

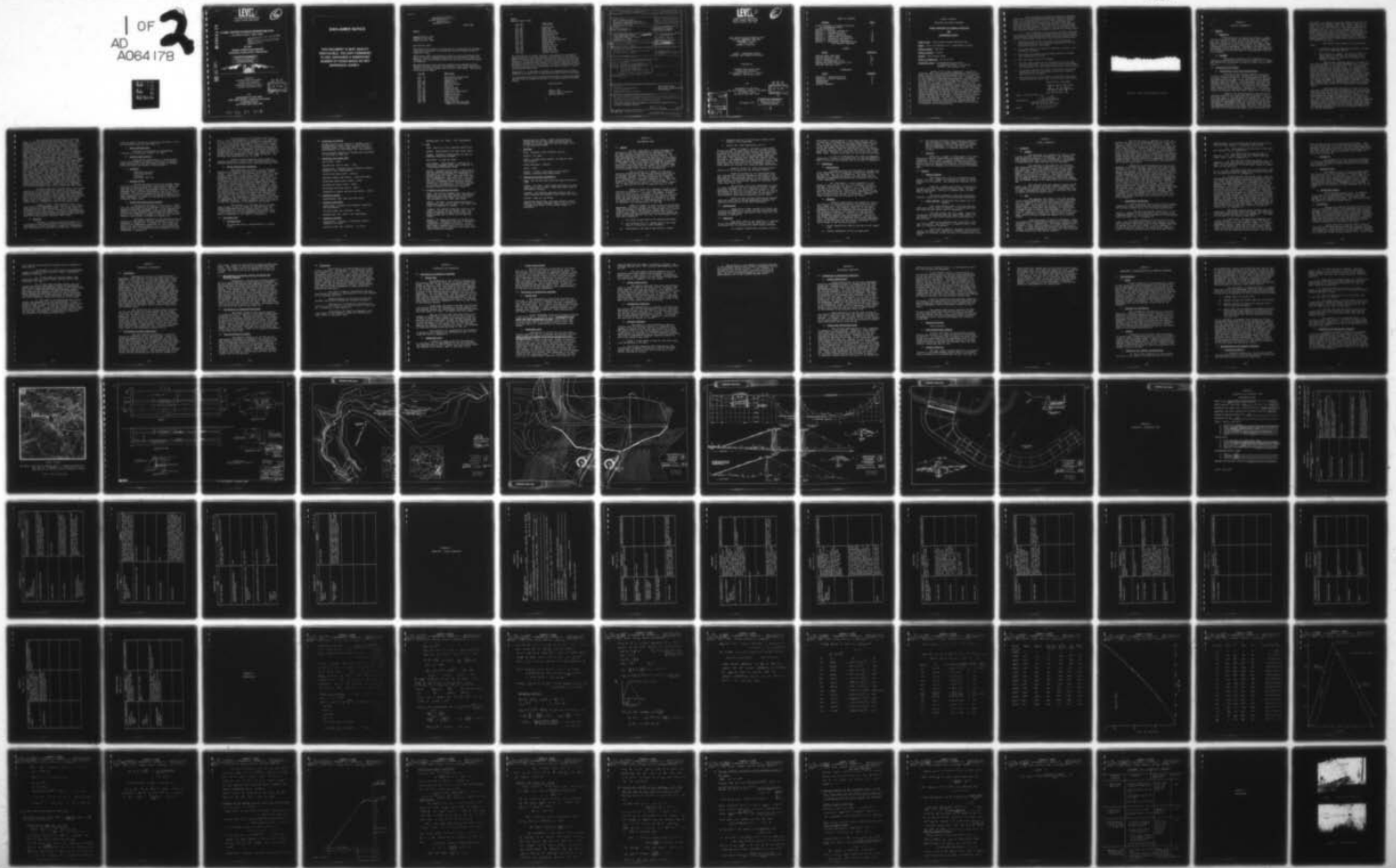
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NEW YORK STATE DEPT OF ENVIRONMENTAL CONSERVATION ALBANY F/G 13/2  
NATIONAL DAM SAFETY PROGRAM. CORK CENTER STORAGE RESERVOIR DAM --ETC(U)  
SEP 78 G S SALZMAN DACW51-78-C-0035

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1 OF 2  
AD A064178





# LEVEL

MOHAWK RIVER WATERSHED  
KECKS CENTER CREEK BASIN

6

AD A 064178

## CORK CENTER STORAGE RESERVOIR DAM FULTON COUNTY, NEW YORK

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NY 658

### PHASE I INSPECTION REPORT NATIONAL DAM SAFETY PROGRAM /

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Prepared by  
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For  
DEPARTMENT OF THE ARMY  
NEW YORK DISTRICT, CORPS OF ENGINEERS  
26 FEDERAL PLAZA  
NEW YORK, NEW YORK 10007

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DEPARTMENT OF THE ARMY  
U. S. ARMY ENGINEER DISTRICT, NEW YORK  
26 FEDERAL PLAZA  
NEW YORK, NEW YORK 10007

2 OCT 1978

NANEN-F

Honorable Hugh L. Carey  
Governor of New York  
Albany, New York 12224

Dear Governor Carey:

The purpose of this letter is to inform you of a clarification of the guidelines used by this office in assessing dams under the National Program of Inspection of Dams.

Office of the Chief of Engineers has recently provided a clarification that dams with seriously inadequate spillways are to be assessed as unsafe, non-emergency, until more detailed studies prove otherwise or corrective measures are completed.

The following dams in your state have previously been assessed as having seriously inadequate spillways, with capability to pass safely only the percentage of the probable maximum flood as noted in each report. They are now to be assessed as unsafe:

<u>I.D. NO.</u>	<u>NAME OF DAM</u>
N.Y. 59	Lower Warwick Reservoir Dam
N.Y. 4	Salisbury Mills Dam
N.Y. 45	Amawalk Dam
N.Y. 418	Jamesville Dam
N.Y. 685	Colliersville Dam
N.Y. 6	Delta Dam
N.Y. 121	Oneida City Dam
N.Y. 39	Croton Falls Dam
N.Y. 509	Chadwick Dam (Plattenkill)
N.Y. 66	Boyds Corner Dam
N.Y. 397	Cranberry Lake Dam
N.Y. 708	Seneca Falls Dam
N.Y. 332	Lake Sebago Dam
N.Y. 338	Indian Brook Dam
N.Y. 33	Lower(S) Wicoppee Dam (Lower Hudson W.S. for Peekskill)

NANEN-F  
Honorable Hugh L. Carey

<u>I.D. NO.</u>	<u>NAME OF DAM</u>
N.Y. 49	Pocantico Dam
N.Y. 445	Attica Dam
N.Y. 658	Cork Center Dam
N.Y. 153	Jackson Creek Dam
N.Y. 172	Lake Algonquin Dam
N.Y. 318	Sixth Lake Dam
N.Y. 13	Butlet Storage Dam
N.Y. 90	Putnam Lake (Bog Brook Dam)
N.Y. 166	Pecks Lake Dam
N.Y. 674	Bradford Dam
N.Y. 75	Sturgeon Pool Dam
N.Y. 414	Skaneateles Dam
N.Y. 155	Indian Lake Dam
N.Y. 472	Newton Falls Dam
N.Y. 362	Buckhorn Lake Dam

The classification of "unsafe" applied to a dam because of a seriously inadequate spillway is not meant to connote the same degree of emergency as would be associated with an "unsafe" classification applied for a structural deficiency. It does mean, however, that based on an initial screening, and preliminary computations, there appears to be a serious deficiency in spillway capacity so that if a severe storm were to occur, overtopping and failure of the dam would take place, significantly increasing the hazard to loss of life downstream from the dam.

Consequently, it is advisable to implement the recommendations previously furnished in the reports for the above-mentioned dams as soon as practicable.

It is requested that owners of these dams be furnished a copy of this letter and that copies be permanently appended to all reports previously furnished to you.

Sincerely yours,

CLARK H. BENN  
Colonel, Corps of Engineers  
District Engineer

REPORT DOCUMENTATION PAGE		READ INSTRUCTIONS BEFORE COMPLETING FORM
1. REPORT NUMBER	2. GOVT ACCESSION NO.	3. RECIPIENT'S CATALOG NUMBER
4. TITLE (and Subtitle) Phase I Inspection Report Cork Center Storage Reservoir Dam Mohawk River Watershed, Fulton Co. N.Y. Inventory No. N.Y. 658		5. TYPE OF REPORT & PERIOD COVERED Phase I Inspection Report National Dam Safety Program
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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. Cork Center Reservoir Dam was judged to be unsafe-non-emergency due to a seriously inadequate spillway.		

393 970 *Lee*

# LEVEL II

6

MOHAWK RIVER WATERSHED  
KECKS CENTER CREEK BASIN  
FULTON COUNTY, NEW YORK

CORK CENTER STORAGE RESERVOIR DAM  
CITY OF JOHNSTOWN, NEW YORK  
DEPARTMENT OF WATER  
NDS # NY 658  
NYSDEC # 172C-3191

PHASE I INSPECTION REPORT  
NATIONAL DAM INSPECTION PROGRAM

Prepared by

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Caldwell, New Jersey 07006

For

DEPARTMENT OF THE ARMY  
New York District, Corps of Engineers  
26 Federal Plaza  
New York, New York 10007

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29 August 1978

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PHASE I REPORT  
NATIONAL DAM SAFETY PROGRAM

BRIEF ASSESSMENT OF GENERAL CONDITIONS

AND

RECOMMENDED ACTION

Name of Dam: Cork Center Storage Reservoir Dam  
Owner: City of Johnstown, N.Y.; Department of Water  
State Located: New York  
County Located: Fulton  
Stream: Keck Center Creek  
Date of Inspection: 20 July 1978  
Inspection Team: Converse Ward Davis Dixon  
91 Roseland Avenue, P. O. Box 91  
Caldwell, New Jersey 07006

Based on our visual inspection, a review of available data, and calculations performed as part of this study, the Cork Center Storage Reservoir Dam is judged to be functioning satisfactorily at this time. However, based on the screening guidelines established by the Department of Army, Office of the Chief of Engineers (OCE), the spillway capacity is rated as inadequate. In addition, the spillway is seriously inadequate since it satisfies all the conditions established by the OCE guidelines for determining seriously inadequate spillway capacity. Since this assessment was based on OCE screening criteria, a detailed hydrologic and hydraulic evaluation of the watershed and spillway should be performed by the use of more precise and sophisticated methods and procedures. Following such an investigation, the need for, and type of, mitigating measures should be determined. Until such a study is completed and the spillway adequacy issue resolved, around-the-clock surveillance of the dam should be provided during periods of unusually heavy precipitation.

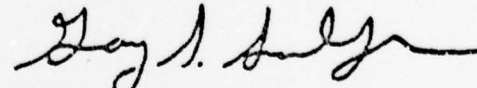
The occurrence of excessive amounts of seepage along the toe of the embankment warrants further investigation. It is recommended that the nature, strength properties and seepage characteristics of the embankment and foundation materials be established through a soil exploratory and testing program. The stability of the embankment may then be analyzed in the light of the new findings, and any necessary measures to reduce or control flow established.

Our assessment of the general physical condition of the Cork Center Storage Reservoir Dam has led us to make the following recommendations which should be implemented as soon as practicable, certainly within the next three years:

1. Injection grouting for fixing the leak between the downstream slope of the overflow spillway and the left abutment should be undertaken.
2. All minor damages to concrete (spalling, scaling, etc.) should be repaired.
3. The intake structure access bridge should be scraped and subsequently painted.
4. The gate house should be lighted.
5. The low woody growth on the upstream face of the dam should be removed. Shallow rooted trees on the embankment should be cut down; deep rooted trees should remain.
6. A specific program of periodic maintenance of the dam embankment and its appurtenant structures should be established and followed.
7. The flow from the lake should be controlled from the intake structure rather than from the lower gate house.

Respectfully submitted,


CONVERSE WARD DAVIS DIXON



Gary S. Salzman, P.E.

Date: 29 August 1978

Approved by:



Colonel Clark H. Benn  
New York District Engineer

Date:

21 September 78



OVERVIEW - CORK CENTER STORAGE RESERVOIR

ABSTRACT

SECTION 1

PROJECT INFORMATION

1.1 General

a. Authority

The authority to conduct this Phase I inspection and evaluation comes from the National Dam Inspection Act (P.L. 92-367) of 1972 in which the Secretary of the Army was authorized to initiate, through the Corps of Engineers, a program of safety inspections of non-federal dams throughout the United States. Management and execution of the program within the State of New York has been undertaken by the New York State Department of Environmental Conservation (NYSDEC).

b. Purpose

The primary purpose of the inspection is to evaluate available data and to give an opinion as to whether the subject dam constitutes a hazard to human life or property.

ABSTRACT

1.2 Description of Project

a. Description of Dam and Appurtenances

The Cork Center Storage Reservoir was built in 1919, and is an earth fill structure with a concrete core wall (Plates IV and V). It is approximately 460 feet in length along its crest and 49 feet high at its deepest section (near the middle). Longitudinally, the earth embankment generally lies in the north-south direction with its crest at elevation 1059. The upstream and downstream faces slope at approximately 1 vertical to 3 horizontal, and 1 vertical to 2 horizontal, respectively. An unpaved access road extends over the crest, which is 10 feet wide.

Based on the design drawings, the top of the concrete core wall is 2 feet wide and is located 2 feet below the crest of the dam along its centerline. It tapers at 1 horizontal to approximately 19 vertical on both its upstream and downstream sides, and its height varies with the depth of embankment above the original ground surface. Its maximum base width is 6 feet. Near the right end of the embankment where the dam height is small, the concrete core wall is of uniform thickness, being 3 feet from top

to bottom. The depth of core wall penetration below the original ground surface varies from about 7 feet to 12 feet, and it is embedded into rock, roughly between Stations 0+70 and 2+90 (Plate V). The main purpose of the concrete core wall is to inhibit seepage through the embankment and to reduce the amount of seepage under it. Plate V also shows that the rip-rap on the upstream face of the embankment extends from elevation 1057 down to original ground surface except where original surface is below elevation 1021, in which case rip-rap is carried to elevation 1021 only.

The outlet works consist of the following items:

- i. A 500-foot long concrete-lined spillway, 60 feet wide at the overflow section.
- ii. Two 24-inch C.I. pipes. One pipe is used for water supply to the Town of Johnstown. The second 24-inch C.I. mud pipe is available for emergency conditions.

The spillway overflow section is located at the right abutment of the earth embankment and in a direction perpendicular to it (Plate IV). It is a concrete gravity structure and ungated (Plate II). Wooden sheet piling was apparently installed below it (Plate IV). The 60-foot length of the overflow section is divided into three equal bays by two piers of 1-foot width each. These piers and the end abutments support a concrete footbridge over the spillway. The spillway crest was raised by 15 inches and its plans for reconstruction were approved by N.Y. State Department of Public Works on October 17, 1963. The rounded crest of the remodelled spillway is at elevation 1055.25 feet, which is 3.75 feet below the abutment seats.

Reconstruction included a 12-inch thick reinforced concrete slab and wall dowelled into the upstream slab and along the vertical upstream face of the spillway, and raising the crest level by 15 inches (Plate II). The downstream face slopes at approximately 1 horizontal to 1.33 vertical and meets the downstream slab, also raised by 1 foot, at elevation 1048.0 feet. The spillway discharge channel (Plates IV and VI), constructed of concrete side-retaining walls and bottom concrete slab, takes two sharply curved turns and empties into the natural streambed about 150 feet downstream of the toe of the deepest section of the earthen embankment.

The intake structure as-built is not that shown on the original design drawings (Plate IV). The intake structure actually built in 1919, as shown in a photograph at the Water Department offices, consists of a vertical concrete cylindrical shell with an 11-foot inside diameter and 6-inch thick wall. Its top is at elevation 1060 and its vertical axis is about 98 feet upstream from the centerline of the crest of the embankment. The approach to the intake structure is a steel angle iron bridge built in 1963. Two valves located at 17-foot and 25-foot depths below the top of the intake (Fig. 69, Appendix E) let water into a 24-inch C.I. supply pipe, laid under the embankment, leading to a lower gate house located downstream of the dam, and then into the creek channel. There is a third valve at the intake structure for the 24-inch C.I. mud pipe, which is 40 feet below the top of intake (elevation 1020). The second 24-inch C.I. pipe (mud pipe) extends 30 feet farther from the intake structure into the lake, and its end is marked by a floating buoy. The mud pipe, which is used as an emergency outlet, follows the alignment of the supply pipe, namely under the embankment to the gate house and then to the natural drainage creek. The valves at the intake structure are kept open throughout the year, but they are worked once a year to check if the mechanism is functioning satisfactorily. Water supply is regulated by the 24-inch supply line valve in the lower gate house.

The gate house has a wooden floor, with stairs leading to the lower level for access to the gates. The gate house is not equipped with any lighting arrangement. In the gate house, there is one 24-inch gate valve across each of the two main lines (water supply line and mud line) and a 16-inch gate valve on the pipe that joins the two pipes (Fig. 70, Appendix E). In addition to the gate valve, there is a butterfly valve on the supply line.

Plates IV and V (1917 drawings) show a 24-inch valve at the embankment crest, but it was not encountered during the field inspection. Similarly, there is no aerator at the site, nor a 16-inch C.I. pipe bypass on the right side of the spillway, as shown on these drawings. It appears that during construction some changes were made from the original plans, deleting these items.

#### b. Location

The dam is locally called "High Daddy" and is located on Keck Center Creek in Fulton County, N.Y., approximately 5 miles northwest of the City of Johnstown, N.Y. The location of the dam is  $N43^{\circ}02'14''$ ,  $W74^{\circ}27'55''$ ,

shown on Plate I, which is a portion of the USGS 7.5 minute Quadrangle Sheet of Peck Lake, N.Y.

c. Size Classification

The dam is classified as intermediate (storage = 445 acre-feet; height = 49 feet).

d. Hazard Classification

This dam is classified as a "high hazard" due to the existence of danger to more than a few human lives (Miller farm, barn and house; trailer off Wemple Road near creek; two houses in Kecks Center).

e. Ownership

Department of Water  
City of Johnstown  
City Hall  
Johnstown, New York

f. Purpose of Dam

The dam was built to act as a storage reservoir for the City of Johnstown water supply system. Its watershed is approximately 2.6 square miles. The flow from the 24-inch supply pipe empties into the natural streambed about 150 feet downstream of the toe of the embankment. The natural channel carries this water to a weir, where water is diverted into the purification plant. After treatment, the water is distributed in the City of Johnstown supply lines.

g. Design and Construction History

The dam was designed in 1917 by W. E. Natan-son, the then Johnstown City Engineer, and James P. Wells, Consulting Engineer, Rochester, N.Y. It was approved by George D. Pratt, Commissioner, Conservation Commission, Division of Inland Waters, State of New York (Ref: back side of 1917 drawings 1 through 7; four of which are presented in this report as Plates III through VI).

Hydrologic computations and detailed stability analysis of the spillway section, performed in 1917, are presented in Appendix E. A summary of this analysis in the form of a memorandum dated July 24, 1917, from John Henry, the then junior engineer, N.Y. State Conservation Commission, to A. H. Perkins, division engineer, is also documented in Appendix E.



An application on Form E-61A1(2/62) dated Oct. 7, 1963, copy presented in Appendix E, was submitted by the City of Johnstown to the Bureau of Waterways, Division of Construction, Department of Public Works, Albany, N.Y., for raising the crest of the spillway by 15 inches to elevation 1055.25. One of the associated drawings prepared by Morrell Vrooman Engineers, Consulting Engineers, Gloversville, N.Y., is included in this report as Plate II. Another drawing in this set details the construction of a bridge from the embankment to the intake structure. The drawings were approved on Oct. 17, 1963.

During visual inspection, minor signs of pressure grouting or possibly just surface guniting were noticed around the spillway with a scratched date of 1963.

#### h. Normal Operational Procedure

The Department of Water, City of Johnstown, controls and regulates the water in the system. Water levels in the reservoir are checked twice a day for 7 days a week. Generally, in late fall, water is drawn down to approximately 1 foot below spillway crest and is maintained at that level through the winter. By April, water is purposely drawn down to about 6 feet below spillway crest so as to pass the spring and summer storms safely, with only a small amount overflowing the spillway. The reservoir level is brought down again in autumn. Usually water does not flow over the spillway; however, the highest water level reported is 2 feet overtopping the spillway (around 1971). Water from the Cork Center Reservoir is drawn through the intake structure and regulated by a gate valve on the 24-inch C.I. supply pipe at the lower gate house. Just downstream of the gate house, this water is discharged into the natural stream which carries it to a treatment facility. After treatment, water is supplied to the City according to the demand of the system.

The 24-inch C.I. mud pipe acts as an emergency outlet in case of danger of overtopping of the earthen embankment during a heavy storm. A copy of "Instructions for State of Emergency Conditions", hung on the office wall of the Department of Water in Johnstown, is attached to this report in Appendix E.

### 1.3 Pertinent Data

#### a. Drainage Area

The drainage area is approximately 2.6 square miles.

b. Discharge at Damsite

Maximum flood at the damsite is unknown, but it corresponds to about 2 feet of water over the spillway crest. This would result in a flow of approximately 645 cfs passing over the spillway.

Total spillway capacity at maximum pool elevation of 1059 feet is 1649 cfs.

c. Elevation (ft. above MSL)

Top of dam: 1059.

Maximum pool (top of dam): 1059.

Normal pool: Variable during different seasons; generally 1054 (approximate).

Overflow spillway crest: 1055.25.

Upstream supply pipe invert: 1043± and 1035±.

Downstream supply pipe invert: 1002±.

Upstream mud pipe invert: 1020±.

Downstream mud pipe invert: 1002±.

Streambed at supply and mud pipe outlet: 1002±.

d. Reservoir Length

Approximately 4000 feet from USGS Quad.

e. Storage (acre-feet)

Normal pool: Variable with seasons; generally 404 (estimated).

Spillway crest (El. 1055.25): 445.

Maximum pool (El. 1059): 601 (estimated).

f. Reservoir Surface

Normal pool: Variable in different seasons; generally 41 acres.

Spillway crest (El. 1055.25): 41 acres.

Maximum pool (El. 1059): 42.2 (estimated).

g. Dam

Type: Earth fill with concrete cutoff wall.

Length: Approximately 460 feet along crest.

Height: Variable; approximately 49 feet at greatest height near center.

Top width: 10 feet at crest.

Side slopes: Approximately 1 vertical to 3 horizontal upstream slope, and 1 vertical to 2 horizontal downstream slope.

Cutoff: Concrete cutoff wall, 2 feet wide at top and tapering upstream and downstream at 1 horizontal to approximately 19 vertical to a maximum width of 6 feet. Near the right end of the embankment, the cutoff wall is of uniform thickness of 3 feet from top to bottom. The wall is embedded below original ground surface for a minimum of 7 feet and a maximum of 12 feet, and penetrates to rock roughly between Stations 0+70 and 2+90 (Plate V).

h. Diversion and Regulating Works

Type: Two 24-inch diameter cast iron pipes at elevations 1043 and 1035 leading into the intake structure from whence only one 24-inch pipe proceeds to the lower gate house.

Length: 300 feet ± from intake structure to lower gate house, and 70 feet ± from lower gate house to discharge at stream.

Closure: Two manually operated valves at the intake structure for the supply line. Two manually operated valves at the lower gate house, either one of which can be used for control.

Access: To intake structure via bridge from embankment. Regulation from top of structure (intake box is locked with chain). At the gate house, regulation on the first floor of wood frame building (stairs to the lower level where valves are located).

Regulating facilities: Supply line valves at intake open at all times and regulation accomplished by 24-inch gate valve on the supply line at the lower gate house.

i. Spillway

Type: Concrete, round crested, gravity.

Length: 512 feet.

Width: 20 feet along channel, 60 feet at overflow section.

Crest elevation: 1055.25.

Gates: None.

Piers: 2; each 1 foot wide at the overflow section, carrying the bridge above.

j. Regulating Outlets (emergency)

Type: One 24-inch cast iron mud pipe at elevation 1020.

Length: 330 feet ± from intake structure to lower gate house, and 70 feet ± from lower gate house to discharge at stream.

Closure: One manually operated valve at the intake structure and another at the lower gate house.

Access: Same as 1.3h above.

Regulating facilities: Mud pipe valve at intake open at all times, whereas valve on the mud pipe at lower gate house is kept closed except in emergency.

SECTION 2  
ENGINEERING DATA

2.1 Design

In 1917, John Henry, a junior engineer for the State of New York Conservation Commission (NYSCC), did some hydraulic computations for the design of the spillway overtopping section and spillway channel. He also performed an analysis of the stability of the gravity section of the dam. His computations along with his memorandum summarizing his findings are presented in Appendix E. However, on the first page of his memo he refers to 7 items that could have been very useful for further understanding of the design of this dam as envisioned in 1917, but unfortunately only Item #7 is available now.

Item #7 refers to 7 sheets of blue print drawings showing plans, sections and details descriptive of the dam, spillway channel, reservoir and structures appurtenant thereto. All these drawings are signed by W. E. Natanson, City Engineer, Rochester, N.Y. They were approved on August 3, 1917 by Division Engineer A. H. Perkins and Commissioner George D. Pratt of NYSCC, Albany, N.Y. A description of each of these drawings is given below.

Sheet #1 (Plate III) presents a general location map on a scale of 1" = 1 mile and plan of the reservoir on a scale of 1" = 100 feet. The number 73 acres marked on this drawing is apparently the size of the property, and should not be mistaken for the reservoir area, which has been checked and found to be close to 41 acres as reported in the Oct. 7, 1963 application, of the City of Johnstown, for reconstruction of the dam.

Sheet #2 (Plate IV) shows a plan of the earthen embankment, spillway overfall section, spillway channel and other appurtenant structures on a contour map with a contour interval of 2 feet. The following items shown on this drawing were probably eliminated from the final design and never constructed because they were not encountered during the field inspection:

- i. 24" valve on the crest of the earthen embankment
- ii. 16" valve and 16" C.I. bypass pipe on the right side of the spillway right abutment
- iii. Footbridge at the end of the spillway channel

iv. Expanded section of the spillway channel downstream of the footbridge

v. Aerator and items associated with it.

Sheet #3 (Plate V) provides a longitudinal section through the embankment and the concrete spillway. It displays the original ground surface, the approximate bottom of the concrete core wall, and wooden sheet piling under the concrete spillway to approximately elevation 1033 feet. This drawing also shows four typical cross sections of the embankment at Stations 0+200, 0+60, 0+86 and 2+05. The 24-inch valve and the related vertical concrete shaft from the crest of the embankment, shown in cross section at Station 2+05, have not been constructed.

Sheet #4 (Plate VI) shows plan and typical sections of the spillway and the spillway channel.

Sheets #5 through #7, not reproduced here, show features not directly related to the safety of the dam, such as plan and cross section of the aerator (not constructed), longitudinal section of the spillway channel, cross sections of the spillway channel at different stations, gate house, etc.

Two drawings (the first reproduced as Plate II) dated September 1963 and prepared by Morrell Vrooman Engineers, Gloversville, N.Y., and the October 7, 1963 application by the City of Johnstown, are the only source of information about raising the crest of the spillway from elevation 1054 feet to elevation 1055.25 feet.

There are only minimal hydrological computations and there are no design parameters available for checking the stability of the earth embankment.

## 2.2 Construction

There are no formal records of original construction or remodeling of the spillway crest available. Concrete strength tests, reported in 1919 (3000 psi) are included in Appendix E.

## 2.3 Operation

The water level in the reservoir is recorded by Mr. Tim Newhouse twice a day for seven days a week. Downstream flow is monitored for water supply purposes.

As a general operational procedure, water is

drawn down to approximately 1 foot below spillway crest in late fall and maintained at that level through the winter. By April, the pool is purposely drawn down to about 6 feet below spillway crest for safely containing the spring and summer floods and passing only a slight amount. In autumn, the pool level is drawn down again. The highest water level reported is 2 feet over top of spillway in the early 1970s. Usually water does not flow over the spillway.

A list of "Instructions for State of Emergency Conditions" is hung on the office wall of the Department of Water in the City of Johnstown and is included in Appendix E.

## 2.4 Evaluation

### a. Availability

Engineering data were provided by the New York State Department of Environmental Conservation (NYSDEC) and by the owner, the City of Johnstown. The owner's City Engineer, Mr. Charles Ackerbauer, and the valve crew explained and demonstrated operational procedures during the visual inspection.

Mr. John Henry, a junior engineer, in his letter of July 24, 1917 to Mr. A. H. Perkins, Division Engineer, acknowledges the receipt of 7 items concerning the design of this dam. Items 1, 2 and 3, namely the original application for construction, the engineer's report, and specifications, could supply additional information on the original concept of the design. However, these documents were not available for review.

### b. Adequacy

Although computations with respect to stability analysis of the gravity section of the dam are available, there is no justification provided for the basic assumptions made. For example, stability of the dam is dependent on the uplift pressures and the properties of the foundation material, yet neither of them has been investigated or evaluated. There is no information on the embankment material and its properties. Similarly, the nature and amount of hydrology data is also very limited. Consequently, the stability of the earth embankment could not be analyzed, and the overall assessment is primarily based on the following factors:

- i. Visual observations made on the day of the inspection
- ii. Overall assessment of the available data

- iii. The analyses performed using hydrologic modelling data available in Upper Hudson and Mohawk River Basins Hydrologic Flood Routing Models, prepared for the Department of Army, New York District Corps of Engineers, by Resource Analysis, Inc. in 1976.

c. Validity

There are a number of items shown on the drawings which were not constructed as determined by visual inspection. This raises a question whether the concrete cutoff wall in the earth embankment was constructed or not, because excessive seepage was observed, on the downstream side, at the left abutment and along the toe of the embankment.

2.5 Geology

a. General Geology

The damsite and lake lie in southern Fulton County, New York. There is an extensive cover of glacial deposits at the surface. The underlying lithology is unknown.

There is a normal fault about a mile east of the dam, with the dam on the downthrown side. The fault trends north-south. There is a linement about 2 miles west of the dam.

The region underwent glaciation during the Wisconsin stage and is part of the glaciated Adirondacks.

b. Local Geology (Interpreted from stereo pair air photos)

Soil cover appears deep. The rock type beneath the glacial cover is unknown. The downstream channel looks dry and fairly free of vegetation.

The lake slopes look very steep. There are indications of siltation near the north inlet shore. In the photos, the lake level appears much lower than the dam spillway.

There is an apparent fault line about 2,500 feet downstream. The faultline is traceable about 4,000 feet in a southerly direction.

There were no geologic features (stratification, faults, cavities, etc.) detected or suspected that could be expected to adversely affect the dam or its appurtenant structures.



## SECTION 3

### VISUAL INSPECTION

#### 3.1 Findings

##### a. General

The reservoir was inspected on July 20, 1978, a sunny warm day with 80°F temperature. The inspection team consisted of Messrs E. A. Nowatzki and G. S. Salzman from Converse Ward Davis Dixon, and Messrs C. Ackerbauer, T. Newhouse and R. Lake from the City of Johnstown.

The approach to the reservoir is through a dirt road which is in good condition; it runs in a northerly direction towards the reservoir from a light-duty Old State Road. The reservoir is locally called "High Daddy". This reservoir is one of the sources of water supply for the City of Johnstown. Cork Center Reservoir was the only component of this supply system inspected, and it appeared to be in generally good condition, except for the seepage noted on the downstream side of the earthen dam.

The overflow from the reservoir during floods passes over the concrete spillway, whereas normally the only outflow is through the 24-inch C.I. supply line, and in case of emergencies another 24-inch "mud pipe" can also be opened to discharge excess water.

##### b. Dam

The approach road leads to the left abutment of the earthen embankment from whence a 10-foot wide road (Fig. 1, Appendix D) in fairly good condition continues over the crest to the junction of the right abutment with the spillway. Just before the spillway, the embankment takes a 90° turn such that the direction of the spillway is perpendicular to that of the embankment. Over the spillway, there is a 3-foot wide concrete footbridge (Figs. 2 and 3, Appendix D) leading to a small length of embankment, which is heavily wooded and appears to be natural ground.

Near the center of the main embankment, there is an angle-iron access bridge leading to a concrete intake structure (Fig. 1, Appendix D). The access bridge was constructed in 1963, and is generally in good condition but rusted. It needs scraping and painting, but one should be careful to avoid contamination of water with lead compounds or other potentially detrimental material.

The rip-rap (Fig. 4, Appendix D) on the upstream face comes up to approximately spillway crest level, and its visible portion is in good condition. There is vegetation above the rip-rap to the edge of the crest, and small woody growth is developing which should be checked at this stage (Figs. 1 and 4, Appendix D). The downstream face of the embankment is very heavily wooded with tall pines and deciduous trees (Fig. 1, Appendix D). Widespread animal burrows were also observed on the downstream slope. Animal burrows and decaying roots of any dying trees could create seepage channels through the embankment, but development of such a condition is inhibited by the presence of the concrete cutoff wall.

Inspection of the downstream face of the embankment at the left abutment (junction with natural soil) revealed a 10' x 10' wet zone about half way down. There was no flow noticed. Farther down to the left, water disappears and emerges again several times along the junction of the embankment with the natural ground surface (the crotch). The presence of several small springs indicates that the flow is coming from the embankment. Seepage also extends along the toe of the dam to the right abutment but does not go up the right abutment crotch. The bottom 5 to 10 feet of slope is wet and spongy. Water can be heard running below the ground surface before emergence. The springs (Fig. 5, Appendix D) empty into the creek where the 24-inch supply and mud pipes empty. Seepage water is clear, with no sloughing or erosion observed. Decay of vegetation in the seepage path (Fig. 6, Appendix D) indicates it has been going on for some time. The observation of seepage indicates that the cutoff wall is not fully effective.

### c. Appurtenant Structures

1. Gravity spillway section and spillway channel: Inspection of the concrete gravity overflow section and the spillway channel led to the following observations:

i. The overflow section itself (Fig. 2, Appendix D) appears to be in generally good condition with a moderate amount of spalling and erosion of its downstream slope and piers (Fig. 7, Appendix D).

ii. About one-third of the way down to the base of the overflow section along its junction with the left wing wall, there are signs of minor seepage indicated by wetness (but not flow) (Figs. 8 and 9, Appendix D). On top of the left wing wall of the spillway, there are signs of pressure grouting which might have been done to

control leaks, or the concrete may be the result of surface guniting; there is a scratched date of 1963.

iii. Minor spalling and scaling of wing walls has started (Fig. 8, Appendix D).

iv. The footbridge over the spillway is scaling, with a large spall near the midspan exposing steel (Figs. 3 and 7, Appendix D).

v. The spillway discharge channel (Fig. 10, Appendix D) has minor spalls at wall joints and on the wall. At the lower end of the spillway, there is a major spall on the left wall.

vi. The last floor slab of the spillway channel is cracked and seepage comes out from below the slab at its lower lip (Fig. 11, Appendix D).

2. Intake structure and outlet pipes: The intake structure consists of a vertical concrete shell with an 11-foot inside diameter and 6-inch thick wall. The top of the intake structure has a concrete floor with a central rectangular entrance to the intake box (Fig. 12, Appendix D). The entrance is covered with steel plate and locked with a chain. There is a circular pipe railing around the platform and three valve stems for the gates on top of the platform (Fig. 13, Appendix D). Gates for the supply line are located at 17-foot and 25-foot depths below the intake platform. The mud pipe extends 30 feet farther from the intake into the lake and is at a 40-foot depth. All three gate valves were turned and found to function smoothly. The intake box cover was removed, but the valves could not be seen because water inside the intake structure was at the pond level. The gate platform and intake structure wall are moderately spalled and scaled (Fig. 13, Appendix D).

The supply and mud pipes traverse under the embankment, through the lower gate house, and then empty into the natural stream (Fig. 14, Appendix D). The steel on the pipes at the outfall looked good.

3. Gate house: The gate house appeared to be in good condition. It has a wooden floor with stairs leading to lower level for access to gates. No lighting arrangement exists. There are four valves in the gate house. Two 24-inch valves are on the main water supply line, one 24-inch valve on the mud line, and one 16-inch valve on the pipe connecting the two lines. All valves

were cracked and found to be functioning smoothly. They are well maintained. Very muddy effluent resulted at the cracking of the mud pipe valve which was closed immediately. The lower gate house floor is a little wet because of seepage into the house, but the pipes and valves under the floor look good and dry.

d. Foundation

The foundation for the dam was not observed. Our geologic evaluation of the site indicates that there is an extensive cover of glacial deposits at the surface. (See Article 2.5)

e. Reservoir Area

The reservoir area is heavily wooded except for one naturally sandy area that is covered with moss and scrub only (Fig. 15, Appendix D). Side slopes of the reservoir are steep; about 1½ horizontal to 1 vertical. However, there is no evidence of slope failure. There is a moderate amount of sedimentation at the upstream entrance and an indication of sedimentation at the dam, suggested by the turbidity of water released from the mud pipe.

f. Downstream Channel

The downstream channel (Fig. 16, Appendix D) is clear of any obstruction or debris. The upper reaches of the channel form part of the water supply channel to the chlorination house. The valley and slopes are generally wooded and appear stable.

3.2 Evaluation

Seepage was observed starting about half-way down the downstream slope of the embankment along its left abutment, continuing along the toe to the right abutment, but does not go up along the crotch of the right abutment. The amount of seepage appears to be substantial, indicating that the cutoff wall is not entirely effective. Although there are indications that this condition has existed for some time and there are no signs of erosion or failure, further investigation is definitely advisable.

About one-third of the way down to base of the spillway overflow section along the junction with the left wing wall, there are signs of minor seepage indicated by wetness and discoloration of the concrete. At present this situation is not of major concern, but it could worsen

with time and the structural concrete may be affected by frost action.

