CONNECTICUT RIVER BASIN
PLYMOUTH, CONNECTICUT

OLD MARSH POND DAM

CT 00285

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

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

JANUARY 1979

UNCLASSIFIED

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

Old Marsh Pond Dam is a zoned earth embankment about 1,100 ft. long with a concrete core wall, is 33 ft. wide at the crest, and has a maximum height of about 32 ft. Both the dam and dike appear bo be in good condition. The full PMF test flood would not overtop the dam (provided that the flashboards were not in place), but would overtop the dike by 1 ft. Based on maximum storage capacity, the dam is classified intermediate in size. It has been classified as having a high hazard potential.

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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

OCT 2 1979

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

Dear Governor Grasso:

I am forwarding to you a copy of the Old Marsh 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. A brief assessment is included at the beginning of the report. I have approved the report and support the findings and recommendations described in Section 7 and ask that you keep me informed of the actions taken to implement them. This follow-up action is a vitally important part of this program.

A copy of this report has been forwarded to the Department of Environmental Protection, the cooperating agency for the State of Connecticut. In addition, a copy of the report has also been furnished the owner, City of Bristol, Connecticut 06010.

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

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

Sincerely,

MAX B. SCHEIDER
Colonel, Corps of Engineers
Division Engineer

Incl
As stated

OLD MARSH POND

CT 00285

CONNECTICUT RIVER BASIN
PLYMOUTH, CONNECTICUT

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

NATIONAL DAM INSPECTION PROGRAM
PHASE I INSPECTION REPORT

Identification No.	CT 00285
Name of Dam:	Old Marsh Pond Dam
Town:	Plymouth
County and State:	Litchfield County, Connecticut
Stream:	Marsh Brook
Date of Inspection:	9 November 1978

BRIEF ASSESSMENT

Old Marsh Pond Dam is a zoned earth embankment about 1,100 ft. long with a concrete core wall, is 33 ft. wide at the crest, and has a maximum height of about 32 ft. Marsh Road traverses the crest of the dam. The dam is operated as a water supply facility for the City of Bristol. The closure dike some 1,000 ft. to the right of the dam is a homogeneous embankment about 526 ft. long, 14 ft. high and 24 ft. wide at the crest.

The main dam has a spillway consisting of a 60 ft. long ungated overflow ogee crest 6.3 ft. below the top of the dam. Fourteen (14) in. high flashboards are fixed on the crest from spring to fall. A 16 in. dia. outlet pipe leads downstream about one mile to Bristol Reservoir No. 1.

Maximum storage capacity of the reservoir to top of dam is 3,094 acre-ft. and the drainage area is 2.34 square miles. The reservoir is about 7,300 ft. long and has a surface at normal storage (without flashboards) of 183 acres. Based on maximum storage capacity, the dam is classified as intermediate in size. Because of the threat to life and property which would result if the dam or dike were breached, it has been classified as having a high hazard potential.

Both the dam and dike appear to be in good condition. The full PMF test flood would not overtop the dam (provided that the flashboards were not in place), but would overtop the dike by 1 ft. The facility could handle 75% of the test flood without overtopping the dike. While the spillway crest is adequate to pass the test flood outflow, the spillway chute and stilling basin walls would all be overtopped. At a 0.5 PMF outflow, the stilling basin walls would be overtopped and the left chute wall would be on the verge of overtopping.

There is considerable seepage in the vicinity of the center of the dam, which has produced a swampy area below the downstream slope. The stilling basin side walls have deflected inwards relative to the spillway chute walls. The outlet is too small to permit rapid drawdown of the reservoir. There is some minor displacement of riprap, some local brush growth and a few animal burrows on the dam.

Within two years after receipt of this Phase I Inspection Report, the owner, the City of Bristol, should retain the services of a registered professional engineer to make further investigations, and should implement the results. These studies should cover: (1) whether the dike should be raised; (2) whether spillway channel modifications are required; (3) elimination of use of flashboards, or modifications to facilitate their quick removal; (4) source of leakage producing a swamp below the dam; (5) cause of relative deflection of spillway chute walls; and (6) whether a new outlet for rapid drawdown of the reservoir is required.

The owner should also implement the following measures: (1) monitor the seepage monthly at the center of the dam; (2) monitor the marshy area right of the maintenance building access ramp monthly; (3) restore riprap in the vicinity of the spillway walls; (4) control growth on the right downstream slope; (5) fill animal burrows and eliminate animal infestation; (6) repair the top of the concrete wall which extends from the right stilling basin wall; (7) remove plywood forms from the underside of the bridge deck, make necessary repairs, paint all bridge steelwork; (8) institute procedures for a biennial periodic technical inspection; and (9) develop a formal surveillance and warning plan.

Peter B. Dyson
Project Manager

COMMONWEALTH OF MASSACHUSETTS
PETER BRIAN DYSON
No. 18452
REGISTERED PROFESSIONAL ENGINEER

Frederick Esper
Frederick Esper
Vice President

COMMONWEALTH OF MASSACHUSETTS
FREDERICK ESPER
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REGISTERED PROFESSIONAL ENGINEER

This Phase I Inspection Report on Old Marsh 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.

JOSEPH W. FINEGAN, JR., MEMBER
Water Control Branch
Engineering Division

CARNEY M. TERZIAN, MEMBER
Design Branch
Engineering Division

JOSEPH A. MCELROY, CHAIRMAN
Chief, NED Materials Testing Lab.
Foundations & Materials 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, a finding that a spillway will not pass the test flood should not be interpreted as necessarily posing a highly inadequate condition. The test flood provides a measure of relative spillway capacity and serves as an 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.

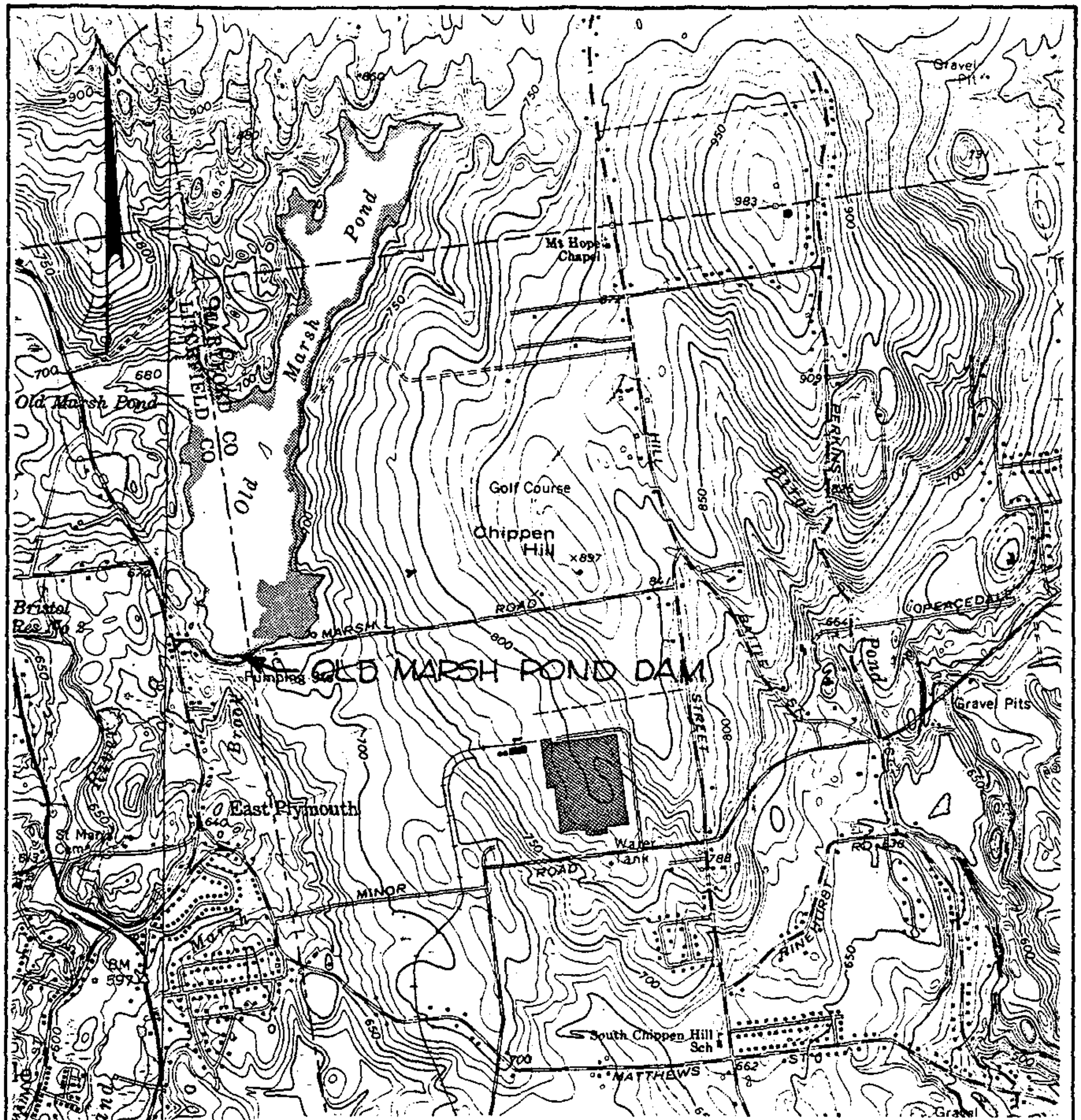
TABLE OF CONTENTS

Page

OLD MARSH POND DAM

Overview of dam from right abutment of dam.

Overview of closure dike from left abutment of dike.



LOUIS BERGER & ASSOC.,INC
WELLESLEY, MASS.
ARCHITECT · ENGINEER

U.S. ARMY ENGINEER DIV. NEW ENGLAND
CORPS OF ENGINEERS
WALTHAM, MASS.

NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS

OLD MARSH POND DAM

BRISTOL - THOMASTON QUADRANGLES

CONNECTICUT RIVER BASIN — STATE - CONN.

SCALE 1:24000
DATE

PHASE I INSPECTION REPORT

OLD MARSH POND DAM CT 00285

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. Louis Berger & Associates, 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 was issued to Louis Berger & Associates, Inc. under a letter of 27 October 1978 from Max B. Scheider, Colonel, Corps of Engineers. Contract No. DACW33-78-C-0371 Job Change No. 1 has been assigned by the Corps of Engineers for this work.

b. Purpose

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

3. Update, verify and complete the National Inventory of Dams.

1.2 Description of Project

a. Location

Old Marsh Pond Dam is located in the Town of Plymouth, Litchfield County, about 4 miles northwest of the City of Bristol, Hartford County, in Central Connecticut. The dam is reached by proceeding west 1¼ miles from Main Street in Bristol to Hill Street, then north 1½ miles to Marsh Road and one mile west to the dam. About a third of the reservoir is in Plymouth and two thirds is in Bristol. The dam is situated on Marsh Brook, a tributary of the Poland River which joins the Pequabuck River in Terryville. The Pequabuck is a tributary of the Farmington River which flows into the Connecticut River.

The dam and reservoir are operated as a water supply facility for the City of Bristol.

b. Description of Dam and Appurtenances

1. Dam

Marsh Pond Dam is a zoned earthfill embankment about 32 ft. high at its maximum section and about 1,100 ft. long. The dam is a reconstructed and raised structure built in 1952 on top of and immediately downstream from an existing dam, which was built about 1913. The original dam had a crest elevation of about 683; the new dam has its crest at elevation 694 MSL.

The original dam was constructed of loamy clay, sand, gravel, and boulders, with its 2 to 1 sloped upstream face covered with stone paving. A central core wall built of boulder concrete on the centerline of the dam was carried from about 2.5 ft. below the crest down to bedrock foundation. The old dam was left in place, except for surface stripping, when the new embankment was added, and it now forms the upstream toe of the new dam.

The new dam was built with its centerline 30 ft. downstream from that of the original dam. A new core wall was constructed of reinforced concrete with its centerline one ft. upstream from the new dam centerline, extending from 3 ft. below crest level to firm rock foundation. To reach bedrock a 7 ft. wide vertical sheeted trench was excavated through the downstream face of the old dam. After the core wall was completed the trench was backfilled with impervious material.

The new dam has a 33 ft. top width at elevation 694, with a 1½ to 1 upstream slope intersecting the crest level of the old dam, and with heavy random riprap placed on the old dam face on about a 2 to 1 slope below the old crest level. The downstream face of the new dam is at 1½ to 1 slope from the crest down to elevation 683, and then at 2 to 1 slope below that level. The surface of the slope is covered with loam soil and is grassed.

The central body of the new dam is compacted impervious embankment. A heavy random riprap on 1½ to 1 covers the upstream face above the old dam. The downstream portion of the dam below elevation 683, with a thickness of 17.5 ft. at that level increasing to as much as 30 ft. at the base of the dam, is composed of pervious material. A graded sand and gravel filter separates this pervious fill from the foundation. Below elevation 668.7 this horizontal filter terminates in a 5 ft. deep toe trench and intercepting drain located along the toe of the dam. A rock-filled toe berm 5 ft. wide covers the intercepting drain.

The foundation of the main portion of the dam and right abutment is overburden of varying thickness, overlying micaceous gneiss and schist ranging from decomposed to firm. The core wall along this portion of the dam was extended at least 2 ft. into sound rock. On the left abutment east of the spillway the bedrock falls away steeply and the core wall was carried through overburden only to what was considered to be impervious stratum. The core wall extends the entire length of the dam below elevation 691. Plans and profiles of the dam and structures are included in Appendix B.

2. Closure Dike

A dike to close off a low saddle is located along the reservoir rim about 1,000 ft. to the right of the dam. The dike is a homogeneous embankment about 14 ft. high above natural ground and about 526 ft. long. The dike, which was built in 1952, replaces a lower embankment closing off the saddle, and was located just downstream so that its upstream portion covers a portion of the old embankment. A deep cutoff trench was excavated and backfilled under the dike, extending to bedrock along the entire length of the embankment. The cutoff has a 10 ft. bottom width and 1 to 1 side slopes, and at its deepest was excavated to more than 30 ft. below ground surface. The dike was built to crest elevation 693, one ft. below the crest level of the main dam. The dike has a top width of 20 ft., with 2 to 1 upstream and downstream slopes. The upstream slope is protected with an 18 in. layer of riprap, placed on a gravel and sand filter. A 5 ft. deep downstream toe drain trench filled with coarse stone and a horizontal filter blanket are provided along the downstream toe of the dike. Plans of the dike are included in Appendix B.

3. Spillway

When the dam was raised the old spillway was abandoned and a new structure was built to the left, higher on the abutment. The spillway is an ungated overflow crest and converging curved chute structure, terminating with a hydraulic jump stilling basin. The ogee crest is 60 ft. in length, with crest level at elevation 687.7, or 6.3 ft. below the top of the dam.

The ogee crest is an overflow gravity dam which is built up from bedrock, resulting in a structure having a maximum height of about 27 ft. This dam has a vertical upstream face and a downstream face with an average slope of 0.76 to 1. The top of this dam is ogee and curved to conform to the underside of a nappe over a sharp crested weir, shaped for a design head of about 2.7 ft.

The channel downstream from the crest turns 22.5 degrees to the right and converges from the 60 ft. crest width to a 25 ft. width. The 25 ft. chute length continues about 130 ft. to a 45 ft. long stilling basin with its floor at elevation 650. The channel walls are 7.25 ft. high below the converged section, 3.5 ft. high along the chute and 12 ft. high at the stilling basin. A 60 ft. span steel beam bridge with concrete deck crosses the spillway at the crest of the dam. Details of the spillway design are included in Appendix B.

4. Outlets

The low level outlet consists of a 24 in. dia. cast iron pipe which was constructed under the old dam, and which then connects directly into a 16 in. dia. line which leads downstream to Bristol Reservoir No. 1. The invert of the inlet to the 24 in. dia. pipe is at elevation 663.7. A 16 in. dia. cast iron high level inlet, with its invert level at elevation 678.3, is carried through the new embankment and connects to the 16 in. dia. line downstream from the toe of the dam. Control for the low level line is by a 24 in. dia. valve, reached from a submerged gate well near the upstream toe, and by a 16 in. dia. gate valve operated from a gate chamber near the downstream toe. The high level outlet is controlled by a 16 in. dia. gate valve operated from a gate chamber located near the upstream edge of the crest of the dam. All outlet flows are carried directly to Reservoir No. 1 via a 16 in. dia. delivery pipe about one mile in length. A 6 in. dia. blowoff line takes off from the 16 in. dia. line about 100 ft. downstream from the toe of the dam. This blowoff line is carried into the spillway stilling basin and is controlled by a 6 in. dia. gate valve.

Details of the outlet pipes are shown on drawings included in Appendix B.

c. Size Classification

Old Marsh Pond Dam is about 32 ft. high, impounding a maximum storage of about 1,800 acre-feet to spillway crest level and about 3,100 acre-feet to the top of the dam. In accordance with the size and capacity criteria given in the Recommended Guidelines for Safety Inspection of Dams, storage governs and therefore the project is classified as intermediate in size.

d. Hazard Classification

A breach failure of Old Marsh Pond Dam would release water down Marsh Brook and into the Poland and Pequabuck Rivers. These streams traverse through the urbanized areas of East Plymouth and Terryville in Plymouth, and through the City of Bristol. There are a number of homes in East Plymouth near stream level along Marsh Brook; major highways, including State Route 72 and part of U. S. Route 202, follow along the Poland and Pequabuck Rivers; and there are a number of highway crossings of these rivers. Consequently, Old Marsh Pond Dam has been classified as having a high hazard potential in accordance with the Recommended Guidelines for the Safety Inspection of Dams.

e. Ownership

Old Marsh Pond Dam is owned by the City of Bristol, Connecticut.

f. Operator

John Burns, Superintendent
Water Department
119 Reservoir Avenue
Bristol, CT 06010

Telephone: (203) 582-7431

g. Purpose of Dam

Old Marsh Pond reservoir is operated in conjunction with other water storage facilities to supply municipal water to the City of Bristol.

h. Design and Construction History

The original Old Marsh Pond Dam was built in 1913. The engineer in charge of constructing the dam was Mr. Nicholas Fogg, employed by the firm of Sperry and Buell. The dam was built for Andrew Terry Co. of Terryville in the Town of Plymouth, who used it as a power generation facility for their manufacturing plant. In 1944 the dam and water rights were acquired by the City of Bristol Water Commission, to supplement the City's water resources. The original dam and reservoir were operated until about 1950, when the reservoir was raised about 8 ft. by the construction of the new dam, which, being 11 ft. higher, provided for an additional 3 ft. of surcharge.

The design of the dam enlargement was prepared by Metcalf and Eddy, Engineers, of Boston, MA.

i. Normal Operational Procedure

There are no written operating procedures. All withdrawals from Old Marsh Pond reservoir are transported by pipeline to Bristol Reservoir No. 1 about one mile downstream, and day to day regulation of withdrawals is not required. Water Department personnel are available for regulating reservoir releases.

1.3 Pertinent Data

a. Drainage area

The drainage area contributing to the Old Marsh Pond reservoir encompasses about 2.34 square miles. The reservoir extends along nearly the full length of the area, with contributing inflows draining off the slopes to the sides through five small inlet streams. These tributary inlets vary from 3,600 ft. to 6,000 ft. in length, with gradients varying from 161 ft. per mile to 267 ft. per mile.

The drainage area measures about 2.4 miles in length and 1.6 miles in maximum width. The eastern edge of the area is relatively open and occupied by a golf course; the remaining area is heavily wooded.

b. Discharge at Damsite

1. Outlet works

Outlet releases are limited to the discharge which can be carried through the 16 in. dia. line to the Bristol Reservoir No. 1. A 6 in. dia. blowoff pipe is provided in this line, controlled by a 6 in. dia. valve. The blowoff line empties into the spillway stilling basin. It is estimated that with all valves open the capacity of the line will be about 9 cfs.

2. Maximum Flood at Damsite

No records are available of flood inflows into Old Marsh Pond reservoir, nor of spillway releases and surcharge heads during such inflows.

3. Ungated Spillway Capacity

The spillway at Old Marsh Pond Dam is an ungated ogee overflow 60 ft. in length, which appears to have been designed for a capacity of about 900 cfs at 2.4 ft. surcharge head. The spillway control will be able to handle about 3,500 cfs with reservoir to top of dike, elevation 693; and about 4,000 cfs with reservoir to top of dam, elevation 694. Discharge curves and computations are shown on Figure 1 and sheets D-1 to D-3, Appendix D.

The spillway chute and stilling basin have been designed to provide for the 900 cfs flow, with adequate sidewall freeboards to prevent an overtopping of the walls. At a 1,400 cfs flow, conjugate depth in the hydraulic jump basin will be equal to the basin wall height; and flows in excess of 1,400 cfs will overtop the basin walls. At about a 2,000 cfs flow, because of the sharp convergence of the left wall into the general direction of flow, it is expected that the resulting wave will ride up and overtop the wall. At about a 2,700 cfs discharge, the flow depth will be equal to the height of the chute walls, without considering wave action and swell owing to air entrainment. At a 4,000 cfs discharge, conjugate depth will be 4 ft. higher than the basin walls. It is thus seen that although the spillway overflow can accommodate 4,000 cfs with the reservoir lower than the top of the dam, the spillway chute and stilling basin can handle no more than 1,400 cfs without incurring damage from an overtopping of its walls. An overtopping would probably wash out the backfill behind the walls and undermine their foundation, thus causing a failure of the spillway chute. Such a failure could threaten the lower portion of the main dam embankment. Computations of spillway channel flow are shown on Sheets D-4 and D-5.

4. Ungated Spillway Capacity at Test Flood Elevation

At the test flood elevation of 694.0, the discharge through the spillway will be 3,940 cfs.

5. Flashboarded Spillway Capacity

With the intent of increasing the reservoir yield by capturing and storing runoffs above spillway crest level, 14 in. high flashboards are installed on the crest each spring and removed in early fall. The flashboards are held in place by angle iron frames, which are inserted into the crest at their bottom and then supported at their top by bolting onto the upstream bridge beam.

With the flashboards in place, the spillway capacity for heads up to reservoir level 692 will be reduced by more than half the capacity without the flashboards; for a head to reservoir level 694 the spillway capacity will be 55 percent of that of an unobstructed crest. A discharge curve for flow over the crest with the flashboards installed is shown on Fig. 1, Sheet D-1, Appendix D.

