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MERRIMACK VALLEY FLOOD CONTROL

ANALYSIS OF DESIGN

BLACKWATER DAM

BLACKWATER RIVER

1940

CORPS OF ENGINEERS, U.S. ARMY

U.S. ENGINEER OFFICE BOSTON, MASS.

E.W. WEBER

407 - 670A

NOTE:

Appendices A, B, and C are bound separately.

Appendix A is being transmitted with the Analysis of Design proper.

Copies of the folders comprising Appendices B and C were submitted January 10, 1939 and November 20, 1939, respectively. Copies of new cover sheets to be substituted thereon are attached herewith.

A N A L Y S I S O F D E S I G N

B L A C K W A T E R D A M

Table of Contents

Paragraph	Subject	Page
	I. INTRODUCTION	
1-01	Comprehensive Plan for Flood Control - Merrimack Valley ...	I-1
1-02	Blackwater Initial Development	I-1
1-03	Blackwater Ultimate Development	I-2
1-04	Effect of Blackwater Reservoir in Reducing Floods	I-2
1-05	Conservation Storage	I-2

Figures

I-1 Map of Merrimack River Basin
I-2 Blackwater Ultimate Development - General Plan and Elevations
I-3 Blackwater Ultimate Development - Sections

	II. SELECTION OF SITE AND TYPE OF DAM	
2-01	Selection of the Dam Site	II-1
2-02	Selection of Type of Dam	II-1
	III. FLOOD CONTROL HYDRAULICS	
3-01	Hydrology Report	III-1
3-02	General Hydrology	III-1
3-03	Precipitation	III-1
3-04	Run-off	III-2
3-05	Reservoir Capacity	III-3
3-06	Outlet Hydraulics	III-3
3-07	Spillway Hydraulics	III-5

Figures

III-1 Rainfall Studies - Location of Precipitation Stations
III-2 Inflow Unit Hydrograph
III-3 Area and Capacity Curves
III-4 Reservoir Design Flood - Inflow, Outflow, Stage, Hydrographs - Initial Development
III-5 Effect of Blackwater Reservoir on the 1936 Flood - Initial Development
III-6 Percent of Computed Spillway Flood vs Peak Flow and Pool Elevation - Initial Development
III-7 Inflow - Outflow Stage Hydrograph for Spillway Design Flood - Initial Development
III-8 Percent of Computed Spillway Flood vs Peak Flow and Pool Elevation - Ultimate Development
III-9 Inflow - Outflow Stage Hydrograph for Spillway Design Flood - Ultimate Development

Table of Contents (Cont'd.)

Paragraph	Subject	Page
	IV. HYDRAULIC DESIGN	
4-01	Location of Discharge Structures	IV-1
4-02	Spillway Design	IV-1
4-03	Outlet Design	IV-3
4-04	Freeboard	IV-5
4-05	Model Studies	IV-6
4-06	Conclusions	IV-6

Figures

IV-1 Spillway Discharge Rating Curves
IV-2 Intake Velocities - Gated Outlets
IV-3 Discharge Capacity of Gated Outlets
IV-4 Discharge Capacity of Gated Outlets
IV-5 Discharge Capacity of Ungated Outlet
IV-6 Discharge Capacity of Ungated Outlet
IV-7 Hydraulic Characteristics of Outlets
IV-8 Rating Curves of Outlets
IV-9 Rating Curve of Outlets - Initial Development
IV-10 Rating Curve of Outlets - Ultimate Development
IV-11 Outlets - Energy and Hydraulic Gradient - Ultimate Development
IV-12 Energy and Hydraulic Gradient in Gated Outlet

	V. INVESTIGATIONS OF FOUNDATIONS AND MATERIALS FOR DAM EMBANKMENT, DIKES, AND CONCRETE AGGREGATE	
5-01	Field Exploration of Foundations	V-1
5-02	Exploration of Materials for the Dam Embankment, Dikes, and Concrete Aggregate	V-2
5-03	Method and Extent of Field Tests	V-3
5-04	Methods and Extent of Laboratory Tests and Investigations	V-3

Figure

V-1 Plan of Borrow Exploration

	VI. DESIGN OF DAM EMBANKMENT AND DAM EMBANKMENT FOUNDATION	
6-01	General	VI-1
6-02	Geological Features and Characteristics of Foundation Materials	VI-2

Table of Contents (Cont'd.)

Paragraph	Subject	Page
6-03	Availability and Characteristics of Dam Embankment Materials	VI-2
6-04	Dam Embankment Sections.	VI-4
6-05	Location of Centerline	VI-5
6-06	Economy of Construction.	VI-6
6-07	Statical Analysis.	VI-6
6-08	Flow Failure Analysis.	VI-7
6-09	Seepage Analysis	VI-8
6-10	Miscellaneous Analyses	VI-9

Figures

VI-1 Rock Contour Map - Blackwater Dam Site
VI-2 Summary of Soil Test Data - Dam Embankment
VI-3 Grain Size Curves - Typical Foundation Materials Upstream from Centerline, Dam Embankment
VI-4 Critical Void Ratio and Shearing Strength Test Results - Upstream Foundation Material, Dam Embankment
VI-5 Consolidation Test Results - Upstream Foundation Material, Dam Embankment
VI-6 Grain Size Curves - Materials Tested for Critical Void Ratio, Dam Embankment
VI-7 Grain Size Curves - Range of Pervious and Impervious Materials, Dam Embankment
VI-8 Critical Void Ratio and Shearing Strength Test Results - Taylor-Fidler Method, Pervious Material, Dam Embankment
VI-9 Critical Void Ratio and Shearing Strength Results - Harvard Method, Pervious Material, Dam Embankment
VI-10 Consolidation Test Results - Pervious Material, Dam Embankment
VI-11 k-e Curves - Representative Embankment Materials
VI-12 Shearing Strength Test Results - Impervious Material, Dam Embankment
VI-13 Optimum Moisture Content Curve - Impervious Material, Dam Embankment
VI-14 Major Sections Considered - Dam Embankment
VI-15 Results of Stability Analysis - Dam Embankment
VI-16 Details of Stability Analysis - Dam Embankment
VI-17 Details of Stability Analysis Computations - Dam Embankment
VI-18 Computations for Drainage of Compacted Pervious Material - Dam Embankment
VI-19 Flow Net and Seepage Analysis - Dam Embankment

Table of Contents (Cont'd.)

Paragraph	Subject	Page
	VII. DESIGN OF DODGE ESTATE DIKE	
7-01	Characteristics of Site	VII-1
7-02	Description of Dike Section	VII-1
7-03	Analysis of Section	VII-1

Figures

VII-1 Grain Size Curves - Range of Fill and Foundation Materials, Dodge Estate Dike
VII-2 Summary of Soil Test Data - Dodge Estate Dike

	VIII. DESIGN OF LITTLE HILL DIKE	
8-01	General	VIII-1
8-02	Geological Features and Characteristics of Foundation Materials	VIII-1
8-03	Availability and Characteristics of Dike Materials	VIII-2
8-04	Dike Sections	VIII-3
8-05	Economy of Construction	VIII-4
8-06	Statical Analysis	VIII-5
8-07	Flow Failure Analysis	VIII-5
8-08	Seepage Analysis	VIII-5
8-09	Miscellaneous Analyses	VIII-6

Figures

VIII-1 Summary of Soil Test Data - Little Hill Dike
VIII-2 Grain Size Curves - Typical Foundation Materials, Little Hill Dike
VIII-3 Grain Size Curves - Typical Pervious Material, Little Hill Dike
VIII-4 Critical Void Ratio and Shearing Strength Test Results - Pervious Material, Little Hill Dike
VIII-5 Grain Size Curves - Materials Tested for Critical Void Ratio or Shearing Strength, Little Hill Dike
VIII-6 Consolidation Test Results - Pervious Material, Little Hill Dike
VIII-7 Grain Size Curves - Range of Impervious Material, Little Hill Dike
VIII-8 Shearing Strength Test Results - Impervious Material, Little Hill Dike
VIII-9 Optimum Moisture Content Curve - Impervious Material, Little Hill Dike

Table of Contents (Cont'd.)

Paragraph	Subject	Page
	Figures	
	VIII-10 Major Sections Considered - Little Hill Dike	
	VIII-11 Results of Stability Analysis - Little Hill Dike	
	VIII-12 Flow Net and Seepage Analysis - Little Hill Dike	
	IX. STRUCTURAL DESIGN	
9-01	General Standards	IX-1
9-02	Unit Weights	IX-1
9-03	Load Assumptions	IX-1
9-04	Concrete Design Standards	IX-1
9-05	Structural Steel Standards	IX-2
9-06	Structure Analysis	IX-3
	Figure	
	IX-1 Stability Analysis for Masonry Structures	
	X. CONSTRUCTION PROCEDURE	
10-01	Sequence of Operations	X-1
10-02	Field Control for Construction of Dam Embankment and Dikes	X-1
10-03	Dam Embankment Piezometers and Settlement Gages	X-3
10-04	Grouting	X-3
10-05	Concrete Construction	X-3
10-06	Structural Steel Construction	X-3
	XI. SUMMARY OF COST	
11-01	Estimated Cost	XI-1
	XII. CHANGES MADE DUE TO EXPERIENCE GAINED FROM CONSTRUCTION	
	(To be made part of the Analysis of Design after construction of the project is complete)	

APPENDICES (in separate volumes)

Letter		Title
A	-	Structural Design Computations
B	-	Compaction Tests and Critical Density Investigation of Earth Embankment Materials
C	-	Design Conference, November 10, 1939 - Synopsis and Folio

ANALYSIS OF DESIGN

BLACKWATER DAM

I INTRODUCTION

1-01. Comprehensive Plan for Flood Control - Merrimack Valley. (a) A "system of flood-control reservoirs in the Merrimack River Basin for the reduction of flood heights in the Merrimack Valley generally" was authorized by the Flood Control Act approved June 22, 1936. The "general comprehensive plan for flood control and other purposes", as outlined in H. Doc. No. 689, 75th Cong., 3d Sess., was approved and the project modified by the Flood Control Act approved June 28, 1938. The plan outlined in H. Doc. No. 689 provides for a system of reservoirs and related flood control works which may be found justified by the Chief of Engineers at an estimated cost of $13,000,000 for construction and $8,000,000 for lands and damages.

(b) Several alternate combinations of reservoirs to serve as a general comprehensive plan of flood control in the Merrimack Basin have been proposed, in all of which the Franklin Falls Reservoir now under construction forms a component part. Other reservoirs proposed for selection as parts of these systems are located mainly in the Contoocook Valley. Blackwater Reservoir is common to all of the most favorable alternate combinations of reservoirs in the Contoocook Valley and has been selected as the second reservoir of the comprehensive plan, as recommended in Definite Project Report for Blackwater Reservoir submitted September 15, 1939, and revised December 16, 1939. The locations of the Franklin Falls and Blackwater Reservoirs are shown on Figure I-1.

1-02. Blackwater Initial Development. (a) General. - The proposed initial flood control development of the Blackwater Dam will consist of a concrete spillway, a concrete penstock intake, a concrete non-overflow section, a rolled-earth retaining section, and two earth dikes (see Sheets 7, 11, 12 and 13 of the contract drawings).

(b) Concrete Structures. - The concrete spillway will consist of a wide-base, gravity-type Ogee section 240 feet long with the crest at Elevation 566.0 M.S.L. The outlet works will consist of 4 rectangular conduits through the spillway. A concrete penstock intake for a 14-foot diameter penstock will be constructed adjacent to the right end of the spillway. A concrete non-overflow section will connect the penstock intake and the dam embankment.

(c) Dam Embankment. - The dam embankment will be a semi-wide-base structure, about 750 feet long at the top, Elevation 584.0 M.S.L., with a maximum height of approximately 60 feet, a top width of 38 feet 10 inches, and slopes as shown on Sheet 8 of the contract drawings. It will be constructed of pervious material with an impervious core, a dumped rock toe, dumped rock fill on the upstream face, and seeded topsoil on the downstream face.

(d) Foundations. - All masonry structures, the impervious core, and the drainage toe of the dam embankment will be placed on bedrock, except that where depths to bedrock exceed 8 feet below the stripping line, the drainage toe will not be carried to bedrock.

(e) Dikes. - In addition to the dam, the initial development includes two rolled-earth dikes, each with top elevation of 584.0 M.S.L. and side slopes as shown on Sheets 11 and 12 of the contract drawings. Dodge Estate Dike will be about 420 feet long with a top width of 15 feet and a maximum height of approximately 18 feet. Little Hill Dike will be about 1230 feet long with a top width of 22 feet and a maximum height of approximately 30 feet.

1-03. Blackwater Ultimate Development. In view of the value of the Blackwater site for future combined storage and power development, provisions have been made in the design of the initial flood control development which will facilitate future alteration for the ultimate multi-purpose development. The nature of the alterations required for the ultimate development is indicated by Figures I-2 and I-3. The ultimate elevation of the top of the dam embankment and of the tops of the dikes is 600 M.S.L., with spillway crest at Elevation 584 M.S.L.

1-04. Effect of Blackwater Reservoir in Reducing Floods. For the initial development, Blackwater Reservoir will have a storage capacity of 46,000 acre-feet, equivalent to 6.8 inches of run-off over its drainage area of 127.5 square miles. The area controlled comprises 17 percent of the Contoocook watershed and approximately 2.6 percent of the entire Merrimack drainage basin. The annual flood control benefits for Blackwater Reservoir considered above are estimated at $140,000, and stage reductions for a flood similar to that of March 1936 at Manchester, N. H., will be about 1.1 feet. The Blackwater Reservoir, in conjunction with the Franklin Falls Reservoir, will control 22.6 percent of the total Merrimack Basin area and will afford flood control benefits of $763,700 annually.

1-05. Conservation Storage. (a) Initial Development. - The initial development, for flood control only, will have no portion of the storage allocated for conservation or other purposes.

(b) Ultimate Development. - The ultimate development will provide the requisite flood control storage of 46,000 acre-feet between Elevations 573 and 584 M.S.L. and, in addition, conservation storage of 69,000 acre-feet below Elevation 573 M.S.L. Economic studies indicate that this conservation storage capacity is the most economical, resulting in the lowest unit cost of storage benefits. Power development at the reservoir site, involving a 10,000-foot penstock and a 10,000-kw. installation, can also be undertaken as part of the ultimate development.

