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NATIONAL DAM SAFETY PROGRAM. LOCK 3 DAM AT MECHANICVILLE (INVEN--ETC(U)
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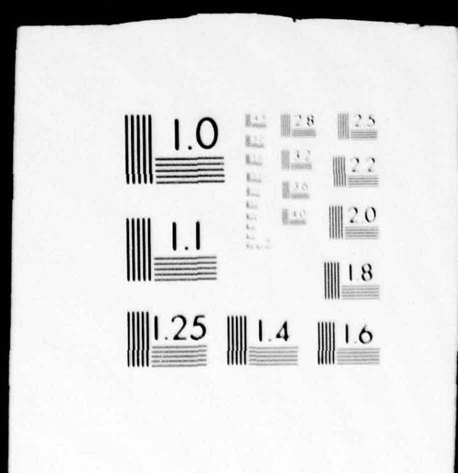

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Lock 3 Dam at Mechanicville
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Inventory No. NY 215

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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.
Lock 3 Dam at Mechanicville was found to have an inadequate spillway capacity, additional investigation to better determine the structural stability of the dam was also recommended.

UPPER HUDSON RIVER BASIN

LOCK 3 DAM AT MECHANICVILLE

RENSSELAER - SARATOGA COUNTY

NEW YORK

INVENTORY NO. N.Y. 215

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

CONTRACT NO. DACW-51-79-C0001

NEW YORK DISTRICT CORPS OF ENGINEERS

FEBRUARY ,1979

PREFACE

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

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

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

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

PHASE 1 INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM
LOCK 3 DAM @ MECHANICVILLE
I.D. No. NY - 215
(#119 - UH)
UPPER HUDSON RIVER BASIN
RENSSELAER - SARATOGA COUNTY, NEW YORK

TABLE OF CONTENTS

PHASE 1 REPORT
NATIONAL DAM SAFETY PROGRAM

Name of Dam:	Lock 3 Dam @ Mechanicville I.D. No. NY-215 (#119-UH)
State Located:	New York
County Located:	Rensselaer - Saratoga
Watershed:	Upper Hudson River Basin
Stream:	Hudson River
Date of Inspection:	October 26, 1978

ASSESSMENT

Examination of available documents and a visual inspection of the dam did not reveal conditions which constitute an immediate hazard to human life or property. However, additional studies should be undertaken to further evaluate conditions affecting the dam.

Additional data and analysis is needed to ascertain the condition and stability of the wrought-iron framework supporting the apron slabs. Also, a more detailed structural stability analysis of the dam using the site-specific foundation material characteristics is also recommended. Such analyses should be performed in accordance with the Corps of Engineers Guidelines, Chapter 4, paragraph 4.4, included in Appendix G.

All additional data gathering, investigations, and analyses should be completed within one year of the date of this Phase 1 report. During the interim period, a detailed emergency-operation plan and warning system should be developed and implemented.

The spillway, not having sufficient discharge capacity for passing one-half the Probable Maximum Flood (PMF), is considered to be inadequate. For such a large storm event, a high tailwater condition would most likely occur resulting in the flooding of the downstream hazard areas. Hence, dam failure from overtopping would not significantly increase the hazard to loss of life downstream from that which would exist just before overtopping failure.

Minor deficiencies found during the visual inspection were limited to concrete surface deterioration and cracking and the overgrowth of vegetation on the earth dike. Such deficiencies should be corrected during normal maintenance operations before the next period of anticipated high river flows (Spring 1980).

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

Approved By:

Col. Clark H. Benn
New York District Engineer

Date:

22 June 79

LOCK 3 DAM @ MECHANICVILLE
(Looking West)

LOCK 3 DAM @ MECHANICVILLE
(Looking East)

PHASE 1 INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM
LOCK 3 DAM @ MECHANICVILLE
I.D. No. NY - 215
(#119 - UH)
UPPER HUDSON RIVER BASIN
RENSSELAER - SARATOGA 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
This inspection was conducted to evaluate the existing conditions of the 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
The Lock 3 Dam at Mechanicville is a masonry gravity dam having a reinforced concrete cap and a downstream precast concrete slab apron supported by a wrought iron framework. The masonry overflow section is 700 feet long and 19 feet high. In addition to the masonry "main dam" portion, easterly extensions include the 98 foot wide navigation Lock 3 and a 520 foot long concrete faced earth dike. Extending westerly is a 135 foot wide gatehouse-forebay structure containing 12 gates leading to the West Virginia Pulp and Paper Company manufacturing plant water supply channel.

b. Location
The dam is located on the Hudson River, northeast of the City of Mechanicville and east of State Routes 4, 32, and 67.

c. Size Classification
This dam is 37 feet high and impounds a reservoir of 3420 acre-feet. It is classified as an "intermediate" size dam (storage capacity between 1000 and 50,000 acre-feet).

d. Hazard Classification
The dam is classified "high" hazard because of the immediate downstream populations located at Hemstreet Park in the Town of Schaghticoke and the City of Mechanicville.

e. Ownership

The Lock 3 Dam is owned by the State of New York - Department of Transportation (NYS-DOT), Waterways Maintenance Subdivision. It is located in DOT-Region 1, whose headquarters are in Albany, New York.

The original dam constructor/owner was the Hudson River Water-Power and Paper Company (HRW-PPC). The West Virginia Pulp and Paper Company (WVPPC), successor in title to the dam from HRW-PPC, transferred ownership to New York State in January 1922. In February 1979, the property of the Saratoga Board Mills Corporation, successor in title to the gatehouse-forebay structure from WVPCC, was obtained in a foreclosure sale by the City National Bank of Detroit, Michigan, one of the principal lienholders against the Mechanicville property. More recent developments regarding the gatehouse-forebay structure ownership is contained in a newspaper article included in Appendix D.

Waterways Maintenance Subdivision:

New York State - DOT
Main Office - State Campus
1220 Washington Avenue
Albany, New York 12232

Director - Mr. Joseph Stellato
(AC - 518) 457-4420

Region One:

New York State - DOT
84 Holland Avenue
Albany, New York 12208

Waterways Maintenance:
Engineer - Mr. John Hulchanski
(AC 518) 474-6715

f. Purpose of Dam

The primary purpose is for navigation through Lock 3 of the Champlain Barge Canal. The impounded waters of the Hudson River provide a storage pool used for gravity inflow to Lock 3. Supplementary purposes include flood control and possible hydroelectric generation from the presently non-operational power station located on the westerly side of the river.

g. Design and Construction History

The original dam at this site was constructed by the Hudson River Water-Power and Paper Company about the year 1882. In 1912, it existed as a masonry dam with a timber downstream apron supported by an iron-steel framework. The dam was reconstructed to its present reinforced concrete crest cap and precast concrete slab apron in 1965.

h. Normal Operational Procedures

Water flows unregulated over the "main dam" spillway. Flow diversions from the storage pool occur by gravity through the intakes of the Lock during boat passage and minimally through the forebay gates.

1.3 PERTINENT DATA

a. Drainage Area (square miles) 4500

b. Discharges at Dam (cfs)

Top of Dam (Top of Earth Dike)	122,100
Top of Lock 3 (River-side Abutment)	77,600
Hydroelectric power station existing machinery (12 units)	
Design:	6,043 (max.)
Operating:	5,893

c. Elevations (Barge Canal Datum - BCD)

Top of Dam (Top of Earth Dike)	83.0
Top of Lock 3 (River-side Abutment)	77.0
Spillway Crest	67.5
Maximum Tailwater (March 19, 1936)	60.70
Minimum Tailwater (March 7, 1944)	46.55
Reservoir Pool (USGS Datum) (USGS Mechanicville, NY 7.5' Quad. - 1954)	66.0

Datum Conversion:

USGS 0.0 equals BCD 1.18

d. Reservoir

	Surface Area (acres)
Top of Dam (Top of Earth Dike) Max. Pool	350
Top of Lock 3 - Maximum Normal Pool	350
Spillway Crest - Normal Pool	260

e. Storage Capacity

	(acre-feet)
Top of Dam (Top of Earth Dike)	8785
Top of Lock 3	6685
Spillway Crest	3420

f. Dam

Type: Masonry, with a reinforced concrete cap and a precast concrete slab apron.

		(feet)
Length:	Spillway Crest	700
	Lock 3	98
	Earth Dike (embankment)	520
	Gatehouse - Forebay	135

		(feet)
Height:		
	Structural	37

		(feet)
Width @ Crest:		
	Spillway (radius)	2.3
	Earth Dike @ Abutment	39
	Normal	60

		(V : H)
Slopes:		
	Spillway - upstream face	1:2
	- downstream apron	1:1.75
	Earth Dike - upstream face	1:1
	- downstream face	1:1

g. Spillway

Type: Uncontrolled, gravity masonry structure with a rounded reinforced concrete cap and a precast concrete slab apron.

Weir Length (feet)	700
Crest Elevation (Barge Canal Datum)	67.5

h. Reservoir Drain

None

SECTION 2: ENGINEERING DATA

2.1 DESIGN

a. Geology

The Lock 3 Dam is located in the Hudson Valley Lowlands physiographic province of New York State. Rock in this area was formed during the Ordovician period. The rock in these areas is predominantly limestone and dolostone. The present surficial soils have resulted primarily from glaciations during the Cenozoic Era; the Wisconsin glaciation being the most recent event to affect this area, having occurred approximately 11,000 years ago.

b. Subsurface Investigations

No records of subsurface investigations were available. Based on the plans which were available for this structure, it appears that the structure is founded on bedrock.

c. Dam and Appurtenant Structures

Records indicate that the dam was constructed about the year 1882. No information was available concerning the original design of the dam. The dam has been reconstructed several times since first constructed. Drawings for the latest reconstruction performed under DOT Contract M65-6 in 1965, are included in Appendix H.

2.2 CONSTRUCTION RECORDS

No records were available for the original dam construction. The only records available were from the 1965 reconstruction.

2.3 OPERATION RECORD

The dam is visually inspected on an irregular basis by engineers from NYS-DOT. Mean daily water levels are recorded at locations both upstream and downstream of the lock. These records began in 1917 and are on file at the N.Y.S. DOT Region One, Waterways Office.

2.4 EVALUATION OF DATA

The data presented in this report was obtained from the files of the Department of Environmental Conservation, the New York State Department of Transportation, the New York State Electric and Gas Corporation and the Federal Energy Regulatory Commission. The information available appears to be adequate and reliable for Phase I inspection purposes.

SECTION 3: VISUAL INSPECTION

3.1 FINDINGS

a. General

Visual inspection of the Lock 3 Dam at Mechanicville and the surrounding area was conducted on October 16, 1978. The weather was cloudy and overcast with the temperature near 50° F. Depth of flow over the spillway crest was approximately 1.65 feet.

b. Dam - Spillway

The "main portion" of the dam, the spillway, could not be observed because of submergence. However, the vertical and horizontal alignment of the crest appeared to be satisfactory.

The following deficiencies were observed:

1. Concrete spalling on the river-side face of the lock upstream protection pier.

2. Smaller areas of spalled concrete on the outer lock walls both upstream and downstream of the crest, in the zone of flow aeration.

3. At the spillway-gatehouse contact, seepage through one masonry block horizontal joint immediately adjacent to the crest was observed.

c. Appurtenant Structures

The navigation Lock 3 was in satisfactory condition. The functioning of the 12 forebay gates could not be ascertained although some flow was occurring downstream of the gatehouse.

The following deficiencies were observed:

1. The concrete surfacing on the Lock-earth dike masonry abutment was deteriorated considerably; having aggregate exposed and surface corners well-rounded from weathering.

Earth Dike-Embankment:

2. The concrete facing on the dike embankment's upstream slope protection was cracked.

3. The vertical alignment of the crest was somewhat irregular sloping downward in the upstream direction; a vehicle path existed on the top of the dike.

4. Several large trees existed on the top of the embankment.

5. Small trees and brush existed on the downstream slope of the embankment.

SECTION 3: VISUAL INSPECTION

3.1 FINDINGS

a. General

Visual inspection of the Lock 3 Dam at Mechanicville and the surrounding area was conducted on October 26, 1978. The weather was cloudy and overcast with the temperature near 50° F. Depth of flow over the spillway crest was approximately 1.65 feet.

b. Dam - Spillway

The "main portion" of the dam, the spillway, could not be observed because of submergence. However, the vertical and horizontal alignment of the crest appeared to be satisfactory.

The following deficiencies were observed:

1. Concrete spalling on the river-side face of the lock upstream protection pier.

2. Smaller areas of spalled concrete on the outer lock walls both upstream and downstream of the crest, in the zone of flow aeration.

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Earth Dike-Embankment:

2. The concrete facing on the dike embankment's upstream slope protection was cracked.

3. The vertical alignment of the crest was somewhat irregular sloping downward in the upstream direction; a vehicle path existed on the top of the dike.

4. Several large trees existed on the top of the embankment.

5. Small trees and brush existed on the downstream slope of the embankment.

Forebay:

6. Seepage through the east wall was occurring in three areas which were all in excess of 200 feet downstream of the axis of the dam.

7. Cracked concrete and joint separation in the masonry was evident along much of the east wall.

d. Reservoir

A low area exists along the edge of the river due east of the north end of the upstream protection pier, the apparent result of an excavation made to allow vehicle access to the river edge.

There was no noticeable signs of soil instability in the reservoir area.

e. Downstream Channel

No unusual conditions were noticed in the downstream Hudson River channel.

3.2 EVALUATION OF OBSERVATIONS

Visual observations did not reveal any problems which would affect the immediate safety of the dam. The deficiencies observed can be corrected during normal maintenance operations. The functioning of the forebay gates is an uncertainty requiring further investigation.

SECTION 4: OPERATION AND MAINTENANCE PROCEDURES

4.1 PROCEDURE

Normal surface is at or slightly above the uncontrolled spillway crest. Flow diversions occur through the navigation Lock and to a lesser extent, through the forebay gates. Presently, the forebay diversion is minimal since neither the paper mill nor the hydroelectric machinery is operating.

4.2 MAINTENANCE OF DAM

Maintenance of the spillway portion of the dam is minimal because of continuous submergence of the crest. During the 1965 reconstruction work, inspection of the iron and steel apron support framework revealed no major structural deterioration.

4.3 MAINTENANCE OF APPURTENANT STRUCTURES

Maintenance of Lock 3 is satisfactory. However, the earth dike has not been maintained as evidenced by trees and brush growing on the downstream slope. The concrete slope protection facing also requires some remedial work to prevent further deterioration.

Maintenance work conducted in the gatehouse-forebay area is unknown. However, since the paper mill has been substantially inoperative for several years, maintenance has probably been minimal.

4.4 WARNING SYSTEM IN EFFECT

No apparent warning system is present.

4.5 EVALUATION

Operation and maintenance of the Lock is satisfactory. Additional maintenance is needed to remedy the earth dike deficiencies stated above. In addition, all masonry and concrete structures including the gatehouse-forebay should be repaired as necessary.

SECTION 5: HYDROLOGIC/HYDRAULIC

5.1 DRAINAGE AREA CHARACTERISTICS

The delineation of the contributing watershed to this dam is shown on the map titled "Drainage Area - Lock 3 @ Mechanicville" (Appendix D). With the drainage area encompassing some 4500 square miles including portions of Vermont and Massachusetts, the Hudson River main stem travels approximately 140 miles from its headwaters south of Lake Placid to the Lock 3 Dam. Major tributaries to the Hudson River are the Cedar, Indian, Boreas, Schroon, Sacandaga, and Hoosic Rivers and the Batten Kill. Numerous lakes including Brant, Schroon, Piseco and Saratoga lie within the basin as well as three major reservoirs; Indian Lake, the Tomhannock, and the Sacandaga Reservoir. Approximately one-half to two-thirds of the basin lies within the Adirondack Mountain area where elevations rise to +5344 at Mount Marcy. Elevations at the east abutment of the dam are near +70. Developed land use has occurred in the lower portion of the basin; the larger developments being the municipalities of Warrensburg, Glens Falls, Hudson Falls, Saratoga Springs; Arlington, Vermont; Greenwich, Schuylerville, Cambridge; Bennington, Vermont; Adams, North Adams, and Williamstown, Massachusetts; and Hoosick Falls.

5.2 ANALYSIS CRITERIA

No hydrologic/hydraulic information was available regarding the original design for this dam. Therefore, the analysis of the spillway capacity of the dam was performed using streamflow gaging station records (Appendix D) and data contained in a Corps of Engineer report entitled "Upper Hudson and Mohawk River Basins Hydrologic Flood Routing Models". The methodology described in this report employed the Corps of Engineers HEC-1 computer program in developing a model that correlated well with past known major storm events; i.e., the storms of October, 1945, December, 1948 and June, 1972. No direct computer analysis using HEC-1 was performed. The spillway design flood selected for analysis was the PMF in accordance with recommended guidelines of the U.S. Army Corps of Engineers.

5.3 SPILLWAY CAPACITY

The single, masonry and concrete, overflow spillway acts as the dam in forming the reservoir pool for the navigation Lock. The 700 foot long ungated spillway has a rounded reinforced concrete cap and a sloping, precast concrete slab apron on the downstream face.

For computed discharges, a discharge coefficient C, of 3.8 was used, determined from the gage readings of March 28, 1913. Computed discharges are as follows:

STAGE	DISCHARGE (cfs)
Top of Dam (Top of Earth Dike)	122,170
Top of Lock 3 (River-side Abutment)	77,670

Maximum discharges through the hydroelectric power station existing machinery (12 units) was determined to be 6,043 cfs.

The spillway does not have sufficient capacity for discharging the peak outflow from one-half the PMF. For this storm event, the peak inflow and peak outflow is 191,000 cfs, whereas the PMF peak discharge is 382,000 cfs. However, the dam has conveyed a maximum discharge of 120,000 cfs. The computed peak discharge capacity is 122,170 cfs.

5.4 RESERVOIR CAPACITY

The normal water surface is at or slightly above the spillway crest. Storage capacity for that crest elevation is 3420 acre-feet. Surcharge storage capacity to the Top-of-Lock elevation is 3265 acre-feet. The total storage capacity to the Top-of-Dike elevation is 8785 acre-feet. The upstream limits of the reservoir are the Lock 4 gates, the low dam across the river approximately 700 feet south of the State Route 67 bridge, and the lower reaches of the Hoosic River.

5.5 FLOODS OF RECORD

The maximum known discharge occurred on March 28, 1913 when a flow of 120,000 cfs was recorded. The computed water surface elevation for this flow is 82.8 (BCD) (81.62 - USGS) or a flow depth to within 5 inches of the top of the earth dike.

5.6 OVERTOPPING POTENTIAL

Analysis indicates the spillway does not have sufficient discharge capacity for either the PMF or one-half the PMF. The computed depths of overtopping are 13.9 feet and 5.7 feet respectively. All storms exceeding approximately 32% of the PMF would result in overtopping of the earth dike.

5.7 EVALUATION

The spillway capacity is inadequate for the peak outflow from one-half the PMF. For such large storm events, a high tailwater condition would most likely occur resulting in the flooding of the downstream hazard areas. Hence, the spillway capacity is not considered to be seriously inadequate since dam failure from overtopping (at elevation of the top of earth dike) would not significantly increase the hazard to loss of life downstream from that which would exist just before overtopping failure.

SECTION 6: STRUCTURAL STABILITY

6.1 EVALUATION OF STRUCTURAL STABILITY

a. Visual Observations

No direct visual observation of the spillway crest was possible because of submergence. However, no significant irregularities in the water surface profile along the axis of the crest were observed. There were no indications of instability at the navigation Lock, along the earth dike, or along the riverside forebay wall.

b. Design and Construction Data

Design computations or other data for the structural stability of the original dam or as it existed in 1912 were not available.

c. Data Review and Stability Evaluation

The NYS-DOT plans (included in Appendix H) show a cross-section of the spillway portion of the dam, both prior to 1965 and the reconstructed portion as it presently exists. A stability analysis was performed using the cross-section information shown plus certain simplifying assumptions made in the analysis. The resistances offered by the toe and the apron sections were calculated and converted to an equivalent passive resistance acting on the downstream face of the spillway masonry section.

Conditions analyzed were:

1) Normal conditions with the water level at the spillway crest elevation.

2) Conditions as in 1) plus a 5000 lb/ft ice load.

3) Water level at the elevation of the maximum known flood (82.8 - BCD); a flow depth of 15.3 feet.

4) Water level at the elevation of one-half PMF (88.7 - BCD); a flow depth of 21.2 feet.

The safety factors for overturning and sliding obtained from the analyses are:

	FACTORS OF SAFETY	
CONDITION	OVERTURNING	SLIDING
1) Normal water level	2.11	2.19
2) Ice load plus 1)	1.45	1.57
3) Maximum known flood	1.01	1.03
4) One-half PMF	0.92	0.88

The analyses indicate less than desireable factors of safety for normal water level conditions and a critical deficiency for the occurrence of a large storm event. The PMF storm event (flow depth of 29.4 feet) was not analyzed because of the results obtained during the one-half PMF analysis. The dam did withstand the 1913 storm event flows although the safety factors from the analysis are approximately equal to 1.00. Hence, the analysis is suspect due to the lack of more detailed information necessary to perform a complete in-depth study.

The condition of and the manner in which the wrought-iron framework is connected to the foundation could not be determined. Therefore, a structural analysis of the framework was not done.

d. Post-Construction Changes

The spillway apron portion of the dam was reconstructed in 1965 from a timber plank surface to one of precast reinforced concrete slabs. In addition, the spillway crest was replaced with a broader and thicker reinforced concrete cap. Detailed plans for this work are included in Appendix H.

e. Seismic Stabiltiy

This dam is located in seismic Zone 1. A seismic stability analysis is not warranted.

SECTION 7: ASSESSMENT/RECOMMENDATIONS

7.1 ASSESSMENT

a. Safety

The Phase 1 inspection of the Lock 3 Dam did not reveal conditions which constitute an immediate hazard to human life or property. The spillway, navigation Lock 3, earth dike, and gatehouse-forebay are not considered to be unstable.

The spillway, not having sufficient discharge capacity for passing one-half the PMF, is considered to be inadequate. During periods of unusually heavy precipitation, continuous surveillance should be provided both at the dam and in the downstream areas to warn of hazardous flooding conditions. Such surveillance procedures and other measures should be documented in a detailed emergency-operation plan for the dam. Also, a warning system should be developed and placed in readiness for future use.

b. Adequacy of Information

The information available appears to be adequate for the Phase 1 inspection purposes except for the following:

1) The physical condition of the wrought-iron framework supporting the apron slabs and the concrete surfaces of both the crest and apron.

2) The physical condition and operational status of the forebay gates.

c. Urgency

Those deficiencies within the zone of water level fluctuations (below elevation 77.0 - BCD) should be corrected prior to the next period of anticipated high flows (Spring 1980). All other deficiencies observed during the visual inspection can be corrected during normal maintenance operations.

d. Necessity for Additional Investigations

Further structural analysis of the wrought-iron framework and a more detailed structural stability analysis of the dam to include the foundation materials' characteristics is recommended.

Additional investigations are also warranted to determine the following:

1) The physical condition of the wrought-iron framework supporting the apron slabs and the concrete surfaces of both the crest and apron.

2) The actual method of anchorage between the wrought-iron framework and the underlying foundation.

3) The physical condition and operational status of the forebay gates.

7.2 RECOMMENDED MEASURES

The following actions should be undertaken:

a) Rehabilitate all deteriorated and cracked concrete surfaces.

b) Remove the trees and brush from the earth dike embankment.

c) Regrade the top of the earth dike to provide a level embankment crest.

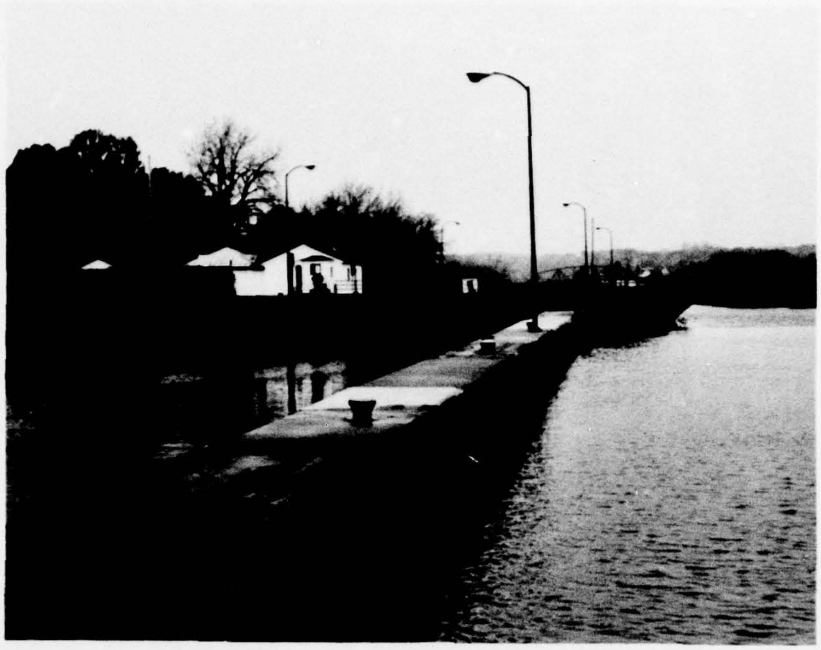
d) Perform periodic maintenance of the dam and all appurtenant structures.

e) Develop and implement a detailed emergency-operation plan and warning system.

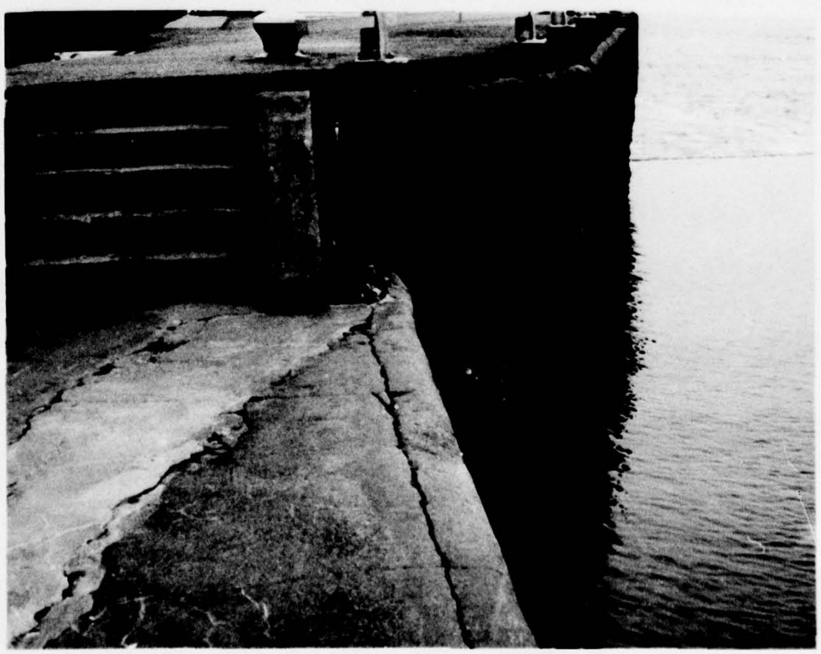
f) As a result of the completed additional investigations, remedial measures deemed necessary should be completed within two years of the date of this report.

APPENDIX A

PHOTOGRAPHS



UPSTREAM APPROACH TO LOCK 3

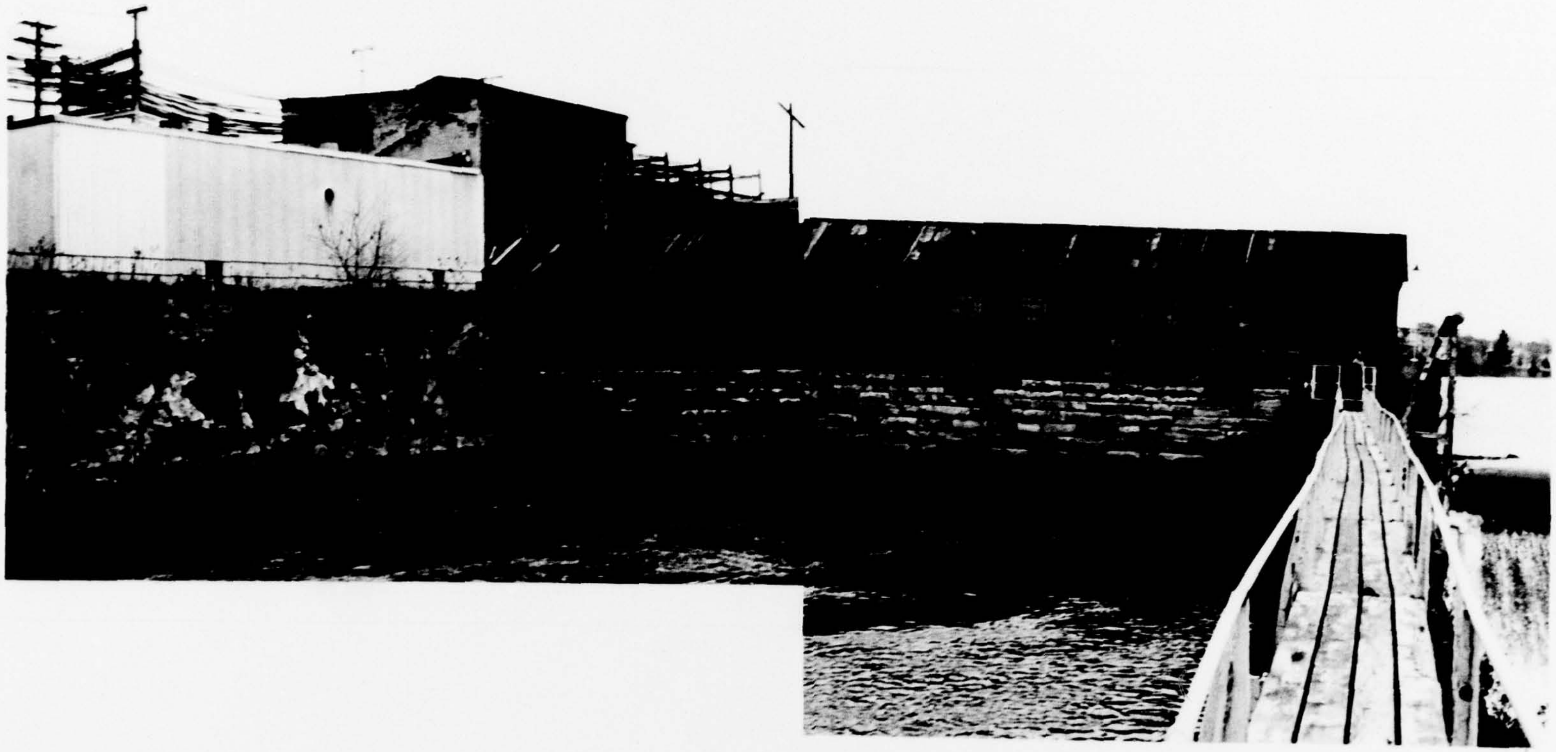


CONCRETE SURFACE DETERIORATION @
LOCK - SPILLWAY CONTACT

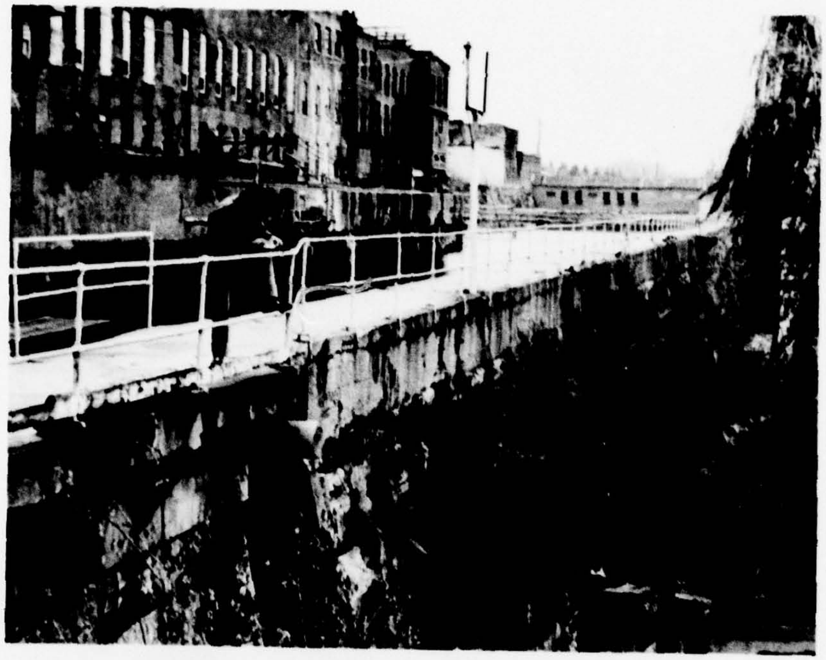
CONCRETE DETERIORATION @ ABUTMENT
OF EARTH DIKE - LOCK 3

(UPSTREAM) EARTH DIKE (DOWNSTREAM)

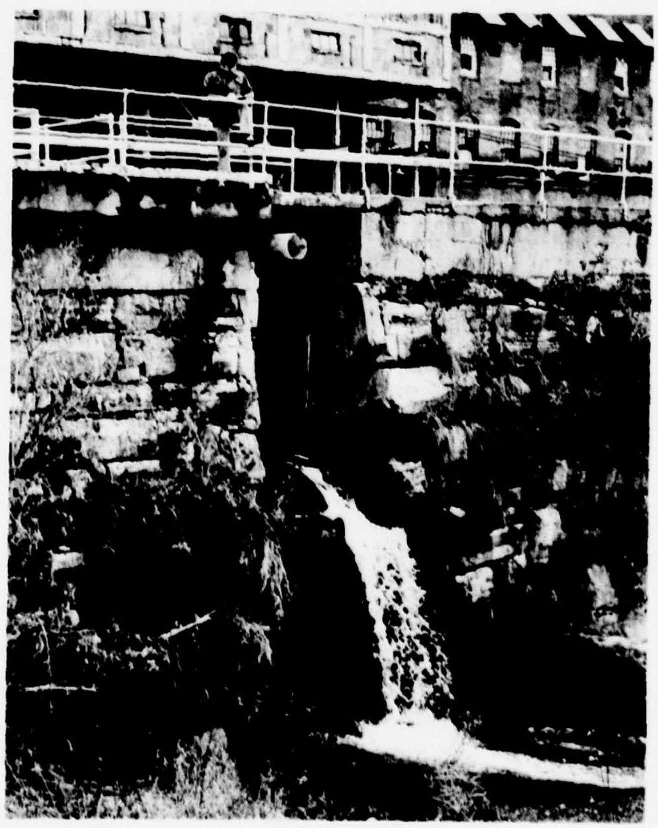
UPSTREAM SLOPE PROTECTION ON EARTH DIKE



GATEHOUSE - FOREBAY
(LOOKING UPSTREAM)



OUTER FOREBAY WALL
(GATEHOUSE - BACKGROUND CENTER)



SLUICE GATE - FOREBAY WALL

APPENDIX B

ENGINEERING DATA CHECKLIST

Check List
Engineering Data
Design Construction Operation

Name of Dam LOCK 3 @ MECHANICVILLE
I.D. # NY-215
(#119-UH)

Item	Remarks		
	Plans	Details	Typical Sections
Dam	YES		YES
Spillway(s)	YES		YES
Outlet(s)			
Design Reports		N/A	
Design Computations		N/A	
Discharge Rating Curves		N/A	
Dam Stability		N/A	
Seepage Studies		N/A	
Subsurface and Materials Investigations		N/A	

LOCK 3 DAM @ MECHANICVILLE

Item	Remarks
Construction History	LIMITED (1913 & 1916 DAM REPORTS)
Surveys, Modifications, Post-Construction Engineering Studies and Reports	NA
Accidents or Failure of Dam Description, Reports	NONE
Operation and Maintenance Records Operation Manual	MEAN [redacted] DAILY WATER LEVEL RECORDS @ LOCK 3 (NYS-DOT GAGE) USGS GAGE (RECORDS 10/1887 TO 9/1956)

APPENDIX C

VISUAL INSPECTION CHECKLIST

VISUAL INSPECTION CHECKLIST

1) Basic Data

a. General

Name of Dam LOCK 3 @ MECHANICVILLE

I.D. # NY-215 (#119-UPPER HUDSON)

Location: Town ~~SCHAGHTICOKE~~ STILLWATER County ~~RENSSELAER~~ SARATOGA

Stream Name HUDSON RIVER

Tributary of N/A

Longitude (W), Latitude (N) 73°-40'-42" W 42°-54'-42" N

Hazard Category C

Date(s) of Inspection 10/26/78

Weather Conditions ±50°F OVERCAST ; RAIN

b. Inspection Personnel J. HUNTINGTON (NYSDOT REGION ONE - WATERWAYS)

R. WARRENDER W. LYNICK

c. Persons Contacted NYSDOT [WATERWAYS SUBDIV. - MAIN OFFICE & REGION]

MR. A. B. CARLSON (SARATOGA BOARD MILLS CORP.)

d. History:

Date Constructed [ORIGINALLY - 1882±] 1965

Owner NYS-DOT WATERWAYS MAINT. SUBDIVISION (ONLY TO WEST END OF THE SPILLWAY CREST) (OWNERSHIP ON 1/18/1922)

Designer NA

Constructed by [ORIGINALLY - HUDSON RIVER WATER-POWER & PAPER COMPANY] ← WEST VIRGINIA PULP & PAPER CO. (SUCCESSOR IN TITLE TO)

2) Technical Data

Type of Dam MASONRY GRAVITY DAM W/ CONC. APRON (IN 1912 - MASONRY W/ TIMBER APRON)

Drainage Area 4500 SQ MILES

Height 37' Length 700' (+)

Upstream Slope 1V : 2H Downstream Slope NA

EARTH BACKFILL

PRECAST CONCRETE SLABS SUPPORTED BY A WROUGHT IRON FRAMEWORK

2) Technical Data (Cont'd.)

External Drains: on Downstream Face N/A @ Downstream Toe N/A

Internal Components:

Impervious Core NONE

Drains NONE

Cutoff Type N/A

Grout Curtain NONE

3) Embankment

EARTH DIKE - EAST OF LOCK 3

a. Crest

(1) Vertical Alignment SATISFACTORY ; SLIGHT DOWNSLOPE FROM THE CENTER OF DIKE TOWARD THE UPSTREAM FACE

(2) Horizontal Alignment SATISFACTORY

(3) Surface Cracks NONE

(4) Miscellaneous

b. Slopes

(1) Undesirable Growth or Debris, Animal Burrows BRUSH & SEVERAL LARGE TREES (>6" DIAM) ON DOWNSTREAM SLOPE

(2) Sloughing, Subsidence or Depressions NONE

(3) Slope Protection CONCRETE FACED - UPSTREAM SLOPE

(4) Surface Cracks or Movement at Toe NONE

(5) Seepage NONE

(6) Condition Around Outlet Structure N/A

EARTH DIKE - EAST OF LOCK 3

c. Abutments

(1) Erosion at Embankment and Abutment Contact NONE

(2) Seepage along Contact of Embankment and Abutment NONE

(3) Seepage at toe or along downstream face NONE

d. Downstream Area - below embankment

(1) Subsidence, Depressions, etc. NONE

(2) ~~Seepage~~, unusual growth SOME BRUSH & WEEDS

(3) Evidence of surface movement beyond embankment toe NONE

(4) Miscellaneous

e. Drainage System

N/A

4) Instrumentation

(1) Monumentation/Surveys WATER SURFACE STAFF GAGES BOTH ABOVE AND BELOW THE NAVIGATION LOCK

(2) Observation Wells NONE

(3) Weirs NONE

(4) Piezometers NONE

(5) Other

5) Reservoir

a. Slopes N/A HUDSON RIVER SHORELINE

b. Sedimentation N/A

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

SUBMERGED @ TIME OF INSPECTION

a. General

b. Principle Spillway A SMOOTH WATER SURFACE PROFILE ALONG THE AXIS OF THE SPILLWAY CREST INDICATES HORIZ & VERT. ALIGNMENT IS SATISFACTORY

c. Emergency or Auxiliary Spillway NONE

d. Condition of ~~Tail race~~ DOWNSTREAM channel – HUDSON RIVER SATISFACTORY

e. Stability of Channel side/slopes N/A

7) Downstream Channel

HUDSON RIVER

a. Condition (debris, etc.) SATISFACTORY

b. Slopes N/A

c. Approximate number of homes 100(+) HEMSTREET PARK IN TOWN OF SCHAGHTICOKE & CITY OF MECHANICVILLE

8) Miscellaneous

9) Structural

a. Concrete Surfaces SPALLED AREAS @ RIVER-SIDE OF LOCK UPSTREAM PROTECTION PIER, ON OUTER SURFACES OF LOCK WALLS IN THE ZONE OF AERATION (DEPTH < 3") EARTH DIKE - LOCK ABUTMENT SIGNIFICANT CONCRETE DETERIORATION

b. Structural Cracking NONE APPARENT

c. Movement - Horizontal & Vertical Alignment (Settlement) NONE

d. SPILLWAY Junctions with Abutments or Embankments

EAST - SATISFACTORY

WEST - SATISFACTORY

e. Drains - Foundation, Joint, Face N/A

f. Water passages, conduits, sluices

PAPER MILL - FOREBAY: MASONRY; JOINTS (CRACKING & SOME SEPARATION)

g. Seepage or Leakage EAST - NONE

WEST - SEEPAGE @ ONE HORIZ. JOINT IN STONE BLOCK @ GATEHOUSE-CREST CONTACT

LEAKAGE THROUGH OUTER FOREBAY WALL IN 3 AREAS @ PLACES 200'(+) BELOW CREST OF DAM

h. Joints - Construction, etc. N/A

i. Foundation APPEARS TO BE ON ROCK (EVIDENT WHEN LOCK WAS DEWATERED; ROCK VISIBLE BENEATH UPSTREAM DOORS WHICH ARE IN SAME LOCATION AS AXIS OF CREST)

j. Abutments N/A

k. Control Gates NONE

l. Approach & Outlet Channels N/A

m. Energy Dissipators (plunge pool, etc.) NONE

n. Intake Structures NONE

o. Stability N/A

p. Miscellaneous

APPENDIX D

HYDROLOGIC/HYDRAULIC

ENGINEERING DATA AND COMPUTATIONS

LOCK 3 DAM @
MECHANICVILLE

CHECK LIST FOR DAMS
HYDROLOGIC AND HYDRAULIC
ENGINEERING DATA

AREA-CAPACITY DATA:

BARGE CANAL DATUM - BCD

	Elevation (ft.)	Surface Area (acres)	Storage Capacity (acre-ft.)
1) Top of Dam	83.0	350	8785
2) Design High Water (Max. Design Pool)	NA		
3) ~~Auxiliary Spillway Crest~~ TOP OF LOCK	77.0	350	6685
4) Pool Level with Flashboards	NA		
5) Service Spillway Crest	67.5	260	3420

DISCHARGES

	Volume (cfs)
1) Average Daily	NA
2) Spillway @ ~~Maximum High Water~~ TOP OF LOCK	77,600
3) Spillway @ Design High Water	NA
4) Spillway @ Auxiliary Spillway Crest Elevation	NA
5) Low Level Outlet	NA
6) Total (of all facilities) @ Maximum High Water	122,100
7) Maximum Known Flood	120,000
8) HYDROELECTRIC STATION EXISTING MACHINERY (12 UNITS)	~~5100 (MAX)~~
DESIGN -	6043 (MAX)
OPERATING -	5893

CREST: ELEVATION: 67.5 (BCD)

Type: REINFORCED CONCRETE CAP OVER STONE BLOCK; PRECAST CONC. APRON SLAB ON DOWNSTREAM FACE

Width: 36' @ BASE (INCL. APRON) Length: 700' (+)

Spillover ENTIRE LENGTH OF CREST

Location

SPILLWAY:

PRINCIPAL		EMERGENCY
67.5 BCD	Elevation	
REINFORCED CONC. CAP OVER STONE BLOCK	Type	NONE
36' @ BASE (INCL. APRON)	Width	
	Type of Control	
✓	Uncontrolled	
	Controlled:	
	Type (Flashboards; gate)	
	Number	
700'	~~Size~~/Length	
	Invert Material	
	Anticipated Length of operating service	
N/A	Chute Length	
4'	Height Between Spillway Crest & Approach Channel Invert (Weir Flow)	

OUTLET STRUCTURES/~~EMERGENCY DRAWDOWN FACILITIES~~:

Type: Gate ✓ Sluice ______ Conduit ______ Penstock ______

Shape : NAVIGATION LOCK 3

Size: 46' x 410'

Elevations: Entrance ~~Invert~~ NORMAL POOL @ 67.5+

Exit ~~Invert~~ NORMAL TAILWATER @ 48.0±

Tailrace Channel: Elevation N/A

HYDROMETEROLOGICAL GAGES:

Type : STAFF GAGE & WATER-STAGE RECORDER / STAFF GAGES @ LOCK 3

Location: ON RT. BANK JUST UPSTREAM OF DAM / UPSTREAM & DOWNSTREAM

Records:

Date - (USGS) 10/1887 TO 9/1956 / (NYS-DOT) 10/1916 TO PRESENT

Max. Reading - HEAD = 11.67' 3/28/1913 DISCHARGE = 120,000 cfs / UPSTR: 79.4 ON 1/1/1949 DOWNSTR: 60.7 ON 3/19/1936 Q = 118,000 cfs

FLOOD WATER CONTROL SYSTEM:

Warning System: NONE

Method of Controlled Releases (mechanisms):

NONE

DRAINAGE AREA: 4500 SQ MILES

DRAINAGE BASIN RUNOFF CHARACTERISTICS:

Land Use - Type: 1/2 - 2/3 OF AREA IN ADIRONDACK MOUNTAIN AREA

Terrain - Relief: ELEVATIONS (+5344 TO +70 @ DAM)

Surface - Soil: VARIES

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

N/A

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

N/A

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

N/A

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

Location: @ SHORELINE, DUE EAST OF END OF THE UPSTREAM PROTECTION PIER

Elevation: 72±

Reservoir:

Length @ Maximum Pool 2± (Miles)

Length of Shoreline (@ Spillway Crest) N/A (Miles)

GA-17 (4/70) NEW YORK STATE DEPARTMENT OF ENVIRONMENTAL CONSERVATION

PROJECT GRID

JOB	SHEET NO.	CHECKED BY	DATE
LOCK 3 DAM @ MECHANICVILLE	1/		
SUBJECT: STREAMFLOW GAGE DATA		COMPUTED BY: WCL	DATE: 2/79

USGS WSP 1302: (RECORDS TO SEPT. 1950)

GAGE #61: MAX. Q = 120000 cfs ON 3/28/1913 ←

MAX. Q = 118000 cfs ON 1/1/1949
[OVER SPILLWAY + THRU WHEELS + THRU LOCK]
SINCE 9/30/1915

USGS WSP 1722: (RECORDS OCT. 1950 TO SEPT. 1960)

GAGE #3355: (DISCONTINUED SEPT. 1956)

MAX. Q — SAME AS ABOVE

AFTER SEPT. 1956:

GAGE #3580 (−) GAGE #3575 = APPROX. LOCK 3 FLOW

#3580 — HUDSON RIVER @ GREEN ISLAND, NY
#3575 — MOHAWK RIVER @ COHOES, NY

APRIL 4 & 5, 1960:

#3580 134,900 cfs
#3575 83,300 cfs
} 51,600 cfs [NOT A MAX. Q] ←

MARCH 6, 1964: MEAN DAILY

#3580 141,000 cfs 96000 cfs [NOT A MAX. Q] ←

#3575 143,000 cfs { RATING CURVE LOGARITHMIC EXTRAPOLATION ABOVE 100,000 cfs } 77500 cfs

3 cfs DIVERSION THRU LOCK 6 @ BARGE CANAL (WINTER SHUTDOWN)

JUNE 24, 1972 (HURRICANE AGNES):

#3580 78,900 cfs [NOT A MAX. Q] ←

#3575 58,100 cfs 50,900 cfs

GA-17 (4/70) NEW YORK STATE DEPARTMENT OF ENVIRONMENTAL CONSERVATION

PROJECT GRID

JOB	SHEET NO.	CHECKED BY	DATE
LOCK 3 DAM @ MECHANICVILLE	21		
SUBJECT		**COMPUTED BY**	**DATE**
STREAMFLOW GAGE DATA & SIMULATION DATA		WCL	2/79

"UPPER HUDSON & MOHAWK RIVER BASINS HYDROLOGIC FLOOD ROUTING MODELS"

GREATEST ANNUAL PEAKS OF RECORD - TABLE 4.1

3/28/1913	119,000 cfs [APPROX. MAX Q] ←
12/31/1948	94,400 cfs
4/12/1922	72,900 cfs

TABLE 6.39: (RESULTS FROM SIMULATION)

SPF = 191,121 cfs

TRANSPOSED AGNES = 251,604 cfs

PMF ≈ 2 × SPF

SPS RAINFALL - 9.5"

PMP RAINFALL - 20" (TP-40)
(10 SQ MILES - 6 HR)

TRANSPOSED AGNES RAINFALL } - 11"

HRR #33 PMP RAINFALL 19.5"

78% (1000 SQ MI - 24 HR) - 15.2"

86% (-48 HR) - 16.8"

∴ PEAK DISCHARGE (PMF) = 382,242 cfs [USE 382,000 cfs ←

PROJECT GRID

JOB	SHEET NO.	CHECKED BY	DATE
LOCK 3 DAM @ MECHANICVILLE	3/		
SUBJECT: DISCHARGE COEFFICIENT		COMPUTED BY: WCL	DATE: 2/79

ROUNDED CREST: RADIUS = 2'-3 5/8" = 2.302'

LENGTH = 700' (MAIN SPILLWAY PORTION)

DRAWING # M3082 (APPENDIX F):

GAGE D: WS ELEV. = 82.58' ON 3/28/1913

TOP-OF-DAM = 70.91

HEAD = 11.67'

USGS GAGE # 61 MAX. Q = 120,000 cfs ON 3/28/1913

$Q = CLH^{3/2}$

$$C = \frac{Q}{LH^{3/2}} = \frac{120000}{(700)(11.67)^{1.5}}$$

[ASSUMED: ENTIRE 3/28/1913 Q FLOWED ONLY OVER THE SPILLWAY

$C = 4.30$

{INDICATES A SHARP-CRESTED WEIR CONDITION

$$C = \frac{120000}{(795.7)(11.67)^{1.5}}$$

{FOR H = 11.67', INUNDATION OF THE LOCK WOULD HAVE OCCURRED

$L = L' - 0.2H$ $L' = 798'$ $H = 11.67'$

$L = 795.7$

$C = 3.78$

USE $C = 3.8$ ←

GA-17 (4/70) NEW YORK STATE DEPARTMENT OF ENVIRONMENTAL CONSERVATION

PROJECT GRID

JOB	SHEET NO.	CHECKED BY	DATE
LOCK 3 DAM @ MECHANICVILLE	4/		
SUBJECT		**COMPUTED BY**	**DATE**
DEPTH - FLOWS (COMPUTED)		WCL	2/79

$Q = CLH^{3/2}$ $L = L' - 2(K_p)H$

SPILLWAY CREST ELEV:

$C = 3.8$ (FOR ANALYSIS - SHT 3) $K_p = 0.1$

USGS (DATUM) - 66.32
BARGE CANAL (DATUM) - 67.5

ELEVATIONS - BARGE CANAL DATUM

TOP - LOCK 3 (RIVER-SIDE ABUTMENT) ELEV. = 77.0 (@ PRESENT)

$L = 700 - 2(0.1)(9.5)$
$L = 698.1$

$Q = (3.8)(698.1)(9.5)^{3/2}$ $H = 9.5'$

$Q = 77676$ cfs

TOP - EARTHEN DIKE ELEV. = 83.0 (@ PRESENT)

$L = 798 - 2(0.1)(6)$
$H = 9.5 + 6$ $L = 796.8$

$Q = 77676 + (3.8)(796.8)(6)^{3/2}$

$= 77676 + 44500$

$Q = 122176$ cfs USE 122170 cfs ←

FOR THE GIVEN Q; COMPUTE H: TOP - EARTHEN DIKE = 83.0

$Q = 120000$ cfs $L = 798$

$H^{3/2} = \frac{Q}{CL} = \frac{(120000 - 77676)}{(3.8)(798)}$ $H = 5.8'$ ELEV = 82.8

(½ PMF) $Q = 191000$ cfs $L = 798 + 520 = 1318$

$H^{3/2} = \frac{(191000 - 122176)}{(3.8)(1318)}$ $H = 5.7'$ ELEV = 88.7

(PMF) $Q = 382000$ cfs

$H^{3/2} = \frac{(382000 - 122176)}{(3.8)(1318)}$ $H = 13.9'$ ELEV = 96.9

GA-17 (4/70) NEW YORK STATE DEPARTMENT OF ENVIRONMENTAL CONSERVATION

PROJECT GRID

JOB	SHEET NO.	CHECKED BY	DATE
LOCK 3 DAM @ MECHANICVILLE	5/		
SUBJECT		**COMPUTED BY**	**DATE**
FLOW AREAS - PLANIMETERED		WCL	3/79

MAP (SCALE 1:6000) 1 IN^2 = 5.739 ACRES

AREA (DAM TO ≈ 870) AREA (870 TO 850)

15.78 IN^2 10.26 IN^2

AREA (850 TO 820) AREA (820 TO LOCK 4; DAM @ 790)

18.53 IN^2 15.44 IN^2

4.41 IN^2 - HOOSIC RIVER

SUB-AREAS (820 - UPSTREAM) AREA

ALONG MAIN STEM OF RIVER - TO ELEV ±70 : 1.87 IN^2

ALONG LOCK 4 APPROACH - TO ELEV ±70 : 1.48 IN^2

ALONG HOOSIC RIVER - TO ELEV ±70 : 0.47 IN^2

STAGE -	AREA (IN^2)	ACRES
66	48.39	278
70	48.39	278
MAIN STEM - 70+	60.48	347 } 370
HOOSIC RIVER - (70-75)	3.94	23 }

USGS 7.5' QUAD - MECHANICVILLE (1954) SCALE: 1:24000 1 IN^2 = 91.827 ACRES

ELEV	AREA
66	3.11 IN^2
	(-) 0.29 IN^2 (GREEN ISLAND)
66-70 :	2.82 IN^2

ABOVE 70:
MAIN STEM - 0.78 IN^2
HOOSIC RIVER - 0.22 IN^2

STAGE -	AREA (IN^2)	ACRES
66	2.82	259
70	2.82	259
MAIN STEM - 70+	3.60	331 } 351
HOOSIC RIVER - (70-75)	0.22	20 }

GA-17 (4/70) NEW YORK STATE DEPARTMENT OF ENVIRONMENTAL CONSERVATION

PROJECT GRID

JOB	SHEET NO.	CHECKED BY	DATE
LOCK 3 DAM @ MECHANICVILLE	6		
SUBJECT		**COMPUTED BY**	**DATE**
STAGE - STORAGE DATA		WCL	2/79

SURFACE AREAS (SHT 5):

STAGE	AREA (ACRES)	Δh (FT)	ΔVOL. (AC-FT)	STORAGE (AC-FT)	
					HT OF DAM ≈ 19'
66(-)	100				DECREASE AREA @ 5% PER FT ELEV.
		19	3420		
66	260			3420	— (SPILLWAY CREST) ←
		4	1040		
70	260			4460	
		5	1525		
70(+) 75 (TO LIMITS)	350			5985	
		2	700		
77	350			6685	
		6	2100		
83	350			8785	

GA-17 (4/70) NEW YORK STATE DEPARTMENT OF ENVIRONMENTAL CONSERVATION

PROJECT GRID

JOB	SHEET NO.	CHECKED BY	DATE
LOCK 3 DAM @ MECHANICVILLE	7/		
SUBJECT		**COMPUTED BY**	**DATE**
GATEHOUSE – FOREBAY – EXISTING HYDRO : DISCHARGES		WCL	2/79

[SHT # M-5596]

11 GATES 8' WIDE x 10' HIGH
1 GATE 4' WIDE x 10' HIGH

[SHT # M 3265]

SPILLWAY CREST – 70.91

BASE OF DAM – 51.5±
@ RIVER BOTTOM

19.4'

(ASSUMPTION):
BOTTOM OF GATES ARE @ ELEV. 53.0

∴ CENTER OF GATES @ ELEV. 58.0

UPSTREAM HEAD = 12.9'

[SHT # M 3280]

BOTTOM OF CANAL @ ELEV. 53.0

[SHT # M 2990]

HYDRO-ELECTRIC PLANT HEAD GATES

6 GATES
CLEAR OPENINGS 12' x 12'
OPENING INVERT – ELEV. 57.50

FERC PERMIT APPLICATION #2793:

MAXIMUM DISCHARGE (12 UNITS) = 5893 cfs ←

I.D. No. NY-215

EXHIBIT M

GENERAL SPECIFICATIONS - EXISTING HYDROELECTRIC POWER STATION

Hydraulic Turbine Information

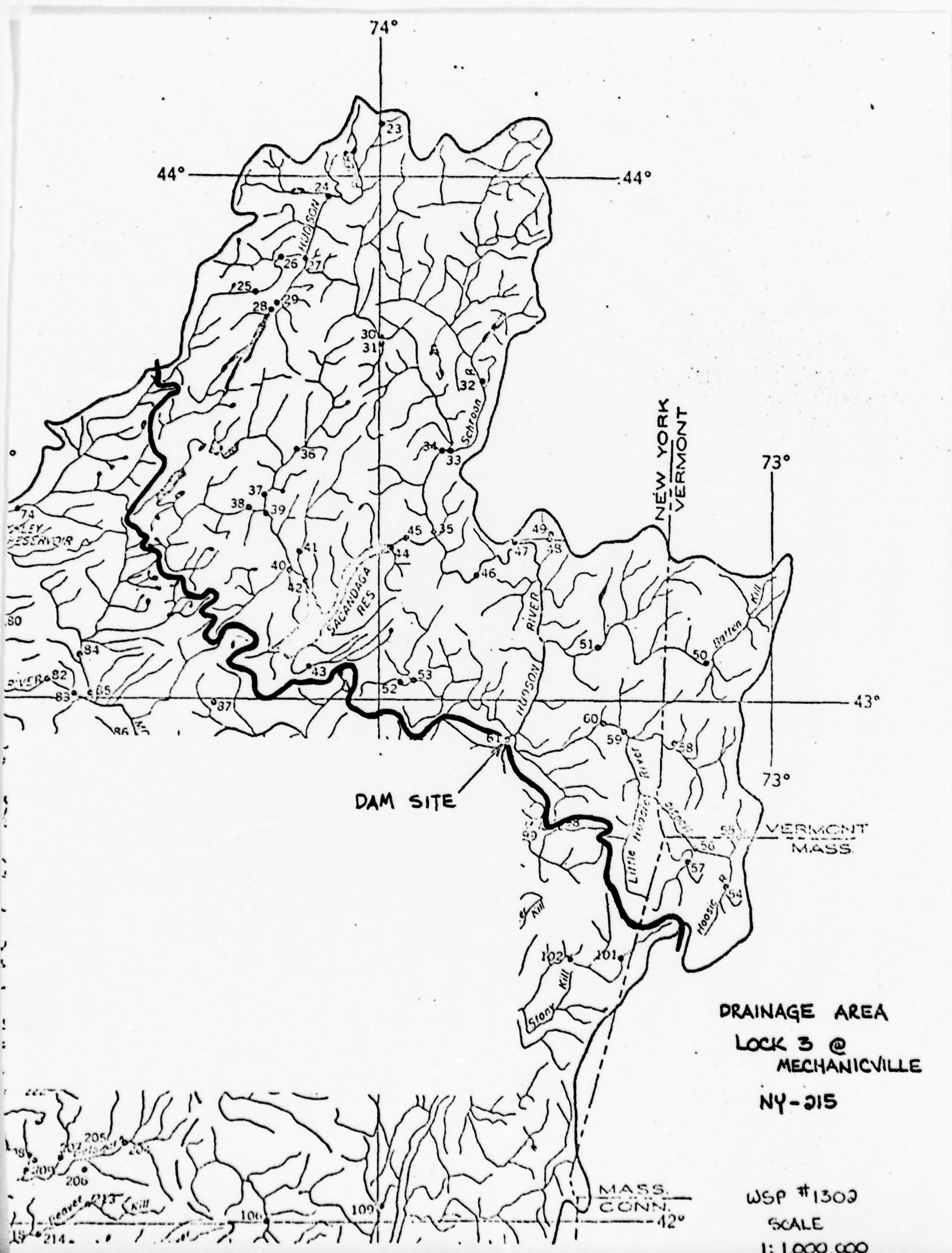
	#3	#4	#7	#8	#9	#10	#11	#12	#13	#14	#15	#16
Make	S.M.S.	S.M.S.	A.C.	S.M.S.	S.M.S.	S.M.S.	Leffel	S.M.S.	S.M.S.	S.M.S.	Leffel	Leffel
Type	Type U1 #8632	8633	AV	-	-	-	-	-	-	-	-	-
Size	1445 HP	-	76"	65"	65"	50"	2-47-1/2	2-45"	2-45"	2-42"	2-47-1/2"	2-47-1/2"
Position	Vert.	Vert.	Vert.	Vert.	Vert.	Vert.	Hor.	Hor.	Hor.	Hor.	Hor.	Hor.
R.P.M.	214	214	187-1/2	250	250	125	150	150	150	150	150	150
Head, feet	19/23	19/23	19/23	19/23	19/23	19/23	19/21	19/21	19/21	19/21	19/21	19/21
C.F.S.	775/780	775/760	636/646	440/475	440/475	400/425	393/413	443/466	433/466	372/291	393/413	393/413

