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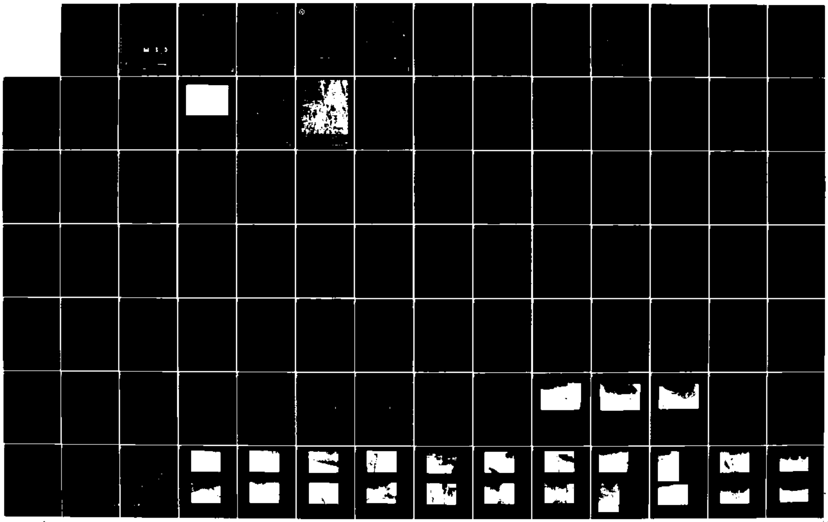

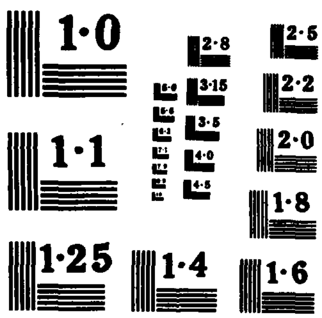
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RICHELIEU RIVER BASIN
TOWN OF WEST RUTLAND
RUTLAND COUNTY, VERMONT

YOUNGS BROOK DAM
VT 00165

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
WALTHAM, MA 02154

JANUARY 1980

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Cover program reads: Phase I Inspection Report, National Dam Inspection Program; however, the official title of the program is: National Program for Inspection of Non-Federal Dams; use cover date for date of report.

19. KEY WORDS (Continue on reverse side if necessary and identify by block number)

DAMS, Visual INSPECTION, DAM SAFETY.

Richelieu river Basin — Youngs Brook
Town of West Rutland
Rutland County, VT.

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

Youngs Brook Dam ~~The dam~~ is an earth embankment about 254 ft. long and 46 ft. high. The dam is in poor condition. There were various significant problems noted. The spillway is inadequate to pass the test flood without overtopping the dam. It is intermediate in size with a significant hazard potential. There are various remdial measures which must be undertaken by the owner. Keywords:

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
424 TRAPELO ROAD
WALTHAM, MASSACHUSETTS 02254

REPLY TO
ATTENTION OF:

NEDED

JAN 06 1981

Honorable Richard A. Snelling
Governor of the State of Vermont
State Capitol
Montpelier, Vermont 05602

Dear Governor Snelling:

Inclosed is a copy of the Youngs Brook Dam (VT-00165) Phase I Inspection Report, which was prepared under the National Program for Inspection of Non-Federal Dams. This report is presented for your use and is based upon a visual inspection, a review of the past performance and a brief hydrological study of the dam. A brief assessment is included at the beginning of the report. I have approved the report and support the findings and recommendations described in Section 7 and ask that you keep me informed of the actions taken to implement them. This follow-up action is a vitally important part of this program.

A copy of this report has been forwarded to the Department of Water Resources, the cooperating agency for the State of Vermont. In addition, a copy of the report has also been furnished the owner, West Rutland Fire District No. 1, West Rutland, VT.

Copies of this report will be made available to the public, upon request, by this office under the Freedom of Information Act. In the case of this report the release date will be thirty days from the date of this letter.

I wish to take this opportunity to thank you and the Department of Water Resources for your cooperation in carrying out this program.

Sincerely,

WILLIAM E. HODGSON, JR.
Colonel, Corps of Engineers
Acting Division Engineer

Incl
As stated

RICHELIEU RIVER BASIN
TOWN OF WEST RUTLAND
RUTLAND COUNTY, VERMONT

YOUNGS BROOK DAM
VT 00165

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

GORDON E. AINSWORTH & ASSOCIATES, INC.

Engineers, Surveyors and Planners

20 SUGARLOAF ST. SOUTH DEERFIELD, MASS. 01373

PROJECT NO. 21.06.79105

1730 NATIONAL DAM INSPECTION PROGRAM

PHASE I INSPECTION REPORT

Identification No. VT 00165

Name of Dam: Youngs Brook Dam

Town: West Rutland

County and State: Rutland County, Vermont

Stream: Youngs Brook

Date of Inspection: 8 November 1979

BRIEF ASSESSMENT

1. Project Description

Youngs Brook Dam is used by the Town of West Rutland as an emergency water supply for both fire protection and potable water. The dam is an earth embankment about 254 feet long by about 46 feet high. Included in the length of the dam are two adjacent spillways at the left abutment, the newer, lower spillway being about 28 feet long and the old spillway being about 19.3 feet long. Top width varies from about 15 feet on the left next to the new spillway to as much as about 125 feet at the right abutment. The upper portion of the upstream face is a vertical concrete retaining wall. Upstream of the wall the surface consists of sloped terraces separated by stone walls. The overall upstream slope is about 1.15H:1V. The downstream slope is covered with trees and is about 1.5H:1V.

Normal pool elevation of 5 feet below the top of the dam is maintained by a free overfall spillway near the left abutment. Right next to this spillway and extending to the left abutment is the older, original chute spillway with a crest 2 feet higher than the newer spillway. Both spillways have concrete weir crests, and discharge channels with concrete training walls and bedrock bottoms. The discharge channels share a common training wall, but the newer spillway discharge channel is much deeper than the older discharge channel.

1731 2. Significant Findings and Assessment

The dam is in POOR condition. Significant problems include questionable stability of both the steep upstream and downstream slopes; heavy tree growth on the downstream slope; water pipe under pressure passing through the embankment; severe distortion and deterioration of the concrete wall on the right side of the upstream face; and cracking, spalling, and deterioration to varying degrees of the concrete training walls of both the old and new spillway discharge channels. Also, seepage was observed exiting from one point at the downstream toe and from the bedrock abutment beneath the left training wall just downstream from the new spillway.

3. Hydraulic and Hydrologic Findings

The spillway is INADEQUATE to pass the test flood without overtopping the dam. In accordance with recommended guidelines of the Corps of Engineers, the dam is classified as INTERMEDIATE in size and as having a SIGNIFICANT hazard potential. Accordingly, a TEST FLOOD equal to ONE-HALF PMF (probable maximum flood) was judged as appropriate within the recommended range of one-half PMF to full PMF. The test flood overtops the dam by a maximum of about 0.6 of a foot with duration of overtopping of about 3 hours. Peak inflow for the test flood is 1910 cfs. Peak outflow is not reduced by reservoir routing and is the same as peak inflow. Total project discharge capacity at the top of the dam is due only to the two spillways (outlet works assumed closed), and is equal to 1370 cfs, or 72% of the test flood peak outflow.

4. Recommended Action

a. IMMEDIATELY after their receipt of this Phase I Inspection Report, the Owner should lower the reservoir to about EL 828 (i.e. to the invert of the outlet conduit about 13 feet below the new spillway crest) and remove pressure from the pipes passing through the dam. The reservoir should be maintained at that level until the dam is repaired or permanently breached.

b. WITHIN ONE YEAR after their receipt of this Phase I Inspection Report, the Owner should implement the following recommendations:

1) Engage a registered engineer qualified in the design of dams to do such work as: (1) determine the stability of the slopes under critical conditions and make recommendations for improvements, (2) redesign the upstream concrete face where it is tilted and cracked, (3) perform a detailed hydraulic and hydrologic study to better assess the adequacy of spillway capacity, and (4) make recommendations and design repairs or replacement as appropriate for various parts of the concrete training walls of both the old and new spillway discharge channels.

2) Remove the trees and roots from the downstream slope and backfill the root holes with material recommended by the engineer engaged. Apply necessary slope protection.

3) At least quarterly monitor the seepage from the toe of the dam and from the bedrock abutment beneath the left training wall just downstream from the new spillway.

Additional recommendations and remedial measures that should be implemented by the Owner WITHIN ONE YEAR after their receipt of this Phase I Inspection Report are described in Section 7.

GORDON E. AINSWORTH & ASSOCIATES, INC.

Kenneth J. Male, P.E.

LICENSED PROFESSIONAL ENGINEER
KENNETH J. MALE
29004
UNIVERSITY OF THE STATE OF NEW YORK
& LAND SURVEYOR

This Phase I Inspection Report on Youngs Brook Dam has been reviewed by the undersigned Review Board members. In our opinion, the reported findings, conclusions, and recommendations are consistent with the Recommended Guidelines for Safety Inspection of Dams, and with good engineering judgment and practice, and is hereby submitted for approval.

RICHARD DIBUONO, MEMBER
Water Control Branch
Engineering Division

ARAMAST MAHTESIAN, MEMBER
Geotechnical Engineering Branch
Engineering Division

CARNEY M. TERZIAN, CHAIRMAN
Design Branch
Engineering Division

APPROVAL RECOMMENDED:

JOE B. FRYAR
Chief, Engineering Division

PREFACE

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

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

It is important to note that the condition of a dam depends on numerous and constantly changing internal and external conditions, and is evolutionary in nature. It would be incorrect to assume that the present condition of the dam will continue to represent the condition of the dam at some point in the future. Only through continued care and inspection can there be any chance that unsafe conditions will be detected.

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

The Phase I Investigation does not include an assessment of the need for fences, gates, no-trespassing signs, repairs to existing fences and railings and other items which may be needed to minimize trespass and provide greater security for the facility and safety to the public. An evaluation of the project for compliance with OSHA rules and regulations is also excluded.

8946

YOUNGS BROOK DAM

PHASE I INSPECTION REPORT

TABLE OF CONTENTS

f. Reservoir Surface (acres)

1)	Normal Pool	2.2
2)	Flood Control Pool	N/A
3)	Spillway Crest Pool (new spillway)	2.2
4)	Top of Dam	2.5
5)	Test Flood Pool	2.6

g. Dam

1) Type - Earth.

2) Length - 254 feet including spillways.

3) Height - Hydraulic Height - 46 feet.
- Structural Height - 46 feet.

4) Top Width - Varies from about 15 feet to 125 feet.

5) Side Slopes - Upstream - Vertical concrete wall and sloped terraces separated by stone walls, about 1.15H:1V overall.
- Downstream - 1.5H:1V overall.

a) Approximate Volume of Dam - 35,000 cubic yards

6) Zoning - None. Concrete wall on upstream face. This wall apparently does not penetrate to the foundation but acts only as a retaining wall for the upper part of the embankment.

7) Impervious Core - None known.

8) Cutoff - None known.

9) Grout Curtain - None known.

10) Other - No comment.

h. Diversion and Regulating Tunnel - N/A

i. Spillway

1) Old Spillway

a) Type - Chute, with concrete overflow weir cap founded on rock.

used in this report are based on this map (plus 143 feet) and on field measurements made during the inspection (see Appendix B2-2), and are in <u>approximate feet above mean sea level NGVD</u> (National Geodetic Vertical Datum of 1929).

1)	Natural Stream Bed at Toe of Dam - D/S	800 ±
	- U/S	803 ±
2)	Bottom of Cutoff	Unknown
	a) Lowest foundation surface	800 ±
	b) Core Wall	Unknown
3)	Maximum Tailwater	Unknown
4)	Normal Pool	841 +
5)	Full Flood Control Pool	N/A
6)	Spillway Crest (ungated)	
	- New spillway crest	841
	- w/flashboards	842.7 ±
	- Old spillway crest	843
7)	Design Surcharge	Unknown
8)	Top of Dam	846
9)	Test Flood Surcharge	846.6

d.) <u>Reservoir</u> (length in feet)

1)	Normal Pool	675 ±
2)	Flood Control Pool	N/A
3)	Spillway Crest Pool (new spillway)	675 ±
4)	Top of Dam	750 ±
5)	Test Flood Pool	760 ±

e. <u>Storage</u> (acre-feet)

1)	Normal Pool	38
2)	Flood Control Pool	N/A
3)	Spillway Crest Pool (new spillway)	38
4)	Top of Dam	50
5)	Test Flood Pool	51

4) Area - 1.85 square miles, or 1182 acres.

5) Topography - Steep wooded slopes averaging 15% to 25% slope. Elevations vary from EL 841 to EL 2726.

b. <u>Discharge at Dam Site</u> (cfs)

1) Outlet Works

a) Outlet Conduit
2-feet square, intake and discharge invert EL 828 ±, discharge capacity about 79 cfs at top of dam EL 846.

b) Drain Pipe
8-inch diameter, intake invert EL 798 ±, discharge invert unknown, discharge capacity about 12 cfs at top of dam EL 846.

c) Water Supply Main
12-inch diameter, intake invert EL 803 ±, discharge invert unknown, discharge capacity about 25 cfs at top of dam EL 846.

2) Maximum Known Flood - Unknown.

3) Ungated spillway capacity at top of dam (2 spillways at different elevations), 1370 cfs @ EL 846.

4) Ungated spillway capacity at test flood pool, 1675 cfs @ EL 846.6.

5) Gated spillway capacity at normal pool - N/A.

6) Gated spillway capacity at test flood pool - N/A.

7) Total spillway capacity at test flood pool, 1675 cfs @ EL 846.6.

8) Total project discharge capacity at top of dam, 1370 cfs @ EL 846.

9) Total project discharge capacity at test flood pool, 1910 cfs @ EL 846.6.

c. <u>Elevation</u> (feet - NGVD)

From USGS topographic mapping and from field measurements at the site, it was determined that the crest of the old spillway is at about EL 843 NGVD. An old topographic map of the reservoir (included as Appendix B2-1) indicates that the old spillway crest is at EL 700 map base. Therefore, all elevations

h. Design and Construction History

Youngs Brook Dam was originally constructed for a private owner sometime in the early 1920's. The designer and construction contractor for the dam are unknown.

The present owner of the dam, West Rutland Fire District No. 1, built a new spillway for the dam in 1948 and 1949 after the old one was damaged by a flood in 1947. The designer and construction contractor for the new spillway construction are unknown.

No other construction, modification, or major repair work is known to have occurred. Refer to Section 2 of this report for a complete discussion of the design, construction and operation history.

i. Normal Operation Procedures

There are no written operation and maintenance procedures for the dam. The dam Operator normally visits the dam once a week. During the winter the Operator visits the dam every other day to control ice build-up on the spillway by chopping the ice away. During storm events, the reservoir level is visually monitored by the Operator, who may then open the outlet conduit to increase outflow. The results of the visits to the dam are not recorded.

Normal water level is maintained by flow over the new spillway crest. The outlet conduit slide gate is normally shut, as is the 8-inch drain, or mud pipe. The 12-inch water supply main is normally open, although little to no outflow occurs as long as the Town's well field, its primary water supply, is operable.

Refer to Section 4 of this report for a complete discussion of operation and maintenance procedures.

1.3 Pertinent Data

a. Drainage Area

1) Location - West central Vermont in the foothills of the Green Mountains.

2) River Basin - Youngs Brook to Clarendon River, then to Otter Creek, to Lake Champlain, to Richelieu River.

3) Shape - Roughly rectangular, 12,000 feet by 5,000 feet.

d. Hazard Classification

In accordance with recommended guidelines (References 1 and 18) involving loss of life and economic loss, Youngs Brook Dam is classified as having a SIGNIFICANT hazard potential. The dam is located in a predominantly rural or agricultural area. However, the increase in flow due to a dam failure would wash out an unpaved Town road and large culvert, flood about 35 acres of farmland on both sides of Town Route 133 to a depth of less than a foot, do some damage to Town Route 133, and flood around the foundations of about 8 houses, 2 house trailers, 3 barns, and miscellaneous outbuildings. The high flow velocity of 10 fps around most of the structures and the velocities of 7 to 10 fps over the farmland would probably do significant erosion damage. Total economic loss is judged appreciable. Loss of life is judged unlikely, but more than a few inhabitable structures are affected. The dam failure analysis is developed in Section 5.5 of this report.

e. Ownership

Youngs Brook Dam was constructed originally for a private owner in the early 1920's. Presently the dam and reservoir are owned by:

West Rutland Fire District No. 1
West Rutland, Vermont 05777

Attention: William F. Harvey III, Chairman
(802) 438-5771

It is not known how much of the watershed, if any, is owned by the Fire District.

f. Operator

Day to day operation of the dam is the responsibility of:

Joseph F. Skaza
21 Clarendon Avenue
West Rutland, Vermont 05777

(802) 438-5771 & 438-2907

g. Purpose of Dam

The dam, when acquired by the present owner, was originally used as an active water supply for the Town of West Rutland. At the present time West Rutland uses a well field as their source of supply, and the reservoir is used as an emergency water supply for both fire protection and potable water.

The older, original spillway extends from the new spillway to the left abutment, and has a 19.3-foot long ungated weir about 3 feet lower than the top of dam. The weir is followed by an inclined drop of several feet into a steeply sloping chute discharge channel with a natural bedrock bottom. The discharge channel is much higher than the discharge channel for the new spillway just to the right.

Just to the right of the new spillway, a 2-foot square concrete outlet conduit starts at the face of the concrete retaining wall and extends for a short distance through the side of the right training wall of the new spillway discharge channel, discharging into the channel. Control is provided by a slide gate on the face of the retaining wall on the upstream side of the dam. The slide gate is operated by a floor stand with handwheel located directly above on top of the dam.

Upstream of the dam at about its left third point there is a normally submerged water supply manhole (shown on Appendices B2-1 and C-1). Entering the manhole is a 12-inch diameter intake pipe from the deepest part of the reservoir. Leaving the manhole is a 12-inch diameter supply main extending through the dam and continuing to the distribution system. Also, there is a 8-inch diameter drain, or mud pipe, exiting from the bottom of the manhole, extending through the dam, and discharging into Youngs Brook somewhere downstream. Both the water supply main and drain pipe are controlled by separate gate valves next to the manhole on the upstream side of the dam. The reservoir must be drawn down about 13 feet by using the outlet conduit in order to get to the valves to operate them.

There is apparently a smaller reservoir in the bottom of the main reservoir that was used when the main reservoir was drained periodically for cleaning. An 8-inch diameter supply main from the small reservoir joins with the 12-inch supply main just downstream of the 12-inch control valve next to the water supply manhole. The 8-inch main is controlled by a separate valve just upstream of the 12-inch main. As with the other control valves, the reservoir must be drawndown with the outlet conduit in order to get to the 8-inch valve and operate it.

c. Size Classification

In accordance with recommended guidelines (Reference 1), Youngs Brook Dam is classified as INTERMEDIATE in size because its hydraulic height is 46 feet (within the 40 to 100-foot range). The maximum storage capacity at the top of the dam is 50 acre-feet.

Access to the dam is from Town Route 133 to the northeast and up the hill via a Town road (see Drainage Area Map, Appendix D-1).

The popular name of the dam is the West Rutland Reservoir Dam, and the impoundment is popularly called the West Rutland Reservoir. The official names are Youngs Brook Dam and Youngs Brook Reservoir. The reservoir is aligned along a northeast - southwest axis with the dam located at the northeast end.

The dam is built across Youngs Brook, a tributary of the Clarendon River. The nearest downstream community is West Rutland, population estimated at 1500, located about 2 river miles downstream of the dam on the west side of the Clarendon River. West Rutland is not an incorporated village or city but is a post office location together with houses and other structures located in the Township of West Rutland.

b. <u>Description of Dam and Appurtenances</u>

Youngs Brook Dam is an earth embankment with a partially exposed concrete retaining wall along the upstream face. The dam is about 254 feet long (including the two spillways) by about 46 feet high. Top width varies from about 15 feet on the left next to the new spillway to as much as about 125 feet at the right abutment. The upper portion of the upstream face is a vertical concrete retaining wall. Upstream of the wall the surface consists of sloped terraces separated by stone walls. The overall upstream slope is about 1.15H:1V. The downstream slope is covered with trees and is about 1.5H:1V.

The materials in the embankment are unknown. No impervious core or zoning are known. No cutoff is known. The foundation conditions are unknown, except that the spillways and left abutment appear to be founded on bedrock.

34 1 Near the left abutment there are two adjacent spillways with concrete weir crests 2 feet different in elevation. The newer, lower spillway is farthest from the abutment and is of the free overfall type with a 28-foot long ungated weir about 5 feet lower than the top of dam. The weir is followed by a nearly vertical drop of over 20 feet to the bedrock bottom of the discharge channel. On the right side of the discharge channel there is a concrete training wall against the embankment, and on the left, there is natural bedrock part way up with a concrete training wall on top of this bedrock. This concrete training wall is also the right side of the older spillway discharge channel.

NATIONAL DAM INSPECTION PROGRAM

PHASE I INSPECTION REPORT

NAME OF DAM: YOUNGS BROOK DAM, ID NO. VT 00165

SECTION 1

PROJECT INFORMATION

1.1 General

a. Authority

The National Dam Inspection Act, Public Law 92-367, August 8, 1972, authorized the Secretary of the Army through the Corps of Engineers to initiate a national program of dam inspection throughout the United States. The New England Division of the Corps of Engineers has been assigned the responsibility of supervising the inspection of dams within the New England Region. Gordon E. Ainsworth and Associates, Inc., has been retained by the New England Division to inspect and report on selected dams in the State of Vermont. Authorization and notice to proceed was issued to Gordon E. Ainsworth and Associates, Inc., under a letter from William E. Hodgson, Jr., Colonel, Corps of Engineers. Contract No. DACW33-80-C-0012 has been assigned by the Corps of Engineers for this work.

b. Purpose of Inspection

1) Perform technical inspection and evaluation of non-Federal dams to identify conditions which threaten the public, and thus permit correction in a timely manner by non-Federal interests.

2) Encourage and assist the States to initiate quickly effective dam safety programs for non-Federal dams.

3) To update, verify, and complete the National Inventory of Dams.

1.2 Description of Project

a. Location

Referring to the Location and Vicinity Maps at the beginning of this report, Youngs Brook Dam is located in west central Vermont in the Town of West Rutland, Rutland County, about 6 miles southwest of the City of Rutland. The dam at its maximum section is at Latitude 43 degrees - 34.3 minutes North, Longitude 73 degrees - 3.1 minutes West.

APPROX. SCALE IN MILES

0 2 4 8 12

DATUM-NGVD 1929, 100' CONTOUR INTERVAL

BASE MAP - 1:250,000 U.S.G.S. TOPO MAP

GLENS FALLS, N.Y., VT., N.H.,
1966, LIMITED REVISION 1967

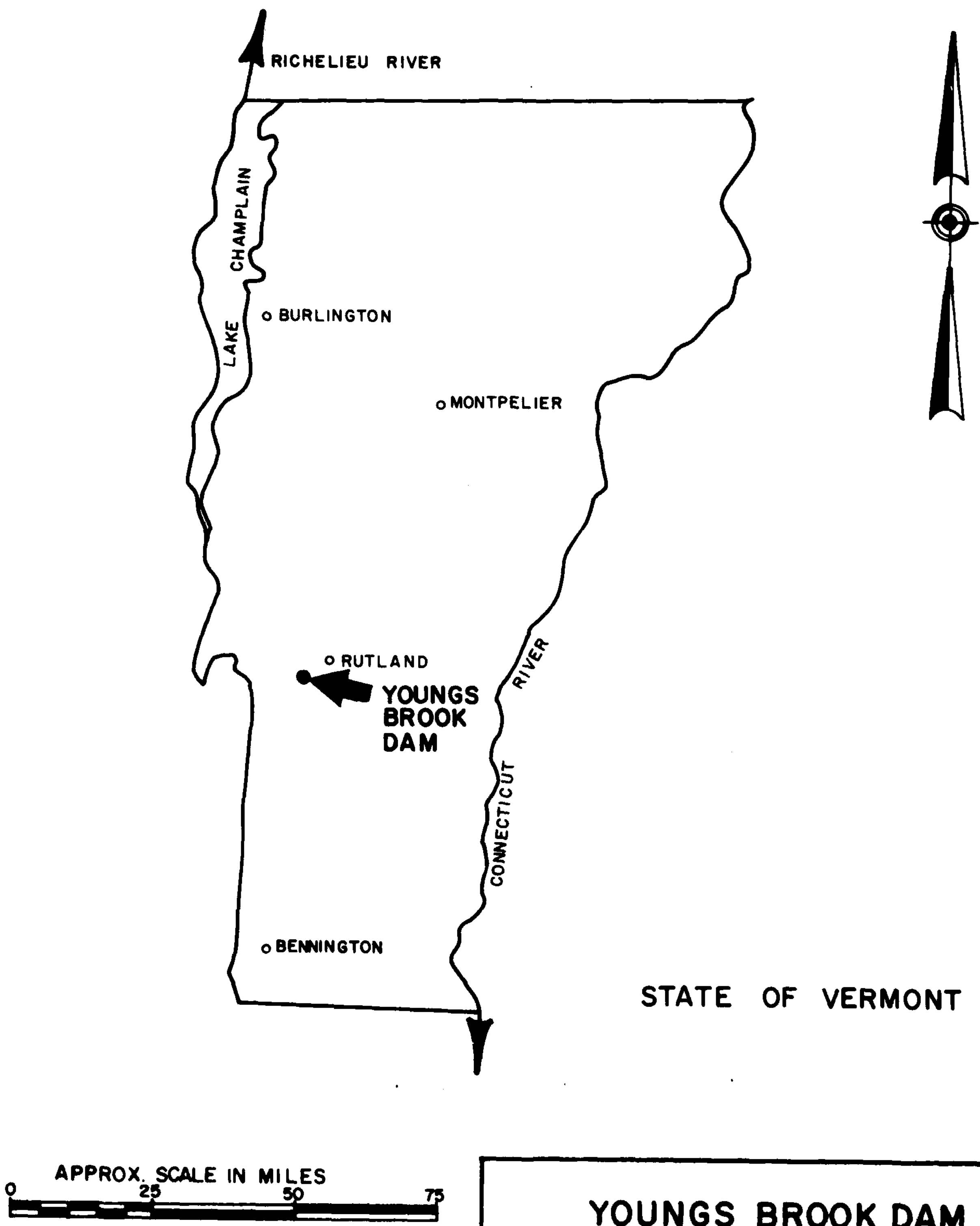
YOUNGS BROOK DAM
VICINITY MAP

GORDON E. AINSWORTH & ASSOCIATES, INC.