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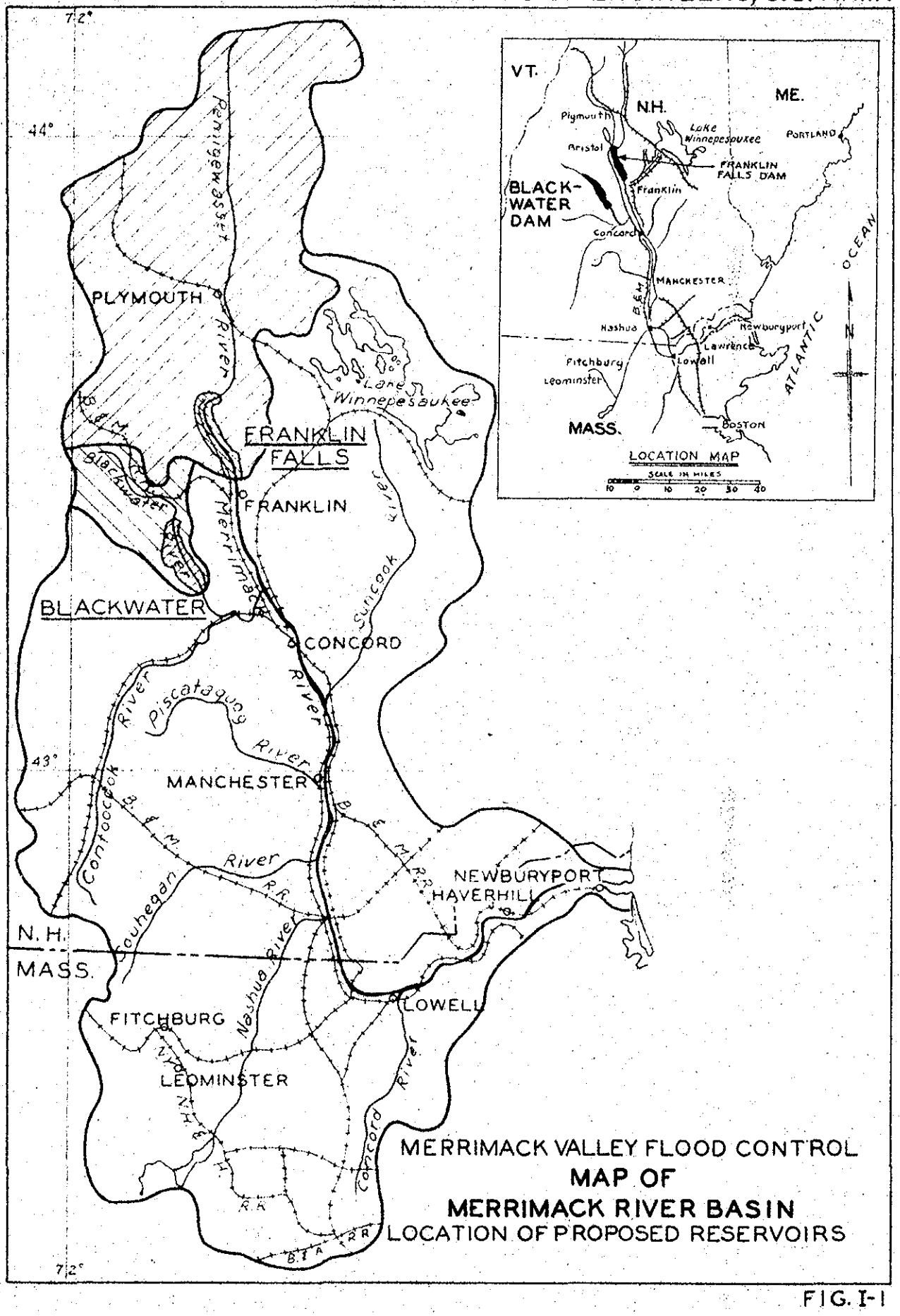
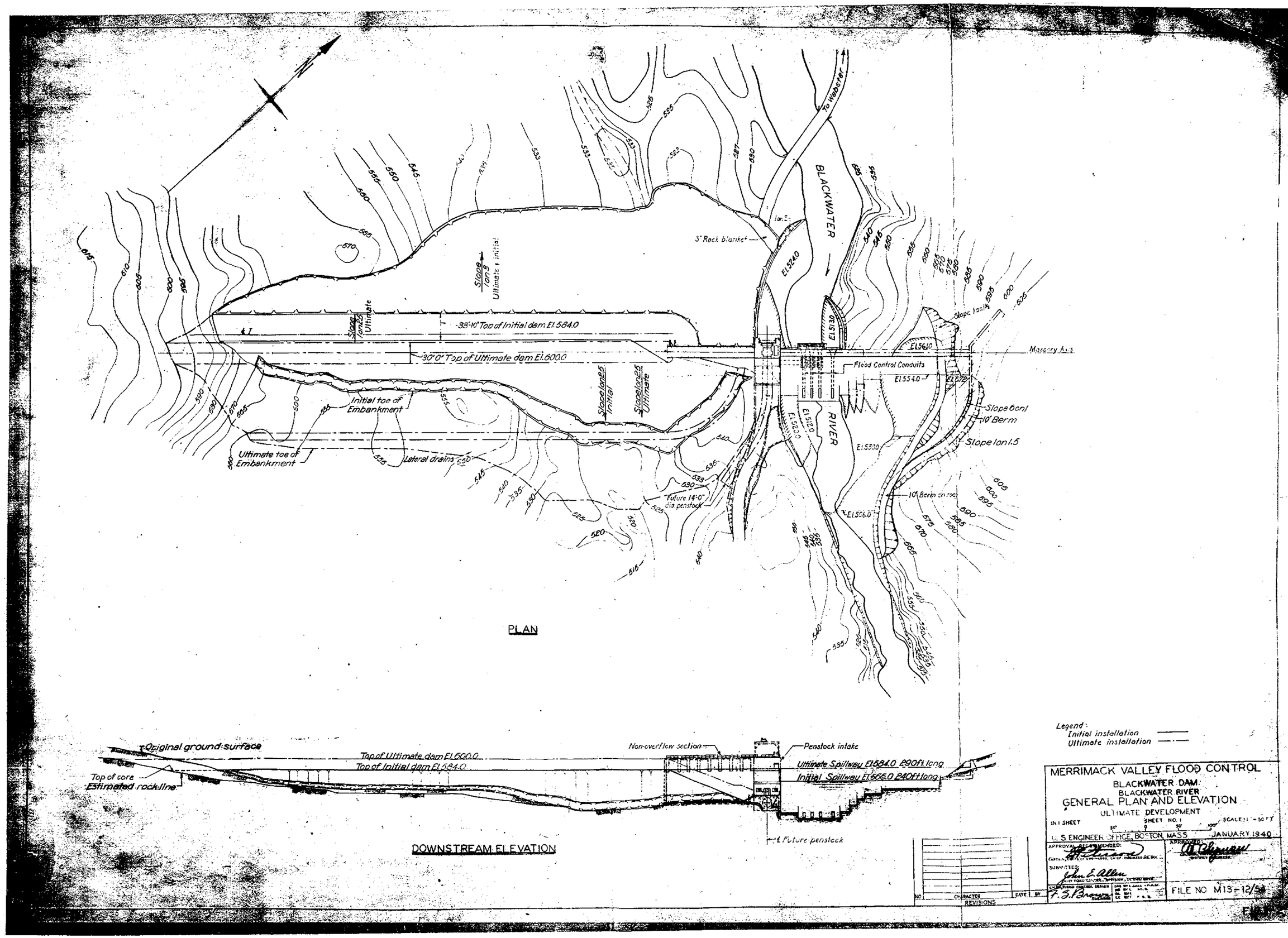
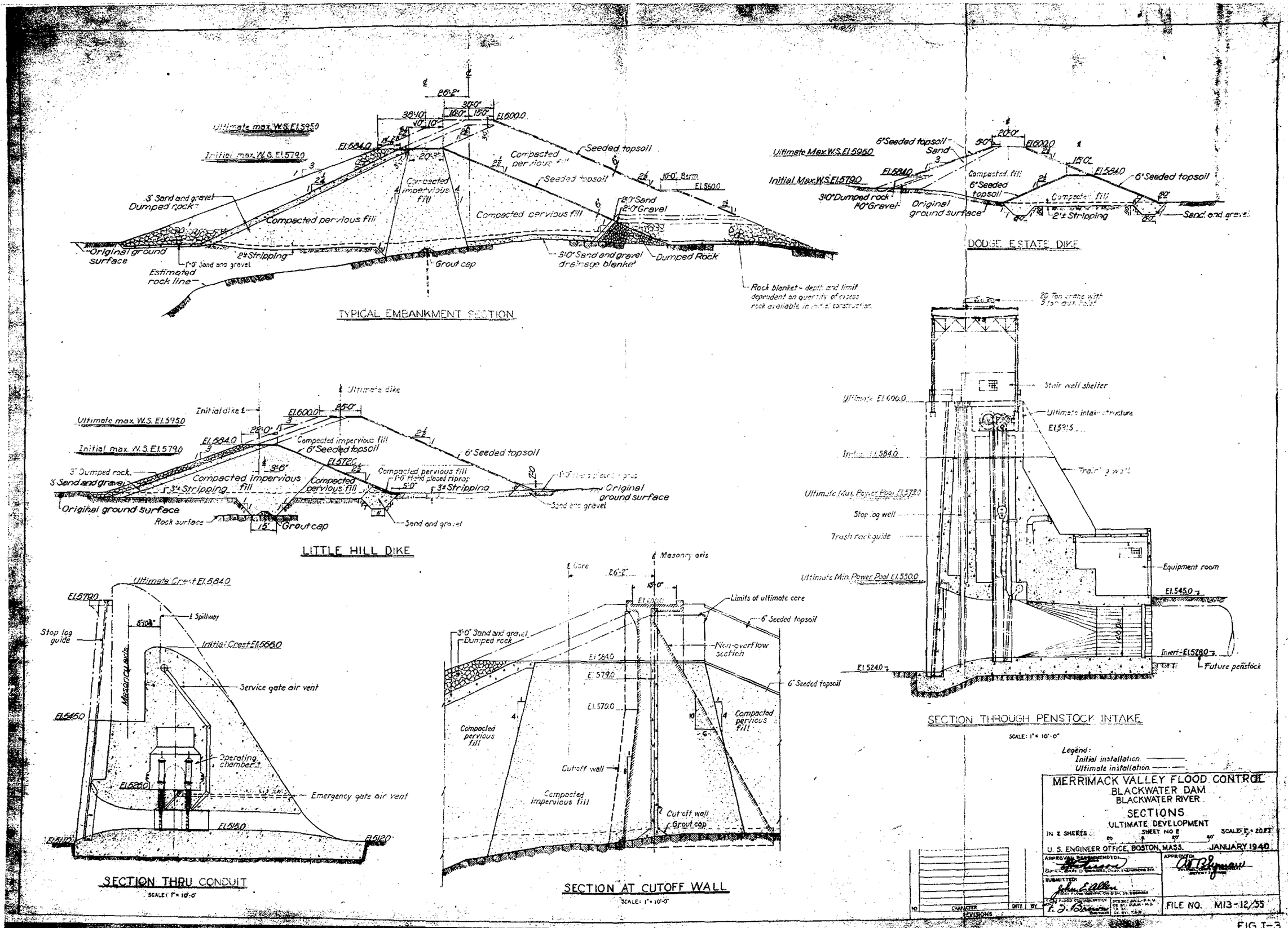


MERRIMACK VALLEY FLOOD CONTROL
MAP OF
MERRIMACK RIVER BASIN
LOCATION OF PROPOSED RESERVOIRS

FIG. I-1



To Webster
BLACKWATER
RIVER
3' Rock blanket
Slope 1 on 3 Ultimate & initial
Slope 1 on 2.5 Ultimate
38'-10" Top of Initial dam El. 584.0
30'-0" Top of Ultimate dam El. 600.0
Initial toe of Embankment
Ultimate toe of Embankment
Lateral drains
Slope 1 on 2.5 Initial
Slope 1 on 2.5 Ultimate
Future 14'-0" dia penstock
Flood Control Conduits
Masonry Axis
Slope 1 on 1½
Slope 6 on 1
10' Berm
Slope 1 on 1.5
10' Berm on rock
El. 524.0
El. 513.0
El. 561.0
El. 554.0
El. 572
El. 512.0
El. 520.0
El. 550.0
El. 506.0
PLAN
Original ground surface
Top of core
Estimated rock line
Top of Ultimate dam El. 600.0
Top of Initial dam El. 584.0
Non-overflow section
Penstock intake
Ultimate Spillway El. 584.0 290 ft long
Initial Spillway El. 566.0 240 ft long
℄ Future penstock
DOWNSTREAM ELEVATION
Legend:
Initial installation
Ultimate installation
MERRIMACK VALLEY FLOOD CONTROL
BLACKWATER DAM
BLACKWATER RIVER
GENERAL PLAN AND ELEVATION
ULTIMATE DEVELOPMENT
IN 1 SHEET
SHEET NO. 1
SCALE 1" = 50 FT
U. S. ENGINEER OFFICE, BOSTON, MASS.
JANUARY 1940
APPROVAL RECOMMENDED:
SUBMITTED:
John E. Allen
APPROVED:
FILE NO. M13-12/54
NO.
CHARACTER
DATE
BY
REVISIONS



TYPICAL EMBANKMENT SECTION
Ultimate max. W.S. El. 595.0
Initial max. W.S. El. 579.0
El. 584.0
El. 600.0
Seeded topsoil
Compacted pervious fill
Compacted impervious fill
3' Sand and gravel
Dumped rock
Original ground surface
2'± Stripping
1'-0" Sand and gravel
Estimated rock line
Grout cap
3'-0" Sand and gravel drainage blanket
Dumped Rock
Rock blanket - depth and limit dependent on quantity of excess rock available in initial construction
DODGE ESTATE DIKE
Ultimate Max. W.S. El. 595.0
Initial Max. W.S. El. 579.0
6" Seeded topsoil
Sand
Compacted fill
3'-0" Dumped rock
1'-0" Gravel
Original ground surface
2'± Stripping
Sand and gravel
LITTLE HILL DIKE
Ultimate dike
Initial dike
Compacted impervious fill
6" Seeded topsoil
3' Dumped rock
3' Sand and gravel
3'± Stripping
Compacted pervious fill
Original ground surface
Rock surface
Grout cap
Sand and gravel
SECTION THROUGH PENSTOCK INTAKE
SCALE: 1" = 10'-0"
Stair well shelter
Ultimate intake structure
Training wall
Ultimate Max. Power Pool El. 572.0
Stop log well
Trash rack guide
Equipment room
Ultimate Min. Power Pool El. 550.0
El. 545.0
Invert - El. 528.0
Future penstock
El. 524.0
SECTION THRU CONDUIT
SCALE: 1" = 10'-0"
Ultimate Crest El. 584.0
El. 579.0
Stop log guide
Spillway
Initial Crest El. 566.0
Masonry axis
Service gate air vent
El. 545.0
Operating chamber
El. 526.0
Emergency gate air vent
El. 515.0
El. 511.0
El. 512.0
SECTION AT CUTOFF WALL
SCALE: 1" = 10'-0"
Masonry axis
Core
Limits of ultimate core
6" Seeded topsoil
3'-0" Sand and gravel
Dumped rock
Non-overflow section
El. 584.0
El. 579.0
El. 570.0
Compacted pervious fill
Cutoff wall
Compacted impervious fill
Grout cap
Legend:
Initial installation
Ultimate installation
MERRIMACK VALLEY FLOOD CONTROL
BLACKWATER DAM
BLACKWATER RIVER
SECTIONS
ULTIMATE DEVELOPMENT
IN 2 SHEETS
SHEET NO 2
U. S. ENGINEER OFFICE, BOSTON, MASS.
JANUARY 1940
APPROVAL RECOMMENDED:
APPROVED:
SUBMITTED:
FILE NO. M13-12/55
REVISIONS

FIG. I-3

II SELECTION OF SITE AND TYPE OF DAM

2-01. Selection of the Dam Site. In the general reach of the river available for location of the Blackwater Dam Site, the selected site affords the most favorable rock foundation conditions accompanying minimum length of dam.

2-02. Selection of Type of Dam. (a) Geology of the Site Influencing Selection of Type of Dam. - The Blackwater Dam Site is located at a point where the river has incised a narrow bedrock gorge more than 20 feet deep immediately below its winding course across an old glacial lake flat. Adjoining the narrow gorge on the right is a secondary valley of much broader expanse but shallower depth, the bottom of which has been eroded to bedrock by the occasional overflow of flood discharges into this valley from the main river above the dam site. Bedrock is either exposed or accessible over the entire dam site at depths rarely exceeding 15 feet. Ample borrow materials suitable for impervious and pervious fill in a rolled-earth embankment are available in close proximity to the dam site. The nearest deposits which would provide a suitable and adequate source of materials for the production of concrete aggregates in quantity sufficient for an all-concrete dam are located more than 4 miles from the dam site. Localized deposits of sand and gravel within 2-1/2 miles of the dam site are suitable and adequate for processing for concrete aggregate in quantity sufficient for the concrete structures of an earth and concrete dam. The dikes will involve no foundation or borrow difficulties for either the initial or the ultimate developments.

(b) Types of Dam Studied. - Because of the general availability of bedrock throughout the entire area of the dam site, numerous design possibilities were studied, including three types o' dam: (1) All-concrete gravity; (2) All-concrete hollow; (3) Earth and concrete. Detailed cost estimates were prepared for each type, the results of which indicated the earth and concrete design to be the most economical. Concurrence in the recommendations of the Boston District that the latter type be adopted was expressed by Messrs. McAlpine and Wallace, representing the Office, Chief of Engineers, Messrs. Stuck and Boesch, representing the North Atlantic Division Office, and Mr. Uhl, Consultant, at the Design Conference held at this office November 10, 1939 (see Appendix C).

III FLOOD CONTROL HYDRAULICS

3-01. Hydrology Report. As required by paragraph 19 of Engineer Bulletin, R. & H. No. 9, 1938, a hydrology report was submitted for approval as part of the Definite Project Report for Blackwater Reservoir, dated September 15, 1939 (E.D. File 7402(Merrimack R.-Blackwater Res.) -8). The Definite Project Report was approved by the Chief of Engineers subject to further hydrological studies. The results of these additional studies were submitted in the form of revised sections of the Definite Project Report on December 16, 1939. The following paragraphs on the hydrology and hydraulic factors affecting the design of the Blackwater Project constitute, therefore, a summary of the hydrology studies referred to above.

3-02. General Hydrology. The mean annual precipitation is about 39 inches, fairly evenly distributed throughout the year. Precipitation during the months of December, January, February and March is principally in the form of snow which may accumulate for most of the winter since temperatures generally remain below freezing during this period. Hence, this period has low run-off and is free from flood threats. Because of snow accumulation, however, Blackwater River is subject to heavy run-off during the spring thaw. Records show that the "spring flood" generally occurs during March or April, although one moderate freshet affected by snow run-off is recorded in the middle of January (January 1935). The high temperature causing snow melt is usually accompanied by rain and is the most frequent cause of floods. The maximum flood of record, that of March 1936, was caused primarily by snow run-off. For the remainder of the year the basin is subject to occasional heavy rainfall from general summer and fall storms. The drainage area is small enough to be seriously affected by intense local storms. The flood produced by the summer storm of September 1938 was the second largest of record.

3-03. Precipitation. (a) Records. - The precipitation observation stations in and adjacent to the Blackwater River are shown on Figure III-1. General data on these stations are listed in the following table:

TABLE III-1 - PRECIPITATION RECORDS

Name & Location of Station	Elevation in feet Above MSL	Length of Record (Years)	Annual Precipitation in Inches: Mean	Maximum: Amount	Maximum: Year	Minimum: Amount	Minimum: Year
So. Danbury (in basin)	724	2	49.4	51.5	1937	47.3	1938
Webster (in basin)	550	2	51.6	53.0	1938	50.3	1937
Grafton (10 miles N.)*	863	23	38.6	50.7	1902	30.0	1908
Franklin (10 miles E.)	390	37	40.2	49.7	1920	30.9	1930
Concord (20 miles SE.)**	350	86	36.4	54.3	1863	26.1	1930

* U.S. Weather Bureau Station with fragmentary records from 1879 to 1917.
** Continuous recording gage.

(b) Rainfall Intensity. - The maximum rainfall recorded over the entire Blackwater watershed for a given storm was 7.8 inches for the period September 18 to 21, 1938. The maximum rainfall at a single station was 8.8 inches during the same period with a maximum intensity of 4.4 inches in 24 hours on September 21. Depth-duration curves for rainfall at stations in and near the Blackwater area were constructed in conformity with the continuously recorded data available for the Concord, N. H. Station.

(c) Selection of Maximum Precipitation Values. - The U. S. Weather Bureau, cooperating with the Department, has prepared depth-area curves of maximum rainfall considered applicable to the New England area. Values from these curves have been used for the Blackwater Reservoir except that a higher value has been used for a 6-hour period upon recommendation of the Hydrometeorological Section of the Office, Chief of Engineers. The selected values, which are intended for use only as maximum conditions for spillway design purposes, are as follows:

TABLE III-2 - MAXIMUM PRECIPITATION VALUES

Period in Days	1/4	1/2	3/4	1	1-1/4	1-1/2	1-3/4	2
Period in Hours	6	12	18	24	30	36	42	48
Precipitation, Inches	11.4	13.9	15.9	17.1	18.2	18.8	19.2	19.4

3-04. Run-off. (a) Records. - A record of stream flow is available at the U. S. Geological Survey Station, located about two miles downstream from the damsite. Two major floods have occurred during this period of record: In March 1936, peak flow 11,000 c.f.s.; in September 1938, peak flow 6880 c.f.s. The November 1927 flood peak flow was 2620 c.f.s. General data on this record is shown in the following table.

TABLE III-3 - STREAM FLOW RECORDS, BLACKWATER RIVER

Station	Drainage Area: Sq. Mi.	Period of Record	Discharge in Cu. Ft. per Sec. Mean	Maximum	Minimum
Original location near Contoocook	134	1918-1920 1927-1935	255	11,000	20
Present location near Webster Recording gage	129	1935-1938			

(b) Frequency of Floods. - The stream flow records on Blackwater River are too short for use in computing a satisfactory frequency relation. By comparison with computed discharge frequencies for watersheds of similar size, it appears that the peak flows of 11,000 c.f.s. in March 1936 and 6880 c.f.s. in September 1938 have a frequency of about 106 and 19 years, respectively. Similarly, it appears that flows of 70 and 85 c.f.s. per square mile have a frequency of about once in 50 and 100 years, respectively.

(c) Unit Hydrographs. - The maximum flow of record uninfluenced by snow run-off was 6880 c.f.s. in September 1938. This flood was used for derivation of run-off distribution values. The resulting unit graph, shown as a dotted line on Figure III-2, was used in computations of reservoir design floods. However, for spillway design purposes where peak values are important, it was considered desirable to use a higher peaked unit graph, as shown in solid line on Figure III-2.

3-05. Reservoir Capacity. (a) Flood Control. - Area and capacity curves are shown on Figure III-3. Based on considerations discussed in following paragraphs, a capacity of 46,000 acre-feet, equivalent to 6.8 inches of run-off, was selected for flood control. In the initial development of the reservoir, this capacity is obtained with the spillway lip at Elevation 566.

(b) Conservation. - Based on analysis of stream flow and cost data for development of the reservoir at various elevations, a storage capacity of 69,000 acre-feet (10.1") below Elevation 573 was found to be the optimum capacity for conservation and power purposes at this site. The required storage of 46,000 acre-feet for flood control is then obtainable between Elevation 573 and Elevation 584. This plan was adopted for the ultimate development.

(c) Capacity Conditions Affecting Outlet and Spillway Design. - The principal factors affecting outlet and spillway design for this project were (1) desirability of sufficient outlet capacity to permit rapid emptying of the reservoir after floods, and (2) the necessity for outlet and spillway plans adaptable to a two-stage development.

(d) Computed Spillway Flood. - As defined in Engineer Bulletin, R. & H. No. 9, 1938, the "Computed Spillway Flood" is "the flood computed by means of the forecast worst storm, the highest run-off factors and the worst run-off producing combinations for the drainage area in question". For Blackwater Reservoir, the worst storm conditions were taken from the maximum precipitation values described in paragraph 3-03(c) with a total rainfall of 19.4 inches in 48 hours. Using a minimum average infiltration rate of 0.083 inch per hour, as determined in the hydrology report referred to in paragraph 3-01, the resulting run-off amounts to 15.8 inches. Including a base flow of 2 c.f.s. per square mile, the volume of the computed spillway flood is 110,600 acre-feet and the peak inflow, 49,300 second-feet.

3-06. Outlet Hydraulics. (a) Control Required. - The physical conditions at the Blackwater site are such that storage may be obtained over a wide range at a very low cost. Also, channel capacity immediately downstream is not a limiting factor because of the absence of possible flood damage. It is physically possible, therefore, to obtain a high degree of control economically.

