

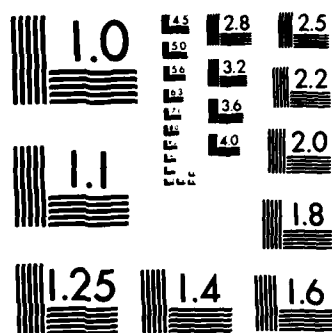
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WASHINGTON STREET DAM. (U) CORPS OF ENGINEERS WALTHAM
MA NEW ENGLAND DIV MAY 79

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MERRIMACK RIVER BASIN
HUDSON, MASSACHUSETTS

WASHINGTON STREET DAM
MA 00447

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM



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

MAY 1979

UNCLASSIFIED

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19. KEY WORDS (Continue on reverse side if necessary and identify by block number) DAMS, INSPECTION, DAM SAFETY, Merrimack River Basin Hudson, Massachusetts Assabet River		
20. ABSTRACT (Continue on reverse side if necessary and identify by block number) The dam consists of a stone masonry spillway 67 feet long and 8 feet high with 15 feet high masonry walls and embankments on each side. The dam is in poor condition. The dam is intermediate in size and has a hazard potential of significant. Investigations are recommended to determine the structural stability of the spillway and to determine the present condition of the former sluiceway.		

WASHINGTON STREET DAM
MA 00447

MERRIMACK RIVER BASIN
HUDSON, MASSACHUSETTS

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

**PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM**

Identification No.: MA 00447
Name of Dam: WASHINGTON STREET DAM
Town: HUDSON
County and State: MIDDLESEX, MASSACHUSETTS
Stream: ASSABET RIVER
Date of Inspection: 8 November 1978

BRIEF ASSESSMENT

Washington Street Dam consists of a stone masonry spillway 67 feet long and 8 feet high with 15 feet high masonry walls and embankments on each side. The original length of the dam is unknown due to development on each side of the spillway. The dam, which reportedly was constructed in the 1860's, impounds the waters of the Assabet River in the Town of Hudson, Mass. The dam was originally constructed to supply water to an adjacent mill. An outlet works which contains a single gate is present at the left abutment of the spillway.

The dam is in poor condition. A number of pressure leaks are present near the bottom of the spillway on the left side. There is a bulge in the face of the spillway in the same area. There are indications that local areas may have settled behind the right downstream channel wall.

Based on the size classification, intermediate, and hazard classification, significant, in accordance with Corps of Engineer Guidelines, the spillway test flood is the 1/2 Probable Maximum Flood (1/2 PMF). Hydraulic analysis indicates that the spillway can safely pass the test flood of 3,790 cfs with a reservoir stage approximately 0.1 feet below the top of dam. Maximum spillway capacity was estimated to be 3,820 cfs.

Investigations are recommended to determine the structural stability of the spillway and to determine the present condition of the former sluiceway. The plugging of leaks in the spillway and the repair of leaks at the outlet gate should be performed with the investigations. Remedial measures recommended for this facility include the removal and patching of deteriorated concrete at the outlet works, the removal of vegetation and/or debris from the channel walls and spillway crest and the repair of a concrete joint in the left downstream channel wall. The Owner should develop a formal maintenance program, operational procedure, emergency procedures plan and institute a program of annual technical inspections. The remedial measures and recommendations should be performed within 1 year of receipt of this report by the Owner. Until the repairs to the spillway have been accomplished, the dam should be kept under surveillance during periods of high precipitation and high reservoir levels.

Camp Dresser & McKee Inc.

Roger H. Wood
Roger H. Wood
Vice President



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

CHARLES G. TIERSCH, Chairman
Chief, Foundation and Materials Branch
Engineering Division

FRED J. RAVENS, Jr., Member
Chief, Design Branch
Engineering Division

SAUL COOPER, Member
Chief, Water Control Branch
Engineering Division

APPROVAL RECOMMENDED:

JOE B. FRYAR
Chief, Engineering Division

PREFACE

This report is prepared under guidance contained in the Recommended Guidelines for Safety Inspection of Dams, for Phase I Investigations. Copies of these guidelines may be obtained from the Office of Chief of Engineers, Washington, D.C. 20314. The purpose of a Phase I Investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general condition of the dam is based upon available data and visual inspections. Detailed investigation, and analyses involving topographic mapping, subsurface investigations, testing, and detailed computational evaluations are beyond the scope of a Phase I Investigation; however, the investigation is intended to identify any need for such studies.

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

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

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

TABLE OF CONTENTS

<u>Section</u>	<u>Page</u>
Letter of Transmittal	
Brief Assessment	
Review Board Page	
Preface	i
Table of Contents	ii & iii
Overview Photo	iv
Location Map	v

REPORT

1. PROJECT INFORMATION

1.1 General	
a. Authority	1-1
b. Purpose of Inspection	1-1
1.2 Description of Project	
a. Location	1-1
b. Description of Dam and Appurtenances	1-2
c. Size Classification	1-2
d. Hazard Classification	1-2
e. Ownership	1-3
f. Operator	1-3
g. Purpose of Dam	1-3
h. Design and Construction History	1-3
i. Normal Operational Procedures	1-3
1.3 Pertinent Data	1-3

2. ENGINEERING DATA

2.1 Design	2-1
2.2 Construction	2-1
2.3 Operation	2-1
2.4 Evaluation	2-1

3. VISUAL INSPECTION

3.1 Findings	
a. General	3-1
b. Dam	3-1
c. Appurtenant Structures	3-1
d. Reservoir Area	3-2
e. Downstream Channel	3-3
3.2 Evaluation	3-3

4. OPERATIONAL PROCEDURES

4.1 Procedures	4-1
4.2 Maintenance of Dam	4-1
4.3 Maintenance of Operating Facilities	4-1
4.4 Description of any Warning System in Effect	4-1
4.5 Evaluation	4-1

TABLE OF CONTENTS (Cont'd)

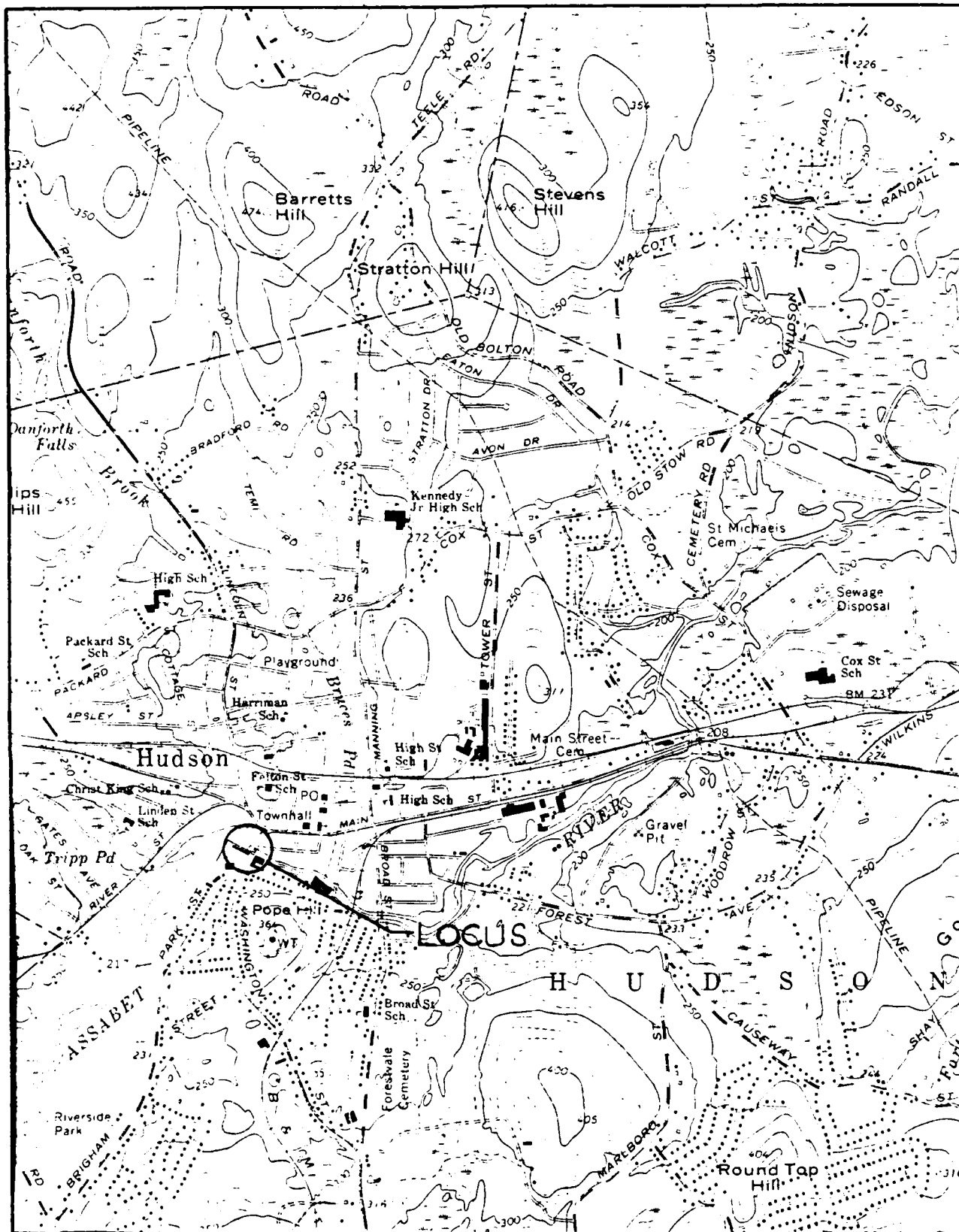
	<u>Page</u>
5. HYDRAULIC/HYDROLOGIC	
5.1 Evaluation of Features	5-1
a. General	5-1
b. Design Data	5-1
c. Experience Data	5-1
d. Visual Observations	5-1
e. Test Flood Analysis	5-2
f. Dam Failure Analysis	5-2
6. STRUCTURAL STABILITY	
6.1 Evaluation of Structural Stability	6-1
a. Visual Observation	6-1
b. Design and Construction Data	6-1
c. Operating Records	6-1
d. Post-Construction Changes	6-1
e. Seismic Stability	6-1
7. ASSESSMENT, RECOMMENDATIONS AND REMEDIAL MEASURES	
7.1 Dam Assessment	7-1
a. Condition	7-1
b. Adequacy of Information	7-1
c. Urgency	7-1
d. Need for Additional Investigation	7-1
7.2 Recommendations	7-1
7.3 Remedial Measures	7-2
a. Operation and Maintenance Procedures	7-2
7.4 Alternatives	7-2

APPENDIXES

APPENDIX A - INSPECTION CHECKLIST	A-1
APPENDIX B - ENGINEERING DATA	B-1
APPENDIX C - PHOTOGRAPHS	C-1
APPENDIX D - HYDROLOGIC AND HYDRAULIC COMPUTATIONS	D-1
APPENDIX E - INFORMATION AS CONTAINED IN THE NATIONAL INVENTORY OF DAMS	E-1



1. Overview of Dam from Washington St. Bridge.



DAM WASHINGTON STREET

IDENTIFICATION NO. 00447



LOCATION MAP
USGS QUADRANGLE
HUDSON, MASSACHUSETTS

APPROX. SCALE: 1" = 2000'

NATIONAL DAM INSPECTION PROGRAM
PHASE I INSPECTION REPORT
WASHINGTON STREET DAM
MA 00447

SECTION 1: PROJECT INFORMATION

1.1 General

- a. Authority - Public Law 92-367, 8 August 1972, authorized the Secretary of the Army, through the Corps of Engineers, to initiate a national program of dam inspection throughout the United States. The New England Division of the Corps of Engineers has been assigned the responsibility of supervising the inspection of dams within the New England Region.

Camp Dresser & McKee Inc. has been retained by the New England Division to inspect and report on selected dams in the State of Massachusetts. Authorization and notice to proceed was issued to Camp Dresser & McKee Inc. under letters of 12 July 1978 and 23 October 1978 from Colonel John P. Chandler, Corps of Engineers. Contract No. DACW 33-78-C-0354 has been assigned by the Corps of Engineers for this work. Haley and Aldrich, Inc. has been retained by Camp Dresser & McKee Inc. for the soils and geological portions of the work.

- b. Purpose - The primary purpose of the investigation is to:

- (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 Washington Street Dam (sometimes called Mill Dam) is located on the Assabet River in the downtown section of the Town of Hudson, Massachusetts, approximately 25 feet upstream of the Washington Street bridge. Access to the dam is directly off of Washington Street.

- b. Description of Dam and Appurtenances - The actual length of the constructed dam is unknown. Evidence of the original embankment areas have been hidden by the property development on each side of the spillway and the construction of the road and bridge immediately downstream of the spillway. Due to the presence of blocked-off openings in the right downstream channel wall, it is assumed that sluiceways were present to the right of the present spillway. The remaining portion of the dam, that which must be considered the present dam, has a length of 67 feet of which 61 feet is spillway and 6 feet is the outlet works. The height of the present dam is approximately 15 feet with the spillway crest elevation being 7 feet below dam crest.

The downstream face of the spillway is vertical while the upstream face is sloped approximately 3 horizontal to 1 vertical. Principal construction of the spillway is stone masonry with mortared joints. The right abutment is of the same construction and serves as the foundation wall of a hardware building which was formerly a mill. The structure is understood to have no basement. Openings in this wall were blocked off with concrete after the flood of August 1955.

The left side of the spillway contains a 6 foot by 8 foot concrete outlet structure. A 3 foot by 4 foot wood sluice gate is located in this structure. The invert of the sluice gate is approximately 1 foot above the downstream elevation of the spillway. The operating controls are located on top of the outlet structure.

The left abutment, training wall and downstream channel wall is a concrete retaining wall. The top of the wall is approximately 7 feet above the spillway crest elevation and 2 feet above the outlet structure. The area to the left of the abutment has a ground elevation which is fairly flat, extending to a service station approximately 50 feet away.

- c. Size Classification - The height of the dam is approximately 15 feet and the estimated total storage capacity at the top of the dam is 1,570 acre-feet. According to guidelines established by the Corps of Engineers, the dam is classified in the intermediate category based on the storage capacity.
- d. Hazard Classification - The dam failure analysis indicates a potential for some loss of life in addition to appreciable economic loss. Downstream of the dam, several business and residential structures would be in the path of the suddenly rising water. The proximity of the residential structures to the Assabet River indicate a possible loss of life. Economic losses would result from the flooding of businesses located immediately downstream. Therefore, the Washington Street Dam is classified as having a significant hazard potential.

