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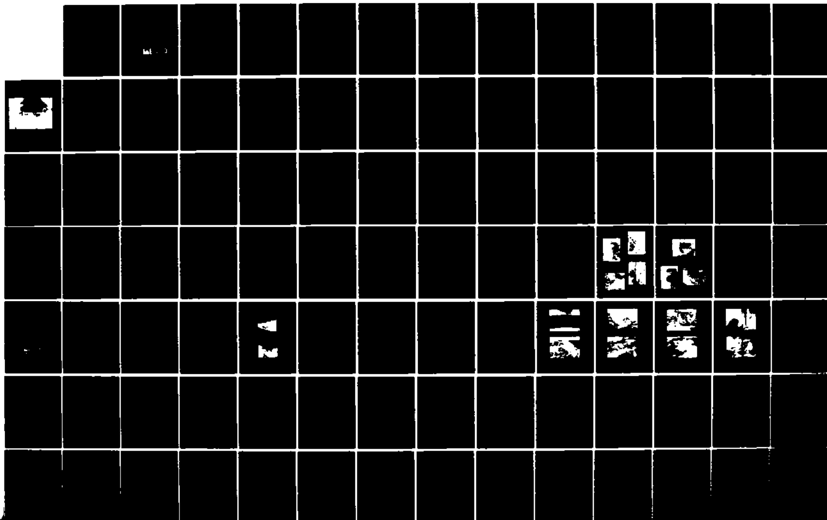

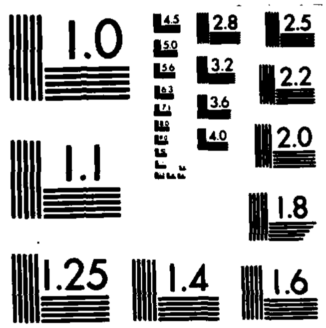

MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

CONNECTICUT RIVER BASIN

WASHINGTON, NEW HAMPSHIRE

BUTTERFIELD POND DAM

NH 00233

NHWRB NO. 245.01

PHASE I INSPECTION REPORT

NATIONAL DAM INSPECTION PROGRAM

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

MARCH 1980

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

DAMS, INSPECTION, DAM SAFETY,

Connecticut River Basin,

Washington, New Hampshire, and

Ashuelot River.

(Fr. p.ii)

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

The dam is a stone filled gravity structure about 210 ft. long and 12.5 ft. high. The dam is ocnsidered to be in very poor condition. It is small in size with a significant hazard potential. The recommendations and remedial measure should be implemented as soon as possible. [illegible]

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

REPLY TO
ATTENTION OF:

NEDED

7 OCT 198[illegible]

Honorable Hugh J. Gallen
Governor of the State of New Hampshire
State House
Concord, New Hampshire 03301

Dear Governor Gallen:

Inclosed is a copy of the Butterfield Pond Dam 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.

Butterfield Pond Dam has been rated as being in very poor condition. The brief assessment and Section 3 of this report contain a discussion as to the condition of the dam. 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 Water Resources Board, the cooperating agency for the State of New Hampshire. In addition, a copy of the report has also been furnished the owner, The State of New Hampshire Resource and Economic Development Dept., Division of Parks and Recreation.

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 Water Resources Board for your cooperation in carrying out this program.

Sincerely,

Max B. Scheider

MAX B. SCHEIDER
Colonel, Corps of Engineers
Division Engineer

Incl
As stated

BUTTERFIELD POND DAM
NH 00233
NHWRB 245.01

CONNECTICUT RIVER BASIN
WASHINGTON, NEW HAMPSHIRE

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

NATIONAL DAM INSPECTION PROGRAM
PHASE I - INSPECTION REPORT
BRIEF ASSESSMENT

Identification No:	NH 00233
Name of Dam:	Butterfield Pond Dam
Town:	Washington
County and State:	Sullivan, New Hampshire
Stream:	Ashuelot River
Date of Inspection:	December 6, 1979

Butterfield Pond Dam is a stone-filled gravity structure about 210 feet in overall length and 12.5 feet high from crest of dam to toe of slope. Located in the center of the dam is the principal overflow section which is 57 feet long and consists of a concrete capped, stone weir with concrete training walls. Near the middle of the overflow section is a 16.2 feet wide by 0.2 feet deep low flow spillway weir cast into the concrete cap. Located at the right training wall of the overflow section is the outlet structure which consists of a reinforced concrete sluice gate structure containing a wood plank sluice gate. Both the left and right embankments consist of unmortared stone. There is no emergency spillway.

The dam impounds Butterfield Pond and adjoining May Pond and the discharge flows through the Ashuelot River in a southwesterly direction approximately 6.0 miles to Ashuelot Pond. The original purpose of the dam is reported to have been to supply power to a mill, but its present use is recreational. The pond is 1.25 miles in length with a surface area of about 126 acres. The maximum storage capacity is about 590 acre feet. to 1473

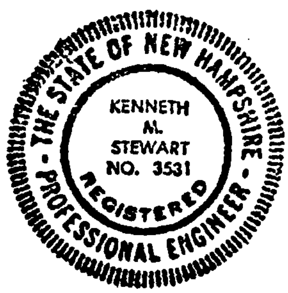
As a result of the visual inspection of this facility, the dam is considered to be in VERY POOR condition. Major concerns are: a sinkhole in the earthfill on the upstream side of the right stone embankment with pond water flowing into the sinkhole; subsidence of the crest and bulging of the downstream slope of the left stone embankment; severely broken and eroded condition of the concrete cap and the downstream concrete facing of the overflow section; and significant leakage and seepage at numerous locations along the downstream face of the dam.

This dam is classified as SMALL in size and a SIGNIFICANT hazard structure in accordance with the recommended guidelines established by the Corps of Engineers. The test flood for this dam therefore, ranges from a 100-year flood to one-half the Probable Maximum Flood (1/2 PMF). Due to the very poor condition of the dam, the 1/2 PMF was selected for this hydrologic analysis. The test flood inflow was estimated to be 7,500 cfs and resulted in a routed test flood outflow equal to 5,430 cfs which would overtop the dam crest by about 5.3 feet. The maximum spillway discharge capacity with the water level at the dam crest was estimated to be 160 cfs or about 3 percent of the routed test flood outflow. A major breach with the reservoir surface at the dam crest would overtop New Hampshire Route 31, by 2 to 3 feet, where it crosses the channel 350 feet below the dam. This could result in significant damage to the bridge and roadway. Although the potential for loss of life exists if the bridge were to wash out, no loss of life is anticipated.

It is recommended that the owner engage a qualified engineer to: investigate the sinkhole, crest subsidence, erosion channel on the downstream slope, and seepage at the left end of the right stone embankment; investigate the subsidence of the crest, sinkhole in the upstream earthfill, bulging of the downstream slope and seepage at the downstream toe of the left stone embankment; investigate the structural condition of the overflow section and the sluice gate; and do a detailed hydrologic-hydraulic investigation to assess further the potential of overtopping the dam, the adequacy of the spillway to pass the test flood, and the need for and means to increase project discharge capacity. It is also recommended that the owner clear brush and trees from a zone 25 feet wide on each side of the discharge channel between the dam and the highway bridge downstream of the dam.

The recommendations and remedial measures are described in Section 7 and should be addressed by the owner within one year after receipt of this Phase I Inspection Report.

Kenneth M. Stewart



Kenneth M. Stewart
Project Manager
N.H.P.E. 3531

S E A Consultants Inc.
Rochester, New Hampshire

This Phase I Inspection Report on Butterfield Pond 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.

ARAMAST MAHTESIAN, MEMBER
Geotechnical Engineering Branch
Engineering Division

CARNEY M. TERZIAN, MEMBER
Design Branch
Engineering Division

RICHARD DIBUONO, CHAIRMAN
Water Control 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 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, 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 trespassing 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.

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APPENDICES

OVERVIEW PHOTO - BUTTERFIELD POND DAM

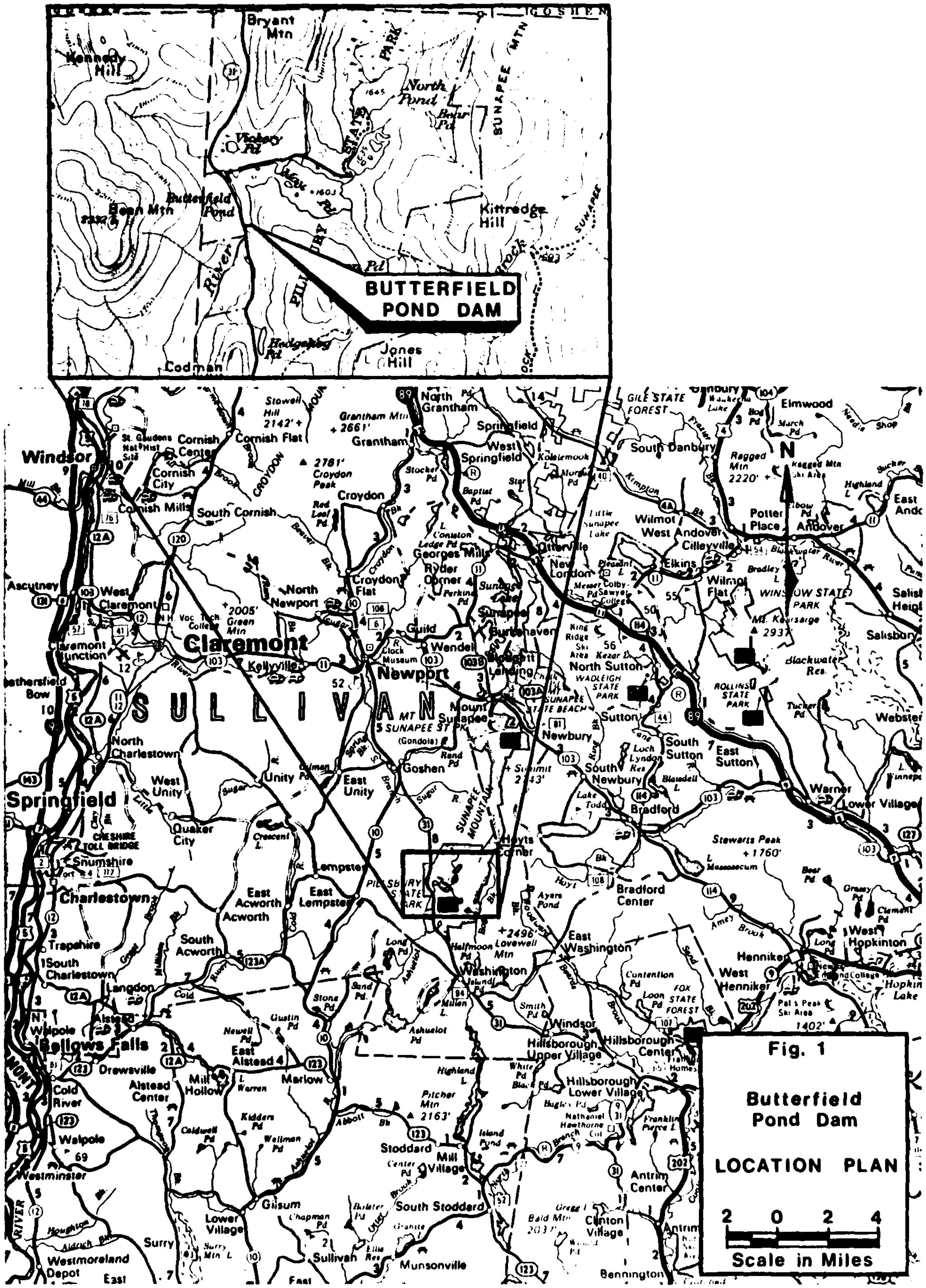


Fig. 1

Butterfield Pond Dam

LOCATION PLAN

NATIONAL DAM INSPECTION PROGRAM
PHASE I INSPECTION REPORT
BUTTERFIELD POND DAM

SECTION 1
PROJECT INFORMATION

1.1 General

a. Authority. 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. S E A Consultants Inc. has been retained by the New England Division to inspect and report on selected dams in the State of New Hampshire. Authorization and notice to proceed were issued to S E A Consultants Inc. under a letter of November 5, 1979 from William Hodgson, Jr., Colonel, Corps of Engineers. Contract No. DACW33-80-C-0008 has been assigned by the Corps of Engineers for this work.

b. Purpose

(1) To perform technical inspection and evaluation of non-federal dams to identify conditions which threaten the public safety and thus permit correction in a timely manner by non-federal interests.

(2) To encourage and prepare 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. The Butterfield Pond Dam is located in the town of Washington, New Hampshire, at the south end of Butterfield Pond, just east of New Hampshire Route 31. The dam impounds water from Butterfield Pond and adjoining May Pond which, after passing over the spillway, flows through the Ashuelot River in a southwesterly direction for approximately 6.0 miles where it discharges into Ashuelot Pond. The dam is shown on U.S.G.S. Quadrangle, Lovewell Mountain, New Hampshire, with coordinates approximately N43°13'33", W72°07'08", Sullivan County, New Hampshire. (See Location Plan)

b. Description of Dam and Appurtenances. Butterfield Pond Dam is a stone-filled gravity structure with a concrete capped overflow section and a reinforced concrete sluice gate structure. The dam is approximately 210 feet in overall length and 12.5 feet high from crest of dam to toe of slope. Both embankments consist of unmortared stone and have a crest width of approximately

6.0 feet. The left embankment has a downstream slope of unmortared stone which extends from crest of dam to toe of slope at approximately 1.5 feet vertical to 1.0 foot horizontal (1.5:1). The right embankment has a downstream slope of earthfill at approximately 1.0 foot vertical to 2.0 feet horizontal (1:2).

Located in the center of the dam is the principal overflow section which is 57 feet long and consists of a concrete capped, stone weir with concrete training walls. Near the middle of the overflow section is a 16.2 feet wide by 0.2 feet deep low flow spillway weir cast into the concrete cap.

Located at the right training wall of the overflow section is the outlet structure which consists of a reinforced concrete sluice gate structure containing a wood plank sluice gate. All mechanical equipment to operate the sluice gate has been removed and the sluice gate is split, leaking and inoperable. Flow passing through the sluice gate structure discharges into a 12 feet wide stone-lined sluiceway that extends approximately 56 feet to the main channel.

c. Size Classification. Small (height - 12.5 feet; storage - 590 acre-feet) based on storage (less than 1,000 acre-feet and greater than or equal to 50 acre-feet) as given in the Recommended Guidelines for Safety Inspection of Dams.

d. Hazard Classification. Significant Hazard. Failure of the dam could result in damage to a state bridge and highway (NH Route 31), since the capacity of the highway bridge is nearly 2,000 cfs less than the dam failure discharge and the roadway would be overtopped by 2 to 3 feet. There are no dwellings located near the downstream channel until the river discharges into Ashuelot Pond. However, at this point the stage would decrease rapidly to less than a foot and dwellings located on the pond would not be impacted. Although the potential for loss of life would exist if the bridge were to wash out, no loss of life is anticipated.

e. Ownership. No information regarding the original owner was found, but according to the files of the State of New Hampshire Water Resources Board, the original dam was built to create a pond and provide power for what was called Butterfield Mill. The dam was reconstructed in 1934 by the Civilian Conservation Corp., and at that time was owned by the State of New Hampshire Forestry Reservation. Since that time, the dam has always been owned by an agency of the State of New Hampshire, and is presently owned by Pillsbury State Park; more specifically, the State of New Hampshire Resources and Economic Development Department, Division of Parks and Recreation, Post Office Box 856, Concord, New Hampshire 03301. Telephone No. (603) 271-3254.

f. Operator. The dam is maintained and operated by Pillsbury State Park, under the State of New Hampshire Resources and Economic Development Department, Division of Parks and Recreation, Post Office Box 856, Concord, New Hampshire 03301. Telephone No. (603) 271-3254.

g. Purpose of Dam. The original purpose of the dam was to provide power to a mill. The present purpose of the dam is recreational.

h. Design and Construction History. No information regarding the original design or construction of the dam was found. Early records indicate that it was last rebuilt in 1934. A set of plans dated 1934, showing plan, elevation, and section of an existing structure and proposed reconstruction prepared by R. D. Chapin, Civil Engineer, Newport, New Hampshire, are on file at the State of New Hampshire Water Resources Board. None of the details shown on these plans are consistent with the configuration of the present structure. Photographs taken in 1937 that are on file substantially agree with the detail of the present structure.

i. Normal Operating Procedures. The Butterfield Pond Dam is used primarily to retain the waters of Butterfield Pond and adjoining May Pond for recreational use at Pillsbury State Park. There is no normal operating procedure for this dam.

1.3 Pertinent Data

a. Drainage Area. The drainage area above the Butterfield Pond Dam covers nearly 7.15 square miles (approximately 4576 acres), consisting of steeply sloped terrain surrounding Butterfield Pond and adjoining May Pond, and other smaller ponds located upstream from Butterfield Pond. The topography in the drainage basin ranges from 2332 feet (NGVD) on top of Bean Mountain to approximately 1592 feet (NGVD) at the base of the dam. The majority of the basin is heavily wooded and generally undeveloped. The development which does exist consists of structures associated with Pillsbury State Park.

b. Discharge at Damsite. Discharge at the damsite normally occurs over the overflow section located between the two concrete training walls. A 16.2 feet wide by 0.2 foot deep low flow spillway is located near the middle of the overflow section. The invert of the spillway weir is at elevation 1603.0 feet (NGVD) and has a capacity of nearly 4 cfs. A 6.1 foot wide by 6.05 foot high sluice gate is located adjacent to the right training wall. The sluice gate is normally closed, and presently is inoperable and leaking through a split in the gate. This gate, if operable, would allow the reservoir to be lowered to an elevation of 1595.2 feet.

(1) The capacity of the sluice gate was estimated to be 435 cfs with the water surface at the top of dam (elevation 1604.2 feet) and 595 cfs with the water surface at the test flood elevation (elevation 1609.5 feet).

(2) Maximum known flood at damsite - unknown

(3) The ungated spillway capacity with the water surface elevation at the top of the dam (elevation 1604.2 feet) was estimated to be 160 cfs

(4) The ungated spillway capacity with the water surface elevation at the test flood elevation (elevation 1609.5 feet) was estimated to be 2,375 cfs

(5) N/A

(6) N/A

(7) The total spillway capacity at the test flood elevation was estimated to be 2,375 cfs at 1609.5 elevation

(8) The total project discharge at the top of the dam was estimated to be 210 cfs at 1604.2 elevation (with the sluice gate closed) and 630 cfs at 1604.2 elevation (with the sluice gate open)

(9) The total project discharge at the test flood elevation was estimated to be 5,430 cfs at 1609.5 elevation

c. Elevation (feet, NGVD) based on elevation 1603.0 shown on U.S.G.S. quad sheet assumed to be pool elevation at top of permanent spillway crest

(1) Streambed at toe of dam - 1591.9

(2) Bottom of cutoff - unknown

(3) Maximum tailwater - unknown

(4) Recreation pool - 1603.2

(5) Full flood control pool - N/A

(6) Spillway crest - 1603.0

(7) Design surcharge (Original Design) - unknown

(8) Top of dam - 1604.2

(9) Test flood design surcharge - 1609.5

d. Reservoir (length in feet) (Butterfield Pond and adjoining May Pond)

(1) Normal pool - 6,565

(2) Flood control pool - N/A

(3) Spillway crest pool - 6,550

(4) Top of dam - 6,630

(5) Test flood pool - 6,990

e. Storage (acre-feet) (Butterfield Pond and adjoining May Pond)

(1) Normal pool - 465

(2) Flood control pool - N/A

(3) Spillway crest pool - 440

(4) Top of dam - 590

(5) Test flood pool - 1,415

f. Reservoir Surface (acres) (Butterfield Pond and adjoining May Pond)

(1) Normal pool - 126

(2) Flood control pool - N/A

(3) Spillway crest - 125

(4) Test flood pool - 175

(5) Top of dam - 134

g. Dam

(1) Type - stone-filled gravity structure with concrete capped overflow section

(2) Length - 210 feet overall

(3) Height - 12.5 feet maximum

(4) Top Width - 6.0 feet (at stone embankments)
8.0 feet (at overflow section)

(5) Side Slopes - 1.5 V to 1.0 H downstream slope (left embankment)
1.0 V to 2.0 H downstream slope (right embankment)

(6) Zoning - unknown

(7) Impervious core - unknown

(8) Cutoff - unknown

(9) Grout curtain - none

(10) Other - none

h. Diversion and Regulating Tunnel

Not applicable (see Section j below)

i. Spillway

(1) Type - stone fill, concrete capped overflow section with concrete training walls

(2) Length of weir - 57 feet (entire overflow section)
16.2 feet (low flow spillway section)

(3) Crest elevation - 1603.0 (invert low flow spillway)
1603.2 (invert main overflow section)

(4) Gates - N/A

(5) U/S Channel - The banks of Butterfield Pond and May Pond are tree lined. For the most part the slopes appear to be stable, although some debris has blocked the sluice gate. Other than the debris blocking the sluice gate, no evidence of significant sedimentation was observed.