(b) Determination of Reservoir Capacity. - The reservoir design flood was based on rainfall of about 100-year frequency plus a prior ground-saturating storm of moderate frequency. To arrive at such a storm, long-time rainfall records at Concord were studied. It was found from

these records that a 100-year rainfall of 3 days' duration amounted to about 8 inches. With this storm a high run-off factor of 90 percent was applied. In order to justify this high run-off factor, a 3-day storm of 1-1/2 inches of rainfall per day was added prior to the 100-year 3-day storm. This gave a flood volume and peak that, in its effect on reservoir capacities, was about equivalent to the 1936 flood, the maximum flood of record. The flood obtained from these assumptions, however, must be considered as one of less frequent occurrence than once in 100 years because of the assumptions made of the 3-day prior rainfall and the high run-off coefficient. This flood, when routed through the reservoir with an outlet discharge of about 1,800 c.f.s., gave a spillway lip or maximum pool elevation of 566 feet. The storage capacity (46,000 acre-feet) at this elevation was adopted. As a check on the adequacy of this capacity, a design flood approaching the maximum probable flood was derived from the September 1932 storm, the greatest of record in the New England region. This check flood, as shown on Figure III-4, reaches an elevation of 567.1 in the reservoir, requires a maximum outlet discharge of 2000 second-feet and a moderate spillway discharge of 1200 second-feet. This is believed to be a satisfactory balance between storage and outlet capacity and represents a high degree of control of major floods.

(c) Determination of Outlet Capacity. - The dam will be built in two stages, initially for flood control only and ultimately raised to provide for flood control and power storage. Selection of the outlet capacities therefore required adjustment so that without permitting part-gate openings in operation the reservoir design outflow of 2000 c.f.s. during the peak of a flood would not be exceeded in either development. Since it will be practicable to operate the power plant of the final project during floods with a discharge of about 1000 c.f.s., it is necessary to provide outlets of such size that preferably but one outlet may be closed to compensate for this discharge and maintain the total reservoir outflow at approximately 2000 c.f.s. Excess gated outlet capacity was desirable in order to expedite emptying the reservoir after the flood. Also, it is desirable to keep the gate sizes the same. After consideration and correlation of the foregoing, the outlet capacity of 2000 c.f.s. is provided in the initial development with one 3'-6" x 6'-6" ungated outlet and one 3'-6" x 5'-3" gated outlet. Two additional 3'-6" x 5'-3" gated outlets are to be installed initially but not used for flood regulation. These additional outlets are available, however, for use in decreasing the time required to empty the reservoir after floods. In the ultimate development the ungated outlet will be closed off and no longer used. The design discharge of 2000 c.f.s. will then be attained at Elevation 584 by using two of the three gated openings. The third gated outlet is then available for additional capacity for emptying purposes. In the ultimate plan, the outflow can be held at approximately 2000 c.f.s. with the power plant in operation by using only one flood control outlet.

(d) Discharge of Freshets. - The storage capacity of the reservoir is comparatively large and with additional outlet capacity available for emptying after floods, the reservoir is entirely adequate for the discharge of freshets. It will rarely be filled to the spillway lip. The September 1938 flood routed through the reservoir showed a maximum

reservoir stage below Elevation 555 and maximum storage of about 20,000 acre-feet.

(e) Time Required to Empty Reservoir. - With an assumed natural inflow of about 5 c.f.s. per square mile, the time required to empty the reservoir from full pool in the initial plan is about 9 days (see Figure III-4). For the ultimate plan the time required is 10 days. For the initial development the 1936 flood volume requires about 8 days to empty with an average inflow during this period of 1100 c.f.s. (see Figure III-5).

(f) Flood Discharge During Construction. - The contractor will be required to conduct his operations in such manner that the dam embankment and dikes shall be kept sufficiently in advance of the concreting operations in the spillway so that with a discharge of 10,000 c.f.s. over the spillway as constructed, the river will not overtop the dam embankment or the dikes. This discharge of 10,000 c.f.s. in addition to available outlet capacities will permit safe passage of a flood somewhat in excess of the maximum of record.

(g) Operation of Reservoir. - As outlined in paragraph 3-06(c), one ungated and three gated outlets are provided. In the initial plan the desired regulation will be obtained by using the ungated outlet and one gated outlet fully opened at all times. The two additional gated outlets will be kept closed unless it is desired to use them after floods for decreasing the time required to empty the reservoir. In the ultimate development, the ungated outlet will be closed off. The desired regulation is then obtained by operating two of the three gated outlets when required or by using one outlet with the power plant in operation. The third outlet is available as additional capacity for emptying the reservoir. Partial gate openings will not be required. The effect of Blackwater Reservoir on the maximum flood of record (March 1936) is shown on Figure III-5.

3-07. Spillway Hydraulics. (a) Criterion for Capacity. - The spillway is required to have sufficient capacity to pass the spillway design flood with a freeboard allowance of 5 feet for wave action and wind set-up under the following conditions: the reservoir full to the spillway lip at the start of the flood and all outlets blocked or inoperative throughout the flood period.

(b) Spillway Design Flood. - The computed spillway flood (see paragraph 3-05(d)) was routed through the reservoir to obtain the reservoir discharge and the maximum water surface elevation. The inflow ordinates of the computed spillway flood were then increased 50 and 100 percent and similarly routed through the reservoir. The data derived from these three routings are shown graphically on Figure III-6 for the initial plan and Figure III-8 for the ultimate development, where the percent of computed spillway flood is plotted (a) against the pool elevation in feet above mean sea level, and (b) against the peak inflows and outflows. After carefully considering all the factors itemized in paragraph 11 of Engineer Bulletin R. & H. No. 9, 1938, Subject: "Spillway Capacities", it was decided to adopt, as the "Spillway Design Flood" for the Blackwater

Dam, the computed spillway flood plus a factor of safety of 35 percent. Inflow, outflow and stage hydrographs for the spillway design flood under the initial plan are shown on Figure III-7. Corresponding data for the ultimate development are given on Figure III-9. General data concerning the flood are summarized as follows:

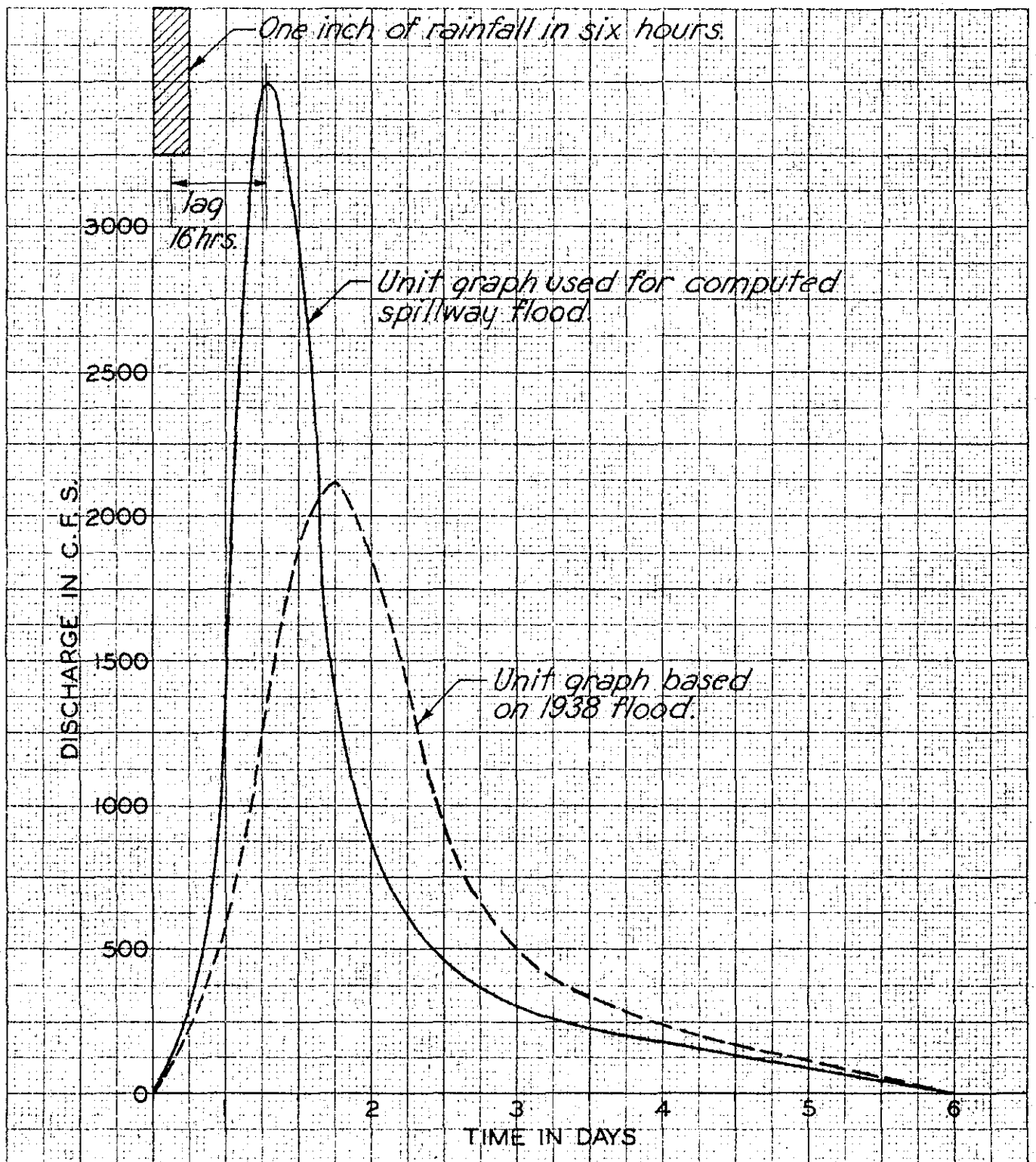
Spillway Design Flood

Percent of increase over Computed Spillway Flood	35%
Volume in acre-feet (including base flow)	148,300
Equivalent volume in inches of run-off	22
Peak inflow (c.f.s.)	66,500
Myer Rating of peak inflow	5880 $\sqrt{DA}$
Peak outflow (c.f.s.) - initial stage	42,800
- ultimate development	40,500
Maximum pool elevation - initial stage	579.0
- ultimate development	595.0
Freeboard provided - initial stage	5.0
- ultimate development	5.0

Meyer

LEGEND
Areas for Thiessens Weighting
Outline of Drainage Areas
Location of Rainfall Stations
Location of Stream Gaging Stations
MERRIMACK VALLEY FLOOD CONTROL
CONTOOCOOK AND
PISCATAQUOG BASINS
RAINFALL STUDIES
U.S. ENGINEER OFFICE BOSTON, MASS.
SEPT. 1939
FILE NO.

FIG. III-1



MERRIMACK VALLEY FLOOD CONTROL

BLACKWATER RESERVOIR

INFLOW UNIT HYDROGRAPH

U.S. ENGINEER OFFICE, BOSTON, MASS.

SEPT, 1939. FILE NO.

FIG. III-2

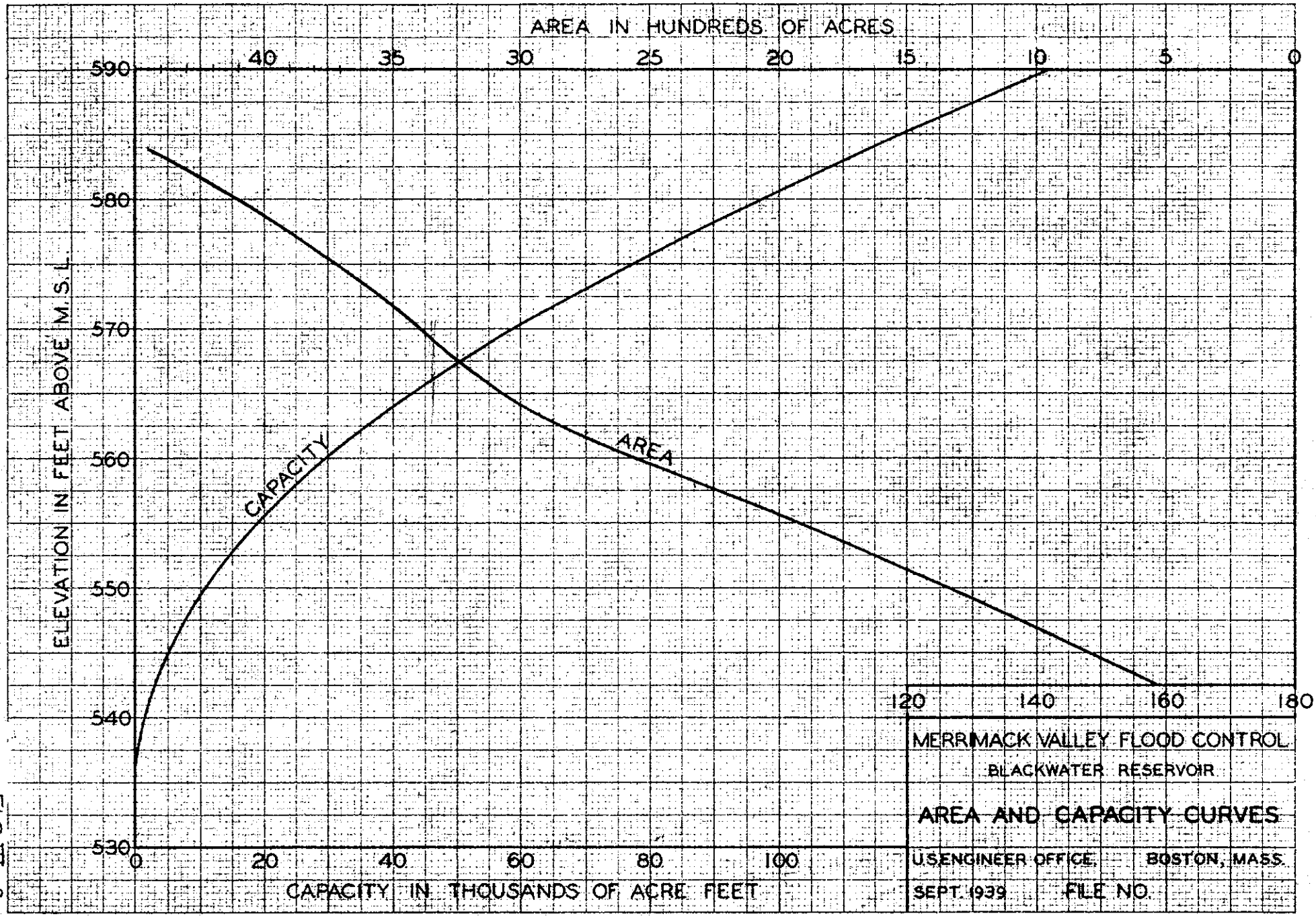
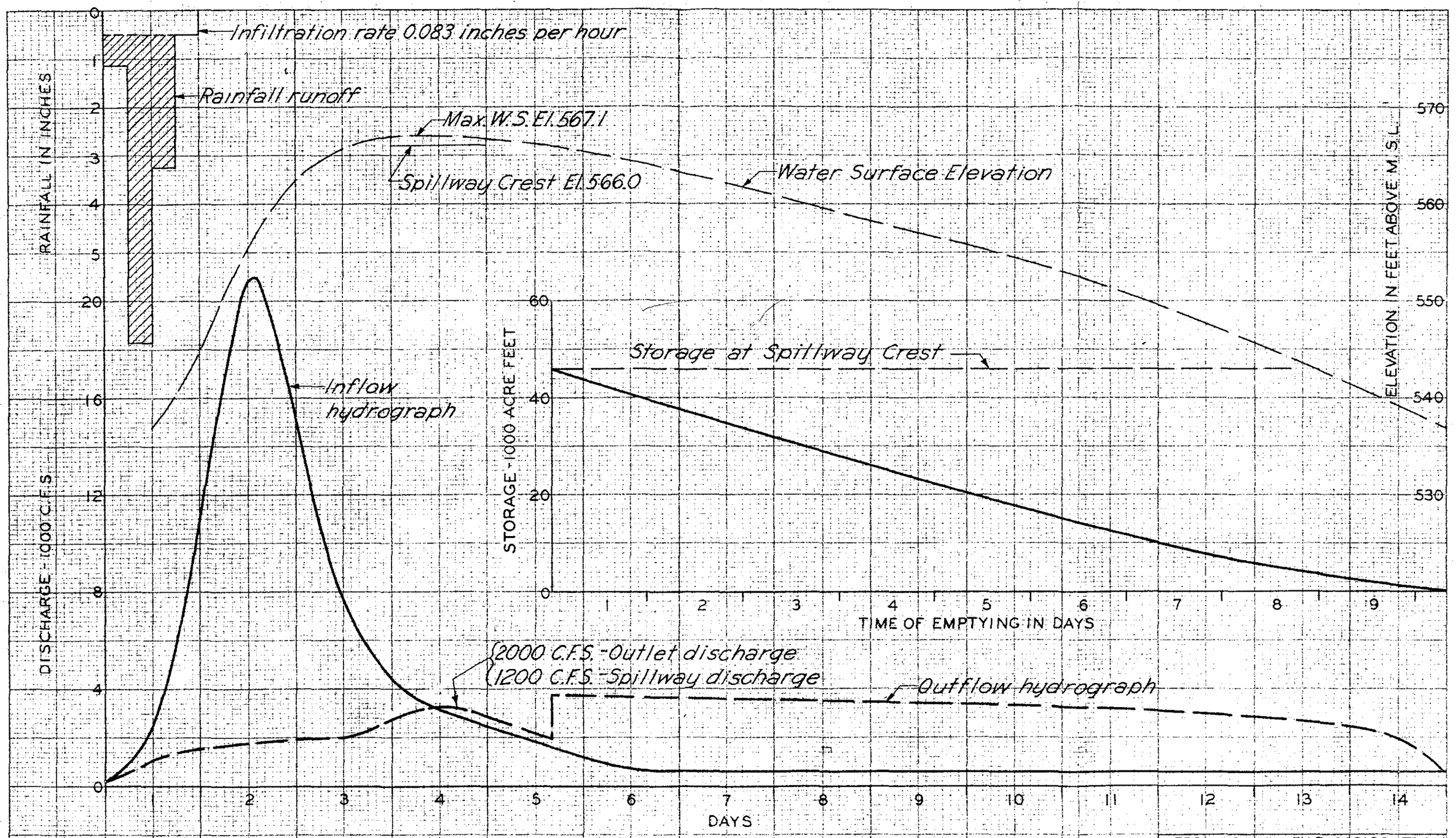

AREA IN HUNDREDS OF ACRES
40
35
30
25
20
15
10
5
0
590
580
570
560
550
540
530
ELEVATION IN FEET ABOVE M.S.L.
CAPACITY
AREA
120
140
160
180
0
20
40
60
80
100
CAPACITY IN THOUSANDS OF ACRE FEET
MERRIMACK VALLEY FLOOD CONTROL
BLACKWATER RESERVOIR
AREA AND CAPACITY CURVES
U.S. ENGINEER OFFICE, BOSTON, MASS.
SEPT. 1939
FILE NO.

FIG. III-3



Note:
Inflow hydrograph based on 1932 rainfall and unit hydrograph derived from analysis of the Sept. 1938 flood

MERRIMACK VALLEY FLOOD CONTROL
BLACKWATER RESERVOIR

RESERVOIR DESIGN FLOOD INFLOW-OUTFLOW-STAGE HYDROGRAPHS

INITIAL DEVELOPMENT
U.S. ENGINEER OFFICE BOSTON MASS.
SEPT. 1939 FILE NO.

