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NEW YORK STATE DEPT OF ENVIRONMENTAL CONSERVATION ALBANY F/G 13/13
NATIONAL DAM SAFETY PROGRAM. INGHAMS DAM (INVENTORY NUMBER NY 1--ETC(U)
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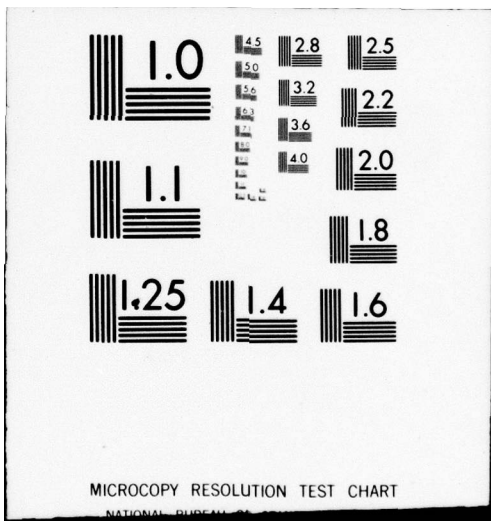
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20. ABSTRACT (Continue on reverse side if necessary and identify by block number) This report provides information and analysis on the physical condition of the dam as of the report date. Information and analysis are based on visual inspection of the dam by the performing organization. The examination of documents and visual inspection of Inghams Dam and appurtenant structures did not reveal conditions which would constitute a hazard to life or property. The dam has a number of problem areas which require further investigation and remedial action. These areas are: → include:		

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(cont) → (1) Additional hydrologic investigations are required to more accurately determine the site specific characteristics of the watershed. Using the Corps of Engineer's Screening Criteria for initial review of spillway adequacy, it has been determined that the embankment would be overtopped for all storms exceeding approximately 24% of the Probable Maximum Flood (PMF) spillway capacity = 21,000 cfs. A flood wave analysis was not conducted since the location of the power plant near the dam was visually determined to be within any flood wave during dam failure. Also, several homes are located along the banks of the downstream channel. The spillway is, therefore, adjudged as "seriously inadequate", and the dam is assessed as unsafe, non-emergency. The classification of "unsafe" applied to a dam because of a seriously inadequate spillway is not meant to connote the same degree of emergency as would be associated with an "unsafe" classification applied for a structural deficiency. It does mean that there appears to be a serious deficiency in spillway capacity, and if a severe storm were to occur, overtopping and failure of the dam could take place, significantly increasing the hazard to loss of life downstream of the dam. A detailed emergency operation plan and warning system should be developed and around-the-clock surveillance should be provided during periods of unusually heavy precipitation.

→ (2) The stability analysis conducted in 1970 did not include the condition of ice loading pressures on the dam. Further, the results of the analysis conducted indicate that the location of the resultant falls outside the middle 1/3 of the base, with the development of tension at the heel for all cases investigated. Therefore, additional investigation is required to determine the type and extent of remedial action required.

(3) The reservoir drain has not been operated since completion of the dam. The drain should be investigated and returned to operating condition; and

(4) An inspection of the spillway should be conducted during no flow conditions to assess the integrity of the spillway. ↗

Within 3 months, these investigations must be initiated and completion scheduled within 1 year of notification. Remedial action should then be completed in the following year.

The following deficiencies were observed which require remedial action:

1. Surface spalling observed on the downstream face and deterioration of the parapet walls should be monitored and repaired as required.
2. If erosion is initiated in the backfill at the north abutment of the spillway, then repair of the training wall at the base of the spillway should be completed.
3. Initiate a program of periodic inspection and maintenance of the dam and appurtenances. Document this information and develop an operations manual.

MOHAWK RIVER BASIN

INGHAMS DAM

**HERKIMER AND FULTON COUNTIES, NEW YORK
INVENTORY NO. N.Y. 183**

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



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NEW YORK DISTRICT CORPS OF ENGINEERS

MARCH, 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 "Probable 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.

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PHASE 1 INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM
INGHAMS DAM I.D. No. NY 183
(formerly Kyser Lake Dam)
DEC #142D-572
MOHAWK RIVER BASIN
HERKIMER-FULTON COUNTY, NEW YORK

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PHASE 1 REPORT
NATIONAL DAM SAFETY PROGRAM

Name of Dam: Inghams Dam (I.D. No. NY 183)
(formerly Kyser Lake Dam)

State Located: New York

County: Herkimer - Fulton

Stream: East Canada Creek
(tributary of the Mohawk River)

Dates of Inspection: October 16, 1978 and March 21, 1979

ASSESSMENT

The examination of documents and visual inspection of Inghams Dam and appurtenant structures did not reveal conditions which would constitute a hazard to life or property. The dam has a number of problem areas which require further investigation and remedial action. These areas are:

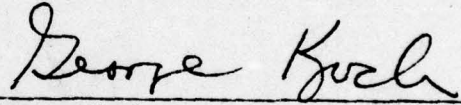
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2. The stability analysis conducted in 1970 did not include the condition of ice loading pressures on the dam. Further, the results of the analysis conducted indicate that the location of the resultant falls outside the middle 1/3 of the base, with the development of tension at the heel for all cases investigated. Therefore, additional investigation is required to determine the type and extent of remedial action required.

3. The reservoir drain has not been operated since completion of the dam. The drain should be investigated and returned to operating condition.
4. An inspection of the spillway should be conducted during no flow conditions to assess the integrity of the spillway.

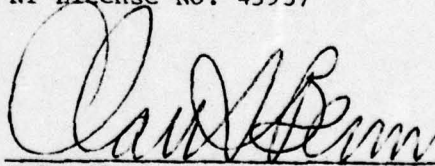
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1. Surface spalling observed on the downstream face and deterioration of the parapet walls should be monitored and repaired as required.
2. If erosion is initiated in the backfill at the north abutment of the spillway, then repair of the training wall at the base of the spillway should be completed.
3. Initiate a program of periodic inspection and maintenance of the dam and appurtenances. Document this information and develop an operations manual.



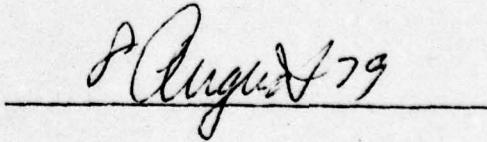
George Koch
Chief, Dam Safety Section
New York State Department
of Environmental Conservation
NY License No. 45937



Col. Clark H. Benn
New York District Engineer

Approved By:

Date:





Photograph #1

Overview of Inghams Dam
Downstream Face



Photograph #2

Overview of Spillway & Tailrace Channel
from top of dam



Photograph #3

Overview of Non-overflow Section
from top of dam

PHASE 1 INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM
INGHAMS DAM I.D. No. NY 183
(formerly Kyser Lake Dam)
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MOHAWK RIVER BASIN
HERKIMER-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

Evaluation of the existing conditions of the subject dam to identify deficiencies and hazardous conditions, determine if they constitute hazards to life and property and recommend remedial measures where necessary.

1.2 DESCRIPTION OF PROJECT

a. Description of the Dam and Appurtenant Structures

Inghams Dam consists of a 480 feet long concrete gravity non-overflow section and a 205 feet long concrete ogee spillway. The maximum height of the dam is 125 feet. The upstream face of the dam is vertical. The crest of the non-overflow section is 12.5 feet wide and consists of two 2.5 feet high parapets. The downstream face of the dam is initially vertical, changing to a 3.2 to 10 batter and then changing to a 6.5 to 10 batter. The structure is founded on bedrock except for a limited section where a deep channel has been eroded in the rock (see drawing in Appendix G). The dam is founded on glacial till at this location.

The ungated concrete spillway is approximately 26 feet high and is also founded on bedrock. Flashboards 4.5 feet in height are used to provide additional storage capacity. The spillway crest is 8.5 feet below the top of the non-overflow section.

b. Location

Inghams Dam is located on the East Canada Creek between Herkimer and Fulton Counties and between the Towns of Manheim and Oppenheim, approximately 4.5 miles east of the Village of Little Falls.

c. Size Classification

The dam is 125 feet high and is classified as a "large" dam. (More than 100 feet).

d. Hazard Classification

The dam is classified as "high" hazard because of the presence of a number of homes along East Canada Creek.

e. Ownership

The dam is owned and operated by the Niagara Mohawk Power Corporation, 300 Erie Boulevard West, Building D-2, Syracuse New York, 13202, Tel: (315) 474-1511, Ext. 1869.

f. Purpose of the Dam

The dam provides storage for power development. In addition, the reservoir is used for recreational purposes.

g. Design and Construction History

The dam was constructed in 1911. The dam was designed by Viele, Blackwell and Buck, Consulting Engineers, 49 Wall Street, New York, NY. No engineering information pertaining to construction history was available.

h. Normal Operating Procedures

Water stored in the reservoir is used for the generation of electricity by the two turbines housed in the power plant approximately 600 feet below the dam. The water from the reservoir passes through the intake located in the gate house near the south abutment. Flow regulation is provided by an electrically operated slide gate at the gate house. Water passes through a screen to the intake chamber then to the 9 feet diameter steel penstock and through the surge tank to the power house. Flow is distributed to the turbines by hydraulically operated wicket gates. Flow not used in the generation of electricity is allowed to spill over the flashboards.

1.3

PERTINENT DATA

a. <u>Drainage Area</u> (sq. mi)	278
Height of dam (feet)	125
b. <u>Discharge at Dam Site</u> (cfs)	
<u>Maximum known Flood</u>	19,300
Spillway at Design Pool (El. 665.5)	20,000
Spillway at Maximum Pool (El. 665.8)	21,000
Maximum Capacity of Reservoir drains	0
Total Discharge, Max. Pool	21,000
Average Daily Discharge	Unknown
c. <u>Elevation</u> (ft. above MSL-Datum)	
Top of Dam	665.8
Design Pool	665.5
Spillway Crest	657.3
Tailrace Channel	539.0
Invert Reservoir Drain Outlet	560.0
d. <u>Reservoir</u>	
Length of maximum Pool, miles	2.5
Length of Shoreline (Spillway Crest) miles	5.7
Surface area (Spillway Crest) acres	188

e. Storage, (Acre-feet)

Spillway Crest	3,100
Maximum Design Pool	4,500
Top of Dam	4,600

f. Dam

Type:	Gravity	Concrete
Length (ft.)		480
Upstream slope		Vertical
Downstream slope		1:0.65
Impervious Core	Concrete core wall keyed to the existing ground.	

Crest elevation, ft.	665.8
Crest Width, ft.	12
Grout curtain	None

g. Spillway

Type:	Ogee
Length, ft.	205
Crest Elevation MSL	657.3
Upstream Channel	Not Visible
Downstream Channel	Bedrock and in good condition.

h. Regulating Outlet

The 9 feet diameter penstock is regulated by an electrically operated slide gate at the upstream side of the dam. Maximum flow through penstock is 700 cfs. Most efficient flow is 600 cfs. An adjacent 6 feet diameter penstock has been plugged since the completion of construction.

i. Reservoir drain

2-6.5 feet diameter drains are inoperative.

SECTION 2: ENGINEERING DATA

2.1 DESIGN

a. Geology

The Inghams 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 soils in the vicinity of the dam are Cazenovia and Mohawk. These soils are of glacial till origin and residuum from shale, siltstone limestone and small amounts of sandstone. They consist generally of stony or shaley silts and clays. Boulders are common. Rock outcrops in the spillway downstream channel and at both abutments of the dam. The internal drainage is poor and the rate of run-off dependent upon the degree of slope.

c. Dam and Appurtenant Structures

The dam was designed by Viele, Blackwell and Buck Consulting Engineers, 49 Wall St., New York, NY. Drawing Number R-3573, "Plan and Elevation of Dam and Spillway" is included in Appendix G. The design includes a concrete gravity dam and ogee spillway, founded on and keyed into bedrock till.

2.2 CONSTRUCTION RECORDS

The only information available concerning the construction of the dam is the year of construction (1911) and a newspaper report included in Appendix G. It was noted that a field change was made during construction from that shown on the drawing included in Appendix G. The south abutment of the dam was extended 63 feet southward from the intake structure and from that point, the concrete core wall was extended 108 feet southward. Elevations shown on this drawing are 6.7 feet higher than the actual U.S.G.S. elevations.

2.3 OPERATION RECORD

All 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 Inghams Dam and the surrounding watershed was conducted on October 16, 1978 and March 21, 1979. The weather was clear and temperatures ranged in the thirties. The reservoir level at the time of the first inspection was 662.3 (USGS) due to the presence of 4.5 feet high flashboards and 657.8 (USGS) during the second inspection due to the loss of the flashboards caused by spring runoff.

b. Concrete Non-Overflow Section

The non-overflow section has not been rehabilitated since its original construction. There are no signs of horizontal or vertical misalignment and no indication of distress or cracking. No seepage was evident at any location. Some surface spalling of the concrete was evident on the downstream face with the maximum depth estimated to be 3 inches and the average depth of approximately 1 inch (see photographs #4,5 & 6). Surface spalling (see photographs #8,9 & 10) was also apparent on the parapet walls. No internal drainage system was provided. However, 6 borings were progressed in June 1970 to determine uplift pressures and these holes have remained open. No flow was observed from these holes. Boring logs for this work have been included in Appendix F "Stability Analysis". The intake structure for the penstock, located near the south end of the dam is in good condition. Some minor ice caused damage was apparent in the concrete walls of intake structure. (See photographs #8 & 12).

c. Spillway

The concrete ogee spillway section has not been repaired since its original construction. No signs of distress or movement were observed. Flow at the time of both inspections prohibited the complete examination of the spillway. A section of the training wall adjacent to the toe at the north end of the spillway was missing (see photographs #16 & 20). The remainder of this wall is in poor condition (see photographs #17, 18 and 21). The original purpose of the wall was to protect the north slope from erosion. Since the flow from the spillway generally does not reach the slope (the deteriorated wall provides little protection now) repair of the wall is not considered significant unless erosion is initiated in the vicinity of the north abutment of the spillway. Flashboards (4.5 feet high) observed during the first inspection were destroyed prior to the second inspection during a period of high runoff (see photographs #16 & 20). The tail race channel is exposed bedrock and appeared to be in good condition.

d. Regulating Outlets

Two penstock intakes are located in the non-overflow section of the dam. The 9 feet diameter penstock has one electrically operated slide gate located in the intake structure and is operational. (See photograph #8). An additional penstock intake was constructed for future electrical generating capacity. However, the expansion of facilities was not undertaken and this 6' diameter intake remains plugged.

e. Reservoir Drain

The two reservoir drains located at the center of the dam are 6.5 feet indiameter. The southerly pipe has been plugged and the northerly pipe has not been operated since the dam was completed. A manually operated butterfly valve is thought to control the flow of the northerly pipe and substantial seepage was observed discharging from this pipe. The controls for this valve have been destroyed. (See photograph #6).

f. Downstream Channel

The downstream channel is primarily bedrock formed and appears in good condition. (See photographs #19 & 22). A bridge located below the power house controls the flow of the channel. Some debris was observed in the downstream channel.

g. Reservoir

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

3.2

EVALUATION OF OBSERVATIONS

No deficiencies were observed which would indicate that the dam is in imminent danger or which may develope into a hazardous condition. All deficiencies observed are of a minor nature and may be corrected by maintenance forces, with the exception of the inoperative reservoir drain.

SECTION 4: OPERATION AND MAINTENANCE PROCEDURE

4.1 PROCEDURE

Inghams Dam is a power dam for Niagara Mohawk Power Corporation. A 9 feet diameter steel pipe (penstock) carries water from the reservoir to the power plant approximately 600 feet below the dam. Flow through the penstock is controlled by an electrically operated slide gate located in the intake structure near the south abutment of the dam. Hydraulically operated wicket gates at the turbines control the flow from the penstock. Two 6.5 feet reservoir drains, one plugged and the other inoperative, have not been used since completion of the dam. The inoperative drain is believed to be controlled by a leaking butterfly valve, the mechanical controls of which have been destroyed.

4.2 MAINTENANCE OF DAM

There is no operation and maintenance manual for the dam. The dam is maintained in generally good condition. Some deterioration of the parapet walls and the downstream face of the non-overflow section was observed. A section of the deteriorated training wall at the northern toe of the spillway is missing.

4.3 MAINTENANCE OF OPERATING FACILITIES

The penstock intake slide gate is operational. The reservoir drain has not been operated since 1911.

4.4 WARNING SYSTEM IN EFFECT

There is no warning system in effect or in preparation.

4.5 EVALUATION

The structure is in need of some maintenance. A program of periodic inspection and maintenance of the dam and appurtenances should be initiated. This information should be documented for future reference. Also, develop an operations manual.

SECTION 5: HYDRAULIC/HYDROLOGIC

5.1 DRAINAGE AREA CHARACTERISTICS

Inghams Dam is located on the East Canada Creek about 3.5 miles northeast of Manheim between Herkimer and Fulton Counties, New York. The drainage area at dam site is 278 square miles. The topography is characterized by mild to steep slopes interspersed by swamps and lakes.

5.2 ANALYSIS CRITERIA

Information on the PMF for Inghams 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 Inghams Dam and its watershed are located within the sub-areas 13 and 14 of the Mohawk River Basin, Little Falls, N.Y. to Mouth. The computed outflow resulting from one half PMF and PMF are 45,000 cfs and 89,000 cfs respectively (Appendix D). Since the storage is very little compared to inflow, the outflow is about the same as inflow.

5.3 SPILLWAY CAPACITY

The ungated concrete ogee spillway is 205 feet long and the maximum head possible between the crest of the spillway and the top of the dam is 8.5 feet. The lake level is raised by 4 feet 6 inches high collapsible flashboards erected on the crest of the spillway. The discharge capacity (source: discharge capacity curve supplied by Niagara Mohawk Power Corporation - see Appendix D) of the spillway is 21,000 cfs when the lake level is at the top of the dam (El. 665.8).

5.4 RESERVOIR CAPACITY

The capacities of the Inghams Lake at spillway level and at flashboard level are 3,100 acre-feet (AF) and 3,900 AF respectively. These volumes are above the intake level. The computed surcharge storage between the crest of the spillway and top of the dam is 1,500 AF which is equivalent to a runoff depth of .1 inch over the entire drainage area. Therefore, the effect of the reservoir in reducing the peak inflow is negligible for all practical purposes.

5.5 FLOODS OF RECORD

The highest flow occurred on March 18, 1936. The flow was 14,750 cfs and resulted in an elevation of 663.8 at the dam site. However, the highest flow at the dam site occurred as a result of an upstream dam failure on or about October 4, 1945. The flow was not recorded at the dam site but amounted to 19,300 cfs at the Dolgerville Gaging Station (drainage area = 261 square miles) approximately 3.5 miles upstream of Inghams Dam and floodwaters reached the elevation of El. 666.3 at the dam site. No records of low flows were recorded.

5.6 OVERTOPPING POTENTIAL

The PMF and $\frac{1}{2}$ PMF peak outflows are 89,000 cfs and 45,000 cfs respectively, compared to spillway capacity of 21,000 cfs. Hence, the dam will be overtopped by 7.7 feet and 3.5 feet of water due to PMF and $\frac{1}{2}$ PMF respectively.

5.7 EVALUATION

The spillway is considered inadequate to pass all storms in excess of 24% of the PMF. With regard to structural stability, the non-overflow section does not meet the minimum recommended factors of safety as defined by the Corps of Engineers. In the event of dam failure, the flood wave would pose a significant danger to the residents and workers within the power plant.

The spillway is, therefore, adjudged as seriously inadequate, and the dam is assessed as unsafe, non-emergency.

SECTION 6: STRUCTURAL STABILITY

6.1 EVALUATION OF STRUCTURAL STABILITY

a. Visual Observations

The visual inspections did not indicate any signs of major distress in connection with the dam.

b. Design and Construction Data

No design computations or construction information prior to completion of the dam in 1911 are available. However, a structural stability analysis was conducted by Uhl, Hall & Rich, Consulting Engineers, Boston, Massachusetts in 1970. The results of this investigation are as follows:

- Case 1 - Normal Water surface El. 663.3 Tailwater 563.3
- Case 2 - Design Flood (20,000 cfs) El. 665.3 Tailwater 563.3
- Case 3 - Probable Maximum Flood (100,000 cfs) El. 675.5 Tailwater 573.3
- Case 4 - Seismic Condition - Normal Water El. 663.3 Tailwater 653.3
Earthquake Force = 0.05g horizontal

All cases assumed uplift as shown in Plate I Appendix F, which was the actual measured uplift conditions.

