HOUSATONIC RIVER BASIN
ANSONIA, CONNECTICUT

QUILLINAN RESERVOIR DAM
CT 00024

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

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

February 1980

UNCLASSIFIED

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

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

DAMS, INSPECTION, DAM SAFETY,

Housatonic River Basin, Ansonia, Conn.
Quillinan Reservoir Dam

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

The dam built in 1880 and reconstructed in 1884, has a total length of 510 ft. and consists of a stone and mortar masonry gravity section with right and left earthfill embankments. The top of the masonry section has an elevation of 138.5 and the top of the embankments ranges in elevation from 138.0 to 140.0. The masonry section is 100 ft. long and 18± ft. in height above the streambed of Beaver Brook. The spillway is a 35.0 ft. long broad crested weir, is located at the center of the masonry section and has a crest elevation of 135.0. The outlet facilities are a 2.5 ft. by 2.5 ft. square conduit at the right end of the spillway and an 8 in. cast iron supply line at the center of the right section of embankment.

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DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
424 TRAPELO ROAD
WALTHAM, MASSACHUSETTS 02154

REPLY TO
ATTENTION OF:
NEDED-E

JUL 01 1980

Honorable Ella T. Grasso
Governor of the State of Connecticut
State Capitol
Hartford, Connecticut 06115

Dear Governor Grasso:

Inclosed is a copy of the Quillinan Reservoir Dam Phase I Inspection Report, which was prepared under the National Program for Inspection of Non-Federal Dams. The report is based upon a visual inspection, a review of past performance, and a preliminary hydrological analysis. A brief assessment is included at the beginning of the report.

The preliminary hydrologic analysis has indicated that the spillway capacity for the Quillinan Reservoir Dam would likely be exceeded by floods greater than 15 percent of the Probable Maximum Flood (PMF), the test flood for spillway adequacy. Our screening criteria specifies that a dam of this class which does not have sufficient spillway capacity to discharge fifty percent of the PMF, should be adjudged as having a seriously inadequate spillway and the dam assessed as unsafe, non-emergency, until more detailed studies prove otherwise or corrective measures are completed.

The term "unsafe" applied to a dam because of an inadequate spillway does not indicate the same degree of emergency as that term would if applied because of structural deficiency. It does indicate, however, that a severe storm may cause overtopping and possible failure of the dam, with significant damage and potential loss of life downstream.

It is recommended that within twelve months from the date of this report the owner of the dam engage the services of a professional or consulting engineer to determine by more sophisticated methods and procedures the magnitude of the spillway deficiency. Based on this determination, appropriate remedial mitigating measures should be designed and completed within 24 months of this date of notification. In the interim a detailed emergency operation plan and warning system should be promptly developed. During periods of unusually heavy precipitation, round-the-clock surveillance should be provided.

NEDED-E
Honorable Ella T. Grasso

I have approved the report and support the findings and recommendations described in Section 7, with qualifications as noted above. I request that you keep me informed of the actions taken to implement these recommendations since this follow-up is an important part of the non-Federal Dam Inspection Program.

A copy of this report has been forwarded to the Department of Environmental Protection, the cooperating agency for the State of Connecticut. This report has also been furnished to the owner of the project, Ansonia-Derby Water Company, Ansonia, Connecticut.

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

I wish to take this opportunity to thank you and the Department of Environmental Protection for the cooperation extended in carrying out this program.

Sincerely,

Max B. Scheider

MAX B. SCHEIDER
Colonel, Corps of Engineers
Division Engineer

HOUSATONIC RIVER BASIN
ANSONIA, CONNECTICUT

QUILLINAN RESERVOIR DAM
CT 00024

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

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

February 1980

BRIEF ASSESSMENT

PHASE I INSPECTION REPORT

NATIONAL PROGRAM OF INSPECTION OF DAMS

Name of Dam:	QUILLINAN RESERVOIR DAM
Inventory Number:	CT 00024
State Located:	CONNECTICUT
County Located:	NEW HAVEN
Town Located:	ANSONIA
Stream:	BEAVER BROOK
Owner:	ANSONIA-DERBY WATER COMPANY
Date of Inspection:	JANUARY 16, 1980
Inspection Team:	PETER M. HEYNEN, P.E.
	HECTOR MORENO, P.E.
	JAY A. COSTELLO
	MIRON PETROVSKY
	ROBERT JAHN

The dam, built in 1880 and reconstructed in 1884, has a total length of 510 feet and consists of a stone and mortar masonry gravity section (including the spillway) with right and left earth-fill embankments. The top of the masonry section has an elevation of 138.5 and the top of the embankments ranges in elevation from 138.0 to 140.0. The masonry section is 100 feet long and 18± feet in height above the streambed of Beaver Brook. The spillway is a 35.0 foot long broad crested weir, is located at the center of the masonry section (See Sheet B-1) and has a crest elevation of 135.0. The outlet facilities are a 2.5 foot by 2.5 foot square conduit at the right end of the spillway and an 8 inch cast iron supply line at the center of the right section of embankment. (See Sheet B-1).

Based upon the visual inspection at the site and past performance, the project is judged to be generally in poor condition. No evidence of instability was observed in the embankment, masonry section or appurtenant structures. However, there are areas which require monitoring and maintenance such as seepage at the right end of the masonry section, the uneven crest of the embankment and spalling of the concrete apron at the base of the spillway.

In accordance with the Army Corps of Engineers' Guidelines for size (Small) and hazard (High) classification of the dam, the test flood range to be considered is from one-half the Probable Maximum Flood (½ PMF) to the Probable Maximum Flood (PMF). The test flood for Quillinan Reservoir Dam will be considered equivalent to the ½ PMF. Peak inflow to the reservoir at the ½ PMF is 2,600 cubic feet per second (cfs); peak outflow is 2,500 cfs with the dam overtopped 1.1 feet. The spillway capacity (not including low point overflows) with the reservoir level to the top of the dam is 720 cfs, which is equivalent to 29% of the routed test flood outflow.

It is recommended that the owner retain the services of a registered professional engineer to perform a more detailed hydraulic/hydrologic analysis to determine the adequacy of the project discharge. Recommendations should be made by the engineer and implemented by the owner. Other items of importance are grading the top of the dam to eliminate low areas, inspection of the spillway and spillway apron during no flow conditions, seepage through the masonry sections and the effect of the fill at the downstream toe of the left embankment.

The above recommendations and further remedial measures presented in Section 7 should be instituted within one year of the owner's receipt of this report.

Peter M. Heynen, P.E.
Project Manager
Cahn Engineers, Inc.

STATE OF CONNECTICUT
PETER M. HEYNEN
NO. 9618
REGISTERED
PROFESSIONAL ENGINEER

Edgar B. Vinal, Jr., P.E.
Senior Vice President
Cahn Engineers, Inc.

STATE OF CONNECTICUT
EDGAR B. VINAL, JR.
No. 4198
REGISTERED
PROFESSIONAL ENGINEER

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

Richard J. DiBuono

RICHARD DIBUONO, MEMBER
Water Control Branch
Engineering Division

Aramast Mahtesian

ARAMAST MAHTESIAN, MEMBER
Geotechnical Engineering Branch
Engineering Division

Carney M. Terzian

CARNEY M. TERZIAN, CHAIRMAN
Design Branch
Engineering Division

APPROVAL RECOMMENDED:

Joe B. Fryar

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 inspection. 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 would necessarily represent the condition of the dam at some point in the future. Only through continued care and inspection can there be any chance that unsafe conditions will be detected.

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

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

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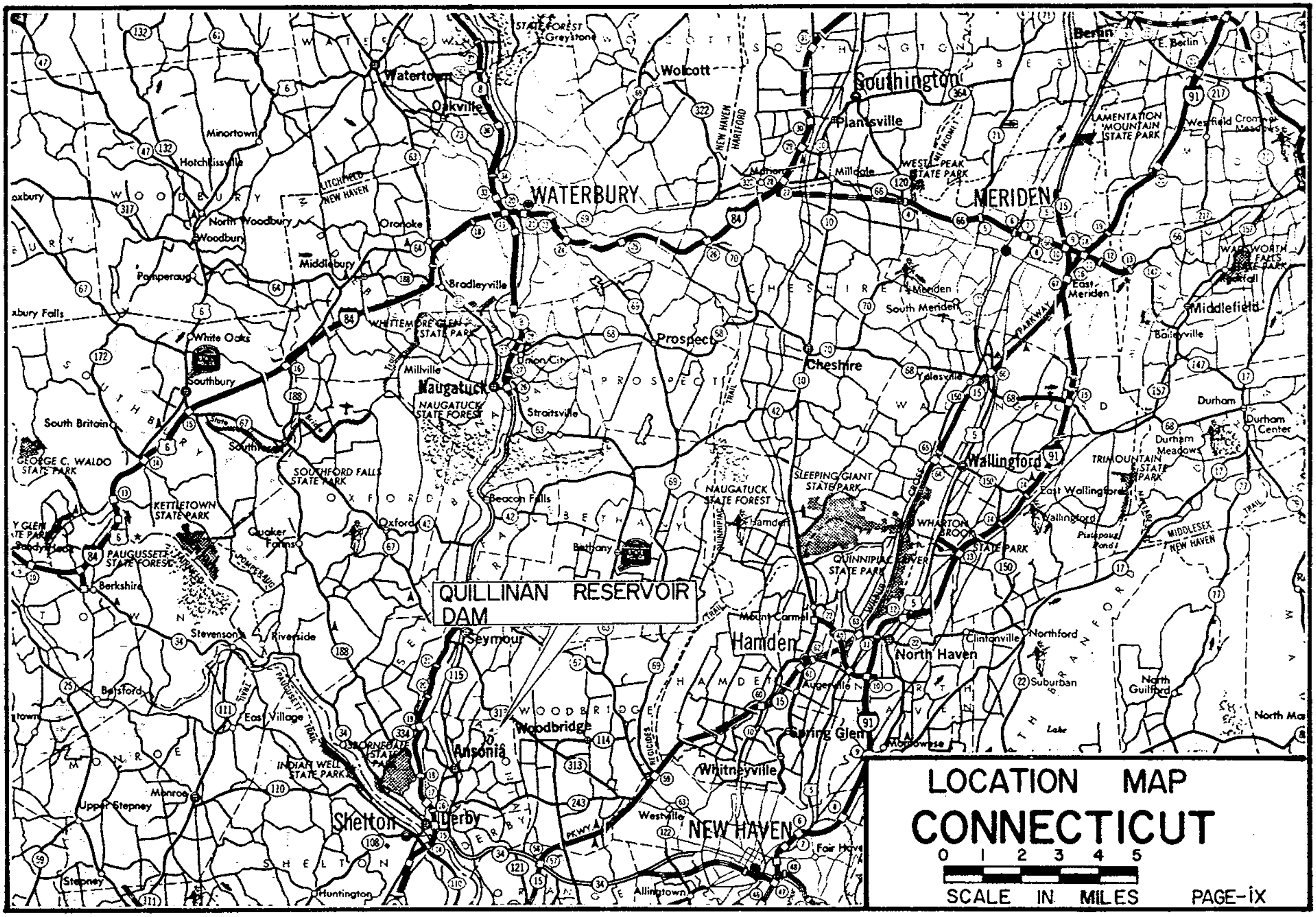
OVERVIEW PHOTO
(January, 1980)

US ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS.	NATIONAL PROGRAM OF INSPECTION OF NON-FED DAMS	Quillinan Reservoir Dam	Ansonia	DATE Feb., 1980
CAHN ENGINEERS INC. WALLINGFORD, CONN. ENGINEER		Beaver Brook	CONNECTICUT	CE #27660KD PAGE Viii


QUILLINAN RESERVOIR DAM
LOCATION MAP
CONNECTICUT
0 1 2 3 4 5
SCALE IN MILES
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WATERBURY
MERIDEN
NEW HAVEN
Southington
Plantsville
Watertown
Oakville
Wolcott
Naugatuck
Cheshire
Prospect
Wallingford
Hamden
North Haven
Seymour
Ansonia
Derby
Shelton
Woodbridge
Southbury
Middlebury
Middlefield
Durham
Berlin
Whitneyville
Westville
Oxford
Beacon Falls
Bethany
Mount Carmel
Spring Glen
Yalesville
Northford
Clintonville
East Village
Stevenson
Riverside
Monroe
Upper Stepney
Stepney
Botsford
Berkshire
Sandy Hook
Huntington
Allingtown
Fair Haven
NAUGATUCK STATE FOREST
SLEEPING GIANT STATE PARK
QUINNIPIAC RIVER STATE PARK
WHARTON BROOK STATE PARK
KETTLETOWN STATE PARK
PAUGUSSETT STATE FOREST
SOUTHFORD FALLS STATE PARK
WHITTEMORE GLEN STATE PARK
GEORGE C. WALDO STATE PARK
INDIAN WELL STATE PARK
OSBORNEDALE STATE PARK
LAMENTATION MOUNTAIN STATE PARK
WEST PEAK STATE PARK
TRIMOUNTAIN STATE PARK
WADSWORTH FALLS STATE PARK

PHASE I INSPECTION REPORT

QUILLINAN RESERVOIR DAM

SECTION I - 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. Cahn Engineers, Inc. has been retained by the New England Division to inspect and report on selected dams in the State of Connecticut. Authorization and notice to proceed were issued to Cahn Engineers, Inc. under a letter of October 15, 1979 from William E. Hodgson, Jr. Colonel, Corps of Engineers. Contract No. DACW 33-79-C-0059 has been assigned by the Corps of Engineers for this work.

b. Purpose of Inspection Program - The purposes of the program are to:

1. Perform technical inspection and evaluation of non-federal dams to identify conditions requiring correction in a timely manner by non-federal interests.

2. Encourage and prepare the States to quickly initiate effective dam inspection programs for non-federal dam.

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

c. Scope of Inspection Program - The scope of this Phase I inspection report includes:

1. Gathering, reviewing and presenting all available data as can be obtained from the owners, previous owners, the state and other associated parties.

2. A field inspection of the facility detailing the visual condition of the dam, embankments and appurtenant structures.

3. Computations concerning the hydraulics and hydrology of the facility and its relationship to the calculated flood through the existing spillway.

4. An assessment of the condition of the facility and corrective measures required.

It should be noted that this report does not pass judgement on the safety or stability of the dam other than on a visual basis. The inspection is to identify those features of the dam which need corrective action and/or further study.

1.2 DESCRIPTION OF PROJECT

a. Location - The dam is located on Beaver Brook in a rural area of the town of Ansonia, county of New Haven, State of Connecticut. The dam is shown on the Ansonia USGS Quadrangle Map having coordinates latitude W41°20.9' and longitude N73°07.1'.

b. Description of Dam and Appurtenances - The dam has a total length of 510 feet which is comprised of a 100 foot long stone and mortar masonry gravity section with a 360 foot section of earth embankment to the left and a 50 foot section of embankment to the right.

The top of the earth embankment is irregular, ranges in elevation from 138.0 to 140.0 and is 4 to 6 feet wide. The upstream slope has a grass cover and an inclination of 1.2 horizontal to 1 vertical (or flatter) above the water line, and flattens to 2 horizontal to 1 vertical with stone riprap protection below the water line. The downstream slope is inclined at 2 horizontal to 1 vertical and has a grass cover. A fill of concrete rubble, earth, scrap metal, tree stumps, etc. is being dumped by the owner along the downstream toe of the left section of embankment. This fill extends from the old concrete foundation near the center of the dam to 65± feet from the left end of the dam (See Sheet B-1).

The stone masonry section of the dam is part of the original dam built in 1880 (raised and rebuilt in 1884) and contains the spillway and low-level outlet conduit. The top of the masonry is at elevation 138.5, which is 18± feet above the streambed of Beaver Brook and 3.5 above the spillway crest.

The spillway is approximately at the center of the masonry section and is a 35 foot long broad crested weir. The crest is 5 feet wide and is at elevation of 135.0. Water flowing over the spillway drops free-fall approximatly 11 feet to a concrete apron at the base of the masonry section. The apron is 7 feet wide and extends 37 feet from the left spillway training wall to the conduit outlet channel at the right end of the spillway (See Sheet B-1).

