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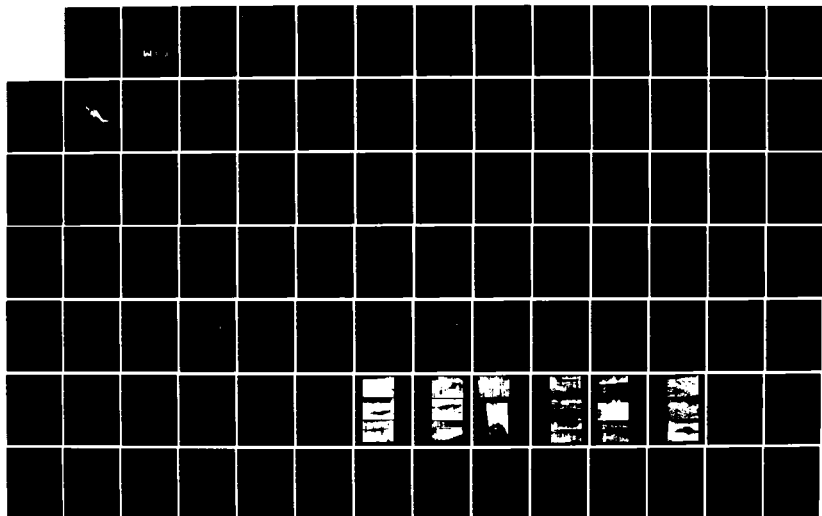
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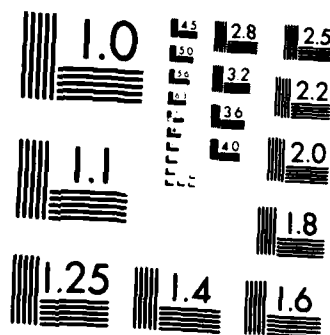
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HOUSATONIC RIVER BASIN
MONTEREY, MASSACHUSETTS

LAKE GARFIELD DAM
MA 00249

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM



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JUN 10 1985
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DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
WALTHAM, MASS. 02154

MARCH 1980

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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
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19. KEY WORDS (Continue on reverse side if necessary and identify by block number) DAMS, INSPECTION, DAM SAFETY, Housatonic River Basin Monterey, Massachusetts Konkapot River (Tributary to Housatonic River)		
20. ABSTRACT (Continue on reverse side if necessary and identify by block number) The dam is an earthfill embankment about 350 in length, and 19.5 feet in height. The overall dam condition assessment has been rated as good. The size is intermediate and the hazard classification is high. There are recommended remedial measures for things such as erosion protection in the report.		



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

REPLY TO
ATTENTION OF:

NEDED

JAN 06 1981

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

Dear Governor King:

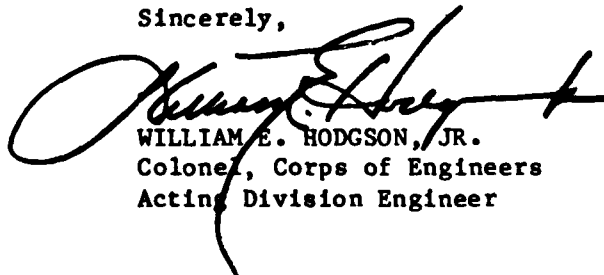
Inclosed is a copy of the Lake Garfield Dam (MA-00249) Phase I Inspection Report, which was prepared under the National Program for Inspection of Non-Federal Dams. This report is presented for your use and is based upon a visual inspection, a review of the past performance and a brief hydrological study 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 Quality Engineering, the cooperating agency for the Commonwealth of Massachusetts. In addition, a copy of the report has also been furnished the owner, Town of Monterey, Mass.

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 Quality Engineering for your cooperation in carrying out this program.

Sincerely,



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

Incl
As stated

LAKE GARFIELD DAM

MA 00249

HOUSATONIC RIVER BASIN
MONTEREY, MASSACHUSETTS

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

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NATIONAL DAM INSPECTION PROGRAM
PHASE I INSPECTION REPORT

Identification No.: MA 00249
Mass. D.P.W. No.: 1-2-193-6
Name of Dam: Lake Garfield Dam
Town: Monterey
County and State: Berkshire County, Massachusetts
Stream: Konkapot River (Tributary to Housatonic River)
Date of Inspection: November 14, 1979

BRIEF ASSESSMENT

The Lake Garfield Dam, No. MA 00249, is located on the headwaters of the Konkapot River a tributary to the Housatonic River. The dam is located in the Town of Monterey, Massachusetts. The dam site is approximately 0.85 miles upstream of the Village of Monterey and is located off of Beartown Mountain Road. The dam is a multiple purpose recreation and flood protection facility which is owned by the Town of Monterey and is under the responsibility of the Board of Selectmen. It was designed by the consulting firm of R.G. Brown & Associates of Pittsfield, Massachusetts and constructed by the firm of Petricca Construction Company of Pittsfield, Massachusetts. Supervision of construction was by the Commonwealth of Massachusetts, Department of Public Works, Division of Waterways, District No. 1. The dam was completed in 1973. The dam is an earthfill embankment about 350 feet in length, and 19.5 feet in height, and has a reinforced concrete principal spillway which maintains the recreation pool level and controls the release of stored floodwater. The facility has a 130 foot wide earth excavated emergency spillway channel around the left abutment. The dam impounds approximately 3660 acre feet at top of dam elevation 1295.08 NGVD.

The dam and appurtenances were found to be in good condition, and the emergency spillway channel was found to be in fair condition due to sloughing of the left excavated embankment, and inadequate erosion protection of the right training embankment. The condition of the emergency spillway is not considered to be critical and does not affect the floodwater discharge characteristics. Therefore the overall dam condition assessment has been rated as GOOD. Some maintenance and minor remedial work is required as listed in Section 7.

The test flood for this dam has been determined to be the Probable Maximum Flood, based on a classification of Intermediate size and HIGH hazard. The drainage area is approximately 4.0 square miles and the test flood inflow (PMF) is 9,200 CFS. Routing the test flood through the reservoir, with the initial pool level at the principal spillway overflow weir crest, results in a test flood outflow of 4,450 CFS from the dam at a pond stage of 1293.4 ft. NGVD. Lake Garfield Dam has a combined spillway capacity of 6,800 CFS which allows a remaining free board of 1.7 feet to the top of the dam with the test flood flow. The spillway capacity is 152% of the routed test flood outflow from the reservoir.

Prior to the assumed breach with the flood pool at the test flood elevation of 1,293.4 ft. NGVD, there is a threat to approximately 18 houses, 1 store, and 18 culverts. Failure of the dam would pose a serious threat to approximately 1 additional house, plus 17 of the houses flooded by pre-failure flows, and most of the culverts. The effects of the dam failure, therefore, add significantly to the damage anticipated and indicate the HIGH hazard classification is appropriate.

The recommended remedial measures as listed in Section 7 including the additional erosion protection along the right training embankment of the emergency spillway, maintenance of the downstream embankment, and clearing of vegetation from the downstream channel, should be implemented within two years of receipt of this report by the Owner.



John W. Powers
Massachusetts Registration 23106

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

Aramast Mahtesian

ARAMAST MAHTESIAN, MEMBER
Geotechnical Engineering Branch
Engineering Division

Carney M. Terzian

CARNEY M. TERZIAN, MEMBER
Design Branch
Engineering Division

Richard J. DiBuono

RICHARD DIBUONO, 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 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 "Probably Maximum Flood" for the region (greatest reasonably possible storm runoff), or fractions thereof. Because of the magnitude and rarity of such a storm event, a finding that a spillway will not pass the test flood should not be interpreted as necessarily posing a highly inadequate condition. The test flood provides a measure of relative spillway capacity and serves as an aide in determining the need for more detailed hydrologic and hydraulic studies, considering the size of the dam, its general condition, and the downstream damage potential.

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

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SECTION 3 - VISUAL INSPECTION

3.1 Findings

(a) General

The Lake Garfield Dam, No. MA 00249 was in GOOD condition at the time of the inspection.

(b) Dam

1) Earth Embankment (See photos 1, 3 & 4)

The upstream embankment, top of the dam, and downstream embankment were all found to be in very good condition. No apparent movement, sloughing, slides or settlement was visible. The washed gravel surface of the upstream embankment was in good condition and extends up the embankment to an elevation of approximately 2 feet above the recreation pool level.

The grass growth areas, which cover all portions of the dam embankment except for the upstream face below elevation 1288.33, were found to be well groomed and well maintained. The grass growth was very well developed with a healthy, thick growth being found on all areas except portions of the lower downstream embankment where some broadleaf weed growth occurs. There was one small area of damaged grass area on the downstream embankment to the right of the principal spillway conduit outlet and just below the horizontal berm section. This damaged area appears to be due to pedestrian type traffic on the embankment. Also, in this same area was an animal burrow approximately 10 inches in diameter and 6 to 8 inches deep.

At the time of the inspection, the pond was drained. The downstream toe was found to be slightly wet along most of the right side of the dam. This appears to be due to surface drainage and the impervious soil conditions. Rainfall did occur during the night and morning before the inspection. There was no visible seepage throughout the entire length of the downstream toe, with the pond being drained.

The foundation drain outlets were found to be submerged by the stilling basin water level, therefore, a determination of water flow from the drains could not be made.

2) Emergency Spillway (See photos 11, 12, 13 & 14)

The emergency spillway channel is in fair condition. The floor of the approach channel was found to be very wet, with standing water 1 to 2 inches deep. Most of the approach channel was covered with a heavy growth of grass and legumes, however, some reed growth was found.

SECTION 2 - ENGINEERING DATA

2.1 Design Data

The design data for the Lake Garfield Dam includes hydrology and hydraulic computations, soil mechanics, structural designs, survey computations, and quantity computations. The design data is in the files of R.G. Brown Associates, Inc. of Pittsfield, Massachusetts, who were the design consultants for the project. The design of the dam and appurtenances was completed during 1970 and 1971.

This design data was reviewed and found to be based on sound engineering practice.

2.2 Construction Data

"As Built" record drawings are not available for the Lake Garfield Dam. Discussions with the Commonwealth of Massachusetts Department of Public Works, who were responsible for the construction supervision, indicated that except for the 4 inch change in elevation as discussed in Section 1.2, paragraph h, the dam was constructed essentially in accordance with the design drawings. The design drawings have been reviewed and found to show good agreement with the visual inspection.

Appendix B contains copies of the design drawings. These copies have been made from a set of design drawings provided by the design consultants on behalf of the Town of Monterey Board of Selectmen.

2.3 Operational Data

The dam is self-regulating, therefore, no operational data is available. Under normal conditions the hydraulics of the principal spillway maintain a recreation pool. The impoundment may be lowered via a sluice gate controlled pond drain conduit located at the dam and an ungated pond drain conduit located at Tyringham Road. The pipes invert elevations are 1277.5 and 1277.6 ft. NGVD, respectively.

2.4 Evaluation of Data

(a) Availability

Sufficient data is available to permit an evaluation of the dam when combined with findings of the visual inspection.

(b) Adequacy

There is sufficient design and construction data to permit an assessment of dam safety when combined with the visual inspection, past performance, and sound engineering judgment.

(c) Validity

Since the observations of the inspection team generally confirm the available data, a satisfactory evaluation for validity is indicated.

spillway riser structure with its invert at elevation 1277.5. The floor stand operator is located on the top of the principal riser which can only be accessed by boat when the reservoir pool is at the flood stage level. The gate is a Rodney Hunt, non seating head type, with a rising stem operator having the following identification:

27291-2
S-5002

The gate is normally in the closed position, and usually opened to drain the pond during the fall of each year. The pond drain has a maximum capacity of 169 CFS with the water level at the top of the dam.

At Tyringham Road there is a 36 inch dia. low level pond drain at invert elevation 1277.6 ft. NGVD. The drain is ungated and serves to hydraulically connect the impoundment areas to the invert elevation of the pipe.

(h) Diversion and Regulating Tunnel

Not applicable

(i) Spillways

1) Type:

- a) Principal spillway: Reinforced concrete drop inlet
- b) Emergency spillway: Grass and legume covered earth excavated channel with level control section

2) Length of weir:

- a) Principal spillway weirs: 2 @ 15 ft. = 30 ft.
- b) Emergency spillway: 130 feet

3) Crest Elevation

- a) Principal spillway weirs: 1286.3
- b) Emergency spillway: Control section-1288.33

4) Gates: None

5) Upstream channel:

- a) Principal Spillway: Reservoir
- b) Emergency Spillway: Grass and legume covered earth excavated channel. 150± ft. to control section.

6. Downstream Channel:

- a) Principal Spillway: Riprapped stilling basin discharging to the natural stream channel.
- b) Emergency Spillway: Grass and legume covered, earth excavated channel 125± ft. to natural stream channel directly downstream of dam.

(j) Regulating Outlets

The regulated outlet from the dam is the pond drain which is controlled by a manually operated 36 inch sluice gate. The gate is located on the inside face of the pond side wall of the principal

- 3) Spillway crest pool
 - a) Principal spillway crest - 1,100
 - b) Emergency spillway crest - 1,650
- 4) Top of dam - 3,600
- 5) Test flood pool - 3,250
- (f) Reservoir Surface (acres)
 - 1) Normal pool - 260
 - 2) Flood-control pool - 278 (Design High Water)
 - 3) Spillway crest
 - a) Principal spillway crest - 260
 - b) Emergency spillway crest - 270
 - 4) Test flood pool - 298
 - 5) Top of dam - 306
- (g) Dam
 - 1) Type - Earth embankment
 - 2) Length - 350 feet
 - 3) Height - 19.5 feet
 - 4) Top Width - 13.5 feet
 - 5) Side Slopes - Upstream face - 4 horizontal on 1 vertical to elevation 1286, 3 horizontal on 1 vertical from 1286 to top of dam. Downstream embankment - 3 horizontal on 1 vertical with horizontal berm at elevation 1287.0.
 - 6) Zoning - A combination of impervious borrow and pervious sand borrow.
 - 7) Impervious Core - Yes
 - 8) Cutoff - Variable width and depth, impervious earthfill cutoff trench.
 - 9) Grout curtain - None

- 5) Gated spillway capacity at normal pool elevation - None
 - 6) Gated spillway capacity at test flood elevation - None
 - 7) Total spillway capacity at test flood elevation - 4450 cfs at elevation 1293.40 ft. NGVD (same as No. 4)
 - 8) Total project discharge (principal and emergency spillway) at top of dam - 6800 cfs at elevation 1295.08 ft. NGVD (same as No. 3)
 - 9) Total project discharge at test flood elevation - 4450 cfs at elevation 1293.40 ft. NGVD (1.7 feet of free board remaining)
- (c) Elevation (feet NGVD)
- 1) Streambed at toe of dam - 1275.50
 - 2) Bottom of cutoff - 1271.7± (low point)
 - 3) Maximum tailwater - Unknown
 - 4) Normal pool - 1286.03 (principal spillway crest)
 - 5) Full flood control pool - 1288.33
 - 6) Emergency spillway crest elevation - 1288.33
 - 7) Design surcharge - 1289.5
 - 8) Top of dam - 1295.08
 - 9) Test flood surcharge - 1293.40 (1.7 feet of free board remaining)
- (d) Reservoir (Length in feet)
- 1) Normal pool - 10,300± feet
 - 2) Flood Control pool - 10,650± feet (Design High Water)
 - 3) Emergency spillway crest pool - 10,500± feet
 - 4) Top of dam - 10,750± feet
 - 5) Test flood pool - 10,700± feet
- (e) Storage (acre-feet)
- 1) Normal pool - 1,100
 - 2) Flood control pool - 2,000 (Design High Water)

1.3 Pertinent Data

(a) Drainage Area

The drainage area for the Lake Garfield Dam covers approximately 4.0 square miles. Nearly all of the drainage area is mountainous type wood land. There are a number of secondary type roads located within the watershed area with some development of homes along these routes. Major portions of the watershed area, however, are undeveloped woodland areas.

The Konkapot River, on which the Lake Garfield Dam is located, originates at the dam which is the only outlet from Lake Garfield. The river which originates at the Lake Garfield Dam, and flows to the Housatonic River, has many tributary streams which add to the flow downstream of the dam. The major tributary downstream confluences are with Loom Brook, Rawson Brook, and Swann Brook.

(b) Discharge at Dam Site

Normal discharge at the site is via the principal spillway through the 60 inch diameter outlet conduit to the downstream channel. If flood flows occur of sufficient magnitude and duration to fill the flood water storage available, then excess flow will be discharged around the dam via the emergency spillway channel. The impoundment may be lowered to elevation 1277.6 ft. NGVD upstream of Tyringham Road and 1277.5 ft. NGVD downstream of Tyringham Road via a sluice gate controlled, 36 inch diameter pond drain located at the dam and an ungated 36 inch diameter pond drain located at Tyringham Road.

1) Outlet Works:

Pond drain, 36 inch dia. RCP, invert elev. 1277.5 ft. NGVD, maximum capacity 169 CFS.

2) Maximum known flood at dam site - Unknown

3) Ungated spillway capacity at top of dam -

With the water level at the top of the dam (elev. 1295.08 ft. NGVD) spillway capacities are as follows:

principal spillway	600 CFS
emergency spillway	6,200 CFS
Total	6,800 CFS

4) Ungated spillway capacity at test flood elevation -

With the water level at the test flood elevation (1293.40 ft. NGVD) spillway capacities are as follows:

principal spillway	450 CFS
emergency spillway	4,000 CFS
Total	4,450 CFS

(e) Ownership

The Lake Garfield Dam is owned by the Town of Monterey, Massachusetts, and is under the responsibility of the Board of Selectmen.

(f) Operator

The operation of the Lake Garfield Dam is the responsibility of the Town of Monterey, Massachusetts, through the Board of Selectmen.

(g) Purpose of Dam

The Lake Garfield Dam is a multiple purpose dam which maintains a recreation pool level and provides some flood storage capacity to reduce downstream flooding from the dam's drainage area. Stored flood water is gradually released through the principal spillway structure.

(h) Design and Construction History

The Lake Garfield Dam was designed by the Consulting Firm of R.G. Brown Associates, Inc., of Pittsfield, Massachusetts. The dam was constructed by the firm of Petricca Construction Company of Pittsfield, Massachusetts and supervision of construction was by the Commonwealth of Massachusetts, Department of Public Works, Division of Waterways, District No. 1. The dam was completed in 1973 and has been in operation since that time. There have been no modifications to the dam since the original construction was completed. However, during the construction of the dam, the normal recreation pool level was increased by 4 inches. The increase in the pond level is reported to have been the result of a petition from lake front property owners which was made during construction. This resulted in raising the principal spillway overflow weirs, the top of the impervious borrow within the embankments, the top of the dam, and the control section of the emergency spillway, all by four inches. No other modifications to the original design are known to have been made during or after construction of this dam.

(i) Normal Operation Procedure

The Lake Garfield Dam is normally self-regulating with the only controlled outlet being the gated pond drain located at the dam. The impoundment area is divided by the Tyringham Road crossing. At this location, an ungated low level pond drain has been provided at invert elevation 1277.6 ft. NGVD. The gated pond drain located at the dam is at invert elevation 1277.5 ft. NGVD. The gated pond drain is normally opened during the fall of each year to drain the impoundment to the level of the pond drain pipes. This is done to allow persons owning camps and residences along the impoundment to repair dock facilities, beaches, retaining walls, etc.

3) Emergency Spillway (See B-2, B-3 & B-5)

The emergency spillway consists of a legume covered earth channel excavated through natural ground on the left abutment of the dam. The spillway channel has a control section approximately at elevation 1288.33 which is 130 feet wide and 30 feet long. The spillway approach channel has a slope of 2% upwards towards the control section and is approximately 150 feet in length. The discharge channel has a downward slope of 3% and is approximately 125 feet in length. The emergency spillway channel curves to the right on the approach and discharge sides and discharges into the original stream channel of the headwaters of the Konkapot River just downstream of the dam. The side slopes of the spillway excavation are 2 horizontal on 1 vertical. The control section is 6.75 feet below the top of the dam.

4) Foundation and Embankment Drainage (See B-6)

A trench drain consisting of gravel extends into the foundation material beneath the downstream embankment. The trench drain extends from the principal spillway about 236 feet right with about 156 feet of 8" diameter perforated A.C. pipe. The trench drain extends from the principal spillway about 35 feet left with about 13 feet of 8" diameter perforated A.C. pipe. Both of the drainage pipes discharge into the stilling basin at invert elevation 1275.8.

A blanket drain of pervious fill material extends vertically between the two zones of impervious fill and horizontally along the foundation of the downstream embankment. This allows drainage of accumulated seepage to be conveyed to the foundation drain trench.

For additional details of the drainage features, refer to pages B-3 and B-6 of Appendix B.

(c) Size Classification

The dam has a maximum impoundment (computed to the top of the dam) of about 3660 acre-feet and a structural height of approximately 19.5 feet. The dam is therefore in the INTERMEDIATE size classification due to impoundment size in accordance with the Corps of Engineers Recommended Guidelines.

(d) Hazard Classification

The hazard potential classification for this dam is HIGH because of the potential for loss of more than a few lives and significant property damage which may occur in the event of a dam failure during a PMF occurrence. There is a potential for severely damaging 18 homes as well as 1 store and 18 culverts crossing roadways.

2) Principal Spillway (See B-4, B-7, B-8 & B-9)

The principal spillway consists of a reinforced concrete drop inlet structure with a sluice gate controlled pond drain pipe at invert elevation 1277.50. The drop inlet structure has two uncontrolled overflow weirs at elevation 1286.03 which control the recreation pool level of the impoundment. The outlet conduit from the principal spillway is a 60 inch diameter reinforced concrete pipe with an inlet invert elevation of 1276.50.

The riser structure is 13 feet 3/8 inches in height from the base of the foundation to the top of the structure. The inside dimensions are 8 feet x 15 feet with 12 inch thick reinforced concrete walls. The inside bottom elevation of the riser structure is elevation 1276.50. The overflow weirs are formed by the top of the riser section side-wall, and each has a length of 15 feet for a total of 30 feet of weir crest at elevation 1286.03. The top of the riser structure is a solid reinforced concrete slab with a 2 foot diameter access opening covered by a cast iron manhole cover. The overflow weirs are protected by a trash rack system consisting of a galvanized iron frame with vertical and horizontal bars located above the crest elevation, and 4-2½ inch diameter pipes located horizontally across the structure below the crest elevation. For additional details regarding the reinforced concrete structure and trash rack system refer to pages B4, B-7 and B-8 of Appendix B.

The sluice gate which controls the 36 inch diameter pond drain is a Rodney Hunt sluice gate, mounted on a type F wall thimble, and is a 36 inch, non-seating head sluice gate located on the inside of the upstream face of the principal spillway riser. The gate is operated by a rising stem, manual crank operated floorstand located on the top of the riser structure.

The pond drain pipe consists of about 16 feet of 36 inch diameter reinforced concrete conduit with a reinforced concrete headwall structure. The headwall structure has a trash rack system consisting of horizontal reinforcing bars embedded into the concrete over the inlet to the pipe. The pond drain pipe enters the structure through the upstream face.

The principal spillway structure has a 60 inch diameter outlet conduit to a stilling basin located at the downstream toe of the dam. The 60 inch diameter conduit consists of about 120 feet of reinforced concrete pipe with a continuous concrete bedding and four anti-seep collars. The pipe has an inlet elevation of 1276.50 and an outlet elevation of 1275.50 providing a slope of 0.0083 feet per foot.

The stilling basin is constructed of an excavated basin from natural ground and is lined with riprap. The 60 inch diameter outlet conduit does not have a headwall at its outlet end.

Monterey. The dam impounds the headwater of the Konkapot River which is a tributary to the Housatonic River. The dam and impoundment is located off of Beartown Mountain Road and is approximately 0.9 miles from the center of Monterey.

The dam is located on U.S.G.S. Monterey, Mass., quadrangle at longitude E 73°-12'-50" and latitude N 42°-11'-26". Refer to the location plans, and Appendix B for additional information.

(b) Description of Dam and Appurtenances

The dam consists of an earthfill embankment, a principal spillway consisting of a reinforced concrete inlet structure, a 60" diameter reinforced concrete outlet conduit, and an earth excavated stone-lined stilling basin at the conduit outlet. An emergency spillway is located on the left abutment and consists of a legume and grass covered channel, excavated in natural ground.

1) Embankment (See B-2, B-3, & B-4)

The following information has been taken from the construction drawings dated 1971, and verified where possible by the visual inspection.

The dam embankment is approximately 350 feet long and has a maximum structural height of approximately 19.5 feet above the original stream channel at the downstream toe. The upstream slope of the embankment is 4 horizontal on 1 vertical from the upstream toe to elevation 1286. From elevation 1286 to the top of the dam the upstream slope is 3 horizontal on 1 vertical. The upstream slope is covered with a blanket of 2 inch washed gravel from the toe to elevation 1288.33, and is grass covered from this point to the top of the dam.

The downstream slope is 3 horizontal on 1 vertical and has a berm (horizontal section) at elevation 1287. The entire downstream slope and top of the dam is grass covered.

The earthfill material consists of basically three materials which are impervious borrow, sand borrow, and gravel. The major portions of the earthfill material below elevation 1288.33 consists of the impervious borrow material. Above elevation 1288.33, the earthfill material is pervious consisting mainly of sand borrow. The impervious borrow material has been placed in two separate sections with the downstream section being separated from the upstream section by a vertical section of sand borrow material. The sand borrow also is placed under the impervious borrow material on the downstream embankment, to allow drainage of accumulated seepage from the central portion of the embankment. The upstream embankment and the top of the upstream impervious borrow section has been covered with a gravel blanket. For further details regarding the earthfill material of the dam embankment, refer to page B-3 of Appendix B.

NATIONAL DAM INSPECTION PROGRAM
PHASE I INSPECTION REPORT

LAKE GARFIELD 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. Tighe & Bond/SCI has been retained by the New England Division to inspect and report on selected dams in Massachusetts. Authorization and notice to proceed were issued to Tighe & Bond/SCI under a letter of October 24, 1979 from Colonel William E. Hodgson, Jr., Corps of Engineers. Contract No. DACW-33-80-C-0005 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 prepare the states to initiate quickly effective dam safety programs for non-federal dams.
- 3) Update, verify, and complete the National Inventory of Dams.

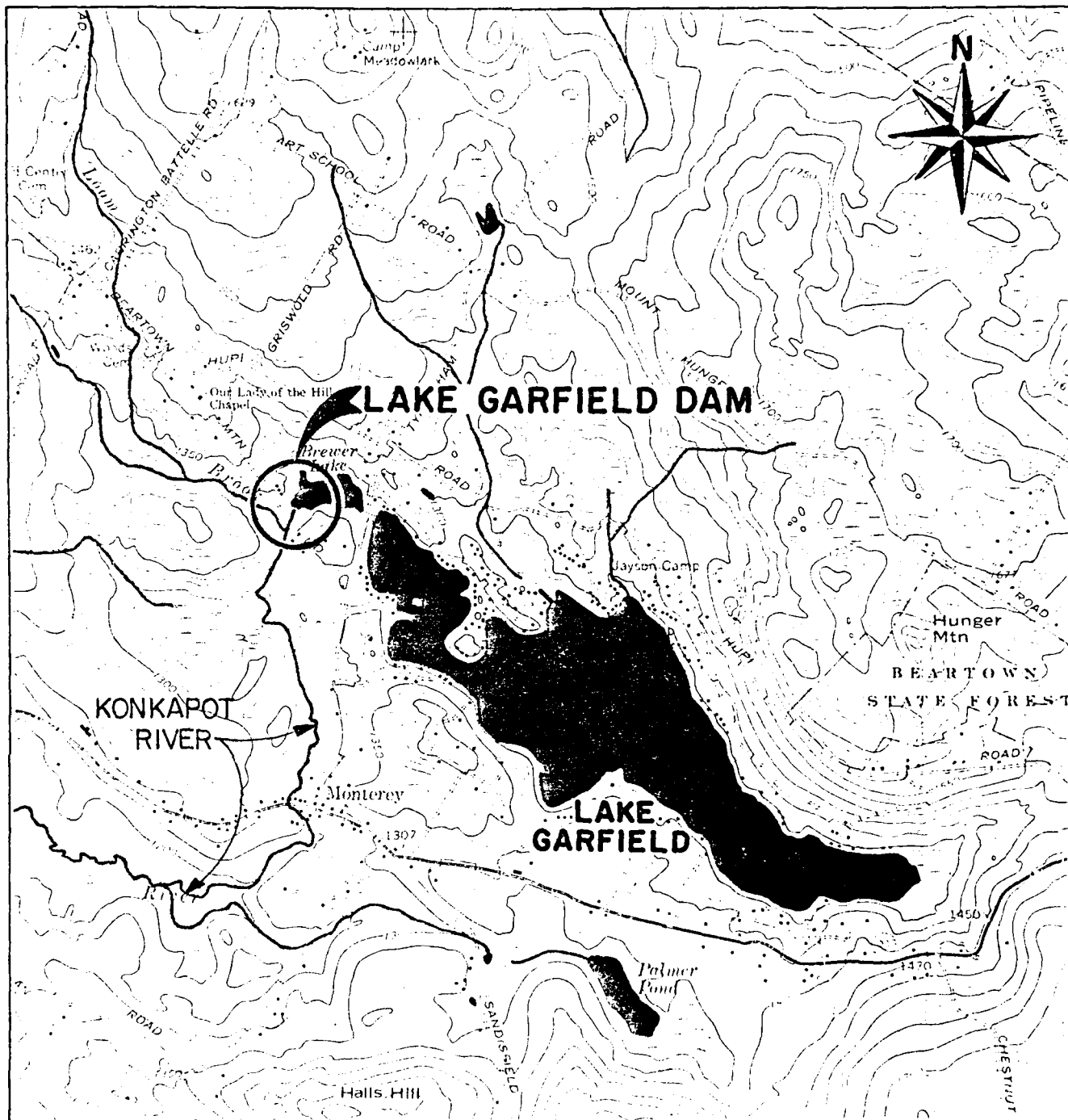
(c) Scope

The program provides for the inspection of non-federal dams in the high hazard potential category based upon location of the dams, and those dams in the significant hazard potential category believed to represent an immediate danger based on condition of the dams.

1.2 Description of Project

(a) Location

The Lake Garfield Dam is located within the Town of Monterey, Massachusetts, about 0.85 miles upstream from the Village of



- SCALE -
1000' 0 1000' 2000' 3000'

FROM: U.S.G.S. MONTEREY, MASS.



QUADRANGLE LOCATION

TIGHE & BOND / SCI
CONSULTING ENGINEERS
EASTHAMPTON, MASS.

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

NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS

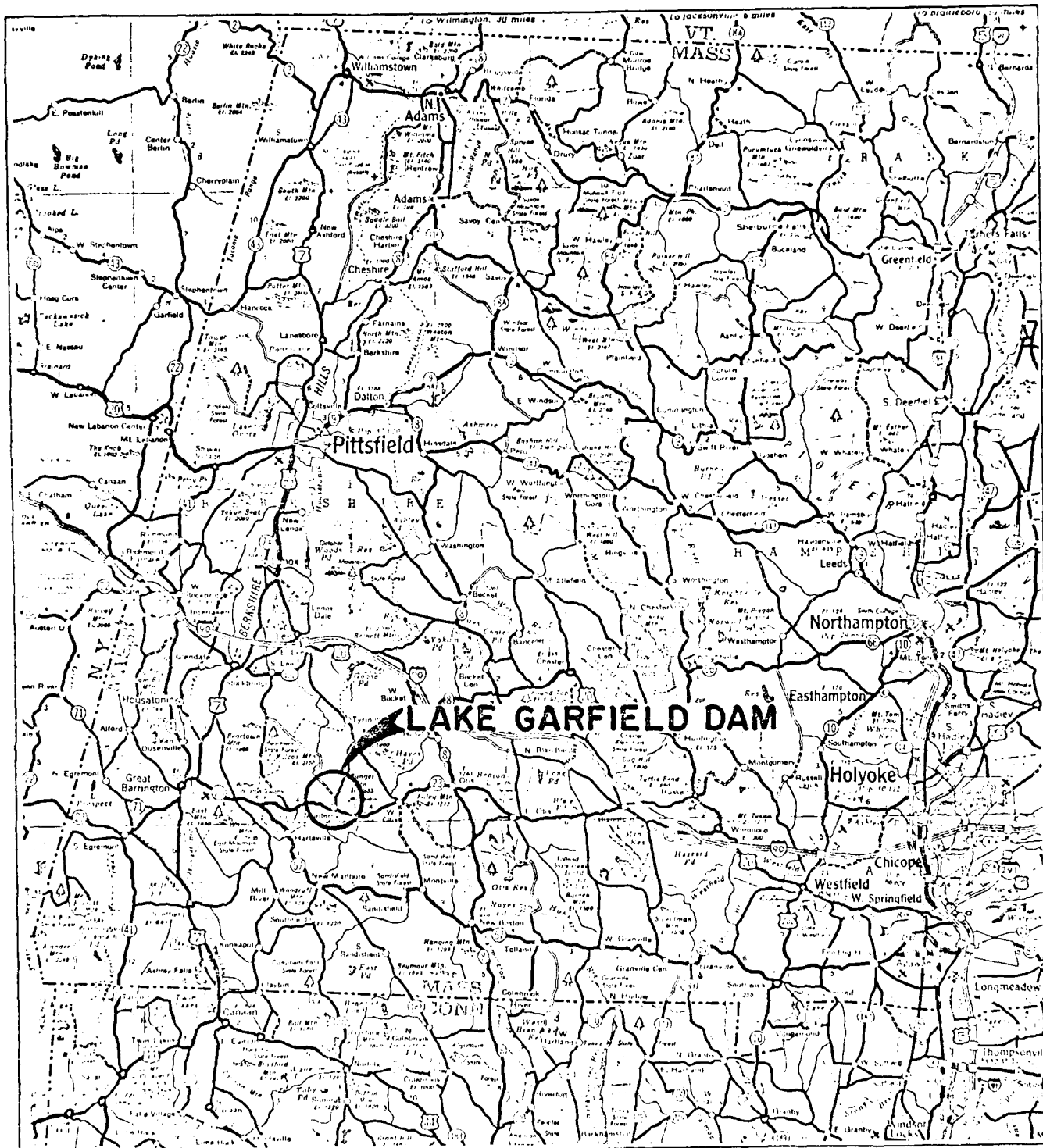
LOCUS PLAN 2

LAKE GARFIELD DAM (MA 00249)
BERKSHIRE COUNTY

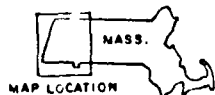
MONTEREY
MASSACHUSETTS

SCALE: AS NOTED

DATE: MARCH 1980



5 0 5 10
SCALE IN MILES



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CONSULTING ENGINEERS
EASTHAMPTON, MASS.

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

NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS

LOCUS PLAN I

LAKE GARFIELD DAM (MA00249)
BERKSHIRE COUNTY

MONTEREY
MASSACHUSETTS

SCALE: AS NOTED
DATE: MARCH 1980

The floor of the discharge channel was found to be generally dry, with a good growth of grass and legumes. The excavated embankment, on the left side of the emergency spillway discharge channel was found to have sloughing occurring about 108 feet downstream of the control section. The sloughing is occurring along the toe of the embankment, and also approximately 20 feet up the embankment. The grass growth has been established over the entire area with no exposed earth areas being visible, therefore, it appears that this embankment movement occurred at least a few years ago and has somewhat stabilized since that time. The slope movement appears to have been caused by a combination of the impervious material in this area and side hill seepage from the right abutment area. This condition does not appear to be critical and does not endanger the capacity of the emergency spillway.

The entire emergency spillway channel was found to be free of debris and did not have any overhanging trees or channel obstructions.

The downstream embankment of the dam is protected from the emergency spillway flow by a training embankment along the right side of the emergency spillway channel. The embankment essentially forms a berm between the principal spillway and the downstream area of the dam. The embankment is only about 4 to 5 feet above the spillway channel, and has a cross sectional shape which is nearly triangular. The training embankment does not have any erosion protection other than the grass cover, and due to its design, could quite easily be eroded by flood flows through the emergency spillway. Some animal borrows were found within the training embankment on the emergency spillway side.

(c) Appurtenant Structures

1) Drop Inlet Principal Spillway (See photos 5, 6, 7 & 8)

The principal spillway riser was found to be in good condition. The structure appeared to be structurally sound with no visible cracking, spalling, seepage, or efflorescence.