Case	Shear Friction Safety Factor	Location of Result from Toe	Overturning Safety Factor
1	5.75	19.6	1.35
2	5.60	17.8	1.30
3	4.88	7.0	1.10
4	5.04	14.5	1.24

These results indicate that the structure does not meet the minimum recommended factors of safety for overturning and location of resultant as defined by the Corps of Engineers "Guidelines" for all cases analyzed. The middle 1/3 of the base ranges from 26.3 to 52.7. Since all resultants analyzed fall outside the middle 1/3, tension will result at the heel. Also, no ice loading conditions were investigated. Further information concerning the stability analysis is included in Appendix F.

It is recommended that additional structural stability investigations be conducted and remedial measures instituted if necessary to achieve the minimum requirements for stability.

c. Operating Records

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

d. Post-Construction Changes

To aid in the structural stability analysis, 6 borings were progressed in June 1970 on the downstream face of the non-overflow section to determine the actual uplift pressures at the base of the dam. These boring logs have been included in Appendix F. Analysis of the water levels in the borings indicated that the actual uplift pressures were lower than uplift pressures generally assumed in design (100% at heel to 100% tailwater). The reduced uplift considerable improved the predicted factors of safety.

e. Seismic Stability

The dam is located in Seismic Zone 2. A seismic analysis was conducted in Case 4 described above.

SECTION 7: ASSESSMENT/RECOMMENDATIONS

7.1 ASSESSMENT

a. Safety

The Phase 1 inspection of Inghams Dam revealed that the spillway is "seriously inadequate" and outflows from any storm in excess of 24% of the PMF would overtop the dam, resulting in a significant increase of the hazard to downstream residents. For this reason, the dam has been assessed as unsafe, non-emergency.

In addition, the structural stability analysis of the non-overflow section indicates that the factors of safety for all cases fall below the minimum requirements of the Corps of Engineers.

b. Adequacy of Information

The information available is adequate for Phase 1 inspection purposes. It should be noted that the design and construction information is extremely limited.

c. Urgency

The following investigations should be initiated within 3 months and completed within 1 year of notification: a detailed hydraulic/hydrologic analysis using the site specific characteristics of the watershed, structural stability including ice loading conditions, and investigation of the reservoir drain and its return to operational status. Remedial action as a result of these investigations should be completed in the following year. The remaining recommended measures described below should be completed during the next construction season.

d. Need for Additional Investigation

Investigate the hydraulic/hydrologic character of the watershed, and the structural stability of the dam including the case concerning ice loading pressures. Investigate the condition of the existing reservoir drain and institute remedial measures to restore the drain to its proper operational capacity. Observe the spillway under no flow conditions. The NYS Department of Environmental Conservation Dam Safety Section will be available to assist in this investigation. Telephone: (518) 457-6310.

7.2 RECOMMENDED MEASURES

- a. Results of the required investigations will determine the remedial measures necessary.
- b. Surface spalling observed on the downstream face and deterioration of the parapet walls should be repaired as required.
- c. Repair the spillway training wall at the base of the spillway if erosion is initiated at the north abutment.
- d. Initiate a program of periodic inspection and maintenance of the dam and appurtenances. Document this information for future reference. Also, develop an operations manual.
- e. A detailed emergency operation plan and warning system should be developed. Also, around-the-clock surveillance should be provided during periods of unusually heavy precipitation.

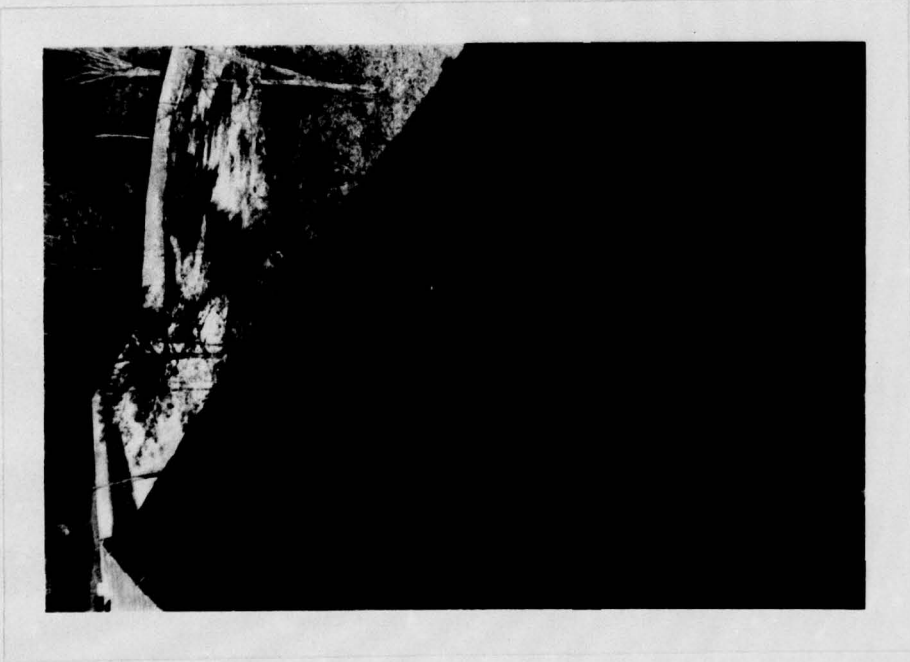
APPENDIX A

PHOTOGRAPHS



Photograph #5

Non-Overflow Section Looking South



Photograph #4

Non-Overflow Section Looking North



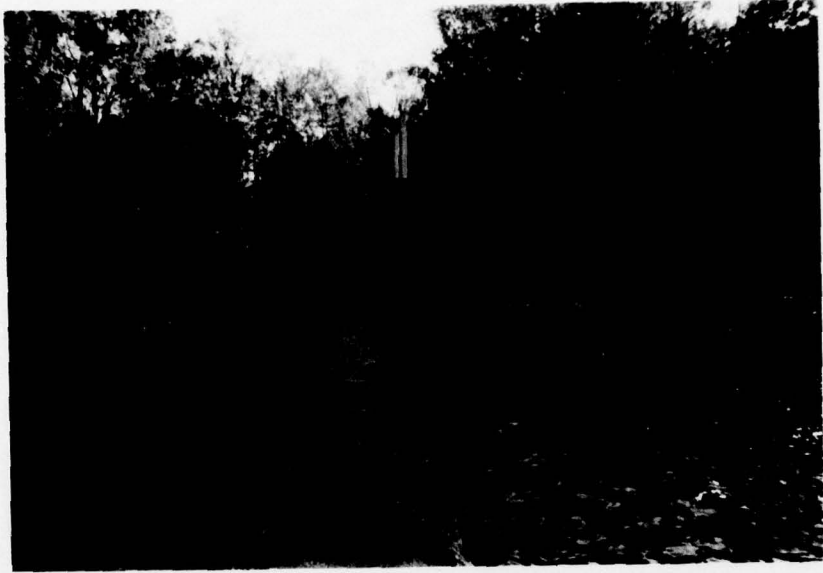
Photograph #6

Non-Overflow Section viewed from top of dam
note leaking reservoir drain



Photograph #7

Old Photograph (undated)
probably during construction



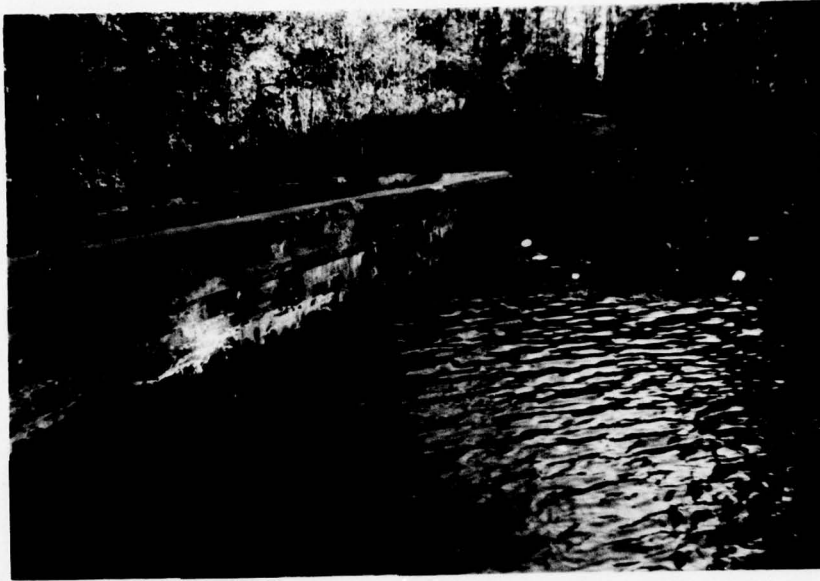
Photograph #8

Top of Non-Overflow Section Looking South
at Intake Structure



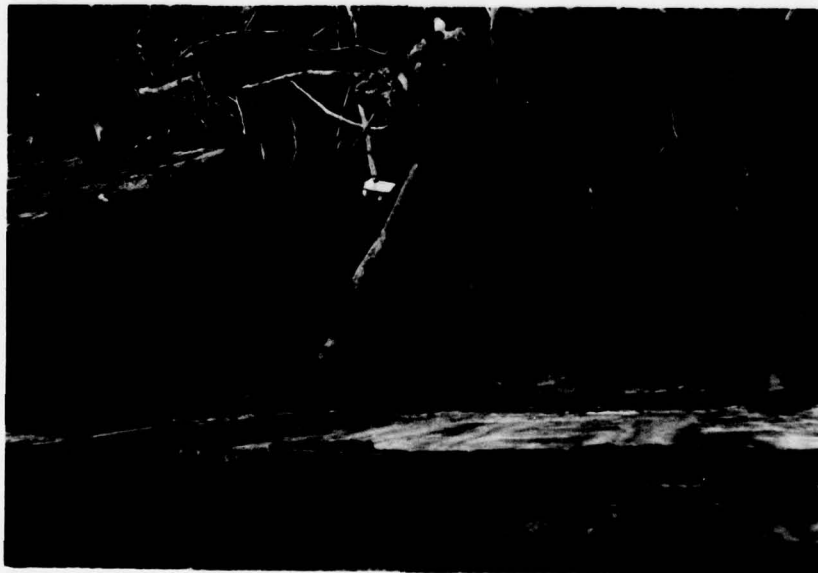
Photograph #9

Top of Non-Overflow Section Looking South
note deterioration of parapet walls



Photograph #10

South Abutment of Non-Overflow Section Looking South



Photograph #11

South Abutment of Non-Overflow Section
downstream face as viewed from top



Photograph #12

Upstream Face of Non-Overflow Section
and Intake Structure



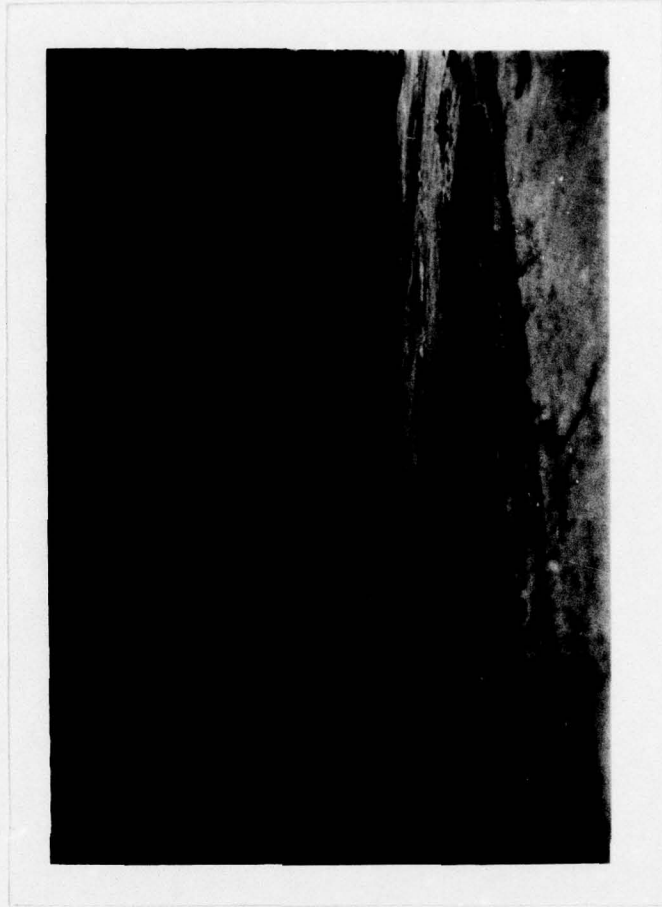
Photograph #13

Trash Screen at Upstream Face
of Intake Structure



Photograph #14

South Spillway Wall, as Viewed from Top of Dam



Photograph #15

Intersection of South Spillway Wall and
Non-Overflow Section, as viewed from top of dam



Photograph #17

Tailrace Channel Below Spillway



Photograph #16

Ogee Spillway with flashboards looking north



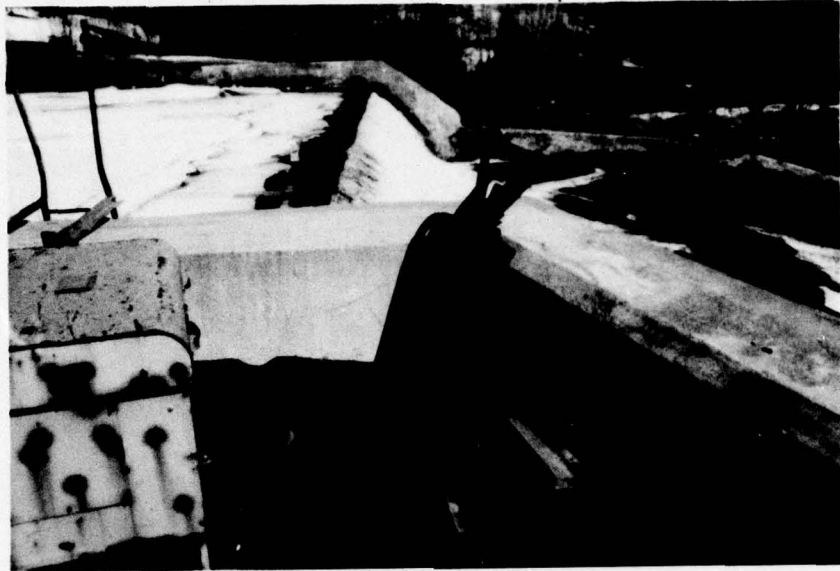
Photograph #18

Area between Spillway Channel
and Non-Overflow Section Channel



Photograph #19

Downstream Channel below Non-Overflow
Section, Looking east



Photograph #20

Ogee Spillway without flashboards looking north
Note bent bars salvaged after failure of flashboards



Photograph #21

Channel Below Spillway where it enters
Non-Overflow Section Channel
(photo is adjacent to photo #1)



Photograph #22

Downstream Channel
as viewed from downstream bridge
below power house

APPENDIX B

ENGINEERING DATA CHECKLIST

Check List
 Engineering Data
 Design Construction Operation

Name of Dam Inghams Dam

I.D. # NY 183

Item	Remarks		
	Plans	Details	Typical Sections
Dam			
Spillway(s)	YES	-	YES
Outlet(s)	-	-	-
Design Reports	NONE		
Design Computations	NONE		
Discharge Rating Curves			
Dam Stability	1970 Report by UH1, Hall & Reich		
Seepage Studies	NONE		
Subsurface and Materials Investigations	Borings from Dam Stability Report		

Item	Remarks
------	---------

Construction History

NONE

Surveys, Modifications,
Post-Construction Engineering
Studies and Reports

ONE REPORT TO STUDY UPLIFT PRESSURE

Accidents or Failure of Dam
Description, Reports

NONE

Operation and Maintenance Records
Operation Manual

All Records kept at Power House
No Manual

APPENDIX C

VISUAL INSPECTION CHECKLIST

VISUAL INSPECTION CHECKLIST

1) Basic Data

a. General

INGHAMS DAM

Name of Dam (KYSER LAKE DAM)

I.D. # N.Y. 183

Location: Town MANHEIM & OPPENHEIM County HERKIMER & FULTON

Stream Name EAST CANADA CREEK

Tributary of MOHAWK RIVER

Longitude (W), Latitude (N) 74°-45'-59" / 43°-3'-42"

Hazard Category C

Date(s) of Inspection OCTOBER 16, 1978
March 21, 1979

Weather Conditions 30° SUNNY

b. Inspection Personnel ROBERT McCARTY, MUHAMMAD ISLAM

ROBERT, LEVETT, LOU PRATT

c. Persons Contacted ROBERT LEVETT, NIAGRA MOHAWK POWER CORPORATION, SYRACUSE, N.Y. 13202 TEL (315) 474-1511

d. History:

Date Constructed 1911

Owner NIAGRA MOHAWK

Designer VIELE BLACKWELL AND BUCK, 49 WALL ST. N.Y.C.

Constructed by unknown

2) Technical Data

Type of Dam CONCRETE AND MASONRY

Drainage Area 270 SQUARE MILES

Height 125 feet Length 400 FEET

Upstream Slope VERTICAL Downstream Slope 1:0.65

2) Technical Data (Cont'd.)

External Drains: on Downstream Face NONE @ Downstream Toe NONE

Internal Components:

Impervious Core Concrete core wall at south abutment

Drains -

Cutoff Type -

Grout Curtain -

3) Embankment

None

a. Crest

(1) Vertical Alignment _____

(2) Horizontal Alignment _____

(3) Surface Cracks _____

(4) Miscellaneous _____

b. Slopes

(1) Undesirable Growth or Debris, Animal Burrows _____

(2) Sloughing, Subsidence or Depressions _____

(3) Slope Protection _____

(4) Surface Cracks or Movement at Toe _____

(5) Seepage _____

(6) Condition Around Outlet Structure _____

c. Abutments

No Earth Embankment

(1) Erosion at Embankment and Abutment Contact _____

(2) Seepage along Contact of Embankment and Abutment _____

(3) Seepage at toe or along downstream face _____

d. Downstream Area - below embankment

(1) Subsidence, Depressions, etc. _____

(2) Seepage, unusual growth _____

(3) Evidence of surface movement beyond embankment toe _____

(4) Miscellaneous _____

e. Drainage System

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

(2) Discharge from Drainage System _____

4) Instrumentation

(1) Monumentation/Surveys NONE

(2) Observation Wells _____

6 borings progressed in June 1970 were used to determine uplift beneath the dam and were left open

(3) Weirs NONE

(4) Piezometers NONE

(5) Other _____

5) Reservoir

a. Slopes O.K.

b. Sedimentation NONE Observed

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

a. General 205 feet long Ogee type approximately
26 feet high flashboards 4 feet in height
are used to store extra water in reservoir

b. Principle Spillway CONDITION OF THE SPILLWAY COULD
NOT BE DETERMINED BECAUSE WATER WAS FLOWING
OVER FLASHBOARDS OVER SPILLWAY

c. Emergency or Auxiliary Spillway NONE

d. Condition of Tail race channel LOOKS CLEAN AS FAR AS
VISIBLE.

e. Stability of Channel side/slopes Looks stable.

7) Downstream Channel

a. Condition (debris, etc.) CLEAN

b. Slopes STABLE

c. Approximate number of homes THERE ARE 3 HOMES ABOUT 300 FEET
BLOW THE DAM. ONE IS 20 FEET ABOVE CREEK BED
AND OTHER 2 ABOUT 30 FEET ABOVE THE CREEK BED. THERE ARE
A FEW MORE HOUSES FURTHER DOWNSTREAM.

8) Miscellaneous POOL IMMEDIATELY DOWNSTREAM OF DAM

HAS SOME DEBRIS SUCH AS LOGS, DRUM, ETCETERA.

9) Structural

- a. Concrete Surfaces CONCRETE SURFACE IS SPALLED
AT DIFFERENT SECTIONS OF DAM. Particularly
the downstream face max depth of spalling is 3 inches approx
& average depth of 1 inch
- b. Structural Cracking PARAQUET WALL CRACKED IN PLACES.
- c. Movement - Horizontal & Vertical Alignment (Settlement) _____
NONE OBSERVED
- d. Junctions with Abutments or Embankments _____
good condition
- e. Drains - Foundation, Joint, Face NONE
6 borings located near the toe of non-overflow section
were progressed in June 1970 to determine uplift pressures
these holes remain open
- f. Water passages, conduits, sluices 9' diameter steel penstock to power house
elect. operated hydraulic operated
with operational slide gate at intake and wicket gates at the turbines
2 - 6'6" reservoir drains have not been operated since filling of the dam.
South pipe is believed plugged, North pipe has bottomly valve which is leaking
- g. Seepage or Leakage _____
no seepage evident

- h. Joints - Construction, etc. _____
 _____ generally good condition
 _____ parapet joints need repair

- i. Foundation _____ no problems observed

- j. Abutments _____ good condition

- k. Control Gates _____ operational for penstock to power
 _____ plant, inoperative reservoir drain

- l. Approach & Outlet Channels _____ good condition

- m. Energy Dissipators (plunge pool, etc.) _____ rock channel
 _____ no problem

- n. Intake Structures _____ good condition
 _____ some limited ice damage to intake walls

- o. Stability _____ appears good

- p. Miscellaneous _____ A section of the training wall at the north end of the
 _____ spillway adjacent to the toe of the spillway is missing
 _____ no erosion was apparent in the area behind the wall (Rock foundation)
 _____ repair if erosion is initiated.

