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NATIONAL DAM SAFETY PROGRAM. DELTA DAM (INVENTORY NUMBER NY 6),--ETC(U)
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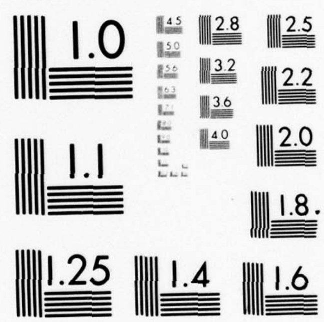

MICROCOPY RESOLUTION TEST CHART
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MOHAWK RIVER BASIN

DELTA DAM

ONEIDA COUNTY, NEW YORK
INVENTORY NUMBER NY 6

PHASE 1
INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

APPROVED FOR PUBLIC RELEASE;
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CONTRACT NO. DACW-51-78-C-0025

...ared by

...OBERT KIMBALL and ASSOCIATES
...W. Highland Ave. Ebensburg, Pa.

Prepared For

DEPARTMENT OF THE ARMY
NEW YORK DISTRICT, CORPS OF ENGINEERS
NEW YORK, NEW YORK

DISCLAIMER NOTICE

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SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

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10 R. Jeffrey/Kimball

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Dam Safety	Oneida County
National Dam Safety Program	Mohawk River
Visual Inspection	Delta Dam
Hydrology, Structural Stability	

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.

Delta Dam was judged to be unsafe, non-emergency due to a seriously inadequate spillway. Additional studies and investigations were also recommended.

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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:

I.D. NO.	NAME OF DAM
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. 421	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) Wiccopee Dam (Lower Hudson W.S. for Peekskill)

I.D. NO.	NAME OF DAM
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

MOHAWK RIVER BASIN

DELTA DAM

ONEIDA COUNTY, NEW YORK
INVENTORY NUMBER NY 6

PHASE 1
INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

Prepared by

L. ROBERT KIMBALL and ASSOCIATES
615 W. Highland Ave. Ebensburg, Pa.

Prepared For

DEPARTMENT OF THE ARMY
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NEW YORK, NEW YORK

Table of Contents

Description of Photographs
Delta Dam

Plate

1. Overall view of the dam from left abutment.
   Note: Seepage visible on section of dam below gate house.

APPENDIX C

2. Left abutment portion of dam from downstream.
   Note: Deterioration of gunite facing, seepage.
   Visible: Old gate house at toe, stilling pools for principal and emergency spillway.

3. Right portion of dam from downstream.
   Note: Deterioration of gunite facing, sprays in spillway apparently caused by deterioration in concrete.
   Visible: Stilling pool, rip rap protection.

4. View of immediate downstream area from top of dam.

5. Looking at major seepage area below gate house from top of dam.
   Note: Vegetative growth in face, rolling and bulging of gunite facing. Amount of seepage depicted by ground saturation and puddles at toe.

6. Close up view from downstream of right abutment of dam.
   Note: Vegetative growth and deterioration of gunite facing.

7. Close up view of downstream face of left abutment of dam.

8. Close up of deterioration of gunite facing on downstream face of left abutment of dam.

9. Upstream face and gate house from left abutment.

10. Upstream face from gate house.
    Note: Erosion of concrete on upstream face at water line.

Phase I Report

Name of Dam: Delta Dam

State Located: New York

County Located: Oneida

Stream: Mohawk River

Date of Inspection: May 5, 1978

ASSESSMENT

The visual inspection and evaluation of Delta Dam did not reveal any problems which would require immediate emergency action. This is not to imply that planning and implementation of followup analyses, design and construction should be put off. As soon as practical, the following studies should be initiated by the owner.

1. Flood routing completed for this structure indicated that the spillway is seriously inadequate as defined by ETL 1110 "Review of Spillway Adequacy". The dam cannot safely pass either the SPF or PMF. Either additional spillway facilities or lowering of the pool elevation to provide additional storm storage or a combination of the two should be studied and a plan implemented in the near future.

2. A thorough evaluation of the condition of the structure including test borings, sampling, and testing and stability analyses should be conducted. Results of the stability analyses should reflect whether the structure can withstand overtopping and used as a design parameter for spillway modifications. The condition of the concrete within the dam and the affect of seepage throughout the dam should be determined.

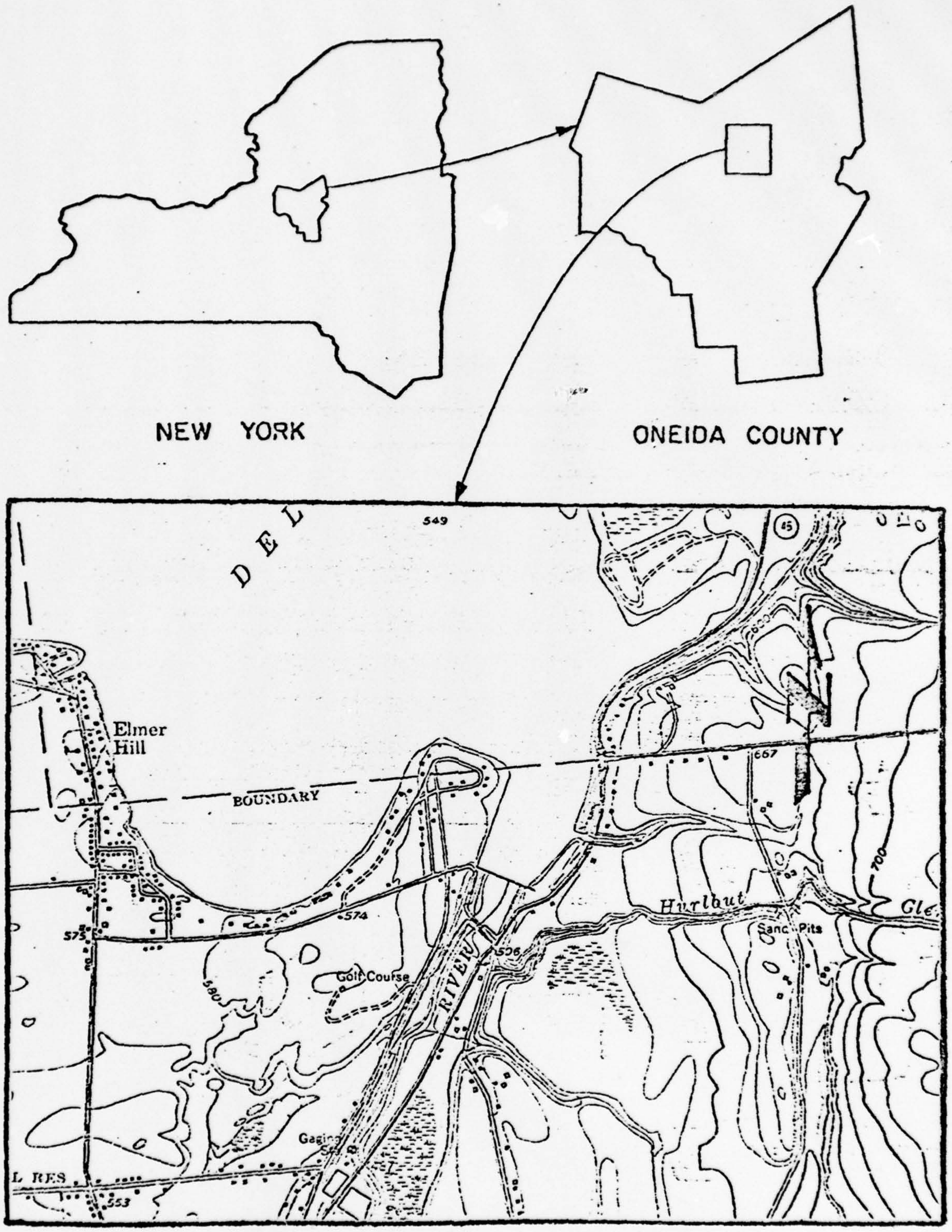
Approved by: R Jeffrey Kimball

R. Jeffrey Kimball, P.E.
L. ROBERT KIMBALL & ASSOCIATES
Registration No. PA 26275 E

Approved by: Clark H. Benn

CLARK H. BENN
Colonel, Corps of Engineers
District Engineer
30 Jun 78

PLATE 1



Portion of U.S.G.S. 7.5' Westernville Quadrangle

DELTA DAM

SITE LOCATION MAP

SCALE : 1"=2000'

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM
DELTA DAM ID# NY 6

SECTION 1: PROJECT INFORMATION

1.1 General

a. Authority: Authority is provided by the National Dam Inspection Act, Public Law 92-367,1972; Contract No. DACW 51-78-C-0025.

b. Purpose of Inspection: Evaluation of non-Federal dams to identify dams which are a threat to life and property.

1.2 Description of Project:

a. Description of Dam and Appurtenances: Delta Dam is a cyclopean masonry gravity structure about 100' high and 1000' long. The dam was completed in 1912 and is used to supply water to the canal system. There are two large (331 and 385' long) abutment sections separated by a 300' long concrete ogee spillway. On the left abutment section is a gatehouse which controls valves for four 60" steel pipes that are used as the outlet works. The top of of the spillway crest is 550.0' and the top of dam is 558.0'.

b. Location: Delta Dam is located approximately 3-1/2 miles north of the City of Rome, Oneida County, New York. The latitude is 43°-16.2' and the longitude is 75°-26'.

c. Size Classification: The storage capacity at normal pool is approximately 63,000 ac-ft. and the height is 100 ft. On this basis the size classification is large.

d. Hazard Classification: The dam is classified as high hazard because of the presence of approximately 20 homes immediately downstream and the City of Rome.

e. Ownership: The dam is owned by the State of New York Department of Transportation.

f. Purpose of Dam: Delta Dam was originally constructed and is currently being used to supply water for the canal system.

g. Design and Construction History: The dam is believed to be designed by the State of New York. Construction began in 1909 and completed in 1912. The contractor was Arthur McMullin. Construction drawings are available at the Department of Transportation offices. Little information was available on the construction history of the dam.

Evaluation of seepage through the dam was completed in 1976-1978 by the New York State DOT with recommendations made for stopping the seepage.

h. Normal Operational Procedures: The reservoir supplies water for the New York Canal System and water is drawn off the lake as needed. The water is regulated by opening the four 60" diameter pipes. If a large storm is expected, the caretaker may lower the lake level to store some of the runoff.

1.3 Pertinent Data:

a. Drainage Areas: The Delta Dam impounds waters of the Mohawk River. It has a drainage area of 150 square miles of open rolling country.

b. Discharge at Damsite:

Maximum Known Flood at Damsite: 4.5' over spillway: 9,000 cfs

Spillway Capacity at Maximum Design Pool Elev.: Unknown

Spillway Capacity at Top of Dam: 21,600 cfs

Outlet works Capacity at Normal Pool: 2,900 cfs

Outlet works Capacity at Top of Dam: 3,100 cfs

Total Spillway Capacity at Top of Dam: Assumed 24,700 cfs

c. Elevation (Ft. above MSL):

Top of Dam: 558

Maximum Pool Design Surcharge: Unknown

Spillway Crest: 550

Streambed at Centerline of Dam: 480

Maximum Tailwater: 490

d. Reservoir:

Length of Normal Pool: 2,300'

Length of Maximum Pool: 2,600' (Top of Dam)

e. Storage (ac-ft):

Normal Pool: 63,000

Design Surcharge: Unknown

Top of Dam: 87,500

f. Reservoir Surface (acres):

Top of Dam: 3,200

Normal Pool: 2,670

Spillway Crest: 2,670

g. Dam:

Type: Cyclopean masonry gravity dam

Length: 1000'

Height: 100'

Top Width: 10.5'

Side Slopes: Upstream 1:30
Downstream 1:6.5

Zoning: N/A

Impervious Core: N/A

Cutoff: Approximately 8' wide and 8' deep into rock

Grout Curtain: Unknown - 1924 report says that grouting will be done, however no record of grouting was found.

h. Diversion & Regulating Tunnel:

Type: Four 60" diameter pipes

Length: Approximately 70 feet

Closure: Opened and closed by electric motors

Access: At gatehouse on crest of dam

Regulating Facilities: Valves with electric motors regulate the flow to the Mohawk River.

i. Spillway:

Type: Concrete ogee

Length of Weir: 300'

Crest Elevation: 550'

Gates: None

U/S Channel: None

D/S Channel: Approximately 65' drop at 1:6.7 slope

j. Regulating Outlets: None

SECTION 2: ENGINEERING DATA

2.1 Design: Pre-construction drawings are available at the Department of Transportation office in Utica. No hydrologic or hydraulic design data was available. A report by Thos. H. Wiggin in 1924 summarizes stability.

2.2 Construction: No construction records were available for review.

2.3 Operation: No operating instructions were made available to indicate proper operation of the outlet works.

2.4 Evaluation: Some of the hydrologic and hydraulic data necessary to perform a detailed analysis of the structure was not available. The material that is available appears to be valid.

SECTION 3: VISUAL INSPECTION

3.1 Findings:

a. General: Delta Dam was inspected by personnel of L. Robert Kimball and Associates and the New York State Department of Transportation on May 5, 1978.

b. Dam: The dam appears to conform to the construction drawings. The dam has had considerable deterioration in the past which has been repaired. Deterioration and seepage problems are continuing to progress.

c. Appurtenant Structures: All drawdown facilities appeared to be in good working condition.

d. Reservoir Area: The impoundment retains waters of the Mohawk River. The overburden in the area is relatively thin and the slopes are moderate.

e. Downstream Channel: The immediate downstream channel is medium wide and open except for several bridges which are of no effect. The channel to the City of Rome is a medium wide flood plain with some development along the river.

3.2 Evaluation: Visual inspection revealed that the dam has seepage passing through the two abutment monoliths. These two monoliths show significant deterioration of the shotcrete and leaching of the shotcrete or concrete. Some of the leaching may be coming from the concrete, since severe deterioration was noted at an early age (1924).The dam is founded on a dark gray fissile shale which is nearly horizontally bedded. This type of rock usually exhibits significant seepage, but it is not present. The drawdown facilities appear to be in good working condition. The downstream area would be affected severly if Delta Dam were to fail.

SECTION 4: OPERATIONAL PROCEDURES

4.1 Procedures: The dam is operated as a water supply for the New York Barge Canal System and water is drawn off the reservoir as needed. At times, if a storm is expected, water is expelled from the reservoir so storm waters can be stored.

4.2 Maintenance of Dam: Day to day maintenance is performed on the dam as needed.

4.3 Maintenance of Operating Facilities: Normal maintenance of operating facilities is performed as needed.

4.4 Description of Any Warning System In Effect: No warning system is present.

4.5 Evaluation: The dam and appurtenant structures appear to be operated at regular intervals and the operational equipment is well maintained.

SECTION 5: HYDRAULIC/HYDROLOGIC

5.1 Evaluation of Features:

a. Design Data: No hydrologic design data was available for the structure. The review information available included stability analysis with a maximum water level 2 feet below the top of dam at elevation 556 feet.

Delta Dam is used primarily for canal water supply. When practical the reservoir level is lowered to accommodate flood waters. The caretaker reports that principal spillway gates are opened when storms are predicted.

b. Experience Data: Reservoir water level records are kept by the caretaker. They are not formally recorded by the owner.

A report titled "Upper Hudson and Mohawk River Basins Hydrologic Flood Routing Models" prepared by the New York District Corps of Engineers, 1976 contains information on Delta Dam.

There is a U.S.G.S. gauge downstream of the dam. The Corps report indicates a peak discharge of 21,819 cfs for their model of the Transposed Agnes event. The caretaker reports a discharge of approximately 9,000 cfs during the Agnes storm.

c. Visual Observations: At the time of the inspection approximately 0.4 feet (5 inches) of water was discharging over the spillway crest (water elevation 550.4').

One of the four spillway gates was open and discharging. All four gates were operated successfully during our inspection.

The downstream toe stilling pool including concrete walls and rip rap were in good condition.

The immediate downstream channel is obstructed by a canal aquaduct and a highway bridge. These structures do not present a concern to overflow capacity.

d. Overtopping Potential: To determine the overtopping potential of Delta Dam, flood routing was conducted.

This potential was investigated through the development of the probable maximum flood (PMF) for the watershed and the subsequent routing of the PMF through the reservoir system. The PMF is that hypothetical flow induced by the most critical combination of precipitation, minimum infiltration losses, and concentration of run-off at a specific location, that is considered reasonably possible for a particular drainage area.

The drainage area contributing to Mohowk River at Delta Reservoir is approximately 150 square miles. Snyder coefficients were developed through watershed modeling done by the Corps. An average Cp = 0.75 and tp = 12.3 were established to define the basic hydrologic working tool, the unit hydrograph. Using Hydrometeorological Report No. 23, the PMP index rainfall was determined to be 19.0 inches for a 24 hour duration, 200 square mile basin. The percentages of the index rainfall applied to other durations were interpolated from the plot of drainage area versus percent of 24 hours, 200 square miles. The computed PMF peak flow was 83,000 CFS. After routing the PMF through the impounded storage, the peak flow was reduced to 67,900 CFS. A plot of the PMF inflow and outflow hydrographs is included in the Appendix. Assumptions made concerning the discharge-storage capacity of the dam were:

1. The reservoir pool was assumed to be at elevation 550.0' (spillway crest).

2. It was assumed that all four gated spillway pipes were closed in developing a discharge rating. This condition is possible and leads to a slightly conservative analysis.

3. A weir coefficient of 3.18 was calculated from available spillway capacity data. The coefficient was assumed accurate for H > 4'. A total spillway length of 300' was assumed correct from construction drawings. For discharges above top of dam elevation (558') a weir length of 662' with a coefficeint of 2.8 was added to the spillway discharge to allow for flow over the dam.

4. Elevation - Storage data was calculated using U.S.G.S. topographic maps.

The ability of the Delta Dam to discharge the standard project flood (SPF) was also evaluated. The SPF peak flow of 49,600 CFS was routed through the impounded storage and reduced to 37,400 CFS. The SPF outflow is indicative of a pool elevation of 560.2 feet above MSL. The dam is overtopped by 2.2 feet, the spillway crest by 10.2 feet. The PMF outflow of 67,900 CFS is equivalent to 5.2 feet over the dam (13.2 feet above the spillway crest).

Summary of Flood Routing
Delta Reservoir

Elevation Top of Dam = 558.0'

Elevation Crest of Spillway = 550.0'

PMF Routing

PMF Peak - 83,000 cfs

PMF After Routing Through Reservoir - 67,900 cfs

Elevation of Routed PMF Corresponding to 67,900 cfs - 563.2'

Dam Overtop - 5.2'

Spillway Surcharge - 13.2'

SPF Routing

SPF Peak - 49,600 cfs

SPF After Routing Through Reservoir - 37,400 cfs

Elevation of Routed SPF Corresponding to 37,400 cfs - 560.2'

Dam Overtop - 2.2'

Spillway Surcharge - 10.2'

5.2 Hydraulic Evaluation of Flood Wave

a. General: For the dam break analysis the flood wave for both the total and partial failure were calculated. The dam is a cyclopean masonry gravity dam founded on rock. Partial failure appears to be the more probable of the two cases.

A summary of flood wave calculations follows this discussion. For both total and partial failure damage to structures 21,000 feet downstream in Colonial Park can be expected. The total failure would flood portions of the city of Rome, New York. Partial failure flooding would be limited to structures within 15 to 20 feet of river level. For partial failure structures located along Route 46 within 2 miles of the dam would be damaged.

Hydraulic Evaluation of Flood Wave:

Total storage = 86,800 AF @ El. 558

	distance from Dam
A) Full breach	
$W_b = W_d = 950'$ $D_b = Y_0 = 90'$ $K = 1$,	0'
$Q_{max} = 1.36 \times 10^6$ cfs	
Reach 1 $L = 2000'$ $W = 1200'$	2000'
$Q_{max} = 1.25 \times 10^6$ (cfs)	
$D_{DS} = 70'$	
Reach 2 $L = 2000'$, $W = 1200'$	4000'
$D_{DS} = 52'$	
$Q_{max} = 1.13 \times 10^6$ cfs	
Reach 3 $L = 4000'$ $W = 3000'$	8000'
$D_{DS} = 34'$	
$Q_{max} = 1.0 \times 10^6$ cfs	
Reach 4 $L = 2000'$ $W = 2500'$	10,000'
$D_{DS} = 36'$	
$Q_{max} = 900 \times 10^3$ cfs	
	13,000'
Reach 5 $L = 3000'$ $W = 2500'$	@ Ridge Mills
$D_{DS} = 33'$	
$Q_{max} = 830 \times 10^3$ cfs	

Reach 6 $L = 4500'$ $W = 2500'$ $17{,}500'$

$D_{DS} = \underline{31.5'}$

$Q_{max} = \underline{750 \times 10^3 \text{ cfs}}$

Reach 7 $L = 3600'$ $W = 3000'$ $21{,}100'$

Floyd Ave Br. over Mohawk River

$D_{DS} = \underline{28'}$

$Q_{max} = \underline{750{,}000} \text{ cfs}$

b) Partial Fail

Assume $W_b = 50'$ $D_b = Y_0 = 90'$

$K = \frac{950}{50} = 19$ $K^{0.28} = 2.28$

$Q_{max} = \underline{163{,}000 \text{ cfs}}$

distance from dam

Reach 1 $L = 2000'$ $W = 300'$ 2,000'

$\underline{D_{DS} = 23'}$

$Q_{max} = \underline{148{,}000 \text{ cfs}}$

Reach 2 $L = 2{,}000'$ $W = 1{,}200'$

$D_{DS} = \underline{16.5'}$ $Q_{max} = \underline{135{,}200 \text{ cfs}}$ 4,000'

Reach 3 $L = 4{,}000'$ $W = 1{,}100'$ 8,000

$D_{DS} = \underline{16.5'}$ $Q_{max} = \underline{124{,}000 \text{ cfs}}$

Reach 4 $L = 2000'$ $W = 1{,}000'$ 10,000'

$D_{DS} = \underline{16.3'}$ $Q_{max} = \underline{110{,}000 \text{ cfs}}$

Reach 5 $L = 3000'$ $W = 800'$ 13,000'

$D_{DS} = \underline{17.5'}$ $Q_{max} = \underline{98{,}000 \text{ cfs}}$

Reach 6 $L = 4500'$ $W = 900'$ 17,500'

$D_{DS} = 15'$ $Q_{max} = \underline{88{,}000 \text{ cfs}}$

Reach 7 $L = 3600'$ $W = 900'$ 21,100'

$D_{DS} = \underline{14'}$ $Q_{max} = \underline{79{,}250 \text{ cfs}}$

SECTION 6: STRUCTURAL STABILITY

6.1 Evaluation of Structural Stability:

a. Visual Observations: No misalignment or settlement of the structure was observed. Water was flowing over the spillway and prevented a close examination of the crest slab.

b. Design and Construction Data: Design data are not available except from the 1924 report which is presented in this report. Construction data are not available. Dam sections were reviewed by Thos. H. Wiggin in 1924 with the water surface at Elev. 556.0 (6' above spillway crest) with and without water pressure beneath the dam. The line of resistance falls within the middle third assuming no water pressure beneath the dam and falls a short distance outside of the middle third if water pressure is assumed to penetrate 2/3 of the area beneath the dam.

c. Operating Records: No information was available on operating records pertaining to the stability of the dam.

d. Post Construction Changes: There have been no post construction changes which should affect the stability of the structure.

e. Seismic Stability: Seismic stability computations are not available. The reservoir is located on the border between seismic zones 1 and 2 and is assumed to present no hazard unless static conditions are unfavorable or marginal.

SECTION 7: ASSESSMENT/REMEDIAL MEASURES

7.1 Dam Assessment:

a. Safety: This dam does not appear to present an immediate danger to life or property. However, the cracks, seepage and leaching of the concrete may increase with time and reduce the stability of the structure. The dam does not appear to present any serious operational deficiencies. The spillway is not adequate to pass the SPF.

b. Adequacy of Information: The information available is inadequate for complete analysis of the dam. The validity of the information available appears to be good. Analysis of stability for maximum highwater and actual uplift pressure is needed.

c. Urgency: The condition of Delta Dam is considered to be a non-emergency situation not requiring immediate action to protect down-stream development. However, due to the presence of cracking, seepage and leaching,a follow upstudy should be conducted before the situation becomes worse. The hydrologic analysis indicates that remedial action is necessary in the near future to provide adequate hydrologic controls for the PMF.

d. Necessity for Future Analyses:

1. A test boring, pressure testing, and laboratory testing program should be conducted to evaluate the internal integrity of the structure.

2. Piezometers should be installed to monitor the uplift pressure on the structure.

3. The stability of the structure should be re-evaluated using the data obtained above and consider maximum high water.

4. The cause of the seepage should be further investigated as suggested in the Materials Bureau report (see appendix D).

7.2 Remedial Measures:

a. Alternatives: The Materials Bureau report should be followed up using the alternative decided upon by the New York Department of Transportation.

b. Remedial modifications should be made within the next two years to increase storm storage and/or spillway capacities if the structural stability of the dam at the high water level (PMF) is inadequate. Additional spillway facilities may be added at the abutments and/or the reservoir pool lowered.

c. An adequate regulation plan and warning system should be developed for use in the event of a threatened failure.