6. Total Project Discharge at Test Flood Elevation

At the test flood elevation of 694.0, the discharge over the dike will be 1,475 cfs, giving a total project discharge of 5,415 cfs.

c. Elevation (ft. above MSL)

1.	Top of dam	694.0
2.	Maximum pool-design surcharge	691.0
3.	Spillway crest (ungated)	687.7
4.	Diversion outlet invert	663.7
5.	Streambed at centerline of dam	663 ±

d. Reservoir

1.	Length of pool (normal W.S.)	7,300 ft.
2.	Average width of pool (normal W.S.)	1,100 ft.

e. Storage (acre-feet)

1.	At normal storage pool (El. 687.7)	1,808
2.	At design surcharge pool (El. 691)	2,454
3.	At top of dike (El. 693)	2,876
4.	At top of dam (El. 694)	3,094

f. Reservoir surface (acres)

1.	At top of dam (El. 694)	220
2.	At top of dike (El. 693)	216
3.	At design surcharge pool (El. 691)	206
4.	At top of flashboards (El. 688.87)	192
5.	At spillway crest (El. 687.7)	183

g. Main Dam

1. Type - Zoned earthfill embankment
2. Length 1,100 ft.
3. Height 32 ft.
4. Top width 33 ft.
5. Side slopes - 1½ to 1 and 2 to 1 upstream; 1½ to 1 and 2 to 1 downstream.
6. Zoning - Concrete core wall; impervious fill for upstream zone and center portion of downstream zone; pervious material at outer portion of downstream zone; horizontal filter under downstream zone; rock fill toe; upstream rock fill facing.
7. Impervious core - Concrete core wall to bedrock along main dam, to impervious stratum along east abutment; core wall carried to within 3 feet of top of dam.
8. Cutoff - core wall in trench to bedrock or to impervious stratum.
9. Grout curtain - none
10. Other - Horizontal filter under downstream pervious zone of dam.

h. Closure Dike

1. Type - Homogeneous earthfill embankment.
2. Length 526 ft.
3. Height 14 ft.
4. Top width 20 ft.
5. Side slopes - 2 to 1 upstream and downstream
6. Zoning - Compacted earthfill. Most impervious material upstream, less impervious material downstream.
7. Impervious core - none
8. Cutoff - Deep trench to bedrock backfilled with impervious fill material. Cutoff trench to depths up to 35 ft. maximum, 10 ft. wide at bottom.
9. Grout curtain - none
10. Other - Upstream slope riprap - 18 in. thick on 6 in. gravel and sand filter layers. 5 ft. deep drain trench under downstream toe backfilled with coarse stone. Rockfill at downstream toe.

i. Spillway

1. Type - Ungated overflow crest and converging chute structure, terminating with hydraulic jump stilling basin.
2. Length of weir 60 ft.
3. Crest elevation 687.7 MSL

4. Gates - None, but provision for 14 in. flashboards.
5. Upstream channel - Inlet directly from reservoir through upstream face of dam.
6. Downstream channel - Converging curved chute and downstream hydraulic jump basin.
7. General - Crest control adequate to release up to 4,000 cfs before overtopping of dam. Chute and stilling basin adequate to discharge only 1,400 cfs before overtopping of walls takes place.

j. Regulating Outlets

1. Invert - Low-level at elevation 663.7
   High level at elevation 678.3

2. Size - Low level - 24 in. dia. connected to 16 in. dia. line to downstream reservoir.
   - High Level - 16 in. dia. connecting into 16 in. dia. line to downstream reservoir.

3. Control mechanism - Low level: 24 in. dia. valve near upstream end and 16 in. dia. valve near downstream toe of dam.
   - High level: 16 in. dia. valve near upstream edge of crest.

SECTION 2 - ENGINEERING DATA

2.1 Design

1943 correspondence in the files of the State Department of Environmental Protection mentions the availability of plans of the original dam, but these have not been retrieved. It is presumed that they were available to the engineering firm of Metcalf and Eddy (now Metcalf and Eddy, Inc.) of Boston, MA, who prepared the detailed plans for the enlarged dam and dike in 1950. Reduced scale copies of those plans which are pertinent to an assessment of dam safety are included in Appendix B. Neither design data nor design criteria leading to this design have been retrieved.

2.2 Construction

No information has been recovered regarding construction of the original dam in 1913. No construction reports or histories of construction have been found documenting construction details or results of the 1952 alterations and additions. The design drawings included in Appendix B have been revised to show as-built details and constitute the record plans for this work.

2.3 Operation

The reservoir is operated by personnel of the City of Bristol Water Department, in conjunction with other reservoir storage facilities in their water supply system. All releases are transported to a lower reservoir and day to day operation of the outlet is not required. Flashboards on the spillway crest are installed in the spring and removed in early fall. No operation or maintenance criteria, or guideline manuals have been prepared for this installation.

2.4 Evaluation

a. Availability

The plans of the enlargement of the dam and dike provide insufficient engineering data for an assessment to be made of the structural stability and safety of the embankments.

b. Adequacy

The lack of in-depth engineering data precludes a definitive review and assessment of this dam. The evaluation is based primarily on visual inspection and engineering judgment.

c. Validity

The validity of the engineering data acquired covering the dam and dike is considered acceptable and is not challenged.

SECTION 3 - VISUAL INSPECTION

3.1 Findings

a. General

The visual inspection of Old Marsh Pond Dam took place on 9 November 1978. At that time, the reservoir was about 3 ft. below spillway crest level. It was not determined whether storage was being released through the pipeline to Bristol Reservoir No. 1.

Both the main dam and closure dike were judged to be in good condition. There was no evidence of any major maintenance problems. Fairly heavy seepage was noted in the vicinity of the original stream channel below the dam, but its source was not determined.

b. Dam

The crest of the dam is a well travelled paved roadway which is in good condition (Appendix C, Photo No. 1).

The riprapped top of the original dam upon which the present dam was superimposed was visible as a berm slightly below the water level. The base of an abandoned intake tower, damaged by ice, was visible in the reservoir opposite the approximate center of the dam.

The upstream face of the dam is riprapped with 6 in. to 18 in. stone almost up to the gravel shoulder. Locally, such as immediately west of the spillway, the riprap has displaced downslope, possibly attributable to surface drainage channelized against the spillway wall. Riprap appears to be less massive than called for on the drawings.

At the left abutment there is an access road to the pumping station and maintenance building, beneath which a culvert passes, draining to a low area at the toe of the dam. Here, the ground is boggy and displays characteristic marsh growth.

Opposite the highest section of the dam in the original stream channel, about 200 ft. west of the spillway, prominent seepage issues from an area about 10 ft. wide in the vicinity of the toe, where a considerable amount of peastone has been placed, and flows southerly some 200 ft. to the end of the spillway stilling basin. When it reaches the end of the spillway channel, the flow issues from a boggy plateau and cascades over a concrete retaining wall adjacent to the spillway stilling basin wall. Additional seepage comes from the side of the wall, and the combined flow was estimated to be about 5 gpm. The top of this wall is badly deteriorated, probably due to freezing and thawing action from the seepage overtopping the wall. The line of seepage from the dam coincides with the alignment of the water supply line, and a swath some 20 ft. wide has been cut in the growth for the full length of the pipe. (Appendix C, Photo Nos. 2, 3 & 4) According to the Water Department's representative, this marshy area and its seepage was present, in about the same condition, as long ago as 1968.

There is some evidence of rodent infestation at mid-dam, on the lower part of the downstream slope, with one hole having recently been filled by the maintenance crew. (Appendix C, Photo No. 5) The downstream slope is grass covered and kept mowed.

c. Closure Dike

The upstream face of the dike is riprapped with large stone in good condition. The crest and downstream slope are clear of excessive growth. No seepage was apparent.

d. Spillway

Except as noted below, the spillway alignment appears to be good and concrete surfaces appear to be in good condition. No cracks or spallings of consequence were detected (Appendix C, Photo No. 6).

At the left stilling basin wall, an apparent leaning in of the wall has taken place, indicated by offsets at the joints of the two 12 ft. high wall panels. A 2 in. offset outward of the upper panel was measured at Station 0+76 (see Sheet 15 of 19, Appendix B); an outward offset of 2 in. of the lower panel was measured at Station 0+95; and an offset outward of 3½ in. of the lower panel was measured at Station 1+18. In an apparent belief that the walls were unstable and were being pushed over by backfill loadings in excess of design loads, in 1968 four steel beams were placed across the basin at the tops of the walls to act as struts (Appendix C, Photo No. 7). It was noted that granular soil is deposited on the channel bottom, which may have passed through the weep holes from the back of the wall, where loss of ground is evident.

An examination of the plans shows that the two stilling basin wall panels are cantilever walls, with a thin face wall cantilevered off the base. The wall panels upstream of the two cantilever sections are of much more rigid gravity wall construction and the end wall section downstream is rigidly supported by the buttress forming the control opening at the end of the stilling basin. It would thus be expected that the tops of the cantilever wall sections would deflect inwards because of cantilever action, while the sections above and below, being of more rigid construction, would deflect less. The left walls have deflected more than the right walls, which could be explained by greater backfill loadings occasioned by freeze and thaw action of water, which collected between the wall and backfill contact. A cursory examination showed no signs of distress in the walls and it is believed that they are in no danger of failing, either with or without the steel beam struts.

As was discussed in Section 1.3b, when spillway flows approach about 900 cfs, there is great danger of an overtopping of the left side chute wall where it converges into the direction of flow below the crest; and at the stilling basin, where the theoretical conjugate depth will be 10 ft., or to an average water surface 2 ft. below the tops of the walls (Appendix C, Photo No. 8). At flows exceeding about 1,400 cfs, the stilling basin walls will be overtopped; at about 2,700 cfs flow through the spillway, all walls will be overtopped.

Although no flashboards were on the spillway crest at the time of the inspection, outline stains of the boards were visible on the crest and holes have been drilled into the crest where such flashboards have been installed. According to Water Department personnel, these flashboards are installed each year in early spring and remain on the crest until fall, in order to capture additional runoff storage above spillway crest level and thereby increase the yield of the reservoir. The flashboards are 14 in. high, bolted to angle iron frame and supports inserted into the crest at their lower end and bolted to the upstream bridge beam at their top.

The use of flashboards on the spillway crest will significantly reduce the ability of the project to handle the larger magnitude floods, and is not consistent with the original design intent of the selected spillway capacity and reservoir surcharge and freeboard storage space. Once installed, it does not appear that the flashboards could be quickly removed in anticipation of a severe storm; and if suddenly removed while being overtopped, the flood wave occasioned by their removal would cause a severe flood wave down river, which would not be the case if gradual spills were released over the uncontrolled crest.

The inspection team was informed that in the past four years, only once did the reservoir rise above the tops of the boards, when several inches spilled over. In three of the last four years the reservoir storage reached above spillway crest level.

The concrete in the upstream curb of the spillway bridge deck is severely deteriorated, exposing the reinforcement. The original plywood forms on the underside of the deck between the steel beams have not been removed. These appear to trap water and roadway salts, leading to accelerated deterioration of the concrete. The bridge beams are badly rusted, presumably from the entrapment and dripping of water and roadway salts. It was also noted that the expansion joints at both bridge seats were open about 1½ in. in rather cool weather.

e. Reservoir Area

The reservoir shoreline and slopes upstream from the dam on both abutments are stable, with no evidences of slides or sloughing. The reservoir is in a restricted water supply preserve which is generally forested and no homes are constructed along the shoreline. There would be no damage upstream from the dam owing to a reservoir rise within the surcharge and freeboard space of the reservoir.

f. Downstream Channel

Spillway releases empty into Marsh Brook, which traverses a narrow valley for approximately one mile through the Town of East Plymouth, to merge with the Poland River near Terryville. Immediately downstream of the spillway stilling basin, the channel is somewhat overgrown with brush and saplings.

The Poland River continues through Terryville about one-half mile to merge with the Pequabuck River, which continues eastward through the City of Bristol. Many homes have been built near stream level along Marsh Brook in East Plymouth, and State Highway 72 follows along the left bank of Poland River for the entire reach from the Marsh Brook confluence to the Pequabuck. A constricted waterway exists under US Highway 6 and 202 crossing near the Poland-Pequabuck confluence, to limit river discharges at that point.

Discharges owing to an overtopping of the dike to the northwest of the dam would flow down a small tributary channel to the west of the main dam, which joins Marsh Brook about 1,500 ft. downstream from the dam. About five homes are built within the flow-way of this tributary channel, which would be affected by a spill over the dike.

3.2 Evaluation

The visual inspection of the dam and dike adequately revealed key characteristics as they may relate to its stability and integrity, permitting an assessment to be made of those features affecting the safety of the structure. The Old Marsh Pond Dam, the dike, and the appurtenant works are judged to be in generally good condition.

SECTION 4 - OPERATIONAL PROCEDURES

4.1 Procedures

The Old Marsh Pond Dam is operated by personnel of the Bristol Water Department. Reservoir operation entails mainly the release of stored water from the reservoir as water supply needs warrant. Flashboards are fixed to the spillway each spring and removed each fall. No documented operating procedures have been prepared.

4.2 Maintenance of Dam and Dike

Little maintenance is required except for the periodic cutting of brush and other growth on the dam embankment, and restoration of displaced riprap. No documented maintenance instructions have been prepared, but routine maintenance appears to be regularly carried out.

4.3 Maintenance of Operating Facilities

All gate valves are said to be serviceable and inspected regularly, except for the upstream 24 in. dia. valve, which is normally submerged and no longer used. No specific maintenance program is in effect.

4.4 Warning System

There is no formal warning system or program at this dam.

4.5 Evaluation

The Old Marsh Pond Dam is of relatively modern construction, with simple operating devices, and therefore does not require detailed operating procedures. Maintenance involves periodic growth removal from the dam and dike, and surveillance regarding seeps, slope damage, animal burrows, etc. Inspection observations noted that the facility appears to be generally well maintained. A formal warning and emergency evacuation system should be developed.

SECTION 5 - HYDRAULIC/HYDROLOGIC

5.1 Evaluation of Features

a. Design Data

1. Reservoir Area and Capacity

The active reservoir capacity to normal water surface elevation 687.7 is indicated by the Bristol Water Department to be 589 million gallons, or about 1,808 acre-feet. The surface area measured from contours on the USGS map is about 183 acres. From these values the reservoir capacities below normal level were reconstructed and computed, and are shown on Figure 2, Sheet D-6 and Sheet D-7, Appendix D. For surcharge storage above normal reservoir level, areas were planimetered from contours delineated on the USGS 2,000 feet per inch quadrangle sheets. Area and capacity curves and tables, for use in flood routings, are also shown on Sheets D-6 and D-7.

2. Flood Hydrology

Old Marsh Pond Dam is about 32 ft. high and impounds about 3,100 acre-feet of storage to top of dam, and as noted in Section 1.2c, is classified as intermediate in size. As noted in Section 1.2d, the hazard potential is classified as high. The Recommended Guidelines for Safety Inspection of Dams require that for hydraulic evaluation the dam adequacy be tested for floods of PMF magnitude.

Precipitation data was obtained from Hydrometeorological Report No. 30, which for the Connecticut area approximates 24.3 in. of 6 hour point rainfall over a 10 square mile area. This value was then reduced by 20 percent to allow for basin size, shape and fit factors. The 6-hour rainfall-duration curve of a total of 19.2 inches was then distributed and rearranged as suggested in Design of Small Dams. A constant loss factor of 0.1 inch per hour was deducted from the precipitation values to give the excess rainfall used to prepare an inflow hydrograph.

To prepare the inflow hydrograph a triangular incremental unitgraph was adopted, using a computed lag time value of about 1.5 hours to derive a time-to-peak for the triangular hydrograph of 1.5 hours (see computations on

Sheets D-8 and D-9, Appendix D). A PMF inflow hydrograph is shown on Figure 3 Sheet D-10, Appendix D, indicating a peak inflow of about 8,500 cfs or a CSM of about 3,600.

Flood routings were made of the PMF for two premises: first, for the present condition with the top of the dike at elevation 693.0; and second, for the condition where the dike had been raised 1.3 ft. so that its overtopping would not occur. Results of the routings are shown on Figure 4, Sheet D-11, and are summarized as follows:

	Present Condition Top of Dike at Elev. 693.0	Dike Raised With Top to Elev. 694.3
Maximum reservoir water surface	694.0	694.25
Maximum discharge thru spillway	3,940 cfs	4,175 cfs
Maximum total discharge over dam	0	306 cfs
Maximum unit discharge over dam	0	0.35 cfs/ft.
Total volume discharged over dam	0	12 A.F
Duration of overtopping of dam	0	1.4 hours
Maximum total discharge over dike	1,475 cfs	0
Maximum unit discharge over dike	2.8 cfs/ft.	0
Total volume discharged over dike	172 A.F	0
Duration of overtopping of dike	2.7 hours	0

From the above it can be seen that, with present conditions, for a PMF event the dike will be overtopped for a period of almost 3 hours, with a discharge over the dike of up to about 1,500 cfs and a total release of about 170 acre-feet; all assuming that the dike does not fail during the overtopping.

A flood routing of a 0.75 PMF through the reservoir and spillway results in a maximum surcharge to elevation 693.0, the top of the dike. This flood routing is shown on Figure 5, Sheet D-12. Floods in excess of this magnitude will result in an overtopping of the dike.

If the dike was raised about 1.3 feet to elevation 694.3 it would not be overtopped during a PMF event, while the dam would be overtopped by only 0.25 feet, with about a 300 cfs overflow for a total release of only about 12 acre-feet. In effect, the surcharge storage space and spillway capacity could handle about 95 percent of a PMF without an overtopping of the dam and dike. It is presumed that the thinking behind the designer's decision to construct the dike 1 ft. lower than the dam was to provide an emergency spillway and thereby avoid an overtopping of the main dam. From the flood routing results it would appear that an overtopping of the dike could be much more serious than an overtopping of the main dam, which would point to the desirability of consideration of raising the dike to prevent such an overtopping.

It is to be noted that the dam has two core walls extending to bedrock across its entire length, with the top of the higher core wall being 3 ft. below crest level. If the crest of the dam were to erode owing to an overtopping, it would not be expected that a sudden failure for the entire height would occur, but rather that the failure would be slowed by the core wall. On this premise, it would be safer that the dam rather than the dike be overtopped in the event of a maximum flood.

The installation of the 14 in. flashboards to gain additional storage within the surcharge storage space reduces the capability of the spillway and remaining surcharge to accommodate higher magnitude floods. On Figure 7, Sheet D-14, Appendix D, a flood routing of a 0.5 PMF shows that, if the flashboards are left in place and the reservoir is at the top of the flashboards at the start of the flood event, a maximum reservoir level of about 692.9 is obtained, just short of the top of the dike. On this basis, it would appear that if the dam is to be safe for handling floods up to a PMF event, the storage of inflows above spillway crest and the use of the flashboards during potential flood periods should be discontinued.

While the spillway crest has sufficient capacity to discharge about 4,000 cfs with reservoir level to the top of the dam, the spillway chute and stilling basin have apparently been designed for about 900 cfs, with what was considered adequate freeboard to safely confine outflows within the chute and basin waterway. As noted in Section 3.1d, computations show that the stilling basin will flow full to top of walls at a discharge of 1,400 cfs, and that an overtopping of the chute walls would occur when discharges exceed about 2,000 cfs. Shown on the following page, as plotted from Figure 6, Sheet D-13, are the spillway outflows resulting from various magnitude flood routings, and the expected consequences resulting at the spillway chute and basin from such outflows:

% of PMF	Spillway Outflow cfs	Spillway Channel Condition*	Remarks
28	900	3 ft. or more wall free-board in channel. 2.2 ft. wall freeboard in stilling basin.	Chute and basin will handle outflow with no wall over-topping threatened
40	1,400	About 1.5 ft. freeboard along chute walls. Stilling basin walls will overtop.	Stilling basin threatened with failure.
55	2,000	Left channel wall below crest will be overtopped. Stilling basin walls will be overtopped.	Overtopping of left chute wall will threaten dam left of spillway with wash-out and failure.
67	2,700	All walls will be over-topped, up to 4 feet at stilling basin.	Probable complete washout of chute and basin and undermin-ing of dam embankment.

*See channel hydraulic computations, Sheets D-4 and D-5, Appendix D.

b. Experience Data

No records have been retrieved in regard to past operation of the reservoir, or of surcharge encroachments and spills through the spillway. The maximum past inflows are not known.

c. Visual Observations

There is no visible evidence, either along the reservior or in the downstream channel, to indicate high water levels or signs of major spillway outflows.

d. Overtopping Potential

For a test flood of full PMF, the reservoir surcharge will reach to elevation 694.0, the top of the main dam. At this head, the dike 1,000 feet north of the right abutment of the dam will be overtopped by 1 ft., with a maximum release over the dike of about 1,475 cfs. This amount of overtopping could cause serious erosion of the dike, and a breach failure would be a distinct possibility.

e. Drawdown Capacity

Drawdown of the reservoir is possible only through the 16 in. dia. line which leads to Bristol Reservoir No. 1. On the basis of an average discharge of about 8 cfs, evacuation would be limited to about 16 acre-feet per day. To empty the 1,800 acre-feet of active storage below spillway crest would require almost 4 months, assuming no inflow in the interim. In the event of an emergency, a rapid drawdown could be carried out only by a controlled breaching of the dam or dike.

f. Downstream Hazard

As discussed in Section 5.d, for a PMF test flood the dike will be overtopped by 1 ft. and a breach failure would be likely. The dike could also conceivably fail structurally, such as by piping or sloughing. On the basis of a trapezoidal gap failure, with a bottom width of 20 ft. at the level of the bottom of the dike, and with slopes at about 1.4 to 1 (angle of repose), a sudden release of a flood wave approaching 7,600 cfs could result. This outflow would reduce to about 4,000 cfs in about 80 minutes during which time about 650 acre-feet of storage would be spilled down Marsh Brook. (See computations on Sheet D-15, Appendix D.)

If the dam were to be overtopped and fail, or to be breached due to structural failure, and the breach is a rectangular gap (assuming 50 ft. of core wall collapses), a sudden release of a flood wave of about 20,000 cfs could result. This outflow would reduce to about 10,000 cfs in 80 minutes during which time about 1,700 acre-feet of storage would be spilled down Marsh Brook. (See computations on Sheet D-15, Appendix D.) It should be noted, however, that the dam has two core walls to varying levels, and it is therefore unlikely that a sudden breach such as is demonstrated above would develop from either an overtopping or from a piping or sloughing failure.

As noted in Sections 1.2d and 3.1f, a number of homes are located along the lower reach of Marsh Brook and a major State Highway traverses the Poland and Pequabuck river below the Marsh-Poland confluence. Stage-discharge computations at a stream section downstream from the populated area on Marsh Brook and below the confluence of the Poland-Pequabuck Rivers show that flood stages of up to 20 ft. could prevail for a 20,000 cfs flood wave from the reservoir. (See computations on Sheet D-16.) Since the river channel storage in this reach of Marsh Brook and Poland River are small, the flood wave would be only slightly diminished until it passed a considerable distance down the Pequabuck River.

Delineated on Figure 8, Sheet D-17, (quad sheet graphic) are the areas which could be flooded by a breach failure of the dam or dike described above.

SECTION 6 - STRUCTURAL STABILITY

6.1 Evaluation of Structural Stability

a. Visual Observation

The field investigations of the embankment revealed no significant displacement or distress which would warrant the preparation of slope stability computations based on assumed soil properties and engineering factors.

However, while the dam is in good condition, the visual observations described in Section 3 indicate that attention should be given to a few items, which are noted in Section 7.3.

b. Design and Construction Data

Plans for construction of the original dam are unavailable, although the owners of the dam in 1943, the Andrew Terry Co., were then in possession of a plan of the dam prepared by Sperry and Buel, "engineers in charge of building the dam". In 1950, plans for the Bristol Board of Water Commissioners were prepared by Metcalf and Eddy to raise the dam about 10 ft. with an improved road on the crest, and to construct a new closure dike.

The design of the new dam and dike appear generally consistent with good earth dam embankment design practice at the time. The dam has a concrete core wall anchored 2 ft. deep in sound rock or impervious stratum, an ogee spillway and discharge channel, and intercepting toe drains. The dam is of impervious material, with a pervious shell downstream on a coarse sand filter. Hand placed 18-in. riprap was called for on the crest of the old dam, with heavy random riprap above that elevation. The upstream slope of the old dam was flattened and riprapped prior to superimposition of the new construction.

An apparent leaning in of the spillway stilling basin walls led to the installation, in 1968, of four steel beam struts between the tops of the walls. Since the plans show these walls to be of cantilever design, whereas the adjacent chute walls are of gravity design, it is believed that some differential deflection can be expected and that these walls are in no danger of failing, either with or without the struts.

Pertinent drawings are included in Appendix B.

c. Operating Records

Records are kept by the Bristol Water Department. A rain gauge is maintained at the filter plant about 1 mile downstream from the dam.

d. Post Construction Changes

Designs for the raising of the original dam and reconstruction of the original dike were well conceived, were in accord with accepted practice of the period, and would not adversely affect stability or integrity.

e. Seismic Stability

The dam is located in Seismic Zone No. 1, and in accordance with Phase I guidelines, does not warrant seismic analysis.

