MERRIMACK VALLEY FLOOD CONTROL

DEFINITE PROJECT REPORT

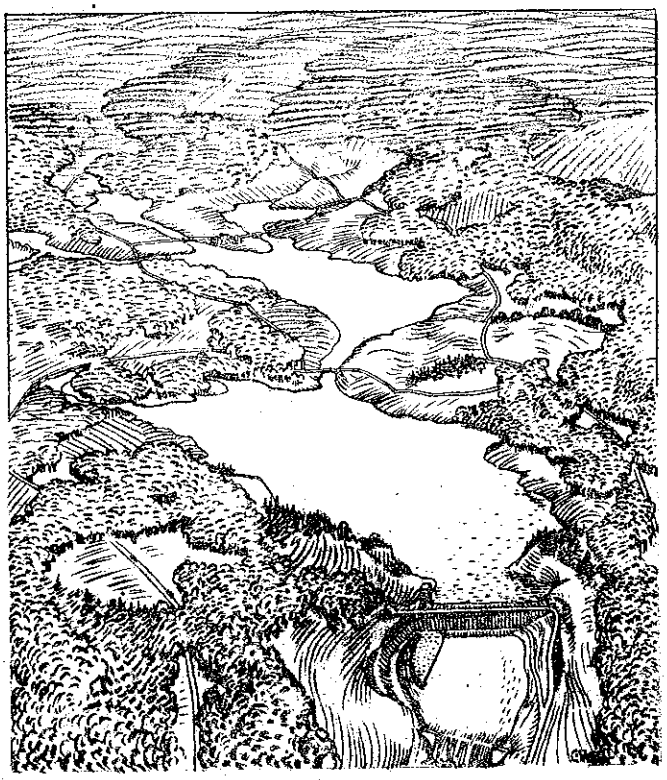
on

WEST PETERBORO RESERVOIR

NUBANUSIT BROOK

NEW HAMPSHIRE

MAY... ...1945



CORPS OF ENGINEERS...U. S. ARMY

U. S. ENGINEER OFFICE . BOSTON, MASSACHUSETTS

JOHN E. ALLEN

DEFINITE PROJECT REPORT

ON

WEST PETERBORO RESERVOIR
NUBANUSIT BROOK
NEW HAMPSHIRE

PREPARED IN THE U.S. ENGINEER OFFICE,

BOSTON, MASSACHUSETTS. DATED MAY, 1945

APPROVED BY THE CHIEF OF ENGINEERS 1945

E.D. FILE _____

THIS REVISED REPORT SUPERSEDES REPORT
DATED 5 AUGUST, 1940 (REVISED 14 DEC., 1940)

DATE	NEW PAGES OR DRAWINGS	DATE APPROVED BY C. OF E.

DEFINITE PROJECT REPORT

on

WEST PETERBORO RESERVOIR

NUBANUSIT BROOK

NEW HAMPSHIRE

Prepared in the U. S. Engineer Office,

Boston, Massachusetts, dated May 1945
revised June 1946

Approved by the Chief of Engineers _____ 1946

E. D. File

REVISIONS

(See following page)

TABLE OF REVISIONS

TEXT OF REPORT

Revised	Page	Par. No.	Title	Date Approved by C. of E.
June 1946	a1	3	Stream Flow Data	
June 1946	a2	4	Dam	
		5	Outlets	
		6	Spillway	
		8	Quantities	
June 1946	a3	9	Estimated Cost	
June 1946	b1		Section B - Syllabus	
June 1946		2	Investigations	
	4	2b	Previous Investigations	
June 1946		3	Local Cooperation	
	6	3b	Status of Approval by State of New Hampshire	
June 1946		4	Definite Project Plan	
	7	4d(2)	Spillway Design Flood	
	8	4f	Estimated Cost	
June 1946		5	Structures and Improvements	
	9	5b	Outlet Works	
	9&10	5c	Spillway	
	10	5d	Culverts	
June 1946		7	Hydroelectric Power	
	11	7b	Ratio of Benefits to Carrying Charges	
June 1946		10	Construction Time Required and Schedule of Operations	
	13	10c	Funds Required by Fiscal Years	
	13&14	10d	Preparation of Plans and Specifications	
June 1946	14&15	11	Clearance with Other Agencies	
June 1946	15&16	12	Operation and Maintenance	
June 1946	16&17	13	Malaria Control	
June 1946		14	Cost Estimates	
	17	14a	Total Costs	
	17,18,&19	14b	Unit Costs	
	20	14c	Carrying Charges	
June 1946	21&22	15	Economic Study	

APPENDICES

APPENDIX I - HYDROLOGY

Revised	Page	Par. No.	Title	Date Approved by C. of E.
June 1946	I-2&I-3	d	Project Description	
June 1946	I-7	i	Stream Flow Data	
June 1946	I-10	m	Reservoir Routing Assumptions	
June 1946	I-11	o	Computed Spillway Floods	
June 1946	I-12	p	Spillway Design Flood	
June 1946	I-13	r	Determination of Top of Dam	
June 1946	I-14&I-15	v	Operation During Floods	
June 1946	I-15&I-16	w	Effect on 1936 and 1938 Floods	
June 1946	I-16	x	Time to Empty Reservoir	
June 1946	I-17	y	Backwater in Reservoir	
June 1946	I-17&I-18	z	Cofferdam Design	
June 1946	I-18&I-19	aa	Stage Frequency Curve (Plate I-22)	

APPENDIX II

Exploration, Geology, Construction Materials, Soil Data, and Analyses

June 1946		a	Exploration	
	II-2&II-3	a(2)(c)	Foundation Exploration of Spillway	
June 1946		b	Geology	
	II-3	b(1)(a)	Nubanusit Valley	
June 1946		c	Construction Materials	
	II-5	c(1)	Materials Required	
	II-6	c(2)	Materials Available in Required Excavation	
	II-7	c(3)(d)	Rock for Slope Protection (Deleted)	
June 1946	II-8&II-9	c(4)	Materials Summary	

APPENDIX III - HYDRAULIC DESIGN

June 1946	III-1	b	Design Criteria	
June 1946	III-1&III-2	d	Size of Gates and Conduit	
June 1946	III-2	e	Stilling Basin	
June 1946	III-3	f	Description	
		g	Spillway Weir	
June 1946	III-3&III-4	h	Approach Channel	
June 1946	III-4&III-5	i	Discharge Channel	

APPENDIX IV - STRUCTURAL DESIGN

June 1946		c	Description of Structure	
	IV-2	c(3)	Spillway	

APPENDIX V - HYDROELECTRIC POWER

Revised	Page	Par. No.	Title	Date Approved by C. of E.
June 1946		2	Estimated Cost	
	V-1	2b	Costs	
June 1946	V-2	3	Basis of Cost Analysis	
		4	Conclusion	
June 1946	V-3&V-4		Table A	

APPENDIX VI - RELOCATIONS

June 1946	VI-1	a	Introduction	

Added VII-1 APPENDIX VII - MALARIA CONTROL STUDIES

DRAWINGS

Revised	Plate No.	Title	Date Approved by C. of E.
June 1946	A-6	Aerial Photo with Location of Spillway	
June 1946 (Added)	I-17A	Spillway Design Flood	
June 1946	I-19	Time to Empty Reservoir	
June 1946	I-20	Reservoir Operation Effect on March 1936 Flood	
June 1946	I-21	Reservoir Operation Effect on Sept. 1938 Flood	
June 1946	I-22	Stage Frequency Curve	
June 1946	II-1	Plan of Foundation Exploration at Site	
June 1946	II-4	Record of Foundation Exploration at Dam Site and Borrow Area	
June 1946	II-5	Profile and Typical Sections of Dam and Outlet	
June 1946	II-6	Geological Profiles at Dam Site	
June 1946	II-7	Plan of Foundation Exploration at Spillway	
June 1946	II-8	Record of Foundation Exploration at Spillway	
June 1946	II-9	General Plan and Typical Sections at Spillway	
June 1946	II-10	Plan and Profile for Spillway Discharge and Protective Dike	
June 1946	III-1	Conduit Discharge Curves	
June 1946	III-2	Hydraulic Data for Conduit	
June 1946	III-3	Stilling Basin	

Revised	Plate No.	Title	Date Approved by C. of E.
June 1946 (Added)	III-4	Spillway Discharge Curve	
June 1946	IV-1	General Plan of Dam and Outlet	
June 1946	IV-2	Profile and Typical Sections of Dam and Outlet	
June 1946	IV-3	Sections of Outlet Works	
June 1946	IV-4	General Plan and Typical Sections at Spillway	
June 1946	IV-5	Plan and Profile for Spillway Discharge and Protective Dike	
June 1946	IV-6	Stability Analyses Concrete Structures	

DEFINITE PROJECT REPORT

WEST PETERBORO RESERVOIR

MERRIMACK VALLEY FLOOD CONTROL

TABLE OF CONTENTS

SECTION A	PERTINENT DATA	Pages a1 to a3
SECTION B	SYLLABUS	Page b1
SECTION C	TEXT OF REPORT:	

Paragraph	Title	Page
1.	Project Authorization	1
2.	Investigations	3
3.	Local Cooperation	5
4.	Definite Project Plan	6
5.	Structures and Improvements	9
6.	Foundations	10
7.	Hydroelectric Power	11
8.	Relocations	11
9.	Availability of Construction Materials	12
10.	Construction Time Required and Schedule of Operations	12
11.	Clearance with Other Agencies	14
12.	Operation and Maintenance	15
13.	Malaria Control	16
14.	Cost Estimates	17
15.	Economic Study	21
16.	Recommendations	23

SECTION D APPENDICES:

Appendix	Title	Page
I	Hydrology	I-1 to I-18
II	Exploration, Geology, Construction Materials, Soil Data and Analyses	II-1 to II-18
III	Hydraulic Design	III-1 to III-4
IV	Structural Design	IV-1 to IV-3
V	Hydroelectric Power	V-1 to V-4
VI	Relocations	VI-1 to VI-2
VII	Malaria Control Studies	VII-1

(Revised June 1946)

SECTION A.

PERTINENT DATA

DEFINITE PROJECT REPORT
WEST PETERBORO RESERVOIR

SECTION A - PERTINENT DATA

There is presented in the following table a summary of the pertinent data and governing criteria for the selection, design, construction, operation and cost of the West Peterboro Reservoir, Merrimack River Basin, New Hampshire, as described in this project report:

1. Project Location . . . . . Nubanusit Brook, tributary of Contoocook River approximately 15 miles east of Keene, New Hampshire, and 2.5 miles northwest of Peterboro, New Hampshire, approximately at river mile 161.3 above the mouth of the Merrimack River.

2. Reservoir Data

Drainage area	44 sq. miles
Flood control storage capacity	12,800 acre-feet
Inches of storage capacity on drainage area	5.5
Reservoir area at elevation of spillway crest:	
Wooded land, containing thin stand of mixed hardwood (45%)	382 acres
Wooded land, containing fair stand of mixed hard and soft woods, small amount merchantable (18%)	153 acres
Tillable and cleared land (4%)	34 acres
Water and swamp land (33%)	281 acres
Total Reservoir Area (100%)	850 acres
Length of Reservoir	3.7 miles
Maximum wave fetch	1.5 miles

3. Stream Flow Data

Average flow	80 c.f.s.
Average annual flood	800 c.f.s.
Flood of 5 November 1927 (estimated)	1,000 c.f.s.
Flood of 19 March 1936	4,140 c.f.s.
Flood of 21 September 1938	4,100 c.f.s.
Spillway design flood - inflow to reservoir .	36,300 c.f.s.
Spillway design flood - outflow	17,800 c.f.s.

(Revised June 1946)

4. Dam

Type	Rolled earth
Top of dam elevation	967 feet (M.S.L.)
Total length of dam	1,100 feet
Maximum height of earth embankment	69 feet
Top width of earth dam	25 feet
Freeboard	5.2 feet
Allowance for failure of upstream dams	1.0 foot
Power development	None

5. Outlets

Location	Single conduit through embankment
Size of conduit	7'-0" x 7'-0"
Number of service gates	3
Type of gates	Mechanically operated sluice gates
Size of gate openings	4'-0" x 5'-0"
Elevation of gate sills	904 feet (M.S.L.)
Maximum discharge capacity (Res.El. 946)	1,680 cu. feet per sec.
Downstream channel capacity	1,000 cu. feet per sec.
Time of emptying 90% of capacity	8 days

6. Spillway

Type of spillway	Low concrete weir
Crest elevation	946 feet (M.S.L.)
Length	100 feet
Spillway design flood discharge	17,800 cubic feet per second
Maximum reservoir elevation (Spillway design flood)	980.8 feet (M.S.L.)

7. Foundations

Dam, General	Silty till.
Dam, Core	Bed rock on west abutment and stream channel - silty till on east abutment.
Outlet works	Bed rock.
Spillway	Bed rock.

8. Quantities

	Earth Dam	Spillway
Embankment		
Impervious fill	85,000 C.Y.	33,000 C.Y.
Pervious fill	107,000 C.Y.	-
Rock slope protection	19,000 C.Y.	10,000 C.Y.
Sand and gravel fill	24,500 C.Y.	4,500 C.Y.
Total Embankment	235,500 C.Y.	47,500 C.Y.

8. Quantities (Cont'd)

	Earth Dam	Spillway
Excavation - earth	137,000 C.Y.	340,000 C.Y.
Excavation - rock	10,000 C.Y.	30,000 C.Y.
Concrete	4,100 C.Y.	3,300 C.Y.

9. Estimated Cost (Including Contingencies Overhead, etc.)

Reservoir clearing	$ 5,300
Land, relocations, etc.	209,000
Construction of dam and spillway	1,511,700
Total	$1,726,000

Cost per Acre-Foot of Total Storage $134.84

(Rev. June 1946)

SECTION B.

SYLLABUS

DEFINITE PROJECT REPORT

WEST PETERBORO RESERVOIR

SECTION B - SYLLABUS

The West Peterboro Reservoir was approved by the Chief of Engineers on 30 April 1940 as a part of the comprehensive flood control plan for the Merrimack River Basin authorized by the Flood Control Act approved 22 June 1936 and amended by the Flood Control Act approved 28 June 1938.

The reservoir site is located on the Nubanusit Brook, New Hampshire, and is designed to protect the lower reaches of the Nubanusit Brook and the town of West Peterboro from all floods, and to reduce flood stages on the Contoocook River and the Merrimack River from floods originating in the drainage basins of the Nubanusit and Stanley Brooks. The proposed dam consists of an earth-filled embankment section with the concrete outlet works at the dam site on the south side of the reservoir and a saddle spillway with an earth-filled dike on the north side. The embankment section is 1100 feet long and has a maximum height of 69 feet. It is constructed of rolled earth fill with a central impervious earth core which extends to rock on the west side and into the impervious till on the east side of the present river. The upstream face of the embankment is protected by a rock blanket; the downstream slope is topsoiled and seeded, and the toe drain is covered with a layer of hand-placed riprap. The outlet works include an intake structure, conduit and stilling basin. The conduit passes through the embankment and has three gated outlets. The intake structure is connected with the embankment by an access bridge. The spillway is located in a remote saddle and is a concrete gravity section which discharges onto an excavated rock floor. A steel girder highway bridge with two concrete piers crosses the approach channel approximately 370 feet above the spillway weir. This arrangement was found to be more economical than the previous arrangement whereby bridge abutments and spillway weir abutments were combined. An earth-filled dike is provided downstream from the spillway discharge channel to prevent flooding of the existing orchards in that region and, also, to restrict spreading and to guide the flow of water directly into Ferguson Brook. The estimated cost for construction of the dam, including the operator's quarters, is $1,726,000, based on a construction period of two years. Local financial cooperation is not required for this project. This definite project is currently revised to incorporate modifications in the design of the outlet works and spillway discharge channel. Authorization for preparation of a definite project report on the West Peterboro Reservoir is contained in telegram from the North Atlantic Division Office, New York, New York, dated 1 May 1940

SECTION C.

TEXT OF REPORT

War Department
United States Engineer Office
Boston, Massachusetts

MERRIMACK RIVER BASIN FLOOD CONTROL PROJECT

DEFINITE PROJECT REPORT
WEST PETERBORO RESERVOIR
CONTOOCOOK RIVER, N. H.

SECTION C - TEXT OF REPORT
May 1945

1. Project Authorization.- The West Peterboro Reservoir for flood control protection in the Contoocook River Basin, New Hampshire, as described herein, is proposed as an element of the comprehensive plan for flood control reservoirs and related flood control work for the Merrimack River Basin authorized by the following portions of the Flood Control Acts of 1936 and 1938.

a. Flood Control Act, approved 22 June 1936 (Public - No. 738 - 74th Congress):

"FLOOD CONTROL ACT OF 1936

"Sec. 5. That pursuant to the policy outlined in Sections 1 and 3, the following works of improvements, for the benefit of navigation and the control of destructive flood waters and other purposes, are hereby adopted and authorized to be prosecuted, in order of their emergency as may be designated by the President, under the direction of the Secretary of War and supervision of the Chief of Engineers in accordance with the plans in the respective reports and records hereinafter designated: Provided, That penstocks or other similar facilities, adapted to possible future use in the development of adequate electric power may be installed in any dam herein authorized when approved by the Secretary of War upon the recommendation of the Chief of Engineers.

****

"MERRIMACK RIVER, NEW HAMPSHIRE AND MASSACHUSETTS

"Construction of a system of flood control reservoirs in the Merrimack River Basin for the reduction of flood heights in the Merrimack Valley generally; estimated construction cost, $7,725,000; estimated cost of lands and damages, $3,500,000,"

b. Flood Control Act, approved 28 June 1938 (Public No. 761 - 75th Congress, 3rd Session):

"Sec. 4. That the following works of improvement for the benefit of navigation and the control of destructive flood waters and other purposes are hereby adopted and authorized to be prosecuted under the direction of the Secretary of War and supervision of the Chief of Engineers in accordance with the plans in the resepctive reports hereinafter designated: Provided, That penstocks or other similar facilities adapted to possible future use in the development of hydroelectric power shall be installed in any dam herein authorized when approved by the Secretary of War upon the recommendation of the Chief of Engineers and of the Federal Power Commission.

"MERRIMACK RIVER BASIN

"The general comprehensive plan for flood control and other purposes, as approved by the Chief of Engineers pursuant to preliminary examinations and surveys authorized by the Act of June 22, 1936, is approved and the project for flood control in the Merrimack River Basin, as authorized by the Flood Control Act approved June 22, 1936, is modified to provide, in addition to the construction of a system of flood control reservoirs, related flood control works which may be found justified by the Chief of Engineers".

Approval of the definite project report dated 5 August 1940 by the Chief of Engineers is contained in the 2nd Indorsement dated 22 September 1940, (File No. 7402 (Merrimack R.-West Peterboro Res)-4) to letter forwarded by the District Engineer, U. S. Engineer Office, Boston, Massachusetts, to the Chief of Engineers, U. S. Army, dated 6 August 1940, Subject: "Definite Project Report, West Peterboro Reservoir".

A description of the approved project was contained in the Annual Report for Fiscal Year 1942 - River and Harbor and Flood Control Projects, dated 1 August 1942, Paragraph 20, as submitted by the Boston District, as follows:

"The dam will be of the rolled-earth type, with a dumped rock toe and a dumped rock blanket on the upstream face, 950 feet long at the top, with a maximum height of 65 feet and top width of 30 feet, and side slopes of 1 on 3 in general. It will contain about

290,000 cubic yards of earth and rock fill. A gravity type spillway 100 feet long, 32.3 square feet of gate-controlled outlet, and a 9-foot diameter penstock, involving about 22,000 cubic yards of concrete, will be constructed. The reservoir will provide a storage capacity of 16,000 acre-feet, equivalent to 6.8 inches of run-off over the 44-square-mile drainage area controlled. The estimated cost is $1,170,000. The project is a unit of the comprehensive plan for flood control and other purposes in the Merrimack River Basin which was authorized by the Flood Control Act approved June 22, 1936, and modified by the Flood Control Act approved June 28, 1938".

The proposed reservoir, as described in this revised definite project report, is located in Hillsboro and Cheshire Counties, New Hampshire. The dam site is about one-half mile upstream from the village of West Peterboro, New Hampshire, on Nubanusit Brook, a tributary of the Contoocook River. The dam consists of a rolled earth embankment section with a reinforced concrete intake structure, outlet conduit, and stilling basin which are located approximately 1000 feet above the existing Verney Mills Dam. The spillway is a gravity concrete weir section and is located in a natural saddle at the north side of the reservoir. The spillway discharge flows into Ferguson Brook which, in turn, discharges into the proposed Bennington Reservoir.

Local financial cooperation is not required for this project.

2. Investigations.-

a. Scope of Investigations, Studies and Planning.- Investigations and studies have been made of all factors affecting the construction of the West Peterboro Reservoir as a unit of the comprehensive flood control reservoir plan for the Merrimack River Basin. Data have been compiled, studied and analyzed on climatical, hydrological and geological conditions, flood heights, frequencies and losses, power possibilities, economics of construction, and benefits to be derived from construction of the project. All local, State and Federal agencies and private owners concerned with the development of the project have been consulted relative to the design, construction and operation of the reservoir. Detailed descriptions and results of the investigations, analytical studies and planning are contained in other sections of this report and in previously prepared reports as noted below.

b. Previous Investigations.- Results of preliminary studies of the West Peterboro Reservoir were included in the report published as House Document No. 689. This report placed

emphasis on the Riverhill site, located approximately 7 miles north westerly of the city of Concord, New Hampshire, for control of the Contoocook River. The Riverhill site was abandoned due to local opposition, and projects for the construction of the West Peterboro, Mountain Brook and Hopkinton-Everett Reservoirs were substituted. Subsequently, in the spring of 1946, local opposition to Mountain Brook developed and this project was abandoned.

Preliminary definite project reports on Mountain Brook and West Peterboro Reservoirs were submitted on 25 April 1940 as a part of the comprehensive flood control plan, and were approved by the Chief of Engineers on 30 April 1940. The preliminary report on the West Peterboro Reservoir proposed locating the dam at the southerly side and a remote saddle spillway on the northerly side of the reservoir as recommended in this report.

At a hearing held by the New Hampshire Water Resources Board in Concord, New Hampshire, on 8 May 1940, the Federal Power Commission proposed a plan of multiple-purpose development for the Contoocook River Basin as an alternate to the flood control plan of development proposed by this office. The alternate plan did not affect the West Peterboro Reservoir since this project was included in both plans of development.

c. Previous Definite Project Plan.- Subsequent to the submission of the preliminary definite project report, dated 25 April 1940, bedrock was discovered in the west bank of the Nubanusit Brook, about 1000 feet upstream from the Verney Mills Dam. The layout of the reservoir was then modified to locate the spillway on the underlying bedrock of the west abutment at the upstream dam site. This relocation of the spillway from the saddle site at the northerly end of the reservoir to the dam site reduced the estimated cost of the project from $1,300,000 to $1,011,000. A definite project report incorporating these modifications was submitted 6 August 1940, and was revised and resubmitted 14 December 1940 to include modifications suggested by the Chief of Engineers. The cost of the modified project as submitted was $1,170,000.

d. Revised Definite Project Plan.- Further geological investigations, made after submission of the definite project report dated 5 August 1940 (Revised 14 December 1940), indicated that the rock on the west bank of the brook is badly weathered, that a concrete lining of the spillway discharge channel would be required, and that the excavated rock would be unsatisfactory for use as slope protection riprap. These factors increased the estimated cost of the approved project to the estimated cost of the original plan which included the saddle spillway. Consideration of the equal costs, together with the direct benefits to be derived in complete downstream protection afforded by the diversion of the spillway

design flood discharge through a saddle spillway into the proposed Bennington Reservoir, led to the decision to locate the spillway in the saddle at the northerly end of the reservoir. Accordingly, the layout of the dam has been modified to construct the dam approximately 1000 feet above the existing Verney Mills Dam and to construct a remote spillway in the saddle between Nubanusit Brook and Ferguson Brook. The present revision of the definite project report incorporates the results of studies and investigations pertaining to this modification of layout and design.

A new area-capacity curve, computed from a 5-foot contour interval survey made in 1944 by aerial survey methods, has resulted in a decrease in reservoir storage capacity from 16,000 acre-feet to 12,800 acre-feet at the limiting spillway crest elevation 946. The original area-capacity curve was based upon standard topographic plans with 20-foot contour intervals, prepared by the U. S. Geological Survey.

3. Local Cooperation.-

a. Flood Control Act Requirements.- Local financial cooperation is not required as Section 2 of the Flood Control Act approved 28 June 1938 (Public No. 761 - 75th Congress, 3rd Session) applies to this project.

b. Status of Approval by State of New Hampshire.- The Governor and Council of the State of New Hampshire passed a resolution on 24 December 1941 approving the acquisition of land for construction of the West Peterboro Reservoir for flood control purposes. The resolution passed by the State is as follows:

> "RESOLVED: That pursuant to and subject to the Laws of 1939, Chapter 149, the Governor and Council consent to and approve of the recommendations of the New Hampshire Water Resources Board for the acquisition of lands and interests therein by the United States by purchase, condemnation or otherwise required for use only in connection with the construction, maintenance and operation of the following projects:
>
> 1. Mountain Brook Reservoir on the Mountain Brook in the Town of Jaffrey in accordance with "Definite Project Report for Mountain Brook Reservoir", Mountain Brook, dated June 1, 1940, revised December 16, 1940.

which report is on file and open to public inspection in the records of the New Hampshire Water Resources Board.

2. West Peterborough Reservoir on the Nubanusit Brook in the Towns of Peterborough, Dublin, Harrisville and Hancock, in accordance with "Definite Project Report for West Peterborough Reservoir", dated August 5, 1940, revised December 14, 1940.

which report is on file and open to public inspection in the records of the New Hampshire Water Resources Board."

The design of the dam has been modified by relocating the spillway from the dam site to the saddle between Half Moon Pond and Ferguson Brook subsequent to the passage of the above resolution. Approval by the State of the acquisition of land for the West Peterboro Reservoir in accordance with this revised definite project plan was requested by letter dated 10 May 1945 to the Honora ble Charles M. Dale, Governor of the State of New Hampshire.

The Governor and Council of the State of New Hampshire passed a resolution on 15 March 1946 approving the acquisition of land for construction of the remote saddle spillway at West Peterborough Reservoir. The resolution passed by the State is as follows:

"RESOLVED: That consent be given to the acquisition by the United States through purchase, condemnation, or otherwise, consistent with applicable provisions of the Constitution of the United States of any lands or interest in lands necessary to construct, maintain and operate a remote saddle spillway at the West Peterborough Flood Control Reservoir, as recommended by the Water Resources Board."

c. Verney Mills Power Pool.- The proposed West Peterboro Dam site is located approximately 1000 feet upstream from the storage and intake dam of the Verney Mills, Inc., and is in the existing power pool formed by the dam. It is proposed to replace the existing 18-inch flashboards by 36-inch flashboards on the existing Verney Mills Dam to compensate for the reduction in power storage caused by the construction of the flood control dam. Representatives of the Verney Mills, Inc., have been consulted and have agreed informally to the proposed plan. A formal agreement is being prepared by the legal staff of the Verney Mills, Inc., and will be incorporated as an appendix to this report when the formalities are completed.

4. Definite Project Plan.-

a. General.- The West Peterboro Reservoir is one of several reservoirs comprising the comprehensive plan for flood control reservoirs and related flood control works for the Merrimack River Basin. The reservoir controls 90 percent of the drainage area of the Nubanusit Brook.

b. Location.- The proposed reservoir is located on the Nubanusit Brook, New Hampshire, in the towns of Peterboro, Dublin, Hancock and Harrisville, in the southwestern part of the State of New Hampshire. The dam is situated approximately one-half mile upstream (north) of the village of West Peterboro.

c. Description of Reservoir Area.- A large part of the land within the reservoir basin is wooded and covered with a second growth composed mainly of white pine, hemlock, white and red maple, gray and silver birch, and white and red oak. Scattered about are small clumps of white pine and hemlock of

sufficient size to be merchantable, but there is not enough mature timber on any one unit to make commercial operations profitable. It is very noticeable that within the thin stands of birch, and the like, there is a good undergrowth of young white pine. These young trees appear thrifty and are making good growth.

The few cleared acreages are rather small in area and contain large boulders that make tillage operations difficult. Only three acres of the thirty-four acres of tillable land within the proposed reservoir have been used for tillage crops within the past years, and most of the fields have been producing little more than a hay crop for a number of years.

d. Design Floods. -

(1) Reservoir Design Flood. - The two largest floods of record on Mubanusit Brook occurred in March 1936 and September 1938. The larger of these, March 1936, had a peak discharge of 4140 c.f.s. and an estimated volume of about 35,000 acre-feet in a 16-day period, equivalent to about 15.0 inches of run-off over the watershed. The September 1938 flood had a peak discharge of 4100 c.f.s. and an estimated volume of about 15,000 acre-feet in a 6-day period, equivalent to about 6.5 inches of run-off over the watershed. These floods were used in studies of storage capacity.

(2) Spillway Design Flood. - The storm used for the spillway design flood comprises the computed maximum possible rainfall over the 44 square mile drainage area, or 19.5 inches of rainfall in 12 hours with 17.3 inches occurring in 6 hours. Run-off was computed by means of a synthetic unit hydrograph, assuming an infiltration rate of .05 inches per hour and base flow of 220 c.f.s., or 5 c.f.s. per square mile over the watershed. The resulting spillway design flood has a peak inflow of 36,300 c.f.s. and a vo lume of 18.9 inches over the watershed. The maximum spillway discharge when routed through the reservoir is 17,800 c.f.s. at a maximum reservoir stage of 960.8 or 14.8 feet above spillway crest.

e. Project Description. - As shown on Plates IV-1 to IV-5 accompanying Appendix IV of this report, the impounding structures consist of a low concrete weir overflow section and a rolled earth embankment. The weir is founded on bed rock, in a natural saddle on the northerly side of the reservoir. The spillway approach and discharge channel is mostly cut through rock and has an earth dike on the northerly side to train the spread of the discharge and protect existing orchards. The outlet works are incorporated into the earth embankment section. The outlet works include a reinforced concrete intake structure, conduit

and stilling basin with gravity type guide walls, all of which are founded on rock. The proposed dam has a crest length of 1100 feet and is founded on bed rock on the west bank and river section, and on glacial till on the east bank of the river. It is proposed to construct the dam for flood control storage only, with no provision made for ultimate raising. The reservoir, which controls a drainage area of 44 square miles, has an area of 850 acres at spillway crest elevation 946. The reservoir has a capacity of 12,800 acre-feet which is equivalent to 5.5. inches of run-off on the drainage area.

f. Estimated Cost.- A detailed estimate of cost for construction of the reservoir is given in Paragraph 14. A summary of the estimated cost is as follows:

Reservoir Costs	$ 209,000
Reservoir Clearing	5,300
Construction Costs	1,511,700
Total Estimated Cost	$1,726,000

g. Method of Operation.- The West Peterboro Reservoir will be operated primarily to regulate the flood discharges of the Nubanusit Brook so as to provide maximum benefits at downstream damage centers and to reduce flows in the brook below the Verney Mills Dam to amounts equal to or less than channel capacity. Sufficient water will be discharged at all times to provide storage equivalent to the existing storage for use of the Verney Mills, Inc.

h. Alternative Layouts.- A number of engineering studies have been made for various locations within the vicinity of the proposed dam site and considerable drill hole information has been obtained for the embankment and saddle spillway locations. Detailed analyses of two different layouts were previously presented in the following reports: (a) Preliminary Definite Project Report, dated 25 April 1940, which had the embankment and outlet works located directly over the existing Verney Mills Dam and the spillway placed in the saddle at the northerly end of the reservoir, and (b) Definite Project Report, dated 5 August 1940 (revised 14 December 1940), which had the embankment, spillway and outlet works located approximately 1000 feet upstream from the Verney Mills Dam.

i. Ultimate Development.- Provision is not being made in the design for ultimately raising the dam, in view of the unfavorable results of analysis of this project for future conservation storage in addition to flood control storage, as presented in Appendix V.

5. Structures and Improvements.-

a. Embankment.- The embankment is designed as an earth fill section consisting of a compacted impervious core backed with compacted random fill and flanked with a shell of compacted pervious fill, as indicated in the sections on Plate IV-2. The upstream slope and downstream drainage toes are protected by stone blankets. The embankment section has a top width at elevation 967 of 25 feet; an upstream slope of 1 on 2.75; a downstream slope of 1 on 2.5; a maximum height of 69 feet and a total length of 1100 feet. The impervious core cutoff extends to bed rock on the west abutment and into the underlying glacial till on the east abutment between Station 5 + 50 and the easterly end of the embankment. An upstream impervious blanket tied into the core is provided on the upstream side of the dam, as indicated on Plates IV-1 and IV-2.

b. Outlet Works.- The outlet works consist of an intake gate house, reinforced concrete conduit and stilling basin which are founded entirely on rock. The gate house is a reinforced concrete structure which contains three outlets, 4'-0" x 5'-0", each of which is equipped with a commercially available cast iron sluice gate of stock size for control of the discharge. Each gate is equipped with a motorized screw type gate hoist. Stop log recesses are provided so that the stream flow can be blocked during maintenance operations. Access is gained to the equipment room of the gate house from the top of the dam embankment by means of a structural steel foot bridge. The gated outlets converge through a smooth transition into a single reinforced box conduit, 7'-0" x 7'-0", which passes through the embankment and discharges into the stilling basin. The stilling basin is composed of a reinforced concrete floor mat, baffles and end sill and concrete gravity side walls all founded on rock. See Plate IV-6 for design analysis of masonry structures. The discharge channel is an earth cut with riprapped side slopes, terminating in the pool created by the Verney Mills Dam. The existing 18 inch high flashboards on the Verney Mills Dam are to be replaced with 36 inch flashboards to compensate for the reduced pondage which will result from construction of the flood control dam across the existing mill pond.

c. Spillway.- The spillway is located in a natural saddle separating the Nubanusit Brook and Ferguson Brook drainage areas. The spillway is a low concrete weir founded on bed rock with a crest length of 100 feet at elevation 946. The approach channel is approximately 1900 feet long, varying from 104 to 140 feet in width with the bottom at elevation 941. The discharge channel at the spillway is at elevation 940 and gradually slopes with a 1 percent grade to the present ground surface. The channel has a bottom width of 100 feet for a distance of approximately

700 feet from the spillway. Sections of the channel not in rock cut are protected by riprap where such protection is required. A low earth filled dike is provided downstream from the discharge channel to guide the spillway discharge, during periods of high water, until it reaches Ferguson Brook and to protect existing orchards back of the proposed dike. The spillway training walls are gravity type concrete sections. A maximum water surface of elevation 958.2 immediately upstream from the weir, equivalent to a pool elevation of 960.8 will be required to pass the spillway design flood discharge of 17,800 c.f.s.

d. Culverts. - A reinforced concrete box culvert, with two 6' x 11' openings, is provided across Farm Road, as shown on Plate IV-5. This culvert is designed to provide for a spillway discharge up to 1000 c.f.s., which is more than sufficient for the 1936 flood spillway discharge.

e. Operator's Quarters. - It is proposed to construct a brick masonry house, with finished interior, to serve as an office and as living quarters for the operator of the dam. This house is to be located in the vicinity of the east abutment on the Government reservation and can be serviced from the access road.

6. Foundations. -

a. Dam Site. - The pre-glacial stream channel of the Nubanusit Brook was a bed rock valley composed of moderately to highly fractured silicious schist. The topography of the region was modified by glaciation and the existing stream is now located on the west wall of the bed rock valley. The foundation soil is a compact sandy to silty glacial till of high shear strength and low permeability with lenses of more pervious sand. A shallow surface layer of the till has been altered by weathering and is less compact than the main till deposit. Bed rock occurs in the west abutment and under the stream channel at shallow depths of 10 to 20 feet, and is buried deeply in the east abutment. An area of weathered bed rock located in the west abutment was encountered during the foundation explorations. The embankment is founded on the glacial till and the outlet conduit and control works are founded on sound but fractured bed rock. A summary of the foundation investigations and analyses is presented in Appendix II of this report.

b. Spillway. - The spillway is located approximately three miles due north of the dam and in a saddle which separates

Nubanusit Brook and Ferguson Brook drainage areas. Bed rock which forms the saddle is fresh, unweathered, moderately fractured porphyritic granite. The bed rock outcrops adjacent to the spillway location but dips steeply northward and is overlain by compact silty till. Cobbles and boulders are densely strewn over the area. The spillway weir and the adjacent portions of the approach and discharge channels are located in sound bed rock. Investigations and soil analyses for the spillway are presented in Appendix II.

7. Hydroelectric Power. -

a. Studies. - Results of studies of the feasibility of developing the West Peterboro site as a multiple-purpose reservoir for flood control, and for conservation storage for stream regulation and hydroelectric power indicate that such development cannot be economically justified. The optimum additional reservoir capacity resulting from second stage construction after completion of the flood control reservoir is 31,200 acre-feet. Second stage construction would involve the acquisition of 1,020 acres of additional reservoir area, the condemnation of a valuable summer camp, the relocation of 5.6 miles of highway, and the acquisition of the water rights at Verney Mills Dam in addition to raising the dam and spillway.

b. Ratio of Benefits to Carrying Charges. - Evaluation of the construction costs and the benefits to be derived from such construction indicate that the resultant ratio of annual benefits to annual carrying charges would be only 0.59 for development of the dam to provide flood and conservation storage and only 0.66 for the development of the dam to provide flood and conservation storage and to develop hydroelectric power at the site.

c. Potential Operation of Verney Mills Hydroelectric Power Installation. - The possibility of abandonment of the Verney Mills Dam at some future time is provided for in the design of the outlet conduit. Extension of the existing penstock at the Verney Mills Dam to the proposed dam has been provided for in the selection of the conduit size and shape. The penstock would be extended from its present location to the stilling basin end of the conduit and then through the conduit to the intake structure. The pipe would enter the side of the conduit immediately upstream from the stilling basin with a "Y" connection and an additional sluice gate would be required at the downstream end of the conduit to permit the discharge of water into the stilling basin.

8. Relocations. - Construction of the proposed West Peterboro Reservoir involves the relocation of only short lengths of roads and a small number of utilities.

Approximately one mile of gravel roadway, running parallel to and on the easterly side of the brook between Swetts Dam Bridge and the proposed dam is subject to inundation. It is proposed to abandon this portion of the road and replace it by regrading and resurfacing an existing graveled farm road which ties into High Wind Road, approximately one-half mile east of the dam site, as shown on Plate VI-1. The relocation of this road also requires the relocation of the local power and telephone service lines, adjacent to the road. A section of the existing graveled roadway in the vicinity of the Sargents Camp requires raising above spillway crest elevation for a length of approximately 0.5 mile. This road, at the present time, is only open for summer traffic.

The masonry abutments of four timber deck bridges must be modified in order to raise the deck and provide the necessary clearance when the water is at spillway crest level. The location of the bridges to be raised is indicated on Plate VI-1.

There are no cemeteries or major pipe lines within the reservoir area.

9. Availability of Construction Materials.-

a. All materials removed from required structure excavations at the dam site are scheduled for use in the embankment. Earth excavation at the spillway site, except for the protective dike fill and grading at the spillway, is wasted because equally suitable materials are available in borrow sources close to the dam. Rock excavated from the spillway and discharge channel is scheduled to be used for slope protection at both the spillway and dam sites. Ample quantities of compact silty glacial till, suitable for the impervious core, occur in both banks of the valley. The site selected for development of the impervious borrow area is on the east wall of the valley and upstream from the center of the dam. Large quantities of glacial sand and gravel deposited in eskers, kames and terraces are located on the east side of the stream and are within one mile upstream from the dam site. These deposits contain sufficient suitable materials for the pervious fill and filter materials. Concrete aggregates may be obtained from the same source by processing.

b. A summary of the materials available from excavation with the indicated disposition, and the materials required for construction and their proposed source, are presented in the tabulation given in Paragraph c of Appendix II.

10. Construction Time Required and Schedule of Operations.-

a. Required Construction Time.- The construction of

the dam and spillway is scheduled to be accomplished by contract during the summer months over a two-year period due to the severe winter weather in this area.

b. Schedule of Operations.- The schedule of construction operations based upon completion of the dam and spillway in two years is as follows:

First Season:

Clear and strip the dam site and spillway approach and discharge channel; excavation for the outlet works, except for closure section adjacent to the brook; excavation for core trench and a portion of the spillway approach and discharge channels; construct the dam embankment on the east and west abutments; grading and paving for the access road; construct outlet works approach channel, conduit, stilling basin, and intake structure to elevation 967; install gates in intake structure.

Second Season:

Construct upstream and downstream cofferdams; divert the river through conduit; construct gate house; complete dam embankment and construct dike downstream from spillway; complete excavation for spillway; construct spillway weir, concrete bridge at spillway and reinforced concrete box culvert under Farm Road; install new flashboards on Verney Mills Dam; relocate roads and raise bridges; construct operator's quarters.

c. Funds Required by Fiscal Years.- The funds required during each fiscal year of the two-year construction period, for the accomplishment of the project, including land acquisition and relocations, construction operations, and engineering, contingencies and overhead, are estimated to be as follows:

First Year	$ 810,000
Second Year	916,000
Total	$1,726,000

The estimated fund requirements are based upon the execution of all construction by contract work.

It is estimated that a period of six months will be required to prepare the contract plans and specifications, at an estimated cost of $50,000.

e. Employment Analysis.- In comparison with similar projects constructed in the Boston District, it is estimated that the project reported herein will create the following number of man hours of labor:

(1)	At-Site Labor:	
	Skilled labor	100,000 man hours
	Unskilled Labor	400,000 man hours
	Other	50,000 man hours
	Total	550,000 man hours
(2)	Off-Site Labor:	
	Total	140,000 man hours

11. Clearance with Other Agencies.- In accordance with Circular Letter No. 2652, dated 1 January 1944, subject: "Quadripartite Agreement", and Circular Letter No. 2306, dated 1 August 1944, subject: "Fish and Wild Life Service Cooperation", information and pertinent data on the construction of the proposed West Peterboro Reservoir have been imparted to the Soil Conservation Service of the Department of Agriculture; the Federal Power Commission; and the Fish and Wild Life Service of the Department of the Interior.

Local representatives of the Fish and Wild Life Service have been consulted and the details of the project have been discussed with them. Plans and other information have been furnished at their request, and it is understood that a complete investigation and study is being made of the project by this Service.

Several conferences have been held in the Boston District Office with representatives of the Federal Power Commission on the construction and relationship of the proposed West Peterboro Reservoir as part of the comprehensive plan for flood control of the Merrimack River Basin. Folios prepared for the Board of Consultant's conferences containing complete plans and analysis of the dam were furnished the Federal Power Commission for its use. In a reply dated 27 March 1946 and addressed to the Secretary of War, the Commission concludes that "the plans for the West Peterboro project provide adequately for its possible use in the development of power, and does not recommend any further provisions for power development."

In accordance with the provisions of the Flood Control Act approved 22 December 1944 (Public Law 534 - 78th Congress - 2nd Session) and as this project in its entirety lies east of the ninety-

seventh meridian, the Department of the Interior has not been consulted with reference to matters of land reclamation.

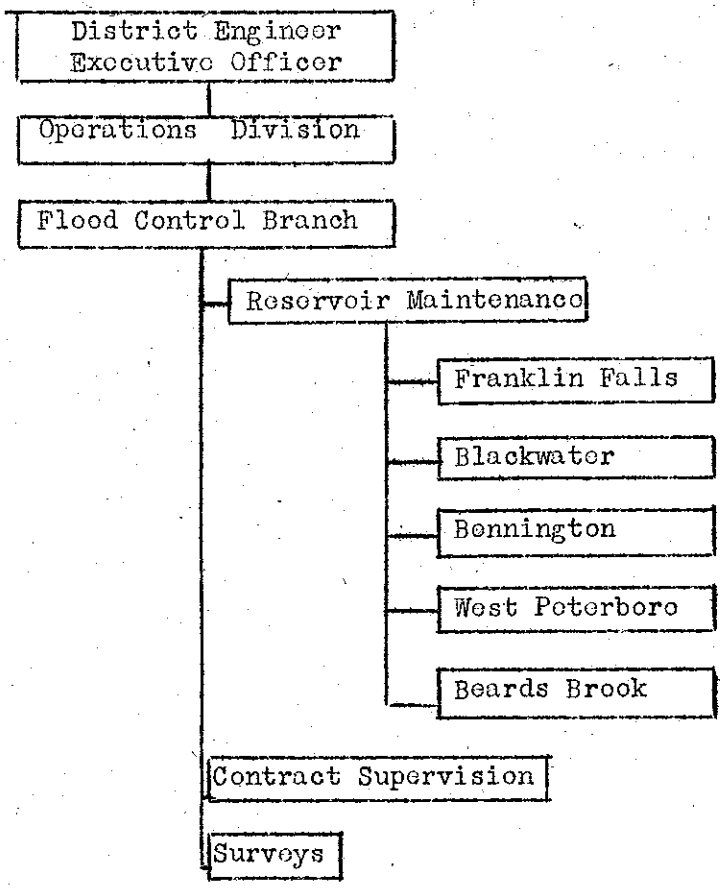
The Soils Conservation Service, Department of Agriculture, has been informed of the proposed flood control program being planned by the Boston District Office for co-ordination of planning for soil conservation in the development of the projects. The Department of Agriculture acknowledged receipt of the data furnished and forwarded its "Report - Soil Erosion Conditions in New Hampshire", for review and wishes to be informed of any hearings which may be held. Further action by the Department has not been indicated to date.

12. Operation and Maintenance.- The operation and maintenance of the dam is a Federal function.

Maintenance of equipment at the operating house and of the dam structure, including slopes, drains and access road, will be necessary. After periods of high water, any debris that may have accumulated will be removed by maintenance personnel at the dam. Policing of the reservoir area will be an additional duty.

The permanent organization at the site will consist of a dam tender, CPC-6, $2469, who will be provided with quarters at the site. Any assistance required in an emergency will be secured locally, as will labor for such special jobs as removal of large amounts of debris from the reservoir.

The operation and maintenance force will be under the supervision of the Flood Control Branch of the Operations Division of the U. S. Engineer Office, Boston District. The organizational structure of the operation and maintenance force is shown on the following chart.



The estimated annual cost of operation and maintenance is as follows:

Item of Work	Estimated Cost
Services of dam tender and part-time helper	$2,900.
Power, lubricants and fuel for gate operation	50.
Heating operating structure	50.
Clearing and removing debris from reservoir area	400.
Maintenance of access road, surface drains and unprotected slopes	175.
Repairs and painting	100.
Miscellaneous	75.
District Office overhead	250.
Total	$4,000.

13. Malaria Control.- In accordance with instructions contained in Circular Letter No. 3606, dated 9 March 1945, subject: "Malaria Control at River and Harbor and Flood Control Reservoirs", this office requested the advice and recommendations of the U. S. Public Health Service with respect to the need and requirements for malaria control at the West Peterboro Reservoir. Plans

(Rev. June 1946)

and pertinent data relative to the reservoir were forwarded to the Public Health Service. Recommendations of the Public Health Service have been received and are presented in Appendix VII of this project.

14. Cost Estimates.-

a. Total Costs.- The estimated costs, including engineering, contingencies and overhead, for the principal elements of the project, as outlined in this report, are as follows:

Lands, Damages, Relocations	$ 209,000
Reservoir Clearing	5,300
Construction of Dam and Outlet	801,000
Construction of Saddle Spillway and Protective Dike	710,700
Total Estimated Cost	$1,726,000

The total estimated cost for West Peterboro Reservoir as given in the Definite Project Report, dated 5 August 1940 (Revised 14 December 1940) is $1,170,000. Geological investigations subsequent to the submission of this estimated cost, revealed a badly weathered rock at the proposed dam site which would require special treatment and increase the construction costs of the through dam spillway to approximately equal the estimated cost of the remote spillway proposed herein.

b. Unit Costs.- The above estimated costs of principal elements are based upon the estimated quantities and current unit prices (i. e. Summer 1946) summarized in the following table. The estimated costs of lands and damages are based on a revised appraisal made in September 1945 by the Boston District Real Estate Sub-office, Boston, Massachusetts, and are believed to represent a reasonable evaluation. The reservoir area at elevation 946 contains 850 acres of land, of which approximately 18% is wooded, 45% is scrub growth, 4% is tillable, and 33% is inundated.

DETAILED ESTIMATE OF COST

1. RESERVOIR COSTS

	Total Cost
Land and Improvements) Riprarian and Water Rights)	$ 80,000
Relocation of Telephone and Power Lines and Highways	85,000
Sub-total - Reservoir Costs	$165,000
Contingencies, (15%)	25,000
	190,000
Government Expenses (10%)	19,000
TOTAL RESERVOIR COSTS	$209,000

II. CONSTRUCTION COSTS

	Quantity	Unit	Unit Cost	Total Cost
(a) Earth Dam and Controlled Outlet				
Stream Diversion and Unwatering Site	-	-	L.S.	$ 20,000
Clearing and Grubbing	10	Acre	$400.00	4,000
Stripping	30,000	C.Y.	.70	21,000
Excavation - Common	107,000	C.Y.	.55	58,850
Excavation - Rock	10,000	C.Y.	3.00	30,000
Borrow - Pervious	123,000	C.Y.	.60	73,800
Borrow - Impervious	56,000	C.Y.	.50	28,000
Sand and Gravel	24,000	C.Y.	2.50	60,000
Screened Gravel	500	C.Y.	3.00	1,500
Structural Backfill	6,000	C.Y.	.80	4,800
Rolled Fill - Pervious	107,000	C.Y.	.15	16,050
Rolled Fill - Impervious	85,000	C.Y.	.18	15,300
Rolled Fill - Random Mat'l	34,000	C.Y.	.15	5,100
Rock Fill (Dumped) Transportation & Placing Only	15,000	C.Y.	1.50	22,500
Riprap-Hand Placed Transportation & Placing Only	4,000	C.Y.	3.00	12,000
Drilling Holes for Grouting	2,500	L.F.	1.70	4,250
Pressure Grouting	2,500	C.F.	1.60	4,000
Concrete - Walls and Stilling Basin	2,000	C.Y.	18.00	36,000
Concrete - Gate Structure and Conduit	2,100	C.Y.	23.00	48,300
Reinforcing Steel	175,000	Lb.	.09	15,750
Gates, Guides, Hoists, and Trashbars	-	-	L.S.	28,000
Operating Superstructure including Crane	-	-	L.S.	6,000
Bridges, Incl. Piers	-	-	L.S.	7,000

(Rev. June 1946)

	Quantity	Unit	Unit Cost	Total Cost
II. CONSTRUCTION COSTS (Cont'd)				
(a) Earth Dam and Controlled Outlet (Cont'd)				
Seeded Topsoil	4	Acre	$600.00	$ 2,400
Access Road	-	-	L.S.	12,000
Operator's Quarters and Miscellaneous Items	-	-	L.S.	20,400
SUB-TOTAL - EARTH DAM AND CONTROLLED OUTLET				$ 557,000
(b) Saddle Spillway and Protective Dike				
Clearing and Grubbing	25	Acre	500.00	12,500
Excavation - Common	340,000	C.Y.	.70	238,000
Excavation - Rock	30,000	C.Y.	3.00	90,000
Concrete:				
Weir and Walls	2,200	C.Y.	20.00	44,000
Bridge Piers and Abutments	1,100	C.Y.	23.00	25,300
Sand and Gravel	4,500	C.Y.	2.50	11,250
Structural Backfill	4,000	C.Y.	.80	3,200
Riprap - Hand Placed	7,000	C.Y.	4.00	28,000
Dumped Rock	3,000	C.Y.	.70	2,100
Seeded Topsoil	13	Acre	400.00	5,200
Highway Bridge	-	-	L.S.	18,000
Compacted Fill	33,000	C.Y.	.18	5,940
Miscellaneous Items	-	-	L.S.	15,510
SUB-TOTAL - SADDLE SPILLWAY AND PROTECTIVE DIKE				$499,000
(c) Reservoir Clearing	-	-	L.S.	4,000
Sub-Total - Construction Costs (a, b and c)				$1,060,000
Contingencies, 10%				106,000
TOTAL CONSTRUCTION COST				$1,166,000
Definite Project Report, Plans and Specifications				$ 129,000
Supervision and Inspection of Construction				116,000
Completion Report				6,000
Division and District Overhead				100,000
TOTAL ENGINEERING AND CONSTRUCTION COST				$1,517,000
III. TOTAL PROJECT COST (I + II)				
Reservoir Costs				$ 209,000
Construction and Engineering Cost				1,517,000
TOTAL ESTIMATED COST OF PROJECT				$1,726,000
Cost per Acre Foot of Total Storage				134.84

c. Carrying Charges.- The total annual carrying charge for the reservoir, based upon interest on the investment, on amortization of structures and equipment, and on operation and maintenance, is $73,919, as summarized in the following table.

ANNUAL COSTS AND CARRYING CHARGES

FEDERAL INVESTMENT

1. Total First Cost:

a.	Structures with 50 year life	$1,676,000
b.	Equipment with 25 year life	50,000
c.	Total	$1,726,000

2. Interest During Construction:
(3% for one-half construction period)

a.	On structures with 50 year life	50,280
b.	On equipment with 25 year life	1,500

3. Total Investment:

a.	Structures with 50 year life	1,726,280
b.	Equipment with 25 year life	51,500
c.	Total Federal Investment	$1,777,780

ANNUAL FEDERAL CARRYING CHARGES

1.	Interest on Investment @ 3%	$53,333
2.	Amortization:	
	a. Structures with 50 year life (0.887%)	15,312
	b. Equipment with 25 year life (2.743%)	1,274
3.	Operation and Maintenance	4,000
4.	Total Annual Federal Carrying Charge	$73,919

Construction Period - 2 years

15. Economic Study.- The ratio of annual benefits to annual carrying charges is 1.11 as shown in the accompanying table "Summary of Benefits and Costs" and justifies construction of the West Peterboro flood control reservoir. It is believed that the above ratio will increase substantially when a review of annual benefits, now in process, is completed.

Studies were also conducted to determine the feasibility of a multiple-purpose development of the West Peterboro Reservoir in the interests of flood control and of power storage for stream regulation and hydroelectric power generation. The results of these studies are presented in Appendix V and indicate that second stage construction cannot be economically justified. The ratio of annual benefits to carrying charges for stream regulation storage is 0.59 and for stream regulation and hydroelectric power generation at the site is 0.66. These low ratios do not warrant making provision in the initial construction for the ultimate development of the reservoir for multiple-purpose use.

SUMMARY OF BENEFITS AND COSTS

1. Construction Costs

a.	West Peterboro Reservoir	$ 1,726,000
b.	Other Flood Control Reservoirs (x)	19,342,000
c.	Local Protection Projects (xx)	1,658,600
	Total Construction	$22,726,600

2. Annual Carrying Charges

a.	West Peterboro Reservoir	$ 73,919
b.	Other Flood Control Reservoirs (x)	890,000
c.	Local Protection Projects (xx)	79,600
	Total Carrying Charges	$ 1,043,519

3. Total Annual Benefits

Based on comprehensive flood control program including reservoirs and local protection (Local benefits attributed to Mountain Brook Reservoir, now abandoned, have been removed. The figure used is being revised to reflect the increased cost of flood damages). $ 1,159,000

4. Ratio of Benefits to Carrying Charges

Ratio of Total Annual Benefits to Annual Carrying Charges 1.11

(x) Includes complete reservoirs at Franklin Falls and Blackwater and proposed reservoirs at Beards Brook and Bennington

	Cost		Annual Charges
Franklin Falls	$ 7,883,000	- Completed	$371,000
Blackwater	1,275,000	- Completed	65,000
Beards Brook	4,200,000	- Estimated	189,000
Bennington	5,984,000	- Estimated	265,000
	$19,342,000		$890,000

(xx) Includes completed local protection project at Lowell and proposes projects at North Andover and Lawrence, Mass. and Nashua, N.H.

	Cost		Annual Charges
Lowell, Mass.	$ 490,600	- Completed	$ 28,300
North Andover, Ms.	440,000	- Estimated	18,900
Lawrence, Mass.	460,000	- Estimated	19,800
Nashua, N.H.	268,000	- Estimated	12,600
	$ 1,658,600		$ 79,600

(Rev. June 1946)

16. Recommendation. - It is recommended that the construction of the West Peterboro Reservoir for the control of floods on the Nubanusit Brook be authorized, the construction to conform with the project plan as established herein.

C. T. HUNT
Colonel, Corps of Engineers
District Engineer

SECTION D.

APPENDICES

WAR DEPARTMENT
United States Engineer Office
Boston, Massachusetts

DEFINITE PROJECT REPORT

WEST PETERBORO RESERVOIR

GENERAL PLANS OF PROJECT

To accompany definite project report
dated May 1945 (Revised June 1946)

DEFINITE PROJECT REPORT

WEST PETERBORO RESERVOIR

GENERAL PLATES

Plate	Title
A-1	Flood Control Projects, Merrimack River Basin
A-2	River Profiles Showing Proposed Comprehensive Flood Protection System
A-3	Contoocook Valley, Location Map of Proposed Reservoirs and Drainage Basins
A-4	Reservoir Map
A-5	Aerial Photo with Location of Dam
A-6	Aerial Photo with Location of Spillway

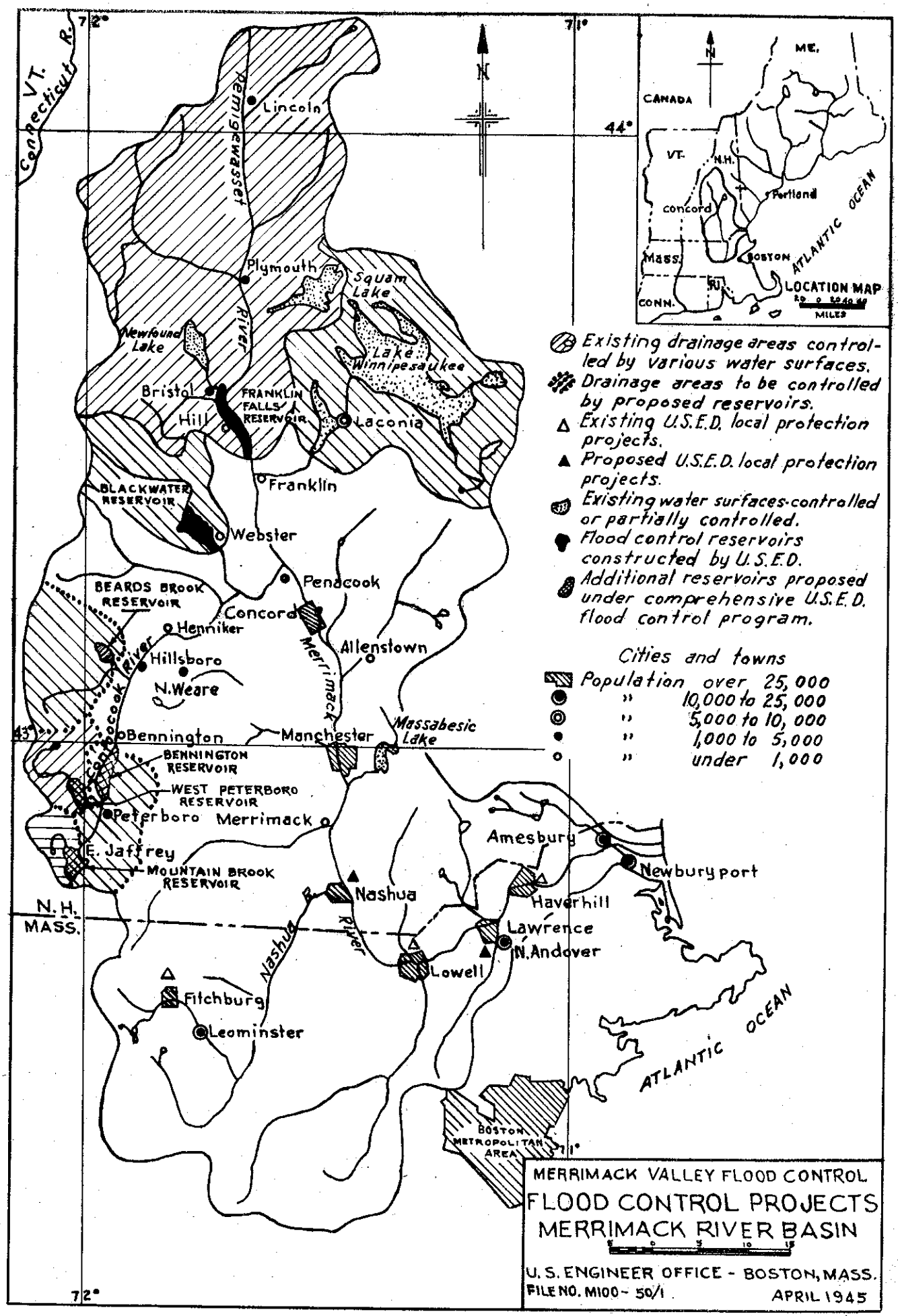
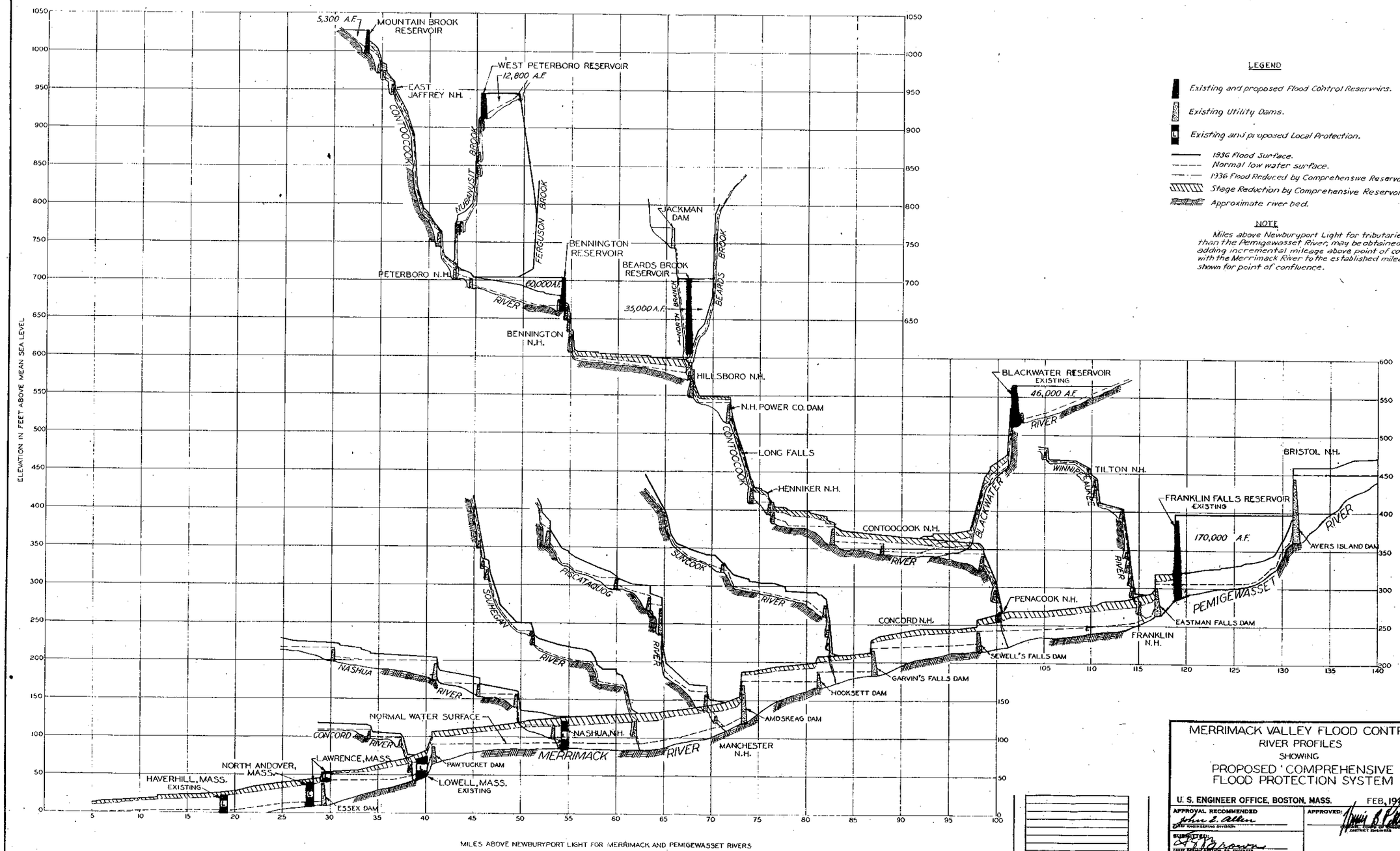

Connecticut R.
VT.
Pemigewasset
River
Lincoln
Plymouth
Squam Lake
Newfound Lake
Lake Winnipesaukee
Bristol
Hill
FRANKLIN FALLS RESERVOIR
Laconia
Franklin
BLACKWATER RESERVOIR
Webster
Penacook
BEARDS BROOK RESERVOIR
Concord
Henniker
Merrimack
Allenstown
Hillsboro
N. Weare
Contoocook River
Massabesic Lake
Bennington
Manchester
BENNINGTON RESERVOIR
WEST PETERBORO RESERVOIR
Peterboro
Merrimack
E. Jaffrey
MOUNTAIN BROOK RESERVOIR
Amesbury
Newburyport
Nashua
Haverhill
Lawrence
N. Andover
Nashua River
Lowell
Fitchburg
Leominster
N. H.
MASS.
BOSTON METROPOLITAN AREA
ATLANTIC OCEAN
72°
71°
44°
43°
CANADA
ME.
VT.
N.H.
Portland
Concord
Mass.
BOSTON
CONN.
R.I.
LOCATION MAP
MILES
Existing drainage areas controlled by various water surfaces.
Drainage areas to be controlled by proposed reservoirs.
Existing U.S.E.D. local protection projects.
Proposed U.S.E.D. local protection projects.
Existing water surfaces controlled or partially controlled.
Flood control reservoirs constructed by U.S.E.D.
Additional reservoirs proposed under comprehensive U.S.E.D. flood control program.
Cities and towns
Population over 25,000
" 10,000 to 25,000
" 5,000 to 10,000
" 1,000 to 5,000
" under 1,000
MERRIMACK VALLEY FLOOD CONTROL
FLOOD CONTROL PROJECTS
MERRIMACK RIVER BASIN
U. S. ENGINEER OFFICE - BOSTON, MASS.
FILE NO. M100-50/1
APRIL 1945

PLATE A-1



LEGEND

- Existing and proposed Flood Control Reservoirs.
- Existing Utility Dams.
- Existing and proposed Local Protection.
- 1936 Flood Surface.
- Normal low water surface.
- 1936 Flood Reduced by Comprehensive Reservoir System.
- Stage Reduction by Comprehensive Reservoir System.
- Approximate river bed.

NOTE

Miles above Newburyport Light for tributaries, other than the Pemigewasset River, may be obtained by adding incremental mileage above point of confluence with the Merrimack River to the established mileage shown for point of confluence.

MERRIMACK VALLEY FLOOD CONTROL
RIVER PROFILES
SHOWING
PROPOSED COMPREHENSIVE FLOOD PROTECTION SYSTEM

U. S. ENGINEER OFFICE, BOSTON, MASS. FEB. 1945

APPROVAL RECOMMENDED: John E. Allen, CHIEF ENGINEERING DIVISION

SUBMITTED: A. B. Brown, CHIEF DESIGN SECTION, SA. ENGINEER

Ellis F. Childs, ENGINEER

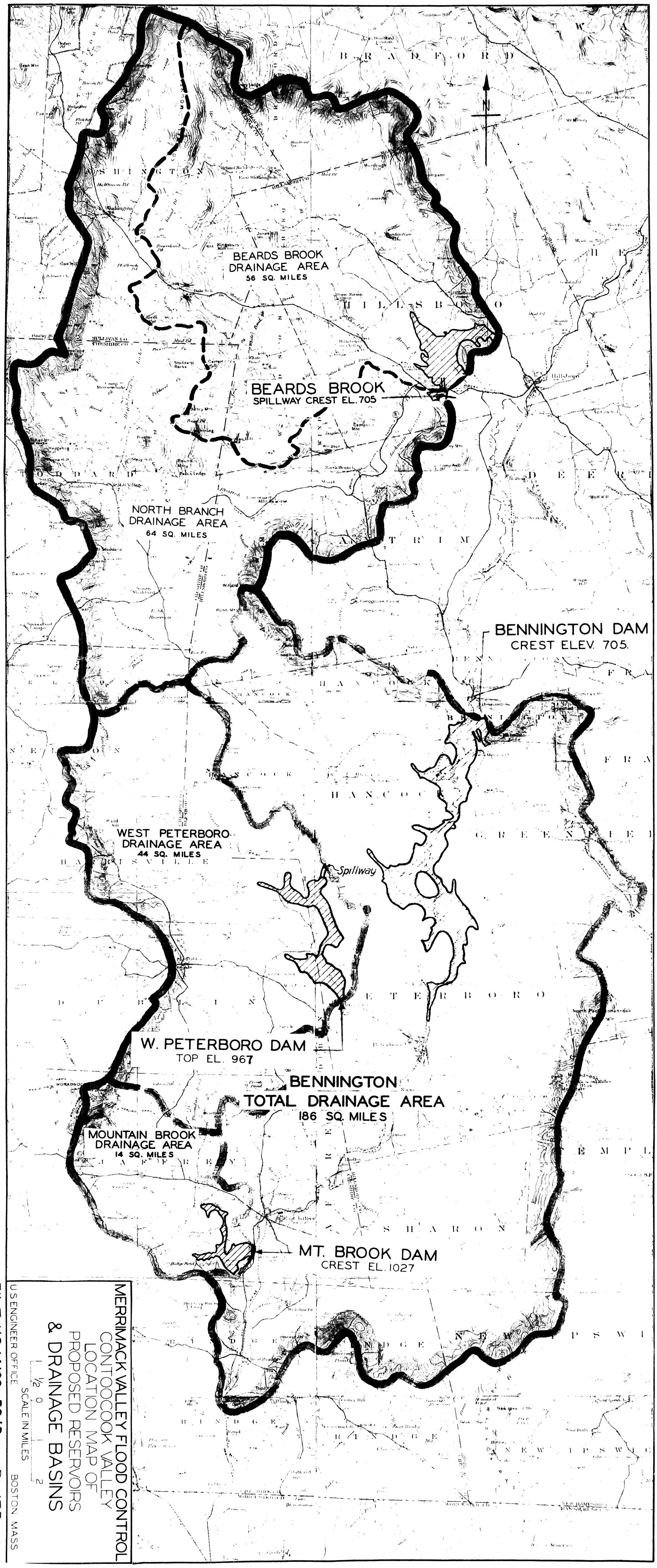
APPROVED: [signature], COLONEL, CORPS OF ENGINEERS, DISTRICT ENGINEER

NO.	CHARACTER	DATE	BY

REVISIONS

FILE NO. M100-50/2

PLATE A-2



MERRIMACK VALLEY FLOOD CONTROL
CONTOOCOOK VALLEY
LOCATION MAP OF
PROPOSED RESERVOIRS
& DRAINAGE BASINS

½ 0 2
SCALE IN MILES

U S ENGINEER OFFICE BOSTON MASS

FILE NO. M100-50/3 PLATE A-3

MERRIMACK VALLEY FLOOD CONTROL

WEST PETERBORO RESERVOIR

RESERVOIR MAP

SCALE IN FEET

1000 0 1000 2000 3000

U. S. ENGINEER OFFICE, BOSTON, MASS

MAY, 1945 FILE NO. M 78-12-35

WAR DEPARTMENT CORPS OF ENGINEERS U.S. ARMY

MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR

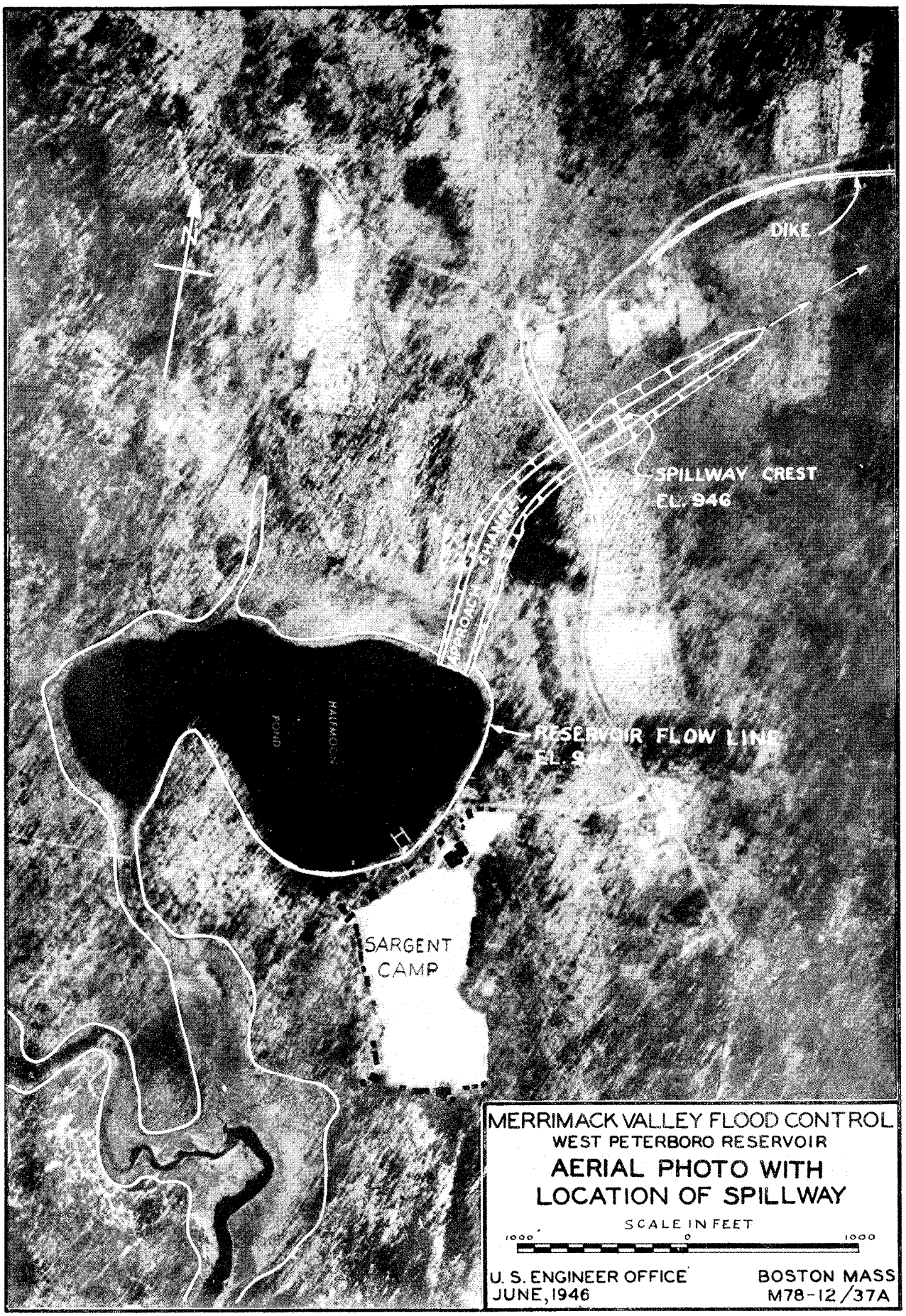
AERIAL PHOTO WITH LOCATION OF DAM

SCALE IN FEET
1000 0 1000

U.S. ENGINEER OFFICE
MAY 1945

BOSTON, MASS.
M78-12/36

PLATE A-5



MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR

AERIAL PHOTO WITH LOCATION OF SPILLWAY

SCALE IN FEET
1000' 0 1000

U. S. ENGINEER OFFICE
JUNE, 1946

BOSTON MASS
M78-12/37A

War Department
United States Engineer Office
Boston, Massachusetts

DEFINITE PROJECT REPORT

WEST PETERBORO RESERVOIR

APPENDIX I

HYDROLOGY

To accompany definite project report
dated May 1945 (Revised June 1946)

DEFINITE PROJECT REPORT

WEST PETERBORO RESERVOIR

APPENDIX I - HYDROLOGY

C O N T E N T S

Paragraph	Title	Page
a.	Scope	I-1
b.	General Discussion of Climate	I-1
c.	History of Floods	I-2
d.	Project Description	I-2
e.	Basin Characteristics	I-3
f.	Natural Storage	I-3
g.	Stream Characteristics	I-3
h.	Precipitation Records	I-4
i.	Stream Flow Data	I-7
j.	General Criteria	I-8
k.	Maximum Storm Studies	I-8
l.	Unit Hydrograph	I-8
m.	Reservoir Routing Assumptions	I-10
n.	Comparison of Seasonal Floods	I-10
o.	Computed Spillway Floods	I-11
p.	Spillway Design Flood	I-12
q.	Effect of Valley Storage	I-12
r.	Determination of Top of Dam	I-12
s.	Downstream Channel Capacity	I-13
t.	Reservoir Capacity	I-13
u.	Normal Operation	I-14
v.	Operation During Floods	I-14
w.	Effect on 1936 and 1938 Floods	I-15
x.	Time to Empty Reservoir	I-16
y.	Backwater in Reservoir	I-17
z.	Cofferdam Design	I-17
aa.	Stage Frequency Curve	I-18

DEFINITE PROJECT REPORT

WEST PETERBORO RESERVOIR

APPENDIX I - HYDROLOGY (Cont'd)

PLATES

Plate	Title
I-1	Drainage Area
I-2	Area and Capacity Curves
I-3	Drainage Areas for Hydrologic Studies
I-4	Basin Characteristics
I-5	Hydrograph at W. Peterboro, N.H., 1921 to 1931
I-6	Hydrograph at W. Peterboro, N.H., 1932 to 1942
I-7	South Branch Ashuelot River, Near Webb, N. H. Unit Hydrograph Determination (Storm of September 1938)
I-8	Unit Hydrograph Determination, South Branch Ashuelot River - Data Sheet.
I-9	Otter Brook Near Keene, N. H. - Unit Hydrograph Determination (Storm of September 1938)
I-10	Unit Hydrograph Determination, Otter Brook - Data Sheet
I-11	Baker River at Wentworth, N. H. Unit Hydrograph Determination (Storm of June 14-15, 1942)
I-12	Unit Hydrograph Determination, Baker River - Data Sheet
I-13	Unit Hydrograph Data
I-14	Curves of Limiting Rainfall
I-15	Comparison of Seasonal Floods
I-16	Computed Spillway Floods
I-17	Spillway Design Flood
I-17A	Spillway Design Flood (Revised June 1946)
I-18	Flood Discharges, New England States
I-19	Time to Empty Reservoir
I-20	Reservoir Operation, Effect on March 1936 Flood
I-21	Reservoir Operation, Effect on September 1938 Flood
I-22	Stage - Frequency Curve

(Rev. June 1946)

DEFINITE PROJECT REPORT
WEST PETERBORO RESERVOIR

APPENDIX I - HYDROLOGY

INTRODUCTION

a. Scope.- The data contained in this appendix constitute a report on the hydrology and topographic features of the drainage basin of the West Peterboro Reservoir as affecting the spillway design flood and spillway requirements in accordance with the procedure outlined in the Engineer Bulletin R. & H. No. 9, 1938. In addition, the report contains the data and procedure used in determining the reservoir design flood.

b. General Discussion of Climate.- A general summary of the climate prevailing over the Contoocook River watershed can best be described by reference to Page 5, Chapter 1 of Hydrometeorological Report No. 1 "Maximum Possible Precipitation Ompompanoosuc Basin", which states:

"The entire region is so situated geographically that it receives a maximum frequency of visitation of cyclonic storm activity through the entire year, in fact, this area eventually comes under the influence of the majority of cyclonic disturbances which affect the United States. No seasonable variation in precipitation is important, annual rainfall being evenly distributed throughout the months. In general, short period rainfall intensities are greatest in late spring and summer and excessive rains of longer durations occur in late summer and early fall. The extremes of average annual precipitation in New England range from 35 inches in northern Vermont and New Hampshire, with the exception of Mt. Washington where it is considerably higher due to orographic effects in the White Mountains, to 47 inches in southwestern Connecticut and 48 inches on the central coast of Maine. Excessive 24-hour amounts during winter months, December to March, of more than three inches are rare, but summer rainfall excesses are more frequent because of high intensities experienced in thunderstorms. A very even distribution of thunderstorm occurrence makes all parts of the region liable to high short-period rainfall intensities during the summer months.