- e. Ownership - The dam is presently owned by the Hudson Light and Power Department, of the Town of Hudson, MA. Mr. H. Huehmer of the Light and Power Department, 44 Forest Ave., Hudson, MA. 01749, is the owners' representative.
- f. Operator - Mr. Julian Dubois, Distribution Supervisor for Hudson Light and Power Company, Hudson, MA., Tel. 617-568-8736 is the owners' operator.
- g. Purpose of the Dam - The Washington Street Dam, at one time, supplied water to an old mill on the spillway right abutment. At this time, there is no known purpose for the dam, other than for aesthetic reasons.
- h. Design and Construction History - The dam was constructed in the 1860's. In 1958, work was done on the Washington Street bridge and bridge abutments in the close vicinity of the dam's side walls.
- i. Normal Operational Procedures - There are no operational procedures currently in effect for this structure.

1.3 Pertinent Data

Elevations given in this report are on National Geodetic Vertical Datum (NGVD) formerly referred to as Mean Sea Level (MSL). The elevation assigned to the spillway crest was taken from Massachusetts Geodetic Survey High Water Data Flood of March 1936 in Massachusetts.

- a. Drainage Area - The dam impounds waters of the Assabet River in the Town of Hudson, Massachusetts. The watershed above the dam is 63.7 square miles. The reservoir occupies a negligible percentage of the total drainage area. The watershed is very flat, with extensive reaches of swamp and marsh areas. The remaining portions of the watershed are forested rolling terrain with very light development.
- b. Discharge at Dam Site - Although there is no recorded information for discharge at the dam site, information is available concerning water surface elevations of the pond upstream of the dam during periods of high flow. Peak water surface elevations occurred in November 1927, March 1936, August 1955, October 1962 and March 1968. Based upon the spillway configuration and recorded water surface elevations, the discharge for the August 1955 flood was approximately 3000 cfs. This flood is generally considered the flood of record for the Assabet River.
 - (1) Outlet Works - 3 ft by 4 ft sluice gate at approximate invert elev. 199.4.

- (2) The maximum known discharge occurred in August 1955, and is estimated to have been 3,000 cfs.
- (3) Ungated spillway capacity at top of dam abutment is 3,820 cfs at elev. 212.7.
- (4) Ungated spillway capacity at test flood discharge is 3,790 cfs at elev. 212.6.
- (5) Gated spillway capacity at normal pool elevation-----N/A
- (6) Gated spillway capacity at test flood elevation-----N/A
- (7) Total spillway capacity at test flood elevation is 3,790 cfs at elev. 212.6.
- (8) Total project discharge at test flood elevation is 3,790 cfs at elev. 212.6.

c. Elevation (NGVD)

- (1) Streambed at centerline of dam-----197.7
- (2) Test flood tailwater-----208.0
- (3) Normal pool-----205.7
- (4) Spillway crest-----205.7
- (5) Original spillway design surcharge-----Unknown
- (6) Top of dam (abutment)-----212.7
- (7) Test flood design surcharge-----212.6

d. Reservoir

- (1) Length of test flood pool-----3220 ft (Est.)
- (2) Length of normal pool-----2170 ft (Est.)

e. Storage (acre-feet)

- (1) Normal pool-----100
- (2) Top of dam (abutment)-----1570
- (3) Test flood pool-----1540

f. Reservoir Surface (acres)

- (1) Normal pool-----55
- (2) Spillway crest-----55
- (3) Test flood pool-----363
- (4) Top of dam (abutment)-----365

g. Dam (See also Spillway Data)

- (1) Type-----Stone masonry spillway with probable adjacent embankments
- (2) Length-----67 ft plus
- (3) Height-----15 ft
- (4) Top width-----Unknown
- (5) Side slopes-----D/S Vertical
- (6) Zoning-----Unknown
- (7) Impervious core-----Unknown
- (8) Cutoff-----Unknown
- (9) Grout curtain-----Probably none

h. Diversion and Regulating Tunnel-----None

i. Spillway

- (1) Type-----Broad crested stone masonry
- (2) Length of weir-----61.0 ft
- (3) Crest elevation-----205.7
- (4) Gates-----3 foot by 4 foot sluice gate
- (5) U/S Channel-----Assabet River
- (6) D/S Channel-----3 arch culverts under Washington Street each 18 feet in diameter

j. Regulating Outlet - There is a 3 foot by 4 foot sluice gate on the left side of the spillway. Reportedly the gate is in poor operational condition. A backhoe will usually be used to seat the gate after being operated.

SECTION 2: ENGINEERING DATA

2.1 Design

There are no known design records for the dam.

2.2 Construction

No records of the original construction were located.

2.3 Operation

There are no known operational records other than County and State inspection reports.

2.4 Evaluation

- a. Availability - There are no known records on the dam except for County and State inspection reports.
- b. Validity - No data was located for the dam.
- c. Adequacy - In the absence of engineering data on the dam, the evaluation for this investigation must be based on the visual examination.

SECTION 3: VISUAL INSPECTION

3.1 Findings

- a. General - The Phase I Visual Examination of the Washington Street Dam was conducted on 8 November 1978.

The dam was observed to be in poor condition based on observed pressure leaks present at the spillway and probable loss of fill material in the abandoned sluiceway. In addition, the outlet works gate is believed to be only marginally operable.

Visual inspection checklists for the site visit are included in Appendix A and selected photographs are given in Appendix C.

- b. Dam - The dam, due to adjacent development of the area, is basically a stone masonry spillway between a stone masonry channel wall on the right side and a concrete channel wall on the left side. An outlet works structure is present between the spillway weir and the left channel wall. Debris is present along the spillway crest. The left half of the spillway contains 18 or more pressure leaks near the base of the structure. They are clearly visible from the downstream bridge. The face of the spillway appears to be bulging downstream in the area of the leaks.

While the dam presently has no discernable earth embankments as such, the fill materials behind each masonry abutment wall also serve to retain the water stored by the dam. There is no visual evidence of wall or backfill settlement or lateral movement, or major seepage, but there is some question as to the present condition of the former sluiceway around the right abutment of the dam.

There is a sag in the northeast corner of the hardware store floor and a large bituminous concrete patch in the sidewalk in front as shown in Photo 11. These conditions, in conjunction with the possible incomplete closure of the water level opening in the channel wall below the dam, as shown in Photo 10, may indicate either loss or consolidation of existing sluiceway fill. A loss of material into the channel would be concealed by the river flow.

- c. Appurtenant Structures

- (1) Approach Channel - The approach channel is formed by two walls extending about 120 feet upstream of the spillway weir. The right wall is a grouted stone masonry wall. It

was a part of the foundation of a mill building but now is part of a hardware store foundation. The openings for the former sluiceways of the mill have been plugged with concrete. Brush is growing in the joints of this wall. The left wall near the spillway is of concrete construction and is in good condition. The remaining portion of the wall is grouted stone masonry which is in good structural condition but has a heavy vine growth.

- (2) Outlet Works - The 3.0 foot by 4.0 foot slide gate in the outlet structure is leaking around the edges and top. There is also a small leak developing through the slide gate in the upper left hand corner. The concrete at the spillway side of the structure has badly spalled and erosion is taking place at the crest level. A crack is present in the concrete on the downstream face of the structure.
- (3) Discharge Channel The left wall is a concrete retaining wall about 15 feet high matching into the grouted stone abutment of the bridge on Washington Street. The wall has three weep holes about 5 feet above the wall footing with the two closest to the dam leaking water. The concrete below the weep holes is badly stained. More water and staining was observed at the bottom of the vertical joints in the concrete wall. Although seepage was observed, soil particles were not evident in the seepage flow. The second joint downstream from the dam appears to be an expansion joint with joint filler either missing or badly disintegrated. A piece of concrete has broken off from the top of the downstream face of the expansion joint. There is an exposed 3 foot high by 8.5 foot wall footing visible just below the water surface. There was an indication that slight movement has taken place at the top of the wall.

The right wall is of the same construction and condition as the right approach channel wall except that a sluiceway opening at the bottom of the wall has been sealed with pre-cast concrete rather than cast in place concrete. It appears that the downstream edge of the closure slab is being supported by reinforcing bars grouted into the joint of the stone masonry.

During the inspection of the dam, a heavy smell of gasoline could be detected and a petroleum product could be seen floating in the water under the bridge. The amount observed was much greater than one would expect from normal roadway discharge.

- d. Reservoir Area - There is a slight increase in the width of the river channel upstream of the Washington Street Dam. Development is sparse with a few dwellings and businesses immediately

upstream of the dam. Relatively minor flooding with no appreciable damage to structures would occur upstream of the dam at test flood elevation. No significant potential was observed for landslides into the general pool area of the dam which would create waves that might overtop the abutments of the dam. No conditions were noted that would result in a sudden increase in sediment load into the upstream pool.

- e. Downstream Channel - Immediately downstream of the Washington Street Dam, flow must pass under Washington Street (Route 85). The bridge consists of three arch-type openings, each approximately 18 feet in diameter and approximately 50 feet in length. Downstream of Washington Street, on the right bank, is a brick and concrete structure constructed on the river bank.

There are approximately 4 residential structures located further downstream on the left bank, set back somewhat from the normal river edge.

Extensive shrubbery exists on and alongside both banks. Elevations rise somewhat sharply on the right bank, while the elevations on the left bank are much flatter. Approximately 1,300 feet downstream of the dam, is the Houghton Street Bridge. The bridge consists of three rectangular openings, each 24 feet wide and 7 feet high. Average slope of the river bed between Washington Street and Houghton Street is .0017.

3.2 Evaluation

Based on the visual examination during the site visit on 8 November 1978, the dam was found to be in poor condition due to the observed line of leaks at the bottom of the spillway and the apparent bulging of a portion of the spillway face. Other deficiencies noted during the examination included brush and vine growth on the walls, deterioration of concrete at the outlet structure, leaks at the outlet works gate and seepage at the downstream walls. The pressure leaks in the spillway, the bulging of the spillway face and the indicated slight movement of the left downstream channel wall are all conditions that could affect the stability of the structure. The pressure leaks and bulging will be further discussed in Section 6. The indicated movement of the downstream channel wall is so slight that it is not of immediate concern.

The abutment-area fills at the Washington Street Dam appear to be performing adequately at the present time, but the uncertain condition of the apparent former sluiceway around the right abutment offers some potential for uncontrolled water flow past the dam.

SECTION 4: OPERATIONAL PROCEDURES

- 4.1 Procedures - In general, there is no established routine for the operation of the dam.
- 4.2 Maintenance of Dam - There is no established formal procedure for the maintenance of the dam.
- 4.3 Maintenance of Operating Facilities - There is no formal procedure for maintenance of operating facilities.
- 4.4 Description of any Warning System in Effect - There is no established warning system or emergency preparedness plan in effect for the dam.
- 4.5 Evaluation - There is no formal operational procedures in effect for the dam. Operational procedures, maintenance programs, warning systems and an emergency preparedness plan should be established for the dam. Periodic maintenance should be performed to insure the gate is operational and to minimize deterioration of the structure.

SECTION 5: HYDRAULIC/HYDROLOGIC

5.1 Evaluation of Features

- a. General - The Washington Street Dam is located on the Assabet River in the downtown section of the Town of Hudson, MA. approximately 25 feet upstream of Washington Street. The dam is a stone masonry structure having a maximum height of approximately 15 feet and a total length of 67 feet. The spillway is 61 feet in length and rises approximately 8 feet above the downstream river bed. The dam creates an impoundment of 55 acres and an estimated total storage capacity of approximately 100 acre-feet, at its spillway crest elevation of 205.7. The pool at the top of dam (approx. elev. 212.7) comprises 365 acres and an estimated total storage capacity of 1,570 acre-feet. The upstream pool is reported to be heavily silted and the project is basically a run of the river type with minimal upstream surcharge-storage.
- b. Design Data - There are no plans or records available concerning design data or construction details for this dam. All hydraulic and hydrologic criteria used in this report were developed by utilizing the U.S.G.S. quadrangle maps, flood records, and other data gathered for this investigation.
- c. Experience Data - Significant flooding has occurred on the Assabet River in November 1927, March 1936, August 1955, October 1962, and March 1968. The flood in 1955 is, according to the Corps of Engineers' Flood Plain Information Report, the flood of record for the Assabet River. The estimated flow over the Washington Street dam was approximately 3,600 cfs and reached an elevation of 212.4. The second greatest flood occurred in March, 1936. Estimated peak flow for this flood was 3,000 cfs and the maximum water surface was approximately 211.6.
- d. Visual Observation - At the time of the inspection of the dam on 8 November 1978, the water surface over the crest of the spillway was approximately 2-3 inches, with a flow estimated at 20 cfs. The spillway appeared to be in good hydraulic condition. There is a 3 foot by 4 foot sluice gate located on the left side of the spillway. This gate was in the closed position at the time of the inspection, but it has been reported that it is marginally operable should the need arise to drain the pond upstream of the dam. The sluice gate control is located on a 6 by 8 foot concrete structure 5 feet above the spillway crest. There is evidence that at some time in the past, a sluiceway existed in the basement of the building on the right abutment. The entrance and exitway for the sluiceway appears to have been blocked.

Approximately 25 feet downstream from the dam, flow must pass under the Washington Street bridge. This bridge has three arch-type openings and appears to be in good condition. Downstream of Washington Street, there is a building on the right bank of the Assabet River. The left bank has little development with small areas of vegetation growing in the river bed.