SECTION 7 - ASSESSMENT, RECOMMENDATIONS & REMEDIAL MEASURES

7.1 Dam Assessment

a. Condition

On the basis of the Phase I visual examination, both Old Marsh Pond Dam and the closure dike appear to be in good condition and functioning adequately. The deficiencies revealed are not of major concern, but tend to indicate that a small amount of additional routine maintenance is required, and that the fairly heavy seepage should be monitored frequently.

The closure dike is 1 ft. lower than the main dam. The reservoir cannot be drawn down rapidly because the blowoff on the 16 in. dia. outlet pipe to the filter plant is only 6 in. dia. The spillway crest capacity is only sufficient to accommodate about 75% of the PMF test flood without overtopping the dike.

At the test flood outflow, the spillway chute and stilling basin walls will all be overtopped. At a 0.5 PMF outflow, the stilling basin walls will be overtopped and the left channel wall below the crest will be on the verge of overtopping. Overtopping of the walls would result in washouts of backfill along the walls, and could result in the collapse of the entire spillway chute and stilling basin. A washout of the spillway channel so close to the dam could threaten a washout of the dam for inflows of much less than the test flood.

b. Adequacy of Information

The information recovered is considered adequate for the purpose of making an assessment of the performance of the dam.

c. Urgency

The recommendations and remedial measures enumerated below should be implemented by the owner within two years after receipt of the Phase I Inspection Report.

d. Need for Additional Investigation

The visual inspection identified a number of potential problems. Additional investigations are required as recommended in Para. 7.2.

7.2 Recommendations

It is recommended that the owner should retain the services of a competent registered professional engineer to make investigations and studies of the following items, and, if proved necessary, design appropriate remedial works:

1. Review the situation regarding the dam overtopping versus the dike overtopping, and determine whether the dike should be raised.

2. Review spillway flow conditions below the crest of the dam at the converging wall, and determine whether modifications are required to improve flow and increase carrying capacity in the chute, and to forestall failure from wall overtopping.

3. Review the use of flashboards on the spillway crest and determine the feasibility of either eliminating their use altogether, or modifying them to facilitate quick removal in anticipation of a storm.

4. Determine the source of leakage producing the swampy area below the dam. Since the thread of the seepage stream coincides with the line of the outlet pipe and its associated gates and valves, it is advisable to inspect carefully each valve and gate chamber, including those abandoned, for possible contributing sources to the leakage. While it is probable that most of the seepage issues from the rock toe drain, faced by the supplementary peastone, the presence of other channels which may have been induced by reconstruction cannot be discounted. Confining the seepage to a constructed, well-defined channel would be useful for monitoring, would improve soil conditions at the toe, and would prevent further damage from freeze-thaw cycles to the wall which extends from the right stilling basin wall.

5. Investigate possible causes of rotation and/or deflection of spillway chute wall, and examine corrective alternatives.

6. Provision of a new outlet facility to permit rapid drawdown of the reservoir.

7.3 Remedial Measures

The owner should take the following actions:

1. Monitor on a monthly basis the fairly heavy seepage at the center of the dam, noting changes in volume and turbidity.

2. Monitor once per month the marshy area to the right of the maintenance building access ramp, at the toe of the slope.

3. Restore riprap in the vicinity of the spillway's upstream walls.

4. Control growth on the right downstream slope.

5. Fill animal burrows, and patrol the downstream slope frequently to detect and eliminate future infestations.

6. Repair the top of the concrete wall which extends from the right stilling basin wall.

7. Remove the plywood forms from the underside of the bridge deck; make all necessary repairs; paint all steelwork.

a. Operation and Maintenance Procedures

The owner should institute procedures for a biennial periodic technical inspection of the dam and appurtenant works, with supplementary inspections for any suspect items. A checklist for periodic inspections should be developed and records should be kept of all maintenance and repair work performed. A formal surveillance, flood warning and emergency evacuation plan should also be developed.

7.4 Alternatives

There appear to be no practical alternatives to the above recommendations.

APPENDIX A

VISUAL INSPECTION CHECKLIST

VISUAL INSPECTION
PHASE I

Identification No. CT 00285 Name of Dam: Old Marsh Pond Dam

Date of Inspection: 9 November 1978

Weather: Clear Temperature: 55°F

Pool Elevation at Time of Inspection: 684 MSL

Tailwater Elevation at Time of Inspection: Not applicable

INSPECTION PERSONNEL

Pasquale E. Corsetti	Louis Berger & Associates, Inc.	Acting Proj. Manager
Carl J. Hoffman	Louis Berger & Associates, Inc.	Hydraulics, Structures
Thomas C. Chapter	Louis Berger & Associates, Inc.	Hydrology, Soils
James H. Reynolds	Goldberg Zoino Dunnicliff & Associates, Inc.	Soils

OWNER'S REPRESENTATIVE

John M. Knibbs	City of Bristol	Assistant Superintendent, Water Department

VISUAL INSPECTION CHECKLIST

Identification No. CT 00285 Name of Dam: Old Marsh Pond Dam Sheet 1

VISUAL EXAMINATION OF	OBSERVATIONS AND REMARKS
EMBANKMENT Vertical alignment and movement	Alignment of dam and dike good. No movement observed.
Horizontal alignment and movement	Alignment of dam and dike good. No movement observed.
Unusual movement or cracking at or near the toe	None observed.
Surface cracks	None observed.
Animal burrows and tree growth	Two fairly large burrows about mid-dam, mid-slope on d/s face.
Sloughing or erosion of slopes	Minor sloughing on upstream face near spillway walls.
Riprap slope protection	Generally good condition. Some displaced downslope at spillway walls.

VISUAL INSPECTION CHECKLIST

Identification No. CT 00285 | Name of Dam: Old Marsh Pond Dam | Sheet 2

VISUAL EXAMINATION OF	OBSERVATIONS AND REMARKS
Seepage	Marshy area, toe of dam, right of access ramp. Heavy seepage from 10 ft. zone at toe drain, 150 ft. right of spillway; marshy zone with free water extends to end of spillway channel; total combined flow about 5 gpm.
Piping or boils	None noted.
Junction of embankment and abutment, spillway and dam	Minor sloughing and displacement of riprap each side of spillway.
Foundation drainage	Granular filter zone under permeable shell; intercepting granular drains; cross underdrains under spillway channel; toe drains.
OUTLET WORKS Approach channel	None.
Outlet conduit concrete surfaces	None.
Intake structure	Concrete valve chamber on u/s slope, condition good.

VISUAL INSPECTION CHECKLIST

Identification No. CT 00285 | Name of Dam: Old Marsh Pond Dam | Sheet 3

VISUAL EXAMINATION OF	OBSERVATIONS AND REMARKS
Outlet structure	None.
Outlet channel	None (16 in. dia. buried pipe).
Drawdown facilities	16 in. dia. pipe to Reservoir No. 1 one mile d/s, with 6 in. dia. blowoff to spillway stilling basin.
<u>SPILLWAY STRUCTURES</u> Concrete weir	Good condition.
Approach channel	None.
Discharge channel	Converging chute with concrete walls and retarding sill; good condition.
Stilling basin	Hydraulic jump type with concrete walls which have deflected inwards and are braced with 4 steel beam struts.

VISUAL INSPECTION CHECKLIST

Identification No. CT 00285 | Name of Dam: Old Marsh Pond Dam | Sheet 4

VISUAL EXAMINATION OF	OBSERVATIONS AND REMARKS
Bridge and piers	Plywood deck forms not removed; concrete curbs deteriorated; exp. joints open 1½"; steel work corroded.
Control gates and operating machinery	None.
INSTRUMENTATION Headwater and tailwater gages	None.
Embankment instrumentation	None.
Other instrumentation	None.
RESERVOIR Shoreline	Gentle slopes, wooded, stable.
Sedimentation	None observed.
Upstream hazard areas in event of backflooding	None.

VISUAL INSPECTION CHECKLIST

Identification No. CT 00285 | Name of Dam: Old Marsh Pond Dam | Sheet 5

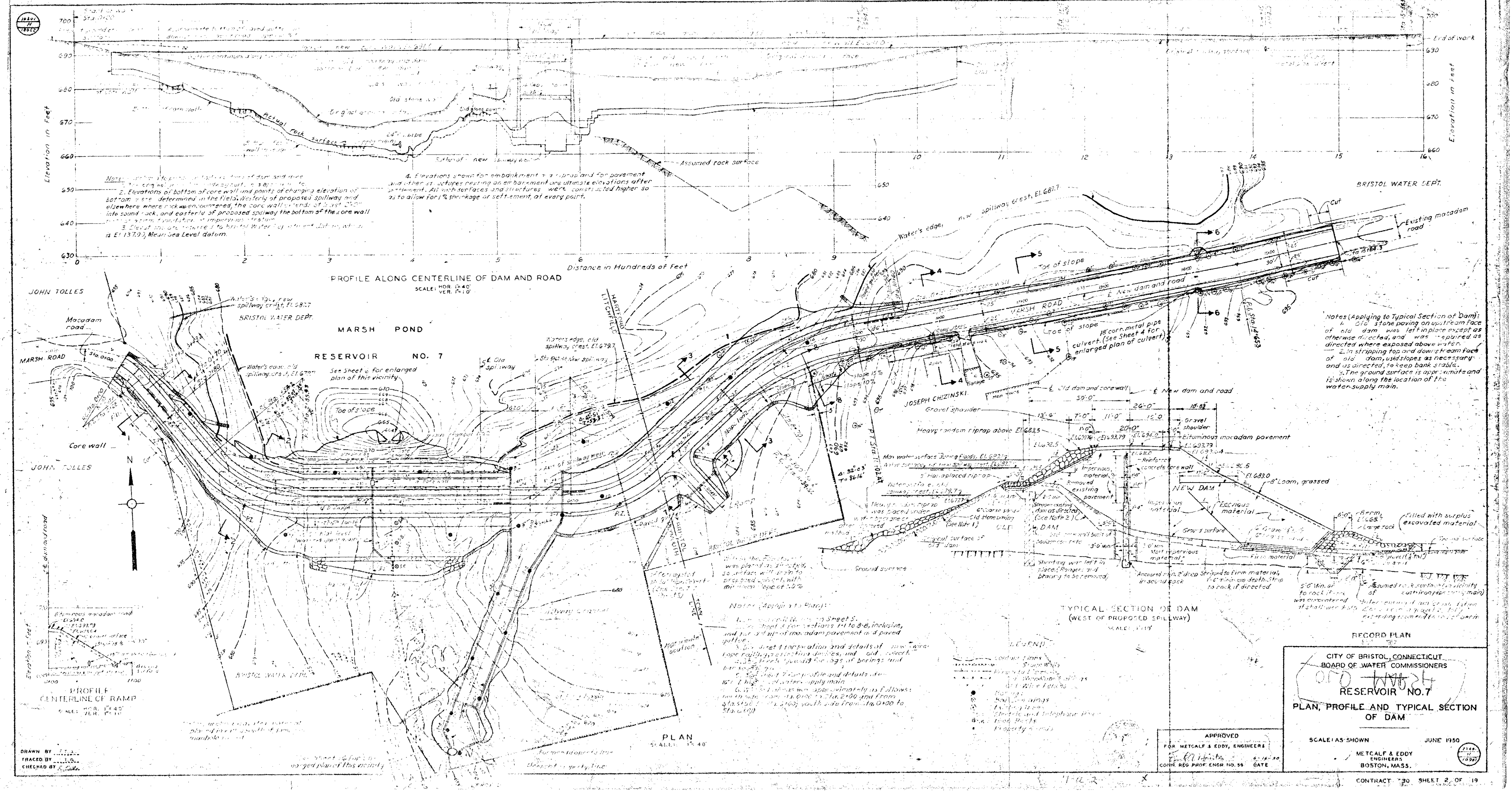
VISUAL EXAMINATION OF	OBSERVATIONS AND REMARKS
Alterations to watershed affecting runoff	None apparent.
DOWNSTREAM CHANNEL Constraints on operation of dam	None.
Valley section	Wide, natural.
Slopes	Gentle, wooded.
Approx. No. of homes/population	At least 15 homes in E. Plymouth and Terryville close to channel.
OPERATION & MAINTENANCE FEATURES Reservoir regulation plan, normal conditions	No formal plan. 14 in. flashboards added in spring, removed in fall, for additional storage.
Reservoir regulation plan, emergency conditions	No formal plan.
Maintenance features	Brush cut, grass mowed, animal burrows filled, etc.

APPENDIX B

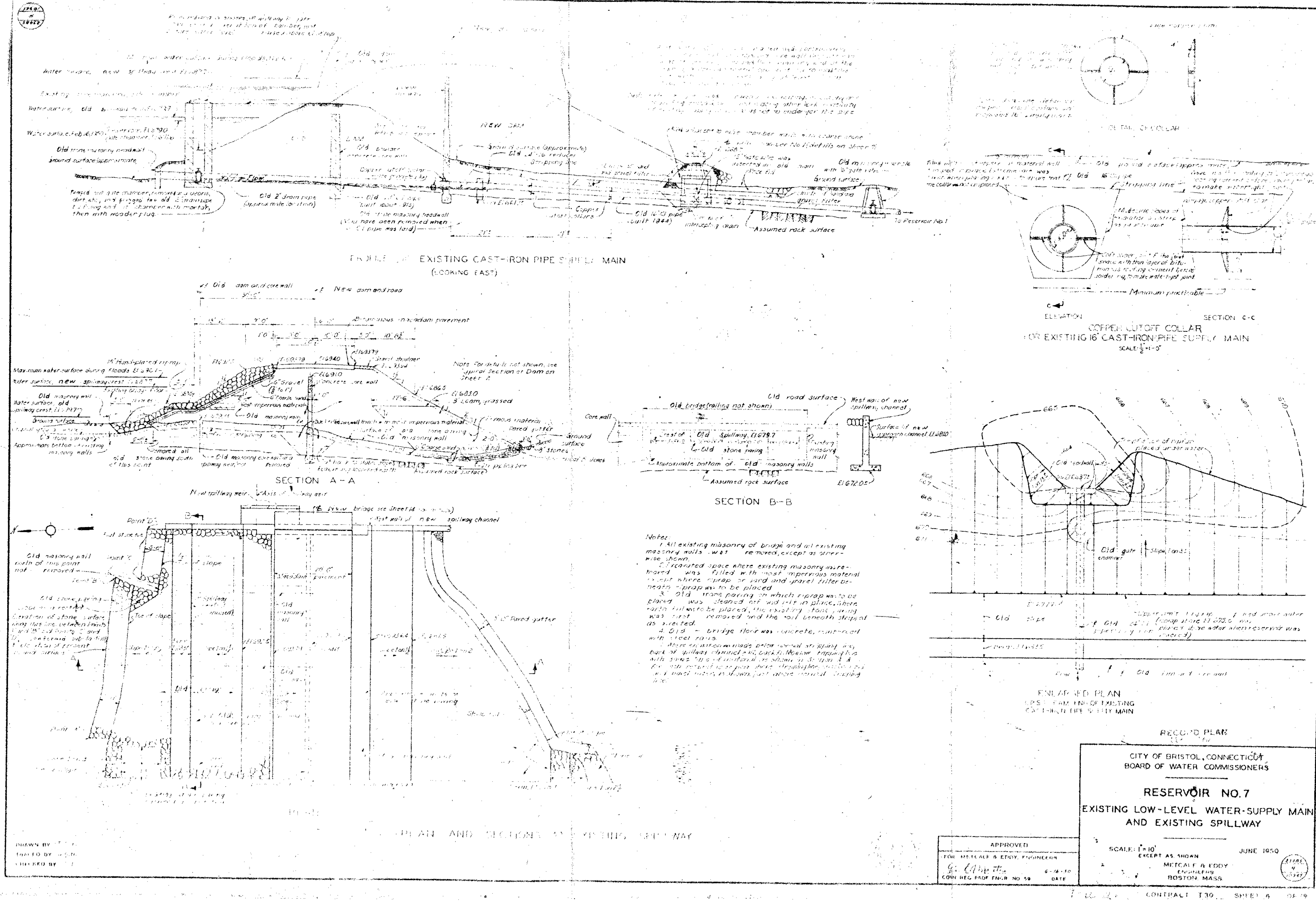
PLANS & RECORDS

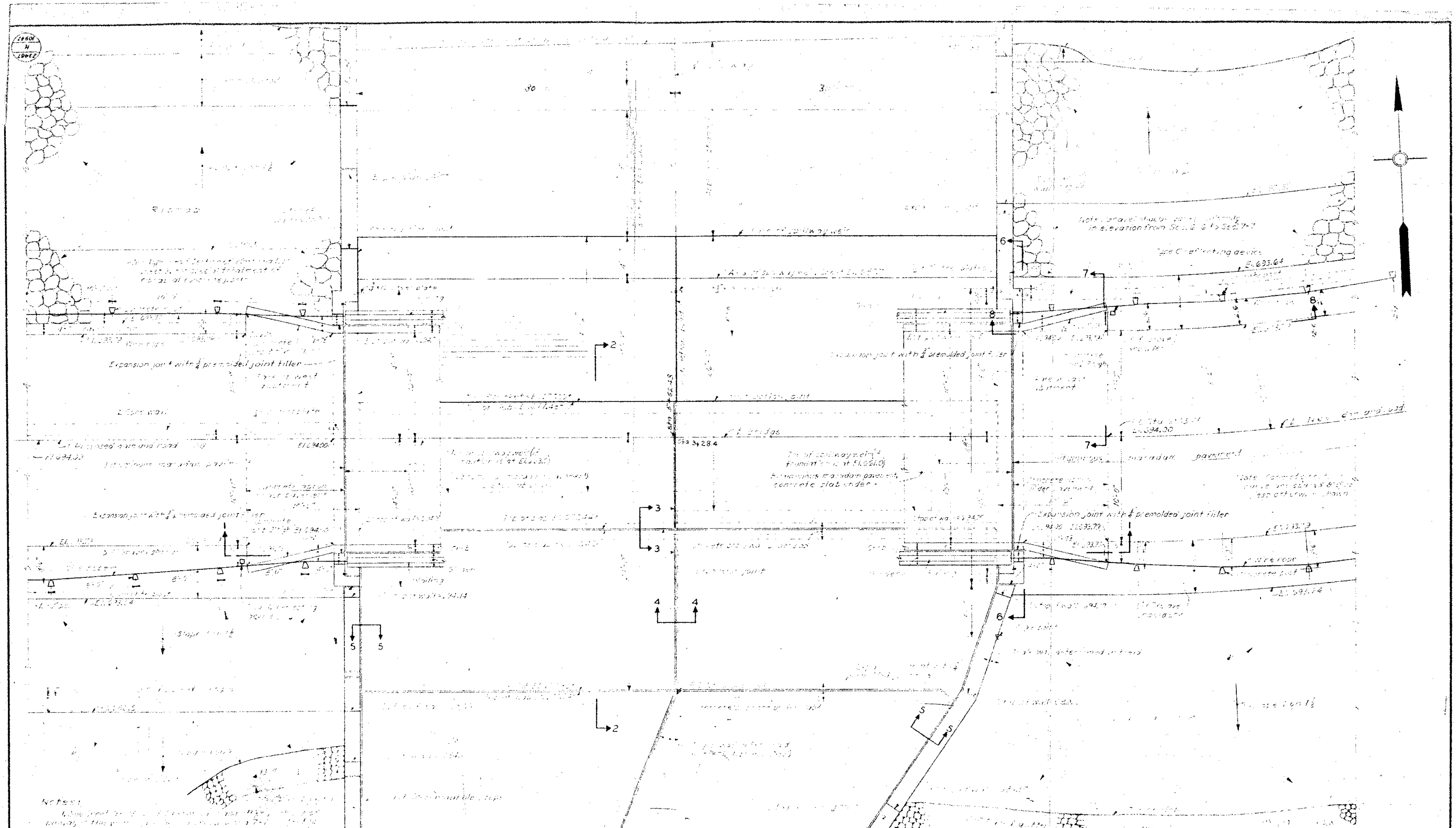
Plans obtained from Water Department, City of Bristol, CT:

Plan, Profile & Typical Section of Dam	Sheet 2 of 19
Existing Low-Level Water Supply Main and Existing Spillway	Sheet 6 of 19
Bridge & Adjacent Spillway; General Plan	Sheet 8 of 19
Bridge & Adjacent Spillway; Plan & Elevation of East Abutment	Sheet 9 of 19
Bridge & Adjacent Spillway; Misc. Details	Sheet 13 of 19
Spillway Channel & Walls; Plan, Profile & Typical Section	Sheet 15 of 19
Spillway Channel & Walls; Misc. Details Vicinity of Outlet	Sheet 16 of 19
Plan, Profile & Sections of Dike	Sheet 18 of 19
Borings & Test Pits	Sheet 19 of 19



Elevation in Feet
Assumed rock surface
Distance in Hundreds of Feet
PROFILE ALONG CENTERLINE OF DAM AND ROAD
SCALE: HOR. 1"=40' VER. 1"=10'
End of work
BRISTOL WATER DEPT.
Existing macadam road
Toe of slope
Cut
MARSH ROAD
New dam and road
Notes (Applying to Typical Section of Dam):
1. Old stone paving on upstream face of old dam was left in place except as otherwise directed, and was repaired as directed where exposed above water.
2. In stripping top and downstream face of old dam, used slopes as necessary and as directed, to keep bank stable.
3. The ground surface is approximate and is shown along the location of the water-supply main.
JOHN TOLLES
Macadam road
MARSH ROAD
BRISTOL WATER DEPT.
MARSH POND
RESERVOIR NO. 7
Toe of slope
Core wall
JOSEPH CHIZINSKI
Old dam and core wall
New dam and road
Gravel shoulder
Bituminous macadam pavement
NEW DAM
Berm
Filled with surplus excavated material
Ground surface
TYPICAL SECTION OF DAM
(WEST OF PROPOSED SPILLWAY)
LEGEND
PROFILE
CENTERLINE OF RAMP
PLAN
SCALE: 1"=40'
DRAWN BY
TRACED BY
CHECKED BY
APPROVED
FOR METCALF & EDDY, ENGINEERS
CONN. REG. PROF. ENGR. NO. 59
6-16-50
DATE
RECORD PLAN
CITY OF BRISTOL, CONNECTICUT
BOARD OF WATER COMMISSIONERS
RESERVOIR NO.7
PLAN, PROFILE AND TYPICAL SECTION OF DAM
SCALE: AS SHOWN
JUNE 1950
METCALF & EDDY
ENGINEERS
BOSTON, MASS.
CONTRACT 730 SHEET 2 OF 19


PROFILE OF EXISTING CAST-IRON PIPE SUPPLY MAIN
(LOOKING EAST)
NEW DAM
To Reservoir No. 1
Assumed rock surface
SECTION A – A
SECTION B – B
Old road surface
Old bridge (railing not shown)
Assumed rock surface
El.672.0±
DETAIL OF COLLAR
ELEVATION
SECTION C-C
COPPER CUTOFF COLLAR
FOR EXISTING 16" CAST-IRON PIPE SUPPLY MAIN
SCALE: 1/2"=1'-0"
ENLARGED PLAN
UPSTREAM END OF EXISTING
CAST-IRON PIPE SUPPLY MAIN
PLAN AND SECTIONS OF EXISTING SPILLWAY
Notes:
1. All existing masonry of bridge and all existing masonry walls was removed, except as otherwise shown.
2. Excavated space where existing masonry was removed was filled with most impervious material except where riprap or sand and gravel filter beneath riprap was to be placed.
3. Old stone paving on which riprap was to be placed was cleaned off and left in place. Where earth fill was to be placed, the existing stone paving was first removed and the soil beneath stripped as directed.
4. Old bridge floor was concrete, reinforced with steel rods.
RECORD PLAN
CITY OF BRISTOL, CONNECTICUT
BOARD OF WATER COMMISSIONERS
RESERVOIR NO. 7
EXISTING LOW-LEVEL WATER-SUPPLY MAIN
AND EXISTING SPILLWAY
SCALE: 1"=10'
EXCEPT AS SHOWN
JUNE 1950
METCALF & EDDY
ENGINEERS
BOSTON, MASS.
APPROVED
FOR METCALF & EDDY, ENGINEERS
CONN. REG. PROF. ENGR. NO. 59
DATE
CONTRACT T30
SHEET 8