The interior of the riser structure was found to be free of any debris or blockage. The sluice gate operator appears to be in good condition, and was found in the open position with the pond drained.

2) Pond Drain Inlet Pipe (See photos 7 & 8)

At the time of the inspection, the pond was drained, therefore, the inlet pipe and headwall structure were visible for inspection. Both the pipe and reinforced concrete headwall were found to be in very good condition. The trash rack system over the inlet to the pond drain was free of any large debris or blockages, and was free flowing.

3) Outlet Conduit (See photos 9 & 10)

The 60 inch diameter conduit was found to be in good condition. All joints visible from the outlet end were found to have good alignment and appeared to be dry above the flow line. The interior of the conduit is in good condition with no visible spalling, cracking, or efflorescence.

4) Stilling Basin (See photo 9 & 10)

The stilling basin at the outlet of the principal spillway outlet conduit, was found to be in good condition. Riprap protection on the embankments was in good condition, and the basin did not appear to have accumulated any visible sediment. The stilling basin was clear of debris with free unobstructed outflow in the downstream channel.

(d) Impoundment Area (See photo 2, 15, 16, 17)

The shoreline of the impoundment area formed between Tyringham Road, which is the location of the original dam, and the present location of the Lake Garfield Dam, was found to be in very good stable condition.

At the time of the Lake Garfield Dam construction, Tyringham Road was reconstructed, and the old masonry dam replaced with three, 84 inch diameter ACCMP culverts, and a 36 inch diameter pond drain for the reservoir area upstream of Tyringham Road. The triple 84 inch diameter culverts were found to be in good condition with good alignment, no erosion, and no unusual wear. Both the inlet and outlet end of the three culverts contained a large amount of stones, which appear to have been dropped into the pipe from the roadway above by children. Both the inlet end and outlet end of the 36 inch diameter pond drain has a reinforced concrete headwall with vertical reinforcing bars embedded in the concrete over the inlet and outlet of the pipe. The inlet end had a large accumulation of weeds and debris but the flow was not seriously impeded. The roadway embankments on both the upstream and downstream sides of the ACCMP culverts were protected by riprap. The riprap was found to be in good condition and did not show any signs of settlement, movement, or sloughing.

(e) Downstream Channel (See photo 10)

The downstream channel is in fair condition with a substantial amount of vegetation growth just downstream of the stilling basin. The channel was free of any debris, fallen trees, or other obstructions, except for the vegetation growth.

3.2 Evaluation

The dam is generally in good condition with the following deficiencies noted:

- (a) Small animal hole and patch of damaged turf on the lower portion of the downstream embankment just to the right of the principal spillway outlet conduit.
- (b) The emergency spillway right training berm area lacks adequate erosion protection.
- (c) The channel downstream of the stilling basin has a substantial amount of vegetation growth which reduces the flow capacity of the channel.

SECTION 4 - OPERATION AND MAINTENANCE PROCEDURES

4.1 Operational Procedures

(a) General

No written operation procedures are available for this dam. The dam is normally self-regulating. The sluice gate on the pond drain is normally in the closed position and is annually opened to lower the pond during the fall of each year.

(b) Description of Warning System in Effect

There is no written warning system in effect.

4.2 Maintenance Procedures

(a) General

The Town of Monterey is responsible for the maintenance of this facility. Maintenance items routinely conducted by the Town include mowing of the embankments and removal of debris from the principal spillway riser structure.

The dam and appurtenant features were inspected by the Commonwealth of Massachusetts Department of Public Works, Division of Waterways, on March 7, 1974. Included in Appendix B is a copy of the inspection report. In 1976 jurisdiction over the non-federally owned dams in the Commonwealth of Massachusetts was transferred to the Department of Environmental Quality Engineering. The Commonwealth of Massachusetts DEQE has not established a program of routine inspection for the dams under their jurisdiction. No inspection reports are available other than the 1974 report included in Appendix B.

(b) Operational Facilities

The only facility requiring routine operation is the pond drain sluice gate located within the principal spillway riser. This facility is routinely operated on an annual basis for draining the pond.

4.3 Evaluation

A formal, written downstream emergency flood warning system should be developed for the dam. A routine program of annual inspections should be started with formal written reports and a listing of required maintenance items.

SECTION 5 - EVALUATION OF HYDRAULIC/HYDROLOGIC FEATURES

5.1 General

The Lake Garfield Dam, No. MA 00249, is a multiple purpose recreation and flood water storage facility which was designed by the consulting firm of R.G. Brown & Associates and is owned by the Town of Monterey, Massachusetts.

The dam is located on the headwaters of the Konkapot River, and forms the beginning of the river with the Lake Garfield drainage area being the source. The Konkapot River flows to the Housatonic River and the Lake Garfield dam is located approximately 19 miles upstream of the Housatonic River confluence. Many small streams converge with the main flow of the Konkapot River between its source, Lake Garfield and the Housatonic River.

The drainage area upstream of the dam is approximately 4.0 square miles (2,550 acres) and consists mainly of wooded, mountainous terrain.

Development within the watershed consists of a number of secondary back roads with a moderate number of single-family residences and a few farms.

The dam itself is about 350 feet long and 19.5 feet high and is an earthfill embankment having both impervious and pervious fill zones. The facility has a principal spillway which maintains a recreation pool level and discharges all normal flows via a 60 inch diameter conduit through the dam. An emergency spillway, consisting of a 130 foot wide earth channel, excavated in natural soil with a legume and grass cover, carries flood flows which exceed the storage capacity of the impoundment around the dam to the downstream channel.

The impoundment area is divided by Tyringham Road which traverses across the lake upstream of the dam. A major portion of the impoundment is upstream of the roadway embankment. The two sections of the lake are hydraulically connected by three, 84 inch diameter culverts and one, 36 inch diameter pond drain. A major portion of the watershed runoff flows through the road culverts prior to reaching the dam. Flood flows are attenuated upstream of the Tyringham roadway according to the surcharge storage characteristics of the upstream impoundment and the hydraulic capacity of the culverts.

5.2 Design Data

A review of the design data for the Lake Garfield Dam, indicates that the "Upper Darby Method" of flood routing was utilized for the hydraulic analysis of the facility. The top of the dam elevation was based on a design rainfall of 22.8 inches which resulted in 18.4 inches of runoff over the drainage area. The design computations assumed a pool elevation of 1285.7 ft. at the beginning of the test storm. This elevation was the original normal pool design level. The peak design inflow was approximately 12,800 CFS which resulted in a peak spillway outflow of approximately 7000 CFS at a pond elevation of 1294.75. The emergency spillway crest has been designed to provide impoundment storage to retard a 100 year

frequency design storm without discharge occurring from the emergency spillway. The design normal recreation pool level was maintained at the level of the previously existing Lake Garfield formed by the old dam which was located at the Tyringham Road crossing. This elevation was increased by 4 inches during the construction of the dam. Raising the normal recreation pool level resulted in raising the crest of the principal spillway weirs, the crest of the emergency spillway control section and the top of the dam, all by 4 inches over the design elevations.

5.3 Experience Data

No records of flow or stage are known to be available for the Lake Garfield Dam.

5.4 Test Flood Analysis

The selection of the test flood is based on the Corps of Engineers "Recommended Guidelines for Safety Inspection of Dams," dated November 1976. These guidelines state that dams classified as "INTERMEDIATE" in size, and HIGH in hazard potential be tested against the Probable Maximum Flood (PMF).

The determination of the PMF for the Lake Garfield Dam is based on the Corps of Engineers "Preliminary Guidance for Estimating Maximum Probable Discharges in Phase I Dam Safety Inspections" dated March 1978. For a drainage area of 4.0 square miles using the mountainous terrain condition, a unit discharge of 2,350 cfs per square mile is obtained from the guidance curves. This results in a PMF test flood of 9,200 cfs.

The purpose of this Phase I inspection is to assess the dams overtopping potential and its ability to store and/or discharge the test flood. This requires determining the storage characteristics of the impoundment area and the stage vs. discharge characteristics of the spillways. These computations have been performed and are included in Appendix D.

The test flood has been routed through the reservoir using the iteration process as outlined in the Corps of Engineers "Preliminary Guidance for Estimating Probable Maximum Discharges in Phase I Dam Safety Inspection."

Two test flood routing analyses were made as follows:

- a) Routing including the effects of Tyringham Road located upstream of the dam.
- b) Routing assuming the Tyringham Road embankment is removed.

The results of routing the test flood assuming Tyringham Road is in place, indicate that the storage capacity of the impoundment area will reduce the test flood inflow of 9,200 CFS to a reservoir outflow of approximately 4200 CFS at a pond storage of 1293.1 feet NGVD. This assumes that the level of the pond is at the recreation pond level of 1286.03 at the start of the test storm.

This analysis indicates that Tyringham Road will be overtopped at the culvert location by approximately 2 feet and is, therefore, subject to substantial damage potential. Breaching of the roadway will reduce the attenuation effects and increase the test flood outflow. The worst case for routing purposes would be with the roadway embankment completely removed.

The results of routing the test flood assuming the roadway embankment completely removed indicate that the test flood inflow of 9,200 CFS would be reduced to a reservoir test flood outflow of 4,450 CFS at a pond stage of 1293.4 feet NGVD.

The combined spillways have a discharge capacity with the water level at the top of the dam of about 6,800 CFS. Using the "worst case" analysis for test flood routing, the reservoir test flood outflow is 4450 CFS. The combined spillways have a capacity of 152% of this test flood outflow, and will allow a remaining freeboard of approximately 1.7 feet to the top of the dam with a test flood pond stage of 1293.4 feet NGVD.

With a capacity to discharge the PMF test flood outflow of the reservoir while maintaining freeboard of 1.7 feet, the dam is concluded to have adequate spillway capacity.

5.5 Dam Failure Analysis

A dam failure analysis using the procedures in the Corps of Engineers "Rule of Thumb Guidance for Estimating Downstream Failure Hydrographs" dated April 1978, was performed for the Lake Garfield Dam. The assumed conditions are as follows:

1. The water level of the impoundment prior to breach occurring is at the test flood elevation of 1293.40.
2. Stream flow at the time of breach is the spillway test flood outflow.
3. For determining the storage volume available to sustain dam failure flow, the "worst case" analysis assuming that Tyringham Road is either breached by flood flows or removed has been used. This provides the entire impoundment storage below the test flood pond stage for dam failure flow analysis.

For an assumed breach equal to 40 percent of the dam width computed at half height, the breached width is approximately 95 feet. The resulting dam failure flow using a water depth of 18.2 feet is 12,400 cfs.

The first damage area impacted by dam failure flow is directly downstream of the dam. The test flood flow prior to the dam breach occurring is 4,450 cfs resulting in a river stage of about 5.5 feet. Dam failure flow is 12,400 cfs resulting in a river stage of about 7.5 feet. There are no structures or development directly downstream of the dam, therefore, any damage will not be significant.

The second damage area impacted by dam failure flow is at the crossing of Beartown Mountain Road about 400 feet downstream of the dam. There is one culvert at this location. Prior to dam breach, test flood flow is 4,450 cfs resulting in a river stage of about 5.5 feet. The culvert has a capacity with the water level at the roadway elevation of approximately 260 cfs, therefore, it will be inundated and the roadway overtopped by about 4 feet. The dam failure attenuated flow is 12,400 cfs resulting in a river stage of about 7.5 feet. This will increase the depth of flow over the roadway by about 3.5 feet and significantly increase the probability of severe damage to the roadway.

The third damage area impacted by the dam failure flow is the Village of Monterey and a crossing of Route 23 in the center of the Village, which is located about 4,500 feet downstream of the dam. There is one culvert at this location. Prior to dam breach, the test flood flow is 4,450 cfs resulting in a river stage of about 10 feet. The culvert has a surcharged capacity of approximately 1,200 cfs, therefore, it will be inundated and the roadway overtopped by about 1.5 feet. The pre-failure flood stage will result in about 5 homes plus 1 store being flooded by about 1.5 feet. The dam failure attenuated flow is 12,100 cfs resulting in a river stage of about 14 feet. This will increase the depth of flow over the roadway by about 4 feet, to a depth of 5.5 feet. The 5 homes plus 1 store flooded by pre-failure flow will be flooded an additional 4 feet, resulting in 5 homes plus 1 store flooded by 5.5 feet, and 1 additional home flooded by 5.5 feet. This will, in effect, significantly increase the probability of severe damage to the primary roadway, and to all of the flooded structures.

The fourth damage area impacted by dam failure flow is a crossing of Curtis Road about 14,000 feet downstream of the dam. There is one culvert at this location. Prior to dam breach, the test flood flow is 4,450 cfs, which results in a river stage of approximately 8 feet. The actual capacity of the Curtis Road culvert is unknown, but it is definitely much less than the pre-failure flood flow, and a river stage of 8 feet will result in the roadway being overtopped by about 2 feet. The dam failure attenuated flow is 12,100 cfs resulting in a river stage of approximately 12.5 feet. The increased flooding of approximately 4.5 feet will significantly add to the potential for damage to the roadway.

The fifth downstream area impacted by dam failure flow is just downstream of the confluence of the Swann Brook which is approximately 15,000 feet downstream of the dam. The Konkapot River at this location flows parallel to the Route 23 roadway, and a number of homes are located in the area. Prior to dam breach, the test flood flow is 4,450 cfs resulting in a river stage of about 8 feet. Pre-failure flood levels will not result in a significant hazard potential. Dam failure attenuated flow is 11,600 cfs resulting in a river stage of about 12 feet. This will result in all flood levels being increased by about 4 feet, but will not significantly add to the dam failure damage potential.

The sixth downstream area impacted by dam failure flow is the Village of Hartsville, which is approximately 27,000 feet downstream of the dam. There is one culvert at this location. Prior to dam breach,

the test flood flow is 4,450 cfs, which results in a river stage of approximately 3 feet. The Route 57 roadway culvert capacity will be exceeded and the roadway overtopped by about 1 foot. The pre-failure flooding will result in about 13 homes being flooded approximately 1 foot. The dam failure attenuated flow is 11,100 cfs resulting in a river stage of approximately 5 feet. This will increase the depth of flow over the roadway to a depth of 3 feet. The 13 homes flooded by pre-failure flow will be flooded an additional 2 feet, resulting in a flooded depth of 3 feet. This will, in effect, significantly increase the probability of severe damage to the primary roadway, and the structures flooded.

The seventh downstream area impacted by dam failure flow is the Village of Mill River, which is located about 50,000 feet downstream of the dam. There are three roadway culverts at this location. Prior to dam breach, the test flood flow is 4,450 cfs resulting in a river stage of about 5.5 feet. The pre-failure flood flow will exceed the capacity of each culvert and overtop the roadways by about 4.5 feet. No homes will be flooded by pre-failure flood levels. Dam failure attenuated flow is 8,000 cfs resulting in a river stage of about 10 feet. This will result in an increase flooding of about 5.5 feet over the roadways and significantly add to the probability of severe damage to the primary roadways. No structure damage is anticipated.

The eight downstream area impacted by dam failure flow is the reach of the Konkapt River downstream of the Village of Mill River to the confluence with the Housatonic River. This reach is approximately 55,000 feet in length and includes ten roadway culverts. Prior to dam breach, the test flood flow is 4,450 cfs, resulting in a river stage of about 7 feet. Pre-failure flooding is expected to exceed the capacity of all the roadway culverts. Most of the culverts are located at fairly broad flood plain areas, therefore, flooding will be shallow and probably in the order of 1 foot. Just upstream of the Village of Clayton, there is one house which will be flooded by pre-failure flows to about a depth of 5 feet. The dam failure attenuated flow is 7,600 cfs and results in a river stage of about 9.5 feet. This will increase all flood levels by 2.5 feet and only slightly increase the damage potential.

Downstream of the confluence with the Housatonic River, dam failure flows will not constitute a significant increase in the flood damage potential.

Probable Downstream Impact Before and After Dam Failure
Lake Garfield Dam MA 00249

<u>Location</u>	<u>No. of Houses or Struc.</u>	<u>Other Damage</u>	<u>Flow Rates (cfs)</u>		<u>River Stage (ft)</u>		<u>Comments</u>
			<u>Before Failure</u>	<u>After Failure</u>	<u>Before Failure</u>	<u>After Failure</u>	
1. Downstream Dam	0	0	4,450	12,400	5.5	7.5	No significant damage
2. 400' D.S. at Beartown Mtn. Road Crossing	0	1 culvert	4,450	12,400	5.5	7.5	Before failure roadway overtopped 4 ft.; after failure roadway overtopped 7½ feet
3. 4,500' D.S. at Village of Monterey and Rt. 23 Crossing	5	1 store 1 culvert	4,450	12,100	10	14	Before failure roadway overtopped 1.5 ft., 5 houses and 1 store flooded 1.5 ft.; after failure all flooding increased 4 ft., 1 house flooded 3.5 ft.
4. 14,000' D.S. at Curtis Road Crossing	0	1 culvert	4,450	12,100	8	12.5	Before fail- ure roadway overtopped 2 ft.; after failure flooding increased 4.5 ft.
5. 15,000' D.S. at Rt. 23 D.S. of Swann River Confluence	0	1 culvert	4,450	11,600	8	12	No significant damage.
6. 27,000' D.S. at Village of Hartsville and Rt. 57 Crossing	13	1 culvert	4,450	11,100	3	5	Before failure roadway flooded 1± ft., 13 houses flooded 1± ft.; after fail- ure flooding increased 2 f

Probable Downstream Impact Before and After Dam Failure
Lake Garfield Dam MA 00249
 (Continued)

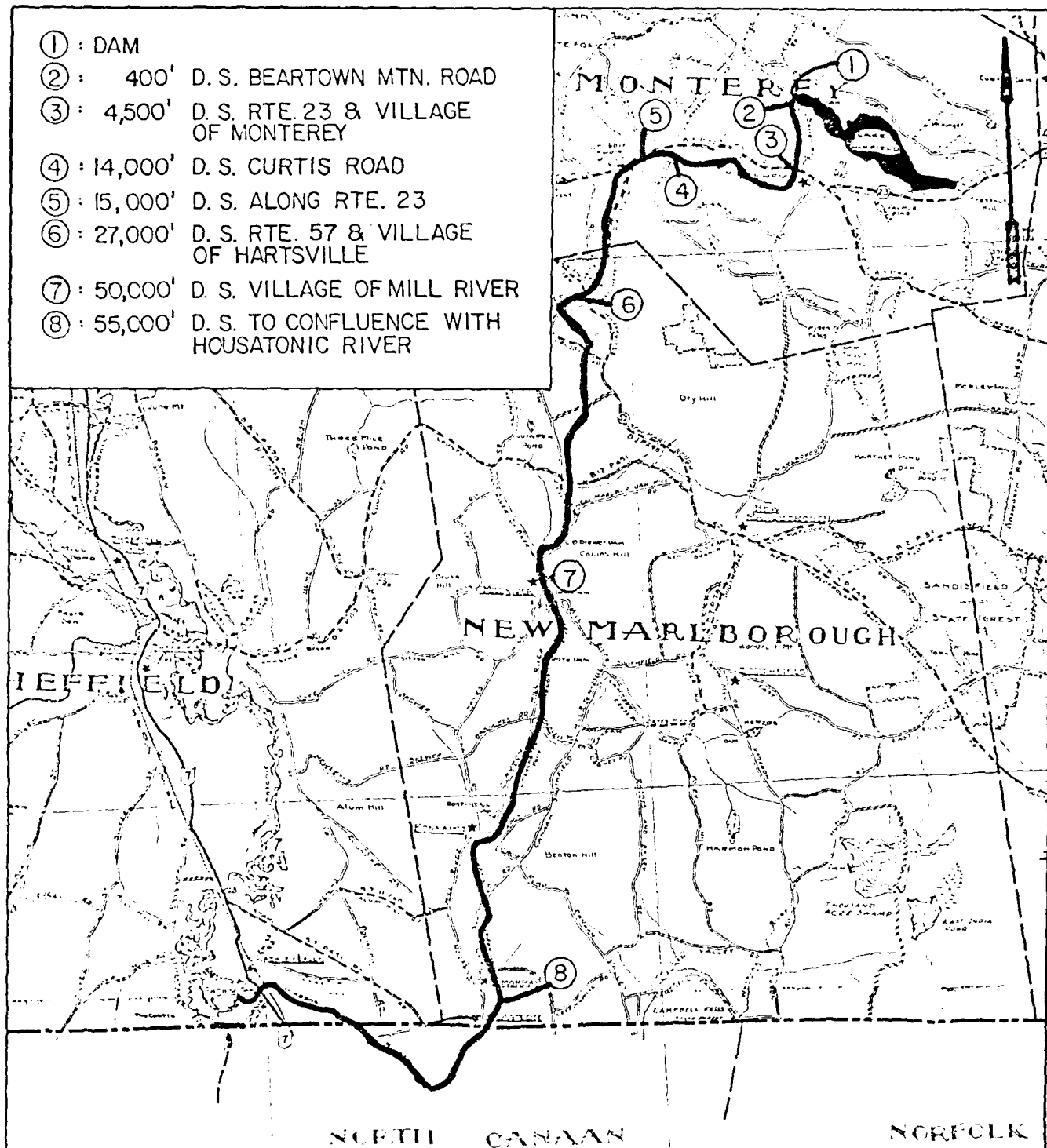
<u>Location</u>	<u>No. of Houses or Struc.</u>	<u>Other Damage</u>	<u>Flow Rates (cfs)</u>		<u>River Stage (ft)</u>		<u>Comments</u>
			<u>Before Failure</u>	<u>After Failure</u>	<u>Before Failure</u>	<u>After Failure</u>	
7. 50,000' D.S. at Village of Mill River and 3 Roadway Crossings	0	3 culverts	4,450	8,000	5.5	10	Before failure 3 roadways flooded 4.5± ft.; after failure flooding increased 5.5 ft.
8. 55,000' Reach from Village of Mill River to Confluence w/Housatonic Rvr.	1	10 culverts	4,450	7,600	7	9.5	Before failure 10 culverts flooded 1± ft., 1 house flooded 5± ft.; after failure flooding increased 2.5± ft.

Total number of houses flooded before failure = 19

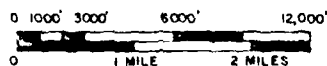
Total number of houses flooded after failure = 20

No. of homes with significant additional flooding = 18

- ① : DAM
- ② : 400' D. S. BEARTOWN MTN. ROAD
- ③ : 4,500' D. S. RTE. 23 & VILLAGE OF MONTEREY
- ④ : 14,000' D. S. CURTIS ROAD
- ⑤ : 15,000' D. S. ALONG RTE. 23
- ⑥ : 27,000' D. S. RTE. 57 & VILLAGE OF HARTSVILLE
- ⑦ : 50,000' D. S. VILLAGE OF MILL RIVER
- ⑧ : 55,000' D. S. TO CONFLUENCE WITH HOUSATONIC RIVER



-SCALE-



QUADRANGLE LOCATION

TIGHE & BOND / SCI
CONSULTING ENGINEERS
EASTHAMPTON, MASS.

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

NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS

LOCATION AND DOWNSTREAM HAZARD MAP

LAKE GARFIELD DAM (MA 00249)
BERKSHIRE COUNTY

MONTEREY
MASSACHUSETTS

SCALE: AS NOTED

DATE: MARCH 1980

SECTION 6 - EVALUATION OF STRUCTURAL STABILITY

6.1 Visual Observation

The visual inspection of the dam embankments did not identify any conditions indicating instability of the slopes. No settlement, sloughing, or piping was observed, and no cracking of the surface could be detected.

6.2 Design and Construction Data

(a) Embankment

Analyses carried out during the design by the design consultant included embankment slope stability analysis. Our review of this information indicates that the embankments are designed based on sound engineering practice.

(b) Appurtenant Structures

A review of the structural calculations for the design of the principal spillway structure and the outlet conduit indicated that these structures have been designed on the basis of sound engineering practice.

6.3 Post-Construction Changes

There are no known post-construction modifications to the Lake Garfield Dam.

6.4 Seismic Stability

The Lake Garfield Dam is located in seismic zone 1. According to the recommended Corps of Engineers Guidelines, a seismic analysis is not warranted.

SECTION 7 - ASSESSMENT, RECOMMENDATIONS AND REMEDIAL MEASURES

7.1 Dam Assessment

(a) Condition

The dam and its appurtenances are in GOOD condition.

(b) Adequacy of Information

There is sufficient design and construction data to permit an assessment of dam safety. In general, available data, past performance of the dam, and sound engineering judgement were sufficient to conduct the analyses presented in this report.

(c) Urgency

The remedial measures described herein should be implemented by the Owner within two years of receipt of this Phase I inspection report.

7.2 Recommendations

The recommendations of this Phase I investigation are that no additional studies are required.

7.3 Remedial Measures

The recommendations of this Phase I investigation are that the following remedial and/or maintenance items be carried out:

- (a) The animal hole and damaged grass area on the downstream embankment area should be repaired.
- (b) The small animal burrows in the right training embankment of the emergency spillway channel should be filled in and additional erosion protection should be provided on this training embankment.
- (c) The downstream channel should be cleared of all vegetation and growth.
- (d) Institute a program of annual technical inspection by a registered professional engineer qualified in dam design and inspection.
- (e) Develop an "Emergency Action Plan" that will include an effective preplanned downstream warning system, locations of emergency equipment, materials and manpower, authorities to contact and potential areas that require evacuation.

7.4 Alternatives

There are no practical alternatives to the above recommendations.

Downstream Face of Dam: Condition: 1. Good____. 2. Minor Repairs X.
3. Major Repairs____. 4. Urgent Repairs____.

Comments: _____

Emergency Spillway: Condition: 1. Good X. 2. Minor Repairs____.
3. Major Repairs____. 4. Urgent Repairs____.

Comments: _____

Water level @ time of inspection: 0.3 ft. above X below____.
top of dam____.
principal spillway X____.
other____.

Summary of Deficiencies Noted:

Growth [Trees and Brush] on Embankment____.
Animal Burrows and Washouts____.
Damage to slopes or top of dam____.
Cracked or Damaged Masonry____.
Evidence of Seepage____.
Evidence of Piping____.
Erosion X____.
Leaks____.
Trash and/or debris impeding flow____.
Clogged or blocked spillway____.
Other____.

INSPECTION REPORT - DAMS AND RESERVOIRS

Location: ~~City~~/Town Monterey.Dam No. 1-2-193-6.Name of Dam Lake Garfield.Inspected by: RDJordan-PFFezzie.Date of Inspection 3-7-74.

Owner/s: per: Assessors_____.

Prev. Inspection x.

Reg. of Deeds_____.

Pers. Contact_____.

1. Town of Monterey Monterey, MA
 Name St. & No. City/Town State Tel. No.

2. _____
 Name St. & No. City/Town State Tel. No.

3. _____
 Name St. & No. City/Town State Tel. No.

Caretaker [if any] e.g. superintendent, plant manager, appointed by absentee owner, appointed by multi owners.

_____ Name St. & No. City/Town State Tel. No.

No. of Pictures taken 3.

Degree of Hazard: [if dam should fail completely]*

1. Minor_____ 2. Moderate_____

3. Severe x_____ 4. Disastrous_____

*This rating may change as land use changes [future development]

Outlet Control: Automatic_____ Manual x_____Operative x yes _____ no.

Comments:_____

Upstream Face of Dam: Condition:

1. Good x_____ 2. Minor Repairs_____

3. Major Repairs_____ 4. Urgent Repairs_____

Comments:_____

APPENDIX B
ENGINEERING DATA
INDEX

1. Design and Construction Records:

The following design records are kept on file by the consulting firm of R.G. Brown Associates of Pittsfield, Massachusetts. Their office is located in the Berkshire Commons Complex on the third floor:

Preliminary Report Titled "Report of Study, Lake Garfield, Monterey, Massachusetts", dated January 20, 1970.

Hydrology and Hydraulics
Soil Mechanics
Structural Design
Survey Calculations
Quantity Computations

Construction records consisting of correspondence, soil testing reports, and pay estimates are kept on file by the Commonwealth of Massachusetts, DPW, Division of Waterways, District 1, located at 270 Pittsfield Road in Lenox, Massachusetts.

2. Inspection Reports (appended)

<u>Date</u>	<u>Inspection Agency</u>
3/7/74	Mass. DPW, Division of Waterways, District 1

3. Design Drawings

<u>Page No.</u>	<u>Description</u>
B-1	Title Sheet and Index
B-2	Plan of Dam and Lake
B-3	Section - Profile - Seeding Plan of Dam
B-4	Plan - Profile Principal Spillway
B-5	Dam Sections - Miscellaneous
B-6	Details - Drainage
B-7	Details - Riser
B-8	Details - Trash Racks
B-9	Details - Dam - Miscellaneous
B-10	Plan - Profile - Road Relocation - Sta. 38+50 to 48+00
B-11	Plan - Profile - Road Relocation - Sta. 48+00 to 53+00
B-12	Profile Culverts - Pond Drain
B-13	Details - Seeding Plan of Road Relocation
B-14	Log of Test Pits and Borings I
B-15	Log of Test Pits and Borings II
B-16	Log of Test Pits and Borings III

APPENDIX B
ENGINEERING DATA

INSPECTION CHECK LIST

PROJECT Lake Garfield Dam

DATE _____

PROJECT FEATURE _____

NAME _____

DISCIPLINE _____

NAME _____

AREA EVALUATED

CONDITION

CURTAIN WORKS - SERVICE BRIDGE

N/A

a. Super Structure

Bearings

Anchor Bolts

Bridge Seat

Longitudinal Members

Under Side of Deck

Secondary Bracing

Deck

Drainage System

Railings

Expansion Joints

Paint

b. Abutment & Piers

General Condition of Concrete

Alignment of Abutment

Approach to Bridge

Condition of Seat & Backwall



INSPECTION CHECK LIST

PROJECT Lake Garfield Dam DATE

PROJECT FEATURE NAME

DISCIPLINE NAME

AREA EVALUATED	CONDITION
<u>OUTLET WORKS - SPILLWAY WEIR, APPROACH AND DISCHARGE CHANNELS</u>	Emergency Spillway
a. Approach Channel	
General Condition	Heavy grass and legumes with some reed type growth
Loose Rock Overhanging Channel	N/A
Trees Overhanging Channel	None
Floor of Approach Channel	Very wet, standing water 1"-2" deep over most of approach
b. Weir and Training Walls	N/A
General Condition of Concrete	
Rust or Staining	
Spalling	
Any Visible Reinforcing	
Any Seepage or Efflorescence	
Drain Holes	
c. Discharge Channel	
General Condition	Heavy grass and legumes, left side embankment sloughing about 108 ft. D.S. of control section.
Loose Rock Overhanging Channel	None
Trees Overhanging Channel	None
Floor of Channel	Mostly dry, but left side embankment toe area is wet
Other Obstructions	None
Notes:	
1. Left embankment sloughing appears to to have occurred a number of years ago and now stablized.	3. The right side training embankment should have additional erosion protection.
2. Side-hill seepage from left abutment appears to be cause of wet conditions	4. Small animal burrow found in right training embankment.

INSPECTION CHECK LIST

PROJECT Lake Garfield Dam

DATE _____

PROJECT FEATURE _____

NAME _____

DISCIPLINE _____

NAME _____

AREA EVALUATED	CONDITION
<u>OUTLET WORKS - OUTLET STRUCTURE AND OUTLET CHANNEL</u>	
General Condition of Concrete	N/A
Rust or Staining	N/A
Spalling	N/A
Erosion or Cavitation	N/A
Visible Reinforcing	N/A
Any Seepage or Efflorescence	N/A
Condition at Joints	60"Ø pipe joints appear good
Drain holes	N/A
Channel	Riprap lined stilling basin good condition
Loose Rock or Trees Overhanging Channel	None
Condition of Discharge Channel	Some vegetation growth in channel downstream of stilling basin Note: Stilling basin is 38+ feet wide and 80+ feet long. No sedimentation visible on bottom.

INSPECTION CHECK LIST

PROJECT Lake Garfield Dam

DATE _____

PROJECT FEATURE _____

NAME _____

DISCIPLINE _____

NAME _____

AREA EVALUATED

CONDITION

OUTLET WORKS - TRANSITION AND CONDUIT

General Condition of Concrete

60" Ø R.C.P. Conduit

Pipe Very Good

Rust or Staining on Concrete

None Visible

Spalling

None Visible

Erosion or Cavitation

None

Cracking

None

Alignment of Monoliths

Good

Alignment of Joints

Good alignment - dry joints

Numbering of Monoliths

N/A

INSPECTION CHECK LIST

PROJECT Lake Garfield Dam : DATE _____
 PROJECT FEATURE _____ NAME _____
 DISCIPLINE _____ NAME _____

AREA EVALUATED	CONDITION
<u>OUTLET WORKS - TRANSITION AND CONDUIT</u>	60" Ø R.C.P. Conduit
General Condition of Concrete	Pipe Very Good
Rust or Staining on Concrete	None Visible
Spalling	None Visible
Erosion or Cavitation	None
Cracking	None
Alignment of Monoliths	Good
Alignment of Joints	Good alignment - dry joints
Numbering of Monoliths	N/A

INSPECTION CHECK LIST

PROJECT Lake Garfield Dam

DATE _____

PROJECT FEATURE _____

NAME _____

DISCIPLINE _____

NAME _____

AREA EVALUATED	CONDITION
<u>OUTLET WORKS - CONTROL TOWER</u>	(Principal Spillway)
a. Concrete and Structural	
General Condition	Very Good
Condition of Joints	Very Good
Spalling	None
Visible Reinforcing	None
Rusting or Staining of Concrete	None
Any Seepage or Efflorescence	None
Joint Alignment	Very Good
Unusual Seepage or Leaks in Gate Chamber	The gate was open at the time of inspection
Cracks	None
Rusting or Corrosion of Steel	None
b. Mechanical and Electrical	No Electrical
~~Air Vents~~	Pond Drain Sluice Gate:
~~Float Wells~~	1. Rodney Hunt 27291-2 S-5002
~~Crane Hoist~~	gate operating mechanism looks good and has been operated by the Owner to drain the pond.
~~Elevator~~	
~~Hydraulic System~~	None: There are no other Mechanical or Electrical features.
Service Gates	
~~Emergency Gates~~	
~~Lightning Protection System~~	
~~Emergency Power System~~	
~~Wiring and Lighting System in Gate Chamber~~	

INSPECTION CHECK LIST -

PROJECT Lake Garfield Dam

DATE _____

PROJECT FEATURE _____

NAME _____

DISCIPLINE _____

NAME _____

AREA EVALUATED

CONDITION

OUTLET WORKS - INTAKE CHANNEL AND INTAKE STRUCTURE

a. Approach Channel

Slope Conditions

Bottom Conditions

Rock Slides or Falls

Log Boom

Debris

Condition of Concrete Lining

Drains or Weep Holes

b. Intake Structure

Condition of Concrete

Stop Logs and Slots

N/A



INSPECTION CHECK LIST

PROJECT Lake Garfield Dam

DATE _____

PROJECT FEATURE _____

NAME _____

DISCIPLINE _____

NAME _____

AREA EVALUATED	CONDITIONS
<u>DAM EMBANKMENT</u>	
Crest Elevation	1295.08 - from Construction Records
Current Pool Elevation	Pond Drained
Maximum Impoundment to Date	Unknown
Surface Cracks	None Visible
Pavement Condition	N/A
Movement or Settlement of Crest	None Apparent
Lateral Movement	None Apparent
Vertical Alignment	Good
Horizontal Alignment	Good
Condition at Abutment and at Concrete Structures	Good
Indications of Movement of Structural Items on Slopes	None Apparent
Trespassing on Slopes	Slight damage to turf on D.S. embankment
Vegetation on Slopes	Well groomed grass
Sloughing or Erosion of Slopes or Abutments	None Apparent
Rock Slope Protection - Riprap Failures	U.S. embankment has 2" washed gravel surface - good condition
Unusual Movement or Cracking at or near Tees	None visible
Unusual Embankment or Downstream Seepage	None visible
Piping or Boils	None
Foundation Drainage Features	Foundation drains to stilling basin. Drain outlets submerged in stilling basin.
Tee Drains	
Instrumentation System	N/A

PARTY ORGANIZATION

DATE 11/14/79

WEATHER Cloudy-40°F

W.S. ELEV. _____ U.S. _____ DN.S. _____

1. <u>John W. Powers, P.E., Proj. Manager</u>	6. _____
Hydrology/	
2. <u>George H. McDonnell, P.E., Hydraulics</u>	7. _____
3. <u>Edward A. Moe, P.E., Soils/Hydraulics</u>	8. _____
4. <u>Omer H. Dumais, Jr., P.E., Civil</u>	9. _____
5. _____	10. _____

REMARKS

1. All project features were inspected by all party members
- 2.
- 3.
- 4.
- 5.
- 6.
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- 8.
- 9.
- 10.