APPENDIX D

HYDROLOGIC/HYDRAULIC

ENGINEERING DATA AND COMPUTATIONS

CHECK LIST FOR DAMS
HYDROLOGIC AND HYDRAULIC
ENGINEERING DATA

1

AREA-CAPACITY DATA:

	<u>Elevation</u> (ft.)	<u>Surface Area</u> (acres)	<u>Storage Capacity</u> (acre-ft.)
1) Top of Dam	<u>665.8</u> usgs	_____	<u>4600</u>
2) Design High Water (Max. Design Pool)	<u>665.5</u>	_____	<u>4545</u>
3) Auxiliary Spillway Crest	_____	_____	_____
4) Pool Level with Flashboards	<u>661.8</u>	_____	<u>3,860</u>
5) Service Spillway Crest	<u>657.3</u>	_____	<u>3082</u>

DISCHARGES

	<u>Volume</u> (cfs)
1) Average Daily	<u>UNKNOWN</u>
2) Spillway @ Maximum High Water	<u>21,000</u>
3) Spillway @ Design High Water	<u>20,000</u>
4) Spillway @ Auxiliary Spillway Crest Elevation	-
5) Low Level Outlet	<u>NOT OPERABLE</u>
6) Total (of all facilities) @ Maximum High Water	<u>21,000</u>
7) Maximum Known Flood due to natural flow	<u>14,750</u>
due to an upstream dam break	19,300

CREST:

ELEVATION: 665.8

Type: CONCRETE

Width: BASE: 83 FEET; TOP: 12.5 FEET Length: 480 FEET

Spillover CONCRETE OGEE 205 FEET LONG

Location NORTH SIDE OF DAM

SPILLWAY:

PRINCIPAL

EMERGENCY

657.3 Elevation _____

CONCRETE OGEE Type _____

205 FEET Width _____

Type of Control

_____ Uncontrolled _____

Controlled:

4.5 FEET HIGH FLASHBOARDS Type _____
(COLLAPSIBLE) (Flashboards; gate)

- Number _____

- Size/Length _____

Invert Material _____

Anticipated Length
of operating service _____

- Chute Length _____

26 FEET Height Between Spillway Crest
& Approach Channel Invert
(Weir Flow) _____

OUTLET STRUCTURES/EMERGENCY DRAWDOWN FACILITIES:

Type: Gate _____ Sluice _____ Conduit 2-6' Penstock _____

Shape : ONE DRAIN PIPE WAS SHUT OFF AND THE OTHER IS

Size: NON-OPERABLE . WATER LEAKS THRU ONE OF THE PIPES .

Elevations: Entrance Invert _____ - _____

Exit Invert _____ 560 _____

Tailrace Channel: Elevation _____ 539 _____

HYDROMETEROLOGICAL GAGES:

Type : _____ NONE _____

Location: _____

Records:

Date - _____

Max. Reading - _____

FLOOD WATER CONTROL SYSTEM:

Warning System: _____ NONE _____

Method of Controlled Releases (mechanisms):

ONLY THRU PENSTOCK AND POWERHOUSE .

DRAINAGE AREA: 278 SQUARE MILES

DRAINAGE BASIN RUNOFF CHARACTERISTICS:

Land Use - Type: _____

Terrain - Relief: _____

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 2.5 (Miles)

Length of Shoreline (@ Spillway Crest) 5.7 (Miles)

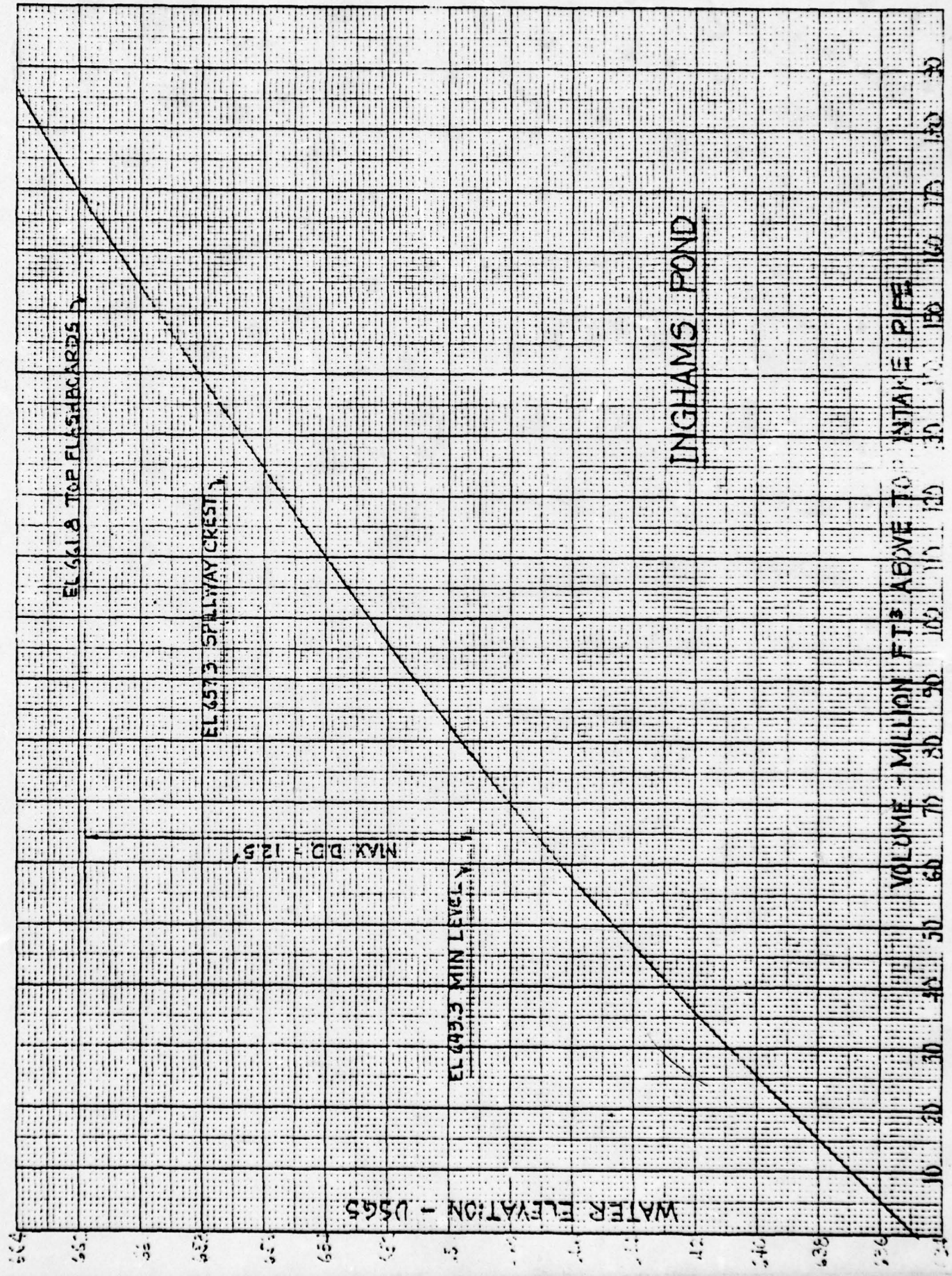
RESERVOIR DETENTION VOLUME

All figures from curve supplied by
Niagara Mohawk Power Corporation.

All storage above intake level (634.8).

Elevation, ft. USAS	Volume in million ft. ³	Volume in acre-feet	Remarks.
634.8	0	0	Top of intake pipe
636	5		
638	15.5		
640	25.5		
642	36.0		
644	46.5		
646	58.0		
648	70.0		
650	83.0		
652	96.0		
654	100.0		
656	124.5		
657.3	134.0	3082	spillway crest
660	155.0		
661.8	168.0	3860	Top of flashboards
664	186.0	4278	
665.8	200.0	4500	Top of dam

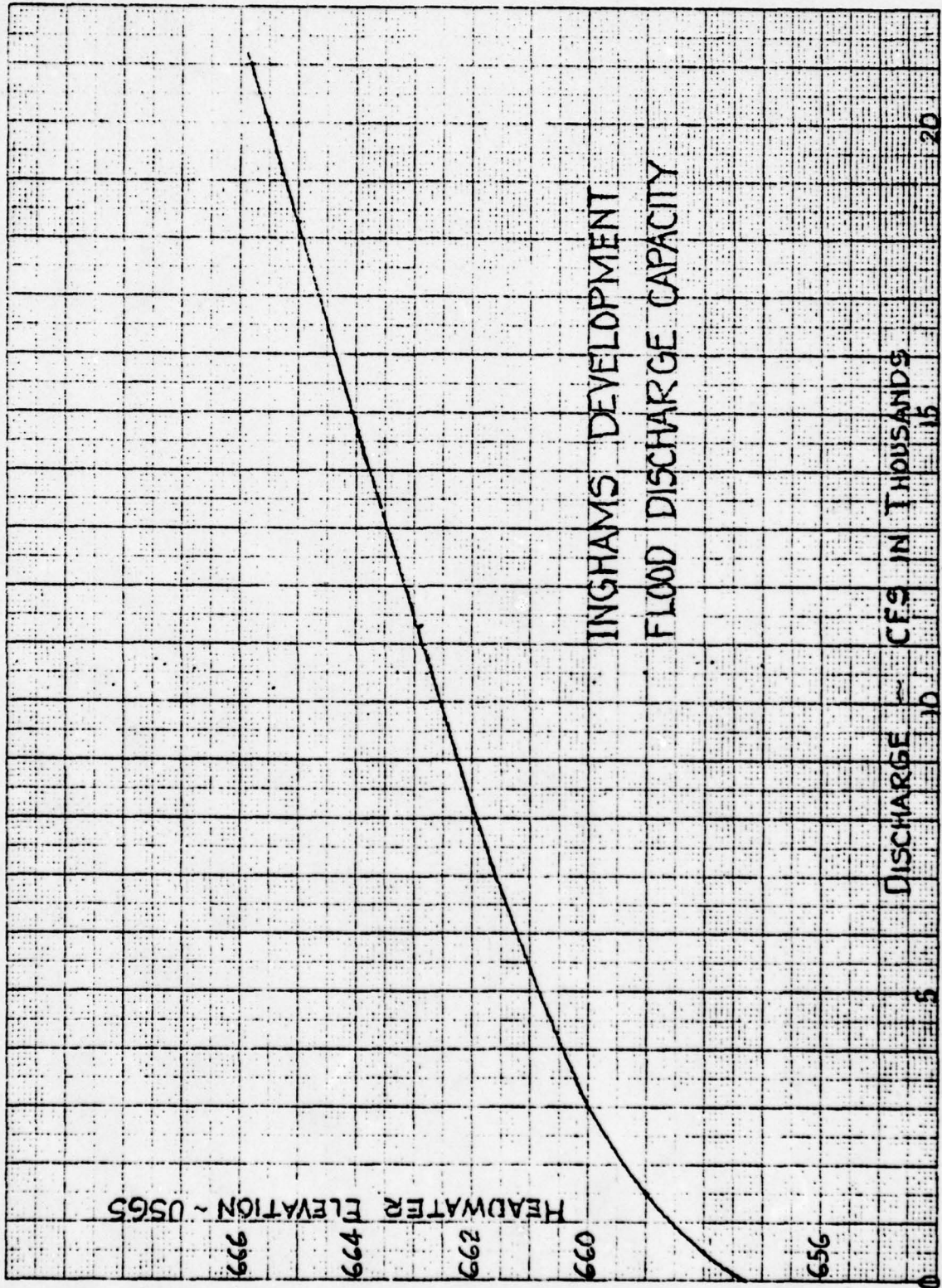
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SPILLWAY CAPACITY

All figures below are from Flood Discharge Capacity curve supplied by Niagara Mohawk Power Corporation.

Elevation, ft	Discharge, cfs
657.3	0 Spillway crest
659.0	1,500
660.0	3,000
661.0	3,500
661.8	7,600 ← TOP of
662.0	8,200 flashboards
663.0	11,500
664.0	14,700
665.0	18,300
665.8	21,000 TOP of dam



INGHAMS DEVELOPMENT
FLOOD DISCHARGE CAPACITY

HEADWATER ELEVATION - USGS

DISCHARGE - CFS IN THOUSANDS

INGHAMS DAM

Drainage area = 278 square miles.

Name of stream : East Canada Creek

The following calculations are based on "Upper Hudson & Mohawk River Basins Hydrologic Flood Routing Models" study by Resource Analysis, Incorporated, for Corps of Engineers, New York District. Reference pages 97 - 107.

Subdivision 16 of Mohawk River Basin (Little Falls N.Y. to Mouth) lies entirely in the drainage area ^{also} of the Inghams dam. Major part of subdivision 14_n lies in the same drainage area.

Area of subdivision 13 = 261 square miles

Area of subdivision 14 = 30 square miles.

But only 17 square miles of subdivision 14 lies in the drainage area of the Inghams dam.

Standard Project Flood (SPF) = $\frac{1}{2}$ Probable Maximum Flood (PMF)

$\frac{1}{2}$ PMF outflow at USGS gage # 3480 of East Canada Creek at East Creek is 46,206 cfs. and drainage area is 291 sq. miles.

$$\left(\frac{A_1}{A_2}\right)^{3/4} = \frac{\frac{1}{2} PMF_1}{\frac{1}{2} PMF_2}$$

$$\text{or } \left(\frac{278}{291}\right)^{3/4} = \frac{\frac{1}{2} PMF}{46,206}$$

or $\frac{1}{2}$ PMF at dam site = 44,649 \approx 44,700 cfs.

\therefore PMF at dam site = $2 \times 44,700 = 89,400$ cfs.

OVERTOPPING

$$\text{Length of dam} = 480 \text{ feet} = L_1$$

$$\text{Length of spillway} = 205 \text{ feet} = L_2$$

$$Q = C L H^{3/2}, \quad C_1 = 4.13, \quad L_1 = \text{ft}, \quad L_2 = 205 \text{ ft}$$

H_1 = height over dam

H_2 = Height over spillway.

Q = Total discharge

$$H_2 = 8.5 \text{ ft} + H_1$$

PMF

$$89,000 = C_1 L_1 H_1^{3/2} + C_2 L_2 H_2^{3/2}$$

(PMF)

$$= 3.33 \times 480 \times H_1^{3/2} + 4.13 \times 205 \times H_2^{3/2}$$

$$= 1598.4 H_1^{3/2} + 846.65 (8.5 + H_1)^{3/2}$$

$$H_1 = 7.7 \text{ feet}$$

\therefore the dam will be overtopped by 7.7 feet of water due to PMF.

$\frac{1}{2}$ PMF

$$45,000 = 1598.4 H_1^{3/2} + 846.65 (8.5 + H_1)^{3/2}$$

$$H_1 = 3.5 \text{ feet}$$

\therefore the dam will be overtopped by 3.5 feet of water due to $\frac{1}{2}$ PMF.

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

UHL, HALL & RICH

Engineers

441 STUART STREET, BOSTON, MASSACHUSETTS 02116 • AREA CODE 617-262-3220
Southern Office: 1301 East Morehead Street, Charlotte, North Carolina 28204 • Area Code 704-376-4336

May 30, 1969

2225-2

Subject: Inghams Dam
Uplift Pressure System

Mr. R. J. DeStefano, System Structural Engineer
Niagara Mohawk Power Corporation
126 State Street
Albany, New York 12201

Dear Mr. DeStefano:

In compliance with your request we have reviewed the procedure previously outlined for drilling an exploratory hole in the tailrace area, to be followed by three pairs of observation holes through the concrete dam structure.

In answer to Mr. Clancy's question to you regarding the need for additional information pertaining to our proposed method of observation hole monitoring for determination of uplift pressures at the hole locations, we feel that the explanation and description of the planned procedure as follows would be adequate and sufficient for the F.P.C. to consider:

"It is planned to make an exploratory drill hole in the foundation immediately downstream of the dam at a point in the tailrace adjacent to the tallest section of dam. Cores will be taken at this hole to a depth of 25 feet below the base of the structure. Should this hole indicate normally sound foundation conditions, as is anticipated, then a series of observation holes will be made through the dam.

It is proposed that three sets of holes be cored through the dam from the downstream face. These three lines of holes will be located at the tallest section of the dam within the original river bed foundation. Spacing between the lines of holes will be approximately 50 feet, which will allow full coverage of the foundation at this deepest section of the structure. Each line of holes will consist of an angle hole and a vertical hole extending into the foundation a maximum of 3 feet. With the collar of these holes located in the vicinity of El. 573, which is above maximum expected tailwater, the vertical hole will contact the foundation at a point approximately 25 feet from the toe. The angle hole will then meet the foundation at about 60 feet from the toe or about 25 feet from the heel of the structure. The enclosed sketch will illustrate the location of the proposed observation holes.

From these holes, a piezometric height of water can be determined and translated into foundation pressures at the particular foundation contact points. It is planned to insert threaded pipe ends at the collars of the holes in order to allow attachment of pressure gages to be used should foundation drainage flows reach this elevation. These holes will normally be capped to prevent foreign material from entering the holes.

Readings would be taken at monthly intervals during the first year of operation of the monitoring system, assuming normal headwater - tailwater relation. Should operating conditions require a rapid or unusual drawdown of the reservoir, more frequent readings would be taken in order to establish trends in uplift conditions versus headwater elevations for the first year.

It is proposed that the readings at the angle holes and vertical holes respectively, would be averaged to be applied over the length of the base, applicable only at the deepest section of the structure. It is intended to treat the pressure at the upstream face of the structure as full headwater pressure, and likewise the pressure at the downstream face will be taken at tailwater pressure. Utilizing the average pressure readings determined from the observation holes, an uplift intensity can then be determined and compared to the extreme assumption of full headwater varying to full tailwater over the full base area. This calculated uplift intensity would then be used in determining the required safety factors of the structure against overturning under normal and flood conditions.

Should the results of these readings indicate that uplift intensities are still too high to provide the required minimum safety factors, then other remedial work would have to be considered to accomplish this."

Mr. R. J. DeStefano

-3-

May 30, 1969

In answer to Mr. Clancy's request that we provide some back up to you on the proposed procedure, we offer the following:

The principle involved here has been employed on many recent structures in a slightly modified way and has been found to be acceptable. The main variation in the method being that the same approach in general is taken except that provision is made to establish these check points at the foundation contact at the time of construction of the dam structures. A so called "pressure cell" is normally built in at selected points at the foundation contact and piping is then carried to some convenient point, usually to a grouting gallery or inspection gallery. At these locations drainage flows, if any, are measured and observed. If deemed necessary, pressure readings are taken. This system has been used by us on several dams and powerhouse structures.

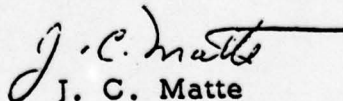
The use of the actual system of drilling holes after construction has been used on many structures, but the majority of these have foundations which have been grouted during or after construction and also have a system of foundation drains installed downstream of the grout curtain. This eliminates the possibility of any qualitative comparison or projection of results at the Inghams site.

However, one installation of note where a system of drilled holes was installed through a dam and foundation which was not grouted or drained, this was at Holtwood Dam on the Susquehanna River. The dam is an overflow ogee spillway about 70 feet high. The foundation was a granitic schist containing some fault seams. Holes were drilled in several lines with 4 holes per line starting at seven feet back from the upstream face and spaced at fifteen feet between holes. All holes were vertical. Results of readings showed a deviation from the typical headwater to tailwater uplift assumption.

We trust that the above will suffice for your present needs. If further information is required please contact us.