CITY OF BRISTOL, CONNECTICUT
BOARD OF WATER COMMISSIONERS

RESERVOIR NO. 7
BRIDGE AND ADJACENT SPILLWAY
GENERAL PLAN

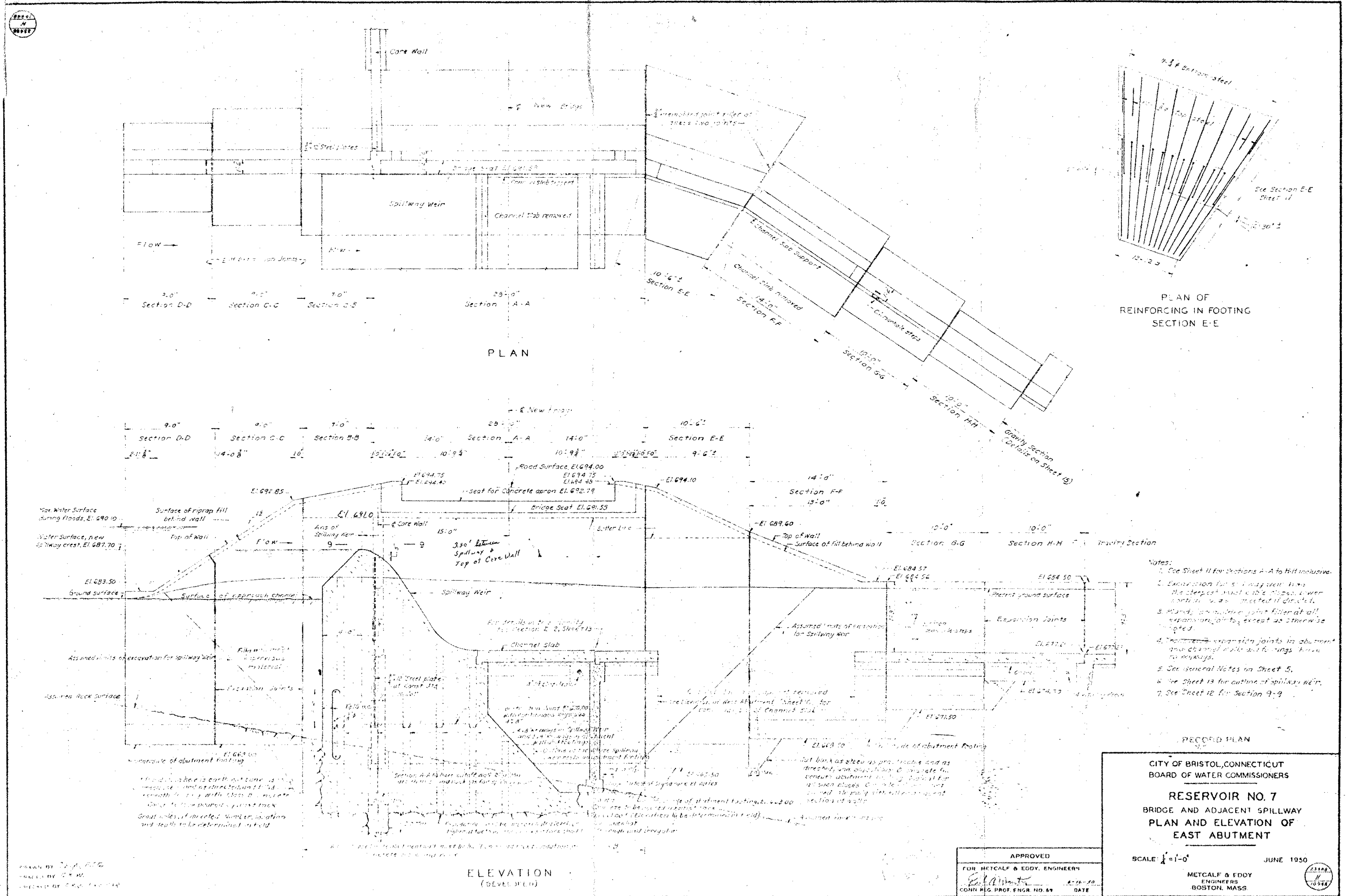
SCALE: $\frac{1}{4}$" = 1'-0"

JUNE 1950

METCALF & EDDY
ENGINEERS
BOSTON, MASS.

APPROVED
FOR METCALF & EDDY, ENGINEERS
CONN. REG. PROF. ENGR. NO. 59 DATE

CONTRACT T30 SHEET 8 OF 19



CITY OF BRISTOL, CONNECTICUT
BOARD OF WATER COMMISSIONERS

RESERVOIR NO. 7

BRIDGE AND ADJACENT SPILLWAY

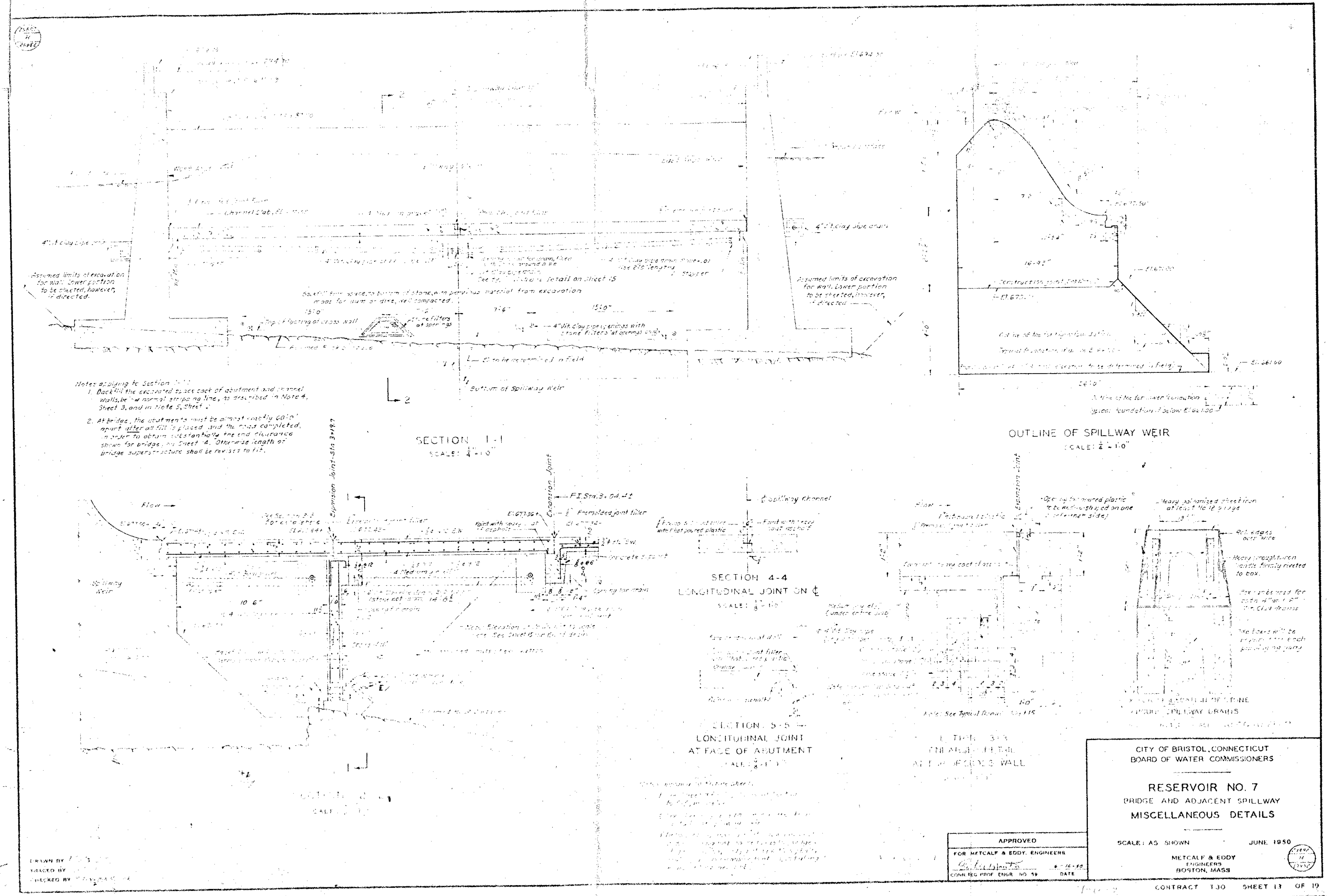
PLAN AND ELEVATION OF EAST ABUTMENT

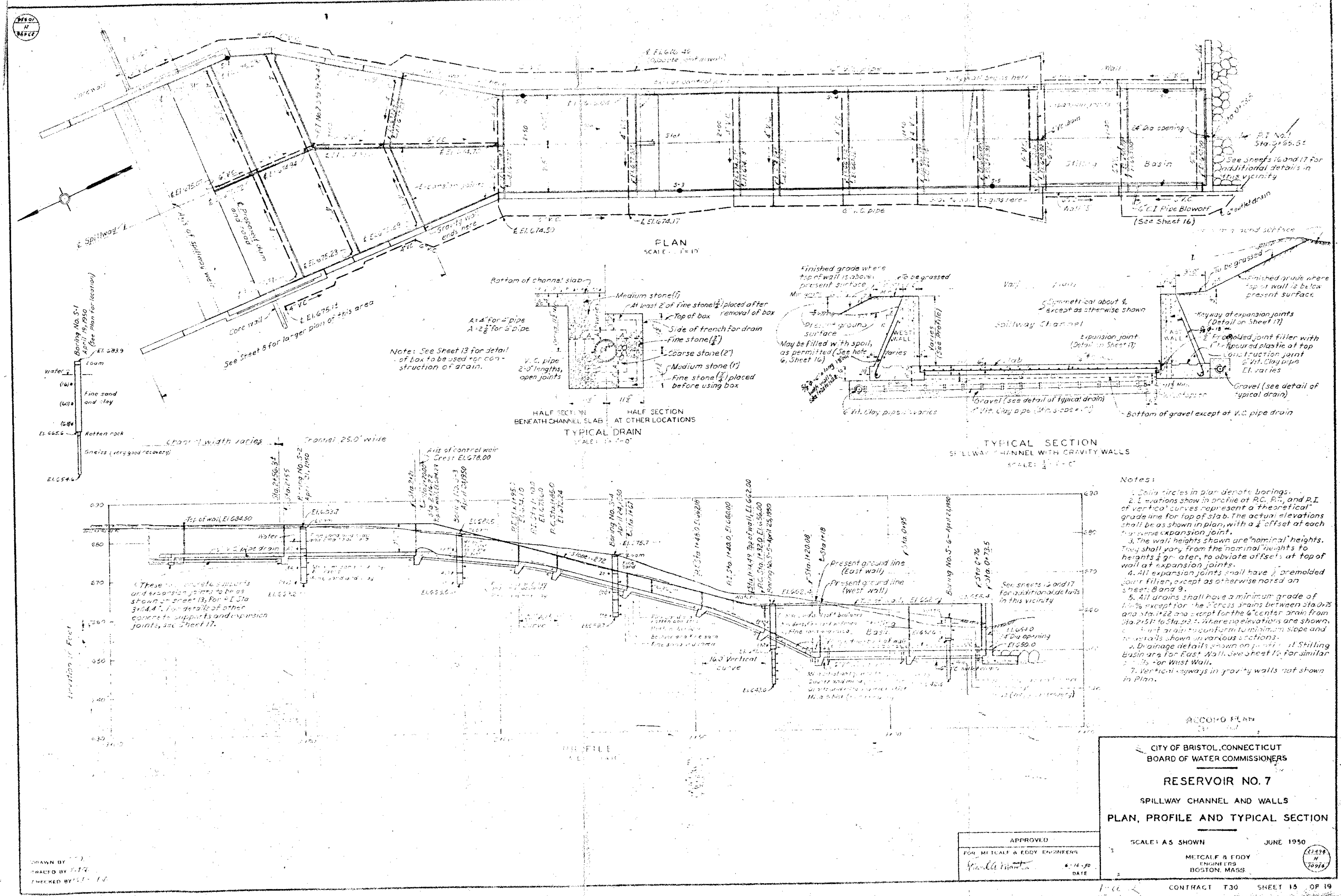
SCALE: 1/4" = 1'-0"   JUNE 1950

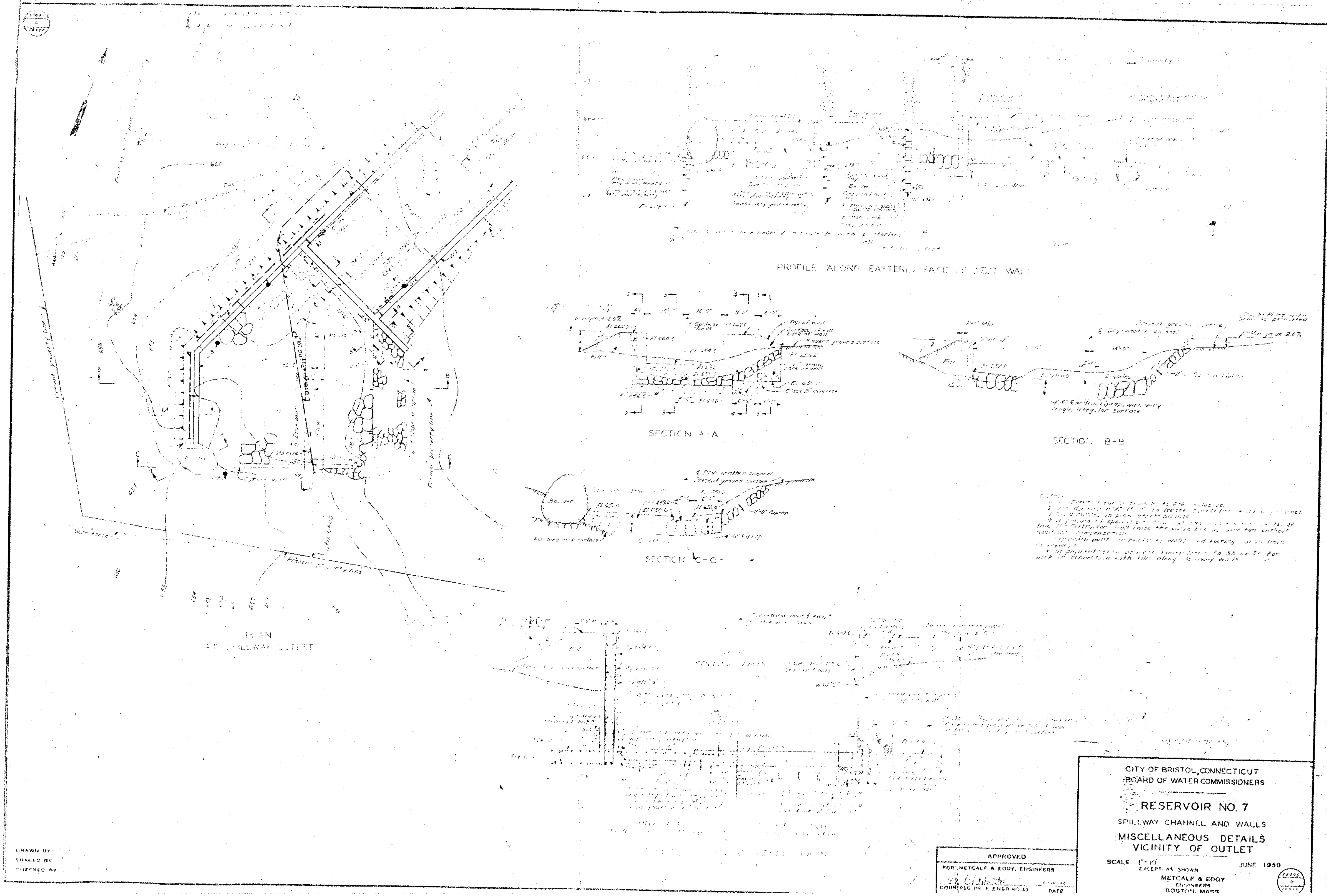
METCALF & EDDY
ENGINEERS
BOSTON, MASS.

APPROVED
FOR METCALF & EDDY, ENGINEERS
CONN. REG. PROF. ENGR. NO. 69   DATE

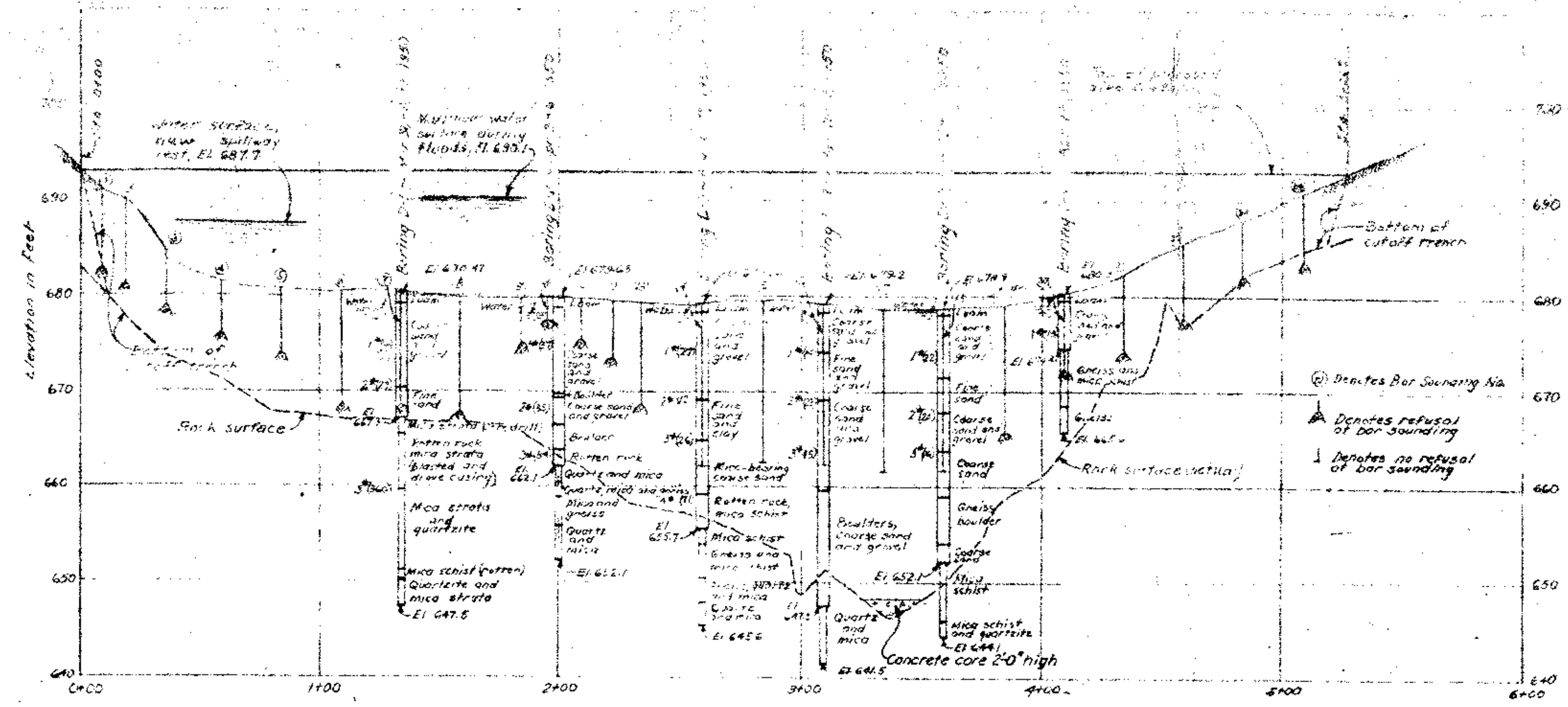
CONTRACT T30   SHEET 9 OF 19


SECTION 1-1
SCALE: 1/4" = 1'-0"
Notes applying to Section 1-1:
1. Backfill the excavated space back of abutment and channel walls, below normal stripping line, as described in Note 4, Sheet 3, and in Note 5, Sheet 5.
2. At bridge, the abutments must be almost exactly 60'0" apart after all fill is placed and the road completed, in order to obtain substantially the end clearance shown for bridge on Sheet 14. Otherwise length of bridge superstructure shall be revised to fit.
Assumed limits of excavation for wall. Lower portion to be sheeted, however, if directed.
4" Vit. clay pipe openings with stone filters at openings
Bottom of Spillway Weir
OUTLINE OF SPILLWAY WEIR
SCALE: 1/2" = 1'-0"
Construction joint
Flow
Spillway Weir
Expansion Joint
Premolded joint filler
F.I. Sta. 3+04.41
SECTION 4-4
LONGITUDINAL JOINT ON ℄
℄ Spillway Channel
SECTION 5-5
LONGITUDINAL JOINT
AT FACE OF ABUTMENT
Heavy galvanized sheet iron
Heavy wrought iron
CITY OF BRISTOL, CONNECTICUT
BOARD OF WATER COMMISSIONERS
RESERVOIR NO. 7
BRIDGE AND ADJACENT SPILLWAY
MISCELLANEOUS DETAILS
SCALE: AS SHOWN
JUNE 1950
METCALF & EDDY
ENGINEERS
BOSTON, MASS.
APPROVED
FOR METCALF & EDDY, ENGINEERS
CONN. REG. PROF. ENGR. NO. 59
DATE
DRAWN BY
TRACED BY
CHECKED BY
CONTRACT T30
SHEET 13 OF 19