"Since great accumulations of snow may occur and since the types of cyclonic disturbances which visit this region may result in rapid melting, snow is an extremely important factor in flood production. At Boston, Massachusetts, the average annual snowfall is 43.8 inches, with extremes of 96.4 inches during the winter of 1873-74 and 10 inches during the winter of 1875-76. The

average seasonal snowfall at first order Weather Bureau stations ranges from 50.4 inches at Albany, New York, 60.4 inches at Portland, Maine, and 73.0 inches at Concord, New Hampshire, to 38.6 inches at New Haven, Connecticut, and 32.4 inches at Providence, Rhode Island. The greatest average seasonal snowfall from any reporting station is 168.3 inches at Pittsburgh, New Hampshire."

c. History of Floods.- Records of great floods on the Nubanusit Brook are incomplete. The peak discharges of the March 1936 flood, Q = 4140 c.f.s., and the September 1938 flood, Q = 4100 c.f.s., were determined by the U. S. Geological Survey at the Verney Mills Dam, (Drainage Area, 45 square miles). A hydrograph is available for the 1927 flood in West Peterboro, (Drainage Area, 48 square miles), from the U. S. Geological Survey gaging station which was in operation from 1920 to 1931. This flood was not severe and the peak discharge of 1010 c.f.s. or 21 c.f.s. per square mile is only comparable to a heavy spring freshet.

d. Project Description.- The West Peterboro dam site is located about one-half mile upstream from West Peterboro on Nubanusit Brook, a tributary of the Contoocook River. (Plate I-1.) The proposed reservoir has a tributary drainage area of 44 square miles, which is 90 per cent of the drainage area of Nubanusit Brook and approximately 36 per cent of the total drainage area above Peterboro, New Hampshire. The reservoir is designed to provide flood protection principally for West Peterboro and Peterboro and to supplement to some extent the protection to be provided by the proposed Bennington Reservoir for the Contoocook River Valley between Bennington and Penacook, New Hampshire. The flood control storage to be provided is 12,800 acre-feet, equivalent to 5.5 inches over the drainage area. The area and capacity curves are shown on Plate I-2. The proposed reservoir, designed for retarding basin operation, has a single conduit through the embankment section. Three (3) gates in the conduit are provided for control of flood discharges. All three gates remain open during normal conditions, but during flood periods gates will be closed to reduce the discharge to safe channel capacity, for stream regulation and protection downstream. The proposed spillway, located about three miles from the dam site, will divert water into the adjacent Ferguson Brook watershed. The spillway crest is designed as a concrete

weir 100 feet long with crest at elevation 946 feet. The location of the spillway at this site requires an approach channel approximately 1500 feet long and a discharge channel approximately 700 feet long.

e. Basin Characteristics.- The watershed of Nubanusit Brook above the West Peterboro dam site is an elongated basin with a maximum length of 11 miles and a width of 5 miles. The long axis of the reservoir lies in a north and south direction (See Plate I-1). The area is drained by two principal tributaries; Nubanusit Brook which rises in the northern portion and flows generally southeasterly; and Stanley Brook which rises in the southern section and flows northeasterly. These tributaries unite within the proposed reservoir area about one mile above the proposed dam site. The drainage areas of the two tributaries are approximately equal. It is probable that flood flows from both areas will arrive in the reservoir at about the same time when produced by simultaneous uniform rainfall over the entire watershed.

f. Natural Storage. - The upper sections of both tributaries contain several comparatively large natural lakes which act as retarding basins. These lakes on Nubanusit Brook are Spoonwood Pond, Nubanusit Lake, Harrisville Pond, and Skatutakee Lake. Nubanusit Lake, the largest in the group, has an area of about one and one-half square miles and a storage capacity of about 4,600 acre-feet which capacity amounts to about 14 inches over its drainage area of 6 square miles. The basins on Stanley Brook are Thorndike Pond and Frost Pond and are located in the southern part of its drainage area. Most of these lakes have been increased somewhat beyond their natural size by the construction of small dams at the outlets. The additional capacity obtained by these artifical means is not the major portion of the capacity of the lakes, and the failure of these small dams would not result in the release of the entire volume of water stored in the lakes. These lakes are used to some extent for flow regulation, but are generally maintained at a maximum level for scenic and recreational purposes and cannot be counted on for appreciable flood control use. However on the previous recommendation of the Office, Chief of Engineers, an additional allowance of one foot in computed maximum water surface of the West Peterboro Reservoir is made to allow for the possible failure of these upstream embankments during the spillway design flood.

g. Stream Characteristics. - The outer margin of the drainage area is at high elevations, the west border averaging elevation 1800 feet with a mountain peak of 2800 feet, and the east border averaging elevation 1100 feet with a peak of 2000 feet. The steep slopes on the margin of the drainage area are

TABLE I. PRECIPITATION - INCHES

Station Years of Record	Fitzwilliam 21 years Average Monthly	Franklin 39 years Average Monthly	Keene 49 years Average Monthly	Manchester 66 years Average Monthly	Nashua 57 years Average Monthly	Newport 12 years Average Monthly
January	3.21	3.03	2.89	3.29	3.43	3.26
February	2.64	2.70	2.68	2.96	3.32	2.26
March	3.63	3.33	3.21	3.61	3.70	3.65
April	3.71	3.53	3.09	3.19	3.32	3.77
May	3.30	3.19	3.11	3.10	3.00	3.01
June	4.36	3.73	3.23	3.20	3.13	3.63
July	4.24	3.77	3.80	3.43	3.38	3.64
August	3.97	3.47	3.86	3.39	3.53	3.37
September	4.15	3.99	3.60	3.38	3.42	3.78
October	3.20	2.88	2.76	3.02	3.00	2.57
November	3.88	3.25	3.03	3.33	3.25	3.01
December	3.13	3.05	2.99	3.25	3.34	2.76
Average Annual	43.42	39.92	38.25	39.15	39.82	38.71

TABLE II. TEMPERATURE - FAHRENHEIT

Station Years of Record	Franklin 40 years Average Monthly Normal	Keene 48 years Average Monthly Normal	Manchester 12 years Average Monthly Normal	Nashua 15 years Average Monthly Normal
January	19.9	21.1	24.1	24.2
February	20.0	21.2	23.9	24.4
March	30.9	32.0	32.9	33.3
April	42.9	43.7	43.6	43.8
May	55.1	55.3	56.4	56.3
June	63.9	63.6	65.8	65.2
July	69.5	68.8	70.3	70.2
August	66.6	66.3	68.8	68.2
September	59.4	59.4	60.5	60.8
October	48.1	48.6	49.0	49.8
November	35.9	36.7	38.9	39.1
December	23.9	24.8	27.4	27.3
Annual Normal	44.7	45.1	46.8	46.9

TABLE III.

CLIMATOLOGICAL DATA FROM OBSERVATIONS AT
U. S. WEATHER BUREAU, CONCORD, N. H.

Month	TEMPERATURE Mean Max.	TEMPERATURE Mean Min.	TEMPERATURE Mean Monthly	Extremes Highest	Extremes Lowest	PRECIPITATION IN INCHES Monthly Mean	PRECIPITATION IN INCHES Greatest in 24 hrs.	UNMELTED SNOWFALL IN INCHES Monthly Mean	UNMELTED SNOWFALL IN INCHES Greatest in 24 hrs.
Length of Record Years	74	74	74	74	74	91	74	74	42
Jan.	29.2	8.9	19.0	72	-35	2.97	2.10	17.6	19.0
Feb.	31.0	9.6	20.3	68	-37	2.65	2.06	17.3	15.0
Mar.	38.4	19.8	29.1	82	-16	3.15	2.59	11.9	12.9
Apr.	52.5	30.6	41.6	92	7	2.96	2.37	4.6	18.3
May	64.8	40.9	52.9	98	22	3.07	3.05	0.1	2.6
June	73.5	49.9	61.7	101	32	3.23	4.47	T	T
July	79.0	55.9	67.4	102	38	3.70	5.11	0	0
Aug.	76.4	54.6	65.5	99	33	3.50	3.32	0	0
Sept.	69.0	47.1	58.0	96	20	3.48	5.97	T	T
Oct.	58.5	37.1	47.8	92	16	3.26	3.45	0.1	1.0
Nov.	44.4	26.4	35.4	80	-17	3.29	4.04	5.4	13.3
Dec.	32.9	15.2	24.0	65	-24	2.92	2.43	11.9	9.5
Year	54.2	33.0	43.6	102	-37	38.18	5.97	68.9	19.0

i. Stream Flow Data.- The U. S. Geological Survey maintained a gaging station on Nubanusit Brook from 1920 to 1931, which has a drainage area of 48.1 square miles. The maximum peak discharge during this period was 1,010 c.f.s., recorded during the November 1927 flood. Peak discharge values determined by the U. S. Geological Survey at a dam near West Peterboro (Drainage Area, 45.2 square miles) for the 1936 flood and the 1938 flood are 4,140 and 4,100 c.f.s., respectively.

Discharge records for other drainage areas nearby or adjacent were studied to obtain run-off characteristics. Records for the following stations were used in this study.

Basin	Name of Stream	Station Location	Drainage Area Square Miles	Max. Discharge c.f.s./ Sq.Mi.
Merrimack	Contoocook River	East Jaffrey	36.1	99
	Nubanusit Brook	West Peterboro	45.2	92
	No. Branch Contoocook River	Antrim	54.8	85
	Warner River	Bradford	19.7	120
	Souhegan River	Greenville	29.9	206
	Stony Brook	Wilton	33.2	174
Connecticut	Moss Brook	Wendell Depot	12.2	126
	So. Branch Ashuelot River	Webb	36.6	163
	Otter Brook	Keene	41.8	147

A hydrograph for the period 1920 to 1942 has been constructed for general information, and is shown on Plates I-5 and I-6. The first half of this hydrograph is plotted directly from U. S. Geological Survey records for the Nubanusit Brook station mentioned above. The last half is estimated by comparison with run-off from the adjacent Otter Brook watershed, (Drainage Area, 41.8 square miles).

The following table illustrates the magnitude of the discharges from the daily average to the spillway design flood:

Average flow	80 c.f.s.
Average annual flood	800 c.f.s.
Flood of November 1927	1,000 c.f.s.
Flood of March 1936	4,140 c.f.s.
Flood of September 1938	4,100 c.f.s.
Spillway Design Flood (Reservoir Inflow)	36,300 c.f.s.
Spillway Discharge	17,800 c.f.s.

DETERMINATION OF SPILLWAY CAPACITY

j. General Criteria.- The spillway capacity for West Peterboro is determined by procedures set up in Engineer Bulletin R. & H. No. 9, 1938. The spillway design flood is based on the highest possible rainfall and very high run-off factors and is assumed to occur when all outlets are inoperative and the reservoir is filled to spillway lip. All storage of the spillway design flood is surcharge storage based on these assumptions. The elevation of the top of the dam is determined by adding a minimum free-board of 5 feet to the maximum water surface elevation produced by the spillway design flood. The analytical investigations made for the selection of the spillway design flood and spillway capacity are described in the succeeding paragraphs.

k. Maximum Storm Studies.- Detailed studies were made of both the summer-fall and winter-spring limiting storm conditions to be expected over the watershed. The rainfall intensity curve for summer-fall conditions shown on Plate I-14 is based on the maximum possible rainfall for the Contoocook River Basin as determined by the Hydrometeorological Section of the U. S. Weather Bureau. The flood-producing storm is of 24-hour duration with a total rainfall of 19.9 inches. The winter-spring limiting storm conditions with run-off from snow melt are taken from an extrapolation of data on Figure 41 in the Hydrometeorological Report No. 1, "Maximum Possible Precipitation over the Ompompanoosuc Basin above Union Village, Vermont", (March 1940). This storm is of 30-hour duration, comprising 12.6 inches of rain and 10.8 inches of snow melt. The distribution of rainfall and snowmelt in these two storms is shown graphically on Plate I-15.

l. Unit Hydrograph.- (1) Lack of Nubanusit Brook Data.- An analysis of the stream flow data available on Nubanusit Brook discloses that the data is unsuitable for determining unit hydrographs applicable to spillway design floods. The flood of November 1927 is the largest on which adequate data are available, but the peak flow of this flood was only 1,130 c.f.s. Some information was obtained during two moderately heavy storms in June and September 1944, but the data were too unreliable to be used for unit hydrograph study.

(2) Data from Adjacent Watersheds.- Consideration was given to records available on other New England streams of about 44 square miles drainage area with the object of obtaining data on the largest flood on streams with physical characteristics conducive to flood peak values comparable to the Nubanusit Brook. The best information obtainable was the records of the U. S. Geological Survey gaging stations for South Branch Ashuelot River at Webb, New Hampshire, (Drainage Area - 36.6 square miles)

and Otter Brook near Keene, New Hampshire, (Drainage Area 41.8 square miles) See Plate I-3. Both areas are in the Connecticut River Basin adjacent to Nubanusit Brook and have recorded stage hydrographs for the 1938 flood. The flood record of the Bakers River at Wentworth, New Hampshire (Drainage Area 59 square miles) was also analyzed. The terrain of the Bakers River Basin is more mountainous than the Nubanusit drainage area, and the relationship is not quite as comparable as the South Branch Ashuelot River and Otter Brook Basins. The unit hydrograph study of these basins conforms to the instructions contained in the letter from the Office of the Chief of Engineers, dated 23 October 1943, Subject: "Compilation of Basic Data for Determination of Unit Hydrographs", and are shown on Plates I-7 to I-12. None of the recorded floods approach the magnitude of the spillway design flood and therefore they cannot be used directly for determining unit hydrographs applicable to the spillway design storm.

(3) Synthetic Graphs.- Synthetic graphs were developed using the empirical method developed by F. F. Snyder for use in determining peak discharge, lag, and duration of unit hydrograph. An explanation of the methods and coefficients used is contained in the Article "Synthetic Unit Graphs" by F. F. Snyder, Transactions American Geophysical Union, 1938. The fundamental formulas for the analysis of the unit hydrograph are:

$$(1) \qquad t_p = C_t \, (L_{co} L)^{0.3}$$

$$(2) \qquad q_p = C_p \, 640/t_p$$

The nomenclature corresponds to the established symbols for hydrograph study and is briefly described in the tabulation on Plate I-13. In analysing flood hydrographs of record the values of L_{co} and L are obtained from topographic maps, values of t_p and q_p are obtained from the records of precipitation and discharge records, and the coefficients C_t and C_p are computed by the formula. Construction of unit hydrographs for tributaries without any stream-flow records involves determination of the L_{co} and L values from topographic maps, assuming values of C_t and C_p based on derived coefficients from the records of comparable basins, and the computation of the unit graph characteristics t_p and q_p.

The values of C_p and C_t selected for West Peterboro Reservoir are based on computed values derived from the South Branch of the

Ashuelot River, Otter Brook, and Bakers River. Three pairs of coefficients are assumed and used to develop three unit-hydrographs of various concentrations and peak discharges. This procedure is followed to provide a range in the computed spillway floods and to determine the effect of various unit hydrographs on the spillway requirements. By increasing the severity of the unit hydrograph an allowance also is made for the more intense rate of precipitation used in the computed spillway floods as compared with the smaller rates of rainfall used in deriving unit hydrographs from floods of record.

m. Reservoir Routing Assumptions. - It is assumed that the reservoir would be filled to normal maximum pool elevation of 946 feet above M.S.L., the crest of the spillway, at the beginning of the spillway design flood. The outlets are assumed to be inoperative. The spillway rating curve (Plate III-3) is computed for a free overfall Ogee spillway using the weir formula $Q = CLH\ 3/2$, where "L" = 100 feet and values of "C" of an Ogee section are used from 3.0 to 3.7 for the maximum head. It is necessary to compute friction and other losses in determining the relation between spillway discharge and reservoir elevation because of the length of the approach channel. A more detailed analysis of the spillway is included in Appendix III. Since the unit hydrograph is synthetic, it is assumed that it applies to reservoir inflow, consequently valley storage is neglected and the spillway floods are routed through the reservoir using the gross storage above the spillway crest.

n. Comparison of Seasonal Floods. - Spillway floods are computed, using unit hydrograph No. 1 shown on Plate I-13 applied to both summer-fall and winter-spring precipitation conditions. An infiltration rate of 0.05 inches per hour and a base flow of 5 c.f.s. per square mile are used. When these two floods are routed through the spillway surcharge storage, it is found that the spillway discharges are practically the same magnitude. Plate I-15 shows the comparison of these two floods and the results of the routing. Pertinent data concerning the computed summer-fall and winter-spring floods are shown in the following table:

		Computed Summer-Fall Flood	Computed Winter-Spring Flood
Duration of storm	hours	24	30
Rainfall	inches	19.9	12.6
Snow melt	inches	none	10.8
Total	inches	19.9	23.4
Rate of infiltration,	inches per hour	0.05	0.05
Runoff	inches	18.9	21.9
Volume	acre feet	44,300	51,400
Peak inflow	c.f.s.	27,200	20,400
Peak outflow	c.f.s.	15,300	15,500
Maximum Reservoir El.	feet	958.9	959.0

Despite the similarity between the spillway discharges from the two seasonal floods the winter-spring hydrologic conditions are discarded in favor of the summer-fall storm because it is concluded that the higher rates of precipitation to be expected during the summer and fall justify the application of the higher unit hydrographs. The snow-melt of the winter-spring storm runs off at a comparatively slow constant rate thus justifying the lower unit hydrograph (No. 1) but not the more severe unit hydrographs (Nos. 2 and 3).

o. Computed Spillway Floods.- Plate I-16 shows the three computed spillway floods derived from the maximum possible precipitation of the summer-fall flood combined with the three empirical unit hydrographs shown on Plate I-13. The three unit hydrographs of different characteristics are used to obtain a range in computed spillway floods in order to determine the effect on the design criteria of the spillway. It is assumed that unit hydrograph No. 1 is applicable to all periods of rainfall less than two inches in a three-hour period, unit hydrograph No. 2 is used for rainfall excess values exceeding two inches to produce hydrograph "N", and graph No. 3 is used for rainfall excess periods to produce hydrograph "O". Routing these three hydrographs through the reservoir with the spillway length of 100 feet results in maximum spillway surcharges that vary between 12 and 15 feet. (See summary and graph on Plate I-16*.) This small difference in surcharge indicates that for a constant volume of inflow hydrograph, the spillway discharge does not change materially even with considerable variation in the shape and intensity of the hydrograph. To determine the effect of variations in volume, the ordinates of hydrograph "M" were increased 25%, 50% and 100%, and the resulting hydrographs were routed through the surcharge storage. The hydrographs are not illustrated but the effect of the increased volume is shown on

*All routing computations are based on original Discharge Curves shown on Plate III-3, dated May, 1945.

Plate I-16 by the curve of maximum water surface elevation versus the per cent increase in volume. A 25% volume increase raises the surcharge from 13 to 16 feet but still results in a freeboard of five feet. The curve indicates that the dam would not be overtopped until the volume had increased approximately 90%, a percentage which is more than adequate as a factor of safety.

p. Spillway Design Flood.- After consideration of the various factors involved in the methods of analyzing precipitation, run-off and constructing flood hydrographs, their relative degree of accuracy, and their effect on the spillway requirements, it was decided to adopt hydrograph "N" as the spillway design flood. Pertinent data relative to the spillway design flood is tabulated below:

Rainfall in inches in 24 hours	19.9
Rate of infiltration (inches per hour)	0.05
Run-off in inches	18.7
Run-off volume in acre-feet	43,800
Peak inflow in c.f.s.	36,300

The Meyer Coefficient for the inflow peak of 36,300 c.f.s., used in the expression $Q = c \sqrt{\text{Drainage Area}}$, is approximately 5500. The peak of the spillway design flood expressed in terms of c.f.s. per square mile is plotted against the drainage area on Plate I-18.

The adopted spillway design flood is shown on Plates I-17 and I-17A with the spillway discharge reservoir stage elevation, pluvigraph and unit hydrographs. Plate I-17 is based on the discharge rating curves of the original layout of the spillway weir and discharge channels dated May 1945, while Plate I-17A is based on the discharge rating curves of the revised spillway layout dated June 1946. Data relative to the reservoir discharges and water stages for the spillway design flood, as shown on the revised Plate I-17A are as follows:

Peak Spillway discharge in c.f.s.	17,800
Maximum water surface (ft. above MSL)	960.8
Surcharge storage utilized, acre-feet	18,600
Surcharge storage utilized, inches	8.0

q. Effect of Valley Storage.- Evaluation of the valley storage in the reservoir area has not beem made. The storage might be appreciable during a spillway flood, but the evaluation would be applying a questionable refinement to a hydrograph that is based on a theoretical flood-producing storm and synthetic unit hydrographs. Consequently, the effect of valley storage is neglected and the spillway flood is considered an inflow to the reservoir and the gross surcharge storage is considered effective in the routing computations.

r. Determination of Top of Dam.- The elevation of the top of dam is determined by adding the required freeboard to the maximum water surface elevation at the time of the maximum

head on the spillway. The theoretical freeboard required for wave height is computed by the Stevenson formula from the following data:

Fetch in miles	1.5
Wind velocity, miles per hour..	80
Angle of wind and fetch	0
Depth of water in feet	30

Since the theoretical freeboard is less than the minimum value of 5 feet prescribed in Engineers Bulletin R. & H. No. 9, 1938, the latter value is used. Due to the questionable stability of the upstream timber-crib dams and their possible failure during a severe flood, an additional allowance of one foot in the computed maximum water surface was made on the recommendation of the Office of the Chief of Engineers.

The top elevation of the West Peterboro Dam is determined by the following data:

Elevation, crest of spillway	946.0
Maximum head on spillway from spillway design flood plus backwater losses	14.8
Freeboard	5.0
Allowance for failure of upstream dams ...	1.0
	966.8
Adopted elevation for top of dam	967.0

RESERVOIR DESIGN FLOOD

s. Downstream Channel Capacity.- Field investigations indicate that the channel capacity of the Nubanusit Brook is approximately 1000 c.f.s. There are six dams between the site of the proposed dam and the mouth of the Nubanusit Brook. The lower dam of the Verney Mills limits the safe reservoir discharge to the 1000 c.f.s. with a reasonable freeboard below abutments. The other dams further downstream are adequate for higher discharges, but the river channel between West Peterboro and Peterboro is sluggish and will not allow much more flow without considerable overflow and some encroachment on residential property.

t. Reservoir Capacity.- Results of studies for flood control reservoirs in this district indicate that, if possible, it is desirable to provide approximately 6 inches of storage to fully control the maximum flood of record without exceeding the downstream channel capacity. Six inches of storage at West Peterboro would require 14,100 acre-feet with the spillway crest at elevation 948.0. The maximum reservoir stage, however, is limited to elevation 946.0 by the Sargent School Camp of Boston

University, which is located near Half Moon Pond. The camp represents a considerable investment and is only a few feet above the water level of the Pond which is at elevation 945.2. Consequently, the spillway crest is established at elevation 946.0, which provides 12,800 acre-feet of storage, or 5.5 inches. This storage is slightly less than the prescribed amount of 6 inches, but as discussed further in paragraphs v. and w., the protection to the downstream communities is guaranteed by the fact that spillway discharges caused by excessively large floods is diverted from the Nubanusit Brook directly into Bennington Reservoir.

u. Normal Operation.- The existing effective storage capacity of the Verney Mills Dam lies between elevations 906.5 and 911.1 (top of present flashboard) and is estimated to be approximately 300 acre-feet. The maximum discharge capacity of the power installation is 120 c.f.s. As a considerable portion of this storage is upstream from the West Peterboro Dam, it is proposed to compensate the hydraulic loss of head through the discharge conduit by replacing the existing 18" flashboards on the Verney Mills Dam with 36" flashboards. The higher flashboard elevation provides some additional storage capacity and allows for the loss of head in the conduit. Consequently, during normal river conditions, the flood control dam will require no operation, the outlet works gates will be left open and all regulation of the flow will take place at the Verney Mills Dam similar to the present conditions.

v. Operation During Floods.- The dam will be operated to provide the maximum flood protection to downstream areas, particularly to the Town of Peterboro. The combination of a saddle spillway and three outlet gates affords alternate methods of regulating the flood discharge as discussed below. The method selected in each instance will probably depend upon the magnitude of the flood, the extent of damage being wrought in Peterboro, and the experience obtained during previously controlled floods. The operation of the reservoir will be governed primarily by flood stages in the Town of Peterboro, Two methods of operation are as follows:

(1) The reservoir may be operated with continuous discharge keeping all three gates wide open until a discharge of about 1000 c.f.s. is reached (El. 922 ±). From this elevation up to El. 936 ± the gates are throttled to limit the discharge to about 1000 c.f.s. Above El. 936 the head is considered too great for partial gate openings; therefore the side gates are closed and the center gate opened wide. The discharge thru this gate will vary from about 770 c.f.s. at El. 936 to about 920 c.f.s. at Spillway Crest, El. 946. Operation by this method reduces discharges throughout the flood and gives more effective flood control than the usual retarding basin operation.

(2) The reservoir may be operated as above until such time as damaging discharges occur at Peterboro on the Contoocook River. All gates would then be closed for a period of one to two days, thus reducing the discharge in Nubanusit Brook to the inflow from the uncontrolled 4 sq. miles below the damsite. This period of closure would be governed by the magnitude of the flood and by local conditions. With extremely large floods this operational procedure will result in spillway discharge but this is not objectionable because spillway discharge by-passes Nubanusit Brook and flows directly into the proposed Bennington Reservoir.

w. Effect on 1936 and 1938 Floods.- The two maximum floods of record, occurring in March 1936 and September 1938, have been used as reservoir design floods to check the adequacy of the storage design discharge, the proposed method of operation, and the probability of spillway discharges during the operation scheme. These two floods are selected to illustrate the range in the reservoir's effectiveness; the 1936 being a prolonged three-storm flood with considerable volume, and the 1938 a single peak flood caused by one concentrated storm producing practically the same peak magnitude as the 1936 flood. The hydrographs of the two floods are based on computed peak discharges at the Verney Mills Dam with the shape and volume of the hydrographs determined from flood records on adjacent and comparable rivers. The constructed hydrographs were assumed to represent the reservoir inflow and consequently were routed through the gross reservoir storage.

Plates I-20 and I-21 show the effect of the reservoir on the 1936 and 1938 flood with two alternate methods of operation. In one case it is assumed that the reservoir is utilized as a simple retarding basin, and in the second case, all gates are closed during the period of damaging stages on the Contoocook River in Peterboro. Pertinent data concerning these two floods with the two described methods of operation and the flood reduction in Peterboro are summarized as follows:

	1936 Flood		1938 Flood	
	Gates Open	Gates Closed During Peak	Gates Open	Gates Closed During Peak
Peak inflow, c.f.s.	4,140	4,140	4,100	4,100
Maximum reservoir stage, MSL	945.9	947.3	937.8	944.0
Outlet discharge at max. stage, c.f.s.	920	930	800	890
Spillway discharge, c.f.s.	0	560	0	0
Total discharge at max. stage, c.f.s.	920	1,490	800	890
Flood control, storage used, Acre Ft.	12,700	13,650	8,200	11,200

	1936 Flood		1938 Flood	
	Gates Open	Gates Closed During Peak	Gates Open	Gates Closed During Peak
Maximum uncontrolled discharge on Nubanusit Brook at Peterboro, c.f.s.	4,600	4,600	4,550	4,550
Maximum controlled discharge on Nubanusit Brook at Peterboro, c.f.s.	1,300	500	1,200	500
Maximum uncontrolled discharge on Contoocook River at Peterboro, c.f.s.	9,000	9,000	10,000	10,000
Maximum controlled discharge on Contoocook River at Peterboro	6,600	5,800	7,400	6,700

The table indicates the justification and advantage in the method of gate operation in respect to reduced discharges in the Town of Peterboro. Closing the gates in the West Peterboro Dam decreases the discharge in the Contoocook River an additional 800 c.f.s., or an increase in the reservoir effectiveness of practically 15%. The location of the spillway in the reservoir design makes this plan of operation practical and assures positive flood control for Nubanusit Brook. If closing the gates results in over-topping the spillway, the discharge bypasses the village where it might cause damage, and flows directly into the proposed Bennington Reservoir.

x. Time to Empty Reservoir.- Assuming the reservoir at spillway crest, elevation 946, with continuous operation as described in Paragraph v (1), and a constant inflow of 5 c.f.s. per square mile, it will take about ten (10) days to completely empty the reservoir (Plate I-19). However, 3 inches of storage, equivalent to 55% of the total storage, will be available in about five days and 5 inches, equivalent to 90%, in about eight days. This is considered adequate protection against a possible second flood. Emptying the reservoir is shown also on Plates I-20 and I-21 for the March 1936 flood and the September 1938 flood.

y. Backwater in Reservoir.- The West Peterboro Reservoir is different from the usual reservoir inasmuch as the spillway is located in the upstream section of the reservoir instead of at the dam site. The effect of backwater tends to raise the water surface elevation at the dam site during the spillway discharges instead of the customary backwater extending upstream. On Plates A-4 and I-1 it is to be noted that the points of inflow to the reservoir may be divided principally between two tributaries, (1) the Nubanusit Brook entering from the northwest with a drainage area of approximately 24 square miles, and (2) the Stanley and Brush Brooks that enter from the southwest with a drainage area of approximately 20 square miles. The inflow from Nubanusit Brook flows only a short distance through the upper reach of the reservoir before being diverted northeasterly into Half Moon Pond and to the saddle spillway. The present outlet to Half Moon Pond to the Nubanusit Brook flows southerly through a wooded section. It is proposed to clear an area through this section 200 feet wide to allow the spillway discharge to flow into Half Moon Pond with the minimum lost head. It should be noted that when the reservoir stages rise to elevation 960 (spillway design flood) the reservoir widens considerable beyond the flow line shown on Plate A-4 and that many of the apparent restrictions disappear. The inflow from the southern part of the reservoir flows north to the saddle spillway, but the cross-sectional area is sufficiently large to eliminate any appreciable difference in water surface elevations from friction losses. The relative timing of the peak inflow and the maximum reservoir stage is important in considering the possible effect of backwater on the design height of the dam. Plate I-17 shows that the maximum inflow of 36,300 c.f.s. occurs when the reservoir stage is approximately elevation 957, and that the inflow during the maximum stage, elevation 960.8, has decreased to about 18,000 c.f.s. The freeboard allowance of 6.2 feet above the maximum reservoir stage provided for the embankment section is adequate provision for any small backwater effect that may occur in the length of the reservoir during the flow condition of the maximum reservoir stages.

z. Cofferdam Design.- The construction schedule requires cofferdams to be constructed in the spring of the second year for construction of the embankment section. All the river flow then will be diverted through the conduit completed during the first construction year. The cofferdams will be placed following the spring run-off from snow-melt and consequently the annual high flows in April can be neglected. The selected elevation 918.0 for the upstream cofferdam is based on providing approximately 2 feet of freeboard for a flood with an inflow peak of about 1000 c.f.s. The pool at elevation 916.0 provides 1200

acre-feet of storage (0.5 inches) and the outlet has a discharge capacity of about 720 c.f.s. The downstream cofferdam will be constructed to elevation 915.0 which provides sufficient freeboard over the maximum pool elevation that may be anticipated above the Verney Mills Dam.

aa. Stage Frequency Curve (Plate I-22).- A study was made to determine the probable reservoir stage equalled or exceeded in a designated number of years. In this connection, two distinct steps were required: (1) to construct a natural discharge volume frequency curve for the West Peterboro site, and (2) to construct a stage frequency for the reservoir which is a direct result of the inflow volume. Available stream flow data on the Nubanusit Brook, required for the first step, is limited to 11 years of record near Peterboro, New Hampshire, (Drainage Area, 48 square miles) from November 1920 to September 1931. No large floods occurred during this period. Beca use of the lack of essential stream flow data on the Contoocook River, it was necessary to utilize records of an adjacent watershed. Otter Brook near Keene, New Hampshire, (Drainage Area, 41 square miles) with records available from October 1923 up to the present time was selected for this purpose. Comparison of the Nubanusit Brook records with the concurrent Otter Brook records indicates that differences of considerable magnitude frequently occur in comparable one day volumes, however, comparable maximum three-day volumes per square mile from the two drainage areas check very closely. It was also determined that the reservoir stage is dependent principally on volume, and that the type of hydrograph, that is a flash flood or a long flat flood, has little influence on the reservoir sta ge. It was, therefore, concluded that stream flow records on Otter Brook, modified for the West Peterboro drainage area could be utilized after 1931 when the records on Nubanusit Brook was discontinued. From these data, a three-day volume frequency curve was constructed using the formula, $Y = \frac{M}{N - 0.5}$ in which Y = the probability of occurrence in years, M = the years of available records, and N = the summation of occurrences.

The 1936 flood, however, was arbitrarily set as the 1% chance of occurrence (i.e. 100 year flood) in order to agree with frequency curves for nearby streams with longer records. Several floods similar in shape to the 1936 flood were routed through the reservoir for both methods of gate operation to determine maximum reservoir stage. In this connection, it should be noted that the 1936 flood was a 3-peak flood; when routed thru the reservoir, the second peak takes up most of the storage, but the third peak causes the maximum water surface elevation. Use of the 1936 flood or smaller floods similar to it for the stage frequency curves is quite conservative, as is shown by the points for the 1938 flood routed through the reservoir. The period of gate closure is shorter with small floods because the period of damaging discharges at Peterboro correspondingly is shorter; hence, it is believed

(Rev. June 1946)

that with floods of about 10% chance of occurrence no period of gate closure is required, and therefore the two curves of reservoir stage converge at about the 10 year frequency.

Plate I-22 shows the stage-frequency relationship for the two described methods of operation (paragraph v), that is, (a) with continuous conduit discharge, and (b) with gates completely closed during period of maximum discharges in the town of Peterboro. It is to be noted that with the continuous discharge it is estimated the reservoir will be filled to spillway crest once in a 100-year frequency, while with the gates closed during the maximum discharges at Peterboro, the reservoir will be filled once in a 60-year frequency. In considering the likelihood of damage to the Sargent Camp during excessive spillway discharges, the difference in the probability of obtaining spillway discharge by the two methods of operation does not appear to be too serious. The following table indicates the anticipated pool elevations for the two methods of operation and for various frequencies:

Frequency	Reservoir Pool Elevations	
	Continuous Discharge	Gates Closed Temporarily
1	914	914
2	916	916
5	918	918
10	921	921
20	925	928
40	933	940
60	941	946
80	944	946.7
100	946	947.3

(Rev. June 1946)

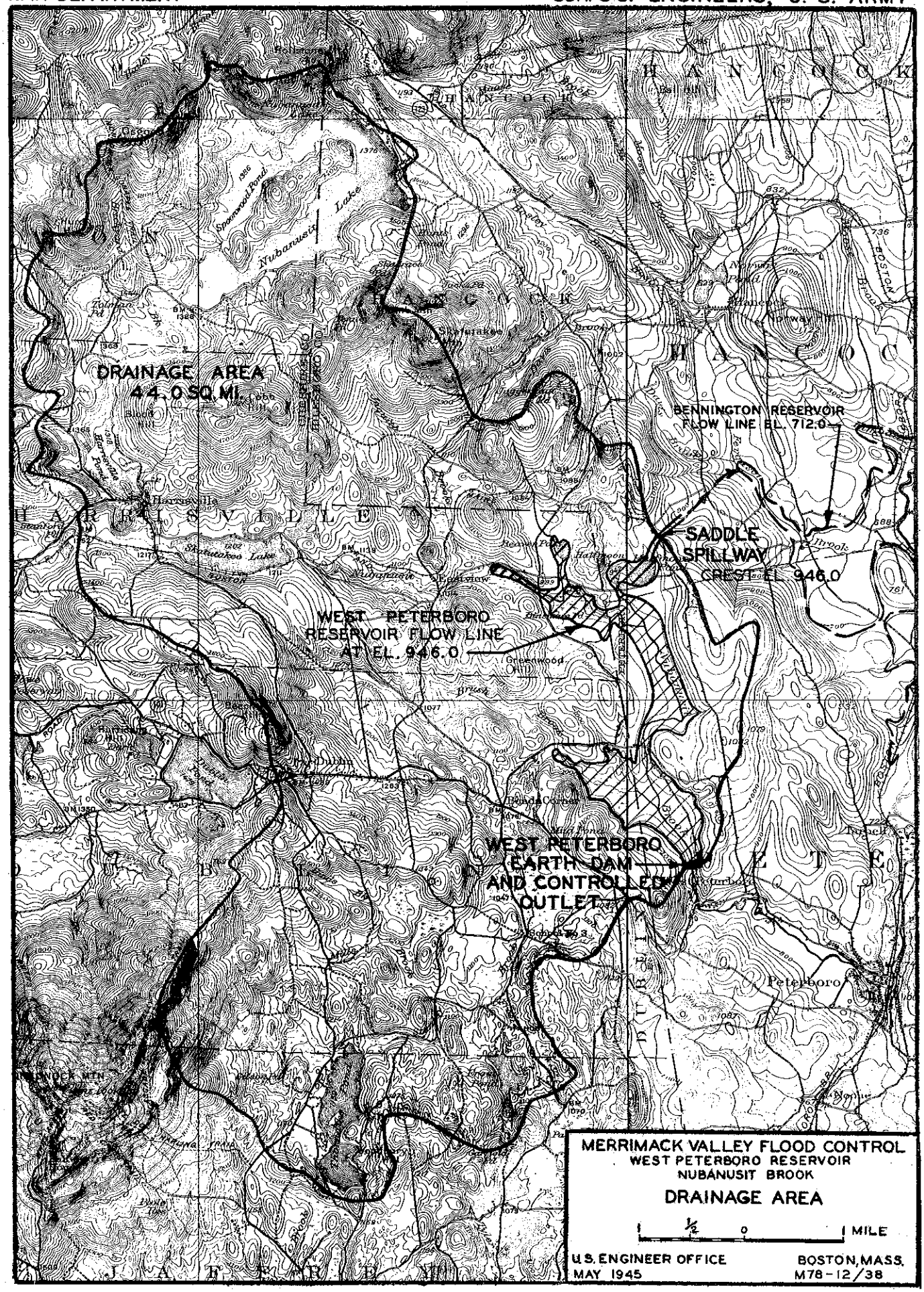
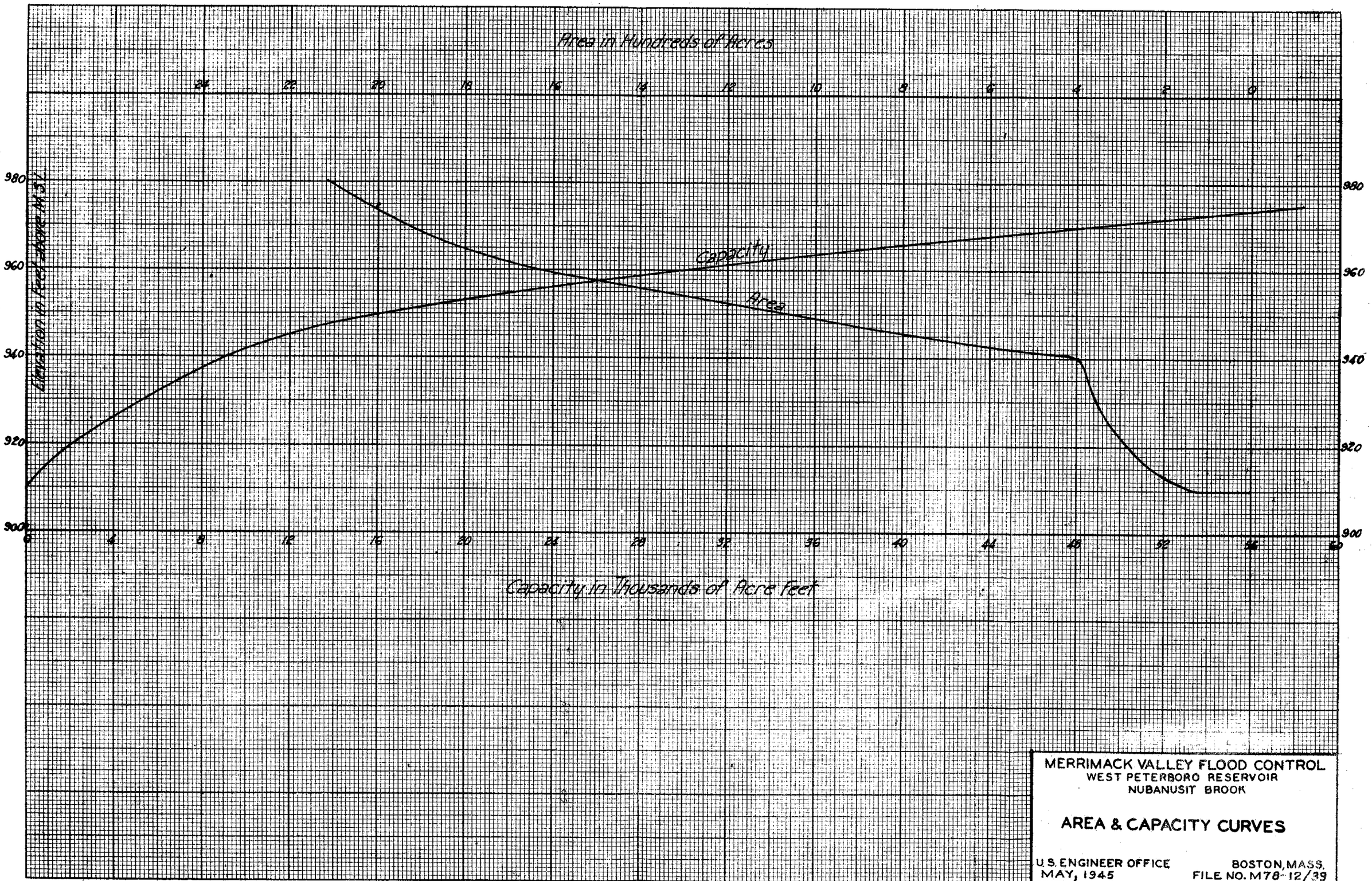

DRAINAGE AREA
44.0 SQ. MI.
BENNINGTON RESERVOIR
FLOW LINE EL. 712.0
SADDLE
SPILLWAY
CREST EL. 946.0
WEST PETERBORO
RESERVOIR FLOW LINE
AT EL. 946.0
WEST PETERBORO
EARTH DAM
AND CONTROLLED
OUTLET
Nubanusit Lake
Harrisville
Peterboro
MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
NUBANUSIT BROOK
DRAINAGE AREA
1 ½ 0 1 MILE
U.S. ENGINEER OFFICE
MAY 1945
BOSTON, MASS.
M78-12/38

PLATE I-1

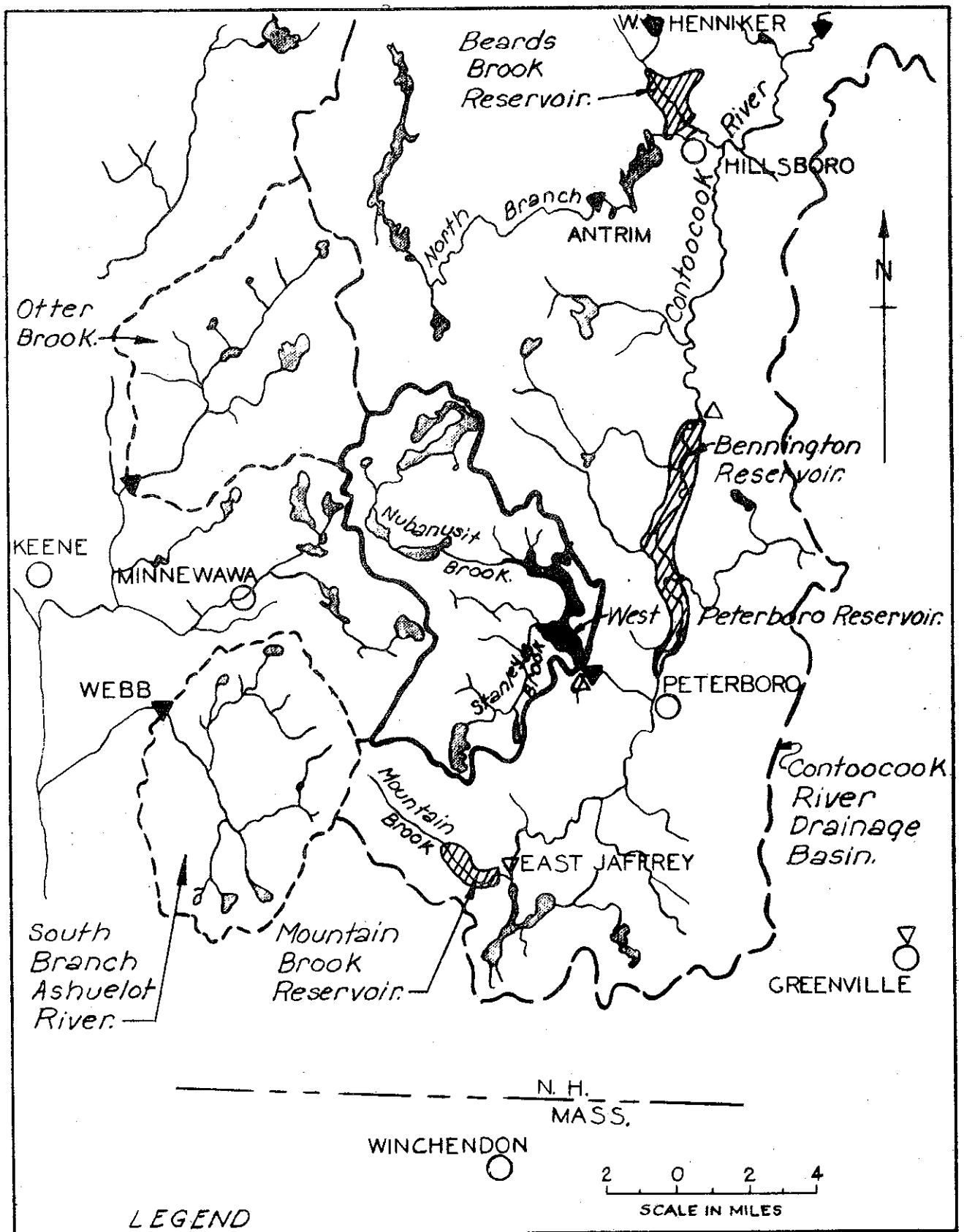


MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
NUBANUSIT BROOK

AREA & CAPACITY CURVES

U.S. ENGINEER OFFICE
MAY, 1945

BOSTON, MASS.
FILE NO. M78-12/39



LEGEND

○ Precipitation Stations.

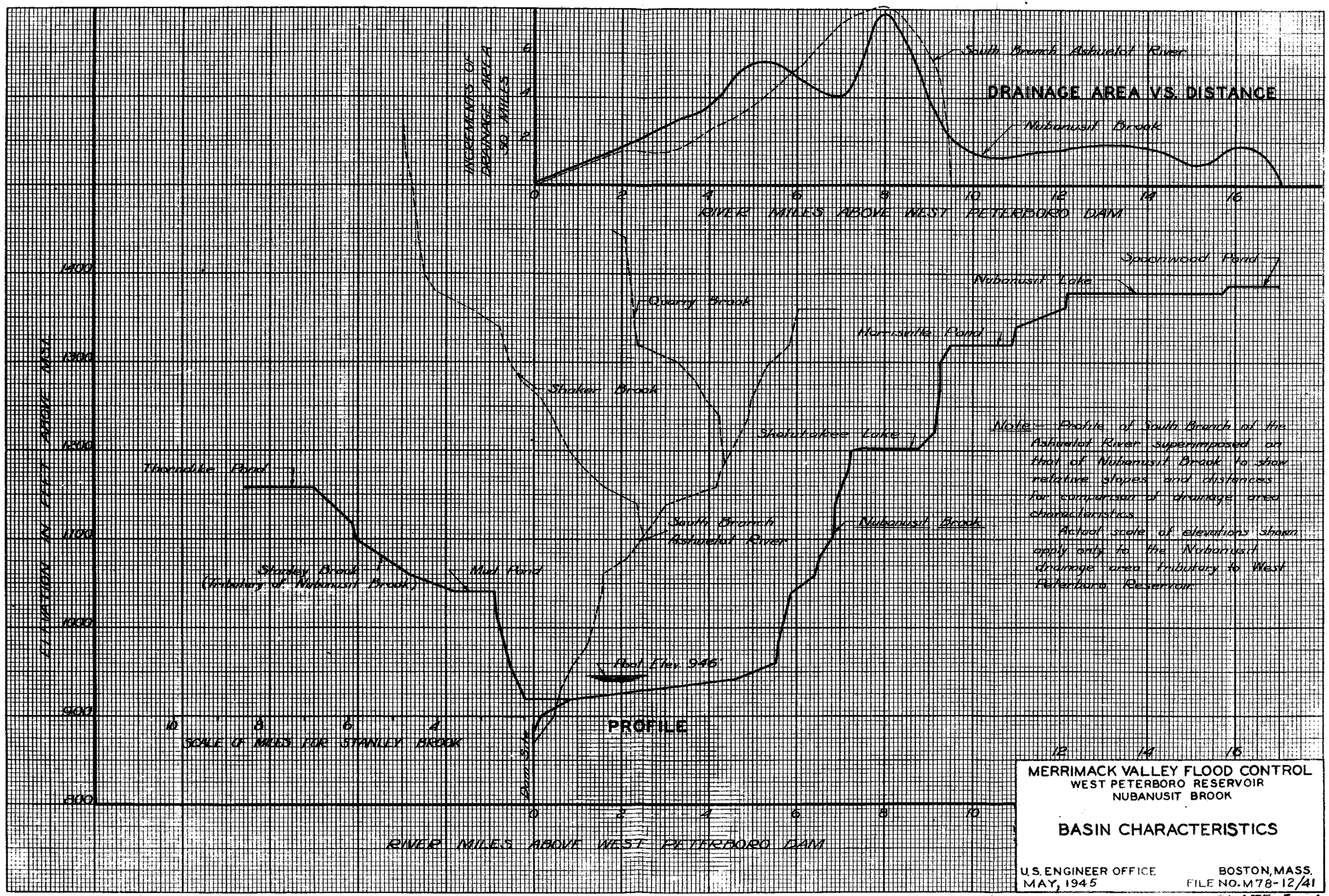
▼ Regular Stream Gaging Stations.

▽ Points of Peak Flow Measurements.

MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
NUBANUSIT BROOK
DRAINAGE AREAS FOR HYDROLOGIC STUDIES

U.S. ENGINEER OFFICE — BOSTON, MASS.
MAY, 1945 — FILE NO. M78-12/40

PLATE T-3



MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
NUBANUSIT BROOK

BASIN CHARACTERISTICS

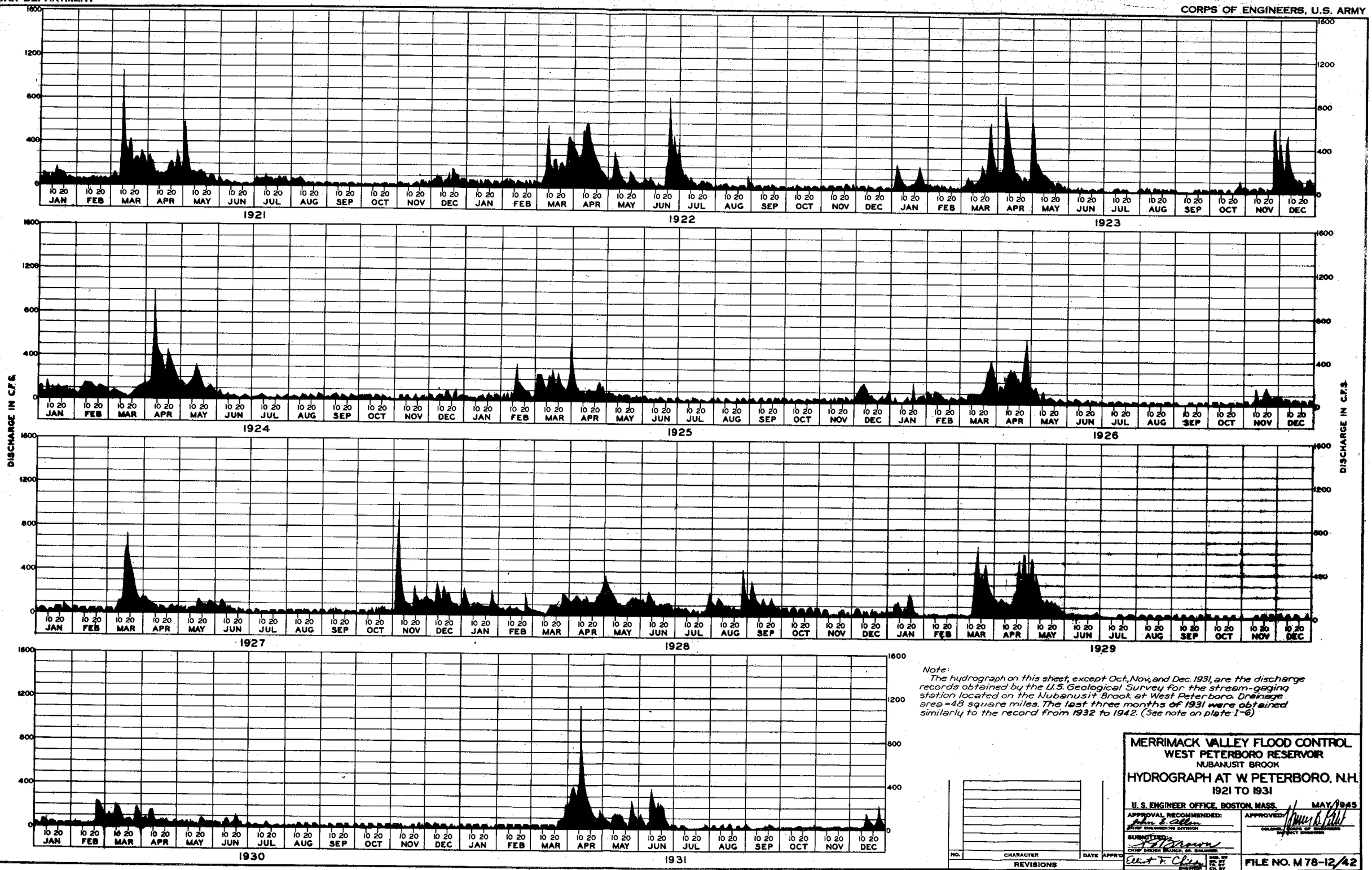
U.S. ENGINEER OFFICE
MAY, 1945

BOSTON, MASS.
FILE NO. M78-12/41

PLATE I-4

WAR DEPARTMENT

CORPS OF ENGINEERS, U.S. ARMY



Note:
The hydrograph on this sheet, except Oct., Nov., and Dec. 1931, are the discharge records obtained by the U.S. Geological Survey for the stream-gaging station located on the Nubanusit Brook at West Peterboro. Drainage area = 48 square miles. The last three months of 1931 were obtained similarly to the record from 1932 to 1942. (See note on plate I-6)

NO.	CHARACTER	DATE	APPR'D
	REVISIONS		

MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
NUBANUSIT BROOK

HYDROGRAPH AT W. PETERBORO, N.H.
1921 TO 1931

U. S. ENGINEER OFFICE, BOSTON, MASS. MAY, 1945

APPROVAL RECOMMENDED: John E. Allen, CHIEF ENGINEERING DIVISION

SUBMITTED: [signature], CHIEF DESIGN BRANCH, SR. ENGINEER

Elliot F. Childs, ENGINEER

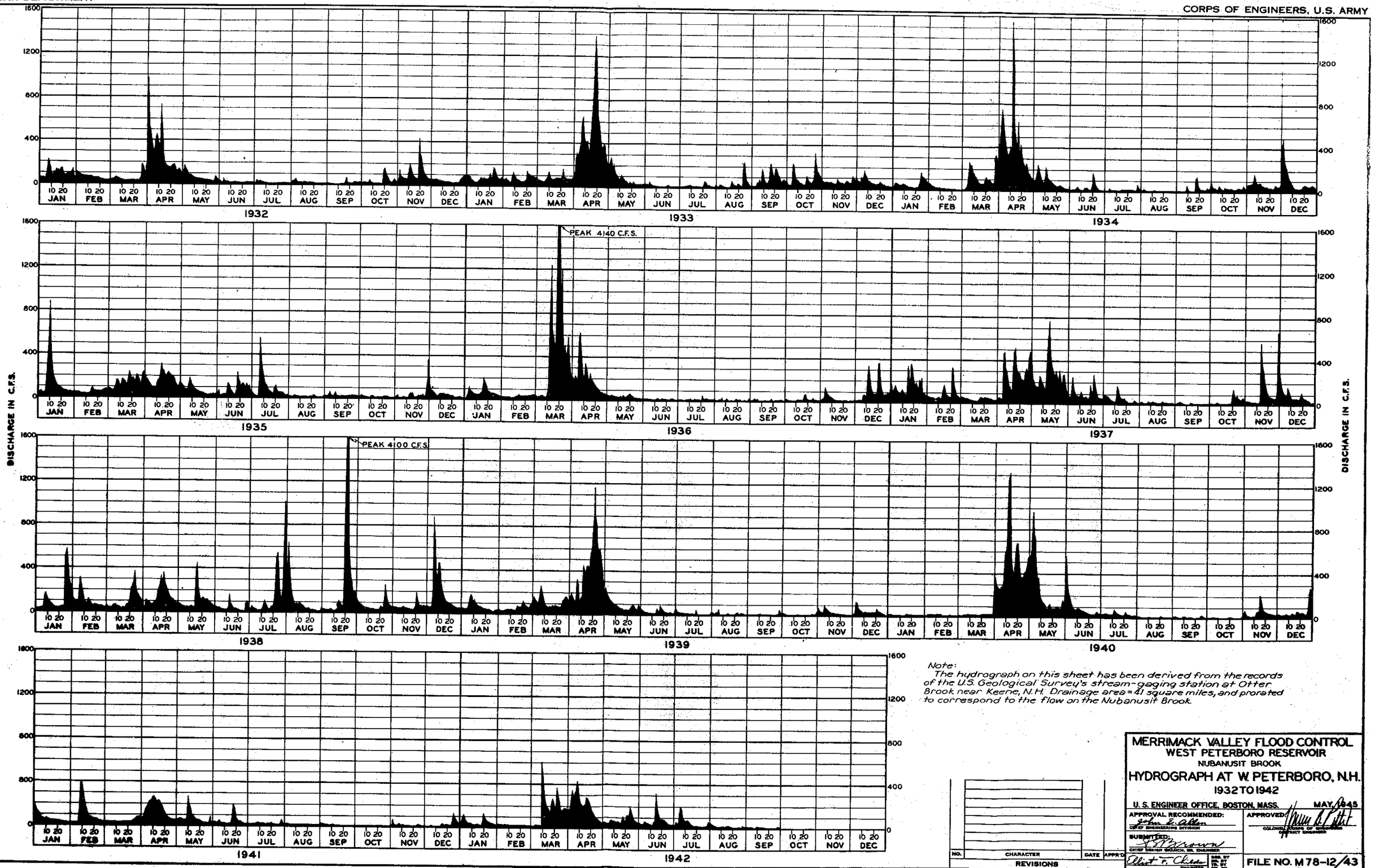
APPROVED: [signature], COLONEL, CORPS OF ENGINEERS, DISTRICT ENGINEER

FILE NO. M 78-12/42

PLATE I-5

WAR DEPARTMENT

CORPS OF ENGINEERS, U.S. ARMY



Note:
The hydrograph on this sheet has been derived from the records of the U.S. Geological Survey's stream-gaging station at Otter Brook near Keene, N.H. Drainage area = 41 square miles, and prorated to correspond to the flow on the Nubanusit Brook.

NO.	CHARACTER	DATE	APPR'D
	REVISIONS		

MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
NUBANUSIT BROOK

HYDROGRAPH AT W. PETERBORO, N.H.
1932 TO 1942

U. S. ENGINEER OFFICE, BOSTON, MASS. MAY, 1945

APPROVAL RECOMMENDED: John E. Allen, CHIEF ENGINEERING DIVISION

SUBMITTED: R. B. Brown, CHIEF DESIGN BRANCH, SR. ENGINEER

Elliot F. Childs, ENGINEER

APPROVED: [signature], COLONEL, CORPS OF ENGINEERS, DISTRICT ENGINEER

FILE NO. M78-12/43

PLATE I-6

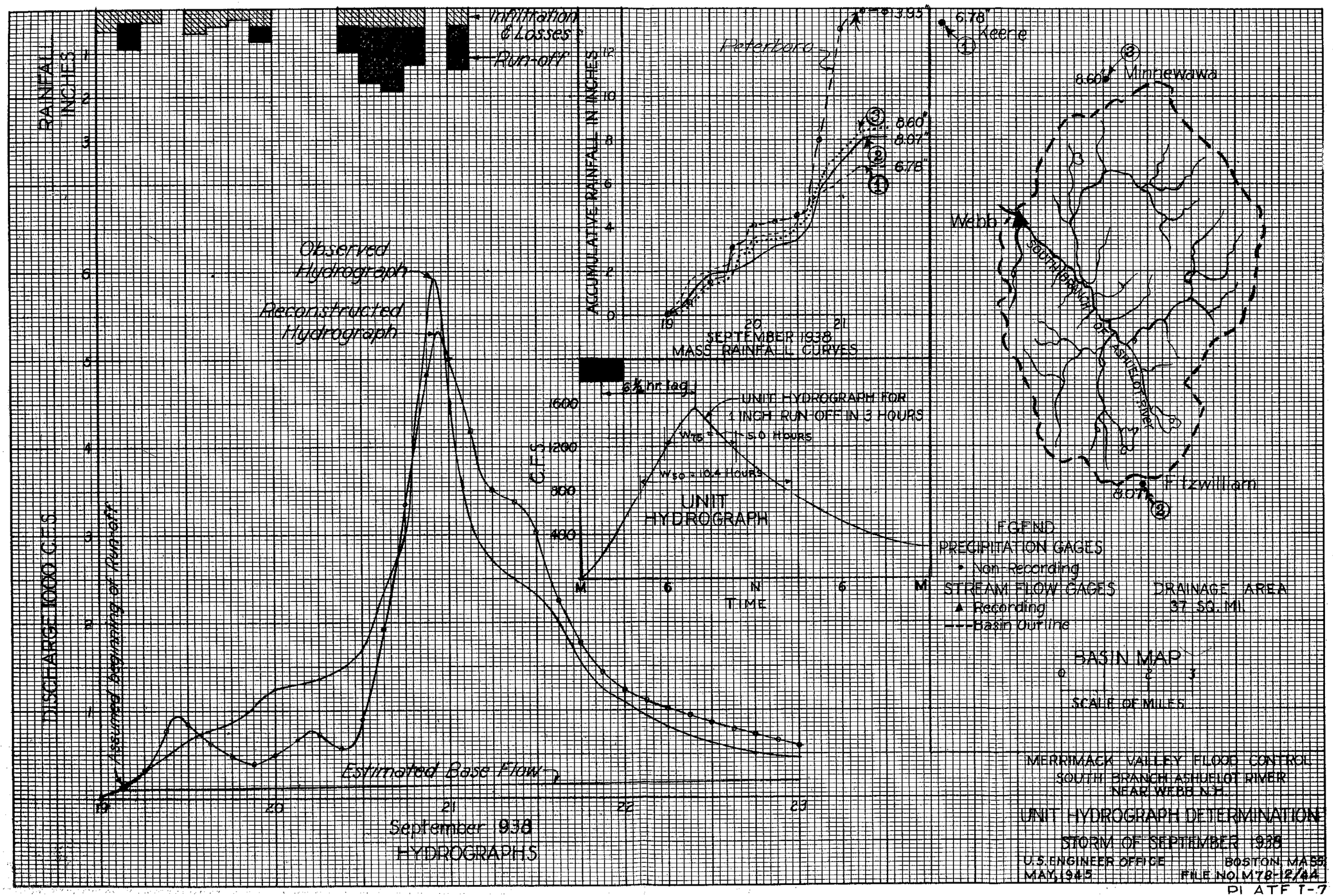

RAINFALL INCHES
Infiltration & Losses
Run-off
Observed Hydrograph
Reconstructed Hydrograph
DISCHARGE 1000 C.F.S.
Assumed beginning of Run-off
Estimated Base Flow
September 1938 HYDROGRAPHS
ACCUMULATIVE RAINFALL IN INCHES
Peterboro
3.95"
6.78"
Keene
8.60"
8.07"
6.78"
Minnewawa
Webb
South Branch
Ashuelot River
Fitzwilliam
SEPTEMBER 1938 MASS RAINFALL CURVES
6½ hr lag
UNIT HYDROGRAPH FOR 1 INCH RUN OFF IN 3 HOURS
W75 = 5.0 Hours
W50 = 10.4 Hours
UNIT HYDROGRAPH
TIME
LEGEND
PRECIPITATION GAGES
• Non-Recording
STREAM FLOW GAGES
▲ Recording
---Basin Outline
DRAINAGE AREA 37 SQ. MI.
BASIN MAP
SCALE OF MILES
MERRIMACK VALLEY FLOOD CONTROL
SOUTH BRANCH ASHUELOT RIVER NEAR WEBB N.H.
UNIT HYDROGRAPH DETERMINATIONS
STORM OF SEPTEMBER 1938
U.S. ENGINEER OFFICE BOSTON, MASS.
MAY, 1945 FILE NO. M78-12/44

PLATE I-7

WAR DEPARTMENT CORPS OF ENGINEERS, U. S. ARMY

Computed by _____
Date _____

UNIT HYDROGRAPH DETERMINATION

STREAM SOUTH BRANCH ASHUELOT LOCATION WEBB, N.H.

DRAINAGE AREA 37 SQ. MI.

STORM OF SEPTEMBER 1938 PREPARED BY BOSTON DIST. N.E. DIV.

AV. RAINFALL 11.50 IN.; RAINFALL-EXCESS 6.00 IN.; F_{av}, 0.61 IN./HR.

L 7.2 mi; L_{co} 3.9 mi; $(LL_{co})^{0.3}$ 2.72; t_R 3 hrs;

LAG(t_{pR}) 6.5 hrs; C_{tR} 2.39; q_{pR} 41.9 c.f.s./sq.mi.; C_p 640 272

LAG(t_{cmR}) 16.5 hrs; W_{50} 10.4 hrs; W_{75} 5.0 hrs; SLOPE _____

TIME 1938 SEPT.	OBSERVED DISCHARGE C.F.S.	ESTIMATED BASE FLOW C.F.S.	STORM RUNOFF C.F.S.	OBSERVED 3 HOUR UNIT HYDROGRAPH C.F.S.	ADJUSTED HOUR UNIT HYDROGRAPH C.F.S.	REPRODUCED STORM HYDROGRAPH C.F.S.
19-3 P	116	100	16	0		100
6	307	104	203	530		422
9	609	108	501	1200		828
12	688	112	576	1395		949
3	718	116	602	1025		731
6	888	120	768	740		560
9	953	124	829	530		442
20-N	1270	128	1142	384		544
3	1300	132	1168	285		723
6	1350	136	1214	217		754
9	1450	140	1310	168		600
12	1670	144	1526	138		804
3	2370	148	2222	108		1779
6	3010	152	2858	85		3443
9	5300	156	5144	67		4846
21-N	4800	160	4640	54		5032
3	3120	164	2956	40		4162
6	2670	168	2502	29		3070
9	2630	172	2458	20		3351
12	2330	176	2154	16		2966
3	1910	180	1730	11		2244
6	1530	184	1346	5		1722
9	1210	188	1022	1		1346
22-N	1030	192	838	0		1077
3	897	196	701			880
6	800	200	600			732
9	718	204	514			620
12	637	208	429			536
3	562	212	350			461
6	512	216	296			404
9	476	220	256			361
23-N	443	224	219			330
3	406	228	178			301
6	382	232	150			275
9	359	236	123			259
12	341	240	101			258
3	313	244	69			257
6	291	248	43			256
9	283	252	31			256
24-N	256	256	0			256

FILE NO. M78-12/45

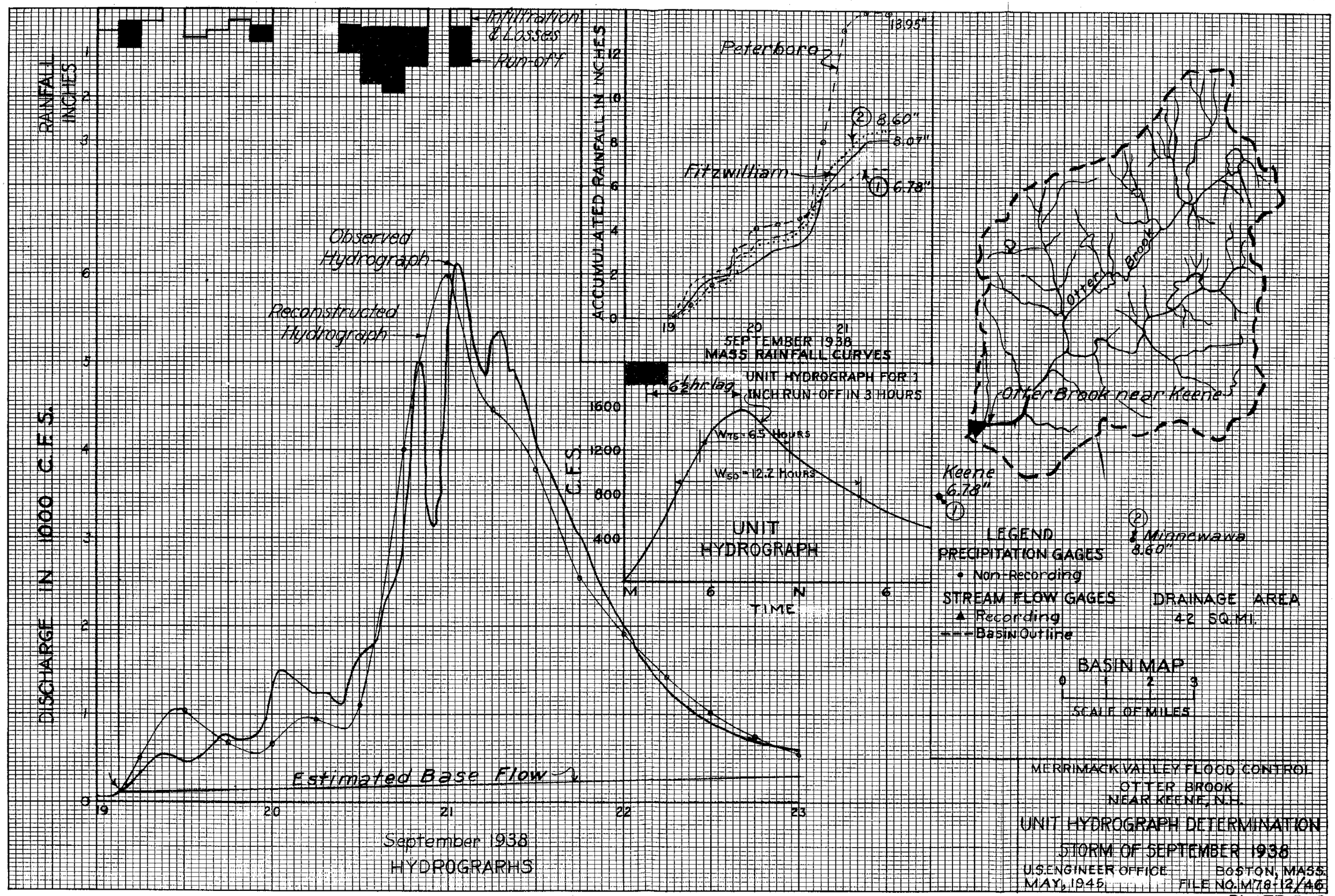

RAINFALL INCHES
Infiltration & Losses
Run-off
DISCHARGE IN 1000 CFS
Observed Hydrograph
Reconstructed Hydrograph
Estimated Base Flow
September 1938
HYDROGRARHS
ACCUMULATED RAINFALL IN INCHES
Peterboro
13.95"
8.60"
8.07"
Fitzwilliam
6.78"
SEPTEMBER 1938
MASS RAINFALL CURVES
6½hr lag
UNIT HYDROGRAPH FOR 1 INCH RUN-OFF IN 3 HOURS
W75 = 6.5 Hours
W50 = 12.2 Hours
CFS
UNIT HYDROGRAPH
TIME
Otter Brook
Otter Brook near Keene
Keene
6.78"
Minnewawa
8.60"
LEGEND
PRECIPITATION GAGES
• Non-Recording
STREAM FLOW GAGES
▲ Recording
--- Basin Outline
DRAINAGE AREA
42 SQ. MI.
BASIN MAP
SCALE OF MILES
MERRIMACK VALLEY FLOOD CONTROL
OTTER BROOK
NEAR KEENE, N.H.
UNIT HYDROGRAPH DETERMINATION
STORM OF SEPTEMBER 1938
U.S. ENGINEER OFFICE
BOSTON, MASS.
MAY, 1945
FILE NO. M78-12/46

PLATE I-9

WAR DEPARTMENT CORPS OF ENGINEERS, U. S. ARMY

Computed by _____
Date _____

UNIT HYDROGRAPH DETERMINATION

STREAM OTTER BROOK LOCATION KEENE, N.H.

DRAINAGE AREA 42 SQ. MI.

STORM OF SEPTEMBER, 1938 PREPARED BY BOSTON DIST. N.E. DIV.

AV. RAINFALL 11.50 IN.; RAINFALL-EXCESS 6.04 IN.; F_{av}, 0.61 IN./HR.

L 10.0 mi; L_{co} 6.3 mi; $(LL_{co})^{0.3}$ 3.46 ; t_R 3 hrs;

LAG(t_{pR}) 6.5 hrs; C_{tR} 1.88 ; q_{pR} 37.1 c.f.s./sq.mi.; C_p 640 241

LAG(t_{cmR}) 14.5 hrs; W_{50} 12.2 hrs; W_{75} 6.5 hrs; SLOPE _____

TIME 1938 SEPT.	OBSERVED DISCHARGE C.F.S.	ESTIMATED BASE FLOW C.F.S.	STORM RUNOFF C.F.S.	OBSERVED HOUR UNIT HYDROGRAPH C.F.S.	ADJUSTED HOUR UNIT HYDROGRAPH C.F.S.	REPRODUCED STORM HYDROGRAPH C.F.S.
19-3P	100	100	0	0		100
6	374	108	266	650		498
9	524	116	408	1390		950
12	485	124	361	1500		1024
3	562	132	430	958		707
6	763	140	623	875		665
9	737	148	589	758		603
20-N	1330	156	1174	660		780
3	1390	164	1226	510		951
6	1240	172	1068	390		931
9	1140	180	960	300		695
12	1560	188	1372	225		1019
3	2060	196	1864	180		2216
6	2820	204	2616	140		4100
9	3810	212	3598	112		5477
21-N	4820	220	4600	85		5517
3	5310	228	5082	65		4585
6	5170	236	4934	50		4357
9	4900	244	4656	40		4556
12	4030	252	3778	30		4154
3	3340	260	3080	20		3142
6	3090	268	2822	16		2605
9	2500	276	2224	10		2143
22-N	2030	284	1746	0		1788
3	1620	292	1328			1459
6	1290	300	990			1209
9	1060	308	752			1011
12	950	316	634			854
3	810	324	486			743
6	737	332	405			654
9	659	340	319			592
23-N	607	348	259			538
3	573	356	217			496
6	546	364	182			462
9	512	372	140			437
12	491	380	111			416
3	465	388	77			412
6	442	396	46			410
9	420	404	16			409
24-N						

FILE NO. M78-12/47

PLATE T-10

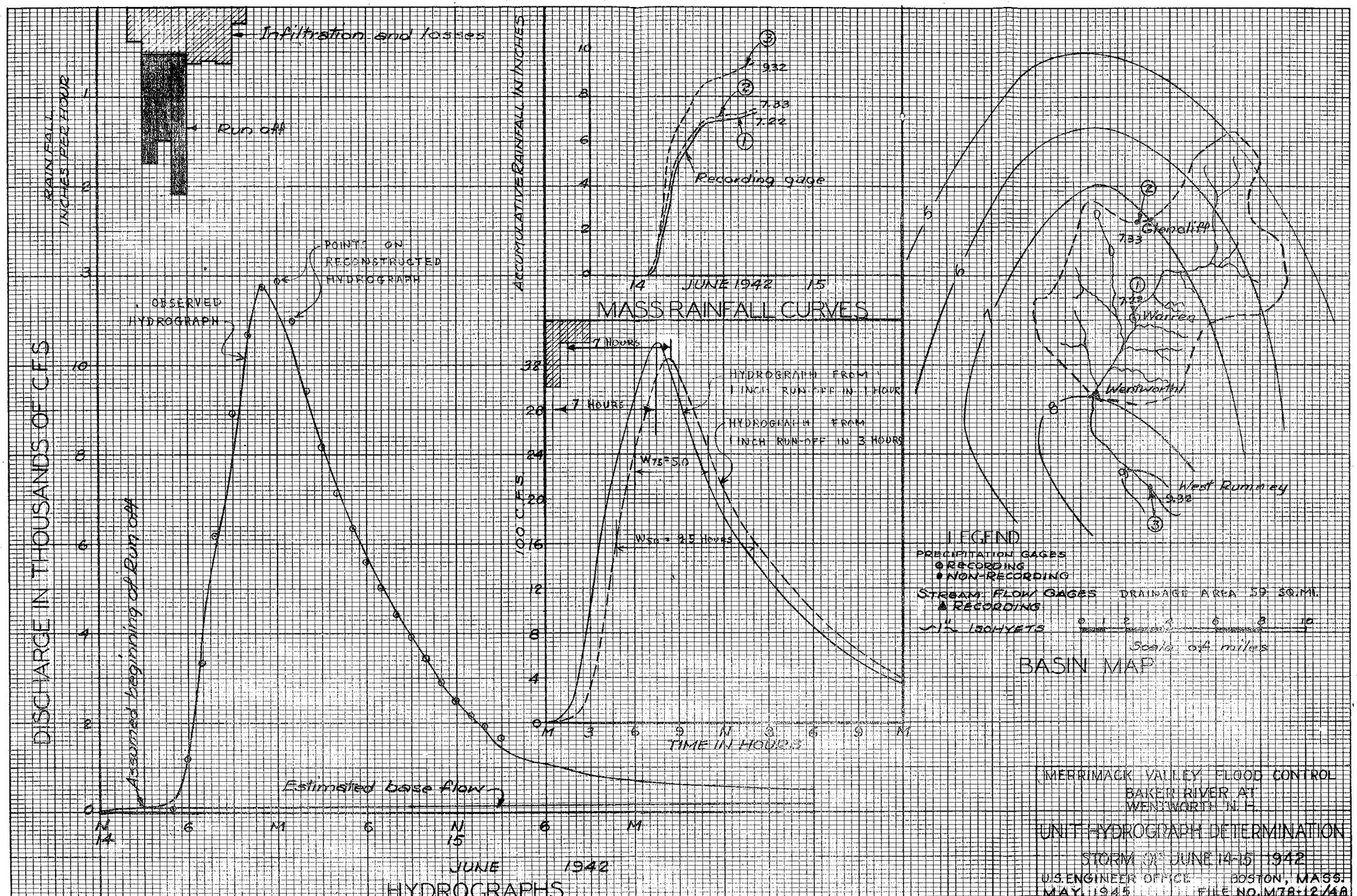

Infiltration and losses
Run off
RAINFALL INCHES PER HOUR
POINTS ON RECONSTRUCTED HYDROGRAPH
OBSERVED HYDROGRAPH
DISCHARGE IN THOUSANDS OF CFS
Assumed beginning of Run off
Estimated base flow
JUNE 1942
HYDROGRAPHS
ACCUMULATIVE RAINFALL IN INCHES
9.32
7.33
7.22
Recording gage
JUNE 1942
MASS RAINFALL CURVES
7 HOURS
7 HOURS
HYDROGRAPH FROM 1 INCH RUN-OFF IN 1 HOUR
HYDROGRAPH FROM 1 INCH RUN-OFF IN 3 HOURS
W75 = 5.0
W50 = 2.5 HOURS
CFS
TIME IN HOURS
Glendale
Warren
Wentworth
West Rumney
LEGEND
PRECIPITATION GAGES
RECORDING
NON-RECORDING
STREAM FLOW GAGES
RECORDING
DRAINAGE AREA 59 SQ.MI.
ISOHYETS
Scale of miles
BASIN MAP
MERRIMACK VALLEY FLOOD CONTROL
BAKER RIVER AT WENTWORTH N.H.
UNIT HYDROGRAPH DETERMINATION
STORM OF JUNE 14-15 1942
U.S. ENGINEER OFFICE BOSTON, MASS.
MAY, 1945
FILE NO. M78-12/48

PLATE I-11

WAR DEPARTMENT CORPS OF ENGINEERS, U. S. ARMY

Computed by _____
Date _____

UNIT HYDROGRAPH DETERMINATION

STREAM BAKER RIVER LOCATION WENTWORTH, N.H.

