AD-A155 502 NATIONAL PROGRAM FOR INSPECTION OF NON-FEDERAL DAMS 1/2
CHICOPEE RESERVOIR (M..(U) CORPS OF ENGINEERS WALTHAM
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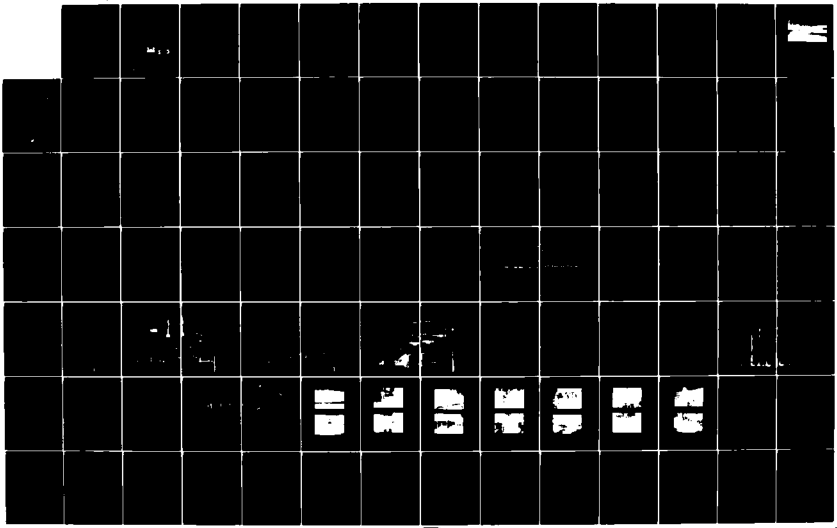

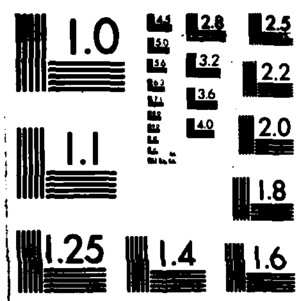

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AD-A155 502

CONNECTICUT RIVER BASIN

CHICOPEE, MASSACHUSETTS

CHICOPEE RESERVOIR
MA 00720

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

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

JUNE 1981

UNCLASSIFIED

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

REPORT DOCUMENTATION PAGE

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1. REPORT NUMBER: MA 00720
2. GOVT ACCESSION NO.: AD-A155502
3. RECIPIENT'S CATALOG NUMBER:
4. TITLE (and Subtitle): Chicopee Reservoir

   NATIONAL PROGRAM FOR INSPECTION OF NON-FEDERAL DAMS
5. TYPE OF REPORT & PERIOD COVERED: INSPECTION REPORT
6. PERFORMING ORG. REPORT NUMBER:
7. AUTHOR(s): U.S. ARMY CORPS OF ENGINEERS NEW ENGLAND DIVISION
8. CONTRACT OR GRANT NUMBER(s):
9. PERFORMING ORGANIZATION NAME AND ADDRESS:
10. PROGRAM ELEMENT, PROJECT, TASK AREA & WORK UNIT NUMBERS:
11. CONTROLLING OFFICE NAME AND ADDRESS: DEPT. OF THE ARMY, CORPS OF ENGINEERS NEW ENGLAND DIVISION, NEDED 424 TRAPELO ROAD, WALTHAM, MA. 02254
12. REPORT DATE: June 1981
13. NUMBER OF PAGES: 80
14. MONITORING AGENCY NAME & ADDRESS (if different from Controlling Office):
15. SECURITY CLASS. (of this report): UNCLASSIFIED

15a. DECLASSIFICATION/DOWNGRADING SCHEDULE:

16. DISTRIBUTION STATEMENT (of this Report)

APPROVAL FOR PUBLIC RELEASE: DISTRIBUTION UNLIMITED

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

18. SUPPLEMENTARY NOTES

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

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

DAMS, INSPECTION, DAM SAFETY,

Connecticut River Basin
Chicopee, Massachusetts
Cooley Brook Tributary of the Chicopee River

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

THe dam is a 545 ft. long, 47 ft. high earth embanment dam. There are deficiencies which shauld be corrected to assure the continued performance of the dam. Generally the dam is in fair condition. The size is intemediate and the hazard potential is high.

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DEPARTMENT OF THE ARMY
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"Original cont... plates: ... ions will be in ... white"

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

Dear Governor King:

Inclosed is a copy of the Chicopee Reservoir (MA-00720) Phase I Inspection Report, prepared under the National Program for Inspection of Non-Federal Dams. This report is based upon a visual inspection, a review of the past performance and a brief hydrological study of the dam. I approve the report and support the findings and recommendations described in Section 7 and ask that you keep me informed of the actions taken to implement them. This follow-up action is vitally important.

Copies of this report have been forwarded to the Department of Environmental Quality Engineering. Copies will be available to the public in thirty days.

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

Sincerely,

C. E. Edgar III

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

Incl
As stated

NATIONAL DAM INSPECTION
PROGRAM

PHASE I INSPECTION REPORT

BRIEF ASSESSMENT

Identification No. MA 00720

Name of Dam: Chicopee Reservoir (Cooley Dam)

City: Chicopee

County and State: Hampden County, Massachusetts

Stream: Cooley Brook Tributary of the Chicopee River

Date of Inspection: March 4, 1981

Chicopee Reservoir Dam is a 545-foot long, 47-foot high earth embankment dam constructed circa 1926. The storage capacity of the Reservoir at the top of the dam is 695 acre-feet. There is a reinforced concrete core wall in the dam with steel sheet piling indicated below the core wall. The spillway is a 44-foot long ogee crest which is located at the right abutment of the dam and has a crest elevation (El) 168.0. The outlet conduits are a 24-inch cast-iron former water supply line and a 30-inch cast-iron blowoff. Both pipes are located between the center of the dam and the right abutment. There are two 24-inch intake conduits which are controlled by sluicegates in the gatehouse where they discharge into the intake chamber and the 24-inch former supply line to the City of Chicopee water system. Flow through the 30-inch blowoff is controlled by a gate valve located in the gatehouse. The 30-inch blowoff discharges into a small reservoir immediately downstream from the dam. Formerly, Chicopee Reservoir was used to provide water storage and to regulate its release as part of the supply system for the City of Chicopee, Massachusetts. The reservoir is presently used for recreational purposes as part of Chicopee Memorial State Park.

The following deficiencies were observed at the site: high degree of erosion of the principal spillway chute structure and outlet training walls; erosion gullies on the right abutment above the dam crest; brush growth on the downstream slope of the embankment; and seepage from the left abutment downstream of the dam. Generally, the dam is in fair condition.

Based upon size classification, intermediate, and hazard potential, high, in accordance with the Corps of Engineers Guidelines, the adopted Spillway Test Flood is equal to the Probable Maximum Flood which yields a peak test flood inflow of 7565 cfs. Hydraulic analyses indicate that the spillway, without flashboards, with the water surface at the top of the dam can discharge 3,950 cfs which is less than the total routed test flood outflow of 6,450 cfs. Thus, the spillway can discharge 60% of the routed test flood outflow.

It is recommended that the owner employ a qualified Registered Professional Engineer to undertake an investigation of the seepage emanating from the left abutment and to prepare plans for rehabilitation of all spalling and erosion of the principal spillway. In addition, the owner should repair the deficiencies listed above, as described in Section 7.3. The owner should also implement a program of annual technical inspections, a plan for surveillance of the dam during and after periods of heavy rainfall, and a plan for notifying downstream residents in the event of an emergency at the dam.

The measures outlined above and in Section 7 should be implemented by the owner within a period of one (1) year after receipt of this Phase I Investigation Report.

COMMONWEALTH OF MASSACHUSETTS
WILLIAM S. PARKER
No. 25373
REGISTERED PROFESSIONAL ENGINEER

Cullinan Engineering Co., Inc.

William S Parker

William S. Parker, PE
Director of Engineering
Project Manager

This Phase I Inspection Report on Chicopee Reservoir (MA-00720) has been reviewed by the undersigned Review Board members. In our opinion, the reported findings, conclusions, and recommendations are consistent with the Recommended Guidelines for Safety Inspection of Dams, and with good engineering judgement and practice, and is hereby submitted for approval.

Carney M. Terzian

CARNEY M. TERZIAN, MEMBER
Design Branch
Engineering Division

Joe Finegan

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

Aramast Mahtesian

ARAMAST MAHTESIAN, CHAIRMAN
Geotechnical Engineering Branch
Engineering Division

APPROVAL RECOMMENDED:

Joe B. Fryar

JOE B. FRYAR
Chief, Engineering Division

PREFACE

This report is prepared under guidance contained in Recommended Guidelines for Safety Inspection of Dams, for a Phase I Investigation. 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 investigations, and analyses involving topographic mapping, subsurface investigations, testing, and detailed computational evaluations are beyond the scope of a Phase I Investigation; however, the investigation is intended to identify any need for such studies.

In reviewing this report, it should be realized that the reported condition of the dam is based on observations of field conditions at the time of inspection along with data available to the inspection team. In cases where the reservoir was lowered or drained prior to inspection, such action, while improving the stability and safety of the dam, removes the normal load on the structure and may obscure certain conditions which might otherwise be detectable if inspected under the normal operating environment of the structure.

It is important to note that the condition of a dam depends on numerous and constantly changing internal and external conditions, and is evolutionary in nature. It would be incorrect to assume that the present condition of the dam will continue to represent the condition of the dam at some point in the future. Only through continued care and inspection can there be any chance that unsafe conditions be detected.

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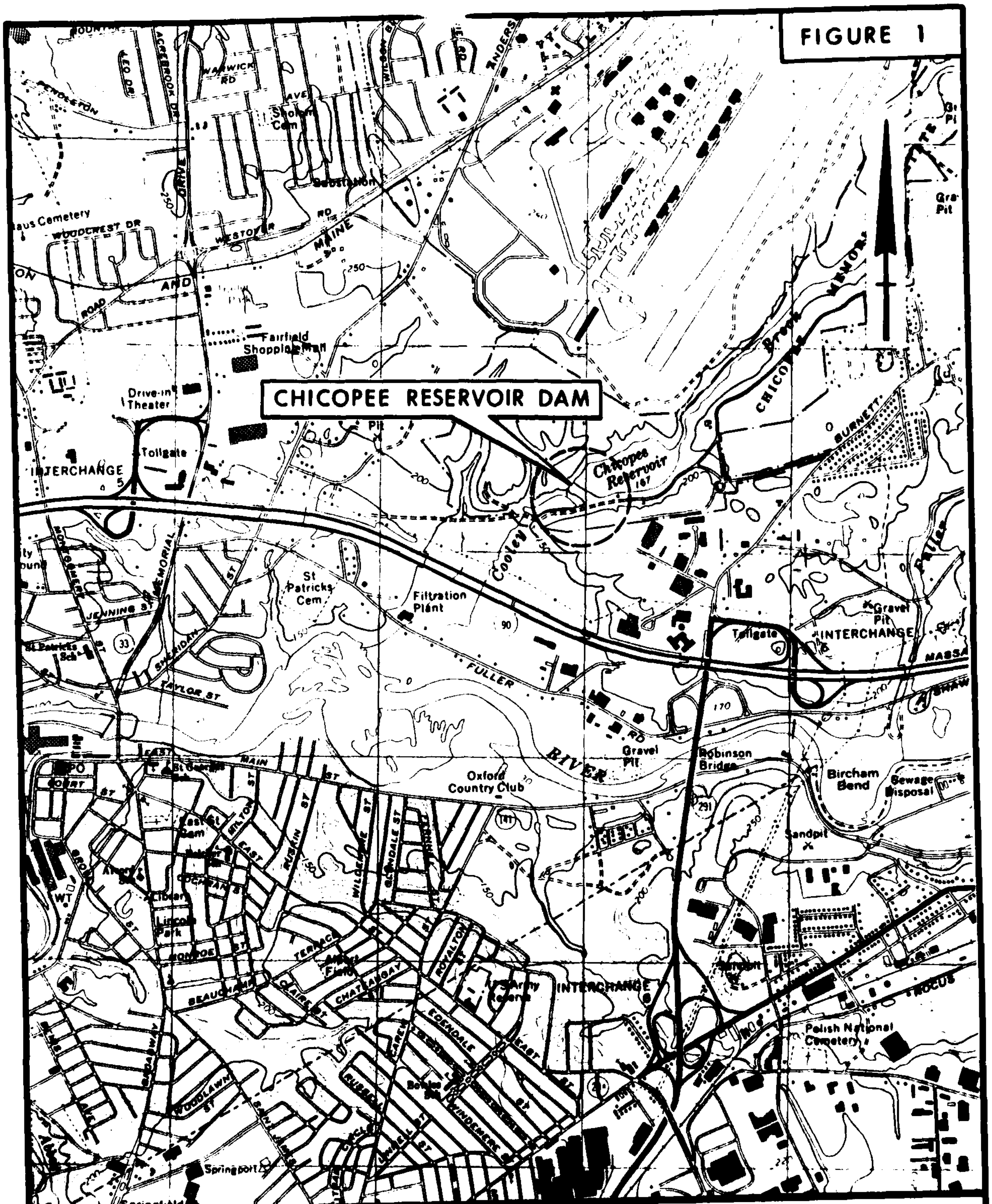
APPENDIXES

OVERVIEW

U.S. ARMY CORPS OF ENGINEERS NEW ENGLAND DIVISION WALTHAM, MASSACHUSETTS	NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS	Chicopee Reservoir Dam Cooley Brook Chicopee, MA MA 00720 March 4, 1981
CULLINAN ENGINEERING CO., INC. CIVIL ENGINEERS AUBURN-BOSTON, MASSACHUSETTS		

FIGURE 1



LOCATION MAP

CHICOPEE RESERVOIR DAM
CHICOPEE , MASS.
1" = 2083'

CULLINAN ENGINEERING CO., INC. SPRINGFIELD NORTH, MASS QUADRANGLE 1979

NATIONAL DAM INSPECTION
PROGRAM

PHASE I INSPECTION REPORT

CHICOPEE RESERVOIR

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 Engineer has been assigned the responsibility of supervising the inspection of dams within the New England Region. Cullinan Engineering Co., Inc., has been retained by the New England Division to inspect and report on selected dams in the State of Massachusetts. Contract No. DACW 33-81-C-0025, dated December 19, 1980, has been assigned by the Corps of Engineers for this work.

(b) Purpose:

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

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

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

1.2 DESCRIPTION OF PROJECT

(a) Location. The dam is located on Cooley Brook, a tributary of the Chicopee River, in the City of Chicopee, Hampden County, Massachusetts (see Location Map). The coordinates of this location are latitude 42 degrees 10.2 minutes north and longitude 72 degrees 33.3 minutes west.

(b) Description of Dam and Appurtenances. Chicopee Reservoir Dam is an earthfill dam 545-feet long and 47-feet high. The top of the earth embankment is 15-feet wide and covered with grass, and is at El 174.0.

A concrete core wall forms the crest of the dam (see Photo No. 2), projecting approximately 2.7 feet above the top of the earth embankment to El 176.7 and the exposed portion of the wall has been rebuilt in recent years. The core wall extends down to the base of the dam, and below the base of the concrete core wall is a steel sheet piling wall extending to depths of 30 feet below the original ground surface. Both the sheet pile wall and the concrete core wall terminate in the abutment at the end of the dam and, therefore, it appears that there is a possibility of seepage around each abutment because the core wall was not extended into the abutment to a point where the high water line intersects natural impervious soil (see Sheet 3 of construction plans in Appendix B). The top portion of the upstream slope is 3:1 to El 156.0 and is concrete paving on an indicated base of 6 inches of gravel and 6 inches of sand. In recent years the dam has been modified with an asphaltic concrete overlay approximately 18 feet wide over the concrete slope paving (see Photo No. 2). Construction plans indicate that at the base of the concrete paving is the upper 24-inch intake pipe at El 156 and a 5-foot wide selected gravel or broken stone berm. The lower portion of the upstream slope is indicated to be 3.5:1 with selected gravel or broken stone fill, approximately to El 130.0. The downstream slope is 2.5:1 and grass covered (see Photo No. 13) with an 8-inch channel pipe drain and berm traversing the slope at El 152.0. At two points along this pipe run the plans indicate that an 8-inch pipe intersects the drain and conveys flow to discharge points at the toe of slope (see construction plans Appendix B). There is a gravel drain with a 6-inch pipe, located approximately 18 inches downstream of the corewall, following the slope of the original ground which collects seepage through the embankment dam. This drain is connected to a 6-inch drain with open joints at right angles to the centerline of the dam which discharges at the downstream toe of the dam (see construction plans Appendix B).

The concrete spillway is an ogee crest weir located at the right abutment of the dam with a concrete nosewall in the center of the spillway. There are concrete training walls, right and left, both upstream and downstream of the spillway (see Photos No's. 4 and 6). The concrete chute is 44-feet wide at the inlet end and tapers to 22-feet wide at the outlet with three cascades over its length (see Photos No's. 5, 7 and 8). The walls of the chute vary in height from 4 feet (through most of length) to a maximum of 13 feet (at spillway). The crest of the ogee weir is at El 168.0 and has pipe slots for flashboard stanchions, located at 2 feet on centers over the entire length of the crest. No flashboards were on the crest or at the site of the dam. A concrete paved slope serves as the spillway approach with a concrete training wall extending 53 feet from the spillway along the northerly shoreline.

Situated approximately in the center of the dam on the upstream face of the embankment is a brick gatehouse outlet structure (see Appendix B and Photo No. 2). The brick gatehouse is built on top of a concrete foundation which forms a 10-foot by 10-foot intake well for the single 24-inch cast-iron outlet (former water supply main). Construction plans show the 24-inch outlet and 30-inch blowoff line to be screened and manually gated. Invert elevations of the inlet pipes at the gatehouse as shown on Sheet 10 of the construction plans (see Appendix B) are as follows: upper 24-inch inlet El 153.0; lower 24-inch inlet El 132.0. A total of three pipe collars are constructed around the 24-inch outlet and 30-inch blowoff pipes from the gatehouse, as shown in Appendix B. Manually operated sluice gates control the 24-inch inlet lines while the 30-inch blowoff is continuous through the gatehouse structure and is regulated by a handwheel operated gate valve. All control valve operating mechanisms are contained in the gatehouse (see Photo No. 3). Immediately downstream from the dam, the blowoff line discharges into a small reservoir (see Photo No. 10). The 24-inch former water supply main feeds into another gatehouse at the dam located approximately 900-feet downstream and then discharges into the existing downstream reservoir.

A creosoted railroad tie crib wall is located on the right reservoir rim approximately 500-feet upstream of the right abutment (see Photo No. 1). This wall is approximately 100-feet long, 10-feet high and 6 to 8 foot wide at the crest and consists of an open-work timber box backfilled with stone and sand and gravel fill. This wall retains a slope approximately 25-feet higher than the wall and, as a result, the total volume of material it holds back is of the order of 4,000 cubic yards. The wall appears to be in fair condition and shows no evidence of distress.

Also along the right rim of the reservoir, southwest of the crib wall, there is a hand-placed rubble wall.

(c) Size Classification. According to the Corps of Engineers' Recommended Guidelines for Safety Inspection of Dams, a dam is classified as "Intermediate" in size if the height is between 40 feet and 100 feet, or the dam impounds between 1,000 Acre-Feet and 50,000 Acre-Feet. The dam has a maximum height of 47 feet and a maximum storage capacity of 695 Acre-Feet. Therefore, the dam is classified as "Intermediate" in size based on height.

(d) Hazard Classification. The results of the dam failure analysis indicate that the failure outflow would travel south through an impoundment (an old reservoir) just below the dam and continue south following the stream bed for Cooley Brook. When the flow reaches the Massachusetts Turnpike, it will be controlled by the culverts under the highway with the peak failure outflow overtopping 240 feet of the roadway with a maximum depth of about 3 feet at the low point of the highway. South of the highway overland sheet flow is anticipated with potential damage ti as many as eight homes and a filtration plant before the flow would reach the Chicopee River. It is anticipated that both the filtration plant and the houses will be inundated by about 2 feet of water following failure (as compated to no flooding prior to failure - see Appendix D). Thus, with an appreciable economic loss and a potential loss of more than a few lives, the dam is classified in the "High" hazard category.

(e) Ownership. The dam and reservoir are owned by the Commonwealth of Massachusetts and are part of the Chicopee Memorial State Park. The owner is represented by Mr. Robert Authier of the Division of Environmental Management, 570 Burnett Road, Chicopee Falls, Massachusetts 01020 (Phone 413-594-9416).

(f) Operator. The facility is operated by the owner, represented by Mr. Robert Authier, as stated above.

(g) Purpose of the Dam. Chicopee Reservoir was formerly used to provide water storage and to regulate its release as part of the supply system for the City of Chicopee, Massachusetts. Presently, the reservoir is used for recreational purposes as part of Chicopee Memorial State Park, with a public beach on the southerly shoreline of the reservoir.

(h) Design and Construction History. Plans for the construction of the dam were developed in 1926 for the City of Chicopee by Morris Knowles, Inc. Engineers. It is assumed that the dam was constructed immediately thereafter. The plans indicate an earth embankment dam with a reinforced concrete core wall and steel sheet piling extending approximately 30-feet below the bottom of the core wall. The ogee crest spillway is located at the right abutment. The plans denote a reservoir capacity of 145 M.G.

(i) Normal Operating Procedure. Maintenance personnel for the State Park facility are at the site on a daily basis. During the summer months when the impoundment is in use for recreational purposes, all valves in the gatehouse are normally closed and all discharge is over the spillway. During the winter months, the impoundment is normally drawn down to a reported elevation of 156.

1.3 Pertinent Data. The 1926 construction plans drawn by Morris Knowles, Inc. Engineers show a spillway crest elevation of 168.0. It is noted on the plans that "Elevations are in Feet from Mean Low Water Charlestown Navy Yard Datum". All other elevations reported herein are based upon a spillway elevation of 168.0.

(a) Drainage Area. The drainage area tributary to the dam is 3.93 square miles. The Reservoir is surrounded by moderately rolling hills with elevations ranging from a low of 167 at the reservoir to a high of 400 at the easterly end of the watershed. There is some moderate residential development and a golf course in the southeasterly section of the watershed. The northwesterly portion of the watershed is comprised of a large section of Westover Air Force Base, including most of the major runway. Chicopee Reservoir accounts for approximately 1.2 percent of the total drainage area. There are no significant upstream ponds, however, marshlands account for 3.4 percent of the total watershed.

(b) Discharge at Dam Site. A notable flood occurred at the location in August 1955. No information was available for an estimated discharge. Normal discharge is over a concrete ogee crest spillway. The weir has a length of 44-feet and a crest elevation of 168.0. Though pipe slots for flashboard stanchions are located over the entire length of the crest, there are no flashboards on the crest or at the dam site. Flow over the weir discharges down a concrete chute with three (3) cascades over the length of the spillway. The concrete chute tapers in width from 44-feet at the top to 22-feet at the outlet. The spillway discharges into a small reservoir at the toe of the downstream slope. In addition to the spillway, there is a 30-inch blowoff line located between the center of the dam and the spillway. This 30-inch pipe is controlled by a gate valve in the gatehouse and discharges at the downstream toe of the slope.

The following is a list of pertinent values relative to discharge:

1.	Outlet Works (Conduit) Size:	(a) 30 inch (b) 24 inch (c) 24 inch
	Invert Elevation:	(a) 129.7 (b) 132.0 (c) 153.0
	Discharge Capacity:	Assumed outlet control with pool at dam crest. (a) 115 cfs (b) 98 cfs (c) 72 cfs
2.	Maximum Known Flood at Dam Site:	Unknown
3.	Ungated Spillway Capacity at Top of Dam: Elevation:	3950 cfs 176.7
4.	Ungated Spillway Capacity at Test Flood Elevation: Elevation:	2975 cfs 175.2
5.	Gated Spillway Capacity at Normal Pool Elevation: Elevation:	N/A
6.	Gated Spillway Capacity at Test Flood Elevation: Elevation:	N/A
7.	Total Spillway Capacity at Test Flood Elevation: Elevation:	2975 cfs 175.2
8.	Total Project Discharge at Top of Dam: Elevation:	3950 cfs 176.7
9.	Total Project Discharge at Test Flood Elevation: Elevation:	6450 cfs 177.4

(c) Elevation - Feet Above Mean Low Water Charlestown Navy Yard Datum

1. Streambed at Toe of Dam: 130.0
2. Bottom of Cutoff: 99.0
3. Maximum Tailwater: Unknown
4. Normal Pool: 168.4
5. Full Flood Control Pool: N/A
6. Spillway Crest: 168.0
7. Design Surcharge - Original Design: 173.0
8. Top of Dam: 176.7
9. Test Flood Surcharge: 175.2

(d) Reservoir - Length in Feet

1. Normal Pool: 4000 feet
2. Flood Control Pool: N/A
3. Spillway Crest Pool: 4000 feet
4. Top of Dam: 6500 feet
5. Test Flood Pool: 5000 feet

(e) Storage - Acre-Feet

1. Normal Pool: 322 acre-feet
2. Flood Control Pool: N/A
3. Spillway Crest Pool: 310 acre-feet
4. Top of Dam: 695 acre-feet
5. Test Flood Pool: 685 acre-feet

(f) Reservoir Surface - Acres

1. Normal Pool: 31 acres
2. Flood-Control Pool: N/A
3. Spillway Crest: 29 acres
4. Test Flood Pool: 50 acres
5. Top of Dam: 54 acres

(g) Dam

1.	Type:	Earthfill
2.	Length:	545 feet (including spillway)
3.	Height:	47 feet
4.	Top Width:	15 feet
5.	Side Slopes:	3.5 horizontal to 1 vertical upstream 2.5 horizontal to 1 vertical downstream
6.	Zoning:	See Plan in Appendix B
7.	Impervious Core:	Concrete core wall
8.	Cutoff:	Steel sheet piles
9.	Grout Curtain:	N/A
10.	Other:	N/A

(h) Diversion and Regulating Tunnel — N/A

(i) Spillway

1.	Type:	Chute spillway with ogee crest weir
2.	Length of Weir:	44 feet
3.	Crest Elevation: with Flashboards: without Flashboards:	N/A 168.0
4.	Cates:	N/A
5.	Upstream Channel:	Reservoir
6.	Downstream Channel:	Concrete chute 44 feet wide at weir tapering to 22 feet at spillway outlet
7.	General:	Three cascades over length

(j) Regulating Outlets

1. Invert:	(a) 129.7 (b) 132.0 (c) 153.0
2. Size:	(a) 30 inch (b) 24 inch (c) 24 inch
3. Description:	(a) 30 inch cast-iron pipe acting as blowoff and reservoir drain (b)(c) 24 inch cast-iron pipes acting as inlets for the outlet well serving the 24 inch outlet (former water supply line)
4. Control Mechanism:	(a) 30 inch gate valve (operating mechanism in gatehouse on dam). (b)(c) 24 inch sluice gates with controls (lifting mechanism in gatehouse on dam)
5. Other:	N/A

SECTION 2
ENGINEERING DATA

2.1 DESIGN RECORDS

The design records consisted of a full set of prints of the original construction drawings, dated July 29, 1926, by Morris Knowles, Inc., Engineers. The plans show details for construction of the dam, core wall, spillway, gatehouse and appurtenances. Original design computations and construction specifications are not available. Also, records for the installation of the bituminous concrete apron, construction of the foot bridge over the spillway, and addition of the concrete cap to the core wall were not located.

2.2 CONSTRUCTION RECORDS

The construction records available are the 1926 plans referred to in Section 2.1. There are no as-built drawings for the dam, spillway or outlet structures.

2.3 OPERATING RECORDS

No formal operating records are kept, however, the dam and appurtenances are checked daily by a member of the park maintenance crew.

2.4 EVALUATION

(a) Availability. Documents described above are available for review at Chicopee Memorial State Park, 570 Burnett Road, Chicopee Falls, Massachusetts 01020.

(b) Adequacy. The available data, in combination with the visual inspection and hydraulic and hydrologic calculations, is adequate for the purpose of Phase I Investigation.

(c) Validity. Comparison of the available data with the field survey conducted during the Phase I Inspection indicates that the information is valid.

SECTION 3
VISUAL OBSERVATIONS

3.1 FINDINGS

(a) General. The Chicopee Reservoir Dam is in FAIR condition at the present time. This classification is primarily based on the observation of a high degree of erosion of the principal spillway chute structure and outlet training walls; erosion gullies on the right abutment above the dam crest; brush growth on the downstream slope of the embankment; and seepage issuing from the left abutment downstream of the dam.

(b) Dam. Generally, the earth embankment is in good condition. Alignment of the upstream slope is good. There is a recently placed asphaltic concrete overlay, on the concrete upstream slope paving, approximately 2 inches thick and 18 feet wide from the parapet wall forming the crest of the dam (see Photo No. 2). The concrete parapet wall is in excellent condition. The upstream slope was inspected and generally found to be in good condition with some minor grass growth in a horizontal seam located within the asphalt overlay and minor cracking on the upstream face of the concrete slope paving.

Two erosion gullies were found on the gravel hill forming the right abutment (see Photo Location Plan). One erosion gully is in the direction of the dam alignment and runs towards the spillway while the second erosion gully runs downstream. The gullies are some 25 to 30 feet in height above the crest of the dam so they pose no immediate danger to the dam, however, they could be a source of continuing erosion and, therefore, should be checked periodically whenever inspections are made. One of the erosion gullies has been backfilled with brush and debris (see Photo No. 14).