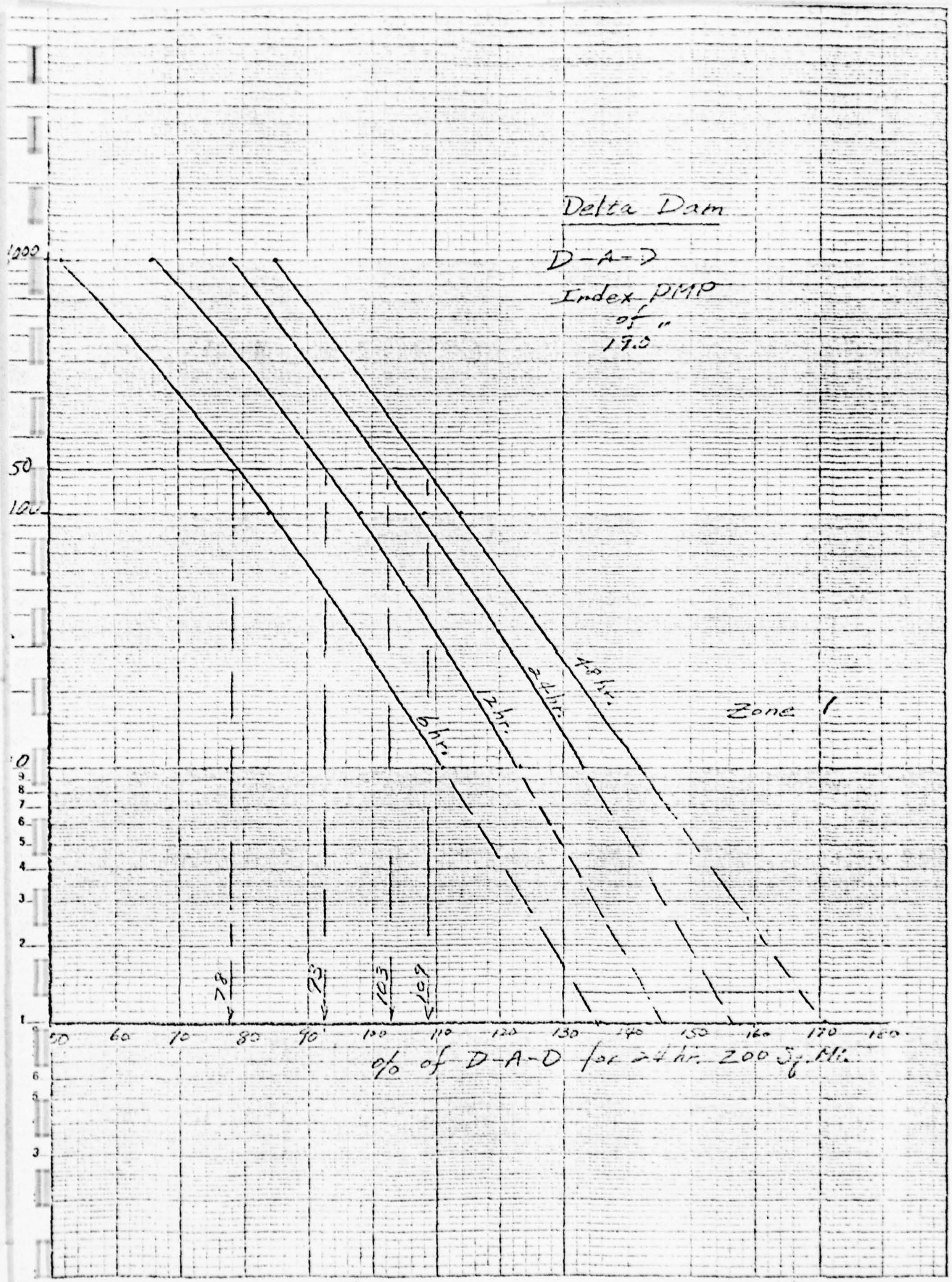
APPENDIX A
GEOLOGY

DELTA RESERVOIR

The bedrock in this region is composed chiefly of Utica Shale, which is of Middle Ordovician Age. These rocks are basically horizontally bedded with only a slight dip to the south. The area is relatively stable tectonically.

During the late Cenozoic Era, this area was heavily glaciated. This produced sediments of various thicknesses and consistencies, particuarly in the valley bottoms.

APPENDIX B
HYDROLOGIC COMPUTATIONS


Delta Dam
D-A-D
Index PMP of 19.0"
Zone 1
6 hr.
12 hr.
24 hr.
48 hr.
78
93
103
109
% of D-A-D for 24 hr. 200 Sq. Mi.

DELTA DAM

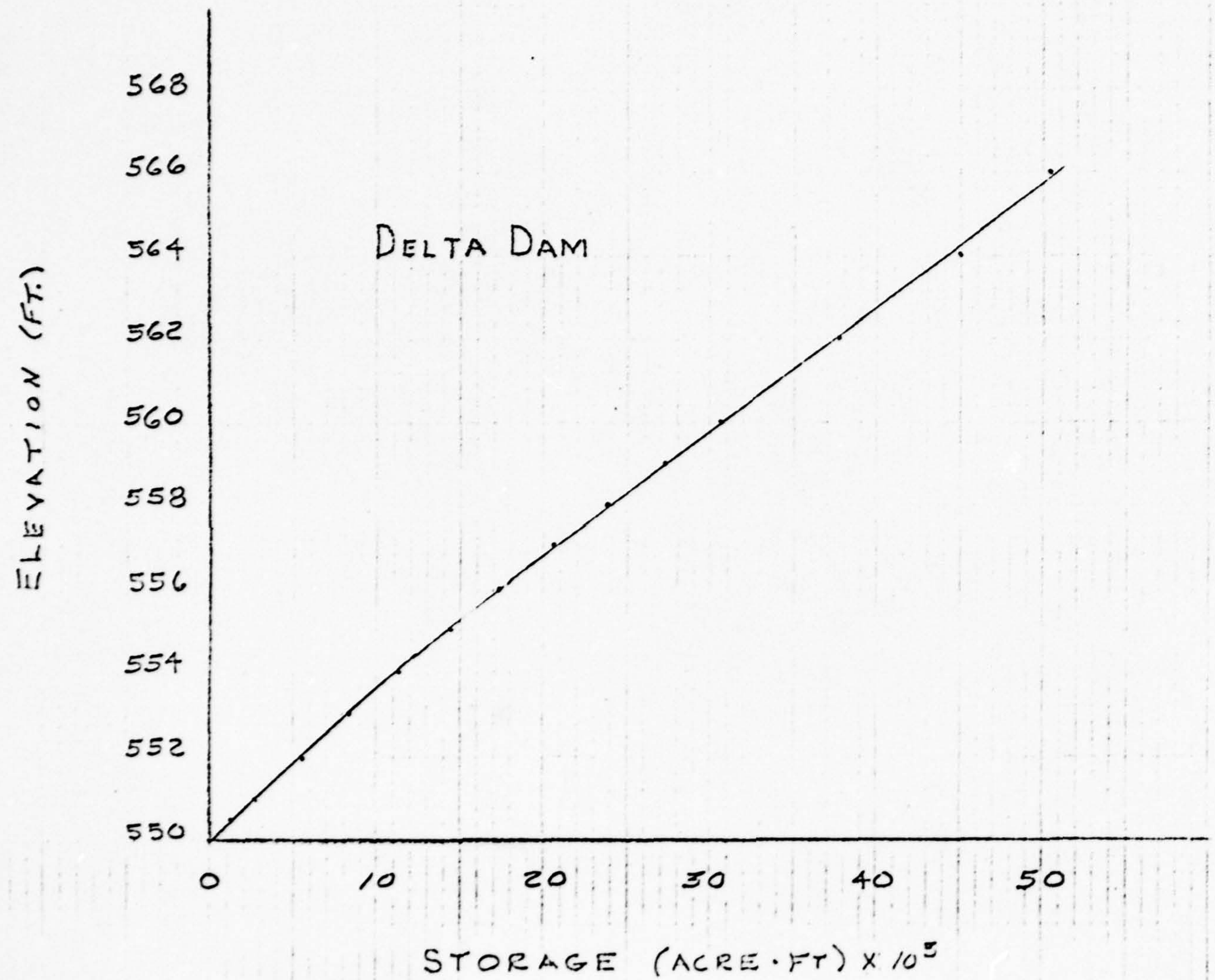
ELEVATION - DISCHARGE RELATIONSHIP

ELEV (FT)	OVERFLOW SECTION C	H	Q (C.F.S)	NON-OVERFLOW SECTION C	H	Q (C.F.S.)	TOTAL Q (C.F.S.)
550.0		0	0	—	—	—	0
550.5	3.18	.5	337	—	—	—	337
551.0	3.18	1	954	—	—	—	954
552.0	3.18	2	2,700	—	—	—	2,700
553.0	3.18	3	4,957	—	—	—	4,957
554.0	3.18	4	7,632	—	—	—	7,632
555.0	3.18	5	10,666	—	—	—	10,666
556.0	3.18	6	14,021	—	—	—	14,021
557.0	3.18	7	17,668	—	—	—	17,668
558.0	3.18	8	21,586	—	—	—	21,586
559.0	3.18	9	25,757	2.8	1	1,854	27,611
560.0	3.18	10	30,167	2.8	2	5,243	35,410
562.0	3.18	12	39,656	2.8	4	14,829	54,485
564.0	3.18	14	49,972	2.8	6	27,242	77,214
566.0	3.18	16	61,054	2.8	8	41,942	102,996

DELTA DAM

ELEVATION - STORAGE RELATIONSHIP

ELEV. (FT.)	SURFACE AREA (ACRES)	Δ ELEV (FT.)	TOTAL STORAGE (AC.-FT.)	DISCHARGE TOTAL Q (C.F.S.)
550.0	2,673		0	0
		0.5		
550.5	2,712		1,346	337
		0.5		
551.0	2,751		2,712	954
		1.0		
552.0	2,829		5,502	2,700
		1.0		
553.0	2,907		8,370	4,957
		1.0		
554.0	2,985		11,316	4,632
		1.0		
555.0	3,063		14,340	10,666
		1.0		
556.0	3,141		17,442	14,021
		1.0		
557.0	3,219		20,622	17,668
		1.0		
558.0	3,297		23,880	21,586
		1.0		
559.0	3,375		27,216	27,611
		1.0		
560.0	3,453		30,630	35,410
		2.0		
562.0	3,609		37,692	54,485
		2.0		
564.0	3,765		45,066	77,214
		2.0		
566.0	3,921		52,752	102,996


DELTA DAM
ELEVATION (FT.)
568
566
564
562
560
558
556
554
552
550
0
10
20
30
40
50
STORAGE (ACRE·FT) X 10³

Delta PMF I

```
******************************
HEC-1 VERSION DATED JAN 1973
UPDATED AUG 74
CHANGE NO. 01
******************************
                 DELTA DAM
                 TEST PMF
                 RESERVOIR AT TOP OF FLOOD CONTROL POOL

                              JOB SPECIFICATION
                    NQ   NHR  NMIN  IDAY   IHR  ITIN METRC  IPLT  IPRT NSTAN
                    60     2     0     0     0     1     0     2     1     0
                                   JOPER     NWT
                                       3       0

     *********       *********       *********       *********       *********

                          SUB-AREA RUNOFF COMPUTATION
                    ISTAQ  ICOMP  IECON  ITAPE   JPLT   JPRT  INAME
                        1      0      0      0      0      0      0

                              HYDROGRAPH DATA
      IHYDG   IUHG  TAREA   SNAP  TRSDA  TRSPC  RATIO  ISNOW  ISAME  LOCAL
          1      1 150.00    0.0 150.00    0.0    0.0      0      0      0

                                PRECIP DATA
               SPFE    PMS     R6    R12    R24    R48    R72    R96
                0.0  19.50  77.50  93.00 103.50 104.00    0.0    0.0
TRSPC COMPUTED BY THE PROGRAM IS 0.870

                                 LOSS DATA
      STRKR  DLTKR  RTIOL  ERAIN  STRKS  RTIOK  STRTL  CNSTL  ALSMX  RTIMP
        0.0    0.0   1.00    0.0    0.0   1.00   1.00   0.07    0.0   0.02

                           UNIT HYDROGRAPH DATA
                      TP= 12.30    CP=0.76    NTA= 0

                               RECESSION DATA
                   STRTQ=  250.00    QRCSN= 1900.00    RTIOR= 1.30
APPROXIMATE CLARK COEFFICIENTS FROM GIVEN SNYDER CP AND TP ARE TC= 7.70 AND R= 3.59 INTERVALS

        UNIT HYDROGRAPH 24 END-OF-PERIOD ORDINATES, LAG= 12.19 HOURS, CP= 0.74  VOL= 1.00
      302.   1404.   2704.  [illegible]  [illegible]  [illegible]  5976.   5451.   4271.   3226.
     2475.   1840.   1395.   1050.    791.    594.    452.    342.    258.    195.
      147.    111.     84.     63.

                           END-OF-PERIOD FLOW
                    TIME   RAIN   EXCS   COMP Q
                      1    0.01   0.00    244.
                      2    0.01   0.00    235.
                      3    0.01   0.00    232.
                      4    0.05   0.00    227.
                      5    0.05   0.00    224.
                      6    0.05   0.00    221.
                      7    0.14   0.00    221.
                      8    0.35   0.01    225.
                      9    0.14   0.00    236.
                     10    0.02   0.00    249.
                     11    0.02   0.00    262.
                     12    0.02   0.00    271.
                     13    0.23   0.05    [illegible]
```

14	0.23	0.04	380.
15	0.23	0.04	551.
16	0.86	0.71	1097.
17	0.86	0.71	2373.
18	0.86	0.71	4535.
19	3.45	3.21	8588.
20	6.84	6.59	17160.
21	2.71	2.56	31242.
22	0.35	0.20	48101.
23	0.35	0.20	64374.
24	0.35	0.20	75940.
25	0.0	0.0	82948.
26	0.0	0.0	81354.
27	0.0	0.0	73504.
28	0.0	0.0	60435.
29	0.0	0.0	47132.
30	0.0	0.0	36151.
31	0.0	0.0	27552.
32	0.0	0.0	20478.
33	0.0	0.0	15743.
34	0.0	0.0	11451.
35	0.0	0.0	8349.
36	0.0	0.0	6856.
37	0.0	0.0	5197.
38	0.0	0.0	3442.
39	0.0	0.0	2493.
40	0.0	0.0	2246.
41	0.0	0.0	1881.
42	0.0	0.0	1832.
43	0.0	0.0	1788.
44	0.0	0.0	1739.
45	0.0	0.0	1694.
46	0.0	0.0	1650.
47	0.0	0.0	1607.
48	0.0	0.0	1565.
49	0.0	0.0	1525.
50	0.0	0.0	1485.
SUM	16.14	15.46	763671.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	82944.	80566.	54256.	21115.	763672.
INCHES		6.00	13.46	15.71	15.79
AC-FT		39971.	107670.	125709.	126292.

OVF

STATION 1

HYDROGRAPH ROUTING

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME
2	1	0	0	2	0	0

ROUTING DATA

QLOSS	CLOSS	AVG	IRES	ISAME
0.0	0.0	0.0	1	0

NSTPS	NSTDL	LAG	AMSKK	X	TSK	STORA
1	0	0	0.0	0.0	0.0	-1.

STORAGE=	0.	3053.	6125.	[illegible]	15315.	21441.	24404.	27557.	37692.	45060.
OUTFLOW=	0.	958.	2700.	7630.	15000.	17005.	21550.	27611.	54485.	77214.

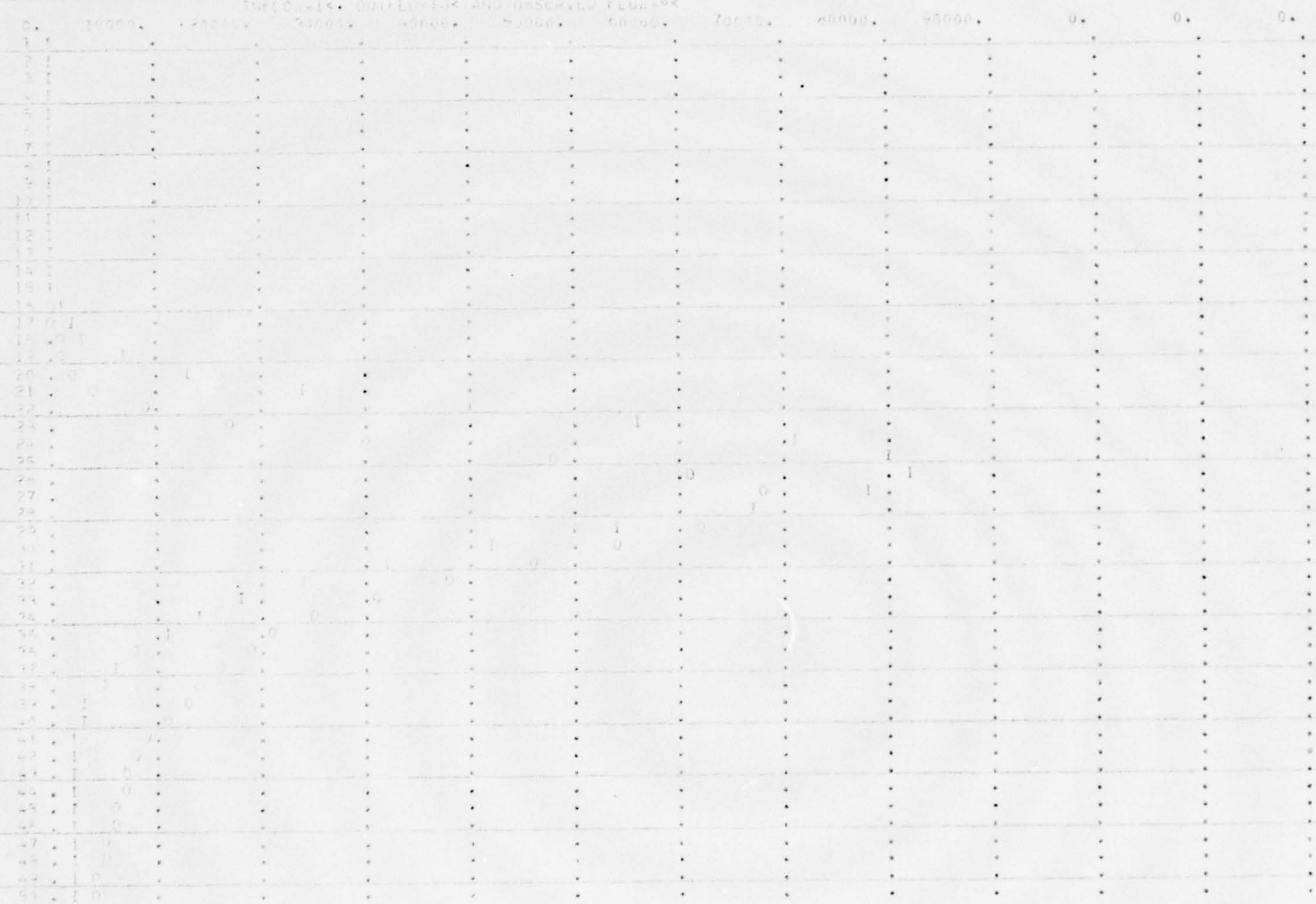
TIME	EOP STOR	AVG IN	EOP OUT
1	762.	244.	244.
2	762.	241.	243.
3	760.	235.	243.
4	776.	231.	242.
5	775.	225.	242.
6	772.	223.	241.
7	764.	221.	240.
8	767.	223.	239.
9	765.	231.	238.
10	766.	243.	239.
11	769.	250.	239.
12	772.	205.	241.
13	780.	283.	243.
14	795.	337.	248.
15	831.	470.	254.
16	923.	829.	287.
17	1155.	1735.	368.
18	1655.	3456.	515.
19	2627.	6546.	818.
20	4547.	12859.	1794.
21	8338.	24231.	4240.
22	13511.	39671.	8479.
23	[illegible]	60431.	14803.
24	[illegible]	70601.	30143.
25	33284.	79914.	47958.
26	[illegible]	84424.	61137.
27	42234.	77652.	67655.
28	41913.	66470.	67646.
29	40107.	53784.	61428.
30	37427.	41042.	53781.
31	34454.	31651.	45891.
32	31515.	24219.	38042.
33	[illegible]	18335.	37484.
34	26487.	13872.	24447.
35	24134.	10500.	21368.
36	22328.	7453.	18803.
37	20405.	6027.	16487.
38	19406.	4567.	14436.
39	16852.	3407.	12437.
40	15453.	2520.	10825.
41	14117.	2054.	9680.
42	12853.	1857.	8328.
43	11985.	1804.	7391.
44	11080.	1762.	6689.

45	10310.	1716.	6288.
46	9625.	1672.	5520.
47	8075.	1628.	5034.
48	[illegible]	1585.	4604.
49	8017.	1545.	4222.
50	7590.	1505.	3883.
SUM			723446.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	67455.	65740.	46031.	29087.	723546.
INCHES		4.54	11.41	14.49	14.96
AC-FT		32625.	91249.	115114.	119673.

STATION

INFLOW=I<, OUTFLOW=O< AND OBSERVED FLOW=*<



RUNOFF SUMMARY, AVERAGE FLOW

		PEAK	6-HOUR	24-HOUR	72-HOUR	AREA
HYDROGRAPH AT	1	67998.	60565.	44250.	21115.	150.00
ROUTED TO	2	67655.	55700.	40001.	20007.	150.00

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****************************
HEC-1 VERSION DATED JAN 1973
UPDATED AUG 74
CHANGE NO. 01
****************************

           DELTA DAM
           TEST SPF
           RESERVOIR AT TOP OF FLOOD CONTROL POOL

                                 JOB SPECIFICATION
                   NO   NHR  NMIN  IDAY   IHR  IMIN METRC  IPLT  IPRT NSTAN
                   50     2     0     0     0     0     0     2     1     0
                                   JOPER    NWT
                                       3      0

          *********       *********       *********       *********       *********

                               SUB-AREA RUNOFF COMPUTATION
                     ISTAQ   ICOMP   IECON   ITAPE    JPLT    JPRT   INAME
                         1       0       0       0       0       0       0

                                    HYDROGRAPH DATA
              IHYDG    IUHG   TAREA    SNAP   TRSDA   TRSPC   RATIO   ISNOW   ISAME   LOCAL
                  1       1  150.00     0.0  150.00     0.0     0.0       0       0       0

                                      PRECIP DATA
                    SPFE     PMS      R6     R12     R24     R48     R72     R96
                   12.00     0.0     0.0     0.0     0.0     0.0     0.0     0.0
TRSPC COMPUTED BY THE PROGRAM IS 0.876

                                       LOSS DATA
             STRKR   DLTKR   RTIOL   ERAIN   STRKS   RTIOK   STRTL   CNSTL   ALSMX   RTIMP
               0.0     0.0    1.00     0.0     0.0    1.00    1.00    0.07     0.0    0.02

                                 UNIT HYDROGRAPH DATA
                           TP= 12.30     CP=0.75     NTA=  0

                                     RECESSION DATA
                     STRTQ=  250.00     QRCSN= 1800.00     RTIOR= 1.30
APPROXIMATE CLARK COEFFICIENTS FROM GIVEN SNYDER CP AND TP ARE TC= 7.70 AND R= 3.58 INTERVALS

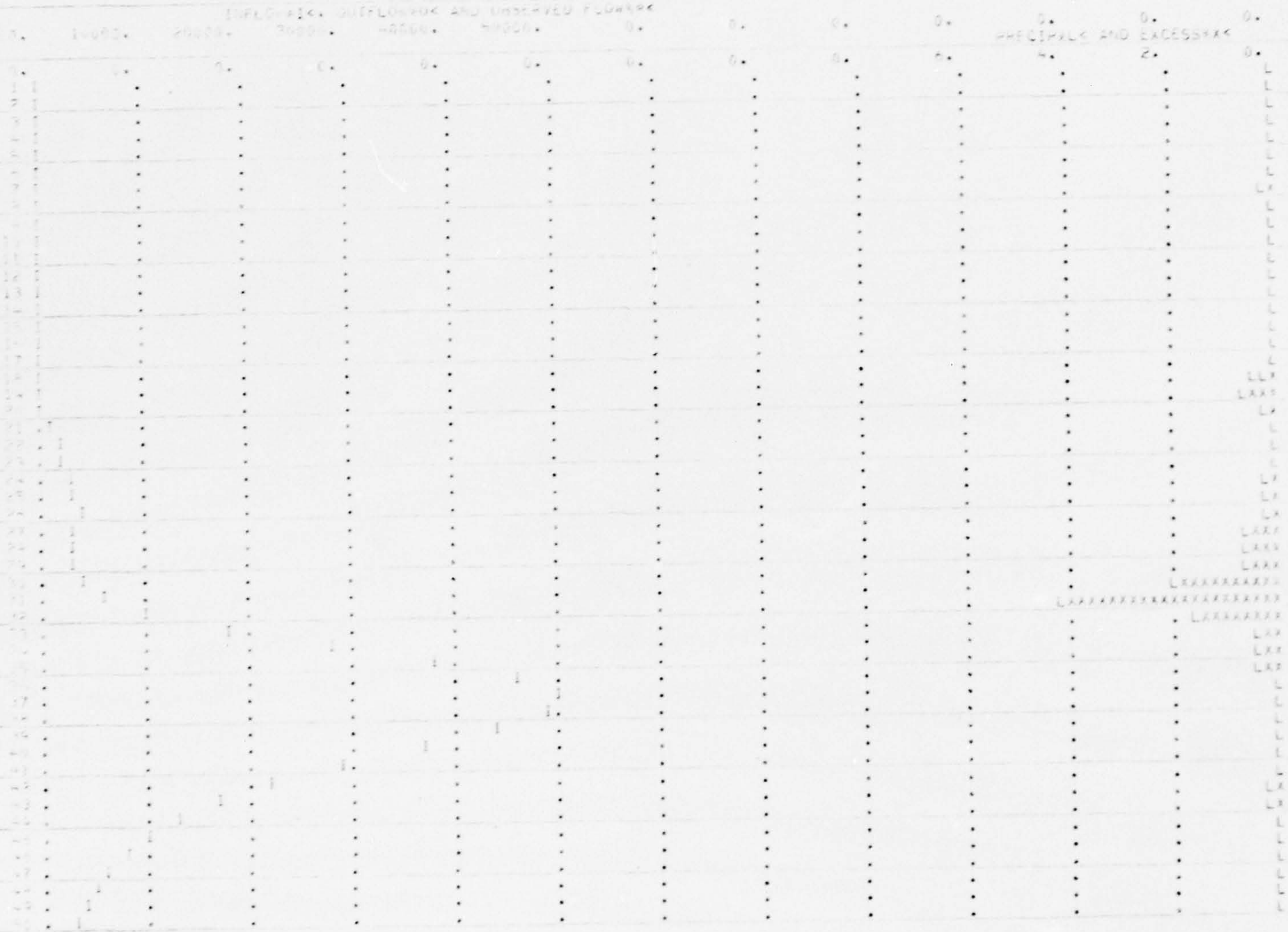
            UNIT HYDROGRAPH 24 END-OF-PERIOD ORDINATES, LAG= 12.19 HOURS, CP= 0.74  VOL= 1.00
          392.      1404.      2704.      4165.      5215.      5659.      5934.      5351.      4271.      3226.
         2436.      1840.      1390.      1050.       793.       599.       452.       342.       258.       195.
          147.       111.        84.        63.