The presence of large trees on the embankment slopes of earthfill dams ordinarily poses a potentially dangerous condition.

a) If the trees are shallow rooted, they could blow over in a major storm, carrying part of the embankment with them.

b) If the trees are deep rooted, the root systems may extend transversely through the embankment. Death of the trees and subsequent decay of the root systems may result in the formation of water passages (pipes). Such pipes provide natural channels for the seepage of water through the embankment; this may result in erosion of the embankment or in the generation of seepage forces that would adversely affect the stability of the slope.

c) The trees on the downstream face of the subject dam appeared to be well established. A study should be made to establish whether the trees are shallow rooted or deep rooted. If they are shallow rooted, removal is in order. If they are deep rooted, removal would be potentially more dangerous than leaving them in place; for this dam, the danger is substantially mitigated by the presence of the concrete cutoff wall.

## SECTION 4

### OPERATIONAL PROCEDURES

#### 4.1 Procedures

Personnel of the City of Johnstown Water Department (JWD) informed us that the water level in the reservoir is recorded on a twice-a-day basis for 7 days a week. Mr. Tim Newhouse of the Water Department is assigned to this study, in addition to recording water levels at other locations such as at the weir, farther downstream. There are no written procedures made available; however, we were informed by JWD personnel that, according to their established practice, water is lowered in late fall to about 1 foot below the spillway crest and retained there throughout the winter. The lake freezes in winter and the normal ice thickness is between 18 and 24 inches. By April, the pool level is further lowered to about 6 feet below spillway crest to receive and safely pass the spring and summer floods. Normally, water does not flow over the spillway; however, the highest water level reported in the early 1970s is about 2 feet above the spillway crest.

Two valves for the 24-inch supply line and one for the mud pipe, all three located at the intake structure, are kept open at all times. They are worked once a year to verify the satisfactory functioning of the mechanism. Water outflow from the reservoir is regulated by one of two supply line 24-inch valves located in the lower gate house. The other 24-inch valve, also located in the lower gate house, but operating the mud pipe, is kept closed except in emergencies. Water from the supply line (and mud pipe) drains out into a natural channel which leads to the downstream diversion works.

#### 4.2 Maintenance of Earth Embankment

The only apparent maintenance is periodic cutting of vegetation on the upstream face. Except for downstream seepage along the left abutment and toe, the earth embankment seems to be in generally good condition. There are no visible signs of sloughing, erosion or cracking of the embankment. The rip-rap on the upstream face extends up to the spillway crest level with no visible failures. There are extensive animal burrows but they should not pose any hazard in view of the barrier provided by the concrete cutoff wall. The downstream edge of the crest, and the downstream face are heavily wooded with

tall pines. There are some deciduous trees on downstream slope. Tree growth is starting on upstream face above rip-rap line, where cutting was apparently omitted for a while. The roadway on top of the dam is in good condition.

#### 4.3 Maintenance of Concrete Gravity Structure and Spillway Channel

Structurally, the gravity structure appears to be in good condition but needs general maintenance. The last general maintenance was apparently in 1963. The downstream face and piers have spalled and eroded moderately. Minor spalling and scaling of wing walls have started. The bridge is badly scaled with a large spall near midspan, exposing steel. About one-third of the way down to the base of the spillway overflow section along the junction with the left wing wall, there are signs of minor seepage indicated by wetness and discoloration. The spillway channel has minor spalls at wall joints and on the wall. At the lower end of the spillway channel there is a major spall on the left wall. The last floor slab of the spillway channel is cracked, and seepage comes out from below the slab at the lower lip.

#### 4.4 Maintenance of Operating Facilities

The access bridge to the intake structure is rusted and does not appear to have been painted since installation in 1963. The gate platform at the intake structure is moderately spalled and scaled. Valve stems at the intake structure and gate house turned, and were observed to function very well with little or no slack. At the outfall, the metal pipes appear to be in good condition. The lower gate house floor is a little wet from seepage into the house, but pipes and valves under the floor look good and dry. There is no lighting arrangement at the lower gate house.

#### 4.5 Warning Systems in Effect

The general condition of the dam and its appurtenant structures are checked daily as part of the pool elevation monitoring procedure. "Instructions for State of Emergency Conditions" have been written and posted on the office wall of the Department of Water so that all employees are familiar with them. Names of people to be notified in case of emergency also appear on those instructions. Access to the controls is maintained in the winter by plowing away the snow from the access road at high priority.

#### 4.6 Evaluation

Functionally, all parts seem to be in good working condition. Maintenance of the operating facilities appears to be satisfactory. However, some routine maintenance needs attention. Small woody growth on the upstream slope above rip-rap level should be taken out and no more woody growth allowed in the future. Minor concrete repairs on the spillway, wing walls, footbridge, spillway channel and intake structure should be undertaken. The rust on the access bridge to intake structure needs scraping followed by painting (being careful with the use of lead components or other potentially deleterious materials).

As a part of general maintenance, the mud pipe should be flushed at least annually to avoid clogging of its upstream open end.

Seepage between the spillway and the left abutment should be eliminated by injection grouting.

Provision of an electrical connection for light at the gate house or any other source of light such as a flashlight is very essential.

Instructions for handling emergency situations appear to be adequate, except for exceptionally high flows, as discussed later.



## SECTION 5

### HYDRAULICS AND HYDROLOGY

#### 5.1 Evaluation of Hydraulic Features

##### a. Design Data

The dimensions of the overflow spillway and the spillway channel are found on, or can be scaled from, Plates II and VI. Based on some referenced tables, that we are not familiar with, the overflow spillway capacity in the original file was calculated as 1630 cfs (Appendix E) for a 4-foot head, which is not too far from our calculation of the overflow spillway capacity as 1814 cfs (Appendix C) for the same head. However, with the raised crest level, the available head over the spillway is 3.75 feet without overtopping the earthen embankment, and for that head the spillway capacity according to our computations drops to 1650 cfs (Appendix C), which is very close to the original design.

Using Chezy and Kutter formulae, the original file arrives at 1670 cfs (Appendix E) as the capacity of the spillway channel for the designed dimensions and slope and water depth of 5 feet. We concur with this result.

The two 24-inch pipes have been shown to be capable of discharging a total of 150 cfs for a head of 20 feet of water. Their method and computations are correct but the available head of water at the upstream end of the mud pipe is 40 feet. We have not been able to establish the depth of the supply line, but the upstream head may be assumed as a minimum of 25 feet. Our estimate of discharge through these two pipes is 320 cfs.

The original flow computations are presented in Appendix E and computations performed as part of this study are found in Appendix C.

##### b. Experience Data

A record of water levels at the reservoir is available, but no measurements of flow are available. The maximum observed head of water over the spillway crest was reported to be about 2 feet in the early 1970s.

c. Visual Observations

The pool elevation on the day of the inspection was 16 inches below that of the spillway crest, so the spillway was not observed in operation; however, there is no reason to believe that it would not function satisfactorily. The maximum height of water that the spillway can accommodate without overtopping of the dam is 3.75 feet. The gate valves on the supply and mud lines at intake and lower gate house were turned and found to function satisfactorily. Only a small amount of flow was being maintained through the supply line. Opening of the mud pipe gate valve was followed by very muddy effluent, so the valve was closed immediately.

5.2 Evaluation of Hydrologic Features

a. Design Data

For calculation of the flood discharge for this dam, in the original computations of 1917, an empirical formula developed by Mr. McKim, the then inspector of dams, has been used. This formula is based on the drainage area of the watershed and gives a flood discharge of 1314 cfs for this watershed. A more detailed analysis was attempted later but left incomplete. These computations are found in Appendix E.

To our knowledge, there are no gaging stations in the local basin. According to the Recommended Guidelines for Safety Inspection of Dams, Department of the Army, OCE, the recommended Spillway Design Flood (SDF) for the subject dam is the Probable Maximum Flood (PMF) since the dam is of intermediate size and poses a high hazard.

b. Experience Data

Information on the PMF for the Cork Center Storage Reservoir and its watershed was obtained from the Upper Hudson and Mohawk River Basins Hydrologic Flood Routing Models prepared in 1976 for the New York District of the U.S. Army Corps of Engineers (USACE) by Resource Analysis, Inc. In this study, the rainfall-runoff mathematical model HEC-1 was used to reconstitute the major historical floods and to simulate the Standard Project Flood (SPF). In addition to the SPF simulation, the rainfall pattern for Tropical Storm Agnes was transposed to fall directly on the basins under study, and the discharges resulting from this rainfall were determined by an application of the calibrated model. In a telephone conversation with Mr. Thomas Smyth, USACE New York District, we

were informed that for Phase I hydrologic analyses, the Probable Maximum Flood (PMF) could be considered as twice the SPF.

The Cork Center Storage Reservoir and its drainage basin were located within Subarea 22 of the Mohawk Basin, Little Falls, N.Y. to Mouth. Computations for routing the PMF through the Cork Center Storage Reservoir are found in Appendix C of this report.

c. Visual Observations

The maximum observed flood over the spillway crest is about 2 feet in the early 1970s. Normally, the water does not flow over the spillway. This appears to be verified by the observable water marks on the wing walls at about spillway crest level (Fig. 2, Appendix D). We were informed by JWD personnel that the pool of the Cork Center Storage Reservoir is lowered to about 6 feet below the crest level. This may explain why heavy rains of the early 1970s were passed without difficulty.

d. Overtopping Potential

The computations in Appendix C indicate that the subject dam will be overtopped by the PMF. The maximum height of water that can flow over the spillway without the dam being overtopped is 3.75 feet. At that height the spillway passes approximately 1650 cfs. The routed PMF is approximately 3630 cfs. Therefore, the spillway can pass only 45 percent of the PMF.

e. Spillway Adequacy

The results of the hydrological analysis indicate that the spillway capacity is inadequate with respect to passing the PMF, and the topping of an earth dam often results in the rapid washout of a dam section. In addition, the spillway is considered seriously inadequate because it satisfies all of the following conditions set forth in DAEN-CWE-HY Engineer Technical Letter No. 1110-2-234 dated 10 May 1978:

1. There is high hazard to loss of life from large flows downstream of the dam.

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

3. The spillway is not capable of passing one-half of the Probable Maximum Flood without overtopping the dam and potentially causing failure. It may, however, be pointed out that if the supply and mud pipes are kept open, the total discharging capacity will increase to approximately 54% of the Probable Maximum Flood.

## SECTION 6

### STRUCTURAL STABILITY

#### 6.1 Evaluation of Structural Stability

##### a. Visual Observations

Visual observations of the earth embankment disclosed considerable amount of seepage along the downstream toe. Seepage starts at the junction of the left embankment with the virgin hillside about half-way down the downstream slope. It turns into flowing water along the toe and extends to the right abutment but does not go up along the crotch of the right abutment. The decay of vegetation in the seepage path indicates it has been going on for some time. The bottom 5 to 10 feet of the slope is wet and spongy but there is no sign of sloughing or erosion. The vertical and horizontal alignments of the embankment appear to have been maintained, and there is no evidence of cracks. Some roots of the pines growing on the downstream slope cross the embankment crest transversely; this condition may not exist at depth because of the presence of the concrete cutoff wall.

About one-third of the way down to base of the overflow spillway and along the junction with the left wing wall, there are signs of minor seepage indicated by wetness. At present, this situation is not of any concern, but it could worsen with time and the structural concrete could deteriorate due to frost action.

##### b. Design and Construction Data

The original computations of 1917, presented in Appendix E, were reviewed. Generally, these computations were found appropriate, except that in one case, the coefficient of active earth pressure was used for calculating passive resistance. Assuming 66% uplift pressure on the base of the dam, it was found safe against overturning about the toe. However, it was found unsafe with respect to sliding because the passive resistance to the cutoff walls was not considered. At a later stage in those computations, resistance to cutoff wall was taken into account but with an active earth pressure coefficient instead of a passive earth pressure coefficient, and the resistance of the downstream key was still not accounted for. Under these conditions the spillway was found to be just safe against sliding with a Factor of Safety of 1, assuming a coefficient of friction of 0.45 between the

concrete and the foundation soil. In the original analysis, ice pressure was neglected.

In the present analysis, the stability of the spillway section prior to 1963 and after remodeling in 1963 has been analyzed for various conditions of pool elevations and ice thrust. The results are shown in a tabular form in Appendix C. The uplift pressure on the base of the dam was taken as 50%, and the coefficient of friction against sliding as 0.3. The only critical condition encountered is in overturning when the pool is at spillway crest level and ice thrust of 4 kips/linear foot is applied 1 foot below the crest. It is, therefore, advisable that pool level during winter be kept at least 3 feet below the spillway crest level. This is mitigated somewhat by the fact that the spillway section has received ice thrusts for about 15 years without visible harm.

Since no information was available regarding the nature of the embankment and foundation materials and their properties, neither stability nor seepage analyses for the embankment could be performed as part of this study.

The present operational procedure provides for the water supply pipe and the mud pipe to be under pressure beneath the dam. This is considered undesirable, as a leak in the line could eventually result in washout of embankment soil, which could endanger the integrity of the dam.

c. Operating Records

None available.

d. Post Construction Changes

In 1963, the crest of the overflow spillway section was raised by 15 inches according to the drawing on Plate II. At the same time, an angle iron steel access bridge was constructed from the embankment crest to the intake structure. Probable pressure grouting or surface grouting around the spillway is indicated about the same time by a scratched date of 1963.

e. Seismic Stability

The Cork Center Storage Reservoir is nominally located on the border between Seismic Zone 1 and Seismic Zone 2 according to the Algermissen Seismic Risk Map. The

USACE guidelines suggest that in the event of doubt about the proper zone, the higher zone should be used. Although earthquakes that cause moderate damage can be expected to occur in Zone 2, the design and construction practices conventionally used for small earth dams are considered to be adequate in areas of low seismicity and the safety factors used for static conditions should preclude major damage for all but the most catastrophic earthquakes. However, no computations were performed to evaluate the effect of earthquakes on the subject dam.

## SECTION 7

### ASSESSMENT, RECOMMENDATIONS AND REMEDIAL MEASURES

#### 7.1 Dam Assessment

##### a. Safety

Visual inspection of the system and a review of the limited available engineering data indicate that the dam embankment and the overflow spillway are in generally good condition and functioning satisfactorily at this time. Our approximate hydrologic/hydraulic calculations indicate that the discharge capacity of the overflow spillway is seriously inadequate according to the OCE screening criteria. Although no signs of sloughing, erosion or cracking of the earthen embankment were observed, substantial amount of seepage along the downstream toe deserves further investigation. The stability of the embankment may then be analyzed in the light of the findings.

##### b. Adequacy of Information

The information available to us is not adequate for a detailed analysis of the stability of the embankment including seepage effects. However, the stability of the overflow spillway has been verified with reasonable degree of certainty, although the nature and properties of the foundation material are not clearly defined. Since there were no hydrologic data available, our assessment of the overtopping potential is based solely on transpositioning modelling results from nearby areas to the subject drainage basin.

##### c. Urgency

Inasmuch as the spillway capacity appears to be seriously inadequate according to the OCE screening criteria, there is some urgency in performing the additional study recommended below. Likewise, occurrence of seepage along the downstream toe of the embankment slope requires investigation at high priority. These investigations should be performed as soon as practicable; this should be within one year.

##### d. Necessity for Further Investigations

In view of the inadequacy of the overflow spillway in its inability to pass at least one half of



the computed PMF without overtopping the dam, and in view of the fact that overtopping in the case of earthfill dams is usually disastrous, the actual capacity of the spillway should be determined using more precise and sophisticated methods and procedures. This further investigation should be performed as soon as possible. Following this study, the need for and type of mitigating measures should be determined. Until such a study is completed, around-the-clock surveillance of the structure should be provided during periods of unusually heavy precipitation.

The occurrence of excessive amount of seepage along the toe of the embankment indicates that the cutoff wall has not been totally effective, which could be caused by any one or a combination of the following possibilities:

- i. Seepage under the cutoff wall.
- ii. Seepage through the cutoff that may have cracked.
- iii. Seepage around the cutoff wall through virgin soil at the left abutment.
- iv. Since there are no construction records available and also some items shown on drawings were not found during field inspection, the possibility that the cutoff wall was not constructed cannot be ruled out.

It is, therefore, recommended that borings be drilled through the downstream slope of the embankment, penetrating into the virgin soil, to establish the properties of both the embankment and the foundation materials. Later, piezometers may be installed in these borings to establish the seepage characteristics through the embankment. Subsequent stability analysis will provide a better understanding of the safety of this dam. If it were found safe, necessary protective measures to prevent piping failures (e.g. a subdrainage system and/or injection grouting) would then be recommended. Test pits should be dug along the centerline of the dam crest to verify the existence of the cutoff wall; the vertical dimension of the wall should be checked by coring.

## 7.2 Recommendations and Remedial Measures

### a. Alterations/Repairs

1) Injection grouting for fixing leak between the downstream slope of the overflow spillway section and the left spillway wall should be undertaken.

2) All minor damages to concrete (spalling, scaling, etc.) at spillway piers, spillway downstream slope, abutment walls, spillway channel walls, concrete footbridge over spillway and intake structure, should be repaired.

3) Rust should be scraped from the intake structure access bridge and the bridge subsequently painted, taking care not to contaminate the water with lead or other deleterious materials.

4) The gate house shaft should be lighted, either by an electrical circuit or by a system of battery-operated emergency lights.

5) The low woody growth on the upstream face of the dam should be removed.

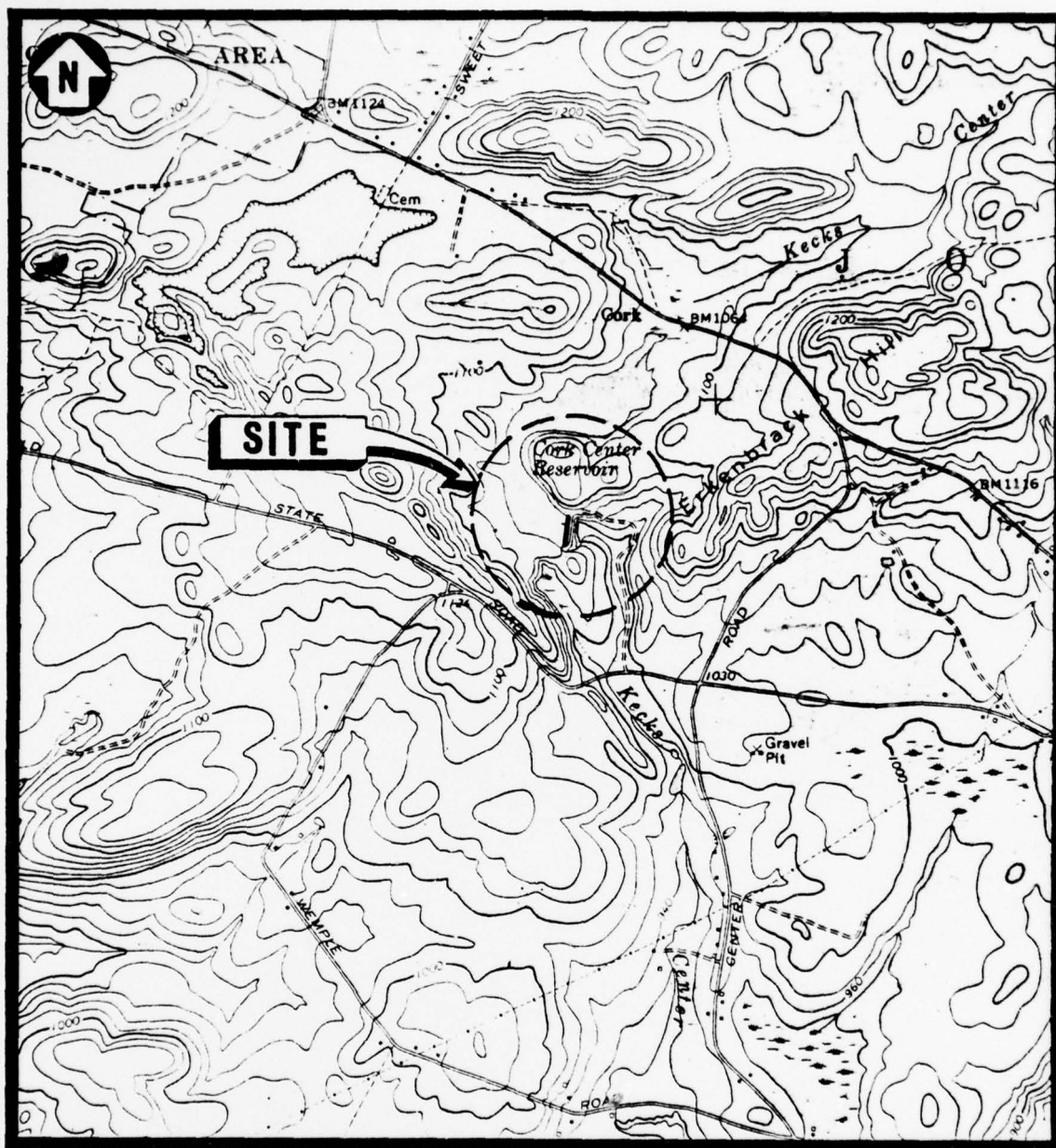
6) The large trees on the embankment should be investigated to determine whether they are shallow rooted or deep rooted. If shallow rooted, they should be cut down; if deep rooted, they should remain.

7) The flow from the lake should again be controlled from the intake structure rather than from the lower gate house, to avoid having pipes under pressure beneath the dam.

The remedial work recommended above is not critical in terms of urgency. It should be done as soon as practicable. Items 1, 5, 6 and 7 can be accomplished this year; all recommendations should be completed within the next three years.

b. Operations and Maintenance Programs

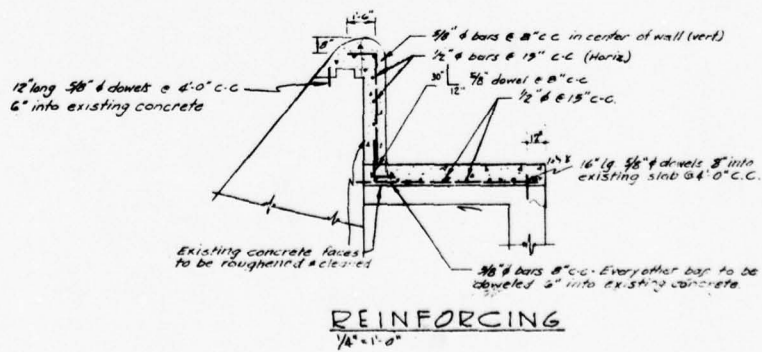
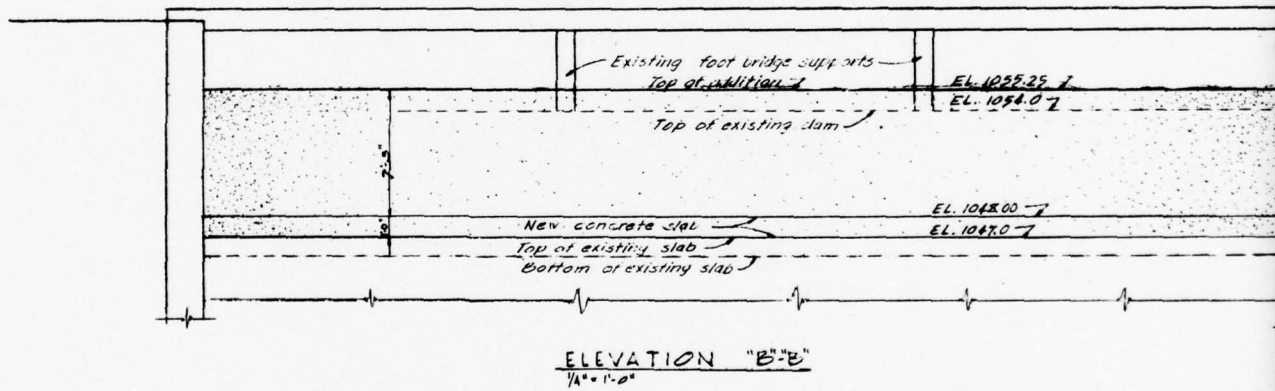
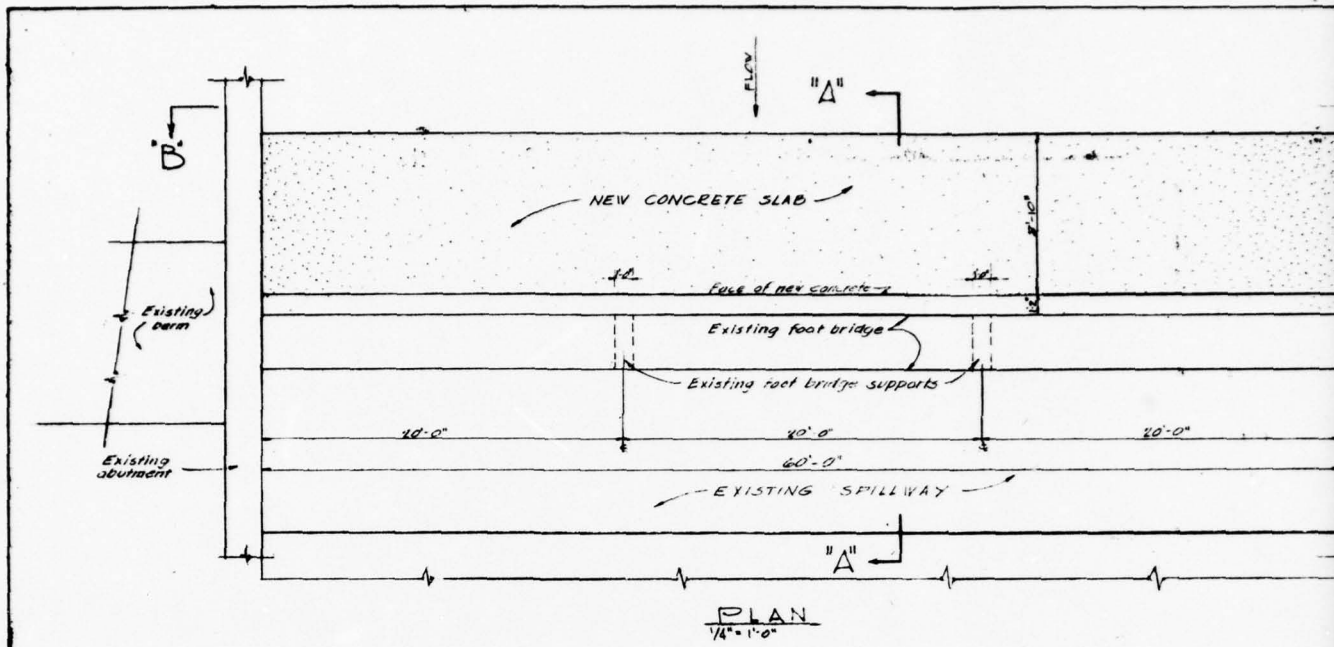
A specific program of periodic maintenance of the dam embankment and its appurtenant structures should be established and followed. This would include definite times for trimming of vegetation on the upstream slope, inspection and repair of concrete structures, testing of control valves for leakage, timely repair of the access road, etc. Periodically, water should be allowed to flow through the mud pipe (dirty water may be bypassed at the treatment plant) to avoid clogging of its open end with silt in the reservoir.



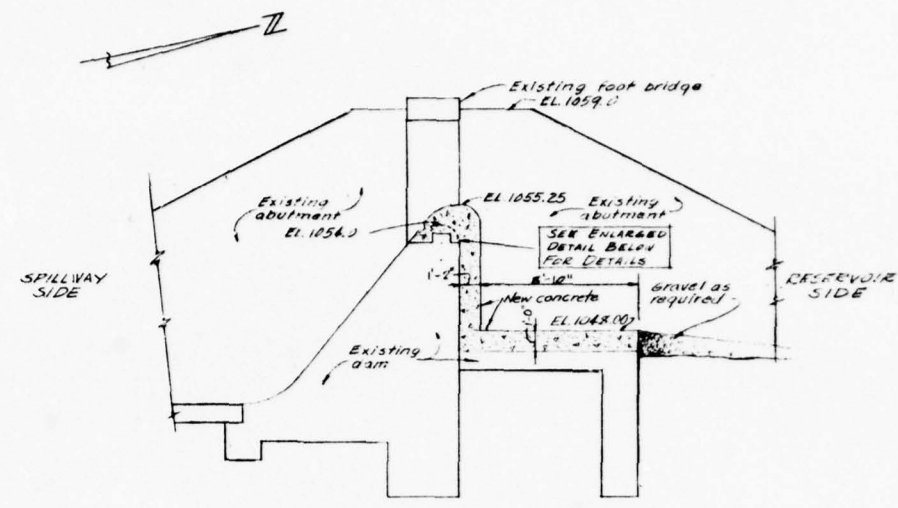
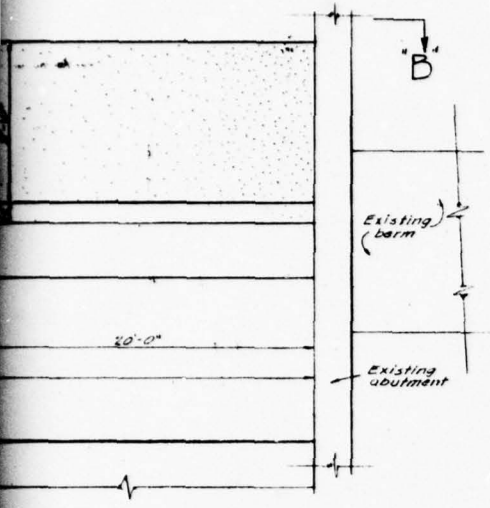
SCALE: 1"=2000'

MAP SOURCE: BASE MAP WAS ADAPTED FROM U.S. GEOLOGICAL SURVEY MAP, PECK LAKE, N.Y. QUADRANGLE, 7.5 MINUTE SERIES 1970. (BASE MAP MAY NOT REFLECT RECENT CARTOGRAPHIC CHANGES)

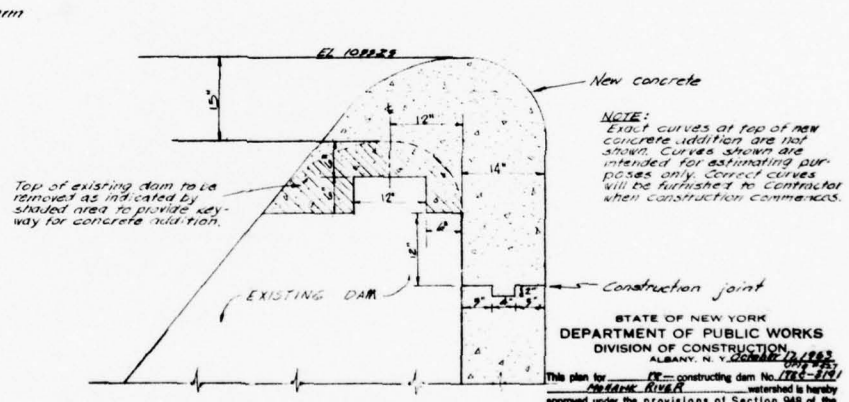
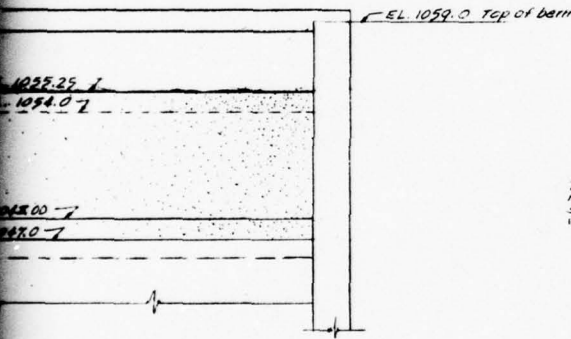
PLATE I SITE LOCATION MAP



DRAWN BY BOWEN  
CHECKED BY SIMON  
ENGRS BY E



SECTION "A"- "A"  
1/4" = 1'-0"



ENLARGED DETAIL  
1" = 1'-0"

STATE OF NEW YORK  
DEPARTMENT OF PUBLIC WORKS  
DIVISION OF CONSTRUCTION  
ALBANY, N. Y. *Stamp: 10/1/78*

This plan for *the State of New York* constructing dam No. *1785-214* watershed is hereby approved under the provisions of Section 94B of the Conservation Law.

Examined and recommended to the Chief Engineer for approval.  
*Alvin V. ...*  
ASSOCIATE CIVIL ENGINEER

APPROVED  
CHIEF ENGINEER  
Department of Public Works  
By *Robert D. ...*  
Chief Engineer

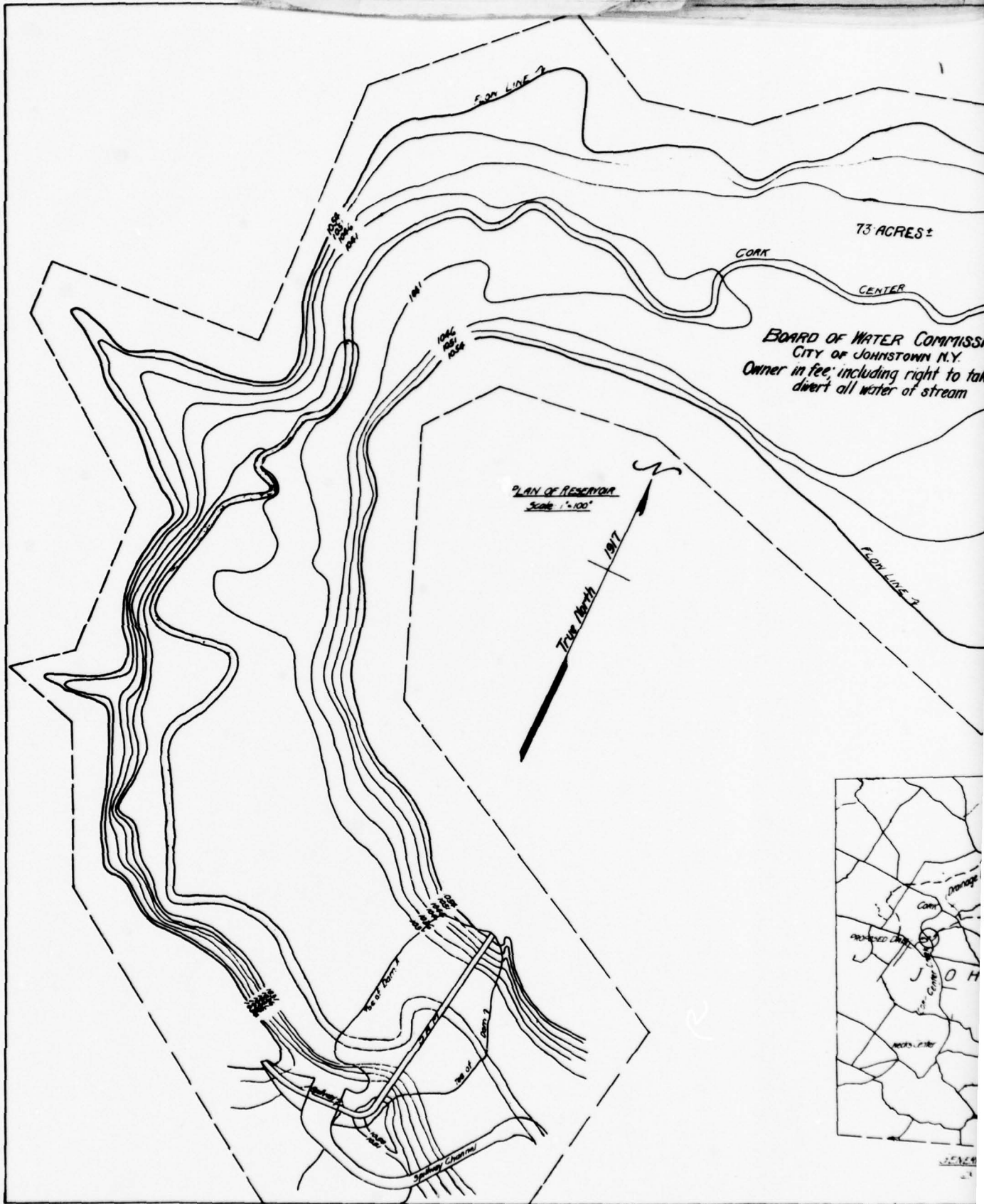
GENERAL NOTES

- Contractor shall be responsible for all dimensions and conditions of the construction site and be responsible for the accuracy of same.

