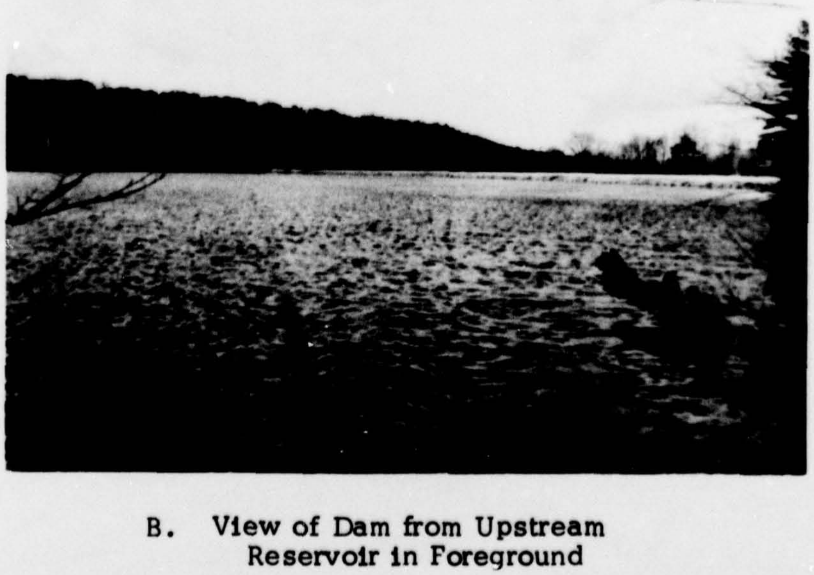
*APPENDIX D*

PHOTOGRAPHS

WILLIAMS BRIDGE DAM



A. Screen Chamber Building to Right of Main Spillway Looking Upstream



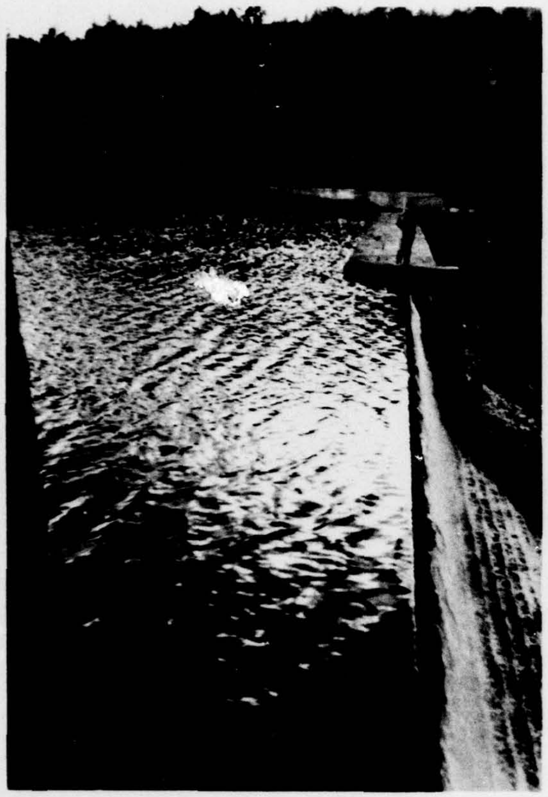
B. View of Dam from Upstream Reservoir in Foreground

WILLIAMS BRIDGE DAM

C. Left Approach Wall of Auxiliary Spillway
Showing Low Area Behind Wall

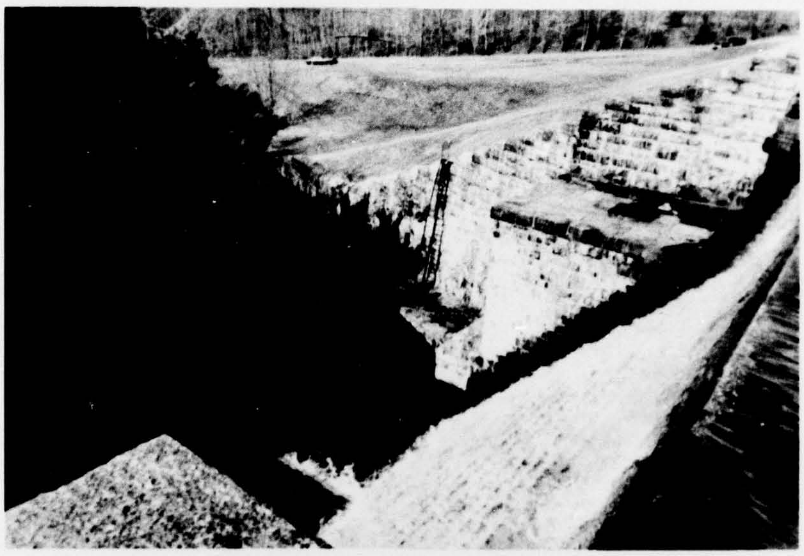
D. Concrete Extension of Masonry Core Wall
Extending Above Top of Embankment