Engineers, Surveyors and Planners

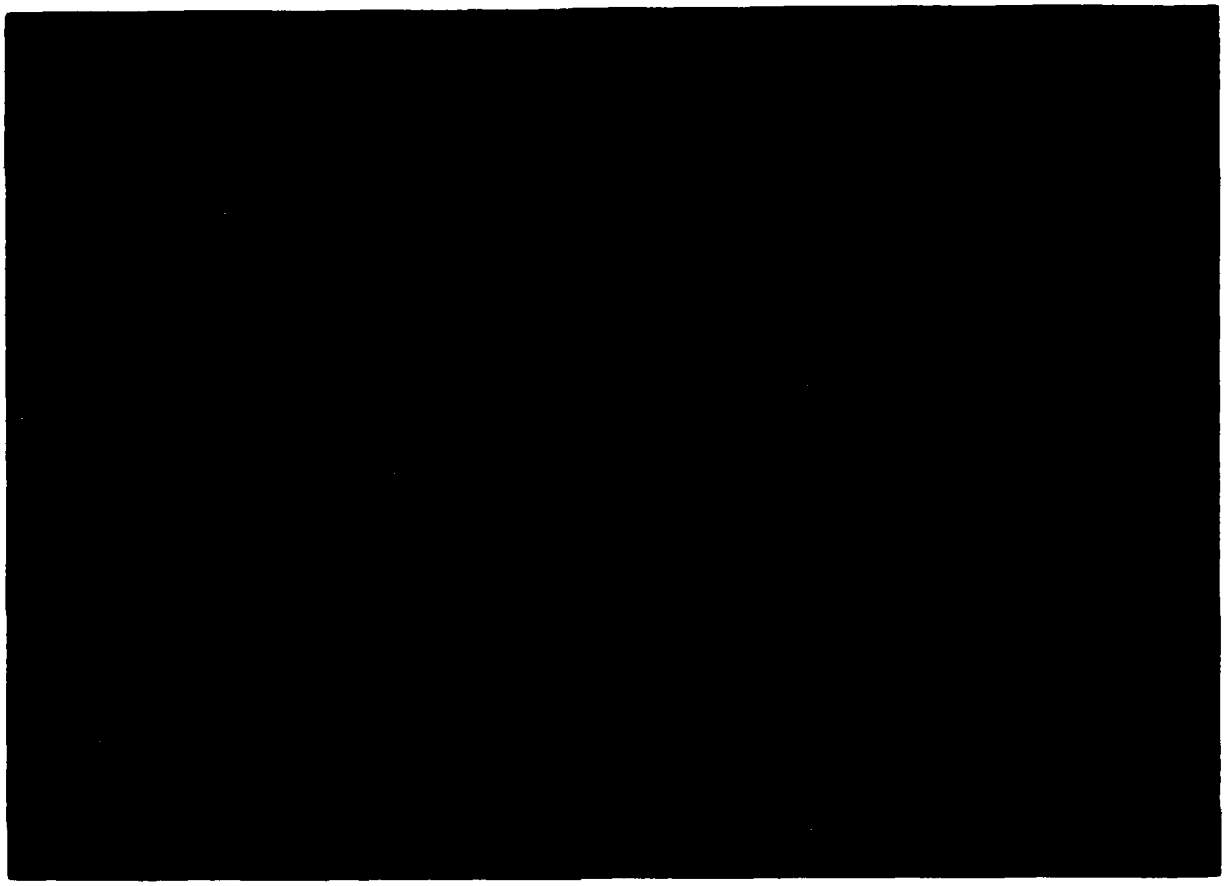
20 SUGARLOAF ST. SOUTH DEERFIELD, MASS. 01373

1x



YOUNGS BROOK DAM LOCATION MAP

GORDON E. AINSWORTH & ASSOCIATES, INC.
Engineers, Surveyors and Planners
20 SUGARLOAF ST. SOUTH DEERFIELD, MASS. 01373



Overview Photo - Youngs Brook Dam - 11/30/79

9734

APPENDICES

TABLES

b) Length of Weir - 19.3 feet.

c) Crest Elevation - w/o flashboards - 843
- with flashboards - N/A

d) Gates - None.

e) Upstream Channel - Sloping bottom of reservoir.

f) Downstream Channel - Concrete-paved channel for about 10 feet, then a bedrock channel along left abutment down to the toe of the dam.

g) General - No comment.

2) New Spillway

a) Type - Free overfall, with concrete weir section about 20 feet deep founded on rock.

b) Length of Weir - 28 feet.

79 4 c) Crest Elevation - w/o flashboards - 841
- with flashboards - 842.7 ±
(Wooden flashboards are in disrepair and Operator plans to permanently remove them.)

d) Gates - None.

e) Upstream Channel - Sloping bottom of reservoir.

f) Downstream Channel - Spillway weir drops off over 20 feet onto bedrock bottom of discharge channel, which is parallel to but much lower than old spillway discharge channel.

g) General - No comment.

j. Regulating Outlets

1) Outlet Conduit

a) Invert - Intake and discharge EL 828 ±.

b) Size - 2-feet square.

c) Description - Short concrete conduit through right training wall of new spillway discharge channel emptying into the channel.

d) Control Mechanism - Slide gate on upstream end of conduit on face of concrete wall with floor stand and handwheel located directly above and just to right of new spillway.

e) Other - No comment.

2) Drain Pipe (Mud Pipe)

a) Invert - Intake EL 798 ±, discharge unknown.

b) Size - 8-inch diameter.

c) Description - Probably cast iron pipe, extending from bottom of water supply manhole, through the dam, and discharging into Youngs Brook somewhere downstream.

d) Control Mechanism - 8-inch gate valve next to water supply manhole just upstream of dam. Valve operator normally submerged.

e) Other - No comment.

3) Water Supply Main

a) Invert - Intake EL 803 ±, discharge unknown.

b) Size - 12-inch diameter.

c) Description - Probably cast iron pipe, extending from the water supply manhole, through the dam, and continuing to the distribution system.

d) Control Mechanism - 12-inch gate valve next to water supply manhole just upstream of dam. Valve operator normally submerged.

e) Other - An 8-inch diameter supply main from the small reservoir in the bottom of the main reservoir joins with the 12-inch supply main just downstream of the 12-inch control valve. The 8-inch main is valved separately just upstream of its junction with the 12-inch main.

SECTION 2

ENGINEERING DATA

2.1 Design Data

It is believed that the dam was designed in the early 1920's for a private owner. The original designer of the dam is unknown. Only one plan of the reservoir (see Appendix B2-1) and a sketch of the water supply manhole (see Appendix B3-5) were available for this dam.

No other design data or drawings were available. The construction specifications were not available.

2.2 Construction Data

a. Initial Construction

It is also believed that the dam was originally constructed in the 1920's. The construction contractor for the dam is unknown. No records on the actual construction of the dam are known.

b. Modifications

According to the present Operator of the dam, the original spillway was damaged by a flood in 1947. In 1948 and 1949 a new spillway was constructed to the right of the old spillway. The designer and construction contractor for the new spillway are unknown. No records, plans, or specifications for the new spillway construction are known.

c. Repairs and Maintenance

According to the Operator, the reservoir used to be drained and cleaned every three years. Photos on Appendices B3-1 through B3-3 show the reservoir drained and being cleaned on two separate occasions. The reservoir was reportedly last drained and cleaned in 1972, and 3 feet of gravel was then placed on the reservoir bottom. No other records of any repairs or maintenance of the dam or reservoir are known.

d. Pending Remedial Work

During the visual inspection, the Operator indicated that he intended to permanently remove the 20-inch high wooden flashboards from the new spillway within the week. The flashboards were in a state of disrepair. The Owner has no plans for any other pending remedial work.

2.3 Operation Data

a. Inspections

Only one inspection report was available and it is included as Appendix B3-4. The inspection was performed by Stephen H. Haybrook, on behalf of the State of Vermont, in June 1952. The report contains some background data on the dam and it is indicated that the dam was generally in satisfactory condition. Also, it is noted that the embankment slope may be steep and that the concrete was in good condition but there was some spalling.

The Operator of the dam indicates that he routinely visits and checks the dam once a week. In the winter, he reports that he checks the spillway for ice build-up every other day. There are no records of any of these visits.

b. Performance Observations

There is no instrumentation in the dam. Other than observations made during the inspection previously discussed in Section 2.3.a, there are no other known records of performance observations.

c. Water Levels and Discharges

There are no known records of routine water levels and discharges from the dam. The Operator indicated that the high water level in the spring was typically about 3 feet higher than the new spillway crest.

d. Past Floods

The only known flood at the dam is the one which occurred in 1947 and damaged the old spillway. There are no known records of this or of any other floods which may have occurred at the dam.

e. Previous Failures

The only known previous failure of the dam occurred as a result of the flood in 1947. The flood damaged the original spillway. No records or details of the flood damage are known.

2.4 Evaluation

a. Availability

As listed in Appendix B1, some engineering data and records are available in the files of the Owner and of the Dam Safety Engineer of the Vermont Department of Water Resources. This data was reviewed, and copies of the records significant to the dam (a limited amount) are included in chronological order in Appendices B2 and B3. Discussion of the data starts at the beginning of this section of the report.

b. Adequacy

Available data consisted of a drawing of the reservoir as it existed in 1931, capacity data for the reservoir with a rough sketch of the water supply manhole and piping, several old photos of the reservoir while drained, and one inspection report. Such data as the design calculations, construction specifications, data on the foundation and embankment soils, and detailed operation and performance data were not available. The lack of such in-depth engineering data does not permit a comprehensive review. Therefore, the adequacy of this dam could not be assessed with respect to reviewing design, construction, and operation data.

c. Validity

Based on visual observation and checking, the limited data available generally appears valid. Exceptions noted are:

1) The existing drawing on Appendix B2-1 shows that the concrete wall on the upstream face of the dam does not extend all the way to the right side of the reservoir. Visual inspection (see Overview Photo) indicates that the concrete wall does extend all the way to the right side of the reservoir and that it appears longer in relation to the rest of the dam than portrayed on the drawing.

2) The old sketch of the water supply manhole on Appendix B3-5 indicates a valve in the intake pipe to the supply manhole located about at the wall of the manhole. No such valve is shown on the existing drawing on Appendix B2-1. It is not known if the valve actually exists.

SECTION 3

VISUAL INSPECTION

3.1 Findings

a. General

Youngs Brook Dam was inspected on November 8, 1979. The inspection party (see Appendix A-1) was accompanied by Mr. Joseph Skaza, Water System Operator, who represented the Owner. The weather was overcast. The water surface was at about EL 841.3, about 0.3 of a foot above the crest of the new spillway, and was drawn down during the inspection by opening the outlet slide gate. The Visual Inspection checklist is included as Appendix A, while selected photos taken during the inspection are included in Appendix C. Appendix C-1 is a photo index map. The Overview Photo at the beginning of the report as well as a couple of the photos in Appendix C are aerial photos taken from a helicopter on November 30, 1979.

b. Dam

The entire downstream slope of this dam is forested with trees up to 12 inch size (see Overview Photo and Photos C-2A, C-2B, and C-3). These trees are a mixture of deciduous (mostly birch) and small evergreens. The tree types and the tree population on the slope are quite different from those on the adjacent naturally-forested areas (see Overview Photo and Photo C-10A). The photo on Appendix B3-1, taken in spring 1933, from a point near the upstream end of the empty reservoir, indicates that these trees had not grown above the crest elevation at that time, which was about 13 years after the dam had been built. Photos on Appendices B3-2 and B3-3, believed taken about 1950, also indicate no significant tree growth above the top of dam.

Both the upstream and downstream slopes of this dam are rather steep, being overall about 1.15H:1V and 1.5H:1V, respectively. A shallow hole dug into the surface of the downstream slope revealed a gray, widely-graded, slightly clayey glacial till. Such a material in the embankment can remain standing with steep slopes under static conditions so long as it does not become saturated with water. Upon saturation, e.g., due to overtopping or seepage while the downstream face is frozen, slumping is likely to occur.

Due to the steep slope, the surface materials on the downstream slope are slowly creeping downhill, as may be inferred from the bowed shapes of the trees in Photo C-3A. The surface of the slope is largely barren between the trees, although the leaves on the ground tend to obscure this fact, as shown in Photo C-3B. In addition, there is a small scarp a few inches high just below the downstream crestline near the middle of the dam. This scarp may be the result of frost effects.

Clear seepage through this dam was observed exiting at the downstream toe in one location at a rate of about 2 to 4 gpm. The location of this seep is at about Sta 0+90 just to the left of an 8-foot high berm-like pile of dirt at the toe. It appears as though the seep may have been pushed to the left and concentrated there when the pile of dirt was placed. One can infer from this relationship that seeps may have been occurring earlier and that the berm material was placed to stabilize the area. In any case, the seepage is exiting not more than one foot above the toe. Also, the left abutment and perhaps half of the dam appear to be founded on bedrock. The seep may be at the interface between the dam and its foundation.

Clear seepage was also observed exiting from the bedrock abutment on the left side of the new spillway at a rate of less than 2 gpm. The location is at the extreme right in Photo C-7A about 6 to 8 feet below the spillway crest.

The upstream concrete wall on the right side of the dam is severely distorted and cracked. It is shown in Photo C-4A. A view along the wall from the right abutment, Photo C-4B, shows that the wall is tilted upstream. The soil behind the wall has settled and formed a small scarp 3 feet behind the wall, as shown in Photo C-5A.

c. Appurtenant Structures

1) Water Supply Manhole and Piping

The water supply manhole is located upstream of the dam about 90 feet from the left abutment. The manhole and its associated piping were not inspected because they were completely underwater, as they are normally.

12 3 The only drawing available showing the layout of the water supply manhole and piping is included as Appendix B2-1. From this drawing and from field measurements shown on Appendix B2-2, it appears that the top of the manhole is at about EL 827.5, or 13.5 feet below the normal water level at the new spillway crest. The water main control valves next to the manhole appear to be several feet higher. According to the Operator, in order to get to the manhole or operate the valves, the reservoir must be lowered using the outlet slide gate and conduit next to the new spillway. Such difficult access to the valves is a potential problem if the valves need to be operated or closed rapidly. Field measurements on Appendix B2-2 indicate that the outlet conduit might be able to lower the water level to about EL 828, about 13 feet below the new spillway crest, or about to the elevation of the top of the manhole. Access to the manhole and valves when the reservoir is lowered is aided by a partial set of metal steps embedded in the concrete wall (steps visible in the right of Photo C-6A).

The outlet of the mud pipe, or drain, is supposed to discharge into the stream channel downstream of the dam. The outlet end of the mud pipe was not able to be found during the visual inspection.

The drawing on Appendix B2-1 indicates a small reservoir in the bottom of the larger reservoir. The body of water shown in the right of an old photo on Appendix B3-1, which was taken in spring 1933 when the larger reservoir was drained for cleaning, appears to be the small reservoir.

2) <u>Outlet Slide Gate and Conduit</u>

The outlet slide gate is on the upstream face of the concrete wall just to the right of the new spillway. It is controlled by a floor stand with handwheel located directly above on top of the wall (see Photo C-6A). The slide gate was completely underwater and was not inspected. The inspection checklist for the control mechanism is on Appendix A-5.

The outlet conduit appears to be a very short 24-inch square concrete section which runs from the slide gate to a discharge opening through the right training wall of the new spillway just downstream of the face of the spillway. A close-up of the discharge end of the conduit is shown in Photo C-6B. The outlet conduit appeared to bend to the right looking upstream and, therefore, only the inside of its downstream end was observable. The inspection checklist is on Appendix A-6.

The bottom left corner of the concrete overhang to the left of the outlet slide gate control mechanism is spalled off (see Photo C-6A). Also, there is some hairline cracking and efflorescence on the concrete wall in the area of the mechanism. Otherwise, anchorage of the floor stand appeared sound.

There is some surface rust on the floor stand anchorage and handwheel. The top of the handwheel is visible in Photo C-8A. The handwheel and slide gate were operated but they moved stiffly. There was no grease on the stem as it rose out of the top of the handwheel.

With the slide gate open 9 inches, it took about 1.25 hours to drop the reservoir from the new spillway crest to 0.6 of a foot below, which averages about half a foot an hour. Photo C-7A shows the downstream end of the outlet conduit while it was discharging, while Photo C-7B was taken with the slide gate closed. Some small flow (visible in Photo C-6B) was observed out of the end of the outlet conduit when the gate was closed. It is presumed that the slide gate leaks slightly.

Along the top inside of the discharge end of the outlet conduit there were remains of wooden forms. These are just visible in Photo C-6B.

3) Spillway and Discharge Channel

The dam has two spillways at its left end (see Overview Photo and Photos C-5B and C-8A). The older, narrower, higher spillway is at the left abutment. It consists of a concrete weir cap on top of bedrock followed by a inclined drop of several feet into a steeply sloping chute discharge channel with a natural bedrock bottom. The inspection checklist is on Appendix A-9.

The newer, wider, lower spillway is just to the right of the old spillway (see Photo C-5B). The right training wall of the old spillway is the left training wall of the new spillway, but the new discharge channel is much lower and has much less slope than the old discharge channel. The new spillway consists of a concrete weir followed by a nearly vertical drop of over 20 feet to the bedrock bottom of the new discharge channel. The downstream face of the new spillway is all concrete and is slightly convex with an overall slope of about 1H:1.6V or steeper (see Appendix B2-2). The incline on the face is evident in Photos C-8A and C-9A, with an overall view of the face in Photos C-7A and C-7B. The inspection checklist for the new spillway is on Appendix A-8.

The approach channel is the same for both the old and new spillways. It consists of the sloping reservoir bottom, which appeared to be silt covered with leaves. There were some logs and debris in front of the new spillway and on the weir crest as seen in Photo C-5B.

The new spillway had 20-inch high wooden flashboards that were in disrepair (see Photos C-5B and C-8A). During the visual inspection, the Operator indicated that he intended to permanently remove the flashboards within the week. The cable across the new spillway, visible in Photos C-5B and C-8A, is used as a safety line when the Operator has to cross the spillway weir, e.g., in the winter when he chops ice build-up off of the weir.

The new concrete spillway weir is in fair condition. Four vertical cracks were observed in the top of the weir, with some of the cracks extending down the downstream face. Also, there appeared to be a horizontal crack on the upstream vertical face of the weir about 3 feet below the crest. This crack did not appear to extend through to the downstream face. No particular seepage from any spot on the downstream face of the weir was observed when flow over the weir was stopped (see Photos C-7A and C-7B).

The old concrete spillway weir was in fair condition. One vertical crack about 1/16-inch wide was observed in the top of the weir at about its center line. Also, there was a small seep at about the center line of the weir cap where its downstream edge contacts bedrock.

The short, left concrete training wall of the old spillway is in poor condition. As seen in the background of Photo C-8A, the upstream end is crumbling and there is spalling in other spots.

The right concrete training wall of the old spillway (i.e., the common training wall between the two spillways) is also in poor condition. Both the right and left sides of the wall are visible in Photos C-8A and C-8B, respectively. The upstream end of the wall next to the weir is crumbling, there is exposed reinforcing on top of the wall, and there is spalling, cracking, and efflorescence in various places. A vertical crack about 1/2 inch wide and about 5 feet high full height of the wall is visible at the center of Photo C-8B. The wall is only about 1-foot thick with vertical sides and appears to sit precariously at the edge of the much deeper new spillway discharge channel to its right (see Photo C-7B).

The right, concrete training wall of the new spillway is shown in Photos C-9A and C-9B. The wall is quite substantial, being about 2 feet thick at its top with a batter on its exposed face. However, as evident in the photos, there is significant surface cracking with efflorescence and rust stains coming from the cracks and from several horizontal construction joints.

9650 There are several logs lying in the bottom of the new spillway discharge channel as well as some brush and small trees growing (see Photos C-7B and C-9B.) Also, the old spillway discharge channel is becoming overgrown with brush and small trees.

d. Reservoir Area

No excessive reservoir sedimentation was observed. Reportedly, the reservoir was last drained and cleaned in 1972 and 3 feet of gravel was then placed on the bottom. No potential landslide areas were noted around the reservoir. Also, there is no potential hazard due to backwater flooding of the reservoir. No features were observed that might cause excessive alteration of the drainage area or increased inflow.

e. Downstream Channel

The downstream channel is an unnamed stream, popularly called Youngs Brook, which runs from the end of the spillway discharge channel about 1.2 stream miles to the Clarendon River. The Drainage Area Map, Appendix D-1, maps the downstream channel and also indexes photos that cover the downstream area. Photo C-10A is an aerial overview of the dam and reservoir looking downstream. Youngs Brook roughly follows the evergreen trees in the photo. Photo C-10B is an aerial overview taken from about the Clarendon River looking upstream. The reservoir is not quite visible at the top center of the photo about 1/2 inch below the top edge.

For about the first 0.7 of a mile downstream of the dam (to about Sta 36+00), Youngs Brook is a fairly steep, rocky stream in a forested valley. Only about 0.2 of a mile downstream (about at Sta 10+00), the stream runs under a road through about a 12-foot diameter structural plate corrugated metal pipe culvert (see Photo C-11A).

For about the next 0.5 of a mile from about Sta 36+00 to the Clarendon River, Youngs Brook flattens out in a tree-lined channel through open farmland. Photo C-11B is looking downstream from about Sta 36+00 where the stream emerges from the forested valley and starts across the open farmland. This same spot can be located in the upper right of Photo C-10B where the tree-lined stream emerges from the evergreens just below the road. The red barn just visible in the background of Photo C-11B is seen clearly at about the center of Photo C-10B next to the highway.

1737 In Photo C-10B, Youngs Brook can then be traced as it emerges from the forested valley at the upper right, runs down to the right next to a tree line, turns just off of the right edge of the photo, crosses Town Route 133 (formerly a State highway), and then runs toward the bottom left of the photo next to the trees between the fields. The stream joins the Clarendon River just off of the bottom left of the photo. The hazard area appears to be the houses and farm located along Route 133. Photos C-12A and C-12B are close-ups of the hazard area. The red barn in the close-ups is the same one visible at about the center of Photo C-10B.

3.2 Evaluation

The upstream and downstream slopes of this dam are steep and their stability should be evaluated.

The water lines that pass through the dam are continuously under pressure. Leakage from these lines could cause a breach. See Section 6 for further discussion.

The roots of the trees on the downstream slope offer the potential of seepage paths opening through the dam toward root holes. These trees should be removed and the root holes filled with appropriate materials. Also, the slope should be protected against erosion.

The seepage through the dam and the left abutment should be monitored regularly to determine whether any changes are occurring with time. Also, the small seep at the concrete weir cap/bedrock contact at about the center of the old spillway should be watched.

The concrete wall on the right side of the upstream face is severely distorted and should be replaced.

The reservoir should be drained with the outlet conduit and the outlet conduit and slide gate should be thoroughly inspected.

The remains of the wooden forms should be removed from the top inside of the outlet conduit. Leakage of the slide gate should be stopped and the slide gate and handwheel should be greased to permit freer operation. Further rusting of the handwheel, floor stand, and floor stand anchorage should be prevented by painting.

While the reservoir is drained with the outlet conduit, the control valves next to the water supply manhole should be checked and operated.

17 3 A means should be provided to operate the water main control valves without first having to lower the reservoir with the outlet conduit.

The reservoir should be drained further with the mud pipe and the water supply manhole and intake piping should be inspected. The outlet end of the mud pipe downstream of the dam should be located and kept clear. The small reservoir and supply main from the small reservoir, both in the bottom of the larger reservoir, may not be important anymore since the larger reservoir is no longer used as a primary source of water supply.

When the reservoir is drained, it should be checked for sediment and cleaned if necessary, since the last draining and cleaning was reportedly in 1972.

Spalled concrete on the overhang just to the left of the outlet slide gate control mechanism should be repaired. Also, hairline cracking and efflorescence in the area of the concrete wall around the control mechanism should be watched and repaired if it gets worse.

Logs and debris in front of and on the new spillway weir should be removed.

The operator should carry out his plans to permanently remove the flashboards from the new spillway weir.

Cracks in both old and new concrete spillway weirs should be repaired.

Crumbling and spalling of the left concrete training wall of the old spillway discharge channel should be repaired. It may be easier to replace this short wall entirely.

The significant surface cracking and resulting efflorescence and rust staining on the large right concrete training wall of the new spillway discharge channel should be repaired.

The common concrete training wall between the two spillway discharge channels is in poor condition. However, it appears to serve no useful purpose. The wall could probably be removed and thereby eliminate an unnecessary flow obstruction between the spillway discharge channels.

The brush and trees should be cleared from both spillway discharge channels. Also, the logs should be cleared from the bottom of the new spillway discharge channel.

SECTION 4

OPERATION AND MAINTENANCE PROCEDURES

4.1 Operation Procedures

a. General

Youngs Brook Reservoir is presently used only as an emergency source of water supply for the Town of West Rutland. The reservoir used to be the main source of supply for the Town but it has been replaced by water supply wells.

There are no written operation procedures for the dam and reservoir. The Operator normally visits and checks the dam once a week. During the winter, the Operator visits the dam every other day to control ice build-up on the spillway by chopping the ice away. The results of the visits to the dam are not recorded.

Under normal operation, the 12-inch valve on the water supply main is left open and the reservoir is connected to the water system even when the Town's well system is in use. The mud pipe, or drain, is normally closed. Also, the outlet conduit is closed during normal operation. The new spillway is normally in operation with water flowing over its concrete weir crest. The 20-inch high flashboards that are presently in place are in poor condition and do not have much of an effect. During the visual inspection, the Operator indicated that he intended to permanently remove the flashboards within the week.

b. Emergency Action Plan and Warning System

During storm events the level of the reservoir is visually monitored by the Operator. At these times the outlet conduit may be opened to increase outflow. There is no other emergency action plan and warning system in effect for Youngs Brook Dam.

4.2 Maintenance Procedures

a. General

The reservoir used to be drained and cleaned every three years (See Section 2.2.c). There are no other current maintenance procedures for the dam and reservoir and their operating facilities. Brush and trees cover the downstream slope and it appears that growth on this slope has been allowed since the original construction. The last significant maintenance was the reservoir draining and cleaning in 1972.

In 1947 a flood occurred which damaged the old original spillway, which is seen in the background (on the left side of the dam) in Photo C-8A. Large concrete blocks now exist in the downstream channel. These may be parts of the old spillway.

In 1948 and 1949 a new spillway was constructed, which is shown in the foreground in Photo C-8A. The new spillway is about 9 feet longer and 2 feet lower than the old spillway. Previously, the embankment extended to the left all the way to the right training wall of the old spillway. (This training wall is now on the left side of the new spillway discharge channel.)

This change in the spillway probably improved the overall structural stability of the dam, but no information is available concerning the design or the methods used to ensure good contact between the embankment and the rock foundations.

At present there are seeps that exit from the bedrock beneath the left training wall downstream from the new spillway, as noted in Section 3.1.b. The exit points are only a few feet downstream from the downstream face of the spillway. Alternate freezing and thawing may eventually cause movements of various slabs of the rock foundation. Movement of such slabs could cause a release of water much like that which now occurs each spring over the spillway. Erosion would not proceed too far since the flow would be confined by bedrock at the left and the new spillway at the right. However, this zone should be inspected and photographed each year after the spring thaw to determine whether any significant changes are occurring.

As pointed out in Section 3.1.b, trees have been allowed to grow on the downstream face of the dam during the last half century. These trees could endanger the stability of the dam by creating channels for piping. The trees currently slow erosion of the steep downstream slope. Therefore, removal of the trees will lead to erosion of that slope unless erosion protection is placed immediately. The trees and roots should be removed, but only after careful consideration of the need to alter the downstream slope based on structural stability considerations (see Section 6.1).

6.4 Seismic Stability

This dam is in Seismic Zone 2. Therefore, according to recommended guidelines (Reference 1), a seismic stability analysis is not warranted.

SECTION 6

EVALUATION OF STRUCTURAL STABILITY

6.1 Visual Observations

The downstream slope of 1.5H:1V is steep. Its surface is bare between the trees, creep is occurring, and a small scarp a few inches high is visible near the downstream crestline. The latter scarp appears to be shallow and may be due to frost effects.

The upstream slope is also steep, about 1.15H:1V overall, but it was not observable on the day of inspection to determine its condition. The concrete wall on the right side of the upstream face is severely distorted. That distortion appears to be a local stability problem of the wall itself, rather than being related to the overall stability of the upstream slope.

Analysis of the stability of both slopes under critical conditions is required. For that purpose it is necessary to make borings in the dam, make strength measurements, and measure the water levels within the embankment. Based on the existing slope, it would appear unlikely that the downstream slope would be proven stable if it were saturated. Such a condition could occur when the downstream face freezes to substantial depth. To avoid this condition it may be necessary to drain the water within the downstream slope or to add a pervious downstream shell. Such drainage provisions do not appear to exist in this dam.

Water lines under pressure pass through the dam. The composition, condition, and foundations of these lines are not known. Leakage could lead to erosion and breach of the dam. It is recommended that the pressure be relieved from these lines until an overall evalution of the dam is carried out.