DRAINAGE AREA 59 SQ. MI.

STORM OF JUNE 1942 PREPARED BY BOSTON DIST. N.E. DIV.

AV. RAINFALL 5.3 IN.; RAINFALL-EXCESS 3.74 IN.; F_{av}, 0.52 IN./HR.

L 12.4 mi; L_{co} 6.6 mi; $(LL_{co})^{0.3}$ 3.75; t_R 3 hrs;

LAG(t_{pR}) 7 hrs; C_{tR} 1.87; q_{pR} 55.1 c.f.s./sq.mi.; C_p 640 386

LAG(t_{cmR}) 13 hrs; W_{50} 9.5 hrs; W_{75} 5.0 hrs; SLOPE _____

TIME 1942 JUNE	OBSERVED DISCHARGE C.F.S.	ESTIMATED BASE FLOW C.F.S.	STORM RUNOFF C.F.S.	OBSERVED 1 HOUR UNIT HYDROGRAPH C.F.S.	ADJUSTED 3 HOUR UNIT HYDROGRAPH C.F.S.	REPRODUCED STORM HYDROGRAPH C.F.S.
14-3A	40	40	0	0	0	40
4	50	40	10	20	7	64
5	100	45	55	100	40	186
6	1150	50	1100	860	326	1228
7	4200	55	4145	1900	953	3364
8	6000	60	5940	2450	1737	6220
9	8000	65	7935	2900	2417	8925
10	10300	70	10230	3300	2663	10700
11	11800	75	11725	3300	3167	11795
M	11200	80	11120	2870	3157	11900
1	10500	85	10415	2460	2877	10995
2	9200	90	9110	2100	2477	9490
3	8400	95	8315	1850	2137	8205
4	7500	100	7400	1660	1870	7185
5	6500	105	6395	1480	1663	6395
6	5800	110	5690	1310	1483	5715
7	5200	115	5085	1150	1313	5075
8	4500	120	4380	1000	1153	4485
9	4000	125	3875	860	1003	3925
10	3500	130	3370	740	867	3417
11	3000	135	2875	630	743	2951
15-N	2500	140	2360	540	633	2560
1	2200	145	2035	460	543	2206
2	1830	150	1680	400	467	1921
3	1470	155	1315	350	403	1682
4	1220	160	1060	310		1498
5	1130	165	965	280		1351
6	1050	170	880	250		1228
7	985	175	810	225		1127
8	914	180	734	200		1030
9	823	185	638	175		941
10	783	190	593	160		865
11	738	195	543	150		804
M	730	200	530	135		759
1	700	205	495	120		715
2	672	210	462	110		669
3	660	215	445	100		630
4	651	220	431	90		598
5	620	225	395	80		595
6	598	230	368	70		533
7	578	235	343	60		500
8	559	240	319	50		468

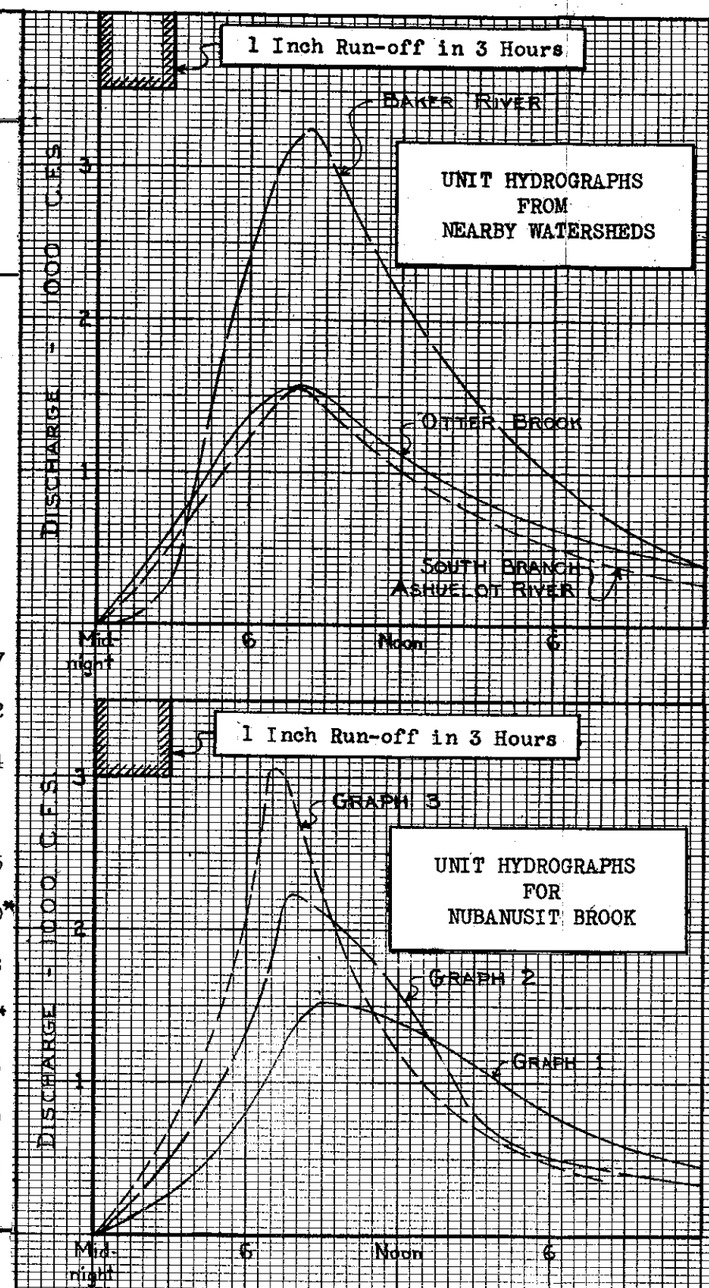
FILE NO. M78-12/49

ANALYSIS OF UNIT HYDROGRAPH

ITEM	SYMBOL	EXPLANATION	UNIT	BAKER RIVER WENTWORTH N.H.	OTTER BROOK KEENE N.H.	SOUTH BRANCH ASHUELOT WEBB N.H.	NUBANUSIT BROOK WEST PETERBORO N.H. GRAPH #1-	GRAPH -2-	GRAPH -3-
(1)	D.A.	Drainage Area	Sq.Mi.	59	42	37	44	44	44
(2)		Storm of		June 1942	Sept. 1938	Sept. 1938	COMPUTED		
(3)		Total Weighted Rainfall over Basin	In.	5.30	11.50	11.50			
(4)		Rainfall Excess	In.	3.74	6.04	6.00			
(5)	F_{av}	Infiltration	In/Hr	0.52	0.61	0.61			
(6)		Peak Discharge	c.f.s.	11,800	6,100	5,900			
(7)		Peak Discharge per Sq. Mi.	c.s.m.	200	145	160			
(8)	L	Length of Drainage Basin	Miles	12.4	10.0	7.2	15.7	15.7	15.7
(9)	L_{co}	Length from C.G. to Point of Discharge	Miles	6.6	6.3	3.9	6.2	6.2	6.2
(10)	$(LL_{co})^{0.3}$			3.75	3.46	2.72	3.94	3.94	3.94
(11)	t_R	Unit Rainfall Duration	Hours	3	3	3	3	3	3
(12)	$Lag(t_{pR})$	Time of Peak Concentration	Hours	7	6.5	6.5	7.4	6.3	5.5
(13)	C_{tR}	Drainage Basin Coefficient		1.87	1.88	2.39	1.87*	1.60*	1.40*
(14)	q_{pR}	Peak Discharge of Unit Hydrograph	cfs/sq mi	55.1	37.1	41.9	34.6	50.8	69.8
(15)	C_p640	Drainage Basin Coefficient		386	241	272	256 *	320 *	386 *
(16)	$Lag(t_{cmR})$	Time of Volume Concentration	Hours	13	14.5	16.5	20	15	12.5
(17)	w_{50}	Width at 50% of Peak Discharge	Hours	9.5	12.2	10.4	12.5	8.6	6.0
(18)	w_{75}	Width at 75% of Peak Discharge	Hours	5.0	6.5	5.0	7.7	4.9	2.8

*Assumed Values

For detailed explanation of use of above symbols refer to "Synthetic Unit Graphs" by Franklin F. Snyder, Transactions American Geophysical Union, 1938.



NUBANUSIT BROOK

UNIT HYDROGRAPH VALUES

TIME Hour	GRAPH 1 c.f.s	GRAPH 2 c.f.s	GRAPH 3 c.f.s
Midnight	-	-	-
1	80	140	170
2	160	300	400
3	260	500	680
4	400	740	1000
5	580	1020	1450
6	800	1300	2050
7	1080	1900	3070
8	1400	2240	2750
9	1520	2140	2250
10	1500	2000	1800
11	1460	1800	1480
Noon	1400	1600	1220
1	1330	1380	1000
2	1240	1100	850
3	1140	800	720
4	1020	650	620
5	900	580	540
6	810	530	480
7	720	490	420
8	660	450	370
9	600	420	330
10	540	390	300
11	500	360	270
Midnight	460	340	240

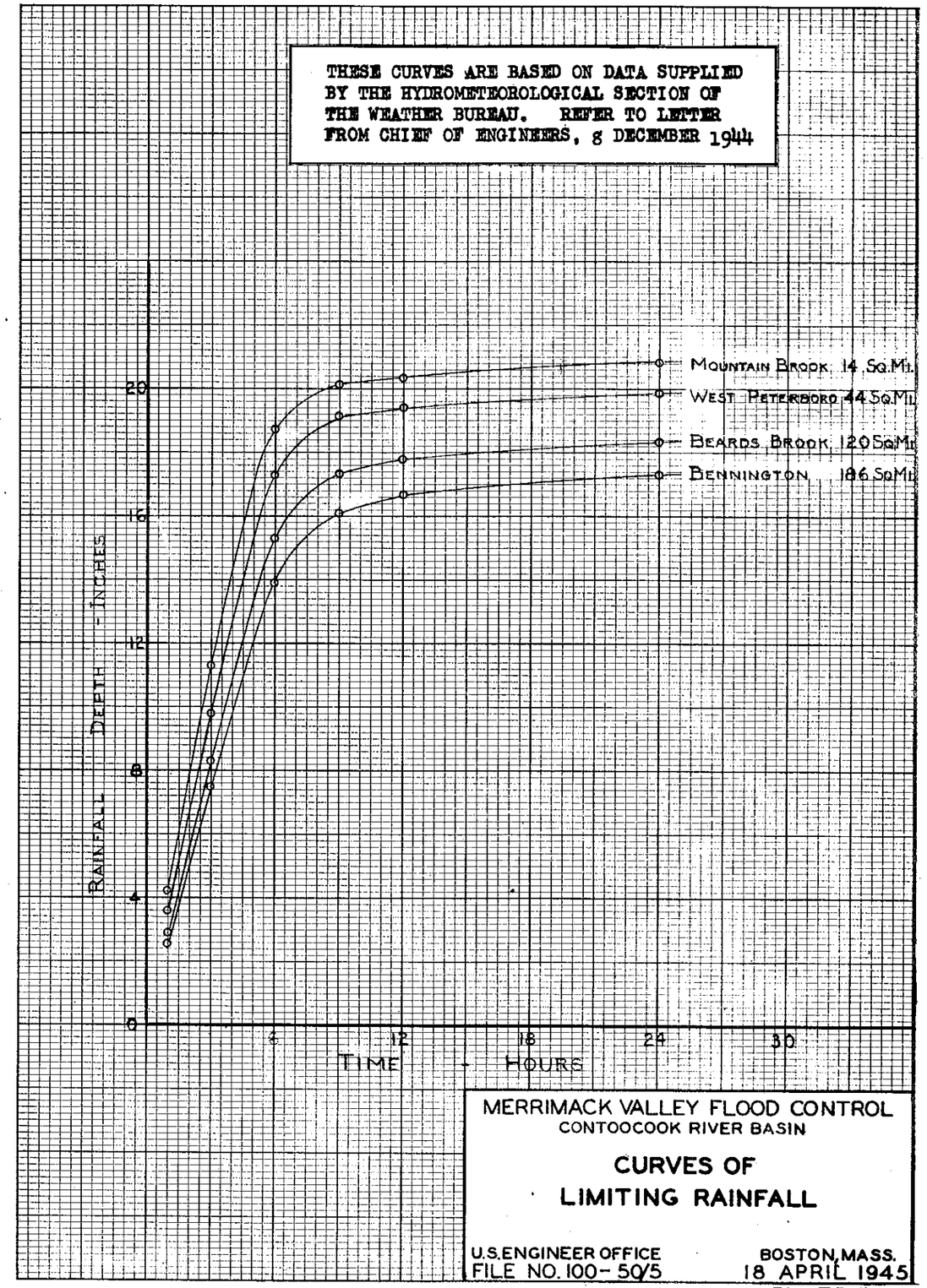
MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
NUBANUSIT BROOK

UNIT HYDROGRAPH DATA

U.S. ENGINEER OFFICE
MAY, 1945

BOSTON, MASS
FILE NO. M78-12/50

PLATE T-12



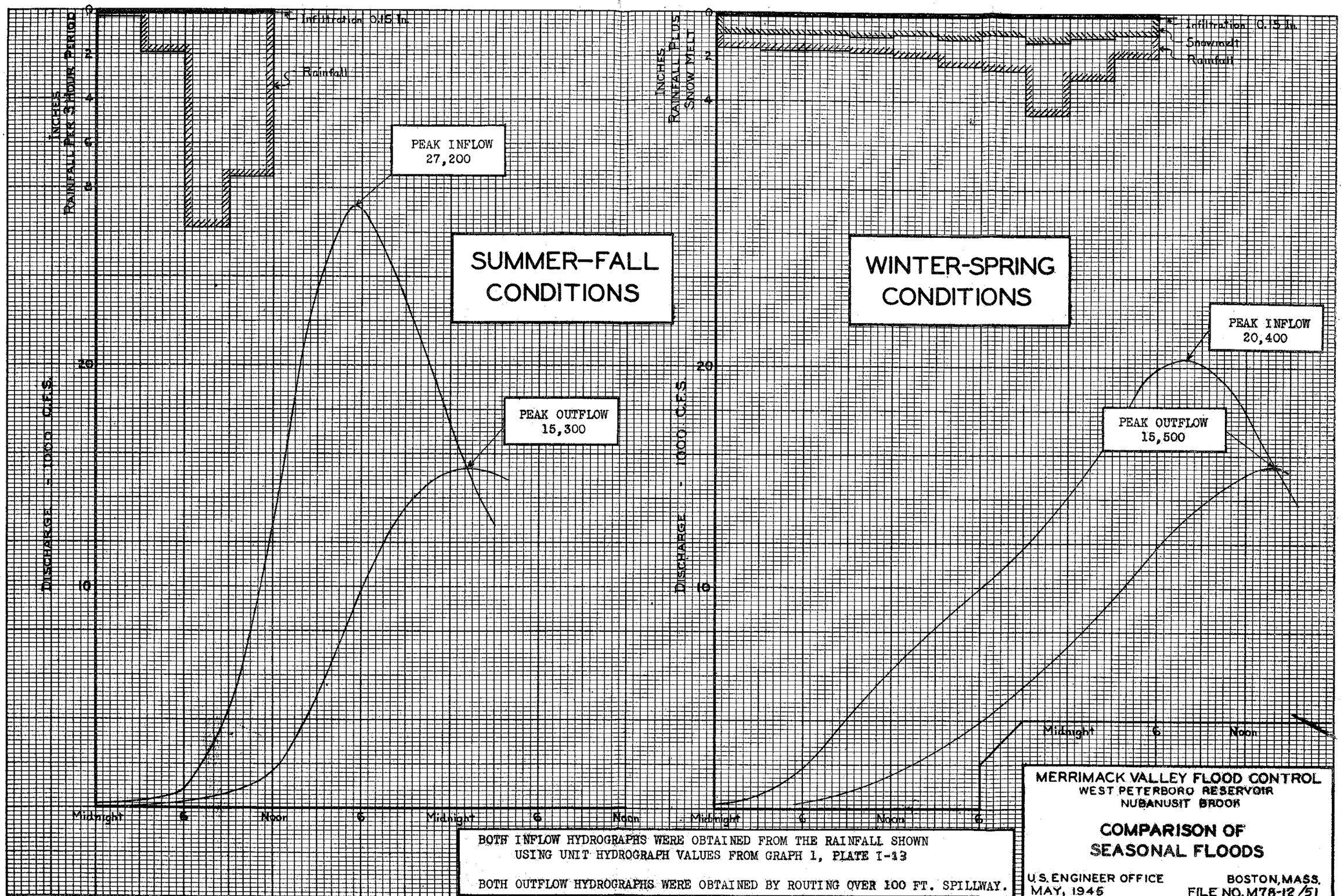
MERRIMACK VALLEY FLOOD CONTROL
CONTOOCOOK RIVER BASIN

CURVES OF LIMITING RAINFALL

U.S. ENGINEER OFFICE
FILE NO. 100-50/5

BOSTON, MASS.
18 APRIL 1945

PLATE I-14



BOTH INFLOW HYDROGRAPHS WERE OBTAINED FROM THE RAINFALL SHOWN USING UNIT HYDROGRAPH VALUES FROM GRAPH 1, PLATE I-13

BOTH OUTFLOW HYDROGRAPHS WERE OBTAINED BY ROUTING OVER 100 FT. SPILLWAY.

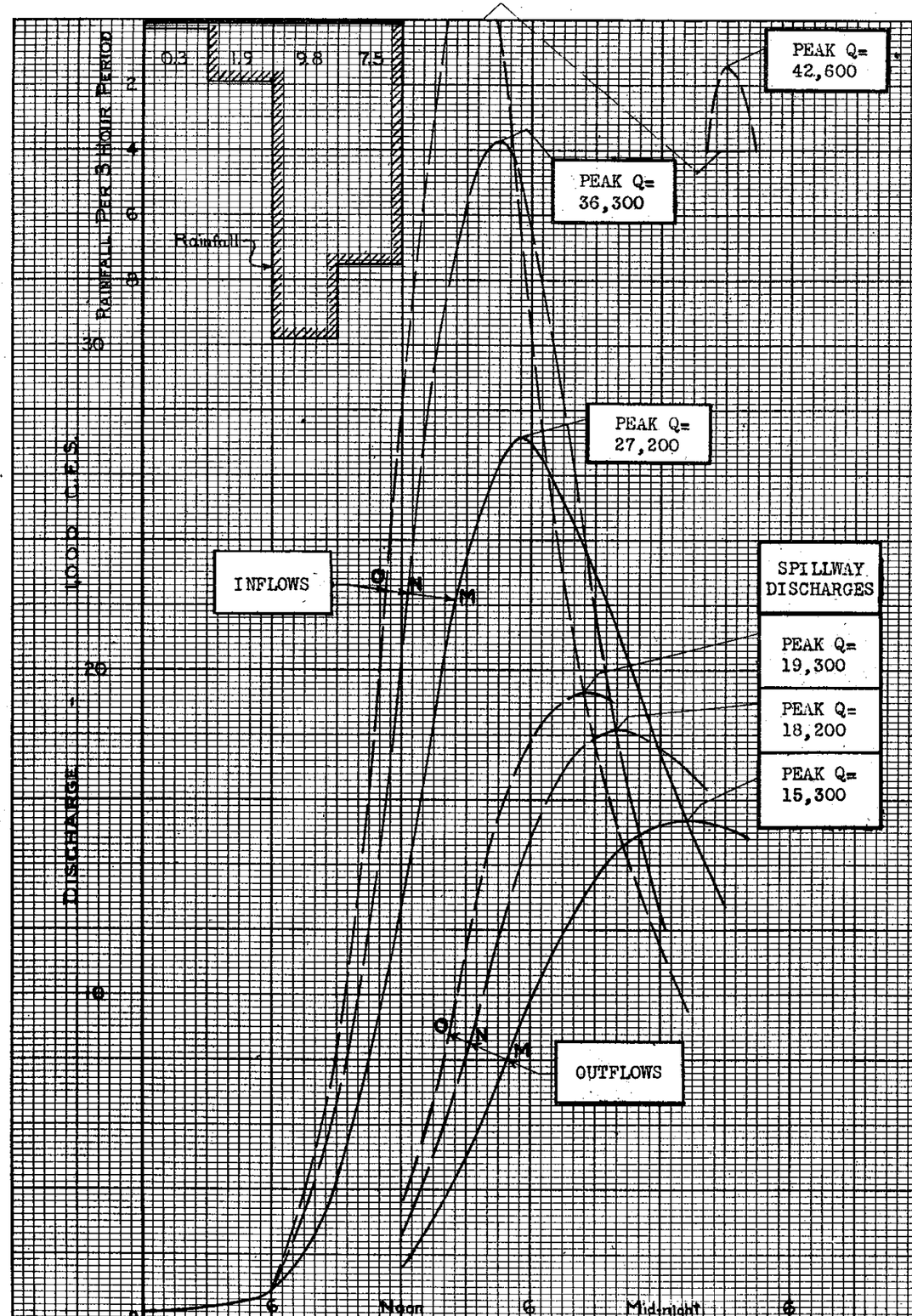
MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
NUBANUSIT BROOK

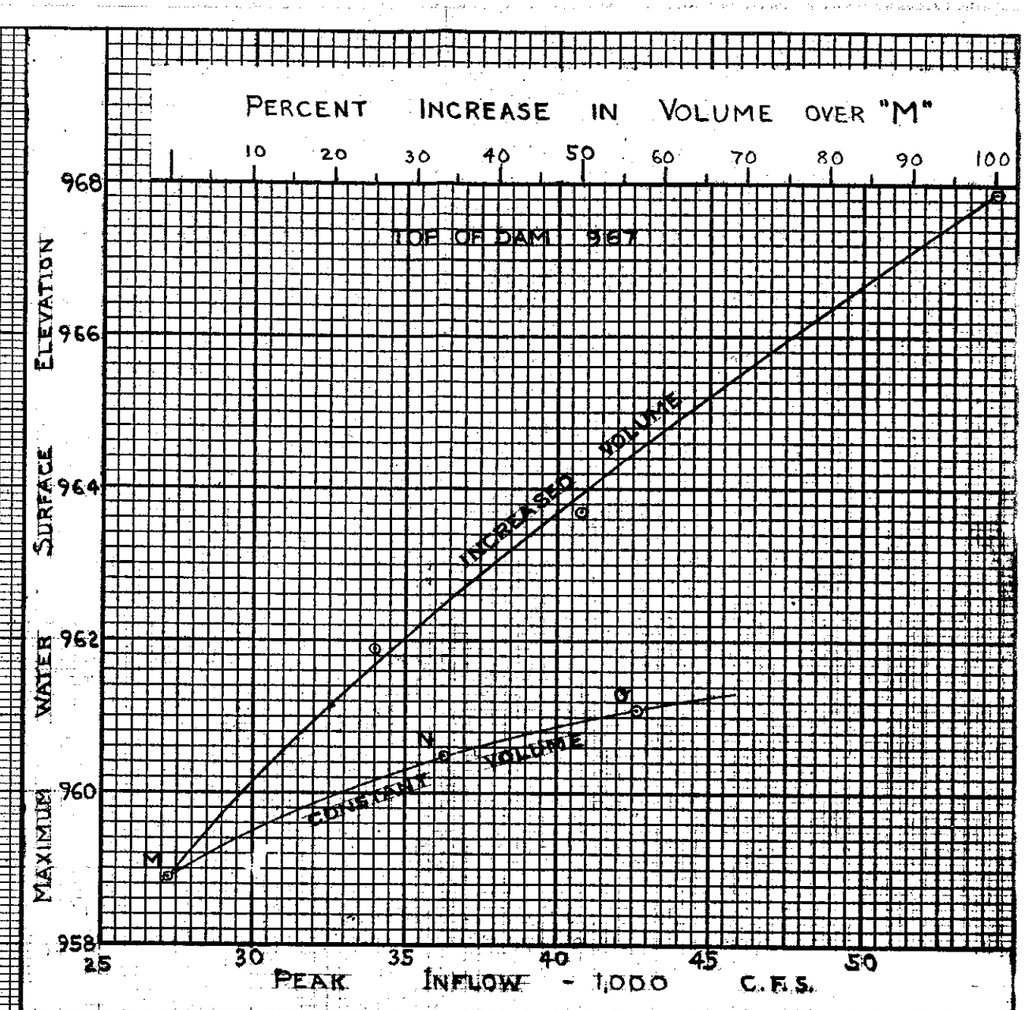
COMPARISON OF SEASONAL FLOODS

U.S. ENGINEER OFFICE
MAY, 1945

BOSTON, MASS.
FILE NO. M78-12/51

PLATE I-15





SPILLWAY FLOODS

FLOOD	VOLUME In.	PEAK INFLOW c.f.s	PEAK OUTFLOW c.f.s.	MAX. W.S. ELEV.
M	18.9	27,200	15,300	958.9
N	18.9	36,300	18,200	960.5
O	18.9	42,600	19,300	961.1
M+25%	23.6	34,000	20,600	961.9
M+50%	28.4	40,800	23,800	963.7
M+100%	37.8	54,400	31,700	967.9

HYDROGRAPH CONSTRUCTION

INFLOW HYDROGRAPH	RAINFALL VALUES 0.3	1.9	9.8	7.5
M	Graph 1		Graph 1	
N	Graph 1		Graph 2	
O	Graph 1		Graph 3	

Refer to Plate I-13 for Unit Hydrograph values for Graphs 1, 2, and 3

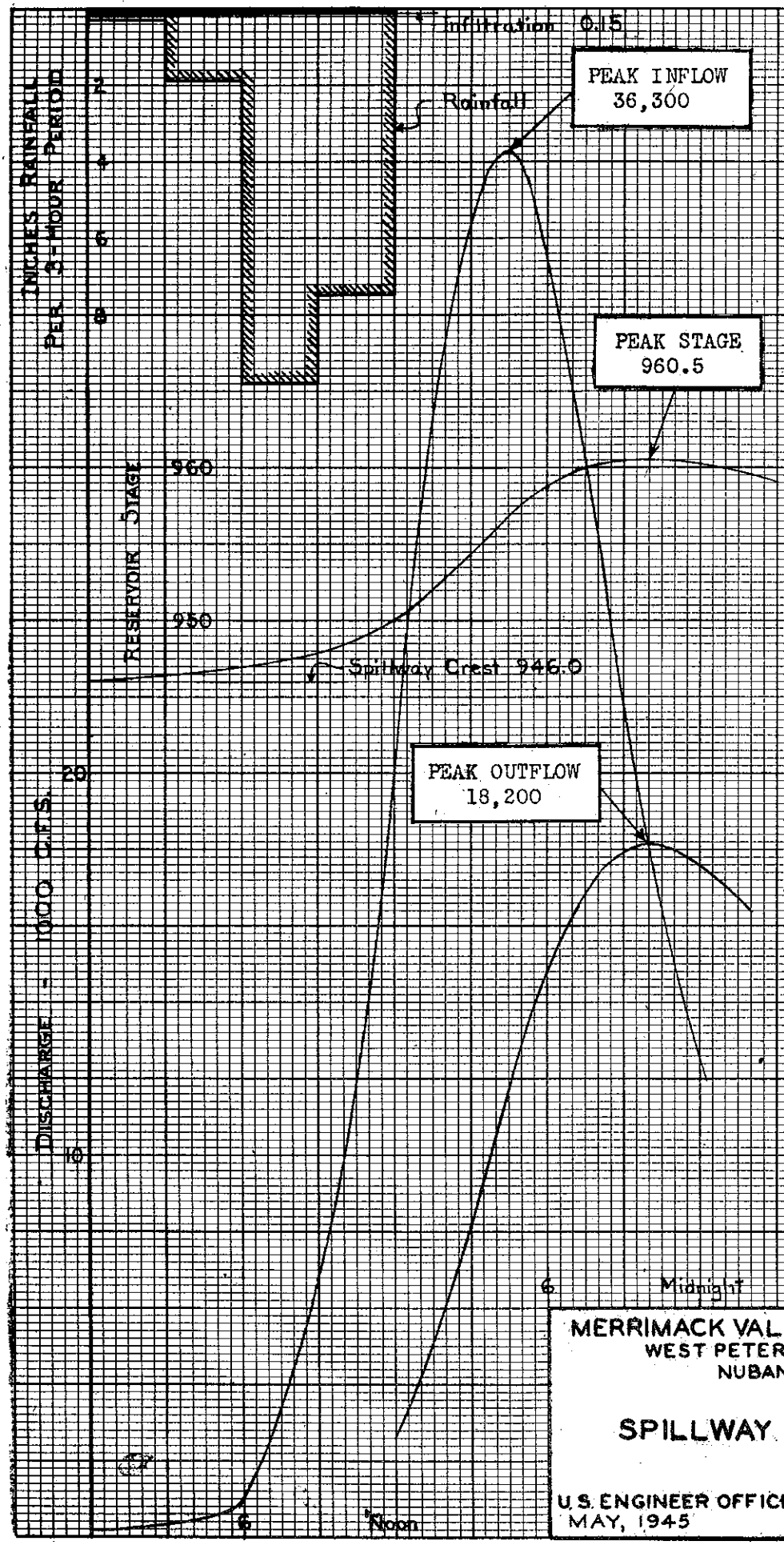
MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
NUBANUSIT BROOK

COMPUTED SPILLWAY FLOODS

U.S. ENGINEER OFFICE
MAY, 1945

BOSTON, MASS.
FILE NO. M78-12/52

PLATE I-16



UNIT HYDROGRAPH VALUES

TIME Hours	RAINFALL OVER 2 In. c.f.s.	RAINFALL UNDER 2 In. c.f.s.
Mid-night	-	-
1	80	140
2	160	300
3	260	500
4	400	740
5	580	1020
6	800	1300
7	1080	1900
8	1400	2240
9	1520	2140
10	1500	2000
11	1460	1800
NOON	1400	1600
1	1330	1380
2	1240	1100
3	1140	800
4	1020	650
5	900	580
6	810	530
7	720	490
8	660	450
9	600	420
10	540	390
11	500	360
Mid-night	460	340

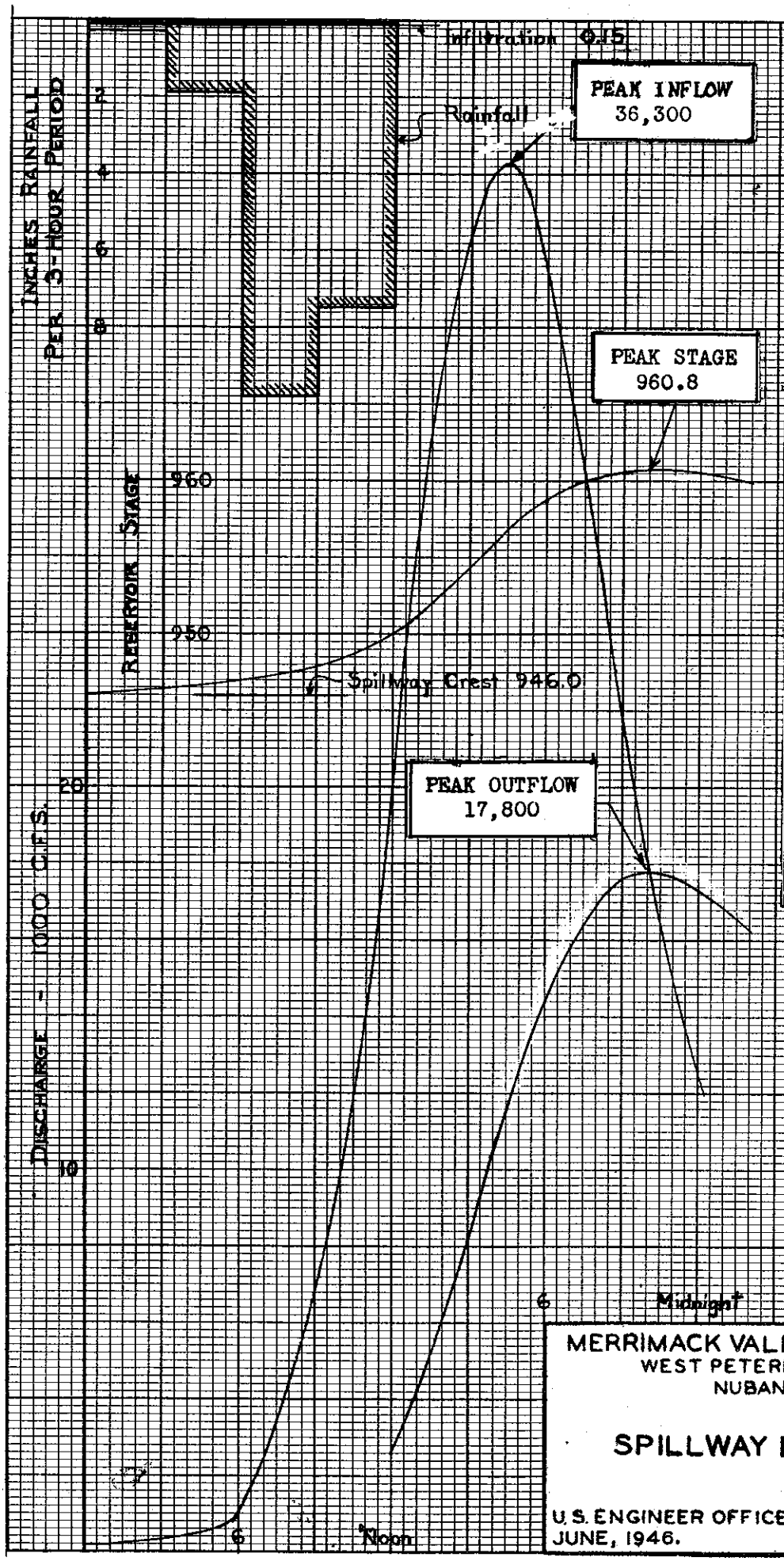
MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
NUBANUSIT BROOK

SPILLWAY DESIGN FLOOD

U.S. ENGINEER OFFICE
MAY, 1945

BOSTON, MASS.
FILE NO. M78-12/53

PLATE I-17



UNIT HYDROGRAPH VALUES

TIME Hours	RAINFALL OVER 2 In. c.f.s.	RAINFALL UNDER 2 In. c.f.s.
Mid-night	-	-
1	80	140
2	160	300
3	260	500
4	400	740
5	580	1020
6	800	1300
7	1080	1900
8	1400	2240
9	1520	2140
10	1500	2000
11	1460	1800
NOON	1400	1600
1	1330	1380
2	1240	1100
3	1140	800
4	1020	650
5	900	580
6	810	530
7	720	490
8	660	450
9	600	420
10	540	390
11	500	360
Mid-night	460	340

MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
NUBANUSIT BROOK

SPILLWAY DESIGN FLOOD

U.S. ENGINEER OFFICE
JUNE, 1946.

BOSTON, MASS.
FILE NO. M78-12/53A

PLATE I-17A

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY

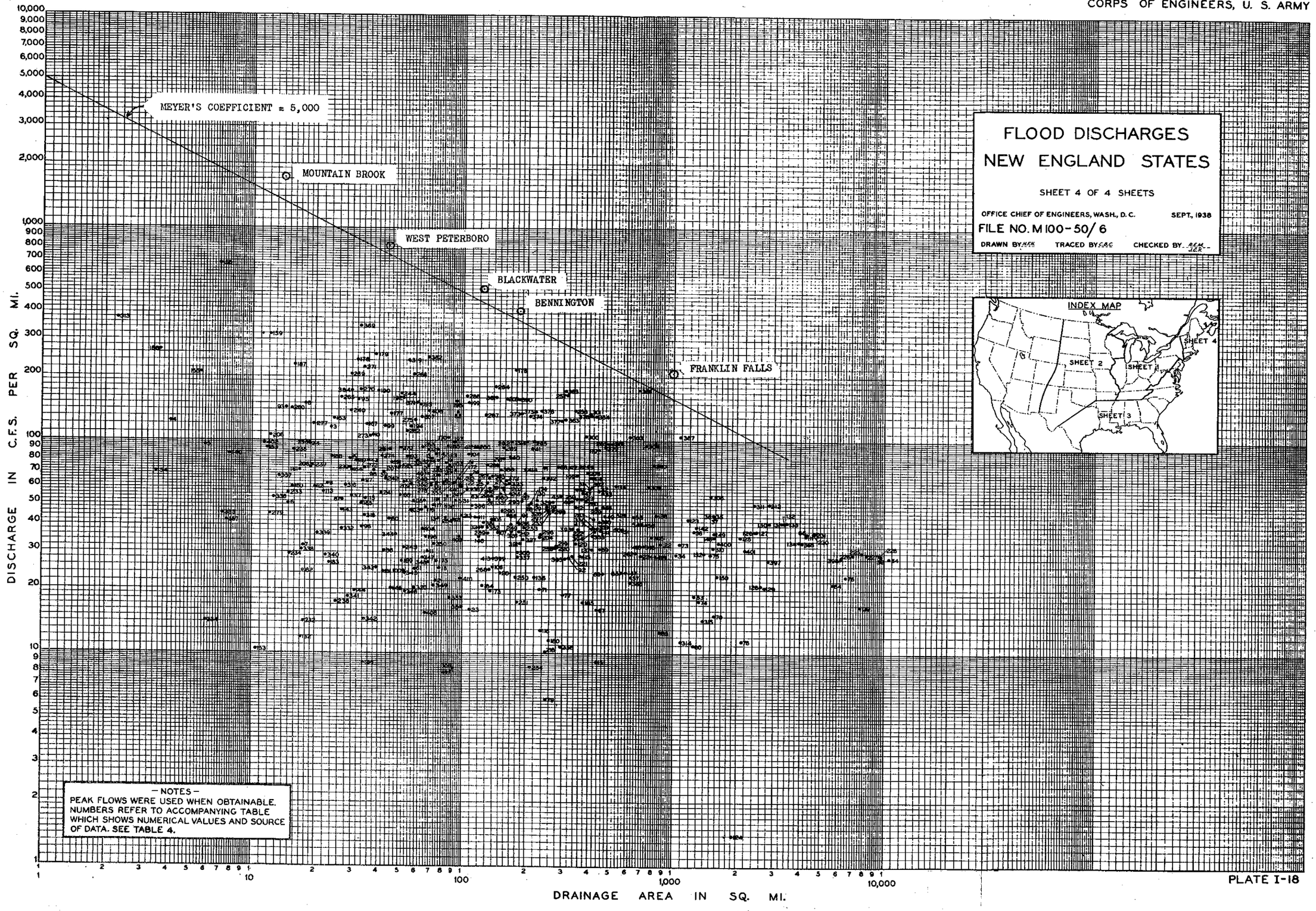
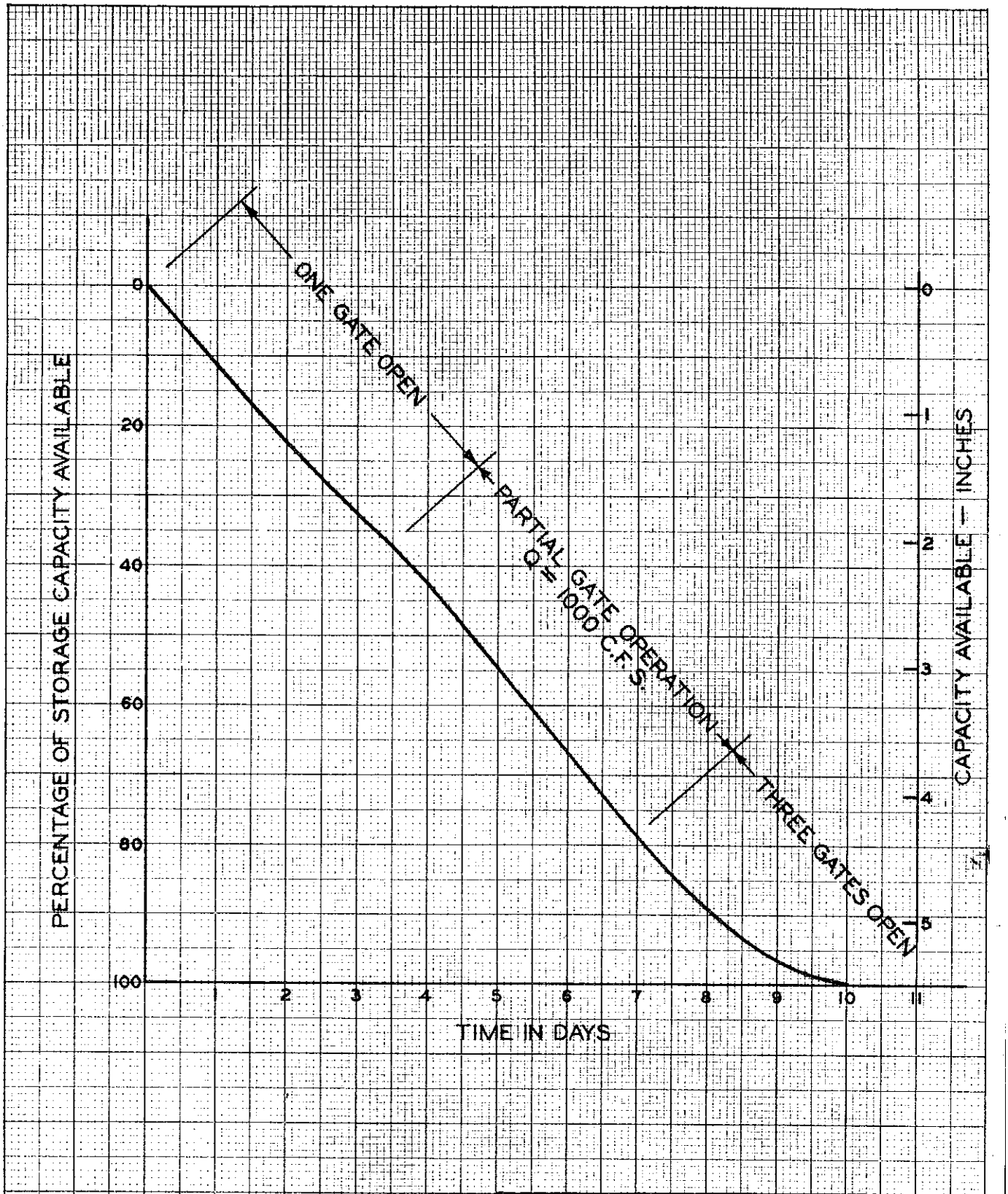


PLATE I-18



MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
NUBANUSIT BROOK

TIME TO EMPTY RESERVOIR

U. S. ENGINEER OFFICE, BOSTON, MASS.
JUNE, 1946. FILE NO. M78-12/54 A

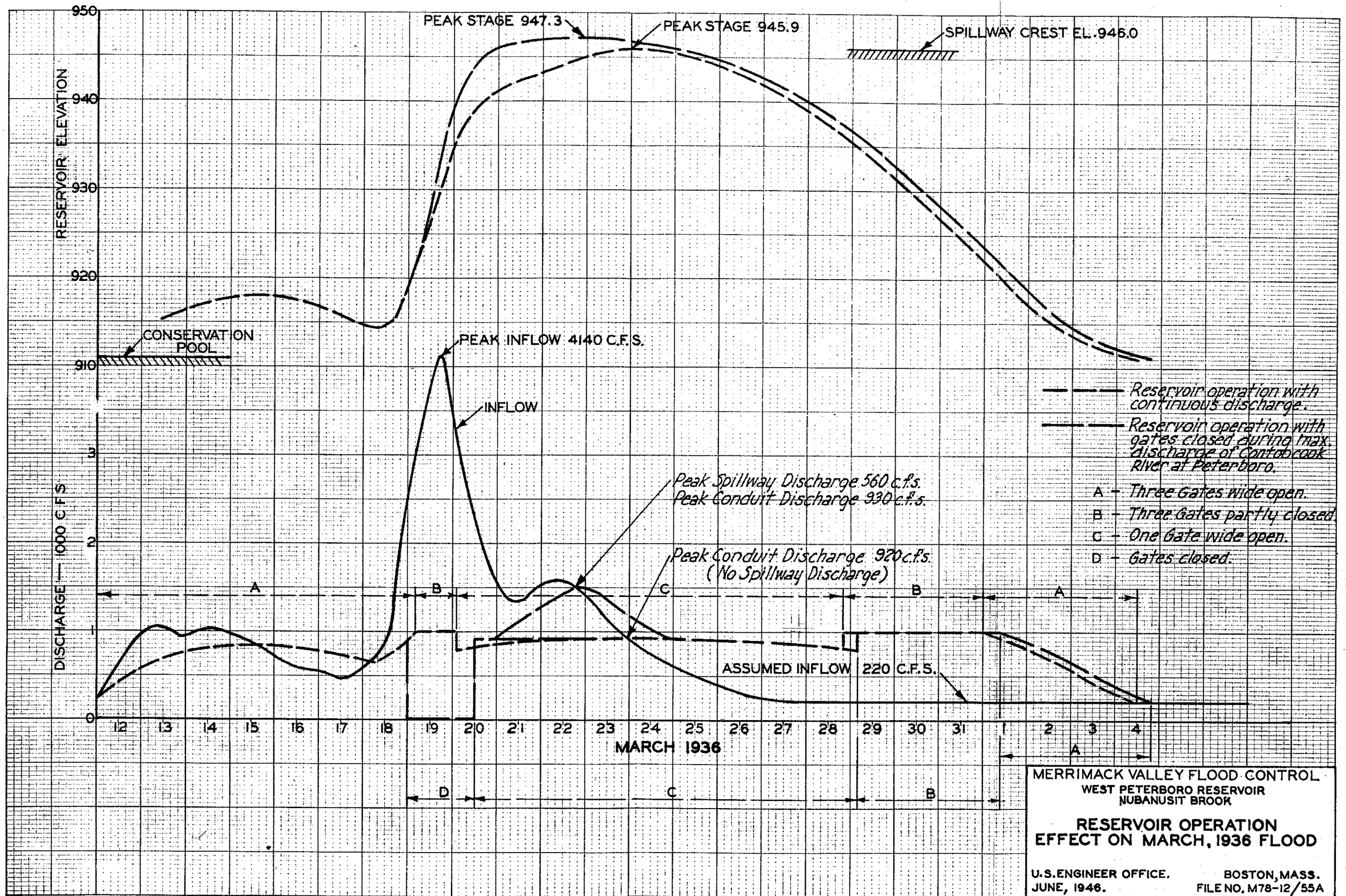
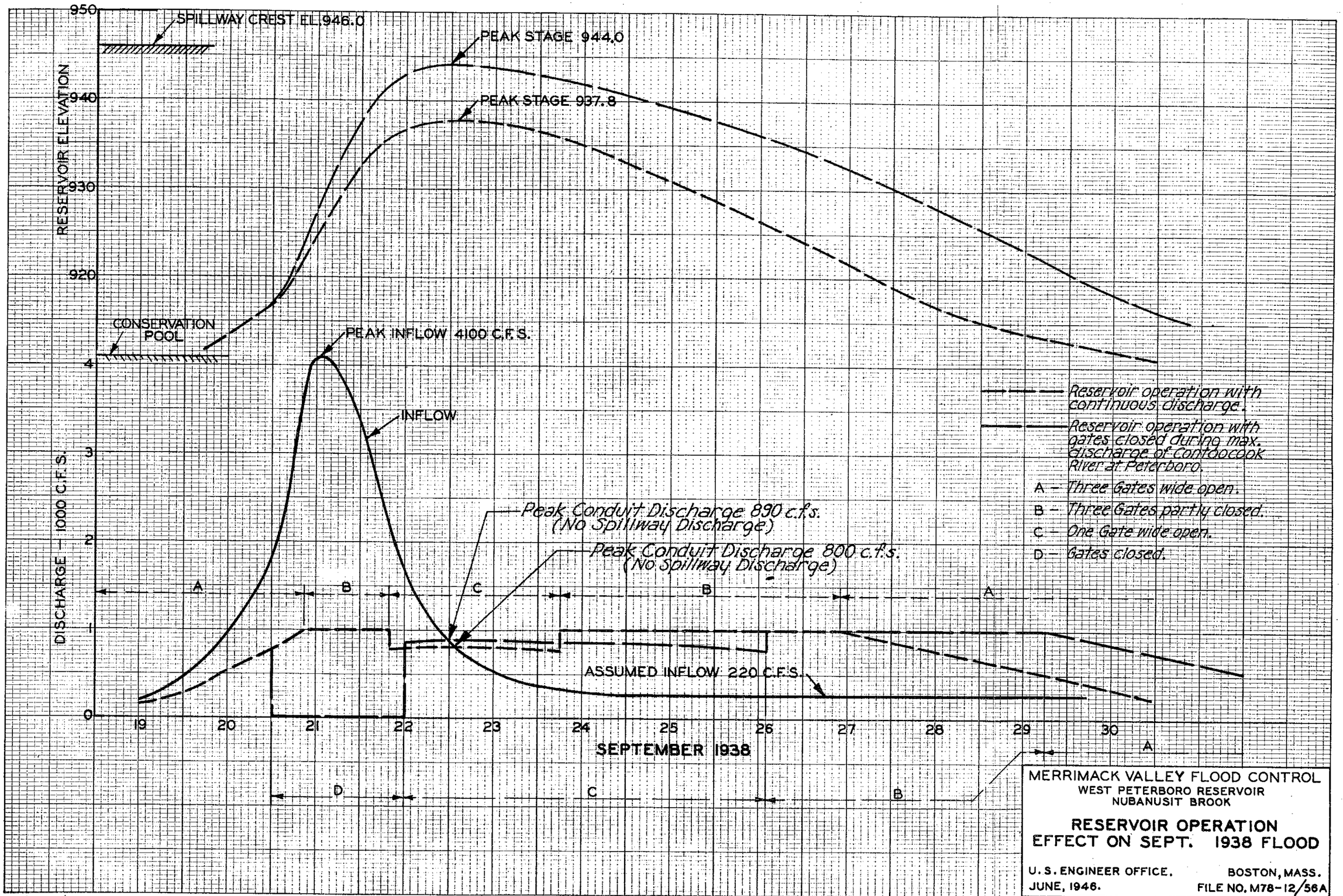

PEAK STAGE 947.3
PEAK STAGE 945.9
SPILLWAY CREST EL. 946.0
RESERVOIR ELEVATION
950
940
930
920
910
CONSERVATION POOL
PEAK INFLOW 4140 C.F.S.
INFLOW
DISCHARGE — 1000 C.F.S.
3
2
1
0
Peak Spillway Discharge 560 c.f.s.
Peak Conduit Discharge 930 c.f.s.
Peak Conduit Discharge 920 c.f.s.
(No Spillway Discharge)
A
B
C
D
ASSUMED INFLOW 220 C.F.S.
12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30 31 1 2 3 4
MARCH 1936
Reservoir operation with continuous discharge.
Reservoir operation with gates closed during max. discharge of Contoocook River at Peterboro.
A - Three Gates wide open.
B - Three Gates partly closed.
C - One Gate wide open.
D - Gates closed.
MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
NUBANUSIT BROOK
RESERVOIR OPERATION
EFFECT ON MARCH, 1936 FLOOD
U.S. ENGINEER OFFICE.
JUNE, 1946.
BOSTON, MASS.
FILE NO. M78-12/55A

PLATE I - 20

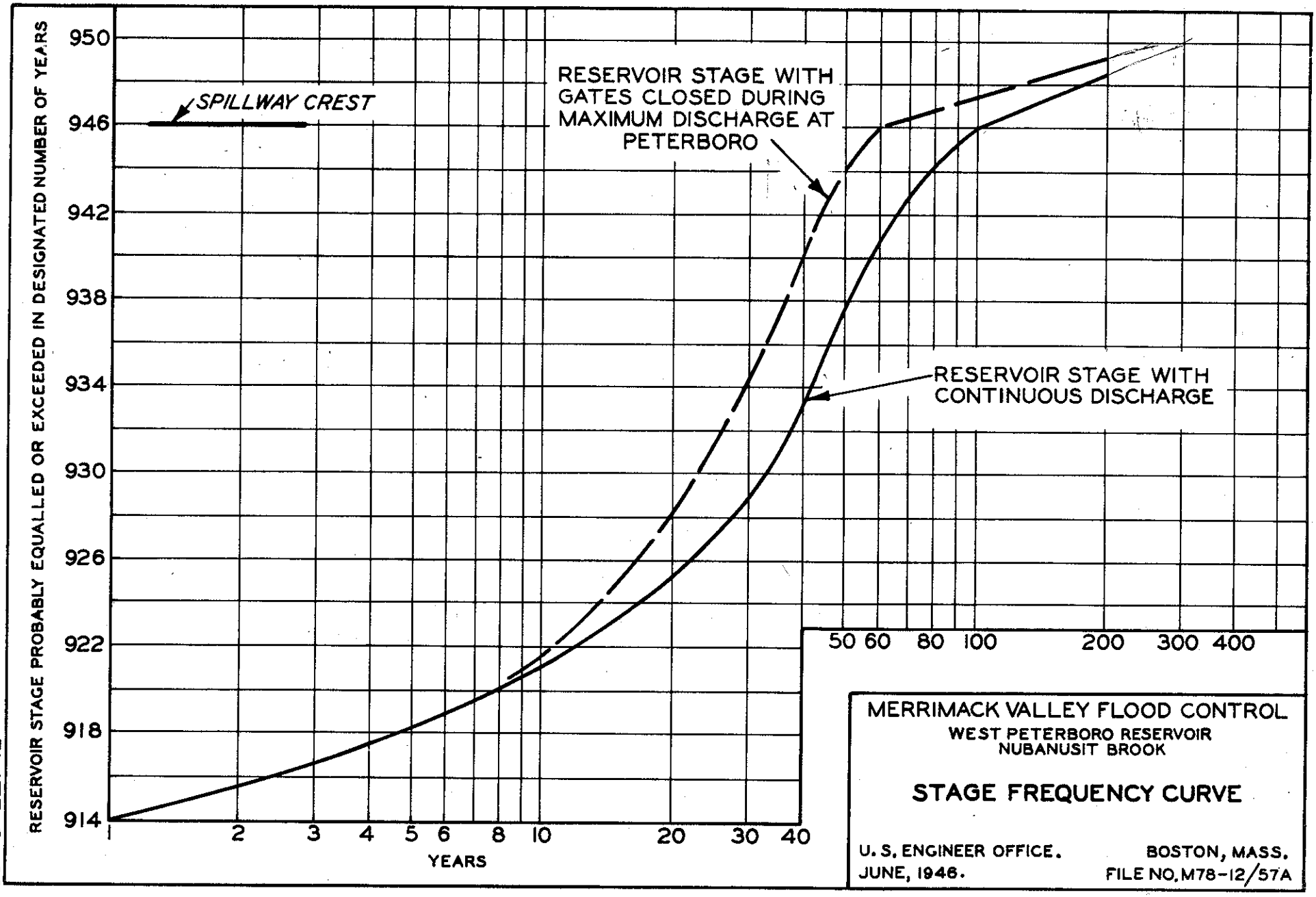


MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
NUBANUSIT BROOK

RESERVOIR OPERATION
EFFECT ON SEPT. 1938 FLOOD

U. S. ENGINEER OFFICE, BOSTON, MASS.
JUNE, 1946. FILE NO. M78-12/56A

PLATE T — 21


RESERVOIR STAGE PROBABLY EQUALLED OR EXCEEDED IN DESIGNATED NUMBER OF YEARS
950
946
942
938
934
930
926
922
918
914
SPILLWAY CREST
RESERVOIR STAGE WITH GATES CLOSED DURING MAXIMUM DISCHARGE AT PETERBORO
RESERVOIR STAGE WITH CONTINUOUS DISCHARGE
1
2
3
4
5
6
8
10
20
30
40
50
60
80
100
200
300
400
YEARS
MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
NUBANUSIT BROOK
STAGE FREQUENCY CURVE
U. S. ENGINEER OFFICE.
JUNE, 1946.
BOSTON, MASS.
FILE NO. M78-12/57A

War Department
United States Engineer Office
Boston, Massachusetts

DEFINITE PROJECT REPORT

WEST PETERBORO RESERVOIR

APPENDIX II

EXPLORATION, GEOLOGY, CONSTRUCTION MATERIALS, SOIL DATA AND ANALYSES

To accompany definite project report
dated May 1945

DEFINITE PROJECT REPORT

WEST PETERBORO RESERVOIR

APPENDIX II

EXPLORATION, GEOLOGY, CONSTRUCTION MATERIALS, SOIL DATA AND ANALYSES

C O N T E N T S

Paragraph	Title	Page
a.	Exploration	II-1
b.	Geology	II-3
c.	Construction Materials	II-5
d.	Soil Data	II-10
e.	Embankment Design	II-11
f.	Outlet Works Foundation Design	II-15
g.	Spillway Foundation Conditions	II-16
h.	Construction Procedure	II-17

PLATES

Plate	Title
II-1	Plan of Foundation Exploration at Dam Site
II-2	Record of Foundation Exploration at Dam Site
II-3	Record of Foundation Exploration at Dam Site
II-4	Record of Foundation Exploration at Dam Site
II-5	Profile and Typical Sections of Dam and Outlet
II-6	Geological Profiles at Dam Site
II-7	Plan of Foundation Exploration at Spillway
II-8	Record of Foundation Exploration at Spillway and Borrow Exploration
II-9	General Plan and Typical Sections at Spillway
II-10	Plan and Profile for Spillway Discharge and Protective Dike
II-11	Plan and Record of Borrow Exploration
II-12	Soil Data Summary
II-13	Seepage and Stability Analyses
II-14	Stability Analysis and Filter Design

DEFINITE PROJECT REPORT
WEST PETERBORO RESERVOIR

APPENDIX II

EXPLORATION, GEOLOGY, CONSTRUCTION MATERIALS, SOIL DATA AND ANALYSES

a. Exploration.- (1) Reconnaissance.- Reconnaissance was conducted in the general vicinity selected for the project location. Topography and geography were studied and surficial examination was made to investigate the principal geological formations. Observations indicated that a satisfactory foundation could be developed and that suitable construction materials were available within reasonable hauling distance.

(2). Subsurface Exploration.- (a) Methods of Exploration.- Subsurface explorations were conducted by the following methods.

1. Seismic Lines.- Seismic exploration was conducted in accordance with procedures developed by Mr. E. R. Shepard*, Office, Chief of Engineers.

2. Drill Holes.- Exploration to recover soil and rock samples for classification and for determination of extent of occurrence were obtained from 3-inch cased drill holes from which 2-inch soil samples were recovered by drive sampling into undisturbed material and from which 1-5/8" rock cores were recovered using a diamond drill. Smaller samples were obtained from 2-inch cased holes for borrow exploration. Undisturbed samples of silt were recovered from one 6-inch diameter drill hole.

3. Test Pits.- Shallow test pits approximately 6 feet deep were used to obtain large samples for determination of natural soil properties and for use in laboratory determination of remolded properties. Test pits were extended by auger boring where possible. Where large samples were required from depths of more than 6 feet below ground surface test pits were sheathed and braced as required.

4. Pressure Tests in Rock.- Where cores from bedrock indicated badly weathered and fractured materials, drill holes were pressure tested in the zone of bedrock penetration in 5-foot segments and for total depth under pressure of approximately one pound per square inch per foot of depth below ground surface.

* "The Seismic Method of Exploration applied to Construction Projects", published in The Military Engineer, September-October 1939.

5. Observation Well.- One observation well was installed in a drill hole where it was desired to determine ground water surface.

(b) Foundation Exploration at Dam Site.- 1. Foundation exploration at the dam site consisted of seven seismic lines, fifty-three 3-inch drill holes, one 6-inch drill hole, eight shallow test pits and one deep sheeted test pit. Location of explorations is shown on Plate II-1 and record of explorations is shown on Plates II-2, II-3, and II-4. Samples were obtained from all explorations for classification and undisturbed samples were obtained from one 6-inch drill hole and from test pits for determination of natural and remolded soil properties.

2. Pressure tests were performed in 13 drill holes in bedrock at this site. In each hole a highly shattered capping existed and the pressure testing equipment could not be sealed. The depth of this zone varied from 1 to 37 feet. Below this zone the rock becomes less fractured with depth, usually becoming relatively tight within 5 to 10 feet. In drill hole D26 a large seam capable of accommodating a flow of 8.9 g.p.m. was encountered between depths of 79 and 82 feet.

3. One observation well was installed in exploration D-61 to determine ground water surface. The point of the observation well rests on decomposed bedrock at 10.8 feet below ground surface. Periodic observations at this well since its installation in July 1944 have not encountered water in the well except in the bottom few inches of the well point which indicates that ground water is more than 10 feet below ground surface.

(c) Foundation Exploration at Spillway.- The initial exploration at the spillway site conducted early in 1940 consisted of 3 drill holes and 25 seismic lines. Additional drilling consisting of 11 drill holes was performed in 1944 and 1945. Exploration locations, rock contours, and pertinent data are shown on Plate II-7 and record of explorations is shown on Plate II-8.

(d) Borrow Exploration.- Exploration for borrow materials consisted of field reconnaissance, 9 drill holes and 65 shallow test pits. Three 3-inch holes were drilled in the pervious borrow area and six 2-inch holes were drilled in the impervious borrow area. Samples of each soil stratum encountered in drill holes and test pits were obtained for classification. Representative samples of principal materials encountered in excavation of test pits were obtained for laboratory determination of soil properties. Natural density of pervious materials was determined by sand displacement method in the field. Undisturbed

chunk samples of cohesive materials were obtained for density tests in the laboratory. A plan of borrow exploration is shown on Plate II-11 and selected logs of borrow exploration are shown on Plates II-4 and II-11.

b. Geology.- (1) Regional.- (a) Nubanusit Valley.- The rocks of the Nubanusit Valley are metamorphic and igneous of Paleozoic age. The metamorphic rocks are schists and gneisses which are deposited originally as flat beds below the water surface. A period of diastrophism which occurred subsequent to this deposition resulted in the crumbling and folding of the layers until they appeared at every attitude. Large amounts of magma were injected into the metamorphosed rocks, where relatively slow cooling occurred and granite or related granitic rocks formed. A period of regional uplift followed and streams began the gradual erosion of the overlying metamorphic rocks. As the drainage pattern developed, the streams cut downward through the metamorphic rocks and exposed large areas of the granite. The granite and schist are not durable rocks and have been affected by weathering to depths of as much as 50 feet. At the beginning of the Pleistocene period, the Nubanusit valley had been developed and the brook was flowing in a bed-rock channel. During the Pleistocene period, the valley was covered by a portion of the continental glacier that occupied Canada and the Northern part of the United States. The glacier modified existing topography by erosion and deposition. Erosion by the glacier consisted of bevelling the northern slopes of hills and steepening their southern slopes. In addition, it scoured and widened north-south valleys and tended to deepen and widen saddles transverse to its line of motion. Deposition by the glacier caused the greatest change in topography. Glacial deposits are of two types; the unsorted, unstratified glacial till deposited directly from the ice, and the sorted gravels, sands, silts and clays that are washed from the ice and deposited in water. Glacial till may consist of a heterogeneous mixture of clay, silt, sand, gravel, cobbles and boulders, or some of these sizes may be lacking. In general, the till of the Nubanusit Valley does not have material as fine as the clay sizes in any appreciable amount. Most of the till may be broken down into two classes; sandy till, which has a predominance of sand sizes, and silty till which has a predominance of silt sizes. Glacial till is highly stable, capable of furnishing good support and is practically impervious. It was deposited in elliptical shaped hills known as drumlins, and as blankets which were smeared across the valleys, partially filling them and disorganizing the drainage system. In many cases interstream divides were buried by glacial deposits and streams now pass from one old drainage basin to another. Where streams still flow within the old valley, they are

seldom in the old channel, therefore when a stream has a rock floor, there may be a buried channel nearby. The sorted materials consist of gravels, sands, silts and clays or, as in the case of a modified glacial drift, combinations of these sizes. These materials have been washed from the ice, sorted or partially sorted and deposited in the form of terraces, eskers, and delta deposits. When the ice sheet retreated from the region, it left the Nubanusit Valley filled with glacial deposits. It is within these deposits that the present channel has been developed.

(b) Vicinity of Dam Site.- In the vicinity of the West Peterboro dam site, the Nubanusit Brook flows southward in a valley composed principally of glacial till. The present channel is located high on the west wall of the pre-glacial bedrock channel, and has cut downward onto the bedrock at a few scattered points. Large quantities of glacio-fluvial materials occur a short distance upstream from the dam site.

(c) Source of Data.- Information pertaining to historical and existing geology of the Nubanusit Valley has been abstracted from reports on the West Peterboro site presented to the Boston District Office by Mr. Sidney Paige, Geologist, U. S. Engineer Office, North Atlantic Division, New York City; by Mr. Charles P. Berky, consulting geologist, Columbia University, and by Mr. Irving B. Crosby, consulting geologist, Boston, Massachusetts.

(2) Dam Site.- (a) General.- The geology of the dam site is discussed in the following sub-paragraphs according to principal conditions encountered. A plan of foundation exploration showing location of drill holes, seismic lines and contours of bedrock is shown on Plate II-1, and graphic logs of individual explorations are shown on Plates II-2 through II-4. Geological profiles are shown on Plates II-5 and II-6. A thin layer of pervious material immediately on top of bedrock wasindicated in reconnaissance investigation but was not proven by the drilling.

(b) West Abutment.- The west abutment of the dam and the entire western wall of the valley are composed of sandy to silty glacial till overlying bedrock at depths of 2 to 40 or more feet below the ground surface. The upper zone of the till body has been affected by weathering and frost action and is less compact than the underlying material. Many boulders are strewn over the ground surface with particular concentration in the vicinity of the outlet discharge channel. The bedrock is silicious schist that has been severely weathered within localized zones, and is highly fractured throughout its upper few feet (Plate II-1).

The same general conditions prevail upstream and downstream of the da m.

(c) East Abutment.- The east abutment and eastern valley wall are composed principally of compact sandy to silty till that overlies bedrock at depths of 60 to 135 feet or more. The upper zone of the till to a depth of four to six feet has been affected by weathering and frost action and is less compact than the underlying till. Scattered boulders occur on the ground surface. Bedrock continues to dip downward toward the east while the ground surface rises, thus increasing the thickness of the till body. Upstream of the dam the foundation conditions remain the same. A short distance downstream from the da m, a localized deposit of compact laminated silt occurs at a depth of approximately 18 to 30 feet.

(3) Spillway.- The spillway is located approximately three miles due north of the dam and in a saddle which separates drainage between Nubanusit Brook and Ferguson Brook. Bedrock which forms the sa ddle is fresh, unweathered moderately fractured porphyritic granite. Bedrock outcrops adjacent to the spillway location but dips steeply northward and is overlain by compact silty till. Cobbles and boulders are densely strewn over the area, with concentrations at areas shown on Plate II-7.

(4) Borrow Sources.- Ample quantities of compact silty glacial till suitable for impervious fill occur in both banks of the valley. The area selected for development is on the east wall of the valley and upstream from the dam. Large quantities of glacial sand and gravel deposited in eskers, kames and terraces are located on the east side of the stream within one (1) mile upstream from the dam site. These deposits contain sufficient suitable materials for pervious fill and filter materials, and concrete aggregates may be obtained from the same source by processing.

c. Construction Materials.- (1) Materials Required.- Principal materials required for construction of the project are summarized in the following tabulation.

Item	Quantity C.Y. (Emb. Meas.)
Impervious Fill	85,000
Random Fill	77,000
Pervious Fill	107,000
Concrete Aggregate and Special Gravel	39,000
Rock for Slope Protection	29,000
Topsoil	17,000

(2) Materials Available in Required Excavations.- Materials from required excavations will be used for embankment purposes where suitable. Of the total excavation approximately 166,000 cubic yards of material is suitable for use in the embankment for the following purposes.

Item	Quantity C.Y. (Exc. Meas.)
Impervious Fill	47,000
Random Fill	82,000
Rock for Slope Protection	20,000
Topsoil	17,000

Rock for slope protection is taken from rock excavation at the spillway although part of the necessary rock may be obtained from boulders in required excavations.

(3) Materials Available for Borrow.- (a) Impervious Borrow.- Massive deposits of sandy and silty glacial till form the walls of the present stream valley. These deposits are excellent sources of material for impervious embankment construction. The area selected for impervious borrow is on the east bank of the stream a short distance upstream from the dam site (Plate II-11). The materials selected for use is the unweathered and unmodified till which occurs under a surface cover of three to ten feet.

(b) Pervious Borrow.- Large quantities of glacio-fluvial deposits occur a short distance upstream from the dam site and exist as kames, eskers and terraces composed of clean sand and gravel. The area selected for development is on the east side of the stream approximately 3/4 mile upstream from the dam site. Only shallow stripping is required to expose suitable materials.

(c) Processed Aggregate.- A survey to locate all possible commercial sources of processed aggregates is in process. Information to date indicates that processing plants are now operating at the following locations.

Locations	Approximate Haul Distance
Manchester, New Hampshire	30 miles
Keene, New Hampshire	25 miles
Fitchburg, Massachusetts	40 miles

Since all these plants are at considerable distance, it may be economical to esta blish a processing plant in the pervious borrow area for the project. The suitability of aggregates processed from

this source has not been determined; however, experience with similar deposits indicates that satisfactory aggregates may be obtained.

(4) Materials Summary.- A summary of construction materials required, with indicated source, and materials available from required excavation and selected borrow areas, with indicated disposition, is shown in the following tabulation. Indicated embankment quantities include allowance for shrinkage and settlement.

CONSTRUCTION MATERIALS

MATERIALS REQUIRED FOR EMBANKMENT

Type	Quantity, C.Y.				Source Excavation Measure, C.Y.
	Embankment Measure			Excavation Measure	
	Total	Dam	Spillway		
Impervious Fill	85,000	85,000	- - -	98,000	47,000 Structure Excavation at Dam Site 51,000 Impervious Borrow
Random Fill	71,000	34,000	- - -	39,000	39,000 Structure Excavation at Dam Site
		- - -	37,000	43,000	43,000 Structure Excavation at Spillway
Pervious Fill	107,000	107,000	- - -	123,000	123,000 Pervious Borrow Area
Conc. Aggregate and Scr. Gravel	39,000	30,000	9,000	60,000	Available from pervious borrow area or commercial sources within 21 miles.
Rock for Slope Protection	29,000	19,000	10,000	20,000	20,000 Rock excavation at spillway
Topsoil	17,000	7,000	10,000	17,000	17,000 Selected stripping at Dam Site and Spillway

MATERIALS AVAILABLE FROM EXCAVATION AND BORROW

Type	Quantity, C.Y. Excavation Measure			Disposition Excavation Measure, C. Y.
	Total	Dam	Spillway	
Stripping Foundation	30,000	30,000	- - -	7,000 Topsoil 23,000 Waste
Structure Foundation (Earth)	365,000	107,000	- - -	47,000 Compacted Impervious Fill 39,000 Compacted Random Fill 21,000 Waste
		- - -	340,000	43,000 Backfill grading and dike 297,000 Waste
Structure Excavation (Rock)	57,000	10,000	- - -	10,000 Upstream Cofferdam
			30,000	20,000 Slope Protection 10,000 Excess excavation not required for this project
Impervious Borrow	56,000	- - -	- - -	5,000 Stripping and Waste 51,000 Compacted Impervious Fill
Pervious Borrow	133,000	- - -	- - -	10,000 Stripping and Waste 123,000 Pervious Fill

d. Soil Data.- (1) Scope and Extent of Laboratory Investigation.- All samples of material encountered in field exploration have been submitted to the laboratory for final classification and testing. Selected representative samples have been tested to determine compaction characteristics, shear strength, permeability and consolidation characteristics. Investigation of these soil properties is continuing but sufficient data have been obtained to determine the general range of results.

(2) Laboratory Procedures.- (a) Mechanical Analysis.- Mechanical analysis of selected representative samples have been made using a standard sieve analysis with a minimum sieve of 100 meshes per inch, and hydrometer analysis of all sizes passing that sieve.

(b) Specific Gravity.- The specific gravity of principal materials has been obtained by the water displacement method (A.A.S.H.O. T100-38).

(c) Density.- Density of cohesive soils has been determined from undisturbed paraffined chunk samples. Density of cohesionless soils has been determined in the field by the sand displacement method.

(d) Water Content.- The natural water content of principal materials was determined from samples obtained in the field and transported to the laboratory in sealed jars. Results are reported in terms of percentage of oven dry weight.

(e) Compaction Characteristics.- Compaction characteristics of cohesive soils are determined from the moisture density relation obtained using standard and modified Proctor Compaction Tests. Compaction characteristics for cohesionless soils include determination of minimum dry density by placing soil in a container in the loosest possible condition without vibration or impact, and maximum dry density obtained by impact compaction.

(f) Shear Strength.- Shear strength of principal materials was determined using a triaxial compression device in accordance with procedures outlined by A. Casagrande and R. E. Fadum, in "Notes on Soil Testing for Engineering Purposes", a publication from Harvard University Graduate School of Engineering.

(g) Permeability.- Permeability of materials was determined using de-aired water in a falling head type apparatus with a plastic, transparent permeameter following the general

procedure described by G. E. Bertram in "An Experimental Investigation of Protective Filters", a publication of Harvard University Graduate School of Engineering.

(h) Consolidation.- Laboratory consolidation characteristics are determined by using fixed ring consolidation test apparatus for a 4-1/4-inch diameter sample of 1-1/4-inch in initial thickness.

(3) Test Results.- (a) Classification of Materials Encountered.- Classification of materials encountered in field explorations are shown by graphic logs of explorations on Plates II-2, 3, 4, 8 and 11. This classification includes color, compactness, consistency, plasticity and basic soil type of each stratum encountered.

(b) Soil Data Summary.- A summary of soil test data compiled from laboratory tests performed on samples of principal foundation and borrow soils appears on Plate II-12. On this plate, Figures 1, 2 and 3 show range of gradation of principal foundation and borrow materials. Figure 4 shows the moisture density relations for silty till encountered in both foundation and impervious borrow areas. Figures 5 and 6 are plots of shear strength for principal impervious and pervious embankment materials. Figure 7 is a summary of properties of materials encountered in required excavation and of materials required for embankment. Conservative estimated values shown in Figure 7 are based on results of these tests augmented by tests on similar materials for other sites, particularly at Franklin Falls and Blackwater Dam.

e. Embankment Design.- (1) Design Criteria.- The design of the dam embankment and dam embankment foundation involved a study of the foundation conditions and characteristics, a study of the characteristics of the available embankment materials, the choice of a section which utilizes economically the available embankment materials and is safe under any condition. This section includes a description of the results of those investigations which are pertinent to the design, a discussion of the choice and economy of the embankment section, and the analysis demonstrating that the section is satisfactory for the following criteria:

(a) The slopes of the embankment must be such that no shear slide can occur in the embankment or foundation materials.

(b) The void ratio of all materials in which a flow slide might occur must be less than the critical void ratio.

(c) Seepage must be controlled so that no detrimental uplift pressures or transportation of material can occur.

(d) Provision should be made to compensate for the settlement of the embankment after construction to insure the design free board height.

(2) Preliminary Embankment Design.- The embankment design presented in the initial project report, dated August 1940, included a large central impervious section and a minor downstream random section with upstream and downstream surface protection essentially as shown in the present design.

(3) Definite Project Design Features.- (a) General.- The definite project design features presented herein are the same as presented for review by the Board of Consultants in December 1944. Pertinent features are discussed in the following paragraphs.

(b) Foundation Conditions.- The foundation conditions for the immediate vicinity of the dam have been described in Paragraph b. Where bedrock is close to the surface in the west bank of the valley and under the present stream channel grouting is accomplished to a depth of 20 feet (Plate II-5). In the east side of the valley, foundation for the embankment consists of compact, sandy and silty glacial till.

(c) Impervious Section.- The central impervious section of the dam consists of silty glacial till extending to bedrock in the west side of the valley and under the present stream channel. In the east side of the valley an inspection and cut-off trench extends through the upper zone of weathered more pervious till to 20 feet below existing ground surface. In the lower portion of the valley between the outlet conduit and approximate station 6 + 70, an impervious blanket 5 feet in thickness extends from the central core under both the upstream pervious section and the upstream toe into the upstream cofferdam. This blanket is provided to obtain maximum cut-off of water seeping under the dam through possible pervious channels in the badly fractured bedrock surface and through local more pervious zones in the till.

(d) Random Section.- The random section of the embankment between the central core and the downstream pervious section of the dam has been provided to utilize the estimated quantity of usable material from structure excavation which does not meet requirements for impervious or pervious fill and to provide a transition between these two materials.

(e) Pervious Sections.- The pervious sections of the dam have been designed to provide adequate stability of the upstream slope of the dam under the most severe condition of sudden drawdown of pool elevation and to hold the maximum line of seepage in the downstream section well below ground surface for condition of sustained high upstream pool elevation.

(f) Drainage Features.- Drainage through and under the earth embankment is collected in a layer of very pervious sand and gravel under the downstream portion of the embankment. Typical details of the drainage design features are shown on Plate II-5. Drainage wells at the downstream side of the dam as proposed in the report of August 1940 are now considered unnecessary.

(g) Slope Protection.- The upstream slope of the earth embankment is protected from wave action and surface erosion by a dumped rock fill on a layer of screened gravel (Plate II-5). Above the rock on the down stream slope, topsoil is used for protection of erosion by wind and surface water.

(4) Design Studies.- (a) Design of Filters.- In all sections of the embankment and its foundation through which water passes, a study has been made to insure that the gradation of adjacent soils is such that the finer sizes of one soil will not be transported by the seeping water into the voids of an adjoining soil. In areas where seepage water is collected, the gradations of adjacent soils fulfill the above criteria and in addition the soils become progressively several times more pervious in the direction of discharge. The criteria used for these analyses is that contained in Chapter XXI of the Military Engineering Manual, OCE. The determination of the general range of material for the filter blanket in the downstream section of the earth embankment is illustrated on Plate II-14.

(b) Study of Seepage.- The seepage through and under the embankment has been studied by use of a flow net (Plate II-13). Based on this preliminary study the total seepage through and under the embankment is estimated at less than 0.1 c.f.s. for sustained maximum pool at elevation 960.5.

(c) Embankment Stability.- 1. Method of Analysis.- Using the most dangerous circle method, the stability ratio for shear failure of the dam embankment and its foundation was determined by investigating the forces tending to cause movement and those producing potential resistance to movement on several circular sliding surfaces of weakness which were selected by systematic trial. The analysis of flow slide failure is described

in a following subparagraph. In the analysis the driving forces include the rotating effect of the weight of the soil mass and water above the surface of failure and also the forces generated by water pressure. The forces producing potential resistance consist of the shear strength generated along the sliding surface. The ratio of the potential resisting force and the driving force is termed the stability ratio. A sufficient number of potential surfaces were analyzed to determine the position of the surface having the least stability ratio, termed the "minimum" stability ratio. A minimum stability ratio of unity indicates equality of driving and potential resisting forces and implies that the embankment is on the verge of failure, while a minimum stability ratio of greater than unity indicates that the structure possesses reserve strength.

