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NATIONAL DAM SAFETY PROGRAM. EPHRATAH DAM (INVENTORY NUMBER NY --ETC(U)
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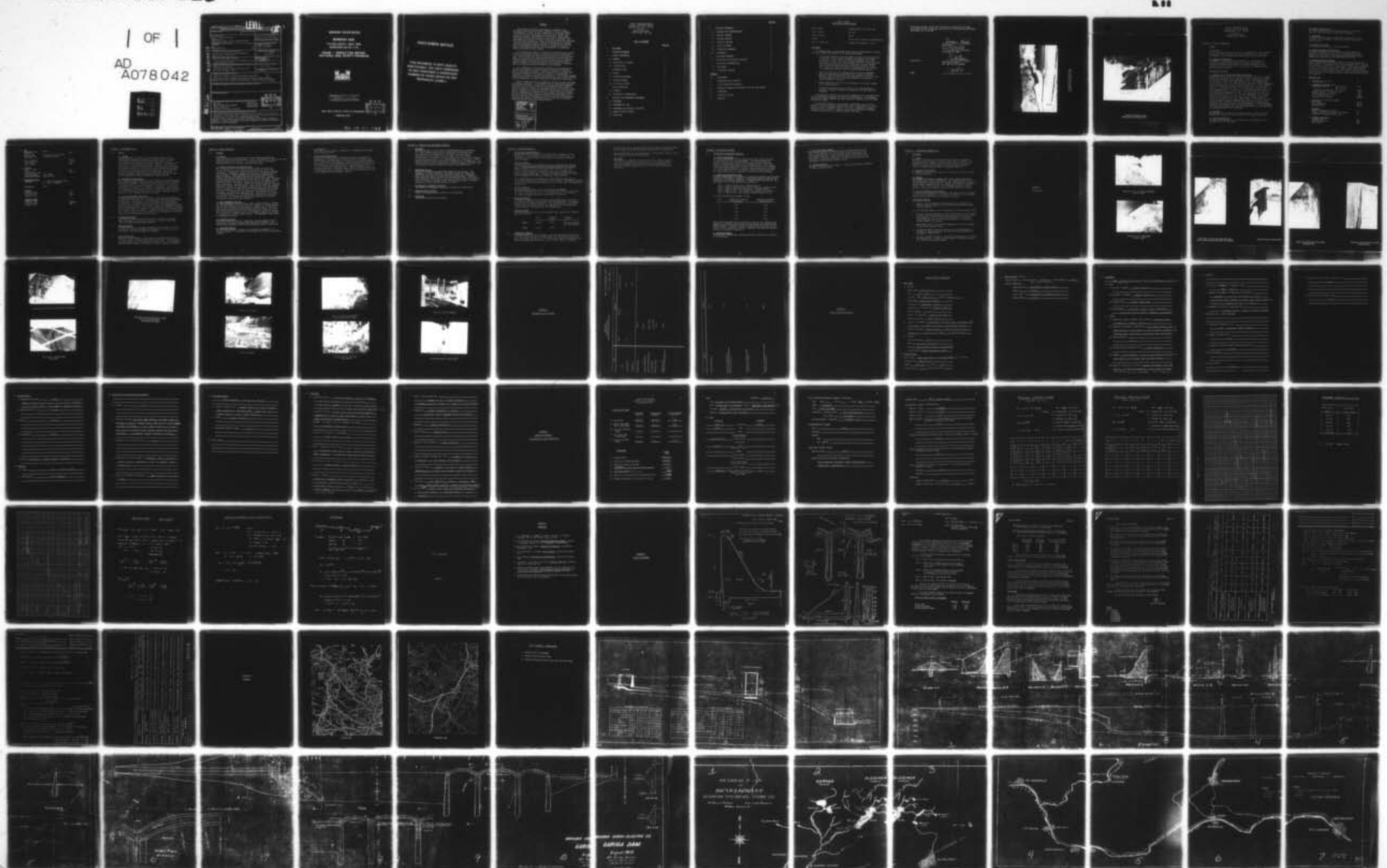

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Ephratah Dam
Fulton County
Caroga Creek

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

This report provides information and analysis on the physical condition of the dam as of the report date. Information and analysis are based on visual inspection of the dam by the performing organization.
Ephratah Dam was found to have several deficiencies which require immediate attention. Further investigation concerning seepage at the dam, and stability analysis was also recommended.

MOHAWK RIVER BASIN

EPHRATAH DAM

FULTON COUNTY, NEW YORK
INVENTORY NO. N.Y. 178

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

APPROVED FOR PUBLIC RELEASE;
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NEW YORK DISTRICT CORPS OF ENGINEERS

FEBRUARY, 1979

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PREFACE

This report is prepared under guidance contained in the Recommended Guidelines for Safety Inspection of Dams, for Phase I Investigations. Copies of these guidelines may be obtained from the Office of Chief of Engineers, Washington, D.C. 20314. The purpose of a Phase I Investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general condition of the dam is based upon available data and visual inspections. Detailed investigation, and analyses involving topographic mapping, subsurface investigations, testing, and detailed computational evaluations are beyond the scope of a Phase I Investigation; however, the investigation is intended to identify any need for such studies.

In reviewing this report, it should be realized that the reported condition of the dam is based on observations of field conditions at the time of inspection along with data available to the inspection team. In cases where the reservoir was lowered or drained prior to inspection, such action, while improving the stability and safety of the dam, removes the normal load on the structure and may obscure certain conditions which might otherwise be detectable if inspected under the normal operating environment of the structure.

It is important to note that the condition of a dam depends on numerous and constantly changing internal and external conditions, and is evolutionary in nature. It would be incorrect to assume that the present condition of the dam will continue to represent the condition of the dam at some point in the future. Only through frequent inspections can unsafe conditions be detected and only through continued care and maintenance can these conditions be prevented or corrected.

Phase I inspections are not intended to provide detailed hydrologic and hydraulic analyses. In accordance with the established Guidelines, the Spillway Test flood is based on the estimated "Probably Maximum Flood" for the region (greatest reasonably possible storm runoff), or fractions thereof. Because of the magnitude and rarity of such a storm event, a finding that a spillway will not pass the test flood should not be interpreted as necessarily posing a highly inadequate condition. The test flood provides a measure of relative spillway capacity and serves as an aide in determining the need for more detailed hydrologic and hydraulic studies, considering the size of the dam, its general condition and the downstream damage potential.

PHASE 1 INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM
EPHRATAH DAM I.D. No. NY 178
DEC #456
MOHAWK RIVER BASIN
FULTON COUNTY, NEW YORK

TABLE OF CONTENTS

PHASE 1 REPORT
NATIONAL DAM SAFETY PROGRAM

Name of Dam:	Ephratah Dam (I.D. No. NY 178)
State Located:	New York
County Located:	Fulton
Stream:	Caroga Creek (tributary of Mohawk River)
Dates of Inspection:	October 16 & November 3, 1978

ASSESSMENT

The Ephratah Dam is a buttressed concrete and cyclopean masonry structure. The visual inspection revealed the following deficiencies:

1. The concrete portions of the dam exhibit signs of major deterioration as indicated by the extensive seepage which appears on the downstream face. Gunite and epoxy injection treatments have failed to control the flow. Cold joints formed during the concrete pouring operations are believed to be the primary path for this seepage. This deterioration may diminish the structural integrity of the dam. Therefore, seepage and stability investigations are required to determine the type and extent of remedial measures needed.

2. Repair the erosion on the upstream face of the earth embankment near the north spillway buttress and place erosion protection material. Monitor periodically the void observed at the toe of the north spillway buttress on the downstream face to determine if further movement is occurring.

3. Remove debris and silt which has accumulated in the outlet channel of the reservoir drain.

4. Periodically monitor the steep slopes of the earth embankment to determine if remedial measures are required to prevent erosion or sloughing.

An engineering study should be initiated immediately and completed within 1 year concerning the seepage and stability investigations. The remaining deficiencies require remedial work which should be completed within the next construction season.

The discharge capacity of the spillway is inadequate for all flow in excess of 17% of the Probable Maximum Flood (PMF). The spillway is not considered seriously inadequate, based on the Corps of Engineer's Screening Criteria, since the dam is a gravity structure and the stability analyses indicate that the dam is not unstable during a PMF event.

Additional spillway capacity may be achieved by permitting flow over the arched sections of the dam, in which case the spillway capacity would approximate 44% of the PMF.

George Koch

George Koch
Chief, Dam Safety Section
New York State Department
of Environmental Conservation
NY License No. 45937

Approved By: Clark H. Benn

Col. Clark H. Benn
New York District Engineer

Date: 17 April 79

Overview of Ephratah Dam
Upstream Face Looking North

Overview of Ephratah Dam
Downstream Face Looking North

PHASE 1 INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM
EPHRATAH DAM I.D. No. NY 178
DEC #456
MOHAWK RIVER BASIN
FULTON COUNTY, NEW YORK

SECTION 1: PROJECT INFORMATION

1.1 GENERAL

a. Authority
The Phase 1 Inspection reported herein was authorized by the Department of the Army, New York District, Corps of Engineers, to fulfill the requirements of the National Dam Inspection Act, Public Law 92-367.

b. Purpose of Inspection
The purpose of this inspection and report is to investigate and evaluate the existing conditions of the subject dam in order to identify deficiencies and hazardous conditions; determine if they constitute hazards to human life or property and recommend remedial measures where necessary.

1.2 DESCRIPTION OF PROJECT

a. Description of Dam and Appurtenant Structures
The Ephratah Dam is a 760 feet long, buttressed concrete and cyclopean masonry dam containing an ogee spillway, 3 arched sections, a concrete gravity section and an earth embankment section. The maximum height of the dam is 65 feet. The ogee spillway is 251 feet long and has a crest elevation of 974.15. The three buttressed arch sections immediately south of the spillway are each 60 feet wide and have a crest elevation of 977.15. The north buttress forms the north edge of the spillway and has a top elevation of 979.15, 5 feet above the spillway crest. An earth embankment with a concrete core wall extends northward from the north buttress a distance of 245 feet. The top of the core wall is at elevation 978 ± and the top of the earth embankment is at elevation 979±. The top width of the core wall is 2 feet and is sloped with a batter of 1/2 inch per foot. A 3 feet diameter reservoir drain is located at the base of the middle arched section. The flow of the drain is controlled by a manually operated butterfly valve located in the building at the toe of the dam. The power generation intake system is located on the upstream face of the south abutment. A 6.5 feet diameter wood stave pipe carries water from the intake 2 miles to the power plant. All regulating outlets are operational.

b. Location
The Ephratah dam is located on the Caroga Creek, a tributary of the Mohawk River, at about 3/4 miles south of the Village of Ephratah.

c. Size Classification
The dam is 65 feet high and is classified as an "intermediate" dam (between 40 and 100 feet high).

d. Hazard Classification
The dam is classified as high hazard because of its location, about 3/4 miles upstream of the Village of Ephratah.

e. Ownership
The dam is owned by Niagara Mohawk Power Corporation, 300 Erie Boulevard West, Building D-2, Syracuse, New York, 13202, Tel: (315) 474-1511.

f. Purpose of the Dam
The dam provides storage for power development.

g. Design and Construction History
The dam was designed by Barclay Parsons & Clapp, Consulting Engineers, 60 Wall Street, New York for Mohawk Hydro-Electric Company, 60 Wall Street, New York and was constructed by Empire Engineering Corporation in the summer and fall of 1910.

h. Normal Operating Procedure
Water releases from the Ephratah Reservoir are passed through the intake located in the gate house on the southeastern side of the dam. Flow regulation is provided by electrically operated inlet regulating valves at the entrance to the inlet chamber. Water passes through a screen to the inlet chamber then to the penstock and through the surge tank to the power house. Quantity of flow through the penstock, used for generation of hydroelectricity depends on the water level in the reservoir.

1.3 PERTINENT DATA

a. Drainage Area (sq. mi)		52
b. Discharge at Dam Site (cfs)		
Maximum known Flood:	March 18, 1936 (El. 975.6)	1,870
Spillway at Design Pool	(El. 977.15)	4,280
Maximum Capacity of Reservoir Drain		260
Maximum Capacity of Penstock		320
Total Discharge, Max. Pool		4,860
Average Daily Discharge		Unknown
c. Elevation (ft. above MSL-Datum)		
Top of Dam		979.15
Max. Design Pool		977.15
Spillway Crest		974.15
d. Reservoir		
Length of maximum pool, miles		0.8
Length of Shoreline (Spillway Crest) miles		1.5
Surface area (Spillway Crest) acres		30
e. Storage, (Acre-feet)		
Spillway Crest		600
Maximum Design Pool		690
Top of Dam		750

f. Dam

Embankment Type:	Earth	
Length (ft.)		330
Buttress Dam	3 arches @ 60 ft. =180 ft.	
Impervions Core	Concrete Core Wall	
Crest Elevation		979.15
Crest Width, ft.		12.00
Grout curtain		None

g. Spillway

Type: Ogee		
Length, ft.		251
Crest Elevation MSL		974.15
Upstream Channel	Not visible	
Downstream Channel	Slate bottom	

h. Regulating Outlet

Upstream -	1. Inlet of reservoir drain 2. Inlet of penstock	
Downstream -	None	

Penstock

Diameter (feet)	6.5
Elevation, MSL	949.0
Capacity (cfs)	320

Reservoir Drain

Diameter (feet)	3
Elevation, MSL	Unknown
Capacity (cfs)	260

SECTION 2: ENGINEERING DATA

2.1 DESIGN

a. Geology

The Ephratah Dam is located in the northwestern portion of the "Hudson-Mohawk Lowlands" physiographic province of New York State. The province resulted from erosion along outcrop belts of weak rocks between the Adirondack and Catskill Mountains. Generally, the province is of low elevation and relief. Bedrock in the vicinity of the dam is primarily Ordovician (500-435 million years ago) shales and sandstones which have been exposed by the southward and westward stripping off of Silurian and Devonian limestones. The present surficial soil deposits have resulted from glaciations during the Cenozoic Era (most recent 65 million year period), the last of which was the Wisconsin ice sheet approximately 11,000 years ago.

b. Subsurface Investigations

The "General Soil Map of New York State" prepared by Cornell University Agriculture Experiment Station indicates that the soil in the vicinity of the dam is Charlton. Charlton soils are of glacial till origin and residuum from chist, gneiss, and granite. This soil is a stony silt and sand with a trace of clay. Boulders are numerous. Rock outcrops occur frequently. However, the depth to bedrock is extremely variable. The overall drainage is good with a rate of runoff dependent upon the degree of slope. Internal drainage is moderate.

c. Dam and Appurtenant Structures

The dam was designed by Barclay, Parsons & Klapp Consulting Engineers, 60.Wall Street, New York, NY. All drawings available have been included in Appendix F. The design of this dam includes a cyclopean masonry spillway which is buttressed at both ends, three reinforced concrete buttressed arch sections a gravity dam section, and an embankment section with a concrete core wall. All concrete elements are founded on and keyed into the shale bedrock or hardpan. A reservoir drain is located at the toe of the center arch. An intake is located at the south end of the upstream face. Water is drawn-off from the reservoir and transmitted to the power house via a 78 inch diameter wood stave pipe.

2.2 CONSTRUCTION RECORDS

No information regarding the construction of the dam was available other than the year of construction and the contractor, that being 1910-11 and Empire Engineering Corporation.

2.3 OPERATION RECORD

The reservoir level is recorded continuously at the intake. Any other information concerning discharges and maintenance is on file at the power house. No operating manual is available.

2.4 EVALUATION OF DATA

Some of the data presented in this report has been made available by Mr. Robert Levett of Niagara Mohawk Power Corporation. This information has been invaluable in the preparation of this report. All information gathered appears adequate and reliable for Phase 1 Inspection purposes.

SECTION 3: VISUAL INSPECTION

3.1 FINDINGS

a. General

Visual inspection of Ephratah Dam and the surrounding watershed was conducted on October 16 and November 3, 1978. The weather was clear and the temperature ranged in the thirties. The reservoir level at the time of inspection was 973.0 or 1.15 feet below spillway crest.

b. Concrete Structure including Spillway

The concrete portion of the dam was gunited between 1969 and 1971 on the upstream face of the arched sections and on the entire downstream face. Considerable cracking and spalling of the gunite surfaces were observed. Extensive seepage was also evident coming through and under the gunite. Some downstream face drains placed through the gunite to relieve the seepage pressure were flowing full. Cracking of structural concrete was observed at the crest of the arches. Cracking extended completely through the top of the dam, but the gunite covering masked the depth of the crack. Cracks were previously repaired and have cracked again. Photographs taken by Mr. Levett during the reservoir drawdown in October 1978 were viewed. They indicated that the upstream face of the spillway is experiencing separation along horizontal planes which are assumed to be the location of cold joints during the pouring operations. The separations are considered the potential source of the seepage observed on the downstream face. The structure is founded on shale bedrock which also provides energy dissipation at the toe of the spillway. The seepage and cracking noted above may be adversely affecting the stability of the structure.

c. Earth Embankment Sections

The earth embankment portions show no signs of major distress. However, the following problem areas were observed: Erosion of the upstream face near the north buttress of the spillway is probably due to wave action and the lack of energy dissipating material such as riprap. The void and seepage observed near the toe of the spillway at the north buttress may be related to seepage along the earth-concrete interface or related to seepage from the jointed shale bedrock. The embankment slopes appear to be excessively steep.

d. Regulating Outlets

The 36 inch reservoir drain is operational, but was unobserved due to siltation of the outlet channel during the previous drawdown. The hydro-electric generation intake and transmission system is operational.

e. Downstream Channel

The downstream channel is primarily shale bedrock and appears to be in good condition. The reservoir drain channel has silted-in and requires cleaning.

f. Reservoir

There are no visible signs of instability or sedimentation problems in the reservoir area.

3.2 EVALUATION OF OBSERVATIONS

Serious deficiencies were observed which should be investigated immediately to determine what corrective action is required, these are the seepage and development of structural concrete deterioration, and erosion of the earth embankment slopes near the north buttress of the spillway near the reservoir level on the upstream face and at the toe of the dam on the downstream face. These deficiencies do not represent conditions of imminent danger, however, remedial action must be undertaken as soon as possible. The minor deficiencies may be corrected or monitored by maintenance forces.

SECTION 4: OPERATION AND MAINTENANCE PROCEDURE

4.1 PROCEDURE

The Ephratah Dam is a power dam for Niagara Mohawk Power Corporation. A 6.5 feet diameter wood stave pipe (penstock) carries water from the reservoir to the power plant. The penstock is connected to a surge tank before it reaches the power plant. The flow through the penstock is controlled by a valve connected to an electrically operated control mechanism located on the south-eastern side of the dam. A 3 feet diameter reservoir drain located below the middle arch is operational. The flow through the drain is controlled by a manually operated butterfly valve, located in the control building at the base of the middle arch.

4.2 MAINTENANCE OF DAM

There is no operation and maintenance manual for the project. The embankment is in good condition. The ogee spillway was treated with gunite and face drains were installed about 8 years ago. The gunite deteriorated and spalled in many places exposing wire mesh. There are voids underneath the spillway toe and near the base of the north buttress. There are cracks in the three arches and some calcification is evident. Seepage was observed through spillway and arches.

4.3 MAINTENANCE OF OPERATING FACILITIES

The valves in the reservoir drain and in penstock are operational.

4.4 WARNING SYSTEM IN EFFECT

There is no warning system in effect or in preparation.

4.5 EVALUATION

The spillway and arches need repairs.

SECTION 5: HYDRAULIC/HYDROLOGIC

5.1 DRAINAGE AREA CHARACTERISTICS

The Ephratah Dam is located on the Caroga Creek, a tributary of the Mohawk River. The drainage area at dam site is 52 square miles. The topography is characterized by steep slopes interspersed by numerous lakes and swamps.

5.2 ANALYSIS CRITERIA

Information on the PMF for Ephratah Dam and its watershed was obtained from the UPPER HUDSON AND MOHAWK RIVER BASINS HYDROLOGIC FLOOD ROUTING MODELS prepared in 1976 for the New York District of the U.S. Army Corps of Engineers by Resource Analysis, Inc. In this study, the rainfall-runoff mathematical model HEC-1 was used to reconstitute the major historical floods and to simulate the Standard Project Flood (SPF). Probable Maximum Flood (PMF) was considered as twice the SPF.

The Ephratah Dam and its watershed are located within the sub-area 16 of the Mohawk River Basin, Little Falls, N.Y. to Mouth. The computed outflow resulting from one half PMF and PMF are 12,500 cfs and 25,000 cfs respectively (Appendix D).

5.3 SPILLWAY CAPACITY

The ungated ogee spillway is 251 feet long and the maximum head possible between the crest of the spillway and the top of the arches is 3 feet. The top of the dam is 2 feet above the top of the arches. The computed maximum capacity of the spillway is 4280 cfs.

5.4 RESERVOIR CAPACITY

The reservoir capacities at the crest of the spillway and at the top of the arches are 600 acre-feet and 690 acre-feet respectively. The storage capacity curve is shown in Appendix D. The curve indicates a surcharge storage above the spillway crest of 91 acre-feet which is equivalent to a runoff depth of .033 inches over the drainage area.

5.5 FLOODS OF RECORD

The highest and lowest water levels recorded since completion of Ephratah Dam are as follows:

	Date	Elevation (feet)	Discharge (cfs)
Highest	3/18/36	975.6	1550 over spillway 320 thru penstock
Lowest	10/5/78	923.0	0

5.6 OVERTOPPING POTENTIAL

The maximum capacities of the spillway, penstock and reservoir drain are 4,280 cfs, 320 cfs and 260 cfs respectively. The PMF outflow being 25,000 cfs, the spillway can pass only 17% of PMF and the spillway coupled with the penstock and reservoir drain can pass 19% of PMF. Again, the spillway alone can pass 34% of one half PMF (12,500 cfs) and the spillway coupled with the penstock and low-level drain can pass 39% of one half PMF.

The dam, therefore, will be overtopped by 27 inches and 4 inches of water due to PMF and one half PMF respectively.

5.7 EVALUATION

The spillway is inadequate to pass one half PMF. However, based on the Corps of Engineer's Screening Criteria, it is not considered seriously inadequate, since the stability analyses conducted (see section 6) achieved adequate factors of safety during overtopping (PMF event).