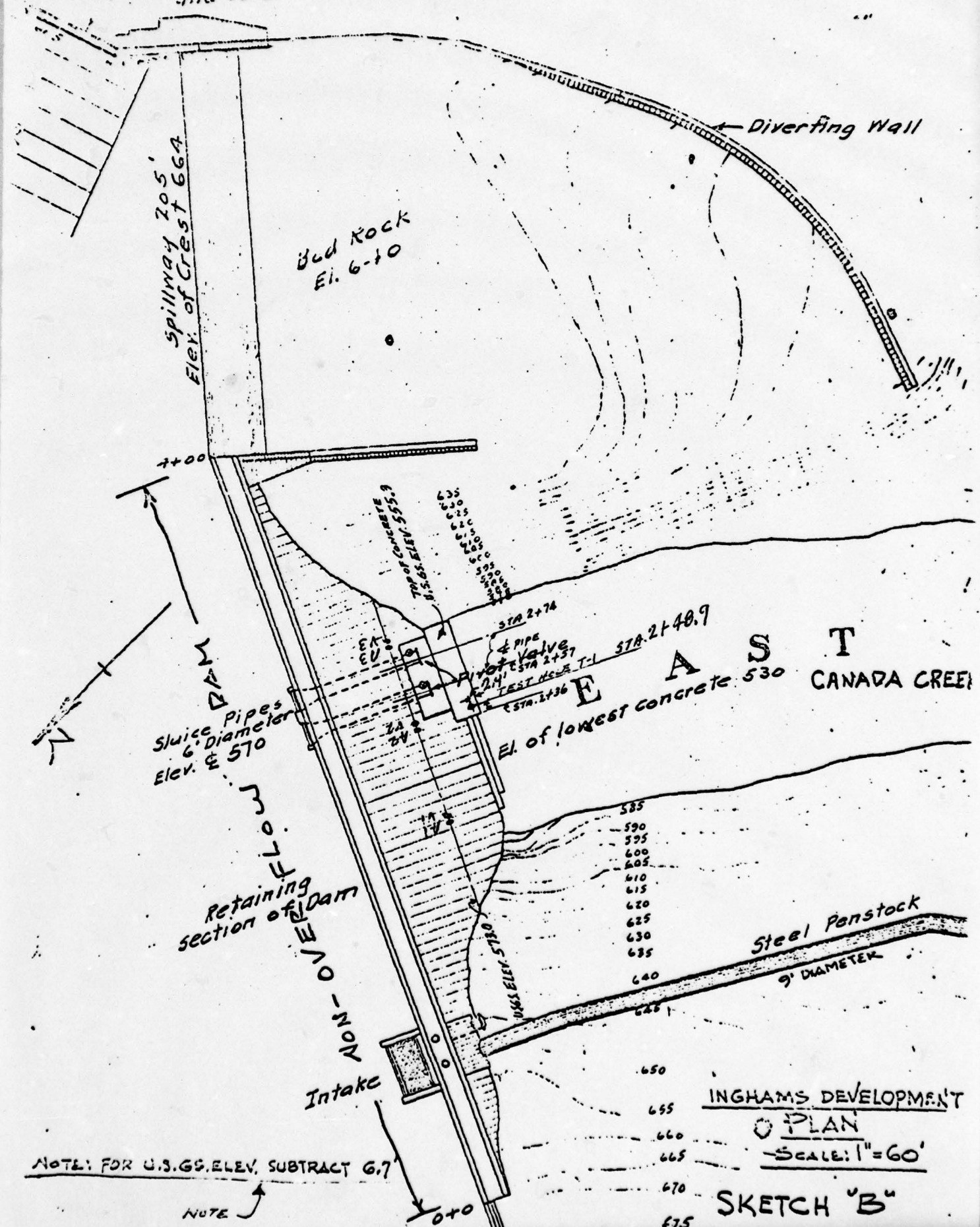
Very truly yours,

UHL, HALL & RICH


J. C. Matte

JCM/mf

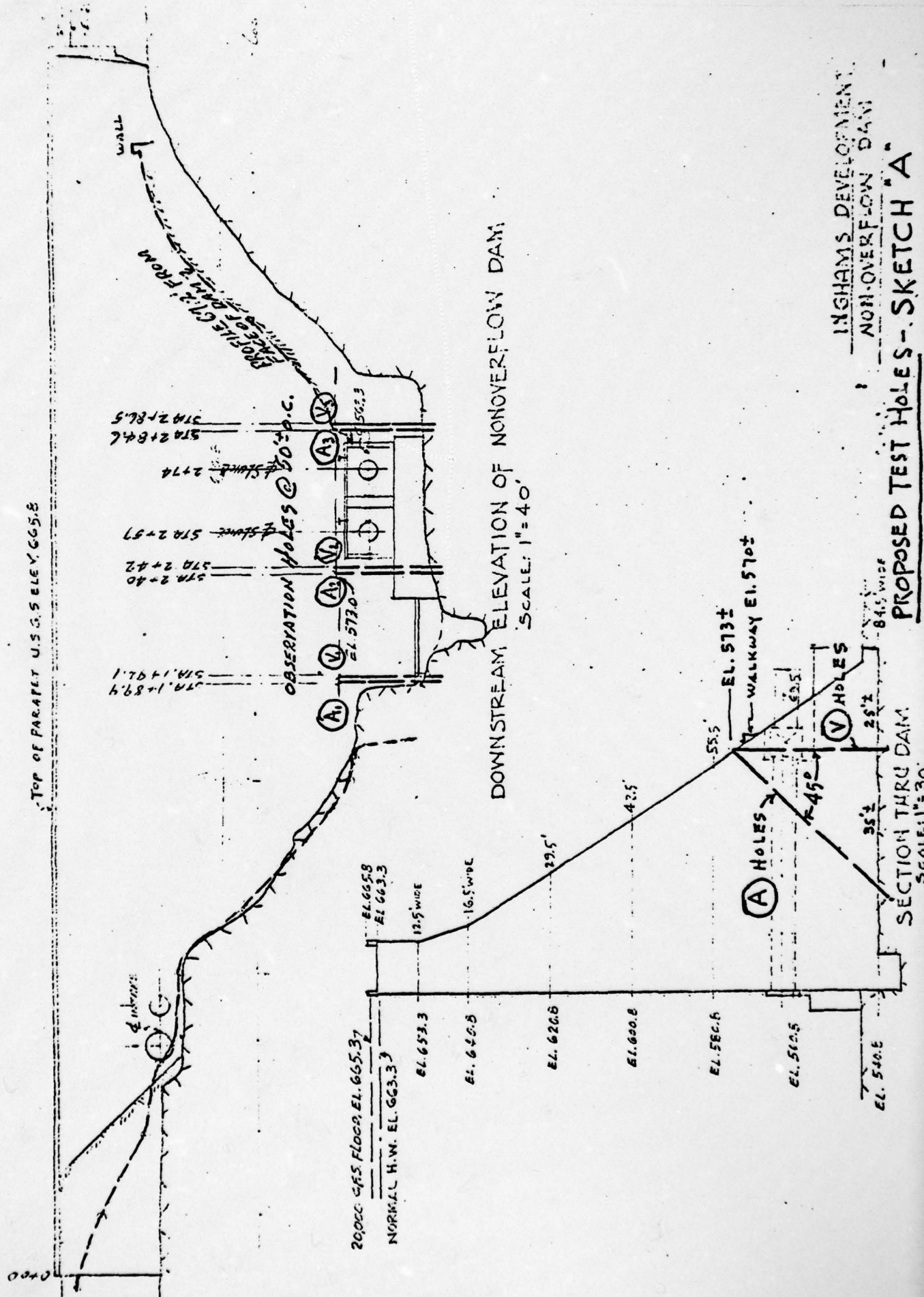
Top of abutment
and Core Wall 6/2.5



NOTE: FOR U.S.G.S.ELEV. SUBTRACT 6.7

NOTE

INGHAMS DEVELOPMENT
 PLAN
 SCALE: 1"=60'
 SKETCH "B"



NIAGARA MOHAWK POWER CORPORATION

INGHAMS DEVELOPMENT

EAST CANADA CREEK PROJECT NO. 2648

REPORT OF

STABILITY ANALYSIS

NON-OVERFLOW SECTION

INGHAMS DAM

UHL, HALL & RICH, ENGINEERS

Boston, Massachusetts

November 1970

2225-3

UHL, HALL & RICH

Engineers

441 STUART STREET, BOSTON, MASSACHUSETTS 02116 • AREA CODE 617-262-3220
Southern Office: 1301 East Morehead Street, Charlotte, North Carolina 28204 • Area Code 704-376-4336

November 16, 1970

2225-3

**SUBJECT: Report of Stability Analysis
Inghams Dam Non-Overflow Section
East Canada Creek Project No. 2648**

Mr. T. J. Brosnan, Vice President and Chief Engineer
Niagara Mohawk Power Corporation
300 Erie Boulevard West
Syracuse, New York 13202

Dear Mr. Brosnan:

We are pleased to transmit herewith our report on the stability of the non-overflow section of Inghams Dam.

The report includes a summary of the correspondence and discussions with the Federal Power Commission relating to the question of stability at this structure since the time of application for license in 1967.

As part of the stability investigation, an exploratory program was carried out through the concrete structure in order to determine actual uplift pressures at the base of the structure. It was determined by means of these exploratory or observation holes that actual uplift conditions were less than those required to be assumed in the absence of specific data.

It is intended that this report shall evidence the fact that the lessening of uplift forces employed in the stability analysis permits the satisfying of the minimum factors of safety established for the structure. In so doing, it is anticipated that no reinforcement of the structure will be required, as was considered earlier.

We trust that you will find this report to be complete and satisfactory.

Very truly yours,

UHL, HALL & RICH


J. C. Matto
Project Engineer

JCM/ss

UHL, HALL & RICH

Engineers

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STABILITY ANALYSIS REPORT

OF THE

INGHAMS DAM NON OVERFLOW SECTION

EAST CANADA CREEK PROJECT NO. 264A

SUMMARY

The East Canada Creek Project encompasses the Beardslee and Inghams Hydro developments, located on East Canada Creek in the towns of Manheim and Oppenheim, New York.

In July of 1967, the Federal Power Commission staff in reviewing the application for license of these developments, requested that a report on the stability of the structures at both projects be submitted. This report was required to include a flood study for the purpose of determining reservoir surcharge at each project.

In complying with the F.P.C. request, Uhl, Hall & Rich was engaged by the applicant to perform the stability analyses on the earth dam and concrete spillway structures at the Beardslee Development, and on the concrete spillway and non overflow dam sections at the Inghams Development. The required flood studies were performed by the applicant, Niagara Mohawk Power Corporation.

The report was prepared and submitted on September 27, 1967.

Further correspondence was received from the Chief of the Bureau of Power of the Federal Power Commission in a letter dated October 30, 1967.

Essentially, this letter contained the following opinions and comments:

1. The F.P.C. staff was of the opinion that in the event of dam failure there would be probable loss of life.
2. Spillway design flood should be based upon probable maximum precipitation rather than the 100 year flood derived by flood frequency analysis, as proposed by the applicant.
3. Once the spillway design flood was determined, stability analyses should be prepared for the normal reservoir loading condition and for the maximum flood reservoir loading condition.
4. Since no drains were provided at the base of the concrete structures, uplift pressures should be assumed as acting over 100 percent of the base areas rather than over two thirds of the base area as previously assumed. Any uplift assumptions different from those noted above should be verified by tests on the structures.
5. Information was requested as to whether soils properties as used in the stability analysis of the earth embankment at Beardslee Dam were based upon assumptions or soil tests.

Further studies were made by the applicant considering the statements and opinions as stated in the above letter. Both flood analyses and stability analyses were performed using the proposed criteria of the F.P.C.

The applicant took the position that flood analysis based on maximum probable precipitation assumptions for these existing developments was not appropriate. However, stability analyses were performed with uplift considered over 100 percent of the base area of the structures, with reservoir elevations as determined by a spillway design flood of 20,000 cfs. This latter figure was the magnitude proposed by the applicant as appropriate to

the developments under consideration.

A meeting was arranged and held at F.P.C. offices in Washington, D.C., with members of the F.P.C. staff, the applicant, and Uhl, Hall & Rich on March 7, 1969, to discuss in total all considerations relating to flood analysis and stability of the project structures.

The essential results of this meeting were as follows:

1. All previous questions regarding the safety and stability of the Beardslee Development structures were satisfied.
2. Factors of safety of the Inghams structures were discussed and it was determined that the spillway structure could be considered adequate in this regard while the non-overflow section of the dam had safety factors which were not satisfactory when full uplift was applied.
3. It was determined that methods of improving the factor of safety against overturning of the non-overflow section should be investigated. It was agreed that a satisfactory range of safety factor would be 1.25 or greater for normal conditions, including a design flood of 20,000 cfs. For a check of extreme conditions a factor of safety of 1.0 would be satisfactory for reservoir conditions corresponding to a maximum probable precipitation flood of 100,000 cfs.

Studies and preliminary cost estimates were made of various methods of dam reinforcement required for improvement of stability factors. Included among these were drilled in prestressed rods and rockfill embankment to be placed at the downstream face of the structure. The latter appeared to offer the only practical solution.

In recognition of the fact that uplift forces were of significant influence in the determination of safety factors, it was decided to investigate actual uplift conditions at the base of the structure and compare these to conditions assumed.

A proposed plan was developed to drill a total of six holes through the structure commencing at the downstream face at an elevation above normal expected tailwater. Three of these holes were to be vertical and three were to be angle holes inclined downward in the upstream direction. These holes, paired off as one vertical and one angle hole at about fifty feet between pairs would provide a piezometric measurement of uplift pressure at the points where the drilled holes intersected bedrock at the base of the dam. It was intended that individual pressure readings at each hole could be averaged and applied as actual uplift forces at the base of the structures. If these actual pressures were less than the intensities assumed on the conventional full headwater to full tailwater gradient as recommended by the F.P.C., then they would be applied in the stability analysis. In any case uplift pressures obtained would be applied over 100 percent of the base area.

The applicant submitted the proposed plan to the F.P.C. by letter dated July, 11, 1969 requesting that they review and comment on the proposed procedure.

A reply to the above request was received from the F.P.C. by letter dated August 5, 1969. The commission staff had determined that the proposal to determine actual uplift pressures was satisfactory to them, and stated also that the stability analyses should include the normal water surface condition, seismic condition, the probable maximum precipitation flood condition, and other loading condition which may be critical.

Specifications for the drilling work were prepared and a contractor selected for the work. It was considered preferable not to attempt this work under severe winter conditions and actual performance of the work was necessarily deferred until spring of 1970. The work was commenced in May and completed during the latter part of June 1970.

Daily water level readings were taken at each hole at completion and continually on a daily basis thereafter through July 2, 1970. Since, by that time it could be observed that either a steady state condition or a diminishing water level condition was commencing in the holes, it was then decided to take readings on a weekly basis.

The stability analysis results as included in this report utilized the actual uplift readings from these test holes. Since actual results proved to be less critical than the assumed uplift loading it was possible to obtain stability factors required by the F.P.C. as minimums, thereby alleviating the need to provide additional means of reinforcing the non-overflow dam section against the forces assumed in the analysis.

OBSERVATION HOLES

The six observation holes through the dam were drilled with rotary drills of standard NX size. Cores of the concrete structure and bedrock at the base of the structure were obtained and examined. All cores indicated sound concrete at all locations. Penetration into bedrock was limited to a maximum of about five feet at all holes. Excellent recovery of core was made in both concrete and bedrock at all holes.

The collar of all six holes was set at El. 573.0 nominally. All water surface elevations observed within the holes were referenced to this elevation.

With the exception of hole V-3, all other holes had flows out of the

collar of the hole at the time the holes were completed. Hole V-3 was a shallow hole which extended only 12 feet before reaching bedrock contact.

With time, the angle holes which extended furthest in the upstream direction, maintained water to El. 573, while two vertical holes which intersected bedrock about 18 feet upstream of the toe of the dam exhibited a diminishing water level. Hole V-3 maintained a relatively constant level, with water observed slightly above El. 572.

Headwater and tailwater elevations were read daily along with the observation hole readings. There appeared to be no apparent direct correlation between reservoir elevation and elevation of the water surface in the holes.

The drilled holes served another important function in that they established the elevation of bedrock at the base of the structure at each location. It was found that the average base elevation of the deep portion of the structure was approximately at El. 547, disregarding information from hole V-3, which was obviously high locally.

From the above observations it was determined that it would be reasonable to assume that the most representative base elevation for making the stability analysis would be El. 547. Also, that a safe and conservative value of piezometric head to be used in uplift pressure determination would be to El. 573 at both the angle and vertical hole intersections at the base of the structures.

Included in the appendix of this report are:

1. Drillers logs of drill holes - Plates III through XI
2. Water Level Observations - Plate XII

STABILITY ANALYSIS

The analysis was made to include the loading conditions which were established for the non-overflow section. These can be summarized as follows:

Case 1 - Normal Water Surface Condition

Headwater El. 663.3

Tailwater El. 563.3

Uplift

Case 2 - Design Flood Condition (20,000 cfs Flood)

Headwater El. 665.3

Tailwater El. 565.3

Uplift

Case 3 - Maximum Probable Flood Condition (100,000 cfs Flood)

Headwater El. 675.5

Tailwater El. 573.0

Uplift

Case 4 - Seismic Conditions

Headwater El. 663.3

Tailwater El. 563.3

Earthquake Forces 0.05g Horizontal Direction

Uplift

Uplift forces in all cases were based on the assumption of full headwater pressure at the upstream face of the structure varying linearly to the observed pressures at the intercept of the two lines of observation holes, and then varying linearly from the line of vertical holes to tail-

water pressure at the downstream face of the structure.

In calculating factors of safety against sliding, it was obvious that in many cases that the sliding factor exceeded the allowable value of friction for concrete to rock. It was therefore necessary to employ the shear value in the rock and to then check the combined shear - friction factor of safety provided.

Results of stability analyses for all cases considered are tabulated on Plate I in the appendix. A tabulation of values and assumptions used in the stability analysis are included on Plate II in the appendix.

CONCLUSIONS

Based on the results of the uplift observations and the ensuing stability calculations it is concluded that the non-overflow structure at Inghams Dam as analyzed satisfies the values of the safety factors prescribed as minimums in all cases. Inspection of the factor of safety against overturning in Case IV as calculated is so close numerically to 1.25 that for all practical purposes it was judged satisfactory.

Since the uplift forces influence to a great degree the numerical factors of safety as calculated, we should like to point out that based on the piezometric readings in the observation holes in the structure, it appears that the uplift pressure diagram used still provides a somewhat larger factor of safety than is apparent. This results from the assumption of full headwater pressure being applied at the upstream face of the structure and then varying linearly to the observed pressure at the first line of holes.

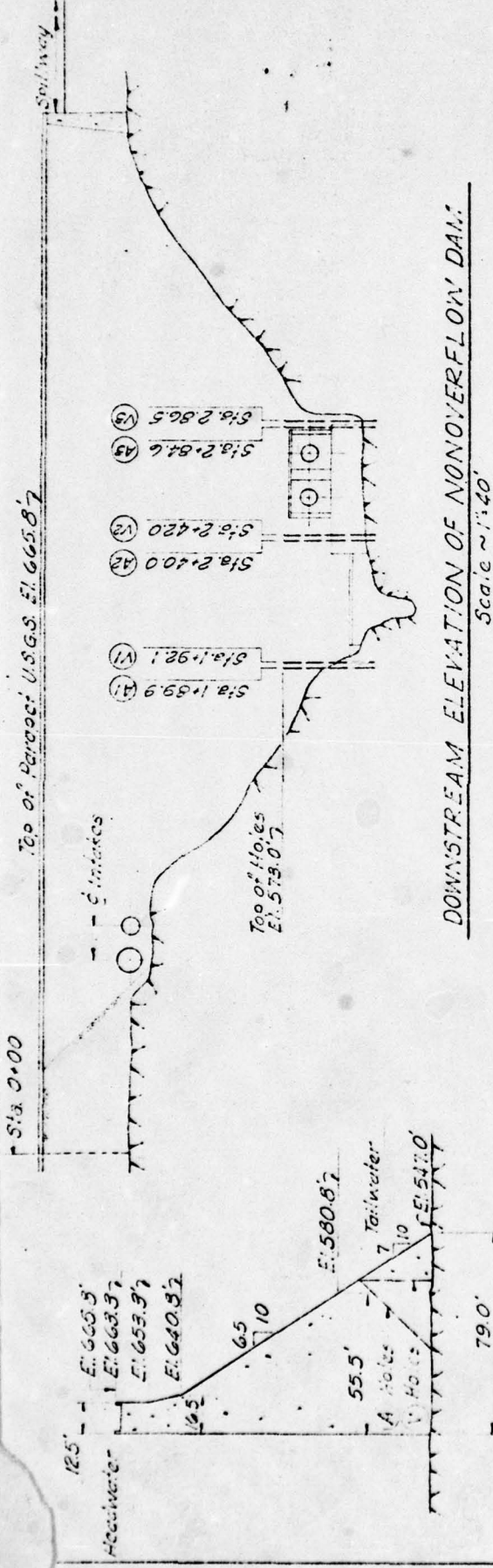
Although no observation holes were provided at or near the upstream face, it would seem unlikely that the loss in head between this face and the first line of holes would be in the order of 90 feet (El. 663 - El. 573 = 90) in a horizontal distance of 35 feet. Then remain, as observed at a constant

value for the next 26 feet, at all three transverse lines of observation holes. It seems more reasonable to assume that in actuality full headwater pressure is not present at the line of the upstream face. Should this be the case, then the uplift effect would thereby be reduced in all cases, resulting in improved factors of safety over those tabulated on Plate I.