                               END-OF-PERIOD FLOW
                        TIME    RAIN   EXCS    COMP Q
                           1    0.01   0.00      244.
                           2    0.01   0.00      237.
                           3    0.01   0.00      232.
                           4    0.02   0.00      226.
                           5    0.02   0.00      221.
                           6    0.02   0.00      217.
                           7    0.07   0.00      214.
                           8    0.14   0.00      212.
                           9    0.06   0.00      213.
                          10    0.01   0.00      215.
                          11    0.01   0.00      217.
                          12    0.01   0.00      217.
                          13    0.03   0.00      215.
```

14	0.03	0.00	211.
15	0.03	0.00	206.
16	0.07	0.00	201.
17	0.07	0.00	195.
18	0.07	0.00	190.
19	0.31	0.01	184.
20	0.63	0.47	374.
21	0.25	0.10	447.
22	0.05	0.00	1613.
23	0.05	0.00	2385.
24	0.35	0.00	3067.
25	0.19	0.04	3484.
26	0.19	0.04	3539.
27	0.19	0.04	3473.
28	0.50	0.35	3144.
29	0.50	0.35	3224.
30	0.50	0.35	3524.
31	2.07	1.72	5585.
32	4.22	4.07	10194.
33	1.67	1.52	14249.
34	0.31	0.16	24141.
35	0.31	0.16	37644.
36	0.31	0.16	45571.
37	0.01	0.00	44577.
38	0.01	0.00	49217.
39	0.01	0.00	44452.
40	0.03	0.00	35754.
41	0.03	0.00	28524.
42	0.03	0.00	22213.
43	0.12	0.00	16477.
44	0.24	0.10	12706.
45	0.10	0.00	9570.
46	0.02	0.00	7540.
47	0.02	0.00	5792.
48	0.02	0.00	4758.
49	0.0	0.0	3902.
50	0.0	0.0	3337.
SUM	13.53	9.86	474899.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	49577.	46122.	32564.	13106.	474900.
INCHES		2.96	8.08	9.75	9.82
AC-FT		22874.	64623.	78025.	78536.

STATION 1


INFLOW(I), OUTFLOW(O) AND OBSERVED FLOW(*)
0. 10000. 20000. 30000. 40000. 50000. 0. 0. 0. 0.
PRECIP(L) AND EXCESS(X)
0. 0. 0.
4. 2. 0.

```
**********          **********          **********          **********          **********

                                HYDROGRAPH ROUTING
                 ISTAQ   ICOMP   IECON   ITAPE   JPLT    JPRT   INAME
                     2       1       0       0      2       0       0
                                   ROUTING DATA
                         QLOSS   CLOSS     AVG    IRES   ISAME
                           0.0     0.0     0.0       1       0

                 NSTPS   NSTOL     LAG   AMSKK       X     TSK   STORA
                     1       0       0     0.0     0.0     0.0     -1.