- e. Test Flood Analysis - Based upon Corps of Engineers Guidelines, the recommended test flood for the size (Intermediate) and hazard potential (Significant) is within the range of 1/2 PMF to full PMF (Probable Maximum Flood). The size classification, based on the storage capacity of the dam, barely exceeds the "small" category. For this reason, the test flood selected was the 1/2 PMF. The 1/2 PMF was determined using the guideline curves as presented by the New England Division of the Corps in "Estimating Maximum Probable Discharges" for the Phase I, Dam Safety Investigations". The watershed for the Assabet River is very flat, with extensive swampy areas. Because of these characteristics, an inflow of 4,150 cfs was adopted which is slightly less than the recommended value for flat and coastal terrain. Surge-storage routing of the 1/2 PMF inflow through the ponding area upstream of the dam resulted in a 1/2 PMP outflow of approximately 3,940 cfs. In 1966, the New England Division of the Corps of Engineers published a Flood Plain Information report for the Assabet River. The Standard Project Flood (SPF) developed for this report was approximately 5,320 cfs at the Maynard Gage which is on the Assabet River approximately 10.5 miles downstream of the Washington Street Dam with a drainage area of 116 square miles. Using the drainage area relationships, the SPF at the Washington Street Dam would be approximately 3,790 cfs. According to published data, the SPF by definition, is approximately equal to 1/2 the PMF. For the purposes of this report, the peak flow for the test flood will be 3,790 cfs. This will result in a peak water level above the dam of approximately Elev. 212.6. The spillway is considered to be just adequate to pass the test flood.
- f. Dam Failure Analysis - Based on Corps of Engineers Guidelines for Estimating Dam Failure hydrographs and assuming that the breach width would be 40 percent of the dam, with the water level at the top of the spillway abutments (Elev. 212.7), the failure would result in a peak outflow rate of 4640 cfs. This flow will result in moderate flooding downstream of Washington Street, especially on the left bank. The constriction of Washington Street bridge and two other bridges downstream will cause some backwater effect. Due to some storage between Washington and Houghton streets, and between Houghton and Broad streets, the peak flows will be reduced to 4,355 cfs and 4,275 cfs, respectively.

The above dam failure analysis is based on the assumption that failure would occur during a full spillway discharge of approximately 3,820 cfs. The increase of flow, due to the dam failing, would amount to approximately 4,640 cfs. It is recognized that just prior to the dam failing, a general condition of flooding would already be occurring downstream. The increase in water surface downstream would be approximately 1 foot. This may or may not present any additional hazard beyond that already existing, due to the high spillway discharge. However, it is recognized that should the dam fail at some point in time when the spillway discharge is somewhat less than maximum, the increase in flow resulting from a failure would be such as to have a significant effect on economic losses and would increase the potential for loss of life to the inhabitants of approximately 4 homes located on the left downstream bank. The Washington Street Bridge would not be overtopped as a result of an increase in flow due to a dam failure

SECTION 6: STRUCTURAL STABILITY

6.1 Evaluation of Structural Stability

- a. Visual Observation - The multi-pressure leaks at the bottom of the spillway and the apparent bulging of a portion of the face of the spillway indicate questionable structural stability of the left half of the spillway. The presence of seepage from the sealed abandoned sluiceway together with the observed evidence of settlement behind the channel right side wall place this wall in question. Although evidence of slight past movement at the top of the left downstream wall is present, the indicated movement is so slight it should not be considered evidence of structural instability at this time.
- b. Design and Construction Data - There are no known design and construction data on the dam thus precluding a theoretical analysis of structural stability.
- c. Operating Records - Inspection reports indicate the pressure leaks at the bottom of the spillway have been in existence for at least ten years. This coupled with the present condition of the dam indicates that the dam has inherent stability but it is deteriorating.
- d. Post-Construction Changes - Without design or "as-built" drawings, the extent of post-construction changes is not known. The existence of the concrete outlet works and the concrete portion of the left side wall as compared to the stone masonry in other areas indicates these structures were constructed at a later date. The sealed outlets on the right side wall indicate that the area to the right of the spillway has been modified since the original construction.
- e. 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 visual examination of the Washington Street Bridge Dam did not reveal any evidence of conditions which would warrant emergency remedial treatment. However, the presence of pressure leaks in the spillway and the face bulge in the spillway cause this project to be considered in poor condition. There is need for maintenance and additional investigation that are outlined hereinafter.
- b. Adequacy of Information - All of the information for the Phase I Investigation had to be obtained from the visual examination, limited field measurements and previous inspection reports. While this information has been sufficient for the purpose of this investigation, it does not permit a detailed evaluation of stability and seepage.
- c. Urgency - The recommended additional investigations and remedial measures outlined in Sections 7.2 and 7.3, respectively, should be undertaken within one year of receipt of this report by the Owner.
- d. Need for Additional Investigations - Additional investigations should be performed by the Owner as outlined in the following section.

7.2 Recommendations

The Owner should engage the services of a qualified registered professional engineer to perform the following investigations:

1. An investigation of the structural stability of the spillway. The investigation should be based on detailed measurements of the spillway, an inspection of the damage to the downstream face of the spillway, and a sampling of the materials of construction by core borings. The investigation should include the dewatering of the pool at which time the upstream face of the spillway should be inspected, the joints in the stone work mortared to reduce leakage, and the outlet gate repaired and made operational under the direction of a professional engineer.
2. An investigation to determine the location, original construction and present condition of the former sluiceway around the right abutment, in order to establish whether or not it presents any hazard to the safety of the dam. The investigation should include further research into historical records, and examination of any crawl spaces under the building and the water level opening below the dam. If a potential hazard does exist, corrective measures should be developed.

7.3 Remedial Measures

a. Operation and Maintenance Procedures - It is recommended that the following operation and maintenance procedures be adopted by the Owner to correct deficiencies noted during the visual examination:

- (1) Remove deteriorated concrete on the outlet works structure and patch with mortar, including all spalled areas. Repair the cracks with epoxy.
- (2) Remove vegetation from the training and channel walls and clear debris from the spillway crest.
- (3) Repair the sealant and concrete at the second wall joint downstream of the dam on the left side.
- (4) Develop a formal maintenance program, operational procedure, emergency procedures plan and warning system in cooperation with downstream officials.
- (5) Due to the condition of the spillway and the right abutment, the dam should be kept under surveillance during periods of high precipitation and high reservoir levels.
- (6) Institute a program of annual technical inspections.

7.4 Alternatives - An alternative to the recommendations and remedial measures would be to breach the dam. The environmental impact of breaching the dam should be investigated before taking this action.

APPENDIX A

INSPECTION TEAM ORGANIZATION AND CHECKLIST

Page No.

VISUAL INSPECTION PARTY ORGANIZATION

A-1

VISUAL INSPECTION CHECK LIST

Dam Embankment, Main Dam
Spillway
Spillway (cont'd)
Outlet Works

A-2
A-3
A-4
A-5

VISUAL INSPECTION PARTY ORGANIZATION
NATIONAL DAM INSPECTION PROGRAM

DAM: Washington Street Bridge Dam

DATE: November 8, 1978

TIME: 8:30 a.m.

WEATHER: 50° F - Overcast - Drizzle

WATER SURFACE ELEVATION UPSTREAM: 2" over weir crest

STREAM FLOW: 15 cfs

INSPECTION PARTY:

1. Robert P. Howard - CDM - Structural/Operations
2. Francis E. Luttazi - CDM - Structural/Operations (Ass't)
3. Charles E. Fuller - CDM - Hydraulic/Hydrology
4. Joseph E. Downing - CDM - Hydraulic/Hydrology (Ass't)
5. Peter L. LeCount - Haley & Aldrich - Soils
6. _____

PRESENT DURING INSPECTION:

1. _____
2. _____
3. _____
4. _____

VISUAL INSPECTION CHECK LIST NATIONAL DAM INSPECTION PROGRAM

DAM: Washington St. Br., Hudson

DATE: 8 November 1978

EMBANKMENT: _____

CHECK LIST	CONDITION
1. Upstream Slope a. Vegetation b. Sloughing or Erosion c. Rock Slope Protection - Riprap Failures d. Animal Burrows 2. Crest a. Vegetation b. Sloughing or Erosion c. Surface cracks d. Movement or Settlement 3. Downstream Slope a. Vegetation b. Sloughing or Erosion c. Surface cracks d. Animal Burrows e. Movement or Cracking near toe f. Unusual Embankment or Downstream Seepage g. Piping or Boils h. Foundation Drainage Features i. Toe Drains 4. General a. Lateral Movement b. Vertical Alignment c. Horizontal Alignment d. Condition at Abutments and at Structures e. Indications of Movement of Structural Items f. Trespassing g. Instrumentation Systems	Note: Dam has substantially no earth embankments. Stone masonry and/or concrete walls on each side support adjacent-area fill & confine flow. Condition notations below apply to abutment areas, as applicable: 1. a. N/A b. N/A c. Stone masonry walls in reasonably good condition. d. N/A 2. a. N/A b. N/A c. N/A d. Apparent past settlement of bldg. floor & pav't on rt. side over old sluiceway. 3. a. N/A b. N/A c. N/A d. N/A e. N/A f. Slight seepage from weep holes & at base of wall below dam on left. g. N/A h. N/A i. N/A 4. a. N/A b. N/A c. N/A d. Seepage in abutment areas associated with dam structure. e. None observed f. N/A g. None

VISUAL INSPECTION CHECK LIST
NATIONAL DAM INSPECTION PROGRAM

DAM: Washington Street

DATE: November 8, 1978

SPILLWAY: _____

CHECK LIST	CONDITION
1. Approach Channel a. General Condition b. Obstructions c. Log Boom etc. 2. Weir a. Flashboards b. Weir Elev. Control (Gate) c. Vegetation d. Seepage or Efflorescence e. Rust or Stains f. Cracks g. Condition of Joints h. Spalls, Voids or Erosion i. Visible Reinforcement j. General Struct. Condition 3. Discharge Channel a. Apron b. Stilling Basin c. Channel Floor d. Vegetation e. Seepage f. Obstructions g. General Struct. Condition 4. Walls a. Wall Location Upstream of Spillway Left & Right (1) Vegetation (2) Seepage or Efflorescence (3) Rust or Stains (4) Cracks (5) Condition of Joints (6) Spalls, Voids or Erosion (7) Visible Reinforcement (8) General Struct. Condition	1. a. Good b. Heavy bush growth along left wall, and minor growth on right wall. c. None 2. a. None b. See Control Facility c. Large build up of growth and debris along spillway crest. d. Leaks through joints at 18 or more locations along the bottom of the left half of the spillway. e. None observed f. None visible as observed from downstream bridge. g. Condition of joints not observable except where leaks are present. h. None visible as observed from downstream bridge. i. N/A j. Fair 3. a. Not visible - submerged. b. N/A c. Not visible - submerged. d. None observed e. None visible - Base of discharge channel submerged. f. None observed upstream of bridge. Trees & brush downstream. g. Not observable 4. a. (1) Heavy growth on grouted stone wall on left side and minor growth on grouted granite slab stone wall on right side. (2) None observed above water line. (3) None observed (observed from bridge) (4) None observed (observed from bridge) (5) Not observable (6) None observed (observed from bridge) (7) None observed (8) Good

VISUAL INSPECTION CHECK LIST
NATIONAL DAM INSPECTION PROGRAM

DAM: Washington Street

DATE: November 8, 1978

SPILLWAY: _____

CHECK LIST	CONDITION
4. b. Wall Location Downstream of Spillway Left. (1) Vegetation (2) Seepage or Efflorescence (3) Rust or Stains (4) Cracks (5) Condition of Joints (6) Spalls, Voids or Erosion (7) Visible Reinforcement (8) General Struct. Condition	4. b. (1) None observed (2) Slow seepage at two of three weep holes upstream of bridge and bottom of stone masonry joint at upstream end of bridge. Slow seepage at bottom of joint in concrete wall upstream of bridge. Very slow seepage from two or three locations at base of wall downstream of bridge. (3) Two of three weep holes and bottom of joints show rust and stain. (4) Upper downstream corner of expansion joint has cracked and fallen off. (5) Construction joints good. Joint filler in expansion joint has disintegrated. (6) See 4b(4) (7) None observed (8) Good
4. c. Wall Location Downstream of Spillway Right (1) Vegetation (2) Seepage or Efflorescence (3) Rust or Stains (4) Cracks (5) Condition of Joints (6) Spalls, Voids or Erosion (7) Visible Reinforcement (8) General Struct. Condition	4. c. (1) None observed (2) None observed through wall. Possible seepage through blocked rectangular outlet at water line close to upstream edge of bridge. There is a 16" sq. ft. + drain halfway up the wall downstream of the bridge. (3) Stains on concrete of concrete blocked opening in mortared joint stone wall. (4) None observed (5) Good (6) Concrete used to block up existing openings in grouted stone wall shows some minor erosion. (7) Bars exposed on downstream side of opening at water level near bridge. (8) Good.

VISUAL INSPECTION CHECK LIST NATIONAL DAM INSPECTION PROGRAM

DAM: Washington Street

DATE: November 8, 1978

OUTLET WORKS: Control Facility

CHECK LIST	CONDITION
1. Control Facility	1.
a. Structure	a. Concrete gate structure approx. 6 ft. wide by 8 ft. deep with the top of the structure approx. 5 ft. above the left side of the spillway crest. Concrete badly spalled and eroded on the spillway side at the crest elevation. There is cracking on the downstream face at about the crest elevation.
b. Screens	b. None observed
c. Stop Logs	c. None
d. Gates	d. There is a 3.0 ft. wide by 4.0 ft. high slide gate. Gate leaks around edges and through a pin hole in gate.
e. Conduit	e. None
f. Seepage or Leaks	f. See 1d.
g. General Struct. Condition	g. The structure is in generally good condition. The slide gate appears to be in poor condition, and should be replaced.

APPENDIX B
LIST OF AVAILABLE DOCUMENTS AND
PRIOR INSPECTION REPORTS

<u>LIST OF AVAILABLE DOCUMENTS</u>		<u>Page No.</u>
		None
<u>PRIOR INSPECTION REPORTS</u>		
<u>Date</u>	<u>By</u>	
September 5, 1968	Metcalf & Eddy Engineers	B-1
December 5, 1974	Mass. Dept. of Public Works	B-3, 4, 5, 6
	w/ Description of Dam	B-7, 8, 9

CABLE ADDRESS - METCOO - 80513
(817) 423-5600

[illegible]

* : 2000000000
 2 3000000000
 3 4000000000

J - 8941

September 5, 1968

Mr. John Shaughnessy
Middlesex County Engineer.
Court House
East Cambridge, Massachusetts

Dear Mr. Shaughnessy:

In response to the request in your letter of August 20, the writer inspected the dam on the Assabet River (R-6) in Hudson, Massachusetts, which belongs to the Hudson Light and Power Department. The inspection was made on September 4 in the company of Mr. H. Huehmer, Department Manager and Mr. Julian DuBois, an employee of the Department.