Considerable clear and clean seepage is emanating from the left abutment, particularly along the toe of the main embankment slope where it intersects the fill forming the left abutment for a distance of approximately 25 feet downstream of the toe of the embankment, and approximately 100 feet to the right of the intersection of the toe of the dam with the left abutment. A board with a V-notch weir cut in it has been placed downstream of the toe of the dam (see Photo No. 10). Approximately 100 GPM of seepage was observed flowing over this weir. About 30 feet upstream of the weir there is a shallow ponded area where a small seepage boil is coming up through the pond from the underlying ground (see Photo No. 11). This ponded area is approximately 100 foot long and 30 foot wide and contains about 6-inches of water.

Also, seepage estimated to be 100 GPM is emanating from the left abutment from the natural slope at the toe of the dam. This seepage is clear and clean and is additive to the seepage noted at the weir so that the total seepage is estimated to be of the order of 200 GPM. Clearly, this seepage should be monitored, and warrants the engagement of a registered professional engineer to make a study of the seepage, particularly at the left abutment.

On the downstream slope of the embankment, there is considerable brush growth, particularly near the left abutment (see Photo No. 13). Also, approximately 25 feet downstream of the toe of the embankment where it intersects the left abutment there is a 24-inch concrete pipe having a length of approximately 15 feet where it intersects a headwall. This concrete pipe is issuing about 5 gallons a minute of seepage which is believed to be seepage going around the left abutment No evidence of a toe drain that reportedly exists (see plans in Appendix B) was found. The existence of this drain should be determined and an analysis of its condition and adequacy should be performed by a qualified registered professional engineer.

(c) Appurtenant Structures

1. Left Core Wall

The original concrete portion of this structure has been capped with new concrete (see Photo No. 2). Its present width is 3 feet. The concrete is in good condition without any evidence of spalls, cracks, or efflorescence. A single rail steel fence consisting of 4 x 4 inch posts and 6 inch channel rail 25 inches high is in good condition. The concrete apron upstream of this core wall is in good condition with the exception of surface erosion. Bituminuous concrete paving 2"± thick has been placed over the entire length of the apron and extends towards the impoundment pool approximately 18-feet (see Photo No. 2). A continuous open joint in the overlay is overgrown with grass and is located at the spillway crest elevation.

2. Principal Spillway

This ogee structure is divided by an intermediate pier 12 inches thick and 8.5 feet long normal to the spillway axis and has its origin at the spillway crest (see Photos No's. 4 and 6). Pipe slcts for flashboard stanchions are located 2-feet on center over the entire length of the crest. No flashboards were on the crest or at the site of the dam.

The downstream surface of the spillway exhibits surface erosion which has exposed the concrete aggregate. The base of the upstream nosing of the intermediate pier has been subjected to spalling approximately 10 inches high and 3 inches deep. The downstream base of this pier has been subjected to erosion for a height of 12 inches and up to 4 inches deep on three sides (see Photo No. 6). Horizontal and vertical reinforcing steel is exposed and rusted and some vertical rods have completely detiorated. A formed keyway was cast in the downstream vertical face of the pier.

A level concrete apron is located upstream of the spillway crest and approximately 12 inches below the crest elevation (see Photo No. 4). Extending upstream into the impoundment pool for a distance of approximately 20 feet, the apron then slopes downward at the rate of of 3 to 1 into the reservoir. The top surface of this apron has been subjected to a considerable amount of surface spalling up to 2 inches deep. The interface of the left spillway training wall and the concrete apron adjacent to the crest has a formed void 5 inches high, 2 feet long and up to 14 inches in depth.

3. Right Training Wall

This structure is approximately 50 feet long and 2 feet in top width (see Photos No's. 4 and 6). A rectangular shaped buttress, which acts as a flow deflector and energy dissipator is located at midlength of the wall. The front face of the portion of this wall upstream of the flow deflector is in good condition with the exception of minor efflorescence observed at a series of fine horizontal cracks. Erosion has occurred at the base of the flow deflector for a height of 14 inches up to 3 inches deep. The downstream portion of this wall has been subjected to horizontal and vertical cracking and associated efflorescence. A vertical construction joint which is located on this wall approximately 3-1/2 feet upstream of the spillway crest has opened at the base by approximately 1/2 inch and has been subjected to surface spalling up to 2-1/2 feet in height. A vertical crack, which is located 18 inches upstream of the spillway crest has displaced outward by 1/2 inch for the full height of the wall with minor spalling along the crack.

Inspection of the upstream end of this wall has revealed that the wall was faced and backed up with concrete to form the thickness of the existing wall. The remains of a cemented stone masonry wall is in evidence at the splayed end of this concrete wall. Remains of a dry stone masonry wall extend upstream.

A steel pipe rail fence in good condition approximately 20 feet long is located at the upstream end of this wall. Sockets in the top of the wall revealed that this fence originally was in place up to the spillway crest.

4. Chute

This flume type structure is divided into three sections by means of vertical drops (5 feet high) (see Photos No's. 5, 7, and 8). The lowest section of the chute has a vertical drop of 6 feet into a channel bed reinforced with energy dissipating blocks. Three trapezoidal shaped energy dissipators are located on both sidewalls at all three sections of the chute. The sidewalls have been faced and backed up with concrete to form 2 foot thick walls. A two span steel framed footbridge with a concrete deck and metal railing spans over the upstream chute section (see Photo No. 5). The depth of flow in the chute at the bridge section was calculated to be approximately 2.4 feet (see calculations in Appendix D). Since the height of the chute walls is 4.0 feet, the bridge will not impede flow in the chute with water surface in the reservoir at the top of the dam.

The left end wall and right abutment are incorporated in the upstream chute section. Starting immediately downstream of the spillway the right abutment has eroded over its entire length at the interface with the concrete apron up to the first cascade. This erosion is up to 6 inches in height and up to 6 inches in depth and is more pronounced at the energy dissipators where it is up to 12-inches high and 6-inches deep. The downstream wall continuation in this reach is in fair condition; minor surface cracking and associated efflorescence being observed. Inspection of the back side of the upstream half of this wall revealed soil erosion exposing the back of this wall. The wall was backed up with minimal effort and in some instances soil has eroded up to 18 inches below the backing exposing vertical reinforcing steel. Erosion has also occurred at the interface with the concrete apron in a similar fashion as the right wall. A portion of the left wall has been treated with a surface coat of mortar which has been subjected to considerable cracking which exhibits minor efflorescence. A two rail steel pipe fence is located on the sloping section of this wall adjacent to the flight of concrete stairs. Both the fence and stairs are in good condition. The concrete apron in this reach of the chute downstream of the footbridge has eroded up to 3 inches deep whereas the section of the apron upstream of the footbridge to the spillway is in good condition. Erosion has occurred at the interface of the first reach of the chute and the first vertical cascade face over a distance of approximately 6 feet and for a depth of 18 inches.

With the exception of minor surface cracking at the top of the intermediate concrete pier, the footbridge is in good condition with no evidence of spalls or erosion. The steel framing of the footbridge which consists of two 14 inch by 7 inch stringers is in good condition and well maintained. However, the underside of the concrete slab has been subjected to a high degree of hairline cracking, efflorescence, exudation and stalactites.

The left wall of the second or middle chute section has been subjected to surface spalling over its entire length. Erosion of the concrete energy dissipators has occurred for a depth of up to 4 inches and a height of 2 feet. Additional erosion has occurred of the entire length of the interface with the right wall in this reach is in good condition with the exception of minor surface cracking and associated efflorescence. With the exception of its downstream lip, the concrete apron is in good condition. Spalling and erosion have occurred over 90% of the length of the interface of this apron and the second cascade drop (see Photo No. 7). This spalling is up to 15 inches in height and 8 inches in depth. A 6 x 6 inch drain opening is located on the face of the cascade wall. Investigations revealed that a 6 inch subdrain outlets through this opening. There was no discharge at the time of inspection (see Photo No. 7).

The left wall at the third and lowest chute section is in fair condition with the exception of minor surface erosion at the energy dissipators. However, the right wall exhibits a considerable amount of horizontal hairline cracks with associated efflorescence and considerable surface spalling. The interface of the apron and the lowest cascade drop has spalled and eroded over 50% of its length (see Photo No. 8) and a triangular spall 4 feet long and 4 feet high and up to 6 inches in depth is located adjacent to the right wall. Other spalled areas are up to 12 inches high and 6 inches deep. A 7 foot square access manhole which houses a cast-iron frame and 34 inch cover is located along the centerline of the concrete apron as an extension to the cascade drop. The access manhole functions as a cleanout for an 8 inch subdrain. At the downstream face of the concrete manhole, the cast-iron subdrain extension into the outlet channel has ruptured. Spalling has occurred at the downstream corner of this structure for a height of approximately 2 feet and up to 8 inches in depth.

5. <u>Outlet Training Walls</u>

The downstream outlet training walls are 12 inches thick with no back batter and approximately 25 feet long with 4 foot 90° returns. The downstream end of the left end wall and return have been subjected to a high degree of erosion from its base at the channel bed for a vertical distance of 3 feet, up to 8 inches deep, and exposing reinforcing steel. Spalling has occurred at the end of the 90° return of the right wall up to 12 inches high and 4 inches deep. Clear and clean seepage is emanating through a crack in the left wall at the approximate rate of 1/10 gallon per minute.

Twelve reinforced concrete energy dissipating blocks are located immediately downstream of the lowest cascade wall and between the outlet training walls. These blocks are approximately 12 inches square; the tops being approximately 4 feet above the channel bed. All of the blocks are severely deteriorated with all reinforcing steel exposed and in four instances they are completely void of concrete (see Photo No. 9).

6. <u>Gatehouse</u>

This is a brick bearing wall structure housing three bench stands and is supported by a concrete foundation (see Photos No's. 2 and 3). Two bench stands are adjacent to the upstream wall. The left bench stand has a 24 inch diameter wheel operator and the right stand has a hand crank. Both have rising stems and operate sluice gates which outlet into a 24 inch conduit which was formerly part of the Chicopee water supply. The left gate was open by approximately 4 inches at the time of inspection. All of the gates are well maintained.

The brick masonry walls of the gatehouse are in good condition, however, the wood framed roof has been subjected to leakage.

(d) <u>Reservoir Area</u>. The only development along the shore of Chicopee Reservoir is a reacreational beach, bathhouse, and maintenance facility for Chicopee Memorial State Park. The remainder of the immediate area is undeveloped, wooded and hilly. There is little potential that further development will occur in the reservoir area as the surrounding land is part of Chicopee Memorial State Park.

On the right rim of the reservoir is a hand-placed rubble wall which appears to be stable with no evidence of movement of the slope or instability (see Photo No. 1).

Located on the right reservoir rim approximately 500 feet upstream of the right abutment is a creosoted railroad tie crib wall (see Photo No. 1). This wall is approximately 100 feet long and 10 feet high, and 6 to 8 feet wide at the crest. It is an open-work timber box backfilled with stone, sand, and gravel fill which retains a slope approximately 25 feet higher than the wall and, as a result, the total volume of material it holds back is of the order of 4,000 cubic yards. The wall appears to be in good condition and shows no evidence of distress. Should this slope fail, it is not likely that a major wave would develop in the reservoir which would go over the dam. The reservoir rim appears to be stable with no evidence of movement or potential of major slides into the reservoir.

(e) Downstream Channel. The concrete spillway, 24-inch outlet and 30-inch blowoff line discharge into a small reservoir at the toe of the downstream slope. The outflow then follows the natural streambed of Cooley Brook in a southwesterly direction, passing under the Massachusetts Turnpike and Fuller Road before discharging into the Connecticut River, approximately 4600 feet downstream of the dam.

3.2 EVALUATION

In general the dam and its appurtenant structures are in fair condition. The problem areas noted during the visual inspection are listed as follows:

(a) High degree of erosion of the principal spillway chute structure and outlet training walls which compromises the integrity of the structure and could lead to further deterioration.

(b) Minor grass growth at the seam between the asphaltic overlay and the concrete apron on the upstream slope.

(c) Erosion gullies on the right abutment above the dam crest. If the gullies are allowed to increase in size and depth, a potential for failure of the right abutment would exist through diversion and focusing of runoff into the area.

(d) Brush growth on the downstream slope of the embankment is considered deleterious due to the destructive action of roots upon a structural earthfill.

(e) Seepage issuing from the left abutment downstream of the dam. A study of the origin of the seepage and the possible transportation of fines should be made as this seepage could lead to piping of soils and result of internal erosion of the abutment.

(f) The existence and condition of the toe drain should be investigated.

SECTION 4
OPERATIONAL AND MAINTENANCE PROCEDURES

4.1 OPERATIONAL PROCEDURES

(a) General. There is no formally established routine for the operation of the dam. On an informal basis, the reservoir is lowered in the early spring and the shoreline of the recreational area is cleaned. The reservoir is not filled again until after the threat of a spring flood has passed.

(b) Warning System. There is no established warning system or emergency preparedness plan in effect for this structure.

4.2 MAINTENANCE PROCEDURES

(a) General. Maintenance of the dam is performed on an informal basis rather than on a formally established routine or procedure. The dam is generally maintained in fair condition.

(b) Operating Facilities. There are no operating facilities for the spillway. The outlets are operable and are opened on a yearly basis. It is reported that the 30-inch outlet gate valve can not be opened until the water surface in the reservoir is down to about El 155± (approximately 13-feet below the spillway crest).

4.3 EVALUATION

Currently, there is no operational procedure in effect for Chicopee Reservoir Dam and maintenance is performed on an informal basis only. Maintenance programs, operational procedures, warning system and emergency preparedness plans should be established. It is recommended that a program be established by the Owner to monitor the seepage on a weekly basis until the recommendations of an engineering study have been implemented. In addition, the Owner should implement a program of annual technical inspections by a qualified registered engineer. These programs should be implemented as recommended in Section 7.3.

SECTION 5
EVALUATION OF HYDRAULIC/HYDROLOGIC FEATURES

5.1 General. Cooley Dam at Chicopee Reservoir is a 47-foot high earthfill dam built in about 1926. It has a concrete core wall with steel sheet piling indicated below the core wall. The spillway is a 44-foot long, ungated, ogee crest weir. The dam was built to provide storage for a water supply to the City of Chicopee. It is located on Cooley Brook. The reservoir presently functions as a recreational facility and is no longer used as a municipal water supply.

5.2 Design Data. Hydraulic and hydrologic computations are not available for the design of the spillway.

5.3 Experience Data. There are no formal records kept pertaining to the performance of the dam. Water level and functional aspects of the dam and appurtenances are monitored through regular inspections made by employees of Chicopee Memorial State Park.

5.4 Test Flood Analysis. Based on the Corps of Engineers Guidelines, the recommended test flood for the size (intermediate) and hazard (high) is the probable maximum flood (PMF). Consequently, the PMF was adopted as the test flood inflow. The watershed has mostly rolling terrain with a gentle slope (about 0.8%), no significant amount of ponded water upstream, and a marshland that accounts for about 3.4% of the total drainage area. For Chicopee Reservoir watershed, the PMF rate was calculated to be 1,925 cfs per square mile of drainage area. Applying the PMF to the 3.93 square miles of drainage area results in a calculated peak flood flow of 7,565 cfs as the inflow test flood. By adjusting the inflow test flood for surcharge storage, the maximum discharge rate was established as 6,450 cfs, with a water surface at El 177.8. Thus, with the nominal top of dam at El 176.7, the dam would be overtopped by 1.1 feet.

Calculations show that with the water surface at the top of dam, the spillway without flashboards can discharge 3,950 cfs, or 60% of the routed test flood outflow.

5.5 Dam Failure Analysis. Based on the Corps of Engineers Guidelines for estimating dam failure hydrographs, and assuming a breach width of 136 feet which represents 40% of the mid-height length of 340 feet at a water surface elevation of 168.0 (spillway elevation) the dam failure outflow would be 53,500 cfs. As a result of the dam failure, some overhead electric lines and the Massachusetts Turnpike would receive severe damage. When the flow reaches the turnpike, it will be controlled by the culverts under the highway with the peak failure outflow overtopping the roadway for a length of 240 feet with a maximum depth of approximately 3 feet at the low point of the highway. South of the highway, the anticipated overland sheet flow would cause damage to as many as eight homes and a filtration plant before reaching the Chicopee River. This compares to no overtopping of the roadway and no encroachment upon the homes prior to failure (see Appendix D). Consequently, with an appreciable economic loss and a potential loss of more than a few lives, the overall potential hazard from a dam failure of Cooley Dam at Chicopee Reservoir would be "high".

SECTION 6
STRUCTURAL STABILITY

6.1 VISUAL OBSERVATIONS

The field inspection of the dam and spillway indicated that these structures are in fair condition. Considerable spalling and erosion was observed in the outlet chute, outlet training walls and energy dissipating devices. No structural deficiencies were noted which would warrant further investigations. There are several items of a maintenance nature which will require remedial attention as outlined in Section 7. Further in depth engineering studies will be required by a registered professional engineer to investigate seepage issuing from the left abutment area. A program of yearly technical inspections by a qualified registered engineer should be implemented to monitor any changes in the conditions of the dam and spillway structure.

6.2 DESIGN AND CONSTRUCTION DATA

Definitive plans of the dam and spillway were reviewed. The drawings consist of 11 sheets developed by Morris Knowles, Inc. Engineers, Pittsburgh, Pennsylvania, dated July 29, 1926. The plans appear to be consistent with the superficial features observed during the field inspection. There is evidence of some recent superficial changes to the dam including an asphaltic concrete overlay on the upstream slope, repair of the concrete parapet wall on the upstream slope, and construction of a crib wall to retain a reservoir slope approximately 500-feet upstream of the right abutment. Laboratory test data of the soils forming the embankments was not available. Calculations pertaining to the stability of the dam and spillway structure are also not available.

6.3 POST-CONSTRUCTION CHANGES

Post-construction changes undertaken within the last five years include the following:

(1) Construction of an asphaltic concrete overlay approximately 18 feet wide and 2 inches thick over the upstream concrete slope paving from the parapet wall outward for a distance of 18 feet.

(2) Repair by new concrete overlay of the upstream parapet wall.

(3) Installation of a pipe guardrail on the parapet wall.

(4) Construction of an open crib wall of creosoted railroad ties to retain a slope approximately 100 feet upstream of the right abutment.

6.4 SEISMIC STABILITY

The dam is located in Seismic Zone No. 2, and in accordance with recommended Phase I Guidelines, does not warrant seismic analysis.

SECTION 7
ASSESSMENT, RECOMMENDATIONS, AND REMEDIAL MEASURES

7.1 DAM ASSESSMENT

(a) Condition. The Chicopee Reservoir Dam is in FAIR condition at the present time. The erosion of the principal spillway consisting of the outlet chute, outlet training walls and energy dissipating devices should be repaired. Other items of concern include the presence of brush growth on the downstream slope of embankment, erosion gullies on the right abutment above the crest of the dam, and grass growth in a seam located within the concrete asphaltic overlay on the upstream slope. In addition to the above maintenance type items, there is seepage issuing from the left abutment downstream of the dam and will require further in depth engineering studies as outlined below.

(b) Adequacy of Information. The original design drawings are available for the embankment and spillway. Consequently, the adequacy of engineering data is considered good. No formal operating records are kept for the dam. Thus, assessment of this dam is based on a knowledge of these design drawings plus the visual inspection conducted on March 4, 1981.

(c) Urgency. The remedial measures enumerated in Section 7.3 below should be implemented by the owner within one year of receipt of this Phase I inspection report, except that a qualified registered engineer should be retained immediately to investigate the seepage emanating from the left abutment.

7.2 RECOMMENDATIONS

It is recommended that the services of a qualified registered professional engineer be retained to:

(1) Perform a detailed hydrologic-hydraulic investigation to assess further the potential of overtopping the dam and the need for and the means to increase project discharge capacity.

(2) Prepare plans for rehabilitation of all spalling and erosion of the principal spillway, including repairs to the buttress on the right training wall upstream of the spillway and the interface of the parapet wall and the ogee spillway.

(3) Undertake an investigation of the seepage emanating from the left abutment.

(4) Remove trees varying in size from 2 to 18 inches in diameter from the downstream slope and within 15 feet of the toe, and backfill with suitable compacted material under the direction of an engineer.

(5) Investigate the reported inoperability of the 30-inch gate valve and prepare plans for restoring it to an operational state.

(6) Investigate the existence and the condition of the toe drain shown on the construction plans (see Appendix B) and prepare plans, if required, to restore it to a functional state.

(7) The owner should implement the recommendations of the above engineering studies.

7.3 REMEDIAL MEASURES

(a) Operation and Maintenance Procedures

(1) Remove minor grass growth and seal the seam located within the concrete asphaltic overlay on the upstream slope with suitable bitumastic material.

(2) Remove the brush and debris from erosion gullies on the right abutment above the dam crest and repair gullies with suitable compacted fill, topsoil, and grass seed.

(3) Monitor seepage on a weekly basis with particular attention paid to the quantity and clarity until the recommendations of the engineering study have been implemented.

(4) Remove brush growth on the downstream slope of the embankment and within 15 feet of the toe.

(5) Implement a program of yearly technical inspections by a qualified registered engineer.

(6) Develop a plan for surveillance of the dam during and immediately after periods of heavy rainfall and for warning of downstream officials in the event of an emergency.

(7) Implement and intensify a program of diligent and periodic inspection.

7.4 ALTERNATIVES

There are no practical alternatives to the above recommendations and remedial measures.

INSPECTION TEAM ORGANIZATION

Date: March 4, 1981

Project: MA 00720
Chicopee Reservoir (Cooley Dam)
Chicopee, Massachusetts

Weather: Clear, cold

INSPECTION TEAM

William S. Parker	Cullinan Engineering Co., Inc. (CEC)	Team Captain
Kenneth W. Hodgson, Jr.	CEC	Hydraulics
Gregory M. Valiton	CEC	Hydraulics
William S. Zoino	Goldberg, Zoino & Associates (GZ)	Soils
Steve Trettel	GZ	Soils
Andrew Christo	Andrew Christo Engineers, Inc (ACE)	Structures
Paul Razgha	ACE	Structures
Carl Razgha	ACE	Structures

Owner was not represented at inspection

NOTE: Observed water surface elevation in reservoir at time of inspection = El 164.0±

CHICOPEE RESERVOIR DAM
MA 00720

March 4, 1981

CHECKLISTS FOR VISUAL INSPECTION

AREA EVALUATED	BY	CONDITION & REMARKS
UPSTREAM SLOPE		
Vegetation	GZ	Minor grass growth in longitudinal seam in asphalt overlay
Sloughing or Erosion		Minor erosion right abutment 25' above dam crest
Rock Slope Protection - Riprap Failures		None - concrete apron
Animal Burrows		None
CREST		
Vegetation		Trees along crest
Sloughing or Erosion		None
Surface Cracks		None
Movement or Settlement		None
DOWNSTREAM SLOPE		
Vegetation		Brush and occasional small trees
Sloughing or Erosion		None
Surface Cracks		None
Animal Burrows		None
Movement or Cracking Near Toe		None
Unusual Embankment or Downstream Seepage	GZ	100 to 200 GPM at toe of embankment at left abutment clear and clean

CHICOPEE RESERVOIR DAM
MA 00720

March 4, 1981

CHECKLISTS FOR VISUAL INSPECTION

AREA EVALUATED	BY	CONDITION & REPAIRS
Piping or Boils	GZ	One boil observed in area of left abutment toe of embankment
Foundation Drainage Features		None, but a drain collector added on the left abutment slope
Toe Drains		None
<u>GENERAL</u>		
Lateral Movement		None
Vertical Alignment		Good
Horizontal Alignment		Good
Condition at Abutments and at Structures		Good - 2 minor erosion gullies 25' above right abutment
Indications of Movement of Structural Items		None
Trespassing		None
Instrumentation Systems	GZ	None
<u>LEFT CORE WALL</u>		
Condition of Concrete	ACE	Good
Spalling		None noted
Erosion		None noted
Cracking		None noted
Efflorescence		None noted
Rusting or Staining of Concrete		None noted
Visible Reinforcing	ACE	None noted

CHICOPEE RESERVOIR DAM
MA 00720

March 4, 1981

CHECKLISTS FOR VISUAL INSPECTION

AREA EVALUATED	BY	CONDITION & REMARKS
Seepage	ACE	None noted
Condition of Steel Railing		Good - Steel elements painted without any evidence of rust
PRINCIPAL SPILLWAY		
Condition of Concrete		Fair
Spalling		None noted
Erosion		Downstream face eroded exposing course aggregate
Cracking		None noted
Efflorescence		None noted
Rusting and Staining of Concrete		None noted
Visible Peinforcing		None noted
Seepage		None noted
INTERMEDIATE PIER		
Condition of Concrete		Poor
Spalling		See erosion
Erosion		Upstream nosing eroded 10" high x 3" deep. Downstream end eroded 12" high and up to 4" deep on three sides.
Rusting or Staining of Concrete		At interface with eroded concrete
Visible Reinforcing	ACE	Horizontal and vertical reinforcing steel exposed. Steel is rusted and some vertical rods completely deteriorated.

CHICOPEE RESERVOIR DAM
MA 00720

March 4, 1981

CHECKLISTS FOR VISUAL INSPECTION

AREA EVALUATED	BY	CONDITION & REMARKS
UPSTREAM APRON		
Condition of Concrete	ACE	Fair
Spalling		Up to 2" deep. Interface of left abutment adjacent to the crest has a formed void 5" high, 2" long and 14" in depth.
Erosion		See spalling
Cracking		None noted
Efflorescence		None noted
Rusting or Staining of Concrete		None noted
Visible Reinforcing		None noted
Seepage		None noted
RIGHT TRAINING WALL		
Condition of Concrete		Fair
Spalling		On top of wall between spillway crest and flow deflector 2' x 1". Base of construction joint 2.5' high.
Erosion		At base of flow deflector 14" high x 3" deep.
Cracking	ACE	Fine horizontal cracks upstrean of flow deflector. Horizontal and vertical cracks downsteam of flow deflector. Fine parallel hairline cracks at top of wall. Vertical crack opened and wall displaced outward by 1/2".

CHICOPEE RESERVOIR DAM
MA 00720

March 4, 1981

CHECKLISTS FOR VISUAL INSPECTION

AREA EVALUATED	BY	CONDITION & REMARKS
Efflorescence	ACE	Located at horizontal and vertical cracks
Rusting or Staining of Concrete		None noted
Visible Reinforcing		None noted
Seepage		None noted
CHUTE		
UPSTREAM SECTION		
Condition of Concrete		Fair
Spalling		See erosion
Erosion		Right abutment and wall eroded at interface with concrete apron up to 6" high and 6" deep. Energy dissipators eroded 12" high and 6" deep. Left wall eroded at interface with concrete apron similar to right wall. Concrete apron downstream of foot-bridge eroded up to 3" deep. Interface of apron and cascade wall eroded over 6' in length x 18" deep
Cracking		Right wall downstream subjected to minor surface cracking. Considerable surface cracking at mortar facing on left wall.
Efflorescence		Minor at location of cracks.
Rusting or Staining of Concrete		None noted
Visible Reinforcing		At back side of right wall.
Seepage	ACE	None noted

CHICOPEE RESERVOIR DAM
MA 00720

March 4,1981

CHECKLISTS FOR VISUAL INSPECTION

AREA EVALUATED	BY	CONDITION & REMARKS
FOOTBRIDGE		
Condition of Concrete Pier	ACE	Fair. Minor surface crack at top of pier with associated efflorescence
Condition of Structural Steel		Good
Condition of Bridge Railing		Rusting at weldments
Concrete Deck		Fair. High degree of cracking efflorescence, exudation and stalactites on underside.
MIDDLE SECTION		
Condition of Concrete		Poor
Spalling		Left wall and energy dissipators over entire length. The interface of the apron and the cascade drop spalled over 90% of length; 15" high x 8" deep.
Erosion		Energy dissipators on left wall eroded 4" deep x 2' high. Interface of left wall and apron eroded over entire length. Minor surface erosion on apron.
Cracking		Minor surface cracking on right wall
Efflorescence		At surface cracks on right wall
Rusting or Staining of Concrete		None noted
Visible Reinforcing		None noted
Seepage	ACE	None noted

CHICOPEE RESERVOIR DAM
MA 00720

March 4, 1981

CHECKLISTS FOR VISUAL INSPECTION

AREA EVALUATED	BY	CONDITION & REMARKS
LOWER SECTION		
Condition of Concrete	ACE	Fair
Spalling		Considerable surface spalling at downstream end of right wall. Interface of apron and cascade drop spalled 50% of length. 4' x 4' triangular spall 6" deep at right wall; balance up to 12" high x 6" deep. D.S. end of manhole structure spalled 2' high x 8" deep.
Erosion		Minor at energy dissipators.
Cracking		Considerable horizontal hairline cracks on right wall
Efflorescence		At surface cracks on right wall.
Rusting or Staining of Concrete		None noted
Visible Reinforcing		None noted
Seepage		None noted
Concrete Drain Outlet		8" C.I. pipe ruptured.
OUTLET TRAINING WALLS		
Condition of Concrete		Fair
Spalling		End of right 90° return wall spalled 12" high x 4" deep
Erosion		Downstream end of left end wall and return eroded 3' high x 8" deep.
Cracking		None noted
Efflorescence	ACE	None noted

CHICOPEE RESERVOIR DAM
MA 00720

March 4, 1981

CHECKLISTS FOR VISUAL INSPECTION

AREA EVALUATED	BY	CONDITION & REMARKS
Rusting or Staining of Concrete	ACE	None noted
Visible Reinforcing		Downstream end of left wall and return.
Seepage		Seepage behind left wall at the rate of 1/10 gallon per minute.
ENERGY DISSIPATING BLOCKS		
Condition of Concrete		Poor. Severely deteriorated with reinforcing steel exposed. In 4 cases blocks completely void of concrete.
GATEHOUSE		
Condition of Building		Fair
Condition of Brick Masonry		Good
Condition of Roof		Wood framed roof subjected to leaking.
Bench Stands	ACE	Well maintained and in operable condition.

