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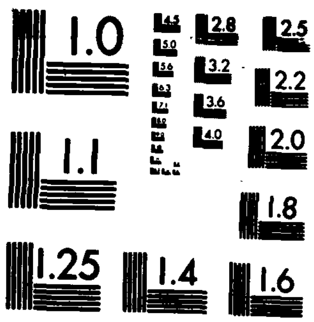

MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

MERRIMACK RIVER BASIN
MARLBOROUGH, MASSACHUSETTS

TYLER DAM
MA 01195

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

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

JUNE, 1981

UNCLASSIFIED

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

REPORT DOCUMENTATION PAGE

READ INSTRUCTIONS BEFORE COMPLETING FORM

1. REPORT NUMBER
MA 01195

2. GOVT ACCESSION NO.

3. RECIPIENT'S CATALOG NUMBER

4. TITLE (and Subtitle)
Tyler Dam

NATIONAL PROGRAM FOR INSPECTION OF NON-FEDERAL DAMS

5. TYPE OF REPORT & PERIOD COVERED
INSPECTION REPORT

6. PERFORMING ORG. REPORT NUMBER

7. AUTHOR(s)
U.S. ARMY CORPS OF ENGINEERS
NEW ENGLAND DIVISION

8. CONTRACT OR GRANT NUMBER(s)

9. PERFORMING ORGANIZATION NAME AND ADDRESS

10. PROGRAM ELEMENT, PROJECT, TASK AREA & WORK UNIT NUMBERS

11. CONTROLLING OFFICE NAME AND ADDRESS
DEPT. OF THE ARMY, CORPS OF ENGINEERS
NEW ENGLAND DIVISION, NEDED
424 TRAPELO ROAD, WALTHAM, MA. 02254

12. REPORT DATE
June 1981

13. NUMBER OF PAGES
55

14. MONITORING AGENCY NAME & ADDRESS (if different from Controlling Office)

15. SECURITY CLASS. (of this report)
UNCLASSIFIED

15a. DECLASSIFICATION/DOWNGRADING SCHEDULE

16. DISTRIBUTION STATEMENT (of this Report)
APPROVAL FOR PUBLIC RELEASE: DISTRIBUTION UNLIMITED

17. DISTRIBUTION STATEMENT (of the abstract entered in Block 20, if different from Report)

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

19. KEY WORDS (Continue on reverse side if necessary and identify by block number)
DAMS, INSPECTION, DAM SAFETY,
Merrimack River Basin
Marlborough, Massachusetts
Assabet River

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)
The dam is an earth embankment structure with a silt 6oe wall. It is 1490 ft. long and has a hydraulic height if 34.4 ft. The dam is considered to be in fair condition. The dam is classified as intermediate in size with a hazard potential of high. Impoundent readings should be taken during flood periods for future reference.

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DEPARTMENT OF THE ARMY

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

REPLY TO
ATTENTION OF:
NEDED

AUG 0 6 1981

Honorable Edward J. King
Governor of the Commonwealth of
Massachusetts
State House
Boston, Massachusetts 02133

Dear Governor King:

Inclosed is a copy of the Tyler Dam (MA-01195) Phase I Inspection Report, prepared under the National Program for Inspection of Non-Federal Dams. This report is based upon a visual inspection, a review of the past performance and a brief hydrological study of the dam. I approve 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 vitally important.

Copies of this report have been forwarded to the Department of Environmental Quality Engineering, and to the owner, Commonwealth of Massachusetts, Department of Environmental Management, Water Resources Commission, Boston, MA. Copies will be available to the public in thirty days.

I wish to thank you and the Department of Environmental Quality Engineering for your cooperation in this program.

Sincerely,

C. E. EDGAR, III
Colonel, Corps of Engineers
Commander and Division Engineer

Incl
As stated

TYLER DAM

MA 01195

MASSACHUSETTS/RHODE ISLAND COASTAL BASIN
MARLBOROUGH, MASSACHUSETTS

PHASE I - INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

NATIONAL DAM INSPECTION PROGRAM
PHASE I - INSPECTION REPORT
BRIEF ASSESSMENT

Identification No.:	MA 01195
Name of Dam:	Tyler
City:	Marlborough
County and State:	Middlesex County, Massachusetts
Stream:	Assabet River
Date of Inspection:	December 8, 1980

Tyler Dam is owned by the Commonwealth of Massachusetts and is used solely for flood control. The dam is an earth embankment structure with a silt core wall. It is 1,490 feet long and has a hydraulic height of 34.4 feet. The storage is 5,700 acre-feet. The 275-foot long emergency spillway discharges to the Assabet River and is located on the east side of the site. A conduit 9 feet wide by 7 feet high also discharges to the Assabet River. The dam was completed in 1980 by the Soil Conservation Service.

As a result of the visual inspection and a review of available data, Tyler Dam is considered to be in fair condition. Major concerns are: sinkholes at the interface of the embankment and riprap; extensive trespassing by motorbikes with consequent erosion of the dam slopes; significant erosion at the upstream slope and right abutment and between the downstream slope and left abutment; irregularity of the dam crest; and lack of erosion protection for the ditch at the downstream toe.

The dam is classified as intermediate in size and a high hazard structure in accordance with the recommended guidelines established by the Corps of Engineers. The test flood for this dam equals the Probable Maximum Flood (PMF). Since the dam is in the intermediate size range and is a high hazard, the PMF was utilized for the hydrologic analysis. The test flood inflow was estimated to be 25,100 cubic feet per second (cfs) and resulted in an outflow discharge estimated to be 22,100 cfs, which would be approximately 2 feet below the top of dam. The maximum spillway capacity with the water level at the dam crest was estimated to be 30,800 cfs, which is about 1.2 times the test flood discharge. A major breach to Tyler Dam would increase the stage along the immediate downstream channel of the Assabet River to approximately 22 feet. Such a breach would cause Robin Hill Road downstream of the dam to be overtopped by about 9 feet, Interstate Route 290 to be overtopped by about 5.5 feet, Bigelow Street to be overtopped by about 8 feet, Chapin Road to be overtopped by about 5 feet, Riverside Park to be overtopped by about 6 feet and Washington Street to be overtopped by about 6 feet. It is estimated that several houses and buildings within the study area would be inundated by 2-8 feet.

It is recommended that the Commonwealth of Massachusetts engage a qualified registered professional engineer to: determine the cause of the small sink holes; investigate the cause of irregularity of the dam crest; specify and oversee procedures for construction of erosion protection where needed; inspect the dam for evidence of seepage when there is water in the reservoir; inspect the dam during each period of significant flood impoundment; and evaluate the seismic stability of the dam. The owner should also repair the sink holes and animal burrows and take meaure to prevent unauthorized vehicular access to the site. The owner should implement and intensify a program of diligent and periodic maintenance.

Impoundment readings should be taken during flood periods for future reference. A surveillance program should be established for use during and after a heavy rainfall, and a downstream warning program developed.

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

Howard Shaevitz, P.E.
Project Manager
M.P.E. No. 28447

SCHOENFELD ASSOCIATES, INC.
Boston, Massachusetts

This Phase I Inspection Report on Tyler Dam (MA-01195) 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 judgement and practice, and is hereby submitted for approval.

Aramast Mahtesian

ARAMAST MAHTESIAN, MEMBER
Geotechnical Engineering Branch
Engineering Division

Carney M. Terzian

CARNEY M. TERZIAN, MEMBER
Design Branch
Engineering Division

J W Finegan

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

APPROVAL RECOMMENDED:

Joe B. Fryar

JOE B. FRYAR
Chief, Engineering Division

PREFACE

This report is prepared under guidance contained in the Recommended Guidelines for Safety Inspection of Dams, for Phase I Investigations. Copies of these guidelines may be obtained from the Office of Chief of Engineers, Washington, D.C. 20314. The purpose of a Phase I Investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general condition of the dam is based upon available data and visual inspections. Detailed investigation and analysis 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 aide in determining the need for more detailed hydrologic and hydraulic studies, considering the size of the dam, its general condition, and the downstream damage potential.

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

TYLER DAM

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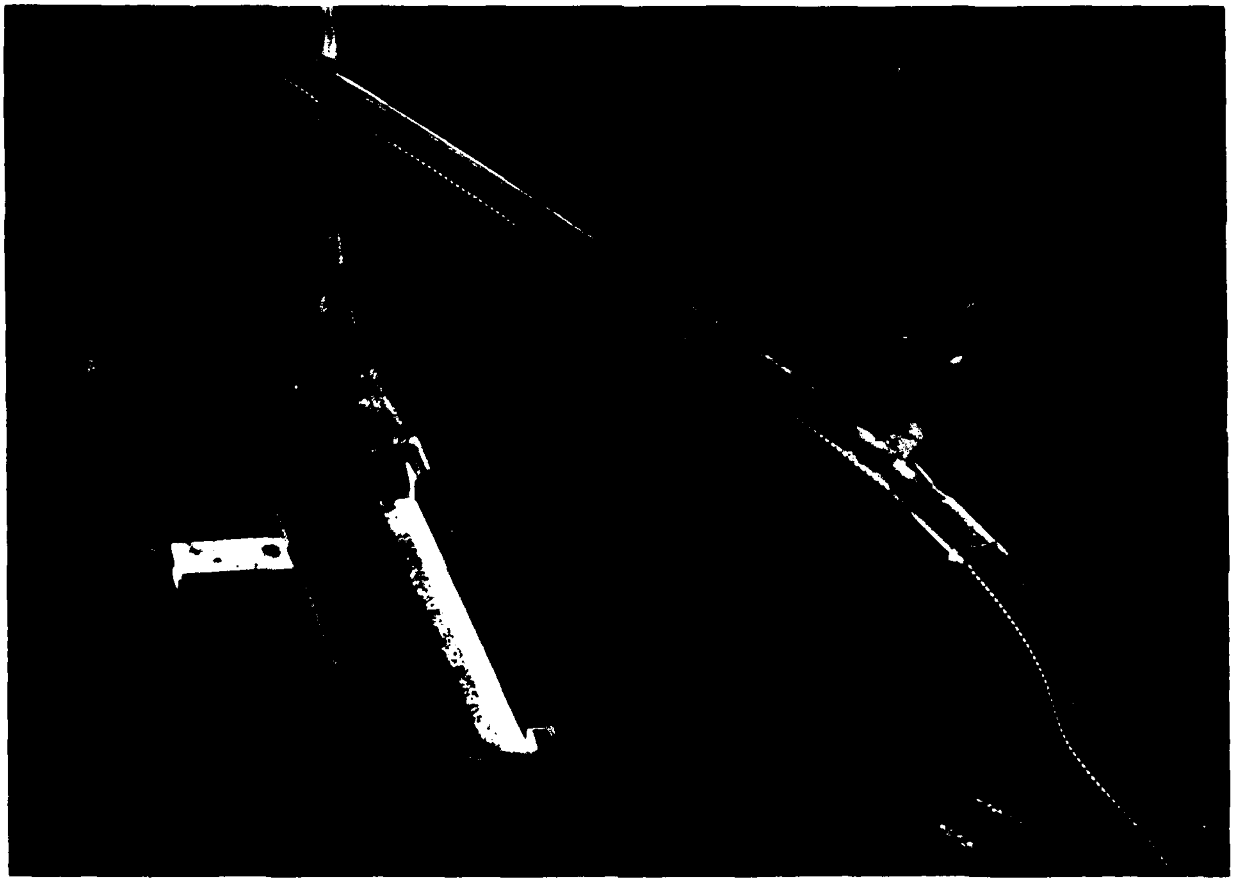
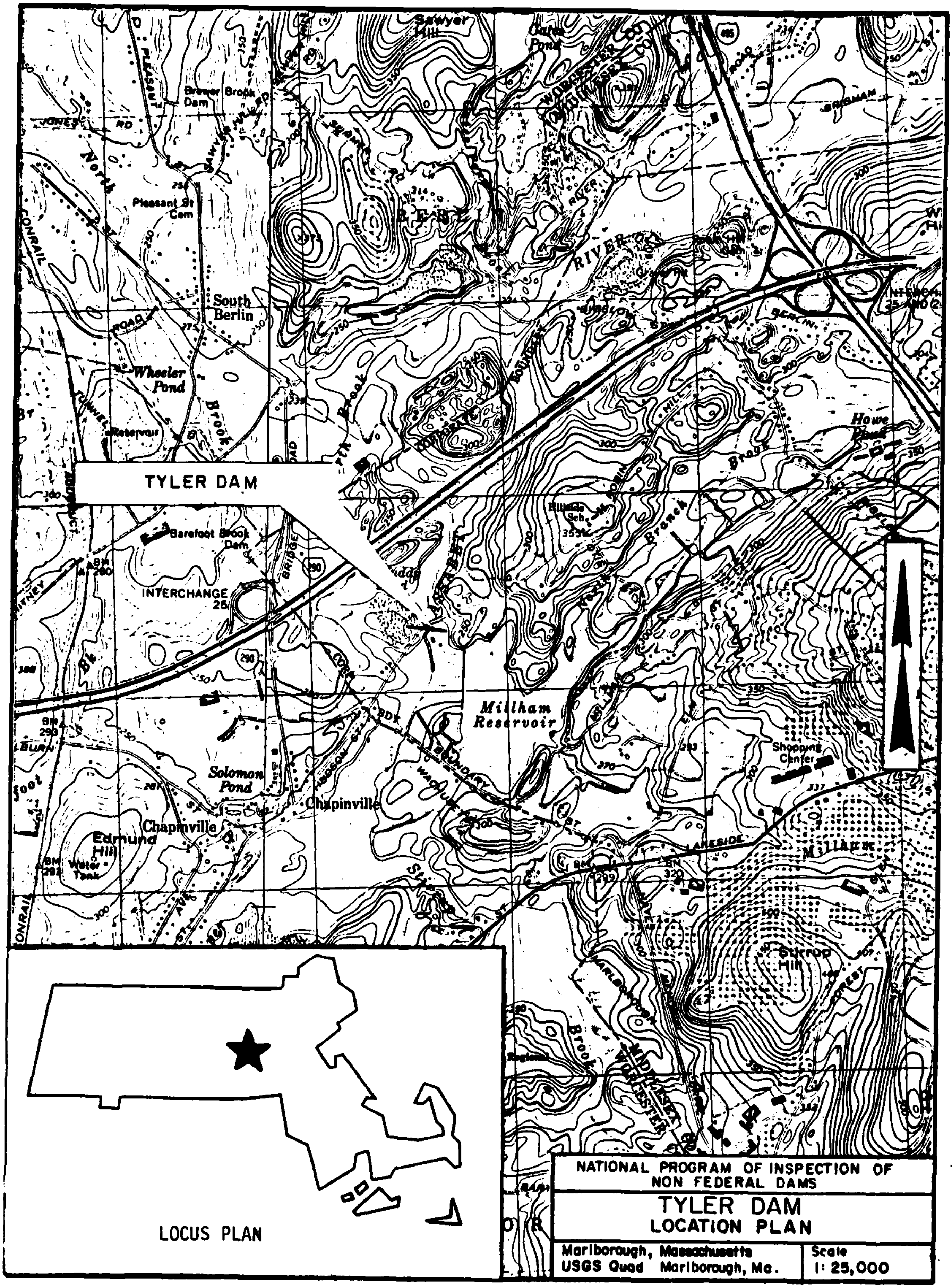


OVERVIEW PHOTOGRAPHY
TYLER DAM


TYLER DAM
LOCUS PLAN
NATIONAL PROGRAM OF INSPECTION OF NON FEDERAL DAMS
TYLER DAM
LOCATION PLAN
Marlborough, Massachusetts
USGS Quad Marlborough, Ma.
Scale
1: 25,000
South Berlin
Wheeler Pond
Barefoot Brook Dam
INTERCHANGE 25
Millham Reservoir
Solomon Pond
Chapinville
Edmund Hill
Shopping Center
Howe Pond
Pleasant St Cem
Brewer Brook Dam
Sawyer Hill

NATIONAL DAM INSPECTION PROGRAM
PHASE I - INSPECTION REPORT
TYLER DAM

SECTION 1
PROJECT INFORMATION

1.1 General

a. Authority. Public Law 92-367, August 8, 1972, authorized the Secretary of the Army, through the Corps of Engineers, to initiate a National Program of Dam Inspection throughout the United States. The New England Division of the Corps of Engineers has been assigned the responsibility of supervising the inspection of dams within the New England Region. Schoenfeld Associates, Inc. has been retained by the New England Division to inspect and report on selected dams in the Commonwealth of Massachusetts. Authorization and notice to proceed were issued to Schoenfeld Associates, Inc. under a letter of October 30, 1980 from Colonel William E. Hodgson, Jr., Deputy Division Engineer. Contract No. DACW33-81-C-0010 has been assigned by the Corps of Engineers for this work.

b. Purpose

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

(2) To encourage and prepare the states to initiate quickly effective dam safety programs for nonfederal dams.

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

1.2 Description of Project

a. Location. Tyler Dam is located in the western portion of Marlborough, Massachusetts and is situated on the Assabet River approximately 2,800 feet (0.53 mile) upstream of Interstate Route 290 and 17,300 feet (3.28 mile) upstream of Interstate 495. The dam is shown on the U.S.G.S. quadrangle sheet for Marlborough, Massachusetts. Its approximate coordinates are N42°-20'-48" and W71°-36'-54". The location of the dam is shown on the preceeding page.

b. Description of Dam and Appurtenances. Tyler Dam is a single-purpose floodwater-retarding structure. The dam is a zoned earthfill structure placed on a sandy silt and silt foundation. The length of the dam, including the spillway, is 1,490 feet. The hydraulic height is 34 feet. The slope of the upstream face of the embankment is 1.0 vertical to 3.5 horizontal. The slope on the downstream face is 1.0 vertical to 3.0 horizontal. A dike extends 530 feet westward from the left abutment of the dam, which is Robin Hill Road.