FIG. III-4

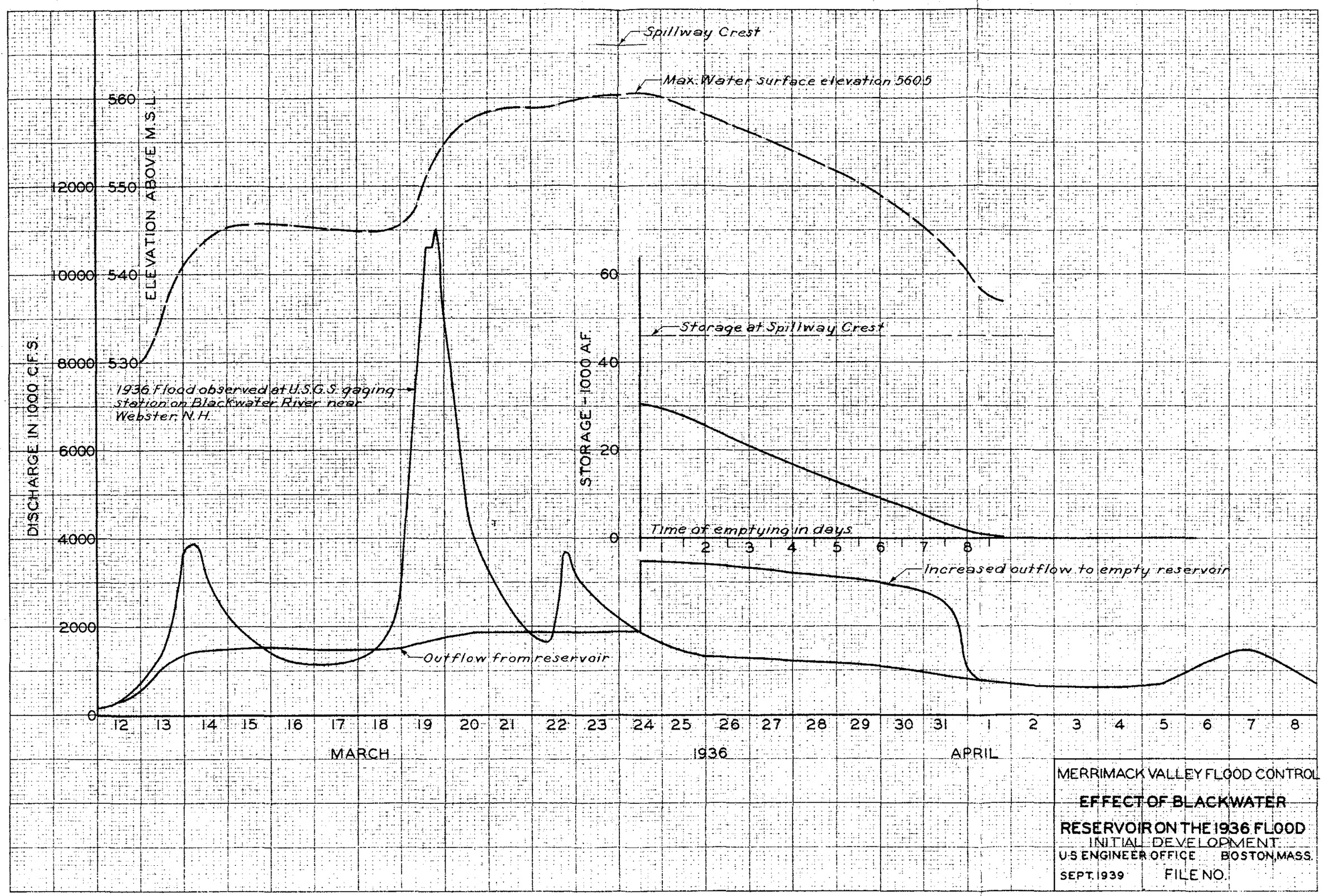

Spillway Crest
Max. Water surface elevation 560.5
ELEVATION ABOVE M.S.L.
5.60
5.50
5.40
5.30
DISCHARGE IN 1000 C.F.S.
12000
10000
8000
6000
4000
2000
0
1936 Flood observed at U.S.G.S. gaging station on Blackwater River near Webster, N.H.
Outflow from reservoir
STORAGE - 1000 A.F.
60
40
20
0
Storage at Spillway Crest
Time of emptying in days
2
3
4
5
6
7
8
Increased outflow to empty reservoir
12
13
14
15
16
17
18
19
20
21
22
23
24
25
26
27
28
29
30
31
1
2
3
4
5
6
7
8
MARCH
1936
APRIL
MERRIMACK VALLEY FLOOD CONTROL
EFFECT OF BLACKWATER
RESERVOIR ON THE 1936 FLOOD
INITIAL DEVELOPMENT
U.S. ENGINEER OFFICE BOSTON, MASS.
SEPT. 1939
FILE NO.

FIG. III-5

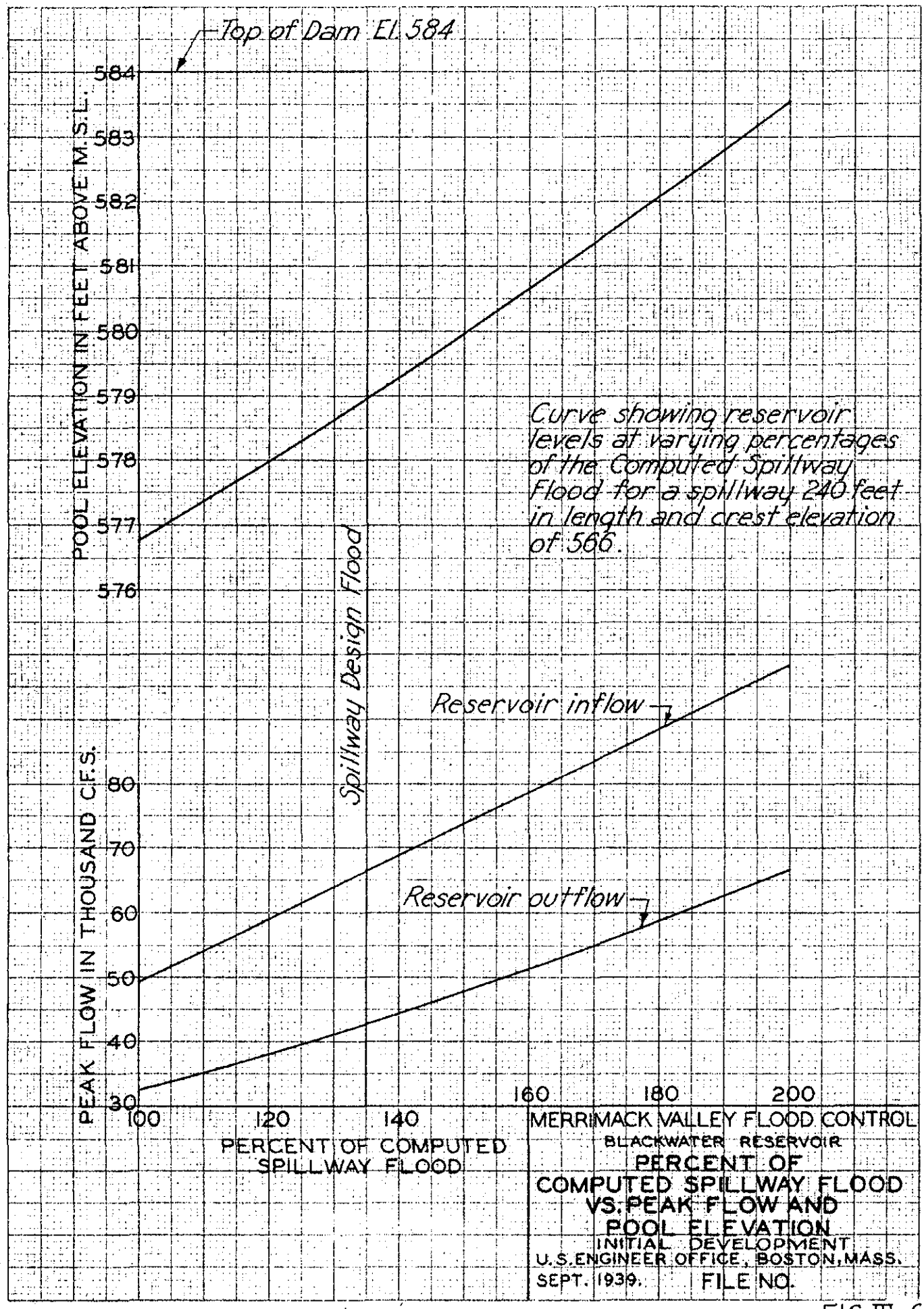
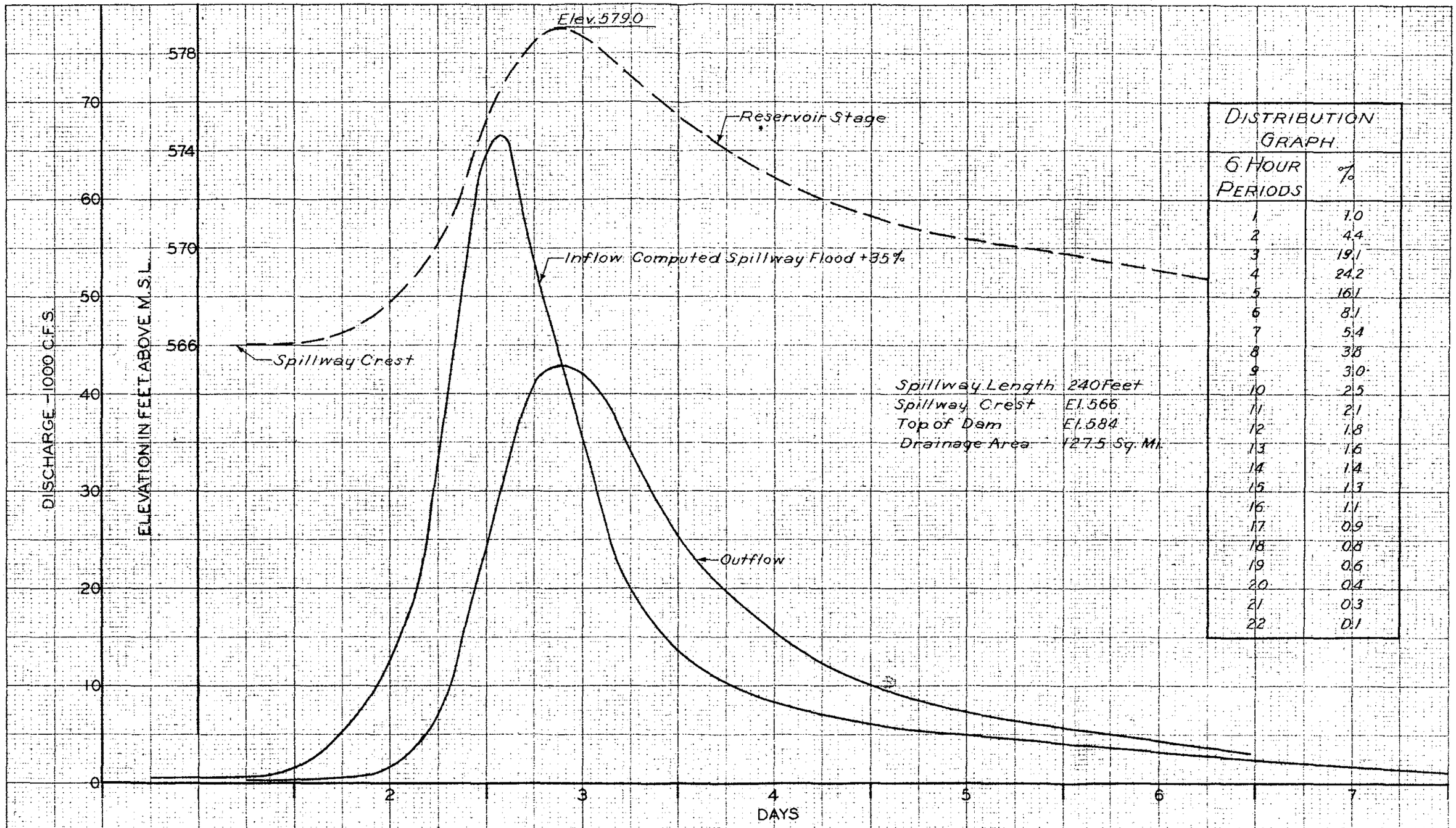

Top of Dam El. 584
584
583
582
581
580
579
578
577
576
POOL ELEVATION IN FEET ABOVE M.S.L.
Curve showing reservoir levels at varying percentages of the Computed Spillway Flood for a spillway 240 feet in length and crest elevation of 566.
Spillway Design Flood
Reservoir inflow
Reservoir outflow
80
70
60
50
40
30
PEAK FLOW IN THOUSAND C.F.S.
100
120
140
160
180
200
PERCENT OF COMPUTED SPILLWAY FLOOD
MERRIMACK VALLEY FLOOD CONTROL
BLACKWATER RESERVOIR
PERCENT OF COMPUTED SPILLWAY FLOOD VS. PEAK FLOW AND POOL ELEVATION
INITIAL DEVELOPMENT
U.S. ENGINEER OFFICE, BOSTON, MASS.
SEPT. 1939.
FILE NO.

FIG. III-6



Spillway Length	240 Feet
Spillway Crest	El. 566
Top of Dam	El. 584
Drainage Area	127.5 Sq. Mi.

DISTRIBUTION GRAPH	
6 HOUR PERIODS	%
1	1.0
2	4.4
3	19.1
4	24.2
5	16.1
6	8.1
7	5.4
8	3.8
9	3.0
10	2.5
11	2.1
12	1.8
13	1.6
14	1.4
15	1.3
16	1.1
17	0.9
18	0.8
19	0.6
20	0.4
21	0.3
22	0.1

MERRIMACK VALLEY FLOOD CONTROL
BLACKWATER RESERVOIR

INFLOW - OUTFLOW STAGE HYDROGRAPH FOR SPILLWAY DESIGN FLOOD

INITIAL DEVELOPMENT

U.S. ENGINEER OFFICE BOSTON, MASS.
SEPT. 1939 FILE NO.

FIG. III - 7

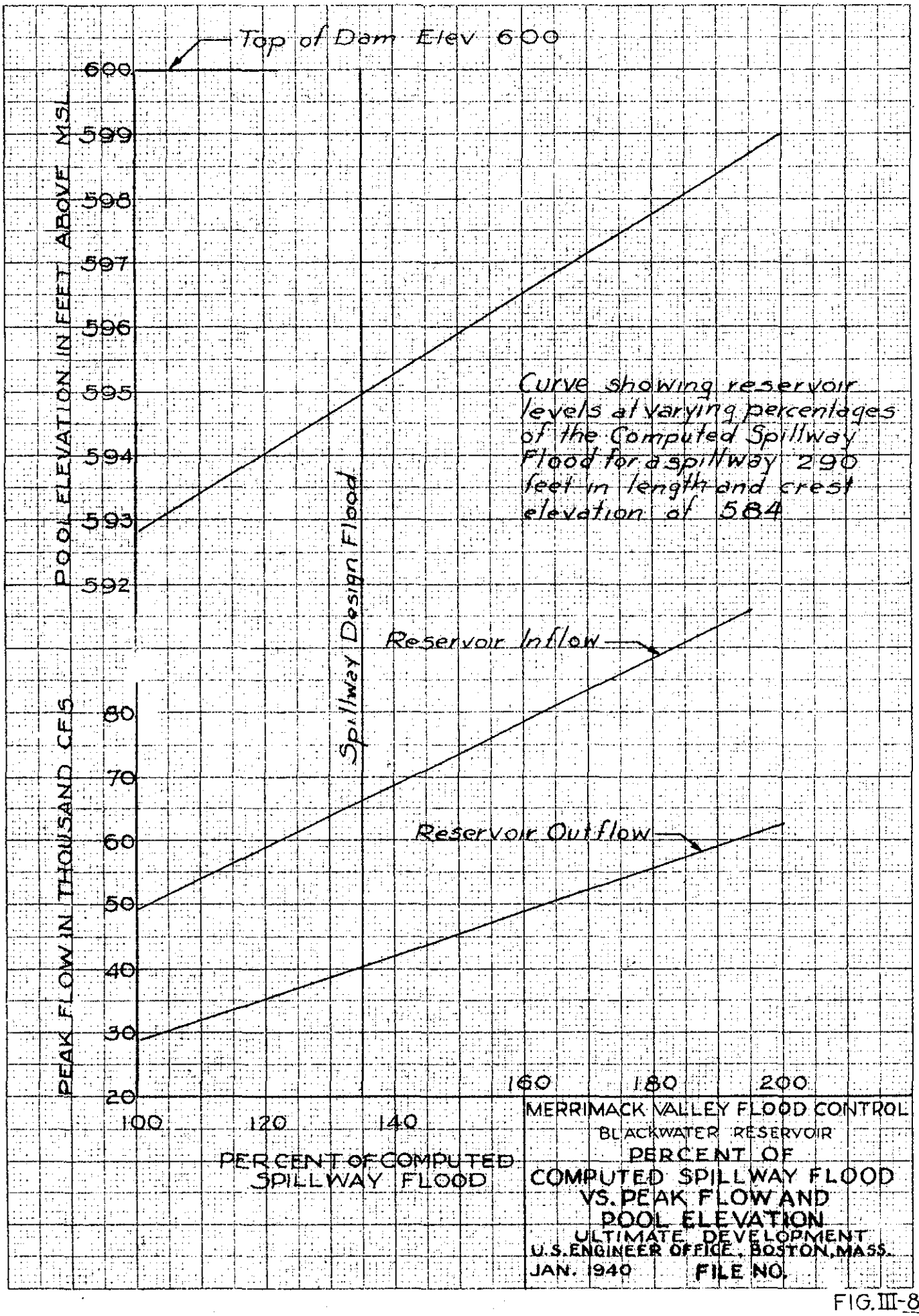

Top of Dam Elev 600
POOL ELEVATION IN FEET ABOVE MSL
600
599
598
597
596
595
594
593
592
Curve showing reservoir levels at varying percentages of the Computed Spillway Flood for a spillway 290 feet in length and crest elevation of 584
Spillway Design Flood
Reservoir Inflow
Reservoir Outflow
PEAK FLOW IN THOUSAND CFS
80
70
60
50
40
30
20
100
120
140
160
180
200
PERCENT OF COMPUTED SPILLWAY FLOOD
MERRIMACK VALLEY FLOOD CONTROL
BLACKWATER RESERVOIR
PERCENT OF COMPUTED SPILLWAY FLOOD VS. PEAK FLOW AND POOL ELEVATION
ULTIMATE DEVELOPMENT
U.S. ENGINEER OFFICE, BOSTON, MASS.
JAN. 1940
FILE NO.