Governor Information

	#3	#4	#7	#8	#9	#10	#11	#12	#13	#14	#15	#16
Make	S.M.S.	S.M.S.	W	W	W	W	W	W	W	W	W	W
Type	TS	TS	HR	HR	HR	HR	LR	VR10	VR10	Mech	LR	LR
Serial	256	257	5363	7364	160466	7381	202529	3123	3122	x	202530	27346
Cap.ft.lbs.	16,500	16,500	16,300	10,500	10,500	8400	10,500	10,000	10,000	x	10500	10,500

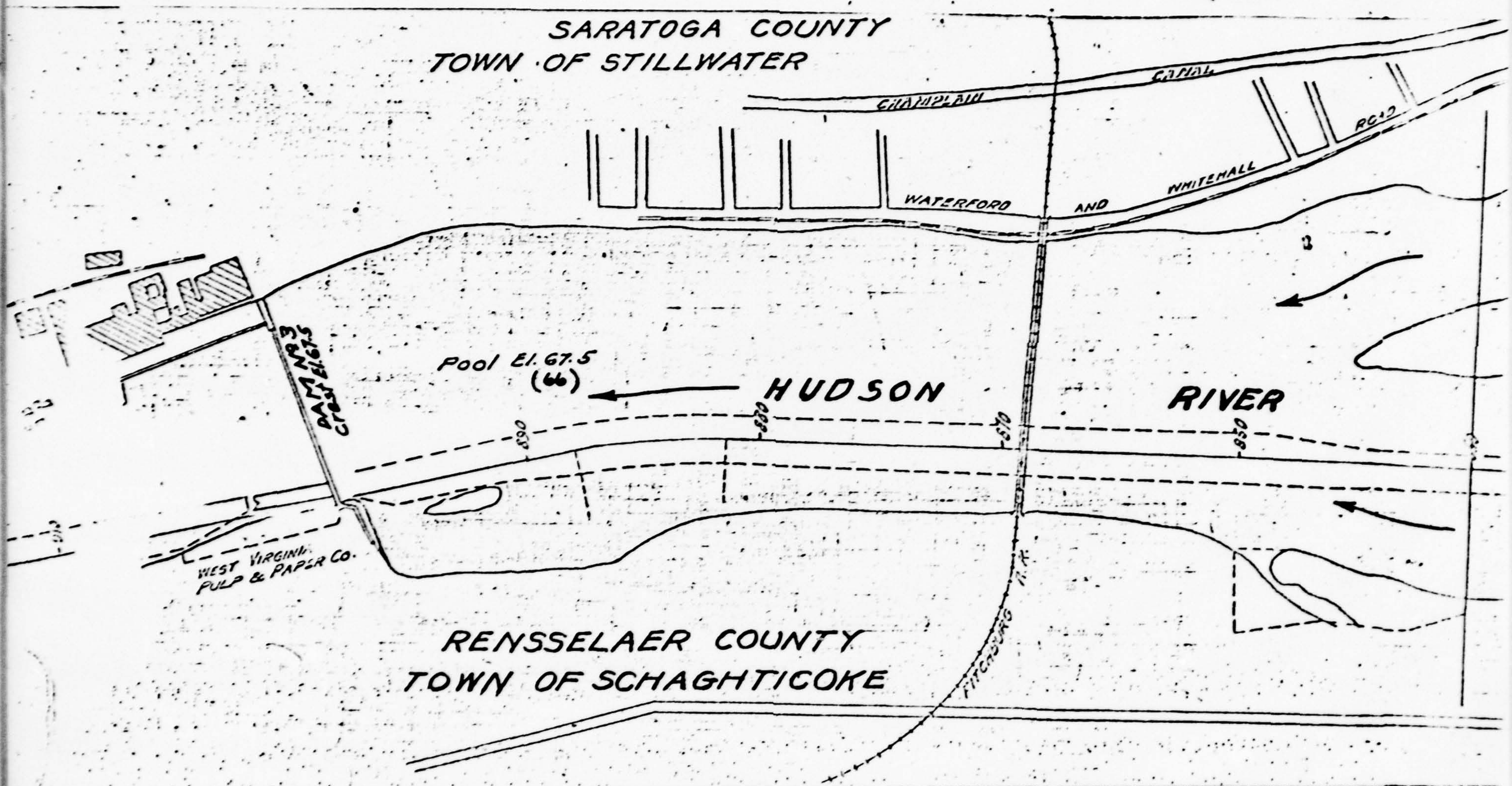
SMS - S. Morgan Smith
A.C. - Allis-Chalmers
W - Woodward

fraction values = Operating / Design


74°
44°
44°
73°
43°
73°
42°
HUDSON
Schroon R.
SACANDAGA RES
RIVER
HUDSON
NEW YORK
VERMONT
Batten Kill
Little Hoosic River
Hoosic R.
VERMONT
MASS.
Stony Kill
MASS.
CONN.
DAM SITE
DRAINAGE AREA
LOCK 3 @
MECHANICVILLE
NY-215
WSP #1302
SCALE
1: 1,000,000

RESERVOIR AREA

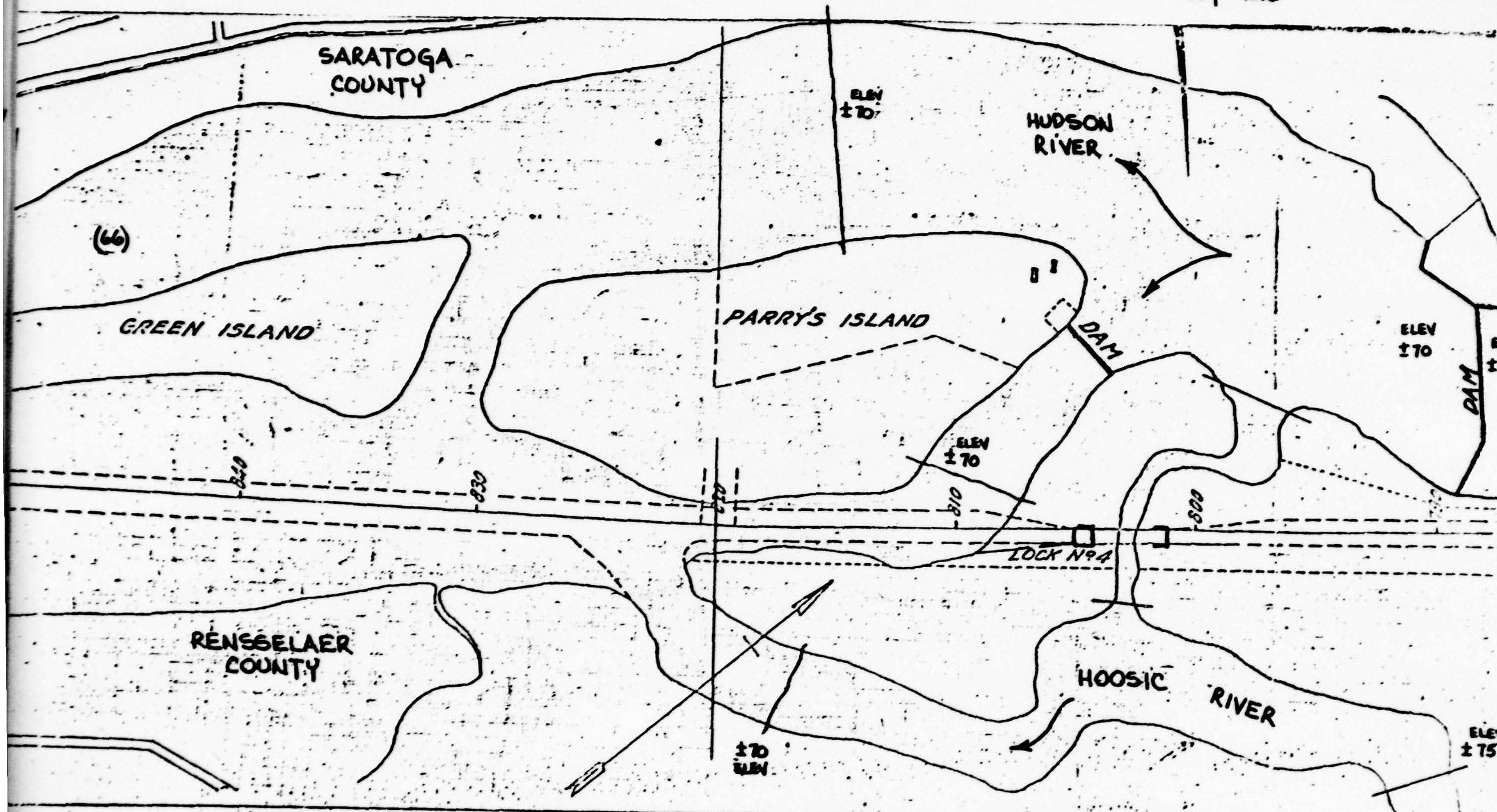
LOCK 3 DAM @ MECHANICVILLE
NY-215



(APPROXIMATE) SCALE: 1:6000

RESERVOIR AREA

LOCK 3 DAM @ MECHANICVILLE
NY-215



(APPROXIMATE) SCALE: 1:6000

61. Hudson River at Mechanicville, N. Y.

Location.--Lat 42°54'45", long 73°40'45", on right bank at dam of West Virginia Pulp & Paper Co., at Mechanicville, Saratoga County, three quarters of a mile upstream from Anthony Kill, and 1½ miles downstream from Hoosic River.

Drainage area.--4,500 sq mi.

Gage.--Water-stage recorder. Datum of gage is 66.63 ft above mean sea level, datum of 1929. Prior to 1911, staff gage at same site and datum.

Average discharge.--63 years (1887-1950), 7,370 cfs, revised (unadjusted).

Extremes.--1887-1950: Maximum discharge, 120,000 cfs Mar. 28, 1913; practically no flow for short periods when plant was shut down.
Maximum known discharge prior to 1913, 70,000 cfs April, 1869 (Report of U. S. Board of Engineers on Deep Waterways).
Since 1930, maximum discharge, 118,000 cfs Jan. 1, 1949.

Remarks.--Discharge computed from flow over spillway, through wheels, and through lock of Champlain Canal since Sept. 30, 1915. Flow appreciably regulated by Indian Lake since 1898 (see p. 45), and Sacandaga Reservoir since Mar. 27, 1930 (see p. 62). During canal navigation season, water is diverted through Glens Falls feeder, Bond Creek (see pp. 66, 67), and Champlain Canal into Lake Champlain basin and occasionally may receive water from that basin through summit level of Champlain Canal at Dunham basin. No adjustment made for these diversions.

Cooperation.--Records of discharge over spillway and through wheels furnished by West Virginia Pulp & Paper Co.

Monthly and yearly mean discharge, in cubic feet per second

Water year	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	The year
1888	2,365	4,190	8,018	6,367	3,714	6,845	21,200	21,420	4,917	1,537	1,725	2,851	7,096
1889	4,608	10,642	10,014	10,983	3,790	8,280	13,690	8,871	6,869	5,727	4,272	1,963	7,476
1890	3,740	7,888	13,226	11,272	7,913	11,129	15,053	17,931	7,392	1,950	2,019	8,844	9,030
1891	9,215	9,121	3,244	8,284	11,664	17,736	20,021	5,533	3,200	2,337	2,666	2,040	7,922
1892	1,473	4,098	8,577	18,857	9,263	10,929	21,554	19,622	12,395	9,287	5,485	4,448	10,498
1893	2,819	7,604	4,031	3,192	4,805	8,250	17,889	22,285	4,801	2,521	5,005	6,870	7,506
1894	3,865	3,639	7,217	6,757	4,836	14,738	11,135	7,566	7,097	3,168	2,456	1,889	6,197
1895	3,649	6,379	4,367	3,876	3,543	4,204	23,822	6,850	2,816	2,559	3,901	2,629	5,716
1896	2,631	8,421	10,899	6,791	4,668	13,600	24,972	4,610	4,738	2,772	2,442	2,879	7,452
1897	4,106	11,352	6,913	4,007	3,895	12,214	19,074	12,167	11,855	11,109	8,241	2,756	8,974
1898	2,574	9,995	14,392	8,173	6,038	19,617	13,047	10,525	5,069	2,751	5,029	3,810	8,413
1899	7,516	8,978	5,291	6,437	5,147	9,316	24,607	9,521	2,539	2,402	1,417	2,054	7,108
1900	2,616	6,066	7,303	5,841	12,484	7,740	22,614	8,992	4,093	2,352	2,703	1,886	7,058
1901	2,128	5,077	5,331	3,087	2,688	8,095	28,268	11,658	7,806	3,531	4,661	4,024	7,214
1902	4,264	3,732	8,491	‡5,255	4,741	25,130	15,222	†9,749	6,315	8,934	6,286	3,643	‡8,480
1903	6,894	7,076	9,162	7,034	9,835	30,938	14,013	3,509	8,472	4,863	5,905	4,118	9,322
1904	10,117	5,769	5,297	5,805	6,858	11,055	20,737	13,373	6,765	‡3,953	‡5,483	6,660	‡8,485
1905	‡11,029	4,644	3,922	6,097	3,544	9,388	22,748	8,171	9,552	6,935	5,631	12,032	‡8,641
1906	6,067	‡5,853	‡8,510	9,310	7,650	9,000	20,600	13,400	9,290	5,780	3,490	3,070	‡8,500
1907	3,370	5,130	4,870	10,300	4,140	9,570	16,600	13,900	5,560	4,120	‡2,370	6,150	‡7,170
1908	9,560	14,000	12,000	8,410	9,690	14,000	22,300	17,900	3,760	2,140	1,770	1,020	9,710
1909	1,380	1,850	2,010	5,210	11,600	9,220	25,800	17,000	6,020	1,980	1,520	1,460	7,090
1910	1,560	1,680	1,570	5,190	5,480	21,400	17,600	9,230	10,200	1,600	1,990	2,270	6,650
1911	2,540	3,280	2,010	5,270	3,370	5,780	16,800	9,260	4,430	1,360	1,180	2,000	4,760
1912	7,240	7,560	10,000	4,760	3,400	10,000	27,600	12,900	5,540	1,310	1,270	†2,490	7,830
1913	6,930	10,800	8,640	12,700	5,220	25,300	16,300	†6,920	†3,930	1,780	1,070	†1,170	‡8,420
1914	2,720	6,780	4,840	2,310	3,070	7,500	33,700	12,900	2,330	2,140	1,450	1,930	6,750
1915	1,240	2,130	2,590	6,480	10,500	9,110	12,900	5,340	1,940	8,500	7,880	3,850	6,010
1916	4,300	4,200	6,630	10,000	10,100	8,910	22,900	13,000	6,900	3,930	1,710	1,920	7,850
1917	2,240	4,020	6,740	4,500	2,750	10,900	21,300	10,100	14,200	4,360	1,870	1,770	7,060
1918	3,660	6,130	2,600	1,440	4,720	12,000	21,800	10,500	4,870	2,310	1,470	3,130	6,210
1919	4,940	8,800	8,160	6,510	3,430	11,000	15,400	13,200	4,080	2,680	1,970	3,710	6,990
1920	5,020	9,950	6,480	2,210	1,970	13,600	26,400	9,530	3,350	2,620	2,660	1,970	7,140
1921	3,980	6,070	13,000	5,110	3,780	21,500	12,700	5,750	1,780	4,390	2,010	1,390	6,830
1922	2,010	5,830	7,070	3,100	4,010	14,800	30,300	11,300	11,800	5,770	3,120	2,450	8,450
1923	2,320	2,420	1,830	4,930	2,980	7,080	25,200	13,100	3,970	1,870	1,300	1,600	5,710
1924	2,400	4,370	9,110	10,200	3,790	5,530	25,400	20,300	3,630	2,040	1,870	3,000	7,640
1925	5,560	3,620	4,320	1,600	9,470	18,200	18,100	8,600	4,810	6,470	4,800	4,730	7,510
1926	6,860	12,800	8,240	4,890	4,070	6,160	27,900	13,300	5,660	3,170	2,510	2,670	7,760
1927	5,340	10,700	3,830	3,570	4,330	15,800	10,600	9,820	4,130	2,070	2,300	2,930	6,310
1928	4,530	17,400	17,600	5,630	7,140	8,230	23,800	12,300	8,850	6,410	4,510	3,220	9,950
1929	2,690	3,270	3,670	5,080	3,820	19,800	24,300	16,000	5,310	3,960	2,040	2,480	7,720
1930	2,960	3,910	4,880	8,350	7,160	12,800	10,700	6,680	6,410	3,110	2,430	2,570	5,990
1931	2,480	2,540	2,700	2,060	1,610	3,440	10,400	7,170	5,150	6,130	3,390	3,690	4,240
1932	4,170	5,090	7,550	10,500	7,580	5,950	14,900	9,360	3,830	4,280	3,790	3,750	6,720
1933	7,050	13,200	7,830	7,810	7,100	7,460	22,300	9,020	3,640	2,970	3,830	4,070	8,000
1934	3,895	4,212	5,315	6,207	3,507	9,315	18,560	5,807	4,687	3,127	2,963	2,867	5,873
1935	3,337	4,423	5,064	9,037	6,283	9,703	9,796	8,669	5,973	12,740	5,579	4,339	7,095
1936	4,337	8,372	8,325	4,999	2,859	25,600	18,240	8,195	3,566	2,736	2,561	3,068	7,759
1937	4,955	8,117	7,386	12,830	7,023	6,309	13,030	13,750	7,264	5,148	4,597	4,227	7,889
1938	4,673	6,958	6,778	7,746	10,430	10,510	8,658	5,317	3,394	4,248	4,408	11,320	7,003
1939	5,795	5,700	17,410	5,965	5,925	10,020	20,310	9,971	4,732	3,352	3,035	2,977	7,519
1940	2,698	3,828	7,781	2,439	2,240	4,483	18,840	15,490	8,404	5,861	3,663	4,736	6,282

† Corrected.
‡ Not previously published; partly estimated on basis of records in reports of State engineer and surveyor of New York.

Monthly and yearly mean discharge, in c

Water year	Oct.	Nov.	Dec.	Jan.	Feb.
1941	3,762	6,846	8,013	8,006	6,167
1942	2,378	4,066	4,733	5,174	3,287
1943	6,590	8,976	8,592	6,780	7,005
1944	3,533	6,884	5,213	3,006	3,832
1945	4,203	4,283	3,915	6,116	5,230
1946	13,840	12,850	8,678	8,142	5,869
1947	4,813	4,497	5,047	7,595	[illegible]
1948	3,024	4,156	3,412	2,603	4,223
1949	2,669	4,676	8,475	18,480	10,900
1950	3,532	4,667	7,342	10,890	7,328

Monthly a

Water year	Oct.	Nov.	Dec.	Jan.	Feb.
1888	0.61	1.04	2.05	1.62	0.89
1889	1.18	2.64	2.57	2.81	.88
1890	.96	1.96	3.39	2.89	1.83
1891	2.36	2.56	.83	[illegible]	[illegible]
1892	.38	1.01	2.27	4.[illegible]	[illegible]
1893	.72	1.89	1.03	.82	[illegible]
1894	.99	.90	1.85	1.73	1.12
1895	.94	1.58	1.12	.99	.82
1896	.69	2.08	2.79	1.74	1.12
1897	1.05	2.87	1.78	1.03	.90

Yearly disc

Year	W.S.P. no.	Water y: Maximum day: Discharge	Date
1888	24	-	-
1889	24	-	-
1890	24	-	-
1891	24	-	-
1892	24	-	-
1893	24	-	-
1894	24	-	-
1895	24	-	-
1896	35	-	-
1897	35	31,060	July 15, 1897
1898	47	39,231	Mar. 14, 1898
1899	47	41,475	Apr. 26, 1899
1900	47	43,546	Apr. 23, 1900
1901	65	54,862	Apr. 24, 1901
1902	82	42,940	Mar. 18, 1902
1903	97	56,283	Mar. 25, 1903
1904	166	36,305	Apr. 11, 1904
1905	166	48,877	Apr. 1, 1905
1906	202	40,300	Apr. 16, 1906
1907	241	36,700	Apr. 1, 1907
1908	241	34,300	Apr. 28, 1908
1909	261	46,300	Apr. 16, 1909
1910	281	37,800	Apr. 3, 1910
1911	301	26,200	May 3, 1911
1912	321	47,275	Apr. 8, 1912
1913	351	a120,000	Mar. 28, 1913
1914	381	64,788	Apr. 22, 1914
1915	401	33,185	Feb. 25, 1915
1916	431	35,845	Apr. 2, 1916
1917	451	36,300	June 13, 1917
1918	471	35,500	Apr. 3, 1918
1919	501	31,600	Apr. 13, 1919
1920	501	36,100	Apr. 6, 1920
1921	521	33,000	Mar. 22, 1921
1922	541	72,900	Apr. 17, 1922
1923	561	43,700	Apr. [illegible], 1923
1924	581	39,800	Apr. [illegible], 1924
1925	601	44,300	Mar. 30, 1925
1926	621	51,800	Apr. 26, 1926
1927	641	35,600	Mar. 21, 1927
1928	661	70,000	Nov. 4, 1927
1929	681	40,200	Mar. 25, 1929
1930	696	23,600	Apr. 8, 1930
1931	711	24,300	July 22, 1931
1932	726	27,700	Apr. 17, 1932
1933	741	46,700	Apr. 19, 1933
1934	756	29,400	Apr. 12, 1934
1935	781	34,000	Jan. 10, 1935

‡ Not previously published.
a Maximum peak discharge.

' West Virginia Pulp &
of a mile upstream from

an sea level, datum of

adjusted).

[illegible]; practically no flow

[illegible] (Report of U. S. Board

...eels, and through lock of
...ated by Indian Lake since
... (see p. 62). During
...s feeder, Bond Creek (see
and occasionally may re-
...n Canal at Dunham basin.

...eels furnished by West

...eet per second

July	Aug.	Sept.	The year
1,537	1,725	2,851	7,096
5,727	4,272	1,963	7,476
1,350	2,019	8,844	9,030
2,337	2,666	2,040	7,922
9,287	5,485	4,448	10,498
2,521	5,005	6,870	7,506
3,188	2,456	1,869	6,197
1,559	3,901	2,629	5,716
2,772	2,442	2,879	7,452
1,109	8,241	2,756	8,974
2,751	5,029	3,810	8,413
2,402	1,417	2,054	7,108
2,352	2,703	1,886	7,058
3,551	4,661	4,024	7,214
8,934	6,286	3,643	#8,480
4,863	5,905	4,118	9,322
#3,953	#5,463	6,660	#8,485
6,935	5,631	12,032	#8,641
5,780	3,480	3,070	#8,500
4,120	#2,370	6,150	#7,170
2,140	1,770	1,020	9,710
1,990	1,520	1,460	7,090
1,600	1,990	2,270	6,650
1,360	1,180	2,000	4,760
1,310	1,770	#2,490	7,850
1,760	1,070	#1,170	#8,420
2,140	1,450	1,930	6,750
8,500	7,880	3,850	6,010
[illegible]	1,710	1,920	7,850
[illegible]	[illegible]	1,770	7,060
[illegible]	[illegible]	3,130	6,210
[illegible]	1,970	3,710	6,990
[illegible]	2,660	1,970	7,140
4,390	2,010	1,390	6,830
5,770	3,120	2,450	8,450
1,870	1,300	1,600	5,710
2,040	1,870	3,000	7,640
6,470	4,800	4,730	7,510
3,170	2,510	2,670	7,760
2,070	2,300	2,930	6,310
6,410	4,540	3,220	9,950
3,960	2,040	2,480	7,720
[illegible]	2,430	2,570	5,990
[illegible]	3,390	3,690	4,240
4,280	3,790	3,750	6,720
2,970	3,830	4,070	8,000
3,127	2,963	2,867	5,873
[illegible]	5,579	4,339	7,095
2,736	2,561	3,068	7,759
5,148	4,597	4,227	7,889
4,248	4,408	11,320	7,003
3,352	3,035	2,977	7,519
5,861	3,663	4,736	6,282

in reports of State engineer and

Monthly and yearly mean discharge, in cubic feet per second, of Hudson River at Mechanicville, N. Y.--Continued

Water year	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	The year
1941	3,762	6,846	8,013	8,006	6,167	5,395	10,430	2,622	2,009	1,937	1,604	2,169	4,898
1942	2,378	4,066	4,733	5,174	3,287	9,743	13,210	5,957	7,587	3,702	3,075	4,475	5,616
1943	6,598	8,976	8,592	6,780	7,005	13,720	11,340	21,880	7,600	4,564	5,374	3,955	8,890
1944	3,535	6,884	5,213	3,006	3,832	8,929	17,480	9,474	7,422	4,557	3,188	3,577	6,413
1945	4,203	4,283	3,915	6,116	5,230	18,150	11,340	17,690	8,290	7,444	4,742	5,568	8,113
1946	13,840	17,850	8,678	8,146	5,969	15,180	5,654	8,930	7,911	3,836	3,816	3,388	8,210
1947	4,813	4,497	5,047	7,595	9,116	9,592	16,460	20,350	16,750	7,505	4,963	3,181	9,145
1948	3,024	4,156	3,412	2,603	4,273	15,800	13,800	10,490	7,235	4,117	3,470	2,567	6,245
1949	[illegible]	4,676	8,475	18,480	10,900	10,162	[illegible]	4,845	2,941	2,365	2,596	[illegible]	6,657
1950	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	3,498	[illegible]	7,124

Monthly and yearly runoff, in inches

Water year	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	The year
1888	0.61	1.04	2.05	1.62	0.89	1.75	5.26	5.49	1.22	0.39	0.44	0.71	21.47
1889	1.18	2.64	2.57	2.81	.88	2.12	3.39	2.27	1.70	1.47	1.09	.49	22.61
1890	.96	1.96	3.39	2.89	1.83	2.85	3.73	4.59	1.83	.50	.52	2.19	27.24
1891	2.36	2.26	.83	2.12	2.70	4.55	4.97	1.42	.79	.60	.68	.51	23.79
1892	.38	1.01	2.27	4.83	2.22	2.80	5.35	5.03	3.08	2.38	1.41	1.10	31.86
1893	.72	1.89	1.03	.82	1.09	2.11	4.44	5.71	1.19	.65	1.28	1.70	22.63
1894	.99	.90	1.85	1.73	1.12	3.78	2.76	1.94	1.76	.81	.63	.47	18.74
1895	.94	1.58	1.12	.99	.82	1.08	5.91	1.76	.70	.66	1.00	.65	17.21
1896	.69	2.08	2.79	1.74	1.12	3.49	6.20	1.18	1.18	.72	.63	.71	[illegible]
1897	1.05	2.82	1.78	1.03	.90	3.12	4.73	3.11	2.93	2.85	2.11	.68	[illegible]

Yearly discharge, in cubic feet per second

Year	W.S.P. no.	Water year ending Sept. 30: Maximum day: Discharge	Maximum day: Date	Minimum day	Mean	Per square mile	Runoff in inches	Calendar year: Mean	Calendar year: Runoff in inches
1888	24	-	-	-	7,096	1.58	21.47	7,957	24.16
1889	24	-	-	-	7,476	1.66	22.61	7,442	22.53
1890	24	-	-	-	9,030	2.01	27.24	8,757	26.38
1891	24	-	-	-	7,922	1.76	23.79	7,302	22.00
1892	24	-	-	-	10,498	2.33	31.86	10,524	31.84
1893	24	-	-	-	7,506	1.67	22.63	7,528	22.73
1894	24	-	-	-	6,197	1.38	18.74	6,170	18.64
1895	24	-	-	-	5,716	1.27	17.21	6,346	19.13
1896	35	-	-	-	7,457	1.66	22.53	7,487	22.62
1897	35	31,060	July 15, 1897	2,180	8,674	1.99	27.11	9,352	28.27
1898	47	39,231	Mar. 14, 1898	1,163	8,413	1.87	25.40	7,987	24.08
1899	47	41,475	Apr. 26, 1899	304	7,108	1.58	21.52	6,625	19.91
1900	47	43,546	Apr. 23, 1900	713	7,058	1.57	21.11	6,770	20.17
1901	65	54,862	Apr. 24, 1901	1,377	7,214	1.60	21.64	7,544	22.66
1902	82	42,940	Mar. 18, 1902	1,400	#8,640	#1.88	#26.07	#9,034	#27.36
1903	97	56,283	Mar. 25, 1903	1,067	9,322	2.07	27.70	9,160	27.63
1904	166	36,305	Apr. 11, 1904	705	#8,485	#1.89	#25.30	#8,353	#24.89
1905	166	48,877	Apr. 1, 1905	-	#8,641	#1.92	#26.07	#8,711	#26.24
1906	202	40,300	Apr. 16, 1906	187	#8,500	#1.89	#25.59	7,910	23.82
1907	241	36,700	Apr. 1, 1907	872	#7,170	#1.59	#21.69	#9,020	#27.31
1908	241	34,300	Apr. 28, 1908	673	9,710	2.16	29.35	7,190	21.69
1909	261	46,300	Apr. 16, 1909	235	7,090	1.58	21.25	7,050	21.12
1910	281	37,800	Apr. 3, 1910	345	6,650	1.48	20.04	6,900	20.80
1911	301	26,200	May 3, 1911	333	4,760	1.06	14.38	6,190	18.67
1912	321	47,375	Apr. 8, 1912	489	7,850	1.74	23.68	7,950	24.08
1913	351	a120,000	Mar. 28, 1913	[illegible]	#8,420	#1.87	#25.42	#7,370	#22.25
1914	381	64,700	Apr. 22, 1914	404	6,750	[illegible]	[illegible]	[illegible]	[illegible]
1915	401	33,165	Feb. 25, 1915	750	6,010	1.34	18.14	[illegible]	[illegible]
1916	431	35,845	Apr. 2, 1916	882	7,850	1.74	23.75	7,700	23.24
1917	451	36,300	June 13, 1917	899	7,060	1.57	21.31	7,010	21.14
1918	471	35,500	Apr. 3, 1918	576	6,210	1.38	18.71	7,010	21.13
1919	501	31,600	Apr. 13, 1919	1,060	6,990	1.55	21.14	6,960	21.01
1920	501	36,100	Apr. 6, 1920	1,090	7,140	1.59	21.60	7,280	22.04
1921	521	33,000	Mar. 22, 1921	715	6,830	1.52	20.61	6,140	18.53
1922	541	72,900	Apr. 12, 1922	725	8,450	1.88	25.50	7,750	23.38
1923	561	43,700	Apr. 9, 1923	743	5,710	1.27	17.23	6,500	19.59
1924	581	39,800	Apr. 19, 1924	820	7,640	1.70	23.10	7,440	22.52
1925	601	44,300	Mar. 30, 1925	696	7,510	1.67	22.65	8,700	26.22
1926	621	51,800	Apr. 26, 1926	995	7,760	1.72	23.41	7,080	21.39
1927	641	35,600	Mar. 21, 1927	613	6,310	1.40	19.05	7,960	24.03
1928	661	70,000	Nov. 4, 1927	933	9,950	2.21	30.13	7,460	22.58
1929	681	40,700	Mar. 25, 1929	685	7,720	1.72	23.27	7,900	23.80
1930	696	23,600	Apr. 8, 1930	805	5,990	1.33	18.05	5,650	17.04
1931	711	24,300	July 22, 1931	614	4,240	.942	12.77	5,000	15.08
1932	726	27,700	Apr. 12, 1932	1,470	6,720	1.49	20.33	7,660	23.15
1933	741	46,700	Apr. 19, 1933	1,420	8,000	1.78	24.14	6,780	20.45
1934	756	29,400	Apr. 12, 1934	1,170	5,873	1.31	17.71	5,821	17.57
1935	781	34,000	Jan. 10, 1935	1,520	7,095	1.58	21.43	7,781	23.49

Not previously published.
a Maximum peak discharge.

Yearly discharge, in cubic feet per second, of Hudson River at Mechanicville, N. Y.--Continued

Year	W.S.P. no.	Water year ending Sept. 30							Calendar year	
		Momentary maximum		Minimum day	Mean	Per square mile	Runoff in inches		Mean	Runoff in inches
		Discharge	Date							
1936	801	72,700	Mar. 19, 1936	1,500	7,759	1.72	23.46		7,711	23.31
1937	821	28,500	May 16, 1937	1,780	7,889	1.75	23.80		7,719	23.30
1938	851	65,600	Sept. 22, 1938	1,670	7,003	1.56	21.14		7,473	22.56
1939	871	33,800	Apr. 29, 1939	1,060	7,519	1.67	22.71		6,284	18.97
1940	891	40,100	Mar. 31, 1940	1,320	6,282	1.40	19.01		7,062	21.37
1941	921	22,600	Dec. 31, 1940	896	4,898	1.09	14.78		4,274	12.90
1942	951	27,100	Apr. 8, 1942	843	5,616	1.25	16.96		6,705	20.24
1943	971	33,100	May 13, 1943	2,050	8,890	1.98	26.80		8,171	24.64
1944	1001	32,800	Apr. 26, 1944	1,060	6,413	1.43	19.41		6,146	18.60
1945	1031	32,600	Mar. 22, 1945	1,170	8,113	1.80	24.46		10,040	30.26
1946	1051	31,500	Mar. 9, 1946	1,070	8,210	1.82	24.75		6,448	19.44
1947	1081	40,700	June 4, 1947	1,070	9,145	2.03	27.56		8,826	26.60
1948	1111	40,100	Mar. 23, 1948	773	6,245	1.39	18.88		6,686	20.22
1949	1141	94,400	Dec. 31, 1948	886	6,657	1.48	20.07		6,634	20.01
1950	1171	77,000	Apr. 5, 1950	1,640	7,124	1.58	21.50		-	-

Note.--Monthly figures of discharge per square mile and runoff, in inches, since October 1898, previously published in water-supply papers, may be subject to considerable error because of diversions, and storage and evaporation in Indian Lake since 1898, and in Sacandaga Reservoir since Mar. 27, 1930. These figures are not published herein.

62. Black River Canal (flowing south) near Boonville, N. Y.

Location.--Lat 43°27'20", long 75°19'25", gage 1 on left bank at lock 69, 2 miles south of Boonville, Oneida County, and gage 2 on right bank of Lansingkill spillway, 100 ft downstream from spillway headgates, 600 ft upstream from lock 70, and half a mile upstream from lock 69.

Gage.--Two water-stage recorders and concrete controls. Datum of gage 1 is 1,105.56 ft above mean sea level, datum of 1929. Prior to June 7, 1929, station was operated as a slope station on summit level of canal. September 1915 to September 1942 station was operated only during canal season.

Extremes.--1915-50: Maximum daily discharge recorded, 323 cfs Nov. 30, 1915; practically no flow at times when no water is being diverted.

Remarks.--Records include combined flow at gages 1 and 2 and represents total diversion from Black River at Forestport, through Forestport feeder, into Delta Reservoir in Mohawk basin. Discharge during periods when no water was diverted is made up of leakage through headgates and runoff from area draining into canal above station.

Monthly and yearly mean discharge, in cubic feet per second

Water year	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	The year
1916	230	254	-	-	-	-	-	-	220	206	210	200	-
1917	210	205	-	-	-	-	-	-	189	183	173	169	-
1918	176	182	-	-	-	-	-	-	177	154	161	158	-
1919	137	131	-	-	-	-	-	-	-	176	164	169	-
1920	184	-	-	-	-	-	-	-	-	151	136	118	-
1921	144	-	-	-	-	-	-	-	146	166	156	158	-
1922	156	-	-	-	-	-	-	-	-	123	142	144	-
1923	145	-	-	-	-	-	-	-	-	84.6	97.7	129	-
1924	141	-	-	-	-	-	-	-	-	111	105	109	-
1925	125	-	-	-	-	-	-	-	-	47.6	40.8	37.6	-
1926	35.7	-	-	-	-	-	-	-	108	151	161	153	-
1927	130	-	-	-	-	-	-	-	-	97.5	93.8	87.5	-
1928	-	-	-	-	-	-	-	-	25.5	51.7	85.1	85.7	-
1929	-	-	-	-	-	-	-	-	-	34.7	19.7	7.30	-
1930	4.65	-	-	-	-	-	-	-	29.9	64.9	84.5	91.1	-
1931	98.1	-	-	-	-	-	-	12.5	26.8	20.4	49.2	72.5	-
1932	84.1	-	-	-	-	-	-	-	-	-	30.5	38.0	-
1933	47.5	-	-	-	-	-	-	-	-	78.4	89.2	121	
1934	155	-	-	-	-	-	-	-	-	52.4	56.3	62.6	-
1935	-	-	-	-	-	-	-	-	22.7	31.1	50.4	60.5	-
1936	57.8	43.0	-	-	-	-	-	-	30.1	41.9	45.6	51.3	-
1937	65.4	-	-	-	-	-	-	-	16.5	33.6	56.8	67.4	-
1938	69.9	-	-	-	-	-	-	-	28.0	42.3	42.6	51.9	-
1939	36.5	-	-	-	-	-	-	-	49.9	52.0	51.4	54.6	-
1940	42.3	-	-	-	-	-	-	6.78	30.5	39.4	42.9	14.2	-
1941	62.3	-	-	-	-	-	-	43.0	46.7	55.5	53.0	58.2	-
1942	62.7	-	-		-	-	9.2	44.3	48.2	55.3	56.6	61.4	-
1943	27.3	34.2	9.66	4.58	4.24	14.5	16.8	19.6	12.6	34.2	47.7	55.7	23.5
1944	62.3	17.1	6.11	2.06	1.81	97.4	16.0	8.29	38.5	21.0	18.1	29.2	19.2
1945	28.7	12.6	4.76	6.42	4.96	16.2	15.8	25.0	14.9	54.5	57.3	48.6	24.3
1946	22.6	9.32	4.86	5.71	5.48	12.1	4.29	7.50	18.5	22.8	23.8	29.8	14.0
1947	46.9	12.6	1.16	.95	.81	.83	1.46	1.29	1.26	7.00	8.45	61.7	12.0
1948	72.0	80.7	4.20	1.88	4.17	12.6	3.52	33.7	38.1	44.1	58.3	66.5	35.0
1949	63.9	50.3	1.65	1.73	1.98	1.87	1.64	27.0	63.1	62.5	62.2	76.3	34.6
1950	62.1	37.9	1.75	1.93	1.65	2.18	4.26	8.99	19.0	33.2	24.7	62.0	21.7

63. Delta Re

Location.--Lat 43°16'20", long 75°25'
Mohawk River 4 miles upstream from

Drainage area.--145 sq mi.

Gage.--Staff gage. Datum of gage is

Remarks.--Dam completed Aug. 3, 1912
able began May 1, 1913. Usable
2,800,000 cu ft. Reservoir is

Cooperation.--Records not previously
State Department of Public Works.

Contents, in milli

Water year	Oct.	Nov.	Dec.	Jan.*
1913	-	-	-	-
1914	542	1,470	1,980	2,120
1915	1,470	1,759	1,718	1,550
1916	2,482	2,493	1,804	[illegible]
1917	1,498	2,290	[illegible]	[illegible]
1918	2,680	1,782	1,334	[illegible]
1919	1,900	2,205	1,705	[illegible]
1920	2,848	2,770	2,273	962
1921	1,282	2,180	2,806	2,728
1922	1,920	2,312	839	1,198
1923	2,020	1,995	1,422	1,970
1924	2,145	2,185	1,100	22
1925	2,185	2,135	1,995	1,110
1926	3	4	1	2
1927	2,356	2,890	1,486	530
1928	1,718	2,926	2,608	2,504
1929	1,955	1,905	1,790	1,286
1930	1,145	1,229	1,714	1,010
1931	1,286	1,274	1,330	1,120
1932	2,045	2,383	2,818	2,740
1933	2,884	2,686	2,462	2,170
1934	1,822	2,155	2,175	1,502
1935	1,546	1,905	1,975	1,450
1936	1,885	2,608	2,322	1,354
1937	1,865	2,515	2,872	2,581
1938	2,120	2,842	2,405	1,723
1939	1,134	624	11	394
1940	1,114	896	1,013	750
1941	1,606	2,100	2,950	1,486
1942	2,100	1,462	1,506	[illegible]
1943	2,438	2,559	2,312	[illegible]
1944	2,385	2,372	1,466	[illegible]
1945	1,895	1,696	1,678	[illegible]
1946	2,225	2,130	1,574	1,514
1947	2,339	2,328	2,045	2,642
1948	1,426	1,358	1,100	1,010
1949	1,162	1,840	2,356	1,975
1950	1,615	1,746	2,422	2,454

64. Mohawk River

Location.--Lat 43°15'50", long 75°2
1 mile downstream from Delta Dam.

Drainage area.--150 sq mi.

Gage.--Water-stage recorder. Datum
datum). Prior to Jan. 24, 1937.

Average discharge.--29 years (1921-

Extremes.--1927-50: Maximum discha
minimum, 30 cfs Sept. 27, 1943
Jan. 17, 1931.

Remarks.--During canal navigation
Forestport feeder and Black River
tion (see p. 82). Flow almost
station). Small quantity of wat
and later returned to river, par

Cooperation.--Records for 1921-27,
nished by State engineer and sur

Bridge, N. Y.

k three-quarters of a mile down-
of village of Eagle Bridge,

23 to September 1960.

ft above mean sea level, datum of
1923, to July 18, 1936, water-
m at different datum.

cfs.

0 cfs Dec. 31, 1948 (gage height,
rating curve extended above 13,000
contracted-opening measurements at
t. 14, 1913; minimum daily, 30 cfs

caused by powerplants above station.

ubic feet per second

June	July	Aug.	Sept.	The year
510	748	565	443	1,144
1,624	293	250	265	1,193
455	203	156	127	978
905	252	267	597	864
397	188	419	222	897
713	268	139	377	1,064
456	348	148	152	659
380	395	245	291	830
421	250	198	178	796
557	471	410	1,561	1,217

in inches

June	July	Aug.	Sept.	The year
1.12	1.69	1.28	0.97	30.45
3.55	.66	.56	.58	31.83
.99	.46	.35	.28	26.03
1.98	.57	.60	1.31	23.01
.87	.42	.95	.49	23.86
1.56	.61	.31	.82	28.40
1.00	.79	.33	.33	17.55
.83	.89	.55	.64	22.08
.92	.56	.45	.39	21.17
1.22	1.06	.93	3.41	32.47

et per second

30		Calendar year	
Per square mile	Runoff in inches	Mean	Runoff in inches
-	-	1,019	27.12
2.24	30.45	1,218	32.44
2.34	31.83	960	25.61
1.92	26.03	1,002	26.67
1.69	23.01	1,011	26.89
1.76	23.86	917	24.42
2.09	28.40	951	25.38
1.29	17.55	635	16.91
1.63	22.08	820	21.82
1.56	21.17	960	25.54
2.39	32.47	-	-

3355. Hudson River at Mechanicville, N. Y.

Location.--Lat 42°54'45", long 73°40'45", on right bank at dam of West Virginia Pulp & Paper Co., at Mechanicville, Saratoga County, three-quarters of a mile upstream from Anthony Kill and 1½ miles downstream from Hoosic River.

Drainage area.--4,500 sq mi.

Records available.--October 1887 to September 1956.

Gage.--Water-stage recorder. Datum of gage is 66.63 ft above mean sea level, datum of 1929. Prior to 1911, staff gage at same site and datum.

Average discharge.--69 years (1887-1956), 7,431 cfs (unadjusted).

Extremes.--1887-1956: Maximum discharge, 120,000 cfs Mar. 28, 1913; practically no flow for short periods when plant was shut down.
Maximum known discharge prior to 1913, 70,000 cfs April 1869 (report of U.S. Board of Engineers on Deep Waterways).
Since 1930, maximum discharge, 118,000 cfs Jan. 1, 1949.

Remarks.--Discharge computed from flow over spillway, through wheels and through lock of Champlain (Barge) Canal since Sept. 30, 1915. Flow appreciably regulated by Indian Lake since 1898 and Sacandaga Reservoir since Mar. 27, 1930. During canal navigation season, water is diverted through Glens Falls feeder, Bond Creek, and Champlain (Barge) Canal into Lake Champlain basin and occasionally may receive water from that basin through summit level of Champlain (Barge) Canal at Dunham Basin. No adjustment made for these diversions. Records of water temperatures for the period October 1954 to September 1960 are published in reports of the Geological Survey.

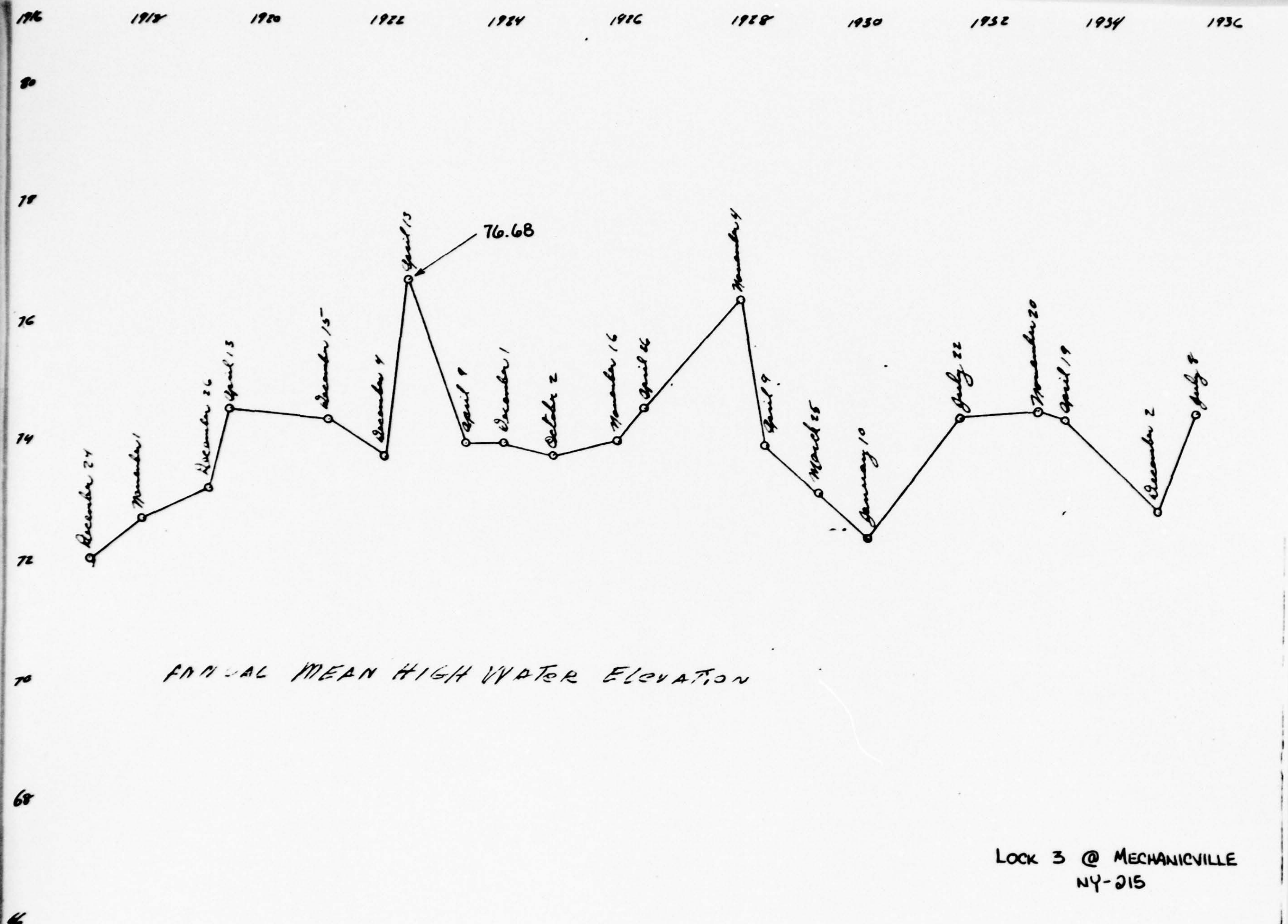
Cooperation.--Records of discharge over spillway and through wheels furnished by West Virginia Pulp & Paper Co.

Monthly and yearly mean discharge, in cubic feet per second

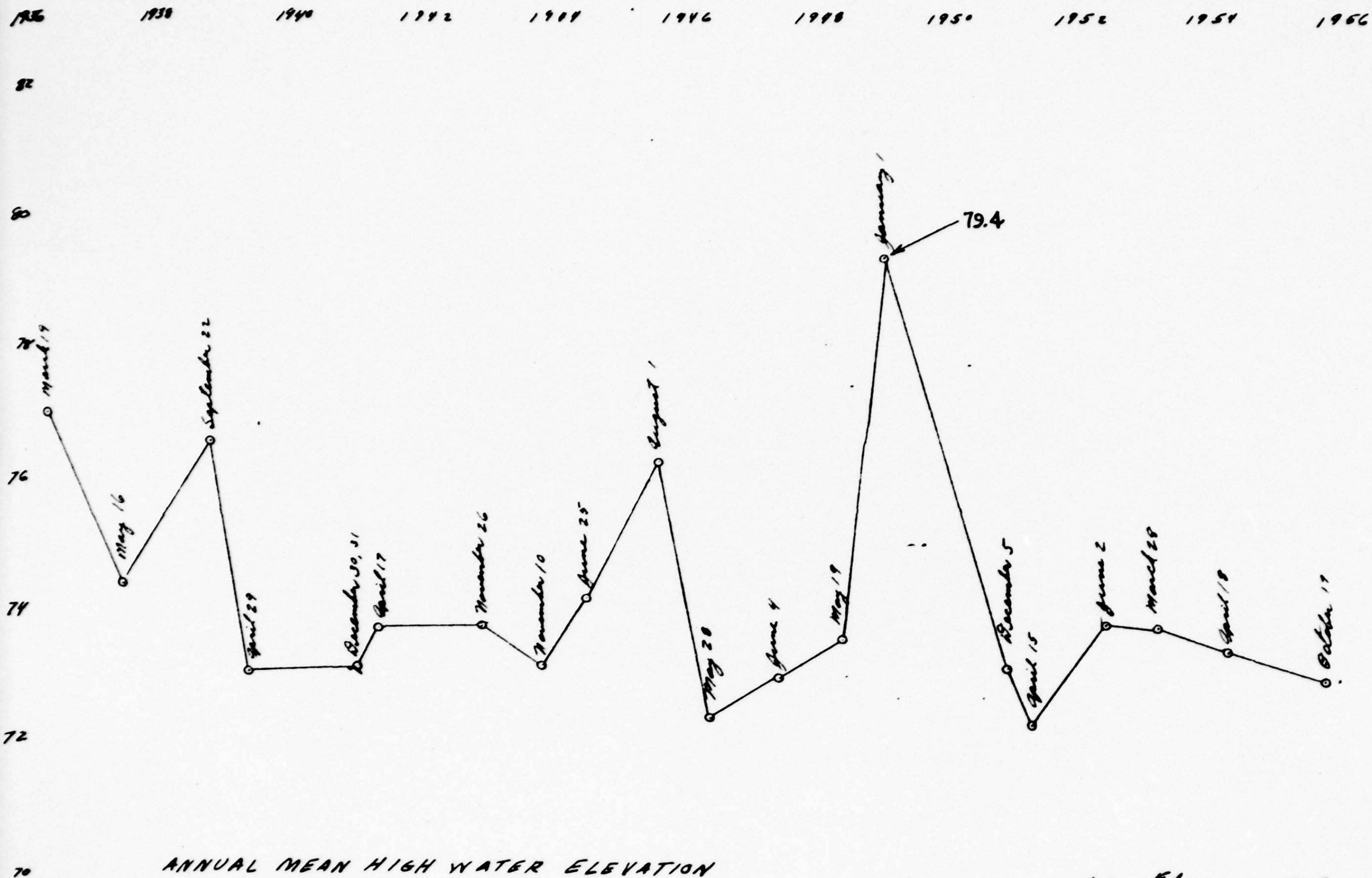
Water year	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	The year
1951	3,409	5,937	9,855	7,528	11,000	10,810	23,840	7,013	4,557	7,770	5,151	4,995	8,454
1952	6,610	12,380	11,190	11,230	9,938	11,780	26,450	12,880	9,522	4,209	3,541	3,426	10,240
1953	3,035	2,898	7,803	7,941	8,490	14,950	16,390	17,730	4,048	3,177	3,275	3,154	7,747
1954	3,238	3,245	6,748	5,746	10,340	9,182	19,190	16,460	9,573	3,215	2,898	4,280	7,809
1955	3,558	7,401	8,030	5,823	5,939	9,818	19,790	5,853	5,329	2,918	3,712	3,151	6,760
1956	7,130	11,520	6,388	5,945	5,006	7,234	16,930	12,210	7,472	4,065	3,864	4,748	7,698
1957	-	-	-	-	-	-	-	-	-	-	-	-	-
1958	-	-	-	-	-	-	-	-	-	-	-	-	-
1959	-	-	-	-	-	-	-	-	-	-	-	-	-
1960	-	-	-	-	-	-	-	-	-	-	-	-	-

Yearly discharge, in cubic feet per second

Year	WSP	Water year ending Sept. 30							Calendar year	
		Momentary maximum		Minimum day	Mean	Per square mile	Runoff in inches		Mean	Runoff in inches
		Discharge	Date							
1950	-	-	-	-	-	-	-		7,452	22.41
1951	1202	39,400	Mar. 31, 1951	1,770	8,454	1.88	25.49		9,369	28.26
1952	1232	46,300	Apr. 6, 1952	1,290	10,240	2.28	30.97		8,870	26.84
1953	1272	44,400	Mar. 27, 1953	1,130	7,747	1.72	23.35		7,703	23.21
1954	1332	38,200	Apr. 18, 1954	1,070	7,809	1.74	23.54		8,286	24.98
1955	1382	33,000	Apr. 16, 1955	1,010	6,760	1.50	20.39		7,262	21.92
1956	1432	32,900	Apr. 30, 1956	2,030	7,698	1.71	23.29		-	-
1957	-	-	-	-	-	-	-		-	-
1958	-	-	-	-	-	-	-		-	-
1959	-	-	-	-	-	-	-		-	-
1960	-	-	-	-	-	-	-		-	-


1916
1918
1920
1922
1924
1926
1928
1930
1932
1934
1936
80
78
76
74
72
70
68
66
76.68
December 24
November 1
December 26
April 13
December 15
December 4
April 13
April 9
December 1
October 2
November 16
April 26
November 4
April 9
March 25
January 10
July 22
November 20
April 19
December 2
July 8
ANNUAL MEAN HIGH WATER ELEVATION
LOCK 3 @ MECHANICVILLE
NY-215

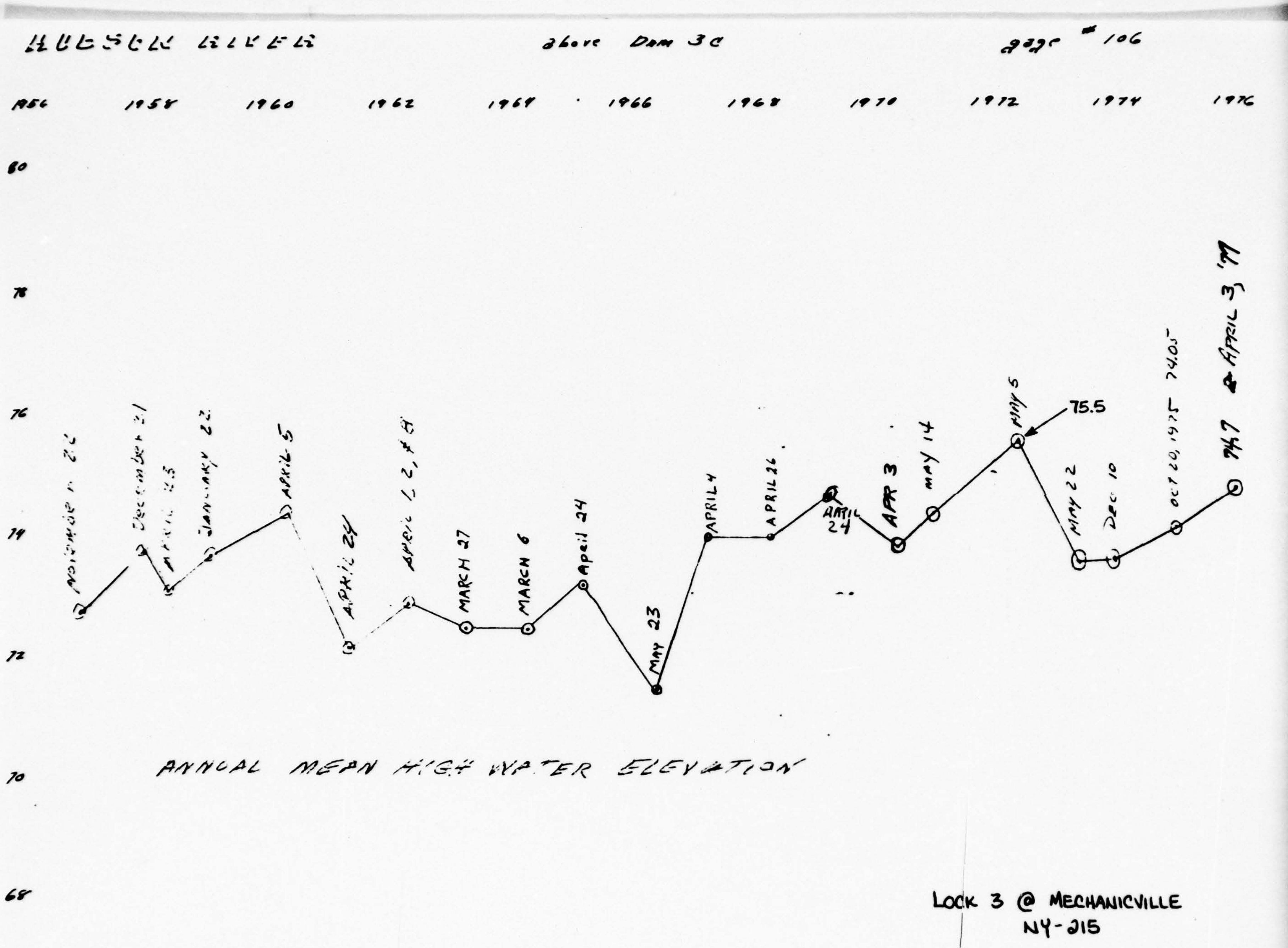
HUDSON RIVER above DAM # 5 C gage # 106

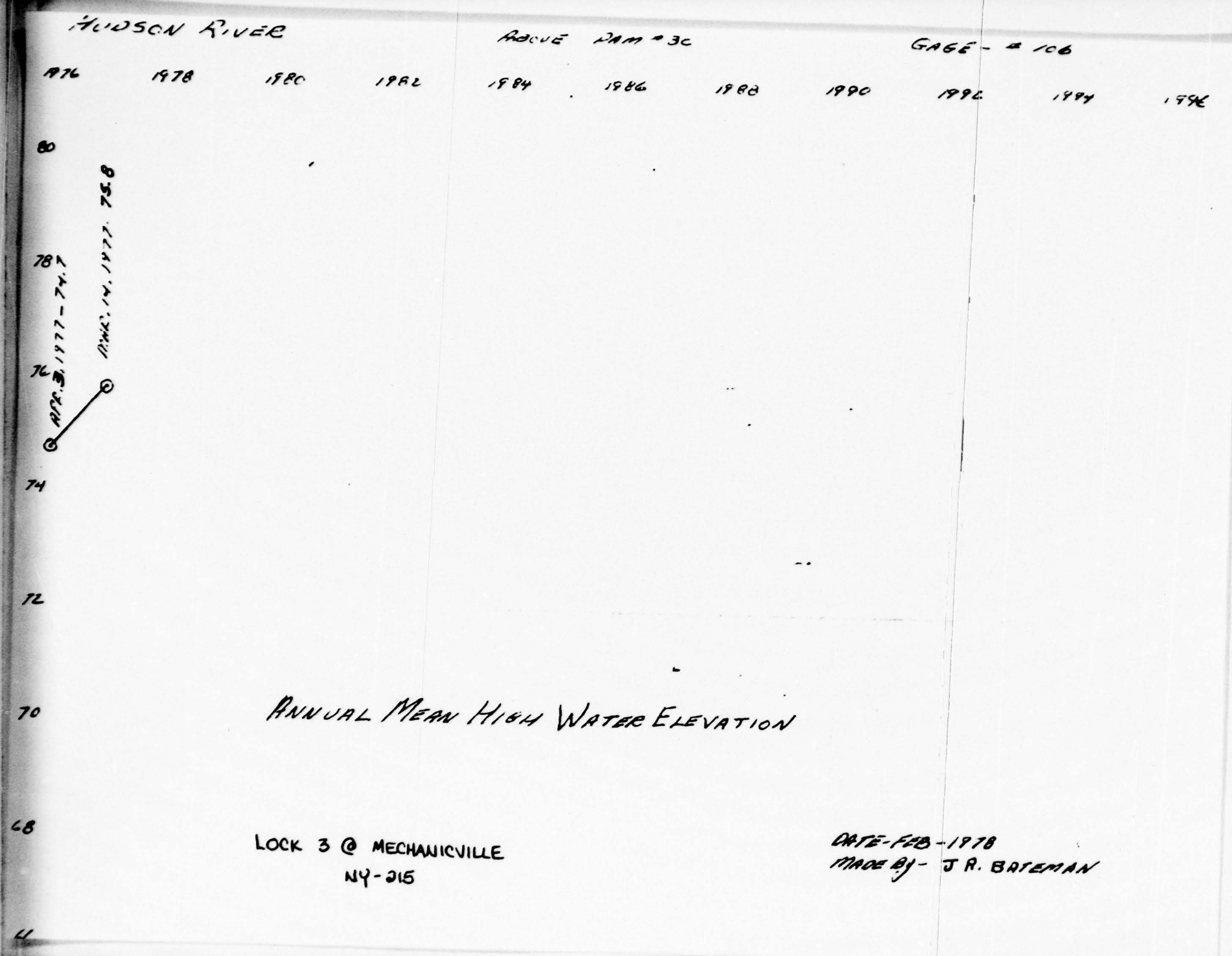


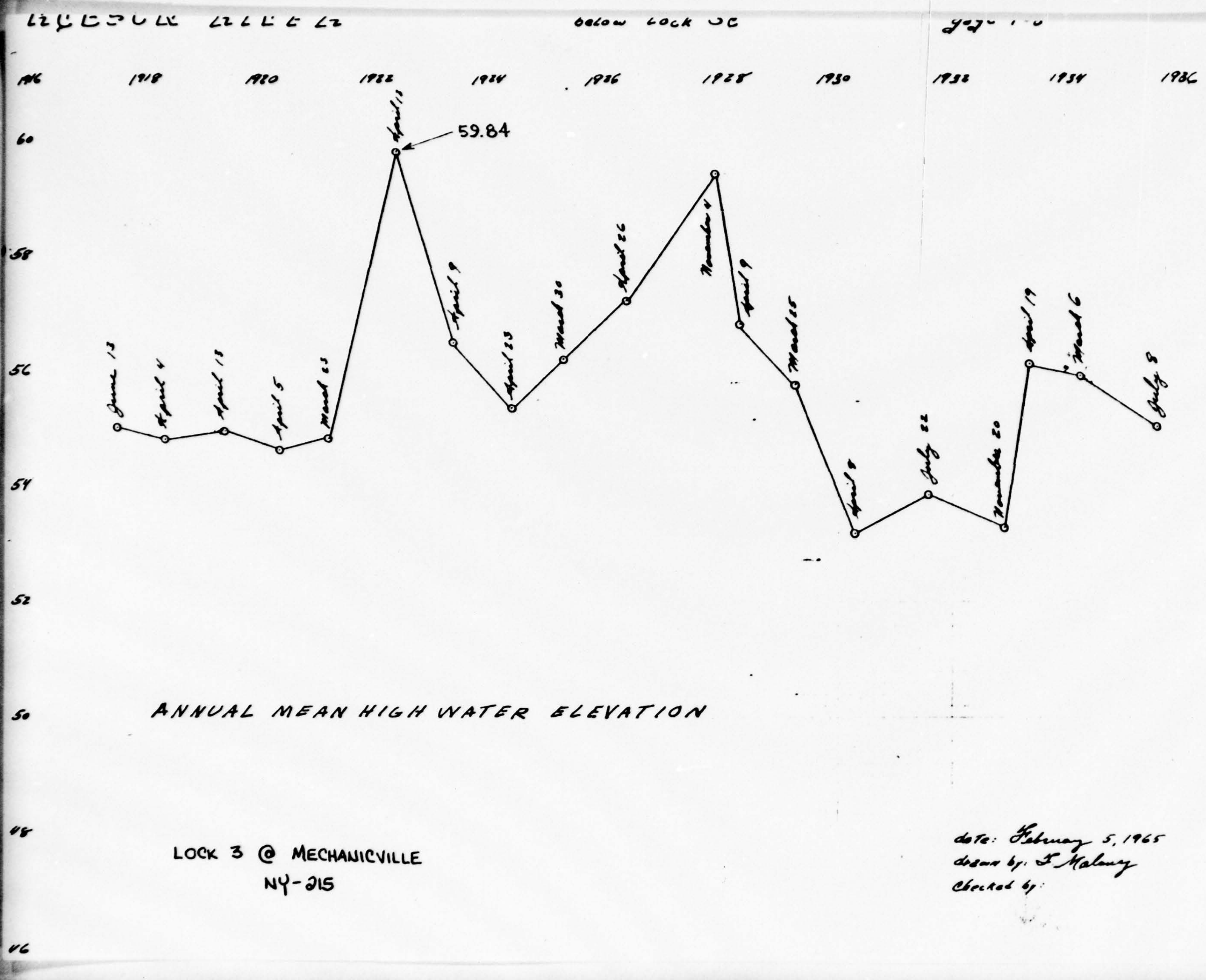
ANNUAL MEAN HIGH WATER ELEVATION

LOCK 3 @ MECHANICVILLE
NY-215

date: February 10, 1965
drawn by: J. Malony
Checked by:


above Dam 3C
gage #106
1956
1958
1960
1962
1964
1966
1968
1970
1972
1974
1976
80
78
76
74
72
70
68
December 21
January 22
April 5
April 24
April 1, 2, & 8
March 27
March 6
April 24
May 23
April 4
April 26
April 24
Apr 3
May 14
May 5
75.5
May 22
Dec 10
Oct 20, 1975 74.05
74.7 @ April 3, '77
ANNUAL MEAN HIGH WATER ELEVATION
LOCK 3 @ MECHANICVILLE
NY-215