(6) D/S Channel. The dam's overflow section discharges into a natural river channel (Ashuelot River) which is about 20 feet wide and 3.5 feet deep. Approximately 350 feet downstream from the dam, the river passes beneath a state highway (NH Route 31). The bridge opening (perpendicular to the centerline of the channel) measures 24.3 feet wide by 10.4 feet high. After passing through the bridge, the river travels in a southerly direction until it discharges into Ashuelot Pond, approximately 6 miles downstream from the dam.

j. Regulating Outlets

(1) Invert - Sluice gate - 1595.2 (bottom of gate opening)

(2) Size - Sluice gate - 6.1 feet wide x 6.05 feet high opening

(3) Description - Sluice gate - 5 inch thick wooden planks, 7.1 feet wide, bolted together to form gate in 6.1 feet wide opening

(4) Control Mechanism - Sluice gate - Manual crank lift by cables. Lifting mechanism removed. Gate planks split and leaking.

SECTION 2
ENGINEERING DATA

2.1 Design

No design data were disclosed for Butterfield Pond Dam. A set of plans dated 1934 showing plan, elevation, and section of an existing structure and proposed reconstruction of the dam by R.D. Chaplin, Civil Engineer, Newport, New Hampshire are on file at the State of New Hampshire Water Resources Board. None of the details shown on those plans were consistent with the configuration of the present structure.

2.2 Construction

No construction records were disclosed.

2.3 Operation

No engineering operational data were found.

2.4 Evaluation

a. Availability. No engineering data were available for Butterfield Pond Dam. A search of the files of the State of New Hampshire Water Resources Board revealed a limited amount of recorded information.

b. Adequacy. The final assessments and recommendations of this investigation are based on the visual inspection and the hydrologic and hydraulic calculations.

c. Validity. The field investigation indicated that the external features of the Butterfield Pond Dam almost completely disagree with the detail shown on the plans on file at the State of New Hampshire Water Resources Board.

SECTION 3
VISUAL INSPECTION

3.1 Findings

a. General. Butterfield Dam impounds a pond of small size. The drainage area above the dam consists of steeply sloped terrain surrounding Butterfield Pond and adjoining May Pond, and other smaller ponds located upstream from Butterfield Pond. The majority of the basin is heavily wooded and generally undeveloped. The development which does exist consists of structures associated with Pillsbury State Park. The downstream area is undeveloped except for the bridge crossing of NH State Route 31.

The field inspection of Butterfield Pond Dam was made on December 6, 1979. The inspection team consisted of personnel from S E A Consultants Inc. and Geotechnical Engineers, Inc. Inspection checklists, completed during the visual inspection, are included in Appendix A. At the time of inspection, water was passing approximately 2-1/2 inches deep over the 16.2 feet wide low flow spillway. The pool elevation was at approximately 1603.2 feet (NGVD). The upstream face of the dam could only be inspected above this water level.

b. Dam. Butterfield Pond Dam is a stone-filled gravity structure about 210 feet in overall length and 12.5 feet high from crest of dam to toe of slope. (See Plans and Detailing in Appendix B.)

The central portion of the dam consists of a stone-masonry overflow section about 57 feet long with concrete training walls and a stone weir, concrete capped on the crest and downstream side. (See Photo Nos. 2 and 7.) The crest of the overflow section is about 8 feet wide and the downstream face is vertical. (See Photo Nos. 3 and 7.) The upstream side of the overflow section is not completely visible beneath the water surface, but does indicate the existence of an unmortared stone apron. Located near the middle of the overflow section cast into the concrete cap is the low flow spillway which is 16.2 feet wide and 0.2 feet deep. (See Photo Nos. 7 and 8.) The concrete cap on the crest of the overflow section is broken and severely eroded at numerous locations. (See Photo Nos. 8 and 10.) At one location there is a small eddy where water is flowing down into a hole on the crest. Water is leaking from the downstream face at the contact between the overflow section and the foundation bedrock. (See Photo No. 9.) Major leakage is discharging from loose rocks at the toe of the right end of the overflow section.

Between the sluice gate at the right training wall of the overflow section and the right abutment, there is a stone embankment which appears to consist of a vertical dry-stone-masonry wall with earthfill against the upstream and downstream sides. In the fill immediately adjacent to the upstream side of the wall, there is a ditch in which water is flowing from the pond toward a sinkhole which is about 15 to 20 feet to the right of the concrete sluice gate structure. (See Plans and Details in Appendix B.) It appears that the water which flows into this sinkhole is discharging at the base of the downstream end of the right training wall of the sluice gate structure. (See Photo No. 13.) About 5 feet to the right of the concrete sluice gate structure, the crest of the stone embankment has subsided about 2 feet. Directly in line with this subsidence, there is an apparent erosion channel that extends from the crest to the toe of the downstream slope of the embankment. This channel is filled with weeds and brush, and there are stumps of some small trees in the channel. Some brush and one small tree are growing on the earthfill on the upstream side of the stone embankment. Brush and weeds are growing on the earthfill on the downstream side of the stone embankment. (See Photo No. 4.)

Between the left training wall of the overflow section and the left abutment there is a stone embankment which has a downstream slope inclined at about 1.5V:1H and which has an earthfill against its upstream side. There appears to be a major bulge in the downstream slope of this stone embankment close to the overflow section of the dam. (See Photo No. 5.) Major seepage is discharging at the toe of the stone embankment next to the overflow section. The crest of the stone embankment has settled about 1 to 1 1/2 feet within about 10 feet of the overflow section and the crest of the earthfill on the upstream side of the embankment has a sinkhole about 3 to 4 feet deep above pond level at a location about 25 feet to the left of the overflow section. (See Photo No. 6.) Brush and small trees are growing on the earthfill on the upstream side of the embankment. Brush and trees are growing at the downstream toe of the embankment. (See Photo No. 4.)

c. Appurtenant Structures. Located at the right training wall of the overflow section is the dam's outlet structure which consists of a reinforced concrete sluice gate structure that discharges into a 12 foot wide stone-lined sluiceway that extends approximately 56 feet to the main channel. (See Photo Nos. 10 and 11.) The sluice gate itself consists of 5-inch thick wood planks that are secured together by two long vertical bolts. The gate is approximately 6.1 feet wide and 6.05 feet high and is raised and lowered through steel slots embedded in the sides of the concrete sluice gate structure. Near the top of the gate, a severe crack has developed between the wood planks and water is pouring through and discharging into the sluiceway. (See Photo No. 12.)

A 6-inch thick concrete slab cast on top of the sluice gate structure acts as a control tower for the gate. The lifting mechanism has been removed, and the gate is jammed in the closed position. The upstream face of the left wall of the concrete sluice gate structure is being undermined and is deteriorated, exposing reinforcing steel.

d. Reservoir Area. The slopes of the ponds appear to be stable. No evidence of significant sedimentation was observed. The approach channel to the spillway is wide and unobstructed.

e. Downstream Channel. The dry-stone-masonry wall on the right side of the sluiceway downstream of the sluice gate structure is in poor condition. Some brush is growing in the channel downstream of the sluiceway. Some trees overhang the channel downstream of the overflow section of the dam, and one tree has blown over across the channel. (See Photo Nos. 14, 15 and 16.)

3.2 Evaluation

On the basis of the results of the visual inspection, Butterfield Pond Dam is considered to be in very poor condition.

A major sinkhole into which water from the pond is flowing on the upstream side of the stone embankment at the right end of the dam, subsidence of the crest of the right stone embankment, an apparent erosion channel on the downstream slope of the right embankment, and a major discharge of water from the base of the right training wall of the sluice gate structure are all signs of serious stability problems of the right embankment. It is possible that this embankment could fail at any time.

A major subsidence of the crest of the stone embankment at the left end of the dam, a major sinkhole in the earthfill on the upstream side of the left stone embankment, apparent bulging of the downstream slope of the left embankment, and a major discharge of water from the downstream toe of the left embankment are all signs of serious stability problems of the left embankment. It is possible that this embankment could fail at any time.

The broken and eroded condition of the concrete cap and downstream facing of the overflow section of the dam, leakage from cracks in the downstream facing, leakage at the contact between the overflow section of the dam and the bedrock foundation, and the flow of pond water into a hole on the crest of the overflow section are all signs of serious stability problems in the overflow section of the dam.

A large crack between the wood planks of the sluice gate and the water pouring through and discharging into the sluiceway, and the absence of any lifting mechanism are signs of considerable deterioration of the gate. It is possible that the gate could fail at any time.

Trees growing at the downstream toe of the dam, and brush which will eventually attain tree-size on the earthfills on the upstream side of the left stone embankment and on the upstream and downstream sides of the right stone embankment may lead to erosion and seepage problems if a tree blows over and pulls out its roots, or if a tree dies or is cut and its roots rot.

SECTION 4
OPERATIONAL AND MAINTENANCE PROCEDURES

4.1 Operational Procedures

a. General. The Butterfield Pond Dam is used primarily to retain the waters of Butterfield Pond and adjoining May Pond. There are no written or routine operational procedures.

b. Description of Any Warning System in Effect. No written warning system exists for the dam.

4.2 Maintenance Procedures

a. General. The owner, the New Hampshire Resources and Economic Development Department, Division of Parks and Recreation, is responsible for the maintenance of the dam. No formal plan for maintenance was discussed.

b. Operating Facilities. No formal plan for maintenance of operating facilities was disclosed.

4.3 Evaluation

The current operation and maintenance procedures for the Butterfield Pond Dam are inadequate to insure that all problems encountered can be remedied within a reasonable period of time. The owner should establish a written operation and maintenance procedure, as well as establish a warning system to follow in event of flood flow conditions or imminent dam failure.

SECTION 5
EVALUATION OF HYDROLOGIC/HYDRAULIC FEATURES

5.1 General. Butterfield Pond Dam is a stone-filled gravity structure approximately 210 feet in overall length and 12.5 feet high from crest of dam to toe of slope. Located near the center of the dam is the principal overflow section which is 57 feet long and consists of a concrete capped, stone weir with concrete training walls. Near the middle of the overflow section is a 16.2 feet wide by 0.2 foot deep low flow spillway weir. Adjacent to the right training wall of the overflow section is a 6.1 feet wide by 6.05 feet high sluice gate housed in a reinforced concrete structure. The sluice gate discharges into a 12 feet wide stone-lined sluiceway which extends approximately 56 feet to the main channel. At this time, the wooden plank sluice gate is inoperable and is severely leaking through a gap between two of the planks.

In addition to Butterfield Pond, five other ponds are located in the drainage area upstream from Butterfield Pond. Consequently, nearly two-thirds of the runoff from the watershed is intercepted by these ponds before flowing into Butterfield Pond.

5.2 Design Data. No hydrological or hydraulic design data were disclosed.

5.3 Experience Data. No experience data were disclosed. Maximum flood flows or elevations are unknown.

5.4 Test Flood Analysis. Due to the absence of detailed design and operational information, the hydrologic evaluation was performed utilizing data gathered during field inspection, watershed size and an estimated test flood determined from the Corps of Engineers guide curves. For this dam (small size and significant hazard) the test flood ranges from a 100-Year Flood to one-half the Probable Maximum Flood (1/2 PMF). Due to the very poor condition of the dam the 1/2 PMF was selected for this analysis. Since the drainage area consists of steeply sloping terrain, the "mountainous" curve, from the Corps of Engineers set of guide curves, was used to estimate the maximum probable peak flow rate.

Based on an estimated maximum probable flood peak flow rate of 2,100 cfs per square mile and a drainage area of 7.15 square miles, the test flood inflow was estimated to be 7,500 cfs. The test flood was routed through the reservoir in accordance with the Corps of Engineers procedure for Estimating Effect of Surcharge Storage on Maximum Probable Discharge. The routed test flood outflow was estimated to be 5,430 cfs. This analysis indicated that the dam crest would be overtopped by approximately 5.3 feet. The maximum spillway capacity (assuming that the sluice gate is closed) with the water level at the dam crest was estimated to be 160 cfs, which is only about 3 percent of the routed test flood outflow.

5.5 Dam Failure Analysis. The impact of dam failure with the reservoir surface at the dam crest was assessed utilizing the "Rule of Thumb" Guidance for Estimating Downstream Dam Failure Hydrographs published by the Corps of Engineers. The analysis covered a reach extending approximately 6 miles downstream to Ashuelot Pond. Based on this analysis, the Butterfield Pond Dam has been classified as a significant hazard.

Failure of the Butterfield Pond Dam would increase the stage along the immediate downstream channel by 7.5 feet, with an associated discharge of 5,950 cfs. Since this discharge exceeds the capacity of the highway bridge by nearly 2,000 cfs, it is probable that the pool formed by the flow restriction of the bridge would overtop the roadway by 2 to 3 feet and could cause significant damage to the bridge and roadway. The stage of the river would be reduced to about 4.5 feet by the time it discharges into Ashuelot Pond. The stage, however, would decrease rapidly, to less than a foot, as the flow passes through the wider portions of the pond. Although the potential for loss of life would exist if the bridge were to wash out, no loss of life is anticipated.

SECTION 6
EVALUATION OF STRUCTURAL STABILITY

6.1 Visual Observations

The visual inspection indicates the following potential structural problems:

(1) A major sinkhole into which water from the pond is flowing on the upstream side of the stone embankment at the right end of the dam, subsidence of the crest of the right stone embankment, an apparent erosion channel on the downstream slope of the right embankment, and a major discharge of water from the base of the right training wall of the sluice gate structure are all signs of serious stability problems in the right embankment. It is possible that this embankment could fail at any time.

(2) A major subsidence of the crest of the stone embankment at the left end of the dam, a major sinkhole in the earthfill on the upstream side of the left stone embankment, apparent bulging of the downstream slope of the left embankment, and a major discharge of water from the downstream toe of the left embankment are all signs of serious stability problems in the left embankment. It is possible that this embankment could fail at any time.

(3) The broken and eroded condition of the concrete cap and downstream facing of the overflow section of the dam, leakage from cracks in the downstream facing, leakage at the contact between the overflow section of the dam and the bedrock foundation, and the flow of pond water into a hole on the crest of the overflow section are all signs of serious stability problems in the overflow section of the dam.

(4) The large crack between the wood planks of the sluice gate and the water pouring through and discharging into the sluiceway, and the absence of any lifting mechanism are signs of considerable deterioration of the gate. It is possible that the gate could fail at any time.

(5) Trees growing at the downstream toe of the dam, and brush which will eventually attain tree-size on the earthfills on the upstream side of the left stone embankment and on the upstream and downstream sides of the right stone embankment, may lead to erosion and seepage problems if a tree blows over and pulls out its roots, or if a tree dies or is cut and its roots rot.

6.2 Design and Construction Data

No information regarding the original design or construction of the dam was found.

6.3 Post-Construction Changes

Early records indicate that the dam was rebuilt in 1934. A set of plans dated 1934, showing plan, elevation, and section of an existing structure and proposed reconstruction prepared by R.D. Chapin, Civil Engineer, Newport, New Hampshire, are on file at the New Hampshire Water Resources Board. None of the detail shown on these plans are consistent with the configuration of the present structure. Photographs taken in 1937 that are on file substantially agree with the detail of the present structure.

6.4 Seismic Stability

This dam is located in Seismic Zone 2 and, in accordance with the Phase I guidelines, does not warrant seismic analysis.

SECTION 7
ASSESSMENT, RECOMMENDATIONS, AND REMEDIAL MEASURES

7.1 Dam Assessment

a. Condition. The visual examination indicates that Butterfield Pond Dam is in very poor condition. The major concerns with respect to the integrity of the dam are:

(1) Sinkhole in the earthfill on the upstream side of the right stone embankment, with pond water flowing into the sinkhole.

(2) Major subsidence of the crest of the left stone embankment.

(3) Bulging of the downstream slope of the left stone embankment.

(4) Severely broken and eroded condition of the concrete cap and the downstream concrete facing of the overflow section.

(5) Leakage from cracks in the downstream facing of the overflow section and at the contact between the overflow section and the foundation bedrock.

(6) Subsidence of the crest of the right stone embankment.

(7) Erosion channel from the crest to downstream toe of the right embankment.

(8) Major seepage at the base of the right training wall of the sluice gate structure.

(9) Sinkhole above pond level in the earthfill on the upstream side of the left stone embankment.

(10) Major seepage at the downstream toe of the left embankment.

(11) Leakage through a large crack between the wood planks of the sluice gate.

(12) Trees overhanging the discharge channel downstream of the overflow section of the dam and one tree which has blown over across the channel.

(13) Inadequacy of the spillway to pass the test flood.

b. Adequacy of Information. The information available from the visual inspection is adequate to identify the problems that are listed in 7.2. These problems will require the attention of a qualified registered professional engineer who will have to make additional engineering studies to design or specify remedial measures. No additional information is needed for the purpose of this Phase I investigation.

c. Urgency. The owner should implement the recommendations in 7.2 and 7.3 immediately upon receipt of this Phase I report.

7.2 Recommendations

The owner should retain a registered professional engineer qualified in the design and construction of dams to:

(1) Investigate the sinkhole, crest subsidence, erosion channel on the downstream slope, and seepage at the left end of the right stone embankment, and design remedial measures as needed.

(2) Investigate the subsidence of the crest, sinkhole in the upstream earthfill, bulging of the downstream slope, and seepage at the downstream toe of the left stone embankment, and design remedial measures as need

(3) Investigate the structural condition of the overflow section and design remedial measures as needed.

(4) Investigate the structural condition of the sluice gate and design remedial measures as needed.

(5) Do a detailed hydrologic-hydraulic investigation to assess further the potential of overtopping the dam, the adequacy of the spillway to pass the test flood, and the need for and means to increase project discharge capacity.

The owner should carry out the recommendations made by the engineer.

7.3 Remedial Measures

a. Operating and Maintenance Procedures. The owner should:

(1) Clear brush and trees from a zone 25 feet wide on each side of the discharge channel between the dam and the highway bridge downstream of the dam.

(2) Visually inspect the dam and appurtenant structures once a month.

(3) Engage a registered professional engineer qualified in the design and construction of dams to make a comprehensive technical inspection of the dam once every year.

(4) Establish a surveillance program for use during and immediately after heavy rainfall, and also a warning program to follow in case of emergency conditions.

7.4 Alternatives

There are no practical alternatives to the recommendations of Section 7.2 and 7.3 except removal of the dam.

APPENDIX A

INSPECTION CHECKLIST

INSPECTION CHECK LIST

PARTY ORGANIZATION

PROJECT: Butterfield Pond Dam, NH

DATE: December 6, 1979

TIME: 9:00 a.m.

WEATHER: Cool, partly cloudy

W.S. ELEV. 1603.2 U.S. 1591.9 DN.S.
(NGVD)

PARTY:

1.	Kenneth Stewart, S E A	6.	Kenneth Stern, NHWRB
2.	Robert Durfee, S E A	7.	Richard DeBood, NHWRB
3.	Bruce Pierstorff, S E A	8.	
4.	Philip Ricardi, S E A	9.	
5.	Ronald Hirschfeld, GEI	10.	

	PROJECT FEATURE	INSPECTED BY	REMARKS
1.	Structural Stability	K. Stewart/R. Durfee	
2.	Hydrology/Hydraulics	B. Pierstorff/P. Ricardi	
3.	Soils and Geology	R. Hirschfeld	
4.			
5.			
6.			
7.			
8.			
9.			
10.			