FIG. III-8

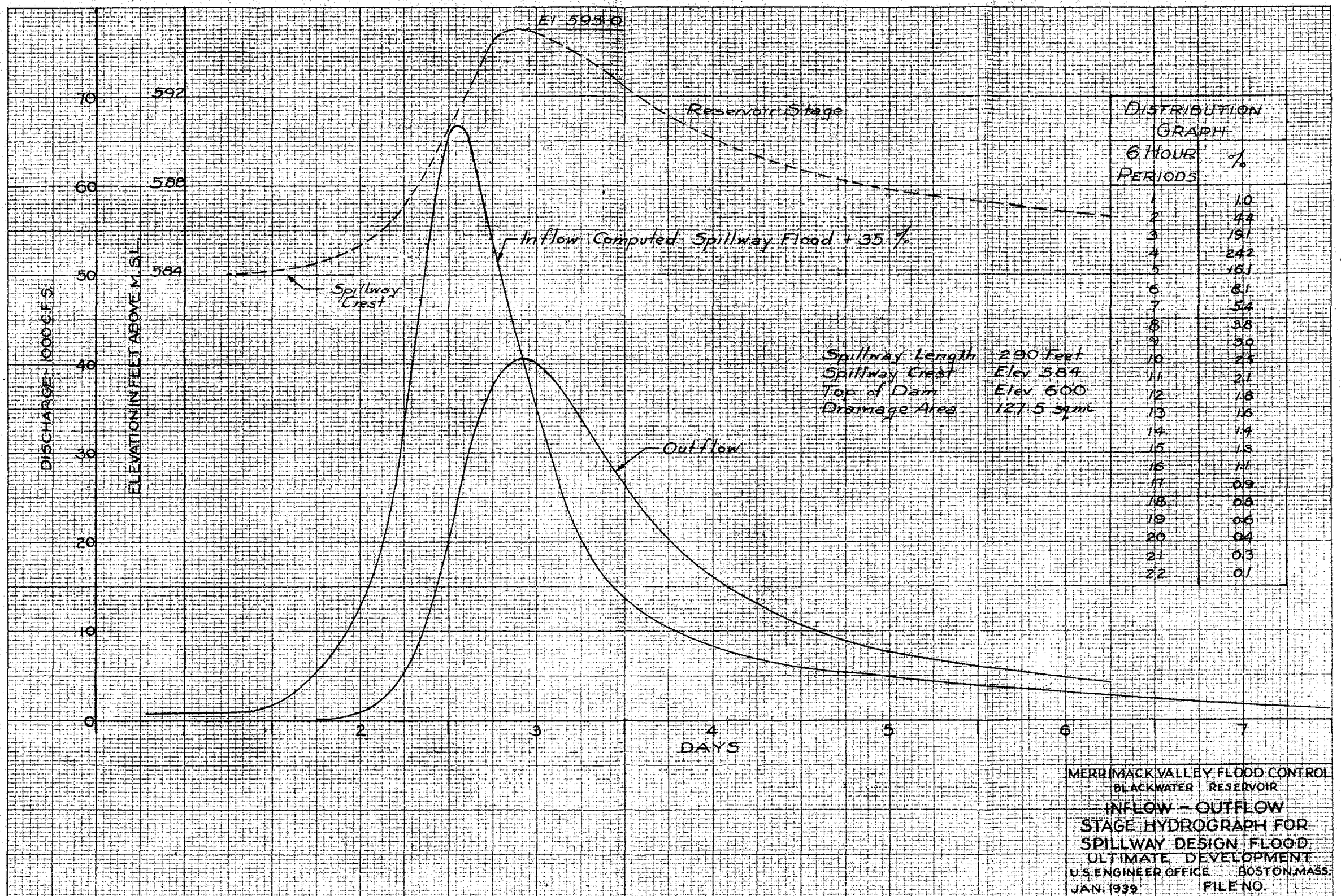

El 595.0
Reservoir Stage
Inflow Computed Spillway Flood + 35 %
Spillway Crest
Outflow
DISCHARGE - 000 C.F.S.
ELEVATION IN FEET ABOVE M.S.L.
70
60
50
40
30
20
10
0
592
588
584
1
2
3
4
5
6
7
DAYS
Spillway Length 290 Feet
Spillway Crest Elev 584
Top of Dam Elev 600
Drainage Area 127.5 sq mi.
DISTRIBUTION GRAPH
6 HOUR PERIODS
%
1 1.0
2 4.4
3 19.1
4 24.2
5 18.1
6 8.1
7 5.4
8 3.8
9 3.0
10 2.3
11 2.1
12 1.8
13 1.6
14 1.4
15 1.3
16 1.1
17 0.9
18 0.8
19 0.6
20 0.4
21 0.3
22 0.1
MERRIMACK VALLEY FLOOD CONTROL
BLACKWATER RESERVOIR
INFLOW - OUTFLOW
STAGE HYDROGRAPH FOR
SPILLWAY DESIGN FLOOD
ULTIMATE DEVELOPMENT
U.S. ENGINEER OFFICE BOSTON, MASS.
JAN. 1939 FILE NO.

FIG. III-9

IV. HYDRAULIC DESIGN

4-01. Location of Discharge Structures. The spillway structure is located at the left end of the dam in the granite rock gorge forming the Blackwater River channel. The outlets are through the concrete spillway section. Discharge from the spillway and outlets is confined to this rock channel, entirely separate from the dam embankment.

4-02. Spillway Design. (a) Requirements.- The spillway is of concrete, with free overfall and is adequate to pass the spillway design flood with a 5-foot freeboard allowance. This requires a capacity of 42,800 c.f.s. in the initial development and 40,500 c.f.s. in the ultimate development. The spillway profile is designed to eliminate negative pressures on the crest and downstream face for a head equal to the surcharge at maximum discharge. The design of the spillway required that the lower portion of the spillway profile for the initial development must also satisfy the discharge conditions for the ultimate development when the spillway is raised to Elevation 584 M.S.L. The masonry line of both initial and ultimate developments was made to conform to that recommended by Creager and Justin (Hydro-Electric Handbook, Table 32), for a head of 13 feet initially and 11 feet ultimately. Adjustment of the required initial and ultimate spillway profiles in relation to each other was made by trial in order that the downstream spillway slope common to both would be satisfactorily tangent to the upper portion of the spillway profiles. The selected masonry line was checked by: (1) the theoretical lower nappe as given by Creager and Justin's Hydro-Electric Handbook, and (2) the theoretical formula $x^2 = 1.8\ hy$ as used in the Technical Reports, Part 7, of the Miami Conservancy District. Both theoretical curves lie within the proposed masonry line for both initial and ultimate developments.

(b) Description of Spillway, and Approach and Discharge Channels.- For the initial development, the spillway has a length of 240 feet, crest at Elevation 566 M.S.L., and was designed for 13-foot maximum surcharge. For the ultimate development, the spillway has a length of 290 feet, crest at Elevation 584 M.S.L., and was designed for 11-foot maximum surcharge. A short bucket is provided at the base of the spillway to prevent direct impact of the falling sheet of water on the rock channel bed, the radius of the bucket being about one-fourth of the distance from Elevation 595 M.S.L. to the channel bed. No stilling basin or protection works was deemed necessary in this channel, as the rock is capable of withstanding high velocities without endangering the structure by erosion. In order to fit the 240-foot spillway of the initial development into the narrow gorge, the base of the spillway was stepped to conform to the channel, the left end of the spillway being provided with a minimum drop of 12 feet. The approach channel is the natural river channel, widened on the left bank with minimum approach depth of 5 feet below the spillway lip. The cross-section of the approach channel, taken at the upstream end of the penstock gatehouse, is about 6,800 square feet at Elevation 566 M.S.L. and 10,100 square feet at Elevation 579 M.S.L. This represents the minimum area of the approach channel. About 100 feet upstream from the dam the area is in excess of 12,000 square

feet, at Elevation 579 M.S.L., giving a mean velocity of approach less than 3.6 feet per second for the spillway design flood. The discharge channel has ample capacity for all anticipated flood discharges. For the spillway design flood, the channel below the dam has ample capacity to prevent submergence. On the left bank, the channel has a minimum depth below the crest of the dam of 12 feet and a bed slope steep enough to carry the flow away at high velocities, thus preventing a hydraulic jump or submergence of this portion of the dam. However, to prevent possible overflow of the right bank with resulting flow near the toe of the dam embankment for a flood equal to the spillway design flood, the narrow rock gorge is to be enlarged to provide sufficient area at the most restricted section to give a mean velocity of about 22 feet per second. The flow conditions below the dam cannot be determined from computations. It appears that a hydraulic jump may form at the base of the highest portion of the spillway. In this event, the water surface would rise to about Elevation 538 M.S.L., with a mean velocity below the jump of but 7 feet per second. The flow over the spillway on either side, which accounts for about 3/4 of the total discharge will not have sufficient tail-water depths to form a hydraulic jump, therefore the major portion of the energy derived from the fall over the dam, with velocities varying from 25 feet per second to about 50 feet per second, will be retained in the flow. The channel area 80 feet below the dam is sufficient to provide for a mean velocity of about 15 feet per second at water-surface Elevation 538 M.S.L. It is desirable not to exceed this elevation because of anticipated wave action resulting from the mixing of flows with varying energy content and flow directions. Undoubtedly the energy content in the water due to velocity head is greater than that due to the average velocity of 15 feet per second. Assuming this energy head equal to about 6.0 feet (corresponding to a mean velocity of 20 feet per second) and deducting friction losses, the velocity at the smallest section (160 feet downstream) will be about 22 feet per second, the corresponding water-surface will be at Elevation 536 M.S.L. Below this control section the channel capacity increases.

(c) Spillway Discharge Capacity.- The spillway discharge capacity and corresponding pool elevation as affecting the dam was computed by means of the weir formula, $Q = CLH^{3/2}$, in which: Q = the total discharge in c.f.s.; L = the effective length of spillway weir; H = total head in feet on the spillway crest, the difference between the elevation of the energy gradient at the weir and the crest of the weir. Because of the deep approach channel, frictional losses can be neglected. The maximum value of C used is 3.8, which is considered conservative for this type of weir. The rating curves for the spillway for both the initial and ultimate developments, and the coefficients used at the several intermediate heads below maximum, are shown in Figure IV-1.

4-03. Outlet Design. (a) The number of outlets was limited to the space available in the river gorge and the size, aside from capacity, to one that would readily pass debris. For these reasons, and the discharge capacities required, 4 outlets were found to be the optimum number.

(b) The size of outlets was determined by the gated outlet size required in the ultimate development to pass approximately 1000 second-feet each with reservoir at spillway crest. The cross-sectional area of such an outlet was computed to be approximately 18-1/2 square feet. It was considered desirable to have all the gates the same size and the height of the gate about twice its width, with no transitions or changes in area in the outlets other than the bellmouth entrance. The selected dimensions of the outlet and gate passage are 5 ft. 3 in. high and 3 ft. 6 in. wide. Three such gated outlets are provided.

(c) The capacity of this size of outlet was then determined with reservoir Elevation 566 M.S.L. the spillway crest elevation of the initial development, and found to be 870 c.f.s. Since it is proposed to limit the outlet design discharge in the initial development to 2000 second-feet, without regulation by partial gate openings, it was found advantageous to provide an ungated outlet to discharge approximately 1130 second-feet. This, with one gated outlet left open at all times, gives the required flood control outlet capacity, no gate operation being necessary except after a major flood when the remaining two gated outlets will be opened to permit rapid emptying of the reservoir. Similarly, in the ultimate development, during a flood exceeding the power storage at Elevation 573 M.S.L., two of the gated outlets will be opened as required.

(d) Description of Outlets.- (1) The approach channel for the outlets is located in the deepest part of the rock gorge forming the river channel. The channel has ample capacity for all flows permitting low velocities of approach. The invert of the 3 gated outlets is about 5 feet above the natural rock bed elevation; the transition from the natural channel to the intake elevation is a flat curve of about 50-foot radius. The single ungated outlet is located at the left side of the channel and requires excavating the approach channel for about 75 feet upstream. The channel bed is to be excavated to Elevation 513 M.S.L.. 2 feet below the outlet invert.

(2) Trash bars are provided for the 4 outlets. They are semi-elliptical in section, their long axis set parallel to the flow to minimize flow disturbance at the entrance. Mean velocities through the trash bars will not exceed about 12 feet per second.

(3) The transition of the gated outlets, from entrance section at the upstream end of the trash bars to the gate passage, is a bellmouth, about 17 feet long, laid out on easy curves extending to within 5 feet of the emergency gates. The top of the transition is shaped to a quarter ellipse, with the horizontal axis about twice the vertical axis. Mean velocities for this intake are given in Figure IV-2 for reservoir level at Elevation 584 and 566, M.S.L. The bellmouth for the ungated outlets is similar to that for the gated outlets, the transition from the entrance area of 145

square feet (10-3/4 ft. x 13-1/2 ft.) to the 3-1/2 ft. x 6-1/2 ft. passageway being made in a distance of 15 feet.

(4) The gated outlets are provided with air vents in the roof immediately downstream from both the emergency and service gates. It was considered advisable to vent the emergency gate as well as the service gate to permit its use as a spare gate, although its vent was not carried above maximum tailwater but well above that anticipated with all 4 outlets discharging. Vent capacity provided was made to conform to the data in Figure 2 given on page 180 of "Dams and Control Works", U. S. Bureau of Reclamation (second edition).

(5) The gate guide slots produce considerable disturbance to flow, necessitating lining the gate passageways in the vicinity of these slots.

(6) The downstream end of all outlets was curved downward, the inverts meeting the bucket of the spillway on a parabolic curve that is flatter than the outflow trajectory with maximum pool elevation. The outlets downstream from the bellmouth intake are uniform in cross-section and area throughout their length.

(7) No stilling basin is provided as the outflow is into a granite rock channel. Consideration was given to the use of a splitter at the outlet portal similar to that devised for the Pittsburgh District, but these were not considered necessary from the standpoint of channel erosion downstream. Their use could be justified to prevent ride-up along the right bank of the high velocity jets from the outlets. To prevent this, it is proposed to carry the channel at Elevation 512 M.S.L. for a distance of about 100 feet downstream, in line with the outlet nearest the right bank. At this point the channel bank makes an angle of about 20 degrees with the line of flow and would not cause excessive ride-up; also, since there will not be sufficient tailwater depth to drown out the outlet discharge, there will be considerable spreading of the flow along the channel floor, with resulting dissipation of energy and velocity.

(e) Computation of Discharge Capacity.- For outlets flowing full, the discharge capacity was determined on the basis of the outlets discharging as an orifice, i.e., $Q = A\sqrt{2gh}$, in which Q = the total discharge in c.f.s., A = the cross-sectional area of the outlet in square feet, and h = the net head in feet. The total head was taken as the head between pond elevation and a point just below the crown of the outlet portal. The net head, h, was determined by deducting from the gross head all head losses in the outlet, including velocity head at the portal. Computations for head losses are given in Figures IV-3 to IV-6. Hydraulic characteristics of the outlets are shown in Figure IV-7. For outlets flowing part full, pond elevations were computed for several discharges, considering the control near the downstream end at the point where the outlet invert is at critical slope for the given discharge. The rating curves for the ungated and gated outlets are shown in Figure IV-8. Rating curves shown in Figure IV-9 are for the outlets operated in the initial development as outlined in paragraph 3-06(g); one curve for the

ungated and one gated outlet remaining open at all times, the other curve for all outlets discharging after the flood, if required to empty the reservoir rapidly. Figure IV-10 shows the rating curves for one, two, and three gated outlets of the ultimate development, wherein one outlet is used for flood control as required in combination with the power plant discharge, two outlets are used (with no power plant discharge) as required for discharge during floods causing the power pool elevation to exceed 573 M.S.L., and all three outlets if required to empty the reservoir after the flood. Figure IV-11 shows the computations for determining the hydraulic and energy gradients in the gated outlets for pool Elevation 584 M.S.L., and Figure IV-12 shows the hydraulic and energy gradients in the gated outlets with reservoir at spillway crest for both the initial and final developments. There is no tailwater rating curve available for the outlets. However, studies made on the discharge channel capacity indicated that for maximum outlet discharge the tailwater could not rise high enough to affect the discharge capacities of the outlets.

4-04. Freeboard. (a) General. - Sufficient freeboard is provided against overtopping the dam by wave action at the maximum water surface. Because of the unusually sheltered location of the Blackwater Dam with respect to the maximum possible conditions producing wave action and wind set up, the minimum freeboard of 5 feet was provided. This value of freeboard has been checked in accordance with the general rules set forth in Engineer Bulletin R. & H. No. 9, 1938, and found adequate.

(b) Allowance for Wind Set-up. - Using the formula of the Lorentz Zuiderzee Commission of Holland,

$$S = .00125 \frac{V^2 F}{D} \cos A$$

in which S = wind set-up in feet, V = velocity of wind in miles per hour (taken as 80 miles per hour for this vicinity), F = fetch in miles (about 5-1/2 miles for a maximum value), D = depth of reservoir in feet (about 40 feet with reservoir Elevation 579) and A = angle of wind and fetch (taken as zero for most extreme conditions), the wind set-up is computed as 0.2 foot per mile of fetch or 1.1 feet for a fetch of 5-1/2 miles.

(c) Allowance for wave action was computed by the Stevenson formula as modified by Molitor (see page 984, Trans. Am. Soc. C.E., Vol. 100, 1935)

$$h = 0.17 \sqrt{VF} + 2.5 - \sqrt[4]{F}$$

in which h = wave height in feet and V and F are defined and evaluated as in the preceding paragraph.

(d) Allowance for ride-up of waves was taken as 1.4 times the total height of wave as computed in the foregoing paragraph.

(e) Total Freeboard Required. - Two possibilities were tried for the dam. That producing the maximum condition was for the maximum reservoir fetch of 5.5 miles for wind set-up, assuming that the maximum water surface produced by this set up is transmitted to the dam. Since it appears that the wind waves will be dissipated in passing perpendicular to their path of travel to the pool above the dam, the maximum fetch applicable above the dam is about 1/2 mile. The computed value of wind set-up and wave height produced under these conditions is less than 5 feet. For the dikes a fetch of 1-1/2 miles was used and the total computed value of wind set-up and wave height found to be less than 5 feet.

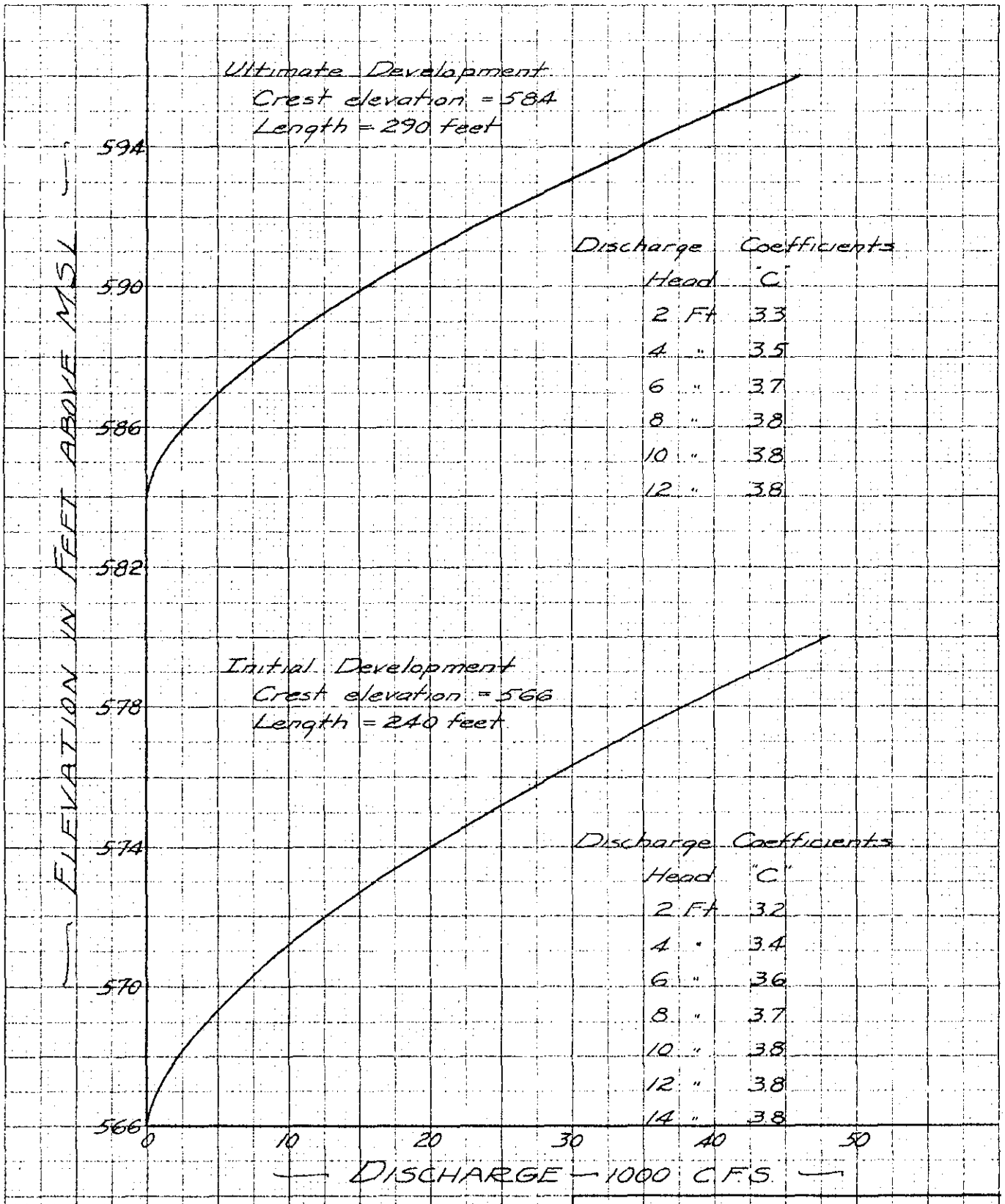
4-05. Model Studies. No hydraulic model tests have been made. Such tests would be useful only for checking the adequacy of the channel below the spillway to prevent overtopping the right bank (see paragraph 4-02(b)).