The outlet facilities are a square low-level conduit at the right end of the spillway and a water supply line located 30± feet to the right of the spillway. The conduit is 2.5 feet by 2.5 feet and is controlled with a butterfly valve, which is operated by hand from the gate house situated directly above the conduit at the top of the masonry section. The outlet for the conduit is located at the base of the masonry section adjacent to the right spillway training wall. The supply outlet is an 8 inch cast iron pipe with two 8 inch sluice gate intakes located in a concrete gate tower, which is situated in the reservoir, 13± feet off shore (See Sheet B-1). The two sluice gates allow water into a screened intake well in the gate tower before the water flows to a pumping station just downstream from the dam. The supply line passes through an abandoned chlorinator and three valve chambers before reaching the pumping station (See Sheet B-1).

c. Size Classification - (SMALL) - The dam impounds 175 acre-feet of water with the reservoir level to the top of the dam which at elevation 138.5, is 18± feet above the streambed of Beaver Brook. According to recommended guidelines, a dam with this height and maximum storage is classified as small in size.

d. Hazard Classification - (HIGH) - If the dam were breached, there is potential for loss of life and extensive property damage just downstream where Beaver Brook passes through a fully developed section of Ansonia. Because of the minimal dissipation of the flood flow by channel storage, structures at street crossings will be overtopped through a major portion of the industrial and commercial zones of Ansonia. Also, there are several industrial buildings spanning Beaver Brook in the flood path, and flood waters will overtop a conduit section of the brook (4000± feet downstream from the reservoir) with potential flooding of a large shopping center.

e. Ownership - Ansonia-Derby Water Company
230 Beaver Street
Ansonia, Conn. 06401
Mr. Fredrick Elliott (Superintendent)
(203)-735-1888 (Business)
(203)-734-0288 (Home)

The original dam was owned and built by a Mr. Quillinan. After a flood in 1884, the dam was purchased and rebuilt by the Ansonia Water Company for use as a water supply facility. This company has now become the Ansonia-Derby Water Company.

f. Operator - Mr. William Clark (203)-734-6641

g. Purpose of Dam - Water Supply - After being rebuilt in 1884, the dam was used to store water for an ice house and other small businesses, as well as for water supply. Now however, the sole purpose is for water supply.

h. Design and Construction History - The following information is believed to be accurate based upon the plans and correspondence available. The original dam was built around 1880 by a Mr. Quillinan. A flood, March 24, 1884, substantially damaged this dam. The Ansonia Water Company purchased the property in 1884 and rebuilt the dam to its present configuration. There are no plans for the original dam, but the rebuilding and raising in 1884 are reported to be designed by a Mr. Hull.

i. Normal Operational Procedures - The butterfly valve at the low-level sluice is opened during periods of high water in the reservoir and is operated at least twice a year for maintenance. As of this date the dam has not been used for water supply since August 1979. The reservoir level is usually maintained at the spillway crest or elevation 135.0.

1.3 PERTINENT DATA

a. Drainage Area - 2.6 square miles of relatively undeveloped, rolling, wooded terrain (See Sheet D-1).

b. Discharge at Damsite - Discharge is over the spillway, through the low-level rectangular conduit and through the 8 inch supply line.

1. Outlet works (Conduits):

2-1/2 feet by 2-1/2 feet low-level conduit @ downstream invert el. 121.5	150 cfs (Head to top of dam)
8 inch supply line	Unknown

2. Maximum known flood @ damsite:	Dam overtopped 1955
3. Ungated spillway capacity @ top of dam el. 138.5:	720 cfs
4. Ungated spillway capacity @ test flood el. 139.6:	1100 cfs
5. Gates spillway capacity @ normal pool:	N/A
6. Gated spillway capacity @ test flood:	N/A
7. Total spillway capacity @ test flood el. 139.6:	1100 cfs
8. Total project discharge @ test flood el. 139.6:	2,500 cfs

c. Elevations (National Geogetic Vertical Datum based on assumed spillway elevation of 135.0 taken from Ansonia USGS Quadrangle Map, 1972)

1. Streambed @ toe of Dam:	120.5
2. Maximum tailwater:	Unknown
3. Upstream portal invert diversion tunnel:	N/A
4. Normal pool:	135.0
5. Full flood control pool:	N/A
6. Spillway crest (ungated):	135.0
7. Design surcharge (original design):	Unknown
8. Top of dam:	138.5 (Masonry section) 138.0± to 140.0± (Embankments)
9. Test flood surcharge:	139.6

d. Reservoir

1. Length of maximum pool:	1,800 ft.
2. Length of normal pool:	1,500 ft.
3. Length of flood control pool:	N/A

e. Storage

1. Normal pool:	123 acre-ft.
2. Flood control Pool:	N/A
3. Spillway crest pool:	123 acre-ft.
4. Top of dam:	175 acre-ft.
5. Test flood pool:	192 acre-ft.

f. Reservoir Surface

1. Normal pool:	13.3 acres
2. Flood control pool:	N/A
3. Spillway crest:	13.3 acres
4. Top of dam:	16 acres
5. Test flood pool:	17 acres

g. Dam

1. Type:	Masonry gravity section earth embankment
2. Length:	510 ft. total 100 ft. (Masonry) 410 ft. (Embankments)
3. Height:	18 ft.
4. Top width:	4 to 6 ft.
5. Side slopes:	1.2H to 1V (Upstream and above waterline) 2.5H to 1V (Upstream and below waterline) 2H to 1V (Downstream)
6. Zoning:	N/A
7. Impervious Core:	N/A
8. Cutoff:	N/A
9. Grout curtain:	N/A
10. Other:	N/A

h. Diversion and Regulating Tunne -N/A

i. Spillway

1. Type:	Broad-crested stone masonry
2. Length of weir:	35 ft.
3. Crest elevation:	135.0
4. Gates:	N/A
5. Upstream Channel:	Earthfill
6. Downstream Channel:	Vertical drop to natural streambed
7. General:	N/A

j. Regulating Outlets

Low-level conduit

1. Invert:	121.5 (downstream)
2. Size:	2.5' x 2.5'
3. Description:	Square opening at base of masonry section at right end of spillway
4. Control Mechanism:	Hand operated butterfly valve with gate stand located directly above in gate house
5. Other:	N/A

Supply outlet

1. Invert:	Unknown
2. Size:	8 inch
3. Description:	Cast iron
4. Control Mechanism:	Two 8 inch sluice gates with two hand operated gate stands and located at supply intake and gate tower
5. Other:	8 inch pipe extends to pumping station directly downstream, which pumps water to Fountain Hill Reservoir.

SECTION 2: ENGINEERING DATA

2.1 DESIGN

a. Available Data - The available data consists of 2 drawings and one inspection report. The drawings are available at the Ansonia-Derby Water Company and include a bathymetric map of the lake with a layout of the dam and buildings, dated 1915, and a drawing of the proposed dam dated April 30, 1884. The inspection report was prepared by A. M. MacKenzie, C.E. in April 1966, and is available at the Connecticut Department of Environmental Protection.

b. Design Features - The drawings and inspection report indicate the design features stated previously herein.

c. Design Data - There are no engineering values, assumptions, test results or calculations available for the original construction or subsequent rebuilding and raising of the dam.

2.2 CONSTRUCTION

a. Available Data - No information is available.

b. Construction Considerations - No information is available.

2.3 OPERATIONS

Lake level readings are taken daily at the dam. No formal operation records are known to exist.

2.4 EVALUATION

a. Availability - Existing data was provided by the owner and the State of Connecticut. The owner made the project available for visual inspection.

b. Adequacy The 1884 drawing of the dam was damaged in a flood in 1955, making parts of the drawing illegible. The limited amount of detailed engineering data available is inadequate to perform an in-depth assessment of the dam, therefore, the assessment of this dam must be based primarily on visual inspection, performance history, hydraulic computations of spillway capacity and approximate hydrologic judgements.

c. Validity - A comparison of record data and visual observations reveals no observable significant discrepancies in the record data.

SECTION 3: VISUAL INSPECTION

3.1 FINDINGS

a. General - The general condition of the project is poor. The inspection revealed several areas requiring maintenance and monitoring. At the time of the inspection, the reservoir level was at elevation 135.1, i.e. 3.4 feet below the crest of the dam with water flowing over the masonry spillway.

b. Dam

Crest - The crest of the earth embankment is very irregular and ranges from 0.5± feet below to 1.5± feet above the top of the masonry section of the dam (Photos 1, 2 and 4). Several paths (or ruts) from pedestrian traffic were noted on the left embankment.

Upstream Slope - Erosion was noted along the water line of the upstream slope of the left section of embankment (Photo 1 and 2). This erosion extends from the riprap protection (just below the water line) to 3± feet up the slope. A small area of erosion was also noted at the embankment to the right of the spillway. This area is 4± feet long and is located at the water line just opposite the supply intake and gate tower. Small brush was also observed on several portions of the upstream slope.

Downstream Slope - The downstream slope of the masonry section is covered with grass, weeds and brush, which is growing out between the masonry joints of the stone masonry (Photos 3 and 5). Seepage through the joints was noted at the lower portion of the masonry section and in several joints approximately 7 feet below the top of the masonry. The total seepage flow is approximately 0.5 to 1 gallon per minute (gpm). Many of the joints in the masonry are cracked and leaching at the areas of seepage, leaving the mortar soft and non-cohesive (Photo 6). The toe of the masonry section is also wet and covered with brush (Photo 3). The downstream slope of the left earth embankment has a grass cover with some brush. No cracks or seepage was observed.

The fill at the toe of the left embankment has a weed cover on the top and terminates to the right at a concrete foundation (Photos 4 and 5, Sheet B-1). For 65± feet at the left end of the dam, there is no fill dumped as yet. In this area the embankment is 8 to 10 feet in height with the downstream slope and toe covered with brush and trees.

Spillway - The downstream face of the masonry spillway is slightly deteriorated near the top. The spillway crest is in good condition. The spillway apron is severely damaged with spalling at the central and left portions (Photo 3). A scour area in the discharge channel approximately 2 feet wide and 2 feet deep was noted along the toe of the spillway apron.

The masonry spillway training walls are in fair to poor condition and have cracks in the mortar joints, some erosion (the left downstream wall) and displacement of the stone masonry (the right downstream wall).

c. Appurtenant Structures - The gate house at the right end of the spillway and low-level conduit, including the butterfly valve and valve stand, are in good condition (Photos 3 and 7).

The concrete gate tower for the supply intake is deteriorated; including exposed aggregate, severe spalling, and cracking (Photo 8).

The valve chamber and the (apparently abandoned) chlorinater chamber for water supply, located at the toe of the masonry section of the dam, are dry-laid stone structures. The floor in both chambers were not visible because of debris and siltation, but seepage was observed at the base of the chlorinator chamber. One seep, with a flow of 0.5 gpm, was in the right upstream corner, with the direction of flow nearly parallel to the dam. Another seep, with a flow of approximately 0.6 gpm, was located in the left downstream corner (Photo 9). There was an indication of hydraulic pressure in this seep with water flowing up out of the ground. Some deposits of brown silt sediments were also noted in this area. There is a 4 inch tile drain pipe at the base of the downstream wall of the chlorinater chamber. The drain was silted sufficiently to reduce by 1/2 the diameter of the pipe. Seepage water was flowing out of the chamber through this pipe but the actual direction of this drain could not be determined.

d. Reservoir Area - The area surrounding the reservoir is generally wooded and undeveloped.

e. Downstream Channel - The downstream channel runs in the natural streambed of the old Beaver Brook. It is moderately developed, steep-sided and wooded to the initial impact area (Photo 10).

3.2 EVALUATION

Based upon the visual inspection, the project is assessed as being generally in poor condition. The following features which could influence the future condition and/or stability of the project were identified.

1. Seepage through the masonry section of the dam, accompanied by leaching of the cement mortar joints, could weaken the masonry and create stability problems.

2. Seepage at the chlorinator chamber could be caused by permeable zones in the base of the masonry section and in the foundaton, or leaks from a damaged water supply line. The origin of the seepage should be investigated.

3. The earth embankment does not have sufficient erosion protection at the present time. Erosion along the length of the upstream slope could continue to expand and increase seepage through the embankment.

4. The deteriorated masonry of the spillway and training walls could result in erosion at the toe of the dam.

5. Scouring at the toe of the concrete spillway apron will lead to further deterioration of the apron if not repaired. Spalling of the concrete of this apron will lead to cracking of the aprong and possible erosion at the foundation of the masonry section.

6. The irregular crest elevation of the embankment sections of the dam could lead to erosion in these areas and along the toe if the dam should be overtopped.

SECTION 4: OPERATONAL PROCEDURES

4.1 REGULATING PROCEDURES

The 8 inch supply line has not been used in 7 months, but any water drawn through this outlet would be pumped to Fountain Hill Reservoir and distributed for water supply from there. The low-level conduit outlet is used to release water during excessively high water in the reservoir. The reservoir water level is normally maintained at elevation 135.0 and lake level readings are taken daily.

4.2 MAINTENANCE OF DAM

The grass is cut on the embankment several times a year. The dam is inspected by the operator on a daily basis. Any repair work is done by the Ansonia-Derby Water Company.

4.3 MAINTENANCE OF OPERATING FACILITIES

The butterfly valve at the low-level conduit and the two gates for supply intake are cleaned and serviced at least twice a year. The gate stands are also greased and checked at this time.

4.4 DESCRIPTION OF ANY FORMAL WARNING SYSTEM IN EFFECT

Watchmen present at the dam would contact Mr. Fredrick Elliott (Superintendent) should a problem arise at the dam. He would contact the Police Department, Fire Department or Civil Defense.

4.5 EVALUATION

The operation and maintenance procedures are generally fair, however there are areas requiring improvement. A formal program of operation and dam maintenance procedures should be implemented, including documentation to provide complete records for future reference. Other remedial operation and maintenance recommendations are presented in Section 7.

SECTION 5: HYDRAULIC/HYDROLOGIC

5.1 EVALUATION OF FEATURES

a. General - The watershed is 2.6 square miles of undeveloped, rolling, wooded terrain. The Quillinan Reservoir is the furthest downstream in a series of 3 reservoirs along Beaver Brook. The cumulative watershed for each of the reservoirs is as follows: Peat Swamp Reservoir - 0.52 square miles, Middle Reservoir -0.57 square miles, and Quillinan Reservoir - 2.6 square miles.

The Quillinan Dam is a masonry gravity structure, which includes a masonry spillway, and adjacent earth embankments. The dam is basically a low surcharge storage - high spillage project used for water supply storage. The storage that is available will reduce the Probable Maximum Flood (PMF) from 5,200 cfs to 5,000 cfs, and the ½ PMF from 2600 cfs to 2500 cfs.

b. Design Data - No computatons could be found for the original dam construction or the raising and rebuilding of the dam in 1884.

c. Experience Data - The original dam, built in 1880, was breached and partially removed by a flood on March 24, 1884. At this time the present structure was built.

d. Visual Observatons - The masonry dam appears in sound condition and the spillway free of debris, however the embankments have an irregular crest profile (Appendix D-4) and are rutted from trespassing.

e. Test Flood Analysis - Based upon the Army Corps of Engineers' "Preliminary Guidance for Estimating Maximum Probable Discharge" dated March 1978, the watershed classification (rolling) and area (2.6 square miles), a Probable Maximum Flood (PMF) of 5200 cfs, or 2000 cfs per square mile (CSM) is expected at the dam site. In accordance with the size (Small) and hazard (High) classification, the test flood range to be considered is from the ½ PMF to the PMF. The test flood for Quillinan Reservoir Dam is considered to be equivalent to the ½ PMF.

Peak inflow to the Reservoir at the ½ PMF is 2600 cfs and the peak outflow is 2500 cfs (Appendix D-2) with the masonry section of the dam overtopped by 1.1 feet (elevation 139.6) and the earth embankment sections overtopped by an average of 0.6 feet (Appendix D-7, D-15). The spillway capacity with the reservoir level to the top of the dam is 720 cfs, which is 29% of the outflow. The outlet discharge capacity (based on head to top of dam) of the low-level conduit is estimated to be 150 cfs. This capacity is not included in the peak outflow computations.

Peak inflow to the reservoir at the PMF is 5200 cfs and peak outflow is 5000 cfs with the masonry section of dam overtopped by 2.0 feet (elevation 140.5) and the earth embankment sections overtopped by an average of 1.5 feet.

f. Dam Failure Analysis - The dam failure analysis is based on the Army Corps of Engineers' "Rule of Thumb Guidance for Downstream Dam Failure Hydrographs" April, 1978. Peak outflow before failure of the dam would be about 720 cfs and the peak failure outflow from the dam breaching would total about 5,000 cfs. A breach of the dam would result in a rise of about 4.4 feet in the water level of the stream at the initial impact area, which corresponds to an increase in the water level from a depth of 4.2 feet just before the breach to a depth of 8.6 feet shortly after the breach. Because of the minimal dissipation of flood waters by channel storage, structures at street crossings will be overtopped through a major portion of the industrial and commercial zones of Ansonia. Industrial buildings spanning Beaver Brook would be jeopardized upon failure of the dam, as well as overflowing of a conduit section of brook 4000 feet downstream with potential flooding of a large shopping center in this area.

SECTION 6: STRUCTURAL STABILITY

6.1 EVALUATION OF STRUCTURAL STABILITY

a. Visual Observation - The visual inspection did not reveal any indications of immediate stability problems. There are areas of seepage, deterioration, and erosion, as described in Section 3, however they are not considered stability concerns at the present time.

b. Design and Construction Data - The drawings and data available and listed in Appendix B were not sufficient to perform an in-depth stability analysis of the dam. No engineering assumptions, data or calculations could be found for the original design of the dam.

c. Operating Records - The operating records available do not include any indication of stability problems at the dam since it's reconstruction in 1884.

d. Post Construction Changes - The only indication of post-constructon changes since the project was re-built in 1884 is a fill along the downstream toe of the embankment and the addition of a concrete apron at the base of the spillway. The dumping of this fill has been in progress for 12$\pm$ years.

e. Seismic Stability - The project is in Siesmic Zone 1 and according to the Recommended Guidelines, need not to be evaluated for seismic stability.