SECTION 6: STRUCTURAL STABILITY

6.1 EVALUATION OF STRUCTURAL STABILITY

a. Visual Observations

The visual observations did not indicate any signs of major distress in connection with the earth embankment. The concrete structure, however, exhibits signs of major deterioration as indicated by the excessive seepage which appears on the downstream face. Gunite and epoxy injection treatments have failed to control this flow of seepage. Cold joints formed during the concrete pouring operations are believed to be the primary path for this seepage. Extensive engineering study and rehabilitation will be required to control the flow of seepage.

b. Design and Construction Data

No design computations or construciton information regarding the structural stability of the dam are available. A structural stability analysis was conducted by Mr. Ralph J. DeStefano of Niagara Mohawk dated July 16, 1971. The results of this investigation are as follows:

Case 1 - Water at normal pool: Elevation 974.15
Case 2 - Water at 100 year storm level (3,000 cfs): Elevation 976.2
Case 3 - Water at PMF level (46,000 cfs): Elevation 984.6
Case 4 - Same as Case 1 and includes ice pressure (5,000 lb/lf)
Case 5 - Same as Case 1 and includes Seismic analysis (Zone 2)

All cases assumed uplift pressure at 100%.

Case	Safety Factor Against Overturning	Base Shear (Sliding) Safety Factor
1	2.26	5.80
2	2.05	5.48
3	1.46	4.38
4	1.99	5.62
5	1.59	5.36

These results indicate that the structure is safe for all loading cases. However, the potential failure planes located along cold joints have not been evaluated. A stability analysis including this factor must be conducted as soon as possible, Further information concerning the stability analysis is included in Appendix F.

c. Operating Records

No operational problems were reported which would influence the stability of the structure.

d. Post-Construction Changes

The concrete portion of the dam was gunited between 1969 and 1971, first on the upstream face of the arched sections and then on the entire downstream face. Epoxy injection of the southern most arch was started in the fall of 1976 and the spring of 1977, but discontinued due to budgetary conditions. The injection attempted to bond the gunite to the structure without success.

e. Seismic Stability

The dam is located in Seismic Zone 2. A seismic analysis was conducted in Case 5 described above.

SECTION 7: ASSESSMENT/RECOMMENDATIONS

7.1 ASSESSMENT

a. Safety

The Phase 1 Inspection of Ephratah Dam did not indicate conditions which constitute an immediate hazard to human life or property. The embankment is not considered to be unstable. However, the concrete portion of the dam requires immediate investigation and remedial action to prevent the development of hazardous conditions.

b. Adequacy of Information

Information reviewed for the purposes of the Phase 1 Inspection report is considered adequate.

c. Urgency

The seepage and stability investigations of the concrete portions of the dam should be initiated immediately, and completed within 1 year from motification. Upon completion of these investigations, construction should commence immediately and the remedial work should be completed within 2 years of notification. The recommended measures listed in section 7.2 should be completed within the next construction season.

d. Need for Additional Investigation

To prevent the development of potentially hazardous conditions, investigations should be undertaken to determine the influence of the observed seepage upon the stability of the concrete structure.

7.2 RECOMMENDED MEASURES

a. Results of the aforementioned investigations will determine the remedial measures required for the observed seepage and its affect upon the stability.

The following improvements can be accomplished by the maintenance forces:

b. Erosion of the upstream face near the north buttress of the spillway requires repair and placement of erosion protection material. The void observed at the toe of the north buttress of the spillway (downstream face) must be periodically monitored to determine if further movement is occurring.

c. Remove debris and silt which has accumulated in the outlet channel of the 36 inch reservoir drain.

d. Periodically monitor the steep slopes of the earth embankment to determine if remedial measures are required to prevent erosion or sloughing of these slopes.

e. Initiate a program of periodic inspection and maintenance of the dam and appurtenances. Document this information for future reference. Also, develop an operations manual.

APPENDIX A

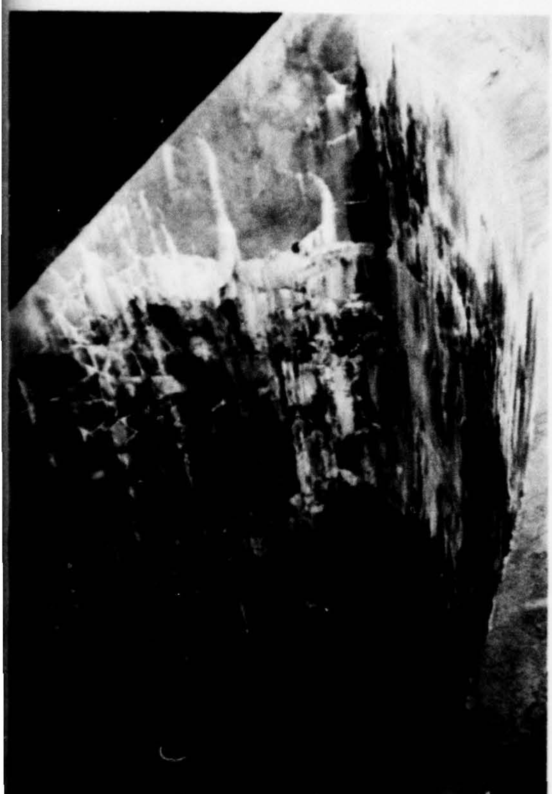
PHOTOGRAPHS

Downstream Face of Dam and Embankment
Looking South

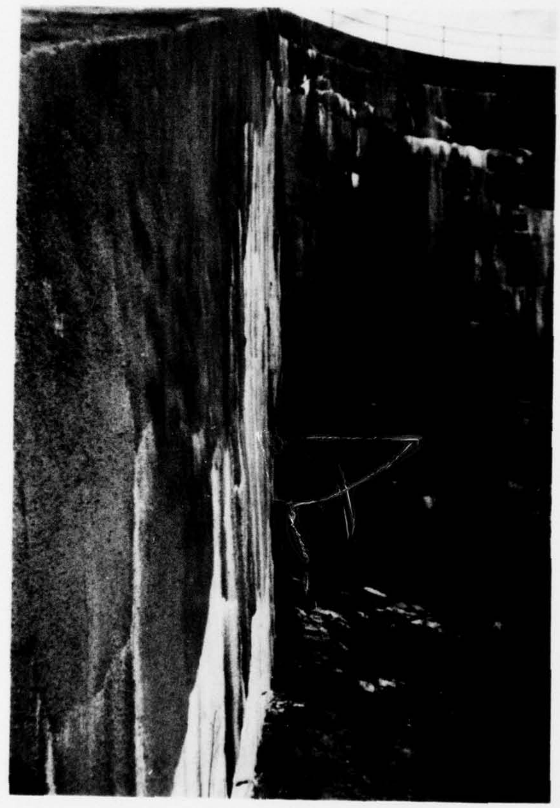
Upstream Face of Embankment
Looking South

North Wall of Third Arch (Northern most)
note cracked and spalled gunite and seepage

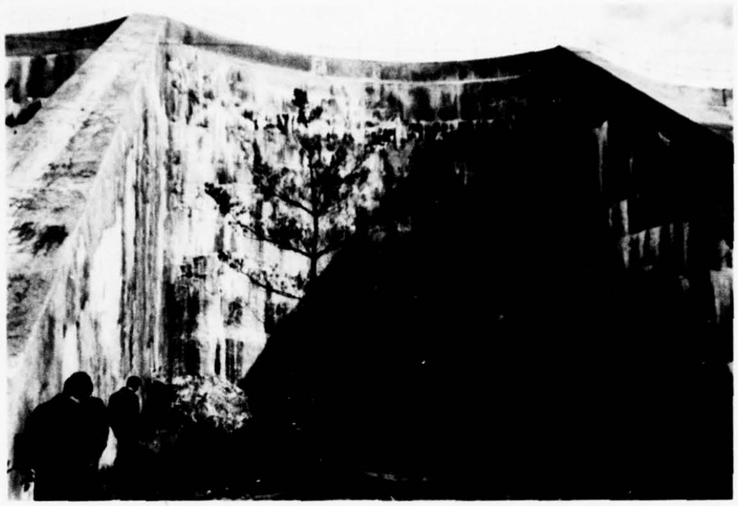
Spillway Crest Looking North



First Arch Viewed from Top of Dam
Looking North



Third Arch Viewed from Top of Dam
Looking North



Second Arch and low-level Outlet Control Building

Top of Dam - Arched Section
Note Crack

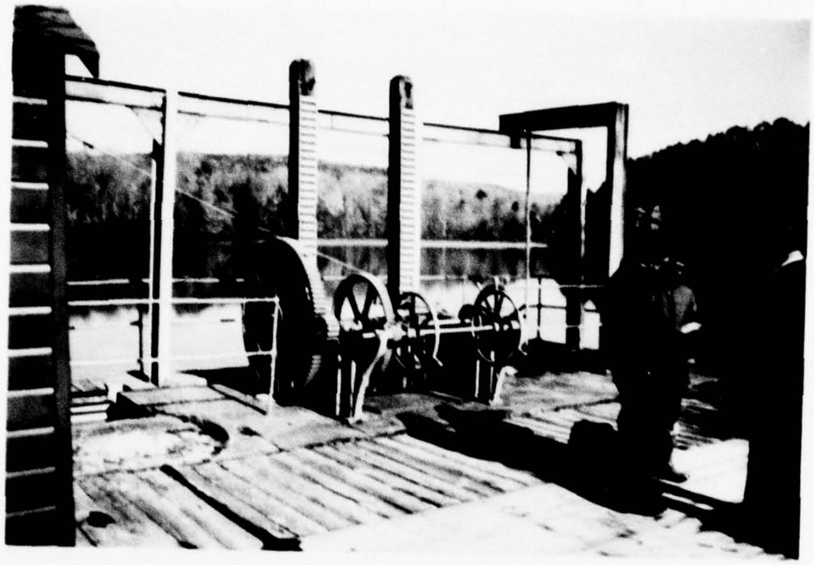
Spillway Face Near North Wall of Arch
Note Deteriorated Gunite
and exposed wire mesh

Spillway Looking North

Base of Spillway

Base of Spillway Looking North
Note Void in Circled Area

Close-up View of Void Area
Circled Above



Penstock Control Mechanism

Downstream Channel Looking West

APPENDIX B

ENGINEERING DATA CHECKLIST

Name of Dam EPHRATAH DAM

I.D. # NY 178

Check List
Engineering Data
Design Construction Operation

Item	Remarks		
	Plans	Details	Typical Sections
Dam			
Spillway(s)	YES	-	YES
Outlet(s)	-	-	YES
Design Reports	NONE		
Design Computations	NONE		
Discharge Rating Curves	INCLUDED		
Dam Stability	INCLUDED		
Seepage Studies	NONE		
Subsurface and Materials Investigations	NONE		

Item	Remarks
Construction History	NONE
Surveys, Modifications, Post-Construction Engineering Studies and Reports	NONE
Accidents or Failure of Dam Description, Reports	NO
Operation and Maintenance Records Operation Manual	NONE

APPENDIX C

VISUAL INSPECTION CHECKLIST

VISUAL INSPECTION CHECKLIST

1) Basic Data

a. General

Name of Dam Ephratah

I.D. # NY 178

Location: Town Ephratah County Fulton

Stream Name Caroga Creek

Tributary of Mohawk River

Longitude (W), Latitude (N) 74°31'4", 43°2'2"

Hazard Category C

Date(s) of Inspection October 16, 1978

Weather Conditions 30's Sunny

b. Inspection Personnel R. McCarthy, R. Warrender M. Islam (DEC)

R. Levett L. Pratt, J. Pichard, D. Guenga (Ni.Mo.)

c. Persons Contacted Robert Levett Niagara Mohawk Power Corporation Syracuse NY 13202 Tel: (315) 474-1511

d. History:

Date Constructed 1910-11

Owner Niagara Mohawk

Designer Barclay, Parsons & Klapp 60 Wall St. NYC

Constructed by Empire Engineering Corp.

2) Technical Data

Type of Dam Concrete buttress with arches (3) & earth abutments

Drainage Area 52 square miles

Height 65 feet Length 760

Upstream Slope — Downstream Slope —

2) Technical Data (Cont'd.)

External Drains: on Downstream Face NONE other @ Downstream Toe NONE

than small gunite face drains

Internal Components:

Impervious Core CONCRETE CORE WALL

Drains NONE

Cutoff Type CONCRETE CUTOFF WALL

Grout Curtain NONE

3) Embankment

280' Earth Embankment north of spillway with concrete core wall

a. Crest

(1) Vertical Alignment good condition

(2) Horizontal Alignment good condition

(3) Surface Cracks none observed

(4) Miscellaneous gunite work has disturbed a portion of the crest (dumping of concrete)

b. Slopes

(1) Undesirable Growth or Debris, Animal Burrows numerous trees and brush at abutments and toe

(2) Sloughing, Subsidence or Depressions very steep slopes, void and some erosion near base of north buttress (downstream face) upstream face - some erosion at waterline near north buttress

(3) Slope Protection none

(4) Surface Cracks or Movement at Toe None observed

(5) Seepage some seepage in area of void (downstream - n. buttress) and from rock outcrop west of north buttress

(6) Condition Around Outlet Structure Reservoir level dropped in Oct, 1978 resulting in siltation of downstream channel below valve house for the 36" low level outlet pipe - pipe unobservable

c. Abutments

(1) Erosion at Embankment and Abutment Contact

See "3.b"

(2) Seepage along Contact of Embankment and Abutment

seepage in void at north buttress could be coming along abutment or around core wall, or thru rock but it is impossible to tell its source

(3) Seepage at toe or along downstream face

none evident except as noted in 3.b.5

d. Downstream Area - below embankment

(1) Subsidence, Depressions, etc.

none - exposed shale bedrock

(2) Seepage, unusual growth

none

(3) Evidence of surface movement beyond embankment toe

none

(4) Miscellaneous

e. Drainage System

none

(1) Condition of relief wells, drains, etc. ____________________

none

(2) Discharge from Drainage System ____________________

none

4) Instrumentation

(1) Monumentation/Surveys NONE

Reservoir level - continuous strip chart recorder

survey bench marks, rain gage at generating station

(2) Observation Wells NONE

(3) Weirs NONE

(4) Piezometers NONE

(5) Other AUTOMATIC WATER LEVEL INDICATOR.

5) Reservoir

a. Slopes O.K. where visible

b. Sedimentation NONE REPORTED.

6) Spillway(s) (including tail race channel)

a. General —

b. Principle Spillway Concrete ogee spillway 251 feet wide. The spillway was gunite treated between 1969 and 1971. Gunite ~~spalled~~ cracked and spalled in many places exposing wiremesh. Some of the face drains placed through gunite are flowing full. There is considerable seepage through the spillway.

c. Emergency or Auxiliary Spillway none

d. Condition of Tail race channel Exposed shale bedrock, some surface weathering, but in general good condition

e. Stability of Channel side/slopes good condition of side slopes

7) Downstream Channel

Rock exposed in much of the channel

a. Condition (debris, etc.) good condition - main channel poor condition - low level outlet: filled with fine sand and silt from recent drawdown

b. Slopes good condition

c. Approximate number of homes Numerous

8) Miscellaneous

9) Structural

a. Concrete Surfaces considerable spalling of gunite with seepage extensive cracking of gunite surfaces and where exposed cracking along cold joints of concrete structure (observed from photos taken @ drawdown)

b. Structural Cracking cracking at crest of buttress sections completely thru top of dam (see photo). Cracks were repaired and have cracked again

c. Movement - Horizontal & Vertical Alignment (Settlement) no problems observed

d. Junctions with Abutments or Embankments good condition visually

e. Drains - Foundation, Joint, Face downstream face drains thru gunite to relieve water pressure - observed some drains flowing full

f. Water passages, conduits, sluices good condition where observed

g. Seepage or Leakage extensive seepage thru cracks in gunite in spillway and buttress-arch sections also some calcification. difficult to tell where seepage is starting due to spalling of gunite face (see photos)

h. Joints - Construction, etc. ______

appear to be in good condition

however gunite masks the majority of the structure

i. Foundation Rock foundation - shale

weathered where exposed to the environment

j. Abutments good condition

k. Control Gates operational

l. Approach & Outlet Channels good condition

m. Energy Dissipators (plunge pool, etc.) none - rock

channel is the energy dissipator

n. Intake Structures appears to be in good condition

(operational) - water level masks most of intakes

o. Stability No stability problems observed other

than deteriorated gunite, seepage may present some problems

suggest a stability analysis taking this & cracking into account

p. Miscellaneous void near toe of spillway adjacent

to north buttress - possible scour, minor

problem.

APPENDIX D

HYDROLOGIC/HYDRAULIC

ENGINEERING DATA AND COMPUTATIONS

CHECK LIST FOR DAMS
HYDROLOGIC AND HYDRAULIC
ENGINEERING DATA

AREA-CAPACITY DATA:

		Elevation (ft.)	Surface Area (acres)	Storage Capacity (acre-ft.)
1)	Top of Dam	979.15	30.94	750
2)	Design High Water (Max. Design Pool)	977.15	30.56	690
3)	Auxiliary Spillway Crest	–	–	–
4)	Pool Level with Flashboards	–	–	–
5)	Service Spillway Crest	974.15	30.00	600

DISCHARGES

		Volume (cfs)
1)	Average Daily	Unknown
2)	Spillway @ Maximum High Water	4,280
3)	Spillway @ Design High Water	4280
4)	Penstock ~~Spillway @ Auxiliary Spillway Crest Elevation~~	320
5)	~~Low Level Outlet~~ Reservoir Drain	260
6)	Total (of all facilities) @ Maximum High Water	4,860
7)	Maximum Known Flood 3.18.1936 (EL.975.6)	1,870

CREST: ELEVATION: 979.15

Type: CONCRETE BUTTRESS DAM

Width: 4'-0" BUTTRESS, 12'-0" EMBANKMENT Length: 760 FEET INCLUDING SPILLWAY AND EARTH EMBANKMENT.

Spillover OGEE, CYCLOPEAN CONCRETE

Location NORTHWEST OF BUTTRESS

SPILLWAY:

PRINCIPAL		EMERGENCY
974.15	Elevation	NONE
OGEE	Type	
	Width	
	Type of Control	
UNCONTROLLED	Uncontrolled	
	Controlled:	
NONE	Type (Flashboards; gate)	
-	Number	
-	Size/Length	
	Invert Material	
	Anticipated Length of operating service	
-	Chute Length	
54 FEET	Height Between Spillway Crest & Approach Channel Invert (Weir Flow)	

OUTLET STRUCTURES/EMERGENCY DRAWDOWN FACILITIES:

Type: Gate ______ Sluice ______ Conduit YES Penstock YES

Shape : CIRCULAR

Size: 6.5' DIAMETER

Elevations: Entrance Invert EL. 949 PENSTOCK

Exit Invert EL. 664 "

Tailrace Channel: Elevation EL 670

HYDROMETEROLOGICAL GAGES:

Type : NONE

Location: ______

Records:

Date - ______

Max. Reading - ______

FLOOD WATER CONTROL SYSTEM:

Warning System: NONE

______

Method of Controlled Releases (mechanisms):

ONLY THROUGH PENSTOCK. GATE ON PENSTOCK OPERATED ELECTRICALLY.

DRAINAGE AREA: 52 Square Miles

DRAINAGE BASIN RUNOFF CHARACTERISTICS:

Land Use - Type: Wooded

Terrain - Relief: HILLY

Surface - Soil: —

Runoff Potential (existing or planned extensive alterations to existing (surface or subsurface conditions)

NONE

Potential Sedimentation problem areas (natural or man-made; present or future)

NONE

Potential Backwater problem areas for levels at maximum storage capacity including surcharge storage:

NONE

Dikes - Floodwalls (overflow & non-overflow) - Low reaches along the Reservoir perimeter:

Location: NONE

Elevation: ____________

Reservoir:

Length @ Maximum Pool 0.76 (Miles)

Length of Shoreline (@ Spillway Crest) 1.52 (Miles)

SPILLWAY RATING CURVE

SPILLWAY SECTION

$C = 3.27 + 0.40\frac{H}{h}$ for Ogee Spillway

where C = Coefficient of Discharge

$L = L' - 0.1NH$ H = Head Over Spillway

h = Height of spillway

$Q = C L H^{3/2}$ L = Crest length of spillway

L' = Measured Length of spillway

$L' = 251$ Ft., $N = 2$ N = Number of End Contractions

EL. FT.	H, FT.	h, FT.	C	L, FT	Q, CFS	REMARKS *
975.15	1	54	3.28	250.8	823	
976.15	2	54	3.28	250.6	2325	
977.15	3	54	3.29	250.4	4281	
978.15	4	54	3.30	250.2	6605	7195
979.15	5	54	3.31	250.0	9252	10918

* Adding discharges over spillway and arch sections.