4-06. Conclusions. The foregoing analysis of the hydraulic design shows that the structures provided satisfy the following criteria:

(1) The spillway and outlet capacity is so great that there is no danger of overtopping.

(2) The freeboard is so great that there is no danger of overtopping by waves.

(3) There is no opportunity for the free passage of water from the upstream to the downstream face.

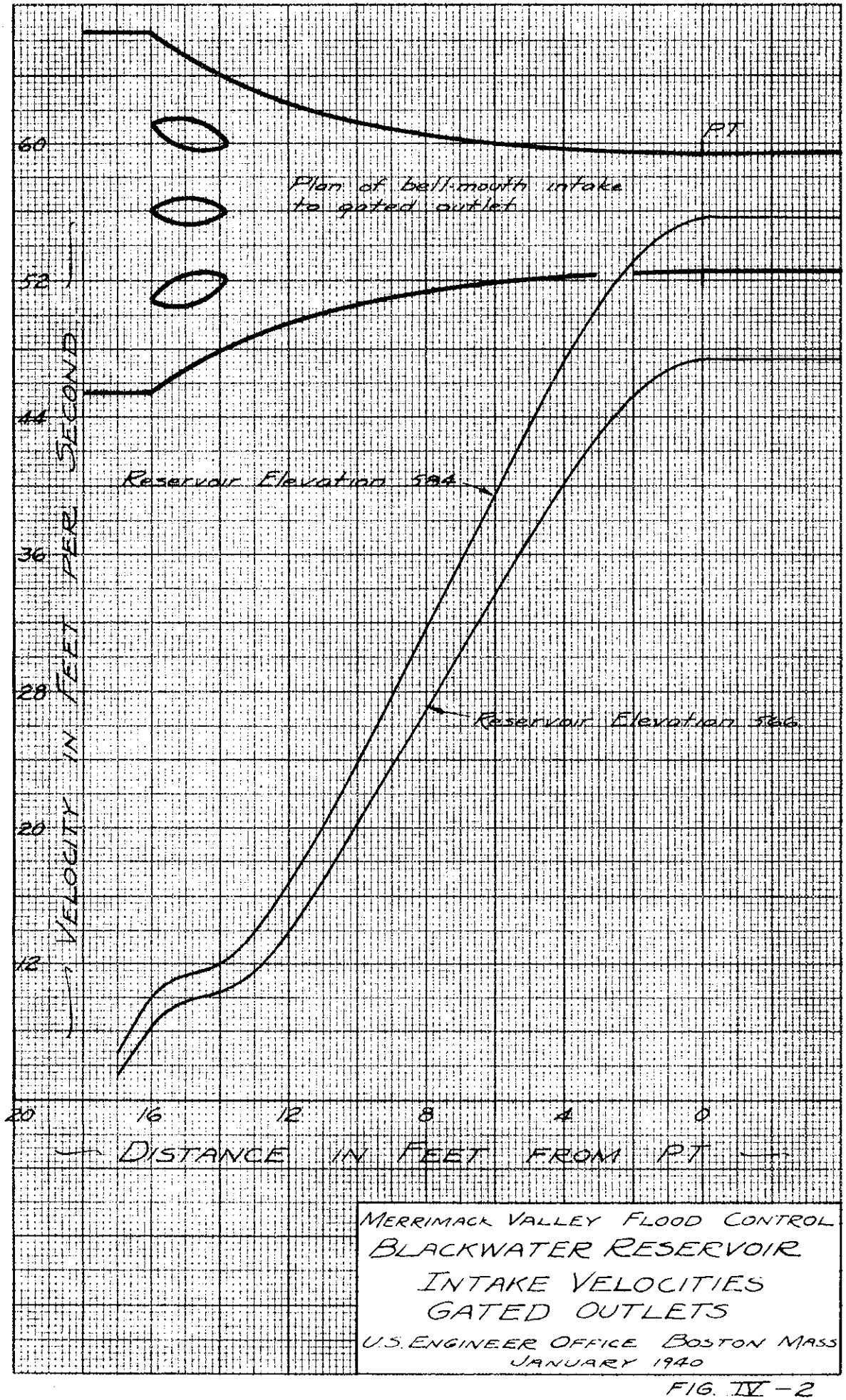


MERRIMACK VALLEY FLOOD CONTROL

BLACKWATER RESERVOIR
SPILLWAY DISCHARGE
RATING CURVES

U.S. ENGINEER OFFICE BOSTON MASS.
JANUARY 1940

FIG. IV-1



MERRIMACK VALLEY FLOOD CONTROL
BLACKWATER RESERVOIR
INTAKE VELOCITIES
GATED OUTLETS
U.S. ENGINEER OFFICE BOSTON MASS
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FIG. IV-2

WAR DEPARTMENT

U. S. ENGINEER OFFICE, BOSTON, MASS.

Page

Subject: Blackwater Dam

Computation: Discharge Capacity of Gated Outlets

Computed by: K. J. Checked by: EC Date: 12/15/39

Outlet Dimensions

Bell-mouth intake — length 17 feet
— area at entrance 158 sq. ft.

Gate passage — length 47 ft.
— height 5.25 ft.
— width 3.50 "
— invert elev. 515 "

Discharge required per outlet for ultimate development (pool elev. 584 ft.) = 1000 cfs

Hydraulic Characteristics

Area — $A = 5.25 \times 3.50$ = 18.4 sq. ft.
Wetted perimeter = $2(5.25 + 3.50)$ = 17.5 ft.
Hydraulic radius — $R = 18.4 \div 17.5$ = 1.05 "
Assumed value of "n" (Kutters) = 0.013
Chezy coefficient "C" = 118

Head Losses

Let v = velocity in gate section

1. h_1 = Entrance loss in bell-mouth (including trash bars) $= 0.05 \frac{v^2}{2g}$

2. h_2 = Friction losses $= \frac{64.4\,L}{C^2 R} \cdot \frac{v^2}{2g}$

where $L = 54$ (47 ft. of section 5.25 x 3.5' and 7 ft. of bell-mouth section upstream from point of tangency with gate passage.)

$C = 118$
$R = 1.05$

$$h_2 = \frac{64.4 \times 54}{118^2 \times 1.05} \cdot \frac{v^2}{2g} = 0.24 \frac{v^2}{2g}$$

3. h_3 = Eddy losses from gate slots = 0.06 "

4. h_4 = Velocity head at outlet portal = 1.00 "
(Assuming no recovery of head)

FIG. IV - 3

Subject: Blackwater Dam
Computation: Discharge Capacity of Gated Outlets (con.)
Computed by: K. J. Checked by: EC Date: 12/15/39

Total head, $H = \Sigma(h_1, h_2, h_3, h_4) = 1.35 \frac{V^2}{2g}$

Discharge per outlet –

$$V = \sqrt{\frac{2gh}{1.35}} = 6.91\sqrt{H}$$

$$Q = AV = 18.4 \times 6.91\sqrt{H} = 127\sqrt{H}$$

Datum for head for outlets flowing full taken as elevation 519, slightly below top of outlet portal.

Total head to spillway crest –
Initial development = 566 − 519 = 47 ft.
Ultimate " = 584 − 519 = 65 "

Outlet discharge with water surface at spillway crest.

Initial development

$$Q = 127\sqrt{47} = 870 \text{ cfs}$$

$$V = 6.9\sqrt{47} = 47 \text{ ft./sec.}$$

Ultimate development

$$Q = 127\sqrt{65} = 1020 \text{ cfs}$$

$$V = 6.9\sqrt{65} = 56 \text{ ft./sec.}$$

WAR DEPARTMENT

U. S. ENGINEER OFFICE, BOSTON, MASS.

Page

Subject: Blackwater Dam

Computation: Discharge Capacity of Ungated Outlet

Computed by: K. J. Checked by: EL Date: 12/15/39

Outlet dimensions

Bell-mouth intake – length 15 ft.
area at entrance 145 sq. ft.

Conduit – length 41 ft.
height 6.5 "
width 3.5 "

Discharge capacity required about 1130 cfs with reservoir at elevation 566. This discharge combined with discharge of 870 cfs from one gated outlet equals outlet design discharge of 2000 c.f.s. The ungated outlet will be used only for the initial development, and permanently closed in the ultimate development.

Hydraulic Characteristics

Area – $A = 6.50 \times 3.50 =$ 22.8 sq. ft.
Wetted perimeter $= 2(6.5+3.5) =$ 20.0 ft.
Hydraulic radius – $R = 22.8 \div 20.0 =$ 1.14 "
Assumed value of "n" (Kutters) = 0.013
Chezy coefficient "C" = 119

Head Losses

Let v = velocity in conduit section.

1. h_1 = Entrance loss in bell-mouth (Including trash bars) $= 0.05 \frac{v^2}{2g}$

2. h_2 = Friction losses $= \frac{64.4\,L}{C^2 R} \cdot \frac{v^2}{2g}$

$L = 46$ (41 ft. of section 6.5' × 3.5' and 5 ft. of bell-mouth section upstream from point of tangency)

$C = 119$

$R = 1.14$

$$h_2 = \frac{64.4 \times 46}{119^2 \times 1.14} \times \frac{v^2}{2g} = 0.18 \frac{v^2}{2g}$$

FIG. IV-5

Subject Blackwater Dam
Computation Discharge Capacity of Ungated Outlet (con)
Computed by K. J. Checked by EC Date 12/15/39

3. h_3 = Velocity head at outlet portal $= 1.00 \frac{v^2}{2g}$
(Assuming no recovery of head)

Total Head, $H = \Sigma(h_1 h_2 h_3) = 1.23 \frac{v^2}{2g}$

Discharge —

$$v = \sqrt{\frac{2gh}{1.23}} = 7.24\sqrt{H}$$

$$Q = vA = 22.8 \times 7.24\sqrt{H} = 165\sqrt{H}$$

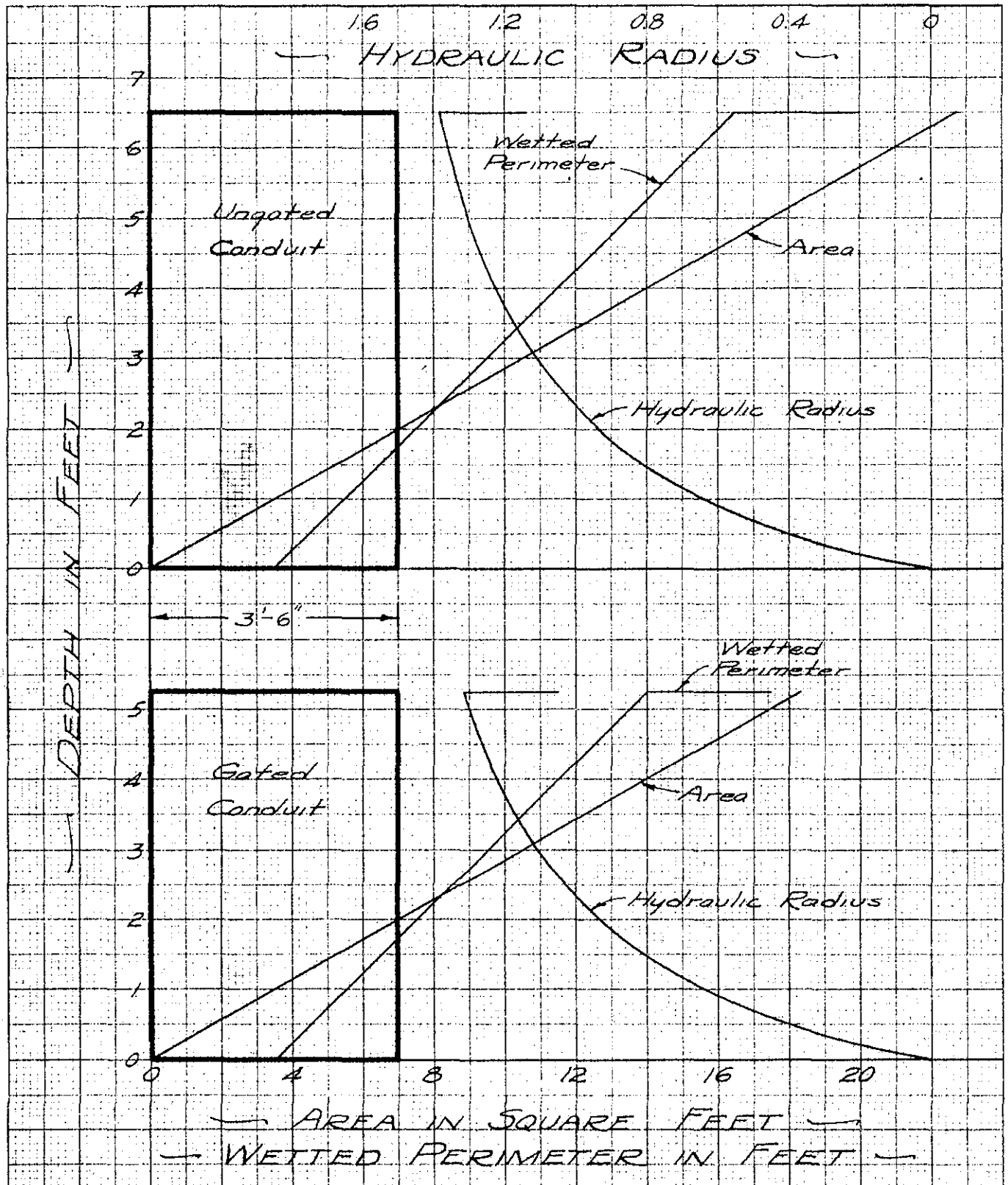
Datum for head for outlets flowing full) taken as elevation 520, slightly below top of outlet portal

Total head to spillway crest
$= 566 - 520 = 46$ ft.