SECTION 7: ASSESSMENT, RECOMMENDATIONS AND REMEDIAL MEASURES

7.1 PROJECT ASSESSMENT

a. Condition - Based upon the visual inspection of the site and past performance, the project appears to be in poor condition. No evidence of immediate structural instability was observed in the dam, spillway or appurtenant structures. However, the masonry section and embankments are generally in poor condition with areas which require maintenance, repair and monitoring.

Based upon the Army Corps of Engineers' "Preliminary Guidance for Estimating Maximum Probable Discharge" dated March, 1978, and hydraulic/hydrologic computations, the peak inflow to the reservoir at test flood is 2,600 cubic feet per second (cfs) and the peak outflow is 2,500 cfs with the dam overtopped 1.1 feet and the water to elevation 139.6. Based upon our hydraulic computations, the spillway capacity with the reservoir level to the top of the dam is 720 cfs, which is equivalent to approximatley 29% of the routed test flood outflow.

b. Adeuqacy of Information - The information available is such that an assessment of the condition and stability of the project must be based solely on visual inspection, past performance and sound engineering judgement.

c. Urgency - It is recommended that the measures presented in Section 7.2 and 7.3 be implemented within 1 (one) year of the owner's receipt of this report.

7.2 RECOMMENDATIONS

It is recommended that further studies be made by a registered professional engineer qualified in dam design and inspection pertaining to the following:

1. A detailed hydraulic/hydrologic analysis to determine the adequacy of the project discharge and existing outlet facilities. Recommendations should be made by the engineer and implemented by the owner.

2. An inspection of the 8 inch water supply pipe through the masonry section for possible leaks.

3. An inspection of the 2.5 foot by 2.5 foot conduit through the spillway for potential seepage.

4. The irregular crest of the left embankment should be graded to the design elevation of the structure and no lower than 138.5, the elevation of the stone masonry section. The right section of embankment should also be raised to elevation 138.5 to eliminate flow through this low area.

5. Repair of the concrete intake and gate tower for the supply line.

6. Origin and significance of seepage at the abandoned chlorinator chamber and location of the 4 inch drain pipe.

7. A comprehensive program for further investigation of the dam. Of particular importance are:

    a. Condition of the masonry spillway and the concrete apron when no water is flowing over the spillway. This should include investigation into the extent of the scouring at the toe of the apron and the affect of this erosion on the stability of the concrete apron.

    b. Effect of the fill at the toe of the left embankment on possible seepage through the dam and monitoring of this seepage.

    c. Development of a program to reduce or stop seepage through the masonry section of the dam.

7.3 REMEDIAL MEASURES

a. Operation and Maintenance Procedures - The following measures should be undertaken by the owner within the time period indicated in Section 7.1.c, and continued on a regular basis.

1. Round-the-clock surveillance should be continued by the owner during periods of heavy precipitation or high project discharge.

2. A formal program of operation and maintenance procedures should be instituted and fully documented to provide accurate records for future references.

3. A comprehensive program of inspection by a registered professional engineer qualified in dam inspection should be instituted on an annual basis.

4. Seepage quantities through the masonry section of the dam and in the chlorinater chamber should be monitored periodically to measure any changes in seepage. The 4 inch tile drain in the chamber should be cleaned.

5. Cracked masonry joints of the spillway training walls should be sealed to prevent further deterioration.

6. The concrete damage at the spillway apron should be repaired or the apron replaced. Erosion at the toe of the apron should be filled and riprapped.

7. Erosion along the upstream slope of the embankment should be filled and riprap protection placed to well above the water line.

8. Provide means for access to the supply intake tower.

9. The cutting of grass, brush and trees on the crest, slopes and toe of the masonry and earth embankment sections should be performed and continued as part of the routine maintenance procedure.

7.4 ALTERNATIVES

This study has identified no practical alternatives to the above recommendations.

APPENDIX A

INSPECTION CHECKLIST

VISUAL INSPECTION CHECK LIST
PARTY ORGANIZATION

PROJECT Quillinan Reservoir Dam

DATE: January 16, 1980

TIME: 9:00am - 1:30pm

WEATHER: Sunny, 34°F

W.S. ELEV. 135.1 U.S. ______ DN.S

PARTY:	INITIALS:	DISCIPLINE:
1. Peter M. Heynen	PMH	Geotechnical
2. Miron Petrovsky	MP	Geotechnical
3. Jay Costello	JC	Geotechnical
4. Hector Moreno	HM	Hydraulic/Hydrologic
5. Moshe Norman	MN	Survey
6.		

PROJECT FEATURE	INSPECTED BY	REMARKS
1. Masonry Dam	PMH, MP, JC, HM, MN	
2. Earthfill Embankment	PMH, MP, JC, HM, MN	
3. Gate House	PMH, JC	
4. Low-Level Outlet	PMH, MP, JC, HM	
5. Intake Gate Tower	PMH, MP, JC	
6. Chlorinator Chamber	PMH, MP, JC	
7. Masonry Spillway	PMH, MP, JC, HM, MN	
8.		
9.		
10.		
11.		
12.		

PERIODIC INSPECTION CHECK LIST

PROJECT Quillinan Reservoir Dam DATE Jan. 16, 1980

PROJECT FEATURE Masonry Dam BY PMH, MP, JC, HM, MN

AREA EVALUATED		CONDITION
DAM EMBANKMENT		
Crest Elevation		138.5
Current Pool Elevation		135.1
Maximum Impoundment to Date		Unknown
Surface Cracks		Some, d/s slope
Pavement Condition		N/A
Movement or Settlement of Crest		None observed
Lateral Movement		
Vertical Alignment		Appears good
Horizontal Alignment		
Condition at Abutment and at Concrete Structures		Good
Indications of Movement of Structural Items on Slopes		None observed
Trespassing on Slopes		
Sloughing or Erosion of Slopes or Abutments		Some erosion of masonry joints on d/s slope and erosion on u/s slope.
Rock Slope Protection-Riprap Failures		Deteriorated riprap on u/s slope
Unusual Movement or Cracking at or Near Toes		None observed
Unusual Embankment or Downstream Seepage		Seepage on d/s slope
Piping or Boils		None observed
Foundation Drainage Features		N/A
Toe Drains		
Instrumentation System		

PERIODIC INSPECTION CHECK LIST

Page A-3

PROJECT Quillinan Reservoir Dam DATE Jan. 16, 1980

PROJECT FEATURE Earthfill Embankment BY PMH, MP, JC, HM, MN

AREA EVALUATED		CONDITION
DIKE EMBANKMENT		
Crest Elevation		139.0 ±
Current Pool Elevation		135.1
Maximum Impoundment to Date		Unknown
Surface Cracks		None observed
Pavement Condition		N/A
Movement or Settlement of Crest		Very irregular crest
Lateral Movement		None observed
Vertical Alignment		Appears poor
Horizontal Alignment		Appears fair
Condition at Abutment and at Concrete Structures		Good
Indications of Movement of Structural Items on Slopes		N/A
Sloughing or Erosion of Slopes or Abutments		Erosion along u/s slope
Rock Slope Protection-Riprap Failures		Riprap displacement
Unusual Movement or Cracking at or Near Toes		None observed
Unusual Embankment or Downstream Seepage		None observed
Piping or Boils		None observed
Foundation Drainage Features		N/A
Toe Drains		N/A
Instrumentation System		N/A
Trespassing on Slopes		N/A

PERIODIC INSPECTION CHECK LIST

Page A-4

PROJECT Quillinan Reservoir Dam DATE Jan. 16, 1980

PROJECT FEATURE Gate House BY PMH, JC

AREA EVALUATED		CONDITION
OUTLET WORKS-CONTROL TOWER		
a) Concrete and Structural		
General Condition		Good
Condition of Joints		N/A
Spalling		Not observed
Visible Reinforcing		N/A
Rusting or Staining of Concrete		
Any Seepage or Efflorescence		None observed
Joint Alignment		N/A
Unusual Seepage or Leaks in Gate Chamber		Not observed
Cracks		Not observed
Rusting or Corrosion of Steel		N/A
b) Mechanical and Electrical		
Air Vents		
Float Wells		
Crane Hoist		N/A
Elevator		
Hydraulic System		
Service Gates		2.5'×2.5 butterfly valve, operable.
Emergency Gates		
Lightning Protection System		N/A
Emergency Power System		
Wiring and Lighting System		

PERIODIC INSPECTION CHECK LIST

PROJECT Quillinan Reservoir Dam DATE Jan. 16, 1980

PROJECT FEATURE Upper Gatehouse BY PMH, JC

AREA EVALUATED	CONDITION
OUTLET WORKS-OUTLET STRUCTURE AND OUTLET CHANNEL	2.5'×2.5' sluice in masonry
General Condition of Concrete	Good
Rust or Staining	None observed.
Spalling	Some
Erosion or Cavitation	Not observed
Visible Reinforcing	N/A
Any Seepage or Efflorescence	Not observed
Condition at Joints	
Drain Holes	N/A
Channel	
Loose Rock or Trees Overhanging Channel	Some debris
Condition of Discharge Channel	Fair

PERIODIC INSPECTION CHECK LIST

Page A-6

PROJECT Quillinan Reservoir Dam DATE Jan. 16, 1980

PROJECT FEATURE Intake Gate Tower BY PMH, MP, JC

AREA EVALUATED	CONDITION
OUTLET WORKS-CONTROL TOWER	
a) Concrete and Structural	
General Condition	Poor
Condition of Joints	N/A
Spalling	Extensive spalling and cracking
Visible Reinforcing	N/A
Rusting or Staining of Concrete	N/A
Any Seepage or Efflorescence	None observed
Joint Alignment	N/A
Unusual Seepage or Leaks in Gate Chamber	Not observed
Cracks	Some
Rusting or Corrosion of Steel	N/A
b) Mechanical and Electrical	
Air Vents	N/A
Float Wells	N/A
Crane Hoist	N/A
Elevator	N/A
Hydraulic System	N/A
Service Gates	2- 8" sluice gates, operable
Emergency Gates	N/A
Lightning Protection System	N/A
Emergency Power System	N/A
Wiring and Lighting System	N/A

PERIODIC INSPECTION CHECK LIST

PROJECT Quillinan Reservoir Dam DATE Jan. 16, 1980

PROJECT FEATURE Chlorinator Chamber BY PMH, MP, JC

AREA EVALUATED		CONDITION
OUTLET WORKS-OUTLET STRUCTURE AND OUTLET CHANNEL		Stone masonry Structure
General Condition of Concrete		Fair
Rust or Staining		N/A
Spalling		
Erosion or Cavitation		None observed
Visible Reinforcing		N/A
Any Seepage or Efflorescence		Seepage on floor
Condition at Joints		N/A
Drain Holes		4" tile pipe w/ extensive siltation
Channel		
Loose Rock or Trees Overhanging Channel		N/A
Condition of Discharge Channel		

PERIODIC INSPECTION CHECK LIST

PROJECT Quillinan Reservoir Dam DATE Jan. 16, 1980

PROJECT FEATURE Masonry Spillway BY PMH, MP, JC, HM, MN

AREA EVALUATED		CONDITION
OUTLET WORKS-SPILLWAY WEIR, APPROACH AND DISCHARGE CHANNELS		
a) Approach Channel		
General Condition		Good
Loose Rock Overhanging Channel		N/A
Trees Overhanging Channel		N/A
Floor of Approach Channel		Not observed
b) Weir and Training Walls		
General Condition of Concrete		Fair
Rust or Staining		N/A
Spalling		Deteriorated masonry joints on d/s face & training walls
Any Visible Reinforcing		N/A
Any Seepage of Efflorescence		Not observed
Drain Holes		N/A
c) Discharge Channel		
General Condition		Fair
Loose Rock Overhanging Channel		None observed
Trees Overhanging Channel		None observed
Floor of Channel		Natural streambed
Other Obstructions		Boulders, brush & dead trees

APPENDIX B

ENGINEERING DATA AND CORRESPONDENCE

Quillinan Reservoir Dam

Existing Plan

"Plan of Proposed Dam Across Beaver Brook"
Ansonia Water Company
Ansonia, Conn.
April, 1884
1 sheet

"Contour Map of Quillinan Reservoir"
Ansonia Water Company
Ansonia, Conn
1915
1 sheet

SUMMARY OF DATA AND CORRESPONDENCE

DATE	TO	FROM	SUBJECT	PAGE
April 8, 1966	State of Connecticut Water Resources Commission	A.M. McKenzie, C.E.	Inspection of Dam	B-3
April 5, 1972	File	Victor F. Galgowski Supt. of Dam Maintenance	Inspection of Dam	B-5
July, 1973	File	Connecticut Board for the Supervision of dams	Inventory Data	B-6

A. M. McKENZIE
CIVIL ENGINEER
M. AM. SOC. C. E.

HYDRAULICS
WATER SUPPLY
LAND DEVELOPMENT

1300 MAIN STREET
SOUTH MERIDEN, CONN.

April 8, 1966.

Water Resources Commission,
State of Connecticut,
State Office Building,
Hartford, 15,
Connecticut.

Ref: Quillinan and Fountain Lake
Reservoirs - Town of Ansonia.
Ansonia Quad.

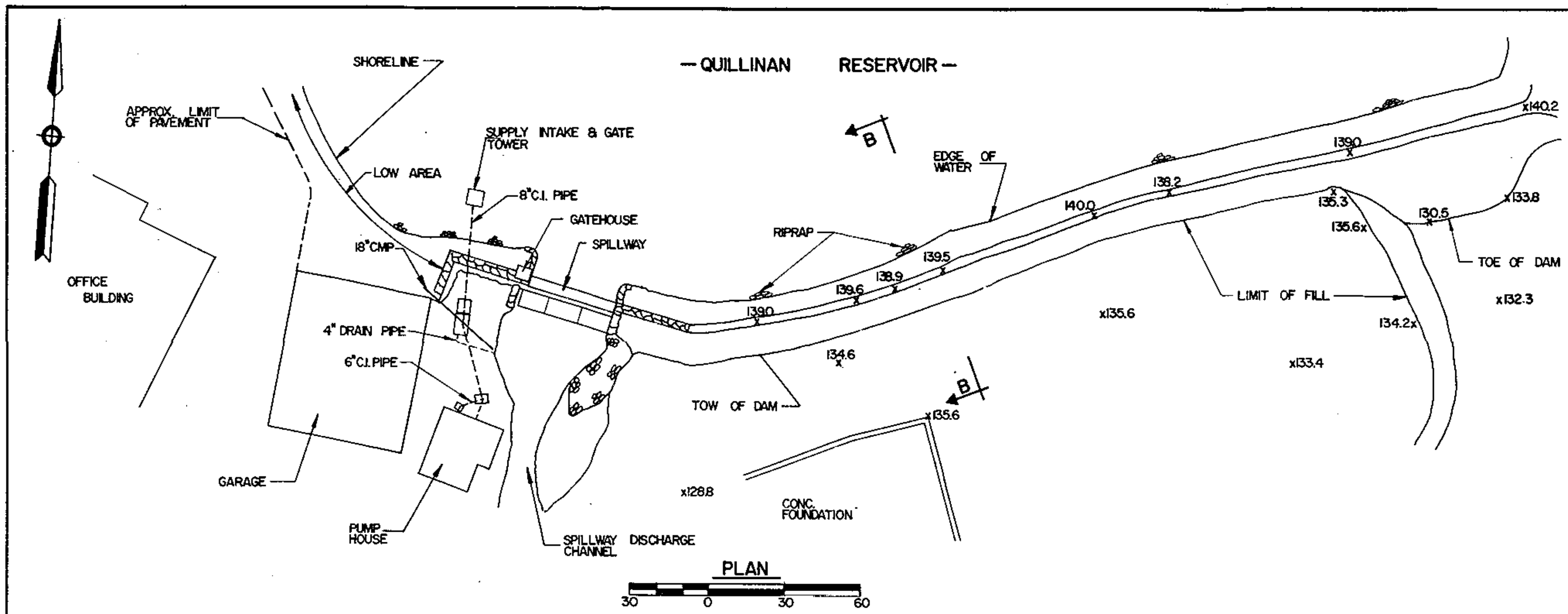
Gentlemen:

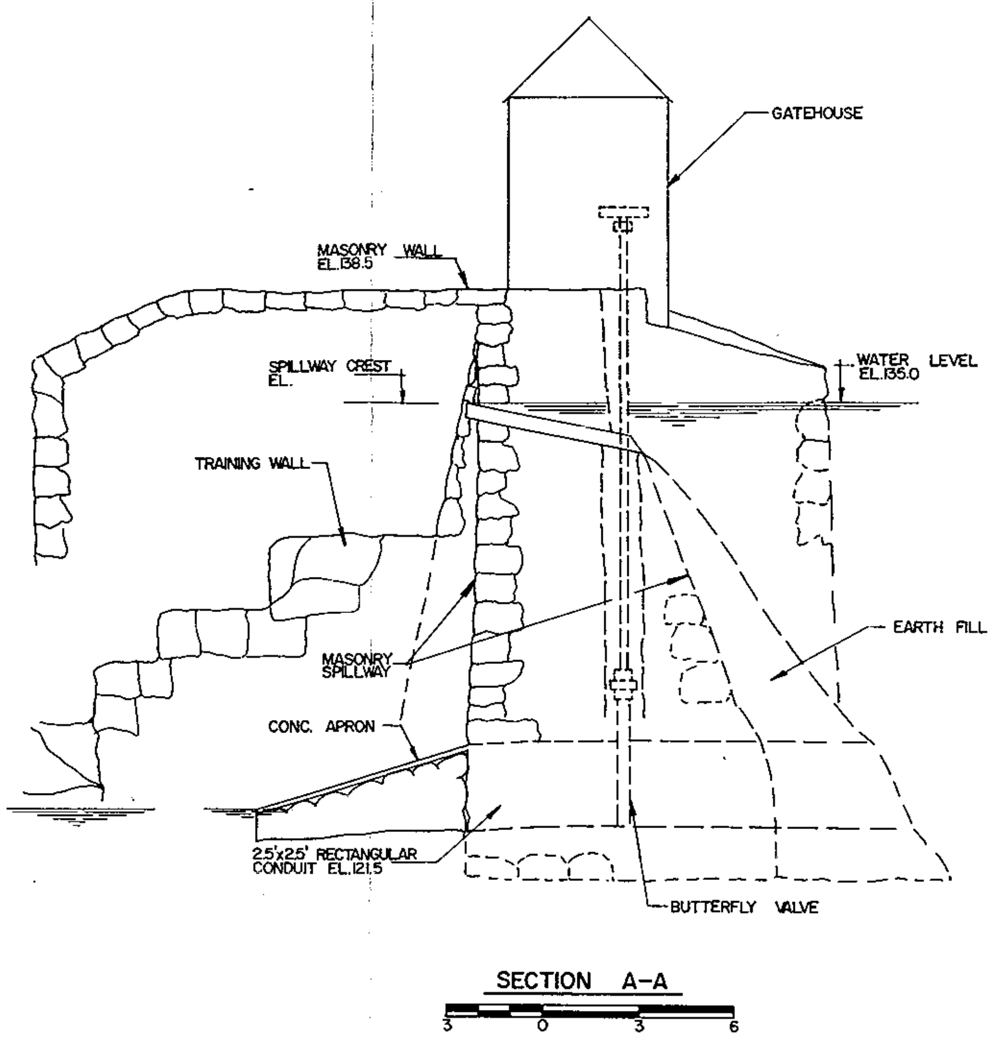
As instructed in your letter of Marcg 16, I have inspected the dams at the two reservoirs mentioned above and submit the following report.

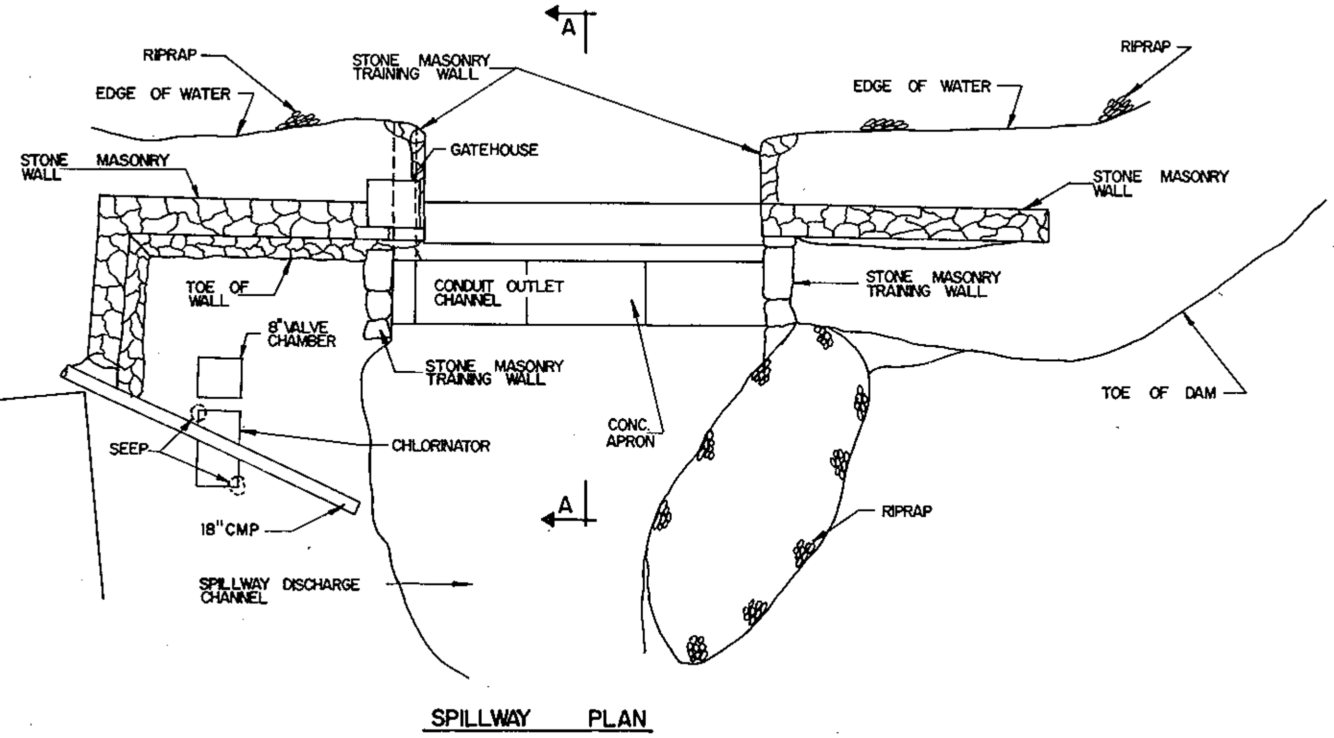
Quillinan Reservoir, a part of the City's water supply, is just east of Beaver Street on the east side of the City. The west end of the dam is not more than 100' from the Ansonia Water Company's office which faces on Beaver Street

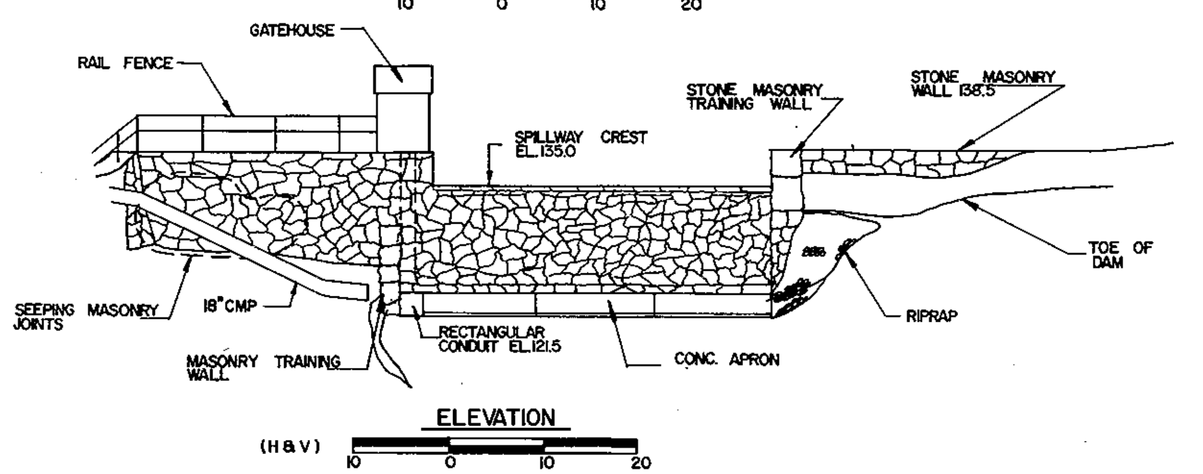
The dam is made up of a stone masonry section 100' long, including a 40' spillway, with a 50' earth embankment on the west end and a 360' long earth embankment on the east end. The maximum height of the masonry section is 20'; the spillway section is 3'-6" lower. The earth embankment varies in height from 0 to 11'. The downstream slope is about 1½:1 and the upstream slope is 2 : 1 or flatter and is protected with stone rip-rap to well below the water line. The earth fill is from 8' to 10' wide on top and is well sodded. At the east end the freeboard is up to 5'.

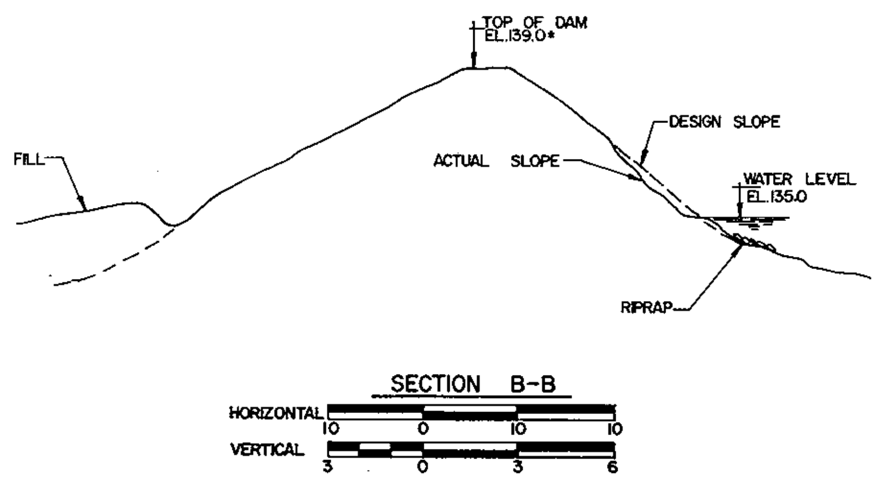
The masonry section of the dam is of a fair quality local stone, probably a type of granite. Most of the joints are well pointed with cement mortar tho there is a very slight seepage thru at several places - nothing of any importance. At the top the masonry is 3' thick with the downstream face battered slightly. There is very little of the upstream face visible but it is probably a gravity section. The dam was originally built in the early 1880s. A drain thru the dam, 2' x2', controlled by a gate upstream, can be seen in the lower left corner of the spillway section in photo # 7. On the inspection date there was a very small stream flowing over the spillway











NOTES

1. THIS PLAN WAS COMPILED FROM AN EXISTING DRAWING "PLAN OF PROPOSED DAM ACROSS BEAVER BROOK" BY THE ANSONIA WATER COMPANY, 1884 AND SUPPLEMENTARY SURVEY BY CAHN ENGINEERS, JANUARY 1980.
2. ALL ELEVATIONS ARE N.G.V.D. BASED ON AN ASSUMED SPILLWAY CREST ELEVATION EQUAL TO THE SURFACE ELEVATION OF 135.0 GIVEN ON THE ANSONIA U.S.G.S QUADRANGLE MAP, 1964 (P.R. 1972). ALL OTHER ELEVATIONS ARE REFERENCED TO THE ASSUMED SPILLWAY CREST ELEVATION.

CAHN ENGINEERS INC. WALLINGFORD, CONNECTICUT ENGINEER	U.S. ARMY ENGINEER. DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS.

NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS

PLAN AND SECTIONS

QUILLINAN RESERVOIR DAM

BEAVER BROOK — ANSONIA, CONNECTICUT

DRAWN BY	CHECKED BY	APPROVED BY	SCALE: AS NOTED	
M. Johnson	JAC	PMH	DATE: FEBRUARY 1980	SHEET B-1

A. M. McKENZIE
CIVIL ENGINEER
M. AM. SOC. C. E.

HYDRAULICS
WATER SUPPLY
LAND DEVELOPMENT

1300 MAIN STREET
SOUTH MERIDEN, CONN.

According to information from an official of the Water Company, this reservoir is used mainly for storage purposes and, immediately below the dam, is a pump station from which the water is transfered to the Fountain Hill Reservir, which is at a much higher elevation and from which it is distributed to the mains.

The water shed above Quillinan Reservoir is 2.3 sq. miles and includes Peat Swamp Reservoir with an area of about .14 sq. miles. It is estimated that a 100 year flood might produce a flow of 470 c.f.s. at the dam which would result in a head of a little over 2' on the spillway. This reasonable. The entire dam is in good condition and is well maintained. It is not considered that the dam might fail under any foreseeable condition and there is no hazard involved. An inspection of the dam should not be necessary at intervals of less than five years.

Fountain Hill Reservoir,
Towns of Seymour and Ansonia,
Ansonia Quad.

Fountain Lake Reservoir is just east of and close to Fountain Lake Road which, at that point in in the town of Seymour. The Town Line between Seymour and Ansonia passes thru the reservoir so that the dam is in both Towns. The normal water surface is at an elevation just above 230 and the water supplies, by gravity, a part of the City of Ansonia.

The dam is a flattened, "S" shape structure, in plan, of stone masonry backed by earth fill upstream and, at each end, there is a section entirely of earth fill. The stone is of fair quality and probably of local origin. The joints are well pointed up and on the downstream face there are indications of recent repairs, including pressure grouting where the pipe stubs have been left in place. Some of the grouting was done with an epoxy which the Water Company found to be very successful.

The overall length of the dam is 350' with a spillway section near the center 22' long. The thickness of the stone masonry at the top is 6' and the maximum height is 20', with the downstream face slightly battered. The spillway is 22" below the top of the dam. On top of the masonry, upstream, there has been poured a concrete wall 18" thick and 8" high and this low wall also extends along the wing walls at the spillway.

IM PUR STO 200

INTERDEPARTMENT MAIL

DATE	April 5, 1972
TO: File	DEPARTMENT: Water & Related Resources
FROM: Victor F. Galgowski, Supt. of Dam Maintenance	DEPARTMENT: Water & Related Resources
SUBJECT: Quillinan Reservoir Dam, Ansonia 1 N1.5B1.0	

This site was inspected on March 7, 1972 by the undersigned. In general the structure appeared to be sound. Slight seepage was noted in a area of the west embankment near the spillway abutment. Water depth over the spillway was four inches.

Victor F. Galgowski
Supt. of Dam Maintenance

VFG:ljg

C-24

STATE BOARD FOR THE SUPERVISION OF DAMS
INVENTORY DATA

NAME OF DAM OR POND Quillinan Reservoir

CODE NO. N 1.5 B1.0

LOCATION OF STRUCTURE:

Town Ansonia

Name of Stream Beaver Brook

U.S.G.S. Quad. Ansonia Long. 73-07-05 Lat. 41-20-53

OWNER: Ansonia Water Company

Address 230 Beaver Ave Ansonia ok 7/83

Telephone (1884)

1880

Pond Used For: Drinking Water DA 2.6 SM

Dimensions of Pond: Width Length Area ~~10-12 A~~ 11 Acres

Depth of Water below Spillway Level (Downstream) 15

Total Length of Dam 300 Length of Spillway 25

Height of Abutments above Spillway 3

Type of Spillway Construction stone

Type of Dike Construction stone and earth

Downstream Conditions Built up area

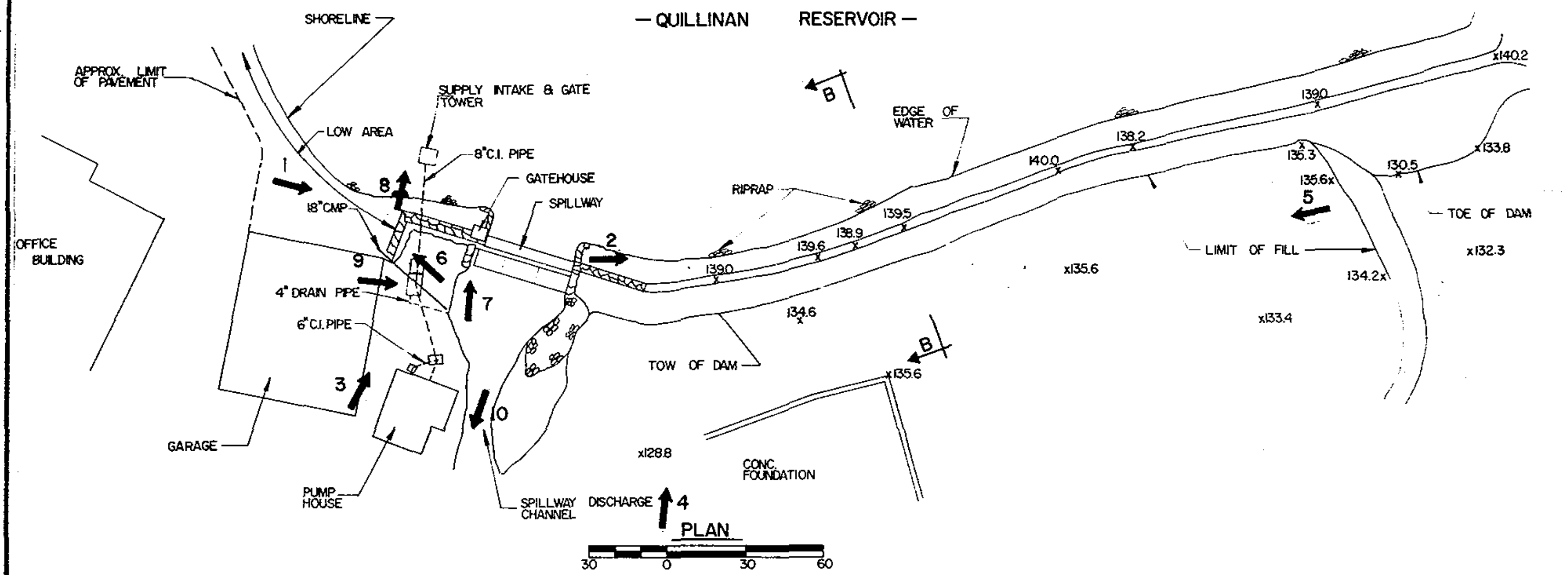
Summary of File Data

Remarks This is a structure of major importance. Board Member should inspect.