As a matter of interest, a comparison of the effect of the uplift as used in the calculations and the conventional assumption of full headwater at the heel varying to full tailwater at the toe was made. Headwater and tailwater elevations were taken as observed on June 19, 1970 as El. 662.5 and El. 559.7, respectively.

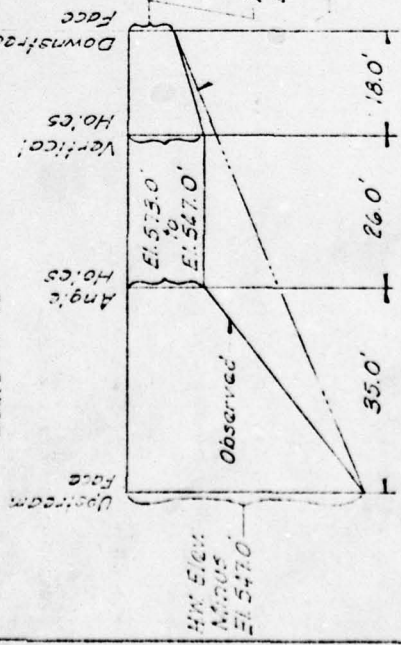
A comparison of the vertical effect showed that observed forces were 69.2 percent of the conventional assumption. This indicates that a net effect at this project would be the equivalent of the conventional uplift gradient of full headwater to tailwater applied over two thirds of the base area versus 100 percent of the area as normally considered at structures with no drainage provisions at the foundation.

A comparison of the overturning effect about the toe was in about the same range, with the observed effect being about 73.5 percent of the conventional assumption.



DOWNSTREAM ELEVATION OF NONOVERFLOW DAM
Scale ~ 1/40'

DAM SECTION
Scale ~ 1/40'



UPLIFT PRESSURE DIAGRAM - HEAD IN FEET
Not to Scale

DAM STABILITY SUMMARY

Condition	Elev	Σ V Kips	Σ H Kips	Σ H Σ V	S sf	Resultant From Top	Σ Mr Ft. Kips	Σ Mo Ft. Kips	Conv. Slabs F.S.		
									Sec.	Toe	
Case 1	547.0	495.7	415.7	0.84	5.75	19.6	37867	28130	1.85	-22	110
Case 2	547.0	492.1	427.5	0.87	5.60	17.8	37894	29117	1.80	-28	111
Case 3	547.0	485.8	490.9	1.03	4.88	7.0	38073	34698	1.10	-63	148
Case 4	547.0	495.7	475.2	0.96	5.04	14.5	37867	30683	1.24	-59	127

CASE 1 - NORMAL WATER SURFACE CONDITION HEADWATER EL. 663.3 TAILWATER EL. 547.0 UPLIFT
 CASE 2 - DESIGN FLOOD CONDITION (3,000 cfs FLOOD) " " " 665.9 " " 545.3 UPLIFT
 CASE 3 - MAX. PROBABLE FLOOD " (100,000 cfs FLOOD) " " " 675.5 " " 573.3 UPLIFT
 CASE 4 - SEISMIC CONDITION, 0.05g HORIZONTAL NIAGARA MOHAWK POWER CORPORATION SYRACUSE, N. Y. " " 663.3 " " 543.3 UPLIFT

INGHAM'S DEVELOPMENT
 EAST CANADA PROJECT NO. 2648
 STABILITY ANALYSIS TABULATION
 UHL, HALL & RICH ENGINEERS BOSTON, MASS.
 NOVEMBER, 1970

TABULATION OF VALUES & ASSUMPTIONS

PLATE II

USED IN STABILITY ANALYSIS

1. Unit weight of concrete - 150 lb/cu. ft,
2. Unit weight of water - 62.4 lb/cu. ft.
3. Uplift in accordance with pressure diagram - Plate I
4. Earthquake acceleration - 0.05g
5. Shear - friction factor of safety - S_{s-f}

$$*S_{s-f} = \frac{f \sum (v) + r S_g A}{\sum H}$$

Where $r = 0.5$

$S_g = 380$ p.s.i.

$A =$ Area of base

$f = 0.5$

$\sum V =$ Summation of vertical forces

$\sum H =$ Summation of horizontal forces

* For discussion and explanation of terms see Hydroelectric Handbook
by Creager and Justin, Wiley & Sons, Inc.

SOILTESTING, INC.

120 Mountain Road
Seymour, Connecticut

FOREMAN - DRILLER
DH - AM - RS

INSPECTOR

CLIENT: **NIAGARA MOHAWK POWER CORP.**
P.O. #23138

PROJECT NO
B-742

PROJECT NAME
INGHAMS HYDRO NON-OVERFLOW DAM

LOCATION
**TOWN OF MANHEIM
HERKIMER CO., NEW YORK**

SHEET 1 OF 1
HOLE NO. V-1

BORING LOCATIONS
as instructed by Engineer

OFFSET

GROUND WATER OBSERVATIONS
AT _____ FT AFTER _____ HOURS
AT _____ FT AFTER _____ HOURS

CASINO SAMPLER CORE BAR
TYPE _____
SIZE I.D. _____
HAMMER WT _____
HAMMER FALL _____
NXM
Diamond

Date Start **6/17/70** Date Finish **6/19/70**
SURFACE ELEV **573.0'**
GROUND WATER ELEV ***see note**

DEPTH	CASING BLOWS PER FOOT	SAMPLE					BLOWS PER 6" ON SAMPLER (FORCE ON TUBE)			CORING TIME PER FT (MIN)	DENSITY OR CONSIST MOIST	STRATA CHANGE DEPTH ELEV	FIELD IDENTIFICATION OF SOIL REMARKS INCL COLOR, LOSS OF WASH WATER, SEAMS IN ROCK, ETC
		NO	TYPE	PEN	REC	DEPTH @ BOT	0-6	6-12	12-18				
									8		Run #1	Gray Concrete solid from face of Dam.	
		1	C	24"	23"	2'0"			11		2'0"		
									10		Run #2		
		2	C	24"	25"	4'0"			10		4'0"		
									11		Run #3		
5		3	C	24"	24"	6'0"			11		6'0"		
									12		Run #4		
									14				
									15				
									15				
10		4	C	60"	60"	11'0"			17		11'0"	(Dolomite seam from 12'0" to 12'8")	
									17		Run #5		
									15				
									18			(Dolomite seam from 16'5" to 17'5")	
15		5	C	60"	60"	16'0"			16		16'0"		
									17		Run #6		
									18			(Dolomite seam from 21'3" to 22'5")	
									20				
									20				
20		6	C	60"	60"	21'0"			25		21'0"	22'9" - Concrete	
		7	C	12"	12"	22'0"			45		22'0"		
									30		Run #7	half Concrete; half Dolomite vertical seam to 24'3"	
									28		Run #8		
25		8	C	36"	36"	25'0"			29		25'0"	Concrete seam to 25'8"	
									25		Run #9		
									27			Gray/White Dolomite (fractured)	
									27				
									28			End of Boring - 30'0"	
30		9	C	60"	60"	30'0"			28		30'0"		

*Note: Hole filled with water on completion-- water level stable for 2 hours.

GROUND SURFACE TO _____ FT. USED _____ CASING THEN _____ CASING TO _____ FT. HOLE NO **V-1**

WASHED PIPIT AUGER UP UNDISTURBED PISTON C=COARSE
UNDISTURBED BALL CHECK T THINWALL V VANE TEST M=MEDIUM
SPLIT TUBE SAMPLER H. S. A. HOLLOW STEM AUGER F=FINE
TRACE = 0-10%, LITTLE = 10-20%, SOME = 20-35%, AND = 35-50%.

SOIL TESTING, INC.

120 Mountain Road
Seymour, Connecticut

CLIENT: **NIAGARA MOHAWK POWER CORP.**

P. O. #23138

SHEET 1 OF 1

HOLE NO. V-2

PROJECT NO

B-742

BORING LOCATIONS

as instructed by Engineer

FOREMAN - DRILLER

DH - AM - RS

PROJECT NAME

INGHAMS HYDRO NON-OVERFLOW DAM

INSPECTOR

LOCATION **TOWN OF MANHEIM**

HERKIMER CO., NEW YORK

OFFSET

GROUND WATER OBSERVATIONS

AT _____ FT. AFTER _____ HOURS

AT _____ FT. AFTER _____ HOURS

CASING SAMPLER CORE BAR
TYPE _____ SIZE 10 _____
HAMMER WT _____ BIT
HAMMER FALL _____ **Diamond**

Date St. 6/11/70 Date Fin. 6/12/70

SURFACE ELEV 573.0'

GROUND WATER ELEV *see note

DEPTH	CASING BLOWS PER FOOT	SAMPLE				DEPTH @ BOT	BLOWS PER 6" ON SAMPLER (FORCE ON TUBE)			CORING TIME PER FT (MIN)	DENSITY OR CONSIST	STRATA CHANGE DEPTH	FIELD IDENTIFICATION OF SOIL REMARKS INCL COLOR, LOSS OF WASH WATER, SEAMS IN ROCK, ETC
		NO	TYPE	PEN	REC		0-6	6-12	12-18				
							MOIST	ELEV					
									7		Run #1	Gray Concrete (first 18" shattered)	
	1	C	18"	12"	1'6"				8		1'6"		
									8		Run #2		
	2	C	24"	24"	3'6"				10		3'6"		
5									10		Run #3		
									11				
	3	C	60"	60"	8'6"				11		8'6"		
									11		Run #4		
10									10				
									10				
	4	C	60"	60"	13'6"				9		13'6"		
									9		Run #5		
15									10				
									11				
	5	C	60"	60"	18'6"				11		18'6"		
									11		Run #6		
20									12				
									12				
	6	C	60"	60"	23'6"				12		23'6"		
									12		Run #7		
25									12				
									11				
									14				
									15				
	7	C	60"	60"	28'6"				15		28'6"		
									15				
30									15				
									18				
												25'10" - Top of Rock	
												Gray/White Dolomite - fractured	
												End of Boring - 28'6"	

*Note: filled hole with water at completion - no elevation change in 3 hours.

GROUND SURFACE TO _____ FT. USED _____ CASING THEN _____ CASING TO _____ FT. HOLE NO. **V-2**

D: DRY W: WASHED P: PIT A: AUGER UP: UNDISTURBED PISTON C: COARSE
 UB: UNDISTURBED BALL CHECK T: THINWALL V: VANE TEST M: MEDIUM
 O.E. - OPEN END SAMPLER S.S. - SPLIT TUBE SAMPLER H.S.A. - HOLLOW STEM AUGER F: FINE
 PROPORTIONS USED: TRACE = 0-10%, LITTLE = 10-20%, SOME = 20-35%, AND = 35-50%.

SOILTESTING, INC.

120 Mountain Road
Seymour, Connecticut

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DD - AM - RS

INSPECTOR

CLIENT: **NIAGARA MOHAWK POWER CORP.**
P.O. #23138

PROJECT NO

B-742

PROJECT NAME

INGHAMS HYDRO NON-OVERFLOW DAM

LOCATION

**TOWN OF MANHEIM
HERKIMER CO., NEW YORK**

SHEET 1 OF 1

HOLE NO. V-2
Phase 2

BORING LOCATIONS

as instructed by Engineer

OFFSET

GROUND WATER OBSERVATIONS

AT _____ FT AFTER _____ HOURS

AT _____ FT AFTER _____ HOURS

CASING SAMPLER CORE BAR
TYPE _____ SIZE 1 D _____
HAMMER WT _____ HAMMER FALL _____
BIT
Diamond

Date Start 6/17/70 Date Fin 6/17/70

SURFACE ELEV 573.0'

GROUND WATER ELEV _____

DEPTH	CASING BLOWS PER FOOT	SAMPLE				DEPTH @ BOT	BLOWS PER 6" ON SAMPLER (FORCE ON TUBE)			CORING TIME PER FT (MIN)	DENSITY OR CONSIST	STRATA CHANGE DEPTH	FIELD IDENTIFICATION OF SOIL REMARKS INCL COLOR, LOSS OF WASH WATER, SEAMS IN ROCK, ETC
		NO	TYPE	PEN	REC		0-6	6-12	12-18				
5												<p>Engineer requested drilling an additional 2 feet.</p> <p>28'6" start Phase 2</p> <hr/> <p>Run #1 30'6" Gray/White Dolomite with vertical and horizontal seams.</p> <hr/> <p>End of Boring - 30'6"</p>	
10													
15													
20													
25													
30		1	C	24"	24"	30'6"			18 20				
35													
40													
45													
50													
55													
60													
65													
70													

GROUND SURFACE TO _____ FT, USED _____ " CASING TWEN _____ " CASING TO _____ FT

D: DRY W: WASHED P: PIT A: AUGER U: UNDISTURBED PISTON
UB: UNDISTURBED BALL CHECK T: THINWALL V: VANE TEST

O.E. - OPEN END SAMPLER S.S. - SPLIT TUBE SAMPLER H.S.A. - HOLLOW STEM AUGER

PROPORTIONS USED: TRACE = 0-10%, LITTLE = 10-20%, SOME = 20-35%, AND = 35-50%

HOLE NO. V-2

Phase 2

C = COARSE

M = MEDIUM

F = FINE

SOILTESTING, INC.

120 Mountain Road
Seymour, Connecticut

FOREMAN - DRILLER

DH - AM - RS

INSPECTOR

CLIENT: NIAGARA MOHAWK POWER CORP.
P.O. #23138

PROJECT NO

B-742

PROJECT NAME

INGHAMS HYDRO NON-OVERFLOW DAM

LOCATION TOWN OF MANHEIM
HERKIMER CO., NEW YORK

SHEET 1 OF 1

HOLE NO. V-3

BORING LOCATIONS

as instructed by Engineer

OFFSKT

GROUND WATER OBSERVATIONS

AT _____ FT AFTER _____ HOURS

AT _____ FT AFTER _____ HOURS

CASING _____ SAMPLER _____ CORE BAR
TYPE _____ SIZE I.D. _____ NYM
HAMMER BY _____ BIT
HAMMER FALL _____ Diamond

Date Start 6/4/70 Date Fin. 6/5/70

SURFACE ELEV 573.0'

GROUND WATER ELEV *see ncte

DEPTH	CASING BLOWS PER FOOT	SAMPLE					BLOWS PER 6" ON SAMPLER (FORCE ON TUBE)			CORING TIME PER FT (MIN)	DENSITY OR CONSIST	STRATA CHANGE DEPTH	FIELD IDENTIFICATION OF SOIL REMARKS INCL COLOR, LOSS OF WASH WATER, SEAMS IN ROCK, ETC
		NO	TYPE	PEN	REC	DEPTH @ BOT	0-6	6-12	12-18				
									7		Run #1	Concrete	
		1	C	24"24"	2'0"				7		2'0"		
									7		Run #2		
		2	C	24"24"	4'0"				8		4'0"	Concrete	
5									9		Run		
									10		#3	Concrete; trace of Dolomite.	
									10				
									13				
		3	C	60"60"	9'0"				11		9'0"		
10									12		Run		
									11		#4		
									11			12'0" - Top of Rock	
									15			Gray/White Shale and Dolomite, Limestone.	
		4	C	60"60"	14'0"				15		14'0"		
15												End of Boring = 14'0"	
20													
25													
30													
15													
40													

*Note: Filled hole with water on completion-water down 21" on June 6.

GROUND SURFACE TO _____ FT, USED _____ CASING THEN _____ CASING TO _____ FT

HOLE NO. V-3

O = DRY W = WASHED P = PIT AT AUGER UP = UNDISTURBED PISTON C = COARSE
 UB = UNDISTURBED BALL CHECK T = THINWALL V = VANE TEST M = MEDIUM
 O.E. = OPEN END SAMPLER SS = SPLIT TUBE SAMPLER H.S.A. = HOLLOW STEM AUGER F = FINE
 PROPORTIONS USED: TRACE = 0-10%, LITTLE = 10-20%, SOME = 20-35%, AND = 35-50%.

SOILTESTING, INC.

120 Mountain Road
Seymour, Connecticut

CLIENT **NIAGARA MOHAWK POWER CORP.**
P.O. #23138

SHEET 1 OF 1
HOLE NO. V-3
Phase 2

FOREMAN - DRILLER
DH - AM - RS

PROJECT NO
B-742

BORING LOCATIONS
as instructed by Engineer

INSPECTOR

PROJECT NAME
INGHAMS HYDRO NON-OVERFLOW DAM
LOCATION
**TOWN OF MANHEIM
HERKIMER CO., NEW YORK**

OFFSET

GROUND WATER OBSERVATIONS

AT _____ FT AFTER _____ HOURS
AT _____ FT AFTER _____ HOURS

CASING TYPE _____
SAMPLER _____
CORE BAR **NXM**
SIZE I.D. _____
HAMMER WT _____
HAMMER FALL _____
BIT **Diamond**

Date Start **6/10/70** Date Fin **6/10/70**
SURFACE ELEV **573.0'**
GROUND WATER ELEV ***see note**

DEPTH	CASING BLOWS PER FOOT	SAMPLE				DEPTH @ BOT	BLOWS PER 6" ON SAMPLER (FORCE ON TUBE)			CORING TIME PER FT (MIN)	DENSITY OR CONSIST	STRATA CHANGE DEPTH	FIELD IDENTIFICATION OF SOIL REMARKS INCL COLOR, LOSS OF WASH WATER, SEAMS IN ROCK, ETC
		NO	TYPE	PEN	REC		0-6	6-12	12-18				
5												Moved back on boring at request of Engineer.	
15									15		14'0"	START Phase 2	
									12		Run #1	Gray/white Dolomite-Limestone.	
									11		15'0"	Concrete	
									11				
		1	C	60"	60"	19'0"			11		19'0"		
									12		Run #2		
20									13				
									14				
									14				
		2	C	54"	54"	23'6"			15		23'6"		
									18		Run #3	Gray/white Dolomite.	
25									17				
									17				
									16				
		3	C	60"	60"	28'6"			17		28'6"		
30												End of Boring - 28'6"	
35													
												*Note: water running out of boring, very slow.	
40													

GROUND SURFACE TO _____ FT. USED _____ " CASING THEN _____ " CASING TO _____ FT

HOLE NO. **V-3**
Phase 2

O. DRY W. WASHED P. PIT A. AUGER U.P. UNDISTURBED PISTON C. COARSE
 U.B. UNDISTURBED BALL CHECK T. THINWALL V. VANE TEST M. MEDIUM
 O.E. - OPEN END SAMPLER S.S. - SPLIT TUBE SAMPLER H.S.A. - HOLLOW STEM AUGER F. FINE
 PROPORTIONS USED: TRACE = 0-10%, LITTLE = 10-20%, SOME = 20-35%, AND = 35-50%.

SOILTESTING, INC.

120 Mountain Road
Seymour, Connecticut

CLIENT: **NIAGARA MOHAWK POWER CORP.**
P. O. #23138

SHEET 1 OF 1
HOLE NO. A-1

FOREMAN - DRILLER

DH - AM - RS

PROJECT NO

B-742

BORING LOCATIONS
as instructed by Engineer

INSPECTOR

PROJECT NAME

INGHAMS HYDRO NON-OVERFLOW DAM

LOCATION **TOWN OF MANHEIM**

HERKIMER CO., NEW YORK

OFFSET

GROUND WATER OBSERVATIONS

AT _____ FT AFTER _____ HOURS

AT _____ FT AFTER _____ HOURS

CASING SAMPLER CORE BAR
TYPE _____ SIZE 1 D _____ HAMMER WT _____ HAMMER FALL _____
BIT
Diamond

Date Start **6/19/70** Date Fin. **6/23/70**

SURFACE ELEV **573.0'**

GROUND WATER ELEV ***see note**

DEPTH	CASING BLOWS PER FOOT	SAMPLE					BLOWS PER 6" ON SAMPLER (FORCE ON TUBE)			CORING TIME PER FT (MIN)	DENSITY OR CONSIST	STRATA CHANGE DEPTH	FIELD IDENTIFICATION OF SOIL REMARKS INCL COLOR, LOSS OF WASH WATER, SEAMS IN ROCK, ETC
		NO	TYPE	PER	REC	DEPTH @ BOT	0-6	6-12	12-18				
							MOIST	ELEV					
									10		Run #1	Gray Concrete	
		1	C	24"	22"	2'0"			9		2'0"		
									10		Run #2		
		2	C	24"	22"	4'0"			10		4'0"		
									12		Run #3	5'2"	
5									15			Layer Dolomite (from 5'2" to 7'8" 7'8")	
									18				
									16				
		3	C	60"	62"	9'0"			14		9'0"	Concrete (from 7'8" to 12'0")	
									14				
10									14		Run #4		
									13			12'0"	
									17				
		4	C	60"	62"	14'0"			19		14'0"	Gray/White Dolomite (from 12'0" to 14'0")	
									14		Run #5		
15									12			Concrete (from 14'0" to 34'6")	
									13				
									13				
		5	C	60"	60"	19'0"			14		19'0"		
									12		Run #6		
20									14				
									12				
									13				
		6	C	60"	60"	24'0"			13		24'0"		
									14		Run #7		
25									13				
									14				
									14				
		7	C	60"	60"	29'0"			14		29'0"		
									13		Run #8		
30									13				
									14				
									15				
		8	C	60"	60"	34'0"			14		34'0"		
									17		Run #9	34'6"	
15									18			Gray/White Dolomite-fractured	
									19				
		9	C	48"	48"	38'0"			20		38'0"		

End of Boring - 38'0"

*Note: Filled hole - water stable in boring--no loss or gain.