The dam is located about 25 feet upstream from the bridge on State Route 85. The dam is not used for any purpose. The structure is approximately 10 feet high with a vertical downstream face and a sloping upstream face (approximately 3:1). The downstream face is stone block with mortared joints. The upstream face appears to be covered with a concrete apron. The construction of the interior of the dam could not be determined but it possibly consists of stone block. The dam is an overflow type, 67 feet in length. The spillway is 61 feet long. A 6 by 8 foot concrete gate structure is located on the northerly abutment. A 3 ft. by 4 ft. wooden sluice gate is operated by a mechanism on the top of the gate structure 5 feet above the 1-ft. wide spillway crest. Each abutment consists of concrete retaining walls extending up and downstream. The pond level was about at the spillway crest at the time of the inspection.

Water was flowing from approximately 18 holes in the downstream face of the dam. The leaks appeared to be confined to the northerly half of the spillway length and emerged from

INDEX/NOTATIONS : REPORTS : RESIONS : ADVICE DURING CONSTRUCTION : ADVICE ON OPERATION

APPENDIX B-1

Mr. John Shaughn
September 5, 1968

2

2 to 3 feet above the base of the dam. The remainder of the exposed portion of the dam appeared to have no appreciable deficiencies.

From our study of the structure we conclude that the leaks should be stopped in order to preserve the stability of the structure. The work would best be undertaken during a period of low stream flow when the pond could be drawn down through the gate and a low coffer-dam placed around the upstream toe. The method of stopping the leaks could be determined when the sources of the leaks were discovered. Repair of the upstream face apron and/or cement grouting of the stone blocks in the dam or the rock foundation might be among the methods employed.

The question was raised by Mr. Huehmer as to whether it would be permissible to place flashboards on the crest of the dam. Flashboards would reduce the discharge capacity of the spillway. Our studies also show that flashboards would throw additional stresses on the dam which could lead to endangering its stability.

We therefore recommend that:

1. The leakage in the dam be eliminated.
2. No flashboards be installed on the dam.

Very truly yours,

METCALF & EDDY, INC.

Gordon E. Thomas
Gordon E. Thomas
Project Engineer

GET:jg

DESCRIPTION OF DAM
DISTRICT 4

Submitted by C. Thomas & V. Murphy
Date 12/5/74

Dam No. 4-9-141-1
City/Town Hudson, N.J.
Name of Dam Cash St. Dam

1. Location Topo Sheet No. 23B
Provide 8 1/2" x 11" in clear copy of topo map with location of dam clearly indicated.
2. Year built: 1864 Year/s of subsequent repairs 1958
3. Purpose of dam: Water Supply _____
Irrigation _____ Recreational _____
Other None
4. Drainage Area: 5 ± Sq. mi. 3.200 Acres.
5. Normal Ponding Area: 16 ± acres Ave Depth 6' ±
Impoundment: 31 M gals 96 acre ft.
6. No. and type of dwellings located adjacent to pond or reservoir
i.e., summer homes etc. Permanent business
7. Dimensions of Dam: Length 780' ± Max. Height 10'
Steepest Upstream Face Vert. _____
Downstream Face Vert. _____
Width across top 1'
8. Classification of Dam by Materials:
Earth ☒ Concrete masonry ☒ Stone masonry ☒
Timber _____ Rockfill _____ Other _____
9. A. Description of present land usage downstream of dam: 80% rural
20% Urban
B. Is there a storage area of flood water downstream of dam which could result in the failure of a dam, etc. due to failure
to _____ ☒ Assabel River

DAM NO. 4-9-141-1

10. Risk to life and property in event of complete failure.

No. of people

No. of homes

No. of businesses

No. of industries

No. of utilities

Type

Type

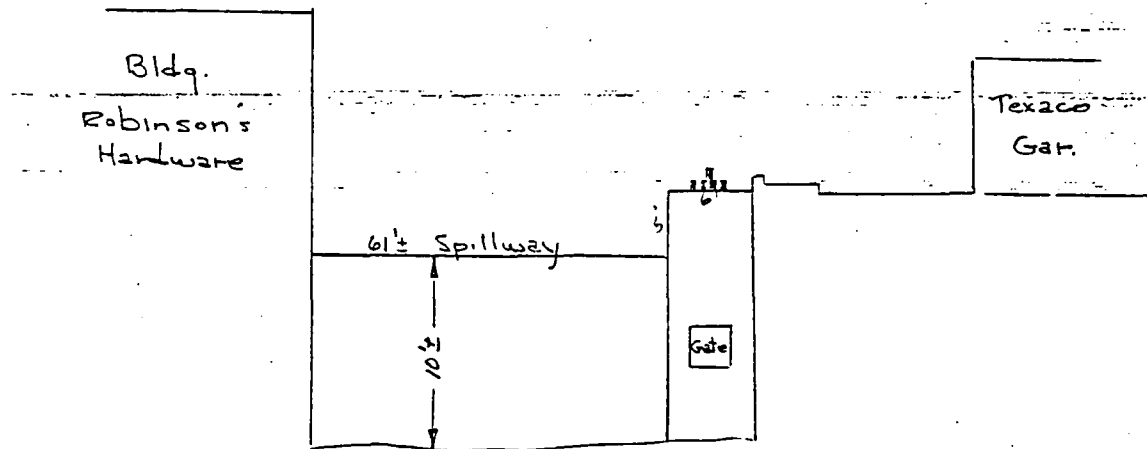
Railroads

Other dams

Other

11. Attach sketch of dam to this form showing section and plan 8 1/2" x 11" sheet

Dam No. 4-9-141-1



Front View
Not To Scale

INSPECTION REPORT - DAMS AND RESERVOIRS

(1) Location: City/Town Hudson Dam No. 4-9-141-1
 Name of Dam Washington St. Dam Inspected by: C. Johnson & V. Murphy
 Date of Inspection 12/5/74

(2) Owners: per: Assessors _____ Prev. Inspection _____
 Reg. of Deeds _____ Pers. Contact ☒
 1. Hudson Light & Power Dept. 49 Forest St, Hudson 1-528-8736
 Name St. & No. City/Town State Tel. No.
 2. _____
 Name St. & No. City/Town State Tel. No.
 3. _____
 Name St. & No. City/Town State Tel. No.

(3) Caretaker: (if any) e.g. superintendent, plant manager, appointed by
 absentee owner, appointed by multi owners.
Mr. H. F. Palmer, Hudson Light Dept. 1-528-8736
 Name St. & No. City/Town State Tel. No.

(4) No. of Pictures taken 4

(5) Degree of Hazard: (if dam should fail completely)
 1. Minor _____ 2. Moderate ☒
 3. Severe _____ 4. Disasters _____
 * This rating may change as land use changes (future development)

(6) Outlet Controls: Automatic _____ Manual ☒
 Operating ☒ Yes _____ No _____

Comments: Open at time of inspection

(7) Estimated Water of De _____

_____ Appears to be adequate

(8) Downstream Face of Dam: Condition: 1. Good _____ 2. Minor Repairs ☒
3. Major Repairs _____ 4. Urgent Rep _____

Comments: Small leaks were noted during inspection

(9) Emergency Spillway: Condition: 1. Good _____ 2. Minor Repairs _____
3. Major Repairs _____ 4. Urgent Repairs _____

Comments: None

(10) Water Level at time of inspection 0.5 ft. above ☒ below _____
top of dam _____ Principal Spillway ☒
Other _____

(11) Summary of Deficiencies Noted

Growth (Trees and Brush) on Embankment ☒

Animal Burrows and Washouts

Damage to slopes or top of dam

Cracked or damaged masonry

Evidence of Seepage

Evidence of Piping

Erosion

Leaks ☒

Trash and/or debris impounding

Clogged or blocked spillway

Other

(12) Remarks & Recommendations: (Fully Explain)

Small leaks noted on downstream face of dam should be repaired to preserve the stability of the structure.

Brush on east embankment should be cut

(13) Overall Condition:

1. Safe ✓
2. Minor repairs needed ✓
3. Conditionally safe - major repairs needed
4. Unsafe
5. Reservoir impounded no longer viable (permanent)
Recommend removal from inspection list

APPENDIX C

SELECTED PHOTOGRAPHS OF PROJECT

LOCATION PLAN

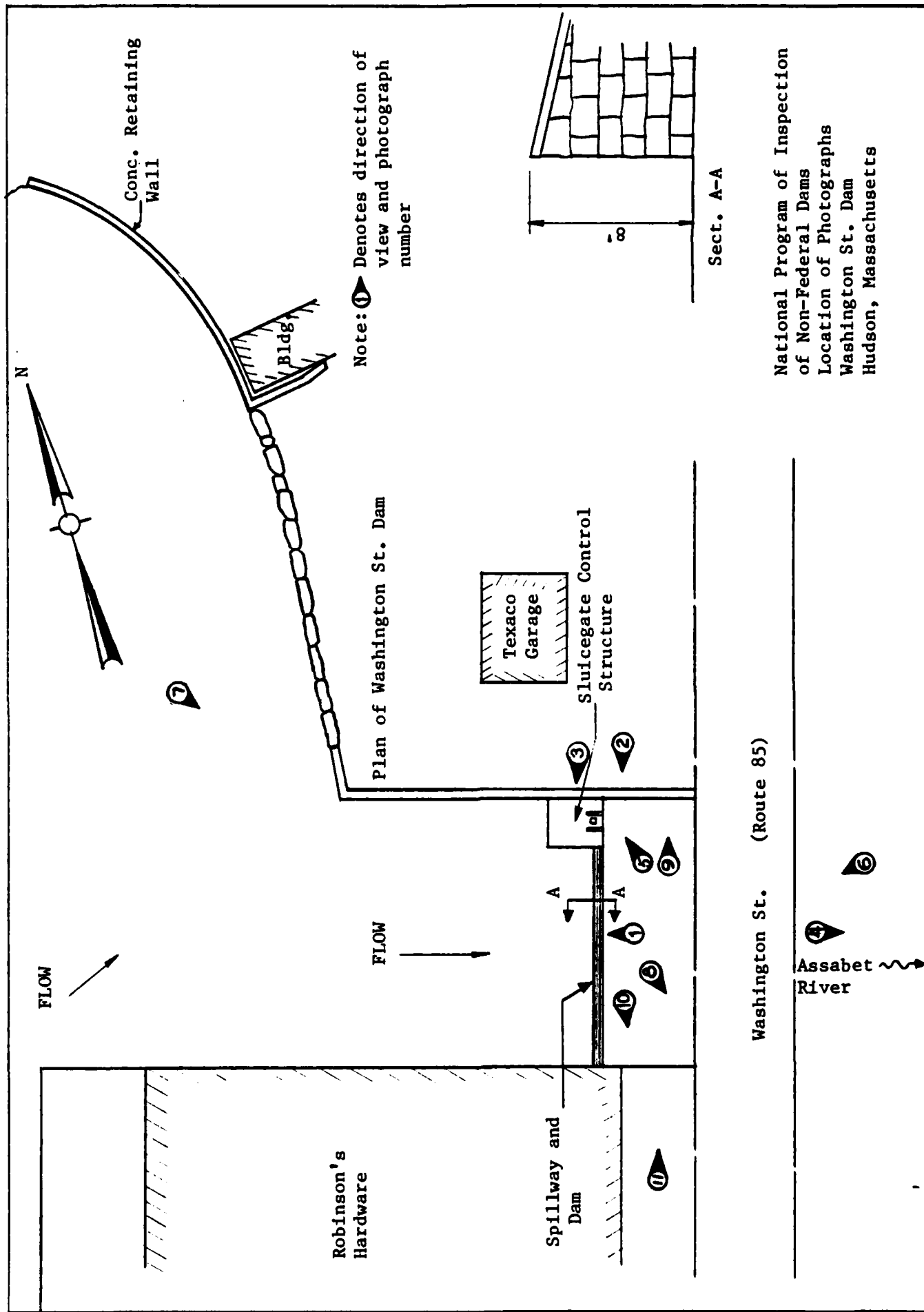
Page No.

Location of Photographs

C-1

PHOTOGRAPHS

<u>No.</u>	<u>Title</u>	<u>Page No.</u>
2.	View of Dam and Spillway from Left Abutment. Sluice Gate Operator is in Foreground.	C-2
3.	Sluice Gate Operator	C-2
4.	View of downstream channel from Washington Street Bridge	C-3
5.	View of Left Abutment showing Weep Holes and Control Works Outlet	C-3
6.	View of Downstream Face of Washington Street Bridge. Dam is in Background.	C-4
7.	View towards Crest of Spillway from North Shore of Storage Pool	C-4
8.	View of Upstream Face of Washington Street Bridge from Sluice Gate Control Structure	C-5
9.	View of Left Abutment and Left Side of Spillway from Washington Street Bridge	C-5
10.	View of Right Abutment and Right Side of Spillway from Washington Street Bridge	C-6
11.	Depression in Sidewalk Adjacent to Washington Street and Downstream Channel Right Wall	C-6



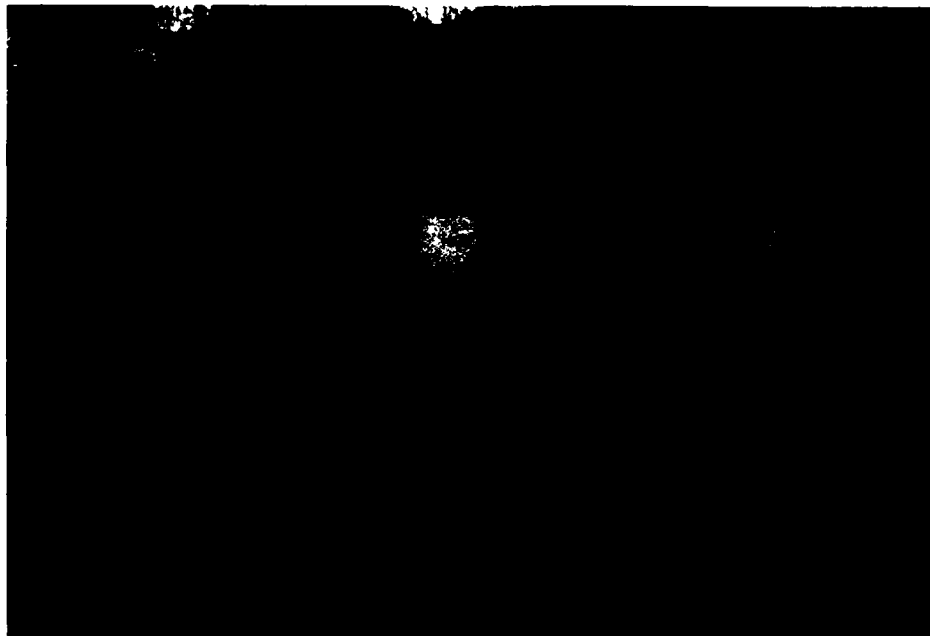
National Program of Inspection
of Non-Federal Dams
Location of Photographs
Washington St. Dam
Hudson, Massachusetts