2. Section Analyzed.- Analysis was made for the typical valley section which is the largest section of the entire project. In the analysis the embankment and its foundation were considered to be integral and sliding surfaces were allowed to pass through foundation and embankment without delineation. The soil characteristics used in this analysis are contained in a tabulation on Plate II-12.

3. Upstream Slope.- The results of 6 trials of potential failure surfaces in the upstream section of the dam are shown on Plate II-13. The minimum stability ratio, considering the embankment and its foundation as a unit, and for conditions existing immediately after a sudden drawdown of the upstream pool is 1.83. Analysis was made, using the method of slices with the usual approximation that soil forces on either side of each slide balance each other. It is assumed that the rock slope protection, the gravel bedding and the pervious section of the dam will drain as rapidly as the pool is drawn down but that the impervious section remains saturated.

4. Downstream Slopes.- Three trial sliding surfaces were investigated for the downstream slope with a minimum stability factor of 2.11 (Plate II-13).

5. Foundation and Abutments.- Stability of the foundation and abutment in analyses for possible shear failure was investigated in conjunction with analyses of the upstream and downstream slopes. The abutment and minor portions of the embankment are considered of greater stability than the principal embankment section for which the analysis is shown on Plate II-13.

(d) Flow Failure Analyses.- A flow failure is defined as the liquefaction by shock of a mass of loose, saturated

cohesionless material. Based upon the experience gained by the detailed study of flow failure made in connection with the design of the Franklin Falls Dam, a flow failure of either the upstream compacted pervious or random sections of the embankment or the embankment foundation is considered highly improbable. An analysis of the possibility of a flow failure will be made in connection with the final design.

(e) Surface Slides.- Surface slides may occur during the period of frost melting in soils affected by frost action. Such surface slides will not occur in the embankment since all materials, with the exception of the turf, in the range of frost penetration (at this site abour four feet) are cohesionless and not susceptible to frost action.

(f) Settlement Analysis.- Based on experience gained as a result of settlement observations on the completed Franklin Falls Dam, a settlement of the embankment at its maximum section is expected to be approximately 1 to 2 inches due to foundation consolidation and 2 to 4 inches due to consolidation of the compacted impervious section under its own weight. Total Settlement will occur gradually over a period of several years. Upon further analysis and before final design plans are prepared, definite values will be established and the embankment constructed to sufficient initial height to allow for this settlement.

f. Outlet Works Foundation Design.- The location of the outlet structures was selected after considerable subsurface exploration work had been carried on in the vicinity. The location provides a bedrock foundation for all of the outlet structures.

(1) Intake Structure.- The intake structure is in bedrock excavation. The depth of excavation is sufficient to remove the loose shattered capping of bedrock which prevails throughout the area. The structure has a concrete lining which provides a smooth flow surface and which aids in the prevention of underseepage.

(2) Conduit.- The outlet conduit is constructed of concrete in a shallow cut in bedrock which is badly weathered and fractured. Seepage along the line of conduit is prevented by concrete collars and grouted rings in the sides and bottom of the excavation. Backfill, placed around the conduit from the intake portal to the downstream limit of the impervious section, consists of compacted impervious material. Below the impervious section, the conduit has been backfilled for a short distance with random material to provide a transition from the impervious

material to the sand and gravel that is used as backfill under the downstream random and pervious sections. The sand and gravel backfill provides free drainage of water that seeps through, around and under the conduit.

(3) Stilling Basin.- The stilling basin is a concrete structure founded on bedrock. The depth of bedrock excavation is sufficient to penetrate the zone of highly weathered and fractured rock and to provide a foundation of fresh moderately fractured schist.

(4) Discharge Channel.- The discharge channel is principally in earth excavation with a small portion in rock cut at the end of the stilling basin. Protection against erosion is provided by 18 inches of riprap over nine inches of screened gravel and nine inches of sand. This protection extends from the bottom of the channel upward to elevation 915 and protects the natural soil against frost action. Above elevation 915 the slopes are protected by nine inches of seeded topsoil. Investigation was made of stability of the west side of the discharge channel with respect to shear failure. Results shown on Plate II-14 indicate a minimum stability ratio of 1.39.

g. Spillway Foundation Conditions.- (1) General.- Preliminary design considered two possible spillway locations. One of these locations was at the dam site and the other was a saddle approximately three miles north of the dam site. The dam site location provided for a spillway just below the west abutment of the embankment, and approach and discharge channels, excavated from overburden and rock to serve the spillway. This site was proposed in the initial project report and it was planned to use the rock, which was excavated, as slope protection material. Further subsurface exploration at the site indicated that the rock to be excavated would not be usable for riprap and that the stability of the side hill cut was questionable. When these facts were ascertained, additional investigation was conducted at the saddle location, where subsurface conditions were found to be more favorable.

(2) Intake Channel.- The intake channel is approximately 1500 feet long and composed mostly of earth excavation with some rock excavation in the vicinity of the spillway weir. Overburden consists of a compact sandy to silty glacial till with many boulders of varying sizes. Earth side slopes of the channel have been selected as 1 on 2 and are considered stable under conditions of rapid drawdown of the water surface in the channel. Where appreciable water velocities occur, a protective cover of 18 inches of riprap over a gravel filter has been provided, other

slopes are protected by 6 inches of seeded topsoil.

(3) Spillway Weir.- The spillway weir is a concrete structure founded directly upon bedrock of fresh, moderately fractured, porphyritic granite. The entire channel in the vicinity of the weir is excavated in rock to the desired bottom elevation. Since the rock dips toward the north in this section, the northern wall of the channel is extended upward to the desired top elevation by a concrete wall.

(4) Discharge Channel.- The discharge channel is approximately 1300 feet long, of which the first 700 feet is largely rock excavation. Overburden consists of sandy to silty glacial till containing large quantities of boulders of varying sizes. Side slopes of the excavation have been selected as 1 on 2 and are considered stable under conditions of rapid drawdown of the water surface in the channel. Where high water velocities occur, the earth slopes are protected by two feet of riprap over a gravel filter. All other slopes are protected by 6 inches of seeded topsoil.

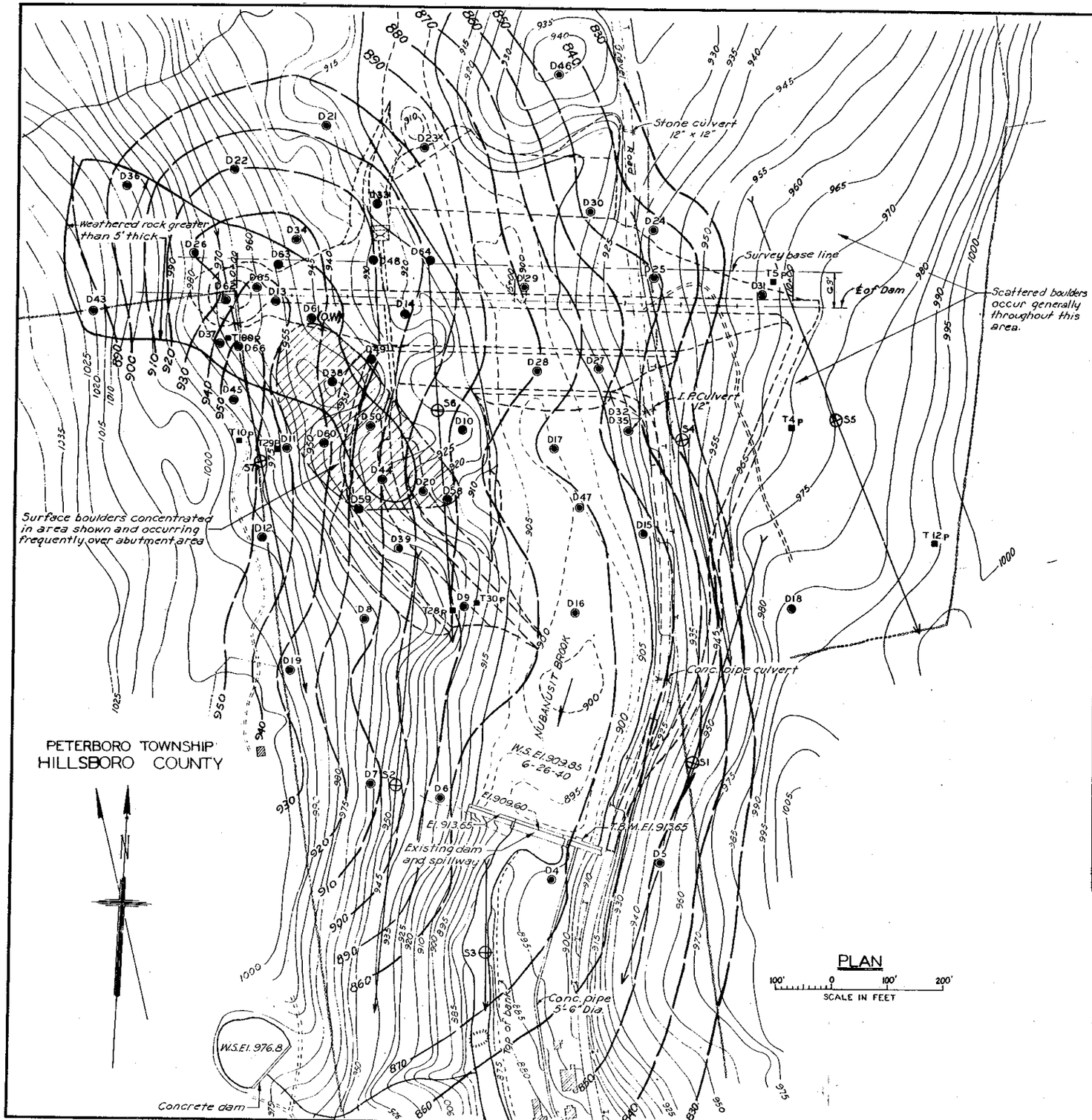
(5) Protective Dike.- A low protective dike approximately 2600 feet long is located downstream from the discharge channel to protect more valuable land west of the present drainage channel. The dike is constructed of material excavated from the approach and discharge channels and is founded on the original ground surface. The slopes of the dike are 1 on 2 and are considered satisfactory for this protective work. The south side of the dike is protected against rapid erosion by 3 feet of dumped rock. The remainder of the dike is protected against normal erosive action by 6 inches of seeded topsoil.

h. Construction Procedure.- (1) Sequence of Operations.- The design contemplates a two-season construction which will permit completion of the outlet works during the first season and which will provide sufficient time to construct the embankment in the second season during the relatively short construction period between normal spring floods and occurrence of freezing temperatures in the fall.

(2) Main Cofferdams.- The main upstream and downstream cofferdams are to be constructed with material obtained from excavation for cut-off trench and outlet structures. Badly fractured rock from outlet works excavation is used on the upstream side of the upstream cofferdam. Preparation of the foundation of the dam and construction of the embankment is to be performed in a dry condition. The upstream cofferdam is to be retained as part of the

completed structure at the end of the upstream impervious blanket. The downstream cofferdam is to be removed upon completion of the embankment.

(3) Embankment Construction.- Construction of the compacted earth embankment requires the simultaneous placement of the compacted impervious, random and pervious sections in parallel layers for full width of dam with the top sloped to drain readily at all times.



DRILL HOLE DATA

DRILL HOLE	ROCK ELEVATION	SURFACE ELEVATION	DRILL HOLE	ROCK ELEVATION	SURFACE ELEVATION	DRILL HOLE	ROCK ELEVATION	SURFACE ELEVATION
D 4	873.3	893.4	D 25	851.2	940.0	D 49	920.6	948.2
D 5	No rock	951.4	D 26	930.9	975.8	D 50	918.5	938.1
D 6	882.6	928.6	D 27	863.1	914.6	D 51		
D 7	914.8	958.3	D 28	881.9	901.1	D 52		
D 8	914.7	968.0	D 29	879.8	901.6	D 53		
D 9	888.1	930.8	D 30	898.1	916.3	D 54		
D 10	885.1	927.8	D 31	No rock	961.5	D 55		
D 11	943.8	967.0	D 32	859.3	922.6	D 56		
D 12	948.0	987.5	D 33	918.0	920.2	D 57		
D 13	942.1	956.2	D 34	933.3	945.7	D 58	896.2	921.8
D 14	910.3	918.5	D 35	* No rock	924.4	D 59	924.3	947.8
D 15	867.0	913.2	D 36	906.0	1001.0	D 60	934.2	944.8
D 16	876.2	897.3	D 37	947.6	972.6	D 61	933.4	949.7
D 17	867.2	898.0	D 38	923.9	958.9	D 62	930.4	967.8
D 18	No rock	986.8	D 39	908.6	941.4	D 63	937.3	955.3
D 19	925.1	992.9	D 43	887.0	1018.8	D 64	909.9	917.9
D 20	894.2	923.2	D 44	913.0	931.0	D 65	945.2	964.0
D 21	906.0	923.4	D 45	948.9	980.2	D 66	952.6	971.2
D 22	912.2	956.2	D 46	844.3	939.2			
D 23	897.2	906.2	D 47	Approx. 872.5	Approx. 900.5			
D 24	843.7	940.4	D 48	921.2	929.6			

For location of drill holes at spillway See Dwg. File No. M78-12/63 (Plate II-7)

* 6 inch hole to obtain undisturbed samples.

SEISMIC DATA

SHOT	ROCK ELEVATION	SURFACE ELEVATION
S1	852.3	946.3
2-4+20B	886.3	944.3
2-3+20B	897.5	960.5
2-2+20B	892.2	952.2
S2	902.1	948.0
2-1+20A	905.2	943.0
2-2+70A	894.1	939.0
2-3+70A	870.3	942.0
3-0+90B	863.9	879.0
S3	871.0	875.0
3-1+10A	864.1	872.0
4-3+20B	830.0	941.8
S4	831.0	942.6
4-3+70A	835.0	946.3
S5	Deeper than 135	976.0
6-2+20B	892.0	916.6
S6	890.0	925.7
6-2+20A	878.0	927.0
7-1+20B	953.0	980.5
S7	957.0	935.8
7-1+20A	958.0	987.5

INDEX FOR SUBSURFACE EXPLORATION

No. OF EXPLORATION	PURPOSE	PLAN PLATE No.	GRAPHIC LOG PLATE No.
D1 - D3	Foundation investigation at Spillway	II-7	II-8
D4 - D39	" " at Damsite	II-1	II-2 & II-3
D40 - D42	Pervious borrow investigation	II-11	II-11
D43 - D50	Foundation investigation at Damsite	II-1	II-3
D51 - D57	" " at Spillway	II-7	II-8
D58 - D66	" " at Damsite	II-1	II-3 & II-4
T1p - T6p	Foundation & Impervious Borrow investigation	II-11	II-11
T10p & T12p	" " " "	II-1	II-11
T28p - T30p	Foundation investigation at Damsite.	II-1	II-4
T45p - T54p	Pervious borrow investigation.	II-11	II-11
T63p & T64p	" " "	II-11	II-11
T67p	Foundation investigation at Spillway.	II-7	II-8
T100p	" " at Damsite.	II-1	II-4
T71p, D103 - D106	Impervious borrow investigation	II-11	II-4
D67 - D77	Foundation investigation at Spillway	II-7	II-8

LEGEND

- Earth contours.
- Rock contours.
- Outline of structures.
- B —⊕— A Seismic exploration.
- Rock outcrop.
- ◉ D38 Drill hole
- ■ T4p Test pit
- ◉ (O.W.) Observation well

Note:
Elevations are in feet and are based on U.S.C.&G.S. datum of mean sea level.

NO.	CHARACTER	DATE	APP'D
1	General Revisions	6/7/62	WJ

REVISIONS

MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
NUBANUSIT BROOK

PLAN OF FOUNDATION EXPLORATION AT DAM SITE

U. S. ENGINEER OFFICE, BOSTON, MASS. MAY, 1945

APPROVAL RECOMMENDED: John E. Allen, CHIEF ENGINEERING DIVISION

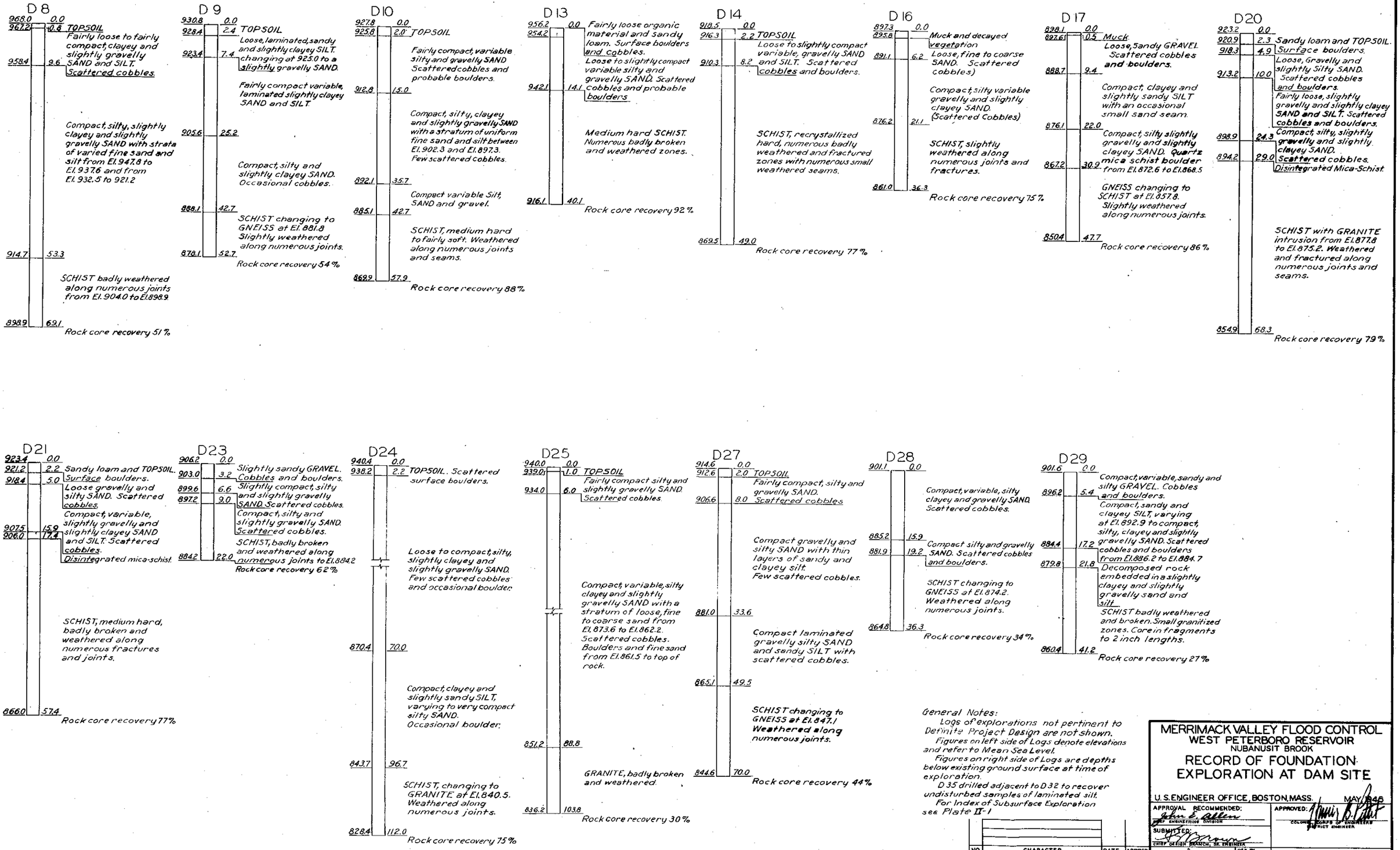
APPROVED: Paul B. Pratt, COLONEL, CORPS OF ENGINEERS, DISTRICT ENGINEER

SUBMITTED: CHIEF DESIGN BRANCH, SR. ENGINEER

DES. BY
DR. BY
TR. BY
CH. BY

FILE NO. M78-12/58

PLATE II-1



General Notes:

Logs of explorations not pertinent to Definite Project Design are not shown.

Figures on left side of Logs denote elevations and refer to Mean Sea Level.

Figures on right side of Logs are depths below existing ground surface at time of exploration.

D 35 drilled adjacent to D 32 to recover undisturbed samples of laminated silt.

For Index of Subsurface Exploration see Plate II-1

NO.	CHARACTER	DATE	APPR'D
	REVISIONS		

MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
NUBANUSIT BROOK

RECORD OF FOUNDATION EXPLORATION AT DAM SITE

U. S. ENGINEER OFFICE, BOSTON, MASS. MAY 1948

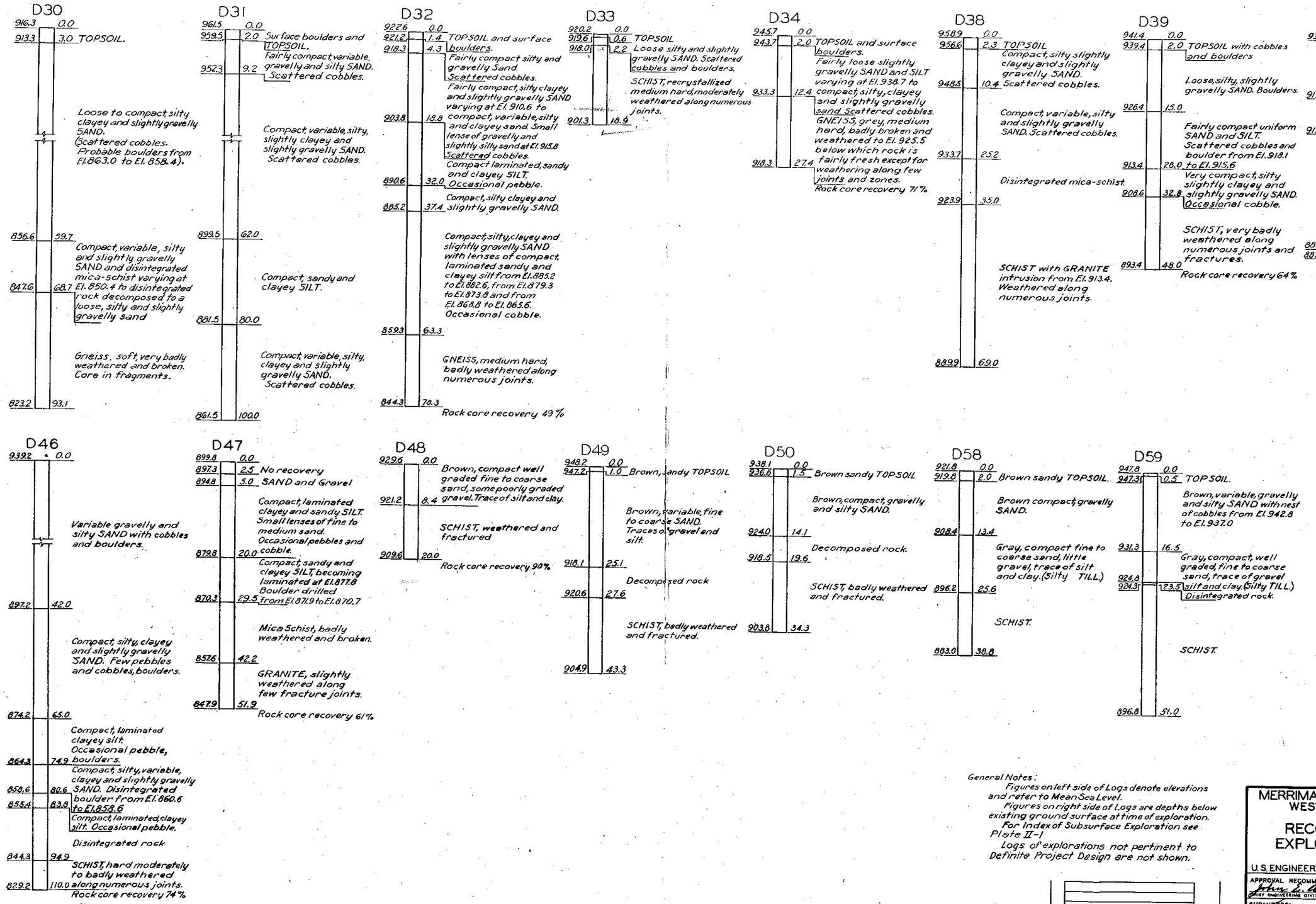
APPROVAL RECOMMENDED: John E. Allen, CHIEF ENGINEERING DIVISION

APPROVED: Francis J. Pratt, COLONEL, CORPS OF ENGINEERS, DISTRICT ENGINEER

SUBMITTED: W. L. Shannon, CHIEF DESIGN BRANCH, SR. ENGINEER

DES. BY W. L. Shannon; DR. BY; TR. BY; CH. BY

FILE NO. M 78-12/59



General Notes:

Figures on left side of Logs denote elevations and refer to Mean Sea Level.

Figures on right side of Logs are depths below existing ground surface at time of exploration.

For Index of Subsurface Exploration see Plate II-1

Logs of explorations not pertinent to Definite Project Design are not shown.

NO.	CHARACTER	DATE	APPR'D
	REVISIONS		

MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
NUBANUSIT BROOK

RECORD OF FOUNDATION EXPLORATION AT DAM SITE

U.S. ENGINEER OFFICE, BOSTON, MASS. MAY 1945

APPROVAL RECOMMENDED: John L. Allen
CHIEF ENGINEERING DIVISION

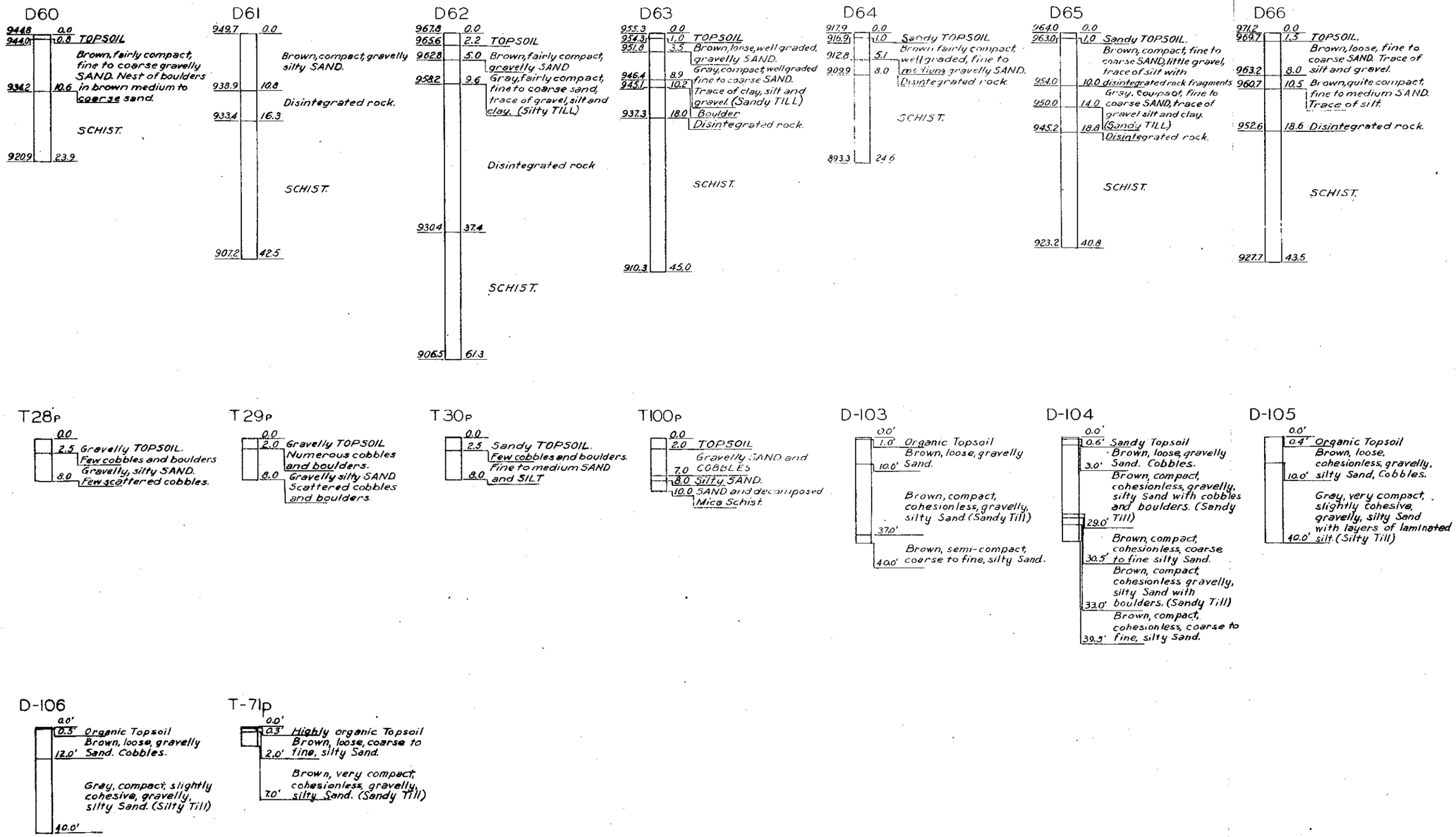
APPROVED: Thomas B. Pitt
COLONEL, CORPS OF ENGINEERS
DISTRICT ENGINEER

SUBMITTED: F. B. Brown
CHIEF DESIGN BRANCH, DR. ENGINEER

W. L. Shannon
ENGINEER

DES. BY
DR. BY
TR. BY
CH. BY

FILE NO. M 78-12/60



General Notes

Figures on left side of Logs denote elevations and refer to Mean Sea Level.

Figures on right side of Logs are depths below existing ground surface at time of exploration.

For Index of Subsurface Exploration see Plate II-1

NO.	CHARACTER	DATE	APPR'D
1	Logs added	5/7/46	[illegible]

REVISIONS

MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
NUBANUSIT BROOK

RECORD OF FOUNDATION EXPLORATION AT DAM SITE AND BORROW AREA

U.S. ENGINEER OFFICE, BOSTON, MASS. MAY 1945

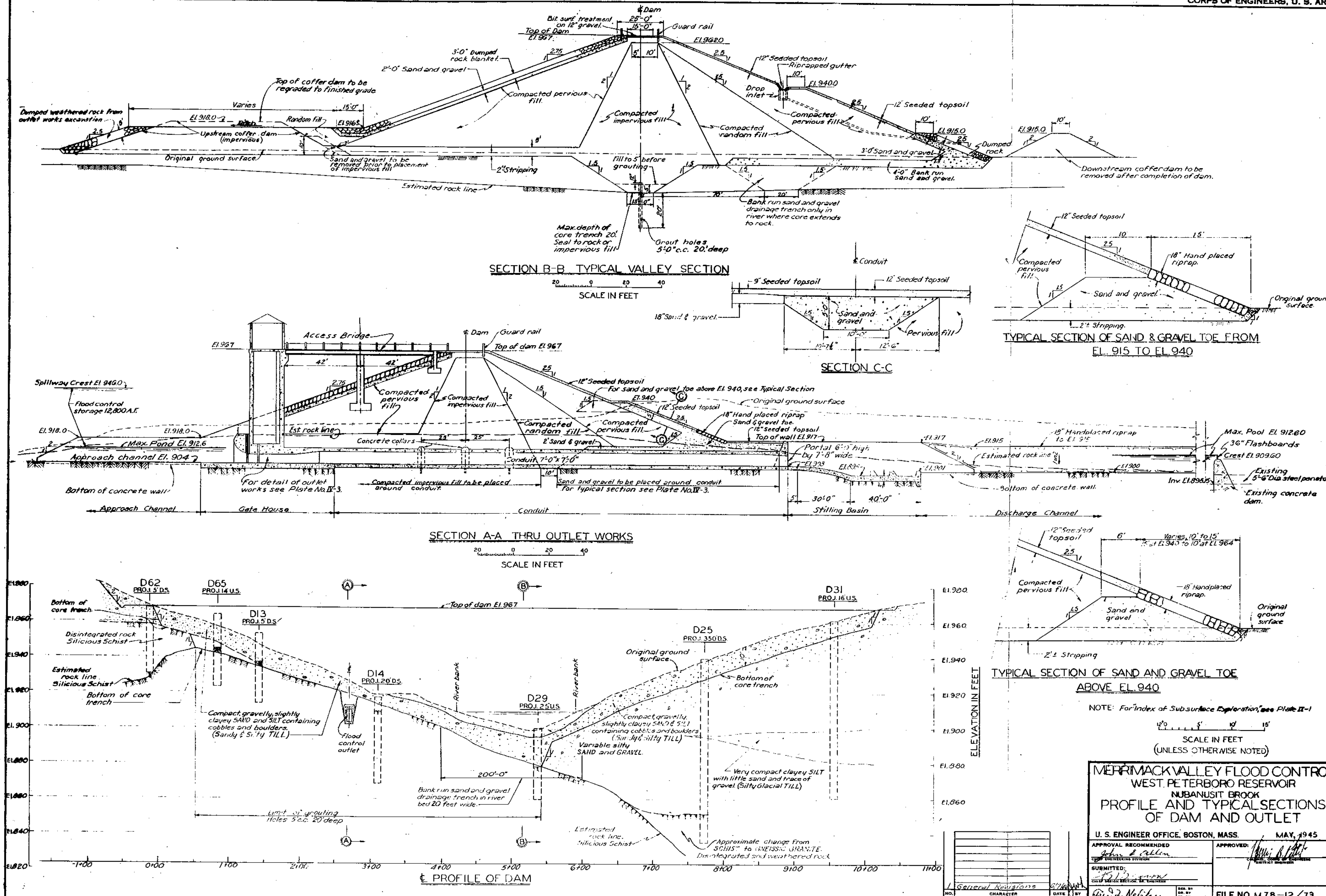
APPROVAL RECOMMENDED: John E. Allen, CHIEF ENGINEERING DIVISION

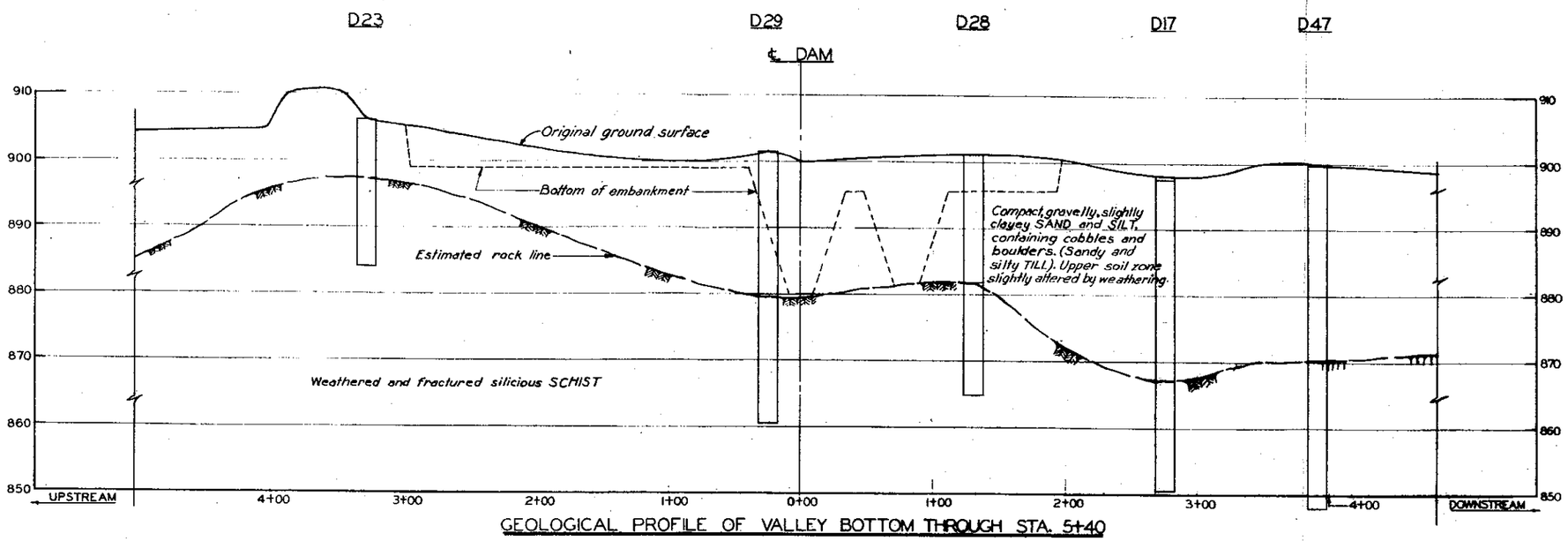
APPROVED: [signature], COLONEL, CORPS OF ENGINEERS, DISTRICT ENGINEER

SUBMITTED: [signature], CHIEF DESIGN BRANCH, SR. ENGINEER

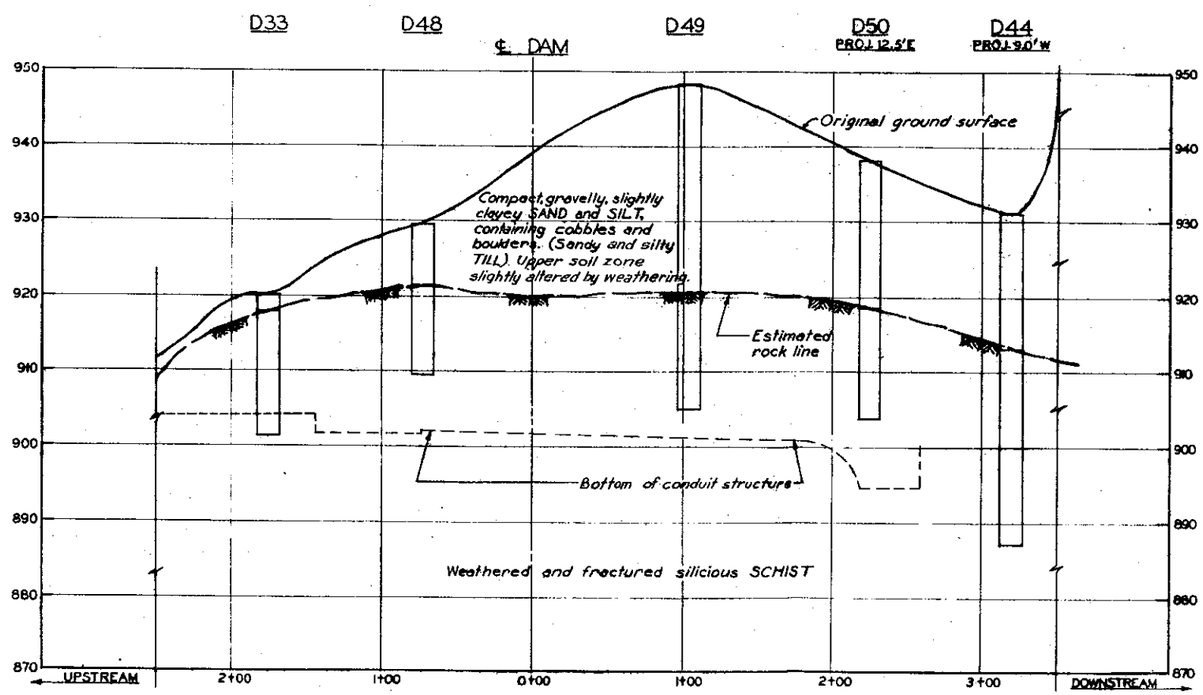
[signature], ENGINEER

FILE NO. M78-12/61


SECTION B-B TYPICAL VALLEY SECTION
SCALE IN FEET
¢ Dam
Bit. surf. treatment on 12" gravel.
25'-0"
15'-0"
Guard rail
Top of Dam El. 967.
El. 962.0
3'-0" Dumped rock blanket.
2'-0" Sand and gravel
12" Seeded topsoil
Riprapped gutter
El. 940.0
Drop inlet
Top of coffer dam to be regraded to finished grade
Varies
15'-0"
Dumped weathered rock from outlet works excavation
El. 918.0
Random fill
El. 916.5
Upstream coffer dam (impervious)
Compacted pervious fill.
Compacted impervious fill
Compacted random fill
Compacted pervious fill
12" Seeded topsoil
El. 915.0
Dumped rock
3'-0" Sand and gravel
Original ground surface
Sand and gravel to be removed prior to placement of impervious fill
2' Stripping
Fill to 5' before grouting
4'-0" Bank run sand and gravel.
Downstream cofferdam to be removed after completion of dam.
Estimated rock line
Bank run sand and gravel drainage trench only in river where core extends to rock.
Max. depth of core trench 20'. Seal to rock or impervious fill
Grout holes 5'-0" c.c. 20' deep
SECTION C-C
¢ Conduit
9" Seeded topsoil
12" Seeded topsoil
18" Sand & gravel
Sand and gravel
Pervious fill
10'-7½"
12'-6"
TYPICAL SECTION OF SAND & GRAVEL TOE FROM EL. 915 TO EL. 940
12" Seeded topsoil
18" Hand placed riprap.
Compacted pervious fill
Sand and gravel
Original ground surface
2'± Stripping
SECTION A-A THRU OUTLET WORKS
SCALE IN FEET
Access Bridge
El. 967
¢ Dam
Guard rail
Top of dam El. 967
Spillway Crest El. 946.0
Flood control storage 12,800 A.F.
El. 918.0
Max. Pond El. 912.6
Approach channel El. 904
Compacted pervious fill
Compacted impervious fill
12" Seeded topsoil
For sand and gravel toe above El. 940, see Typical Section
Original ground surface
El. 940
18" Hand placed riprap
Sand & gravel toe.
12" Seeded topsoil
Top of wall El. 917
Est rock line
Compacted random fill
Compacted pervious fill.
Concrete collars
2' Sand & gravel
Conduit 7'-0" x 7'-0"
Portal 6'-0" high by 7'-8" wide
18" Handplaced riprap to El. 915
Estimated rock line
Max. Pool El. 912.60
36" Flashboards
Crest El. 909.60
Existing 5'-6" Dia. steel penstock
Inv. El. 895.15
Existing concrete dam.
Bottom of concrete wall
For detail of outlet works see Plate No. IV-3.
Compacted impervious fill to be placed around conduit.
Sand and gravel to be placed around conduit. For typical section see Plate No. IV-3.
Bottom of concrete wall.
Approach Channel
Gate House
Conduit
Stilling Basin
Discharge Channel
30'-0"
40'-0"
TYPICAL SECTION OF SAND AND GRAVEL TOE ABOVE EL. 940
12" Seeded topsoil
6'
Varies 10' to 15' 15' at El. 940 to 10' at El. 964
Compacted pervious fill
18" Handplaced riprap.
Sand and gravel
Original ground surface
2'± Stripping
NOTE: For "Index of Subsurface Exploration," see Plate II-1
SCALE IN FEET (UNLESS OTHERWISE NOTED)
¢ PROFILE OF DAM
D62 PROJ. 5' D.S.
D65 PROJ. 14' U.S.
D13 PROJ. 5' D.S.
D14 PROJ. 20' D.S.
D29 PROJ. 25' U.S.
D25 PROJ. 350' D.S.
D31 PROJ. 16' U.S.
Top of dam El. 967
Bottom of core trench
Disintegrated rock Silicious Schist
Estimated rock line Silicious Schist
Compact, gravelly, slightly clayey SAND and SILT containing cobbles and boulders. (Sandy & Silty TILL)
Flood control outlet
River bank
Original ground surface
Bottom of core trench
Variable silty SAND and GRAVEL.
Very compact clayey SILT with little sand and trace of gravel. (Silty Glacial TILL)
200'-0"
Bank run sand and gravel drainage trench in river bed 20 feet wide.
Limit of grouting Holes 5' c.c. 20' deep
Estimated rock line. Silicious Schist
Approximate change from SCHIST to GNEISSIC GRANITE. Disintegrated and weathered rock
ELEVATION IN FEET
-1+00 0+00 1+00 2+00 3+00 4+00 5+00 6+00 7+00 8+00 9+00 10+00 11+00
General Revisions
REVISIONS
MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
NUBANUSIT BROOK
PROFILE AND TYPICAL SECTIONS OF DAM AND OUTLET
U. S. ENGINEER OFFICE, BOSTON, MASS. MAY, 1945
APPROVAL RECOMMENDED
SUBMITTED:
APPROVED:
FILE NO. M78-12/73



GEOLOGICAL PROFILE OF VALLEY BOTTOM THROUGH STA. 5+40



GEOLOGICAL PROFILE AT STA 2+69.5
(APPROXIMATE LOCATION OF CONDUIT)

NOTE:
For "Index of Subsurface Exploration" see Plate II-1
Elevations are in feet above Mean Sea Level.

NO.	CHARACTER	DATE	APPR'D
1	General Revisions	6/7/46	[illegible]

REVISIONS

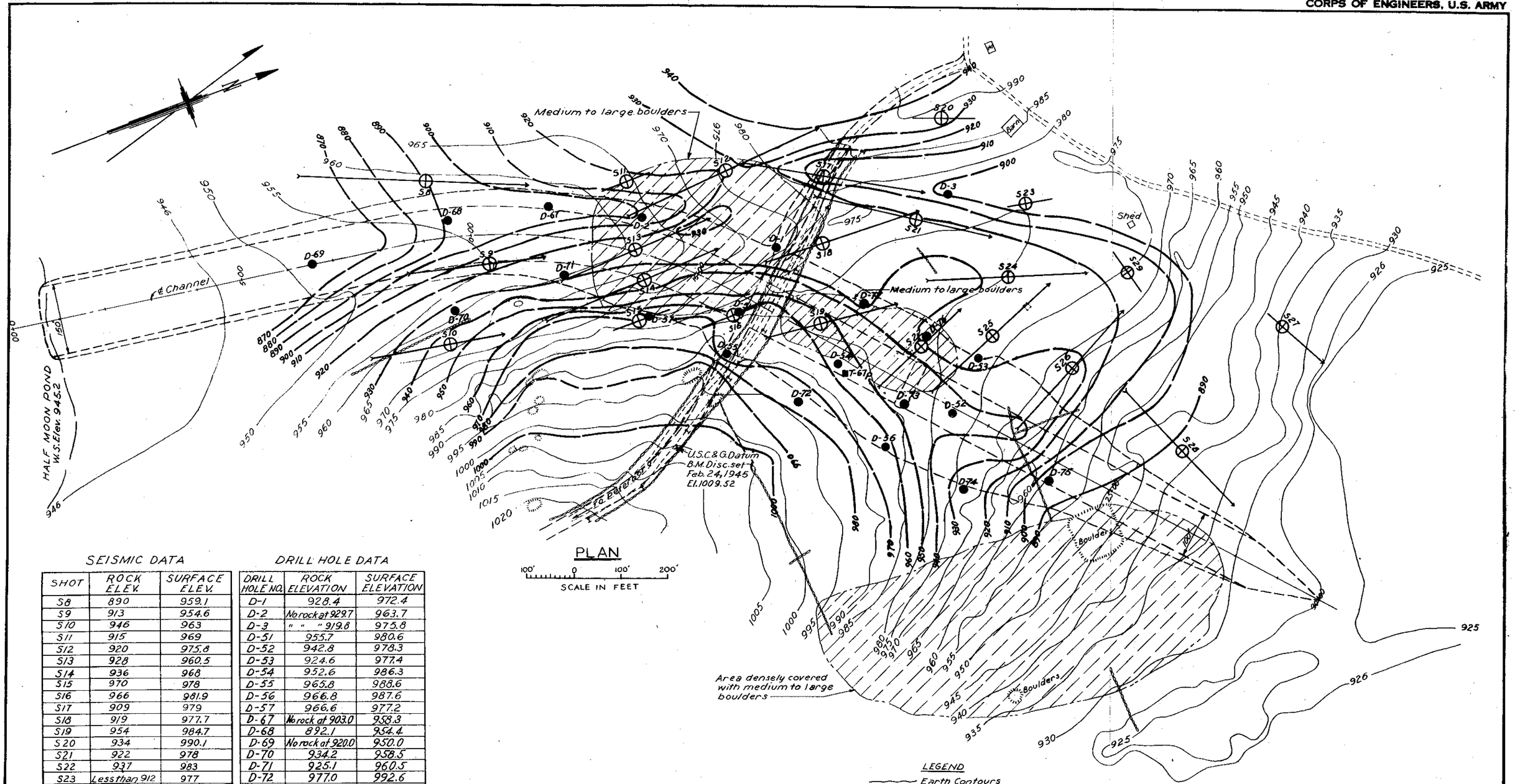
MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
NUBANUSIT BROOK

GEOLOGICAL PROFILES
AT DAM SITE

U.S. ENGINEER OFFICE, BOSTON, MASS. MAY 19-5

APPROVAL RECOMMENDED: John E. Allen, CHIEF ENGINEERING DIVISION
SUBMITTED: [signature], CHIEF DESIGN BRANCH, SR. ENGINEER
APPROVED: [signature], COLONEL, CORPS OF ENGINEERS, DISTRICT ENGINEER
W.L. Shannon, ENGINEER
DES. BY / DR. BY / TR. BY / CK. BY

FILE NO. M 78-12/62



SEISMIC DATA

SHOT	ROCK ELEV.	SURFACE ELEV.
S8	890	959.1
S9	913	954.6
S10	946	963
S11	915	969
S12	920	975.8
S13	928	960.5
S14	936	968
S15	970	978
S16	966	981.9
S17	909	979
S18	919	977.7
S19	954	984.7
S20	934	990.1
S21	922	978
S22	937	983
S23	Less than 912	977
S24	927	979.2
S25	943	977.8
S26	932	969.6
S27	853 at A	931.5
S28	888	942
S29	Less than 928	975

DRILL HOLE DATA

DRILL HOLE NO.	ROCK ELEVATION	SURFACE ELEVATION
D-1	928.4	972.4
D-2	No rock at 929.7	963.7
D-3	" " " 919.8	975.8
D-51	955.7	980.6
D-52	942.8	978.3
D-53	924.6	977.4
D-54	952.6	986.3
D-55	965.8	988.6
D-56	966.8	987.6
D-57	966.6	977.2
D-67	No rock at 903.0	958.3
D-68	892.1	954.4
D-69	No rock at 920.0	950.0
D-70	934.2	958.5
D-71	925.1	960.5
D-72	977.0	992.6
D-73	946.9	983.8
D-74	924.4	974.5
D-75	894.6	959.9
D-76	921.8	975.5
D-77	928.2	981.0

Note:
Elevations are in feet and are based on U.S.C.&G.S. datum of mean sea level.
For "Index of Subsurface Exploration" see PLATE II-1

LEGEND

- Earth Contours
- Rock Contours
- Outline of Structures
- B → A Seismic Exploration
- Rock Outcrop
- Boulder Area
- ● D-53 Drill Hole
- ■ T-67p Test Pit

NO.	CHARACTER	DATE	APPR'D
	REVISIONS		

MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
PLAN OF FOUNDATION
EXPLORATION AT SPILLWAY

U. S. ENGINEER OFFICE, BOSTON, MASS. JUNE 1946

APPROVAL RECOMMENDED: John E. Allen, CHIEF ENGINEERING DIVISION

SUBMITTED: John W. Fuslie, CHIEF DESIGN BRANCH, SR. ENGINEER

W. L. Shannon, SR. ENGINEER

APPROVED: COLONEL, CORPS OF ENGINEERS, DISTRICT ENGINEER

FILE NO. M78-12/63 A

THIS SHEET SUPERSEDES SHEET NO. M78-12/63

PLATE II-7

D-1

Elev.	Depth	Description
El. 972.4	0.0'	
El. 969.4	3.0'	Topsoil, cobbles and boulders
El. 967.0	5.4'	Fairly compact, sandy slightly silty Gravel.
El. 952.4	20.0'	Compact, variable, gravelly silty Sand.
El. 943.9	28.5'	Cobbles and boulders
		Boulders
El. 928.4	44.0'	Compact, variable gravelly silty Sand. Cobbles and boulders
El. 911.9	60.5'	Granite-Gneiss slightly weathered along numerous joints.

D-2

Elev.	Depth	Description
El. 963.7	0.0'	
El. 961.7	2.0'	Topsoil, vegetation and boulders.
El. 949.3	14.4'	Compact, variable gravelly Sand and Silt Cobbles.
		Boulders imbedded in gravelly Sand and Silt.
El. 933.7	30.0'	
El. 929.7	34.0'	Compact gravelly Sand and Silt.

D-3

Elev.	Depth	Description
El. 975.8	0.0'	
El. 972.8	3.0'	Topsoil and loam
El. 965.8	10.0'	Compact gravelly Sand and Silt. Boulders.
El. 957.8	18.0'	Compact, sandy, silty Gravel. Numerous boulders.
		Very compact, variable gravelly Sand and Silt. Scattered cobbles and boulders.
El. 919.8	56.0'	

D-51

Elev.	Depth	Description
El. 980.6	0.0'	
El. 980.1	0.5'	Topsoil
El. 955.7	24.9'	Brown variable density from loose to very compact, poorly graded fine to coarse Sand, some gravel with trace of clay and silt.
El. 941.8	38.8'	Very coarse grained Porphyritic Granite-badly fractured. Fracture vanes are weathered and dip from horizontal to 45°. Pieces of core range from a ground mass to 3" in length. Quartz and Feldspar are the predominant minerals. Rock very hard. 70 % Recovery.

D-52

Elev.	Depth	Description
El. 978.3	0.0'	
El. 942.8	35.5'	Dark gray, loose above 10.0' to compact well graded fine to coarse Sand. Little gravel with trace of clay and silt. Boulders encountered at 6.5' and cobbles from 30.0' to 35.0' which appear as crushed rock at that depth, overlaying bedrock.
El. 928.0	50.3'	Coarse grained Porphyritic Granite showing unweathered horizontal fracturing throughout. Pieces of core were recovered in lengths from 1" to 1.5' and contain Biotite, Quartz, Feldspar and Hornblende. 96 % Recovery.

D-53

Elev.	Depth	Description
El. 977.4	0.0'	
El. 975.9	1.5'	Topsoil
El. 956.4	21.0'	Brown, becoming gray 5.0', fairly compact well graded, fine to coarse Sand. Trace of gravel, silt and clay.
El. 952.9	24.5'	Boulder of Porphyritic Granite.
El. 924.6	52.8'	Dark gray, variable density, fairly well graded, fine to coarse Sand, becoming compact at 48.0' with increased gravel content
El. 914.6	62.8'	Coarse grained Porphyritic Granite fractured throughout at about 2" intervals on horizontal plane. Consists of Biotite, Feldspar, Quartz and Hornblende 100 % Recovery

D-54

Elev.	Depth	Description
El. 986.3	0.0'	
El. 984.8	1.5'	Topsoil
El. 976.3	10.0'	Brown, slightly compact to compact fairly well graded, fine to coarse Sand. Trace of gravel and silt.
El. 952.6	33.7'	Gray, fairly compact, well graded, fine to coarse Sand, little gravel. Trace of clay and silt.
El. 940.5	45.8'	Coarse grained Porphyritic Granite. Core recovered in lengths of 1½" to 2" which was fractured on horizontal planes by drilling action. Rock unweathered. Constituent minerals, Quartz, Biotite Mica, Feldspar and Hornblende. 86 % Recovery.

D-55

Elev.	Depth	Description
El. 988.6	0.0'	
El. 987.1	1.5'	Sandy topsoil
El. 965.8	22.8'	Brown, changing to gray at 12.0', fairly compact well graded, fine to coarse Sand. Trace of silt, clay and gravel.
El. 953.3	35.3'	Coarse grained Porphyritic Granite, densely impregnated with Muscovite Mica, particularly along fracture planes. Large phenocrysts of Feldspar and Quartz. Rock shows only slight weathering and fractures are generally horizontal. 85 % Recovery

D-56

Elev.	Depth	Description
El. 987.6	0.0'	
El. 986.6	1.0'	Topsoil
El. 966.8	20.8'	Gray, variable density well graded fine to coarse Sand with trace of silt clay and gravel.
El. 954.0	33.6'	Coarse grained Granite Porphyry. Core extracted in lengths of 1.0' to 2.0', fresh and hard. Slight weathering, fracturing horizontal. 97.5 % Recovery

D-57

Elev.	Depth	Description
El. 977.2	0.0'	
El. 975.7	1.5'	Topsoil
El. 965.6	11.6'	Brown loose becoming very compact at 10.6', fairly well graded fine to coarse Sand. Little gravel, trace of clay and silt
El. 953.7	23.5'	Coarse grained Porphyritic Granite. Core fractured throughout in variable lengths from ground mass to 6" caused by drill action. Very slightly weathered. Fracturing on horizontal plane. 97 % Recovery.

D-67

Elev.	Depth	Description
El. 958.3	0.0	
El. 957.5	0.8	Highly organic Topsoil
El. 952.7		Brown-gray, loose, cohesionless, medium to fine, silty, Sand.
El. 941.2	5.6	Light brown, compact, gravelly, silty, Sand.
	11.1	Grey, compact, slightly cohesive, gravelly, silty, Sand. (Silty Till) Boulders drilled from El. 943.5 to El. 941.1, El. 929.5 to El. 926.9, El. 921.9 to El. 920.8, El. 918.8 to El. 918.3 and El. 917.8 to El. 915.9
El. 903.0	55.3	

D-68

Elev.	Depth	Description
El. 954.4	0.0'	
El. 953.4	1.0'	Organic Topsoil
El. 949.6	4.8'	Brown, loose, gravelly Sand.
El. 934.8	19.6'	Brown, grey, compact, slightly cohesive, gravelly, silty Sand (Silty Till) Boulder encountered from El. 938.4 to El. 937.9 and El. 936.4 to El. 936.2.
El. 917.4	37.0'	Gray, compact, gravelly Sand. Boulder drilled from: El. 930.6 to El. 929.1.
El. 892.1	62.3'	Grey, compact, slightly cohesive gravelly, silty Sand. (Silty Till) Boulders drilled from El. 915.2 to El. 914.6 and El. 899.8 to El. 897.3.
El. 876.9	77.5'	Porphyritic Granite-weathered and fractured to contact with micaceous garnet Schist at El. 889.6. Schist only slightly weathered to El. 880.4 Porphyritic Granite from El. 880.4 (74.0) weathered and fractured. 80 % Recovery

D-69

Elev.	Depth	Description
El. 950.0	0.0'	
El. 948.3	1.7'	Topsoil
El. 941.0	9.0'	Brown, loose, cohesionless, gravelly, silty, Sand-with cobbles.
El. 939.7		Brown semi-compact, cohesionless, gravelly silty Sand. (Sandy Till)
El. 932.7	17.3'	Gray, semi-compact slightly cohesive, gravelly silty Sand-Cobbles. (Silty Till)
El. 920.0	30.0'	Gray very compact cohesionless gravelly Sand. (Boulder drilled from El. 566.0 to El. 564.3)

D-70

Elev.	Depth	Description
El. 958.5	0.0'	
El. 957.5	1.0'	Organic Topsoil
El. 954.6	3.9'	Brown, loose, cohesionless, coarse to fine, silty Sand.
El. 951.2	7.3'	Brown, loose, gravelly Sand-Cobbles.
El. 942.0	16.5'	Brown, compact, gravelly, Sand-with lenses of laminated silt-cobbles.
El. 940.1		
El. 934.2	18.4'	Gray, semi-compact, cohesionless, gravelly, silty Sand.
El. 918.8	24.3'	Gray, very compact, slightly cohesive, gravelly silty, Sand. (Silty Till)
	39.7'	Porphyritic Granite 94% Recovery

D-71

Elev.	Depth	Description
El. 960.5	0.0'	
El. 959.8	0.7'	Topsoil
El. 957.8		Brown, loose, cohesionless, coarse to fine, silty Sand-Cobbles.
El. 964.0	2.7'	Brown, loose, gravelly Sand Cobbles.
	14.5'	Gray, semi-compact, slightly cohesive, gravelly, silty, Sand. (Silty Till)
El. 925.1	35.4'	Porphyritic Granite 78.3 % Recovery
El. 910.4	50.1'	

D-72

Elev.	Depth	Description
El. 992.6	0.0'	
El. 991.0	1.6'	Organic Topsoil Boulder Area
El. 977.0	15.6'	Brown-gray, compact, slightly cohesive, gravelly silty Sand-with cobbles and boulders. (Silty Till)
El. 932.6	60.0'	Porphyritic Granite Unweathered with network of talcosed interstices 90 % Recovery

D-73

Elev.	Depth	Description
El. 983.8	0.0'	
El. 982.2	1.6'	Sandy Topsoil
		Brown-gray. slightly cohesive, gravelly, silty Sand (Silty Till) Boulders encountered at El. 975.6 and El. 974.3.
El. 946.9	36.9'	Small boulder encountered at El. 954.8.
El. 928.8	55.0'	Porphyritic Granite coarse grained and unweathered with extrusive contact of granite Gneiss. Fracturing along calsite filled planes of schistosity. 97% Recovery

D-74

Elev.	Depth	Description
El. 974.5	0.0'	
El. 973.7	0.8'	Sandy Topsoil
El. 972.5		Light brown, semi-compact cohesionless, silty, Sand.
	2.0'	
		Brown-gray, compact, slightly cohesive, gravelly, silty Sand-with boulders. (Silty Till)
El. 924.4	50.1'	Porphyritic Granite. Fresh and unweathered. Fracturing along natural interstices. 93% Recovery.
El. 909.4	65.1'	

D-75

Elev.	Depth	Description
El. 959.9	0.0'	
El. 959.2	0.7'	Topsoil
		Brown-gray, compact, slightly cohesive, gravelly, silty, Sand-with cobbles and boulders. (Silty Till)
		(Boulder drilled from El. 943.3 to El. 940.1)
El. 894.6	65.3'	(Ground water at Sur. El. 17 May 45)
El. 879.2	80.7'	Granitic Gneiss Slightly weathered Mechanical fracturing along interstic planes. 80% Recovery.

D-76

Elev.	Depth	Description
El. 975.5	0.0'	
El. 974.7	0.8'	Organic Topsoil (Boulder Area)
El. 970.5	5.0'	Brown-gray, loose, gravelly silty Sand.
El. 968.3	7.2'	Brown-gray, loose, cohesionless, gravelly, silty Sand-With boulders. (Sandy Till)
El. 921.8	53.7'	Gray, compact, slightly cohesive, gravelly, silty Sand. (Silty Till)
El. 906.8	68.7'	Porphyritic Granite coarse grained, unweathered. Horizontal mechanical fracturing. 92% Recovery

D-77

Elev.	Depth	Description
El. 981.0	0.0'	
El. 980.0	1.0'	Organic Topsoil
El. 977.5	3.5'	Brown loose Sand and Gravel.
El. 928.2	52.8'	Brown-gray, compact, slighty cohesive, gravelly, silty, Sand. (Silty Till) (Boulder drilled from El. 969.9 to El. 965.1) (Ground water El. 973.5 - 30 May 45)
El. 914.3	66.7'	Micaceous Schist and Porphyritic Granitic. Contact from El. 928.2 to El. 924.6 (Schist) From El. 924.6 to El. 914.3 Porphyritic Granite-feldspathic with vertical natural and horizontal mechanical fracturing. 85% Recovery.

T-67p

Elev.	Depth	Description
	0.0'	
	1.5'	Topsoil
		Compact, dry, fine to coarse Sand. Little gravel, trace of silt.
	3.5'	Compact, fine to medium Sand and Silt, trace of gravel. (Glacial Till)
	5.5'	

General Notes:

Logs of explorations not pertinent to Definite Project Design are not shown.

Figures on left side of logs denote elevations and refer to Mean Sea Level.

Figures on right side of logs are depths below existing ground surface at time of exploration.

For "Index of Subsurface Exploration" see Plate II-1

NO.	CHARACTER	DATE	APPR'D
1	Log Numbers Revised	6/7/64	

REVISIONS

MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR

RECORD OF FOUNDATION EXPLORATION AT SPILLWAY

U. S. ENGINEER OFFICE, BOSTON, MASS. MAY 1943

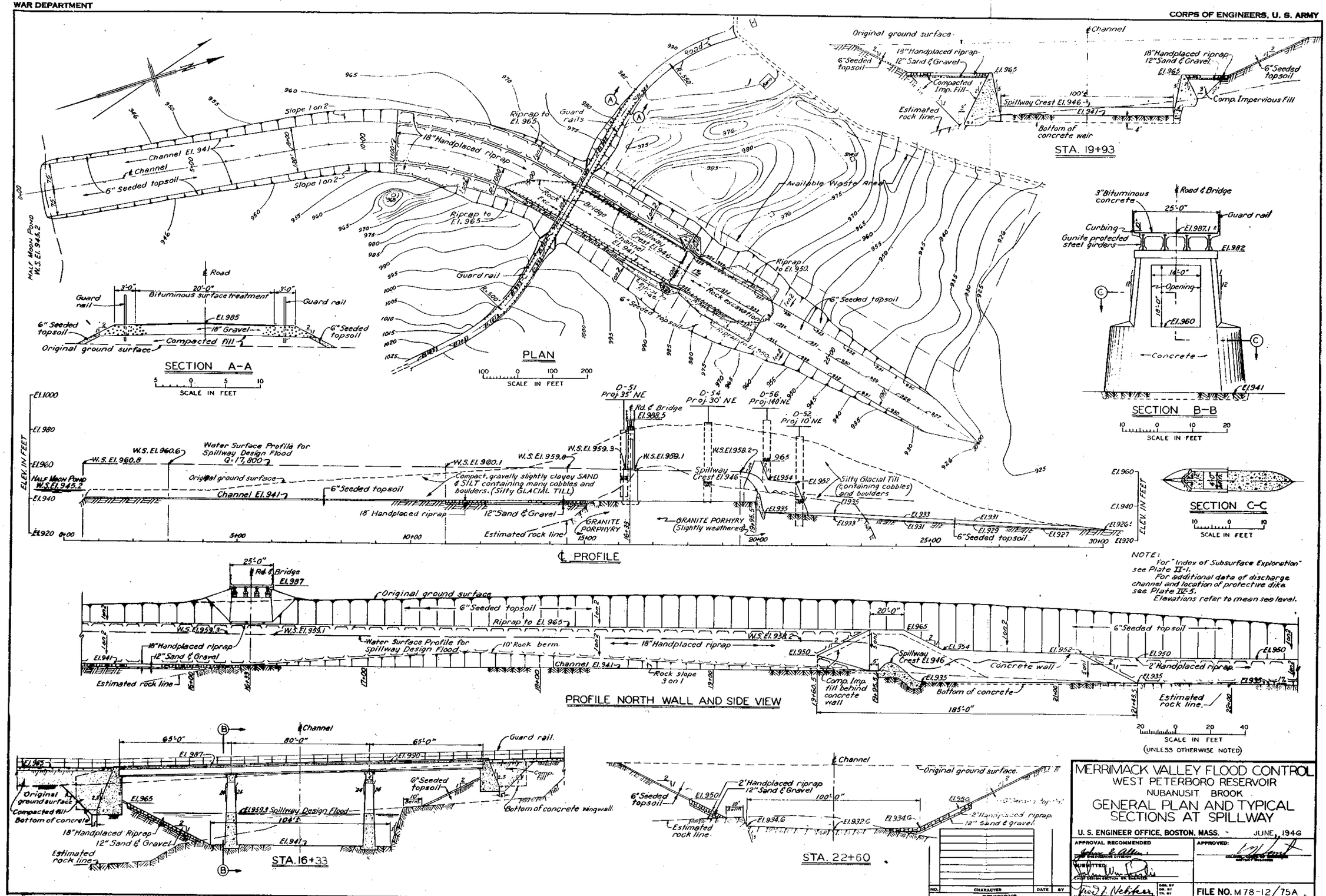
APPROVAL RECOMMENDED: CHIEF ENGINEERING DIVISION

APPROVED: COLONEL, CORPS OF ENGINEERS, DISTRICT ENGINEER

SUBMITTED: CHIEF DESIGN BRANCH, SR. ENGINEER

ENGINEER — DRN. BY, DES. BY, TR. BY, CH. BY

FILE NO. M 78-12/64



NOTE:
For "Index of Subsurface Exploration" see Plate II-1.
For additional data of discharge channel and location of protective dike see Plate IV-5.
Elevations refer to mean sea level.

(UNLESS OTHERWISE NOTED)

MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
NUBANUSIT BROOK
GENERAL PLAN AND TYPICAL SECTIONS AT SPILLWAY

U. S. ENGINEER OFFICE, BOSTON, MASS. JUNE, 1946

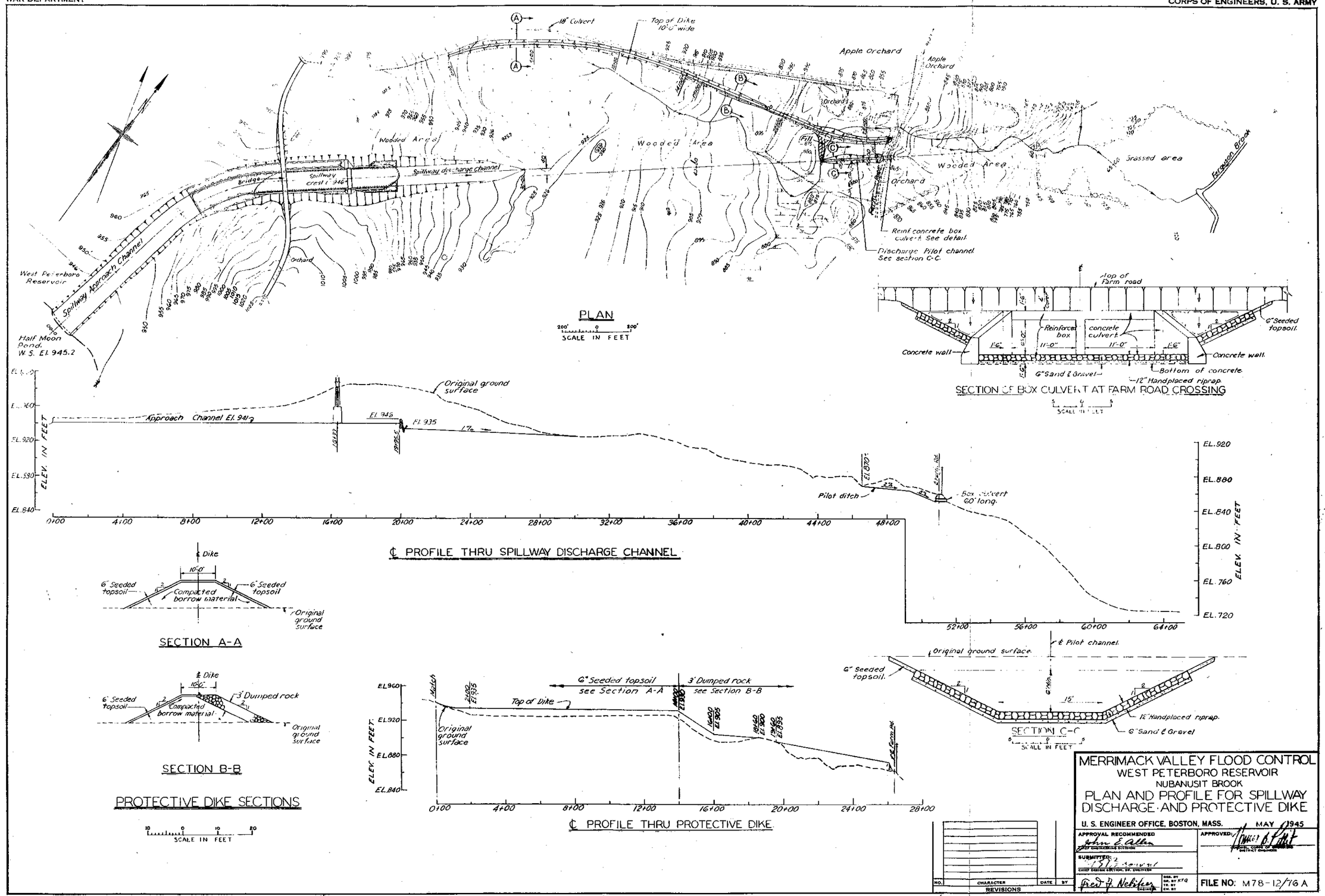
APPROVAL RECOMMENDED: Chief, Engineering Division
SUBMITTED: Chief, Design Section, Ex. Engineer
APPROVED: Colonel, Corps of Engineers, District Engineer

NO.	CHARACTER	DATE	BY
REVISIONS			

DES. BY
DR. BY
TR. BY
CH. BY

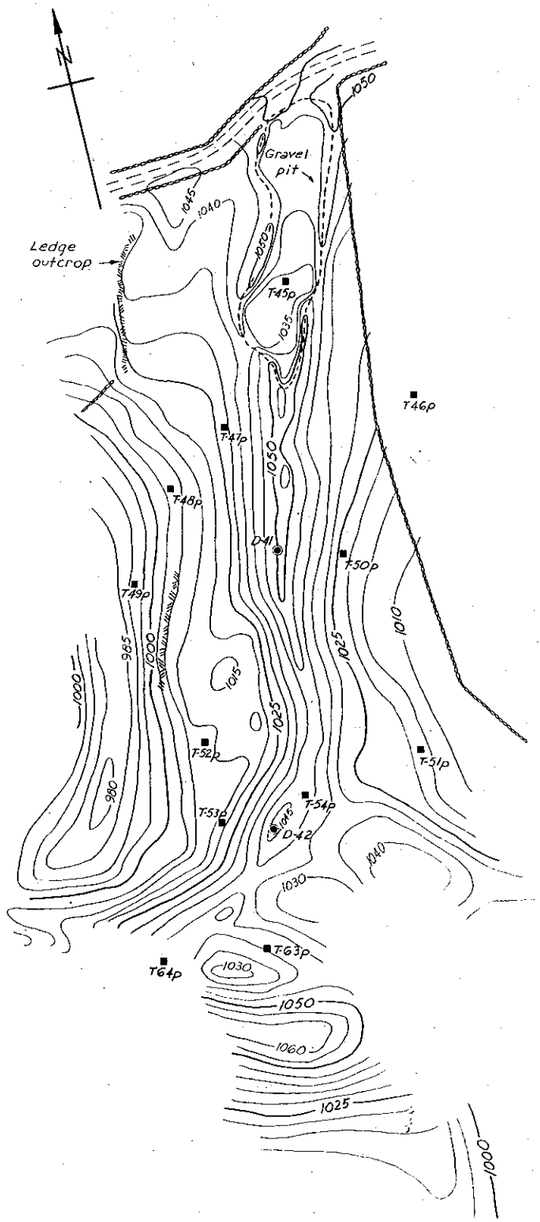
FILE NO. M 78-12/75A

THIS SHEET SUPERSEDES SHEET NO. M 78-12/75



THIS SHEET SUPERSEDES SHEET NO. M78-12/76

PLATE II-10



DETAIL PLAN OF PERVIOUS BORROW AREA

100' 0 100' 200'

SCALE IN FEET

D-40

0.0'	
2.0'	Topsoil
	Loose, variably gravelly Sand.
48.5'	
	Loose, slightly gravelly Sand. Small stratum of slightly silty uniform Sand at 76.0'
82.0'	
85.0'	Loose to slightly compact, slightly silty and slightly gravelly Sand.
100.0'	Loose coarse Sand varying at 90.0' to slightly compact silty Sand.

D-41

1049.3	0.0'	
1046.9	2.4'	Topsoil
1041.8	7.5'	Loose, gravelly sand.
1036.8	12.5'	Loose, uniform sand.
		Fairly compact gravelly sand varying at 30' to gravelly and silty sand. (Scattered cobbles.)
1018.2	31.1'	
1015.4	33.9'	Schist hard, slightly weathered along numerous joints
1008.2	41.1'	Gneiss hard, slightly weathered along few joints.
		Rock core recovery 92%

D-42

1047.8	0.0'	
1045.4	2.4'	Sandy Topsoil.
1040.8	7.0'	Loose gravelly sand.
1032.8	15.0'	Loose sandy gravel.
		Loose, gravelly sand varying at 17.5 to sandy gravel (Occasional cobble)
1020.8	27.0'	
1017.8	30.0'	Fairly compact gravelly sand
		Fairly compact sandy gravel changing at 41.9' to a gravelly sand. (Boulders from 32.0' to 34.0')
1004.5	43.3'	
994.5	53.3'	Schist, quartz, mica slightly mineralized Hard, badly weathered and fractured at 53.3'
		Rock core recovery 55.0%

T-1p

0.0'	
1.5'	Loose, brown topsoil and sandy loam
6.0'	Compact, silty, slightly clayey and slightly gravelly Sand. (Few scattered cobbles)

T-2t

0.0'	
1.0'	Topsoil and sandy loam. (Few cobbles and boulders.)
6.0'	Compact, silty, slightly clayey and slightly gravelly Sand. (Few scattered cobbles.)

T-3p

0.0'	
1.5'	Loose, brown topsoil and sandy loam. (Few cobbles and boulders.
6.0'	Loose to compact silty and gravelly Sand. (Few scattered cobbles.)

T-4p

0.0'	
1.5'	Loose, brown topsoil and sandy loam. Few cobbles and boulders.
6.0'	Ground water at 1.0' Fairly loose, slightly silty Sand and Gravel varying to gravelly and silty Sand (Few cobbles and boulders.)

T-5p

0.0'	
1.5'	Loose, brown topsoil and sandy loam.
6.0'	Numerous cobbles and boulders. Ground water at 1.5 Fairly loose, gravelly and silty Sand varying to fairly compact, slightly clayey and slightly gravelly Sand and silt. (Few cobbles and boulders.)

T-6p

0.0'	
1.5'	Loose, brown topsoil and sandy loam.
6.0'	Loose, slightly gravelly Sand and Silt varying to fairly loose gravelly and silty Sand. Ground water at 5.0' (Few cobbles and boulders from 2.4 to bottom of hole.)

T-10p

0.0'	
2.0'	Loose topsoil and sandy loam with few cobbles and boulders.
6.0'	Fairly loose, gravelly and silty Sand with a stratum of very compact, slightly clayey and slightly gravelly Sand and Silt from 3.8' to 4.7' (Few scattered cobbles.)

T-12p

0.0'	
2.0'	Loose topsoil and sandy loam
6.0'	Loose, slightly silty Sand and Gravel. (Numerous cobbles and boulders to 5.5') Ground water at 5.0'

T-45p

0.0'	
4.0'	Loose sandy gravel (Numerous cobbles and boulders) Ground water at 3.5' Probable ledge at 4.0'

T-46p

0.0'	
1.5'	Loose topsoil and sandy loam.
3.5'	Fairly loose, gravelly sand varying at 2.4' to silty, gravelly and slightly clayey sand. (Few cobbles and boulders.) Probable ledge at 3.5'

T-47p

0.0'	
1.8'	Loose topsoil, sandy loam and gravel. Few cobbles and boulders.
6.0'	Loose variable, sandy gravel with a stratum of gravelly sand from 2.4' to 3.1' (Few scattered cobbles)

T-48p

0.0'	
2.0'	Loose topsoil and sandy loam
6.0'	(Few cobbles and boulders.) Loose, variable, sandy gravel. (Few scattered cobbles and boulders

T-49p

0.0'	
2.5'	Topsoil and sandy loam.
6.0'	Few cobbles and boulders Loose sand and gravel. Cobbles and large boulders towards bottom of hole.

T-50p

0.0'	
2.0'	Loose topsoil and sandy loam.
6.0'	Loose gravelly sand varying at 3.5' to sand and gravel with a few cobbles and boulders.

T-51p

0.0'	
1.5'	Loose topsoil and sandy loam.
6.0'	Loose, gravelly and slightly silty sand varying at 4.5' to sand and gravel. (Few scattered cobbles.)

T-52p

0.0'	
2.5'	Fairly loose topsoil and sandy loam.
8.5'	Loose sandy gravel with a few cobbles varying at 4.5' to loose gravelly sand.