APPENDIX B

ENGINEERING DATA

CITY OF CHICOPEE, MA

BOARD OF WATER CO

PLANS FO

IMPOUNDING DAM A

1926

MORRIS KNOWLES INC.,

PITTSBURGH, PA. CLEV

(1)

COPEE, MASSACHUSETTS.

WATER COMMISSIONERS.

PLANS FOR

DAM AND RESERVOIR.

1926

KNOWLES INC., ENGINEERS.

RGH, PA. CLEVELAND, OHIO.

Approved By
Board Of Water Commissioners

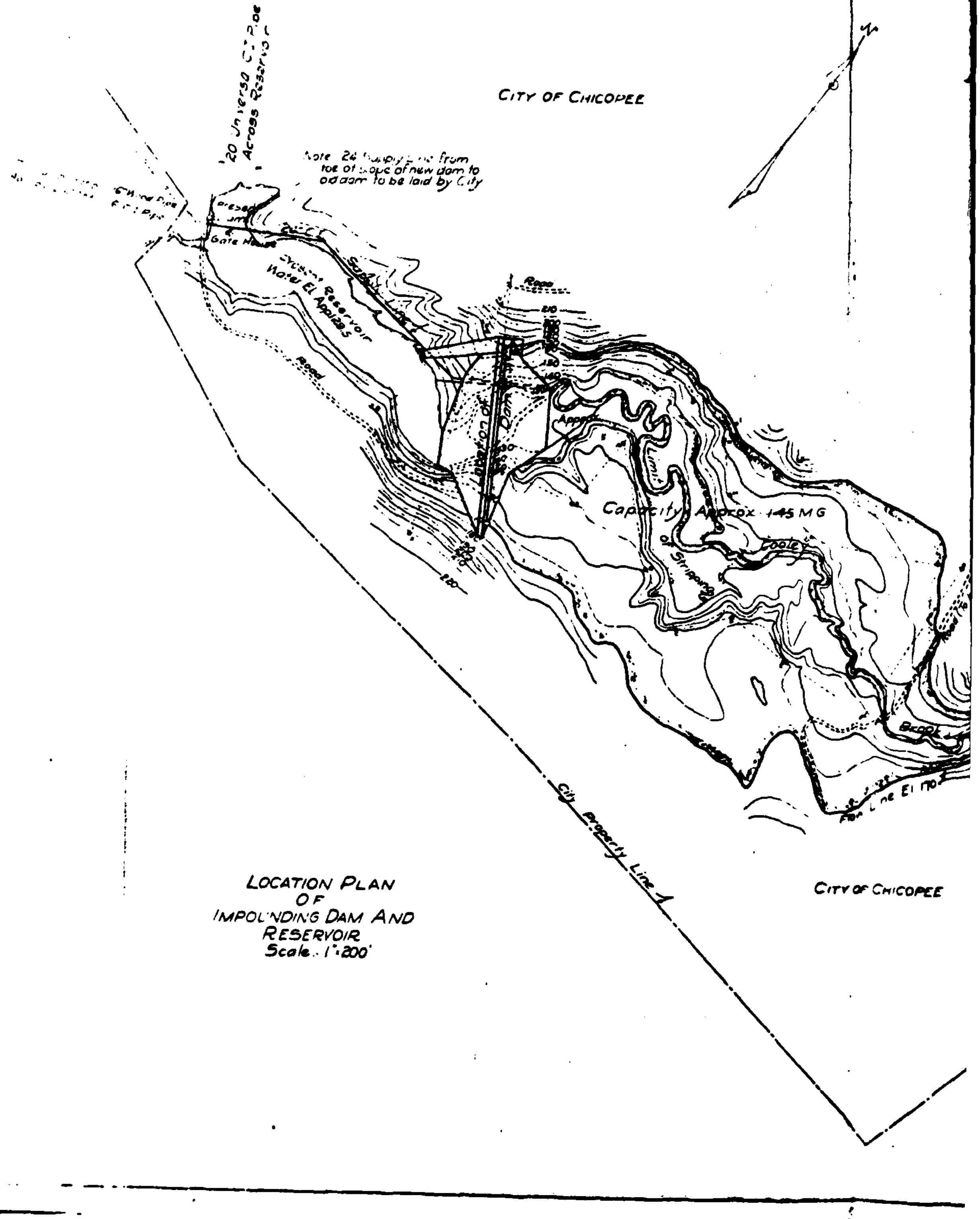
Chairman

Mayor

John F. Sullivan
Superintendent Water Department

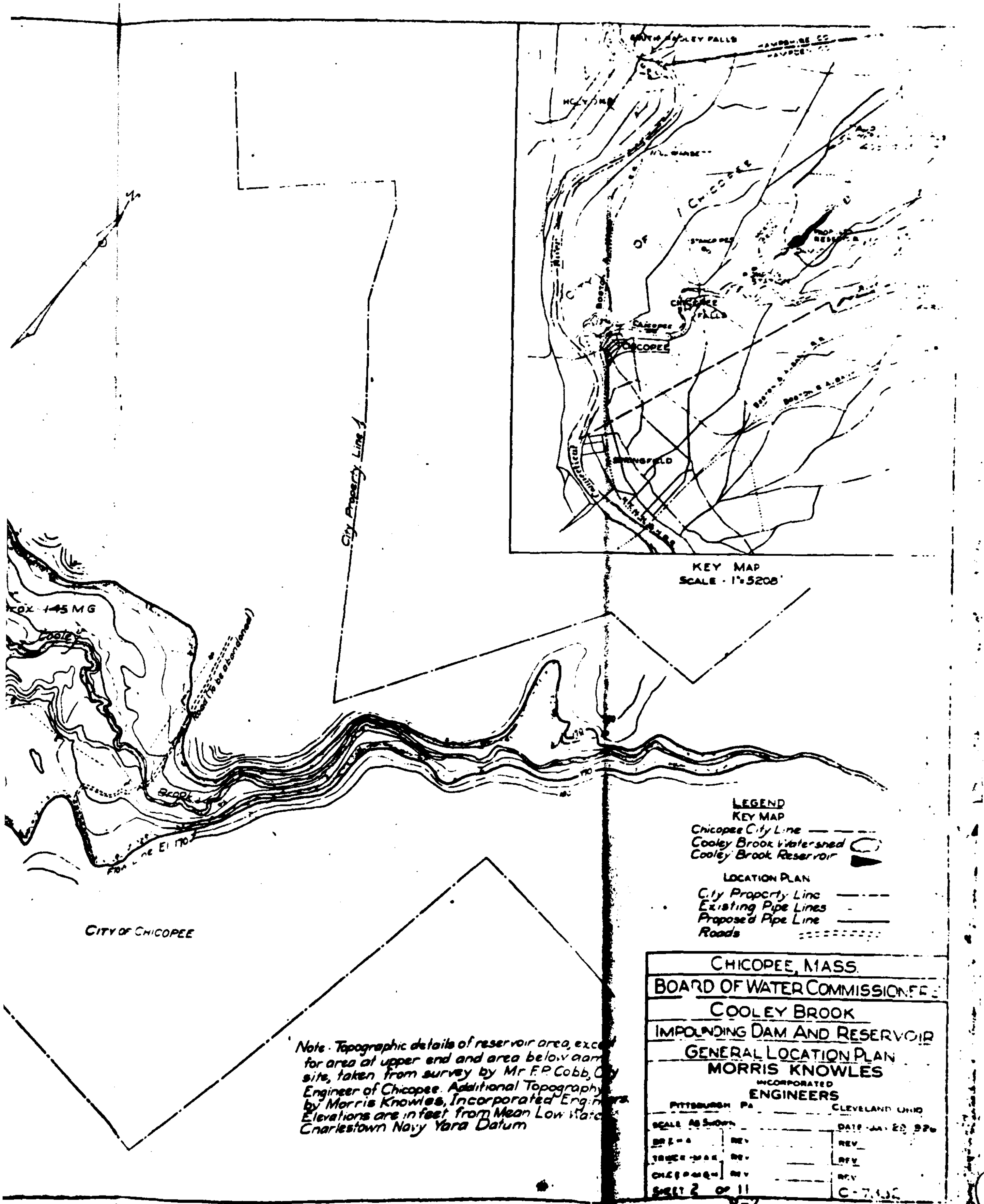
B-1

2

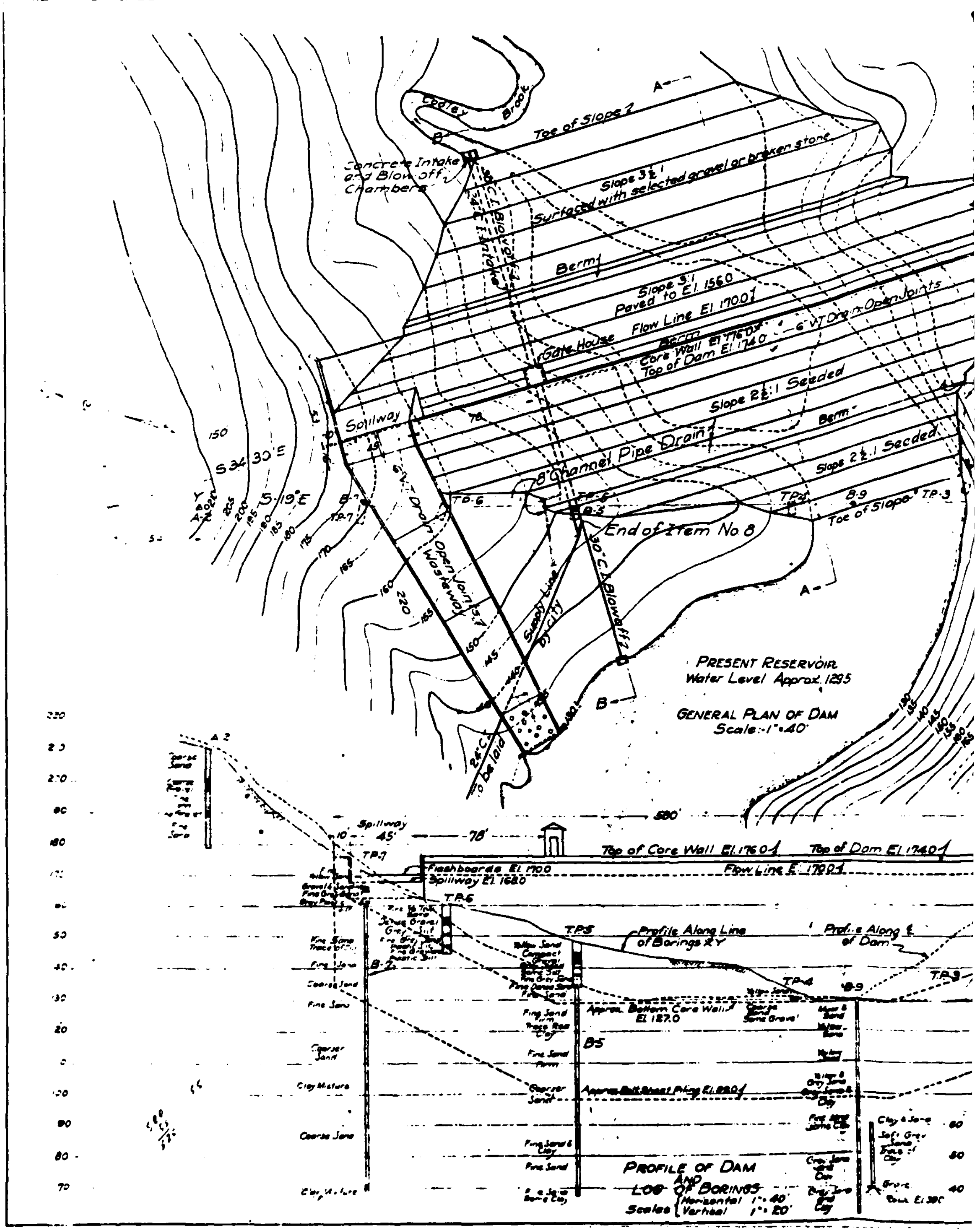


LOCATION PLAN
OF
IMPOUNDING DAM AND
RESERVOIR
Scale 1"=200'

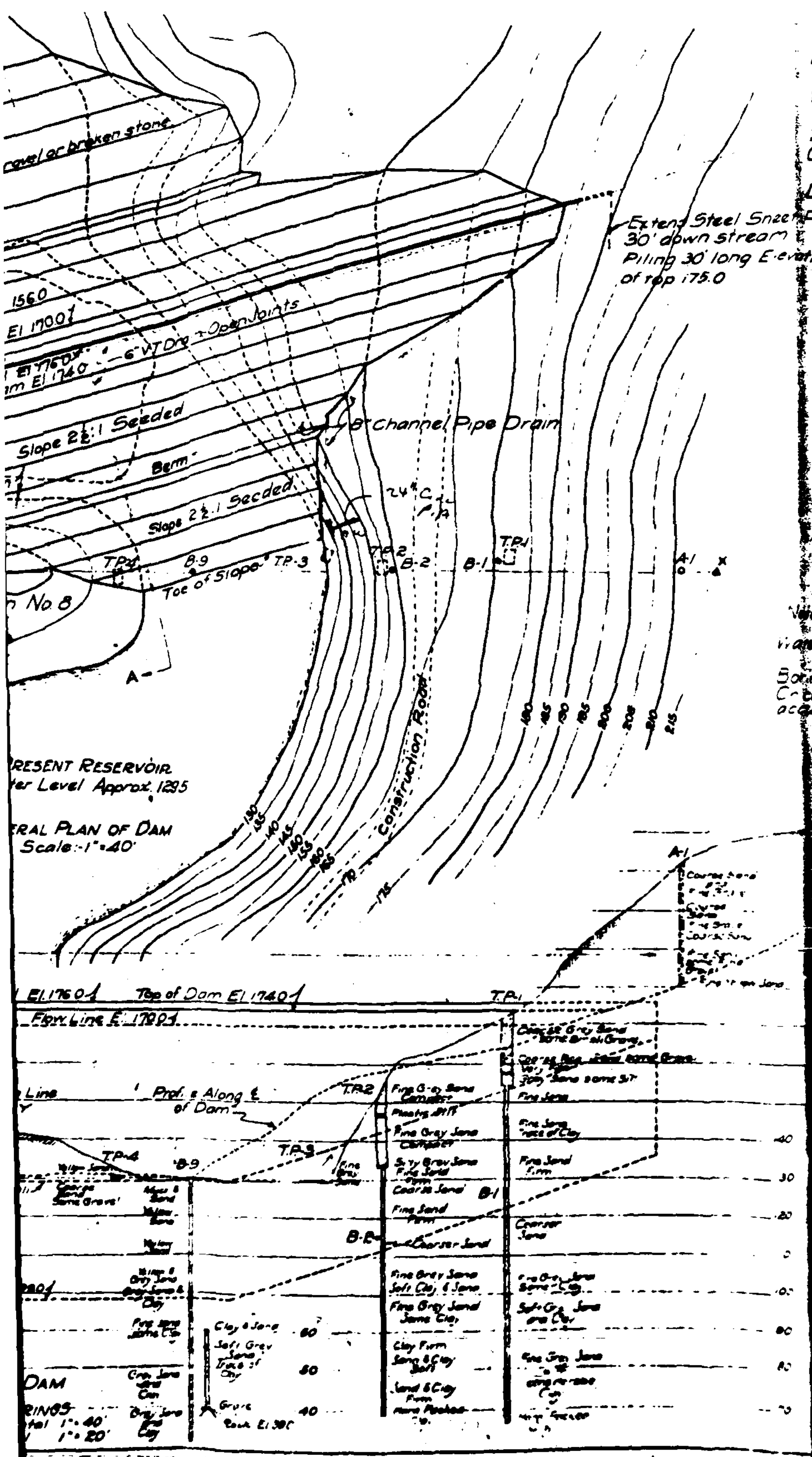
(1.)


KEY MAP
SCALE · 1"=5208'
City Property Line
Cooley Brook
Brook
CITY OF CHICOPEE
LEGEND
KEY MAP
Chicopee City Line
Cooley Brook Watershed
Cooley Brook Reservoir
LOCATION PLAN
City Property Line
Existing Pipe Lines
Proposed Pipe Line
Roads
Note · Topographic details of reservoir area, except for area at upper end and area below dam site, taken from survey by Mr F.P. Cobb, City Engineer of Chicopee. Additional Topography by Morris Knowles, Incorporated Engineers. Elevations are in feet from Mean Low Water Charlestown Navy Yard Datum
CHICOPEE, MASS.
BOARD OF WATER COMMISSIONERS
COOLEY BROOK
IMPOUNDING DAM AND RESERVOIR
GENERAL LOCATION PLAN
MORRIS KNOWLES
INCORPORATED
ENGINEERS
PITTSBURGH PA
CLEVELAND OHIO
SCALE AS SHOWN
REV
SHEET 2 OF 11

(2)



Copley Brook
Toe of Slope
Concrete Intake and Blow-off Chambers
Slope 3½:1 Surfaced with selected gravel or broken stone
Berm
Slope 3:1 Paved to El. 156.0
Gate House
Flow Line El. 170.0
Berm
Core Wall El. 176.0
Top of Dam El. 174.0
6" V.T. Drain Open Joints
Slope 2½:1 Seeded
Spillway
8" Channel Pipe Drain
Slope 2½:1 Seeded
Toe of Slope
End of Item No 8
30" C.I. Blow-off
Supply Line by City
Open Joint 6" V.T. Drain
Wasteway
Present Reservoir Water Level Approx. 129.5
General Plan of Dam
Scale:-1"=40'
S 34° 30' E
S 19° E
Spillway
Top of Core Wall El. 176.0
Top of Dam El. 174.0
Flashboards El. 170.0
Flow Line El. 170.0
Spillway El. 168.0
Profile Along Line of Borings X-Y
Profile Along ℄ of Dam
Approx. Bottom Core Wall El. 127.0
Approx. Butt Sheet Piling El. 99.0
Profile of Dam and Log of Borings
Scales Horizontal 1" = 40' Vertical 1" = 20'



Note:-

For Location Plan of Dam and Reservoir see Dwg C-7332

For Sections A-A and B-B see Dwgs C-7334

For Details of Slope Paving Core Wall etc see Dwg C-7335

For Details of Spillway and Wasteway see Dwgs C-7336, C-7337 C-7338 and C-7339

For Details of Gate House see Dwgs. C-7340 and C-734

LEGEND

- Test Pits
- Wash Borings (Dry Samples)
- Auger Borings

Test Pits and Borings

- Sand
- Sand, Some Gravel
- Gravel
- Sand, Some Clay
- Clay
- Silt
- Muck
- Rock

Note:

Elevations are in feet from Mean Low Water Charlestown Navy Yard Datum

Information shown by Test Pits and Borings is believed to be correct. The City of Chicopee, however, does not guarantee the accuracy of this information.

CHICOPEE MASS

BOARD OF WATER COMMISSIONERS

COOLEY BROOK

IMPOUNDING DAM AND RESERVOIR

PLAN AND PROFILE OF IMPOUNDING DAM

MORRIS KNOWLES

ENGINEERS

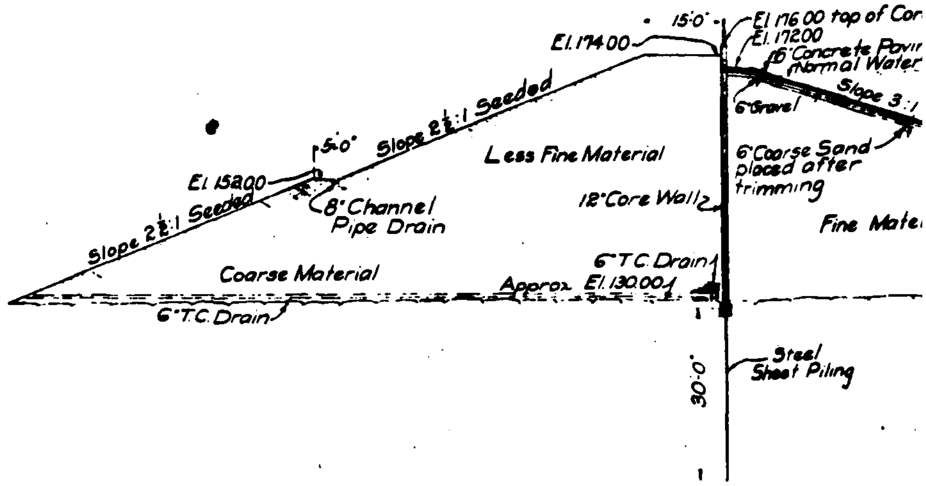
PITTSBURGH PA — CLEVELAND OHIO

Date Jan 29 1925

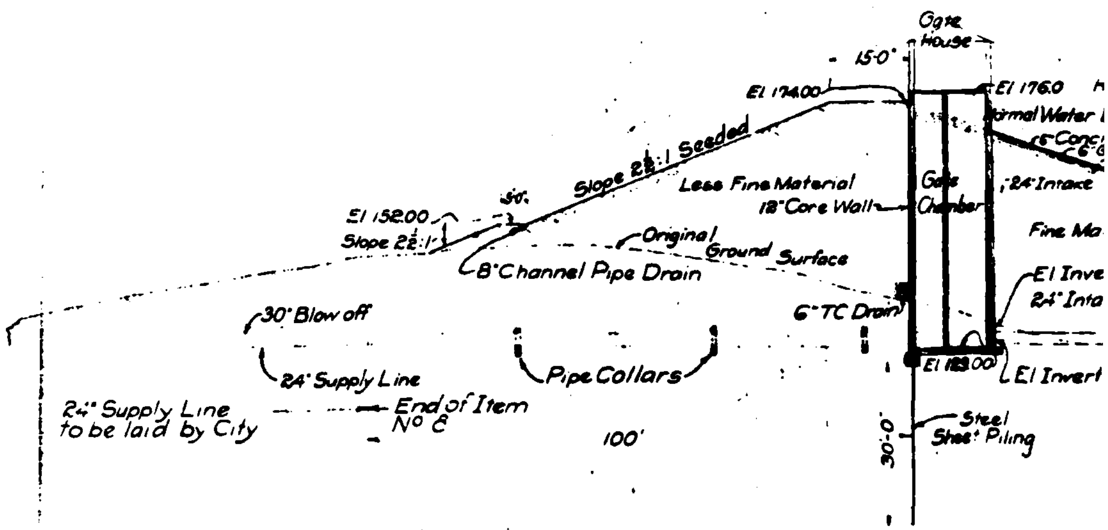
C-7333

B-3

2

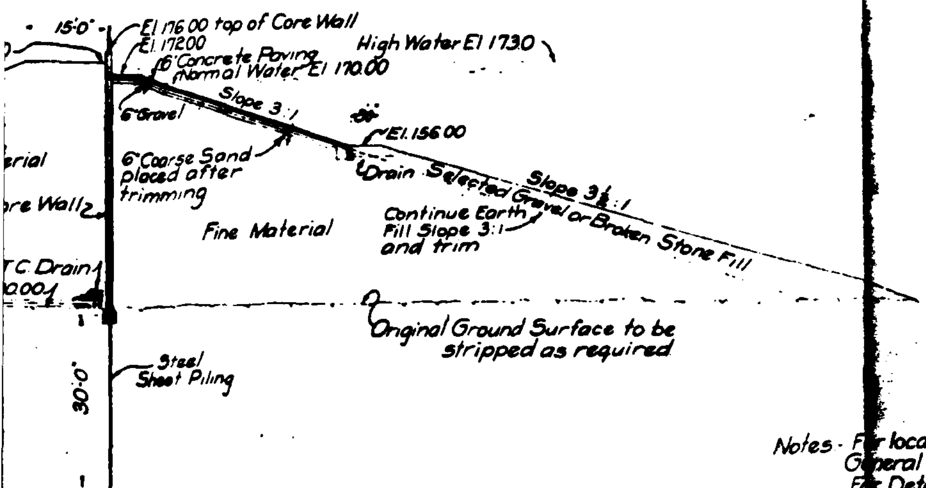


TYPICAL DAM SECTION A-A
Scale 1/16":1'-0"



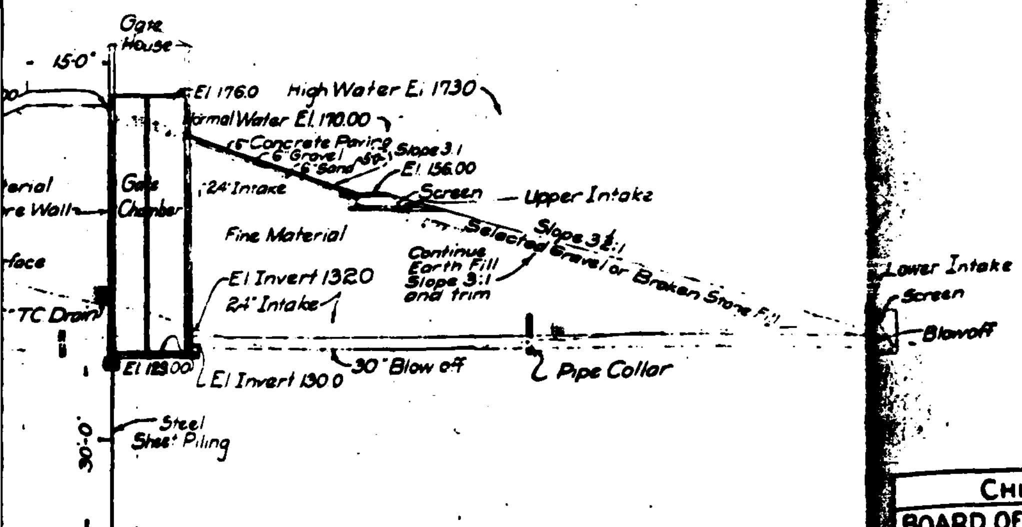
DAM SECTION B-B
Scale 1/16":1'-0"

1



TYPICAL DAM SECTION A-A
Scale 1/16":1'0"

Notes - For location of Sections A-A and B-B and General Plan of Dam see Dwg C-7333
For Details of Core Wall, Slope Paving, Inlets, etc. see Dwg C-7335
For Details of Gate House see Dwg C-7340
For Details of Spillway and Wasteway see Dwgs C-7336, C-7337, C-7338 and C-7339



DAM SECTION B-B
Scale 1/16":1'0"