The principal spillway consists of a two-stage riser and a monolithic conduit which is designed to handle the 100-year frequency storm at controlled discharge rates without discharge occurring at the emergency spillway. The principal spillway riser structure is 50 feet long by 9 feet wide by 20 feet high. The orifice to the riser is 6.37 feet wide x 6.75 feet long with an invert at the same elevation at the principal spillway (207.61). The riser crest consists of 4 openings on each of the long sides of the riser structure. Each opening is 12.5 feet wide x 2.5 feet wide with a weir elevation of 227.00. The 9-foot wide x 7-foot high principal spillway carries water from the riser through the dam.

The emergency spillway is a reinforced concrete spillway 275 feet long with a drop of 23.5 feet. It discharges to the Assabet River and is located on the right side of the dam.

c. Size Classification. The dam is considered to be intermediate in size because the hydraulic height is 34.4 feet and the storage is 5,700 acre-feet. This is in accordance with the Recommended Guidelines for Safety Inspections for Dams, which defines an intermediate dam as having a storage capacity of 1,000 to 50,000 acre-feet.

d. Hazard Classification. The potential for hazard posed by Tyler Dam is classified as high. This is in accordance with the Recommended Guidelines for Safety Inspection for Dams, which defines a high hazard structure as one which poses a threat to more than a few lives. A major breach to Tyler Dam would result in the overtopping of Robin Hill Road by approximately 9 feet, Interstate Route 290 by 5.5 feet, Bigelow Street by 8 feet, Chapin Road by 5 feet, Riverside Park by 6 feet, and Washington Street by 6 feet. In addition, several houses would be inundated by 2-8 feet after the breach.

e. Ownership. The dam is owned by the Commonwealth of Massachusetts.

f. Operator. The dam is operated and maintained by the Commonwealth of Massachusetts, Department of Environmental Management, Water Resources Commission, Division of Water Resources, 100 Cambridge Street, Boston, Massachusetts 02202. The senior civil engineer is Mr. Michael Beshara. His telephone number is (617) 727-3267.

g. Purpose of Dam. The dam is a single purpose structure designed to retard floodwaters in conjunction with nine other structures on the upper Assabet River.

h. Design and Construction History. Tyler Dam was designed and built by the SCS as part of its upper Assabet River Watershed Study. The design was completed in 1973. Construction began in 1976 by G. Bonazzoli & Sons, Inc. of Hudson, Massachusetts. The SCS stopped work on the project in 1977 in order to modify the design. The modification included strengthening buttresses and training walls to prevent overturning. Work was resumed in 1979 and the final inspection occurred in October, 1980.

Plans, calculations, and the design folder were obtained from the SCS, 451 West Street, Amherst, Massachusetts 01002. The telephone number is (413) 256-0441.

i. Normal Operation Procedures. There are no normal operating procedures for Tyler Dam.

1.3 Pertinent Data

a. Drainage Area. The total drainage area for Tyler Dam is 39.5 square miles. Of this total, 18.4 square miles are controlled and 21.1 square miles are uncontrolled.

The Assabet River begins at the Assabet River Dam (George H. Nichols Dam) in western Westborough, Massachusetts and travels in a generally northerly direction to and beyond the Tyler Dam. The Tyler Dam drainage area of 39.5 square miles ranges in elevation from approximately 600 at Green Hill in Shrewsbury to 207 at the invert to the Tyler Dam conduit. Based upon the SCS's Flood Hazard Analysis, Upper Assabet River Tributaries, Massachusetts, approximately 57 percent of the upper Assabet area can be considered to be forest. Of the remaining portions, 19 percent are urbanized, 10 percent are croplands, 5 percent is water-covered, and 9 percent is classified as other. The area around the dam is mostly wooded. There are no cottages or dwellings along the shoreline.

b. Discharge at Dam Site

(1) Outlet works for Tyler Dam consist of a conduit 9 feet wide by 7 feet high with an invert elevation of 207.6. When the water surface elevation is at the emergency spillway elevation of 231.07, the conduit capacity is 1,400 cfs.

(2) Daily records of maximum water surface elevation are not maintained.

(3) The emergency spillway and outlet capacity with the water surface at the top of the dam is approximately 30,700 cfs at elevation 242.0.

(4) The emergency spillway and outlet capacity with the water surface elevation at the test flood elevation of 240.0 is approximately 23,000 cfs.

(5) The total project discharge at the test flood elevation of 240.0 is approximately 23,000 cfs.

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

(1) Streambed at centerline of dam - 207.6

(2) Bottom of cutoff - N/A

(3) Maximum tailwater - unknown

(4) Normal pool - 212.0 (sediment pool)

(5) Flood control pool - 231.07

(6) Emergency spillway crest - 231.07

(7) Design surcharge - 234.07

(8) Test flood surcharge - 240.0

(9) Top of dam - 242.0

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

(1) Normal pool - 1,700

(2) Flood control pool - 6,600

(3) Spillway crest pool - 6,200

(4) Test flood pool - 7,000

(5) Top of dam - 7,600

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

(1) Normal pool - 15

(2) Flood control pool - 2,160

(3) Spillway crest pool - 1,960

(4) Test flood pool - 5,240

(5) Top of dam - 5,700

f. Reservoir Surface (acres)

(1) Normal pool - 18

(2) Flood control pool - 265

(3) Spillway crest pool - 220

(4) Test flood pool - 355

(5) Top of dam - 385

g. Dam

(1) Type - compacted earthfill placed in a sandy silt and silt foundation

(2) Length - 1,490 feet

(3) Hydraulic height - 32.4 feet

(4) Top width - 14.0 feet

(5) Side slopes - 3.5:1 upstream; 3.0:1 downstream

(6) Zoning - Zone 1: silt, 6-inch maximum rock size, 9-inch maximum lift, Class A compaction; Zone 2: silty sand; 12-inch maximum rock size, 18-inch maximum lift, Class C compaction.

(7) Impervious core - none

(8) Cutoff - bottom of cutoff trench varies from elevation 201 to elevation 236

(9) Grout curtain - N/A

(10) Other - N/A

h. Diversion and Regulating Tunnel - N/A

i. Spillway

(1) Type - emergency: reinforced concrete drop spillway
principal: located in riser structure; low level orifice 6.37 feet x 6.75 feet; spillway crest is comprised of 8 sections 12.5 feet x 2.5 feet

(2) Length of weir - emergency: 275 feet
principal: 100 feet

(3) Crest elevation - emergency: 231.07
principal: 227.00

(4) Gates - N/A

(5) U/S channel - 207.61

(6) D/S channel - 207.61

(7) General - emergency: spillway is supported from overturning by 12 buttresses
principal: low level orifice (6.37 feet x 6.75 feet) at same elevation as principal spillway (207.61)

j. Regulating Outlet

(1) Invert - 207.61 (level for entire 73.3-foot length)

(2) Size - 9 feet wide x 7 feet high

(3) Description - concrete box, floor 1'-5", roof 1'-4", walls 1'-3" to 0'-11"

(4) Control mechanism - none

(5) Other - riser crest at elevation 227.0

SECTION 2
ENGINEERING DATA

2.1 Design

A complete set of design drawings and design calculations for Tyler Dam has been prepared by the SCS. The drawings are dated 1972 and 1973 and were modified by additional drawings dated 1974, 1976, 1978, 1979 and 1980.

2.2 Construction

No construction records were available for use in evaluating the dam. The dam was constructed between 1976-79 by the SCS. There was a delay in construction of about 18 months in 1977-1979 to allow for strengthening of buttresses and training walls. The work resumed in 1979. Final inspection occurred in October, 1980.

2.3 Operation

No engineering operation data were found.

2.4 Evaluation

a. Availability. The engineering data used in the preparation of this report are presented in Appendix B.

b. Adequacy. Available engineering data and design drawings are considered adequate for a Phase I investigation, although seepage problems could not be evaluated because there was no water in the reservoir.

c. Validity. The field investigation indicated that the external features of Tyler Dam have not changed substantially from the design drawings of 1972 and 1973, as modified by the drawings of 1974, 1976, 1978, 1979 and 1980.

SECTION 3
VISUAL INSPECTION

3.1 Findings

a. General. The visual inspection of Tyler Dam was conducted on December 8, 1980. The field inspection team consisted of personnel from Schoenfeld Associates, Inc., D. Baugh Associates, Inc., and Geotechnical Engineers, Inc. Representatives of the Soil Conservation Service and the Massachusetts Water Resources Commission were also present. Inspection checklists, completed during the field site visit, are included in Appendix A. Selected photographs of the dam site are included in Appendix C.

Tyler Dam is a flood-control dam. At the time of the inspection there was little water in the reservoir. Consequently, it was not possible to determine whether significant seepage occurs through the embankment, foundations, and abutments when there is water in the reservoir.

In general, the overall condition of the dam and its appurtenant structures is fair.

b. Dam. Tyler Dam is an earth embankment structure with a 275-foot long spillway and training walls. The embankment was seeded in the summer of 1980. At the time of the inspection grass was growing on the crest, upstream slope, and downstream slope, but it was not yet well established (Photo Nos. 1 and 2). The crest of the dam is 1 to 2 feet higher at the spillway abutments. A portion of the crest about 50 to 150 feet left of the spillway is quite irregular (Photo No. 3). This irregularity does not appear to be the result of slumping on either the upstream or downstream slopes, and may be the result of post-construction settlement of the embankment. Evidence of extensive trespassing by motorbikes on the upstream slope and consequent erosion in some of the tracks was observed.

There is significant erosion at the contact between the upstream slope and the right abutment and between the downstream slope and the highway fill at the left abutment. Newly seeded grass is beginning to grow in the eroded areas at the contacts and may eventually stabilize them against further erosion.

Both abutments consist of soil. The left abutment is a highway fill at Robin Hill Road, and there is a dike which extends westward from the highway fill. The results of the visual inspection of the dike are described below.

c. Appurtenant Structures. There is a concrete overflow spillway structure with an earth embankment approach near the right end of the dam (Photo Nos. 4 and 5). Immediately adjacent to the upstream side of the concrete weir in the spillway structure, there is a zone of rockfill (Photo No. 6) which is about 4 feet wide and consists of pieces of rock up to about 2 feet in size which are slush grouted at the surface. Immediately upstream of the rockfill is a sandy earthfill which has developed extensive shallow sinkholes (Photo No. 7), apparently the result of piping into the large voids in the rockfill. Coarse riprap has been placed on the lower part of the downstream slope of the embankment immediately adjacent to the wingwall at each end of the concrete spillway structure (Photo Nos. 8 and 9). The large riser is in very good condition with no indications of any problems (Photo Nos. 10 and 11).

A dike extends about 530 feet westward from the highway embankment which constitutes the left abutment of the dam. The crest, upstream slope, and downstream slope of the dike have a sparse cover of grass and weeds. There are motorbike tracks and erosion channels on the dike, especially on the downstream slope, and a 6-inch diameter animal burrow on the downstream slope about 100 feet from the street. Drainage from a depression downstream of the dike (enclosed by the dike, the highway fill, and natural ground on the downstream side of the dike) is conveyed under the highway fill in a culvert and flows from the culvert to the main river channel in an open ditch along the downstream toe of the left end of the dam. There is no erosion protection on the bottom and sides of this channel (Photo No. 12) and the culvert is partially filled with sediment that has been eroded from fields in the depression.

d. Reservoir Area. This is a flood-control dam and there was little water in the reservoir at the time of inspection.

The area immediately adjacent to the reservoir is gently sloped and moderately vegetated. In addition, directly east and upstream of the dam and adjacent to the reservoir area is the Millham Reservoir and its dam. At present, there are no signs of sloughing or erosion along the shoreline of the Tyler Reservoir. A rapid rise in the water level of the reservoir will not endanger life or property.

e. Downstream Channel. A highway embankment at Robin Hill Road crosses the channel immediately downstream of the dam (Photo Nos. 13 and 14). The natural channel is in good condition. The banks of the channel between the dam and Robin Hill Road are generally free of trees and brush until the channel passes under Hudson Street, at which point the channel is overhung by light vegetation.

3.2 Evaluation

On the basis of the visual inspection the dam is judged to be in fair condition.

Extensive trespassing by motorbikes on the upstream slope of the dam and the downstream slope of the dike and consequent erosion in some places could lead to further erosion and possible breaching of the dam or dike if not controlled.

Significant erosion has occurred at the contact between the upstream slope and the right abutment and between the downstream slope and the highway fill which constitutes the left abutment. These areas have been seeded, but the grass has not yet become well enough established to prevent erosion, which could lead to breaching of the dam if not controlled.

Irregularity of a portion of the crest of the dam left of the spillway abutment, although not associated with visible slumping of either the upstream or downstream slope of the dam, may indicate some sort of embankment problem that could lead to instability of the dam.

An animal burrow on the downstream slope of the dike could become a focus for the development of seepage or piping problems when there is water in the reservoir.

A culvert under the highway fill at the left abutment is partially clogged with sediment, which reduces its capacity to drain water from the depression at the downstream side of the dike. If the culvert became completely plugged, rainfall would be ponded in this depression.

A ditch at the downstream toe of the left end of the dam has no erosion protection. Water flowing in this ditch could erode the toe of the dam and result in instability of the downstream slope.

Sinkholes in the sand fill which is adjacent to the coarse rockfill against the upstream side of the concrete emergency spillway weir appear to be the result of piping of the sand into large voids of the rockfill. They do not appear to be evidence of any stability problem.

SECTION 4
OPERATIONAL AND MAINTENANCE PROCEDURES

4.1 Operational Procedures

a. General. Tyler Dam is used solely for flood control. It is one of 10 structures designed by the SCS along the upper Assabet River to retard flood flows.

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

4.2 Maintenance Procedures

a. General. The Water Resources Commission is responsible for maintenance of the dam. There are no established procedures or manuals. Current procedures include a yearly inspection, usually in the spring, by representatives of the Water Resources Commission, the Soil Conservation Service, and the City of Marlborough. Any deficiencies found would be corrected by a contractor engaged by the owner during the summer months. The work would then be inspected for compliance with the contract.

b. Operating Facilities. No formal maintenance procedures for the operating facilities were disclosed.

4.3 Evaluation

Even though there are no mechanical or electrical components at Tyler Dam to warrant daily or weekly maintenance, a formal inspection plan should be prepared to insure that no acute problems arise because of lack of concern. Additionally, a formal warning system should be prepared to insure against injuries or loss of life at Robin Hill Road and/or Interstate Route 290 or in case of an emergency at the dam. The project should be monitored during flooding periods.

SECTION 5
EVALUATION OF HYDROLOGIC/HYDRAULIC FEATURES

5.1 General

Tyler Dam is an earth embankment structure. According to design drawings, the dam is 1,490 feet long and has a maximum structural height of 46 feet. The concrete emergency spillway has a length of 275 feet and is located on the right side of the site. The crest elevation is 231.07. The spillway discharges to the Assabet River.

The normal outlet is a 9-foot wide by 7-foot high drain located on the east side of the reservoir and discharges to the Assabet River. The reservoir is used for flood protection.

5.2 Design Data

Hydrological or hydraulic design data were obtained from the Soil Conservation Service, 451 West Street, Amherst, Massachusetts 01002.

5.3 Experience Data

Daily readings of the water surface elevations are not taken.

5.4 Test Flood Analysis

The hydrologic evaluation was performed utilizing data obtained from the Soil Conservation Service, data gathered during the field inspection, watershed size, and an estimated test flood equal to the Probable Maximum Flood (PMF). The full PMF test flood was selected because the dam is a high hazard structure. The controlled drainage basin is controlled by four dams built by the Soil Conservation Service. Routed flows from these dams approximates the PMF conditions. Routed outflows from controlled drainage areas were obtained from SCS design data. The uncontrolled drainage basin is considered rolling. Therefore, the "rolling" curve from the Corps of Engineers set of guide curves was used to determine the inflow from the uncontrolled drainage area.

The routed flood peak inflow from the controlled drainage area (18.4 square miles) was 12,500 cfs. The estimated maximum probable flood peak flow rate of 620 cfs per square mile and an uncontrolled drainage area of 21.1 square miles yielded a test flood inflow of 13,100 cfs. The total flood peak inflow when time is taken into consideration (19.75 hours to peak) is 25,100 cfs. The test flood was routed through the dam in accordance with the Corps of Engineers procedure for Estimating Effect of Surcharge Storage on Maximum Probable Discharge. The reservoir water surface was assumed to be at elevation 212.0 prior to the flood routing. The project discharge was estimated to be 22,100 cfs. This analysis indicated that the dam crest would not be overtopped by the test flood, but that the water surface elevation would be about 2 feet below the dam crest. The maximum spillway capacity with the water level at the dam crest was estimated to be 30,800 cfs. Therefore, the 275-foot long by 10-foot deep emergency spillway channel has adequate capacity to handle the test flood discharge. The capacity of the spillway was estimated to be approximately 30,800 cfs, which is 1.2 times the test flood discharge.

5.5 Dam Failure Analysis

The impact of dam failure with the reservoir surface at the dam crest was assessed utilizing the "Rule of Thumb" Guidance for Estimating Downstream Dam Failure Hydrographs provided by the Corps of Engineers. The analysis covered a reach extending approximately 5.4 miles downstream to a point where the Assabet River passes under Washington Street with the potential for causing the loss of more than a few lives. Based on this analysis, Tyler Dam was classified as a high hazard.

A dry breach was assumed. With the water surface at the top of spillway crest, the flow through the principal spillway was 1,400 cfs. The breach of the emergency spillway was estimated to result in an outflow of 23,600 cfs.

A major breach to Tyler Dam would increase the stage along the immediate downstream channel of the Assabet River by approximately 22 feet. Such a breach would cause Robin Hill Road downstream of the dam to be overtopped by about 9 feet, Interstate Route 290 to be overtopped by about 5.5 feet, Chapin Road to be overtopped by about 5 feet, Riverside Park to be overtopped by about 6 feet, and Washington Street to be overtopped by about 6 feet. Approximately six houses on the west bank upstream of Chapin Street approximately 10 feet above the stream prior to the breach would be subject to a possible breach stage of 18 feet. Businesses at Washington Street approximately 6 feet above the stream would be subject to a possible breach stage of 11 feet.