APPENDIX A
INSPECTION CHECKLIST

12. Remarks & Recommendations: [Fully Explain]

In general, the dam is in very good condition. The earthen emergency spillway is covered with a good turf cover. The embankments are stable, showing no signs of settlement or sloughing.

There is one minor deficiency located on the downstream slope. An area extending up the slope and about six feet wide has had all the turf removed by motor bike wheels. This strip is subject to erosion and should be reshaped and reseeded as early as possible.

It was noted that there is no protective cover on the drawdown gate stem. The town should be advised to cover the stem to protect the mechanism from the elements. The town should also be advised to "exercise" the gate periodically to insure proper functioning in case of an emergency.

In my opinion, the dam , in its present condition, is safe.

A description of the structure was submitted in 1972. There are no changes to be noted.

For location see Topo 6-A.

13.

Overall Condition:

1. Safe X
2. Minor repairs needed _____
3. Conditionally safe - major repairs needed _____
4. Unsafe _____
5. Reservoir impoundment no longer exists [explain]
Recommend removal from inspection list _____

COMMONWEALTH OF MASSACHUSETTS

DEPARTMENT OF PUBLIC WORKS

DIVISION OF WATERWAYS

PROPOSED EARTH DAM

AND

RELOCATION OF TYRINGHAM ROAD

AT

LAKE GARFIELD

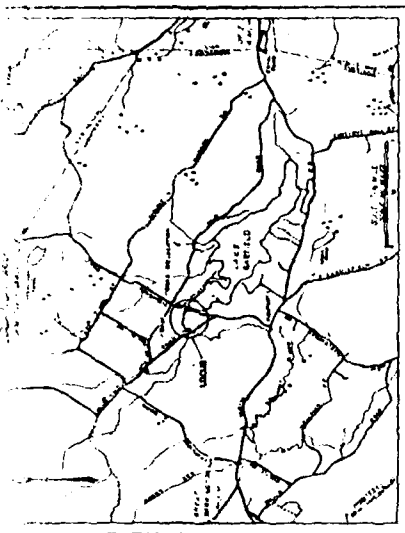
MONTEREY, MASSACHUSETTS

1971

AUTHORIZED UNDER CHAP. 262 ACTS OF 1969

INDEX

1. TITLE SHEET & INDEX
2. PLAN OF DAM & LAKE
3. SECTION-PROFILE-SEEDING PLAN OF DAM
4. PLAN-PROFILE OF PRINCIPAL SPILLWAY
5. DAM SECTIONS - MISCELLANEOUS
6. DETAILS - DRAINAGE
7. DETAILS - RISER
8. DETAILS - TRASH RACKS
9. DETAILS - DAM - MISCELLANEOUS
10. PLAN-PROFILE - ROAD RELOCATION - STA. 38+50 TO STA. 48+00
11. PLAN-PROFILE - ROAD RELOCATION - STA. 48+00 TO STA. 53+00
12. PROFILE - CULVERTS & POND DRAIN
13. DETAILS - SEEDING PLAN OF ROAD RELOCATION
14. LOSS OF TEST PITS & BORINGS I
15. LOSS OF TEST PITS & BORINGS II
16. LOSS OF TEST PITS & BORINGS III



DRAINAGE AREA
2,592 ACRES
TOP OF DAM
EL. 1294.75
EMERGENCY SPILLWAY CREST
EL. 1288.00
WATER SURFACE AREA
170 ACRES
HEIGHT OF DAM
19 FEET
FLOODWATER RETARDING STORAGE
770 ACRES FEET
TOTAL STORAGE
1090 ACRES FEET
DESIGN HIGH WATER
EL. 1299.8



PROPOSED EARTH DAM
RELOCATION OF TYRINGHAM ROAD
LAKE GARFIELD
MONTEREY, MASS.

TITLE SHEET & INDEX

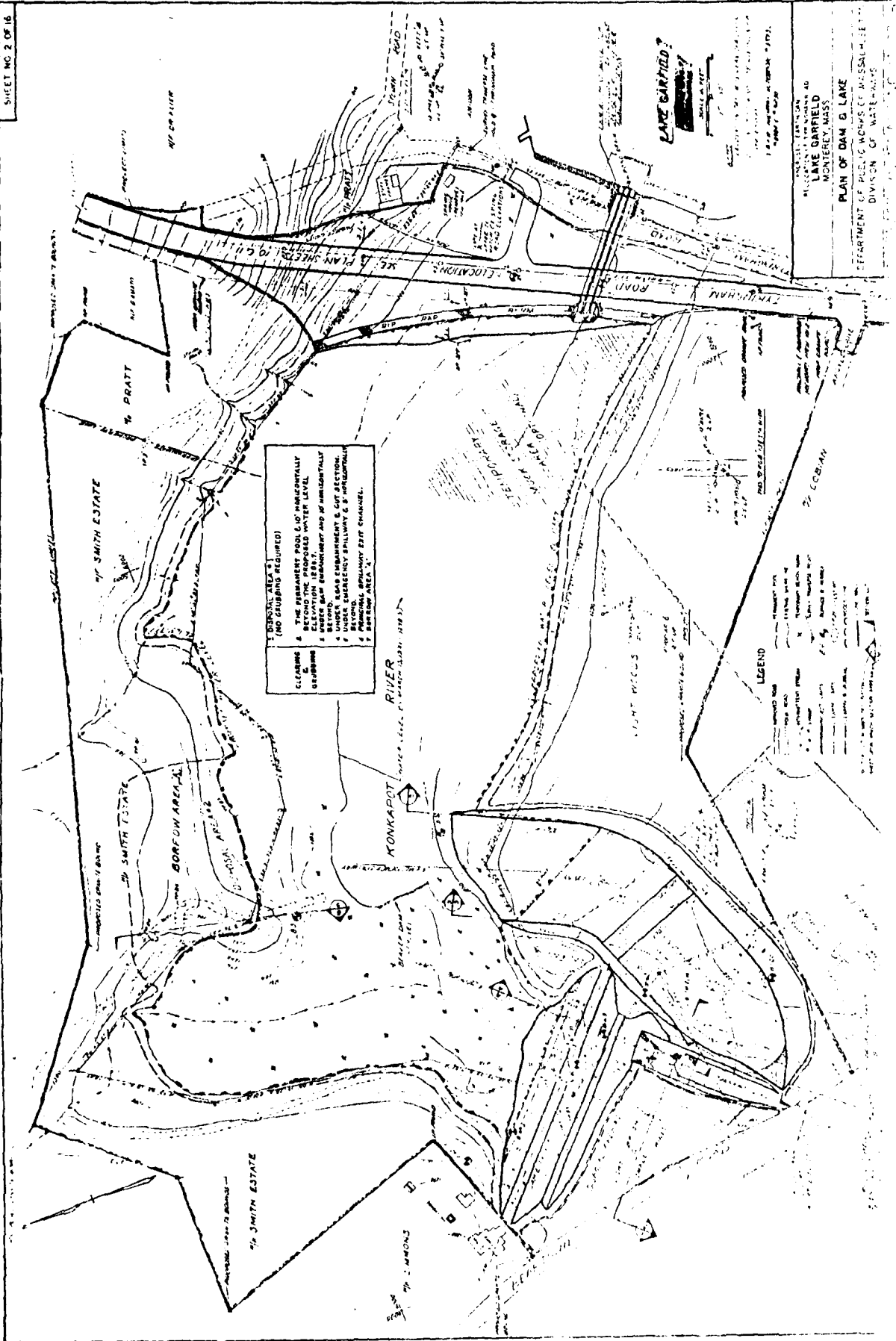
DEPARTMENT OF PUBLIC WORKS OF MASSACHUSETTS
DIVISION OF WATERWAYS

ADVISORY BOARD ON WATERWAYS
JULY 1971

OCTOBER 1971
REVISION NO. 1

CONTRACT NO. 2738 ACC NO. 04893-4

200-9-1000-10



LEGEND

1. DRAINAGE AREA (NO GROUPING REQUIRED)

2. THE PERMANENT POOL END HORIZONTALLY BEYOND THE PROPOSED WATER LEVEL

3. ELEVATION 1000.0

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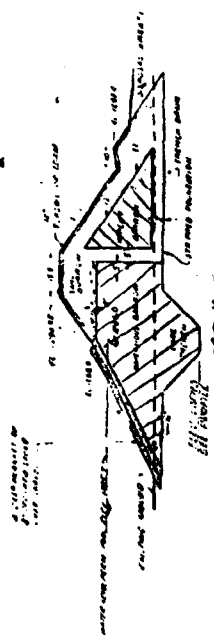
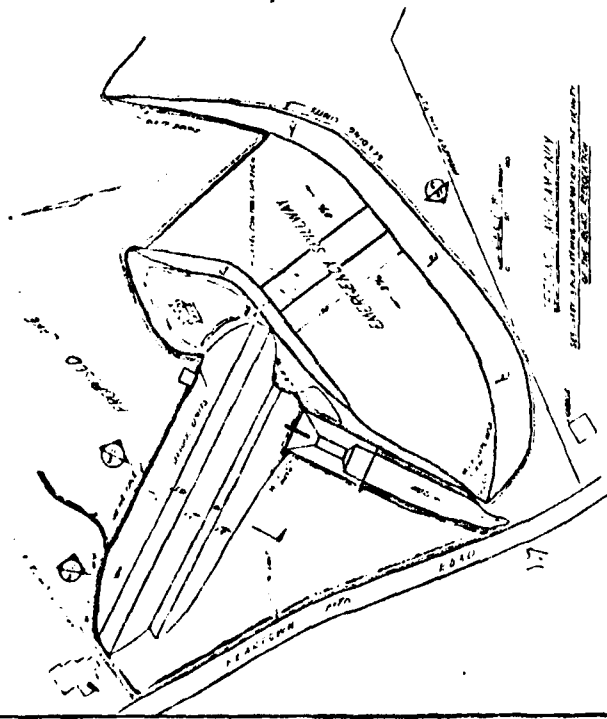
97. ELEVATION 1000.0

98. ELEVATION 1000.0

99. ELEVATION 1000.0

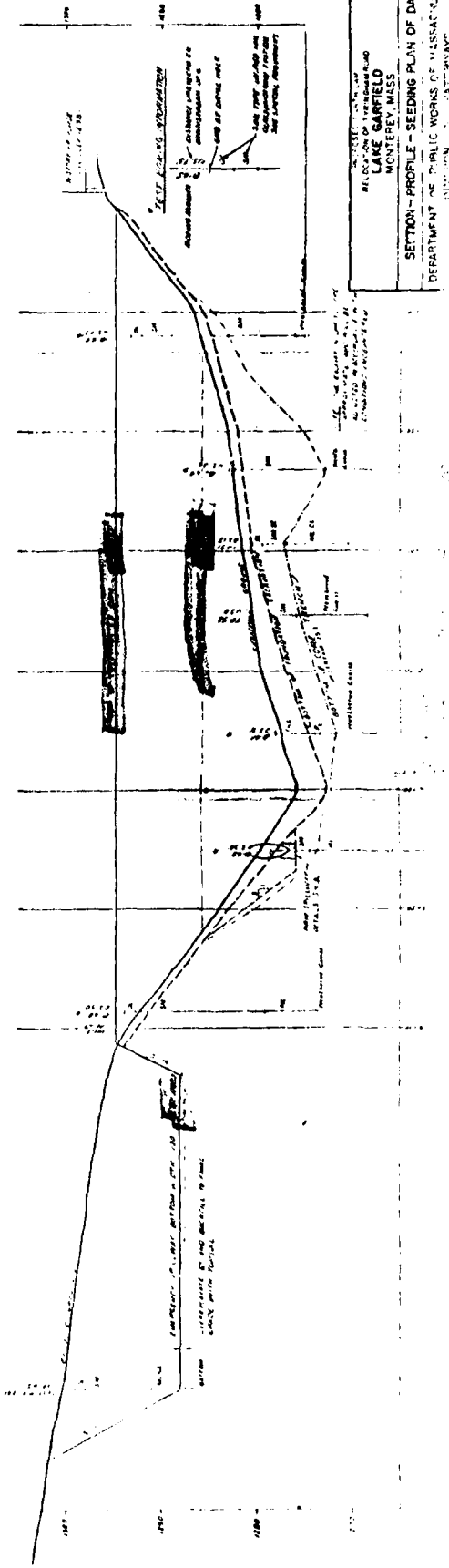
100. ELEVATION 1000.0

RECEIVED AT THE MONROE AD
LAKE GARFIELD
MONTEREY, MASS.
PLAN OF DAM & LAKE
DIVISION OF WATERWAYS
DEPARTMENT OF PUBLIC WORKS OF MASSACHUSETTS

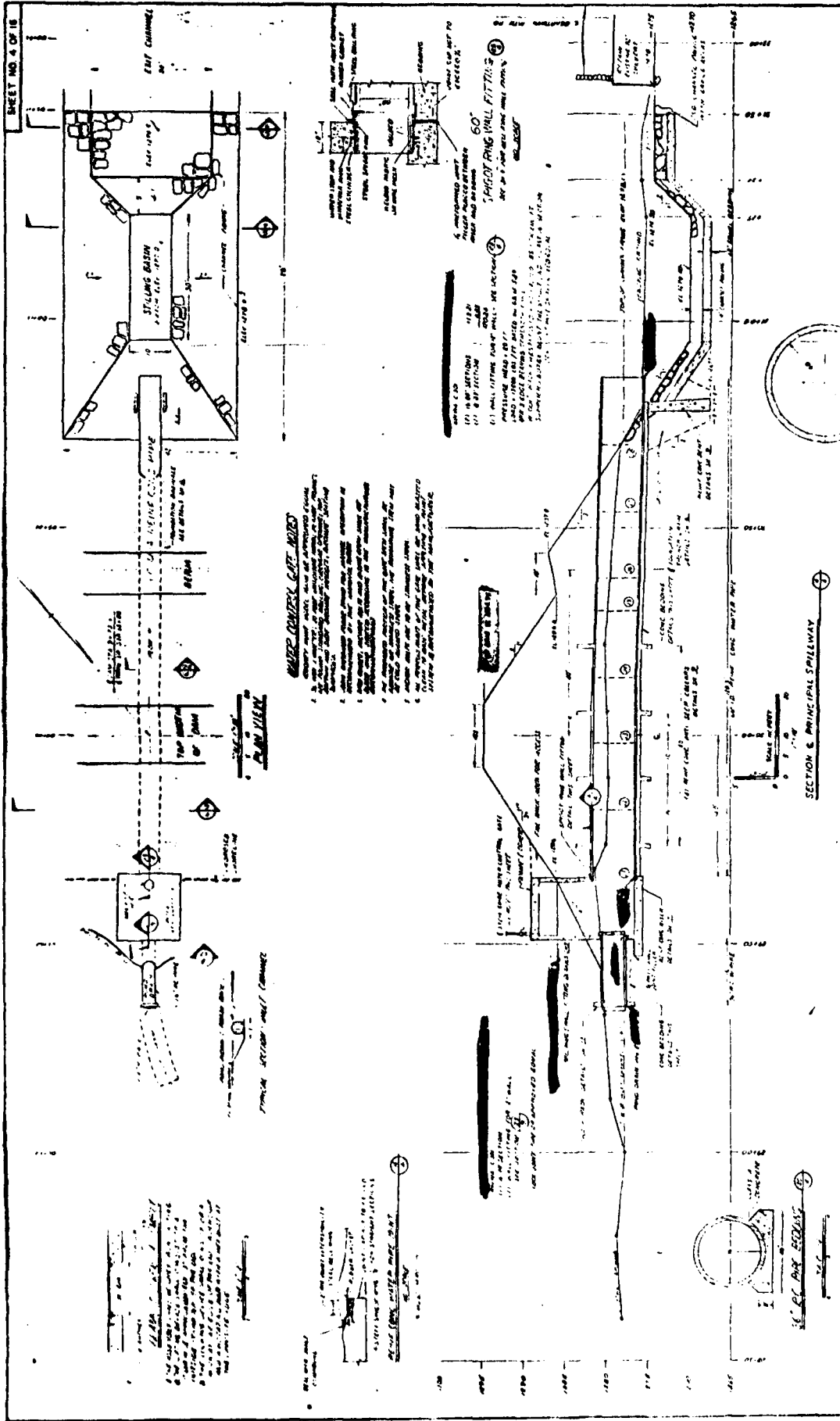


CROSS SECTION

ITEM	DESCRIPTION	QUANTITY	UNIT	PRICE	TOTAL
1	Concrete dam structure	10,000	cuyd	1.00	10,000.00
2	Concrete spillway structure	5,000	cuyd	1.00	5,000.00
3	Concrete abutment structure	2,000	cuyd	1.00	2,000.00
4	Concrete foundation structure	1,000	cuyd	1.00	1,000.00
5	Concrete dam structure	10,000	cuyd	1.00	10,000.00
6	Concrete spillway structure	5,000	cuyd	1.00	5,000.00
7	Concrete abutment structure	2,000	cuyd	1.00	2,000.00
8	Concrete foundation structure	1,000	cuyd	1.00	1,000.00
9	Concrete dam structure	10,000	cuyd	1.00	10,000.00
10	Concrete spillway structure	5,000	cuyd	1.00	5,000.00
11	Concrete abutment structure	2,000	cuyd	1.00	2,000.00
12	Concrete foundation structure	1,000	cuyd	1.00	1,000.00



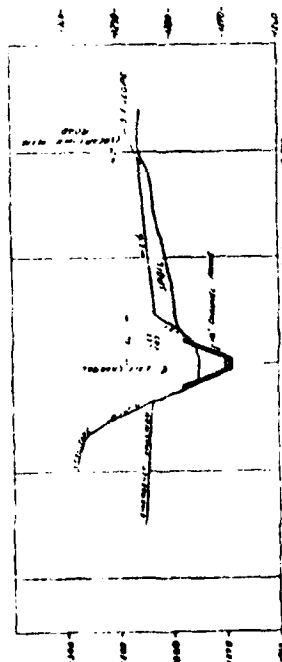
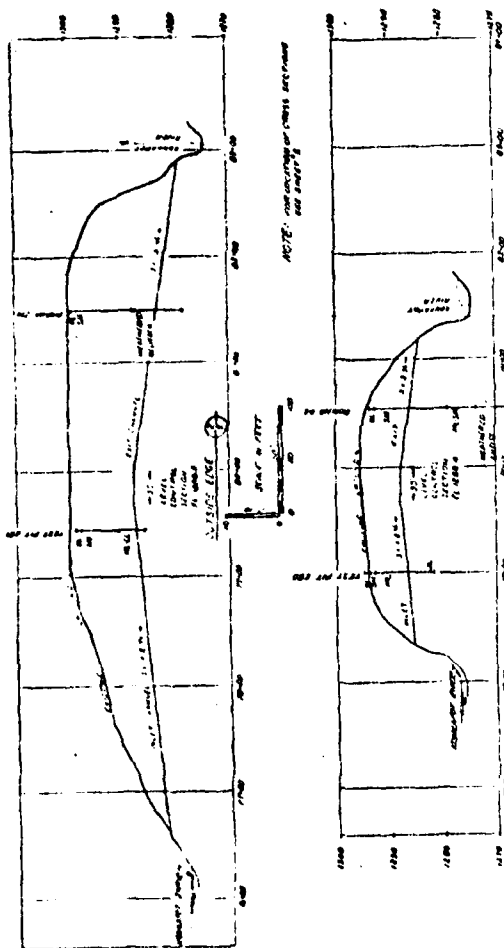
SECTION - PROFILE - SEEDING PLAN OF DAM
DEPARTMENT OF PUBLIC WORKS OF MASSACHUSETTS



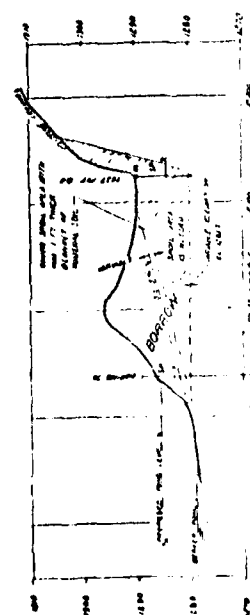
PROJECT TITLE: PLAN-PROFILE OF THE PRINCIPAL SPILLWAY
 LOCATION: MONTEREY, CALIFORNIA
 DRAWN BY: [Name]
 CHECKED BY: [Name]
 DATE: [Date]

ITEM	DESCRIPTION	QUANTITY	UNIT
1	Concrete Spillway Structure	1000	cu yd
2	Water Control Gate	1	unit
3	Stilling Basin	1	unit
4	60° Spaced Ring Wall Fitting	1	unit
5	Water Control Gate	1	unit
6	Stilling Basin	1	unit
7	60° Spaced Ring Wall Fitting	1	unit
8	Water Control Gate	1	unit
9	Stilling Basin	1	unit
10	60° Spaced Ring Wall Fitting	1	unit

SECTION A: PRINCIPAL SPILLWAY
 SECTION B: PRINCIPAL SPILLWAY
 SECTION C: PRINCIPAL SPILLWAY
 SECTION D: PRINCIPAL SPILLWAY
 SECTION E: PRINCIPAL SPILLWAY
 SECTION F: PRINCIPAL SPILLWAY
 SECTION G: PRINCIPAL SPILLWAY
 SECTION H: PRINCIPAL SPILLWAY
 SECTION I: PRINCIPAL SPILLWAY
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 SECTION V: PRINCIPAL SPILLWAY
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 SECTION Z: PRINCIPAL SPILLWAY

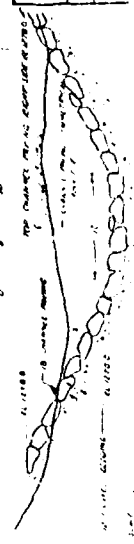
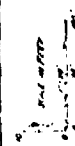


CROSS SECTION - STA. 31.55 - REMARKS: SUSPENSION AREA



CROSS SECTION - STA. 31.55 - REMARKS: SUSPENSION AREA

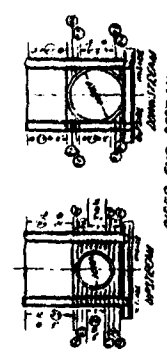
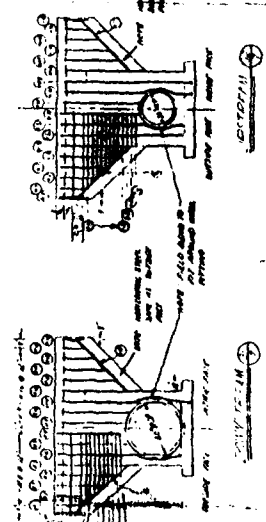
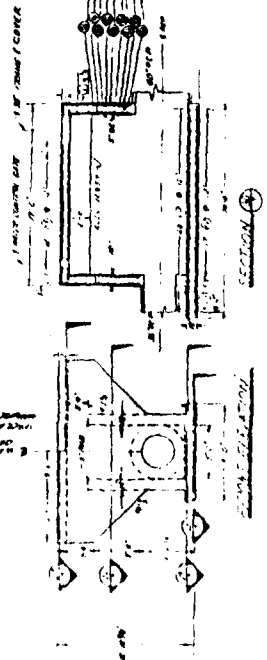
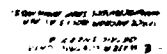
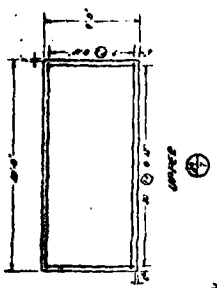
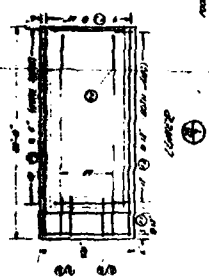
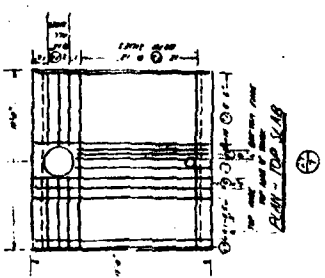
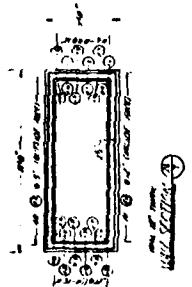
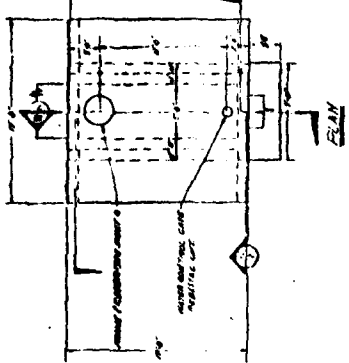
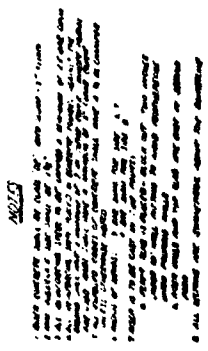
ENERGIZED SPILLWAY PROFILES



MASSACHUSETTS
DEPARTMENT OF PUBLIC WORKS
DIVISION OF WATERWAYS
DAM SECTIONS - MISCELLANEOUS
LAKE GARFIELD
MONTEREY, MASS.

37501

The image shows a page from a Braille manuscript, likely a musical score. It contains two tables of musical notation. The top table has 10 rows, and the bottom table has 8 rows. Each row contains musical notes and rests in Braille format. The notation is arranged in a structured, grid-like manner, typical of early Braille musical notation.

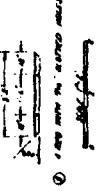
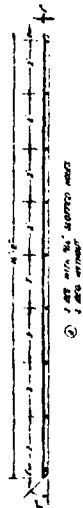
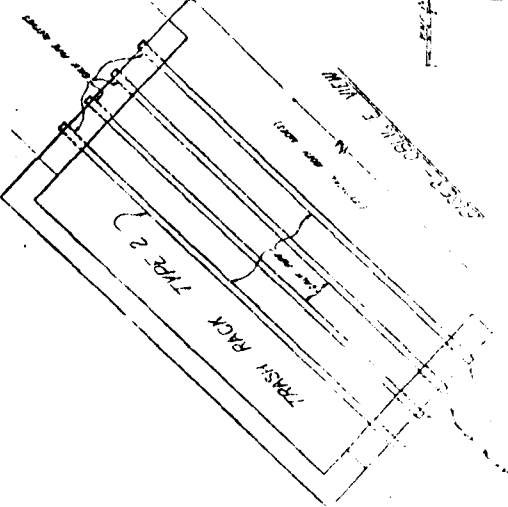
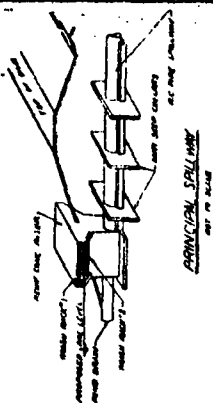
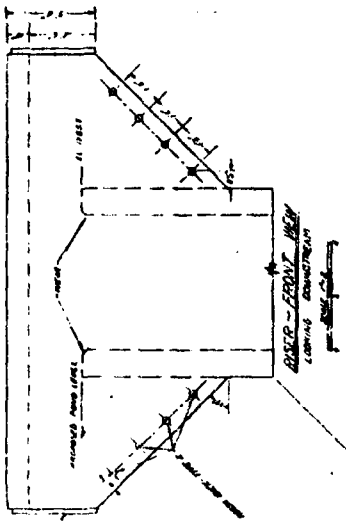
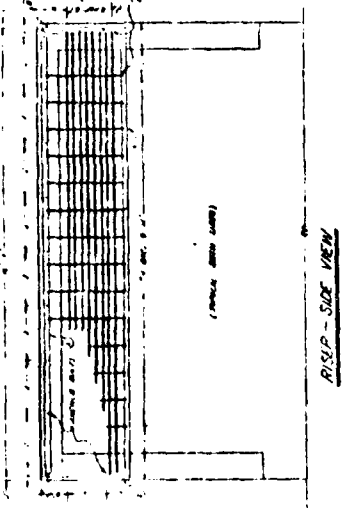


ENTER END SECTION

PROPOSED EARTH DAM
LOCATION OF TRINGHAM ROAD
LAKE GARFIELD
MONTEREY, MASS

DETAILS - RIDER

DEPARTMENT OF PUBLIC WORKS OF MASSACHUSETTS
DIVISION OF WATERWAYS



STANDARD DETAILS - TRASH RACK TYPE 1

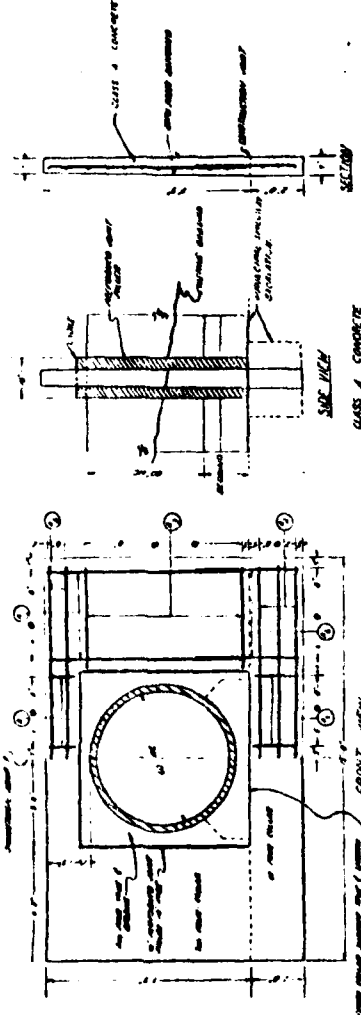
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2. RACK	1	EA	1	EA
3. RACK	1	EA	1	EA
4. RACK	1	EA	1	EA
5. RACK	1	EA	1	EA
6. RACK	1	EA	1	EA
7. RACK	1	EA	1	EA
8. RACK	1	EA	1	EA
9. RACK	1	EA	1	EA
10. RACK	1	EA	1	EA

NOTE: TRASH RACKS ARE CONSTRUCTED OF GALVANIZED STEEL.

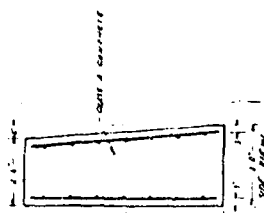
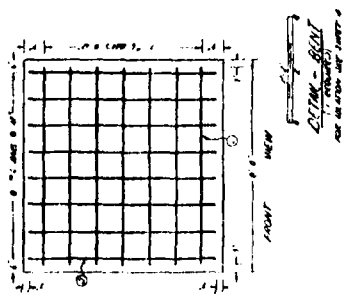
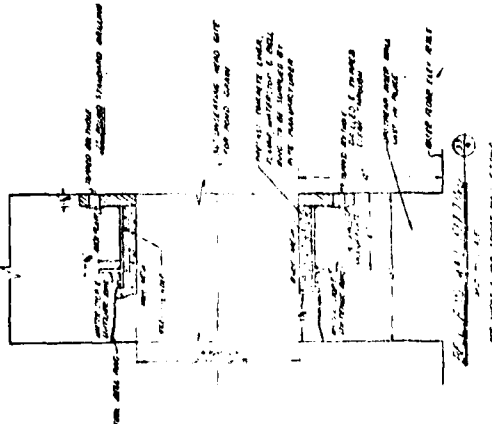
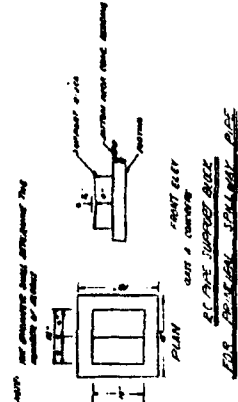
DESIGNED BY: [Name]
 CHECKED BY: [Name]
 DATE: [Date]
 DEPARTMENT OF PUBLIC WORKS OF MASSACHUSETTS
 DIVISION OF [Division Name]

ANTI-SLIP COLLAR ONLY

NO. 1	NO. 2	NO. 3	NO. 4	NO. 5	NO. 6	NO. 7	NO. 8	NO. 9	NO. 10



ANTI-SLIP COLLAR (REQUIRED)



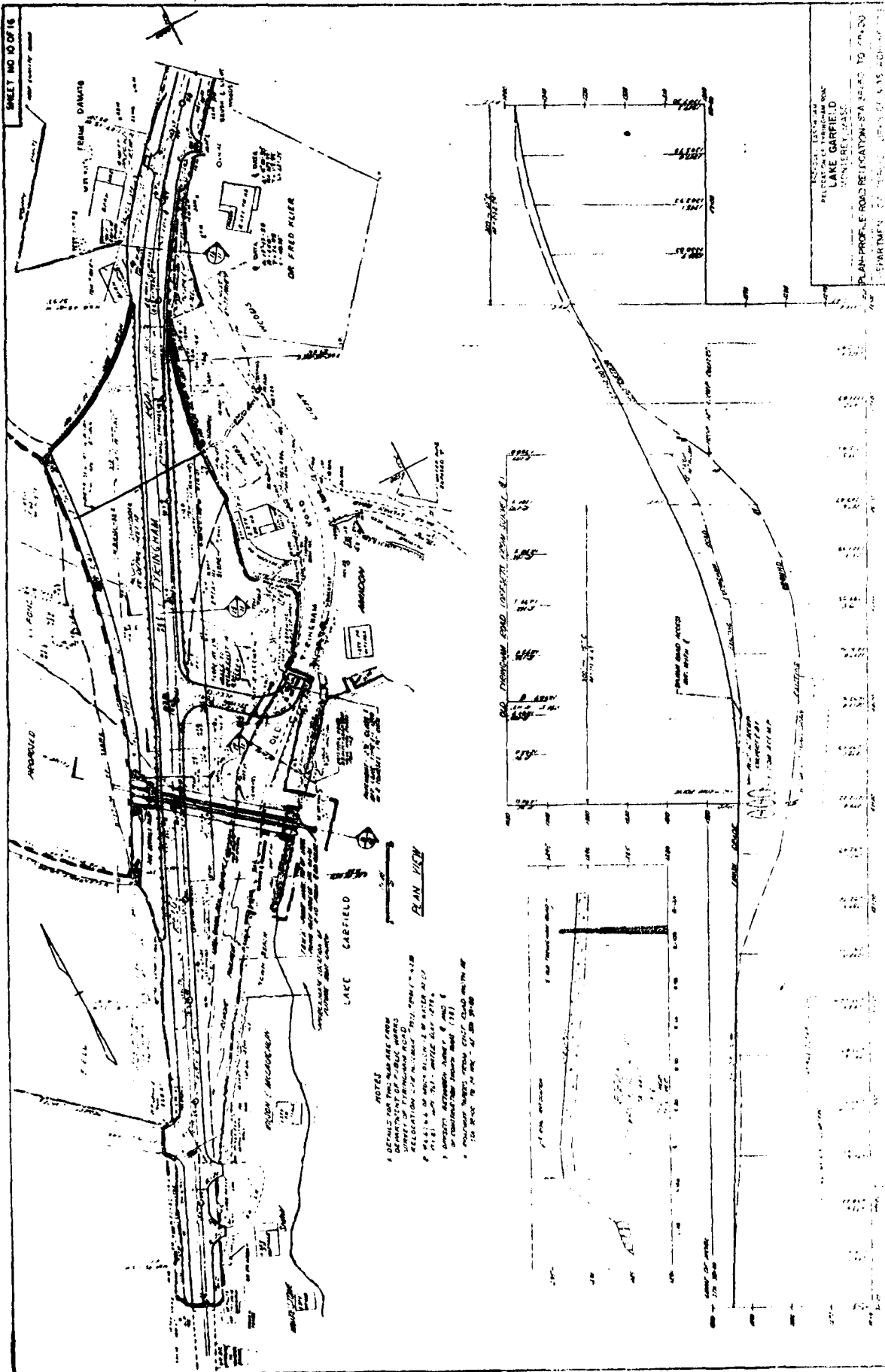
DEPARTMENT OF PUBLIC WORKS OF MASSACHUSETTS

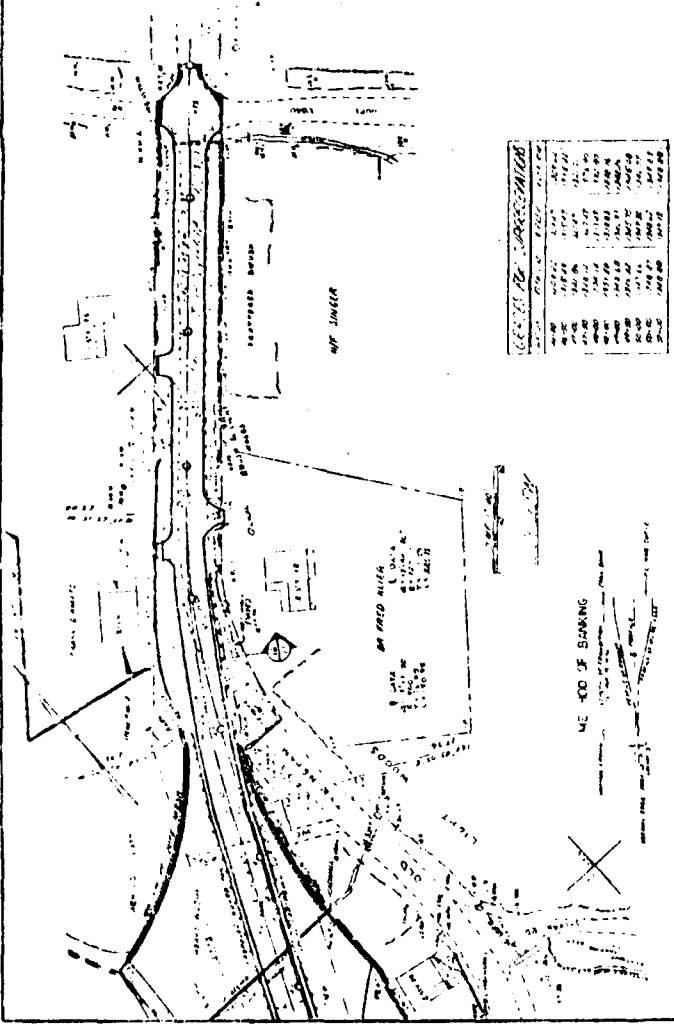
DETAILS - DAM - MISC.