HUDSON RIVER
ABOVE DAM #3C
GAGE - #106
1976
1978
1980
1982
1984
1986
1988
1990
1992
1994
80
78
76
74
72
70
68
APR. 3, 1977 - 74.7
MAR. 14, 1977. 75.8
ANNUAL MEAN HIGH WATER ELEVATION
LOCK 3 @ MECHANICVILLE
NY-215
DATE - FEB - 1978
MADE BY - J R. BATEMAN


below Lock 3C
1916
1918
1920
1922
1924
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1928
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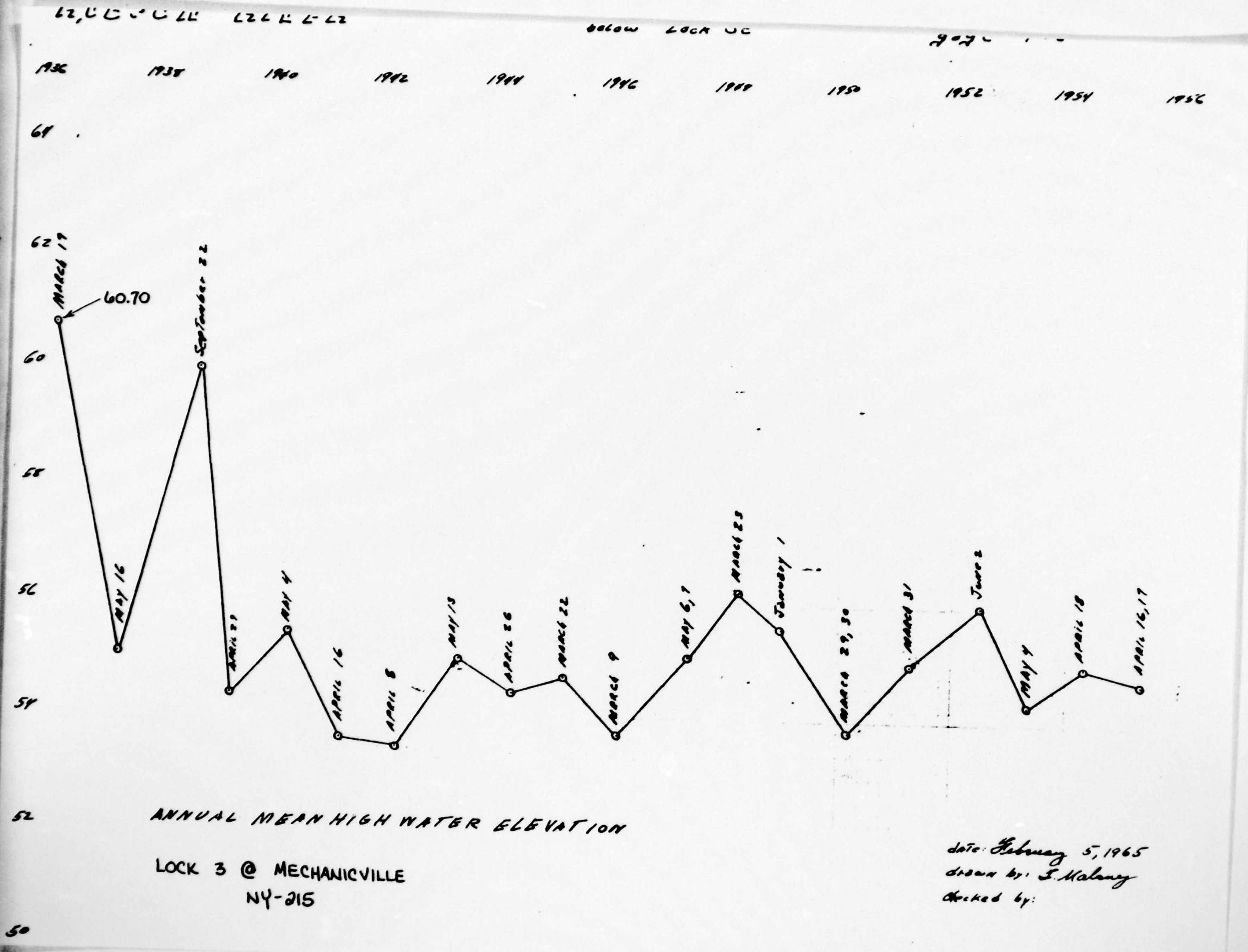
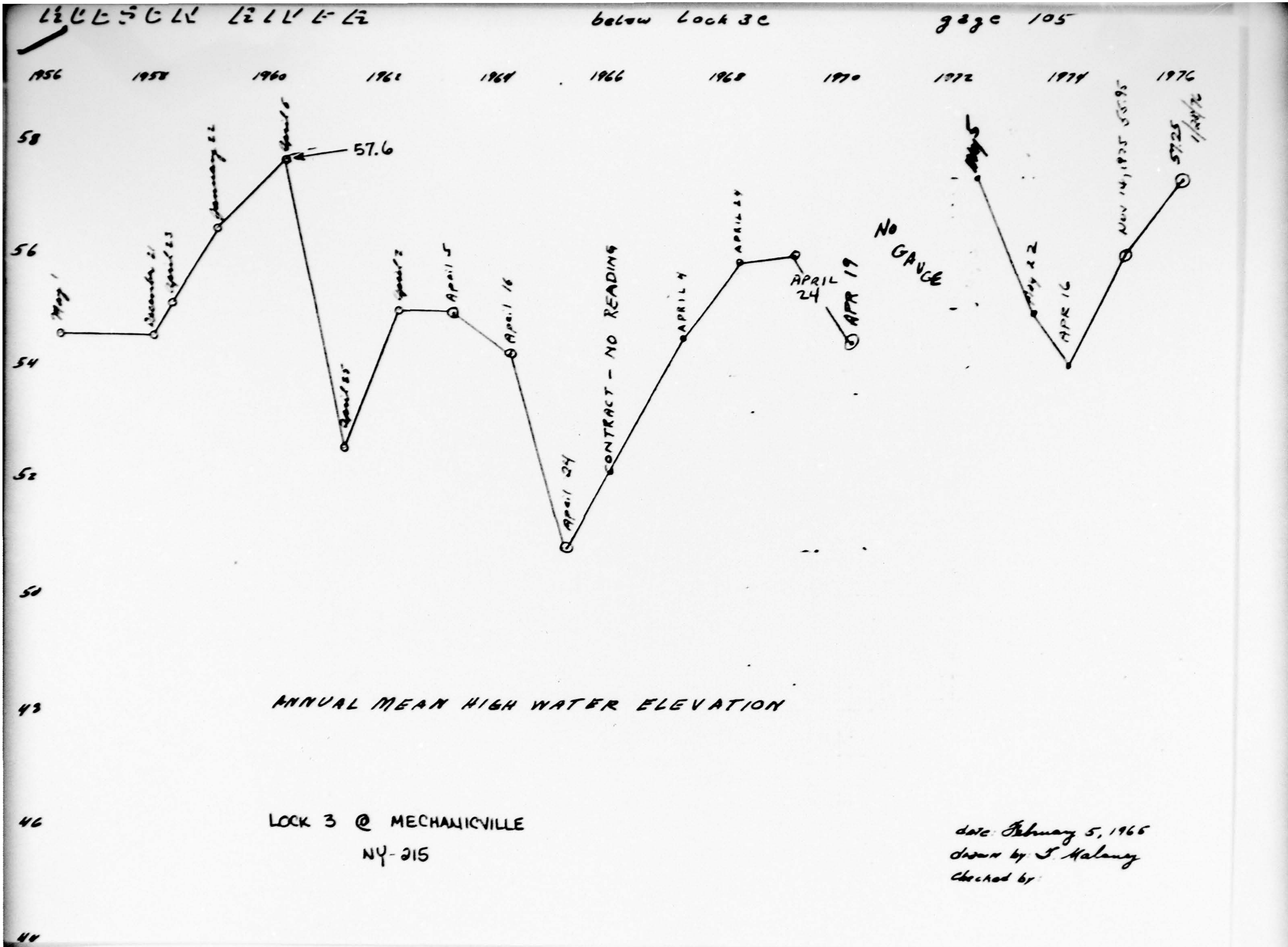
1932
1934
1936
60
58
56
54
52
50
48
46
59.84
June 13
April 4
April 13
April 5
March 23
April 13
April 9
April 23
March 30
April 26
November 4
April 9
March 25
April 8
July 22
November 20
April 19
March 6
July 8
ANNUAL MEAN HIGH WATER ELEVATION
LOCK 3 @ MECHANICVILLE
NY-215
date: February 5, 1965
drawn by: F. Maloney
Checked by:



ANNUAL MEAN HIGH WATER ELEVATION

LOCK 3 @ MECHANICVILLE
NY-215

date: February 5, 1965
drawn by: J. Maloney
checked by:


below Lock 3C
gage 105
1956
1958
1960
1962
1964
1966
1968
1970
1972
1974
1976
58
56
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52
50
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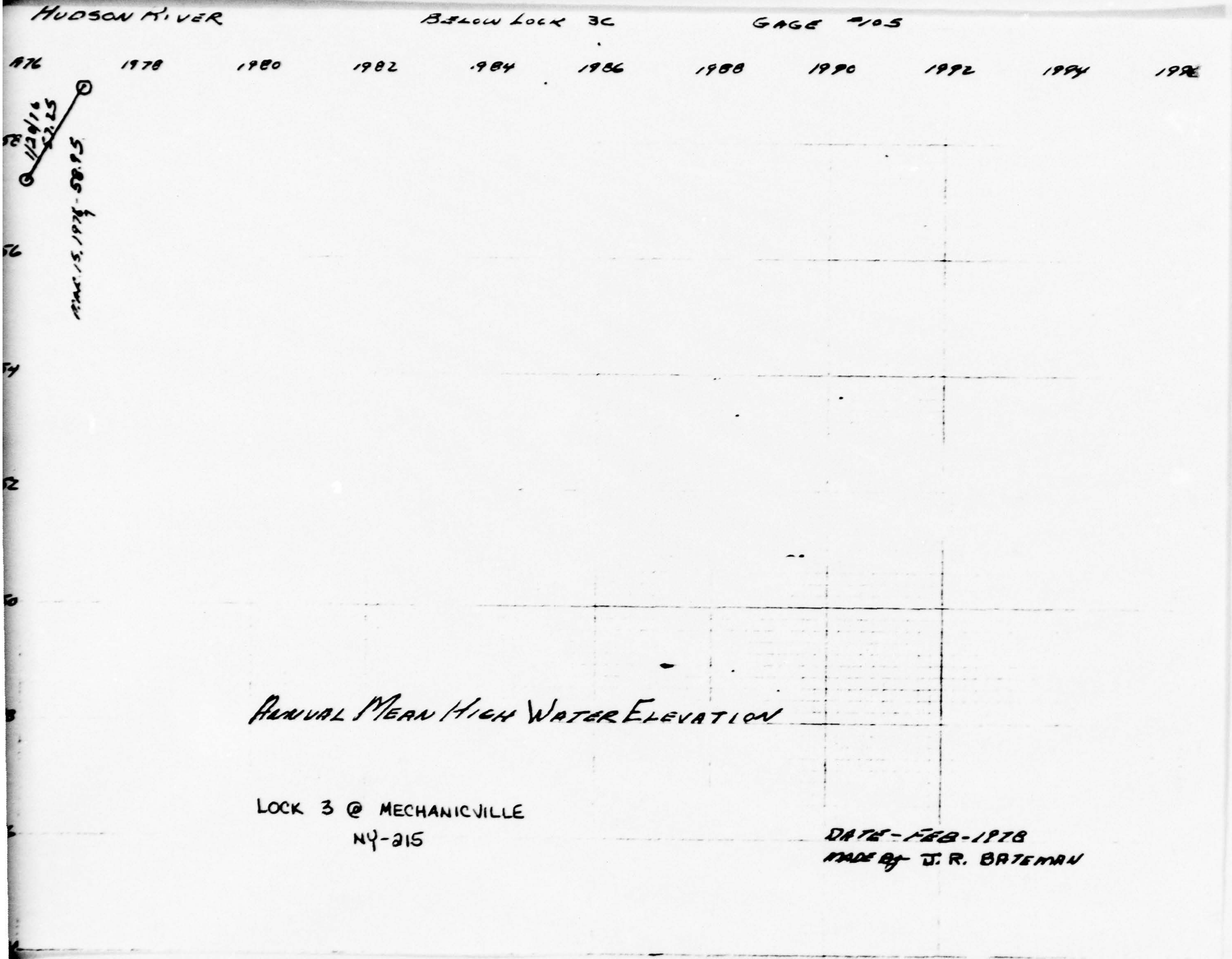
46
44
57.6
April 5
April 16
April 24
CONTRACT - NO READING
APRIL 4
APRIL 24
APRIL 24
APR 19
NO GAUGE
APR 16
Nov 14, 1975 55.95
57.25
ANNUAL MEAN HIGH WATER ELEVATION
LOCK 3 @ MECHANICVILLE
NY-215
date: February 5, 1966
drawn by: J. Malaney
checked by:


HUDSON RIVER
BELOW LOCK 3C
GAGE #105
1976
1978
1980
1982
1984
1986
1988
1990
1992
1994
1996
58
56
54
52
50
11/20/76 57.25
MAR. 15, 1976 - 58.95
ANNUAL MEAN HIGH WATER ELEVATION
LOCK 3 @ MECHANICVILLE
NY-215
DATE - FEB - 1978
MADE BY - J. R. BATEMAN

I.D. No. NY-215 LOCK 3 DAM @ MECHANICVILLE

Water rights to utility

3/9/79

By RICHARD BANTTARI
Staff Writer

MECHANICVILLE — The water rights of the former Saratoga Board Mills Corporation, described by a study of the New York State Army Engineers as one of the 10 most desirable sources of low - cost energy in the United States, will be sold to the New York State Electric & Gas Corporation.

Robert Lockwood, counsel for the City National Bank of Detroit which owns the paper mill property, said from Detroit Thursday, that the bank has signed a "tentative agreement" with NYSE&G. According to Lockwood, the utility will pay "less than" $2.1 million for the rights.

Neither Lockwood nor Daniel Collins, Berkshire District Area manager for NYSE&G, would elaborate on the sale. It was not known if the purchase by the utility would involve the Mill's defunct hydroelectric plant which includes 25 - cycle regenerative equipment.

Collins said the utility would issue a press release sometime today or Saturday.

Lockwood said one of the officers for NYSE&G, Larry Sweetland, an industrial development manager based in corporate headquarters in Binghamton, has been handling negotiations with the bank since City National successfully bid $2,000,001 for the mill property Feb. 19.

A spokesman for NYSE&G in Binghamton said Sweetland was unavailable for comment because he was on a Buffalo business trip.

NYSE&G originally filed a letter of intent nearly two years for the purchase of the mill's water rights to former Saratoga Board Mills Corporation President Hy Sweet. According to a Nov. 29 referee's report filed by foreclosure attorney Pat Keniry, the utility was interested in purchasing approximately 1.2 acres of mill land together with certain easements and water rights.

Keniry's report listed the NYSE&G offer at $2,075,000.

Lockwood said the bank was awaiting finalization from the utility for the offer. Asked when the sale would be made, the attorney could offer no date. "We're ready," he said.

The Detroit lawyer said the bank has received at least three or four offers for the water rights since NYSE&G and the bank began to negotiate. Some of the offers, he said were as high as $3 and $4 million. But, Lockwood said, the additional offers appeared to include contingencies.

"We're not in the paper business," Lockwood said, referring to the bank's willingness to sell off the mill property. He said he is tentatively scheduled to be in Mechanicville Thursday and said the bank is "still open" to any offers from local concerns.

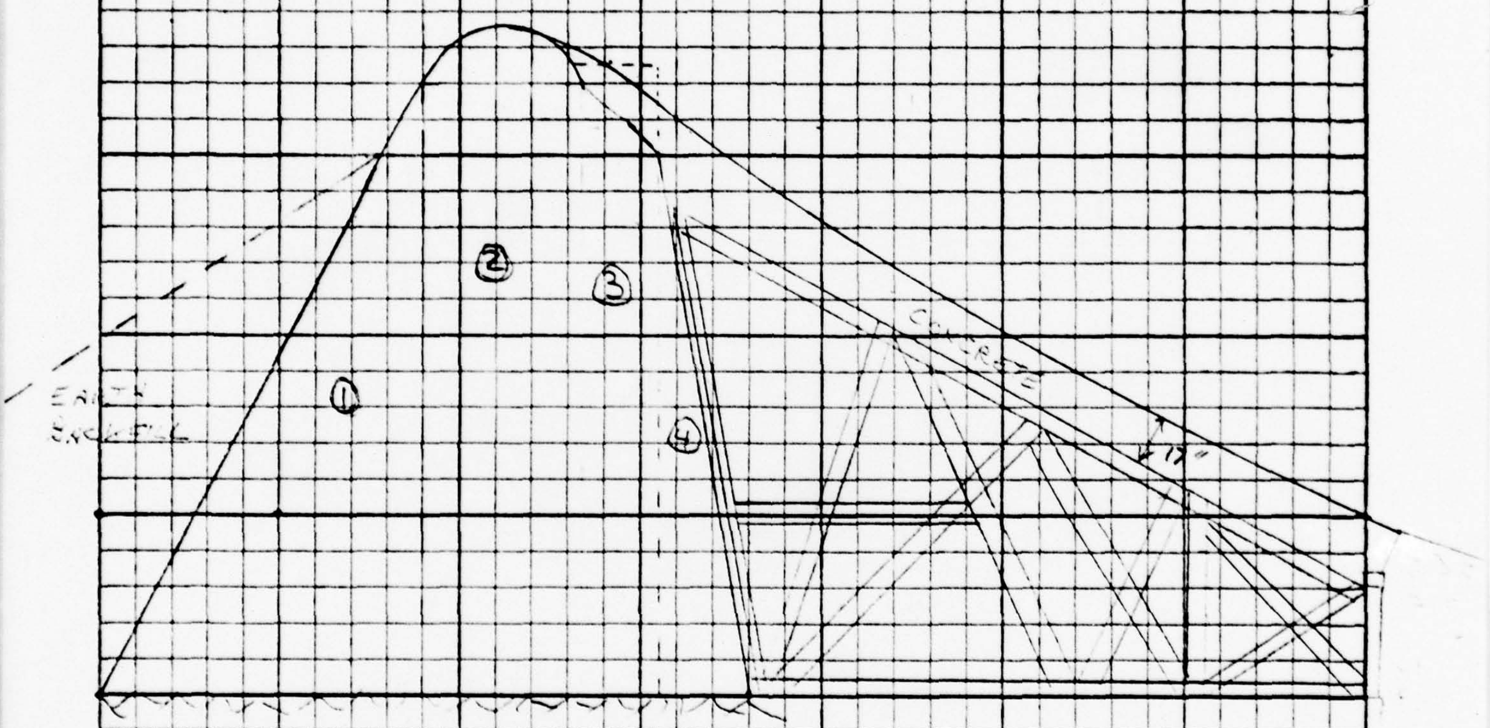
APPENDIX E

STRUCTURAL STABILITY ANALYSES

GA-17 (4/70) NEW YORK STATE DEPARTMENT OF ENVIRONMENTAL CONSERVATION

PROJECT GRID

JOB	SHEET NO.	CHECKED BY	DATE
Lock No. 3	1		
SUBJECT		**COMPUTED BY**	**DATE**
Stability Analysis			



			$\bar{X}$
AREA ①	(9)(17)(½)	= 76.5	12′
(2a)	½ (π)(2.3)²	= 8.3	6.75′
(2b) +	(4.5)(17)	= 76.5	6.75′
		84.8	
AREA ②		= 84.8	6.75
AREA ③	(17.5)(2.1)	= 36.75	3.35
AREA ④	½ (17)(2.3)	= 19.55	1.53

GA-17 (4/70) NEW YORK STATE DEPARTMENT OF ENVIRONMENTAL CONSERVATION

PROJECT GRID

JOB	SHEET NO.	CHECKED BY	DATE
Lock No 3	2		
SUBJECT		COMPUTED BY	DATE
Stability Analysis (cont)		RLW	3/12/79

RESOLVE FORCES ACTING ON MAIN SECTION DUE TO STEEL FRAMEWORK & CONCRETE KEY INTO ONE PASSIVE FORCE ACTING AT HEIGHT OF 12.5'

1. DETERMINE APPROXIMATE WEIGHT FOR APRON

FRAMEWORK MADE OF 2 ANGLES $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{1}{2}$ ∴ 22#/ft

FRAMEWORK (117 ft)(22#/ft) = 2.57k

TRANSVERSE BRACES

(5 BRACES)(4 ANGLES/BRACE)(1 ft)(11#/ft/ANGLE) = .220k

(5 BRACES)(13#/ft of PLATE)(1 ft) = .068k

.288k

DETERMINE WEIGHT OF CONCRETE APRON

(1 ft)(1.42 ft)(23 ft)(150#/ft³) = 4.90k

SLIDING FORCE = (7.76)(.6) = 4.65k

2. EFFECT OF KEY AT TOE (PASSIVE RESISTANCE)

LIMESTONE & DOLOSTONE $\phi = 55°$ $S_s = 1000$ psi

$$P_p = W \tan(\phi + \alpha) + \frac{SA}{\cos\alpha(1 - \tan\phi \tan\alpha)}$$

$$= (1.29k)(\tan(72.5)) + \frac{(1 ksi)(9.91 ft^2)\left(\frac{1 ft^2}{144 in^2}\right)}{\cos(\alpha)(1 - \tan 72.5)}$$

$P_p = 4.16k$

GA-17 (4/70) NEW YORK STATE DEPARTMENT OF ENVIRONMENTAL CONSERVATION

PROJECT GRID

JOB	SHEET NO.	CHECKED BY	DATE
Lock No 3	3		
SUBJECT: Stability Analysis (Cont)		COMPUTED BY: RLW	DATE: 3/13/79

3 Effect of Key At Toe (Sliding Resistance)

Volume of Block = (5.5)(11.5)(1) = 63.25 ft^3

$F_{sliding}$ = (63.25 ft^3)(.15 k/ft^3)(.6) = 5.69 K

Total Force To Be Added As A Passive Resistance

4.65 + 4.16 + 5.69 = 14.50 K

P_p = 17.61 K This Has To Be Modeled As A Passive Resistance Acting At Point of Intersection of Masonry

$P_p = \frac{1}{2}\gamma H^2 K_p \Rightarrow K_p = \frac{P_p}{\frac{1}{2}\gamma H^2} = \frac{14.50 K}{\frac{1}{2}(.055)(37.5)^2} = .38$

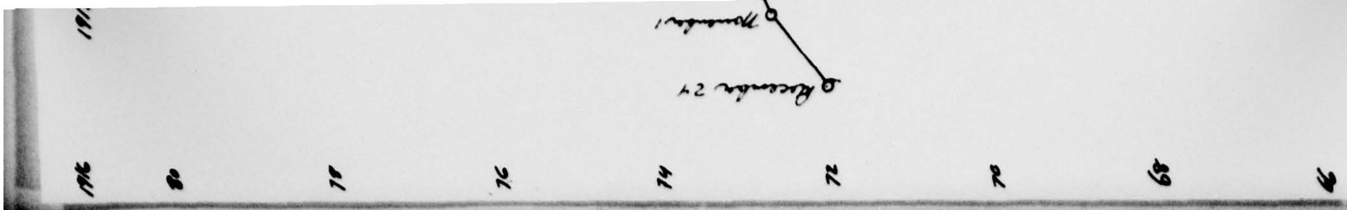
γ = .055 - Already Input Into Program

H = 37.5' - To Get Resultant To Act At Proper Point

$\therefore K_p$ = .38 γ = .055 H_p = 37.5

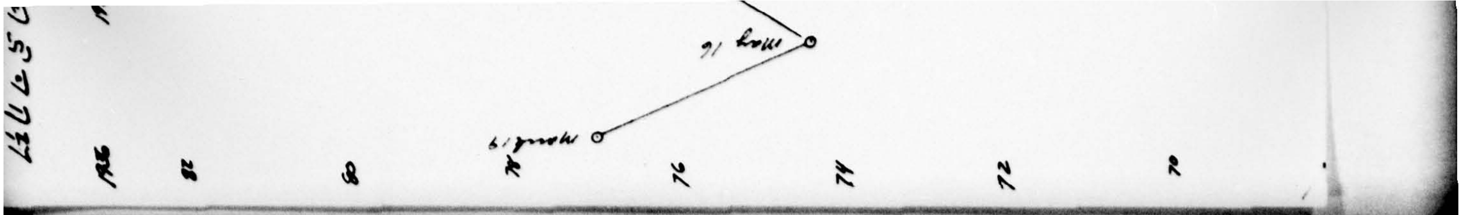
INPUT TO STABILITY ANALYSIS PROGRAM

INPUT ENTRY	PROGRAM No.
Unit Weight of Dam (K/ft^3)	0
Area of Segment No. 1 (ft^2)	1
Distance from Center of Gravity of Segment No. 1 to Downstream Toe (ft)	2
Area of Segment No. 2 (ft^2)	3
Distance from Center of Gravity of Segment No. 2 to Downstream Toe (ft)	4
Area of Segment No. 3 (ft^2)	5
Distance from Center of Gravity of Segment No. 3 to Downstream Tow (ft)	6
Base Width of Dam (Total) (ft)	7
Height of Dam (ft)	8
Ice Loading (K/L ft.)	9
Coefficient of Sliding	10
Unit Weight of Soil (K/ft^3)	11
Active Soil Coefficient - Ka	12
Passive Soil Coefficient - Kp	13
Height of Water over Top of Dam or Spillway (ft)	14
Height of Soil for Active Pressure (ft)	15
Height of Soil for Passive Pressure (ft)	16
Height of Water in Tailrace Channel (ft)	17
Weight of Water (K/ft^3)	18
Area of Segment No. 4 (ft^2)	19
Distance from Center of Gravity of Segment No. 4 to Downstream Toe (ft)	20
Height of Ice Load or Active Water (ft)	46



Normal Conditions		Ice Load of 5000 psf	
0.15	RCL 1	0.15	RCL 1
76.5 76.5	RCL 2	76.5 76.5	RCL 2
12. 12.	RCL 3	12. 12.	RCL 3
84.8 84.8	RCL 4	84.8 84.8	RCL 4
6.8 6.8	RCL 5	6.8 6.8	RCL 5
36.5 36.5	RCL 6	36.5 36.5	RCL 6
3.4 3.4	RCL 7	3.4 3.4	RCL 7
18. 18.	RCL 8	18. 18.	RCL 8
18.75 18.75	RCL 9	18.75 18.75	RCL 9
0. 0.	RCL 10	5. 5.	RCL 10
0.6 0.6	RCL 11	0.6 0.6	RCL 11
0.055 0.055	RCL 12	0.055 0.055	RCL 12
0.3 0.3	RCL 13	0.3 0.3	RCL 13
0.38 0.38	RCL 14	0.38 0.38	RCL 14
0. 0.	RCL 15	0. 0.	RCL 15
14.8 14.8	RCL 16	14.8 14.8	RCL 16
37.5 37.5	RCL 17	37.5 37.5	RCL 17
0. 0.	RCL 18	0. 0.	RCL 18
0.0624 0.0624	RCL 19	0.0624 0.0624	RCL 19
19.6 19.6	RCL 20	19.6 19.6	RCL 20
1.5 1.5	RCL 46	1.5 1.5	RCL 46
18.75		18.75	

Normal Conditions		Ice Load of 5000 psf
2.114081436	**F.S. Overturning**	1.448057541
10.28454668		6.030622769
2.18720134	**F.S. Sliding**	1.571983559



	Largest Storm On Record		1/2 PMF	
	0.15	RCL 1	0.15	RCL 1
	76.5 76.5	RCL 2	76.5 76.5	RCL 2
	12. 12.	RCL 3	12. 12.	RCL 3
	84.8 84.8	RCL 4	84.8 84.8	RCL 4
	6.8 6.8	RCL 5	6.8 6.8	RCL 5
	36.5 36.5	RCL 6	36.5 36.5	RCL 6
	3.4 3.4	RCL 7	3.4 3.4	RCL 7
	18. 18.	RCL 8	18. 18.	RCL 8
	18.75 18.75	RCL 9	18.75 18.75	RCL 9
	0. 0.	RCL 10	0. 0.	RCL 10
	0.6 0.6	RCL 11	0.6 0.6	RCL 11
	0.055 0.055	RCL 12	0.055 0.055	RCL 12
	0.3 0.3	RCL 13	0.3 0.3	RCL 13
	0.38 0.38	RCL 14	0.38 0.38	RCL 14
	15.3 15.3	RCL 15	21.2 21.2	RCL 15
	14.8 14.8	RCL 16	14.8 14.8	RCL 16
	37.5 37.5	RCL 17	37.5 37.5	RCL 17
	23. 23.	RCL 18	26. 26.	RCL 18
	0.0624 0.0624	RCL 19	0.0624 0.0624	RCL 19
	19.6 19.6	RCL 20	19.6 19.6	RCL 20
	1.5 1.5	RCL 46	1.5 1.5	RCL 46
	18.75		18.75	
F.S.-Overturning	1.00[illegible]392562		.920[illegible]042241	
	9.08[illegible]79[illegible]665		12.0[illegible]32223	
F.S.-Sliding	1.028220468		.8815[illegible]9513	

APPENDIX F

REFERENCES

REFERENCES

1) US Army Corps of Engineers; New York District; Upper Hudson and Mohawk River Basins Hydrologic Flood Routing Models, October 1976.

2) US Geological Survey; Compilation of Records of Surface Waters of the United States, Part 1-B North Atlantic Slope Basins;

   Water Supply Paper 1302 (Through September 1950), 1960.

   Water Supply Paper 1722 (October 1950 to September 1960), 1964.

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

4) E.E. Seelye; Design, 3rd edition, John Wiley and Sons, Inc., 1960.

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

6) *U.S. Department of the Interior, Bureau of Reclamation;* Design of Small Dams, 2nd edition (rev. reprint), 1977.

APPENDIX G

CORPS OF ENGINEERS
GUIDELINES

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

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

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

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

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

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

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

4.4.3. Embankment Dams.

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

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

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

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

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

i_c = Critical gradient

i = Design gradient

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

H_c = The critical uplift

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

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

γ_w = The unit weight of water

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

4.4.4. Concrete Dams and Appurtenant Structures.

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

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

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

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

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

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

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

where

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

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

V = Summation of vertical forces (including uplift)

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

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

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

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

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

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

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

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

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

P_p = passive resistance of rock wedge

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

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

α = angle between inclined failure plane and horizontal

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

A = area of inclined plane of resistance

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

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

where

D = Thickness of the rock strut

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

$$S_{s\text{-}f} = \frac{R_R}{H} \qquad (8)$$

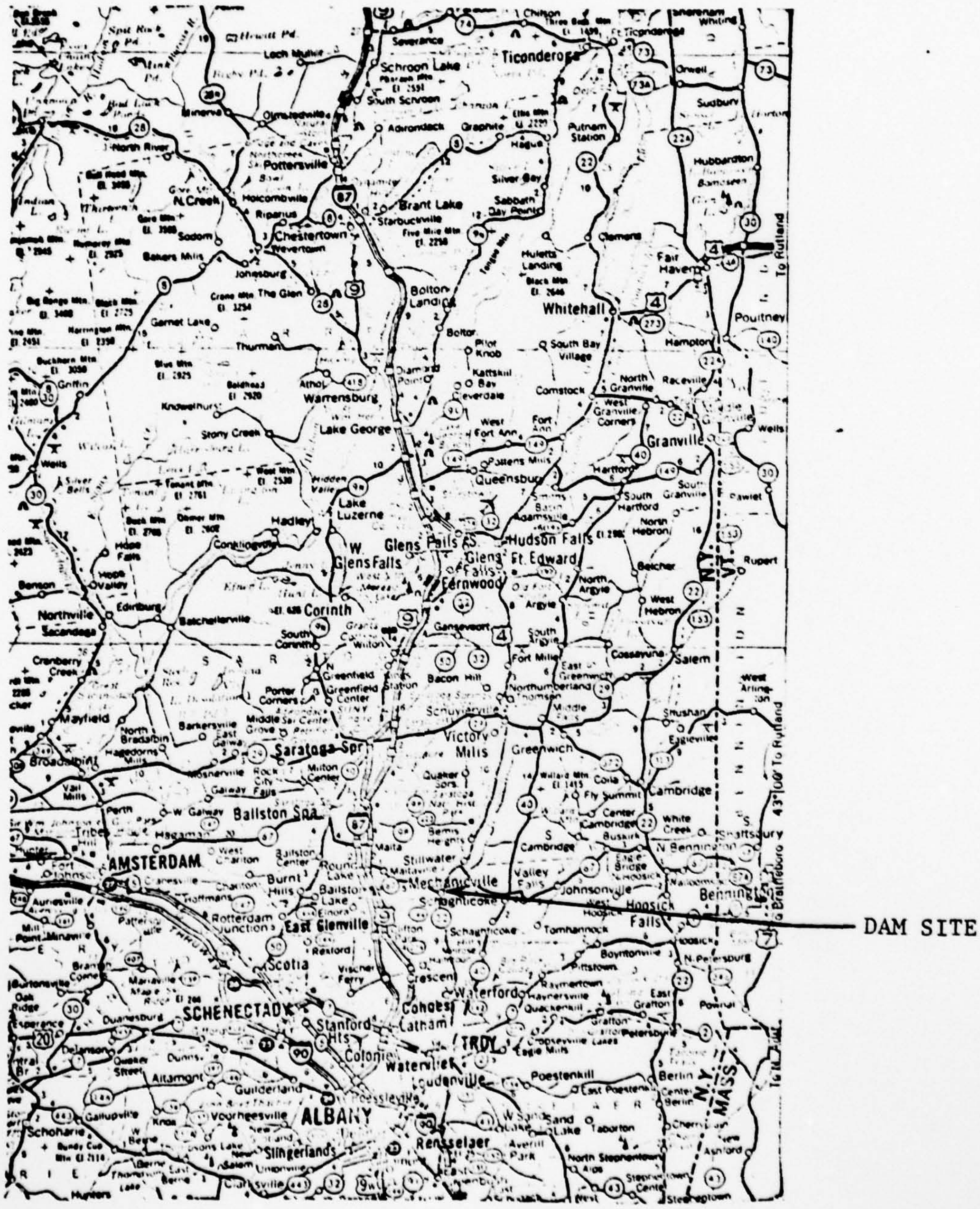
When the downstream passive wedge contributes to the sliding resistance, the shear fruction safety factor formula becomes:

$$S_{s\text{-}f} = \frac{R_R + P_p}{H} \qquad (9)$$

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

APPENDIX H

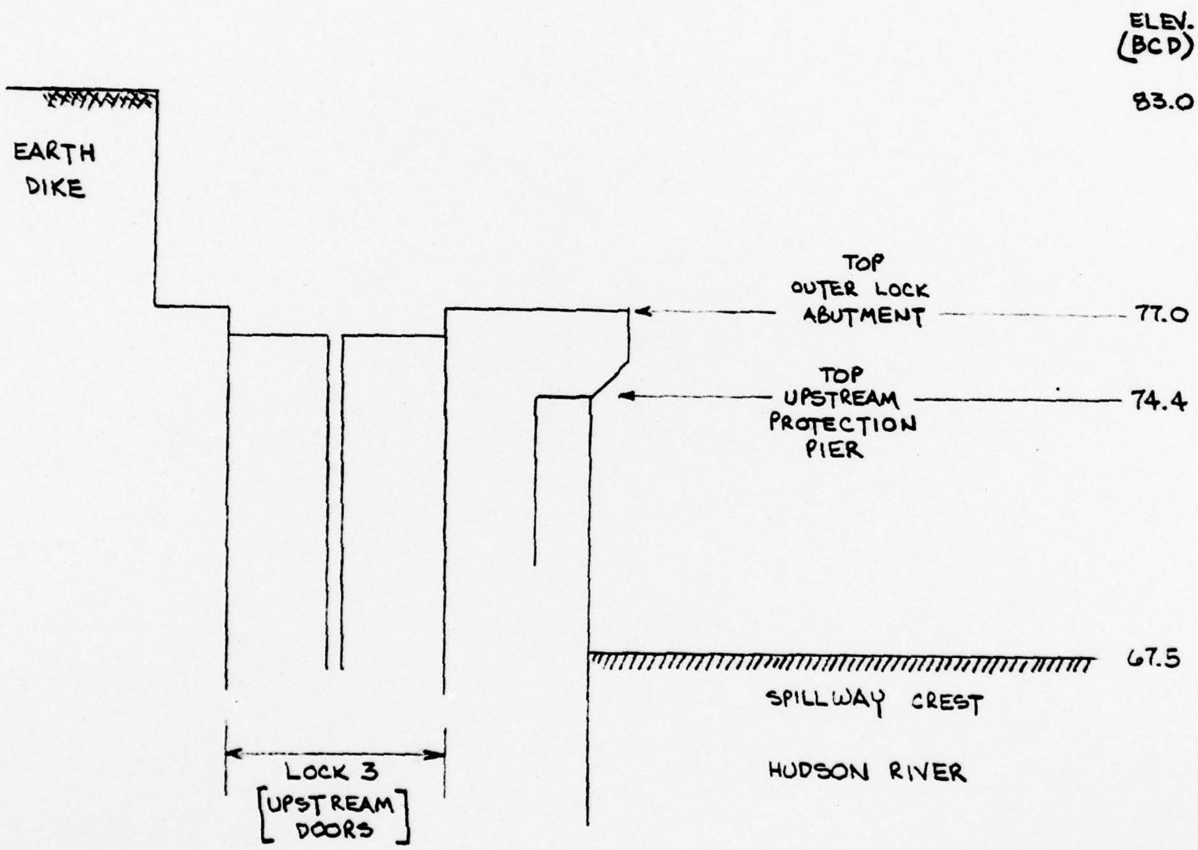
DRAWINGS



VICINITY MAP

LOCK 3 DAM @ MECHANICVILLE

LOCK 3 DAM @ MECHANICVILLE



DATUM:

NYS-DOT BARGE CANAL DATUM (BCD) USGS

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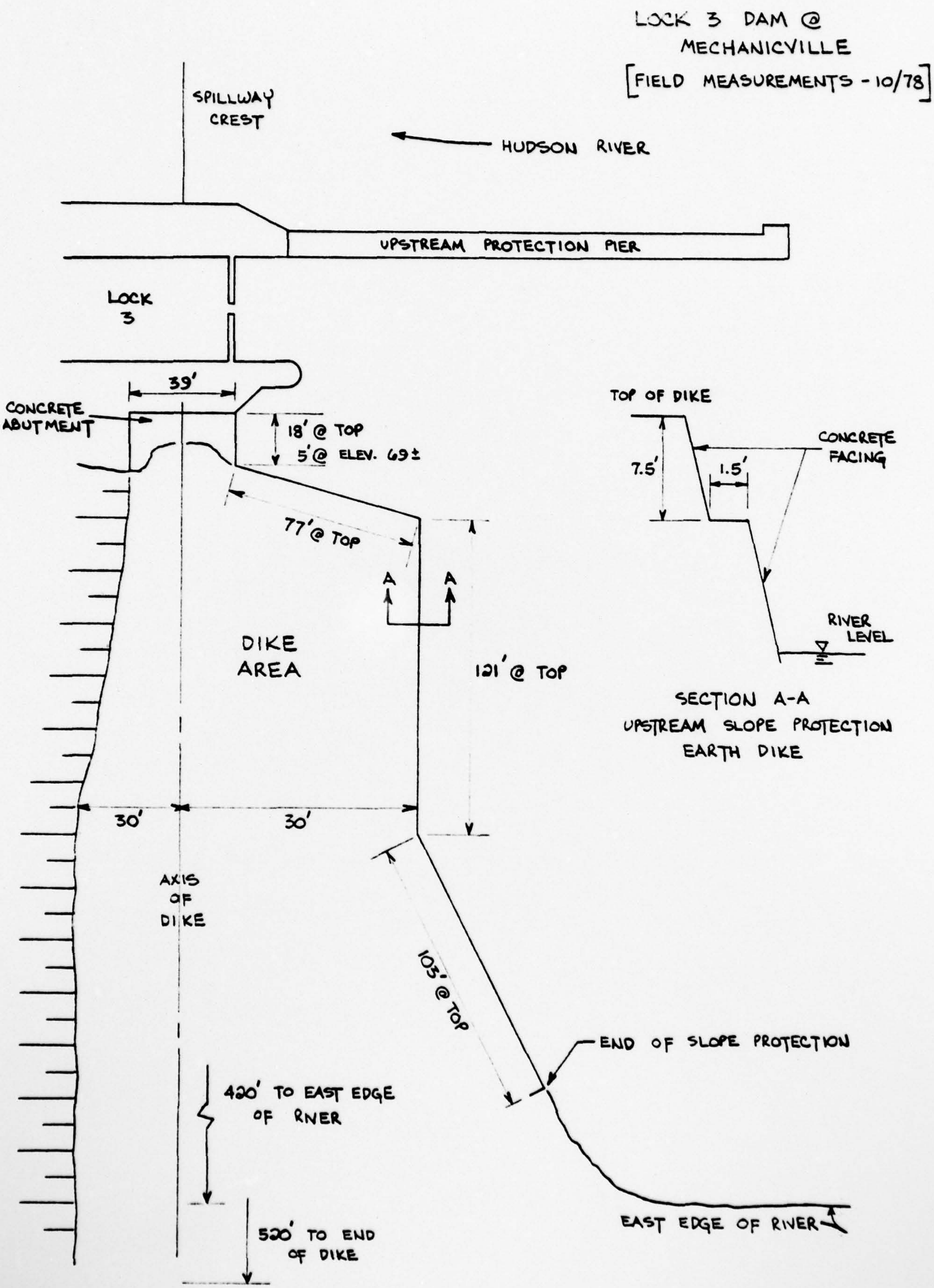
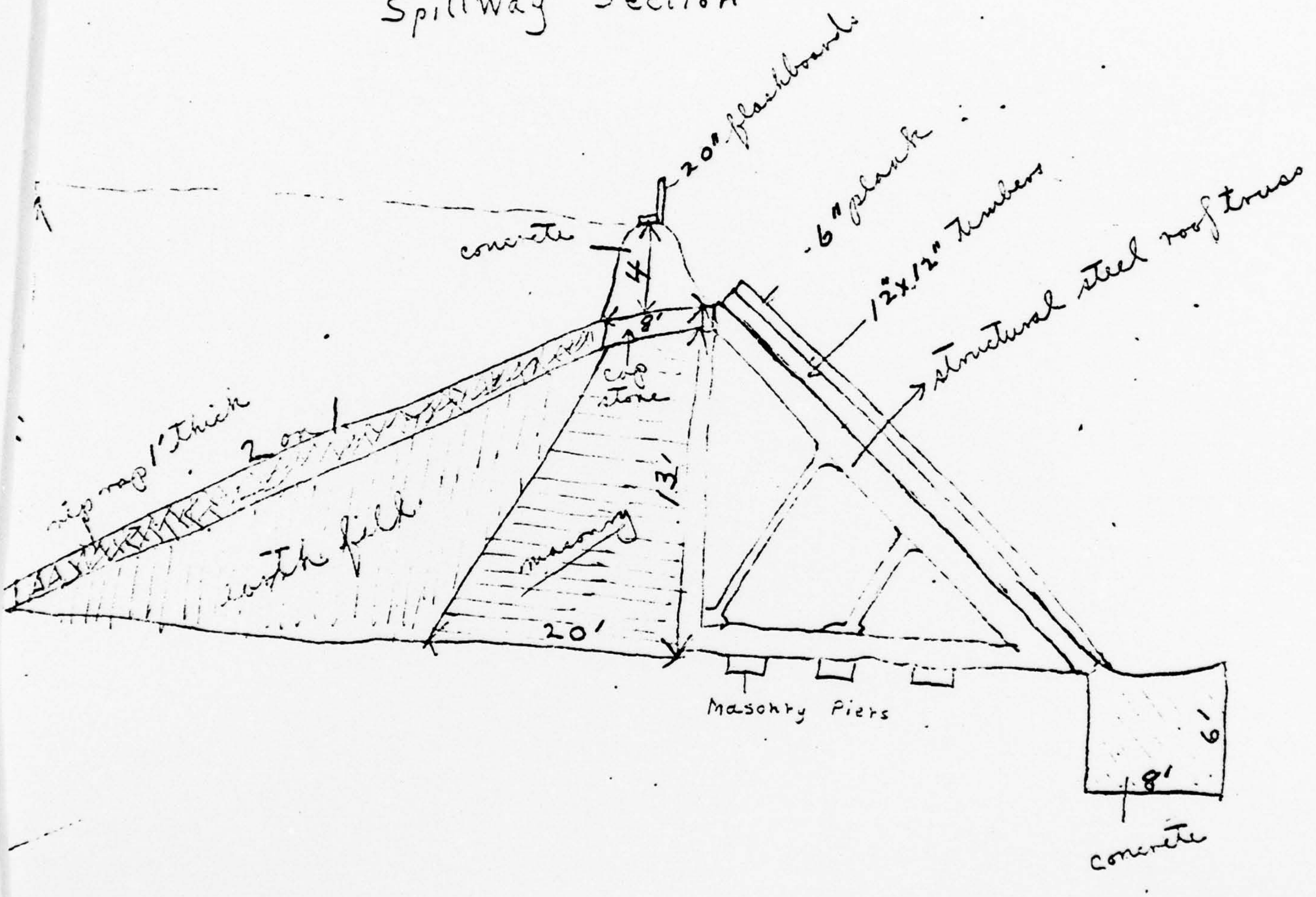

LOCK 3 DAM @
MECHANICVILLE
[FIELD MEASUREMENTS - 10/78]
SPILLWAY CREST
HUDSON RIVER
UPSTREAM PROTECTION PIER
LOCK 3
39'
CONCRETE ABUTMENT
18' @ TOP
5' @ ELEV. 69±
77' @ TOP
A
A
DIKE AREA
121' @ TOP
30'
30'
AXIS OF DIKE
103' @ TOP
END OF SLOPE PROTECTION
420' TO EAST EDGE OF RIVER
520' TO END OF DIKE
EAST EDGE OF RIVER
TOP OF DIKE
CONCRETE FACING
7.5'
1.5'
RIVER LEVEL
SECTION A-A
UPSTREAM SLOPE PROTECTION
EARTH DIKE

(In the space below, make one sketch showing the form and dimensions of a cross section through the spillway or waste-weir of this dam, and a second sketch showing the same information for a cross section through the other portion of the dam. Show particularly the greatest height of the dam above the stream bed, its thickness at the top, and thickness at the bottom, as nearly as you can learn.)

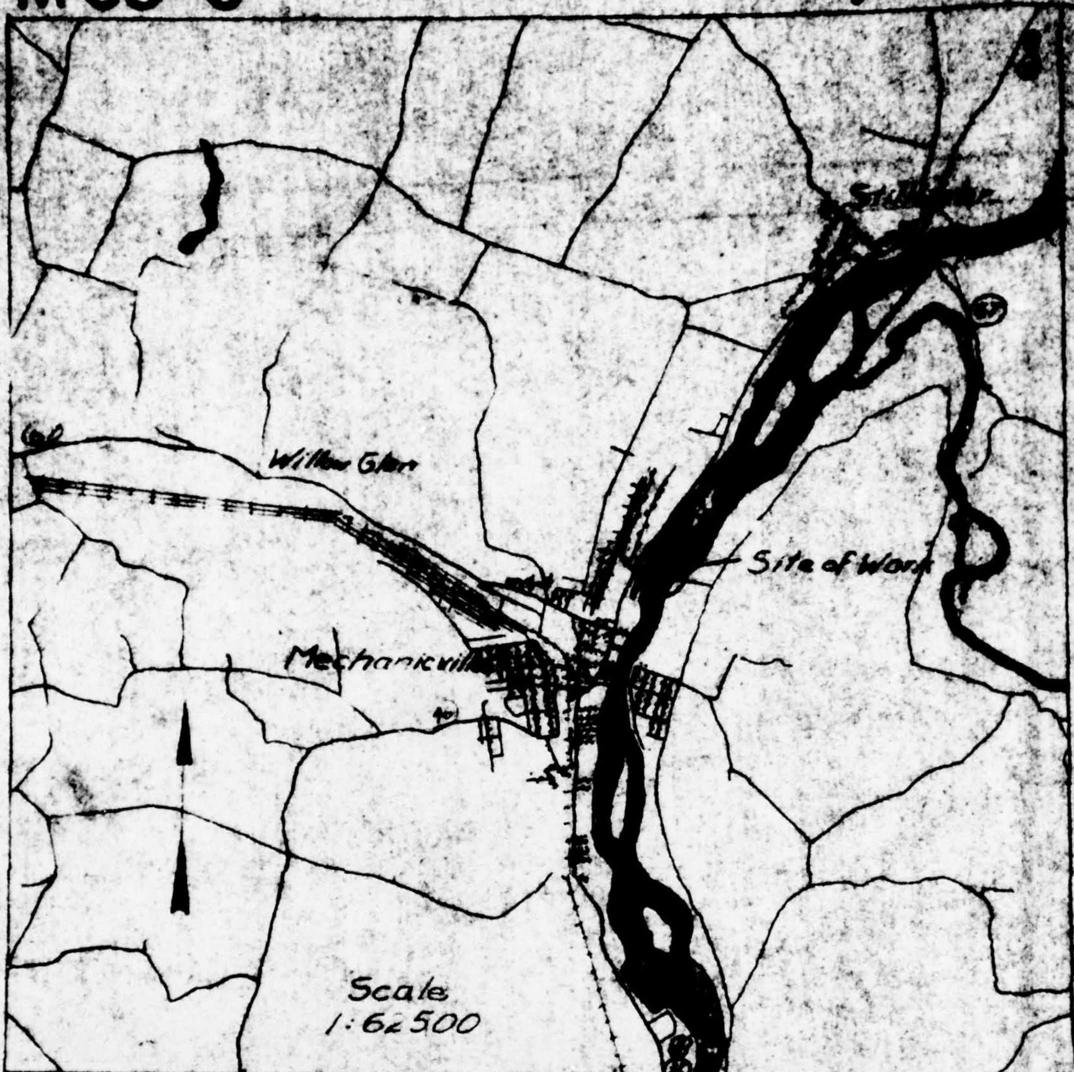
LOCK 3 DAM @ MECHANICVILLE, N.Y. 6/24/16

Spillway Section



M65-6

1



Location: The project is located at Lock 3-C on the Hudson River Mechanicville, N.Y. East of the West Virginia Paper Co.

ESTIMATE OF QUANTITIES

NO.	ITEM	UNIT	N
68A	Concrete Cylinder Curing Box	Ea.	
6EX	Engineers Office, Type A	L.S.	
13AX	3" Steel Conduit	L.F.	
18	Class A Concrete for Structures	C.Y.	
25F	Steel Fabric Reinforcement	S.Y.	
28X	Bar Reinforcement for Structures	Lb.	
81AX	Removing Existing Superstructure	L.S.	
201X	Chipping and Removing Conc. and Stone Masonry	C.Y.	
202	Drilling Holes	L.F.	
208	Precast Concrete Slab	S.F.	