INSPECTION CHECK LIST

PROJECT: Butterfield Pond Dam, NH

DATE: December 6, 1979

PROJECT FEATURE: Dam Embankment

NAME:

DISCIPLINE:

NAME:

AREA EVALUATED	CONDITIONS
DAM EMBANKMENT	
Crest Elevation	1603.0
Current Pool Elevation	1603.2
Maximum Impoundment to Date	Unknown
Surface Cracks	None observed
Pavement Condition	Not paved
Movement or Settlement of Crest	One sinkhole in crest to right of sluice gate structure, one sinkhole in crest near left end of overflow section
Lateral Movement	Bulging of downstream dry stone masonry wall between left end of overflow section and left abutment in vicinity of sinkhole on crest
Vertical Alignment	Sinkholes, as noted above
Horizontal Alignment	See "Lateral Movement" above
Condition at Abutment and at Concrete Structures	Fair
Indications of Movement of Structural Items on Slopes	None observed
Trespassing on Slopes	No evidence observed
Vegetation on Slopes	Brush and some small trees on upstream side of embankment, on abutments, and downstream of toe of dam
Sloughing or Erosion of Slopes or Abutments	Major erosion channel on downstream slope next to training wall on right side of sluiceway
Rock Slope Protection - Riprap Failures	No riprap
Unusual Movement or Cracking at or near Toe	None observed
Unusual Embankment or Downstream Seepage	Major seepages at several locations
Piping or Boils	None observed
Foundation Drainage Features	None observed
Toe Drains	None observed
Instrumentation System	None observed

INSPECTION CHECK LIST

PROJECT: Butterfield Pond Dam, NH DATE: December 6, 1979

PROJECT FEATURE: Dike Embankment NAME: ______

DISCIPLINE: ______ NAME: ______

AREA EVALUATED	CONDITIONS
DIKE EMBANKMENT	No dike
Crest Elevation	
Current Pool Elevation	
Maximum Impoundment to Date	
Surface Cracks	
Pavement Condition	
Movement or Settlement of Crest	
Lateral Movement	
Vertical Alignment	
Horizontal Alignment	
Condition at Abutment and at Concrete Structures	
Indications of Movement of Structural Items on Slopes	
Trespassing on Slopes	
Vegetation on Slopes	
Sloughing or Erosion of Slopes or Abutments	
Rock Slope Protection - Riprap Failures	
Unusual Movement or Cracking at or near Toe	
Unusual Embankment or Downstream Seepage	
Piping or Boils	
Foundation Drainage Features	
Toe Drains	
Instrumentation System	

INSPECTION CHECK LIST

PROJECT: Butterfield Pond Dam, NH   DATE: December 6, 1979

PROJECT FEATURE: Intake Channel   NAME: ______

DISCIPLINE: ______   NAME: ______

AREA EVALUATED	CONDITIONS
<u>OUTLET WORKS - INTAKE CHANNEL AND INTAKE STRUCTURE</u>	
a. Approach Channel	
Slope Conditions	Good
Bottom Conditions	Not visible beneath pond surface
Rock Slides or Falls	None
Log Boom	None
Debris	Debris built up against sluice gate
Condition of Concrete Lining	Loose stone lining
Drains or Weep Holes	None
b. Intake Structure	
Condition of Concrete	Fair to poor. Exposed reinforcing steel and numerous cracks.
Stop Logs and Slots	Wooden gate (not operable) split and leaking.

INSPECTION CHECK LIST

PROJECT: Butterfield Pond Dam, NH — DATE: December 6, 1979

PROJECT FEATURE: Control Tower — NAME: ______

DISCIPLINE: ______ — NAME: ______

AREA EVALUATED	CONDITIONS
OUTLET WORKS - CONTROL TOWER	
a. Concrete and Structural	
General Condition	Fair
Condition of Joints	Good
Spalling	Minor
Visible Reinforcing	Visible reinforcement on leading edge of both sides of intake channel
Rusting or Staining of Concrete	Minor
Any Seepage or Efflorescence	Minor
Joint Alignment	Good
Unusual Seepage or Leaks in Gate Chamber	None observed
Cracks	Numerous
Rusting or Corrosion of Steel	Rusting of visible reinforcing steel
b. Mechanical and Electrical	
Air Vents	Not applicable
Float Wells	Not applicable
Crane Hoist	None
Elevator	Not applicable
Hydraulic System	Not applicable
Service Gates	Not accessible - water pouring through apparent split in wooden gate
Emergency Gates	Same as service gates
Lightning Protection System	Not applicable
Emergency Power System	Not applicable
Wiring and Lighting System	Not applicable

INSPECTION CHECK LIST

PROJECT: Butterfield Pond Dam, NH DATE: December 6, 1979

PROJECT FEATURE: Transition and Conduit NAME: ____

DISCIPLINE: ____ NAME: ____

AREA EVALUATED	CONDITIONS
<u>OUTLET WORKS - TRANSITION AND CONDUIT</u>	Not applicable
General Condition of Concrete	
Rust or Staining on Concrete	
Spalling	
Erosion or Cavitation	
Cracking	
Alignment of Monoliths	
Alignment of Joints	
Numbering of Monoliths	

INSPECTION CHECK LIST

PROJECT: Butterfield Pond Dam, NH DATE: December 6, 1979

PROJECT FEATURE: Outlet Structure NAME:

DISCIPLINE: NAME:

AREA EVALUATED	CONDITIONS
OUTLET WORKS - OUTLET STRUCTURE AND OUTLET CHANNEL	
General Condition of Concrete	Fair
Rust or Staining	Minor
Spalling	Minor
Erosion or Cavitation	Both wing walls undermined and eroded. (Left side more serious.)
Visible Reinforcing	None observed
Any Seepage or Efflorescence	Some efflorescence
Condition at Joints	Cracking at lift boundaries
Drain holes	None
Channel	
Loose Rock or Trees Overhanging Channel	Trees overhanging channel. Dry stone masonry wall on the right side of the sluiceway channel is in poor condition.
Condition of Discharge Channel	Fair

INSPECTION CHECK LIST

PROJECT: Butterfield Pond Dam, NH   DATE: December 6, 1979

PROJECT FEATURE: Spillway Weir   NAME:

DISCIPLINE: NAME:

AREA EVALUATED	CONDITIONS
<u>OUTLET WORKS - SPILLWAY WEIR, APPROACH AND DISCHARGE CHANNELS</u>	
a. Approach Channel	
General Condition	Good
Loose Rock Overhanging Channel	None observed
Trees Overhanging Channel	None observed
Floor of Approach Channel	Not visible beneath pond surface
b. Weir and Training Walls	
General Condition of Concrete	Extensively deteriorated
Rust or Staining	None observed
Spalling	Large sections of concrete cap broken away
Any Visible Reinforcing	None
Any Seepage or Efflorescence	Extensive seepage
Drain Holes	None
c. Discharge Channel	
General Condition	Fair
Loose Rock Overhanging Channel	None observed
Trees Overhanging Channel	Trees in channel and overhanging channel
Floor of Channel	Boulder-covered
Other Obstructions	One tree has fallen across channel

INSPECTION CHECK LIST

PROJECT: Butterfield Pond Dam, NH

DATE: December 6, 1979

PROJECT FEATURE: Service Bridge

NAME:

DISCIPLINE:

NAME:

AREA EVALUATED	CONDITIONS
OUTLET WORKS - SERVICE BRIDGE	No service bridge
a. Super Structure	
Bearings	
Anchor Bolts	
Bridge Seat	
Longitudinal Members	
Under Side 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 B

ENGINEERING DATA

AVAILABLE ENGINEERING DATA

No Engineering Data other than past inspection reports from the State of New Hampshire Water Resource Board were available.

PAST INSPECTION REPORTS

MEMO

December 7, 1979

To: Vern Knowlton

From: Ken Stern K

Re: Corps Inspection of May Pond Dam

(Butterfield Dam) 245.01, Washington

On December 6, 1979 I accompanied the inspection team from SEA Consultants. This dam is in poor condition. The concrete is extremely deteriorated, the rock abutments have settled, there is major leakage at several locations.

The only structure threatened should the dam fail is a highway bridge on state route 31. This bridge has a large clear opening. If the dam were to fail gradually there may be no damage to the bridge.

I discussed the dam with Gary, who has been there, and we agree that major reconstruction is needed. Once work is considered, total reconstruction may be inevitable.

I recommend that the stoplogs be removed and the pond lowered until remedial action is taken. This would reduce the hydraulic pressure on the dam and reduce the amount of water discharged should the dam fail. The lowered water level would redistribute the location and magnitude of the ice pressures on the structure.

I recommend lowering the pond. A decision should be made and action taken now.

KS/ln

May Pond, Dam No. 245.01, Washington, New Hampshire

This is a stone-fill, gravity with concrete abutments and spillway cap dam type structure. It is approximately 225' long and with a maximum height of 18'. The present structure has several serious leaks through the stonework, and cracks and holes in the concrete. It also contains a pond drain gate near the base of the dam which also leaks. The present configuration of the dam and spillway does not permit the passage of the estimated 100 year flood flow (1,450cfs) without the dam being overtopped. The Board's proposal includes work to stop the leakage and increase the discharge capacity to equal the 100 year flood flows.

The proposal incorporates constructing an access road, removing the leaking spillway stones and concrete cap, the leaking gate section abutments, constructing steel reinforced concrete face walls and abutments, and a new concrete spillway with flashboards. This will require the removal of accumulated silt and debris from the upstream side of the dam. The project also includes constructing a stoplog section to act as a pond drain which may require some channel excavation to improve the hydraulics of the downstream channel.

The attached cost estimate reflects the materials of construction and labor costs of this proposal to be constructed not later than the end of 1980.

MAY POND DAM (#245.01)

WASHINGTON, NEW HAMPSHIRE

1. Dam originally constructed to create a mill pond, but now used to maintain a recreation pond for users of Pillsbury State Park

2. Pond area - 103 Acres

3. Ratio of net drainage to pond area - 37:1

4. 100 year flood flows - 1450 cfs

5. Shoreline - 3± Miles

6. Altitude - 1632 feet

7. Watershed - Connecticut

8. River system - Ashuelot River

9. Inlets - Ashuelot River

10. Color of water - colorless

11. Ownership- State, Division of Parks

MAY POND DAM (#245.01)

PILLSBURY STATE PARK

WASHINGTON, N. H.

At the present time, the dam on May Pond does not have capacity to flow the 100 year storm frequency flow without overtopping the dam. The present design standard requires dams to pass storms equal to 100 year frequency flood flows. The dam also has several serious leaks through the stonework.

The design for this project includes:

1. Removing existing spillway and construction a permanent concrete crest with automatic flashboards.
2. Stoplog section construction.
3. New concrete abutments and cut-off and upstream face walls to prevent the leak which is now occurring.

The follwing is a cost estimate:

1.	Access Road	$ 8,000.00
2.	Remove cut brush and grass	2,000.00
3.	Remove existing stone spillway, debres and silt	10,000.00
4.	Concrete, reinforcements, etc. (200 cy)	70,000.00
5.	Stoplog construction	8,000.00
6.	Backfill & clean-up	6,000.00
	SUB-TOTAL	$104,000.00
	20% Engineering & Contingencies	20,800.00
	TOTAL	$124,800.00
	ROUNDED TOTAL	$125,000.00

2/7/79

M E M O

TO: Vernon A. Knowlton
Chief Engineer

DATE: August 16, 1978

FROM: Gary Kerr
Water Resources Engineer

SUBJECT: Dam Inspection #245.01 - Report of Leakage

DATE OF INSPECTION: August 14, 1978

Via a letter from the S.C.S. office in Claremont I was instructed to reinspect the subject dam for a serious leak. Below are listed my observations and please refer to the accompaning photos and file for clarity.

1. Dam is founded on ledge and consists of piled stones embankment and spillway with a concrete cap.

2. A stoplog section with concrete abutments.

3. Serious leakage occuring thru the structure at several places:
   a. Right embankment
   b. Left side pier of the stoplog section
   c. Thru the hole in the spillway cap

This structure is listed as a menace dam because of the pondage (approx. 103 acres), DA, of 7.3 mi^2, and downstream development.

The Ashuelot River flows thru May Pond, under Route 31 and into Ashuelot Pond, Washington. There is considerable domestic development around Ashuelot Pond and points South.

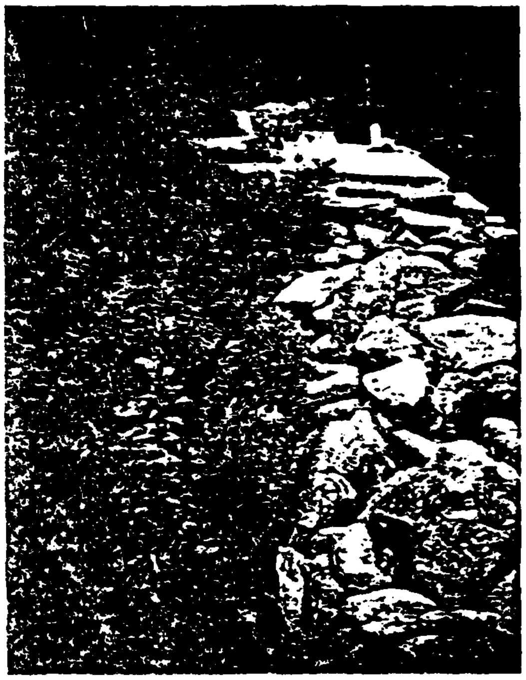
In F.C.M.'s inspection report of 1971, he states that the dam's flood capacity is sufficient to pass the estimated 100 flood with 1/2' of freeboard and no gate (now stoplogs) operation. He also indicated "it appeared (through openings in the snow) to be well built, substanial, and water tight." Unfortunately now, these assumptions are not entirely true. The dam has deteriorated, rocks have moved, the spillway cap is broken and the structure does leak seriously.

I strongly suggest that the pond be lowered or the dam sealed, sufficiently enough to stop the leakage thru the embankment and spillway cap. This may require a drawdown of 2-3', and since we are approching the hurricane season, the drawdown would redue the protential flooding of a full pond plus runoff from the storm should the dam fail.

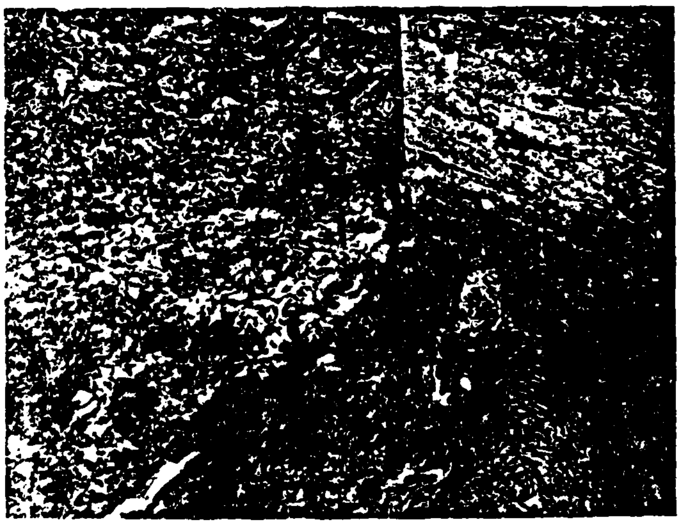
CLK/kn

MAY POND DAM #245.01

LIGHT STONE EMBANKMENT
ARROWS DENOTE FLOW PATHS

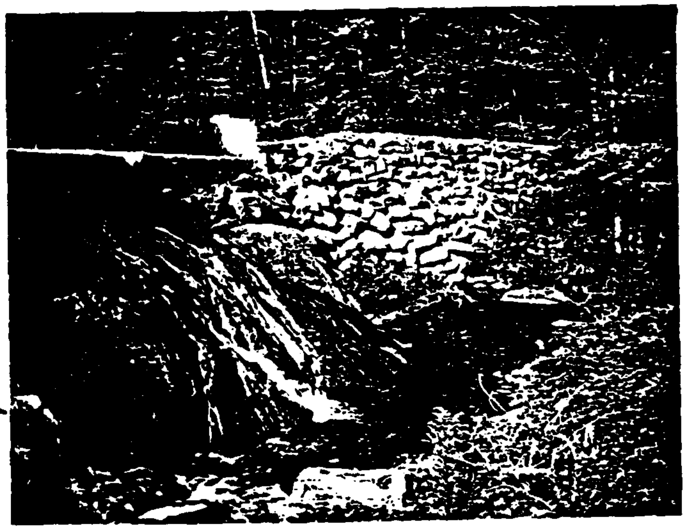


ENTRANCE PATH



EXIT AT BASE OF STONE EMBANKMENT NEAR STOPLOG SECTION

MAY POND DAM
245.01



LEAKAGE PROBABLY
FROM STOPLOG
SECTION

HOLE IN SPILLWAY
(CONCRETE CAP)

EXIT FOR HOLE
IN CAP.

STATE OF NEW HAMPSHIRE

INTER-DEPARTMENT COMMUNICATION

DATE August 16, 1978

FROM George M. McGee, Sr.
Chairman

AT (OFFICE) Water Resources Board

SUBJECT Leakage through dam #245.01 at May Pond

TO Theodore Natti, Director
Division of Resources Development

This office has been alerted to the fact that your"Butterfield Dam" (245.01), at May Pond, Pillsbury State Park is leaking quite badly. An engineer re-inspected this dam and filed his report. Please be aware that this dam was inspected, per your request, in September, 1975 and a copy of the suggested repairs was sent to you. The inspector noted that none of the suggested repairs, short term or otherwise, were implamented and now the dam condition has deteriorated seriously.

As a result of this inspection the following items require your immediate attention:

1. The right hand piled stone embankment (looking downstream) no longer acts as a pond retaining structure as water freely flows through it.
2. This same embankment appears to have sloughed, to the extent that it no longer retains the shape of a stone wall with vertical sides.
3. Because of the present pond elevation and erosion on the upstream side of this embankment, leakage is occuring through the right hand embankment (please see photos).
4. Leakage is also freely flowing through the enlarged hole in the concrete spillway cap (please see photo).
5. Leakage is also evident adjacent to the left hand pier for the stoplog section on the downstream side of the spillway.

All of the above constitute a hazardous condition and threatens the stability of the dam and as such require corrective action.

Because this dam is a menace structure, we require that you send us a schedule of your proposed repairs within 30 days. We do suggest that you reduce the pond level 2-3 feet, or more, effectively immediately and remain lowered until your repairs are completed, or the causes of the leakage eliminated.

If you have any questions, please contact us.

Sincerely yours,

George M. McGee, Sr.

GMMG/GK/kn
Enclosure

N. H. WATER RESOURCES BOARD
Concord, N. H. 03301

DAM SAFETY INSPECTION REPORT FORM

Town: WASHINGTON Dam Number: 245.01

Inspected by: GARY L. KERR Date: 30 Sep 1975

Local name of dam or water body: MAY POND

Owner: PILLSBURY STATE PARK Address:

Owner ~~was~~/was not interviewed during inspection.

Drainage Area: 6.01 sq. mi. Stream: ASHUELOT RIVER

Pond Area: 103 (F&G Book) Acre, Storage ______ Ac-Ft. Max. Head ______ Ft.

Foundation: Type LEDGE & ROCKS, Seepage present at toe - ~~Yes~~/No, No

Spillway: Type BROAD CRESTED CONCRETE CAP, Freeboard over perm. crest: 2'±,

Width 40'±, Flashboard height 3'± MAX,

Max. Capacity ______ c.f.s.

Embankment: Type ROCK, Cover ROCKS Width ~~5'~~ 6',

Upstream slope VERT to 1; Downstream slope VERT to 1

Abutments: Type CONCRETE, Condition: ~~Good~~ Fair ~~Poor~~

Gates or Pond Drain: Size 5' LONG Capacity ______ Type STOP LOGS

Lifting apparatus NONE Operational condition YES w/EQUIP

Changes since construction or last inspection:

Downstream development:

This dam would/~~would not~~ be a menace if it failed. WASH OUT RD

Suggested reinspection date:

Remarks: HOLE IN CONCRETE CAP OF SPILLWAY

MINOR EROSION OF ABUTMENTS

APPROX 3" FLOW OVER SPILLWAY

0/LMAC

DATE: February 8, 1971

FROM: Francis C. Moore, P.E.
Water Resources Engineer

SUBJECT: May Pond at Pillsbury State Park, Washington - #254.01

TO: Vernon A. Knowlton
Chief Water Resources Engineer

On January 25, 1971, I inspected the dam called Butterfield dam that flows back into May Pond in Washington at the head-waters of Ashuelot River. This dam was well rebuilt by C.C.C. forces in 1934, consisting of rock fill dam with concrete capped spillway.