UNIVERSITY OF MASSACHUSETTS

UNIVERSITY OF MASSACHUSETTS

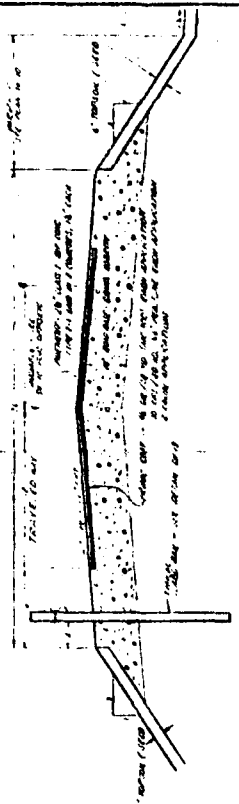
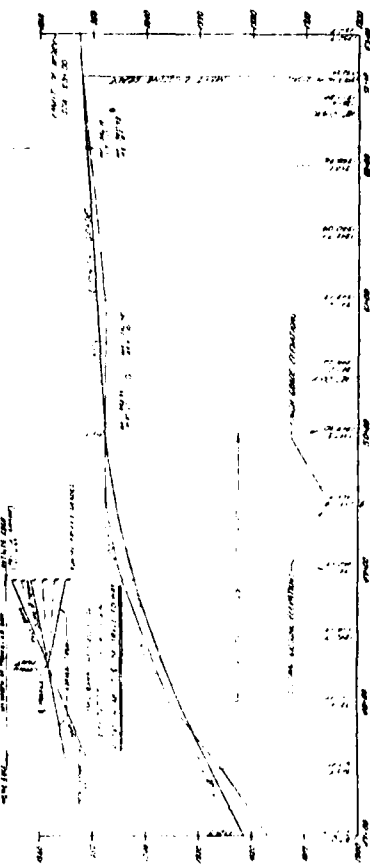
UNIVERSITY OF MASSACHUSETTS



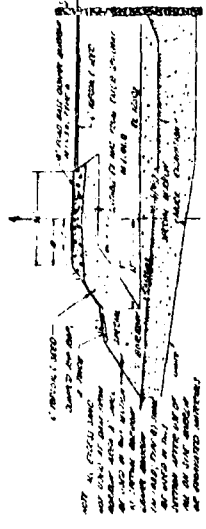


PROPOSED ROAD

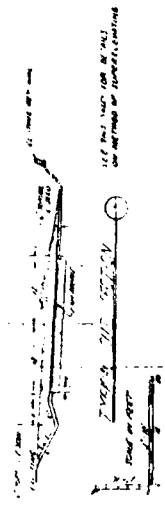
STATION	1+00	2+00	3+00	4+00	5+00	6+00	7+00	8+00	9+00	10+00
ELEVATION	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00
WIDTH	10.00	10.00	10.00	10.00	10.00	10.00	10.00	10.00	10.00	10.00
AREA	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00



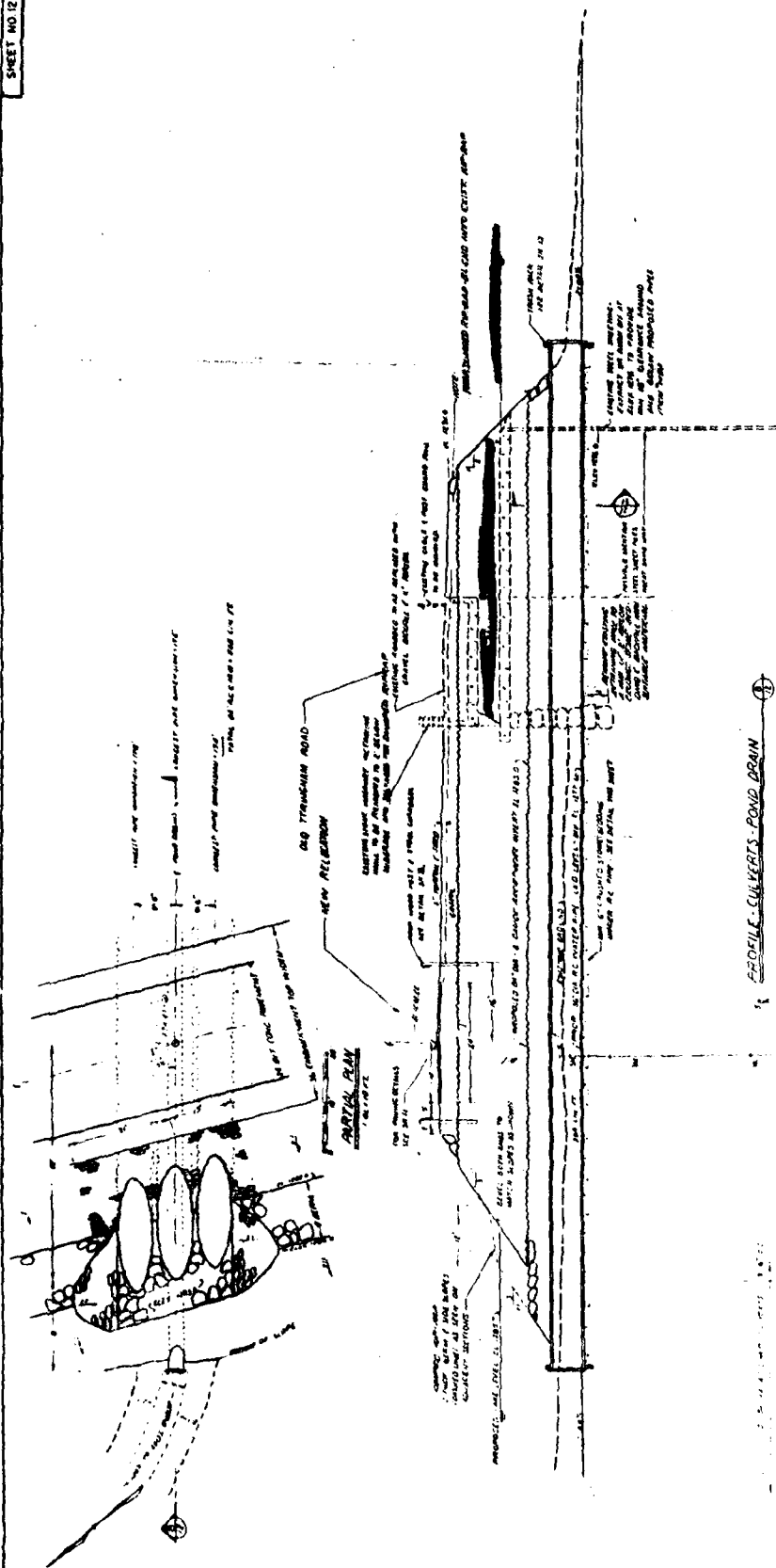
TYPICAL PAVEMENT SECTION



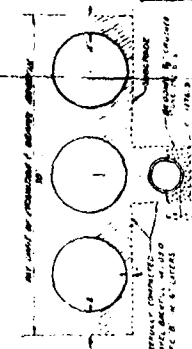
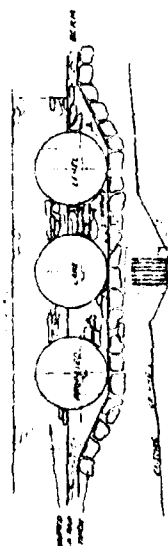
TYPICAL BRIDGE SECTION



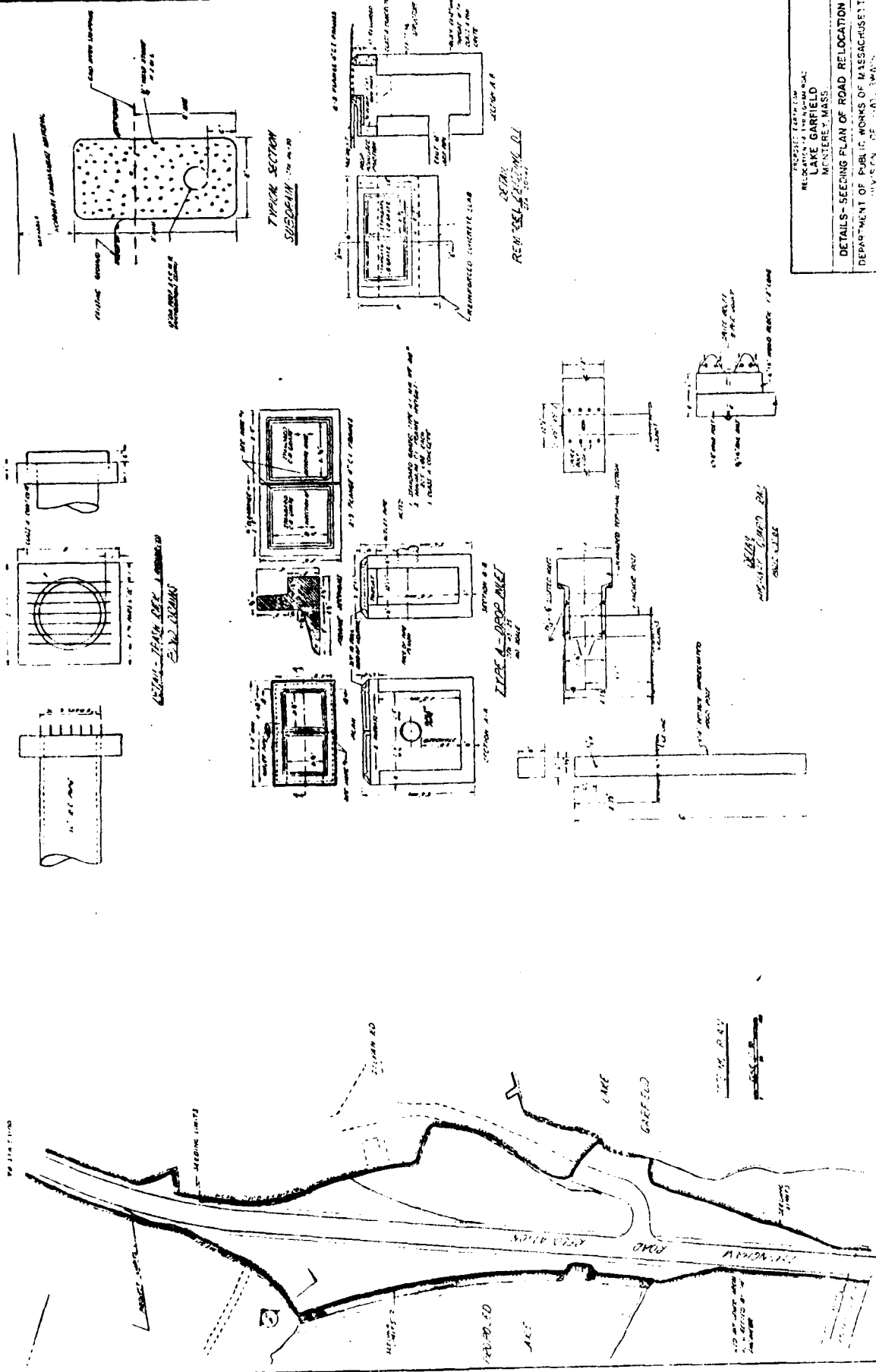
PROPOSED ELEVATION
RELOCATION OF THE MAIN ROAD
LAKE GARFIELD
MONTREY, MASS.
PLAN PROFILE-ROAD RELOCATION - STA. 48+00 TO 53+00
DEPARTMENT OF PUBLIC WORKS OF MASSACHUSETTS
DIVISION OF HIGHWAYS



PROFILE - CULVERTS - POND DRAIN



PROPOSED EARTH DAM
RELOCATED CULVERTS
LAKE GARFIELD
MONTEREY, MASS.
PROFILE - CULVERTS - POND DRAIN
DEPARTMENT OF PUBLIC WORKS OF MASSACHUSETTS
DIVISION OF HIGHWAYS



REQUEST: 7-20-73
RELOCATION OF HIGHWAY 1
LAKE GARFIELD
MONTEREY, MASS.
DETAILS - SEEDING PLAN OF ROAD RELOCATION
DIVISION OF ROAD BUILDING
DEPARTMENT OF PUBLIC WORKS OF MASSACHUSETTS

Hydrologic / Hydraulic ComputationsIndexPage

Size & Hazard Classification

D-1

Spillway Adequacy Analysis

D-5

Summary of Downstream Conditions
With Dam Failure During PMF

D-24

Downstream Conditions With
Dam Failure

D-32

Culvert Capacities

D-70

APPENDIX D

HYDROLOGIC AND HYDRAULIC COMPUTATIONS

15

side of triple barrel
art under Tyringham
which crosses impound-
ment.



16

side of triple barrel
showing typical
accumulation.



17

ndament area looking
down of Tyringham Road
arts towards dam
tion. (dam not in photo)





Photo 12

Emergency spillway channel
left side excavated embank-
ment.

Note: Sloughing occurring
but not visible in photo

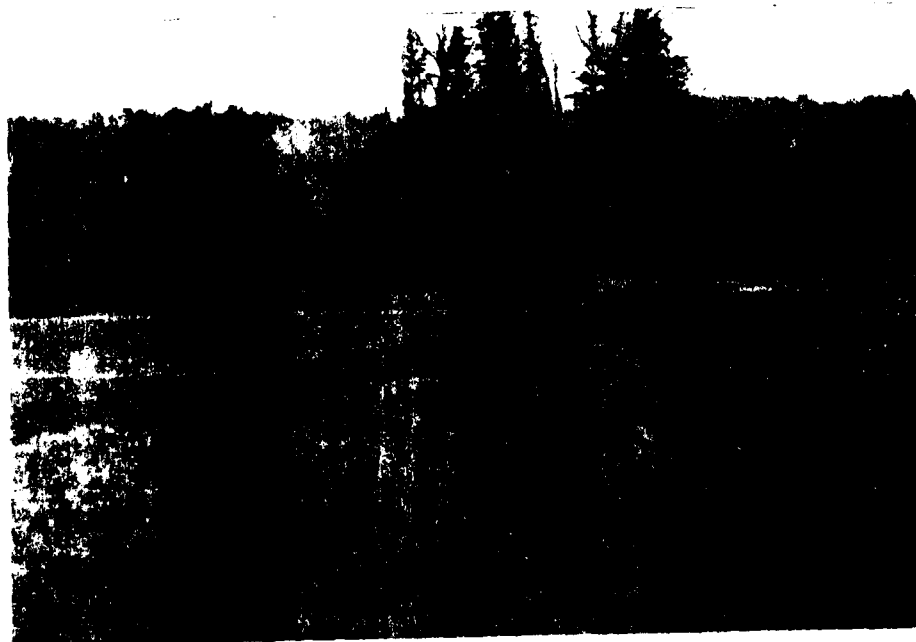


Photo 13

Emergency spillway right
side training embankment
and left side of dam.



Photo 14

Emergency spillway
discharge channel
looking from top of
dam, left side.



Fig. 9

Principal spillway outlet
shift at stilling basin.



Fig. 10

Stilling basin and discharge
channel looking downstream
from outlet conduit.



Fig. 11

Spillway approach
looking from left
of discharge channel.

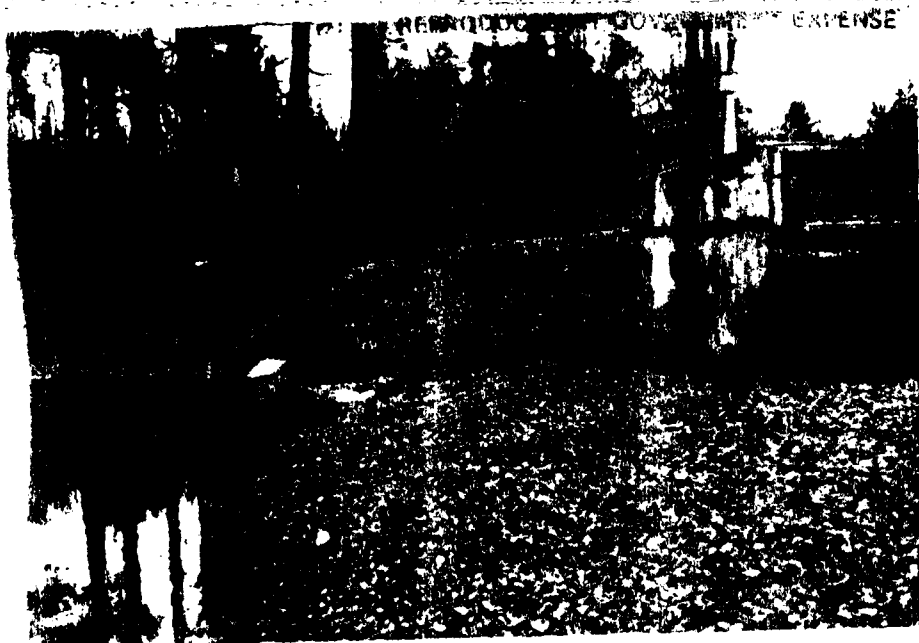


Photo 7

Pond drain inlet conduit and
upstream side of principal
spillway drop inlet structure.



Photo 8

Pond drain control sluice gate
located in principal spillway
drop inlet structure.

photo 4

downstream embankment
sloping towards left
attest from right
side.



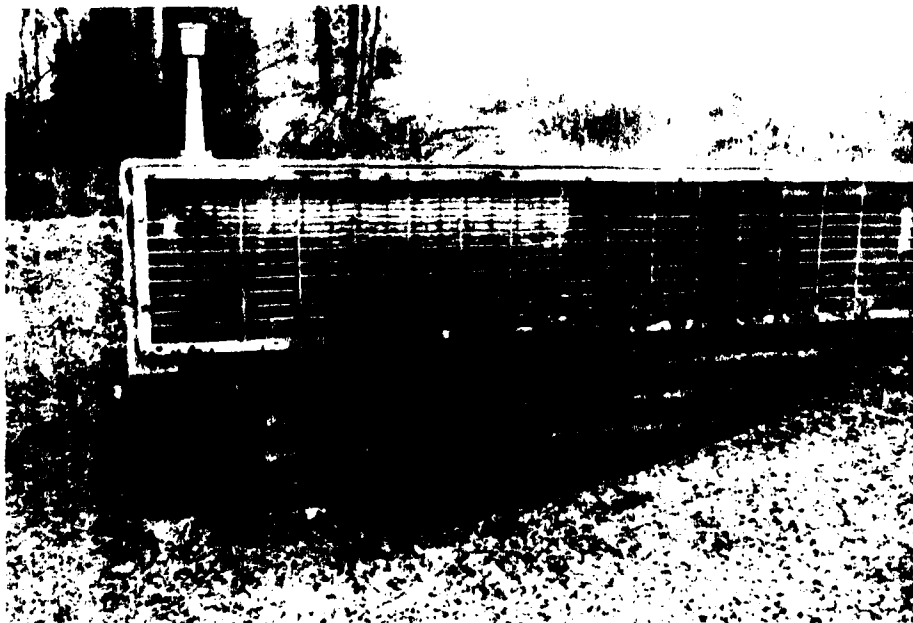
photo 5

principal spillway drop
structure.



photo 6

view of principal
spillway drop inlet
structure.



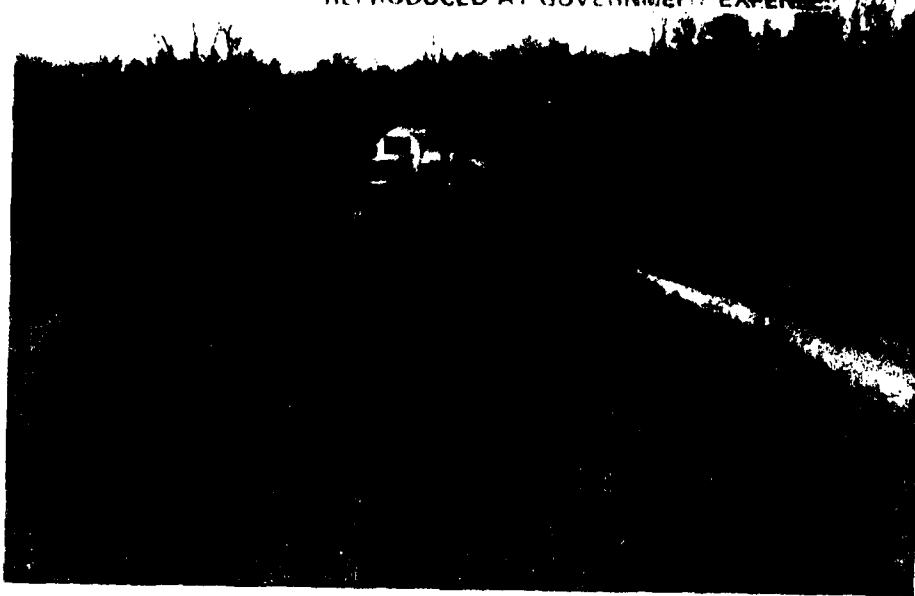


Photo 1

Top of dam looking
towards right abutment
from left side



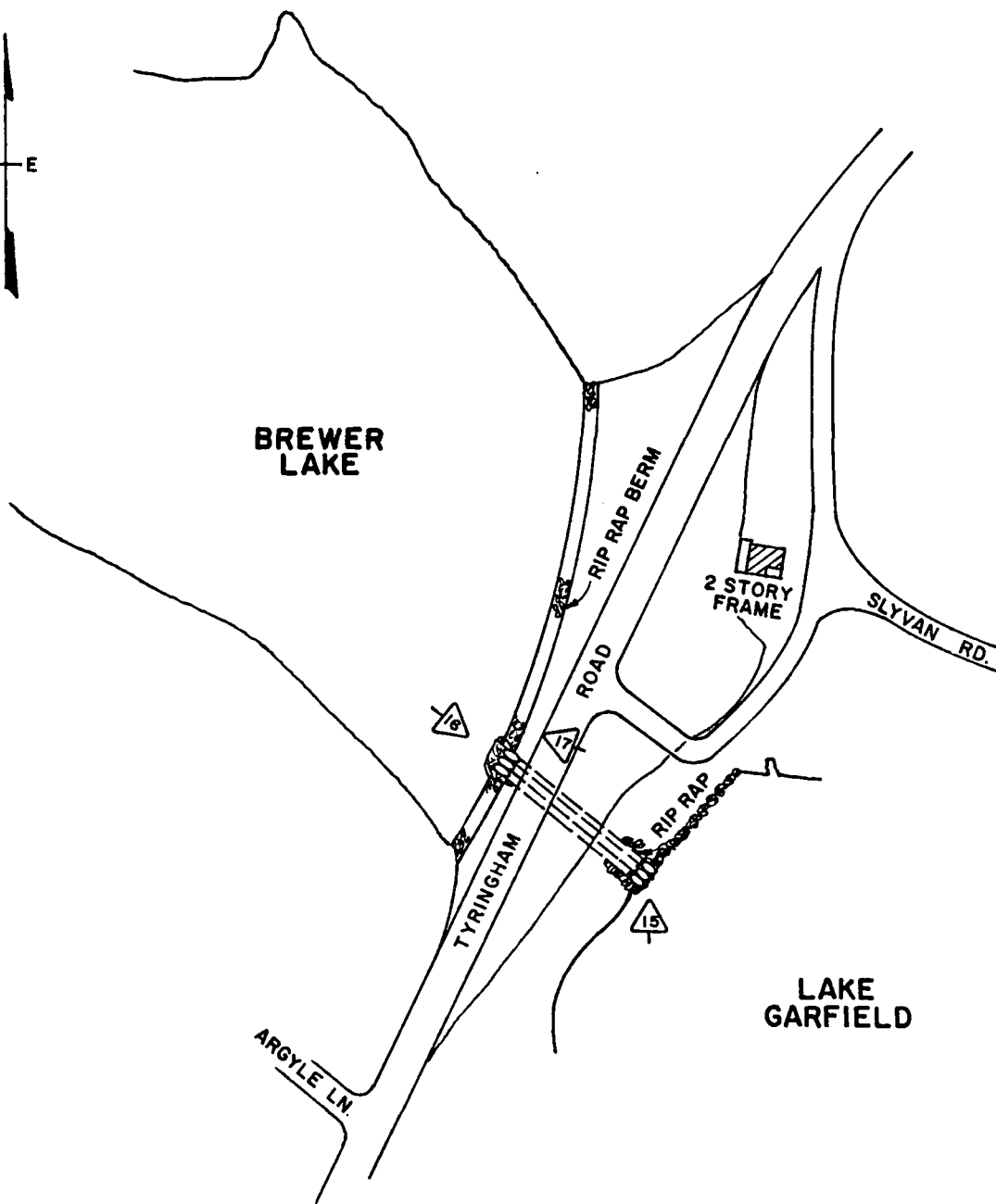
Photo 2

Overview of impoundment
area with pond drained.
Tyringham Road crossing
in upper photo.



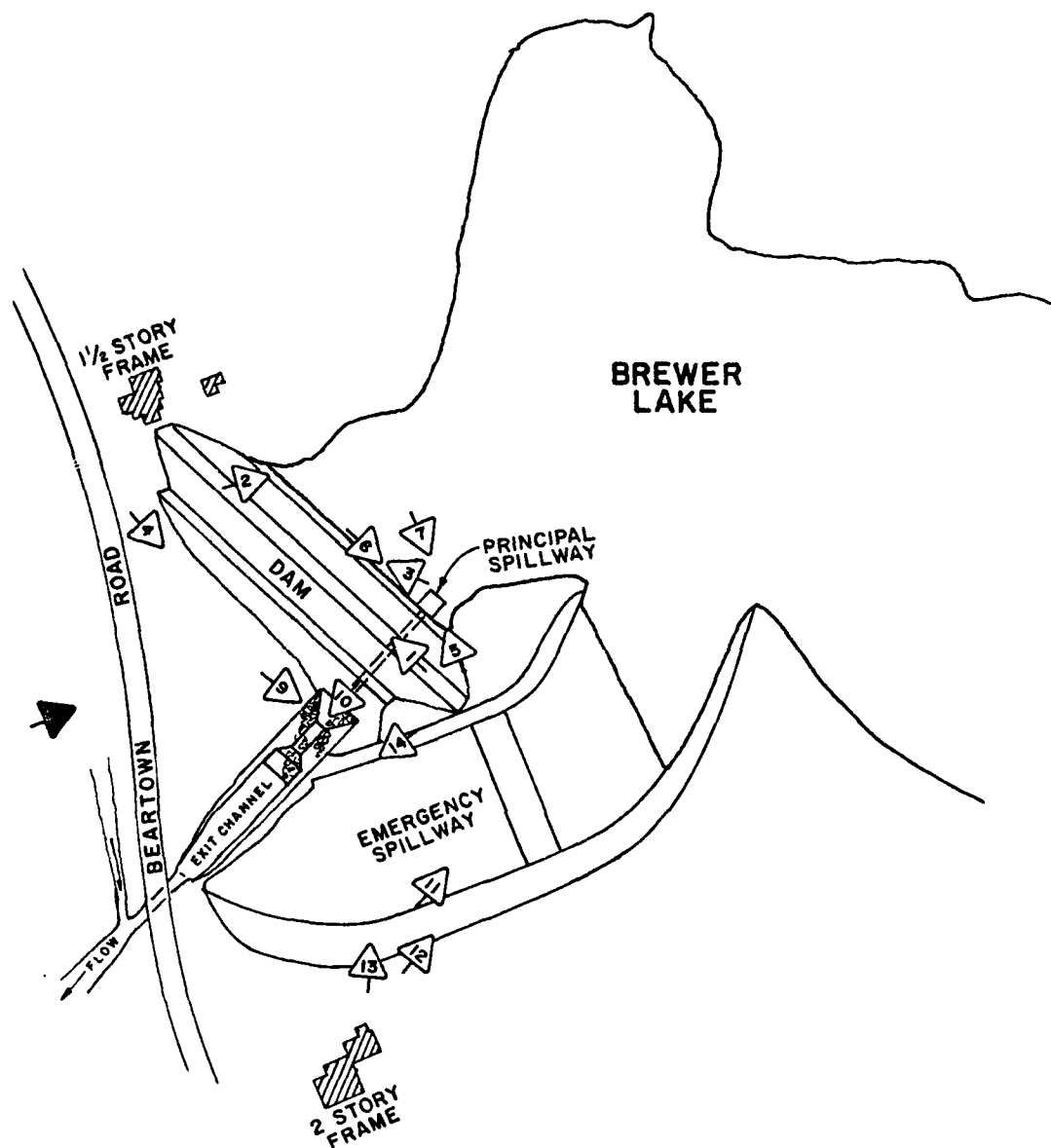
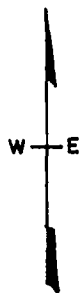
Photo 3

Upstream embankment look-
ing towards right abutment
from left side.



16 APPENDIX C

TIGHE & BOND / SCI CONSULTING ENGINEERS EASTHAMPTON, MASS.		U.S. ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS.	
NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS			
LOCATION AND ORIENTATION OF PHOTOS 2			
LAKE GARFIELD DAM (MA 00249) BERKSHIRE COUNTY		MONTEREY MASSACHUSETTS	
		SCALE: NONE	
		DATE: MARCH 1980	



OVERVIEW (AERIAL)



APPENDIX C

TIGHE & BOND / SCI
CONSULTING ENGINEERS
EASTHAMPTON, MASS.

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

NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS

LOCATION AND ORIENTATION OF PHOTOS I

LAKE GARFIELD DAM (MA00249)
BERKSHIRE COUNTY

MONTEREY
MASSACHUSETTS

SCALE: NONE

DATE: MARCH 1980

APPENDIX C
PHOTOGRAPHS

[illegible][illegible]

1. The first of these is the fact that the
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 9. of the first of these is the fact that the
 10. of the first of these is the fact that the

LAKE GARFIELD
MOUNTAIN VIEW

[illegible]

DATE	TIME	LOCATION	WIND	SEA	TEMP	DEPTH	REMARKS
10/10/50	0800	100-110	10-15	2-3	20-25	100	Underway
10/10/50	0900	110-120	10-15	2-3	20-25	100	Underway
10/10/50	1000	120-130	10-15	2-3	20-25	100	Underway
10/10/50	1100	130-140	10-15	2-3	20-25	100	Underway
10/10/50	1200	140-150	10-15	2-3	20-25	100	Underway
10/10/50	1300	150-160	10-15	2-3	20-25	100	Underway
10/10/50	1400	160-170	10-15	2-3	20-25	100	Underway
10/10/50	1500	170-180	10-15	2-3	20-25	100	Underway
10/10/50	1600	180-190	10-15	2-3	20-25	100	Underway
10/10/50	1700	190-200	10-15	2-3	20-25	100	Underway
10/10/50	1800	200-210	10-15	2-3	20-25	100	Underway
10/10/50	1900	210-220	10-15	2-3	20-25	100	Underway
10/10/50	2000	220-230	10-15	2-3	20-25	100	Underway
10/10/50	2100	230-240	10-15	2-3	20-25	100	Underway
10/10/50	2200	240-250	10-15	2-3	20-25	100	Underway
10/10/50	2300	250-260	10-15	2-3	20-25	100	Underway
10/10/50	2400	260-270	10-15	2-3	20-25	100	Underway
10/10/50	2500	270-280	10-15	2-3	20-25	100	Underway
10/10/50	2600	280-290	10-15	2-3	20-25	100	Underway
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10/10/50	6600	680-690	10-15	2-3	20-25	100	Underway
10/10/50	6700	690-700	10-15	2-3	20-25	100	Underway
10/10/50	6800	700-710	10-15	2-3	20-25	100	Underway
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10/10/50	7300	750-760	10-15	2-3	20-25	100	Underway
10/10/50	7400	760-770	10-15	2-3	20-25	100	Underway
10/10/50	7500	770-780	10-15	2-3	20-25	100	Underway
10/10/50	7600	780-790	10-15	2-3	20-25	100	Underway
10/10/50	7700	790-800	10-15	2-3	20-25	100	Underway
10/10/50	7800	800-810	10-15	2-3	20-25	100	Underway
10/10/50	7900	810-820	10-15	2-3	20-25	100	Underway
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10/10/50	9800	1000-1010	10-15	2-3	20-25	100	Underway
10/10/50	9900	1010-1020	10-15	2-3	20-25	100	Underway
10/10/50	10000	1020-1030	10-15	2-3	20-25	100	Underway

1. 2000-2001
 2. 2001-2002
 3. 2002-2003
 4. 2003-2004
 5. 2004-2005
 6. 2005-2006
 7. 2006-2007
 8. 2007-2008
 9. 2008-2009
 10. 2009-2010
 11. 2010-2011
 12. 2011-2012
 13. 2012-2013
 14. 2013-2014
 15. 2014-2015
 16. 2015-2016
 17. 2016-2017
 18. 2017-2018
 19. 2018-2019
 20. 2019-2020
 21. 2020-2021
 22. 2021-2022
 23. 2022-2023
 24. 2023-2024
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 40. 2039-2040
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 218. 2217-2218
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 221. 2220-2221

[illegible]

Figure 1 shows a series of nine line drawings of a person, arranged in a 3x3 grid. The drawings represent the progression of a child's drawing from age 2 to age 7. The top row shows a simple stick figure with a head, torso, and limbs. The middle row shows a more developed figure with a head, torso, and limbs, and a small head. The bottom row shows a figure with a head, torso, and limbs, and a small head. The drawings are labeled with '2', '3', '4', '5', '6', and '7' in the bottom right corner of each drawing.

[illegible]

Size & Hazard Classification

data reference: Design Drawings dated
1971 & Construction Records.