SECTION 6
EVALUATION OF STRUCTURAL STABILITY

6.1 Visual Observations

The general structural stability of the dam is fair as evidenced by vertical, horizontal, and lateral alignment. Since this dam was recently completed (October 1980), any potential stability problems have not yet had time to manifest. The only area of concern is located just before the spillway weir where several small sinkholes were noted at the grouted riprap/earth embankment interface.

The following conditions observed during the visual inspection, however, are indicative of problems that could result in long-term structural instability.

(1) Extensive trespassing by motorbikes on the upstream slope of the dam and the downstream slope of the dike, along with consequent erosion in some places, could lead to further erosion and possible breaching of the dam or dike if not controlled.

(2) Significant erosion has occurred at the contact between the upstream slope and the right abutment and between the downstream slope and the highway fill which constitutes the left abutment. These areas have been seeded, but the grass has not yet become well enough established to prevent erosion, which could lead to breaching of the dam if not controlled.

(3) Irregularity of a portion of the crest of the dam left of the spillway abutment, although not associated with visible slumping of either the upstream or downstream slope of the dam, may indicate some sort of embankment problem that could lead to instability of the dam.

(4) An animal burrow on the downstream slope of the dike could become a focus for the development of seepage or piping problems when there is water in the reservoir.

(5) A ditch at the downstream toe of the left end of the dam has no erosion protection. Water flowing in this ditch could erode the toe of the dam and result in instability of the downstream slope.

Sinkholes in the sand fill which is adjacent to the coarse rockfill against the upstream side of the emergency spillway appear to be the result of piping of the sand into large voids of the rockfill. They do not appear to be evidence of any stability problem.

Because there was little water in the reservoir at the time of the inspection, it was not possible to determine whether there are any seepage problems when there is water in the reservoir.

6.2 Design and Construction Data

A complete set of design drawings dated 1972 and 1973, is available for this dam. A complete set of modifications on drawings dated 1974, 1976, 1978, 1979 and 1980 is also available.

The drawings indicate that the foundation of the dam consists of sand, silty sand, and silt, underlain by bedrock. It is noted on the drawings that topsoil and peat, where present, are to be removed prior to placement of the embankment.

The core of the dam is comprised of silt. It has a top elevation of 235.0 (6.4 feet below the crest of the dam), a top width of 14 feet, and an upstream slope of 1H:1V. The shell of the dam consists of silty sand, which is specified to have 100% passing the No. 10 sieve and not more than 5% passing the No. 200 sieve.

The two-layer chimney filter-drain is shown between the core and the downstream shell material, from the base of the embankment up to elevation 229, which is 6 feet below the top of the core and 5 feet below the design high-water level. The layer next to the core is specified to have 100% passing the No. 10 sieve and 20 to 50% passing the No. 200 sieve; the second layer, next to the downstream shell material, is specified to have 100% passing the 3-inch sieve and less than 3% passing the No. 200 sieve. A 6-inch asbestos perforated piping is specified to drain the second layer.

The upstream slope is 3.5H:1V and the downstream slope is 3H:1V.

The drawings show a drain fill under the downstream apron of the spillway and a concrete cutoff wall to bedrock under the concrete spillway structure.

It appears that there was no seismic analysis of the stability of the dam.

6.3 Post-Construction Changes

Because the dam is newly constructed, there are no post-construction changes.

6.4 Seismic Stability

This dam is in the boundary region between Seismic Zones 2 and 3. Phase I guidelines recommend, as a minimum, that suitable analysis made by conventional equivalent static load methods should be in record for dams in Zone 3. As far as can be determined, no such analysis has been made.

SECTION 7
ASSESSMENT, RECOMMENDATIONS, AND REMEDIAL MEASURES

7.1 Dam Assessment

a. Condition. After consideration of the available information, the results of the inspection, contact with the owner, and hydrologic/hydraulic computations, Tyler Dam is judged to be in fair condition. The following conditions are indicative of potential long-term problems:

(1) Extensive trespassing by motorbikes on the upstream slope of dam and the downstream slope of the dike, with the consequent erosion in some places, could lead to further erosion and possible breaching of the dam or dike if not controlled.

(2) Significant erosion has occurred at the contact between the upstream slope and the right abutment and between the downstream slope and the highway fill which constitutes the left abutment. These areas have been seeded but the grass has not yet become well enough established to prevent erosion, which could lead to breaching of the dam if not controlled.

(3) Irregularity of a portion of the crest of the dam left of the spillway abutment, although not associated with visible slumping of either the upstream or downstream slope of the dam, may indicate some sort of embankment problem that could lead to instability of the dam.

(4) An animal burrow on the downstream slope of the dike could become a focus for the development of seepage or piping problems when there is water in the reservoir.

(5) A ditch at the downstream toe of the left end of the dam has no erosion protection. Water flowing in this ditch could erode the toe of the dam and result in instability of the downstream slope.

Sinkholes in the sand fill which is adjacent to the coarse rockfill against the upstream side of the concrete emergency spillway weir appear to be the result of piping of the sand into large voids of the rockfill. They do not appear to be evidence of any stability problem.

b. Adequacy of Information. The information obtained from the design drawings and the results of the visual inspection are adequate for the purposes of this Phase I study, with the exception that potential seepage problems could not be evaluated on the basis of the visual inspection because there was no water in the reservoir.

c. Urgency. The owner should implement the recommendations in 7.2 and 7.3 within one year after receipt of this Phase I report.

7.2 Recommendations

The following investigations should be carried out and needed corrections performed under the direction of a registered engineer qualified in the design and construction of dams.

(1) Determine the cause of the small sinkholes at the earth embankment/ spillway riprap interface.

(2) Specify and oversee procedures for repairing erosion on the upstream slope of the dam, the downstream slope of the dike, the contact between the upstream slope of the dam and the right abutment, and the downstream slope of the dam and the highway embankment which comprises the left abutment.

(3) Investigate the cause of the irregularity of the crest in a zone to the left of the spillway abutment, and design and oversee remedial measures, if needed.

(4) Specify and oversee construction of erosion protection for the open ditch at the downstream toe near the left end of the dam.

(5) Inspect the dam for evidence of seepage when there is sufficient water in the reservoir.

(6) Inspect the dam during each period of significant flood impoundment.

(7) Evaluate the seismic stability of the embankment and its foundation.

7.3 Remedial Measures

a. Operating and Maintenance Procedures. The owner should:

(1) Implement and intensify a program of diligent and periodic maintenance including, but not limited to, mowing brush on slopes, backfilling animal burrows or tire ruts with suitable well tamped material, and cleaning debris from spillways and slopes.

(2) Implement measures to prevent unauthorized vehicular access to the site.

(3) Continue the annual technical inspection by representatives of the owner, the SCS and the community. A participant in the inspection team should be a registered professional engineer qualified in the design and construction of dams.

(4) Reservoir impoundment readings should be taken during flood periods for future reference.

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

7.4 Alternatives

There are no practical alternatives to the remedial measures described in Section 7.3.

APPENDIX A

INSPECTION CHECK LIST

VISUAL INSPECTION CHECKLIST
PARTY ORGANIZATION

PROJECT Tyler Dam, Marlborough, MA

DATE Dec. 8, 1980

TIME 9:00 A.M.

WEATHER Cool, Partly Cloudy

W.S. ELEV. 213.3 UPSTREAM
213.3 DOWNSTREAM

PARTY:

1. Howard Shaevitz, SAI
2. Peter Austin, DBA
3. Ronald Hirschfeld, GEI
4. Ernest Struzziero, Mass. WRC
5. Chester Dodge, SCS
6.
7.
8.
9.
10.

	PROJECT FEATURE	INSPECTED BY	REMARKS
1.	Hydrology/Hydraulics	Howard Shaevitz	
2.	Structural Stability	Peter Austin	
3.	Soils and Geology	Ronald Hirschfeld	
4.			
5.			
6.			
7.			
8.			
9.			
10.			

PERIODIC INSPECTION CHECKLIST

PROJECT Tyler Dam, Marlborough, MA DATE Dec. 8, 1980

PROJECT FEATURE Dam Embankment NAME

DISCIPLINE NAME

AREA EVALUATED	CONDITION
DAM EMBANKMENT	
Crest Elevation	231.07
Current Pool Elevation	213.3
Maximum Impoundment to Date	Unknown
Surface Cracks	None observed
Pavement Condition	Not paved
Movement or Settlement of Crest	Crest elevation is irregular from about 50 to 150' left of spillway "island"
Lateral Movement	None observed
Vertical Alignment	Fair
Horizontal Alignment	Good
Condition at Abutment and at Concrete Structures	Good except for sinkholes in fill at upstream edge of coarse rock against upstream side of concrete spillway weir
Indications of Movement of Structural Items on Slopes	None observed
Trespassing on Slopes	Extensive trespassing by motorbikes on upstream slope
Sloughing or Erosion of Slopes or Abutments	Considerable erosion of upstream slope in motorbike tracks; also contact between upstream slope and right abutment
Rock Slope Protection - Riprap Failures	No riprap, except at toe of downstream slope next to spillway training walls
Unusual Movement or Cracking at or Near Toe	None observed
Unusual Embankment or Downstream Seepage	None observed
Piping or Boils	None observed
Foundation Drainage Features	None observed
Toe Drains	None observed
Instrumentation System	None observed
Vegetation	Grass recently seeded

PERIODIC INSPECTION CHECKLIST

PROJECT Tyler Dam, Marlborough, MA DATE Dec. 8, 1980

PROJECT FEATURE Dike Embankment NAME ______

DISCIPLINE ______ NAME ______

AREA EVALUATED	CONDITION
DIKE EMBANKMENT	
Crest Elevation	244.66
Current Pool Elevation	213.3
Maximum Impoundment to Date	Unknown
Surface Cracks	None observed
Pavement Condition	No pavement
Movement or Settlement of Crest	None observed
Lateral Movement	None observed
Vertical Alignment	Good
Horizontal Alignment	Good
Condition at Abutment and at Concrete Structures	Good
Indications of Movement of Structural Items on Slopes	None observed
Trespassing on Slopes	Extensive motorbike trespassing
Sloughing or Erosion of Slopes or Abutments	Extensive erosion on slopes, especially on downstream slope
Rock Slope Protection - Riprap Failures	No riprap
Unusual Movement or Cracking at or Near Toe	None observed
Unusual Embankment or Downstream Seepage	None observed
Piping or Boils	None observed
Foundation Drainage Features	None observed
Toe Drains	None observed
Instrumentation System	None observed
Vegetation	Very sparse

PERIODIC INSPECTION CHECKLIST

PROJECT Tyler Dam, Marlborough, MA DATE Dec. 8, 1980

PROJECT FEATURE Intake Channel NAME

DISCIPLINE NAME

AREA EVALUATED	CONDITION
OUTLET WORKS - INTAKE CHANNEL AND INTAKE STRUCTURE	
a. Approach Channel	
Slope Conditions	Good
Bottom Conditions	Soil
Rock Slides or Falls	None
Log Boom	None
Debris	None
Condition of Concrete Lining	Not applicable
Drains or Weep Holes	Not applicable
b. Intake Structure	
Condition of Concrete	Excellent
Stop Logs and Slots	None

PERIODIC INSPECTION CHECKLIST

PROJECT Tyler Dam, Marlborough, MA DATE Dec. 8, 1980

PROJECT FEATURE Control Tower NAME ____________

DISCIPLINE ____________ NAME ____________

AREA EVALUATED	CONDITION
OUTLET WORKS - CONTROL TOWER	Not applicable
a. Concrete and Structural	
General Condition	
Condition of Joints	
Spalling	
Visible Reinforcing	
Rusting or Staining of Concrete	
Any Seepage or Efflorescence	
Joint Alignment	
Unusual Seepage or Leaks in Gate Chamber	
Cracks	
Rusting or Corrosion of Steel	
b. Mechanical and Electrical	
Air Vents	
Float Wells	
Crane Hoist	
Elevator	
Hydraulic System	
Service Gates	
Emergency Gates	
Lightning Protection System	
Emergency Power System	
Wiring and Lighting System	

PERIODIC INSPECTION CHECKLIST

PROJECT Tyler Dam, Marlborough, MA DATE Dec. 8, 1980

PROJECT FEATURE Transition & Conduit NAME ____________

DISCIPLINE ____________ NAME ____________

AREA EVALUATED	CONDITION
OUTLET WORKS - TRANSITION AND CONDUIT	
General Condition of Concrete	Excellent
Rust or Staining on Concrete	None
Spalling	None
Erosion or Cavitation	None
Cracking	None
Alignment of Monoliths	Good
Alignment of Joints	
Numbering of Monoliths	

PERIODIC INSPECTION CHECKLIST

PROJECT Tyler Dam, Marlborough, MA   DATE Dec. 8, 1980

PROJECT FEATURE Outlet Structure   NAME ______

DISCIPLINE ______   NAME ______

AREA EVALUATED	CONDITION
OUTLET WORKS - OUTLET STRUCTURE AND OUTLET CHANNEL	
General Condition of Concrete	Excellent
Rust or Staining on Concrete	None
Spalling	None
Erosion or Cavitation	None observed
Visible Reinforcing	None
Any Seepage or Efflorescence	None observed
Condition at Joints	Excellent
Drain Holes	Not visible beneath tailwater
Channel	
Loose Rock or Trees Overhanging Channel	None
Condition of Discharge Channel	Good

PERIODIC INSPECTION CHECKLIST

PROJECT Tyler Dam, Marlborough, MA DATE Dec. 8, 1980

PROJECT FEATURE Spillway Weir NAME ____________

DISCIPLINE ____________ NAME ____________

AREA EVALUATED	CONDITION
<u>OUTLET WORKS - SPILLWAY WEIR, APPROACH AND DISCHARGE CHANNELS</u>	
a. Approach Channel	
General Condition	Good
Loose Rock Overhanging Channel	None
Trees Overhanging Channel	None near dam
Floor of Approach Channel	Soil
b. Weir and Training Walls	
General Condition of Concrete	Excellent but there are some sink holes in front of grouted riprap spillway crest
Rust or Staining	None
Spalling	None
Any Visible Reinforcing	None
Any Seepage or Efflorescence	None observed
Drain Holes	Not visible beneath tailwater
c. Discharge Channel	
General Condition	General
Loose Rock Overhanging Channel	None
Trees Overhanging Channel	None
Floor of Channel	Not visible beneath tailwater
Other Obstructions	Highway bridge immediately downstream of dam

PERIODIC INSPECTION CHECKLIST

PROJECT Tyler Dam, Marlborough, MA DATE Dec. 8, 1980

PROJECT FEATURE Service Bridge NAME

DISCIPLINE NAME

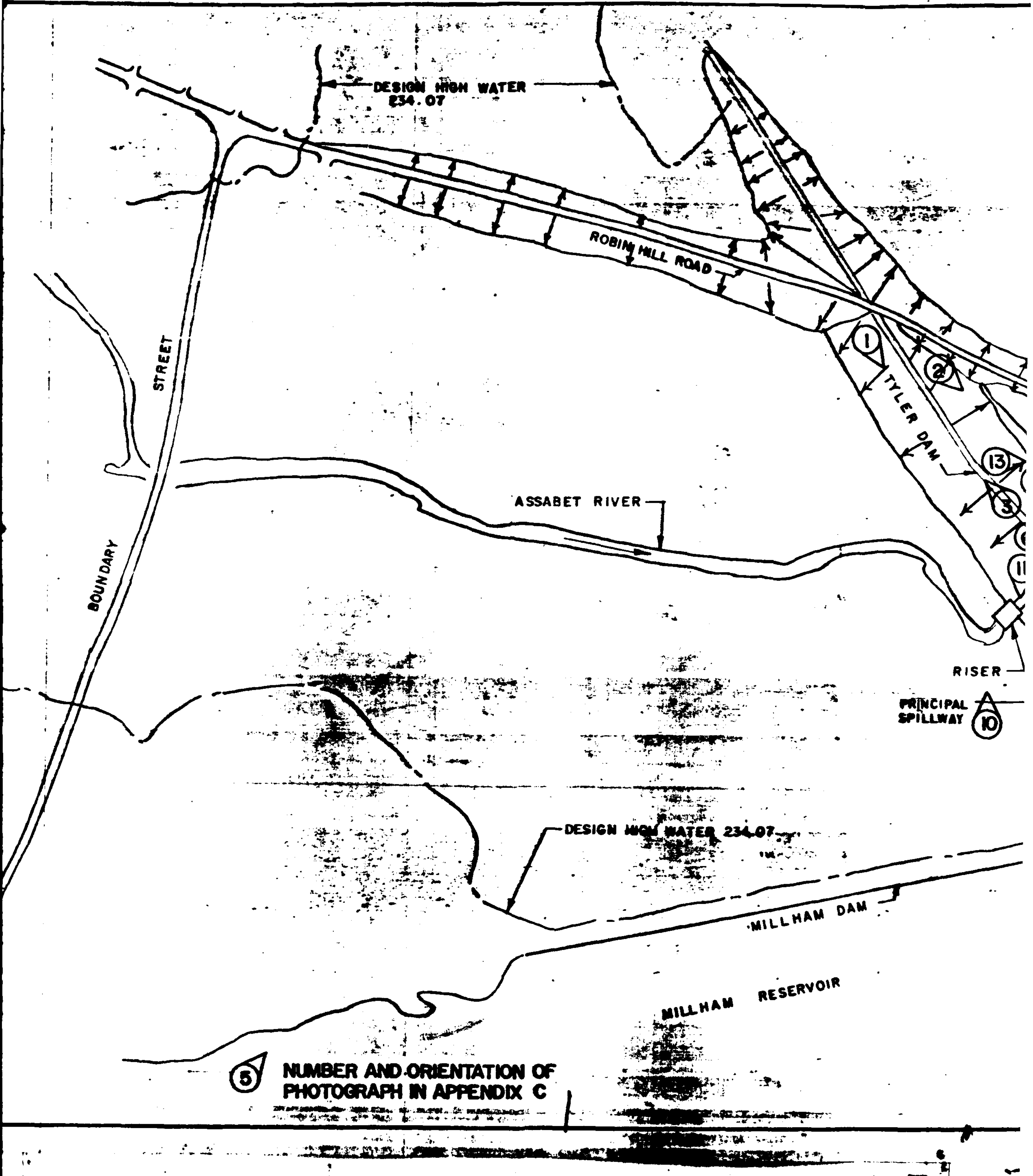
AREA EVALUATED	CONDITION
OUTLET WORKS - SERVICE BRIDGE	Not applicable
a. Super Structure	
Bearings	
Anchor Bolts	
Bridge Seat	
Longitudinal Members	
Underside of Deck	
Secondary Bracing	
Deck	
Drainage System	
Railings	
Expansion Joints	
Paint	
b. Abutment & Piers	
General Condition of Concrete	
Alignment of Abutment	
Approach to Bridge	
Condition of Seat & Backwall	