The capacity of the spillway with 1/2 foot freeboard and no overflow through a 6' x 6' gate is once in 100 years. The dam appears, through openings in the snow, to be well built, substantial and water tight. However, there may be some trees to be removed from dam ($500.), some concrete patching ($3,500.) and miscellaneous work ($1,000.) totaling about $5,000. upon inspection at a later time.

Two views are shown in photos taken at this dam. The gate section could be opened to lower the pond if necessary. This dam is near N. H. H. W. Route #31 midway between Washington and Goshen on the northeast side of the highway.

As seen in the photos, water was going an estimated 4" over the spillway.

FCM/jb

NEW HAMPSHIRE WATER CONTROL COMMISSION
DATA ON DAMS IN NEW HAMPSHIRE

LOCATION STATE NO. 245.01 ✓

Town Washington ✓ : County Washington ✓

Stream May Pond ✓

Basin-Primary Conn. R. ✓ : Secondary Ashuelot R. ✓

Local Name Old Butterfield Mill ✓

Coordinates—Lat. : Long.

GENERAL DATA

Drainage area: Controlled Sq. Mi.: Uncontrolled Sq. Mi.: Total 6.01 ✓ Sq. Mi.

Overall length of dam 225 ✓ ft.: Date of Construction rebuilt 1934 ✓

Height: Stream bed to highest elev 18 ✓ ft.: Max. Structure 15.5 ✓ 15.25 ✓ ft.

Cost—Dam : Reservoir

DESCRIPTION Rockfill-Concrete cap spillway, stone&timber ✓

Waste Gates

Type concrete ✓

Number 1 : Size 6 ✓ ft. high x 6 ✓ ft. wide

Elevation Invert 8.5 ✓ : Total Area 36 ✓ sq. ft.

Hoist

Waste Gates Conduit

Number : Materials

Size ft.: Length ft.: Area sq. ft.

Embankment

Type

Height—Max. ft.: Min. ft.

Top—Width : Elev. ft.

Slopes—Upstream on : Downstream on

Length—Right of Spillway : Left of Spillway

Spillway

Materials of Construction ✓ concrete cap

Length—Total 47.5' high 17.5' low ✓ ft.: Net ft.

Height of permanent section—Max. 15.5 ✓ ft.: Min. 15.25 ✓ ft.

Flashboards—Type : Height ft.

Elevation—Permanent Crest : Top of Flashboard

Flood Capacity cfs.: cfs/sq. mi.

Abutments

Materials:

Freeboard: Max. 2.75 ft.: Min. 2.5 ✓ ft.

Headworks to Power Devel.—(See "Data on Power Development")

OWNER N.H. Forestry Reservation ✓

REMARKS Additional spillway over gate 6' wide, same elevation. ✓
Use-Recreation. Good Condition

Tabulation By RLT B-13 Date 9/22/39

NEW HAMPSHIRE WATER RESOURCES BOARD

INVENTORY OF DAMS AND WATER POWER DEVELOPMENTS

DAM

BASIN Connecticut N°. 245.01

RIVER May Pond MILES FROM MOUTH ____ D.A.SQ.MI 6.01

TOWN Washington OWNER N.H. Forestry Reservation

LOCAL NAME OF DAM Old Butterfield Mill

BUILT rebuilt 1934 DESCRIPTION ~~Stone planked upstream Concrete gate structure~~ ~~+ spillway, Ledge foundation~~ Rock fill - concrete cap spillway Stone, timber + concrete

POND AREA-ACRES 102.65 PA DRAWDOWN FT. ____ POND CAPACITY-ACRE FT. ____

HEIGHT-TOP TO BED OF STREAM-FT. 18 MAX. ____ MIN. ____

OVERALL LENGTH OF DAM-FT. 225' MAX.FLOOD HEIGHT ABOVE CREST-FT. ____

PERMANENT CREST ELEV.U.S.G.S. ____ LOCAL GAGE ____

TAILWATER ELEV.U.S.G.S. ____ LOCAL GAGE ____

SPILLWAY LENGTHS-FT. 47.5 high 17.5 low FREEBOARD-FT. 2.5 and 2.75

FLASHBOARDS-TYPE, HEIGHT ABOVE CREST ____

WASTE GATES-NO.	WIDTH	MAX.OPENING	DEPTH SILL BELOW CREST
1	6	6	8.5

REMARKS Condition ~~ruins psc~~ good. Dam has been rebuilt probably WPA project

I3 into Ashuelot River.

POWER DEVELOPMENT

Coordinates Lat. 43° 13.535'
Long 72° 7.213'

UNITS NO.	RATED HP	HEAD FEET	C.F.S. FULL GATE	KW	MAKE

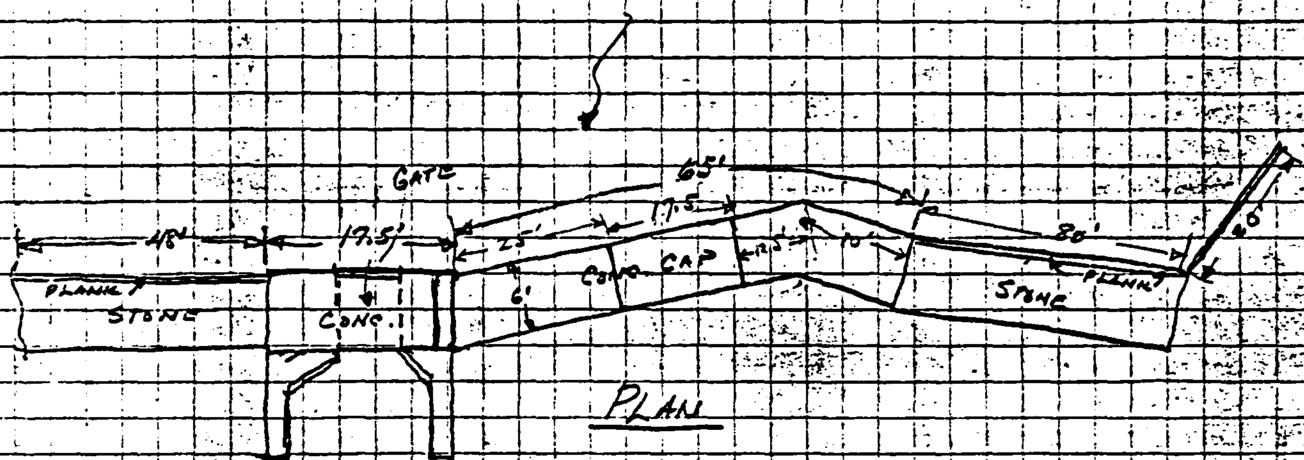
USE Once power for mill now recreation

REMARKS Additional spillway over gate 6' wide. same elevation P.S.C. says ruins 1922.

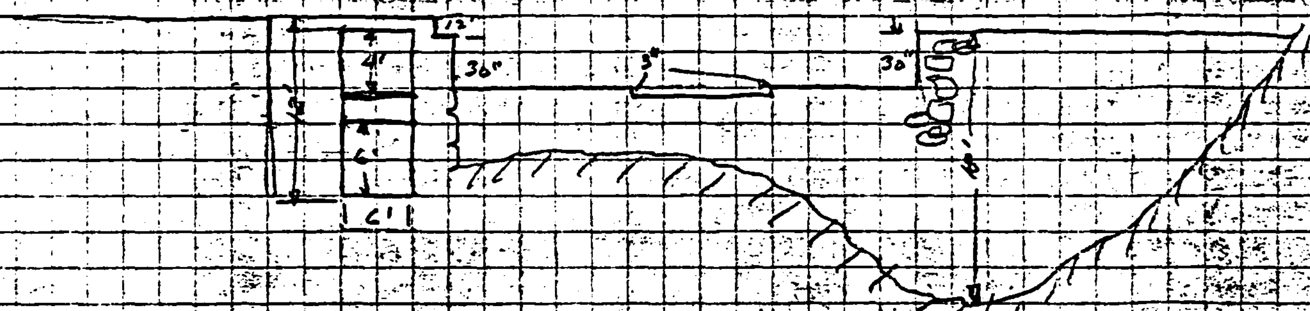
DATE 9/28/37 H & S.H.S.

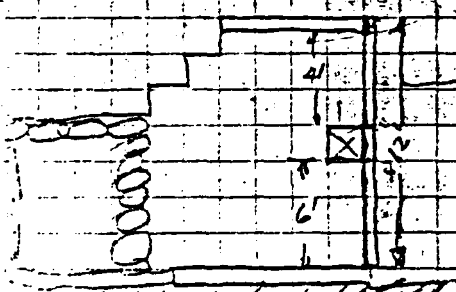
MAY POND DAM – WASHINGTON 9/28/37 245.01

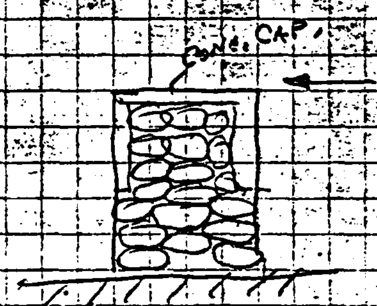
STATE FORESTRY



PLAN



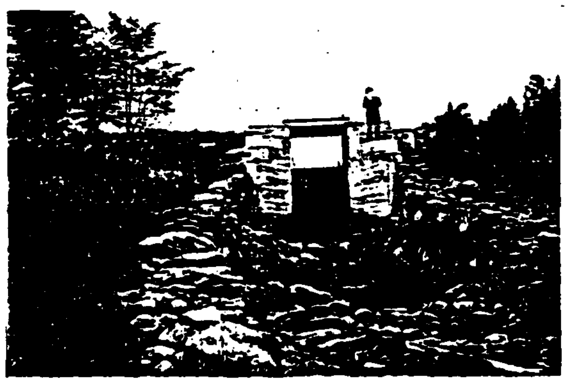




X-SECTION SPILL

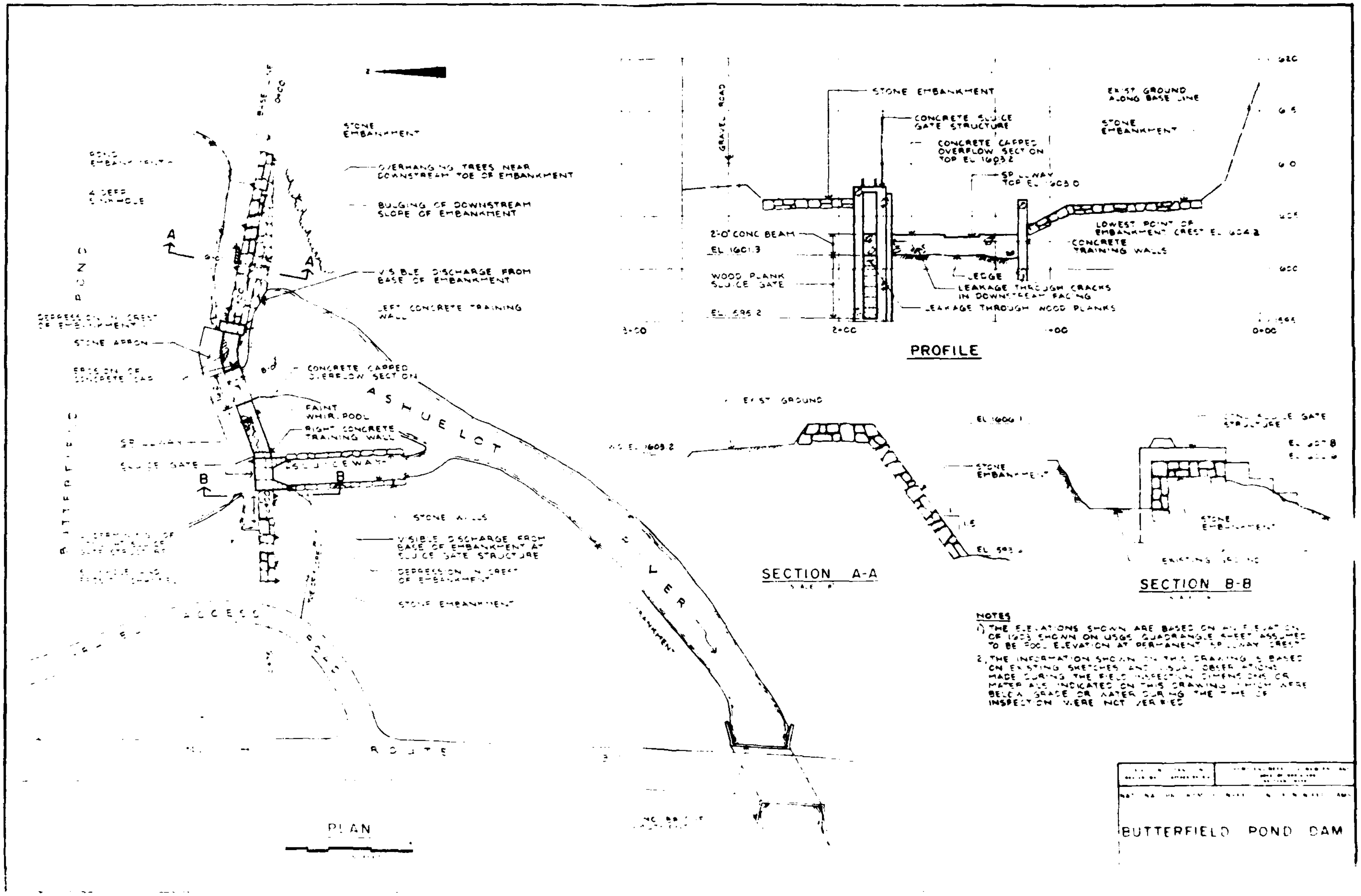
X-SECTION GATE

MAY POND IN WASHINGTON
N. H. Forestry Dept.
September 28,1937



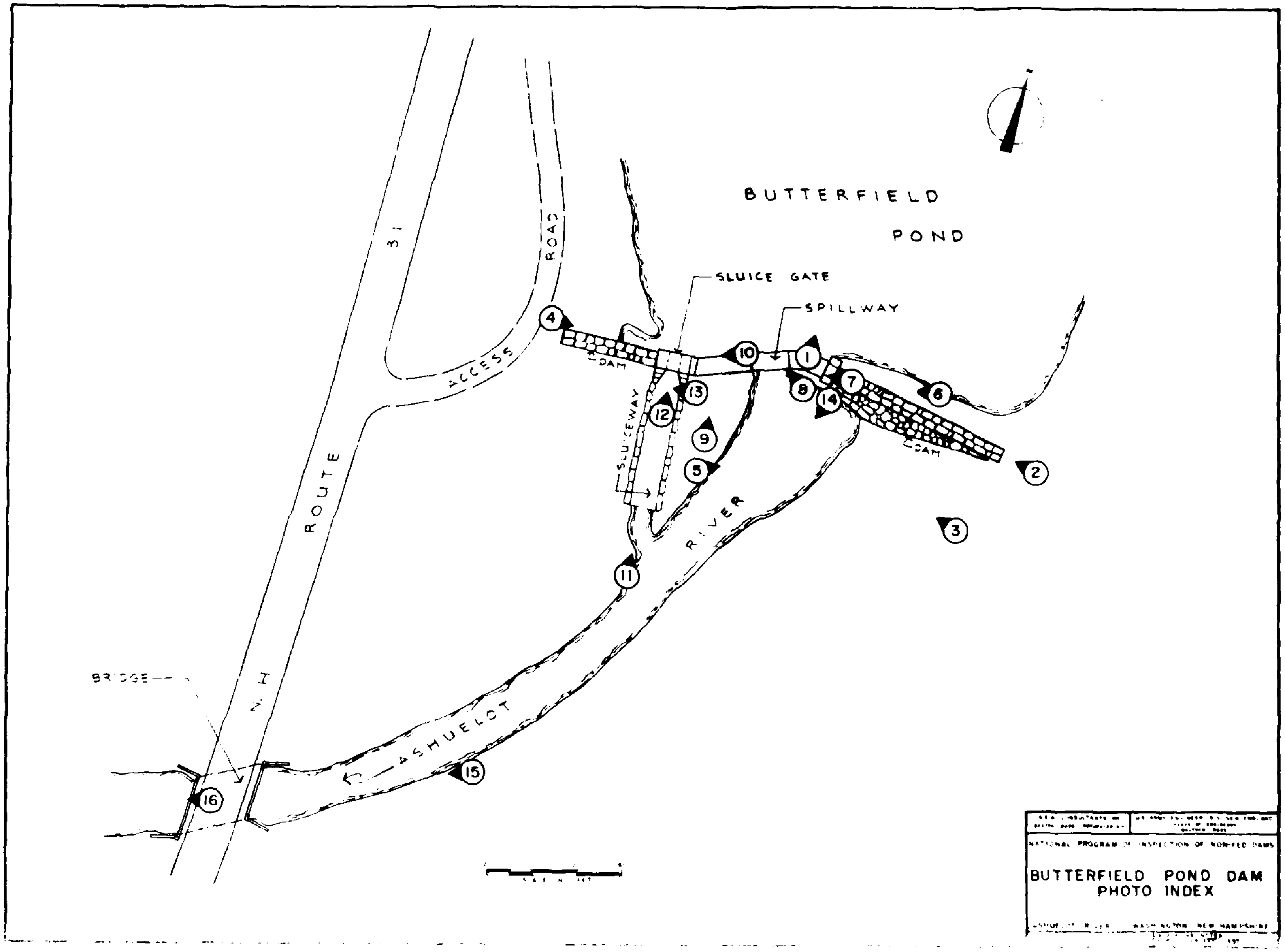
Gate

PLANS AND DETAILS


PLAN
BUTTERFIELD POND
ASHUELOT RIVER
ACCESS ROUTE
BASE LINE
0+00
STONE EMBANKMENT
OVERHANGING TREES NEAR DOWNSTREAM TOE OF EMBANKMENT
BULGING OF DOWNSTREAM SLOPE OF EMBANKMENT
VISIBLE DISCHARGE FROM BASE OF EMBANKMENT
LEFT CONCRETE TRAINING WALL
DEPRESSION IN CREST OF EMBANKMENT
STONE APRON
CONCRETE CAPPED OVERFLOW SECTION
FAINT WHIRLPOOL
RIGHT CONCRETE TRAINING WALL
SPILLWAY
SLUICE GATE
SLUICEWAY
STONE WALLS
VISIBLE DISCHARGE FROM BASE OF EMBANKMENT AT SLUICE GATE STRUCTURE
DEPRESSION IN CREST OF EMBANKMENT
STONE EMBANKMENT
PROFILE
GRAVEL ROAD
STONE EMBANKMENT
CONCRETE SLUICE GATE STRUCTURE
CONCRETE CAPPED OVERFLOW SECTION TOP EL 1603.2
SPILLWAY TOP EL 1603.0
EXIST. GROUND ALONG BASE LINE
STONE EMBANKMENT
LOWEST POINT OF EMBANKMENT CREST EL 1604.2
CONCRETE TRAINING WALLS
2'-0" CONC BEAM
EL 1601.3
WOOD PLANK SLUICE GATE
EL 1595.2
LEDGE
LEAKAGE THROUGH CRACKS IN DOWNSTREAM FACING
LEAKAGE THROUGH WOOD PLANKS
3+00
2+00
1+00
0+00
SECTION A-A
EXIST. GROUND
EL 1606.1
EL 1593.4
SECTION B-B
STONE EMBANKMENT
EXISTING GROUND
NOTES
1) THE ELEVATIONS SHOWN ARE BASED ON AN ELEVATION OF 1603 SHOWN ON USGS QUADRANGLE SHEET ASSUMED TO BE POOL ELEVATION AT PERMANENT SPILLWAY CREST.
2. THE INFORMATION SHOWN ON THIS DRAWING IS BASED ON EXISTING SKETCHES AND VISUAL OBSERVATIONS MADE DURING THE FIELD INSPECTION. DIMENSIONS OR MATERIALS INDICATED ON THIS DRAWING WHICH WERE BELOW GRADE OR WATER DURING THE TIME OF INSPECTION WERE NOT VERIFIED.
BUTTERFIELD POND DAM

APPENDIX C

SELECTED PHOTOGRAPHS



NATIONAL PROGRAM OF INSPECTION OF NON-FED DAMS

BUTTERFIELD POND DAM
PHOTO INDEX

ASHUELOT RIVER WASHINGTON, NEW HAMPSHIRE

Photo No. 1 - General view of reservoir from dam.