CHICOPEE, MASS

BOARD OF WATER COMMISSIONERS

COOLEY BROOK
IMPOUNDING DAM AND RESERVOIR

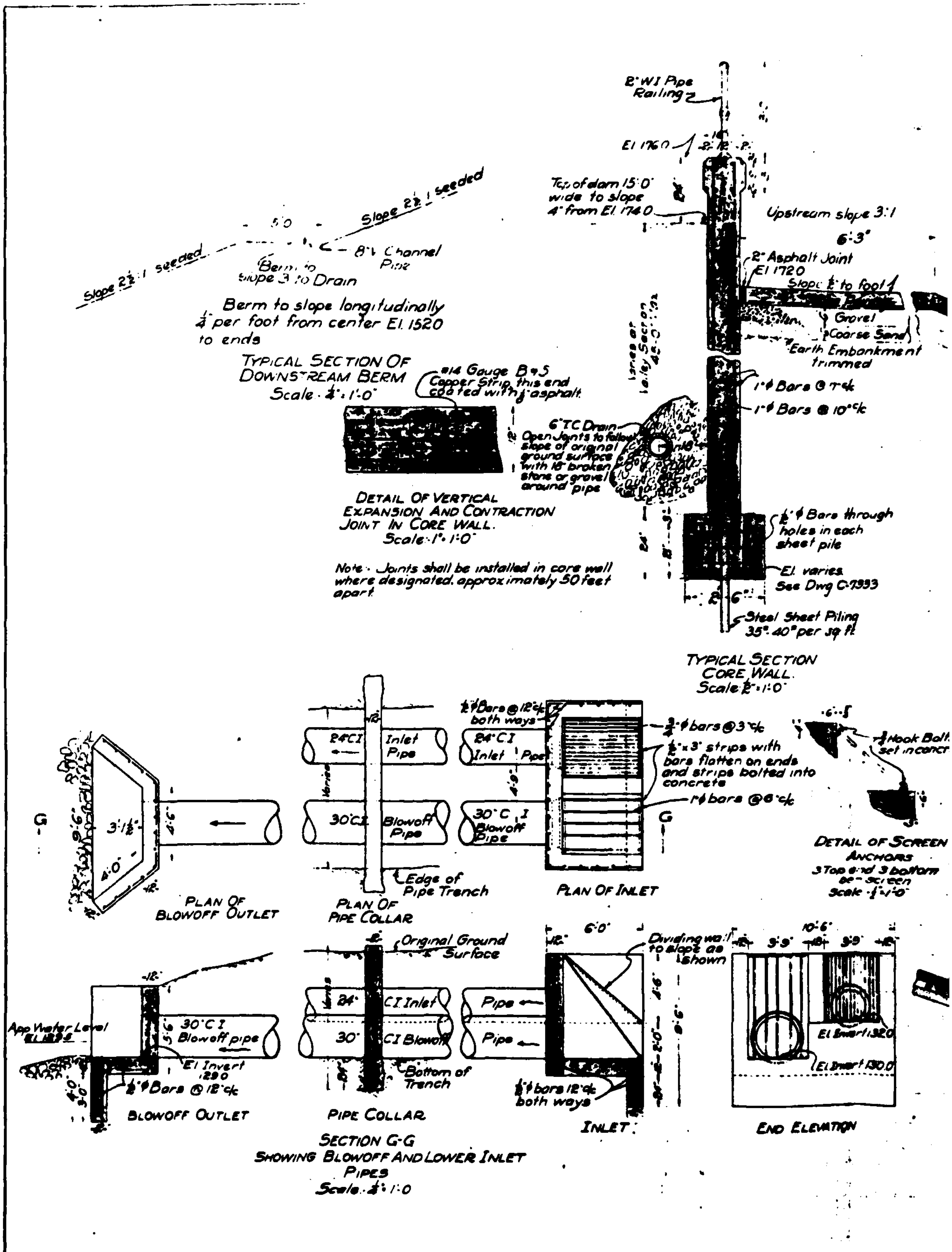
SECTIONS OF IMPOUNDING DAM

MORRIS KNOWLES
INCORPORATED
ENGINEERS

PITTSBURGH PA | CLEVELAND OHIO

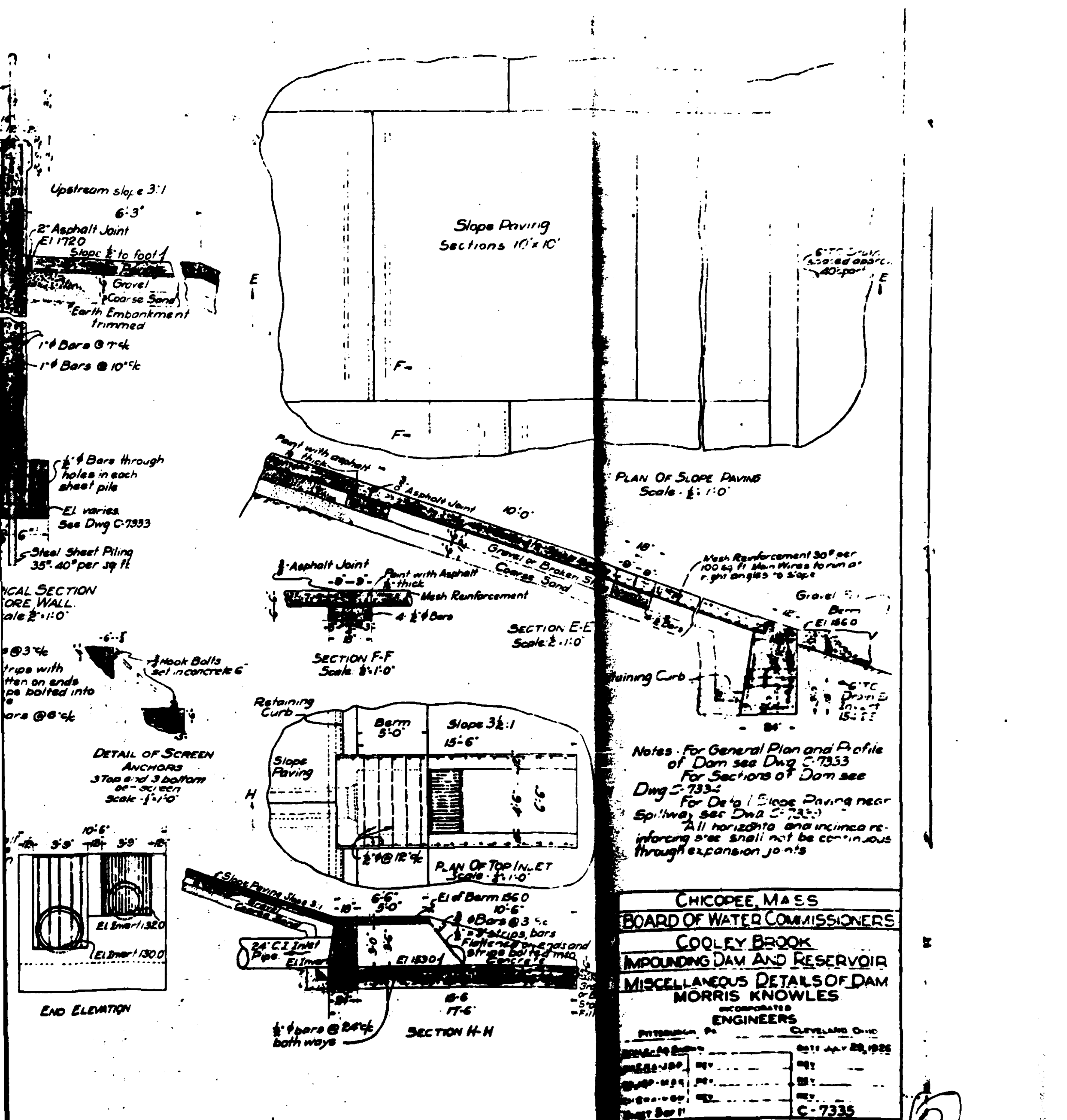
C-7334

2

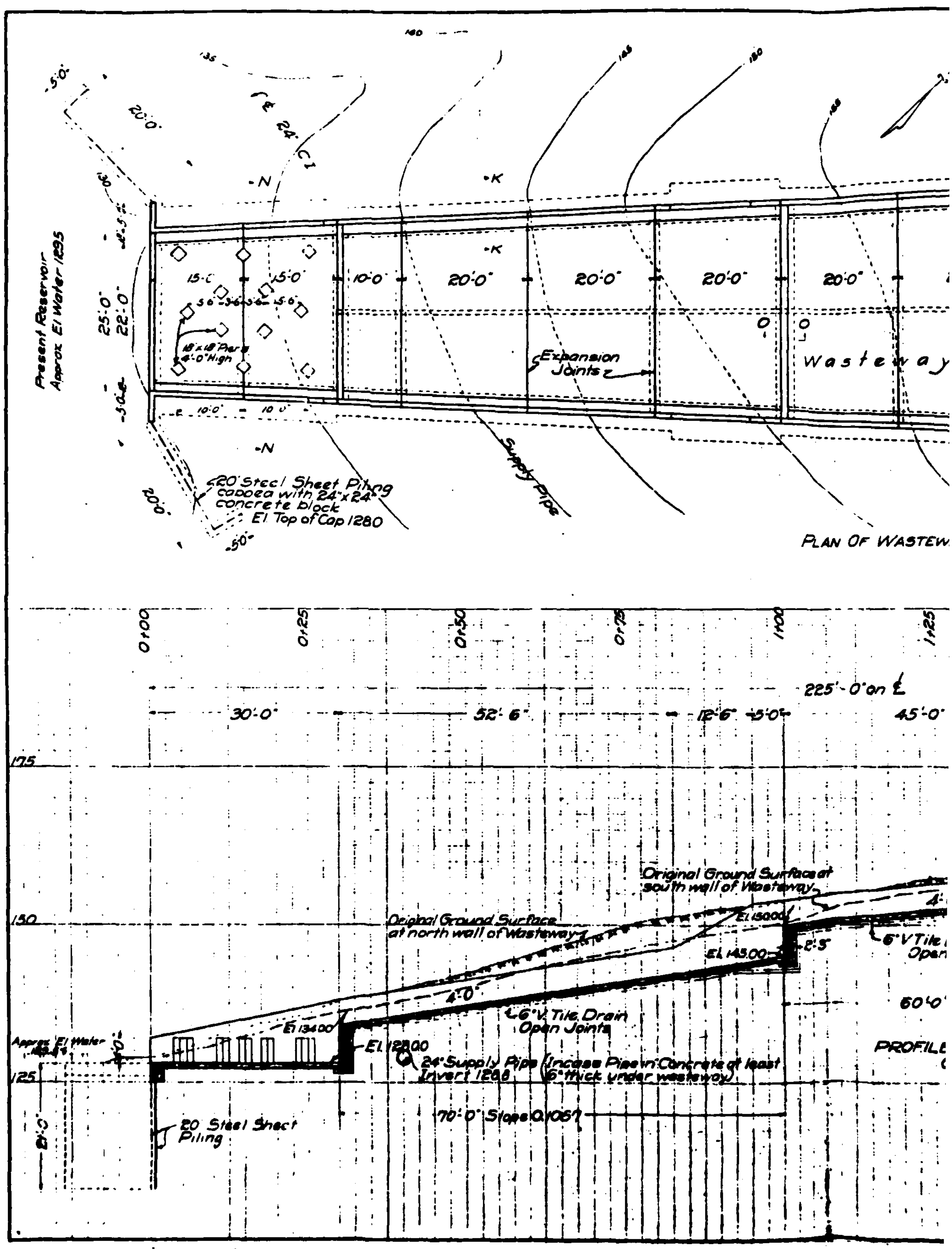


2" WI Pipe Railing
El 176.0
Top of dam 15'0" wide to slope 4" from El 174.0
Upstream slope 3:1
6'3"
2" Asphalt Joint El 172.0
Slope ½" to foot
Gravel
Coarse Sand
Earth Embankment trimmed
1"φ Bars @ 7"c/c
1"φ Bars @ 10"c/c
6" TC Drain
Open Joints to follow slope of original ground surface with 18" broken stone or gravel around pipe
½"φ Bars through holes in each sheet pile
El varies See Dwg C-7393
Steel Sheet Piling 35"-40" per sq ft
TYPICAL SECTION CORE WALL Scale ½"=1'0"
Slope 2½:1 seeded
Slope 2½:1 seeded
5'0
8" Channel Pipe
Berm to slope 3 to Drain
Berm to slope longitudinally ¼" per foot from center El 152.0 to ends
TYPICAL SECTION OF DOWNSTREAM BERM Scale ¼"=1'-0"
#14 Gauge B&S Copper Strip, this end coated with asphalt
DETAIL OF VERTICAL EXPANSION AND CONTRACTION JOINT IN CORE WALL. Scale 1"=1'0"
Note: Joints shall be installed in core wall where designated, approximately 50 feet apart.
G
3'1½"
4'6"
4'0"
PLAN OF BLOWOFF OUTLET
24"CI Inlet Pipe
30"CI Blowoff Pipe
Edge of Pipe Trench
PLAN OF PIPE COLLAR
½"φ Bars @ 12"c/c both ways
24" CI Inlet Pipe
30" C I Blowoff Pipe
¾"φ bars @ 3"c/c
½"x3" strips with bars flatten on ends and strips bolted into concrete
1"φ bars @ 6"c/c
PLAN OF INLET
¾" Hook Bolt set in concr
DETAIL OF SCREEN ANCHORS 3 Top and 3 bottom per screen Scale ½"=1'0"
App Water Level El 133.5
30" CI Blowoff pipe
El Invert 129.0
½"φ Bars @ 12"c/c
BLOWOFF OUTLET
Original Ground Surface
24" CI Inlet
30" CI Blowoff
Bottom of Trench
PIPE COLLAR
6'0"
Dividing wall to slope as shown
Pipe
Pipe
½"φ bars 12"c/c both ways
INLET
10'6"
3'9"
3'9"
El Invert 132.0
El Invert 130.0
END ELEVATION
SECTION G-G SHOWING BLOWOFF AND LOWER INLET PIPES Scale ¼"=1'0

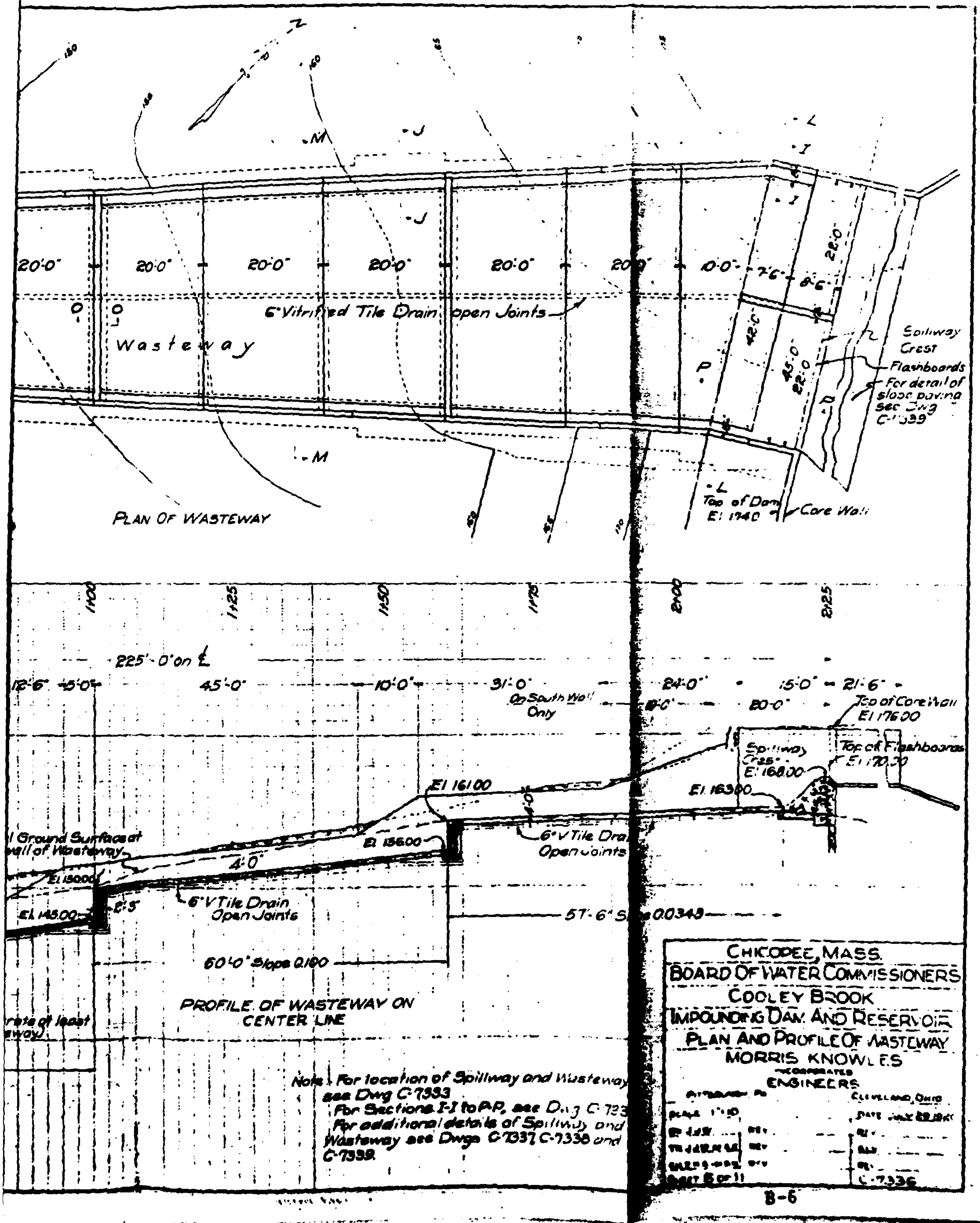
(1)



Upstream slope 3:1
2" Asphalt Joint
El 1720
Gravel
Coarse Sand
Earth Embankment trimmed
1"φ Bars @ 10"c/c
Steel Sheet Piling 35#-40# per sq ft
Slope Paving Sections 10'x10'
PLAN OF SLOPE PAVING Scale · ⅛" = 1'-0"
SECTION E-E
SECTION F-F
Mesh Reinforcement
Retaining Curb
Berm 5'-0"
Slope 3½:1
Slope Paving
PLAN OF TOP INLET
DETAIL OF SCREEN ANCHORS
END ELEVATION
24" C.I. Inlet Pipe
SECTION H-H
Notes · For General Plan and Profile of Dam see Dwg C-7333
For Sections of Dam see Dwg C-7334
All horizontal and inclined reinforcing steel shall not be continuous through expansion joints
CHICOPEE, MASS
BOARD OF WATER COMMISSIONERS
COOLEY BROOK
IMPOUNDING DAM AND RESERVOIR
MISCELLANEOUS DETAILS OF DAM
MORRIS KNOWLES
ENGINEERS
C-7335

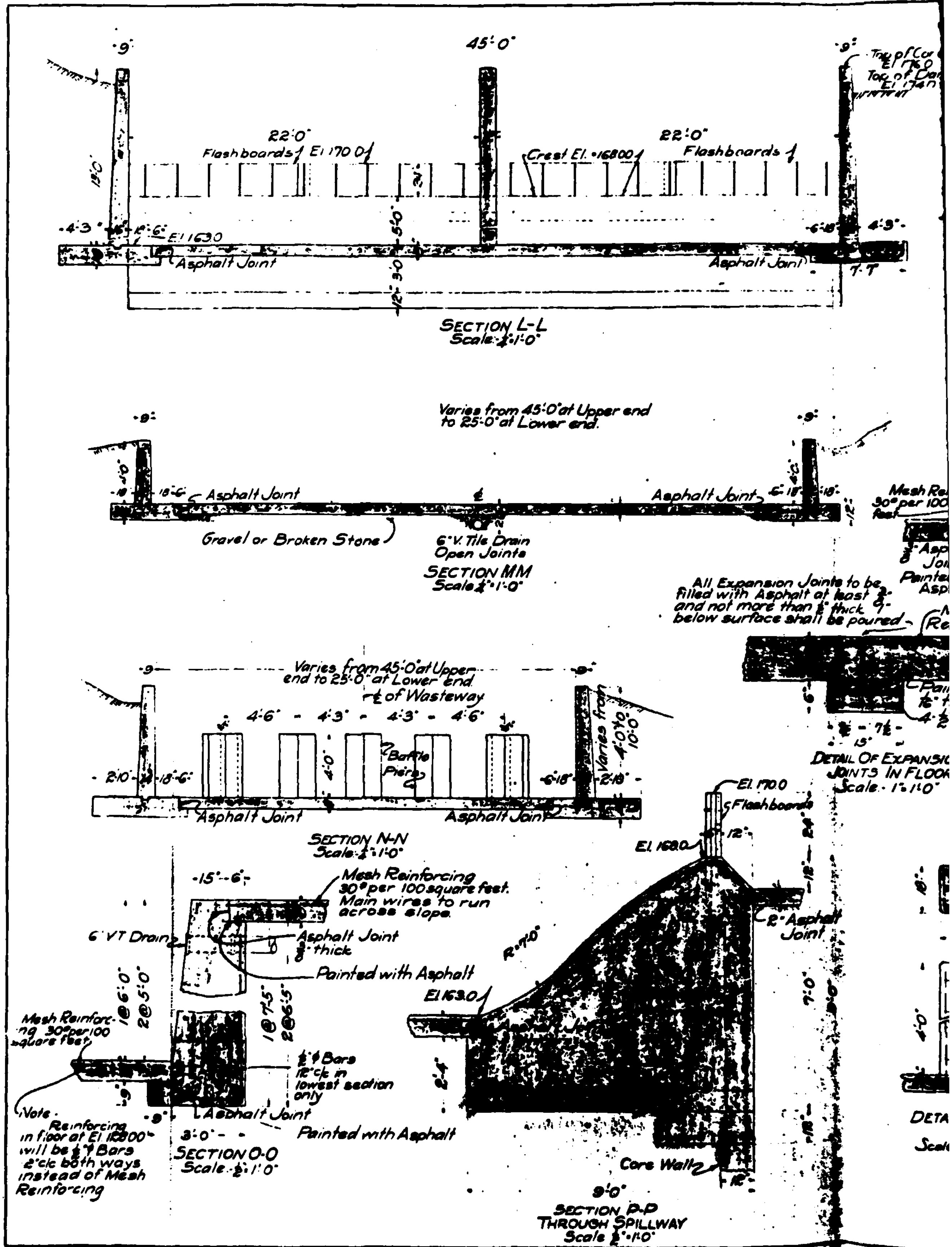


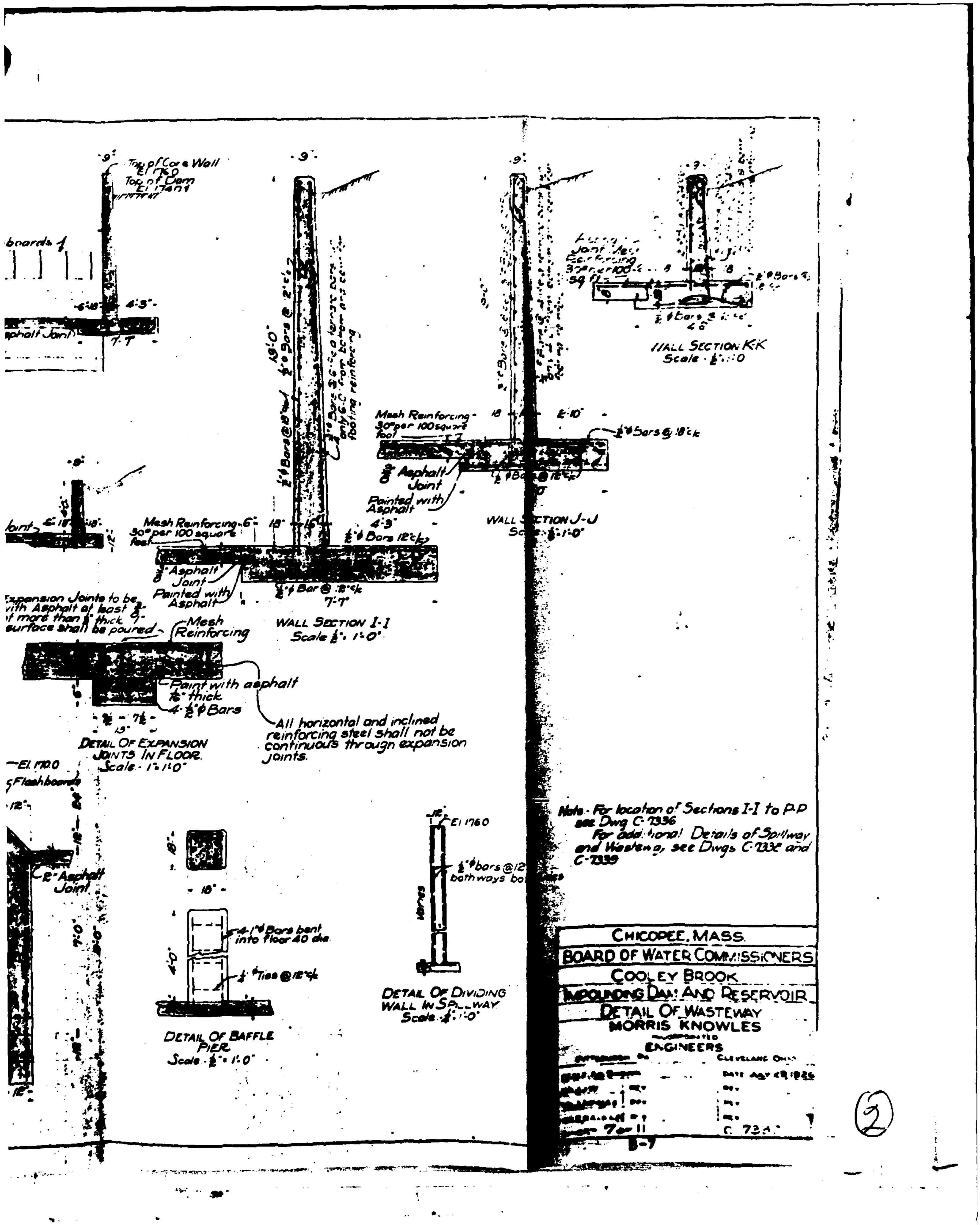
Present Reservoir Approx. El Water 1285
20'0"
5'0"
25'0"
22'0"
15'0"
10'0"
20'0"
18"x18" Piers 4'0" High
Expansion Joints
Wasteway
Supply Pipe
20' Steel Sheet Piling capped with 24"x24" concrete block El Top of Cap 1280
PLAN OF WASTEW
0+00
0+25
0+50
0+75
1+00
1+25
225'-0" on ℄
30'-0"
52'-6"
12'-6"
5'-0"
45'-0"
175
130
125
Original Ground Surface at south wall of Wasteway
Original Ground Surface at north wall of Wasteway
El 150.00
El 145.00
2.5'
4'-0"
6" V Tile Drain Open Joints
60'-0"
El 134.00
El 128.00
24" Supply Pipe (Incase Pipe in Concrete at least 6" thick under wasteway) Invert 126.8
PROFILE
70'-0" Slope 0.1057
20 Steel Sheet Piling


Wasteway
20'-0"
6" Vitrified Tile Drain, open Joints
Spillway Crest
Flashboards
For detail of slope paving see Dwg C-7339
Top of Dam El. 174.0
Core Wall
PLAN OF WASTEWAY
225'-0" on ℄
12'-6"
15'-0"
45'-0"
10'-0"
31'-0"
On South Wall Only
24'-0"
15'-0"
21'-6"
80'-0"
Top of Core Wall El. 176.00
Spillway Crest El. 168.00
Top of Flashboards El. 170.30
El. 161.00
El. 163.00
El. 156.00
6" V Tile Drain Open Joints
Ground Surface at wall of Wasteway
El. 150.00
El. 148.00
4'-0"
57'-6" Slope 0.0348
60'-0" Slope 0.100
PROFILE OF WASTEWAY ON CENTER LINE
Note: For location of Spillway and Wasteway see Dwg C-7333
For Sections I-I to P-P, see Dwg C-733
For additional details of Spillway and Wasteway see Dwgs C-7337, C-7338 and C-7339.
CHICOPEE, MASS.
BOARD OF WATER COMMISSIONERS
COOLEY BROOK
IMPOUNDING DAM AND RESERVOIR
PLAN AND PROFILE OF WASTEWAY
MORRIS KNOWLES
ENGINEERS
CLEVELAND, OHIO
SCALE 1"=10'
SHEET 6 OF 11
C-7336

B-6

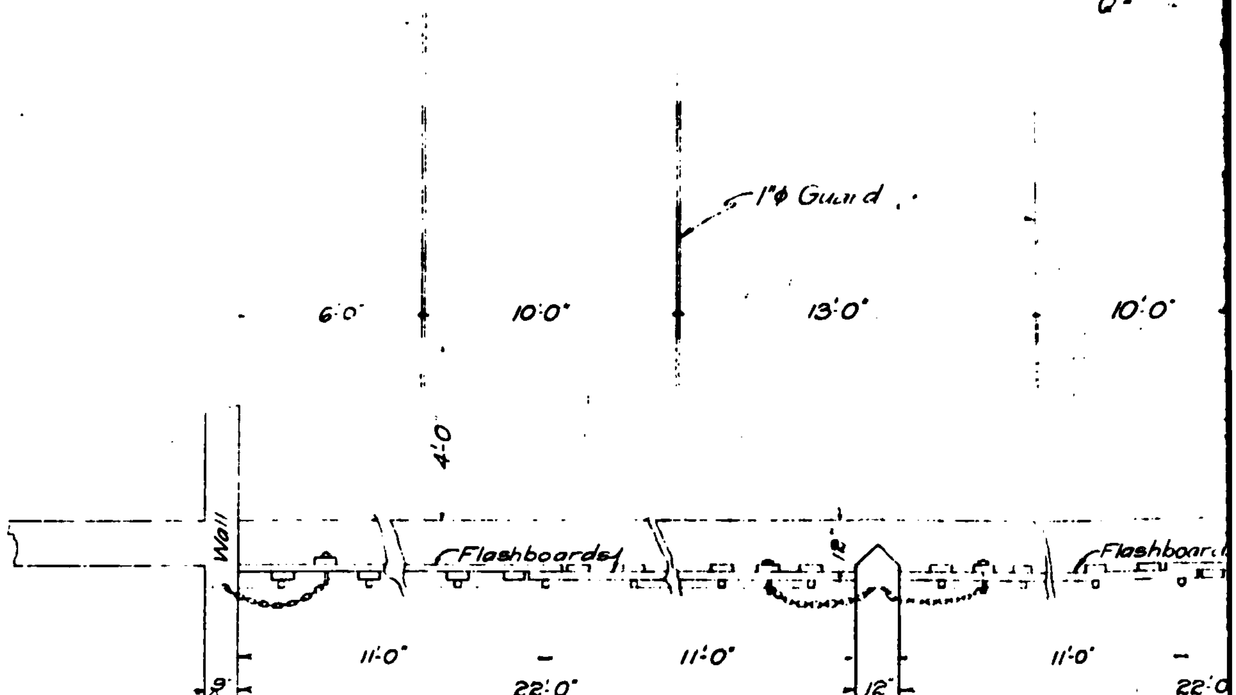
2


45'-0"
22'-0"
Flashboards El 170.0
Crest El. 168.00
22'-0"
Flashboards
El 163.0
Asphalt Joint
Asphalt Joint
SECTION L-L
Scale: 1/4"=1'-0"
Varies from 45'-0" at Upper end
to 25'-0" at Lower end.
Asphalt Joint
Asphalt Joint
Gravel or Broken Stone
6" V. Tile Drain
Open Joints
SECTION MM
Scale 1/4"=1'-0"
All Expansion Joints to be filled with Asphalt at least
and not more than thick
below surface shall be poured
Varies from 45'-0" at Upper end to 25'-0" at Lower end
℄ of Wasteway
4'-6" - 4'-3" - 4'-3" - 4'-6"
Baffle Piers
Varies from 4'-0" to 10'-0"
Asphalt Joint
Asphalt Joint
SECTION N-N
Scale 1/4"=1'-0"
DETAIL OF EXPANSIO
JOINTS IN FLOOR
Scale 1"=1'-0"
El. 170.0
Flashboards
El. 168.0
2" Asphalt Joint
Mesh Reinforcing
30# per 100 square feet.
Main wires to run
across slope.
6" VT Drain
Asphalt Joint
thick
Painted with Asphalt
El 163.0
Mesh Reinforcing 30# per 100 square feet
Bars
12" c/c in
lowest section
only
Asphalt Joint
Painted with Asphalt
Note -
Reinforcing
in floor at El 163.00
will be Bars
c/c both ways
instead of Mesh
Reinforcing
SECTION O-O
Scale 1/4"=1'-0"
Core Wall
9'-0"
SECTION P-P
THROUGH SPILLWAY
Scale 1/4"=1'-0"