Engineer Dist No. 1

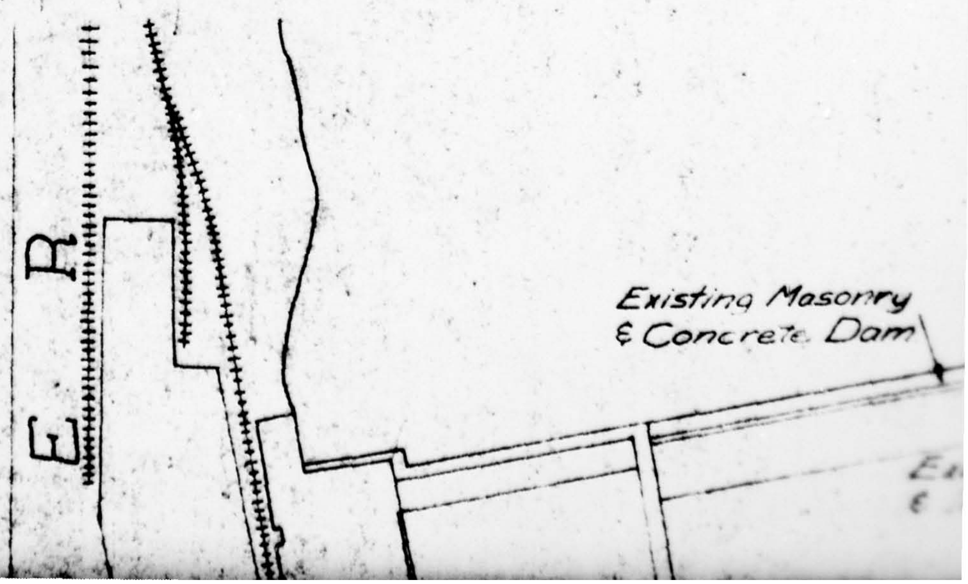
165

2

STATE OF
DEPARTMENT OF
CHAPTER 542 LAW
CONTRACT
FOR M 65
REHABILITATION OF DAM
CHAMPLAIN C
AT
MECHANICVILL
SARATOGA AND RENSSEL
NEW YORK
SCALES AS IND
SHEETS 1 TO

TIES

	NEAT	PROPOSAL
		1
		Nec.
		275
		410
		520
		11,700
		Nec.
		180
		1500
		15,750

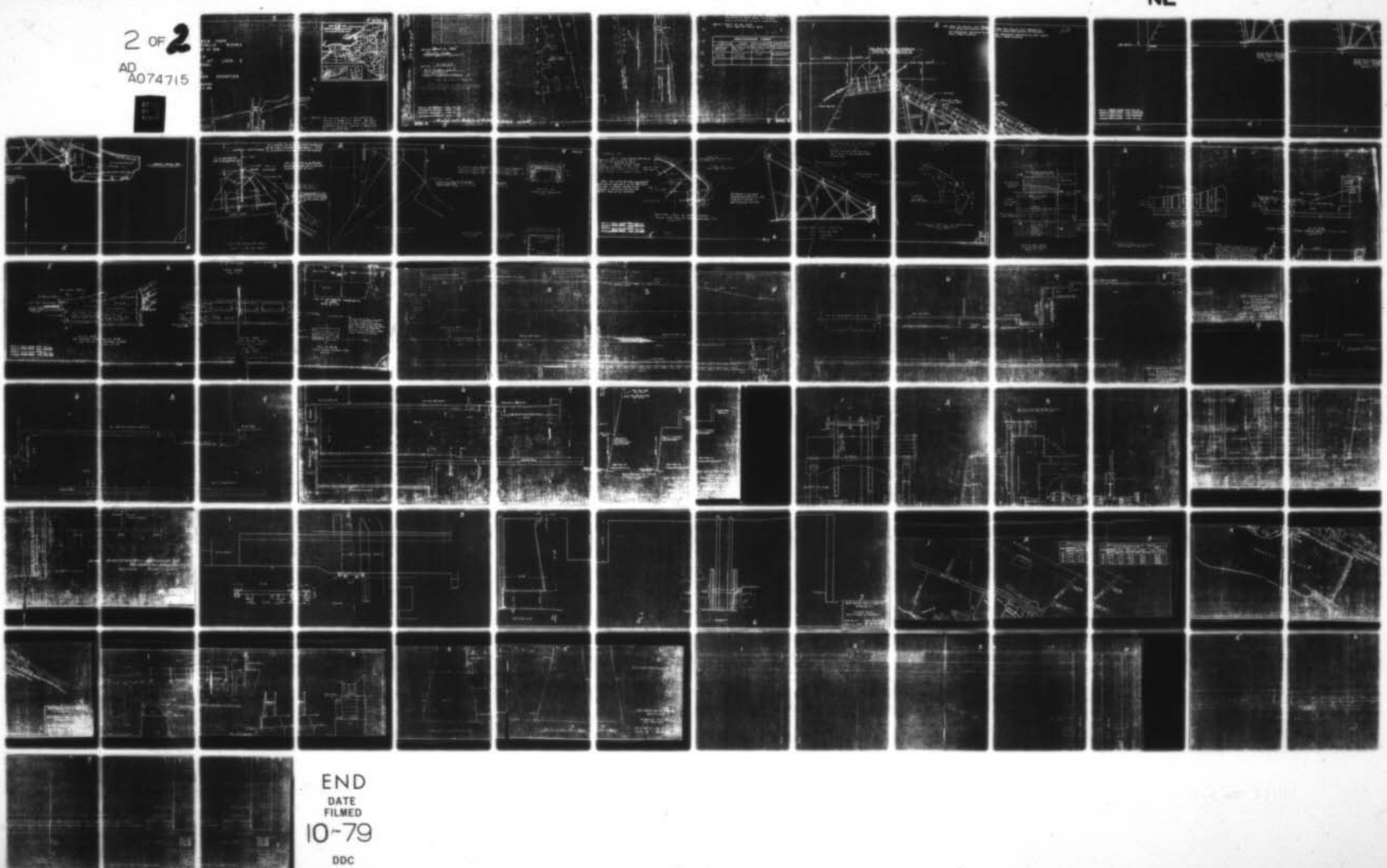


AD-A074 715 NEW YORK STATE DEPT OF ENVIRONMENTAL CONSERVATION ALBANY F/G 13/2
NATIONAL DAM SAFETY PROGRAM. LOCK 3 DAM AT MECHANICVILLE (INVEN--ETC(U)
JUN 79 G KOCH DACW51-79-C-0001

UNCLASSIFIED NL

2 OF 2

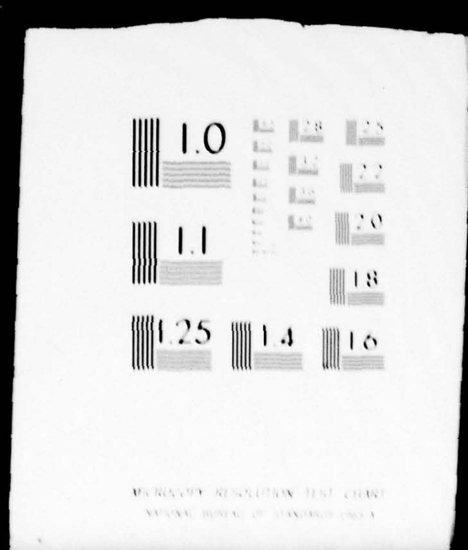
AD A074715



END
DATE
FILMED
10-79
DDC

2 OF 2

AD A074715

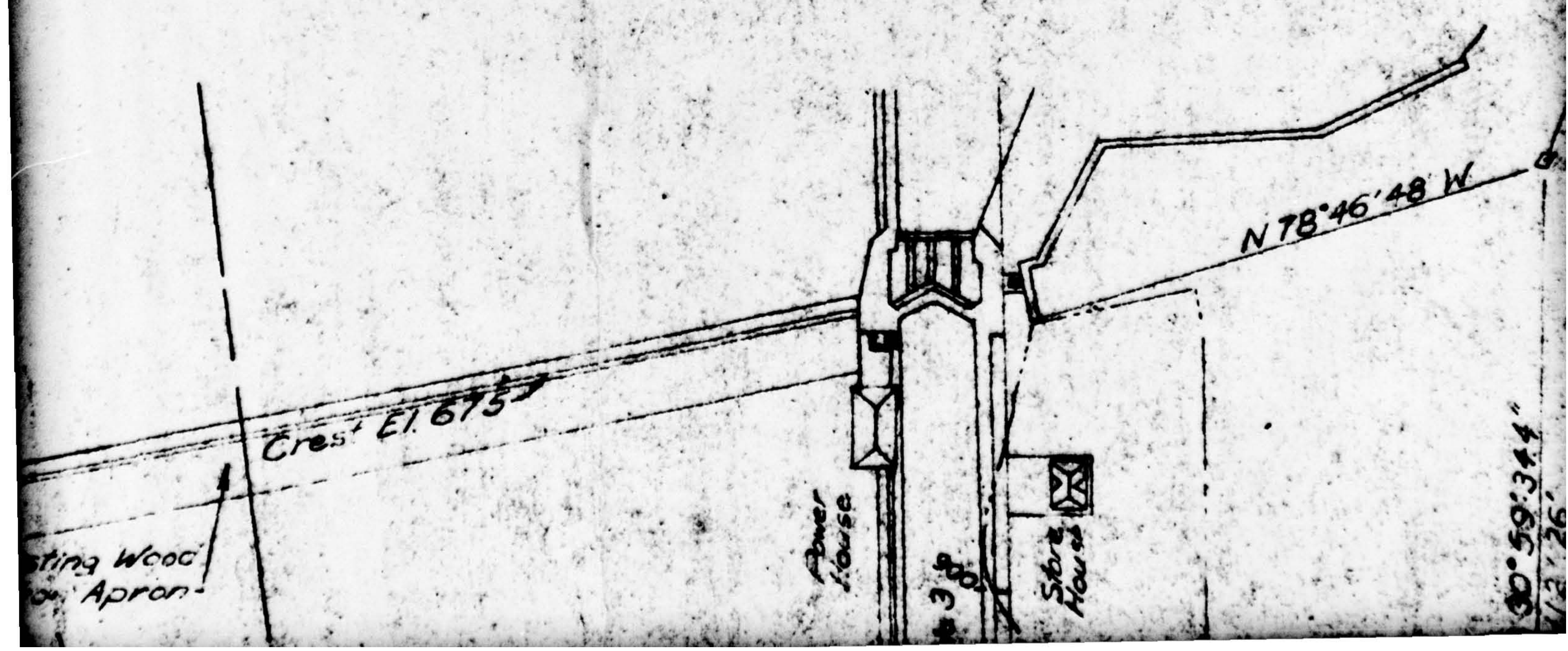


NEW YORK
PUBLIC WORKS
S OF 1939

-6
AT LOCK 3
ANAL

E
AER COUNTIES

CATED
4 INC.



4 M65-6

E

K

51°26'46.6"
162.48'

C

-93.07

I

DATUM: The elevations shown on this contract are referred to Barge Canal Datum. This datum is based on the Greenbush Bench Mark whose established elevation referred to Barge Canal Datum is 14.730 feet.

T

Approved April 13, 1965

Frank J. Fuller

District Engineer, Dist. No. 1

Approved 5-26-65

E. C. [illegible]

Asst. Superintendent

Operations and Maintenance (Highways)

SPECIAL NOTE – COMPOSITION OF CONCRETE ITEMS

In the composition of all [illegible] items in this contract, it is intended that the cement type shall be [illegible] plus an approved air entraining agent added [illegible] of mixing.

MADE BY W. Peterson DATED Nov. [illegible]

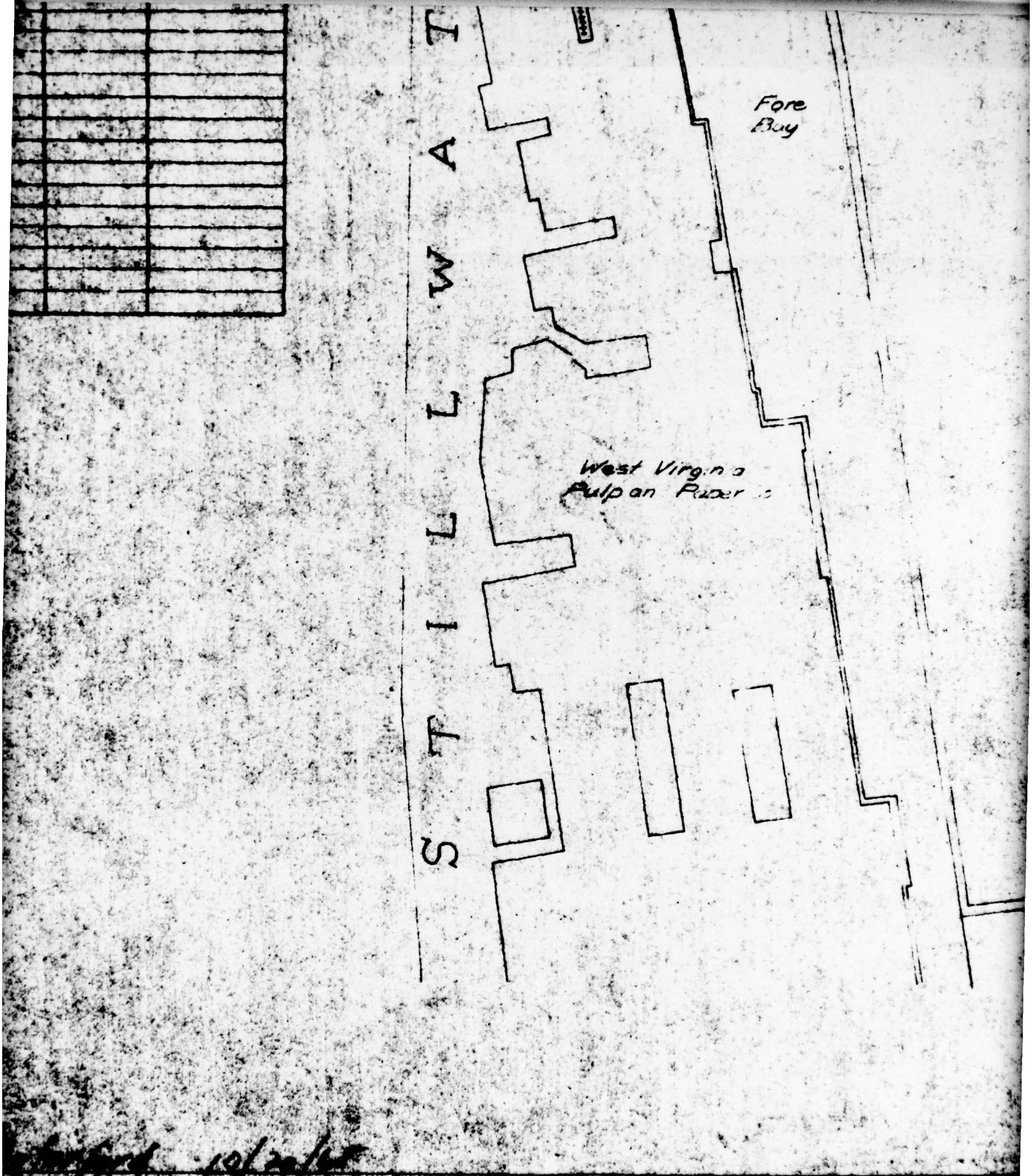
CHECKED BY C. Dupray DATED [illegible]

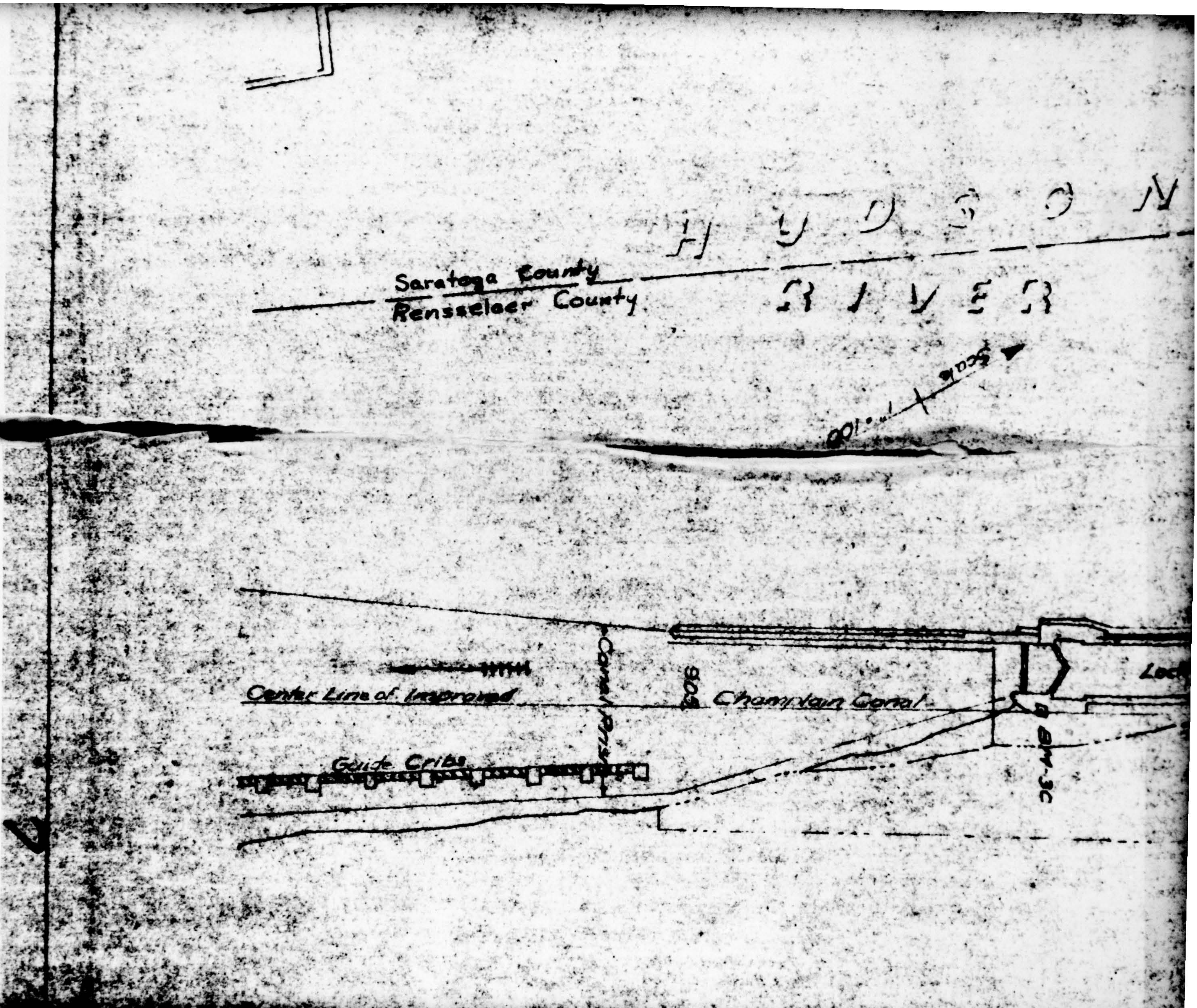
TRACED BY R. Inglis DATED Dec. [illegible]

CHECKED BY W. Peterson DATED [illegible]

M65-6

5


STILLWAT
Fore Bay
West Virgin a
Pulp an Paper


HUDSON
RIVER
Saratoga County
Rensselaer County
Scale 1"=100'
Center Line of Improved Champlain Canal
905
Canal Prism
Guide Cribs
BM-3C

Department of Public Works, Jan. 2, 1962, except as modified by special specifications.

BENCH MARK - 3C Elev. 74.40
Brass Plug - S.E. Thrust Block

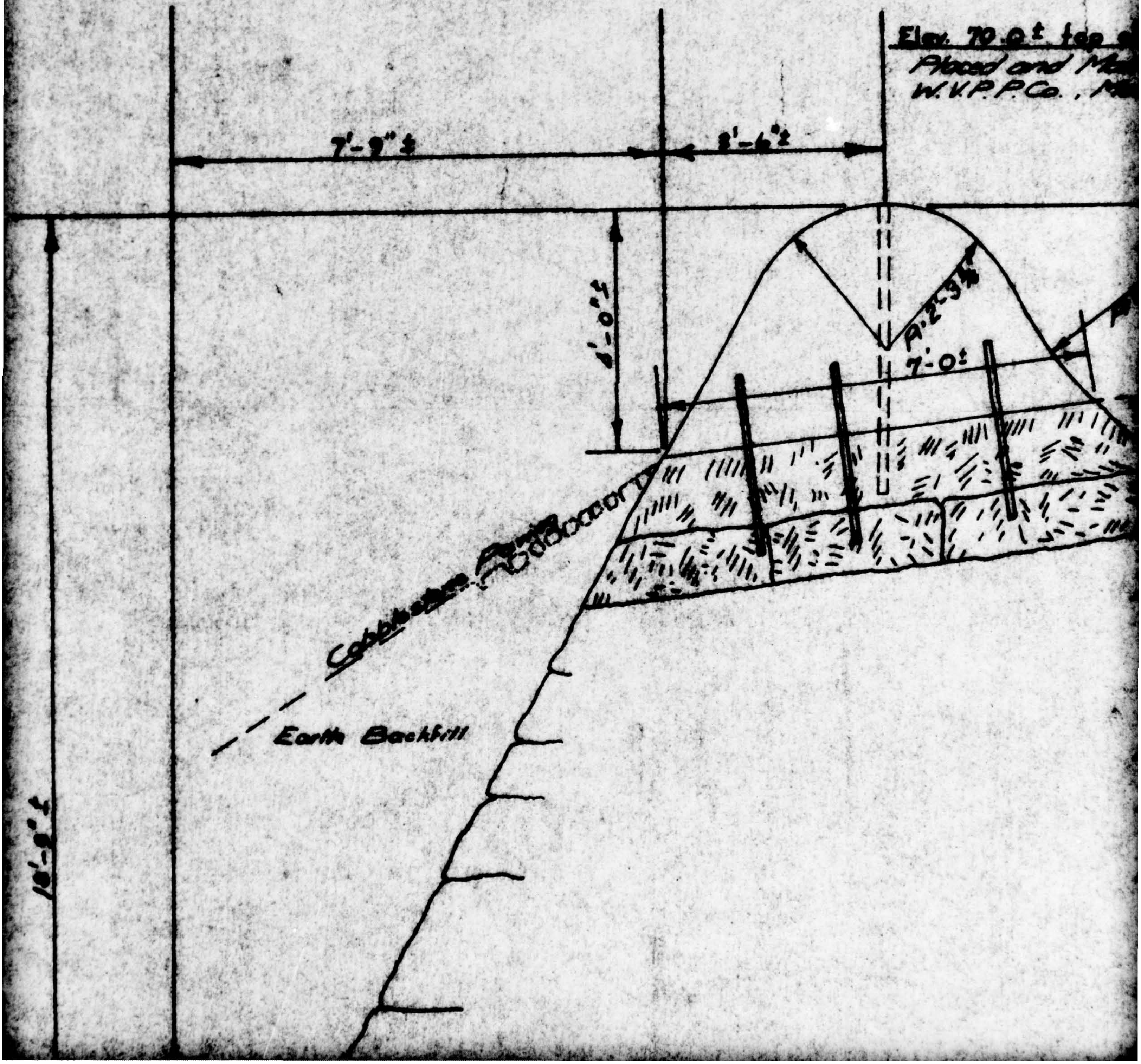
MAINTENANCE TABLE			
STRUCTURE	OWNER	MAINTAINED BY	JURISDICTION
Flashboard Structure	West Virginia Pulp & Paper Co.	West Virginia Pulp & Paper Co	By Agreement
Dam at Lock 3C	State of New York	Canals	By Appropriation

S C H A G H

1

8 M65-6

1

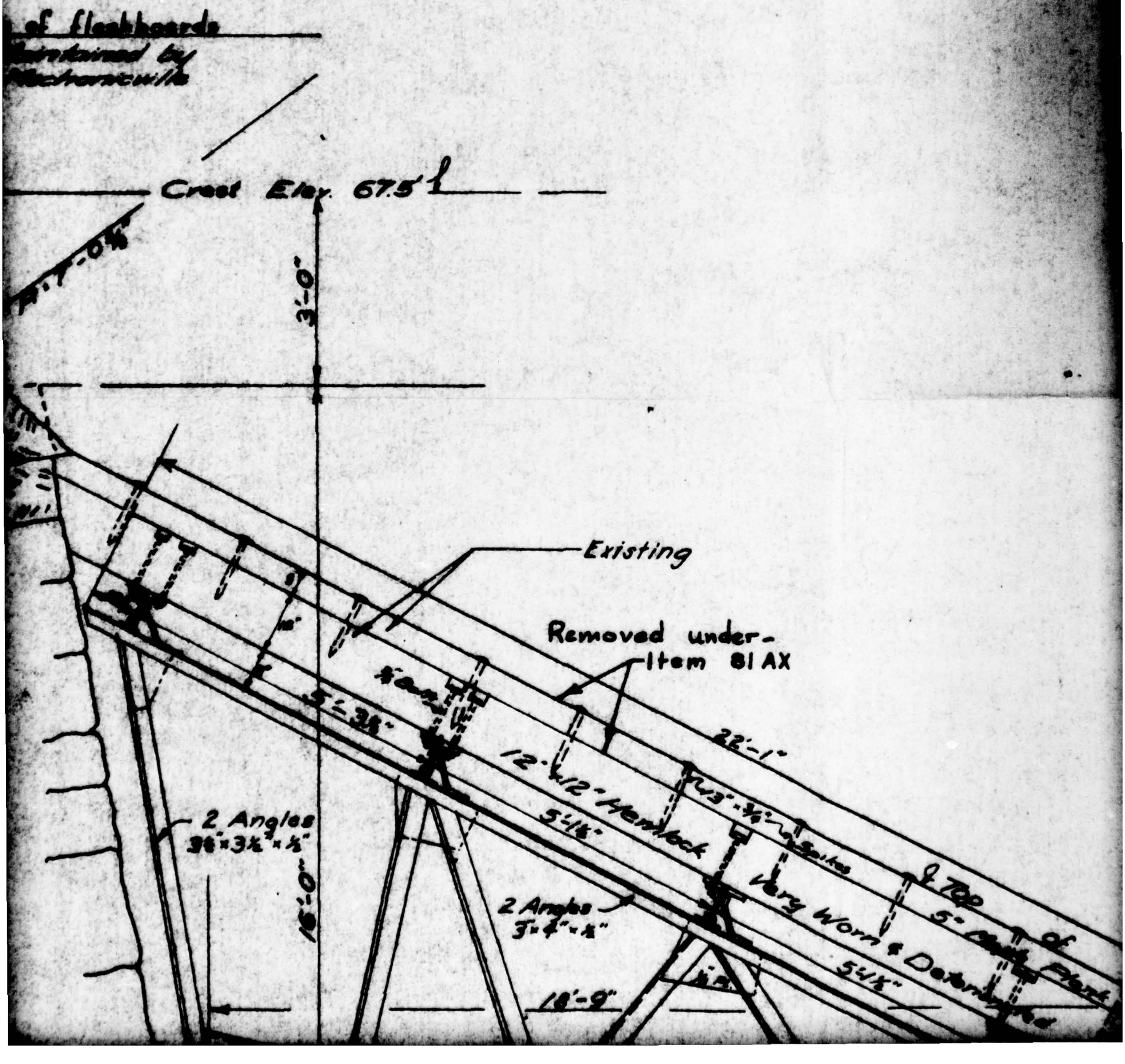


Elev. 70.0± top o
Placed and Ma
W.V.P.P.Co., M
7'-9"±
8'-6"±
4'-0"±
R. 2'-3½"
7'-0±
Cobblestone Paving
Earth Backfill
18'-8"±

2

Note: Upper Pool Elevation 67.5 Requir[...]
N.Y.S. Barge Canal System for Na[...]

30" Flashboards Maintained by W[...]
Pulp & Paper Company.



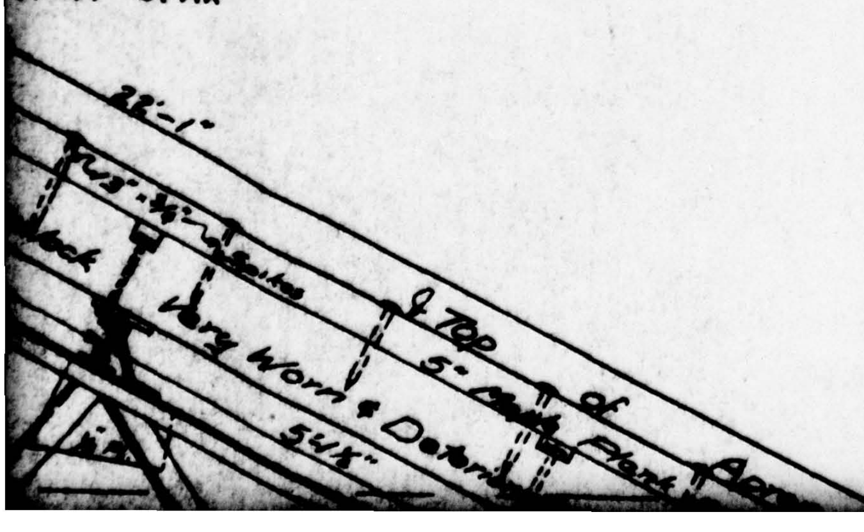
2-A

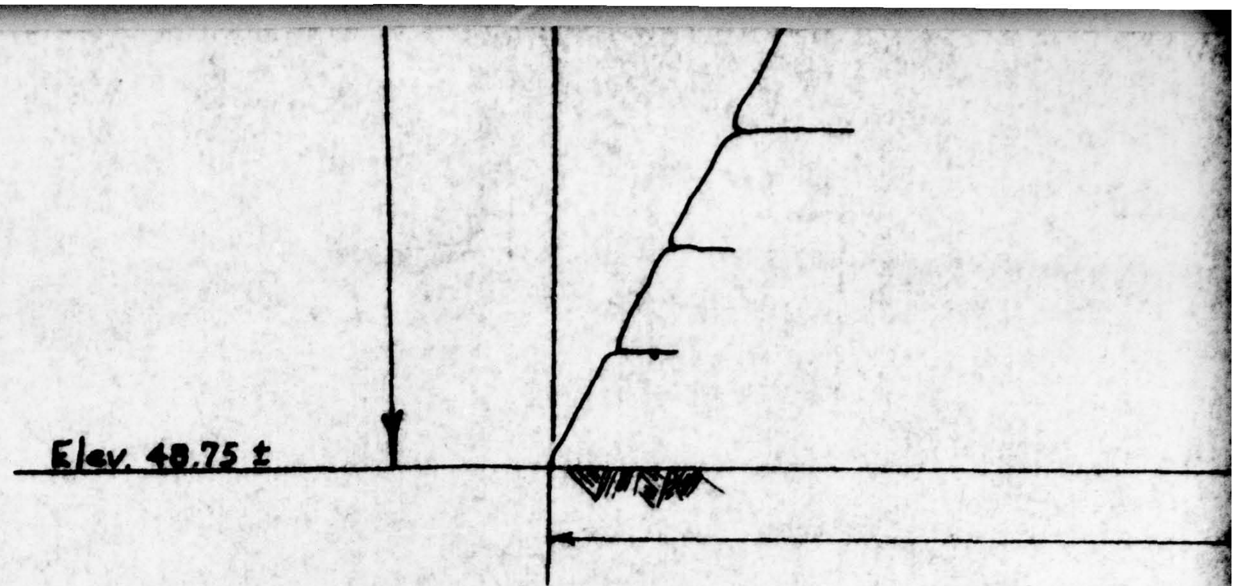
Upper Pool Elevation 67.5 Required by N.Y.S. Barge Canal System for Navigation.

30" Flashboards Maintained by West Virginia Pulp & Paper Company.

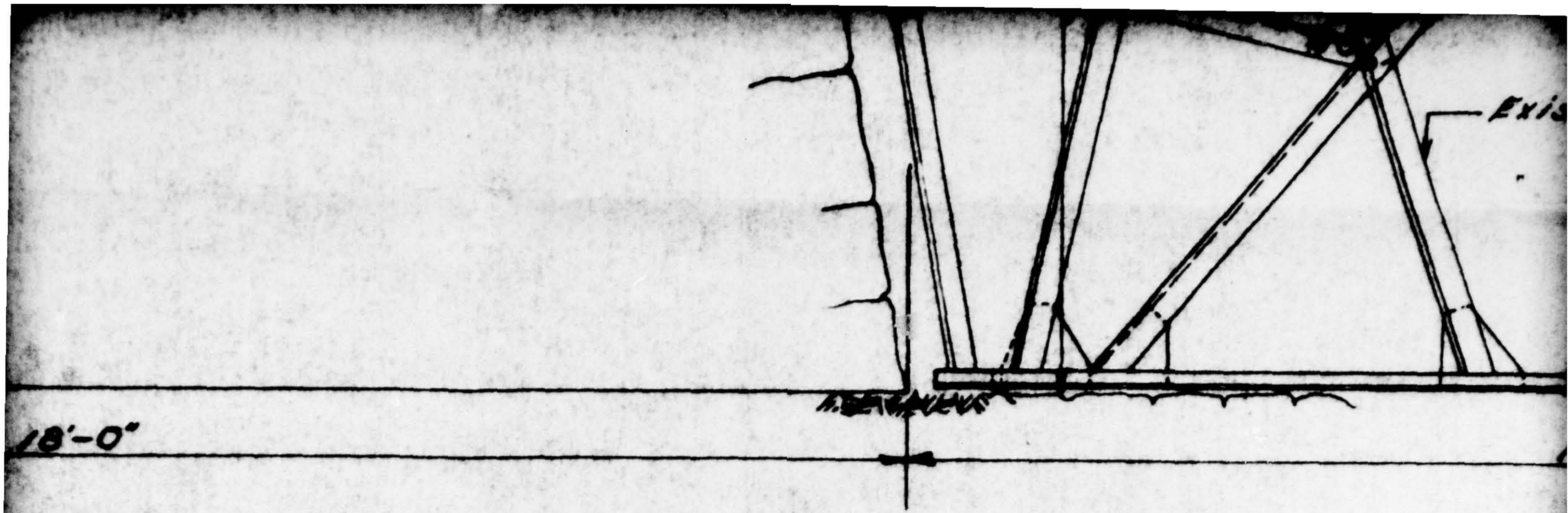
ting

d under-
ltem 81 AX

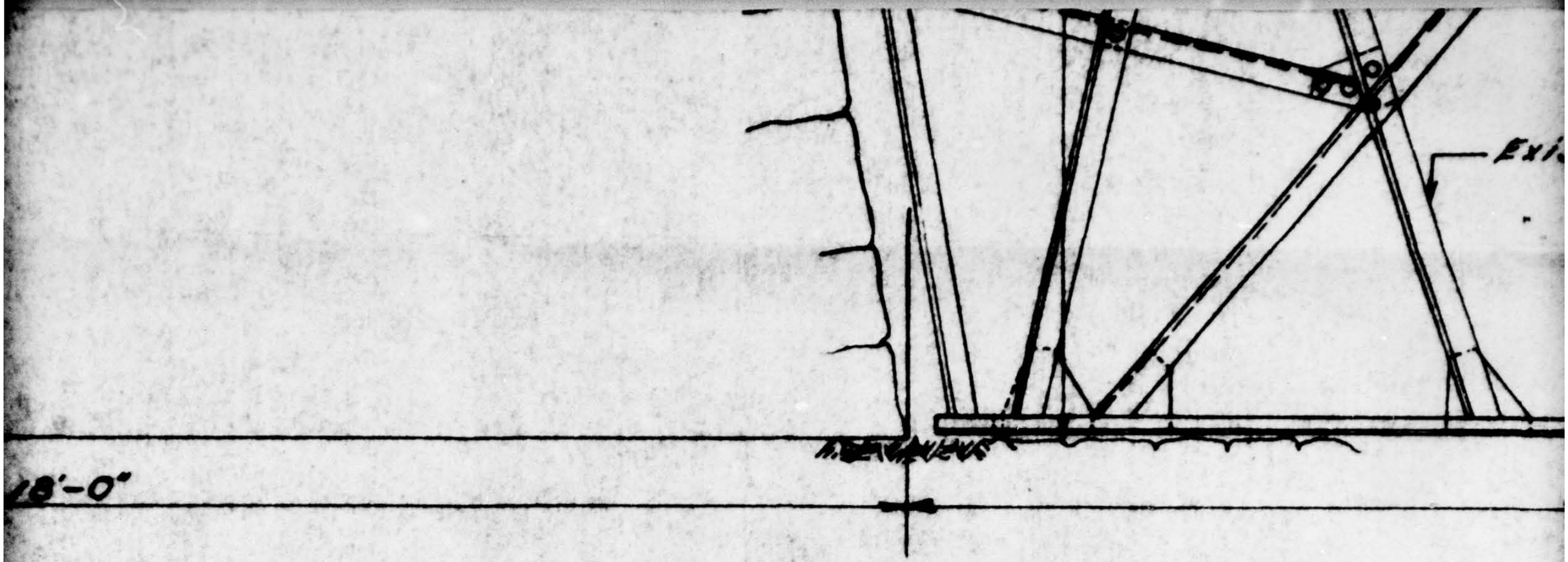




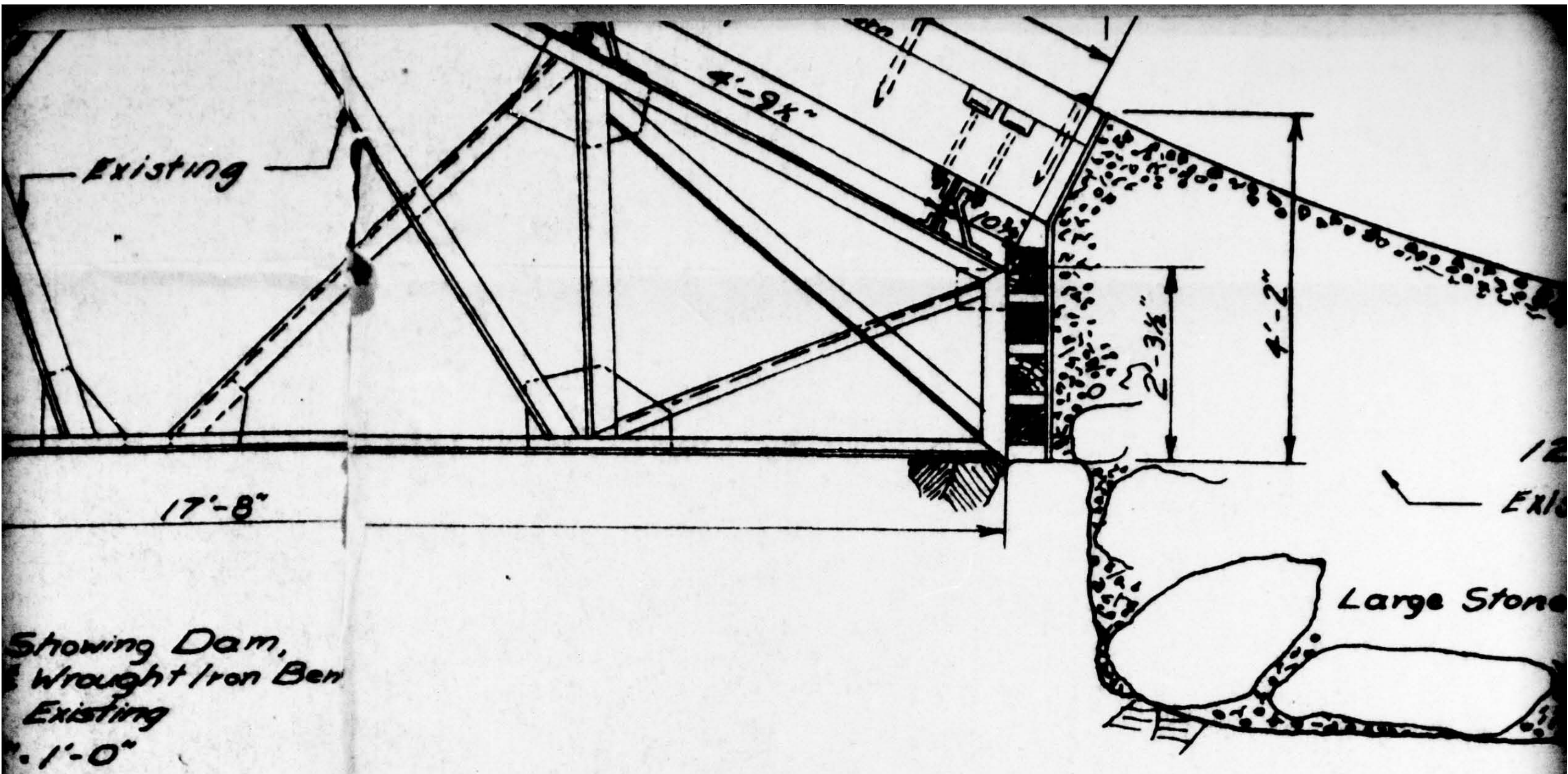
MADE BY [illegible] DATED Sept. '63
CHECKED BY Robert Wood DATED October 1963
TRACED BY Robert Wood DATED October 1963
CHECKED BY [illegible] DATED Oct. '63



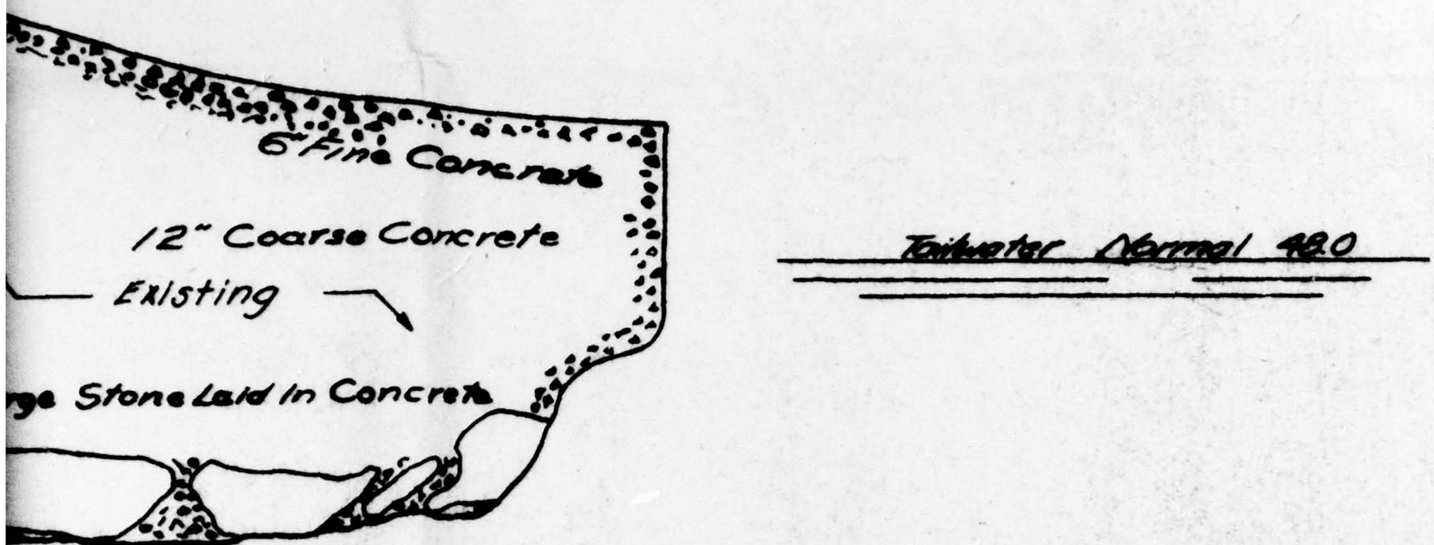
Cross Section Showing
Wooden Apron & Wrought
At Lock 3-C Existing
Scale ½" = 1'-0"



Cross Section Showing
Wooden Apron & Wrough
At Lock 3-C Existing
Scale ⅛" = 1'-0"

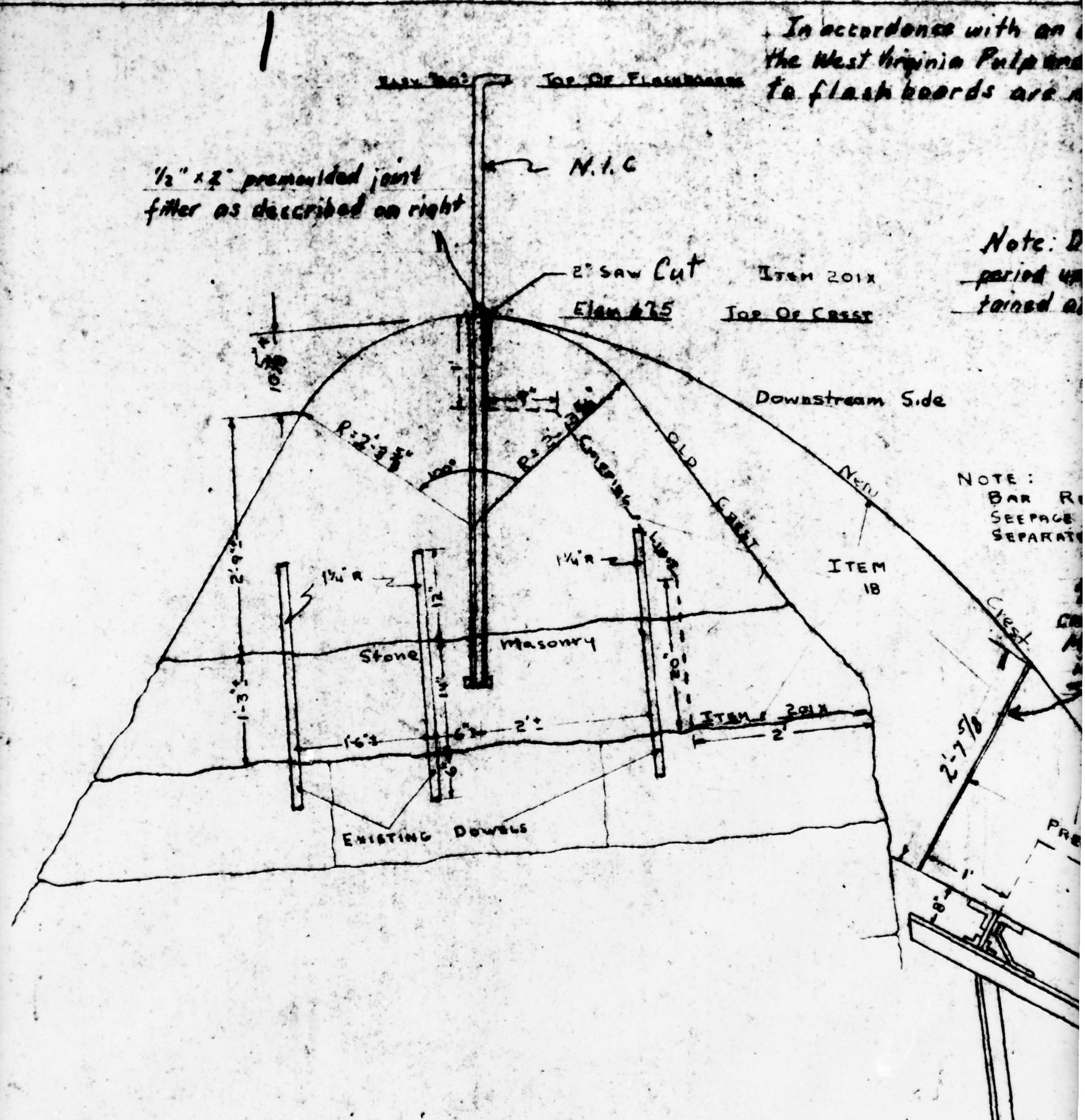


Existing
4'-9½"
10½
4'-2"
2'-3½"
17'-8"
12
Exis
Large Stone
Showing Dam,
Wrought Iron Ben
Existing
". 1'-0"


6" Fine Concrete
12" Coarse Concrete
Existing
Stone Laid In Concrete
Tailwater Normal 48.0

2

6



SECTION VIEW OF CREST

SHOWING OLD AND NEW SECTIONS

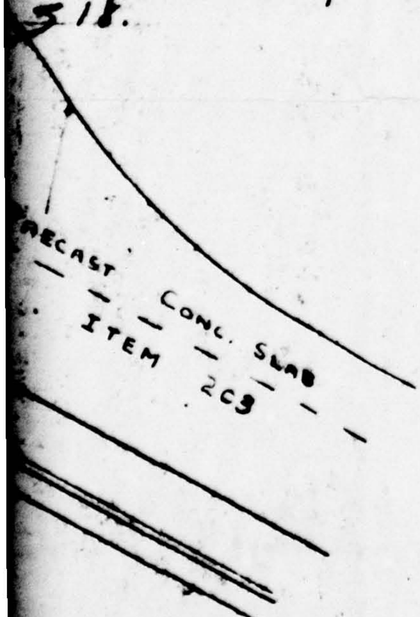
SCALE 3/4" = 1'

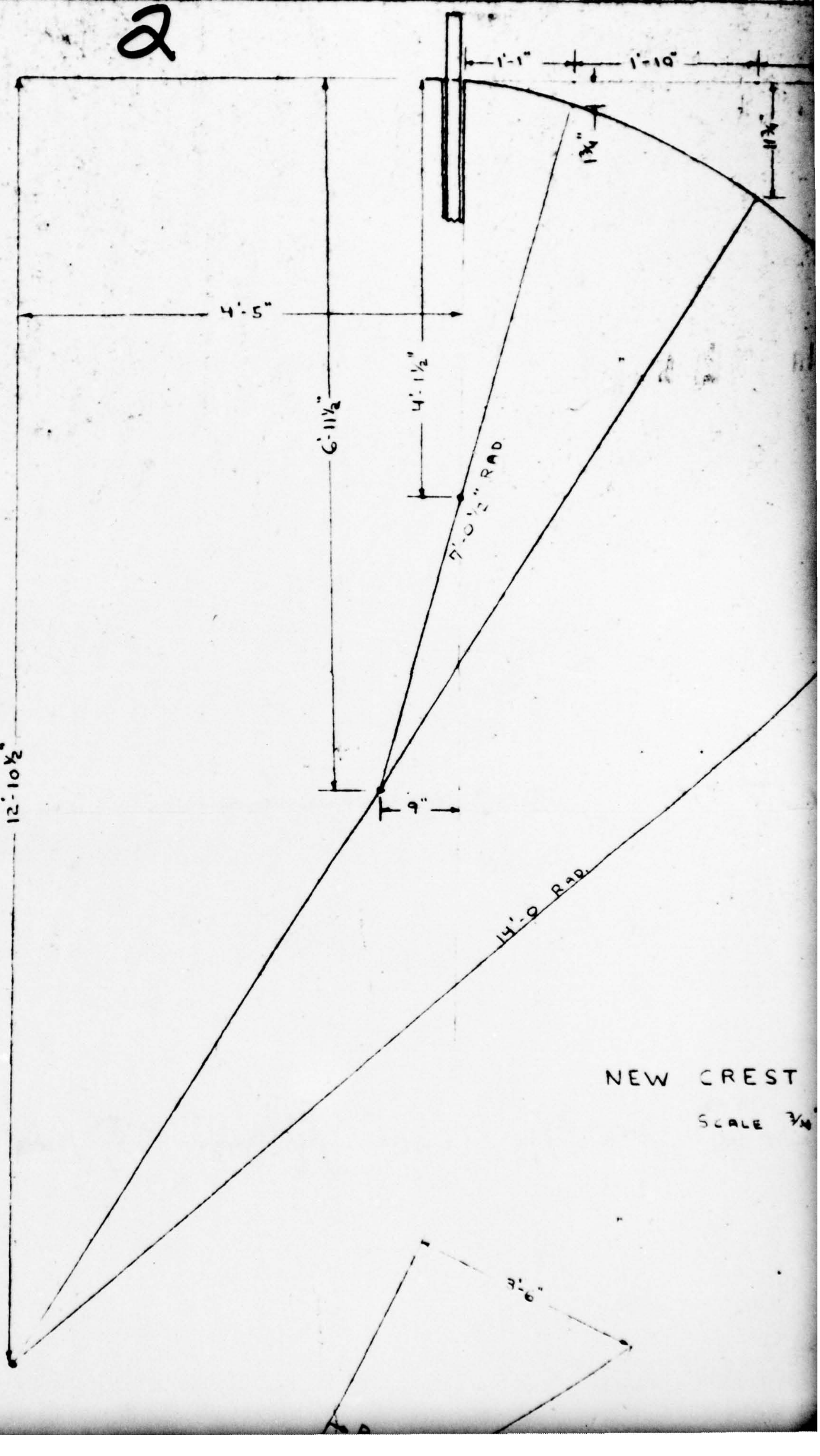
...n understanding with
...nd Paper Co. struts
...not to be replaced

...: During construction
...upper pool will be main-
...t at El. 67.5'.

...Reinforcement &
...e Pipe Shown on
...ate Section Below

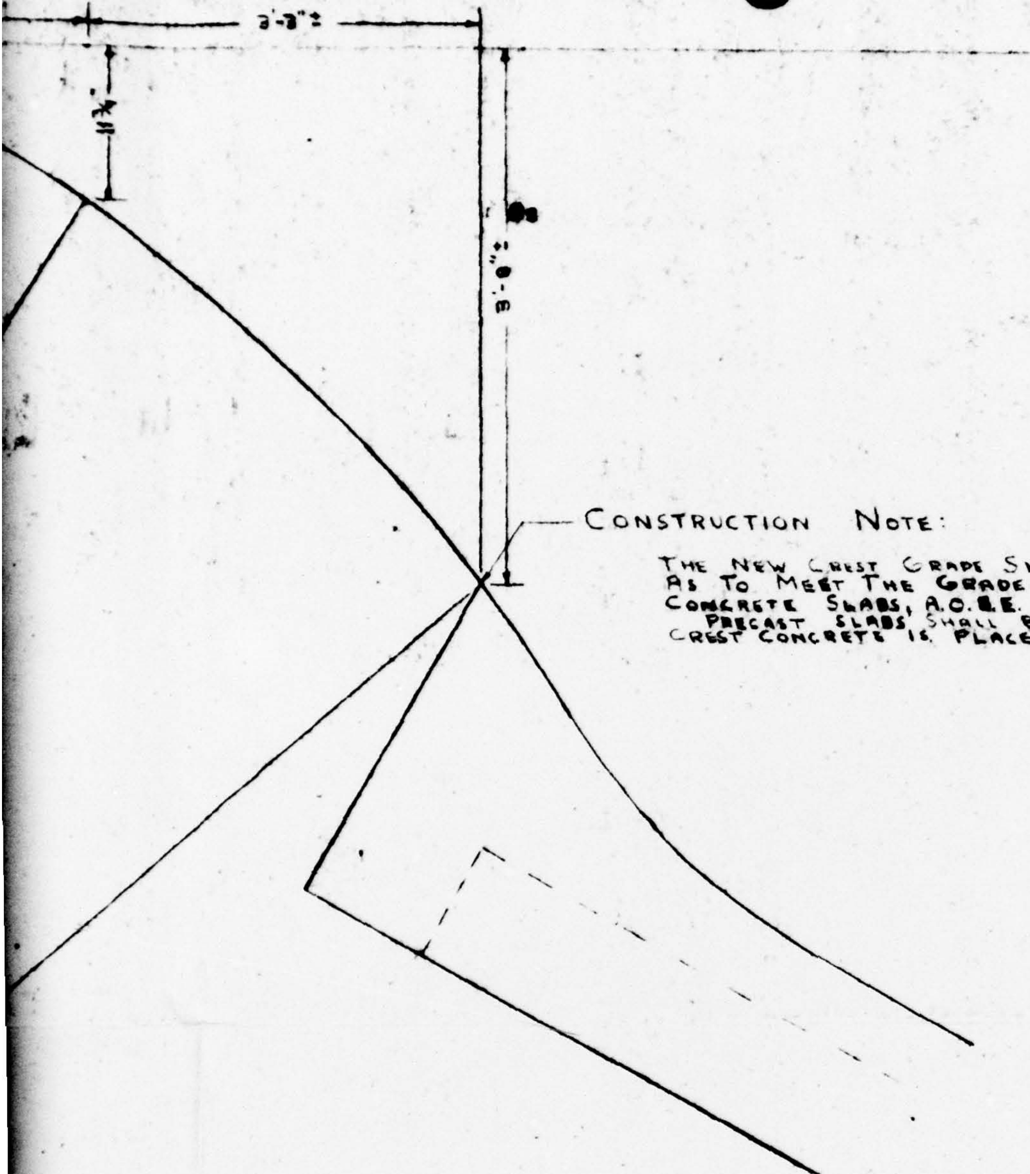
½" premoulded joint filler conforming to M 36, M 36B or M 36C. Cost to be included in price bid for Item 518.





NEW CREST

SCALE ¾"



ALL BARS & STIRRUPS ITEM 203 #4
LONGITUDINAL BARS ALL 21'-6"
LONG. BEGIN STIRRUPS 1' FROM
LOWER END OF PRECAST SLAB. #6

CONSTRUCTION NOTE:

THE NEW CREST GRADE SHALL BE SO PLACED AS TO MEET THE GRADE OF THE PRECAST CONCRETE SLABS, A.O.B.E.
PRECAST SLABS SHALL BE PLACED BEFORE THE CREST CONCRETE IS PLACED.

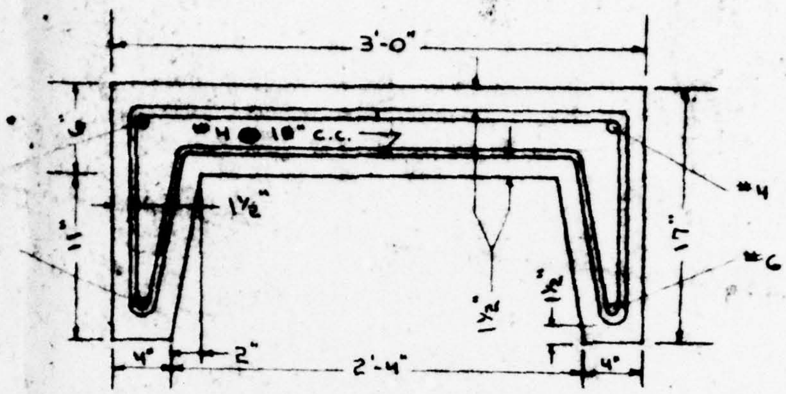
CREST PROFILE

SCALE ¾" = 1'

LOCATE THIS STIRRUP
6" FROM END

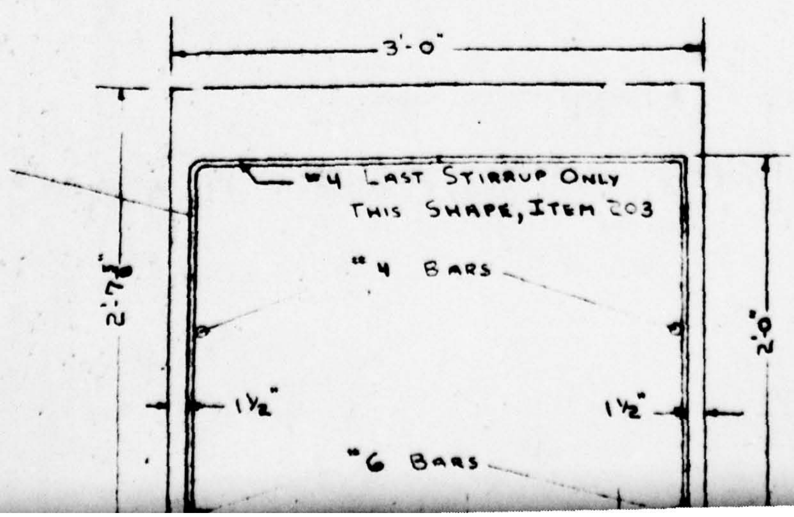
ARS & STIRRUPS ITEM 203 #4
DINAL BARS ALL 21'-6"
EGIN STIRRUPS 1' FROM
ND OF PRECAST SLAB. #6

PLACED
RECAST
BEFORE THE



SECTION A-A
SHOWING BAR REINFORCEMENT
SCALE 1"=1'

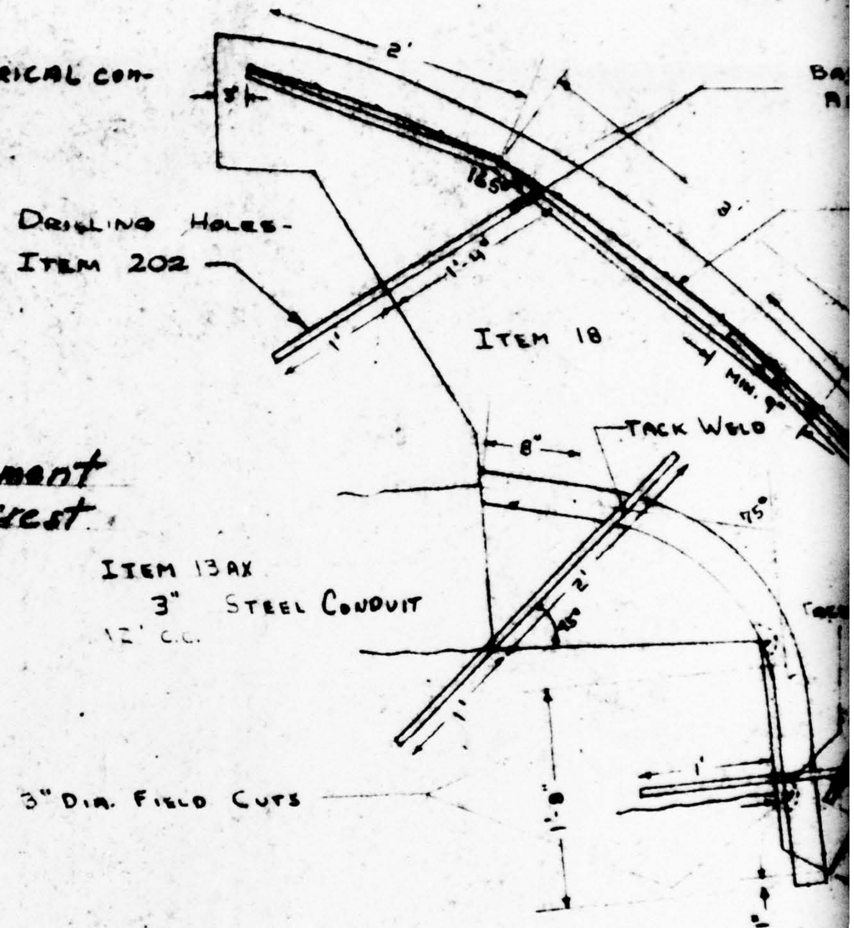
ATE THIS STIRRUP
" FROM END



CONSTRUCTION NOTE:

THE 3" STEEL SEEPAGE ELECTRICAL CONDUIT SHALL BE INSTALLED SO THAT IT WILL NOT BE DISLOCATED, OR DRAIN HOLES BECOME PLUGGED DURING CONCRETE PLACEMENT. 3" DRAIN HOLES SHALL BE FIELD LOCATED TO COINCIDE WITH MASONRY JOINT SEEPAGE PLAINS.

Note: The cost of all labor, equipment and material required to insure that crest concrete is placed in the dry shall be included in the prices bid for the various items of the contract.



SECTION VIEW OF CRE

SHOWING CREST POUR, REINFORC

SCALE 3/4" = 1'

MADE BY ROBERT KIRNER DATED NOV. '64

CHECKED BY [illegible] DATED [illegible]

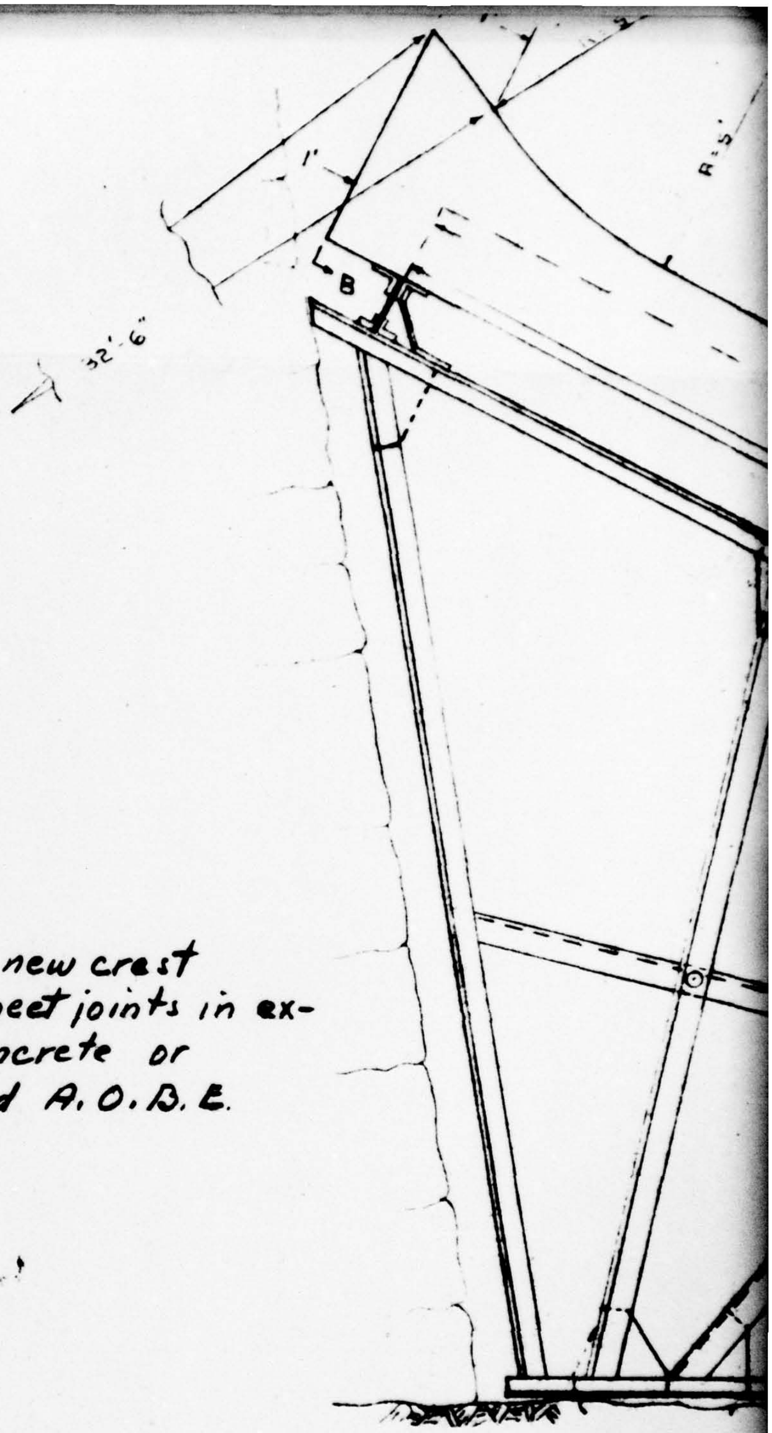
TRACED BY ROBERT [illegible] DATED NOV. '64

CHECKED BY [illegible] DATED [illegible]

5

ITEM 28X
BAR REINFORCEMENT FOR STRUCTURES,
ALL BARS #6, 18" C.C.