In view of the steep slopes, the fact that the dam is apparently homogeneous in cross section, and the water supply is no longer required, the reservoir level should be lowered immediately upon receipt of this report to about EL 828 (i.e., to the invert of the outlet conduit about 13 feet below the new spillway crest) and maintained at that level to the extent possible by leaving the outlet conduit open.

6.2 Design and Construction Data

There are no design and construction data available.

6.3 Post-Construction Changes

According to the Operator, Mr. Joseph Skaza, sediment formerly was removed from the reservoir once every three years. The last cleaning was in 1972.

The flood routing was not carried any farther downstream than Sta 60+00 because no more structures or features that would be impacted exist in the next 500 feet before Youngs Brook joins the Clarendon River. The Clarendon River is wide with a broad vacant flood plain in this area that appears able to absorb a failure of Youngs Brook Dam. The nearest downstream community of West Rutland is about 2000 feet farther downstream and about 1500 feet to the left of the river. West Rutland would be unaffected by a failure of Youngs Brook Dam.

In summary, it appears that the increase in flow due to a failure of the dam would wash out an unpaved Town road and large culvert, flood about 35 acres of farmland on both sides of Town Route 133 to a depth of less than a foot, do some damage to Town Route 133, and flood to a depth of less than a foot around the foundations of about 8 houses, 2 house trailers, 3 barns, and miscellaneous outbuildings. The high flow velocity of 10 fps around most of the structures and the velocities of 7 to 10 fps over the farmland would probably do significant erosion damage. Total economic loss is judged appreciable. Loss of life is judged unlikely, but more than a few inhabitable structures are affected. Therefore, according to recommended guidelines of the Corps of Engineers (Reference 1), the dam is classified as having a significant hazard potential.

Just prior to the dam breach, outflow from the dam was 1370 cfs, and flow 2000 feet downstream was about 3.8 feet deep at about 20 fps. After the breach, peak outflow from the dam increases about 18 times to 24,000 fps. This causes flow at Sta 20+00 to increase about 10 times to 14,000 fps, and the water surface to rise from 3.8 to 10.2 feet deep, an increase of 6.4 feet, which floods an area about 100 feet wide. Velocity increases about 1.5 times to 29 fps.

About 1000 feet downstream of the dam, Youngs Brook runs under an unpaved Town road through a structural plate corrugated metal pipe culvert estimated to be 12 feet in diameter (see Photo C-11A). With about 15 feet of headwater (i.e., water level with the road), the culvert has an estimated capacity of about 1500 cfs (Reference 17). Prior flow of 1370 cfs could probably pass through the culvert without flooding the road. However, the increase in flow to at least 14,000 cfs (computed downstream at Sta 20+00) due to the dam failure would certainly be out-of-channel and going over the roadway and would probably wash out the road and the culvert.

At Sta 47+00, several hundred feet upstream of houses along Town Route 133 (formerly a State highway, see Photos C-10B, C-12A and C-12B), peak flow increases about 6 times to 8400 cfs after the breach. This causes the water to rise from 2.3 to 3.6 feet deep, an increase of 1.3 feet, which floods an area about 1050 feet wide. Velocity decreases slightly to about 10 fps. The channel banks and fields are estimated at EL 560. Prior flow at EL 559.3 appears to be in-channel. The 1.3-foot increase to EL 560.6 due to the dam failure appears to be out-of-channel and spread over the farmland to the right of the stream to a depth of about half a foot. About 10 acres of farmland would be involved. This shallow flow would continue downstream and flood around the foundations of the houses along Route 133 to a depth of about half a foot, but the first floors would probably not be flooded. However, it appears that as many as 8 houses, 1 house trailer, 2 barns, and miscellaneous outbuildings would be involved in this minor flooding (see Photo C-10B). The high flow velocity of 10 fps would probably erode the farmland, ground around the foundations of the structures, and do some damage to Town Route 133.

At Sta 60+00 the flood plain widens significantly. Channel banks and farm fields to the right of the stream are estimated at EL 520. Prior flow at EL 520.1 appears to just flood the fields. The 0.2-foot increase to EL 520.3 due to the dam failure appears to increase flooding of the farmland slightly. About 25 acres of farmland would be involved. The ground around a house trailer and a barn (see bottom right corner of Photo C-10B) would be flooded. The moderate flow velocity of 7 fps would probably cause some erosion of the farmland.

TABLE 5.2

YOUNGS BROOK DAM

DAM FAILURE ANALYSIS

CONDITIONS — Top of Dam EL 846
New Spillway Crest EL 841
Total Project Discharge Capacity at
Top of Dam = 1370 cfs ±
due to two spillways. Outlet works closed.

	Approx. Peak Flow (cfs)	Time to Peak Flow (hours)	Approx. Max. Water Surface			
			Elev. (feet)	Depth (feet)	Top Width (feet)	Avg. Vel. (fps)
PRIOR FLOW AT TOP OF DAM						
Inflow = Outflow = Total Project Discharge Capacity at Top of Dam						
Start Routing at Top of Dam						
Dam	1370	--	846.0	43.0	--	--
Sta 1+00	1370	--	805.1	5.1	--	15
Sta 20+00	1370	--	703.8	3.8	--	20
Sta 36+00	1370	--	602.8	2.8	--	17
Sta 47+00 Near Houses	1370	--	559.3	2.3	--	12
Sta 60+00	1370	--	520.1	1.1	--	7
BREACH AT TOP OF DAM						
Inflow = zero						
Start Routing at Top of Dam						
Start Breach W.S. at Top of Dam						
Time of Failure = 0.00 hour						
Breach Time = 0.03 hour						
Breach Width = 50 feet						
Breach Depth = 43 feet						
Trapezoid, 0.5H:1V side slopes						
Dam	24,000	0.03	846.0	43.0	--	--
Sta 1+00	--	--	--	--	--	--
Sta 20+00	14,000	0.03	710.2	10.2	100	29
Sta 36+00	11,900	0.05	607.1	7.1	100	31
Sta 47+00 Near Houses	8,400	0.07	560.6	3.6	1050	10
Sta 60+00	5,500	0.10	520.3	1.3	3000	7

USGS map that the cross sections were developed from and of the limited 8-point cross section accepted by the program. The third and sixth point on each cross section are defined as the overbank points. Therefore, distinguishing between in-channel and overbank flow cannot be done reliably by simple comparison of computed water surface depth with the defined overbank points. It must be done by judging the calculated quantity, depth, width, and velocity of flow against the real channel cross-section and configuration as it exists.

b. Results of Analysis

The results of the dam failure analysis using the HEC-1 DB program are summarized in Table 5.2. PRIOR FLOW AT TOP OF DAM establishes initial conditions downstream due to steady state total project discharge capacity at the top of dam with no dam breach. The computer input and selected pages of the computer output start on Appendix D-17. Results for all stations are summarized in Table 5.2.

BREACH AT TOP OF DAM is a major sudden failure of the dam under the conditions previously discussed in Section 5.5.a. Results are summarized in Table 5.2 for all stations, with the computer input and selected pages of the computer output starting on Appendix D-21.

From the computer listing and plot of the breach hydrograph on Appendices D-22 and D-23, note that the standard calculation interval selected (1 minute = 0.017 hours) was short enough to permit the interpolated breach hydrograph at the standard time interval to closely approximate the computed breach hydrograph. Only the interpolated breach hydrograph is routed downstream.

Appendix D-24 is a computer plot of the complete outflow hydrograph during and after the breach.

c. Hazard Evaluation

For a sudden major dam failure, BREACH AT TOP OF DAM, the computed maximum water surface elevation for each downstream station is tabulated in Table 5.2 (Sta 1+00 not used for breach routing) and is plotted on each cross section beginning on Appendix D-15. The top widths of flow determined from each cross section are tabulated in Table 5.2 and are plotted on Appendix D-1 to define the limit of the hazard area, i.e., the limit of flooding due to the dam failure.

The average velocity of peak flow (flow divided by total flow area) is also listed in Table 5.2 for each downstream station for both flow cases. For the dam breach case, the flow area calculation is shown on each cross section plot starting on Appendix D-15, and consists of storage for the channel reach defined by the cross section divided by reach length. The channel storage was computed by the HEC-1 DB program for both flow cases.

is shown by the computer plot on Appendix D-12. Total project discharge capacity at the top of the dam is due only to the two spillways (outlet works assumed closed) and is equal to 1370 cfs, or about 72% of the test flood peak outflow.

5.5 Dam Failure Analysis

a. Failure Conditions

In order to evaluate the downstream hazard, the flow just prior to and then due to an assumed major failure or breach of the dam was routed downstream using the HEC-1 DB program. Stream conditions just prior to and after the assumed failure were compared. Corps of Engineers' criteria call for breaching the dam with no inflow flood and with the water surface static at the top of the dam, or static at the test flood pool if a test flood of full PMF does not overtop the dam. Since the overtopping analysis shows that the test flood of one-half PMF does overtop the dam, the dam breach was begun at time zero with the water surface at the top of the dam. The contents of the reservoir were routed through the breach as the breach progressed.

To model a sudden major dam breach, maximum breach geometry was selected as follows: constant trapezoidal shape with moderate 0.5H:1V side slopes, breach width across the bottom of the trapezoid equal to about 40% of the dam length at mid height (approximately 50 feet), and a breach depth below the top of the dam equal to 43 feet (down to EL 803), which approximates a full depth failure that would completely drain the reservoir. Breach geometry is illustrated on Appendix D-20.

Breach time, or time for the breach width to progress from the top to the bottom of the dam, was selected so that the peak outflow using the HEC-1 DB program would approximate that computed by the Corps of Engineers' "Rule of Thumb" method using the same breach width and depth. No additional flow from the spillway was considered, since it is assumed for this dam that the breach could include all or part of the spillway. The selection of breach time is shown on Appendix D-20. Rule of Thumb peak breach outflow is the same as total peak outflow from the dam and is equal to about 23,700 cfs. A breach time of 0.03 hours, or 1.8 minutes, was selected for the HEC-1 DB program, which results in a peak outflow of about 24,000 cfs.

The inputted cross sections defining average downstream channel reaches were developed from and are located on the USGS map included as Appendix D-1. Hand plottings of the cross sections start on Appendix D-15. Normal depth channel routing was performed by the HEC-1 DB program using the Manning's n values for left over bank, channel, and right overbank as listed on each cross section plot. The overbank points and the actual channel section in between are only an approximation of the true natural channel. This is because of the constraints of the small scale

TABLE 5.1

YOUNGS BROOK DAM

OVERTOPPING ANALYSIS

CONDITIONS – Total Drainage Area = 1.85 Square Mile
Start Routing at New Lower Spillway Crest EL 841.
Top of Dam EL 846
Total Project Discharge Capacity at Top of Dam = 1370 cfs±
due to two spillways. Outlet works closed.
Some Values Rounded from Computed Results.

		TEST FLOOD ONE-HALF PMF [a]
INFLOW		
24-hour Rainfall (inches)		11.1 [b]
24-hour Rainfall Excess (inches) [c]		8.5 [d]
Peak Flow	(cfs)	1910
	(csm)	1032
OUTFLOW		
Peak Flow	(cfs)	1910
	(csm)	1032
Time to Peak Outflow (hours)		17.5
Maximum Storage (acre-feet)		51
Max. W.S. Elevation (feet-NGVD)		846.6
Minimum Freeboard (feet)		overtopped
Maximum Depth over Dam (feet)		0.6
Duration of Overtopping (hours)		2.75

(a) One-half of full PMF total runoff, including base flow. For one-half PMF base flow = 2 cfs per square mile = 4 cfs ±

(b) Approximation assuming total losses are the same as for the full PMF. Full PMF 24-hour rainfall equals 19.5 inches.

(c) Rainfall Excess = Rainfall for the Reservoir Surface. For the rest of the drainage area, losses are assumed to be 1.0 inch initially and 0.1 inch per hour thereafter.

(d) Equal to one-half of full PMF value. Full PMF 24-hour rainfall excess for the land surface equals 16.9 inches.

not of precipitation. The HEC-1 DB program applies the ratio to total runoff, including base flow. This method of applying the ratio introduces an increasing error in base flow as the ratio of the PMF gets smaller. However, this error was eliminated by inputting twice the desired base flow to the full PMF, so that one-half PMF, the test flood, would have the correct base flow.

All precipitation was distributed by the program using the built-in Standard Project Storm arrangement of EM-1110-2-1411 (Reference 13), including the percentage distribution for the maximum 6-hour precipitation, and by both the built-in arrangement and percentage distribution from HYDRO-35 (Reference 6) for the maximum 1-hour precipitation.

2722 Appendix D-7 summarizes the subarea, loss rate, and unit hydrograph data input to the program. Only two subareas were used. Subarea 1 consists of all the drainage area around the reservoir, and Subarea 2 consists of just the reservoir surface. For the land in Subarea 1, loss rates were assumed to be 1.0 inch initially and a constant 0.1 inch per hour thereafter. Snyder unit hydrograph parameters were assumed for average conditions per Appendix D-7 and input to the program. A conservative standard lag time was used. The program uses the inputted Snyder coefficients to solve by iteration for approximate Clark coefficients, which are then used to calculate the runoff hydrograph.

For the reservoir surface making up Subarea 2, loss rates were set to zero so that rainfall would equal rainfall excess, or runoff. Assuming no delay in the rainfall/runoff response, a constant unit hydrograph for a rainfall duration equal to the HEC-1 DB calculation interval was developed per Appendix D-7 and input to the model.

f. Overtopping Potential

The results of the overtopping analysis using the HEC-1 DB program are summarized in Table 5.1. The overtopping analysis computer input and complete output for the test flood of one-half PMF are included starting on Appendix D-8.

As noted from Table 5.1, the test flood of one-half PMF overtops the dam by a maximum of about 0.6 of a foot with duration of overtopping of about 3 hours. Peak inflow for the test flood is 1910 cfs, or 1032 csm (cfs per square mile). Peak outflow is not reduced by reservoir routing and is the same as peak inflow, or 1910 cfs, or 1032 csm, and occurs about 17.5 hours after the start of the storm. The peak portion of the inflow and outflow hydrograph for the test flood of one-half PMF foot depth from the top of the dam down to the new spillway crest, it would take about 13 minutes for the spillways to drain the 12 acre-feet of storage between the top of the dam and the new spillway crest, or about 3 minutes per foot, all assuming no inflow.

d. Selection of Test Flood

Based on the dam failure analysis presented later in Section 5.5, Youngs Brook Dam is classified as having a significant hazard potential. The increase in flow due to a failure of the dam would result in appreciable economic loss caused by wash out of an unpaved Town road and large culvert, flooding of about 35 acres of farmland on both sides of Town Route 133 to a depth of less than a foot, probable damage to Town Route 133, and flooding to a depth of less than a foot around the foundations of about 8 houses, 2 house trailers, 3 barns, and miscellaneous outbuildings. The high flow velocity of 10 fps around most of the structures and the velocities of 7 to 10 fps over the farmland would probably do significant erosion damage. Loss of life is unlikely. Since the dam is also classified as intermediate in size (see Section 1.2.c), recommended guidelines of the Corps of Engineers (Reference 1) indicate a test flood in the range of one-half PMF (probable maximum flood) to full PMF. Since the dam is at the lower end of its intermediate size range with regard to height (46 feet within the 40 to 100-foot range), and since loss of life is unlikely in the event of a dam failure, even though economic loss is appreciable, the test flood selected for this evaluation was one-half PMF (per Table 5.1, peak inflow = 1910 cfs, peak outflow = 1910 cfs).

The PMF event is that hypothetical flood flow produced by the most critical combination of precipitation, minimum infiltration loss, and concentration of runoff that is considered reasonably possible for a particular drainage area.

e. Development of Test Flood

The index PMP (probable maximum precipitation) input to the HEC-1 DB program was 18.5 inches for a 24-hour duration, all-season storm over a 200 square mile basin, according to HMR 33 (Reference 4). Maximum 6-hour, 12-hour, and 24-hour precipitation for the actual size of the drainage area (same for 10 square miles or less) were input to the model as percentages of the index PMP in accordance with HMR 33. A storm reduction coefficient was then applied internally by the program in order to transpose or center the storm over the actual total drainage area. Thus, the corrected 24-hour PMP for the actual total drainage area became 19.5 inches.

In accordance with accepted practice, floods as ratios of the PMF (e.g., one-half PMF) were taken as ratios of runoff, abutment, and is of the free overfall type with a concrete control weir about 28 feet long at EL 841. The weir crest is about 3 feet wide, and at the time of the field inspection had 20-inch high wooden flashboards that were in disrepair. Since the Operator indicated that he intended to permanently remove the flashboards within the week, all analysis in this report is for the spillway without flashboards. The sides of the control section are vertical on the left end of the weir and sloped on the right end. The weir is followed by a nearly vertical drop of over 20 feet to the bedrock bottom of the discharge channel.

The old, higher spillway next to the left abutment has a concrete control weir about 19.3 feet long at EL 843. The weir crest is about 1 foot wide and has no provision for flash-boards. The sides of the control section are essentially vertical on both ends of the weir. The weir is followed by an inclined drop of several feet into a steeply sloping chute discharge channel with a natural bedrock bottom. The discharge channel is much higher than the discharge channel for the new spillway just to the right, but the two channels share a common concrete training wall.

The discharge capacity for each of the two spillways was computed assuming critical flow over a rectangular sharp-crested weir. Total spillway capacity was taken as the sum of the two spillways. The formulas used and the results of hand computations are shown on Appendix D-5. With water 5 feet over the new spillway crest (i.e., 3 feet over the old spillway crest and level at the top of dam), the two spillways together have a discharge capacity of about 1040+330 = about 1370 cfs.

42 4 Taking the spillway crests at EL 841 and EL 843, and the top of dam at EL 846, total discharge computations are summarized on Appendix D-5 and graphed on Appendix D-6. Total discharge from the dam is the sum of the discharges from the spillways, plus flow over the dam for the overtopping condition. As discussed previously in Section 5.4.a, the outlet works were assumed closed and not contributing to the total discharge capacity. The sum of the hand-computed discharges for the two spillways was input directly to the HEC-1 DB program. Flow over the dam was computed by the HEC-1DB program assuming critical flow over a rectangular broad-crested weir with a level crest equal to the length of just the dam without the spillways. The top of dam elevation, length, appropriate discharge coefficient, and exponent of head were input into the program. The formula used for the computation as well as the results of hand computation at selected points, are shown on Appendix D-5.

With the reservoir at the top of dam EL 846, 5 feet over the new spillway crest, the total discharge from the dam is about 1370 cfs. This is due to both the new and old spillways. Also, with an average discharge of 685 cfs over the 5-

17 6 5.4 Test Flood Analysis

a. Initial Conditions

The U.S. Army Corps of Engineers Hydrologic Engineering Center's Program HEC-1 DB (Reference 3) was used to develop the test flood hydrology and perform the reservoir routing.

The purpose of this analysis was to evaluate the dam and spillway with respect to their surcharge storage and spillway capacity. Accordingly, it was assumed that the water surface was at the new (lower) spillway crest at the start of the flood routing. Also, the outlet conduit was assumed closed, as it is normally. In addition, the water supply main and drain pipe were assumed to be closed.

A constant base flow of 2 cfs per square mile was chosen to represent average conditions in the drainage area and was input into the program for all subareas.

b. Storage Capacity

Using a bathymetric map of the reservoir (see Appendix B2-1), areas inside contour elevations were measured and the capacity of the reservoir was computed by the method of conic sections. The computations were done both by hand (Appendix D-2) and by the HEC-1 DB computer program, with the results of computer calculations on Appendices D-11 and D-14. A hand tabulation of the input and the computed results is on Appendix D-2.

Total computed storage at the various elevations agrees almost exactly with reported values (see comparison on Appendix D-2 using reported values from Appendix B3-5).

Using the measured and computed values, stage-area and stage-storage curves are presented on Appendices D-3 and D-4, respectively. At the new spillway crest, EL 841, the reservoir has a surface area of 2.2 acres and a total capacity of 38 acre-feet. At the top of dam, EL 846, the surface area increases to 2.5 acres and the capacity to 50 acre-feet, or about 16.3 million gallons. Surcharge storage between the new spillway crest and the top of dam amounts to 12 acre-feet, or about 0.1 inches of runoff from the 1.85 square-mile drainage area. Therefore, the reservoir has very little capacity to attenuate peak inflow.

6413 c. Discharge Capacity

The dam has two adjacent spillways near the left abutment with weir crests 2 feet different in elevation (see Overview photo, photos in Appendic C, and field measurements on Appendix B2-2). The new, lower spillway is farthest from the

SECTION 5

EVALUATION OF HYDRAULICS AND HYDROLOGY

5.1 General

Youngs Brook Dam is shown on the Location and Vicinity Maps at the beginning of this report and on the Drainage Area Map, Appendix D-1. The dam and reservoir are located on Youngs Brook in central Vermont. About 6500 feet downstream of the dam Youngs Brook joins the Clarendon River. The Clarendon River then runs northeasterly about 2 river miles to the Otter Creek. The Otter Creek runs northward and flows into Lake Champlain, which in turn is drained to the north by the Richelieu River.

The total drainage area at the dam is about 1.85 square miles, of which about 0.004 square miles (2.2 acres), or less than 1%, is actual reservoir surface at the new (lower) spillway crest. Being in the foothills of the Green Mountains, the topography is characterized by steep slopes averaging 15% to 25%. Elevations in the drainage area vary from EL 841 to EL 2726.

5.2 Design Data

There are no known records of the hydraulic and hydrologic criteria used in the original design of the dam and reservoir. Other engineering data available, consisting of one old drawing of the reservoir, reservoir capacity data, old photographs, and an inspection report, are discussed in Section 2 of this report.

5.3 Experience Data

As noted in Section 2.3 of this report, there are no known records of routine water levels and discharges, or of past floods at the dam. It is known that a flood in 1947 damaged the spillway sufficiently to require its replacement, but no written records of that flood exist.

According to NOAA Climatological Data for New England (References 20 and 21), the nearest climatological station is No. 6995, Rutland, located in Rutland at Latitude 43 degrees - 36 minutes North, Longitude 72 degrees - 58 minutes West. The station is non-recording and temperature and precipitation observations are made. Years of record start in about 1916. The station is identified on the Vicinity Map at the beginning of this report and is located about 5 miles northeast of Youngs Brook Dam.

97ᴗ3 b. Operating Facilities

(Covered under preceding Section 4.2.a - General.)

4.3 Evaluation

Effective operation and maintenance procedures for this dam do not exist. Such operation and maintenance procedures need to be developed and implemented by the Owner in order to avoid further deterioration of the dam.

As part of the operation procedure, a reservoir regulation plan may need to be developed to maintain normal water level below the spillway crest. This is necessary due to the dam's questionable physical condition and structural stability (see Sections 3, 6, and 7).

An emergency action plan and warning system needs to be developed by the Owner to ensure proper and timely action during critical periods.

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SECTION 7

ASSESSMENT, RECOMMENDATIONS, AND REMEDIAL MEASURES

7.1 Dam Assessment

a. Condition

Youngs Brook Dam is in POOR condition. Significant problems include questionable stability of both the steep upstream and downstream slopes; heavy tree growth on the downstream slope; water pipe under pressure passing through the embankment; severe distortion and deterioration of the concrete wall on the right side of the upstream face; and cracking, spalling, and deterioration to varying degrees of the concrete training walls of both the old and new spillway discharge channels. Also, seepage was observed exiting from one point at the downstream toe and from the bedrock abutment beneath the left training wall just downstream from the new spillway.

The spillway is INADEQUATE to pass the test flood without overtopping the dam. In accordance with recommended guidelines of the Corps of Engineers, the dam is classified as INTERMEDIATE in size and as having a SIGNIFICANT hazard potential. Accordingly, a TEST FLOOD equal to ONE-HALF PMF (probable maximum flood) was judged as appropriate within the recommended range of one-half PMF to full PMF. The test flood overtops the dam by a maximum of about 0.6 of a foot with duration of overtopping of about 3 hours. Peak inflow for the test flood is 1910 cfs. Peak outflow is not reduced by reservoir routing and is the same as peak inflow, or 1910 cfs. Total project discharge capacity at the top of the dam is due only to the two spillways (outlet works assumed closed) and is equal to 1370 cfs, or 72% of the test flood peak outflow.

b. Adequacy of Information

This Phase I Inspection was based primarily on the visual inspection and the hydraulic and hydrologic computations performed, coupled with sound engineering judgement. Available data was limited, and consisted of a drawing of the reservoir as it existed in 1931, capacity data for the reservoir with a rough sketch of the water supply manhole and piping, several old photos of the reservoir while drained, and one inspection report. Such data as the design calculations, construction specifications, data on the foundation and embankment soils, and detailed operation and performance data were not available. The lack of such
1732 in-depth engineering data does not permit a comprehensive review. Therefore, the adequacy of this dam could not be assessed with respect to reviewing design, construction, and operation data.

c. Urgency

IMMEDIATELY after their receipt of this Phase I Inspection Report, the Owner should implement the recommendation given in Section 7.2.a. The remainder of the recommendations, given in Section 7.2.b, and the remedial measures given in Section 7.3 should be implemented by the Owner WITHIN ONE YEAR after their receipt of this report.

7.2 Recommendations

a. IMMEDIATELY after their receipt of this Phase I Inspection Report, the Owner should lower the reservoir to about EL 828 (i.e., to the invert of the outlet conduit about 13 feet below the new spillway crest), remove water pressure from the pipes that pass through the dam, and maintain the reservoir at that level until the dam is repaired or permanently breached.

b. WITHIN ONE YEAR after their receipt of this Phase I Inspection Report, the Owner should engage a registered engineer qualified in the design of dams to do the following work and provide the consequent recommendations. The Owner should implement those recommendations.

1) Determine the stability of the slopes under critical conditions. For this purpose, borings should be made to recover samples for strength testing, and piezometers should be installed to measure the water levels within the dam.

2) Make recommendations on the alterations needed to improve the stability of the dam.

3) After the stability has been evaluated and the future configuration of the downstream slope determined, make recommendations on (1) removing the trees and roots from the downstream slope to a distance of 20 feet downstream from the toe, (2) type of materials and placement methods for filling root holes after the trees are removed, and (3) appropriate slope protection measures to reduce surface erosion.

4) Redesign the upstream concrete face in the zone where it is tilted and cracked.

5) Drain the reservoir with the outlet conduit and thoroughly inspect the outlet conduit and slide gate.

6) Drain the reservoir further with the mud pipe and inspect the water supply manhole and intake piping.

7) Perform a detailed hydraulic and hydrologic study to better evaluate the adequacy of spillway capacity. If necessary, spillway capacity should be increased by new design and construction.

8) Contingent on the results of the detailed hydraulic and hydrologic study, make recommendations on how to best repair the cracks in the concrete spillway weirs and the significant surface cracking on the right concrete training wall of the new spillway discharge channel. Advise how to repair or replace the left concrete training wall of the old spillway discharge channel. Also, determine if the common concrete training wall between the two spillways, which is in the worst condition, can be removed or if it must be repaired or replaced in some manner to maintain stability of the weir sections. Advise how to remove the wall or design repairs or replacement as appropriate.

7.3 Remedial Measures

a. Operation and Maintenance Procedures

WITHIN ONE YEAR after their receipt of this Phase I Inspection Report, the Owner should implement the following operation and maintenance procedures:

1) At least quarterly monitor the seepage from the toe of the dam and from the bedrock abutment beneath the left training wall just downstream from the new spillway. Also, the small seep at the concrete weir cap/bedrock contact at about the center of the old spillway should be watched.

2) Remove the remains of the wooden forms from the top inside of the outlet conduit.

3) Stop leakage of the outlet slide gate and grease the slide gate and handwheel to permit free operation.