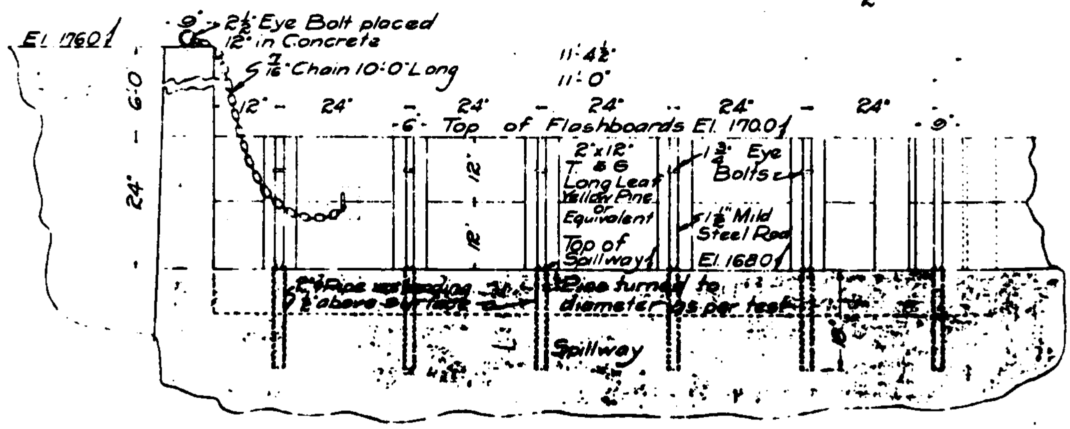


Top of Core Wall El 176.0
Top of Dam El 174.0
Mesh Reinforcing 6" 30# per 100 square feet
3/8" Asphalt Joint Painted with Asphalt
1/2" φ Bars 12" c/c
1/2" φ Bar @ 12" c/c
7'-7"
WALL SECTION I-I
Scale 1/2" = 1'-0"
Mesh Reinforcing 30# per 100 square feet
3/8" Asphalt Joint Painted with Asphalt
1/2" φ Bars @ 18" c/c
WALL SECTION J-J
WALL SECTION K-K
Expansion Joints to be ... with Asphalt at least ... not more than 1/2" thick ... surface shall be poured
Mesh Reinforcing
Paint with asphalt 1/2" thick
4-1/2" φ Bars
All horizontal and inclined reinforcing steel shall not be continuous througn expansion joints.
DETAIL OF EXPANSION JOINTS IN FLOOR.
Scale 1" = 1'-0"
El 170.0
Flashboards
2" Asphalt Joint
4-1" φ Bars bent into floor 40 dia.
1/2" φ Ties @ 12" c/c
DETAIL OF BAFFLE PIER
Scale 1/2" = 1'-0"
El 176.0
1/2" φ bars @ 12" both ways
varies
DETAIL OF DIVIDING WALL IN SPILLWAY
Note - For location of Sections I-I to P-P see Dwg C-7336
For additional Details of Spillway and Wasteway, see Dwgs C-7338 and C-7339
CHICOPEE, MASS.
BOARD OF WATER COMMISSIONERS
COOLEY BROOK
IMPOUNDING DAM AND RESERVOIR
DETAIL OF WASTEWAY
MORRIS KNOWLES
INCORPORATED
ENGINEERS
CLEVELAND OHIO
7 of 11
B-7

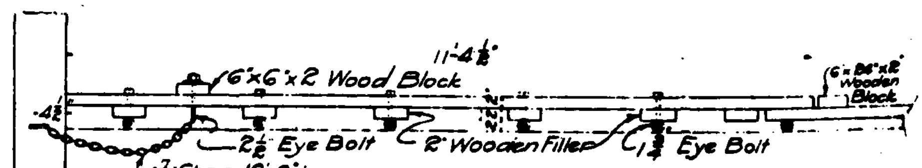
2



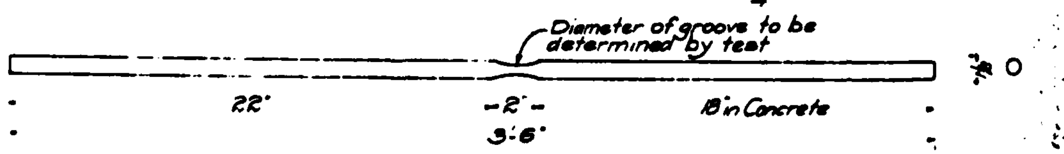
GENERAL PLAN OF FLASHBOARDS
Scale: 1/2" = 1'-0"



ELEVATION OF FLASHBOARD
Scale: 3/4" = 1'-0"

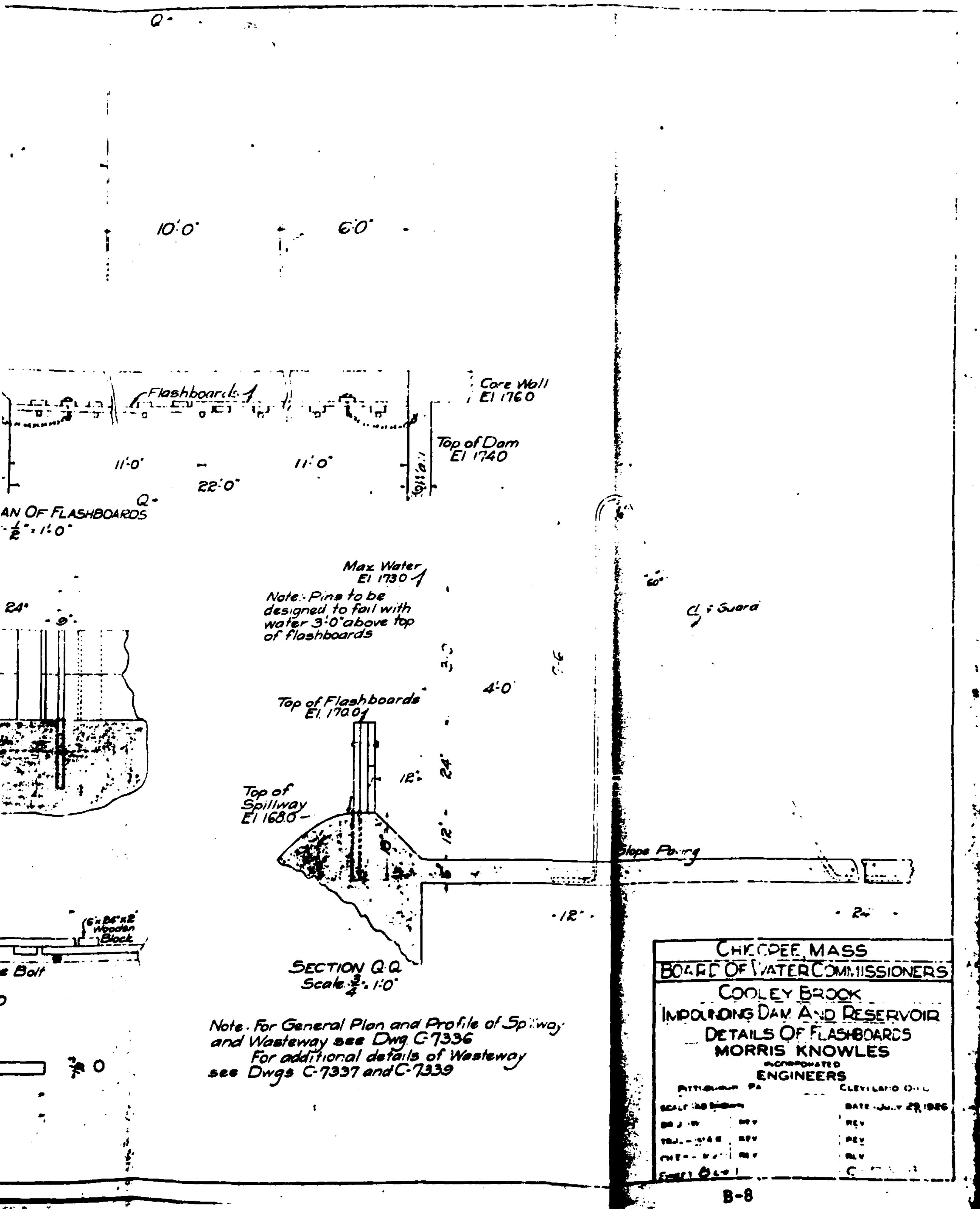


PLAN OF FLASHBOARD
Scale 3/4" = 1'-0"

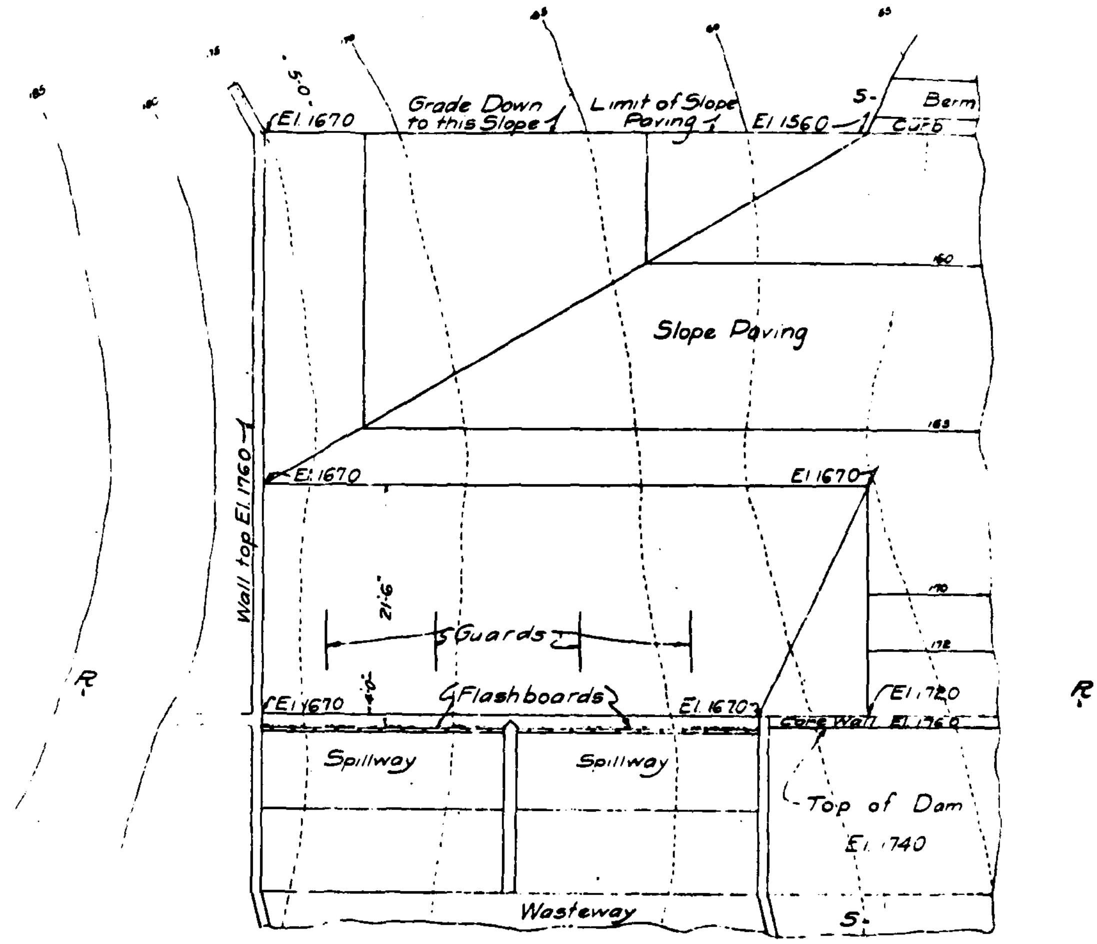


DETAILS OF PIN
Scale 3" = 1'-0"

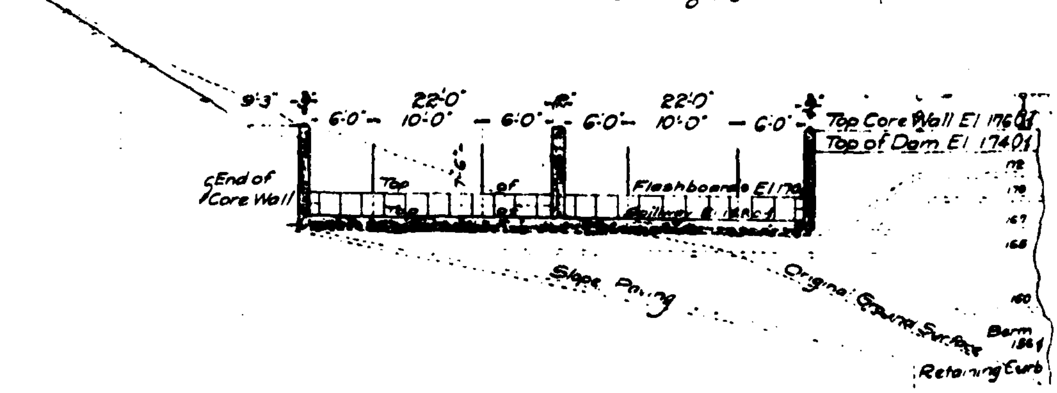
No
an
se



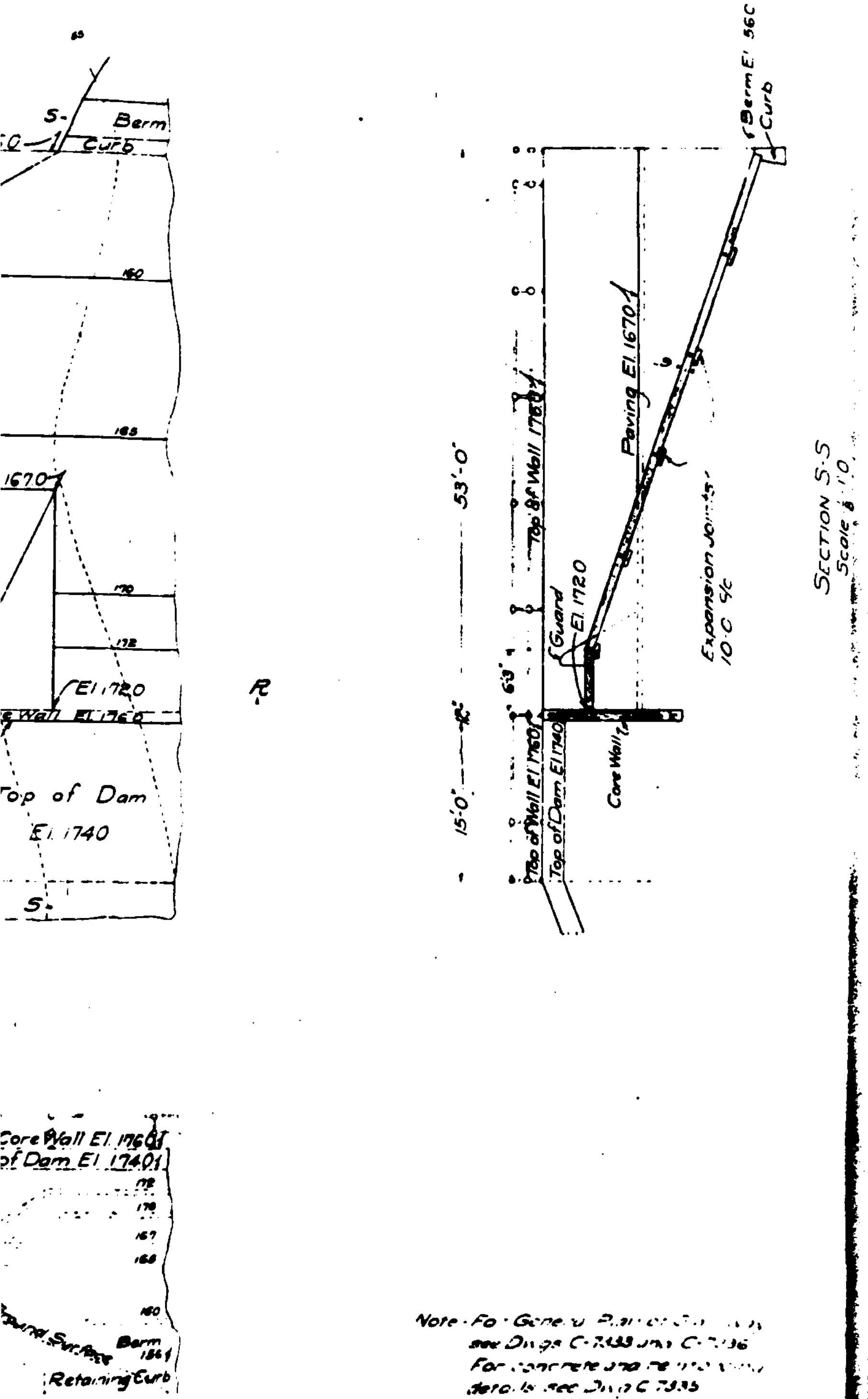
Q
10'0"
6'0"
Flashboards
Core Wall
El 176.0
Top of Dam
El 174.0
11'0"
22'0"
11'0"
AN OF FLASHBOARDS
Max Water
El 173.0
Note: Pins to be designed to fail with water 3'0" above top of flashboards
24"
9"
4'0"
Top of Flashboards
El 170.0
12"
24"
Top of Spillway
El 168.0
12"
Slope Paving
12"
24"
6"x 36" x 2" Wooden Block
Bolt
SECTION Q-Q
Scale 3/4" = 1'0"
Note. For General Plan and Profile of Spillway and Wasteway see Dwg C-7336
For additional details of Wasteway see Dwgs C-7337 and C-7339
CHICOPEE, MASS
BOARD OF WATER COMMISSIONERS
COOLEY BROOK
IMPOUNDING DAM AND RESERVOIR
DETAILS OF FLASHBOARDS
MORRIS KNOWLES
INCORPORATED
ENGINEERS
PITTSBURGH PA
CLEVELAND OHIO
DATE JULY 29, 1926



PLAN OF SLOPE PAVING
AT SPILLWAY
Scale $\frac{1}{8}$"=1'-0"

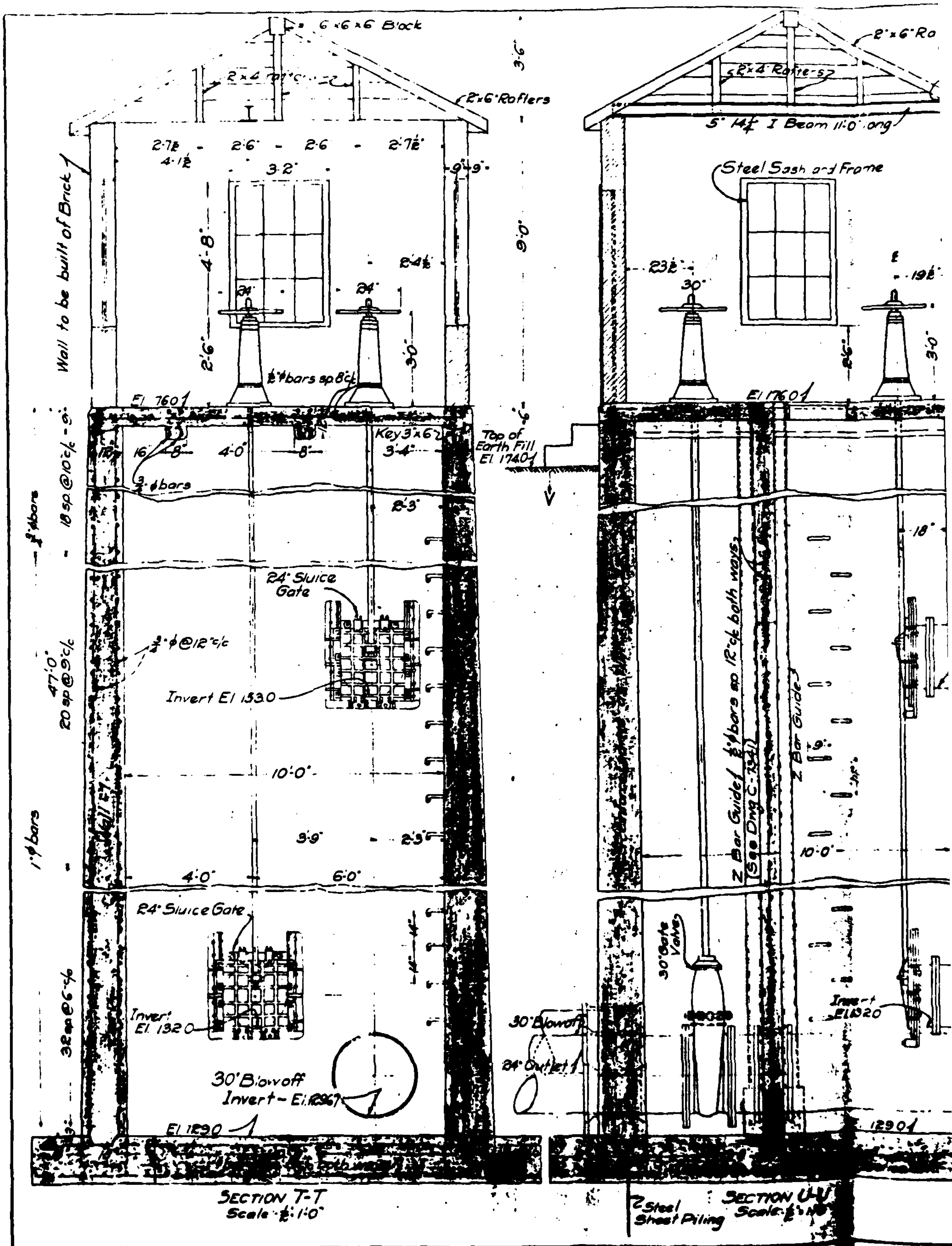


SECTION R-R
SHOWING DEVELOPMENT OF SLOPE
PAVING
Scale - $\frac{1}{8}$"=1'-0"

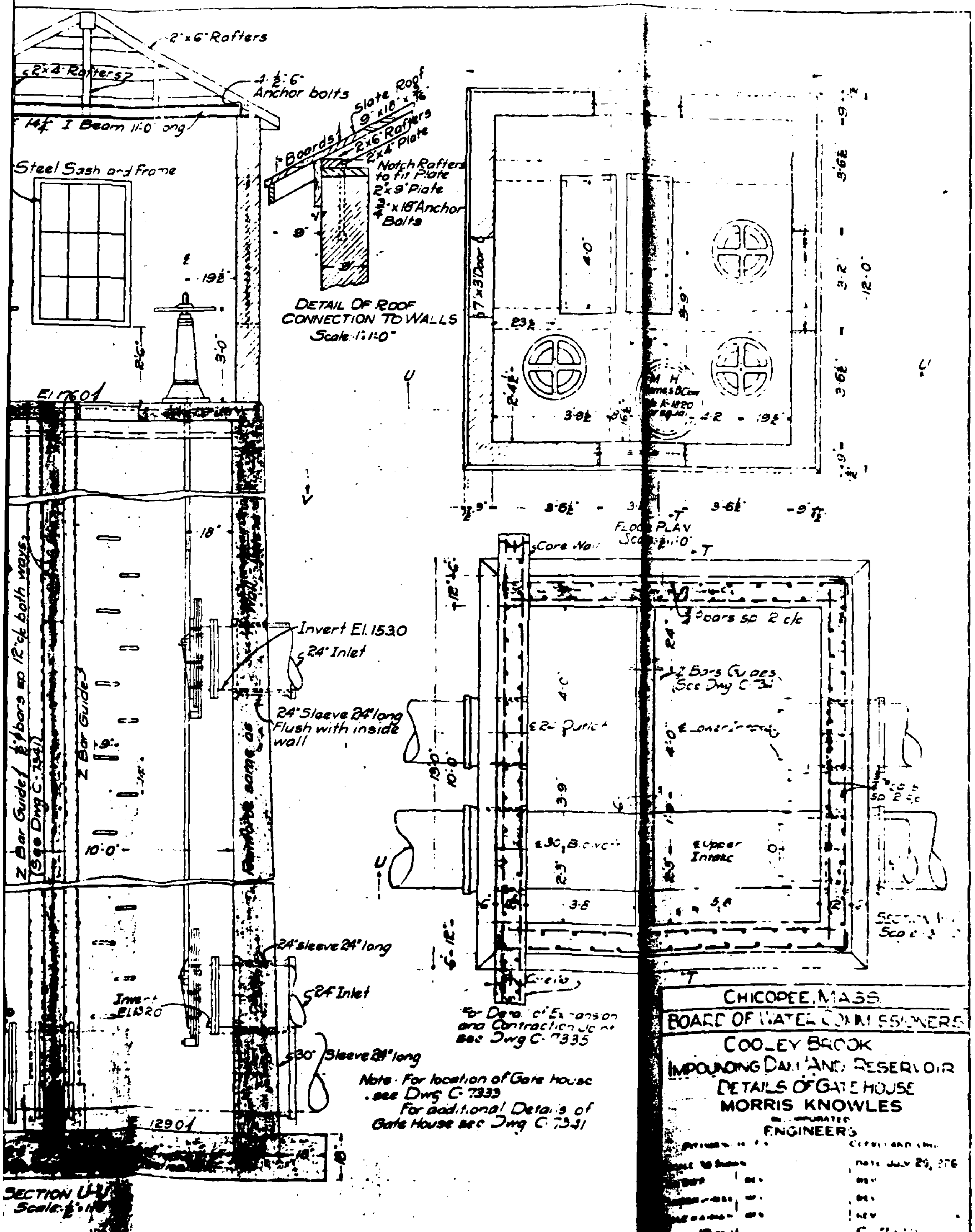


Note - For General Plan ... see Dwgs C-7333 and C-7336. For concrete ... details see Dwg C 7335

CHICOPEE, MASS
BOARD OF WATER COMMISSIONERS
COOLEY BROOK
IMPOUNDING DAM AND RESERVOIR
DETAIL OF SLOPE PAVING & SPILLWAY
MORRIS KNOWLES
ENGINEERS
PITTSBURGH, PA CLEVELAND, OHIO
DATE JULY 20, 192-
C 7339



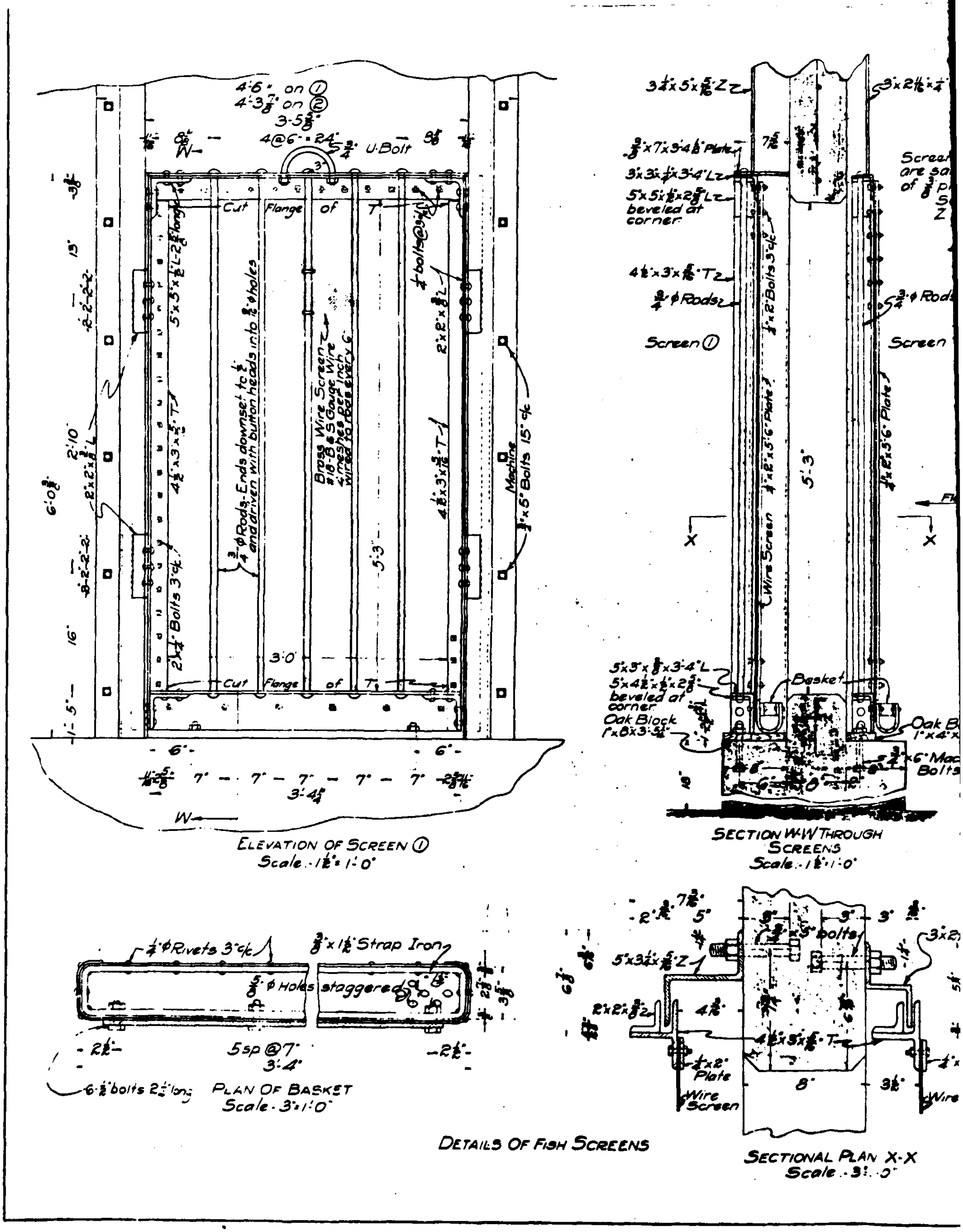
6 x 6 x 6 Block
2 x 4 Rafters
2 x 6 Rafters
Wall to be built of Brick
2·7½ - 2·6 - 2·6 - 2·7½
4·1½
3·2
9·9
3·6
9·0
4·8
2·4½
24
24
2·6
3·0
½ φ bars sp 8 c/c
El 1760
Key 3 x 6
16
8
4·0
8
3·4
Top of Earth Fill El 1740
¾ φ bars
18 sp @ 10 c/c - 9
2·3
24 Sluice Gate
¾ φ @ 12 c/c
Invert El 1530
47·0
20 sp @ 9 c/c
10·0
3·9
2·3
4·0
6·0
1 φ bars
24 Sluice Gate
Invert El 1320
32 sp @ 6 c/c
30 Blowoff
Invert - El 1296
El 1290
30 Blowoff
24 Outlet
SECTION T-T
Scale ¼ : 1·0
2 x 6 Ra
2 x 4 Rafters
5 14½ I Beam 11·0 long
Steel Sash and Frame
23½
30
19½
2·6
3·0
El 1760
18
Z Bar Guide
Z Bar Guide
¾ φ bars sp 12 c/c both ways
(See Dwg C-794)
9
10·0
30 Gate Valve
Invert El 1320
1290
Z Steel Sheet Piling
SECTION U-U
Scale ¼ : 1·0