APPENDIX B

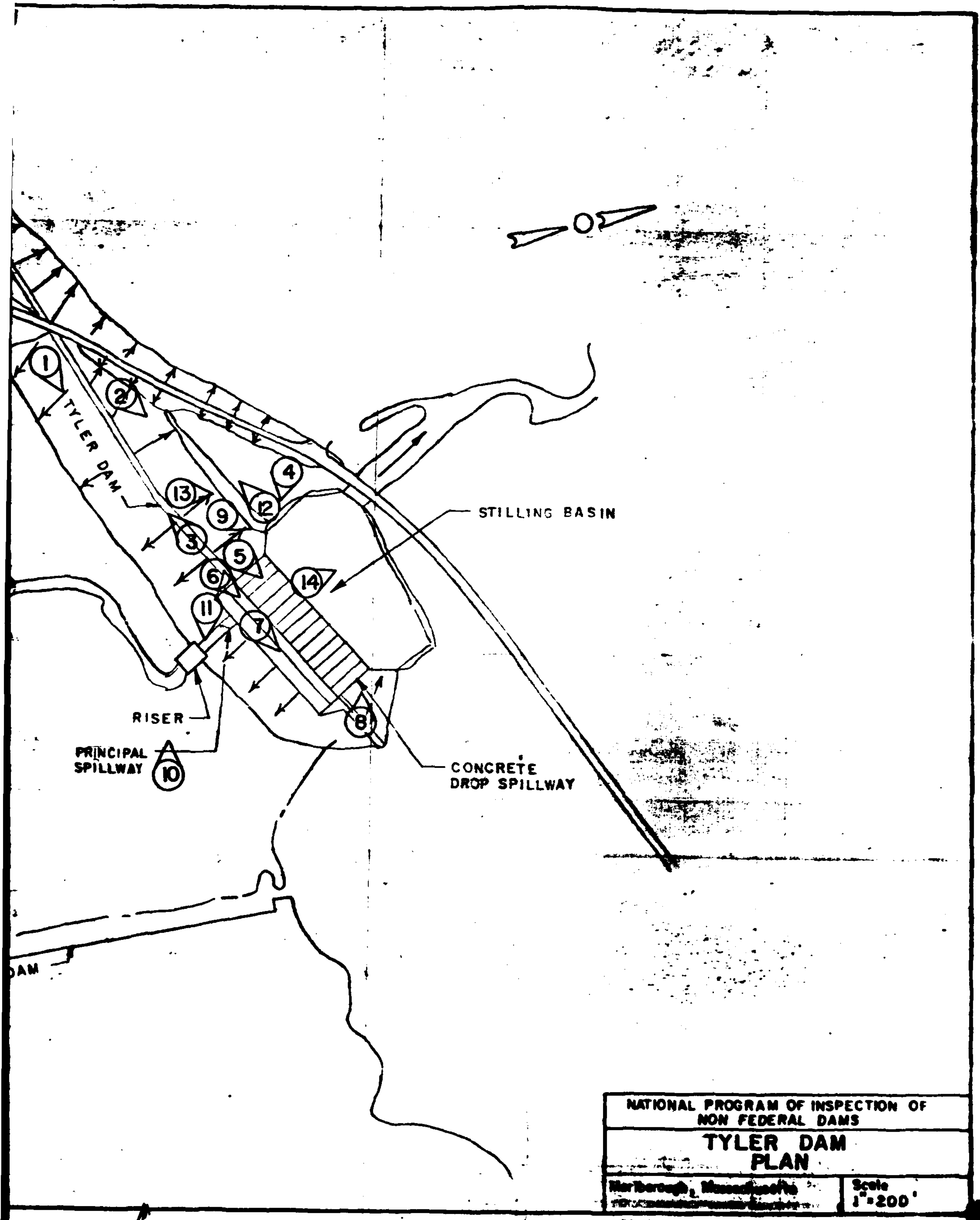
ENGINEERING DATA

Available Engineering Data

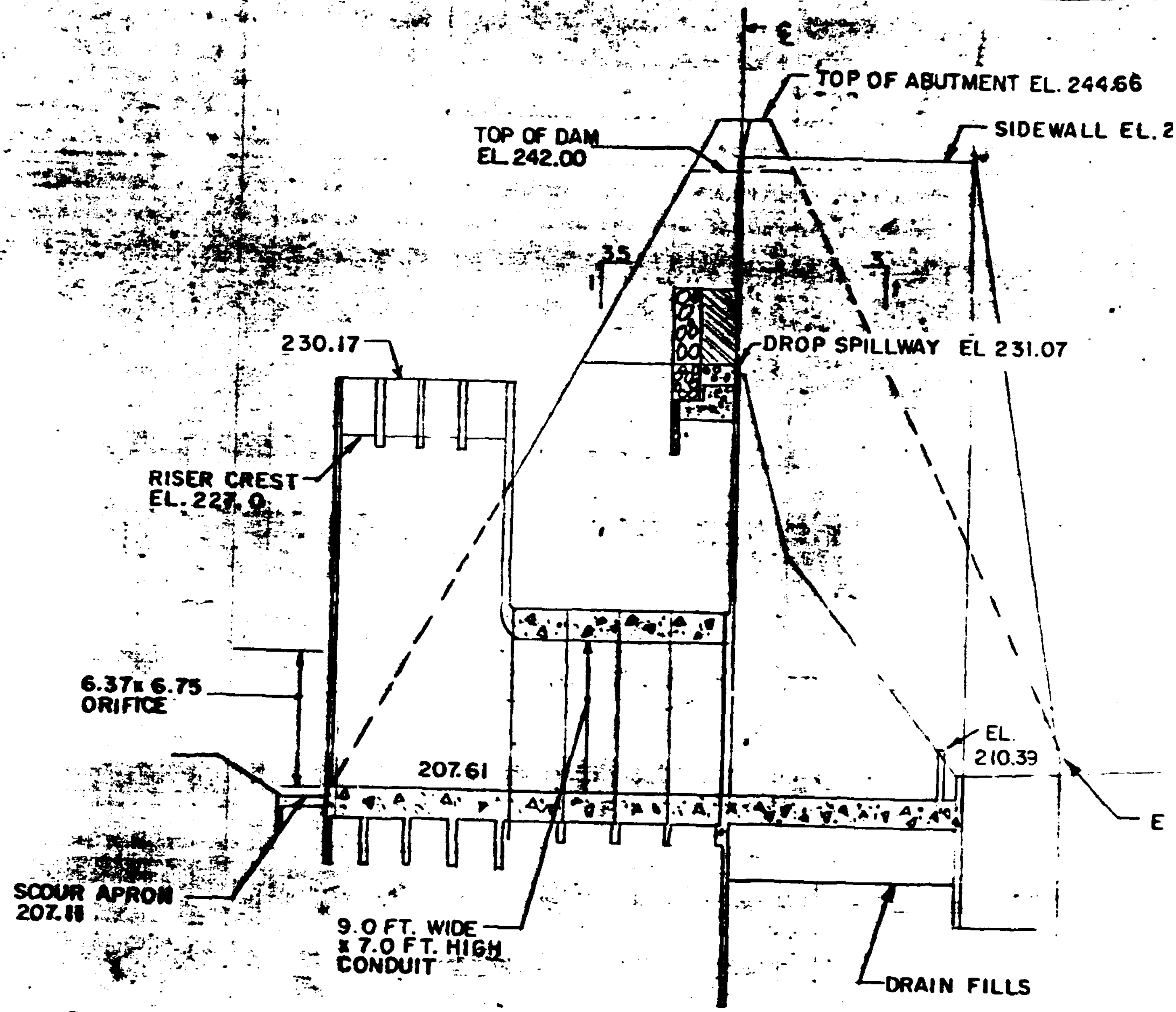
Plans of the reservoir and dam were obtained from the Massachusetts Water Resources Commission, 100 Cambridge Street, Boston, Massachusetts 02202. The original set of drawings are dated 1972 and 1973 and were modified by drawings dated 1974, 1976, 1978, 1979 and 1980.



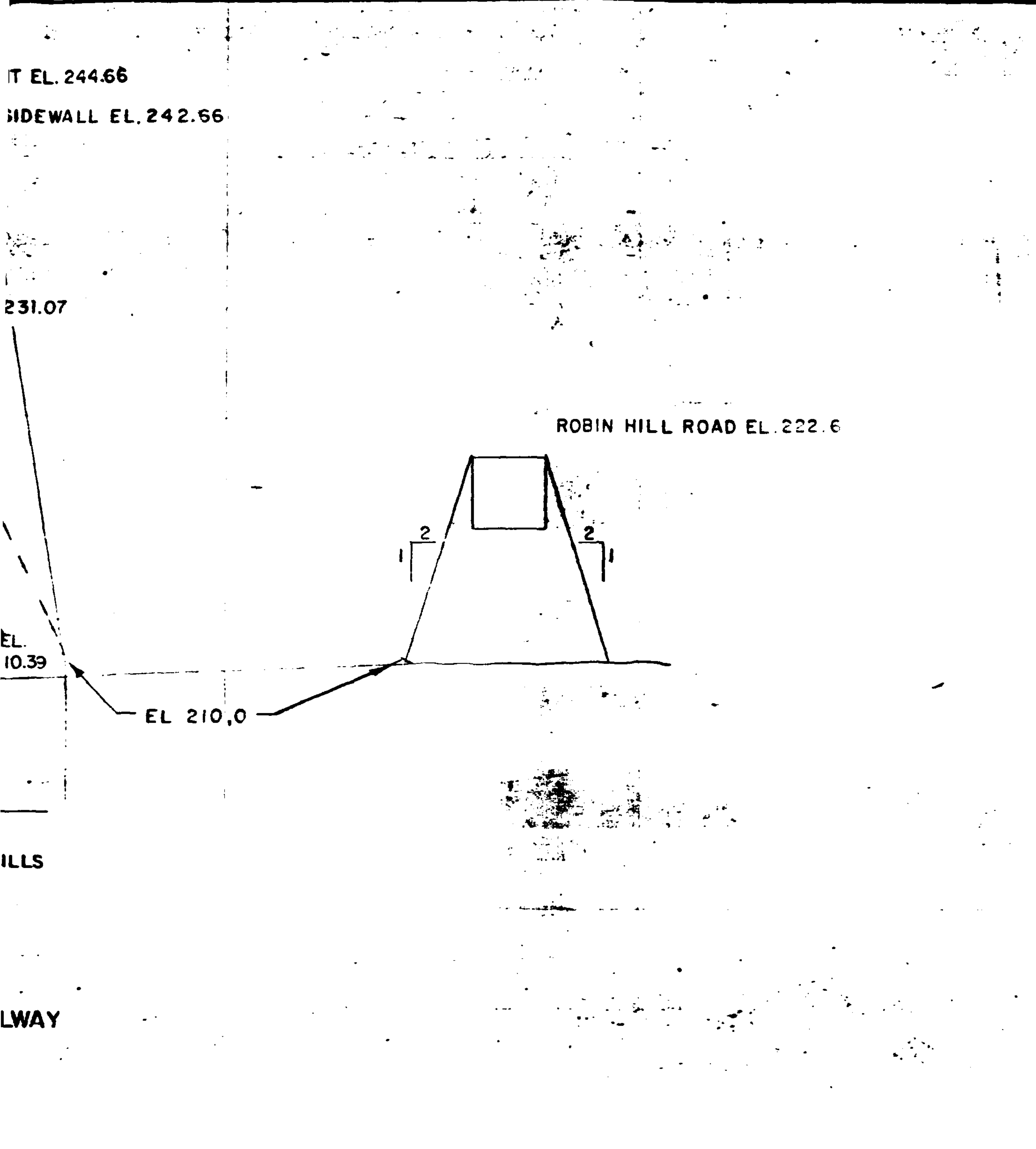
DESIGN HIGH WATER 234.07
ROBIN HILL ROAD
STREET
BOUNDARY
TYLER DAM
1
2
13
3
11
ASSABET RIVER
RISER
PRINCIPAL SPILLWAY
10
DESIGN HIGH WATER 234.07
MILLHAM DAM
MILLHAM RESERVOIR
5
NUMBER AND ORIENTATION OF PHOTOGRAPH IN APPENDIX C



TYLER DAM
STILLING BASIN
RISER
PRINCIPAL SPILLWAY
CONCRETE DROP SPILLWAY
DAM
1
2
3
4
5
6
7
8
9
10
11
12
13
14
NATIONAL PROGRAM OF INSPECTION OF NON FEDERAL DAMS
TYLER DAM PLAN
Marlborough, Massachusetts
Scale 1" = 200'



PROFILE ALONG ℄ PRINCIPAL SPILLWAY



NATIONAL PROGRAM OF INSPECTION OF NON FEDERAL DAMS

TYLER DAM

Marlborough, Massachusetts

Scale
Hor. 1" = 30'
Vert. 1" = 6'

APPENDIX C

SELECTED PHOTOGRAPHS

(The Index to the Photographs in this Appendix is Found in Appendix B)

Photo No. 1 - Upstream slope of dam viewed near road crossing.

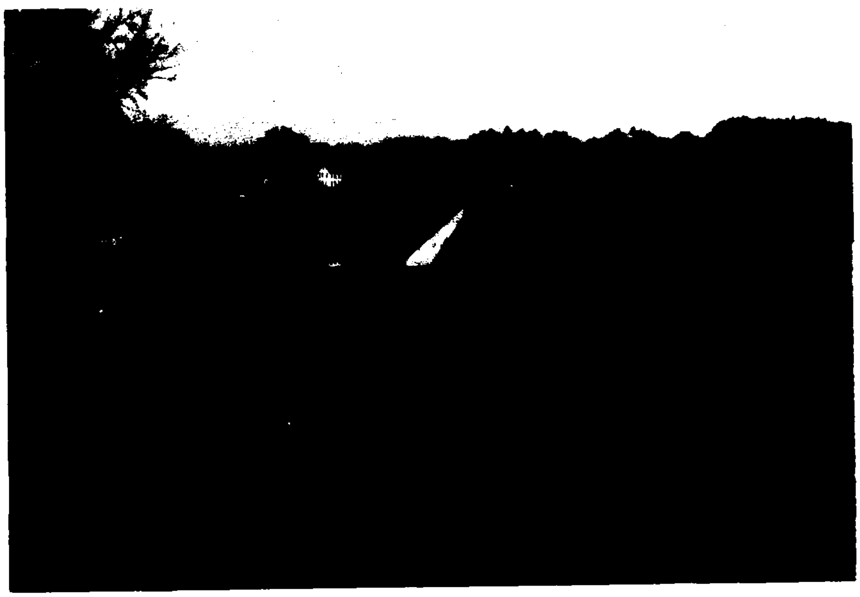


Photo No. 2 - Downstream slope of dam viewed near road crossing.

Photo No. 3 - Crest of dam, Robin Hill Road, and left abutment viewed from left side of spillway structure. Grass been recently planted on crest and netting used for erosion protection is still in place. Grass on crest and both slopes is mowed.

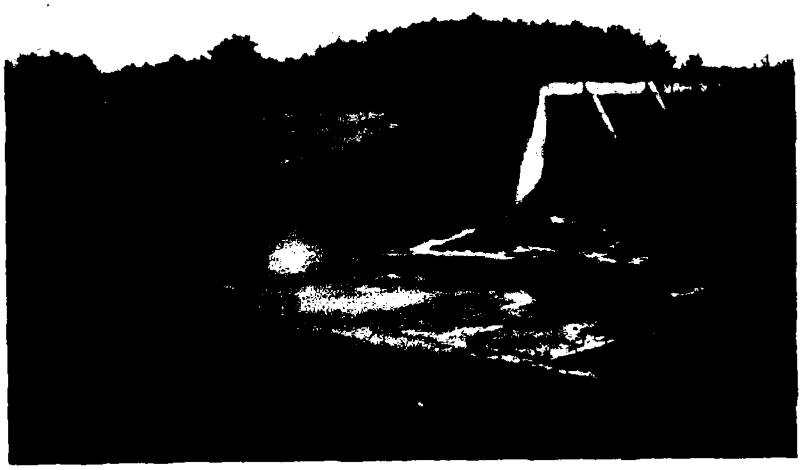


Photo No. 4 - View of stilling basin and concrete spillway structure.

Photo No. 5 - View from crest of embankment at left end of spillway, showing right bank of downstream channel, stilling basin, concrete spillway structure and right abutment of dam.

Photo No. 6 - View of right abutment and spillway; note zone of rockfill on upstream side of concrete weir.

Photo No. 7 - One of several additional sinkholes on upstream edge of coarse rockfill against upstream side of concrete spillway weir.

Photo No. 8 - Fill and riprap behind training wall at right end of spillway structure. (Note: Grass is not yet well established on upstream slope and significant erosion is occuring where motor bike tracks go up slope.)