Dam Height:

$$\begin{array}{r} 1295.08 \\ - 1275.50 \\ \hline 19.58 \end{array}$$

top of dam (raised 0.33' from original
design 9/18/72)
original stream bed @ D.S. toe

use: Height = 19.5 ft

Storage Capacity:

(see note below)

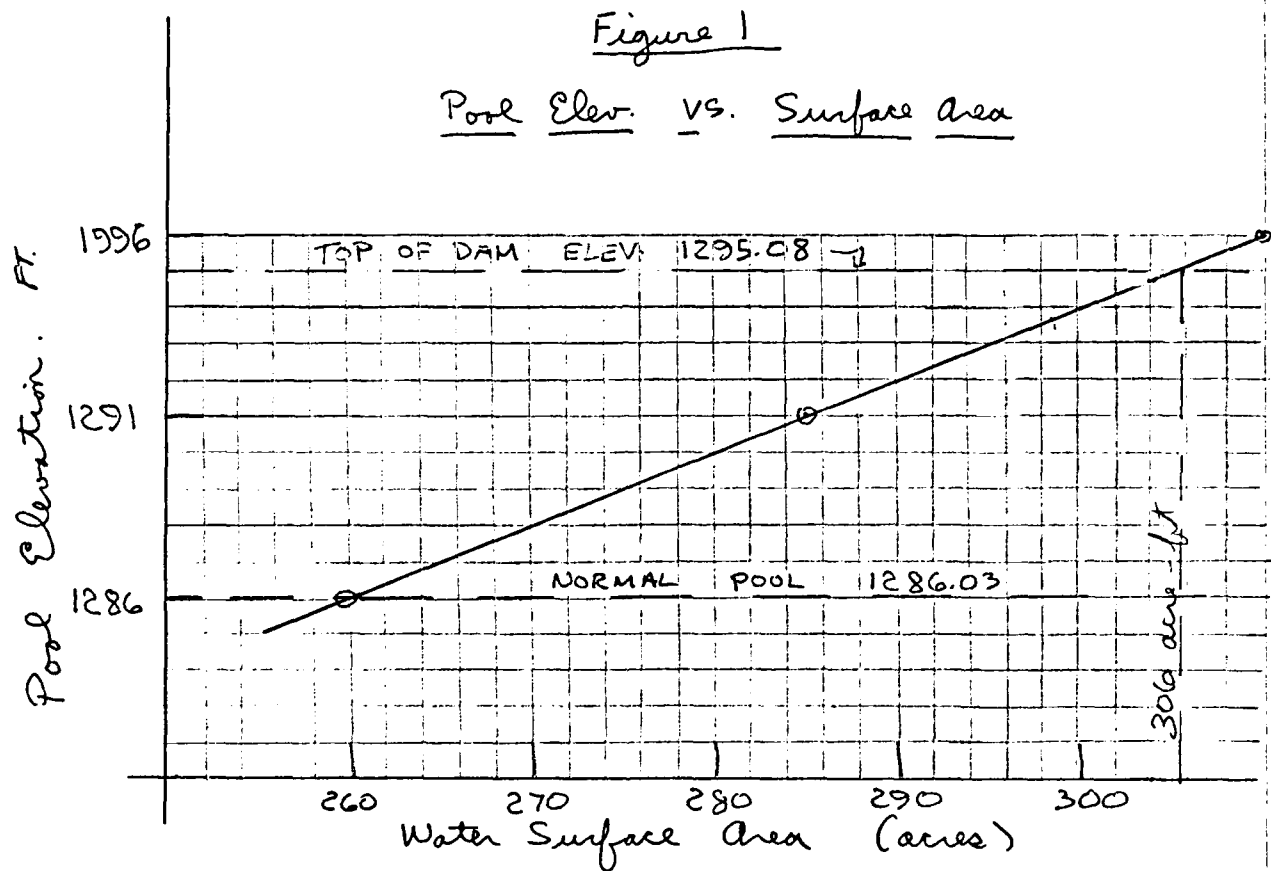
1. Surface Area @ Normal Pool = 259 acres.
(252 original lake + 7.0 new lake) from USGS
2. "Theoretical Area" with Pool 10 ft
above Normal Pool = 310 acres.
(299 original lake + 11 new lake) from USGS

∴ +10 ft depth = +51 acres.

assume stage vs. storage is a
linear function for depths 10 ft
above & below normal pool.

*: The design drawings list 265 acres for original
lake. This value did not agree with planimeter
and USGS, it was confirmed that the original lake
area was 252 acres.

$$\text{Change in area/ft depth} = \frac{51 \text{ acres}}{10 \text{ ft}} = 5.1 \text{ acres/ft}$$



Storage between Normal Pool & Top Dam:

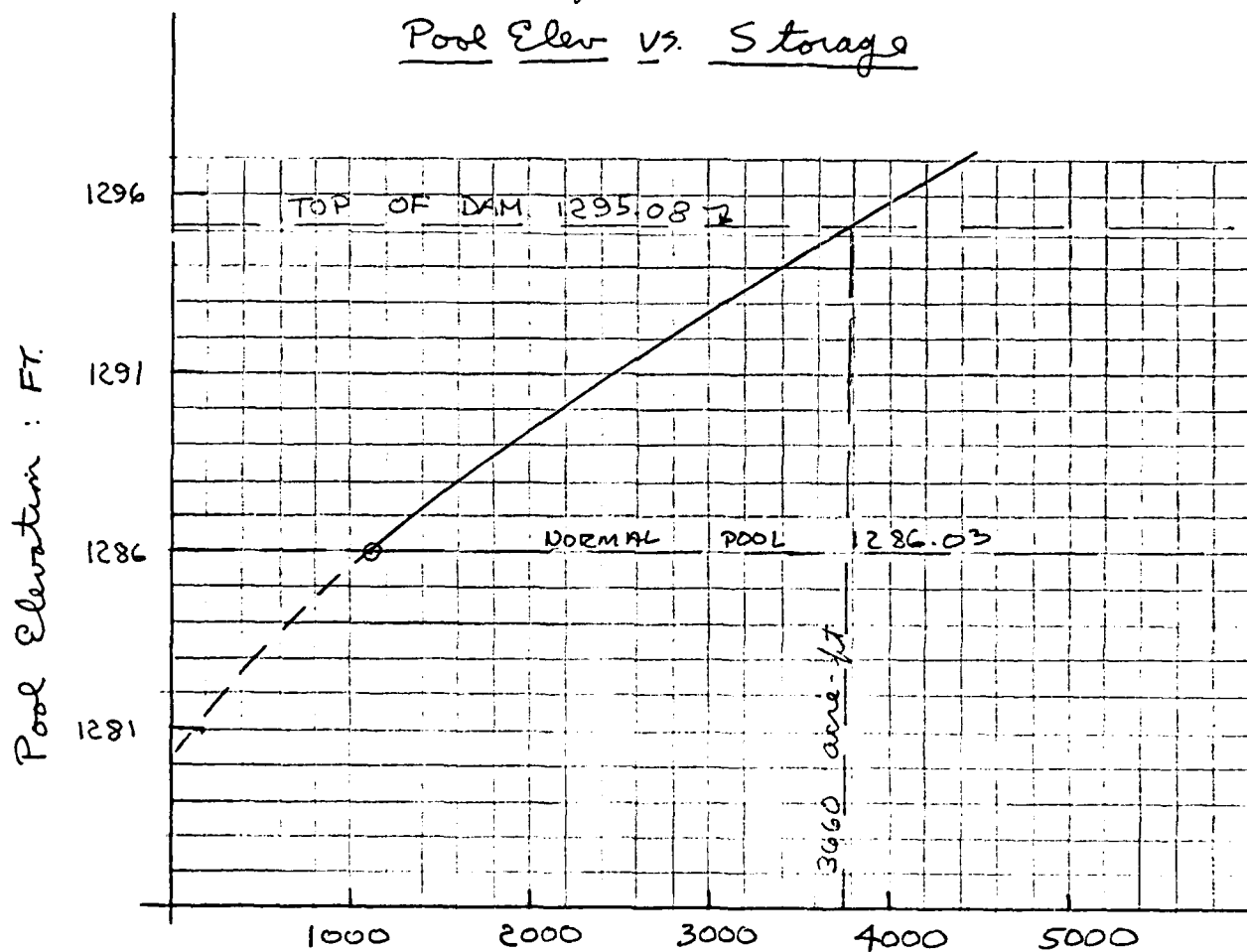
$$\text{Storage} = (9.05) \left[\frac{260 + 306}{2} \right] = 2560 \text{ acre-ft}$$

Design calculations dated 4/14/71 list

Storage @ Normal Pool = 1100 ± CFS

$\therefore$ 1100 acre-ft Normal Pool Storage
2560 acre-ft Normal Pool to Top Dam
 3660 acre-ft total Storage

Figure 2

Pool Elev vs. Storage

Size Class: 19.5 ft < 40 ft $\rightarrow$ small

1000 ac-ft < 3660 ac-ft < 50,000 ac-ft

Intermediate

$\therefore$ Due to impoundment size Lake Garfield is Intermediate

Hazard Class:

The hazard classification appears to be high due to the location of the Village of Monterey being downstream of the dam. There are more than a few homes adjacent to the discharge stream and a major roadway is crossed. High hazard is assumed for the test flood selection pending the results of the dam failure analysis.

$\therefore$ Intermediate Size
High Hazard.

Required Spillway Design Flood = P.M.F.

OAD 4/24/80

1400

U-10-29

Lake Garfield

Spillway Adequacy Analysis

① Test Flood Determination

Lake Garfield Drainage Area = 2550 acres

(From USGS Maps)

(3.98 mi²)

Note: Design drawings indicate 2592 acres.

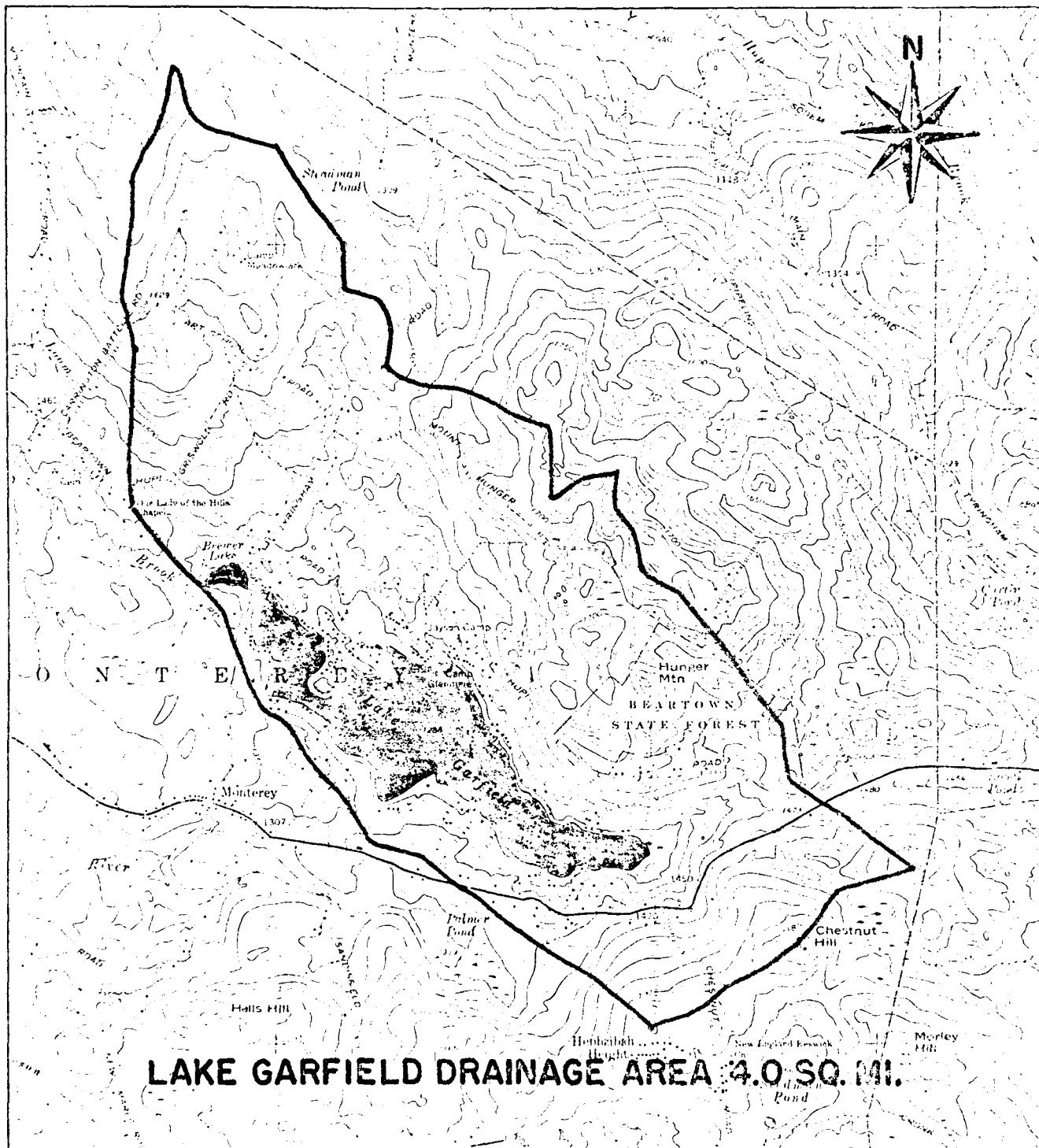
(4.05 mi²)

For computations use 4.0 mi²

From COE guidance curve "Maximum Probable
Flood Peak Flow Rates" Dec. 1977.

Unit Discharge = 2300 CFS/mi² for
Mountainous Terrain

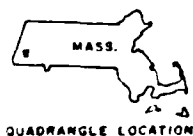
P.M.F. test flood = $4.0 \times 2300 \text{ CFS/mi}^2 = 9,200 \text{ CFS}$



LAKE GARFIELD DRAINAGE AREA 4.0 SQ. MI.

-SCALE-
1000' 0 1000' 2000' 3000' 4000'

FROM: U.S.G.S. MONTEREY, MASS.



TIGHE & BOND / SCI
CONSULTING ENGINEERS
EASTHAMPTON, MASS.

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

NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS

DRAINAGE AREA MAP

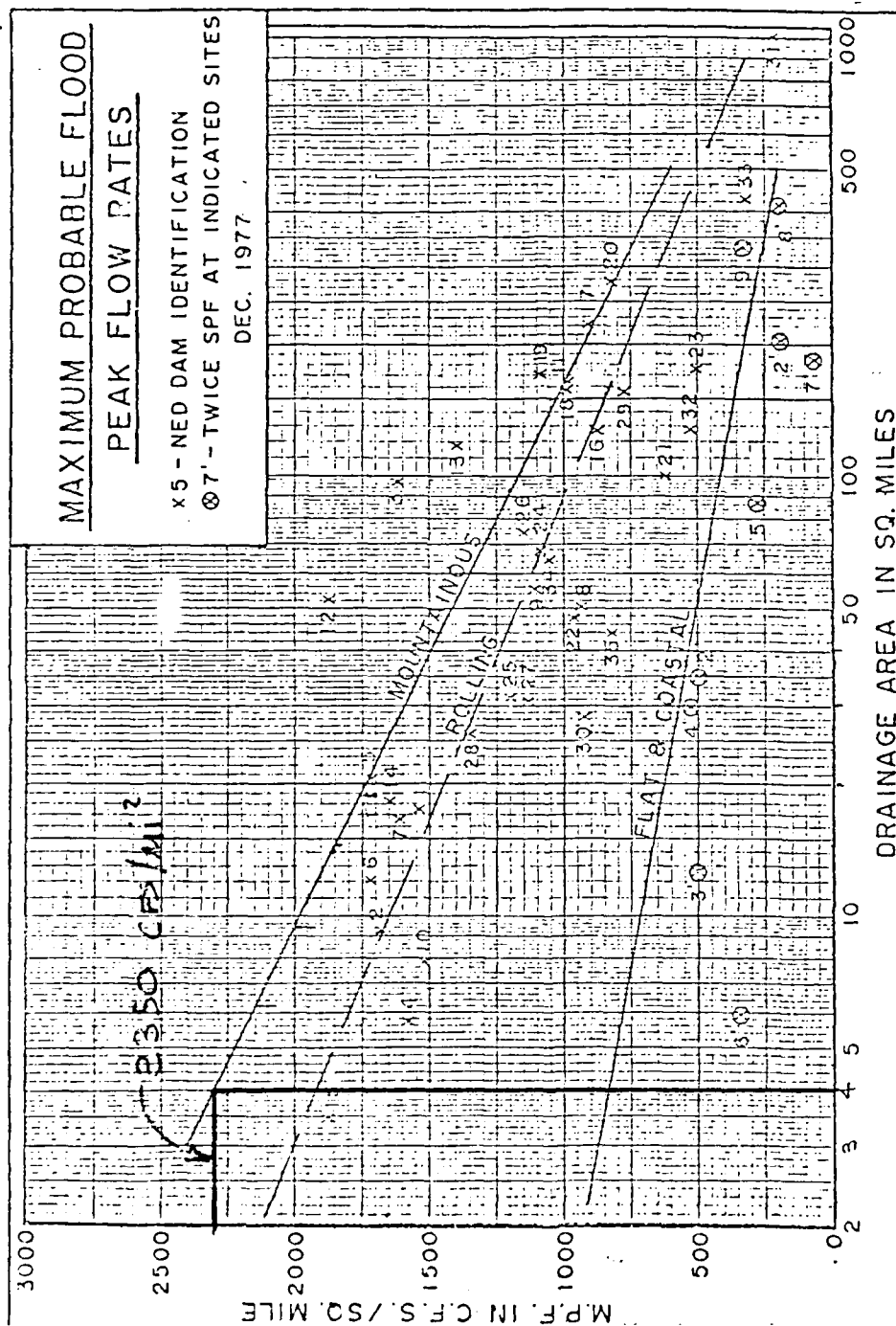
LAKE GARFIELD DAM (MA 00249)
BERKSHIRE COUNTY

MONTEREY
MASSACHUSETTS

SCALE: AS NOTED

DATE: MARCH 1980

Figure 3



(B) Combined Spillways Capacity

1. Principal Spillway

a) Overflow weir : elev : 1286.03

length : 2 @ 15 ft = 30 ft

width : 12" w/ 3" radius

$$Q = 3.33 (L) (H)^{1.5} \quad (\text{weir flow})$$

at water depths greater than 2 ft over the weirs, the top of the structure causes orifice flow to occur.

$$Q = 0.70 A \sqrt{2gH} \quad (\text{orifice flow})$$

Elev.	H	$H^{1.5}$	Q
1286.03	0	0	0
1286.53	0.5	0.35	35 CFS
1287.03	1.0	1.0	99 CFS
1288.03	2.0	2.82	282 CFS

orifice flow

Elev.	H	$\sqrt{2gH}$	Q
1288.53	1.5	9.82	412 CFS
1290.03	3.0	13.90	584 CFS
1294.03	7.0	21.23	892 CFS
1296.03	9.0	24.07	1011 CFS

Outlet Pipe Capacity :

Pipe : 60" ϕ , 120 ft in length

losses = inlet + friction + outlet

$$\text{losses} = 0.5 \frac{V^2}{2g} + f \frac{L}{D} \frac{V^2}{2g} + 1.0 \frac{V^2}{2g}$$

$$f \frac{L}{D} = (0.02) \frac{120}{5} = 0.48$$

$$H = \left[0.5 \frac{V^2}{2g} + 0.48 \frac{V^2}{2g} + 1.0 \frac{V^2}{2g} \right]$$

$$H = 1.98 \frac{V^2}{2g}$$

$$\sqrt{\frac{2gH}{1.98}} = V$$

$$Q = A \sqrt{\frac{2gH}{1.98}}$$

$$A = 19.62 \text{ ft}^2$$

$$Q = 112 \sqrt{H}$$

Elev.	H	$\sqrt{H}$	Q	
1286.03	8'	2.82	316 cfs	
1288.03	10'	3.16	354	} pipe is control limitation
1290.03	12'	3.46	388	
1294.03	16'	4.0	448	
1296.03	18'	4.24	475	

2. Emergency Spillway

control section length = 30'

" " width = 130'

" " elev = 1288.33

$$H = \left(\frac{q}{3.09} \right)^{\frac{2}{3}} + L \left(\frac{q n}{d^{\frac{5}{3}} 1.486} \right)^2$$

$$q = \frac{Q}{\text{width}} = \frac{Q}{130'}$$

$$L = 30'$$

$$n = 0.04 \text{ (Manning Coefficient)}$$

$$d = \frac{2}{3} \left(\frac{q}{3.09} \right)^{\frac{2}{3}} \quad \text{(Manning Relationship)}$$

Q	q	d	H	Elev.
1000 cfs	7.69	1.23	2.48	1290.81
2000 "	15.38	1.96	3.48	1291.81
3000 "	23.07	2.57	4.34	1292.67
4000 "	30.76	3.12	5.12	1293.45
5000	38.46	3.63	5.84	1294.17
6000	46.15	4.1	6.54	1294.87
7000	53.85	4.55	7.19	1295.52

OHD 4/24/80

M.E.

6

3. Broad Crested Weir Flow Over Top of Dam.

$$Q = 3.09 L H^{3/2}$$

$$L = 350 \text{ ft}$$

Elev.	H	Q
1295.08	0	0
1295.33	0.25	135 CFS
1295.83	0.75	702 "
1296.58	1.50	1990 "
1297.08	2.0	3060 "
1298.08	3.0	5600 "

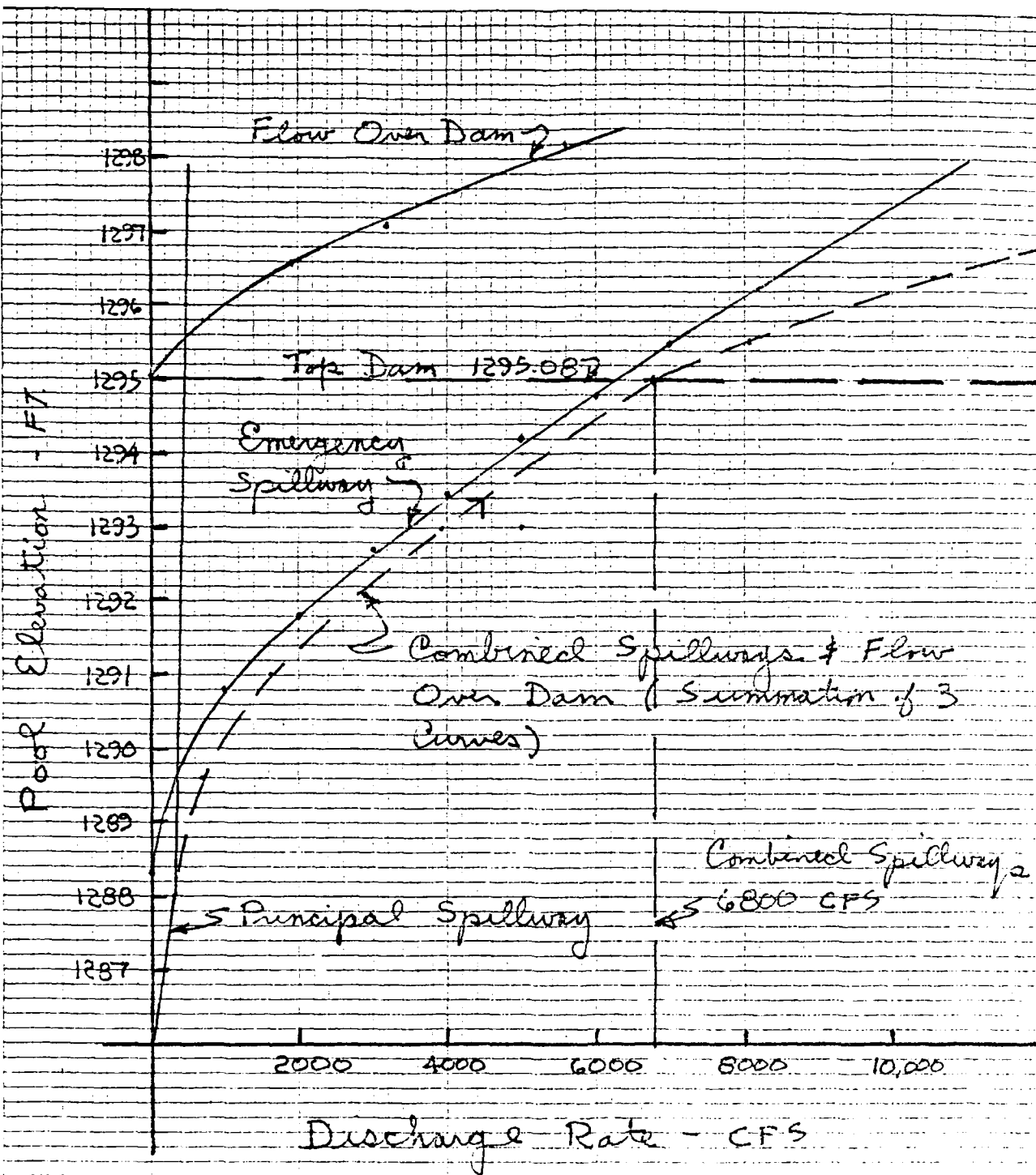


Figure 4

Pool Elev. Vs. Spillway Capacity

4D 10/20/80

4. Tyringham Road Culverts (upstream of dam)

3 - 84" ϕ ACCMP culverts

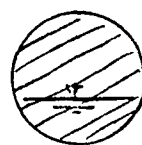
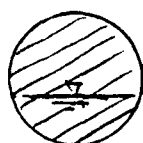
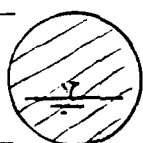
1 - 36" ϕ RCP Ungated Pond Drain

TOP MAIN DAM 1294.75

TYRINGHAM ROAD GRADE 1292.22

TOP 1290.0

INV. 1283.0



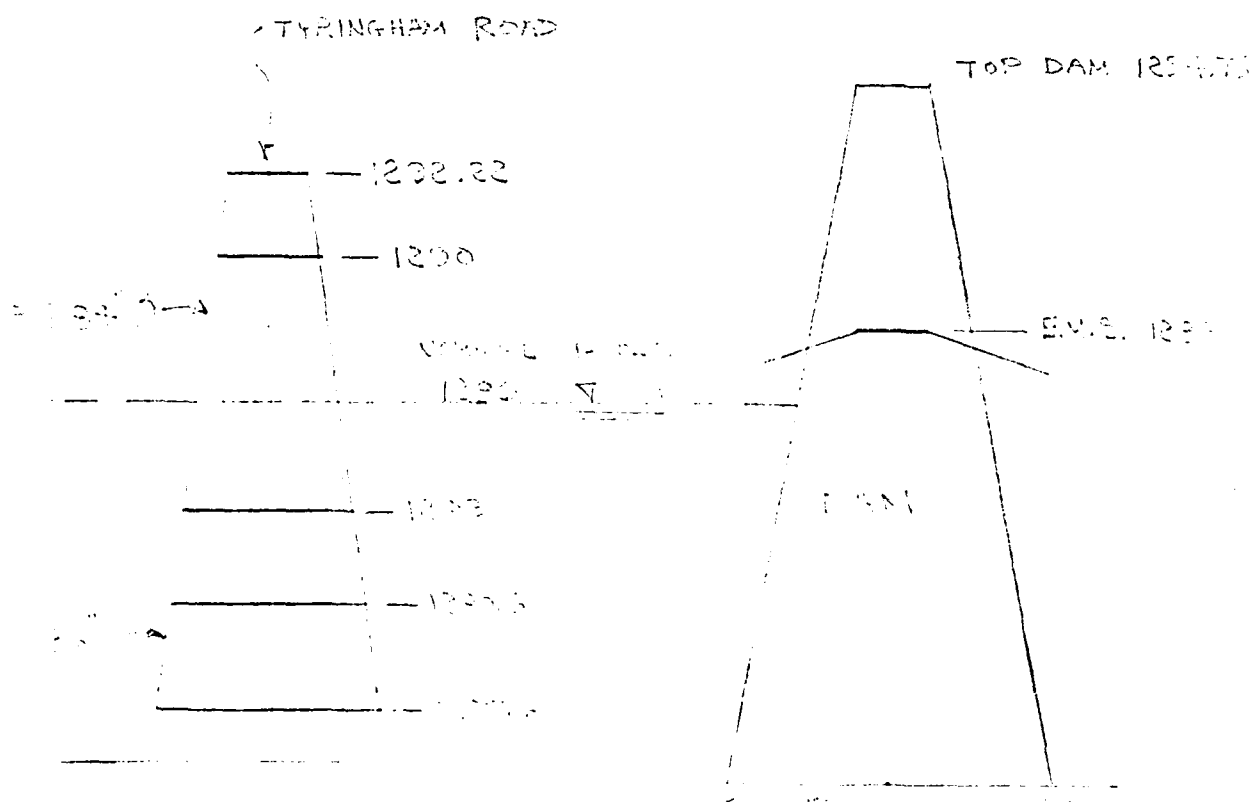
3 @ 84" ϕ

Normal Pond Elev.

INV. 1277.6



1 @ 36" ϕ



D-12

10/20/80

Culverts capacity

$$\text{Head} - \text{losses} = \frac{V^2}{2g}$$

losses = entrance + friction

$$\text{losses} = 0.5 \frac{V^2}{2g} + 0.4 \frac{V^2}{2g}$$

$$\text{losses} = 0.9 \frac{V^2}{2g}$$

$$\text{Head} - 0.9 \frac{V^2}{2g} = \frac{V^2}{2g}$$

$$\text{Head} = 1.9 \frac{V^2}{2g}$$

$$V = \sqrt{\frac{2g(\text{Head})}{1.9}}$$

Elev.	H	Vel	Q
1290	3.5 ft	10.9 FPS	1250 CFS
1292	5.5 ft	13.7 FPS	1570 CFS
1295	8.5 ft	17.0 FPS	1950 CFS

Broad Crested Weir Flow Over Dam

$$Q = 3.09 L H^{3/2}$$

Elev.	H.	L.	Q
1293	0.8 ft	200 ft	440 CFS
1294	1.8 ft	370 ft	2760 CFS
1295	2.8 ft	600 ft	8690 CFS

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NATIONAL PROGRAM FOR INSPECTION OF NON-FEDERAL DAMS
LAKE GARFIELD DAM (MA.) (U) CORPS OF ENGINEERS WALTHAM
MA NEW ENGLAND DIV MAR 80

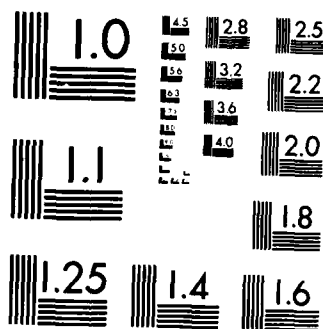
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MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

OHD 10/20/80

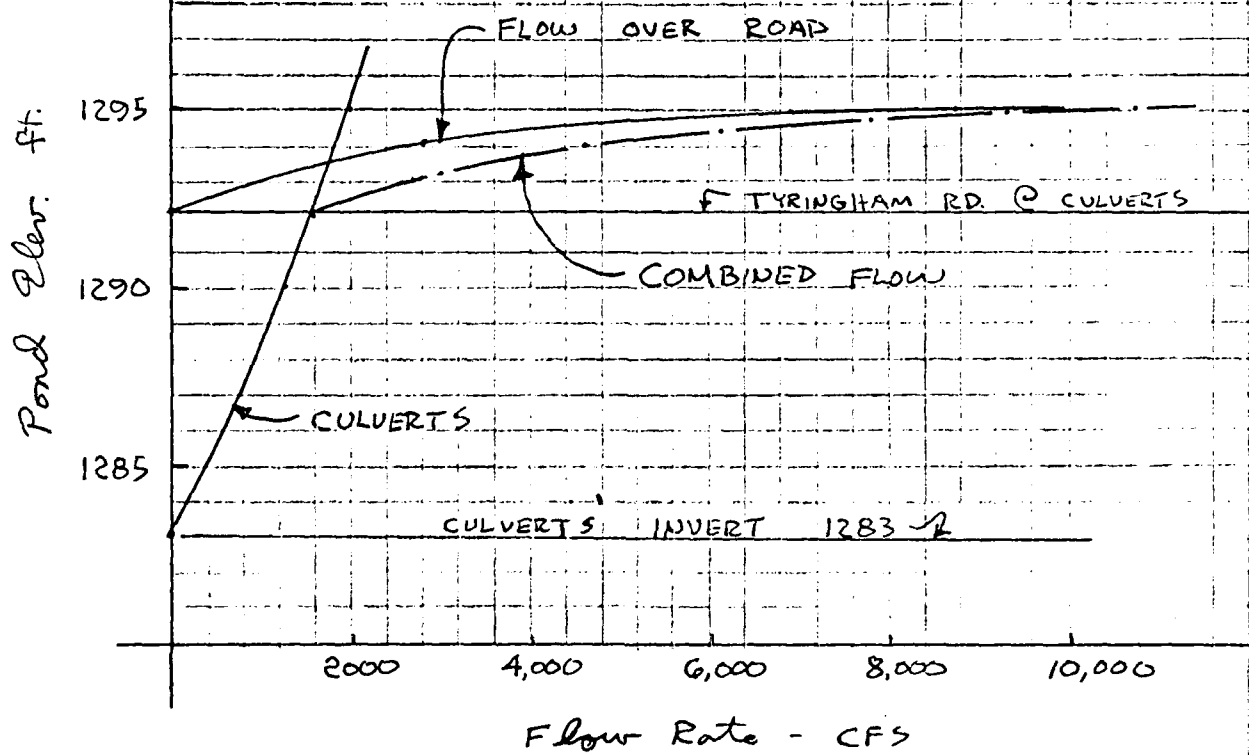


Figure 5

Tyringham Rd. Culvert Capacities.

© Estimated Effects of Surge Storage
On Test Flood Outflow

1. assume at start of test flood
the pool elevation is at the
normal recreation pool level
of 1286.03 .

then elev. 1286 = 0 storage

Refer to Pool Elev vs. Storage Curve
on next page which has been
computed in "Size & Hazard Classification"
Section of calcs.