WILLIAMS BRIDGE DAM



E. Main Spillway and Auxiliary Spillway

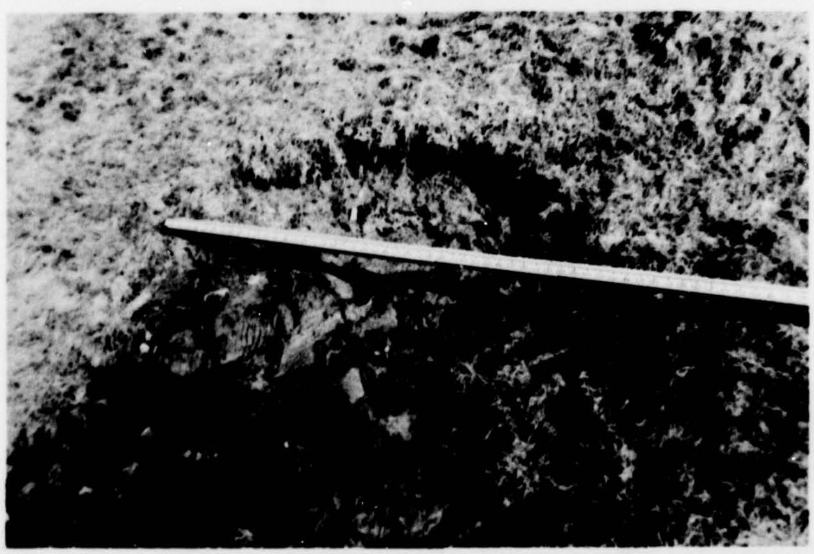
F. Main Spillway and Downstream Training Wall of Auxiliary Spillway



G. Valve House to Right of Main Spillway and with Retaining Wall in Background

H. Disintegrated Concrete at Downstream Face of Auxiliary Spillway

I. Masonry Retaining Wall Between
Embankment and Outlet Channel of Main Spillway
Showing Repaired Diagonal Crack and Disintegration of
Concrete Caps - Wet Area Shown Behind Retaining Wall



J. Wet Area
Adjacent to Main Spillway Retaining Wall

K. 1916 Piezometer and Groundhog Burrowing on Downstream Embankment Surface

L. Stream, Bridge, and Caretaker's Home below Dam Embankment with 1916 Piezometer in Foreground

SUSQUEHANNA RIVER BASIN

STAFFORD MEADOW BROOK, LACKAWANNA COUNTY

PENNSYLVANIA

WILLIAMS BRIDGE DAM

PENNSYLVANIA GAS AND WATER COMPANY

PHASE I INSPECTION REPORT

NATIONAL DAM INSPECTION PROGRAM

MAY 1978

# APPENDIX E

# GEOLOGY

WILLIAMS BRIDGE DAM

APPENDIX E

GEOLOGY

1. General Geology. The damsite and reservoir are located in the northeastern portion of Lackawanna County. Lackawanna County was completely covered with ice during the last continental glaciation of Pleistocene time. The general direction of ice movement was S 35°-40° W. Glacial drift covers the entire County, except where subsequent erosion has removed it. Thick deposits of glacial outwash occur in many places along the Lackawanna River, and are 50 to 100 feet thick near Dickson, Scranton, and Moosic.

The only important structural feature in Lackawanna County is the Lackawanna Syncline, which traverses the County in a southwesterly direction. The syncline enters the County at the northeast corner as a narrow shallow trough, gradually deepens and broadens toward the southwest, and reaches its maximum development in Luzerne County. The rock formations exposed range from the post-Pottsville formations (youngest) through the Pottsville, Mauch Chunk shale, Pocono sandstone to the Damascus formation of the Catskill group (oldest). The rim rocks, the Pottsville formation and Pocono sandstone, have dips that rarely exceed 10° to 20° and form a rather simple syncline. The core rocks, the post-Pottsville formations, are folded into a series of minor anticlines and synclines which trend about N 70° E. The rocks in the northwestern and southeastern parts of the County, outside of the limits of the Lackawanna Syncline, are generally horizontally stratified.

The Lackawanna River, in general, follows the axis of the Lackawanna Syncline. Southeast of the Lackawanna River, the rise in terrain is quite gradual and the crests of the high mountains are several miles from the Lackawanna River. Streams, such as Roaring Brook and Stafford Meadow Brook, have cut deep canyons through the mountains and follow a tortuous course to their confluence with the Lackawanna River near Scranton, Pennsylvania. In the area of interest, the Lackawanna River streambed is founded in post-Pottsville formations. Proceeding uphill from the river, the older Pottsville formation, Mauch Chunk shale, Pocono sandstone, and Catskill continental group are encountered in turn. The tributary streams, in flowing down the mountains, have generally cut through or around the hard sandstone and conglomerate members, and have eroded their streambed into the softer shales and glacial till. The Catskill continental group of rocks underlies the greater part of Lackawanna County.

2. Site Geology. Except for the geologic formations involved, the foundation conditions for Williams Bridge Dam afforded by Stafford Meadow Brook are characteristic of numerous other streams in this

section of the State. The stream has cut through an outcrop of massive gray Pocono sandstone and conglomerate and, at the damsite, is flowing parallel to the interface of Pocono sandstone and Mauch Chunk shale formations. The left abutment of the dam is founded on and keyed into the steep sandstone and conglomerate. The right abutment, as described in the 1914 reports of the Pennsylvania Water Supply Commission, is founded on hardpan that had been covered with loam and boulders. This was probably decomposed Mauch Chunk shale and/or glacial till. The spillways and outlet facilities are founded on rock. The main portion of the embankment is founded on hardpan.