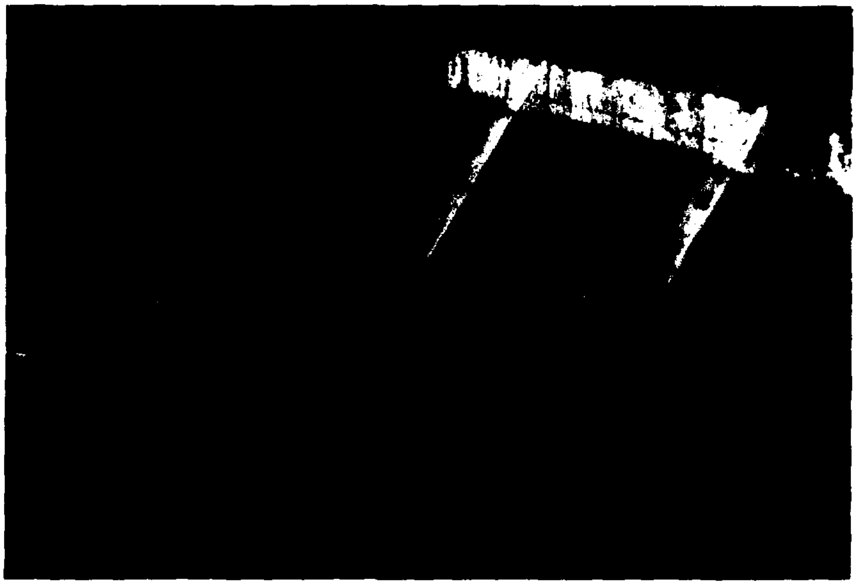


Photo No. 9 - Fill and riprap behind wingwall at left end of spillway.

Photo No. 10 - Drop-inlet spillway structure.

Photo No. 11 - Riser structure at left end of
overflow spillway.

Photo No. 12 - Extensive siltation in drainage channel at toe of dam which carries water from culvert under roadway draining toe of dike. Also erosion on bank of channel which is at toe of dam.

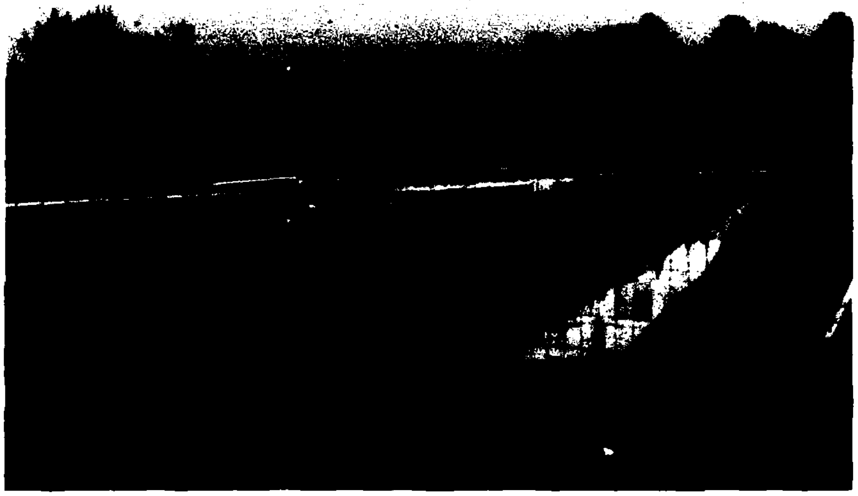


Photo No. 13 - View downstream from crest of dam left of spillway showing Robin Hill Road over Assabet River.

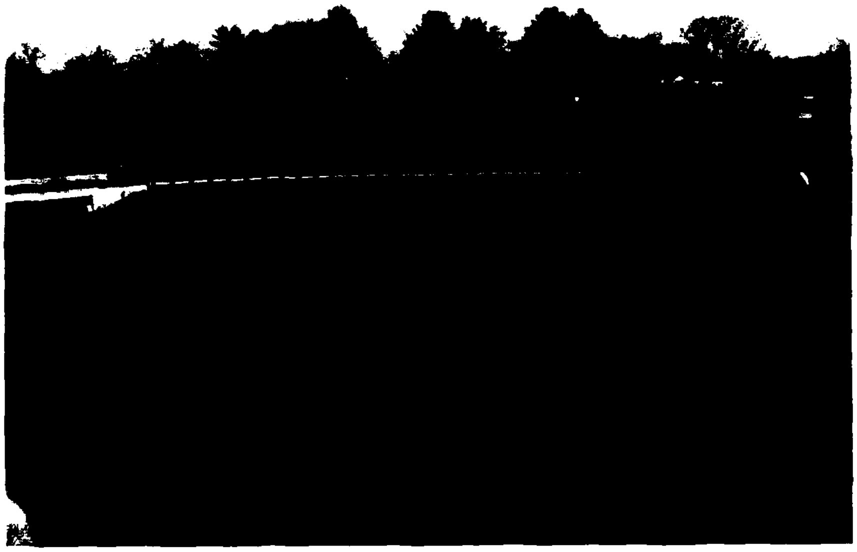
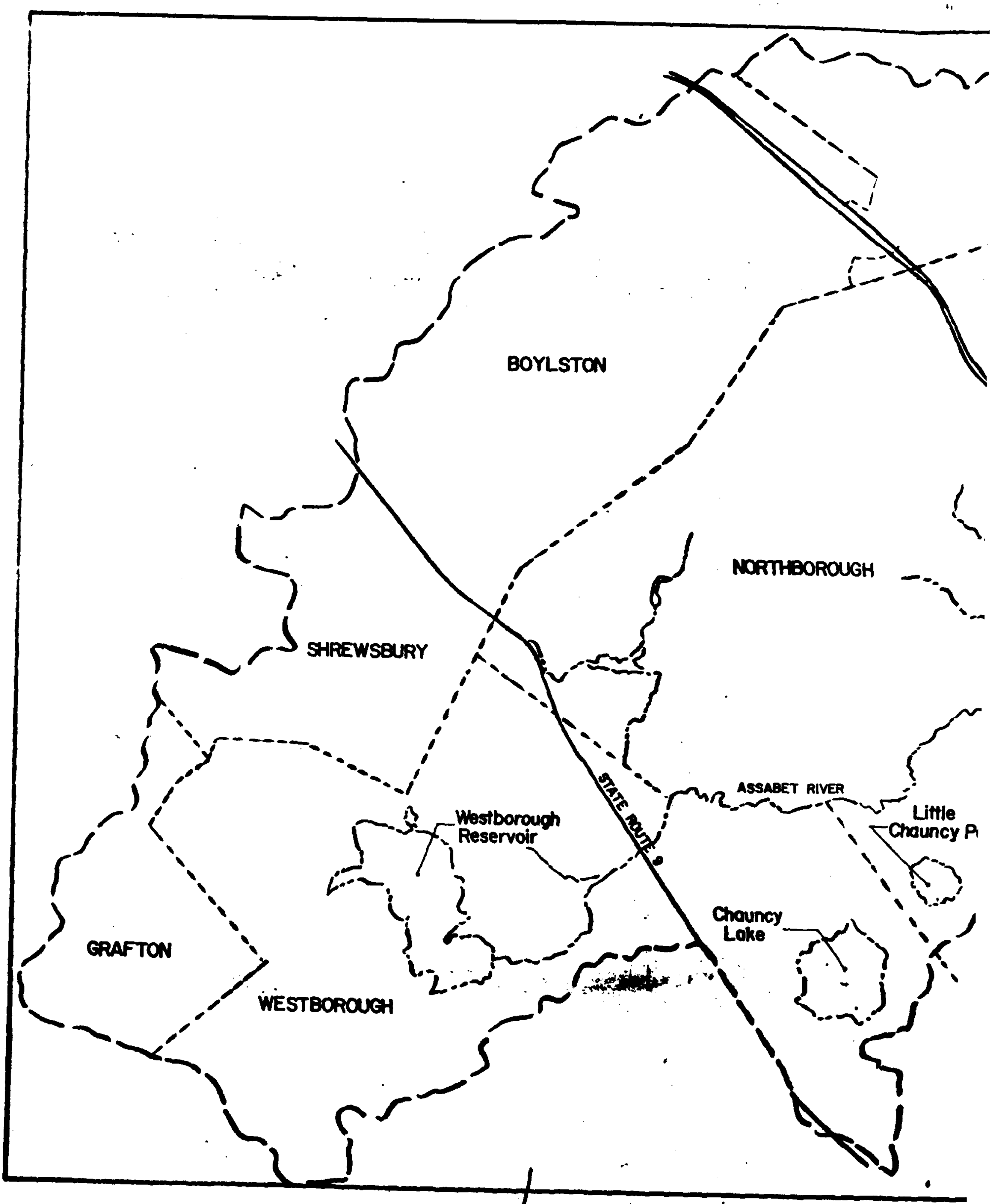
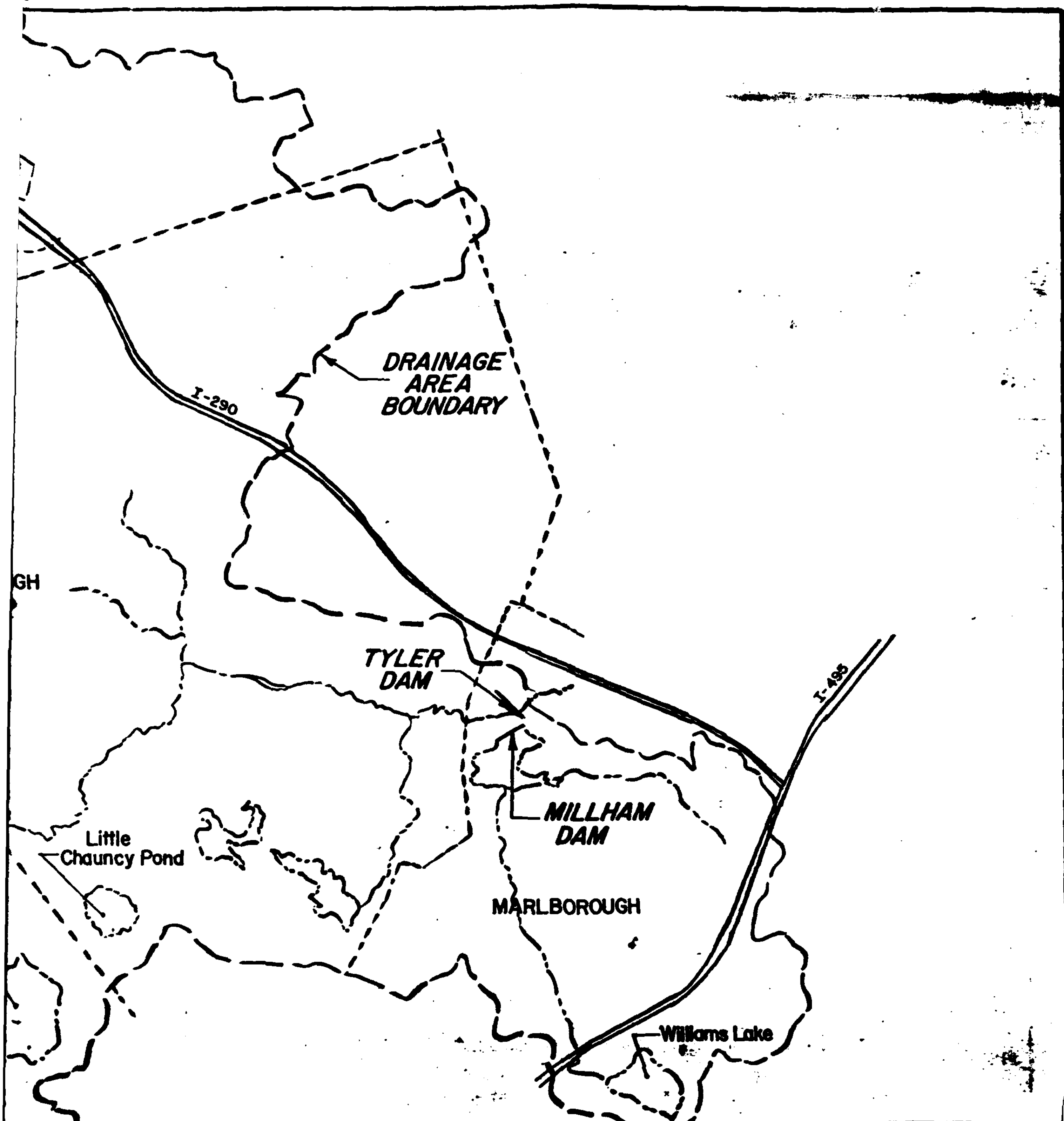


Photo No. 14 - View from crest of embankment at left end of spillway, showing Robin Hill Road crossing downstream channel.

APPENDIX D

HYDROLOGIC AND HYDRAULIC COMPUTATIONS

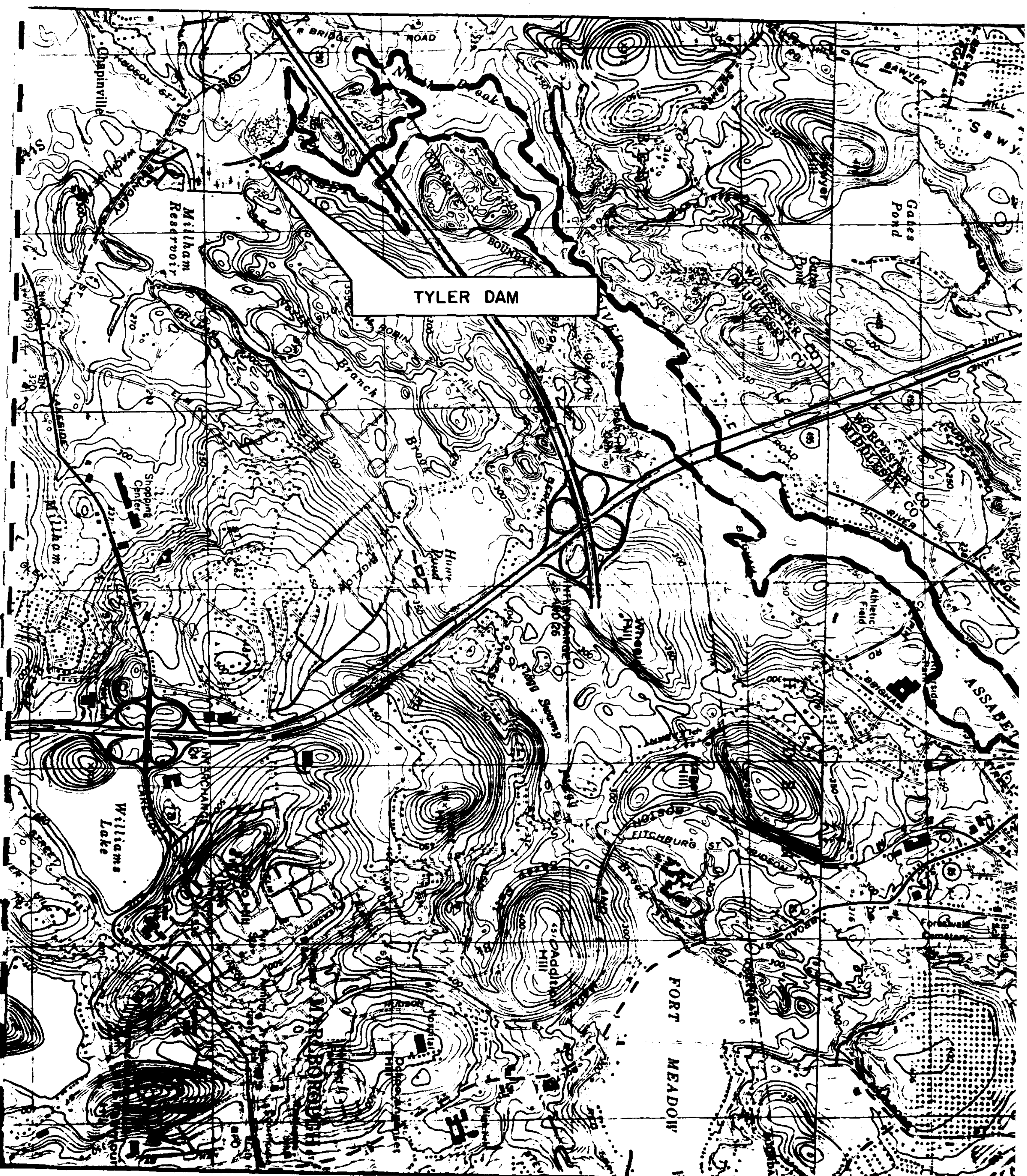

BOYLSTON
NORTHBOROUGH
SHREWSBURY
STATE ROUTE 9
ASSABET RIVER
Westborough Reservoir
Little Chauncy P
Chauncy Lake
GRAFTON
WESTBOROUGH