2. Flood Routing

- a) Routing With Effects of Tyingham
Road Culverts Upstream of Dam

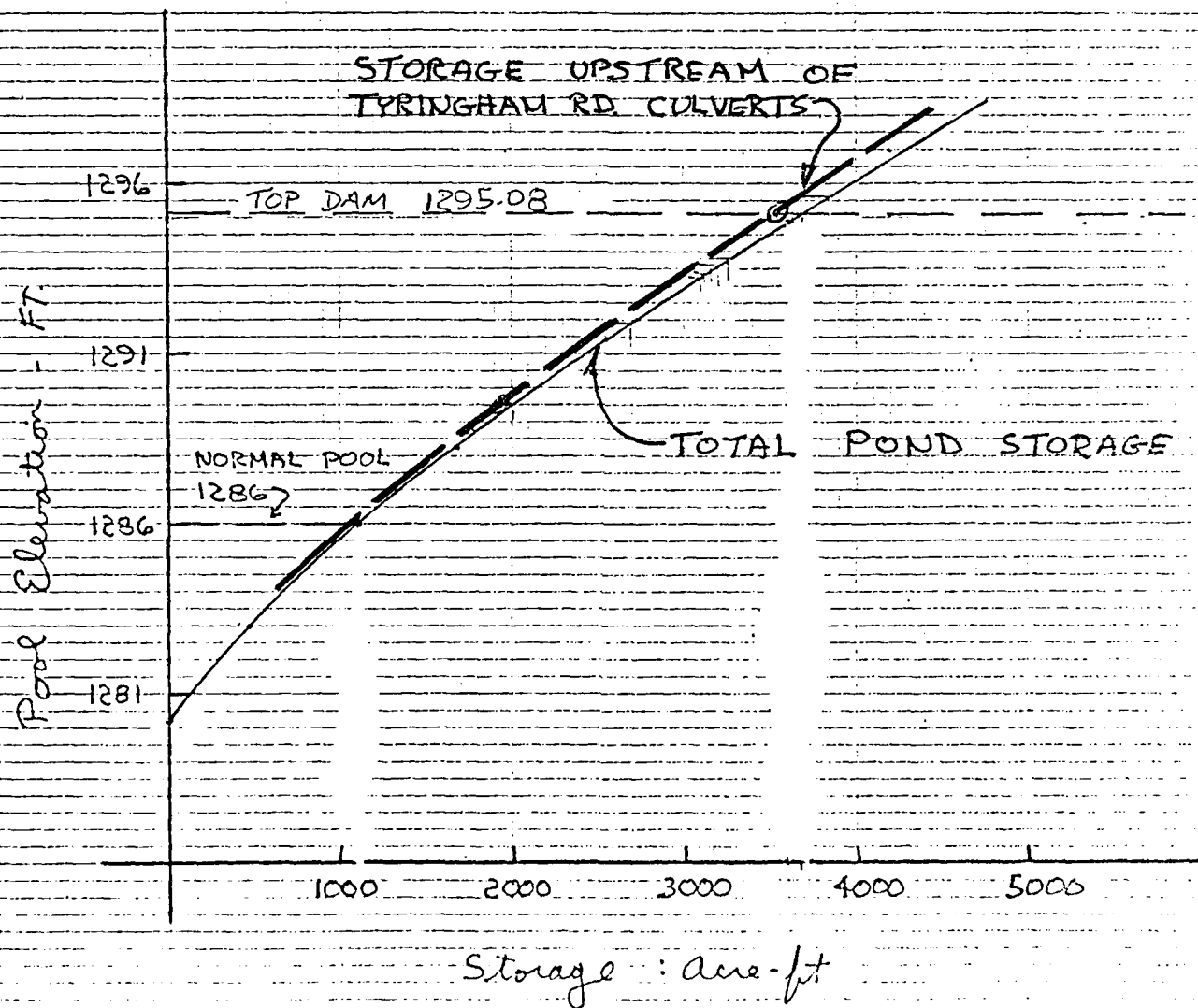


Figure 6

Pool Elev. vs. Storage
Upstream of Tyringham Rd

Trial #1 : Test Flood Inflow 9,200 CFS (PMF)

from figure 5 : elev = 1295

from figure 6 : $Stor = 3500 - 1050 = 2450$
ac.ft

$$2450 \div 2550 \text{ acres} = 0.96 \text{ ft} = 11.53 \text{ inches}$$

$$Q_{P2} = 9200 \left(1 - \frac{11.53}{19}\right) = 3617 \text{ CFS}$$

Trial # 2 : $Q_{P2} = 3617 \text{ CFS}$

Elev = 1294

$Stor_2 = 3200 - 1050 = 2150 \text{ ac. ft}$

$$2150 \div 2550 \text{ acres} = 0.84 \text{ ft} = 10.11 \text{ inches}$$

$$Q_{P3} = 9200 \left(1 - \frac{10.11}{19}\right) = 4300 \text{ CFS}$$

Trial #3 : $Q_{P3} = 4300 \text{ CFS}$

Elev = 1294.2

$Stor_3 = 3250 - 1050 = 2200 \text{ ac. ft}$

$$2200 \div 2550 \text{ acres} = 0.86 \text{ ft} = 10.35 \text{ inches}$$

$$Q_{P4} = 9200 \left(1 - \frac{10.35}{19}\right) = 4190$$

∴ Outflow D.S. of Tyringham Rd is
4200 CFS @ Elev. 1294.1

Roadway Overtopped $2 \pm \text{ ft}$ - damage
potential is high

Downstream of Tyringham Rd. and upstream of the dam there is only $110 \pm$ acre feet of storage, therefore, surcharge storage will not significantly reduce the test flood inflow beyond the attenuation at Tyringham Road

Spillway test flood outflow = 4,200 CFS
@ Elev. 1293.2 ft

Note that Tyringham Road would be substantially overtopped therefore, serious damage to the embankment could reduce the attenuation effects of this barrier. The worst case would be with embankment fully removed.

b) Flood Routing With Tyringham Road Embankment Removed

OND 4/24/80

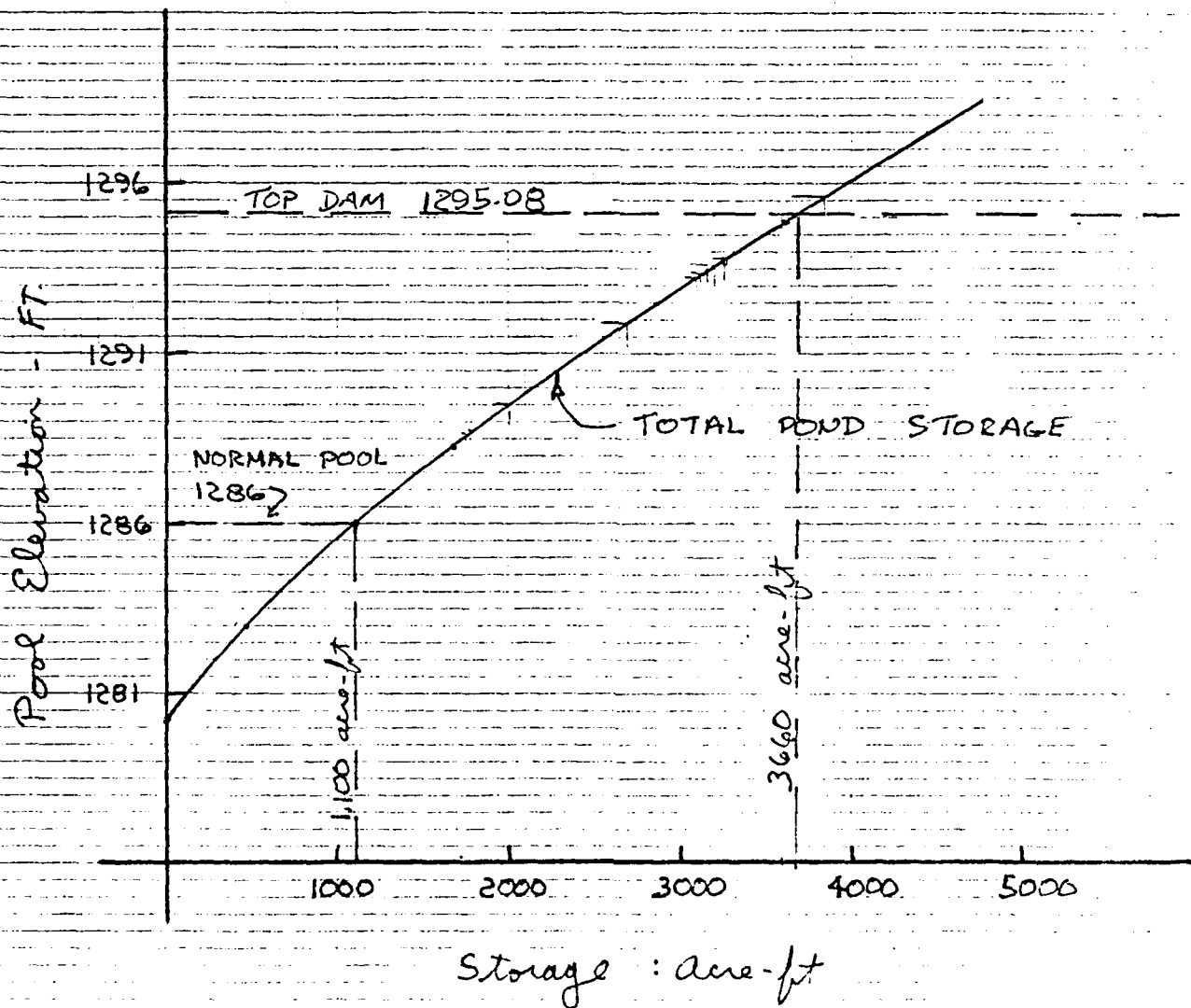


Figure 7

Pool Elev. vs. Storage

(Same as Figure 2)

Trial #1: $Q_{p1} = 9,200$ CFS (PMF test flood)

From Fig 4: Elev = 1295.6

From Fig 7: $Stor_1 = (3850 - 1100) = 2750$ acre-ft

$2750 \text{ acre-ft} \div 2550 \text{ acres} = 1.07 \text{ ft} = 12.94 \text{ inches}$

$$Q_{p2} = 9,200 \left(1 - \frac{12.94}{19}\right) = 2934 \text{ CFS}$$

Trial #2: $Q_{p2} = 2934$ CFS

From Fig 4: Elev = 1291.9

From Fig 7: $Stor_2 = (2700 - 1100) = 1600$ acre-ft

$1600 \text{ acre-ft} \div 2550 \text{ acres} = 0.627 \text{ ft} = 7.53 \text{ inches}$

$$Q_{p3} = 9,200 \left(1 - \frac{7.53}{19}\right) = 5556 \text{ CFS}$$

Trial #3: $Q_{p3} = 5556$ CFS

From Fig 4: Elev = 1293.85

From Fig 7: $Stor_3 = (3250 - 1100) = 2150$ acre-ft

$2150 \text{ acre-ft} \div 2550 \text{ acres} = 0.84 \text{ ft} = 10.11 \text{ inches}$

$$Q_{p4} = 9,200 \left(1 - \frac{10.11}{19}\right) = 4305 \text{ CFS}$$

Trial #4: $Q_{p4} = 4305$ CFS

From Fig 4: Elev = 1293.2

From Fig 7: $Stor_4 = (3100 - 1100) = 2000$ acre-ft

$2000 \text{ acre-ft} \div 2550 \text{ acres} = 0.78 \text{ ft} = 9.41 \text{ inches}$

$$Q_{p5} = 9,200 \left(1 - \frac{9.41}{19}\right) = 4646 \text{ CFS}$$

Trial # 5 : $Q_{p5} = 4646 \text{ CFS}$

From Fig 4 : Elev = 1293.7

From Fig 7 : $Stor_5 = (3200 - 1100) = 2100 \text{ acre-ft}$

$2100 \text{ acre-ft} \div 2550 \text{ acres} = 0.82 \text{ ft} = 9.88 \text{ inches}$

$Q_{p6} = 9200 \left(1 - \frac{9.88}{19}\right) = 4416 \text{ CFS}$

Trial # 6 : $Q_{p6} = 4416 \text{ CFS}$

From Fig 4 : Elev = 1293.4

From Fig 7 : $Stor_6 = (3180 - 1100) = 2080$

$2080 \text{ acre-ft} \div 2550 \text{ acres} = 0.815 \text{ ft} = 9.79 \text{ inches}$

$Q_{p7} = 9200 \left(1 - \frac{9.79}{19}\right) = 4462 \text{ CFS}$

$4416 < \text{Outflow} < 4462$

$\therefore \text{Test Flood Outflow} = \underline{\underline{4450 \text{ CFS}}}$

$4450 \text{ CFS} > 4200 \text{ CFS} \therefore$ using the worst case the spillway test flood outflow will result in a pond elevation of 1293.4 with a remaining freeboard to the top of the dam of 1.75 ft (Spillway Capacity = 6,800 CFS) E-2.

Summary of Routing:

Worst case is with Tyringham Rd.
substantially breached or removed.

Test Flood inflow = 9,200 CFS

Test Flood outflow = 4450 CFS

Pond Elevation 1293.4

Remaining Freeboard
to Top of Dam 1.7 ± ft

Summary of Downstream Conditions
With Dam Failure During PMF

The following area designations refer to the "Location And Downstream Hazard Maps".

① at Dam

② Downstream of Dam

Q before failure = 4450 CFS

Q after failure = 12,400 CFS

The Beartown Mountain Road culvert just downstream of the dam has a capacity of about 260 CFS.

Prefailure flood flows will inundate the culvert and overtop the roadway by about 4 ft.

The post-failure flow will increase the roadway flooding by an additional $3\frac{1}{2}$ ft to $7\frac{1}{2}$ ft.

③ Village of Monterey

Q before failure = 4450 CFS

Q after failure = 12,400 CFS

The Rt. 23 arch culvert has a capacity of about 1200 CFS. Prefailure flood flows will inundate the culvert and overtop the roadway by up to $1.5 \pm$ ft at the low point in the road which is about $100 \pm$ ft east of the culvert. Prefailure flooding will inundate 6 structures (5 homes + 1 store) by 1.5 ft.

Post-failure flow will increase all flooding by about $4 \pm$ ft.

④ Curtis Road Crossing

Q before failure = 4450 CFS

Q after failure = 12,100 CFS

The actual capacity of the Curtis Road culvert is unknown, but it is definitely much less than both the Pre-failure & Post failure flood flows.

The pre-failure flood flow will cause a river stage of $8 \pm$ ft, which will overtop the roadway by $2 \pm$ ft.

Post-failure flows will result in about a $4.5 \pm$ ft increased River stage.

Downstream of Confluence With Swan Brook

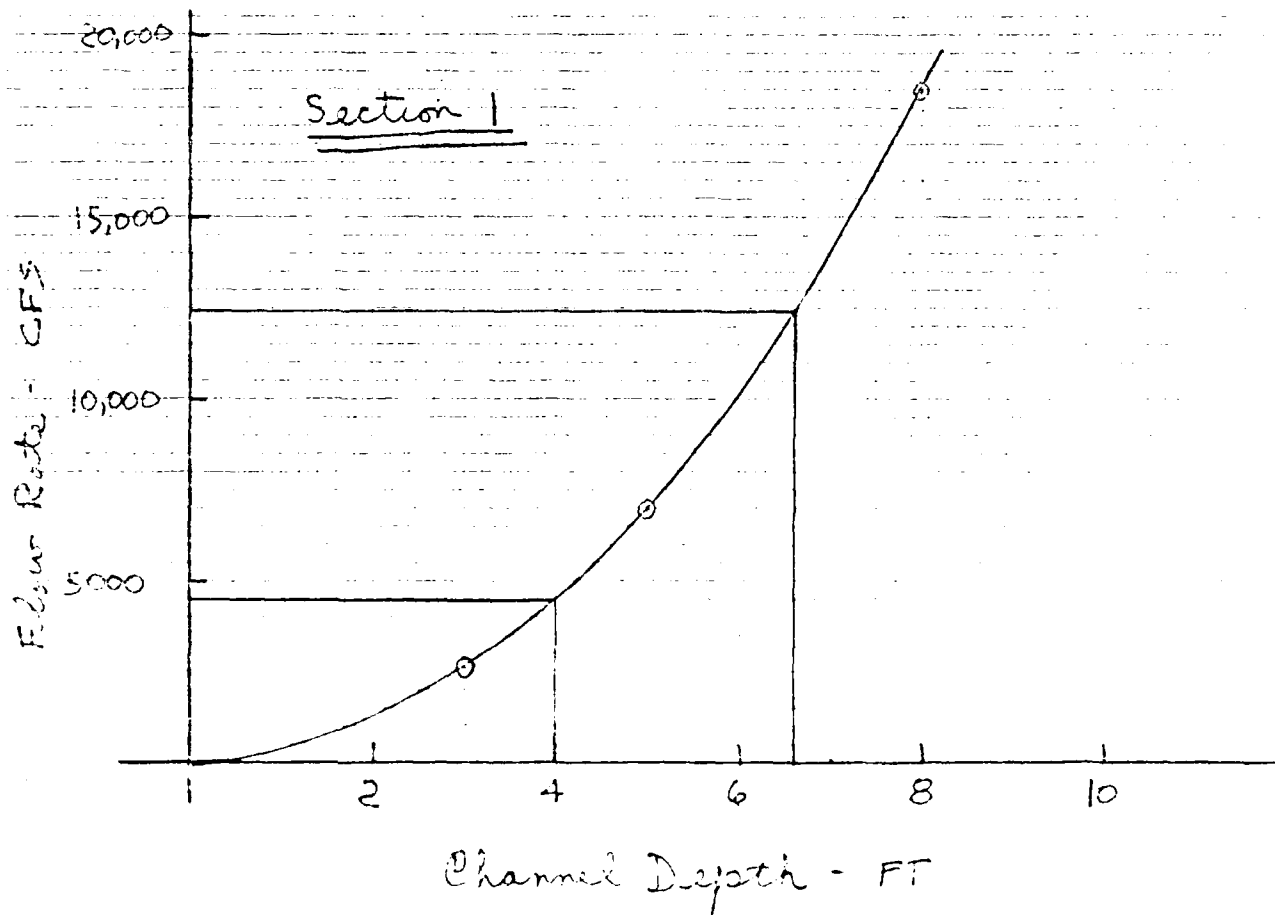
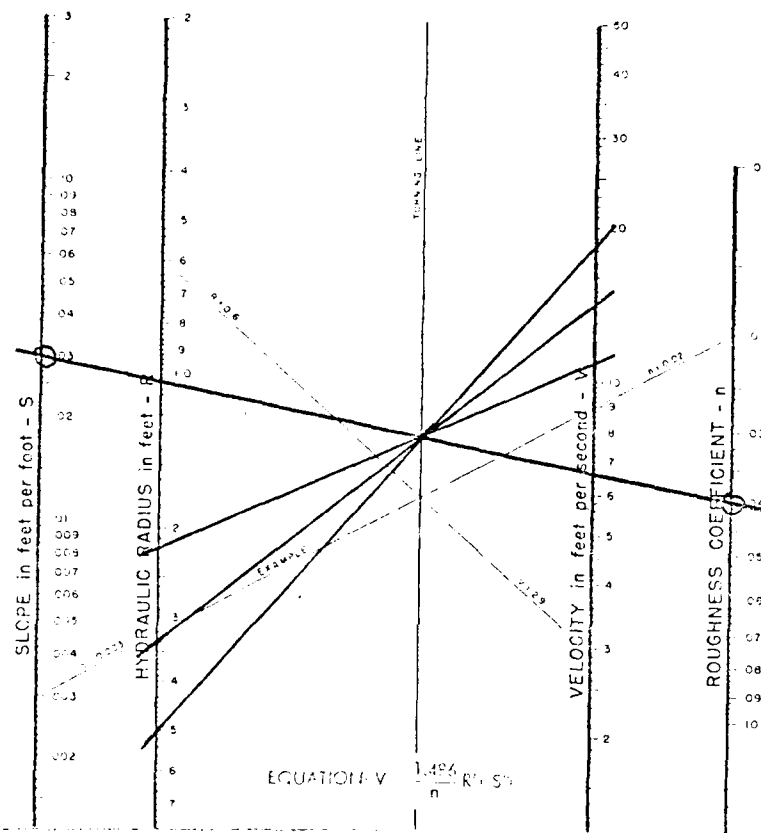
Q before failure = 4450 CFS

Q after failure = 11,600 CFS

Pre-failure flood flows will cause a river stage of $8 \pm$ ft.

This will not result in any flood damage.

Post failure flooding will increase the river stage by $4 \pm$ ft and will not constitute a hazard potential at this location.



b) Depth = 8 ft

Top Width = 200 ft

$$\text{Area} = \frac{200 + 50}{2} \times 8 = 1000 \text{ ft}^2$$

$$\text{Hyd. Rad} = 1000 \div 200 = 5$$

$$\text{Vel} = 18.5 \text{ FPS}$$

$$Q = 18.5 \times 1000 = 18,500 \text{ CFS}$$

c) Depth = 3 ft

Top Width = 110 ft

$$\text{Area} = \frac{110 + 50}{2} \times 3 = 240 \text{ ft}^2$$

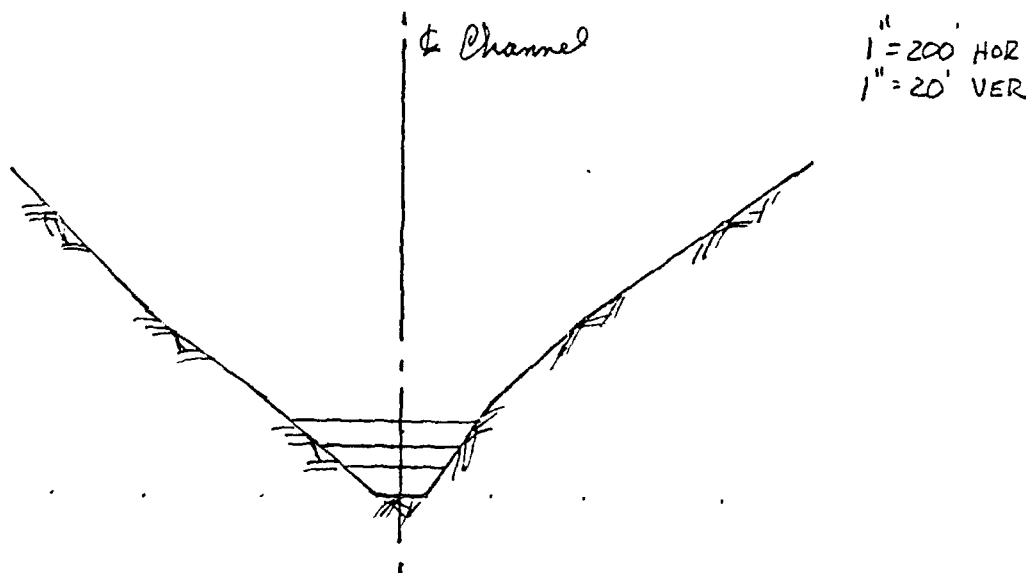
$$\text{Hyd. Rad} = 240 \div 110 = 2.18$$

$$\text{Vel} = 11 \text{ FPS}$$

$$Q = 11 \times 240 = 2,600 \text{ CFS}$$

(E) Channel Stage vs. Flow

Section 1: Downstream of Beartown Mountain
Road Culvert & Confluence
with Loom Brook.



Channel Slope = 0.03 (just u.s. of slope change)
Manning $n = 0.04$

a) Depth = 5 ft

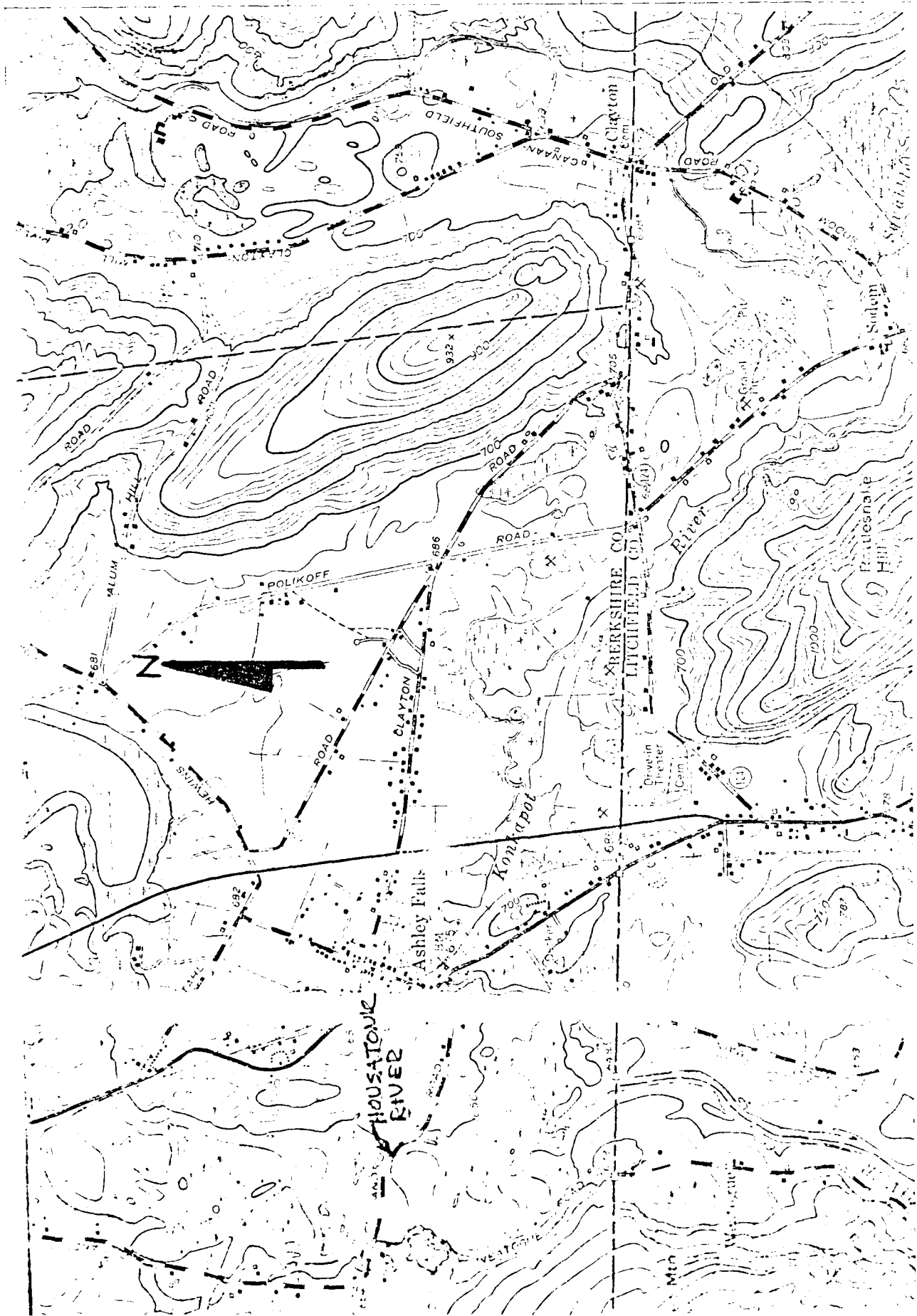
Top Width = 150 ft

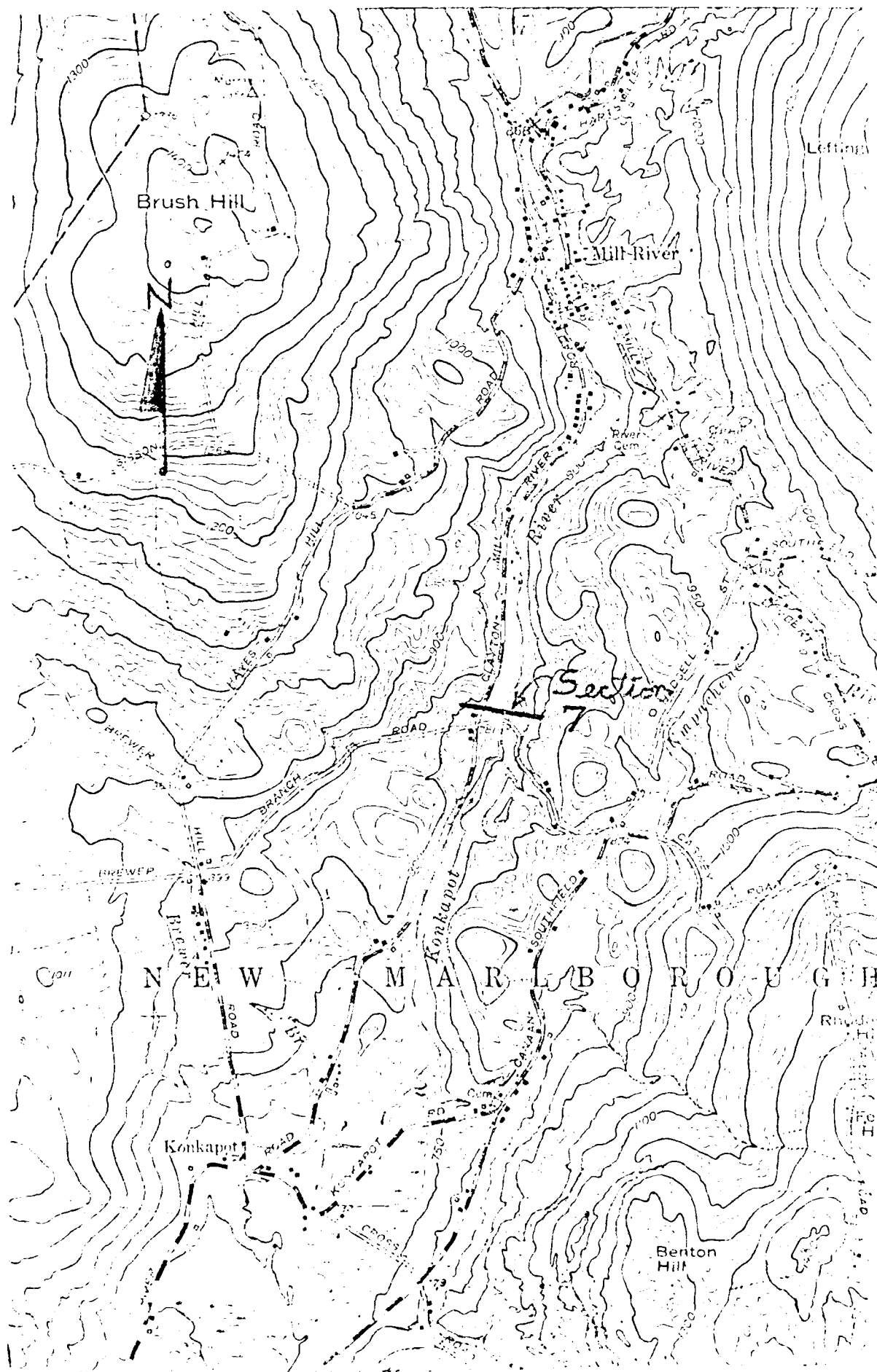
$$\text{Area} = \frac{150 + 50}{2} \times 5 = 500 \text{ ft}^2$$

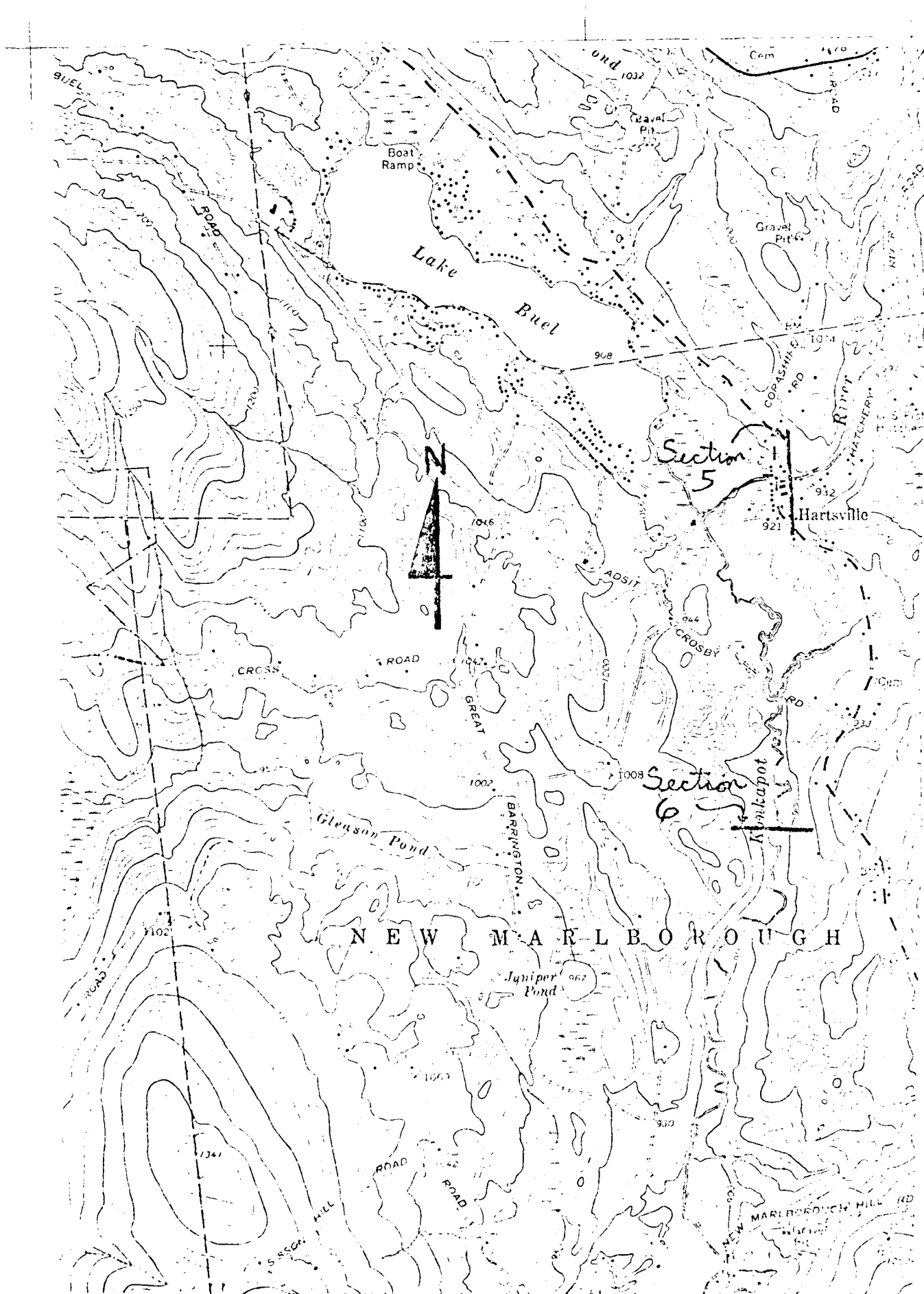
$$\text{Hyd. Radius} = 500 \text{ ft}^2 \div 150 \text{ ft} = 3.33$$

Vel = 14 FPS

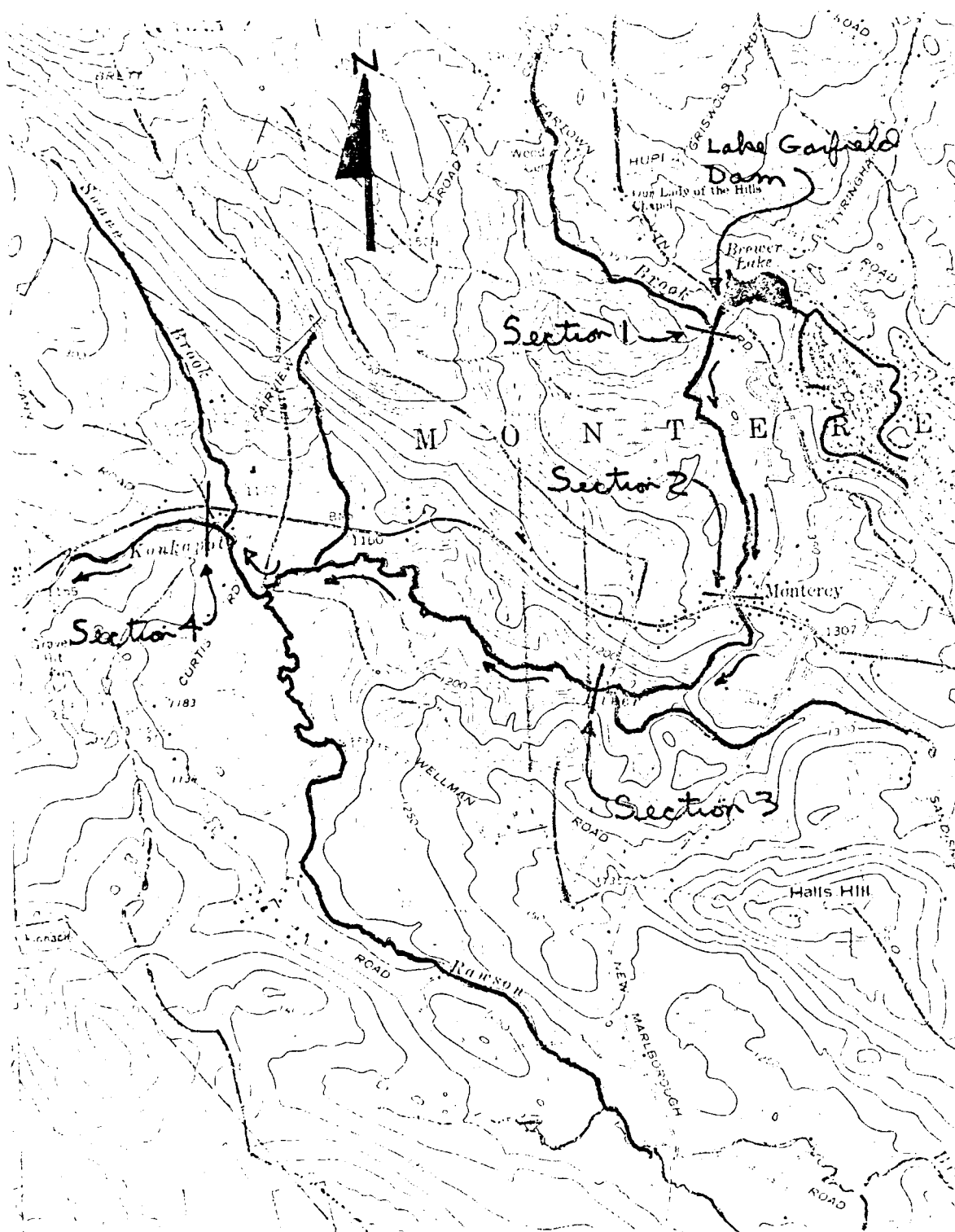
$$Q = 14 \times 500 \text{ ft}^2 = 7000 \text{ CFS}$$







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② Length of dam failure

mid-height dam length = 236 ft

$$40\% \times 236 \text{ ft} = \underline{\underline{95 \text{ ft}}}$$

③ Depth of Water at failure

Test Flood Pool Elev = 1293.

$$\begin{array}{r} \text{O.G. } \& \text{ dam Elev} = -1275. \\ \hline 18.2 \text{ ft} \end{array}$$

④ Peak Failure Outflow

$$Q_P = \left(\frac{8}{27}\right)(95')(\sqrt{32.2})(18.2)^{3/2}$$

$$Q_P = \underline{\underline{12,400 \text{ CFS}}}$$

Downstream Conditions With Dam Failure

reference: "Rule of Thumb Guidance For
Estimating Downstream Dam Failure
Hydrographs", COE, April, 1978.