STORAGE=       0.    3063.    6125.   12252.   15315.   21441.   24504.   27567.   37692.   45066.
OUTFLOW=       0.     954.    2700.    7632.   10656.   17668.   21566.   27611.   54485.   77214.
```

TIME	EOP STOR	AVG IN	EOP OUT
1	782.	244.	244.
2	782.	241.	243.
3	780.	235.	243.
4	778.	229.	242.
5	775.	224.	241.
6	771.	219.	240.
7	767.	215.	239.
8	763.	213.	238.
9	757.	212.	236.
10	755.	214.	235.
11	752.	210.	234.
12	749.	217.	233.
13	747.	216.	233.
14	743.	213.	232.
15	740.	209.	230.
16	735.	203.	229.
17	730.	198.	227.
18	725.	193.	226.
19	719.	190.	224.
20	729.	284.	227.
21	745.	637.	248.
22	957.	1255.	298.
23	1231.	1999.	383.
24	1606.	2723.	501.
25	2055.	3275.	640.
26	2520.	3564.	787.
27	2973.	3556.	925.
28	3354.	3324.	1120.
29	3683.	3207.	1306.
30	4034.	3529.	1507.
31	4534.	4707.	1795.
32	5501.	7891.	2344.
33	7353.	14220.	3688.
34	10376.	23193.	6122.
35	14512.	33020.	9871.
36	19334.	41735.	15259.
37	24166.	47574.	21179.
38	28183.	49397.	29246.
39	30567.	46534.	35574.
40	31249.	40602.	37383.
41	30626.	32759.	35730.
42	29242.	25519.	32056.
43	27552.	19565.	27581.
44	25754.	14438.	24046.

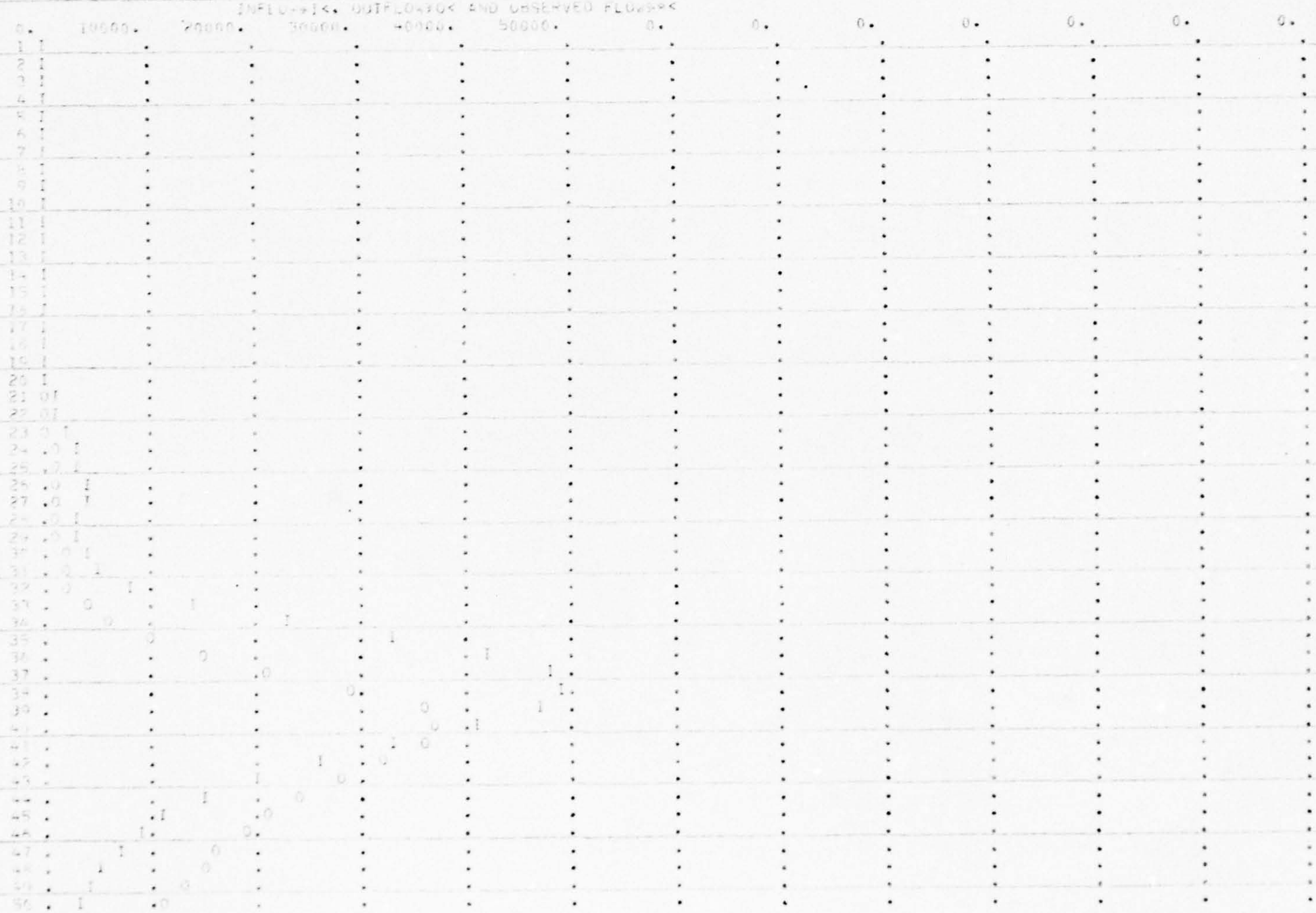
45	23425.	11365.	20647.
46	22118.	8755.	18535.
47	20356.	6816.	16427.
48	18686.	5375.	14520.
49	17140.	4280.	12752.
50	15731.	3420.	11141.
SUM			388512.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	37383.	36229.	26155.	10699.	388512.
INCHES		2.25	6.49	7.96	8.03
AC-FT		17974.	51905.	63699.	64250.

OVF

STATION 2

INFLOW=I<, OUTFLOW=O< AND OBSERVED FLOW=*<



RUNOFF SUMMARY, AVERAGE FLOW

		PEAK	6-HOUR	24-HOUR	72-HOUR	AREA
HYDROGRAPH AT	1	49577.	46122.	32564.	13106.	150.00
ROUTED TO	2	37343.	35229.	26155.	10699.	150.00

APPENDIX C
PHOTOGRAPHS

Description of Photographs
Delta Dam

Plate

1. Overall view of the dam from left abutment.
   Note: Seepage visible on section of dam below gate house.

APPENDIX C

2. Left abutment portion of dam from downstream.
   Note: Deterioration of gunite facing, seepage.
   Visible: Old gate house at toe, stilling pools for principal and emergency spillway.

3. Right portion of dam from downstream.
   Note: Deterioration of gunite facing, sprays in spillway apparently caused by deterioration in concrete.
   Visible: Stilling pool, rip rap protection.

4. View of immediate downstream area from top of dam.

5. Looking at major seepage area below gate house from top of dam.
   Note: Vegetative growth in face, rolling and bulging of gunite facing. Amount of seepage depicted by ground saturation and puddles at toe.

6. Close up view from downstream of right abutment of dam.
   Note: Vegetative growth and deterioration of gunite facing.

7. Close up view of downstream face of left abutment of dam.

8. Close up of deterioration of gunite facing on downstream face of left abutment of dam.

9. Upstream face and gate house from left abutment.

10. Upstream face from gate house.
    Note: Erosion of concrete on upstream face at water line.

PLATE 2

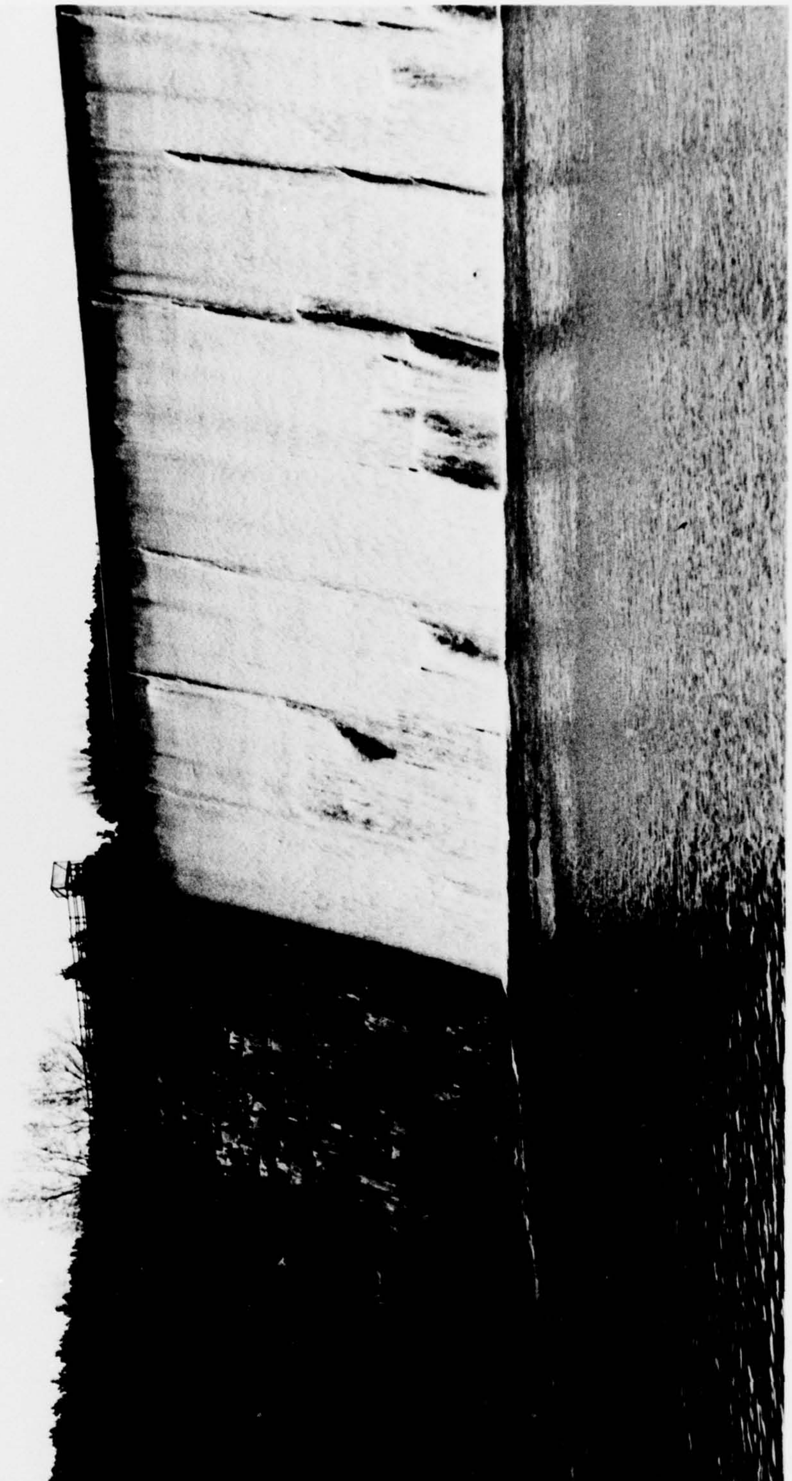


PLATE 3

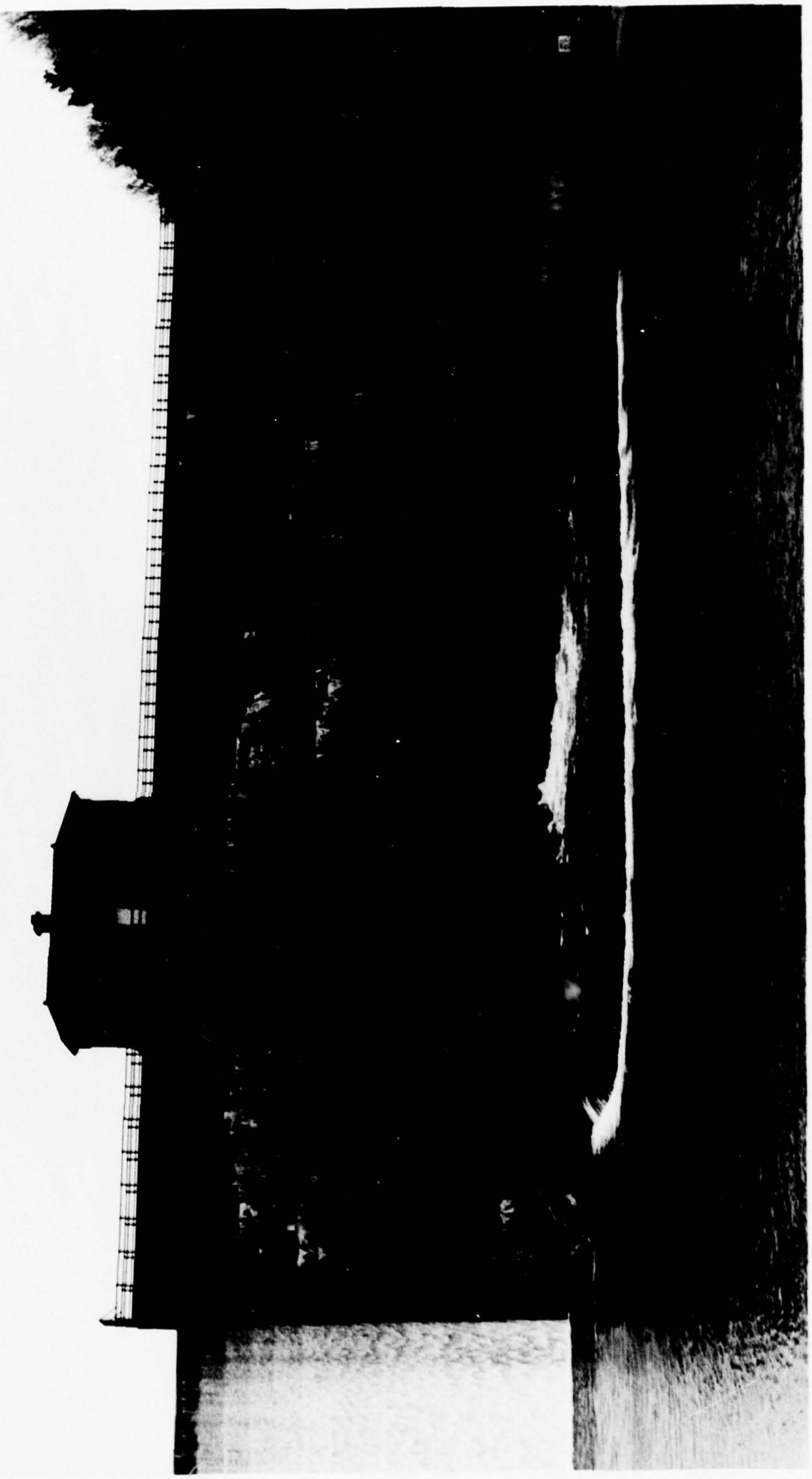


PLATE 4

PLATE 5

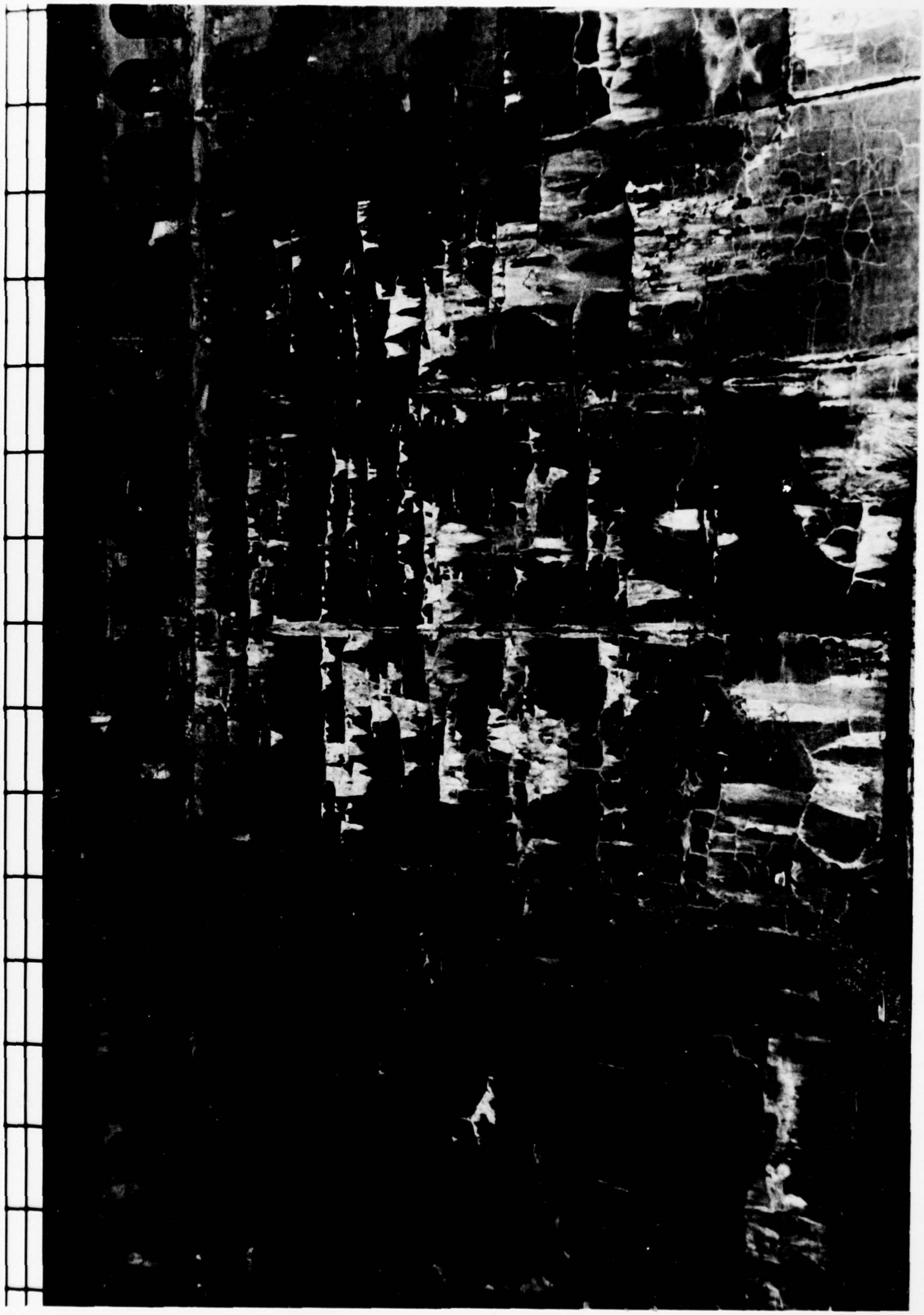


PLATE 6

PLATE 7

PLATE 8

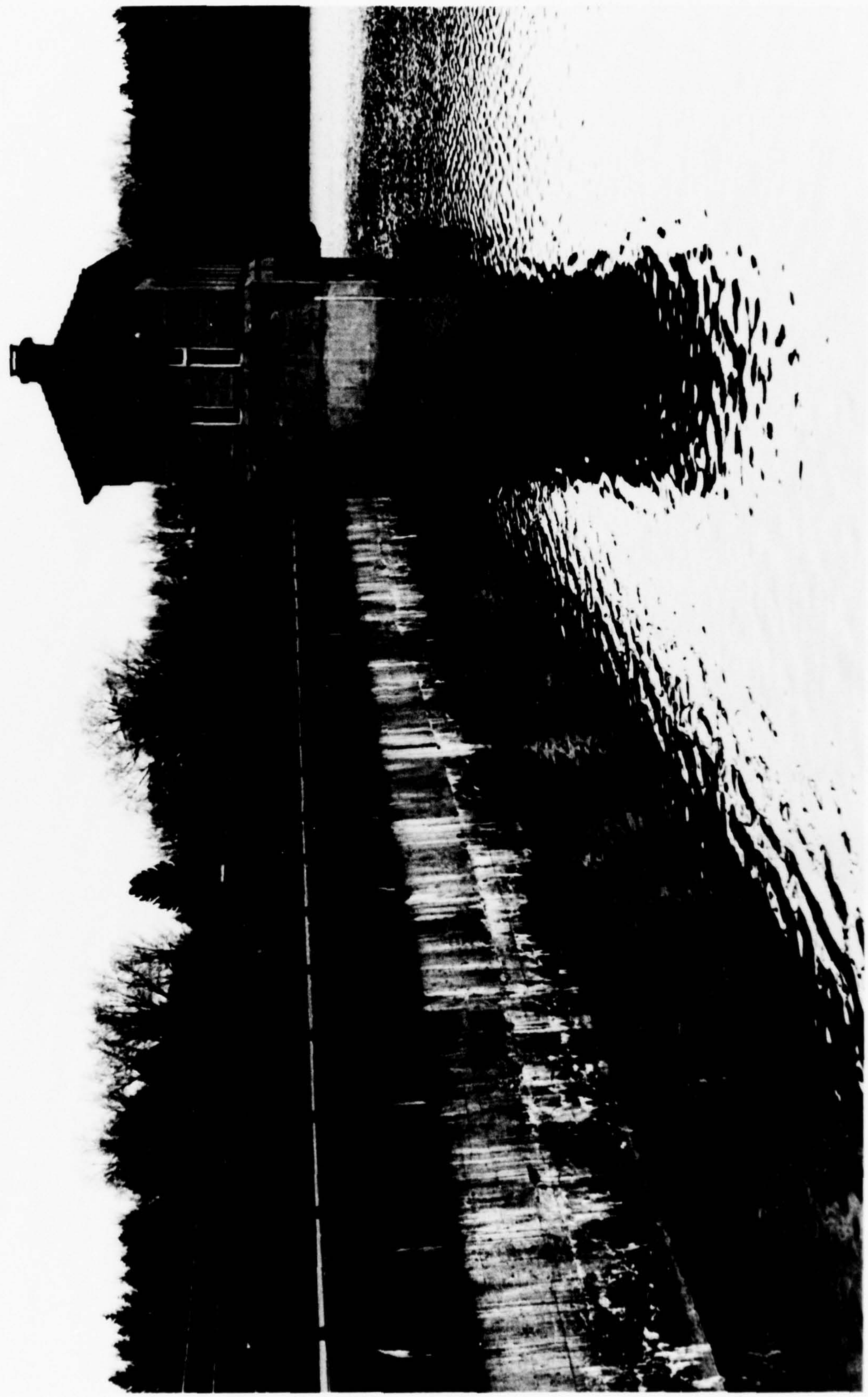


PLATE 9

APPENDIX D
PERTINENT CORRESPONDENCE AND REPORTS

NEW YORK STATE
DEPARTMENT OF TRANSPORTATION
Raymond T. Schuler, Commissioner

1220 Washington Avenue, State Campus, Albany, New York 12226

'76 OCT 28

CONSTRUCTION MANAGEMENT

October 26, 1976

Mr. George Koch, Supervisor
Dam Safety Program
NYS Department of Environmental Conservation
50 Wolf Road
Albany, NY 12233

Dear Mr. Koch:

Re: Dam #935
Delta Reservoir
Mohawk River

Pursuant to your letter of August 12, 1976, an inspection of the subject dam was made by members of the New York State Department of Transportation employees from the Materials Bureau and Soil Mechanics Bureau of the Technical Services Subdivision and from the Main and Regional Offices of the Waterways Maintenance Subdivision.

The inspection group found no soils problems with the dam.

Leaking appears on the downstream face of the dam in several isolated areas beneath the gate house. The amount of water appearing at the face is sufficient to indicate that water is passing through the dam. However, since the dam is a concrete structure which is keyed into bedrock, the leaks are not considered serious enough at this time to classify the situation as an emergency.

The entire dam face including the spillway and buttresses have been repaired with shotcrete at various times. The condition of the shotcrete is poor. Many cracks and deteriorated areas exist in the shotcrete. Also we found that the shotcrete emitted a hollow sound when struck with a hammer in nearly all areas that were reachable. This indicates that the shotcrete is not bonded to the dam.

The shotcrete repairs cover all original vertical and horizontal construction joints. Water may be coming through the dam in a very few locations, most likely the joints, and spreading between the original concrete and the debonded shotcrete layer. This can account for several surface leaks on the dam face.

XXXXXXXX
Peter A.A. Berle,

August 12, 1976

Mr. Joseph R. Stellato, Director
Waterways Maintenance Subdivision
D.O.T. Administration & Engineering Bldg.
State Campus
Albany, New York

Re: Dam #935 Delta Reservoir
Mohawk River

Dear Mr. Stellato:

In conformance with the Dept. of Environmental Conservation Dam Safety program, an inspection was made of the referenced dam on August 8, 1976. The inspection indicates that the concrete in the downstream slope of the dam is spalling and in need of repair. Our previous inspection of this structure was on April 5, 1973. A review of our reports and photos from the two inspections indicates that the concrete deterioration has increased to the extent that now there is a leak in the east abutment approximately two-thirds of the height above the base.

Our records indicate that a letter was sent to your office on April 16, 1973 recommending concrete repair work on the downstream slope. Your letter of April 18, 1973 to Mr. Stanford Zeccolo indicated that a contract would be let to repair the damaged concrete areas. These areas have not been repaired but have deteriorated more.

Because of the large reservoir storage (75,000 A.F.) behind the dam, we feel it is imperative that the concrete maintenance is taken care of before any serious problems develop. After your regional engineer has had an opportunity to inspect the dam, we would like to know of your intentions.

Very truly yours,

George Koch, Supervisor
Dam Safety Program

GK:bt

(2) R. [illegible] [illegible] of structure
D.O.T. [illegible] people took
samples. 10/13/76

cc: Henry B. Zywiak

(1) I Phoned H. Zywiak 9/17/76 He said Engrs from D.O.T. Albany will inspect dam in about a week and he will then receive their decision. G. Koch 17 Sept

Mr. George Koch
October 26, 1976
Page 2

Materials personnel in cooperation with Regional Waterways will conduct an evaluation of the dam. This evaluation will begin by trying to locate the leaks through the dam and the first step will be the opening up of some of the drains on the downstream face which are located at the vertical construction joints. Plugged drains may be a key factor. If the evaluation of the vertical drains does not provide an answer as to where the leak(s) is located, then exploration holes will be made through the shotcrete layer at selected areas. With favorable weather conditions, it is anticipated this evaluation will be completed this fall.

Upon completion of this evaluation, the proper course of action for Waterways to follow will be decided upon.

Waterways will keep you advised on the evaluation and whatever course of action may develop from it.

Sincerely yours,

Edward M. Rowan
for

JOSEPH R. STELLATO
Director of Waterways Maintenance

Bureau of Water Regulation

April 16, 1973

Mr. Joseph R. Stellato, Director
Waterways Maintenance Subdivision
D. O. T. Administration & Eng. Bldg.
State Campus
Albany, New York

Dear Mr. Stellato:

D.O.T.Reg. Dam No. 935 (114-A)
Across Mohawk River at Delta Reservoir
Town of Rome, Oneida County
D. O. T. Reg. Dam No. 932 (115-K)
Across Mohawk River
Town of Rome Oneida County

In conformance with this Department's Dam Safety Program, an inspection was made of the above-referenced dams on April 5, 1973.

A review of our files reveals that these dams are owned by the State of New York. I am assuming that your division is responsible for the maintenance of these structures. If I am in error, please advise.

The Delta dam was constructed in 1912 and consists of a concrete-stone structure approximately 980 feet long and 100 feet high forming an impoundment approximately two miles across at its greatest width and four miles long. The reservoir was created as an addition to the State water storage system for supplying the barge canal.

Our findings reveal that spalding of the concrete has occurred along the wall faces.

Dam No. 932 (115-K) was constructed prior to 1917 and consists of granite blocks mortared together. The structure is approximately 180 feet long and six feet high. The dam is approximately one-quarter of a mile upstream from the canal and is the second dam upstream from the canal. It was constructed to act as an ice breaker to protect the canal system.

NEW YORK STATE
DEPARTMENT OF TRANSPORTATION
Raymond T. Schuler, Commissioner

1220 Washington Avenue, State Campus, Albany, New York 12226

RESOURCE MGT. SERV.
APR 20 1973
BUR. WATER REG.
REC'D.

April 18, 1973

Mr. Stanford Zeccolo
Sr. Hydraulic Engineer
NYS Dept. of Environmental
Conservation
Bureau of Water Regulation
50 Wolf Road
Albany, New York 12201

Dear Mr. Zeccolo:

Re: DOT Reg. Dam No. 935 (114A)
DOT Reg. Dam No. 932 (115K)

We have forwarded your letter of April 16, 1973 concerning the above referenced dams to our Region 2 Office in Utica. The dams are under our maintenance.

The spalling on the concrete wall faces of Delta Dam is a condition we were aware of and monitor closely. We plan eventually to let a contract for the refacing of these areas.

The tree growth occurring at the Upper Retention Dam at Rome will be eliminated by our Section 4 Maintenance Forces.

Thank you for bringing these findings to our attention. All of our dams are scheduled for inspection this August under our Structure Inspection Procedures.

Sincerely yours,

J. Dungan
for
JOSEPH R. STELLATO
Director of Waterways Maintenance

JRS:JD:dl

Our findings concerning this dam reveal that trees are growing in the granite joints along the abutments. This growth could cause serious damage to the structure.

We suggest that your office investigate the above situations. Maintenance at this time could save more expensive repairs in the future.

Very truly yours,

Stanford Zeccolo
Sr. Hydraulic Engineer

SZ/ed
cc: L. Blake

DELTA

DEC DAM INSPECTION REPORT

03	33	12	000935	14573	3	4
B	CTY	YR. AP.	DAM NO.	INS. DATE	USE	TYPE

AS BUILT INSPECTION

1	Location of Spillway and outlet	1	Elevations
1	Size of Spillway and outlet	1	Geometry of Non-overflow section

1 GENERAL CONDITION OF NON-OVERFLOW SECTION

1	Settlement	2	Cracks	1	Deflections
1	Joints	2	Surface of Concrete	1	Leakage ✓
	Undermining	1	Settlement of Embankment	1	Crest of Dam
	Downstream Slope	1	Upstream Slope	1	Toe of Slope

2 GENERAL CONDITION OF SPILLWAY AND OUTLET WORKS

	Auxiliary Spillway	2	Service or Concrete Spillway	1	Stilling Basin
2	Joints	2	Surface of Concrete	2	Spillway Toe
2	Mechanical Equipment	1	Plunge Pool	1	Drain

3	Maintenance	~~B~~ C	Hazard Class
3	Evaluation	5	Inspector

COMMENTS:

Surface concrete spalling

8/5/76 additional spalling + leakage

DAM INSPECTION REPORT
(By Visual Inspection)

Dam Number	River Basin	Town	County	Hazard Class*	Date & Inspector
935	Mohawk	Rome	Oneida	c	8/5/76 KDH B.C.

Type of Construction

- [ ] Earth w/concrete spillway
- [ ] Earth w/drop inlet pipe
- [ ] Earth w/stone or riprap spillway
- [x] Concrete
- [x] Stone
- [ ] Timber

Use

- [x] Water Supply canal
- [ ] Power
- [ ] Recreation
- [ ] Fish and Wildlife
- [ ] Farm Pond
- [ ] No Apparent Use-Abandoned

Estimated Impoundment Size

- [ ] 1-5 acres
- [ ] 5-10 acres
- [x] Over 10 acres

Estimated Height of Dam above Streambed

- [ ] Under 10 feet
- [ ] 10-25 feet
- [x] Over 25 feet ≈80'

Condition of Spillway

- [x] Service satisfactory
- [x] In need of repair or maintenance
- [x] Auxiliary satisfactory
- [x] In need of repair or maintenance

Explain: 4" cap coming off

Condition of Non-Overflow Section

- [ ] Satisfactory
- [x] In need of repair or maintenance

Explain: Concrete cracking small leakage.

Condition of Mechanical Equipment

- [x] Satisfactory ?
- [ ] In need of repair or maintenance

Explain:

Evaluation (From Visual Inspection)

- [x] No defects observed beyond normal maintenance
- [ ] Repairs required beyond normal maintenance

*Explain Hazard Class, if Necessary

Boonville Quadrangle - 114

NOTICE: After filling out one of these forms as completely as possible for each dam in your district, return it at once to the Conservation Commission, Albany.)

STATE OF NEW YORK

CONSERVATION COMMISSION

ALBANY

955 Mohawk

-A

DAM REPORT

6/15, 1915
(Date)

CONSERVATION COMMISSION,

DIVISION OF INLAND WATERS.

GENTLEMEN:

I have the honor to make the following report in relation to the structure known as the Delta Dam.

This dam is situated upon the Mohawk River (Give name of stream)

in the Town of Rome, Oneida County,

about 5 miles (State distance) from the ~~Village~~ or City of Rome.

The distance down (Up or down) stream from the dam, to the highway bridge (Give name of nearest important stream or of a bridge),

is about 750 ft. (State distance)

The dam is now owned by N.Y. State (Give name in full)

and was built in or about the year 1912, and was extensively repaired or reconstructed during the year ———.

As it now stands, the spillway portion of this dam is built of Concrete (State whether of masonry, concrete or timber)

and the other portions are built of concrete (State whether of masonry, concrete, earth or timber with or without rock fill)

As nearly as I can learn, the character of the foundation bed under the spillway portion of the dam is solid rock and under the remaining portions such foundation bed is solid rock.

DAM INSPECTION REPORT
(By Visual Inspection)

Dam Number	River Basin	Town	County	Hazard Class*	Date & Inspector
935	Mohawk	Rome	Oneida	C	8/5/76 KDH B.C.

Type of Construction

- [ ] Earth w/concrete spillway
- [ ] Earth w/drop inlet pipe
- [ ] Earth w/stone or riprap spillway
- [x] Concrete
- [x] Stone
- [ ] Timber

Use

- [x] Water Supply canal
- [ ] Power
- [ ] Recreation
- [ ] Fish and Wildlife
- [ ] Farm Pond
- [ ] No Apparent Use-Abandoned

Estimated Impoundment Size

- [ ] 1-5 acres
- [ ] 5-10 acres
- [x] Over 10 acres

Estimated Height of Dam above Streambed

- [ ] Under 10 feet
- [ ] 10-25 feet
- [x] Over 25 feet ±80'

Condition of Spillway

- [x] Service satisfactory
- [x] In need of repair or maintenance
- [x] Auxiliary satisfactory
- [x] In need of repair or maintenance

Explain: 4" cap coming off

Condition of Non-Overflow Section

- [ ] Satisfactory
- [x] In need of repair or maintenance Explain: Concrete cracking small leakage

Condition of Mechanical Equipment

- [x] Satisfactory ?
- [ ] In need of repair or maintenance Explain:

Evaluation (From Visual Inspection)

- [x] No defects observed beyond normal maintenance
- [ ] Repairs required beyond normal maintenance

*Explain Hazard Class, if Necessary

The total length of this dam is 980 feet. The spillway or waste-weir portion, is about 300 feet long, and the crest of the spillway is about 8 feet below the top of the dam.

The number, size and location of discharge pipes, waste pipes or gates which may be used for drawing off the water from behind the dam, are as follows: 4 waste pipes in bottom at point indicated — each 5 ft. in diameter.

(State briefly, in the space below, whether, in your judgment, this dam is in good condition, or bad condition, describing particularly any leaks or cracks which you may have observed.)

This dam is in very good condition. There are no leaks or cracks in it.

Reported by C W H Douglass,
(Signature)

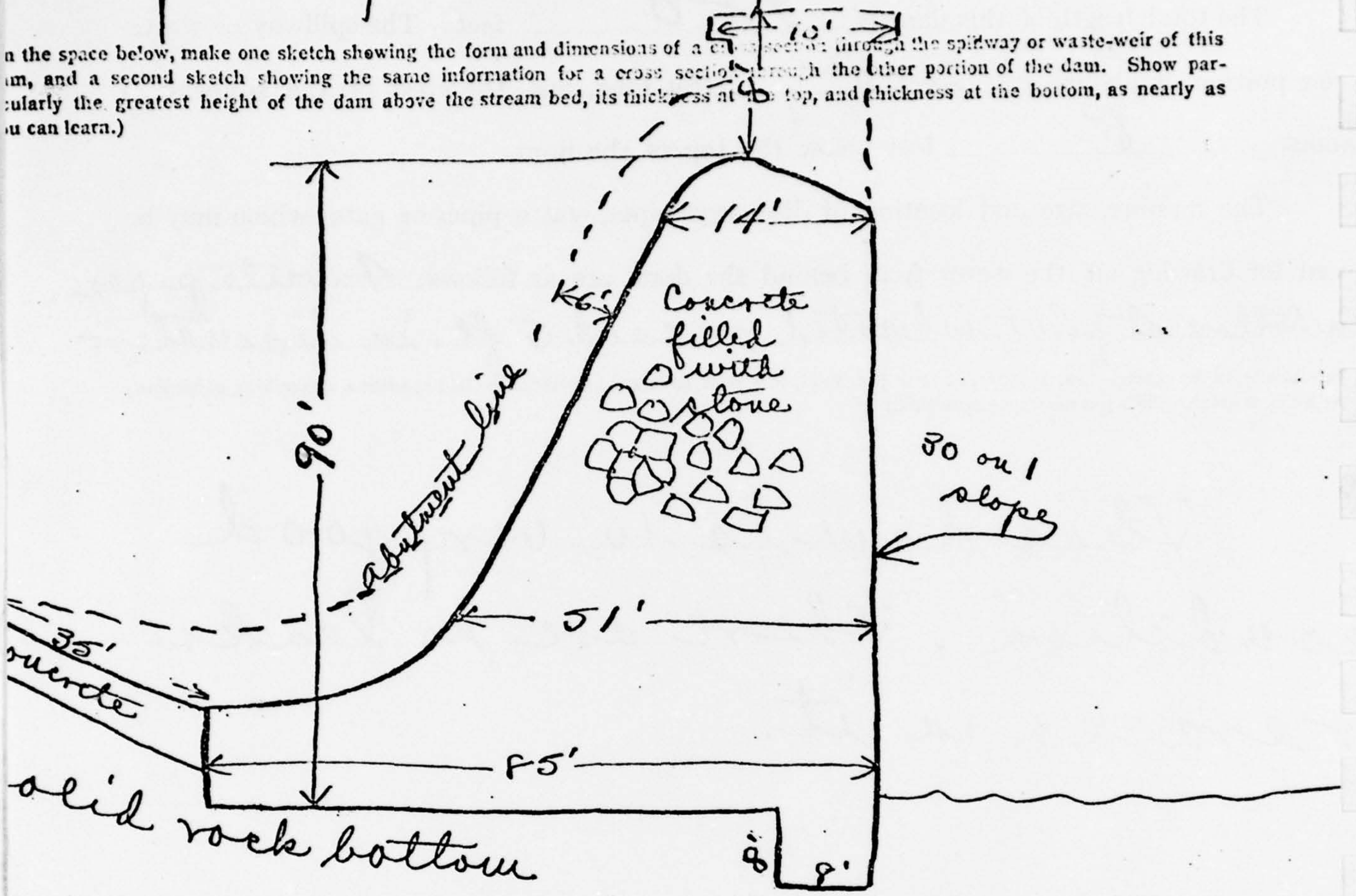
115 Standart St.,
(Address—Street and number, P. O. Box or R. F. D. route)

Syracuse, N. Y.
(Name of place)

(SEE OTHER SIDE)

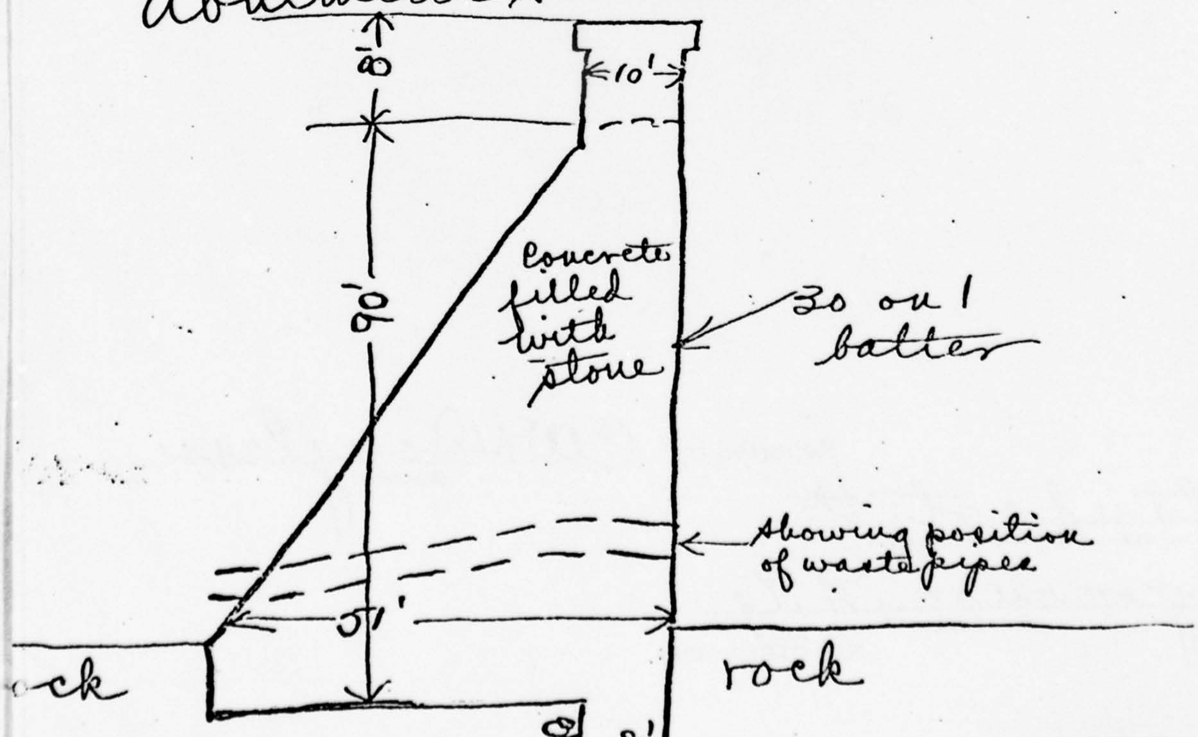
Spillway Section

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



In the space below, make a third sketch showing the general plan of the dam, and its approximate position in relation to buildings or other conspicuous objects in the vicinity.

Abutment section



THOS. H. WIGGIN
CONSULTING ENGINEER
50 CHURCH STREET
HUDSON TERMINAL BUILDING
ROOM 367 TEL. CORT. 7785 7786
NEW YORK

REPORTS. DESIGNS. SUPERVISION. VALUATION
WATER SUPPLY
SEWERAGE
CANALS
FLOOD CONTROL
TUNNELS
REINFORCED CONCRETE
INSPECTION OF SPECIAL MATERIAL
INVESTIGATION OF SPECIAL PROCESSES

NEW YORK June 7th, 1924.

Messrs R. M. Fraser,
A. F. Knight,
J. D. Black.

Committee of the Chamber of Commerce,
Rome, New York.

Gentlemen:-

In accordance with a letter dated March 8th and telegram dated March 14th, 1924 from Mr. Fraser of your Committee, I have made an investigation of the Delta Dam, located on the Mohawk River about 5 miles north of Rome and forming a part of the state Barge Canal feeder system and beg to report as follows:

The Delta Dam is a cyclopean masonry structure about 90' high in the deeper portions and about 1000 feet long. At the center is a spillway section 300 feet long. The dam impounds approximately 20 billion gallons of water, which is used to feed the summit level of the Barge Canal. The investigation was undertaken under the auspices of the Chamber of Commerce, owing to apprehension for the safety of the dam felt by citizens of Rome because the surface of the dam both upstream and downstream, but more particularly downstream, is disintegrated over considerable areas. The disintegrated concrete can be removed by pick or bar to a depth of 6" in places. The disintegration is apparently due in part to seepage of water through the dam (on the coping which also shows disintegration, moisture comes from rain or snow) and the freezing of the saturated concrete in cold weather. Even with seepage such action would not generally be nearly so-marked and the secondary cause is probably weakness and excessive porosity of the surface concrete. Many concrete structures, including a number of well-known dams show similar, though generally less marked deterioration. The attention of many prominent investigators is now focussed on Portland cement and its permanence is being questioned even where under cover of earth or water. However, important disintegration is seldom found where there is not exposure to moisture and frost.

The dam was inspected in company with all the members of your Committee on March 18th last and again on May 9th. A heavy crowbar was used at many points on the downstream face that could be reached from the ground and at some other points higher up which were reached from a short ladder. One bad spot on the upstream face of the easterly bulkhead portion was examined from a hanging ladder. The worst spots were found on the downstream face on the bulkhead section east of the spillway. Some of these places were 6 feet or so long and 4 feet or so in maximum height and the disintegrated material was easily removed to a depth of about 6 inches. The downstream face of the bulkhead portions of the dam to a height of 10 or 15 feet above the ground was rather generally covered with a shell of calcareous matter from 1/2" to 1" in thickness which could be scaled off in large sheets. This material was evidently deposited by water which came through the dam and represents soluble parts of the cement. The action is akin to that by which stalagmites are formed in limestone caves.