T-53p

0.0'	
1.5'	Loose topsoil and sandy loam.
6.0'	Loose, variable, sand and gravel with a stratum of fairly loose, slightly silty, fine sand from 2.6' to 3.1'. (Few scattered cobbles and boulders)

T-54p

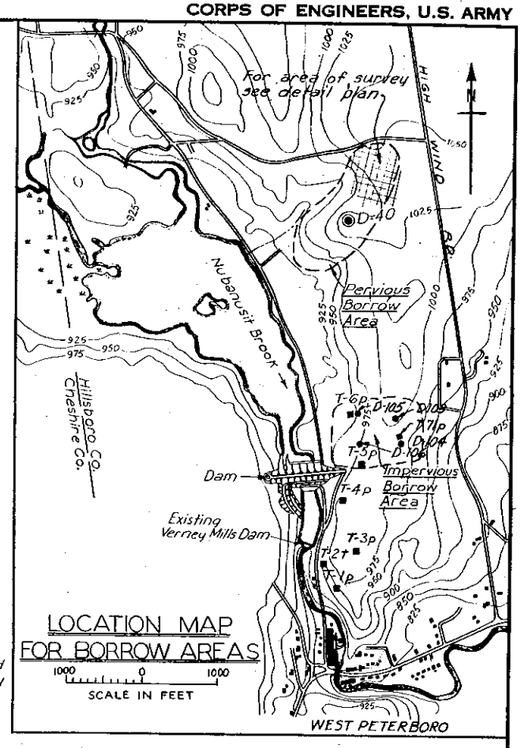
0.0'	
1.0'	Loose topsoil and sandy loam
6.0'	with a few cobbles. Loose sand and gravel. (Few scattered cobbles.)

T-63p

0.0'	
1.0'	Loose topsoil, sandy loam and
8.0'	few cobbles. Loose variable sand and gravel.

T-64p

0.0	
1.0	Loose topsoil, sandy loam and
8.0	few cobbles. Loose, variable sand and gravel with stratum of gravelly and slightly silty sand from 5.0' to 6.0'. (Few scattered cobbles and occassional boulder.)



LOCATION MAP FOR BORROW AREAS

1000 0 1000

SCALE IN FEET

WEST PETERBORO

LEGEND

Indicates ground water.

Notes:-

For "Index of Subsurface Exploration" See PLATE II-1

Figures on left side of logs denote elevations and refer to Mean Sea Level.

Figures on right side of logs are depths below existing ground surface at time of exploration.

NO.	CHARACTER	DATE	APPR'D
	REVISIONS		

MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
NUBANUSIT BROOK

PLAN AND RECORD OF BORROW EXPLORATION

SCALE AS SHOWN

U. S. ENGINEER OFFICE, BOSTON, MASS. MAY, 1945

APPROVAL RECOMMENDED: John E. Allen
CHIEF ENGINEERING DIVISION

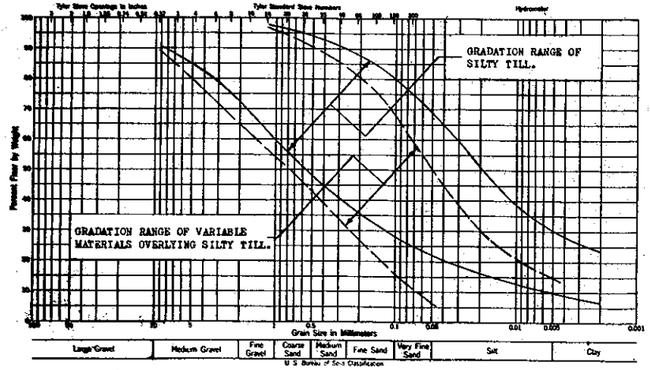
APPROVED: Louis A. Pitts
COLONEL, CORPS OF ENGINEERS
DISTRICT ENGINEER

SUBMITTED: A. S. Brown
CHIEF DESIGN BRANCH, SR. ENGINEER

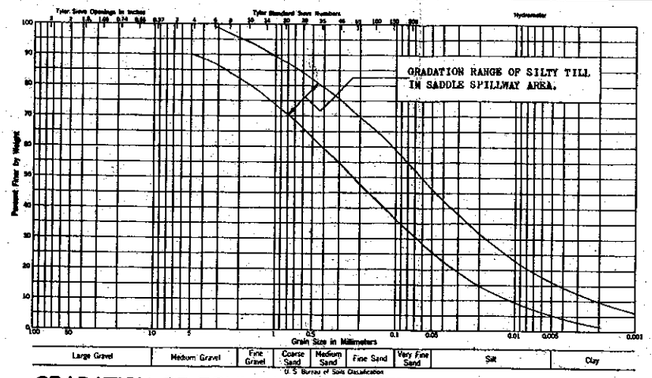
W. L. Shannon
ENGINEER

DSN. BY
DR. BY
TR. BY
CH. BY

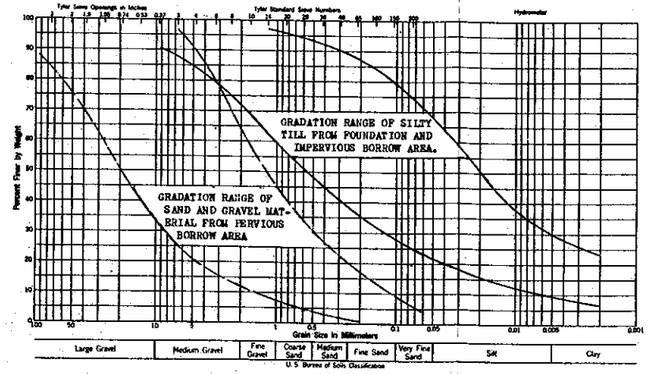
FILE NO. M78-12/65



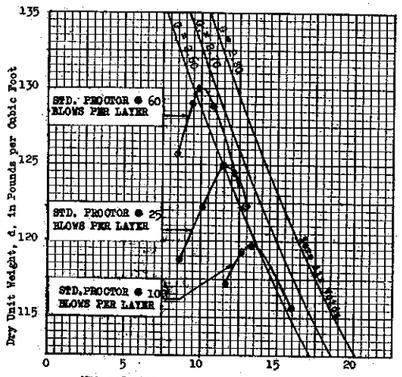
GRADATION OF FOUNDATION MATERIALS AT DAM SITE
FIG. 1



GRADATION OF FOUNDATION MATERIALS AT SPILLWAY
FIG. 2

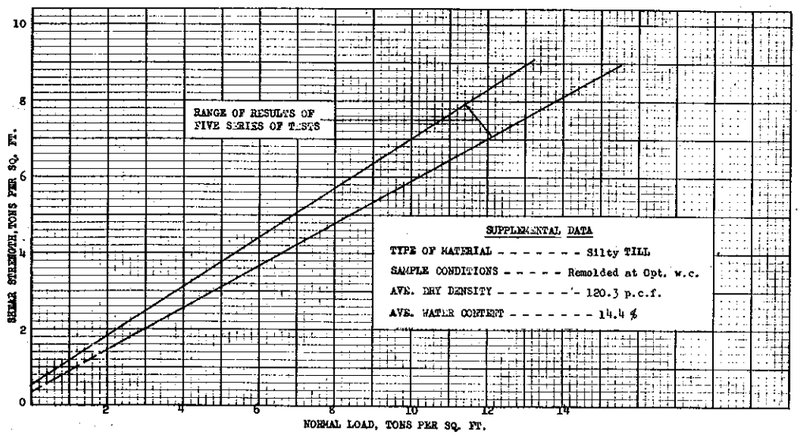


GRADATION OF BORROW MATERIALS
FIG. 3

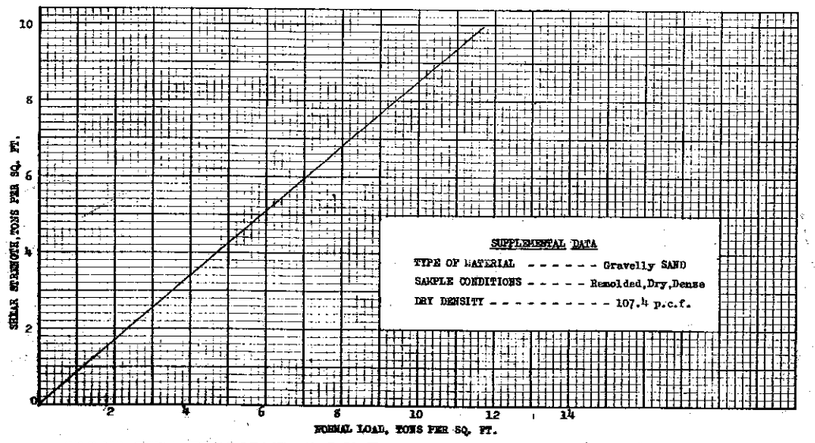


ITEM	UNIT	TEST 1	TEST 2	TEST 3
MAX. DENSITY < ¼" SIEVE	p.c.f.	119.7	125.0	130.0
OPT. WATER CONTENT < ¼"	%	13.4	11.4	9.7
SPECIFIC GRAVITY	—	2.74	2.74	2.74

MOISTURE-DENSITY PLOT FOR SILTY TILL
FIG. 4



ENVELOPES OF MOHR CIRCLES FOR REPRESENTATIVE SAMPLE OF IMPERVIOUS MATERIAL IN CONSOLIDATED QUICK SHEAR TESTS
FIG. 5



ENVELOPE OF MOHR CIRCLES FOR REPRESENTATIVE SAMPLE OF PERVIOUS MATERIAL IN CONSOLIDATED QUICK SHEAR TESTS
FIG. 6

SUMMARY OF PROPERTIES OF MATERIALS ENCOUNTERED

IDENTIFICATION			NATURAL PROPERTIES					REMOLDED PROPERTIES		
TYPE	OCCURRENCE	CLASSIFICATION	WATER CONTENT w % dry wt.	DRY DENSITY d p. c. f.	VOID RATIO e	PERMEABILITY k 10^{-4} cm/sec.	SHEAR STRENGTH ϕ deg.	COMPACTION CHARACTERISTICS	PERMEABILITY k 10^{-4} cm/sec.	SHEAR STRENGTH ϕ deg.
Foundation Materials at Dam	Unconsolidated Surface Deposits	Variable gravelly and silty SAND	17.3 (a) (8) 10.1-24.0	112.6 (8) 100-123	0.50 (8) 0.69-0.36	5.0 (b) est.	35 est.	120 p.c.f. at w =12% est.	1.0 est.	35 est.
	Upper zone of TILL	Weathered brown sandy TILL	12.0 (11) 10.0-14.2	125.6 (10) 113-131	0.35 (10) 0.50-0.29	0.1 est.	35 est.	120 p.c.f. at w =12% est.	0.18 (3) 0.13-0.22	35 est.
	Principal TILL body	Gray compact silty TILL	11.7 (20) 9.2-14.9	136.3 (10) 125-149	0.24 (10) 0.35-0.15	.003 (1)	35 est.	See Figure 4	.066 (6) 0.02-0.10	See Figure 5
Foundation Materials at Spillway	Unconsolidated Surface Deposits	Variable gravelly and silty SAND and sandy TILL	14.0 (24) 9.8-23.6	118.4 (7) 108-125	0.42 (7) 0.56-0.35	3.0 est.	35 est.	120 p.c.f. at w =12% est.	3.0 est.	35 est.
	Principal TILL body	Gray compact silty TILL	11.2 (22) 8.2-14.6	131.7 (6) 128-135	0.28 (6) 0.32-0.25	0.01 est.	35 est.	See Figure 4	.07 (2) 0.06-0.08	35 est.
Embankment Materials	Impervious FILL (Borrow)	Silty TILL	9.4 (11) 8.4-11.5	133.4 (16) 121-147	0.27 (16) 0.39-0.15	0.01 est.	35 est.	See Figure 4	0.01 est.	35 est.
	Random FILL (Structure Excavation)	Silty SAND and sandy TILL	12.8 (17) 3.9-20.0	115.8 (11) 100-127	0.46 (11) 0.69-0.33	1.0 est.	35 est.	120 p.c.f. at w =12% est.	1.0 est.	35 est.
	Pervious FILL (Borrow)	GRAVEL and SAND	5.8 (14) 3.9-7.0	107.0 (3) 103-110	0.59 (3) 0.64-0.54	10-100 est.	35 est.	Max. density (Compacted dry) 115 p.c.f. est.	34 (3) 23.0-54.0	See Figure 6

NOTES

(a) Top figures indicate average data
Figures in ( ) indicate number of tests
Figures below ( ) indicate range of test results

(b) "est" indicates estimated values based on similar materials for other sites.

FIG. 7

NO.	CHARACTER	DATE	APPR'D
	REVISIONS		

MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
NUBANUSIT BROOK

SOIL DATA SUMMARY

U. S. ENGINEER OFFICE, BOSTON, MASS. MAY, 1962

APPROVAL RECOMMENDED: John E. Allen, CHIEF ENGINEERING DIVISION

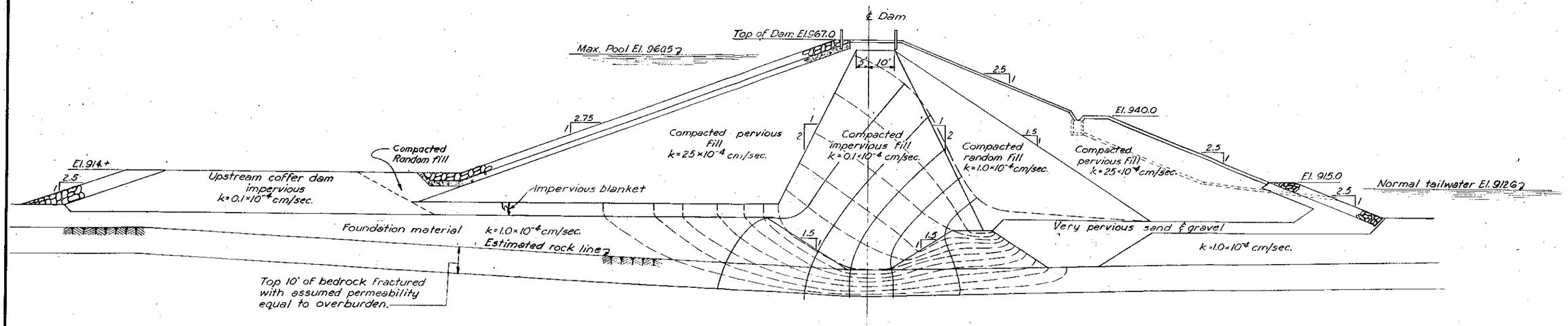
APPROVED: [illegible], COL., CORPS OF ENGINEERS, DISTRICT ENGINEER

SUBMITTED: J. S. Brown, CHIEF DESIGN BRANCH, SR. ENGINEER

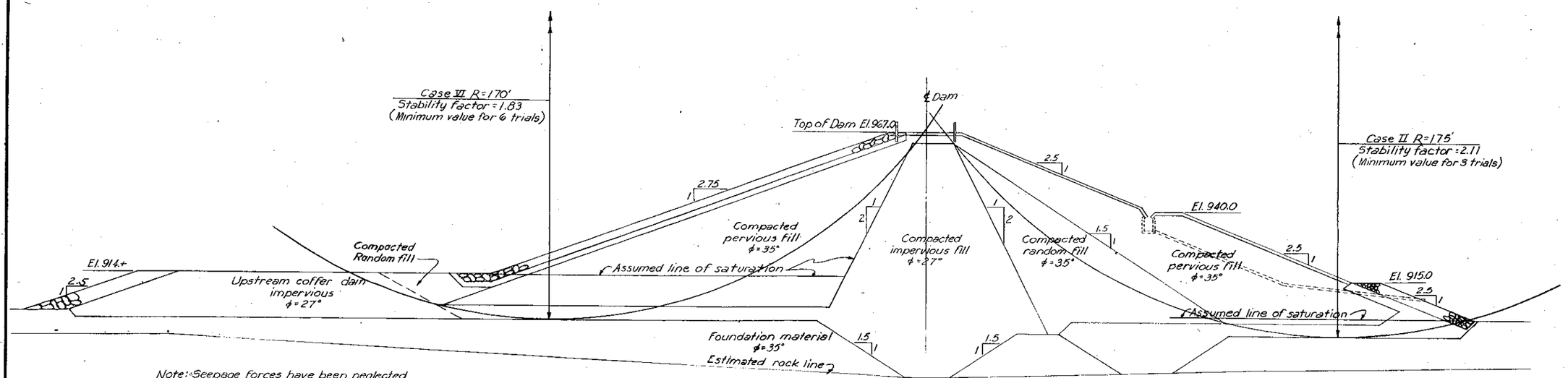
W. L. Shannon, ENGINEER

DES. BY
DR. BY
TR. BY
CK. BY

FILE NO. M 78-12/66



FLOW NET FOR TYPICAL VALLEY SECTION



Note: Seepage forces have been neglected in this analysis.
Cases studied are for sudden draw down.

STABILITY ANALYSIS FOR TYPICAL VALLEY SECTION

NO.	CHARACTER	DATE	APPR'D
	REVISIONS		

MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
NUBANUSIT BROOK

SEEPAGE & STABILITY ANALYSES

SCALE: 1" = 20'

U.S. ENGINEER OFFICE, BOSTON, MASS. MAY 1945

APPROVAL RECOMMENDED: John E. Allen
CHIEF ENGINEERING DIVISION

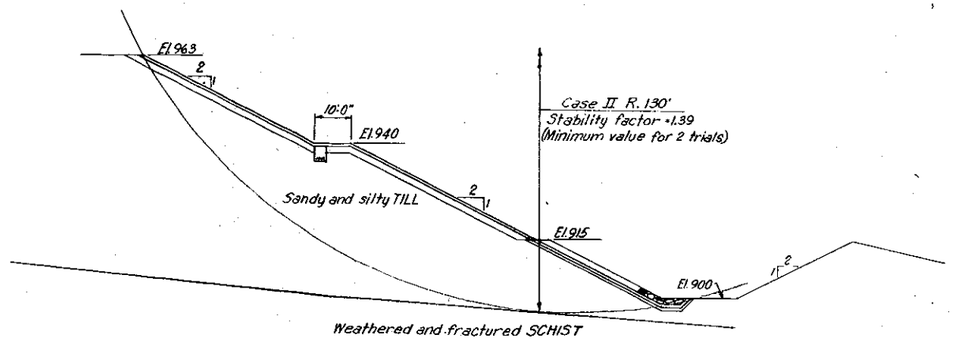
SUBMITTED: J. B. Brown
CHIEF DESIGN BRANCH SR. ENGINEER

W. L. Shannon
ENGINEER

DES. BY
DR. BY
TR. BY
CH. BY

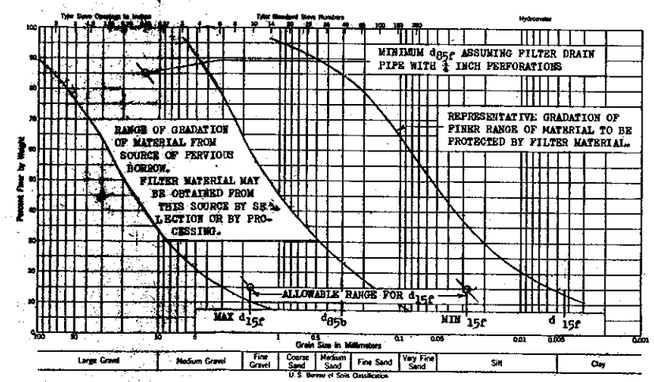
APPROVED: Jonny B. Patt
COLONEL, CORPS OF ENGINEERS
DISTRICT ENGINEER

FILE NO. M78-12/67



STABILITY ANALYSIS OF WEST BANK OF OUTLET DISCHARGE CHANNEL AT DAM SITE

20 0 20 40
SCALE IN FEET



FILTER DESIGN

MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
NUBANUSIT BROOK

STABILITY ANALYSIS AND FILTER DESIGN

U. S. ENGINEER OFFICE, BOSTON, MASS. MAY, 1945

APPROVAL RECOMMENDED: John E. Allen, CHIEF ENGINEERING DIVISION

APPROVED: [signature], COLONEL, CORPS OF ENGINEERS, DISTRICT ENGINEER

SUBMITTED: [signature], CHIEF DESIGN BRANCH, DR. ENGINEER

[signature], ENGINEER

FILE NO. M 78-12/68

NO.	CHARACTER	DATE	APPR'D
	REVISIONS		

PLATE II-14

WAR DEPARTMENT
United States Engineer Office
Boston, Massachusetts

DEFINITE PROJECT REPORT

WEST PETERBORO RESERVOIR

APPENDIX III

HYDRAULIC DESIGN

To accompany definite project report
dated May 1945 (Revised June 1946)

DEFINITE PROJECT REPORT

WEST PETERBORO RESERVOIR

APPENDIX III - HYDRAULIC DESIGN

C O N T E N T S

Paragraph	Title	Page
a.	General	III-1
b.	Design Criteria	III-1
c.	Existing Verney Mills Dam	III-1
d.	Size of Gates and Conduits	III-1
e.	Stilling Basin	III-2
f.	Description	III-3
g.	Spillway Weir	III-3
h.	Approach Channel	III-3
i.	Discharge Channel	III-4

PLATES

Plate	Title
III-1	Hydraulic Data for Conduit
III-2	Stilling Basin
III-3	Discharge Curves
III-4	Spillway Discharge Curve

(Rev. June 1946)

DEFINITE PROJECT REPORT
WEST PETERBORO RESERVOIR

APPENDIX III - HYDRAULIC DESIGN

a. General.- This appendix discusses the hydraulic factors involved in the design of the flood control conduit and related structures and of the spillway, including its approach channel and discharge chute. The project involves no new or unusual features and the various factors used in design computation follow conservative practice for long-established design principles.

DESIGN OF CONDUIT

b. Design Criteria.- West Peterboro Reservoir will normally be operated as a simple retarding basin but the single conduit is equipped with three gates to permit complete closure when desirable. Design of the conduit followed these criteria:

(1) To limit flood flows to the safe channel capacity below the West Peterboro dam site, estimated at about 1000 c.f.s.

(2) To obtain this discharge capacity of about 1000 c.f.s. with a reservoir elevation corresponding to about 25% of the total reservoir capacity.

(3) To empty the reservoir in a reasonable period of time.

(4) To provide adequate diversion capacity during construction.

(5) To permit regulation of the existing Verney Mills Dam power pool without excessive head loss through the conduit.

c. Existing Verney Mills Dam.- The West Peterboro Dam will be located in the pool of the existing Verney Mills Dam. In the development of the West Peterboro Project it is planned to provide the same power storage under future conditions as is provided under existing conditions. The Verney Mills Dam, located about 1000 feet below the West Peterboro dam site, is a concrete gravity dam with spillway crest about 92 feet long at Elevation 909.6. The crest is provided with 18-inch flashboards, resulting in a total usable storage of about 300 acre-feet available with maximum drawdown to Elevation 906.5. Spillway abutments are at elevation 913.6 ±. A steel penstock 5 feet 6 inches in diameter transports water from this dam to the Verney Mills, a distance of about 1000 feet. Flow into the penstock is controlled by manually operated gates.

d. Size of Gates and Conduit.- The conduit selected is 7 feet square with three gates, each 4 feet by 5 feet. The size

of the conduit was dependent on obtaining the specified 1000 c.f.s. discharge capacity with reservoir water surface at elevation 921.5. The shape of the conduit was selected to permit extension of the existing penstock for future power development as described in Appendix V. The length of the conduit is about 320 feet. The conduit invert will slope from elevation 904 at upstream end to elevation 903 at the downstream end. The exit portal of the conduit is designed with a slightly reduced section (6 feet high and 7 feet 8 inches wide) to eliminate possible low pressures within the conduit itself and to start spreading the flow before discharging into the stilling basin. Friction losses in the conduit were computed by the Manning formula, using n =0.013. Other hydraulic losses considered were for trash racks, entrance, eddies at gate slots, and transition. It is estimated that with all gates open and reservoir at spillway crest, the discharge will be about 1680 c.f.s. (Plate III-2). Energy and hydraulic gradients for 1680 c.f.s. (three gates open with reservoir at Elevation 946) and for 1000 c.f.s. (three gates open with reservoir at Elevation 922) are shown on Plate III-2. The discharge of 120 c.f.s. required by Verney Mills can be obtained through the conduit with a total head loss of about 0.25 feet when Verney Mills pool is at Elevation 909.6, and a total head loss of about 0.4 feet when Verney Mills pool is drawn down to Elevation 908.0. Head losses for other elevations of Verney Mills pool are shown on Plate III-2.

e. Stilling Basin.- The stilling basin is designed to dissipate the energy in the conduit discharges obtainable with reservoir stages at maximum elevation during the spillway design flood, and with all gates open. For this discharge (2100 c.f.s.) the tailwater based on the spillway discharge of the Verney Mills Dam is elevation 913.3. To provide for a lower tailwater due to flow through the power conduit and sluice gates and for lower discharges the elevation of the stilling basin floor is established at elevation 898. The basin is 40 feet long and 25 feet wide. Although the tailwater elevation will submerge the portal of the conduit during normal daily flow, the high discharge velocity, obtained when there is any appreciable head on the conduit, will force the tailwater away from the portal and allow the discharge jet to spread into the stilling basin. In designing the concrete stilling basin a length of 40 feet was found to be sufficient because the discharge channel from the basin is in rock excavation and eliminates the possibility of scouring. The hydraulic jump will reduce the portal velocity of 46 feet per second to 5 feet per second at the end of the basin.

f. Description.- The selected spillway site is located in a natural saddle in the eastern rim of the basin about 3 miles from the dam site and about 1900 feet northeast of Half Moon Pond. The cut through the saddle consists of a concrete weir 100 feet long at elevation 946, and an approach channel to the weir about 1900 feet long, varying in width from 140 feet at Half Moon Pond to 100 feet just above the weir. The discharge channel extends approximately 800 feet downstream from the weir with constant width of 100 feet. Approximately 300 feet of the discharge channel is in rock cut. The discharge channel has a constant slope of 1 per cent. Beyond the defined saddle the spillway discharge will flow into a swamp and wooded area as shown on Plate IV-5.

g. Spillway Weir.- The spillway weir is a concrete section with upstream face sloped 1 on 1. The shape of the cross section was selected from data in the U. S. Bureau of Reclamation report "Abstract of Studies of Crests for Overfall Dams, Bulletin 3 - Part VI". The elevation of the crest, 946, is determined by storage requirements of the reservoir, limited by the elevation of Half Moon Pond as discussed in Appendix I. The length of 100 feet was selected for the spillway after preliminary estimates showed that this length gave the lowest combined cost of dam and spillway. The discharge capacity of the spillway is computed by the weir formula $Q - CLH^{3/2}$. At maximum head a coefficient of about 3.8 is possible with the crest and approach channel selected but a value of 3.7 was used with high discharges to allow for the possibility of some end contractions at the weir. The spillway discharge curve is shown on Plate III-4.

h. Approach Channel.- The trapezoidal approach channel varies in width from 140 feet at the northern end of Half Moon Pond to 100 feet just above the weir section. The bottom elevation of 941 was selected as giving the least possible excavation consistent with stable flow at high discharges. Part of the approach channel is to be excavated in rock using side slopes of 3 on 1. Side slopes of the remainder of the cut are 1 on 2. Except for the section cut in rock, the channel for a distance of 900 feet above the weir section is to be lined with 18-inch hand-placed riprap. At maximum head there will be an average velocity of about 8.2 feet per second in the section above the weir. This velocity will be less as the approach channel widens upstream and will be about 5.9 feet per second at the end of the riprap section. It is estimated that the average velocity in the channel above this section will be about 4.4 feet per second since the banks are so low that a considerable portion of the discharge will be overbank flow. Since this channel is about 1900 feet long, from Half Moon Pond to the Weir, it was

necessary to compute friction losses and velocity head differences in determining the relation between reservoir elevation and discharge at the weir. The relation between elevations in the reservoir and at the weir are shown in the following table:

Reservoir Elevation at Half Moon Pond	Velocity Head plus Friction Losses	Water Surface Elevation at Weir	Discharge in c.f.s.
946.0	0	946.0	0
948.0	0.1	947.9	1000
950.0	0.3	949.7	2600
952.0	0.8	951.2	4600
954.0	1.3	952.7	7000
956.0	1.8	954.2	9800
958.0	2.2	955.8	13000
960.8	2.6	958.2	17800

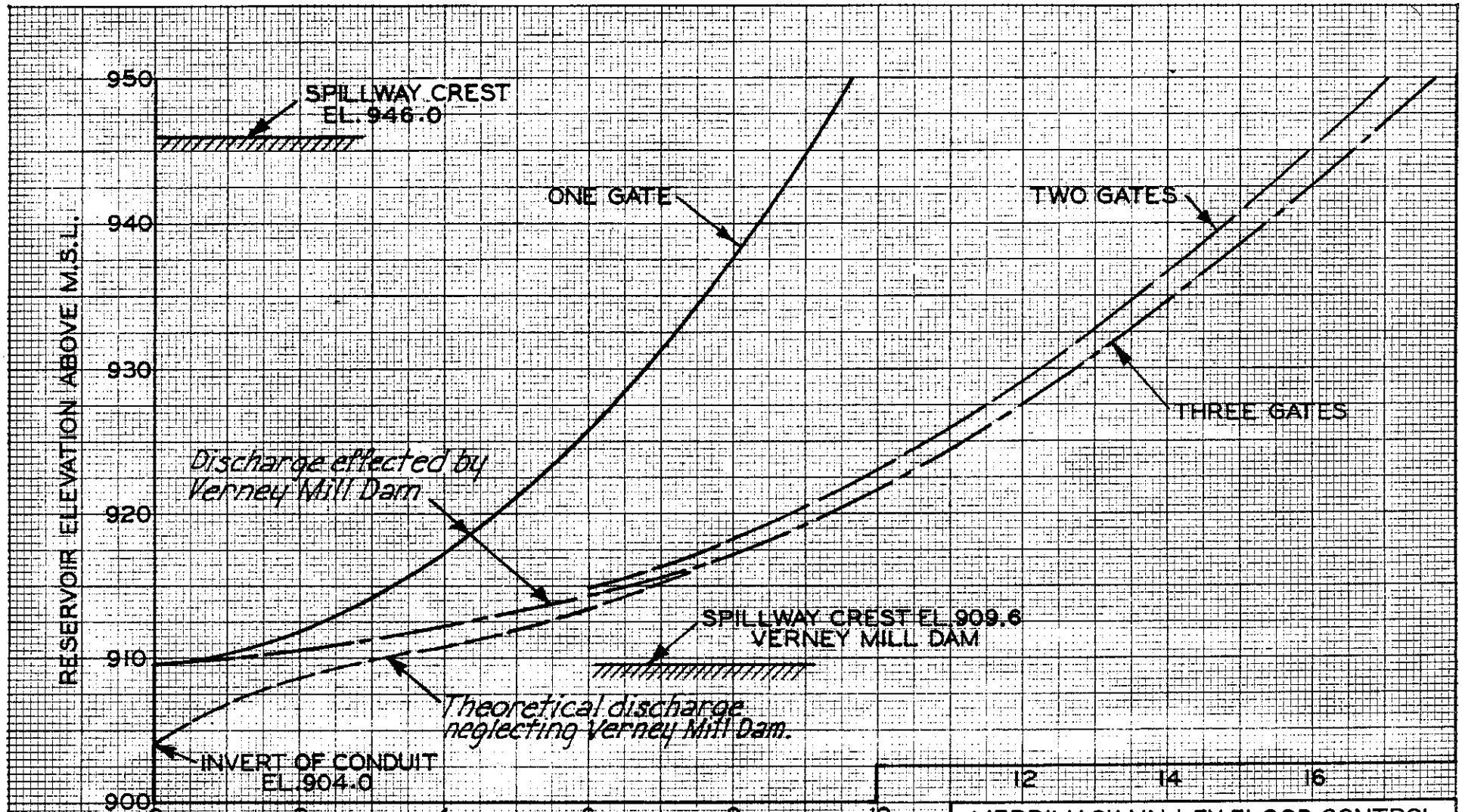
These losses have been used in determining the spillway rating curve shown on Plate III-4.

i. Discharge Channel.- The discharge channel below the weir is a trapezoidal section with bottom 100 feet wide and at elevation 935. The section is changed slightly in a distance of 200 feet, by dropping the bottom to 933 in the center while keeping the elevation of 935 at the side walls. This cross section is maintained for the rest of the cut at a constant slope of 1%. The section has side slopes of 3 on 1 for rock cut and 1 on 2 for earth. The latter is riprapped up to elevation 950 along the left bank for a distance of 350 feet below the crest. Computations indicate that the spillway design discharge of 17,800 c.f.s. will reach a limiting depth of about 12 feet in this spillway channel. This depth, with the bottom elevation of 935, permits the weir to discharge all flows without appreciable submergence. It is expected that the spillway design discharge will be attended by standing waves below the crest. The top of the short concrete wall on the left bank slopes from elevation 954 to elevation 952 and provides reasonable freeboard from these waves.

The discharge channel terminates in a swampy area which is expected to retard low and medium discharges. This swamp is now drained by a small brook which at one point passes through a culvert about 2-1/2 feet square. A limited amount of diking and channel improvement is required here and at a few other points to confine large flows and lead them to the valley of Ferguson Brook. Since the maximum pool for Bennington Reservoir will extend up Ferguson Brook to this point, no improvement is required to permit greater

(Rev. June 1946)

carrying capacity except for enlargement of the water area at the first bridge. As shown on Plate IV-5, it is proposed to replace the small existing road culvert with a large dual culvert with a capacity of approximately 1000 c.f.s. which is sufficient for any nominal spillway discharge that may be anticipated from a reservoir design flood.



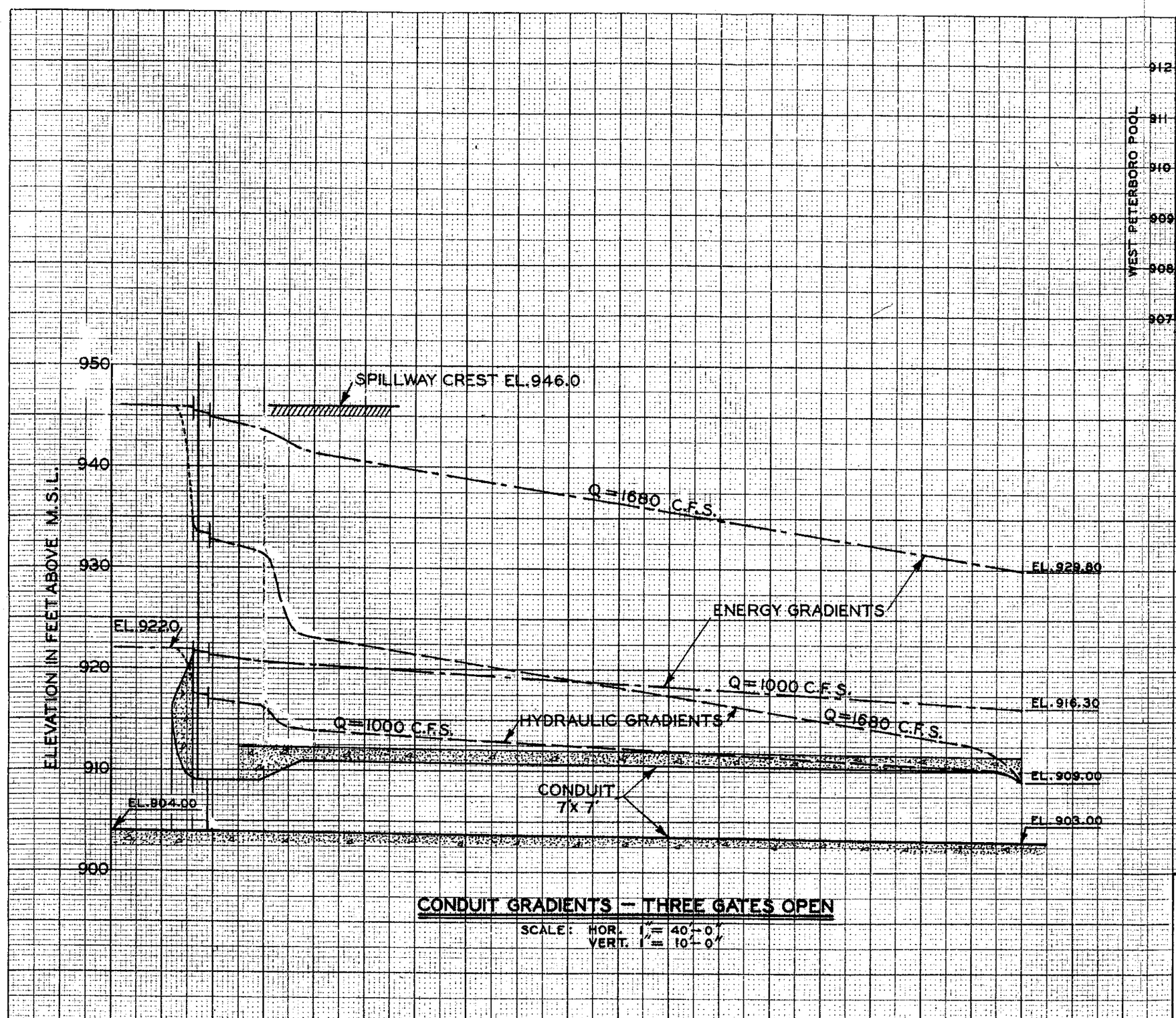
DISCHARGE CURVES FOR 7'X 7' CONDUIT
WITH THREE 5'X 4' GATES

MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
NUBANUSIT BROOK

CONDUIT DISCHARGE CURVES

U. S. ENGINEER OFFICE. BOSTON, MASS.
JUNE, 1946. FILE NO. M78-12/78

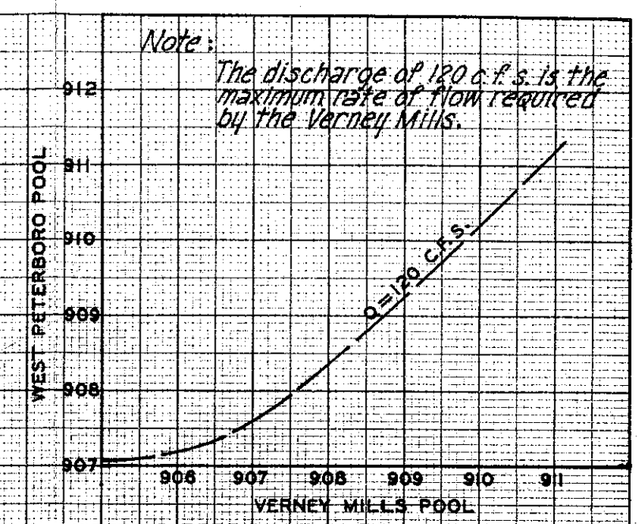
PLATE III-1



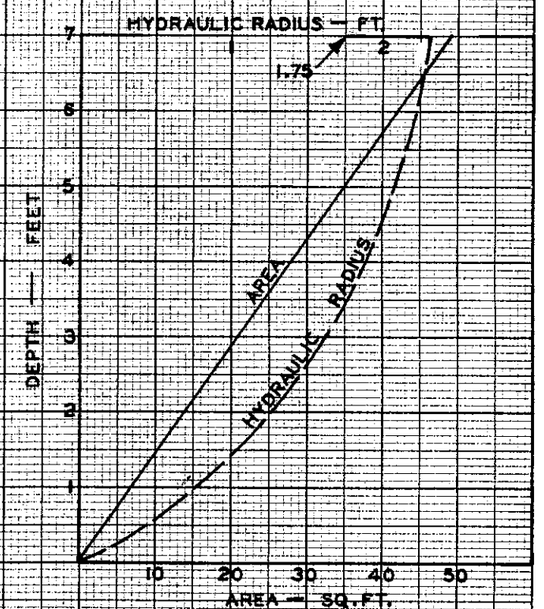
CONDUIT GRADIENTS — THREE GATES OPEN

SCALE: HOR. 1" = 40'-0"
VERT. 1" = 10'-0"

Note:
The discharge of 120 c.f.s. is the maximum rate of flow required by the Verney Mills.



DIFFERENCE IN POOL ELEVATION



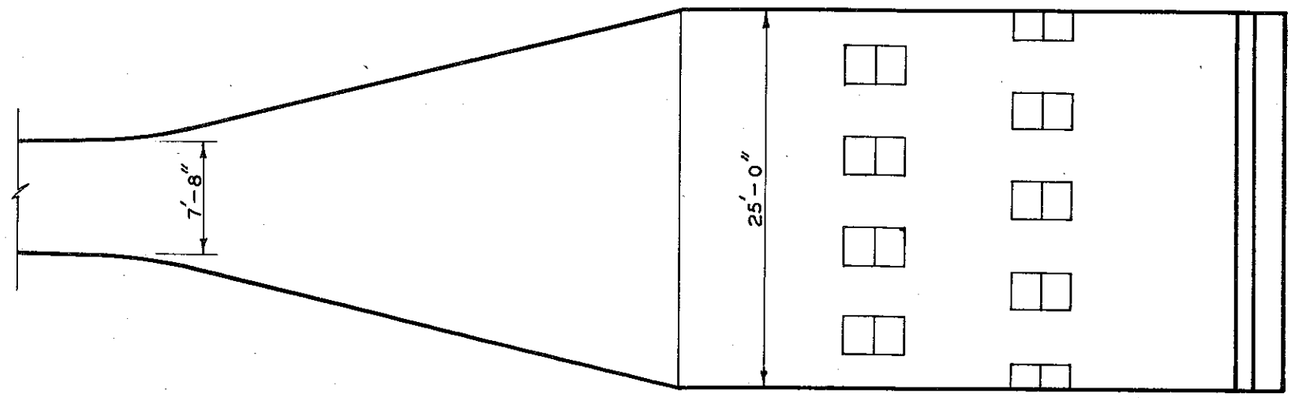
HYDRAULIC CHARACTERISTICS

MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
NUBANUSIT BROOK

HYDRAULIC DATA FOR CONDUIT

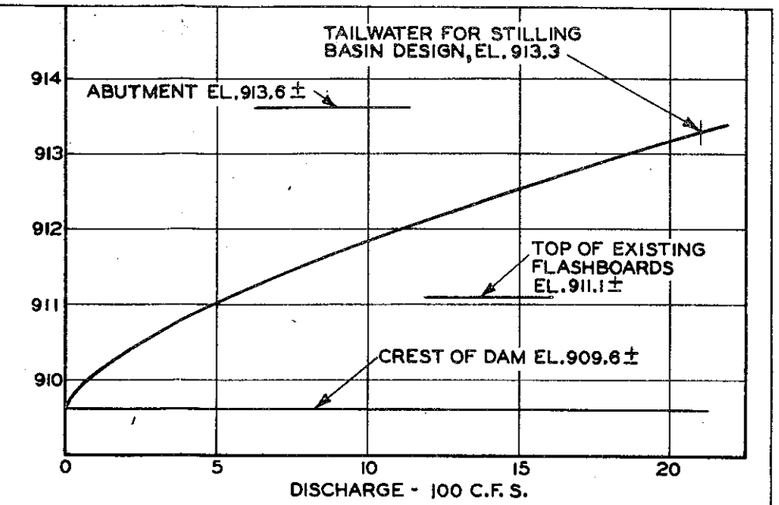
U.S. ENGINEER OFFICE, BOSTON, MASS.
JUNE, 1946. FILE NO M78-12/69A

PLATE III - 2



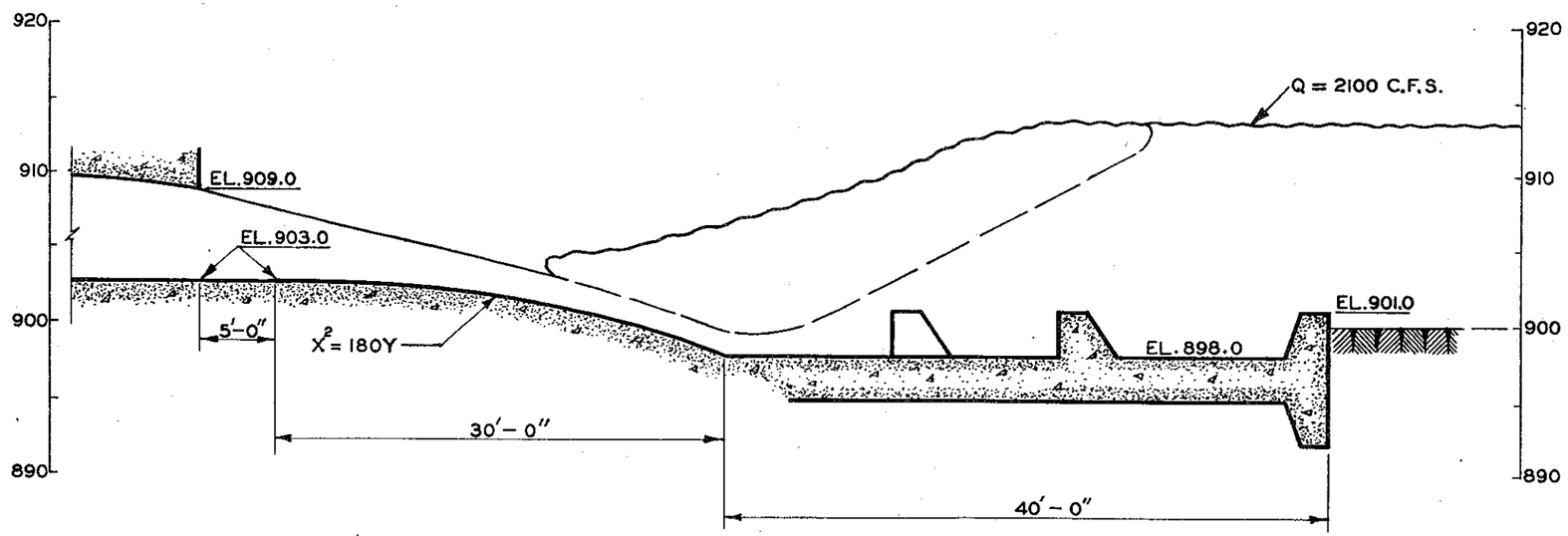
PLAN

SCALE: 1" = 10'-0"



TAILWATER RATING CURVE

BASED ON CONCRETE CREST OF VERNEY MILLS DAM
CREST LENGTH = 92 FT.



SECTION

SCALE: 1" = 10'-0"

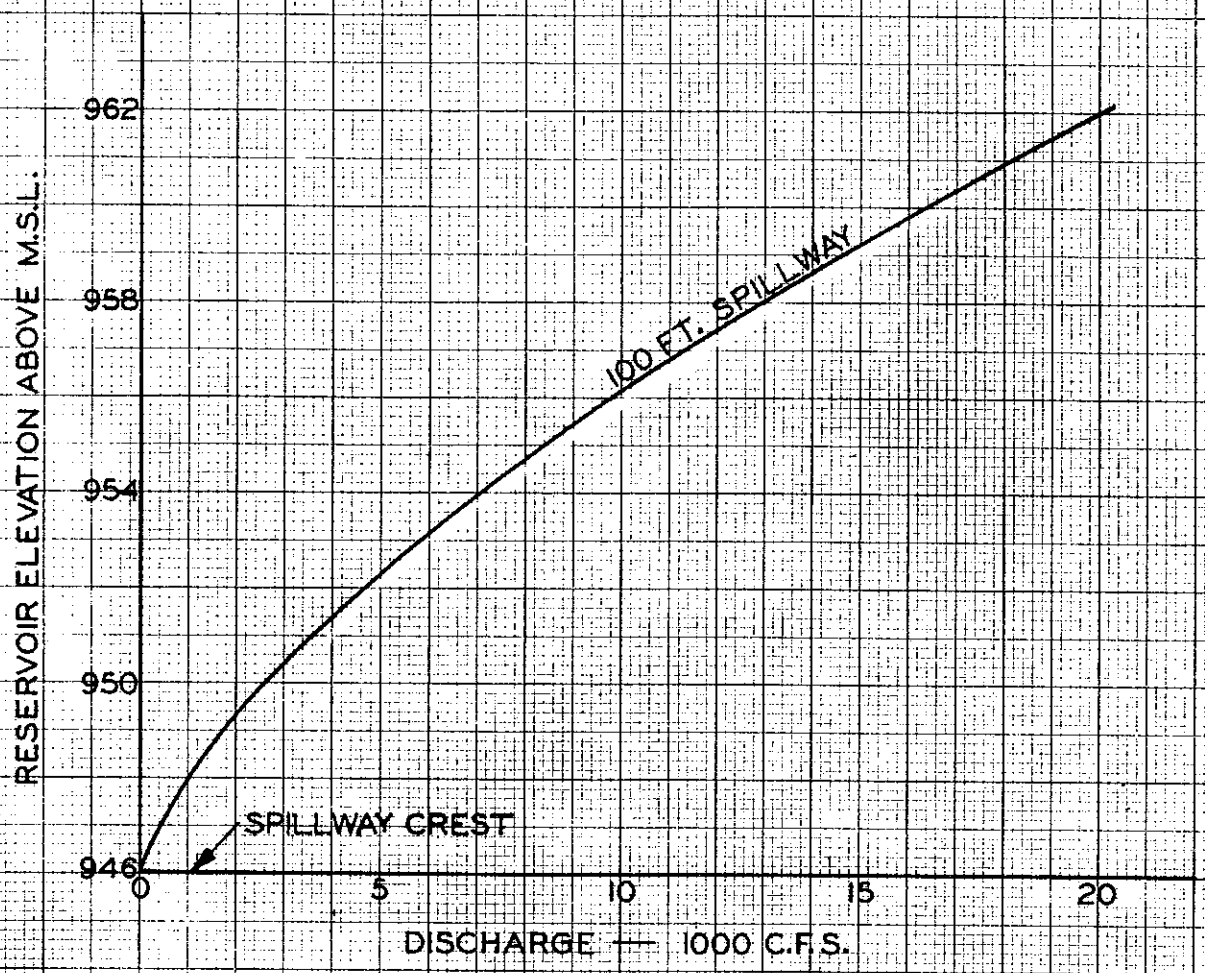
MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
NUBANUSIT BROOK

STILLING BASIN

U. S. ENGINEER OFFICE. BOSTON, MASS.
JUNE, 1946. FILE NO. M 78-12/70A

Note:

Discharges computed from $Q = CLH^{3/2}$ with $L = 100$ ft. and maximum $C = 3.7$ as determined from U.S. Department of Interior bulletin, "Studies of Crests for Overfall Dams". Curve corrected for backwater rise in water surface from Spillway to Reservoir.



MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
NUBANUSIT BROOK

SPILLWAY DISCHARGE CURVE.

U.S. ENGINEER OFFICE. BOSTON, MASS.
JUNE, 1946. FILE NO M78-12/78

War Department
United States Engineer Office
Boston, Massachusetts

DEFINITE PROJECT REPORT

WEST PETERBORO RESERVOIR

APPENDIX IV

STRUCTURAL DESIGN

To accompany definite project report
dated May 1945

DEFINITE PROJECT REPORT

WEST PETERBORO RESERVOIR

APPENDIX IV - STRUCTURAL DESIGN

C O N T E N T S

Paragraph	Title	Page
a.	Selection of Type and Arrangement of Structures	IV-1
b.	Consultants Conferences	IV-1
c.	Description of Structures	IV-2
d.	Protective Dike	IV-3
e.	Box Culvert	IV-3
f.	Structural Stability Analyses	IV-3

PLATES

Plate	Title
IV-1	General Plan of Dam and Outlet
IV-2	Profile and Typical Sections of Dam and Outlet
IV-3	Sections of Outlet Works
IV-4	General Plan and Typical Sections at Spillway
IV-5	Plan and Profile for Spillway Discharge and Protective Dike
IV-6	Stability Analyses, Concrete Structures

DEFINITE PROJECT REPORT
WEST PETERBORO RESERVOIR

APPENDIX IV. STRUCTURAL DESIGN

a. Selection of Type and Arrangement of Structures.- The entire reach of the narrow portion of the Nubanusit Brook Valley extending from the existing Verney Mills Dam to a section approximately 1700 feet upstream has been explored to develop the most economical location for construction of the proposed West Peterboro Dam. Layouts and estimates have been prepared for several types of dams at various locations within the above noted reach and with the spillway located in the remote saddle as described in Paragraph 4. Additional foundation exploration of the remote saddle spillway area, as originally recommended in the preliminary Definite Project Report, dated 25 April 1940, and the dam site spillway area as described in the Definite Project Report, dated 5 August 1940 (Revised 14 December 1940), has disclosed the following:

(1) The estimated costs of both plans are approximately equal.

(2) The construction of the dam site spillway involves excavation of large quantities of disintegrated rock which has little salvage value as riprap. The quality of the rock is such that the discharge channel requires a concrete lining.

(3) The construction of the dam site spillway incurs the possibility of a slide occurring on the planes of laminated silt during deep excavation for the construction of the spillway weir and discharge channel.

(4) The construction of the low saddle spillway presents no construction difficulties.

The locations of the dam and spillway selected, as shown on Plates IV-1 and IV-4 avoid interference with the existing industrial development on the stream and will afford the villages of Peterboro and West Peterboro complete protection from all damaging floods.

b. Consultants Conferences.- During the progress of the design, two Board of Consultants meetings were held in the Boston District Office and the selection of structures and features of design were discussed in detail. The first conference was held on 1 September 1944 and the following members were present:

Mr. W. H. McAlpine - Office, Chief of Engineers, Washington, D.C.
Dr. Arthur Casagrande - Harvard University, Cambridge, Mass., Consultant
Mr. W. F. Uhl - Charles T. Main, Inc., Boston, Mass. Consultant.

The second conference was held on 14-15 December 1944, and the following members were present:

Mr. W. H. McAlpine, Office, Chief of Engineers, Washington, D.C.
Mr. W. F. Uhl - Charles T. Main, Inc., Boston, Mass., Consultant
Mr. J. D. Justin - Philadelphia, Pennsylvania, Consultant
Dr. Arthur Casagrande, Harvard University, Cambridge, Mass Consultant

Informal discussions were held at various times with Mr. W.H. McAlpine and Dr. Arthur Casagrande on various features of design.

Prior to the completion of the final contract plans, one more meeting of the Board of Consultants will be held in order to review the contract plans.

c. Description of Structures. - (1) Dam Embankment. - The proposed dam, as shown on Plate IV-1, consists of a rolled earth embankment with a crest length of approximately 1100 feet and a maximum height of 69 feet with cross section as shown on Plate IV-2, the design of which is discussed in detail in Appendix II.

(2) Outlet Works. - The outlet works consist of a reinforced concrete intake structure, conduit through the dam and stilling basin, sections of which are shown on Plates IV-2 and IV-3. The intake structure contains the gated inlets and is connected with the top of the dam by means of a structural steel foot bridge. The conduit is a square section which passes through the embankment and has a series of cut-off collars for control of seepage through the dam. The size and shape of the conduit has been proportioned, at no additional expense, to permit the future installation of a penstock extension from the Verney Mills Dam as described in Paragraph 7. The stilling basin consists of a reinforced concrete slab with reinforced concrete baffles and end sill. Gravity type sections have been selected for the stilling basin walls in view of their greater durability. The outlet works are founded on bedrock and the stability analyses are for the structures as shown on Plate IV-6.

(3) Spillway. - The spillway is located northeasterly of Half Moon Pond and in a saddle which separates drainage to Mubanusit Brook and to Ferguson Brook. The most economical channel width and weir length were computed in relation to the corresponding dam height required. The spillway channel in the vicinity of the weir is cut through rock. A

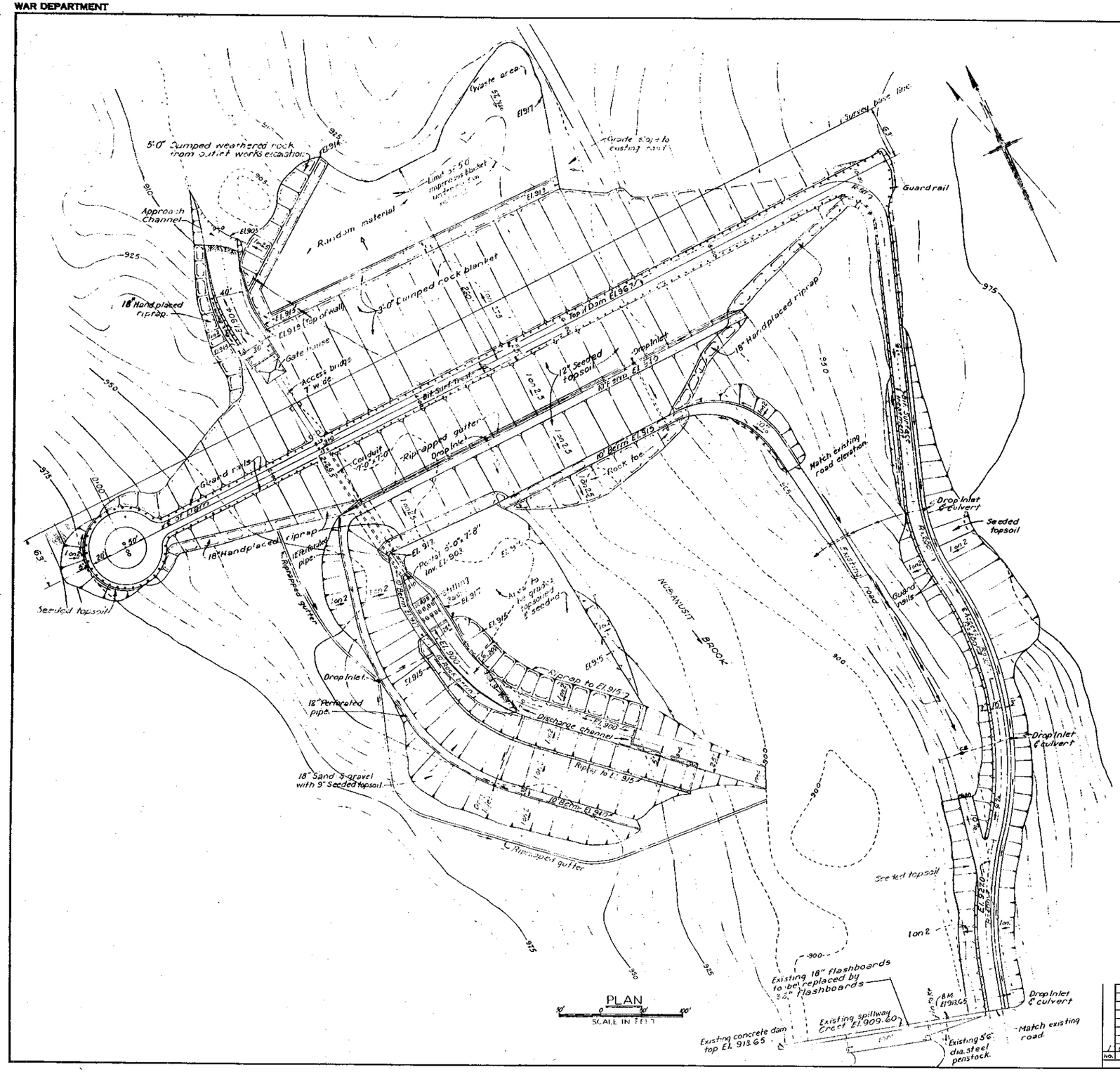
gravity concrete retaining wall, 20 feet high and 200 feet long, is extended downstream from the weir to assure added embankment protection. Slope protection for the earth surface of the approach channel, consisting of riprap and dumped rock, is provided upstream from the weir, for approximately 800 feet, as protection against scour from higher channel velocities. The spillway is a low concrete weir, and is founded on rock as are the training walls. Sections of the spillway structures are shown on Plate IV-4 and the stability analyses for the gravity walls are shown on Plate IV-6.

(4) Protective Dike.- The design of the earth-filled dike, on the north side of and down stream from the spillway discharge channel, is discussed in Appendix II. The location and sections of the dike are shown on Plate IV-5.

(5) Box Culvert.- The box culvert as shown on Plate IV-5 is located across the spillway discharge channel on Farm Road and is designed to permit the passage of a spillway flood equivalent to the computed spillway discharge of the 1936 flood. The structure is designed for standard highway traffic loadings and, with the exception of the gravity type abutments and wing walls, is of reinforced concrete construction.

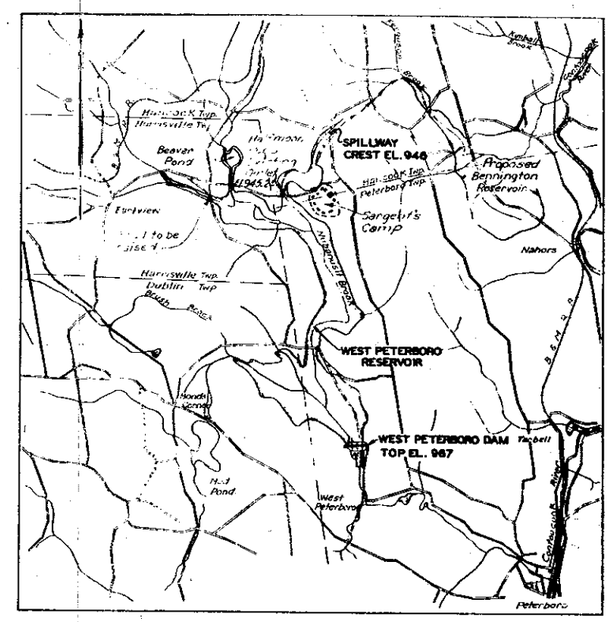
(6) Structural Stability Analyses.- The masonry structures are designed in accordance with standard methods and the results of the analyses are shown on Plate IV-6. The intake structure is analyzed with a water level in the reservoir equivalent to the extreme high water resulting from a spillway design flood and with the reservoir empty. The analysis of the conduit is based on the most severe condition when the reservoir is empty and the resulting earth fill loads applied to the exterior top and sides. The stability analyses for the wall sections are based on the lateral pressure of the earth retained and normal tailwater shortly after a design flood in which the saturation line in the fill is above normal tailwater. Uplift pressures applied to the masonry structures are equal to the full effective head applied over 50% of the base area.

In view of the relatively light foundation loads resulting from the computations and the remoteness of any possible earthquake shock to the dam when the reservoir is full, the structures were not analyzed with additional earthquake loadings.



PLAN

SCALE IN FEET



LOCATION MAP

5000' 0 5000'

SCALE IN FEET

MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
NUBANUSIT BROOK

GENERAL PLAN OF DAM AND OUTLET

U. S. ENGINEER OFFICE, BOSTON, MASS. MAY 1945

APPROVAL RECOMMENDED: John E. Allen, CHIEF ENGINEERING DIVISION

APPROVED: CHIEF, CORPS OF ENGINEERS, DISTRICT ENGINEER

SUBMITTED: CHIEF DESIGN SECTION, SR. ENGINEER

ENGINEER

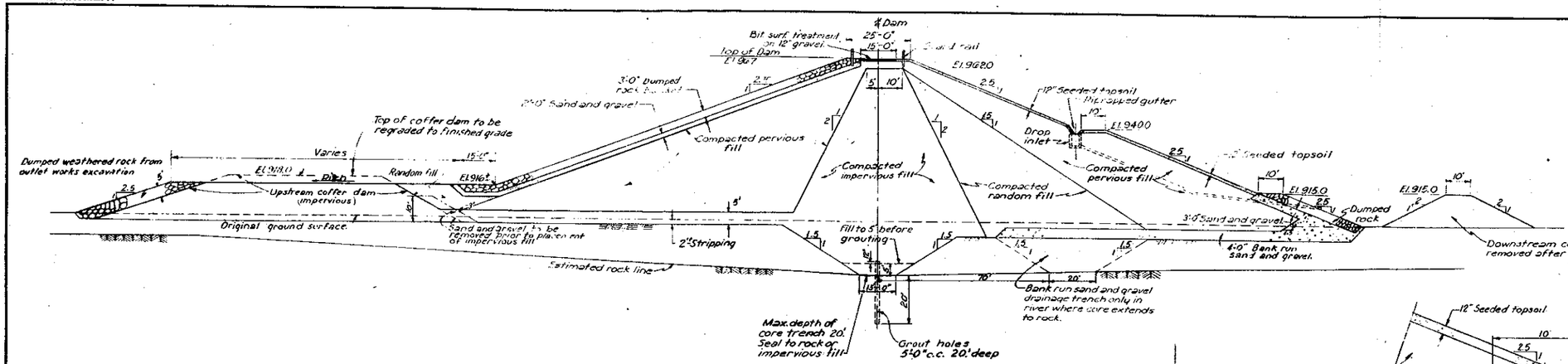
NO.	CHARACTER	DATE	BY
1	General Revisions		

REVISIONS

DES. BY
DR. BY
TR. BY
CK. BY

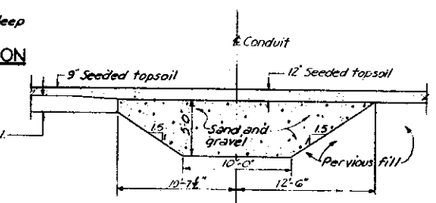
FILE NO. M 78-12/72

PLATE IV-1

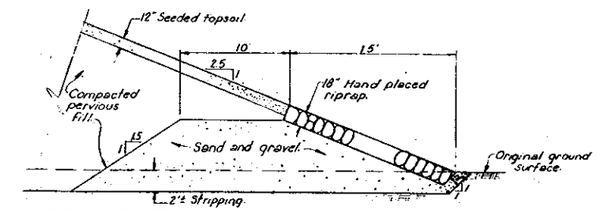


SECTION B-B TYPICAL VALLEY SECTION

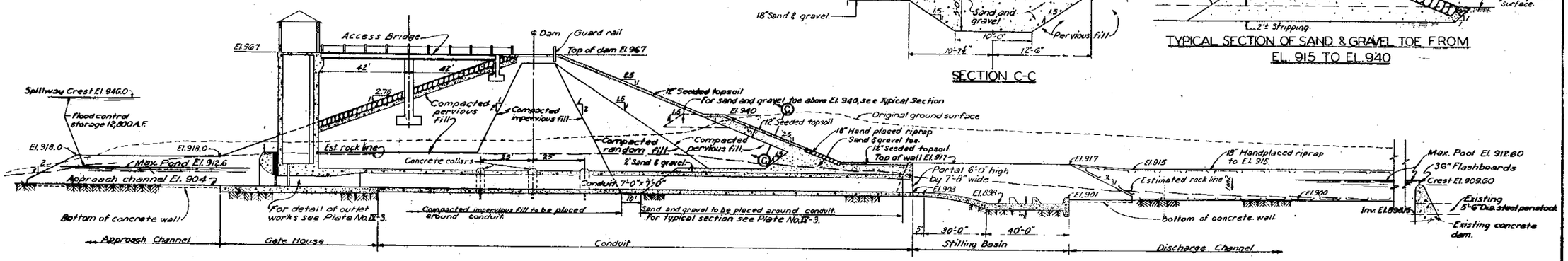
SCALE IN FEET



SECTION C-C

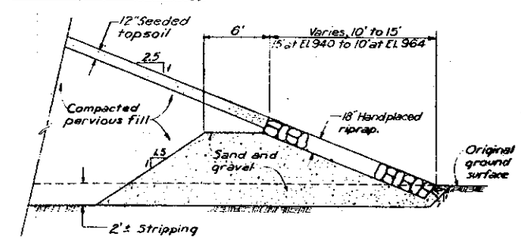


TYPICAL SECTION OF SAND & GRAVEL TOE FROM EL. 915 TO EL. 940



SECTION A-A THRU OUTLET WORKS

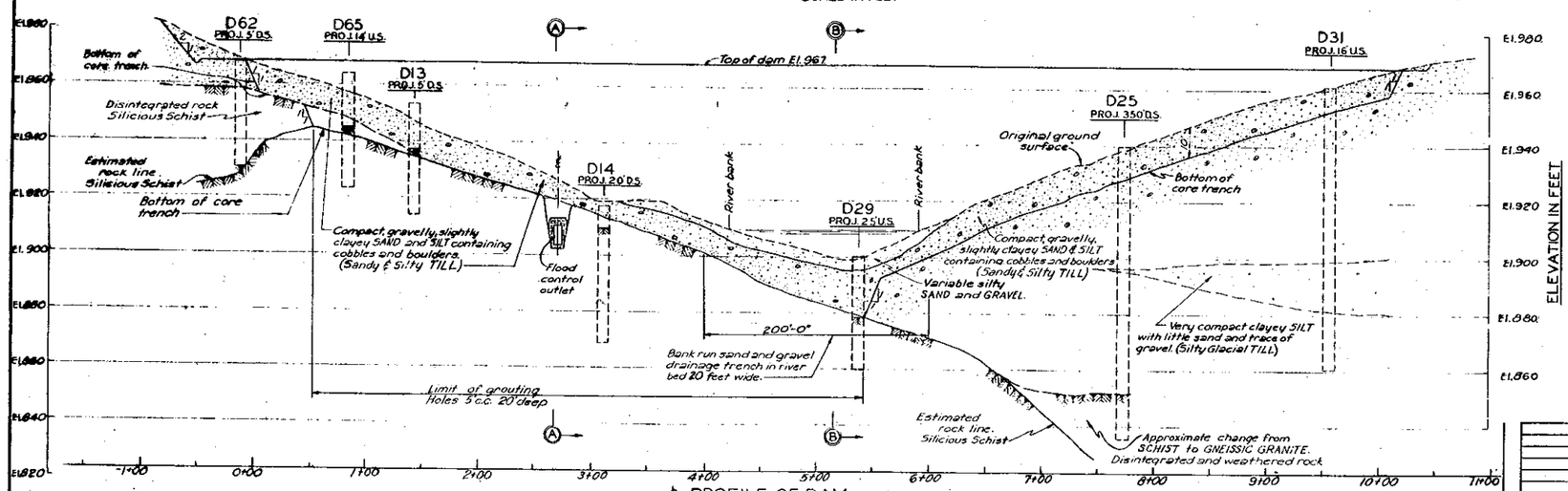
SCALE IN FEET



TYPICAL SECTION OF SAND AND GRAVEL TOE ABOVE EL. 940

NOTE: For index of Subsurface Exploration, see Plate II-1

SCALE IN FEET (UNLESS OTHERWISE NOTED)



℄ PROFILE OF DAM

NO.	CHARACTER	DATE	BY
1	General Revisions	6/16/44	

REVISIONS

MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
NUBANUSIT BROOK

PROFILE AND TYPICAL SECTIONS OF DAM AND OUTLET

U. S. ENGINEER OFFICE, BOSTON, MASS. MAY, 1945

APPROVAL RECOMMENDED: John L. Allen, Chief, Engineering Division

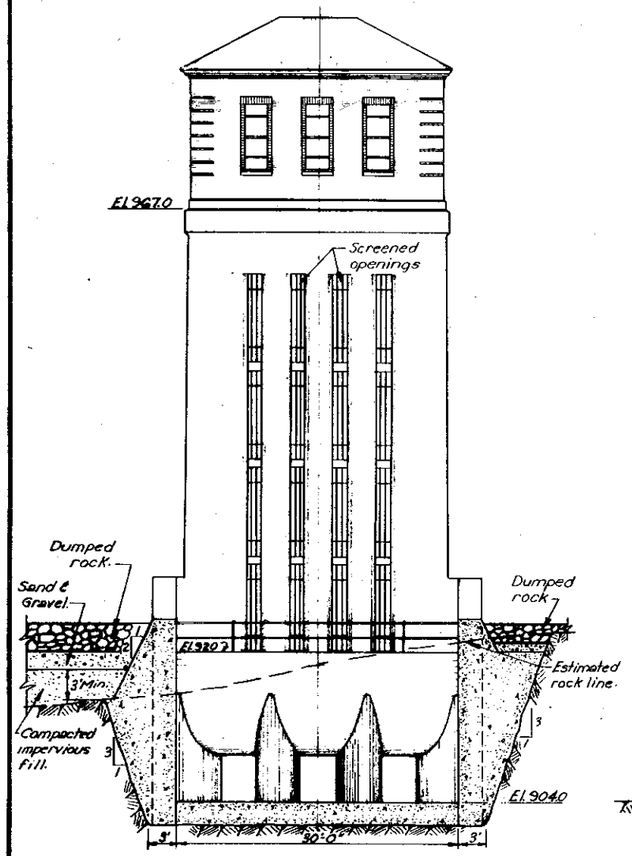
APPROVED: Willis A. Hitchcock, Colonel, Corps of Engineers, District Engineer

SUBMITTED: F. J. Brown, Chief, Design Section, Eng. Division

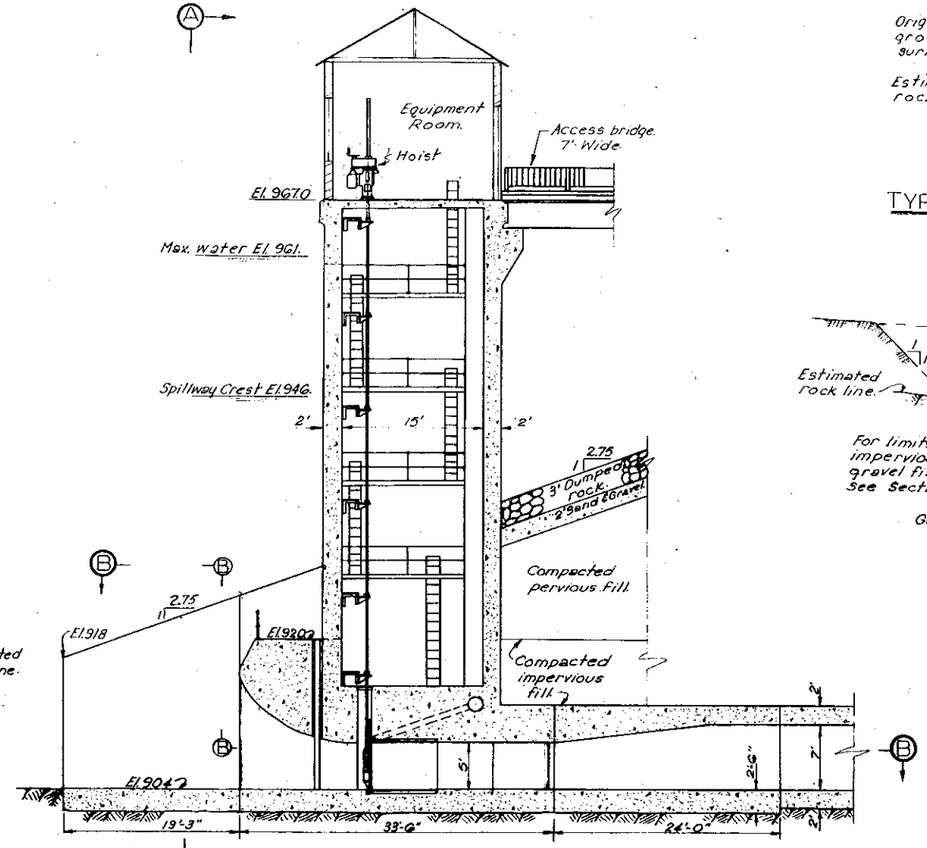
Designed: Richard J. Nelsen

DES. BY / DR. BY / TR. BY / CH. BY

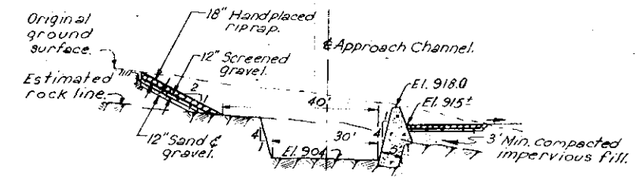
FILE NO. M 78-12/73



VIEW A-A

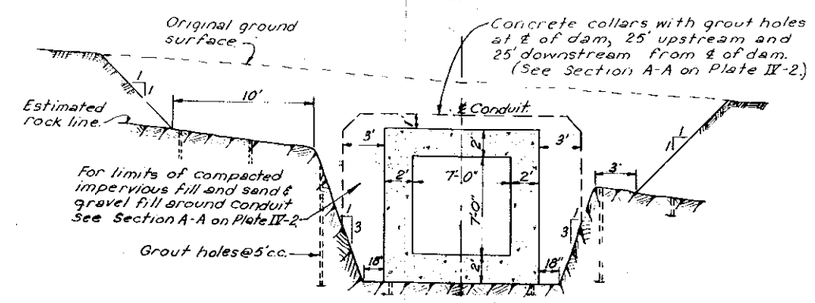


SECTION C-C



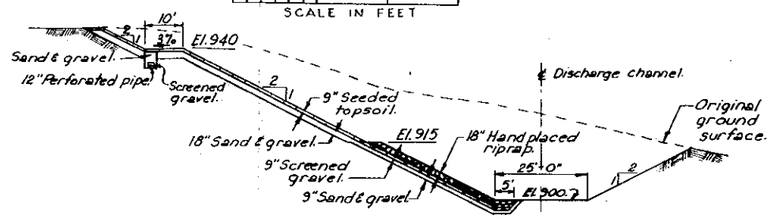
TYPICAL SECTION THRU OUTLET APPROACH CHANNEL

20' 0 20' 40'
SCALE IN FEET



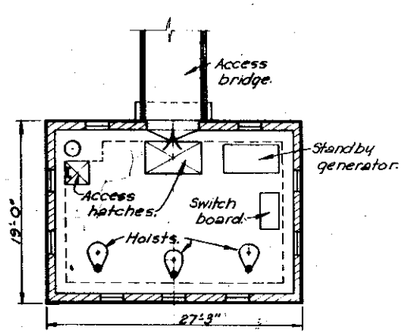
TYPICAL SECTION THRU CONDUIT

12' 0 5' 10' 15'
SCALE IN FEET

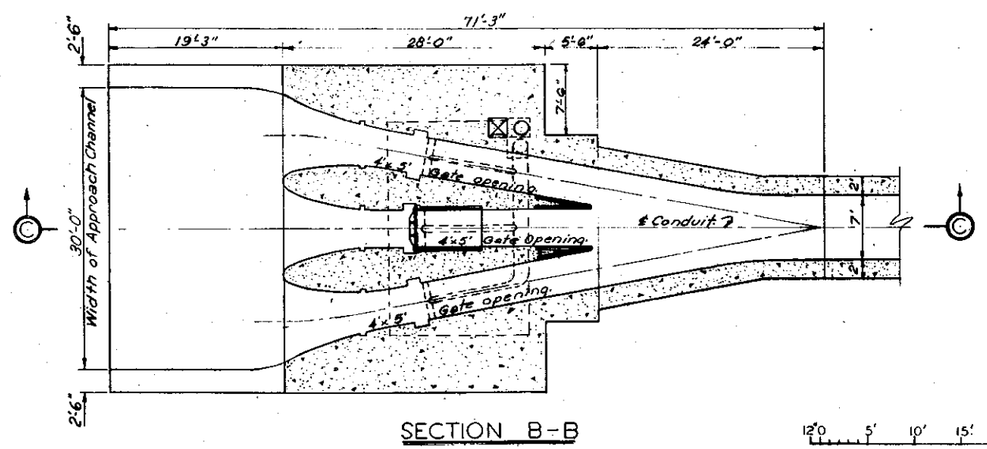


SECTION THRU SIDE SLOPE OF DISCHARGE CHANNEL STA. 8+60±

20' 0 20' 40'
SCALE IN FEET

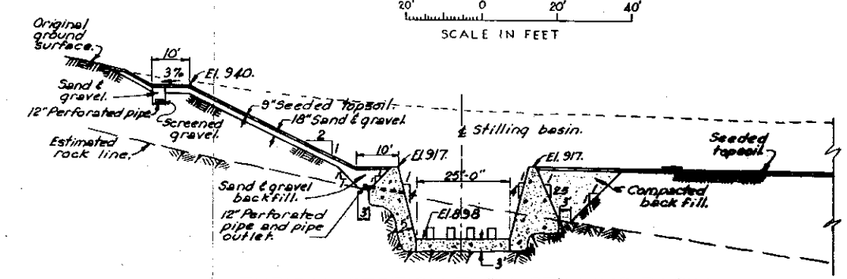


PLAN OF EQUIPMENT ROOM



SECTION B-B

12' 0 5' 10' 15' 20'
SCALE IN FEET
(UNLESS OTHERWISE NOTED)



TYPICAL SECTION THRU STILLING BASIN

20' 0 20' 40'
SCALE IN FEET

MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
NUBANUSIT BROOK

SECTIONS OF OUTLET WORKS

U. S. ENGINEER OFFICE, BOSTON, MASS. JUNE 1946

APPROVAL RECOMMENDED:
CHIEF, ENGINEERING DIVISION

SUBMITTED:
CHIEF, DESIGN BRANCH, ENGINEERING DIVISION

APPROVED:
COLONEL, CORPS OF ENGINEERS
DISTRICT ENGINEER

NO.	CHARACTER	DATE	APPR.