CONVERSE WARD DAVIS DIXON  
CONSULTING ENGINEERS  
PLATE II AUGUST 1978

RECEIVED  
DEPT. CHIEF ENG.  
OCT 1 1978  
FLOOD CONTROL  
CANALS

CITY OF JOHNSTOWN  
FULTON COUNTY, NEW YORK  
WATER WORKS IMPROVEMENTS  
RAISING CORK CENTER  
STORAGE RESERVOIR DAM  
SCALE: AS SHOWN SEPTEMBER 1963  
*Morrell Vrooman, Jr.*  
MORRELL VROOMAN ENGINEERS  
CLOVERVILLE, NEW YORK  
LICENSE NO. N.Y. 22134



2

73 ACRES ±

CORK

CENTER

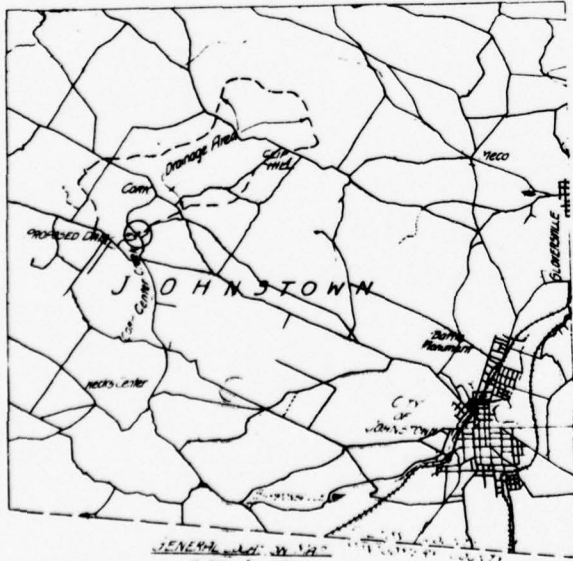
CREEK

BOARD OF WATER COMMISSIONERS  
CITY OF JOHNSTOWN N.Y.  
Owner in fee, including right to take and  
divert all water of stream

FLOOD LINE - A

172000000 Feet

ORIG #427  
NEW 172C-3191  
MOHAWK RIVER  
Adopted & Approved by Resolution  
of the Board of Water Commissioners  
of the City of Johnstown, adopted  
July 9, 1917.  
*Joseph H. ...* President  
*John P. ...* City Clerk

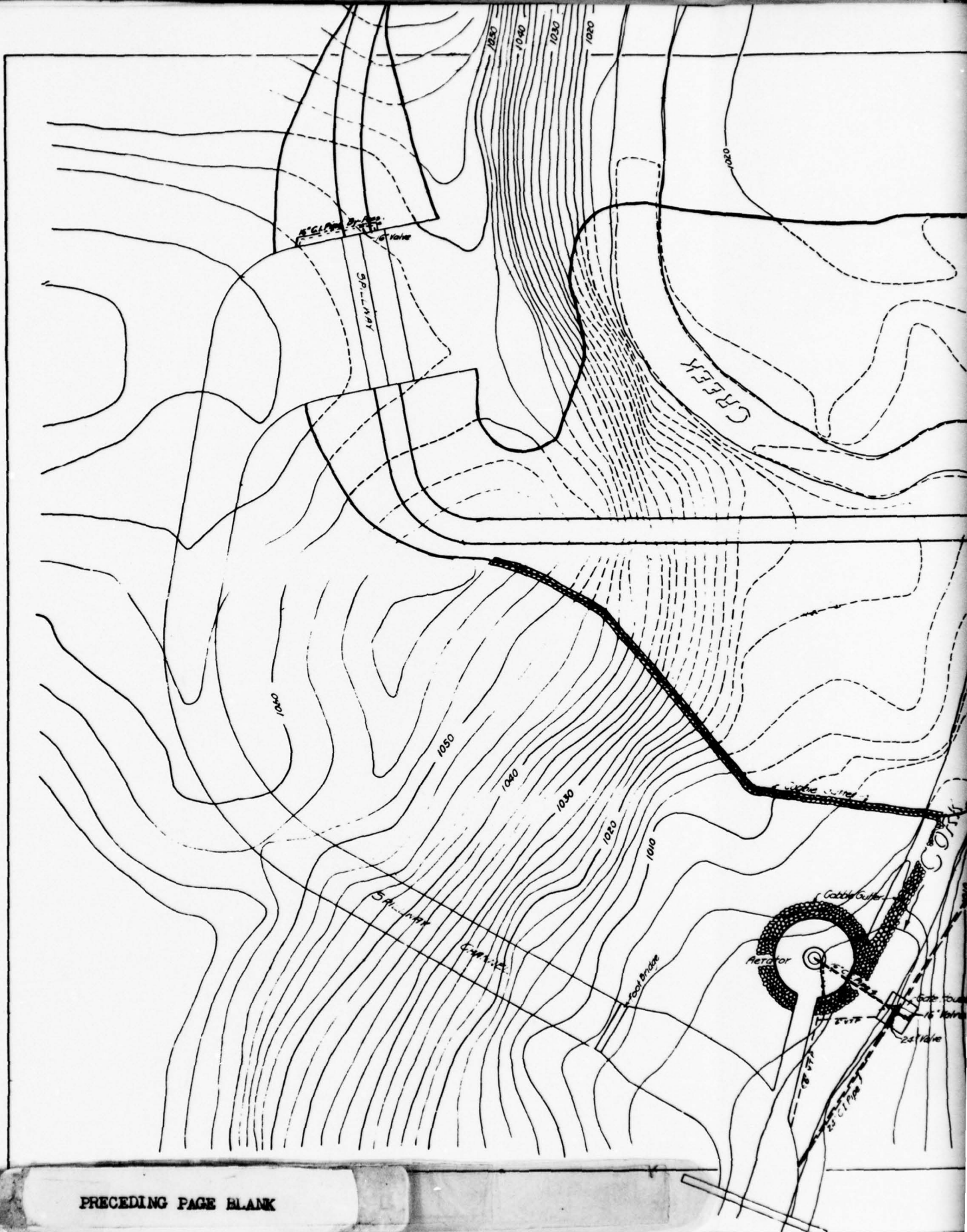


JOHNSTOWN WATER SUPPLY  
CORK CENTER CREEK  
FULTON COUNTY, N.Y.  
RESERVOIR AND LOCATION MAP  
Scale As Indicated

1

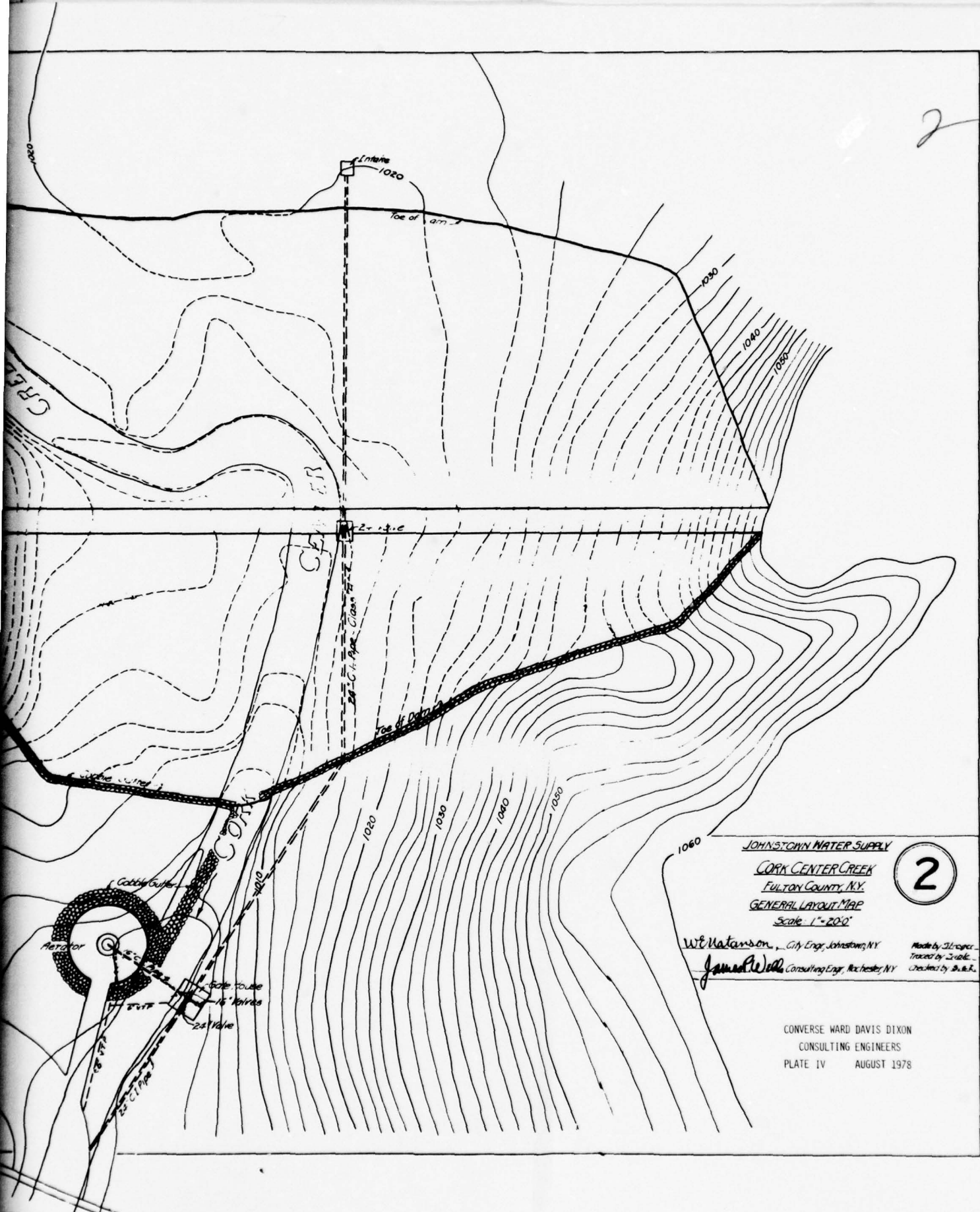
*W. Malanson*, Civil Engr., Rochester, NY  
*James Wells*, Consulting Engr., Rochester, NY  
Made by *George S. ...*  
Traced by *George S. ...*  
Checked by *...*

CONVERSE WARD DAVIS DIXON  
CONSULTING ENGINEERS  
PLATE III AUGUST 1978



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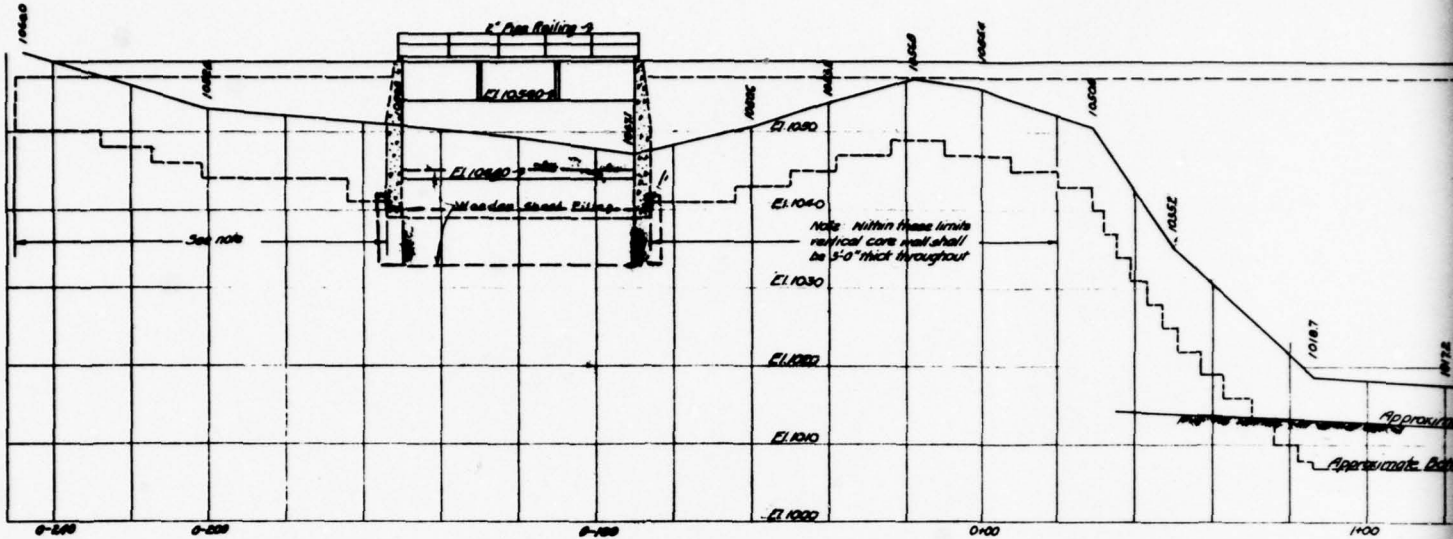
JOHNSTOWN WATER SUPPLY  
 CORK CENTER CREEK  
 FULTON COUNTY, N.Y.  
 GENERAL LAYOUT MAP  
 Scale 1" = 200'

2

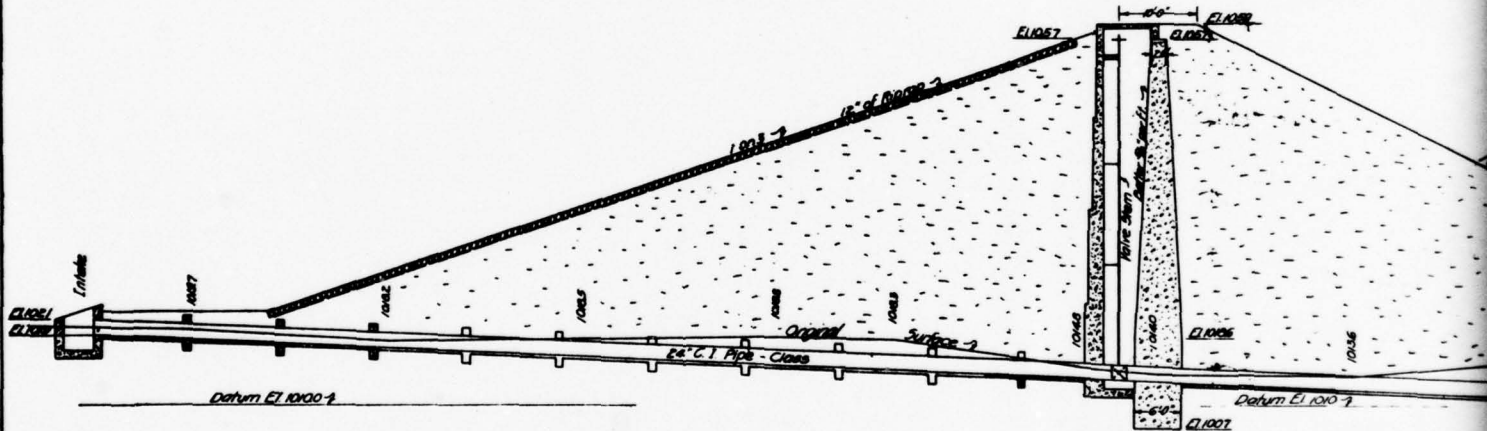
W. E. Watsonson, City Engr., Johnstown, NY  
 James W. Wells, Consulting Engr., Rochester, NY  
 Made by J. L. Rogers  
 Traced by S. G. L. L.  
 Checked by S. G. L. L.

CONVERSE WARD DAVIS DIXON  
 CONSULTING ENGINEERS  
 PLATE IV AUGUST 1978

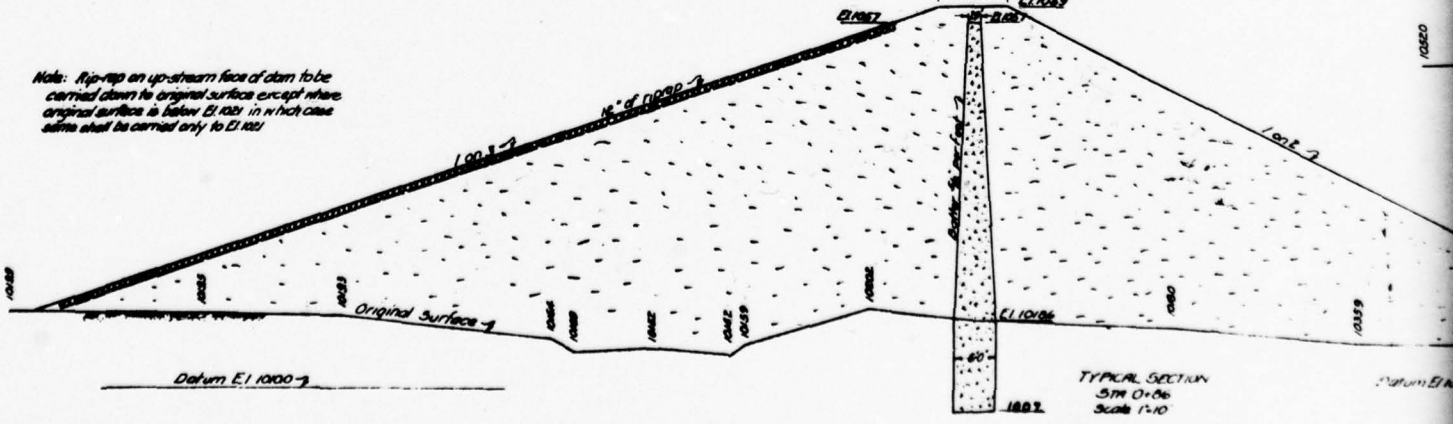
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PROFILE ALONG E OF DAM  
Scale: Hor 1"=20'  
Vert. 1"=10'

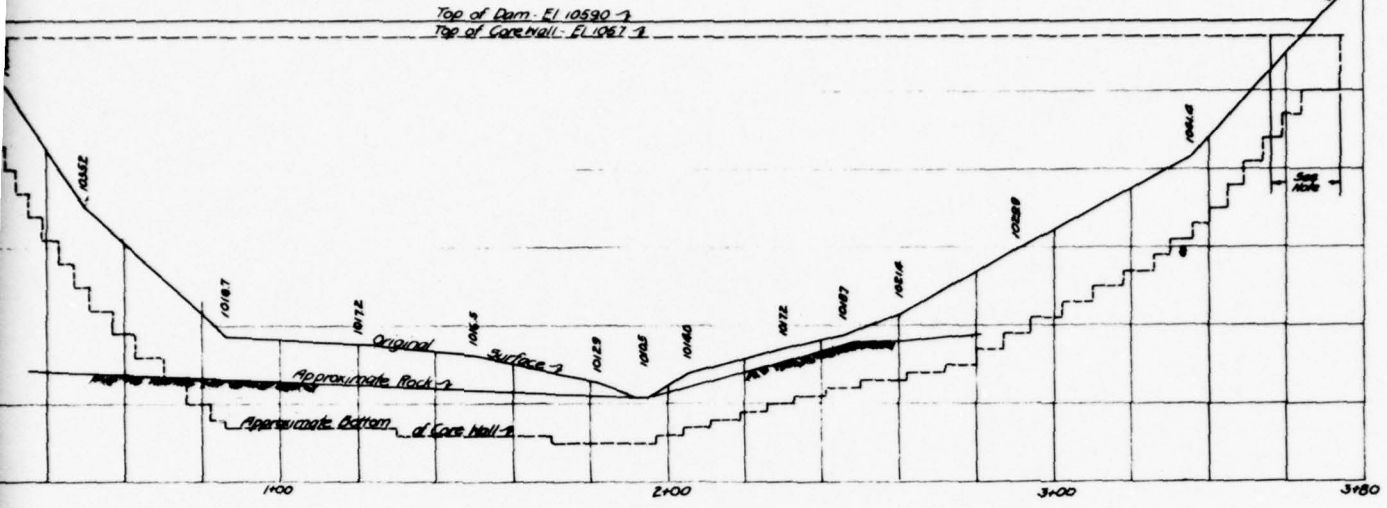


TYPICAL SECTION  
574 2+05'  
Scale: 1"=10'

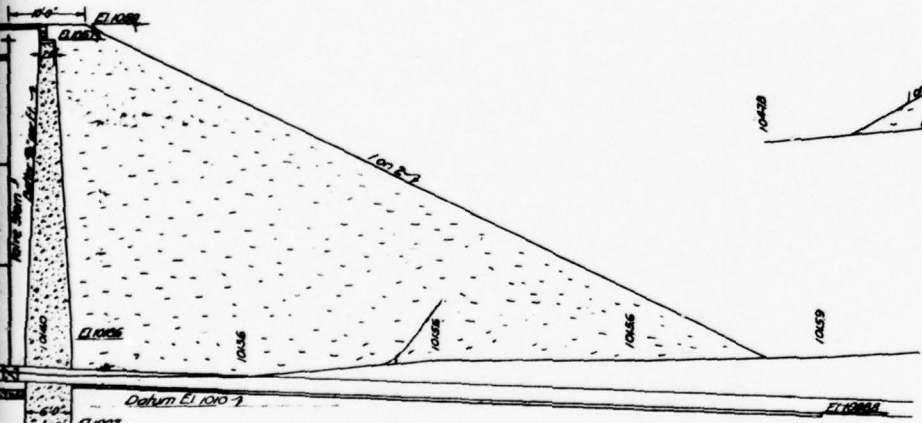


Note: Rip-rap on upstream face of dam to be carried down to original surface except where original surface is below El. 1021 in which case rip-rap shall be carried only to El. 1021

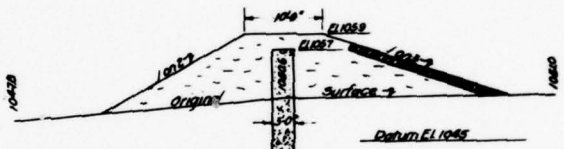
TYPICAL SECTION  
574 0+06'  
Scale: 1"=10'



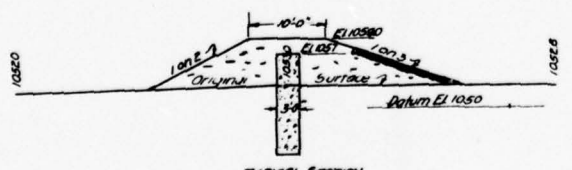
PROFILE ALONG E OF DAM  
 Scale: Hor. 1"=20'  
 Vert. 1"=10'



TYPICAL SECTION  
 Sta 0+25  
 Scale 1"=10'



TYPICAL SECTION  
 Sta 0+60  
 Scale 1"=10'



TYPICAL SECTION  
 Sta 0+86  
 Scale 1"=10'

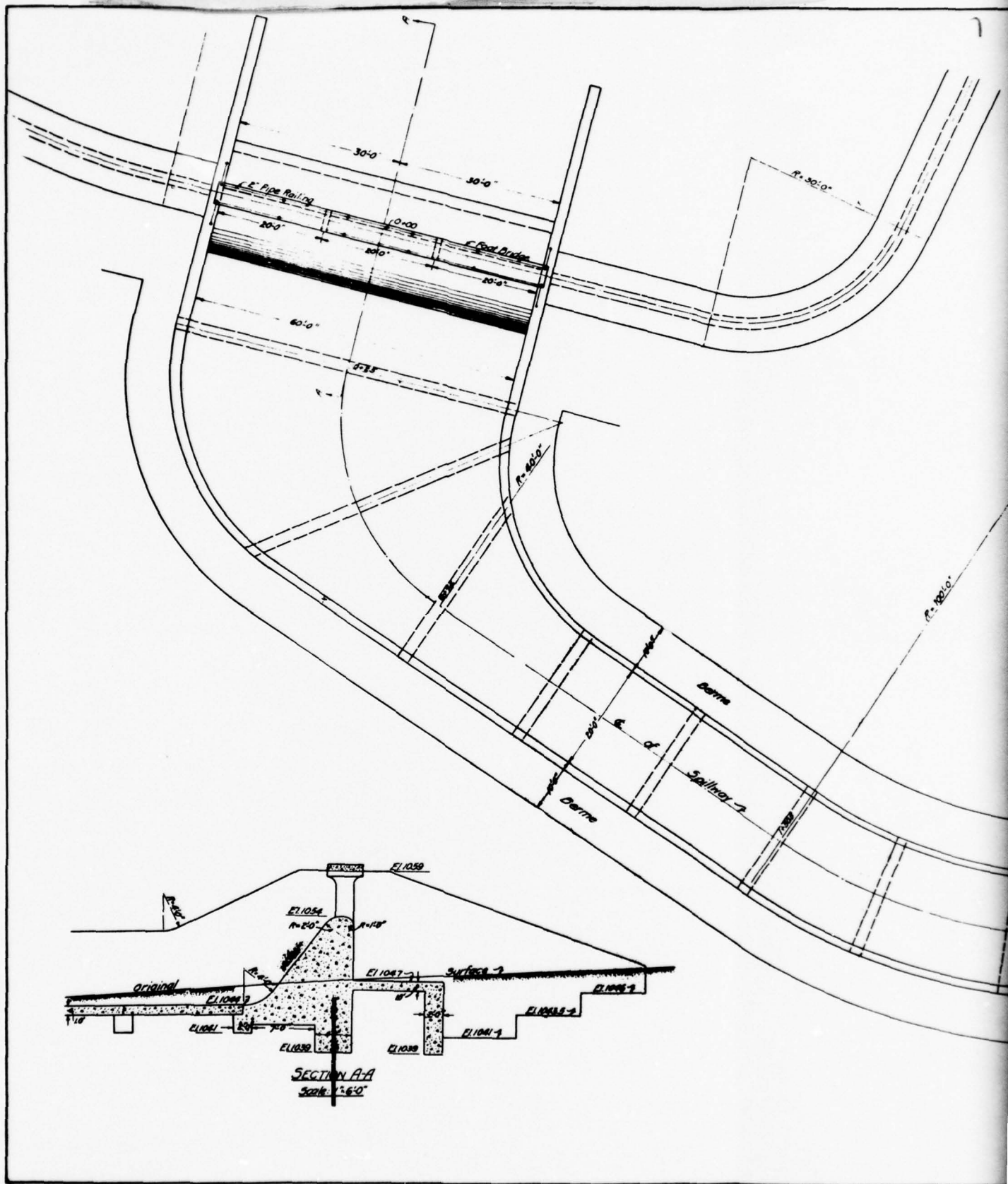
TYPICAL SECTION  
 Sta 0+86  
 Scale 1"=10'

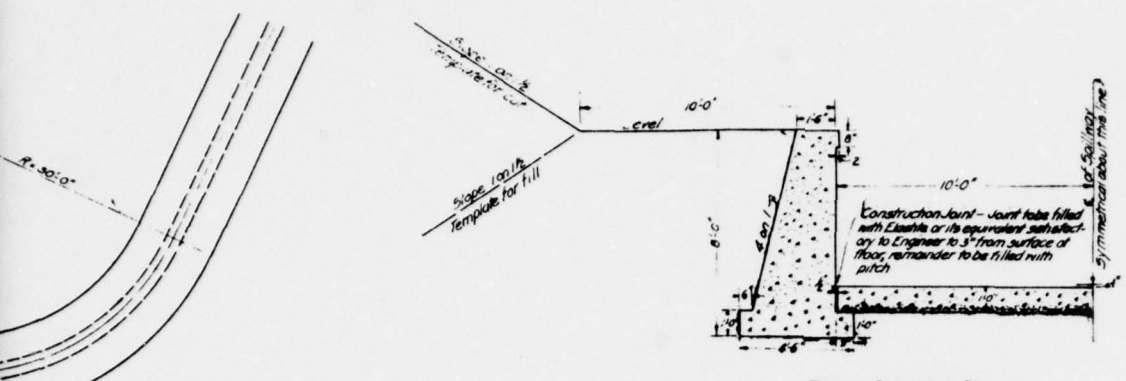
JOHNSTOWN WATER SUPPLY  
 CORK CENTER CREEK  
 FULTON COUNTY, N.Y.  
 PROFILE AND SECTIONS OF DAM  
 Scale: As indicated

W. Materson, City Engr., Johnstown, N.Y.  
 James P. Wells, Consulting Engr., Rochester, N.Y.

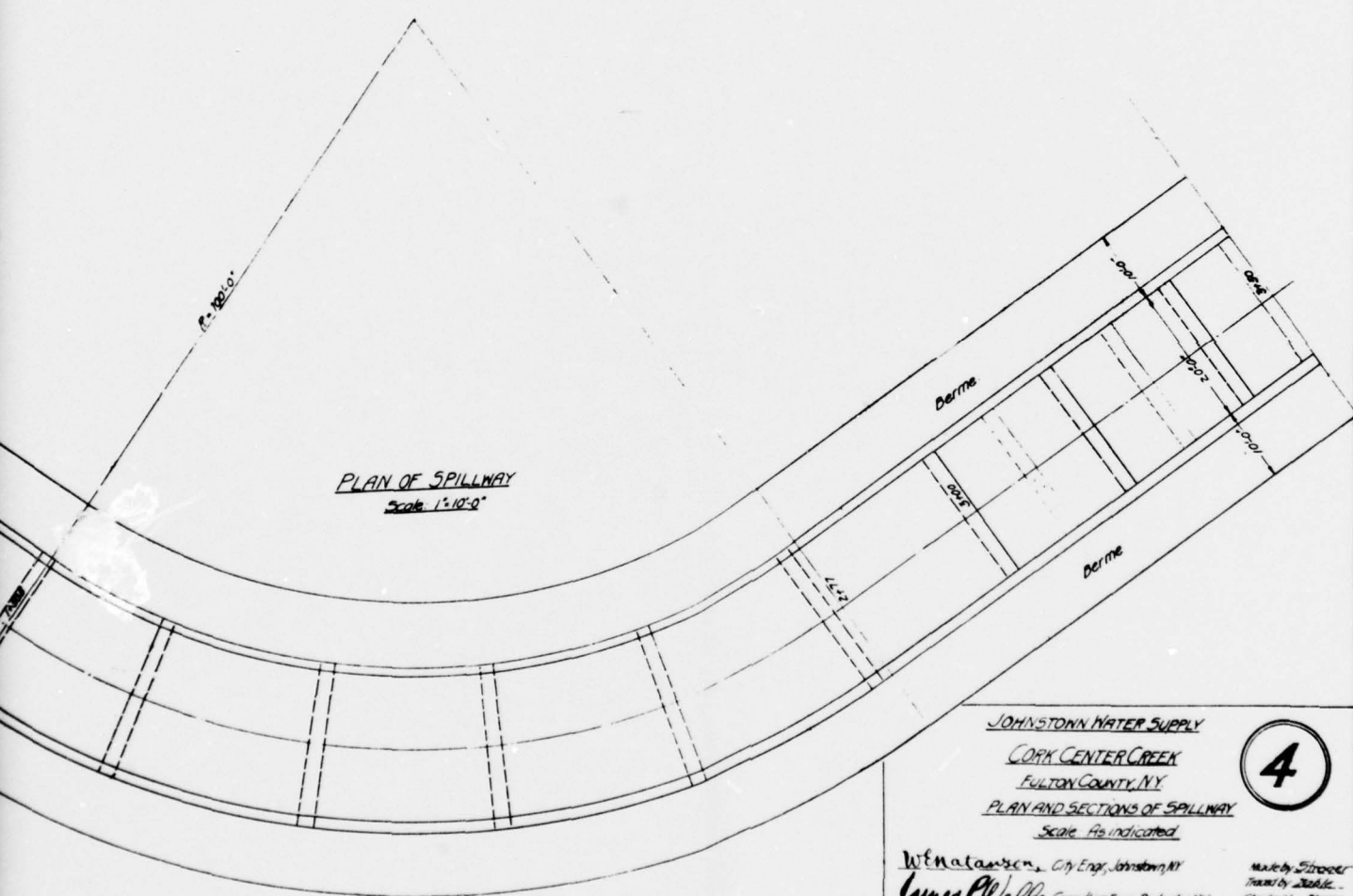
Made by: S. J. ROSE  
 Traced by: J. A. H. K.  
 Checked by: S. J. ROSE







TYPICAL SECTION OF SPILLWAY  
Scale: 1"=5'-0"



PLAN OF SPILLWAY  
Scale: 1"=10'-0"

JOHNSTON WATER SUPPLY  
CORK CENTER CREEK  
FULTON COUNTY, NY  
PLAN AND SECTIONS OF SPILLWAY  
Scale As Indicated

4

Wenatansen, City Eng., Johnston, NY  
James P. Walla, Consulting Eng., Rochester, NY  
Made by Strover  
Drawn by [unclear]  
Checked by [unclear]

CONVERSE WARD DAVIS DIXON  
CONSULTING ENGINEERS  
PLATE VI AUGUST 1978

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APPENDIX A  
CHECKLIST - ENGINEERING DATA

CHECKLIST

HYDROLOGIC AND HYDRAULIC DATA

ENGINEERING DATA

NAME OF DAM: Cork Center Storage Reservoir NDS ID NO.: NY 658  
Dam  
RATED CAPACITY (ACRE-FEET) 445 NYS DEC ID NO.: 172C-3191  
ELEVATION TOP NORMAL POOL (STORAGE CAPACITY): Varies; approx. 1054  
ELEVATION TOP FLOOD CONTROL POOL (STORAGE CAPACITY): 1055.25  
ELEVATION MAXIMUM DESIGN POOL: 1059  
ELEVATION TOP DAM: 1059  
CREST: (Overflow Spillway):

- a. Elevation 1055.25
- b. Type Gravity concrete overflow spillway; rounded-crest
- c. Width Rounded; approximately 3 feet
- d. Length 60 feet
- e. Location Spillover Near right end of embankment
- f. Number and Type of Gates None

OUTLET WORKS:

- a. Type 24-inch C.I. water supply pipe
- b. Location Under the embankment; almost along its deepest
- c. Entrance inverts 1043± and 1035± (supply pipe); \* section
- d. Exit inverts 1002±
- e. Emergency draindown facilities 24-inch C.I. mud pipe

HYDROMETEOROLOGICAL GAGES:

- a. Type None
- b. Location None
- c. Records None

MAXIMUM NON-DAMAGING DISCHARGE: Unknown; 1650 cfs (estimate)

\*1020± (mud pipe)

CHECKLIST

ENGINEERING DATA

DESIGN, CONSTRUCTION, AND OPERATION  
PHASE I

NAME OF DAM: Cork Center Storage Reservoir

NDS ID NO.: NY658NYS DEC ID NO.: 172C-3191

Sheet 1 of 5

ITEM	REMARKS
DRAWINGS	<p>Plate II: Remodelling details of the spillway crest (1963)</p> <p>Plates III through VI: Plans and cross sections of the embankment and associated structures (1917)</p>
REGIONAL VICINITY MAP	<p>Dam shown (Plate I) on USGS 7½ minute quadrangle sheet of Peck Lake, N.Y. (N43°02'14", W74°27'55")</p>
CONSTRUCTION HISTORY	<p>None available</p>
TYPICAL SECTIONS OF DAM	<p>Sections through earthen embankment and overflow spillway shown on Plates V and VI</p>
HYDROLOGIC/HYDRAULIC DATA	<p>USACE Hydrologic Model for Mohawk River Basin and computed capacity vs pool elevation are the only hydrologic data available. Some hydraulic data available.</p>



ITEM	REMARKS
OUTLETS: Plan Details Constraints Discharge Ratings	Changed from what is shown on Plate IV. See Figures 69 and 70 in Appendix E.
RAINFALL/RESERVOIR RECORDS	Records of reservoir water levels available for a certain period extending into the past. No rainfall or any other gages in this basin to our knowledge.
DESIGN REPORTS	Not available
GEOLOGY REPORTS	Very sketchy one-paragraph report (reproduced in Appendix E) by Mr. A. R. McKim, the then inspector of docks and dams for Conservation Commission, State of N.Y.
DESIGN COMPUTATIONS: Hydrology & Hydraulics Dam Stability Seepage Studies	Limited hydrology and hydraulics data and detailed spillway stability analysis (Appendix E). No seepage studies or stability analysis for the earth embankment.

ITEM	REMARKS
MATERIALS INVESTIGATIONS Boring Records Laboratory Field	The reconstruction application (1963) indicates that the embankment is gravel with cobbles and boulders, and the bed is boulders. A letter dated 7-23-17 says natural soil is loamy earth with some boulders. Materials testing of concrete in 1919 indicated greater than 3000 psi strength.
POST-CONSTRUCTION SURVEYS OF DAM	None available
BORROW SOURCES	None available
MONITORING SYSTEMS	None
MODIFICATIONS	24-inch gate valve on the dam crest, 16-inch C.I. bypass pipe on the right of spillway right abutment wall and aeration tank shown on Plate IV were not encountered during field inspection - probably not constructed. In 1963 spillway crest raised by 15 inches from El. 1054 to El. 1055.25; angle-iron access bridge to intake structure constructed. Date of tree planting on downstream slope not known.

ENGINEERING DATA

ITEM	REMARKS
HIGH POOL RECORDS	Maximum observed 2 feet above spillway crest in the early 1970s
POST-CONSTRUCTION ENGINEERING STUDIES AND REPORTS	None
PRIOR ACCIDENTS OR FAILURE OF DAM Description Reports	None reported
MAINTENANCE AND OPERATION RECORDS	None available
SPILLWAY: Plan Sections Details	Plan, elevations and sections in Plates II, V and VI

ENGINEERING DATA

ITEM	REMARKS
<p>OPERATING EQUIPMENT: Plans Details</p>	<p>None available</p>
<p>PREVIOUS INSPECTION Date: Findings</p>	<p>Inspections are performed periodically by NYSDEC. The last inspection report on file is dated 10-23-69. Report, in part, indicates: "Adequate no apparent repairs needed or minor repairs that can be covered by periodic maintenance."</p>

APPENDIX B

CHECKLIST - VISUAL INSPECTION

CHECKLIST

VISUAL INSPECTION

PHASE I

NAME OF DAM: Cork Center Storage Reservoir Dam County: Fulton State: New York NDS ID No.: NY 658  
Keck Creek NYS DEC ID No.: 172C-3191

Type of Dam: Earthfill-Concrete Core Wall Hazard Category: High  
Date(s) Inspection: 20 July 1978 Weather: Sunny, Warm Temperature: 80°F

Pool Elevation at Time of Inspection: 1053.9 msl 16" below top of spillway crest

Tailwater at Time of Inspection: None msl (Below lower gate house)

Inspection Personnel:

E. A. Nowatzki (CWDD) C. Ackerbauer (City of Johnstown)  
G. S. Salzman (CWDD) R. Lake (City of Johnstown)  
T. Newhouse (City of Johnstown)

E. A. Nowatzki Recorder

Remarks:  
Reservoir locally called "High Daddy"

EMBANKMENT

Sheet 1 of 3

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SURFACE CRACKS	None visible - animal burrows throughout	
UNUSUAL MOVEMENT OR CRACKING AT OR BEYOND THE TOE	None visible	
SLOUGHING OR EROSION: Embankment Slopes Abutment Slopes	None visible for either	
VERTICAL AND HORIZONTAL ALIGNMENT OF THE CREST	Both OK	
RIPRAP FAILURES	None visible - rip-rap extends up to about spillway crest elevation. Vegetation above rip-rap to dam crest.	Woody vegetation on upstream slope above rip-rap should be removed.