SPILLWAY RATING CURVE

ARCH SECTION

$C = 3.27 + 0.40 \frac{H}{h}$ — For Ogee Spillway

$L = L' - 0.1NH$

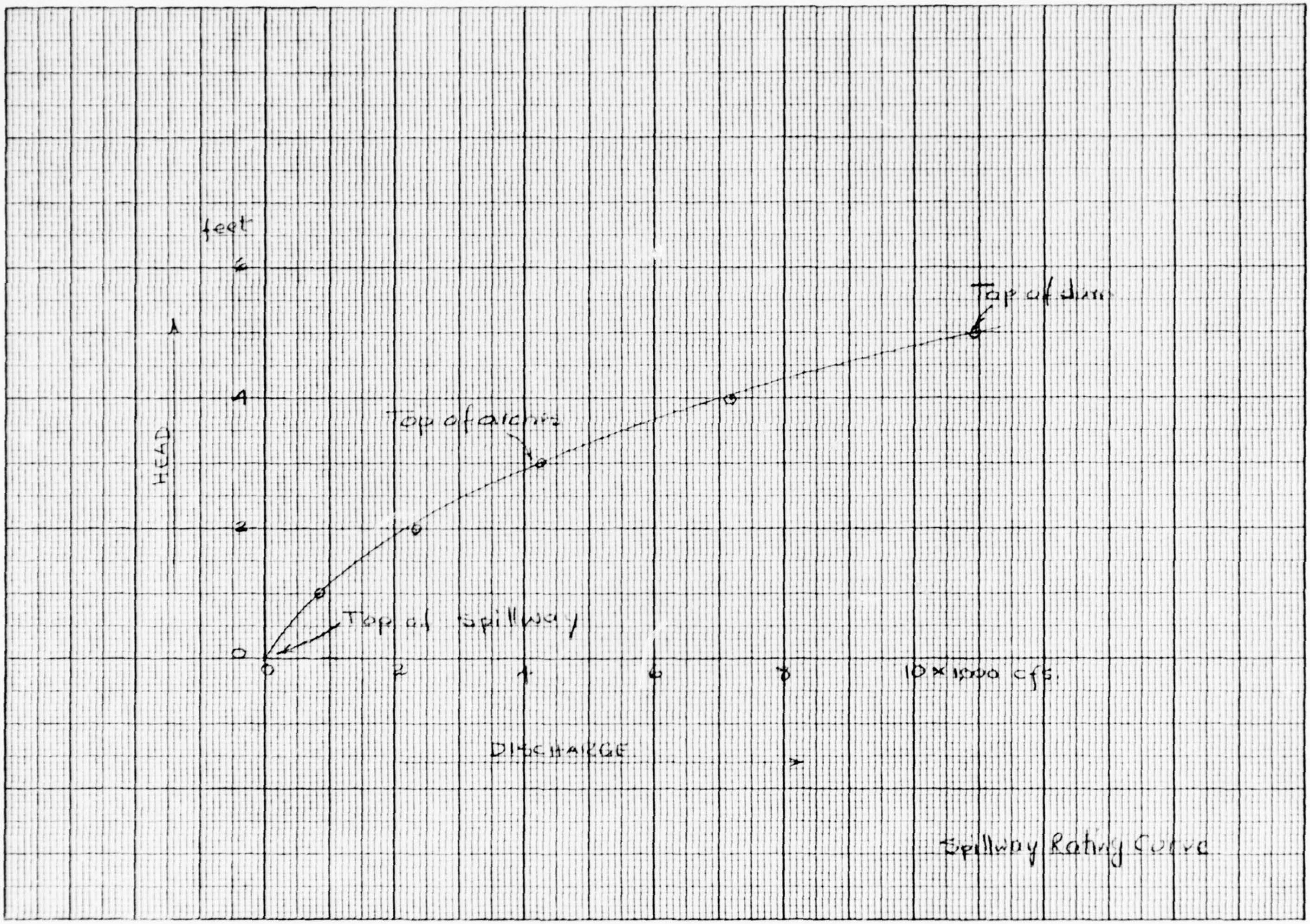
$Q = CLH^{3/2}$

$L' = 180$ Ft., $N = 2$

where
- C = Coefficient of Discharge
- H = Head Over Spillway
- h = Height of spillway
- L = Crest length of spillway
- L' = Measured Length of spillway
- N = Number of End Contractions

EL. FT.	H, FT.	h, FT.	C	L, FT	Q, CFS	REMARKS
978.15	1	54	3.28	179.8	590	
979.15	2	54	3.28	179.6	1666	

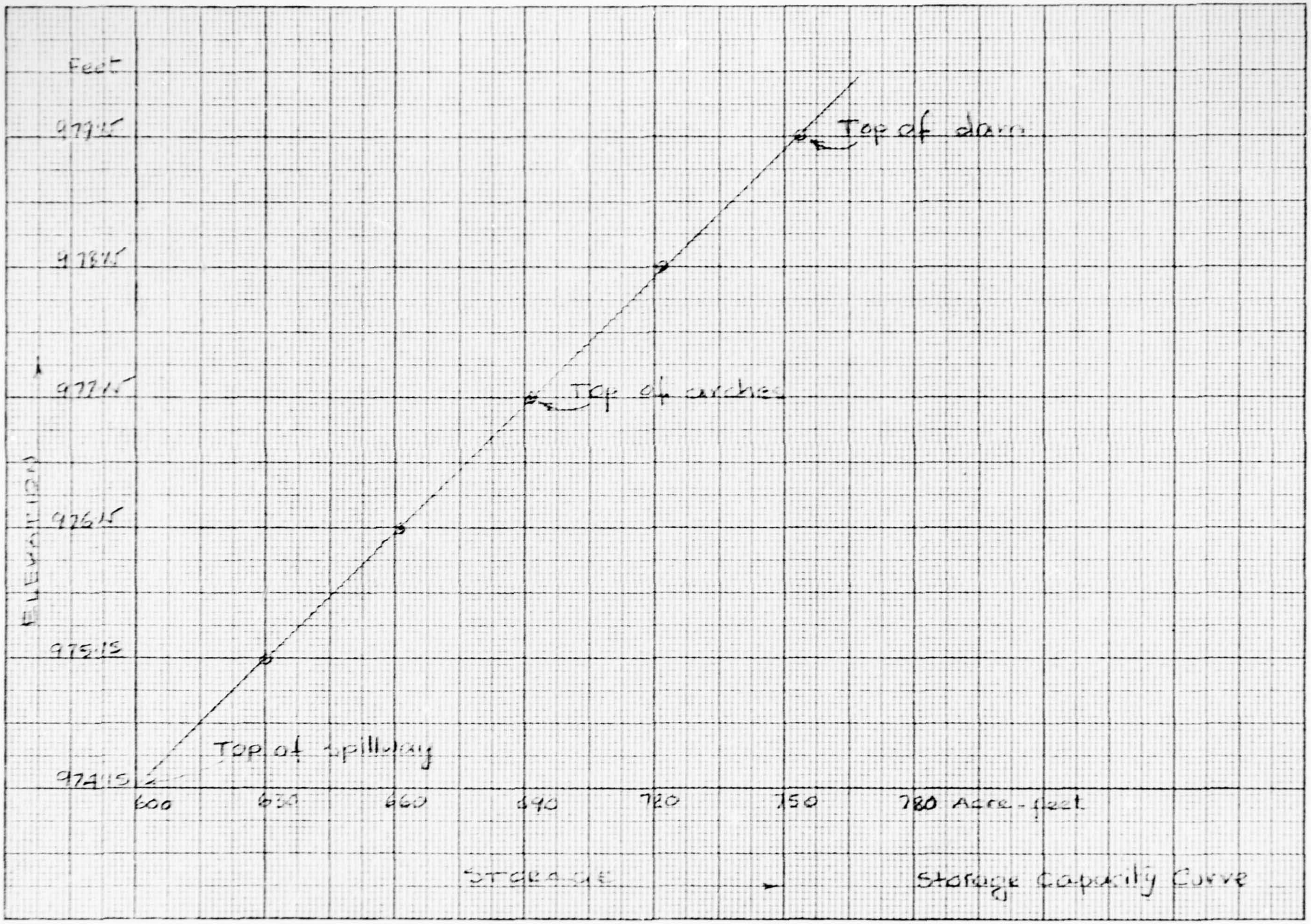
K+E 20 X 20 TO THE INCH 46 1240
7 X 10 INCHES MADE IN U.S.A.
KEUFFEL & ESSER CO.



STORAGE CAPACITY CURVE

ELEVATION Feet	STORAGE Acre-feet
974.15	600
975.15	630
976.15	661
977.15	691
978.15	722
979.15	753

All storage approximate.


Feet
979.15
978.15
977.15
976.15
975.15
974.15
ELEVATION
Top of dam
Top of arches
Top of spillway
600
630
660
690
720
750
780 Acre-feet
STORAGE
Storage Capacity Curve

Ephratah Dam — PMF, 1/2 PMF

Drainage area from USGS sheets = 52 square miles

From "Upper Hudson & Mohawk River Basins Hydrologic Flood Routing Models" study, subdivision 16; pages 97 - 107:

Area of subbasin 16 = 151 square miles

SPF = 1/2 PMF = 27,889 cfs

∴ PMF = 27,889 × 2 = 55,778 cfs

$$\left(\frac{A_1}{A_2}\right)^{3/4} = \frac{PMF_1}{PMF_2} \quad ; \quad \left(\frac{52}{151}\right)^{3/4} = \frac{PMF_1}{55,778}$$

∴ PMF for Ephratah Dam = 25,074 cfs.

≃ 25,000 cfs

For 1/2 PMF

$$\left(\frac{A_1}{A_2}\right)^{3/4} = \frac{1/2\ PMF_1}{1/2\ PMF_2} \quad , \quad \left(\frac{52}{151}\right)^{3/4} = \frac{1/2\ PMF_1}{27,889}$$

1/2 PMF = 12,537 cfs.

≃ 12,500 cfs.

CAPACITIES OF RESERVOIR DRAIN AND PENSTOCK

$Q = C_c \, C_v \, A \sqrt{2gh}$ where

C_v = Coefficient of velocity

C_c = Coefficient of contraction

A = Area of the drain pipe, in ft^2

h = head, in ft

Q = Discharge, in cfs.

use $C_c = 0.66$, $C_v = 0.95$, Diameter of pipe = 3 feet

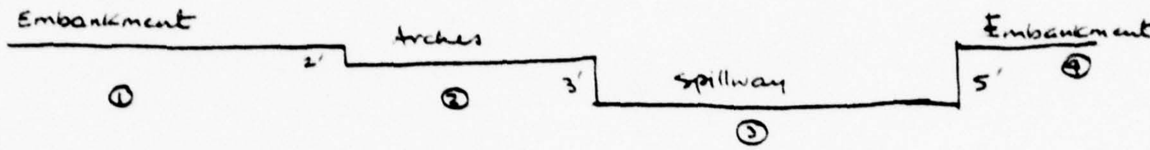
$g = 32.2 \; ft^2/sec$ $h = 54$ feet

$$Q = .66 \times .95 \times \left(\frac{\pi \, 3^2}{4}\right) \sqrt{2 \times 32.2 \times 54}$$

$= 261$ cfs

Capacity of Penstock = 320 cfs.

OVERTOPPING



Lengths: Embankments ①+④ = 329 feet
Arches ② = 180 "
Spillway ③ = 251 "

Total length of dam = 760 "

PMF = 25,000 cfs, ½ PMF = 12,500 cfs.

$Q = CLH^{3/2}$

$$Q = 3.3 \times 329 \times (.33)^{3/2} + 3.3 \times 180 \times (2.33)^{3/2} + 3.3 \times 251 \times (5.33)^{3/2}$$
$$= 206 + 2113 + 10192$$
$$= 12{,}511 \text{ cfs} \approx 12500 \text{ cfs}.$$

Hence, the dam is overtopped by ½ PMF by $.33 \times 12 = 4$ inches.

$$Q = 3.30 \times 329 \times (2.25)^{1.5} + 3.30 \times 180 \times (4.25)^{1.5} + 3.3 \times 251 \times (7.25)^{1.5}$$
$$= 3{,}664 + 5{,}204 + 16{,}169$$
$$= 25{,}038 \approx 25{,}000 \text{ cfs}.$$

Hence, the dam is overtopped by PMF by 27 inches.

LIST OF REFERENCES

APPENDIX E

APPENDIX E

REFERENCES

1) U.S. Department of Commerce, Technical Paper No. 40, Rainfall Frequency Atlas of the United States, May 1961.

2) Soil Conservation Service, National Engineering Handbook, Section 4, Hydrology, August 1972 (U.S. Department of Agriculture).

3) H.W. King and E.F. Brater, Handbook of Hydraulics, 5th edition, McGraw-Hill, 1963.

4) T.W. Lambe and R.V. Whitman, Soil Mechanics, John Wiley and Sons, 1965.

5) W.D. Thornbury, Principles of Geomorphology, John Wiley and Sons, 1969.

6) University of the State of New York, Geology of New York, Education Leaflet 20, Reprinted 1973.

7) Cornell University Agriculture Experiment Station (compiled by M.G. Cline and R.L. Marshall), General Soil Map of New York State and Soils of New York Landscapes, Information Bulletin 119, 1977.

8) Upper Hudson and Mohawk River Basins Hydrologic Flood Routing Models, New York District Corps of Engineers.

APPENDIX F

STABILITY ANALYSES

EPHRATAH HYDROELECTRIC PLANT

SPILLWAY OUTLINE

7/9/71

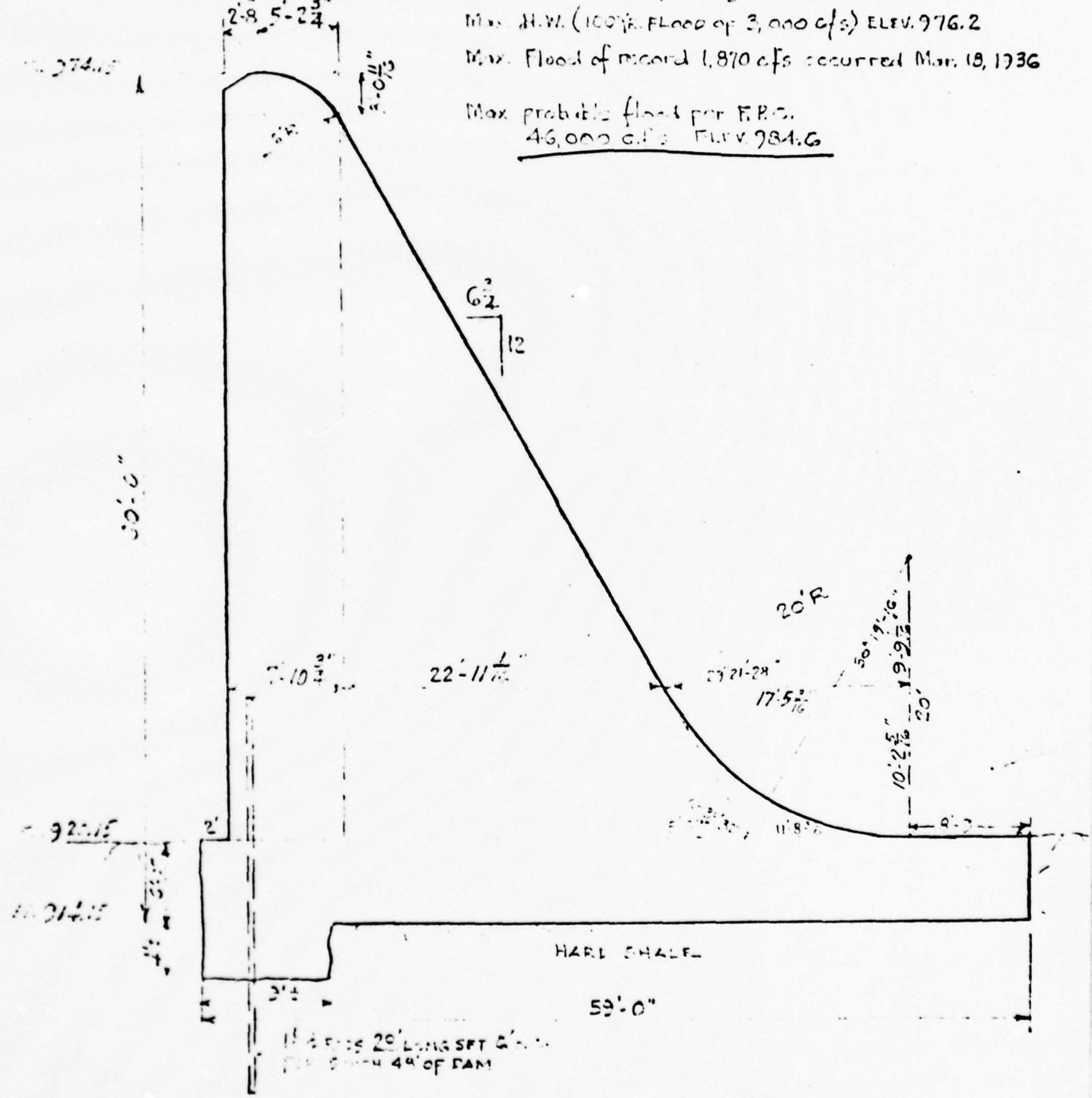
1'5" FLASHBOARDS REMOVED. TOP F.B. ELEV. 975.57

Normal pond and spillway crest = ELEV. 974.15

Max. H.W. (100 Yr. Flood of 3,000 c/s) ELEV. 976.2

Max. Flood of record 1,870 c/s occurred Mar. 18, 1936

Max probable flood per F.P.C.
46,000 c.f.s. ELEV. 984.6



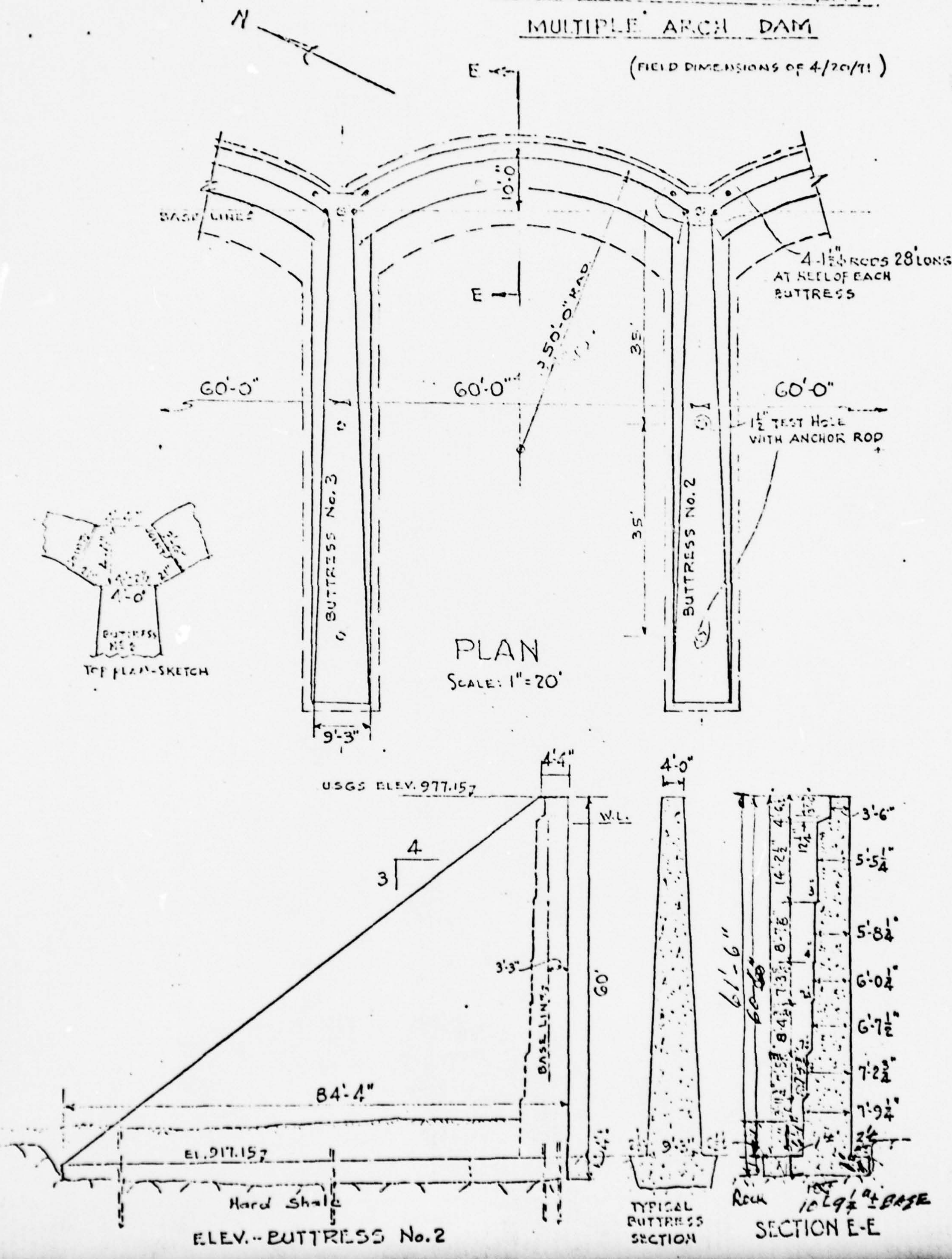
SPILLWAY SECTION

SCALE: 1"=10'

[illegible] DEVELOPMENT

MULTIPLE ARCH DAM

(FIELD DIMENSIONS OF 4/20/71)



DISTRICT Albany

FROM R. J. DeStefano

DATE July 16, 1971 FILE CODE

TO Mr. R. C. Clancy

SUBJECT Ephratah Hydro
Multiple Arch-Non-Overflow Dam
Stability Analysis

My stability analysis of the multiple arch non-overflow dam, Ephratah Hydro, F.P.C. Project No. 2706, based on our recent field dimensions of the dam, indicate that all of the F.P.C. criterias' are satisfied, and the structure is stable against all reasonable expected loadings as assumed. Uplift is not usually considered in an arch structure of this type and therefore was not included in the study. Stability summary, Sketch B, is included in the Appendix.

The five conditions analysed, using rock base at elevation 914.15, are as follows:

Case 1 - Water line at normal pond elevation of 974.15.

Case 2 - Water line at maximum high water elevation of 976.2, which is our computed 100 year flood of 3,000 cfs.

Case 3 - Water line at maximum probable flood elevation of 984.6, which was determined by F.P.C. corresponding to a flow of 46,000 cfs.

Case 4 - Same as Case 1 and includes ice.

Case 5 - Same as Case 1 and includes earthquake.

The full arch span of 60 feet was used for the water face pressure acting on the buttress and only one-quarter of the arch span on each side of the buttress together with the buttress section was considered for base shear and overturning.

The usual allowable limits in the design of dams as determined from F.P.C. discussions are as follows:

Factor of safety against overturning

	Minimum	Preferred
Normal pond	1.50	2.00
Design high water	1.25	1.50
Maximum probable flood	1.00	1.25

Unit shear value, a maximum of 400 pounds per square inch unless based on actual rock shear tests

Rock shear-fricition factor of safety, usually 5, except for the maximum probable flood condition when 4 is permissible.

The computed stability results:

	S.F. against Overturning	Base Unit shear, psi	Base shear-friction F.S.
Case 1	2.26	325	5.80
Case 2	2.05	360	5.48
Case 3	1.46	362	4.38
Case 4	1.99	351	5.62
Case 5	1.59	371	5.36

Note that all of the above computed values are within the F.P.C. limits and therefore should satisfy the F.P.C. requirements.

Previous Discussions:

On December 7, 1970, Mr. P. H. Tucker met informally with Messrs. Carl Marlatt and Jack Shepley of the F.P.C. Bureau of Power in Washington, D.C. to discuss the stability analysis of the Ephratah Dam.

Mr. Marlatt had determined a maximum probable flood of 46,000 cfs with headwater elevation of 984.6 and indicated that our computed rock base unit shear value of 508 psi for the 46,000 cfs flood condition was greater than the 400 psi that the F.P.C. allows when no detailed exploratory information is available.