Beneath this shell the concrete was generally in fair condition. It was noted on the May 9th visit that flood waters which spilled over the dam between the March 18th and the later visit had washed away all the badly disintegrated concrete from the downstream face of the spillway, leaving many depressions.

Some of the soft material that was removed by the bar appeared to have more than the normal quantity of sand but most of it had plenty of stone and the stone seemed to be sufficiently solid. From this inspection it was concluded that the concrete would be able surely and for some years to come to carry its full expected stress out to within 6 or 8 inches of the surface and in the computations that have been made, it was assumed that the exterior 8" did not bear stress but was effective so far as weight is concerned.

The investigation has covered the following points:

1. Materials and workmanship in the dam and foundation conditions.
2. Adequacy of the design.
3. Damage that would be occasioned by failure.
4. Methods of repair.

In the report following these topics will be taken up in order, using the same numbers.

1. MATERIALS AND WORKMANSHIP AND FOUNDATION

The surface defects in this dam are evident to all who have examined it. By the use of the crowbar as previously described, it is found that the concrete is solid except for a thickness of 6" at the maximum and generally it is solid nearly to the surface. The solidity of the interior is indicated in the gate well which can be examined by removing iron covers west of the spillway. It is also noticeable that concrete in the upper part of the dam is generally sound except in parts of the coping and that concrete in the ornamental arches forming the cornice is particularly good, possibly owing to a greater proportion of cement in the concrete. The cornice is also too high up to be affected by leakage through the dam and is well sheltered besides.

The State Engineer's office has kindly furnished me with laboratory reports of sand and cement quoted below and several engineers who were present at times during the construction of the dam have described from memory their impressions.

Cement & Sand Tests - From Records of State Engineer

Cement: Dragon Portland.	Tensile strength of 1 : 3 briquettes. Lbs. per Sq. In.		Ratio to Std. Sand. %		Voids %	Loam %
	7 days	28 days	7 days	28 days		
1. Frenchville bank sand	165	253	76	91	37.0	4.7
2. Do washed	189	273	87	99	-	-
3. Standard sand	216	277	100	100		
4. Crusher dust	289	424	147	132	30.5	-
5. Do with sand, ratio 1:4	185	271	95	84	-	-
6. Standard sand	195	322	100	100		
7. Westernville bank sand	234	345	138	106	31.8	7.0
8. Do washed	193	317	113	95	-	-
9. Standard sand	170	324	100	100		

Coarse aggregate:- Crushed limestone from quarry near Booneville, N. Y.

From such information as I have gathered, the Frenchville sand seems to have been used. I examined the bank with you on May 9th and it is easy to understand references to lumps of clay which got into the sand. The sand itself seems to occur in clean bands and pockets but is covered and more or less interbedded with clay. It is stated that the sand was carefully inspected for clay but was not always satisfactory and that after some time washing at the pit was required. One statement was to the effect that a proportion of crusher dust was sometimes used but no convincing evidence of this is available and the contractor cannot recall that any was used.

I conclude that lines 1, 2 and 5 in the above table of tests represent the material probably used at various times. Two of the 7 day tests with standard sand and one of the 28 day tests are below the present requirements for strength of cement briquettes (viz 200 lbs. at 7 days and 300 lbs. at 28 days) but Portland cement of 2 or 3 decades ago was even less strong and made good masonry fully as often as the present product.

One engineer stated that the broken limestone used in the concrete was not so good as that used for the cyclopean blocks but had thinner beds and tended to make more dust. The pieces of stone found in the rotten concrete seemed hard and strong.

The dam was divided by vertical joints at right angles to the length of the dam into sections 30' long, with keys or dovetails between. These sections were filled alternately with concrete and carried up about 20' at a time. The photographs to which I have had access through the State Engineer and through engineers connected with the work, show this alternate concreting of sections and also show that the dam was brought to approximately one level for nearly if not all its length and then carried up again, saw tooth fashion, for another lift of about 20'. This method of construction provided for the minimum shrinkage cracks across the dam. Keys or dovetails at these vertical joints tend to prevent leakage and to make the dam act as a unit. Large cyclopean stones projecting across every main horizontal joint furnish not only shearing or anti-sliding strength, but some tensile strength as well.

From an excellent descriptive article on the Delta Dam by Emile Low, one of the resident engineers on the Barge Canal, (Engineering & Contracting June 19, 1912, pp. 681-88) the following points bearing on construction methods are briefed:-

(a) Masonry consists of 1:2½:5 concrete, rather soft, with 40% of large cyclopean stones imbedded. These latter were often over a cubic yard in size. They were scrubbed and washed thoroughly before being placed.

(b) Cyclopean stones were kept back 12" from faces of forms and 6" apart horizontally from each other. Layers of stones were placed so as to break joints horizontally and vertically.

(c) At horizontal joints cyclopean stones were left projecting so as to form a bond.

(d) In joining new work to old, the old surfaces were freed of dirt and laitance and washed with thin mortar.

(e) Concrete was mixed in batches containing 2½ bbls. of cement, i.e. about cu yds. of concrete. The mixer was made by Thos. Carlin's Son's Co. and was a 5 ft. cube rotated on a diagonal axis. The average daily output of concrete was about 170 cu. yds.

(f) Both concrete and cyclopean stones were placed by a double line of derricks. The stones were raised and lowered to "work up a bed in the fresh concrete".

(g) During cold weather, heated stones were used, "with steam under canvas to prevent freezing".

(h) The mixture was made "rather wet so that little tamping was required, but shovels were used to work the concrete down in narrow forms or between stones. The coarser aggregate was kept away from the forms by forks. On rock foundation the surface was cleared of all loose pieces, washed and wet cement or mortar scrubbed over it with brooms".

In discussion with engineers who were present during the building of the dam, several difficulties in construction have been brought out. It was mentioned that the slope of the form for the toe of the dam in connection with the attempt to get as many large stones out under this form as possible, caused the concrete between the form and the large stones to be less accessible and more likely to contain an excess of mortar and laitance. The difficulty of keeping a soft bed under the large stones was also mentioned and seepage through the dam under these stones thought to be a possible result. On the other hand another observer mentioned the concrete as being wet and the stones as being lowered into a sloppy matrix. A more permanent resident at the dam spoke of building up the stones locally because it was difficult to keep moist concrete under them if the concrete was spread out over the whole 30 foot section. This seems like a more probable condition than that of extreme sloppiness at the point of placing large stones and corresponds with the writer's observation on the large dams of the Catskill water system of New York City. Surfaces which are left for a little while become partly set and it is impracticable to embed stones unless fresh concrete is placed to sufficient depth to bury the unevennesses in the bottom of the stone. This difficulty is inherent in cyclopean masonry but has been overcome in many dams to the extent necessary to avoid leakage and the probability of leakage should be less than in the case of a rubble masonry dam where mortar has to be placed in every joint under and between stones.

On the whole it is difficult to account for the rather excessive seepage through the dam by any evidence that has been produced. The difficulty of avoiding excess of mortar and laitance in the concrete between the large stones and the sloping forms is a real one and was not encountered in most other cyclopean dams because these in general had either stone or concrete blocks for their exposed faces and these blocks were laid ahead of the concrete and took the place of wooden forms. It is possible that the quality of concrete in the downstream face may have been affected by this cause, accentuated by the use of too much water, which is now known to weaken concrete.

My investigation of causes of weakness in the concrete has been necessarily superficial. However, the evidence in hand and personal acquaintance with the high character of the engineers connected with the design and supervision of the dam and with the good reputation of the contractor leave no room for a shadow of doubt in my mind that the work was done with unusual care and faithfulness and that the defects were due to unknown causes such as are all too common in cement work. Thousands of dollars have been spent on studies of defects in concrete, frequently without disclosing convincing reasons. Investigations of the nature of Portland cement are being conducted by national and municipal agencies as well as by manufacturers' associations, foundations and technical societies. More are planned and the Delta Dam may well be included in examples to be studied.

Test Borings to be Made in Dam.

Since it is evident that a new face must be placed on the dam, it is perhaps not so necessary to know absolutely what is the cause of the present weakness but it is necessary to be assured that the weakness does not exist except near the outer surface and will not progress inward beneath the new faces. I have had several conferences with Col. Frederick S. Greene, Supt. of Public Works, Mr. Roy K. Fuller, Commissioner of Canals and Mr. Dwight B. La Du, State Engineer and Surveyor, with respect to these and other problems and it has been agreed that 3 core borings will be put down from top to bottom of the dam and at least 3 samples cut from the face of the dam in order to see the quality of the interior masonry and obtain some tests thereon. My present opinion is predicated on the assumption that these tests will show the interior concrete to be sufficiently sound for its purpose but until these tests have been completed, the conclusions reached in this report must be taken subject to that condition being found in the tests.

Foundations of Dam.

The foundations for this dam consist of level-bedded Hudson River shale and eye witnesses state that it disintegrates rapidly when uncovered. The stone which is now uncovered adjacent to the dam is very flakey and obviously unsuitable for a foundation, but I have no doubt that the rock under the dam is sufficiently solid for the moderate pressures required in this comparatively low dam. The borings previously mentioned will extend through the dam into the rock and will verify the quality of the rock as well as that of the masonry.

Assuming that the borings will show that the rock under the dam has remained sound, the most important question regarding foundation is whether the dam is adequately keyed into the rock so as to avoid danger of sliding which has figured in most of the limited number of failures of gravity dams that have occurred.

I have examined all the official and also such private photographs as were available and have also examined the official cross sections of excavation. These show that the dam is well keyed into the rock foundation in the whole of the bulkhead sections and in all but about 150 feet of the spillway section, viz. at the westerly end thereof. In this stretch of 150 feet there is practically only the roughness of the nearly level shale foundation and the cut off wall at the rear of the dam to prevent sliding. The apron which is 4 feet thick bears against only a shallow shoulder at the front. The mathematical significance of this defect is explained further along under Adequacy of Design. At this place it will only be stated that the friction of masonry against even the comparatively flat foundation furnishes sufficient resistance unless water pressure gets under the dam or penetrates the horizontal seams in the rock immediately below the dam. Assistance from adjacent sections that are more completely bonded adds to security but improvements in the front footing of this 150 foot stretch of spillway are desirable. The section of bulkhead dam at the gate house is particularly well footed by its own depth into rock and by the walls and floor of the blow-off pool.

The remaining question as to foundation is the permanence of the rock shoulder in front of the dam, particularly at the spillway. The apron and paving below seem to be still intact, judging from the downstream edge which is the only part projecting above the permanent water cushion pool, but no flood has occurred which has caused a depth of more than 1'9" of water over the spillway. Some time will come a flood causing a depth of 6' or more. The condition of the paving and of the rock beneath and downstream therefrom could be carefully examined when the repairs are made. The contract for repairs should be so drawn as to permit relaying part or all of the paving and using cement grout in the joints if found desirable.

Grouting Foundation.

This level bedded shale foundation, while generally tight, may contain some horizontal seams between layers. Reference is made in Mr. Lowe's article above mentioned to a small quantity of leakage collected into drains which were to be grouted. The Olive Bridge Dam of the Catskill Water Works had a somewhat similar foundation and it was found comparatively easy to force grout or liquid mortar down through a vertical 2" hole into the rock and in some cases up into other drill holes as much as 50' distant. I cannot find that any grouting was done in the foundation of the Delta Dam. The disadvantage in omitting this grouting would be that of possible upward pressure exerted under the dam in the level seams in the rock. This matter will be discussed more fully under Adequacy of Design. The best construction would call for a thorough grouting and then a system of drainage beneath the dam for the downstream 2/3 or so of its width so as to remove the possibility of upward pressure. Something can still be done to repair this omission if tests show it to be important and methods are taken up later on in the section entitled Methods of Repair.

2. ADEQUACY OF DESIGN

Perhaps the best way to show how the Delta Dam compares in design with other dams is by drawing several representative dams to the same scale and tracing them one over another on the same sheet. This has been done in Sheets R1 and R2 attached. Sheet R1 shows several typical "bulkhead" dams, that is, dams which are not used as overflows or spillways. These are all "gravity" dams, that is dams which owe their stability to weight alone without any assistance from arch section secured by building the dam on a horizontal curve or arch. It will be noticed that two of the bulkhead dams, namely #5 and #8, are materially lighter than the Delta Dam and #2 which is the so-called New Croton dam of the N. Y. City Water Supply, while much heavier at the top, has a lighter section lower down. The Croton dam is a very high dam and becomes much heavier at lower depths. The Sodom dam, #5 was built before stresses in dams were understood as they are now. It is in a rather narrow valley on very firm gneiss and while quite thin has never shown the least sign of distress. The Austin, Pa. dam, #8, failed by sliding on its foundation. Dam #3, the Olive Bridge dam of the Catskill System and Dam #4, the Croton Falls dam of the New York City Croton system are very much heavier than the Delta Dam, particularly at the tops, which are designed for roadways of liberal width and to withstand heavy ice pressure. Dam #7 which is of a design commonly used by a very large public corporation is of about the same weight as the Delta Dam. In general, it may be said that the Delta Dam has about the same section as would be found often in private or dividend-producing enterprises but is lighter than most of the large modern public dams.

Drawing R2 shows a similar comparison for spillway sections. In this case the Delta Dam down to a depth of 75' is considerably lighter than any other dam of which I have record except #2, the Turlock dam which is heavier at the top but about the same as Delta at depth 75 feet. Another engineer who recently made a similar comparison has informed me that he also was unable to find a lighter section, although there are doubtless lighter ones among the many hundreds of unrecorded dams. The spillway section of Delta dam, however, is of about the same weight as the bulkhead section and would have about the same computed stability in spite of the fact that it is customary to make spillway sections heavier than bulkhead sections. One of the spillway dams shown on R2, namely #6, at Austin, Texas, failed by sliding due to the solubility of its foundation which was limestone.

Computations as to Stability of Delta Dam.

I have made some computations as to stability of and stresses in the Delta dam and these are shown on Sheets R3, R4 and R5. Since making these computations I have had access to computations made by the Barge Canal engineers and find a close agreement.

Sheet R3 shows the stresses in the bulkhead section of the Delta dam on two assumptions, namely (a) that the water pressure does not have access beneath the dam and (b) that the water pressure does have access beneath the dam and acts on 2/3 of the area with an effect diminishing from full head at the upstream edge or heel of the dam to tail water head at the downstream edge or toe of the dam. It will be seen from this diagram that the so-called line of resistance comes within the middle third assuming no water pressure beneath the dam and it comes a short distance outside of the middle third if water pressure is assumed to penetrate beneath the dam. The vertical stresses in either case are not particularly high, namely about six tons per square foot compression at the toe and less than 1 ton tension at the heel assuming water pressure beneath the dam.

Diagrams R4 and R5 give similar computations for the spillway section but in the case of R5 the investigation has been extended to cover ice pressure and shows that the dam would be overstressed if an ice pressure of 40,000# per linear foot could be exerted. This question of ice pressure is one that has never been satisfactorily settled by engineers. Adequate provision for ice pressure results in the very massive design shown for the Ashokan Dam on Sheet R1. There are probably 100 times as many dams that do not provide for ice pressure as there are dams which do provide for ice pressure. Among municipal engineers the reason for providing for ice pressure is generally that the dam impounds a very large body of water above a populus district and that this extra insurance is justified to prevent a shadow of doubt as to loss of life and of valuable property. The Delta dam impounds a very large body of water. The effect of a break in this dam on the City of Rome is take up hereinafter under the heading "Damage that would be occasioned by Failure". In this place I will limit myself to the statement that heavy ice pressure is not likely to occur at the Delta Dam on account of the shape of the reservoir but that the policy (which public documents show was in the minds of the designers) of draining the reservoir so that it is 40 or 50 feet below the crest of the dam through the winter is certainly a commendable one.