ITEM 25F
STEEL FABRIC REINFORCEMENT
6½' LONG



Note. Joints in new crest concrete shall meet joints in existing crest concrete or shall be placed A.O.B.E.

CREST
...NFORCEMENT, PIPE.

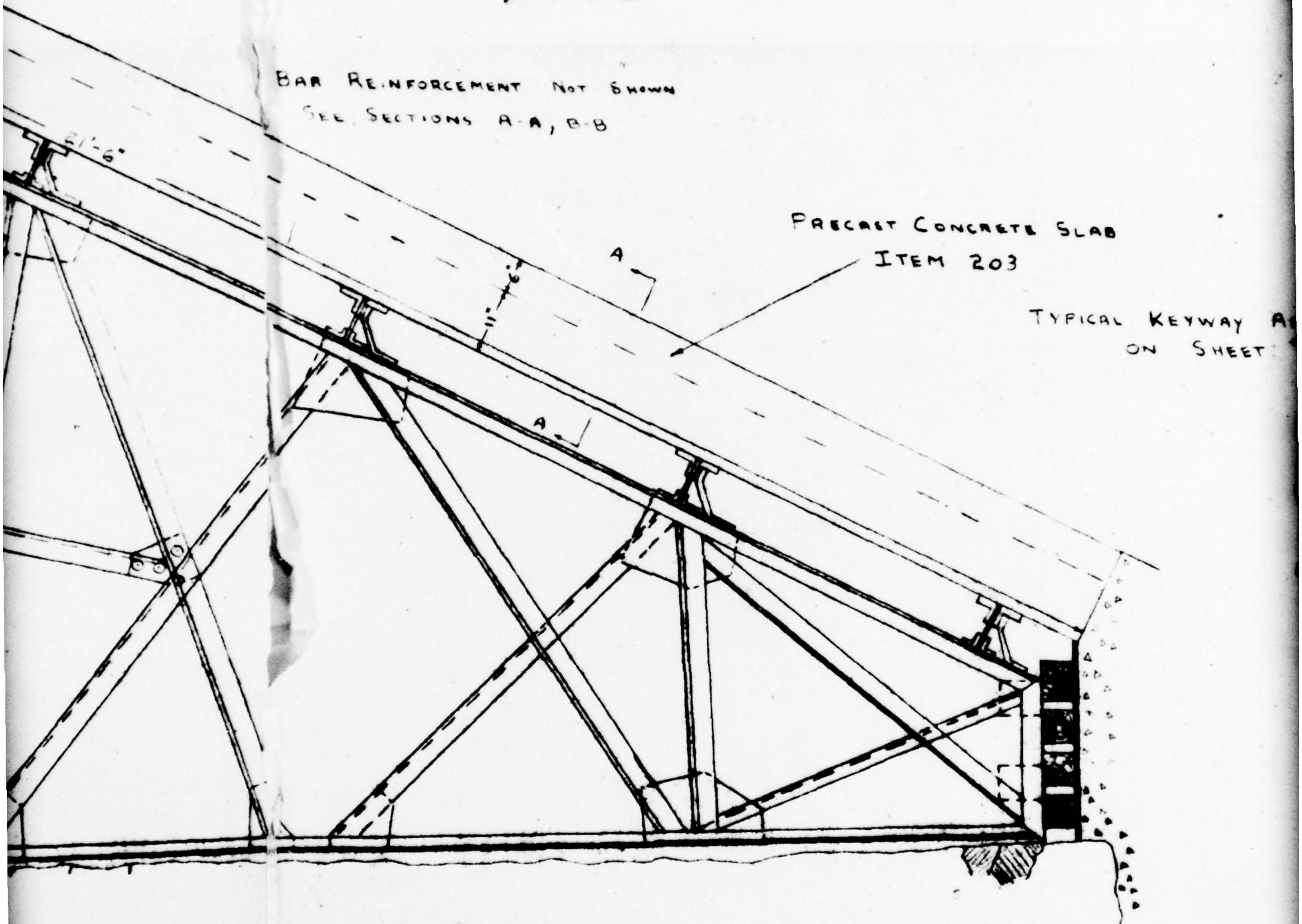
6

M 8

NOTE:

SEE SHEET #4 FOR PLAN VIEW OF PRECAST SLABS, SHOWING NON-TYPICAL PRECAST SLAB SECTIONS AT W.V.P.&P. CO. WALL AND APPROX. CONCRETE POUR AT LOCK WALL.

Precast concrete slabs are to be set in place as close to each other as possible



TYPICAL DAM CROSS SECTION

SHOW WROUGHT IRON BENT

WITH CONC. SLAB

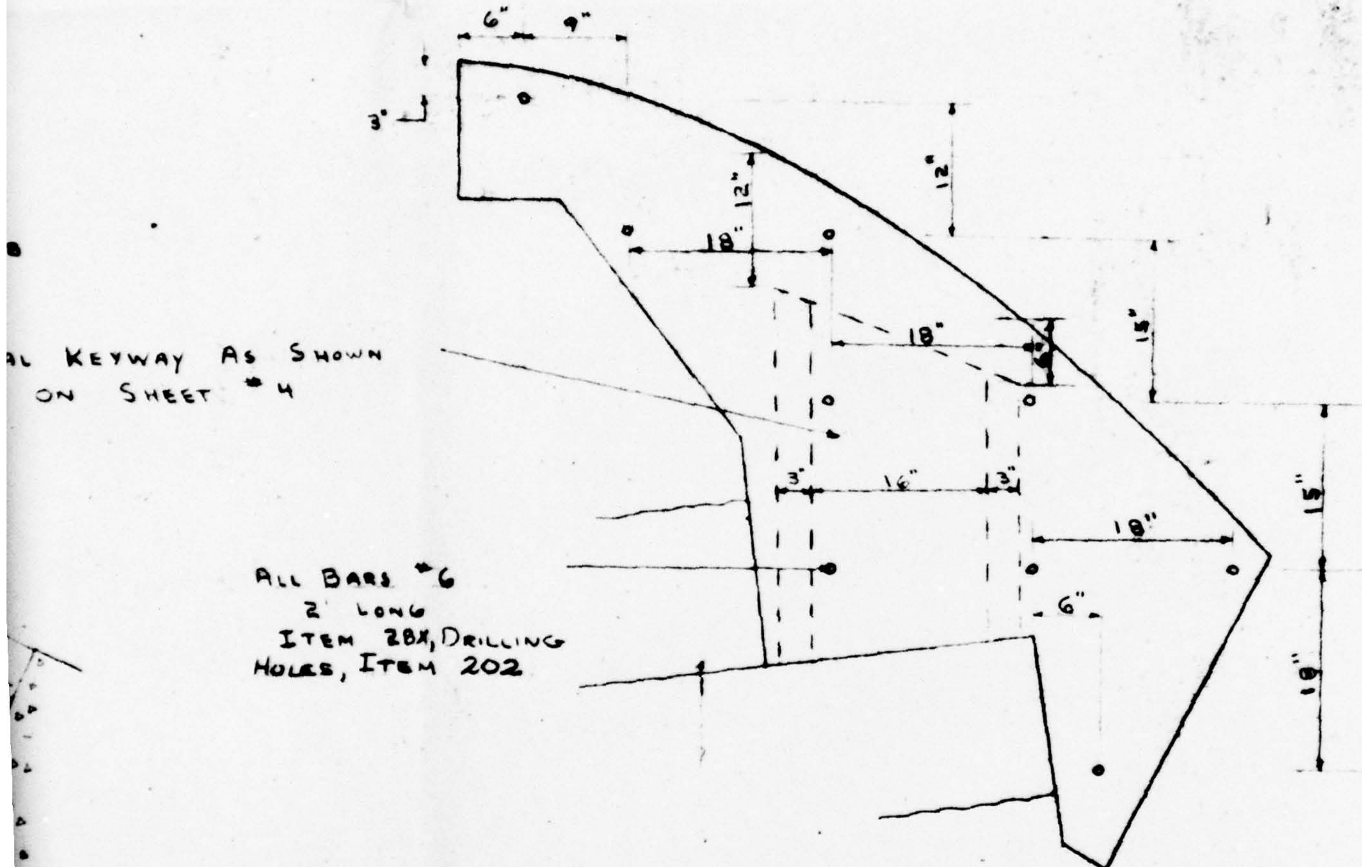
½" = 1'

7

SECTION B-B

End Stirrup

Scale 1"=1'



CREST DOWELLING DETAIL AT LOCK & W.V.P.P.CO. WALLS

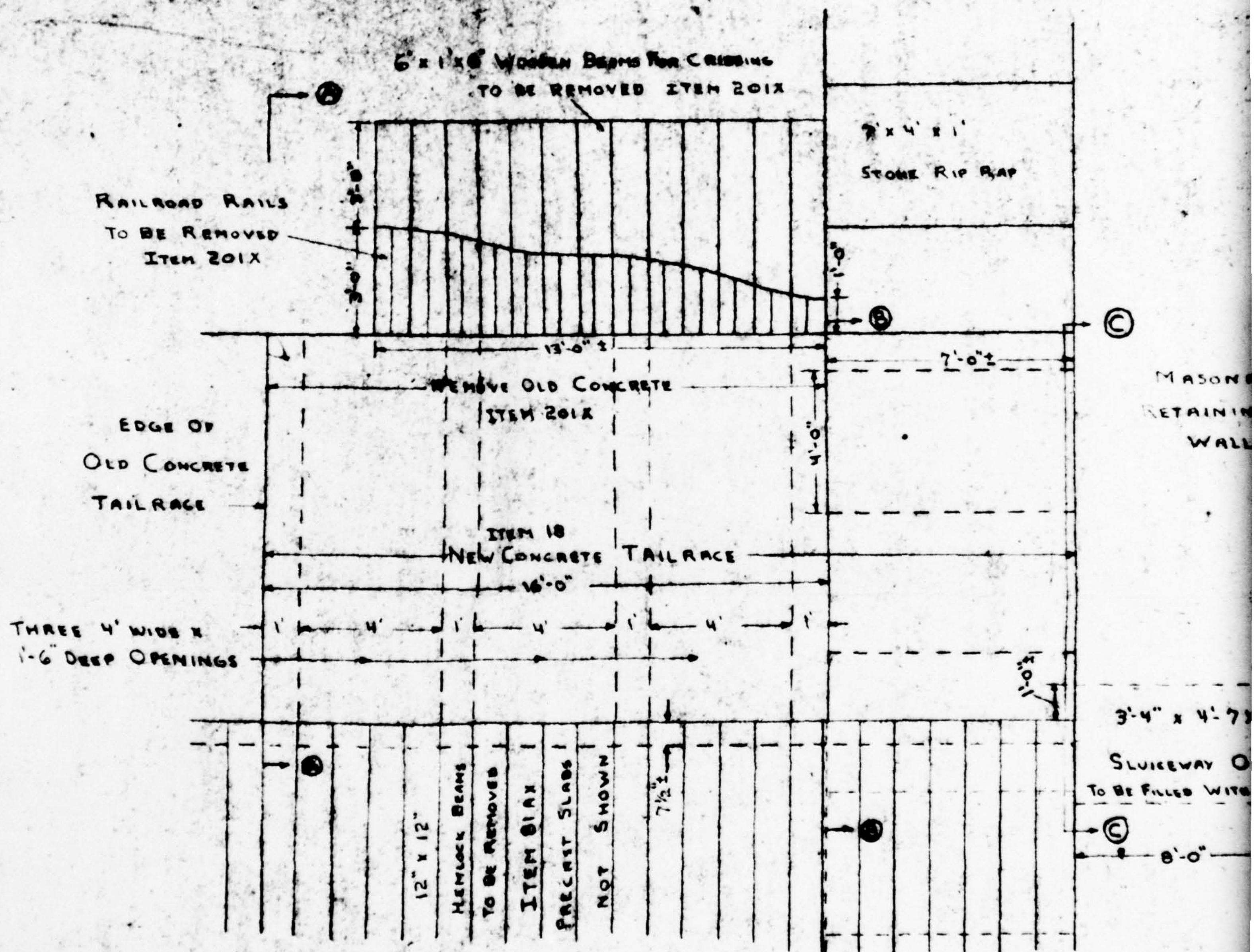
KEYWAY AT CONST. JOINTS

SCALE 3/4"=1'

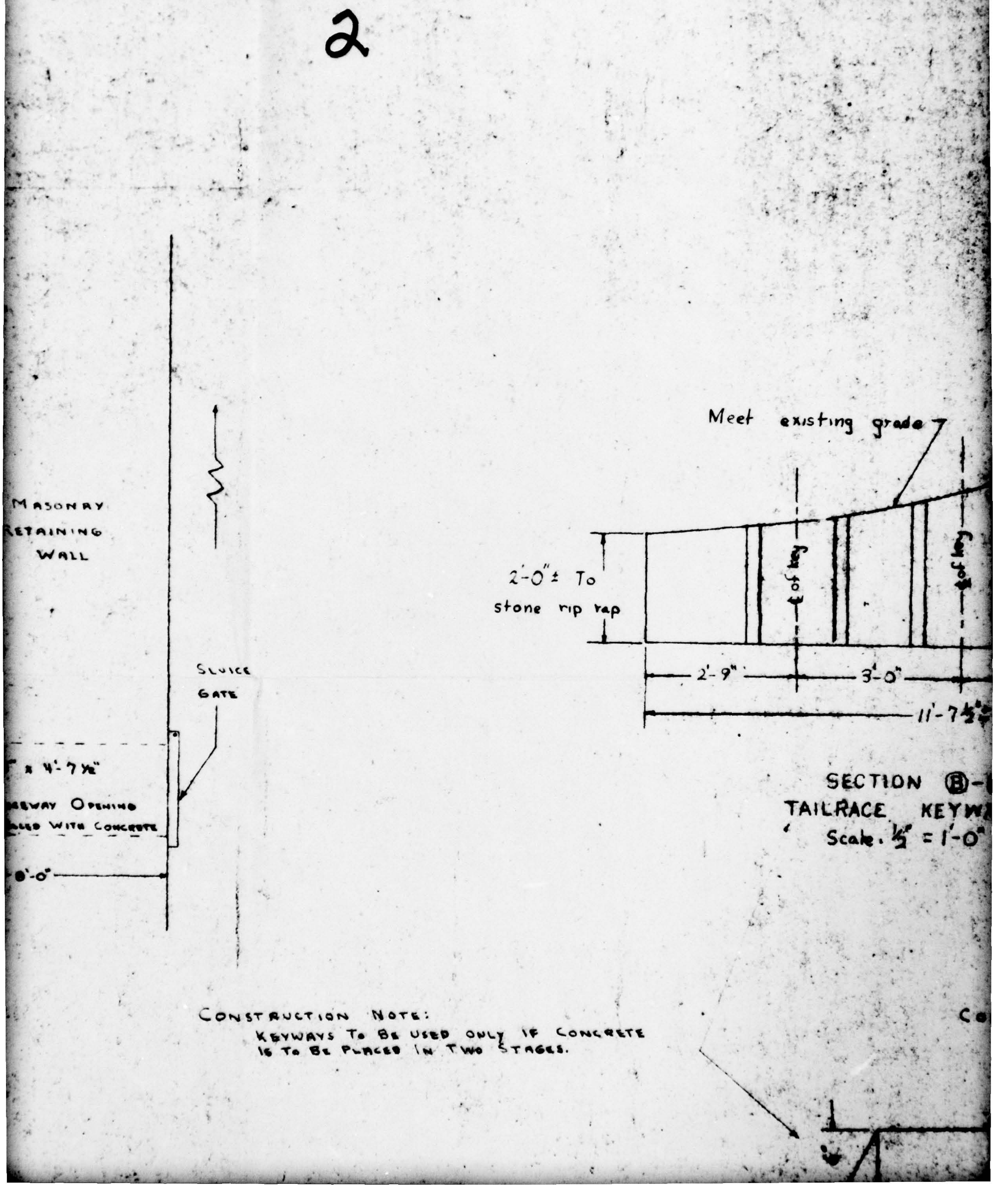
8

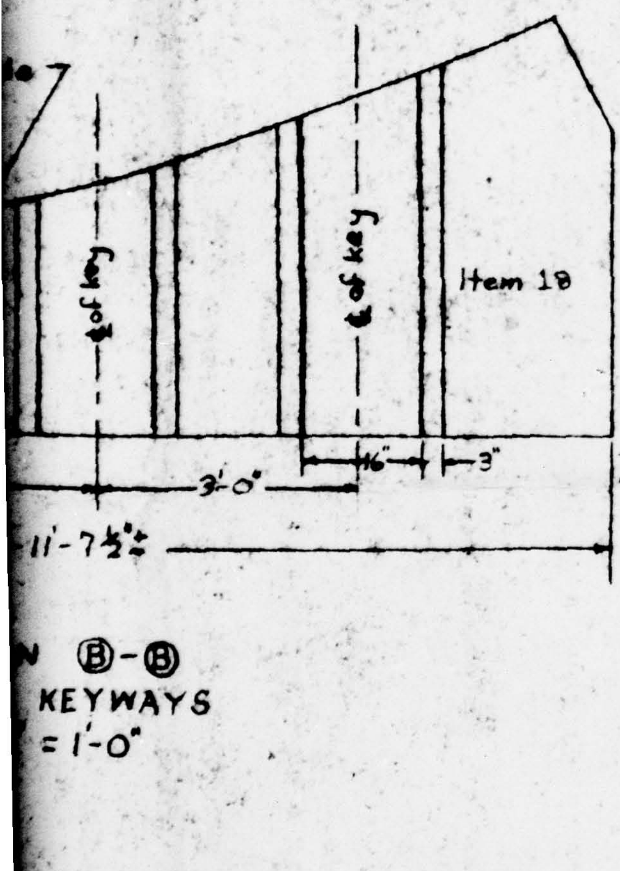
3

1

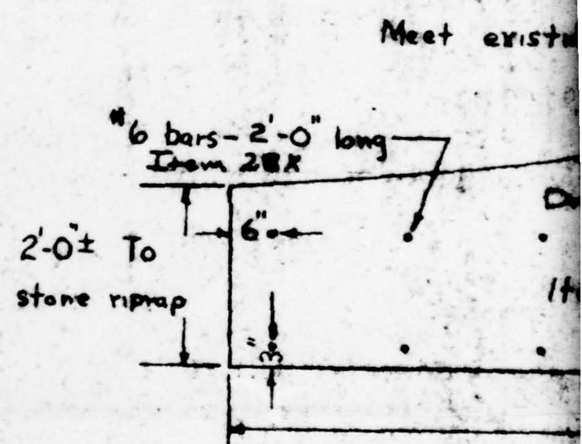


Plan Of New Section
Of Concrete Tailrace
Scale 1/4" = 1'-0"


2
Masonry
Retaining
Wall
Sluice
Gate
x 4'-7½"
eway Opening
aced With Concrete
8'-0"
Meet existing grade
2'-0"± To
stone rip rap
℄ of key
℄ of key
2'-9"
3'-0"
11'-7½"
SECTION Ⓑ-
TAILRACE KEYW
Scale. ½" = 1'-0"
Construction Note:
Keyways To Be Used Only If Concrete
Is To Be Placed In Two Stages.
Co



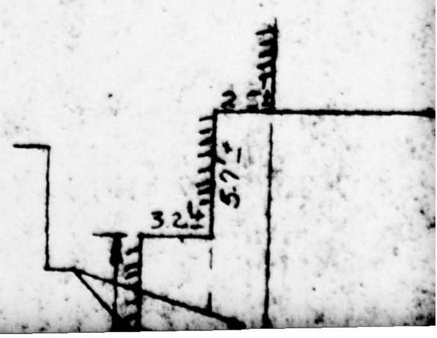
N (B)-(B)
KEYWAYS
= 1'-0"



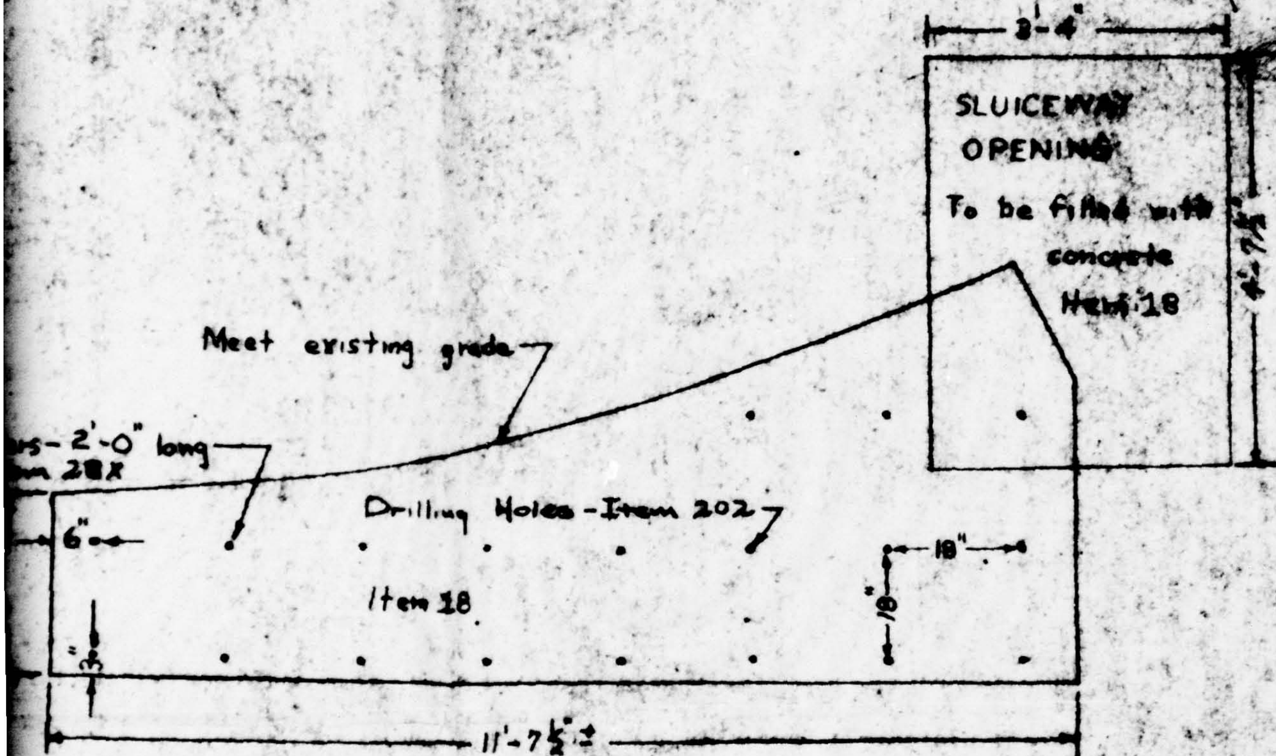
SEC
TAILRACE DOWEL
Scal

CONSTRUCTION NOTE:

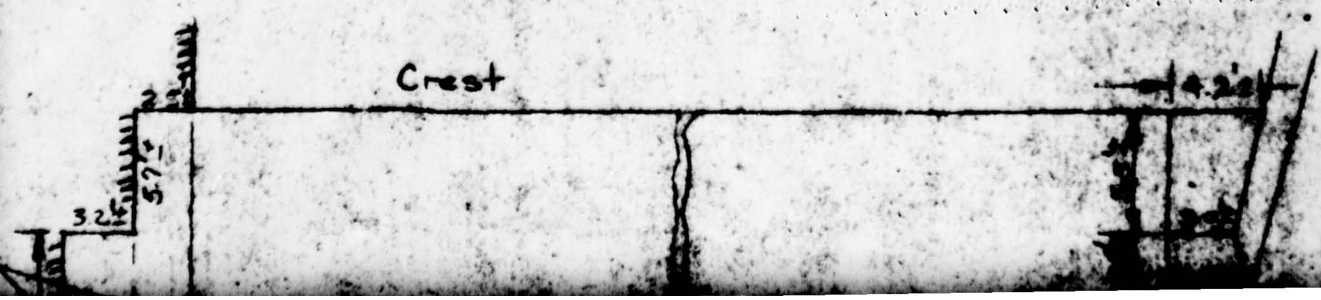
TWO PRECAST CONCRETE SECTIONS WILL BE PLACED IN THIS AREA WITH CONCRETE CROSS-SECTION AND REINFORCING STEEL SIMILAR TO SECTION A-A, SHEET #3. LENGTH AND WIDTH TO BE MEASURED IN FIELD AND SLAB BEVELED AT UPSTREAM END TO FIT AGAINST W.V.P.&P. CO. WALL



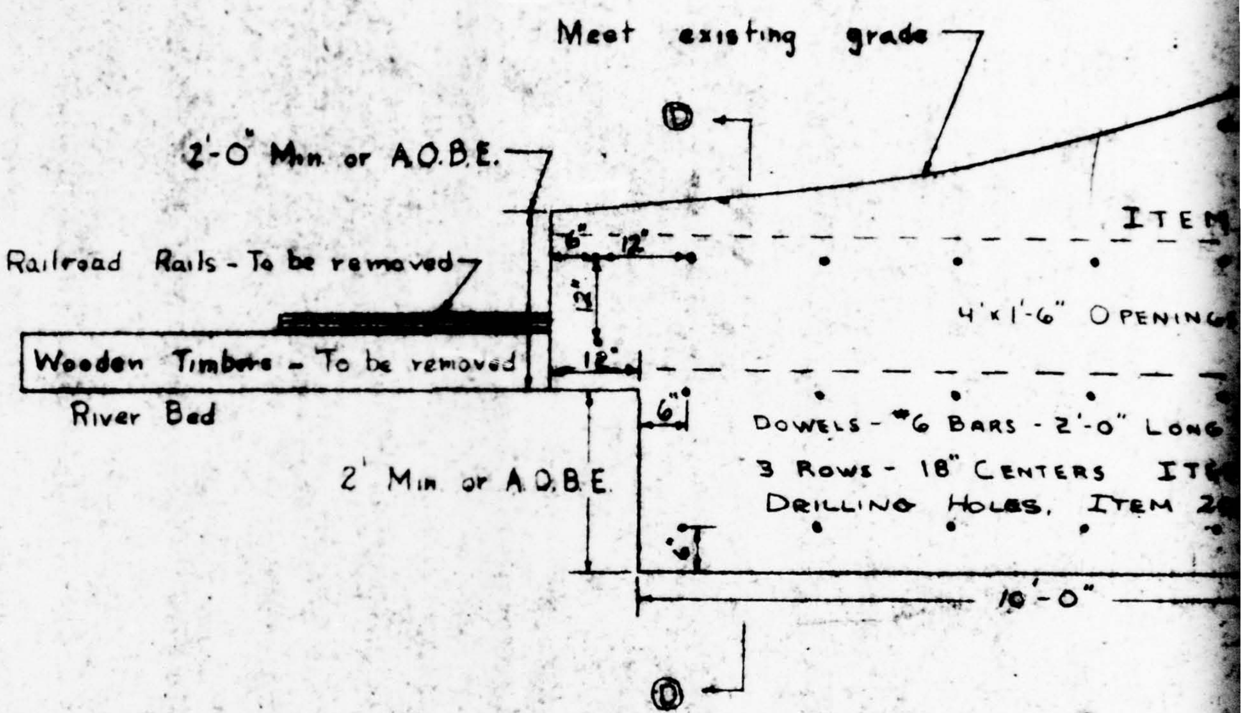
4



SECTION C-C
TAILRACE DOWELING DETAIL AT W.V.P.&P. CO. WALL
Scale: ½" = 1'-0"

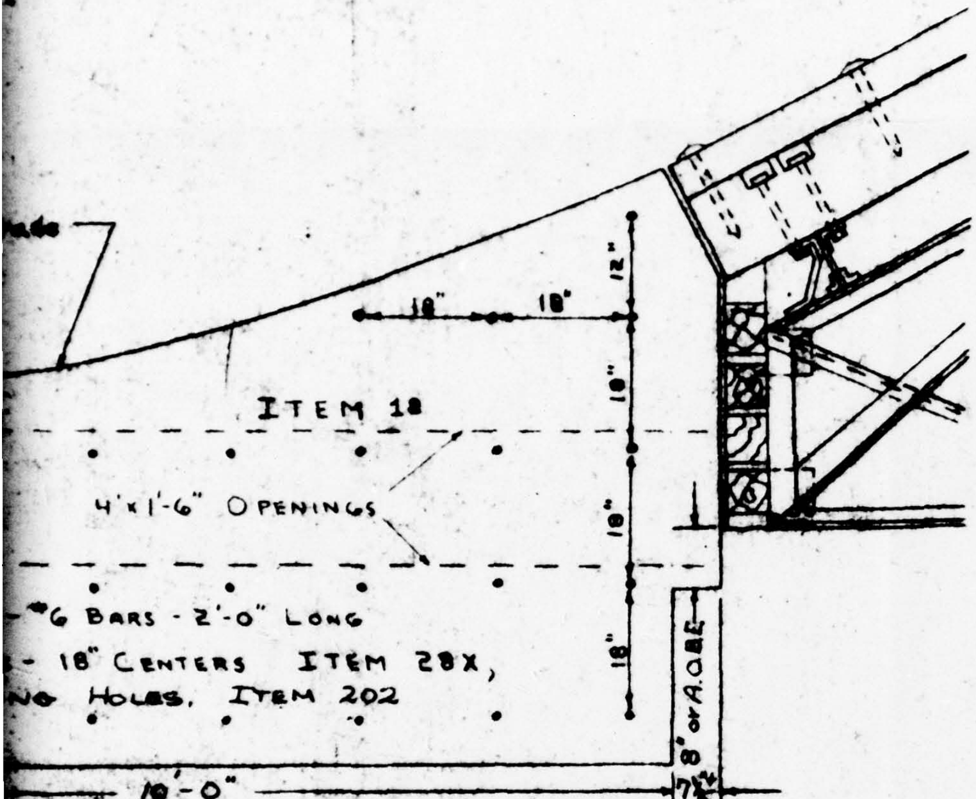


5

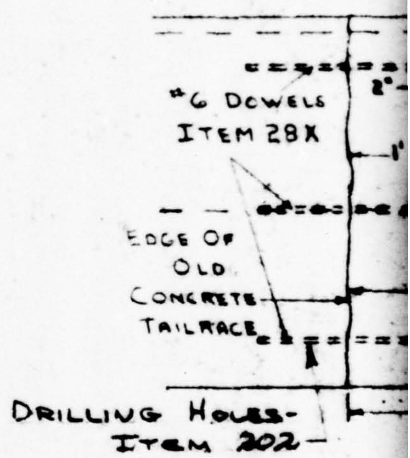


SECTION (A)-(A)
NEW CONCRETE TAIL
Scale: 1/2" = 1'-0"

MADE BY Robert Kisker DATED Nov. '64
CHECKED BY Robert Wood DATED Nov. 1964
TRACED BY Robert Kisker DATED Nov. '64
CHECKED BY Robert Wood DATED Nov. 1964

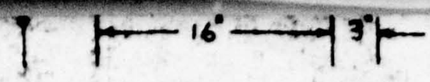


SECTION Ⓐ-Ⓐ
CONCRETE TAILRACE
Scale: 1/2" = 1'-0"

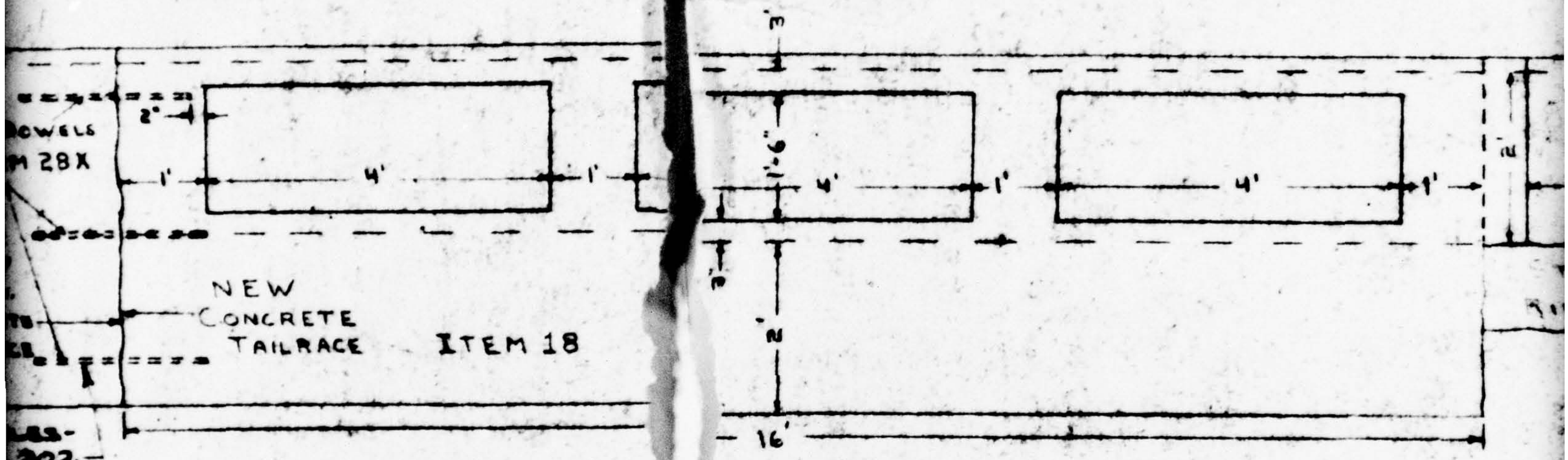


SLABS TO BE PAID FOR UNDER ITEM 2

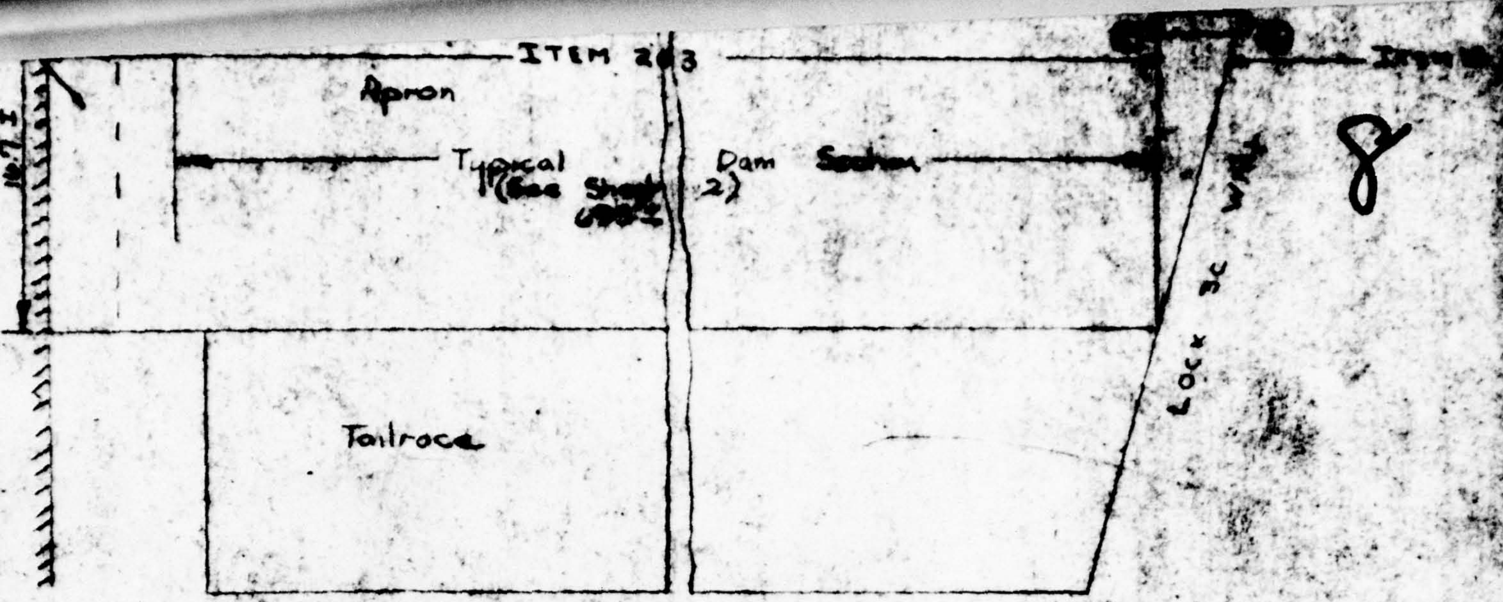
7



TYPICAL KEYWAY
Scale: 1" = 1'-0"



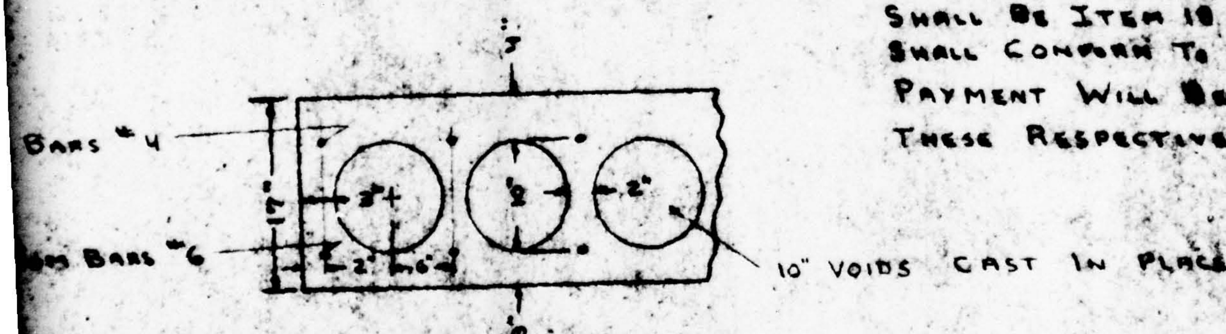
SECTION D-D
NEW CONCRETE TAILRACE
SHOWING OPENINGS
AND
JOINT DETAILS
SCALE 1/2" = 1'



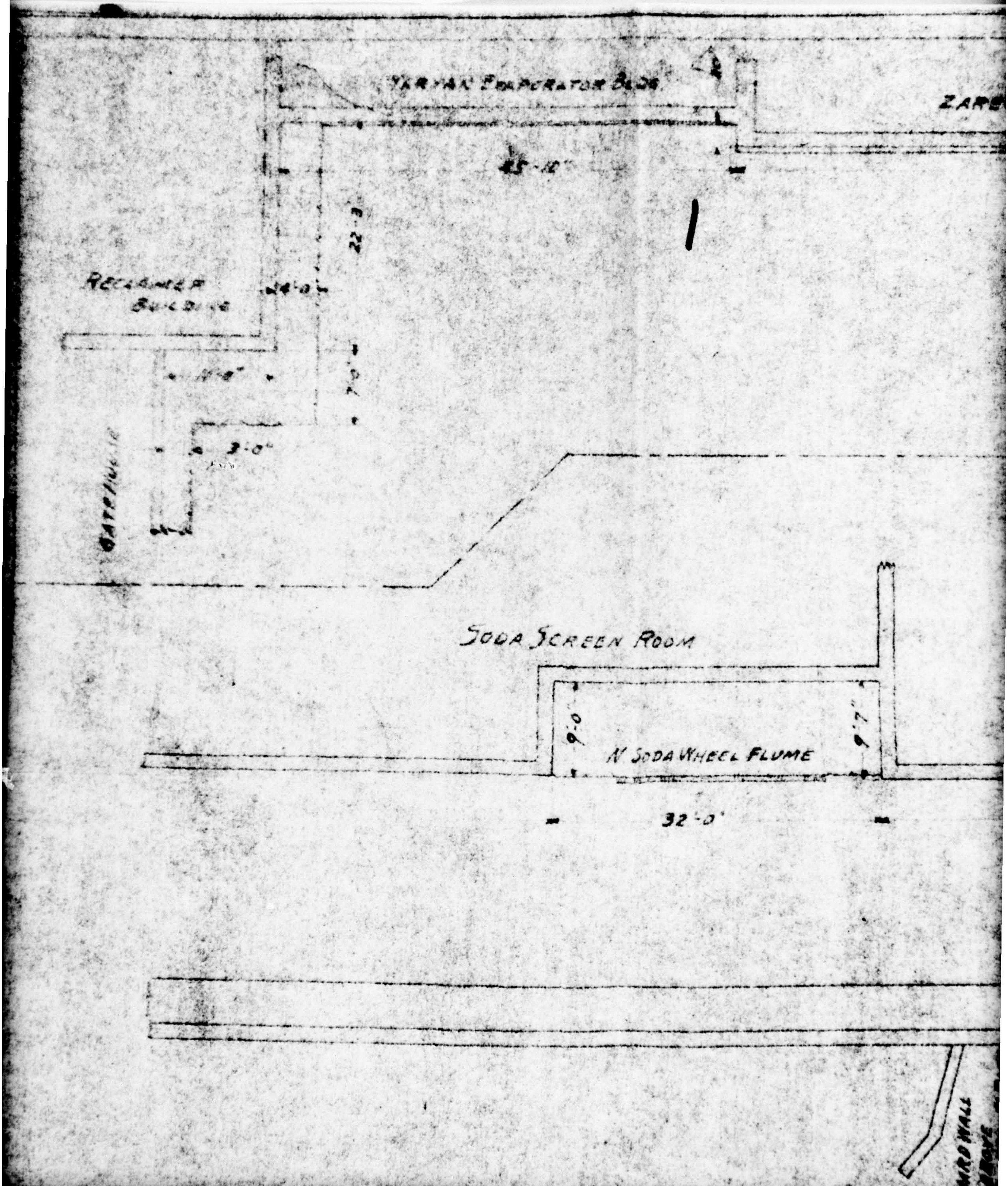
PLAN OF DAM SHOWING IRREGULARITIES
OF APRON
SCALE: $\frac{1}{8}$" = 1'-0"

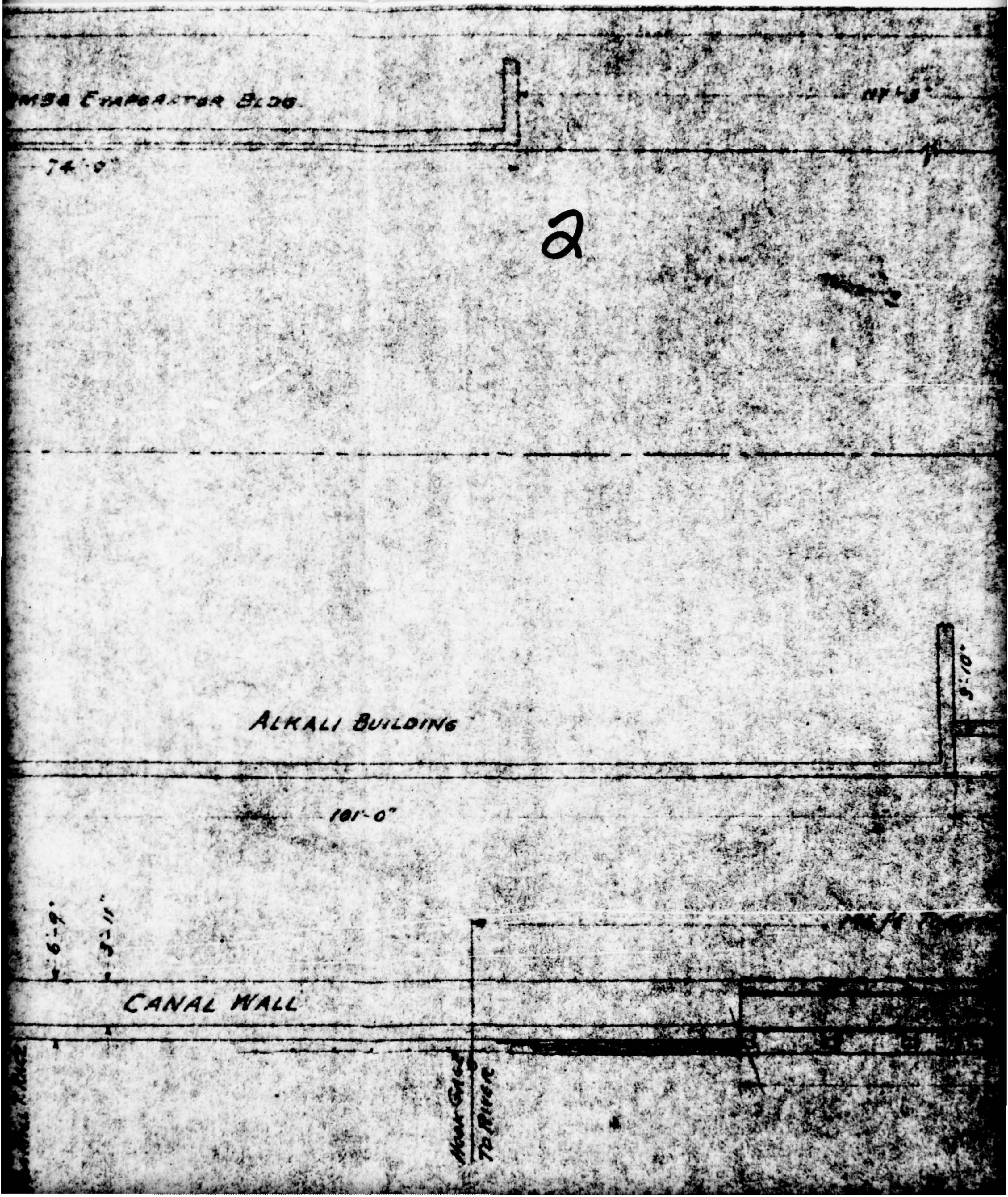
CONST JOINT KEYWAY
SEE ABOVE NOTE

CONSTRUCTION NOTE:
AT EAST END OF DAM CONCRETE WILL BE PLACED U.S.E. TO COMPLETE APRON. THE CONCRETE SHALL BE ITEM 10 AND RE. BARS SHALL CONFORM TO ITEM 20X: PAYMENT WILL BE MADE UNDER THESE RESPECTIVE ITEMS.

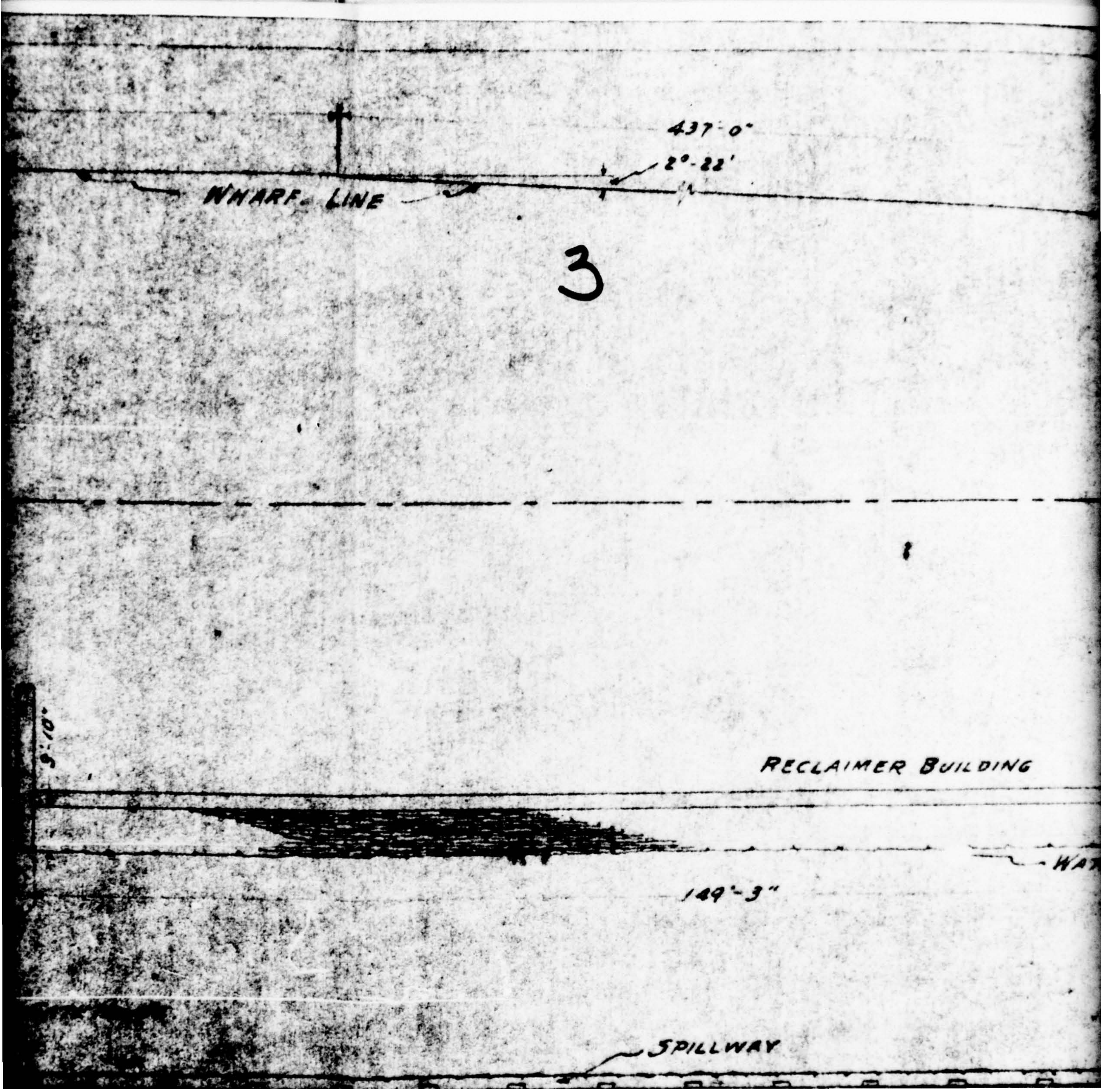


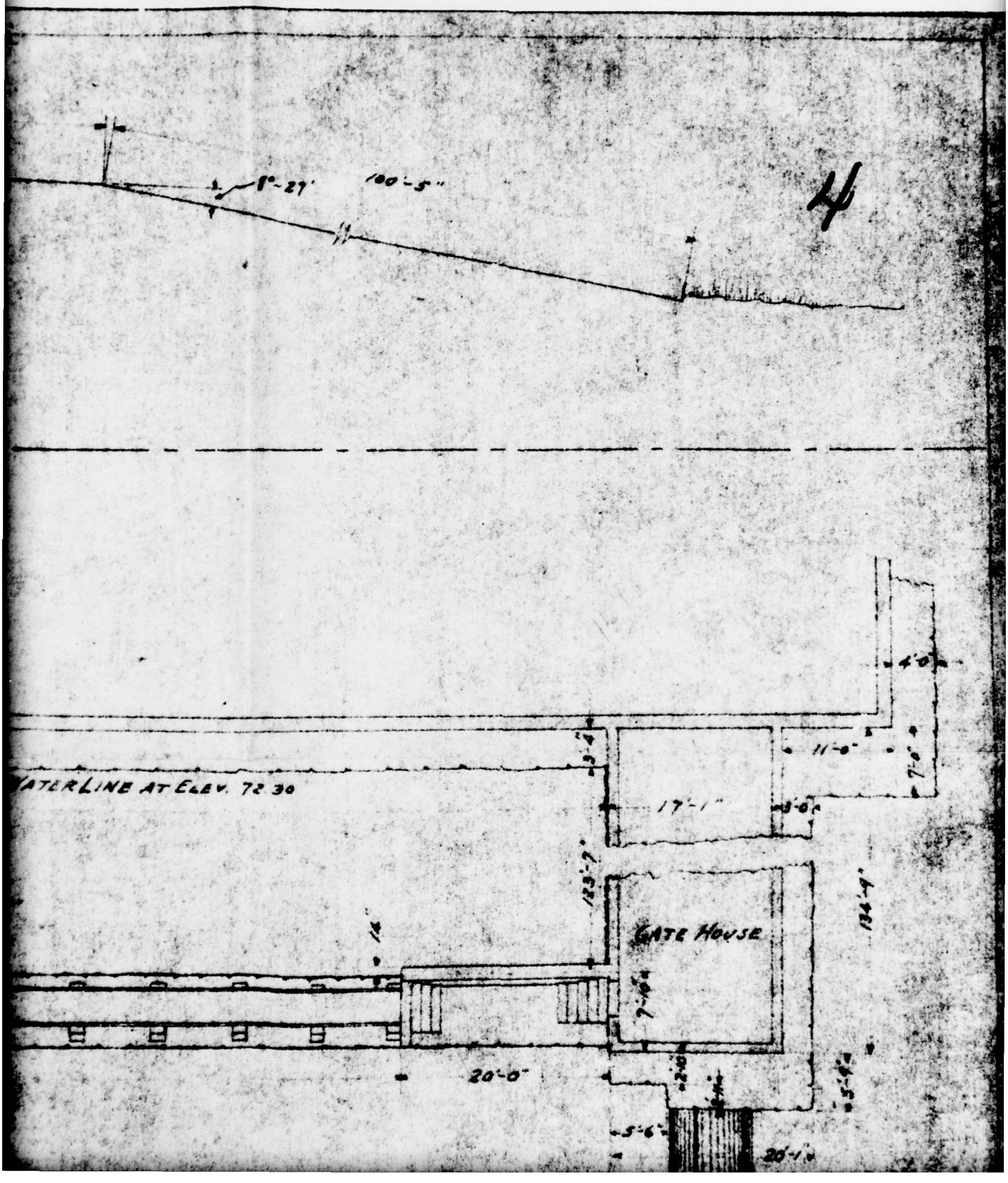
SECTION Ⓔ-Ⓔ
TYPICAL SECTION - POURED IN PLACE
SCALE ¾" = 1'


YARYAN EVAPORATOR BLDG.
ZARB
45'-10"
1
22'-3"
RECLAIMER
BUILDING
14'-0"
7'-0"
3'-0"
SODA SCREEN ROOM
9'-0"
N SODA WHEEL FLUME
9'-7"
32'-0"

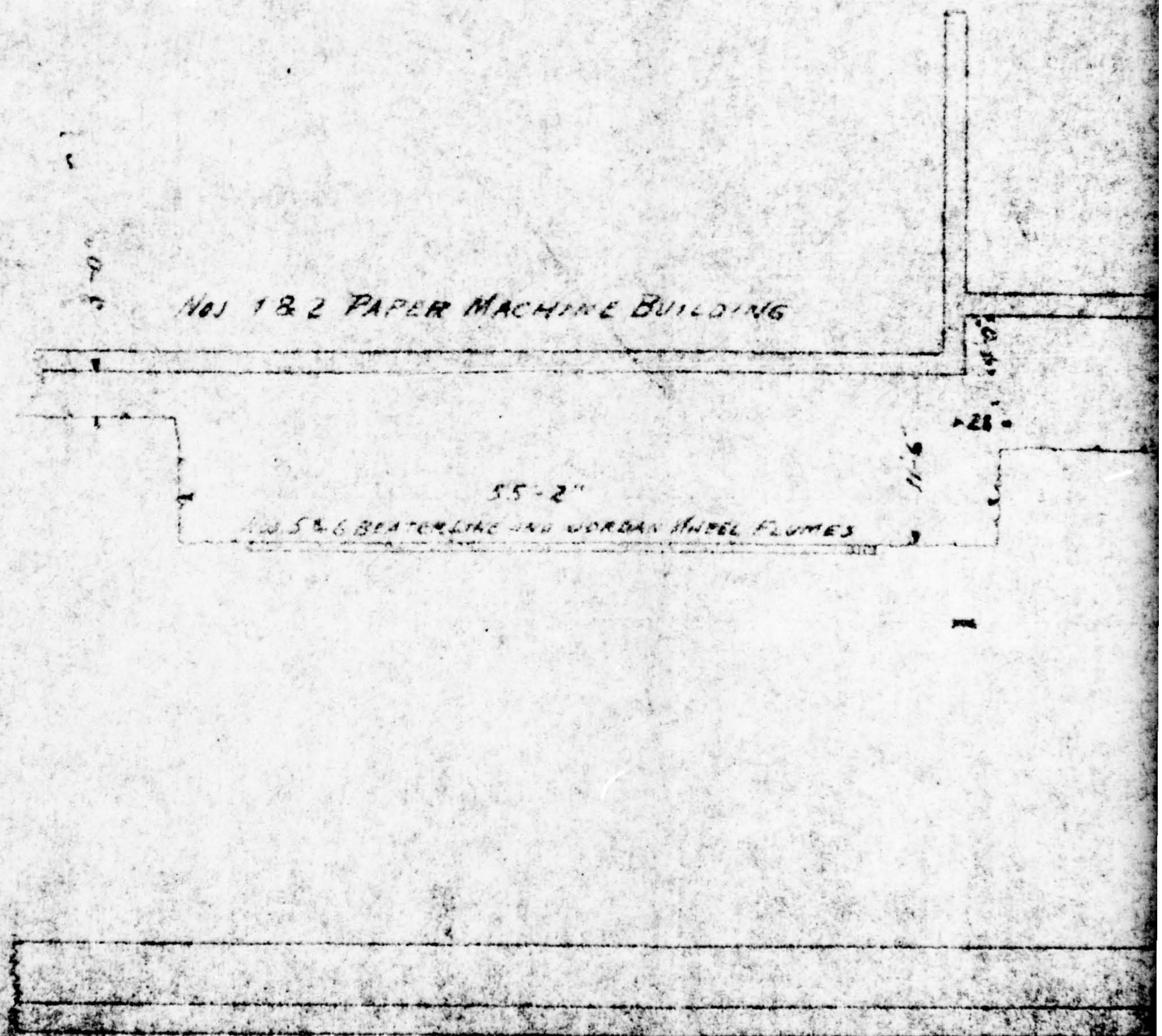


74'-0"
2
ALKALI BUILDING
9'-10"
101'-0"
6'-9"
3'-11"
CANAL WALL
TO RIVER

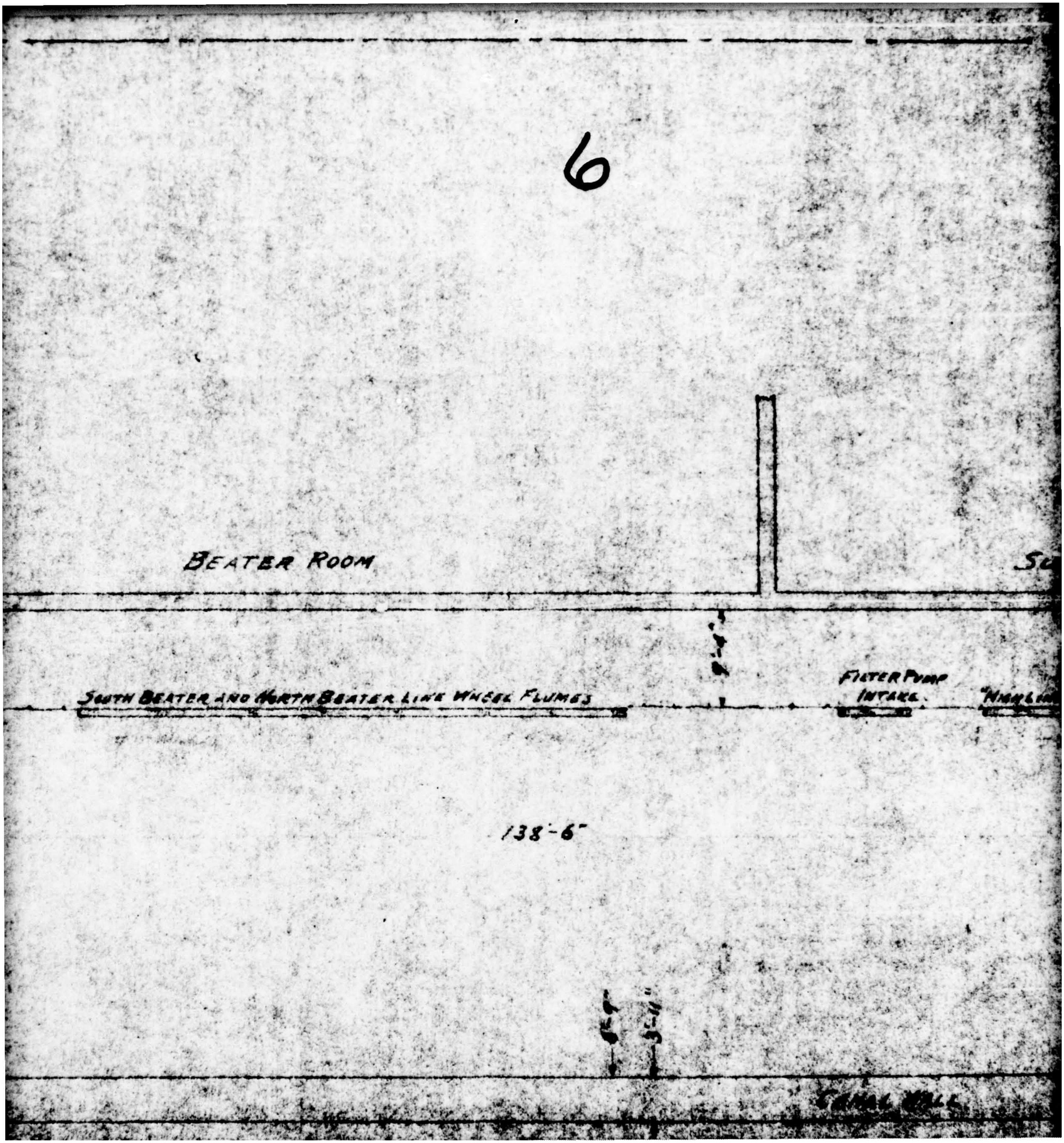

437'-0"
2°-22'
WHARF LINE
3
9'-10"
RECLAIMER BUILDING
149'-3"
SPILLWAY


4
8°-27'
100'-5"
4'-0"
11'-0"
7'-0"
3'-4"
WATERLINE AT ELEV. 72.30
17'-1"
3'-0"
123'-7"
134'-9"
14"
GATE HOUSE
7'-0½"
20'-0"
5'-6"
20'-1"

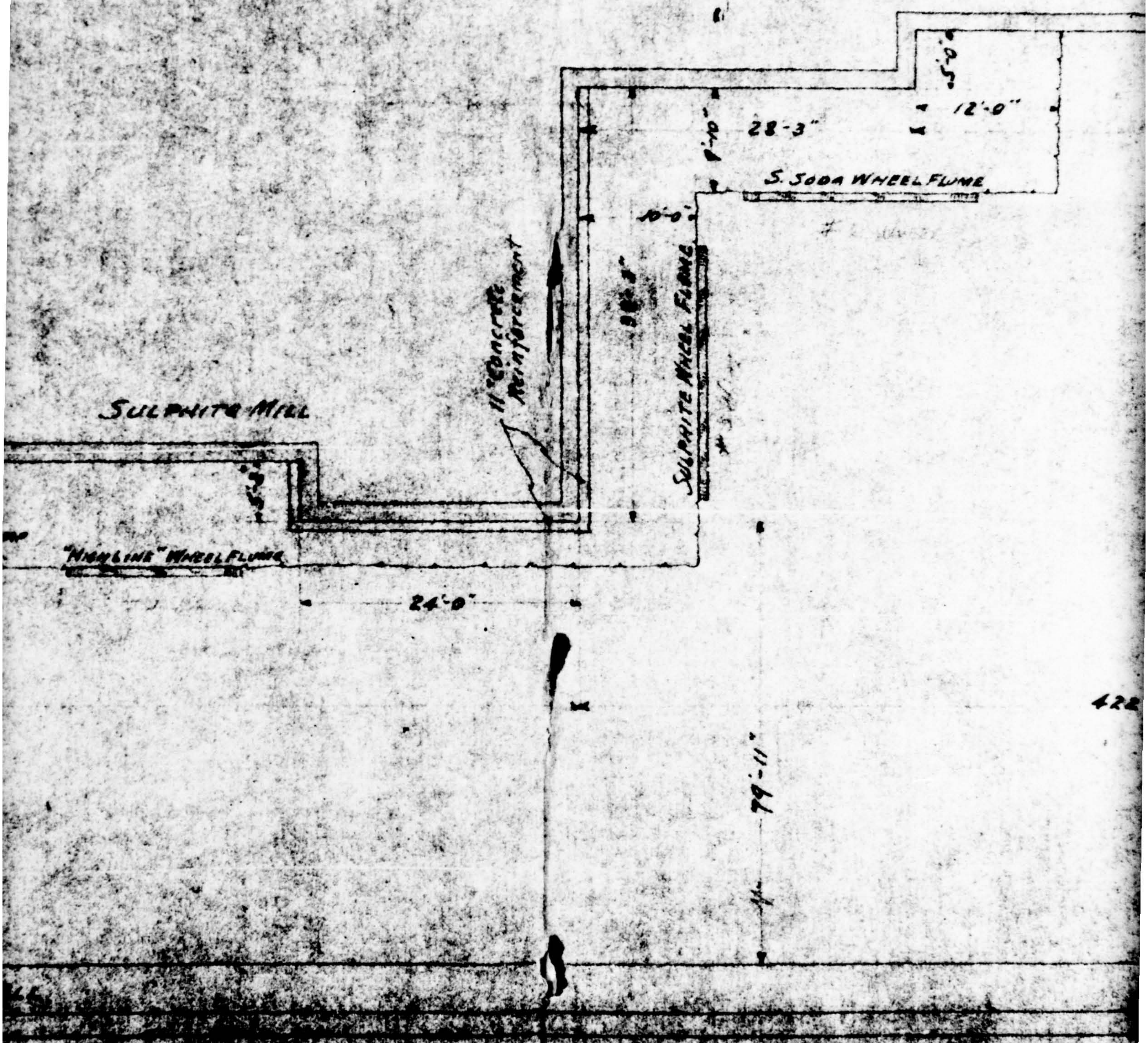
5



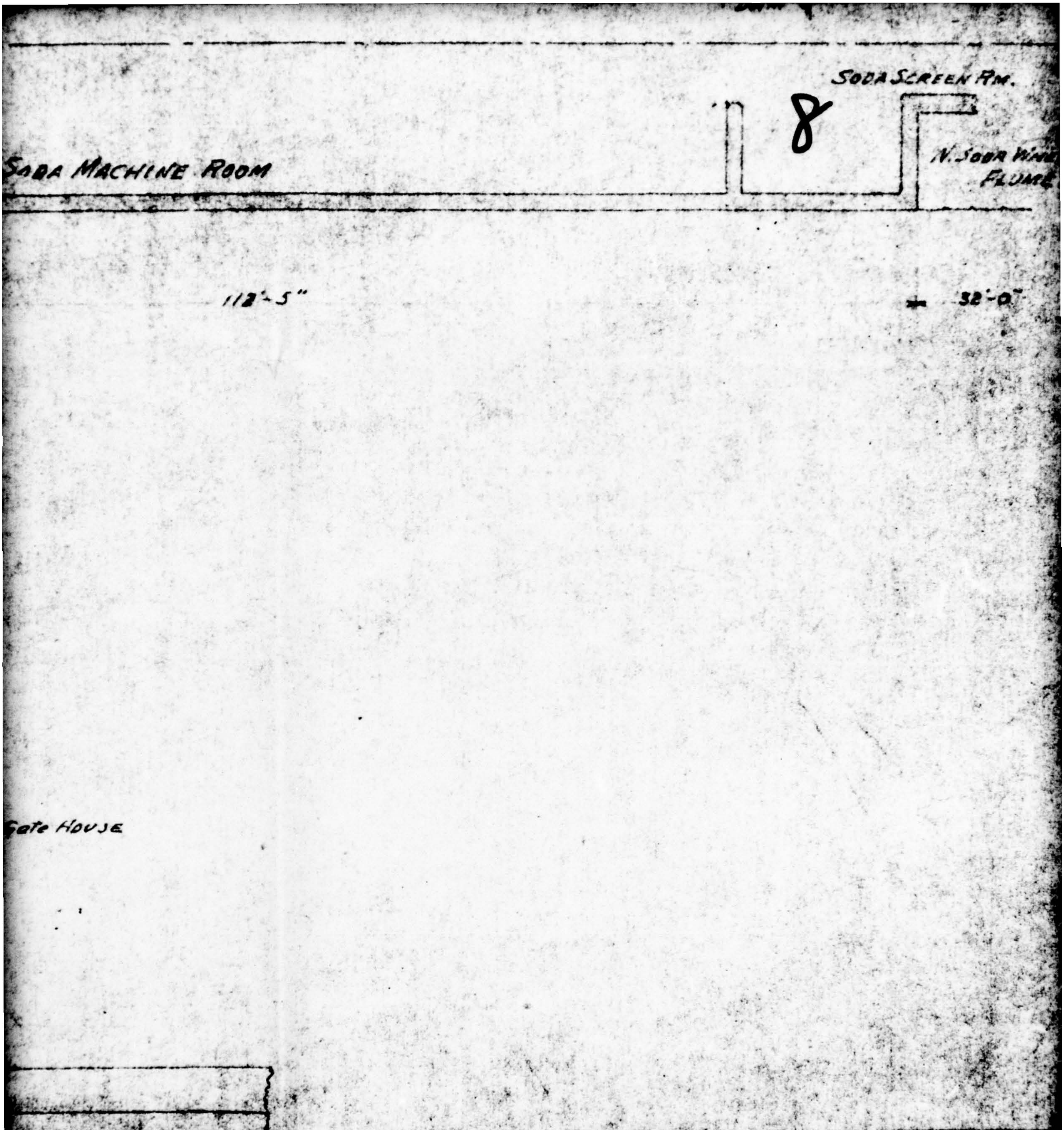
NOS 1 & 2 PAPER MACHINE BUILDING
55'-2"
11'-6"
28
JORDAN WHEEL FLUMES


6
BEATER ROOM
SOUTH BEATER AND NORTH BEATER LINE WHEEL FLUMES
FILTER PUMP INTAKE
138'-6"
CANAL WALL

7



SULPHITE MILL
11" CONCRETE REINFORCEMENT
S. SODA WHEEL FLUME
SULPHITE WHEEL FLUME
"HIGHLINE" WHEEL FLUME
28'-3"
12'-0"
10'-0"
24'-0"
79'-11"



WEST VIRGINIA PULP & PAPER CO.