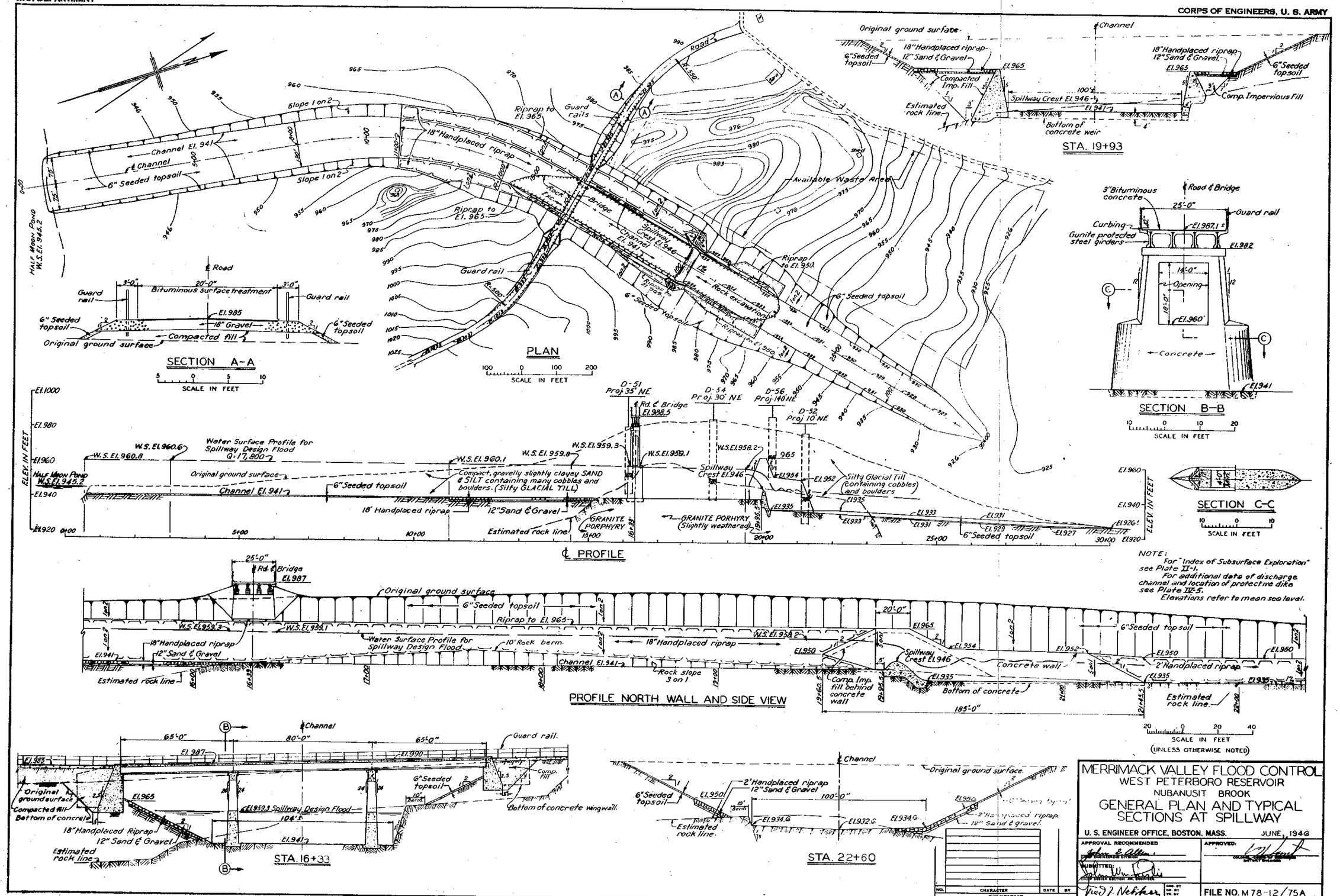
REVISIONS

FILE NO. M78-12/74A

THIS SHEET SUPERSEDES DWG. NO. M78-12/71

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY



NOTE:
For "Index of Subsurface Exploration" see Plate II-1.
For additional data of discharge channel and location of protective dike see Plate IV-5.
Elevations refer to mean sea level.

MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
NUBANUSIT BROOK
GENERAL PLAN AND TYPICAL SECTIONS AT SPILLWAY

U. S. ENGINEER OFFICE, BOSTON, MASS. JUNE, 1946

APPROVAL RECOMMENDED: John E. Allen, Engineering Division
SUBMITTED: John W. Leslie, Chief Design Section, Sr. Engineer
APPROVED: Colonel, Corps of Engineers, District Engineer

REVISIONS: NO. | CHARACTER | DATE | BY

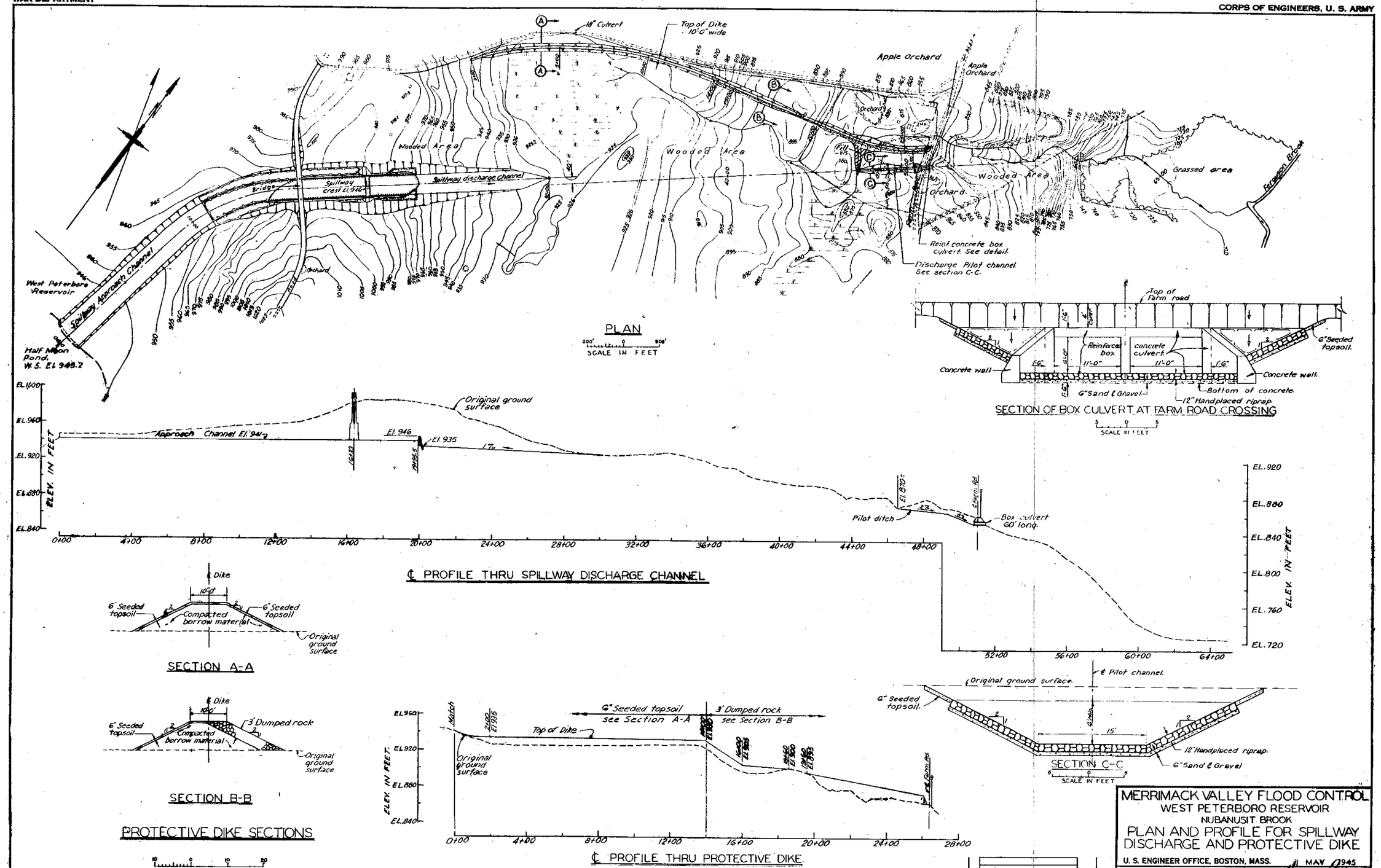
FILE NO. M 78-12/75A

THIS SHEET SUPERSEDES SHEET NO. M 78-12/75

PLATE IV-4

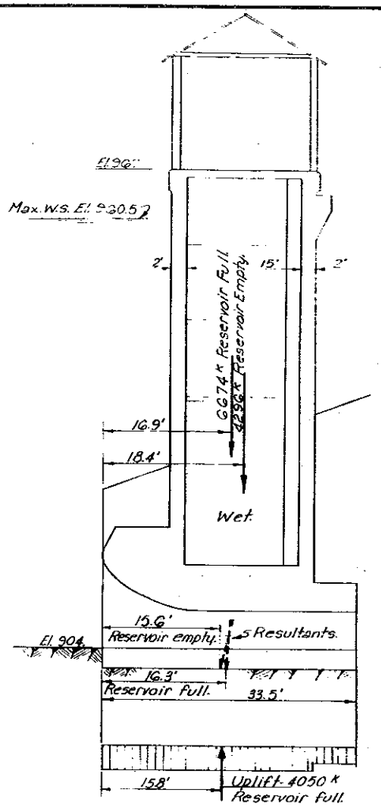
WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY

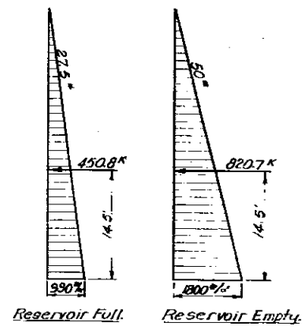


THIS SHEET SUPERSEDES SHEET NO. M78-12/76

PLATE IV-5.

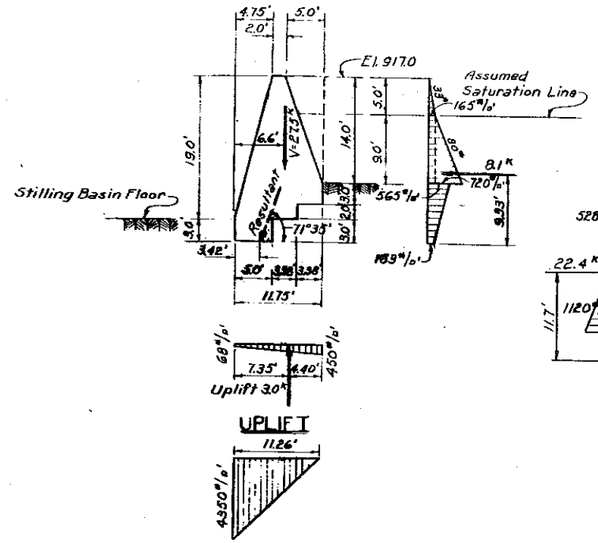


UPLIFT

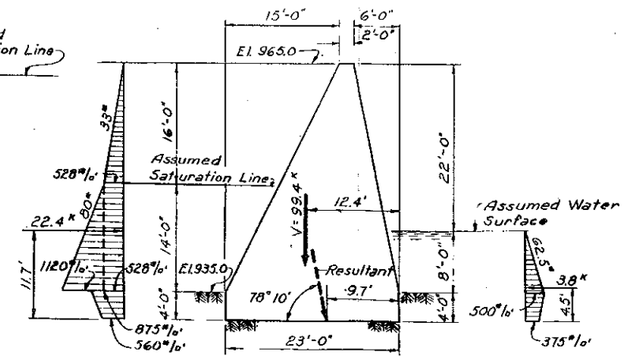


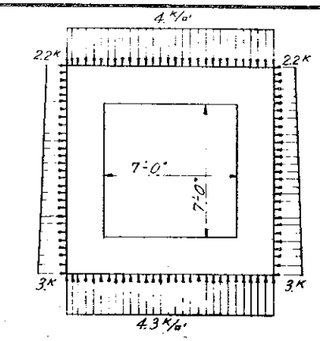
LATERAL EARTH PRESSURE

Note:- Hydrostatic pressures balance and contribute no overturning.

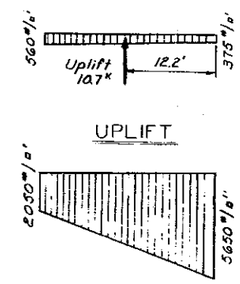


BEARING
STABILITY F.S. 2.1
STILLING BASIN





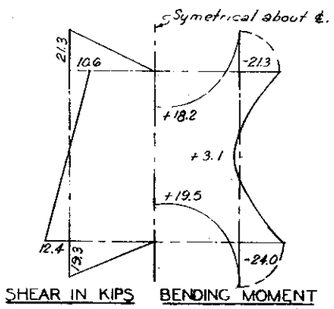
LOADING - CONDUIT EMPTY



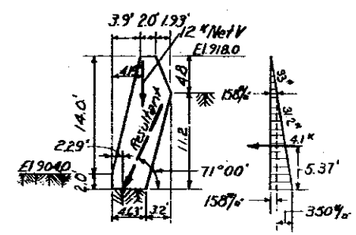
UPLIFT

BEARING
STABILITY F.S. 4.5

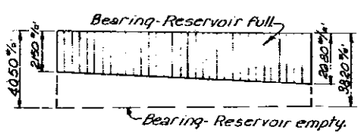
SPILLWAY CHANNEL WALL



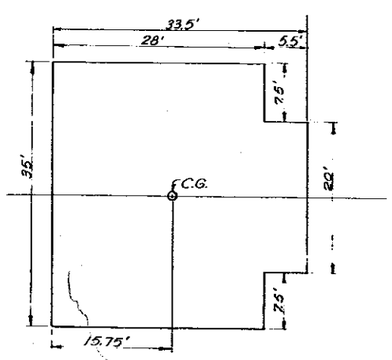
CONDUIT



STABILITY F.S. 1.58
APPROACH CHANNEL



BEARING



PLAN

DESIGN ASSUMPTIONS AND GENERAL NOTES

Unit Weights	
Concrete	*150#/cu. ft.*
Water	*625#/cu. ft.*
Dry Earth	*125#/cu. ft.*
Saturated Earth	*140#/cu. ft.*
Equivalent Fluid Pressures	
Water	*625#/sq. ft.*
Dry Earth	*33#/sq. ft.*
Saturated Earth	*80#/sq. ft (unless otherwise noted)*

Uplift between rock and concrete - 100% over 50% of area.

10 0 10 20
SCALE IN FEET
(UNLESS OTHERWISE NOTED)

SCALE OF PRESSURE DIAGRAMS
HOR. 2000 FT. 0 2000. 4000 FT.
VERT. 4000 FT. 0 4000. 8000 FT.

NO	CHARACTER	DATE	APPR'D
1	General Revisions	6/7/62	

REVISIONS

MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
NUBANUSIT BROOK

STABILITY ANALYSES
CONCRETE STRUCTURES

U.S. ENGINEER OFFICE, BOSTON, MASS. MAY 1945

APPROVAL RECOMMENDED: John E. Allen
CHIEF ENGINEERING DIVISION
SUBMITTED: J. Brown
CHIEF DESIGN BRANCH, SR. ENGINEER
ENGINEER
DES. BY
DR. BY
T.P. BY
CK. BY
APPROVED: John E. Hall
COLONEL, CORPS OF ENGINEERS
DISTRICT ENGINEER

FILE NO. M78-12/77

PLATE IV-6

WAR DEPARTMENT
United States Engineer Office
Boston, Massachusetts

DEFINITE PROJECT REPORT

WEST PETERBORO RESERVOIR

APPENDIX V

HYDROELECTRIC POWER

To accompany definite project report
dated May 1945 (Revised June 1946)

DEFINITE PROJECT REPORT

WEST PETERBORO RESERVOIR

APPENDIX V - HYDROELECTRIC POWER

C O N T E N T S

Paragraph	Title	Page
a.	Structure	V-1
b.	Estimated Cost	V-1
c.	Basis of Cost Analysis	V-2
d.	Conclusion	V-2

DEFINITE PROJECT REPORT
WEST PETERBORO RESERVOIR

APPENDIX V. - HYDROELECTRIC POWER

1. Studies.- The results of studies undertaken to determine the feasibility of obtaining additional storage capacity for the development of a multiple-purpose reservoir indicate that 31,200 acre-feet is the optimum capacity that could be obtained at a minimum cost per acre-foot of yield. This additional storage would provide a regulated minimum flow of 60 c.f.s. for stream regulation. The increased storage could be obtained by raising the dam and spillway to provide a total storage capacity of 44,000 acre-feet, of which 31,200 acre-feet would be for conservation storage and 12,800 acre-feet for flood control purposes. An economic study of utilizing this additional storage capacity for stream regulation, with and without hydroelectric development at the site, has been made and the results summarized in Table A accompanying this appendix.

2. Estimated Cost.-

a. Additional Construction and Land-Takings.- Development of the reservoir for multiple-purpose use, undertaken as a second-stage project subsequent to the completion of the flood control reservoir, would involve the acquisition of 1,020 acres of additional land. This additional area includes the land and buildings of the summer camp owned by the Sargent School of Physical Education, Boston University, and the relocation of an additional 5.6 miles of highway. In addition, it would be necessary to acquire the water rights of the existing 1,000 horsepower industrial installation just below the proposed West Peterboro site. The principal items of construction involved in the second-stage development would be raising the saddle spillway weir by 22 feet, raising the earth dam and gate house, including the access bridge, by 18 feet and the construction of power generation facilities.

b. Costs.- The estimated cost of second-stage development for (b) flood control and conservation storage is $1,387,000 with an annual carrying charge of $55,910 and (a) flood control, conservation storage and hydroelectric power development at the site is $2,051,000 with an annual carrying charge of $94,760. The cost of the hydroelectric power development is based on a plant with 1600 KW installed capacity and a 30 percent load factor, operating with an average net head of 143 feet, and having a 3700 foot penstock, with surge tank.

3. Basis of Cost Analysis. - The evaluation of power benefits to be derived by a hydroelectric plant located at the proposed dam site and the existing downstream hydroelectric installations with the reservoir operated to increase the low water flows, has been based on the estimated costs for the production of equivalent power by a steam plant located at a load center such as Manchester, New Hampshire. The construction cost of such a steam plant is estimated to be $125 per KW., and the annual cost, consisting of the fixed charges and allowance for operation and maintenance, is estimated to be $17.50 per kilowatt per year. This amount is the value assigned per kilowatt of dependable prospective hydroelectric capacity. The value of energy output, based on the current cost of coal fuel at Manchester, is estimated at about 3 mills per kilowatt hour. An allowance of 10% has been made for losses in transmission.

4. Conclusion. - The studies of the benefits to be derived from the development of a multiple-purpose reservoir and the current high level of construction costs, indicate that second-stage construction cannot be economically justified. The ratio of annual benefits to carrying charges for stream regulation storage is 0.59 and for stream regulation and hydroelectric power generation at site is 0.66. These low ratios do not warrant making provision in the initial construction for the ultimate development of the reservoir for multiple-purpose use.

TABLE A

COST ANALYSIS - 2ND STAGE CONSTRUCTION FOR FLOOD CONTROL AND POWER STORAGE

	Plan 1 Raising of Dam and Spillway For Stream Regulation Storage	Plan 2 Same as Plan 1 with Power Installation at Site
1. RESERVOIR DATA		
Top of Dam - Elev.	985	985
Saddle Spillway - Elev.	968	968
Flood Control Storage Capacity	12,800 A.F. 5.5"	12,800 A.F. 5.5"
Conservation Storage Capacity	31,200 A.F. 13.3"	25,000 A.F. -El. 961
Storage at Max. Drawdown		6,200 A.F. -El. 932
Total Storage Capacity	44,000 A.F. 18.8"	44,000 A.F. 18.8"
Average Annual Yield	24,000 A.F.	22,000 A.F.
Water Utilization Factor	80%	60%
2. ESTIMATED COST OF 2ND STAGE CONSTRUCTION (Including Overhead and Contingencies)		
Cost of Reservoir and Dam:		
Reservoir Costs (incl. Bldgs. and road relocations)	$ 780,000	$ 780,000
Earth Dam and Outlet	370,000	370,000
Saddle Spillway	90,000	90,000
Reservoir Clearing	300,000	300,000
Sub-total	$1,540,000	$1,540,000
Cost of Power Installation for 1,600 K.W. Capacity		760,000
TOTAL ESTIMATED COST OF 2ND STAGE CONSTRUCTION	$1,540,000	$2,300,000
Cost per acre foot of conservation storage	$49.36	$61.60
Cost per acre foot of Average Annual Yield	64.17	70.00

TABLE A (Cont'd.)

COST ANALYSIS - 2ND STAGE CONSTRUCTION
FOR FLOOD CONTROL AND POWER STORAGE

	Plan 1 Raising of Dam and Spillway For Stream Regulation Storage	Plan 2 Same as Plan 1 with Power Installation at Site
3. ANNUAL CARRYING CHARGES OF 2ND STAGE CONSTRUCTION COST	$62,000	$105,000
4. ANNUAL POWER BENEFITS		
(a) Power Installation at Site:		
1. 1,600 K.W. @ 17.50		$ 28,000
2. 4,500,000 K.W.H. @ 3 Mills		$ 13,500
(b) Increased Output at Existing Downstream Plants		
Increase in prime peaking capacity		
1,000 K.W. @ 17.50	$ 17,500	
800 K.W. @ 17.50		$ 14,000
Increase in average annual energy output with existing head downstream of 411 feet.		
6,400,000 K.W.H. @ 3 Mills	$ 19,200	
4,500,000 K.W.H. @ 3 Mills		$ 13,500
TOTAL ANNUAL POWER BENEFITS	$ 36,700	$ 69,000
5. RATIO OF ANNUAL BENEFITS TO ANNUAL CARRYING CHARGES	0.59	0.66

War Department
United States Engineer Office
Boston, Massachusetts

DEFINITE PROJECT REPORT

WEST PETERBORO RESERVOIR

APPENDIX VI

RELOCATIONS

To accompany definite project report
dated May 1945

DEFINITE PROJECT REPORT

WEST PETERBORO RESERVOIR

APPENDIX VI - RELOCATION

C O N T E N T S

Paragraph	Title	Page
a.	Introduction	VI-1
b.	Roads	VI-1
c.	Bridges	VI-1
d.	Power and Telephone Lines	VI-2
e.	Procedure	VI-2
f.	Railroads	VI-2
g.	Cemeteries	VI-2
h.	Pipe and Water Supply Lines	VI-2
i.	Water Rights	VI-2
j.	Method of Accomplishing Relocations	VI-2
k.	Source of Information	VI-2

PLATES

Plate	Title
VI-1	Relocations - Roads and Utilities

DEFINITE PROJECT REPORT
WEST PETERBORO RESERVOIR

APPENDIX VI - RELOCATIONS

a. Introduction. - The reservoir basin at spillway crest elevation 946 contains only short sections of roads and a small number of bridges and utilities which will be affected by the inundation of the area during periods of high water stages. Approximately one mile of a third class road, with accompanying power and telephone lines, requires relocations and one-half mile of third class road requires raising. Several small wooden bridges must be raised to provide the necessary clearance for flood flows and a box culvert installed on the Sargent School Road, as indicated on Plate VI-1. In order to provide a connection for High Wind Road at the spillway, it is proposed to construct a steel girder bridge over the spillway approach channel 370 feet upstream from the weir. The total cost of the above-noted relocations is estimated to be $107,500, excluding the steel girder bridge which is included in the construction costs.

b. Roads. - The third class gravel road running parallel to and on the east side of the brook between Swetts Dam Bridge and the proposed dam, is subject to inundation for a distance of approximately one mile and is to be abandoned. It is proposed to replace this portion of the road by regrading and resurfacing an existing farm road which runs from Swetts Dam Bridge to High Wind Road which is approximately one-half mile east of the abandoned road. This farm road is to be improved by regrading and surfacing with gravel.

Low sections in the existing third class gravel surfaced road passing the Sargent School Camp are to be raised approximately three (3') feet in elevation for a distance of about one-half mile and the culvert at the outlet from Half Moon Pond is to be replaced and lengthened.

Highway traffic can be maintained during the various stages of construction by rerouting until such time as the proposed raising of roads and relocations are completed.

The Chief Engineer of the Highway Department, State of New Hampshire has been consulted and has agreed that the relocations proposed are a reasonable solution for raising and relocating the network of roads affected by construction of the proposed dam.

c. Bridges. - The Swetts Dam Bridge, the bridge over Beaver Brook and the two bridges over upper Nubanusit Brook are to be raised from three to five feet in elevation by reconstructing and adding to their masonry abutments. The decks of these bridges are constructed of wood planks laid on steel girders.

The cost of constructing the proposed new concrete box culvert crossing Farm Road as illustrated on Plate IV-4 is included in the estimated cost of the spillway.

d. Power and Telephone Lines.- The pole line paralleling the road through the dam site carries two power lines and two telephone lines. This pole line is to be relocated to connect with an existing pole line along High Wind Road. The pole line leading to Sargents Camp is to be raised in elevation where it is adjacent to the road sections which are to be raised.

e. Procedure.- It is anticipated that the relocation and raising of roads and utilities will be accomplished by the respective owners who will be reimbursed by the Government for such work. The roads, bridges and culverts are owned by the Town of Peterboro, the power lines by the Public Service Company of New Hampshire and the telephone lines by the New England Telephone and Telegraph Company.

f. Railroads.- There are no railroads within the proposed reservoir requiring relocation.

g. Cemeteries.- There are no cemeteries within the proposed reservoir basin.

h. Pipe and Water Supply Lines.- There are no major pipe or water supply lines within the area of the proposed reservoir basin.

i. Water Rights.- The Verney Mills, Inc., of Manchester, N.H., own water rights in Nubanusit Brook and this dam is situated approximately 1000 feet downstream from the proposed dam site. The consideration in this instance will be furnishing the Verney Mills, Inc., with as much water as they are now receiving.

j. Method of Accomplishing Relocations.- Where it is necessary to relocate utilities it is proposed to have the work accomplished by the respective owner through contractual arrangements.

k. Source of Information.- In accordance with the requirements of Circular Letter No. 3570, Real Estate No. 62, dated 21 February 1945, subject,"Real Estate Functions of Division Offices", the method of disposing of utilities lying in the reservoir area was developed in collaboration with representatives of the Real Estate Division of the Office of the Division Engineer, New England Division. The general description of the real estate involved and data as to its acquisition cost were taken from the "Report-Real Estate Cost, Bennington Reservoir,N.H.", dated 21 April 1945, prepared by the Division Engineer, New England Division.

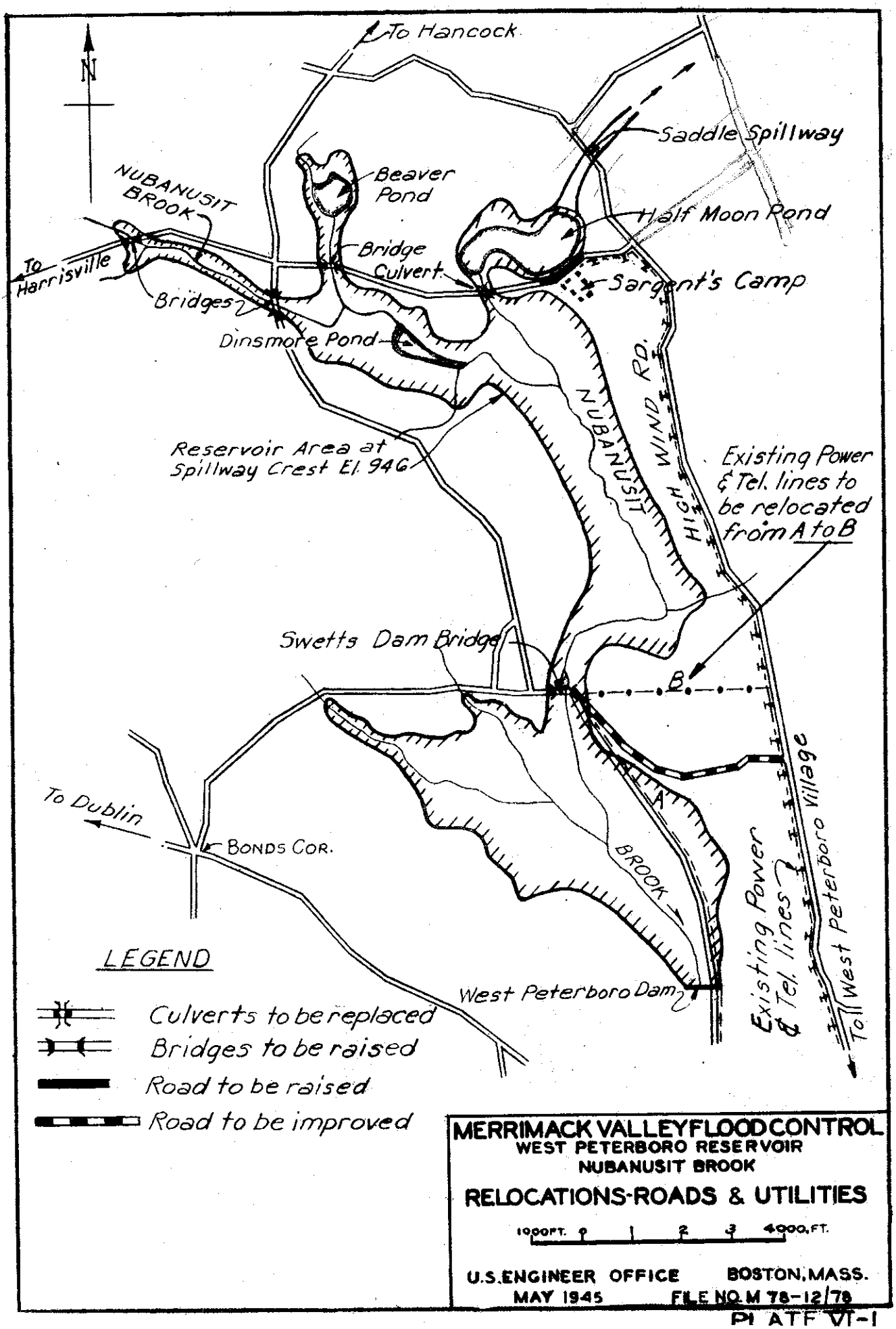

N
To Hancock
Saddle Spillway
Beaver Pond
NUBANUSIT BROOK
Half Moon Pond
Bridge Culvert
To Harrisville
Sargent's Camp
Bridges
Dinsmore Pond
NUBANUSIT
HIGH WIND RD.
Reservoir Area at Spillway Crest El. 946
Existing Power & Tel. lines to be relocated from A to B
Swetts Dam Bridge
B
A
To Dublin
BONDS COR.
BROOK
Existing Power & Tel. lines
To West Peterboro Village
West Peterboro Dam
LEGEND
Culverts to be replaced
Bridges to be raised
Road to be raised
Road to be improved
MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
NUBANUSIT BROOK
RELOCATIONS-ROADS & UTILITIES
1000 FT. 0 1 2 3 4000 FT.
U.S. ENGINEER OFFICE BOSTON, MASS.
MAY 1945 FILE NO. M 78-12/78

PLATE VI-1

WAR DEPARTMENT
UNITED STATES ENGINEER OFFICE
BOSTON, MASSACHUSETTS

DEFINITE PROJECT REPORT

WEST PETERBORO RESERVOIR

APPENDIX VII

MALARIA CONTROL STUDIES

(To accompany definite project report dated
May 1945 (Revised June 1946))

DEFINITE PROJECT REPORT
WEST PETERBORO RESERVOIR

APPENDIX VII - MALARIA CONTROL STUDIES

CONTENTS

Paragraph	Title	Page
a	Scope	VII-1
b	Incidence of Malaria	VII-1
c	Biological Factors	VII-1
d	Recommendations for Malaria Control	VII-1
e	Final Survey	VII-1

DEFINITE PROJECT REPORT
WEST PETERBORO RESERVOIR

APPENDIX VII. - MALARIA CONTROL STUDIES

a. Scope.- This appendix presents the pertinent data contained in the Reconnaissance Malaria Survey Report of the Proposed West Peterboro Reservoir made in November 1945 by U. S. Public Health Service at the request of this office. The report, based on field investigations by the U. S. Public Health Service, was forwarded to this office by letter of 22 March 1946.

b. Incidence of Malaria.- Records of the U. S. Public Health Service Division of Public Health Methods show no cases of malaria in the State of New Hampshire from 1932 to 1941 and four (4) cases from 1941 to 1945. The data do not indicate where the four (4) listed infections were acquired.

c. Biological Factors.- No Anopheles quadrimaculatus adults were found in the reservoir vicinity during an entomological survey in August 1945, though this mosquito is known to inhabit this region. No anopheline larvae were found in the places examined.

d. Recommendations for Malaria Control.- The report made by the Public Health Service contains two recommendations for Malaria control.

"(1) The reservoir basin should be cleared to a contour 2 feet above the maximum conservation pool level.

"(2) Provision should be made for the annual removal of floatage and drift. This may be accomplished by a spring surcharge followed by a rapid drawdown to strand the debris which should then be collected and disposed of. Timber and brush killed by inundation during the growing season should be removed from the flood control zone of drawdown."

e. Final Survey.- The U. S. Public Health Service does not believe that further studies will be necessary.

(Rev. June 1946)

SUPERSEDED DATA

DEFINITE PROJECT REPORT

WEST PETERBORO RESERVOIR

MERRIMACK VALLEY FLOOD CONTROL

TABLE OF CONTENTS

SECTION A	PERTINENT DATA	Pages a1 to a3
SECTION B	SYLLABUS	Page b1
SECTION C	TEXT OF REPORT:	

Paragraph	Title	Page
1.	Project Authorization	1
2.	Investigations	3
3.	Local Cooperation	5
4.	Definite Project Plan	6
5.	Structures and Improvements	9
6.	Foundations	10
7.	Hydroelectric Power	11
8.	Relocations	11
9.	Availability of Construction Materials	12
10.	Construction Time Required and Schedule of Operations	12
11.	Clearance with Other Agencies	14
12.	Operation and Maintenance	15
13.	Malaria Control	16
14.	Cost Estimates	17
15.	Economic Study	21
16.	Recommendations	23

SECTION D APPENDICES:

Appendix	Title	Page
I	Hydrology	I-1 to I-18
II	Exploration, Geology, Construction Materials, Soil Data and Analyses	II-1 to II-18
III	Hydraulic Design	III-1 to III-4
IV	Structural Design	IV-1 to IV-3
V	Hydroelectric Power	V-1 to V-4
VI	Relocations	VI-1 to VI-2

SUPERSEDED

DEFINITE PROJECT REPORT
WEST PETERBORO RESERVOIR

SECTION A - PERTINENT DATA

There is presented in the following table a summary of the pertinent data and governing criteria for the selection, design, construction, operation and cost of the West Peterboro Reservoir, Merrimack River Basin, New Hampshire, as described in this project report:

1. Project Location . . . . . Nubanusit Brook, tributary of Contoocook River approximately 15 miles east of Keene, New Hampshire, and 2.5 miles northwest of Peterboro, New Hampshire, approximately at river mile 161.3 above the mouth of the Merrimack River.

2. Reservoir Data

Drainage area	44 sq. miles
Flood control storage capacity	12,800 acre-feet
Inches of storage capacity on drainage area	5.5
Reservoir area at elevation of spillway crest:	
Wooded land, containing thin stand of mixed hardwood (45%)	382 acres
Wooded land, containing fair stand of mixed hard and soft woods, small amount merchantable (18%)	153 acres
Tillable and cleared land (4%)	34 acres
Water and swamp land (33%)	281 acres
Total Reservoir Area (100%)	850 acres
Length of Reservoir	3.7 miles
Maximum wave fetch	1.5 miles

3. Stream Flow Data

Average flow	80 c.f.s.
Average annual flood	800 c.f.s.
Flood of 5 November 1927 (estimated)	1,000 c.f.s.
Flood of 19 March 1936	4,140 c.f.s.
Flood of 21 September 1938	4,100 c.f.s.
Spillway design flood - inflow to reservoir . .	36,300 c.f.s.
Spillway design flood - outflow	18,200 c.f.s.

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4. Dam

Type	Rolled earth
Top of dam elevation	967 feet (M.S.L.)
Total length of dam	1,100 feet
Maximum height of earth embankment	69 feet
Top width of earth dam	25 feet
Freeboard	5.5 feet
Allowance for failure of upstream dams	1.0 foot
Power development	None

5. Outlets

Location	Single conduit through embankment
Size of conduit	5'-9" x 5'-9"
Number of service gates	2
Type of gates	Hydraulically operated sluice gates
Size of gate openings	3'-3" x 5'-9"
Elevation of gate sills	904 feet (M.S.L.)
Maximum discharge capacity	1,050 cubic feet per second
Downstream channel capacity	1,000 cubic feet per second
Time of emptying 80% of capacity	8 days

6. Spillway

Type of spillway	Low concrete weir
Crest elevation	946 feet (M.S.L.)
Length	100 feet
Spillway design flood discharge	18,200 cubic feet per second
Maximum reservoir elevation (Spillway design flood)	960.5 feet (M.S.L.)

7. Foundations

Dam, General	Silty till.
Dam, Core	Bed rock on west abutment and stream channel - silty till on east abutment.
Outlet works	Bed rock.
Spillway	Bed rock.

8. Quantities

Embankment:	Earth Dam	Spillway
Impervious fill	85,000 C.Y.	40,000 C.Y.
Pervious fill	107,000 C.Y.	-
Rock slope protection	19,000 C.Y.	11,500 C.Y.
Sand and gravel fill	24,500 C.Y.	4,000 C.Y.
Total Embankment	235,500 C.Y.	55,500 C.Y.

SUPERSEDED

8. Quantities (Cont'd.)

	Earth Dam	Spillway
Excavation - earth	135,000 C.Y.	260,000 C.Y.
Excavation - rock	9,000 C.Y.	48,000 C.Y.
Concrete	4,200 C.Y.	5,600 C.Y.

9. Estimated Cost (Including Contingencies Overhead, etc.)

Reservoir clearing	$ 4,500
Land, relocations, etc.	193,000
Construction of dam and spillway	1,196,500
Total	$ 1,394,000

Cost per Acre-Foot of Total Storage . . . . . $ 108.91

SUPERSEDED

DEFINITE PROJECT REPORT

WEST PETERBORO RESERVOIR

SECTION B - SYLLABUS

The West Peterboro Reservoir was approved by the Chief of Engineers on 30 April 1940 as a part of the comprehensive flood control plan for the Merrimack River Basin authorized by the Flood Control Act approved 22 June 1936 and amended by the Flood Control Act approved 28 June 1938.

The reservoir site is located on the Nubanusit Brook, New Hampshire, and is designed to protect the lower reaches of the Nubanusit Brook and the town of West Peterboro from all floods, and to reduce flood stages on the Contoocook River and the Merrimack River from floods originating in the drainage basins of the Nubanusit and Stanley Brooks. The proposed dam consists of an earth-filled embankment section with the concrete outlet works at the dam site on the south side of the reservoir and a saddle spillway with an earth-filled dike on the north side. The embankment section is 1100 feet long and has a maximum height of 69 feet. It is constructed of rolled earth fill with a central impervious earth core which extends to rock on the west side and into the impervious till on the east side of the present river. The upstream face of the embankment is protected by a rock blanket; the downstream slope is topsoiled and seeded, and the toe drain is covered with a layer of hand-placed riprap. The outlet works include an intake structure, conduit and stilling basin. The conduit passes through the embankment and has two gated outlets. The intake structure is connected with the embankment by an access bridge. The spillway is located in a remote saddle and is a concrete gravity section which discharges onto an excavated rock floor. The training walls are gravity type sections with a concrete highway bridge spanning between them directly over the spillway crest. An earth-filled dike is provided downstream from the spillway discharge channel to prevent flooding of the existing orchards in that region and, also, to restrict spreading and to guide the flow of water directly into Ferguson Brook. The estimated cost for construction of the dam, including the operator's quarters, is $1,394,000, based on a construction period of two years. Local financial cooperation is not required for this project. This definite project is currently re-written to incorporate modifications in the design of the dam and spillway. Authorization for preparation of a definite project report on the West Peterboro Reservoir is contained in telegram from the North Atlantic Division Office, New York, N. Y., dated 1 May 1940

SUPERSEDED

emphasis on the Riverhill site, located approximately 7 miles north westerly of the city of Concord, New Hampshire, for control of the Contoocook River. The Riverhill site was abandoned due to local opposition, and projects for the construction of the West Peterboro, Mountain Brook and Hopkinton-Everett Reservoirs were substituted.

Preliminary definite project reports on Mountain Brook and West Peterboro Reservoirs were submitted on 25 April 1940 as a part of the comprehensive flood control plan, and were approved by the Chief of Engineers on 30 April 1940. The preliminary report on the West Peterboro Reservoir proposed locating the dam at the southerly side and a remote saddle spillway on the northerly side of the reservoir as recommended in this report.

At a hearing held by the New Hampshire Water Resources Board in Concord, New Hampshire, on 8 May 1940, the Federal Power Commission proposed a plan of multiple-purpose development for the Contoocook River Basin as an alternate to the flood control plan of development proposed by this office. The alternate plan did not affect the West Peterboro Reservoir since this project was included in both plans of development

c. Previous Definite Project Plan.- Subsequent to the submission of the preliminary definite project report, dated 25 April 1940, bedrock was discovered in the west bank of the Nubanusit Brook, about 1000 feet upstream from the Verney Mills Dam. The layout of the reservoir was then modified to locate the spillway on the underlying bedrock of the west abutment at the upstream dam site. This relocation of the spillway from the saddle site at the northerly end of the reservoir to the dam site reduced the estimated cost of the project from $1,300,000 to $1,011,000. A definite project report incorporating these modifications was submitted 6 August 1940, and was revised and resubmitted 14 December 1940 to include modifications suggested by the Chief of Engineers. The cost of the modified project as submitted was $1,170,000.

d. Revised Definite Project Plan.- Further geological investigations, made after submission of the definite project report dated 5 August 1940 (Revised 14 December 1940), indicated that the rock on the west bank of the brook is badly weathered, that a concrete lining of the spillway discharge channel would be required, and that the excavated rock would be unsatisfactory for use as slope protection riprap. These factors increased the estimated cost of the approved project to the estimated cost of the original plan which included the saddle spillway. Consideration of the equal costs, together with the direct benefits to be derived in complete downstream protection afforded by the diversion of the spillway

SUPERSEDED

2. West Peterborough Reservoir on the Nubanusit Brook in the Towns of Peterborough, Dublin, Harrisville and Hancock, in accordance with "Definite Project Report for West Peterborough Reservoir", dated August 5, 1940, revised December 14, 1940.

which report is on file and open to public inspection in the records of the New Hampshire Water Resources Board."

The design of the dam has been modified by relocating the spillway from the dam site to the saddle between Half Moon Pond and Ferguson Brook subsequent to the passage of the above resolution. Approval by the State of the acquisition of land for the West Peterboro Reservoir in accordance with this revised definite project plan was requested by letter dated 10 May 1945 to the Honorable Charles M. Dale, Governor of the State of New Hampshire.

c. Verney Mills Power Pool.- The proposed West Peterboro Dam site is located approximately 1000 feet upstream from the storage and intake dam of the Verney Mills, Inc., and is in the existing power pool formed by the dam. It is proposed to replace the existing 18-inch flashboards by 36-inch flashboards on the existing Verney Mills Dam to compensate for the reduction in power storage caused by the construction of the flood control dam. Representatives of the Verney Mills, Inc., have been consulted and have agreed informally to the proposed plan. A formal agreement is being prepared by the legal staff of the Verney Mills, Inc., and will be incorporated as an appendix to this report when the formalities are completed.

4. Definite Project Plan.-

a. General.- The West Peterboro Reservoir is one of several reservoirs comprising the comprehensive plan for flood control reservoirs and related flood control works for the Merrimack River Basin. The reservoir controls 90 percent of the drainage area of the Nubanusit Brook.

b. Location.- The proposed reservoir is located on the Nubanusit Brook, New Hampshire, in the towns of Peterboro, Dublin, Hancock and Harrisville, in the southwestern part of the State of New Hampshire. The dam is situated approximately one-half mile upstream (north) of the village of West Peterboro.

c. Description of Reservoir Area.- A large part of the land within the reservoir basin is wooded and covered with a second growth composed mainly of white pine, hemlock, white and red maple, gray and silver birch, and white and red oak. Scattered about are small clumps of white pine and hemlock of

SUPERSEDED

sufficient size to be merchantable, but there is not enough mature timber on any one unit to make commercial operations profitable. It is very noticeable that within the thin stands of birch, and the like, there is a good undergrowth of young white pine. These young trees appear thrifty and are making good growth.

The few cleared acreages are rather small in area and contain large boulders that make tillage operations difficult. Only three acres of the thirty-four acres of tillable land within the proposed reservoir have been used for tillage crops within the past years, and most of the fields have been producing little more than a hay crop for a number of years.

d. Design Floods.-

(1) Reservoir Design Flood.- The two largest floods of record on Nubanusit Brook occurred in March 1936 and September 1938. The larger of these, March 1936, had a peak discharge of 4140 c.f.s. and an estimated volume of about 35,000 acre-feet in a 16-day period, equivalent to about 15.0 inches of run-off over the watershed. The September 1938 flood had a peak discharge of 4100 c.f.s. and an estimated volume of about 15,000 acre-feet in a 6-day period, equivalent to about 6.5 inches of run-off over the watershed. These floods were used in studies of storage capacity.

(2) Spillway Design Flood.- The storm used for the spillway design flood comprises the computed maximum possible rainfall over the 44 square mile drainage area, or 19.5 inches of rainfall in 12 hours with 17.3 inches occurring in 6 hours. Run-off was computed by means of a synthetic unit hydrograph, assuming an infiltration rate of .05 inches per hour and base flow of 220 c.f.s., or 5 c.f.s. per square mile over the watershed. The resulting spillway design flood has a peak inflow of 36,300 c.f.s. and a volume of 18.9 inches over the watershed. The maximum spillway discharge when routed through the reservoir is 18,200 c.f.s. at a maximum reservoir stage of 960.5 or 14.5 feet above spillway crest.

e. Project Description.- As shown on Plates IV-1 to IV-5 accompanying Appendix IV of this report, the impounding structures consist of a low concrete weir overflow section and a rolled earth embankment. The weir is founded on bed rock, in a natural saddle on the northerly side of the reservoir. The spillway approach and discharge channel is mostly cut through rock and has an earth dike on the northerly side to train the spread of the discharge and protect existing orchards. The outlet works are incorporated into the earth embankment section. The outlet works include a reinforced concrete intake structure, conduit

SUPERSEDED

and stilling basin with gravity type guide walls, all of which are founded on rock. The proposed dam has a crest length of 1100 feet and is founded on bed rock on the west bank and river section, and on glacial till on the east bank of the river. It is proposed to construct the dam for flood control storage only, with no provision made for ultimate raising. The reservoir, which controls a drainage area of 44 square miles, has an area of 850 acres at spillway crest elevation 946. The reservoir has a capacity of 12,800 acre-feet which is equivalent to 5.5 inches of run-off on the drainage area.

f. Estimated Cost.- A detailed estimate of cost for construction of the reservoir is given in Paragraph 14. A summary of the estimated cost is as follows:

Reservoir Costs	$ 193,000.
Reservoir Clearing	4,500.
Construction Costs	1,196,500.
Total Estimated Cost	$ 1,394,000.

g. Method of Operation.- The West Peterboro Reservoir will be operated primarily to regulate the flood discharges of the Nubanusit Brook so as to provide maximum benefits at downstream damage centers and to reduce flows in the brook below the Verney Mills Dam to amounts equal to or less than channel capacity. Sufficient water will be discharged at all times to provide storage equivalent to the existing storage for use of the Verney Mills, Inc.

h. Alternative Layouts.- A number of engineering studies have been made for various locations within the vicinity of the proposed dam site and considerable drill hole information has been obtained for the embankment and saddle spillway locations. Detailed analyses of two different layouts were previously presented in the following reports: (a) Preliminary Definite Project Report, dated 25 April 1940, which had the embankment and outlet works located directly over the existing Verney Mills Dam and the spillway placed in the saddle at the northerly end of the reservoir, and (b) Definite Project Report, dated 5 August 1940 (revised 14 December 1940), which had the embankment, spillway and outlet works located approximately 1000 feet upstream from the Verney Mills Dam.

i. Ultimate Development.- Provision is not being made in the design for ultimately raising the dam, in view of the unfavorable results of analysis of this project for future conservation storage in addition to flood control storage, as presented in Appendix V.

SUPERSEDED

5. Structures and Improvements.-

a. Embankment.- The embankment is designed as an earth fill section consisting of a compacted impervious core backed with compacted random fill and flanked with a shell of compacted pervious fill, as indicated in the sections on Plate IV-2. The upstream slope and downstream drainage toes are protected by stone blankets. The embankment section has a top width at elevation 967 of 25 feet; an upstream slope of 1 on 2.75; a downstream slope of 1 on 2.5; a maximum height of 69 feet and a total length of 1100 feet. The impervious core cut-off extends to bed rock on the west abutment and into the underlying glacial till on the east abutment between Station 5+50 and the easterly end of the embankment. An upstream impervious blanket tied into the core is provided on the upstream side of the dam, as indicated on Plates IV-1 and IV-2.

b. Outlet Works.- The outlet works consist of an intake gate house, reinforced concrete conduit and stilling basin which are founded entirely on rock. The gate house is a reinforced concrete structure which contains two outlets, 3'-3" x 5'-9", each of which is equipped with two hydraulically operated sluice gates for control of the discharge. A stop log recess is provided in the event of failure of the hydraulically operated gates and for use during maintenance operations. Access is gained to the equipment room of the gate house from the top of the dam by means of a structural steel bridge. The gate chamber outlets converge into a single reinforced concrete conduit, 5'-9" x 5'-9", which passes through the embankment and discharges into the stilling basin. With the exception of the guide walls which are gravity type concrete, the stilling basin is composed of a reinforced concrete mat, baffles and end sill, all of which are founded on rock. See Plate IV-6 for design analysis of masonry structures. The discharge channel is an earth cut with riprapped side slopes, terminating in the pool created by the Verney Mills Dam. The existing 18-inch flashboards on the Verney Mills Dam are to be replaced with 36-inch flashboards to compensate for the reduced pondage as a result of the construction of the flood control dam.

c. Spillway.- The spillway is located in a natural saddle separating the Nubanusit Brook and Ferguson Brook drainage areas. The spillway is a low concrete weir founded on bed rock with a crest length of 100 feet at elevation 946. The approach channel is approximately 1500 feet long, varying from 104 to 140 feet in width with the bottom at elevation 941. The discharge channel at the spillway is at elevation 940 and gradually slopes with a 1 percent grade to the present ground surface. The channel has a bottom width of 104 feet for a distance of approximately

SUPERSEDED

300 feet from the spillway and narrows to a width of 70 feet in the remaining 400 feet of length. Sections of the channel not in rock cut are protected by riprap where such protection is required. A low earth filled dike is provided downstream from the discharge channel to guide the spillway discharge, during periods of high water, until it reaches Ferguson Brook and to protect existing orchards back of the proposed dike. The spillway training walls are gravity type concrete sections which form the abutments for a new concrete highway bridge spanning between them over the weir. A maximum water surface of elevation 958.3 immediately upstream from the weir, equivalent to a pool elevation of 960.5, will be required to pass the spillway design flood discharge of 18,200 c.f.s.

d. Culverts.- A reinforced concrete box culvert, with two 6' x 11' openings, is provided across Farm Road, as shown on Plate IV-5. This culvert is designed to provide for a spillway discharge up to 1000 c.f.s., which is equivalent to the 1936 flood spillway discharge.

e. Operator's Quarters.- It is proposed to construct a brick masonry house, with finished interior, to serve as an office and as living quarters for the operator of the dam. This house is to be located in the vicinity of the east abutment on the Government reservation and can be serviced from the access road.

6. Foundations.-

a. Dam Site.- The pre-glacial stream channel of the Nubanusit Brook was a bed rock valley composed of moderately to highly fractured silicious schist. The topography of the region was modified by glaciation and the existing stream is now located on the west wall of the bed rock valley. The foundation soil is a compact sandy to silty glacial till of high shear strength and low permeability with lenses of more pervious sand. A shallow surface layer of the till has been altered by weathering and is less compact than the main till deposit. Bed rock occurs in the west abutment and under the stream channel at shallow depths of 10 to 20 feet, and is buried deeply in the east abutment. An area of weathered bed rock located in the west abutment was encountered during the foundation explorations. The embankment is founded on the glacial till and the outlet conduit and control works are founded on sound but fractured bed rock. A summary of the foundation investigations and analyses is presented in Appendix II of this report.

b. Spillway.- The spillway is located approximately three miles due north of the dam and in a saddle which separates

SUPERSEDED

Nubanusit Brook and Ferguson Brook drainage areas. Bed rock which forms the saddle is fresh, unweathered, moderately fractured porphyritic granite. The bed rock outcrops adjacent to the spillway location but dips steeply northward and is overlain by compact silty till. Cobbles and boulders are densely strewn over the area. The spillway weir and the adjacent portions of the approach and discharge channels are located in sound bed rock. Investigations and soil analyses for the spillway are presented in Appendix II.

7. Hydroelectric Power.-

a. Studies.- Results of studies of the feasibility of developing the West Peterboro site as a multiple-purpose reservoir for flood control, and for conservation storage for stream regulation and hydroelectric power indicate that such development cannot be economically justified. The optimum additional reservoir capacity resulting from second stage construction after completion of the flood control reservoir is 31,200 acre-feet. Second stage construction would involve the acquisition of 1,020 acres of additional reservoir area, the condemnation of a valuable summer camp, the relocation of 5.6 miles of highway, and the acquisition of the water rights at Verney Mills Dam in addition to raising the dam and spillway.

b. Ratio of Benefits to Carrying Charges.- Evaluation of the construction costs and the benefits to be derived from such construction indicate that the resultant ratio of annual benefits to annual carrying charges would be only 0.66 for development of the dam to provide flood and conservation storage and only 0.73 for the development of the dam to provide flood and conservation storage and to develop hydroelectric power at the site.

c. Potential Operation of Verney Mills Hydroelectric Power Installation.- The possibility of abandonment of the Verney Mills Dam at some future time is provided for in the design of the outlet conduit. Extension of the existing penstock at the Verney Mills Dam to the proposed dam has been provided for in the selection of the conduit size and shape. The penstock would be extended from its present location to the stilling basin end of the conduit and then through the conduit to the intake structure. The pipe would enter the side of the conduit immediately upstream from the stilling basin with a "Y" connection and an additional sluice gate would be required at the downstream end of the conduit to permit the discharge of water into the stilling basin.

8. Relocations.- Construction of the proposed West Peterboro Reservoir involves the relocation of only short lengths of roads and a small number of utilities.

SUPERSEDED

the dam and spillway is scheduled to be accomplished by contract during the summer months over a two-year period due to the severe winter weather in this area.

b. Schedule of Operations.- The schedule of construction operations based upon completion of the dam and spillway in two years is as follows:

First Season:

Clear and strip the dam site and spillway approach and discharge channel; excavation for the outlet works, except for closure section adjacent to the brook; excavation for core trench and a portion of the spillway approach and discharge channels; construct the dam embankment on the east and west abutments; grading and paving for the access road; construct outlet works approach channel, conduit, stilling basin, and intake structure to elevation 967; install gates in intake structure.

Second Season:

Construct upstream and downstream cofferdams; divert the river through conduit; construct gate house; complete dam embankment and construct dike downstream from spillway; complete excavation for spillway; construct spillway weir, concrete bridge at spillway and reinforced concrete box culvert under Farm Road; install new flashboards on Verney Mills Dam; relocate roads and raise bridges; construct operator's quarters.

c. Funds Required by Fiscal Years.- The funds required during each fiscal year of the two-year construction period, for the accomplishment of the project, including land acquisition and relocations, construction operations, and engineering, contingencies and overhead, are estimated to be as follows:

First Year	$ 450,000.
Second Year	944,000.
Total	$1,394,000.

The estimated fund requirements are based upon the execution of all construction by contract work.

d. Preparation of Plans and Specifications.- Prior to preparation of final plans and specifications, further explorations

SUPERSEDED

of foundation conditions at the saddle are required to determine the most economical spillway location. The estimated cost of the additional explorations is approximately $5,000. It is estimated that a period of four months will be required to prepare the contract plans and specifications, at an estimated cost of $20,000.

e. Employment Analysis.- In comparison with similar projects constructed in the Boston District, it is estimated that the project reported herein will create the following number of man hours of labor:

(1)	At-Site Labor:	
	Skilled Labor	100,000 man hours
	Unskilled Labor	400,000 man hours
	Other	50,000 man hours
	Total	550,000 man hours
(2)	Off-Site Labor:	
	Total	140,000 man hours.

11. Clearance with Other Agencies.- In accordance with Circular Letter No. 2652, dated 1 January 1944, subject: "Quadripartite Agreement", and Circular Letter No. 2306, dated 1 August 1944, subject: "Fish and Wild Life Service Cooperation", information and pertinent data on the construction of the proposed West Peterboro Reservoir have been imparted to the Soil Conservation Service of the Department of Agriculture; the Federal Power Commission; and the Fish and Wild Life Service of the Department of the Interior.

Local representatives of the Fish and Wild Life Service have been consulted and the details of the project have been discussed with them. Plans and other information have been furnished at their request, and it is understood that a complete investigation and study is being made of the project by this Service.

Several conferences have been held in the Boston District Office with representatives of the Federal Power Commission on the construction and relationship of the proposed West Peterboro Reservoir as part of the comprehensive plan for flood control of the Merrimack River Basin. Folios prepared for the Board of Consultant's conferences containing complete plans and analysis of the dam were furnished the Federal Power Commission for its use. A formal reply has not been received by this office in regard to any investigation the Commission might be undertaking at this time.

In accordance with the provisions of the Flood Control Act approved 22 December 1944 (Public Law 534 - 78th Congress - 2nd Session) and as this project in its entirety lies east of the ninety-

SUPERSEDED

seventh meridian, the Department of the Interior has not been consulted with reference to matters of land reclamation.

The Soils Conservation Service, Department of Agriculture, has been informed of the proposed flood control program being planned by the Boston District Office for co-ordination of planning for soil conservation in the development of the projects. The Department of Agriculture acknowledged receipt of the data furnished and forwarded its "Report - Soil Erosion Conditions in New Hampshire", for review and wishes to be informed of any hearings which may be held. Further action by the Department has not been indicated to date.

12. Operation and Maintenance.- The operation and maintenance of the dam is a Federal function.

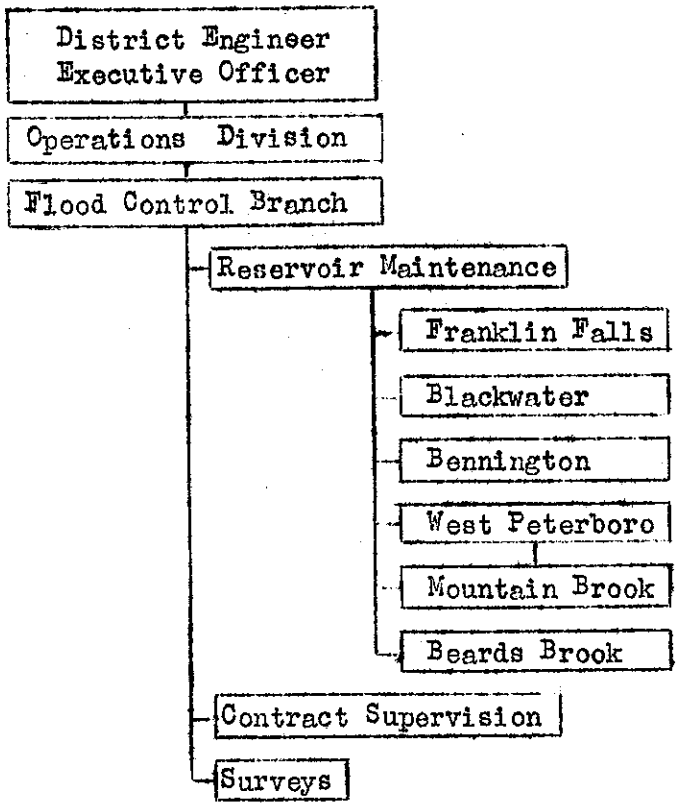
Maintenance of equipment at the operating house and of the dam structure, including slopes, drains and access road, will be necessary. After periods of high water, any debris that may have accumulated will be removed by maintenance personnel at the dam. Policing of the reservoir area will be an additional duty.

It is contemplated that the dam tender at the West Peterboro Reservoir will perform the required maintenance functions at the proposed Mountain Brook Reservoir also. Periodic visits to Mountain Brook Dam will be necessary for the purpose of inspecting and servicing the water level recorder and for regular maintenance functions.

The permanent organization at the site will consist of a dam tender, CPC-6, $1860, who will be provided with quarters at the site. Any assistance required in an emergency will be secured locally, as will labor for such special jobs as removal of large amounts of debris from the reservoir.

The operation and maintenance force will be under the supervision of the Flood Control Branch of the Operations Division of the U. S. Engineer Office, Boston District. The organizational structure of the operation and maintenance force is shown on the following chart.

SUPERSEDED



The estimated annual cost of operation and maintenance is as follows:

Item of Work	Estimated Cost
Services of dam tender and part-time helper	$ 2,900.
Power, lubricants and fuel for gate operation	50.
Heating operating structure	50.
Clearing and removing debris from reservoir area	400.
Maintenance of access road, surface drains and unprotected slopes	175.
Repairs and painting	100.
Miscellaneous	75.
District Office overhead	250.
Total	$ 4,000.

13. Malaria Control.- In accordance with instructions contained in Circular Letter No. 3606, dated 9 March 1945, subject: "Malaria Control at River and Harbor and Flood Control Reservoirs", this office has requested the advice and recommendations of the U. S. Public Health Service with respect to the need and requirements for malaria control at the West Peterboro Reservoir. Plans

SUPERSEDED

and pertinent data relative to the reservoir have been forwarded to the Public Health Service and reply has been received expressing general interest and promising attention as soon as possible. When the recommendations of the Public Health Service have been received, it is proposed to include the data in the definite project report as a supplementary appendix.

14. Cost Estimates.-

a. Total Costs.- The estimated costs, including engineering, contingencies and overhead, for the principal elements of the project, as outlined in this report, are as follows:

Lands, Damages, Relocations	$ 193,000.
Reservoir Clearing	4,500.
Construction of Dam and Outlet	626,400.
Construction of Saddle Spillway and Protective Dike	570,100.
Total Estimated Cost	$1,394,000.

The total estimated cost for West Peterboro Reservoir as given in the Definite Project Report, dated 5 August 1940 (revised 14 December 1940) is $1,170,000. Geological investigations subsequent to the submission of this estimated cost, revealed a badly weathered rock at the proposed dam site which would require special treatment and increase the construction costs of the through dam spillway to approximately equal the estimated cost of the remote spillway proposed herein.

b. Unit Costs.- The above estimated costs of principal elements are based upon the estimated quantities and unit prices summarized in the following table. The estimated costs of lands and damages are based on an appraisal made in April 1945 by the Boston District Real Estate Sub-office, Boston, Massachusetts, and are believed to represent a reasonable evaluation. The reservoir area at elevation 946 contains 850 acres of land, of which approximately 18% is wooded, 45% is scrub growth, 4% is tillable, and 33% is inundated.

SUPERSEDED

and pertinent data relative to the reservoir were forwarded to the Public Health Service. Recommendations of the Public Health Service have been received and are presented in Appendix VII of this report.

14. Cost Estimates. -

a. Total Costs. - The estimated costs, including engineering, contingencies and overhead, for the principal elements of the project, as outlined in this report, are as follows:

Lands, Damages, Relocations	$ 209,000.
Reservoir Clearing	5,300.
Construction of Dam and Outlet	739,300.
Construction of Saddle Spillway and Protective Dike	662,400.
Total Estimated Cost	$ 1,616,000.

The total estimated cost for West Peterboro Reservoir as given in the Definite Project Report, dated 5 August 1940 (Revised 14 December 1940) is $1,170,000. Geological investigations subsequent to the submission of this estimated cost, revealed a badly weathered rock at the proposed dam site which would require special treatment and increase the construction costs of the through dam spillway to approximately equal the estimated cost of the remote spillway proposed herein.

b. Unit Costs. - The above estimated costs of principal elements are based upon the estimated quantities and current unit prices (i.e. Summer 1946) summarized in the following table. The estimated costs of lands and damages are based on a revised appraisal made in September 1945 by the Boston District Real Estate Sub-office, Boston, Massachusetts, and are believed to represent a reasonable evaluation. The reservoir area at elevation 946 contains 850 acres of land, of which approximately 18% is wooded, 45% is scrub growth, 4% is tillable, and 33% is inundated.

SUPERSEDED

DETAILED ESTIMATE OF COST

I. RESERVOIR COSTS

	Total Cost
Land and Improvements / Riparian and Water Rights	$ 80,000.
Relocation of Telephone and Power Lines and Highways	73,000.
Sub-total - Reservoir Costs	$ 153,000.
Contingencies (15%)	23,000.
	$ 176,000.
Government Expenses (10% ±)	17,000.
TOTAL RESERVOIR COSTS	$ 193,000.

II. CONSTRUCTION COSTS

	Quantity	Unit	Unit Price	Total Cost
(a) Earth Dam and Controlled Outlet				
Stream Diversion and Pumping	-	-	Lump Sum	$ 15,000.
Clearing and Grubbing	-	-	Lump Sum	6,000.
Stripping	30,000	c.y.	.	18,000.
Excavation - Common	105,000	c.y.	.50	52,500.
Excavation - Rock	9,000	c.y.	2.50	22,500.
Borrow - Pervious	123,000	c.y.	.60	73,800.
Borrow - Impervious	56,000	c.y.	.50	28,000.
Sand and Gravel	24,000	c.y.	2.50	60,000.
Screened Gravel	500	c.y.	3.00	1,500.
Structural Backfill	6,000	c.y.	.50	3,000.
Rolled Fill - Pervious	107,000	c.y.	.12	12,840.
Rolled Fill - Impervious	85,000	c.y.	.18	15,300.
Rolled Fill-Random Mtl.	34,000	c.y.	.12	4,080.
Rock Fill (Dumped) Transportation & Placing only	15,000	c.y.	1.20	18,000.
Riprap-Hand Placed Transportation & Placing only	4,000	c.y.	3.00	12,000.
Drilling Holes for Grouting	3,500	l.f.	1.50	5,250.
Pressure Grouting	3,500	c.f.	1.50	5,250.
Concrete: Walls and Stilling Basin	2,400	c.y.	15.00	36,000.
Concrete: Gate Structure and Conduit	1,800	c.y.	20.00	36,000.
Reinforcing Steel	180,000	lbs.	.06	10,800.

SUPERSEDED

II. CONSTRUCTION COSTS (Continued)

(a) Earth Dam and Controlled Outlet (Cont'd.)	Quantity	Unit	Unit Price	Total Cost
Gates, Guides, Hoists, and Trashbars	-	-	Lump Sum	$ 20,000.
Operating Superstructure, including Crane	-	-	Lump Sum	5,000.
Bridges, Incl. Piers	-	-	Lump Sum	6,000.
Seeded Topsoil	4	Acres	500.	2,000.
Access Road	-	-	Lump Sum	12,000.
Operator's Quarters and Miscellaneous Items	-	-	Lump Sum	20,180.
SUB-TOTAL - EARTH DAM AND CONTROLLED OUTLET				$501,000.
(b) Saddle Spillway and Protective Dike				
Clearing and Grubbing	20	Acre	360.00	$ 7,200.
Excavation - Common	260,000	c.y.	.60	156,000.
Excavation - Rock	48,000	c.y.	2.50	120,000.
Concrete:				
Weir and Pier	800	c.y.	20.00	16,000.
N. and S. Abutments and Walls	4,800	c.y.	15.00	72,000.
Sand and Gravel	4,000	c.y.	2.50	10,000.
Structural Backfill	7,000	c.y.	.50	3,500.
Riprap - Hand Placed	8,500	c.y.	3.00	25,500.
Dumped Rock	3,000	c.y.	.60	1,800.
Seeded Topsoil	12	Acre	360.00	4,320.
Highway Bridge	-	-	Lump Sum	17,000.
Compacted Fill	40,000	c.y.	.18	7,200.
Miscellaneous Items	-	-	Lump Sum	15,480.
SUB-TOTAL - SADDLE SPILLWAY AND PROTECTIVE DIKE				$456,000.
(c) Reservoir Clearing	-	-	Lump Sum	$ 3,600.
Sub-Total - Construction Costs (a, b and c)				$960,600.
Engineering, Inspection, Overhead and Contingencies (25%±)				240,400.
TOTAL CONSTRUCTION COSTS				$1,201,000.
III. TOTAL ESTIMATED COST (I & II)				$1,394,000.

IV. UNIT STORAGE COST

Cost per Acre-Foot of Storage = $\frac{\$1,394,000}{12,800}$ $ 108.91

SUPERSEDED

c. Carrying Charges.- The total annual carrying charge for the reservoir, based upon interest on the investment, on amortization of structures and equipment, and on operation and maintenance, is $60,518, as summarized in the following table.

ANNUAL COSTS AND CARRYING CHARGES

FEDERAL INVESTMENT

1.	Total First Cost:	
	a. Structures with 50 year life	$ 1,357,000.
	b. Equipment with 25 year life	37,000.
	c. Total	$ 1,394,000.
2.	Interest During Construction: (3% for one-half construction period)	
	a. On structures with 50 year life	$ 40,710.
	b. On equipment with 25 year life	1,110.
3.	Total Investment:	
	a. Structures with 50 year life	$ 1,397,710.
	b. Equipment with 25 year life	38,110.
	c. Total Federal Investment	$ 1,435,820.

ANNUAL FEDERAL CARRYING CHARGES

1.	Interest on Investment @ 3%	$ 43,075.
2.	Amortization:	
	a. Structures with 50 year life (0.887%)	$ 12,398.
	b. Equipment with 25 year life (2.743%)	1,045.
3.	Operation and Maintenance	4,000.
4.	Total Annual Federal Carrying Charge	$ 60,518.

Construction Period - 2 years

SUPERSEDED

15. Economic Study.- The ratio of annual benefits to annual carrying charges is 1.31 as shown in the accompanying table "Summary of Benefits and Costs" and justifies construction of the West Peterboro flood control reservoir.

Studies were also conducted to determine the feasibility of a multiple-purpose development of the West Peterboro Reservoir in the interests of flood control and of power storage for stream regulation and hydroelectric power generation. The results of these studies are presented in Appendix V and indicate that second stage construction cannot be economically justified. The ratio of annual benefits to carrying charges for stream regulation storage is 0.66 and for stream regulation and hydroelectric power generation at the site is 0.73. These low ratios do not warrant making provision in the initial construction for the ultimate development of the reservoir for multiple-purpose use.

SUPERSEDED

SUMMARY OF BENEFITS AND COSTS

1. Construction Costs:

a. West Peterboro Reservoir	$ 1,394,000
b. Other Flood Control Reservoirs (x)	16,830,000
c. Local Protection Projects (xx)	1,313,150
Total Construction Costs . . .	$19,537,150

2. Annual Carrying Charges:

a. West Peterboro Reservoir	$ 60,518
b. Other Flood Control Reservoirs (x)	778,018
c. Local Protection Projects (xx)	63,800
Total Annual Carrying Charges .	$ 902,336

3. Total Annual Benefits:

Based on comprehensive flood control program including reservoirs and local protection	$ 1,183,000

4. Ratio of Benefits to Carrying Charges:

Ratio of total Annual Benefits to Annual Carrying Charges	1.31

(x) Includes completed reservoirs at Franklin Falls and Blackwater and proposed reservoirs at Beards Brook, Mountain Brook and Bennington.

	Cost		Annual Charges
Franklin Falls	$ 7,690,000	- Completed	$ 362,200
Blackwater	1,160,000	- Completed	59,000
Beards Brook	3,500,000	- Estimated	158,000
Mountain Brook	480,000	- Estimated	21,800
Bennington	4,000,000	- Estimated	177,018
	$16,830,000		$ 778,018

(xx) Includes completed local protection project at Lowell and proposed projects at North Andover and Lawrence, Mass., and Nashua, New Hampshire.

	Cost		Annual Charges
Lowell, Mass.	$ 443,500	- Completed	$ 25,500
North Andover, Mass.	323,400	- Estimated	13,880
Lawrence, Mass.	329,250	- Estimated	14,180
Nashua, New Hampshire	217,000	- Estimated	10,240
	$1,313,150		$ 63,800

SUPERSEDED

16. Recommendation.- It is recommended that the construction of the West Peterboro Reservoir for the control of floods on the Nubanusit Brook be authorized, the construction to conform with the project plan as established herein.

HOMER B. PETTIT
Colonel, Corps of Engineers
District Engineer

SUPERSEDED

War Department
United States Engineer Office
Boston, Massachusetts

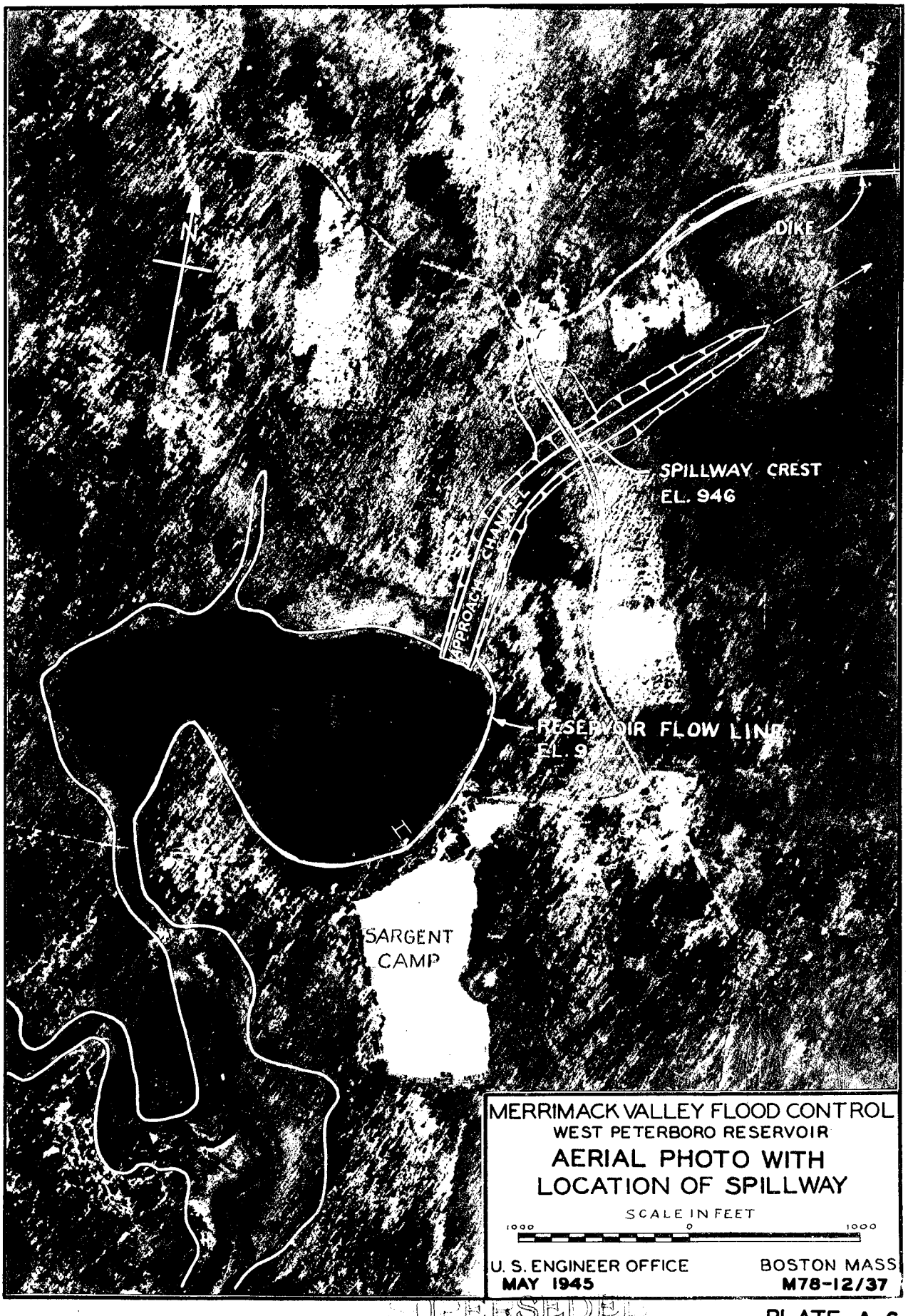
DEFINITE PROJECT REPORT

WEST PETERBORO RESERVOIR

GENERAL PLANS OF PROJECT

To accompany definite project report
dated May 1945

WAR DEPARTMENT CORPS OF ENGINEERS U.S. ARMY



MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
AERIAL PHOTO WITH
LOCATION OF SPILLWAY

SCALE IN FEET
1000 0 1000

U. S. ENGINEER OFFICE BOSTON MASS
MAY 1945 M78-12/37

SUPERSEDED

PLATE A-6

War Department
United States Engineer Office
Boston, Massachusetts

DEFINITE PROJECT REPORT

WEST PETERBORO RESERVOIR

APPENDIX I

HYDROLOGY

To accompany definite project report
dated May 1945

SUPERSEDED

DEFINITE PROJECT REPORT

WEST PETERBORO RESERVOIR

APPENDIX I - HYDROLOGY (Cont'd)

PLATES

Plate	Title
I-1	Drainage Area
I-2	Area and Capacity Curves
I-3	Drainage Areas for Hydrologic Studies
I-4	Basin Characteristics
I-5	Hydrograph at W. Peterboro, N.H., 1921 to 1931
I-6	Hydrograph at W. Peterboro, N.H., 1932 to 1942
I-7	South Branch Ashuelot River, Near Webb, N.H. Unit Hydrograph Determination (Storm of September 1938)
I-8	Unit Hydrograph Determination, South Branch Ashuelot River - Data Sheet.
I-9	Otter Brook Near Keene, N.H. Unit Hydrograph Determination (Storm of September 1938)
I-10	Unit Hydrograph Determination, Otter Brook - Data Sheet
I-11	Baker River at Wentworth, N.H. Unit Hydrograph Determination (Storm of June 14-15, 1942)
I-12	Unit Hydrograph Determination, Baker River - Data Sheet.
I-13	Unit Hydrograph Data
I-14	Curves of Limiting Rainfall
I-15	Comparison of Seasonal Floods
I-16	Computed Spillway Floods
I-17	Spillway Design Flood
I-18	Flood Discharges, New England States
I-19	Time to Empty Reservoir
I-20	Reservoir Operation, Effect on March 1936 Flood
I-21	Reservoir Operation, Effect on September 1938 Flood
I-22	Stage - Frequency Curve

SUPERSEDED

average seasonal snowfall at first order Weather Bureau stations ranges from 50.4 inches at Albany, New York, 60.4 inches at Portland, Maine, and 73.0 inches at Concord, New Hampshire, to 38.6 inches at New Haven, Connecticut, and 32.4 inches at Providence, Rhode Island. The greatest average seasonal snowfall from any reporting station is 168.3 inches at Pittsburgh, New Hampshire".

c. History of Floods.- Records of great floods on the Nubanusit Brook are incomplete. The peak discharges of the March 1936 flood, Q = 4140 c.f.s., and the September 1938 flood, Q = 4100 c.f.s., were determined by the U. S. Geological Survey at the Verney Mills Dam, (Drainage Area, 45 square miles). A hydrograph is available for the 1927 flood in West Peterboro, (Drainage Area, 48 square miles), from the U. S. Geological Survey gaging station which was in operation from 1920 to 1931. This flood was not severe and the peak discharge of 1010 c.f.s. or 21 c.f.s. per square mile is only comparable to a heavy spring freshet.

d. Project Description.- The West Peterboro dam site is located about one-half mile upstream from West Peterboro on Nubanusit Brook, a tributary of the Contoocook River. (Plate I-1). The proposed reservoir has a tributary drainage area of 44 square miles, which is 90 per cent of the drainage area of Nubanusit Brook and approximately 36 per cent of the total drainage area above Peterboro, New Hampshire. The reservoir is designed to provide flood protection principally for West Peterboro and Peterboro and to supplement to some extent the protection to be provided by the proposed Bennington Reservoir for the Contoocook River Valley between Bennington and Penacook, New Hampshire. The flood control storage to be provided is 12,800 acre-feet, equivalent to 5.5 inches over the drainage area. The area and capacity curves are shown on Plate I-2. The proposed reservoir, designed for retarding basin operation, has a single conduit through the embankment section. Two (2) gates in the conduit are provided for control of flood discharges. Both gates remain open at all times during normal operation and throughout the flood period but, if circumstances warrant it, both gates may be closed near the peak of the flood for stream regulation and protection downstream. The proposed reservoir may also be operated with one gate open at certain periods. With the reservoir water surface at spillway crest elevation the maximum discharge to be obtained with both gates open is about 1000 c.f.s. and with one gate open is about 700 c.f.s. The proposed spillway, located about three miles from the dam site, will divert water into the adjacent Ferguson Brook watershed. The spillway crest is designed as a concrete

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weir 100 feet long with crest at elevation 946 feet. The location of the spillway at this site requires an approach channel approximately 1500 feet long and a discharge channel approximately 100 feet long.

e. Basin Characteristics.- The watershed of Nubanusit Brook above the West Peterboro dam site is an elongated basin with a maximum length of 11 miles and a width of 5 miles. The long axis of the reservoir lies in a north and south direction (See Plate I-1). The area is drained by two principal tributaries; Nubanusit Brook which rises in the northern portion and flows generally southeasterly; and Stanley Brook which rises in the southern section and flows northeasterly. These tributaries unite within the proposed reservoir area about one mile above the proposed dam site. The drainage areas of the two tributaries are approximately equal. It is probable that flood flows from both areas will arrive in the reservoir at about the same time when produced by simultaneous uniform rainfall over the entire watershed.

f. Natural Storage.- The upper sections of both tributaries contain several comparatively large natural lakes which act as retarding basins. These lakes on Nubanusit Brook are Spoonwood Pond, Nubanusit Lake, Harrisville Pond, and Skatutakee Lake. Nubanusit Lake, the largest of the group, has an area of about one and one-half square miles and a storage capacity of about 4,600 acre-feet which capacity amounts to about 14 inches over its drainage area of 6 square miles. The basins on Stanley Brook are Thorndike Pond and Frost Pond and are located in the southern part of its drainage area. Most of these lakes have been increased somewhat beyond their natural size by the construction of small dams at the outlets. The additional capacity obtained by these artifical means is not the major portion of the capacity of the lakes, and the failure of these small dams would not result in the release of the entire volume of water stored in the lakes. These lakes are used to some extent for flow regulation, but are generally maintained at a maximum level for scenic and recreational purposes and cannot be counted on for appreciable flood control use. However, on the previous recommendation of the Office, Chief of Engineers, an additional allowance of one foot in computed maximum water surface of the West Peterboro Reservoir is made to allow for the possible failure of these upstream embankments during the spillway design flood.

g. Stream Characteristics.- The outer margin of the drainage area is at high elevations, the west border averaging elevation 1800 feet with a mountain peak of 2800 feet, and the east border averaging elevation 1100 feet with a peak of 2000 feet. The steep slopes on the margin of the drainage area are

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i. Stream Flow Data.- The U. S. Geological Survey maintained a gaging station on Nubanusit Brook from 1920 to 1931, which has a drainage area of 48.1 square miles. The maximum peak discharge during this period was 1,010 c.f.s., recorded during the November 1927 flood. Peak discharge values determined by the U. S. Geological Survey at a dam near West Peterboro (Drainage Area, 45.2 square miles) for the 1936 flood and the 1938 flood are 4,140 and 4,100 c.f.s., respectively.