2"x6" Rafters
2"x4" Rafters
4-1/2"-6" Anchor bolts
14" I Beam 11'-0" long
Steel Sash and Frame
Slate Roof 9"x18"x3/16"
Boards
2"x6" Rafters
2"x4" Plate
Notch Rafters to fit Plate
2"x9" Plate
3/4"x18" Anchor Bolts
DETAIL OF ROOF CONNECTION TO WALLS
Scale 1"=1'-0"
El. 176.0
3'-7"x3' Door
FLOOR PLAN
Core Wall
3/4" bars sp. 2' c/c
Z Bar Guides (See Dwg C-7341)
Invert El. 153.0
24" Inlet
24" Sleeve 24" long Flush with inside wall
Z Bar Guides
Z Bar Guides 1/2" bars sp. 12" c/c both ways (See Dwg C-7341)
24" sleeve 24" long
24" Inlet
Invert El. 132.0
30" Sleeve 24" long
El. 129.0
Upper Intake
Note: For location of Gate house see Dwg C-7333
For additional Details of Gate House see Dwg C-7341
For Detail of Expansion and Contraction Joint see Dwg C-7335
SECTION U-U
CHICOPEE, MASS.
COOLEY BROOK
IMPOUNDING DAM AND RESERVOIR
DETAILS OF GATE HOUSE
MORRIS KNOWLES
ENGINEERS

B-10

(2)



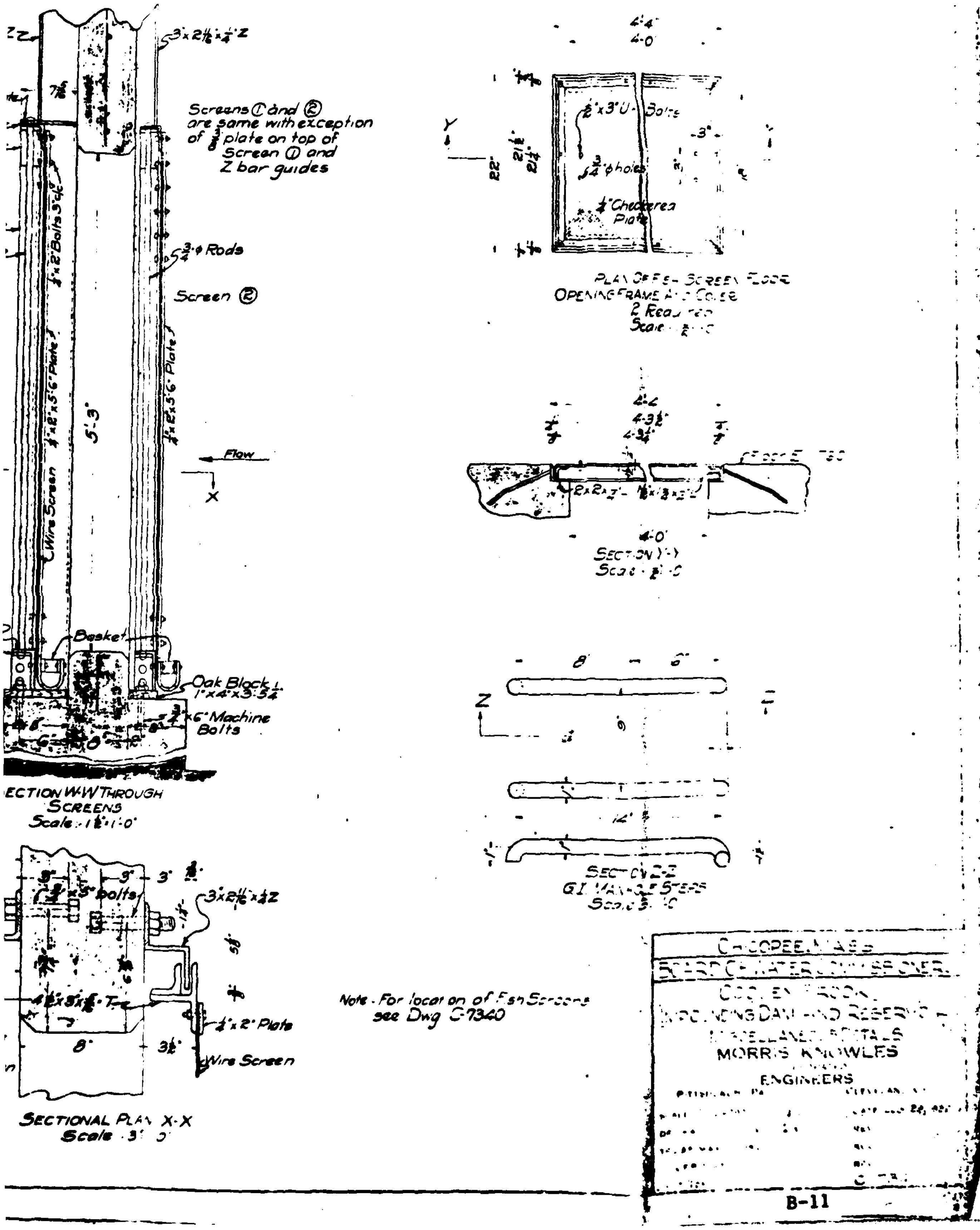
ELEVATION OF SCREEN ①
Scale 1½" = 1'-0"

SECTION W-W THROUGH SCREENS
Scale 1½" = 1'-0"

PLAN OF BASKET
Scale 3" = 1'-0"

SECTIONAL PLAN X-X
Scale 3" = 1'-0"

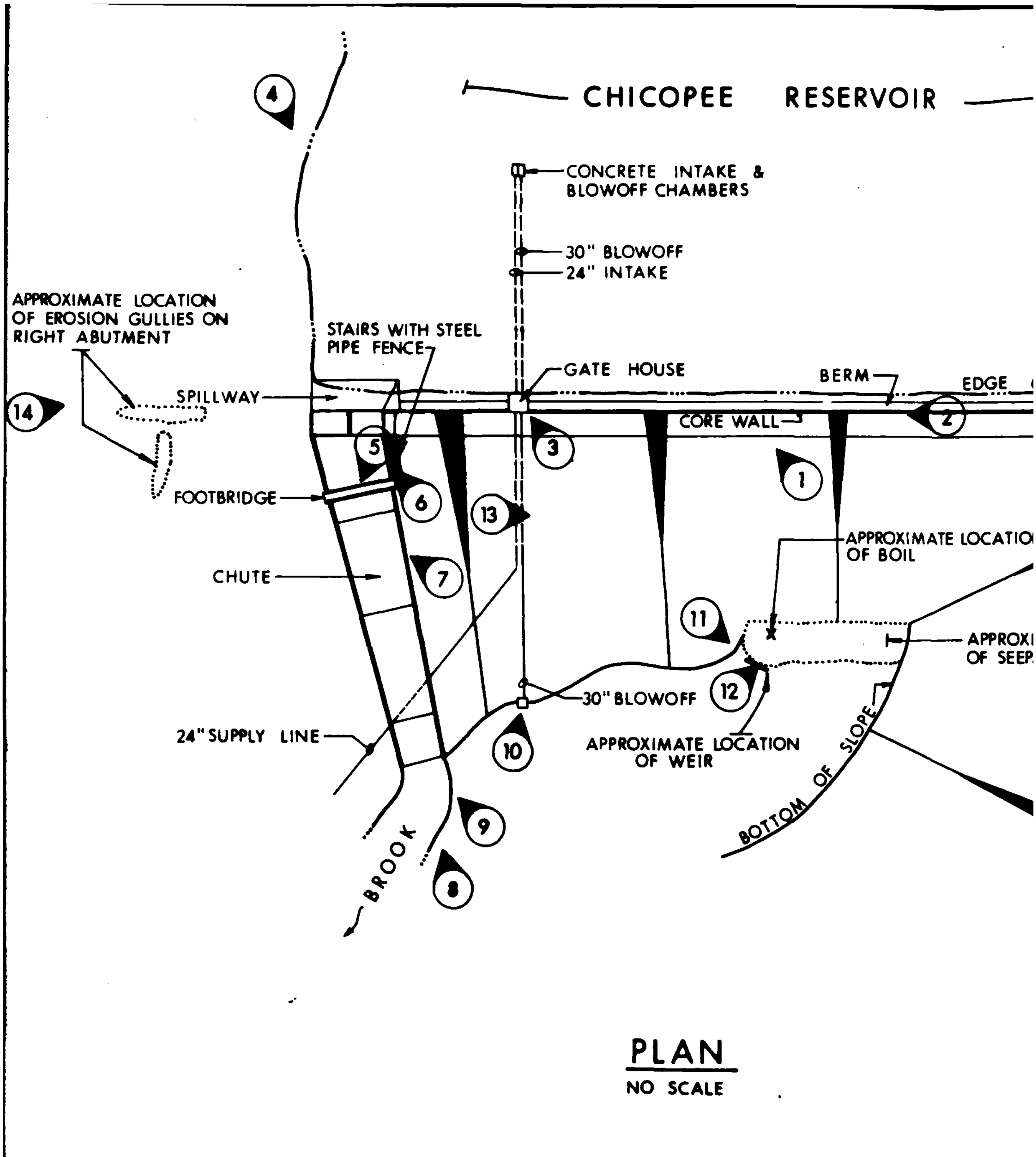
DETAILS OF FISH SCREENS



3"x 2 1/8"x 1/4" Z
Screens ① and ② are same with exception of 3/8" plate on top of Screen ① and Z bar guides
3/4" ϕ Rods
Screen ②
Flow
5'-3"
Wire Screen
Basket
Oak Block 1"x4"x3'-5 1/4"
3/4"x6" Machine Bolts
SECTION W-W THROUGH SCREENS
Scale - 1 1/2" = 1'-0"
3"x 2 1/8"x 1/4" Z
1/4"x 2" Plate
Wire Screen
SECTIONAL PLAN X-X
4'-4"
4'-0"
22"
21 1/4"
1/2"x3" U-Bolts
3/4" ϕ holes
1/4" Checkered Plate
2 Required
4'-0"
Note - For location of Fish Screens see Dwg C-7340
MORRIS KNOWLES
ENGINEERS

APPENDIX C

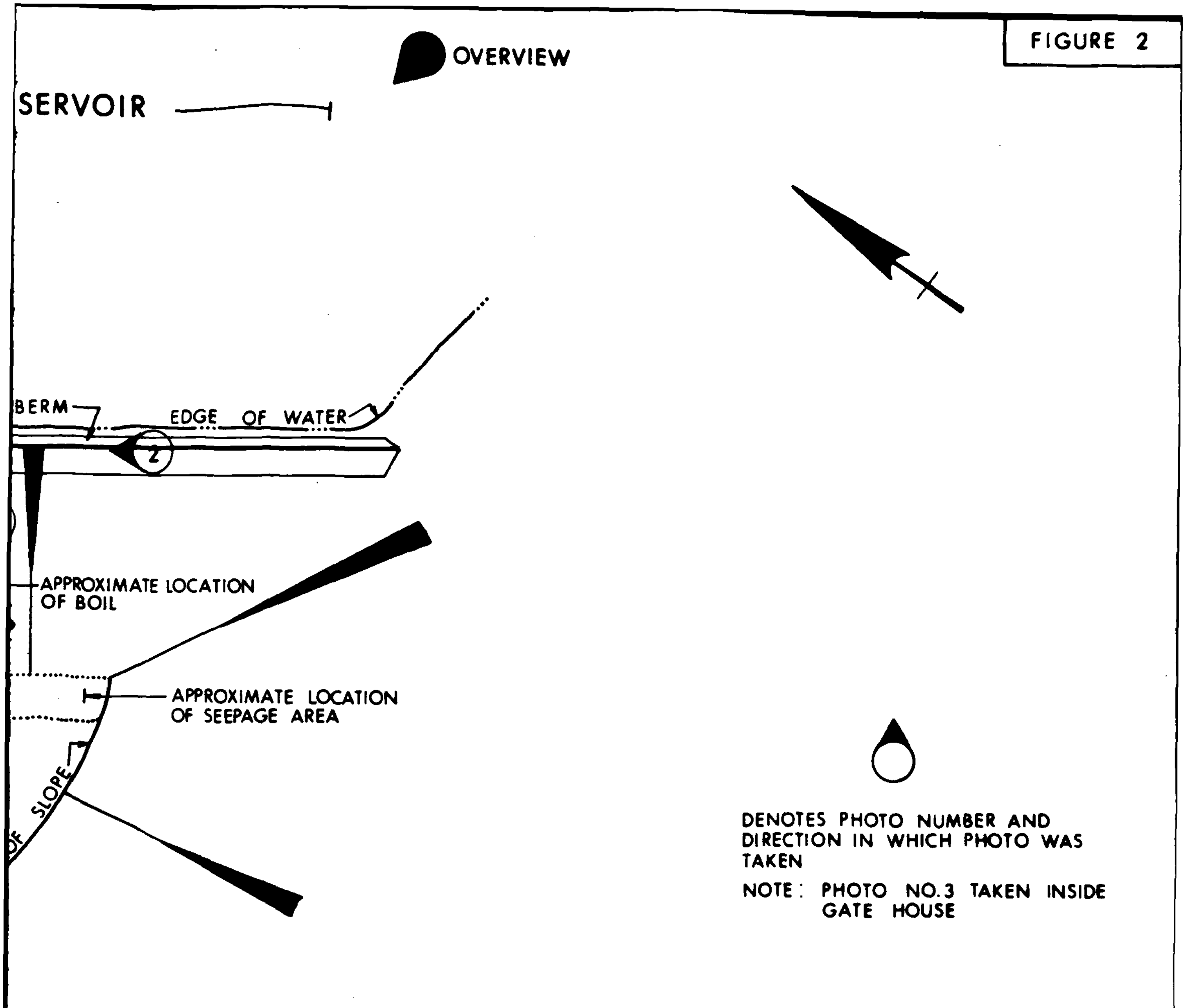
PHOTOGRAPHS



PLAN

NO SCALE

FIGURE 2



U.S. ARMY CORPS OF ENGINEERS
NEW ENGLAND DIVISION
WALTHAM, MASSACHUSETTS

NATIONAL PROGRAM OF INSPECTION
OF NON-FED. DAMS

PHOTO LOCATION PLAN
CHICOPEE RESERVOIR
CHICOPEE, MASS.

SCALE NO SCALE		DATE MARCH 1981	
DWN NP	CKD GV	APP KH	PAGE C-1

CULLINAN ENGINEERING CO., INC.
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AUBURN - BOSTON, MASSACHUSETTS

PHOTO NO. 1
VIEW OF CRIB WALL ON RIGHT RIM

PHOTO NO. 2
VIEW OF UPSTREAM FACE OF DAM
NOTE VEGETATION AT SUMMER WATER LINE

U.S. ARMY CORPS OF ENGINEERS NEW ENGLAND DIVISION WALTHAM, MASSACHUSETTS	NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS	Chicopee Reservoir Dam Cooley Brook Chicopee, MA
CULLINAN ENGINEERING CO., INC. CIVIL ENGINEERS AUBURN-BOSTON, MASSACHUSETTS		MA 00720 March 4, 1981

PHOTO NO. 3
VIEW OF GATE OPERATORS INSIDE
GATE HOUSE

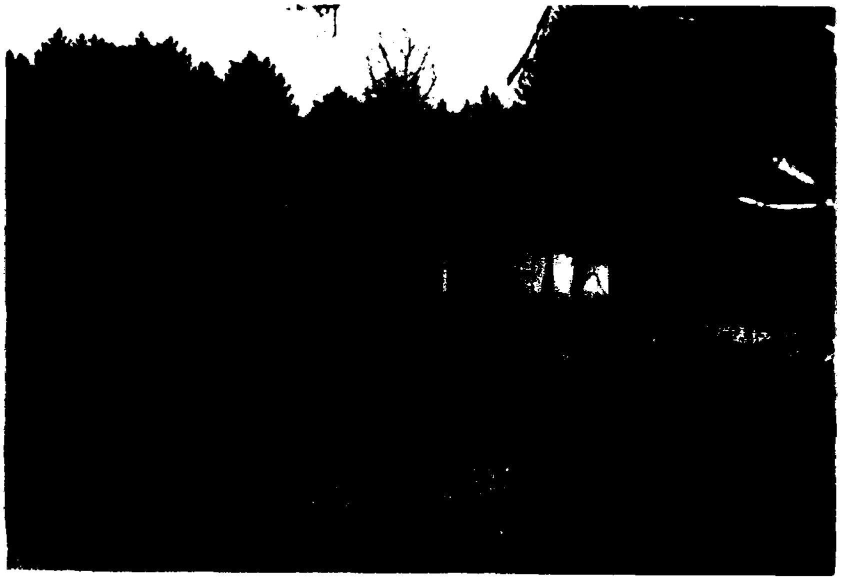


PHOTO NO. 4
VIEW OF UPSTREAM END OF SPILLWAY

U.S. ARMY CORPS OF ENGINEERS NEW ENGLAND DIVISION WALTHAM, MASSACHUSETTS	NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS	Chicopee Reservoir Dam Cooley Brook Chicopee, MA MA 00720 March 4, 1981
CULLINAN ENGINEERING CO., INC. CIVIL ENGINEERS AUBURN-BOSTON, MASSACHUSETTS		

PHOTO NO. 5
VIEW OF CHUTE LOOKING DOWNSTREAM

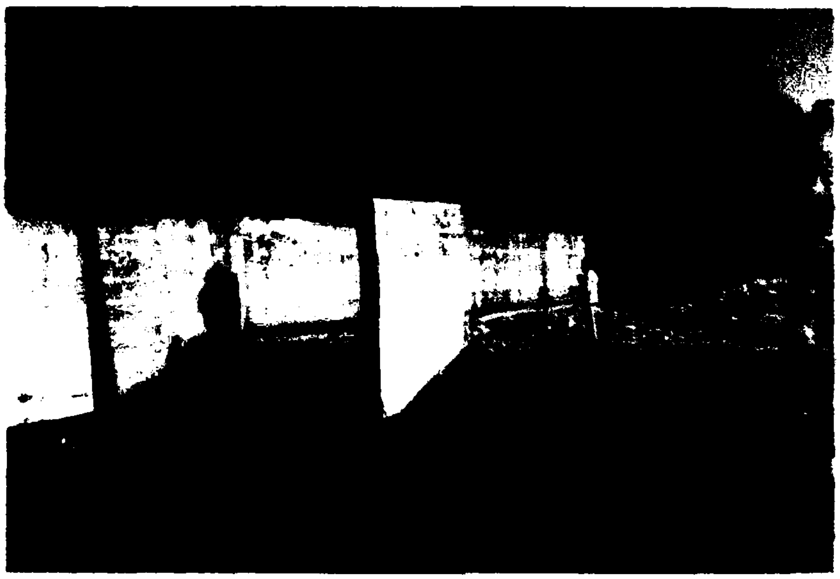


PHOTO NO. 6
VIEW OF SPILLWAY CREST LOOKING UPSTREAM

U.S. ARMY CORPS OF ENGINEERS NEW ENGLAND DIVISION WALTHAM, MASSACHUSETTS	NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS	Chicopee Reservoir Dam Cooley Brook Chicopee, MA MA 00720 March 4, 1981
CULLINAN ENGINEERING CO., INC. CIVIL ENGINEERS AUBURN-BOSTON, MASSACHUSETTS		

PHOTO NO. 7
VIEW OF INTERMEDIATE SECTION OF CHUTE
NOTE DETERIORATION OF CONCRETE

PHOTO NO. 8
VIEW OF CHUTE LOOKING UPSTREAM

U.S. ARMY CORPS OF ENGINEERS NEW ENGLAND DIVISION WALTHAM, MASSACHUSETTS	NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS	Chicopee Reservoir Dam Cooley Brook Chicopee, MA
CULLINAN ENGINEERING CO., INC. CIVIL ENGINEERS AUBURN-BOSTON, MASSACHUSETTS		MA 00720 March 4, 1981

PHOTO NO. 9
VIEW OF CONCRETE BAFFLES AT CHUTE OUTLET
NOTE DETERIORATION OF CONCRETE

PHOTO NO. 10
VIEW OF 30" BLOWOFF OUTLET AT
DOWNSTREAM RESERVOIR

U.S. ARMY CORPS OF ENGINEERS NEW ENGLAND DIVISION WALTHAM, MASSACHUSETTS	NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS	Chicopee Reservoir Dam Cooley Brook Chicopee, MA MA 00720 March 4, 1981
CULLINAN ENGINEERING CO., INC. CIVIL ENGINEERS AUBURN-BOSTON, MASSACHUSETTS		

PHOTO NO. 11
VIEW OF SEEPAGE AT DOWNSTREAM TOE OF SLOPE
NOTE BOIL IN CENTER OF PHOTO

PHOTO NO. 12
VIEW OF V-NOTCH WEIR AT DOWNSTREAM
TOE OF SLOPE

U.S. ARMY CORPS OF ENGINEERS NEW ENGLAND DIVISION WALTHAM, MASSACHUSETTS	NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS	Chicopee Reservoir Dam Cooley Brook Chicopee, MA MA 00720 March 4, 1981
CULLINAN ENGINEERING CO., INC. CIVIL ENGINEERS AUBURN-BOSTON, MASSACHUSETTS		

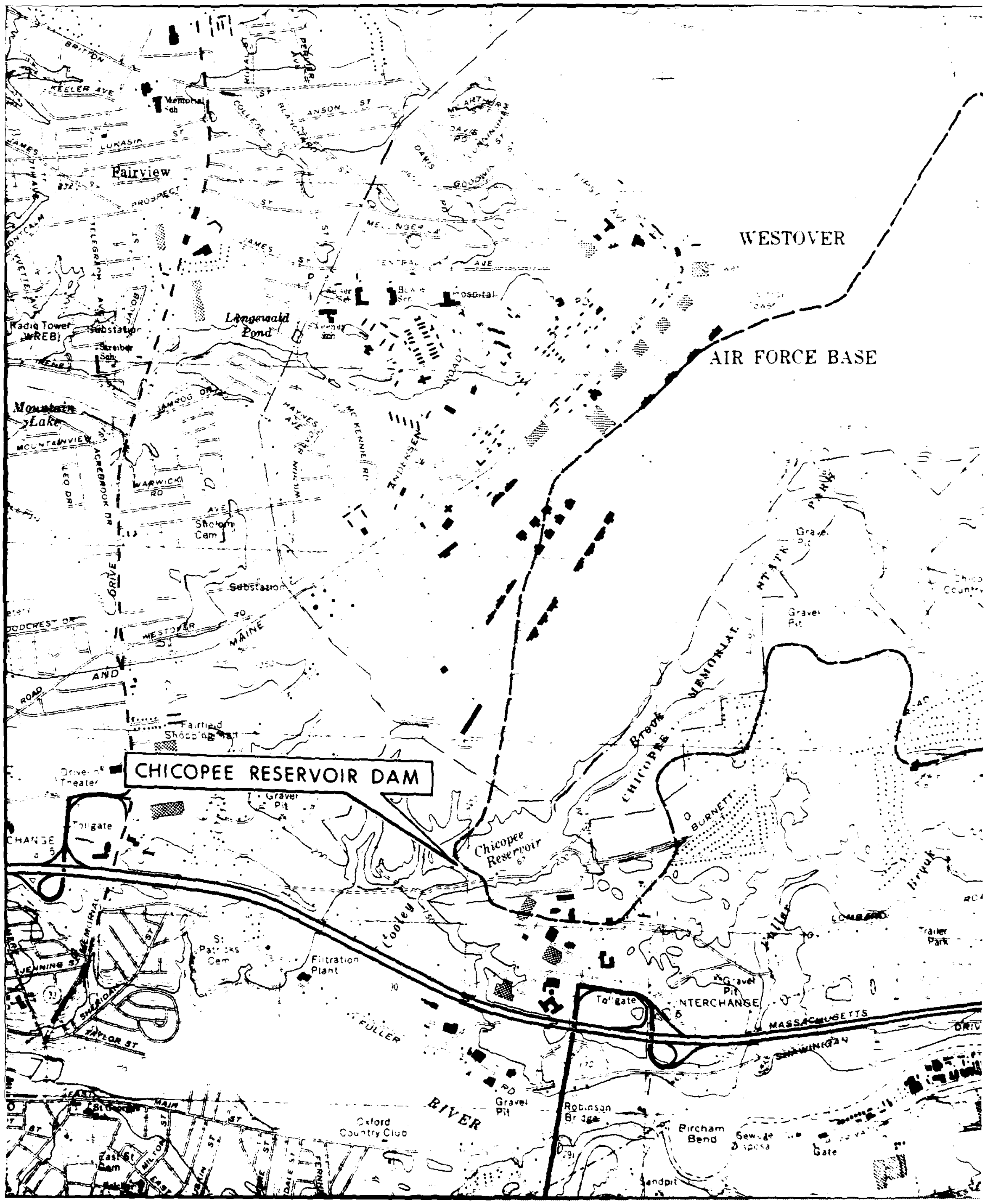
PHOTO NO. 13
VIEW OF DOWNSTREAM FACE OF DAM
NOTE BRUSH AND TREE GROWTH

PHOTO NO. 14
VIEW OF BRUSH FILLED EROSION GULLY
AT RIGHT ABUTMENT

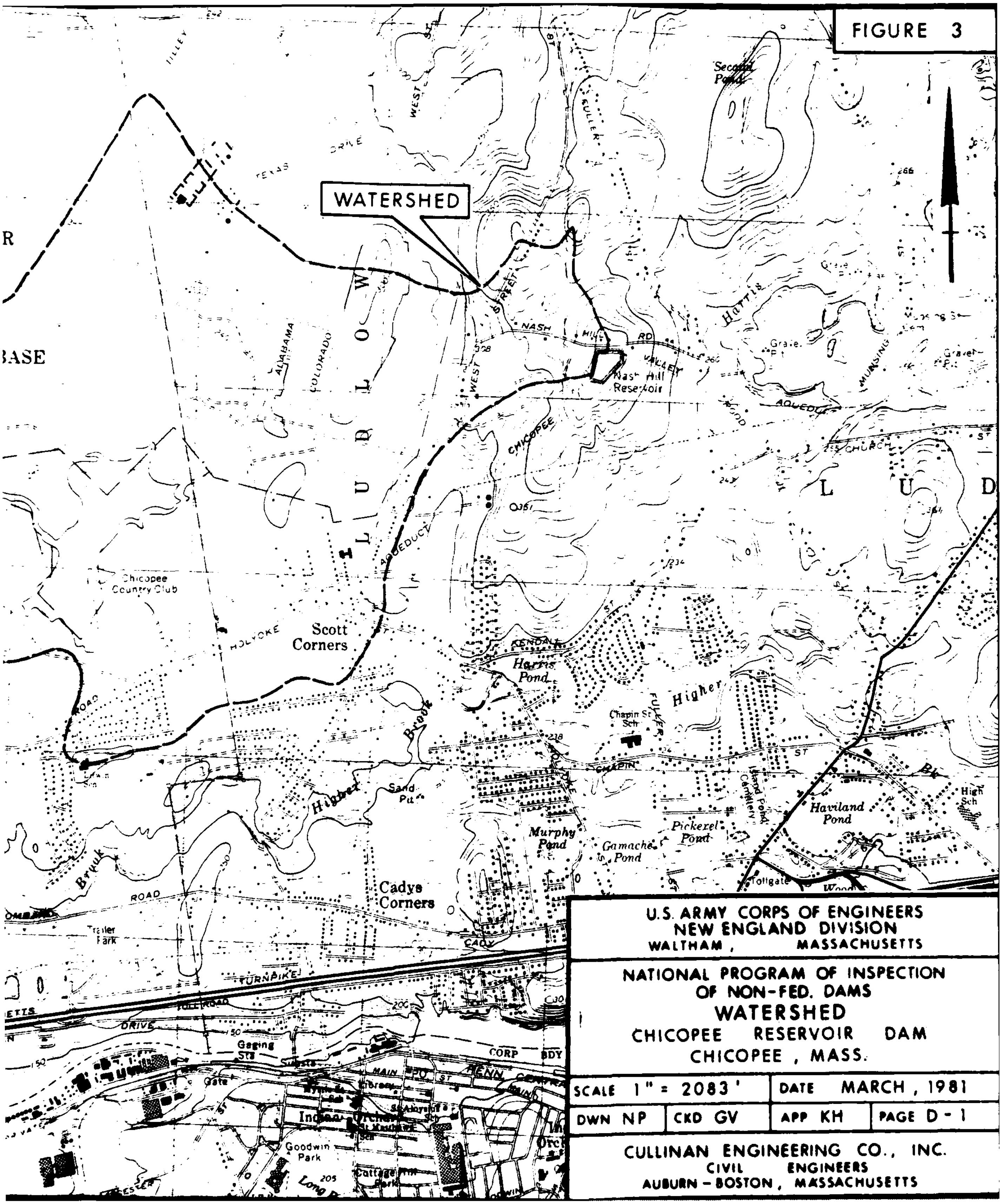
U.S. ARMY CORPS OF ENGINEERS NEW ENGLAND DIVISION WALTHAM, MASSACHUSETTS	NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS	Chicopee Reservoir Dam Cooley Brook Chicopee, MA
CULLINAN ENGINEERING CO.,INC. CIVIL ENGINEERS AUBURN-BOSTON, MASSACHUSETTS		MA 00720 March 4, 1981

APPENDIX D

HYDROLOGIC AND HYDRAULIC COMPUTATIONS



CHICOPEE RESERVOIR DAM
WESTOVER
AIR FORCE BASE
Fairview
Lengewald Pond
Mountain Lake
Chicopee Reservoir
Chicopee Brook
Cooley Brook
Fuller Brook
RIVER
Filtration Plant
Oxford Country Club
Robinson Bridge
Bircham Bend
Tollgate
INTERCHANGE
MASSACHUSETTS
Hospital
Substation
Drive-in Theater
Gravel Pit
Trailer Park
Sandpit



U.S. ARMY CORPS OF ENGINEERS
NEW ENGLAND DIVISION
WALTHAM, MASSACHUSETTS

NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS

WATERSHED

CHICOPEE RESERVOIR DAM
CHICOPEE, MASS.