4) Paint the handwheel, floor stand, and floor stand anchorage to prevent further rusting.

5) While the reservoir is drained with the outlet conduit, check and operate the control valves next to the water supply manhole.

1733 6) Provide a means to operate the water main control valves without first having to lower the reservoir with the outlet conduit.

7) Locate the discharge end of the mud pipe downstream of the dam and keep the end clear.

8) When the reservoir is drained, check it for sediment and clean it as necessary.

9) Repair the spalled concrete on the overhang just to the left of the outlet slide gate control mechanism. Also, watch for worsening hairline cracking and efflorescence in the area of the concrete wall around the control mechanism and repair it promptly.

10) Clear logs and debris from in front of and on the new spillway weir.

11) Carry out plans to permanently remove the flashboards from the new spillway weir.

12) Clear brush and trees from both spillway discharge channels. Also, clear the logs from the bottom of the new spillway discharge channel.

13) Develop and implement effective operation and maintenance procedures to avoid further deterioration of the dam. Included may need to be a reservoir regulation plan to maintain normal water level below the spillway crest until questions about slope stability have been resolved.

14) Develop an "Emergency Action Plan" that will include an effective downstream warning system; locations of emergency equipment, materials, and manpower; authorities to contact; and potential areas that require evacuation.

7.4 Alternatives

No practical alternatives exist to the recommendations and remedial measures contained in this report.

APPENDIX A

INSPECTION CHECKLIST

VISUAL INSPECTION CHECKLIST
DAM INSPECTION

DAM	YOUNGS BROOK DAM	DATE	November 8, 1979
ID NO.	VT 00165	TIME	1045 - 1600
TOWN	West Rutland	WEATHER	Overcast
COUNTY	Rutland	W.S. ELEV.	841.3± UPSTREAM
STATE	Vermont		800 ± DOWNSTREAM

INSPECTION PARTY		RECORDER (X)
1.	Thomas Bennedum, Gordon E. Ainsworth & Assoc., Inc.	X
2.	Edwin Vopelak, Jr., Gordon E. Ainsworth & Assoc., Inc.	
3.	John Kenworthy, Gordon E. Ainsworth & Assoc., Inc.	
4.	Steve J. Poulos, Geotechnical Engineers, Inc.	X
5.	Joseph F. Skaza, Operator, West Rutland	
6.		
7.		
8.		
9.		
10.		

	PROJECT FEATURE/DISCIPLINE	INSPECTOR	REMARKS
1.	H & H	T. Bennedum	--
2.	Geotechnical	S. Poulos	--
3.	Structural	T. Bennedum	--
4.	Mechanical	T. Bennedum	--
5.	Electrical	None	N/A
6.			

VISUAL INSPECTION CHECKLIST

PROJECT YOUNGS BROOK DAM DATE Nov. 8, 1979

PROJECT FEATURE ——— NAME ———

DISCIPLINE Geotechnical NAME S. J. Poulos

	AREA EVALUATED	CONDITION
	DAM EMBANKMENT	
	Crest Elevation	EL 846
	Current Pool Elevation	EL 841.3
	Maximum Impoundment to Date	Unknown.
GEI	Surface Cracks	No cracks evident. Surface is cow pasture.
GEI	Pavement Condition	No pavement.
GEI	Movement or Settlement of Crest	Depression behind wall at Sta 80. Sinkhole 6"-8" deep Sta 57, 6' right of right abutment of spillway due to movement of fines upstream through concrete retaining wall into reservoir. Another larger low spot opposite crack in wall (Sta 1+00, 48 ft rt.). Soil to rt. of right abutment of spillway seems to have eroded down about 6" from former level because surface is depressed within 3-6' from wall. Section of upstream wall from Sta 1+70 to Sta 2+00 is tipped upstream and a scarp has formed 3' downstream from wall causing a 9" low spot. Wall is cracked and slowly failing.
GEI	Lateral Movement	No lateral movement of embankment observable except as noted in "Movement or Settlement of Crest."
GEI	Vertical Alignment	Undulating ± 9".
GEI	Horizontal Alignment	Not observable.
GEI	Condition at Abutment and at Concrete Structures	See sinkholes at "Movement or Settlement of Crest" above.
GEI	Indications of Movement of Structural Items on Slopes	None except as in "Movement or Settlement of Crest."
GEI	Trespassing on Slopes	Free access.
GEI	Sloughing or Erosion of Slopes or Abutments	Top of downstream slope shows apparent scarp just below crest at middle of dam. Material is glacial till probably taken from nearby field. About 40% of downstream slope is eroded bare. Two definite channels. Continual sheet erosion.
GEI	Rock Slope Protection-Riprap Failures	Upstream concrete wall discussed in "Movement or Settlement of Crest."

2A

VISUAL INSPECTION CHECKLIST

PROJECT YOUNGS BROOK DAM — DATE Nov. 8, 1979

PROJECT FEATURE ——— — NAME ———

DISCIPLINE Geotechnical — NAME S. J. Poulos

	AREA EVALUATED	CONDITION
	DAM EMBANKMENT	
GEI	Unusual Movement or Cracking at or Near Toe	None observed.
GEI	Unusual Embankment or Downstream Seepage	Clear seep at 2-4 gpm at downstream toe about Sta 0+90. Located to left of an 8-ft-high pile of dirt at toe.
GEI	Piping or Boils	None observed.
GEI	Foundation Drainage Features	None.
GEI	Toe Drains	None.
GEI	Instrumentation System	None.
GEI	Vegetation	Birch to 12" white pine to 9". Raspberry, blackberry bushes. Growth different from surrounding woods but practically forested.

VISUAL INSPECTION CHECKLIST

DAM: YOUNGS BROOK DAM DATE: Nov. 8, 1979

DISCIPLINE: Structural/H & H INSPECTOR: T. Bennedum

DISCIPLINE: Geotechnical INSPECTOR: S.J. Poulos

AREA EVALUATED	CONDITION
OUTLET WORKS - INTAKE CHANNEL AND INTAKE STRUCTURE	
a. Approach Channel	
GEI Slope Conditions	
G I Bottom Conditions	Same as spillway approach channel.
GEI Rock Slides or Falls	
Log Boom	
Debris	
Condition of Concrete Lining	
GEI Drains or Weep Holes	N/A
b. Intake Structure	
Condition of Concrete	Both water supply MH & intake of outlet conduit underwater and not observable.
Stop Logs and Slots	

VISUAL INSPECTION CHECKLIST

DAM YOUNGS BROOK DAM DATE Nov. 8, 1979

DISCIPLINE Structural/Mechanical INSPECTOR T. Bennedum

DISCIPLINE No Geotechnical Features INSPECTOR --

AREA EVALUATED	CONDITION
OUTLET WORKS - CONTROL TOWER	Floor stand w/handwheel on U/S concrete face of dam to right of new spillway.
a. Concrete and Structural	In area of floor stand.
General Condition	Fair.
Condition of Joints	N/A
Spalling	Bottom corner of overhang left of flood stand.
Visible Reinforcing	None.
Rusting or Staining of Concrete	Some rust stains from floor stand.
Any Seepage or Efflorescence	Efflorescence at H/L cracks.
Joint Alignment	N/A
Unusual Seepage or Leaks in Gate Chamber	N/A
Cracks	H/L cracks on face of wall.
Rusting or Corrosion of Steel	None.
b. Mechanical and Electrical	
Air Vents	N/A
Float Wells	N/A
Crane Hoist	N/A
Elevator	N/A
Hydraulic System	N/A
Service Gates	Slide gate underwater. Operable, needs grease, some surface rust. Leakage observed when closed.
Emergency Gates - None	
Lightning Protection System	None.
Emergency Power System	None.
Wiring and Lighting System	None.
	Gate was opened 9" (rising stem) and reservoir was lowered from new spillway crest down 0.6' in about 1.25 hours.

VISUAL INSPECTION CHECKLIST

DAM: YOUNGS BROOK DAM | DATE: Nov. 8, 1979

DISCIPLINE: Structural/H & H | INSPECTOR: T. Bennedum

DISCIPLINE: No Geotechnical Features | INSPECTOR: --

AREA EVALUATED	CONDITION
<u>OUTLET WORKS - TRANSITION AND CONDUIT</u>	Conduit is very short 24" square section from slide gate on U/S face of dam discharging thru right T.W. of new spillway discharge channel.
General Condition of Concrete	Not readily observable, but condition appears good. Remains of wood forms along top.
Rust or Staining on Concrete	None observed.
Spalling	None observed.
Erosion or Cavitation	None observed.
Cracking	None observed.
Alignment of Monoliths	Appears OK.
Alignment of Joints	Appears OK.
Numbering of Monoliths	N/A

7

VISUAL INSPECTION CHECKLIST

DAM YOUNGS BROOK DAM DATE Nov. 8, 1979

DISCIPLINE Structural/H & H INSPECTOR T. Bennedum

DISCIPLINE Geotechnical INSPECTOR S.J. Poulos

AREA EVALUATED	CONDITION
OUTLET WORKS - OUTLET STRUCTURE AND OUTLET CHANNEL	
General Condition of Concrete	Same as right training wall of new spillway discharge channel.
Rust or Staining	
Spalling	
Erosion or Cavitation	
Visible Reinforcing	
Any Seepage or Efflorescence	
Condition at Joints	
GEI Drain holes	None.
GEI Channel	
GEI Loose Rock or Trees Overhanging Channel	Same as new spillway discharge channel.
GEI Condition of Discharge Channel	

VISUAL INSPECTION CHECKLIST

DAM: YOUNGS BROOK DAM — DATE: Nov. 8, 1979

DISCIPLINE: Structural/H & H — INSPECTOR: T. Bennedum

DISCIPLINE: Geotechnical — INSPECTOR: S.J. Poulos

8

AREA EVALUATED	CONDITION
OUTLET WORKS - SPILLWAY WEIR, APPROACH AND DISCHARGE CHANNELS	NEW SPILLWAY
a. Approach Channel	(Same for new & old spillway.)
General Condition	OK.
Loose Rock Overhanging Channel	None.
Trees Overhanging Channel	Side of reservoir forested with pine trees. Logs on new spillway.
Floor of Approach Channel	Reservoir bottom. Leaves over silt.
b. Weir and Training Walls	
General Condition of Concrete	Fair. 4 cracks in top of weir, some extending down D/S face. Horiz. crack on U/S side of weir 3' below crest. H/L cracks in right T.W.
Rust or Staining	Rust staining on right T.W. at H/L cracks and joints.
Spalling	Along left T.W.
Any Visible Reinforcing	Top of left T.W.
Any Seepage or Efflorescence	Efflorescence on right T.W. at H/L cracks and joints.
Drain Holes	None.
c. Discharge Channel	
General Condition	Satisfactory.
Loose Rock Overhanging Channel	Some loose rock but not significant.
Trees Overhanging Channel	Trees up to 8" dia. on both sides of 15' wide channel.
Floor of Channel	Natural bedrock.
Other Obstructions	Logs. Not significant.

VISUAL INSPECTION CHECKLIST

DAM YOUNGS BROOK DAM DATE Nov. 8, 1979

DISCIPLINE Structural/H & H INSPECTOR T. Bennedum 8

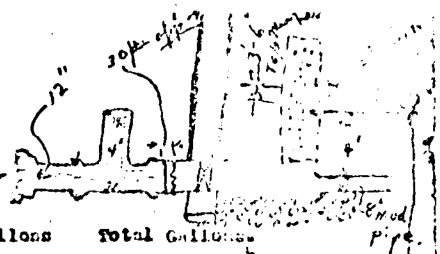
DISCIPLINE Geotechnical INSPECTOR --

	AREA EVALUATED	CONDITION
	OUTLET WORKS - SPILLWAY WEIR, APPROACH AND DISCHARGE CHANNELS	
	a. Approach Channel	Same as for new spillway.
GEI	General Condition	
G[	Loose Rock Overhanging Channel	
G[	Trees Overhanging Channel	
GFI	Floor of Approach Channel	OLD SPILLWAY
	b. Weir and Training Walls	
	General Condition of Concrete	Poor. Weir has about 1/16" vert. crack at C.L. U/S end of left T.W. crumbling. Right T.W. cracked. Top section newer.
	Rust or Staining	Rust from rails across top.
	Spalling	On left T.W.
	Any Visible Reinforcing	Top of right T.W.
	Any Seepage or Efflorescence	Seep at D/S C.L. of weir cap where it meets rock. Some efflorescence on T.W.'s.
GEI	Drain Holes	None.
	c. Discharge Channel	
GI	General Condition	Fair
GFI	Loose Rock Overhanging Channel	Some loose rock but not significant.
GEI	Trees Overhanging Channel	Pine and birch trees on left side.
GI	Floor of Channel	Natural bedrock w/some loose concrete.
GI	Other Obstructions	Some trees and brush in channel.

CAPACITY

of

WEST RUTLAND RESERVOIR.



Intake to supply Manhole. End pipe.

Contour	Area Sq.ft.	Diff.	Cubic	Gallons	Total Gallons
660	0				
661	60	60	30	225	225
662	290	230	165	1,239	1,464
663	1,095	805	692	5,189	6,653
664	9,230	8,135	5,163	38,723	45,376
665	10,855	1,625	10,042	75,315	120,691
666	13,340	2,495	12,103	90,772	211,463
667	17,540	4,200	15,440	115,800	327,263
668	20,340	2,800	18,940	142,050	469,313
669	21,780	1,440	21,060	157,950	627,263
670	23,775	1,995	22,778	170,735	797,998
671	25,799	2,024	24,787	185,903	983,901
672	27,558	1,759	26,678	200,085	1,183,986
673	29,342	1,784	28,450	213,375	1,397,361
674	30,776	1,434	30,059	225,442	1,622,803
675	32,910	2,134	31,843	238,823	1,861,626
676	34,808	1,898	33,859	253,942	2,115,568
677	37,271	2,463	36,040	270,300	2,385,868
678	38,939	1,668	38,105	285,788	2,671,656
679	41,127	2,188	40,033	300,247	2,971,903
680	44,440	2,313	42,783	320,873	3,292,776
681	46,997	2,557	45,719	342,892	3,635,668
682	48,604	1,634	47,635	358,793	3,994,461
683	50,764	2,082	49,725	372,937	4,367,398
684	53,036	2,272	51,902	389,265	4,756,663
685	56,675	3,637	54,856	411,420	5,168,083
686	59,719	3,044	58,197	436,478	5,604,561
687	62,518	2,799	61,118	458,385	6,062,946
688	65,187	2,669	63,833	478,747	6,541,693
	68,176	2,989	66,681	500,107	7,041,800
		3,054	69,708	522,810	7,564,610
		2,571	72,526	543,945	8,108,555
		2,976	75,299	564,743	8,673,298
	[illegible]	2,106	77,840	583,800	9,257,098
	...029	3,136	80,461	603,458	9,860,556
	85,500	3,471	83,764	628,230	10,488,786
...6	89,019	3,519	87,260	654,450	11,143,236
697	92,448	3,429	90,733	680,498	11,823,734

T to 8" To Small Reservoir

Below the 12" main to city but above 8" mud pipe outlet (contours 661–664)

11/27/74
aw

INSPECTION REPORT
ON
Youngs Brook Dam

1. Date of inspection 6/52 2. Water conditions drawn down

GENERAL DATA:

3. Location of dam Youngs Br.; West Rutland
4. Owner and operator West Rutland Fire District
5. Characteristic features of dam earth dam 50' high concrete spillway on ledge - crest 5' lower
6. Other related data Pond area about 2 acres at crest - serves water supply

OBSERVATIONS:

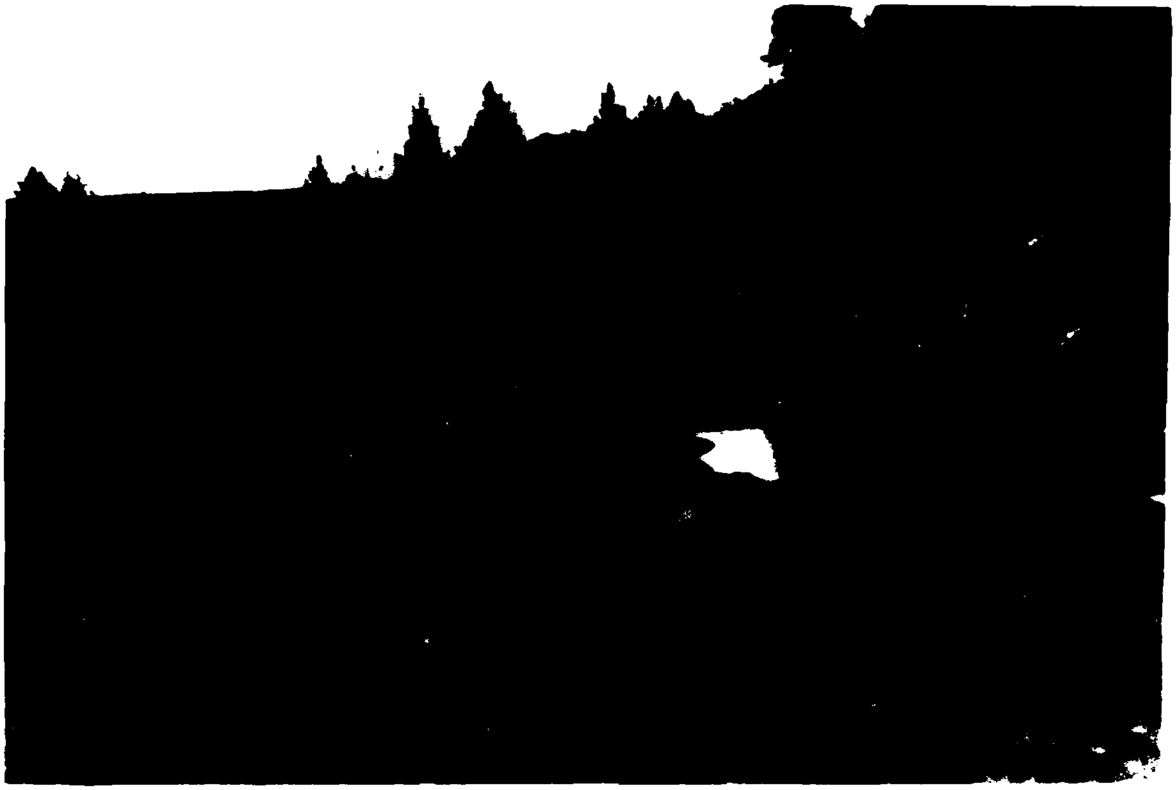
7. Condition of structure embankment slope may be steep. concrete in good condition - some spalling
8. Condition of equipment satisfactory
9. Operation satisfactory
10. Maintenance satisfactory

REMARKS: Dam in isolated Location

Inspected by [illegible]

Youngs Brook Reservoir drained for cleaning - Presumed about 1950

Youngs Brook Reservoir drained for cleaning – Presumed about 1950



Youngs Brook Reservoir drained for cleaning - Spring 1933

APPENDIX B

SECTION B3

COPIES OF PAST INSPECTION REPORTS AND DATA

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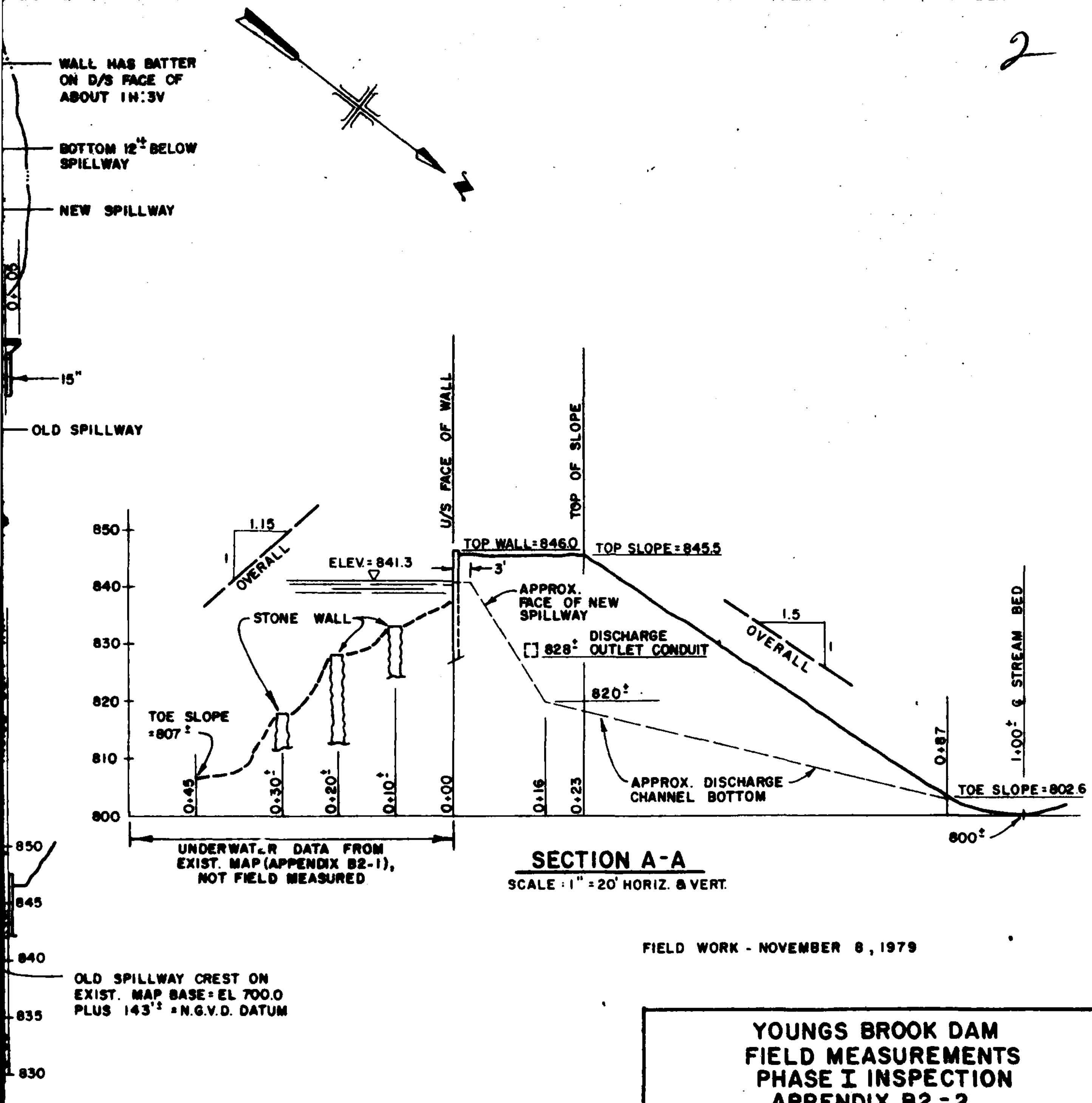
2



SECTION A-A

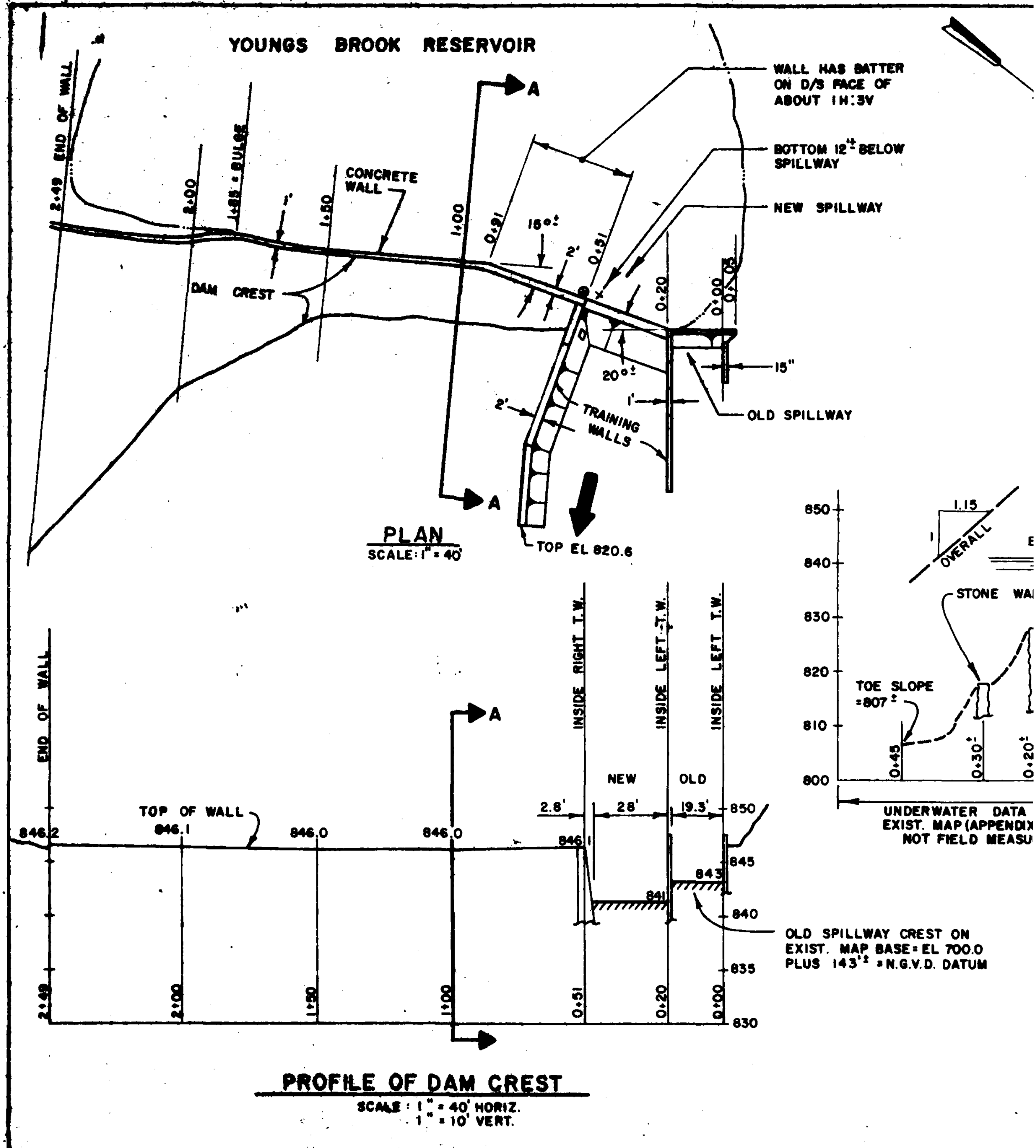
SCALE: 1" = 20' HORIZ. & VERT.