MECHANICVILLE, N.Y.

PLAN OF HYDRAULIC CANAL

SHOWING

ADJACENT BUILDING WALLS

SHEET NO. 1

House

All measurements across power canal are from top of walls.

WEST VIRGINIA PULP & PAPER CO.
MECHANICVILLE, N.Y.

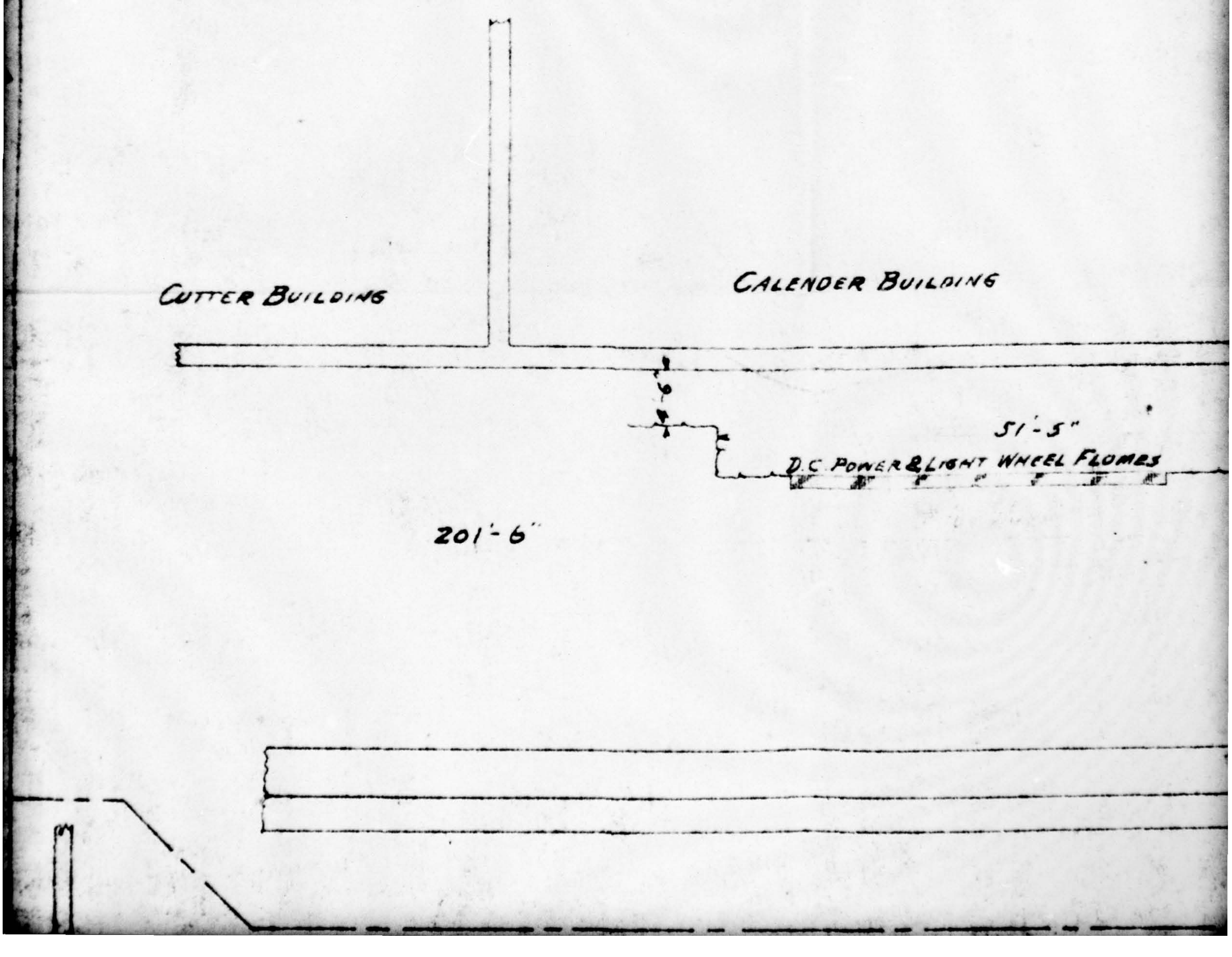
PLAN OF HYDRAULIC CANAL
SHOWING
ADJACENT BUILDING WALLS
SHEET No. 1

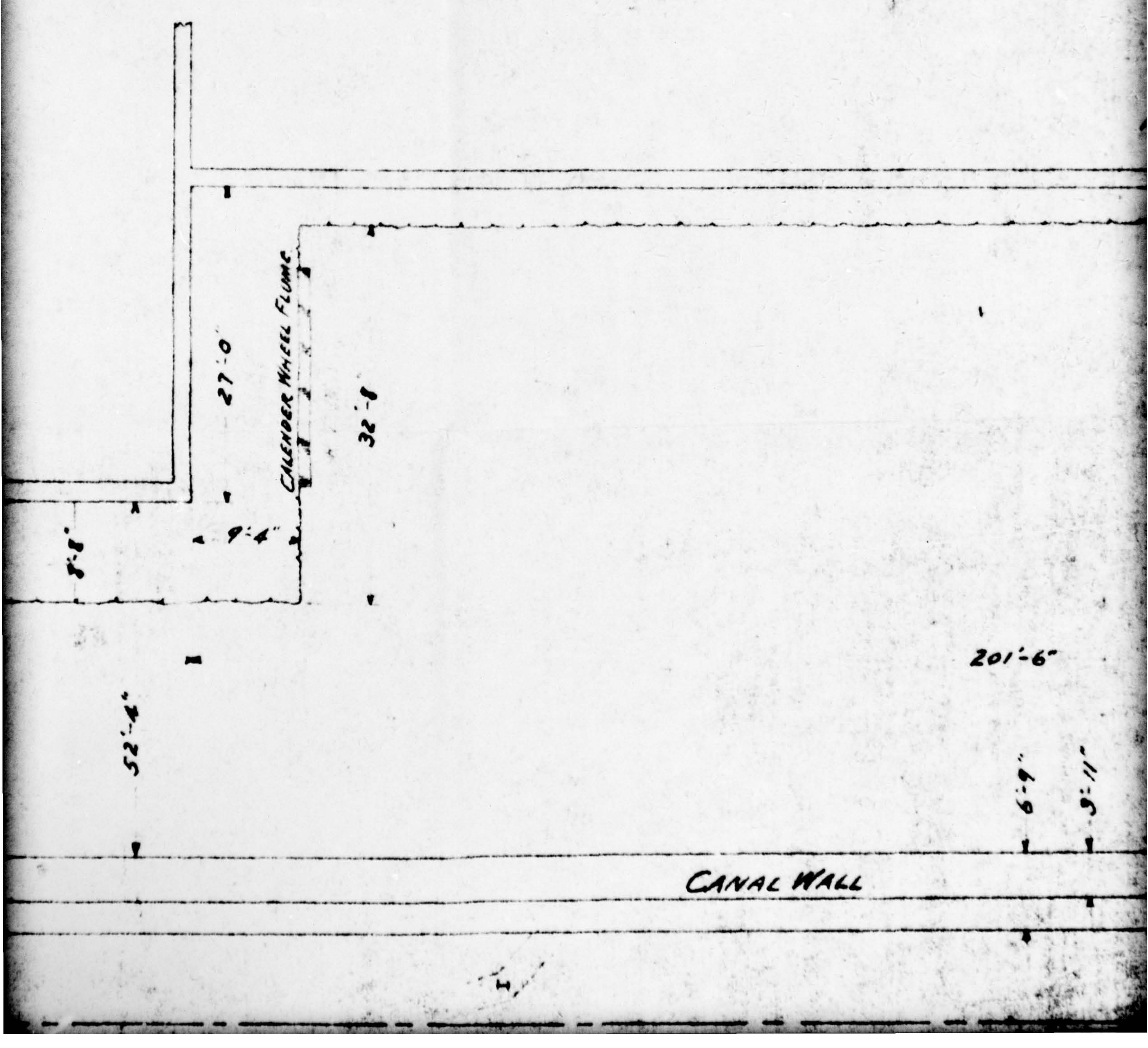
Scale $\frac{1}{32}$" = 12"   Date MAY 25, 1925

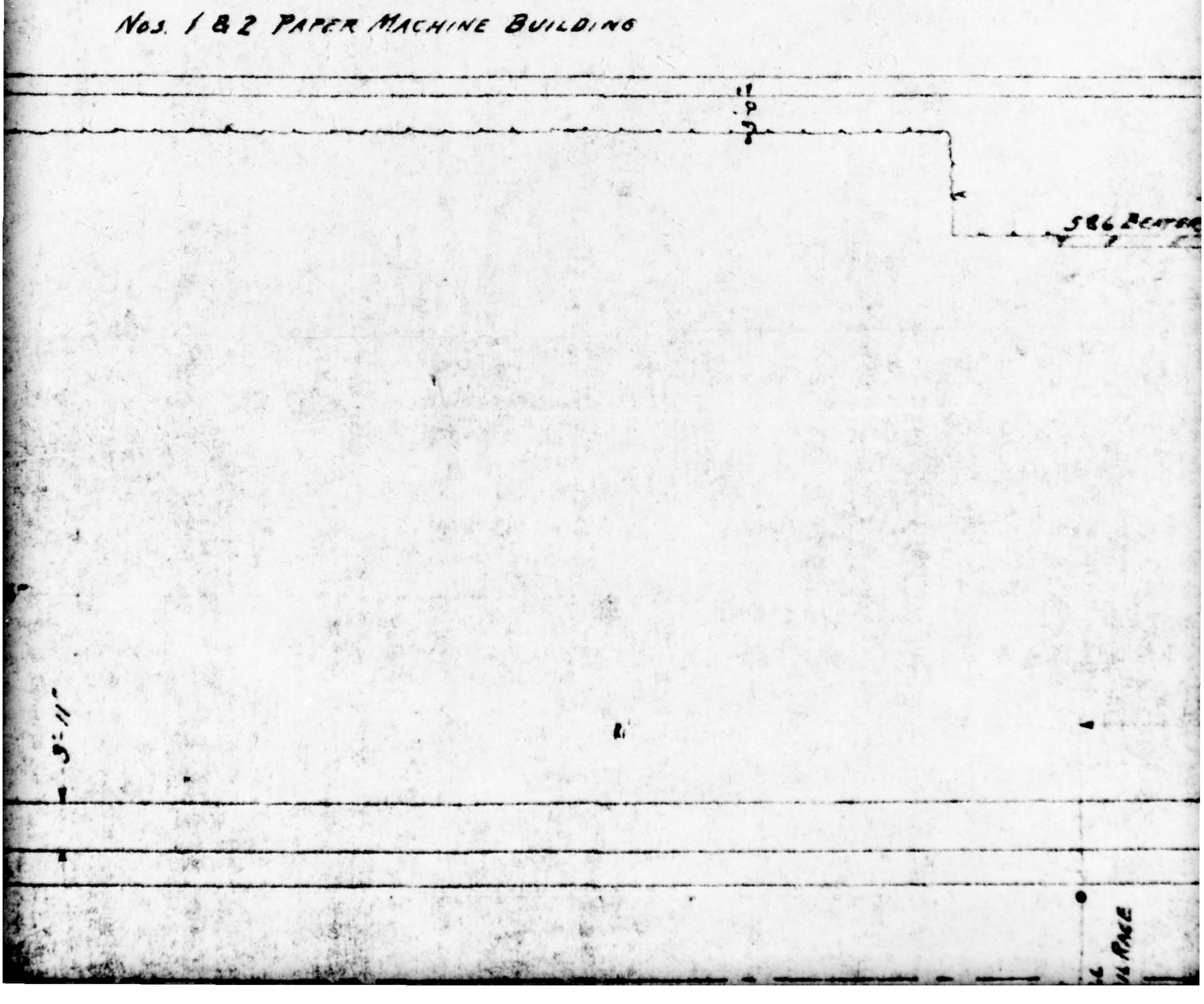
Drawn by WB   No. M 3275

9

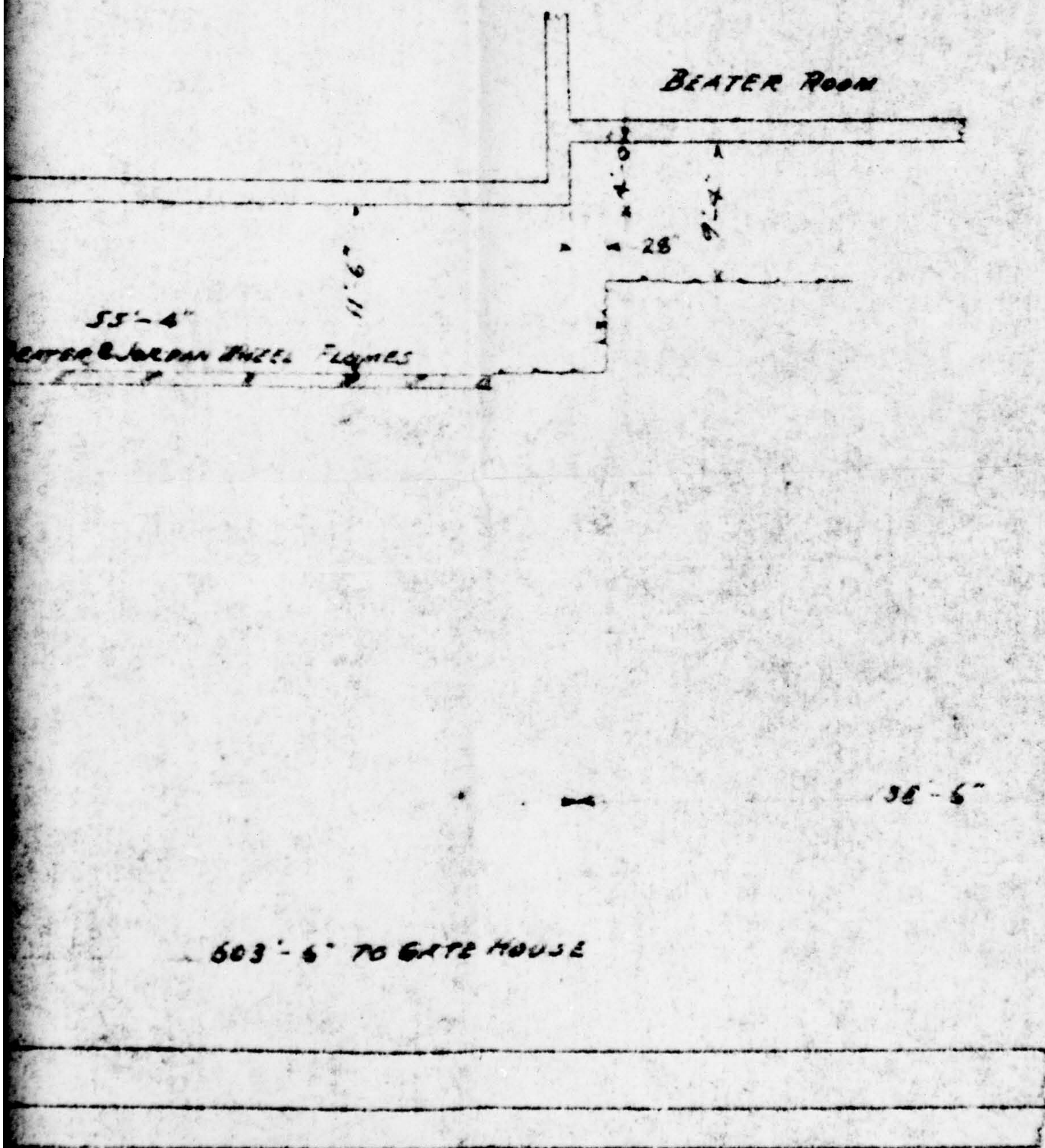
1

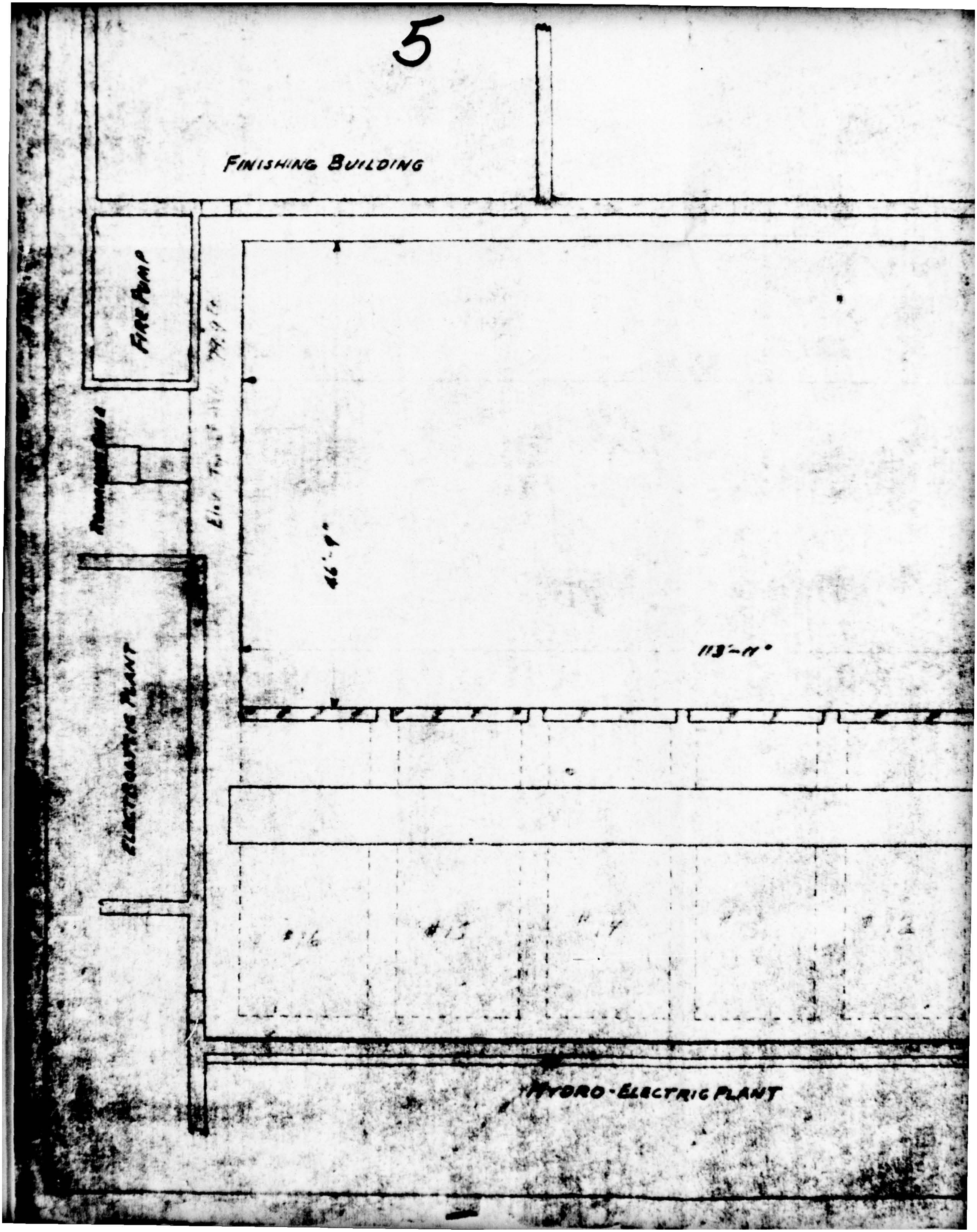

CUTTER BUILDING
CALENDER BUILDING
51'-5"
D.C. POWER & LIGHT WHEEL FLUMES
201'-6"


2
CALENDER WHEEL FLUME
27'-0"
32'-8"
9'-4"
8'-8"
52'-4"
201'-6"
6'-9"
9'-11"
CANAL WALL

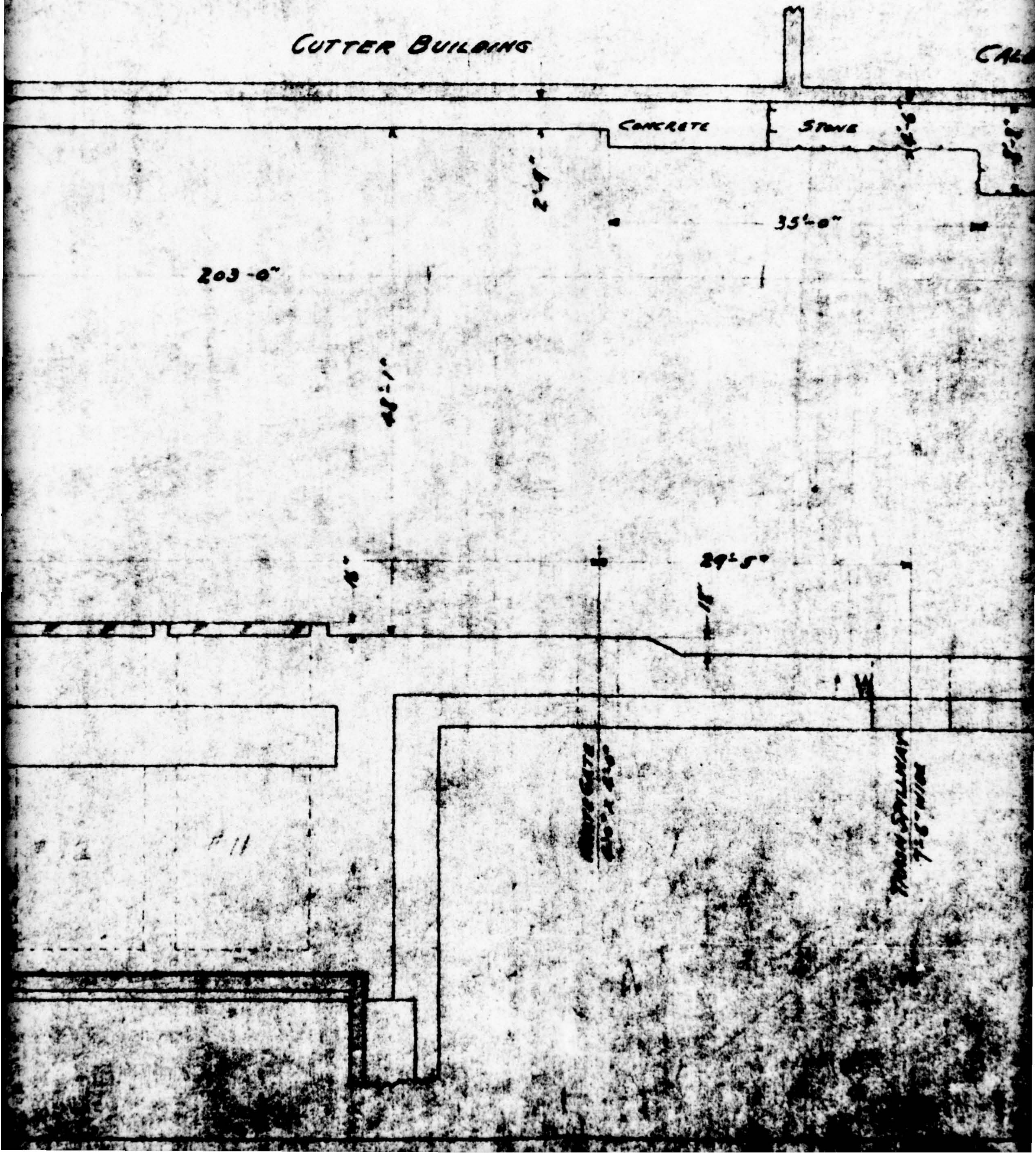

NOS. 1 & 2 PAPER MACHINE BUILDING
9'-11"

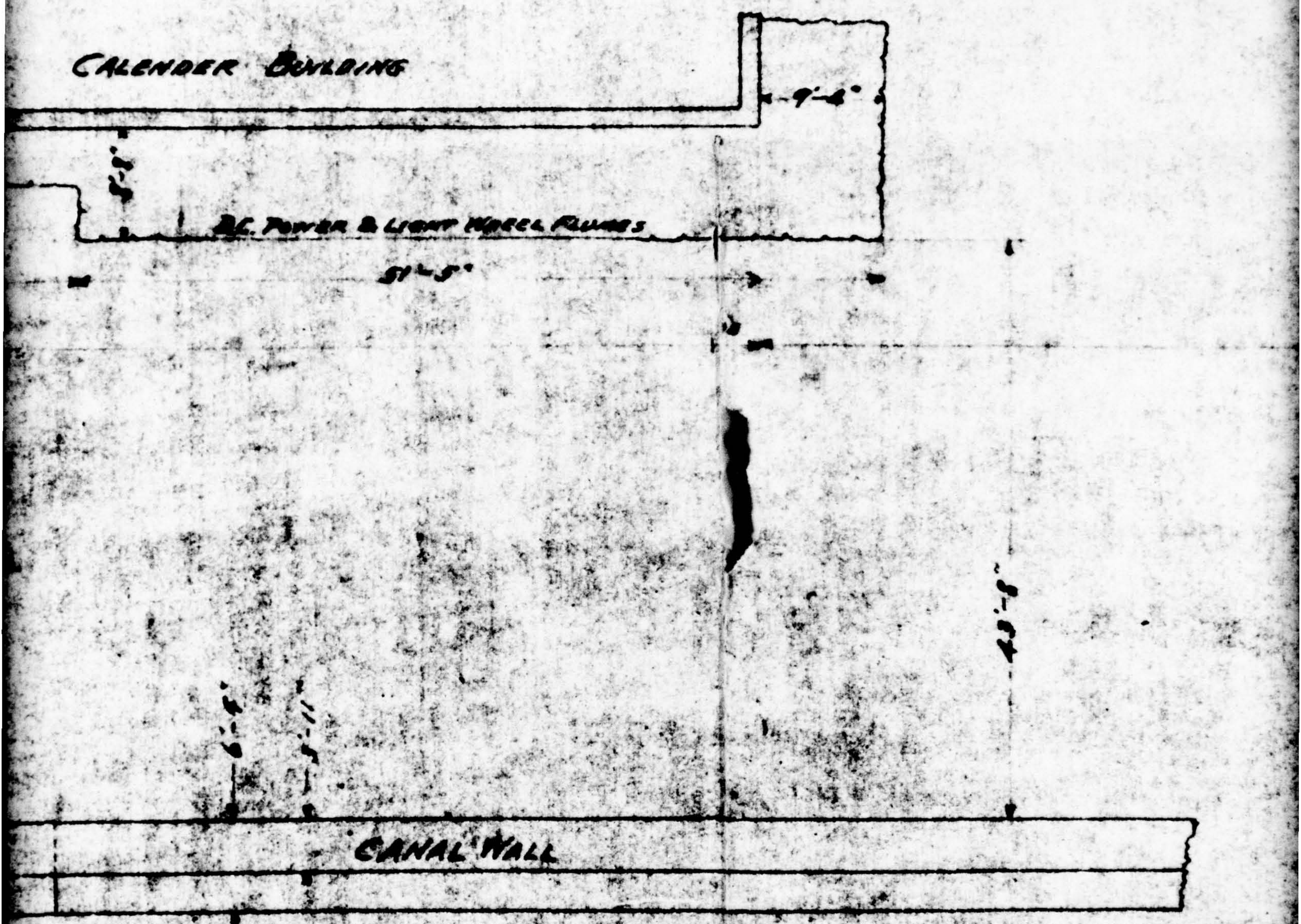
4

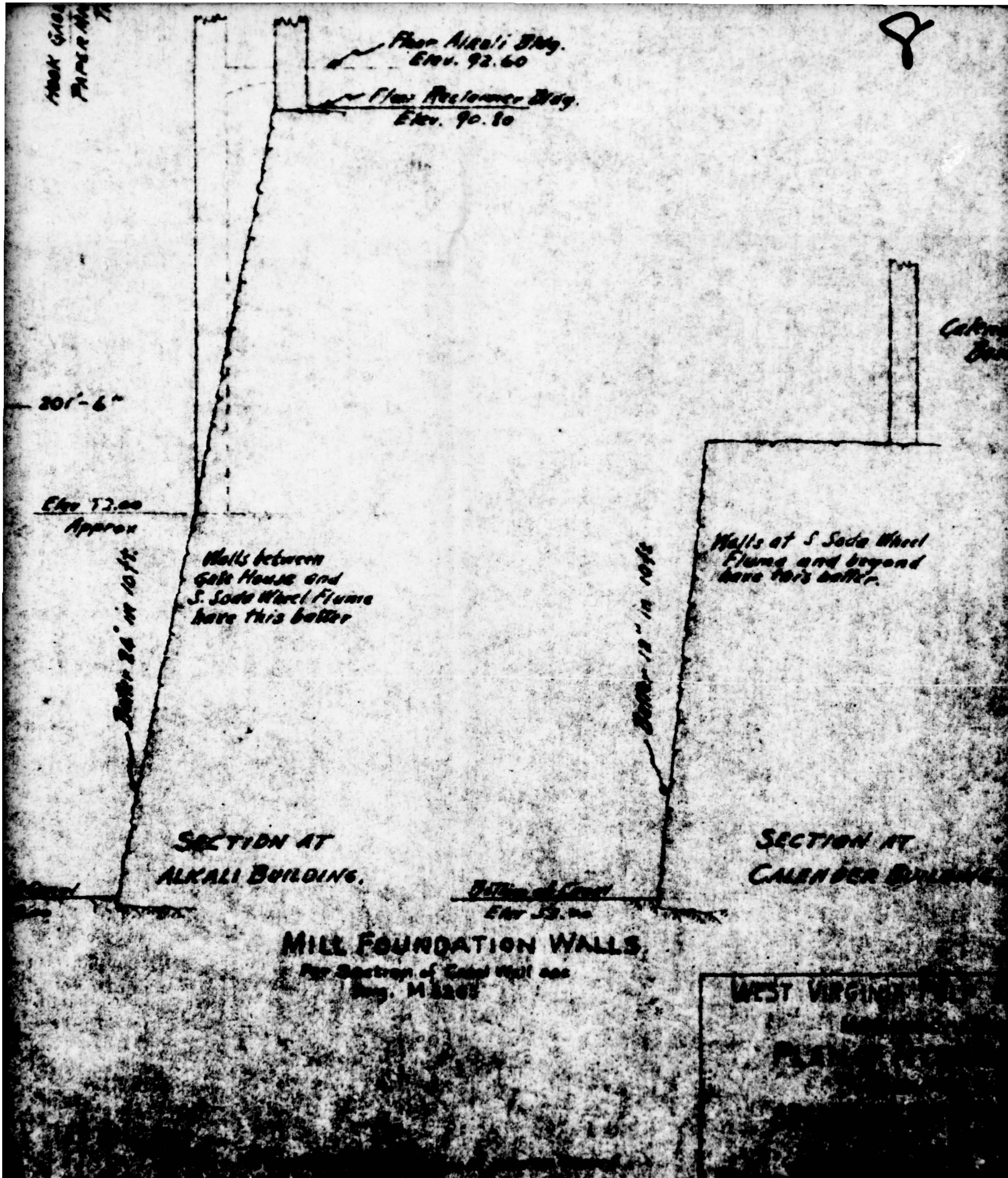


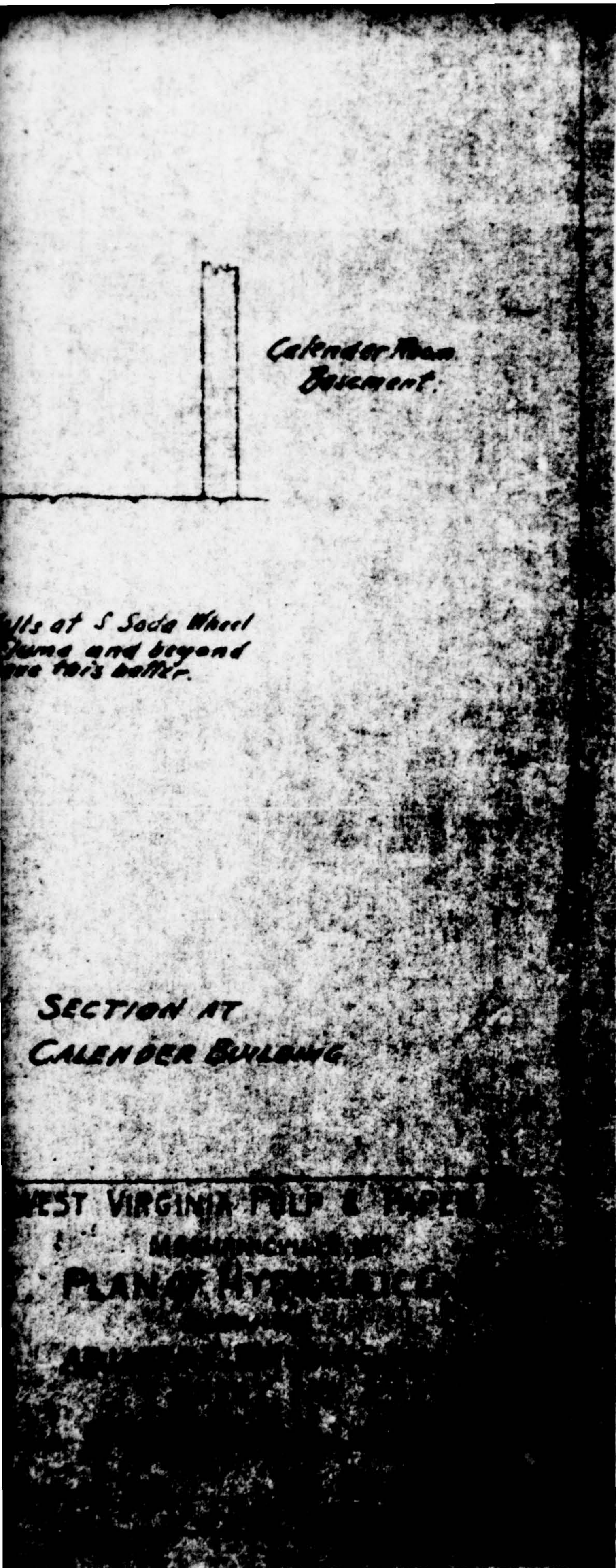

FINISHING BUILDING
FIRE PUMP
46'-9"
113'-11"
ELECTROLYTIC PLANT
HYDRO-ELECTRIC PLANT

6



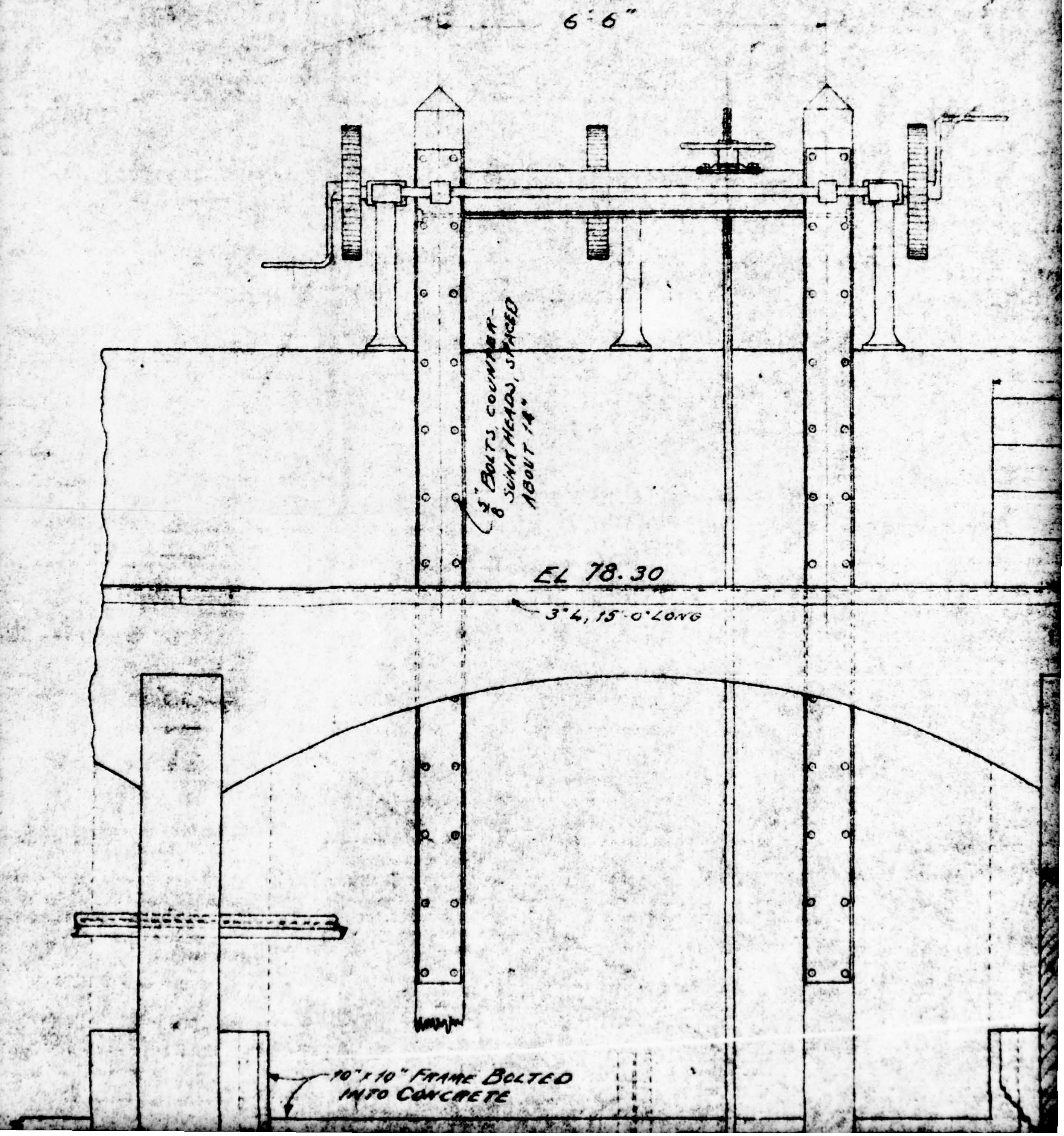

CALENDER BUILDING
9'-8"
D.C. POWER & LIGHT WHEEL FLUMES
51'-5"
CANAL WALL


8
Floor Alkali Bldg.
Elev. 92.60
Floor Reclaimer Bldg.
Elev. 90.80
201'-6"
Elev 72.00
Approx
Batter 24" in 10 ft.
Walls between
Gate House and
S. Soda Wheel Flume
have this batter
Walls at S Soda Wheel
Flume and beyond
have this batter.
Batter 12" in 10 ft
SECTION AT
ALKALI BUILDING.
SECTION AT
CALENDER
MILL FOUNDATION WALLS



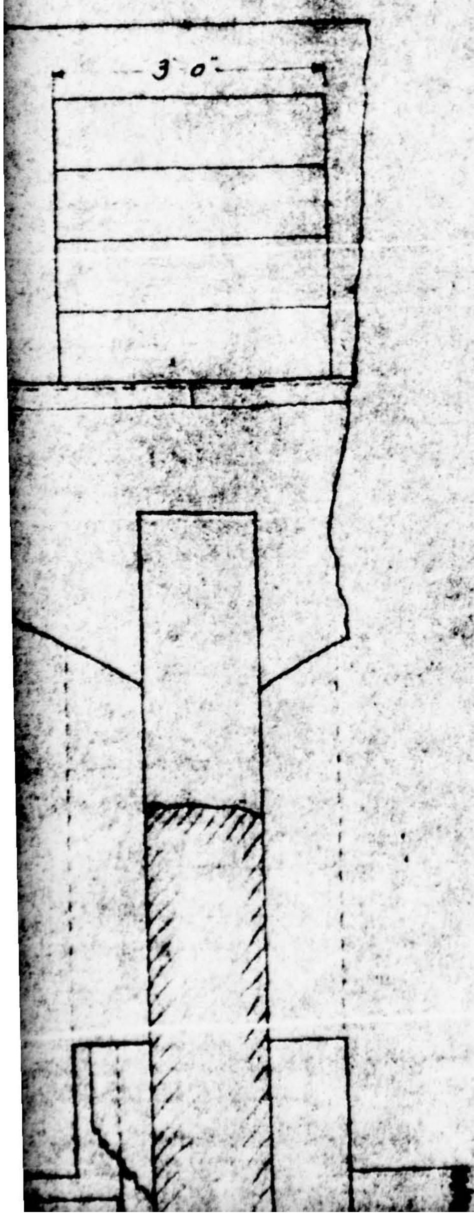
Calender Room
Basement.
lls at S Soda Wheel
Flume and beyond
SECTION AT
CALENDER BUILDING
WEST VIRGINIA PULP & PAPER

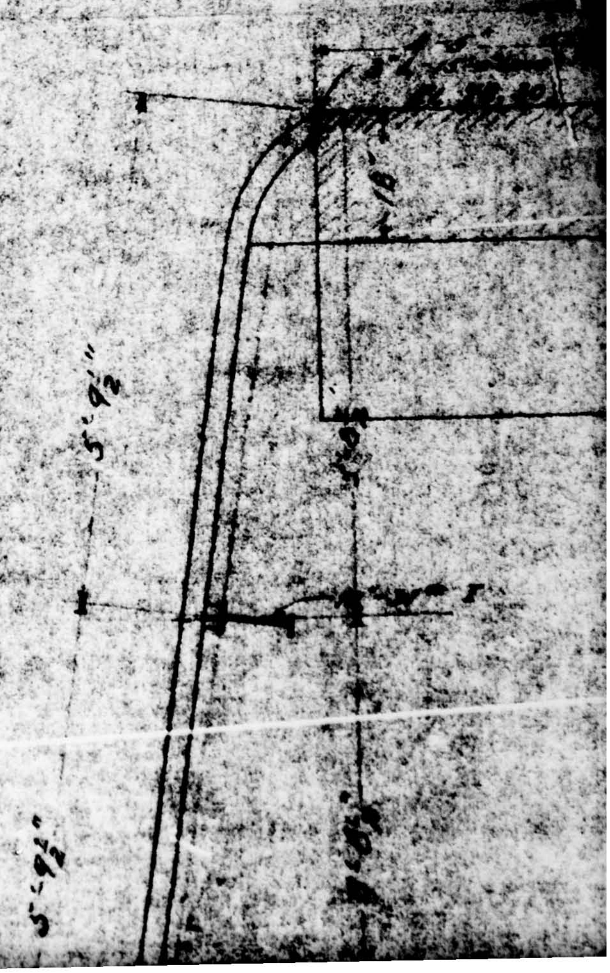
1



6'-6"
5/8" BOLTS, COUNTER-SUNK HEADS, SPACED ABOUT 14"
EL 78.30
3" L, 15'-0" LONG
10" x 10" FRAME BOLTED INTO CONCRETE

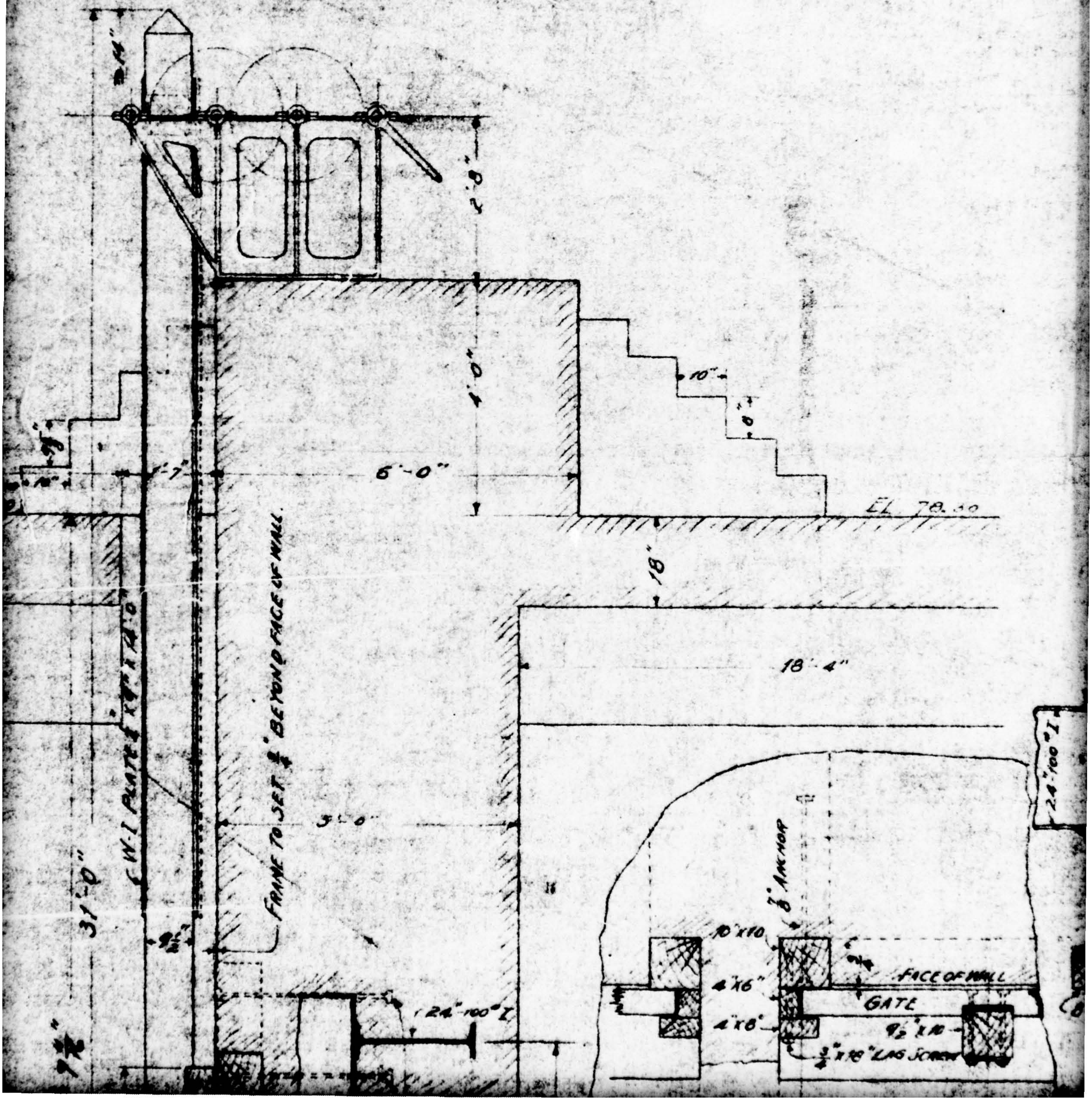
2



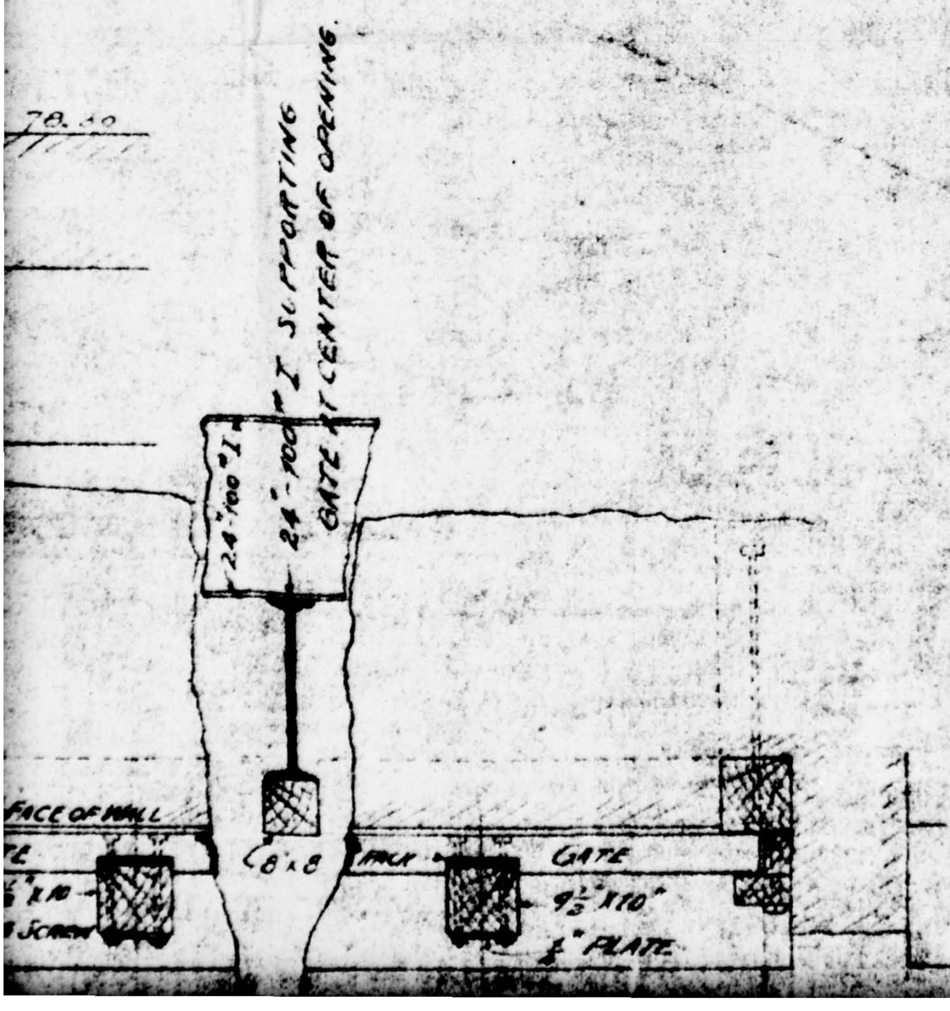


3

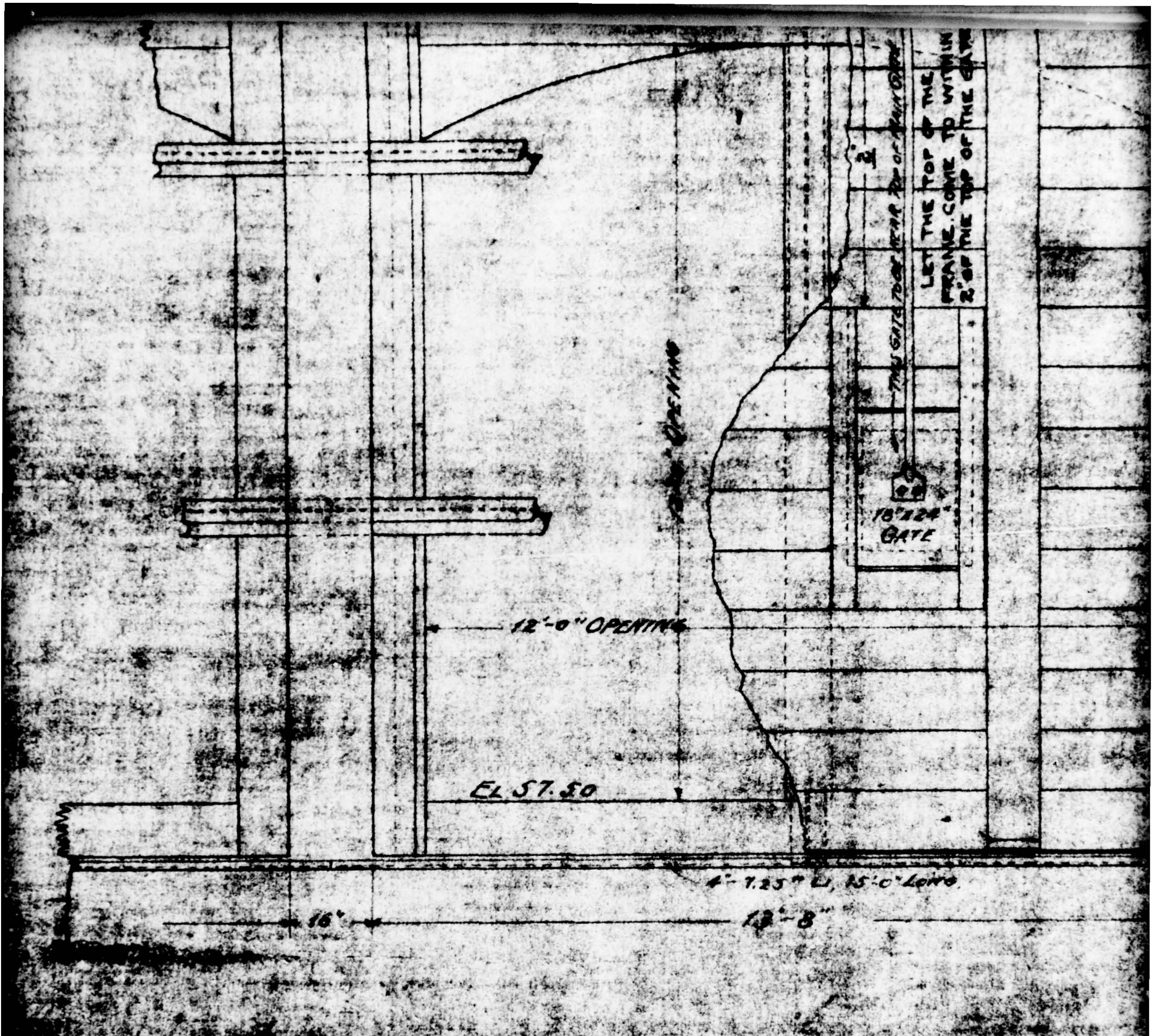
FOR LOCATION OF ANCHOR BOLTS FOR GATE RIG SEE LEFFEL DRAWING No. 13913.