For the case with upward water pressure even without ice pressure, the ratio of the resultant horizontal to vertical force reaches the high value of about .85 and calls for very good slide-resisting bonding at all construction joints and at the foundation. Projecting cyclopean stones seem by the photographs to afford excellent bond at construction joints. Where the foundation is excavated below the surface of the rock, the bonding with the foundation is greatly aided by a bearing against the front of the excavation. The extent to which this aid is needed in the case with upward water pressure may be estimated by assuming that the total sliding force of 231,000 lbs. per linear foot of dam is resisted by a combination of 3 forces described below:

	Lbs.
(a) By friction on the foundation assumed very conservatively as 50% of the net downward weight or pressure. 50% of 273,000	137,000
(b) By bearing of rear cut-off wall against rock at front of cut-off trench. If it were assumed that water pressure against the dam for a height equal to width of cut-off trench were thus resisted the aid would be	26,000
	163,000

(c) Remainder of sliding force, or 231,000 - 163,000 = 68,000 lbs. may be assumed to be resisted by bearing of dam against front of excavation or against intermediate shoulders in the rock. If unit compression of concrete against rock is taken as 200 lbs. per sq. in. or 28,800 lb. per sq. foot, then a total depth of 2.4 feet of shoulder is needed for 68,000 lbs.

It has been previously explained under section entitled Foundation of Dam that the rock shoulders are adequate (generally at least twice the 2.4' depth above noted) except for a stretch of 150 feet in the westerly end of spillway. In this stretch, if under pressure should exist to the extent computed on R3, R4 and R5, an increased friction factor combined with aid from adjacent better supported sections would be called into action and would doubtless prevail though a greater margin of safety should be provided in so important a dam..

The necessity of preserving the rock downstream which prevents the dam from sliding has been explained in the section entitled Foundations of Dam.

It may be concluded that in design the Delta dam follows usual commercial practice in its partial provision against uplift pressure and its absence of provision against ice pressure. It is less conservative than most of the modern large municipal and national dams. The westerly 150 feet of spillway is still less conservative. The deterioration has not yet materially affected the stability of the dam but would become a factor if not checked.

3. DAMAGE THAT WOULD BE OCCASIONED BY FAILURE OF THE DELTA DAM

As a matter of completeness and because engineering judgment in the design and construction of any dam must be affected by such considerations, I have made a brief investigation of the probable damage that would result from a failure of the Delta dam. The dam is constructed in sections 30' long. The least failure of importance that could be conceived as happening would be for three 30' sections to move by partial tipping followed by sliding in the manner that occurred in the case of the Austin dam on the Colorado River in Texas and that other dam at Austin, Pennsylvania. Such an event would occur, if at all, in a time of maximum flood. There are approximately 137 square miles of water shed upstream from the Delta dam and a rainfall of 10 inches in 30 hours such as occurred in New Jersey and Southern New York in 1903 might easily cause a depth of 7' or more over the spillway in spite of the great holding back power of the Delta reservoir. At this time the reservoir which has a full level capacity of about 2,750,000,000 cu. ft. or 20.6 billion gallons would be holding a considerably larger quantity of water, namely about 3,600,000,000 cu. ft. or 27 billion gallons.

A rough computation made by taking cross sections from U. S. Geological Sheet of the stream below the dam, indicates that water would escape through a 90' opening in the dam at the rate of about 180,000 cu. ft. per second. A wall of water about 10' high would advance toward Rome probably at the rate of about 10 miles an hour and after the channel were entirely filled would continue to flow for several hours with a velocity of 6 or 8 miles an hour and with a depth of 10 to 15' in the river north of Rome. The bridge opening under Dominick Street has only about 1,000 sq. ft. of opening as compared with about 15,000 sq. ft. which would be required to carry 180,000 cu. ft. per second at the velocity which it would be likely to attain. The water would therefore back up north of Dominick Street until it overflowed that street for a length of half a mile or so and a depth of 6 or 8 feet. Topography is not available to determine the exact course of the water but the lower parts of the main city, the new developments to the East and the factories south of East Dominick Street would suffer greatly from water escaping West over the divide toward Wood Creek and South and East toward the Canal. The canal section is not large enough to contain such a flood and would be submerged but the Mohawk valley would carry the flood off to the east without excessive depth. There would probably be serious loss of life, greater or less depending on the time of day or of night at which the break occurred.

The lesson that I would draw from this study is that the Delta dam, impounding as it does a very large quantity of water, and located so that a failure would have such serious consequences, should be repaired thoroughly so as to make its factor of safety a little better than it was when originally constructed. In the winter time, as has already been explained, the practice of drawing down the reservoir somewhat, which has the effect of preventing any considerable ice pressure, should be continued as an invariable policy or else reliable means adopted to keep the ice cut away from the dam.

4. METHODS OF REPAIR

The repairs to the Delta dam should be of such a nature as to accomplish the following purposes:-

(a) The cutting off of any seepage between the layers of level-bedded shale under the dam.

(b) The drainage of the rock foundation under the down-stream portion of the dam so as to eliminate upward water pressure so far as practicable.

(c) Increase in the provisions for resisting sliding of the westerly 150 feet of spillway section.

(d) Strengthening, to the extent found desirable after unwatering and inspecting, of the apron protecting the rock in front of the spillway.

(e) The cutting off of seepage through the dam in the parts visible above ground in order both to eliminate the saturation of the surface concrete which has caused disintegration by freezing and to reduce any upward pressure that may exist within the joints. The stoppage of this seeping is particularly necessary in connection with placing repair masonry mentioned below.

(f) The removal of unsound masonry from the exposed faces of the dam. It is assumed that very little unsound masonry exists below the ground line and below the low water level in the rear of the dam.

(g) The placing on upstream and downstream faces of the dam of a sufficiently substantial layer of new masonry well bonded with the old masonry to avoid future disintegration and to increase somewhat the weight and factor of safety of the structure.

Some further explanations of the methods suggested for accomplishing the purposes above enumerated are given below under the same letters.

(a) Cutting Off Seepage Under Dam.

As previously explained in the report, grouting is generally gone in the foundation of a dam to cut off the seepage under the dam and also to reduce the possible upward pressure tending to decrease the stability of the dam in the way described under Design. Core borings can be made from the top of the dam through which grouting of the foundation can be done. The 3 test holes will indicate whether grouting of the foundation is desirable, though if these 3 result in a negative conclusion a few more holes should be tried as a verification when the contract repair work is started. It is possible that these same holes may also be used for grouting joints in the dam above the foundation though grouting from the face may be more effective. A special packer such as has been used in other cases can be employed in these borings in order to limit the grouting below any desired point. A hole every 10 feet along the dam ought to be sufficient.

(b) Draining Rock Foundation.

To still further reduce possibility of upward pressure holes may be drilled into the foundation at a distance of about 1/3 the width of the dam from the upstream edge. These holes could be put in on a slope from the downstream face of the dam and would limit the upward pressure to the height at which the hole was started in this downstream face. Holes for relieving pressure in rock have been used successfully in the Catskill water works in accordance with the writer's design.

(c) Increase in Slide-Resisting Provisions for the Westerly 150 feet (Approx.) of Spillway.

Study will be needed to determine the best method. The easiest way will be to excavate a trench in rock and put in a heavy block of concrete just downstream from the toe of the concrete apron. The surface of the new concrete would be sloped up to fit the profile of the apron paving and would replace a part of this paving.

(d) Strengthening Apron.

Official photos and cross sections indicate paving apparently ungrouted laid partly on earth or fill. After unwatering the pool and inspecting the rock beneath and downstream from the apron at enough places to be assured that the rock has not softened or become eroded in a serious way, it may be found sufficient to relay all disturbed or undermined paving and grout the joints with cement mortar. A 7 foot flood should be in mind in inspecting and repairing this apron. Inspection after every flood is a particularly important duty considering the laminated nature of this shale and its tendency to go to pieces when exposed. It is expected that the continued wetness of the stream bed has prevented deterioration such as has taken place in the rock which has been exposed to air.

(e) Cutting Off Seepage Through Dam.

The test holes which have been previously mentioned in the report will be used not only to test quality of masonry but also to test the possibility of grouting in such a way as to intercept the seepage which is now coming through the dam in numerous places. If this solution is found feasible, which is not at all sure, the holes already described under paragraph (a) above can also be used for this purpose. In any case it should be possible after preparing the upstream face of the dam to caulk and grout any visible joints and waterproof any porous places that will be exposed. Making the upstream face of the dam tight is necessary, otherwise any new masonry placed downstream is likely to retain water behind it and be disturbed by freezing. Furthermore, the old masonry may keep on deteriorating behind the new if it is wet and freezes.

(f) Removing Unsound Masonry.

The unsound masonry can be removed easily by air chisel, possibly by the aid of hose streams before and after using the chisel. The total quantity will be very considerable. There is nothing difficult however about the operation and everybody agrees that this rotten concrete must be removed down to sound material before placing new material for the outside surface. The cleanness of the downstream face of the spillway as observed after the spring overflow suggests that strong hose streams may be used to advantage in removing most of the rotten concrete.

(g) New Masonry on Upstream and Downstream Faces.

In order that there shall be no danger of a repetition of the frost action resulting in scaling off of the surface masonry I believe the new surfacing should have a substantial thickness and should be bonded to the old masonry not alone by grooves but also by steel anchors grouted into drilled holes in the old masonry. I have in mind a thickness of about 2 feet though detailed study may indicate the desirability of making the new masonry thicker than 2 feet at the bottom and perhaps it may be reduced to 15" or 18" at the top. There is a problem in keeping the pleasing appearance of the dam where the new masonry terminates at the top. This is particularly true on the downstream face near the top where there is an ornamental cornice with arches.

On the downstream face at least, except near the ends of the dam where the head is low, the new masonry should be footed in sound rock (or masonry in the spillway) and bonded by shoulders in order to secure the advantage in stability resulting from the increased width of the dam. Below the ground line and pool level on the downstream face of the dam and below the winter water level on the upstream face of the dam, the new masonry can without question be of concrete and sufficient cement should be used to be sure that there will be no further disintegration. A mixture of 1: 2: 4 or perhaps 1: 2½: 4½ approximately would be satisfactory, the exact proportions of sand and stone to be determined so as to give the densest practicable mix. Above the levels previously mentioned, it would seem best to use stone masonry of a permanence equal to the very good stone which was used for the cyclopean blocks and also for the paving which is visible downstream from the dam and shows absolutely no disintegration of even the sharpest corners of the stone blocks.

Discussing further the kind of masonry to be used for the lower parts of the repairs, there will be an advantage in using concrete masonry below ground on the downstream face even if stone could be had for the same price because the concrete will bond more strongly with the present masonry and grooves can be cut to insure the shearing strength between new and old masonry necessary to make the two parts act together in producing stability of the structure.

Referring further to the artistic question of joining new and old masonry at the top of the dam, a thoroughly satisfactory solution can only be worked out on the drafting board which will of course be done by the engineering force under the State Engineer and Surveyor. It is the intention of the State authorities as expressed to me to prepare the designs and contract for the repair work after the test borings have been made. It will then be possible to go to the Legislature next winter and proceed with the repairs the following season.

<u>Work Should be done in Summer if Possible.</u>

There was some discussion between Supt. of Public Works Greene, State Engineer and Surveyor La Du and the writer as to whether the repairs could properly be done in the winter in order that the reservoir might be used for feeding the Canal without interruption during the season of navigation. There is no question that it is physically possible to make the repairs in winter but this would add at least $75,000 I think and perhaps more to their cost since there would have to be a steam heated housing over all parts where masonry was being placed and adequate protection would have to be left in place until the masonry was well set to avoid a repetition of the disintegration.

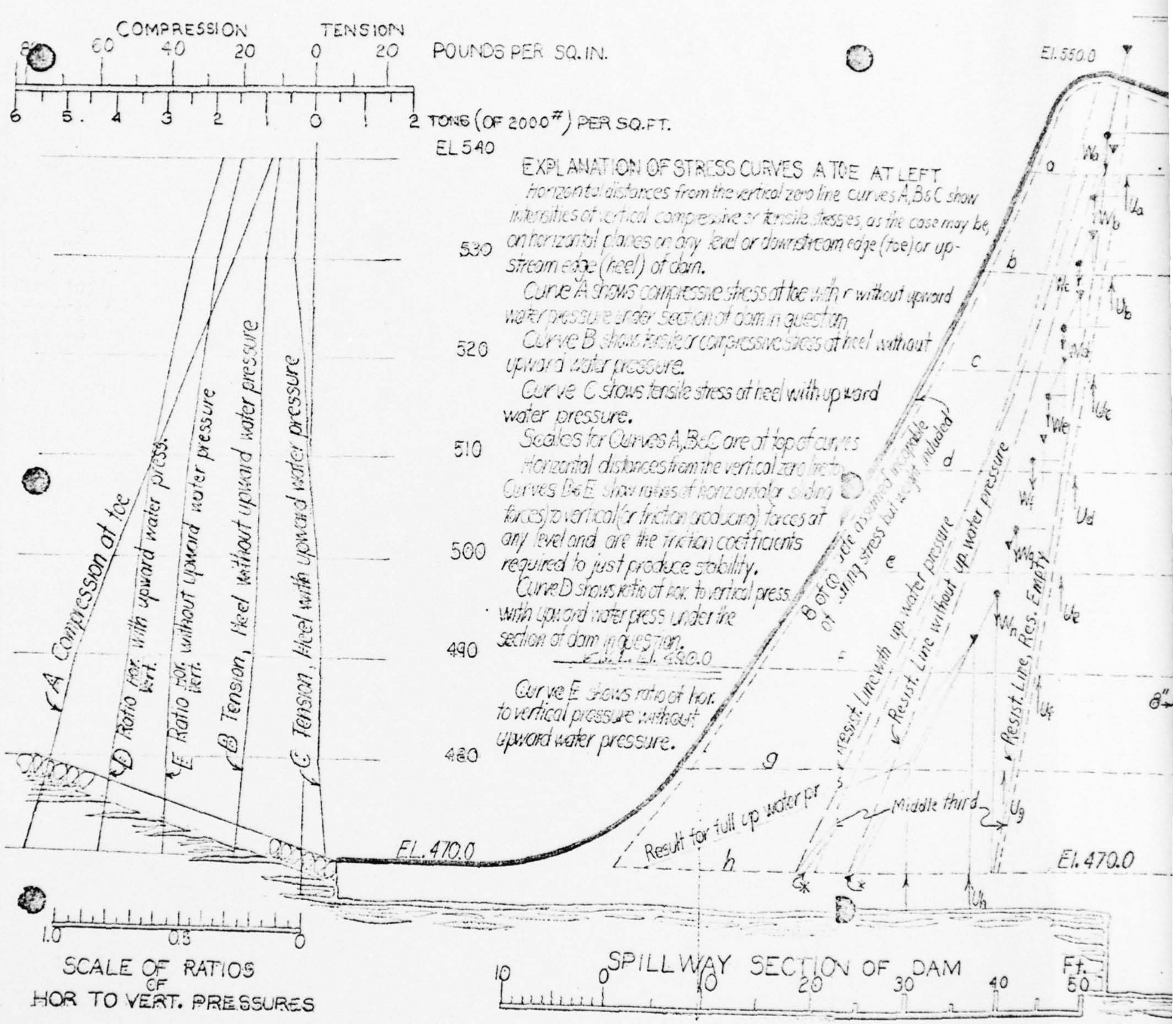
I am inclined to think that a computation of the amount of water required for the canal under present traffic conditions will show that the summit level can be adequately fed from the other available sources. It may be necessary to curtail to some extent the freedom with which pleasure boats can pass the locks; that is it may be necessary to pass them in groups at certain times and not whenever they come along. It is certainly worth considerable care and planning to avoid the expense and constructional disadvantages of making those repairs during freezing weather.

Acknowledgements.

In conclusion I would like to make acknowledgement of the cooperation afforded me by your Committee and by engineers formerly connected with the Barge Canal; also of the courtesies extended on all my visits by Superintendent of Public Works Frederick S. Greene, State Engineer and Surveyor Dwight B. La Du and Commissioner of Canals Roy K. Fuller.

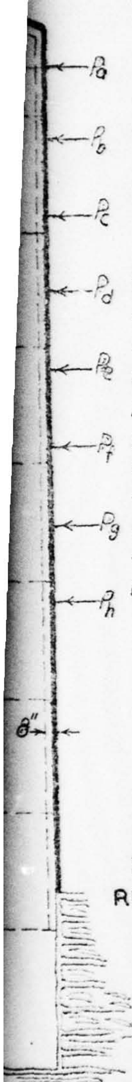
Rome Chamber of Commerce Should Maintain Interest.

The proposed program of Supt. Greene and State Engineer La Du



COMPRESSION
TENSION
POUNDS PER SQ.IN.
TONS (OF 2000#) PER SQ.FT.
EL 540
EXPLANATION OF STRESS CURVES AT TOE AT LEFT
Horizontal distances from the vertical zero line curves A, B & C show intensities of vertical compressive or tensile stresses, as the case may be, on horizontal planes on any level or downstream edge (toe) or upstream edge (heel) of dam.
Curve A shows compressive stress at toe with or without upward water pressure under section of dam in question.
Curve B shows tensile or compressive stress at heel without upward water pressure.
Curve C shows tensile stress at heel with upward water pressure.
Scales for Curves A, B & C are at top of curves
Horizontal distances from the vertical zero line to Curves D & E show ratios of horizontal (or sliding forces) to vertical (or friction producing) forces at any level and are the friction coefficients required to just produce stability.
Curve D shows ratio of hor. to vertical press. with upward water press under the section of dam in question.
Curve E shows ratio of hor. to vertical pressure without upward water pressure.
A Compression at toe
D Ratio hor. with upward water press. vert.
E Ratio hor. without upward water pressure vert.
B Tension, Heel without upward water pressure
C Tension, Heel with upward water pressure
El. 550.0
El. 470.0
EL. 470.0
Resist. Line with up. water pressure
Resist. Line without up. water pressure
Resist. Line, Res. Empty
Middle third
Result for full up water pr
SCALE OF RATIOS OF HOR TO VERT. PRESSURES
SPILLWAY SECTION OF DAM
Ft.

W.L. El. 556.0



FORCES ACTING ON DAM

~ Forces $P_a, P_b \ldots P_j$ are the resultant water pressures on the sections of dam above the respective joints indicated by the subscript letters a, b... j.

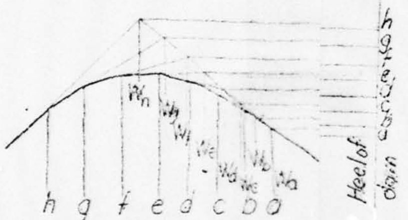
~ Forces $W_a, W_b \ldots W_j$ are the resultant weights of all the sections of dam above the respective joints indicated by the subscript letters a, b... j.

~ Forces $U_a, U_b \ldots U_j$ are the uplift water pressures on the sections of dam above the respective joints indicated by the subscript letters a, b,... j.

~ Uplift pressure is assumed to be uniformly decreasing from heel to toe varying from reservoir head on upstream side at heel to tail water head on downstream side at toe, and is applied to $\frac{2}{3}$ of the area of joint.

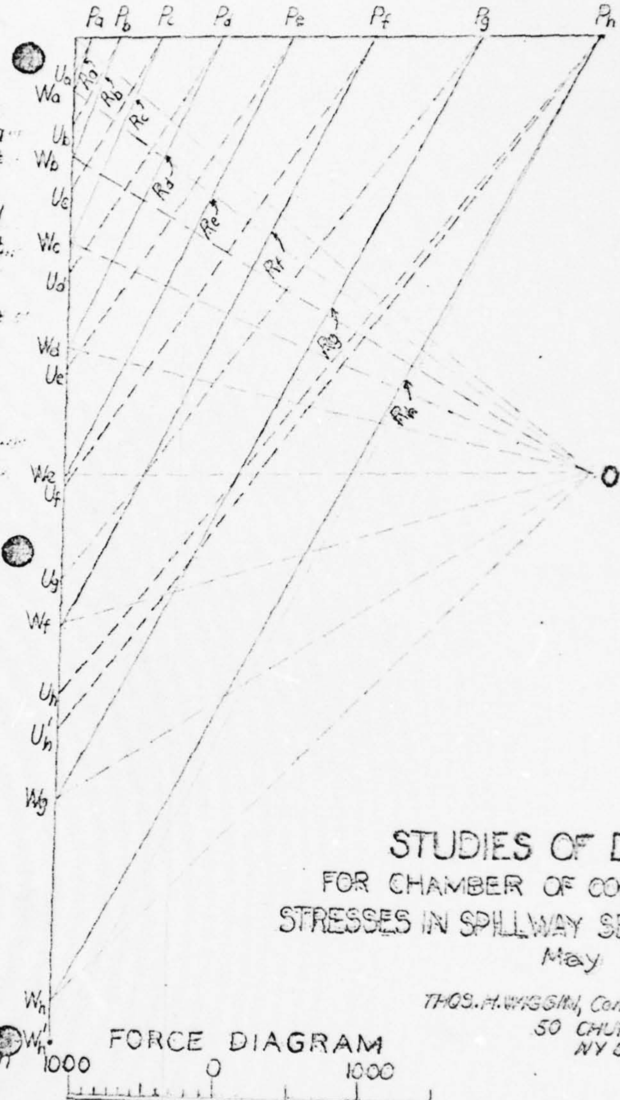
~ All forces are in terms of unit weight of water or 62.5 pounds.

~ Weight of masonry assumed to be 150# per cu. ft., specific gravity 2.4.



POINTS OF APPLICATION OF RESULTANT WEIGHTS $W_a, W_b \ldots W_j$

* Resultants using points W_h' & U_h' on the force diagram which are based on assumption that masonry weighs 155 lbs. per cu. ft.



FORCE DIAGRAM

1000 0 1000

UNIT = Weight of 1 cu. ft. of water = 62.5 lbs.

STUDIES OF DELTA DAM

FOR CHAMBER OF COMMERCE, ROME, N.Y.

STRESSES IN SPILLWAY SECTION - NO ICE PRESSURE

May 1924

THOS. H. WIGGIN, Consulting Engineer
50 CHURCH ST.,
NY CITY

AD-A069 472 KIMBALL (L ROBERT) AND ASSOCIATES EBENSBURG PA F/G 13/2
NATIONAL DAM SAFETY PROGRAM. DELTA DAM (INVENTORY NUMBER NY 6),--ETC(U)
JUN 78 R J KIMBALL DACW51-78-C-0025

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2 OF 2