EMBANKMENT

Sheet 2 of 3

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
<p>JUNCTION OF EMBANKMENT WITH:                      Abutment                      Spillway                      Other Features</p>	<p>Embankment with left abutment: Upstream OK.                      Downstream - moderate seepage starting in crotch about 1/4 way down slope. Ground wet. Some springs noted along (REFER TO SHEET 3)</p>	
<p>ANY NOTICEABLE SEEPAGE</p>	<p>Seepage noted where downstream embankment meets left abutment (see above). Springs noted. Seepage extends across to toe to right abutment but (REFER TO SHEET 3)</p>	<p>See recommendations in text of the report.</p>
<p>RECORDING INSTRUMENTATION</p>	<p>None</p>	
<p>DRAINS</p>	<p>None</p>	
<p>OTHER</p>	<p>Downstream edge of crest and downstream face heavily wooded with tall pines. Also deciduous trees on downstream slope. 2 large pines on (REFER TO SHEET 3)</p>	<p>Roadway on top of dam in good condition. Some pine roots cross it at surface.</p>



VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
<p>JUNCTION OF EMBANKMENT WITH:                      Abutment                      Spillway                      Other Features</p>	<p>crotch and continuing along toe to right embankment crotch. Embankment with right abutment: Upstream OK. Downstream - wet spots in crotch near toe. Embankment with spillway: Shallow embankment sections to left and right of spillway which is located at right angle to main dam at right abutment. No problems evident.</p>	
<p>ANY NOTICEABLE SEEPAGE</p>	<p>does not go up right abutment crotch. Wet zones noted up to about 10-15' vertically above toe. Can hear water running below ground before emergence. Ground wet and spongy. Spring empties into creek channel where 24" supply &amp; mudline pipes empty. Seeping water is clear - no sloughing noted. Decay of vegetation in seepage path indicates it has been going on for some time. No erosion observed.</p>	
<p>OTHER</p>	<p>upstream face near left abutment. Tree growth starting on upstream face above rip-rap line.</p>	

OUTLET WORKS

Sheet 1 of 2

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CRACKING AND SPALLING OF CONCRETE SURFACES IN OUTLET CONDUIT	Outlet is steel pipe - not visible except at discharge point - looks OK.	
INTAKE STRUCTURE	Access bridge rusted. Gate platform moderately spalled and scaled. Valves for 17' and 25' entry open at all times. Turned and observed (REFER TO SHEET 2)	
OUTLET STRUCTURE	Two 24" pipes go from lake intake structure, under dam, to lower gate house, and then to creek channel. 3 valves in lower gate house, 2 on main (REFER TO SHEET 2)	
OUTLET CHANNEL	Two 24" pipes to creek channel. Steel or iron pipes look OK.	
EMERGENCY GATE	Mud pipe serves as emergency outlet.	

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
INTAKE STRUCTURE	to function OK. 24" mud pipe extends about 30 feet into lake. Valve remains opened at all times. Crack valve OK. Water level inside intake structure at lake level; therefore could not see valves.	
OUTLET STRUCTURE	line - tested & functioning. There was a flow being maintained. Mud pipe valve cracked & functions; effluent very muddy - turned off immediately.	Should have light to valves under wood floor. Valves well maintained - little or no slack.

UNGATED SPILLWAY

Sheet 1 of 2

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CONCRETE WEIR	Generally OK - Downstream face moderately spalled and eroded.	
APPROACH CHANNEL	Minor spalling and scaling on wing walls.	
DISCHARGE CHANNEL	Minor spalling at wall joints and on wall. Major spall on left wall at lower end of spillway channel. Last floor slab cracked - some seepage (REFER TO SHEET 2)	Seepage is too far below spillway to influence dam.
BRIDGE AND PIERS	Piers moderately eroded (some spalling). Bridge scaled badly. Large spall (exposed steel) about midspan. Other spalls on concrete walkway.	
JUNCTION WITH LEFT WING WALL	Some minor seepage starting about 1/3 of way down to base of spillway as shown by wetness but not flow.	Injection grouting should be done to avoid further deterioration.

UNGATED SPILLWAY

Sheet 2 of 2

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
DISCHARGE CHANNEL	coming from below slab at lower lip.	

INSTRUMENTATION

Sheet 1 of 1

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
MONUMENTATION/SURVEYS	Downstream flow monitored for water supply purposes.	
OBSERVATION WELLS	None	
WEIRS	Downstream at chlorination house.	
PIEZOMETERS	None	
OTHER	Water level monitored and recorded twice daily at spillway.	

RESERVOIR

Sheet 1 of 1

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SLOPES	<p>Steep down to water - about 1½ horizontal to 1 vertical. One naturally sandy area covered with moss and scrub, otherwise heavily wooded.</p>	<p>No evidence of slope failure.</p>
SEDIMENTATION	<p>Moderate at upstream entrance. Indication of sedimentation at dam by turbidity of water released from mud pipe.</p>	

DOWNSTREAM CHANNEL

Sheet 1 of 1

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
<p>CONDITION Obstructions Debris Other</p>	<p>No obstructions or debris. Upper reaches are controlled for chlorination house (water supply channel).</p>	
<p>SLOPES Cover Stability</p>	<p>Generally wooded - appear good.</p>	
<p>APPROXIMATE NUMBER OF HOMES AND POPULATION</p>	<p>One house and barn (Miller Rd) within mile of dam creek. Trailer off Wemple Rd (about 1½ miles downstream) + 2 houses in Keck's Center (about 2 miles downstream).</p>	<p>Concur with high hazard designation.</p>



APPENDIX C  
COMPUTATIONS

BY J.K. DATE 8/7/78

JOSEPH S. WARD

SHEET NO. 1 OF 13

CHKD. BY P.M. DATE 7/1/78

91 ROSELAND AVE. CALDWELL, N. J.

JOB NO. A-7805-11 F

SUBJECT Cork Center Storage Reservoir - Hydrology

Classification of Dam

Maximum height of dam = 40 ft.  $\rightarrow$  49 ft from sheet # 3 (Plate 5)  
 of Johnston's Water Supply Information obtained from Oct. 7, 1963 application for raising cork center storage reservoir spillway crest

Storage at spillcrest elevation (1055.25') = 145,000,000 gallons  
 = 445 acre ft.

The dam is, therefore, 'intermediate size' based on its height as determined from the Department of the Army Office of the Chief Engineer's "Recommended guidelines for Safety Inspection of Dams". Since the dam has been classified as a high hazard dam, the aforementioned publication requires the spillway to be designed for passing the probable maximum flood (PMF)

Flow over spillway when pool is at the top of dam is El. 1059.0'

Effective length of spillway <sup>crest</sup>  $L = L' - 2(NK_p + K_a) H_e$

$L' = 58'$

$N = 2$

$K_p = 0.02$

$K_a = 0.0$

$H_e = 1059 - 1055.25 = 3.75 \text{ ft.}$

Bureau of Reclamation  
 Design of Small Dams  
 page 113

$\therefore L = 58 - 2(2 \times 0.02 + 0) 3.75 = 57.7 \text{ ft.}$

BY J. E. DATE 8/7/58

JOSEPH S. WARD

SHEET NO. 2 OF 13

CHKD. BY P. DATE 7/9/58 91 ROSELAND AVE. CALDWELL, N. J.

JOB NO. A7805-11F

SUBJECT Cork Center Storage Reservoir - Hydrology

$$Q = C_o L H_o^{3/2}$$

Assume the crest to behave as ogee crest, see from page 378 of Bureau's Design of Small Dam

for  $P = 7.25 \text{ ft.}$ ,  $H_o = 3.75 \text{ ft.}$   $\frac{P}{H_o} = \frac{7.25}{3.75} = 1.93$   
and  $C_o = 3.94$

$$\therefore Q = 3.94 \times 57.7 \times (3.75)^{3/2} = 1650 \text{ cfs}$$

From page 97, 98, 100, 106 and 107 of USACE Model Study of Upper Hudson & Mohawk River Basins use subbasin 22 of the Mohawk drain, Little Falls, N.Y. to Mouth

Subarea	Area (mi <sup>2</sup> ) $A_i$	SPF,	TA (Transposed Areas)
22	23	10655 cfs	7538 cfs

Since SPF is greater than TA the SDF will be based on  $2 \times \text{SPF} = \text{PMF}$

Drainage Area of Cork Center Reservoir =  $A_2 = 2.6 \text{ mi}^2$  {at 14, 1953  
app. to rain  
spillway crest

$$\left(\frac{A_1}{A_2}\right)^{0.75} = \left(\frac{F_1}{F_2}\right)$$

$$\left(\frac{23}{2.6}\right)^{0.75} = \left(\frac{10655}{F_2}\right) \quad \therefore F_2 = \frac{10655}{5.13} = 2077 \text{ cfs}$$

$$\text{PMF} = 2 \text{ SPF} = 2 \times 2077 = 4154 \text{ cfs}$$

BY J.K. DATE 8/7/78

JOSEPH S. WARD

CHKD. BY P.M. DATE 8/2/78

91 ROSELAND AVE. CALDWELL, N. J.

SHEET NO. 3 OF 13

SUBJECT Cove Center Storage Reservoir - Hydrology

JOB NO. A7805-11 F

Assume Max. pool elevation at top of dam = 1059' which is 3.75 ft above normal pool at spillway crest (El. 1055.25')

General slope of the shore line =  $1\frac{1}{2} H : 1 V$  (Field inspection)

Length of shore line = 1.78 miles (from USGS quad.)

Lake size at normal pool elevation = 41 acres (Oct. 14, 1963 approx.)

$$\begin{aligned} \text{Volume change from normal pool to maximum pool} &= \Delta V = \\ &= (1.5 \times 3.75) \times \frac{1}{2} \times 3.75 \times 1.78 \times 5280 \times \frac{1}{43560} + (3.75 \times 41) \\ &= 2.28 + 153.75 = 156 \text{ acre feet.} \end{aligned}$$

$$\begin{aligned} \therefore \text{storage capacity at max. pool} &= \text{storage capacity at normal pool} + \Delta V \\ &= 445 + 156 = 601 \text{ acre feet.} \end{aligned}$$

### Overtopping Potential

$$Q_1 = \text{Max. spillway capacity} = 1650 \text{ cfs}$$

$$Q_2 = \text{PMF} = 4154 \text{ cfs}$$

$p_{\text{PMF}}$  = % of PMF spillway will pass; % of SPF spillway will pass

$$\text{or } p_{\text{PMF}} = \frac{Q_1}{Q_2} = \frac{1650}{4154} = 40\% \quad p_{\text{SPF}} = \frac{1650}{2077} = 79\%$$

$$\begin{aligned} \therefore (1-p) &= \frac{\text{Required reservoir storage}}{\text{Vol. of inflow hydrograph}} = 0.60 \text{ (for PMF)} \\ &= 0.21 \text{ (for SPF)} \end{aligned}$$

BY J.K. DATE 8/7/78 JOSEPH S. WARD SHEET NO. 4 OF 13  
 CHKD. BY P.G.M. DATE 9/8/78 91 ROSELAND AVE. CALDWELL, N. J. JOB NO. A7805-11F  
 SUBJECT Cork Center Storage Reservoir - Hydrology

Solving for  $T_p$  for triangular hydrograph. Assume  $T_p$  is a function of the linear elements of equivalent areas

$$A_1 = 23 = \frac{\pi}{4} d_1^2$$

$$\text{or } d_1 = 5.41$$

$$A_2 = 2.6 = \frac{\pi}{4} d_2^2$$

$$\text{or } d_2 = 1.82$$

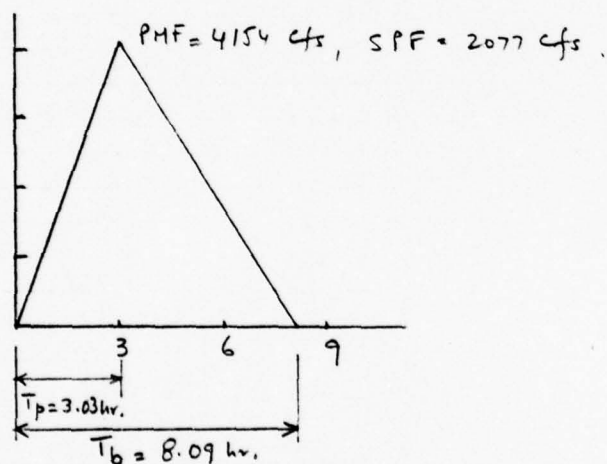
$T_{p1} = 9 \text{ hr.}$  → Upper Hudson and Mohawk river basins flood routing models.

$$T_{p2} = ?$$

$$T_{p2} = \frac{d_2}{d_1} T_{p1} = \frac{1.82}{5.41} \times 9 = 3.03 \text{ hrs.}$$

$$T_{b2} = 2.67 (T_{p2}) = 2.67 \times 3.03 = 8.09 \text{ hrs.} \quad (\text{Design of small dam page 69})$$

Q  
cfs



$$\text{Volume of inflow hydrograph} = V_i = \frac{\text{PMF} \times T_b}{2}$$

$$\text{For PMF} = \frac{1}{2} \times 4154 \times 8.09 \times \frac{3600}{43560} = 1389 \text{ acre ft.}$$

$$\text{For SPF} = 695 \text{ acre ft.}$$

BY J.K. DATE 8/7/78

JOSEPH S. WARD

SHEET NO. 5 OF 13

CHKD. BY PGM. DATE 8/8/78

91 ROSELAND AVE. CALDWELL, N. J.

JOB NO. A7805-11 F

SUBJECT Cork Center Storage Reservoir - Hydrology

$$\begin{aligned} \text{Required reservoir storage} &= 0.6 \times 1389 = 833 \text{ acre ft. for PMF} \\ &= 0.21 \times 695 = 146 \text{ acre ft. for SPF} \end{aligned}$$

But available incremental storage = 156 acre ft. (Sheet 3)

$$156 < 833 \text{ acre ft. (PMF)} \quad 156 > 146 \text{ (SPF)}$$

∴ CORK CENTER RESERVOIR WILL NOT BE ABLE TO CONTAIN THE PMF WITHOUT OVERTOPPING OF THE DAM BUT WILL BE ABLE TO CONTAIN THE SPF WITHOUT OVERTOPPING. However, more accurate analysis follows in the succeeding pages.

BY J.K. DATE 8/7/78

JOSEPH S. WARD

SHEET NO. 6 OF 13

CHKD. BY PGM DATE 8/9/78

91 ROSELAND AVE. CALDWELL, N. J.

JOB NO. H7825-11 F

SUBJECT Cork Center Storage Reservoir - Hydrology

Discharge Q (cfs) vs head over spillway crest.

p = 2.25

Q = C<sub>0</sub> L H<sub>0</sub><sup>3/2</sup>

H <sub>0</sub>	EL.	Q	cfs
0.5	1055.75	= 3.95 x 57.7 (0.5) <sup>3/2</sup>	= 81
1.0	1056.25	= 3.95 x 57.7 (1) <sup>3/2</sup>	= 228
1.5	1056.75	= 3.95 x 57.7 (1.5) <sup>3/2</sup>	= 419
2.0	1057.25	= 3.95 x 57.7 (2) <sup>3/2</sup>	= 645
2.5	1057.75	= 3.95 x 57.7 (2.5) <sup>3/2</sup>	= 901
3.0	1058.25	= 3.945 x 57.7 (3) <sup>3/2</sup>	= 1183
3.5	1058.75	= 3.94 x 57.7 (3.5) <sup>3/2</sup>	= 1489
3.75	1059.00	= 3.935 x 57.7 (3.75) <sup>3/2</sup>	= 1649 ← Max. pool
4.0	1059.25	= 3.93 x 57.7 (4) <sup>3/2</sup>	= 1814
4.5	1059.75	= 3.925 x 57.7 (4.5) <sup>3/2</sup>	= 2162
5.0	1060.25	= 3.92 x 57.7 (5) <sup>3/2</sup>	= 2529
5.5	1060.75	= 3.91 x 57.7 (5.5) <sup>3/2</sup>	= 2910
6.0	1061.25	= 3.905 x 57.7 (6) <sup>3/2</sup>	= 3311

BY J.K. DATE 8/7/78 JOSEPH S. WARD SHEET NO. 7 OF 13  
 CHKD. BY P.G. DATE 8/2/78 91 ROSELAND AVE. CALDWELL, N. J. JOB NO. A7905-11E  
 SUBJECT Cork Center Storage Reservoir - Hydrology

Flood storage vs head above crest or elevation

Lake Area at normal pool elevation = 41 acre } sheet  
 General slope of shore line =  $1\frac{1}{2}H:1V$

Head H	EL.	Vol. = $(H)(41) + \left[ \frac{(H)(1.5H)}{2} \times \text{Length of shore} \times \frac{5280}{43560} \right]$	
0.5'	1055.75	$= 0.5 \times 41 + 0.04$	= 21 acre ft.
1.0'	1056.25	$= 41 + 0.16$	= 41
1.5	1056.75	$= 61.5 + 0.36$	= 62
2.0	1057.25	$= 82 + 0.65$	= 83
2.5	1057.75	$= 102.5 + 1.0$	= 104
3.0	1058.25	$= 123 + 1.46$	= 124
3.5	1058.75	$= 143.5 + 1.98$	= 145
3.75	1059.00	$= 153.8 + 2.28$	= 156 ← Max. pool
4.0	1059.25	$= 164 + 2.59$	= 167
4.5	1059.75	$= 184.5 + 3.28$	= 188
5.0	1060.25	$= 205 + 4.0$	= 209
5.5	1060.75	$= 225.5 + 4.9$	= 230
6.0	1061.25	$= 246 + 5.8$	= 252



J.K.

JOSEPH S. WARD

BY..... DATE 8/7/78

91 ROSELAND AVE. CALDWELL, N. J.

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CHKD. BY PGM DATE 8/3/78

JOB NO. A7805-11 F

SUBJECT Cork Center storage Reservoir - Hydrology

EL. (ft.)	Q (cfs)	Q/2 (cfs)	Flood storage acre-ft.	S Flood storage cfs-hrs.	S/ΔT (0.5 hr)	SI = $\frac{Q}{2} + \frac{S}{\Delta T}$
Normal pool 1055.25	0	0	0	0	0	0
1055.75	81	40	21	254	508	548
1056.25	228	114	41	496	992	1106
1056.75	419	210	62	750	1500	1710
1057.25	645	323	83	1004	2008	2331
1057.75	901	451	104	1258	2516	2967
1058.25	1183	592	124	1500	3000	3592
1058.75	1489	745	145	1755	3510	4255
1059.00	1649	825	156	1888	3776	4601
1059.25	1814	907	167	2021	4042	4949
1059.75	2162	1081	188	2275	4550	5631
1060.25	2529	1265	209	2529	5058	6323
1060.75	2910	1455	230	2783	5566	7021
1061.25	3311	1655	252	3049	6098	7753

BY J.K. DATE 8/7/78

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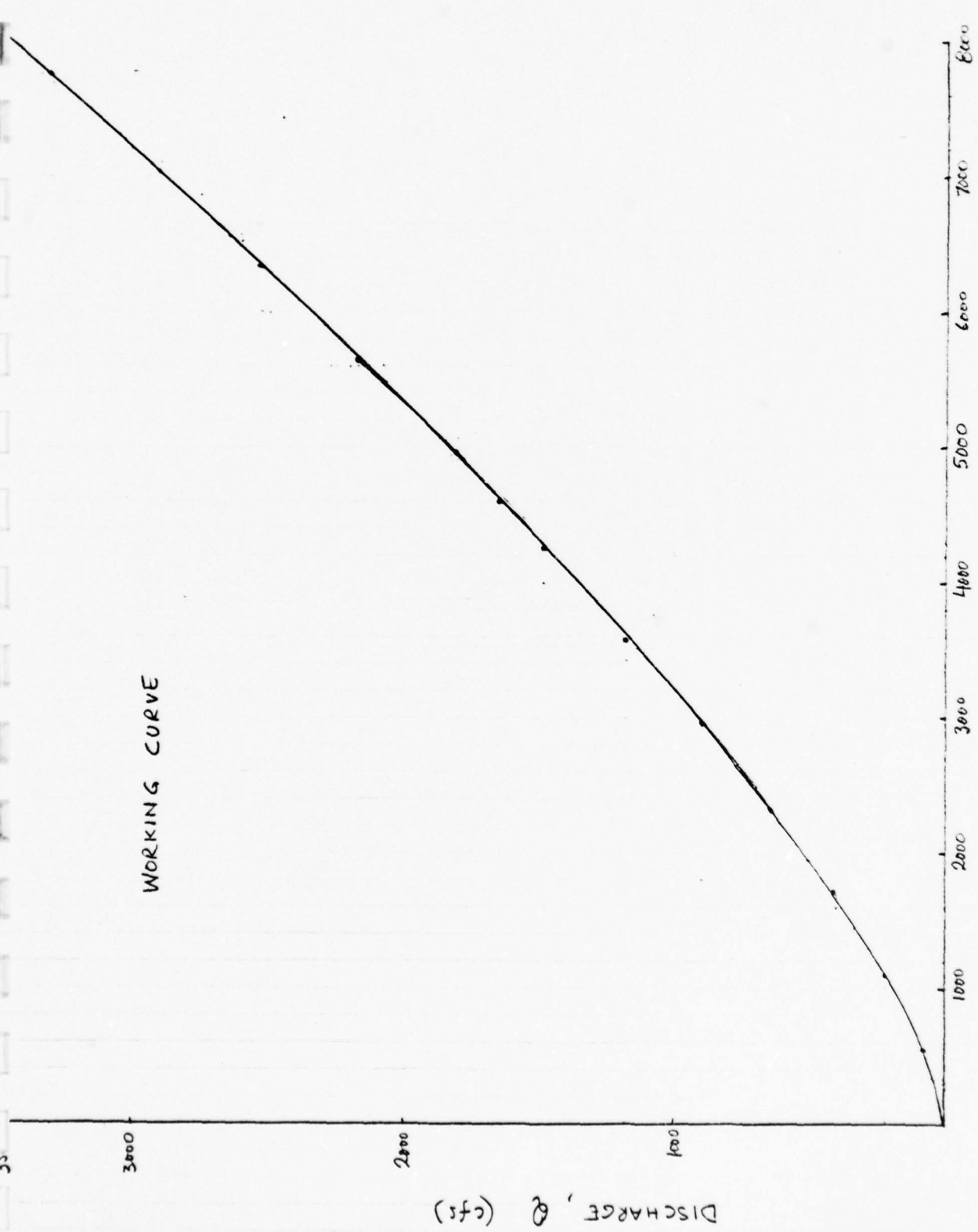
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CHKD. BY PGM DATE 8/2/78 91 ROSELAND AVE. CALDWELL, N. J.

JOB NO. A7825-11F

SUBJECT Cork Center Storage Reservoir - Hydrology

WORKING CURVE



$SI = \frac{Q}{2} + \frac{S}{\Delta T}$

35

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SHEET NO. 10 OF 13

CHKD. BY PSM DATE 8/2/78

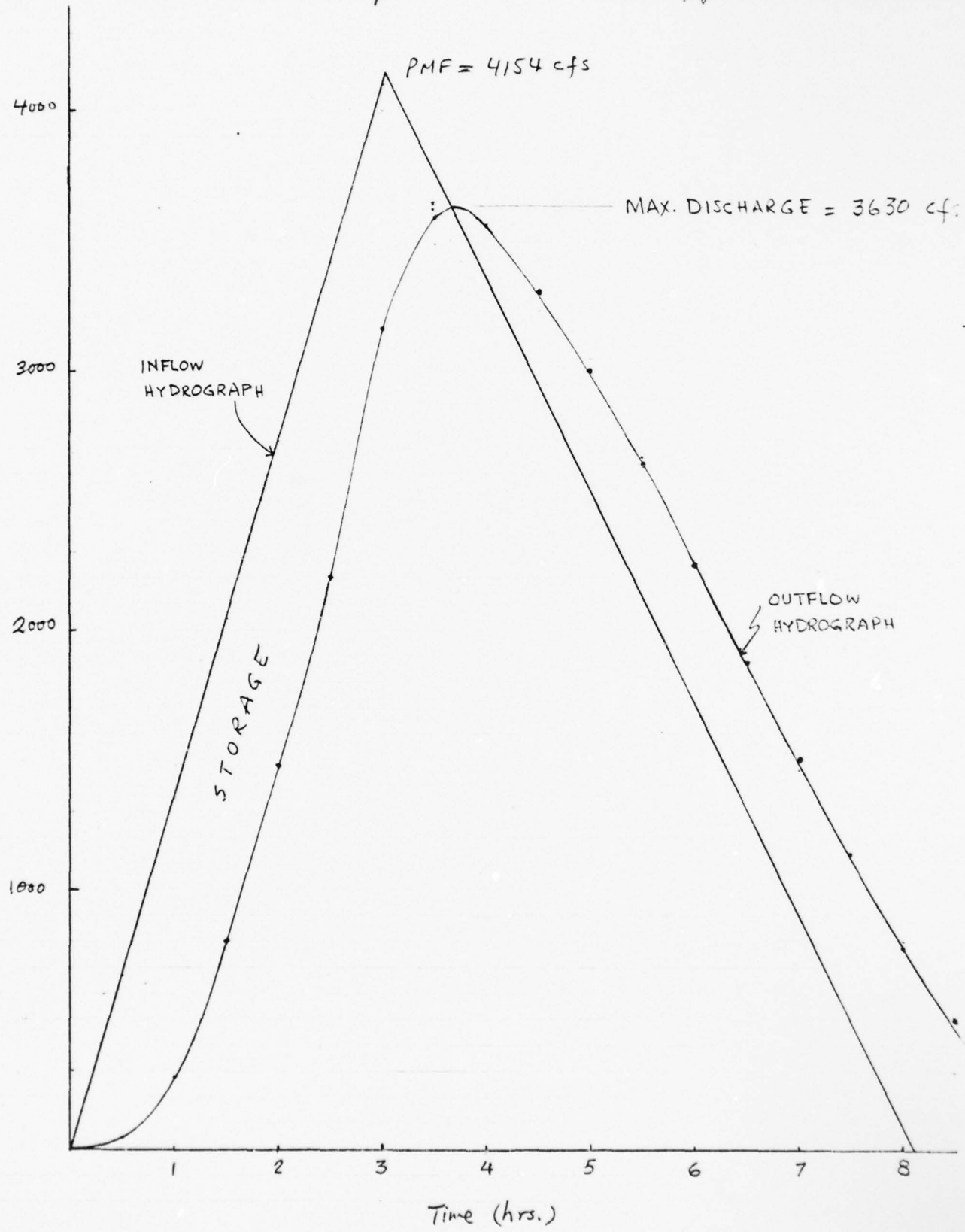
91 ROSELAND AVE. CALDWELL, N. J.

JOB NO. A7805-11 F

SUBJECT Cork Center Storage Reservoir - Hydrology

Time (Hrs)	I (cfs)	$\bar{I}$	$(SI)_N$	Q	$(SI)_{N+1} = (SI)_N - Q_N + \bar{I}_{(N+1)}$
0	0	0	0	0	$0 - 0 + 335 = 335$
0.5	670	335	335	40	$335 - 40 + 1005 = 1300$
1.0	1340	1005	1300	280	$1300 - 280 + 1685 = 2705$
1.5	2030	1685	2705	800	$2705 - 800 + 2375 = 4280$
2.0	2720	2375	4280	1470	$4280 - 1470 + 3065 = 5875$
2.5	3410	3065	5875	2200	$5875 - 2200 + 3755 = 7430$
3.0	4100	3755	7430	3160	$7430 - 3160 + 3930 = 8200$
3.5	3760	3930	8200	3590	$8200 - 3590 + 3555 = 8165$
4.0	3350	3555	8165	3560	$8165 - 3560 + 3145 = 7750$
4.5	2940	3145	7750	3300	$7750 - 3300 + 2740 = 7260$
5.0	2540	2740	7190	3000	$7260 - 3000 + 2335 = 6595$
5.5	2130	2335	6525	2640	$6525 - 2640 + 1925 = 5810$
6.0	1720	1925	5810	2260	$5810 - 2260 + 1520 = 5070$
6.5	1320	1520	5070	1870	$5070 - 1870 + 1110 = 4310$
7.0	900	1110	4310	1500	$4310 - 1500 + 700 = 3510$
7.5	500	700	3510	1140	$3510 - 1140 + 295 = 2665$
8.0	90	295	2665	770	$2665 - 770 + 45 = 1940$
8.5	0	45	1940	500	$1940 - 500 + 0 = 1440$

BY J.K. DATE 8/7/78 JOSEPH S. WARD SHEET NO. 11 OF 13  
 CHKD. BY Pkm DATE 7/5/78 91 ROSELAND AVE. CALDWELL, N. J. JOB NO. A7805-11F  
 SUBJECT Cork Center Storage Reservoir - Hydrology



BY J.K. DATE 8/8/78

JOSEPH S. WARD

SHEET NO. 12 OF 13

CHKD. BY P.W. DATE 8/2/78

91 ROSELAND AVE. CALDWELL, N. J.

JOB NO. A7805-11F

SUBJECT Cork Center Storage Reservoir - Hydrology

From outflow hydrograph, discharge at maximum pool = 3630 cfs

Determine H by trial and error

$$Q = C_o L H_o^{3/2}$$

$$\text{For } H_o = 6.4 \text{ ft}$$

$$Q = 3.90 \times 57.7 (6.4)^{3/2} = 3643 \text{ cfs, close enough}$$

∴ The PMF will raise the pool to  $1055.25 + 6.4 = 1061.65'$

El. 1061.65' is 2.65' above the top of the dam.

% of PMF THAT CAN BE PASSED AT PEAK OUTFLOW IS:

MAX OUTFLOW w/ POOL ELEV. AT DAM CRUST	=	$\frac{1650 \text{ cfs}}{3630 \text{ cfs}} \times 100 = 45\%$
MAX. DISCHARGE FOR PMF INFLOW	=	

Discharge through supply and mud pipes

- upstream head on mud pipe = 40 feet
- " " " supply line = 25 feet (minimum)

From original drawings the difference in upstream and downstream water levels =  $1059 - 1002 = 57$  feet.

Since the actual inlet level at the outfall and the length of pipes are not known, <sup>exactly</sup> the total head loss due to entrance, through gate valves, bends, and friction in pipes may be conservatively taken as 17 feet and discharge will be computed for 40 feet head

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SUBJECT Cork Center Storage Reservoir - Hydrology

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JOB NO. A7805-11F

$$Q = Av = A\sqrt{2gh} = \frac{\pi}{4}(2)^2\sqrt{2 \times 32.2 \times 40}$$
$$= 159 \text{ cfs } \checkmark$$

$\therefore$  discharge through 2 pipes = 320 cfs.  $\checkmark$

If the flow through these two pipes is taken into consideration, the % of PMF that can be passed at peak outflow =  $\frac{1650 + 320}{3630} = 54\% \checkmark$

BY J.K. DATE 8/3/78

JOSEPH S. WARD

SHEET NO. 1 OF 10

CHKD. BY H.R.D. DATE 8/4/78

91 ROSELAND AVE. CALDWELL, N. J.

JOB NO. A7805-11F

SUBJECT Cork Center storage Reservoir Spillway

Stability

The original calculations in the file are quite elaborate and deal in depth with items of <sup>even</sup> minor significance. A rough check of the original analysis revealed it to be in conformance with standard engineering practice except in one case (page 28) where active earth pressure coefficient was used for calculating passive resistance.

Present analyses are performed on the section of the section of the dam shown on the next sheet.

- (1) Stability of the spillway against overturning about its toe  
Resisting forces and moments.

$$\begin{aligned} \text{Weight of the concrete section} &= (2.5 \times 12)140 + \left(\frac{1}{2} \times 9 \times 12\right)140 + (8 \times 12)140 \\ &= 4200 + 7560 + 1680 = 13440 \text{ lb.} \end{aligned}$$

$$\begin{aligned} \text{Moment about pt. P} &= (4200 \times 10.25) + \left(7560 \times 9 \times \frac{2}{3}\right) + (1680 \times 9.5) \\ &= 43050 + 45360 + 15960 = 104370 \text{ ft.-lb.} \end{aligned}$$

$$\begin{aligned} \text{Wt. of triangular section of water over the crest} \\ &= \frac{1}{2} \times 4 \times 2.5 \times 62.4 = 312 \text{ lb.} \end{aligned}$$

$$\text{Moment about toe} = 312 \left(\frac{2}{3} \times 2.5 + 9\right) = 3328 \text{ ft.-lb.}$$

Neglect the active forces on the ups and D/s faces of the cut-off wall because their net result is small.

$$\text{Therefore total resisting moment} = 104370 + 3328 = 107700 \text{ ft.-lb.}$$

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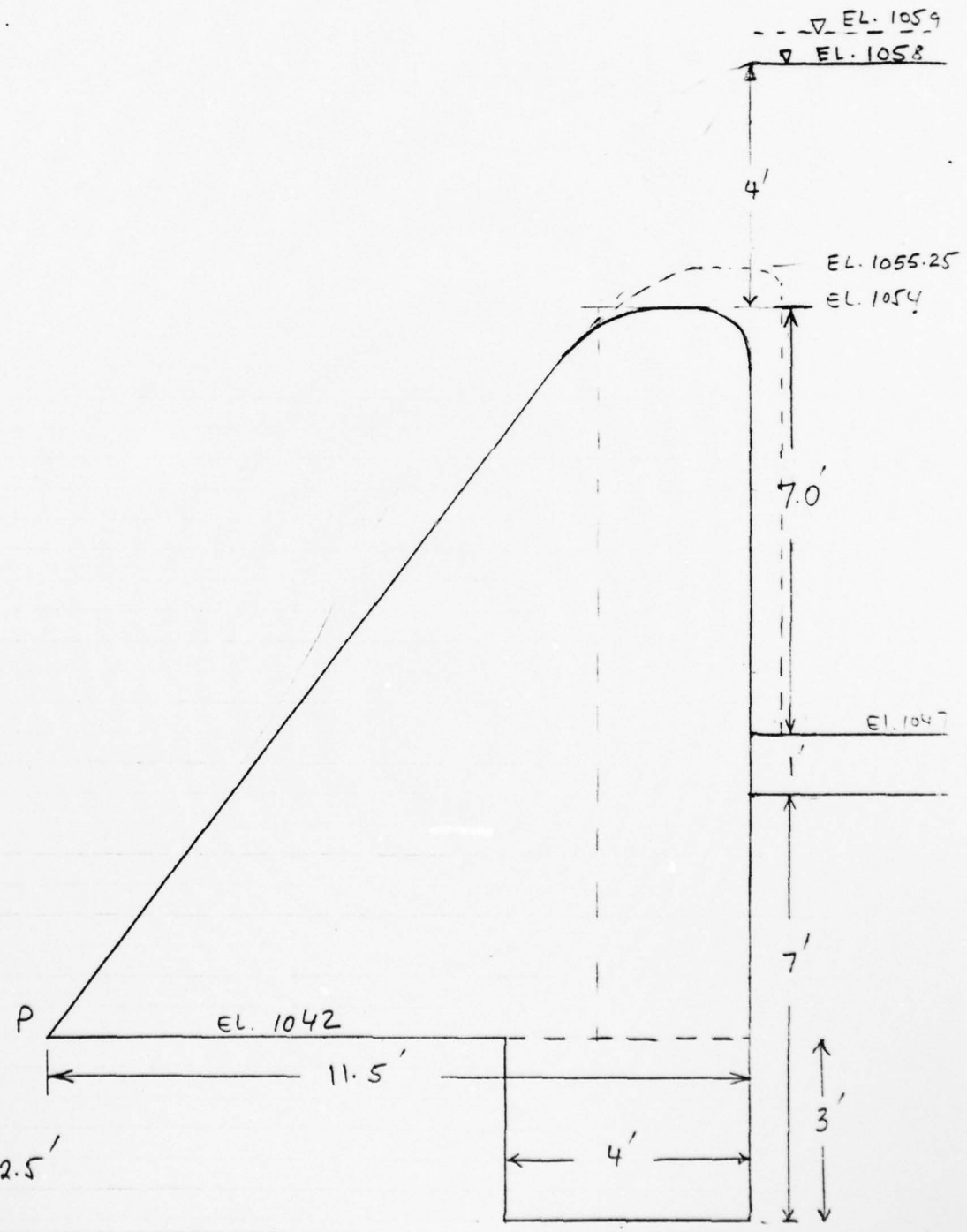
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SHEET NO. 2 OF 10

CHKD. BY HDD. DATE 8/14/78 91 ROSELAND AVE. CALDWELL, N. J.

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SUBJECT Cork Center Storage Reservoir Spillway Stability



Scale 1" = 2.5'



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CHKD. BY ARD. DATE 8/4/78

91 ROSELAND AVE. CALDWELL, N. J.

JOB NO. A7805-11F

SUBJECT Cork Center Storage Reservoir spillway stability

Overturning Forces and Moments

Horizontal Water Thrust above El. 1047

$$= \frac{62.4}{2} [(11)^2 - (4)^2] = 3276 \text{ lb.}$$

Moment about toe =  $3276 \times 8 = 26208 \text{ ft-lb.}$ 

Assuming head of water 7' and 6' at El. 1046' and 1042' respective

This adds to the overturning moment.

$$\text{Horizontal Thrust} = \frac{62.4}{2} \left[ \left( \frac{7+6}{2} \right) \times 4 \right] = 811 \text{ lb.}$$

$$\text{Moment} = 811 \times 2 = 1622 \text{ ft-lb.}$$

Neglect active earth pressure between El. 1046 + 1042

Uplift pressures

The upstream wall below El. 1047 (Drawing #4, Dim 427M), the ups slab and cut-off wall should be effective in reducing the uplift pressure. Assuming that uplift varies from a head of 5.5 ft. at the ups end of the dam base to zero at the downstream end.

$$\text{Hence total uplift pressure} = \frac{1}{2} \times 5.5 \times 11.5 \times 62.4 = 1973$$

$$\text{Moment about toe} = 1973 \times 11.5 \times \frac{2}{3} = 15126 \text{ ft-lb.}$$

$$\text{Net vertical load} = 13440 + 312 - 1973 = 11779 \text{ lb.}$$

If  $\bar{x}$  is the distance of the resultant from the toe,

then

$$11779 \bar{x} = 107700 - 26208 - 1622 - 15126$$

$$\text{or } \bar{x} = \frac{64744}{11779} = 5.5 \text{ ft.}$$

$$\frac{1}{3} \text{ rd base width} = \frac{11.5}{3} = 3.8 \text{ ft.}$$

BY J.K. DATE 8/3/78  
CHKD. BY A.D.D. DATE 8/4/78  
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91 ROSELAND AVE. CALDWELL, N. J.