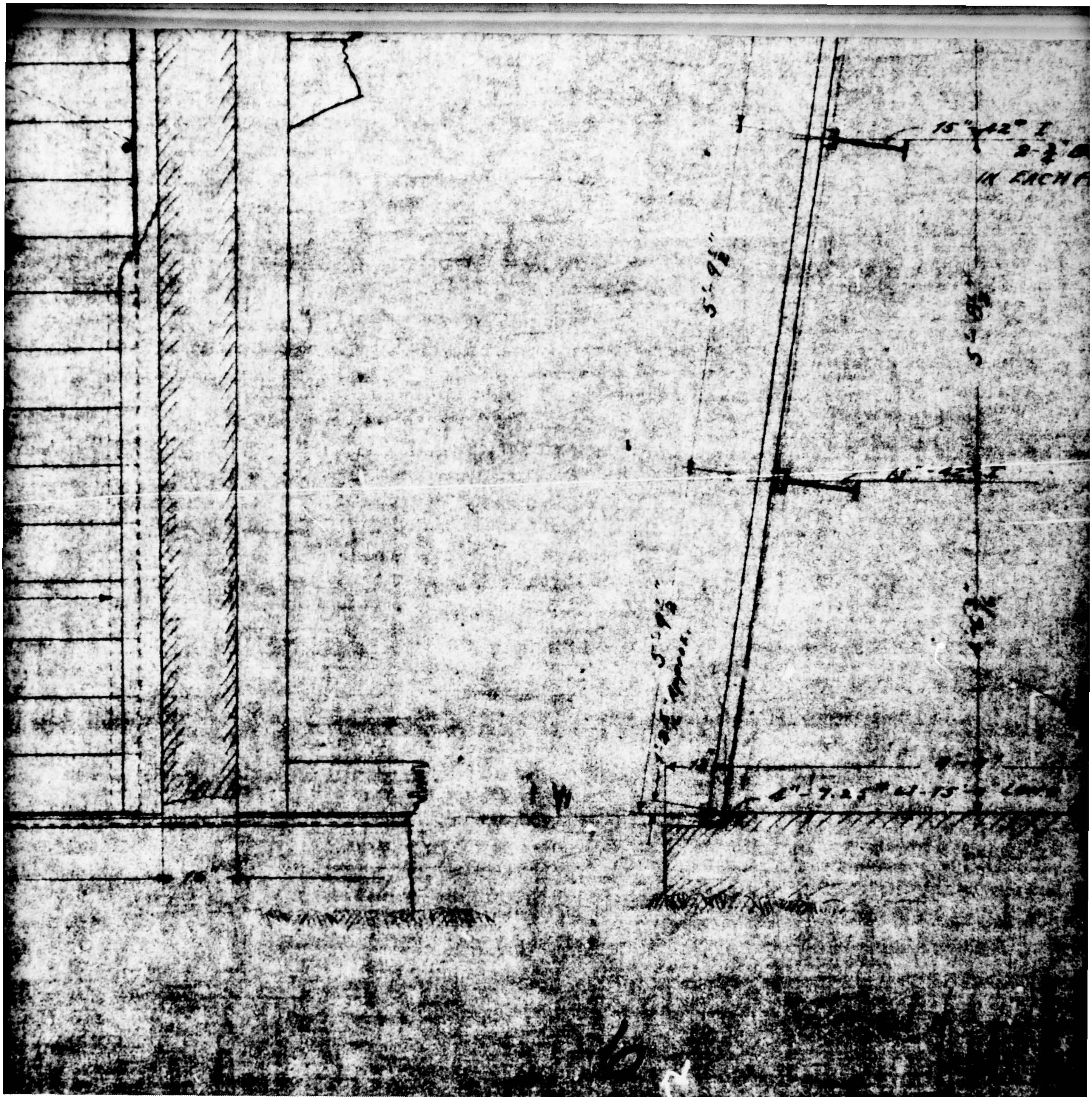


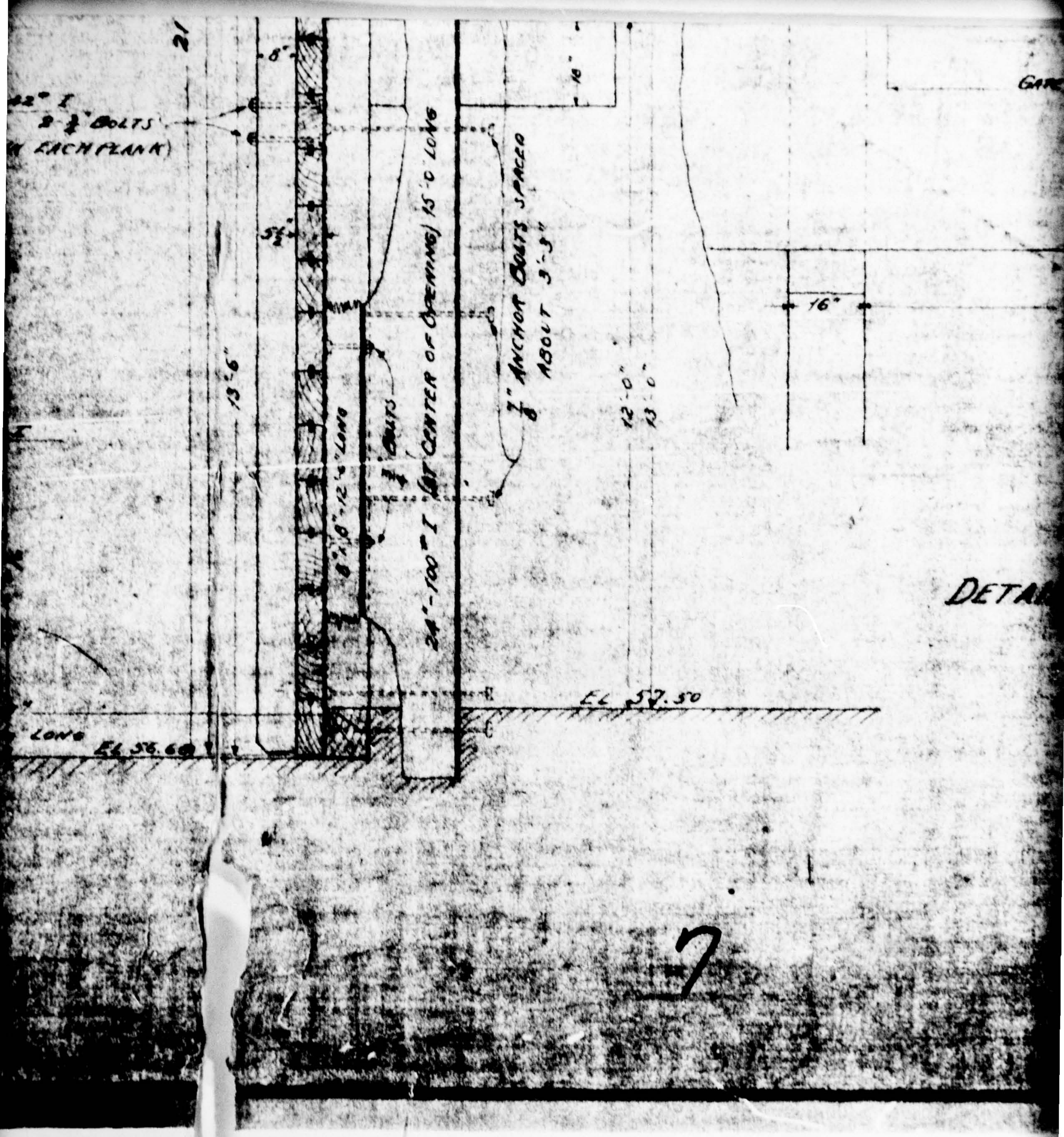
4



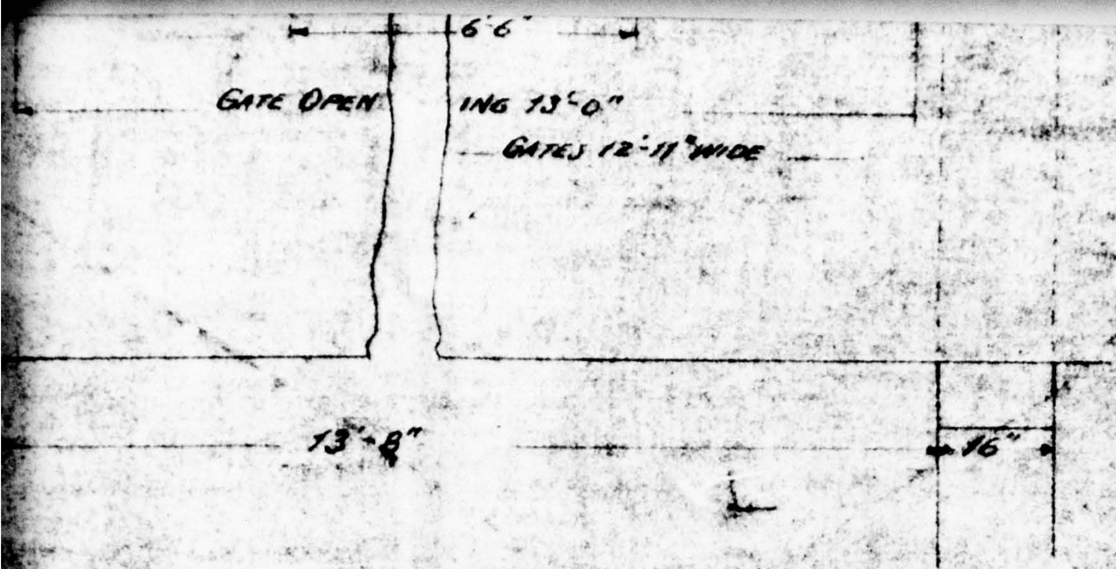
24" 100# I
24" 100# I SUPPORTING
GATE AT CENTER OF OPENING.
FACE OF WALL
8 x 8
GATE
9½" X 10"
PLATE


12'-0" OPENING
EL 57.50
16"
13'-8"
18"x24"
GATE
LET THE TOP OF THE
FRAME COME TO WITHIN
2" OF THE TOP OF THE GATE


15" 42# I
IN EACH
5'-9½"



2-¾" BOLTS
EACH PLANK)
8"
5½"
13'-6"
8"x8"-12'-6" LONG
¾" BOLTS
24"-100# I (AT CENTER OF OPENING) 15'-0" LONG
⅞" ANCHOR BOLTS SPACED ABOUT 3'-3"
10"
12'-0"
13'-0"
16"
EL 57.50
EL 56.60
LONG
DETA
7



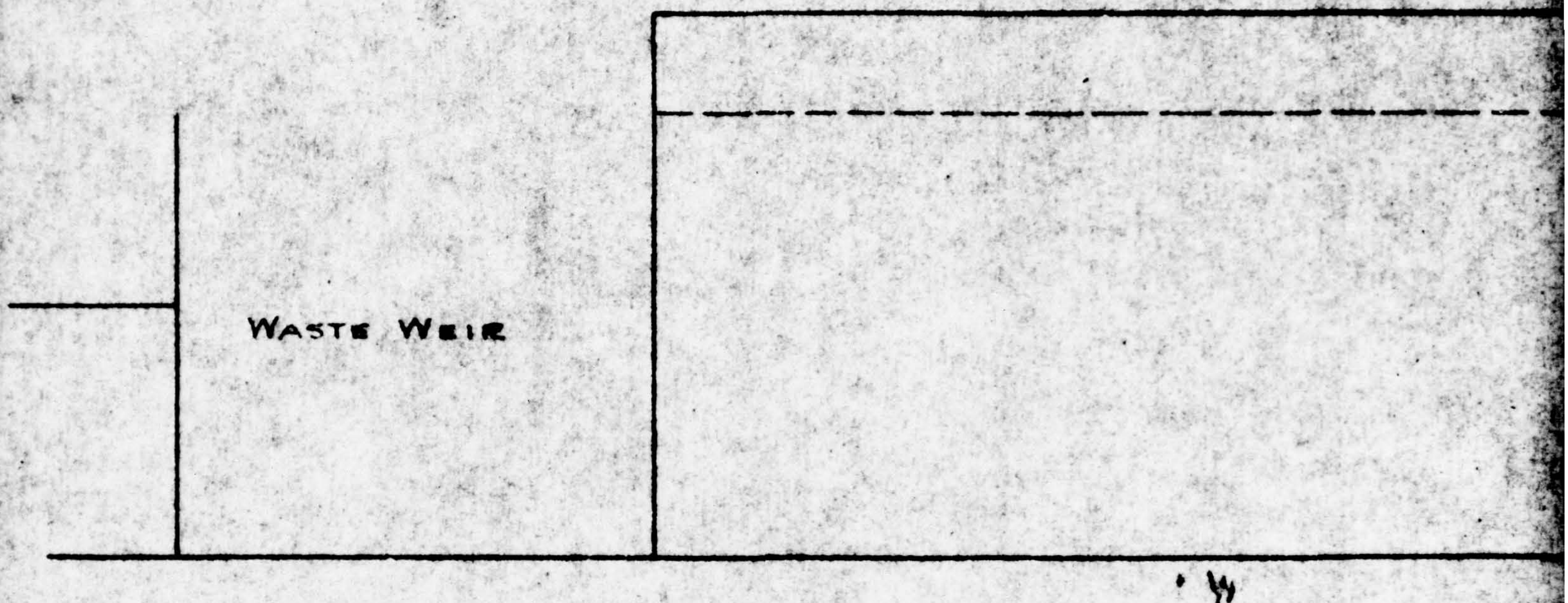
DETAILS OF HEAD GATES – HYDRO-ELECTRIC PLANT.
WEST VIRGINIA PULP & PAPER CO.

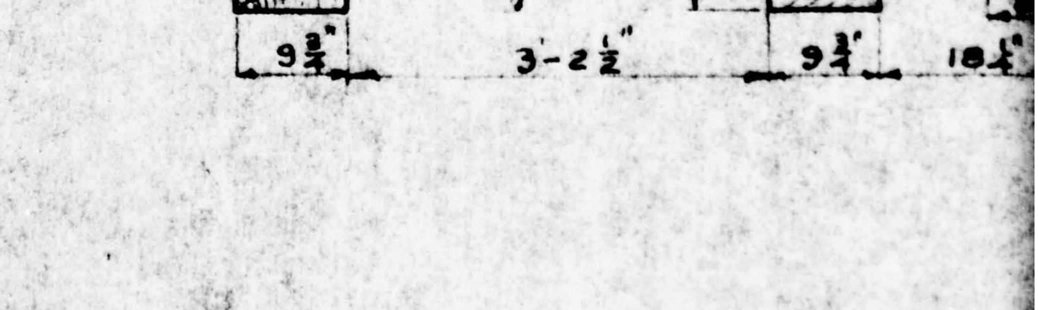
SCALE 1/2" = 1 FT.-0".   MECHANICVILLE, N.Y.

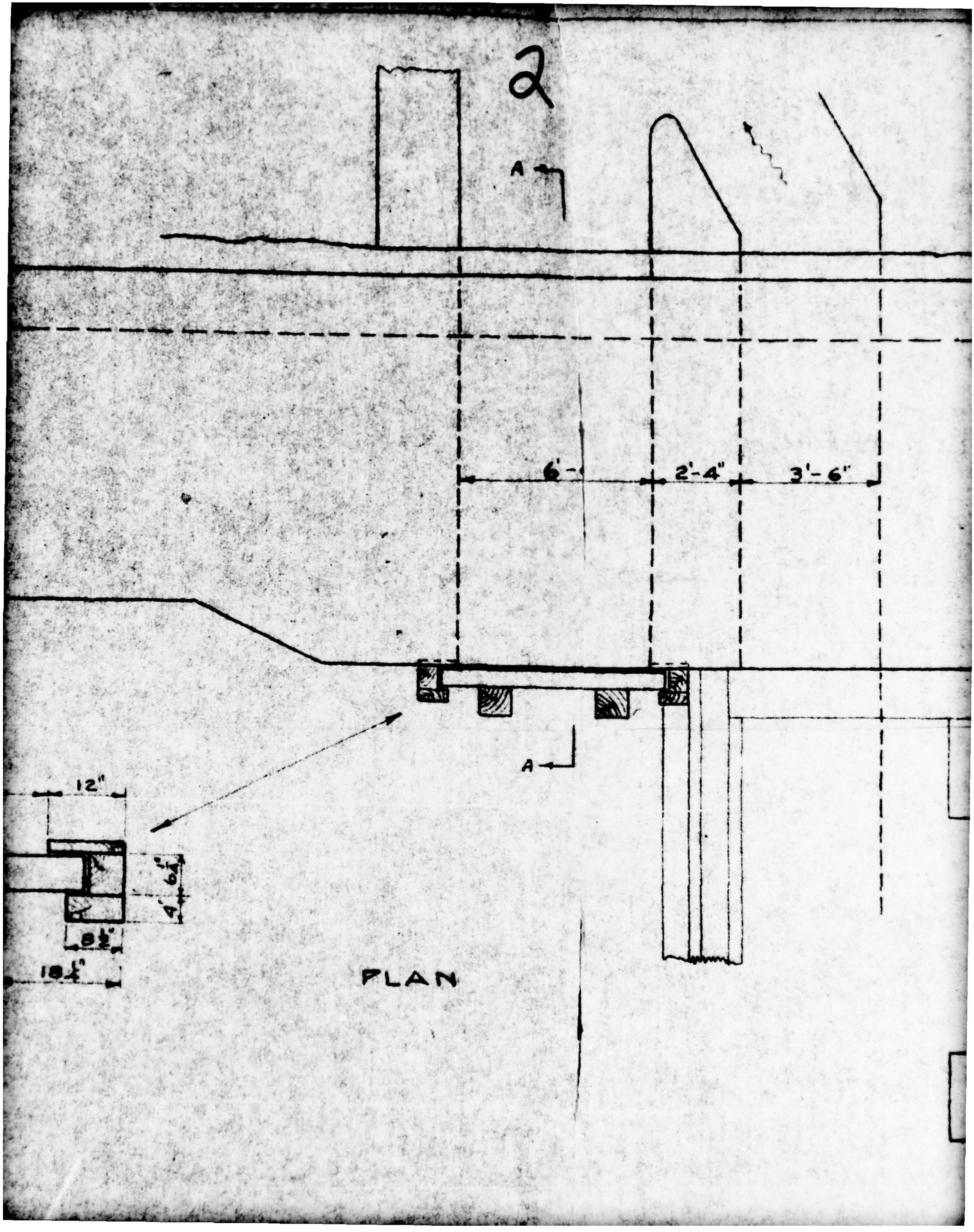
NOV. 16, [illegible]

8

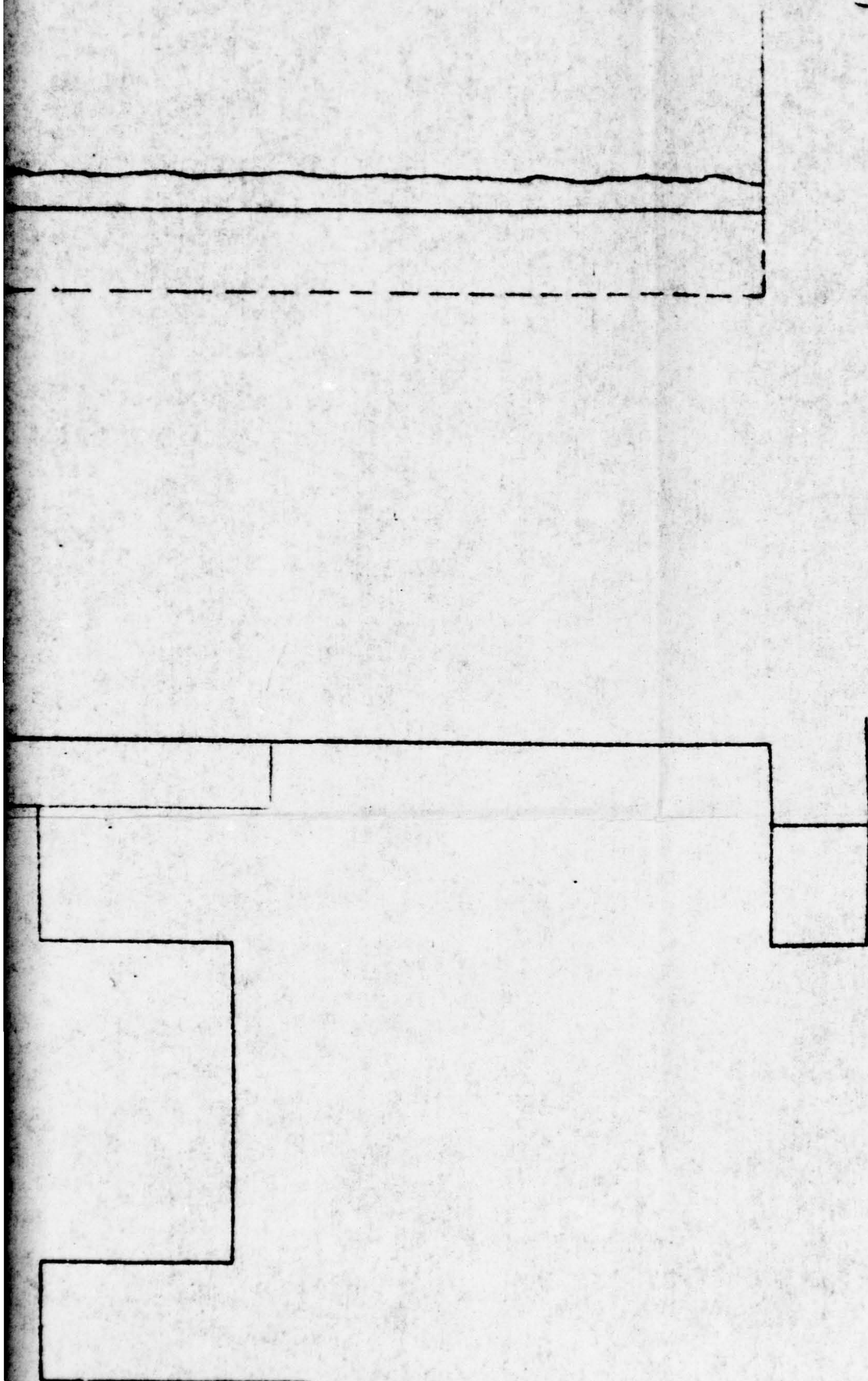
1

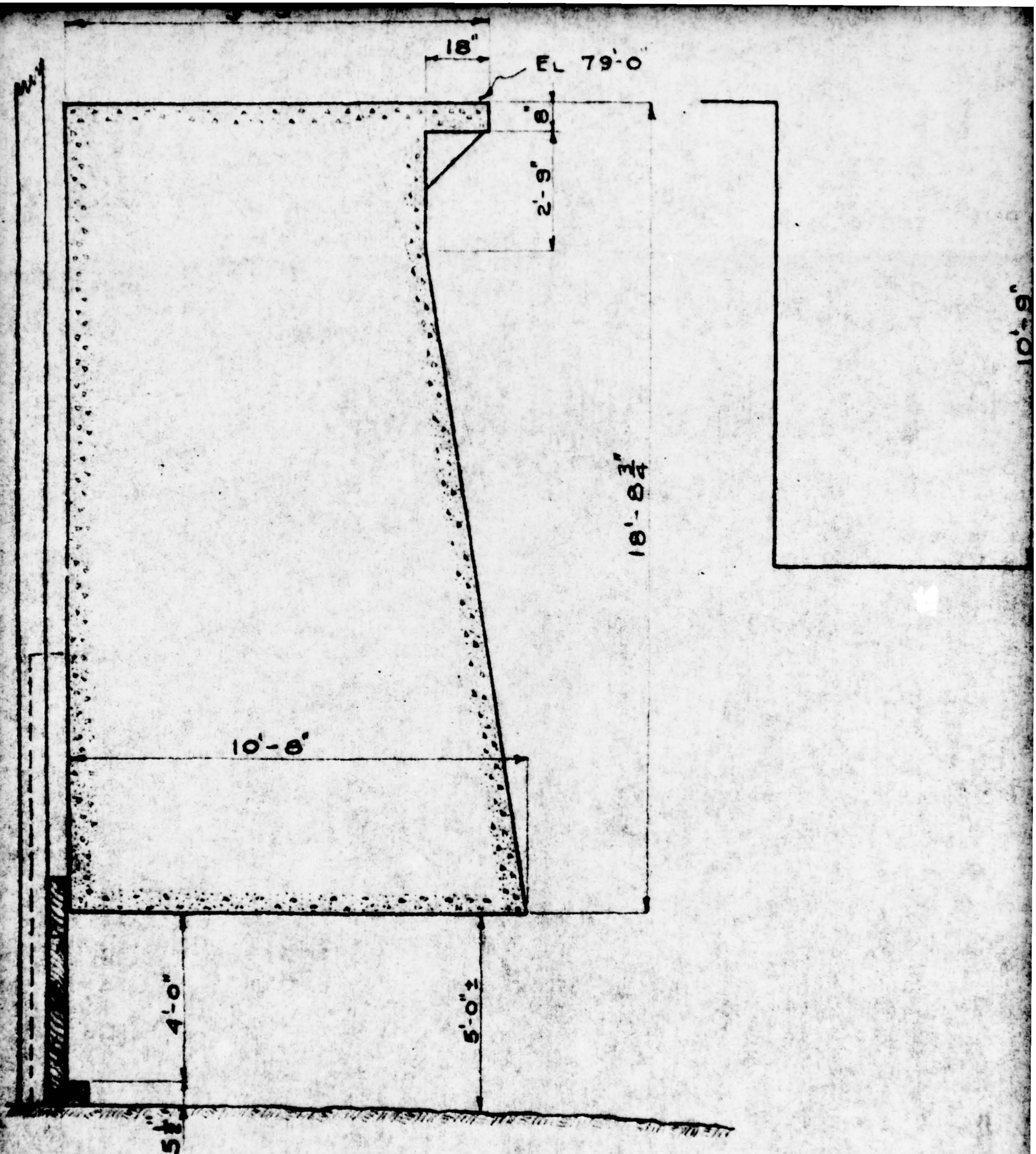





2
A
6'-
2'-4"
3'-6"
A
12"
6¼"
4"
8½"
18¼"
PLAN

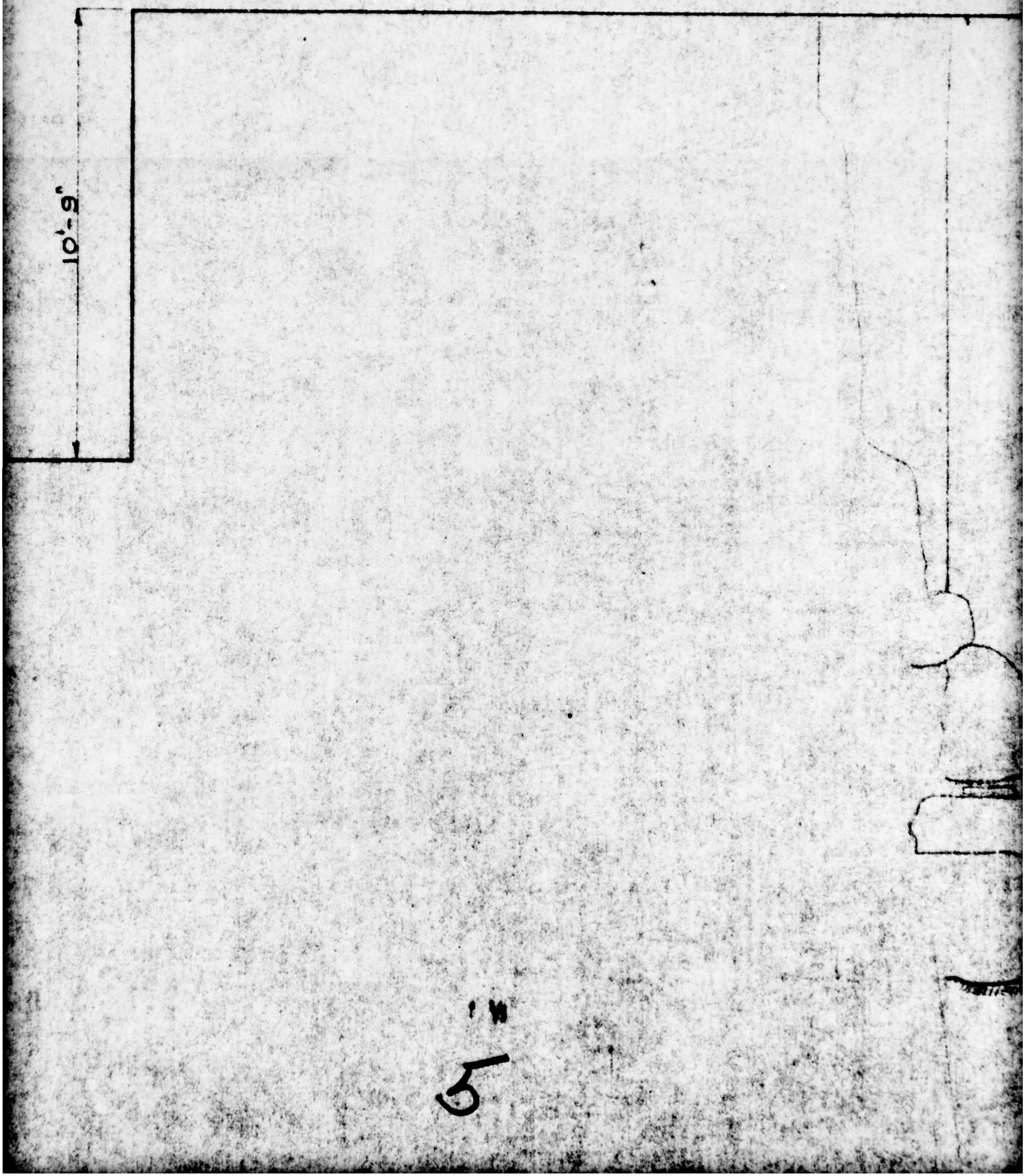
3

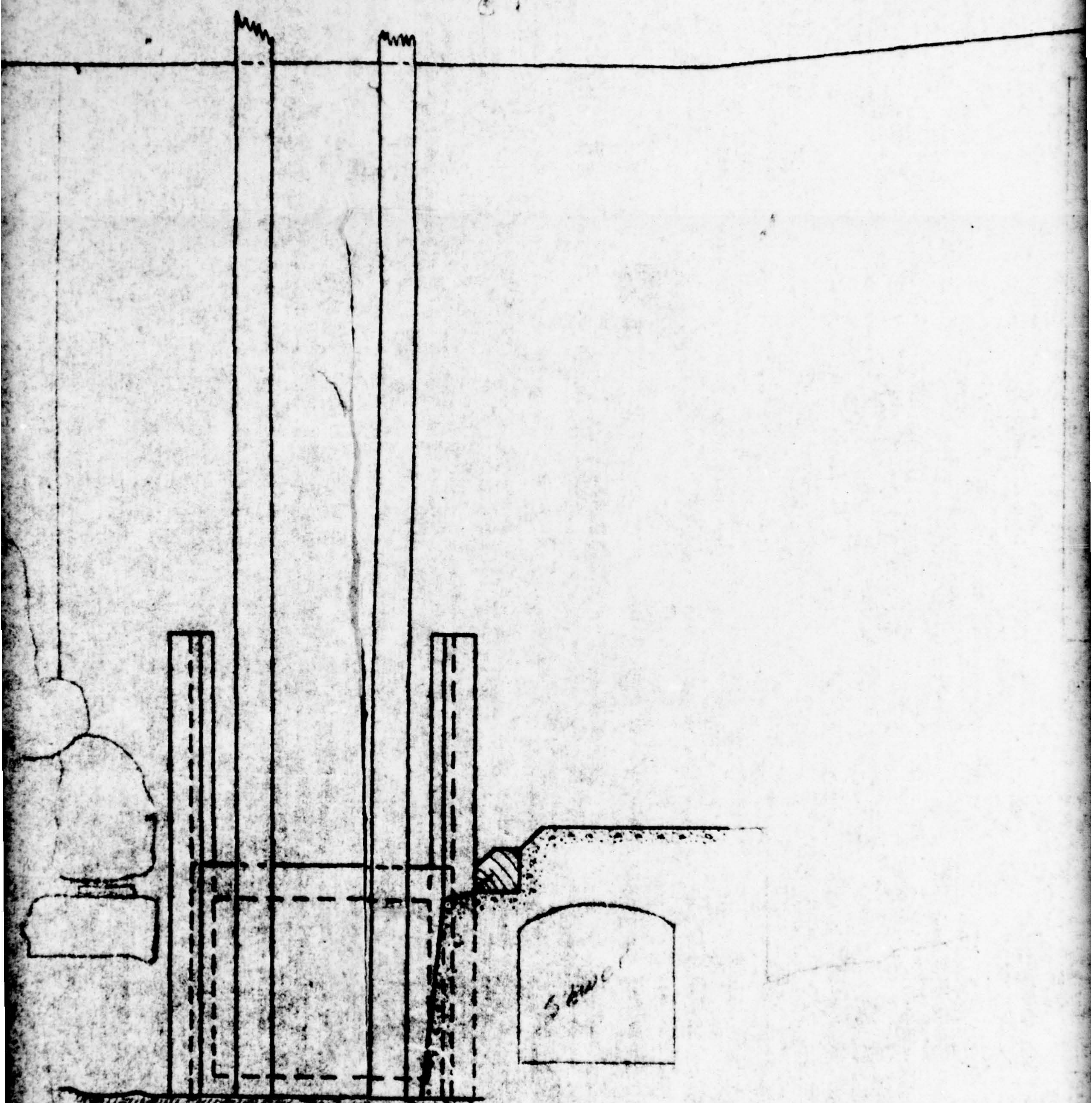




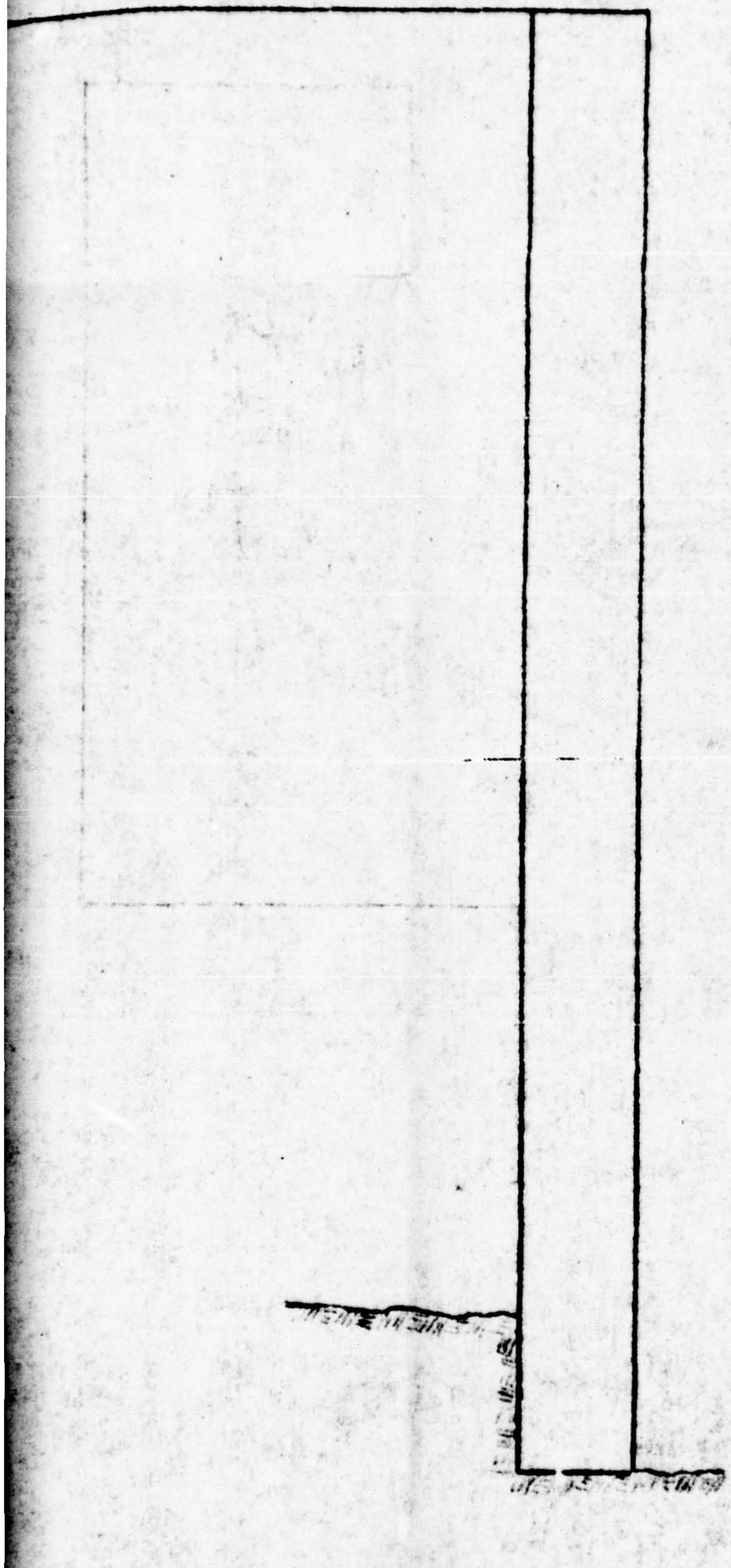
SECTION A-A

H


10'-9"



ELEVATI



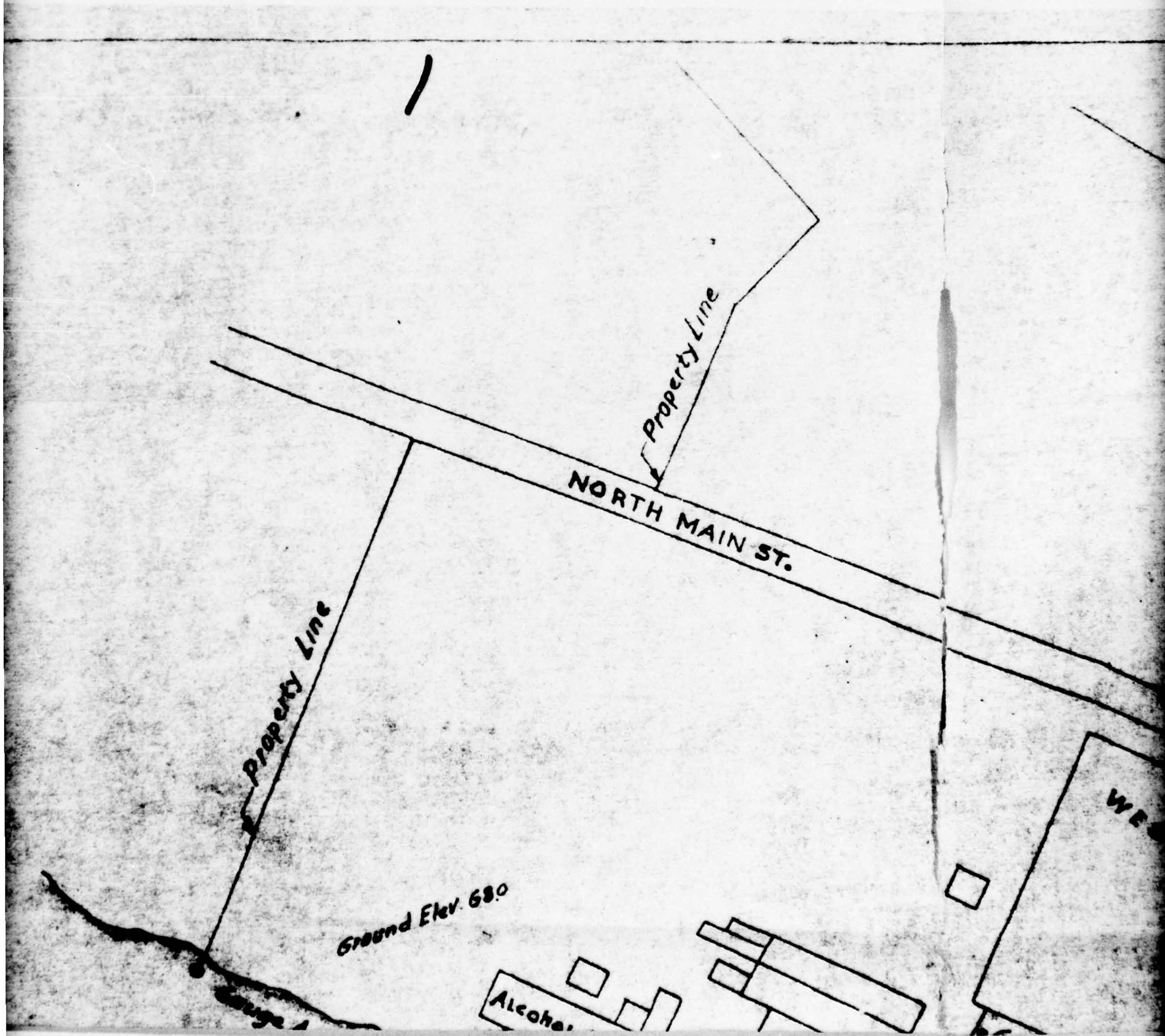
7

WEST VIRGINIA PULP & PAPER CO.

MECHANICVILLE, N. Y.

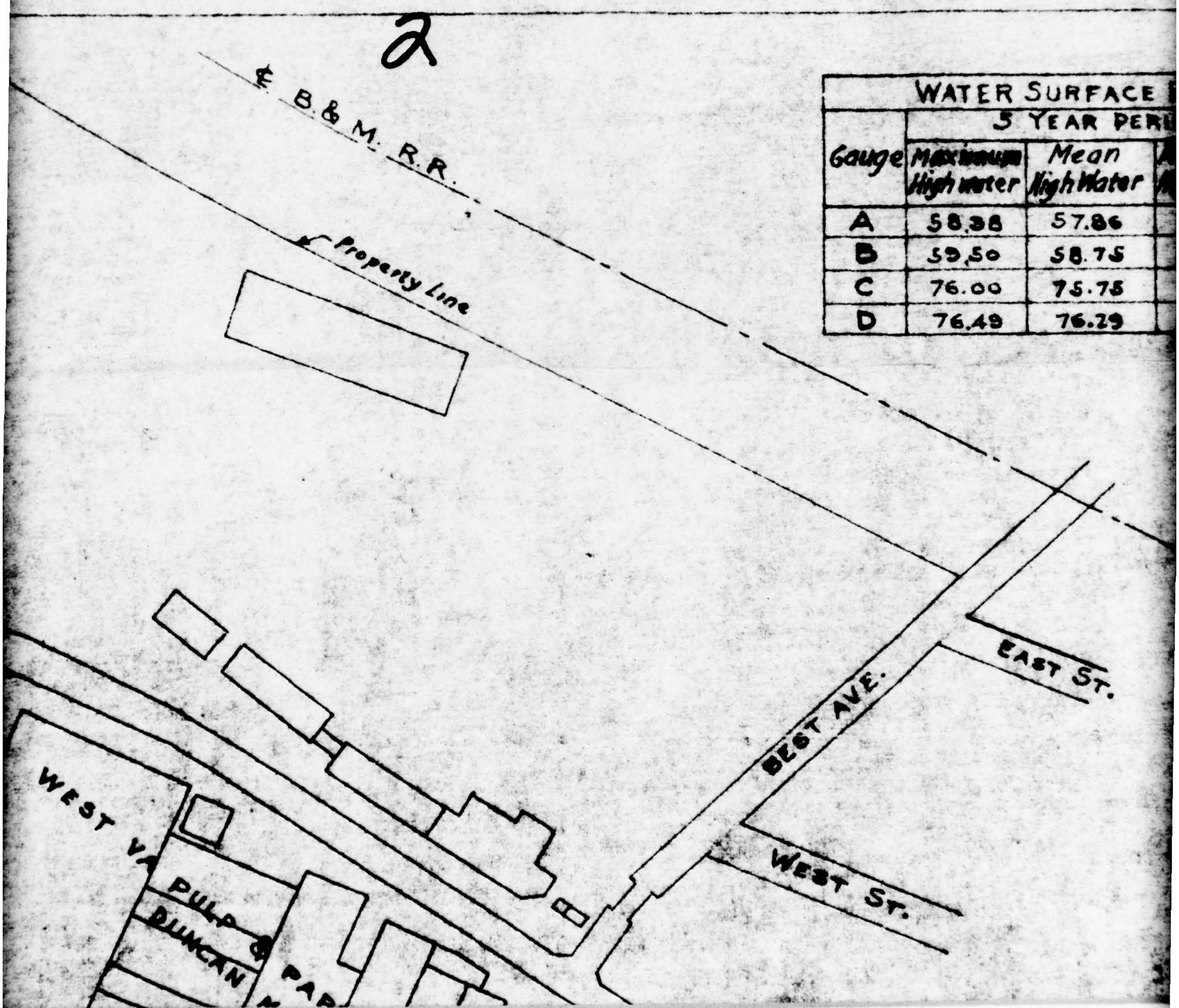
SLUICE GATE
SOUTH END OF FOREBAY

Scale ⅜" = 1 FT. | Date June 19, [illegible]
Drawn by AFW | No M-426[illegible]



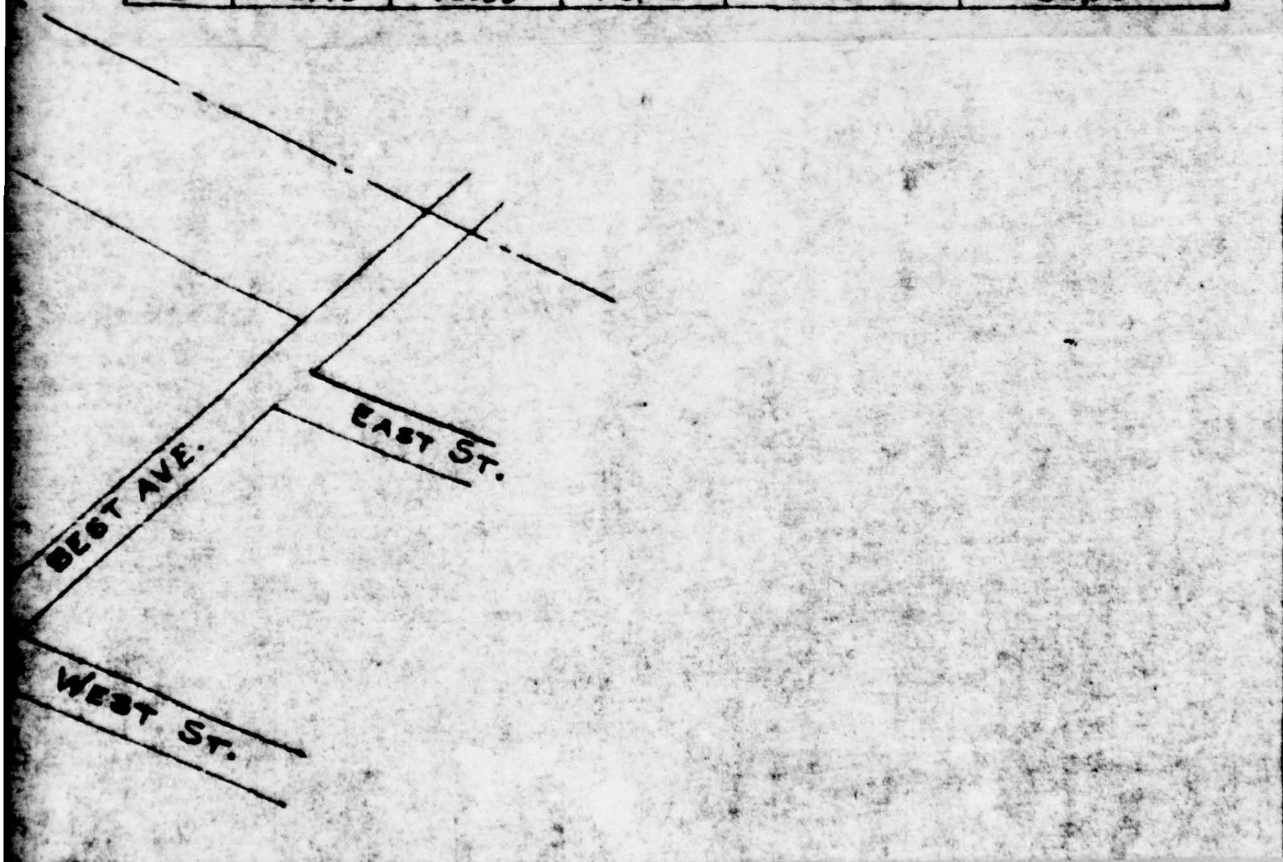
Property Line
NORTH MAIN ST.
Property Line
Ground Elev. 68.0
Alcohol

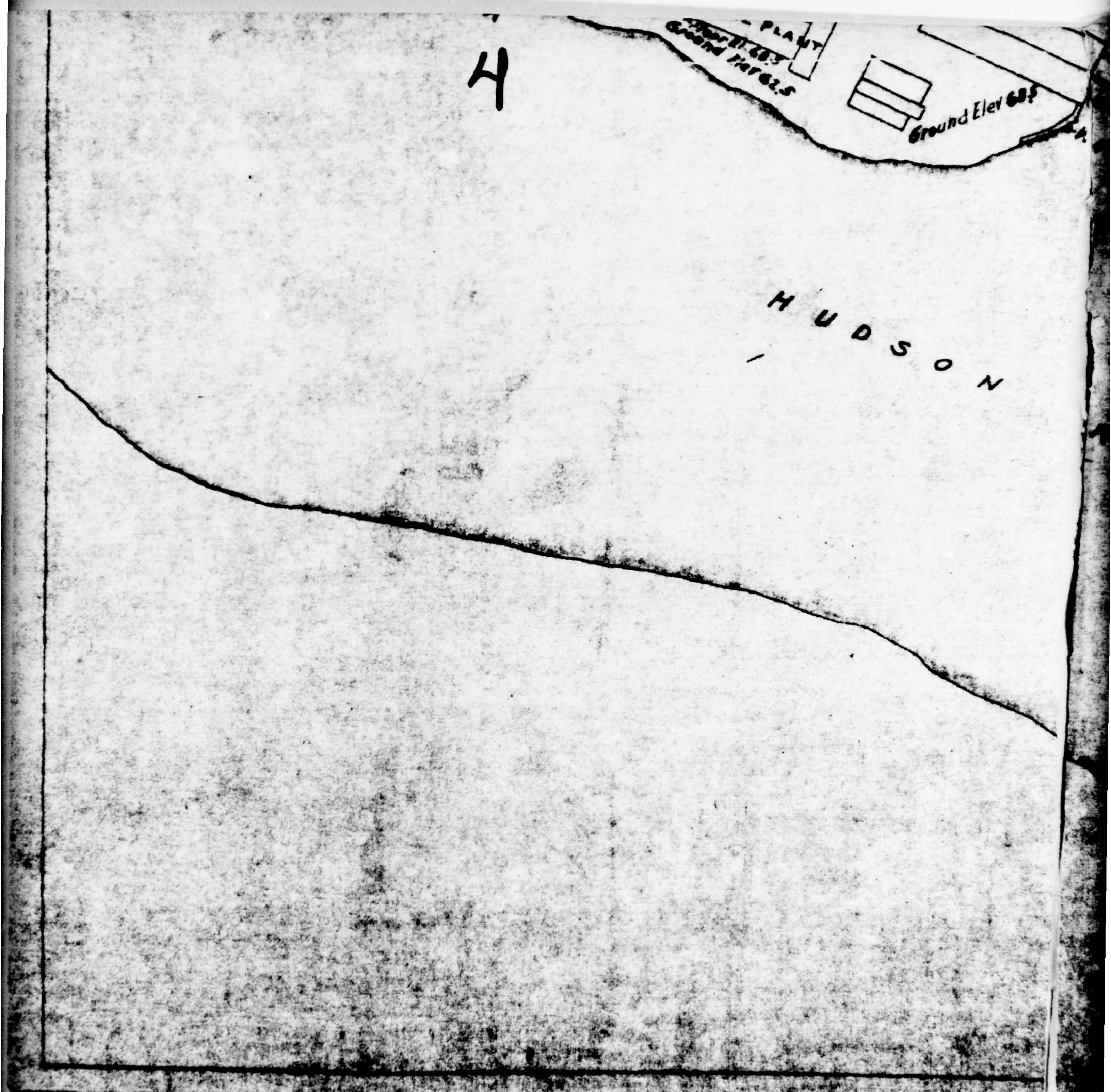
2

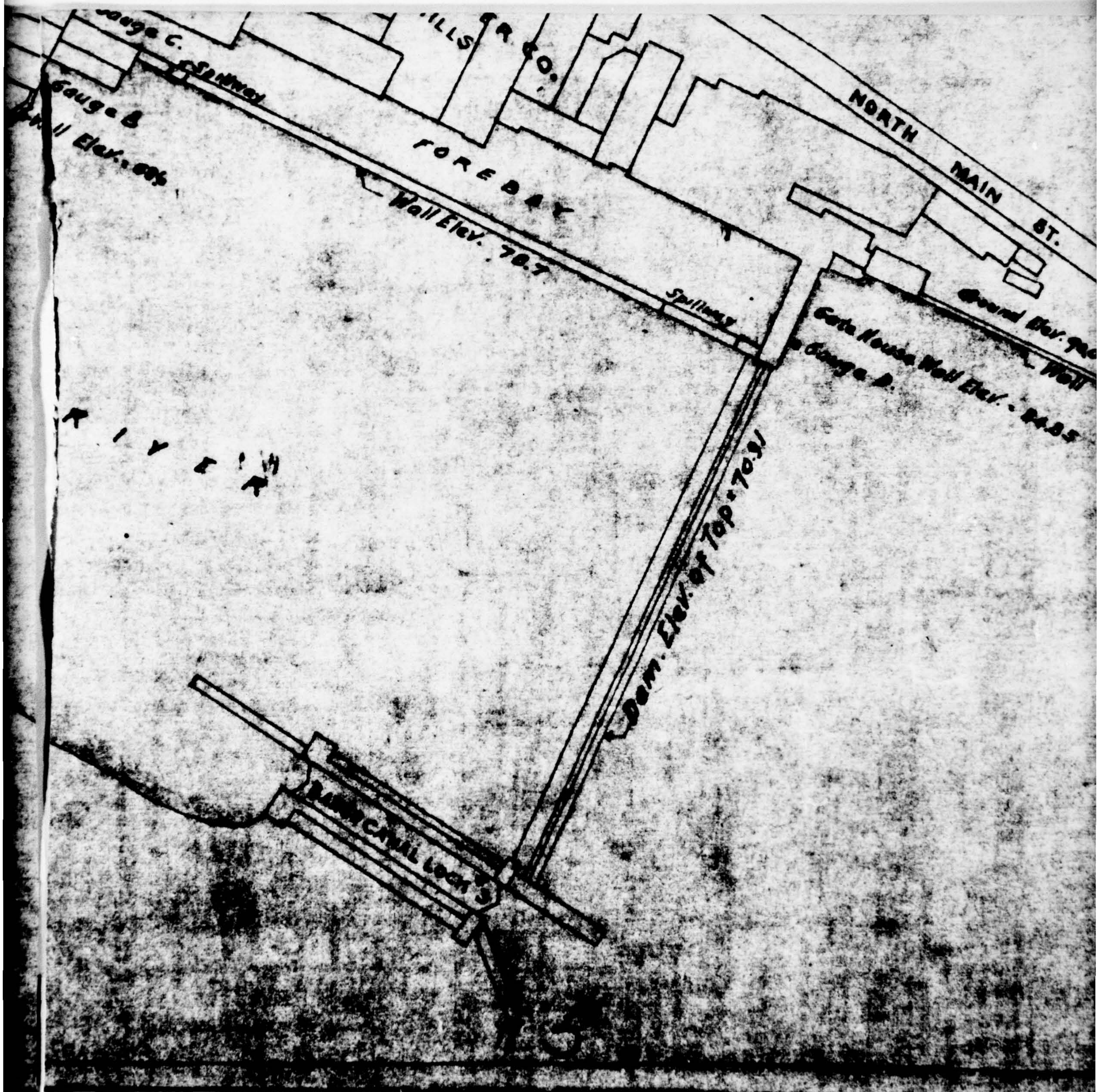


WATER SURFACE		
	5 YEAR PER	
Gauge	Maximum High water	Mean High Water
A	58.38	57.86
B	59.50	58.75
C	76.00	75.75
D	76.49	76.29

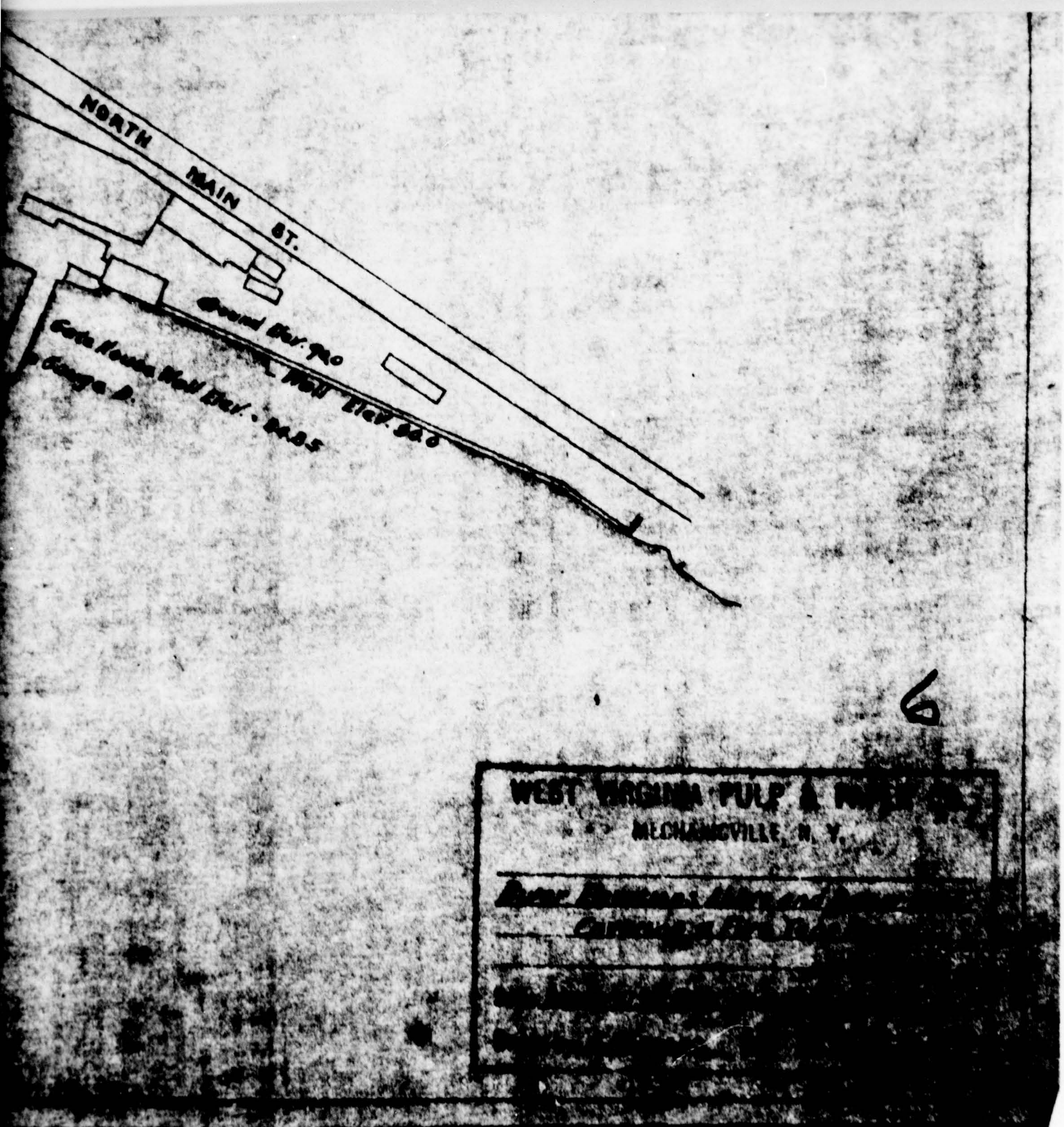
WATER SURFACE ELEVATION AT GAUGES					
	3 YEAR PERIOD TO DATE				Highest Water Recorded March 28 1913
Gauge	Maximum High water	Mean High Water	Minimum High Water	Lowest Water Recorded	
A	58.38	57.86	57.50	49.00	67.50
B	59.50	58.75	58.00	49.00	68.20
C	76.00	75.75	75.50	65.00	76.00
D	76.49	76.29	76.02	66.91	82.88