Outlet discharge with water surface at spillway crest

$$V = 7.25\sqrt{46} = 49 \text{ ft/sec.}$$

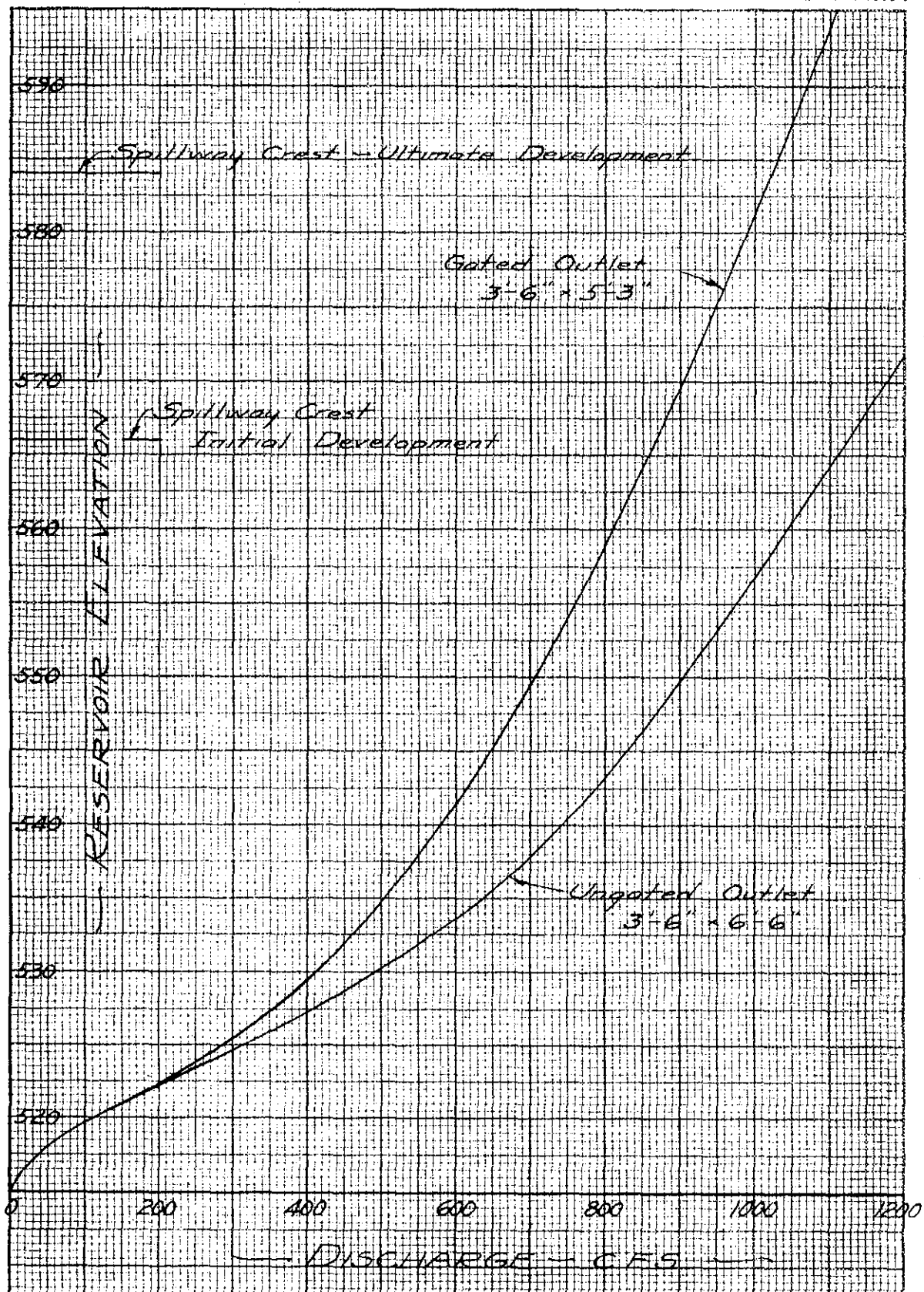
$$Q = 165\sqrt{46} = 1120 \text{ cfs}$$



MERRIMACK VALLEY FLOOD CONTROL
BLACKWATER RESERVOIR
HYDRAULIC CHARACTERISTICS OF OUTLETS

U.S. ENGINEER OFFICE BOSTON MASS.
JANUARY 1940

FIG. IV - 7



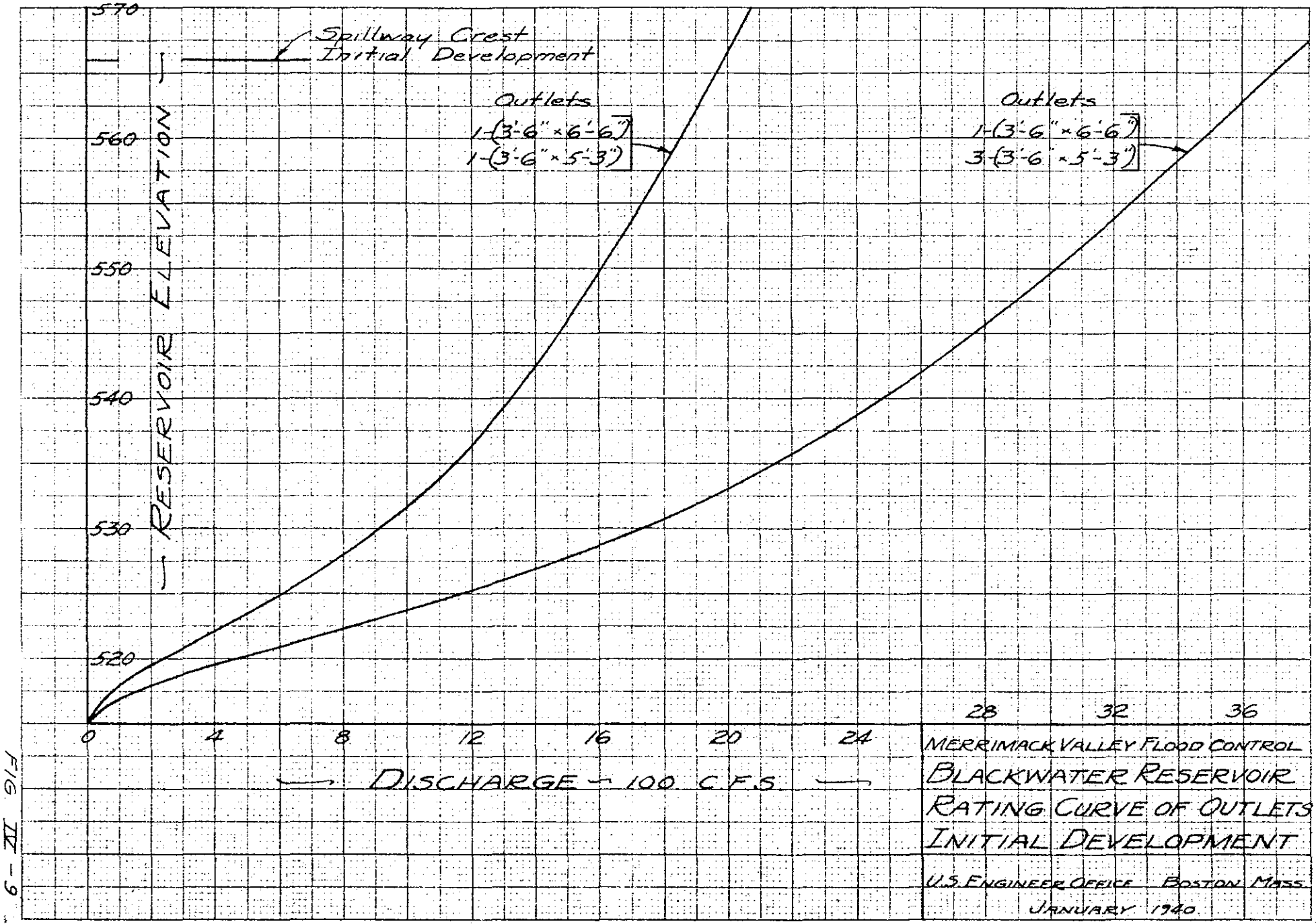
MERRIMACK VALLEY FLOOD CONTROL

BLACKWATER RESERVOIR

RATING CURVES OF OUTLETS

U.S. ENGINEER OFFICE BOSTON, MASS.
JANUARY 1940

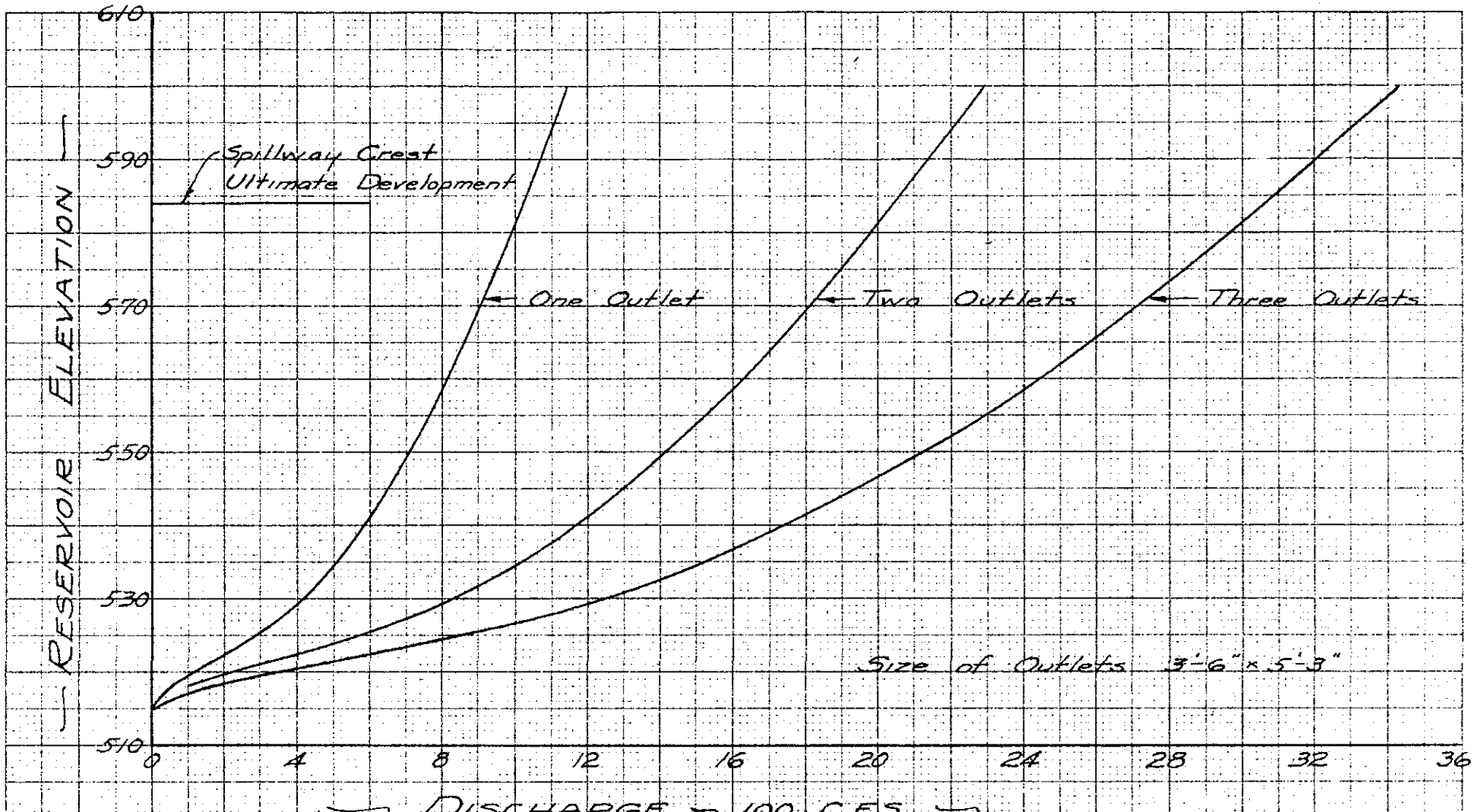
FIG. IV-8



MERRIMACK VALLEY FLOOD CONTROL
BLACKWATER RESERVOIR
RATING CURVE OF OUTLETS
INITIAL DEVELOPMENT

U.S. ENGINEER OFFICE BOSTON MASS.
JANUARY 1940

FIG. XI - 9



MERRIMACK VALLEY FLOOD CONTROL
BLACKWATER RESERVOIR
RATING CURVE OF OUTLETS
ULTIMATE DEVELOPMENT

U.S. ENGINEER OFFICE BOSTON, MASS
JANUARY 1940

FIG. IX-10

WAR DEPARTMENT

U. S. ENGINEER OFFICE, BOSTON, MASS.

Page

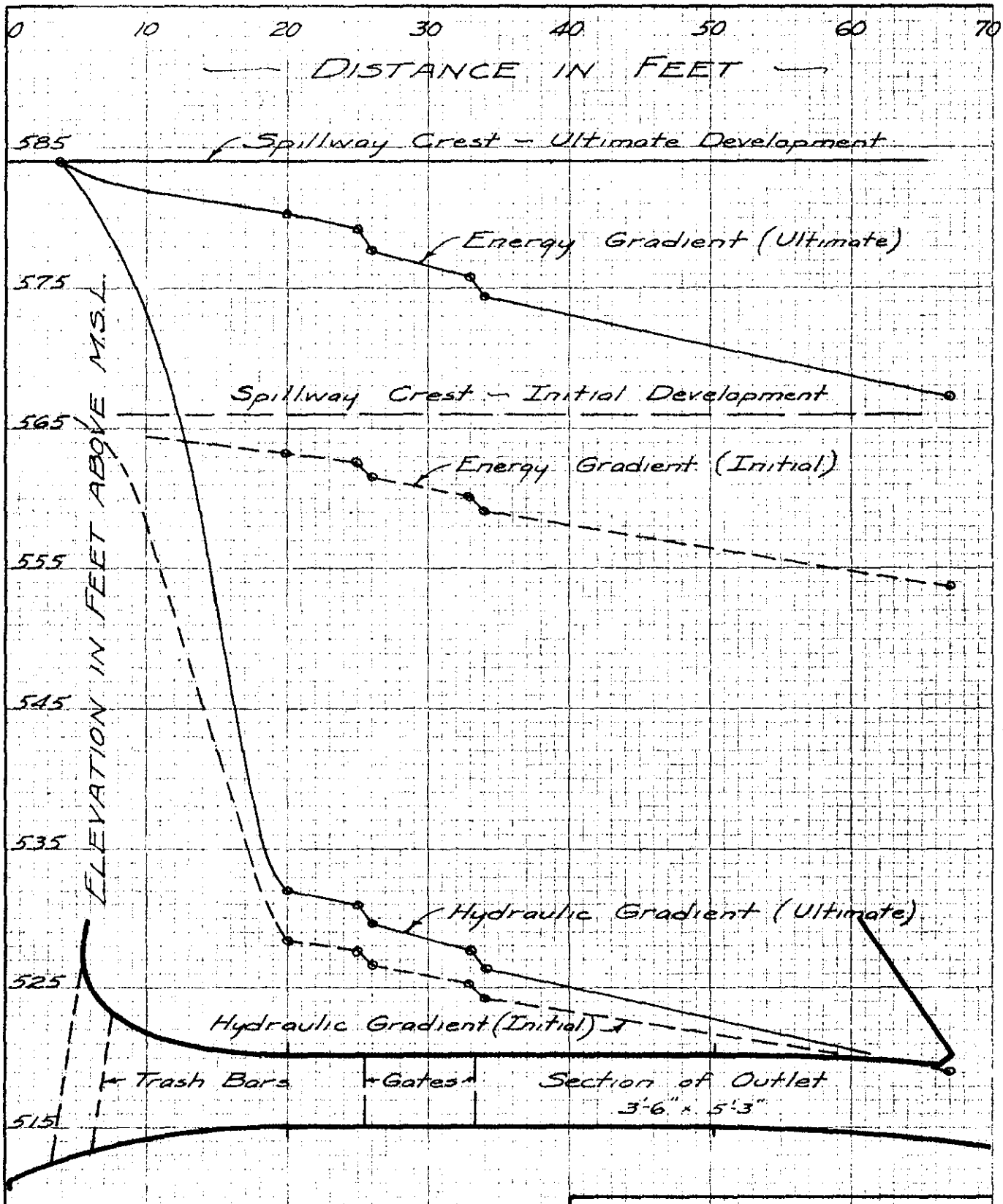
Subject: Blackwater Reservoir – Gated Outlets

Computation: Outlets – Energy & Hydraulic Gradient – Ultimate

Computed by: Ec Checked by: K. J. Date: 1/9/40

Station	Description of Head Loss	Head Loss in terms of velocity head	Head Loss in feet	Elev. of Energy Gradient	Elev. of Hydraulic Gradient	Velocity Head
0+04				584.00	584.0	$\frac{v^2}{2g} = 0$
	Entrance loss	$0.05 \frac{v^2}{2g}$	2.41			$(v = 0)$
	Friction loss $\frac{7}{54} \times 0.24 =$	0.03 "	1.44			
0+20				580.15	532.00	$\frac{v^2}{2g} = 48.15$
	Friction loss $\frac{5}{54} \times 0.24 =$	0.02 "	0.96			$(v = 55.65)$
0+25				579.19	531.04	
	Gate slot	0.03 "	1.44			
0+26				577.75	529.60	
	Friction loss (8 ft.) $\frac{8}{54} \times 0.24 =$	0.04 "	1.93			
0+33				575.82	527.67	
	Gate slot	0.03 "	1.44			
0+34				574.38	526.23	
	Friction loss (34 ft.) $\frac{34}{54} \times 0.24 =$	0.15 "	7.22			
0+67				567.16	519.01	
	Velocity head (Assuming no recovery)	1.00 "	48.15			
				519.01		

FIG. IV-11



MERRIMACK VALLEY FLOOD CONTROL
BLACKWATER RESERVOIR
ENERGY & HYDRAULIC GRADIENT
IN GATED OUTLET
U.S. ENGINEER OFFICE BOSTON, MASS.
JANUARY 1940.

FIG. IV - 12

V INVESTIGATIONS OF FOUNDATIONS AND MATERIALS FOR DAM EMBANKMENT, DIKES, AND CONCRETE AGGREGATE

5-01. Field Exploration of Foundations. (a) General. - Foundation exploration at the dam, Dodge Estate Dike and Little Hill Dike sites included drilling in overburden and rock, excavation and sampling of test pits and auger holes, and the determination of bedrock elevations by the seismic method. The location and extent of the explorations at the three sites are shown on Sheets 3, 11 and 12 of the contract drawings. Complete field logs of the drill holes and test pits and a report of the seismic investigations are on file in the District Office. All soil samples and rock cores are stored in the District Soils Laboratory.

(b) Drill Holes. - Holes were drilled in the overburden and rock with diamond core drilling equipment using 3-inch casing in the overburden and a 1-11/16 inch bit to obtain bedrock cores. The overburden was dry sampled using a 2-inch sampling tube. Soil samples were usually taken at 5-foot intervals or at changes in material. Wash boring between samples was permitted. A total of 20 holes were drilled at the dam site, 1 at the Dodge Estate Dike site and 6 at the Little Hill Dike site with a total footage of 250 feet in overburden and 842 feet in rock.

(c) Test Pits. - Test pits were excavated and sampled at the three sites. Whenever possible, the test pits were extended by auger boring. In general, test pits were excavated to a depth of approximately 6 feet before auger boring. One pit at the Little Hill Dike site was carried to a depth of 12 feet. Bag, jar, density, and undisturbed 12-inch cube box samples were taken. In general, bag samples were taken at intervals of 2 feet and jar samples when required. A total of 7 undisturbed box samples and 17 constant volume density samples were taken at the dam site. A total of 61 pits at the dam site, 5 pits at the Dodge Estate Dike site, and 20 pits at the Little Hill Dike site were excavated, augered, and sampled with a total footage of 377 feet of excavation and 97 feet of auger boring.

(d) Seismic Investigations. - Seismic investigations were conducted at the dam and Little Hill Dike sites to supplement the drill hole information for determining the location of bedrock. The work was conducted by A. E. Wood, Associate Geologist of the Binghamton District, using the method developed by E. R. Shepard, Senior Physicist, Office of the Chief of Engineers (for brief description, see article "The Seismic Method of Exploration Applied to Construction Projects" by E. R. Shepard, The Military Engineer, September-October, 1939). A total of 8 seismic lines at the dam site and 2 at the Little Hill Dike site were fired with an aggregate length of 4220 feet and 119 rock elevation determinations.

5-02. Exploration of Materials for the Dam Embankment, Dikes, and Concrete Aggregates. (a) General. - Exploration to locate suitable materials for the dam embankment, the two dikes, and for concrete aggregates, included test pit excavation and sampling, auger boring, drilling and seismic work. The work included detailed exploration for the initial development and some additional exploration for the ultimate development. Test pits were usually excavated to a depth of approximately 6 feet. Whenever possible, auger boring was used for the exploration and to extend test pits. Bag, jar and density samples were taken from the test pits and bag samples from auger holes. In general, bag samples were taken at 2-foot intervals. Prior to the selection of the final borrow areas, all possible sources of materials in the vicinity of each site were explored (see Figure V-1). Drilling and seismic explorations were used in the vicinity of the final selected areas to determine the depths of suitable materials. The location and extent of the preliminary and final explorations in and near the final selected areas are shown on Sheets 6, 11 and 12 of the contract drawings. Complete field logs of the test pits and auger holes and a report of the seismic investigations are on file in the District Office. All soil samples are stored in the District Soils Laboratory.

(b) Dam Embankment. - (1) Preliminary and final exploration for materials for the dam embankment and for processed sand and gravel for all structures consisted of excavating 144 test pits and auger holes, firing 11 seismic lines, and drilling 1 hole. The selected Borrow Areas A, B and C shown on Sheet 6 of the contract drawings contain impervious, pervious and sandy gravel materials, respectively. Within the boundaries or immediately adjacent to the selected areas, 51 test pits, 8 seismic lines, and 1 drill hole were located. Large exposures of the materials in Borrow Areas A and B were made in connection with compaction tests. In Borrow Area C, 2 of the test pits were excavated to a depth of 20 feet. In Borrow Area C, samples were taken from each test pit and screened in the field to determine the percentage of stones between 1/4-inch and 1 inch, 1 inch and 2 inches, and greater than 2 inches. Representative samples which have been tested in the Central Concrete Laboratory, West Point, N. Y., to determine the suitability of this material for concrete aggregate show excellent possibilities.

(2) No exploration was necessary to determine the location of deposits of suitable rock for the various features of the embankment since a sufficient quantity is available from the structure and borrow material excavations.

(c) Dodge Estate Dike. - Exploration for the embankment material for the main part of the Dodge Estate Dike consisted of excavating 27 6-foot test pits of which 1 is within the final selected area as shown on Sheet 12 of the contract drawings. Exploration for sand and gravel was included in the exploration for borrow materials for the dam.

(d) Little Hill Dike. - To locate embankment materials for the Little Hill Dike, 77 test pits and auger holes, 3 seismic lines, and 1 drill hole were used. Borrow Area H-1 as shown on Sheet 11 of

the contract drawings was finally selected containing both pervious and impervious materials, and Borrow Area H-2 was selected containing sandy gravel for backing, filters, drainage blanket and toe. Within the selected areas were located approximately 15 test pits and auger holes and 1 seismic line. Of the test pits in Borrow Area H-1, 2 pits were excavated to a depth of approximately 20 feet. Suitable rock and riprap may be obtained from outcrop deposits on the right abutment or from numerous stone walls in the vicinity.

5-03. Method and Extent of Field Tests. (a) Field Compaction Tests. - To determine to what extent the impervious and pervious embankment materials from Borrow Areas A and B could be compacted, field compaction tests were performed using full size equipment. A report on these tests is given in Appendix B.

(b) Pressure Testing of Bedrock. - The bedrock was pressure tested at 18 drill holes at the dam site and at 2 drill holes at the Little Hill Dike site. The tests were made using standard pressure pumping equipment. The data of the tests were recorded on the drill hole field logs which are on file in the District Office.

5-04. Methods and Extent of Laboratory Tests and Investigations. (a) General. - To determine those characteristics of the foundation and embankment materials for the dam and the two dikes required for proper design and construction, a comprehensive program of laboratory tests was followed using the samples, cores and data obtained during the field exploration. Most of the tests, which included classification, moisture content, specific gravity, density, permeability, shear, critical void ratio, consolidation, expansion and optimum moisture tests, were made in the District Soils Laboratory. The methods used and the extent of the individual tests are described briefly in the following paragraphs. After studying the results of the tests and the nature of the various materials, values of the characteristics were selected which were used in the design. These values are shown on Figures VI-2, VII-2 and VIII-1, and by curves on figures designated in Sections VI, VII and VIII. A final detailed report of all laboratory tests and investigations will be prepared and placed on file in the District Office.