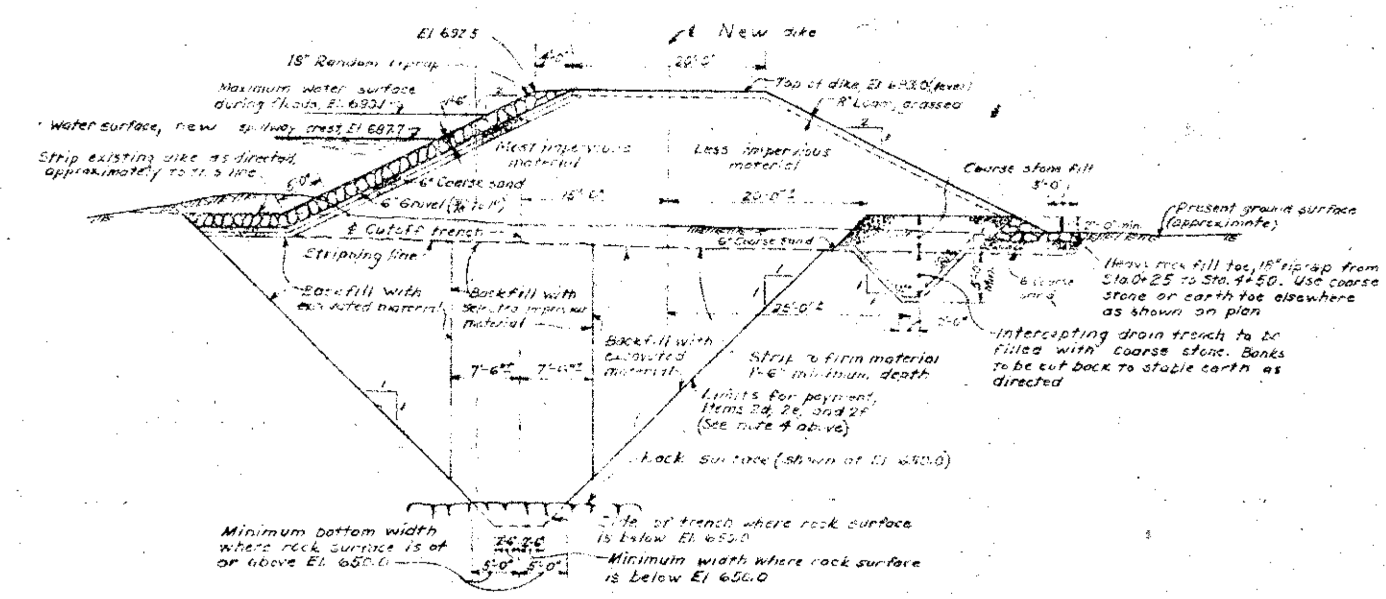

PLAN
TYPICAL DRAIN
HALF SECTION BENEATH CHANNEL SLAB
HALF SECTION AT OTHER LOCATIONS
Note: See Sheet 13 for detail of box to be used for construction of drain.
Bottom of channel slab
Medium stone(1")
At least 2' of fine stone(3/4") placed after removal of box
Top of box
Side of trench for drain
Fine stone(3/4")
Coarse stone(2")
Fine stone(3/4") placed before using box
A=4" for 4" pipe
A=2 3/8" for 6" pipe
V. C. pipe 2'-0" lengths, open joints
TYPICAL SECTION
SPILLWAY CHANNEL WITH GRAVITY WALLS
Symmetrical about ℄ except as otherwise shown
Spillway Channel
Finished grade where top of wall is above present surface
Finished grade where top of wall is below present surface
To be grassed
Present ground surface
May be filled with spoil, as permitted (See Note 6, Sheet 16)
Keyway at expansion joints (Detail on Sheet 17)
Expansion joint (Detail on Sheet 17)
Gravel (see detail of typical drain)
Bottom of gravel except at V.C. pipe drain
6" Vit. Clay pipe El. varies
See Sheets 16 and 17 for additional details in this vicinity
P.I. No.1 Sta.0+65.5±
24" Dia opening
Stilling Basin
6" C.I. Pipe Blowoff (See Sheet 16)
Gravity wall ends here
Core wall
See Sheet 8 for larger plan of this area
℄ Spillway
Boring No. S-1 April 19, 1950
Loam
Fine sand and clay
Rotten rock
Gneiss (very good recovery)
Channel width varies
Channel 25.0' wide
Axis of control weir Crest El. 678.00
Top of wall El. 684.50
Present ground line (East wall)
Present ground line (West wall)
16.3' Vertical curve
Elevation in Feet
PROFILE
Notes:
1. Solid circles in plan denote borings.
2. Elevations shown in profile at P.C., P.T., and P.I. of vertical curves represent a "theoretical" grade line for top of slab. The actual elevations shall be as shown in plan, with a 1/4" offset at each transverse expansion joint.
3. The wall heights shown are "nominal" heights. They shall vary from the "nominal" heights to heights 1/4" greater, to obviate offsets at top of wall at expansion joints.
4. All expansion joints shall have 1/2" premolded joint filler, except as otherwise noted on sheets 8 and 9.
5. All drains shall have a minimum grade of 1.0% except for the cross drains between Sta. 0+75 and Sta. 1+22 and except for the 6" center drain from Sta. 2+51 to Sta. 3+3. Where no elevations are shown, drain to conform to minimum slope and details shown on various sections.
6. Drainage details shown on profile of Stilling Basin are for East Wall. See Sheet 16 for similar details for West Wall.
7. Vertical keyways in gravity walls not shown in Plan.
RECORD PLAN
CITY OF BRISTOL, CONNECTICUT
BOARD OF WATER COMMISSIONERS
RESERVOIR NO. 7
SPILLWAY CHANNEL AND WALLS
PLAN, PROFILE AND TYPICAL SECTION
SCALE: AS SHOWN
JUNE 1950
METCALF & EDDY ENGINEERS BOSTON, MASS.
APPROVED
FOR METCALF & EDDY ENGINEERS
DATE
DRAWN BY
TRACED BY
CHECKED BY
CONTRACT T30
SHEET 15 OF 19


PROFILE ALONG EASTERLY FACE OF WEST WALL
SECTION A-A
SECTION B-B
SECTION C-C
PLAN
AT SPILLWAY OUTLET
DRAWN BY
TRACED BY
CHECKED BY
APPROVED
FOR METCALF & EDDY, ENGINEERS
CONN. REG. PROF. ENGR. NO. 59
DATE
CITY OF BRISTOL, CONNECTICUT
BOARD OF WATER COMMISSIONERS
RESERVOIR NO. 7
SPILLWAY CHANNEL AND WALLS
MISCELLANEOUS DETAILS
VICINITY OF OUTLET
SCALE
EXCEPT AS SHOWN
JUNE 1950
METCALF & EDDY
ENGINEERS
BOSTON, MASS.

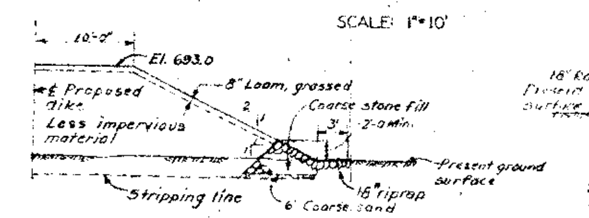
Notes:
1. [illegible] trench shall be carried to rock [illegible]
2. Intercepting drain trench extends from Sta. [illegible] to Sta. 4+25±.
3. Top of embankment [illegible]
4. The 1 to 1 side slopes shown for cutoff trench are for payment for earth excavation and refill. The actual slopes may be different.



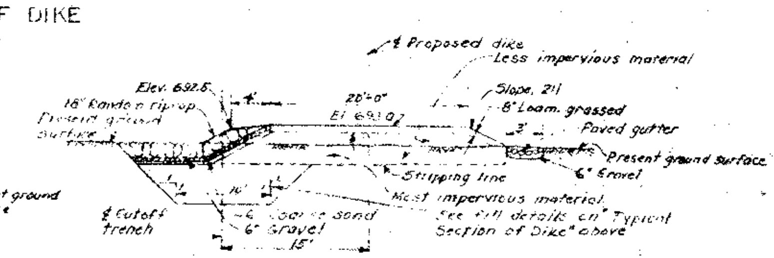
PROFILE ALONG CENTER-LINE
SCALE - HORZ. 1"=40' VERT. 1"=10'



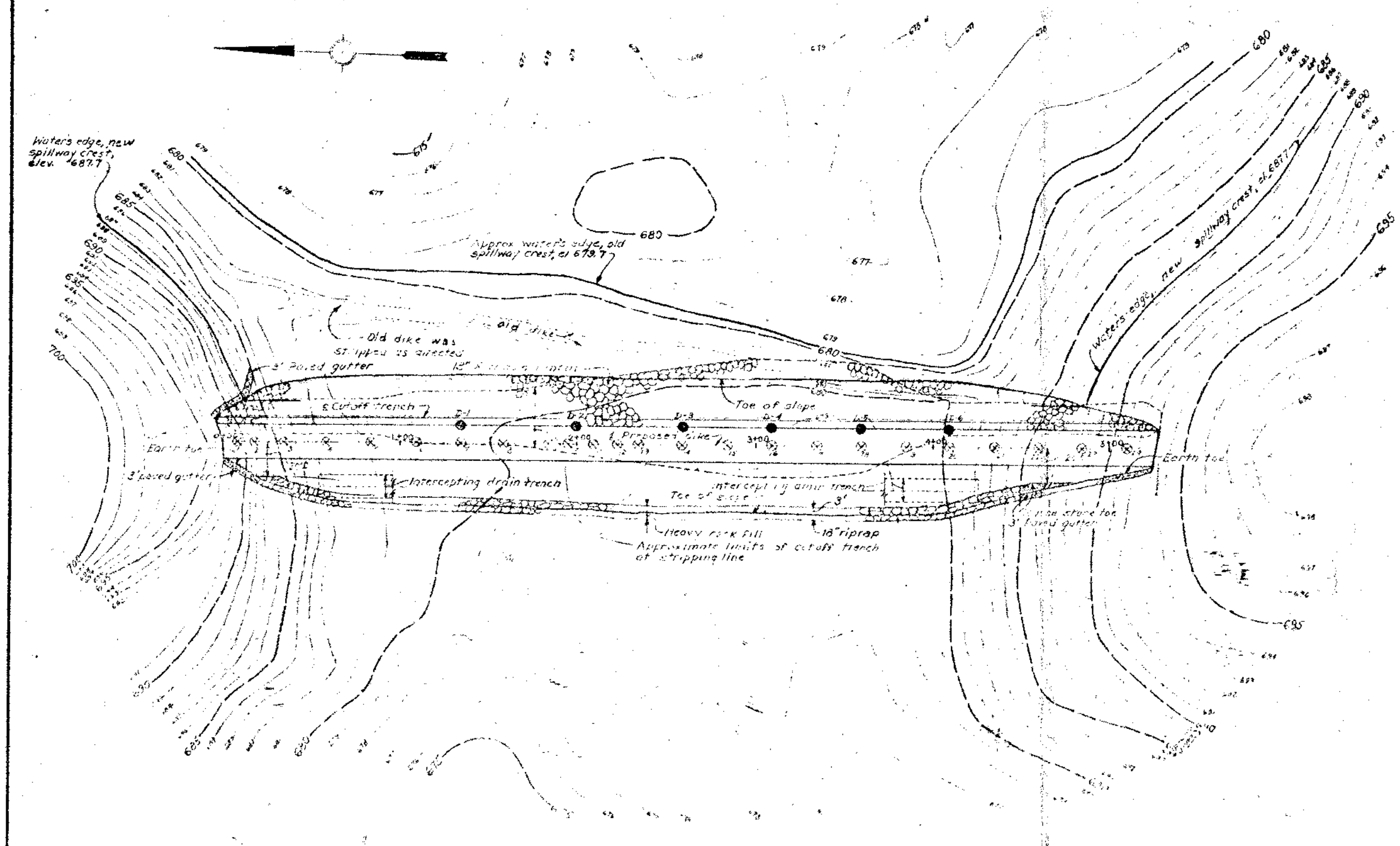
TYPICAL SECTION OF DIKE
SCALE: 1"=10'



HALF SECTION AT STA. 4+50
SHOWING TYPICAL COARSE STONE FILL AT TOE
SCALE: 1"=10'



SECTION AT STA. 5+10
SHOWING TYPICAL DETAILS EACH END OF DIKE
SCALE: 1"=10'



PLAN

Notes:
1. See Sheet 2 for legend, Sheet 5 for General Notes, and Sheet 15 for General Boring Notes.
2. When the borings shown on this sheet were made, the water surface in the reservoir was approximately El. 673.7.
3. Bar soundings shown on this sheet (and also those shown on Sheet 2) were made by forces of Bristol Water Department by driving a 7/8" round pointed bar into the ground. The bar was driven to 7'-0" depth manually by sledge hammer, then to greater depths by use of an air hammer. Various lengths of bar were used, from short to long (17.0) as the depth of the hole increased.
4. Certain shallow test pits were excavated along the center line of the dike by Water Department forces. Records of these test pits and samples therefrom were examined by prospective bidders at the office of the Board of Water Commissioners.

RECORD PLAN

DRAWN BY
TRACED BY
CHECKED BY

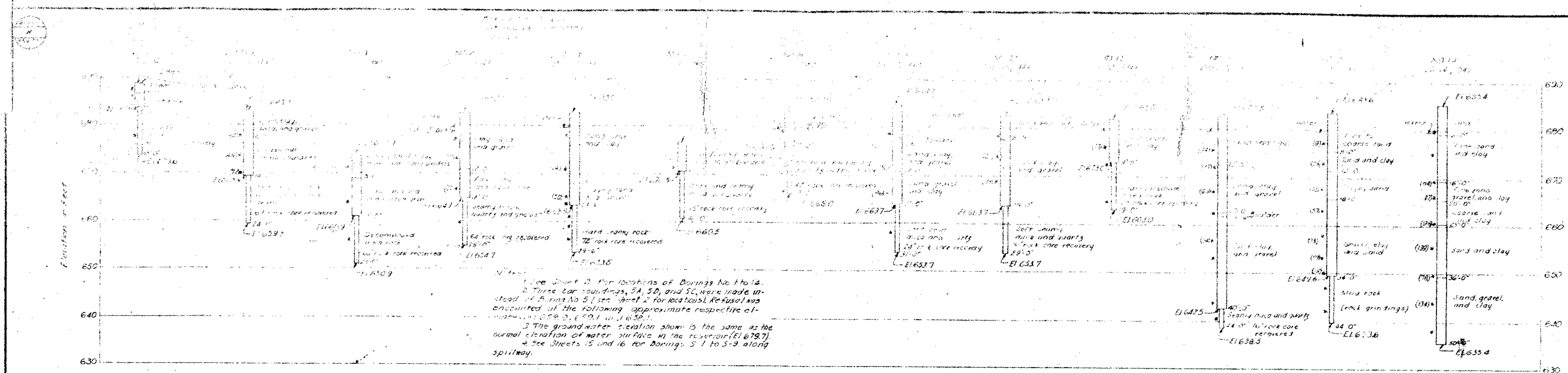
APPROVED
FOR METCALF & EDDY, ENGINEERS
CONN. REG. PROF. ENGR. NO. 32 | DATE

CITY OF BRISTOL, CONNECTICUT
BOARD OF WATER COMMISSIONERS

RESERVOIR NO. 7
PLAN, PROFILE AND SECTIONS OF DIKE

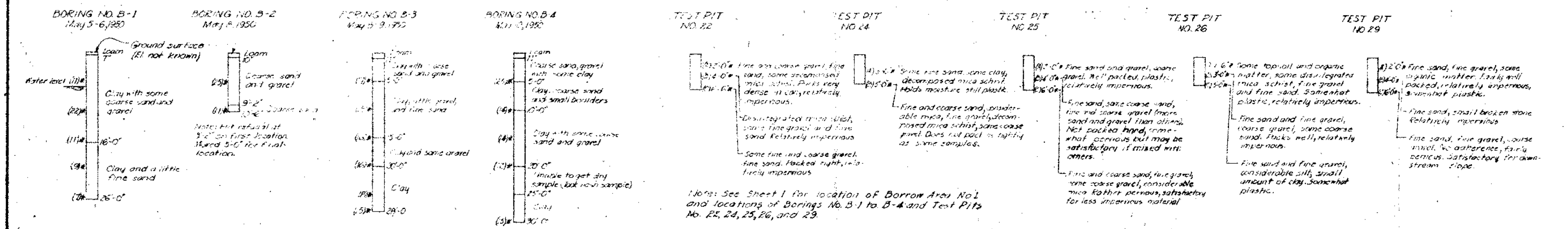
SCALE AS SHOWN — JUNE 1950

METCALF & EDDY
ENGINEERS
BOSTON, MASS.



Notes:
1. See Sheet 3 for locations of Borings No 1 to 14.
2. Three bar soundings, 5A, 5B, and 5C, were made instead of Boring No 5 (see Sheet 2 for locations). Refusal was encountered at the following approximate respective elevations: 659.3, 659.1 and 658.1.
3. The groundwater elevation shown is the same as the normal elevation of water surface in the reservoir (El 679.7).
4. See Sheets 15 and 16 for Borings S-1 to S-9 along spillway.

BORINGS AT DAM, ALONG MARSH ROAD



Note: See Sheet 1 for location of Borrow Area No 1 and locations of Borings No. B-1 to B-4 and Test Pits No. 22, 24, 25, 26, and 29.

BORINGS AND TEST PITS AT BORROW AREA NO. 1

GENERAL BORING AND TEST PIT NOTES

Applying to all borings, bar soundings, and test pits.

Borings in earth were made by the wash-boring method. Borings in rock were made by core drill. Test pits consisted of unsheeted excavations made by hand.

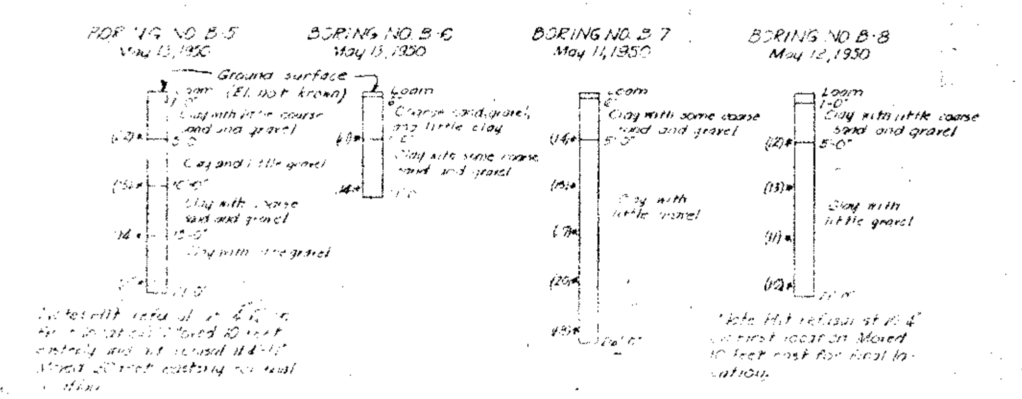
An asterisk (*) indicates a point at which a sample was taken. The samples in wash borings were taken by driving an open-end 1" diameter pipe, or 1½" spoon (for Borings No. 1 to 14), into undisturbed material ahead of the washing.

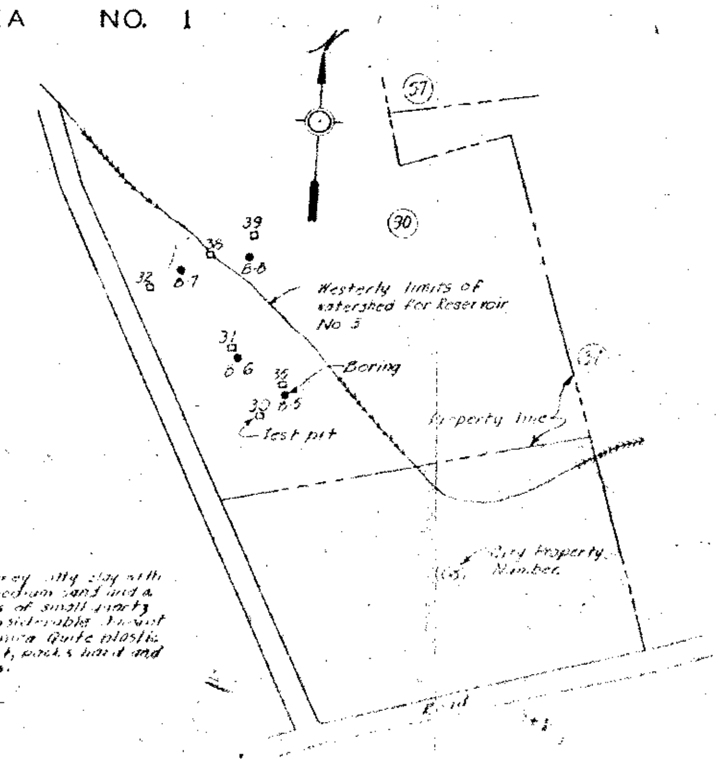
The figure adjacent to the asterisk, in parentheses, indicates the penetration, this being the number of blows required to drive an open-end 1" diameter pipe, or 1½" spoon, into the material a distance of 1'-0" at the point where the penetration was measured, using a 300 lb. or 140 lb. weight falling a distance of approximately 16" or 18" (the latter in each case being for Borings No. 1 to 14), ahead of the washing.

The other figure or letter adjacent to the asterisk, at certain locations, is the sample number or designation.

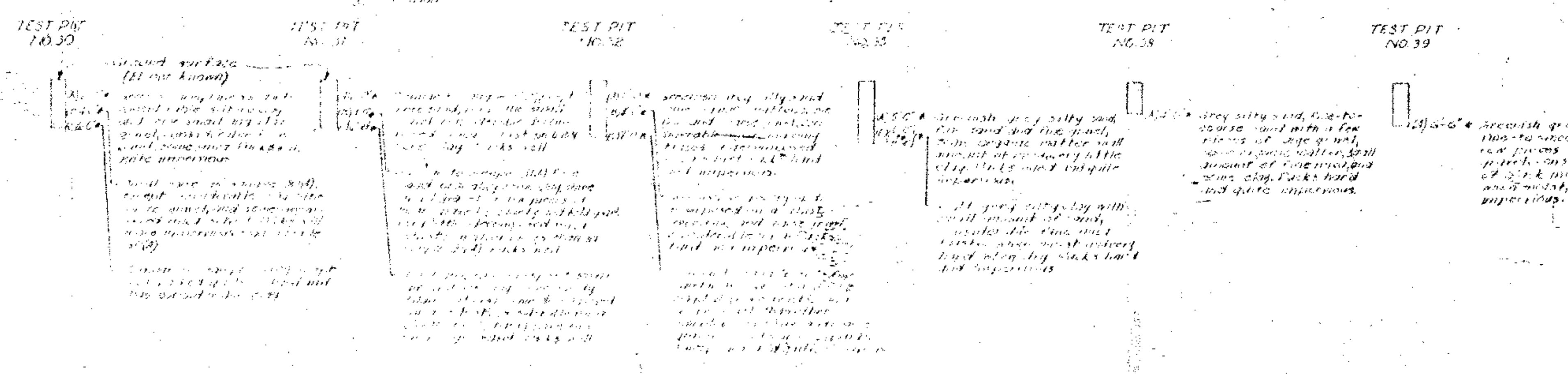
Original boring logs, made by boring contractors, giving additional comments in some cases, together with samples from borings and test pits, are on file at the office of the Board of Water Commissioners where they may be examined by prospective bidders.

No guarantee is given as to the character of the material encountered in any boring or test pit, nor is any guarantee given that borings, bar soundings, or test pits represent the true character of the material which will be excavated or which will underlie the proposed work or that ground water conditions will be as indicated.





Note: See other borings, bar soundings, and test pits shown and listed on Sheets 15, 16, and 18.



BORINGS AND TEST PITS AT BORROW AREA NO. 2

DRAWN BY
TRACED BY
CHECKED BY

B-9

APPROVED
FOR METCALF & EDDY ENGINEERS
CONN. REG. PROF. ENG. NO. 59
DATE

CITY OF BRISTOL, CONNECTICUT
BOARD OF WATER COMMISSIONERS

RESERVOIR NO. 7
BORINGS AND TEST PITS AT DAM AND BORROW AREAS

SCALE: VERT. 1"=10'
JUNE 1950

METCALF & EDDY
ENGINEERS
BOSTON, MASS.

CONTRACT T30 SHEET 19 OF 19

October 19, 1943
William S. Wise

Memorandum to General Wadhams:

Re: Old Marsh Pond
Plymouth-Bristol
Owned by Andrew Terry Co.
Terryville, Conn.

On October 18, I discussed with Mr. George Clark of the Andrew Terry Co. the proposal for using this pond for additional water supply for the City of Bristol. We looked over Old Marsh Pond and also the proposed site for a new reservoir for Bristol.

Mr. Clark's story regarding this proposal is as follows:

A few weeks ago Mr. Wooding of the Bristol Water Co. discussed with Mr. Clark the building of a new reservoir which is very badly needed to supplement their water supply. The site has been selected, (I do not know by whom), and the estimated cost of building the dam was $275,000 pre-war figures. Mr. Clark did not know whether this included clearing of the reservoir site. Mr. Clark was very much disturbed by the suggestion of spending $275,000 and more for the construction of a reservoir on a small brook which is dry most of the summer; it certainly was during our visit. Mr. Clark therefore suggested the purchase of Old Marsh Pond from the Andrew Terry Co. because of its excellent location and good condition and because it is no longer used by this company for power. Mr. Wooding indicated that this pond had never been considered for this purpose.

Mr. Clark has a plan of this pond showing all the properties around it. The company owns all of the property except two or three small pieces. They apparently do not own all of the watershed above it but this is all undeveloped land.

At present the gate to the pond is partly open and the City of Bristol has built a small temporary dam about a mile downstream from which the water is pumped into one of the reservoirs. So, the city at present is using Old Marsh Pond water by consent of the Andrew Terry Co.