SCALE 1" = 2083'		DATE MARCH, 1981	
DWN NP	CKD GV	APP KH	PAGE D-1

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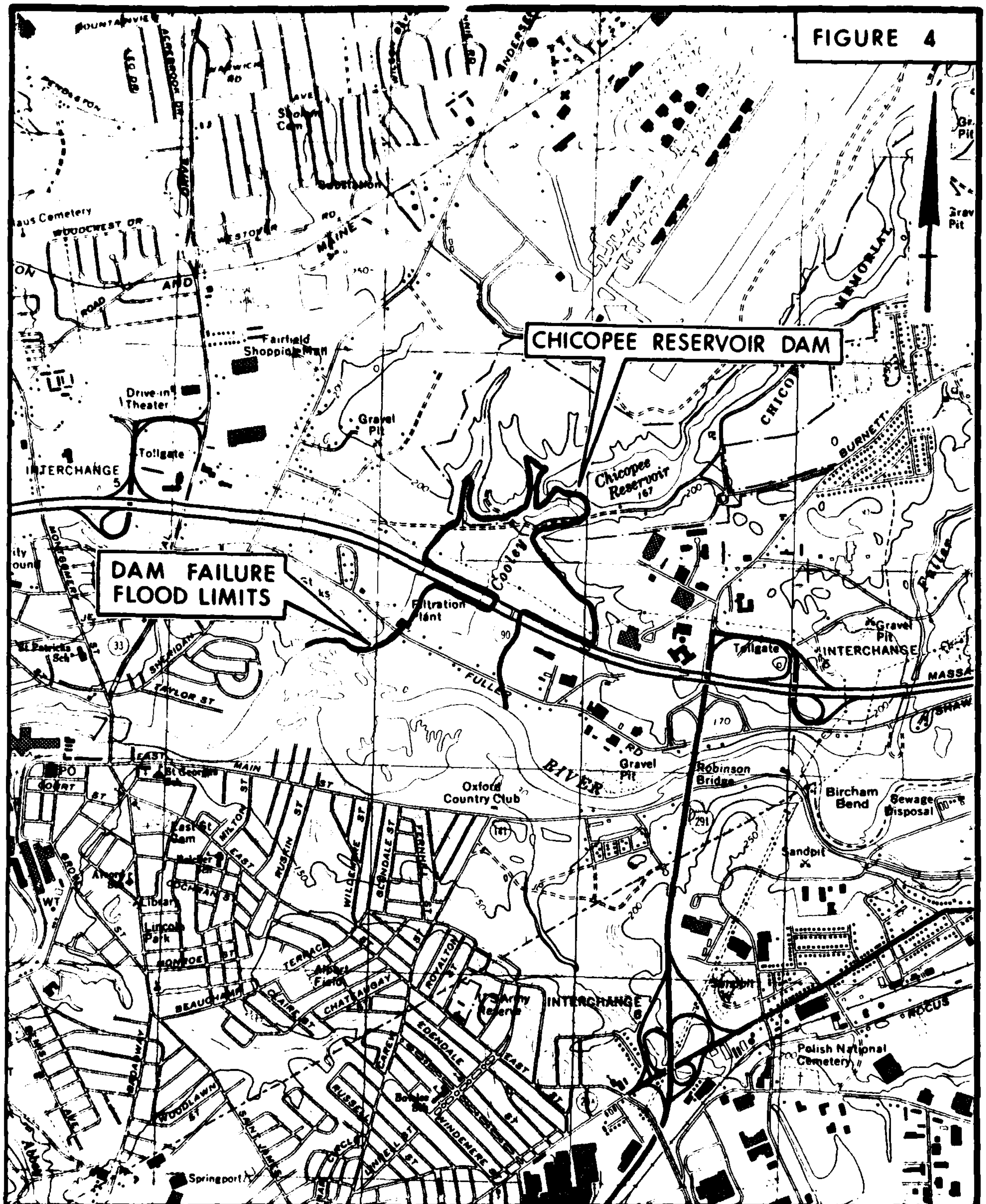

DAM FAILURE FLOOD LIMITS
CHICOPEE RESERVOIR DAM
CHICOPEE , MASS.
1" = 2083'

CULLINAN ENGINEERING CO., INC SPRINGFIELD NORTH , MASS QUADRANGLE 1979

CLIENT / PROJECT U.S. Army COE / Non-Federal Dams DATE 5/20/81 JOB NO. ______
SUBJECT: Chicopee Reservoir Analysis and Evaluation BY GMV CHKD. BY JDP SHEET 1 OF 16

I. Classification:

Size: Storage (max.) = 630 Ac.Ft. ∴ Small
height (struct.) = 48 ft. ∴ Intermediate
∴ overall size classification is Intermediate

Hazard Potential: Analysis indicates that if failure were to occur with the water surface at the top of the dam, the resulting outflow could cause severe damage to overhead electric lines, the Mass. Turnpike, a water filtration plant, and as many as eight homes. With an appreciable economic loss and the potential loss of more than a few lives, therefore, hazard potential is considered High.

II. Spillway Design Flood:

With a high hazard potential and an intermediate size dam, the COE "Recommended Guidelines for Safety Inspection of Dams" indicates that a test flood equal to the Probable Maximum Flood should be used.
∴ Determine SDF using the full PMF

III. Inflow Hydrograph:

Tributary Area = 2513 Acres = 3.93 Sq. Miles
Terrain is Rolling (from inspection of USGS Springfield North Quad)
∴ From COE "Maximum Probable Flood Peak Flow Rates"
PMF (csm) = 1925 CSM
∴ SDF = PMF = 1925 CSM x 3.93 SM = 7565 cfs

Time to peak $t_p = \frac{484AQ}{q_p}$ Where: A = drainage area = 3.93 Sq. Mi.
Q = total runoff = 19.0 In. (PMP)
q_p = peak flow = 7565 cfs

$\therefore t_p = \frac{484 \times 3.93 \times 19.0}{7565} = 4.8$ hrs. (287 min.)

Time base for hydrograph $t_b = 2.67 t_p$

$\therefore t_b = 2.67 \times 4.8 = 12.8$ hrs. (765 min.)

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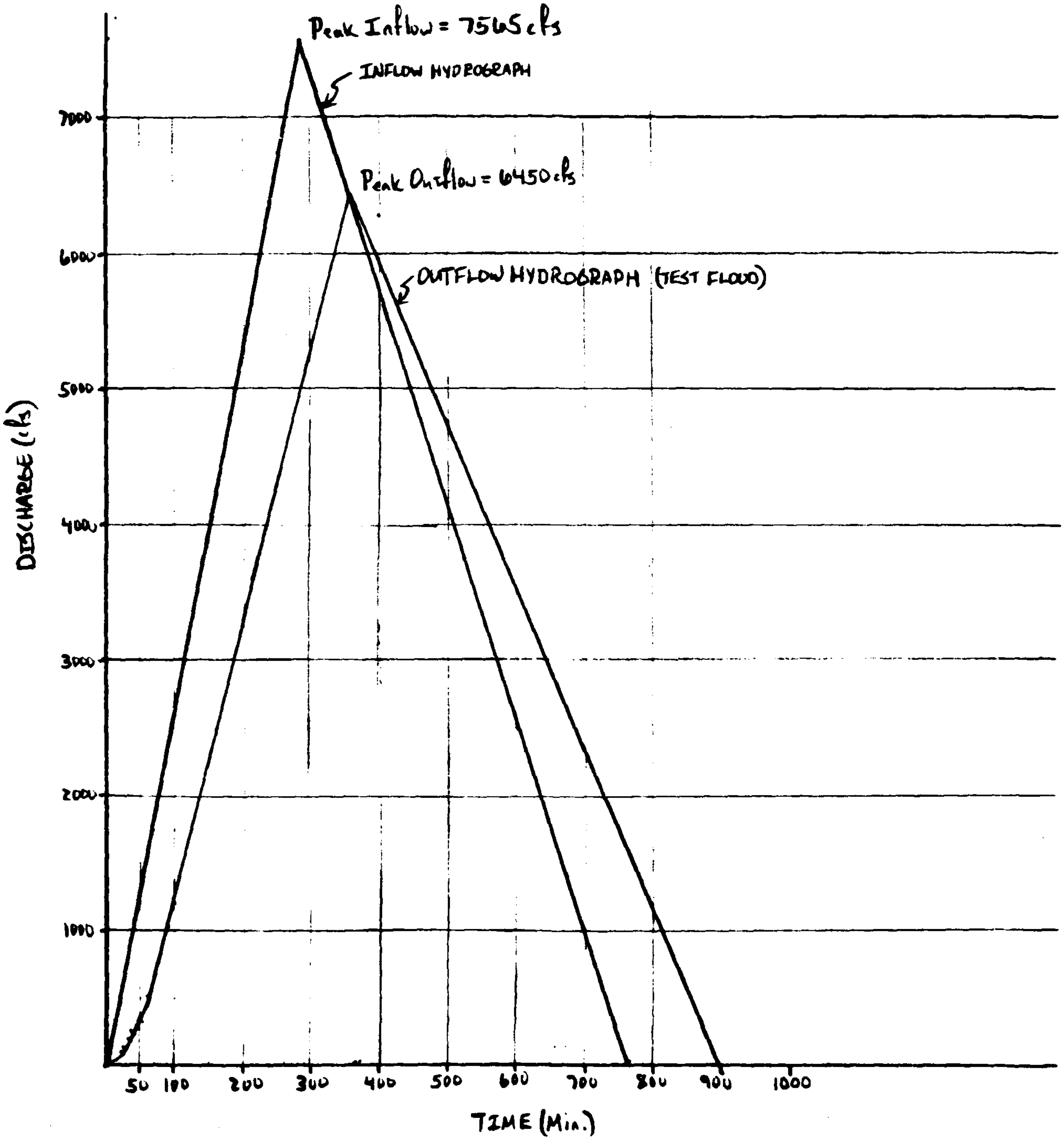
CLIENT / PROJECT U.S. Army COE / Non-Federal Dams DATE 5/20/81 JOB NO

SUBJECT Chicopee Reservoir Analysis and Evaluation BY GMV CHKD BY JDP SHEET 2 OF 16



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CIVIL ENGINEERS — LAND SURVEYORS

IV. Flood Routing:

Stage Discharge Data: information used to develop the stage discharge data is from the 1926 Contract Drawings and field observations.

Outlets – 30" Blowoff @ El. 129.7
(A) 24" Inlet @ El. 132.0
(B) 24" Inlet @ El. 153.0
VALUES GIVEN IN TABLE BUT NOT PLOTTED ON STAGE DISCHARGE CURVE

44' of Ogee Crest Spillway @ El. 168.0
500' of Embankment @ Top of Wall El. 176.7
NOTE: Normal Water (observed level) @ El. 168.4

*Pipe discharge will be given by the outlet control nomograph designated as Chart 9 on pg. 5-32 of HEC#5 assuming $K_e = 0.5$ and L = actual length (from plans) of all pipes.

Discharge over the ogee crest spillway will be given by the equation $Q = 3.5 L H^{3/2}$

Discharge over the embankment given by the equation for a broad crested weir $Q = 3.03 L H^{3/2}$

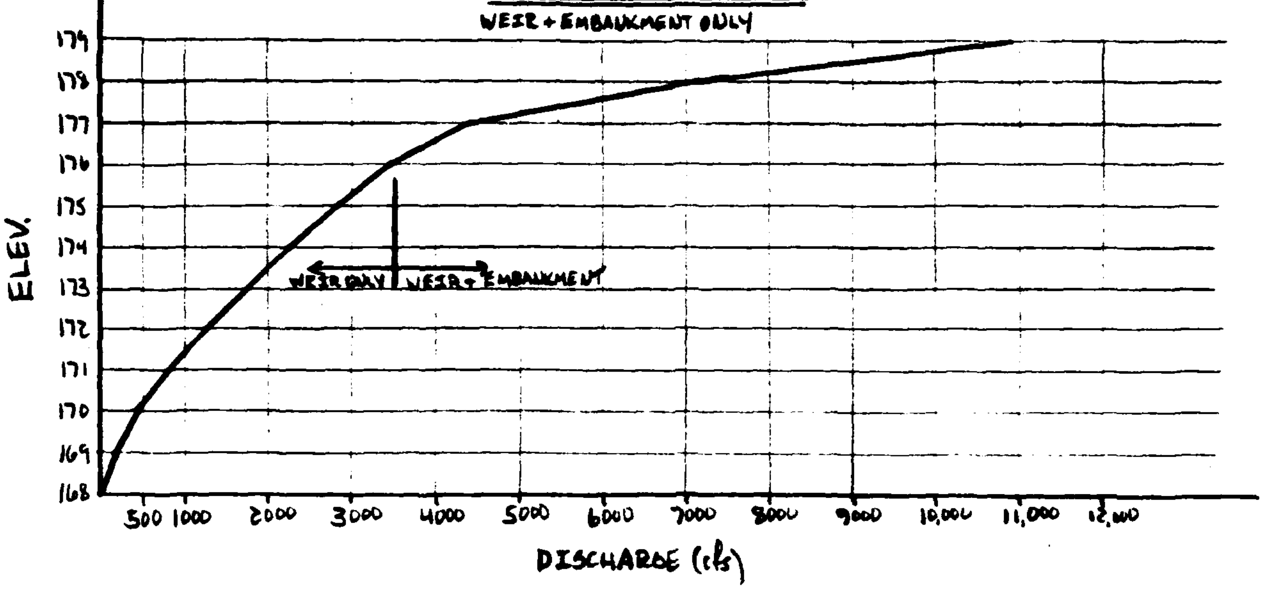
ELEV.	$H_{30"}$	$H_{24"A}$	$H_{24"B}$	H_{WEIR}	$H_{EMB.}$	*$Q_{30"}$	*$Q_{24"A}$	*$Q_{24"B}$	Q_{WEIR}	$Q_{EMB.}$	Q_{TOTAL}
169	39.3'	37'	16'	1.0'	–	105 cfs	79 cfs	69 cfs	154 cfs	–	407 cfs
170	40.3'	38'	17'	2'	–	106	80	71	436	–	693
171	41.3'	39'	18'	3'	–	108	81	73	800	–	1062
172	42.3'	40'	19'	4'	–	109	82	75	1232	–	1498
173	43.3'	41'	20'	5'	–	110	83	77	1722	–	1992
174	44.3'	42'	21'	6'	–	112	84	79	2263	–	2538
175	45.3'	43'	22'	7'	–	113	85	80	2852	–	3130
176	46.3'	44'	23'	8'	–	114	86	82	3485	–	3767
177	47.3'	45'	24'	9'	0.3'	115	87	84	4158	249 cfs	4693
178	48.3'	46'	25'	10'	1.3'	117	88	86	4870	2246	7407
179	49.3'	47'	26'	11'	2.3'	–	–	–	5618	5285	

*For H > 26' $H = \left[1.5 + \frac{29n^2 L}{R^{1.33}}\right] \frac{V^2}{2g}$ to be used

CLIENT / PROJECT U.S. Army COE / Non-Federal Dams DATE 5/20/81 JOB NO. ______
SUBJECT: Chicopee Reservoir Analysis and Evaluation BY GMV CHKD BY JDP SHEET 4 OF 16

IV. Flood Routing: cont.

STAGE DISCHARGE CURVE



Stage Storage Data — to develop the stage storage curve, the areas at normal water (assumed to be 168.4) and at elev 170 & 180 will be determined from the USGS Springfield North Quad and averaged to compute the volume. For purposes of this analysis, the stage storage relationship will be assumed to be linear.

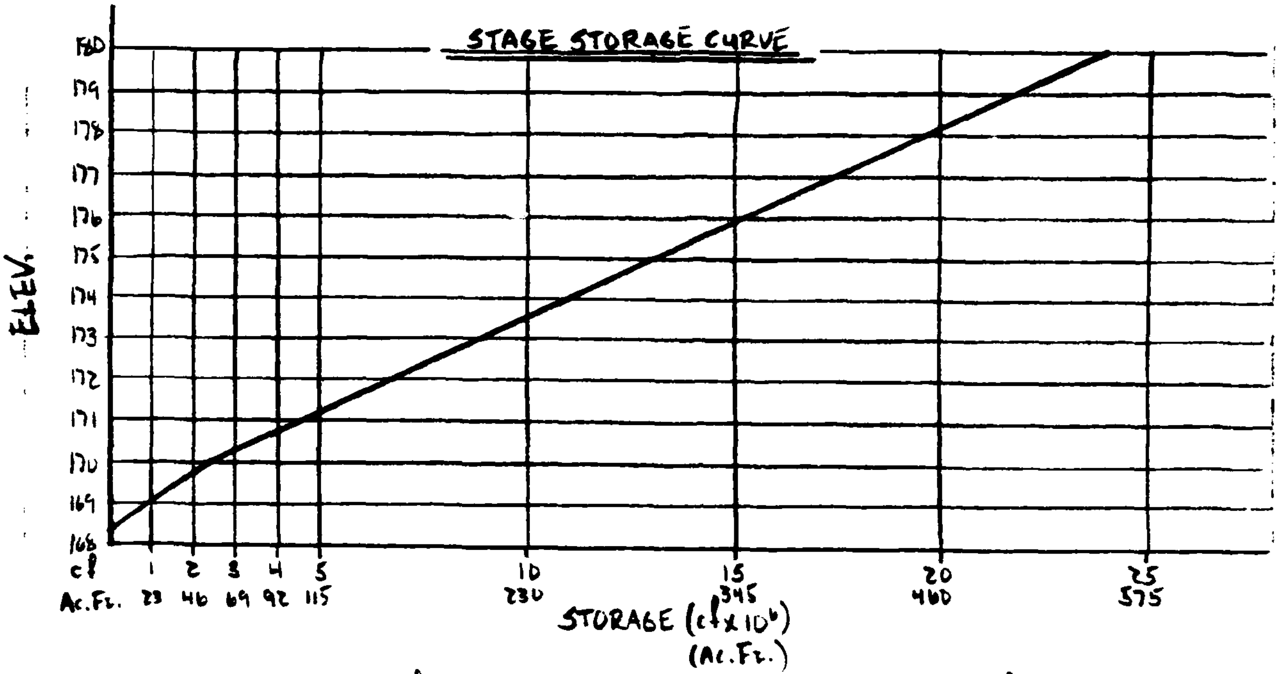
Area at Elevation 167 (Norm. Water 168.4) = 1,063,400 sf
Area at Elevation 170 = 1,649,300 sf
Area at Elevation 180 = 2,669,300 sf

∴ Area at Elev. 168.4 = 1,336,820 sf
∴ Volume @ Elev. 170 = [(1,336,820 + 1,649,300) ÷ 2] x 1.6 = 2,388,900 sf
Volume @ Elev. 180 = [(1,649,300 + 2,669,300) ÷ 2] x 10 + Vol. @ 170
= 23,981,900

CLIENT / PROJECT U.S. Army COE / Non-Federal Dams DATE 2/9/81 JOB NO.

SUBJECT Chicopee Reservoir Analysis and Evaluation BY GMV CHKD. BY JDP SHEET 5 OF 16

IV. Flood Routing: cont.



Routing of the SDF will be performed using the program for Muskingum Method Hydrograph Routing as contained in the text entitled "Hydrologic and Hydraulic Computations on Small Programmable Calculators" by Thomas E. Croley II.

Δt = 10 min.

X = 0 (reservoir routing)

K = approximated as slope of line obtained by plotting storage vs. outflow

Storage @ el. 176 = 15,200,000 cf ±

outflow @ el. 176 = 3485 cfs

$$\therefore K = \frac{15.2 \times 10^6 \text{ cf}}{3485 \text{ cfs}} \times \frac{1 \text{ min.}}{60 \text{ sec.}} = 72.7 \text{ min.}$$

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CLIENT / PROJECT U.S. Army COE / Non-Federal Dams DATE 5/20/81 JOB NO.

SUBJECT: Chicopee Reservoir Analysis and Evaluation BY GMV CHKD BY JDP SHEET 6 OF 16

IV. Flood Routing: cont.

TIME	INFLOW	OUTFLOW	TIME	INFLOW	OUTFLOW
0 min.	0 cfs	0 cfs	250	6590	4734
10	264	*12	260	6853	4989
20	527	61	270	7117	5245
30	791	138	280	7380	5503
40	1054	239	290	7518	5753
50	1318	361	300	7359	5970
60	1582	501	310	7201	6139
70	1845	657	320	7043	6265
80	2109	827	330	6884	6355
90	2372	1009	340	6726	6413
100	2635	1201	350	6568	6443
110	2899	1403	360	6410	6449
120	3163	1612	370	6251	6434
130	3427	1829	380	6093	6400
140	3690	2052	390	5935	6350
150	3954	2279	400	5777	6287
160	4217	2512			
170	4481	2748			
[illegible]	4745	2988			
[illegible]	5008	3231			
200	5272	3477			
210	5535	3725			
220	5799	3975			
230	6063	4227			
240	6326	4480			

Peak Outflow = 6450 cfs

∴ W.S. Elevation = 177.8

∴ Dam overtopped by 1.1 ft.

*Starting Value – $I_1 + I_2 + \cancel{\frac{2S_1}{\Delta t} - O_1}^{0} = \frac{2S_2}{\Delta t} + O_2 \therefore I_1 + I_2 = 264\text{ cfs} = \frac{2S_2}{\Delta t} + O_2$

@ 01.169 $\frac{2S_2}{\Delta t} + O_2 = 3321\text{ cfs}$, $O_2 = 154\text{ cfs}$

$\frac{264}{3321} = \frac{O}{154} \Rightarrow O = 12\text{ cfs}$

CLIENT / PROJECT U.S. Army COE / Non-Federal Dams DATE 5/20/81 JOB NO.
SUBJECT Chicopee Reservoir Analysis and Evaluation BY GMV CHKD. BY JDP SHEET 7 OF 16

V. Dam Failure Outflow:

Assume 40% of mid-height length breaches with the water surface at the top of the dam.

$\therefore W_b = 0.40 \times 340\text{ ft.} = 136\text{ ft.}$

Top of dam @ el. 176.7

Downstream @ el. 130±

$\therefore Y_o = 176.7 - 130 = 46.7\text{ ft.}$

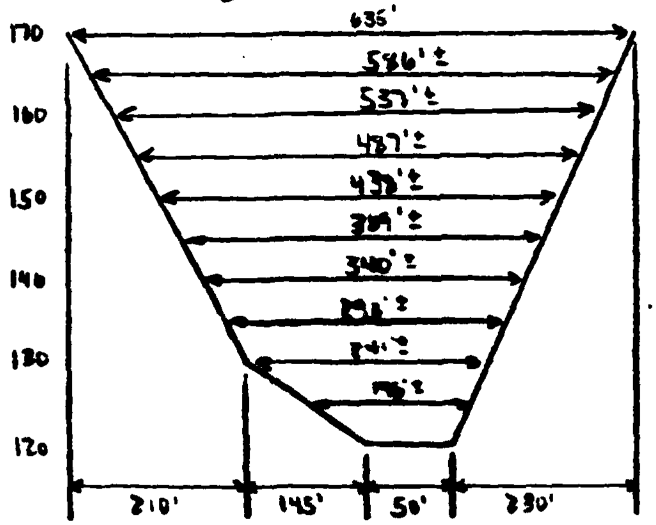
Peak Failure Outflow $Q_{p_1} = \frac{8}{27} W_b \sqrt{g}\, Y_o^{3/2}$

$\therefore Q_{p_1} = \frac{8}{27} \times 136 \times \sqrt{32.2} \times (46.7)^{3/2} = 72{,}974$ (+3956 spillway) = 76,930 cfs

Total Storage S = 630 Ac.Ft. (from inventory sheet) SAY 77,000 cfs

VI. Downstream Failure Analysis:

(1) Section 1000'± downstream of dam (from USGS) 1" = 200' Horiz. 1" = 20' Vert.



L = 1000 ft.±

S = 0.005 ft/ft ±

n = 0.045

ELEV.	AREA (A)	WP	HYDRAULIC RADIUS (R)	$Q = \frac{1.486}{n} A R^{2/3} S^{1/2}$
125	490 sf	146 ft.	490 ÷ 146 = 3.36 ft.	2567 cfs
130	1458 sf	241 ft.	1458 ÷ 241 = 6.05 ft.	11,304 cfs
135	2785 sf	290 ft.	2785 ÷ 290 = 9.60 ft.	29,374 cfs
140	4360 sf	340 ft.	4360 ÷ 340 = 12.8 ft.	55,708 cfs
145	6183 sf	389 ft.	6183 ÷ 389 = 15.9 ft.	91,290 cfs

II. Downstream Dam Failure Analysis : cont.

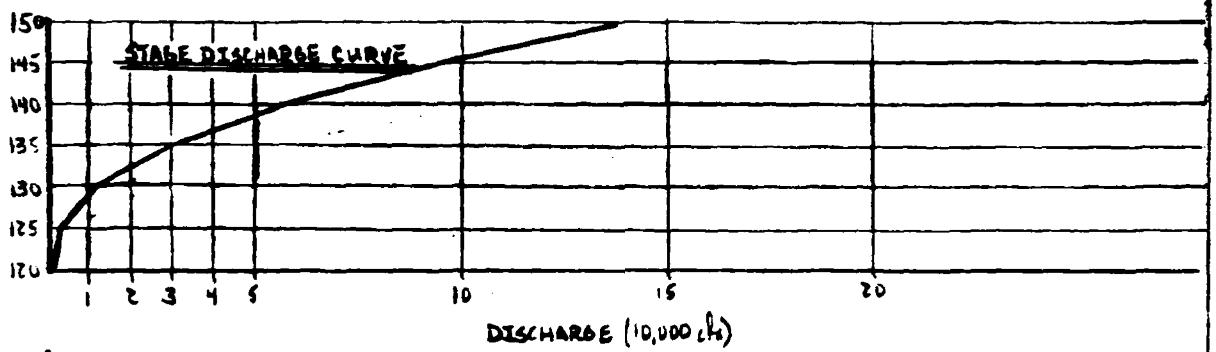
Area at Elev. 130 = 86,800 sf
Area at Elev. 140 = 217,000 sf
Area at Elev. 150 = 434,000 sf
Area at Elev. 160 = 477,500 sf

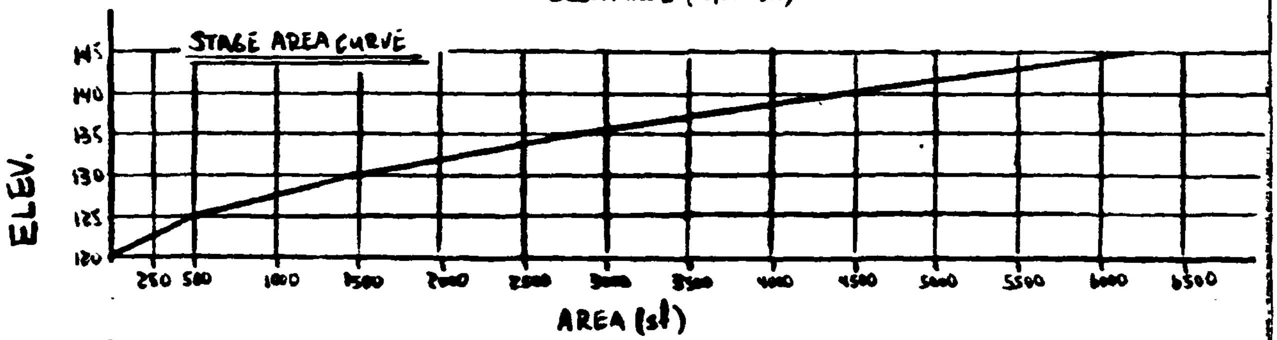
$\therefore$ Vol. @ 130 $= \left[(86{,}800+0) \div 2\right] \times 10 \times \frac{1}{43560} = 10.0$ Ac. Ft.