Router Test Flood Outflow = 4450 CFS

Test Flood Pool Elevation = 1293.4

(Top of Dam elev 1295.08)

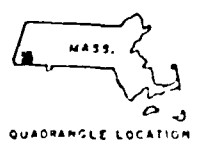
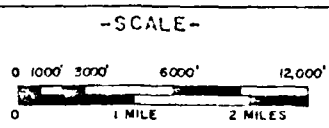
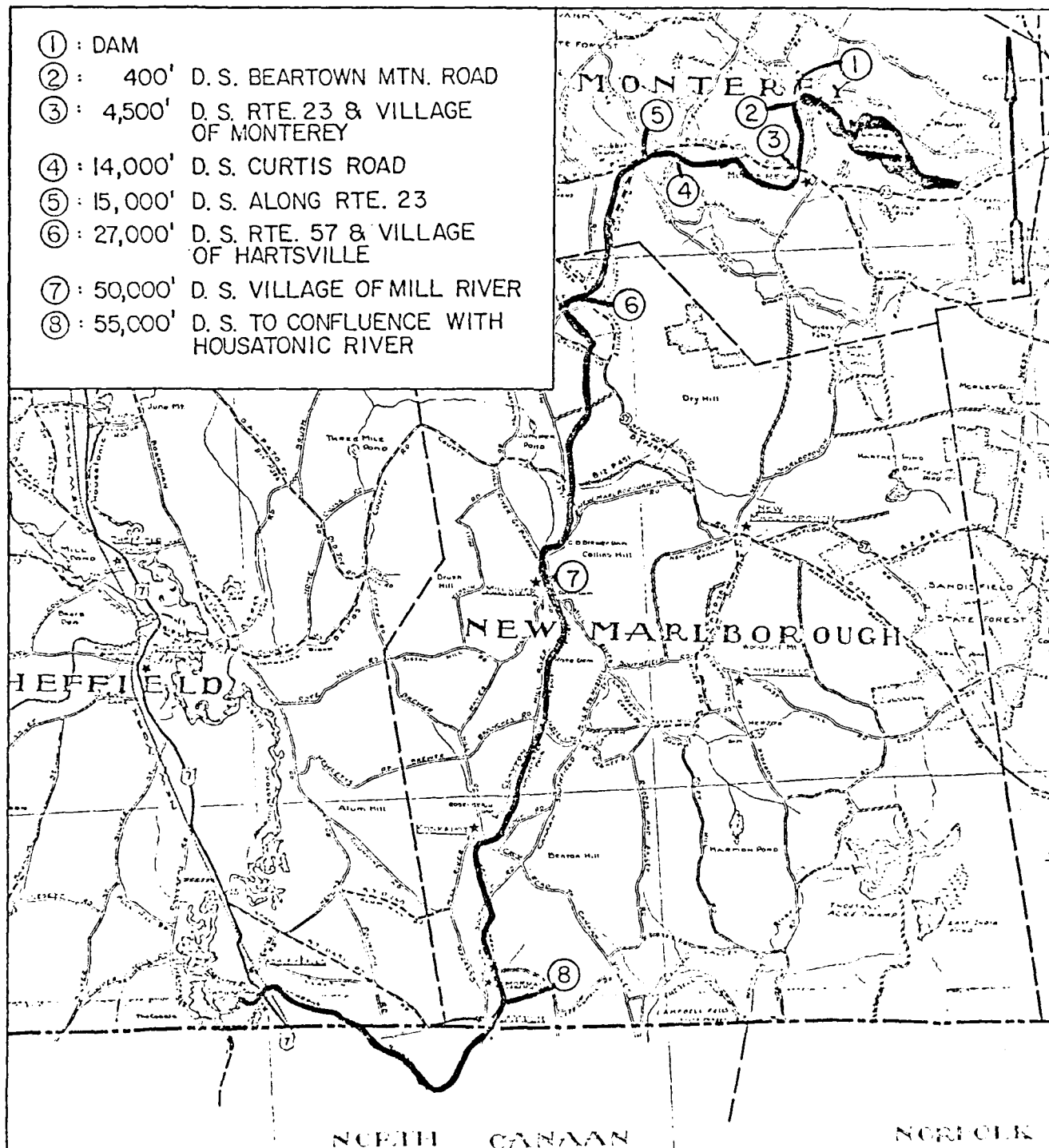
① Reservoir Storage @ Failure

The test flood routing indicated that Tyringham Rd. would be overtopped being subject to embankment breaching. This being the worst case for dam failure analysis would allow the entire Lake Garfield impoundment to be available as dam failure outflow. The impoundment storage at the test flood elevation of 1293.4 is 3200 acre-feet

Total Damage Potential :

	Before Failure	After Failure
②	Secondary road culvert overtopped $4 \pm$ ft.	overtopping increased $3 \frac{1}{2} \pm$ ft
③	Primary road culvert overtopped - road flooding $1.5 \pm$ ft. 6 houses flooded $1.5 \pm$ ft	overtopping increased $4 \pm$ ft 6 house flooded $5.5 \pm$ ft 1 houses flooded $3.5 \pm$ ft
④	Secondary road culvert overtopped $2 \pm$ ft	overtopping increased $4.5 \pm$ ft
⑤	No Damage	No Damage.
⑥	Primary Roadway flooded $1 \pm$ ft 13 houses flooded $1 \pm$ ft	overtopping increased $2 \pm$ ft 13 houses flooded $3 \pm$ ft
⑦	Two - Primary roads flooded $4.5 \pm$ ft	overtopping increased $5.5 \pm$ ft
⑧	10 Road Crossings flooded $4.5 \pm$ ft 1 house flooded $5 \pm$ ft	overtopping increased $2.5 \pm$ ft 1 house flooded $7.5 \pm$ ft

- ① : DAM
- ② : 400' D. S. BEARTOWN MTN. ROAD
- ③ : 4,500' D. S. RTE. 23 & VILLAGE OF MONTEREY
- ④ : 14,000' D. S. CURTIS ROAD
- ⑤ : 15,000' D. S. ALONG RTE. 23
- ⑥ : 27,000' D. S. RTE. 57 & VILLAGE OF HARTSVILLE
- ⑦ : 50,000' D. S. VILLAGE OF MILL RIVER
- ⑧ : 55,000' D. S. TO CONFLUENCE WITH HOUSATONIC RIVER



TIGHE & BOND / SCI
CONSULTING ENGINEERS
EASTHAMPTON, MASS.

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

NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS LOCATION AND DOWNSTREAM HAZARD MAP

LAKE GARFIELD DAM (MA 00249)
BERKSHIRE COUNTY

MONTEREY
MASSACHUSETTS

SCALE: AS NOTED
DATE: MARCH 1980

Downstream of Mill River

Q before failure = 4450 CFS

Q after failure = 7,600 CFS

Pre-failure flood flows will cause a river stage of about $7 \pm$ ft. There are 10 road crossings from Mill River to the confluence with the Housatonic River. All are expected to be exceeded by pre-failure flood flows.

Just upstream of the Village of Clayton there is one structure which will be flooded $5 \pm$ ft.

Post-failure flood flows will cause a river stage of $9.5 \pm$ ft and increase all flood levels by $2.5 \pm$ ft.

② Village of Mill River

Q before Failure 4450 CFS

Q after Failure 8,000 CFS

Pre-failure flood flows will cause a river stage of $5.5 \pm$ ft upstream of Mill River. There are two roadway culverts which will be overtopped by about 4.5 ft. It appears as though all structures are above flood level.

Past-Failure flood flows will cause a river stage of $10 \pm$ ft upstream of Mill River. Roadway flooding will be increased to about 10 ft. No structure flooding is anticipated.

⑤ Village of Hartsville

Q before Failure 4450 CFS

Q after Failure 11,100 CFS

Pre-failure flood flows will cause a river stage of $3 \pm$ ft upstream of Hartsville. The Route 57 culvert's capacity will be exceeded and the roadway overtopped by about 1 ft. Upstream river stage and roadway overtopping will result in flooding about 13 structures $1 \pm$ ft.

Post-failure flows will increase the flood levels by $2 \pm$ ft.

Pre-failure : 13 structures flooded 1 ft
Major Highway Overtopped 1 ft

Post failure : 13 structures flooded 3 ft
Major Highway Overtopped 3 ft.

Test Flood outflow before failure:

4450 CFS spillway outflow

River stage = 4 ft

Dam failure flow:

12,400 CFS dam failure

River stage = 6.6 ft

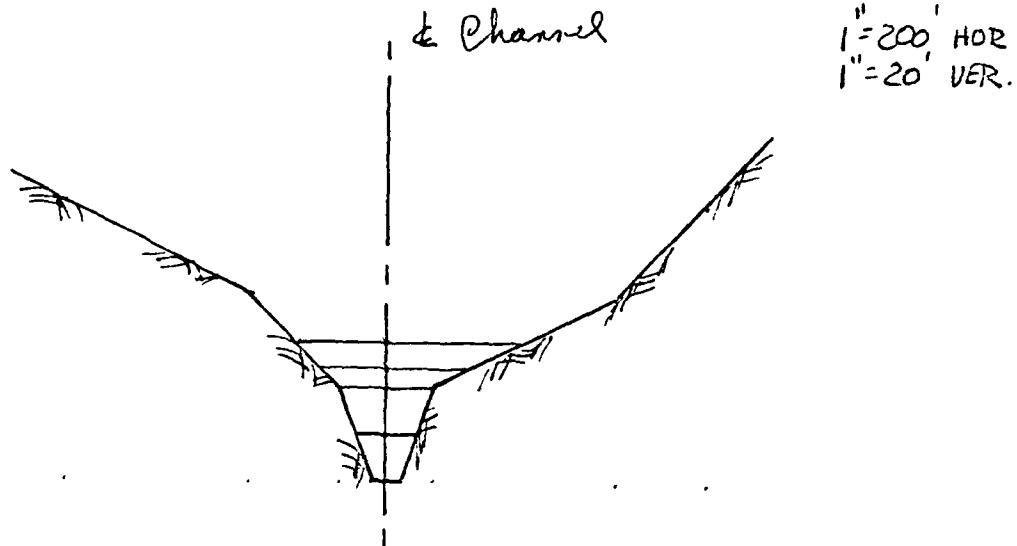
Section 1 is only 500 ft downstream of the dam, and the increase in stream stage is only 2.6 ft, therefore, dampening will be negligible.

Outflow to Reach 2 = 12,400 CFS

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Moe

Section 2 : Upstream of Route 23 at Village
Of Monterey



Channel Slope = 0.003

Manning $n = 0.04$

a) Depth = 5 ft

Top Width = 70 ft

$$\text{Area} = \frac{30 + 70}{2} \times 5 = 250 \text{ ft}^2$$

$$\text{Hyd. Rad.} = 250 \div 70 = 3.57$$

$$\text{Vel} = 4.7 \text{ FPS}$$

$$Q = 4.7 \times 250 = 1175 \text{ CFS}$$

b) Depth = 10 ft

Top Width = 100 ft

$$\text{Area} = \frac{30 + 100}{2} \times 10 = 650 \text{ ft}^2$$

$$\text{Hyd. Rad.} = 650 \div 100 = 6.5$$

$$\text{Vel} = 7.0 \text{ FPS}$$

$$Q = 7.0 \times 650 = 4,550 \text{ CFS}$$

c) Depth = 15 ft

Top Width = 250 ft

$$\text{Area} = \frac{30 + 250}{2} \times 15 = 2100 \text{ ft}^2$$

$$\text{Hyd. Rad.} = 2100 \div 250 = 8.4$$

$$\text{Vel} = 8.3 \text{ FPS}$$

$$Q = 8.3 \times 2,100 = 17,400 \text{ CFS}$$

d) Depth = 12 ft

Top width = 155 ft

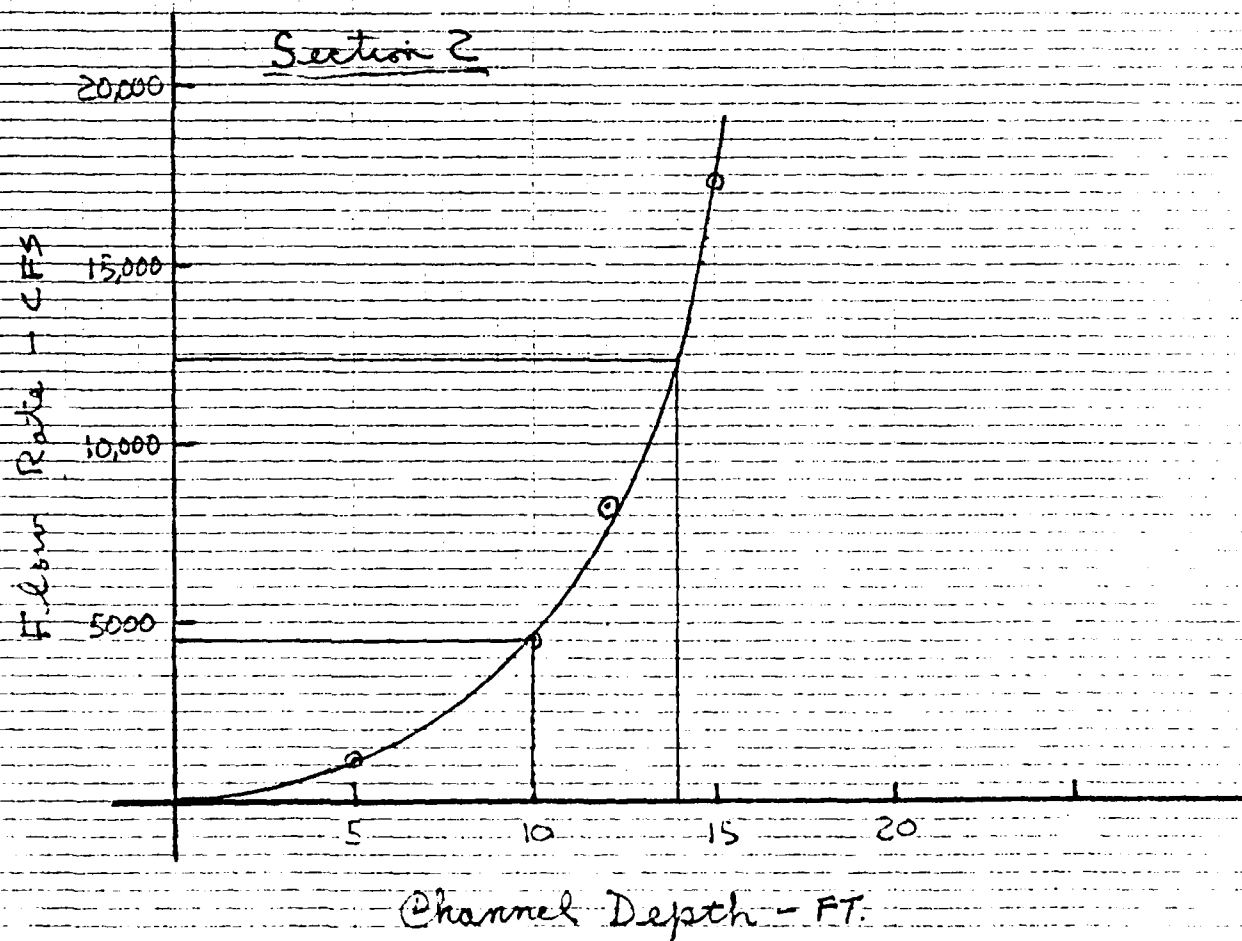
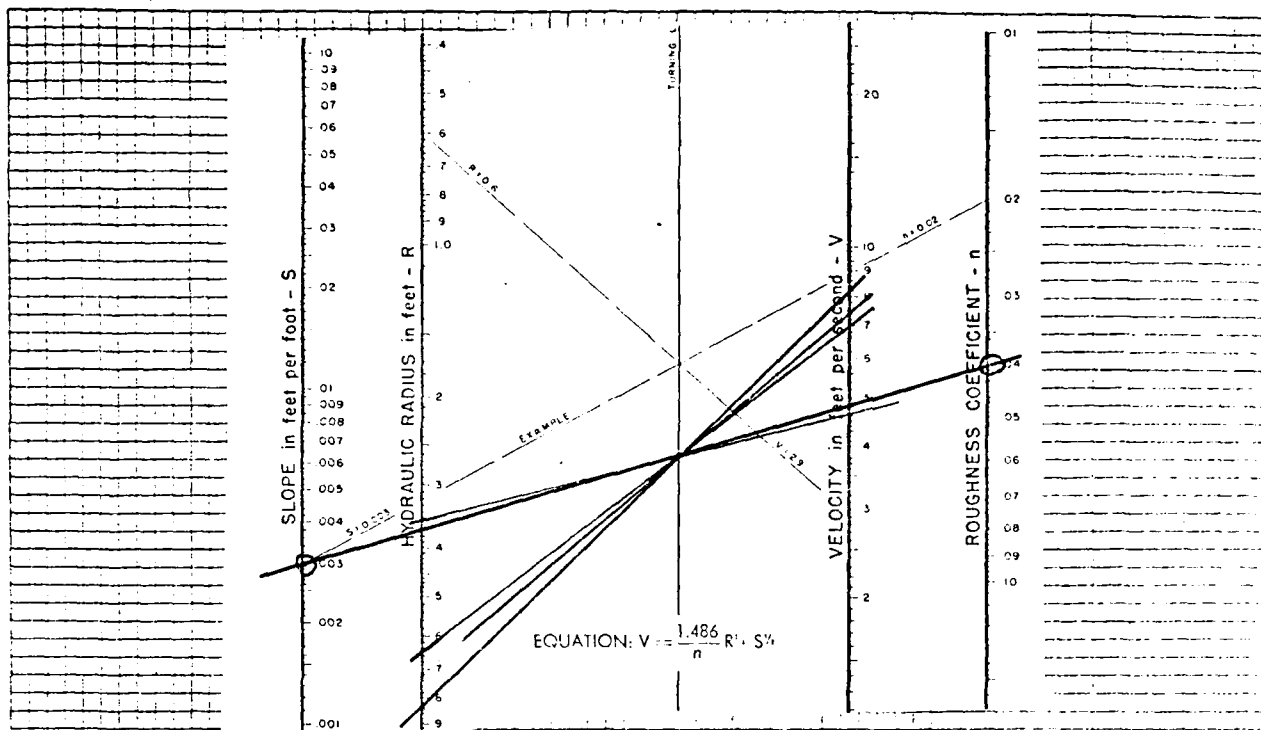
$$\text{Area} = \frac{30 + 155}{2} \times 12 = 1110 \text{ ft}^2$$

$$\text{Hyd. Rad.} = 1110 \div 155 = 7.16$$

$$\text{Vel} = 7.5 \text{ FPS}$$

$$Q = 7.5 \times 1110 = 8,300 \text{ CFS}$$

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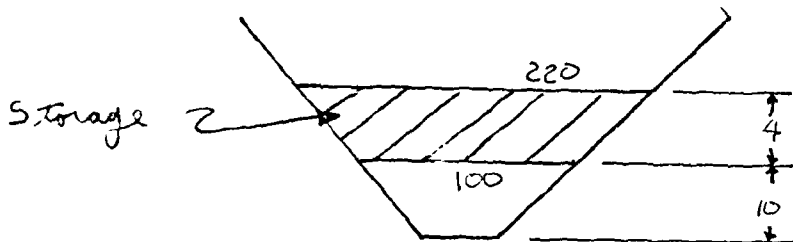
Test Flood Flow before failure : 4450 CFS

River stage = 10.0 ft

Dam failure flow : 12,400 CFS

River stage = 14.0 ft

Flow attenuation :



$$\text{Storage} = \frac{100 + 220}{2} \times 4 \times 3,600 = 53 \text{ acre-ft}$$

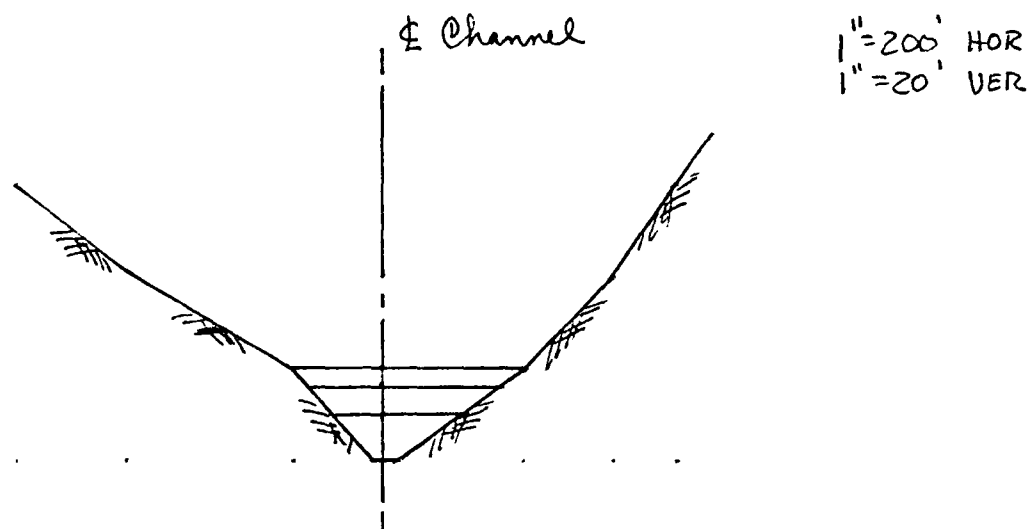
43,560

$$\text{attenuated Flow} = 12,400 \left(1 - \frac{53}{3200}\right) = 12,200 \text{ CFS}$$

Resulting River stage = 13.8 ft

13.8' not significantly lower than
unattenuated flow : outflow to
reach 3 = 12,200 CFS.

Section 3: Downstream of Confluence with
"Palmer Pond" Brook.



Channel Slope = 0.014

Manning $n = 0.04$

a) Depth = 5 ft

Top Width = 140 ft

$$\text{Area} = \frac{30 + 140}{2} \times 5 = 425 \text{ ft}^2$$

$$\text{Hyl. Rad} = 425 \div 140 = 3.03$$

$$\text{Vel} = 9.3 \text{ FPS}$$

$$Q = 9.3 \times 425 = 3950 \text{ CFS}$$

b) Depth = 10 ft

Top Width = 250 ft

$$\text{Area} = \frac{30 + 250}{2} \times 10 = 1400 \text{ ft}^2$$

$$\text{Hyd. Rad.} = 1400 \div 250 = 5.6$$

$$\text{Vel} = 13.5 \text{ FPS}$$

$$Q = 13.5 \times 1400 = 18,900 \text{ CFS}$$

c) Depth = 8 ft

Top Width = 210 ft

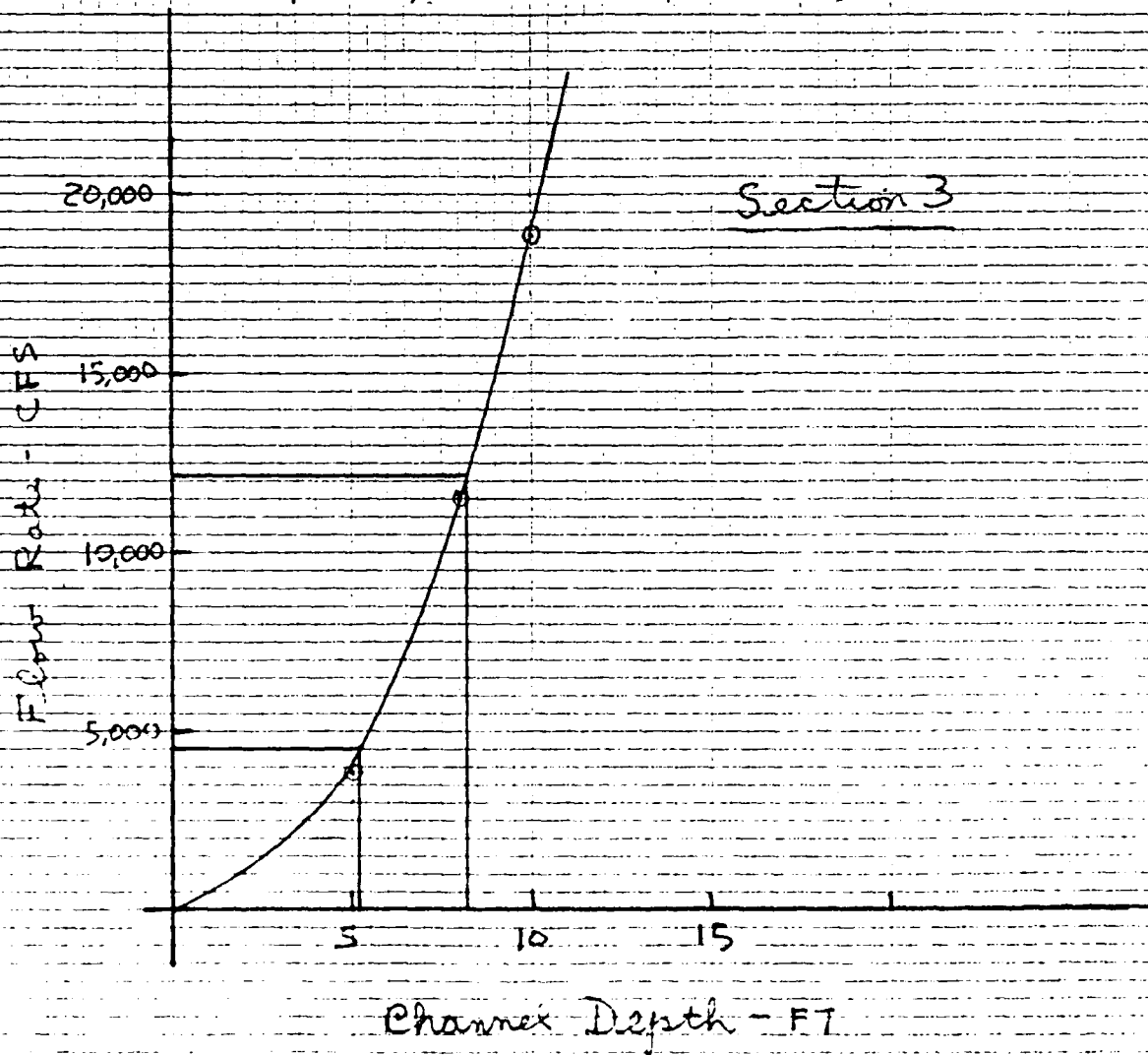
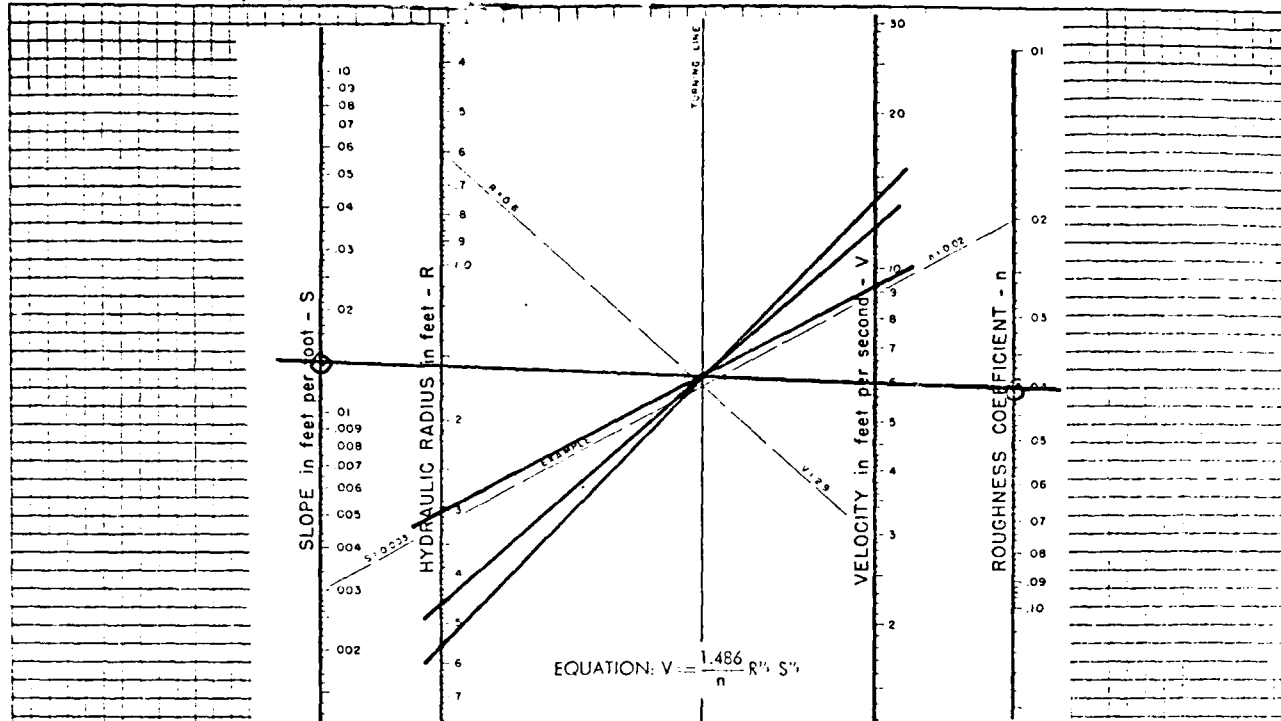
$$\text{Area} = \frac{30 + 210}{2} \times 8 = 960 \text{ ft}^2$$

$$\text{Hyd. Rad.} = 960 \div 210 = 4.57$$

$$\text{Vel} = 12 \text{ FPS}$$

$$Q = 12 \times 960 = 11,500 \text{ CFS}$$

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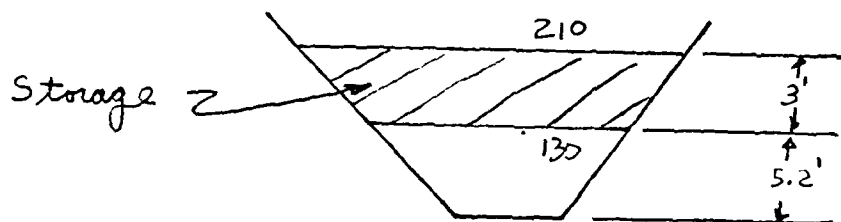
Test Flood Flow before failure : 4450 CFS

River stage = 5.2 ft

Dam failure flow : 12,200 CFS

River stage = 8.2 ft

Flow Attenuation



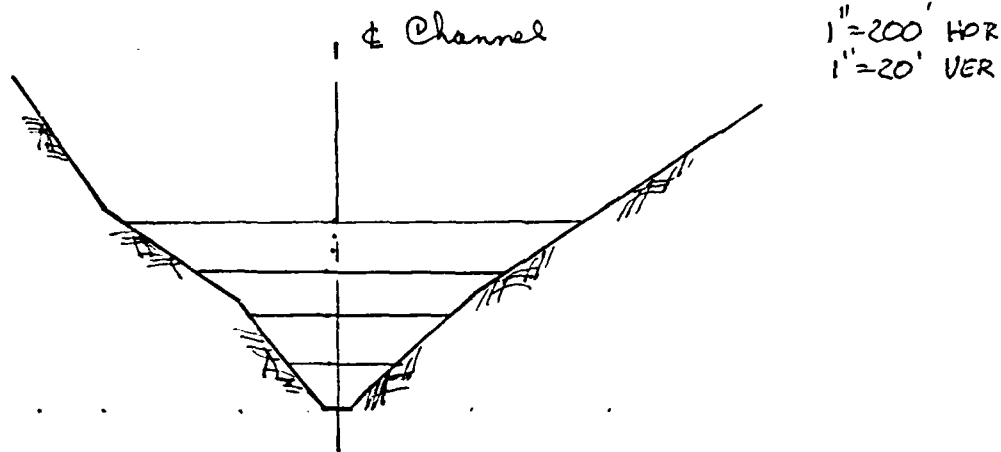
$$\text{Storage} = \frac{\frac{130 + 210}{2} \times 3 \times 3000}{43,560} = 35 \text{ acre-ft}$$

$$\text{attenuated flow} = 12,200 \left(1 - \frac{35}{3200}\right) = 12,100 \text{ CFS}$$

Resulting River stage = 3.1 ft

negligible difference $\therefore$ outflow to reach #4 = 12,100 CFS

Section 4: Downstream of Confluence w/ Swan Brook.