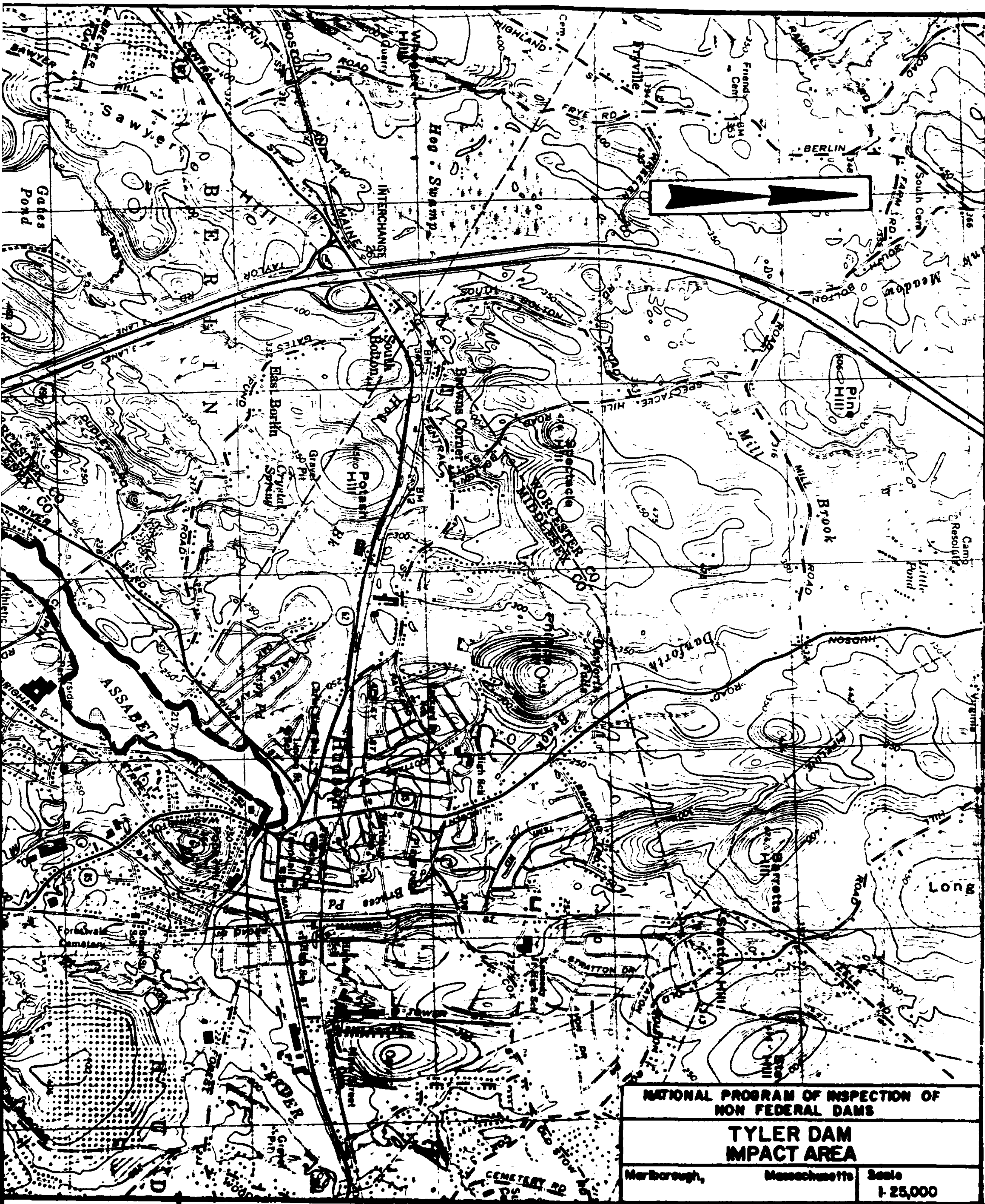


NATIONAL PROGRAM OF INSPECTION OF NON FEDERAL DAMS

TYLER DAM
DRAINAGE AREA

Marlborough, U.S.G.S. Quad.	Massachusetts Marlborough, Mass	Scale 1: 48,000


TYLER DAM
Millham Reservoir
Williams Lake
Gates Pond
Shopping Center
Athletic Field
FORT MEADOW
ASSABET
FITCHBURG ST
BOUNDARY
BRIGHAM
WORCESTER CO
MIDDLESEX CO



NATIONAL PROGRAM OF INSPECTION OF NON FEDERAL DAMS

TYLER DAM
IMPACT AREA

Marlborough, Massachusetts | Scale 1:25,000

SCHOENFELD ASSOCIATES, INC.
Consulting Engineers
210 South Street
BOSTON, MASSACHUSETTS 02111
(617) 423-5541

JOB TYLER DAM
SHEET NO. 1 OF 23
CALCULATED BY G. SHARRY DATE 22 APR 81
CHECKED BY H SHAWITZ DATE APRIL 29, 1981
SCALE

TEST FLOOD ANALYSIS

Choose spillway design flood (SDF)

Classification - Size: Intermediate
Hazard: High

Use probable maximum flood (PMF) as SDF.

Total drainage area = 39.5 mi^2...

controlled DA = 18.4 mi^2
uncontrolled DA = 21.1 mi^2

From PMF guide curves* for uncontrolled DA:

Q_{p_1} = 620 csm or Q_{p_1} = 620(21.1 mi^2) = 13082 cfs

Drainage area of 18.4 mi^2 is controlled by four SCS flood control dams. Three of these dams, Nichols, Hop Brook, and Cold Harbor Brook, discharge directly to waters flowing into the Tyler floodpool.

"Freeboard" routings of these upper sites performed in series with Tyler were supplied by the SCS. Conditions for these routings (rainfall = 30 inches, duration = 24.7 hours) closely resemble PMF conditions.

These hydrographs from the upper sites were combined and a routed peak of 12543 cfs at 17.5 hours was obtained.

*"Flat and Coastal" curve used; uncontrolled DA consists of many swampy areas as well as Millham Reservoir.

SCHOENFELD ASSOCIATES, INC.
Consulting Engineers
210 South Street
BOSTON, MASSACHUSETTS 02111
(617) 423-5541

TEST FLOOD ANALYSIS

The combined hydrograph for the upper sites was then combined with the hydrograph representing conditions over the uncontrolled drainage area. This produced a peak inflow to Tyler Dam of 25100 cfs at 19.75 hours.

$$Q_{p_1} = \underline{\underline{25100}} \text{ cfs}$$

Surcharge Storage Routing

$$Q_{p_2} = Q_{p_1} - Q_{p_1}\left(\frac{STOR}{19}\right)$$

ELEVATION ABOVE NGVD (FT)	SURCHARGE STORAGE* (AC-FT)	STOR (IN)$^{\triangle}$	Q_{p_2} (CFS)
220	250	0.12	24941
222	500	0.24	24783
226	1050	0.50	24439
230	1700	0.81	24030
232	2180	1.03	23739
234	2670	1.27	23422
236	3230	1.53	23079
238	3980	1.89	22603
240	4850	2.30	22062
242	5700	2.71	21520

See surcharge storage routing curve, SH 4/23.

* Normal storage = sediment storage ≈ 15 ac-ft. See SH 5/23.
$^{\triangle}$ Spread over entire drainage area of 39.5 mi^2

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JOB TYLER DAM
SHEET NO. 3 OF 23
CALCULATED BY G. SHARKEY DATE 29 APR 81
CHECKED BY H. SHTROVITZ DATE APRIL 29, 1981
SCALE

TEST FLOOD ANALYSIS

Develop rating curve at Tyler Dam...

Disregard flow through principal spillway riser structure as insignificant at high stages. Tailwater due to I-290 was not considered due to the questionable effect of potential floodwater storage areas located between the dam and I-290. An attempt was made to adjust the "C" value for the emergency spillway weir to account for tailwater effects produced by Robin Hill Road. Adjustment values obtained proved to be negligible. Therefore, use C = 3.1 for emergency spillway, length = 275 feet. Use C = 2.8 and L = 500 feet for flow over earth embankment section.

STAGE ABOVE EMER. SPILLWAY CREST (FT)	Q SPILLWAY (CFS)	Q EMBANK'T (CFS)	Q TOTAL (CFS)
1	853		853
3	4430		4430
5	9531		9531
7	15789		15789
9	23018		23018
10.9	30679		30679
12	35438	1615	37053
13	39959	4260	44219

See rating curve; SH 4/23.

From curve intersection, PMF outflow = <u>22100</u> cfs at elevation 240.0. The PMF would fill the reservoir to within 2 feet of top of dam.

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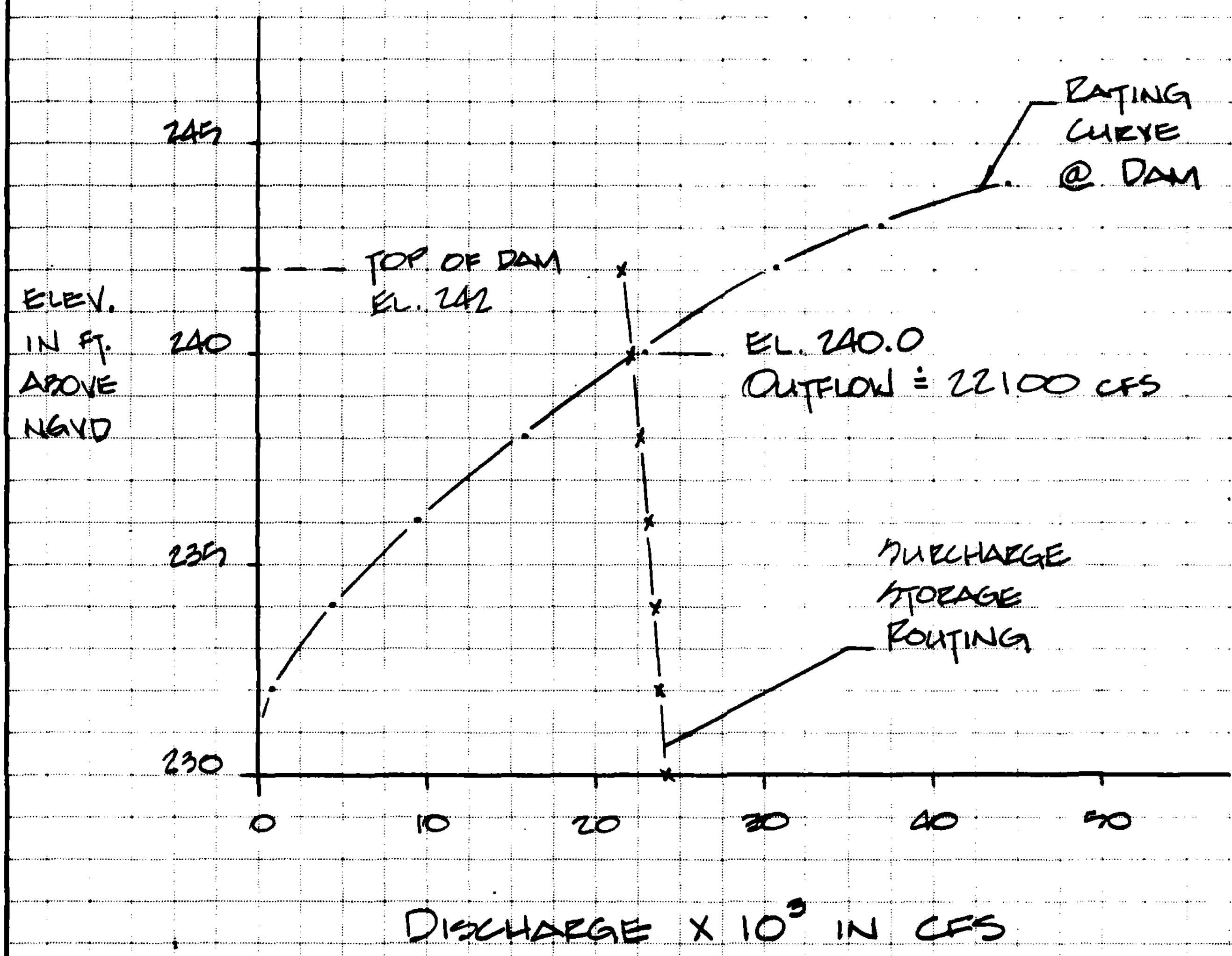
JOB TULER DAM
SHEET NO. 4 OF 23
CALCULATED BY G. SHARRY DATE 21 APR 81
CHECKED BY H. SHMOVITZ DATE APRIL 23, 1981
SCALE

STAGE VS. DISCHARGE*



* NOTE: For principal spillway conduit rating curve, see "attachment A" at end of computations.

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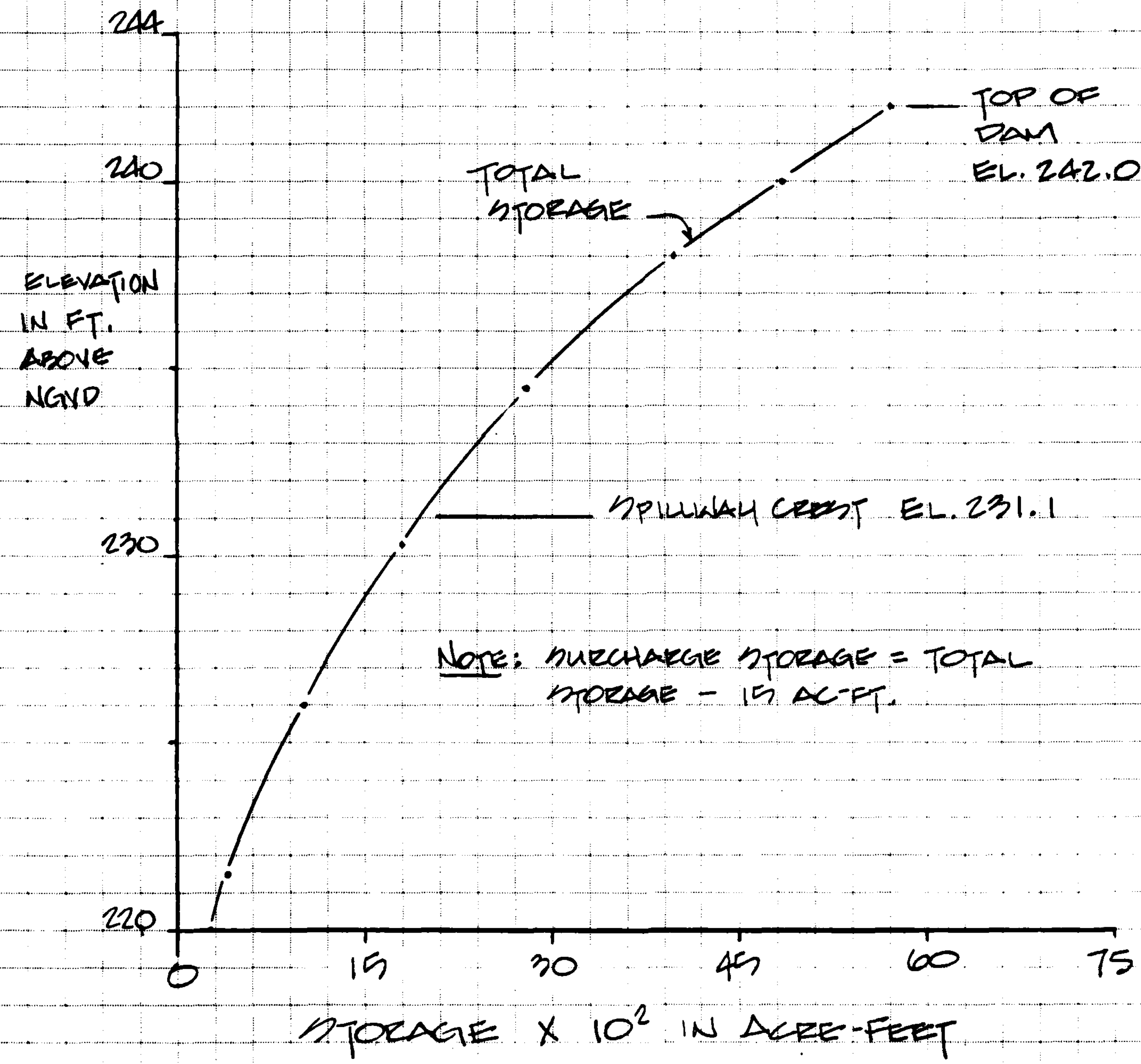
JOB Tyler Dam
SHEET NO. 5 OF 23
CALCULATED BY G. Sharry DATE 13 APR 81
CHECKED BY H. Shmenite DATE April 20, 1981
SCALE

ELEVATION VS. STORAGE*



* From SCS Design Folder, Tyler Dam

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CALCULATED BY G. SHARPY DATE 28APR81
CHECKED BY H. SHMEVITZ DATE APRIL 28, 1981
SCALE

BREACH ANALYSIS

Antecedent Condition

Normally, a water surface at top of dam is assumed prior to breach. However, if this assumption were made at Tyler Dam, a large antecedent stage would result. For instance, at the time of breach just downstream of the dam, actual breach flow would comprise only about 20 % of total outflow. This is due to the large emergency spillway and the effect of tailwater on a breach at Tyler Dam. If this analysis were continued downstream, the increase in flooding damage due to a breach would be negligible when compared to damage already resultant from the antecedent condition. Therefore, a "dry" breach with antecedent stage at emergency spillway crest was selected as the dam failure scenario.

"Dry" Breach

Water surface elevation = 231.0

Tailwater due to Robin Hill Rd., located about 220 feet downstream of dam, will affect breach outflow.

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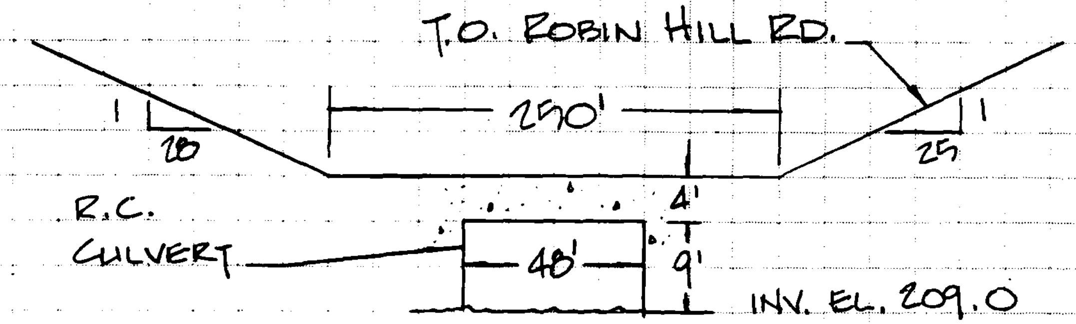
JOB TYLER DAM
SHEET NO. 7 OF 23
CALCULATED BY G. SHARRY DATE 20 APR 81
CHECKED BY H. SHAEVITZ DATE APRIL 28, 1981
SCALE

BREACH ANALYSIS

Develop rating curve at Robin Hill Rd. Roadway and bridge control flow in channel downstream of dam. Use FHA HEC-5 charts to rate culvert flow assuming inlet control. Use Manning equation to rate flow over roadway. Usually, the weir equation would be used here; however, due to topography and vegetation just downstream of Robin Hill Rd., an open channel was chosen as a better representation of actual flow conditions.

$$Q_{ROAD} = \frac{1.49}{n} A R^{2/3} S^{1/2} \qquad n = 0.08 \qquad S = 0.003$$



ELEVATION LOOKING DOWNSTREAM

STAGE ABOVE CHANNEL INV. (FT)	FLOW AREA* (FT²)	WETTED PERIMETER (FT)	Q ROAD* (CFS)	Q CULVERT (CFS)	Q TOTAL (CFS)
6				1728	1728
12				4944	4944
18	1913	515	4680	7104	11784
22	4397	727	14886	8400	23286
26	7729	939	32134	9360	41494
30	11909	1151	57669	10080	67749

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BREACH ANALYSIS

See rating curve, SH 22/23.