SHEET NO. 4 OF 10

JOB NO. A7805-11F

Coxk Center Storage Reservoir Spillway Stability

The resultant falls within the middle third of the base width of the spillway and therefore safe against overturning.

Stability with respect to sliding:

Passive resistance to the cut off wall not considering the weight of the dam =  $\frac{(100-62.4)}{2} \cdot (3)^2 \tan^2(45 + \frac{33}{2})$   
= 574 lb.

The same amount of passive resistance will be developed by the keying effect of the toe. Therefore, total passive resistance = 1148 lb.

Assume  $\mu = 0.3$

Total resistance =  $(0.3 \times 11779) + 1148 = 4682$

Driving forces =  $3276 + 811 = 4087$

F.S. against sliding =  $\frac{4682}{4087} = 1.15$

The calculations in the original file indicated the spillway to be unstable with respect to sliding. This was because of two main differences in the two analyses. (1) The original analysis assumed an open joint between the horizontal slab and the vertical upstream face of the spillway and, therefore, a higher intensity of hydrostatic pressure was taken

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JOSEPH S. WARD

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91 ROSELAND AVE. CALDWELL, N. J.

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SUBJECT

Cork Center Storage Reservoir Spillway Stability

along the vertical upstream side of the section and consequently at the base of the dam also.

(2) No passive resistance to cut off wall and at the toe was considered.

(2) Checking the stability of the spillway with water at the top of the dam i.e. at El. 1059

Resisting forces and moments will practically remain the same.

Horizontal Water pressure above El. 1047

$$= \frac{62.4}{2} [(12)^2 - (5)^2] = 3713 \text{ lb.}$$

$$\text{Moment about toe} = 3713 \times 8 = 29704 \text{ ft-lb.}$$

Horizontal pressure and moment below the upstream slab will hardly be affected and are, therefore, taken the same i.e. 811 lb. and 1622 ft-lb. respectively.

Uplift pressure & moment will also remain unchanged. i.e. 1973 lb. and 15126 ft-lb.

$$\text{Hence } 11779 \bar{x} = 107700 - 29704 - 1622 - 15126$$

$$\text{or } \bar{x} = \frac{61248}{11779} = 5.2 \text{ ft.} > 3.8 \text{ ft}$$

The spillway is still safe against overturning

$$\text{F.S. against sliding} = \frac{4682}{3713 + 811} = 1.03$$

which is just safe against sliding.

BY J.K. DATE 8/16/78

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SHEET NO. 6 OF 10

CHKD. BY P.M. DATE 8/28/78

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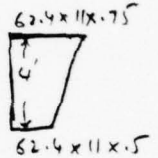
SUBJECT Cork Center storage Reservoir Spillway Stability

(3) Revising stability calculations for the remodelled section of the dam.

Water at El. 1059

$$\text{Horizontal water pressure} = \frac{62.4}{2} [(11)^2 - (3.75)^2] = 3336 \text{ lb.}$$

$$\begin{aligned} \text{Horizontal water pressure on the vertical ups} \\ \text{face of the wall below the ups slab} &= \left( \frac{62.4 \times 11 \times .75 + 62.4 \times 11 \times .5}{2} \right) \times 4 \\ &= 1716 \text{ lb.} \end{aligned}$$



$$\therefore \text{Total driving force} = 3336 + 1716 = 5052 \text{ lb.}$$

Passive resistance from the two keys = 1148 lb. (page 4)

Passive resistance taking into account the weight of the dam =  $\frac{13440}{11.5} \times 3 \times 3 = 10518$  lb/ft. (discounting the nominal weight of water over the crest)

$$\begin{aligned} \text{Wt. of water on the upstream slab less 75\% uplift} \\ = [11 - (.75 \times 11)] \times 8.83 \times 62.4 = 1515 \text{ lb.} \end{aligned}$$

$$\begin{aligned} \text{Wt. of concrete on the upstream slab} &= 140 [(12.25) + (2 \times 8.83)] \\ &= 3767 \text{ lb.} \end{aligned}$$

Wt. of concrete in the gravity section = 13440 lb (Page 1; reflect the small amount of additional concrete on the crest)

Assuming 50% uplift on the base of the dam = 1973 lb (Page 1)

$$\therefore \text{Net downward load} = 1515 + 3767 + 13440 - 1973 = 16749 \text{ lb.}$$

$$\therefore F.S. \text{ against sliding} = \frac{(16749 \times .3) + 1148 + 10518}{5052} = 3.3$$

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CHKD. BY Am DATE 8/22/78

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91 ROSELAND AVE. CALDWELL, N. J.

SHEET NO. 7 OF 10

JOB NO. A7805-11F

Cork Center storage Reservoir spillway stability  
for water at top of dam

Stability against overturning, will not be checked because the original section was very safe (page 3+4) and the new section has got even more weight added to it.

- (4) Checking stability of the remodelled section of the dam with water at El. 1055.25 (dam crest) and ice thrust of 4k per linear foot at El. 1054.25

Stability against overturning:

Resisting moment from page 1 = 104370 ft-lb.

Moment due to weight of the additional concrete in

the remodelled section =  $(1 \times 8.25) \times 140 \times 12 = 13860$  ft-lb.

Total resisting moment =  $104370 + 13860 = 118230$  ft-lb.

Overturning loads & moments:

Horizontal water pressure =  $\frac{62.4}{2} (7.25)^2 = 1640$  lb.

moment =  $1640 \times 8.417 = 13803$  ft-lb.

Horizontal pressure on the vertical upstream face of the spillway below elevation 1046 =  $\left( \frac{62.4 \times 7.25 \times .75 + 62.4 \times 7.25 \times .5}{2} \right) 4$

= 1131 lb.

Its moment =  $1131 \times 2 = 2262$  ft-lb.

Assuming 50% uplift =  $\frac{1}{2} \times (7.5 \times .5) \times 11.5 \times 62.4 = 1346$  lb.

Its moment =  $1346 \times 11.5 \times \frac{2}{3} = 10319$  ft-lb.

BY J.K. DATE 8/16/78

JOSEPH S. WARD

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JOB NO. A 7805-11 F

SUBJECT Cork Center Storage Reservoir Spillway Stability

$$\text{Moment due to ice Thrust} = 4 \times 12.25 = 49000 \text{ ft. lb.}$$

$$\begin{aligned} \text{Total overturning moments} &= 13803 + 2262 + 10319 + 49000 \\ &= \underline{75,384} \text{ ft. lb.} \end{aligned}$$

$$\begin{aligned} \text{Net downward vertical load} &= 13440 + (8.25 \times 140) - 1346 \\ &= 13249 \text{ lb.} \end{aligned}$$

$$\begin{aligned} \therefore \text{point of application of the resultant } \bar{x} &= \frac{118,230 - 75,384}{13249} \\ &= 3.2 \end{aligned}$$

$$\frac{1}{3} \text{rd base width} = \frac{11.5}{3} = 3.8' > 3.2' \quad \therefore \text{unsafe}$$

$$\bar{x} = \frac{118230 - 75384 + 12000}{13249} = 4.1' \quad \therefore \text{OK}$$

However, the stabilising effect of the weight of water over the ups slab has not been considered in the analysis, which will bring the resultant in the middle third of the base. As a precaution during winter, water level should be kept at least 3 ft. below crest level to reduce the overturning moment due to ice. Stability against sliding;

$$\text{Driving forces} = 1640 + 1131 + 4000 = 6771 \text{ lb.}$$

$$\text{Passive resistance from page 6} = 1148 + 10518 = 11666$$

$$\begin{aligned} \text{Weight of water on the upstream slab less 75\% uplift} \\ &= [7.25 - (.75 \times 7.25)] \times 8.83 \times 62.4 = 1000 \text{ lb.} \end{aligned}$$

$$\text{Wt. of concrete on the upstream slab} = 3767 \text{ lb (page 6)}$$

$$\therefore \text{Net vertical downward load} = 1000 + 3767 + 13440 - 1346 =$$

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SHEET NO. 9 OF 10

CHKD. BY Am DATE 8/28/78

91 ROSELAND AVE. CALDWELL, N. J.

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SUBJECT

Cork Center Storage Reservoir Spillway Stability

$$= 16861 \text{ lb.}$$

$$\therefore F.S. \text{ against sliding} = \frac{(16861 \times 0.3) + 11666}{6771} = 2.5$$

BY J.K. DATE 8/17/78 JOSEPH S. WARD  
 CHKD. BY PCM DATE 8/23/78 91 ROSELAND AVE. CALDWELL, N. J. SHEET NO. 10 OF 10  
 SUBJECT Cork Center Storage Reservoir spillway stability JOB NO. A7805-11F

SUMMARY OF STABILITY ANALYSIS

STRUCTURE ANALYZED	ASSUMPTIONS	SAFETY AGAINST OVERTURNING	F.S. (SLIDING)
1) Original shape before 1963	(a) No passive resistance to U/s key from the weight of the dam (b) No sliding resistance from U/s slab. (c) 50% uplift on the base of the dam. (d) U/s water level at El. 1058	Resultant almost in the middle of the base. Therefore, OK.	1.15
2) Original shape before 1963	(a), (b) & c, same as above (d) U/s water level at El. 1059	Resultant in the middle third. Therefore OK.	1.03
3) Remodelled section of the spillway in its existing shape	(a) Passive resistance to U/s key from the weight of the dam considered. (b) sliding resistance from U/s slab accounted for (c) 15% uplift under the U/s slab and 50% uplift on the base of the dam (d) U/s water level at El. 1059	Did not analyze because it will not be much different from the first two conditions. Therefore OK.	3.3
4) Remodelled section of the spillway ie in its present shape	(a), (b) and (c) same as in # 3 above. (d) U/s water level at El. 1055.25 (e) ice thrust of 4 <sup>k</sup> /ft. of dam at El. 1054.25	Resultant outside the middle third. Unsafe. During winter, keep pool level at least 3 ft. below spillway crest.	2.5



APPENDIX D  
PHOTOGRAPHS



FIGURE 1      CREST OF DAM LOOKING LEFT

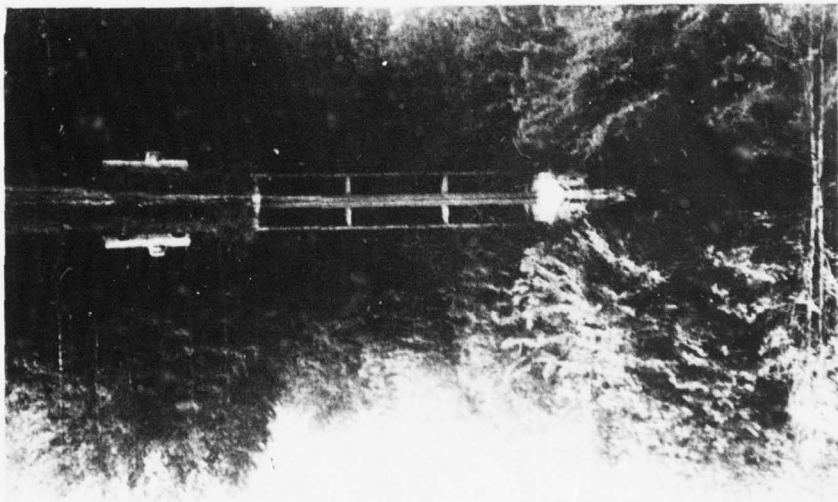


FIGURE 2      SPILLWAY OVERVIEW

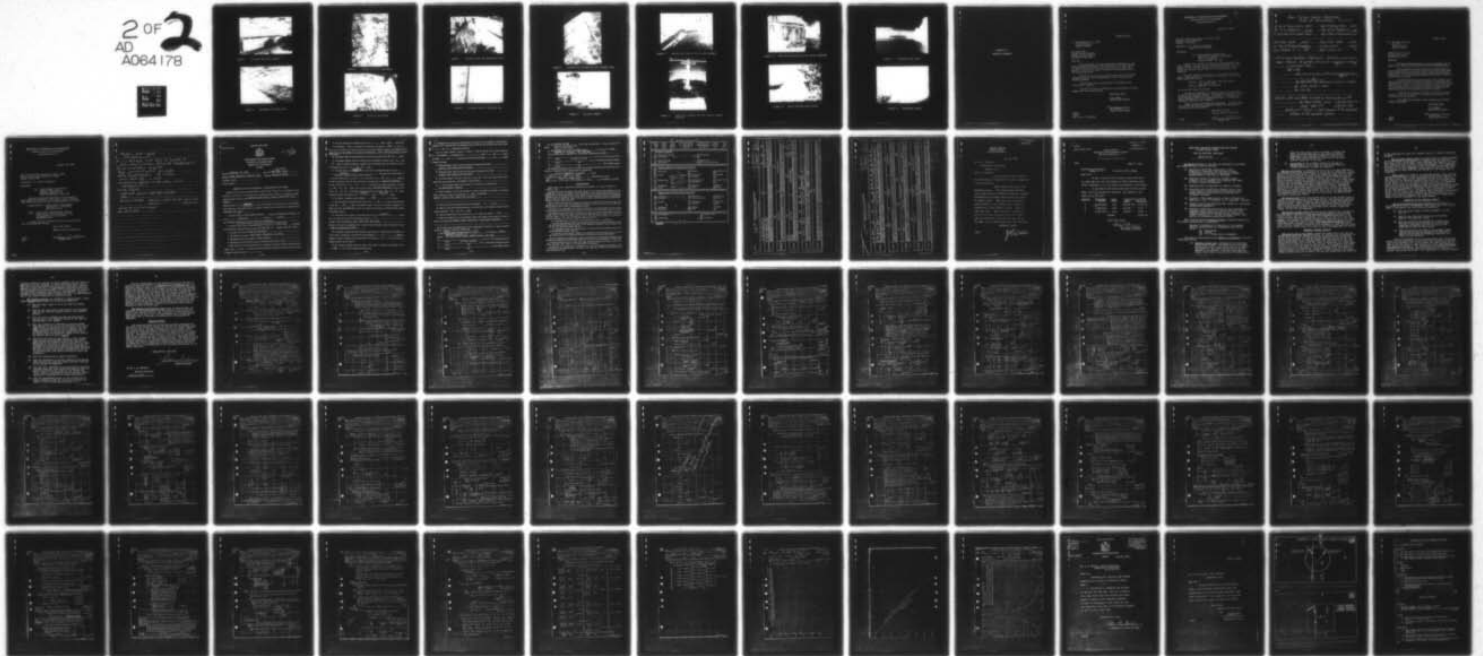
AD-A064 178

NEW YORK STATE DEPT OF ENVIRONMENTAL CONSERVATION ALBANY F/G 13/2  
NATIONAL DAM SAFETY PROGRAM. CORK CENTER STORAGE RESERVOIR DAM --ETC(U)  
SEP 78 G S SALZMAN DACW51-78-C-0035

UNCLASSIFIED

NL

2 OF 2  
AD A064178



END  
DATE  
FILMED  
3-79  
DDC



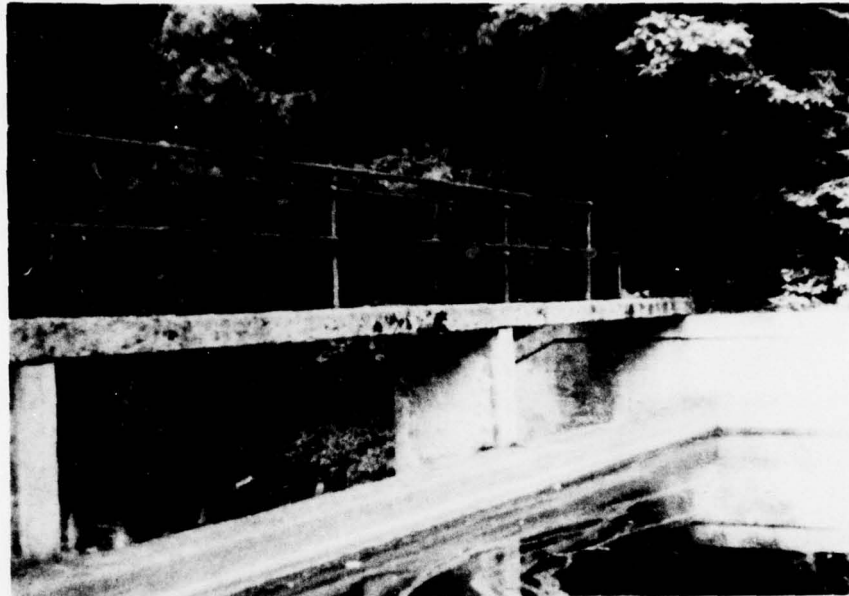


FIGURE 3      SPILLWAY AND RIGHT WINGWALL

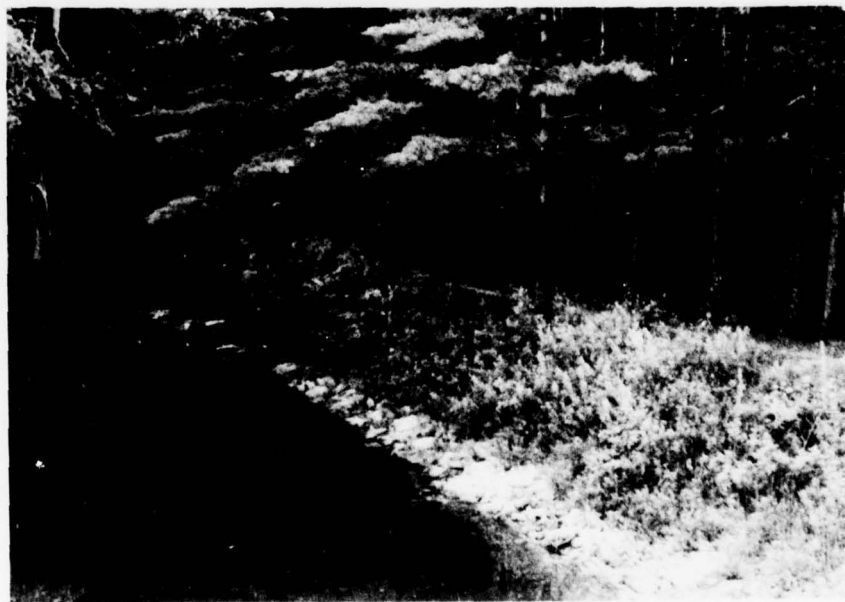


FIGURE 4      EMBANKMENT UPSTREAM SLOPE



FIGURE 5      SEEPAGE AT TOE OF EMBANKMENT



FIGURE 6      DECAY OF VEGETATION

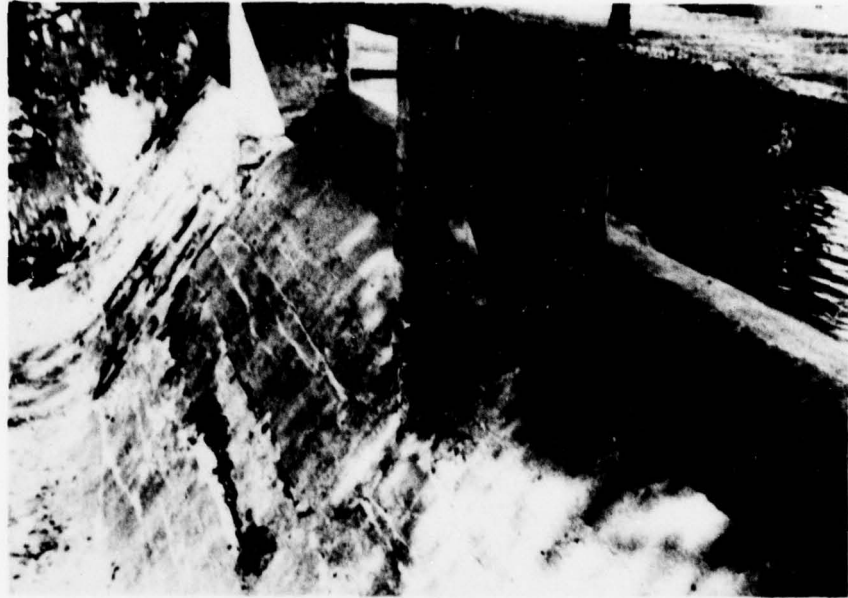


FIGURE 7 SPILLWAY PIERS AND DOWNSTREAM SLOPE

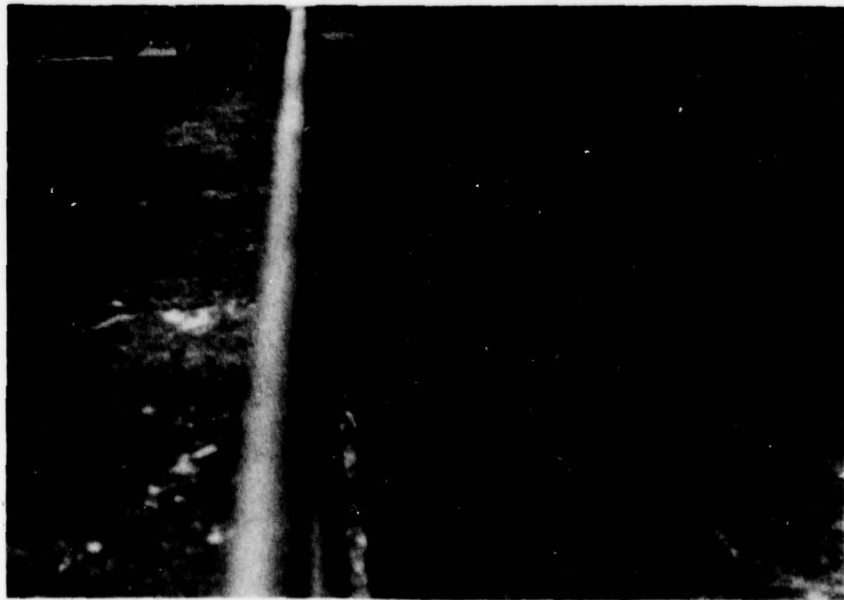


FIGURE 8 SPILLWAY AND LEFT ABUTMENT WALL

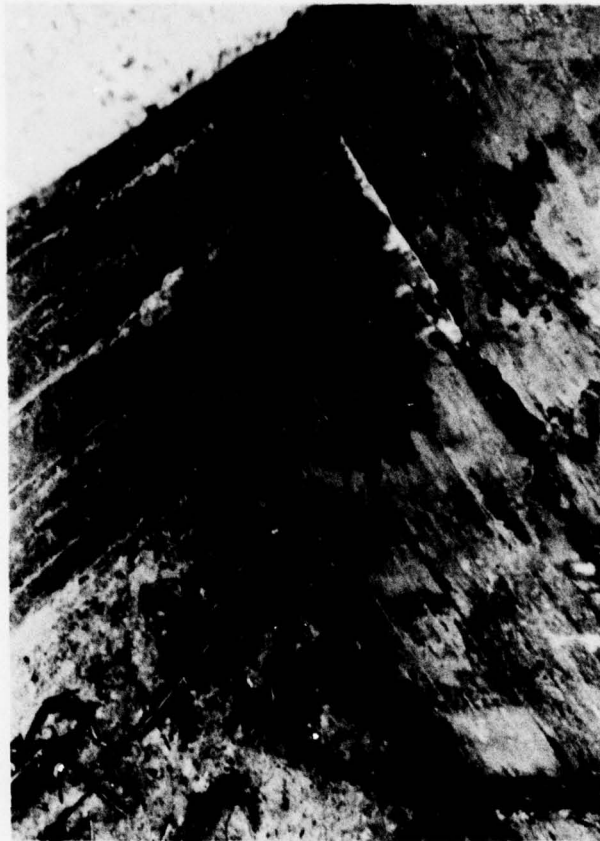


FIGURE 9      CLOSE-UP OF SPILLWAY AND LEFT ABUTMENT WALL



FIGURE 10      SPILLWAY CHANNEL





FIGURE 11      CRACKED LAST SLAB OF THE SPILLWAY CHANNEL

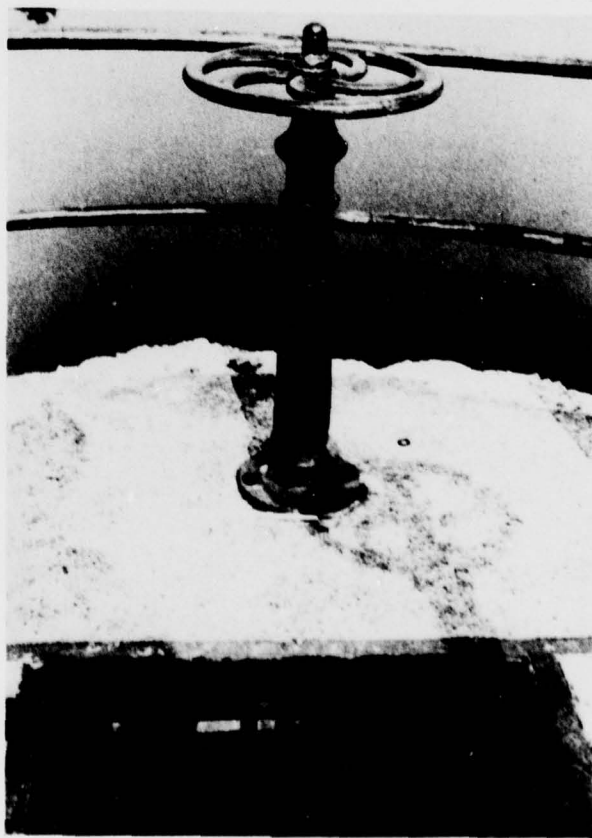


FIGURE 12      INTAKE BOX ENTRANCE AND GATE STEM AT INTAKE STRUCTURE

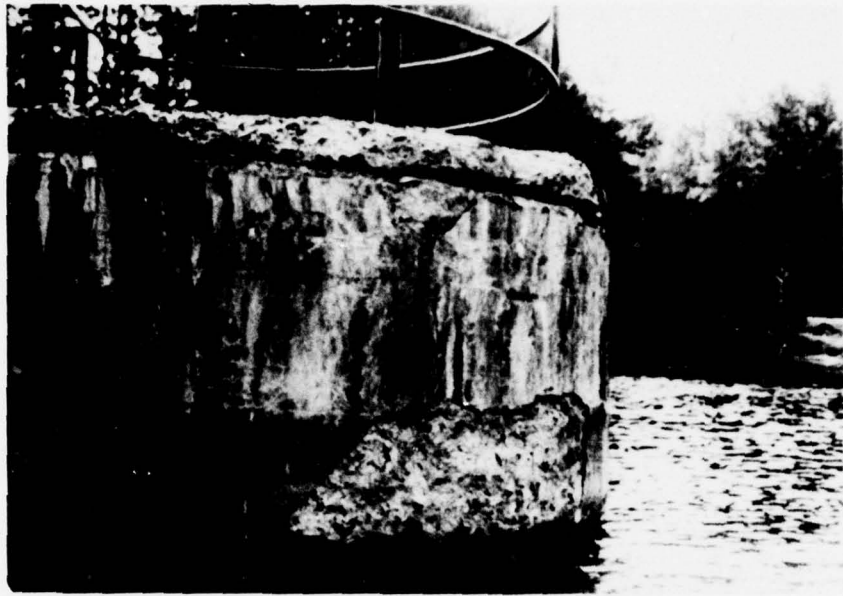


FIGURE 13 SPALLING AND SCALING OF THE INTAKE STRUCTURE



FIGURE 14 SUPPLY AND MUD PIPES OUTFALL



FIGURE 15      RESERVOIR SIDE SLOPES



FIGURE 16      DOWNSTREAM CHANNEL

APPENDIX E  
RELATED DOCUMENTS

October 17, 1963

Re: Reconstruction of Dam #127  
Town of Johnstown  
County of Fulton

Mr. Virgil Eble  
Morrill Wroonan Engineers  
Consultant Civil Engineers  
Cloversville, New York

Dear Sir:

The application, plans and specifications filed by you with this Department in accordance with the provisions of Section 948 of the Conservation Law for the owner City of Johnstown, City Hall, Johnstown, New York, for raising the crest of the existing spillway are satisfactory to us.

The details of the proposed work are approved to the extent of the authority of the Superintendent of Public Works under the aforesaid statute.

This dam bears our new designation of #1726-3191 of the Mohawk River Watershed.

One set of formally approved plans and specifications is being returned for the owner's records.

Very truly yours,

M. A. Faboo  
Deputy Chief Engineer

By: \_\_\_\_\_  
A. Dickinson  
Assoc. Civil Engineer

JEP:ifs  
Encl.  
CC: Mr. F. W. Montanari

MORRELL VROOMAN ENGINEERS  
CONSULTING CIVIL ENGINEERS  
GLOVERSVILLE, N. Y.

April 18, 1963

New York State Department of Public Works  
Division of Construction  
Albany 1, New York

Attention: Mr. Albert Dickinson,  
Assoc. Civil Engineer

Gentlemen:

Re: Water System Improvements  
Dam #427 - Proposed Flashboards  
City of Johnstown  
Fulton County, New York

We have your letter of April 15, 1963 concerning the above project. We are having difficulty checking the discharge of the spillway as noted in paragraph 3 of your letter and we will appreciate it if we can resolve this question.

In our interview with your Mr. John Peck on January 4, 1960 regarding the raising of this dam, the requirements established were:

Run-off - 250 to 300 cubic feet per second  
per square mile  
Minimum freeboard - 12 inches.

We agree with these standards.

We used a watershed of 3 square miles and a run-off of 300 cubic feet per second. The discharge capacity of the spillway was computed by Bazin's formula. The result is quite different from the capacity you note. We would appreciate receiving your comments.

Thank you for the plans you forwarded. It would seem well if we can have the partially filled out application in order to avoid confusion.

Very truly yours,  
MORRELL VROOMAN ENGINEERS

By Virgil Ehle  
Virgil Ehle

VE:ef

# CORK CENTER CREEK RESERVOIR

CITY OF JOHNSTOWN #1732-3111

Elev. top of Embankment = 1059.0'      Elev. of Spillway Crest = 1054.0  
 Less 1'-0" of Freeboard = - 1.0'      Add 1'-6" for Flashboard = + 1.5  
 Assumed Elev. of Max. Water = 1058.0      Elev. of Top of Flashboards = 1055.0

Elev. of Max. Water = 1058.0      Elev. of Max. Water = 1058.0  
 Elev. Top of Flashboards = 1055.5      Elev. base of Weir = 1047.0  
 Depth of head = "H" = 2.5'      Depth of Water = "d" = 11.0

Refer to King's "Handbook of Hydraulics" 1954 edition pages 4-4 & 4.

Bazin Formula:  $Q = \frac{2}{3} \sqrt{2g} L H^{3/2} (0.6075 + \frac{0.01476}{H}) (1 + 0.55 \frac{H^2}{d^2})$   
 $L = 60'-0" - 2'-0" = 58'-0"$

$$\sqrt{2g} = 8.02$$

$$\therefore Q = 0.667 \times 8.02 \times 58.0 \times 2.5^{3/2} (0.6075 + \frac{0.01476}{2.5}) [1 + 0.55 (\frac{2}{11})^2]$$

$$Q = \frac{1225}{0.667 \times 8.02 \times 58.0 \times 3.953} (0.6075 + 0.0059) (1 + 0.0284)$$

$$Q = 1225 \times 0.6134 \times 1.0284$$

$$Q = 770 \text{ cfs}$$

Formula used was: Robert E. Horton's Formula }  $Q = CLH^{3/2}$   
 for Broad Crested Weirs }  $Q = 3.5 \times 58.0 \times (2.5)^{3/2}$   
 King's page 5-5 }  $Q = 804 \text{ cfs.}$

Coefficient "C" of 3.5 is used because of the smoother surfaces of the concrete sections.

April 15, 1963

RE: Proposed Flashboards  
Dam #427  
Town of Johnstown  
County of Fulton

Morrell Vroman, Engineers  
21 North Main Street  
Gloversville, New York

Gentlemen:

The type of flashboards shown on the plan submitted with the application in reference to the above dam is not deemed satisfactory.

The large volume of water that would be suddenly released by automatically controlled flashboards added to flood stage runoff waters could possibly cause considerable damage to installations downstream from the dam. Due to this potential hazard approval cannot be given for construction of this type.

Our investigation shows that the crest of the spillway can be raised 18 inches and still be capable of discharging about 2-1/2 times the anticipated runoff. On this basis, a permanent type of construction; such as raising the crest with reinforced concrete, securely anchoring permanent flashboards, or installation of roller or radial gates to continuously control the water level; would be acceptable.

In the event it is decided to obtain the additional storage capacity as outlined above please submit three sets of prints of the revised plan for our approval.

Your tracing and a photostat copy of Sheet 8 of the intake details are enclosed.

Very truly yours,

E. W. Dayton  
Deputy Chief Engineer

BY: \_\_\_\_\_  
A. Dickinson  
Assoc. Civil Engineer

AD/em  
Encl.



MORRELL VROOMAN ENGINEERS  
CONSULTING CIVIL ENGINEERS  
GLOVERSVILLE, N. Y.

October 10, 1963

New York State Department of Public Works  
State Campus Site, Washington Avenue  
Albany 1, New York

Attention: Mr. Albert Dickinson

Gentlemen:

Re: Water Supply Improvements  
Raising Cork Center  
Storage Reservoir Dam  
City of Johnstown, New York

For your review and approval we are transmitting herewith Application Form E-61A1 (2/62) together with three sets of plans and specifications as follows:

Specifications - Water Supply Improvements  
Contract No. 2--Raising  
Cork Center Dam

Plans - Water Works Improvements, Raising  
Cork Center Storage Reservoir Dam,  
as prepared by us and dated  
September 1963.

We look forward to receiving your early approval for the referenced project.

Very truly yours,

MORRELL VROOMAN ENGINEERS

By Virgil Ehle  
Virgil Ehle

VE:EF  
Enc.

Johnston water supply  
Cork Center Reservoir

Dam is on Keck Center Creek flowing into Cayadutta Cr.

5 miles northwest of Johnston. Reconstructed 1963. RAISED dam 15"

Drainage around lake is 2.65 sq miles

40 year peak rate of runoff is 300 cu. ft./sec.

Spillway designed to discharge 900 cu. ft./sec.

maximum height of dam is 40 FT.

Designed maximum highwater crest above spillway is 2'-9"

with freeboard of 1'.

Concrete abutment

right bank gravel with cobbles & boulders

left "

41 acres of water 128,000,000 gallons of water at normal level

145,000,000 " " " at spillway level

drainage by a 24" cast iron pipe

dam built 1919



DEPARTMENT OF PUBLIC WORKS  
DIVISION OF CONSTRUCTION  
BUREAU OF WATERWAYS  
ALBANY

OK. J. E. Lee  
Chyatt

Received October 14, 1963 Dam No. Orig #427  
1720-3191  
Disposition Design Approved October 17, 1963 Watershed Mohawk River  
Foundation inspected .....  
Structure inspected .....

Application for the Construction or Reconstruction of a Dam

Application is hereby made to the Superintendent of Public Works, Albany, N. Y., in compliance with the provisions of Section 948 of the Conservation Law (Chapter 602, Laws of 1959) for the approval of specifications and detailed drawings, marked Raising Cork Center Storage Reservoir Dam -

Sheet 1

herewith submitted for the { ~~reconstruction~~ } reconstruction of a dam herein described. All provisions of law will be complied with in the erection of the proposed dam. It is intended to complete the work covered by the application about November 22, 1963

(Date)

1. The dam will be on Keck Center Creek flowing into Cayadutta Creek in the town of Johnstown County of Fulton and 5 miles northwest of the City of Johnstown

(Give exact distance and direction from a well-known bridge, dam, village, main cross-roads or mouth of a stream)

2. Location of dam is shown on the attached map or overlay of the Peck Lake quadrangle of the United States Geological Survey at latitude 43° 02' 14" N longitude 74° 27' 55" W

3. The name of the owner is City of Johnstown

4. The address of the owner is City Hall, Johnstown, New York

5. The impounded water will be used for water supply

6. Will any part of the dam be built upon or its pond flood any State lands? No

7. Does Section 179 of the Conservation Law (see page five of this form) apply to the above named stream? Yes.....; No...X..... If answer is yes, give Conservation Department's assigned number for permit to change or modify the stream .....

8. The area draining into the proposed pond or lake is ..... acres; 2.6 square miles.

9. The computed 40 year peak rate of runoff used in the design is 300 cu. ft. per sec.

State criterion of method used in determining the peak rate of runoff N.C.S. Dept. of Public Works - used and not exceeded on similar watersheds - also observed depth on this structure

10. The maximum height of the proposed dam above the bed of the stream will be 40 feet ..... inches.

11. The designed maximum high water elevation above the spillcrest is computed to be 2 feet 9 inches; the designed freeboard as measured from the maximum high water elevation to the top of the proposed dam will be 1 feet 0 inches.

12. The open spillway of the ~~proposed~~ <sup>existing</sup> dam that will control the designed flood flow will be of ..... concrete ..... The width of the control section of  
(State type, such as: vegetated earth, concrete, masonry, timber, rock filled crib, etc.)

the spillway, measured normal to the flow of water at the crest, will be 58 feet ..... inches in the clear; facing down stream, the waters will be held at the right end by a concrete abutment the top of which will be 3 feet 9 inches above the spillcrest, and have a top width of 10 feet 0 inches; and at the left end by a concrete abutment the top of which will be 3 feet 9 inches above the spillcrest and have a top width of 10 feet 0 inches. vertical vertical  
The slope of the sides of the spillway will be ..... on ..... (left) ..... on ..... (right).