Messrs. Marlatt and Shepley concluded that the best means of resolving the rock unit shear value would be to take borings in the bed rock and test the rock cores for the actual shear value and if the unit shear value yielded a shear-friction factor of safety of 4 they felt the dam stability would be adequate.

Conclusions:

My original computations that noted the 508 psi unit shear value were based on our filed drawings that were noted "as built"; and these same dimensions are noted in the F.P.C. application drawing. My field inspection colored photographs indicated a much thicker wall near top so it was decided to take actual field dimensions.

The actual field dimension of the arch and buttress section were taken April 20, 1971. The field dimension show the arch wall thickness near the base of 7 feet $9\frac{1}{4}$ inches compared to the 5 feet 0 inches as was noted on the "as built" drawing.

It is concluded therefore:

1 - Based on the actual arch dimensions, the 59 year old multiple arch non-overflow dam satisfied the F.P.C. criteria and is stable against all reasonable expected loadings as assumed.

2 - In my opinion, the rock shear tests as requested by the F.P.C. are not required as the computed unit shear value is below the F.P.C. allowable value.

3 - The F.P.C. application drawing, showing the arch section, should be revised to agree with the field dimensions and resubmitted to the F.P.C.

4 - This project will require a certified initial inspection report to be submitted to the F.P.C. within 2 years after the date of the issuance of the license. The F.P.C. presumably now requires that the initial inspection report also contain an analysis of spillway adequacy, effect of overtopping, should it occur, and stability of the project structures.

5 - As previously noted, our stability analysis for the multiple arch non-overflow structure is stable against the maximum probable flood of 46,000 cfs, which value was given to us by the F.P.C. and not verified by us to date. Therefore, it is suggested that the final determination of the maximum probable flood together with the spillway adequacy, effect of overtopping and stability of structures be determined by our Consultant at the time we are required to do so by the F.P.C.

6 - The revised stability summary for the multiple arch non-overflow dam should be the basis for our future informal discussions with the F.P.C.

Attached hereto is a copy of the actual field dimensions of the arch dam "Sketch A", and a copy of the stability summary of the Ephratah multiple arch non-overflow dam, "Sketch B".

Reference Letter: Mr. R. C. Clancy to H. D. Philip, dated December 16, 1970, Subject-F.P.C. Project No. 2706-Ephratah.

If you need additional information, kindly advise.

RJD

Ralph J. DeStefano

RJD:cg
Attach.
cc: J.W.Keib
L.Martin
J.J.Miller
H.D.Philip
P.D.Raymond
W.E.Stahlka
P.H.Tucker

EPHRATAH HYDRO RJD 7/14/71

SPILLWAY - STABILITY SUMMARY

CONDITION	BASE ELEV.	ΣH KIPS	ΣV KIPS	ΣH / ΣV	? WITH ? 400 PSI	RESULTANT FROM TOE FT.	ΣMr FT. KIPS	ΣMo FT. KIPS	ΣMr / ΣMo	CONC. STRESS PSI HEEL	CONC. STRESS PSI TOE
CASE 1 W.L. AT NORMAL POND EL.974.15, 100% UPLIFT	914.15	112.5	137.8	0.82	15.72	19.53	9,291.6	6,600.7	1.41	-1.6	+32.6
CASE 2- W.L. AT NORMAL POND 100% UPLIFT EARTHQUAKE	914.15	131.1	125.3	1.04	13.4	14.6	9,291.6	7,457.7	1.24	-7.1	+36.8
CASE 3. NORMAL POND 100% UPLIFT ICE	914.15	117.5	137.8	0.85	15.1	17.4	9,291.6	6,895.0	1.35	-3.7	+36.1
CASE 4. W.L. AT EL.973.12 (100 YR FLOOD OF 3,000 cfs) 100% UPLIFT	914.15	119.37	134.79	0.89	14.79	17.7	9,325.4	6,953.0	1.34	-3.2	+34.9
CASE 5 - MAX. PROBABLE FLOOD 46,000 cfs EL 984.6 100% UPLIFT T.W. AT EL.926.15	914.15	146.95	109.43	1.34	11.95	1.19	9,572.0	9,442.1	1.02	-24.0	+50.0

− INDICATES TENSION
+ " COMPRESSION

SHEET NO. ______ OF ______

SUBJECT ______________________ DATE ______

INDEX OR FILE NO. ______

PREPARED BY ______

CHECKED BY ______

CASE 1 - NORMAL POND EL. 974.15, 100% UPLIFT

CASE 2 - NORMAL POND EL. 974.15, 100% UPLIFT, EARTHQUAKE

CASE 3 - NORMAL POND EL. 974.15, 100% UPLIFT, ICE.

CASE 4 - 100 YR. FLOOD OF 3,000 cfs, EL. 976.12, 100% UPLIFT

CASE 5 - MAX. PROB. FLOOD OF 46,000 cfs, EL. 984.6, 100% UPLIFT, ASSUMED T.W. EL. 925.15

VALUES & ASSUMPTIONS USED IN STABILITY ANALYSIS

UNIT WEIGHT OF CONCRETE 150 lb./cu.ft.

UNIT WEIGHT OF WATER 62.4 lb/cu.ft.

UPLIFT: AT BASE. FULL HEADWATER PRESSURE AT UPSTREAM FACE OF STRUCTURE VARYING LINEARLY TO FULL TAILWATER PRESSURE AT DOWNSTREAM FACE OF STRUCTURE.

EARTHQUAKE: FORCE OF 0.05g APPLIED IN THE HORIZONTAL DIRECTION

ICE: HORIZONTAL FORCE EQUAL TO 5,000 lb/L.F.

SHEAR: S_{SF} = FRICTION FACTOR OF SAFETY

$$S_{SF} = \frac{f \Sigma V + r S_a A}{\Sigma H}$$

where $r = 0.5$

S_a = 380 psi (Per B.R.C. when no tests are made)

A = area of base

f = 0.5

ΣV = Summation of vertical forces

ΣH = Summation of horizontal forces

FACTOR OF SAFETY AGAINST OVERTURNING:

Normal pool condition	MIN. 1.50	PREFER. 2.00
Maximum high water	MIN. 1.25	PREFER. 1.50
Maximum probable flood	MIN. 1.00	PREFER. 1.25

113-2 N B-600

SUBJECT [illegible] PROJECT	SHEET NO. 1 OF
MULTIPLE ARCH [illegible] OVERFLOW DAM	DATE 1/25/71
STABILITY ANALYSIS	INDEX OR FILE NO.
(MAX. FLOOD OF RECORD = 1,870 cfs, EL. 975.6)	PREPARED BY R.J.D.
	CHECKED BY

CASE 1 - Water Line at Normal Pond, EL. 974.15 (NO ICE, NO UPLIFT, NO EARTHQUAKE)

CASE 2 - W.L. at Max. High Water, EL. 976.2 (3,000 cfs 100 YR FLOOD) (" " ")

CASE 3 - W.L. at Max. Probable Flood, EL. 984.6 (46,000 cfs PER F.P.C.) (" " ")

CASE 4 - Same as CASE 1 except includes ice

CASE 5 - Same as CASE 1 except includes earthquake

Arch span = 60' consider only 1/4 span each side of buttress for overturning and base shear
No uplift considered for thin wall arch dam.

Rock foundation [illegible] as hard dense shale
w = weight of water = 62.5 lbs. per cu. ft.
weight of concrete = 150 lbs. per cu. ft.
Frictional resistance $\frac{\Sigma F}{\Sigma W}$ Assumed 0.75 for masonry on good rock

Resistance to sliding $\Sigma F \leq \frac{f \Sigma W + r s A}{F_s}$

where:

s = unit shearing strength, for good rock foundation varies from 600 to 1,400 lbs per sq. in (F.P.C. has 400 psi Max)

F_s = shear-friction factor of safety = usually 5 (For Max. Prob. Flood = 4)

r = ratio of average to maximum shearing stress - Assume r = 0.5

f = coef. of static friction - Assumed = 0.75 for masonry on good rock

A = area of base or joint.

Earthquake effect on water and dam $P_e = \frac{2}{3} C_e \alpha h^2$

Where α = ratio of earthquake intensity to gravity - use 0.05 for [illegible] area

C_e = Assumed as 52 for dam height under 300 ft.

Resultant of water pressure = $y = \frac{4}{3\pi} h_e$ above the base

Factor of safety against overturning - normal condition = min. 1.5 prefer. 2.0

- max. high water = min 1.25 " 1.50
- max prob. flood = min. 1.00 " 1.25

EPHRATAH HYDRO R.J.D. 7/14/71

NON-OVERFLOW MULTIPLE ARCH DAM - STABILITY SUMMARY

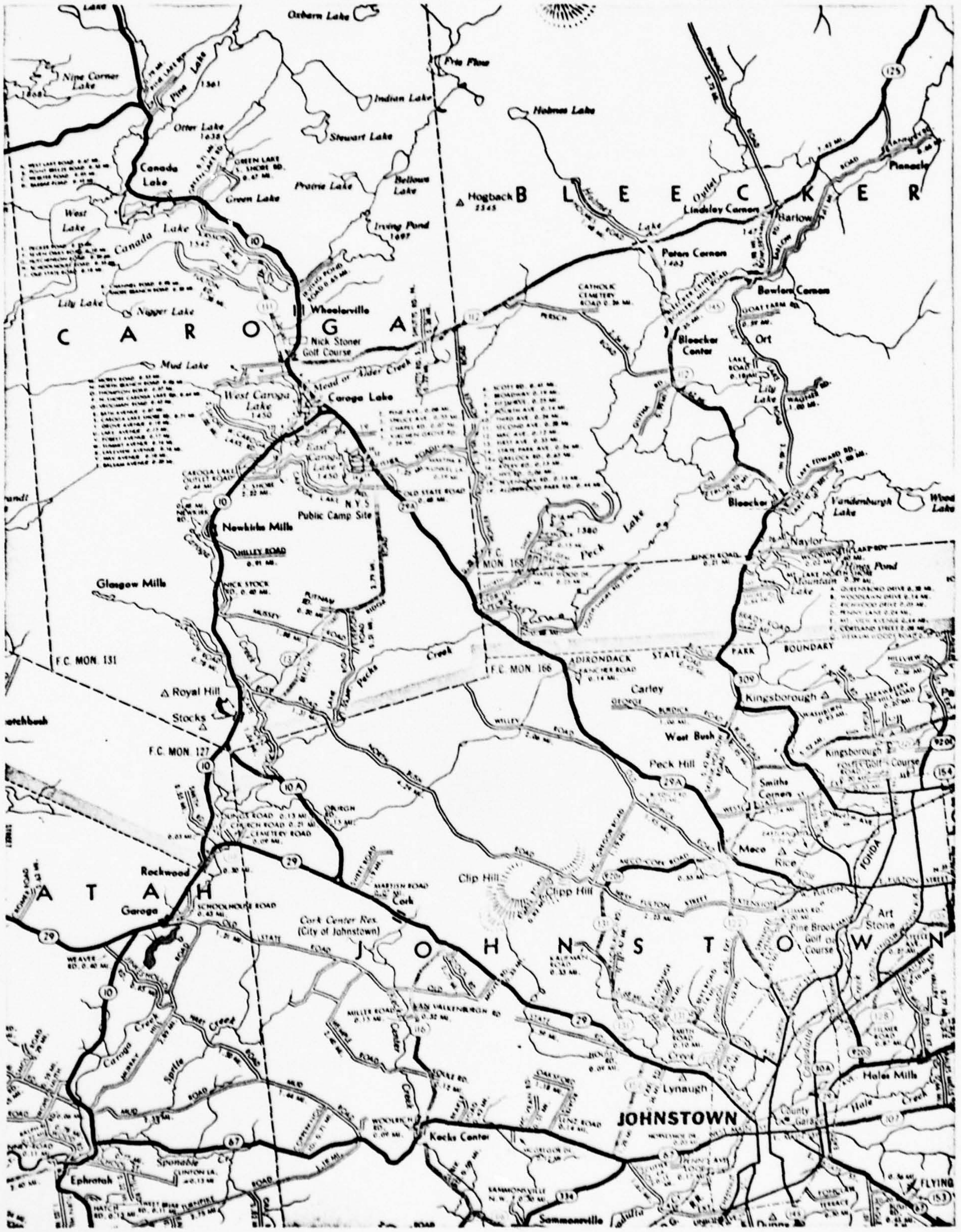
CONDITION		BASE ELEV.	ΣH KIPS	ΣV KIPS	$\frac{\Sigma H}{\Sigma V}$	S_{S-F}	F_S PSI	RESULTANT FROM TOE FT.	ΣM_R FT. KIPS	ΣM_O FT. KIPS	$\frac{\Sigma M_R}{\Sigma M_O}$	CONC. STRESS PSI HEEL	CONC. STRESS PSI TOE
CASE 1	W.L. AT NORMAL POND ELEV. 974.15	914.15	6,750.0	4,894.0	1.38	325	5.80	34.95	306,135.	135,000.	2.26	+8.8	+47.4
CASE 2	W.L. AT MAX. H.W. EL. 976.2 (100 YR FLOOD OF 3,000 c.f.s.)	914.15	7,219.0	4,394.0	1.48	360	5.49	32.55	306,135.	149,305	2.05	+5.1	+51.9
CASE 3	W.L. AT MAX. PROBABLE FLOOD ELEV. 984.6 (46,000 c.f.s. PER F.P.C.)	914.15	9,202.0	4,394.0	1.84	362	4.33	19.85	306,591.	210,645.	1.46	-14.6	+75.5
CASE 4	SAME AS CASE 1 WITH ICE	914.15	7,050.0	4,894.0	1.44	351	5.62	31.20	306,135.	153,315.	1.99	+2.8	+54.4
CASE 5	SAME AS CASE 1 WITH EARTHQUAKE	914.15	7,369.0	4,649.0	1.85	371	5.36	30.40	306,135	165,005.	1.59	+2.5	+54.1

− INDICATES TENSION
+ " COMPRESSION

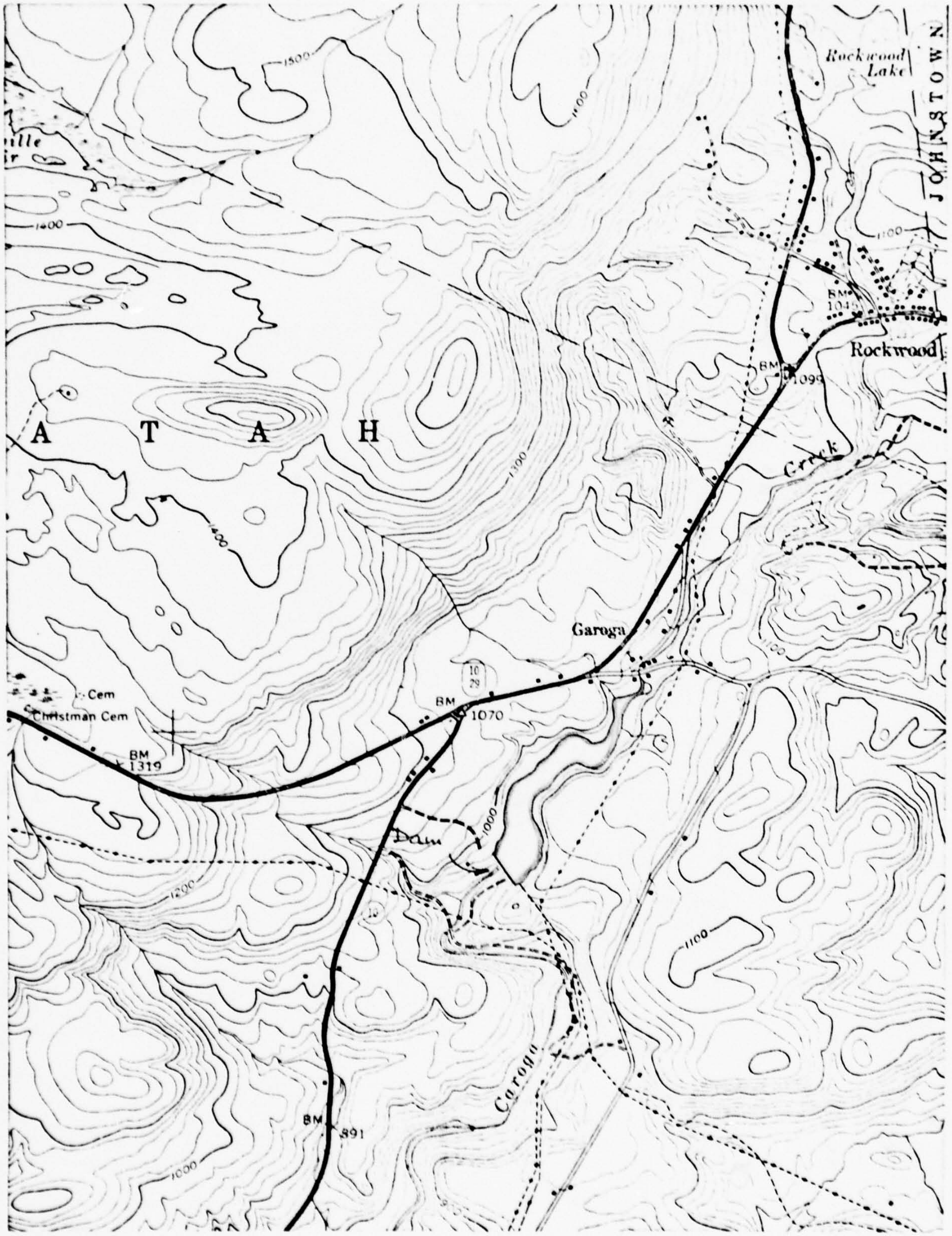
SKETCH B

APPENDIX G

DRAWINGS



VICINITY MAP

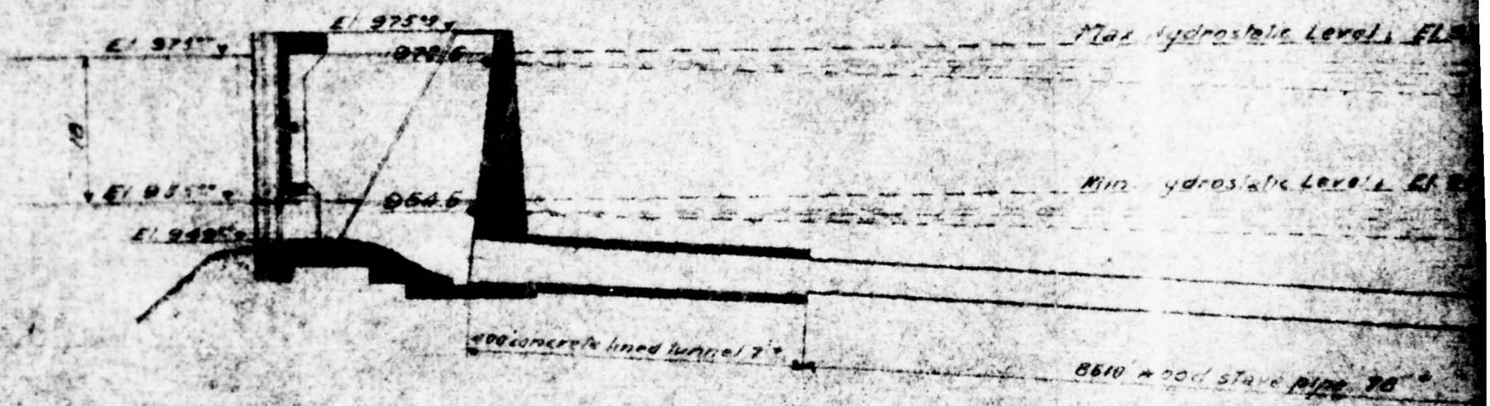


TOPOGRAPHIC MAP

LIST OF DRAWINGS: EPHRATAH DAM

1. General plan of development
2. Elevation and sections of dam
3. Penstock connecting intake, surge tank and power house

INTAKE



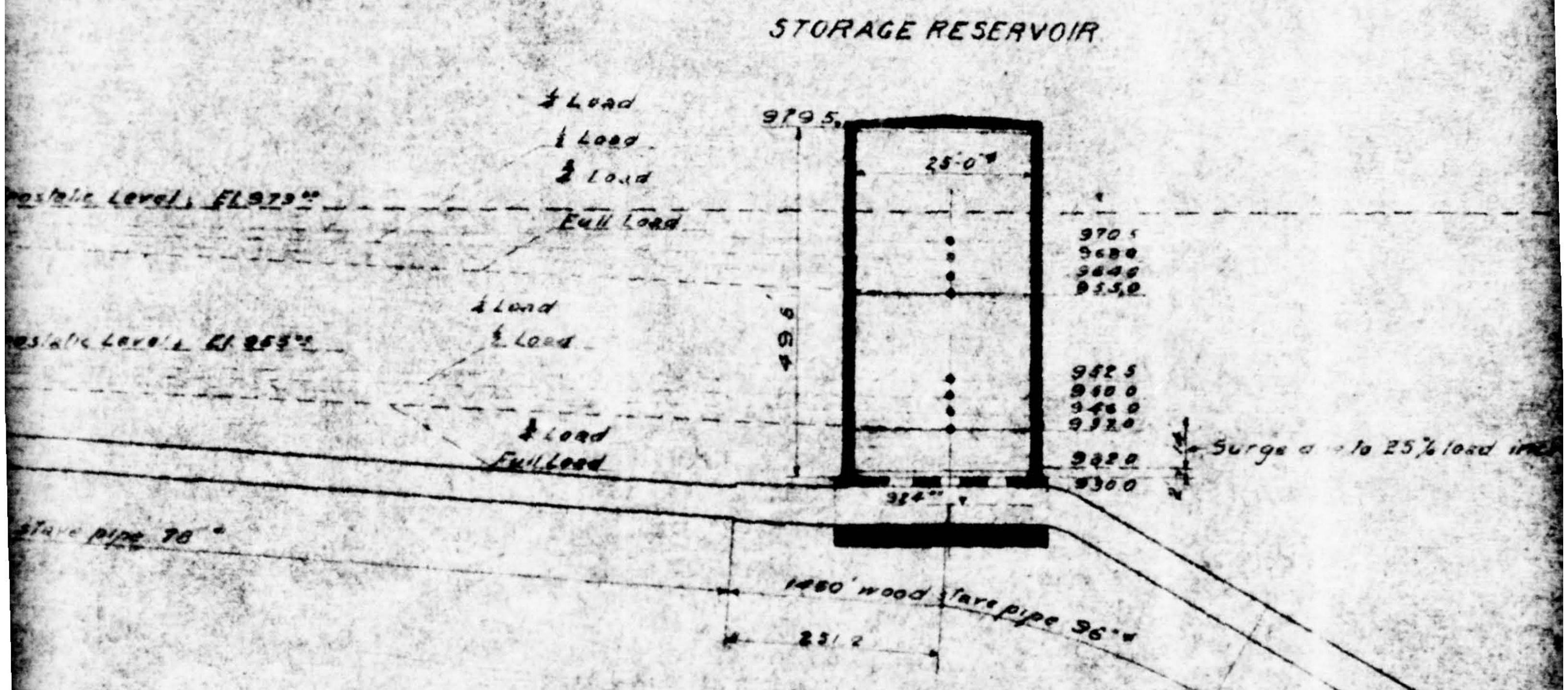
HYDRAULIC ELEMENTS.