2. View of dam and spillway from left abutment.
Sluice gate operator is in foreground.



3. Sluice gate operator.



4. View of downstream channel from Washington St. Bridge.



5. View of left abutment showing weep holes and control works outlet.



6. View of downstream face of Washington St. Bridge.
Dam is in background.



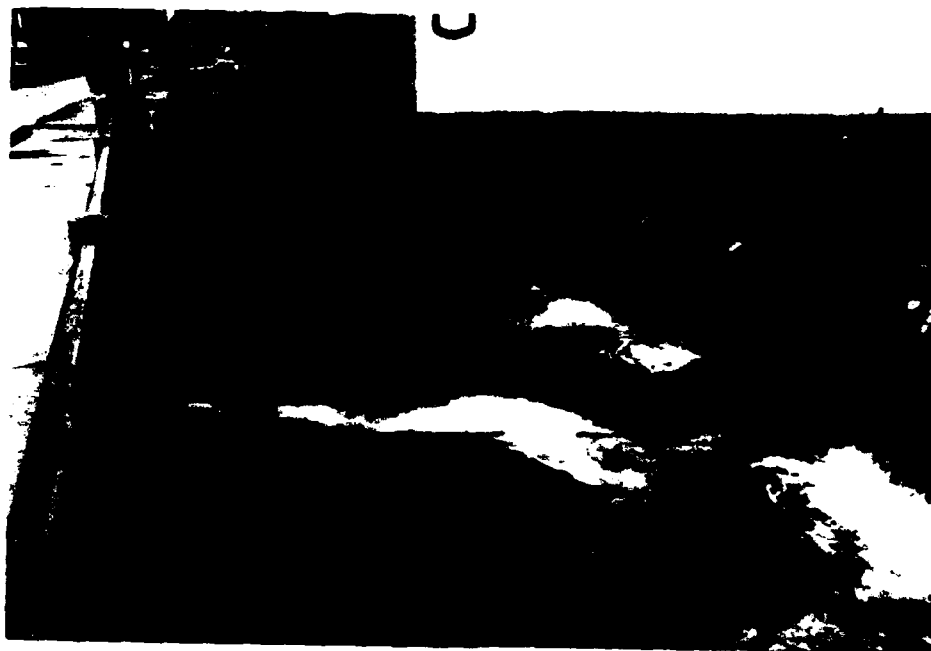
7. View of
downstream face of Washington St. Bridge.



8. View of upstream face of Washington St. Bridge from sluice gate control structure.



9. View of left abutment and left side of spillway from Washington St. Bridge.



10. View of right abutment and right side of spillway
from Washington St. Bridge.



11. Depression in sidewalk adjacent to Washington St.
and downstream channel right wall.

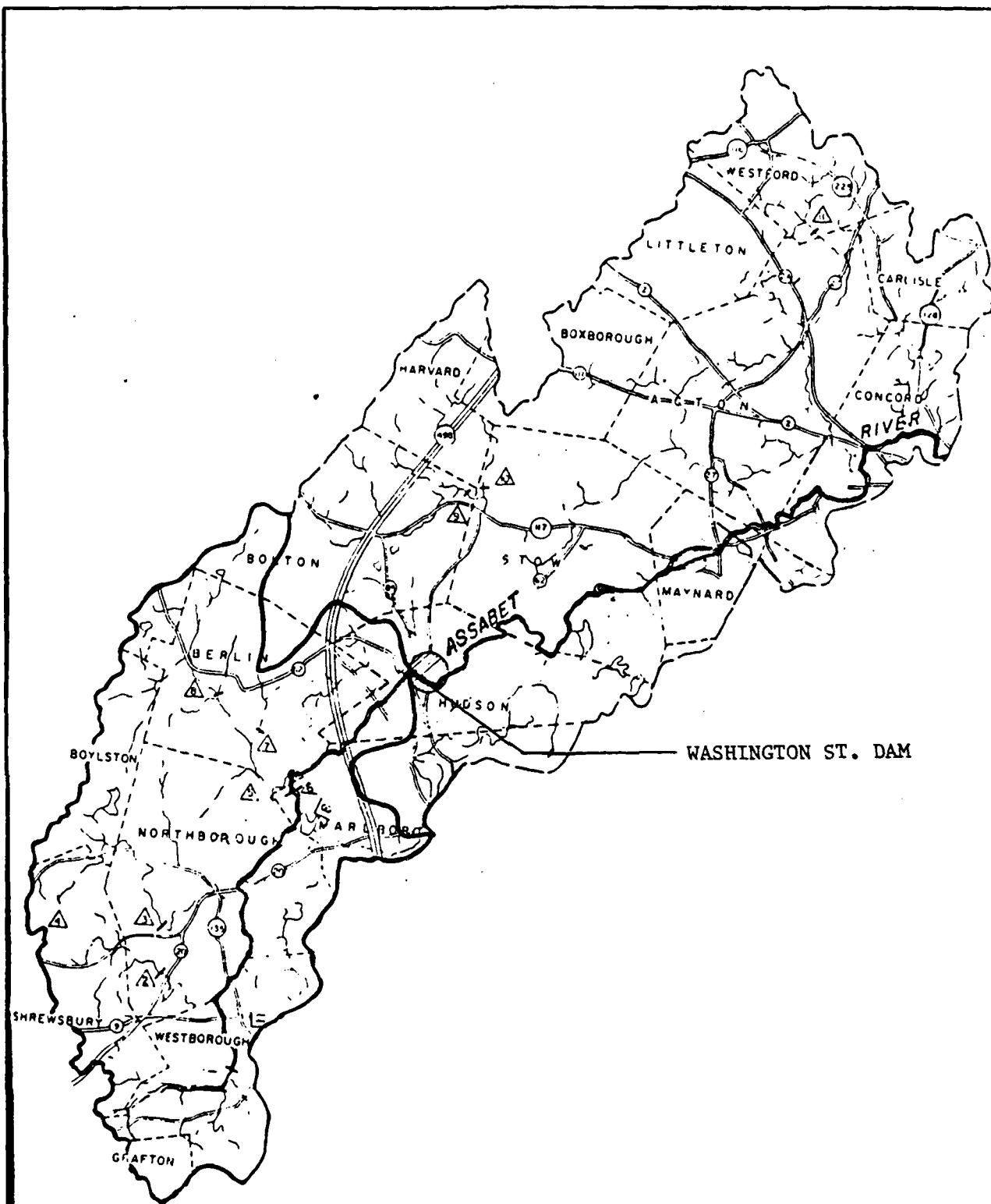
APPENDIX D

OUTLINE OF DRAINAGE AREA AND
HYDRAULIC COMPUTATIONS

COMPUTATIONS

Page No.

Drainage Area Map	D-1
Drainage Area	D-2
Impact Area	D-3
Dam Failure Analysis	D-4
Stage-Discharge Relationships	D-5
Size Classification, Hazard Potential and Test Flood Determination	D-17
Flood Routing	D-19
Tailwater Analysis	D-20



DAM WASHINGTON ST. DAM

IDENTIFICATION NO. 00447



DRAINAGE AREA MAP
FLOOD PLAIN INFORMATION
TECHNICAL REPORT

APPROX. SCALE: 1" = 3 miles

APPENDIX D-1

CAMP DRESSER & MCKEE
Environmental Engineers
Boston, Mass.

CLIENT CofE
PROJECT Wash St Dam
DETAIL Drainage Area

JOB NO. 380-5-17
DATE CHECKED 14 Mar 79
CHECKED BY WAK

PAGE 1/1
DATE 7/3/79
COMPUTED BY P&E

WASHINGTON ST. DAM, ASSABET RIVER

DRAINAGE AREA

40,757.1 Acres

63.68 mi²

WATER SURFACE AREAS

H₂O

55.01 A

.0861 mi²

at Elev 206.4

EL. 210

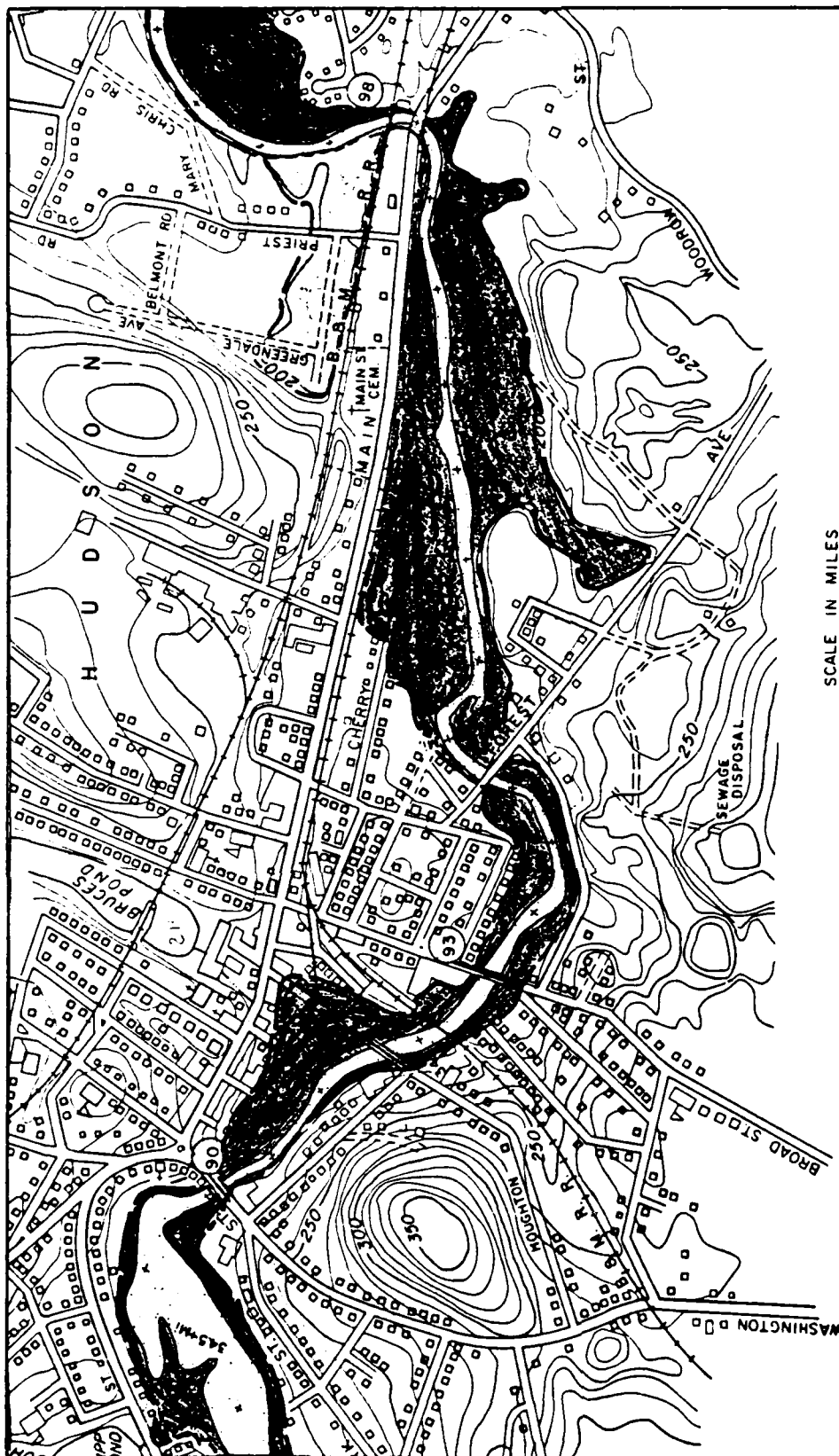
210.28 A

.3286 mi²

EL. 220

676.77 A

1.057 mi²



SCALE IN MILES
0.1 0 0.1 0.2 Mi.

APPROXIMATE LIMITS OF DOWNSTREAM FLOODING

DAM WASHINGTON STREET

IDENTIFICATION NO. 00447



IMPACT AREA

APPROX. SCALE: 1" = 530'

APPENDIX D- 3

Dam Failure Analysis

$$Q_{p1} = \frac{8}{27} W_b \sqrt{g} Y_o^{3/2}$$

where Q_p = Peak Failure Outflow

W_b = Breach width

g = 32.2

Y_o = River Bed to ws at time of failure over dam

Spillway width = 61'

$W_b = .4 (61) = 24.4'$ say 25'

$Y_o = 8'$ (toe dam to crest) + 7' (max ht report over crest)
= 15'

$$Q_{p1} = \frac{8}{27} \times 25 \times 32.2^{1/2} \times 15^{3/2}$$

= 2442 cfs say 2440 cfs

Add to this flow over remaining portion

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

$C = 3.3$

$L = 36'$

$H = 2'$

$$Q = 3.3 \times 36 \times 2^{3/2} = 2200 \text{ cfs}$$

Total flow = 2440 + 2200 = 4640 cfs.