Discharge records for other drainage areas nearby or adjacent were studied to obtain run-off characteristics. Records for the following stations were used in this study.

Basin	Name of Stream	Station Location	Drainage Area Square Miles	Max. Discharge c.f.s./ Sq. Mi.
Merrimack	Contoocook River	East Jaffrey	36.1	99
	Nubanusit Brook	West Peterboro	45.2	92
	No. Branch Contoocook River	Antrim	54.8	85
	Warner River	Bradford	19.7	120
	Souhegan River	Greenville	29.9	206
	Stony Brook	Wilton	33.2	174
Connecticut	Moss Brook	Wendell Depot	12.2	126
	So. Branch Ashuelot River	Webb	36.6	163
	Otter Brook	Keene	41.8	147

A hydrograph for the period 1920 to 1942 has been constructed for general information, and is shown on Plates I-5 and I-6. The first half of this hydrograph is plotted directly from U. S. Geological Survey records for the Nubanusit Brook station mentioned above. The last half is estimated by comparison with run-off from the adjacent Otter Brook watershed, (Drainage Area, 41.8 square miles).

The following table illustrates the magnitude of the discharges from the daily average to the spillway design flood:

Average flow	80 c.f.s.
Average annual flood	800 c.f.s.
Flood of November 1927	1,000 c.f.s.
Flood of March 1936	4,140 c.f.s.
Flood of September 1938	4,100 c.f.s.
Spillway Design Flood (Reservoir Inflow)	36,300 c.f.s.
Spillway Discharge	18,200 c.f.s.

SUPERSEDED

Ashuelot River, Otter Brook, and Bakers River. Three pairs of coefficients are assumed and used to develop three unit-hydrographs of various concentrations and peak discharges. This procedure is followed to provide a range in the computed spillway floods and to determine the effect of various unit hydrographs on the spillway requirements. By increasing the severity of the unit hydrograph an allowance also is made for the more intense rate of precipitation used in the computed spillway floods as compared with the smaller rates of rainfall used in deriving unit hydrographs from floods of record.

m. Reservoir Routing Assumptions.- It is assumed that the reservoir would be filled to normal maximum pool elevation of 946 feet above M.S.L., the crest of the spillway, at the beginning of the spillway design flood. The outlets are assumed to be inoperative. The spillway rating curve (Plate III-3) is computed for a free overfall Ogee spillway using the weir formula $Q = CLH^{3/2}$, where "L" = 100 feet and values of "C" of an Ogee section are used from 3.0 to 3.5 for the maximum head. It is necessary to compute friction and other losses in determining the relation between spillway discharge and reservoir elevation because of the length of the approach channel. A more detailed analysis of the spillway is included in Appendix III. Since the unit hydrograph is synthetic, it is assumed that it applies to reservoir inflow, consequently valley storage is neglected and the spillway floods are routed through the reservoir using the gross storage above the spillway crest.

n. Comparison of Seasonal Floods.- Spillway floods are computed, using unit hydrograph No. 1 shown on Plate I-13 applied to both summer-fall and winter-spring precipitation conditions. An infiltration rate of 0.05 inches per hour and a base flow of 5 c.f.s. per square mile are used. When these two floods are routed through the spillway surcharge storage, it is found that the spillway discharges are practically the same magnitude. Plate I-15 shows the comparison of these two floods and the results of the routing. Pertinent data concerning the computed summer-fall and winter-spring floods are shown in the following table:

SUPERSEDED

		Computed Summer-Fall Flood	Computed Winter-Spring Flood
Duration of storm	hours	24	30
Rainfall	inches	19.9	12.6
Snow melt	inches	none	10.8
Total	inches	19.9	23.4
Rate of infiltration,	inches per hour	0.05	0.05
Runoff	inches	18.9	21.9
Volume	acre feet	44,300	51,400
Peak inflow	c.f.s.	27,200	20,400
Peak outflow	c.f.s.	15,300	15,500
Maximum Reservoir El.	feet	958.9	959.0

Despite the similarity between the spillway discharges from the two seasonal floods the winter-spring hydrologic conditions are discarded in favor of the summer-fall storm because it is concluded that the higher rates of precipitation to be expected during the summer and fall justify the application of the higher unit hydrographs. The snow-melt of the winter-spring storm runs off at a comparatively slow constant rate thus justifying the lower unit hydrograph (No. 1) but not the more severe unit hydrographs (Nos. 2 and 3).

o. Computed Spillway Floods.- Plate I-16 shows the three computed spillway floods derived from the maximum possible precipitation of the summer-fall flood combined with the three empirical unit hydrographs shown on Plate I-13. The three unit hydrographs of different characteristics are used to obtain a range in computed spillway floods in order to determine the effect on the design criteria of the spillway. It is assumed that unit hydrograph No. 1 is applicable to all periods of rainfall less than two inches in a three-hour period, unit hydrograph No. 2 is used for rainfall excess values exceeding two inches to produce hydrograph "N", and graph No. 3 is used for rainfall excess periods to produce hydrograph "O". Routing these three hydrographs through the reservoir with the spillway length of 100 feet results in maximum spillway surcharges that vary between 12 and 15 feet. (See summary and graph on Plate I-16). This small difference in surcharge indicates that for a constant volume of inflow hydrograph, the spillway discharge does not change materially even with considerable variation in the shape and intensity of the hydrograph. To determine the effect of variations in volume, the ordinates of hydrograph "M" were increased 25%, 50% and 100%, and the resulting hydrographs were routed through the surcharge storage. The hydrographs are not illustrated but the effect of the increased volume is shown on

SUPERSEDED

Plate I-16 by the curve of maximum water surface elevation versus the per cent increase in volume. A 25% volume increase raises the surcharge from 13 to 16 feet but still results in a freeboard of five feet. The curve indicates that the dam would not be overtopped until the volume had increased approximately 90%, a percentage which is more than adequate as a factor of safety.

p. Spillway Design Flood.- After consideration of the various factors involved in the methods of analysing precipitation, run-off, and constructing flood hydrographs, their relative degree of accuracy, and their effect on the spillway requirements, it was decided to adopt hydrograph "N" as the spillway design flood. This hydrograph with the spillway discharge, reservoir stage elevation, pluviograph, and unit hydrographs is summarized on Plate I-17. Pertinent data relative to the spillway design flood is tabulated below.

Rainfall in inches in 24 hours	19.9
Rate of infiltration (inches per hour)..	0.05
Run-off in inches	18.7
Run-off volume in acre feet	43,800
Peak inflow in c.f.s.	36,300
Peak spillway discharge in c.f.s.	18,200
Maximum water surface (ft. above M.S.L.)	960.5
Surcharge storage utilized, acre feet	18,200
Surcharge storage utilized, inches	7.8

The Meyer Coefficient for the inflow peak of 36,300 c.f.s., used in the expression $Q = c\sqrt{\text{Drainage Area}}$ is approximately 5500. The peak of the spillway design flood expressed in terms of c.f.s. per square mile is plotted against the drainage area on Plate I-18.

q. Effect of Valley Storage.- Evaluation of the valley storage in the reservoir area has not been made. The storage might be appreciable during a spillway flood, but the evaluation would be applying a questionable refinement to a hydrograph that is based on a theoretical flood-producing storm and synthetic unit hydrographs. Consequently, the effect of valley storage is neglected and the spillway flood is considered an inflow to the reservoir and the gross surcharge storage is considered effective in the routing computations.

r. Determination of Top of Dam.- The elevation of the top of dam is determined by adding the required freeboard to the maximum water surface elevation at the time of the maximum

SUPERSEDED

head on the spillway. The theoretical freeboard required for wave height is computed by the Stevenson formula from the following data:

Fetch in miles	1.5
Wind velocity, miles per hour.	80
Angle of wind and fetch	0
Depth of water in feet.....	30

Since the theoretical freeboard is less than the minimum value of 5 feet prescribed in Engineers Bulletin R. & H. No. 9, 1938, the latter value is used. Due to the questionable stability of the upstream timber-crib dams and their possible failure during a severe flood, an additional allowance of one foot in the computed maximum water surface was made on the recommendation of the Office of the Chief of Engineers.

The top elevation of the West Peterboro Dam is determined by the following data:

Elevation, crest of spillway	946.0
Maximum head on spillway from spillway design flood	14.5
Freeboard	5.0
Allowance for failure of upstream dams ..	1.0
	966.5
Adopted elevation for top of dam	967.0

RESERVOIR DESIGN FLOOD

s. Downstream Channel Capacity.- Field investigations indicate that the channel capacity of the Nubanusit Brook is approximately 1000 c.f.s. There are six dams between the site of the proposed dam and the mouth of the Nubanusit Brook. The lower dam of the Verney Mills limits the safe reservoir discharge to the 1000 c.f.s. with a reasonable freeboard below abutments. The other dams further downstream are adequate for higher discharges, but the river channel between West Peterboro and Peterboro is sluggish and will not allow much more flow without considerable overflow and some encroachment on residential property.

t. Reservoir Capacity.- Results of studies for flood control reservoirs in this district indicate that, if possible, it is desirable to provide approximately 6 inches of storage to fully control the maximum flood of record without exceeding the downstream channel capacity. Six inches of storage at West Peterboro would require 14,100 acre-feet with the spillway crest at elevation 948.0. The maximum reservoir stage, however, is limited to elevation 946.0 by the Sargent School Camp of Boston

SUPERSEDED

University, which is located near Half Moon Pond. The camp represents a considerable investment and is only a few feet above the water level of the Pond which is at elevation 945.2. Consequently, the spillway crest is established at elevation 946.0, which provides 12,800 acre-feet of storage, or 5.5 inches. This storage is slightly less than the prescribed amount of 6 inches, but as discussed further in paragraphs v. and w., the protection to the downstream communities is guaranteed by the fact that spillway discharges caused by excessively large floods is diverted from the Nubanusit Brook directly into Bennington Reservoir.

u. Normal Operation.- The existing effective storage capacity of the Verney Mills Dam lies between elevations 906.5 and 911.1 (top of present flashboard) and is estimated to be approximately 300 acre-feet. The maximum discharge capacity of the power installation is 120 c.f.s. As a considerable portion of this storage is upstream from the West Peterboro Dam, it is proposed to compensate the hydraulic loss of head through the discharge conduit by replacing the existing 18" flashboards on the Verney Mills Dam with 36" flashboards. The higher flashboard elevation provides some additional storage capacity and allows for the loss of head in the conduit. Consequently, during normal river conditions, the flood control dam will require no operation, the outlet works gates will be left open and all regulation of the flow will take place at the Verney Mills Dam similar to the present conditions.

v. Operation During Floods.- The dam will be operated to provide the maximum flood protection to downstream areas, particularly to the Town of Peterboro. The combination of a saddle spillway and two outlet gates affords a number of methods of regulating the flood discharge as discussed below. The method selected in each instance will probably depend upon the magnitude of the flood, the extent of damage being wrought in Peterboro, and the experience obtained during previously controlled floods. The operation of the reservoir will be governed primarily by flood stages in the Town of Peterboro. Three methods of operation are as follows:

(1) The reservoir may be considered as a simple retarding basin with no operation of the gates. The capacity of the conduit will limit the discharges to a maximum of about 1000 c.f.s., the peak of the discharge depending on the magnitude of the flood and the extent that the reservoir storage is utilized.

(2) Close one gate to reduce the flow on the Nubanusit Brook during the peak of the flood on the Contoocook River, and help to prevent damage through the Town of Peterboro.

SUPERSEDED

(3) Close both gates and stop all outflow from the dam during the period of damaging discharges on the Contoocook River. Closing both gates would reduce the flow in the Nubanusit Brook to an absolute minimum, and would be of the most benefit to the Town of Peterboro. Stopping all reservoir discharge and allowing spillway discharge is rather unconventional, but it is a procedure that can always be safely followed due to the fact that the saddle spillway bypasses Nubanusit Brook and discharges directly into the proposed Bennington Reservoir.

w. Effect on 1936 and 1938 Floods.- The two maximum floods of record, occurring in March 1936 and September 1938, have been used as reservoir design floods to check the adequacy of the storage design discharge, the proposed method of operation, and the probability of spillway discharges during the operation scheme. These two floods are selected to illustrate the range in the reservoir's effectiveness; the 1936 being a prolonged three-storm flood with considerable volume, and the 1938 a single peak flood caused by one concentrated storm producing practically the same peak magnitude as the 1936 flood. The hydrographs of the two floods are based on computed peak discharges at the Verney Mills Dam with the shape and volume of the hydrographs determined from flood records on adjacent and comparable rivers. The constructed hydrographs were assumed to represent the reservoir inflow and consequently were routed through the gross reservoir storage.

Plates I-20 and I-21 show the effect of the reservoir on the 1936 and 1938 flood with two alternate methods of operation. In one case it is assumed that the reservoir is utilized as a simple retarding basin, and in the second case, both gates are closed during the period of damaging stages on the Contoocook River in Peterboro. Pertinent data concerning these two floods with the two described methods of operation and the flood reduction in Peterboro are summarized as follows:

	1936 Flood		1938 Flood	
	Gates Open	Gates Closed During Peak	Gates Open	Gates Closed During Peak
Peak inflow, c.f.s.	4,140	4,140	4,100	4,100
Maximum outlet discharge, c.f.s.	1,060	1,060	950	1,050
Maximum spillway discharge, c.f.s.	450	760	0	0
Total discharge, c.f.s.	1,510	1,820	950	1,050
Maximum reservoir stage	947.0	947.6	939.0	945.6
Flood control storage utilized in acre feet	13,000	13,800	8,700	12,500

SUPERSEDED

	1936 Flood		1938 Flood	
	Gates Open	Gates Closed During Peak	Gates Open	Gates Closed During Peak
Maximum uncontrolled discharge on Nubanusit Brook at Peterboro, c.f.s.	4,600	4,600	4,550	4,550
Maximum controlled discharge on Nubanusit Brook at Peterboro, c.f.s.	1,400	500	1,300	500
Maximum uncontrolled discharge on Contoocook River at Peterboro, c.f.s.	9,000	9,000	10,000	10,000
Maximum controlled discharge on Contoocook River at Peterboro (including effect of Mountain Brook Reservoir)	5,500	4,600	6,800	5,900

The table indicates the justification and advantage in the method of gate operation in respect to reduced discharges in the Town of Peterboro. Closing the gates in the West Peterboro Dam decreases the discharge in the Contoocook River an additional 900 c.f.s., or an increase in the reservoir effectiveness of practically 25%. The location of the spillway in the reservoir design makes this plan of operation practical and assures positive flood control for Nubanusit Brook. If closing the gates results in over-topping the spillway, the discharge bypasses the village where it might cause damage, and flows directly into the proposed Bennington Reservoir.

x. Time to Empty Reservoir.- Assuming the reservoir at spillway crest, elevation 946, with both gates open, and a constant inflow of 5 c.f.s. per square mile, it will take about 14 days to completely empty the reservoir (Plate I-19). However, 3 inches of storage, equivalent to 56% of the total storage, will be available in about 5 days, and 4-1/2 inches, equivalent to 84%, in about nine days. This is considered adequate protection against a possible second flood. Emptying the reservoir is shown also on Plates I-20 and I-21 for the March 1936 flood and the September 1938 flood.

SUPERSEDED

y. Backwater in Reservoir.- The West Peterboro Reservoir is different from the usual reservoir inasmuch as the spillway is located in the upstream section of the reservoir instead of at the dam site. The effect of backwater tends to raise the water surface elevation at the dam site during the spillway discharges instead of the customary backwater extending upstream. On Plates A-4 and I-1 it is to be noted that the points of inflow to the reservoir may be divided principally between two tributaries, (1) the Nubanusit Brook entering from the northwest with a drainage area of approximately 24 square miles, and (2) the Stanley and Brush Brooks that enter from the southwest with a drainage area of approximately 20 square miles. The inflow from Nubanusit Brook flows only a short distance through the upper reach of the reservoir before being diverted northeasterly into Half Moon Pond and to the saddle spillway. The present outlet to Half Moon Pond to the Nubanusit Brook flows southerly through a wooded section. It is proposed to clear an area through this section 200 feet wide to allow the spillway discharge to flow into Half Moon Pond with the minimum lost head. It should be noted that when the reservoir stages rise to elevation 960 (spillway design flood) the reservoir widens considerable beyond the flow line shown on Plate A-4 and that many of the apparent restrictions disappear. The inflow from the southern part of the reservoir flows north to the saddle spillway, but the cross-sectional area is sufficiently large to eliminate an appreciable difference in water surface elevations from friction losses. The relative timing of the peak inflow and the maximum reservoir stage is important in considering the possible effect of backwater on the design height of the dam. Plate I-17 shows that the maximum inflow of 36,300 c.f.s. occurs when the reservoir stage is approximately elevation 957, and that the inflow during the maximum stage, elevation 960.5, has decreased to about 18,000 c.f.s. The freeboard allowance of 6.5 feet above the maximum reservoir stage provided for the embankment section is adequate provision for any small backwater effect that may occur in the length of the reservoir during the flow condition of the maximum reservoir stages.

z. Cofferdam Design.- The construction schedule requires cofferdams to be constructed in the spring of the second year for construction of the embankment section. All the river flow then will be diverted through the conduit completed during the first construction year. The cofferdams will be placed following the spring run-off from snow-melt and consequently the annual high flows in April can be neglected. The selected elevation 920.0 for the upstream cofferdam is based on providing approximately 2 feet of freeboard for a flood with an inflow peak of about 1000 c.f.s. The pool at elevation 918.0 provides 1600

SUPERSEDED

acre-feet of storage (0.7 inches) and the outlet has a discharge capacity of about 520 c.f.s. The downstream cofferdam will be constructed to elevation 915.0 which provides approximately 2 feet of freeboard over the maximum pool elevation that may be anticipated above the Verney Mills Dam.

aa. Stage Frequency Curve (Plate I-22).- A study was made to determine the probable reservoir stage equalled or exceeded in a designated number of years. In this connection, two distinct steps were required: (1) to construct a natural discharge volume frequency curve for the West Peterboro site, and (2) to construct a stage frequency for the reservoir which is a direct result of the inflow volume. Available stream flow data on the Nubanusit Brook, required for the first step, is limited to 11 years of record near Peterboro, New Hampshire, (Drainage Area, 48 square miles) from November 1920 to September 1931. No large floods occurred during this period. Because of the lack of essential stream flow data on the Contoocook River, it was necessary to utilize records of an adjacent watershed. Otter Brook near Keene, New Hampshire, (Drainage Area, 41 square miles) with records available from October 1923 up to the present time was selected for this purpose. Comparison of the Nubanusit Brook records with the concurrent Otter Brook records indicates that differences of considerable magnitude frequently occur in comparable one day volumes, however, comparable maximum two-day volumes per square mile from the two drainage areas check very closely. It was also determined that the reservoir stage is dependent principally on volume, and that the type of hydrograph, that is a flash flood or a long flat flood, has little influence on the reservoir stage. It was, therefore, concluded that stream flow records on Otter Brook, modified for the West Peterboro drainage area could be utilized after 1931 when the records on Nubanusit Brook was discontinued. From these data, a two-day volume frequency curve was constructed using the formula, $y = \frac{M}{N - 0.5}$ in which Y = the probability of occurrence in years, M = the years of available records, and N = the summation of occurrences. Several floods of computed frequency were then routed through the reservoir to obtain the maximum reservoir stage. As the final method of gate operation is uncertain, the reservoir routings were based on the assumption that the reservoir was a simple retarding basin with both gates open continuously. The results of these computations are plotted graphically on Plate I-22.

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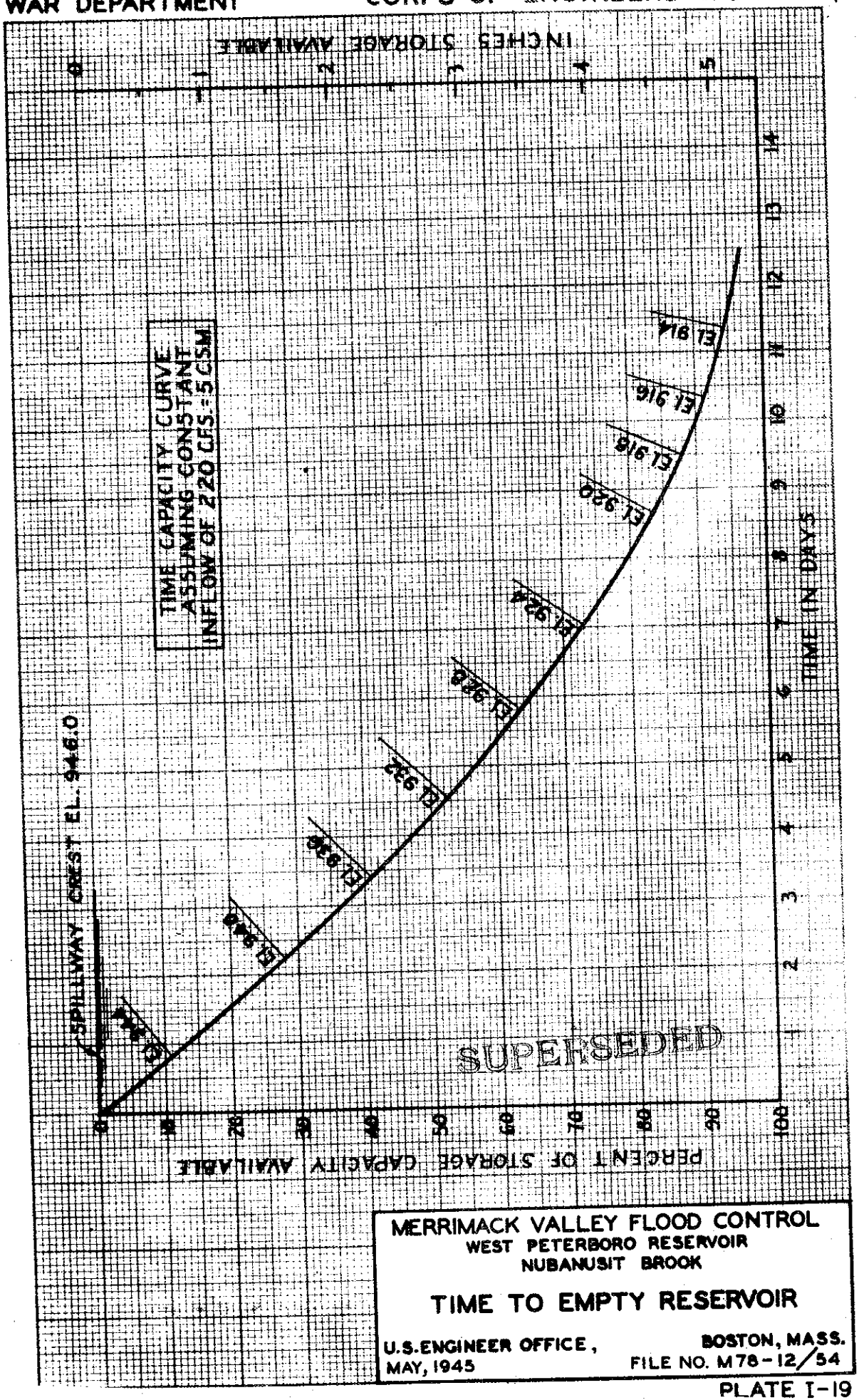


MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
NUBANUSIT BROOK

TIME TO EMPTY RESERVOIR

U.S. ENGINEER OFFICE, BOSTON, MASS.
MAY, 1945 FILE NO. M78-12/54

PLATE I-19

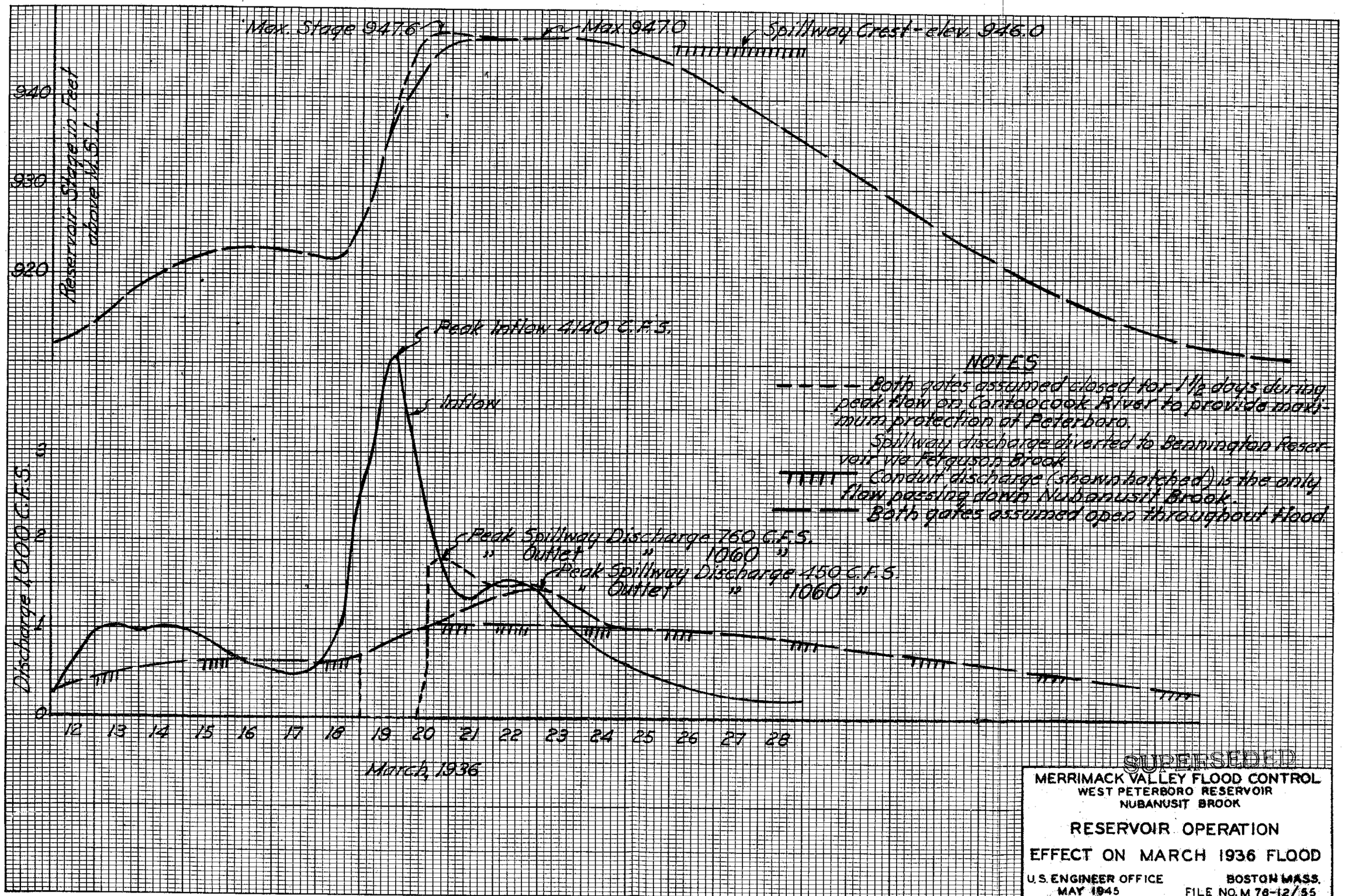
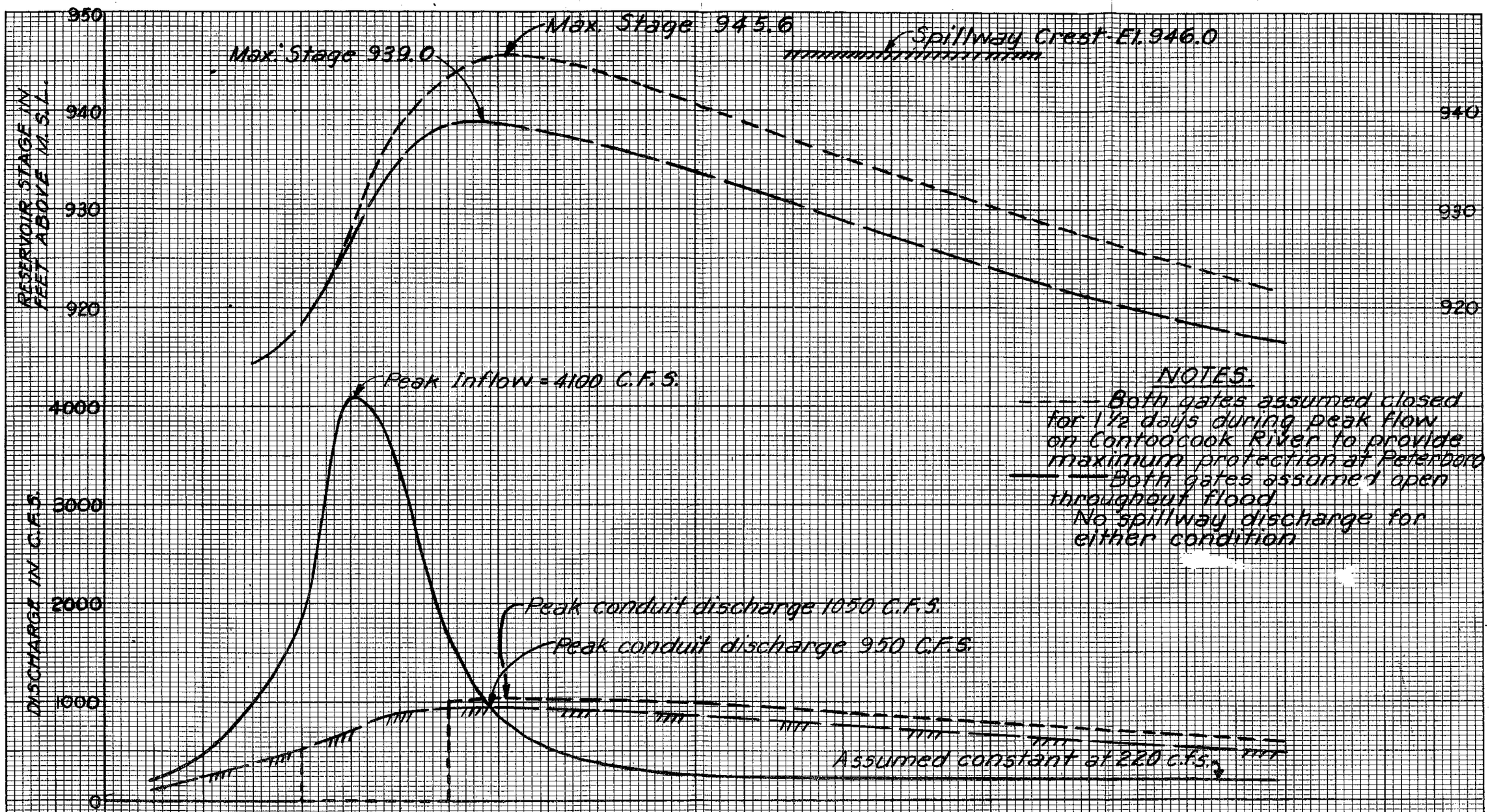

Max. Stage 947.6
Max 947.0
Spillway Crest - elev. 946.0
Reservoir Stage in Feet above M.S.L.
940
930
920
Peak Inflow 4,140 C.F.S.
Inflow
Discharge 1000 C.F.S.
3
2
1
0
Peak Spillway Discharge 760 C.F.S.
" Outlet " 1060 "
Peak Spillway Discharge 450 C.F.S.
" Outlet " 1060 "
12
13
14
15
16
17
18
19
20
21
22
23
24
25
26
27
28
March, 1936
NOTES
Both gates assumed closed for 1½ days during peak flow on Contoocook River to provide maximum protection at Peterboro.
Spillway discharge diverted to Bennington Reservoir via Ferguson Brook
Conduit discharge (shown hatched) is the only flow passing down Nubanusit Brook.
Both gates assumed open throughout flood.
SUPERSEDED
MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
NUBANUSIT BROOK
RESERVOIR OPERATION
EFFECT ON MARCH 1936 FLOOD
U.S. ENGINEER OFFICE
MAY 1945
BOSTON MASS.
FILE NO. M 78-12/55

PLATE I-20



SUPERSEDED

MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
NUBANUSIT BROOK

RESERVOIR OPERATION

EFFECT ON SEPTEMBER 1938 FLOOD

U.S. ENGINEER OFFICE BOSTON, MASS.
MAY 1945 FILE NO. M 78-12/56

PLATE I-21

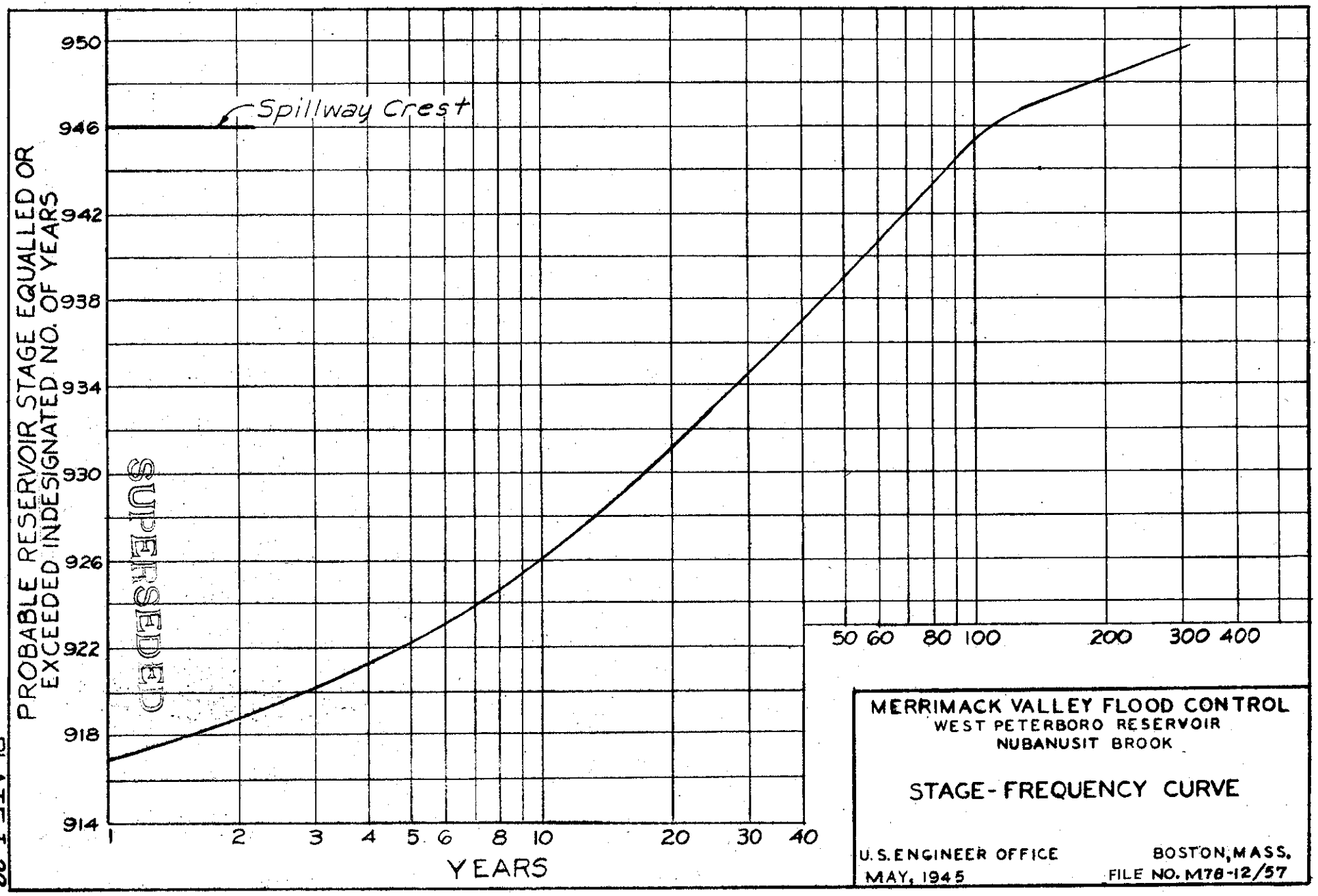

PROBABLE RESERVOIR STAGE EQUALLED OR EXCEEDED INDESIGNATED NO. OF YEARS
950
946
942
938
934
930
926
922
918
914
Spillway Crest
1
2
3
4
5
6
8
10
20
30
40
50
60
80
100
200
300
400
YEARS
SUPERSEDED
MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
NUBANUSIT BROOK
STAGE-FREQUENCY CURVE
U.S. ENGINEER OFFICE
BOSTON, MASS.
MAY, 1945
FILE NO. M78-12/57

PLATE I-22

5. Observation Well.- One observation well was installed in a drill hole where it was desired to determine ground water surface.

(b) Foundation Exploration at Dam Site.- 1. Foundation exploration at the dam site consisted of seven seismic lines, fifty-three 3-inch drill holes, one 6-inch drill hole, eight shallow test pits and one deep sheeted test pit. Location of explorations is shown on Plate II-1 and record of explorations is shown on Plates II-2, II-3, and II-4. Samples were obtained from all explorations for classification and undisturbed samples were obtained from one 6-inch drill hole and from test pits for determination of natural and remolded soil properties.

2. Pressure tests were performed in 13 drill holes in bedrock at this site. In each hole a highly shattered capping existed and the pressure testing equipment could not be sealed. The depth of this zone varied from 1 to 37 feet. Below this zone the rock becomes less fractured with depth, usually becoming relatively tight within 5 to 10 feet. In drill hole D26 a large seam capable of accommodating a flow of 8.9 g.p.m. was encountered between depths of 79 and 82 feet.

3. One observation well was installed in exploration D-61 to determine ground water surface. The point of the observation well rests on decomposed bedrock at 10.8 feet below ground surface. Periodic observations at this well since its installation in July 1944 have not encountered water in the well except in the bottom few inches of the well point which indicates that ground water is more than 10 feet below ground surface.

(c) Foundation Exploration at Spillway.- The initial exploration at the spillway site conducted early in 1940 consisted of 3 drill holes and 25 seismic lines. Additional drilling was performed in 1944 and 1945. Exploration locations, rock contours, and pertinent data are shown on Plate II-7 and record of explorations is shown on Plate II-8.

(d) Borrow Exploration.- Exploration for borrow materials consisted of field reconnaissance, 9 drill holes and 65 shallow test pits. Three, 3-inch holes were drilled in the pervious borrow area and six, 2-inch holes were drilled in the impervious borrow area. Samples of each soil stratum encountered in drill holes and test pits were obtained for classification. Representative samples of principal materials encountered in excavation of test pits were obtained for laboratory determination of soil properties. Natural density of pervious materials was determined by sand displacement method in the field. Undisturbed

SUPERSEDED

chunk samples of cohesive materials were obtained for density tests in the laboratory. A plan of borrow exploration is shown on Plate II-11 and selected logs of borrow exploration are shown on Plates II-8 and II-11.

b. Geology.- (1) Regional.- (a) Contoocook Valley.- The rocks of the Nubanusit Valley are metamorphic and igneous of Paleozoic age. The metamorphic rocks are schists and gneisses which are deposited originally as flat beds below the water surface. A period of diastrophism which occurred subsequent to this deposition resulted in the crumbling and folding of the layers until they appeared at every attitude. Large amounts of magna were injected into the metamorphosed rocks, where relatively slow cooling occurred and granite or related granitic rocks formed. A period of regional uplift followed and streams began the gradual erosion of the overlying metamorphic rocks. As the drainage pattern developed, the streams cut downward through the metamorphic rocks and exposed large areas of the granite. The granite and schist are not durable rocks and have been affected by weathering to depths of as much as 50 feet. At the beginning of the Pleistocene period, the Nubanusit Valley had been developed and the brook was flowing in a bedrock channel. During the Pleistocene period, the valley was covered by a portion of the continental glacier that occupied Canada and the Northern part of the United States. The glacier modified existing topography by erosion and deposition. Erosion by the glacier consisted of bevelling the northern slopes of hills and steepening their southern slopes. In addition, it scoured and widened north-south valleys and tended to deepen and widen saddles transverse to its line of motion. Deposition by the glacier caused the greatest change in topography. Glacial deposits are of two types; the unsorted, unstratified glacial till deposited directly from the ice, and the sorted gravels, sands, silts and clays that are washed from the ice and deposited in water. Glacial till may consist of a heterogeneous mixture of clay, silt, sand, gravel, cobbles and boulders, or some of these sizes may be lacking. In general, the till of the Nubanusit Valley does not have material as fine as the clay sizes in any appreciable amount. Most of the till may be broken down into two classes; sandy till, which has a predominance of sand sizes, and silty till which has a predominance of silt sizes. Glacial till is highly stable, capable of furnishing good support and is practically impervious. It was deposited in elliptical shaped hills known as drumlins, and as blankets which were smeared across the valleys, partially filling them and disorganizing the drainage system. In many cases interstream divides were buried by glacial deposits and streams now pass from one old drainage basin to another. Where streams still flow within the old valley, they are

SUPERSEDED

The same general conditions prevail upstream and downstream of the dam.

(c) East Abutment.- The east abutment and eastern valley wall are composed principally of compact sandy to silty till that overlies bedrock at depths of 60 to 135 feet or more. The upper zone of the till to a depth of four to six feet has been affected by weathering and frost action and is less compact than the underlying till. Scattered boulders occur on the ground surface. Bedrock continues to dip downward toward the east while the ground surface rises, thus increasing the thickness of the till body. Upstream of the dam the foundation conditions remain the same. A short distance downstream from the dam, a localized deposit of compact laminated silt occurs at a depth of approximately 18 to 30 feet.

(3) Spillway.- The spillway is located approximately three miles due north of the dam and in a saddle which separates drainage between Nubanusit Brook and Ferguson Brook. Bedrock which forms the saddle is fresh, unweathered moderately fractured porphyritic granite. Bedrock outcrops adjacent to the spillway location but dips steeply northward and is overlain by compact silty till. Cobbles and boulders are densely strewn over the area, with concentrations at areas shown on Plate II-7.

(4) Borrow Sources.- Ample quantities of compact silty glacial till suitable for impervious fill occur in both banks of the valley. The area selected for development is on the east wall of the valley and upstream from the dam. Large quantities of glacial sand and gravel deposited in eskers, kames and terraces are located on the east side of the stream within one (1) mile upstream from the dam site. These deposits contain sufficient suitable materials for pervious fill and filter materials, and concrete aggregates may be obtained from the same source by processing.

c. Construction Materials.-(1) Materials Required.- Principal materials required for construction of the project are summarized in the following tabulation.

Item	Quantity C.Y. (Emb. Meas.)
Impervious Fill	85,000
Random Fill	74,000
Pervious Fill	107,000
Concrete Aggregate and Special Gravel	44,000
Rock for Slope Protection	30,500
Topsoil	15,000

SUPERSEDED

(2) Materials Available in Required Excavations.- Materials from required excavations will be used for embankment purposes where suitable. Of the total excavation approximately 168,000 cubic yards of material is suitable for use in the embankment for the following purposes.

Item	Quantity C.Y. (Exc. Meas.)
Impervious Fill	47,000
Random Fill	85,000
Rock for Slope Protection	22,000
Topsoil	15,000

Rock for slope protection is taken from rock excavation at the spillway although part of the necessary rock may be obtained from boulders in required excavations.

(3) Materials Available for Borrow.- (a) Impervious Borrow.- Massive deposits of sandy and silty glacial till form the walls of the present stream valley. These deposits are excellent sources of material for impervious embankment construction. The area selected for impervious borrow is on the east bank of the stream a short distance upstream from the dam site (Plate II-11). The materials selected for use is the unweathered and unmodified till which occurs under a surface cover of three to ten feet.

(b) Pervious Borrow.- Large quantities of glacio-fluvial deposits occur a short distance upstream from the dam site and exist as kames, eskers and terraces composed of clean sand and gravel. The area selected for development is on the east side of the stream approximately 3/4 mile upstream from the dam site. Only shallow stripping is required to expose suitable materials.

(c) Processed Aggregate.- A survey to locate all possible commercial sources of processed aggregates is in process. Information to date indicates that processing plants are now operating at the following locations.

Locations	Approximate Haul Distance
Manchester, New Hampshire	30 miles
Keene, New Hampshire	25 miles
Fitchburg, Massachusetts	40 miles

Since all these plants are at considerable distance, it may be economical to establish a processing plant in the pervious borrow area for the project. The suitability of aggregates processed from

SUPERSEDED

this source has not been determined; however, experience with similar deposits indicates that satisfactory aggregates may be obtained.

(d) Rock for Slope Protection.- Rock for slope protection may be obtained from rock excavation for spillway construction or from a combination of this rock excavation and of boulders in required structure excavations. Bedrock excavation at spillway and boulders in earth excavation are composed of hard, durable granite. Preliminary quantity estimates indicate that approximately 26,000 cubic yards of rock excavation at the spillway is not required for use in the West Peterboro Project. This rock excavation is proposed as a possible source of riprap for the Bennington Reservoir Project. If the Bennington Project is constructed prior to West Peterboro and, if this rock source is used, approximately 14,000 cubic yards of rock would be removed. If the West Peterboro Project is constructed before the Bennington Project the excess rock excavation will be stockpiled in such location that it may be reclaimed if desired or may be left without rehandling.

(4) Materials Summary.- A summary of construction materials required, with indicated source, and materials available from required excavation and selected borrow areas, with indicated disposition, is shown in the following tabulation. Indicated embankment quantities include allowance for shrinkage and settlement.

SUPERSEDED

CONSTRUCTION MATERIALS

MATERIALS REQUIRED FOR EMBANKMENT

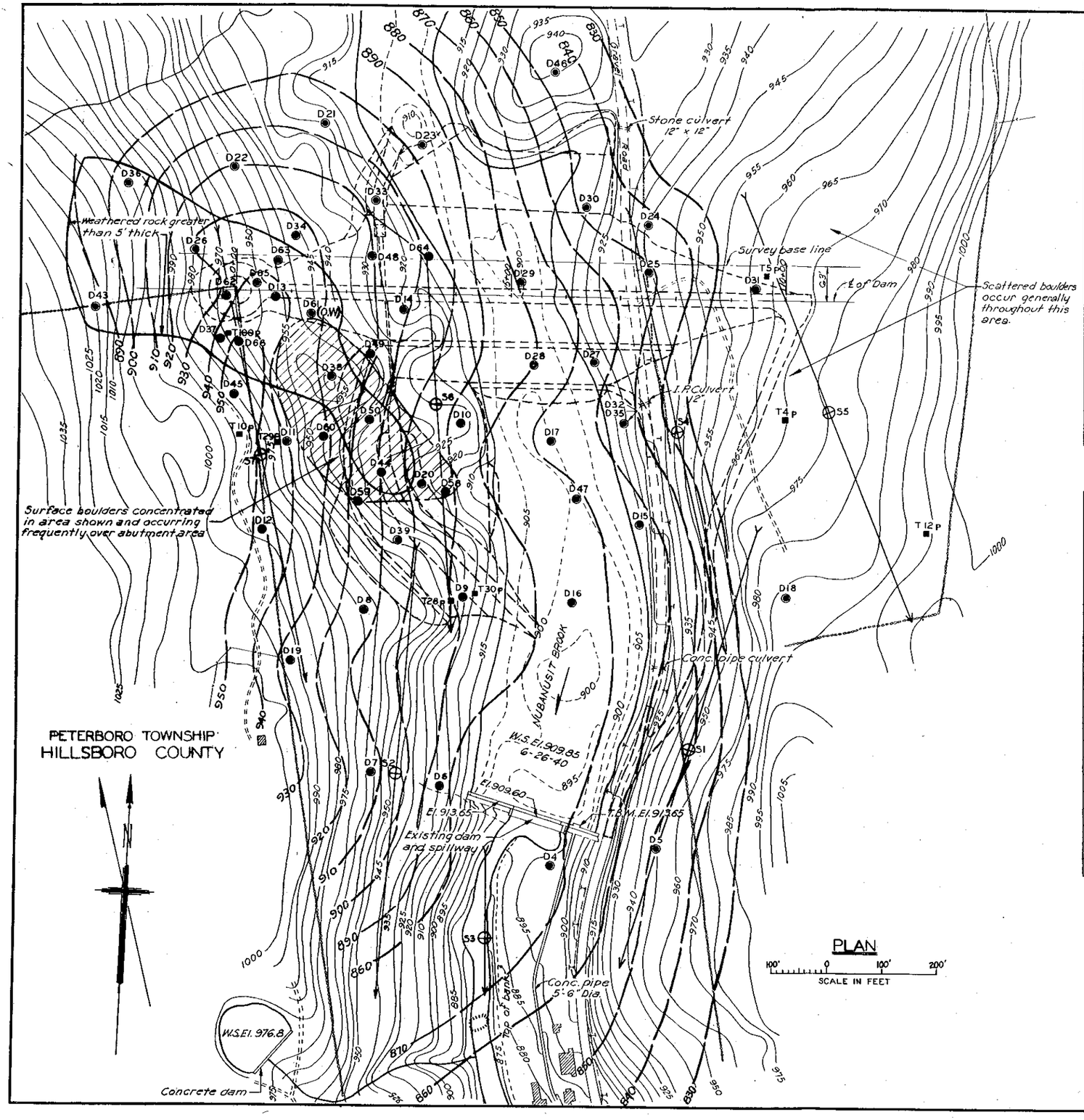
Type	Quantity, C.Y. Embankment Measure Total	Dam	Spillway	Excavation Measure	Source Excavation Measure, C. Y.
Impervious Fill	85,000	85,000	- - -	98,000	47,000 Structure Excavation at Dam Site 51,000 Impervious Borrow
Random Fill	74,000	34,000	- - -	39,000	39,000 Structure Excavation at Dam Site
		- - -	40,000	46,000	46,000 Structure Excavation at Spillway
Pervious Fill	107,000	107,000	- - -	123,000	123,000 Pervious Borrow Area
Conc. Aggregate and Scr. Gravel	44,000	30,000	14,000	60,000	Available from pervious borrow area or commercial sources within 21 miles.
Rock for Slope Protection	30,500	19,000	11,500	22,000	22,000 Rock excavation at spillway
Topsoil	15,000	15,000	- - -	15,000	15,000 Selected stripping at Dam Site

SUPERSEDED

MATERIALS AVAILABLE FROM EXCAVATION AND BORROW

Type	Quantity, C.Y. Excavation Measure			Disposition Excavation Measure, C. Y.	
	Total	Dam	Spillway		
Stripping Foundation	30,000	30,000	- - -	15,000 15,000	Topsoil Waste
Structure Foundation (Earth)	365,000	105,000	- - -	47,000 39,000 19,000	Compacted Impervious Fill Compacted Random Fill Waste
		- - -	260,000	45,000 215,000	Backfill grading and dike Waste
Structure Excavation (Rock)	57,000	9,000	- - -	9,000	Upstream Cofferdam
			48,000	22,000 26,000	Slope Protection Excess excavation not required for this project
Impervious Borrow	56,000	- - -	- - -	5,000 51,000	Stripping and Waste Compacted Impervious Fill
Pervious Borrow	133,000	- - -	- - -	10,000 123,000	Stripping and Waste Pervious Fill

SUPERSEDED



DRILL HOLE DATA

DRILL HOLE	ROCK ELEVATION	SURFACE ELEVATION	DRILL HOLE	ROCK ELEVATION	SURFACE ELEVATION	DRILL HOLE	ROCK ELEVATION	SURFACE ELEVATION
D 4	873.3	893.4	D 25	851.2	940.0	D 49	920.6	948.2
D 5	No rock	951.4	D 26	930.9	975.8	D 50	918.5	938.1
D 6	882.6	928.6	D 27	863.1	914.6	D 51	For location of drill holes at spillway See Dwg. File No. M78-12/ (Plate II-7)	
D 7	914.8	958.3	D 28	881.9	901.1	D 52		
D 8	914.7	968.0	D 29	879.8	901.6	D 53		
D 9	888.1	930.8	D 30	898.1	916.3	D 54		
D 10	885.1	927.8	D 31	No rock	961.5	D 55		
D 11	943.8	967.0	D 32	859.3	922.6	D 56		
D 12	948.0	987.5	D 33	918.0	920.2	D 57		
D 13	942.1	956.2	D 34	933.3	945.7	D 58	896.2	921.8
D 14	910.3	918.5	D 35	* No rock	924.4	D 59	924.3	947.8
D 15	867.0	913.2	D 36	906.0	1001.0	D 60	934.2	944.8
D 16	876.2	897.3	D 37	947.6	972.6	D 61	933.4	949.7
D 17	867.2	898.0	D 38	923.9	958.9	D 62	930.4	967.8
D 18	No rock	986.8	D 39	908.6	941.4	D 63	937.3	955.3
D 19	925.1	992.9	D 43	887.0	1018.8	D 64	909.9	917.9
D 20	894.2	923.2	D 44	913.0	931.0	D 65	945.2	964.0
D 21	906.0	923.4	D 45	948.9	980.2	D 66	952.6	971.2
D 22	912.2	956.2	D 46	844.3	939.2			
D 23	897.2	906.2	D 47	Approx. 872.5	Approx. 900.5			
D 24	843.7	940.4	D 48	921.2	929.6			

* 6 inch hole to obtain undisturbed samples.

SEISMIC DATA

SHOT	ROCK ELEVATION	SURFACE ELEVATION
S1	852.3	946.3
2-4+20B	886.3	944.3
2-3+20B	897.5	960.5
2-2+20B	892.2	952.2
S2	902.1	948.0
2-1+20A	905.2	943.0
2-2+70A	894.1	939.0
2-3+70A	870.3	942.0
3-0+90B	863.9	879.0
S3	871.0	875.0
3-1+10A	864.1	872.0
4-3+20B	830.0	941.8
S4	831.0	942.6
4-3+70A	835.0	946.3
S5	Deeper than 135.	976.0
6-2+20B	892.0	916.6
S6	890.0	925.7
6-2+20A	878.0	927.0
7-1+20B	953.0	980.8
S7	957.0	985.8
7-1+20A	958.0	987.5

Note:
Elevations are in feet and are based on U.S.C.&G.S. datum of mean sea level.

INDEX FOR SUBSURFACE EXPLORATION

No. OF EXPLORATION	PURPOSE	PLAN PLATE No.	GRAPHIC LOG PLATE No.
D1 - D3	Foundation investigation at Spillway	II-7	II-8
D4 - D39	" " at Damsite	II-1	II-2 & II-3
D40 - D42	Pervious borrow investigation	II-11	II-11
D43 - D50	Foundation investigation at Damsite	II-1	II-3
D51 - D57	" " at Spillway	II-7	II-8
D58 - D66	" " at Damsite	II-1	II-3 & II-4
T1p - T6p	Foundation & Impervious Borrow investigation	II-11	II-11
T10p & T12p	" " " "	II-1	II-11
T28p - T30p	Foundation investigation at Damsite.	II-1	II-4
T45p - T54p	Pervious borrow investigation.	II-11	II-11
T63p & T64p	" " "	II-11	II-11
T67p	Foundation investigation at Spillway.	II-7	II-8
T100p	" " at Damsite.	II-1	II-4
T71p, D103 - D106	Impervious borrow investigation	II-11	II-8

SUPERSEDED

LEGEND
- Earth contours.
- Rock contours.
- Outline of structures.
- B —⊕— A Seismic exploration.
- Rock outcrop.
- ◉D38 Drill hole
- ■T4p Test pit
- ◉(O.W) Observation well

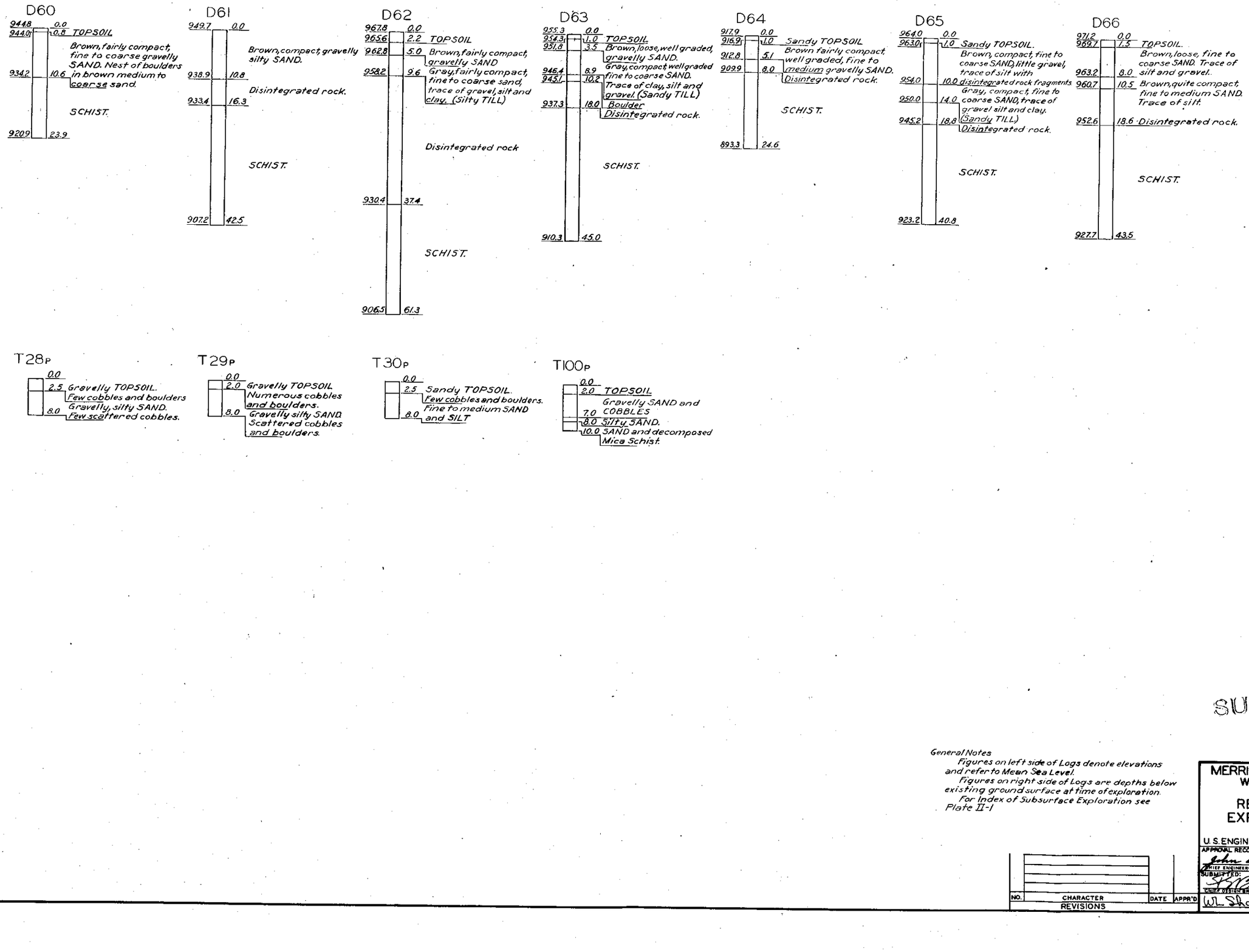
NO.	CHARACTER	DATE	APPR'D
	REVISIONS		

MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
NUBANUSIT BROOK
PLAN OF FOUNDATION EXPLORATION AT DAM SITE

U. S. ENGINEER OFFICE, BOSTON, MASS. MAY, 1945

APPROVAL RECOMMENDED: John E. Allen, ENGINEERING DIVISION
APPROVED: Thomas J. Platt, COLONEL, CORPS OF ENGINEERS, DISTRICT ENGINEER
SUBMITTED: F. F. Brown, CHIEF DESIGN BRANCH, SR. ENGINEER
W. L. Shannon, ENGINEER
DWN. BY DR. BY TR. BY CH. BY

FILE NO. M78-12/58



SUPERSEDED

General Notes
Figures on left side of Logs denote elevations and refer to Mean Sea Level.
Figures on right side of Logs are depths below existing ground surface at time of exploration.
For Index of Subsurface Exploration see Plate II-1

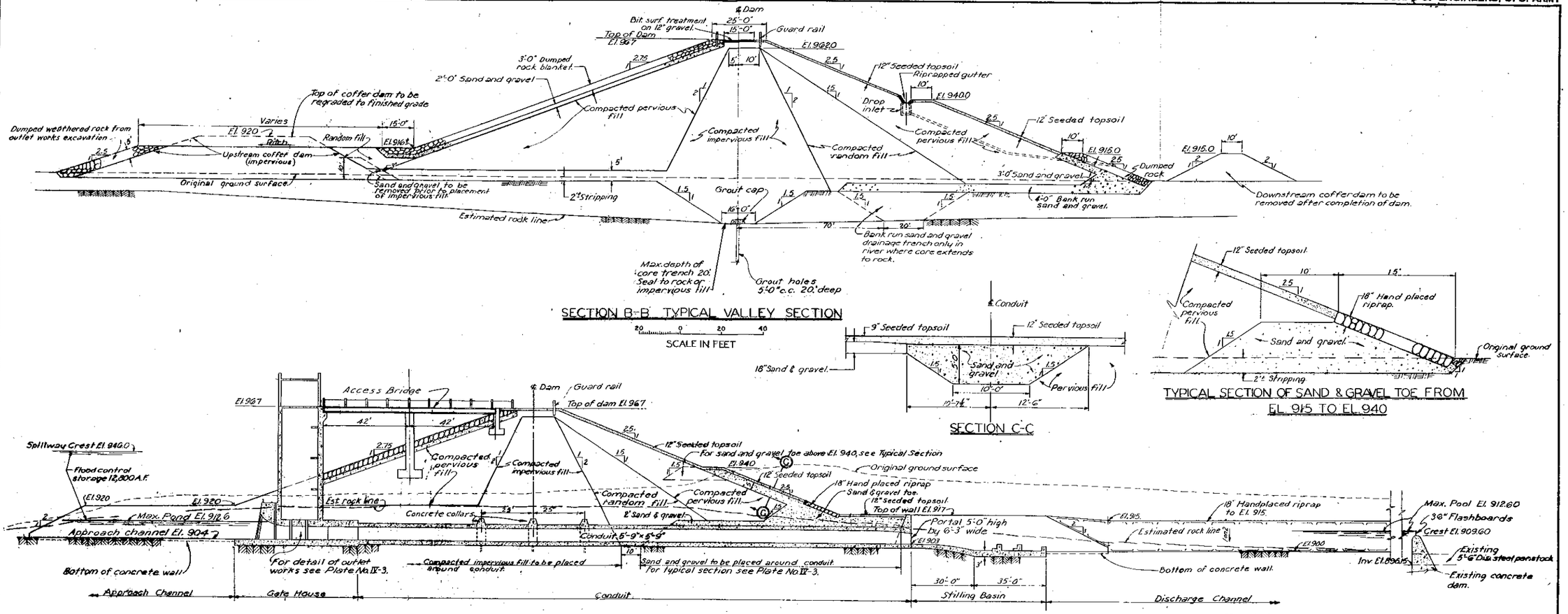
NO.	CHARACTER	DATE	APPR'D
	REVISIONS		

MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
NUBANUSIT BROOK
RECORD OF FOUNDATION EXPLORATION AT DAM SITE

U.S. ENGINEER OFFICE, BOSTON, MASS. MAY 1945

APPROVAL RECOMMENDED: John E. Allen, CHIEF ENGINEERING DIVISION
SUBMITTED: S. G. Brown, CHIEF DESIGN BRANCH, SR. ENGINEER
W. L. Shannon, ENGINEER
APPROVED: [signature], COLONEL, CORPS OF ENGINEERS, DISTRICT ENGINEER

FILE NO. M78-12/61



SECTION B-B TYPICAL VALLEY SECTION

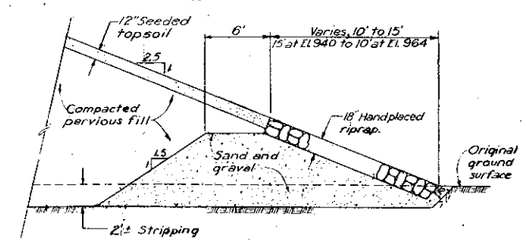
SCALE IN FEET

SECTION C-C

TYPICAL SECTION OF SAND & GRAVEL TOE FROM EL. 915 TO EL. 940

SECTION A-A THRU OUTLET WORKS

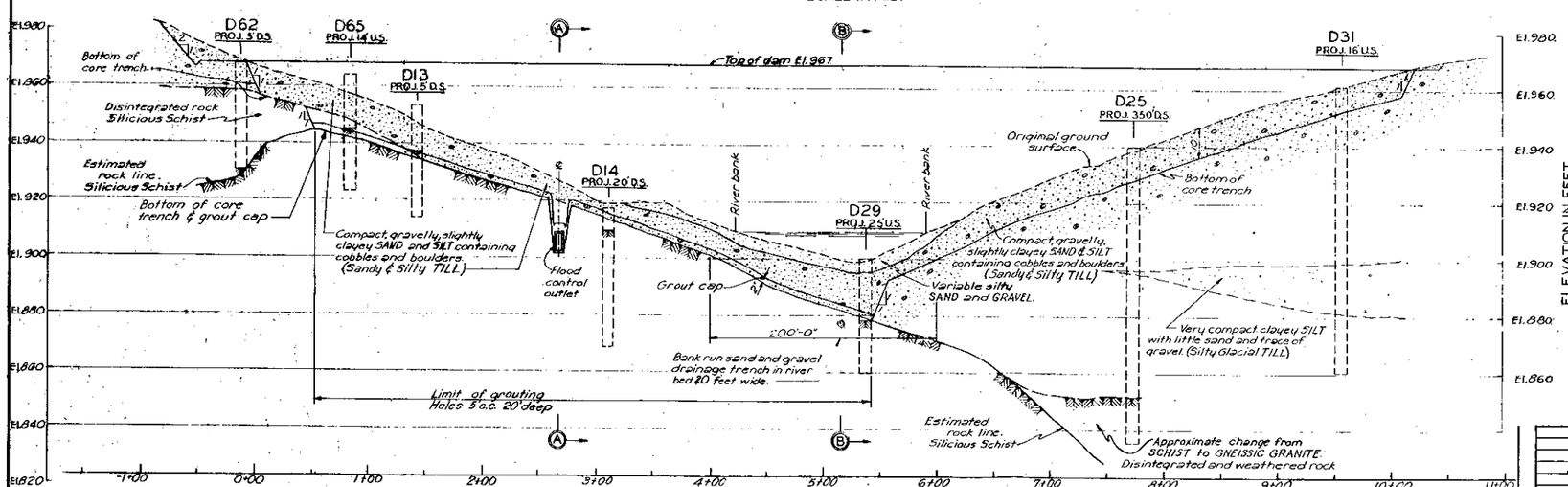
SCALE IN FEET



TYPICAL SECTION OF SAND AND GRAVEL TOE ABOVE EL. 940

NOTE: For Index of Subsurface Exploration, see Plate II-1

SCALE IN FEET (UNLESS OTHERWISE NOTED)



℄ PROFILE OF DAM

NO.	CHARACTER	DATE	BY
	REVISIONS		

MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
NUBANUSIT BROOK
PROFILE AND TYPICAL SECTIONS OF DAM AND OUTLET

U. S. ENGINEER OFFICE, BOSTON, MASS. MAY, 1945

APPROVAL RECOMMENDED: John L. Allen, CHIEF DESIGN BRANCH, DESIGN DIVISION

APPROVED: [illegible], DISTRICT ENGINEER

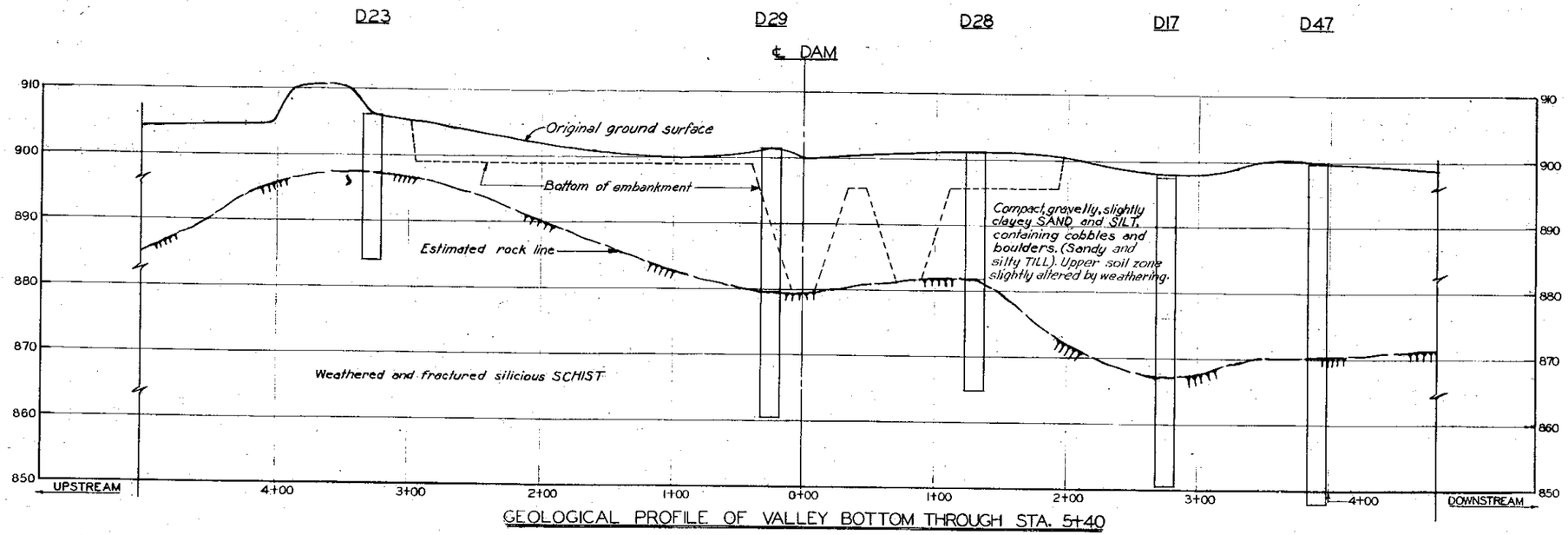
SUBMITTED: [illegible], CHIEF DESIGN SECTION, ENGINEER

DES. BY [illegible]
DR. BY [illegible]
TR. BY [illegible]
CK. BY [illegible]

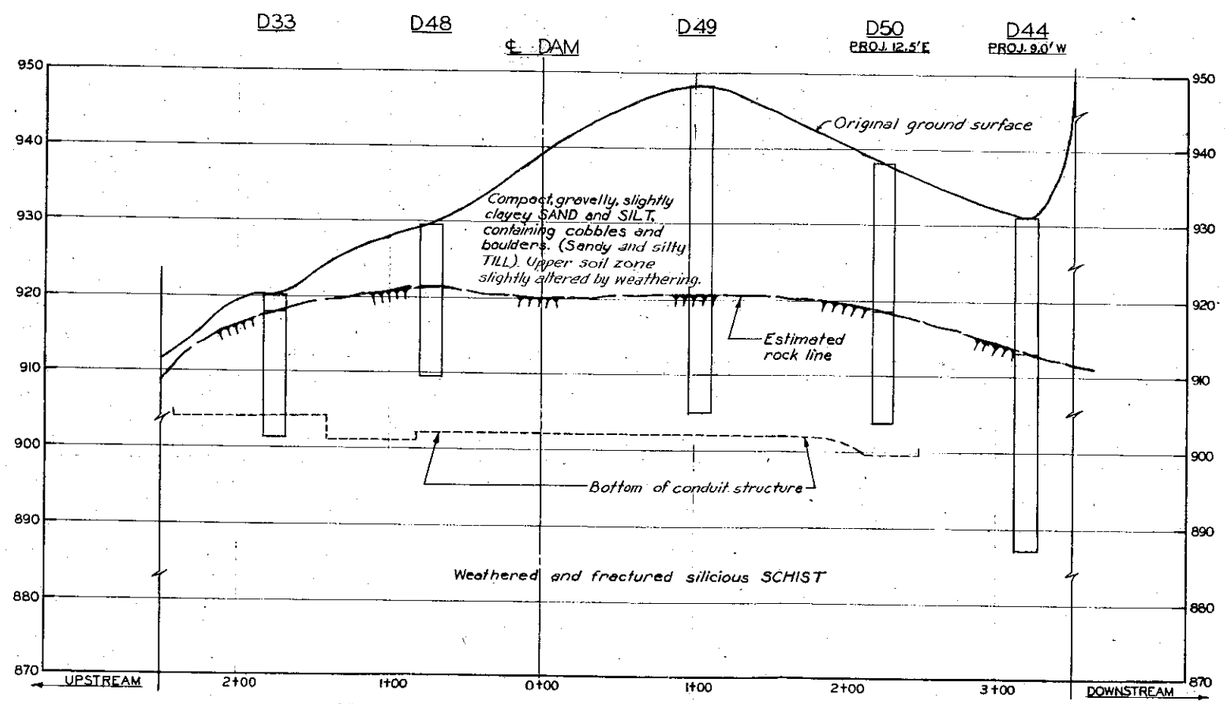
FILE NO. M78-12/73

SUPERSEDED

PLATE II-5



GEOLOGICAL PROFILE OF VALLEY BOTTOM THROUGH STA. 5+40



GEOLOGICAL PROFILE AT STA 2+67
(APPROXIMATE LOCATION OF CONDUIT)

NOTE:
For "Index of Subsurface Exploration" see Plate II-1
Elevations are in feet above Mean Sea Level.