FIELD WORK - NOVEMBER 8, 1979

YOUNGS BROOK DAM
FIELD MEASUREMENTS
PHASE I INSPECTION
APPENDIX B2-2

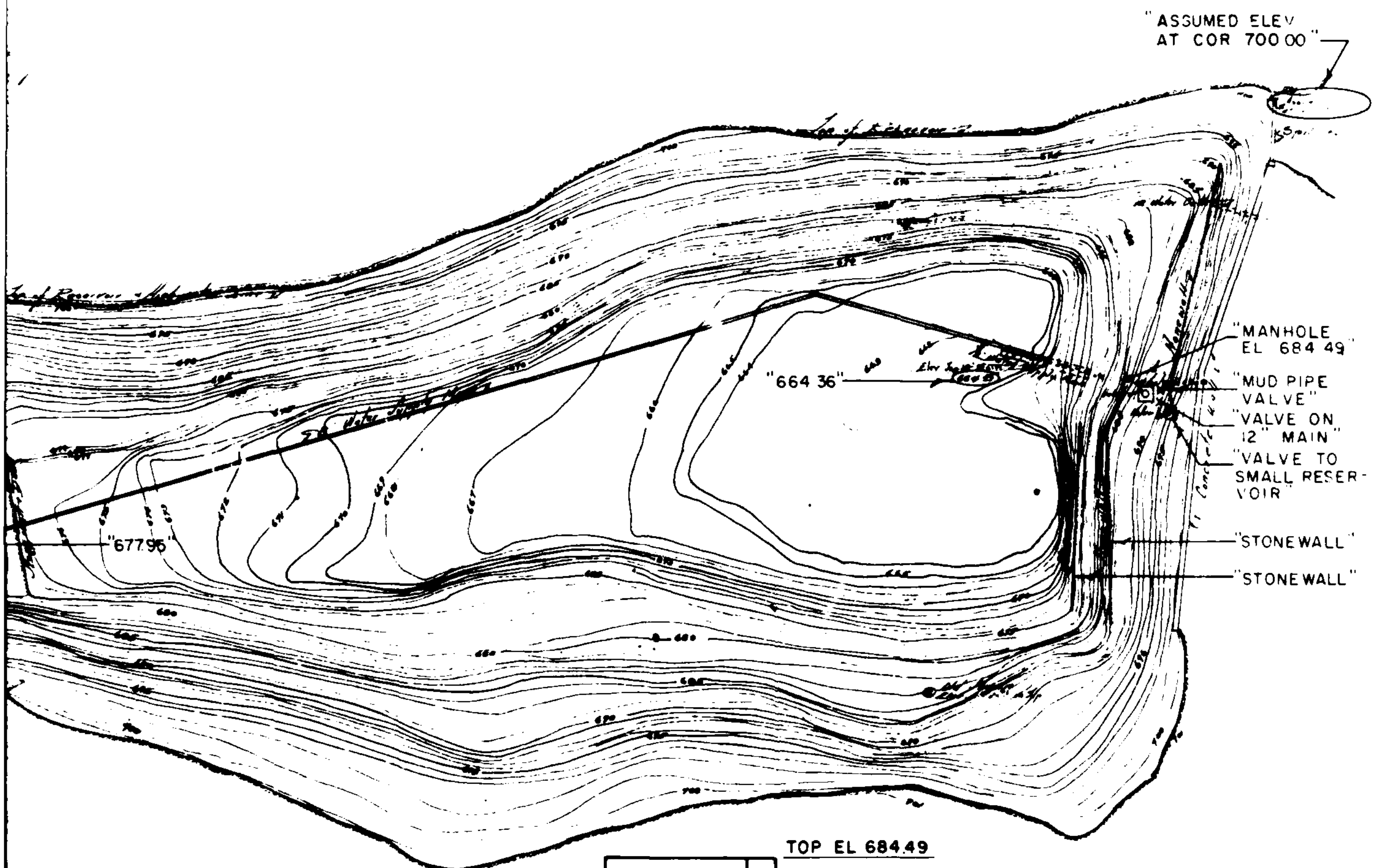
GORDON E. AINSWORTH & ASSOCIATES INC.
Engineers, Surveyors and Planners
20 SUGARLOAF ST. SOUTH DEERFIELD, MASS. 01373

JANUARY 1980 DWG. NO. 80-114



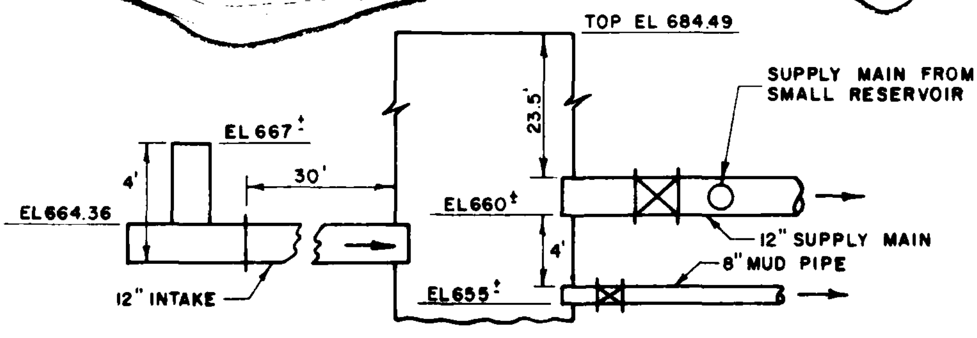
YOUNGS BROOK RESERVOIR
WALL HAS BATTER ON D/S FACE OF ABOUT 1H:3V
BOTTOM 12± BELOW SPILLWAY
NEW SPILLWAY
OLD SPILLWAY
CONCRETE WALL
DAM CREST
END OF WALL
TRAINING WALLS
TOP EL 820.6
PLAN
SCALE: 1" = 40'
TOE SLOPE ±807±
STONE WA
OVERALL
UNDERWATER DATA
EXIST. MAP (APPENDIX
NOT FIELD MEASU
TOP OF WALL
INSIDE RIGHT T.W.
INSIDE LEFT T.W.
NEW
OLD
OLD SPILLWAY CREST ON EXIST. MAP BASE = EL 700.0 PLUS 143'± = N.G.V.D. DATUM
PROFILE OF DAM CREST
SCALE: 1" = 40' HORIZ.
1" = 10' VERT.

2



SERVOIR

MAY 6 1931

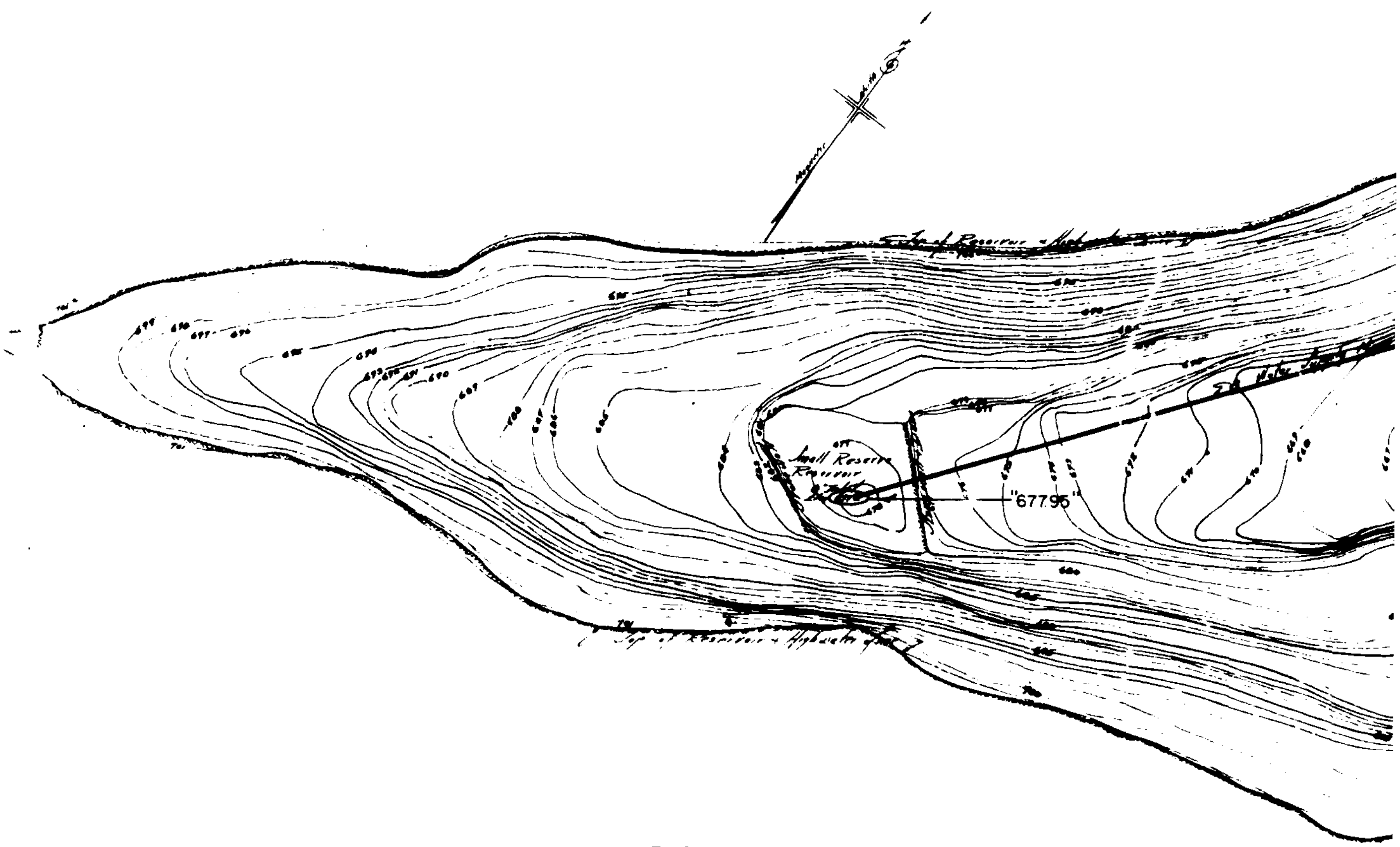


DETAIL OF WATER SUPPLY MANHOLE

NO SCALE

COMPILED FROM THIS DRAWING & FROM OLD SKETCH ON APPENDIX B3-5

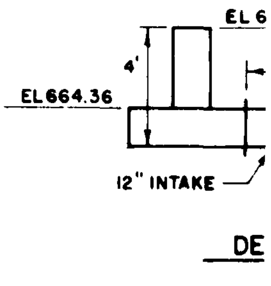
SSOCIATES, INC. 79-397



WEST RUTLAND RESERVOIR

WEST RUTLAND – VT

SCALE 1" = 20 – 0"   MAY 6 1931



REDUCED TO 40% OF ORIGINAL

ASSOCIATES INC 79 -397

APPENDIX B

SECTION B2

DRAWINGS

TABLE OF CONTENTS

APPENDIX B

SECTION B1

LISTING OF LOCATIONS FOR AVAILABLE RECORDS AND DATA

a. Owner: West Rutland Fire District No. 1
West Rutland, Vermont 05777
Attention: William F. Harvey III, Chairman
(802) 438-5771
Joseph F. Skaza, Operator
(802) 438-2907

1) Drawing of reservoir
2) Photographs
3) Reservoir capacity data

b. Original Designer: Unknown.

c. Original Construction Contractor: Unknown.

d. Repair Designer: Unknown.

e. Repair Construction Contractor: Unknown.

f. Agency of Environmental Conservation
Department of Water Resources
Water Quality Division
Montpelier, Vermont 05602
Attention: A. Peter Barranco, Jr., P.E.
Dam Safety Engineer
(802) 828-2761

1) Inspection report.

APPENDIX B

ENGINEERING DATA

Section	Description
B1	Listing of Locations for Available Records and Data
B2	Drawings
B3	Copies of Past Inspection Reports and Data

VISUAL INSPECTION CHECKLIST

DAM: YOUNGS BROOK DAM — DATE: Nov. 8, 1979

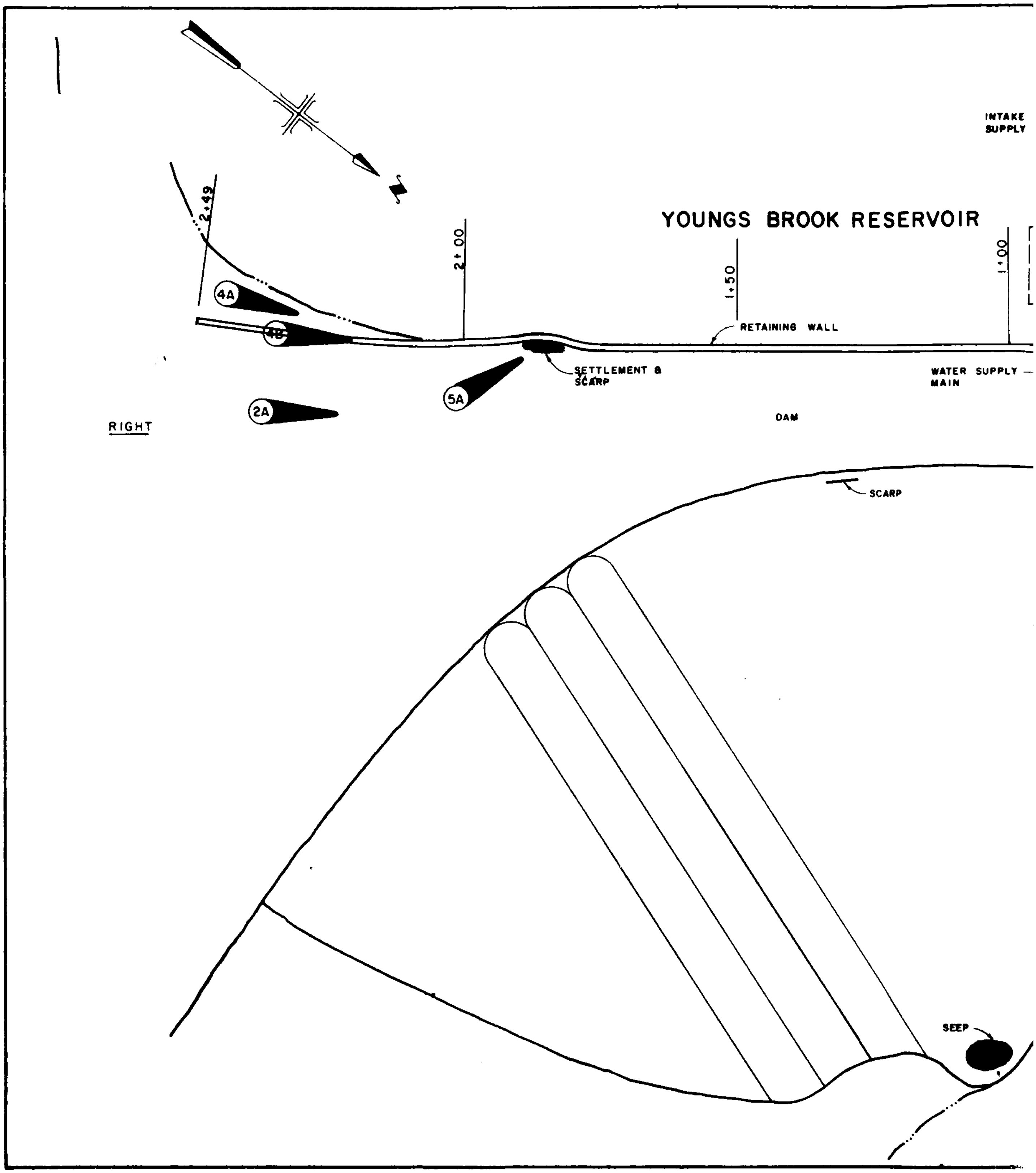
DISCIPLINE: Structural — INSPECTOR: T. Bennedum 9

DISCIPLINE: No Geotechnical Features — INSPECTOR: --

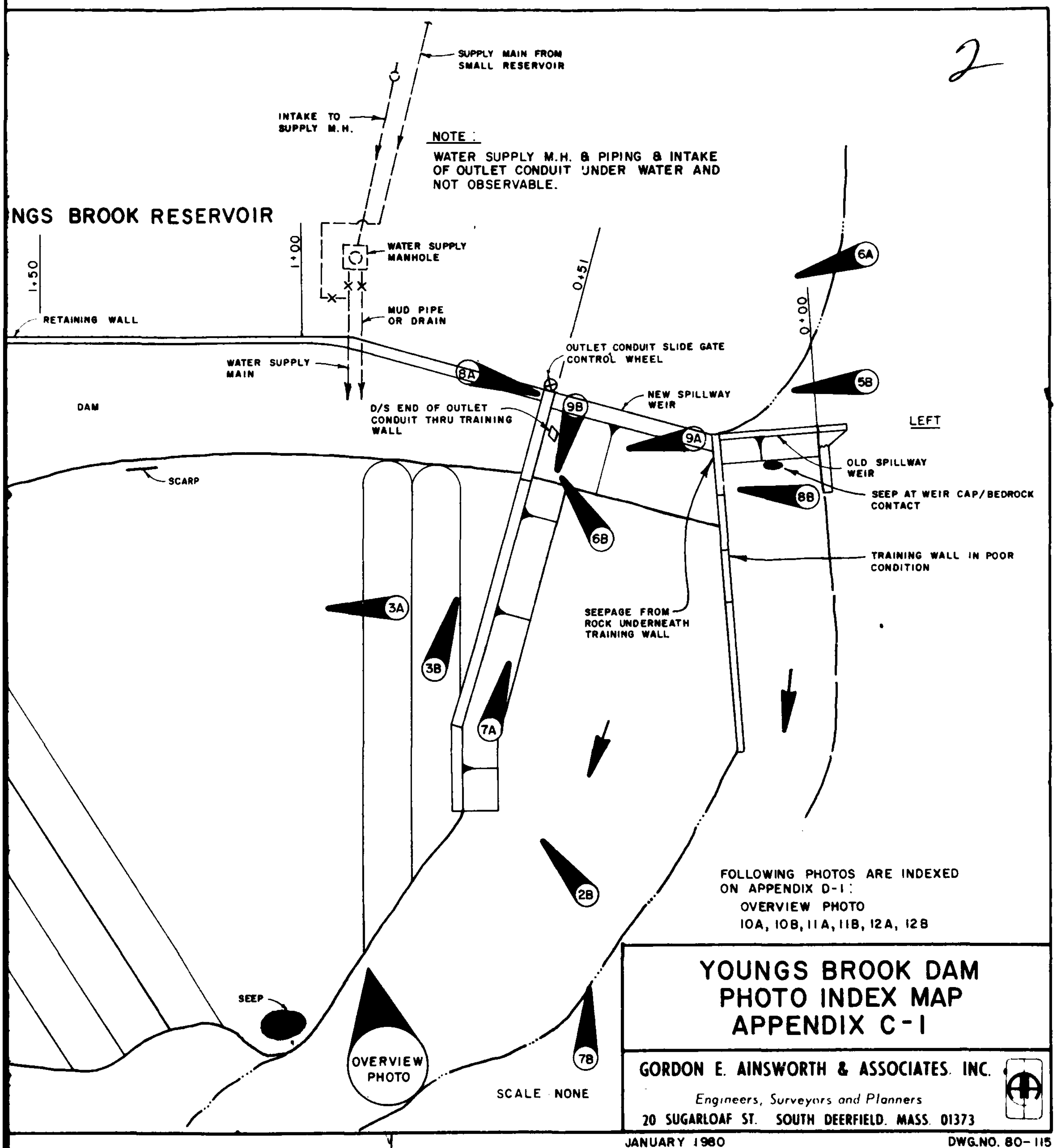
AREA EVALUATED	CONDITION
OUTLET WORKS - SERVICE BRIDGE	N/A - No service bridge.
a. Super Structure	
Bearings	
Anchor Bolts	
Bridge Seat	
Longitudinal Members	
Underside of Deck	
Secondary Bracing	
Deck	
Drainage system	
Railings	
Expansion Joints	
Paint	
b. Abutment & Piers	
General Condition of Concrete	
Alignment of Abutment	
Approach to Bridge	
Condition of Seat & Backwall	

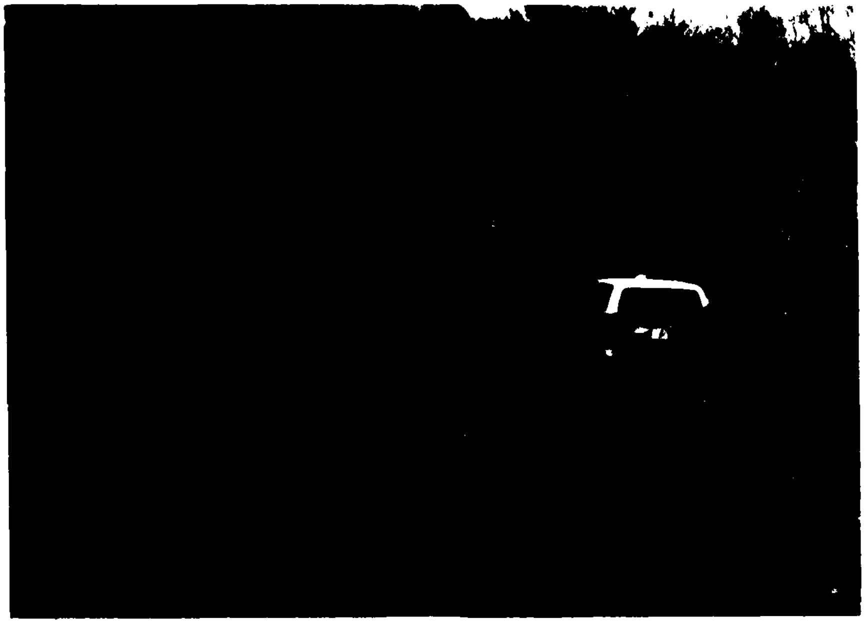
APPENDIX C

PHOTOGRAPHS



INTAKE
SUPPLY
YOUNGS BROOK RESERVOIR
2+49
2+00
1+50
1+00
4A
4B
RETAINING WALL
SETTLEMENT &
SCARP
WATER SUPPLY —
MAIN
5A
2A
RIGHT
DAM
SCARP
SEEP


2
SUPPLY MAIN FROM SMALL RESERVOIR
INTAKE TO SUPPLY M.H.
NOTE :
WATER SUPPLY M.H. & PIPING & INTAKE OF OUTLET CONDUIT UNDER WATER AND NOT OBSERVABLE.
NGS BROOK RESERVOIR
1+50
1+00
0+51
0+00
WATER SUPPLY MANHOLE
MUD PIPE OR DRAIN
RETAINING WALL
WATER SUPPLY MAIN
OUTLET CONDUIT SLIDE GATE CONTROL WHEEL
NEW SPILLWAY WEIR
DAM
D/S END OF OUTLET CONDUIT THRU TRAINING WALL
LEFT
OLD SPILLWAY WEIR
SCARP
SEEP AT WEIR CAP/BEDROCK CONTACT
TRAINING WALL IN POOR CONDITION
SEEPAGE FROM ROCK UNDERNEATH TRAINING WALL
6A
5B
8A
9B
9A
8B
6B
3A
3B
7A
2B
7B
SEEP
OVERVIEW PHOTO
SCALE NONE
FOLLOWING PHOTOS ARE INDEXED ON APPENDIX D-1 :
OVERVIEW PHOTO
10A, 10B, 11A, 11B, 12A, 12B
YOUNGS BROOK DAM
PHOTO INDEX MAP
APPENDIX C-1
GORDON E. AINSWORTH & ASSOCIATES. INC.
Engineers, Surveyors and Planners
20 SUGARLOAF ST. SOUTH DEERFIELD. MASS. 01373
JANUARY 1980
DWG.NO. 80-115



C-2A Dam crest looking toward left abutment - 11/08/79

C-2B Downstream slope of dam looking from new spillway discharge channel toward right abutment - 11/08/79

C-3A Trees on downstream slope of dam looking toward right abutment. Note bowed shape of trees - 11/08/79

C-3B Downstream slope of dam behind right training wall of spillway discharge channel. Note barren nature of slope between trees. 11/08/79

C-4A Concrete wall on upstream face of dam looking from upstream of right abutment. Note deterioration of concrete - 11/08/79

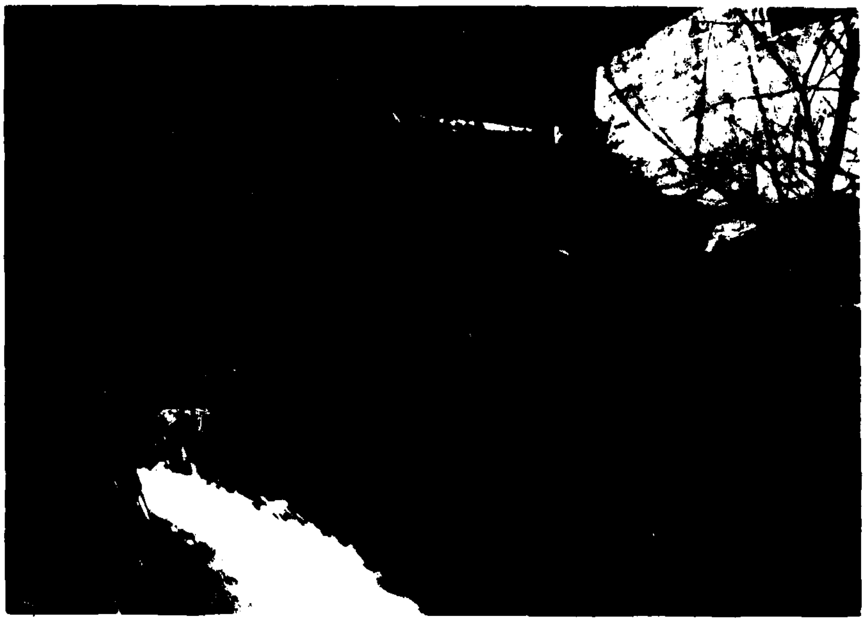
C-4B View along concrete wall on upstream face of dam looking from right abutment. Note bow and tilt of wall toward water - 11/08/79

C-5A Settlement of soil behind the concrete wall near the bow shown in photo C-4B - 11/08/79

C-5B Dam crest looking toward right abutment. Note old spillway weir at lower left, new spillway at center (with flashboards), and outlet control mechanism (floor stand) just to right of new spillway - 11/08/79

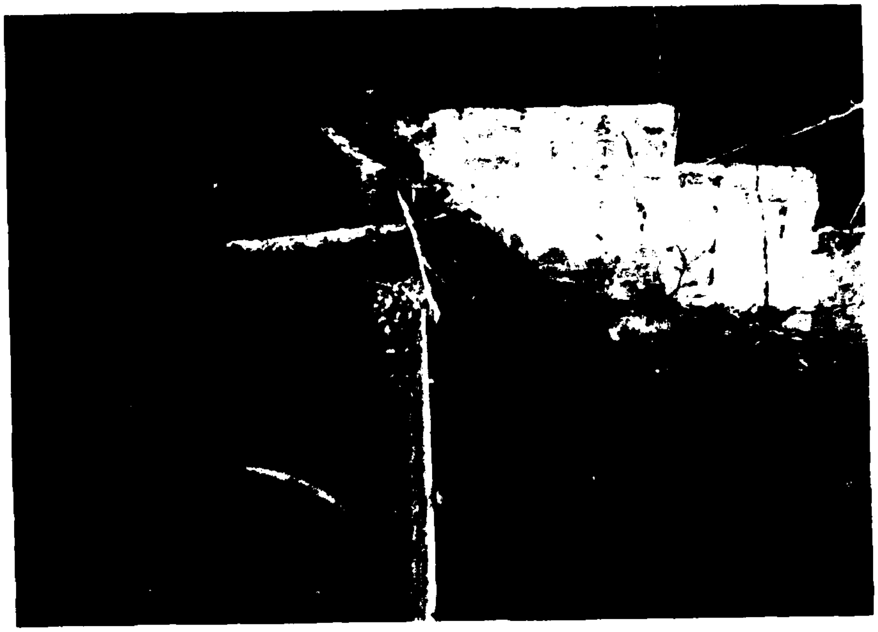
C-6A Outlet slide gate control mechanism (floor stand with handwheel) just to right of new spillway - 11/08/79

C-6B Downstream end of outlet conduit in right training wall of new spillway discharge channel - 11/08/79



C-7A New spillway looking upstream from right side of spillway discharge channel. Note discharge from end of outlet conduit shown in photo C-6B - 11/08/79

C-7B New spillway looking upstream from new spillway discharge channel - 11/08/79