CLIENT Cof E
 PROJECT Wash. St Dam
 DETAIL Dam Ecol / Arch

JOB NO 380
 DATE CHECKED 4/11/79
 CHECKED BY VED

PAGE 2/18
 DATE 14/10/79
 COMPUTED BY WAK

Stage- Discharge Ratings for downstream reaches

From field measurements:

Ground elev at Washington St. Bridge River mile 34.27

$$= 196.6' \pm$$

Ground elev at Houghton St. Bridge River mile 34.03

$$= 194.4' \pm$$

$$\Delta \text{ Elev} = 2.2'$$

$$\text{avg. slope} = \frac{2.2'}{1267} = .0017$$

Capacity of three arches on Washington St. bridge

$$Q = \frac{1.486}{n} \times A \times R^{4/3} \times S^{1/2}$$

$$n = .02$$

$$S = .0017$$

at 2 ft flow $A = 36.3 \text{ ft}^2$ $W_p = 22$ $R = \frac{36.3}{22} = 1.65$

$$Q = \frac{1.486}{.02} \times 36.3 \times 1.65^{4/3} \times .0017^{1/2}$$

$$= 74.3 \times 36.3 \times 1.396 \times .0412$$

$$= 1155 \text{ cfs}$$

$$3 \text{ Openings} = 1155 \times 3 = 465 \text{ cfs}$$

at 4 ft flow $A = 71.0 \text{ ft}^2$ $W_p = 26.2$ $R = 2.71$

$$Q = 74.3 \times 71.0 \times 2.71^{4/3} \times .0412$$

$$= 422 \text{ cfs}$$

$$3 \text{ Opening} = 422 \times 3 = 1266 \text{ cfs}$$

CLIENT C of E
 PROJECT Wash St Dam
 DETAIL Dam Failure Ana

JOB NO 30-5-17
 DATE CHECKED 4/11/79
 CHECKED BY JED

PAGE 3/18
 DATE 1479/79
 COMPUTED BY WAK

At 6 ft flow $A = 102.5 \text{ ft}^2$ $W_p = 30.8$ $R = 3.33$

$$Q = 74.3 \times 102.5 \times 3.33^{1/3} \times .0412$$

$$= 700 \text{ cfs}$$

$$3 \text{ openings} = 700 \times 3 = 2100 \text{ cfs}$$

At 8 ft flow $A = 125 \text{ ft}^2$ $W_p = 37.2$ $R = 3.36$

$$Q = 74.3 \times 125 \times 3.36^{1/3} \times .0412$$

$$= 858 \text{ cfs}$$

$$\text{Three openings} = 858 \times 3 = 2574 \text{ cfs}$$

At 9 ft flow (Arch's flowing full)

$A = 127 \text{ ft}^2$ $W_p = 46.2$ $R = 2.74$

$$Q = 74.3 \times 127 \times 2.74^{1/3} \times .0412$$

$$= 761 \text{ cfs}$$

$$3 \text{ culverts} = 2283 \text{ cfs}$$

For Pressure flow, assume downstream is free outfall, flowing full

$$Q = C A_o \sqrt{2g(h_1 - h_2)}$$

where $C = 0.8$

$D_o = 127$

h_1 = head at upstream end

h_2 = head at downstream end

For full over lap

$h_1 = 10$

$D_o = 127$

$h_2 = 9$

$$Q = 0.8 (127) \sqrt{2(32.2)} \parallel$$

$$= 815 \text{ cfs}$$

$$3 \text{ Culverts} = 2445 \text{ cfs}$$

CAMP DRESSER & MOORE INC.

CLIENT CofE
 PROJECT Wash St Dam
 DETAIL Dam Fail Anal

JOB NO 380-5-17
 DATE CHECKED 4/11/79
 CHECKED BY JED

PAGE 4/19
 DATE 14/12/79
 COMPUTED BY WAK

For 2 ft over top

$$Q = .8 (127) \sqrt{2(32.2)2}$$

$$= 1153$$

3 culverts = 3459 cfs

$h_1 = 11$

$h_2 = 9$

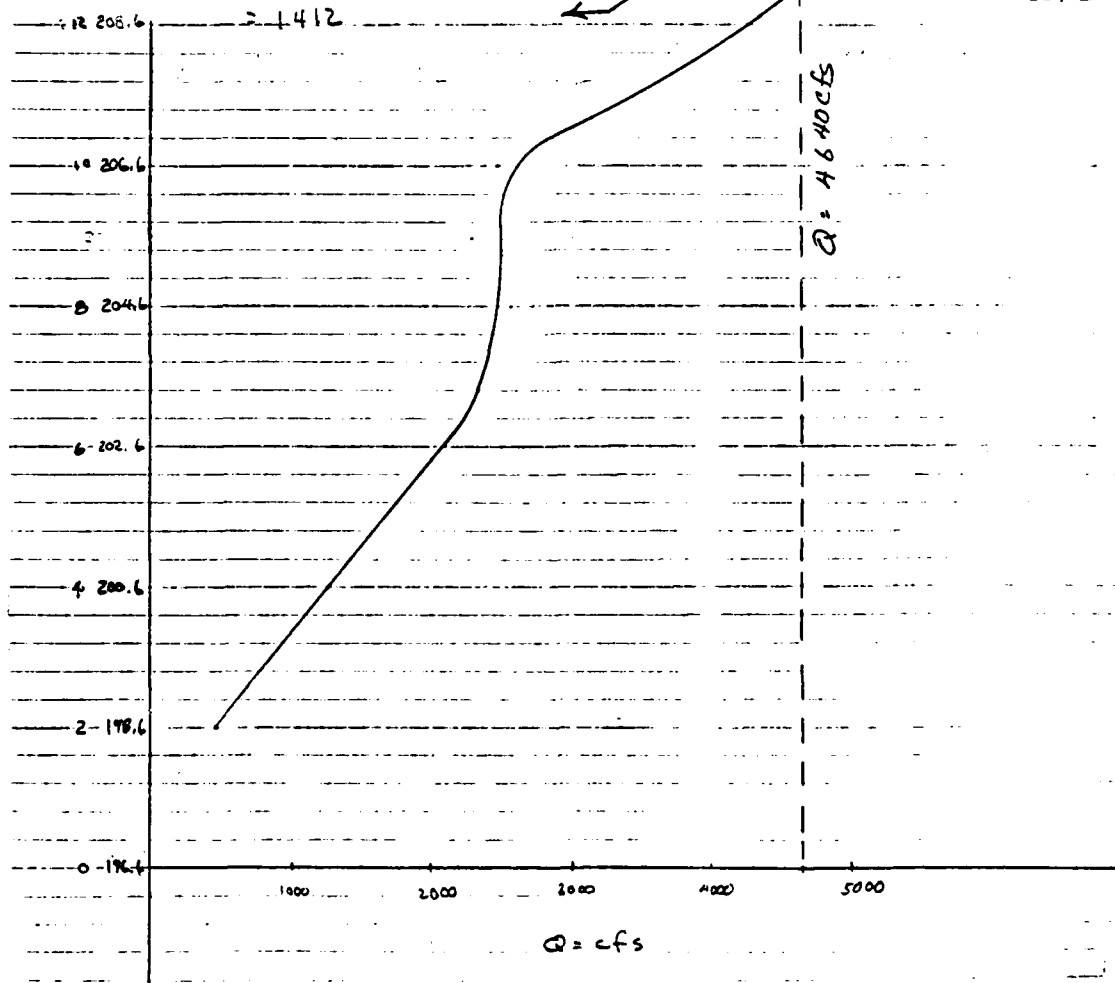
For 3 ft over top

$$Q = .8 (127) \sqrt{2(32.2)(3)}$$

3 culv = 4236 cfs

A' overtop = 4892 cfs

Elev 209.2



CLIENT C of E
 PROJECT Wash St Dam
 DETAIL Fail. Anal

JOB NO 380-5-17
 DATE CHECKED 4/11/79
 CHECKED BY JED

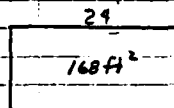
PAGE 5/18
 DATE 14 Mar 79
 COMPUTED BY WAK

With dam failure, as calc. on page 1,
 flow will be approx 3.6' over top of arch.

Houghton St Bridge - Approx 1,270 feet downstr.
 from Washington St.

Typ. opening is

Total of 3 openings



$n = .02$
 $sl = .0017$

$$Q = \frac{1.49}{.02} \times A \times R^{4/3} \times S^{1/2}$$

Depth = 2' $A = 48$ $wp = 28$ $R = 1.21$

$$Q = \frac{1.49}{.02} \times 48 \times 1.21^{4/3} \times .0017^{1/2}$$

= 211 cfs times 3 openings = 633 cfs

Depth = 4' $A = 96 \text{ ft}^2$ $wp = 32$ $R = 1.3$

$$Q = \frac{1.49}{.02} \times 96 \times 1.3^{4/3} \times .0017^{1/2}$$

= 613 cfs times 3 openings = 1839 cfs

Depth = 6' $A = 144 \text{ ft}^2$ $wp = 36$ $R = 1.4$

$$Q = \frac{1.49}{.02} \times 144 \times 1.4^{4/3} \times .0017^{1/2}$$

= 1115 cfs times 3 openings = 3345 cfs

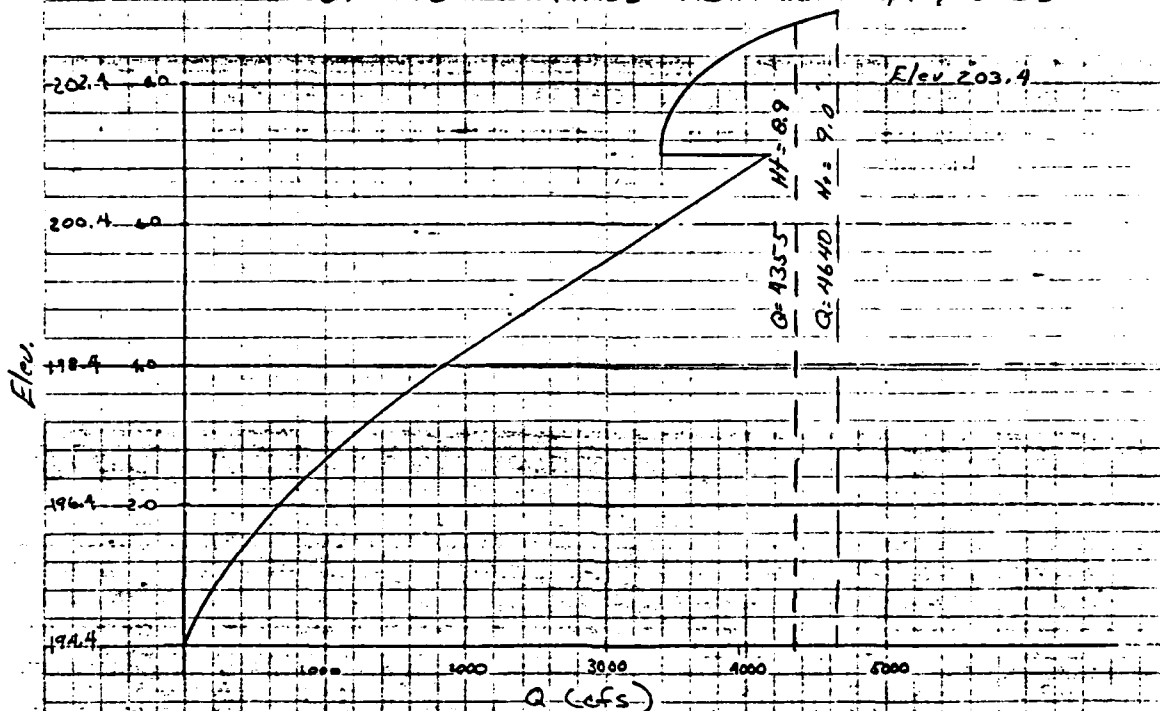
CLIENT C of E
 PROJECT Wash St. Dam
 DETAIL Fall Anal

JOB NO. 380-5-17DATE CHECKED 4/11/79CHECKED BY JEDPAGE 6/18DATE 15/7/79COMPUTED BY WAK

Depth 7' $B = 168 \text{ ft}$ $WP = 38$ $R = 4.42$

$$Q = \frac{1.49}{0.02} \times 168 \times 4.42^{1/3} \times 0.017^{1/2}$$

$$= 1390 \text{ cfs} \quad \text{times } 3 = 4170 \text{ cfs}$$



Pressure Flow

Assume downstream free disch.

Depth = 8' or 1' over top

$$Q = CA \sqrt{2gh} = 8 \times 168 \times \sqrt{2(32.2)(1)}$$

$$= 1020.6 \quad \text{times } 3 = 3235.7 \text{ cfs}$$

Depth = 9'

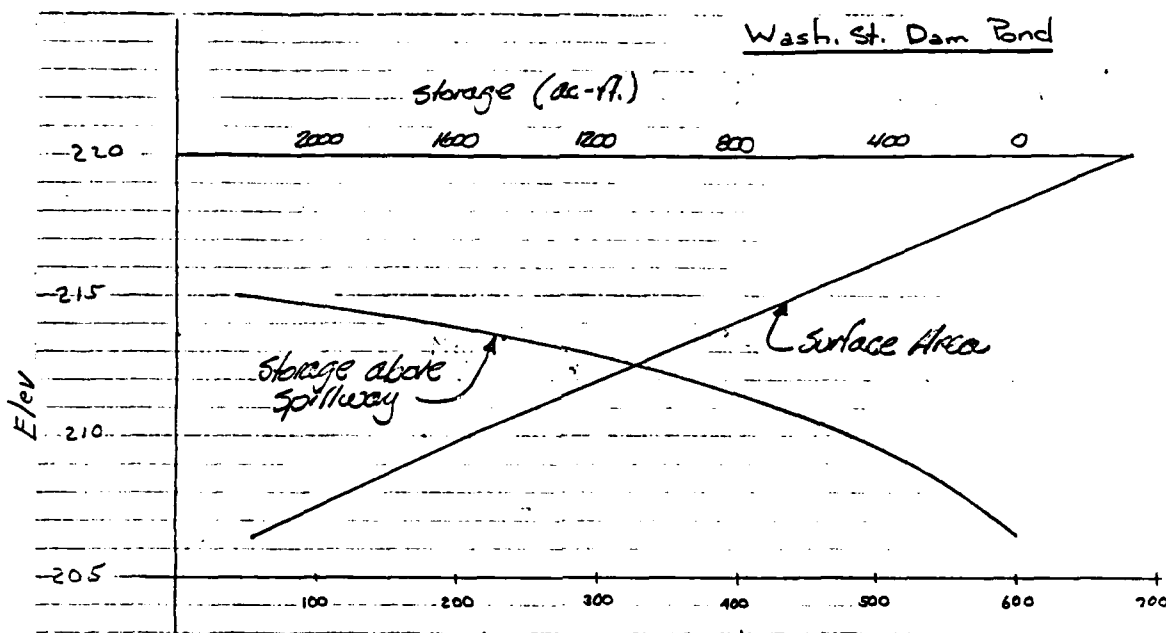
$$Q = CA \sqrt{2gh} = 8 \times 168 \times \sqrt{2(32.2)(2)}$$

$$= 1525.3 \quad \text{times } 3 = 4575.9 \text{ cfs}$$

CLIENT C of E
 PROJECT Wash St Dam
 DETAIL Fall Anal

JOB NO 380-5-17
 DATE CHECKED 4/11/79
 CHECKED BY JED

PAGE 2/1A
 DATE 15 Mar 79
 COMPUTED BY WAK



Surface Area (Heres)

Storage above Res.