Breach $Q = Q_{P_1} = 8/27\, W_b \sqrt{g}\, Y_0^{3/2}$

Use $W_b = 200$ ft. (embankment section to southwest of spillway)

$Y_0 = ?$ Water surface = 231.0
Toe @ spillway = 210.0

Try $Y_0 = 231.0 - 210.0 = 21.0$ ft.

$Q_{P_1} = 8/27 (200) \sqrt{32.2} (21)^{3/2} = 32360$ cfs

From Robin Hill Rd. rating curve, stage = 24.1 ft. = el. 233.1, 233.1 > 231.0, Y_0 too high...

Try $Y_0 = 17$ ft., $Q_{P_1} = 8/27 (200) \sqrt{32.2} (17)^{3/2} = 23570$ cfs

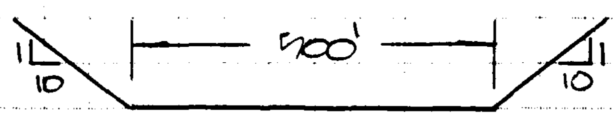
From rating curve, stage = 22.0 ft = el. 231.0
231.0 = spillway crest elevation. Therefore, use $Y_0 = 17$ ft.

"Dry" breach $Q_{P_1} = \underline{\underline{23570}}$ cfs

REACH 1

Downstream limit is Robin Hill Rd. Rating curve was already developed above.

Reach length = 220 ft.



TYPICAL X-SECTION - BACKWATER STORAGE

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JOB: TULER DAM
SHEET NO. 9 OF 23
CALCULATED BY: G. SHARPEY DATE: 28 APR 81
CHECKED BY: H. SHKREVITZ DATE: APRIL 29, 1981
SCALE:

BREACH ANALYSIS

REACH 1 (cont.)

$Q_{p_1} = 23570$ cfs   stage = 22.0 ft.

$V_1 = \text{area}(\text{length}) = \frac{15840(220)}{43560} = 80.0 \text{ ac-ft} < \frac{5700}{2} \therefore \text{OK}$

$Q_{p_2(\text{TRIAL})} = Q_{p_1}\left(1 - \frac{V_1}{S}\right) = 23570\left(1 - \frac{80}{5700}\right) = 23239$ cfs

stage = 22.0 ft. $\therefore V_2 = V_1 = 80.0 \text{ ac-ft} = V_{AVG}$

$Q_{p_2} = Q_{p_1}\left(1 - \frac{V_{AVG}}{S}\right) = 23570\left(1 - \frac{80}{5700}\right) = \underline{\underline{23239}}$ cfs

stage = 22.0 ft.

Robin Hill Rd. would be overtopped by about 9.0 feet. Excessive damage is likely. Loss of life is a remote possibility.

REACH 2

Downstream limit is Interstate 290. Highway embankment acts as a dam. Develop rating curve at I-290. Use FHA HEC-5 charts to rate culvert flow assuming inlet control. Use weir equation, $Q = CLH^{3/2}$, w/ $C = 2.0$ for flow over roadway.

Reach length = 2000 ft.

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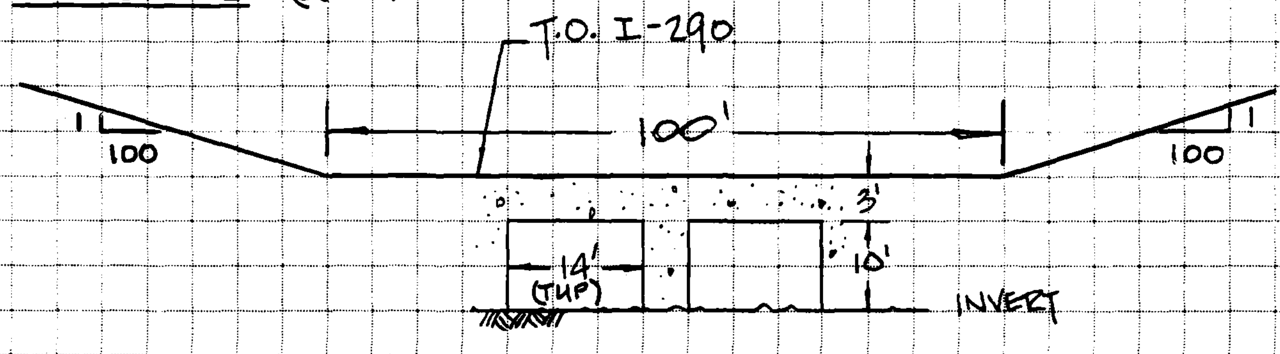
JOB: TYLER DAM
SHEET NO. 10 OF 23
CALCULATED BY: G. SHARPY DATE: 29 APR 81
CHECKED BY: H. SHAEVITZ DATE: APRIL 29, 1981
SCALE:

BREACH ANALYSIS

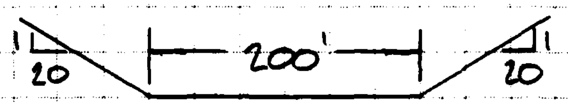
REACH 2 (cont.)



ELEVATION LOOKING DOWNSTREAM

STAGE ABOVE CHANNEL INV (FT)	Q CULVERT (CFS)	Q WEIR (CFS)	Q TOTAL (CFS)
6	1040		1040
12	2940		2940
15	3780	1697	5477
18	4480	13416	17896
18.5	4590	16768	21358
19	4650	20576	25226

See rating curve, SH 22/23.



TYPICAL X-SECTION - BACKWATER STORAGE

$Q_{p_1} = 23239$ cfs

stage = 18.8 ft.

$V_1 = \text{area}(\text{length}) = \frac{10829(2000)}{43560} = 497.2$ ac-ft $< \frac{5700}{2}$ ∴ OK

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BREACH ANALYSIS

REACH 2 (cont.)

$Q_{P2(TRIAL)} = Q_{P1}(1 - \frac{V_1}{S}) = 23239(1 - \frac{497.2}{5700}) = 21212$ cfs

stage = 18.5 ft. $V_2 = \frac{10545(2000)}{43560} = 484.2$ ac-ft

$V_{AVG} = 490.7$ ac-ft

$Q_{P2} = Q_{P1}(1 - \frac{V_{AVG}}{S}) = 23239(1 - \frac{490.7}{5700}) = \underline{\underline{21238}}$ cfs

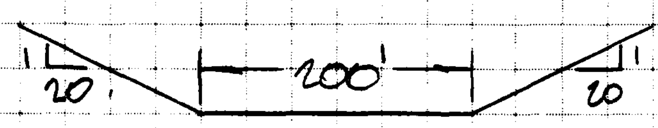
stage = 18.5 ft.

Interstate 290 would be overtopped by about 5.5 ft. Some damage could result and loss of a few lives would be possible.

REACH 3

This reach would act as an open channel with length = 1200 ft. Develop rating curve for reach using the Manning equation...

$Q = \frac{1.49}{n} A R^{2/3} S^{1/2}$ $n = 0.07$ $S = 0.001$



TYPICAL CROSS SECTION

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BREACH ANALYSIS

REACH 3 (cont.)

STAGE ABOVE CHANNEL INV. (FT)	FLOW AREA (FT^2)	WETTED PERIMETER (FT)	Q (CFS)
6	1920	440	3450
12	5280	680	13932
15	7500	800	22438
18	10080	920	33460
19	11020	960	37734
20	12000	1000	42323

See rating curve, SH 22/23.

$Q_{P_1} = 21238$ cfs stage = 14.5 ft.

$V_1 = \text{area}(\text{length}) = \frac{7105(1200)}{43560} = 195.7 \text{ ac-ft} < \frac{5700}{2} \therefore \text{ok}$

$Q_{P_2}(\text{TRIAL}) = Q_{P_1}\left(1 - \frac{V_1}{S}\right) = 21238\left(1 - \frac{195.7}{5700}\right) = 20509$ cfs

stage = 14.2 ft. $V_2 = \frac{6873(1200)}{43560} = 189.3$ ac-ft

$V_{AVG} = 192.5$ ac-ft

$Q_{P_2} = Q_{P_1}\left(1 - \frac{V_{AVG}}{S}\right) = 21238\left(1 - \frac{192.5}{5700}\right) = \underline{\underline{20521}}$ cfs

stage = 14.2 ft.

No damage would be expected along Reach 3.

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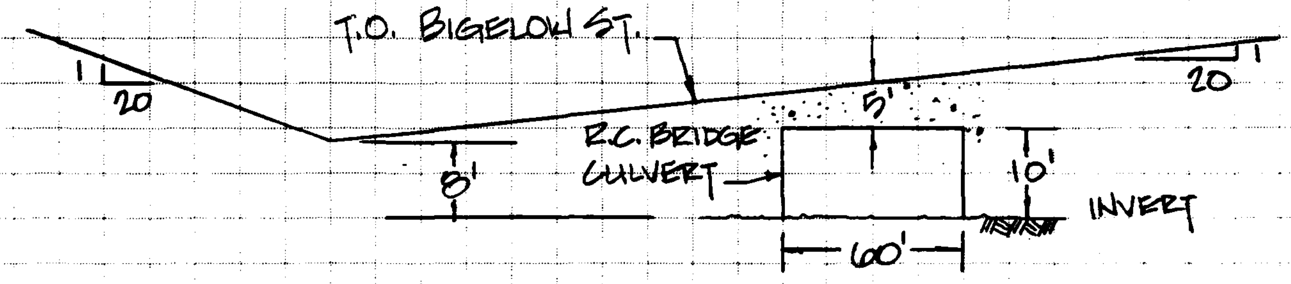
JOB: TYLER DAM

CALCULATED BY: G. SHARRY DATE: 29 APR 81
CHECKED BY: H. SHKEVITZ DATE: APRIL 29, 1981
SCALE:

BREACH ANALYSIS

REACH 4

Downstream limit is Bigelow St. Develop rating curve at bridge as structure will control flow. Use FHA HEC-5 charts to rate flow under bridge assuming inlet control. Use weir equation, $Q = CLH^{3/2}$ w/ $C = 2.5$ to rate flow over road.

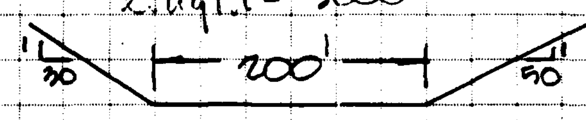


ELEVATION LOOKING DOWNSTREAM

STAGE ABOVE CHANNEL INV. (FT)	Q CULVERT (CFS)	Q WEIR (CFS)	Q TOTAL (CFS)
12	2900	1600	4500
16	8700	9051	17751
18	10000	15811	25811
19	10900	20066	30966
20	11700	24942	36642

See rating curve, SH 22/23.

Length = 3000'



$Q_{p_1} = 20521$ cfs

Stage = 16.7 ft.

TYPICAL X-SECTION - LOOKING UPSTREAM
BACKWATER STORAGE

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JOB TULER DAM

CALCULATED BY G. SHARRY DATE 29 APR 81
CHECKED BY H. SHAEVITZ DATE APRIL 29, 1981
SCALE

BREACH ANALYSIS

REACH 4 (cont.)

$V_1 = \text{area}(\text{length}) = \frac{14496(3000)}{43560} = 998.3 \text{ ac ft} < \frac{5700}{2} \therefore OK$

$Q_{P2(TRIAL)} = Q_{P1}\left(1 - \frac{V_1}{S}\right) = 20521\left(1 - \frac{998.3}{5700}\right) = 16927 \text{ cfs}$

stage = 15.8 ft. $V_2 = \frac{13146(3000)}{43560} = 905.4 \text{ ac-ft}$

$V_{AVG} = 951.9 \text{ ac-ft}$

$Q_{P2} = Q_{P1}\left(1 - \frac{V_{AVG}}{S}\right) = 20521\left(1 - \frac{951.9}{5700}\right) = \underline{\underline{17094}} \text{ cfs}$

stage = 15.8 ft.

The low point on Bigelow St. would be overtopped by about 7.8 feet. One inhabited structure would be subject to about 8 feet of flooding. Appreciable property damage and loss of a few lives are possible.

REACH 5

Downstream limit is I-495. Develop rating curve at highway using Manning equation to rate low flow under bridge. Use orifice equation to rate pressure flow and weir equation, $Q = CLH^{3/2}$, w/ $C = 2.0$ for flow over roadway.

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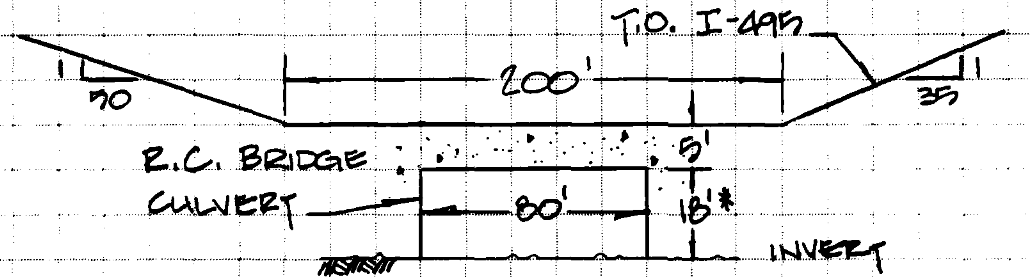
BREACH ANALYSIS

REACH 5 (cont.)

Manning equation: $n = 0.025$ $S = 0.001$

Orifice equation: $Q = CA\sqrt{2g\Delta h}$ (submerged orifice)

$C = 0.6$, Δh = stage − 18'* (orifice is open at median strip)



ELEVATION LOOKING DOWNSTREAM

STAGE ABOVE CHANNEL INV. (FT)	Q CULVERT (CFS)	Q WEIR (CFS)	Q TOTAL (CFS)
5	2038		2038
10	6031		6031
15	11123		11123
20	9806		9806
22	13867		13867
23	15504		15504
24	16984	485	17469
25	18344	1612	19956
26	19611	3403	23014

See rating curve, SH 23/23.

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JOB TYLER DAM
SHEET NO. 16 OF 23
CALCULATED BY G. SHARRY DATE 29 APR 81
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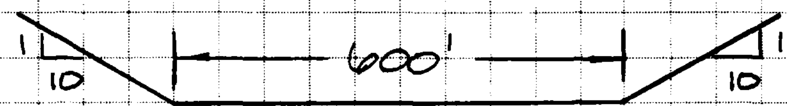
SCALE

BREACH ANALYSIS

REACH 5 (cont)

Reach length = 4600 ft.



TYPICAL X-SECTION - BACKWATER STORAGE

$Q_{P_1} = 17094$ cfs stage = 23.8 ft.

$V_1 = \text{area}(\text{length}) = \frac{19944(4600)}{43560} = 2106.1$ ac-ft $< \frac{5700}{2}$ ∴ OK

$Q_{P_2(\text{TRIAL})} = Q_{P_1}\left(1 - \frac{V_1}{S}\right) = 17094\left(1 - \frac{2106.1}{5700}\right) = 10778$ cfs

stage = 14.9 ft. $V_2 = \frac{11160(4600)}{43560} = 1178.5$ ac ft

$V_{AVG} = 1642.3$ ac ft

$Q_{P_2} = Q_{P_1}\left(1 - \frac{V_{AVG}}{S}\right) = 17094\left(1 - \frac{1642.3}{5700}\right) = \underline{\underline{12169}}$ cfs

stage = 21.2 ft.

The bridge culvert below I-495 would handle the breach flow without occurrence of roadway overtopping. Water would rise to within 1.8 feet of the top of road. One inhabited structure on Bigelow St. south of the I-495 crossing would be subject to less than 2 feet of flooding.

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BREACH ANALYSIS

REACH 6

Downstream limit is Chapin Rd. Develop rating curve at bridge. Use FHA HEC-5 charts to rate culvert flow. Use weir equation, $Q = CLH^{3/2}$ w/ $2.5 = C$, to rate flow over roadway.

ELEVATION LOOKING DOWNSTREAM

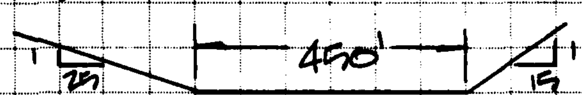
STAGE ABOVE CHANNEL INV (FT)	Q CULVERT (CFS)	Q WEIR (CFS)	Q TOTAL (CFS)
6	962		962
12	2496		2496
15	3016	1949	4965
16.5	3328	5370	8698
18	3576	11023	14599
19	3692	16205	19897
20	3848	22627	26475

See rating curve, SH 23/23.

$Q_{P_1} = 12169$ cfs

stage = 17.5 ft.

length = 4500 ft



TYPICAL X-SECTION BACKWATER STORAGE LOOKING UPSTREAM

REACH 6 (cont.)

V_1 = area (length) = $\frac{14000(4500)}{43560}$ = 1446.3 ac ft < $\frac{5700}{2}$ ∴ OK

$Q_{P2\,(TRIAL)} = Q_{P1}\left(1 - \frac{V_1}{S}\right) = 12169\left(1 - \frac{1446.3}{5700}\right) = 9081$ cfs

stage = 18.3 ft. $V_2 = \frac{12981(4500)}{43560}$ = 1341.0 ac-ft

V_{AVG} = 1393.7 ac-ft

$Q_{P2} = Q_{P1}\left(1 - \frac{V_{AVG}}{S}\right) = 12169\left(1 - \frac{1393.7}{5700}\right) = \underline{\underline{9194}}$ cfs

stage = 16.7 ft.