Old Marsh Pond water surface covers an area of about 160 acres and has a storage capacity of about 166,000,000 gals. The dam and spillway was raised four feet in 1912. At that time, the core wall was designed so that it could be raised another four feet when desired. This increase would probably add another 220,000,000 gallons storage. The raising of the dam would not be very expensive.

176,418,000 gal.

This pond, according to Mr. Clark, has a drainage area of about 2½ sq. mi. but apparently is well fed by springs because the water never drops much even during dry weather. This year the water has been flowing over the spillway until the latter part of July, and now with the gate partly open, for some time,

10/21/43 Mr. Geo. Clark and Mr. Blodgett came in to discuss Marsh Pond & Mr. Clark decided he should have an appraisal made. We are to look for plans of Marsh Dam at Library.

is only about 8" below the spillway.

The pond is only about 1/2 mile from Bristol No.2 Reservoir and, according to Mr. Clark, at an elevation so that the water could be discharged into No. 2 by gravity.

The proposed new dam is located on a small stream, apparently with a small drainage area, according to appearances. It was estimated that it would have a storage capacity of 225,000,000 gals. when full.

Mr. Clark employed Mr. Blodgett for legal advice concerning the proposal to use Old Marsh Pond and also try to get some idea of its value. Mr. Blodgett suggested getting an appraiser from Philadelphia to determine its value. This of course would be more than an ordinary appraisal job because its value would be based on the weighted advantages, disadvantages and cost of developing a new supply.

It is unsafe to make any recommendations concerning this problem from such a very rough investigation. But, on the surface, it would appear that the use of Old Marsh Pond for water supply should be given some consideration if it has not yet been done. The advantages would appear to be - from this rough investigation:

(1) It could be put into use in a comparatively short time.

(2) The pond has been in use for a long time so that is has been seasoned and the water has lost the high color, usual in a new reservoir.

(3) It now has a storage capacity nearly equal to the proposed new reservoir and if the dam was raised it would have a total capacity of 385,000,000 gallons as compared to 225,000,000 gallons for a new one.

(4) It might be purchased and the dam raised for less cost than a new one.

(5) Apparently the watershed would not be difficult to control from a sanitary standpoint.

At present the Andrew Terry Co. permits bathing in the pond just as a courtesy to local people. Two of the small property owners rent boats for fishing and recreation. The Fish and Game Commission stocks the pond but the lease expires with termination of present ownership.

The only conclusion that can be drawn is that if this pond has never been considered as a source of additional water supply, it has merits worthy of consideration. There may be disadvantages that have not appeared in the discussion.

Respectfully submitted,

b

William S. Wise, Chief Engineer

EDGAR L. POND
PRESIDENT

GEORGE C. CLARK
VICE PRESIDENT & TREAS.

DWIGHT W. POND
SECRETARY

The Andrew Terry Co.

MANUFACTURERS OF

ELECTRIC CONDUIT FITTINGS

Terryville, Conn.

December 11, 1943

General Sanford H. Wadhams
Room 317, State Office Building
Hartford, Conn.

Dear General Wadhams:

I was much interested in your letter of the 10th and wish to thank you for it and also thank you for your efforts in having this matter placed on the agenda for the special session of the General Assembly.

We have handed a plan of a dam to Bristol which we think is the correct plan of the dam at the Old Marsh. At any rate, they can ascertain the correctness of same by getting in touch with Nicholas Fogg of Niantic, formerly employed by Sperry and Buel, and who was the engineer in charge of the building of the dam.

Yours very truly,

ANDREW TERRY CO.

George C. Clark

George C. Clark
President

GCC:mfb

12/13/43

Note: If Bristol purchases Old Marsh Pond it seems probable that the Board of Supervision of Dams will ask for approval to raise the dam by perhaps four feet. In that case the plans showing details of construction will be very helpful.

[illegible]

EDGAR L. POND
PRESIDENT

GEORGE C. CLARK
VICE PRESIDENT & TREAS.

DWIGHT W. POND
SECRETARY

The Andrew Terry Co.

MANUFACTURERS OF

ELECTRIC CONDUIT FITTINGS

Terryville, Conn.

RECEIVED
DEC 20 1943
STATE WATER COMMISSION

December 18, 1943

General Sanford H. Wadhams
Room 317, State Office Building
Hartford, Conn.

Dear General Wadhams:

The Superintendent of the Bristol Water Company dropped into my office yesterday and said that they had found a detailed plan of the dam at the Old Marsh in the effects of Mr. Buell.

Metcalf and Eddy had looked the thing over and the only fault they find is a small growth of underbrush on the lower side of the dam.

The only thing holding up the deal now is an infected toe of Bristol's Corporation counsel. We believe that a certain degree of professional efficiency will obliterate all these defects soon.

Yours very truly,

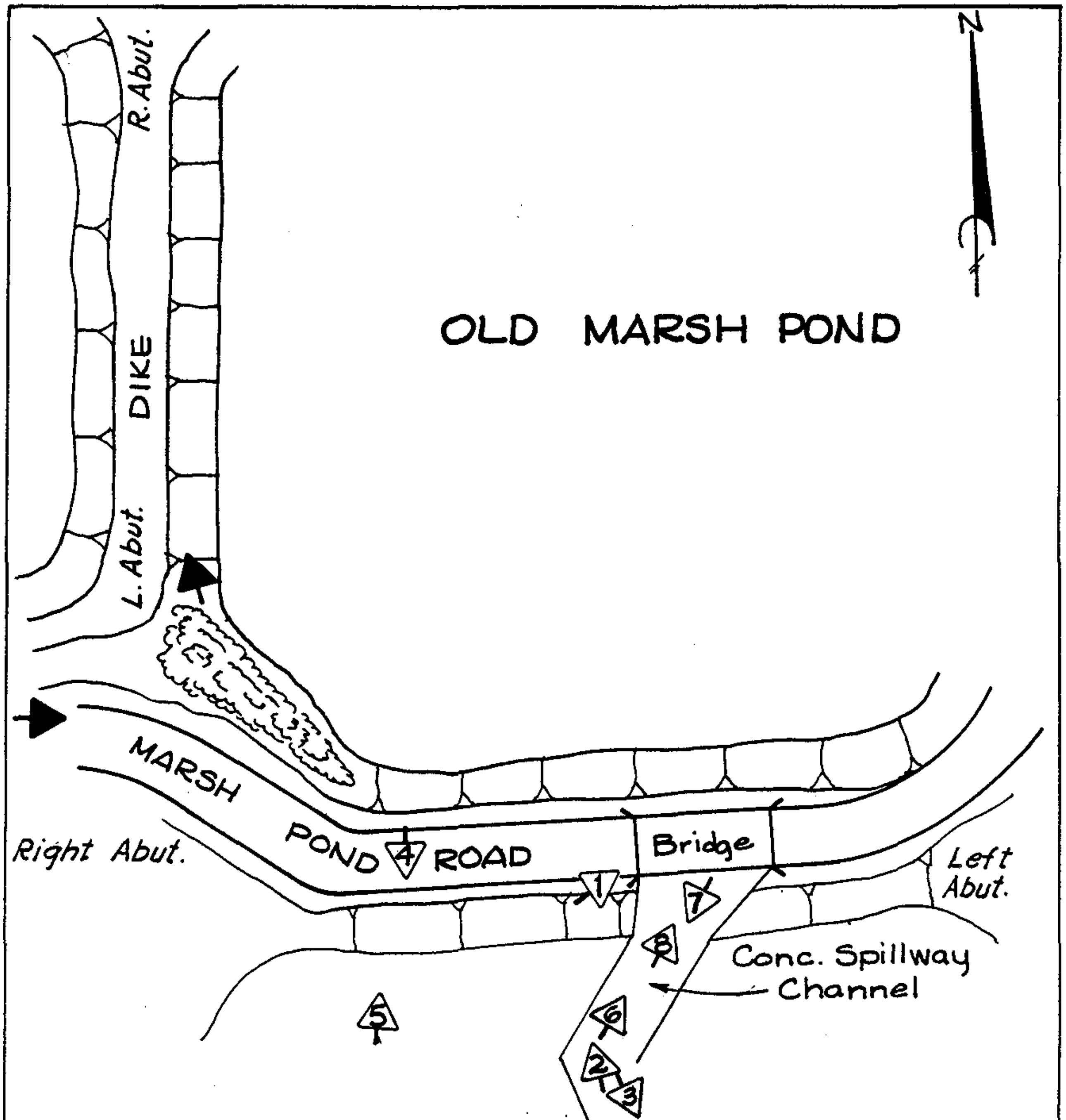
ANDREW TERRY CO.

George C. Clark
President

GCC:mfb

APPENDIX C

SELECTED PHOTOGRAPHS



Appendix "C" Photos

Overview Photos

LOUIS BERGER & ASSOC., INC WELLESLEY, MASS. ARCHITECT · ENGINEER	U.S. ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS.

NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS

OLD MARSH POND DAM

SKETCH PLAN SHOWING LOCATION & ORIENTATION OF PHOTOS

STATE - CT.

			SCALE 1:24000
			DATE

OLD MARSH POND DAM

1. Upstream slope left of spillway bridge.

2. Seepage over wall below stilling basin.

OLD MARSH POND DAM

4. Valve chamber at downstream toe and seepage in old stream bed.

3. Downstream channel from spillway stilling basin.

OLD MARSH POND DAM

5. Animal burrow on downstream face.

6. Spillway channel and stilling basin.

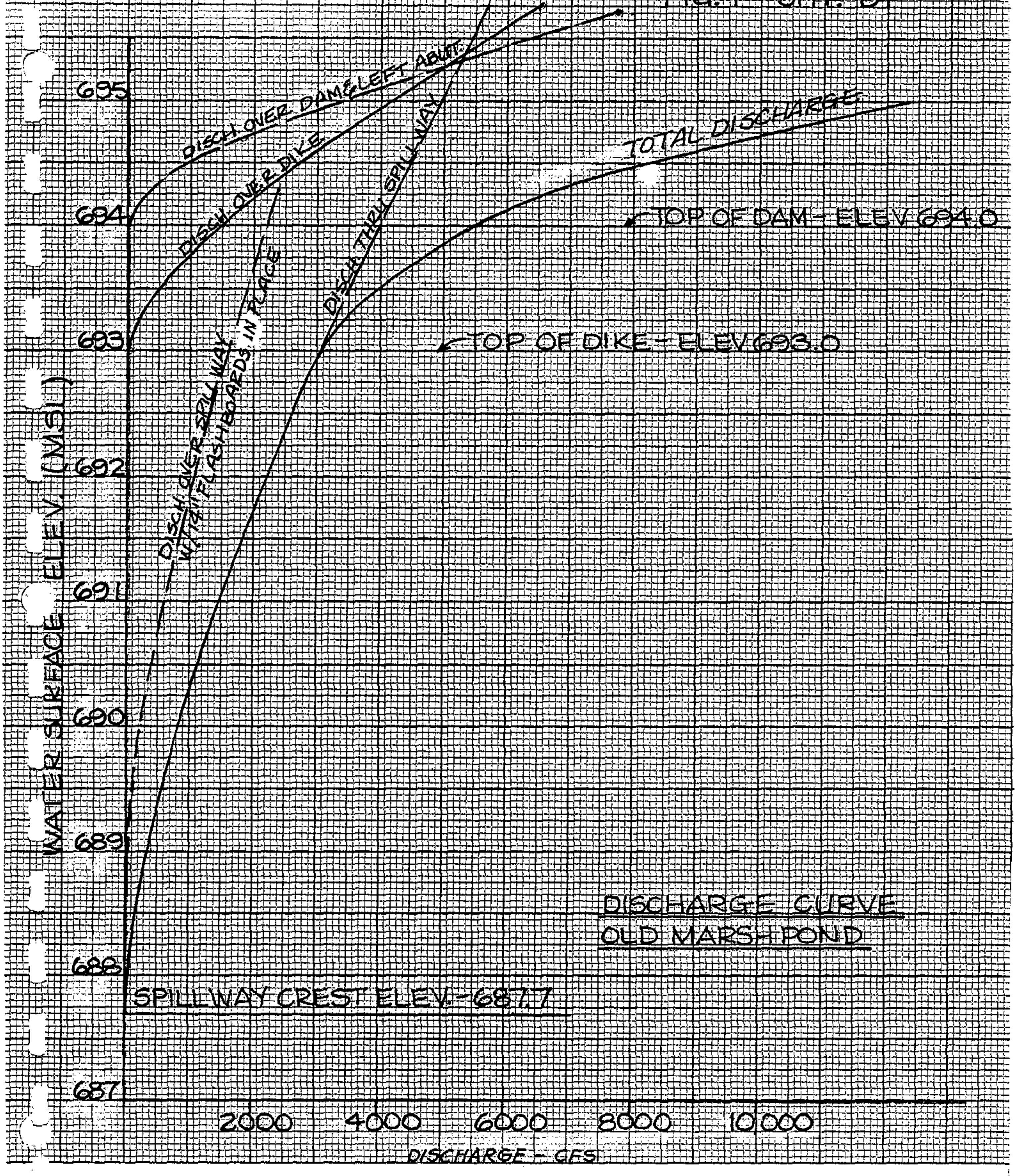
OLD MARSH POND DAM

7. Spillway stilling basin.

8. Spillway channel with retarding sill.

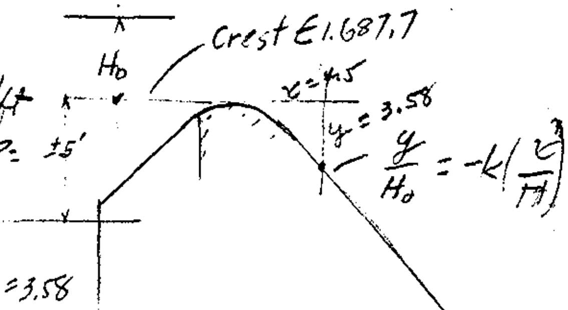
APPENDIX D

HYDROLOGIC & HYDRAULIC COMPUTATIONS


DISCH. OVER DAM & LEFT ABUT.
DISCH. OVER DIKE
DISCH. THRU SPILLWAY
TOTAL DISCHARGE
TOP OF DAM - ELEV 694.0
DISCH. OVER SPILLWAY W/ 1/4' FLASHBOARDS IN PLACE
TOP OF DIKE - ELEV 693.0
WATER SURFACE ELEV. (M.S.L.)
695
694
693
692
691
690
689
688
687
DISCHARGE CURVE
OLD MARSH POND
SPILLWAY CREST ELEV. - 687.7
2000
4000
6000
8000
10,000
DISCHARGE - CFS

BY PEC DATE 12/14/78 **LOUIS BERGER & ASSOCIATES INC.** SHEET NO. D-2 OF ___
CHKD. BY ___ DATE ___ DAM INSPECTION PROJECT W189
SUBJECT OLD MARSH POND #15

Assume $H_o = \pm 3'$ $P + H_o = \pm 8'$
Assume $C = \pm 4$ $q = CH^{3/2} = \pm 20$ cfs/ft
Approach $v = \frac{20}{8} = 2.5$ $h_a = 0.10$
$\frac{h_a}{H_o} = \frac{0.1}{3} = 0.033$



From Fig 247 DSM. $n = 1.765$ $k = .54$ $x = 4.5$ $y = 3.58$

$$\frac{3.58}{H_o} = .54\left(\frac{4.5}{H_o}\right)^{1.765} \qquad 6.63 = H_o\left(\frac{4.5}{H_o}\right)^{1.765} \qquad 6.63 = \frac{(4.5)^{1.765}}{H_o^{.765}}$$

$$H_o^{.765} = 2.145$$
$$H_o = 2.71$$

∴ Design head = 2.71 Say 2.70' @ El 690.4

Fig 249 Des Small Dams
$\frac{P}{H_o} = \frac{5}{2.7} = 1.85$ Coeff for design head = 3.93

Fig 251 For 3:3 slope reduction .995 x Vertical = 3.93 x .995 = 3.90

$Q = CLH^{3/2}$

← Spillway L = 60' → ① —Main Dam L = 875

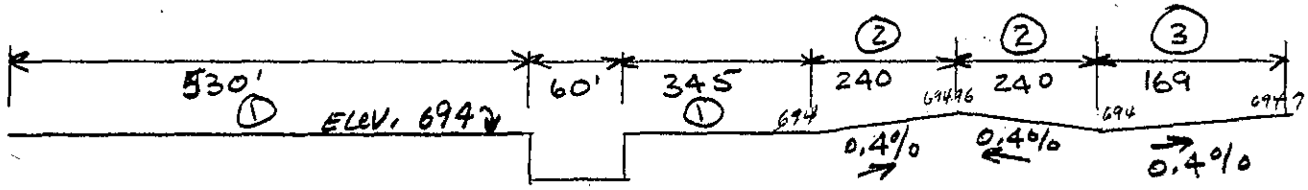
Elev.	H	$\frac{H_e}{H_o}$	$\frac{C}{C_o}$	C	ΔQ	H	C	ΔQ	Left Abut ΔQ	Dike	Total
687.7	0	–									0
688.2	0.5	.20	.85	3.34	71						71
688.7	1.0	.4	.9	3.54	212						212
689.2	1.5	.6	.94	3.70	408						408
689.7	2.0	.8	.97	3.81	647						647
690.2	2.5	1.0	1.00	3.93	932						932
690.7	3.0	1.2	1.03	4.05	1263						1263
691.2	3.5	1.4	1.05	4.13	1623						1623
691.7	4.0	1.6	1.07	4.15	1992						1992
692.2	4.5			4.15	2377						2377
692.7	5.0			4.15	2784						2784
693.7	6.0			4.15	3660					863	4523
694	6.3			4.15	3937	0		0	0	1473	5410
694.7	7.0				4612	0.7	2.8	1435	426	3265	9312
695.7	8.0				5634	1.7	2.8	5431	2307	6534	17599

BY PEC DATE 1/8/79 **LOUIS BERGER & ASSOCIATES INC.** SHEET NO. D-3 OF

CHKD. BY ______ DATE ______ OLD MARSH POND PROJECT ______

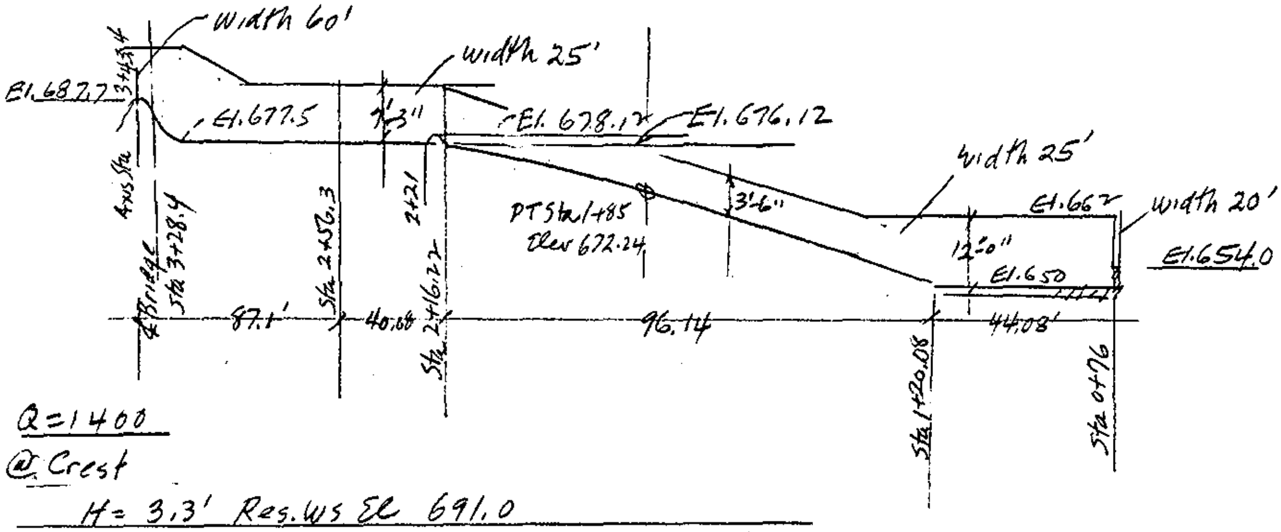
SUBJECT ______

DISCHARGE - Cont.



ELEV	H	C	Q/FT	② - twice (spillway .96 / 240) L	AV. Q/FT	ΔQ	③ - .7 / 169 L	AV. Q/FT	ΔQ	Dike L=526 H	C	ΔQ
687.7												
688.2												
688.7												
689.2												
689.7												
690.2												
690.7												
691.2												
691.7												
692.2												
692.7										~~693~~ 0		
693.7										0.7	2.8	863
694.0	0	2.8	0				0			1.0	2.8	1473
694.7	0.7	2.8	1.64	175	0.82	287	169	0.82	139	1.7	2.8	3265
695.7	1.7	2.8	6.21	240	3.92	1643	169	3.93	664	2.7	2.8	6534

SUBJECT OLD MARSH POND DAM - SPILLWAY HYDRAULICS



Q = 1400

@ Crest

H = 3.3' Res. WS El 691.0

@ Sta 2+22 Bottom El 678.12 $d + 1.2h_v = 691 - 678.12 = 12.88$ $w = 25'$

$d = 2.36$ $A = 59.0$ $v = 23.73$ $h_v = 8.74$ $1.2h_v = 10.49$ $d + 1.2h_v = 12.85$ ok

Wall ht. 5'-3" Freeboard = 2.89'

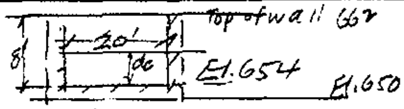
@ Sta 1+85 Bottom El 672.24 $d + 1.2h_v = 691 - 672.24 = 18.76$ $w = 25$

$d = 1.86$ $A = 46.5$ $v = 30.11$ $h_v = 14.07$ $1.2h_v = 16.89$ $d + 1.2h_v = 18.75$ OK

Wall ht 3'-6" Freeboard 1.64'

@ Stilling basin

Downstream end sill



Critical flow over end sill Gradient El. 662

$d_c = 2h_{vc}$ $d_c + h_{vc} = 8'$ $h_{vc} = 2.67$ $v_c = 13.10$ $A = 5.33 \times 20 = 106.7$

$Q_c = 1397$ cfs

Depth in basin upstream $d + h_v = 12'$

$d = 11.65'$ $A = 291.21$ $v = 4.81$ $h_v = 0.35'$ ok

Conjugate depth of Jump. $H_T = 691 - 662 = 29'$

$q = \frac{1400}{25} = 56$ cfs/ft. $d_2 = \pm 12'$ TW El 662.

Q = 4000

@ Crest

H = 6.3 Res WS El 694.0

@ Sta 2+22 Bottom El. 678.12 $d + 1.2h_v = 694 - 678.12 = 15.88$ $w = 25'$

$d = 7.51$ $A = 187.75$ $v = 21.30$ $h_v = 7.05$ $1.2h_v = 8.46$ $d + 1.2h_v = 15.96$ ok

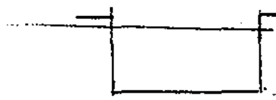
Wall ht = 5'-3" Overtopping 2.25'

@ Sta 1+85 Bottom El. 672.24 $d + 1.2h_v = 694 - 672.24 = 21.76$ $w = 25'$

$d = 5.40$ $A = 135$ $v = 29.63$ $h_v = 13.63$ $1.2h_v = 16.36$ $d + 1.2h_v = 21.76$ OK.

Wall ht = 3'-6" Overtopping 1.90'

Q = 4000 cfs
@ Stilling basin
Downstream end sill



$Q_c = 4000$ $\quad Q_c = 3.087\, b\, H_{E_c}^{3/2}$ $\quad H_{E_c} = \left(\frac{4000}{20 \times 3.087}\right)^{2/3} = 16.1'$

In basin $d + h_v = 16.1$ $\quad d = 14.0$ $\quad A = 350$ $\quad v = 11.43$ $\quad h_v\ 2.03$ $\quad d + h_v = 16.03$ OK

Conjugate depth of jump $\quad H_T = 694 - 664 = 30'$

$q = \frac{4000}{25} = 160$ cfs/ft $\quad d_2 = 21'$

$\therefore$ T.W. depth $= \frac{14}{21} = 0.67$ % of conj. depth.
Sweepout of basin is probable.