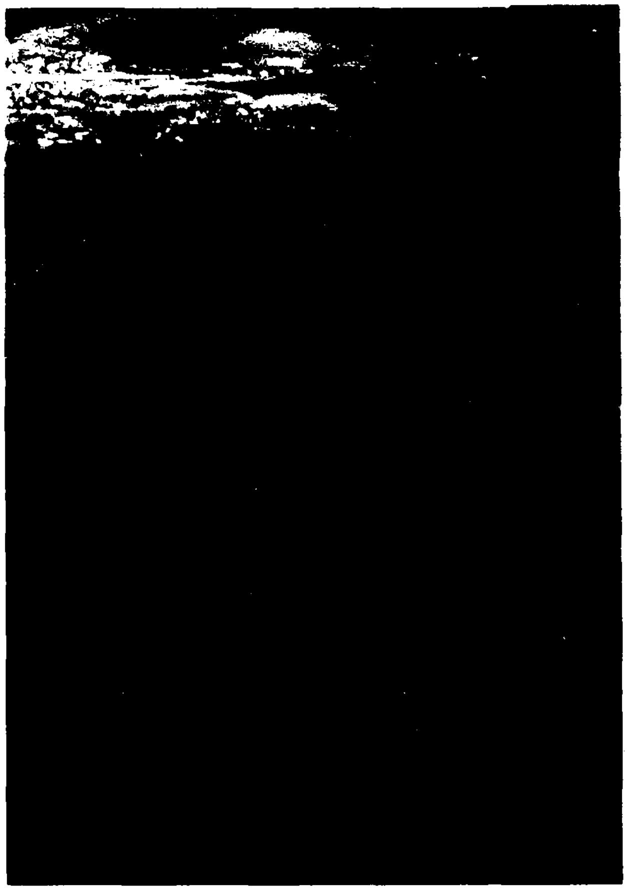


C-8A New spillway weir (in foreground with flashboards) and old spillway weir looking toward left abutment. Left training wall of new spillway separates the two spillways - 11/08/79

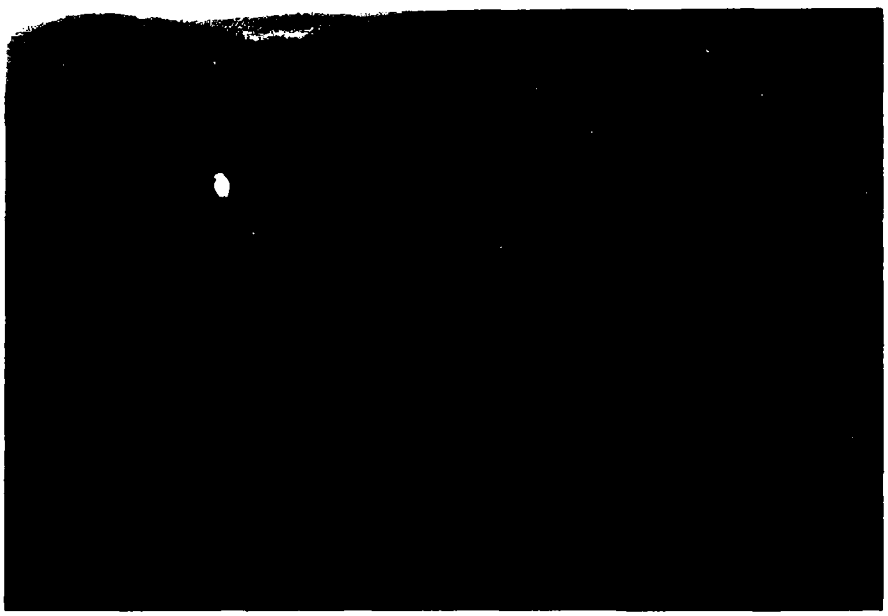
C-8B Right training wall of old spillway (back side of training wall shown in photo C-8A) looking from old spillway discharge channel - 11/08/79

C-9A Right training wall of new spillway looking downstream from spillway crest - 11/08/79

C-9B Discharge channel, looking downstream from new spillway crest 11/08/79



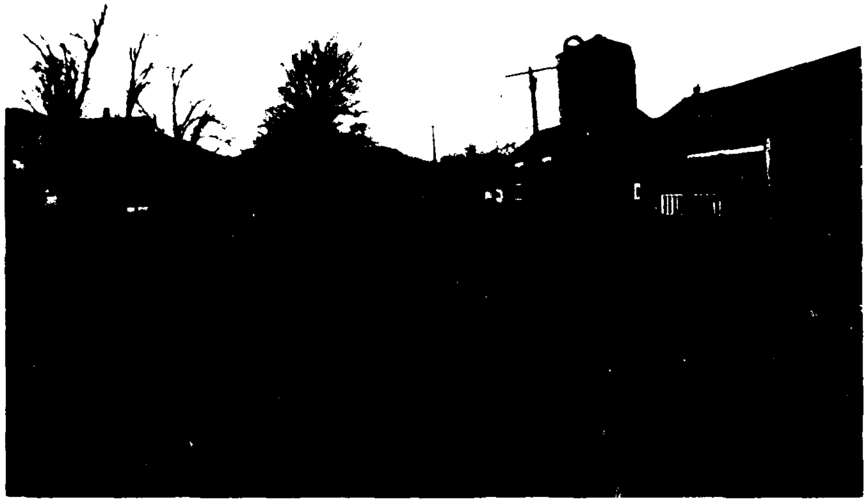
C-10A Aerial overview of dam and reservoir looking downstream - 11/30/79



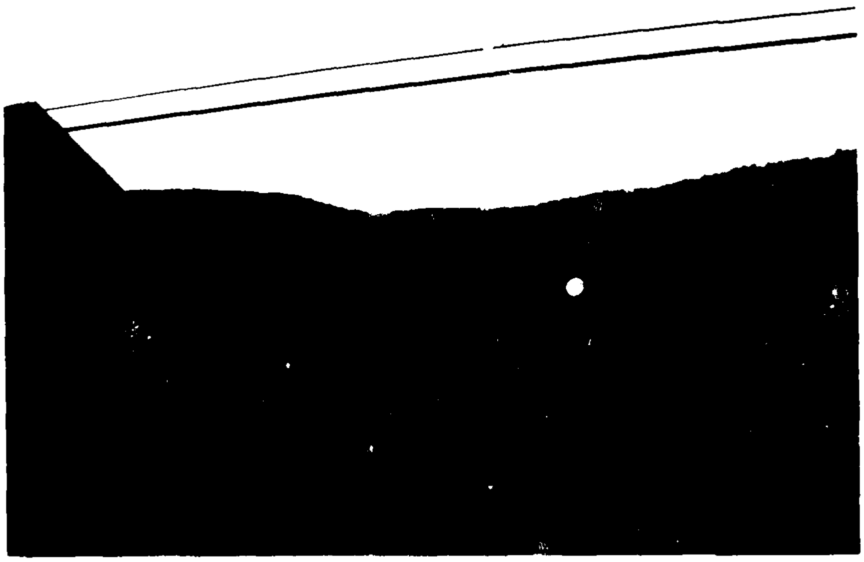
C-10B Aerial overview of downstream hazard area along State Route No. 133, looking upstream toward reservoir - 11/30/79

C-11A Inlet of culvert under road about 1000 feet downstream of dam - 11/08/79

C-11B Building about 3600 feet downstream of dam, looking downstream. Note stream channel toward right center of photo. 11/08/79



C-12A Downstream hazard area along State Route No. 133, looking northwesterly toward stream channel which crosses under the road in background of photo - 11/08/79

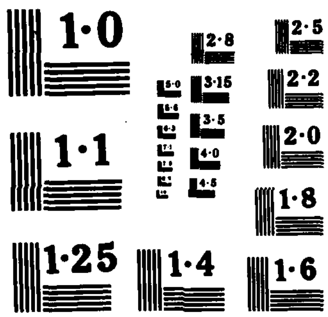


C-12B Downstream hazard area near State Route No. 133, looking northerly toward stream channel in background across field 11/08/79

AD-A156 976 NATIONAL PROGRAM FOR INSPECTION OF NON-FEDERAL DAMS YOUNGS BROOK DAM (VT..(U) CORPS OF ENGINEERS WALTHAM MA NEW ENGLAND DIV JAN 80 2/2

UNCLASSIFIED F/G 13/13 NL

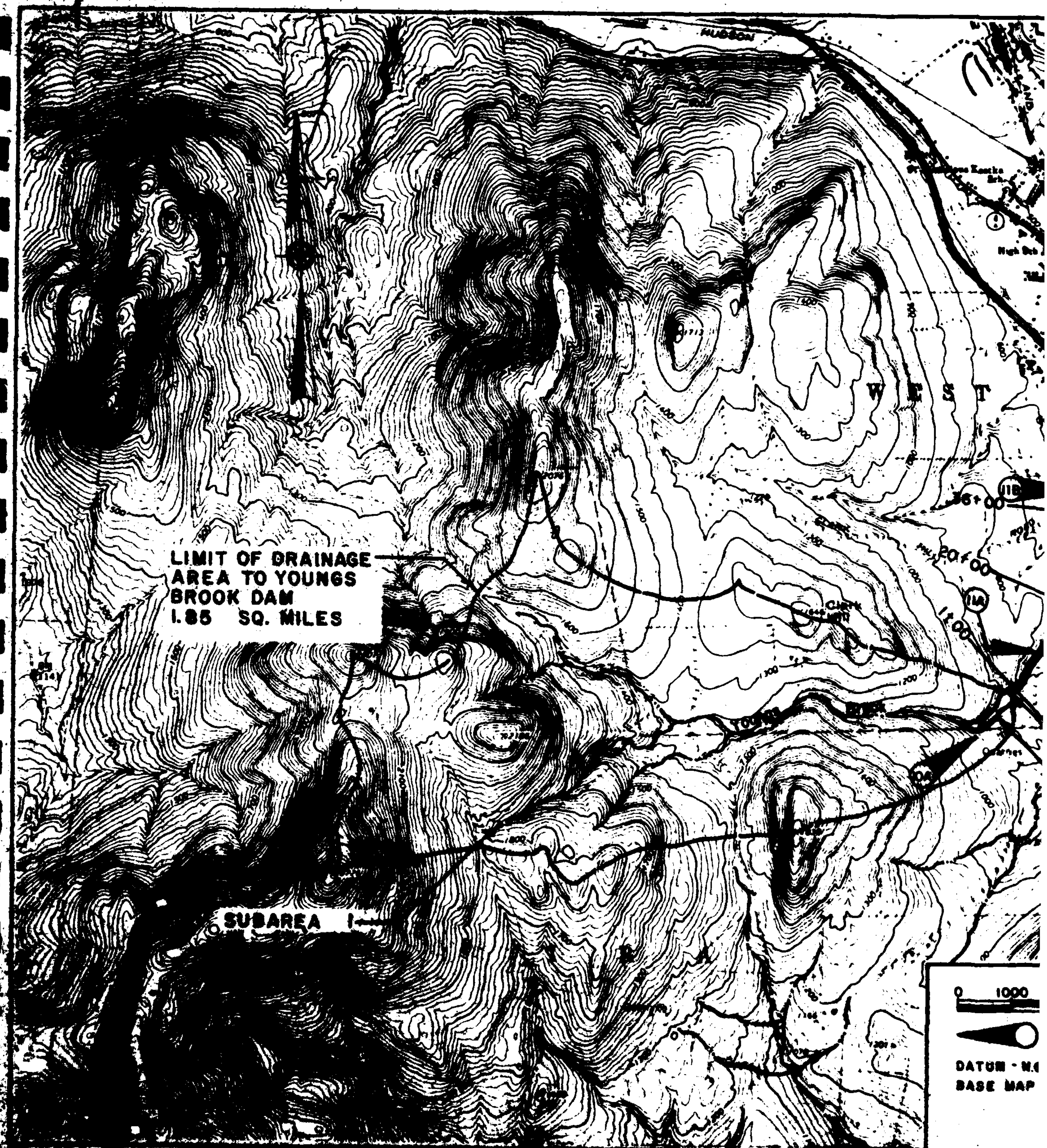
END
DATE
FILMED
8 85

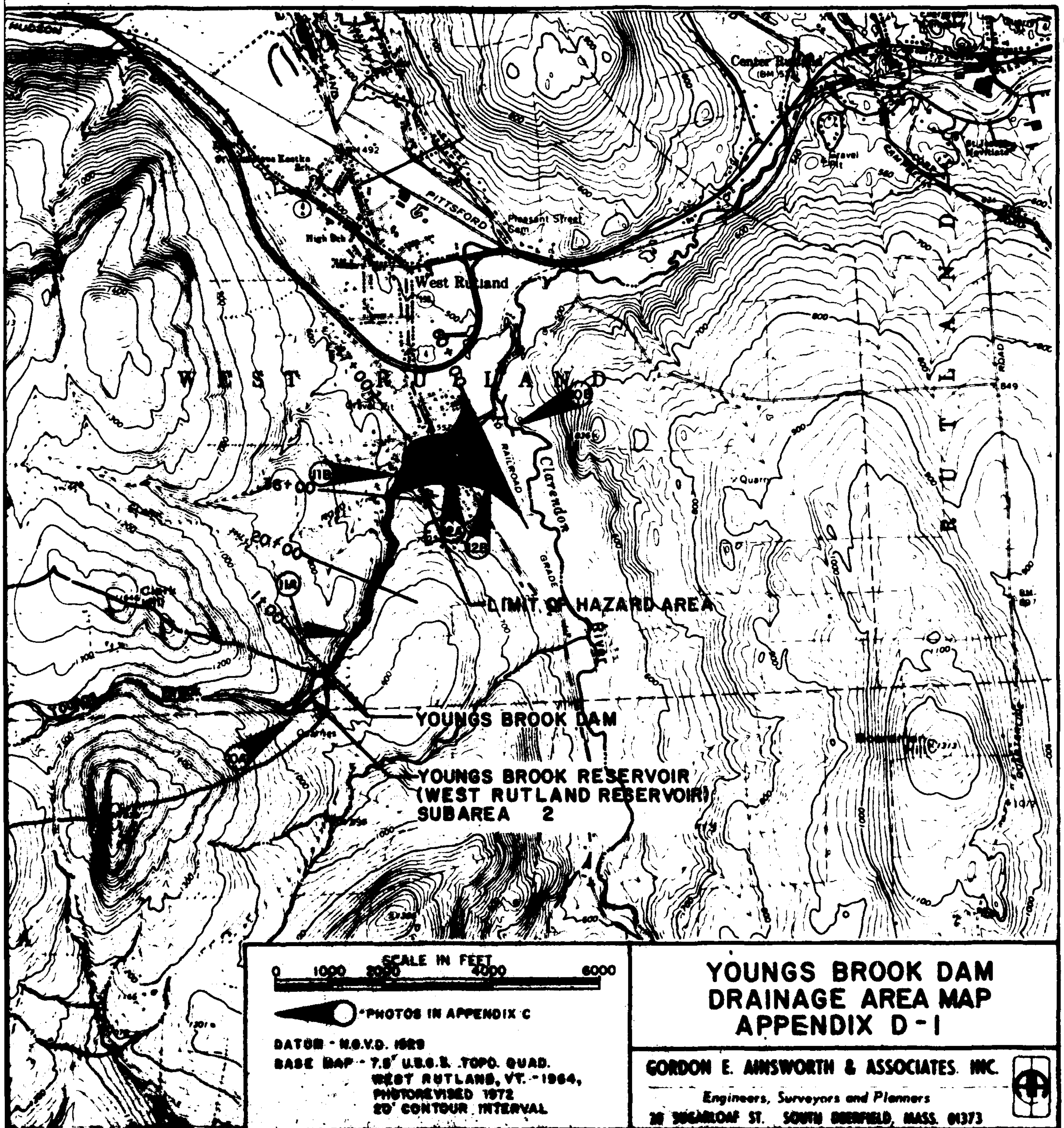

1·0
2·8
2·5
3·15
2·2
3·5
2·0
4·0
1·1
4·5
1·8
1·25
1·4
1·6

APPENDIX D

HYDRAULIC AND HYDROLOGIC COMPUTATIONS

TABLE OF CONTENTS


LIMIT OF DRAINAGE AREA TO YOUNGS BROOK DAM 1.85 SQ. MILES
SUBAREA I
HUDSON
WEST
Clark
0 1000
DATUM - N.
BASE MAP



YOUNGS BROOK DAM
DRAINAGE AREA MAP
APPENDIX D-1

GORDON E. AINSWORTH & ASSOCIATES, INC.

Engineers, Surveyors and Planners

26 SUGARLOAF ST. SOUTH DEERFIELD, MASS. 01373

DWG. NO. 79-39A

GORDON E. AINSWORTH
& ASSOCIATES, INC.
20 Sugarloaf Street
SOUTH DEERFIELD, MASSACHUSETTS 01373
(413) 665-2161

JOB: YOUNGS BROOK DAM
SHEET NO. ______ OF ______
CALCULATED BY: ELY   DATE: 12/7/79
CHECKED BY: TPB   DATE: 9/23/80
SCALE: 21-06-79105

ELEVATION - AREA - STORAGE COMPUTATIONS

RESERVOIR VOLUME: COMPUTED BY METHOD OF CONIC SECTIONS

$$\Delta V_{12} = \frac{h}{3}\left(A_1 + A_2 + \sqrt{A_1 A_2}\right)$$

ELEVATION (NGVD - ft)*	AREA (acres)		h (ft)	ΔV (ac-ft)	V_T (ac-ft)	HEC-1DB V_T	REPORTED V_T**
803 (660)	0	BATHYMETRIC MAP	0	0	0	0 (ac-ft)	0
808 (665)	0.25	BATHYMETRIC MAP	5	0.4	0.4	0	0.4 AF
813 (670)	0.53	BATHYMETRIC MAP	5	1.9	2.3	2	2.4 AF
823 (680)	0.99	BATHYMETRIC MAP	10	7.5	9.8	10	10.1 AF
833 (690)	1.58	BATHYMETRIC MAP	10	12.7	22.5	23	Not Avail.
NEW SPILLWAY → 841 (698)	2.2 ±	(EST.)	—	—	—	38	11,823,734
OLD SPILLWAY → 843 (700)	2.40	BATH. MAP	10	19.8	42.3	42	Gals. =
TOP OF DAM → 846	2.5 ±	(EST.)	—	—	—	50	36 AF
860	3.1	USGS	17	46.6	88.9	89	@ EL 840 (697)

DRAINAGE AREA

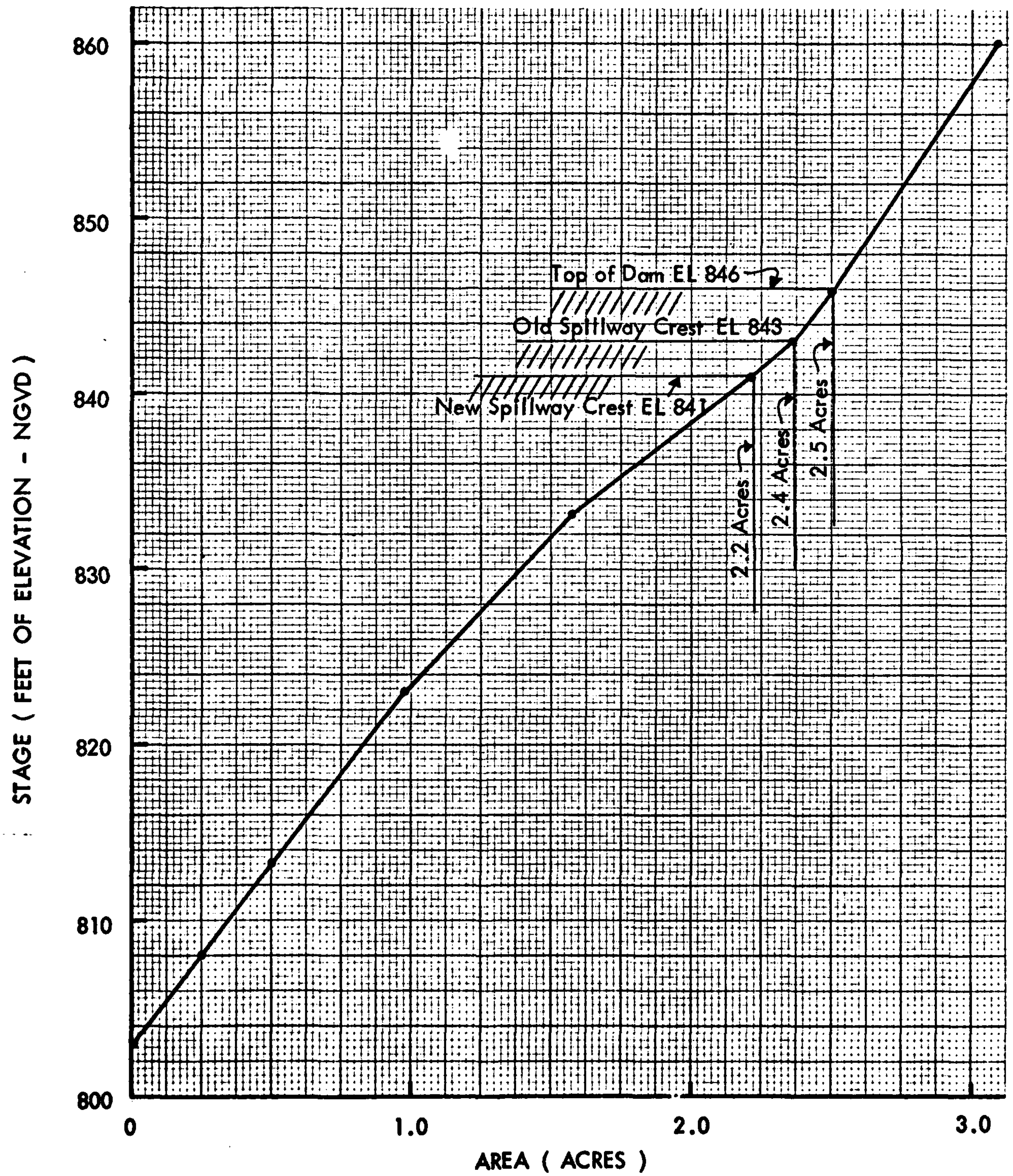
	AREA (acres)	AREA (square miles)	
(BATH. MAP) RESERVOIR SURFACE (SUBAREA 2) @ NEW SPILLWAY CREST EL.=841	2.2	0.004 ±	(0.2%)
(USGS MAP) REMAINDER OF DRAINAGE BASIN (SUBAREA 1)	1179.3	1.842	
TOTAL DRAINAGE BASIN	1181.5	1.846	SAY 1.85

* ELEVATION BASE ON PLAN ENTITLED "WEST RUTLAND RESERVOIR" IS APPROXIMATELY 143' LOWER THAN NGVD ELEVATION.

** STORAGE VOLUMES FOR RESERVOIR FROM TABLE ENTITLED "CAPACITY OF WEST RUTLAND RESERVOIR". APPENDIX B3-5

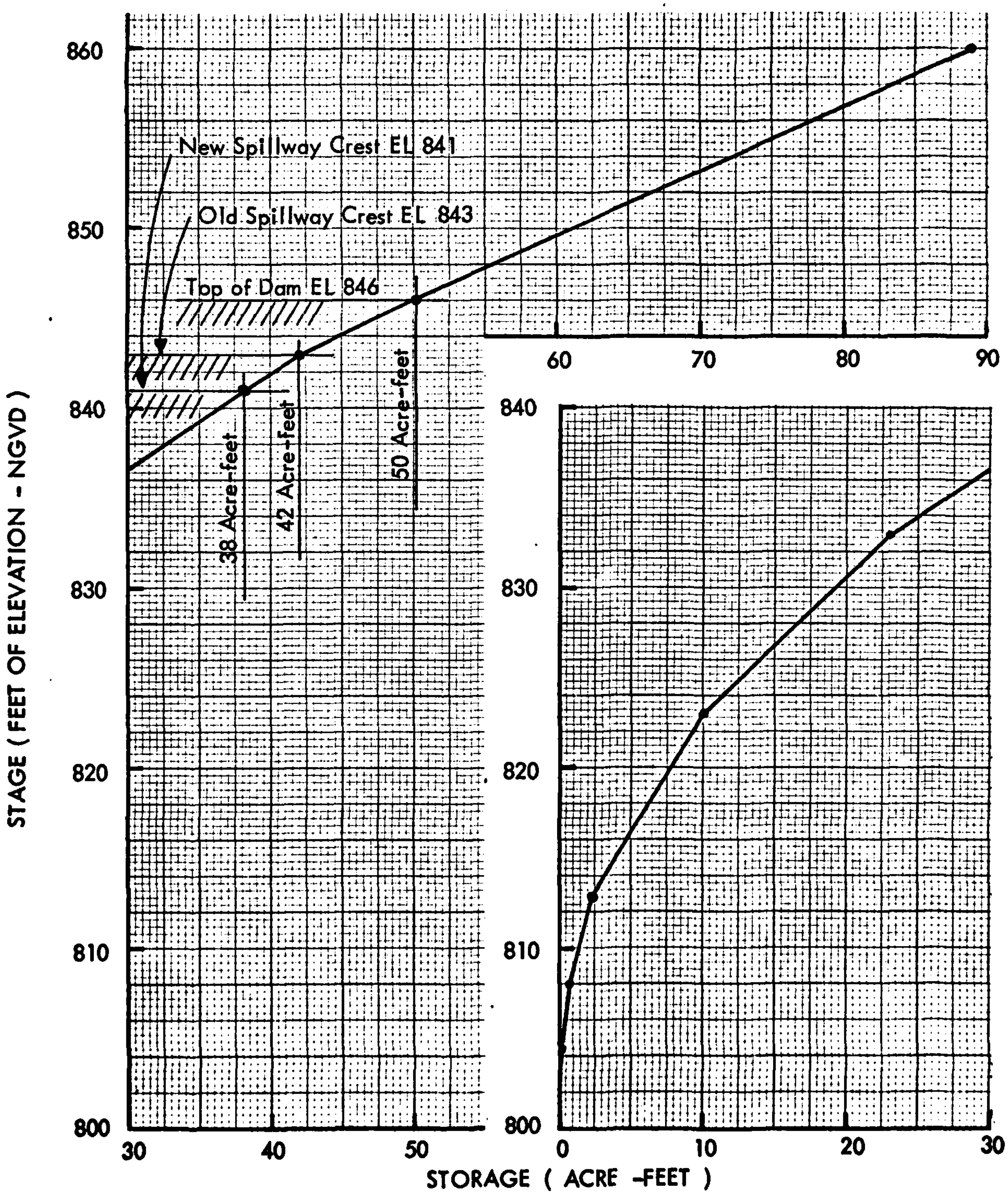
YOUNGS BROOK DAM, WEST RUTLAND, VERMONT

STAGE - AREA



YOUNGS BROOK DAM, WEST RUTLAND, VERMONT

STAGE - STORAGE



G. E. Ainsworth Associates
20 Sugarloaf Street
S. DEERFIELD, MA 01373
Phone 665-2161

JOB YOUNGS BROOK DAM
SHEET NO. ______ OF ______
CALCULATED BY ELV DATE 9/24/80
CHECKED BY [illegible] DATE 9/24/80
SCALE 21-06-79105

DISCHARGE COMPUTATIONS

DAM APPURTENANCE	ELEVATION (NGVD)	SIZE
NEW SPILLWAY (NS)	CREST EL. = 841	28' CREST LENGTH
OLD SPILLWAY (OS)	CREST EL. = 843	19.3' CREST LENGTH
DAM	CREST EL. = 846 (LEVEL)	198' CREST LENGTH
OUTLET WORKS:		
OUTLET CONDUIT	INV. EL. ≈ 828	2'x2' CONC. CONDUIT
WATER SUPPLY MAIN	INV. EL. ≈ 803	12" DIA
DRAIN PIPE (MUD PIPE)	INV. EL. ≈ 798	8" DIA

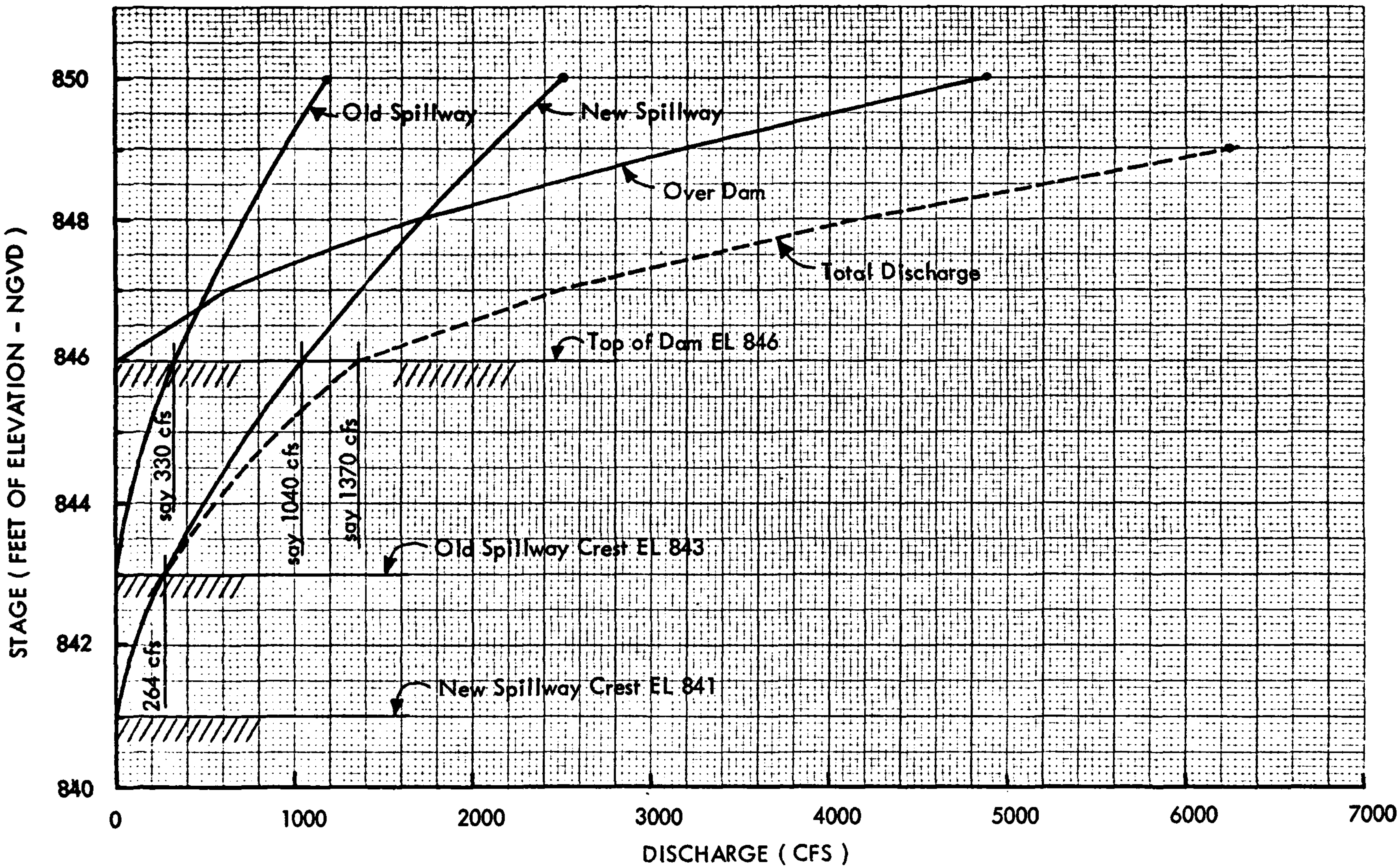
FOR FLOW OVER SPILLWAYS: $Q = 3.33 LH^{1.5}$ (FORMULA FOR CRITICAL FLOW OVER A SHARP-CRESTED WEIR. REF. 9 HAND COMPUTED.)