Class B.

APPENDIX C

DETAIL PHOTOGRAPHS



2 PHOTO NUMBER & DIRECTION

PHOTO LOCATION PLAN

QUILLINAN RESERVOIR DAM

SHEET C-1

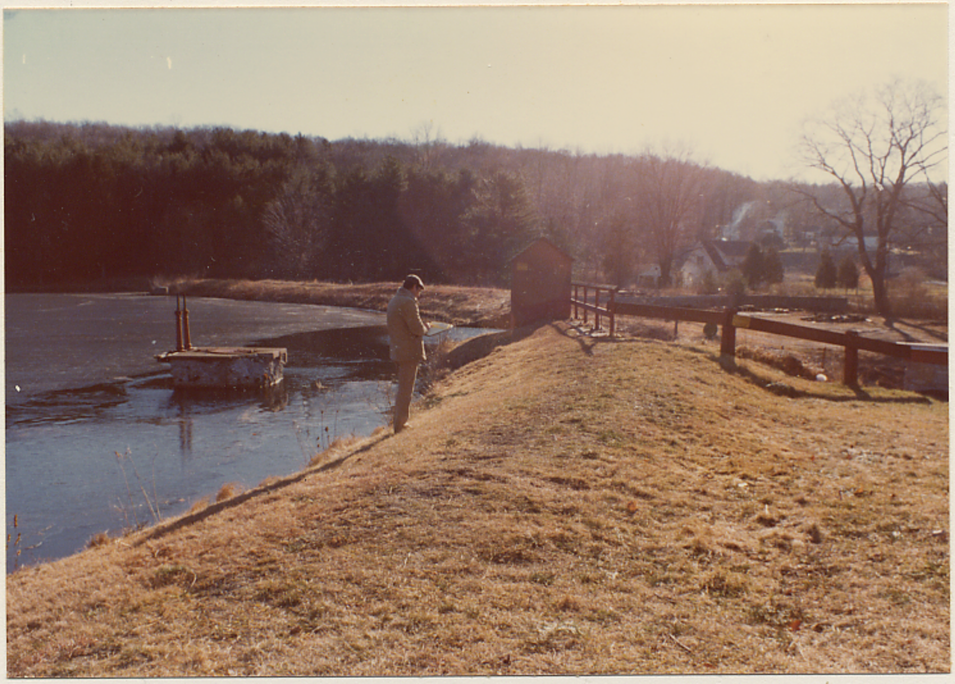


Photo 1 - Upstream slope and top of dam from right abutment. Gate structure for supply intake at left (Jan. 1980)

Photo 2 - Upstream slope and top of embankment left of spillway (Jan. 1980).

US ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS.	NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS	Quillinan Reservoir Dam Beaver Brook Ansonia, Ct.
CAHN ENGINEERS INC. WALLINGFORD, CONN. ENGINEER		CE# 27 660 KD DATE Feb. 80 PAGE C-1

Photo 3 - Downstream face of masonry section with spillway (Jan 1980)

Photo 4 - Downstream slope of embankment left of spillway.
Note irregularity of crest. (Jan. 1980)

Photo 5 - Fill dumped at toe of dam left of Spillway (Jan. 1980)

Photo 6 - Seepage in corner of downstream face of masonry section. (Jan. 1980)

US ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS.	NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS	Quillinan Reservoir Dam Beaver Brook Ansonia, Ct.
CAHN ENGINEERS INC. WALLINGFORD, CONN. ENGINEER		CE# 27 660 KD DATE Feb 1980 PAGE C-3

Photo 7 - Rectangular conduit at right side of spillway (Jan 1980)

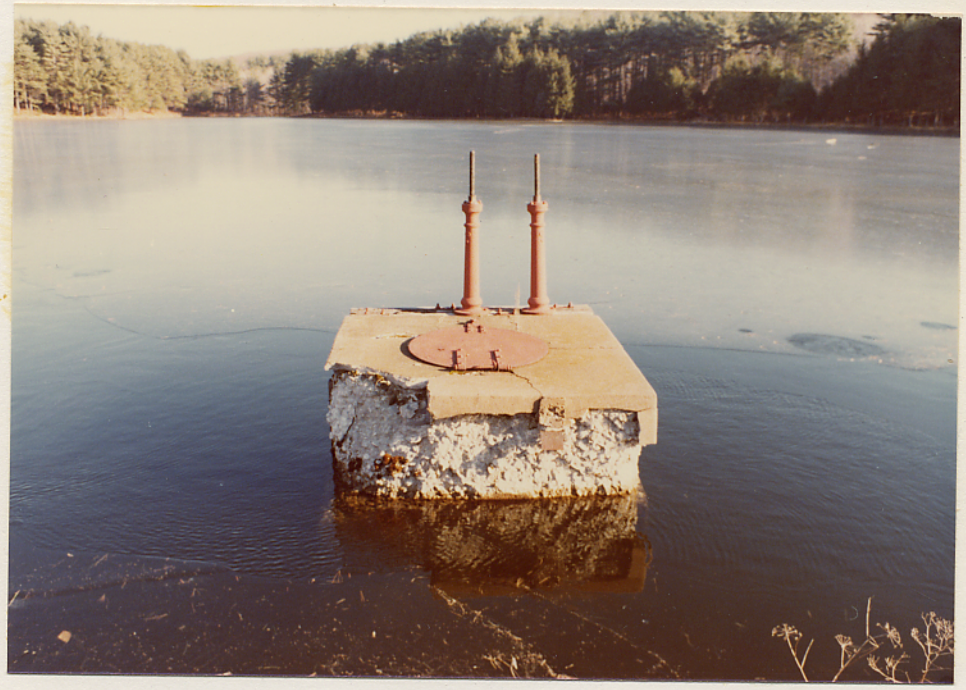


Photo 8 - Gate structure for supply intake (Jan 1980)

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Photo 9 - Chamber for chlorinator. Seepage at base of chamber in left downstream corner, right corner in photo (Jan 1980)

Photo 10 - Downstream channel from spillway (Jan 1980)

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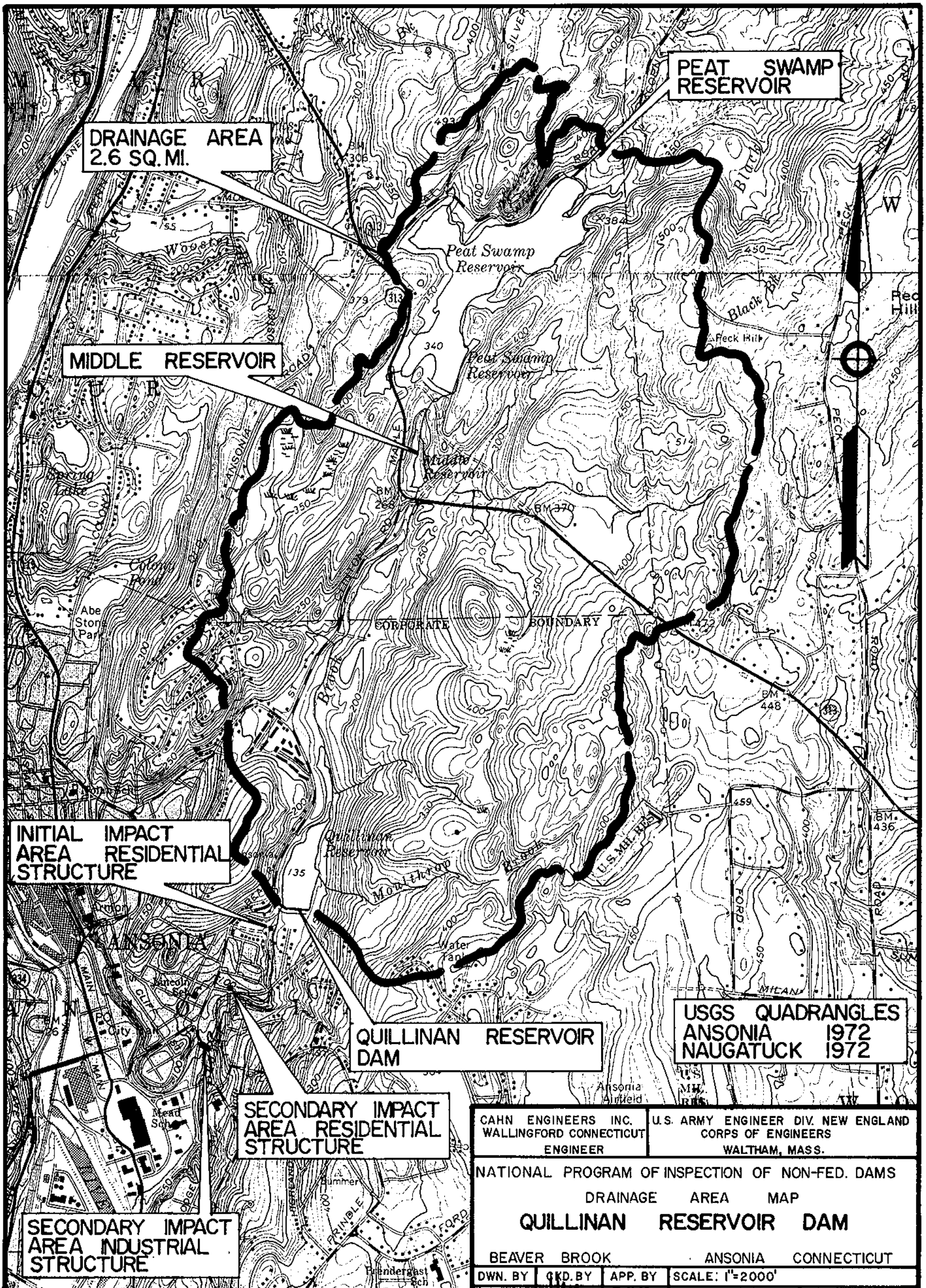
APPENDIX D

HYDRAULICS/HYDROLOGIC COMPUTATIONS


PEAT SWAMP RESERVOIR
DRAINAGE AREA 2.6 SQ. MI.
MIDDLE RESERVOIR
INITIAL IMPACT AREA RESIDENTIAL STRUCTURE
QUILLINAN RESERVOIR DAM
SECONDARY IMPACT AREA RESIDENTIAL STRUCTURE
SECONDARY IMPACT AREA INDUSTRIAL STRUCTURE
USGS QUADRANGLES
ANSONIA 1972
NAUGATUCK 1972
CAHN ENGINEERS INC.
WALLINGFORD CONNECTICUT
ENGINEER
U.S. ARMY ENGINEER DIV. NEW ENGLAND
CORPS OF ENGINEERS
WALTHAM, MASS.
NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS
DRAINAGE AREA MAP
QUILLINAN RESERVOIR DAM
BEAVER BROOK
ANSONIA CONNECTICUT
DWN. BY
CKD. BY
APP. BY
SCALE: 1"=2000'
DATE: FEBRUARY 1980
SHEET D-1

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Project INSPECTION OF NON-FEDERAL DAMS IN NEW-ENGLAND Sheet D-1 of 15
Computed By HW Checked By GAB Date 2/4/80
Field Book Ref. Other Refs. CE # 27-660-HA Revisions

HYDROLOGIC/HYDRAULIC INSPECTION

QUILLINAN RESERVOIR DAM, ANSONIA, CT.

I) PERFORMANCE AT PEAK FLOOD CONDITIONS:

1) PROBABLE MAXIMUM FLOOD (PMF):

a) WATERSHED CLASSIFIED AS "ROLLING" TO "MOUNTAINOUS"

b) WATERSHED AREA:

QUILLINAN RESERVOIR IS LOCATED ON BEAVER BROOK D/S FROM PEAT SWAMP AND MIDDLE RESERVOIRS. THE TOTAL WATERSHED IS SUBDIVIDED AS FOLLOWS*:

i) D.A. TO PEAT SWAMP RESERVOIR: $(DA)_{P.S.} \doteq 0.52^{sq\,mi}$
ii) INCREMENT TO MIDDLE RES.: $\Delta_{PS,M} \doteq .05^{sq\,mi}$
iii) D.A. TO MIDDLE RESERVOIR: $(DA)_M \doteq 0.57^{sq\,mi}$
iv) INCREMENT TO QUILLINAN RES.: $\Delta_{MQ} \doteq 2.02^{sq\,mi}$
v) TOTAL D.A. TO QUILLINAN RESERVOIR: $DA. = (DA)_Q \doteq 2.59^{sq\,mi}$
SAY, 2.6 sq mi

c) PEAK FLOODS (FROM NED-ACE GUIDELINES - GUIDE CURVES FOR PMF):

FROM PEAT SWAMP RESERVOIR DAM (CT 00088) PHASE I INSPECTION REPORT, AUGUST 1978, THE SURCHARGE STORAGE OF THIS RESERVOIR REDUCES THE PMF PEAK INFLOW OF $(Q_{P_1})_{P.S} = 1600^{cfs}$ TO $(\pm)(Q_{P_3})_{P.S.} = 640^{cfs}$. SIMILARLY, THE ½ PMF PEAK INFLOW TO PEAT SWAMP RESERVOIR,

*NOTE: DRAINAGE AREAS FROM CONN. DEP, BULLETIN No 1, 1972 (GAZETTEER OF NATURAL DRAINAGE AREAS) P.66 AND C.E. MEASURE ON USGS NAUGATUCK, CT. AND ANSONIA, CT. QUADRANGLE SHEETS.

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Project NON-FEDERAL DAMS INSPECTION Sheet D-2 of 15
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QUILLINAN RESERVOIR DAM

1, c - Cont'd) PEAK FLOWS

$(Q'_{P_1})_{PS} = 800^{CFS}$ IS REDUCED TO AN OUTFLOW OF (±) $(Q'_{P_3})_{PS} \cong 270^{CFS}$ BY SURCHARGE STORAGE.

HOWEVER, BECAUSE THE WATERSHED AREA REGULATED BY PEAT SWAMP IS ONLY (±) 20% OF THE QUILLINAN RESERVOIR'S D.A., THE EFFECT OF PEAT SWAMP IN REDUCING PEAK INFLOWS AT QUILLINAN RES. WILL BE RELATIVELY SMALL (EXPECTED TO BE MAX (±) 10%) AND CAN BE INCORPORATED BY ADJUSTMENT OF THE UNIT PMF (CSM) VALUE FOR THE TOTAL WATERSHED FROM THE NED-ACE GUIDE CURVES.