Compute Q_{p2}

$$Q_{p2} = Q_{p1} \left(1 - \frac{V_1}{S}\right)$$

V_1 = Volume Water between Washington St & Houghton St.

S = Storage above reservoir at time of failure.

$$V_1 = L_{reach} \times depth_{avg} \times width_{avg}$$

$$= 1270' \times 9.0' \times 370' \div 43560 = 97.1 \text{ ac-ft}$$

$$S = \frac{Area_{205.7} + Area_{212.7}}{2} \times 7.0' = \frac{55 + 365}{2} \times 7.0 = 1470 \text{ ac-ft}$$

Assume storage below dam res_{res} avg depth $= 5 \times 5520 \times \frac{1}{3}$
 $= 100 \text{ ac-ft}$

$$\text{Total } S = 1570 \text{ ac-ft}$$

CLIENT Caf 5
PROJECT Wash. St Dam
DETAIL Failure Anal

JOB NO 380-5-17
DATE CHECKED 4/11/79
CHECKED BY JED

PAGE 8/18
DATE 15 Mar 79
COMPUTED BY WAK

$$Q_{p2} = 4640 \text{ cfs} \left(1 - \frac{97.1}{1570}\right)$$

$$Q_{p2} = 4355 \text{ cfs}$$

New V

$$V_L = 1270' \times 8.8' \times 370'$$

$$= 94.9 \text{ ac-ft}$$

$$Q_{p2} = Q_{p1} \left(1 - \frac{V_{avg}}{S}\right) \quad V_{avg} = \frac{97.1 + 94.9}{2}$$

$$= 4640 \left(1 - \frac{96.0}{1570}\right)$$

$$= 4355 \text{ cfs} \quad \checkmark$$

Broad St. Bridge 690' downstream of Houghton St.

3 openings 8' high x 26' wide each

Inv. 194.0

$$2' Flow \quad Q = 1.49 \times A \times R^{4/3} \times S^{1/2}$$

$$S = .0017$$

$$A = 82$$

$$n = .02$$

$$W_p = 30$$

$$R = 1.73$$

$$= 1.49 \times 52 \times 1.73^{4/3} \times .0017^{1/2}$$

$$= .02$$

$$= 230 \text{ cfs}$$

$$3 \text{ Openings} =$$

$$690 \text{ cfs}$$

4' Flow

$$Q = 24.5 \times 104 \times 3.06^{4/3} \times .0017^{1/2}$$

$$A = 104$$

$$W_p = 34$$

$$R = 3.06$$

$$3 \text{ openings} = 2019 \text{ cfs}$$

6' Flow

$$Q = 24.5 \times 156 \times 4.11^{4/3} \times .0017^{1/2}$$

$$A = 156$$

$$W_p = 38$$

$$R = 4.11$$

$$= 230 \text{ cfs}$$

$$3 \text{ Openings} = 3690 \text{ cfs}$$

APPENDIX D-11

CLIENT G of E
 PROJECT Wash St Dam
 DETAIL Fall Anal

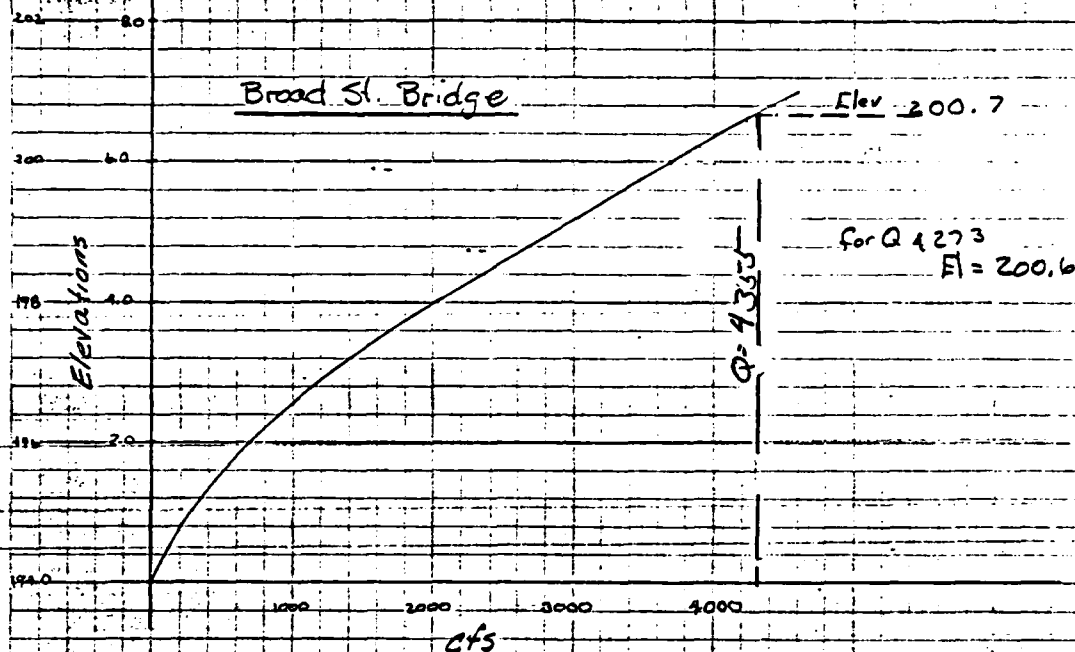
JOB NO 300-5-17
 DATE CHECKED 4/11/79
 CHECKED BY JED

PAGE 9/18
 DATE 15 Mar 79
 COMPUTED BY WAK

$$2' \text{ flow } Q = 74.5 \times 182 \times 4.55^{1/2} \times .0017^{1/2} \\ = 1535 \text{ cfs}$$

$$A = 182 \\ W_p = 40 \\ R = 4.55$$

$$3 \text{ Openings} = 3 \times 1535 = 4605 \text{ cfs}$$



With a Q_p of 4355 cfs, depth of flow is 6.7'

Volume between Houghton St. and Broad St.

$$L = 690' \\ D = 6.7' \\ W = 280' \quad Vol = 690' \times 6.7' \times 280' = 29.7 \text{ ac-ft}$$

$$\text{Storage} = 1570 \text{ ac-ft}$$

$$Q_p = Q_p \left(1 - \frac{29.7}{1570} \right)$$

$$4355 \text{ cfs} (1 - .0210)$$

$$= 4275 \text{ cfs}$$

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CLIENT

C of E

JOB NO

280-5-17

PAGE

10 / 1A

PROJECT

Wash. St Dam

DATE CHECKED

4/11/79

DATE

15 Mar 79

DETAIL

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Now Vol:

$$L = 690'$$

$$D = 6.6'$$

$$W = 280' \text{ avg}$$

$$\text{Vol} = 690 \times 6.6 \times 280 = 29.3$$

$$V_{\text{avg}} = \frac{29.7 + 29.3}{2} = 29.5$$

$$Q_{P3} = 4255 \left(1 - \frac{29.5}{1570} \right)$$

$$= 4273 \text{ cfs}$$

APPENDIX D-13

Reach No. 1 located mid-way between Washington St + Houghton St.

Try ws at 199 d = 5

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

$$S = .0017$$

$$n = .025$$

$$d = 5' \quad A = 125 \quad w_p = 51 \quad R = 2.45$$

$$Q = \frac{1.49}{.025} \times 125 \times 2.45^{2/3} \times .0017^{1/2}$$

$$= 558 \text{ cfs}$$

Try ws at 200

$$A = 195 \text{ ft}^2 \quad w_p = 91 \quad R = 2.14$$

$$Q = \frac{1.49}{.025} \times 195 \times 2.14^{2/3} \times .0017^{1/2}$$

$$= 795 \text{ cfs}$$

Try ws at 201

$$A = 307 \text{ ft}^2 \quad w_p = 137 \text{ ft} \quad R = 2.24$$

$$Q = \frac{1.49}{.025} \times 307 \times 2.24^{2/3} \times .0017^{1/2}$$

$$= 1292 \text{ cfs}$$

Try ws at 202

$$A = 467 \text{ ft}^2 \quad w_p = 187 \text{ ft} \quad R = 2.50$$

$$Q = \frac{1.49}{.025} \times 467 \times 2.50^{2/3} \times .0017^{1/2}$$

$$= 2114 \text{ cfs}$$

CLIENT C of E
 PROJECT Wash St Dgn
 DETAIL Fall Area

JOB NO 380-5-17
 DATE CHECKED 4/11/79
 CHECKED BY ED

PAGE 12/18
 DATE 16 Nov 79
 COMPUTED BY WAK

Try ws 2+203

$$A = 671 \text{ ft}^2 \quad w_p = 232 \text{ ft} \quad R = 2.89$$

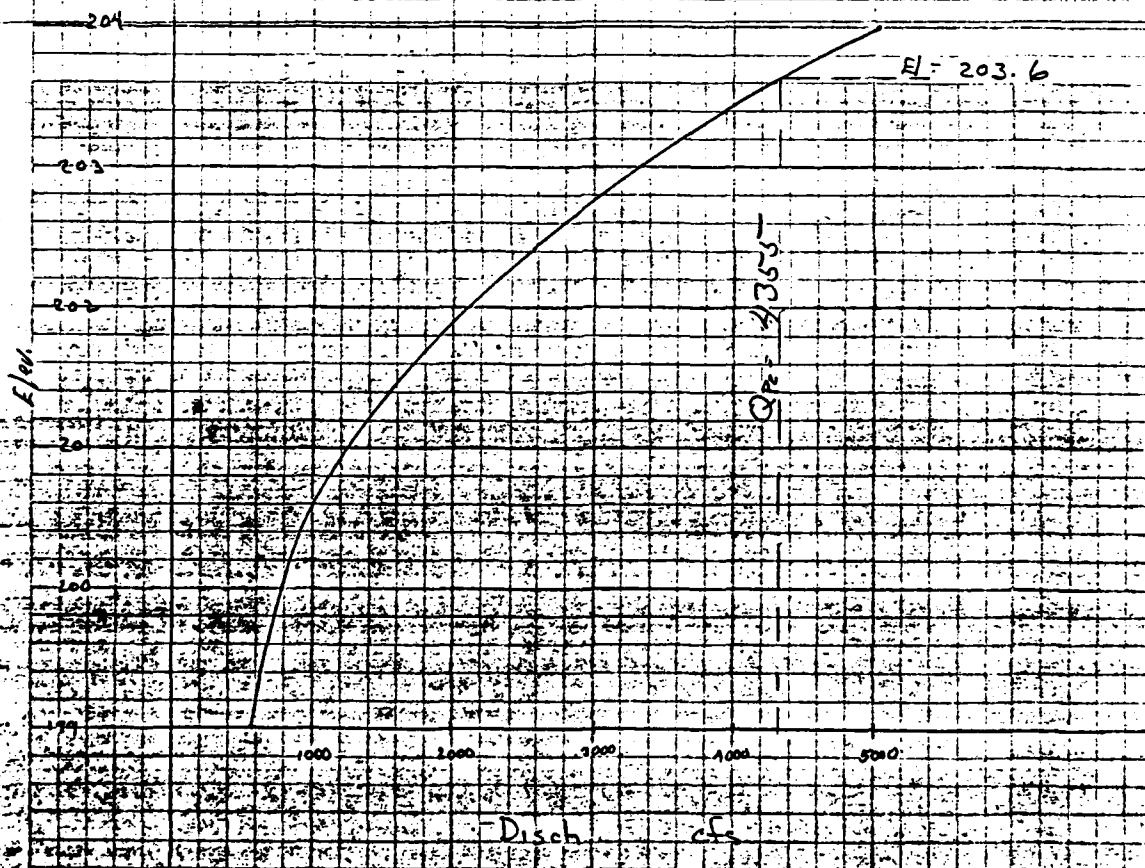
$$Q = \frac{1.49}{.025} \times 671 \times 2.89^{4/3} \times .0017^{1/2}$$

$$= 3345 \text{ cfs}$$

Try ws 2+204

$$A = 919 \text{ ft}^2 \quad w_p = 278' \quad R = 3.31$$

$$Q = \frac{1.49}{.025} \times 919 \times 3.31^{4/3} \times .0017^{1/2} = 5015 \text{ cfs}$$