Photo No. 2 - View of crest of dam from left abutment looking toward right abutment

Photo No. 5 - View of downstream face of left stone embankment (Note depression in crest of embankment)

Photo No. 6 - Closeup view of 4 feet deep sink hole located to left of overflow section

Photo No. 9 - Closeup of seepage and cracks on downstream face of overflow section.

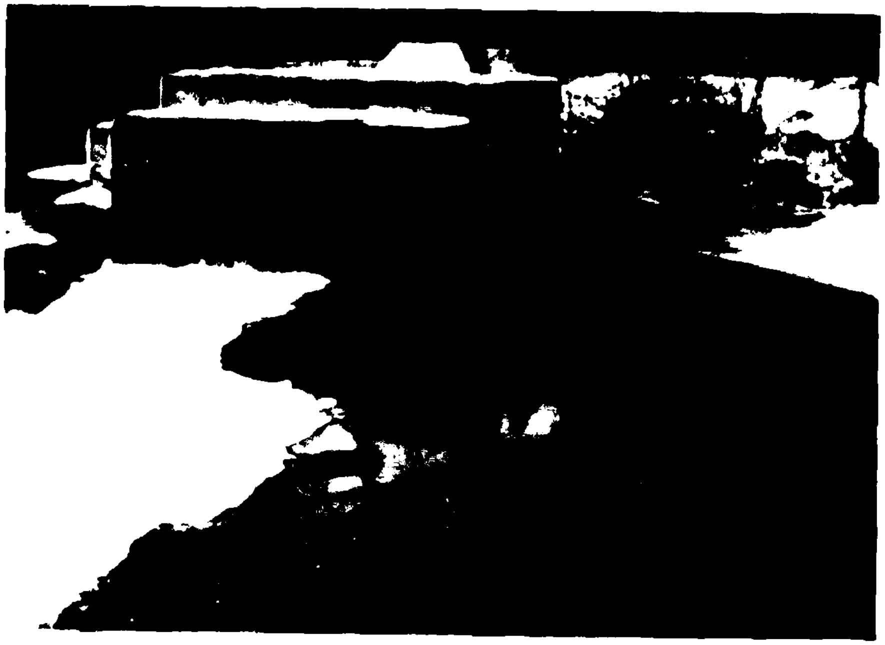


Photo No. 10 - View of upstream face of sluice gate structure and erosion of concrete cap of overflow section.

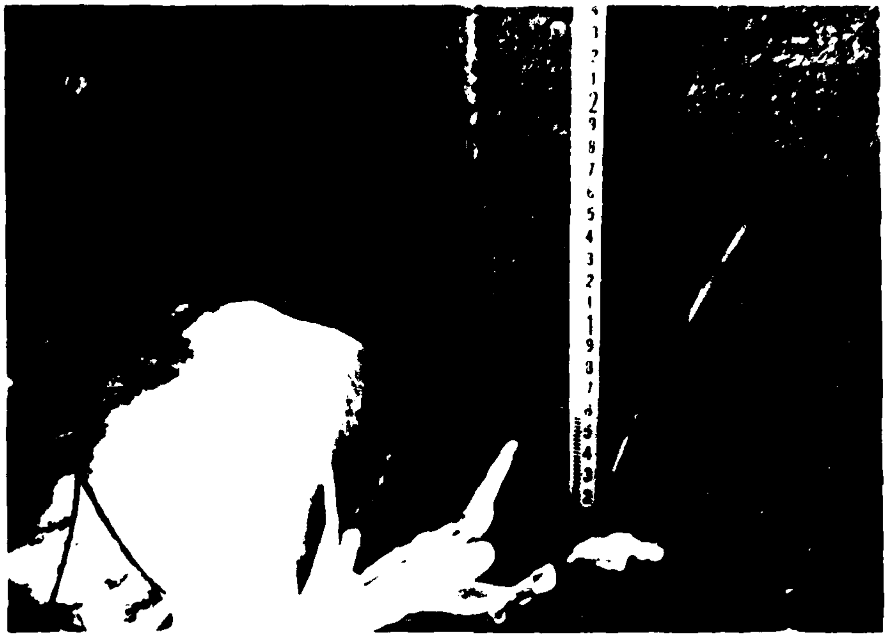
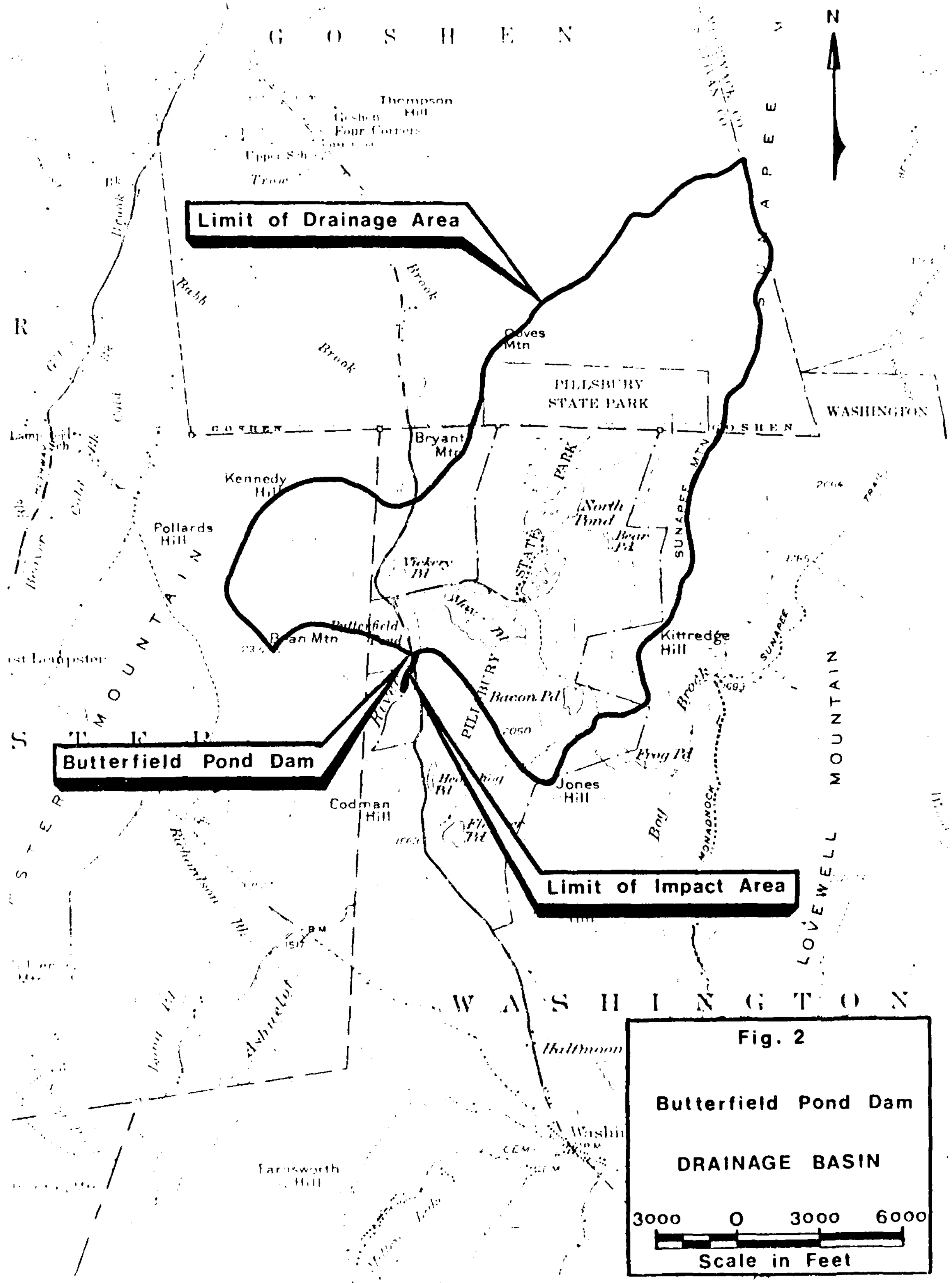


Photo No. 13 - Closeup view of seepage at the downstream end of the right training wall of the sluice gate structure.

Photo No. 14 - General view of downstream channel immediately below dam.

APPENDIX D

HYDROLOGIC AND HYDRAULIC COMPUTATIONS


N
G O S H E N
Thompson Hill
Limit of Drainage Area
Oves Mtn
PILLSBURY STATE PARK
WASHINGTON
Bryant Mtn
Kennedy Hill
Pollards Hill
North Pond
Bear Pd
Kittredge Hill
Bean Mtn
Butterfield Pond
Bacon Pd
Butterfield Pond Dam
Jones Hill
Codman Hill
Limit of Impact Area
LOVEWELL MOUNTAIN
SUNAPEE MTN
W A S H I N G T O N
Farnsworth Hill
Fig. 2
Butterfield Pond Dam
DRAINAGE BASIN
3000
0
3000
6000
Scale in Feet

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ENGINEERS / PLANNERS

BOSTON, MASS.
ROCHESTER, N.H.

CLIENT Army Corps
PROJECT Pittsfield [illegible]
DETAIL Hydrologic Calcs

JOB NO. 274-7901
COMPTD. BY BWP
CK'D. BY KMS

DATE 2/4/80
DATE 2/3/80

I. Basic Data

A. Drainage Area

1. 7.15 square miles — as defined on U.S.G.S. sheets and then planimetered

2. drainage area would classify as mountainous for estimating MPF Peak Flow Rates

B. Dam and Storage Information

1. Size Classification: SMALL based on Storage (< 1000 acreft and ≥ 50 acre-ft)

   as indicated below storage at crest of dam estimated to be 500 acre-ft

2 Hazard Potential: SIGNIFICANT

   Failure could result in damage to state bridge and highway (N.H. Route 3)

3. Storage Information

Descriptive Information	Elevation* (feet)	Surface* Area (acres)	Storage (acre-ft)
1620' contour	1620	255	
Test Flood	1609.5	175	415
Top of left embankment	1606.6	152	935
Top of dam (low point on left stone embankment)	1604.2	134	500
Principal Normal Pool	1603.2	126	465
Spillway weir crest	1603.0	125	440

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CLIENT Army Corps JOB NO. 274-7901 PAGE [illegible]
PROJECT Butterfield Pond COMPTD. BY RWP DATE [illegible]
DETAIL Hydrologic Calcs. CK'D. BY KMS DATE [illegible]

* Notes: (1) elevations : NGVD
(2) normal pool taken to correspond with pool shown on U.S.G.S. sheet, elevation of overflow spillway weir crest equal to 1603.0 feet (NGVD)
(3.) surface area at crest of dam determined by interpolating between the surface areas defined by the pool shown on the U.S.G.S. sheet and the 1620 feet contour
(4) Storage at spillway weir crest estimated by dividing reservoir into pyramidal frustrum sections and determining the volume of each section with the equation for the volume of a pyramidal frustrum.

C. Spillway Information

1. Overflow section located near the center of the dam was a concrete cap. Adjacent to the right training wall is a sluice gate. The gate is constructed of wooden planks enclosed in a poured concrete structure. The gate is presently inoperable, and leaks considerably.

   a. for the subsequent calculations of spillway capacity it was assumed that the sluice gate was closed and not leaking.

2 Discharge over the spillway given by broad-crested weir formula

$$Q = C L H^{3/2}$$ (Standard Handbook for CE's, [illegible])

where : Q = discharge, cfs
C = discharge coeff., use 2.6

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CLIENT Army Corps | JOB No. 274-790 | PAGE 3 of 47
PROJECT Butterfield Pond | COMPTD. BY PJP | DATE 2/4/80
DETAIL Hydrologic Calcs | CK'D. BY KMS | DATE 2/3/80

L = weir length, feet
H = head over weir, feet

II Estimate Effect of Surcharge Storage on Maximum Probable Discharge.

A. Develop stage-discharge curve for outflow from dam-complex

1. define sources of outflow

a. discharge over spillway + overflow structure - above elevation 1603.0' as defined above

b. discharge through opening above sluicegate - above elevation 1603.3'

(1) use broad-crested weir equation from elevation 1603.3 to elevation 1607.3 with C = 2.6

(2) above elevation 1607.3 flow defined by discharge through an orifice

$$Q = C a \sqrt{2gh}$$ (Standard Handbook for CEs Merritt

where Q = discharge, cfs
C = coeff of discharge use 0.6
a = area of orifice, sq. ft
g = acceleration due to gravity, 32.2 ft/sec²
h = head on horizontal center line of orifice, feet

c. discharge over stone embankment adjacent to left abutment - above elevation 1604.2'
(1) use broad-crested weir equation as defined above with C = 2.6

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CLIENT Army Corps — JOB NO. 274-7901 —
PROJECT Butterfield Pond — COMPTD. BY BWP — DATE 3/4/80
DETAIL Hydrologic Calcs. — CK'D. BY VMS — DATE [illegible]

d. discharge over right stone embankment above elevation 1606.5
   (1) use broad-crested weir equation as defined above with C = 2.6

e. discharge over abutments and concrete sluiceway structure — above elevation 1606.6'
   (1) use broad-crested weir equation as defined above with C=2.6

f. discharge over gravel road — above elevation 1607.9
   (1) use broad-crested weir equation — with C = 2.6

2. Discharge over spillway + overflow structure

Elevation (feet NGVD)	C	L (feet)	H (feet)	Q (cfs)
1603.0	—	—	0	0
1604.0	2.6	57	avg ≈ 0.95	137
1605.0	↓	↓	1.95	404
1606.0			2.95	751
1607.0			3.95	1160
1608.0			4.95	1630
1609.0			5.95	2150
1610.0			6.95	2720
1611.0			7.95	3320
1612.0			8.95	3970
1613.0	↓	↓	9.95	4650

3. Discharge through opening above sluicegate
   a between elevations 1603.27' and 1607.27'

Elevation (feet NGVD)	C	L (feet)	H (feet)	Q (cfs)
1603.27	—	—	0	0
1604.0	2.6	6.1	0.73	10
1605.0	↓	↓	1.73	36
1606.0			2.73	72
1607.0	↓	↓	3.73	114

b. above elevation 1607.27'

Elevation (feet NGVD)	C	a (ft^2)	h (feet)	Q (cfs)
1608.0	0.6	24.4	2.73	194
1609.0	↓	↓	3.73	227
1610.0			4.73	256
1611.0			5.73	281
1612.0			6.73	305
1613.0			7.73	327

4. discharge over left stone embankment

a. triangular x-section

Elevation (feet NGVD)	C	L (feet)	H, avg (feet)	Q (cfs)
1604.17	—	—	0	0
1605.0	2.6	9	0.42	6
1606.0	↓	18	0.92	41
1607.0		↓	1.92	125
1608.0			2.92	234
1609.0			3.92	363
1610.0			4.92	511
1611.0			5.92	674
1612.0			6.92	852
1613.0			7.92	1040

b. remainder of left stone embankment

Elevation (feet NGVD)	C	L (feet)	H, avg (feet)	Q (cfs)
1606.08	—	—	0	0
1607.0	2.6	63	0.75	[illegible]5
1608.0	↓	74	1.75	[illegible]
1609.0		76	2.75	[illegible]01
1610.0		78	3.75	[illegible]470

b. remainder of left stone embankment

Elevation (feet NGVD)	C	L (feet)	H (feet)	Q (cfs)
1611.0	2.6	80	4.75	2150
1612.0	↓	82	5.75	2940
1613.0	↓	84	6.75	3830

5. discharge over right stone embankment

Elevation (feet NGVD)	C	L (feet)	H, avg (feet)	Q (cfs)
1606.48	—	—	0	0
1607.0	2.6	46	0.47	39
1608.0	↓	51	1.47	236
1609.0	↓	↓	2.47	515
1610.0	↓	↓	3.47	857
1611.0	↓	↓	4.47	1250
1612.0	↓	↓	5.47	1700
1613.0	↓	↓	6.47	2180

6. discharge over left training wall & sluice gate structure

a. left training wall

Elevation (feet NGVD)	C	L (feet)	H (feet)	Q (cfs)
1606.58	—	—	0	0
1607.0	2.6	4	0.42	3
1608.0	↓	↓	1.42	13
1609.0	↓	↓	2.42	39
1610.0	↓	↓	3.42	66
1611.0	↓	↓	4.42	97
1612.0	↓	↓	5.42	131
1613.0	↓	↓	6.42	169

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PROJECT Butterfield Pond　　COMPTD. BY BWF　　DATE 2/4/83
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b. sluicegate structure

Elevation (feet, NGVD)	C	L (feet)	H (feet)	Q (cfs)
1607.77	—	—	—	0
1608.0	2.6	17.6	0.23	5
1609.0	↓	↓	1.23	62
1610.0			2.23	152
1611.0			3.23	266
1612.0			4.23	396
1613.0			5.23	547

7. discharge over gravel road to west of right stone embankment

Elevation (feet NGVD)	C	L (feet)	H avg (feet)	Q (cfs)
1607.38	—	—	0	0
1608.0	2.6	32	0.37	19
1609.0	↓	35	1.37	146
1610.0		39	2.37	370
1611.0		42	3.37	676
1612.0		45	4.37	1070
1613.0		49	5.37	1590

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CLIENT: Army Corps	JOB NO. 274-7901	PAGE 8 of [illegible]	
PROJECT: [illegible] Pond	COMPTD. BY BWP	DATE 3/5/80	
DETAIL: Hydrologic Calcs.	CK'D. BY [illegible]	DATE [illegible]	

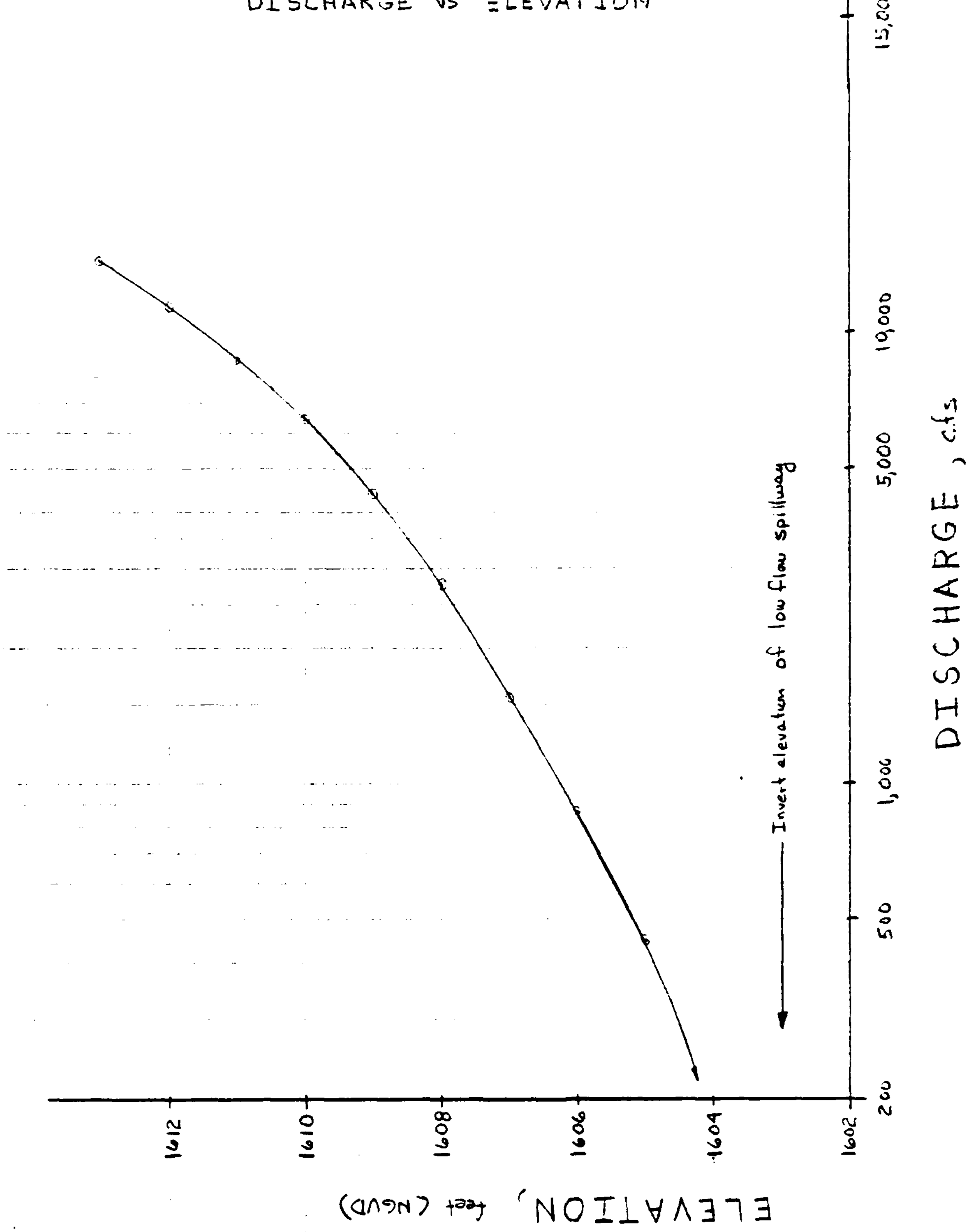
8. Total discharge from project site

Elevation (feet NGVD	Q [illegible] spillway	Q opening [illegible] gate	Q total left embankment	Q right embankment	Q [illegible] abutment + S. [illegible]	Q gravel road	Q TOTAL
1603.0	0	0	0	0	0	0	0
1604.0	137	10	0	0	0	0	147
1605.0	404	36	6	0	0	0	446
1606.0	751	72	41	0	0	0	864
1607.0	1160	114	240	39	3	0	≈ 1560
1608.0	1630	194	679	236	23	19	2780
1609.0	2150	227	1264	515	101	146	4400
1610.0	2720	256	1981	857	218	370	6400
1611.0	3320	231	2324	1250	363	576	9710
1612.0	3970	305	3792	1700	527	1070	11,400
1613.0	4650	327	4870	2180	716	1590	14,300