FOR FLOW OVER DAM: $Q = 3.087 LH^{1.5}$ (FORMULA FOR CRITICAL FLOW OVER A BROAD-CRESTED WEIR. INPUTTED INTO HEC-1 DB COMPUTER PROGRAM FOR DISCHARGE COMPUTATIONS, REF. 9)

ELEVATION (NGVD)		H_{NS} (feet)	H_{OS} (feet)	H_{DAM} (feet)	Q_{NS} (cfs)	Q_{OS} (cfs)	INPUT $Q_{SPILLWAY}$ (cfs)	Q_{DAM} (cfs)	$Q_{OUTLETS}$ (cfs)	Q_{TOTAL} (cfs)
841	NEW SPILLWAY	0	0	0	0	0	0	0	0	0
842		1	0	0	93	0	93	0	ALL OUTLETS ASSUMED CLOSED	93
843	OLD SPILLWAY	2	0	0	264	0	264	0		264
844		3	1	0	484	64	548	0		548
845		4	2	0	746	182	928	0		928
846	TOP OF DAM	5	3	0	1042 SAY 1040	334 SAY 330	1376 SAY 1370	0		1376
847		6	4	1	1370	514	1884	611		2495
848		7	5	2	1727	719	2446	1729		4175
849		8	6	3	2110	945	3055	3176		6231
850		9	7	4	2517	1190	3707	4890	0	8597

YOUNGS BROOK DAM, WEST RUTLAND, VERMONT

STAGE - DISCHARGE



GORDON E. AINSWORTH
& ASSOCIATES, INC.
20 Sugarloaf Street
SOUTH DEERFIELD, MASSACHUSETTS 01373
(413) 665-2161

JOB YOUNGS BROOK DAM
SHEET NO. ____ OF ____
CALCULATED BY ELV DATE 12/7/79
CHECKED BY [illegible] DATE 9/23/80
SCALE 21-06-79105

DRAINAGE AREA DATA FOR HEC-1 DB MODEL

SUBAREA 1: AREA TRIBUTARY DIRECTLY TO RESERVOIR
AREA = 1.842 SQUARE MILES

LOSS RATES: 1.0" - INITIALLY
0.1"/HOUR - CONSTANT LOSS RATE

UNIT HYDROGRAPH PARAMETERS: USED SNYDER METHOD

A = DRAINAGE AREA = 1.842 SQUARE MILES
L = LENGTH ALONG MAIN WATERCOURSE TO UPSTREAM LIMIT OF DRAINAGE AREA = 2.5 MILES
L_{CA} = LENGTH ALONG MAIN WATERCOURSE TO POINT OPPOSITE THE CENTROID OF THE DRAINAGE AREA = 1.25 MILES
C_t = SNYDER'S BASIN COEFFICIENT = 2.0 ASSUMED AVERAGE
C_p = SNYDER'S PEAKING COEFFICIENT = 0.625 ASSUMED AVERAGE
t_p = STANDARD LAG IN HOURS = $C_t(L L_{CA})^{0.3}$ = 2.8 HOURS

ALTERNATE $t_p = \frac{\text{flow length}}{\text{flow velocity}} = \frac{13{,}200'}{2\text{ ft/sec}} = 1.8$ HOURS

∴ USE t_p = 2.0 HOURS (BETWEEN CALCULATED VALUES BECAUSE OF STEEP SLOPES)

SUBAREA 2: RESERVOIR SURFACE, AREA = 0.004 SQUARE MILES (2.23 ACRES)

LOSS RATES: NONE BECAUSE RAINFALL ≈ RUNOFF FOR WATER SURFACE

UNIT HYDROGRAPH PARAMETERS:

FOR U.H. w/ 5 MINUTES DURATION & 1" RAIN

$$\bar{Q} = \frac{A(1")}{t} = \frac{2.2\text{ acres}(1")}{5\text{ minutes}}\left(\frac{43560\text{ sq. ft.}}{1\text{ acre}}\right)\left(\frac{1'}{12"}\right)\left(\frac{1\text{ minute}}{60\text{ seconds}}\right)$$

$\bar{Q}$ = 27 cfs (SINCE NO LOSS RATE)

```
A   RED DAM INSPECTION  CONTRACT: DACW33-80-C-0012      Y0071
A     T. C0165  YOUNGS BROOK DAM. 21-06-79105
A   OVERTOPPING - TEST FLOOD = ONE-HALF PMF
B       288         0         5                                                4
B1        4
J         1         1         1
J1      .50
K         0      SA-1                                    1
K1  SUBAREA 1 RUNOFF COMPUTATION
M         1         1     1.842                   10                           1
P                18.5       111       123        132
T                                                        1.0       .10
W       2.9      .025
X       -4.         0         1
K         0    SA-RES                                    1
K1  SUBAREA RESERVOIR RUNOFF
M         1        -1      .084                   10                           1
P                18.5       111       123        132
T                                                          0         0
J         1
J1       27
X       -4.         0         1
K         2     SA-2C                                    1
K1  COMBINING HYDROGRAPHS 1,2
K         1       RES                              2     1
K1  ROUTING FLOWS  THROUGH RESERVOIR
Y                                       1          1
Y1        1                                             -841        -1
Y4      841       842       843       844        845       846       847       848       849       850
Y5        0        93       264       548        928      1376      1884      2446      3055      3707
SA        0       .28       .53       .99       1.58      2.40       3.1
SE      803       808       813       823        833       843       860
S0      841
SD      846     3.087       1.5       198
K        99
A
A
A
A
A
```

```
**********************************
FLOOD HYDROGRAPH PACKAGE (HEC-1)
DAM SAFETY VERSION      JULY 1978
  LAST MODIFICATION  26 FEB 79
**********************************

RUN DATE: 9/24/80
    TIME: 10:19 AM

              NED DAM INSPECTION  CONTRACT: DACW33-80-C-0012      YBOT1
              VT. 00165  YOUNGS BROOK DAM, 21-06-79165
              OVERTOPPING - TEST FLOOD = ONE-HALF PMF

                                   JOB SPECIFICATION
              NO      NHR     NMIN    IDAY     IHR     IMIN   METRC    IPLT    IPRT   NSTAN
             288        0        5       0       0        0       0       0       4       0
                                     JOPER     NWT   LROPT   TRACE
                                         5       0       0       0

                         MULTI-PLAN ANALYSES TO BE PERFORMED
                            NPLAN= 1 NRTIO= 1 LRTIO= 1
               RTIOS=    0.50

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

                                SUB-AREA RUNOFF COMPUTATION

              SUBAREA 1 RUNOFF COMPUTATION
                       ISTAQ    ICOMP   IECON   ITAPE    JPLT    JPRT   INAME  ISTAGE   IAUTO
                       SA-1         0       0       0       0       0       1       0       0

                                    HYDROGRAPH DATA
              IHYDG    IUHG   TAREA    SNAP   TRSDA   TRSPC   RATIO   ISNOW   ISAME   LOCAL
                  1       1    1.84    0.00   10.00    0.00   1.000       0       1       0

                                     PRECIP DATA
                            SPFE     PMS      R6     R12     R24     R48     R72     R96
                            0.00   18.50  111.00  123.00  132.00    0.00    0.00    0.00
TRSPC COMPUTED BY THE PROGRAM IS 0.800

                                      LOSS DATA
            LROPT   STRKR   DLTKR   RTIOL   ERAIN   STRKS   RTIOK   STRTL   CNSTL   ALSMX   RTIMP
                0    0.00    0.00    1.00    0.00    0.00    1.00    1.00    0.10    0.00    0.00

                                  UNIT HYDROGRAPH DATA
                                TP=  2.00    CP=0.63    NTA=  0

                                    RECESSION DATA
                            STRTQ=   -4.00    QRCSN=    0.00    RTIOR= 1.00

          UNIT HYDROGRAPH100 END-OF-PERIOD ORDINATES, LAG=   1.99 HOURS, CP= 0.62  VOL= 0.98
```

3.	13.	26.	42.	60.	80.	101.	123.	146.	170.
194.	219.	244.	269.	291.	312.	330.	345.	358.	369.
377.	383.	386.	386.	382.	374.	361.	345.	335.	316.
302.	289.	276.	264.	253.	242.	231.	221.	211.	202.
193.	185.	177.	169.	162.	155.	148.	141.	135.	129.
124.	118.	113.	108.	104.	99.	95.	91.	87.	83.
79.	76.	72.	69.	66.	63.	61.	58.	55.	53.
51.	49.	46.	44.	42.	41.	39.	37.	36.	34.
33.	31.	30.	28.	27.	26.	25.	24.	23.	22.
21.	20.	19.	19.	17.	17.	16.	15.	15.	14.

```
                                              END-OF-PERIOD FLOW
 MO.DA  HR.MN  PERIOD   RAIN   EXCS   LOSS     COMP Q       MO.DA  HR.MN  PERIOD   RAIN   EXCS   LOSS    COMP Q

                                                                         SUM  19.54  16.89   2.64   235292.
                                                                            ( 496.)( 429.)(  67.)( 6662.72)

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

                                          SUB-AREA RUNOFF COMPUTATION

             SUBAREA RESERVOIR RUNOFF
                               ISTAG   ICOMP   IECON   ITAPE    JPLT    JPRT   INAME  ISTAGE   IAUTO
                               SA-RES      0       0       0       0       0       1       0       0

                                                HYDROGRAPH DATA
                 IHYDG   IUHG   TAREA    SNAP   TRSDA   TRSPC   RATIO   ISNOW   ISAME   LOCAL
                     1     -1    0.00    0.00   10.00    0.00   0.000       0       1       0

                                                  PRECIP DATA
                         SPFE     PMS      P6     R12     R24     R48     R72     R96
                         0.00   18.50  111.00  123.00  132.00    0.00    0.00    0.00
 TRSPC COMPUTED BY THE PROGRAM IS 0.800

                                                   LOSS DATA
             LROPT   STRKR   DLTKR   RTIOL   ERAIN   STRKS   RTIOK   STRTL   CNSTL   ALSMX   RTIMP
                 0    0.00    0.00    1.00    0.00    0.00    1.00    0.00    0.00    0.00    0.00

                                                RECESSION DATA
                               STRTQ=   -4.00   QRCSN=    0.00   RTIOR= 1.00

                                              END-OF-PERIOD FLOW
 MO.DA  HR.MN  PERIOD   RAIN   EXCS   LOSS     COMP Q       MO.DA  HR.MN  PERIOD   RAIN   EXCS   LOSS    COMP Q

                                                                         SUM  19.54  19.54   0.00     51F.
                                                                            ( 496.)( 496.)(   0.)(  14.67)

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

                                             COMBINE HYDROGRAPHS

             COMBINING HYDROGRAPHS 1,2
                               ISTAG   ICOMP   IECON   ITAPE    JPLT    JPRT   INAME  ISTAGE   IAUTO
                               SA-2C       2       0       0       0       0       1       0       0

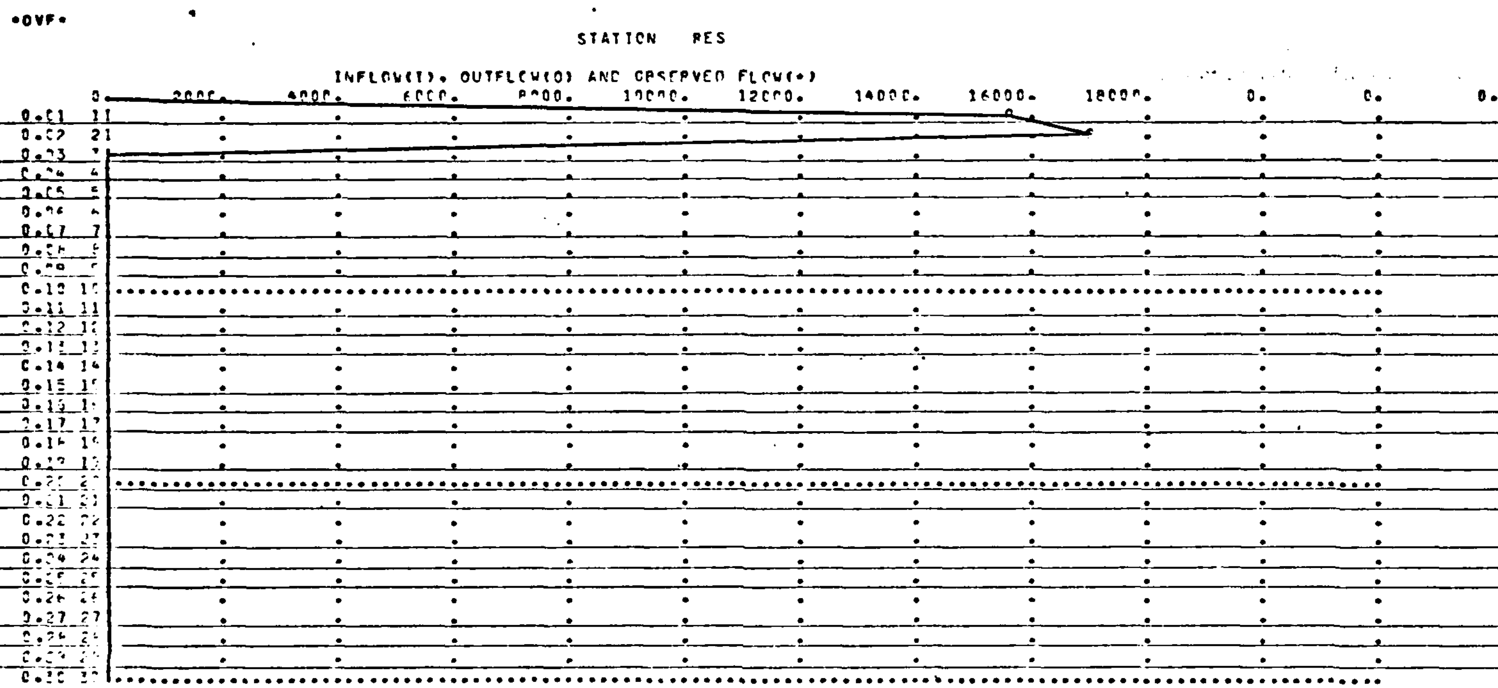
          **********         **********         **********         **********         **********

                                             HYDROGRAPH ROUTING

             ROUTING FLOWS THROUGH RESERVOIR
                               ISTAG   ICOMP   IECON   ITAPE    JPLT    JPRT   INAME  ISTAGE   IAUTO
                                 RES       1       0       0       2       0       1       0       0
                                                 ROUTING DATA
                       QLOSS   CLOSS     AVG    IRES   ISAME    IOPT    IPMP            LSTR
                         0.0   0.000    0.00       1       1       0       0               0

                               NSTPS   NSTDL     LAG   AMSKK       X     TSK    STORA  ISPRAT
                                   1       0       0   0.000   0.000   0.000    -841.      -1

   TIME    841.00    842.00    843.00    844.00    845.00    846.00    847.00    848.00    849.00
```

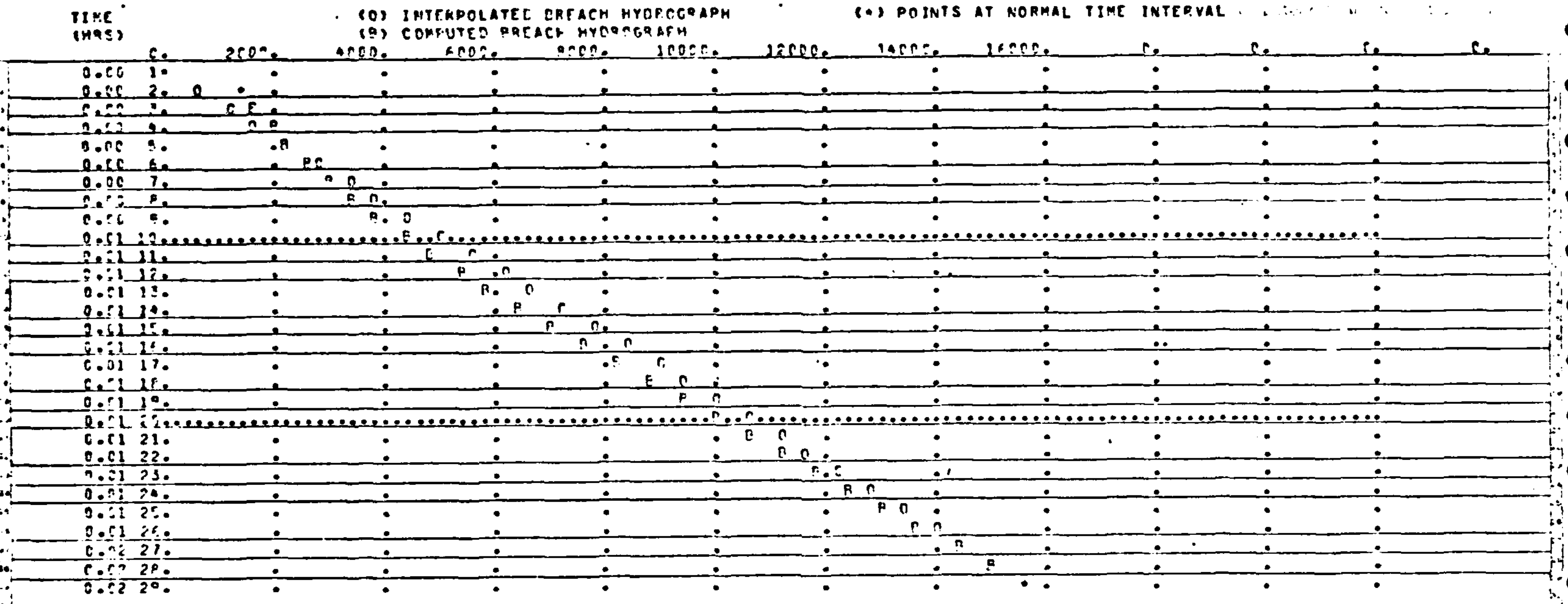

OVF
STATION RES
INFLOW(I), OUTFLOW(O) AND OBSERVED FLOW(*)
0. 2000. 4000. 6000. 8000. 10000. 12000. 14000. 16000. 18000. 0. 0. 0.

•OVF•

STATION RES

(O) INTERPOLATED BREACH HYDROGRAPH (•) POINTS AT NORMAL TIME INTERVAL

(B) COMPUTED BREACH HYDROGRAPH



THE DAM BREACH HYDROGRAPH WAS DEVELOPED USING A TIME INTERVAL OF 0.001 HOURS DURING BREACH FORMATION.
DOWNSTREAM CALCULATIONS WILL USE A TIME INTERVAL OF 0.017 HOURS.
THIS TABLE COMPARES THE HYDROGRAPH FOR DOWNSTREAM CALCULATIONS WITH THE COMPUTED BREACH HYDROGRAPH.
INTERMEDIATE FLOWS ARE INTERPOLATED FROM END-OF-PERIOD VALUES.

TIME (HOURS)	TIME FROM BEGINNING OF BREACH (HOURS)	INTERPOLATED BREACH HYDROGRAPH (CFS)	-	COMPUTED BREACH HYDROGRAPH (CFS)	=	ERROR (CFS)	ACCUMULATED ERROR (CFS)	ACCUMULATED ERROR (AC-FT)
0.000	0.000	0.		0.		0.	0.	0.
0.001	0.001	557.		1480.		-923.	-923.	-0.
0.001	0.001	1114.		1679.		-565.	-1488.	-0.
0.002	0.002	1671.		1941.		-270.	-1758.	-0.
0.002	0.002	2228.		2253.		-25.	-1783.	-0.
0.003	0.003	2785.		2605.		179.	-1604.	-0.
0.004	0.004	3342.		2994.		348.	-1256.	-0.
0.004	0.004	3899.		3414.		484.	-772.	-0.
0.005	0.005	4456.		3862.		593.	-179.	-0.
0.005	0.005	5013.		4335.		678.	499.	0.
0.006	0.006	5570.		4829.		742.	1240.	0.
0.007	0.007	6127.		5343.		784.	2024.	0.
0.007	0.007	6684.		5873.		811.	2834.	0.
0.008	0.008	7241.		6418.		823.	3657.	0.
0.008	0.008	7798.		6976.		822.	4480.	0.
0.009	0.009	8355.		7553.		802.	5282.	0.
0.010	0.010	8912.		8139.		773.	6054.	0.
0.010	0.010	9469.		8733.		736.	6791.	0.
0.011	0.011	10026.		9331.		694.	7485.	0.
0.011	0.011	10583.		9934.		649.	8134.	0.
0.012	0.012	11139.		10545.		595.	8729.	0.
0.012	0.012	11696.		11171.		526.	9254.	0.
0.013	0.013	12253.		11797.		456.	9710.	0.
0.014	0.014	12810.		12422.		389.	10099.	0.
0.014	0.014	13367.		13045.		322.	10421.	1.
0.015	0.015	13924.		13692.		232.	10654.	1.
0.015	0.015	14481.		14331.		151.	10804.	1.
0.016	0.016	15038.		14960.		78.	10883.	1.
0.017	0.017	15595.		15595.		0.	10883.	1.

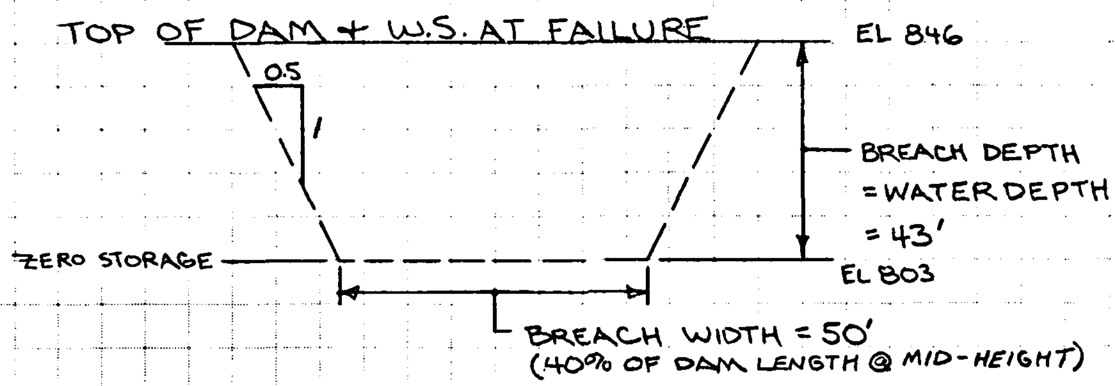
```
A  NED DAM INSPECTION  CONTRACT: DACW33-80-C-0012       YBDP1
A  VT. 00168  YOUNGS BROOK DAM, 21-06-79195
A BREACH AT TOP OF DAM
B       30        0        1                                                    4
B1      5
J       1         1        1
J1    1.0
K       0   SA-RES                        1           1
K1 SUBAREA RESERVOIR RUNOFF
M       1        -1     .704                                                    1
P
T
U       1
U1
X       5         0        1
K       1       RES                       2        1        1
K1 ROUTING FLOWS  THROUGH RESERVOIR
Y                                1        1
Y1      1                                                -846       -1
Y4    841       842      843      844      845      846      847      848      849      850
Y5      0        93      264      548      928     1776     1884     2446     3255     3707
$A      0       .25      .53      .99     1.58     2.40      3.1
$E    807       808      813      823      833      847      860
$$    841
$D    846     3.087      1.5      198
$B     50       5.5      803      .03      846      846
K       1     20+00                                 3        1
K1 CHANNEL ROUTING   STA 20+00
Y                                1        1
Y1      1
Y6    .04       .03      .04      700      719     2000     .052
Y7   1000       750     1090      720     1190      702     1195      700     1205      700
Y7   1205       702     1260      720     1370      750
K       1     36+00                                 3        1
K1 CHANNEL ROUTING STA 36+00
Y                                1        1
Y1      1
Y6    .04       .03      .04      600      619     1600     .063
Y7   1000       640     1200      620     1460      620     1595      600     1605      600
Y7   1720       620     1850      640     2000      640
K       1     47+00                                 3        1
K1 CHANNEL ROUTING STA 47+00
Y                                1        1
Y1      1
Y6    .04       .03      .04      557      576     1100     .039
Y7   1500       580     1000      570     1470      560     1500      557     1520      557
Y7   1560       560     2500      560     2500      580
K       1     60+00                                 3        1
K1 CHANNEL ROUTING STA 60+00
Y                                1        1
Y1      1
Y6    .04       .03      .04      519      538     1300     .029
Y7   1000       540     1000      520     2180      520     2150      519     2210      519
Y7   2220       520     4000      520     4000      540
K      99
A
A
A
A
A
```

G. E. Ainsworth Associates
20 Sugarloaf Street
S. DEERFIELD, MA 01373
Phone 665-2161

JOB: YOUNGS BROOK DAM
SHEET NO. ______ OF ______
CALCULATED BY: ELV DATE: 9-19-80
CHECKED BY: JPB DATE: 9/23/80
SCALE: 21-06-79105

BREACH CRITERIA

EARTH DAM



RULE OF THUMB PEAK OUTFLOW - SUDDEN BREACH

$Q_p = 8/27\, W_b \sqrt{g}\, Y_o^{1.5}$ $\quad w_b$ = BREACH WIDTH = 50'

Q_p = 23700 cfs ± $\quad Y_o$ = WATER DEPTH = 43'

ADDITIONAL FLOW

NONE BECAUSE BREACH IS ASSUMED TO INCLUDE THE SPILLWAY

TOTAL PEAK OUTFLOW FROM DAM

Q_p = 23,700 + 0 = 23,700 cfs SAY 24,000 cfs

HEC-1 DB BREACH PROGRAM

CALCULATION INTERVAL = 1 MINUTE

BREACH TIME	PEAK OUTFLOW (cfs)
.01 HR	43,300
.04 HR	18,800
.03 HR	24,000

SUMMARY OF DAM SAFETY ANALYSIS

PLAN 1 ....