	Full Load	$\frac{3}{4}$ Load	$\frac{1}{2}$ Load	$\frac{1}{4}$ Load
Loss of head Intake	0.5'	0.25'	0.15'	0.1'
" " " Tunnel, per ft.	.00185	.000955'	.00051	.000275'
" " " 78" woodpipe "	.00192'	.000988	.000526	.000285'
" " " 96" " "	.00064'	.000332'	.000177	.0000955
" " " 96" steel " "	.000913'	.000472'	.000252'	.000136'
Maximum net head	277.0'	281.0'	291.5	294.0'
Minimum " "	259.0'	269.0'	273.5'	276.0'
Velocity in tunnel ft. per sec.	7.28'	5.22'	3.82'	2.81'
" " 78" pipe " " "	8.44'	6.06'	4.43'	3.25'
" " 96" " " " "	5.57'	4.01'	2.92'	2.15'

T

Horse Power - 1 Unit	
" 2 "	
" 3 "	
" 4 "	
Efficiency	
Revolutions per mi	
Gate opening	
Discharge cu. ft. sec.	
" " " 4	

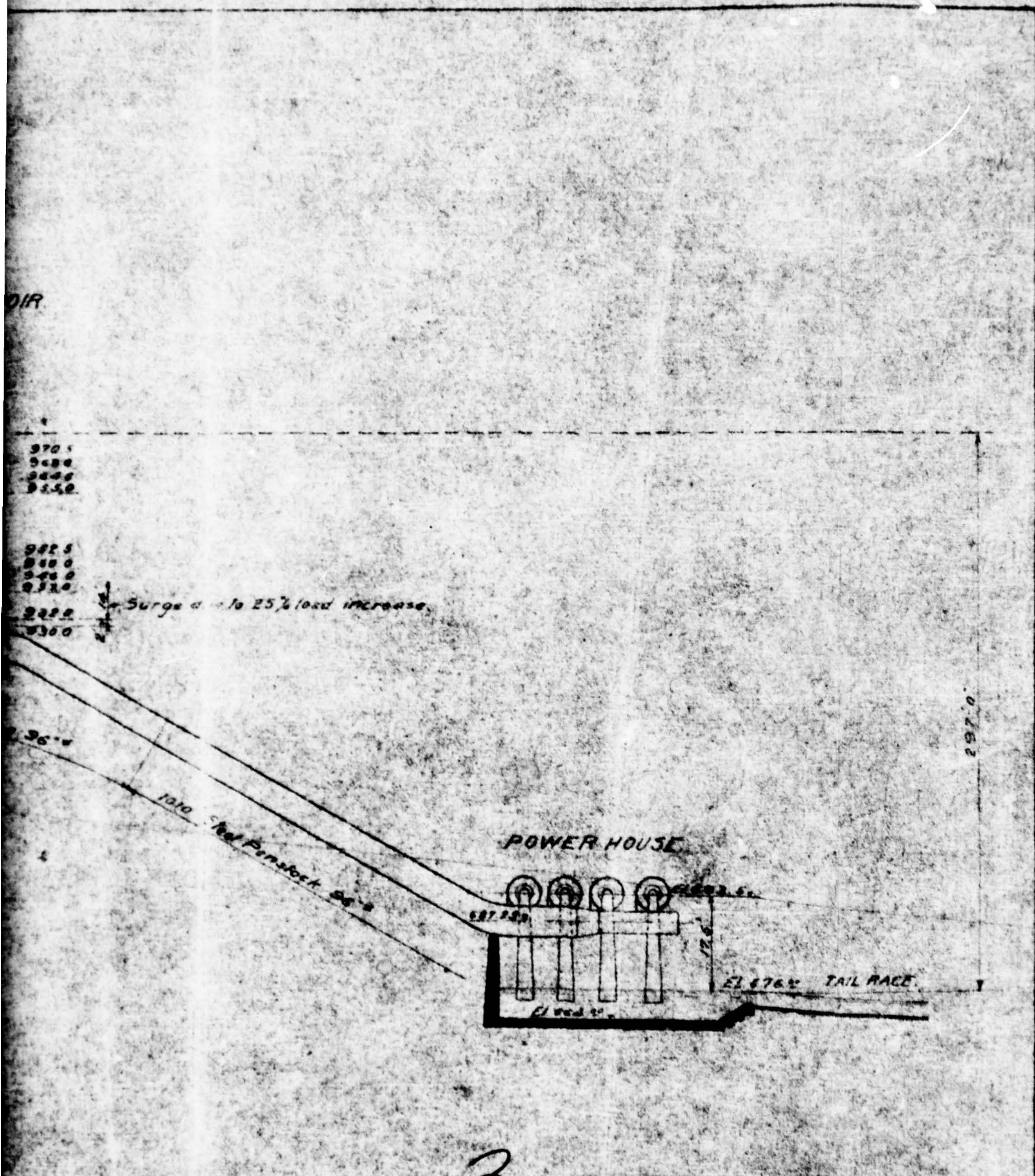
1281.



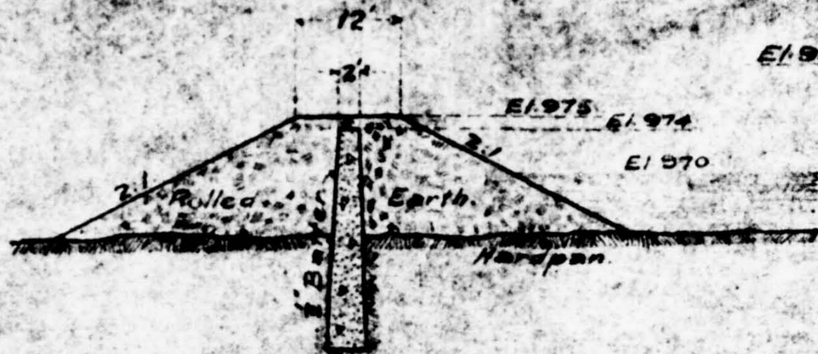
TURBINES ELEMENTS

	Full Load	¾ Load	½ Load	¼ Load
...se Power - 1 Unit	1750	1312	875	440
2 " "	3500	2624	1750	880
3 "	5250	3936	2625	1320
4 "	7000	5250	3500	1750
...ficiency	81%	82%	75%	51%
...volutions per min	720	720	–	–
...te opening	1.0	.74	.56	.40
...charge cu ft sec 1 unit	69	50	37	27
" " " 4 "	280	201	147	108

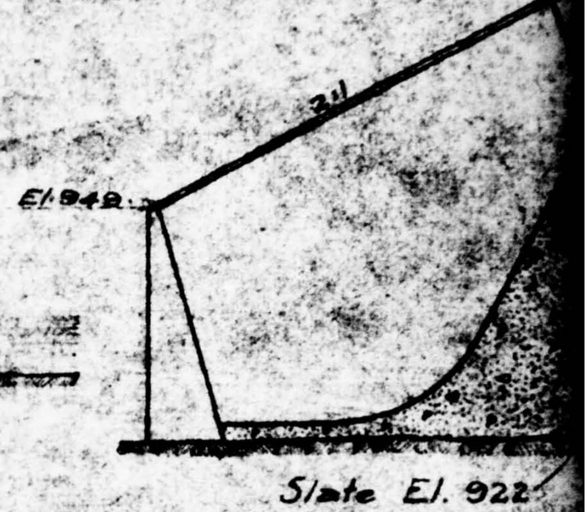
2



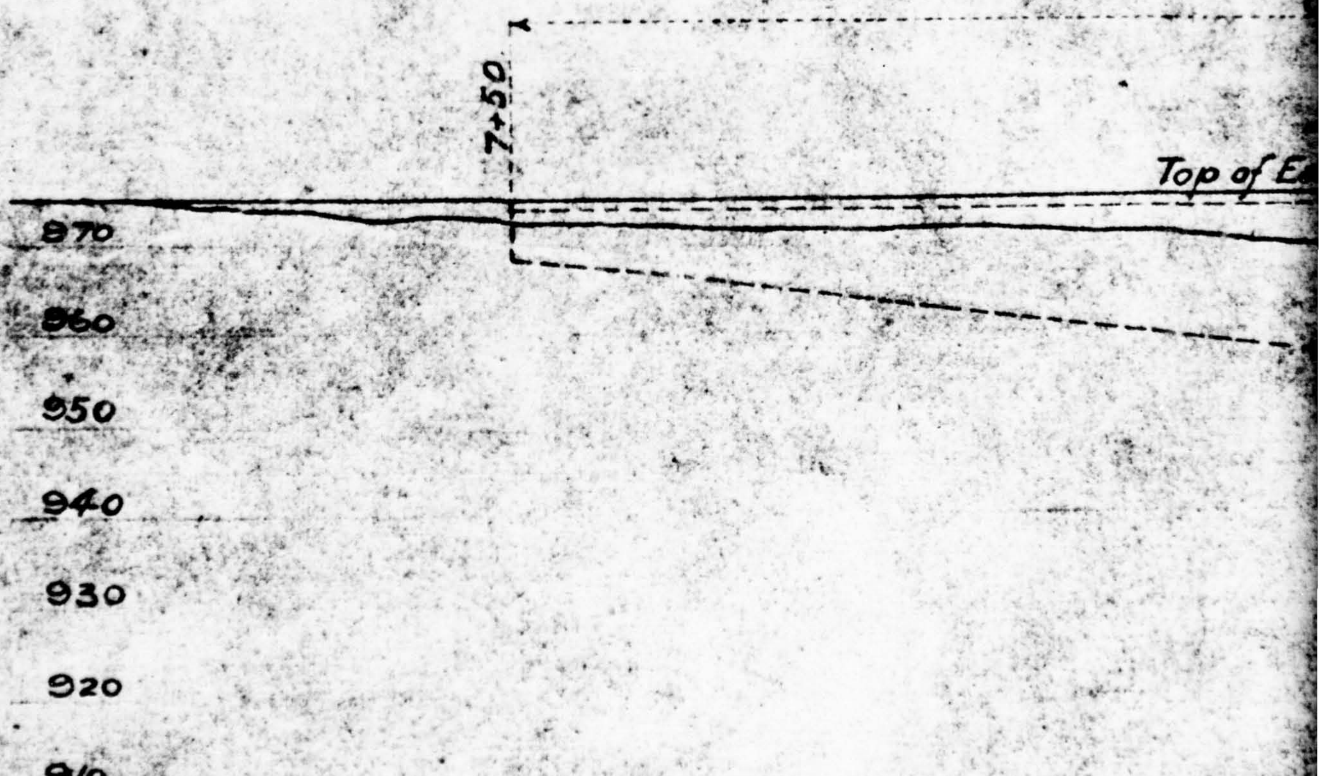
3

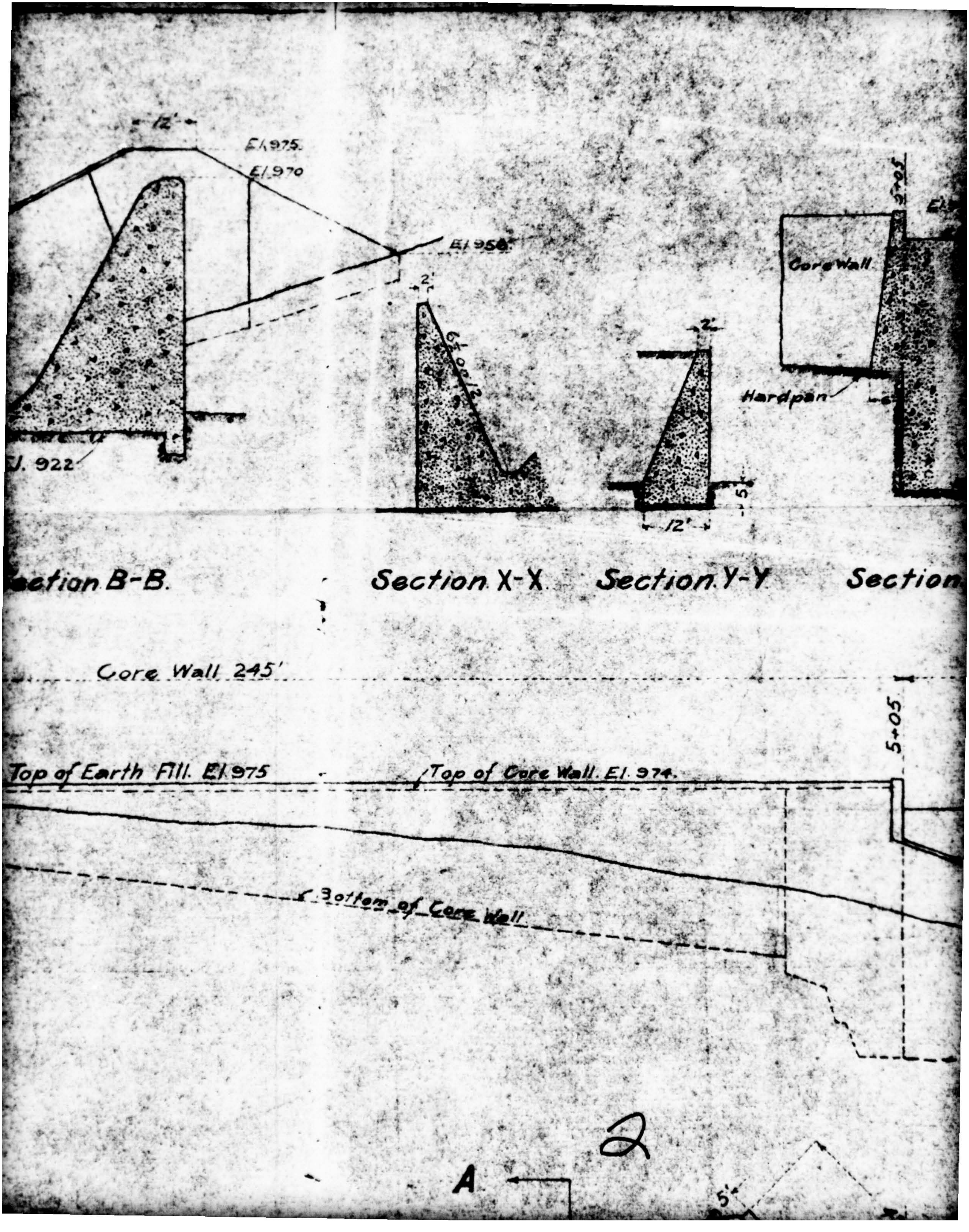


Section A-A

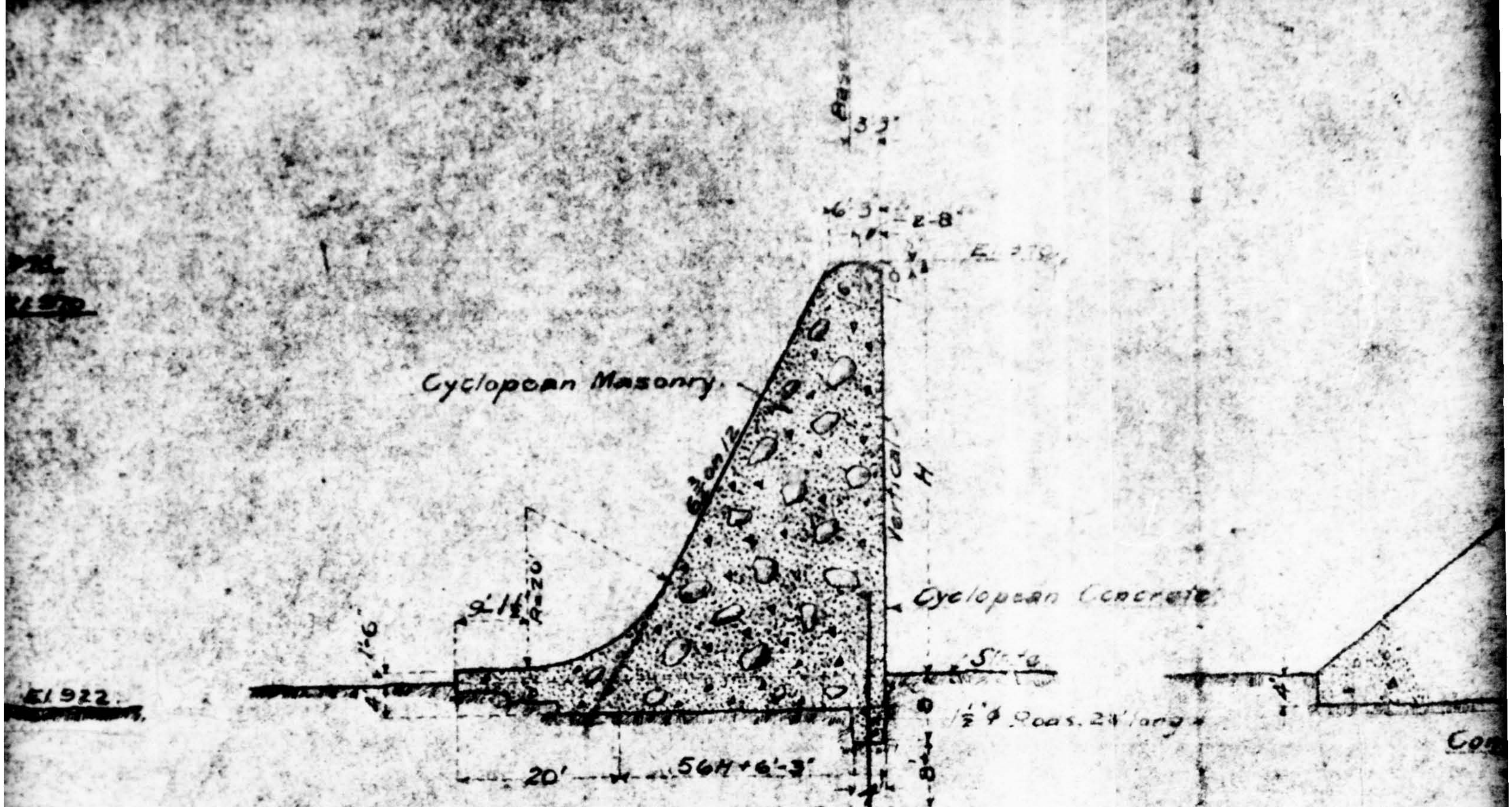


Section

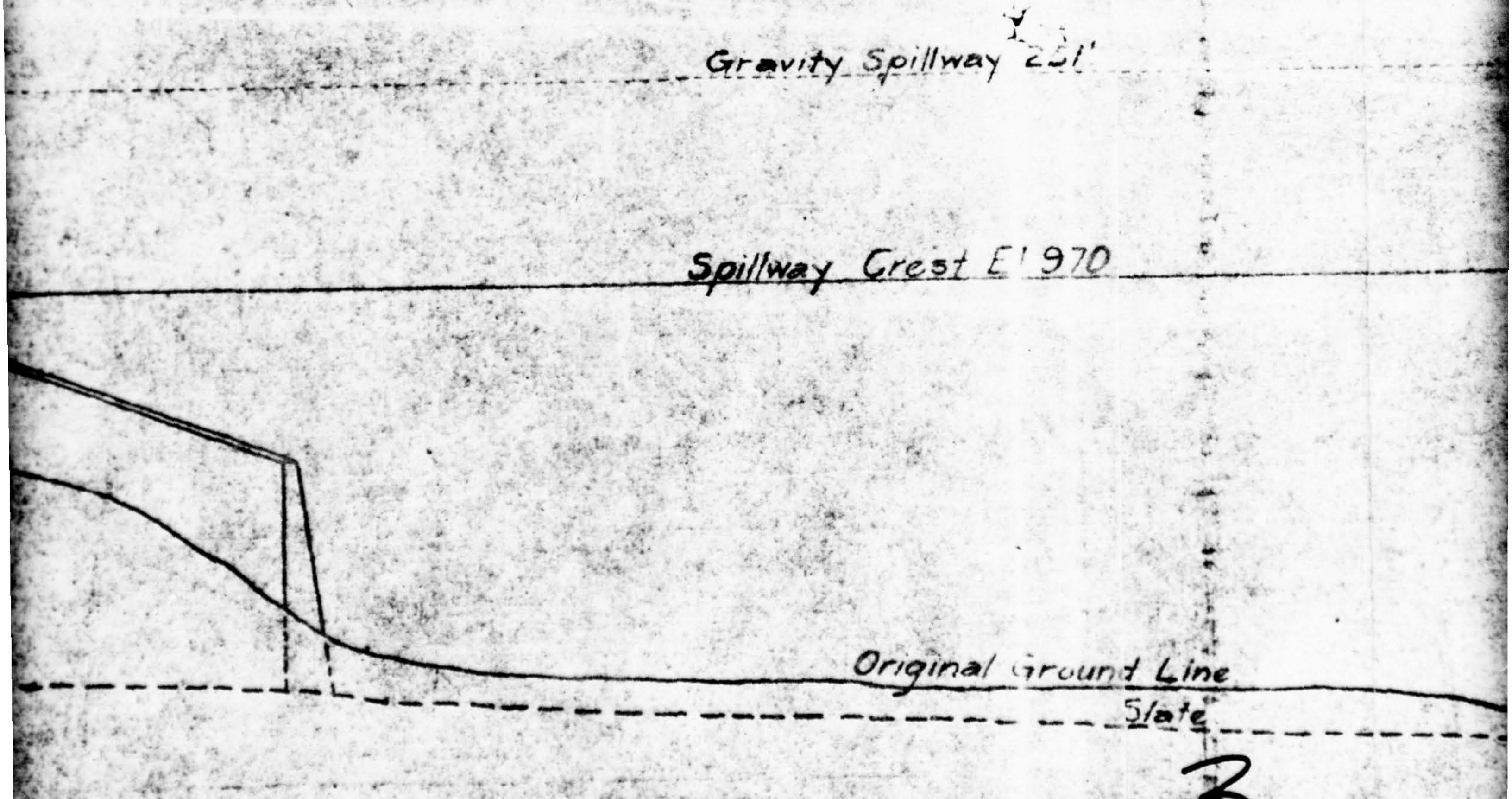




12'
El.975
El.970
El.958
J. 922
2'
Core Wall
Hardpan
2'
12'
5'
ection B-B.
Section X-X
Section Y-Y
Section
Core Wall 245'
5+05
Top of Earth Fill. El.975
Top of Core Wall. El. 974.
Bottom of Core Wall
2
A
5'