Q = 4700 cfs. (Max outflow for PMF)
Downstream end sill @ Critical depth. $\quad H_E = \left(\frac{4700}{20 \times 3.087}\right)^{2/3} = 18.0'$ @ El 672

In basin $d + h_v = 18'$. $d = 15.8$ $\quad A = 395$ $\quad v = 11.9$ $\quad h_v = 2.2$
$\therefore$ Basin WS El 665.8

Conjugate depth of jump $\quad H_T = 694.25 - 665.8 = 28.45$

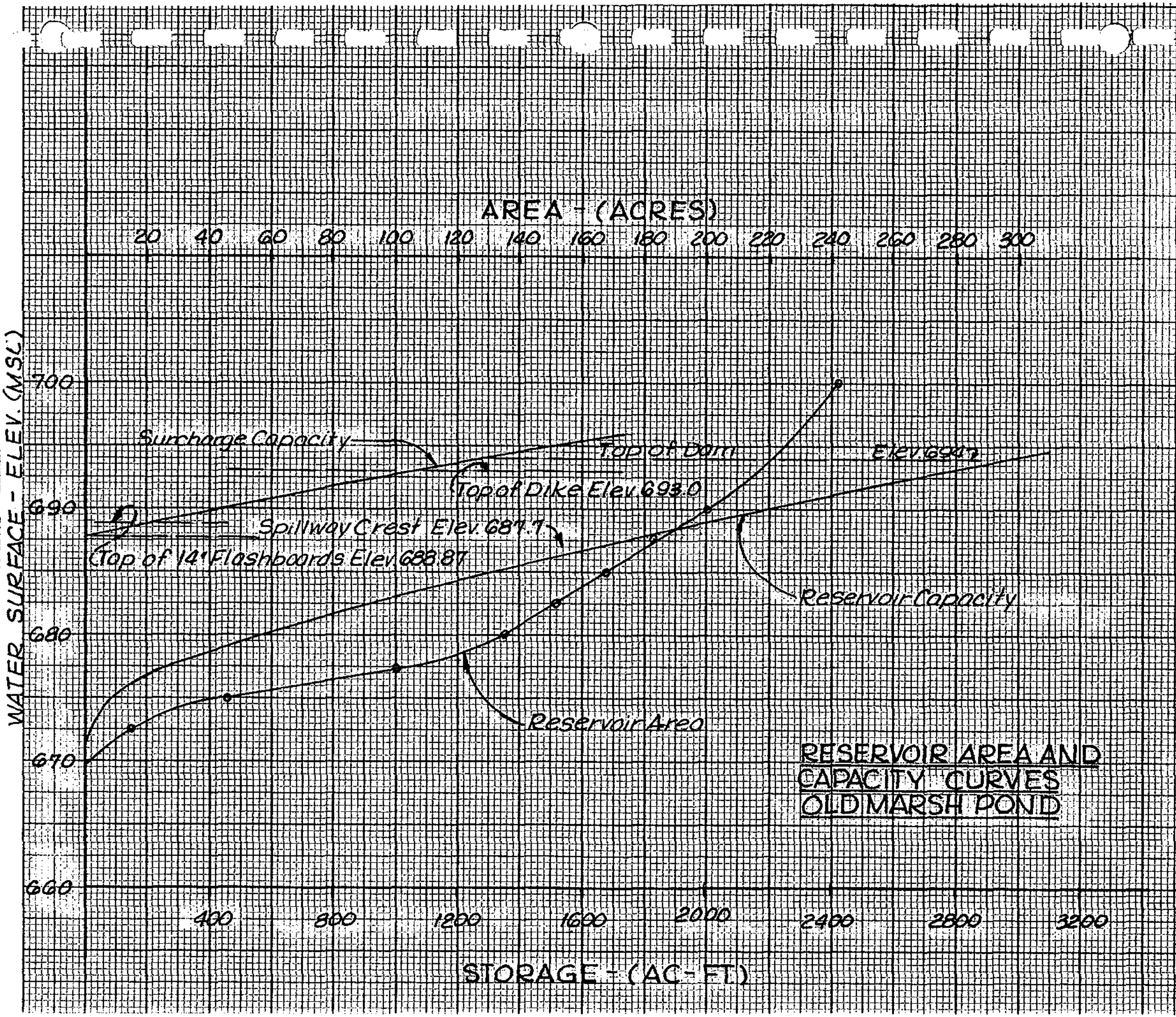
$q = \frac{4700}{25} = 188$ $\quad d_2 = 22$

$\therefore$ TW depth $= \frac{15.8}{22} = 0.72$ % of conj. depth
Sweepout of basin is probable.

SPILLWAY DISCHARGE OVER 14" FLASHBOARDS ON CREST

ELEVATION	H	C	Q cfs
687.7	0		
688.87	0		0
689.0	0.13	3.30	9
690.0	1.13	3.30	238
691.0	2.13	3.28	612
692.0	3.13	3.24	1076
693.0	4.13	3.20	1611
694.0	5.13	3.16	2203.

D-6



KEUFFEL & ESSER CO.
MADE IN U.S.A.

FIG 2 - SHT D-6

BY: ___ DATE: 11-10-78 **LOUIS BERGER & ASSOCIATES INC.** SHEET NO. D-7 OF ___

CHKD. BY ______ DATE ______ INSPECTION OF DAMS - CONN-R.I. PROJECT ______

SUBJECT: OLD MARSH POND RESERVOIR - AREA-CAPACITY and DISCH. CURVES #15

RESERVOIR AREA - CAPACITY

Res Area 183 ac @ El 687.7
201 ac @ 690
247 ac @ El 700

Elev.	Area (Acre)	Av. Area (acre)	Height (ft.)	Incremental Storage (ac-Ft)	Cumulative storage (ac-ft)	Surcharge storage (ac-Ft)	Remarks
670	10	0	0	0	0		24" Φ invert.
672.5	15	7.5	2.5	19	19		
675	46	30.5	2.5	76	95		
677.5	100	73	2.5	183	278		
680	136 (130)	118	2.5	295	573		
682.5	152	144	2.5	360	933		
685	168	160	2.5	400	1334		
687.7	183	175.5	2.7	474	1808	0	Spillway Crest
688	186	184.5	0.3	55.4	1863	55.	
689	193	190.0	1	190.0	2053	245.	
690	201	197.0	1	197.0	2250	442.	
691	206	203.5	1	203.5	2454	645.	
692	211	208.5	1	208.5	2662	854.	
693	216	213.5	1	213.5	2876	1068	Top of DIKE
694	220	218.0	1	218.0	3094	1286	TOP OF DAM
695	224	222.0	1	222	3316	1508.	
696	228	226.0	1	226	3542	1734	
700	247						

14" Flashboards El. 687.7 to 688.87

1.17 × 187.5 = 219 AF storage reduction

Spwy Q over flashboards

C = 3.33 To El. 694 H = 5.13 Q = 2321 < 3937

Drainage area 2.34 sq. mi.
Res. area = 183 acres = 12% of total D.A.
Reservoir capacity to normal storage level 1808 AF.
Spillway crest El. 687.7 L = 60'
Top of dam El. 694.
(See memo Oct 19, 1943 for data on storage capacity, area, etc.

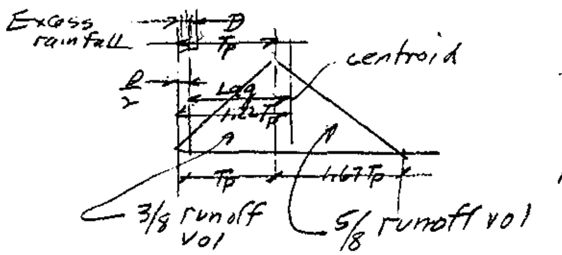
Drainage Area

Stream ③ L = 6000' = 1.14 mi
Max elev. = 988 Res El. 688 H = 300' } Tributary ①
$S = \frac{300}{1.14} = 263$ ft/mi.

Stream 5 L = 6000 = 1.14 El. 935 − 680 = $\frac{255}{1.14}$ = 224'/mi } Tributary ②

Average s = 244'/mi K = 4.25 (Curve B K = 2.5 to 5, Assume K = 4.25)
$Lag = K\left(\frac{L\,L_c}{\sqrt{S}}\right)^{.33} = 4.25\left(\frac{1.14 \times \frac{1.14}{2}}{\sqrt{244}}\right)^{.33} = 1.48$ Say 1.50

Check for velocity.
$v = \frac{6000}{3600 \times 1.50} = 1.11$ '/sec. ≅ 1.0 OK



$\frac{D}{2} + Lag = 1.22\,T_p$
$T_p = \frac{Lag}{1.22} + \frac{D}{2 \times 1.22} = .82\,Lag + .42D$
D = 0.5
$T_p = .82 \times 1.5 + .42 \times 0.5 = 1.44$
say $T_p = 1.5$ hrs.

If: $T_p = 1.5$ Then; $T_p(1.67) = 1.5(1.67) = 2.51$

Tributary streams into reservoir

Tributary	Length	Drop	Slope/mi	SL
①	3600	182	267	961,200
②	4000	122	161	644,000
③	6000	300	263	1,578,000
④	4200	132	166	697,200
⑤	6000	252	224	1344 000
Average	4760		203	1 048 920 ÷ 4760 = 220'/mi.

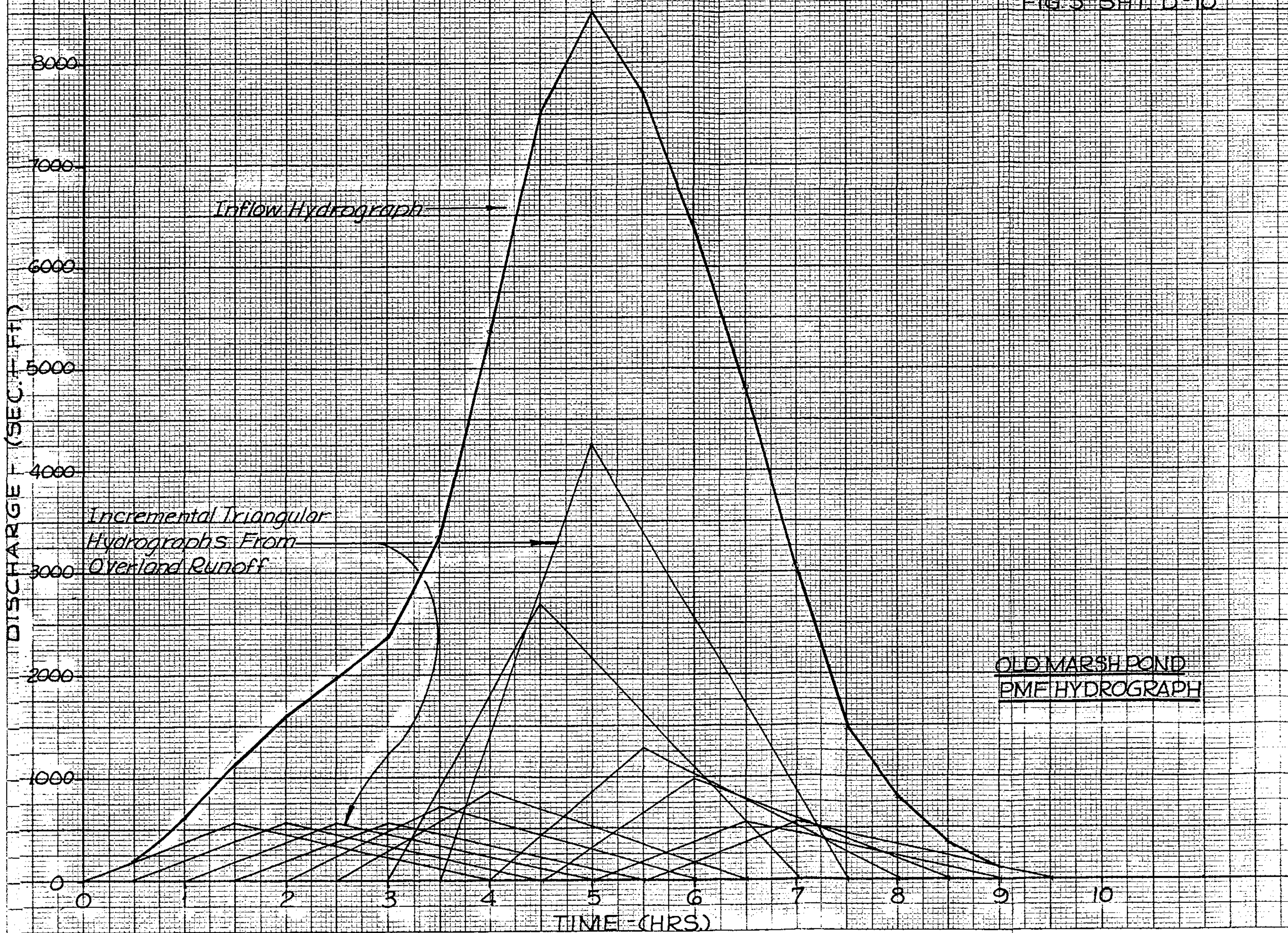
Calculate Q_p:

$$Q_p = \frac{484\,AQ}{T_p} = \frac{484\,(2.34)(1)}{1.5} = 755 \text{ cfs}$$

Rainfall

PMF = 24" x 0.8 (Fit Factor) = 19.2" for 6 hrs.
Adjust for Filtration Loss ∴ 19.2 − 0.4 = 18.8"

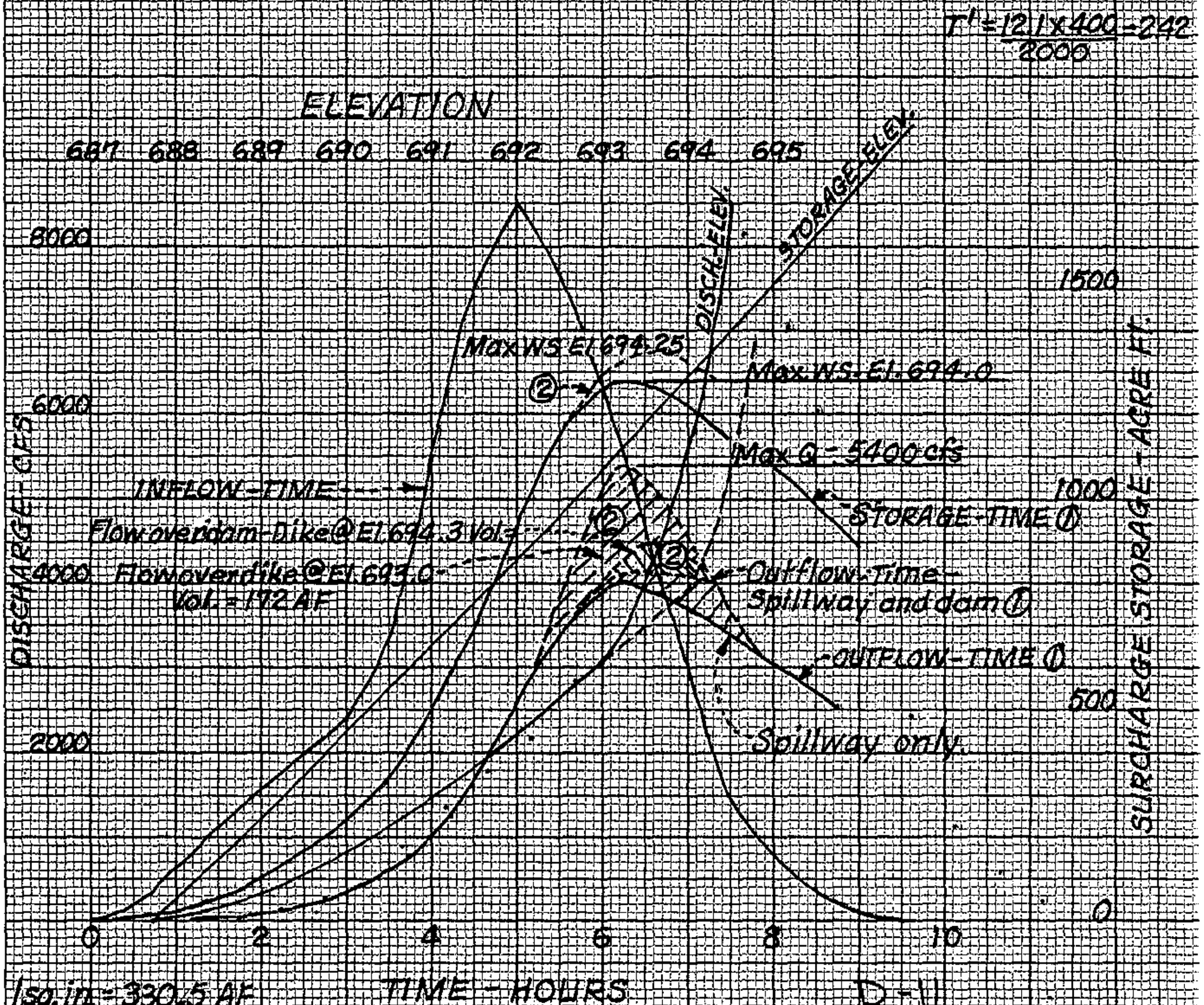
Time	Rain Dist (inches)	Q_p	Begin T	T of Peak	T of End
0					
0.5	0.75	566	0	1.5	4.01
1.0	0.75	566	0.5	2.0	4.51
1.5	0.75	566	1.0	2.5	5.01
2.0	0.75	566	1.5	3.0	5.51
2.5	0.94	710	2.0	3.5	6.01
3.0	1.13	853	2.5	4.0	6.51
3.5	3.57	2695	3.0	4.5	7.01
4.0	5.64	4258	3.5	5.0	7.51
4.5	1.69	1276	4.0	5.5	8.01
5.0	1.32	997	4.5	6.0	8.51
5.5	0.75	566	5.0	6.5	9.01
6.0	0.75	566	5.5	7.0	9.51


Inflow Hydrograph
Incremental Triangular
Hydrographs From
Overland Runoff
DISCHARGE (SEC. FT.)
TIME (HRS.)
OLD MARSH POND
PMF HYDROGRAPH