THEREFORE, NEGLECTING ALSO THE EFFECT OF THE MIDDLE RESERVOIR (W.S. AREA (±) 1.8 AC.), THE PEAK INFLOWS TO QUILLINAN RESERVOIR ARE ESTIMATED AS FOLLOWS:

i) FROM THE GUIDE CURVES, ADJUSTED CSM $\cong 2000^{CFS}/_{SQ.MI}$

∴ ii) PMF $\cong 2000 \times 2.6 \cong \underline{\underline{5200}}^{CFS}$

iii) ½ PMF $\cong \underline{\underline{2600}}^{CFS}$

2) SURCHARGE AT PEAK INFLOWS (PMF AND ½ PMF)

a) OUTFLOW RATING CURVE

i) SPILLWAY

QUILLINAN RESERVOIR DAM SPILLWAY IS A STONE MASONRY BROAD CRESTED TRAPEZOIDAL WEIR WITH INCLINED U/S FACE AND FREE FALL (OVER A LIP) D/S EDGE (SEE SKETCH ON P. D-3). THE CREST, (±) 5' BROAD, SLOPES UPWARD AT (±) 4.5H TO 1V TOWARDS THE

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Project NON-FEDERAL DAMS INSPECTION Sheet D-3 of 15
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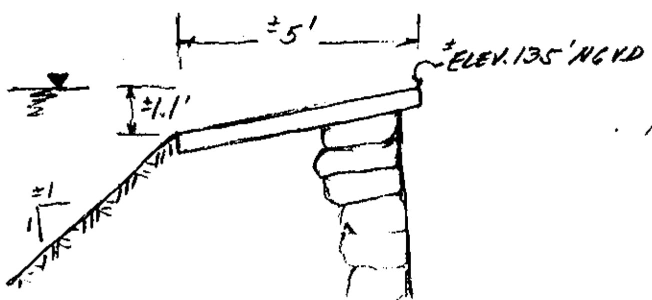
QUILLINAN RESERVOIR DAM

2,a - Cont'd) OUTFLOW RATING CURVE - SPILLWAY

U/S EDGE ((±)ELEV.* 135' NGVD). THE U/S FACE SLOPES AT (±) 1^H TO 1^V ((±)1.35^H TO 1^V ON THE ORIGINAL DWG. - 1884). IN PLAN, THE SPILLWAY LENGTH IS (±)35'. THE HEIGHT BETWEEN THE SPILLWAY CREST AND THE TOP OF THE MASONRY DAM IS (±) H≑3.5'.



SPILLWAY CROSS-SECTION

NOTE: DATA FROM C.E. FIELD OBSERVATIONS ON 1/9/80 (HWL/R.J.) AND DRAWINGS FURNISHED BY THE ANSONIA - DERBY WATER CO.

THEREFORE, ASSUMING THE SPILLWAY DISCHARGE COEFFICIENT C≑3.2 AND USING THE CREST ELEVATION 135' NGVD AS DATUM, THE SPILLWAY DISCHARGE IS APPROXIMATED BY:

$$Q_s \doteq \underline{110 H^{3/2}}$$

ii) EXTENSION OF THE RATING CURVE FOR SURCHARGES OVERTOPPING THE DAM AND/OR ADJACENT TERRAIN.

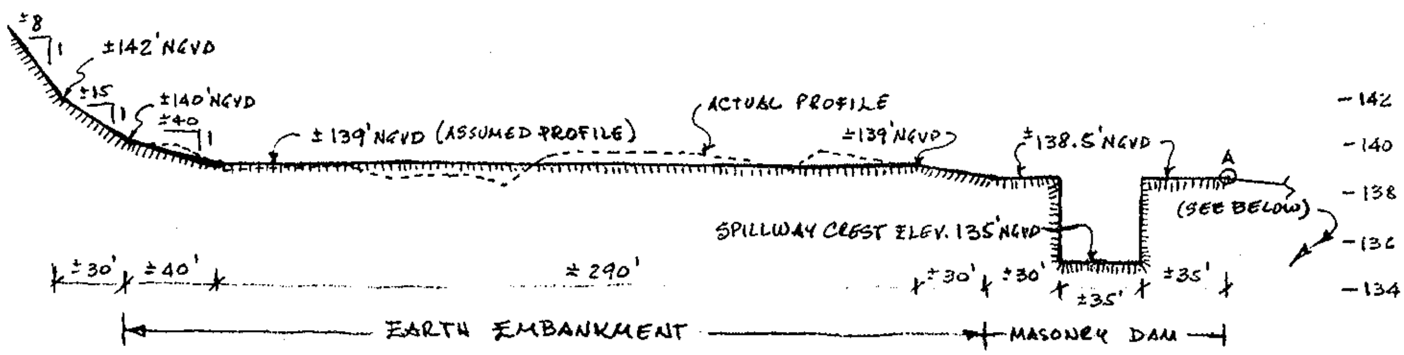
QUILLINAN RESERVOIR DAM IS ESSENTIALLY A STONE MASONRY DAM WITH A TOP ELEVATION OF (±) 138.5' NGVD, ABUTTED AT BOTH SIDES BY EARTH EMBANKMENTS OF VARIED (IRREGULAR) TOP ELEVATION. THE TOTAL LENGTH OF THE MASONRY PORTION AT (±) ELEV. 138.5' NGVD, IS (±) 65'.

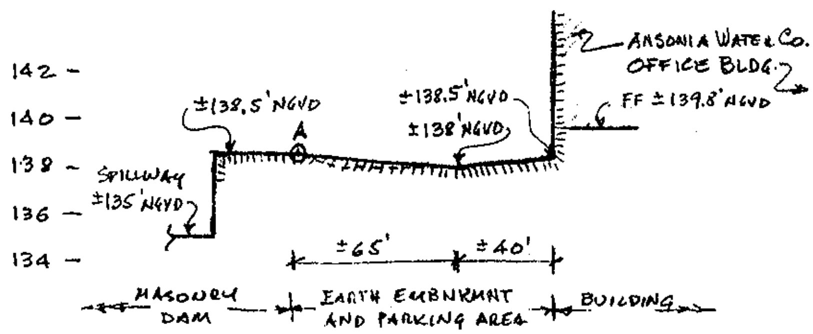
*NOTE: W.S. ELEV. 135' ON THE USGS, ANSONIA, CT. QUADRANGLE SHEET (PHOTOREVISED 1972) IS ASSUMED TO BE THE SPILLWAY CREST ELEVATION ON NATIONAL GEODETIC VERTICAL DATUM (NGVD) AND EQUIVALENT TO ELEV. 138' (DATUM UNKNOWN) SHOWN ON THE ANSONIA WATER CO. DWG. "CONTOUR MAP OF QUILLINAN RESERVOIR," DATED 1915.

QUILLINAN RESERVOIR DAM

2,a - (Cont'd) OUTFLOW RATING CURVE

THE TOP ELEVATION OF THE LEFT EMBANKMENT VARIES FROM (±)138' NGVD TO (±)140' NGVD AND PRESENTS WITHIN THESE LIMITS A VERY IRREGULAR OVERFLOW PROFILE (SEE SKETCH BELOW). THE RIGHT EMBANKMENT FALLS TO (±) ELEV. 138' NGVD ((±) 0.5' LOWER THAN THE TOP OF THE MASONRY DAM), AND TIES WITH THE PARKING AREA OF THE ANSONIA-DERBY WATER CO. BUILDING TO FORM THE OVERFLOW SECTION. IT WILL BE ASSUMED THAT THIS BUILDING (FF. ELEV. (±) 139.8 NGVD) CLOSES VERTICALLY, ABOVE GROUND ELEV. (±) 138.5' NGVD AT THE RIGHT SIDE OF THE OVERFLOW SECTION. TO THE LEFT, THE DAM AND ADJACENT TERRAIN RAISE GRADUALLY AS SHOWN BELOW ON THE OVERFLOW PROFILE ASSUMED FOR THIS COMPUTATION.





QUILLINAN RESERVOIR

- OVERFLOW PROFILE -

NOTE: DATA FROM C.E. FIELD MEASUREMENTS ON 1/9/80 BY HM & R.J.

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Project NON-FEDERAL DAMS INSPECTION Sheet D-5 of 15
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QUILLINAN RESERVOIR DAM

2,a-(Cont'd) OVERFLOW RATING CURVE

THEREFORE, ASSUMING $C=3.0$ FOR THE DAM AND ADJACENT TERRAIN OVERFLOWS AND EQUIVALENT LENGTHS FOR THE SLOPING TERRAIN, THE OVERFLOW CAN BE APPROXIMATED AS FOLLOWS:

1') SLOPING TERRAIN TO THE LEFT ABOVE ELEV. 142' NGVD:

$$(L_L')_1 \approx \underbrace{2/3(8)}_{\sim 5.33}(H-7) \quad \therefore \quad (Q_L')_1 \approx \underline{16\,(H-7)}^{5/2}$$

2') SLOPING TERRAIN TO THE LEFT BETWEEN ELEVS. 140' AND 142' NGVD:

$$(L_L')_2 \approx 2/3(15)(H-5) \quad \therefore \quad (Q_L')_2 \approx \underline{30\,(H-5)}^{5/2} \quad H \le 7'$$

$$(Q_L')_{2'} \approx 3 \times 30\,(H-5.47)^{3/2} \approx \underline{90\,(H-5.47)}^{3/2}$$

FOR $H > 7'$

3') SLOPING LEFT SIDE EMBANKMENT (DAM) ABOVE ELEV. 139' NGVD:

$$(L_D')_1 \approx 2/3(40)(H-4) \quad \therefore \quad (Q_D')_1 \approx \underline{80\,(H-4)}^{5/2} \quad H \le 5'$$

$$(Q_D')_{1'} \approx 3 \times 40\,(H-4.24)^{3/2} \approx \underline{120\,(H-4.24)}^{3/2}$$

FOR $H > 5'$

4') EMBANKMENT (DAM) ASSUMED AT ELEV. 139' NGVD:

$$(Q_D)_2 = 3 \times 290\,(H-4)^{3/2} = \underline{870\,(H-4)}^{3/2}$$

5') SLOPING EMBANKMENT TO THE LEFT OF MASONRY DAM, ABOVE ELEV. 138.5' NGVD.

$$(L_D')_3 \approx 2/3(60)(H-3.5) \quad \therefore \quad (Q_D')_3 \approx \underline{120\,(H-3.5)}^{5/2} \quad H \le 4'$$

$$(Q_D')_{3'} \approx 3 \times 30\,(H-3.62)^{3/2} = \underline{90\,(H-3.62)}^{3/2}$$

FOR $H > 4'$

QUILLINAN RESERVOIR DAM

2,a - Cont'd) OVERFLOW RATING CURVE

6') MASONRY DAM (ELEV. 138.5' NGVD):

$$(Q_D)_4 = 3 \times 65 (H-3.5)^{3/2} = \underline{\underline{195 (H-3.5)^{3/2}}}$$

7') SLOPING EMBANKMENT/PARKING AREA BETWEEN ELEVS. 138.5' AND 138' NGVD:

$$L'_R \approx \tfrac{2}{3}(210)(H-3) \quad \therefore \quad Q'_R \approx \underline{\underline{420(H-3)^{5/2}}} \quad H \leq 3.5'$$

$$Q'_{R'} = 3 \times 105 (H-3.12)^{3/2} = \underline{\underline{315 (H-3.12)^{3/2}}}$$

FOR $H > 3.5$

THEREFORE, THE TOTAL OUTFLOW RATING CURVE IS APPROXIMATED BY:

$$Q \approx 110 H^{3/2} + 16 (H-7)^{5/2} + (Q'_L)_{2*} + (Q'_D)_{1*} + 870 (H-4)^{3/2} + \\ + (Q'_D)_{3*} + 195 (H-3.5)^{3/2} + Q'_{R*}$$

WHERE THE (*) TERMS ARE GIVEN BY THE EQUATIONS (2'); (3'); (5') AND (7') ABOVE AS APPLICABLE TO THE SURCHARGE H FOR WHICH Q IS TO BE DETERMINED

THE RESULTING OUTFLOW RATING CURVE FOR QUILLINAN RESERVOIR IS PLOTTED ON NEXT PAGE (p. D-7).

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Project NON-FEDERAL DAMS INSPECTION — Sheet D-7 of 15
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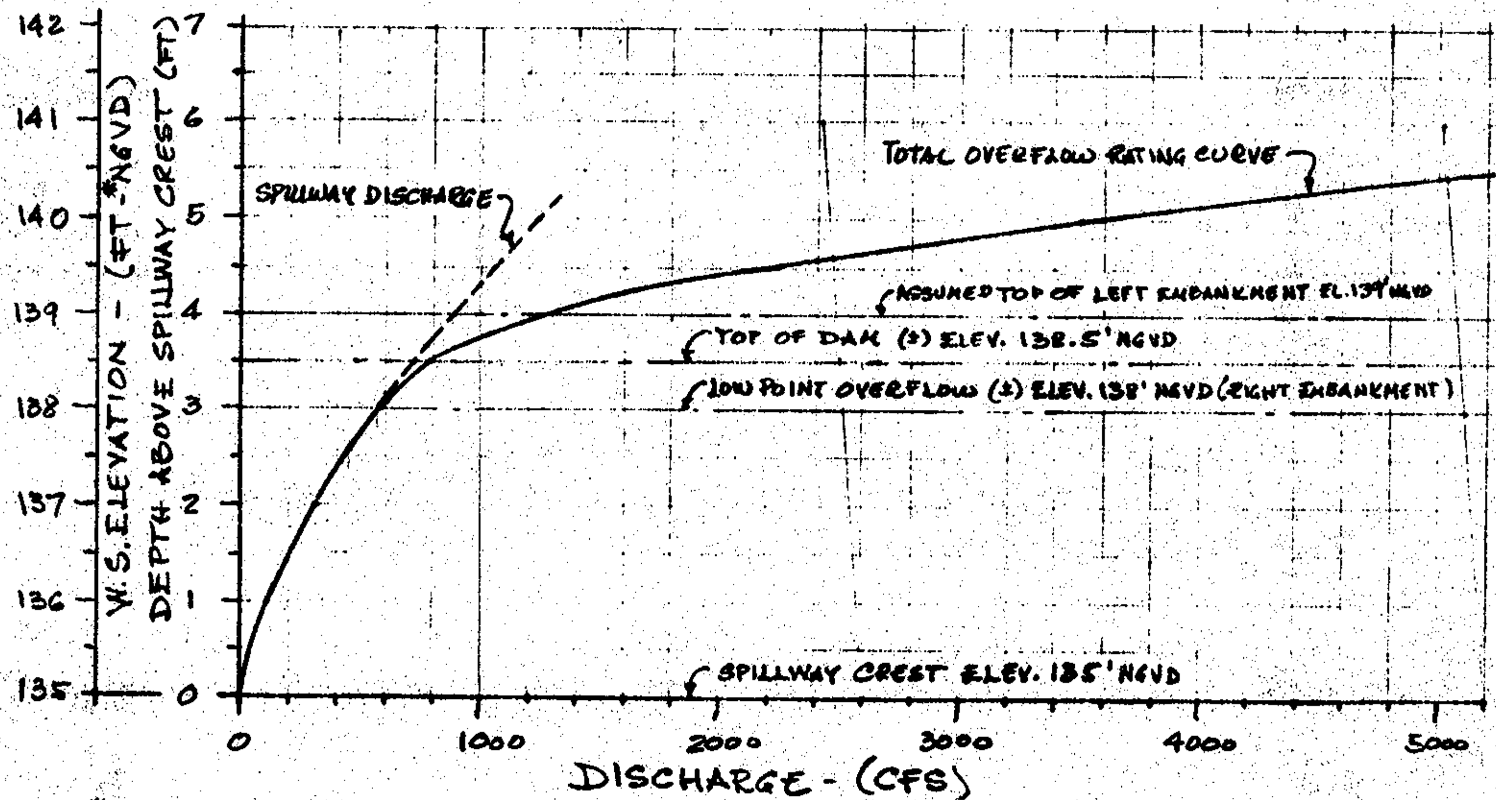
QUILLINAN RESERVOIR DAM

2, a - Cont'd) OUTFLOW RATING CURVE



* SEE NOTE ON DATUM - P. D-3

b) SURCHARGE HEIGHT TO PASS PEAK INFLOWS (Q_{P_1} & Q'_{P_1})

i) @ Q_{P_1} = PMF ≈ 5200 cfs H_1 ≈ **5.5'**

ii) @ Q'_{P_1} = ½ PMF ≈ 2600 cfs H'_1 ≈ **4.7'**

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Project NON-FEDERAL DAMS INSPECTION Sheet D-8 of 15
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QUILLINAN RESERVOIR DAM

2-Cont'd) SURCHARGE AT PEAK INFLOWS (PMF & 1/2 PMF)

c) EFFECT OF SURCHARGE STORAGE ON PEAK OUTFLOWS:

i) AVE. LAKE AREA WITHIN EXPECTED SURCHARGE:
1') LAKE AREA AT FLOW LINE (ELEV. 135' NGVD): $^{*}A_{WL} = 13.3^{AC}$
2') AREA AT CONTOUR 140' NGVD (MSL)*: $A_{140} = 16.9^{AC}$
3') AREA AT CONTOUR 150' NGVD (MSL)*: $A_{150} = 26.0^{AC}$

∴ AREA AT ELEV 140 (MAX. EXPECTED SURCHARGE): $A_{140} \doteq 16.9^{AC}$

∴ AVE. AREA WITHIN EXPECTED SURCHARGE: $\bar{A} = 15.1$ SAY, $\bar{A} \doteq \underline{\underline{15}}^{AC}$

*NOTE: AREA AT FLOW LINE FROM THE ANSONIA WATER CO. CONTOUR MAP OF QUILLINAN RESERVOIR MAP, DATED 1915, (SCALE 1"=50'). AREAS AT CONTOURS 140' & 150' NGVD FROM USGS: ANSONIA CT. QUADRANGLE SHEET (SCALE 1"=2000'). (LAKE AREA ON USGS MAP MEASURES ONLY ±11.3 AC).