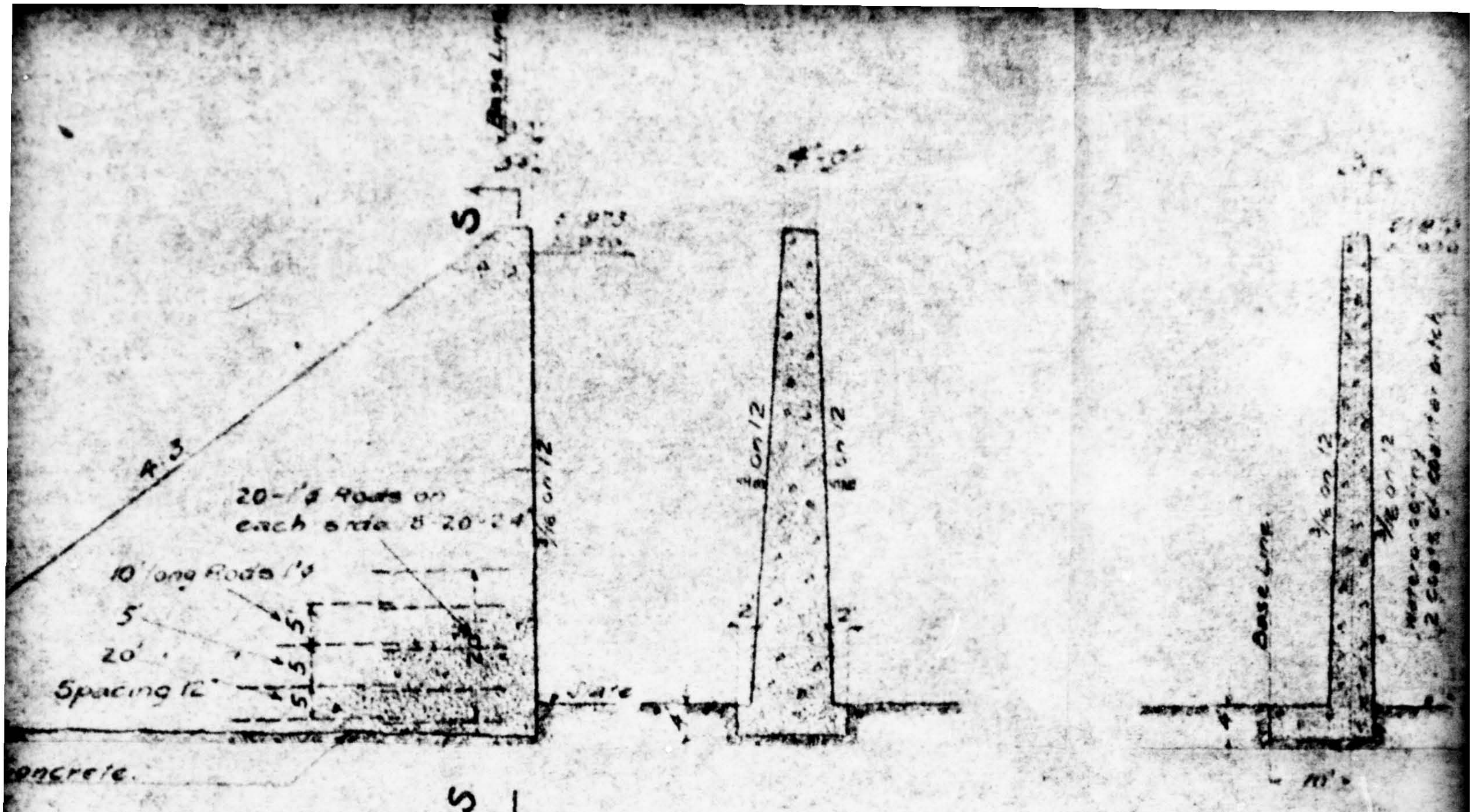
Section C-C.



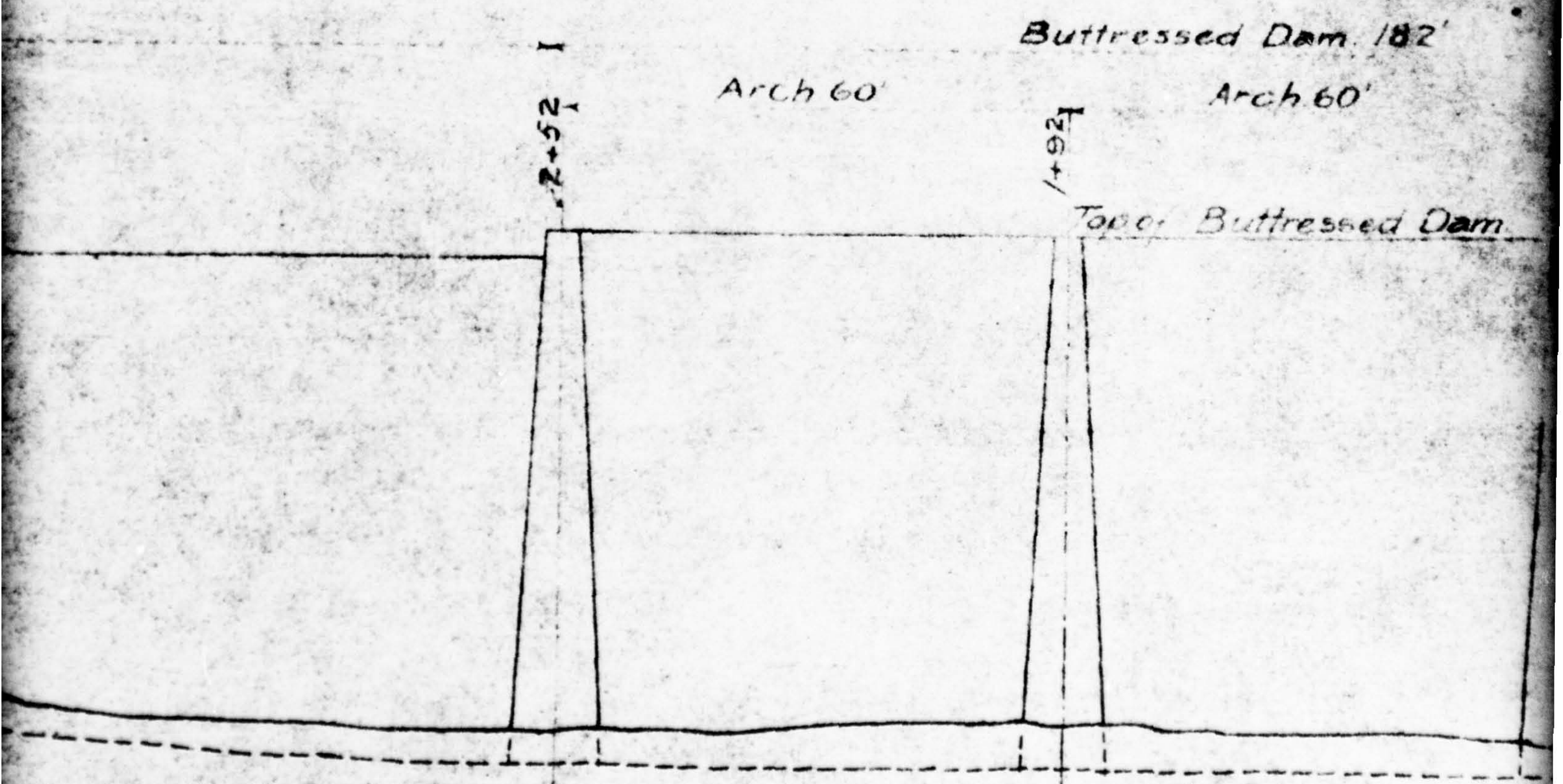
3

Elevation.

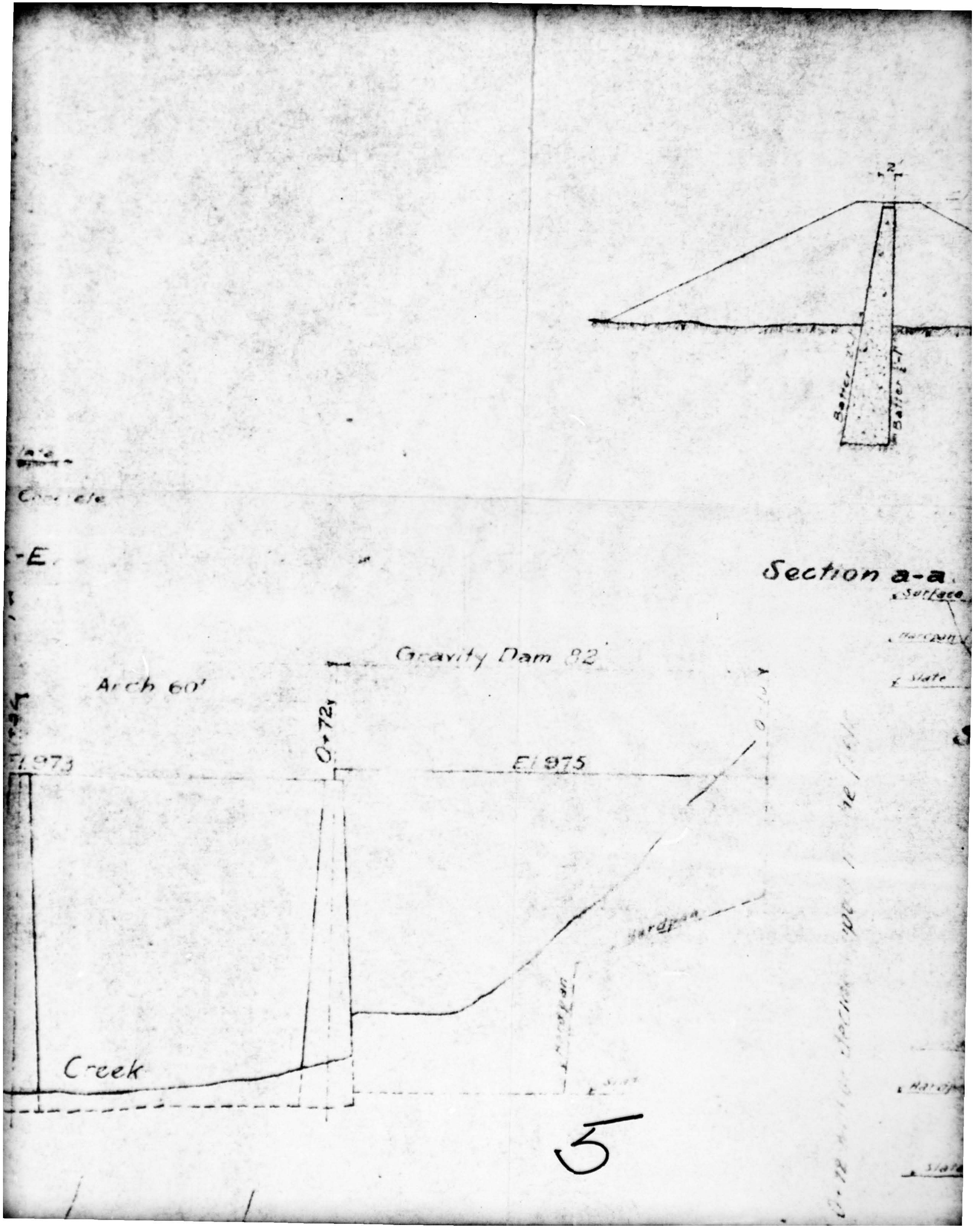
B.