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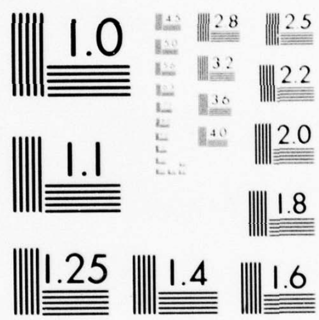
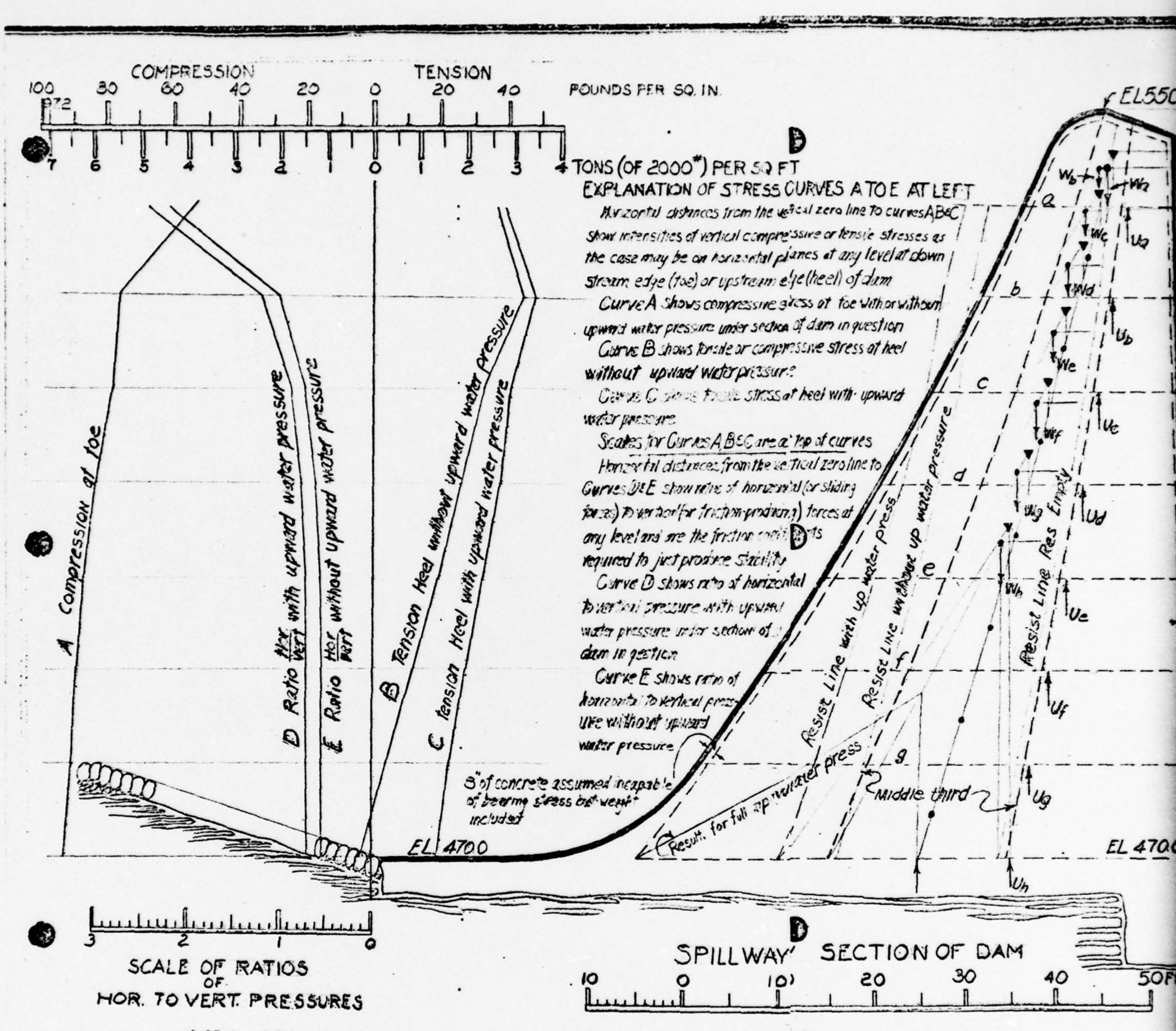
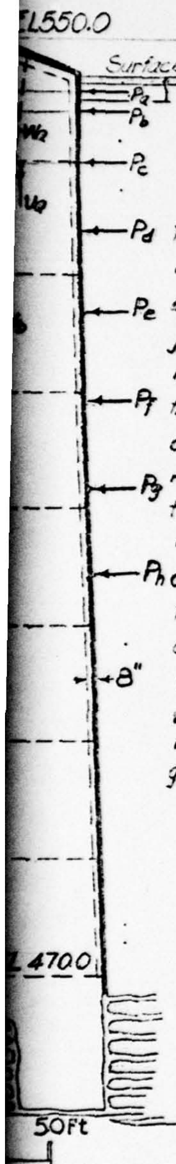


MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A



COMPRESSION
TENSION
100 80 60 40 20 0 20 40
POUNDS PER SQ. IN.
7 6 5 4 3 2 1 0 1 2 3 4
TONS (OF 2000#) PER SQ FT
EXPLANATION OF STRESS CURVES A TO E AT LEFT
Horizontal distances from the vertical zero line to curves A B & C show intensities of vertical compressive or tensile stresses as the case may be on horizontal planes at any level at down stream edge (toe) or upstream edge (heel) of dam
Curve A shows compressive stress at toe with or without upward water pressure under section of dam in question
Curve B shows tensile or compressive stress at heel without upward water pressure
Curve C shows tensile stress at heel with upward water pressure
Scales for Curves A B & C are at top of curves
Horizontal distances from the vertical zero line to Curves D & E show ratio of horizontal (or sliding forces) to vertical (or friction-producing) forces at any level and are the friction coefficients required to just produce stability
Curve D shows ratio of horizontal to vertical pressure with upward water pressure under section of dam in question
Curve E shows ratio of horizontal to vertical pressure without upward water pressure
A Compression at toe
D Ratio Hor/Vert with upward water pressure
E Ratio Hor/Vert without upward water pressure
B Tension Heel without upward water pressure
C tension Heel with upward water pressure
8" of concrete assumed incapable of bearing stress but weight included
EL 4700
EL 470.0
EL 550
a
b
c
d
e
f
g
Resist Line with up water press
Resist Line without up water pressure
Resist Line Res Empty
Result. for full up water press
Middle third
3 2 1 0
SCALE OF RATIOS OF HOR. TO VERT. PRESSURES
SPILLWAY SECTION OF DAM
10 0 10 20 30 40 50 FT



FORCES ACTING ON DAM

⌐ Ice pressure is assumed at 4000 # per lineal foot applied one foot below the surface of the ice

⌐ Forces $P_a, P_b \ldots P_h$ are respective resultants of the ice press and the water pressure on the section above the respective joints indicated by the subscript letters a b etc

⌐ Forces $W_a, W_b, \ldots W_h$ are the resultant weights of all the sections of dam above the respective joints indicated by the subscript letter

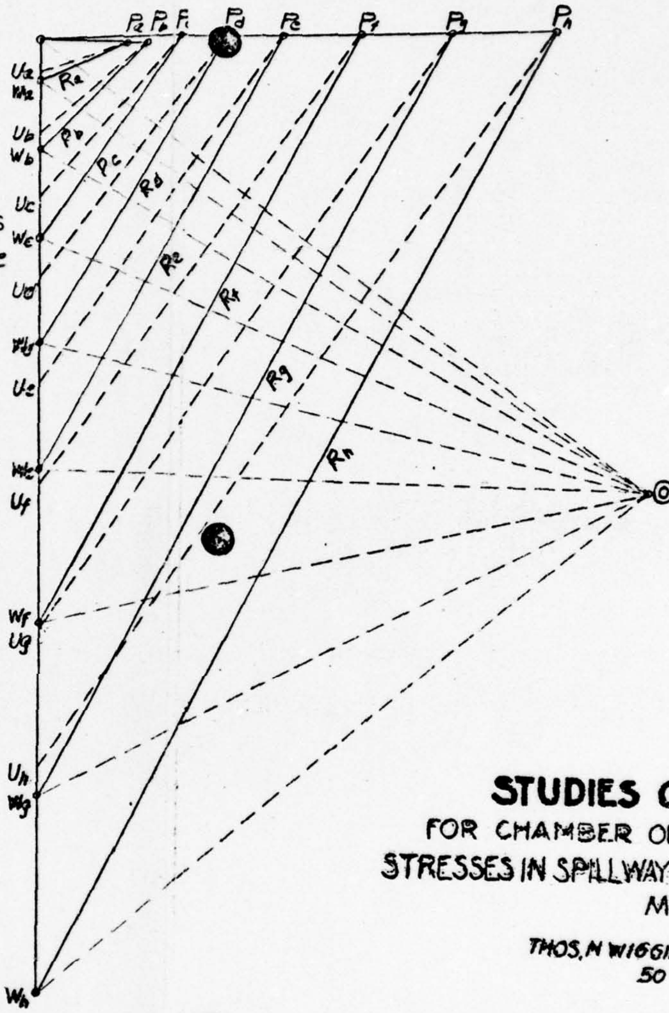
⌐ Forces $U_a, U_b \ldots U_h$ are uplift water pressure on the section of dam above the joint indicated by the subscript letter

⌐ Uplift pressure is assumed to be uniformly decreasing from head to toe varying from [illegible] head at the heel to 0 at the toe and is applied to two thirds of the surface

⌐ All forces are in terms of the unit weight of water of 62.5 pounds

⌐ Weight of masonry assumed to be 150# per cu ft, specific gravity 2.4

POINTS OF APPLICATION OF RESULTANT WEIGHTS $W_a, W_b \ldots W_h$



FORCE DIAGRAM

1000 0 1000 2000 3000

UNIT = weight of 1 cu ft of water = 62.5 lbs.

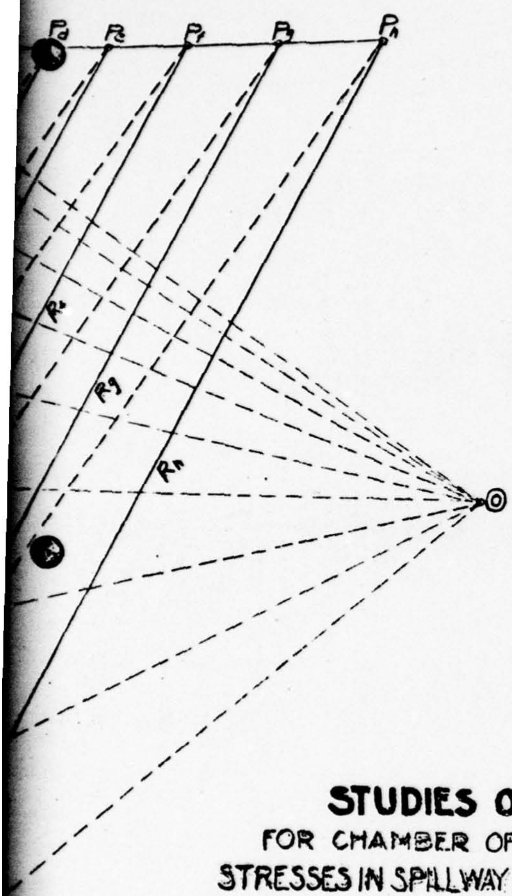
STUDIES OF DELTA DAM

FOR CHAMBER OF COMMERCE, ROME

STRESSES IN SPILLWAY SECTION WITH ICE P

May 1924

THOS. N WIGGIN Consulting Engineer
50 CHURCH ST.
NY CITY



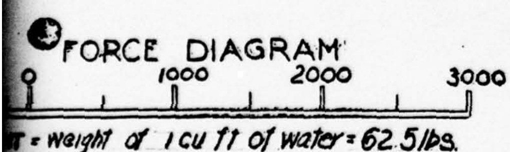
STUDIES OF DELTA DAM

FOR CHAMBER OF COMMERCE, ROME, NY

STRESSES IN SPILLWAY SECTION WITH ICE PRESSURE

May 1924

THOS. H WIGGIN Consulting Engineer
50 CHURCH ST.
NY CITY



1

FORCES ACTING ON DAM

Forces $P_b, P_c, \ldots P_j$ are the resultant water pressures on the sections of dam above the respective joints indicated by the subscript letters b, c … j

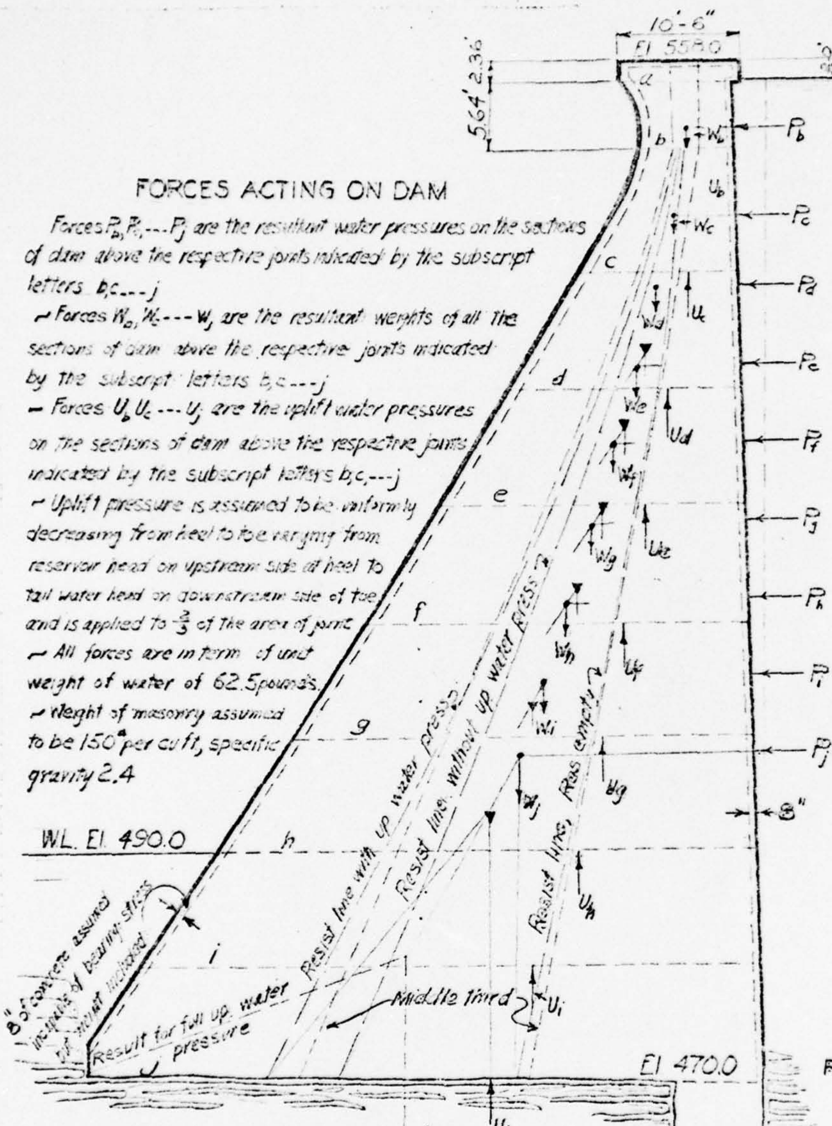
– Forces $W_a, W_c \ldots W_j$ are the resultant weights of all the sections of dam above the respective joints indicated by the subscript letters b, c … j

– Forces $U_b\ U_c \ldots U_j$ are the uplift water pressures on the sections of dam above the respective joints indicated by the subscript letters b, c, … j

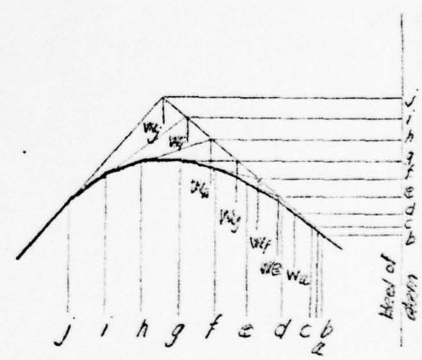
– Uplift pressure is assumed to be uniformly decreasing from heel to toe varying from reservoir head on upstream side at heel to tail water head on downstream side of toe and is applied to $\frac{2}{3}$ of the area of joint

– All forces are in term of unit weight of water of 62.5 pounds.

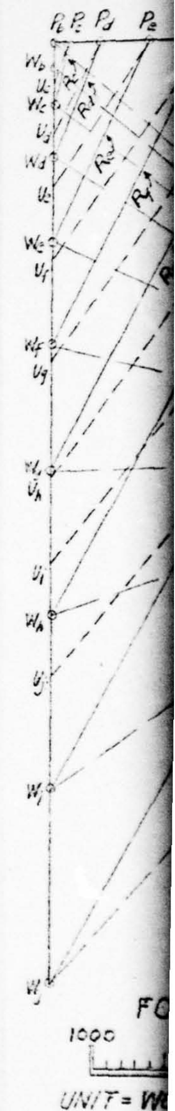
– Weight of masonry assumed to be 150# per cu ft, specific gravity 2.4

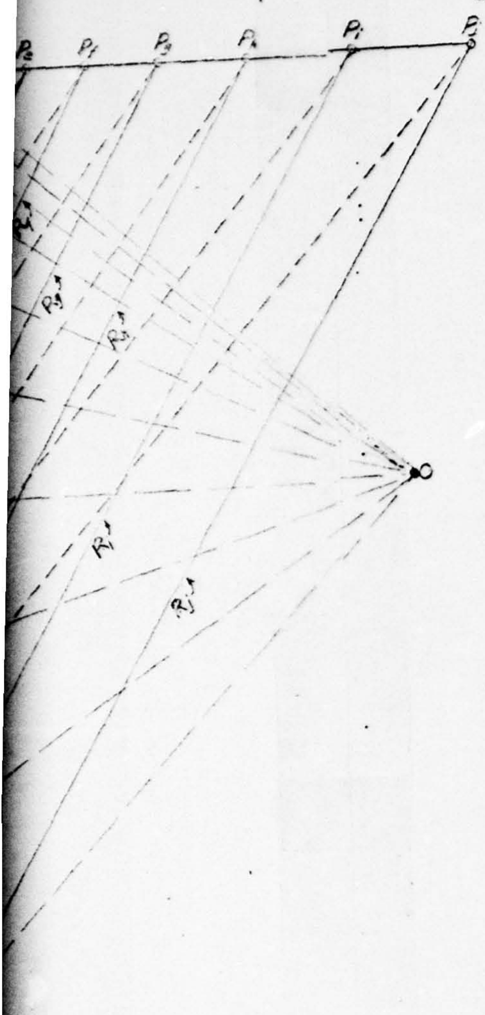


BULKHEAD SECTION OF DAM

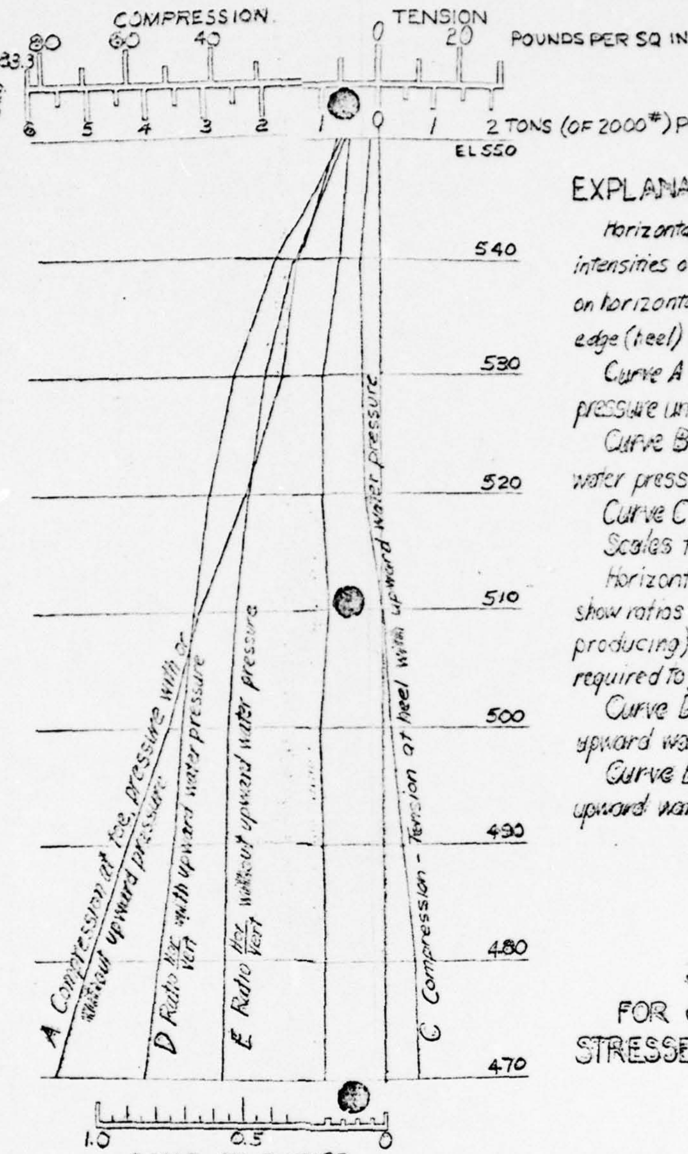


POINTS OF APPLICATION OF RESULTANT WEIGHTS $W_a, W_b \ldots W_j$





FORCE DIAGRAM
0 1000 2000
T = weight of 1 cu.ft of water = 62.5 lbs



SCALE OF RATIOS OF HOR TO VERT PRESSURES

EXPLANATION OF STRESS CURVES A TOE AT L

Horizontal distances from the vertical zero line to curves A, intensities of vertical compressive or tensile stresses as the on horizontal planes at any level at downstream edge (Toe) edge (heel) of dam.

Curve A shows compressive stress at toe with or without pressure under the section of dam in question

Curve B shows tensile or compressive stress at heel wi water pressure.

Curve C shows tensile stress at heel with upward water p

Scales for Curves A, B & C are at top of curves.

Horizontal distances from the vertical zero line to Cur show ratios of horizontal (or sliding forces) to vertical (o producing) forces at any level and are the friction coe required to just produce stability.

Curve D shows ratio of horizontal to vertical pressu upward water pressure under the section of dam in ques

Curve E shows ratio of horizontal to vertical press upward water pressure.

STUDIES OF DELTA DAM

FOR CHAMBER OF COMMERCE, ROME, N

STRESSES IN BULKHEAD SECTION-NO ICE PRE

May 1924

THOS. H. WIGGIN, Consulting Engineer
50 CHURCH St.,
N.Y. CITY.

3

S CURVES A TOE AT LEFT

rtical zero line to curves A, B & C show
 or tensile stresses as the case may be
 at downstream edge (toe) or upstream

stress at toe with or without upward water
am in question
mpressive stress at heel without upward

 heel with upward water pressure
re at top of curves.
 vertical zero line to Curves D & E
liding forces) to vertical (or friction-
 and are the friction coefficients
ity.
rizontal to vertical pressure with
the section of dam in question.
rizontal to vertical pressure without

DELTA DAM
COMMERCE, ROME, N.Y.
SECTION-NO ICE PRESSURE
y 1924

Consulting Engineer
HURCH St.,
Y. CITY.

to make the test boring and other tests this summer preparatory to drawing up plans and a contract for repairs to be made next season is a highly satisfactory one and one in which the people of Rome and particularly your association should be interested to the extent of making their wishes felt when the matter comes up for legislative consideration next winter. The provisions to prevent ice pressure against the dam might well be made a matter of permanent interest and inspection by your association.

Research Work Needed.

I have strongly urged that a committee of the American Society of Civil Engineers or of Engineering Foundation be appointed to settle the doubtful points as to uplift water pressure and ice pressure in dams. Only lack of funds will prevent favorable action. Such committees give their services but cannot pay for clerical and detailed engineering assistants and apparatus; hence small endowments are necessary and the funds available in the societies above mentioned do not nearly suffice for work badly needed.

Yours very truly,

THW:MR.

No Charge

PART I

Materials Bureau Trip Report on Delta Dam, Rome New York

"INSPECTING DELTA DAM FOR SOURCE(S) OF LEAKS EVIDENT ON DAM FACE"

NYS DOT Region 2 Personnel Contacted:

Chester Urbanczyk	Ben Sweeney
Edward Sears	Henry Zwyack

DATE: 12/1/76

PIN: ML7201.70.116

FROM: E. F. DiCocco

TO: R. H. Obuchowski

R. H. Obuchowski and E. F. DiCocco visited the Delta dam on 11/15/76 and 11/17/76. The purpose of the visits was to determine the source of leak(s) evident on the face of the dam. The following persons were contacted at the dam.

Name	Official Capacity
Ed Sears	Gatehouse Attendant at the dam
Chet Urbanczyk	R-2 Canals & Locks Engineer
Ben Sweeney	R-2 Canals & Locks Superintendent
Henry Zwyack	R-2 Head of Canals & Locks

The following contracts on the dam are on microfilm. They are all available from Carl Tiemann of the NYSDOT Waterways unit at telephone (518) 457-4474.

Contract Number	MICROFILM NO. Roll	Frame	Type of Work
229	10	473	Original Construction
M56-6	22	237	Repair
M58-13	22	303	Repair

DELTA DAM HISTORY

1911 - Dam Constructed

1926- 1927 - A layer of concrete was placed across the upstream face of the dam to reduce seepage. The downstream face of the dam was covered with a "weatherproof facing of gunite" to stop the disintegration of the existing downstream face. The work was done by Department forces and lapsed over at least one winter.

1956 - Contract M56-6 issued to grout leaking areas of dam face. We (Materials Bureau) have parts of these contract plans.

1959 - Contract M58-13 issued to "shotcrete" dam spillway and face. I don't believe we have any copies or portions of these plans.

PROBLEM LEAK(S)

Upon initial inspection of the dam leak(s), it was evident that the flow of the leak is steady. This means that the leak(s) are fed by a constant flow and the only obvious source capable of supplying a constant source of water must be the reservoir behind the dam. However, it is not obvious how the water from the reservoir finds its way to the leak(s) at the face of the dam. Diagrams I and II (attached) depicts the situation. Diagram I shows the dam in general view from its downstream face. Diagram II is a closeup view of section C depicted in diagram I.

Trip of 11/15/76 (EFD & RHO) On this trip, R. Obuchowski and E. F. DiCocco took several pictures of the situation, discussed the general problems with Chet Urbanczyk and Ben Sweeney of Region 2 and took the measurements (approximate) given on diagrams I and II of this report. The drains (1 & 2) at the bottom of the dam were also exposed for inspection. We retrieved a sample of a white, sedimented material that was plugging each drain. The Materials Bureau Chemistry Lab analyzed this material and reported (76LUE2476) that it was probably Calcium Hydroxide ($Ca(OH)_2$). This is probably a product of water leaching into and through the dam slowly and carrying the Calcium Hydroxide into the drains and eventually plugging them over the lifetime of the dam. The drain intakes (1 & 2) at the top of the dam were also chipped open for examination. We found that drain 1 was filled with water and sand (apparently) up to about 5.5 feet from the top deck of the dam. The last 5.5 feet were clear. The condition of drain 2 was the same as drain 1 except that the top 4' were clear of water and sand. We do not know whether or not these drains were purposely filled with sand at the time of their construction. If they were, it was probably to keep the flow rate of water low so as to try to minimize erosion, scouring and leaching of the concrete in the dam. Drains 1 & 2 are the 2 drains closest to leak(s) 1 & 2 (See diagram II). More drains are to the right of drains 1 & 2. These were not exposed or inspected. We do not know what their condition is. All vertical drains were placed at vertical (cold) concrete construction joints at the time of dam construction. The inspected drains 1 & 2 at the bottom of the dam exhibited no water draining out. Therefore, we do not feel that the water flow rate in these two drains is sufficient to feed apparent leaks 1 & 2. During this inspection we also noted that the reservoir level was 8.4' from the top of the dam. This corresponds to an elevation of 550' from sea level. In addition, water was flowing over the main dam spillway.

Trip of 11/17/76 (EFD & RHO) On this trip, R. Obuchowski and E. F. DiCocco met with Chet Urbanczyk and Ed Sears. We placed Rhodamine red dye in drains 1 & 2 at the top of the dam. We expected the dye to come out at either of leaks 1 & 2 or both. However, no dye came out at either leak. This confirms that the two drains (1 & 2) are not feeding the two evident leaks (1 & 2). The dye did come out of various construction joints near the two drains tested. Dye leaked from both vertical and horizontal construction joints. To get some idea of the leaking rate of the two drains, we timed the dye and obtained the following results:

Drain 1 (West)

Start Dyeing	11:14 am
Dye visible at horizontal construction joint 10' down from top of dam	11:45 am
Dye not visible at base drain	2:15 pm

Drain 2 (East)

Dye immediately visible (5 minutes) at vertical and horizontal construction joints 10' from top of dam.

During this dyeing process, we refilled each drain 3 times to the top in order to provide enough pressure head and water to carry the dye out. Rhodamine dye is very powerful. It should be used sparingly. A paper cupfull is enough to stain large areas (100 ft.^{2}) of water. Rhodamine is also reported to be harmless to fish and water plant life. We have 8 pictures (yet to be developed) showing this dyeing process and resultant staining. The dyeing confirmed that the 2 vertical drains tested are not feeding the leak(s). Since the other vertical drains are even further from leaks 1 & 2 than are drains 1 & 2, we do not feel that they are connected to and feeding the leaks either.

Trip of 11/23/76 (EFD & RAM) On this trip R. Marcucci and E. F .DiCocco met with Ed Sears and Chet Urbanczyk at the Delta dam. We tried placing dye in the reservoir at the level of leak 1 (30'± from dam top). The dye did not come through at leak 1, but was drawn around the valve house by the tremendous suction of water due to the outlet valve on the opposite side of the valve house.

A ladder was placed on the face of the dam and E. F. DiCocco inspected both leaks (1 & 2). Leak 2 is not a true leak. It is fed by leak 1 and the water spreads out along the shotcrete construction joint at apparent leak 2. However, the water is not leaking out at the construction joint at leak 2. Therefore the only true leak seems to be leak 1. Because we discovered that leak 2 is not a true leak, we did not try dyeing at its level (43' from top of dam) behind the dam in the reservoir.

We then tried dyeing in the westernmost intake well (closest to leak 1) in the valve house itself. We also transferred the open outlet valve draining the reservoir from the stated intake well. The dye in the intake well was well stirred and mixed with the water. No dye came out anywhere. At this point we are not able to report the source of the leak. Obviously, many possibilities exist. We suspect that the water is somehow infiltrating through construction joints to the leak.

We have left the dye with Ed Sears, the valve house attendant. He has tried dyeing various intake wells. To date, 12/1/76 we (the Materials Bureau) have received no word of results. If positive results had been obtained, the Materials Bureau would have been notified.

OPINION. After reviewing the M56-6 contract plans, we found that leak 1 is just about centrally located in an area of grout repair holes. It may be that the original leak(s) of the M56-6 contract were not adequately sealed and that they eventually reworked their way to the present leak. The leak is located at station 3+10 at elevation 530'. This corresponds to within 2'± of a grout drill hole as shown on the plans.

OPTIONS AND FUTURE WORK:

1. Ed Sears & Chet Urbanczyk will continue monitoring the situation. They will notify Materials if anything happens.

2. The reservoir will be lowered to about the level of leak 1 for the winter. We plan to inspect the reservoir side face of the dam when the lake is frozen in the winter.

3. SUGGESTION. Perhaps an infrared photograph of the dam face (downstream) leak area will reveal the extent of water infiltration. This idea might be further explored.

4. If all else fails, exploratory chipping of the leaking area might be done by department forces next spring and summer.

NOTE: We have various plans and photographs and slides for inspection by whoever is interested.

Trip of 1/20/77 (RHO & RCB) Ed Sears, R. C. Babyak & R. H. Obuchowski inspected the Delta Dam. The following conditions at or near the dam were noted:

a.	Top of Dam Elevation	558.4'
b.	Approximate Elevation of Leak One	529.1'
c.	Approximate Elevation Difference (a-b)	29.3'
d.	Reservoir Level on 1/20/77	534.7'

e. About two (2') of snow on the ground.
f. Construction joints on the upstream face of the dam were deteriorated at the edges. Deterioration extended about 4"+ deep. This deterioration may be causing or contributing to leak one.
g. Blocks of ice at upstream side of dam hiding water level at approximate level of leak. This made it very difficult to observe upstream face at approximate level of leak one.
h. Approximately 10" layer of ice buildup covering leak 1. Ed Sears tried dyeing the westernmost intake well prior to 1/20/77. On that same day, no dye was visible at leak one level 4 hours after placing the Rhodamine red dye in intake well. However, red dye was possibly visible on 1/20/77 at leak one location. However, this observation was not certain.

Observations a-h address themselves to options 1 & 2 on pages 3 and 4 of this Trip Report. Referring to options 3 and 4 on page 4 of this Trip Report, we did no exploratory chipping of the concrete in the area of leak one. Neither did we pursue the infrared photograph option any further.

DIAGRAM I: GENERAL VIEW OF DELTA DAM

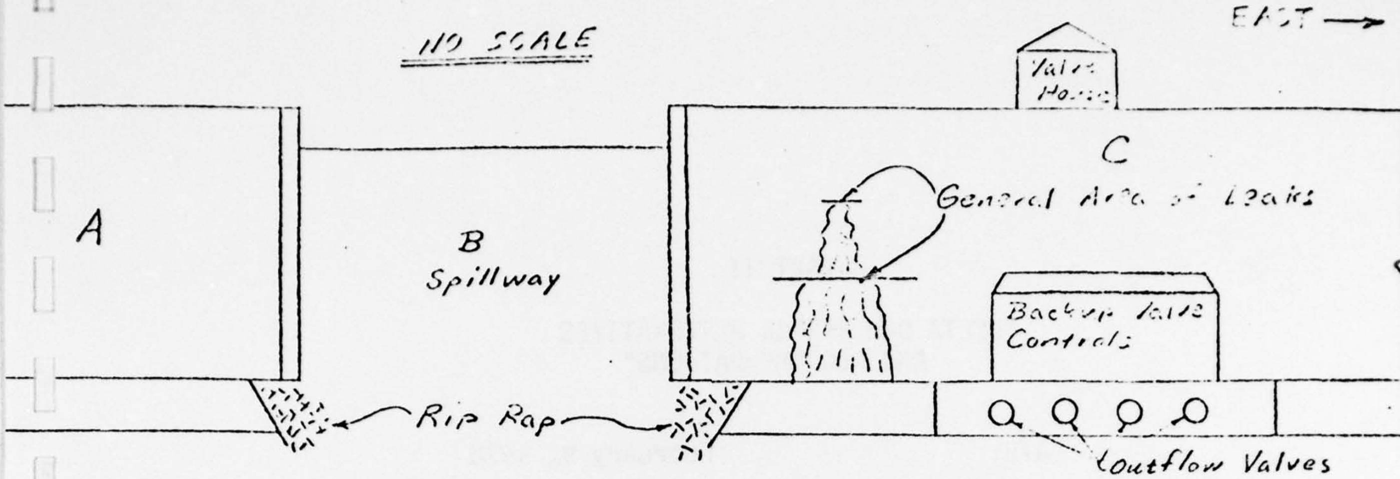
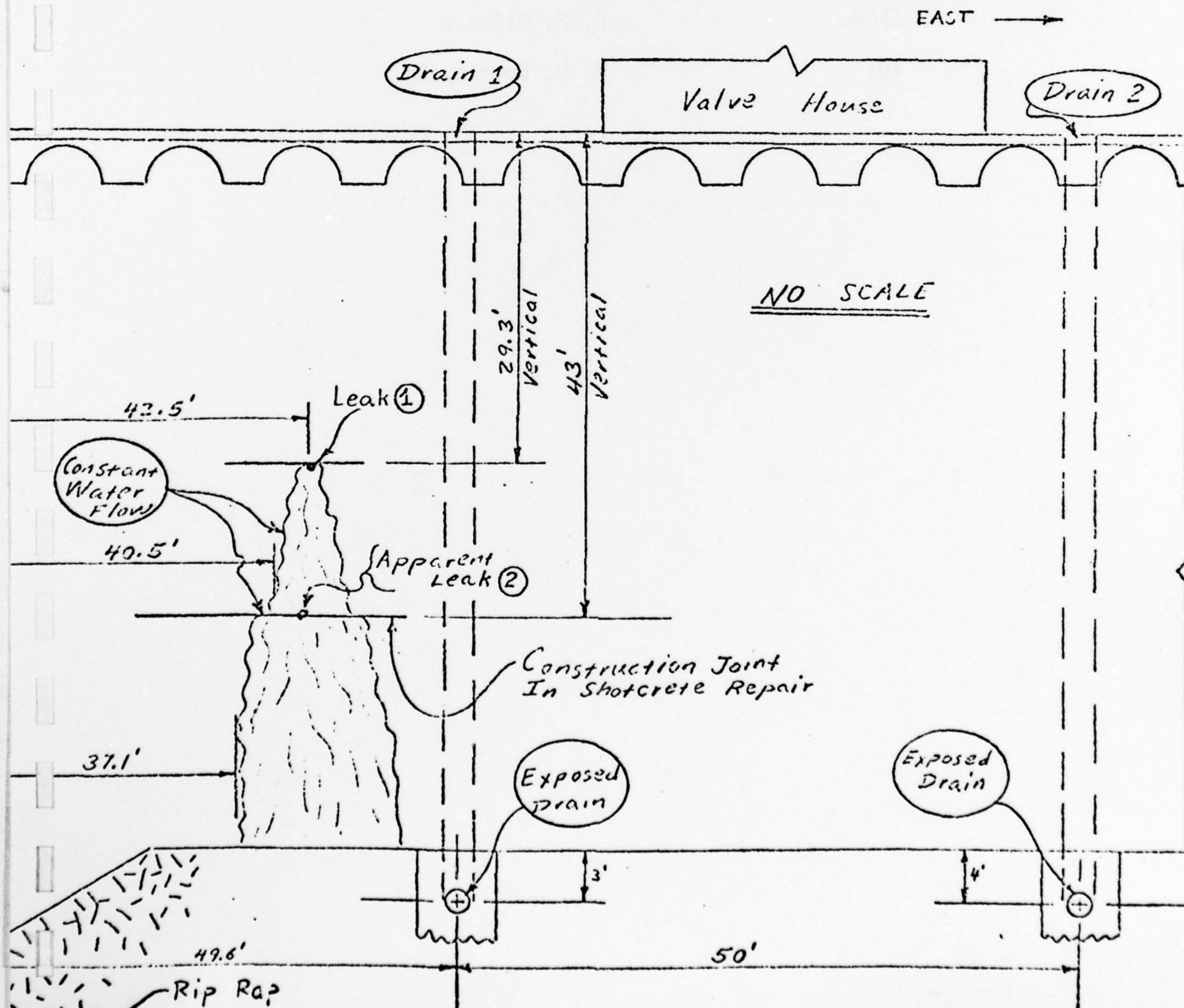


DIAGRAM II: CLOSEUP VIEW OF SECTION C ABOVE



PART II

"DELTA DAM REPAIR ALTERNATIVES
AND RECOMMENDATIONS"

DATE:	February 9, 1978
PIN:	ML 7201.70.116
FROM:	E. F. DiCocco
TO:	R. H. Obuchowski

During the past year, field crews from this office inspected the Delta dam near Rome, NY several times in attempts to determine the source(s) of the dam leak on the downstream face of the eastern end of the dam. As noted in Part I of the trip report (p. 3), we have concluded that leak number one appears to be the only true leak. The approximate location of leak one is shown in the section "C" diagram on page 5 of the trip report. The report is written by E. F. DiCocco to R. H. Obuchowski and dated 12/1/76. It includes and explains the reasoning for some of our conclusions and findings. It also lists previous repair contract identifications and, where copies of the contract plans can be obtained. This 12/1/76 report should be read in order to obtain a better understanding of our findings.

No leaks or problem areas are visible at the western end or spillway of the dam. Therefore, this report only deals with the eastern end of the dam and the known leak.

The leak is flowing steadily, but does not appear likely to cause a catastrophic failure. Its flow is high enough so that, in time, additional material will be leached and washed out of the concrete in the dam causing the leak to worsen. Any increase in flow may cause: erosion problems in the soil at the base of the dam; water problems at the valve control building at the base; further washout in the concrete. The leak should be stopped and prevented from expanding beyond its present size. Repairing the leak will eliminate any need for expensive emergency repairs to it in the future.

There are several other seepage areas at the downstream eastern side of the dam (see 1977 color photos - Table B). Some of the repair alternatives will also help in remedying these seepage problems. However, we do not feel these seepage areas are very critical. The most important problem is leak number one.

The color photos in Table B also reveal that the shotcrete facing originally placed circa 1927 is very deteriorated. Although this deteriorated shotcrete is unsightly, we do not feel that it needs to be removed and replaced to maintain the structural integrity of the dam. However, since the Delta reservoir is part of Delta Lake State Park, and the general public frequently sees the "apparent condition" of the dam, you may want to replace the deteriorated shotcrete with a cosmetic facade. Cost considerations should control this option.