Chapin Rd. would be overtopped by about 4.7 feet. Several residences located on the west bank of the river would be subject to about 4.5 feet of backwater flooding. Excessive property damage and loss of a few lives would be possible.

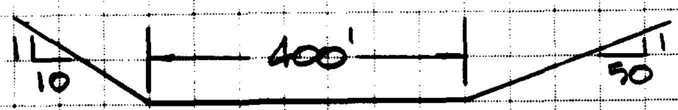
REACH 7

Downstream limit is footbridge at Riverside Park. Footbridge does little to back up flow. Use Manning equation to develop rating curve for reach...

$n = 0.06$

$S = 0.001$

length = 3500 ft.



TYPICAL CROSS SECTION
LOOKING DOWNSTREAM

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JOB TULER DAM

CALCULATED BY G. SHARRY DATE 21 APR 81
CHECKED BY H. SHKOVITZ DATE APRIL 20, 1981
SCALE

BREACH ANALYSIS

REACH 7 (cont.)

STAGE ABOVE CHANNEL INV (FT)	FLOW AREA (FT²)	WETTED PERIMETER (FT)	Q (CFS)
3	1470	580	2145
5	2750	720	5374
7	4270	820	10069
8	5120	880	13000
9	6030	940	16340

See rating curve, SH 23/23.

$Q_{P_1} = 9194$ cfs stage = 6.8 feet

$V_1 = \text{area(length)} = \frac{4107(3500)}{43560} = 330.0$ ac-ft $< \frac{5700}{2}$ ∴ OK

$Q_{P_2}(\text{TRIAL}) = Q_{P_1}\left(1-\frac{V_1}{S}\right) = 9194\left(1-\frac{330}{5700}\right) = 8662$ cfs

stage = 6.4 ft. $V_2 = \frac{3789(3500)}{43560} = 304.4$ ac-ft

$V_{AVG} = 317.2$ ac-ft

$Q_{P_2} = Q_{P_1}\left(1-\frac{V_{AVG}}{S}\right) = 9194\left(1-\frac{317.2}{5700}\right) = \underline{\underline{8682}}$ cfs

stage = 6.5 ft.

Riverside Park would be inundated by backwater up to 6 feet deep. Minor property damage would probably occur.

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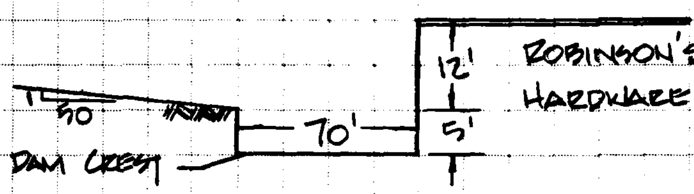
BREACH ANALYSIS

REACH 8

Downstream limit is dam located about 55 feet upstream of Washington St. Dam controls flow throughout reach. Robinson's Hardware is immediately adjacent to the dam causing extreme flow constriction. Use weir equation to rate flow over dam; $Q = CLH^{3/2}$, $C = 3.0$ for dam, $C = 2.0$ for flow over parking lots and Washington St. bridge. When stage is more than 6 ft. above dam crest, use $C = 2.0$ for all weirs.

STAGE ABOVE SPILLWAY CREST (FT)	Q (CFS)
2	594
6	3136
10	7222
11	9517
12	12302
13	15613



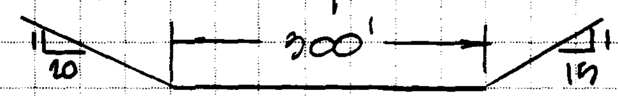
ELEVATION LOOKING DOWNSTREAM

See rating curve, SH 23/23.

$Q_{P1} = 8682$ cfs

Stage = 10.8 ft.

Reach length = 2600'



TYPICAL BACKWATER STORAGE X-SECTION LOOKING UPSTREAM

$V_1 = \text{area}(\text{length}) = \frac{5281(2600)}{43560} = 315.2 \text{ ac-ft} < \frac{5700}{2}$ ∴ OK

$Q_{P2(TRIAL)} = Q_{P1}\left(1 - \frac{V_1}{S}\right) = 8682\left(1 - \frac{315.2}{5700}\right) = 8202$ cfs

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BREACH ANALYSIS

REACH 8 (cont.)

$Q_{P2\,(TRIAL)} = 8202$ cfs stage = 10.7 ft.

$V_2 = \frac{5214(2600)}{43560} = 311.2$ ac ft $V_{AVG} = 313.2$ ac-ft

$Q_{P2} = Q_{P1}\left(1 - \frac{V_{AVG}}{S}\right) = 8682\left(1 - \frac{313.2}{5700}\right) = \underline{\underline{8205}}$ cfs

stage = 10.7 ft.

Washington St. would be overtopped by about 5.7 ft. Upstream of Washington St., Robinson's Hardware and a Texaco gas station would be subject to about 4 feet of flooding. Broad's Ford and a Mobil gas station, both located just downstream of Washington Street, would be inundated by up to 5.5 feet of water. Excessive property damage and loss of more than a few lives would probably result.

Accordingly, Tyler Dam has been classified as <u>High</u> Hazard.

NOTE:

Antecedent flow through principal spillway conduit $\approx$ 1400 cfs, or about 6 % of breach outflow. Therefore, assume stages computed throughout breach analysis are increases in water surface elevation due to breach.

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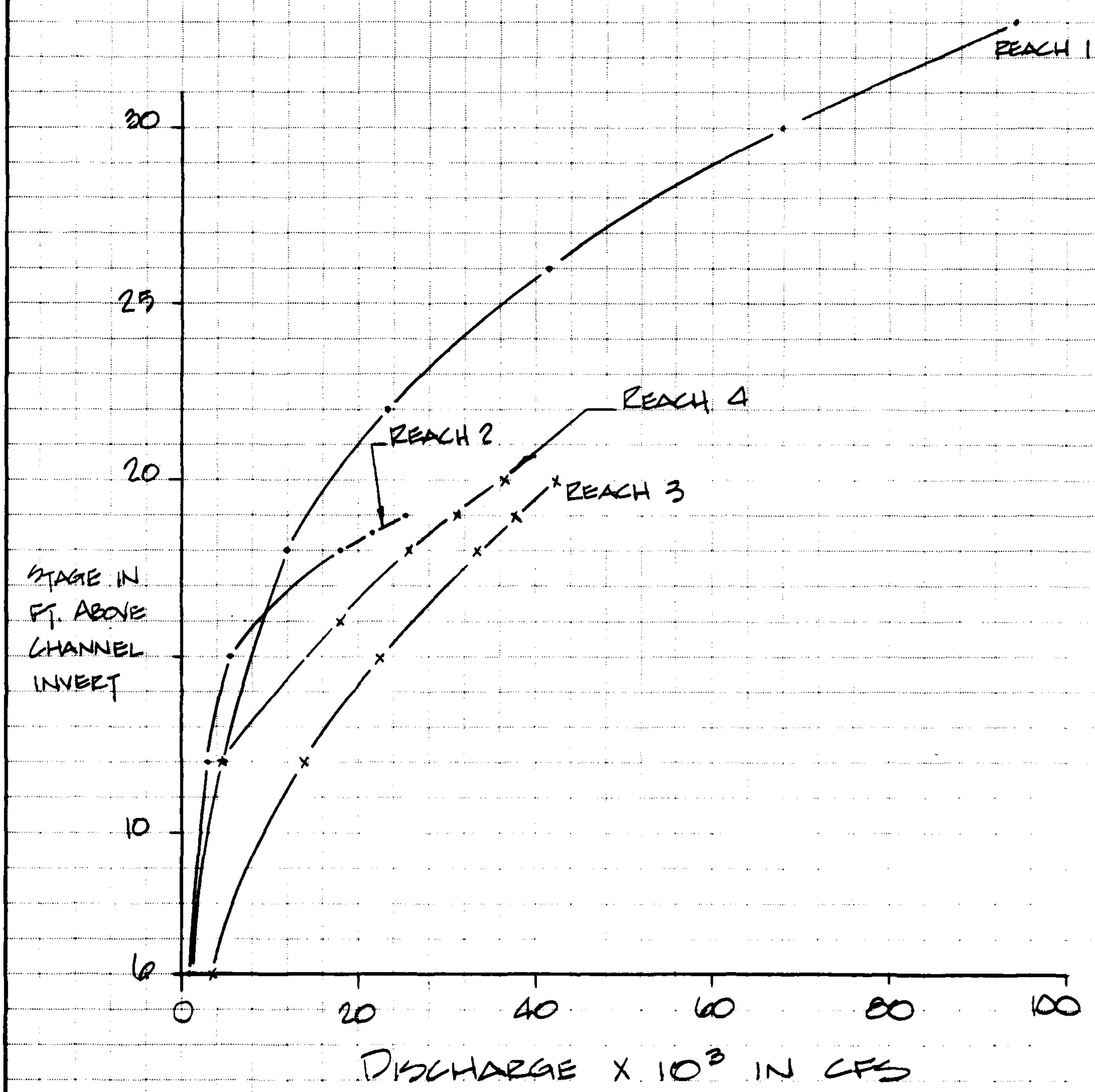
JOB TULER DAM
SHEET NO. 22 OF 23
CALCULATED BY G. SHARRY DATE 13 APR 81
CHECKED BY [illegible] DATE APRIL 29, 1981
SCALE

DOWNSTREAM HAZARD - REACH RATING CURVES



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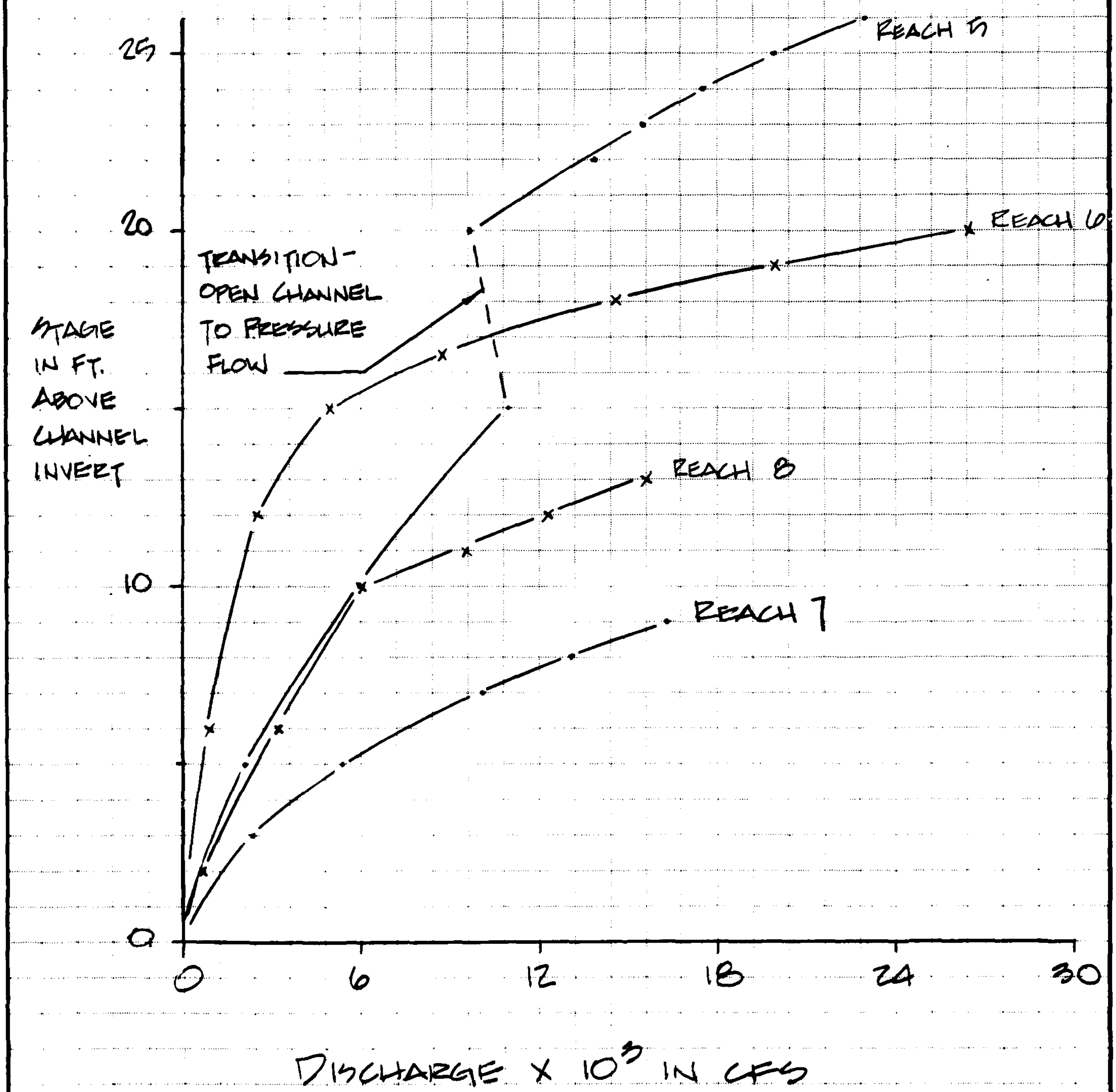
BOSTON, MASSACHUSETTS 02111
(617) 423-5541

JOB TYLER DAM
SHEET NO. 23 OF 23
CALCULATED BY G. SHARRY DATE 16APR81
CHECKED BY H. SHMEVITZ DATE H. SHMEVITZ
SCALE

DOWNSTREAM HAZARD-REACH RATING CURVES



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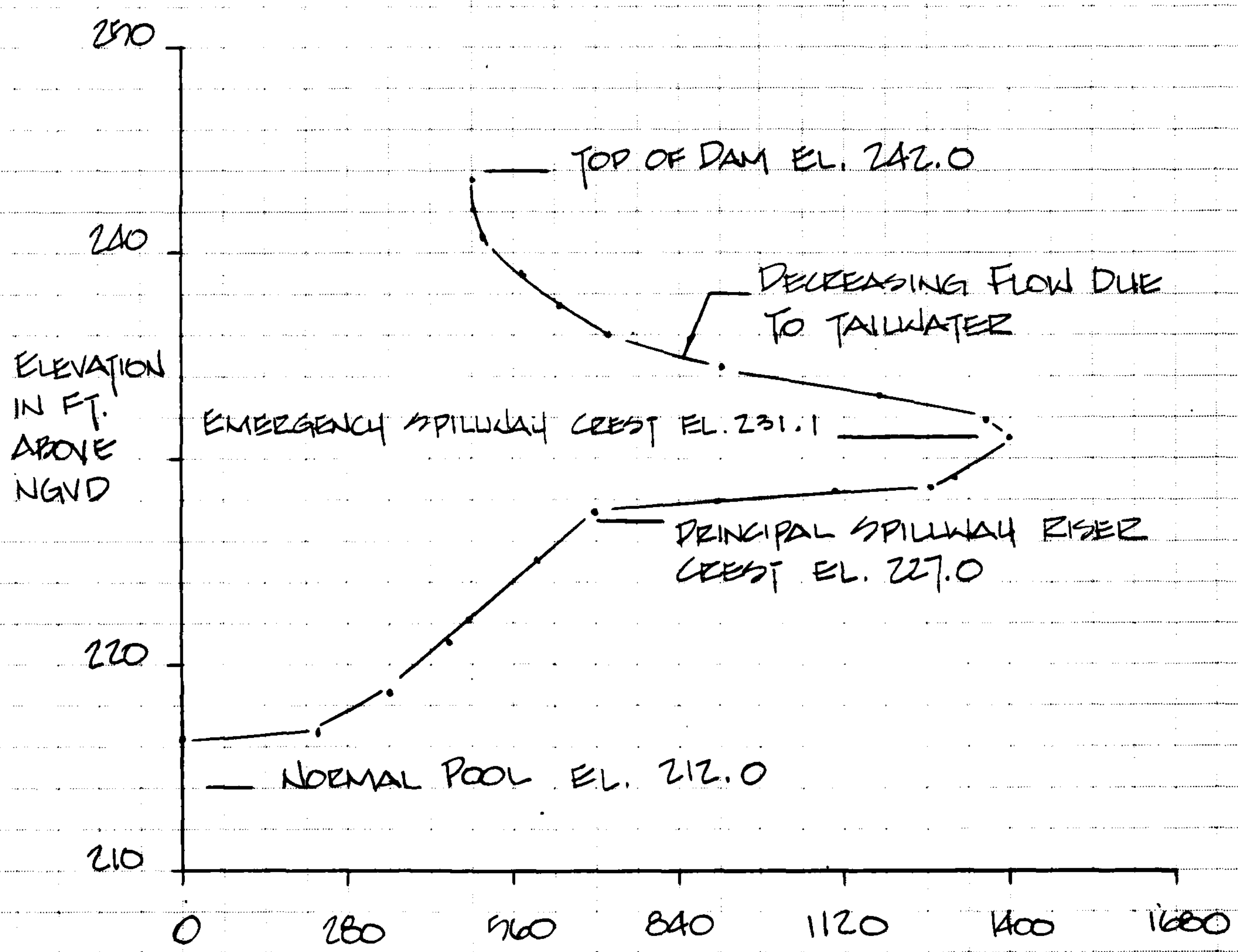
(617) 423-5541

JOB TYLER DAM
SHEET NO. ATTACHMENT A OF
CALCULATED BY G. SHARRY DATE 21 MAY 81
CHECKED BY DATE
SCALE

PRINCIPAL SPILLWAY CONDUIT
DISCHARGE RATING CURVE*



* From design data supplied by the SCS, Amherst, Mass.

APPENDIX E

INFORMATION AS CONTAINED IN THE NATIONAL INVENTORY OF DAMS

NOT AVAILABLE AT THIS TIME

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