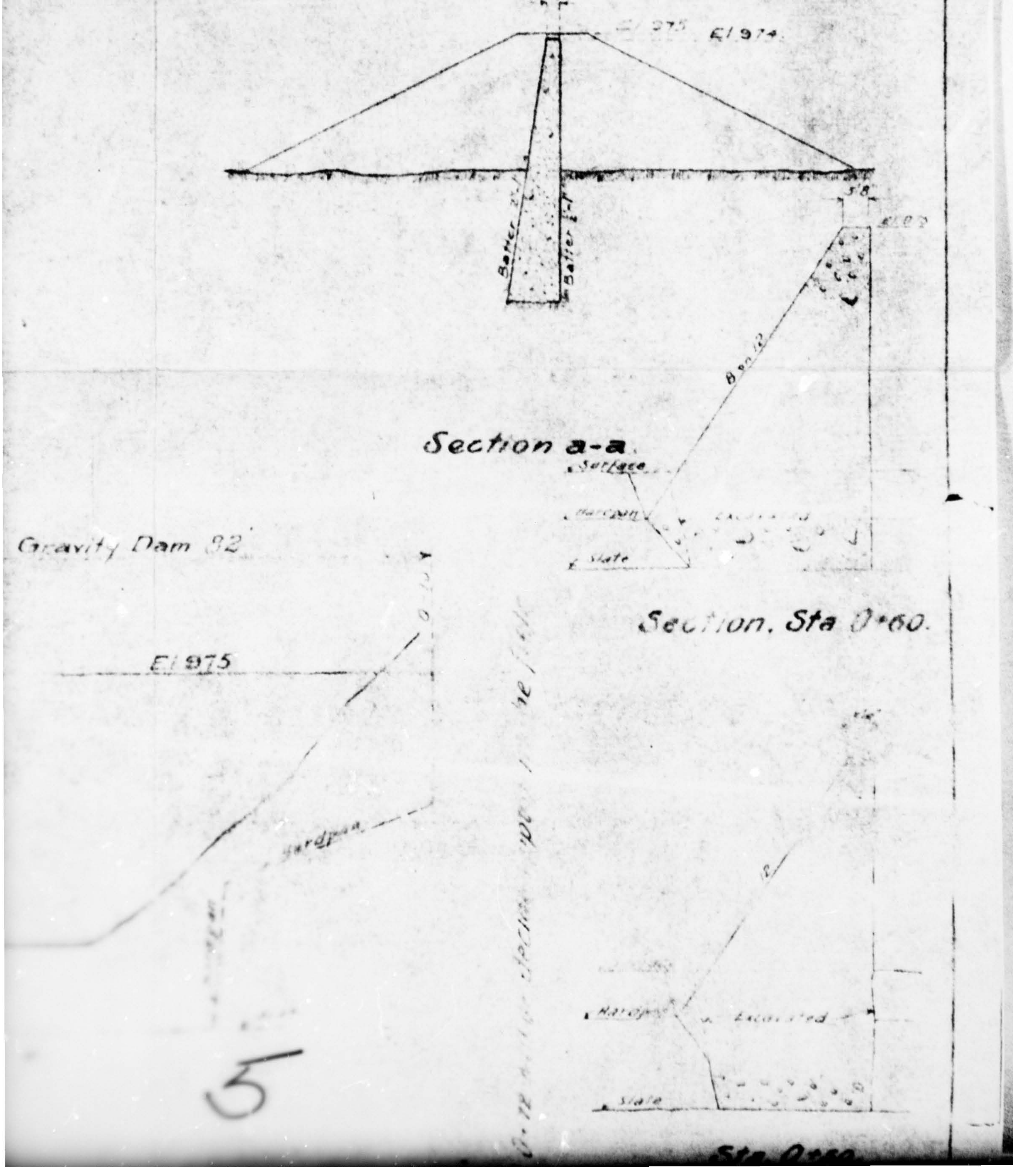
Section D-D.   Section S-S   Section. E

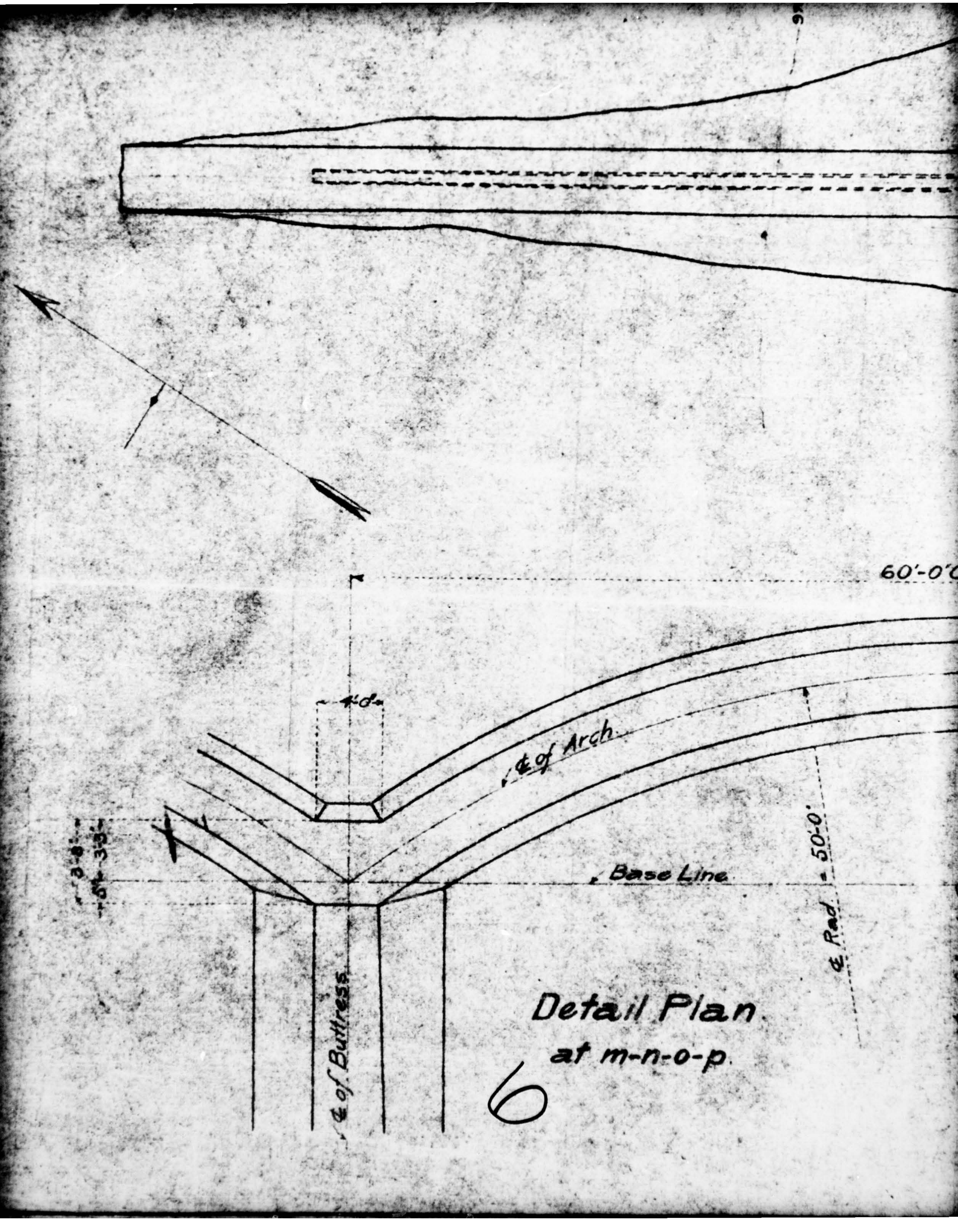


4

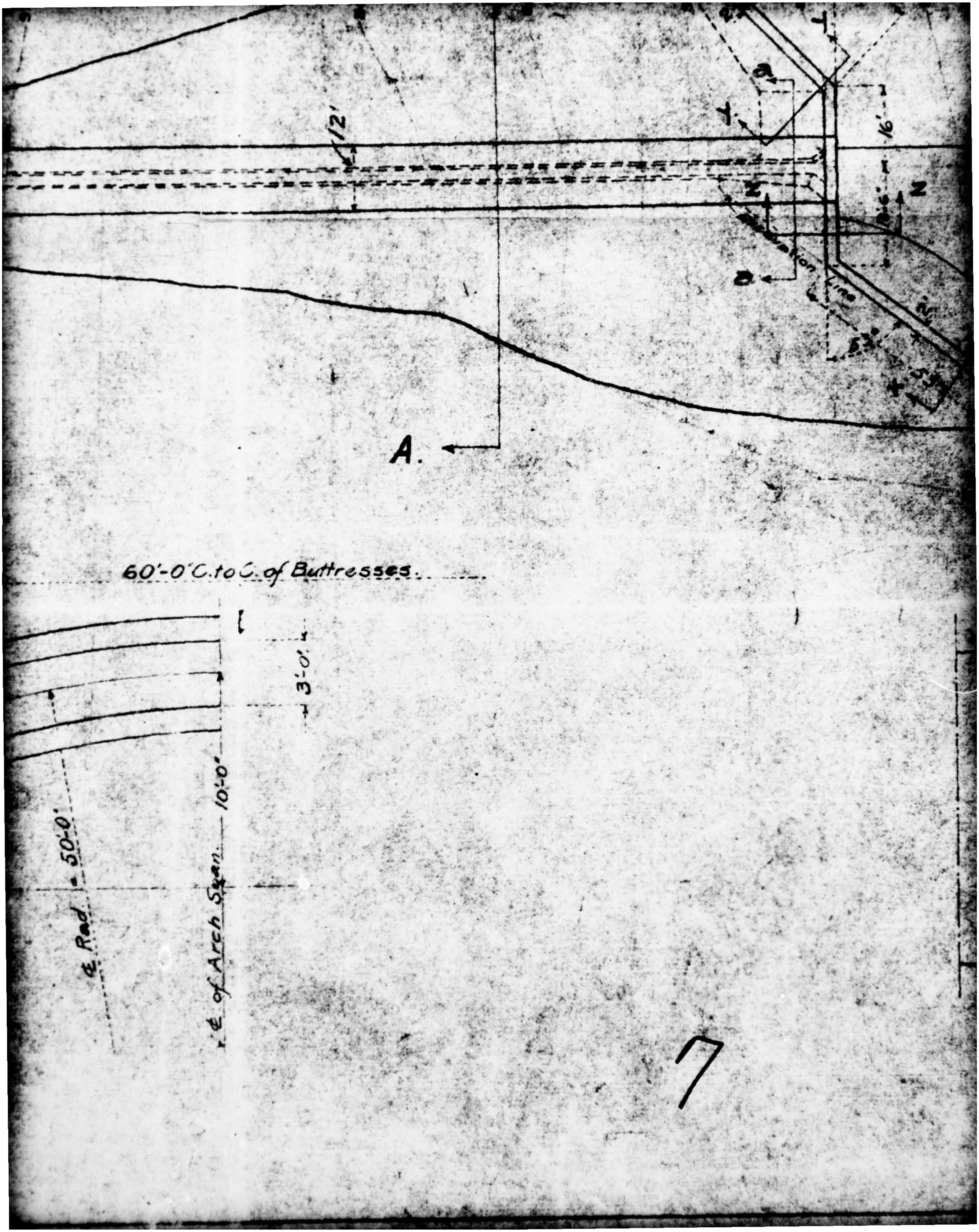


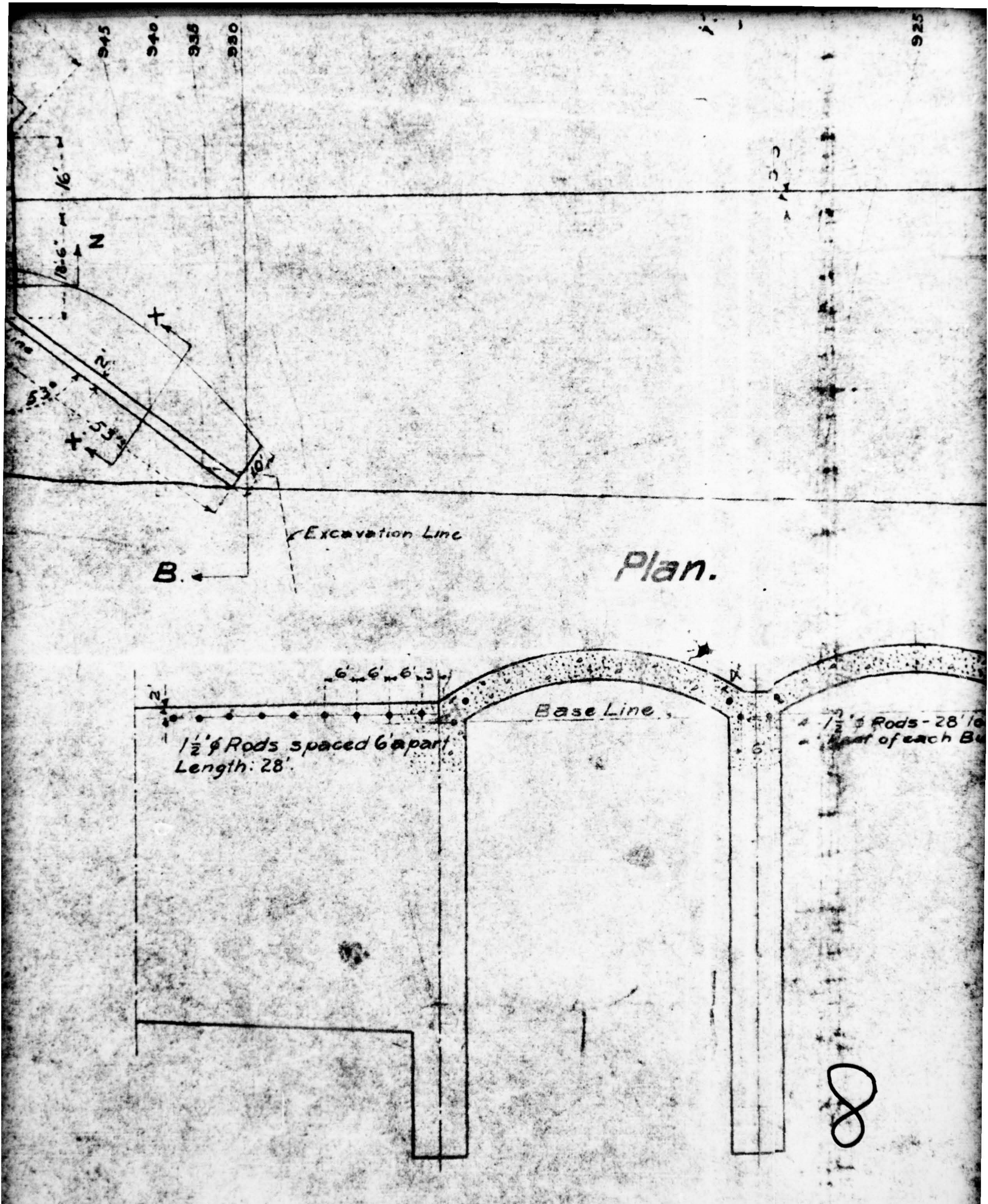
2'
Batter
Batter
Concrete
Section a-a.
Surface
Slate
Gravity Dam 82
Arch 60'
0+72
El 975
Creek
Hardpan
5


El 974
Batter
Batter
Ben 12
Section a-a
Surface
Hardpan
Excavated
Slate
Gravity Dam 82
Section. Sta 0+60
El 975
Hardpan
Hardpan
Excavated
Slate
5

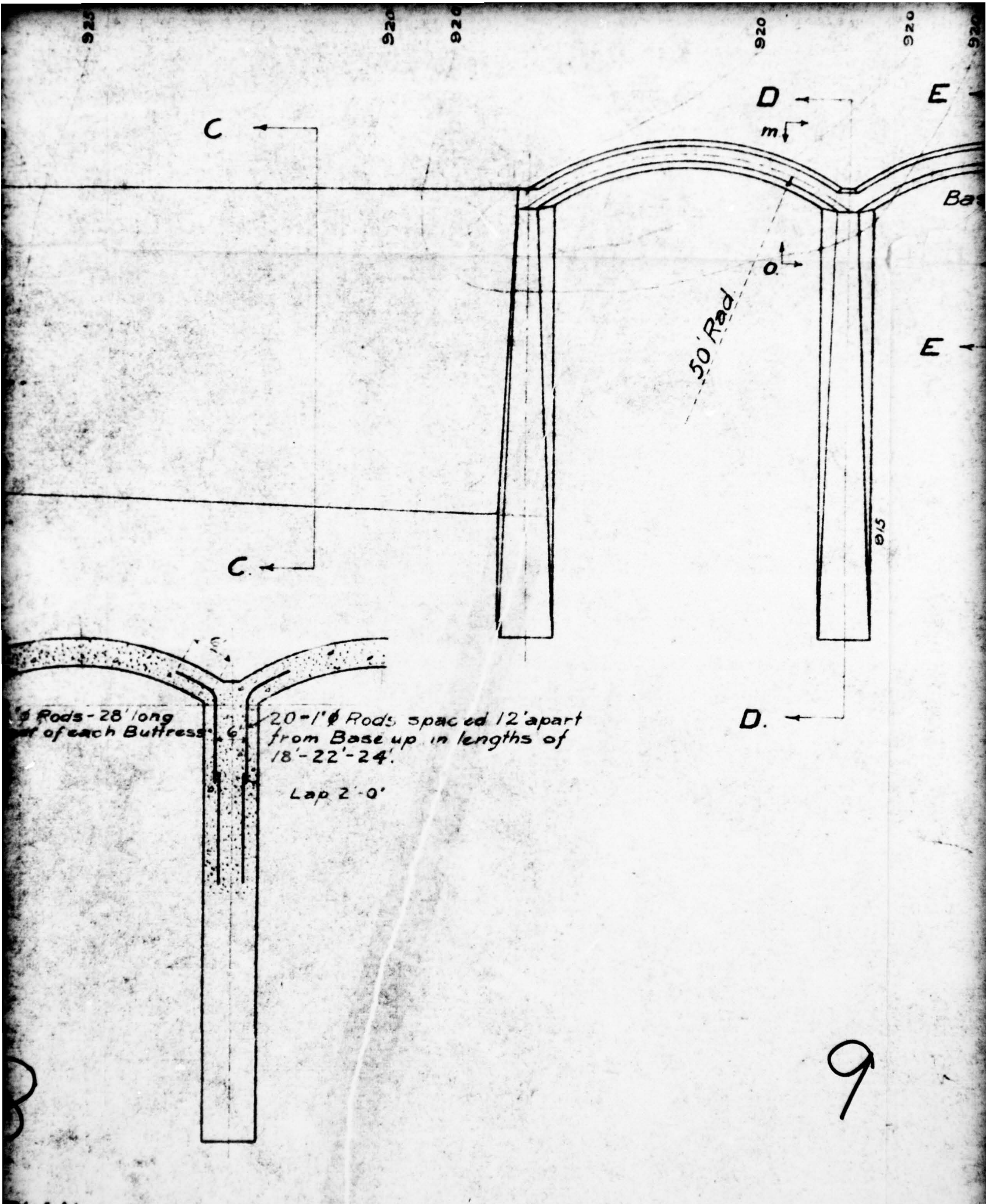


60'-0"
4'-0"
℄ of Arch
3'-6"
3'-3"
5'
Base Line
℄ Rad = 50'-0"
℄ of Buttress
Detail Plan
at m-n-o-p
6


12'
16'
A.
60'-0" C. to C. of Buttresses.
3'-0"
10'-0"
℄ Rad. = 50'-0"
℄ of Arch Span.
7

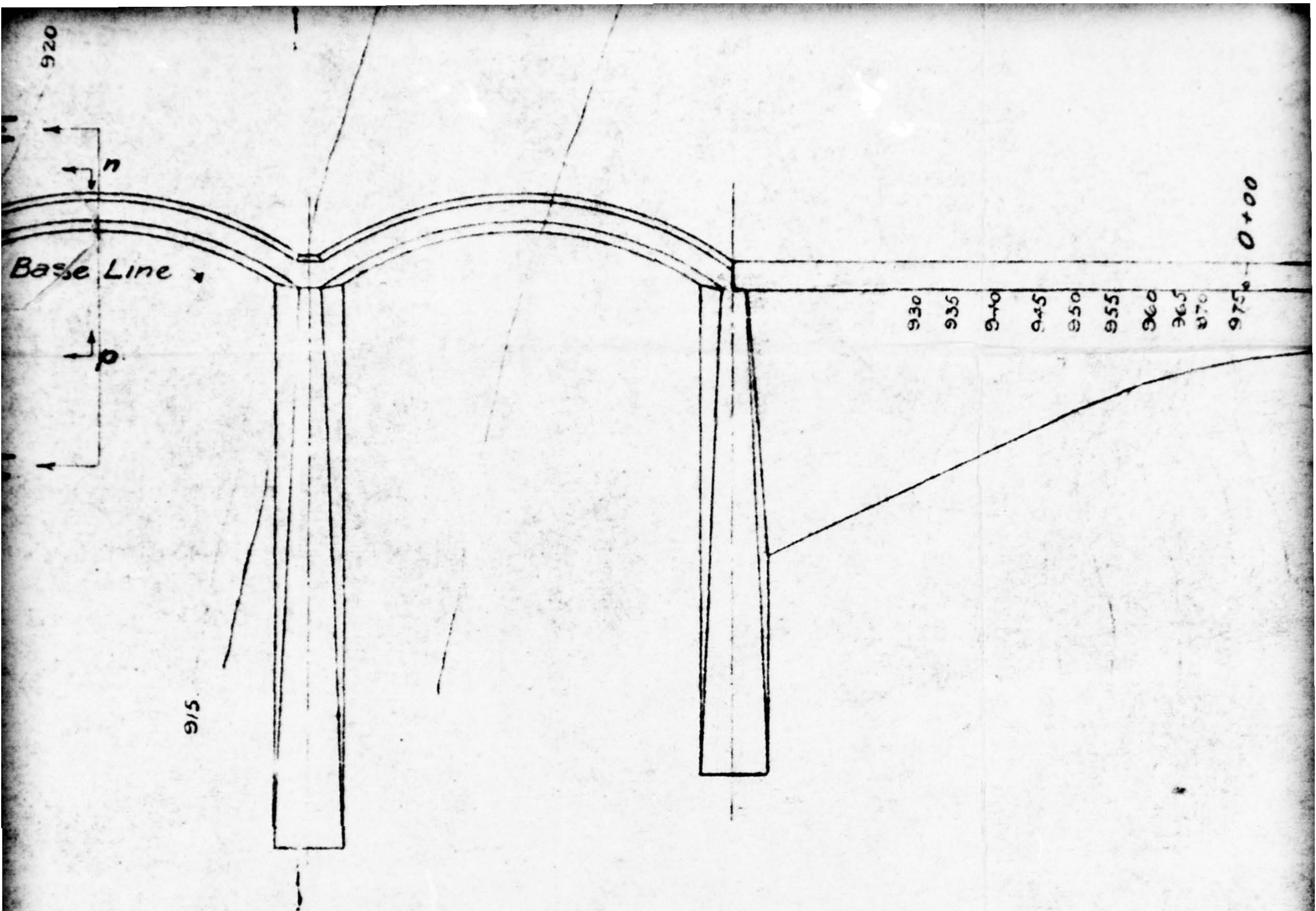


PART FOUNDATION PLAN



925
920
920
920
920
920
C
C.
D
D.
E
E
m
O.
50' Rad.
Bas
915
Rods - 28' long
of each Buttress
6'
6'
20-1"Ø Rods spaced 12' apart from Base up in lengths of 18'-22'-24'.
Lap 2'-0"
9

PLAN.



MOHAWK HY

GARO

Aug

10

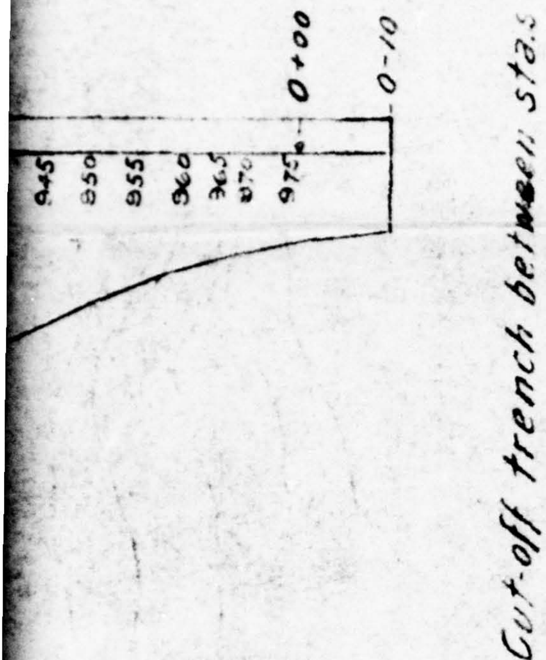
W^m^ Bar

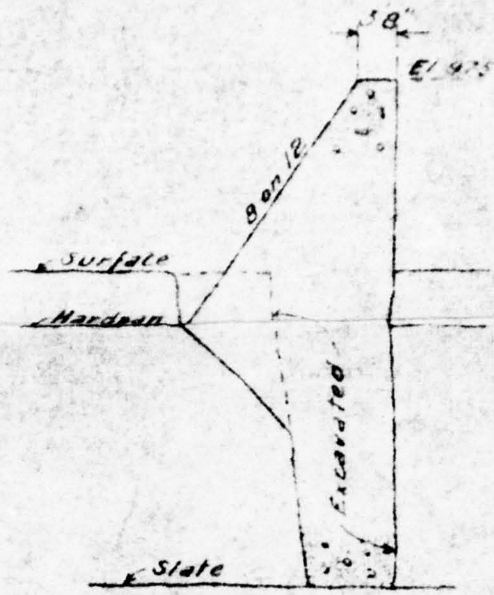
Consult

60, W

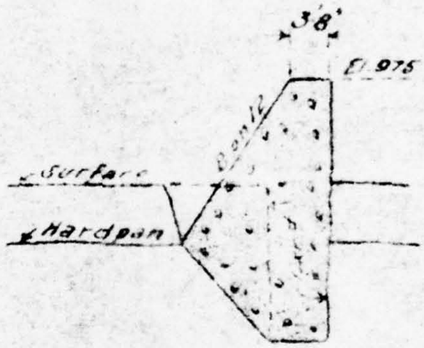
Sc

Revised 8/29 o. Gravity Dam between
Sta.s 0-10 an 0+72 changed.





Sta. 0+30.



Sta. 0+10.

MOHAWK HYDRO-ELECTRIC CO.

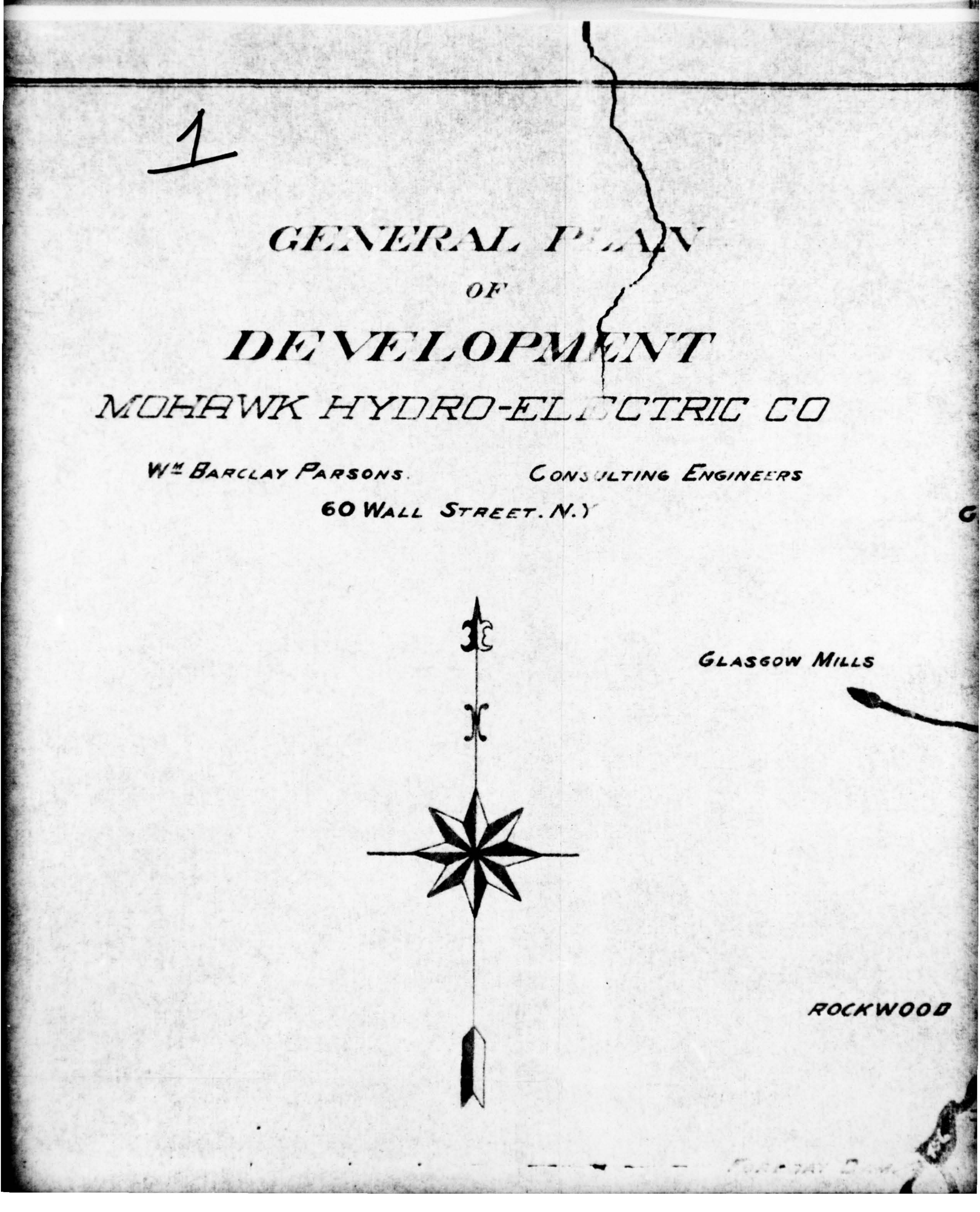
GAROGA DAM.

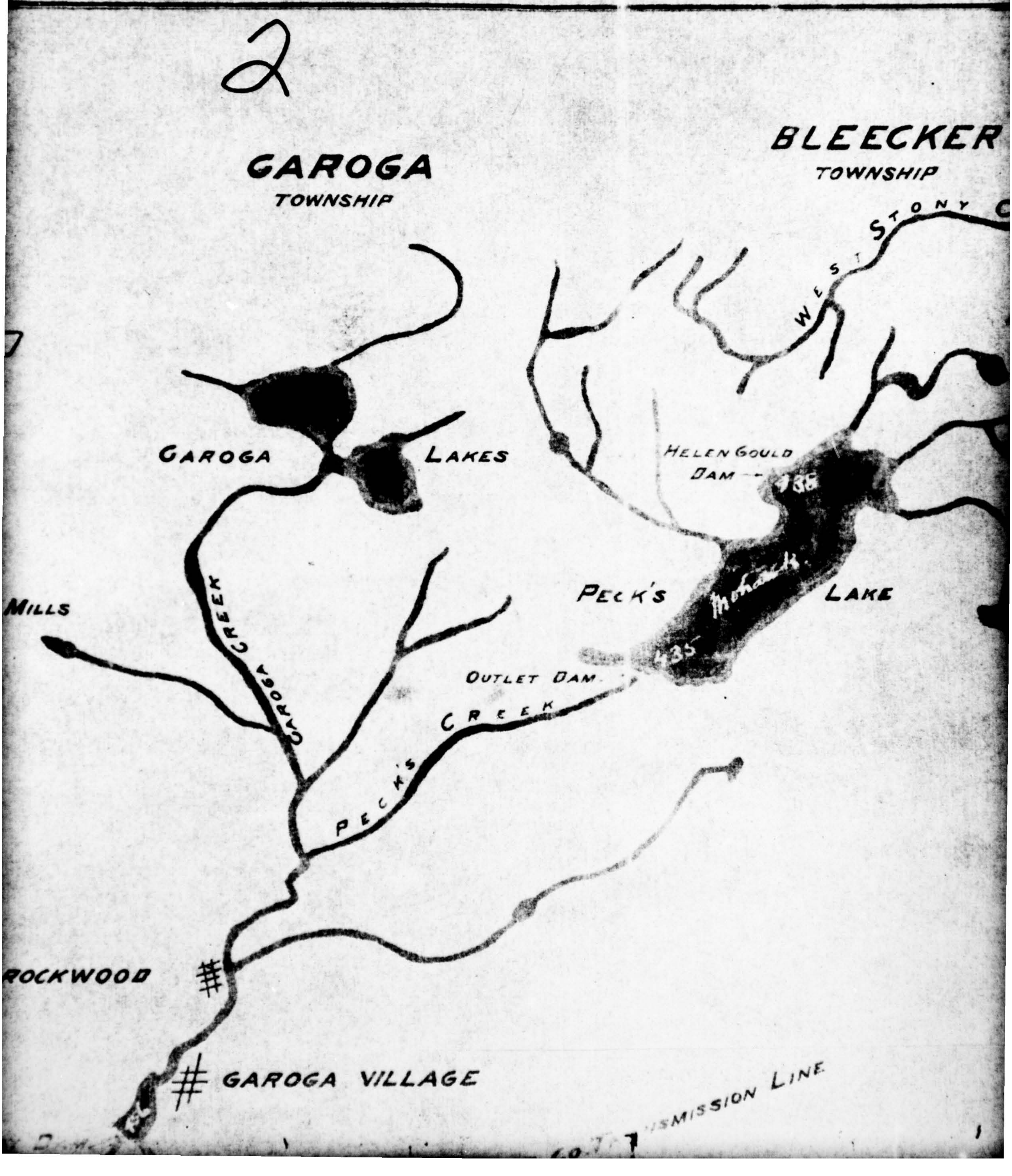
August, 1910.