13. The spillway is designed to safely discharge 900 cu. ft. per sec.

14. The surface area of the proposed pond or lake will be 41 acres at the normal water elevation and 41 acres at the spillcrest elevation; the volume of the water impounded in the pond or lake will be 128,000,000 gallons at the normal water elevation and 145,000,000 gallons at the spillcrest elevation.

15a. The normal water elevation of the proposed pond or lake will be varying feet ..... inches below the spillway crest, and will be maintained by means of a ..... ; the pond or lake will be drained by means of a 24" cast iron pipe ; provision will be made for supplying water to riparian owners downstream, during dry seasons, by means of dam in place since 1919.

15b. In addition to normal water control, provision must be made for a bottom draw-off if the pond is on a trout stream of constant flow. The draw-off will be by means of a ..... , designed to maintain an outflow of one-half of the minimum inflow of the stream of ..... cu. ft. per sec. up to a maximum outflow of one cu. ft. per sec.

16. The maximum discharge through the spillway that controls the normal water elevation will be 900 cu. ft. per sec. during maximum high water.

17. If flashboards are to be used to control flood flow they must be of the automatic or self-tilting type, designed to fail or otherwise permit full discharge through the spillway when the flood waters reach a height of ..... feet ..... inches above the spillcrest.

18. If an overfall structure is used as a spillway, it shall be provided with an apron constructed of in place; the thickness of the ..... will be ..... feet ..... inches, the width ..... feet ..... inches across the stream and the length ..... feet ..... inches parallel to the stream.

19. Facing downstream, what is the nature of material composing the right bank? .....  
Gravel with cobble and boulders

20. Facing downstream, what is the nature of the material composing the left bank? .....  
Gravel with cobble and boulders

21. The natural material of the bed on which the proposed dam will rest is (clay, sand, gravel, boulders, granite, shale, slate, limestone, etc.) in place - no record

22. Are there any porous seams or fissures beneath the foundation of the proposed dam? .....  
No appreciable leakage

23. State the character of the bed and the banks in respect to the hardness, perviousness, water bearing, effect of exposure to air and to water, uniformity, etc. Bed - boulders  
Banks - gravel with cobble and boulders

24. Was the above soil information obtained from soil borings? .....; test pits? .....

25. State how much above the spillcrest elevation is the lowest part of the immediate upstream adjoining property or properties, 4 feet 0 inches.

26. Does this proposed pond or lake constitute any part of a public water supply? Yes

27. State if any damage to life or to any buildings, roads or other property could be caused by any possible failure of the proposed dam Small intake reservoir - remainder farmland

28. The design, plans and specifications have been prepared under the supervision of: (Sign on applicable line below). MURRELL VROOMAN ENGINEERS

(a) By *Murrell Vrooman, Jr.* P. E. License No. 22134  
(Signature) (Title: Engineer or Conservational)  
Address 21 North Main Street, Gloversville, New York

(b) ..... U. S. D. A. Soil Conservation Service  
(Signature) (Title)

(c) ..... N. Y. S. Conservation Department Engineer  
(Signature) (Title)

(d) ..... Other qualified engineer.  
(Signature) (Title)

29. The raising of dam will be under the supervision of: (Sign on applicable line below).  
(State where: Erection, Reconstruction or Repair)

CITY OF JOHNSTOWN  
(a) By William H. McGregor P. E. License No. 27760  
William H. McGregor, City Engineer  
Address City Hall, Johnstown, New York (Telephone 6-9414)

(b) \_\_\_\_\_ U. S. D. A. Soil Conservation Service  
(Signature) (Title: Engineer or Conservator)

(c) \_\_\_\_\_ N. Y. S. Conservation Department Engineer  
(Signature) (Title)

(d) \_\_\_\_\_ Other qualified engineer.  
(Signature) (Title)

The foregoing information is correct to the best of my knowledge and belief, and the construction will be carried out in accordance with the approved plans and specifications.

CITY OF JOHNSTOWN, NEW YORK, Owner  
By Mario H. Costa, authorized agent of owner.  
Address of signer City Hall, Johnstown, New York Date October 7, 1963

### INSTRUCTIONS

Read carefully, the law setting forth the requirements to be complied with in order to construct or reconstruct a dam.

Determine first whether the stream, across which the dam is to be erected or from which water for the proposed pond or lake is to be diverted, is under the jurisdiction of the Conservation Department. This information may be obtained upon request from the manager of the District Fisheries Office of the Conservation Department which has jurisdiction in the County where the stream is located, the Conservation Department, Bureau of Fish, State Campus Site, Albany 1, New York or the New York State Department of Public Works, Bureau of Waterways, Albany 1, New York.

Before a dam may be erected across a natural water-course, the riparian rights of other land owners (both upstream and downstream) must be considered and customarily their consent be obtained as such rights have been adjudged by the civil courts to be inalienable and inviolate.

The elevation of the impounded water should be maintained at a suitable level below the lowest contour of the adjoining properties thereby preventing inundation of the properties during the highest stage of the waters.

Each application for the construction or reconstruction of a dam must be made on this standard form, copies of which will be furnished upon request to the New York State Department of Public Works, Bureau of Waterways, Albany 1, New York. The application, properly executed, must be accompanied by three sets of plans and specifications. The plans must contain the following information:

- a. A topographical plan (with contours) of the impounded area drawn to a suitable scale.
- b. A profile and transverse section of the impounded area showing the proposed excavation, the normal water and possible high water elevations. A 1'-0" minimum of freeboard is to be provided between the top of the dam and the possible high water.
- c. A longitudinal elevation and transverse section of the dam with all the necessary details of the related appurtenances, spillways, drains, etc.
- d. A log of the soil information. Samples of the materials to be used in the dam and of the material upon which the dam is to be founded may be asked for, but need not be furnished unless requested.

No work of construction, reconstruction or repairs of the structure or structures shall be started until after the plans and specifications have been formally approved by the New York State Department of Public Works.

If the dam constitutes a part of a public water supply, application should also be made to the Water Resources Commission under Article V of the Conservation Law, as amended.

An application for the construction or reconstruction of a dam must be signed by the prospective owner of the dam or his duly authorized agent. The address of the signer and the date must be given as provided for in this application form.

DNR DAM INSPECTION REPORT

<input checked="" type="checkbox"/> 3	<input checked="" type="checkbox"/> 18	<input checked="" type="checkbox"/> 63	<input type="checkbox"/> <input type="checkbox"/> <input type="checkbox"/> 3191	<input type="checkbox"/> <input type="checkbox"/> <input type="checkbox"/> <input type="checkbox"/> 102369	<input type="checkbox"/> <input type="checkbox"/> 03	<input checked="" type="checkbox"/> 2
RB	CTY	YR AP.	DAM NO.	INS. DATE	USE	TYPE

AS BUILT INSPECTION

<input checked="" type="checkbox"/> Location of Sp'way and outlet	<input checked="" type="checkbox"/> Elevations
<input checked="" type="checkbox"/> Size of Sp'way and Outlet	<input checked="" type="checkbox"/> Geometry of Non-overflow section

GENERAL CONDITION OF NON-OVERFLOW SECTION

<input checked="" type="checkbox"/> Settlement	<input checked="" type="checkbox"/> Cracks	<input checked="" type="checkbox"/> Deflections
<input checked="" type="checkbox"/> Joints	<input checked="" type="checkbox"/> Surface of Concrete	<input checked="" type="checkbox"/> Leakage
<input checked="" type="checkbox"/> Undermining	<input checked="" type="checkbox"/> Settlement of Embankment	<input checked="" type="checkbox"/> Crest of Dam
<input type="checkbox"/> Downstream Slope	<input checked="" type="checkbox"/> Upstream Slope	<input checked="" type="checkbox"/> Toe of Slope

*no apparent repairs needed or minor repairs that can be covered by maintenance*

GENERAL COND. OF SP'WAY AND OUTLET WORKS

<input checked="" type="checkbox"/> Auxiliary Spillway	<input checked="" type="checkbox"/> Service or Concrete Sp'way	<input checked="" type="checkbox"/> Stilling Basin
<input checked="" type="checkbox"/> Joints	<input checked="" type="checkbox"/> Surface of Concrete	<input checked="" type="checkbox"/> Spillway Toe
<input checked="" type="checkbox"/> Mechanical Equipment	<input checked="" type="checkbox"/> Plunge Pool	<input checked="" type="checkbox"/> Drain

<input checked="" type="checkbox"/> Maintenance	<input checked="" type="checkbox"/> Hazard Class
<input checked="" type="checkbox"/> Evaluation	<input checked="" type="checkbox"/> Inspector

COMMENTS:

*no further action*





**PART II - INVENTORY OF DAMS IN THE UNITED STATES**  
 (PURSUANT TO PUBLIC LAW 87-474)

See reverse side for instructions.

Project Control Sheet  
 DATE: C/E-11

STATISTICS	SPILLWAY			VOL. AMT. OF DAM (CF)			INSTALL. CAP. (CUB. FT.)			PROPOSED (ACR.)			NAVIGATIONAL LOCKS					
	CRGHT LENGTH (FT)	WIDTH (FT)	BASEWIDTH (FT)	WIDTH (FT)	LENGTH (FT)	HEIGHT (FT)	WIDTH (FT)	LENGTH (FT)	HEIGHT (FT)	WIDTH (FT)	LENGTH (FT)	HEIGHT (FT)	WIDTH (FT)	LENGTH (FT)	HEIGHT (FT)	WIDTH (FT)		
1	550	66	706	85	55	5	0	0	0	0	0	0	0	0	0	0		

MISC DATA	OWNER	ENGINEERING BY	CONSTRUCTION BY
1	CITY OF WASHINGTON		

MISC. DATA	DESIGN	CONSTRUCTION	OPERATION	REGULATORY AGENCY	MAINTENANCE
1	DC				

MISC. DATA	INSPECTION BY	INSPECTION DATE	AUTHORITY FOR INSPECTION
1	DC	23 JUL 1967	DC

REMARKS

1120-5111  
Mchewik

**JAMES P. WELLS**  
CONSULTING ENGINEER  
CUTLER BLDG. ROCHESTER, N.Y.

May 20, 1919.

Mr. A. E. Perkins,  
Conservation Commission,  
Albany, N. Y.

Dear Mr. Perkins:-

I am having sent from Johnstown the three blocks of concrete for test in regard to which I wrote you.

These blocks were made in the following manner: The proportions of material were 1 part cement to three (3) parts of a mixture of bank sand and crusher dust and six (6) parts of crushed stone. They were made by taking a small amount at a time from several batches of concrete as it came from the mixer. They were made last June. The concrete was allowed to set and the cubes were then put in water for a week and since that time they have been out in the open and the forms have never been taken off.

Sincerely yours,

JPW/c

*J.P. Wells*

STATE OF NEW YORK  
DEPARTMENT OF STATE ENGINEER AND SURVEYOR  
ERIE CANAL, RESIDENCY NO. \_\_\_\_\_

SUBJECT:

June 10, 1919.

Conservation Commission,  
Albany, N. Y.

Attention of Mr. McKim.

Dear Sir:-

We have received three concrete cubes which bore one of your tags, No. 259, and marked as "From Johnstown Water Works, Johnstown, N. Y., For dam at Cork Center on the Creek".

In accordance with your instruction we have tested these cubes for compressive strength. The results of the tests are as follows:-

Cube No.	Dimensions inches	Area sq. in.	Compressive Strength	
			Total	Per Sq. in.
1	6.0x6.0x6.0	36.0	113,000 lbs	3,140 lbs
2	6.1x5.9x6.0	36.0	109,500 "	3,040 "
3	6.0x6.0x6.0	36.0	121,000 "	3,360 "
Average			3,180 lbs	

Yours very truly,

*Russell S. Sherman*  
Sen. Asst. Engineer,  
in charge of Tests.

MEMORANDUM REGARDING PROPOSED DAM #427 MOHAWK  
ON CORK CENTER CREEK

CITY OF JOHNSTOWN, APPLICANT

Serial No. 275  
-----

The Papers relating to this dam, as received by the writer from Inspector McKim, were as follows:

- (1) Application form IW56, dated July 9, 1917, completed in duplicate, and executed by Louis K. Maylender as President of the Board of Water Commissioners of the City of Johnstown.
- (2) Engineer's Report (3 sheets in backer) signed jointly by W. E. Natanson, City Engineer, and James P. Wells, Consulting Engineer.
- (3) Specifications in duplicate (13 sheets in backer)
- (4) Certified copy of resolution adopted July 9, 1917, by the Board of Water Commissioners approving the plans, ordering the construction of the dam, etc. (4 sheets in backer)
- (5) Pamphlet - 39th Annual Report of Board of Water Commissioners, City of Johnstown, N. Y., dated Dec. 31, 1916.
- (6) General location and watershed map, marked (1) on U.S.G.S. Cloversville quadrangle.
- (7) Portfolio containing blue prints of plans submitted in duplicate, (each containing 7 sheets size 24" x 36" numbered consecutively) showing plans, sections and details descriptive of the dam, spillway channel, reservoir and structures appurtenant thereto.

The following described data is lacking, although required by our printed instructions to applicants:

Engineer's explanation of stability of the overfall section, giving methods of computation and results as to -

- (a) Overturning
- (b) Sliding
- (d) • • • Sufficiency of washwall.

The plans or specifications are lacking in clearness in the following particulars:

- (1) Spillway channel bed. Paragraph 20 of specifications reads in part - "To break the flow of water, rubble-stone will be laid in the bed of the spillway channel so that about half the stone is imbedded in the concrete. All such stone shall be about 2 cubic feet in size and will be spaced about

three (3) feet apart center to center." Neither the plan, profile nor sections of such spillway channel, appearing on the drawings numbered 4, 5 and 6 (Acc. 8059, 8060, 8113) indicate in what portion such projecting stones are to be laid.

- (2) Apron slope of the overfall section of the dam is neither marked on the plans nor were dimensions discovered from which it might be computed.

The site of the dam, as indicated on the U. S. G. S. map attached to the original copy of this memorandum, is in the Town of Johnstown, Fulton County, on Cork Center Creek, a branch of Cayadutta Creek, at a distance, measured along the stream, of about 4-1/2 miles above Sammonsville, and about 8-1/2 miles above the Village of Fonda. The U. S. G. S. sheets show the stream as flowing, throughout its entire length, in a very narrow valley, and the drawings submitted by the applicant indicate a total fall of about 750 feet between the water surface elevation at the proposed dam and that at the Village of Fonda (population 1,125). Immediately above Fonda the stream bed appears to have a fall of about 50 feet to the mile. At a point about 1-3/4 miles below the site of the proposed dam (storage capacity 22,000,000 cu.ft.) the U. S. G. S. sheet indicates a concentrated fall of about 350 feet in 1-1/2 miles, and Sammonsville lies almost immediately below.

The drainage area above the proposed dam site on Cork Center Creek is stated in the application as 2.86 square miles. The slopes are generally rolling or fairly steep throughout, and tributary stream beds short and almost straight. There is only one other small pond shown within the area and the proposed reservoir (water surface 50 acres) only appears to have a storage capacity above the spillway crest of about 2,400 acre inches, or 1.31" on the watershed.

The maximum probable flood from the entire watershed was estimated by the applicant's engineers as 572 cubic feet per second, although they further stated that the spillway had been designed to take care of 560 cubic feet per second per square mile, or a total of 1600 cubic feet per second. The formula for maximum discharge, as developed by our Mr. McKim, indicates a probable flood of about 1315 cubic feet per second from the watershed under consideration while this Commission's enveloping curve (Acc. C-1618) indicates a probable flood discharge of about 270 cubic feet per second per square mile, or a total of about 770 cubic feet per second.

#### SPILLWAY CHANNEL CAPACITY

With stones set in its bottom, projecting upward about one foot and spaced about three feet center to center, as provided for by paragraph 20 of the specifications, and neglecting the retarding effect of the two curves in the channel, it does not appear that the section 20 feet wide, with a 5/10% grade, would discharge more than about 785 cubic feet per second when flowing full to the top, which is equivalent to a clear depth of about 5 ft. above the tops of such stones. This value has been determined by the use of the Chezy and Kutter formulas (assuming a value for the coefficient of roughness of .03 as recommended by R. E. Horton for cement channels with "dry-rubble surface", in good condition) and represents an average velocity of about 7.85 feet per second and only about 49%

of the discharge for which the overfall section is said to have been designed.

With a smooth cement bottom and neglecting the retarding effect of the two curves in the channel, it appears that the section 20 feet wide, with a 5/10% grade, would discharge the quantity for which the overfall section is said to have been designed, with a depth of about 4.9 feet, such value having been determined by the same method as stated in the last paragraph, except that the value of .014 for such channels in good condition was assumed for the coefficient of roughness.

Three curves are shown in the proposed channel, each of which changes the direction of flow by an angle nearly ninety degrees. At the first curve, which occurs immediately at the foot of the overfall section, the designer increases the height of the extreme side of the channel by about 33-1/3% to prevent overflow. At the second curve, which occurs at the middle of the section shown as 20 ft. wide (and for which the discharging capacities were computed as stated in the previous paragraph) both sides of the channel are indicated as the same height of 6 ft. (Section - Sta. 2+50, Sheet 6, Acc. 8113). It is possible that the channel might overflow on the outside of the curve at this point, although the drawings (Sheet 2, Acc. 8057) do not seem to indicate that a failure of such channel would impair the earth embankment to any great extent.

#### STABILITY OF THE OVERFALL SECTION

The section above the natural surface, as indicated by the elevation of 1047', on drawing 4 (Acc. 8059), was first investigated in accordance with the following assumptions:

- (1) That the apron slopes 5.2 feet backward in a rise of 7 ft.
- (2) That an open joint may form along a horizontal plane at the elevation stated.
- (3) That the intensity of hydrostatic pressure upon the base of the section above such a joint might range from maximum at the back to zero at the intersection with the apron slope.
- (4) That two-thirds of the area of the triangle representing the hydrostatic pressure upon the base of such a section would approximately represent the total of such pressure.

From an investigation of the forces acting upon the section formed as described above, it was found that the point of application of the resultant fell 13.6% outside the middle third of the base (using the length of that portion as a unit) when overtopped to a depth of four feet. The corresponding coefficient of stability was about 1.1, and the required coefficient of friction to prevent sliding was about 94%. If a coefficient of friction between

concrete surfaces of 70% may be safely assumed, then the point of application of the resultant of the forces acting would seem to fall well within the middle third when the total static pressure amounts to about 590 lbs., or approximately 33-1/3% of full uplift determined as described above. If a coefficient of friction of only 60% is assumed as safe between such surfaces, then the allowable static pressure would be about 195 lbs., or approximately 11% of full uplift determined as described above.

The complete section, as indicated on drawing sheet 4 (Acc. 8059) was <sup>first</sup> investigated with the following assumptions:

- (1) That the apron slope is to be the same as already stated.
- (2) That an open joint might form between the horizontal slab and the vertical upstream face of the overfall section.
- (3) That the line of seepage follows entirely around the outline of the submerged portion of the dam and cut-off wall (4 ft. thick).
- (4) That the intensity of hydrostatic pressure ranges from maximum at the top of the horizontal slab (Elev. 1047) on the up stream side of the section, and declines progressively along such a seepage line, as described above, to zero at an assumed open joint so located that the horizontal length of the base of the section would be about 11.6 ft.
- (5) That the total area of that portion of the triangle representing the hydrostatic pressure which would apply against the vertical back of the section, and two-thirds of the areas of those portions of the same triangle which would apply against the remainder of the base of the section, would approximately represent the total of such hydrostatic pressure below the top of the horizontal slab behind the section under consideration.
- (6) That ice pressure may be safely neglected.
- (7) That the resistance to sliding offered by the paving slab in front and cohesion between vertical sections of the dam may be neglected.
- (8) That the soil, described by the applicant's engineers as sandy loam will exert additional pressure upon the back of the dam and the front of the cut-off wall (4 ft. thick), in accordance with the general theory, taking into consideration the loads superimposed.
- (9) That the approximate weight of the sub-soil may be safely assumed as 100# per cubic foot and that its plane of repose slopes approximately 2 on 3.

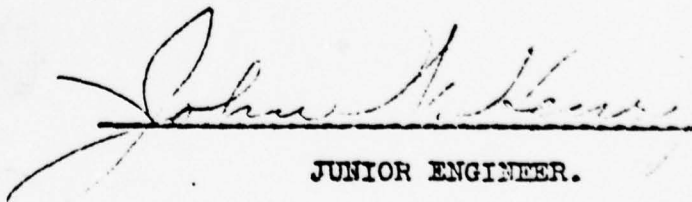
From an investigation of the forces acting upon the section formed as described above, and overtopped to a depth of 4 ft., it was found that the point of application of the resultant fell well within the middle third of the length of the base (on the upstream side of the middle), the value of the coefficient of stability was about 1.5, and that of the coefficient of friction between the material of the dam and the sandy loam or hardpan sub-soil required to prevent sliding would be about 74%, which may not be warranted. It also appears, however, that an intensity of resistance of only 690 lbs. per square foot exerted by the sub-soil in front of the downstream side of the cut-off wall, would reduce the coefficient of friction to about 45%, and it seems quite possible that such an assumption could be safely made.

The maximum sub-foundation load seems to maintain when the elevation of the water surface in the reservoir lies below the base of the overfall section. The intensity of such pressure was determined to be about 1-1/4 tons per square foot, acting upon a damp sandy loam or hard-pan, as described by the applicant's engineers.

#### RETAINING WALLS

The retaining walls for protecting the earth embankment at the ends of the overfall section, as outlined on the profile along the center line of the dam (drawing sheet 3, Acc. 8058), seem to be of rather scant dimensions, ranging as they do from a 2 ft. top width to a maximum width of 4 ft. at a point about 8 ft. below the top. The exposed total height of such walls on the upstream side of the overfall section is about 12 ft. and they must resist some static pressure as well as earth pressure. The length of these sections having a 12 ft. exposed height is short, yet their location is such that should a flood occur immediately after a failure the upstream side of the embankment would be badly eroded and this might result in the undermining of the core wall and failure of the entire structure.

Respectfully submitted,



JUNIOR ENGINEER.

To Mr. A. H. Perkins,  
Division Engineer.

July 24, 1917.



SUBJECT: City of New York, Dept. of Sanitation, Sewerage and Waterworks, 27th St. Station FILE NO. DM 42-11  
Storage Basin for Dam #27 (Mohawk) Serial #275 ACC. NO. C-3617

COMPUTED BY: W. G. ... DATE: July 14, 1917 CHECKED BY: \_\_\_\_\_  
 MADE IN CONNECTION WITH: Investigation of sewerage disposal at 27th St. Station

REFERENCE: Original application and accompanying papers as listed on back of this form

Spillway Channel Capacity (Station 100) (Accessed)  
 (Neglecting Effects of rubble bottom and curves) (Channel Width 20 ft, Accessed)

Maximum (Neglecting Velocity of Approach) Grade = 5% or 1 in 20  
 Using Weir Formula Maximum Depth = 6 ft  
 $C = 3.35 = \frac{Q}{L \cdot H^{3/2}}$   $\log 3.35 = 2.525$   $\log 20 = 1.301$   
 $5 \cdot 20^2 = 200^2 = 40,000$   $\log 40,000 = 4.602$   $\times 10 = 23.01$   
 $= 2.04 = \frac{4.602 - 2.525}{1.301} = 1.57$   $\times 2 = 3.14$   
 $C = 0.309$   $\times 2 = 0.618$   $\times 12 = 7.416$

$Q = C \cdot L \cdot H^{3/2} = 2.04 \times 20 \times 6^{3/2} = 6000 \text{ c.f.s.}$  (This is too high as for weir with velocity of approach. See U.S.G.S. R. 200, p. 27 and III.)  
 Note: This is much smaller than assumed by the designing engineer in his application and report.

Correct for maximum possible velocity of approach.

$Q = A \cdot V = 1600 \text{ c.f.s.}$   $V_m = \frac{Q}{A} = \frac{1600}{120} = 13.33 \text{ ft. per sec.}$   
 $h = 0.155 \cdot V_m^2 = 0.155 \cdot 178 = 27.59 \text{ ft.}$   $H = 6 \text{ ft.}$

$Q = 2.04 \left( b - \frac{h}{10} \right) \left[ \frac{2}{3} H - h \right] = 87 \text{ c.f.s.}$  (Francis Formula)  
 Using Francis formula for sharp crested weir. Probably too low. Note opposite values of G above for weir.

Using coefficient for broad crested weir (U.S.G.S. W.S. R. 200, p. 121 and 122)

$Q = 2.64 \times 20 \times 21.32 = 1125 \text{ c.f.s.}$  (For weir with rounded front and back. Discharge therefore too high if the velocity too high.)

Note: Both of the last two computed discharges were based on the assumption that the spillway channel at Sta. 100 (Accessed) would flow full to top with correction for full velocity head required to pass the 1600 c.f.s. through such a water cross section. Both would probably be much less than six feet and it seems improbable that the resulting high velocity would maintain any such values at discharge. Channel bottom at Sta. 100.

Using Chezy and Kutter Formulae  $V = C \cdot \sqrt{R \cdot S}$

Assuming Channel Flowing Full throughout entire length  $A = 6 \times 20 = 120$   
 $n = 0.03$  (for rubble)  $S = \frac{h}{L} = \frac{5}{100} = .05$   $r = \frac{A}{P} = \frac{120}{32}$   
 $C = 42$  (from 56)  $V = 61.9 \sqrt{0.05 \times .05} = 8.48 \text{ ft. per sec.}$

$Q = A \cdot V_m = 120 \times 8.48 = 1018 \text{ c.f.s.}$  (See Arts. 7 and 15 also.)  
 or  $Q = 120 \times 8.48 = 1018 \text{ c.f.s.}$  (Corrected)

SUBJECT City of Johnston FILE NO.                       
Dam # 27 Mohawk (Serial # 275) ACC. NO. G-3678  
See sheet # 1. SHEET 2

COMPUTER W. Henry July 12, 1917 CHECKED BY                      19                     

MADE IN CONNECTION WITH                     

REFERENCE                      Curr's from Acc. G-3677

Spillway Channel Capacity continued from sheet 1.

Chezy Formula - Continued

If stones are not imbedded in bottom of spillway channel as seemingly required by TP 20 of specifications and bottom was finished smooth then value of "n" of 0.014 might be warranted for 1720

$$v = C \sqrt{rs} = 131.4 \times .137 = 18.0 \text{ ft/sec. (Revised)}$$

$$Q = 120 \times 18 \text{ ft/sec} = 2,160 \text{ c.f.s. with channel flowing full to top which would probably not maintain.}$$

Determining depth required to discharge 16000  
 Using value of "n" for concrete lined channel in good condition = 0.014 (Acc. 1730)

Try depth of 5 ft in channel

$$v = C \sqrt{rs} = 129.5 \sqrt{3333 \times 0.055} = 16.7 \text{ ft/sec. } A = 100$$

$$Q = 100 \times 16.7 = 1,670 \text{ c.f.s. } r = \frac{A}{p} = \frac{100}{30} = 3.33$$

Try depth of 4.9' in channel

$$A = 4.9 \times 20 = 98$$

$$r = \frac{A}{p} = \frac{98}{29.8} = 3.29$$

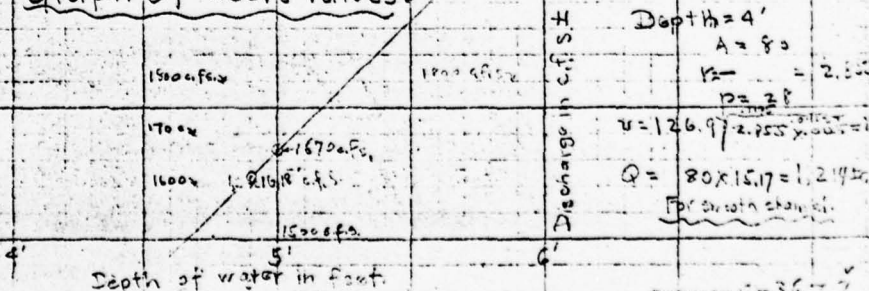
$$p = (2 \times 4.9) + 20 = 29.8$$

$$v = C \sqrt{rs} = 129.4 \sqrt{3.29 \times 0.055} = 16.5 \text{ ft/sec.}$$

$$Q = 98 \times 16.5 = 1,618 \text{ c.f.s.}$$

Curves in channels will tend to reduce the probable discharge while steeper grade below will tend to increase it

Graph of above values:



SUBJECT City of Malheur FILE NO.             
Dam 42.7 Malheur (Serial #275) ACC. NO. C-3679  
See sheet #1. SHEET 3

COMPUTER J. Henry July 11, 1917 CHECKED BY           

MADE IN CONNECTION WITH            Draw's Form Acc. C-3678

REFERENCE Overfall Section - Discharging Capacity

Designing Engineer reports as follows (Ref App #27)

Watershed 2.86 Sq. Mi. ( $2.86 \times 640 = 1,831$  Acres)  
 Water surface 50 Acres. ( $43,560 \times 50 = 2,178,000$  sq. ft.)  
 Reservoir capacity 22 M.C.F. (above natural surface)  
 Spillway Length 60 ft.  
 May be overtopped 4 ft. ( $4 \times 50 = 200$  sq. ft. inches spread)  
 Resulting discharge 1600 c.f.s. ( $200 \times 8 = 1,600$  c.f.s. on watershed)  
 or 560 c.f.s./Sq. Mi.

Spillway divided into 3 sections by bridge supports.

Each <sup>end</sup> section =  $\frac{60}{3} = 19.33$  ft. and Sq.  $\frac{2}{3}$  ends

Middle section 19.33 ft. and Sq.  $\frac{1}{3}$

Breadth Effective as corrected for end contractions

End sections:  $b = 19.33 - \frac{0.3}{2} = 19.03$  ft. (Ends)

Middle Section:  $b = 19.33 - \frac{0.5 \times 2}{2} = 18.73$  ft. (Middle)

Approximate effective length = 56.79 ft. Total.

→ Discharge by W.S.P. 200, Tables 98, 12, section 11:

$28.19 \text{ c.f.s.} \times 1.11 \times 56.79 \text{ ft.} = 1,778 \pm \text{c.f.s.}$

$28.65 \pm \text{c.f.s.} \times 56.79 \text{ ft.} = 1,630 \pm \text{c.f.s. ES. Curve}$

1600 c.f.s. probably conservative

McKinnis Formula for flood discharge:

$Q = 600 A^{\frac{2}{3}} = 600 \times 2.86^{\frac{2}{3}} = 1,314 \pm \text{c.f.s.}$

Capacity at Spillway overfall section OK.

(Height in feet)

1-23-57-3000 (10-2001)

Form 100

Form 975

STATE OF NEW YORK - CONSERVATION COMMISSION

Dm 427M

SUBJECT City of Watkins

FILE NO.

Dam 427 Mohawk (Serial 275)

ACC. NO. C-3680

SHEET A

COMPUTER W. Henry 12 July 1957 CHECKED BY

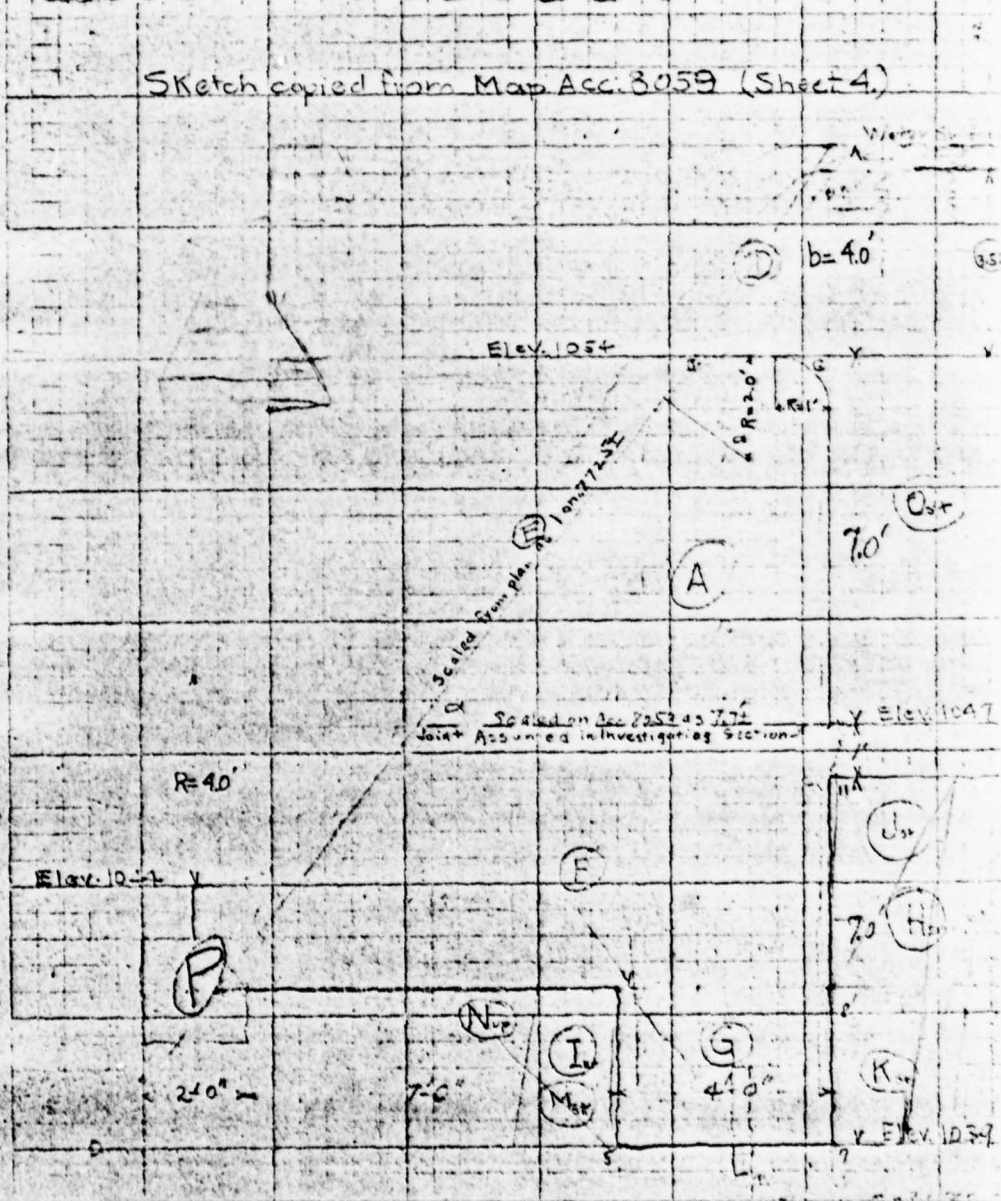
MADE IN CONNECTION WITH

Dist's File Acc. C-3679

REFERENCE

Overfall Section - Resistance against overturning and sliding

Sketch copied from Map Acc. 3059 (Sheet 4)



7-29-72-2500 (10-2061)  
 Acc. 106  
 Form 706

STATE OF NEW YORK - CONSERVATION COMMISSION

Dm 427M

SUBJECT City of Johnstown  
 Dam 42 / Mohawk (Serial = 275)

FILE NO. 2  
 ACC. NO. C-3681

SHEET 5

COMPUTER J. Wherry 12 July 1977 CHECKED BY

MADE IN CONNECTION WITH

Comm's form Acc. C-3680

REFERENCE Overfall Section Resistance (Cont'd.) Sketch on Sheet 4

Neglecting Ice Pressure

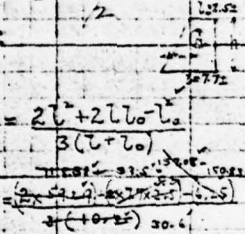
Investigating section 1-ft thick, overtopped 4 ft.

Assuming horizontal joint at intersection of natural surface line with back of overfall section (El. 10+7)

(A) Enveloping trapezoid

Area:  $\frac{5.1 + 7.7}{2} \times 7 = 35.7$  Area Arm Moment

Arm:  $\frac{2L^2 + 2Ll - l^2}{3(L+l)}$



Moment:  $35.7 \times 4.9 = 176.0$

(C) Portion cut from back of crest

Area:  $1 - (7.75 \times 4) = -0.25$

Arm: (Approximately)  $7.7 - 2 = 5.7$

Moment:  $-0.25 \times 5.7 = -1.5$

(B) Portion cut from front of crest

Area:  $\tan \alpha = \frac{1}{7.725} = 1.293^\circ$

$\angle \alpha = 52^\circ 17'$

$\frac{52.33^\circ}{360^\circ} = \frac{1.825}{12.5664}$

$\frac{(2.0 \times 1/2)}{2} = \frac{1.825}{12.5664} = 0.15$

Arm (Approximately)  $7.7 - 2.5 = 5.2$

Total Net Moment (Area x Ft.)  
 $M_{r_{net}} = 140 \times 173.6 = 24,300$

Comm's on Acc.

STATE OF NEW YORK - CONSERVATION COMMISSION

Dm 427.1

SUBJECT City of Yonkers FILE NO. \_\_\_\_\_  
Dam 427 Manawick (Serial No. 275) ACC. NO. C-3682  
See Sheet #1. SHEET 6  
 COMPUTER Sperry July 13, 1917 CHECKED BY \_\_\_\_\_

MADE IN CONNECTION WITH \_\_\_\_\_  
 REFERENCE City's new Act C-3682

Overflow Section - Investigation of Resistance (Contd.)