Discharge vs Elevation shown graphically in Figure 2

FIGURE 1

DISCHARGE vs ELEVATION



B. Effect of surcharge storage on max. prob. discharge

1. Pertinent Data

   a. Drainage area = 7.15 square miles
   b. Characteristics of basin - mountainous
   c. Test flood = 1/2 PMF (SMALL SIZE and SIGNIFICANT HAZARD
   d. Follow Army Corps' procedure

2. STEP 1: Determine Peak Inflow Q_{P1} from Guide Curve

   a. the maximum probable discharge was estimated to be 2100 cfs/sq.mi

$$\therefore \text{PMF} = (2100 \text{ cfs/sq.mi})(7.15 \text{ sq.mi})$$

$$\approx 15{,}000 \text{ cfs}$$

$$1/2\ \text{PMF} = 7{,}500 \text{ cfs}$$

3. STEP 2: Determine surcharge height to pass Q_{P1}, $STOR_1$, and Q_{P2}

   a. from Figure 1 determine surcharge height to pass Q_{P1} = 7,500 cfs

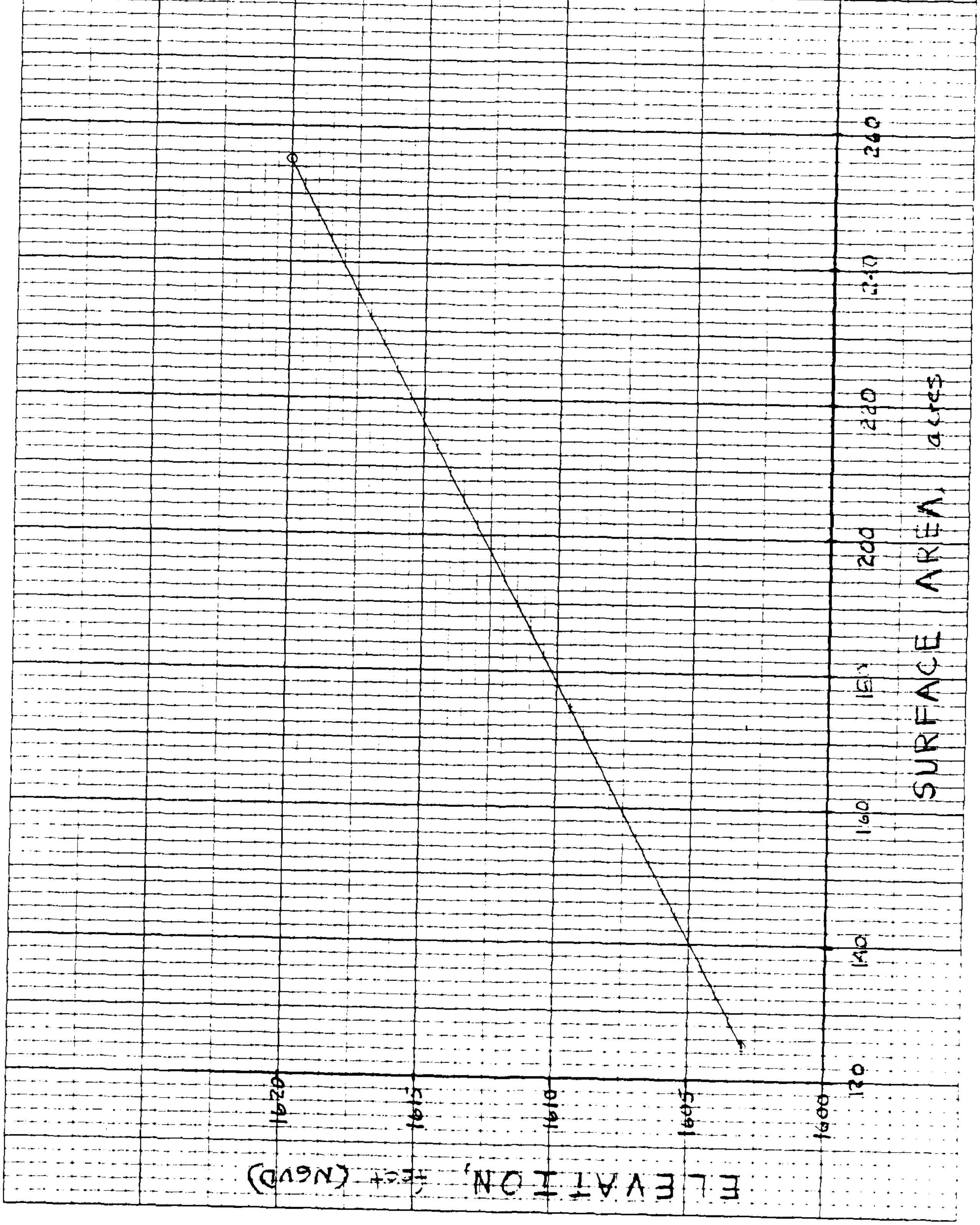
surcharge elevation = 1610.5'
elev. of overflow spillway weir = 1603.0'
surcharge height = 7.5 feet

   b. determine volume of surcharge STOR$_1$ in inches of runoff

first determine volume of storage [illegible]
(1) determine surface area of [illegible] corresponding to surcharge elevation from Figure 2 = 132 acres
(2) average surface area for surcharge [illegible] and spillway crest pool (elevation 1603.0')

FIGURE 2

SURFACE AREA vs ELEVATION



(3) multiply surcharge (depth above elevation 1603.0) times average surface area to determine volume of storage in acre-ft for input into following equation.

$$STOR_1 = \frac{\text{Volume of storage (in acre-inches)}}{\text{drainage area}}$$

$$STOR_1 = \frac{\left(\frac{182 \text{ acres} + 125 \text{ acres}}{2}\right)(7.5 \text{ feet})(12''/\text{ft})}{4576 \text{ acres}}$$

$$STOR_1 = 3.02 \text{ inches}$$

c. determine Q_{P2}

$$Q_{P2} = Q_{P1}\left(1 - \frac{STOR_1}{9.5''}\right)$$

$$Q_{P2} = (7{,}500 \text{ cfs})\left(1 - \frac{3.02''}{9.5''}\right)$$

$$Q_{P2} \approx 5{,}120 \text{ cfs}$$

4. STEP 3: Determine surcharge height [illegible] Q_{P2} and then Q_{P3}

a. From Figure 1 determine surcharge height to pass $Q_{P2} = 5{,}120$ cfs

surcharge elevation	=	1609.4
elev. spillway weir crest	=	1603.0
surcharge height	=	6.4 feet

surface area at 1609.4' = 174 acres

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b. determine $STOR_2$

$$STOR_2 = \frac{\left(\frac{174\,ac + 125\,ac}{2}\right)(6.4\,ft)(12''/ft)}{4576\text{ acres}}$$

$$= 2.51 \text{ inches}$$

c. Average $STOR_1$ and $STOR_2$

$$STOR_{AVG} = \frac{STOR_1 + STOR_2}{2}$$

$$STOR_{AVG} = \frac{3.02'' + 2.51''}{2}$$

$$STOR_{AVG} = 2.77 \text{ inches}$$

d. determine Q_{P3}

$$Q_{P3} = (7{,}500\,cfs)\left(1 - \frac{2.77''}{9.5''}\right)$$

$$Q_{P3} = 5{,}320\ cfs$$

5. STEP 4: Determine surcharge height for Q_{P3} and $STOR_3$

a. from Figure 1 surcharge height for Q_{P3} = 5,320 cfs

surcharge elevation = 1609.5'
elev spillway weir crest = 1603.0'
surcharge height = 6.5 feet

surface area at 1609.5' = 175 acres

b. determine $STOR_3$

$$STOR_3 = \frac{\left(\frac{175\,ac + 125\,ac}{2}\right)(6.5\,ft)(12''/ft)}{4576\text{ acres}}$$

$STOR_3$ = 2.56 inches

c. determine $STOR_{AVG}$

$$STOR_{AVG} = \frac{2.77'' + 2.56''}{2}$$

$STOR_{AVG}$ = 2.67 inches

d. determine Q_{P4}

$$Q_{P4} = (7{,}500 \text{ cfs})\left(1 - \frac{2.67''}{9.5''}\right)$$

Q_{P4} = 5,390 cfs

6. STEP 5: Determine surcharge height for Q_{P4} and $STOR_4$, and Q_{P5}

a. From Figure 1 surcharge height for Q_{P4} = 5,390 c[illegible]

surcharge elevation	=	1609.5
elev. spillway weir crest	=	1603.0
surcharge height	=	6.5

surface area at 1609.5' = 175

b. determine $STOR_4$

$$STOR_4 = \frac{\left(\frac{175ac + 125ac}{2}\right)(6.5 ft)(12''/ft)}{4576 \text{ acres}}$$

$STOR_4$ = 2.56 inches

c. determine $STOR_{AVG}$

$$STOR_{AVG} = \frac{2.67'' + 2.56''}{2}$$

= 2.62 inches

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d. determine Q_{P5}

$$Q_{P5} = (7,500 \text{ cfs})\left(1 - \frac{2.62''}{9.5''}\right)$$

$$Q_{P5} = 5,430 \text{ cfs}$$

6. STEP 6: Determine surcharge height for Q_{P5} and $STOR_5$

a. From Figure 1 surcharge height for Q_{P5} = 5,430 cfs

surcharge height = 1609.5'
elev. spillway weir crest = 1603.0
surcharge height = 6.5 feet

surface area at 1609.5' = 175 acres

b. determine $STOR_5$

$$STOR_5 = \frac{\left(\frac{175\,ac + 125\,ac}{2}\right)(6.5\,ft)(12''/ft)}{4,576 \text{ acres}}$$

$$STOR_5 = 2.56 \text{ inches}$$

c. determine $STOR_{AVG}$

$$STOR_{AVG} = \frac{2.62'' + 2.56''}{2}$$

$$STOR_{AVG} = 2.59 \text{ inches}$$

$STOR_5$ and $STOR_{AVG}$ agree to within about 1%, therefore accept test flood discharge = 5,430 cfs at surcharge elev. = 1609.5 feet

7. In Conclusion

a. Test flood discharge = 5,430 cfs and will overtop over spillway weir crest by 6.5 feet and the dam crest by 5.3 feet

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b. overflow spillway capacity

(1) water surface at low point on left stone embankment - 1604.2'

$$Q = (2.6)(16.2')(1604.2 - 1603.0)^{3/2} + (2.6)(40.8')(1604.2' - 1603.2')^{3/2} \approx \underline{\underline{160 cfs}}$$

(2) water surface at top of left abutment - 1606.6'

$$Q = (2.6)(16.2')(1606.6' - 1603.0')^{3/2} + (2.6)(40.8')(1606.6' - 1603.2)^{3/2} \approx \underline{\underline{450 cfs}}$$

(3) water surface at test flood elevation - 1609.5'

$$Q = (2.6)(16.2')(1609.5' - 1603.0)^{3/2} + (2.6)(40.8')(1609.5 - 1603.2)^{3/2} \approx \underline{\underline{2375 cfs}}$$

C. sluicegate capacity - includes discharge through 6.1' wide by 6.05' high sluicegate - orifice discharge

(1) water surface at low point on left stone embankment - 1604.2'

$$(a)\ Q_{sluicegate} = (0.6)(6.1')(6.05')\left[(2)(32.2)(1604.2' - 1598.25)\right]$$

$$Q_{sluicegate} \approx 435 \text{ cfs}$$

(2) water surface at top of left abutment - 1606.6'

$$(a)\ Q_{sluicegate} = (0.6)(6.1')(6.05')\left[(2)(32.2)(1606.6' - 1598.25')\right]$$

$$Q_{sluicegate} \approx 515 \text{ cfs}$$

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(3) water surface at test flood elevation - 1609.5'

$$(a)\ Q_{sluicegate} = (0.6)(6.1')(6.05')\left[(2)(32.2)(1609.5' - 1598.25')\right]^{1/2}$$

$$Q_{sluicegate} \approx 595 \text{ cfs}$$

d capacity of low flow spillway

$$Q = (2.6)(16.2)(0.2)^{3/2} = \qquad 3.3 \text{cfs}$$

III. Using "Rule of Thumb" Guidance for Estimating Downstream Dam Failure Hydrographs examine impact of dam failure

1. Pertinent Data

   a. Failure occurs when reservoir level at crest of dam - elevation = 1604.2 feet

   b. Storage at crest elevation estimated to be approximately 590 acre-feet

A. Reach 1

1. STEP 1: Determine reservoir storage at time of failure

   from previous calcs. storage = 590 acre-ft

2. STEP 2: Determine Peak Failure Outflow Q_{P1}

$$Q_{P1} = (8/27)\, W_b \sqrt{g}\, Y_o^{3/2}$$

where: W_b = Breach width (use 40% of total length)

$= (210\text{ feet})(0.40)$

$= 84$ feet

Y_o = Total height from channel bed to pool level at failure

$= 12.2$ feet

$$\begin{array}{r} 1604.2' \\ \approx \underline{1592.0'} \\ 12.2 \end{array}$$

$$Q_{P1} = (8/27)(84\text{ feet})(32.2)^{1/2}(12.2\text{ ft})^{3/2}$$

$$Q_{P1} \approx 6{,}000\text{ cfs}$$

Prefailure flow is negligible compared to the dam failure discharge, and [illegible] been included as part of these calculations

3. STEP 3: Prepare stage-discharge curve for Reach I

a. Pertinent Data

(1) Reach length = 350 feet

(2) Channel slope = 0.0155

(3) Manning n = 0.05

(4) Channel shape - trapezoidal

(5) Base width ≈ 20 feet

b. See Figure 3 for stage-discharge curve

4. STEP 4: Estimate Reach Outflow

a. Determine stage for Q_{P1} = 6000 cfs from Figure 3 and find volume in reach

(1) Stage (depth of flow) = 7.6 feet

(2) Volume in reach = (reach length) (cross-sectional area of channel)

$$\text{X-area} = (0.5)(7.6\text{ ft})(20\text{ ft} + 133\text{ ft})$$
$$= 581\text{ ft}^2$$

$$\text{Volume} = V_1 = \frac{(581\text{ ft}^2)(350\text{ ft})}{43{,}560\text{ ft}^2/\text{acre}}$$
$$= 4.7\text{ acre-ft}$$

$$V_1 < \frac{S}{2} \quad \therefore \text{reach length OK}$$

b. Determine Q_P (TRIAL)

$$Q_{P2(\text{TRIAL})} = Q_{P1}\left(1 - \frac{V_1}{S}\right)$$

$$Q_{P2(\text{TRIAL})} = (6000\text{ cfs})\left(1 - \frac{4.7\text{ acre-ft}}{590\text{ acre-ft}}\right)$$

$$Q_{P2(\text{TRIAL})} = 5950\text{ cfs}$$

c. Compute V_2 using $Q_{P2\,(TRIAL)}$

From Figure 3 determine stage for $Q_{P2(TRIAL)}$

Stage = 7.5 feet

X-area = (0.5)(7.5 ft)(20 ft + 132 ft)
= 570 ft²

$$V_2 = \frac{(570\ ft^2)(350\ ft)}{43{,}560\ ft^2/acre}$$

V_2 = 4.6 acre-ft

d. Average V_1 and V_2 and compute Q_{P2}

(1) $$V_{avg} = \frac{V_1 + V_2}{2}$$

$$V_{avg} = \frac{4.7\ ac\text{-}ft + 4.6\ ac\text{-}ft}{2}$$

Vavg = 4.7 acre-ft

(2) $$Q_{P2} = Q_{P1}\left(1 - \frac{V_{avg}}{S}\right)$$

$$Q_{P2} = (6000\ cfs)\left(1 - \frac{4.7}{590}\right)$$

Q_{P2} = 5950 cfs

3. Reach 2

3. STEP 3: Prepare stage-discharge curve for Reach 2

a. Pertinent Data

(1) Reach length = 175 feet

(2) Channel slope = 0.0375

(3) Manning n = 0.05

(4) Channel shape - trapezoidal

(5) Base width ≈ 20 feet

b. See Figure 3 for stage-discharge curve

4. STEP : Estimate Reach Outflow

a. Determine stage for Q_{P2} = 5950 cfs from Figure 3 and find volume in reach

(1) Stage (depth of flow) = 6.3 feet

(2) Volume in reach = (reach length) (cross-sectional area of channel)

$$\text{X-area} = (0.5)(6.3\text{ ft})(20\text{ ft} + 115\text{ ft}) \approx 425\text{ ft}^2$$

$$\text{Volume} = V_1 = \frac{(425\text{ ft}^2)(175\text{ ft})}{43{,}560\text{ ft}^2/\text{acre}} = 1.7\text{ ac-ft}$$

$V_1 < \frac{S}{2}$ ∴ reach length OK

b. Determine $Q_{P3(TRIAL)}$

$$Q_{P3(TRIAL)} = Q_{P2}\left(1 - \frac{V_1}{S}\right)$$

$$Q_{P3(TRIAL)} = (5950\text{ cfs})\left(1 - \frac{1.7}{590}\right)$$

$$Q_{P3(TRIAL)} = 5930\text{ cfs}$$

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c. Compute V_2 using Q_{P3} (TRIAL)

From Figure 3 determine stage for Q_P (TRIAL)

Stage = 6.3 feet

X-area = (0.5)(6.3 ft)(20 ft + 115 ft)
≈ 425 ft²

$$V_2 = \frac{(425\ ft^2)(175\ ft)}{43{,}560\ ft^2/acre}$$

V_2 = 1.7 acre-ft

d. Average V_1 and V_2 and compute Q_{P3}

(1) $Vavg = \frac{V_1 + V_2}{2}$

$$V_{avg} = \frac{1.7\ ac\text{-}ft + 1.7\ ac\text{-}ft}{2}$$

Vavg = 1.7 acre-ft

(2) $Q_{P3} = Q_{P2}\left(1 - \frac{Vavg}{S}\right)$

$$Q_{P3} = (5{,}950\ cfs)\left(1 - \frac{1.7}{590}\right)$$

Q_{P3} = 5,930 cfs

C. Reach 3

3. STEP 3: Prepare stage-discharge curve for Reach 3

a. Pertinent Data

(1) Reach length = 725 feet

(2) Channel slope = 0.0276

(3) Manning n = 0.05

(4) Channel shape - trapezoidal

(5) Base width ≈ 20 feet

b. See Figure 3 for stage-discharge curve

4. STEP 4: Estimate Reach Outflow

a. Determine stage for Q_{P3} = 5,930 cfs from Figure 3 and find volume in reach

(1) Stage (depth of flow) = 6.8 feet

(2) Volume in reach = (reach length) (cross-sectional area of channel)