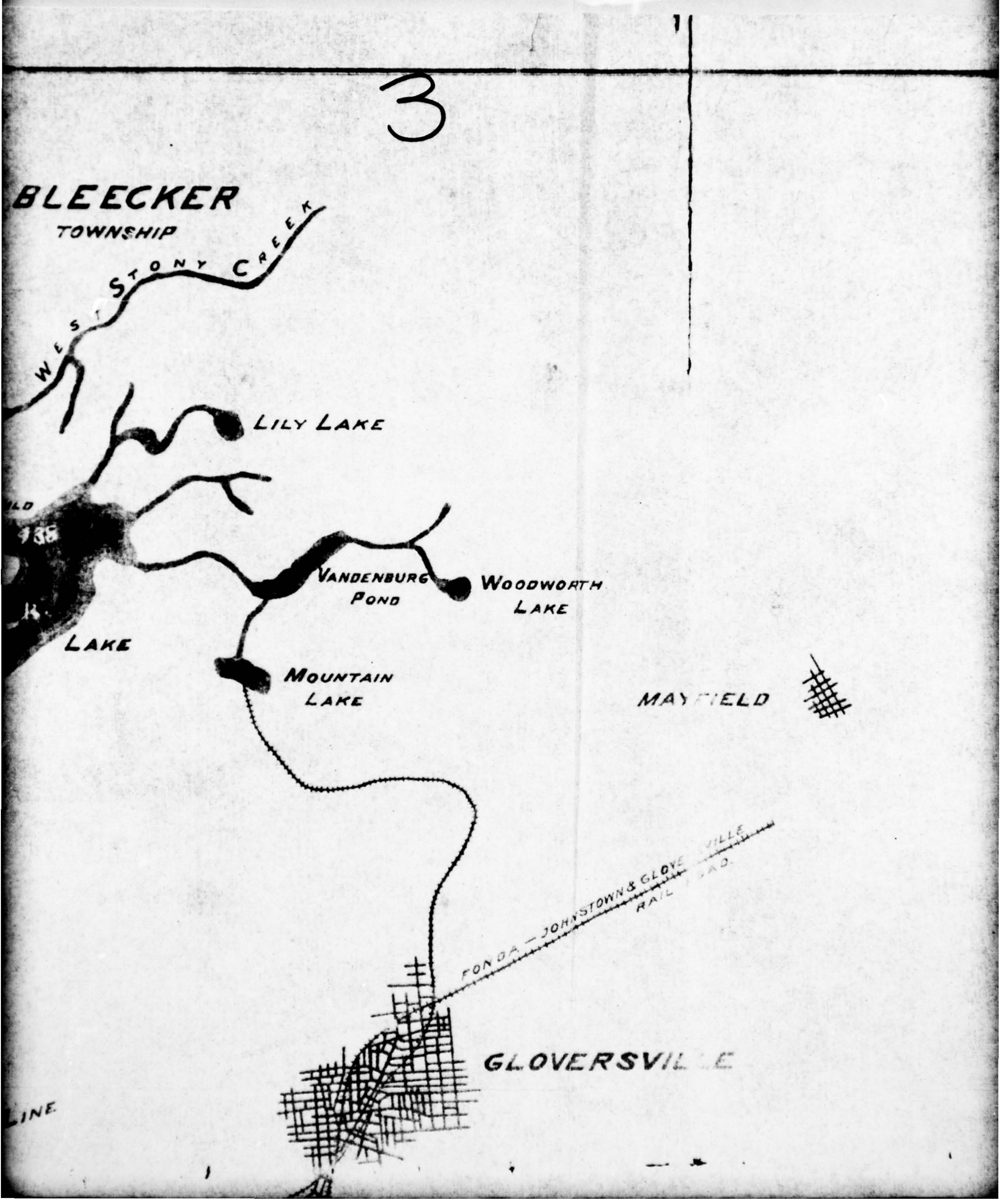
Wm Barclay Parsons,
Consulting Engineers,
60, Wall St., New York.

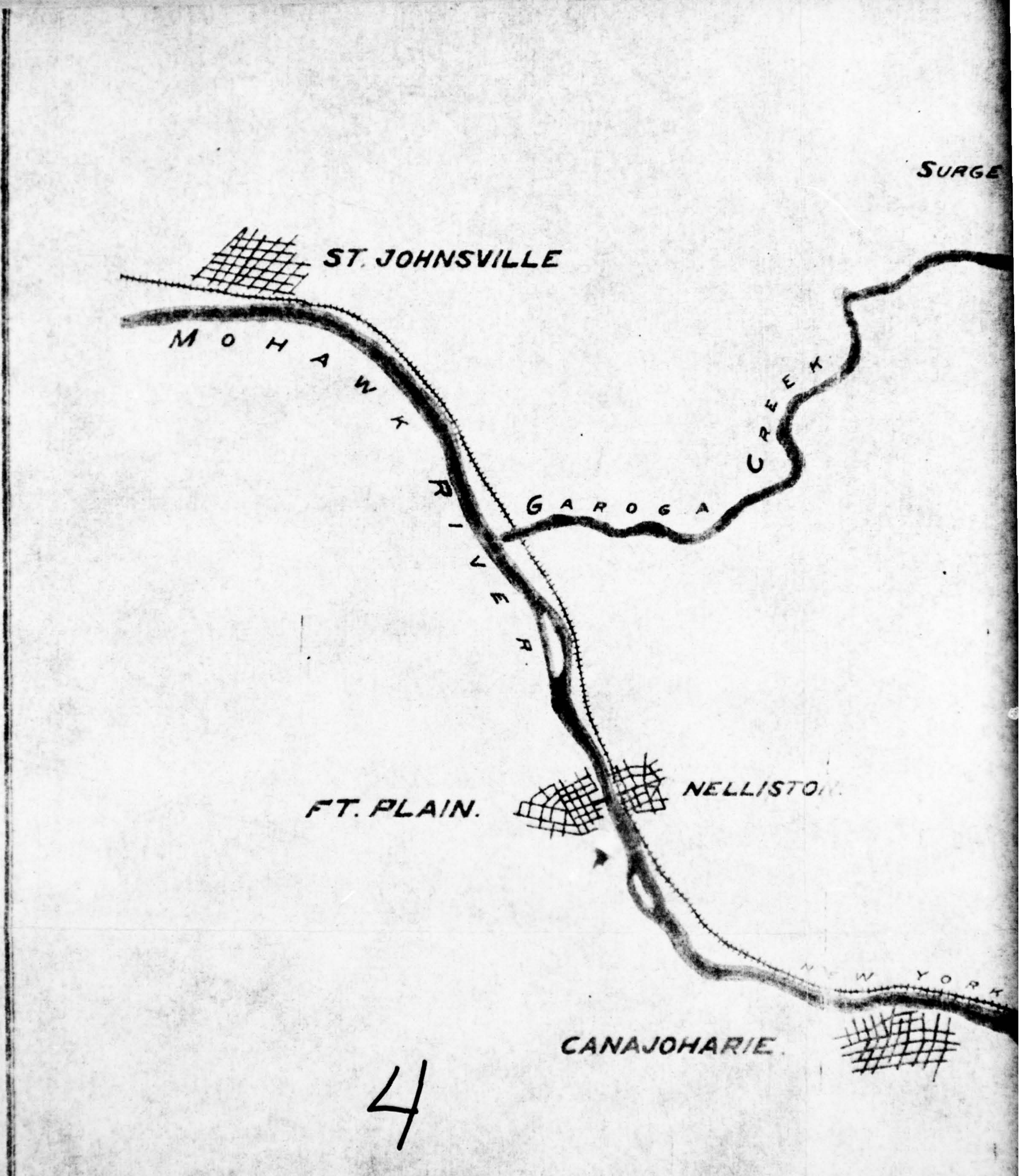
Scale 1"=20'.

11


1
GENERAL PLAN
OF
DEVELOPMENT
MOHAWK HYDRO-ELECTRIC CO
Wm BARCLAY PARSONS. CONSULTING ENGINEERS
60 WALL STREET. N.Y
GLASGOW MILLS
ROCKWOOD


2
GAROGA
TOWNSHIP
BLEECKER
TOWNSHIP
WEST STONY C
GAROGA
LAKES
HELEN GOULD
DAM
MILLS
GAROGA CREEK
PECK'S
LAKE
OUTLET DAM
PECK'S CREEK
ROCKWOOD
GAROGA VILLAGE
SMISSION LINE

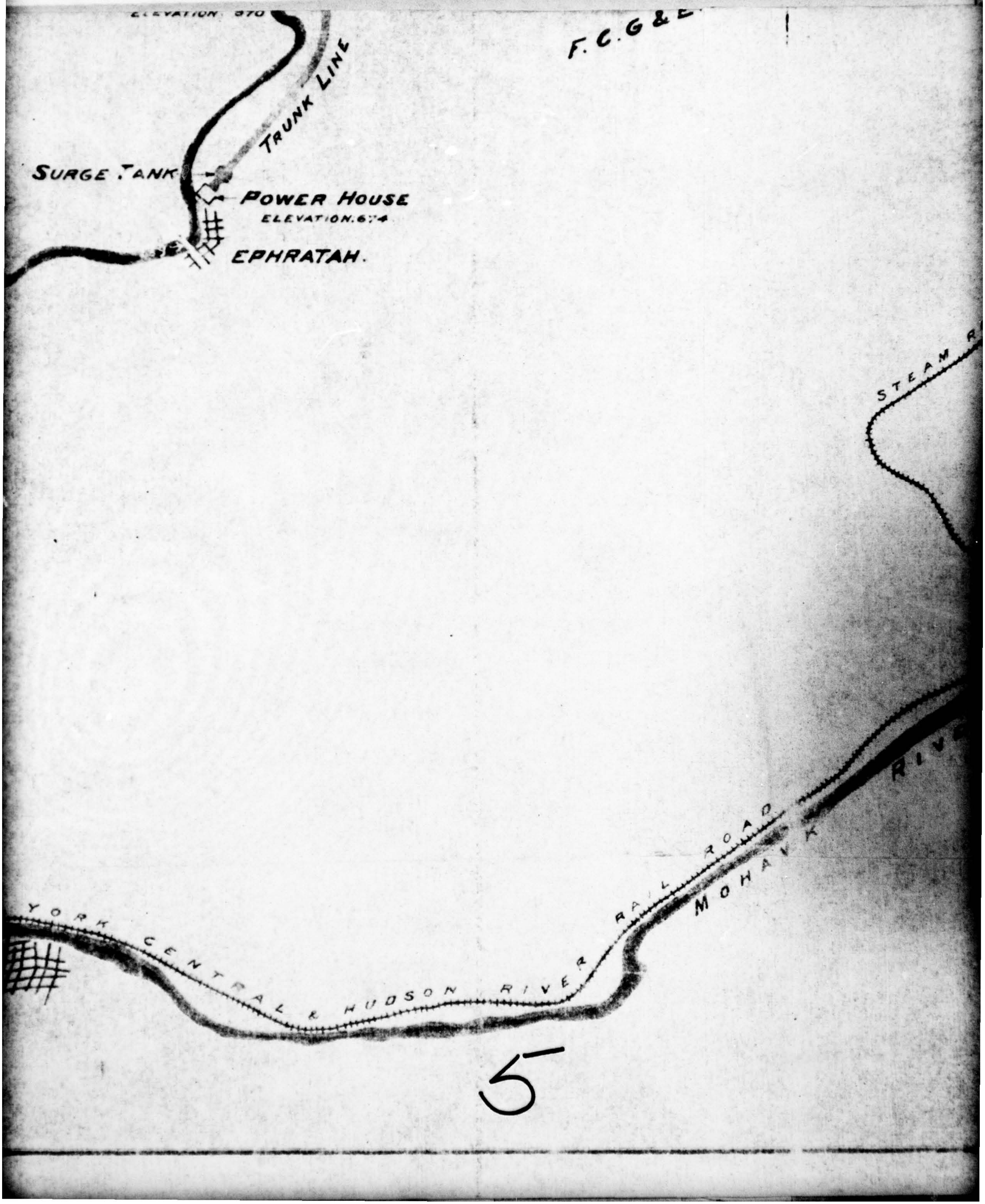

3
BLEECKER
TOWNSHIP
WEST STONY CREEK
LILY LAKE
VANDENBURG
POND
WOODWORTH
LAKE
LAKE
MOUNTAIN
LAKE
MAYFIELD
FONDA — JOHNSTOWN & GLOVERSVILLE
RAIL ROAD
GLOVERSVILLE
LINE

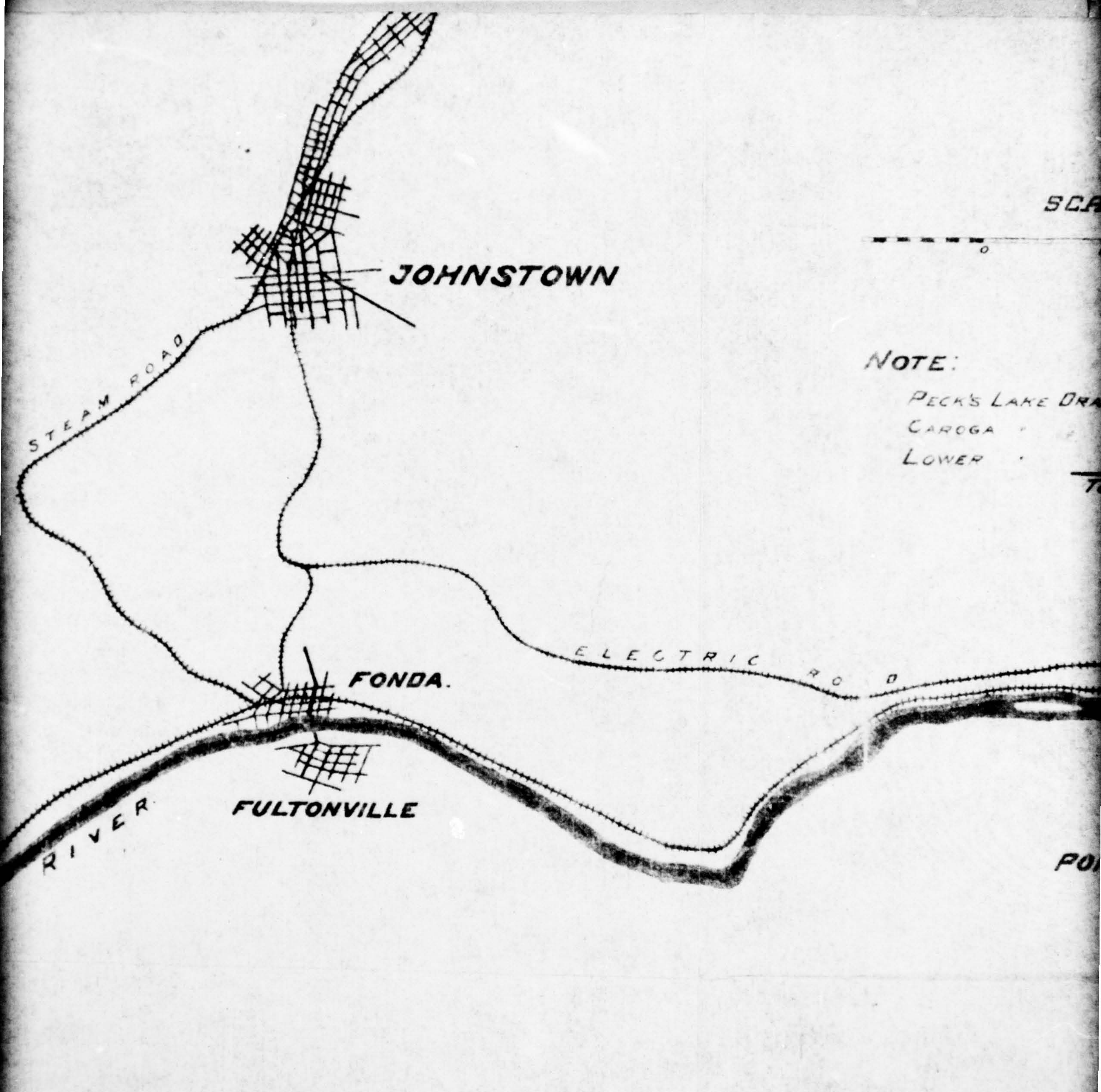


SURGE
ST. JOHNSVILLE
MOHAWK RIVER
GAROGA CREEK
NELLISTO
FT. PLAIN.
NEW YORK
CANAJOHARIE.

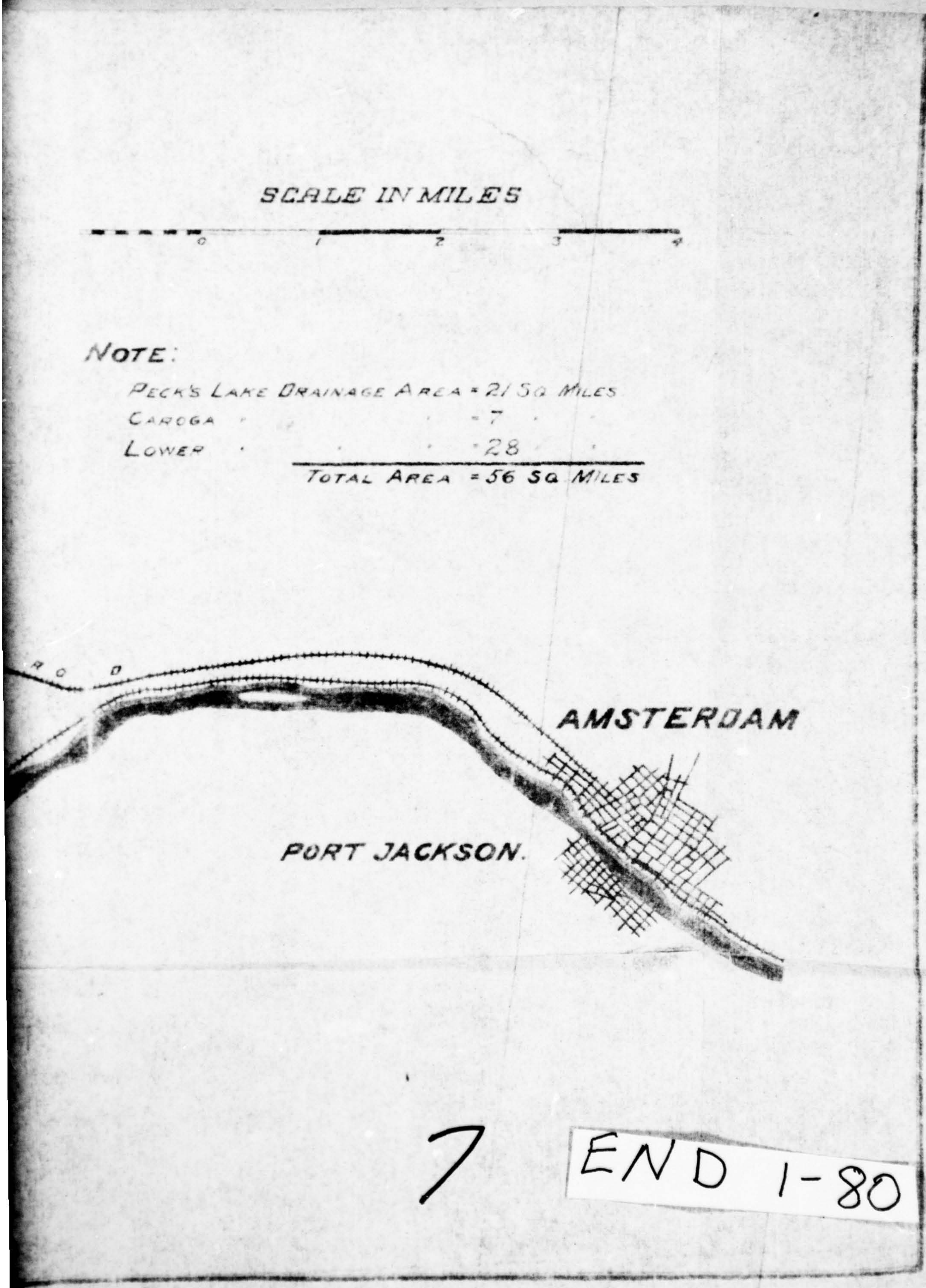
4

1312


F.C.G&
TRUNK LINE
SURGE TANK
POWER HOUSE
EPHRATAH.
STEAM R
YORK CENTRAL & HUDSON RIVER RAIL ROAD
MOHAWK
RIV


JOHNSTOWN
STEAM ROAD
FONDA.
FULTONVILLE
RIVER
ELECTRIC ROAD
SCA
NOTE:
PECK'S LAKE DRA
CAROGA
LOWER
PO

6


SCALE IN MILES
0 1 2 3 4
NOTE:
PECK'S LAKE DRAINAGE AREA = 21 SQ MILES
CAROGA " " " = 7 "
LOWER " " " = 28 "
TOTAL AREA = 56 SQ MILES
R O D
AMSTERDAM
PORT JACKSON.
7
END 1-80