ii) ASSUME NORMAL POOL AT FLOW LINE ELEVATION: ELEV. 135' NGVD

iii) WATERSHED AREA: D.A. ≐ 2.6 sq mi (SEE p. D-1)

iv) DISCHARGE (Q_{P2}) AT VARIOUS HYPOTHETICAL SURCHARGE ELEVATIONS:

$H = 6'$ $V = 15 \times 6 = 90^{AC FT}$ $\therefore S = \frac{90}{2.6 \times 53.3} = 0.65''$

$H = 3'$ $V = 45^{AC FT}$ $\therefore S = 0.32''$

FROM APPROXIMATE ROUTING NED-ACE GUIDELINES AND 19" MAX. PROBABLE R.O. IN NEW ENGLAND:

$$Q_{P2} = Q_{P1}\left(1 - \frac{S}{19}\right) \text{ AND FOR 1/2 PMF: } Q'_{P2} = Q'_{P1}\left(1 - \frac{S}{9.5}\right)$$

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QUILLINAN RESERVOIR DAM

2,c - Cont'd) EFFECT OF SURCHARGE STORAGE ON PEAK OUTFLOWS:

∴ FOR THE PREVIOUS HYPOTHETICAL SURCHARGES:

$H = 6'$ $Q_{P_2} = 5000$ CFS $Q'_{P_2} = 2400$ CFS

$H = 3'$ $Q_{P_2} = 5100$ CFS $Q'_{P_2} = 2500$ CFS

AND FOR $H = 0$ $Q_{P_2} = 5200$ CFS $Q'_{P_2} = 2600$ CFS

d) PEAK OUTFLOWS (Q_{P_3} & Q'_{P_3})

USING NED-ACE GUIDELINES "SURCHARGE STORAGE ROUTING" ALTERNATE METHOD (SEE RATING CURVE p. D-7):

$Q_{P_3} \doteq 5000$ CFS $H_3 \doteq 5.5'$

$Q'_{P_3} \doteq 2500$ CFS $H'_3 \doteq 4.6'$

3) SPILLWAY CAPACITY RATIO TO PEAK INFLOWS AND OUTFLOWS:

a) SPILLWAY CAPACITY TO ELEVATION OF FIRST LOW POINT:
(±) ELEV. 138' NGVD AT EMBANKM/PARKING AREA) ($H = 3'$): $(Q_S)_1 \doteq 570$ CFS

∴ THE SPILLWAY CAPACITY TO FIRST OVERFLOW ELEVATION IS (±) 11% OF BOTH, THE INFLOW (Q_{P_1}) AND THE OUTFLOW (Q_{P_3}) AT PEAK FLOOD = PMF.

LIKEWISE, THE SPILLWAY CAPACITY TO FIRST OVERFLOW ELEVATION IS (±) 22% OF THE INFLOW (Q'_{P_1}) AND (±) 23% OF THE OUTFLOW (Q'_{P_3}) AT PEAK FLOOD = ½ PMF

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Project NON-FEDERAL DAMS INSPECTION Sheet D-10 of 15
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QUILLINAN RESERVOIR DAM

3- Cont'd) SPILLWAY CAPACITY RATIO TO PEAK INFLOWS AND OUTFLOWS:

b) SPILLWAY CAPACITY TO TOP OF DAM (ASSUMING NO LOW POINT OVERFLOW*):
$H = 3.5'$ $\therefore$ $(Q_S)_2 \doteq 720$ CFS

$\therefore$ THE SPILLWAY CAPACITY IS (±)14% OF BOTH, THE INFLOW (Q_{P_1}) AND THE OUTFLOW (Q_{P_3}) AT PEAK FLOOD = PMF.
SIMILARLY, THE SPILLWAY CAPACITY IS (±)28% OF THE INFLOW (Q'_{P_1}) AND (±)29% OF THE OUTFLOW (Q'_{P_3}) AT PEAK FLOOD = ½ PMF.

c) SPILLWAY CAPACITY TO PMF AND ½ PMF SURCHARGES:

i) CAPACITY TO PMF SURCHARGE ($H_3 \doteq 5.5'$): $(Q_S)_3 \doteq 1400$ CFS

$\therefore$ THE SPILLWAY CAPACITY TO PMF SURCHARGE IS (±)27% OF THE INFLOW (Q_{P_1}) AND (±)28% OF THE OUTFLOW (Q_{P_3}) AT PEAK FLOOD = PMF.

ii) CAPACITY TO ½ PMF SURCHARGE ($H'_3 \doteq 4.6'$): $(Q'_S)_4 \doteq 1100$ CFS

$\therefore$ THE SPILLWAY CAPACITY TO ½ PMF SURCHARGE IS (±)42% OF THE INFLOW (Q'_{P_1}) AND (±)44% OF THE OUTFLOW (Q'_{P_3}) AT PEAK FLOOD = ½ PMF.

* THE LOW POINT OVERFLOW IS NOT CONSIDERED TO BE AN ALLOWABLE ADDITIONAL SPILLWAY CAPACITY ($Q \doteq 180$ CFS TO TOP OF DAM).

NOTE: QUILLINAN RESERVOIR DAM HAS A (±)2.5'×2.5' VALVED CONDUIT OUTLET, (±)20' LONG, WITH (±) INVERT ELEV. 121.5' NGVD. THE MAXIMUM CAPACITY OF THIS CONDUIT OUTLET IS ESTIMATED AT (±)150 CFS (W.L. AT TOP OF DAM). THE CAPACITY OF THIS OUTLET ALTHOUGH CONSIDERED USEFUL IN EMERGENCIES TO LOWER THE RESERVOIR'S W.L., IS NOT INCLUDED IN THE PREVIOUS OUTFLOW RATING CURVE/SURCHARGE COMPUTATIONS IN WHICH IT IS ASSUMED TO HAVE A NEGLIGIBLE EFFECT.

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Project NON-FEDERAL DAMS INSPECTION Sheet D-11 of 15
Computed By HM Checked By GAB Date 2/11/80
Field Book Ref. Other Refs. CE #27-660-HA Revisions

QUILLINAN RESERVOIR DAM

II) DOWNSTREAM FAILURE HAZARD

1) POTENTIAL IMPACT AREA

JUST DOWNSTREAM FROM QUILLINAN RESERVOIR, BEAVER BROOK CROSSES A FULLY DEVELOPED SECTION OF THE CITY OF ANSONIA. ALONG THE BROOK, SEVERAL STRUCTURES (RESIDENTIAL, COMMERCIAL & INDUSTRIAL) OF WHICH AT LEAST ONE SPANS THE BROOK, HAVE FIRST FLOORS (±) 8' TO 10' ABOVE THE STREAMBED.
(±) 4000' D/S FROM QUILLINAN RESERVOIR, U/S FROM CENTRAL ST., AT LEAST ONE SECTION OF BEAVER BROOK, (±) 200' LONG, IS PIPED. D/S FROM CENTRAL ST., THE LOWER (±) 3000' REACH OF THE BROOK TO ITS CONFLUENCE WITH THE NAUGATUCK RIVER, IS LEVIED AS A PART OF THE ACE/ANSONIA NAUGATUCK RIVER FLOOD CONTROL SYSTEM.

2) FAILURE AT QUILLINAN RESERVOIR DAM:

a) BREACH WIDTH:

i) HEIGHT OF DAM

TOP OF DAM (±) ELEV. 138.5' NGVD
D/S TOE OF DAM (STREAMBED) - (±) 121' NGVD
$\therefore H \doteq 17.5'$ SAY, $H = \underline{18'}$

ii) MID-HEIGHT OF DAM: (±) ELEV. 130' NGVD $\left(138.5 - \frac{17.5}{2} = 129.8 \text{ SAY } 130' \text{NGVD}\right)$

iii) APPROX. MID-HEIGHT LENGTH: $\ell \doteq$ *83' (*FROM C.E. FIELD MEASUREMENTS ON 1/9/80 BY HM & RJ)

iv) BREACH WIDTH (SEE NED-ACE D/S DAM FAILURE GUIDELINES).

$$W = 0.4 \times 83 \doteq 33' \quad \therefore \text{ ASSUME } W_b = \underline{33'}$$

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Project NON-FEDERAL DAMS INSPECTION Sheet D-12 of 15
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Field Book Ref. Other Refs. CE #27-660-HA Revisions

QUILLINAN RESERVOIR DAM

2-(Cont'd) FAILURE AT QUILLINAN RESERVOIR DAM

b) PEAK FAILURE OUTFLOW (Q_{P1}):

ASSUME SURCHARGE TO TOP OF DAM (ELEV. 138.5' NGVD)

i) HEIGHT AT TIME OF FAILURE: $Y_0 \simeq \underline{18'}$

ii) SPILLWAY DISCHARGE AT TIME OF FAILURE: $Q_S \simeq 720^{CFS}$
(ASSUMING NO LOW POINT OVERFLOW)

iii) BREACH OUTFLOW (Q_b):

$$Q_b = 8/27\, W_b \sqrt{g}\, Y_0^{3/2} \simeq 4240^{CFS}$$

iv) PEAK FAILURE OUTFLOW (Q_{P1}) TO BEAVER BROOK:

$$Q_{P1} = Q_S + Q_b \simeq 4960 \quad \text{SAY,} \quad Q_{P1} \simeq \underline{5000}^{CFS}$$

c) FLOOD DEPTH* IMMEDIATELY D/S FROM DAM:

$$Y \simeq 0.44 Y_0 \simeq 7.9' \quad \text{SAY,} \quad Y \simeq \underline{8'}$$

(*FROM RETREATING WAVE THEORY APPLIED TO DAM FAILURE)

d) ESTIMATE OF D/S FAILURE CONDITIONS AT POTENTIAL IMPACT AREA:

(SEE NED-ACE GUIDELINES FOR ESTIMATING D/S FAILURE HYDROGRAPH)

THE (±) 500' LONG REACH OF BEAVER BROOK D/S FROM QUILLINAN RE-SERVOIR TO THE INITIAL IMPACT AREA IS GENERALLY, V-SHAPED WITH (±) 5^H TO 1^V SIDE SLOPES TO A DEPTH OF (±) 8' TO 20' THE AVERAGE SLOPE OF THE REACH IS (±) 3%.

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Project: NON-FEDERAL DAMS INSPECTION | Sheet D-13 of 15
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Field Book Ref. | Other Refs. CE #27-660-HA | Revisions

QUILLINAN RESERVOIR DAM

2,d-(Cont'd) FAILURE CONDITIONS AT IMPACT AREA

ASSUMING NO SIGNIFICANT PEAK FLOOD REDUCTION IN THE CHANNEL REACH TO THE INITIAL IMPACT AREA, THE D/S CONDITIONS AFTER FAILURE OF QUILLINAN RESERVOIR DAM WILL BE APPROXIMATELY:

$Q_{P_3} \simeq Q_{P_1} \simeq \underline{\underline{5000}}^{cfs}$; $Y_3 \simeq \underline{\underline{8.6'}}$ $(n \simeq 0.010)$

e) APPROXIMATE STAGE BEFORE FAILURE: $Q_S \simeq 720^{cfs}$: $Y_S \simeq \underline{\underline{4.2'}}$

f) RAISE IN STAGE AT IMPACT AREA: $\Delta Y \simeq \underline{\underline{4.4'}}$

NOTE: A SIMILAR COMPUTATION ROUTED OVER A (±) 4000' REACH TO CENTRAL ST. WITH AVE SLOPE $S \simeq 2\%$ GIVES D/S FAILURE CONDITIONS OF APPROX. $Q_3' \simeq 3600^{cfs}$ AND $Y_3' = 8.2'$ $(\Delta Y' \simeq 4.0')$

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Project NON-FEDERAL DAMS INSPECTION

Computed By HGL Checked By GAB Date 3/11/80
Field Book Ref. Other Refs. CE # 27-660-HA Revisions

QUILLINAN RESERVOIR DAM

III) SELECTION OF TEST FLOOD

1) CLASSIFICATION OF DAM ACCORDING TO NED-ACE GUIDELINES:

a) SIZE: *STORAGE (MAX) $\cong$ 175 $^{AC\,FT}$ ($50 < S < 1000$ $^{AC\,FT}$)
*HEIGHT $\cong$ 18' ($H < 25$ ^{FT})

*NOTE: STORAGE: FROM THE ANSONIA WATER CO. "CONTOUR MAP OF QUILLINAN RESERVOIR", 1915, CAPACITY TO FLOW LINE (ELEV. 138' - UNK. DATUM): $S_{FL} = 40^{MG} \cong 123^{AC\cdot FT}$; SURCHARGE STORAGE TO TOP OF DAM: $\Delta S = 15 \times 3.5 \cong 52^{AC\,FT}$ ($\bar{A} \cong 15^{AC}$ - See p. D-8)
$\therefore$ STORAGE TO TOP OF DAM (MAX): $S \cong 175^{AC\,FT}$
HEIGHT (See p. D-11)

$\therefore$ SIZE CLASSIFICATION: SMALL

b) HAZARD POTENTIAL: AS A RESULT OF THE D/S FAILURE ANALYSIS AND IN VIEW OF THE IMPACT THAT THE FAILURE OF QUILLINAN RESERVOIR DAM MAY HAVE ON THE POTENTIAL IMPACT AREA DESCRIBED ON p. D-11, THIS DAM IS CLASSIFIED AS HAVING:

HAZARD CLASSIFICATION: HIGH

2) TEST FLOOD: ½ PMF $\cong$ 2600 ^{CFS}

THIS SELECTION IS BASED ON THE RESULTS OF THE PREVIOUS ANALYSIS AND CLASSIFICATIONS.

Cahn Engineers Inc. Consulting Engineers

Project NON-FEDERAL DAMS INSPECTION Sheet D-15 of 15
Computed By HCl Checked By GAB Date 2/11/80
Field Book Ref. Other Refs. CE#27-660-HA Revisions

QUILLINAN RESERVOIR DAM

IV) SUMMARY AND COMMENTS

1) TEST FLOOD = 1/2 PMF ≅ 2600 CFS
(PARALLEL COMPUTATIONS HAVE BEEN MADE FOR PMF ≅ 5200 CFS AND ARE ALSO SUMMARIZED BELOW)

2) PERFORMANCE AT PEAK FLOOD CONDITIONS:
a) PEAK INFLOWS: Q_{P_1} = PMF ≅ 5200 CFS Q'_{P_1} ≅ 1/2 PMF ≅ 2600 CFS
b) PEAK OUTFLOWS: Q_{P_3} ≅ 5000 CFS Q'_{P_3} ≅ 2500 CFS
c) SPILLWAY CAPACITY:
i) TO FIRST LOW POINT (EMBANK./PARKING AREA); (H ≅ 3'): $(Q_S)_1$ ≅ 570 CFS OR, (±) 11% OF (Q_{P_3}) AND (±) 23% OF (Q'_{P_3})
ii) TO TOP OF DAM (NO LOW POINT OVERFLOW); (H = 3.5'): $(Q_S)_2$ ≅ 720 CFS OR, (±) 14% OF (Q_{P_3}) AND (±) 29% OF (Q'_{P_3})
iii) TO PMF SURCHARGE (H_3 ≅ 5.5'): $(Q_S)_3$ ≅ 1400 CFS OR (±) 28% OF (Q_{P_3})
iv) TO 1/2 PMF SURCHARGE (H'_3 ≅ 4.6'): $(Q'_S)_4$ ≅ 1100 CFS OR (±) 44% OF (Q'_{P_3})

THEREFORE, AT TEST FLOOD Q'_{P_1} = 1/2 PMF, THE DAM IS OVERTOPPED TO A DEPTH OF (±) 1.1' (W.S. (±) ELEV. 139.6' NGVD) OR, TO A SURCHARGE OF (±) 4.6' ABOVE THE SPILLWAY CREST ELEV. 135' NGVD.

SIMILARLY, AT Q_{P_1} = PMF, THE DAM IS OVERTOPPED (±) 2' (W.S. (±) ELEV. 140.5' NGVD) OR, TO A SURCHARGE OF (±) 5.5' ABOVE THE SPILLWAY CREST.