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	846.00	841.00	846.00
STORAGE	50.	38.	50.
OUTFLOW	1376.	0.	1376.

RATIO OF PMF	MAXIMUM RESERVOIR W.S.ELEV	MAXIMUM DEPTH OVER DAM	MAXIMUM STORAGE AC-FT	MAXIMUM OUTFLOW CFS	DURATION OVER TOP HOURS	TIME OF MAX OUTFLOW HOURS	TIME OF FAILURE HOURS
1.00	846.00	0.00	50.	1376.	0.00	0.00	0.00

PLAN 1 STATION 1+00

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
1.00	1376.	805.1	0.02

PLAN 1 STATION 20+00

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
1.00	1376.	70[illegible].8	0.02

PLAN 1 STATION 36+00

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
1.00	1376.	602.8	0.02

PLAN 1 STATION 47+00

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
1.00	1376.	555.3	0.02

PLAN 1 STATION 60+00

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
1.00	1376.	520.1	0.02

PEAK FLOW AND STORAGE (END OF PERIOD) SUMMARY FOR MULTIPLE PLAN-RATIO ECONOMIC COMPUTATIONS
FLOWS IN CUBIC FEET PER SECOND (CUBIC METERS PER SECOND)
AREA IN SQUARE MILES (SQUARE KILOMETERS)

OPERATION	STATION	AREA	PLAN	RATIOS APPLIED TO FLOWS RATIO 1 1.00
HYDROGRAPH AT	SA-RES	0.00	1	1376.
	(	0.01)	(	38.96)(
ROUTED TO	RES	0.00	1	1376.
	(	0.01)	(	38.96)(
ROUTED TO	1+00	0.00	1	1376.
	(	0.01)	(	38.96)(
ROUTED TO	20+00	0.00	1	1376.
	(	0.01)	(	38.96)(
ROUTED TO	36+00	0.00	1	1376.
	(	0.01)	(	38.96)(
ROUTED TO	47+00	0.00	1	1376.
	(	0.01)	(	38.96)(
ROUTED TO	60+00	0.00	1	1376.
	(	0.01)	(	38.96)(

```
A  NED DAM INSPECTION   CONTRACT: DACW33-80-C-0012      YBPF1
A  VT. 00165  YOUNGS BROOK DAM, 21-06-79105
A PRIOR FLOW AT TOP OF DAM
B      10        0        1                                              4
B1      5
J       1        1        1
J1    1.0
K       0   SA-RES                                        1
K1 SUBAREA RESERVOIR RUNOFF
M       1       -1     .004                                              1
P
T
U       1
U1
X    1376        0        1
K       1      RES                               1        1
K1 ROUTING FLOWS  THROUGH RESERVOIR
Y                                  1        1
Y1      1                                             -846      -1
Y4    841      842      843      844      845      846     847     848      849      850
Y5      0       53      264      548      928     1376    1864    2446     3055     3707
SA      0      .25      .53      .99     1.58     2.40     3.1
SE    803      806      813      823      833      843     860
$$    841
SD    846    3.067      1.5      198
K       1     1+00                               3        1
K1 CHANNEL ROUTING  STA 1+00
Y                                  1        1
Y1      1                                               -1
Y6    .04      .03      .04      800      819      100     .03
Y7   1200      840     1120      820     1215      802    1215     800     1225      800
Y7   1225      802     1270      820     1310      840
K       1    20+00                               3        1
K1 CHANNEL ROUTING   STA 20+00
Y                                  1        1
Y1      1                                               -1
Y6    .04      .03      .04      700      719     1000    .054
Y7   1000      750     1090      720     1190      702    1195     700     1205      700
Y7   1205      702     1280      720     1370      759
K       1    36+00                               3        1
K1 CHANNEL ROUTING STA 36+00
Y                                  1        1
Y1      1                                               -1
Y6    .04      .03      .04      600      619     1600    .063
Y7   1550      640     1400      620     1460      620    1595     600     1605      600
Y7   1720      620     1850      640     2000      640
K       1    47+00                               3        1
K1 CHANNEL ROUTING STA 47+00
Y                                  1        1
Y1      1                                               -1
Y6    .04      .03      .04      557      576     1100    .039
Y7   1000      580     1000      570     1470      569    1500     557     1520      557
Y7   1560      560     2500      560     2500      580
K       1    60+00                               3        1
K1 CHANNEL ROUTING STA 60+00
Y                                  1        1
Y1      1                                               -1
Y6    .04      .03      .04      519      538     1100    .020
Y7   1100      540     1300      520     2160      520    2190     519     2210      519
Y7   2320      520     4100      520     4000      540
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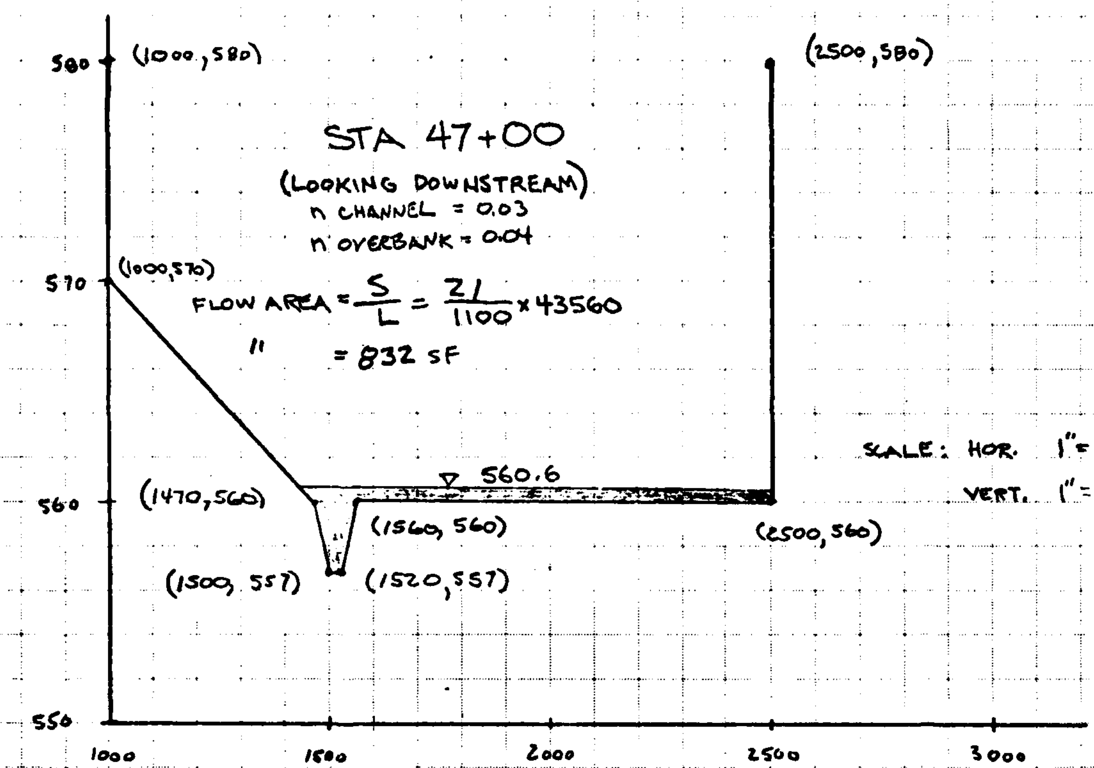
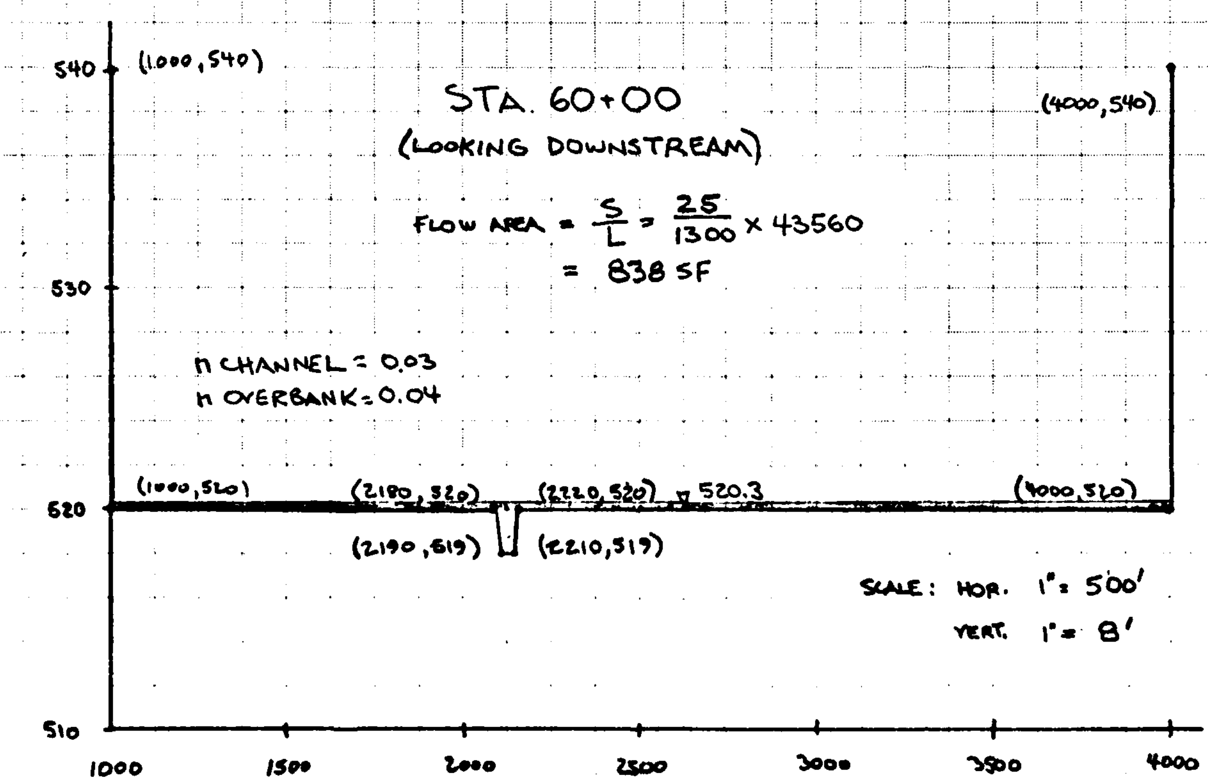
(413) 665-2161

JOB YOUNGS BROOK DAM
SHEET NO. ______ OF ______
CALCULATED BY CLV DATE 1/11/79
CHECKED BY TPB DATE 9/23/80
SCALE 21-06-79110





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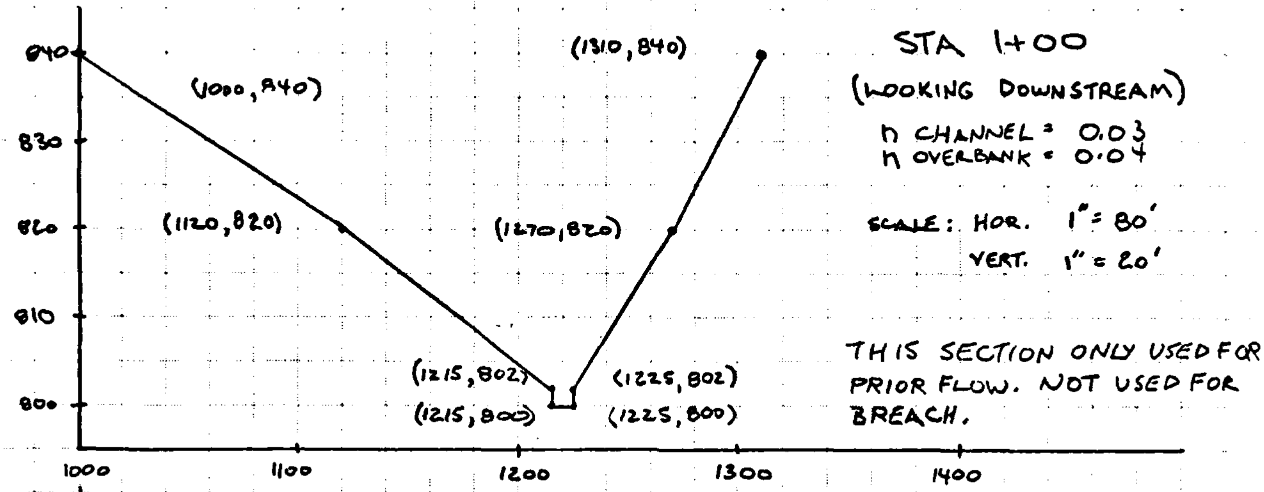
JOB YOUNGS BROOK DAM

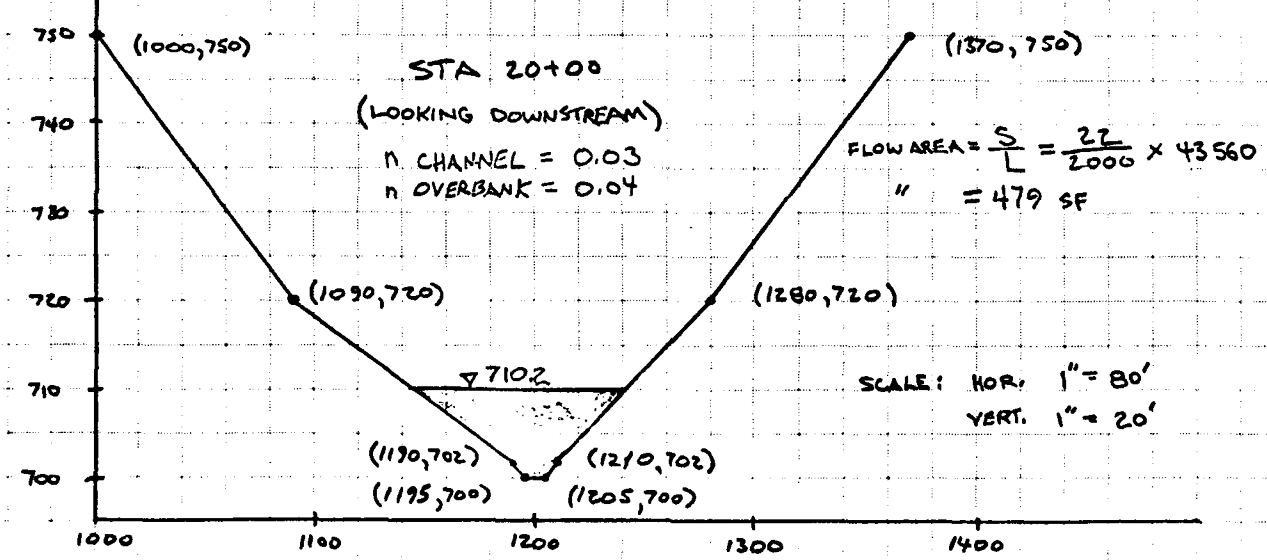
SHEET NO. ______ OF ______

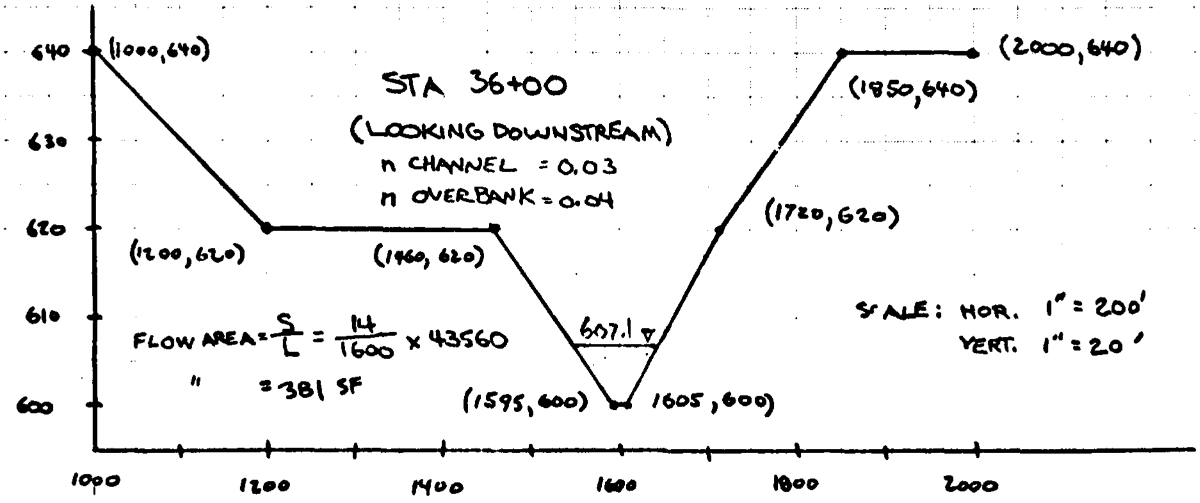
CALCULATED BY ELV DATE 1/11/79

CHECKED BY GPB DATE 9/23/80

SCALE 21-06-79110







SUMMARY OF DAM SAFETY ANALYSIS

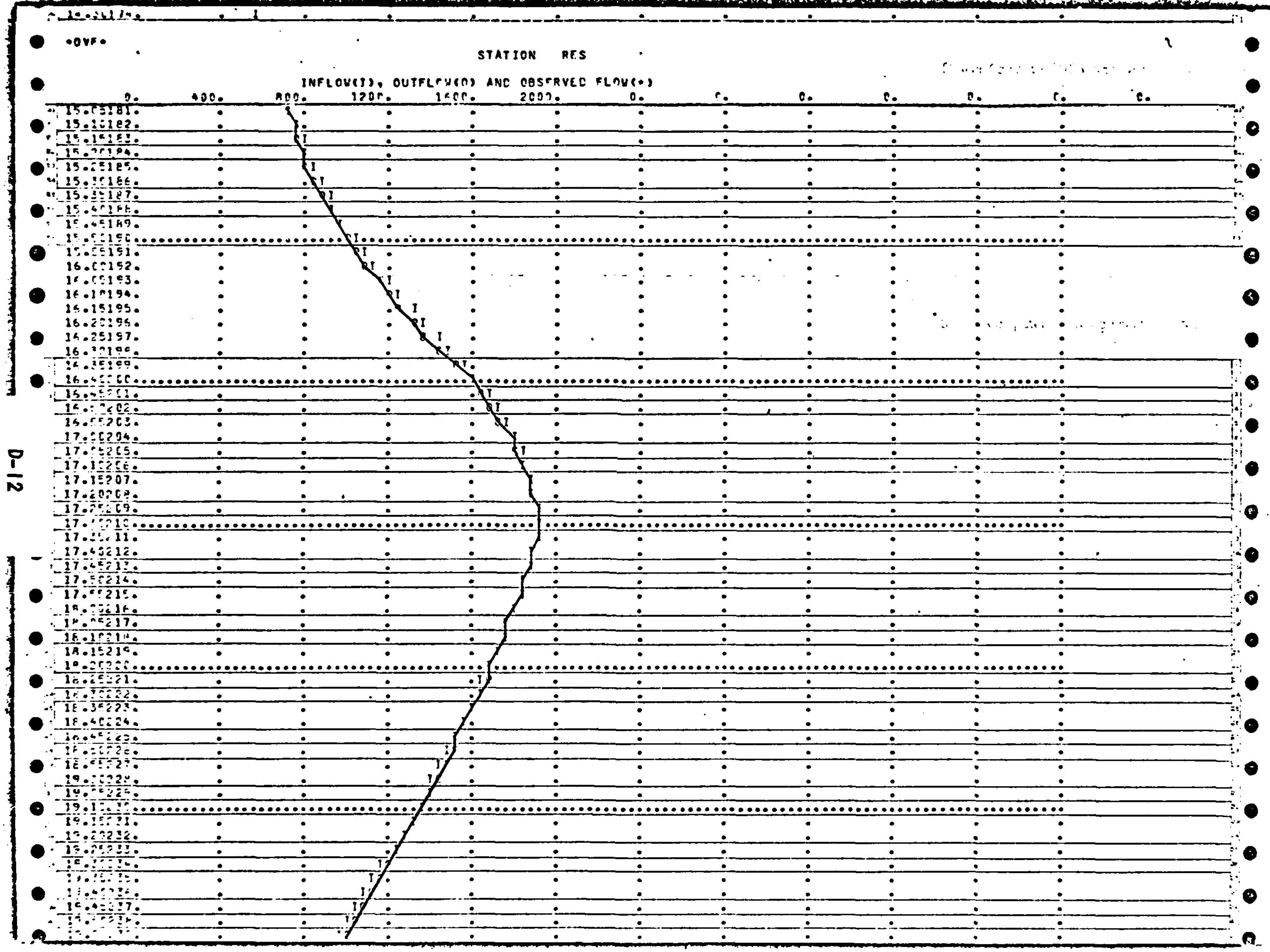
PLAN 1 ....

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	841.00	841.00	846.00
STORAGE	38.	38.	50.
OUTFLOW	0.	0.	1376.

RATIO OF PMF	MAXIMUM RESERVOIR W.S.ELEV	MAXIMUM DEPTH OVER DAM	MAXIMUM STORAGE AC-FT	MAXIMUM OUTFLOW CFS	DURATION OVER TOP HOURS	TIME OF MAX OUTFLOW HOURS	TIME OF FAILURE HOURS
0.50	846.55	0.55	51.	1510.	2.75	17.50	0.00

PEAK FLOW AND STORAGE (END OF PERIOD) SUMMARY FOR MULTIPLE PLAN-RATIO ECONOMIC COMPUTATIONS
FLOWS IN CUBIC FEET PER SECOND (CUBIC METERS PER SECOND)
AREA IN SQUARE MILES (SQUARE KILOMETERS)

OPERATION	STATION	AREA	PLAN	RATIOS APPLIED TO FLOWS RATIO 1 0.50
HYDROGRAPH AT	SA-1	1.84	1	1909.
		(4.77)		(54.06)(
HYDROGRAPH AT	SA-RES	0.00	1	24.
		(0.01)		(0.67)(
2 COMBINED	SA-2C	1.85	1	1911.
		(4.78)		(54.12)(
ROUTED TO	RES	1.85	1	1910.
		(4.78)		(54.10)(


STATION RES
INFLOW(I), OUTFLOW(O) AND OBSERVED FLOW(*)
0. 400. 800. 1200. 1600. 2000. 0. 0. 0. 0. 0. 0. 0.

FLOW	0.00	93.00	264.00	548.00	928.00	1376.00	1884.00	2446.00	3055.00
SURFACE AREA=	0.	0.	1.	1.	2.	2.	3.		
CAPACITY=	0.	0.	2.	10.	23.	42.	89.		
ELEVATION=	803.	808.	813.	823.	833.	843.	860.		

CREL	SPWID	COQW	EXPW	ELEVL	COQL	CAREA	EXPL
841.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

DAM DATA

TOPEL	COQD	EXPD	DAMWID
846.0	3.1	1.5	198.

PEAK OUTFLOW IS 1910. AT TIME 17.50 HOURS

PEAK FLOW AND STORAGE (END OF PERIOD) SUMMARY FOR MULTIPLE PLAN-RATIO ECONOMIC COMPUTATIONS
FLOWS IN CUBIC FEET PER SECOND (CUBIC METERS PER SECOND)
AREA IN SQUARE MILES (SQUARE KILOMETERS)

OPERATION	STATION	AREA	PLAN	RATIOS APPLIED TO FLOWS RATIO 1 1.00
HYDROGRAPH AT	SA-RES	0.00 (0.01)	1	0. (0.00)(
ROUTED TO	RES	0.00 (0.01)	1	16946. (479.86)(
ROUTED TO	20+00	0.00 (0.01)	1	14009. (396.70)(
ROUTED TO	36+00	0.00 (0.01)	1	11863. (335.92)(
ROUTED TO	47+00	0.00 (0.01)	1	8431. (238.74)(
ROUTED TO	60+00	0.00 (0.01)	1	5504. (155.86)(

SUMMARY OF DAM SAFETY ANALYSIS

PLAN 1 ....

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	846.00	841.00	846.00
STORAGE	50.	38.	50.
OUTFLOW	1376.	0.	1376.

RATIO OF PMF	MAXIMUM RESERVOIR W.S.ELEV	MAXIMUM DEPTH OVER DAM	MAXIMUM STORAGE AC-FT	MAXIMUM OUTFLOW CFS	DURATION OVER TOP HOURS	TIME OF MAX OUTFLOW HOURS	TIME OF FAILURE HOURS
1.00	845.97	0.00	50.	23575.	0.00	0.03	0.00

PLAN 1 STATION 20+00

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
1.00	14009.	710.2	0.03

PLAN 1 STATION 36+00

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
1.00	11863.	607.1	0.05

PLAN 1 STATION 47+00

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
1.00	8431.	560.6	0.07

PLAN 1 STATION 60+00

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
1.00	5504.	520.3	0.10

APPENDIX E

INFORMATION AS CONTAINED IN

THE NATIONAL INVENTORY OF DAMS

INFORMATION AS CONTAINED IN

THE NATIONAL INVENTORY OF DAMS

THIS SHEET TO BE FURNISHED BY

THE CORPS OF ENGINEERS

APPENDIX F

REFERENCES

REFERENCES

This is a general list of references pertinent to dam safety investigations. Not all references listed have necessarily been used in this specific report.

1. "Recommended Guidelines For Safety Inspection of Dams", Appendix D of ER 1110-2-106, Dept. of the Army, Office of the Chief of Engineers, Washington, D.C., 26 September 1979.

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17. "Hydraulic Charts For the Selection of Highway Culverts", Hydraulic Engineering Circular No. 5, U.S. Department of Commerce, Bureau of Public Roads, December 1965.

18. 33 CFR Part 22, Final Rule, "Engineering and Design; National Program For Inspection of Non-Federal Dams", ER 1110-2-106, U.S. Army Corps of Engineers, March 24, 1980.

19. "Water Resources Data For New Hampshire and Vermont - Water Year 1977", USGS Water-Data Report NH-VT-77-1, U.S. Geological Survey, Boston, Ma., 1978.

20. "Climatological Data - May 1979 - New England", Volume 91, No. 5, National Oceanic and Atmospheric Administration, National Climatic Center, Asheville, North Carolina.

21. "Climatological Data - Annual Summary - New England", Volume 90, No. 13, National Oceanic and Atmospheric Administration, National Climatic Center, Asheville, North Carolina.

NOT AVAILABLE AT THIS TIME

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