SUPERSEDED

MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
NUBANUSIT BROOK

GEOLOGICAL PROFILES
AT DAM SITE

U.S. ENGINEER OFFICE, BOSTON, MASS. MAY 1945

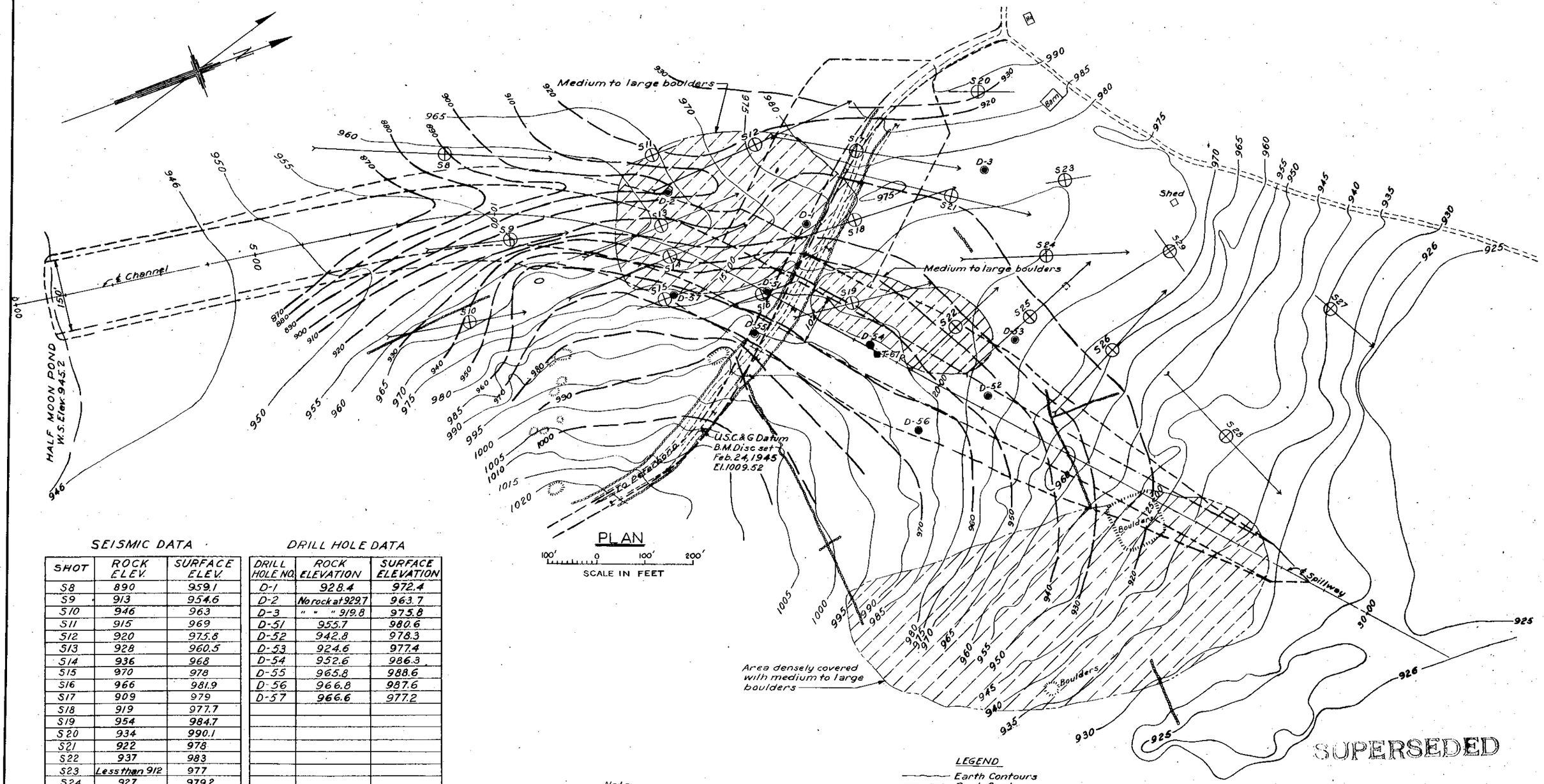
APPROVAL RECOMMENDED: John E. Allen, CHIEF ENGINEERING DIVISION
SUBMITTED: J. M. Brown, CHIEF DESIGN BRANCH, SR. ENGINEER
APPROVED: Walter A. Pitt, COLONEL, CORPS OF ENGINEERS, DISTRICT ENGINEER

NO.	CHARACTER	DATE	APPR'D
	REVISIONS		

W. L. Shannon
DES. BY
DR. BY
TR. BY
CH. BY

FILE NO. M 78-12/62

PLATE II-6



SEISMIC DATA

SHOT	ROCK ELEV.	SURFACE ELEV.
S8	890	959.1
S9	913	954.6
S10	946	963
S11	915	969
S12	920	975.8
S13	928	960.5
S14	936	968
S15	970	978
S16	966	981.9
S17	909	979
S18	919	977.7
S19	954	984.7
S20	934	990.1
S21	922	978
S22	937	983
S23	Less than 912	977
S24	927	979.2
S25	943	977.8
S26	932	969.6
S27	853 at A	931.5
S28	888	942
S29	Less than 928	975

DRILL HOLE DATA

DRILL HOLE NO.	ROCK ELEVATION	SURFACE ELEVATION
D-1	928.4	972.4
D-2	No rock at 929.7	963.7
D-3	" " " 919.8	975.8
D-51	955.7	980.6
D-52	942.8	978.3
D-53	924.6	977.4
D-54	952.6	986.3
D-55	965.8	988.6
D-56	966.8	987.6
D-57	966.6	977.2

Note:
Elevations are in feet and are based on U.S.C.&G.S. datum of mean sea level.
For "Index of Subsurface Exploration" see PLATE II-1

LEGEND
- Earth Contours
- Rock Contours
- Outline of Structures
- B → A Seismic Exploration
- Rock Outcrop
- Boulder Area

NO.	CHARACTER	DATE	APPR'D
	REVISIONS		

MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
PLAN OF FOUNDATION EXPLORATION AT SPILLWAY

U. S. ENGINEER OFFICE, BOSTON, MASS. MAY 1945

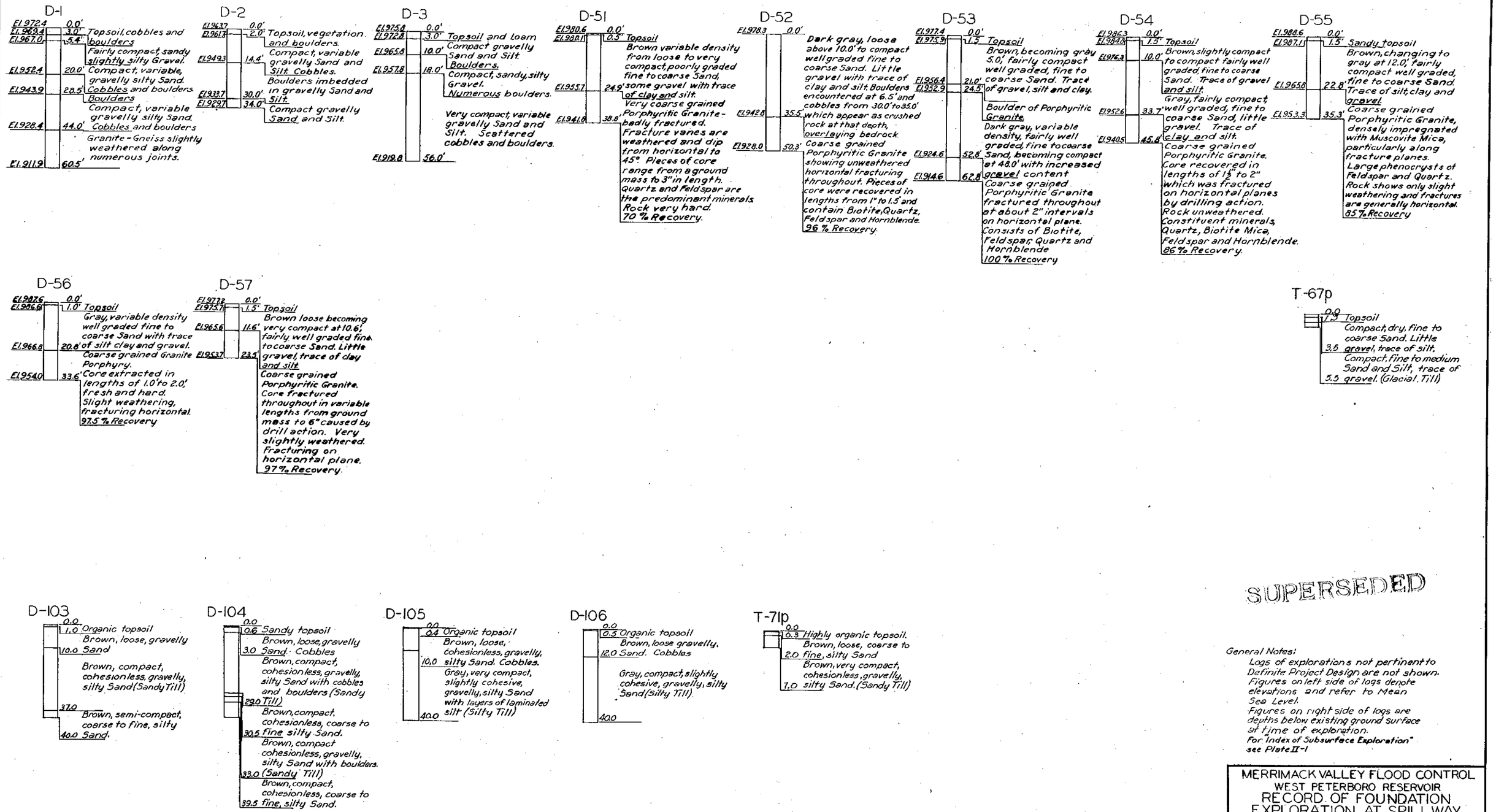
APPROVAL RECOMMENDED: John E. Allen, CHIEF ENGINEERING DIVISION

APPROVED: Paul D. Fitt, COLONEL, CORPS OF ENGINEERS, DISTRICT ENGINEER

SUBMITTED: J. T. Brown, CHIEF DESIGN BRANCH, SR. ENGINEER

W. L. Shannon

FILE NO. M78-12/63



SUPERSEDED

General Notes:

Logs of explorations not pertinent to Definite Project Design are not shown.

Figures on left side of logs denote elevations and refer to Mean Sea Level.

Figures on right side of logs are depths below existing ground surface at time of exploration.

For "Index of Subsurface Exploration" see Plate II-1

NO.	CHARACTER	DATE	APPR'D
	REVISIONS		

MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
RECORD OF FOUNDATION EXPLORATION AT SPILLWAY AND BORROW EXPLORATION

U. S. ENGINEER OFFICE, BOSTON, MASS. MAY 1945.

APPROVAL RECOMMENDED: John E. Allen, CHIEF ENGINEERING DIVISION

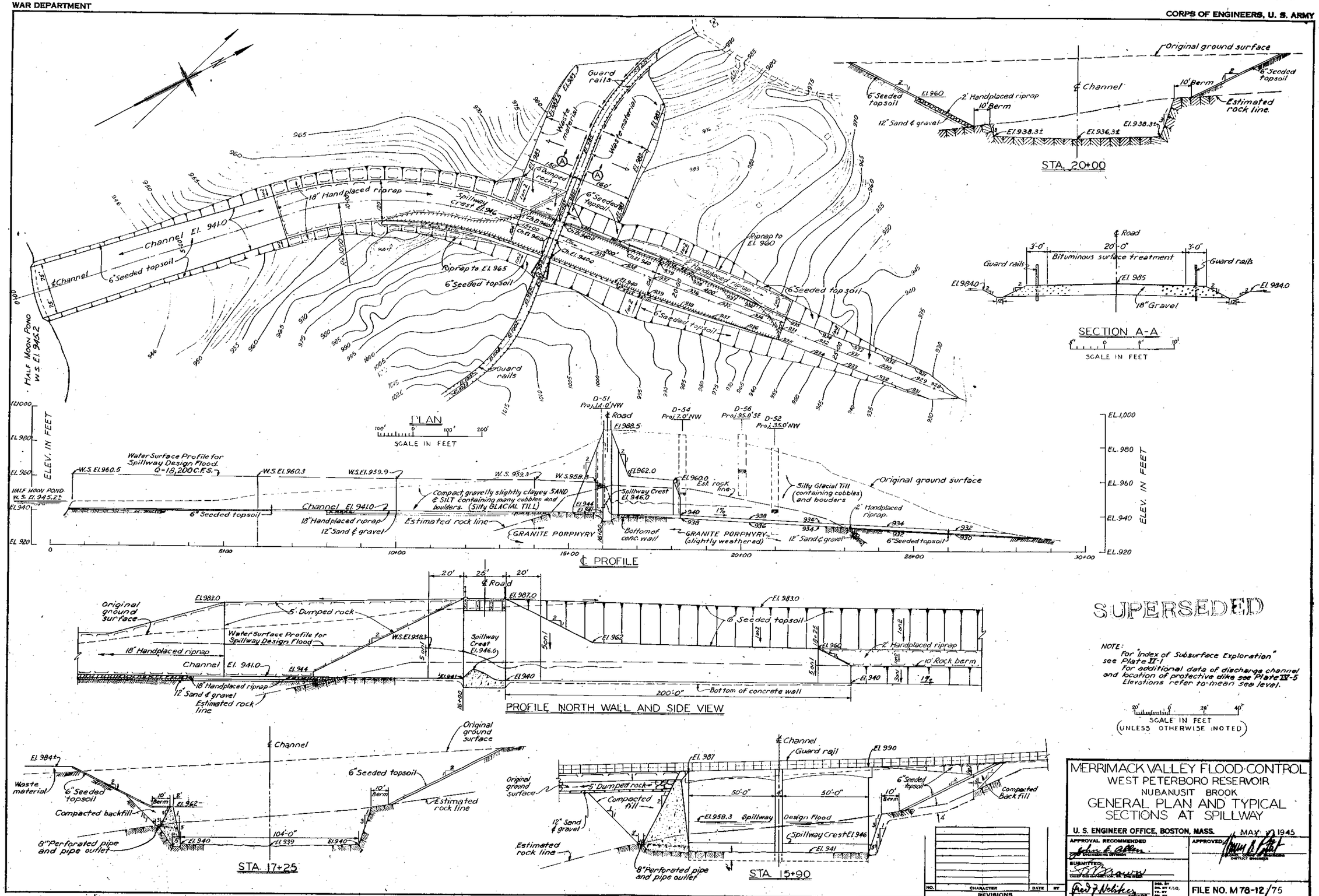
APPROVED: Thomas J. Platt, COLONEL, CORPS OF ENGINEERS, DISTRICT ENGINEER

SUBMITTED: CHIEF DESIGN BRANCH, SR. ENGINEER

W. L. Shannon, ENGINEER

DWN. BY / DES. BY / TR. BY / CH. BY

FILE NO. M 78-12/64



SUPERSEDED

NOTE:
For "Index of Subsurface Exploration" see Plate II-1
For additional data of discharge channel and location of protective dike see Plate II-5
Elevations refer to mean sea level.

SCALE IN FEET (UNLESS OTHERWISE NOTED)

MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
NUBANUSIT BROOK
GENERAL PLAN AND TYPICAL SECTIONS AT SPILLWAY

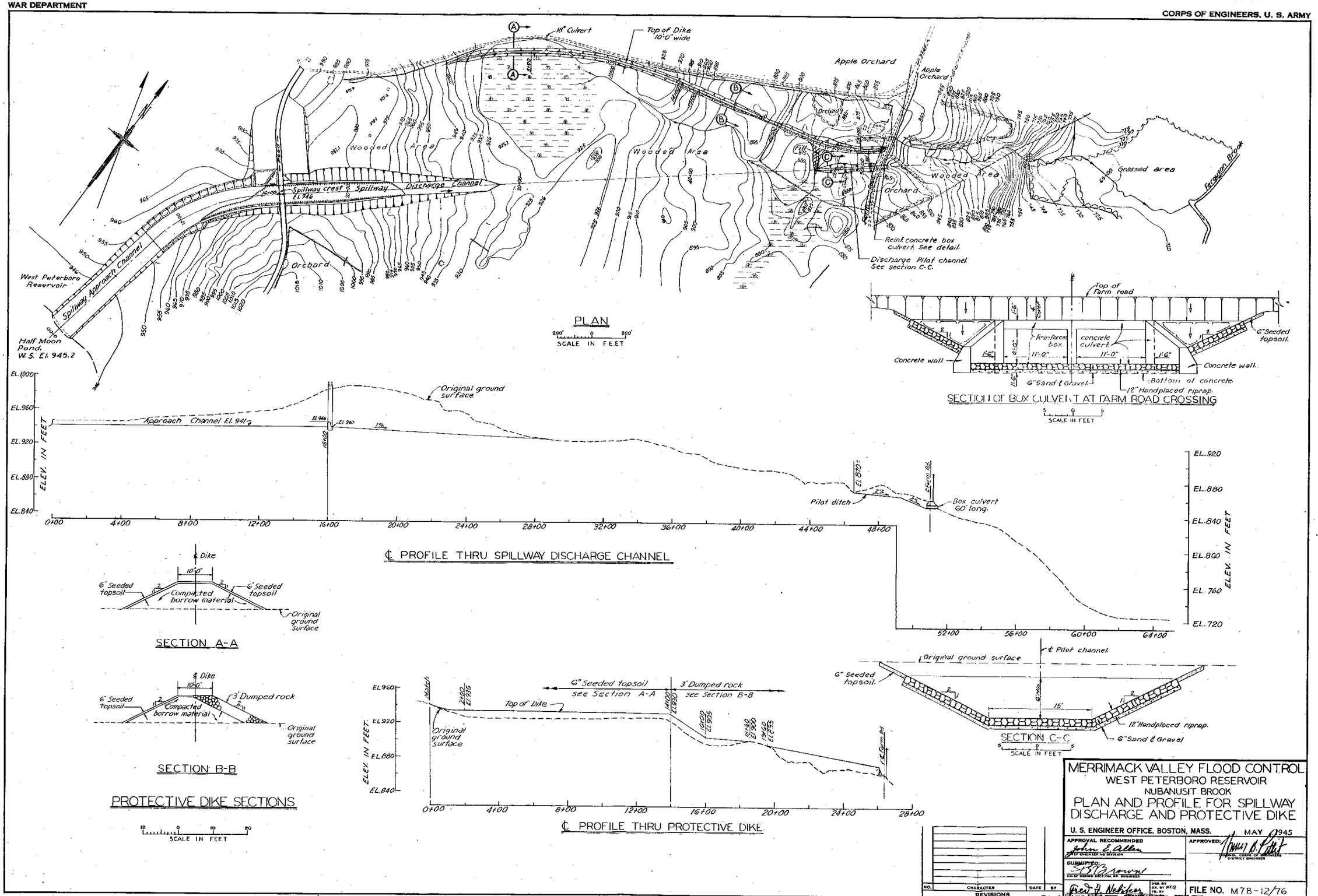
U. S. ENGINEER OFFICE, BOSTON, MASS. MAY 1945

APPROVAL RECOMMENDED: John E. Allen, Chief, Engineering Division
APPROVED: Thomas H. Patton, Colonel, Corps of Engineers, District Engineer
SUBMITTED: Chief, Design Section
Fred J. Nolikes, Engineer

NO.	CHARACTER	DATE	BY

REVISIONS

FILE NO. M78-12/75


18" Culvert
Top of Dike 10'-0" wide
Apple Orchard
Wooded Area
Spillway crest El. 946
Spillway Discharge Channel
Spillway Approach Channel
West Peterboro Reservoir
Half Moon Pond, W.S. El. 945.2
Orchard
Grassed area
Ferguson Brook
Reinf. concrete box culvert. See detail.
Discharge Pilot channel. See section C-C.
PLAN
SCALE IN FEET
Top of Farm road
Reinforced box
concrete culvert
Concrete wall
6" Seeded topsoil
6" Sand & Gravel
Bottom of concrete.
12" Handplaced riprap.
SECTION OF BOX CULVERT AT FARM ROAD CROSSING
Original ground surface
Approach Channel El. 941
Pilot ditch
Box culvert 60' long.
ELEV. IN FEET
℄ PROFILE THRU SPILLWAY DISCHARGE CHANNEL
℄ Dike
Compacted borrow material
SECTION A-A
3' Dumped rock
SECTION B-B
PROTECTIVE DIKE SECTIONS
6" Seeded topsoil
see Section A-A
see Section B-B
Top of Dike
Match
℄ PROFILE THRU PROTECTIVE DIKE
℄ Pilot channel
SECTION C-C
12" Handplaced riprap.
6" Sand & Gravel
MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
NUBANUSIT BROOK
PLAN AND PROFILE FOR SPILLWAY DISCHARGE AND PROTECTIVE DIKE
U. S. ENGINEER OFFICE, BOSTON, MASS. MAY 1945
APPROVAL RECOMMENDED
APPROVED:
SUBMITTED:
REVISIONS
FILE NO. M78-12/76
SUPERSEDED

War Department
United States Engineer Office
Boston, Massachusetts

DEFINITE PROJECT REPORT

WEST PETERBORO RESERVOIR

APPENDIX III

HYDRAULIC DESIGN

To accompany definite project report
dated May 1945

SUPERSEDED

DEFINITE PROJECT REPORT

WEST PETERBORO RESERVOIR

APPENDIX III - HYDRAULIC DESIGN

C O N T E N T S

Paragraph	Title	Page
a.	General	III-1
b.	Design Criteria	III-1
c.	Existing Verney Mills Dam	III-1
d.	Size of Gates and Conduit	III-1
e.	Stilling Basin	III-2
f.	Description	III-2
g.	Spillway Weir	III-3
h.	Approach Channel	III-3
i.	Discharge Channel	III-4

PLATES

Plate	Title
III-1	Hydraulic Data for Conduit
III-2	Stilling Basin
III-3	Discharge Curves

SUPERSEDED

DEFINITE PROJECT REPORT
WEST PETERBORO RESERVOIR

APPENDIX III - HYDRAULIC DESIGN

a. General.- This appendix discusses the hydraulic factors involved in the design of the flood control conduit and related structures and of the spillway, including its approach channel and discharge chute. The project involves no new or unusual features and the various factors used in design computation follow conservative practice for long-established design principles.

CONDUIT DESIGN

b. Design Criteria.- West Peterboro Reservoir will normally be operated as a simple retarding basin but the single conduit is equipped with two gates to permit complete closure when desirable. Design of the conduit followed these criteria:

(1) To limit flood flows to the safe channel capacity below the West Peterboro dam site.

(2) To empty the reservoir in a reasonable period of time.

(3) To provide adequate diversion during construction.

(4) To permit regulation of the existing Verney Mills Dam power pool without excessive head loss through the conduit.

c. Existing Verney Mills Dam.- The West Peterboro Dam will be located in the pool of the existing Verney Mills Dam. In the development of the West Peterboro Project it is planned to provide the same power storage under future conditions as is provided under existing conditions. The Verney Mills Dam, located about 1000 feet below the West Peterboro dam site, is a concrete gravity dam with spillway crest about 92 feet long at Elevation 909.6. The crest is provided with 18-inch flashboards, resulting in a total usable storage of about 300 acre-feet available with maximum drawdown to Elevation 906.5. Spillway abutments are at elevation 913.6 ±. A steel penstock 5 feet 6 inches in diameter transports water from this dam to the Verney Mills, a distance of about 1,000 feet. Flow into the penstock is controlled by manually operated gates.

d. Size of Gates and Conduit.- The conduit selected is 5 feet 9 inches square with two gates, each 3 feet 3 inches by

SUPERSEDED

5 feet 9 inches. This size and shape of conduit were selected to permit extension of the existing penstock for future power development as described in Appendix V. The length of the conduit is about 320 feet. The conduit invert will slope from elevation 904 at upstream end to elevation 903 at the downstream end. The exit portal of the conduit is designed with slightly reduced section (5 feet high and 6 feet-3 inches wide) to eliminate possible low pressures within the conduit itself and to start spreading the flow before discharging into the stilling basin. Friction losses in the conduit were computed by the Manning formula, using n = .013. Other hydraulic losses considered were for trash racks, entrance, eddies at gate slots, and transition. It is estimated that with both gates open and reservoir at spillway crest, the discharge will be about 1050 c.f.s. (Plate III-3). This discharge is approximately equal to the estimated downstream channel capacity and for reasons discussed in the following paragraph, it was deemed inadvisable to reduce the conduit size below 5'-9" square. Energy and hydraulic gradients for 1050 c.f.s. and 700 c.f.s., equivalent to both gates and one gate open with the reservoir at spillway crest elevation are shown on Plate III-1. The size and invert elevation of the conduit allows discharge of the 120 c.f.s. required for the Verney Mills during conditions of extreme drawdown with a lost head of less than 0.5 foot between the upper and lower pools.

c. Stilling Basin.- The stilling basin is designed to dissipate the energy in the maximum conduit discharge obtainable with reservoir at spillway crest and with both gates open. For this discharge (1050 c.f.s.) the tailwater based on the spillway discharge of the Verney Mills Dam is elevation 912. To allow for a lower tailwater due to flow through the power conduit and sluice gates, and flow conditions during dry periods, the elevation of the stilling basin floor is established at elevation 900. The basin is 35 feet long and 20 feet wide. Although the tailwater elevation submerges the portal of the conduit during normal daily flow, the high discharge velocity, obtained when there is any appreciable head on the conduit, will force the tailwater away from the portal and allow the discharge jet to spread into the stilling basin. In designing the concrete stilling basin a length of 35 feet was found to be sufficient because the discharge channel from the basin is in rock excavation and eliminates the possibility of scouring. The hydraulic jump will reduce the portal velocity of 35 feet per second to 5 feet per second at the end of the basin.

DESIGN OF SPILLWAY

1. Description.- The selected spillway site is located in a natural saddle in the eastern rim of the basin about 3 miles from the dam site and about 1500 feet northeast of Half Moon Pond. The cut through the saddle consists of a concrete weir 100 feet

SUPERSEDED

long at elevation 946, and an approach channel to the weir about 1500 feet long, varying in width from 140 feet at Half Moon Pond to 104 feet just above the weir. The discharge channel extends approximately 1300 feet downstream from the weir varying in width from 104 feet to 70 feet. Approximately 700 feet of the discharge channel is in rock cut. The discharge channel has a constant slope of 1 per cent. Beyond the defined saddle the spillway discharge will flow into a swamp and wooded area as shown on Plate IV-5.

g. Spillway Weir.- The spillway weir is a concrete section with upstream face sloped 1 on 1. The shape of the cross section was selected from data in the U.S. Bureau of Reclamation report "Abstract of Studies of Crests for Overfall Dams, Bulletin 3-Part VI." The elevation of the crest 946 is determined by storage requirements of the reservoir, limited by the elevation of Half Moon Pond as discussed in Appendix I. The length of 100 feet was selected for the spillway after preliminary estimates showed that this length gave the lowest combined cost of dam and spillway. The discharge capacity of the spillway is computed by the weir formula $Q = CLH^{3/2}$. At maximum head a coefficient of about 3.8 is possible with the crest and approach channel selected but a value of 3.5 was used with high discharges to allow for the possibility of some backwater submergence of the weir. The spillway discharge curve is shown on Plate III-3.

h. Approach Channel.- The trapezoidal approach channel varies in width from 140 feet at the northern end of Half Moon Pond to 104 feet just above the bridge section. The bottom elevation of 941 was selected as giving the least possible excavation consistent with stable flow at high discharges. Part of the approach channel is to be excavated rock using side slopes of 3 on 1. Side slopes of the remainder of the cut are 1 on 2. Except for the section cut in rock, the channel for a distance of 800 feet above the weir section is to be lined with 18-inch hand placed riprap. At maximum head there will be an average velocity of about 9.5 feet per second in the narrow section above the bridge. This velocity will be less as the approach channel widens upstream and will be about 6.5 feet per second at the end of the riprap section. It is estimated that the average velocity in the channel above this section will be about 4.5 feet per second since the banks are so low that a considerable portion of the discharge will be overbank flow. Since this channel is about 1500 feet long, from Half Moon Pond to the weir, it was necessary to compute friction losses and velocity head differences in determining the relation between reservoir elevation and discharge at the weir. The relation between elevations in the reservoir and at the weir are shown in the following table:

SUPERSEDED

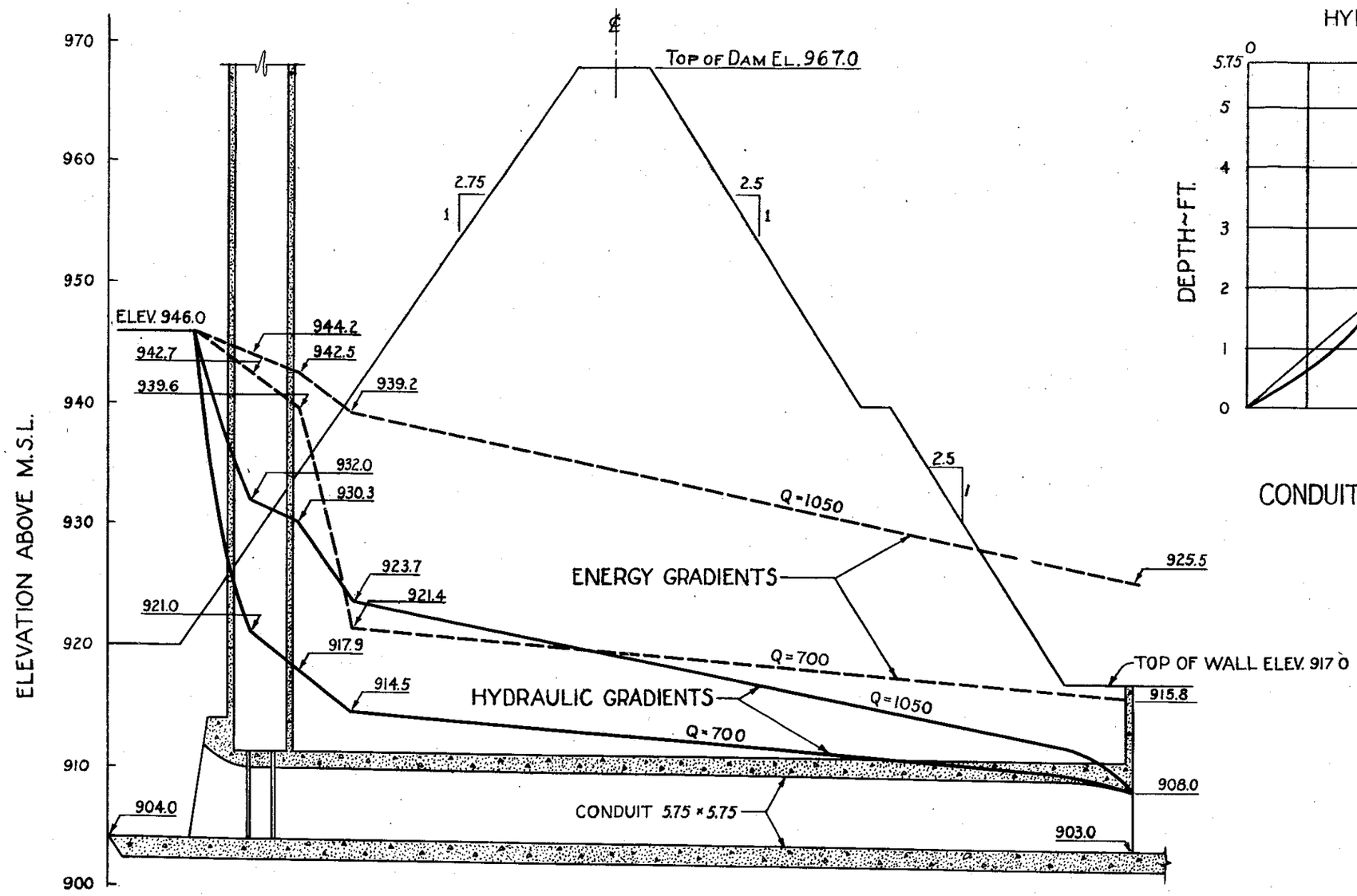
Reservoir Elevation at Half Moon Pond	Velocity Head and Friction Losses	Water Surface Elevation at Weir	Discharge in c.f.s.
946.0	0	946.0	0
948.0	0.2	947.8	1,000
950.0	0.4	949.6	2,700
952.0	0.6	951.4	4,800
954.0	0.9	953.1	7,500
956.0	1.4	954.6	10,400
958.0	1.8	956.2	14,800
960.5	2.2	958.3	18,300

These losses have been used in determining the spillway rating curve shown on Plate III-3.

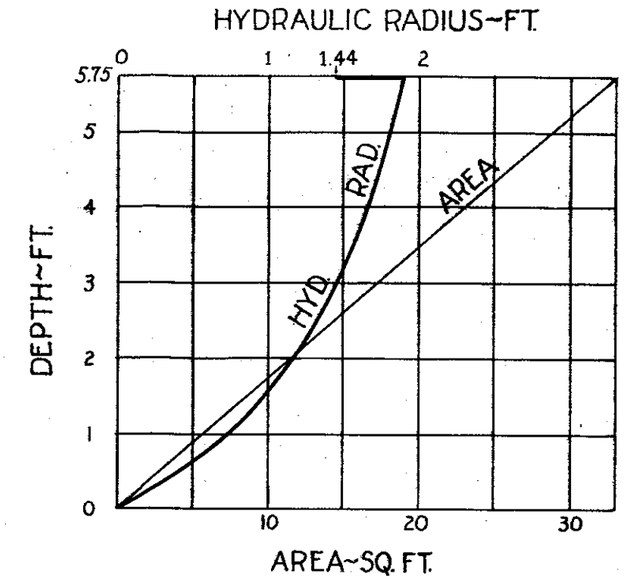
i. Discharge Channel.- The discharge channel for about 300 feet below the weir is trapezoidal in section, the bottom being 104 feet wide and at elevation 940. The right bank is in a rock cut and the left bank is confined by a concrete retaining wall. In the following 400 feet the channel gradually narrows to a bottom width of 70 feet which width extends beyond the rock cut. Earth slopes are 1 on 2 and have a 2-foot depth of hand-placed riprap. Beyond this section the 70-foot width is maintained but the riprap is discontinued. The channel has a constant slope of 1 per cent throughout its length. It is not hydraulically determinable whether an hydraulic jump will take place in the discharge channel during the spillway design discharge of 18,200 c.f.s. Computations indicate, however, that the lower conjugate depth of flow is less than the critical flow for the cross-sectional area and slope, and consequently the flow would be in an unstable condition and must change, either by forming an hydraulic jump or turbulent breakdown, to the upper conjugate depth. The selected coefficient of discharge for the weir is sufficiently low to provide for some small submergence in case the water below the weir should flow at the upper conjugate depth without going through the lower unstable condition.

The discharge channel terminates in a swampy area which is expected to retard low and medium discharges. This swamp is now drained by a small brook which at one point passes through a culvert about 2-1/2 feet square. A limited amount of diking and channel improvement is required here and at a few other points to confine large flows and lead them to the valley of Ferguson Brook. Since the maximum pool for Bennington Reservoir will extend up Ferguson Brook to this point, no improvement is required to permit greater carrying capacity except for enlargement of the water area at the first bridge. As shown on Plate IV-5 it is proposed to replace the small existing road culvert with a large dual culvert with a capacity of approximately 1000 c.f.s. which is sufficient for any nominal spillway discharge that may be anticipated from a reservoir design flood.

SUPERSEDED



SCALE
Hor. 1" = 40'
Ver. 1" = 10'



CONDUIT CHARACTERISTICS

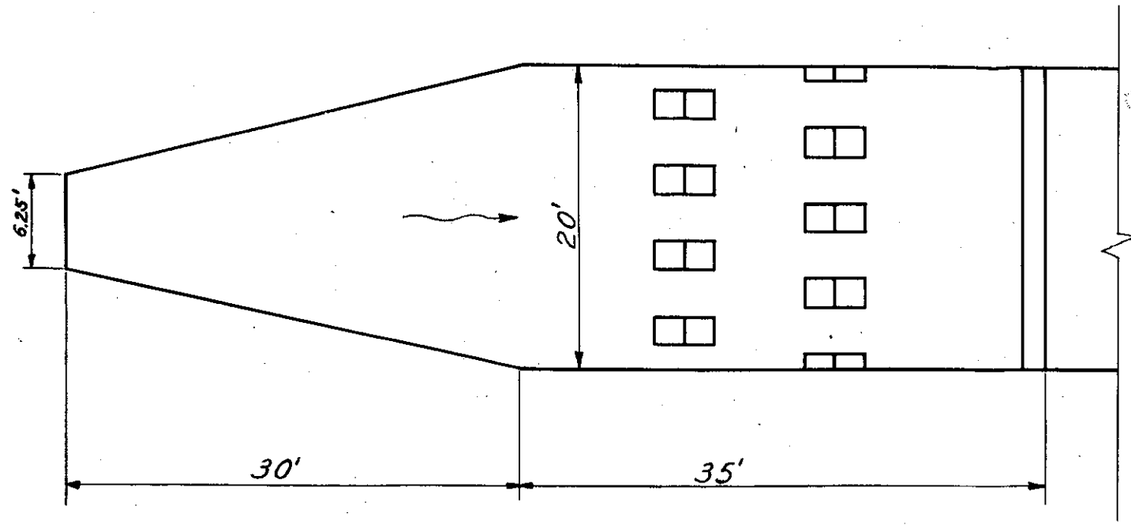
SUPERSEDED

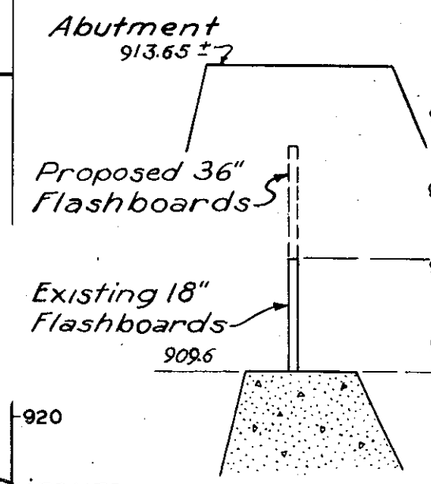
MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR

HYDRAULIC DATA FOR CONDUIT

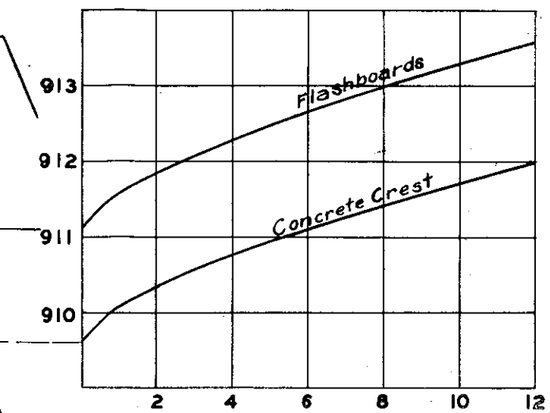
U. S. ENGINEER OFFICE BOSTON, MASS.
MAY, 1945 FILE NO. M78-12/69

PLATE III-1

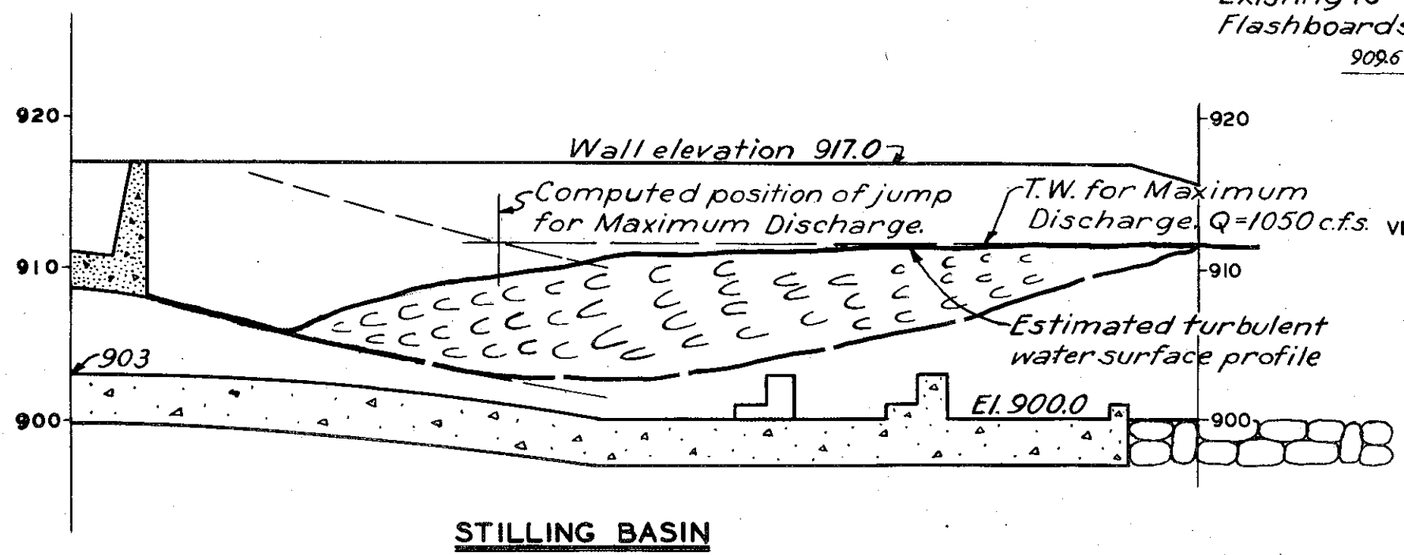




CREST
VERT SCALE 1"=2'-0"



Discharge = 100 c.f.s.
DISCHARGE CURVES



STILLING BASIN
SCALE 1"=10'-0"

VERNEY MILLS DAM

SUPERSEDED

MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
NUBANUSIT BROOK

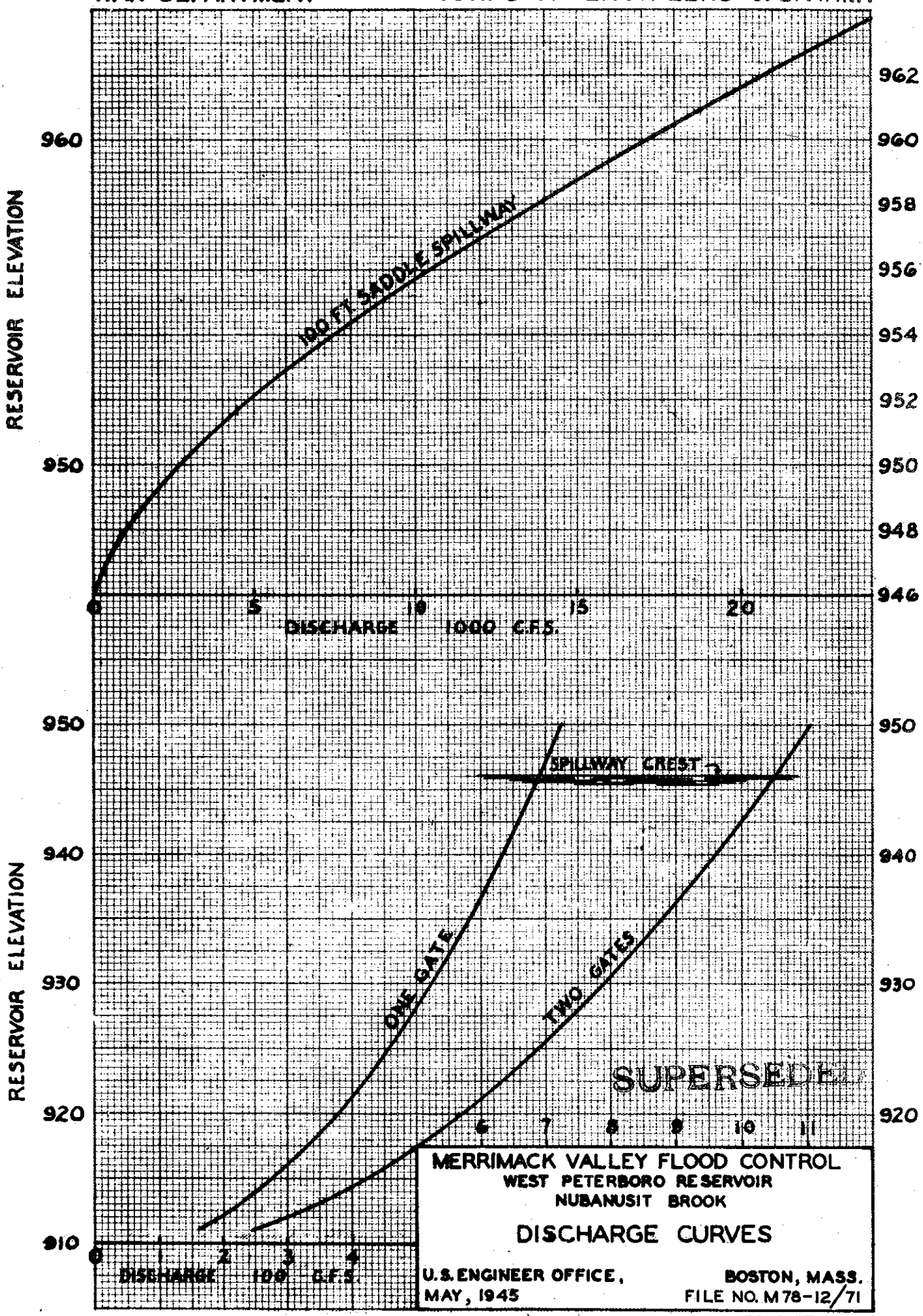
STILLING BASIN

U. S. ENGINEER OFFICE
MAY 1945

BOSTON, MASS.
M78-12/70

PLATE III-2

WAR DEPARTMENT CORPS OF ENGINEERS U.S. ARMY



SUPERSEDED

MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
NUBANUSIT BROOK

DISCHARGE CURVES

U.S. ENGINEER OFFICE, BOSTON, MASS.
MAY, 1945 FILE NO. M78-12/71

PLATE III-3

The second conference was held on 14-15 December 1944, and the following members were present:

Mr. W. H. McAlpine, Office, Chief of Engineers, Washington, D.C.
Mr. W. F. Uhl - Charles T. Main, Inc., Boston, Mass., Consultant
Mr. J. D. Justin - Philadelphia, Pennsylvania, Consultant
Dr. Arthur Casagrande, Harvard University, Cambridge, Mass., Consultant.

Informal discussions were held at various times with Mr. W. H. McAlpine and Dr. Arthur Casagrande on various features of design.

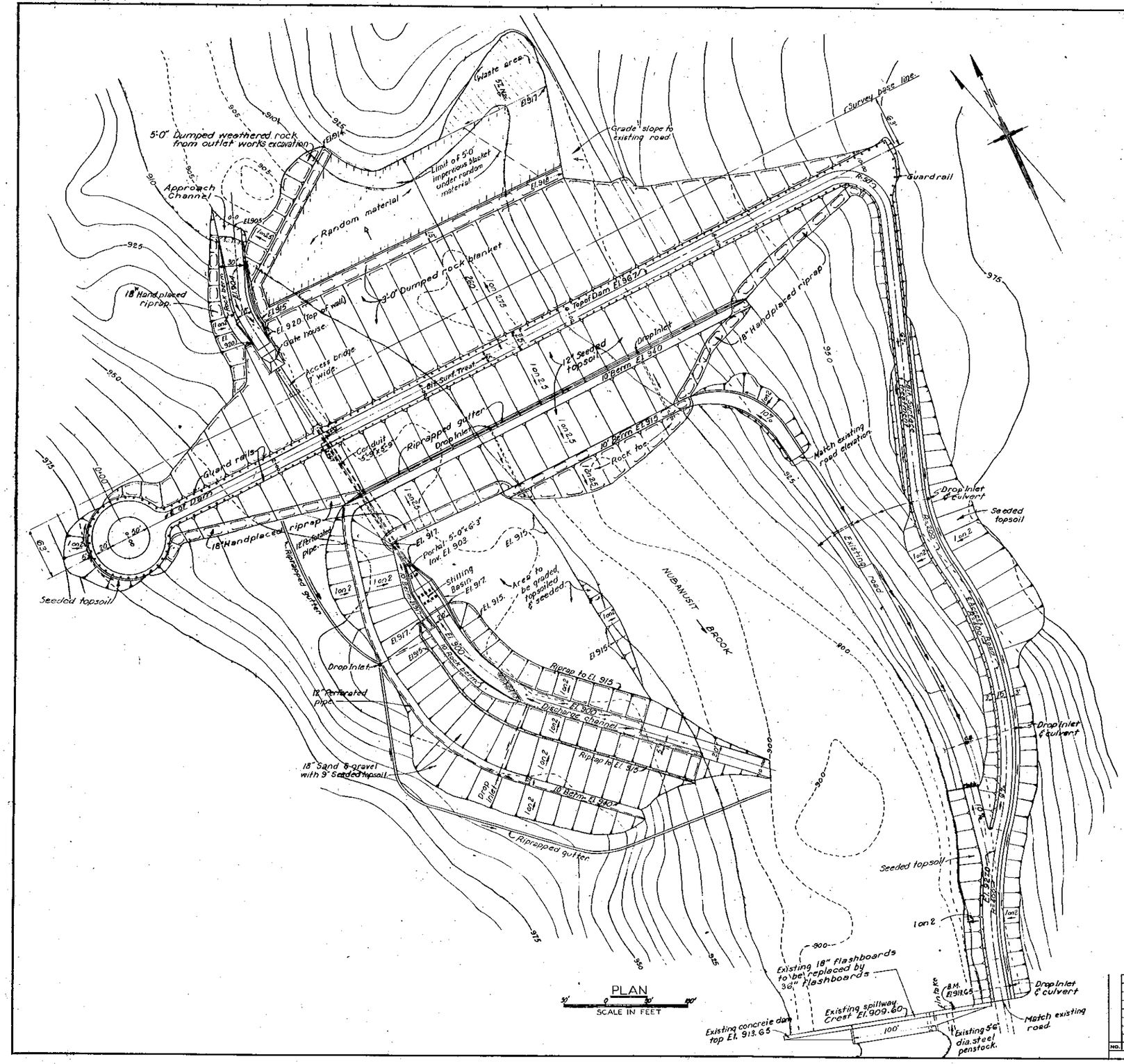
Prior to the completion of the final contract plans, one more meeting of the Board of Consultants will be held in order to review the contract plans.

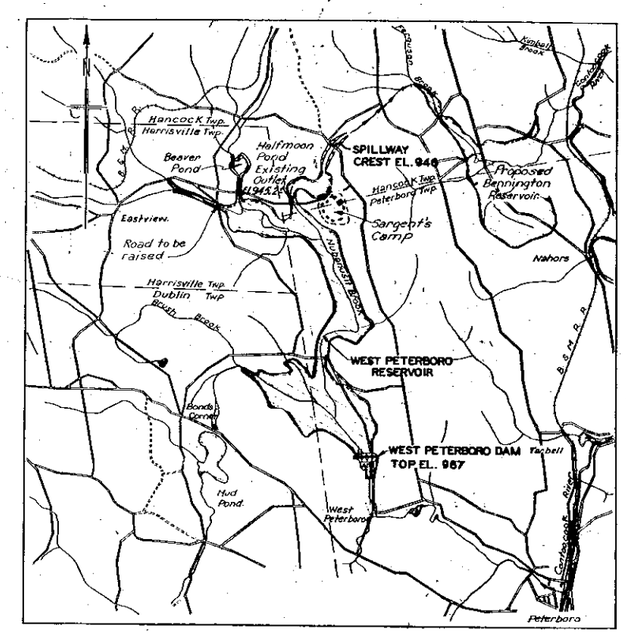
c. Description of Structures.- (1) Dam Embankment.- The proposed dam, as shown on Plate IV-1, consists of a rolled earth embankment with a crest length of approximately 1100 feet and a maximum height of 69 feet with cross section as shown on Plate IV-2, the design of which is discussed in detail in Appendix II.

(2) Outlet Works.- The outlet works consist of a reinforced concrete intake structure, conduit through the dam and stilling basin, sections of which are shown on Plates IV-2 and IV-3. The intake structure contains the gated inlets and is connected with the top of the dam by means of a structural steel foot bridge. The conduit is a square section which passes through the embankment and has a series of cut-off collars for control of seepage through the dam. The size and shape of the conduit has been proportioned, at no additional expense, to permit the future installation of a penstock extension from the Verney Mills Dam as described in Paragraph 7. The stilling basin consists of a reinforced concrete slab with reinforced concrete baffles and end sill. Gravity type sections have been selected for the stilling basin walls in view of their greater durability. The outlet works are founded on bedrock and the stability analyses are for the structures as shown on Plate IV-6.

(3) Spillway.- The spillway is located northeasterly of Half Moon Pond and in a saddle which separates drainage to Nubanusit Brook and to Ferguson Brook. The most economical channel width and weir length were computed in relation to the corresponding dam height required. The spillway channel in the vicinity of the weir is cut through rock, with the height of the north face increased by means of a gravity concrete wall to elevation 987 to facilitate a 25 foot concrete bridge crossing over the weir. The spillway training walls serve as abutments for the bridge. A

SUPERSEDED





LOCATION MAP

5000' 0 5000'
SCALE IN FEET

SUPERSEDED

NO.	CHARACTER	DATE	BY
	REVISIONS		

MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
NUBANUSIT BROOK

GENERAL PLAN OF DAM AND OUTLET

U. S. ENGINEER OFFICE, BOSTON, MASS. MAY 1945

APPROVAL RECOMMENDED: John E. Allen, Engineering Division

APPROVED: Colonel, Corps of Engineers, District Engineer

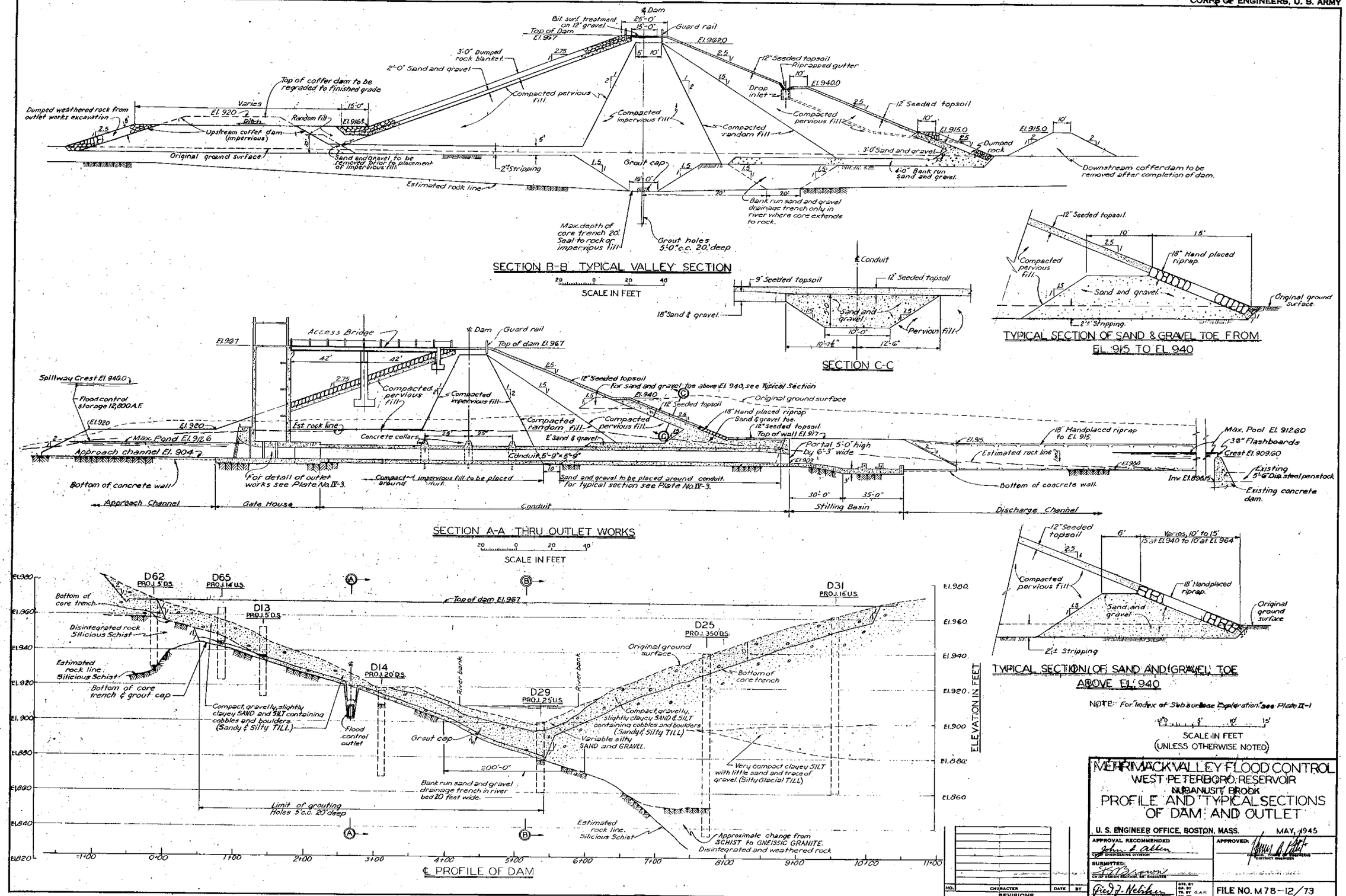
SUBMITTED: Chief Design Section, Eng. Division

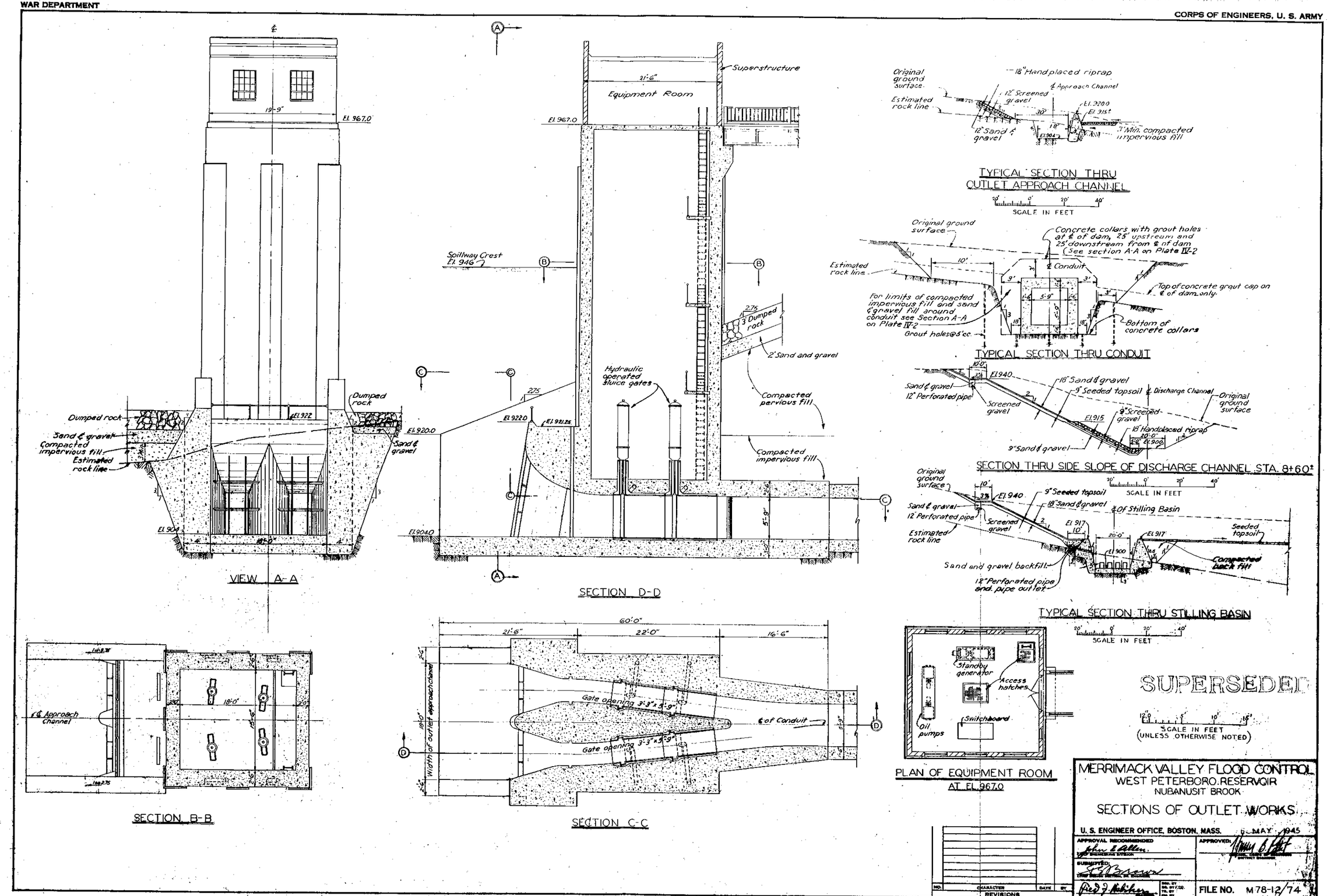
Fred J. Nebiker, Engineer

DES. BY
DR. BY L.S.
TR. BY
CH. BY

FILE NO. M 78 - 12/72

PLATE IV-1


SECTION B-B TYPICAL VALLEY SECTION
SCALE IN FEET
SECTION C-C
TYPICAL SECTION OF SAND & GRAVEL TOE FROM EL. 915 TO EL. 940
SECTION A-A THRU OUTLET WORKS
SCALE IN FEET
TYPICAL SECTION OF SAND AND GRAVEL TOE ABOVE EL. 940
NOTE: For "Index of Subsurface Exploration" see Plate II-1
SCALE IN FEET (UNLESS OTHERWISE NOTED)
℄ PROFILE OF DAM
ELEVATION IN FEET
MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
NUBANUSIT BROOK
PROFILE AND TYPICAL SECTIONS OF DAM AND OUTLET
U. S. ENGINEER OFFICE, BOSTON, MASS.
MAY, 1945
APPROVAL RECOMMENDED
APPROVED:
SUBMITTED:
REVISIONS
FILE NO. M78-12/73
SUPERSEDED
PLATE IV-2



MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
NUBANUSIT BROOK

SECTIONS OF OUTLET WORKS

U. S. ENGINEER OFFICE, BOSTON, MASS. MAY 1945

APPROVAL RECOMMENDED: John E. Allen, CHIEF ENGINEERING DIVISION

APPROVED: [signature], COLONEL, CORPS OF ENGINEERS, DISTRICT ENGINEER

SUBMITTED: [signature], CHIEF DESIGN SECTION, DIV. ENGINEER

[signature] DESIGNER

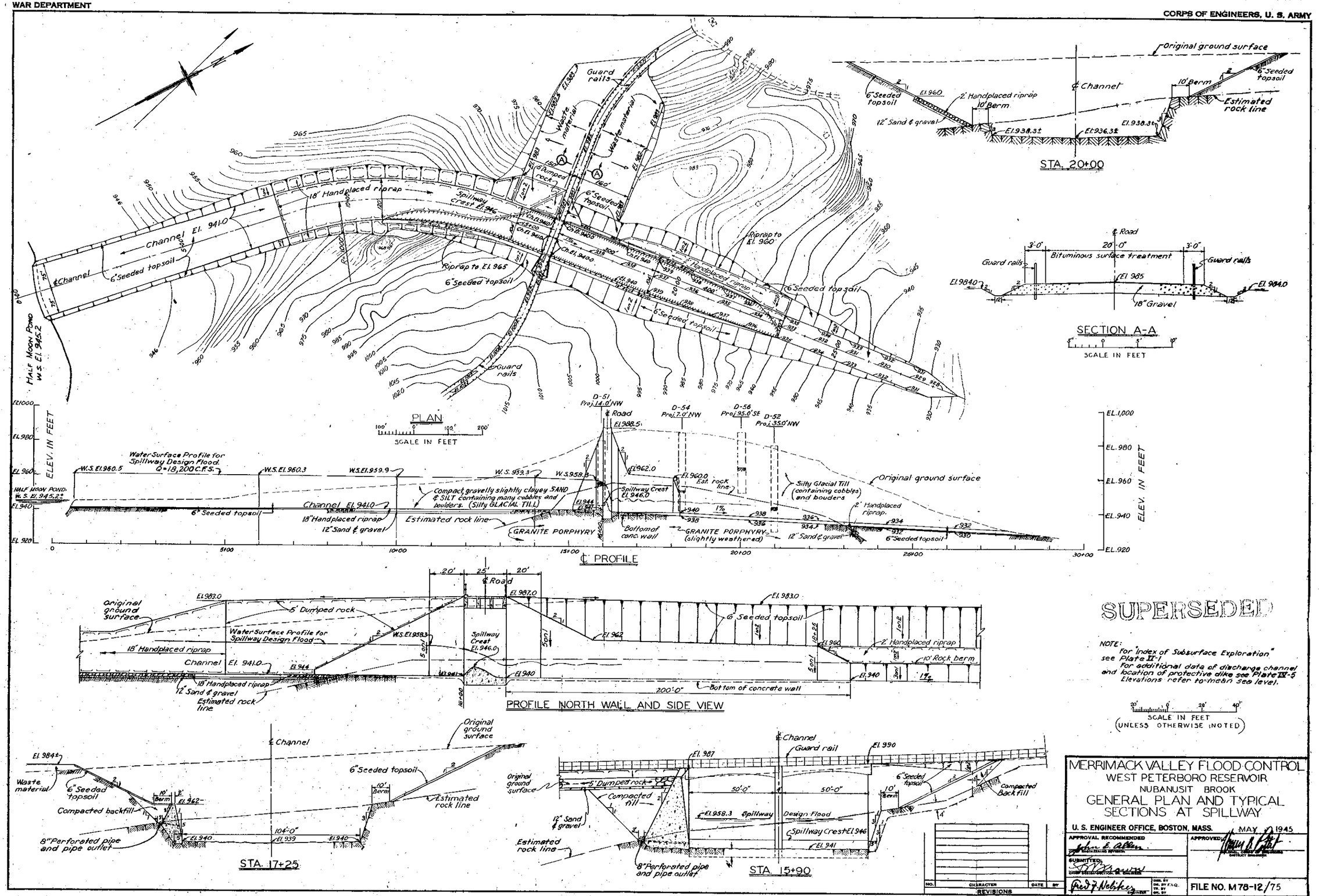
FILE NO. M78-12/74

NO.	CHARACTER	DATE	BY
	REVISIONS		

PLATE IV-3

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY



SUPERSEDED

NOTE:
For "Index of Subsurface Exploration" see Plate II-1
For additional data of discharge channel and location of protective dike see Plate IV-5
Elevations refer to mean sea level.

SCALE IN FEET (UNLESS OTHERWISE NOTED)

MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
NUBANUSIT BROOK
GENERAL PLAN AND TYPICAL SECTIONS AT SPILLWAY

U. S. ENGINEER OFFICE, BOSTON, MASS. MAY 1945

APPROVAL RECOMMENDED: John E. Allen, CHIEF, MAIN ENGINEERING DIVISION
APPROVED: [signature], COLONEL, CORPS OF ENGINEERS, DISTRICT ENGINEER
SUBMITTED: [signature], CHIEF DESIGN SECTION, DR. ENGINEER
Fred J. Nelikes, ENGINEER

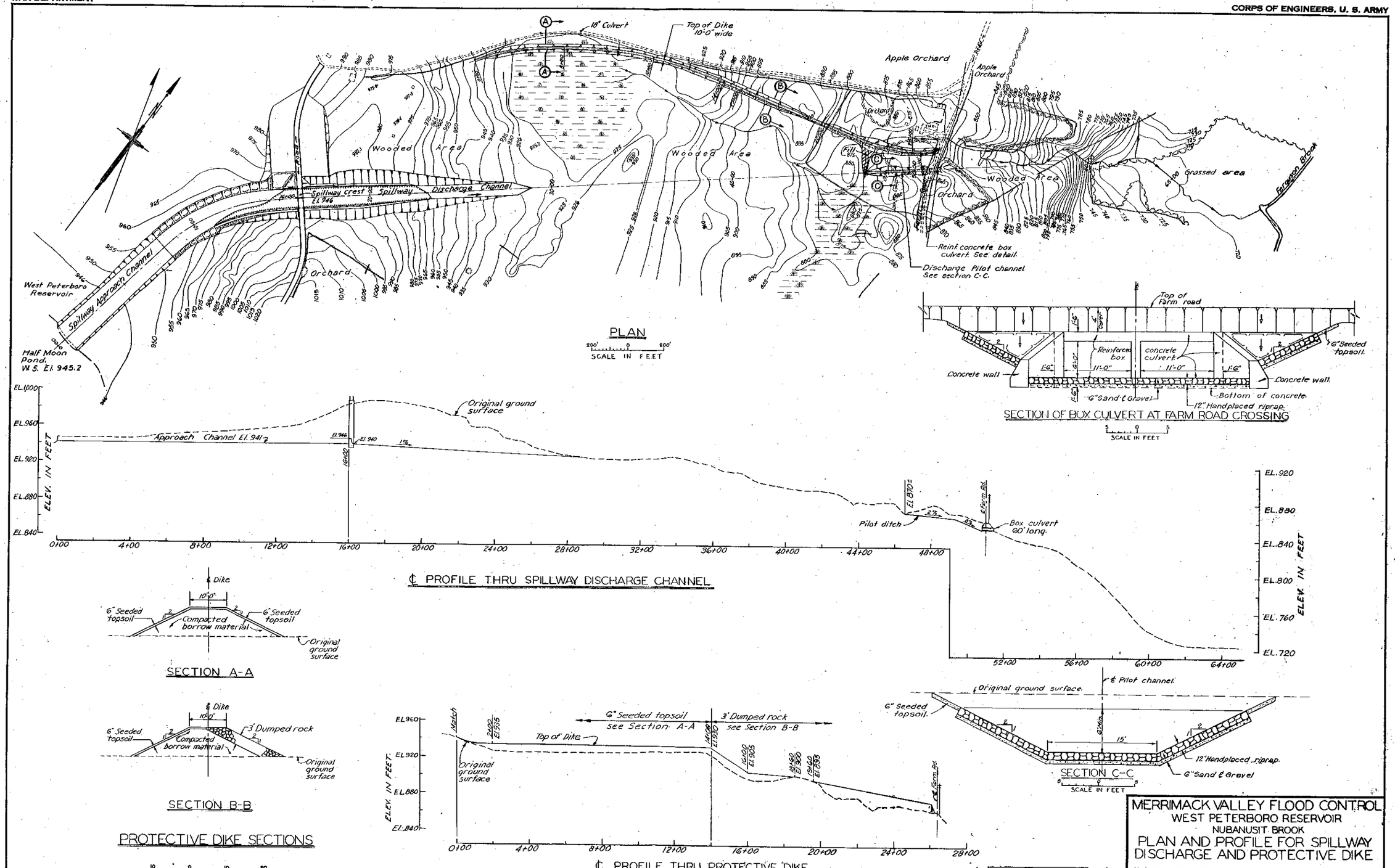
NO.	CHARACTER	DATE	BY
	REVISIONS		

FILE NO. M78-12/75

PLATE IV-4

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY



MERRIMACK VALLEY FLOOD CONTROL
WEST PETERBORO RESERVOIR
NUBANUSIT BROOK
PLAN AND PROFILE FOR SPILLWAY DISCHARGE AND PROTECTIVE DIKE

U. S. ENGINEER OFFICE, BOSTON, MASS. MAY 1945

APPROVAL RECOMMENDED: John E. Allen, CHIEF ENGINEERING DIVISION

APPROVED: Thomas B. Pitt, COLONEL, CORPS OF ENGINEERS, DISTRICT ENGINEER

SUBMITTED: S. B. Brown, CHIEF DESIGN SECTION, DIV. ENGINEER

Fred J. Nebiker, ENGINEER

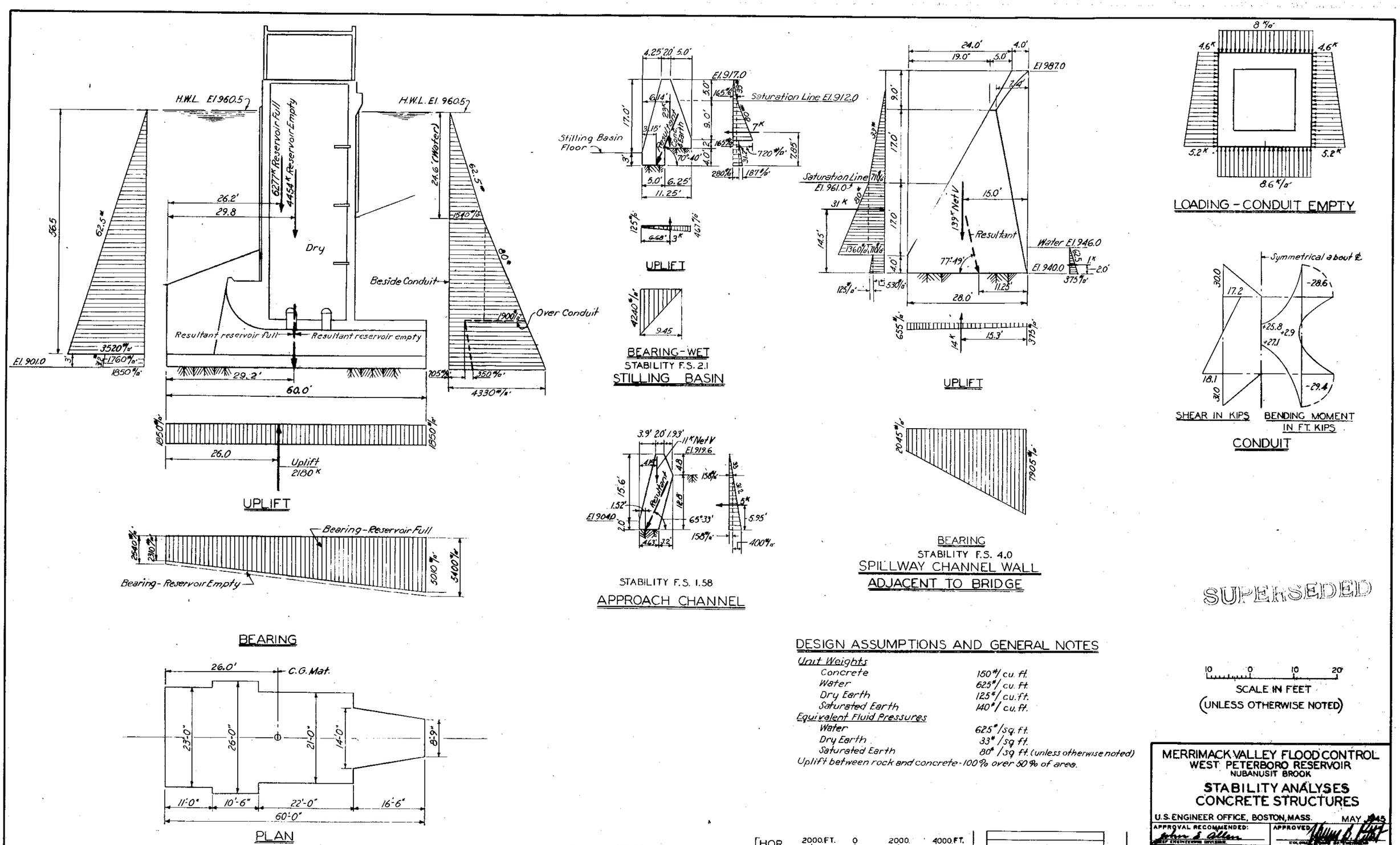
DES. BY, DR. BY, TR. BY, CH. BY

FILE NO. M78-12/76

NO.	CHARACTER	DATE	BY
	REVISIONS		

SUPERSEDED

PLATE IV-5



DESIGN ASSUMPTIONS AND GENERAL NOTES

Unit Weights	
Concrete	150#/cu. ft.
Water	625#/cu. ft.
Dry Earth	125#/cu. ft.
Saturated Earth	140#/cu. ft.
Equivalent Fluid Pressures	
Water	625#/sq. ft.
Dry Earth	33#/sq. ft.
Saturated Earth	80#/sq. ft. (unless otherwise noted)

Uplift between rock and concrete - 100% over 50% of area.

PLATE IV-6

War Department
United States Engineer Office
Boston, Massachusetts

DEFINITE PROJECT REPORT

WEST PETERBORO RESERVOIR

APPENDIX V

HYDROELECTRIC POWER

SUPERSEDED

To accompany definite project report
dated May 1945

3. Basis of Cost Analysis.- The evaluation of power benefits to be derived by a hydroelectric plant located at the proposed dam site and the existing downstream hydroelectric installations with the reservoir operated to increase the low water flows, has been based on the estimated costs for the production of equivalent power by a steam plant located at a load center such as Manchester, New Hampshire. The construction cost of such a steam plant is estimated to be $102 per KW., and the annual cost, consisting of the fixed charges and allowance for operation and maintenance, is estimated to be $17.50 per kilowatt per year. This amount is the value assigned per kilowatt of dependable prospective hydroelectric capacity. The value of energy output, based on the current cost of coal fuel at Manchester, is estimated at about 3 mills per kilowatt hour. An allowance of 10% has been made for losses in transmission.

4. Conclusion.- The studies of the benefits to be derived from the development of a multiple-purpose reservoir and the current high level of construction costs, indicate that second-stage construction cannot be economically justified. The ratio of annual benefits to carrying charges for stream regulation storage is 0.66 and for stream regulation and hydroelectric power generation at site is 0.73. These low ratios do not warrant making provision in the initial construction for the ultimate development of the reservoir for multiple-purpose use.

SUPERSEDED

TABLE A

COST ANALYSIS - 2ND STAGE CONSTRUCTION
FOR FLOOD CONTROL AND POWER STORAGE

	Plan 1 Raising of Dam and Spillway For Stream Regulation Storage		Plan 2 Same as Plan 1 with Power Installation at Site	
1. RESERVOIR DATA				
Top of Dam - Elev.	985		985	
Saddle Spillway - Elev.	968		968	
Flood Control Storage Capacity	12,800 A.F.	5.5"	12,800 A.F.	5.5"
Conservation Storage Capacity	31,200 A.F.	13.3"	25,000 A.F. -El.961	
Storage at Max. Drawdown			6,200 A.F. -El.932	
Total Storage Capacity	44,000 A.F.	18.8"	44,000 A.F.	18.8"
Average Annual Yield	24,000 A.F.		22,000 A.F.	
Water Utilization Factor	80%		60%	
2. ESTIMATED COST OF 2ND STAGE CONSTRUCTION (Including Overhead and Contingencies)				
Cost of Reservoir and Dam:				
Reservoir Costs (incl. Bldgs. and road relocations		$ 710,000		$ 710,000
Earth Dam and Outlet		330,000		330,000
Saddle Spillway		73,000		73,000
Reservoir Clearing		274,000		274,000
Sub-total		$1,387,000		$1,387,000
Cost of Power Installation for 1,600 K.W. Capacity				664,000
TOTAL ESTIMATED COST OF 2ND STAGE CONSTRUCTION		$1,387,000		$2,051,000
Cost per acre foot of conservation storage	$44.46		$55.48	
Cost per acre foot of Average Annual Yield	$57.79		$63.05	

SUPERSEDED

TABLE A (Cont'd.)

COST ANALYSIS - 2ND STAGE CONSTRUCTION
FOR FLOOD CONTROL AND POWER STORAGE

	Plan 1 Raising of Dam and Spillway For Stream Regulation Storage	Plan 2 Same as Plan 1 with Power Installation at Site
3. ANNUAL CARRYING CHARGES OF 2ND STAGE CONSTRUCTION COST	$ 55,910	$ 94,760
4. ANNUAL POWER BENEFITS		
(a) Power Installation at Site:		
1. 1,600 K.W. @ 17.50		$ 28,000
2. 4,500,000 K.W.H. @ 3 Mills		$ 13,500
(b) Increased Output at Existing Downstream Plants		
Increase in prime peaking capacity		
1,000 K.W. @ $17.50	$ 17,500	
800 K.W. @ 17.50		$ 14,000
Increase in average annual energy output with existing head downstream of 411 feet.		
6,400,000 K.W.H. @ 3 Mills	$ 19,200	
4,500,000 K.W.H. @ 3 Mills		$ 13,500
TOTAL ANNUAL POWER BENEFITS	$ 36,700	$ 69,000
5. RATIO OF ANNUAL BENEFITS TO ANNUAL CARRYING CHARGES	0.66	0.73

SUPERSEDED

DEFINITE PROJECT REPORT
WEST PETERBORO RESERVOIR

APPENDIX VI - RELOCATIONS

a. Introduction.- The reservoir basin at spillway crest elevation 946 contains only short sections of roads and a small number of bridges and utilities which will be affected by the inundation of the area during periods of high water stages. Approximately one mile of a third class road, with accompanying power and telephone lines, requires relocations and one-half mile of third class road requires raising. Several small wooden bridges must be raised to provide the necessary clearance for flood flows and a box culvert installed on the Sargent School Road, as indicated on Plate VI-1. In order to provide a connection for High Wind Road at the spillway, it is proposed to construct a concrete bridge over the spillway and use the guide walls as abutments. The total cost of the above-noted relocations is estimated to be $91,000, excluding the spillway bridge which is included in the construction costs.

b. Roads.- The third class gravel road running parallel to and on the east side of the brook between Swetts Dam Bridge and the proposed dam, is subject to inundation for a distance of approximately one mile and is to be abandoned. It is proposed to replace this portion of the road by regrading and resurfacing an existing farm road which runs from Swetts Dam Bridge to High Wind Road which is approximately one-half mile east of the abandoned road. This farm road is to be improved by regrading and surfacing with gravel.

Low sections in the existing third class gravel surfaced road passing the Sargent School Camp are to be raised approximately three (3') feet in elevation for a distance of about one-half mile and the culvert at the outlet from Half Moon Pond is to be replaced and lengthened.

Highway traffic can be maintained during the various stages of construction by rerouting until such time as the proposed raising of roads and relocations are completed.

The Chief Engineer of the Highway Department, State of New Hampshire has been consulted and has agreed that the relocations proposed are a reasonable solution for raising and relocating the network of roads affected by construction of the proposed dam.

c. Bridges.- The Swetts Dam Bridge, the bridge over Beaver Brook and the two bridges over upper Nubanusit Brook are to be raised from three to five feet in elevation by reconstructing and adding to their masonry abutments. The decks of these bridges are constructed of wood planks laid on steel girders.

SUPERSEDED