During our inspections, we noticed that some of the construction joints on the upstream facing were deteriorating. These deteriorating joints may be providing water with access routes to the construction joints in the main body of the dam beneath the two foot (+) facing. This 2'$\pm$ facing is shaded blue on the cross sectional view of Delta Dam on page 5/8 of the M56-6 contract drawing. The water may then be infiltrating along the main construction joints to steadily feed the observed leak. Presuming this to be the problem, we recommend the following four repair alternatives. These repairs might possibly be done with NYSDOT Maintenance forces, by scaling this work to fit their capacity and concentrating on the area of the leak.

The lowest level of the reservoir encountered during our inspections was about 30 feet from the top. This left a water depth of about 50 feet. Because of this, we are not exactly certain what the condition of the dam facing is within the lower 50 feet. The facing may be deteriorated more or less extensively than now suspected.

From our inspections, we propose repair alternatives A, B, C and D to stop the leak. Alternatives A, B and C are listed in order of increasing effort and cost. Alternative D is our primary recommendation and the least costly.

ALTERNATIVE A

Patch the two foot (+) thick facing on upstream eastern end of the dam. The repair should be as follows: Drain the Delta reservoir completely. Then, chip the existing concrete along all vertical and horizontal construction joints in the facing. The chipping should straddle each joint and should be continued into concrete that is not deteriorated. Clean out all deteriorated and loosely bonded existing material in and around the chipped out joints. Then, sandblast these chipped out areas until a sound, clean surface is obtained. Where large patch areas are necessary, anchor steel reinforcing mesh to the sound concrete of the dam. Then, fill in the chipped out joints with a mortar mix proportioned 1:2 (cement:sand) by volume. A stiff mortar mix can be hand-packed into the joints or wooden or metal forms can be placed to shape the plastic mortar to match the original shape of the dam's upstream facing. The freshly placed mortar should be allowed to cure at least one day prior to reimmersing it in water. This length cure should allow the fresh mix to set well enough so that, when it is reimmersed in water, its strength will not be affected due to a disruption of its water/cement (w/c) ratio.

While the reservoir is drained, it would be convenient and advantageous to inspect the full depth of all intake wells carefully. In addition, both sides of the spillway and the upstream western end of the dam should also be carefully inspected at this time.

ALTERNATIVE B

Pressure grout the entire eastern end of the dam. This could be done with conventional methods of pressure grouting into deep vertical predrilled holes through the top of the dam. The grout holes should be drilled approximately 5 feet apart or less. Grouting should progress from the bottom upwards. The grout should be a pure cement slurry or epoxy grout without any sand. Similar pressure grouting was conducted on the Delta dam under repair contract M56-6.

During our inspections, we exposed two vertical drains (drains 1 and 2, p. 5 of the Part I report). Each of these drains was plugged with efflorescence product from about 5' from the top to the bottom of each drain. This efflorescence was caused by water leaching through cracks and seams in the concrete and depositing the residue in the drains. The configuration of these drains is shown in a xerox copy of a photograph taken in October 1909 during the original construction of the dam. This photo is labeled C-1.

The drain, interlocking keyways, and cyclopean masonry are clearly marked. We are concerned that similar efflorescence product may have been deposited in cracks and joints throughout the dam during its lifetime. If this is the case, the pressurized grout material may not bond well to the efflorescence and not completely seal the joints, cracks or leak(s). We have attached an article from the 10/74 issue of Public Works magazine which describes some advantages and problems associated with epoxy grouting procedures.

ALTERNATIVE C

Drain the reservoir fully. On the eastern upstream face of the dam, remove the existing nominal two foot thick (±) facing until original concrete is encountered that is not deteriorated. Also chip out repairs that may have been done under alternative A as necessary. Anchor suitably sized reinforcing bars into the original sound concrete that has been uncovered by removing the nominal 2' (±) facing. Place formwork to reconstruct the facing to the original configuration or thicker. Then resurface the entire eastern upstream face of the dam with suitable NYSDOT Class A concrete. This pour should progress in stages vertically upwards from the base of the dam. The fresh concrete should be properly vibrated as work progresses in each stage. New joints should be formed and sealed using waterstops. The fresh concrete should be reimmersed in water as described in alternative A.

We do not recommend shotcrete as a substitute for this procedure. We believe that if shotcrete were used, rebound losses of up to 30% of the shotcrete projected would occur. This high rebound loss, the thickness required (2'±), and the special procedures and equipment needed, would make shotcreting too expensive.

ALTERNATIVE D

Pressure grout in the immediate vicinity of the leak until water flow is sufficiently negligible or completely stopped. Grouting should progress from leak number one and radiate outward in all directions. The grout can be either an epoxy type or cement slurry. The grout should be capable of rebonding cracks and delaminations in the concrete.

After reviewing several epoxy type grouts, we can recommend one with reasonable certainty of success. This product is "Niklepoxy concrete injection resin - product #3". It is manufactured by Rocky Mountain Chemical Company. This material's primary advantages are claimed to be:

1. It is capable of rebonding cracks and delaminations in portland cement concrete in the presence of water and at temperatures as low as 0°C (32°F).

2. On frost free dry surfaces, the system can be applied and can develop strength at temperatures as low as -23°C (-10°F).

3. The resin can be injected at relatively low pressures (10 to 25 psi) above the counteracting water pressures. Counteracting pressures should

be less than 20 psi for adequate performance.

For the reader's convenience, we have included a copy of the information sheet on this product.

Other epoxy grouts are available from various other manufacturers and distributors. Based on their product literature, we do not feel they will perform as well. However, these other products should be carefully examined to determine their suitability. This Bureau is presently engaged in evaluating these other epoxy materials via consultations with their distributors.

Alternative D is our primary alternative. We feel it is the cheapest method of stopping the leak. It should be tried first. Alternatives A, B, and C should be considered if D fails.

CAUTIONS CONCERNING EPOXY GROUT

Some epoxy mortar grout compounds are very toxic. Others are less toxic. If epoxy grout is used in repairing the Delta dam, care should be exercised to ensure that the Delta Lake reservoir is not contaminated with toxic compounds. This problem may be particularly important in this case because the Delta reservoir is connected with the barge canal system and the Mohawk River. The river is used as a water supply by some municipalities. However, the Delta reservoir itself is not a city water supply. Epoxy grouts can be used safely if placed in accordance with manufacturer's directions.

SUGGESTIONS

During our 2/22/77 inspection of the dam, we inspected the westernmost intake well in the gatehouse. The water level in the well was dropped about 30' from the top. Substantial concrete cracking on the interior walls of the intake well was observed. We believe this observed cracking may be contributing to the downstream face leak and seepage areas. Therefore, any repair of the dam should consider an examination of all intake wells for possible necessary repairs. The intake wells should be repaired where warranted. Photographs B-7, B-8 and B-9 in Table B are typical examples of the intake well deterioration we observed.

We also recommend that all vertical drains plugged with efflorescence be cleaned out. This can probably be accomplished by jetting water, under high pressure, into the drains. Fire hoses might be suitable for this. Water jetting should proceed from the bottom of each drain upwards. This procedure allows loosened materials to wash back out through the bottom of the drains. Once the drains are cleaned out, they will probably be working. Therefore, an interceptor pipe should be placed at the base of the drains to carry seepage water to a suitable drainage area. A possible drainage area outlet for the interceptor pipe is onto the rip-rap near the dam spillway.

We do not recommend placing a waterproof membrane between the chipped and new facings of alternative C for two reasons:

1. The chipped face would have to be smoothed over with mortar so that the rough chipped concrete could not puncture the membrane. This smoothing process requires very careful time consuming and costly repairs.

2. Even if it were feasible to place the membrane in the suggested manner, the membrane would act as a "bond breaker" between the new facing and the old concrete. Obviously, this is not desirable because the new surfacing would likely peel away from the membrane and the dam.

We also do not recommend placing a waterproof membrane over the new upstream face resurfacing. Although this seems a good idea, we strongly believe that repeated winter icing action on the membrane would seriously damage it. This is undesirable because a torn or damaged membrane cannot perform as the intended water barrier.

Since the leak and seepage occurs on the eastern side of the dam, we do not recommend any repairs on the downstream face of the spillway or western end of the dam at this time.

Interesting insight into this dam's construction history can be found in this report dated 6/7/24:

> "Report on Delta Dam to the Chamber of Commerce Committee of Rome, NY" by Thomas H. Wiggin, Consulting Engineer, 50 Church Street, NYC, NY

The report gives background information on prior Delta dam repairs as well as on original construction methods including "cyclopean masonry" and block construction in stages. Five (5) black and white photographs showing original construction operations from 1909 to 1911 are included in the 6/7/24 report. We have included xerox copies of two of these 5 photographs. The xerox copies of the photos are labeled C-1 and C-2. They show the cyclopean masonry, block construction, interlocking keyways and vertical drains. The 1924 report should be studied prior to developing specific repair plans.

To aid the reader of this transmittal in understanding the present situation more fully, we have attached Tables A and B with a total of 17 photographs showing the problem features of the dam. Each photograph is numbered and short descriptions are provided.

Table A includes 8 black and white photographs circa 1956-1959. These pictures show various stages of repair during the M56-6 contract period. The present leak (number 1) is in the chipped out area shown on pictures A-6, A-7, and A-8. The general condition of the dam in 1959 is also evident from these pictures.

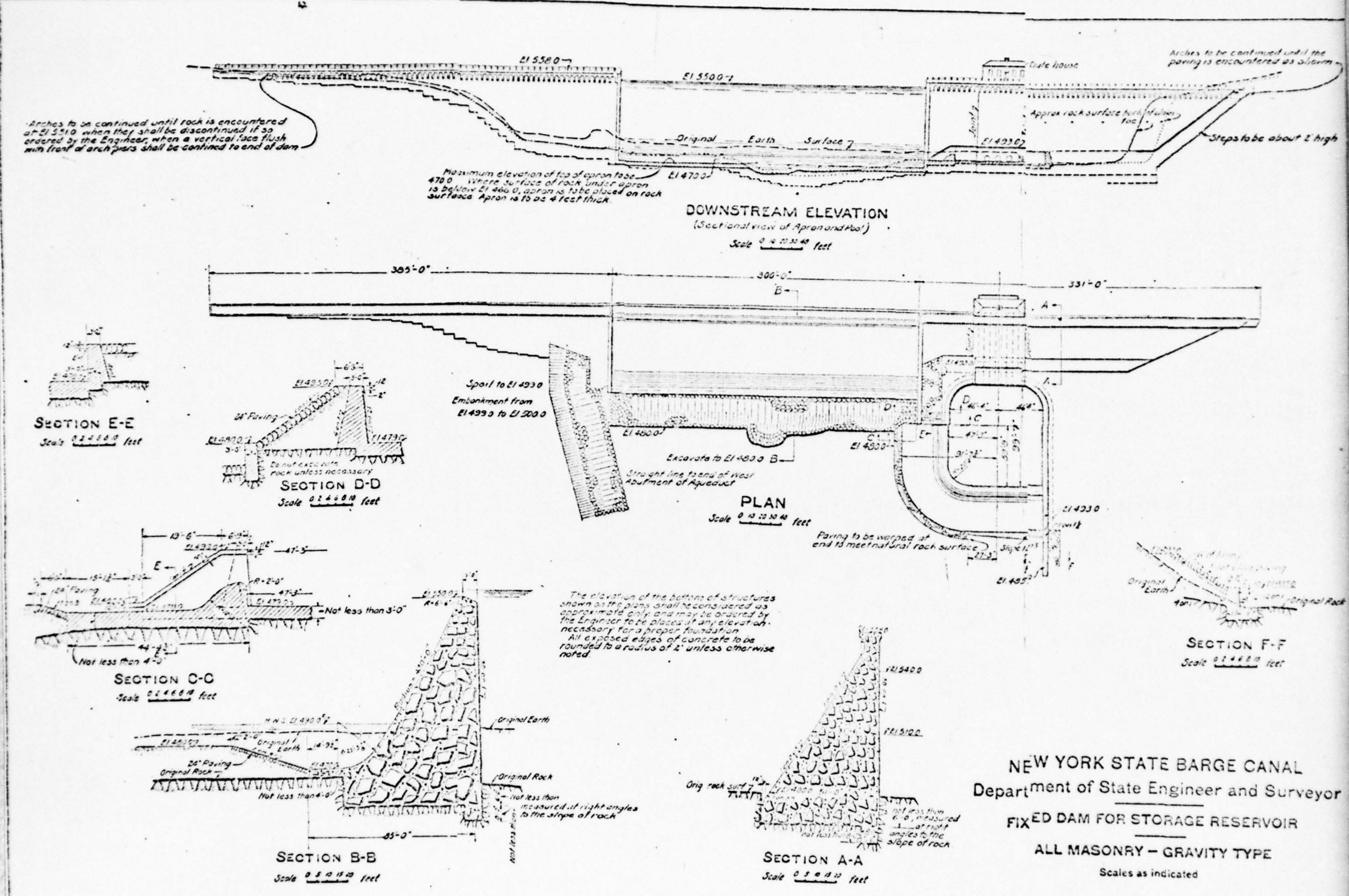
Table B displays color photographs of our most current investigations. Rhodamine red dye is evident in pictures B-1, B-2, and B-3. Its location is circled in each of these 3 photos. Close-up photos of leak one are shown in photos B-2 and B-4. In each of these photos, leak one is designated by a hexagon.

Sheet 5/8 of the M56-6 contact plans is included for your reference. It shows various views of the dam. We have located leak number 1 on this drawing. All dimensions shown in red pencil are approximate.

To reiterate, we feel that the cheapest repair alternative is D. Alternatives A, B, and C follow with increasing cost if D fails. In addition, any alternatives chosen should consider intake well repairs as well as an examination of the entire dam when the reservoir is drained.

APPENDIX E
CONSTRUCTION DRAWINGS

PLATE 30



NEW YORK STATE BARGE CANAL
Department of State Engineer and Surveyor
FIXED DAM FOR STORAGE RESERVOIR
ALL MASONRY – GRAVITY TYPE
Scales as indicated

APPENDIX F
VISUAL CHECK LIST

CHECK LIST
VISUAL INSPECTION
PHASE 1

NAME DAM Delta COUNTY Oneida STATE New York ID# NY 6

TYPE OF DAM Cyclopean masonry gravity structure HAZARD CATEGORY High

DATE(s) INSPECTION May 5, 1978 WEATHER Rain-cloudy TEMPERATURE 50°

POOL ELEVATION AT TIME OF INSPECTION 550.4 M.S.L. TAILWATER AT TIME OF INSPECTION 482.0 M.S.L.

INSPECTION PERSONNEL:

R. Jeffrey Kimball, P.E.   Ed Sears - Caretaker   Jim Kaulfman, NYDOT

James T. Hockensmith   Ed Warner, New York Department of Tran.

John C. Pierchoski, P.E.

John C. Pierchoski, P.E. RECORDER

CONCRETE/MASONRY DAMS

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SURFACE CRACKS CONCRETE SURFACES	Many small and medium cracks on both monoliths adj. to the spillway. Much of the shotcrete has spalled or has not bonded to the dam. Some spalling and cavitation at the water level on the upstream face.	
STRUCTURAL CRACKING	No structural cracks observed, however shotcrete covers face.	
VERTICAL AND HORIZONTAL ALIGNMENT	Good, no movement noted.	
MONOLITH JOINTS	Joints have been heavily shotcreted (looks like severe deterioration in past)	
CONSTRUCTION JOINTS	Considerable patchwork on vertical joints. Looks like they were eroded badly and patched.	
STAFF GAGE OF RECORDER:	Staff gage on side of intake gate house, is read daily.	

CONCRETE/MASONRY DAMS

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
ANY NOTICEABLE SEEPAGE	Considerable minor cracking downstream left face. Considerable evidence of previous seepage now dried up. Fairly high seepage in left Monolith section. Considerable seepage at elevation 527.5 near spillway, and at 19' down from top (elevation	539). Horizontal eroding and spalling at water level (550.5) Right front face showing evidence of extensive past seepage that is now stopped. Spillway wing walls both sides are spider webbed with cracks.
STRUCTURE TO ABUTMENT/EMBANKMENT JUNCTIONS	Left abutment set deeply into thin bedded, black, gray shale. No seepage noticable.	
DRAINS	Small drains may be plugged.	
WATER PASSAGES	None known	
FOUNDATION	Abutments and base appears to be notched into dark gray fissle shale. Appears sound all around.	

OUTLET WORKS

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CRACKING AND SPALLING OF CONCRETE SURFACES IN OUTLET CONDUIT	Four - 60" Diameter steel pipes One pipe was opened 14 inches.	
INTAKE STRUCTURE	Good condition - one minor crack on gate house. Equipment well maintained and operable. Electric motors on all valves.	
OUTLET STRUCTURE	No major damage apparent. Very minor cracking along walls	329 cfs. being discharged according to Mr. Sears.
OUTLET CHANNEL	Bottom not visible, full of water, sides looked good.	
EMERGENCY GATE	All discharge gates and pipes operate smoothly.	

UNGATED SPILLWAY

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CONCRETE WEIR	No distress noted - Ogee spillway, water flowing over spillway did not permit close examination	
APPROACH CHANNEL	None	
DISCHARGE CHANNEL	About 12 places where shotcrete has spalled and is causing water to splash away from spillway. Water flowing over spillway did not permit close examination.	
BRIDGE AND PIERS	Abandoned canal bridge immediately downstream. It will not have a restricting effect.	

GATED SPILLWAY

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CONCRETE SILL	N/A	
APPROACH CHANNEL	N/A	
DISCHARGE CHANNEL	N/A	
BRIDGE AND PIERS	N/A	
GATES AND OPERATION EQUIPMENT	N/A	

DOWNSTREAM CHANNEL

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CONDITION (OBSTRUCTIONS, DEBRIS, ETC.)	Good condition, riprap is large and not displaced. Canal bridge downstream is not an obstruction.	
SLOPES	3 or 4 to 1 and brush and tree covered, stable.	
APPROXIMATE NO. OF HOMES AND POPULATION	At least 20 homes.	

RESERVOIR

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SLOPES	Apparently stable.	
SEDIMENTATION	Very little - will have no affect on storage capacity	

INSTRUMENTATION

VISUAL EXAMINATION	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
MONUMENTATION/SURVEYS	None known	
OBSERVATION WELLS	None known	
WEIRS	None known	
PIEZOMETERS	None known	
OTHER	None known	

APPENDIX G
ENGINEERING DATA CHECK LIST

CHECK LIST
ENGINEERING DATA
DESIGN, CONSTRUCTION, OPERATION
PHASE I

NAME OF DAM Delta Dam

ID# NY 6

ITEM	REMARKS
AS-BUILT DRAWINGS	New York State Department of Transportation Utica, New York
REGIONAL VICINITY MAP	New York State Department of Transportation Utica, New York
CONSTRUCTION HISTORY	New York State Department of Transportation Utica, New York
TYPICAL SECTIONS OF DAM	New York State Department of Transportation Utica, New York
OUTLETS - PLAN - DETAILS - CONSTRAINTS - DISCHANCE RATINGS	New York State Department of Transportation Utica, New York
RAINFALL/RESERVOIR RECORDS	U.S.G.S. gaging station downstream - pool records by New York State Department of Transportation

ITEM	REMARKS
DESIGN REPORTS	New York State Department of Transportation Utica, New York
GEOLOGY REPORTS	New York State Department of Transporation Thomas H. Wiggin Report.
DESIGN COMPUTATIONS	
HYDROLOGY & HYDRAULICS	Unknown
DAM STABILITY	Thomas H. Wiggin Report - New York State Department of Transportation
SEEPAGE STUDIES	Unknown
MATERIALS INVESTIGATIONS	
BORING RECORDS	New York State Department of Transportation
LABORATORY	Unknown
FIELD	Unknown
POST-CONSTRUCTION SURVEYS OF DAM	None known
BORROW SOURCES	Unknown

ITEM	REMARKS
MONITORING SYSTEMS	Staff gage at dam by caretaker and U.S.G.S. gaging station downstream
MODIFICATIONS	None known
HIGH POOL RECORDS	caretaker's office at dam - Hurricane Agnes June 1972 W.E. 554.2
POST CONSTRUCTION ENGINEERING STUDIES AND REPORTS	New York State Department of Transportation - Materials Bureau 1976 Thomas H. Wiggin Report - 1924.
PRIOR ACCIDENTS OR FAILURE OF DAM DESCRIPTION REPORTS	None Known
MAINTENANCE OPERATION RECORDS	New York State Department of Transportation Utica, New York

	REMARKS
SPILLWAY PLAN	
SECTIONS	New York State Department of Transportation Utica, New York
DETAILS	New York State Department of Transportation Utica, New York
OPERATING EQUIPMENT PLANS & DETAILS	New York State Department of Transportation Utica, New York

CHECK LIST
HYDROLOGIC AND HYDRAULIC
ENGINEERING DATA

DRAINAGE AREA CHARACTERISTICS: 150 square miles rolling woodland

ELEVATION TOP NORMAL POOL (STORAGE CAPACITY): 550.0 (63,000 acre feet)

ELEVATION TOP FLOOD CONTROL POOL (STORAGE CAPACITY): N/A

ELEVATION MAXIMUM DESIGN POOL: Unknown

ELEVATION TOP DAM: 558.0

CREST:

a. Elevation 550.0
b. Type concrete ogee
c. Width ----
d. Length 300.0
e. Location Spillover center of structure
f. Number and Type of Gates None

OUTLET WORKS:

a. Type 4-60" steel pipes
b. Location East of center of dam
c. Entrance inverts 487.5
d. Exit inverts 479.0
e. Emergency draindown facilities Above pipes

HYDROMETEOROLOGICAL GAGES:

a. Type Staff gage Stream gage - U.S.G.S.
b. Location Dam Upstream Downstream
c. Records Daily by care taker U.S.G.S.

MAXIMUM NON-DAMAGING DISCHARGE Hurricane Agnes - June 1972 - 4.2' over spillway