X-area = (0.5)(6.8 ft)(20 ft + 122 ft)
≈ 483 ft²

$$\text{Volume} = V_1 = \frac{(483\ \text{ft}^2)(725\ \text{ft})}{43{,}560\ \text{ft}^2/\text{acre}}$$

= 8.0 acre-ft

$V_1 < \frac{S}{2}$ ∴ reach length OK

b. Determine $Q_{P4(TRIAL)}$

$$Q_{P4(TRIAL)} = Q_{P3}\left(1 - \frac{V_1}{S}\right)$$

$$Q_{P4(TRIAL)} = (5{,}930\ \text{cfs})\left(1 - \frac{8.0}{590}\right)$$

$$Q_{P4(TRIAL)} = 5{,}850\ \text{cfs}$$

c. Compute V_2 using $Q_{P4(TRIAL)}$

From Figure 3 determine stage for $Q_{P4(TRIAL)}$

Stage = 6.8 feet

X-area = (0.5)(6.8 ft)(20 ft + 122 ft)

≈ 483 ft^2

$$V_2 = \frac{(483\ ft^2)(725\ ft)}{43{,}560\ ft^2/acre}$$

V_2 = 8.0 acre-ft

d. Average V_1 and V_2 and compute

(1) $$V_{avg} = \frac{V_1 + V_2}{2}$$

$$V_{avg} = \frac{8.0\ acre\text{-}ft + 8.0\ acre\text{-}ft}{2}$$

V_{avg} = 8.0 acre-ft

(2) $$Q_{P4} = Q_{P3}\left(1 - \frac{V_{avg}}{S}\right)$$

$$Q_{P4} = (5{,}930\ cfs)\left(1 - \frac{8.0}{590}\right)$$

Q_{P4} = 5,850 cfs

D. Reach 4

3. STEP 3: Prepare stage-discharge curve for Reach 4

   a. Pertinent Data

      (1) Reach length = 1650 feet
      (2) Channel slope = 0.0077
      (3) Manning n = 0.05
      (4) Channel shape - trapezoidal
      (5) Base width ≈ 20 feet

   b. See Figure 3 for stage-discharge curve

4. STEP 4: Estimate Reach Outflow

   a. Determine stage for Q_{P4} = 5,850 cfs from Figure 3 and find volume in reach

      (1) Stage (depth of flow) = 4.4 feet

      (2) Volume in reach = (reach length) (cross-sectional area of channel)

$$\text{X-area} = (0.5)(4.4\text{ ft})(20\text{ ft} + 580\text{ ft})$$
$$= 1320\text{ ft}^2$$

$$\text{Volume} = V_1 = \frac{(1320\text{ ft}^2)(1650\text{ ft})}{43,560\text{ ft}^2/\text{acre}}$$

$$= 50.0\text{ acre-ft}$$

$$V_1 < \frac{S}{2} \quad \therefore \text{reach length OK}$$

   b. Determine $Q_{P5(TRIAL)}$

$$Q_{P5(TRIAL)} = Q_{P4}\left(1 - \frac{V_1}{S}\right)$$

$$Q_{P5(TRIAL)} = (5,850\text{ cfs})\left(1 - \frac{50}{590}\right)$$

$$Q_{P5(TRIAL)} = 5,350\text{ cfs}$$

c. Compute V_2 using $Q_{P5(TRIAL)}$

From Figure 3 determine stage for $Q_{P5(TRIAL)}$

Stage = 4.2 feet

X-area = (0.5)(4.2 ft)(20 ft + 554 ft)

$\approx$ 1205 ft²

$$V_2 = \frac{(1205\ ft^2)(1650\ feet)}{43{,}560\ ft^2/acre}$$

V_2 = 45.7 acre-ft

d. Average V_1 and V_2 and compute Q_{P5}

(1) $$V_{avg} = \frac{V_1 + V_2}{2}$$

$$V_{avg} = \frac{50.0\ ac\text{-}ft + 45.7\ ac\text{-}ft}{2}$$

V_{avg} = 47.9 acre-ft

(2) $$Q_{P5} = Q_{P4}\left(1 - \frac{V_{avg}}{S}\right)$$

$$Q_{P5} = (5{,}950\ cfs)\left(1 - \frac{47.9}{590}\right)$$

Q_{P5} = 5,390 cfs

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E. Reach 5

3. STEP 3: Prepare stage-discharge curve for Reach 5

a. Pertinent Data

(1) Reach length = 950 feet

(2) Channel slope = 0.0077

(3) Manning n = 0.05

(4) Channel shape - trapezoidal

(5) Base width ≈ 20 feet

b. See Figure 3 for stage-discharge curve

4. STEP 4: Estimate Reach Outflow

a. Determine stage for Q_{P5} = 5,380 cfs from Figure 3 and find volume in reach

(1) Stage (depth of flow) = 6.8 feet

(2) Volume in reach = (reach length) $\begin{pmatrix}\text{cross-sectional}\\ \text{area of channel}\end{pmatrix}$

$$\text{X-area} = (0.5)(6.8\text{ feet})(20\text{ feet} + 234\text{ feet}) \approx 864\text{ ft}^2$$

$$\text{Volume} = V_1 = \frac{(864\text{ ft}^2)(950\text{ feet})}{43{,}560\text{ ft}^2/\text{acre}}$$

$$= 18.8\text{ acre-ft}$$

$V_1 < \frac{S}{2}$ ∴ reach length OK

b. Determine $Q_{P6(TRIAL)}$

$$Q_{P6(TRIAL)} = Q_{P5}\left(1 - \frac{V_1}{S}\right)$$

$$Q_{P6(TRIAL)} = (5{,}380\text{ cfs})\left(1 - \frac{18.8}{590}\right)$$

$$Q_{P6(TRIAL)} = 5{,}210\text{ cfs}$$

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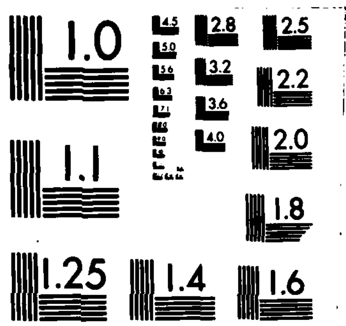

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c. Compute V_2 using $Q_{P6\,(TRIAL)}$

From Figure 3 determine stage for Q_P (TRIAL)

Stage = 6.7 feet

X-area = (0.5)(6.7 feet)(20 feet + 230 feet)
≈ 838 ft²

$$V_2 = \frac{(838\ ft^2)(950\ ft)}{43{,}560\ ft^2/acre}$$

V_2 = 18.3 acre-ft

d. Average V_1 and V_2 and compute Q_{P6}

(1) $$V_{avg} = \frac{V_1 + V_2}{2}$$

$$V_{avg} = \frac{18.8\ acre\text{-}ft + 18.3\ acre\text{-}ft}{2}$$

V_{avg} = 18.5 acre-ft

(2) $$Q_{P6} = Q_{P5}\left(1 - \frac{V_{avg}}{S}\right)$$

$$Q_{P6} = (5{,}380\ cfs)\left(1 - \frac{18.5}{590}\right)$$

Q_{P6} = 5,210 cfs

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F. Reach 6

3. STEP 3: Prepare stage-discharge curve for Reach 6

a. Pertinent Data

(1) Reach length = 1550 feet

(2) Channel slope = 0.00457

(3) Manning n = 0.05

(4) Channel shape - trapezoidal

(5) Base width ≈ 20 feet

b. See Figure 3 for stage-discharge curve

4. STEP 4: Estimate Reach Outflow

a. Determine stage for Q_{P6} = 5,210 cfs from Figure 3 and find volume in reach

(1) Stage (depth of flow) = 7.5 feet

(2) Volume in reach = (reach length) (cross-sectional area of channel)

$$\text{X-area} = (0.5)(7.5\text{ feet})(20\text{ feet} + 255\text{ feet})$$
$$= 1031\text{ ft}^2$$

$$\text{Volume} = V_1 = \frac{(1031\text{ ft}^2)(1550\text{ ft})}{43{,}560\text{ ft}^2/\text{acre}}$$

$$= 36.7\text{ acre-ft}$$

$$V_1 < \frac{S}{2}$$ ∴ reach length OK

b. Determine $Q_{P7(TRIAL)}$

$$Q_{P7(TRIAL)} = Q_{P6}\left(1 - \frac{V_1}{S}\right)$$

$$Q_{P7(TRIAL)} = (5{,}210\text{ cfs})\left(1 - \frac{36.7}{590}\right)$$

$$Q_{P7(TRIAL)} = 4{,}890\text{ cfs}$$

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c. Compute V_2 using $Q_{P7(TRIAL)}$

From Figure 3 determine stage for $Q_{P7(TRIAL)}$

Stage = 7.3 feet

X-area = (0.5)(7.3 feet)(20 ft + 249 ft)

≈ 982 ft²

$$V_2 = \frac{(982\ ft^2)(1550\ ft)}{43{,}560\ ft^2/acre}$$

V_2 = 34.9 acre-ft

d. Average V_1 and V_2 and compute Q 7

(1) $Vavg = \dfrac{V_1 + V_2}{2}$

$$V_{avg} = \frac{36.7\ ac\text{-}ft + 34.9\ ac\text{-}ft}{2}$$

Vavg = 35.8 acre-ft

(2) $Q_{p7} = Q_6 \left(1 - \frac{Vavg}{S}\right)$

$$Q_{p7} = (5{,}210\ cfs)\left(1 - \frac{35.8}{590}\right)$$

Q_{p7} = 4,890 cfs

G Reach 7

3. STEP 3: Prepare stage-discharge curve for Reach 7

a. Pertinent Data

(1) Reach length = 2825 feet

(2) Channel slope = 0.00457

(3) Manning n = 0.05

(4) Channel shape - trapezoidal

(5) Base width ≈ 20 feet

b. See Figure 3 for stage-discharge curve

4. STEP 4: Estimate Reach Outflow

a. Determine stage for Q_{P7} = 4,890 cfs from Figure 3 and find volume in reach

(1) Stage (depth of flow) = 7.4 feet

(2) Volume in reach = (reach length) $\begin{pmatrix}\text{cross-sectional}\\ \text{area of channel}\end{pmatrix}$

$$\text{X-area} = (0.5)(7.4 \text{ feet})(20 \text{ ft} + 245 \text{ ft})$$
$$= 981 \text{ ft}^2$$

$$\text{Volume} = V_1 = \frac{(981 \text{ ft}^2)(2825 \text{ ft})}{43{,}560 \text{ ft}^2/\text{acre}}$$

$$= 63.6 \text{ acre-ft}$$

$V_1 < \frac{S}{2}$ ∴ reach length OK

b. Determine $Q_{P8(TRIAL)}$

$$Q_{P8(TRIAL)} = Q_{P7}\left(1 - \frac{V_1}{S}\right)$$

$$Q_{P8(TRIAL)} = (4{,}890 \text{ cfs})\left(1 - \frac{63.6}{590}\right)$$

$$Q_{P8(TRIAL)} = 4{,}360 \text{ cfs}$$

c. Compute V_2 using $Q_{P8(TRIAL)}$

From Figure 3 determine stage for $Q_{P8(TRIAL)}$

Stage = 7.0 feet

X-area = (0.5)(7.0 feet)(20 ft + 232 ft)

= 882 ft²

$$V_2 = \frac{(882\,ft^2)(2825\,ft)}{43{,}560\,ft^2/acre}$$

V_2 = 57.2 acre-ft

d. Average V_1 and V_2 and compute Q_{P8}

(1) $$Vavg = \frac{V_1 + V_2}{2}$$

$$V_{avg} = \frac{63.6\,ac\text{-}ft + 57.2\,ac\text{-}ft}{2}$$

Vavg = 60.4 acre-ft

(2) $$Q_{P8} = Q_{P7}\left(1 - \frac{Vavg}{S}\right)$$

$$Q_{P8} = (4{,}890\ cfs)\left(1 - \frac{60.4}{590}\right)$$

Q_{P8} = 4,390 cfs

H. Reach 8

3. STEP 3: Prepare stage-discharge curve for Reach 8

a. Pertinent Data

(1) Reach length = 5625 feet

(2) Channel slope = 0.00356

(3) Manning n = 0.05

(4) Channel shape - trapezoidal

(5) Base width ≈ 20 feet

b. See Figure 3 for stage-discharge curve

4. STEP 4: Estimate Reach Outflow

a. Determine stage for Q_{P8} = 4,390 cfs from Figure 3 and find volume in reach

(1) Stage (depth of flow) = 6.1 feet

(2) Volume in reach = (reach length) (cross-sectional area of channel)

$$\text{X-area} = (0.5)(6.1 \text{ feet})(20 \text{ feet} + 355 \text{ feet})$$
$$= 1144 \text{ ft}^2$$

$$\text{Volume} = V_1 = \frac{(1144 \text{ ft}^2)(5625 \text{ ft})}{43,560 \text{ ft}^2/\text{acre}}$$

$$= 148 \text{ acre-ft}$$

$V_1 < \frac{S}{2}$ ∴ reach length OK

b. Determine $Q_{P9(TRIAL)}$

$$Q_{P9(TRIAL)} = Q_{P8}\left(1 - \frac{V_1}{S}\right)$$

$$Q_{P9(TRIAL)} = (4,390 \text{ cfs})\left(1 - \frac{148}{590}\right)$$

$$Q_{P9(TRIAL)} = 3,290 \text{ cfs}$$

c. Compute V_2 using $Q_{P9(TRIAL)}$

From Figure 3 determine stage for $Q_{P9(TRIAL)}$

Stage = 5.4 feet

X-area = (0.5)(5.4 feet)(20 feet + 318 ft)
= 913 ft²

$$V_2 = \frac{(913\ ft^2)(5625\ ft)}{43{,}560\ ft^2/acre}$$

V_2 = 118 acre-ft

d. Average V_1 and V_2 and compute Q_{P9}

(1) $$V_{avg} = \frac{V_1 + V_2}{2}$$

$$V_{avg} = \frac{148\ ac\text{-}ft + 118\ ac\text{-}ft}{2}$$

V_{avg} = 133 acre-feet

(2) $$Q_{P9} = Q_{P8}\left(1 - \frac{V_{avg}}{S}\right)$$

$$Q_{P9} = (4{,}390\ cfs)\left(1 - \frac{133}{590}\right)$$

Q_{P9} = 3,400 cfs

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I Reach 9

3. STEP 3: Prepare stage-discharge curve for Reach 9

a. Pertinent Data

(1) Reach length = 2875 feet
(2) Channel slope = 0.0139
(3) Manning n = 0.08
(4) Channel shape - trapezoidal
(5) Base width ≈ 40 feet

b. See Figure 3 for stage-discharge curve

4. STEP 4: Estimate Reach Outflow

a. Determine stage for Q_{p9} = 3,400 cfs from Figure 3 and find volume in reach

(1) Stage (depth of flow) = 5.5 feet

(2) Volume in reach = (reach length) $\begin{pmatrix}\text{cross-sectional} \\ \text{area of channel}\end{pmatrix}$

$$\text{X-area} = (0.5)(5.5\text{ feet})(40\text{ ft} + 720\text{ ft})$$
$$= 2090\text{ ft}^2$$

$$\text{Volume} = V_1 = \frac{(2090\text{ ft}^2)(2875\text{ ft})}{43{,}560\text{ ft}^2/\text{acre}}$$

$$= 138\text{ acre-ft}$$

$V_1 < \frac{S}{2}$ ∴ reach length OK

b. Determine $Q_{P10(TRIAL)}$

$$Q_{P10(TRIAL)} = Q_{P9}\left(1 - \frac{V_1}{S}\right)$$

$$Q_{P10(TRIAL)} = (3{,}400\text{ cfs})\left(1 - \frac{138}{590}\right)$$

$$Q_{P10(TRIAL)} = 2610\text{ cfs}$$

c. Compute V_2 using $Q_{P10(TRIAL)}$

From Figure 3 determine stage for $Q_{P10(TRIAL)}$

Stage = 4.9 feet

X-area = $(0.5)(4.9\text{ ft})(40\text{ ft} + 600\text{ ft})$
$\approx 1568\text{ ft}^2$

$$V_2 = \frac{(1568\text{ ft}^2)(2875\text{ ft})}{43{,}560\text{ ft}^2/\text{acre}}$$

V_2 = 103 acre-ft

d. Average V_1 and V_2 and compute Q_P

(1) $V_{avg} = \frac{V_1 + V_2}{2}$

$$V_{avg} = \frac{138\text{ ac-ft} + 103\text{ ac-ft}}{2}$$

V_{avg} = 121 acre-ft

(2) $Q_{P10} = Q_{P9}\left(1 - \frac{V_{avg}}{S}\right)$

$$Q_{P10} = (3{,}400\text{ cfs})\left(1 - \frac{121}{590}\right)$$

Q_{P10} = 2,710 cfs

J. Reach 10

3. STEP 3: Prepare stage-discharge curve for Reach 10

a. Pertinent Data

(1) Reach length = 5,300 feet
(2) Channel slope = 0.0011
(3) Manning n = 0.08
(4) Channel shape - trapezoidal
(5) Base width ≈ 40 feet

b. See Figure 3 for stage-discharge curve

4. STEP 4: Estimate Reach Outflow

a. Determine stage for Q_{P10} = 2,710 cfs from Figure 3 and find volume in reach

(1) Stage (depth of flow) = 6.7 feet

(2) Volume in reach = (reach length) (cross-sectional area of channel)

$$\text{X-area} = (0.5)(6.7 \text{ feet})(40 \text{ ft} + 525 \text{ ft})$$
$$= 1893 \text{ ft}^2$$

$$\text{Volume} = V_1 = \frac{(1893 \text{ ft}^2)(5{,}300 \text{ ft})}{43{,}560 \text{ ft}^2/\text{acre}}$$
$$= 230 \text{ acre-ft}$$

$V_1 < \frac{S}{2}$ ∴ reach length OK

b. Determine $Q_{P11(TRIAL)}$

$$Q_{P11(TRIAL)} = Q_{P10}\left(1 - \frac{V_1}{S}\right)$$

$$Q_{P11(TRIAL)} = (2{,}710 \text{ cfs})\left(1 - \frac{230}{590}\right)$$

$$Q_{P11(TRIAL)} = 1{,}650 \text{ cfs}$$

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c. Compute V_2 using $Q_{P11(TRIAL)}$

From Figure 3 determine stage for Q_P (TRIAL)

Stage = 5.4 feet

X-area = (0.5) (5.4 feet) (40 ft + 433 ft)

= 1277 ft²

$$V_2 = \frac{(1277\ ft^2)(5{,}300\ ft)}{43{,}560}$$

V_2 = 155 acre-ft

d. Average V_1 and V_2 and compute Q_P

(1) $Vavg = \frac{V_1 + V_2}{2}$

$$V_{avg} = \frac{230\ ac\text{-}ft + 155\ ac\text{-}ft}{43{,}560\ ft^2/acre}$$

Vavg = 193 acre-ft

(2) $Q_{P11} = Q_{P10}\left(1 - \frac{Vavg}{S}\right)$

$$Q_{P11} = (2{,}710\ cfs)\left(1 - \frac{193}{590}\right)$$

Q_{P11} = 1,830 cfs

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K. Reach 11

3. STEP 3: Prepare stage-discharge curve for Reach 11

   a. Pertinent Data

      (1) Reach length = 5,000 feet

      (2) Channel slope = 0.0011

      (3) Manning n = 0.08

      (4) Channel shape - trapezoidal

      (5) Base width ≈ 40 feet

   b. See Figure 3 for stage-discharge curve

4. STEP 4: Estimate Reach Outflow

   a. Determine stage for Q_{P11} = 1,830 cfs from Figure 3 and find volume in reach