GROUND SURFACE TO _____ FT. USED _____ CASING THEN _____ CASING TO _____ FT. HOLE NO. **A-1**

D = DRY W = WASHED P = PIT A = AUGER UP = UNDISTURBED PISTON C = COARSE
 UB = UNDISTURBED BALL CHECK T = THINWALL V = VANE TEST M = MEDIUM
 O.E. = OPEN END SAMPLER S.S. = SPLIT TUBE SAMPLER H.S.A. = HOLLOW STEM AUGER F = FINE
 PROPORTIONS USED: TRACE = 0-10%, LITTLE = 10-20%, SOME = 20-35%, AND = 35-50%.

SOILTESTING, INC.

120 Mountain Road
Seymour, Connecticut

CLIENT: **NIAGARA MOHAWK POWER CORP.**
P. O. #23138

SHEET 1 OF 2
HOLE NO. A-2

FOREMAN - DRILLER
DH - AM - RS

PROJECT NO
B-742

BORING LOCATIONS
as instructed by Engineer

INSPECTOR

PROJECT NAME
INGHAMS HYDRO NON-OVERFLOW DAM

LOCATION
**TOWN OF MANHEIM
HERKIMER CO., NEW YORK**

OFFSET

GROUND WATER OBSERVATIONS
AT _____ FT AFTER _____ HOURS
AT _____ FT AFTER _____ HOURS

CASINO SAMPLER CORP. BAR
TYPE _____ SIZE 1.0 _____
HAMMER WT _____ BIT
HAMMER FALL _____ **Diamond**

Date Start **6/12/70** Date Fin. **6/16/70**
SURFACE ELEV **573.0'**
GROUND WATER ELEV ***see note**

DEPTH	CASING BLOWS PER FOOT	SAMPLE				DEPTH @ BOT	BLOWS PER 6" ON SAMPLER (FORCE ON TUBE)			CORING TIME PER FT (MIN)	DENSITY OR CONSIST MOIST	STRATA CHANGE DEPTH ELEV	FIELD IDENTIFICATION OF SOIL REMARKS INCL COLOR, LOSS OF WASH WATER, SEAMS IN ROCK, ETC
		NO	TYPE	PEN	REC		0-6	6-12	12-10				
5	1	C	18"	13"	1'6"				7		Run #1	Gray weathered concrete - top 12" to 18"	
									8		1'6"		
	2	C	18"	20"	3'0"				10		Run #2		
									18		3'0"		
									22		Run #3		
10	3	C	54"	54"	7'6"				17		-7'6"	Gray Concrete; some decayed stone.	
									19		Run #4		
									13				
									14				
									17				
15	4	C	60"	55"	12'6"				16		12'6"		
									16		Run #5		
									17				
									25				
									23				
20	5	C	60"	60"	17'6"				26		17'6"		
									26		Run #6		
									26		18'6"		
									28		Run #7		
									25		22'6"		
25	6	C	12"	12"	18'6"				25		Run #8		
									25		20		
									20		Run #9		
									19		27'6"		
									21		Run #10		
30	7	C	48"	48"	22'6"				23		32'6"		
									18		Run #11		
									20				
									20				
									19				
35	8	C	60"	57"	27'6"				21		37'6"	Concrete	
									20		Run #11		
									21				
									20				
									20				
40	9	C	60"	60"	32'6"				20				
									21				
									19				
								20					
								20					
								22					
								21					
								20					
								26					

GROUND SURFACE TO _____ FT. USED _____ CASING THEN _____ CASING TO _____ FT. HOLE NO. **A-2**

D: DRY W: WASHED P: PIT A: AUGER UP: UNDISTURBED PISTON C: COARSE
 UB: UNDISTURBED BALL CHECK T: THINWALL V: VANE TEST M: MEDIUM
 O.E. - OPEN END SAMPLER S.S. - SPLIT TUBE SAMPLER H.S.A. - HOLLOW STEM AUGER F: FINE
 PROPORTIONS USED: TRACE = 0-10%, LITTLE = 10-20%, SOME = 20-35%, AND = 35-50%.

SOILTESTING, INC.

120 Mountain Road
Seymour, Connecticut

CLIENT: **NIAGARA MOHAWK POWER CORP.**
P.O. #23138

SHEET 2 OF 2
HOLE NO. A-2

FOREMAN - DRILLER
DH - AM - RS
INSPECTOR

PROJECT NO
B-742
PROJECT NAME
INGHAMS HYDRO NON-OVERFLOW DAM
LOCATION
**TOWN OF MANHEIM
HERKIMER CO., NEW YORK**

BORING LOCATIONS
as instructed by Engineer
OFFSET

GROUND WATER OBSERVATIONS
AT _____ FT AFTER _____ HOURS
AT _____ FT AFTER _____ HOURS

CASING SAMPLER CORE BAR
TYPE _____ SIZE 10 _____ HAMMER WT _____ HAMMER FALL _____
NMX
BIT
Diamond

Date Start **6/12/70** Date Fin **6/16/70**
SURFACE ELEV **573.0'**
GROUND WATER ELEV **#see note**

DEPTH	CASING BLOWS PER FOOT	SAMPLE					BLOWS PER 6" ON SAMPLER (FORCE ON TUBE)			CORING TIME PER FT (MIN)	DENSITY OR CONSIST	STRATA CHANGE DEPTH	FIELD IDENTIFICATION OF SOIL REMARKS INCL COLOR, LOSS OF WASH WATER, SEAMS IN ROCK, ETC
		NO	TYPE	PEN	REC	DEPTH @ BOT	0-6	6-12	12-18				
40		11	C	48"	41"	41'6"			8			Run #11 41'6" Gray concrete. Brown Sandy wash came out of void between 40'6" and 41'6". Gray cement, no stone between 41' and 41'4".	
									6				
									15			Run #12 43'6" Gray/White fractured Dolomite.	
		12	C	24"	20"	43'6"			20				
45												End of Boring - 43'6"	
50													
55													
60													
65													
70													
75													
80													

Note: Water down to 15' at 4:30 PM on June 15, 1970. On June 16, 1970 water flowing out of hole before last 24" run; after run, water advanced up hole at 30"/min. or better.

GROUND SURFACE TO _____ FT. USED _____ CASING THEN _____ CASING TO _____ FT. HOLE NO. **A-2**

D = DRY W = WASHED P = PIT A = AUGER U = UNDISTURBED PISTON C = COARSE
 UB = UNDISTURBED BALL CHECK T = THINWALL V = VANE TEST M = MEDIUM
 O.E. = OPEN END SAMPLER S.S. = SPLIT TUBE SAMPLER H.S.A. = HOLLOW STEM AUGER F = FINE
 PROPORTIONS USED: TRACE = 0-10%, LITTLE = 10-20%, SOME = 20-35%, AND = 35-50%.

SOILTESTING, INC.

120 Mountain Road
Seymour, Connecticut

CLIENT: **NIAGARA MOHAWK POWER CORP.**
P.O. #23138

SHEET 1 OF 1
HOLE NO. A-3

FOREMAN - DRILLER
DH - AM - RS

PROJECT NO
B-742

BORING LOCATIONS
as instructed by Engineer

INSPECTOR

PROJECT NAME
INGHAMS HYDRO NON-OVERFLOW DAM

LOCATION
**TOWN OF MANHEIM
HERKIMER CO., NEW YORK**

OFFSET

GROUND WATER OBSERVATIONS

AT _____ FT AFTER _____ HOURS

AT _____ FT AFTER _____ HOURS

TYPE _____ CASING _____ SAMPLER _____ CORE BAR
SIZE 1 D _____ _____ _____ **NXM**
HAMMER WT _____ _____ _____ BIT
HAMMER FALL _____ _____ _____ **Diamond**

Date Start **6/5/70** Date Fin. **6/9/70**

SURFACE ELEV **573.0'**

GROUND WATER ELEV ***see note**

DEPTH	CASING BLOWS PER FOOT	SAMPLE				DEPTH @ BOT	BLOWS PER 6" ON SAMPLER (FORCE ON TUBE)			CORING TIME PER FT (MIN)	DENSITY OR CONSIST MOIST	STRATA CHANGE DEPTH ELEV	FIELD IDENTIFICATION OF SOIL REMARKS INCL COLOR, LOSS OF WASH WATER, SEAMS IN ROCK, ETC
		NO	TYPE	PEN	REC		0-6	6-12	12-18				
		1	C	24"	24"	2'0"				9	Run #1		
										10	2'0"	Concrete - with large stone aggregate	
		2	C	24"	18"	4'0"				11	Run #2		
										14	4'0"		
5		3	C	18"	24"	5'6"				60	Run #3		
										14	5'6"		
										12	Run		
										13	#4		
		4	C	48"	48"	9'6"				15	9'6"		
10										14	Run		
		5	C	36"	36"	12'6"				13	#5		
										13	12'6"		
										14	Run		
										16	#6		
15										15	Run		
		6	C	60"	60"	17'6"				15	17'6"		
										14	Run		
										15	#7		
20										15	Run		
		7	C	60"	60"	22'6"				16	22'6"		
										15	Run		
										14	#8		
25										15	Run		
		8	C	60"	60"	27'6"				16	27'6"		
										16	Run		
										14	#9		
30										15	Run		
		9	C	60"	60"	32'6"				16	32'6"		
										16	33'0"	Top of Rock	
										16	Run		
										15	#10	Gray/White Dolomite, Limestone fractured.	
35										17	Run		
										19	Run		
										20	Run		
		10	C	60"	60"	37'6"				20	37'6"	End of Boring - 37'6"	

*Note: Making water; very slow stream; 6/9/70 after completion.

GROUND SURFACE TO _____ FT. USED _____ CASING THEN _____ CASING TO _____ FT

HOLE NO. **A-3**

D: DRY W: WASHED P: PIT A: AUGER UP: UNDISTURBED PISTON C: COARSE
 UB: UNDISTURBED BALL CHECK T: THINWALL V: VANE TEST M: MEDIUM
 O.E.: OPEN END SAMPLER S.S.: SPLIT TUBE SAMPLER H.S.A.: HOLLOW STEM AUGER F: FINE
 PROPORTIONS USED: TRACE = 0-10%, LITTLE = 10-20%, SOME = 20-35%, AND = 35-50%.

NIAGARA MOHAWK POWER CORPORATION

INGHAMS DAM
NON-OVERFLOW SECTION

WATER LEVEL OBSERVATIONS

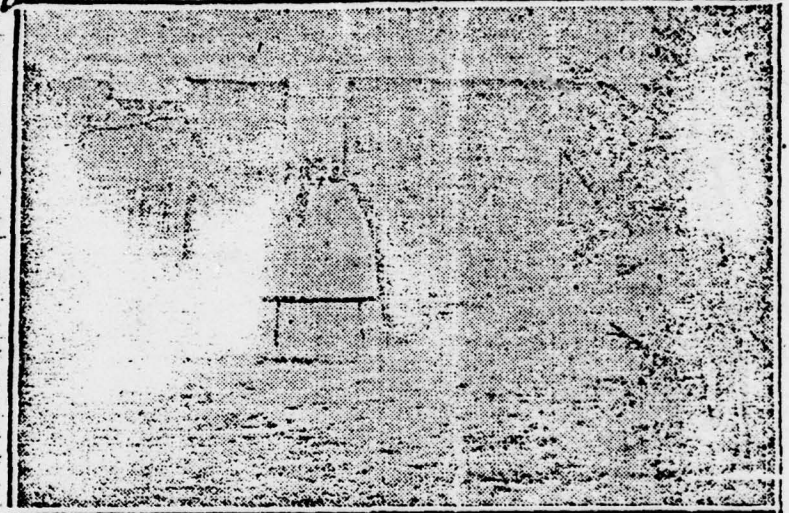
1970	H.W. ELEV.	T.W. ELEV.	AIR TEMP. °F.	ELEVATIONS OF WATER IN HOLES					
				A-1	V-1	A-2	V-2	A-3	V-3
JUNE 12	657.0		68				573.0	573.0	572.29
JUNE 15	658.5		68				573.0	573.0	571.62
JUNE 16	657.3		68				573.0	573.0	571.33
JUNE 17	657.8		68			573.3	573.0	573.0	571.33
JUNE 18	658.0		68			573.3	570.08	573.0	572.25
JUNE 19	662.5	559.7	68		573.0	573.3	573.1	573.0	572.33
JUNE 22	660.3	555.5	57		573.0	573.3	570.75	573.0	572.33
JUNE 23	659.7	555.3	60		572.17	573.3	570.50	573.0	572.25
JUNE 24	656.3	553.3	60	573.0	571.83	573.3	570.00	573.0	572.17
JUNE 25	655.9	552.9	63	573.0	571.37	573.3	569.04	573.0	572.33
JUNE 26	657.7	553.4	67	573.0	570.92	573.3	569.50	573.0	572.17
JUNE 29	661.0	552.5	65	573.0	570.08	573.3	570.08	573.0	572.08
JUNE 30	661.1	553.4	64	573.0	569.83	573.3	569.83	573.0	572.33
JULY 1	661.0	553.5	68	573.0	569.58	573.3	570.33	573.0	572.25
JULY 2	659.0	553.2	68	573.0	569.42	573.3	569.92	573.0	572.33
JULY 7	660.7	553.1	72	573.0	568.67	573.3	569.92	573.0	572.21
JULY 13	660.7	553.1	75	573.0	568.29	573.3	569.67	573.0	572.08
JULY 21	662.0	559.7	74	573.0	568.33	573.3	573.0	573.0	572.29
JULY 27	660.9	553.1	85	573.0	568.08	573.3	571.33	573.0	572.17
AUGUST 4	661.3	553.0	82	573.0	568.67	573.3	570.83	573.0	572.29
AUGUST 11	659.3	552.8	80	573.0	568.33	573.3	570.50	573.0	572.08
AUGUST 18	659.9	553.0	72	573.0	568.25	573.3	570.62	573.0	572.17
SEPT. 1	658.8	552.9	70	573.0	570.08	573.3	571.25	573.0	572.21
SEPT. 9	660.5	553.0	61	573.0	569.83	573.3	571.50	573.0	572.25
SEPT. 15	661.2	553.0	52	573.0	570.00	573.3	571.75	573.0	572.21
SEPT. 21	661.6	553.0	69	573.0	570.17	573.3	572.00	573.0	572.17

APPENDIX G

DRAWINGS

Power Dam at Inghams Mills

Syracuse Herald, Nov. 5, 1911.



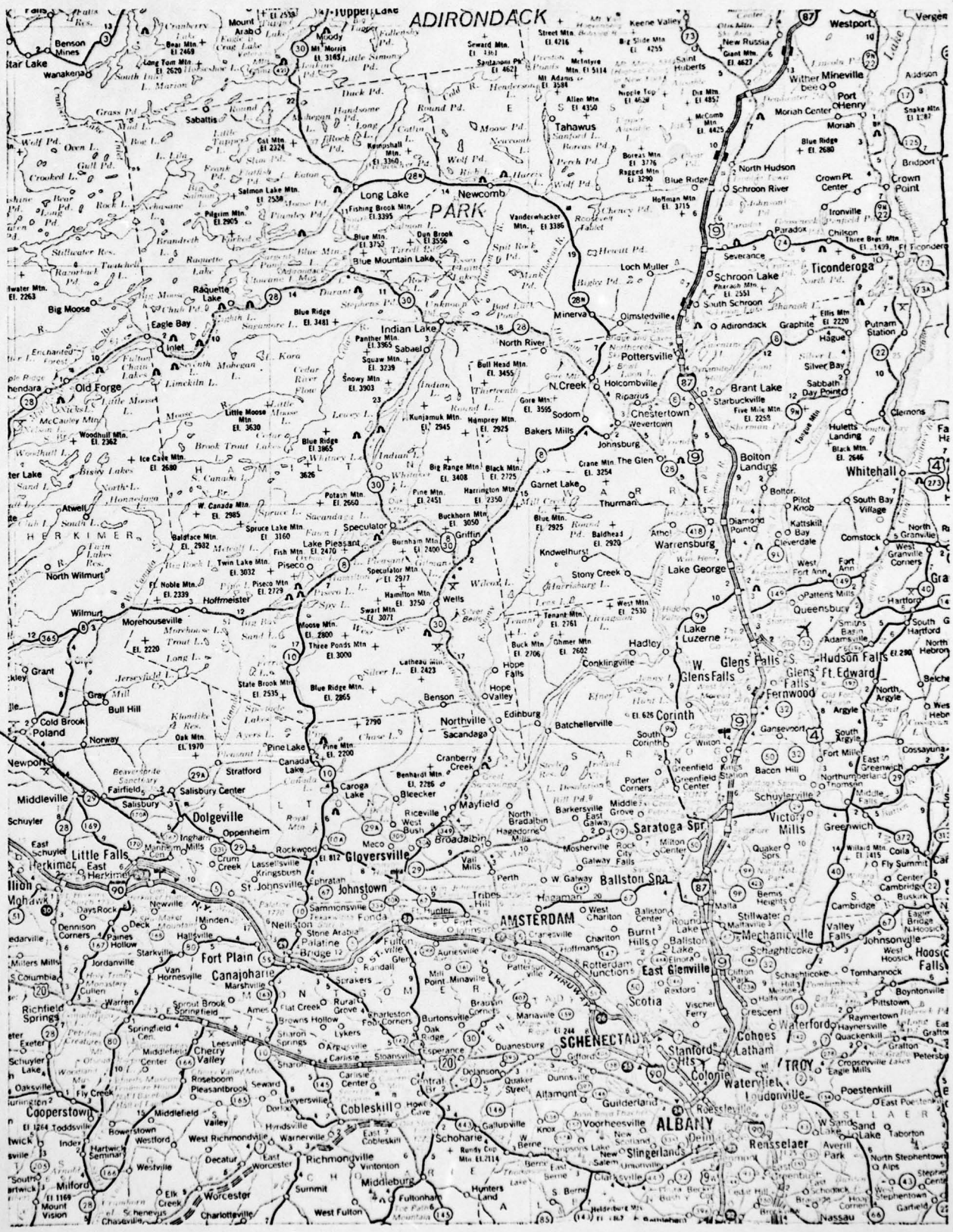
Little Falls, Nov. 4.—The hog dam in the East Canada creek at Inghams Mills is rapidly nearing completion and is attracting many visitors from all parts of the State. The structure is being erected by the East Creek Light and Power company and is three miles east of this city. When completed in about a month it will furnish power that will be sold to the Fonda, Johnstown & Gloversville Railroad company and to the villages in the Mohawk valley east of this city. The proposed Little Falls &

Johnstown Railroad company will also use power from this plant.

The dam is said to be the highest of any in this part of the State and is built in a narrow gully in the East Canada creek. It will flood the valley back as far as Dolgeville. Already the dam is half filled and all that remains for its completion is the building of a small section in the center. The artificial lake that will be made by this large body of water promises to become a popular summer resort for Little Falls.