3) DOWNSTREAM FAILURE CONDITIONS:
a) PEAK FAILURE OUTFLOW: Q_{P_1} ≅ 5000 CFS (W/O LOW POINT OVERFLOW)
b) FLOOD DEPTH IMMEDIATELY D/S FROM DAM: y_0 ≅ 8'
c) CONDITIONS AT THE INITIAL IMPACT AREA D/S FROM DAM (BEAVER BROOK)
i) APPROX. STAGE BEFORE FAILURE: y_S ≅ 4.2' (Q_S ≅ 720 CFS)
ii) APPROX. STAGE AFTER FAILURE: y_3 ≅ 8.6' (Q_S ≅ 5000 CFS)
iii) APPROX. RAISE IN STAGE AFTER FAILURE: Δy ≅ 4.4'

PRELIMINARY GUIDANCE

FOR ESTIMATING

MAXIMUM PROBABLE DISCHARGES

IN

PHASE I DAM SAFETY

INVESTIGATIONS

New England Division
Corps of Engineers

March 1978

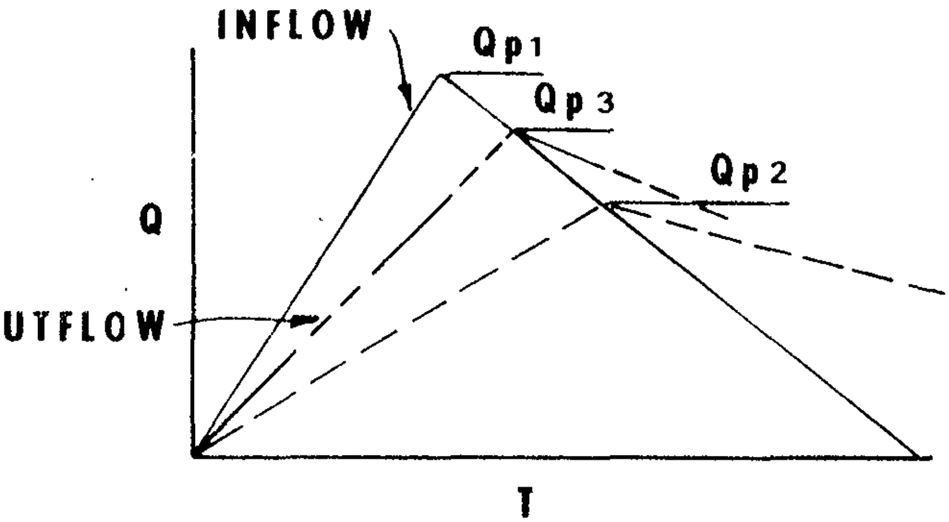
MAXIMUM PROBABLE FLOOD INFLOWS
NED RESERVOIRS

	Project	Q (cfs)	D.A. (sq. mi.)	MPF cfs/sq. mi.
1.	Hall Meadow Brook	26,600	17.2	1,546
2.	East Branch	15,500	9.25	1,675
3.	Thomaston	158,000	97.2	1,625
4.	Northfield Brook	9,000	5.7	1,580
5.	Black Rock	35,000	20.4	1,715
6.	Hancock Brook	20,700	12.0	1,725
7.	Hop Brook	26,400	16.4	1,610
8.	Tully	47,000	50.0	940
9.	Barre Falls	61,000	55.0	1,109
10.	Conant Brook	11,900	7.8	1,525
11.	Knightville	160,000	162.0	987
12.	Littleville	98,000	52.3	1,870
13.	Colebrook River	165,000	118.0	1,400
14.	Mad River	30,000	18.2	1,650
15.	Sucker Brook	6,500	3.43	1,895
16.	Union Village	110,000	126.0	873
17.	North Hartland	199,000	220.0	904
18.	North Springfield	157,000	158.0	994
19.	Ball Mountain	190,000	172.0	1,105
20.	Townshend	228,000	106.0(278 total)	820
21.	Surry Mountain	63,000	100.0	630
22.	Otter Brook	45,000	47.0	957
23.	Birch Hill	88,500	175.0	505
24.	East Brimfield	73,900	67.5	1,095
25.	Westville	38,400	99.5(32 net)	1,200
26.	West Thompson	85,000	173.5(74 net)	1,150
27.	Hodges Village	35,600	31.1	1,145
28.	Buffumville	36,500	26.5	1,377
29.	Mansfield Hollow	125,000	159.0	786
30.	West Hill	26,000	28.0	928
31.	Franklin Falls	210,000	1000.0	210
32.	Blackwater	66,500	128.0	520
33.	Hopkinton	135,000	426.0	316
34.	Everett	68,000	64.0	1,062
35.	MacDowell	36,300	44.0	825

MAXIMUM PROBABLE FLOWS
BASED ON TWICE THE
STANDARD PROJECT FLOOD
(Flat and Coastal Areas)

	River	SPF (cfs)	D.A. (sq. mi.)	MPF (cfs/sq. mi.)
1.	Pawtuxet River	19,000	200	190
2.	Mill River (R.I.)	8,500	34	500
3.	Peters River (R.I.)	3,200	13	490
4.	Kettle Brook	8,000	30	530
5.	Sudbury River.	11,700	86	270
6.	Indian Brook (Hopk.)	1,000	5.9	340
7.	Charles River.	6,000	184	65
8.	Blackstone River.	43,000	416	200
9.	Quinebaug River	55,000	331	330

ESTIMATING EFFECT OF SURCHARGE STORAGE ON MAXIMUM PROBABLE DISCHARGES



STEP 1: Determine Peak Inflow (Q_{p1}) from Guide Curves.

STEP 2: a. Determine Surcharge Height To Pass "Q_{p1}".

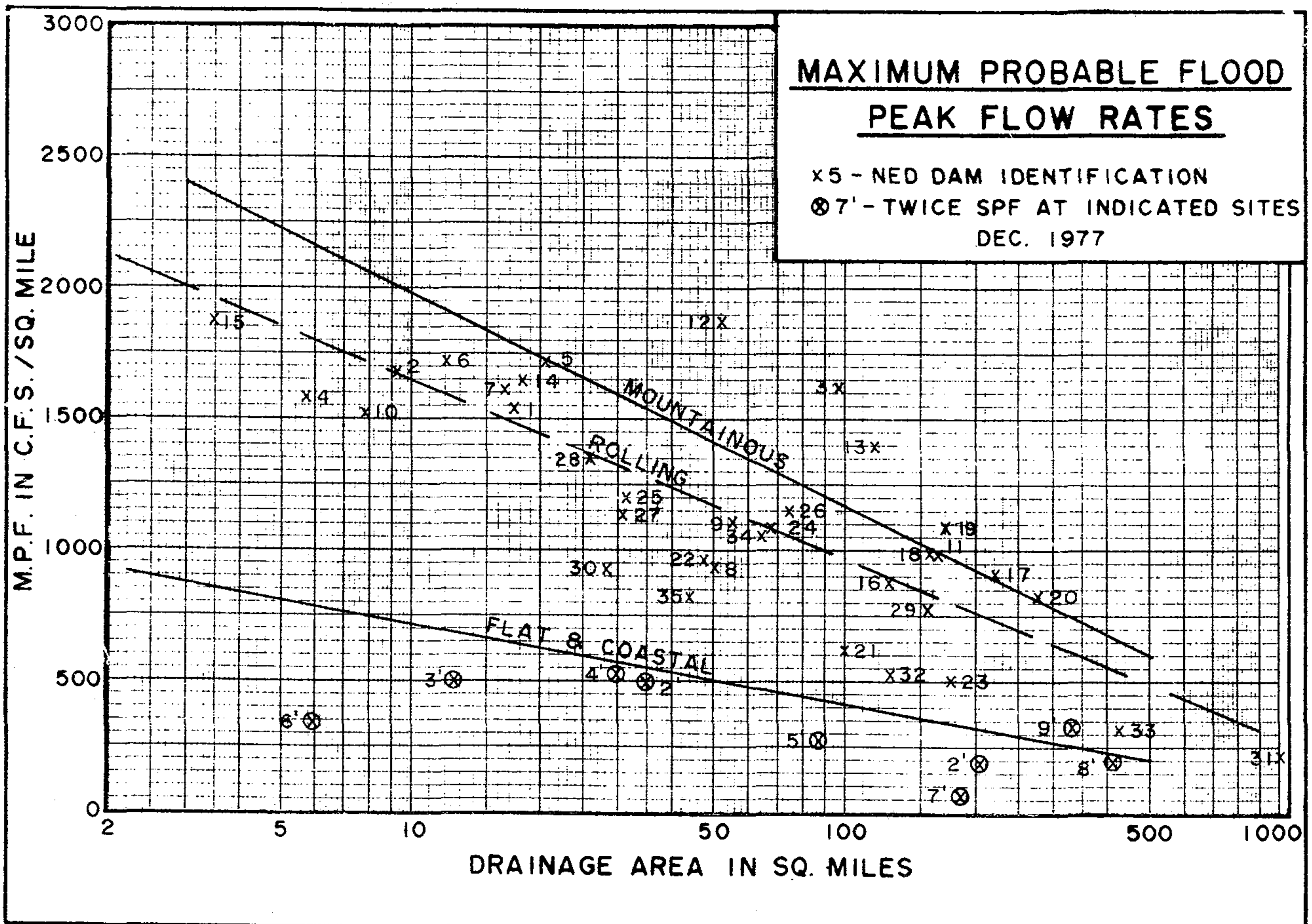
b. Determine Volume of Surcharge ($STOR_1$) In Inches of Runoff.

c. Maximum Probable Flood Runoff In New England equals Approx. 19", Therefore:

$$Q_{p2} = Q_{p1} \times \left(1 - \frac{STOR_1}{19}\right)$$

STEP 3: a. Determine Surcharge Height and "$STOR_2$" To Pass "Q_{p2}"

b. Average "$STOR_1$" and "$STOR_2$" and Determine Average Surcharge and Resulting Peak Outflow "Q_{p3}".



MAXIMUM PROBABLE FLOOD
PEAK FLOW RATES
x5 - NED DAM IDENTIFICATION
⊗7' - TWICE SPF AT INDICATED SITES
DEC. 1977
M.P.F. IN C.F.S. / SQ. MILE
DRAINAGE AREA IN SQ. MILES
MOUNTAINOUS
ROLLING
FLAT & COASTAL
3000
2500
2000
1500
1000
500
0
2
5
10
50
100
500
1000

SURCHARGE STORAGE ROUTING SUPPLEMENT

STEP 3: a. Determine Surcharge Height and "$STOR_2$" To Pass "Qp_2"

b. Avg "$STOR_1$" and "$STOR_2$" and Compute "Qp_3".

c. If Surcharge Height for Qp_3 and "$STOR_{AVG}$" agree O.K. If Not:

STEP 4: a. Determine Surcharge Height and "$STOR_3$" To Pass "Qp_3"

b. Avg. "Old $STOR_{AVG}$" and "$STOR_3$" and Compute "Qp_4"

c. Surcharge Height for Qp_4 and "New $STOR_{Avg}$" should Agree closely

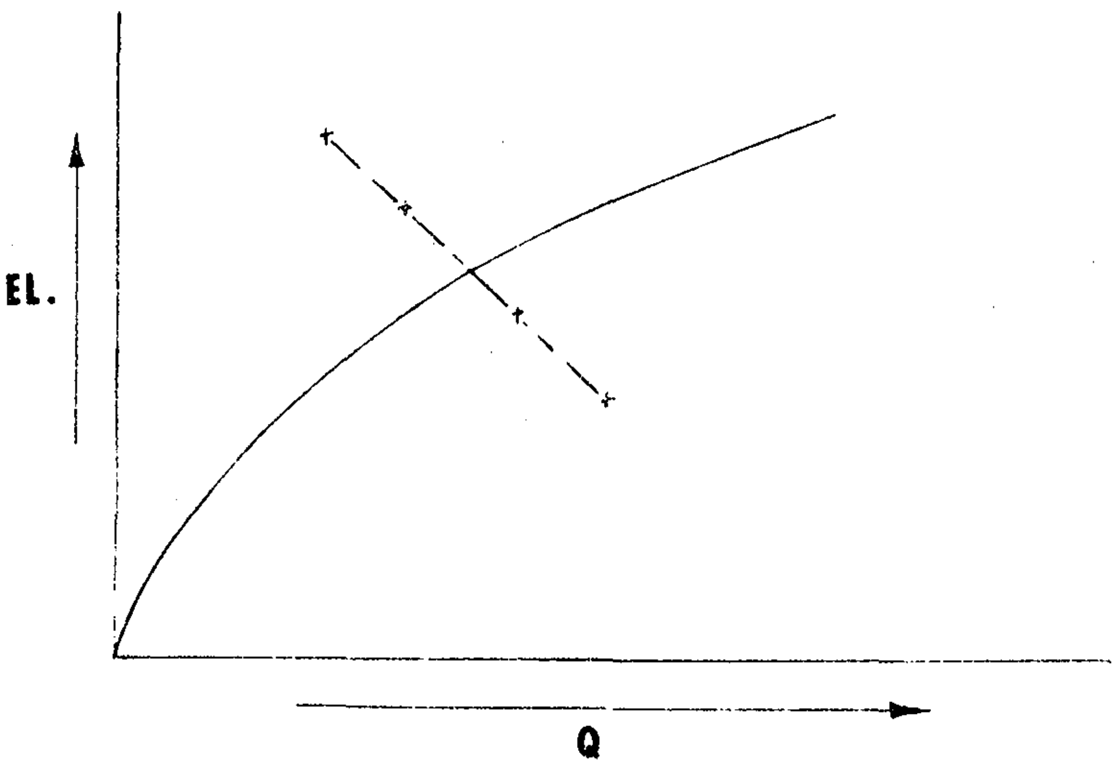
SURCHARGE STORAGE ROUTING ALTERNATE

$$Q_{p2} = Q_{p1} \times \left(1 - \frac{STOR}{19}\right)$$

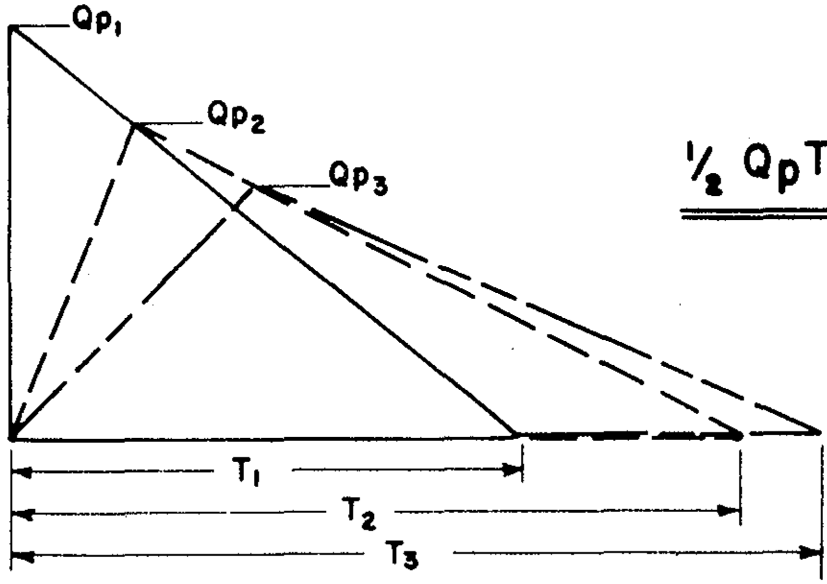
$$Q_{p2} = Q_{p1} - Q_{p1}\left(\frac{STOR}{19}\right)$$

FOR KNOWN Q_{p1} AND 19'' R.O.

Q_{p2}	STOR	EL.



"RULE OF THUMB" GUIDANCE FOR ESTIMATING DOWNSTREAM DAM FAILURE HYDROGRAPHS



$$\frac{1}{2} Q_p T = 12\,S$$

STEP 1: DETERMINE OR ESTIMATE RESERVOIR STORAGE (S) IN AC-FT AT TIME OF FAILURE.

STEP 2: DETERMINE PEAK FAILURE OUTFLOW (Q_{p1}).

$$Qp_1 = \frac{8}{27} W_b \sqrt{g}\, Y_0^{3/2}$$

W_b = BREACH WIDTH - SUGGEST VALUE NOT GREATER THAN 40% OF DAM LENGTH ACROSS RIVER AT MID HEIGHT.

Y_o = TOTAL HEIGHT FROM RIVER BED TO POOL LEVEL AT FAILURE.

STEP 3: USING USGS TOPO OR OTHER DATA, DEVELOP REPRESENTATIVE STAGE-DISCHARGE RATING FOR SELECTED DOWNSTREAM RIVER REACH.

STEP 4: ESTIMATE REACH OUTFLOW (Q_{p2}) USING FOLLOWING ITERATION.

A. APPLY Q_{p1} TO STAGE RATING, DETERMINE STAGE AND ACCOPMANYING VOLUME (V_1) IN REACH IN AC-FT. (NOTE: IF V_1 EXCEEDS 1/2 OF S, SELECT SHORTER REACH.)

B. DETERMINE TRIAL Q_{p2}.

$$Qp_2(\text{TRIAL}) = Qp_1\left(1 - \frac{V_1}{S}\right)$$

C. COMPUTE V_2 USING Q_{p2} (TRIAL).

D. AVERAGE V_1 AND V_2 AND COMPUTE Q_{p2}.

$$Qp_2 = Qp_1\left(1 - \frac{V_{AVE}}{S}\right)$$

STEP 5: FOR SUCCEEDING REACHES REPEAT STEPS 3 AND 4.

APRIL 1978

APPENDIX E

INFORMATION AS CONTAINED IN
THE NATIONAL INVENTORY OF DAMS

NOT AVAILABLE AT THIS TIME