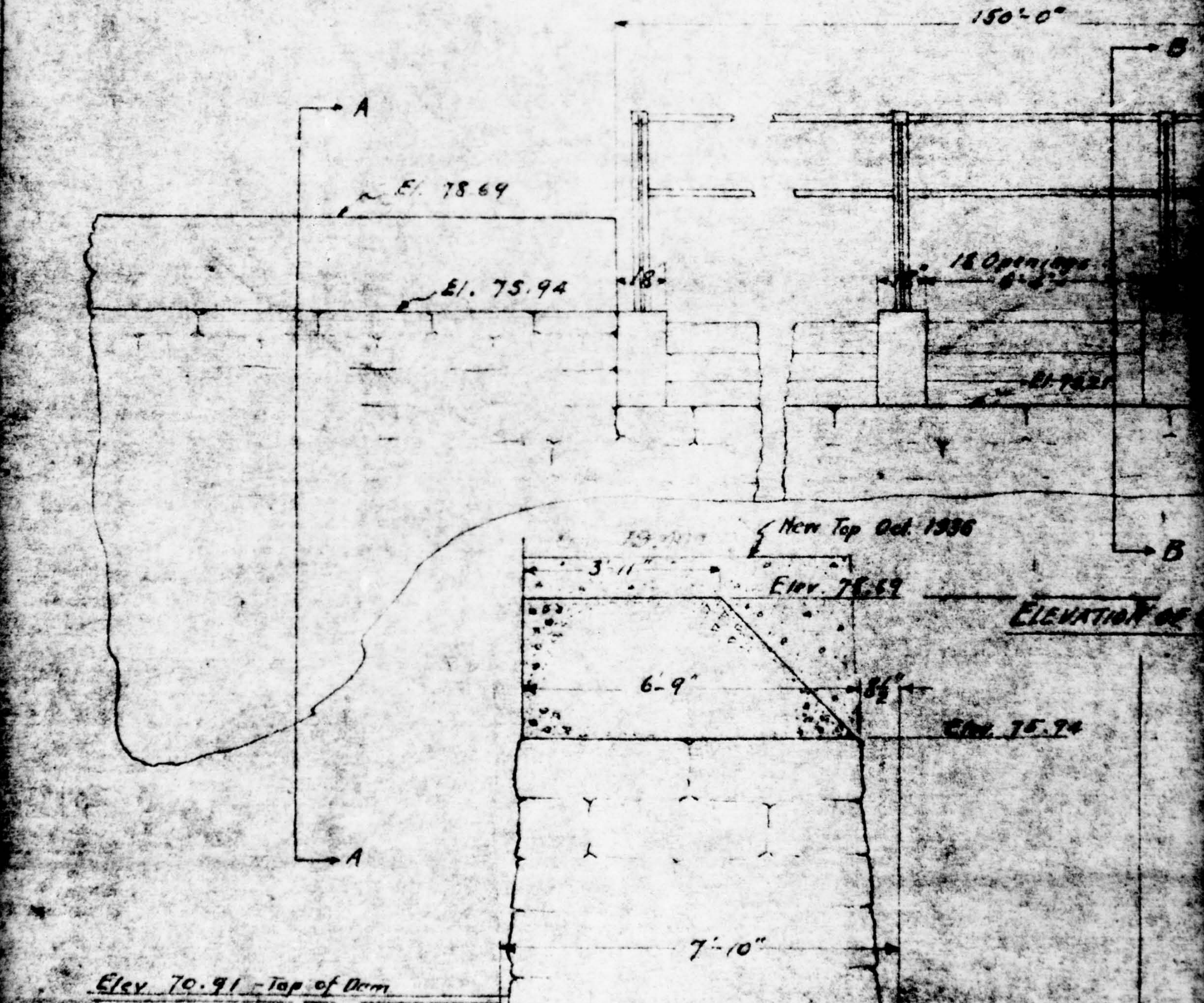

H
PLANT
Ground Elev 62.5
Ground Elev 68.5
HUDSON



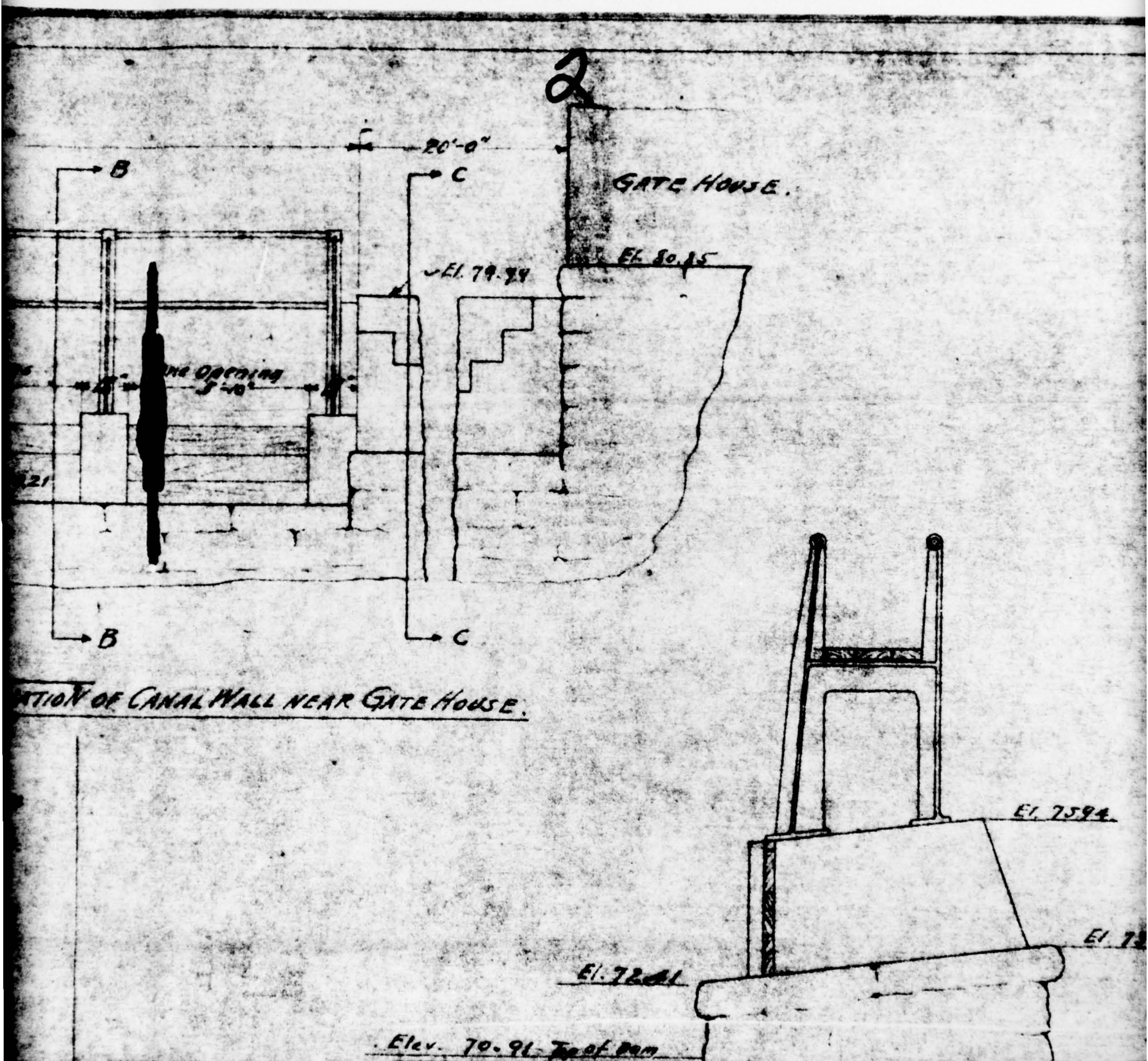
Gauge C.
Spillway
Gauge B
NORTH MAIN ST.
FOREBAY
Wall Elev. 78.7
Spillway
Gate House Wall Elev. = 84.85
Gauge D.
Wall
R I V E R
Dam. Elev. of Top = 70.51
CANAL LOCK 3

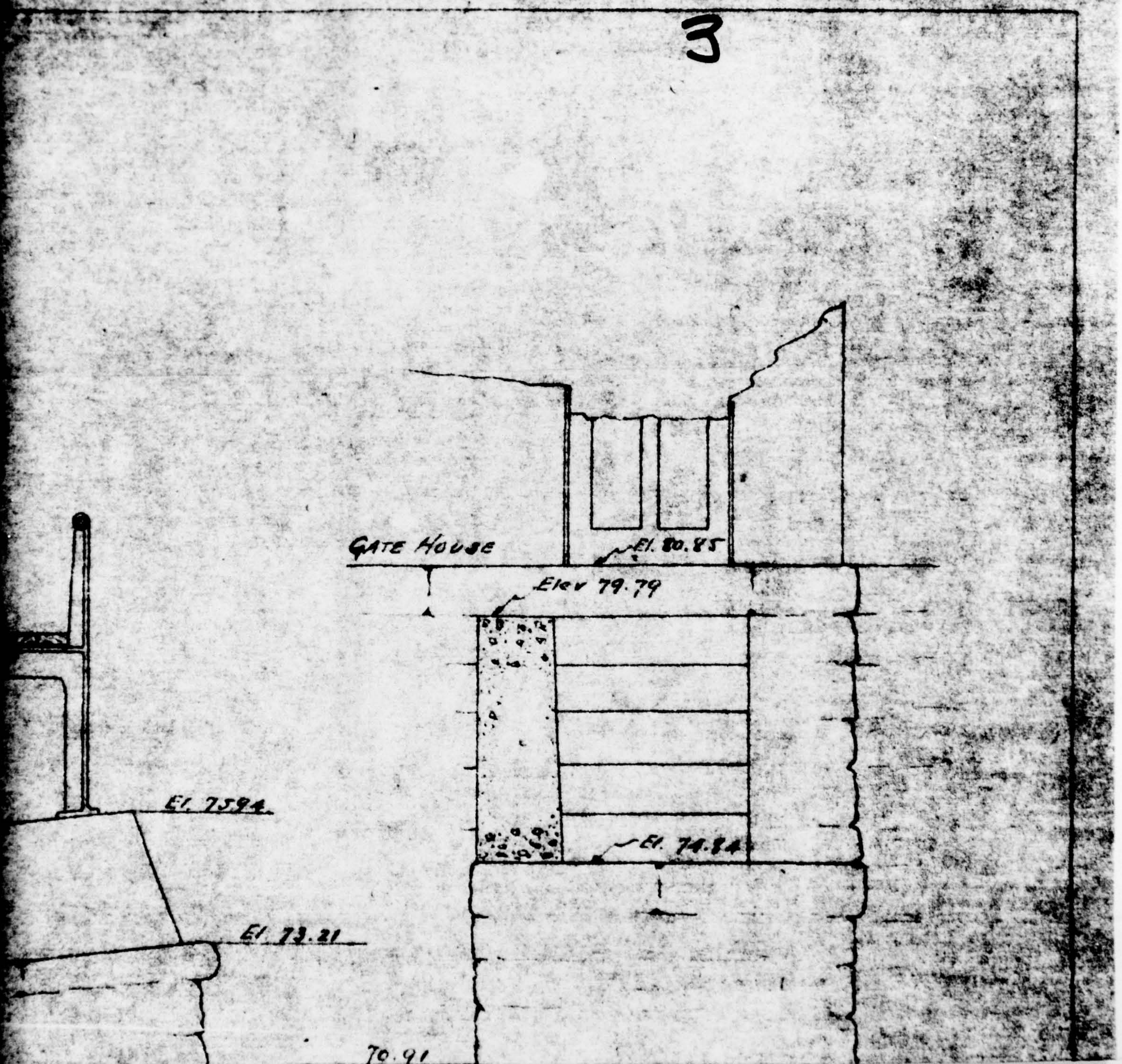

NORTH MAIN ST.
Ground Elev.
Wall Elev.
Gate House Wall Elev.
Gauge D.
6
WEST VIRGINIA PULP & PAPER CO.
MECHANICVILLE, N. Y.

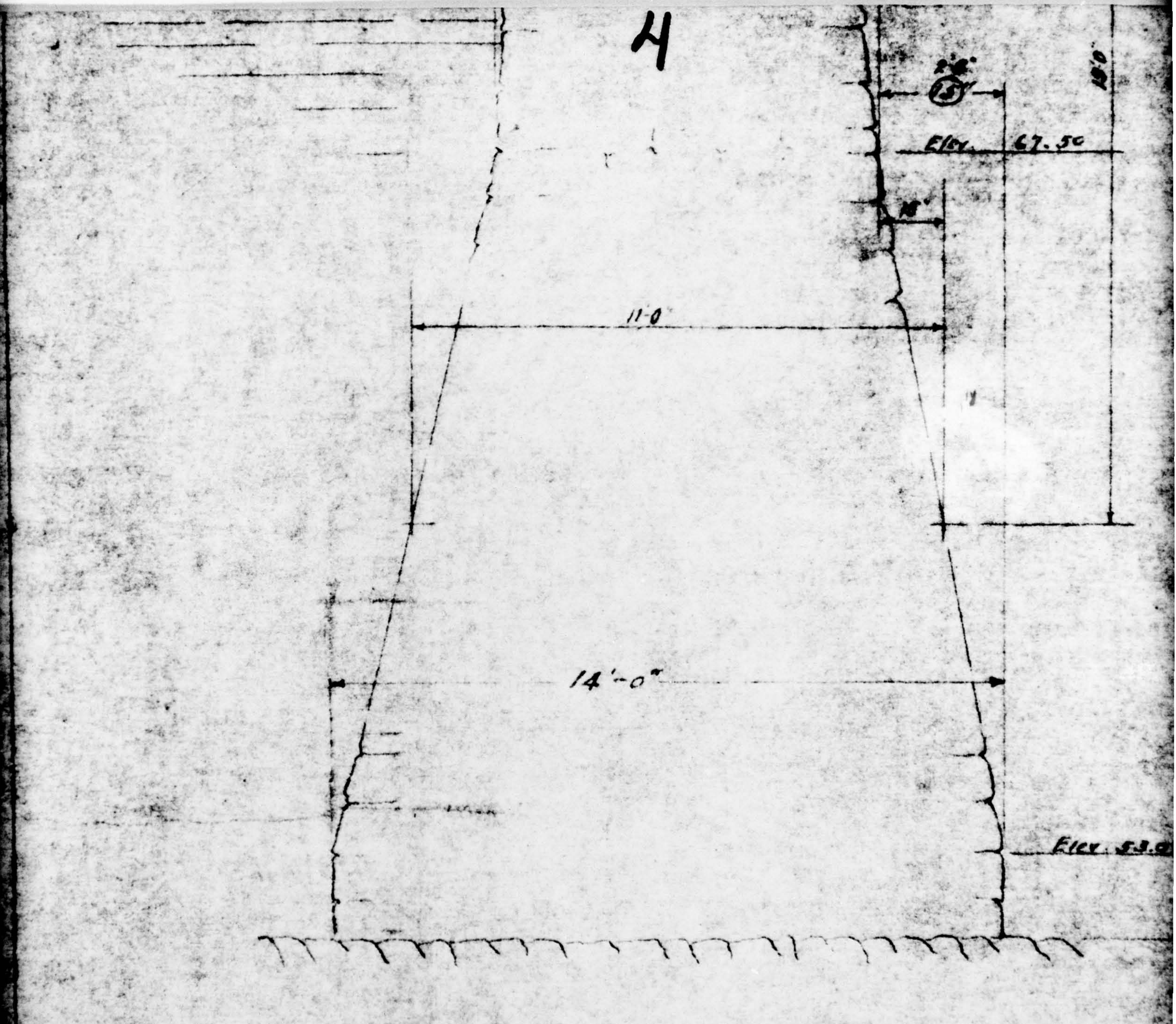
1



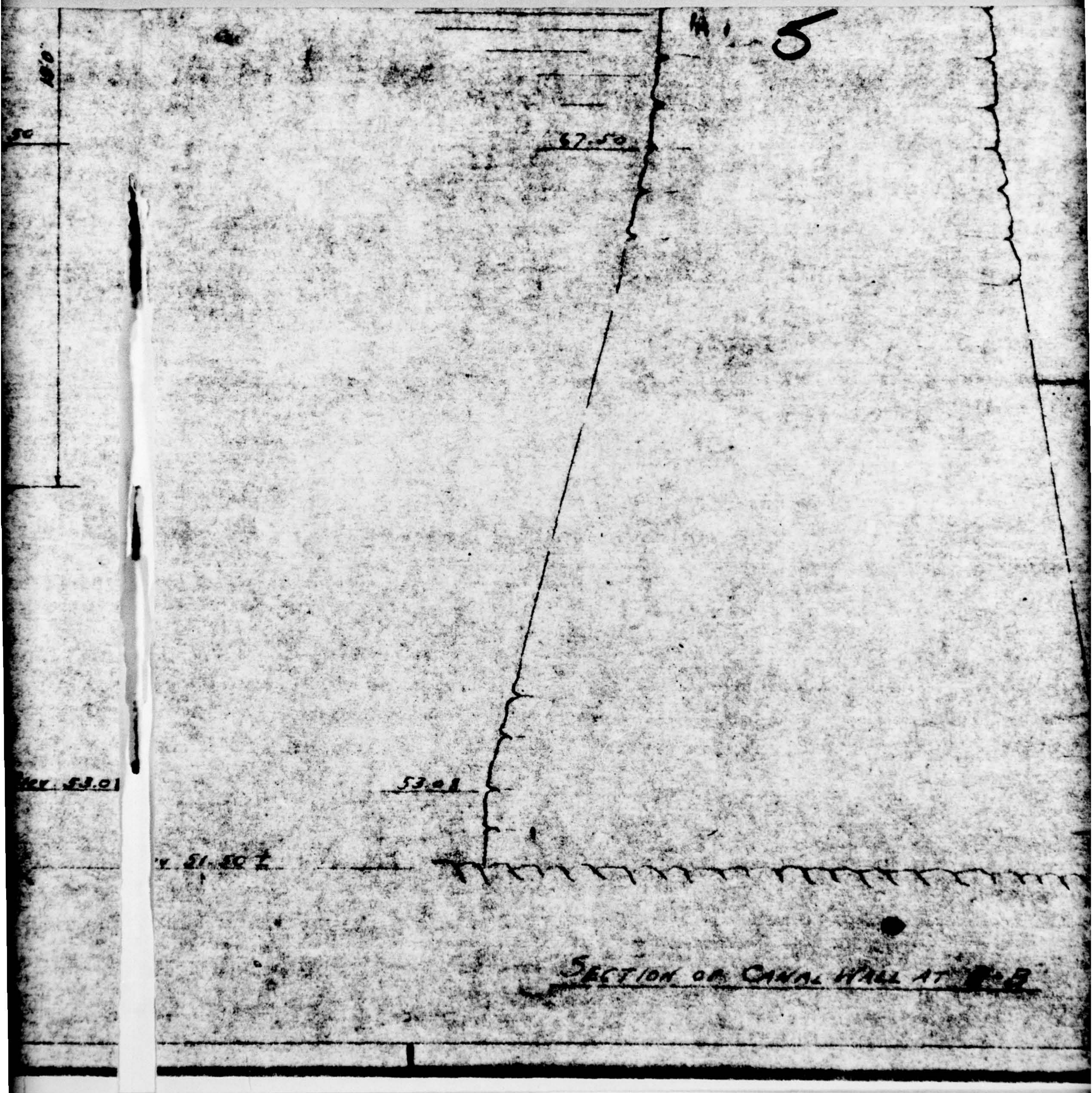
150'-0"
A
A
B
B
El. 78.69
El. 75.94
18
16 Openings
El. 74.21
New Top Oct. 1936
3'-11"
Elev. 78.69
ELEVATION OF
6'-9"
8½"
Elev. 75.94
7'-10"
Elev. 70.91 - Top of Dam


2
20'-0"
B
C
GATE HOUSE.
El. 80.15
El. 79.94
ne Opening
5'-10"
21
B
C
TION OF CANAL WALL NEAR GATE HOUSE.
El. 75.94
El. 7
El. 72.41
Elev. 70.91 - Top of Dam

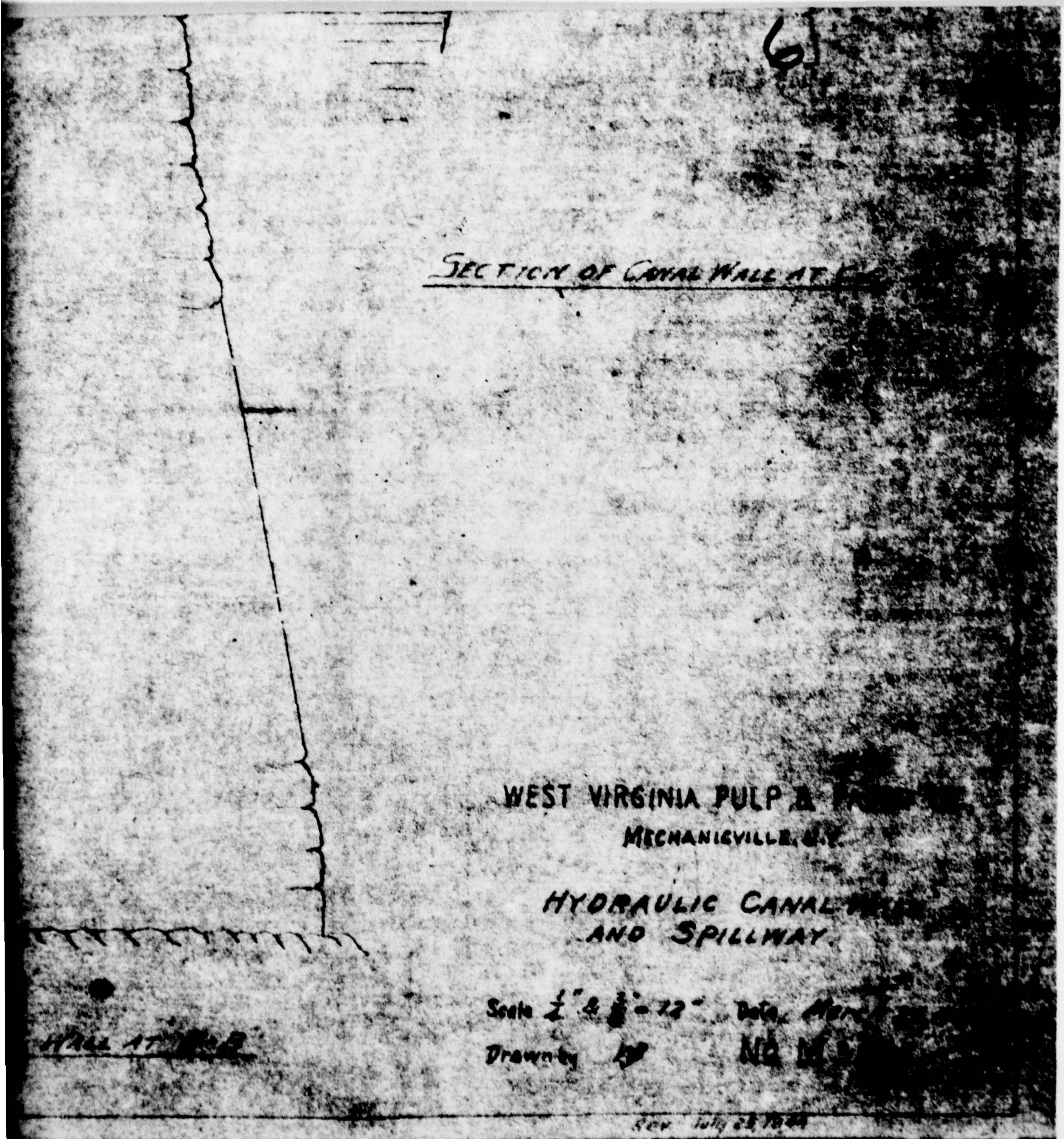

GATE HOUSE
El. 80.85
Elev 79.79
El. 74.84
El. 75.94
El. 73.21
70.91

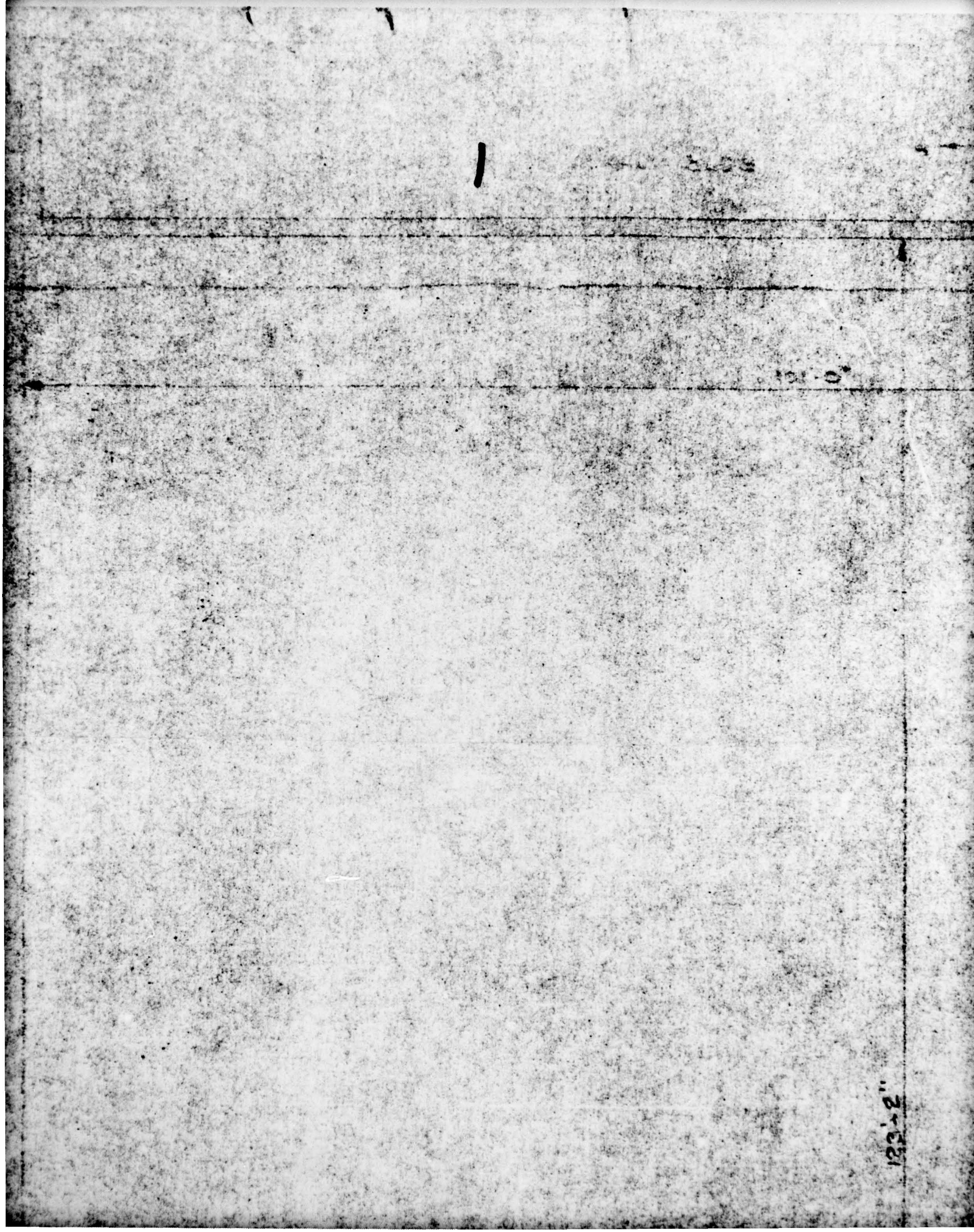


SECTION OF CANAL WALL AT A-A.

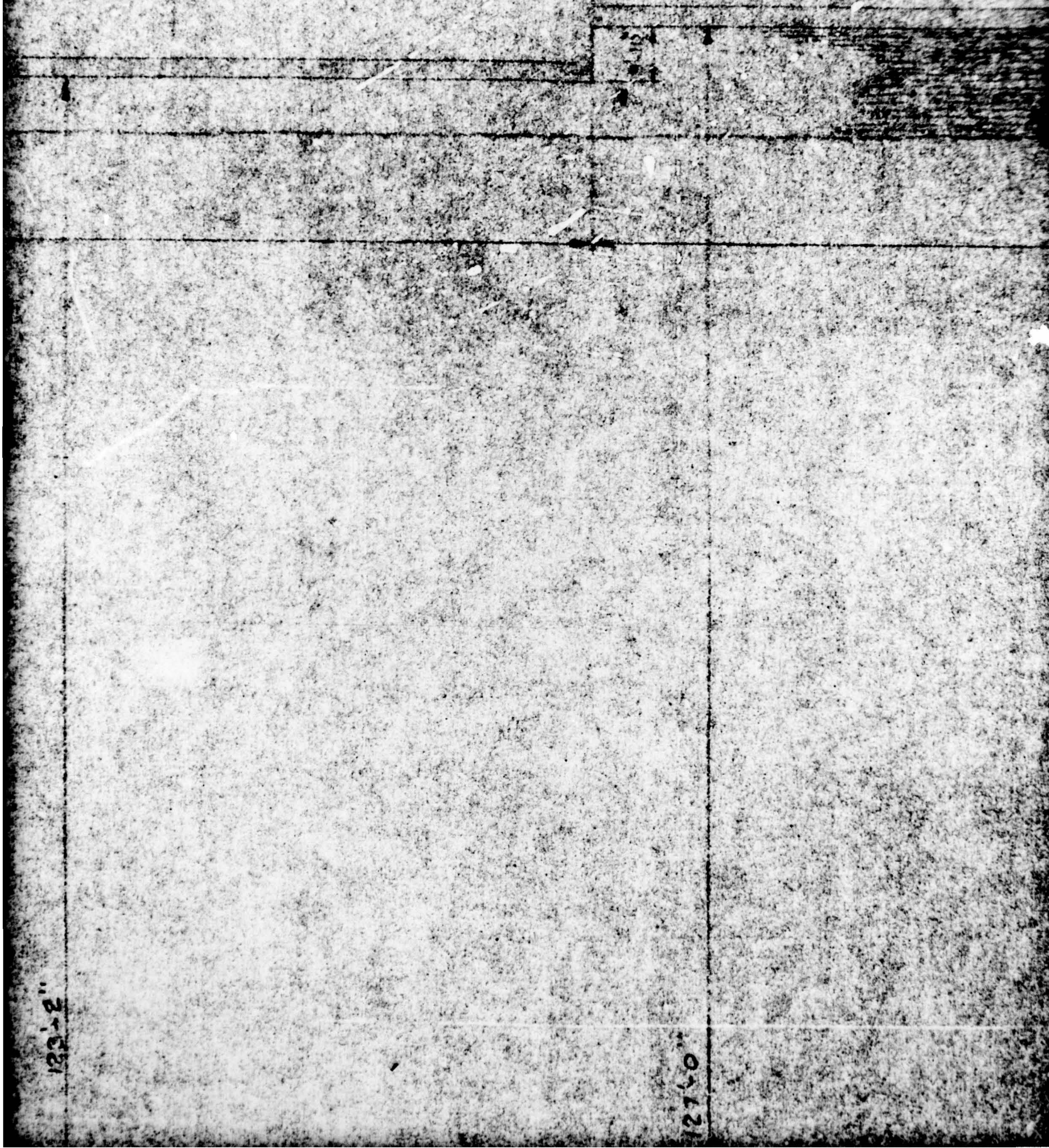


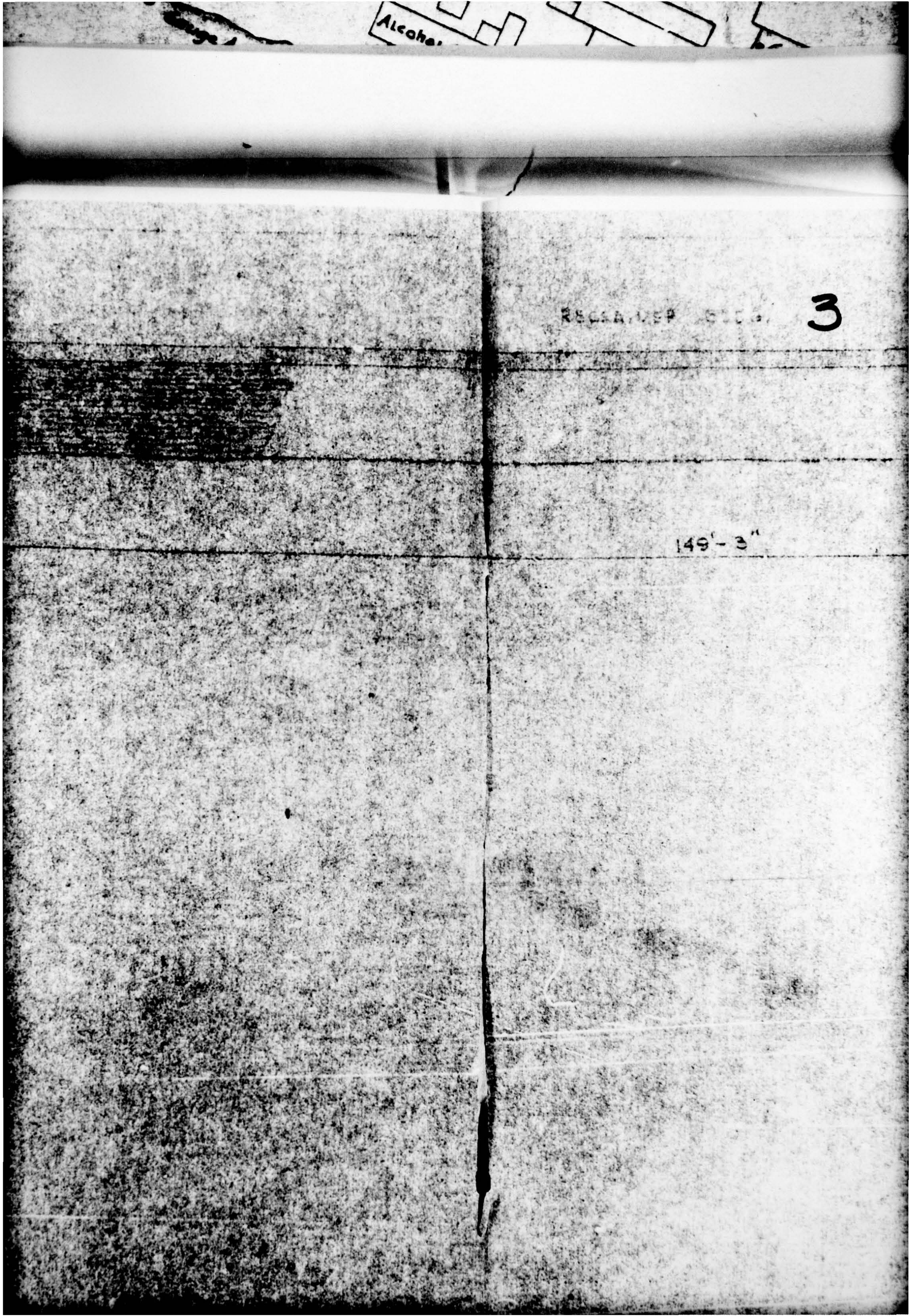
SECTION OF CANAL WALL AT B

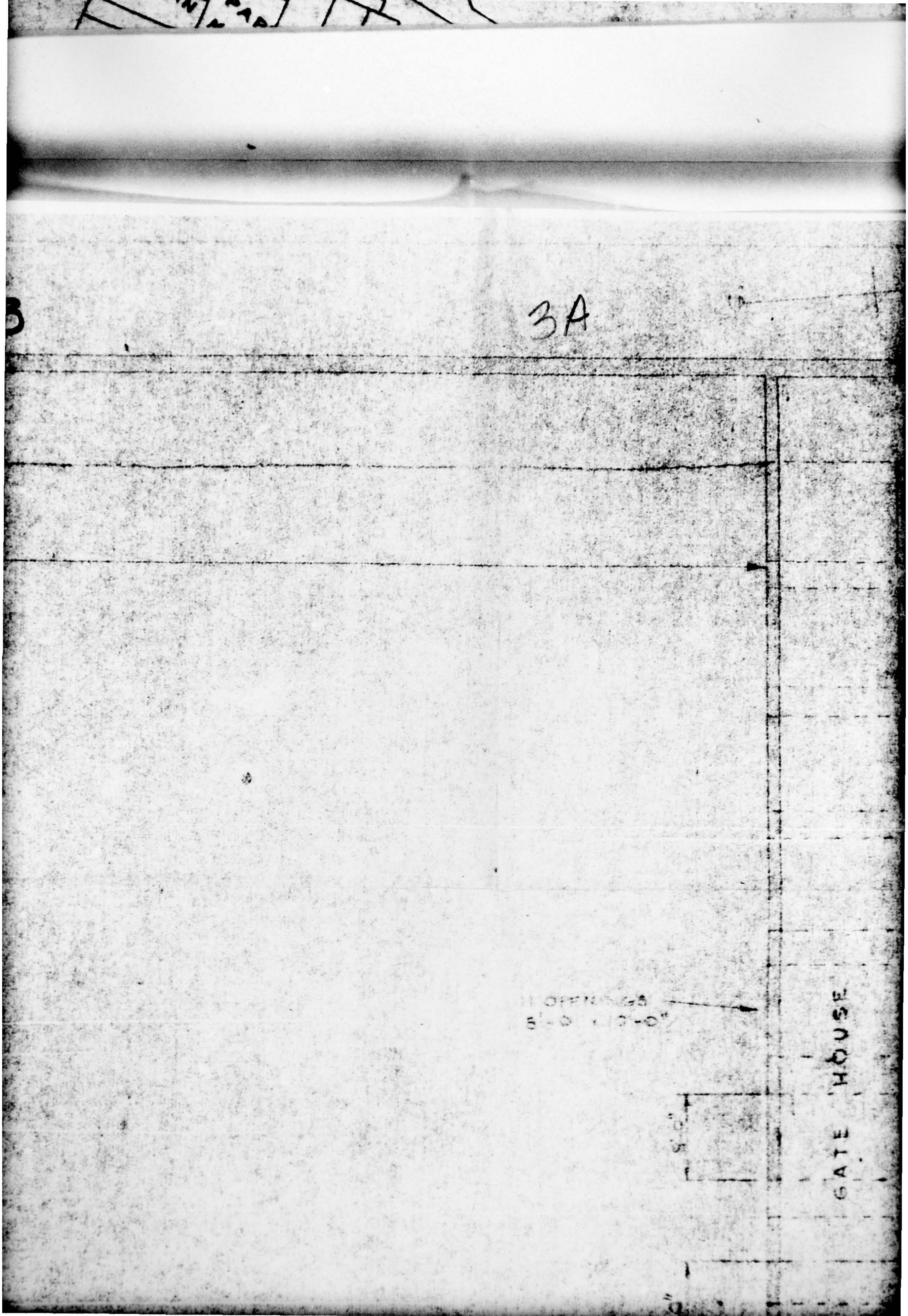

6
SECTION OF CANAL WALL AT
WEST VIRGINIA PULP &
MECHANICVILLE
HYDRAULIC CANAL
AND SPILLWAY
Scale
Date
Drawn by
WALL AT



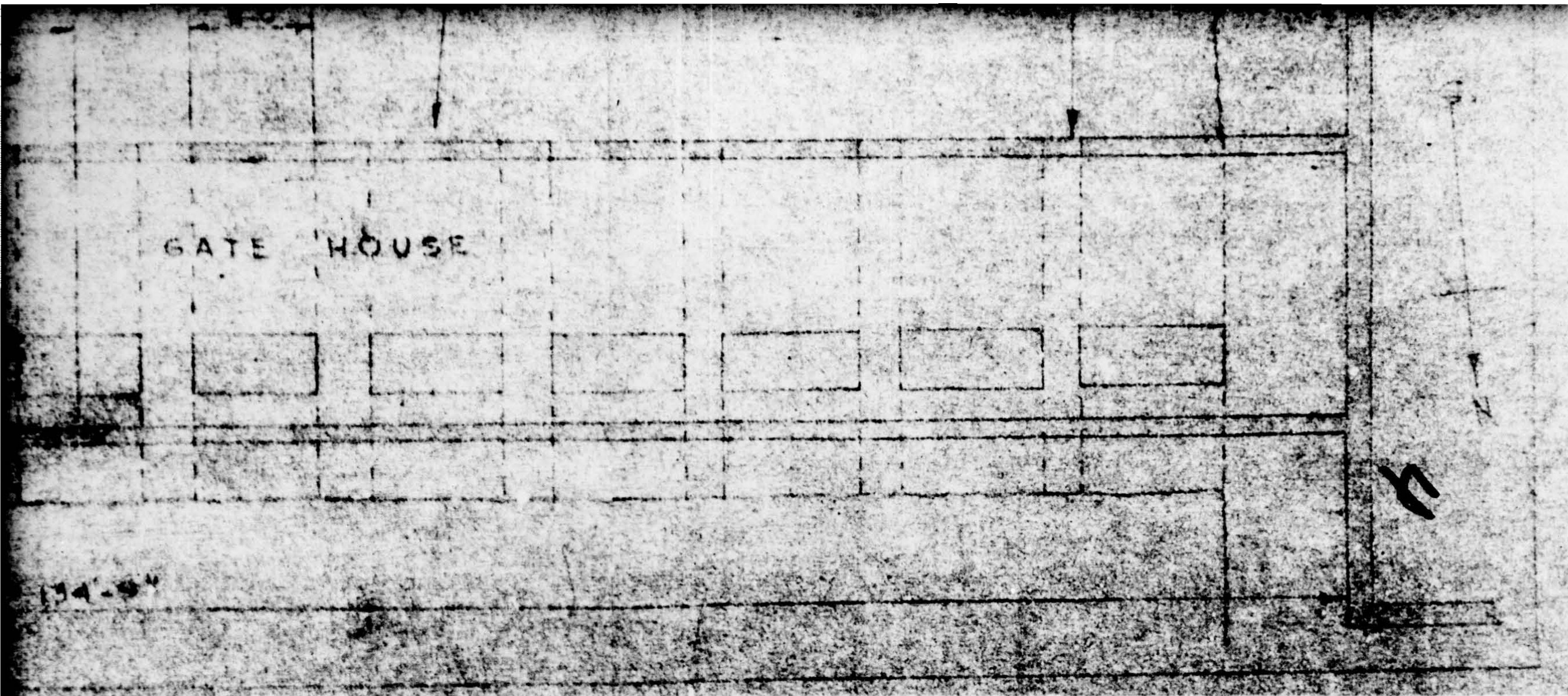
2



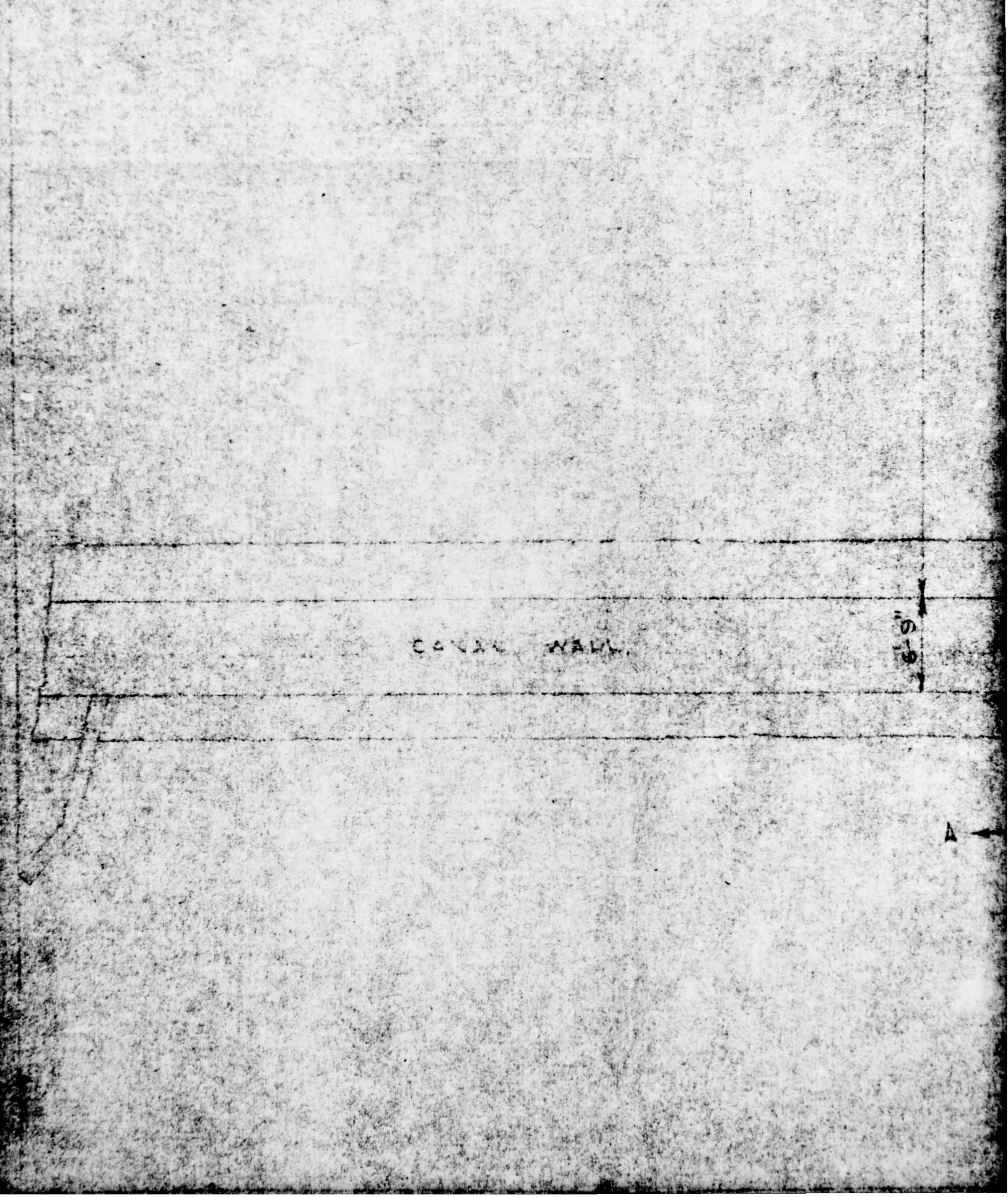

Alcohol
3
149'-3"

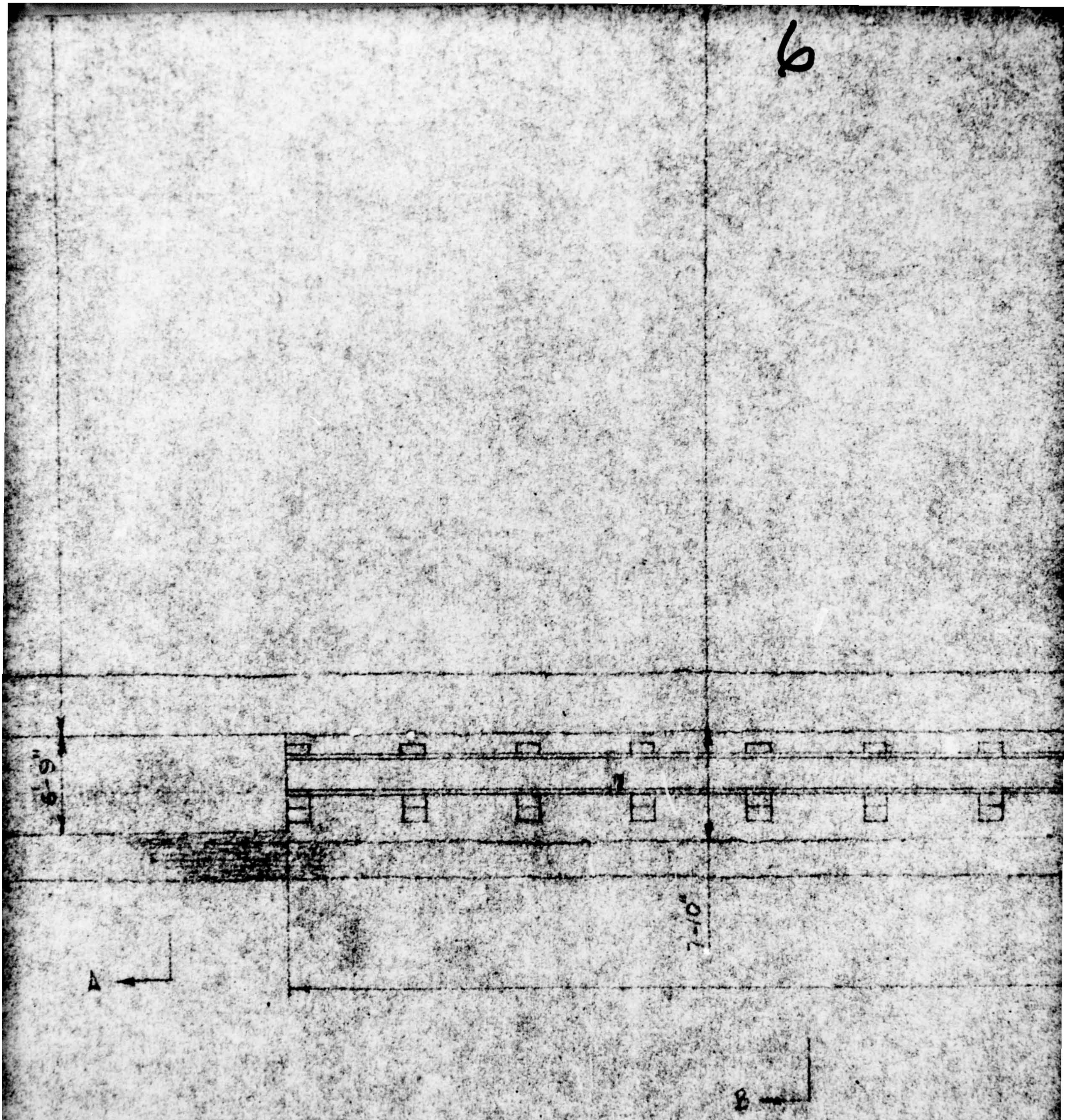


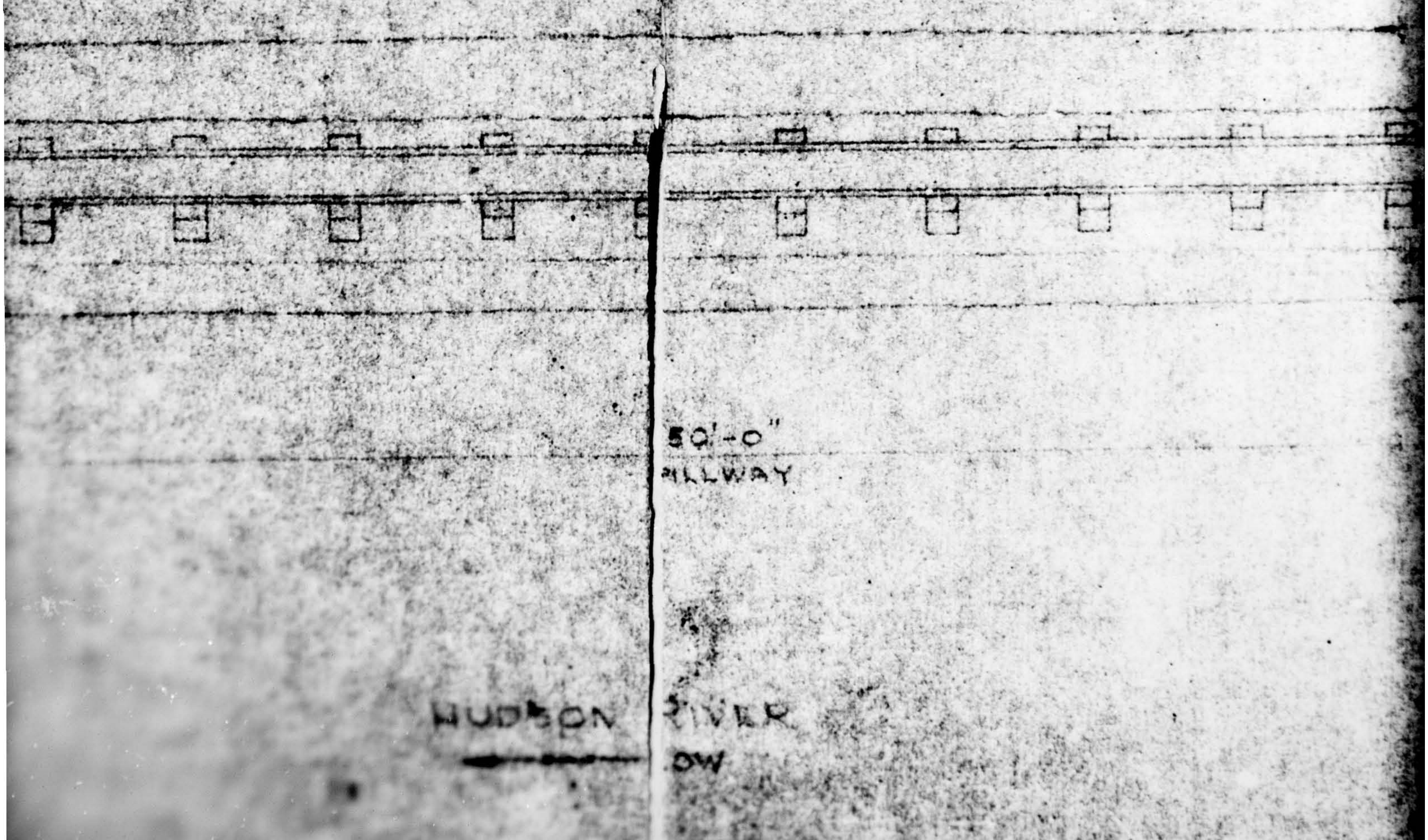
3A
OPENINGS
5'-0"
GATE HOUSE

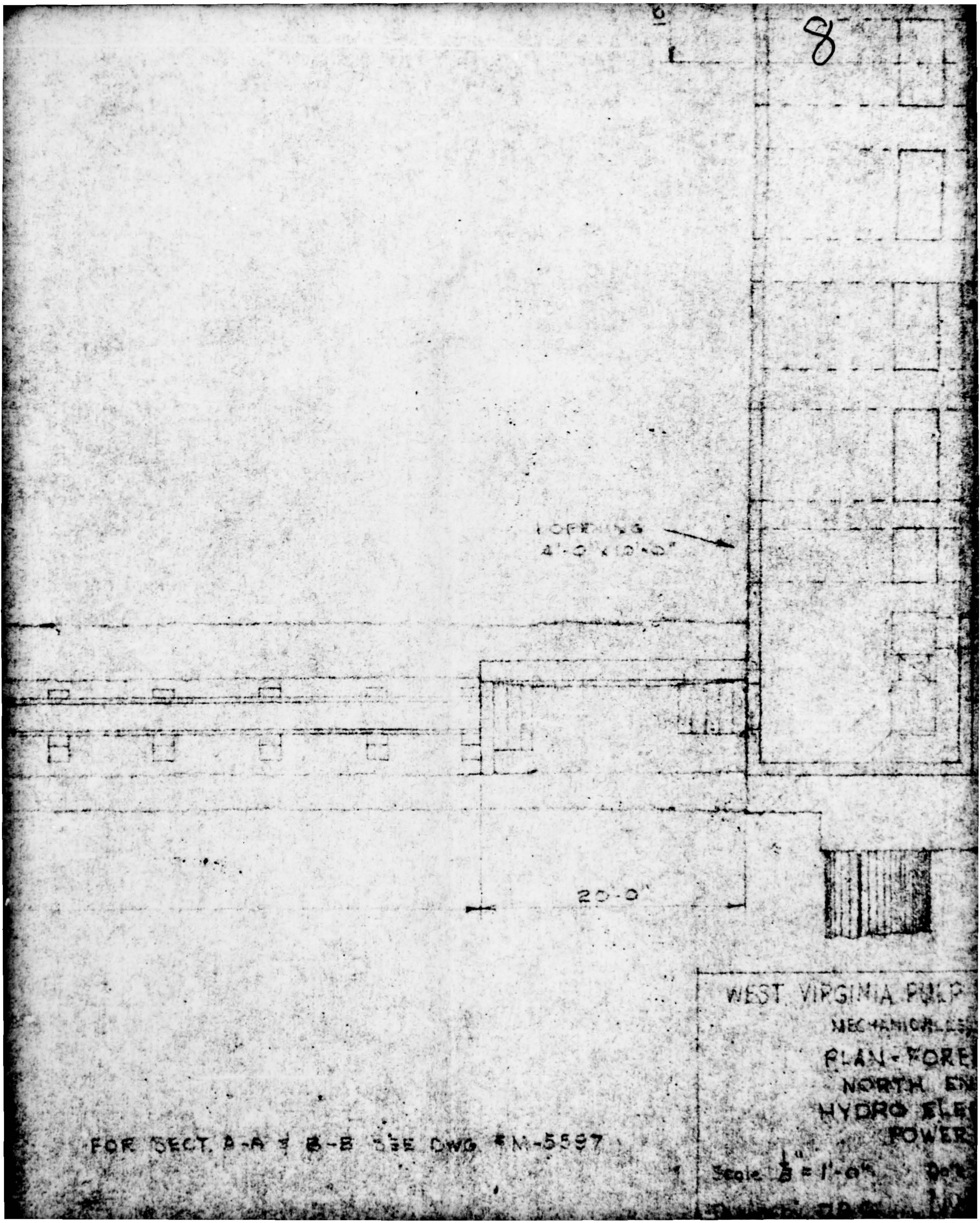

GATE HOUSE

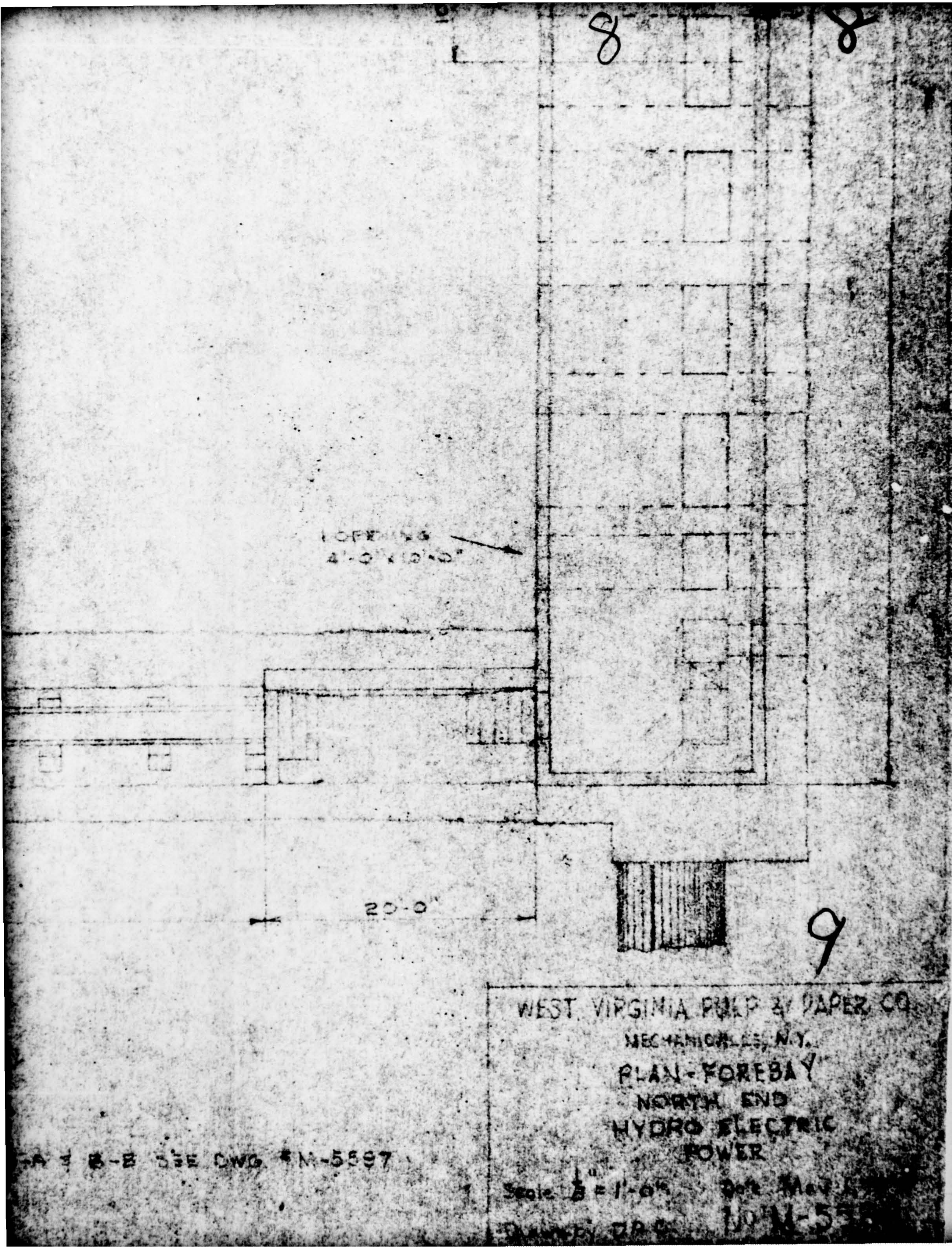
5




6'-9"
7'-10"
A
B


50'-0"
HUDSON RIVER


8
OPENING
20'-0"
FOR SECT. A-A & B-B SEE DWG. #M-5597
WEST VIRGINIA PULP
PLAN - FORE
NORTH EN
HYDRO ELE
POWER
Scale 1/8" = 1'-0"


8
8
20'-0"
9
WEST VIRGINIA PULP & PAPER CO.
MECHANICVILLE, N.Y.
PLAN - FOREBAY
NORTH END
HYDRO ELECTRIC
POWER