Channel Slope = 0.002

Manning $n = 0.04$

a) Depth = 5 ft

Top width = 115 ft

$$\text{Area} = \frac{50 + 115}{2} \times 5 = 412 \text{ ft}^2$$

$$\text{Hgl. Rad} = 412 \div 115 = 3.58$$

$$\text{Vel} = 3.8 \text{ FPS}$$

$$Q = 3.8 \times 412 = 1566 \text{ CFS}$$

b) Depth = 10 ft

Top Width = 220 ft

$$\text{Area} = \frac{50 + 220}{2} \times 10 = 1350 \text{ ft}^2$$

$$\text{Hyd. Rad} = 1350 \div 220 = 6.13$$

$$\text{Vel} = 5.4 \text{ FPS}$$

$$Q = 5.4 \times 1350 = 7300 \text{ CFS}$$

c) Depth = 15 ft

Top Width = 330 ft

$$\text{Area} = \frac{50 + 330}{2} \times 15 = 2850 \text{ ft}^2$$

$$\text{Hyd. Rad} = 2850 \div 330 = 8.63$$

$$\text{Vel} = 6.6 \text{ FPS}$$

$$Q = 6.6 \times 2850 = 18,800 \text{ CFS}$$

d) Depth = 20 ft

Top Width = 500 ft

$$\text{Area} = \frac{50 + 500}{2} \times 20 = 5,500 \text{ ft}^2$$

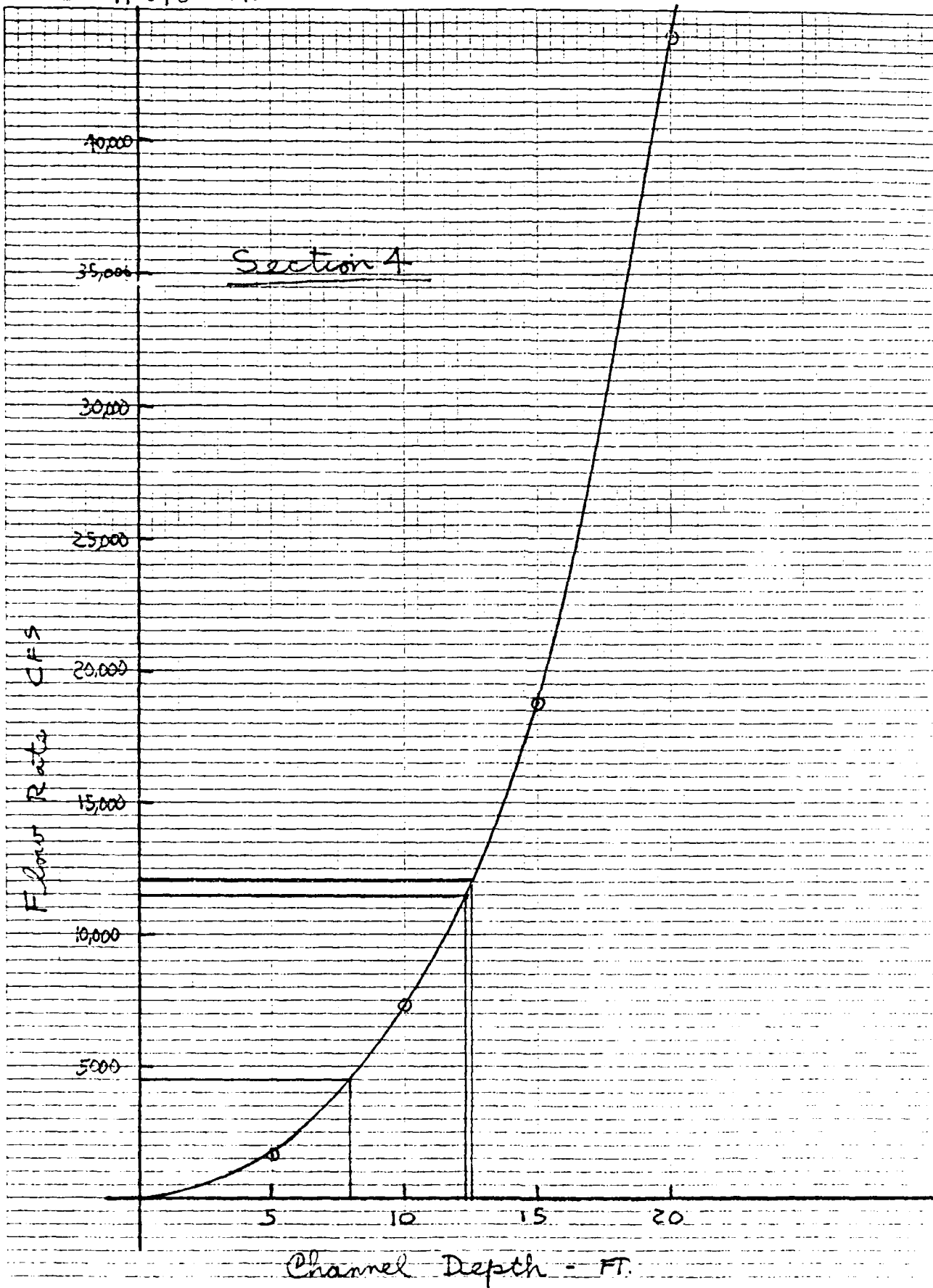
$$\text{Hyd. Rad.} = 5500 \div 500 = 11$$

$$\text{Vel} = 8 \text{ FPS}$$

$$Q = 44,000 \text{ CFS}$$

GHD 4/25/80 Mac

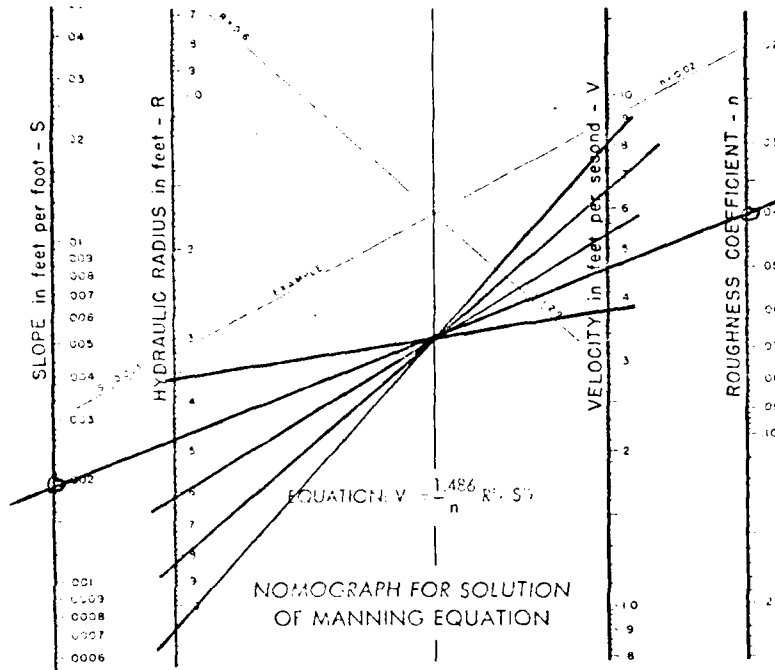
17



DHD 4/25/85

19.0

15



AD 4/25/80

Mac

USED 10/24/80 OHD

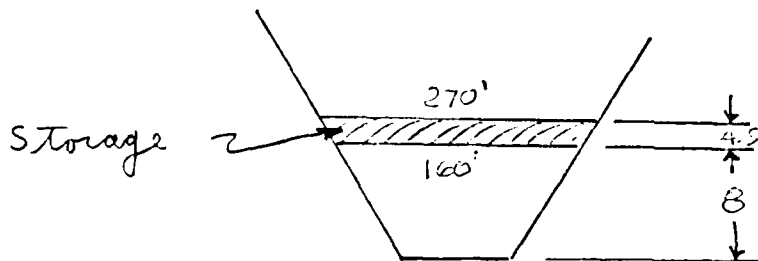
Test Flood flow before failure: 4450 CFS

River Stage = 8.0 ft

Dam Failure Flow: 12,100 CFS

River Stage = 12.5 ft

Flow attenuation:



$$\text{Storage} = \frac{160 + 270}{2} \times 4.5 \times 7,000 = 155 \text{ acre-ft}$$

43,500

$$\text{attenuated flow} = 12,100 \left(1 - \frac{155}{3200}\right) = 11,500 \text{ CFS}$$

Resulting River Stage = 12.1 ft

10/21/80

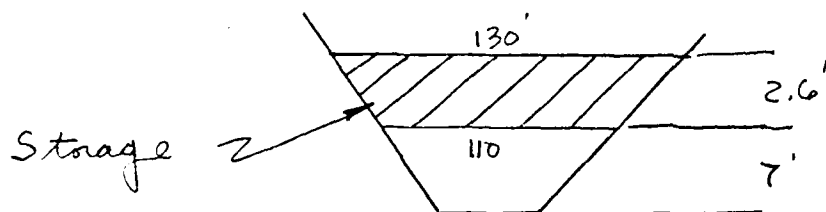
Pre Failure Test Flood Flow : 4450 CFS

River Stage = 7 ft

Dam Failure Flow : 8,000 CFS

River Stage = 9.6 ft

Attenuated Flow

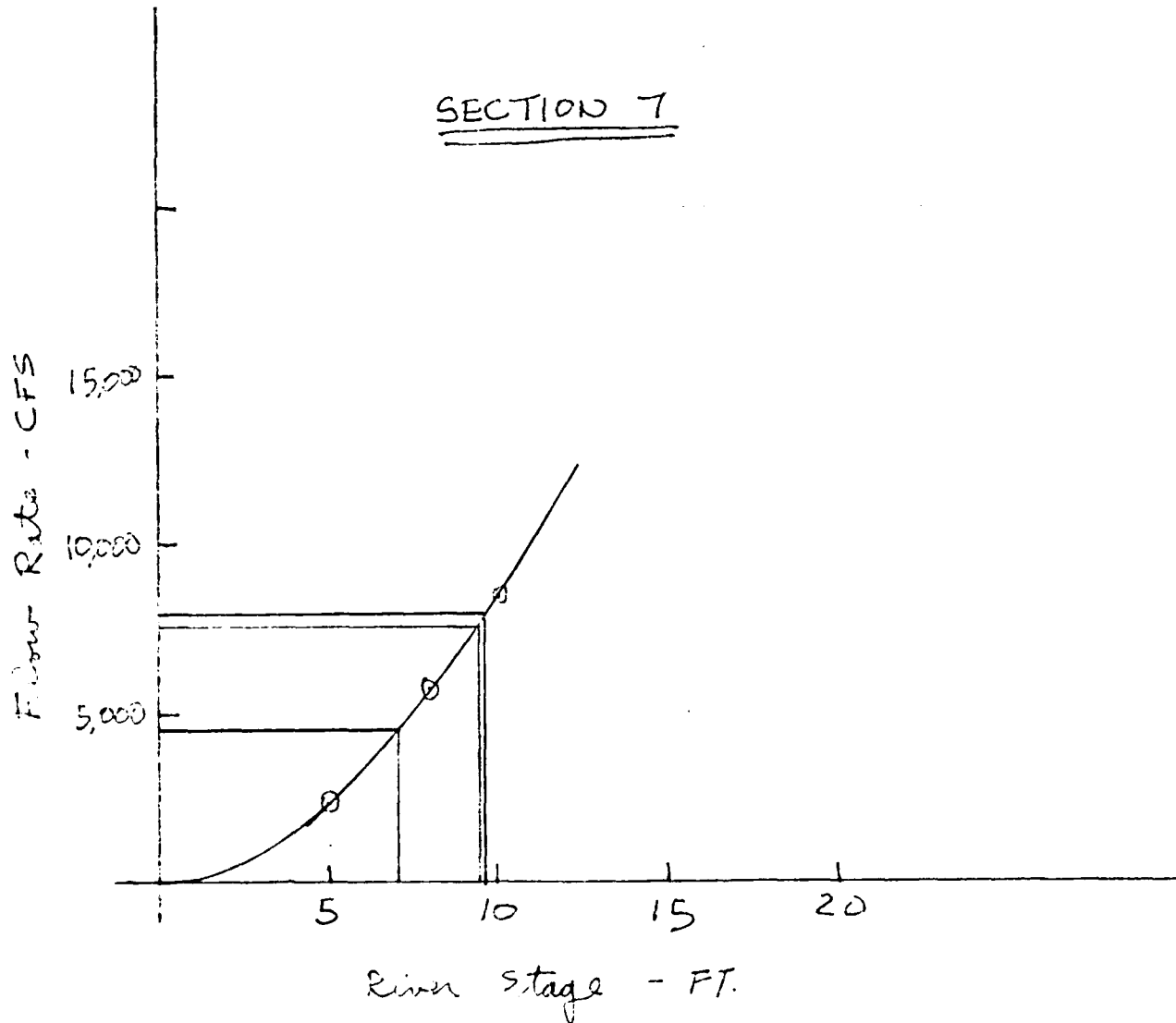
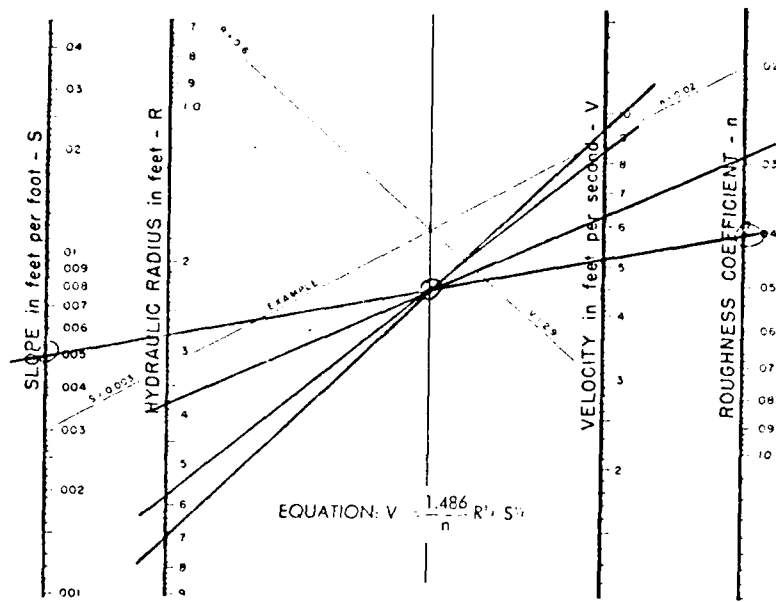


$$\text{Storage} = \frac{\frac{110 + 130}{2} \times 2.6 \times 20,000}{43560} = 143 \text{ acre-ft}$$

$$Q_1 = 8,000 \left(1 - \frac{143}{3200}\right) = 7,600 \text{ CFS}$$

River Stage = 9.5 ft

Outflow to Reach #8 = 7,600 CFS



b) Depth = 10 ft

Top width = 130 ft

$$\text{Area} = \frac{50 + 130}{2} \times 10 = 900 \text{ ft}^2$$

$$\text{Hyt. Rad.} = 900 \div 130 = 6.9$$

$$\text{Vel} = 9.4 \text{ FPS}$$

$$Q = 9.4 \times 900 = 8,460 \text{ CFS}$$

c) Depth = 8 ft

Top width = 120 ft

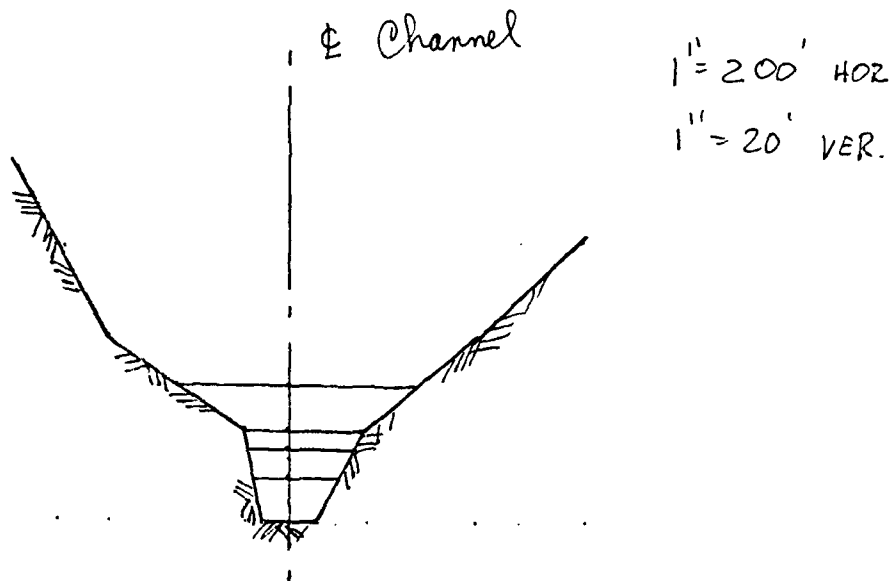
$$\text{Area} = \frac{50 + 120}{2} \times 8 = 680 \text{ ft}^2$$

$$\text{Hyt. Rad} = 680 \div 120 = 5.7$$

$$\text{Vel} = 8.5 \text{ FPS}$$

$$Q = 8.5 \times 680 = 5780 \text{ CFS}$$

Section 7 : Typical Section D.S. of
Village of Mill River



Channel Slope = 0.005

Manning $n = 0.04$

a) Depth = 5 ft

Top Width = 100 ft

$$\text{Area} = \frac{50 + 100}{2} \times 5 = 375 \text{ ft}^2$$

$$\text{Hyd. Rad} = 375 \div 100 = 3.8$$

$$\text{Vel} = 6.3 \text{ FPS}$$

$$Q = 6.3 \times 375 = 2360 \text{ CFS}$$

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$$Q_2 = 11,100 \times \left(1 - \frac{467}{2966}\right) = 9,500 \text{ CFS}$$

$$\text{River Stage} = 9.4 \text{ ft}$$

$$\text{Str 3} = \frac{\frac{550 + 590}{2} \times 3.9 \times 20,000}{43,560} = 1020 \text{ ac-ft}$$

$$Q_3 = 11,100 \left(1 - \frac{1020}{3200}\right) = 7,560 \text{ CFS}$$

$$\text{River Stage} = 7.7 \text{ ft}$$

$$\text{Str 4} = \frac{\frac{550 + 585}{2} \times 2.2 \times 20,000}{43,560} = 573 \text{ ac-ft}$$

$$Q_4 = 11,100 \left(1 - \frac{573}{3200}\right) = 9,100 \text{ CFS}$$

$$\text{River Stage} = 8.8 \text{ ft}$$

$\therefore$ River stage is : $7.7 < \text{R.S.} > 8.8$

$$\text{River stage} = 8.2 \text{ ft}$$

$$\text{Outflow} = 8,000 \text{ CFS}$$

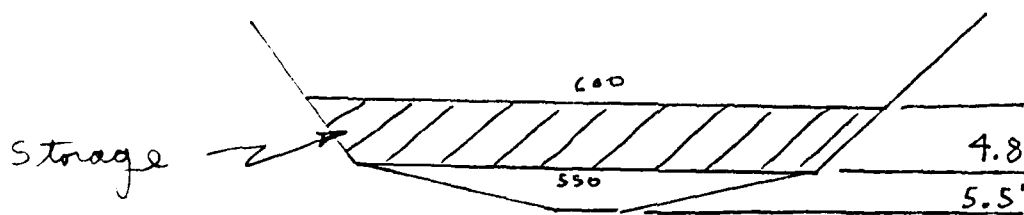
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Test Flood Flow Before Failure : 4450 CFS

River stage = 5.5 ft

Dam Failure Flow : 11,100 CFS

River stage = 10.3 ft

Flow attenuation :

$$\text{Storage} = \frac{550 + 600}{2} \times 4.8 \times 20,000 = 1267 \text{ acre-ft}$$

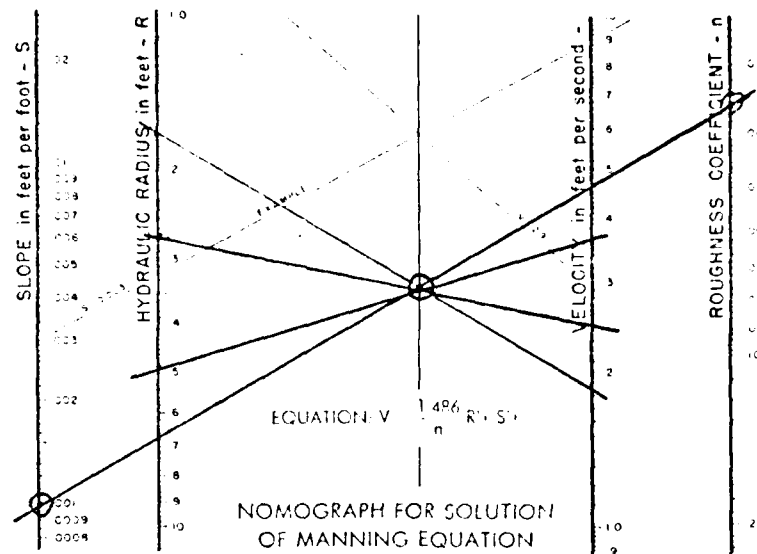
43,560

$$\text{attenuated Flow} = 11,100 \left(1 - \frac{1267}{3200}\right) = 6,700 \text{ CFS}$$

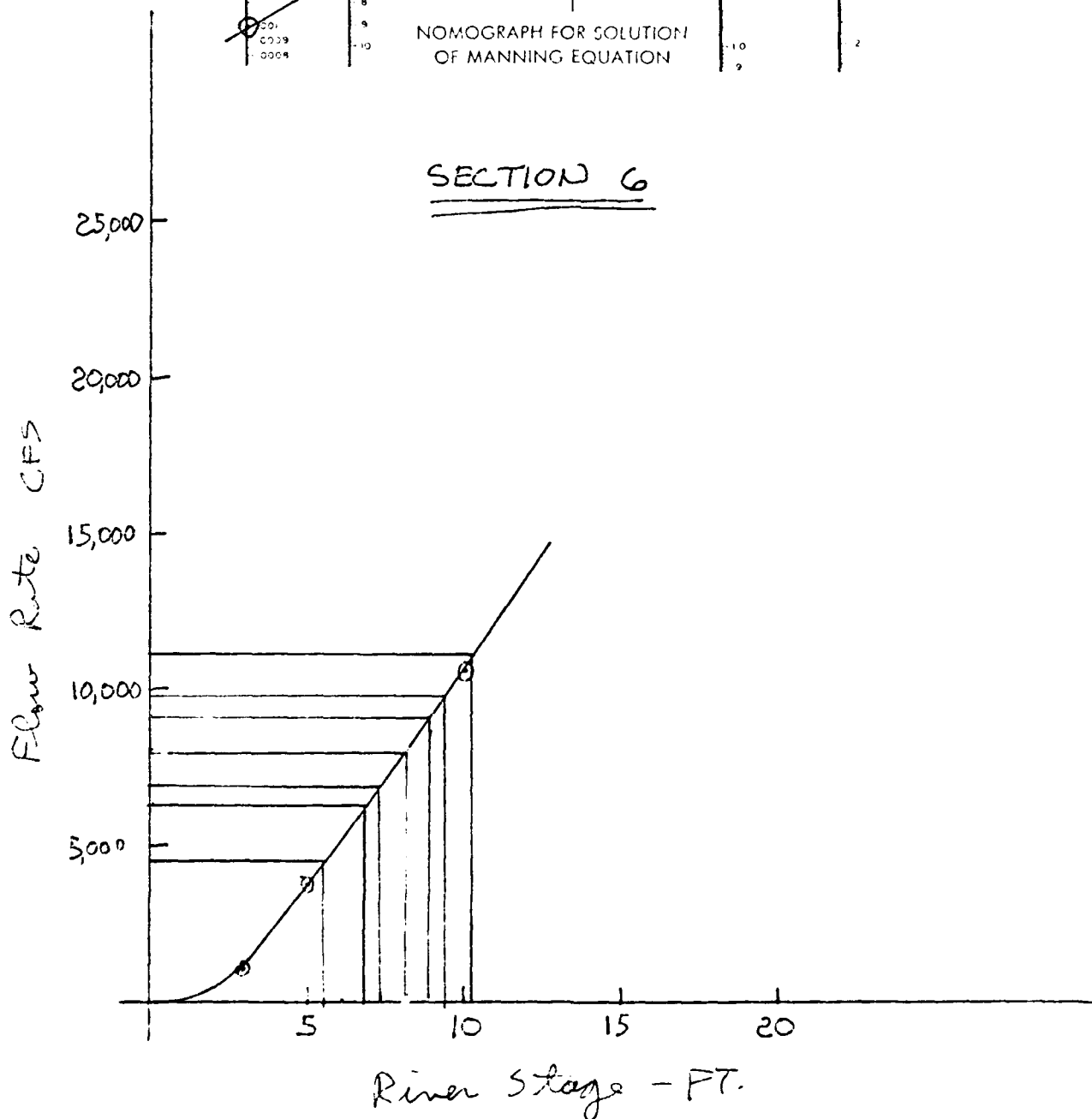
River stage = 7.3 ft

$$\text{Storage} = \frac{550 + 580}{2} \times 1.8 \times 20,000 = 467 \text{ acre-ft}$$

43,560



SECTION 6



b) Depth = 5 ft

Top Width = 550 ft

$$\text{Area} = \frac{50 + 550}{2} \times 5 = 1500 \text{ ft}^2$$

$$\text{Hyd. Rad} = 1500 \div 550 = 2.7$$

$$\text{Vel} = 2.5 \text{ FPS}$$

$$Q = 2.5 \times 1500 = 3750 \text{ CFS}$$

c) Depth = 10 ft

Top Width = 600 ft

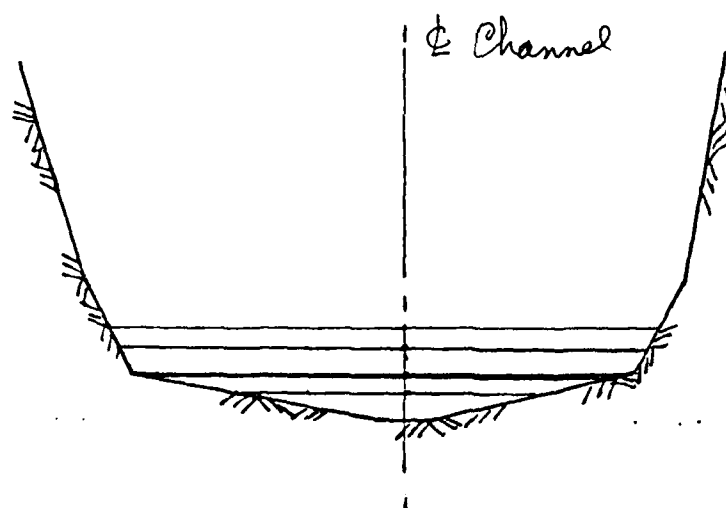
$$\text{Area} = \left(\frac{50 + 550}{2} \times 5 \right) + \left(\frac{550 + 600}{2} \times 5 \right) = 2940 \text{ ft}^2$$

$$\text{Hyd. Rad} = 2940 \div 600 = 4.9$$

$$\text{Vel} = 3.6 \text{ FPS}$$

$$Q = 3.6 \times 2940 = 10,600 \text{ CFS}$$

Section 6: Typical Section Through
Flat, Meandering Section in
New Marlborough.



1" = 200' HOR

1" = 20' VER.

Channel Slope = 0.001

Manning $n = 0.035$

a) Depth = 3 ft

Top Width = 320'

$$\text{Area} = \frac{50 + 320}{2} \times 3 = 555 \text{ ft}^2$$

$$\text{Hyd. Rad} = 555 \div 320 = 1.7$$

$$\text{Vel} = 1.8 \text{ FPS}$$

$$Q = 1.8 \times 555 \text{ ft}^2 = 1000 \text{ CFS}$$

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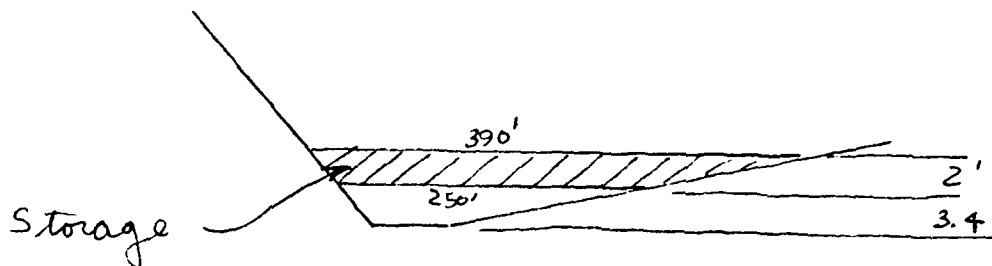
Test Flood Flow Before Failure : 4450 CFS

River Stage = 3.4 ft

Dam Failure Flow : 11,600 CFS

River Stage = 5.4 ft

Flow attenuation



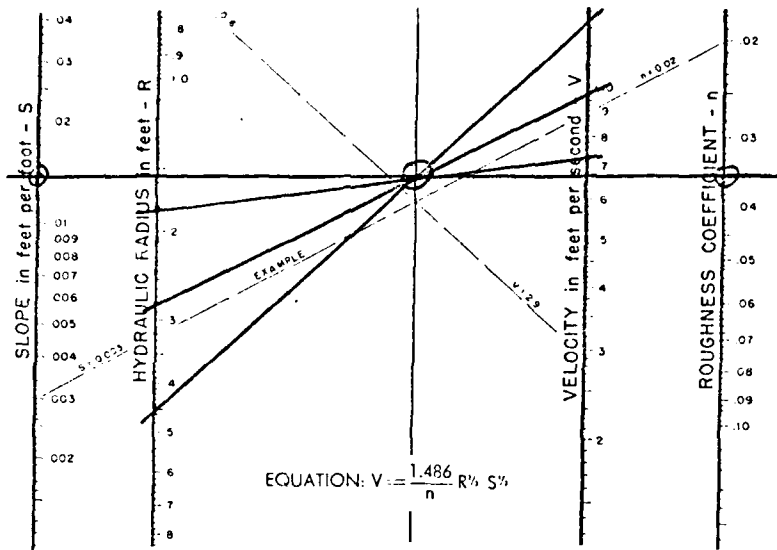
$$\text{Storage} = \frac{\frac{250 + 390}{2} \times 2 \times 11,000}{43,560} = 162 \text{ acre-ft}$$

$$\text{attenuated Flow} = 11,600 \left(1 - \frac{162}{3200}\right) = 11,000 \text{ CFS}$$

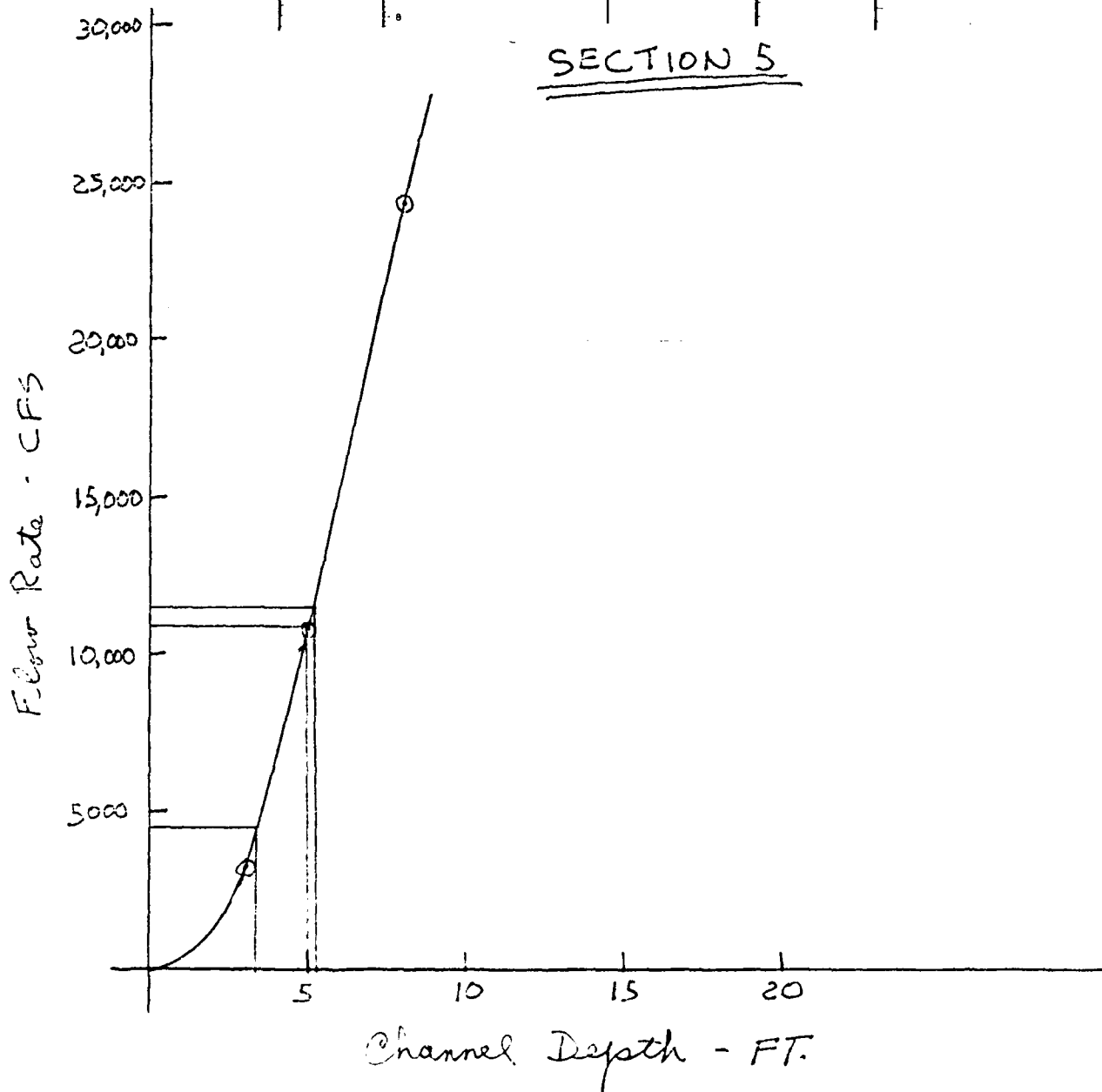
River Stage = 5.0 ft

$$\text{Storage}_2 = \frac{\frac{245 + 390}{2} \times 1.6 \times 11,000}{43,560} = 128 \text{ acre-ft}$$

$$\begin{aligned} \text{attenuated Flow} &= 11,600 \left(1 - \frac{128}{3200}\right) = 11,100 \text{ CFS} \\ \text{Flow to Reach \# 6} &= \underline{\underline{11,100 \text{ CFS}}} \end{aligned}$$



SECTION 5



b) Depth = 3 ft

Top Width = 240 ft

$$\text{Area} = \frac{50 + 240}{2} \times 3 = 435 \text{ ft}^2$$

$$\text{Hyd. Rad} = 435 \div 240 = 1.81$$

$$\text{Vel} = 7.3 \text{ FPS}$$

$$Q = 3175 \text{ CFS}$$

c) Depth = 8 ft

Active Flow Section Top Width = 400'
(all else is only storage)

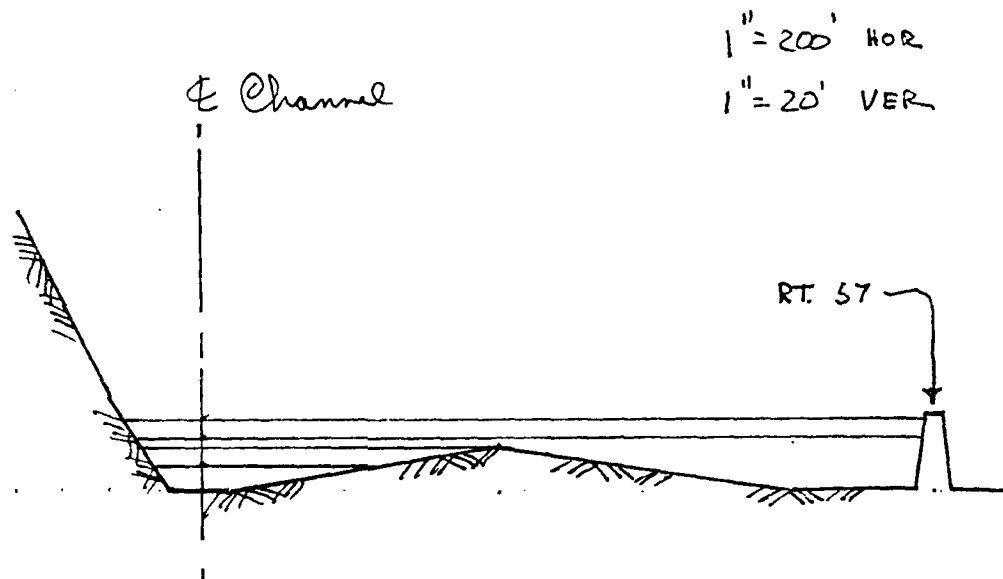
$$\text{Area} = \frac{50 + 400}{2} \times 8 = 1800 \text{ ft}^2$$

$$\text{Hyd. Rad.} = 1800 \div 400 = 4.5$$

$$\text{Vel} = 13.5 \text{ FPS}$$

$$Q = 13.5 \times 1800 = 24,300 \text{ CFS}$$

Section 5: at Crossing of
in Village of Hartsville



Channel Slope = 0.014

Manning $n = 0.035$

a) Depth = 5 ft

Top width = 390 ft

$$\text{Area} = \frac{50 + 390}{2} \times 5 = 1100 \text{ ft}^2$$

$$\text{Hyal. Rad.} = 1100 \div 390 = 2.8$$

$$\text{Vel} = 9.8 \text{ FPS}$$

$$Q = 9.8 \times 1100 = 10,780 \text{ CFS}$$

OHD 4/25/80

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REVISED 10/21/80 OHD

$$\text{Storage}_2 = \frac{\frac{250 + 100}{2} \times 4.1 \times 7000}{43,560} = 135 \text{ acre-ft}$$

$$\text{Attenuated Flow} = 12,100 \left(1 - \frac{135}{3200}\right) = 11,600 \text{ CFS}$$

$$\therefore \text{outflow to reach 5} = \underline{\underline{11,600 \text{ CFS}}}$$

Downstream of New Marlborough the Konkagot River again flattens out into broad flood plain areas and a significant swamp area just upstream of the Housatonic River confluence. The dam failure flow remaining at this point will be attenuated to nearly the test flood pre-failure flow prior to reaching the Housatonic River.

Flood stages less than 10 ft beyond Reach 7 will not create any additional hazard potential.

Culvert Capacities

$$\text{Head} - \text{losses} = \frac{V^2}{2g}$$

$$\text{losses} = \text{entrance} + \text{friction}$$

$$\text{losses} = 0.5 \frac{V^2}{2g} + f \frac{L}{D} \frac{V^2}{2g}$$

$$\text{Head} = \frac{V^2}{2g} + 0.5 \frac{V^2}{2g} + f \frac{L}{D} \frac{V^2}{2g}$$

1. Beartown Mountain Rd. Culvert

$$\text{dia} = 6 \text{ ft}$$

$$\text{length} = 29.5 \text{ ft}$$

Pipe material is steel penstocks tube

$$f = 0.016$$

$$f \frac{L}{D} = 0.016 \left(\frac{29.5}{6} \right) = 0.078 - \text{negligible}$$

$$\text{Head} = \frac{V^2}{2g} + 0.5 \frac{V^2}{2g} = 1.5 \frac{V^2}{2g}$$

$$V = \sqrt{\frac{2gH}{1.5}}$$

ΔH inlet to outlet allow 2' due to downstream backwater.

$$V = \sqrt{\frac{(2)(32.2)(2)}{1.5}} = 9.3 \text{ FPS}$$

$$Q = \frac{(6)^2 \pi}{4} \times 9.3 \text{ FPS} = 262 \text{ CFS}$$

2. Route 23 Culvert @ Monterey

ACCOMP Pipe Arch.

Height of arch = 10 ft

Width of arch = 16 ft

Length = 40.3 ft

$$\text{Area} = 131 \text{ ft}^2$$

$$f = 0.02$$

$$f \frac{L}{D} = 0.02 \left(\frac{40.3}{13} \right) = 0.06 \text{ negligible}$$

$$\text{Head} = 1.5 \frac{V^2}{2g}$$

$$V = \sqrt{\frac{2gH}{1.5}}$$

ΔH inlet to outlet allow 2' due to downstream backwater.

$$V = \sqrt{\frac{(2)(32.2)(2)}{1.5}} = 9.3 \text{ FPS}$$

$$Q = 9.3 \times 131 = 1200 \text{ CFS}$$

APPENDIX E

INFORMATION AS CONTAINED IN THE
NATIONAL INVENTORY OF DAMS

NOT AVAILABLE AT THIS TIME

END

FILMED

7-85

DTIC