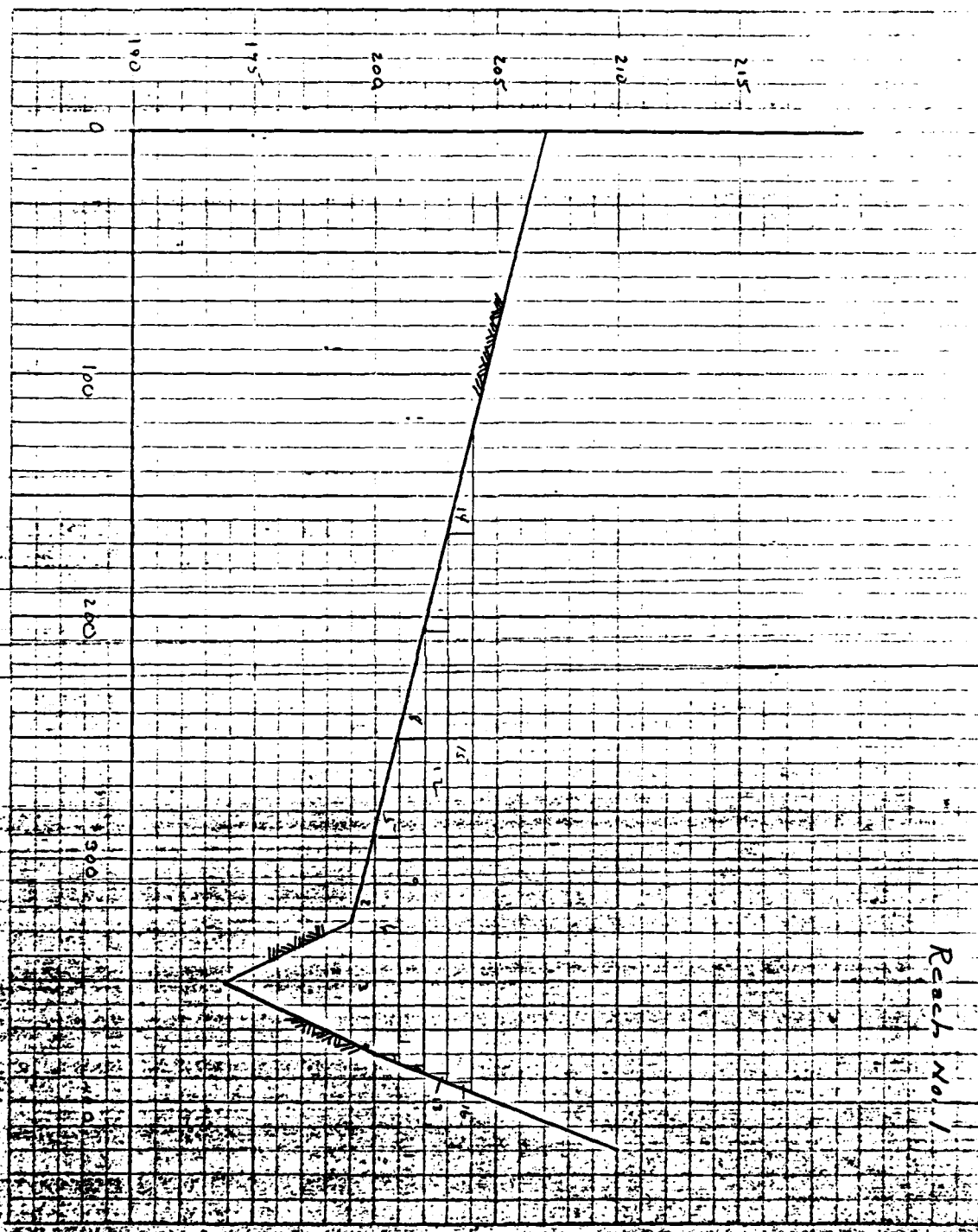


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PROJECT Wish St Dam
DETAIL Fall Am

JOB NO 380-5-17
DATE CHECKED 9/11/79
CHECKED BY JED

PAGE 13/18
DATE 16 Mar 79
COMPUTED BY WAK



Reach No 1

CLIENT C of I
PROJECT Wash St Dam
DETAIL Size Determin.JOB NO 380-S-17
DATE CHECKED 4/11/79
CHECKED BY JEDPAGE 14/18
DATE 17 Mar 79
COMPUTED BY WAK

From terrain analysis and physical investigation, if the dam should fail, some flooding will occur downstream. Loss of life is considered slight, with appreciable economic loss. Therefore the Washington St. dam is classified as Significant with regards to Hazard Potential.

Size Classification

Abutment Elev. 212.7Toe Dam = 197.7ht = 15.0' $\therefore$ Small

Storage above dam

Approx 1470 ac-ft $\therefore$ Intermediate

From Guidelines

Hydrologic Evaluation will be as follows:

Hazard = Significant Size = Intermediate

Test Flood

1/2 PMF to PMF

Since the storage is just slightly into the intermediate range, use 1/2 PMF.

CLIENT CofE
 PROJECT Wash St Dam
 DETAIL Test Flood

JOB NO 380-5-17
 DATE CHECKED 4/11/79
 CHECKED BY JED

PAGE 15/18
 DATE 19 MAR 79
 COMPUTED BY WAK

Test Flood

$$D.A. = 63.7 \text{ mi}^2$$

$$\text{less } 6.5 \text{ mi}^2 \quad (\text{D.A. to Assabet River Dam})$$

$$= 57.2 \text{ mi}^2$$

Assabet River can be classified as a very flat watercourse with much swamp land. From MPF graph, for a very flat watercourse the peak flow rate would be approximately 145 cfs/mi² for a peak flow of 8295 cfs.

1/2 MPF is therefore 4147 cfs.

Sec 158

In 1966 the Corps of Engineers published a Flood Plain Information report for the Assabet River. The Standard Project Flood (SPF) for this report was determined to have a flow rate of 5320 cfs at the Maynard Gage (116 mi²).

On a drainage area relationship, the SPF at the Washington St dam could be determined to be

$$Q_1 = \frac{A_1^{.894}}{A_2^{.894}} (1)$$

$$Q_2 = \frac{A_2^{.894}}{A_1^{.894}} (1)$$

$$Q_1 = \frac{63.7^{.894}}{116^{.894}} (1)$$

$$Q_2 = \frac{116^{.894}}{63.7^{.894}} (1)$$

$$Q_2 = 1.296$$

Where Q_1 = Flow / mi² upstr

Q_2 = Flow / mi² down

A_1 = D.A. upstr

A_2 = D.A. down

$$Q_2 = 5320 \text{ cfs} / 116 \text{ mi}^2 = 45.9 \text{ cfs/mi}^2$$

CLIENT C of EJOB NO 380-5-17PAGE 152/18PROJECT Wash St DamDATE CHECKED 4/11/79DATE 21 MAR 79DETAIL Test FloodCHECKED BY JEDCOMPUTED BY WAK

Effect of Routing $\frac{1}{2}$ PMF through Pond upstream
of dam.

$$Q_{P1} = 4147 \text{ cfs. at Elev } 212.9 \pm$$

$$\text{Storage above spillway} = 1566 \text{ ac-ft}$$

$$\text{D.A.} = 57.2 \text{ mi}^2 \text{ (} 63.7 \text{ mi}^2 \text{ less } 6.5 \text{ mi}^2 \text{ to upst. dam)}$$

$$= 36,608 \text{ ac.}$$

$$\frac{1566 \text{ ac-ft}}{36,608 \text{ ac}} = .04 \text{ ft} = .48" \text{ R.O.} = \text{STOR } 1$$

$$Q_{P2} = Q_{P1} \left(1 - \frac{\text{STOR}_1}{9.5} \right) = 4147 \left(1 - \frac{.48}{9.5} \right)$$

$$= 3940 \text{ cfs at elev } 213.5$$

$$\text{Storage} = 1510 \text{ ac-ft}$$

$$\frac{1510}{36,608} = .04 \text{ ft} = .48" \text{ R.O.}$$

$$Q_{P3} = 3940 \text{ cfs}$$

$$Q_1 = 1.296 \times 45.9 = 59.5 \text{ cfs/mi}^2$$

Total Flow = 3790 cfs = SPF Corps of Engineers
Report

According to published data,

$$\text{SPF} \approx 1/2 \text{ MPF.}$$

For the purposes of this investigation, the test flood will be 3790 cfs, as determined by Corps of Engineers. This flow will be the outflow from the dam.

$$\therefore \text{TEST FLOOD} = 3790 \text{ cfs}$$

Tailwater.

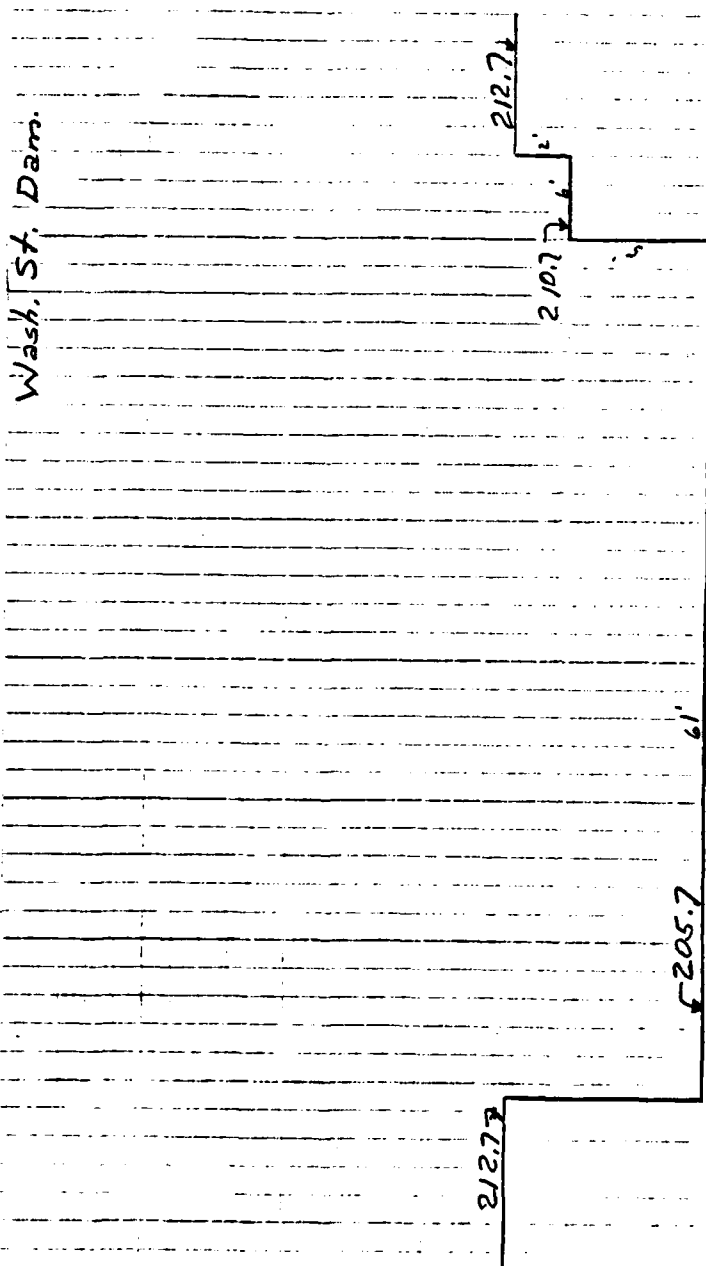
$Q = 3790 \text{ cfs}$ will be trying to pass under Washington St. Bridge. From rating curve, the w.s. upstream of the bridge, but downstream of the dam would approach Elev. 208.0 msl

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CLIENT C of E
PROJECT Wash St Dam
DETAIL Spillway Dam.

JOB NO. 380-5-17
DATE CHECKED 4/11/79
CHECKED BY VED

PAGE 17/18
DATE 20/11/79
COMPUTED BY WAK



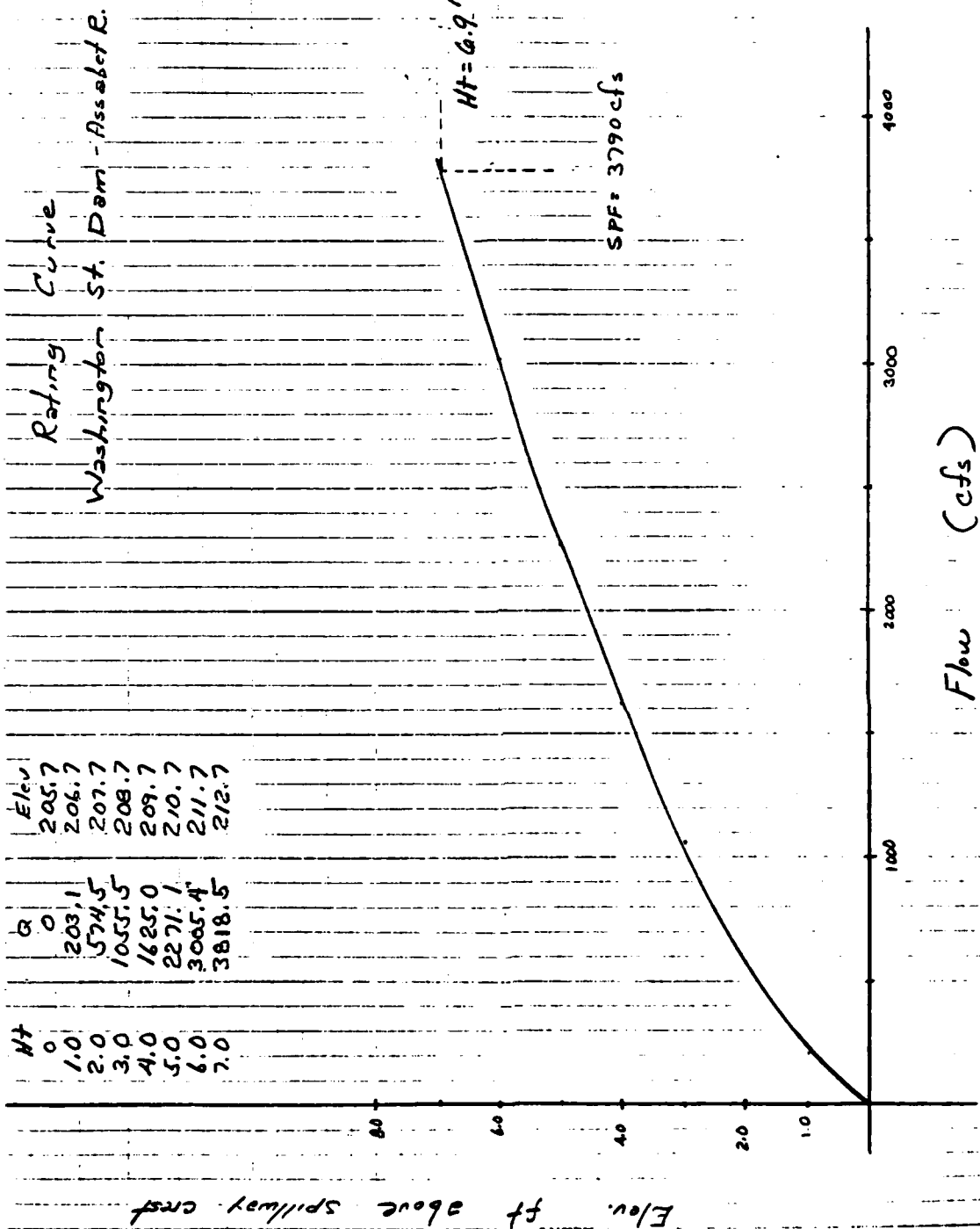
APPENDIX D-21

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 PROJECT Wash St Dam
 DETAIL Spillway Curve

JOB NO. 380-17-17
 DATE CHECKED 4/11/79
 CHECKED BY JED

PAGE 18/18
 DATE 20/12/77
 COMPUTED BY WAK



APPENDIX E
INFORMATION AS CONTAINED IN
THE NATIONAL INVENTORY OF DAMS

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7-85

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