	Area	Arm	Moment
Water Resisting Moments:			
(B) Neglected.			
(C) (From sheet #5)			+ 1.5 ✓
(D) Prism above plane of crest:			
Area: (Assumed as equal to 1/2 the section of prism 2.5 x 4)	+ 5.0 ✓		
Arm: $7.7 - \frac{2.5}{3}$		6.9 ✓	
(E) Moment of Prism of water sliding on apron neglected.			+ 34.5 ✓
Total Net Moment (Area x Ft.)			= 36.0 ✓
$M_{r(water)}$ in Ft. lbs = $(62.5 \times 36.0)$			= 2250 ✓
$M_r = \text{Total Resisting Moment} = M_{r(water)} + M_{r(masonry)}$			= 26,550 ✓

$M_{(masonry)} = \gamma(H-b)^2(H+2b)$   
 $= 62.5 \left( \frac{11 \times 11}{2} \times \frac{11}{3} + \frac{11 \times 11}{2} \times \frac{11}{3} \right)$   
 $M_{(masonry)} = \frac{\gamma H^2 L^2}{2} \times \frac{2}{3} = \frac{62.5 \times 11^2 \times 7.7^2}{2} \times \frac{2}{3} \times \frac{2}{3} \times 7.7$   
 $M_o = \text{Total Overturning Moment} = 18,760$   
 $M_r - M_o = 7,790$

Vertical Component of Resultant

	Areas (Cu. Ft.)	Weight
(A) Masonry	+ 35.7 ✓	
(B+C) Masonry at 140 #/cu. ft.	35.3 ✓	+ 4,950 ✓
(D) Water	+ 5.0 ✓	
(E) Water pressure	5.215 x 62.5 ✓	+ 326 ✓
$+ V_o = \text{Total downward pressure}$		= 5,276 ✓

STATE OF NEW YORK - CONSERVATION COMMISSION Dm 4-27M  
 SUBJECT City of Johnstown FILE NO. 10-10-10-10  
 Dam 427 Mohawk (Serial - 275) ACC. NO. C-3683  
 COMPUTER *M. Murray, 13 dy* 1917 CHECKED BY SHEET 7  
 MADE IN CONNECTION WITH 13  
 REFERENCE Cory's note Acc. C-3683

Overfall Section - Investigation of Stability Contd.

Vertical Component of Resultant Contd.

+Ve (From sht. 6)	+5,276
Uplift (Sht. 6)	-1,766
$V_E = \text{Net}$	<u>+3,510</u>

$U = \frac{M_E - M_o}{V_E} = \frac{7,790}{3,510} = 2.22 \pm$  Arm of Ve  
12.6% outside middle third.

$\frac{L}{3} = \frac{7.7}{3} = 2.567 \pm$  Length of middle third

Note: Since the point of application of the resultant falls outside the middle third, measured by the usual practice, the dam is said to be unsafe as the concrete at the section must resist tension which is not permitted by best practice. (See below and on sheet 6)

Sliding - Stability against

$H_o = \frac{\gamma H + \gamma_b h}{2} \times h = \frac{62.5(4+4)}{2} \times 7 = 3,282 \pm$

$F = \frac{V_E}{H_o} = \frac{3,510}{3,282} = 1.068 \pm$  Coefficient of stability

$f = \frac{H_o}{V_E} = 93.6 \pm$  {Coefficient of friction should not exceed 70 or 75%

Seemingly unsafe against sliding

Neglecting Uplift. (Sheets 6-7)

$\frac{M_E - M_o}{V_E} = \frac{26,550}{5,276} = 5.03 \pm$  Arm of Ve  
19.7% inside middle third

$F_1 = \frac{V_E}{H_o} = \frac{5,276}{3,282} = 1.61 \pm$  Coefficient of stability

$f_1 = \frac{H_o}{V_E} = 62.3 \pm$  {Coefficient of friction should not exceed 70 or 75%}

SUBJECT City of Mohawa (Serial # 275) FILE NO. \_\_\_\_\_  
See Sheet #1 ACC. NO. C-3684  
 SHEET 8

COMPUTER Wickery 19 CHECKED BY \_\_\_\_\_ 19

MADE IN CONNECTION WITH \_\_\_\_\_

REFERENCE Comm. from Acc. C-3682

Overall Section - Investigation of Stability - Control  
Uplift Perm. 5.13 in accordance with previous assumptions

$$U_2 = \frac{2}{3} = \frac{Mr - (M_0 + \frac{2}{3} \times 7.7 P_u)}{V_e - P_u} = \frac{2.567 \times 16,850 - 9,700 - 5.13 P_u}{13.530 \times 5,276 - P_u} = \frac{43,110 - 5.13 P_u}{70,800 - P_u}$$

$$2.567 \times 5,276 - 2.567 P_u = 16,850 - 5.13 P_u$$

$$256 P_u = 3,322$$

(N.B. See Revision 02)  $P_u = 12.95$  or 73.6% of Total Uplift  
 (As far as overturning is concerned)  
 (See Further Revision Below)

Sliding

$$F_2 = \frac{V_e - P_u}{H_0} = \frac{3,981}{3,282} = 1.21 \text{ Coefficient of Stability}$$

$$f_2 = \dots = 0.26 \text{ Coefficient of Friction}$$

Preferably should not exceed .60

$$f_3 = \frac{H_0}{V_e - P_u} = \frac{3,282}{5,276 - P_u} = .60$$

$$.60 \times 5,276 - .60 P_u = 3,282$$

Recomputing allowable uplift for sliding with coefficient of friction .60

$$P_{u3} = \frac{3,282 - 3,165.6}{.60} = 195 \text{ or } 11\% \text{ of total uplift}$$

$$f_4 = \frac{3,282}{5,276 - P_{u4}} = .70$$

$$.70 \times 5,276 - .70 P_{u4} = 3,282$$

$$3,693.2 - .70 P_{u4} = 3,282$$

$$.70 P_{u4} = 411.2$$

$$P_{u4} = 588 \text{ or } 33.3\% \text{ total uplift}$$

Unit Stresses with full upward pressure

$$P_a = \frac{V_e + \frac{6V_e d}{L}}{L} = \frac{456 + \frac{6 \times 1,960 \times (7.7 \times 4.25)}{7.7}}{7.7} = 3,437 \text{ or } 242$$

$$P_b = \frac{V_e - \frac{6V_e d}{L}}{L} = \frac{456 - 3,033}{7.7} = 2,577 \text{ or } 17.88$$

Using class B (1245) as specified, according to Phil. Pa. Masonry  
 experiments factor of safety for  $P_u = 1260$   $F = 13.3$  OK



SUBJECT City of Saratoga FILE NO. DE-1000  
Dam # 27 Mohawk (Serial # 275) ACC. NO. C3685  
See sheet # 1 SHEET 9  
 COMPUTER W. H. Murray BY 16, 19, 17 CHECKED BY \_\_\_\_\_

MADE IN CONNECTION WITH \_\_\_\_\_  
 REFERENCE Cons. Plan Acc. C-3685

Overfall Section Investigation of Stability Control

Investigation at horizontal plane at highest elevation of sub foundation below overfall section (See red cray on line on sketch sht # 4) but assuming that 4ft cut off wall built monolithic with do Neglecting ice pressure as before

Considering section 4ft. thick longitudinally, overtopped <sup>to depth of</sup> 4ft.

Assuming: joint in paving slab behind dam and

(Revised on sheet 23, assuming that cut off wall is 100% efficient)

Intensity of static pressure as maximum at intersection of paving slab with back of dam (Elev. 1047) and declines gradually along a line of sewage following the outline of the section and becoming flat at an assumed joint at the point of intersection of the slope of the apron extended, and the horizontal plane at the base of the dam (See red lines on sketch, sht. #)

Resistance to sliding due to paving in front of the dam has been neglected. Assuming that the sand or loam soil will exert pressures on the back of the section and the front of the cut off wall (4ft. thick) in accordance with the results obtained by the usual formulas taking weight of the loam as 100 lb/cu ft and natural slope of same as 2 on 3

	Area	Moment
(A) $4.92 + (11.2 + 7.7)$	$+35.7$	$8.8$
(B) $5.3 + 3.9$	$-0.175$	$9.2$
(C) $7.5 + 3.9$	$-0.215$	$11.4$
Net for section above El. 1047		$= +310$
(F) Trapezoid below natural surface and above top of cut off wall (see sketch on Sht. 4)	Area: $7.7 + 1.6 \times 5.1$	$+4.72$
Arm: $(2 \times 1.6) + 1.6 \times 7.7 - 7.7$		$6.72$
Moment:		$+324$
Masonry Resisting Moment		$+634$

SUBJECT City of Albany

FILE NO. 557-1-11

Dam 427 Mohawk (Serial # 275)

ACC. NO. C-3687

Sec. Sheet # 1

SHEET 11

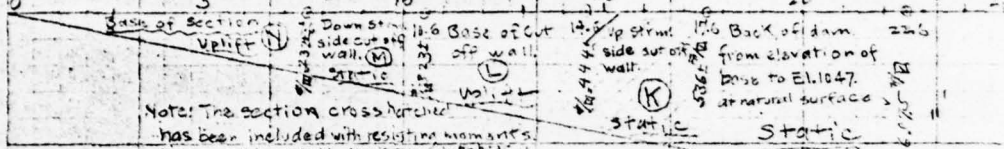
COMPUTER *W. H. ...* DATE *July 16, 1917* CHECKED BY

MADE IN CONNECTION WITH

REFERENCE *Comp's most Acc. C-3686*

Overall Section, Investigation of Stability Continued.  
 Assumptions as stated on sht. 9.)

$M_0 =$  Overturning Moments  
 Uplift and Static above natural surface  $\gamma H = 62.5 \times 11 = 687.5$



(J) Rectangular portion  
 (Total area used)

Force:  $536 \times 5' = 2,680$

Arm:  $\frac{5.0}{2} = 2.5$

Moment:  $6,700 \text{ ft}^{\#}$

Triangular portion  
 (Total area used)

Force:  $\frac{7.5 \times 5}{2} = 151.5$   
 $151.5 \times 5 = 757.5$

Arm:  $\frac{2}{3} \times 5 = 3.33$

Moment:  $1,263 \text{ ft}^{\#}$

(K) Rectangle:  
 (Area used)

Force:  $33 \times 4 \times \frac{2}{3} = 86 \text{ ft}^{\#}$

Arm:  $\frac{4 + 7.6}{2} = 9.6$

Moment:  $8,260 \text{ ft}^{\#}$

Triangle:  
 (Area used)

Force:  $\frac{1.2 \times 4}{2} = 2.4$   
 $2.4 \times \frac{2}{3} = 1.6$

Arm:  $7.6 + (\frac{2}{3} \times 4) = 10.27$

Moment:  $16,540 \text{ ft}^{\#}$

Overturning moments forward to sheet 12

Comp's Acc. C-3687

N.B. Excepting those portions of the above force diagram which act on the back of the dam and buttress (K), only 75% of the areas have been considered as effective.

SUBJECT: City of Albany FILE NO. 2500  
Dam 427 Mohawk (Serial = 275) ACC. NO. C-3686  
See page #1 SHEET 10  
 COMPUTER: W. M. ... JUL 6, 19 17 CHECKED BY: \_\_\_\_\_

MADE IN CONNECTION WITH \_\_\_\_\_  
 REFERENCE Curr's prev Acc. C-3685

Overfall Section - Investigation of Stability Continued			
(Assumptions as stated on Sht. 9)	Area	Arm	Moment FxL
Masonry resisting moment forward from Sht. 9			+634.0
(G) Section of 4ft cut off wall:			
Area: 4' x 3' =	12'±		
Arm: 11.6' - 2'		9.6'	
Moment:			+115.1
Total net masonry resisting moment (Area =)			749.1
M <sub>Masonry</sub> =	u	u	u
M <sub>Water</sub> = (Sht. 6) " water (Error carried)		140' x 749.1 =	104,700
Moment arm too short but usually neglected dryway			+ 2,250
(H) For resisting moment due to pressure of earth in back of dam, see computation on sht. 13-20 for insertion here. (See sheet #20)			
Frictional resistance =	830' (Sht. 20)		
Arm:		11.6'	
Moment:			9,630
(I) (Cutting pressure saps by passed by dam) Friction = 35 x 123.9 =	45.1' (Ref. C-3584)		
Arm and Moment:		7.6'	343
M <sub>Earth</sub> Static Resisting Pressure:			
(K) Rectangular portion: (2/3 of areas used)			
Force: 444' x 3 1/2' =	888		
Arm:		1.5'	
Moment:			1,332
Triangular Portion (2/3 area used)			
Force: 536 - 444 x 3 1/2' =	92		
Arm:	2/3	1.0'	
Moment:			92
M <sub>Earth</sub>			
Mr = Total of Resisting Moments			118,347

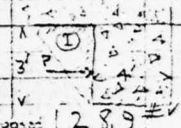
22-17-2500 (16-2961)  
 LCA 700  
 WBS

STATE OF NEW YORK - CONSERVATION COMMISSION  
 City of Johnston  
 Dam 427 Mohawk (Serial # 275)  
 COMPUTER: Steeny 4/17/17 CHECKED BY: \_\_\_\_\_  
 MADE IN CONNECTION WITH: \_\_\_\_\_  
 REFERENCE: \_\_\_\_\_  
 FILE NO. DM 427M  
 ACC. NO. C-3688  
 SHEET 12  
 COPY'S FROM ACC. C-3687

Overfall Section Investigation of Stability Continued

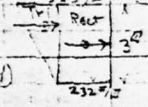
Assumptions as stated on Sht. 9  
Overturning Moments cont'd. Force Arm Moments / #  
 Total of overturning moments forward from Sht. 11 = 17,877

(E) Earth thrust on cut off wall  
 Force: 1289 #  
 (load which reduces moment arm)

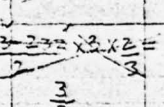


Arm: (Neglecting reduced length of moment arms which would result if a surimposed load is considered) 2.0 #  
 Moment: 258

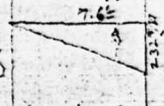
(M) Rectangles: (2/3 area used)  
 Force:  $232 \times 3 \times \frac{2}{3} = 464$  #  
 Arm:  $\frac{3}{2} = 1.5$  #  
 Moment: 696



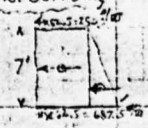
(N) Triangles: (2/3 area used)  
 Force:  $323 \times 2 \times \frac{2}{3} = 91$  #  
 Arm:  $\frac{3}{3} = 1.0$  #  
 Moment: 91



(N) Triangle: (2/3 Area used)  
 Force:  $232 \times 7.6 \times \frac{2}{3} = 588$  #  
 Arm:  $7.6 \times \frac{2}{3} = 5.17$  #  
 Moment: 3,000



(H) Above channel bottom; (Shts. 11 & 12) 2,370 #  
 Moment: 3,033



Overturning moments forward to Sht. 13 = 24,955 #  
 Copy's on Acc.

1-23-17-370 (16-2001)  
 Acc. 108  
 Form 906

STATE OF NEW YORK - CONSERVATION COMMISSION Dm 427M

SUBJECT City of Johnston  
 Dam #27 Mohawk (Serial #275)  
 (See Sheet 1)  
 COMPUTER W. Henry Jy 17, 1917 CHECKED BY  
 MADE IN CONNECTION WITH  
 REFERENCE Carr's Form No. C-3689  
 FILE NO. D-275  
 ACC. NO. C-3689  
 SHEET 13

Overall Section Investigation of Stability Continued  
 (Assumptions as stated on Sht. 9)  
 Overturning moments continued

Overturning moments	Force	Arm	Moment
Overturning moments from sheet #1			24,955#
Continued (Strength)			
Force: $(17 \times 2.5) \times 7 = 1,750$		3.5	6,125#
Arm: $\frac{7}{2}$			
Moment			6,125#
Increase			
Force: $\frac{4375}{2} \times 7 = 1,531$		2.35	3,565#
Arm: $\frac{7}{3}$			
Moment			3,565#
$M_o =$ Total of Overturning Moments			34,645#
$M_r = M_o =$ (From Sht. 10)			83,902#

Vertical Components			
Earth Friction (H) (Sht. 10)			+ 875#
Areas			
Masonry			
(A) 35.3 Cu. Ft. (Sht. 6)			
(P) 48.2 " " (" 9)			
(G) 12.0 " " (" 10)			
95.5 " " $\times 140$ / cu ft			+ 13,380#
Water (Sht. 6)			+ 326#
Total downward pressure			+ 14,581#
Uplift Forces			
(D) 86.1# (Sht. 11)			
(E) 588# (Sht. 12)			
$V_e =$ Net			- 1,449#
$U = M_r - M_o = 6,371$			13,132#
$I = 11.6 = 3.87$			(64.6) Inside wall

Carr's on Acc.

SUBJECT City of Mohawk  
Dam 427 Mohawk (Serial 4275)  
(See sheet 1)  
FILE NO.  
ACC. NO. C-3690  
SHEET 14

COMPUTER *ReNewing* July 17, 17 CHECKED BY  
MADE IN CONNECTION WITH  
REFERENCE Conn's Form Acc. G-3007

Cross Section - Investigation of Stability - Continued  
(Assumptions as stated on Sht. 9)

Sliding

$$F = \frac{V_e}{H_o} = \frac{13,132}{9,000} = 1.46$$

(Ref. Below)

Too high without using the resisting power of earth in computing on sheet 12 which would lower this value. Coefficient of Stability

$$f = \frac{H_o}{V_e} = 0.68 = .736$$

Concrete on sandy loam should not exceed .55 to .45. Coefficient on sheet 20 and 22 and recomputed with partial uplift on sat 23

Sub foundation loads

Section overlapped to depth of 4'  
Maximum

$$P_a = \frac{V_e}{L} \left[ 1 + 6 \frac{U - \frac{V_e}{L}}{L} \right] = \frac{13,132}{11.2} \left[ 1 + 6 \frac{1.25 - \frac{13,132}{11.2}}{11.2} \right] = 1,470$$

$P_b = 300$

Reservoir Empty

Maximum  
 $U = \frac{M_r - M_u}{V_e} = \frac{111,382}{14,255} = 7.84$

2.87 beyond middle of CR. Moment arm between Upstream from point assumed as 2.87

$$P_a = \frac{V_e}{L} \left[ 1 + 6 \frac{U - \frac{V_e}{L}}{L} \right] = \frac{14,255}{11.6} \left[ 1 + 6 \frac{7.84 - \frac{14,255}{11.6}}{11.6} \right] = 2,520$$

Fairly high but probably OK

Horizontal Component of Resultant (Used above)

Dist	+1,750	+1,531	(0.11)
Dist	+2,630	+373	(0.11)
Dist	+2,370	-	(0.12)
Dist	+888	-92	(0.10)
		-129	
		-464	
		-91	
	+7,689		
	+7,590		
	-684		
		-68	

$H_o = \text{Net Total} = 7,226$  as used above

M <sub>r</sub> (masonry) (Sht. 10)	+1,047,700		13,380
M <sub>u</sub> (masonry) (u 10, 12)	+9,630	-3,033	830
M <sub>u</sub> (earth) (u 10, 12)	+114,673	-255	45
	-3,291		
M <sub>r</sub> - M <sub>u</sub>	= 111,382		
		$V_e = 14,255$	(Used above)

DATE  
PAGE NO.

SUBJECT City of Johnstown  
Dam 427 Mohawk (Serial # 275)  
(See sheet # 1)

FILE NO.  
ACC. NO. C-3691  
SHEET 15

COMPUTER Conway Jy 17, 1917 CHECKED BY

MADE IN CONNECTION WITH

REFERENCE Com's Plan Acc. C-3691

Overfall Section - Investigation of Stability Cont'd

Assuming maximum probable flood equal to values computed from formula derived by Mr. McKim ( $Q = 600 A^2$ ) and recomputing stability of the section overtopped to the depth thus indicated.

Neglecting ice pressure

Assuming open joint at intersection of natural surface line with back of overfall section (Elev. 047)

Investigating section 1 ft. thick overtopped to depth as computed below

$Q = 600 \times (2.8'b)^2 = 1,314 c.f.s.$  (Ref - Sht. 3)  
Effective suppressed weir length (Ref Sht. 3)

Total length, end wall to end wall = 60 ft.  
20 ft. section = 19.33 ft.

Determining approximate depth on crest

2 ft. depth discharge of 572 c.f.s. (Fig. 2)

1 ft. " " " " 620 c.f.s. (Sht. 3)

Approximate Locus = Equation

Probable extreme limit?  
Probable mean value?  
Minimum limit?

Min. height 3.4'  
Prob. Max. " 3.5'  
Top of spillway 3.55 ft

572 c.f.s.  
500 c.f.s.

$b$  (For two end sections) =  $19.33 - \left( \frac{2.7 \times .35}{1.01 \times 3.6} \right) = 19.06'$  (prob.)

$b$  (For middle section) =  $19.33 - \left( \frac{1.5 \times .36}{1.10 \times 3.6} \right) = 18.79'$  (prob.)

Approximate effective length = 56.91'

Discharge by E.P.C. curve =  $56.91 \times 2.4 \times 3.5 = 1,365 c.f.s.$

Spillway Channel Capacity with projecting stance in bottom (4 ft) Using n = .03 as available

$v = 60.67 \times 2.33 \times .03 = 4.25$  ft/sec }  $A = 5' \times 20' = 100'$  }  $p = 3.33$  }  $Q = 100 \times 7.25 = 725 c.f.s.$   
 $Q = 100 \times 7.25$  }  $p = 30'$  }  $Q = 100 \times 7.25 = 725 c.f.s.$

Curve on Acc.

STATE OF NEW YORK - CONSERVATION COMMISSION

Dm427M

SUBJECT City of Mohawk FILE NO. \_\_\_\_\_

Dam 427 Mohawk (Serial No 275) ACC. NO. C-3692

(See sheet #1.) SHEET 16

COMPUTER J. Harvey 4/18/1977 CHECKED BY \_\_\_\_\_

MADE IN CONNECTION WITH \_\_\_\_\_

REFERENCE Overfall Section - Investigation of Stability Contd. (Assumptions as stated on Sheet 15)

Depth on Crest - Determination of Continued

$b_1$  (For two end sections) =  $12.33 - \left( \frac{2.5 \times 3.55}{12} \right) = 19.06'$

$b_1$  (For middle section) =  $14.33 - \left( \frac{5.325 \times 3.55}{12} \right) = 13.80'$

Approximate total effective crest length =  $56.9'$

Resulting discharge by E.S. Cullings curve =  $23.5' \times 1.33 = 31.15$

This value plots on curve (Sheet 15) OK.

Reading from such curve 131 ft of 3 =  $3.51'$

3.51' of water on crest will therefore be used.

Resisting Moments:

Masonry in Ft. Lbs. (Sht. 5) Area  $\square$  Arm Ft. Ft. Lbs.  $\times$  12

Water

(B) Neglected

(C) From Sht. 5 +1.5

(D) Prism on crest (Assumed as equal to 1/2 the section of form 12.5' x 3.51')

Arm: 4.39'  $\times$  6.7

Moment: +30.25

(E) Prism of water sliding on apron neglected.

$M_{r(water)} = 31.75' \times 2.5' \times 62.5' / \text{cu ft} = 1,983'$

$M_r = \text{Total Resisting Moment} = +26,283'$

Overturning Moments:

Static  $\times$  26 - Tables C-358(1)

Depth	Force	Moment about Base	Mom. about Top
10.5'	+3,451.90	+12,093.30	+24,186.70
3.51'	-3,852.97	-450.55	-901.10
F = 3,066.27			23,285.60
F x 10.5'			22,250.00
$M_o(\text{static}) =$			8,944'



STATE OF NEW YORK - CONSERVATION COMMISSION DM 427M

SUBJECT: Canal Wall (Serial 271) (Sheet 17) FILE NO. C-3693  
 ACC. NO. C-3693 SHEET 17  
 COMPUTER: 4/19/17 CHECKED BY: 10  
 MADE IN CONNECTION WITH: \_\_\_\_\_  
 REFERENCE: Canal Wall Acc. C-3693

Overall Section Investigation of Stability - Continued (Assumptions as stated on sheet 15)			
<u>Overturning Moments - Continued</u>			
Masonry	Forward from	Sht 16	8,954 <sup>#</sup>
Uplifts (No. 3-3581 - Shts 65 & 67)			
Force: $(65.6 \times 7.7 \times \frac{2}{3}) = 1670$			
Arm:	(Sht. 6)		5.13 <sup>#</sup>
Multiplying			8,659 <sup>#</sup>
$M_o = \text{Total Overturning Moment}$			<u>17,624<sup>#</sup></u>
$M_r - M_o = \text{Net Resisting Moment (Sht. 16)}$			<u>8,659<sup>#</sup></u>
<u>Vertical Components</u>			
Masonry (Sht. 6)	$35.3 \times 145$	Weight	4,950 <sup>#</sup>
Water (Sht. 5)	$0.215$		
(Sht. 16)	$4.39$	$4.6 \text{ cu ft @ } 62.5$	<u>288<sup>#</sup></u>
Total downward pressure			<u>= +5,238<sup>#</sup></u>
Uplift as above			<u>= 1,690<sup>#</sup></u>
$V_e =$			<u>= 3,548<sup>#</sup></u>
$U = M_r - M_o = 2.44$			(Arm of $V_e$ )
$V_e = 7.7$			(5.1% outside middle third)
$\frac{U}{3} = \frac{2.44}{3} = 0.81$			
$F = \frac{V_e}{H_o} = \frac{3,548}{3,067} = 1.16$			Coefficient of Stability
$f = \frac{H_o}{V_e} = \frac{3,067}{3,548} = 0.865$			Coefficient of Friction should not exceed .70 or .75
$P_o = \frac{V_e(1-f)}{2} = \frac{3,548(1-0.865)}{2} = 238$			
Neglecting uplift should be O.K. as on sheet 7.			

3-17-200 (10-2001)  
 Loc. 100  
 Form 710

STATE OF NEW YORK - CONSERVATION COMMISSION Dm 427M

SUBJECT Dam 427 Mohawk (Serial No. 275) FILE NO. \_\_\_\_\_  
 (See sheet-1) ACC. NO. C-3694  
 SHEET 18

COMPUTER W. J. Murray 12/4/17 CHECKED BY \_\_\_\_\_

MADE IN CONNECTION WITH \_\_\_\_\_  
 REFERENCE Form Sht 10, Acc 3694 Dm's map Acc. Sht 10 C-3694

Insert of computation for which there was insufficient space on Sheet 10 whole result was embodied

Head Resisting moment due to pressure of earth on back of dam.  
 Assumptions involved:

- (1) Weight of damp sand or loam 100 lbs/cu. ft.
- (2) Angle of repose of same material about 33° 41'
- (3) Joints assumed in upstream paving slab.
- (4) Load of concrete paving slab and 11 ft. of water superimposed upon prism of earth in pressure.
- (5) Coefficient of friction of .35, between the damp sand or loam and back of dam.
- (6) That the theory of earth pressure gives results sufficiently accurate.

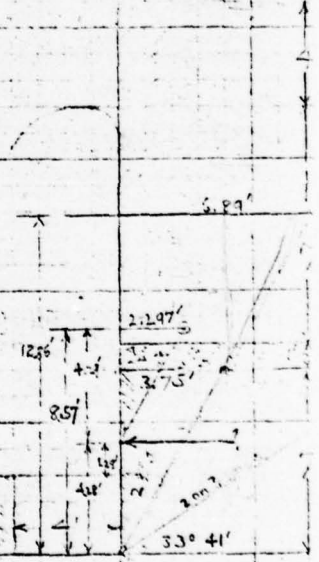
Angle of maximum pressure:

$$90^\circ - 33^\circ 41' = 56^\circ 19'$$

$$28^\circ 9' \frac{1}{2}$$

$$\tan 28^\circ 9' = .535 = \frac{x}{7}$$

$$x = 3.745$$



Superimposed Loads:

$$3.75' \times 11' \times 140 \#/cu. ft. = 52.6 \# \text{ Concrete}$$

$$3.75' \times 11' \times 62.5 \#/cu. ft. = 257.0 \# \text{ Water}$$

Total Load 3,126 #

Equivalent prism of Earth:

$$\frac{3126}{100 \#/cu. ft.} = 31.06 \text{ Cu. Ft.}$$

Areas proportional to square of same dimension.

$$\frac{2 \times 3.75}{44.16} = \frac{7^2}{y^2} \quad y^2 = \frac{49 \times 44.16}{13.1} = 165.5 \pm$$

$$y = 12.86 \pm \quad \frac{12.86}{3} = 4.287 \pm \quad \frac{12.86 \times 12.86}{3} = 55.4 \pm$$

$$\frac{3.75' \times 6.89'}{7' \times 12.86'} = \frac{2.297'}{4.29'}$$

Moment arm about base of dam =  $4.287 - 3' = 1.287'$  (Use on Sht 11) Dm's map Acc. Sht 11 C-3694

SUBJECT: 477 Mohawk Section FILE NO. \_\_\_\_\_  
 COMPUTER: \_\_\_\_\_ 19 \_\_\_\_\_ CHECKED BY: \_\_\_\_\_  
 MADE IN CONNECTION WITH: \_\_\_\_\_  
 REFERENCE: on Sht. 18, Acc. 367 ACC. NO. C-3695  
 SHEET 19

Insert continued from sheet 18

Moments of Prism of Maximum Pressure

About back of section of dam (Sheet 18)

	Area	Arm	Moment
Rectangle:			
Area: $3.75 \times 8.22$	31.1		
Arm: $3.75 \div 2$			58.2
Mom.:			
Triangle:			
Area: $\frac{3.75}{2} \times 7$	13.1		
Arm: $3.75 \div 3$			16.4
Mom.:			74.2
Rectangle:			
Area: $3.75 \times 7$	26.3		
Arm: $\frac{3.75}{2} \times 7$			11.14
Mom.:			346.1
Triangle:			
Area: $\frac{1}{2} \times 7$	3.5		
Arm: $\frac{1}{3} \times 7$			4.6
Moment:			61.2
Totals	44.2		407.2
Resulting Moment arm:			9.22

REVERSED

By data added on Sheet 18

3694

Observed Acc. C-3694

Cur's on loc.

SUBJECT Dam 427 Mohawk (Serial No. 275) FILE NO. 275

ACC. NO. C-3696 SHEET 20

COMPUTER ... 19 7 CHECKED BY (Sheet # 19)

MADE IN CONNECTION WITH ... SHEET 20

REFERENCE (Heads on Sht. 10, Acc 3696) CONT'S FROM ACC. C-3695

Inset - continued from Sheet # 19:

Point of application of resultant (Sketch on sht. 18)

Total Weight of Earth and Superimposed Loads:

$$3,106 \# + \left[ \frac{1310.75 \# \cdot 15725 \cdot 37 \pm 5}{2} \right] = 4,417 \#$$

Horizontal Pressure of earth and superimposed loads:

$$\frac{P}{W} = \frac{3.75}{7} = \frac{2,370 \#}{4,420 \#}$$

$$P = 2,370 \# \text{ (used on Sht. 12)}$$

Frictional Resistance between earth and dam  
Assuming coefficient of friction = .35 = f

$$f = \frac{F_m}{P} = .35 = \frac{830 \#}{2,370 \#}$$

$$F_m = 830 \# \text{ Resistance (used on Sht. 12)}$$

Inset of computation under title "Sliding" on sheet #

$$f_1 = \frac{H_e}{V_e} = .45 = \frac{5,915 \#}{13,132 \#} \dots H_e = H_u = 3,000 \dots V_e = 3,000 \dots$$

(See sheet 20) showing resisting value of  $\frac{1,031 \#}{2,060 \#}$   
Determining whether earth would resist such force:

Exposed area = 3' x 1'  $\therefore$  Unit intensity =  $2,060 = 687 \#$

Assuming coefficient of friction of wet loams = .50

The horizontal movement of  $2 \times 687 = 1374 \#$  of subsoil would resist  $687 \#$

This small amount would undoubtedly be safely resisted by the pressure turned against the bottom of the dam by the cut off wall 4" thick, or by resistance of the earth below and at toe of the dam against sliding, or bulging.  $\tan \angle \text{of repose} = .668 = \frac{4.41}{6.61} = \frac{4.41 \# \text{ Vertical}}{6.61 \# \text{ Horizontal}}$

Point of application of resultant  $3 \times 4.41 = 13.23 \# \text{ Vertical}$



SUBJECT Dam #27 Mohawk (Sec of No. 275) FILE NO. \_\_\_\_\_  
 ACC. NO. C-3698 SHEET 22

COMPUTER W. Slattery 22 July 1917 CHECKED BY \_\_\_\_\_

MADE IN CONNECTION WITH \_\_\_\_\_

REFERENCE (Insert to follow Sht 14, continued) Comm's plan No. 2017  
Effect of Earth Pressure upon Computed Values, continued  
 (Sketch on Sht. 21).

Areas proportional to squares of same dimension

$$\frac{1}{2} \times 1.66^2 = 3^2 \quad \gamma_1^2 = 9 \times 21.66 = 194.94$$

$$\frac{(\frac{3}{2} \times 1.66) + 19.25^2}{2 \times 1.66^2} = \gamma_1^2 \quad \gamma_1 = 9^{\frac{1}{2}}$$

$$\frac{1.66'}{3} = \frac{4.815}{9} = \frac{1,160\#}{2,167\#} \quad \frac{4.815}{3} = 1.605'$$

Loads:  
 (Sht. 21) Superimposed = 1,925#  
 Earth proper = 242#  
 Total = 2,167#

Horizontal pressure = 1,160# (Ref. above)  $\frac{1}{2} \times 1.66 \times 1.66 \times 1.66$   
 Used on Sht. 14 = 129#  
 To resist sliding = 1,031# (Used on Sht. 20)

Retaining Walls at ends of Overfall Section:

Assuming open joint at natural surface behind overfall section (Bl. 1047)

$P = 2,062.8 \text{ lbs.}$  (Ref. - C 584)

Investigating section only 1 ft. thick. (Ignoring static pressure) Fig. 1057

$M_o = 8,251.2$  (Ref. - C 357)

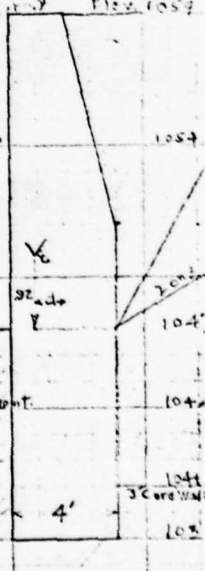
$M_r \left\{ \begin{aligned} 4 \times \frac{1}{2} \times 21.66 \times 21.66 &= +9571 \\ (\frac{3}{2} \times 8) \times \frac{1}{3} &= -32 \end{aligned} \right.$

$C = \frac{8,975}{724\#} = 140\# \times 64'$

$V_e = 725\# = (48 + 9) \times 140\#$

$U = 0.92 \quad \frac{1}{3} = \frac{1}{3} = 1.33$  Point of application of resultant falls 33.3% outside middle of wall without considering static pressure

$P_o = \frac{V_e}{L} - \frac{6V_e d}{L^2} = \frac{725}{4} - \frac{6 \times 725 \times 6}{16} = 172\#/\text{sq ft}$



Note: While this wall stands 12' high for a horizontal distance of only 4' ft. still it must bear more, less static pressure in addition to earth pressure and its location is such that should a flood occur immediately after its failure, the up stream side of the embankment would be badly eroded which might result in the undermining of the core wall and failure of the entire structure.

SUBJECT Dam # 27 Mahanick (Serial No. 270) FILE NO. \_\_\_\_\_  
 ACC. NO. C-3699 SHEET 23

COMPUTER 24 1917 CHECKED BY \_\_\_\_\_

MADE IN CONNECTION WITH \_\_\_\_\_

REFERENCE Overfall Section Investigation of stability (Revision of data on sheets 9-14) Comm's new Acc. C-3699

Overfall Section Investigation of stability (Revision of data on sheets 9-14)

Same assumptions as stated on sheet 9 except that it is further assumed that static pressure acts only upon the upstream face of the dam and upon the base of the outfall wall. This seems warranted if a row of suitable sheet piling is carefully driven to a good depth, and well bonded into the concrete for the entire width of the overfall section and into the embankment at each end.

Resisting Moments:

Mr (masonry) (Sht. 10) 104,700 ✓  
 Mr (water) ( " 10) 2,250 ✓  
 Mr (static) (Combined with Magnetic below)  
 Mr (earth) [H(Sht. 10)] + [L(Sht. 22)] = (9.630 + 3.170) 13,600 ✓

$M_r = 120,550$

Overturing Moments:

Ⓐ (Sht. 12) → uplift under the base of dam 3,033 ✓  
 Ⓑ (Sht. 21-2) Arm so short moment seems negligible 00

Ⓒ Static: (Ref. C-3581) Total Head 19 Ft.

Pressure	Mom. about base	Mom. about V.S.
19 Ft. 11,281	71,448	x 2 = 142,896 ✓
9 Ft. -500	6667	x 2 = 13,334 ✓
		Diff = 141,562 ✓
		= 234,600 ✓

Lower 15 Ft. 10,781 x 19' = 204,939  
 $63,238 \div 1.732 = 36,500$   
 $2.85 \times 10,781 = 30,720$

Ⓓ (revised) Base stresses, 344 2 Ft.

$1,187.5 \times 2 \times (11.6 - \frac{2}{2}) = 25,200$

$M_o = 58,933$

$M_r - M_o = 61,617$

Vertical Components:

(K+I) (Shts 10 and 22) (830 + 521.5) = 1352 ✓  
 Masonry: (Sht 13) + 13,380 ✓  
 Water: (Sht 13) + 326 ✓  
 $+ 15,058$   
 Uplift (above)  $2 \times 11 \times 7.5 = 2,275$  ✓

$V_E = \text{Net } 12,683$

SUBJECT City of Van Hook FILE NO. 12-111  
Dam #27 Mohawk (Serial No. 275) ACC. NO. C-3700  
 SHEET 24  
 COMPUTER Jy 24, 1917 CHECKED BY \_\_\_\_\_  
 MADE IN CONNECTION WITH \_\_\_\_\_  
 REFERENCE \_\_\_\_\_  
 Conn's show Acc. C-3697

Overfall Section - Investigation of Stability  
 Assumptions as stated on Sht. 23.

$$U = \frac{M_r - M_o}{V_e} = \frac{4.87' \cdot \frac{1}{3} = 1.62}{3} = 3.87\frac{1}{2}$$

Point of application of resultant falls 26% within the middle  $\frac{1}{3}$ . According to the usual practice, the dam is therefore said to be safe against overturning about this plane.