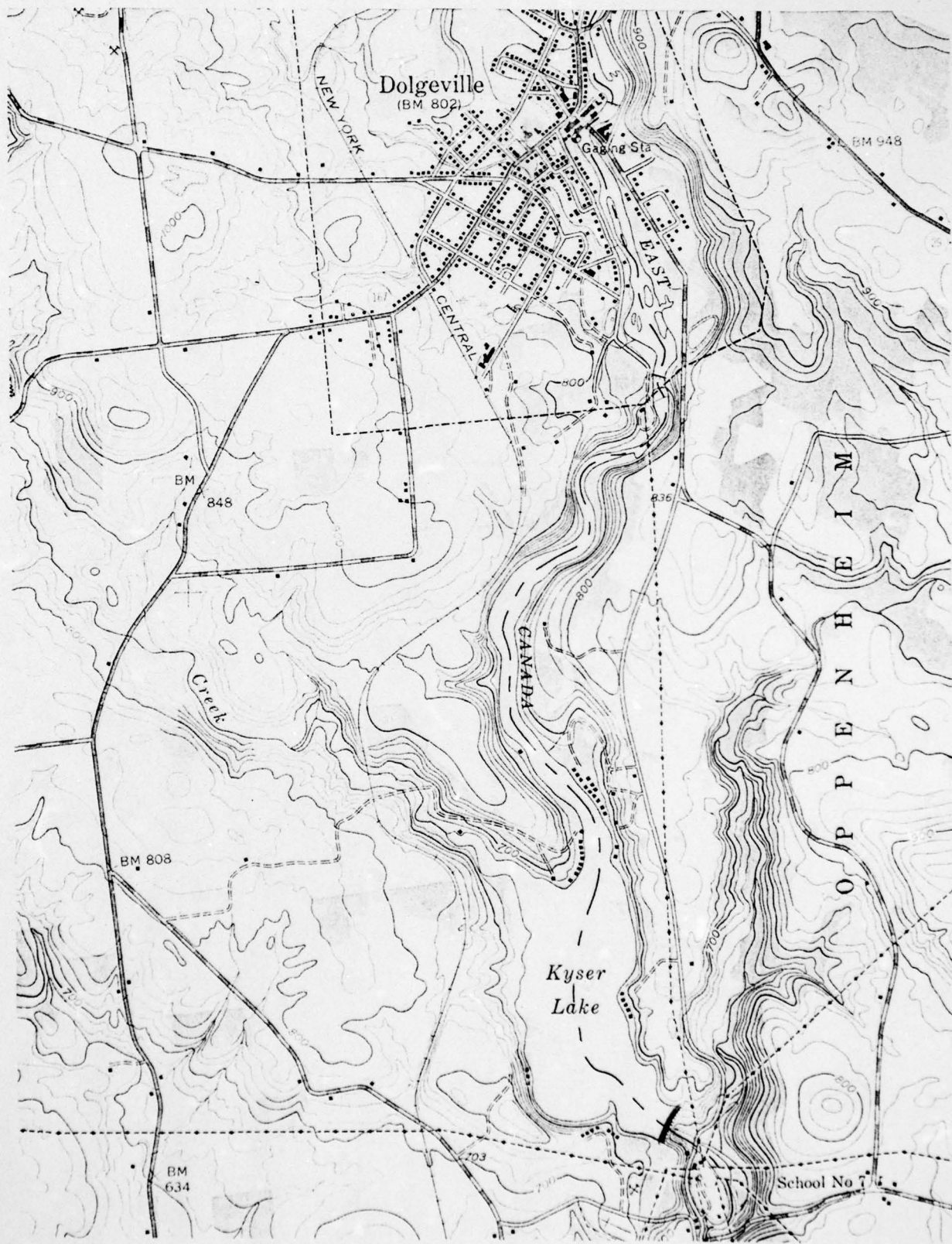
NEWSPAPER CLIPPING FROM SYRACUSE HEARLD

· NOVEMBER 5, 1911



DAM

VICINITY MAP



TOPOGRAPHIC MAP

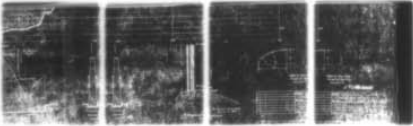
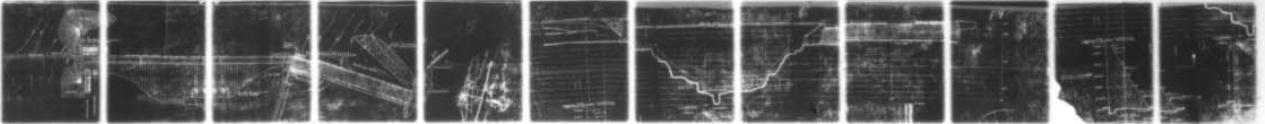
AD-A077 423

NEW YORK STATE DEPT OF ENVIRONMENTAL CONSERVATION ALBANY F/G 13/13
NATIONAL DAM SAFETY PROGRAM. INGHAMS DAM (INVENTORY NUMBER NY 1--ETC(U)
AUG 79 G KOCH DACW51-79-C-0001

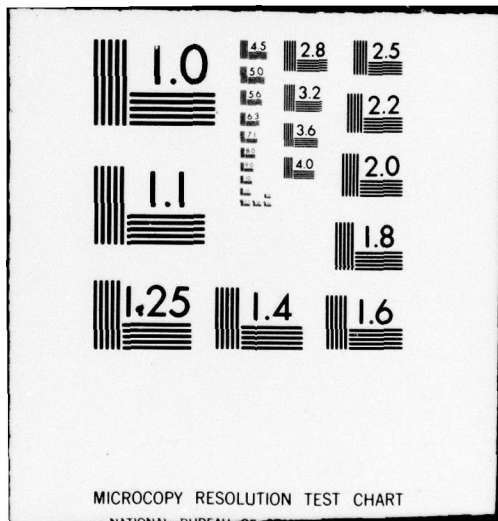
UNCLASSIFIED

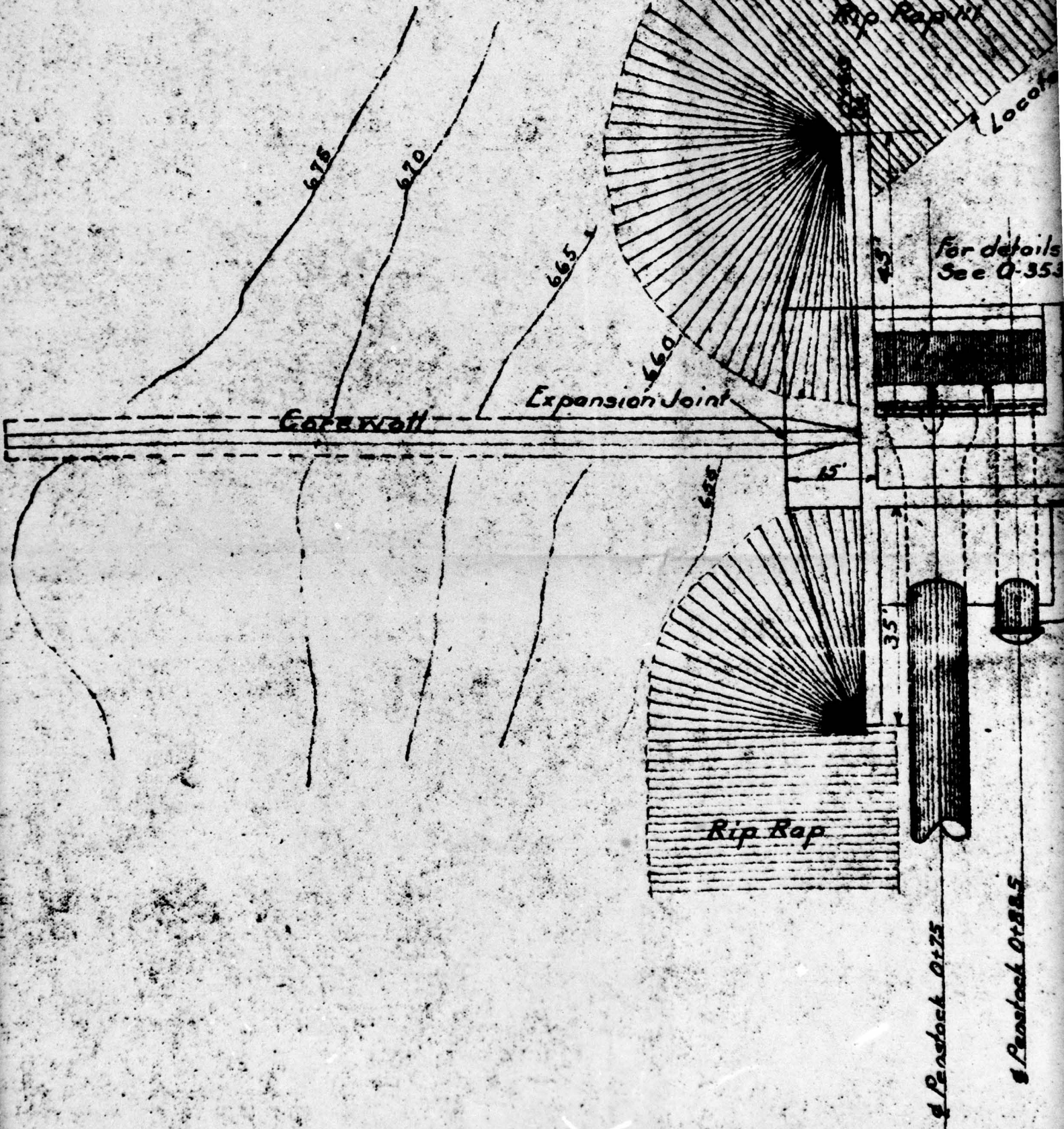
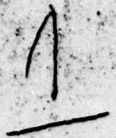
NL

2 OF 2
ADA
077423



END
DATE
FILMED
1-80
DDC





For details
See Q-353

Penstock Q-75

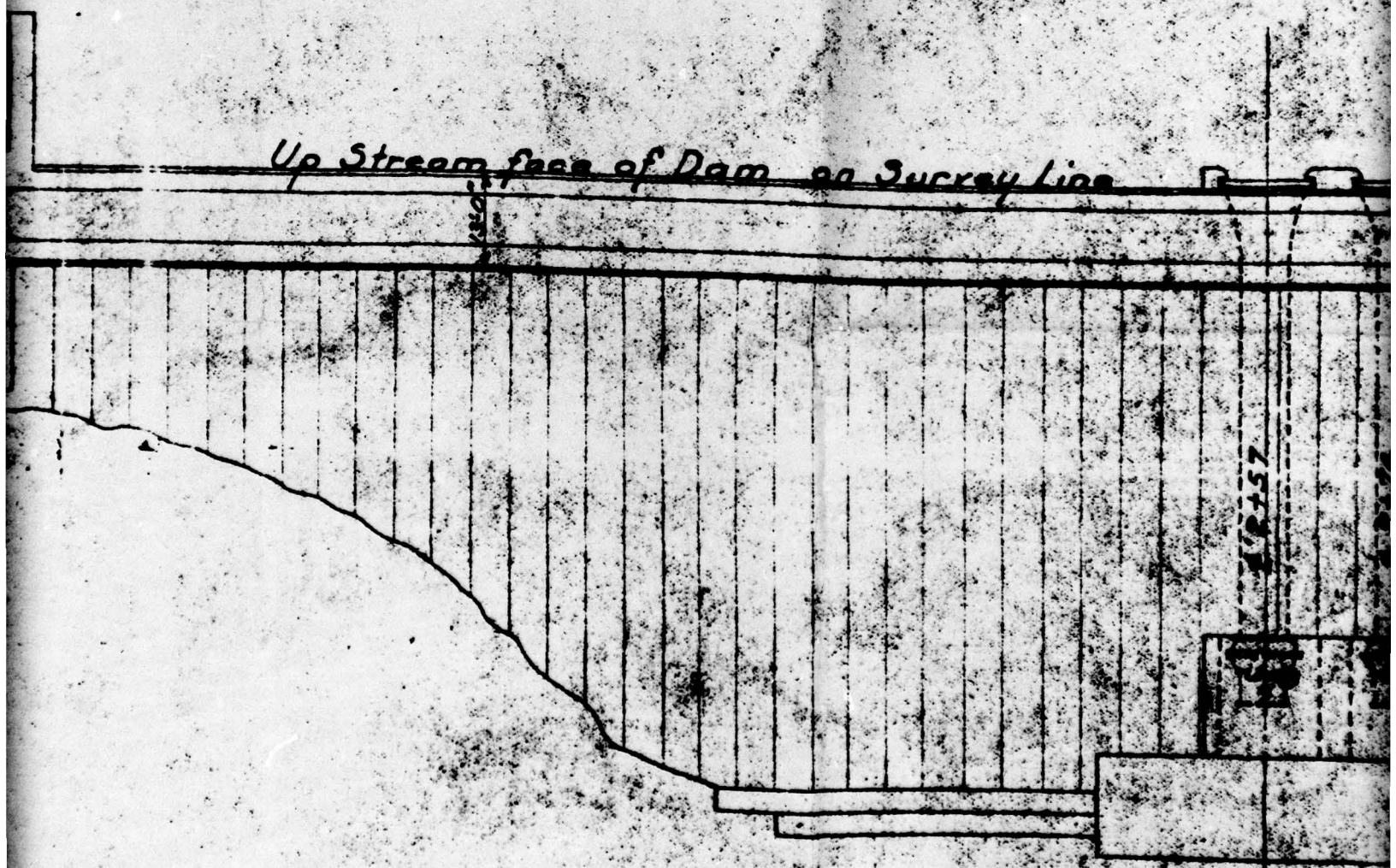
Penstock Q-88.5



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530

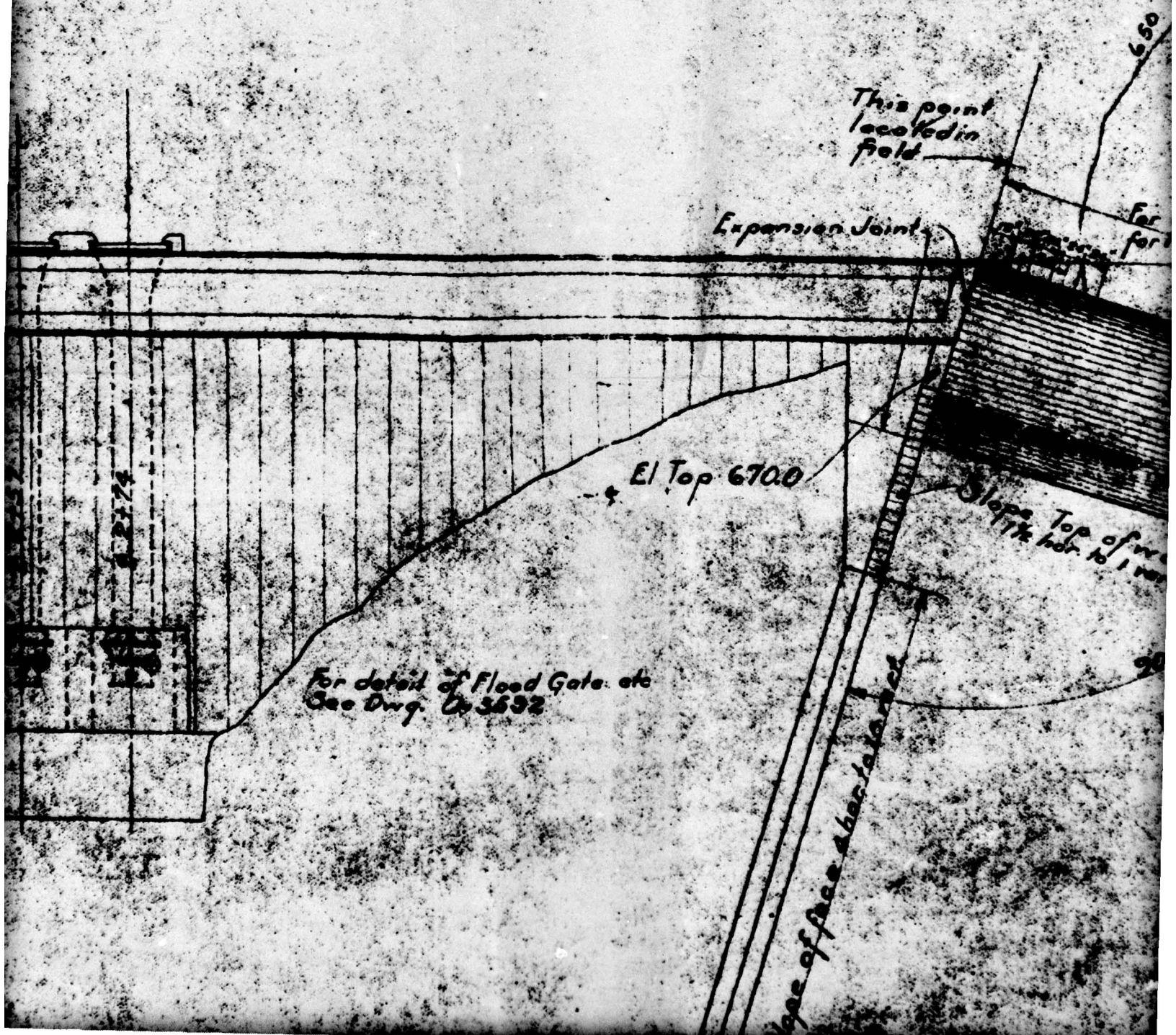
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Up Stream face of Dam on Survey Line