OLD MARSH POND RESERVOIR

FLOOD ROUTINGS OF PMF THRU RESERVOIR AND SPILLWAY

case ① Dike constructed to El. 693.0
case ② Dike raised to Elev. 694.3



OLD MARSH POND RESERVOIR
FLOOD ROUTING OF 0.75 PMF THRU RESERVOIR AND SPILLWAY

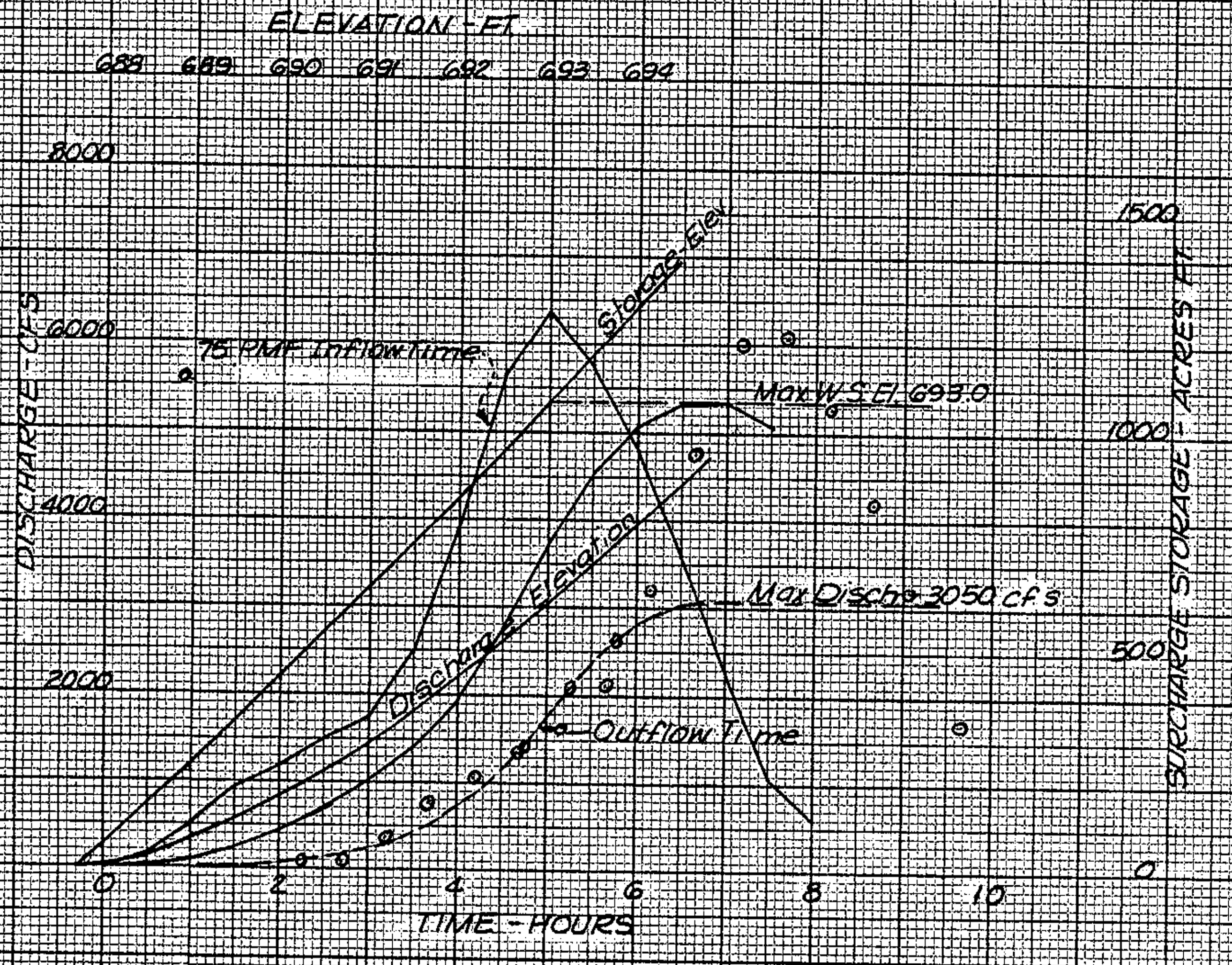
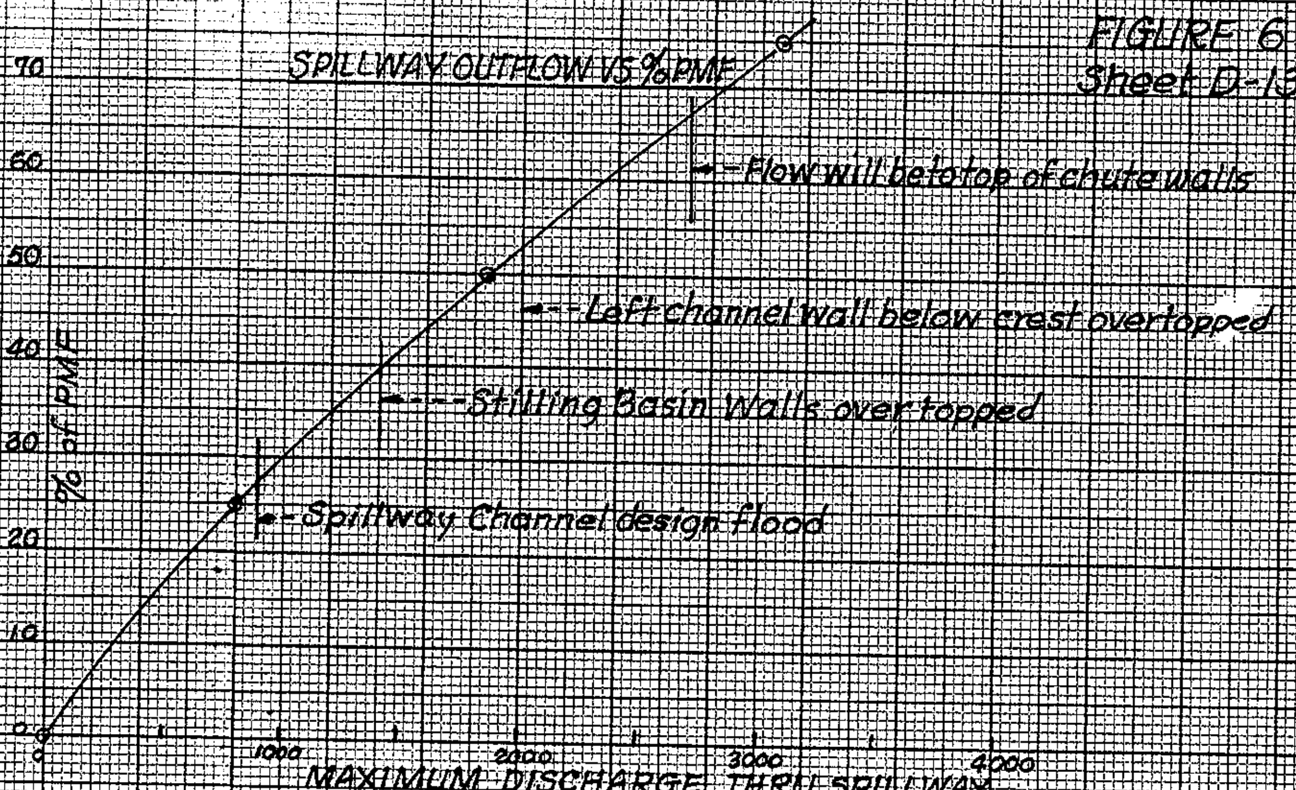
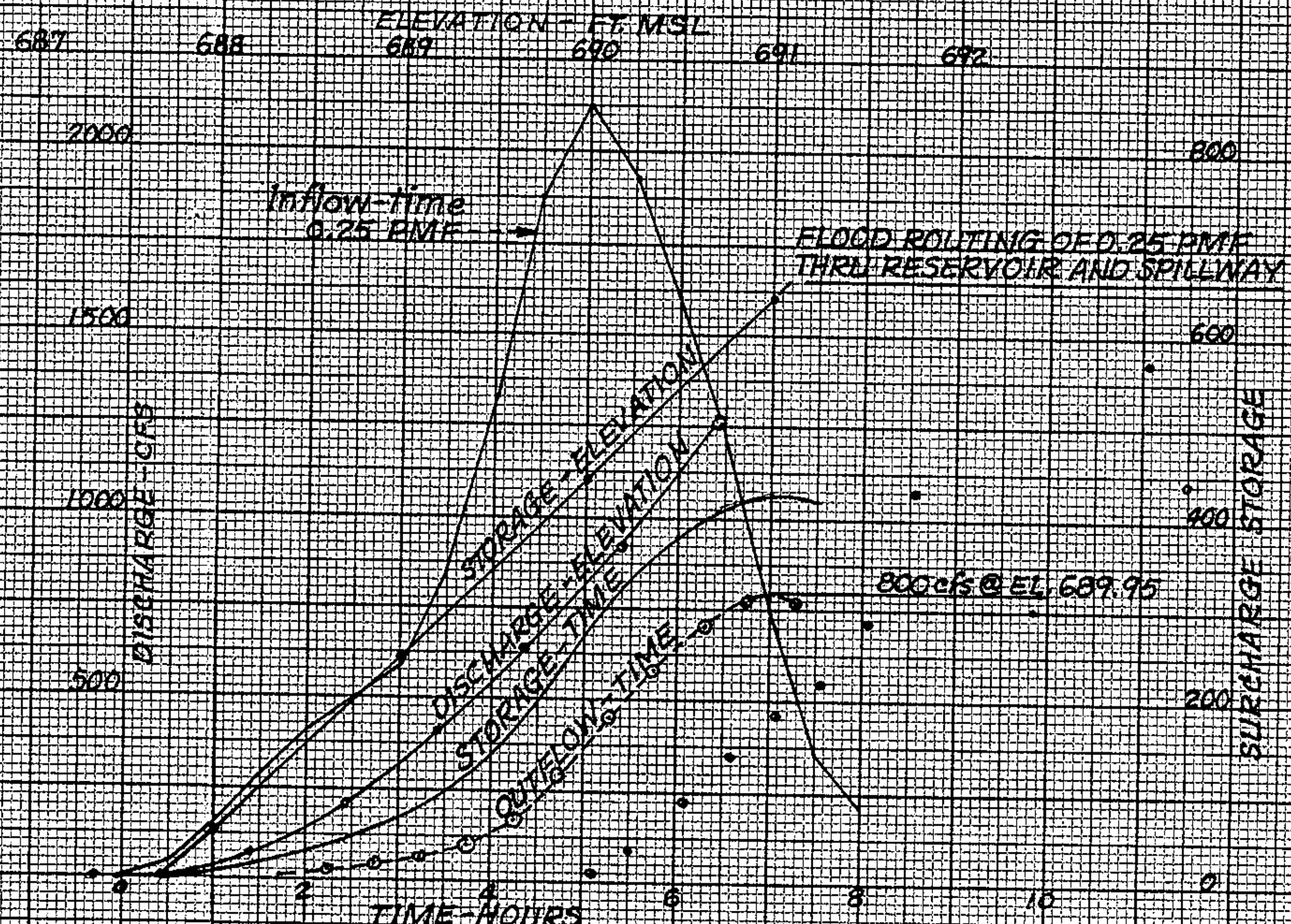


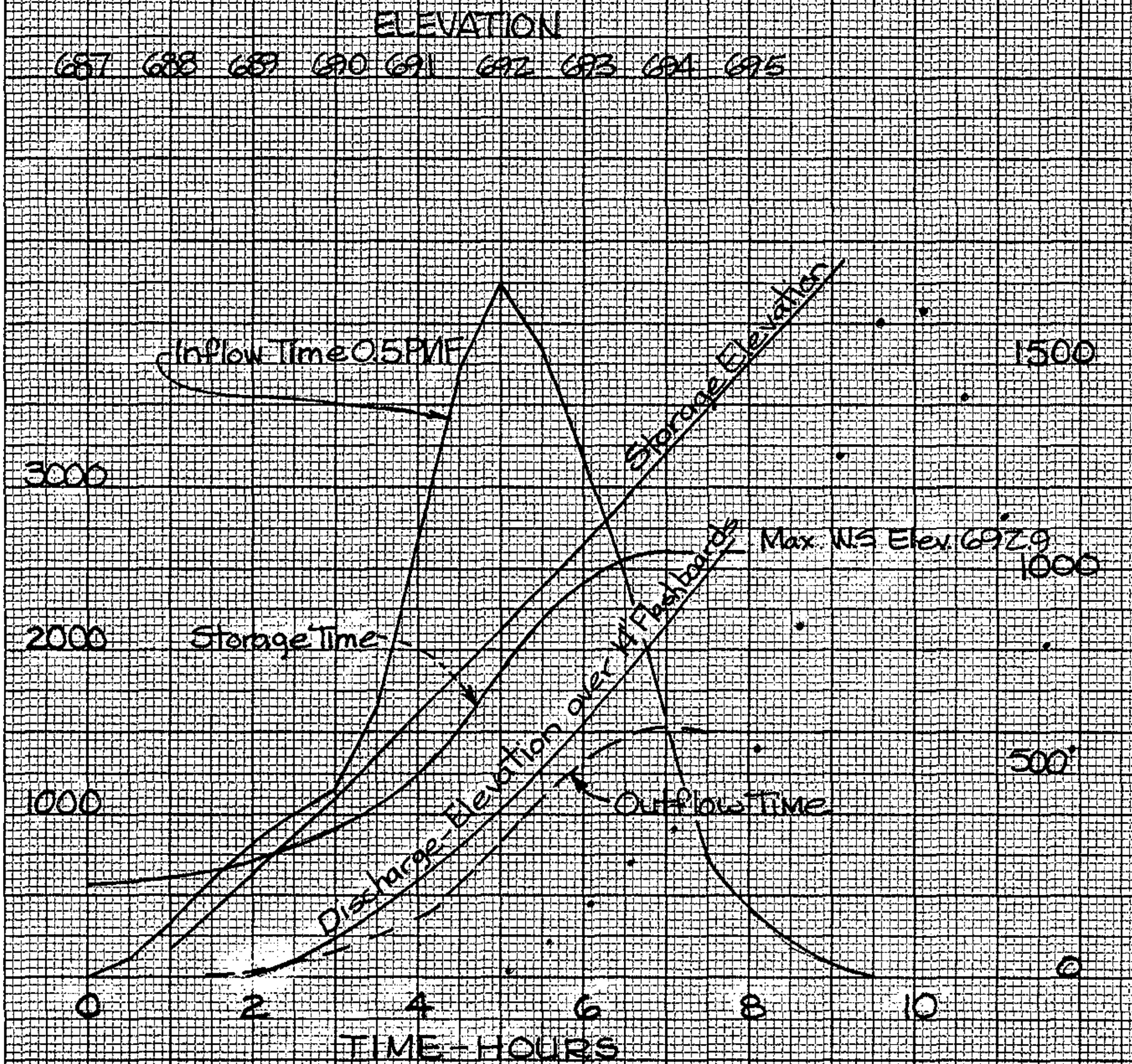
FIGURE 6
Sheet D-13





OLD MARSH POND DAM

OLD MARSH POND RESERVOIR
FLOOD ROUTING OF 0.5 PMF THRU RESERVOIR AND SPILLWAY WITH 14" FLASHBOARDS INSTALLED AND RESERVOIR TO TOP OF FLASHBOARDS AT START OF FLOOD EVENT.

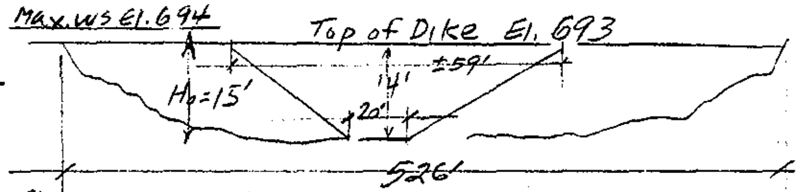


BY CGH DATE 1-24-79 **LOUIS BERGER & ASSOCIATES INC.** SHEET NO. D-15 OF

CHKD. BY ______ DATE ______ INSPECTION OF DAMS - CONN. & R.I. PROJECT ______

SUBJECT OLD MARSH POND DAM - HYDRAULICS OF BREACH FAILURE

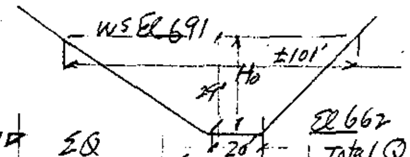
BREACH FAILURE AT DIKE



$Q = \frac{8}{27} \times W \sqrt{g}\, H_o^{3/2} = 1.68 W H_o^{3/2}$

Elev	H	q cfs/ft	ΔQ - U. 20	L 1.4 ◁	ΔQ ◁▷	ΣQ	Spillway Q	Total outflow	Average outflow cfs	Δ Storage AF	Evacuation time - Hrs.
694	15	97.6	1952	25.2	2459	4112	3500	7612			
691	12	69.8	1396	16.8	1173	2568	1500	4068	5840	640	1.33
688	9	45.4	908	12.6	572	1480	50	1530	2800	591	2.56
685	6	24.7	494	8.4	207	702	0	702	1116	529	5.75
682	3	8.7	174	4.2	37	210	0	210	456	473	12.57
679	0	0	0	0	—	0	0	0	105	406	46.87
										2639 AF	169 hrs 7 days.

TRAPEZOIDAL BREACH FAILURE AT DAM



ELEV	H	q/ft cfs	ΔQ H 20	◁ L ▷	ΔQ ◁▷	ΣQ	Spwy & 4000	El 662 Total Q	Ave outflow	Δ Stor.	Evac. time hrs.
~~694~~	~~32~~	~~304.1~~	~~6082~~	~~44.8~~	~~13503~~	~~19585~~		~~23584~~			
691	29	262.4	5248	40.6	10633	15900	1500	17,400	20492	640	0.38
688	26	222.7	4454	36.4	8106	12560	50	12,610	15005	591	0.48
685	23	185.3	3706	32.2	5967	9673	0	9673	11142	529	0.58
682	20	150.3	3006	28.0	4208	7214	0	7214	8413	473	0.68
679	17	117.8	2356	23.8	2804	5160	0	5160	6187	406	0.98
676	14	88.0	1760	19.6	1725	3485	0	3485	4322	287	0.80
673	11	61.3	1226	15.4	944	2170	0	2170	2827	134	0.57
670	8	38.0	760	11.2	426	1186	0	1186	1678	19	0.14
667	5	18.8	376	7	132	508	0	508	847	0	0
662	0	0	0	0		0	0	0	254	0	0
										2439 AF	4.23 hrs

50' RECTANGULAR BREACH FAILURE AT DAM (50' of core wall collapses)

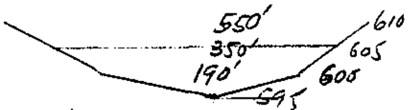
ELEV.	H	q/ft	W=50 Q	Spillway Q	Disch. over dike	Total Outflow
694	32	304.1	15200	4000	1500	20700
690	28	248.9	12450	850	0	13300
685	23	185.3	9260	0	0	9260
680	18	128.3	6420	0	0	6420
675	13	78.7	3940	0	0	3940
670	8	38.0	1900	0	0	1900

STAGE-DISCHARGE - MARSH BROOK IN EAST PLYMOUTH ±4500' Below Dam.

$S = \frac{10}{2500} = .004$ $S^{1/2} = .0632$

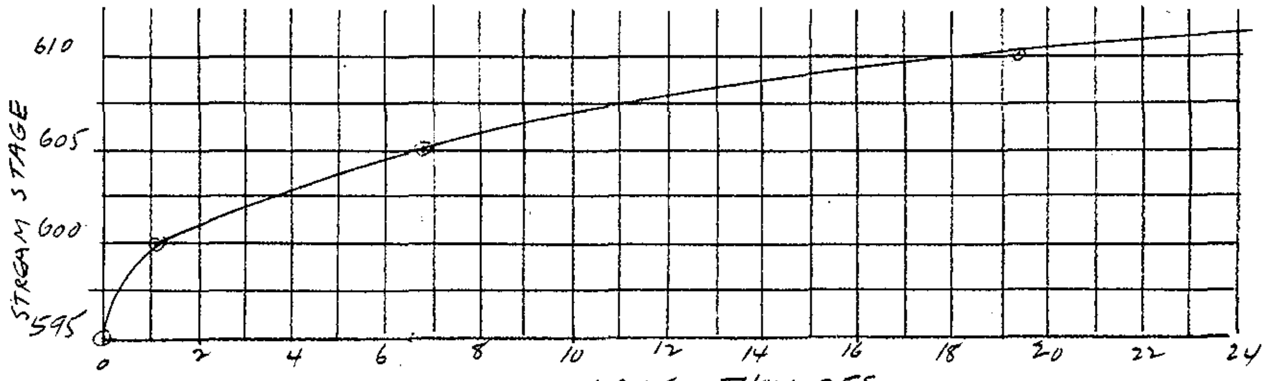
$n = 0.075$

$Q = \frac{1.486}{n} A r^{2/3} S^{1/2} = 1.252 A r^{2/3}$



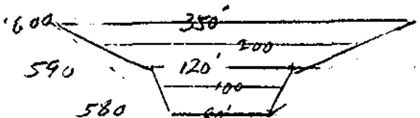
ELEV.	Δ Area	Σ Area	W.P.	r	$r^{2/3}$	Q
595	0		0			0
600	475	475	190.3	2.50	1.84	1094
605	1350	1825	350.6	5.21	3.00	6863
610	2250	4075	550.8	7.40	3.80	19372

$\frac{4075 \times 4500}{43560} = 420 AF$



STAGE-DISCHARGE IN MARSH BROOK

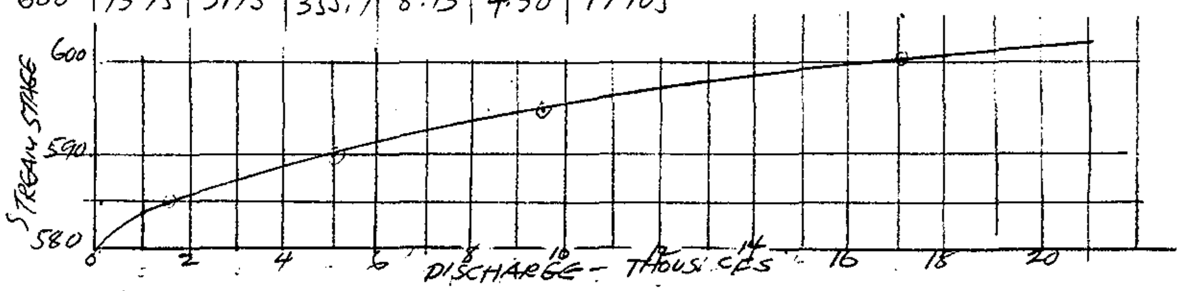
STAGE DISCHARGE - BELOW CONFLUENCE OF POLAND + PEQUABUCK RIVERS



$n = 0.075$ $S = \frac{14}{3100} = .0045$ $S^{1/2} = 0.0672$

$Q = 1.252 A r^{2/3}$

EL.	Δ Area	Σ Area	W.P.	r	$r^{2/3}$	Q
580	-	0				0
585	450	450	102.4	4.40	2.68	1512
590	550	1000	124.8	8.01	4.00	5013
595	800	1800	205.4	8.76	4.25	9579
600	1375	3175	355.7	8.93	4.30	17105



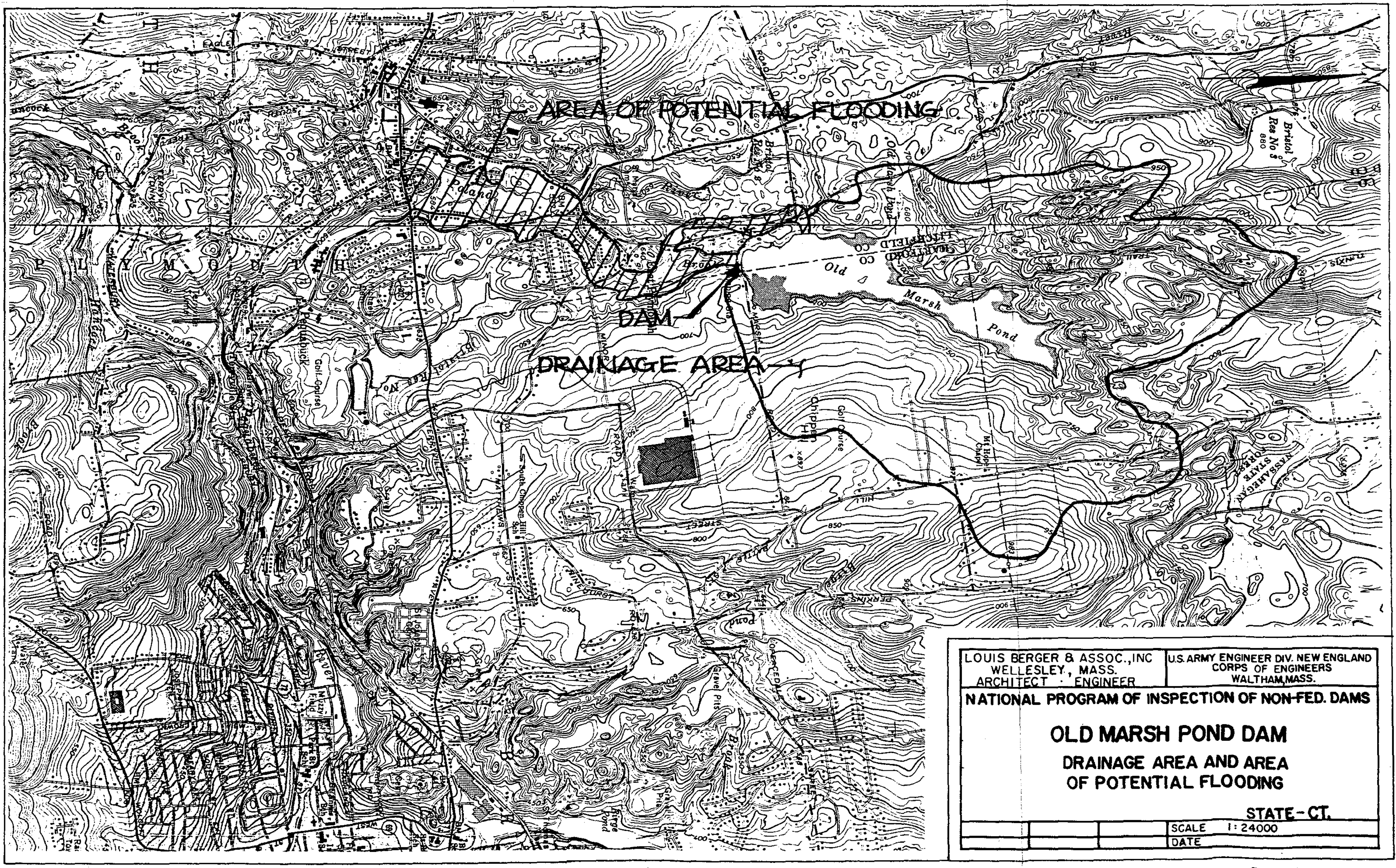

AREA OF POTENTIAL FLOODING
DAM
DRAINAGE AREA
Old Marsh Pond
P L Y M O U T H
LOUIS BERGER & ASSOC., INC
WELLESLEY, MASS.
ARCHITECT · ENGINEER
U.S. ARMY ENGINEER DIV. NEW ENGLAND
CORPS OF ENGINEERS
WALTHAM, MASS.
NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS
OLD MARSH POND DAM
DRAINAGE AREA AND AREA
OF POTENTIAL FLOODING
STATE - CT.
SCALE 1: 24000
DATE

FIG. 8

APPENDIX E

INFORMATION AS CONTAINED IN THE
NATIONAL INVENTORY OF DAMS

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
CT	225	NED	CT	005	[illegible]				OLD MARSH POND DAM	4141.9	7259.8	31JAN79

POPULAR NAME	NAME OF IMPOUNDMENT
	OLD MARSH POND

REGION	BASIN	RIVER OR STREAM	NEAREST DOWNSTREAM CITY–TOWN–VILLAGE	DIST FROM DAM (MI.)	POPULATION
1	[illegible]	MARSH BROOK	EAST PLYMOUTH	1	2500

TYPE OF DAM	YEAR COMPLETED	PURPOSES	STRUCTURAL HEIGHT (FT.)	HYDRAULIC HEIGHT (FT.)	IMPOUNDING CAPACITIES MAXIMUM (ACRE–FT.)	IMPOUNDING CAPACITIES NORMAL (ACRE–FT.)	DIST	OWN	FED R	PRV/FED	SCS A	VER/DATE
RECPRG	1912	S	32	32	3094	1898	NED	N	N	N	N	21FEB79

REMARKS
22-ORIGINAL DAM MODIFIED IN 1950

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.)
1	1100	U	50	4000	126460											

OWNER	ENGINEERING BY	CONSTRUCTION BY
CITY OF BRISTOL	SPERRY + [illegible]	SPERRY + [illegible]

REGULATORY AGENCY			
DESIGN	CONSTRUCTION	OPERATION	MAINTENANCE
[illegible]	NONE	[illegible]	[illegible]

INSPECTION BY	INSPECTION DATE DAY MO YR	AUTHORITY FOR INSPECTION
[illegible] HEPBER + ASSOCIATES, INC.	05JAN78	PL92-367

REMARKS