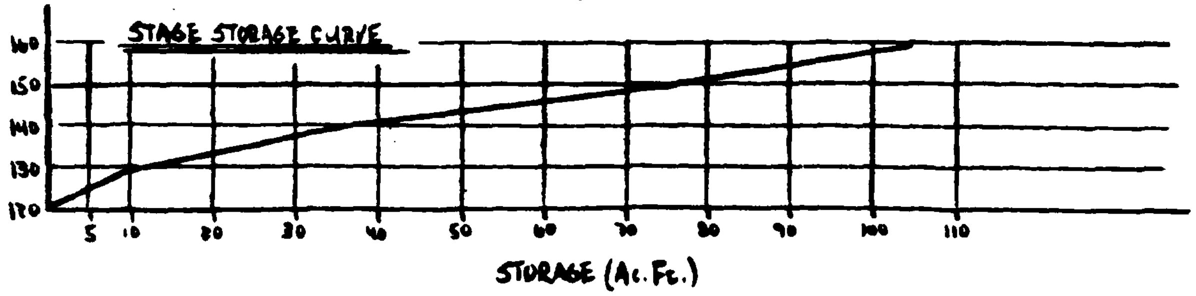
Vol. @ 140 $= \left[(217{,}000+86{,}800) \div 2\right] \times 10 \times \frac{1}{43560} = 34.9$ Ac. Ft.

Vol. @ 150 $= \left[(434{,}000+217{,}000) \div 2\right] \times 10 \times \frac{1}{43560} = 74.7$ Ac. Ft.

Vol. @ 160 $= \left[(477{,}500+434{,}000) \div 2\right] \times 10 \times \frac{1}{43560} = 104.6$ Ac. Ft.







CLIENT / PROJECT U.S. Army COE / Non-Federal Dams DATE 5/20/81 JOB NO
SUBJECT Chicopee Reservoir Analysis and Evaluation BY GMV CHKD BY JDP SHEET 9 OF 16

VI. Downstream Dam Failure Analysis: cont.

@ $Q_{p_1} = 77{,}000$ cfs
$H_1 = 23.0$ ft. ± (El. 143.0±)
$V_1 = 46.8$ Ac.Ft. $< \frac{1}{2}S$ ∴ Reach is OK

$$\therefore Q_{p_2}(\text{TRIAL}) = Q_{p_1}\left(1-\frac{V_1}{S}\right) = 77{,}000\left(1-\frac{46.8}{630}\right) = 71{,}275 \text{ cfs}$$

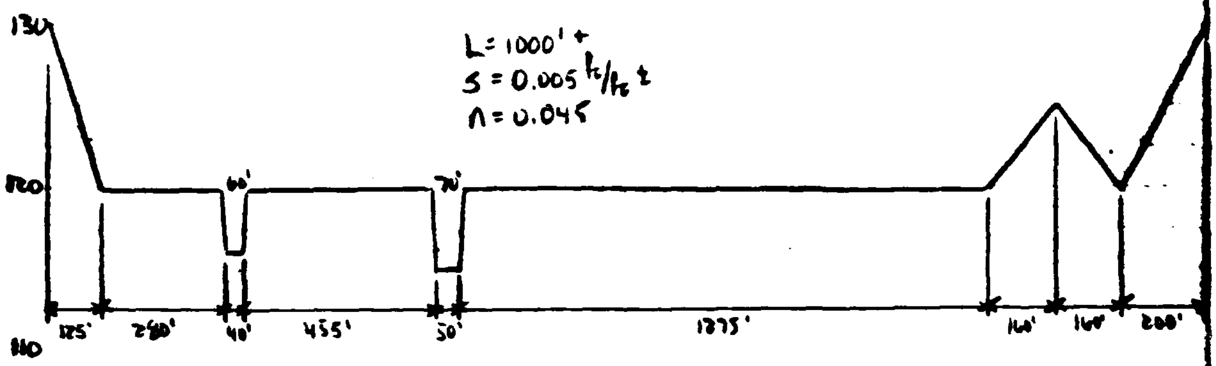
@ $Q_{p_2} = 71{,}275$ cfs
$H_2 = 22.2$ ft. ± (El. 142.2±)
$V_2 = 43.7$ Ac.Ft.

$$\therefore V_{AVG} = \frac{V_1+V_2}{2} = \frac{46.8+43.7}{2} = 45.3 \text{ Ac.Ft.}$$

$$\therefore Q_{p_2} = Q_{p_1}\left(1-\frac{V_{AVG}}{S}\right) = 77{,}000\left(1-\frac{45.3}{630}\right) = 71{,}470 \text{ cfs}$$

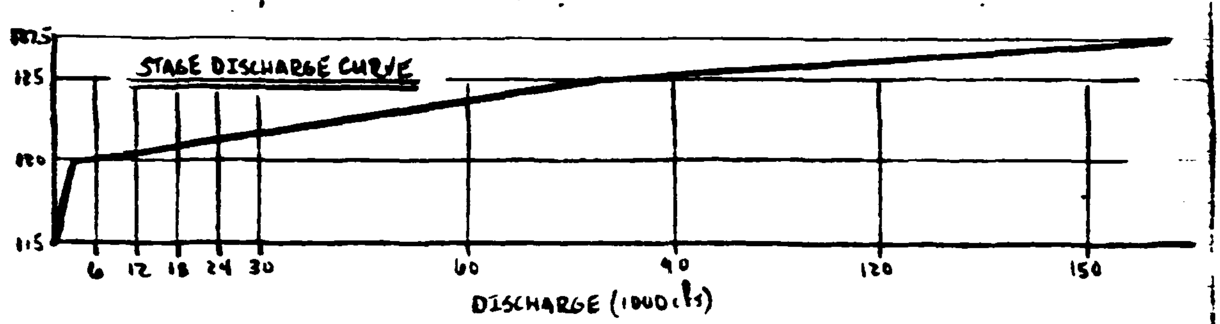
SAY 71,500 cfs H = 22.2 ft.± A = 5170 sf

(2) Section 2000'± downstream of dam (from USGS) 1" = 400' Horiz, 1" = 10' Vert.

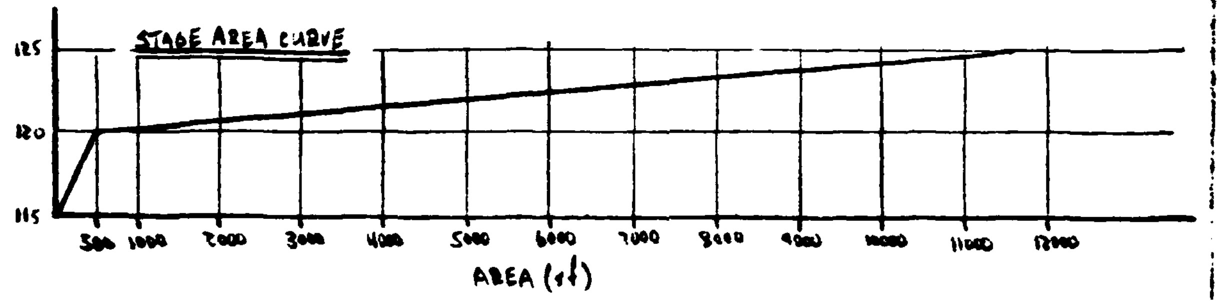


ELEV.	AREA (A)	WP	HYDRAULIC RADIUS (R)	$Q = \frac{1.486}{n} A R^{2/3} S^{1/2}$
120	500 sf	134 ft.	500 ÷ 134 = 3.73 ft.	2808 cfs
125	11,556 sf	2367 ft.	11,556 ÷ 2367 = 4.88 ft.	77,632 cfs
130	25,525 sf	2789 ft.	25,525 ÷ 2789 = 9.15 ft.	260,738 cfs

VI. Downstream Dam Failure Analysis: cont.



ELEV.



@ $Q_{P_2} = 71{,}500$ cfs

$H_1 = 9.6$ ft. ± (El. 124.6 ±)

$V_1 = \frac{5170 + 10{,}650}{2} \times 1000 \times \frac{1}{43{,}560} = 182$ Ac. Ft. $< \frac{1}{2}S$ ∴ Reach is OK

$\therefore Q_{P_3}(\text{TRIAL}) = Q_{P_2}\left(1 - \frac{V_1}{S}\right) = 71{,}500\left(1 - \frac{182}{630}\right) = 50{,}891$ cfs

@ $Q_{P_3} = 50{,}891$ cfs

$H_2 = 8.2$ ft. ± (El. 123.2 ±)

$V_2 = \frac{5170 + 7600}{2} \times 1000 \times \frac{1}{43{,}560} = 147$ Ac. Ft.

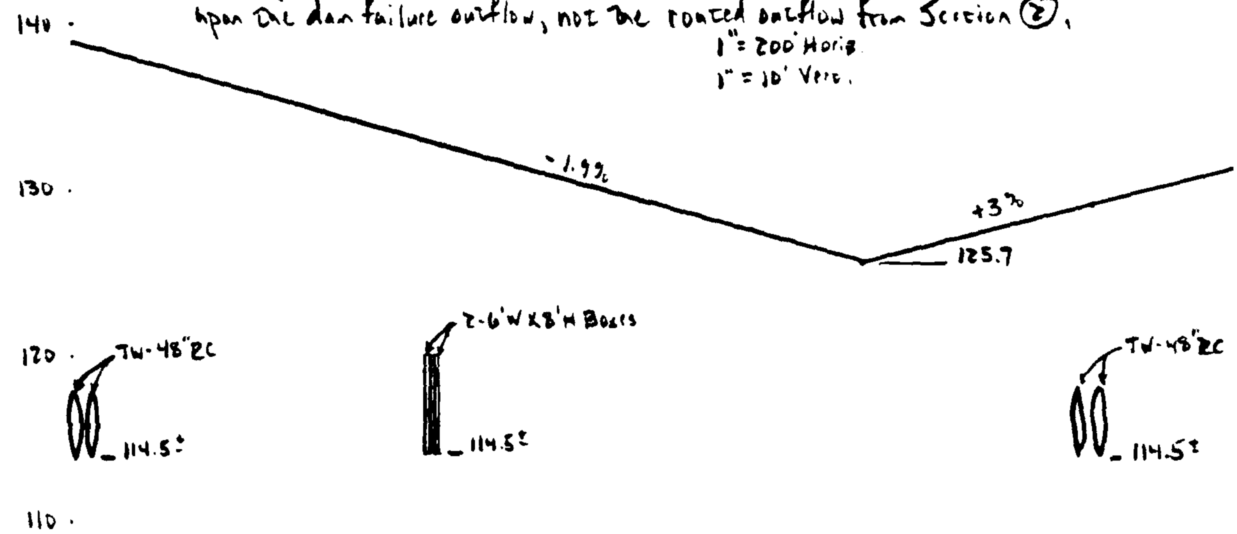
$\therefore V_{AVG} = \frac{V_1 + V_2}{2} = \frac{182 + 147}{2} = 164.5$ Ac. Ft.

$\therefore Q_{P_3} = Q_{P_2}\left(1 - \frac{V_{AVG}}{S}\right) = 71{,}500\left(1 - \frac{164.5}{630}\right) = 52{,}830$ cfs

SAY 52,900 cfs H = 8.3 ft. ± A = 7900 sf ±

VI. Downstream Dam Failure Analysis: cont.

Immediately downstream of Section ②, the peak failure outflow encounters the Mass. Turnpike. Flow under the roadway is controlled by two 6'x8' box culverts at elev. 114.5± and a total of 4-48" RC culverts at elev. 114.5±. The elevation of the low point of the roadway is approximately 125.70 with a 1.9% slope in one direction and a 3.0% slope in the other direction. Discharge through the culverts will be determined using the charts contained in HEC#5 assuming inlet control. Flow over the roadway will be determined assuming that the highway functions as a broad crested weir with discharge given by the equation $Q = 3.03LH^{3/2}$ where H is the average depth over the weir. Since the roadway will function as a control section, inflow to the culverts will be based upon the dam failure outflow, not the routed outflow from Section ②.



ELEV.	$HW_{CULV.}$	(AVG.) $HW_{ROADWAY}$	$Q_{BOX CULV.}$	$Q_{4-48"}$	$Q_{ROADWAY}$	Q_{TOTAL}
116	1.5 ft.	-	± 84 cfs	± 76 cfs	-	160 cfs
118	3.5	-	228	252	-	480
120	5.5	-	450	460	-	910
122	7.5	-	720	600	-	1320
124	9.5	-	940	720	-	1660
126	11.5	0.15 ft.	1200	840	5 cfs (L=25')	2045
128	13.5	1.15	1380	920	750 (L=200')	3050
130	15.5	2.15	1560	1020	3530 (L=370')	6110

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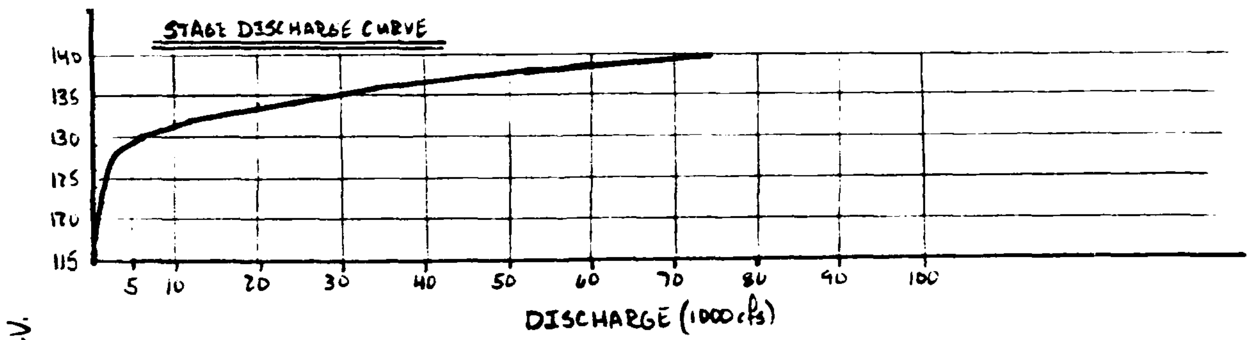
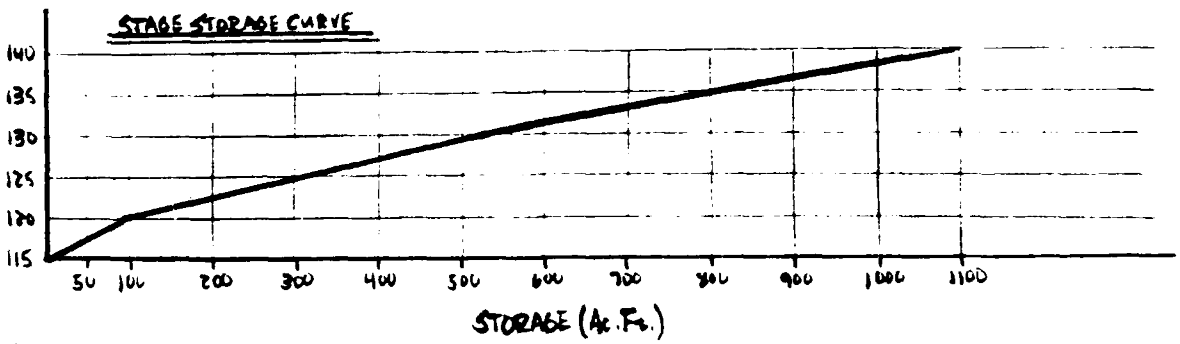
VI. Downstream Dam Failure Analysis: cont.

ELEV.	HW culv.	(AVG.) HW roadway	Q twin culv.	Q 4×4'	Q roadway	Q total
132	17.5	3.15	1680	1120	9150 (L=540')	11,950
134	19.5	4.15	1860	1200	18,440 (L=720')	21,500
136	21.5	5.15	1980	1240	31,160 (L=880')	34,380
138	23.5	6.15	2040	1280	48,980 (L=1060')	52,300
140	25.5	7.15	2160	1320	71,250 (L=1236')	74,730

Stage storage data upstream of the Mass. Turnpike is developed using the topography shown on the USGS Springfield North Quad.

Area at Elev. 114.5 = 0 (assumed)
Area at Elev. 120 = 33.9 Acres ∴ Vol. @ 120 = 93 Ac. Ft.
Area at Elev. 130 = 52.8 Acres ∴ Vol. @ 130 = 527 Ac. Ft.
Area at Elev. 140 = 60.8 Acres ∴ Vol. @ 140 = 1095 Ac. Ft.





AD-A155 502 NATIONAL PROGRAM FOR INSPECTION OF NON-FEDERAL DAMS CHICOPEE RESERVOIR (M..(U) CORPS OF ENGINEERS WALTHAM MA NEW ENGLAND DIV JUN 81 2/2

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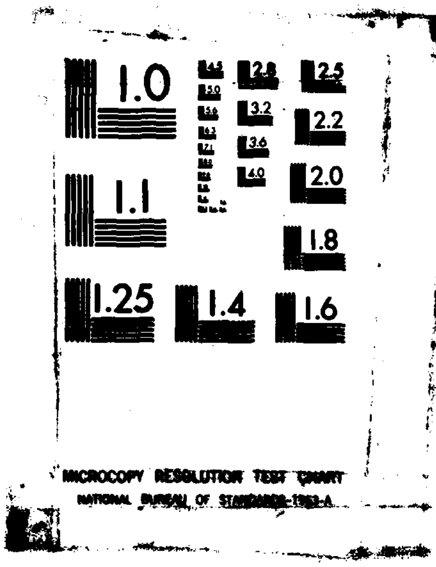
1.0
2.8
2.5
3.2
2.2
3.6
4.0
2.0
1.1
1.8
1.25
1.4
1.6
MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

VI. Downstream Dam Failure Analysis: cont.

In order to route the failure outflow through the Mass. Turnpike, it is necessary to develop an inflow hydrograph based upon the failure outflow and the storage volume. Inspection of the stage discharge curve for the roadway indicates that, with the water surface at the top of the dam, the pre-failure discharge would overtop the roadway, therefore, fail the dam with the water at spillway elevation (168.0).

$\therefore Q_{p_1} = \frac{8}{27} \times 136 \times \sqrt{32.2} \times (38.0)^{3/2} = 53,563$ cfs SAY 53,500 cfs

S = 510 Ac.Ft. ±

ELEV.	Q_o	S	ΔTime	Total Time
168	53,500 cfs	510 Ac.Ft.		
	$\bar{Q}$ = 50,424	ΔS = 40	0.6 Min.	0.6 Min.
165	47,347	470		
	$\bar{Q}$ = 42,460	ΔS = 67	1.1	1.7
160	37,573	403		
	$\bar{Q}$ = 33,213	ΔS = 67	1.5	3.2
155	28,583	336		
	$\bar{Q}$ = 24,518	ΔS = 67	2.0	5.2
150	20,452	269		
	$\bar{Q}$ = 16,868	ΔS = 67	2.9	8.1
145	13,284	202		
	$\bar{Q}$ = 10,258	ΔS = 68	4.8	12.9
140	7,231	134		
	$\bar{Q}$ = 4,894	ΔS = 67	9.9	22.8
135	2,557	67		
	$\bar{Q}$ = 1,279	ΔS = 67	38.0	60.8
130	0	0		

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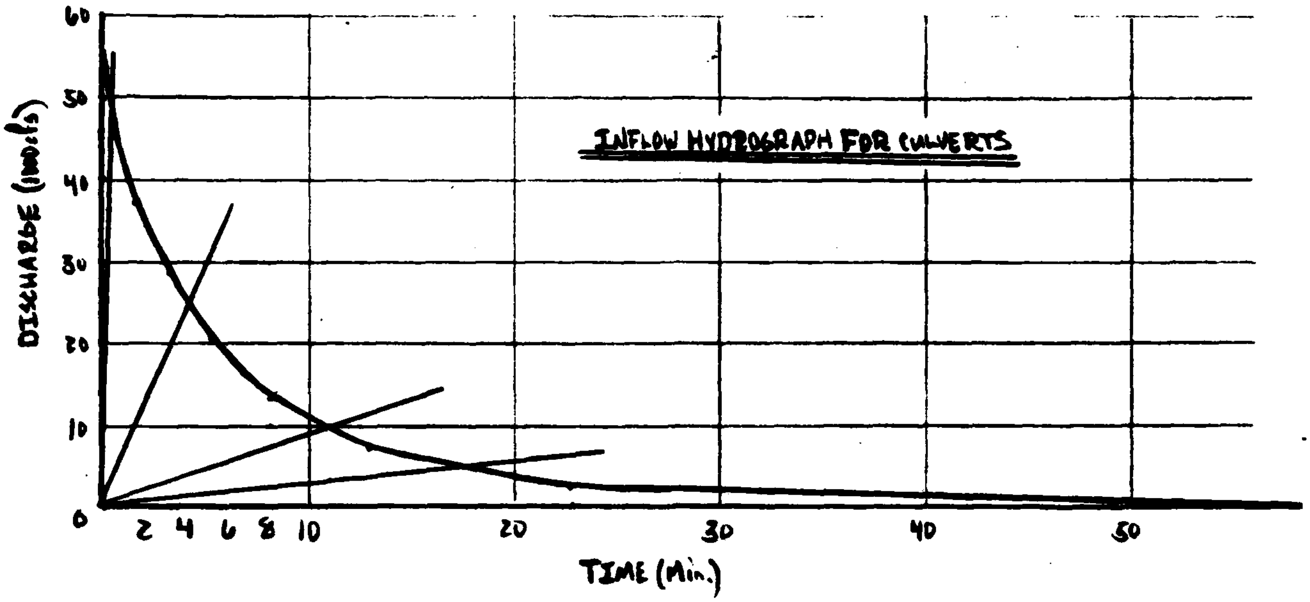
VI. Downstream Dam Failure Analysis: cont.



Route failure outflow through the Mass. Turnpike by plotting Discharge vs. Storage for the roadway and Outflow vs. Storage for Chicopee Reservoir.

FROM STAGE-DISCHARGE AND STAGE STORAGE CURVES FOR THE TURNPIKE –

OUTFLOW	ELEV.	STORAGE
1850 cfs	125	310 Ac.Ft.
6110	130	530
21,500	134	760
52,300	138	1000

HYDROGRAPH RELATIONSHIP –

OUTFLOW	STORAGE
50,000 cfs	24 Ac.Ft.
25,000	142
10,000	280
5,000	369
0	510

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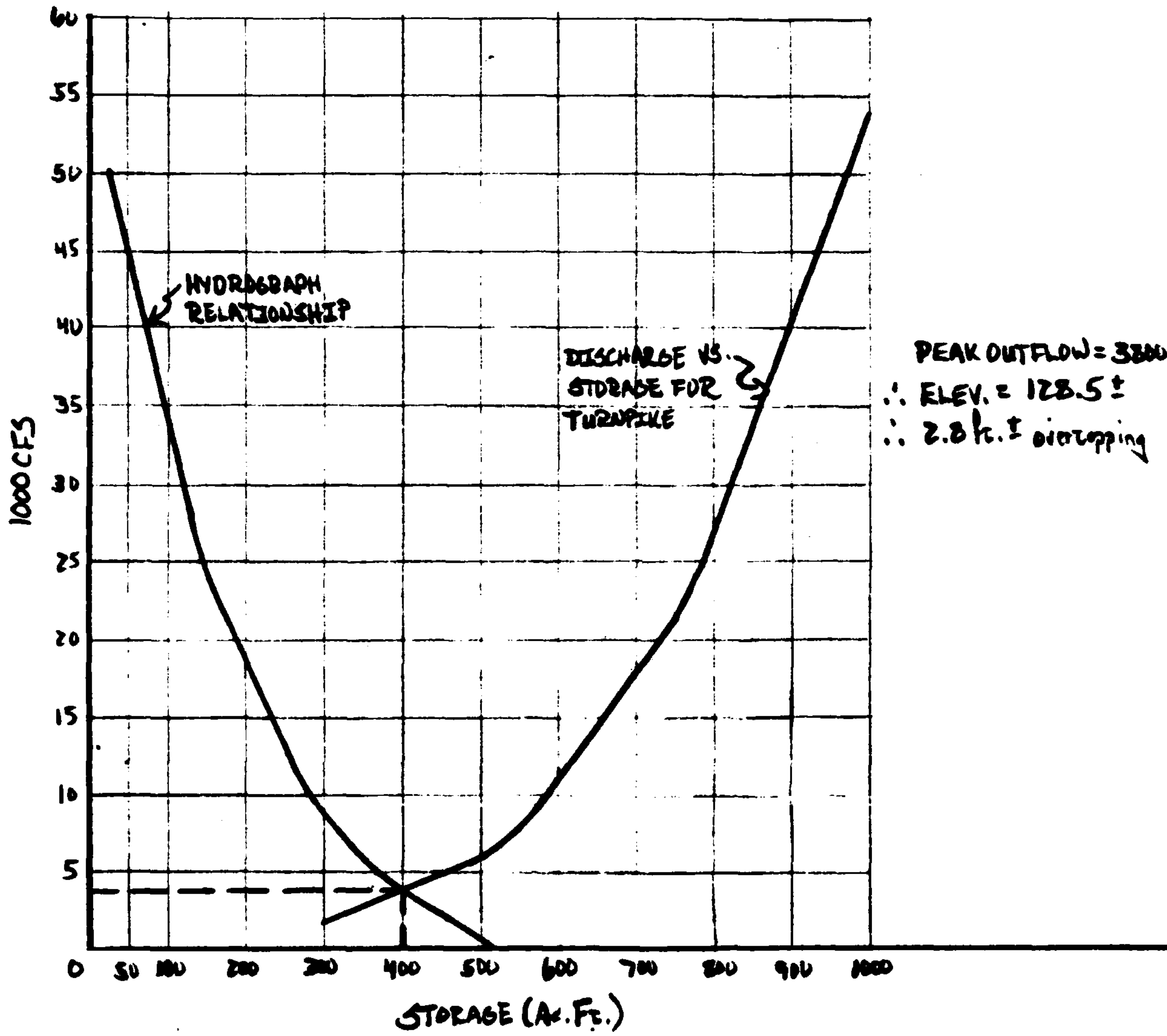
VI. Downstream Dam Failure Analysis : cont.



PEAK OUTFLOW = 3800cfs

∴ ELEV. = 128.5±

∴ 2.8 ft.± overtopping

Examination of the analyses for sections 1 & 2 (with failure occuring with the water surface at the top of the dam) indicates that the elevations reached are not of a magnitude that would cause overtopping of the roadway. Since the preceding analysis indicates overtopping (with a lower failure outflow), it is apparent that the roadway acts as a control section. ∴ neglect sections 1 & 2

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VI. Downstream Dam Failure Analysis: cont.

Flow Depths at damage locations prior to, and following, failure with the water surface at the spillway elevation are as follows:

LOCATION	* PRE-FAILURE ELEV.	POST-FAILURE ELEV.	ELEVATION OF STRUCTURE
MASS. TURNPIKE	—	128.5 ±	125.7 (Low Point)
FILTRATION PLANT	—	112 ± (Estimated)	110 ±
HOUSES ON FULLER RD.	—	112 ± (Estimated)	110 ±

In addition, with the roadway being overtopped by 2.8 ft., failure of the embankment is possible which would increase the depth of flow downstream considerably.

* No outflow from reservoir prior to failure with the water surface at the spillway elevation.

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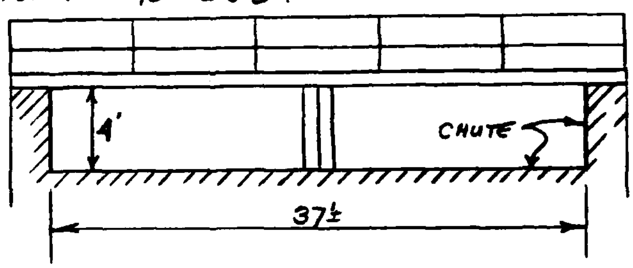
SUBJECT: Chicopee Reservoir - Depth of Flow at Chute Bridge BY JDP CHKD. BY GMJ SHEET 1 OF 2

CALCULATION OF DEPTH OF FLOW UNDER CHUTE BRIDGE AS REQUESTED BY MR. HOLTHAM OF THE COE ON THE DRAFT REPORT REVIEW COMMENT SHEET.

CHUTE SECTION AT BRIDGE.



FOR THE CHANNEL FLOWING FULL,

$A = 37 \times 4 = 148$

$WP = 4 + 37 + 4 = 45$

$R = 3.29$

ASSUME $n = 0.015$

$S = .0348$ (FROM PLANS)

∴ CAPACITY OF CHUTE FLOWING FULL IS

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

$$Q = \frac{1.486}{.015} (148)(3.29)^{2/3} (.0348)^{1/2}$$

$$Q = 6050 \text{ CFS.}$$

WITH THE WATER SURFACE AT THE TOP OF THE DAM, THE SPILLWAY CAN DISCHARGE 3950 CFS.

$$\frac{3950}{6050} = .65$$

FROM DATA BOOK FOR CIVIL ENGINEERS, DESIGN, BY ELWYN E. SEELYE, 3RD EDITION, P 18-73, FIG. A.

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CLIENT / PROJECT U.S. Army COE / Non-Federal Dams DATE 6/4/81 JOB NO.
SUBJECT: Chicopee Reservoir - Depth of Flow BY DP CHKD. BY OMJ SHEET 2 OF 2

FOR 65% OF FULL DISCHARGE, THE DEPTH OF FLOW WILL BE 59% OF THE FULL DEPTH

∴ DEPTH OF FLOW = .59 x 4' = 2.36 FT.

∴ THE BRIDGE WILL NOT IMPEDE FLOW IN THE CHANNEL FOR THE TEST FLOOD OUTFLOW.

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APPENDIX E

INFORMATION AS CONTAINED IN
THE NATIONAL INVENTORY OF DAMS

NOT AVAILABLE AT THIS TIME

END

DATE FILMED

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