      (1) Stage (depth of flow) = 5.7 feet

      (2) Volume in reach = (reach length) (cross-sectional area of channel)

$$\text{X-area} = (0.5)(5.7\ \text{feet})(40\ \text{ft} + 455\ \text{feet})$$
$$= 1411\ \text{ft}^2$$

$$\text{Volume} = V_1 = \frac{(1411\ \text{ft}^2)(5000\ \text{ft})}{43{,}560\ \text{ft}^2/\text{acre}}$$
$$= 162\ \text{acre-ft}$$

$$V_1 < \frac{S}{2}\quad \therefore \text{reach length OK}$$

   b. Determine $Q_{P12(TRIAL)}$

$$Q_{P12(TRIAL)} = Q_{P11}\left(1 - \frac{V_1}{S}\right)$$

$$Q_{P12(TRIAL)} = (1{,}930\ \text{cfs})\left(1 - \frac{162}{590}\right)$$

$$Q_{P12(TRIAL)} = 1{,}330\ \text{cfs}$$

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c. Compute V_2 using $Q_{P12(TRIAL)}$

From Figure 3 determine stage for $Q_{P12(TRIAL)}$

Stage = 5.0 feet

X-area = (0.5) (5.0 ft)(40 ft + 40[illegible]
= 1113 ft²

$$V_2 = \frac{(1113\ ft^2)(5000\ ft)}{43,560\ ft^2/acre}$$

V_2 = 128 acre-ft

d. Average V_1 and V_2 and compute Q_P

(1) $$Vavg = \frac{V_1 + V_2}{2}$$

$$V_{avg} = \frac{162\ acre\text{-}ft + 128\ acre\text{-}ft}{2}$$

Vavg = 145 acre-ft

(2) $$Q_{P12} = Q_{P11}\left(1 - \frac{Vavg}{S}\right)$$

$$Q_{P12} = (1,830\ cfs)\left(1 - \frac{145}{590}\right)$$

Q_{P12} = 1,380 cfs

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L. Reach 12

3. STEP 3: Prepare stage-discharge curve for Reach 12

a. Pertinent Data

(1) Reach length = 5,000 feet

(2) Channel slope = 0.0011

(3) Manning n = 0.08

(4) Channel shape - trapezoidal

(5) Base width ≈ 40 feet

b. See Figure 3 for stage-discharge curve

4. STEP 4: Estimate Reach Outflow

a. Determine stage for Q_{P12} = 1,380 cfs from Figure 3 and find volume in reach

(1) Stage (depth of flow) = 5.0 feet

(2) Volume in reach = (reach length) (cross-sectional area of channel)

$$\text{X-area} = (0.5)(5.0\,ft)(40\,ft + 405\,ft)$$
$$= 1113\,ft^2$$

$$\text{Volume} = V_1 = \frac{(1113\,ft^2)(5000\,ft)}{43{,}560\,ft^2/acre}$$

$$= 128\text{ acre-ft}$$

$V_1 < \frac{S}{2}$ ∴ reach length OK

b. Determine $Q_{P12(TRIAL)}$

$$Q_{P12(TRIAL)} = Q_{P12}\left(1 - \frac{V_1}{S}\right)$$

$$Q_{P12(TRIAL)} = (1{,}380\,cfs)\left(1 - \frac{128}{590}\right)$$

$$Q_{P12(TRIAL)} = 1{,}080\,cfs$$

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c. Compute V_2 using $Q_{P13(TRIAL)}$

From Figure 3 determine stage for $Q_{P13(TRIAL)}$

Stage = 4.6 feet

X-area = (0.5)(4.6 feet)(40 feet + 375 ft)
$\approx$ 955 ft²

$$V_2 = \frac{(955\ ft^2)(5000\ ft)}{43{,}560\ ft^2/acre}$$

V_2 = 110 acre-ft

d. Average V_1 and V_2 and compute Q_{P13}

(1) $$Vavg = \frac{V_1 + V_2}{2}$$

$$V_{avg} = \frac{128\ acre\text{-}ft + 110\ acre\text{-}ft}{2}$$

Vavg = 119 acre-ft

(2) $$Q_{P13} = Q_{P12}\left(1 - \frac{Vavg}{S}\right)$$

$$Q_{P13} = (1{,}380\ cfs)\left(1 - \frac{119}{590}\right)$$

Q_{P13} = 1,100 cfs

M Reach 13

3. STEP 3: Prepare stage-discharge curve for Reach 13

a. Pertinent Data

(1) Reach length = 3,050 feet
(2) Channel slope = 0.0011
(3) Manning n = 0.08
(4) Channel shape - trapezoidal
(5) Base width ≈ 2,080 feet

b. See Figure 3 for stage-discharge curve

4. STEP 4: Estimate Reach Outflow

a. Determine stage for Q_{P13} = 1,100 cfs from Figure 3 and find volume in reach

(1) Stage (depth of flow) = 0.7 feet

(2) Volume in reach = (reach length) (cross-sectional area of channel)

$$\text{X-area} = (0.5)(0.7\text{ feet})(2080\text{ ft} + 2110\text{ ft})$$
$$= 1467\text{ ft}^2$$

$$\text{Volume} = V_1 = \frac{(1467\text{ ft}^2)(3050\text{ ft})}{43,560\text{ ft}^2/\text{acre}}$$

$$= 103\text{ acre-ft}$$

$V_1 < \frac{S}{2}$ ∴ reach length OK

b. Determine $Q_{P14(TRIAL)}$

$$Q_{P14(TRIAL)} = Q_{P13}\left(1 - \frac{V_1}{S}\right)$$

$$Q_{P\ (TRIAL)} = (1,100\text{ cfs})\left(1 - \frac{103}{550}\right)$$

$$Q_{P\ (TRIAL)} = 910\text{ cfs}$$

c. Compute V_2 using Q_{P14}(TRIAL)

From Figure 3 determine stage for Q_{P14}(TRIAL)

Stage = 0.6 feet

X-area = (0.5)(0.6 ft)(2,080 + 2105 ft)
= 1256 ft²

$$V_2 = \frac{(1256\ ft^2)(3050\ ft)}{43,560\ ft^2/acre}$$

V_2 = 88 acre-ft

d. Average V_1 and V_2 and compute Q_{P14}

(1) $$Vavg = \frac{V_1 + V_2}{2}$$

$$V_{avg} = \frac{103\ ac\text{-}ft + 88\ ac\text{-}ft}{2}$$

Vavg = 95 acre-ft

(2) $$Q_{P14} = Q_{P13}\left(1 - \frac{Vavg}{S}\right)$$

$$Q_{P14} = (1,100\ cfs)\left(1 - \frac{95}{590}\right)$$

Q_{P14} = 920 cfs

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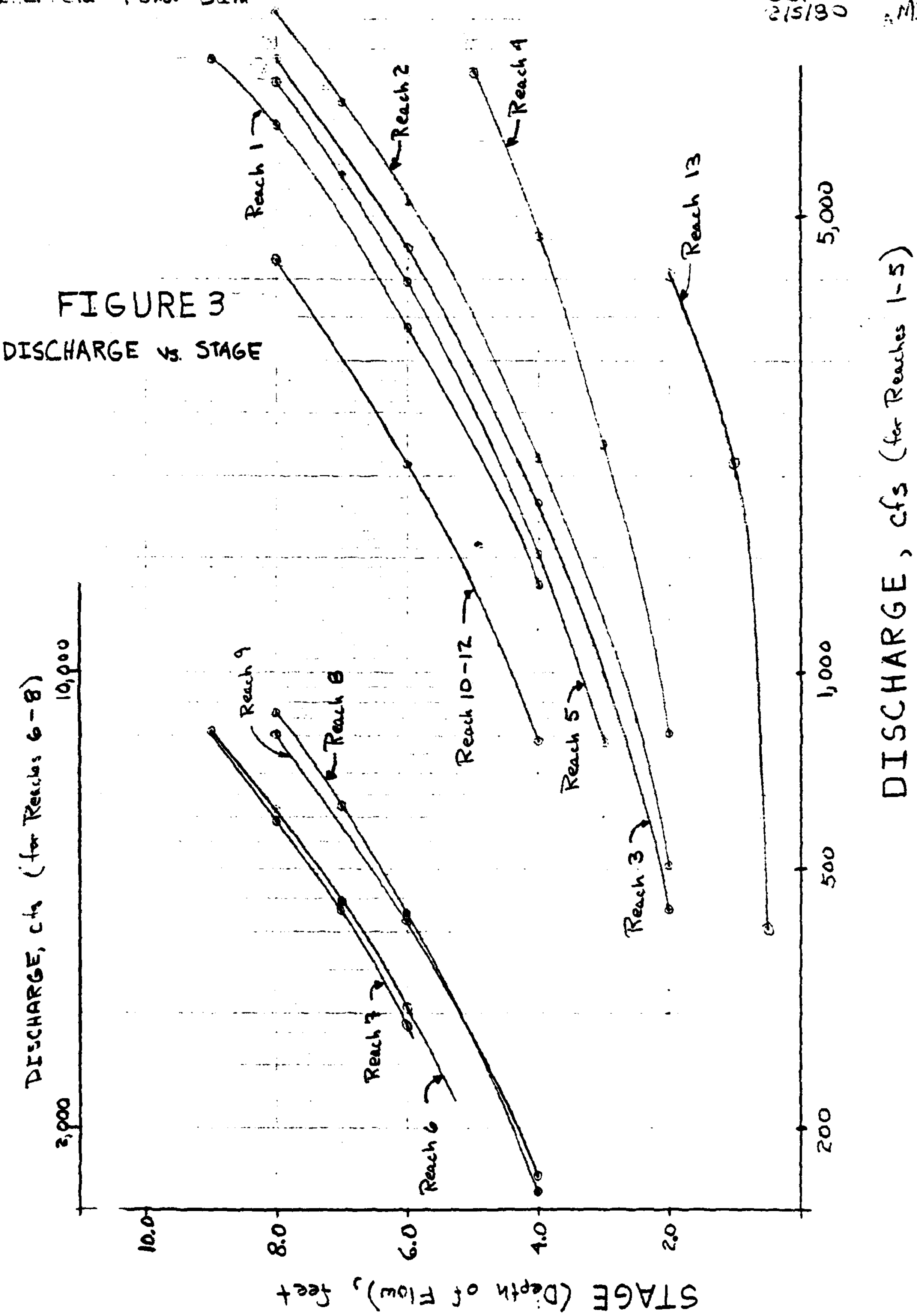
FIGURE 3

DISCHARGE vs. STAGE



Butterfield Pond Dam

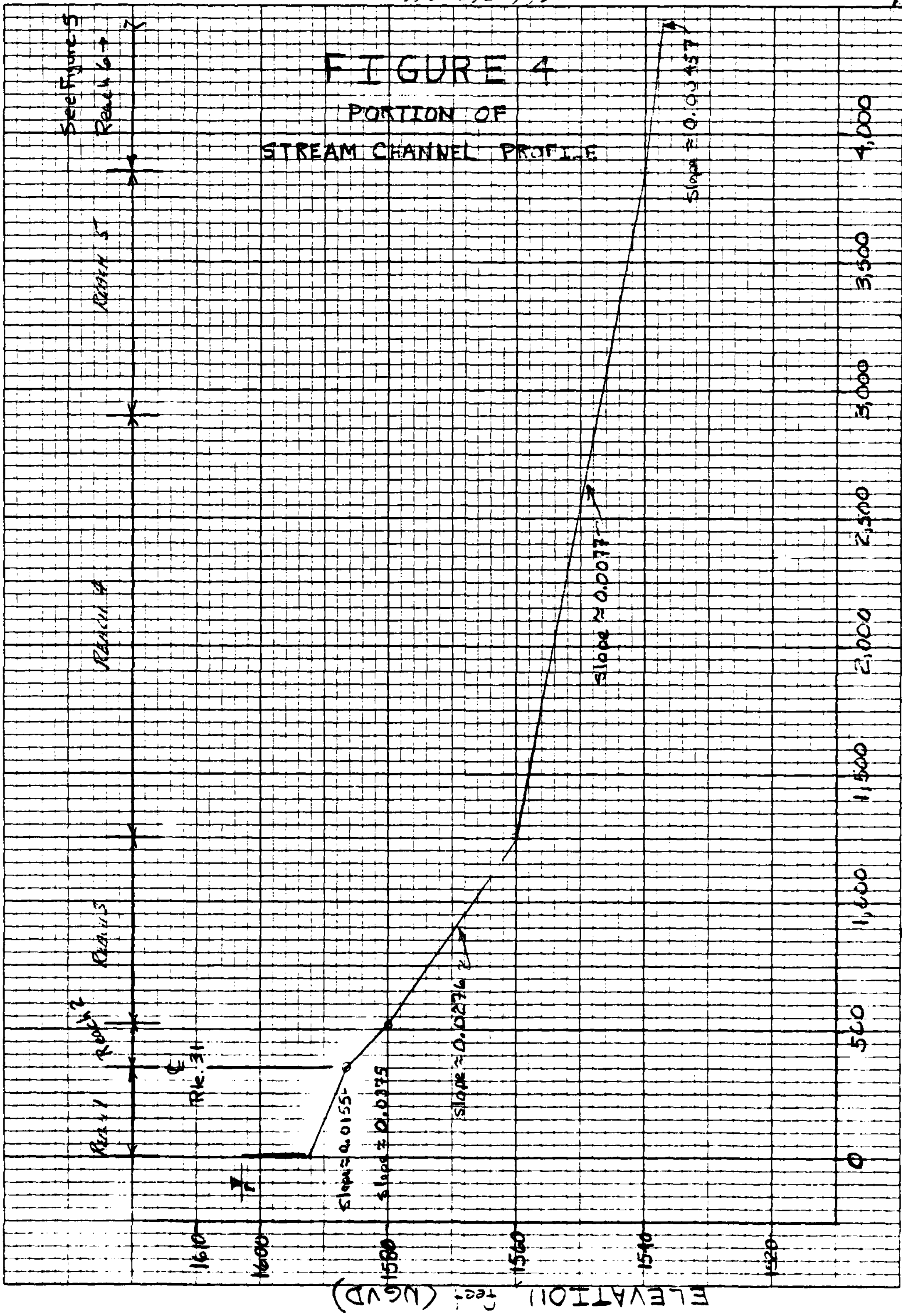


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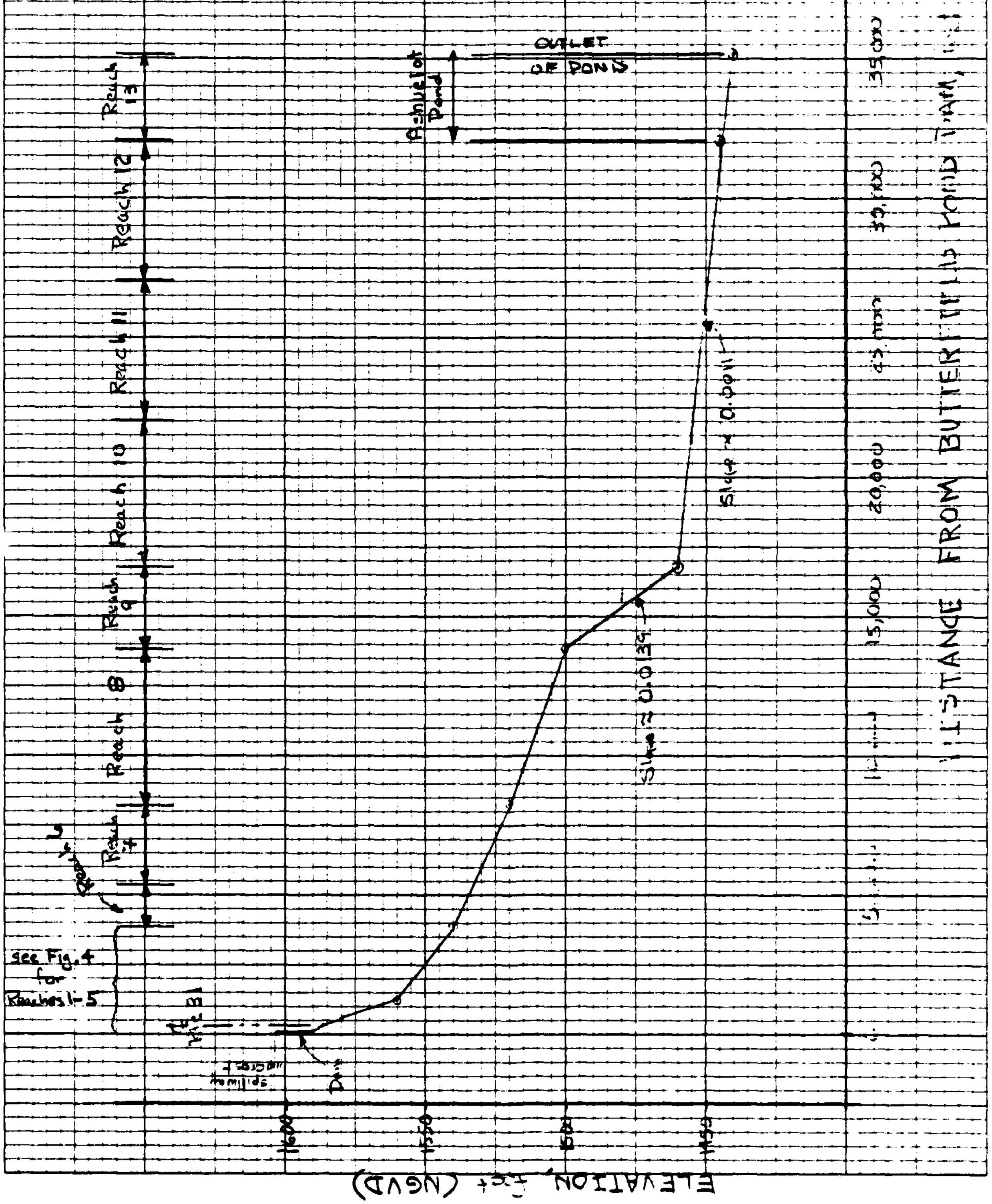
FIGURE 5

STREAM CHANNEL PROFILE



INVENTORY OF DAMS IN THE UNITED STATES

STATE	IDENTITY NUMBER	DIVISION	STATE	COUNTY	CONGR DIST.	STATE	COUNTY	CONGR DIST.	NAME	LATITUDE (NORTH)	LONGITUDE (WEST)	REPORT DATE DAY MO YR
NH	233	NED	NH	019	02				BUTTERFIELD POND DAM	4313.6	7207.1	24MAR80

POPULAR NAME	NAME OF IMPOUNDMENT
	BUTTERFIELD POND + MAY POND

REGION	BASIN	RIVER OR STREAM	NEAREST DOWNSTREAM CITY-TOWN-VILLAGE	DIST FROM DAM (MI.)	POPULATION
01	06	ASHUELOT RIVER	MARLOW	9	516

TYPE OF DAM	YEAR COMPLETED	PURPOSES	STRUCTURAL HEIGHT (FT.)	HYDRAULIC HEIGHT (FT.)	IMPOUNDING CAPACITIES MAXIMUM (ACRE-FT.)	IMPOUNDING CAPACITIES NORMAL (ACRE-FT.)
PGERCT	1934	R	13	13	590	465

DIST	OWN	FED R	PRV/FED	SCS A	VER/DATE
NED	N	N	N	N	

REMARKS

D/S HAS	SPILLWAY CREST LENGTH	SPILLWAY TYPE	SPILLWAY WIDTH (FT.)	MAXIMUM DISCHARGE (FT.)	VOLUME OF DAM (CY)	POWER CAPACITY INSTALLED (MW)	POWER CAPACITY PROPOSED (MW)	NO.	NAVIGATION LOCKS LENGTH (FT.)	WIDTH (FT.)	LENGTH (FT.)	WIDTH (FT.)	LENGTH (FT.)	WIDTH (FT.)	LENGTH (FT.)	WIDTH (FT.)
2	210	U	57	160	400											

OWNER	ENGINEERING BY	CONSTRUCTION BY
NH RES AND ECONOMIC DEV		

REGULATORY AGENCY DESIGN	CONSTRUCTION	OPERATION	MAINTENANCE
NH WATER RES BOARD	NH WATER RES BOARD	NH WATER RES BOARD	NH WATER RES BOARD

INSPECTION BY	INSPECTION DATE DAY MO YR	AUTHORITY FOR INSPECTION
S E A CONSULTANTS INC	06DEC79	P.L. 92 - 367

REMARKS

END

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