Horizontal Components:

Static (Sheet 23)	+10,781#
Earth (H, Sht. 12)	+2,370#
Earth (I, Sht. 22)	+13,151#
	- 12,160#

$H_o = 11,991\#$

$F = \frac{V_e}{H_o} = 1.05 \pm$  Coefficient of stability very low.

$f = \frac{H_o}{V_e} = .945$  Coefficient of friction should not exceed .50 to .60

$f_1 = .60 = \frac{H_{o1}}{V_e}$   $H_{o1} = 7,600\#$   
 $H_o - H_{o1} = 4,391\#$

Note: Excess is due to considering static pressure as increasing below surface of ground. In true resulting intensity of pressure per unit length on down stream side of cut off wall =  $\frac{4,391}{3} = 1,464 \frac{\#}{ft}$

This would cause an upward pressure on bottom of dam of about (Sht 22)  $3,290\#$

$\frac{3}{4.49'} = \frac{3,290\#}{4,391\#}$  Point of application unknown

$\frac{3,290}{65} = 50.6$  Co. Fr. of material would resist it but probably not available.



29-17-2300 (16-2941)  
 CO. 100  
 MAR 920

STATE OF NEW YORK - CONSERVATION COMMISSION Dm 427M

SUBJECT City of Jannstown FILE NO.  
 (See Sheet #1) ACC. NO. C-3770  
 Ser # 275 SHEET 25  
 Filed by ppg da

COMPUTER J. Henry 2724 1017 CHECKED BY \_\_\_\_\_

MADE IN CONNECTION WITH \_\_\_\_\_ Dist's Plan Acc. 3700

REFERENCE Overfall Section - Investigation of stability

Assuming horizontal joint at base of dam (EI: 1042±)

Other assumptions as on Sht. 9 except that joint is to be assumed between cut off wall and base of dam

Crest overtopped to depth of only 3.51' as per flood by McKim formula (Ret - Sht. 16)

Material behind section has effective weight of only  $100 - 62.5 = 37.5$  per cubic foot =  $w$

Resisting Moments Ft. Lbs. Sq. Ft x Ft

$M_r(\text{Masonry, above EI: 1042}) (\text{ref. Sht. 9}) 634 \times 140 = 88,750$

$M_r(\text{water})$

(B) Neglected.

(C) (Sht. 5)  $0.215 \times [7.5 + \frac{11.5}{2}] \times 62.5 = 153$

(D) (Sht. 14)  $4.39 \times [10.5 + \frac{14.5}{2}] \times 62.5 = 2,850$

$M_r(\text{Earth}) (\text{Sht. 26}) .35 \times 174.2 \times 11.6 = 708$

$M_r = +92461$

$M_o = (\text{From Sht. 26}) -45,468$

$M_r - M_o = 46,993$

Vertical Components:

Masonry =  $83.5 \times 140 = +11,660$  (Sht. 15)

Water (Sht. 17) =  $+288$

$+11,948$

Uplift: (Sht. 11) =  $-1,022$

(Sht. 12) =  $-587$

$-1,610$

$V_e = 10,338$

$U = \frac{M_r - M_o}{V_e} = 4.54'$

$\frac{U}{V_e} = \frac{4.54}{10.338} = 0.44$  (Point of application of resultant falls 17.3% within middle third)

Dist's on Acc.

SUBJECT City of Johnstown  
Proposed Cath Center Cr. Dam  
(See Sheet #1)

FILE NO. 122  
ACC. NO. C-3769  
SHEET 26

COMPUTER J. J. J. 27 Jy 1917 CHECKED BY

MADE IN CONNECTION WITH

Comm's Plan Acc. 3770

Overfall Section - Investigation of stability  
(Assumptions as on Sht. #25)

Overturning Moments

$M_{0(Static)} (Ref - Sht. 16)$   
 $\frac{8964 \#}{3,067 \#} = 2,922 \text{ ft moment arm}$   
 $+ 5.0' = (8' - 3')$   
 $7.9 = 2' \text{ Corrected arm}$   
 $24,300 \#$

$M_{0(EARTH)}$  Weight of superimposed slabs  $= 140' \times 0.25' = 77.5 \#/sq \text{ ft}$

$P = (\frac{1}{2}wh + 2vh) \tan^2(45 - \frac{1}{2}\phi)$  (Ref. Am. C. E. PKT. BK p58)

$= (\frac{37.5 \times 16}{2} + 2 \times 0) \tan^2(45 - \frac{1}{2} \times 30) = 174.3$

$Arm = \frac{33sh}{wh + 2v} = \frac{1.33(37.5 \times 4 + 2 \times 77.5)}{37.5 + 4} = 1.67'$

(J) (Sht. 11)  $291 \#$

(L) (Sht. 11)  $7,963 \#$

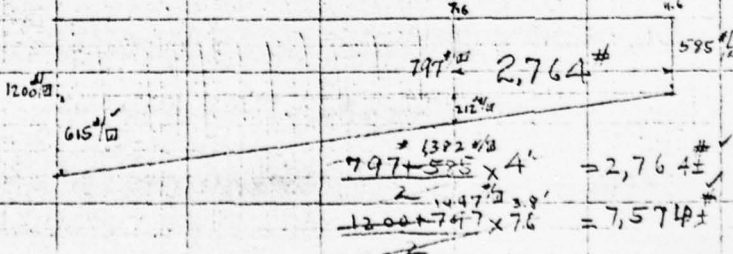
(N) (Sht. 12)  $9,914 \#$

$M_0 = (\text{Used on Sht. 25})$   $3,000 \#$   
 $45,468 \#$

Subfoundation Loading

$P_a = \frac{V_6(1+6e)}{L} = \frac{10,333}{11.6} [1 + \frac{6 \times 0.67}{11.6}] = 1,200 \#/sq \text{ ft}$

$P_b = 895 \times 1 - \frac{0.63 \times 1,200}{11.6} = 585 \#/sq \text{ ft}$



23-17-3500 (16-2961)  
 CE. 106  
 AM WTB

STATE OF NEW YORK - CONSERVATION COMMISSION Lm 427M  
 SUBJECT City of Johnstown FILE NO. 2008-100  
Proposed Clark Center Cr. Dam ACC. NO. C-3768  
 (See Sheet 1.) 2008-100 SHEET 27  
 COMPUTER Frederick 29 July 1917 CHECKED BY Filed by p. 10  
 MADE IN CONNECTION WITH Cur's mon. Acc. C-3769

REFERENCE  
Overflow Section - Investigation of Stability  
(Assumptions as on Sht. 25)

Sliding Resistance (Ref - Sht. 26)

Load on material of cut off wall = 2,764#  
 Coefficient of friction: concrete = .65  
 Resistance offered = 1,800#  
 Load on material of sub foundation = 7,574#  
 Coefficient of friction, concrete on wet clay = .35  
 Resistance offered = 2,650#  
 Total Resistance (INSUFFICIENT) = 4,450#

Horizontal Components (Neglecting back water and apron seepage)

Static (Sht. 16) 3,067#  
 " (Sht. 11)  $\frac{2}{3} \times 3,059$  2,040#  
 Earth (Sht. 26) 174#

Computed after making report. 4/2/16  
 Total Sliding Force 5,281#  
 Unresisted according to above assumptions 831#  
 (The movement of 17 cu. ft. of earth would probably resist this.)  
 If sand subsoil will take 10% of 7,574# more = 757#  
 Yet unresisted. PROBABLY SAFE = 74#

REVISION Assume that cut off wall is to be built monolithic and well bonded with dam.

Earth pressure on back, revised (h = 7 ft.)

$P = \frac{1}{2} (wh^2 + uh) \tan^2(45^\circ - \frac{1}{2}\phi)$  (Sht. 26)  
 $= \frac{1}{2} (-419 \times 49 + 7 \times 77.5) \tan^2(45^\circ - \frac{33^\circ}{2}) = 419$   
 Amount used heretofore = 174#  
 Increase (Forward to Sht. 26) = 245#

Cur's mon. Acc. - 3769

STATE OF NEW YORK - CONSERVATION COMMISSION Lm 427M  
 SUBJECT City of Johnstown FILE NO.                       
 (See Sheet #1.) ACC. NO. C-3767  
 COMPUTER V. Henry Jy 27 1917 CHECKED BY                      SHEET 28  
 MADE IN CONNECTION WITH                      CONV. FROM ACC. C-3767

REFERENCE CONV. FROM ACC. C-3767

Overfall Section - Investigation of Stability  
 (Assumptions as on shts. 25 and 27)

Sliding Resistance (Continued from Sht. 27)  
 (From Sht. 27) Forward + 245#  
 (K) Static at back of cut off wall (Sht. 10) + 980#

Total increase in horizontal component = + 1,225#

Opposing Factors:  
 (G)  $3 \times 4 \times 140 \times \frac{1.680}{12} \times .35 = -588#$  756  
 (P)  $5.5 \times 140 \times \frac{.772}{12} \times .35 = -270#$  347  
 (I)  $\frac{19,332 \times 11.6}{11.6} = 1,037#$  /lin.ft (superimposed on subfoundation)

Note: The use of the average load seems on the side of safety as the prism of maximum pressure lies entirely upstream from the center of the base (Ref- Sht. 21) and the point of application of the vertical component of all forces was shown to intersect the base on the upstream side of the center (Ref- Von Sht. 13).

$$P = \frac{(wh^2 + vh) \tan^2(45^\circ \pm \phi)}{2} \quad (\text{Ref Sht. 26})$$

$$= \frac{(37.5 \times 9 + 1,037 \times 3) \times .286}{2} = 438.5# \quad = -938#$$

Sliding of section above base on wet clay =  $.35 \times 10,338#$  (Ref Sht. 25) = -3,620# 465

Total resistance of active forces = 5,416#

H<sub>0</sub> + 1,225# (Ref above and on Sht. 27) = 6,506#

Unresisted by active forces = 1,090#/ft longitudinal

Note: Since coefficient of friction of only .35 has been used above, and sliding on wet loam with coefficient of .50 is now to be assumed, the difference, or  $.15 \times 7,574#$  may be deducted before considering quantity of earth which must be moved, before failure would occur. (Ref- Sht. 27)

N.B.  $.15 \times 7,574 = 1,131#$   
 Dam therefore seems SAFE AGAINST SLIDING if cut off wall has not been                     

Computed after making written report but I advised Mr. McK and Mr. Perkins of this result.

STATE OF NEW YORK - CONSERVATION COMMISSION  
 SUBJECT: City of Johnstown  
Proposed Cork Center Cr Dam  
 (See Sheet 1)  
 COMPUTER: J. Deary 28 July 1917  
 CHECKED BY: \_\_\_\_\_  
 MADE IN CONNECTION WITH: \_\_\_\_\_  
 FILE NO: \_\_\_\_\_  
 ACC. NO: C-3765  
 SHEET 29  
 Filed by: \_\_\_\_\_

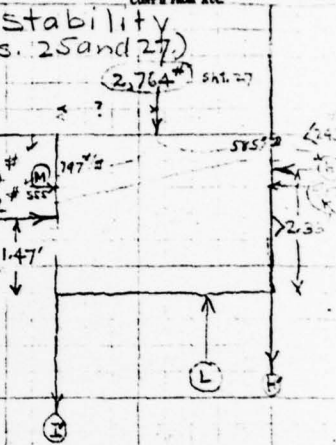
REFERENCE  
Overfall Section - Investigation of stability  
 (Assumptions as on Shts. 25 and 27)  
Cut Off Wall - Stresses

Possible tension due to earth friction

Coefficient may equal .60  
 $H = 245 \# (Sht. 27) \times 60 = 147.0 \#$   
 $V = 6037 \# ( " 28) \times 60 = 622.2 \#$

(I) (Ref. Am. C. E. Pratt, BK. p. 583)

Arm =  $\frac{.33h(wh+2v)}{wh+2v}$   
 $\frac{.33 \times 27(11.25 \times 3 + 2 \times 1,037)}{11.25 \times 3 + 2 \times 1,037} = 1.47$



Note: Probably not worth while to analyze so many forces

The wall is employed merely to resist sliding of the section

Suppose (as an extreme case) that all such resistance were to be exerted as a tangential force about its base, then - (Ref Sht 27)

$(?) 4,450 \# \times 3 \times 12^{36} = 160,000 \# = bd^2f =$   
 $= \frac{12 \times 4 \times f}{6}$

$f = \frac{160,000 \#}{4,608} = 34.7 \# / 3$  Roughly Approximate only.

Conclusions:

It would seem reasonable to require that such cut off wall be reinforced with steel to carry all tension

$3/8$  bars  $4 \frac{3}{4}$  cts would seem O.K. (Roughly approximate only.)

$60 \times 12^3 = 180$

SUBJECT City of Johnstown, FILE NO. D-19-11  
Proposed Clark Center Creek Dam ACC. NO. M5590  
SER. # 275 SHEET 30

COMPUTER J.H. Doe 10.18 CHECKED BY \_\_\_\_\_  
MADE IN CONNECTION WITH Dec. 23, 1918 and Dec. 13, 1918 letters by Jas. P. Well, Engineer, Dec. 27, 1918  
REPORT OF INSPECTION BY our Mr. McKim, as instructed by Div. Engrs. CON'D FROM ACC. C-3765

Equalization Effect of Reservoir as left when work was stopped.  
Capacity of two 24" outlet pipes:

Note: The plans, (Acc. 8058) show but one such 24" pipe with outlet end far below the dam, and at end of the spillway channel, with elevation about 998±

The letters and reports do not state whether the two 24" pipes have their outlets at the same elevation, or at what elevation their ends have been left open

Assumptions:

- ① Gross head 20 ft. as indicated by Mr. McKim's report;
- ② That the ends of both pipes are at the same elevation, and that the effective head upon both will be the same;
- ③ That length of such pipes is about 230 ft. which is the approximate base width of the embankment as shown by the plans (Acc. 8058) for the elevation at which such pipes appear to rest;
- ④ That loss of head from reservoir to inside of pipe is  $\frac{75\%}{2}$

Limits:

Maximum:  $Q_1 = 2A V_1 = 2(A \cdot 7 \cdot 2 \cdot 9h) = 28.02 A \sqrt{h}$   
 $= 8.02 (2 \times 3.1416) \cdot 4.6t$   
 $= 225 \text{ c.f.s.} = 79.5 \text{ c.f.s. / sq. mi.}$   
 (Watershed = 226 sq. mi.)

$V_1 = \frac{Q_1}{2A} = \frac{225}{6.28} = 35.9 \text{ ft/s.}$  (Neglecting losses at head due to entrance friction, etc.)

Try Chezy formula and a value of .011 for "n" (Ref. 6128)

$r = \frac{A}{P} = \frac{3.142}{6.283} = .5$  }  $S = \frac{20t}{230} = .087$

$V_2 = C_2 \sqrt{r S} = 127.6 \sqrt{.5 \cdot .087} = 266 \text{ ft.}$  }  $C = 127.6$   
 $Q_2 = C_2 A \sqrt{r S} = 6.28 \times 230 \times 266 = 3,800,000 \text{ c.f.s.}$  (Correct on Acc. M-111)

$v = \sqrt{\frac{14.14}{2 \times 2.3026}}$

122.8  
132.4

SUBJECT City of Johnstown, N.Y. FILE NO. Emt 27M  
Proposed Gork Center Cr. Dam ACC. NO. M5591  
 COMPUTER Jan 3, 1970 CHECKED BY See Sht. 30 SHEET 31

MADE IN CONNECTION WITH \_\_\_\_\_ DATE'S FROM ACC. M55-70

REFERENCE  
Equalization Effect of Reservoir as work was stopped:  
Capacity of Outlet works, Contd.

(Assumptions as on Sht. 30)

Limits, Continued:

Minimum:

Try Chezy formula and a value of .015 for  $n$  (Ref. 170)

$$v_3 = C \sqrt{RS} = 86 \times .208 = 17.9 \text{ ft/sec.}$$

$$Q_3 = 2(Av_3) = 6.28 \times 17.9 = 112.5 \pm \text{c.f.s.} \approx 3.3 \text{ cfs/sq. mi.}$$

Try  $v_4 = 23.3 \text{ ft/sec.}$

$$\frac{.75 v^2}{2g} = \frac{.75 \times 543}{64.32} = 6.33 \text{ Loss at entrance or in rack, Assumption #3}$$

$$.5Q_4 = A v_4 = 3.142 \times 23.3 = 73.2 \pm \text{c.f.s.}$$

Day  $C = 130$  for Hazen Williams formula

$$\frac{73.2 \text{ c.f.s.}}{1.3} = 56.4 \text{ c.f.s.}$$

$5.7 \text{ ft}/100'$  due to velocity and pipe friction (R=1012)

or  $13.1 \text{ ft.}$  loss of head in 230' by assumption #3.

$$13.1 + 6.3 = 19.4 \pm \text{ft.} \left\{ \begin{array}{l} \text{Total head loss} \\ \text{to discharge } 73.2 \text{ c.f.s.} \end{array} \right.$$

Therefore one 24" pipe would discharge  $75 \pm \text{c.f.s.}$

The two would therefore discharge  $150 \pm \text{c.f.s.}$

or the equivalent of  $52.5 \pm \text{c.f.s.}/\text{sq. mi.}$

provided that the rack and pipe can be kept flowing free and it is not clear by what means the racks are to be cleaned.

Form NY-66  
6-10-1-3000  
(10-10-59)  
Am. 100

STATE OF NEW YORK - CONSERVATION COMMISSION

SUBJECT City of Johnstown, N.Y., FILE NO.             
Proposed Gork Center Cr. Dam ACC. NO. M5592  
Series 275 SHEET 32

COMPUTER R. E. Barhoust - M. Duffy IS CHECKED BY H. E. Barhoust M.D. 19  
 MADE IN CONNECTION WITH See Sht. 30

REFERENCE Conr's from Acc. M5591

Equilization Effect of Reservoir (with dam completed to Elev. 1043)

Area + Capacity Curve Data (Used on Acc. M5597)

Planimetered Areas from Acc. 3056

						Area	Sq. Ft.
Elev. 1031							
40072	8672		38700	8595			
31400	8648	8660	30205	8586	8595	5625	271,000
22752			21617				
Elev. 1036							
37220	11649		31495	11583			
25571	11654	11654	9912	11563	11573	11,613	35,000
13912			48349				
Elev. 1041							
48163	17108		49479	17242			
31055	17160		32237	17227			
13895			15009				
		17154			17215	17,125	539,000
41981			7615	17181			
24810	17171		40434	17210			
			23224				
24791	17176						
7615							
Elev. 1046							
4410	34240		13734	34347			
29170	34181	34210	27337	34272	34310	34,260	1,976,000
35789			45715				
Elev. 1051							
32560	48858		22775	48850			
39702		48875	23925	48850	48850	48,822	1,532,000
			25075				
39625	48893						
40732							
Elev. 1054							
33179	58574						
24605	58585	58580	16820	58403			
33983			7617				
					58402	58,494	1,825,000
			7643	58413			
			19230				
Planimeter tested on Plat 500' x 500' Area = 314,000 sq ft. 1 Plot, Unit = 300,000 7556 = 31,4117 24922 9562 9558 4560 9551 7554 7556 75360 9555 7618 9552 7554 5905 9555 26132 9556							

Conr's on Acc. M5597



STATE OF NEW YORK  
 SUBJECT City of Johnstown, N.Y., FILE NO. Dm 427M  
Proposed Cork Center Cr. Dam ACC. NO. M5597  
 COMPUTER Just. Jan. 24, 1919 CHECKED BY See Sht. 30 SHEET 33  
 MADE IN CONNECTION WITH See Sht. 30 CURV'S FROM ACC. M 5592

REFERENCE Equalization Effect of Reservoir as work was stopped.

Area & Capacity Curve Data - Continued:

El.	Areas	Sum	Sum/2	Interval ft.	Volume Increments	Volume- Integrated
1022.5	43M	314M	157M	7.5	1,385M	
" 1031	271M	636	318	5.0	1,590M	1,385,000
" 1036	365M	904	452	5.0	2,260M	2,975,000
" 1041	539M	1615	807.5	5.0	4,000M	5,235,000
" 1046	1,076M	2,607	1,304	5.0	6,520M	9,275,000
" 1051	1,532M	3,367	1,683.5	3.0	5,050M	15,795,000
" 1054	1,835M	5,202	2,601			20,845,000
		5,661	2,830.5			26,545,000

(Data from Acc. M5592)

Plotted on Acc. 511

46. 380  
Form 1080

STATE OF NEW YORK - CONSERVATION COMMISSION

SUBJECT

City of Johnstown, N.Y.,  
Proposed Cork Center On Dam

FILE NO

Dm 457M

ACC. NO

3085

Seri # 275

SHEET 34

COMPUTER

W.H. Jan 27 1919

CHECKED BY

See Sht. 30

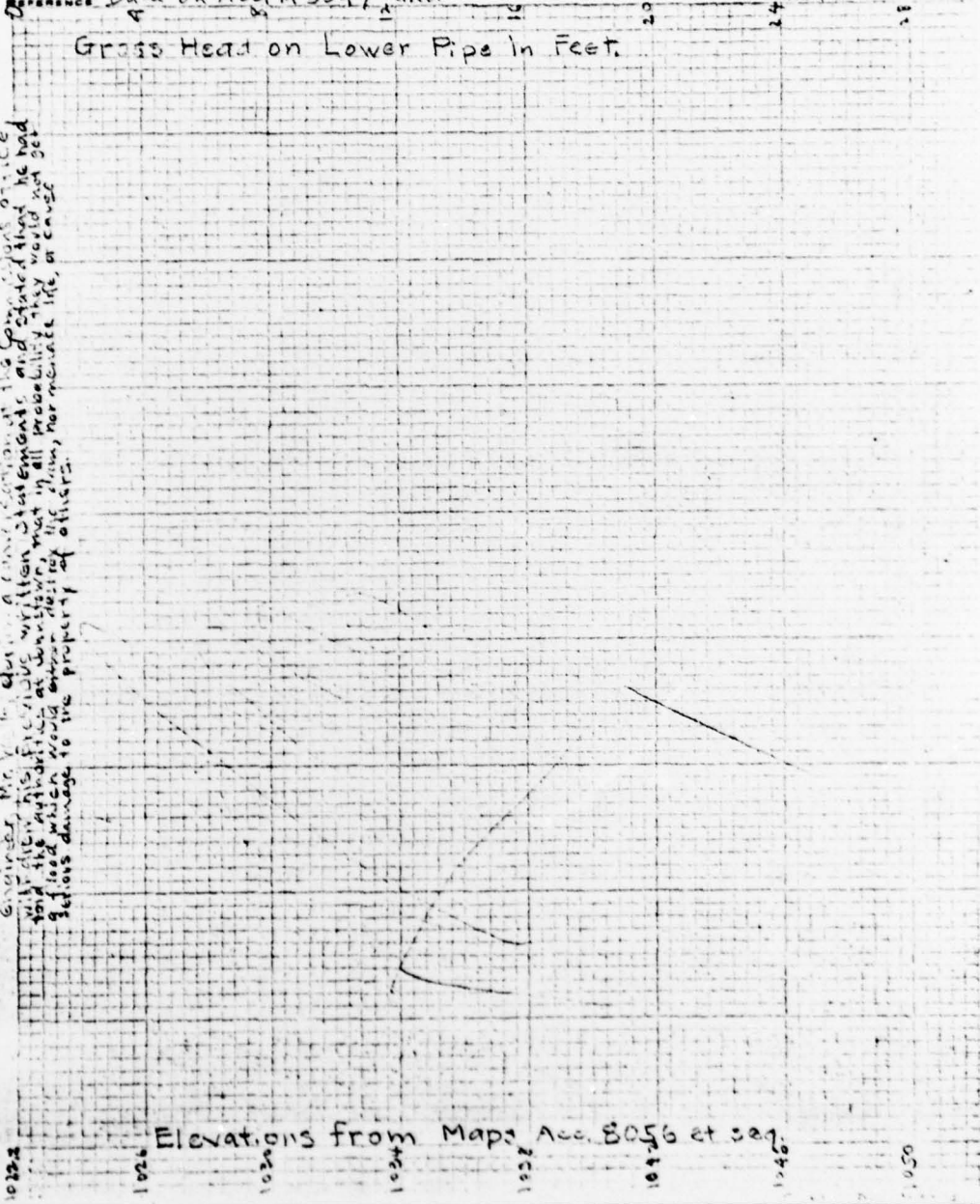
MADE IN CONNECTION WITH

REFERENCE Data on Acc. M5597 and

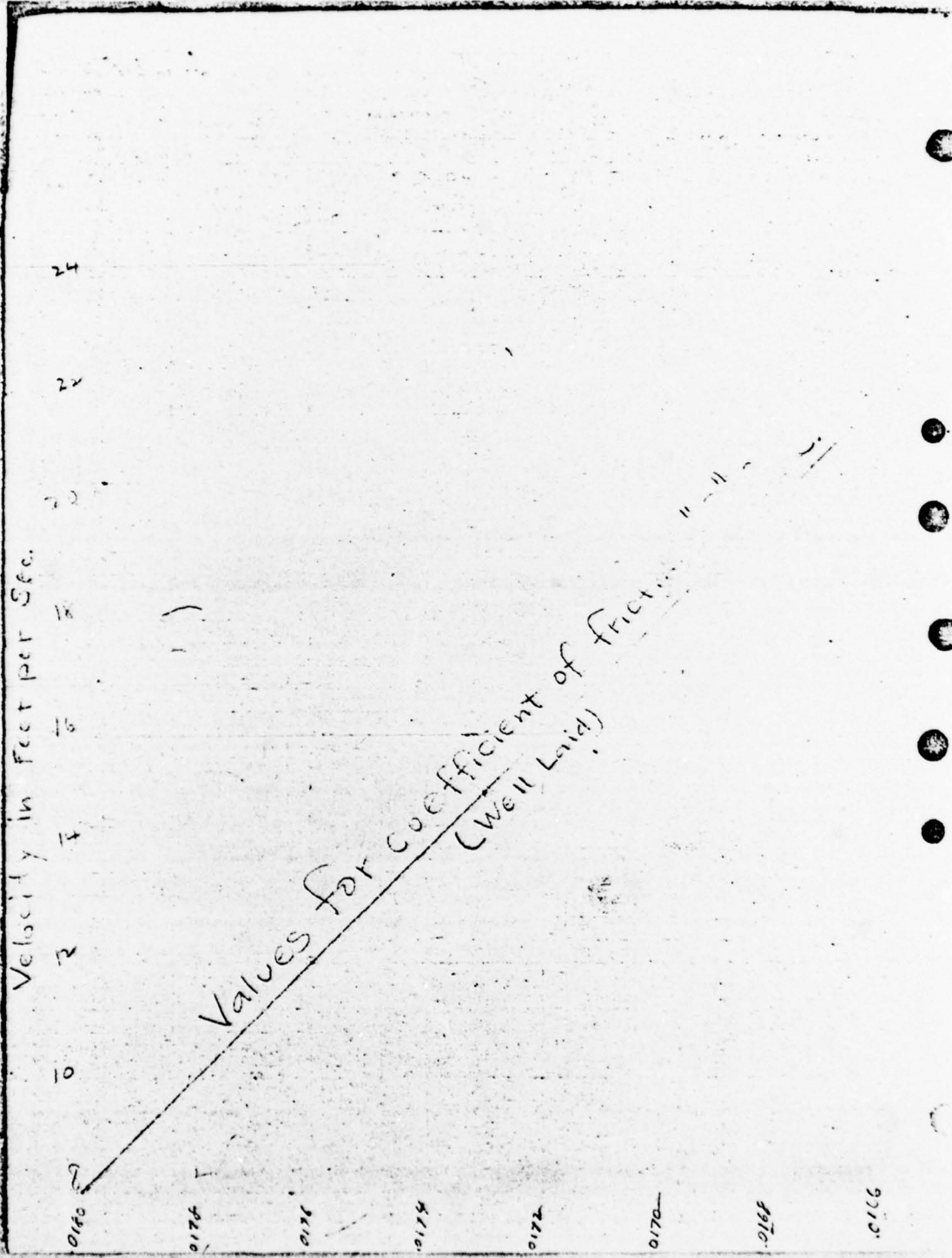
Cont'd from Acc. M5597

Gross Head on Lower Pipe in Feet.

Comments: was not contacted as the applicants  
convinced Mr. Y. in den. a connection at the Conservation office  
with the title records written statements and stated that he had  
paid the title records. However, that in all probability they would not get  
a lead which would ensure that the dam, nor incite life, or cause  
serious damage to the property of others.



Elevations from Maps Acc. 8056 et seq.



STATE OF NEW YORK - CONSERVATION COMMISSION

App. 390  
Form 1960

SUBJECT

City of Johnstown, N. Y.  
Proposed Clark Center Creek dam.

FILE NO Dm 427 M

ACC. NO M-4294

SER. # 275

SHEET 35

COMPUTER

J. H. Jan 24, 1919

CHECKED BY

19

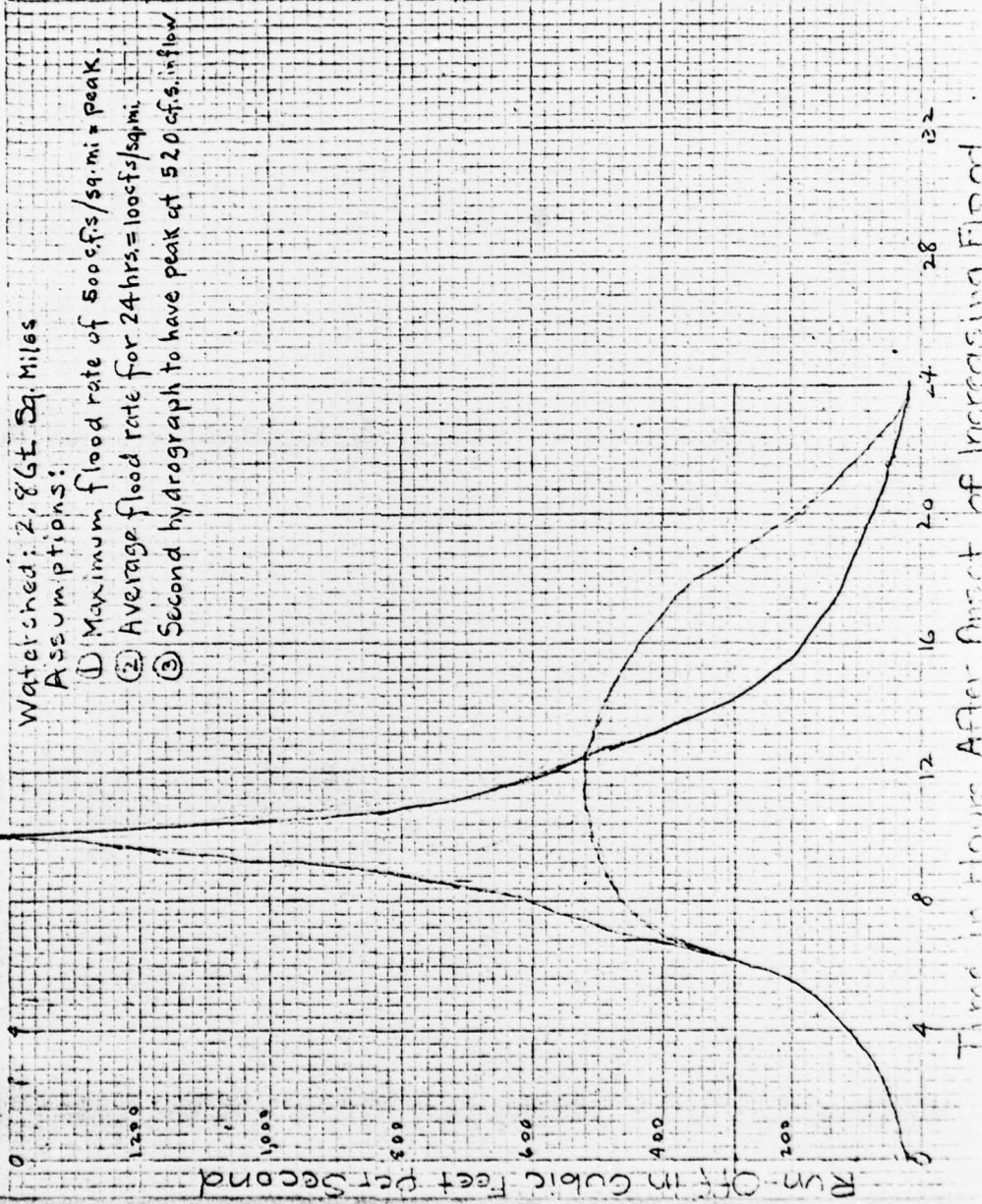
MADE IN CONNECTION WITH

See Sheet 30

REFERENCE

Equalization effect of reservoir as left when work was stopp'd

Cont'd from App. 308



Time in Hours After Onset of Increasing Flood

FOR: D. PRATT, COMMISSIONER  
ALEXANDER MACDONALD, DEPUTY COMMISSIONER  
S. LIGHTON, SECRETARY  
FOR: WILL MCLEAN, DEPUTY ATTORNEY GENERAL

STATE OF NEW YORK



DIVISION OF FISH AND GAME  
LLEWELLYN LEGGE, CHIEF  
DIVISION OF LANDS AND FORESTS  
C. R. PETTIS, SUPERINTENDENT  
DIVISION OF WATERS  
A. H. PERKINS, DIVISION ENGINEER  
DIVISION OF SARATOGA SPRINGS  
J. G. JONES, SUPERINTENDENT  
SARATOGA SPRINGS, N. Y.

CONSERVATION COMMISSION

ALBANY

July 23, 1917.

REPLYING PLEASE REFER TO FILE NUMBER

Mr. A. H. Perkins, Division Engineer,  
Conservation Commission,  
P R E S E N T.

Dear Sir:-

Concerning the reservoir #427 Mohawk  
Watershed for the city of Johnstown (Serial  
#275):-

On July 20 I inspected the vicinity  
of the site for this dam. The soil I found to  
be a loamy earth with some bolders which were  
mostly of granite from a glacial formation.  
I found no outcropping of any ledge rock and  
the excavations will have to be carefully watched  
to see if there is any such.

Respectfully yours,

*Allen Rice Watkins*

INSPECTOR OF DOCKS AND DAMS.

MvK:MH.

150

Nov. 6, 1916.

Mr. W. E. Natanson, City Engineer,  
Johnstown, N. Y.

Dear Sir:-

We are informed that you are going to build a dam and desire information about the jurisdiction of the Conservation Commission over such construction. We, therefore, hand you here-with application blanks and instruction sheet.

Yours truly,

GEORGE D. PRATT, Comm'r.

By.



---

Division Engineer.

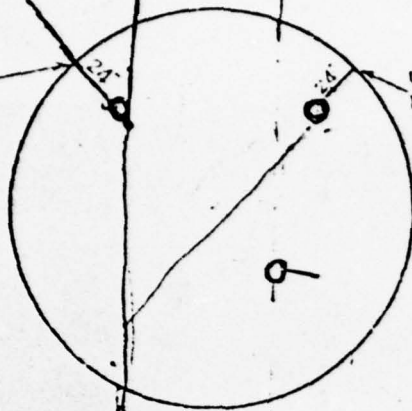
ANP/F

STANDPIPE - CORK CENTER STORAGE RESERVOIR

05  
69

Lower Level Intake  
25' from top

Upper Level Intake  
17' from top



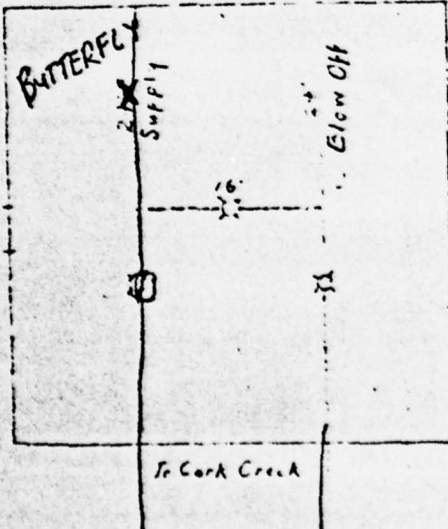
26  
Supply

24  
Blow off

40' DEEP

70

GATE HOUSE -  
CORK CENTER  
STORAGE RESERVOIR



To Cork Creek

INSTRUCTIONS FOR STATE OF EMERGENCY CONDITIONS

- 1.) Notify Abel and McGregor

Cold Brook

- 1.) Open gate in creek culvert under highway if you can get to it.
- 2.) Open mud gate in manhole between reservoir and road.
- 3.) Open hydrants, Johnson, Hall, Akin, John, Wells, Burton, Pearl Oak. Close down gate on Cayadutta Street at Main.

Storage

Open gate.

Open Mud Gate

Pray

Distributing.

- 1.) Open Bypass gate at Aeration Falls but don't let the channel overflow, especially down by Chlorination House.
- 2.) Open Mud Gate in gate house.
- 3.) No water should escape over any part of the dam except the concrete spillway.

RESERVOIR INSTRUCTION

Cold Brook

18" Below sidewalk - turn down gate in cornfield  
Even with sidewalk - turn out intakes  
12" over sidewalk - State of Emergency Exists, - call Skinny, Jr. & WM.H. McGregor

Storage

- 1.) When full, keep it from running over by opening gate. Keep it 2" - 6" down from edge.
- 2.) If not full, operation of gate depends on distributing reservoir operation.

Distributing

- 1.) When storage is not full, do not let intake run over spillway. Intake should range 6" - 42" below spillway level. Adjust by storage gate.
- 2.) When storage is full, intake can run over, but watch depth over spillway and fullness in concrete bypass channel.
- 3.) If any water begins to run over the grassy, earthen dam or around the sides of the spillway, a State of Emergency Exists.