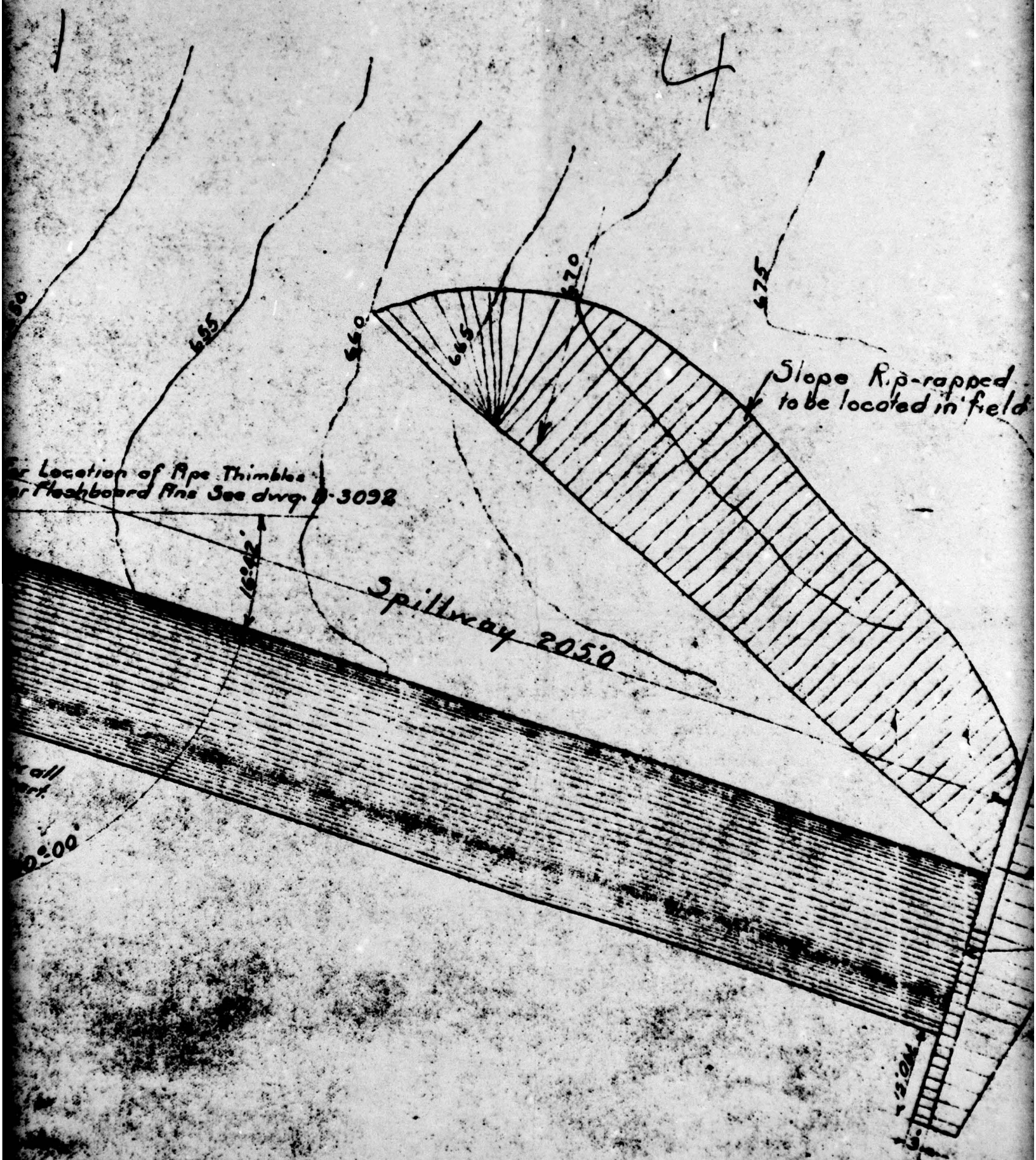


251

3



4



665

660

665

670

675

Slope Rip-rapped to be located in field

For Location of Pipe Thimbles or Flashboard Pins See dwg. D-3098

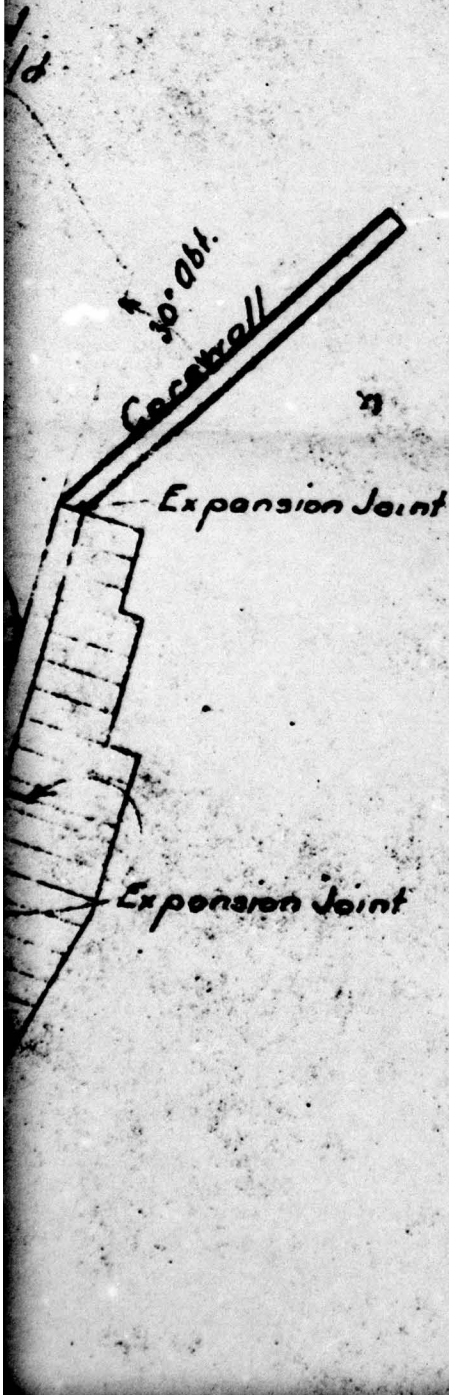
Spillway 205.0

16.42

10:00

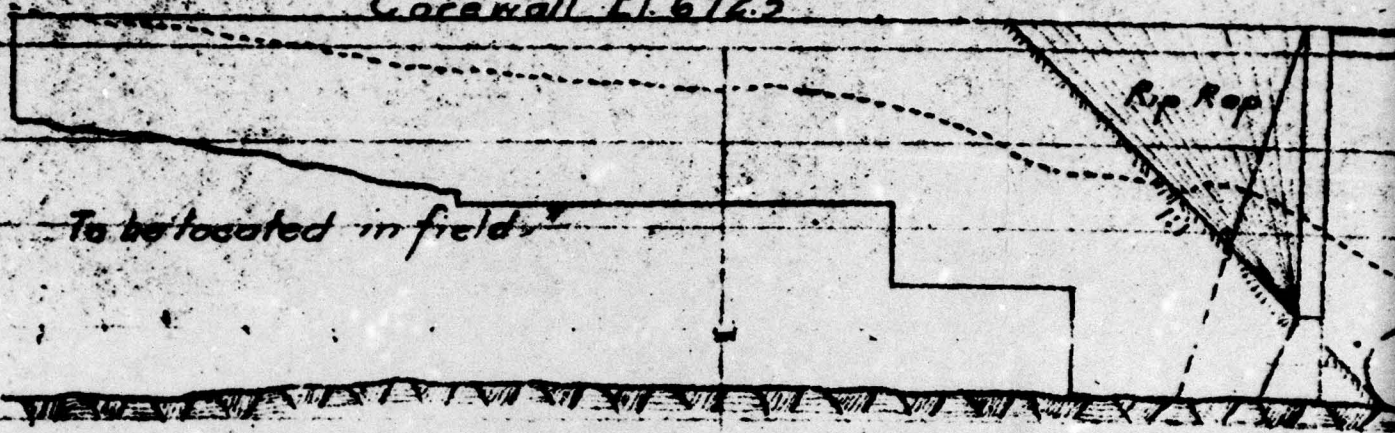
13.04

5



6

Corwall El. 672.5



To be located in field

Rip Rap

H.W. El. 669.0 El. 672.5 El. 670.5

For details of Parapets notes etc. see map G-3530

El. 660.0

12.5' wide

Ditch 32' wide

El. 647.5

12.5' wide

0

50

100

0

Elev. Top of Parapet 6



100

H.W. EL. 6650

100

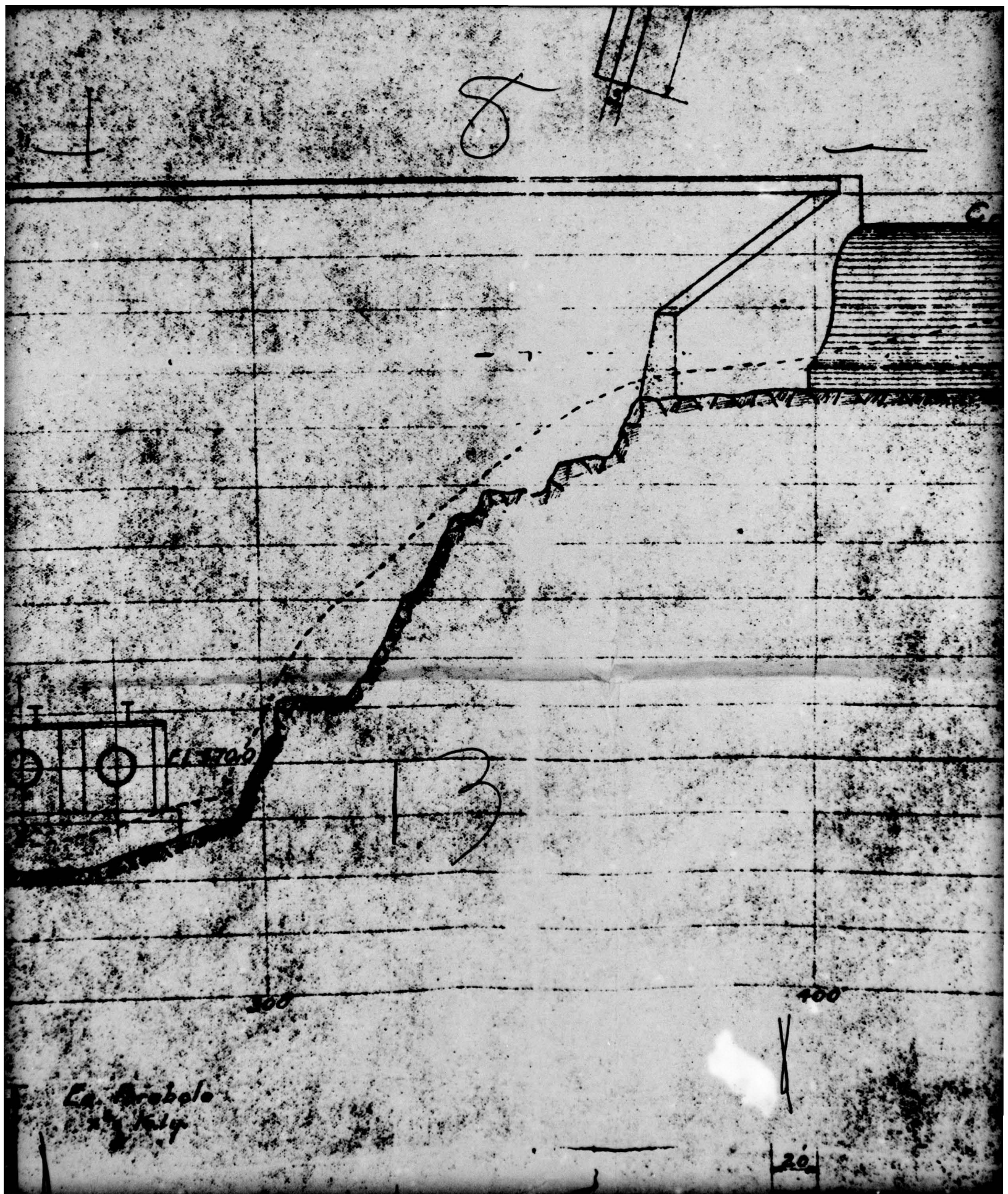
12 Feet space

EL. 6640

See Detail

12





S

C

E. Sirobolo
24 July

B

100

400

20

Rock 636.0

500

El. 668.0

14

El. 672.5'

El. 652.0

10

670

650

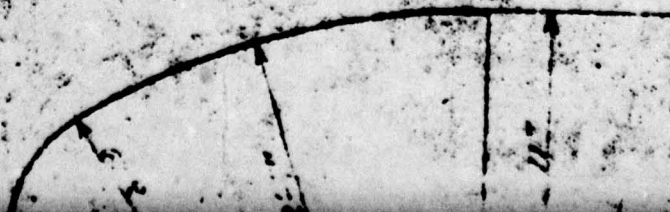
600

550

500

15

16



H.W. El. 669.0 El. 672.5 El. 670.5 For detail of Parapets notes etc. see dwp. C-3530

El. 660.0 12.5' wide
Batter 32' in 10'

El. 647.5 16.5' wide

El. 627.5
Batter 65' in 10'

THIS PAGE IS BEST QUALITY PRACTICE FROM COPY FURNISHED TO DDD.

El. 607.5 42.5'

El. 587.5 55.5'

El. 567.5 69.5' Batter 7' in 10'

El. 547.5 Solid Rock

Section of Dam
July 11, 20

300 8-20

100

200

H.W. EL. 669.0

EL. 664.0

12 Feet

See Detail

12

12'-0"

Quartz

13.5'

13.9'

10'

55.5'

Dotter Tin 10'

H.W. 570.0

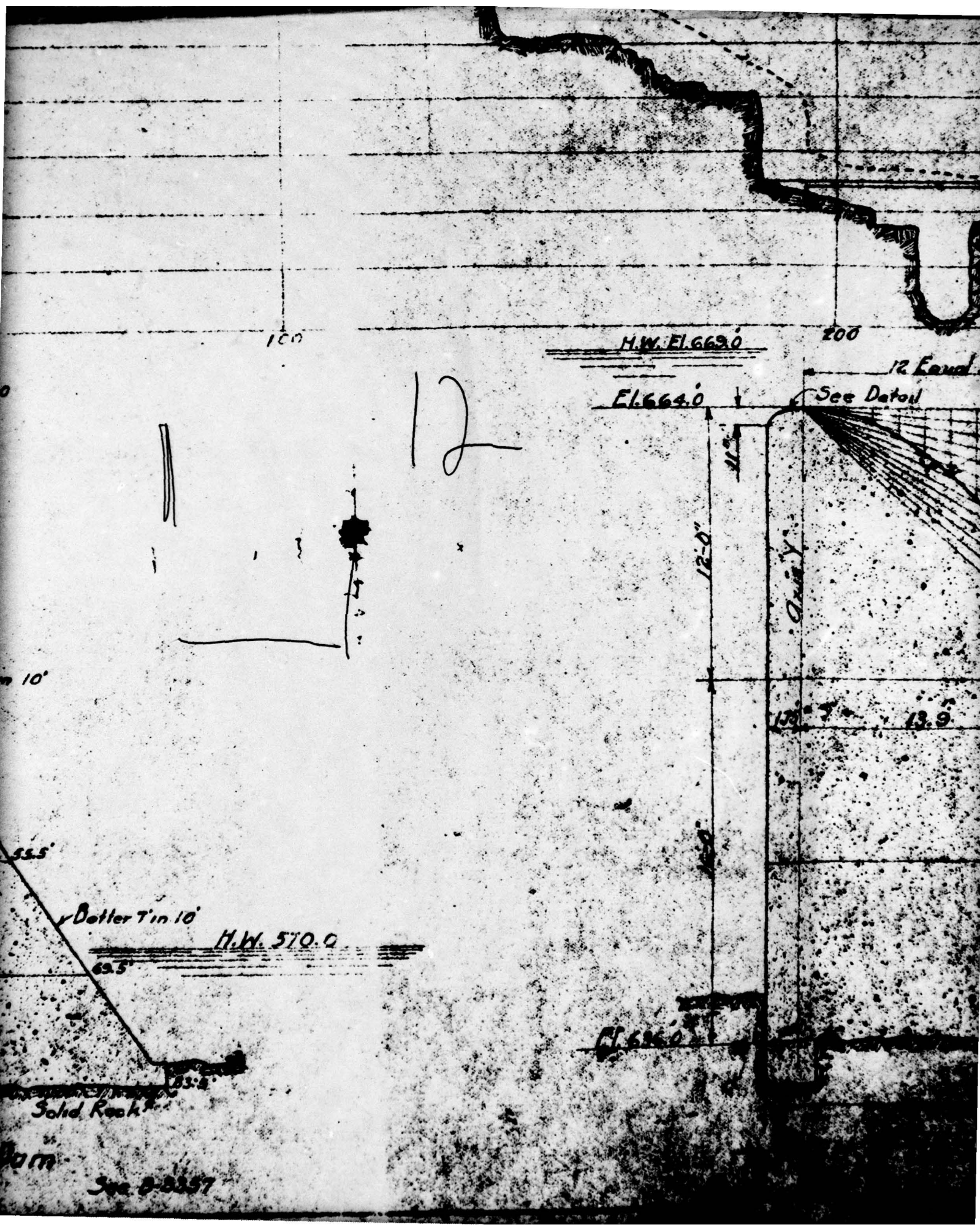
69.5'

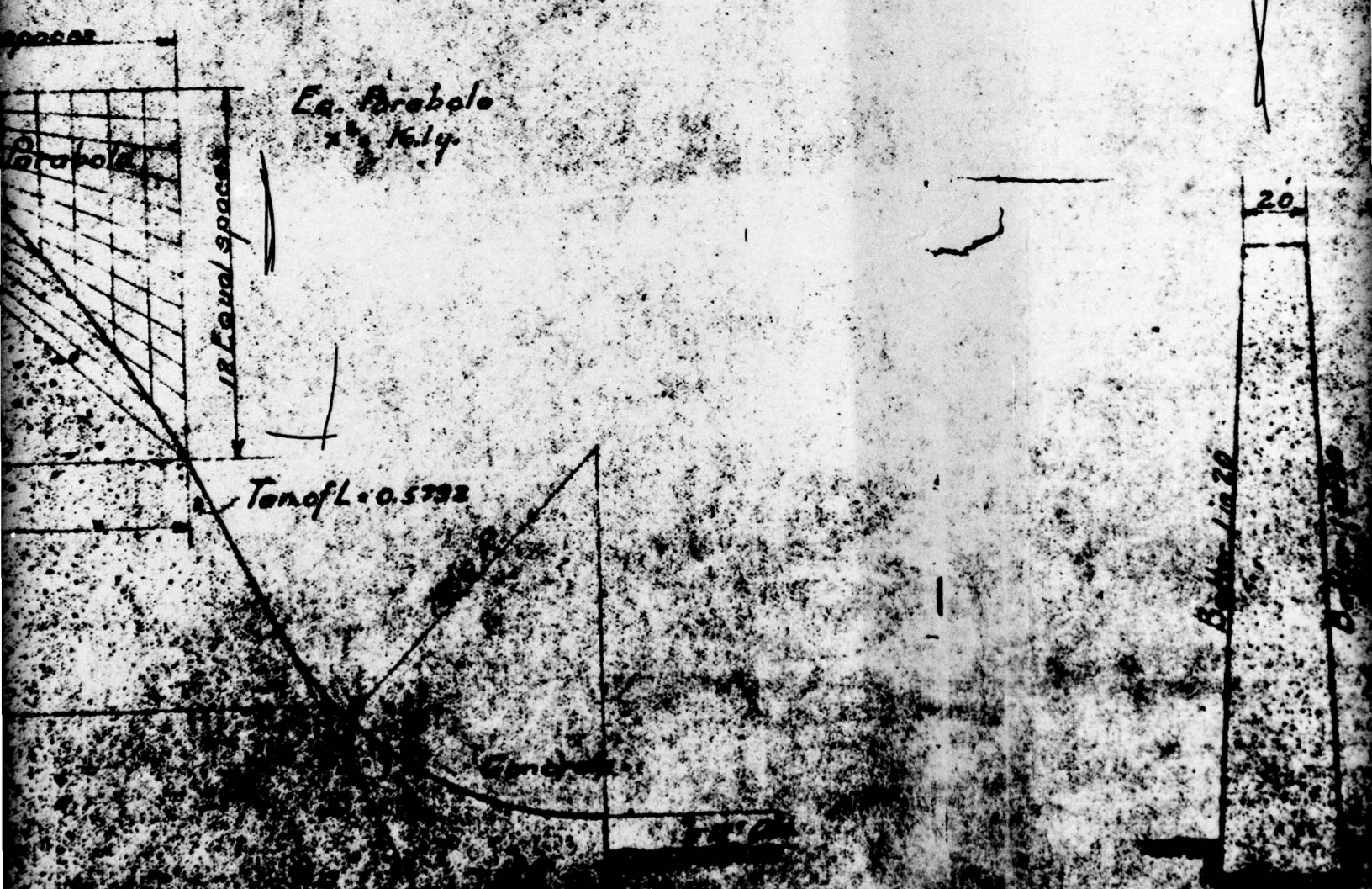
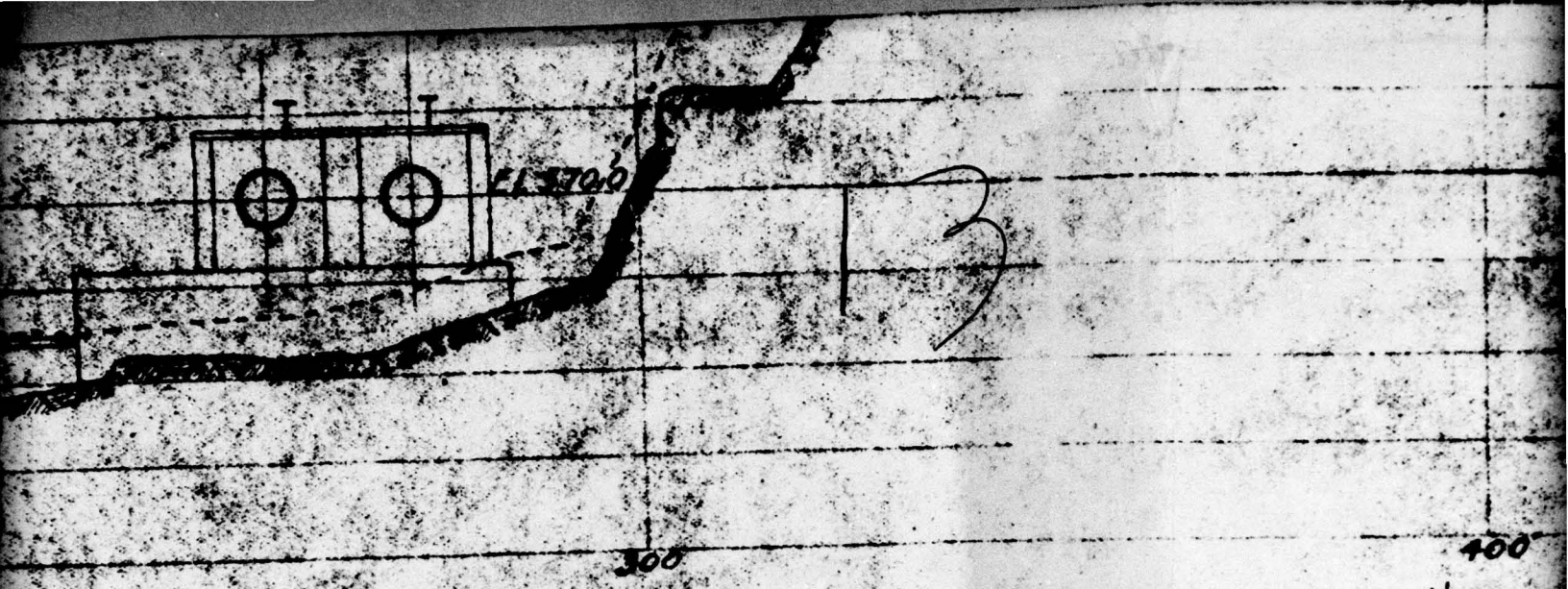
Solid Rock

Dam

See B-3257

EL. 666.0





Section
 Garena
 Con...

100

500

Fl. 66

14

20

Bottom
Bottom

Brass

60

11

6

Section of
General
Case

Arrangement

20

550

700

11"

16

Detail of Inner Edge
Crest of Spillway
Scale 1/2" = 1'-0"

EAST CREEK ELECT. & POWER CO.

DESIGN & CONSTRUCTION
PLAN AND ELEVATION OF SPILLWAY

SCALE 1/2"

YULE CLARKFIELD

CONSTRUCTION

REVISIONS