(b) Classification. - (1) All samples from test pits, drill holes and auger borings were classified using the M.I.T. scale of classification and recorded in a laboratory log for each hole. The grain size distributions of a sufficient number of samples were determined by the usual sieve and hydrometer methods to serve as a guide in the classification of all samples. General geological descriptions of the overburden at each drill hole were prepared after studying the laboratory classifications and the field logs. The bedrock at each drill hole was classified by a geologist after examining the rock cores and field logs.

(2) In addition to the above described classification, samples from test pits in or near Borrow Area C which were screened in the field (see paragraph 5-02(b)) were also screened on the No. 3, 20 and 65 mesh Tyler sieves in the laboratory. The results were combined

with those obtained in the field to determine the suitability of the gradation of the materials for concrete aggregates.

(c) Moisture Content. - Moisture content determinations were made on a few samples taken from the deposits of impervious and pervious materials for the dam embankment and of impervious material for the Little Hill Dike. No moisture content determinations were deemed necessary for the other materials since the natural moisture contents vary appreciably with climatic conditions. The determinations were made by oven drying approximately 100 grams of material.

(d) Specific Gravity. - The specific gravities of the solid phase of several representative soil samples of borrow and foundation materials for all sites were determined using the pycnometer flask method with water as the reference base.

(e) Density. - (1) The natural densities of the foundation materials under the upstream section of the dam embankment, and of the major embankment materials were determined in the laboratory by testing chunk and constant volume samples taken during the field exploration and tests. The averages of the results of void ratio, unit weights and degrees of compaction are shown on Figures VI-2, VII-2 and VIII-1. These averages and the results of individual tests were used to determine the shearing strength and permeability which are discussed in paragraphs 5-04(f) and (g). Natural densities of the foundation materials at the two dike sites and under the downstream section of the dam embankment were not determined, and the values given were estimated on the basis of experience with similar materials and indications from other test data.

(2) The densities to which the impervious and pervious materials for the dam embankment can be economically compacted were determined by field tests described in paragraph 5-03(a). On the basis of these tests, the densities to which the dike materials, except the pervious material for the Little Hill Dike, can be economically compacted have been estimated and are shown on Figures VI-2, VII-2 and VIII-1. The compacted density of the pervious material for the Little Hill Dike was estimated from the results of the critical void ratio tests and other test data. The characteristics discussed in the following paragraphs were determined for the materials compacted to these values.

(f) Permeability. - (1) Permeability tests were made on remolded and undisturbed samples of foundation and borrow materials. All test results were corrected to a temperature of 10 degrees Centigrade. A total of 68, 23 and 23 tests were made on remolded samples for the dam, Dodge Estate Dike and Little Hill Dike, respectively. Each test required one or more runs with the material placed at various void ratios. Tests of single runs were made when no information was available as to the actual value of the natural or compacted void ratio of the materials. Pervious embankment materials and some samples from the foundation under the upstream section of the dam embankment were tested with 4 to 6 runs at various void ratios. The results of these tests were plotted on semi-log paper to show the variation of

the coefficient of permeability with the void ratio. From the curves; the coefficients of permeabilities corresponding to the actual or assumed void ratios of the materials in nature or in the compacted embankment were determined.

(2) In general, remolded samples of pervious materials were tested in "falling head" type of permeameters consisting of a Lucite or glass cylinder varying from 2 to 4 inches in diameter and a small glass tube serving as the "head tube". Some of the very pervious materials were tested in metal permeameters. Deaerated water was used for all tests.

(3) Remolded samples of the impervious materials for the dam embankment and Little Hill Dike were tested in a consolidation device 4-1/4 inches in diameter and 1-1/4 inches high. Material was placed in the device at its plastic limit and was consolidated under a load of 2 tons per square foot. By placing material in this manner, densities were obtained equivalent to those shown for compacted materials on Figures VI-2 and VIII-1. Water was forced upwards through the sample and the coefficient of permeability was calculated from the usual "falling head" measurements.

(4) Tests were made on 7 and 2 undisturbed 12-inch cube box samples taken respectively from the foundation of the dam embankment and from the field test embankment of pervious material, described in paragraph 5-03(a). Each sample was tested in two directions in a large, wooden permeameter of the "falling head" type.

(5) The results of tests on remolded and undisturbed foundation samples were used to determine the permeability of all pertinent foundation samples by comparison of gradation. Based on a complete study of the values and the nature of the foundation deposits at the three sites, the design values were determined.

(6) The design values of the coefficients of permeabilities for the various embankment materials are considered the most representative and were based on a complete study of all test data and compaction characteristics of the materials and the design features of the embankments.

(g) Shearing Strength. - The shearing strengths of representative soil samples of material for the dam embankment and Little Hill Dike and foundation materials at the dam were determined. Values of shearing strength of the foundation materials of the Little Hill Dike and the foundation and embankment materials of the Dodge Estate Dike, based on their similarity to other materials tested, were assigned. The shearing strengths of cohesionless materials were determined from the data of the critical void ratio tests which are described in paragraph 5-04(h). The critical void ratio tests give slightly conservative values for shearing strength since they were run at higher stress intensities than will actually occur in the foundations or embankments. A representative sample of each of the impervious materials for the dam embankment and Little Hill Dike was tested in the Providence District Soils Laboratory using a direct shear machine

similar to that described by Dr. Glennon Gilboy in Engineering News Record, May 1936. The material was thoroughly kneaded to its plastic limit, placed in the shear box, and consolidated under a load of either 2 or 4 tons per square foot before the normal load was adjusted for the test run. Tests in a consolidation device with the materials placed in a similar manner indicate that the void ratios as obtained under a load of 2 tons per square foot are equivalent to those for compacted materials as shown on Figures VI-2 and VIII-1. After consolidation, the sample was submerged, the desired normal load was applied, and the test performed before subsequent consolidation or rebound could occur. A rate of strain of about 0.05 inch per minute and normal loads of 1/4 to 4 tons per square foot were used.

(h) Critical Void Ratio. - (1) The critical void ratios under various intensities of minor principal stress were determined on representative samples of cohesionless material for the dam embankment and Little Hill Dike and of the foundation material under the upstream section of the dam embankment. Tests were conducted on all materials in the District Soils Laboratory by the Taylor-Fidler method, which was described in a paper given by Professor D. W. Taylor of the Massachusetts Institute of Technology at the Atlantic City Convention of the A.S.T.M., June 1939.

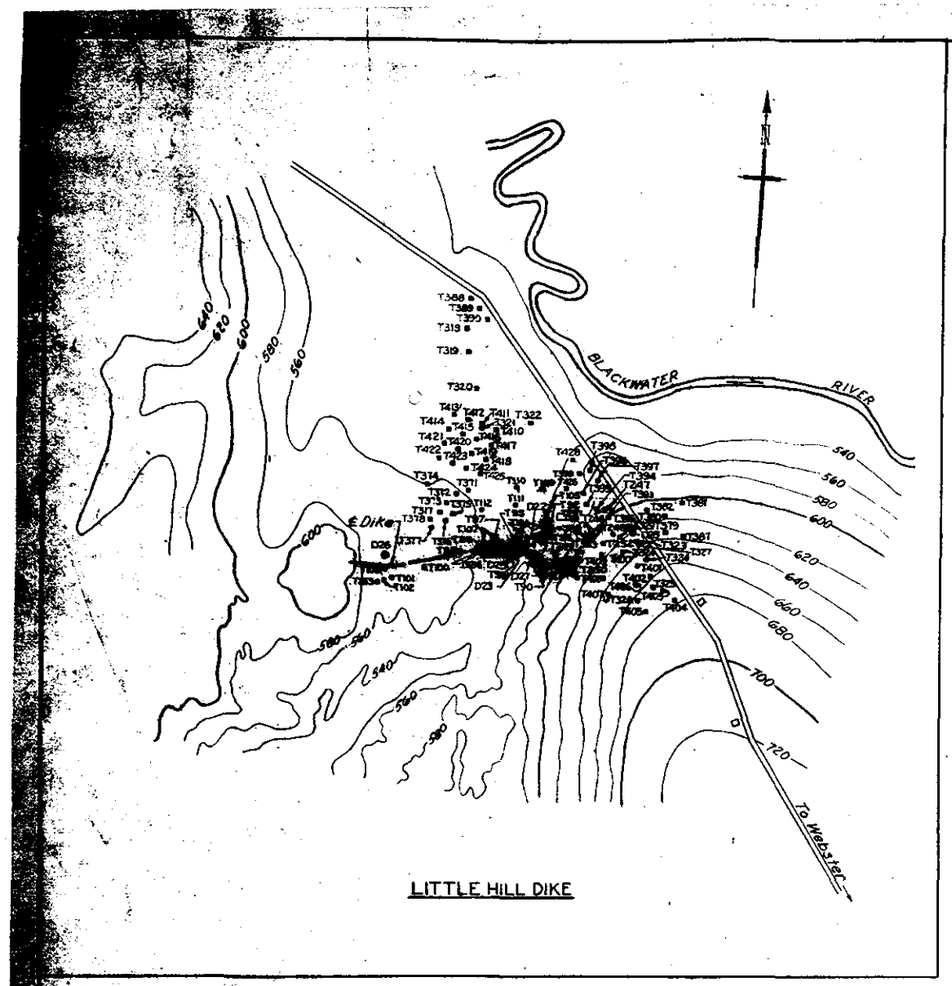
(2) For these tests a triaxial compression device, similar to that described in Appendix BII Analysis of Design, Franklin Falls Dam, was used to test samples 6.9 centimeters in diameter and 20 centimeters high. From these tests the critical void ratio under pressure was determined. Tests were also made at Harvard University by the Harvard method on the pervious materials for the dam embankment as described in Appendix B. The data of tests made at Harvard have been replotted to determine critical void ratio under pressure.

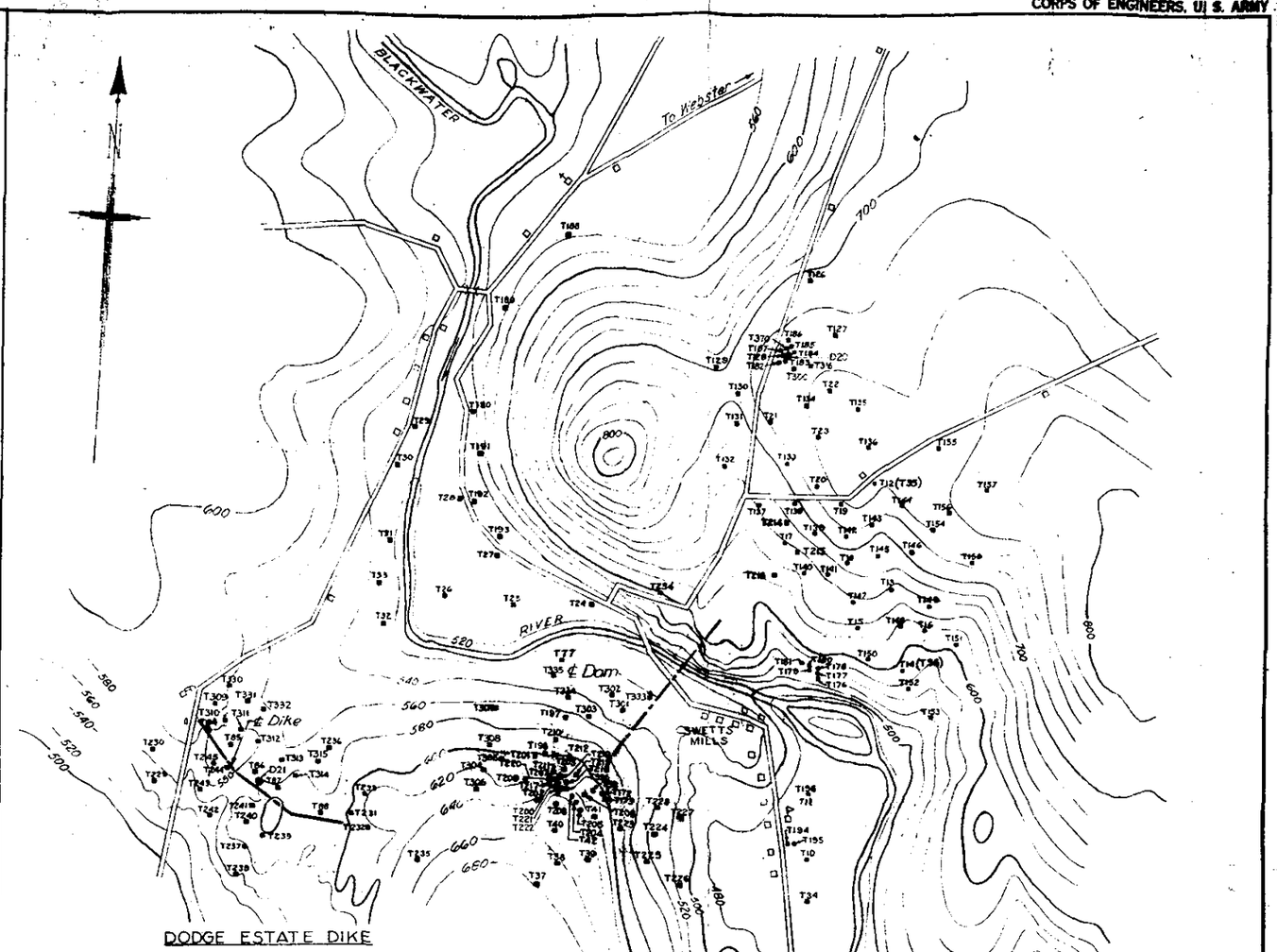
(i) Consolidation. - Consolidation tests, using a consolidation device 4-1/4 inches in diameter and 1-1/4 inches high, were made on all samples tested for critical void ratio to determine the change in void ratio due to the weight of the embankment. The only other consolidation tests required are those on impervious embankment material. Prior to construction, consolidation tests will be made on these materials, the results of which will be used to determine the consolidation during and after construction and the density after construction.

(j) Expansion. - To determine if the impervious materials for the dam embankment and Little Hill Dike will expand on being saturated, tests on representative samples were made. The material was compacted in a consolidation device at various densities and their corresponding optimum moisture contents. After a sample was consolidated under a load of 1/4 ton per square foot, water was applied to the base of the sample and the vertical movement recorded as the sample became saturated. The test data indicated that neither material would expand on becoming saturated if under a pressure of 1/4 ton per square foot.

(k) Optimum Moisture. - Series of Proctor tests with the number of blows varying from 10 to 60 were performed on representative samples of impervious materials for the dam embankment and for the Little Hill Dike to determine the optimum moisture for any desired compacted density. The tests were made on binder material, defined as material passing the 1/4-inch sieve. The optimum moisture content of binder material for a compacted dry weight of 120 pounds per cubic foot is 13 and 12 percent by dry weight for the dam embankment and Little Hill Dike materials, respectively.

(l) Filter. - Two filter tests were made using the pervious and sand filter (Material passing 1/4 inch sieve) materials for the dam embankment and sand and gravel backing and pervious materials for the Little Hill Dike. The tests, made on 2-inch diameter samples, consisted in forcing, for a period of 4 hours, deaerated water vertically downward through a 2-inch layer of embankment material under a hydraulic gradient of 30 and into a 2-inch layer of filter material from where it was discharged. Measurements were made every 30 minutes to determine coefficients of permeability and visual observations were made of the conditions of the samples particularly at the boundary between the filter and embankment material. Toward the end of the tests the samples were tapped and vibrated. In both tests the permeability did not change during the 4-hour period nor was there any visual evidence of transportation of pervious material into the filter material either under steady seepage or under seepage with vibration.





MERRIMACK VALLEY FLOOD CONTROL
BLACKWATER RESERVOIR
BLACKWATER RIVER
PLAN OF BORROW AREA
EXPLORATION

IN SHEETS SHEET NO. SCALE: 1"=400'

U. S. ENGINEER OFFICE, BOSTON, MASS.

APPROVAL RECOMMENDED: DATE:

SUBMITTED:

VI DESIGN OF DAM EMBANKMENT AND DAM EMBANKMENT FOUNDATION

6-01. General. - 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 will utilize economically the available embankment materials and will be safe under any conditions. The methods and the extent of investigations to determine the characteristics of the foundation and embankment materials have been described in Section V. 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:

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

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

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

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

6-02. Geological Features and Characteristics of Foundation Materials. - (a) General. - The geological features and characteristics of the foundation materials influencing the design of the dam embankment and dam embankment foundation are summarized in the following paragraphs.

(b) Description of Site. - The valley at the site, as shown by the topography on Sheet 3 of the contract drawings, is assymetric with the river flowing in a rock gorge against the left valley wall which rises abruptly. To the right of the existing channel is a broad and shallow former flood channel. The right wall of the valley rises from this former valley bottom with a long, fairly gentle slope. Bedrock is generally shallow over the entire site as shown by the center line profile on Sheet 5 of the contract drawings, and the rock contours on Figure VI-1. Numerous outcrops exist in the valley bottom and walls of the river gorge. Bedrock in the right abutment in general rises with the topography at a depth of about 15 feet. Bedrock in the left abutment is exposed in the lower regions and is buried approximately 15 feet near the top of the proposed dam embankment. In the valley bottom, the depth to rock is shallow downstream from the proposed centerline. Upstream from the proposed centerline, it dips so that the average depth of overburden under the upstream section of the proposed embankment will be approximately 15 feet.

(c) Bedrock. - (1) The bedrock over the site is a coarse crystalline granite with local gneissic phases and quartz veins. The classi-

fication of the bedrock at each drill hole is shown on Sheets 4 and 5 of the contract drawings. The bedrock is structurally sound except for the top few feet of weathered rock which is present in the left abutment. An ancient and inactive fault striking North 23 degrees East crosses the river channel about 150 feet downstream from drill hole D29 and dips 45 degrees Southeast. In general, there are three sets of joints. The two major sets, which were formed in the older rock, trend roughly north-south and east-west, magnetic bearing, and some have been filled with younger rocks forming prominent dikes. In addition to the two major sets of joints, there is a third set striking about North 45 degrees East magnetic. These joint planes dip at an angle of about 55 degrees to the Southeast, differing from the two major sets which are nearly vertical.

(2) Some of the joints in the bedrock are open as disclosed by the pressure testing during exploration. Bedrock at the drill holes was pressure tested with depths varying from 20 to 75 feet. The loss of water at some of the holes was negligible while at others, the loss was at a rate of 14 gallons per minute. It is believed that joints in the region of the right abutment are tighter than elsewhere over the site.

(d) Overburden. - The overburden on both abutments is a sandy glacial till. In the valley bottom occur variable water laid deposits of cohesionless sands. The classification of the overburden at each drill hole is shown on Sheets 4 and 5 of the contract drawings. The overburden on the right abutment is similar to the material for the embankment as described in paragraph 6-03 (b). The overburden on the left abutment is quite variable, ranging from slightly silty sand to very silty sand and gravel, and probably contains numerous boulders. The overburden downstream from the centerline in the bottom of the valley is shallow, containing numerous large boulders and pockets of silty to gravelly sand and organic materials. The overburden upstream from the centerline is in general a uniform fine to medium sand. The values of the characteristics shown on Figure VI-2, based on the investigations described in Section V, are considered representative for design purposes. The range of grain size distribution of the samples taken during the exploration upstream from the center line is shown on Figure VI-3. The results of the critical void ratio and consolidation tests on a representative sample are shown on Figures VI-4 and VI-5, respectively. The grain size curve of the material tested is shown on Figure VI-6.

6-03. Availability and Characteristics of Dam Embankment Materials. (a) General. - The materials for the embankment will be obtained from structure excavation and from the borrow areas shown on Sheet 6 of the contract drawings. The borrow areas contain the most economical materials in the vicinity which are suitable for a safe design. The following paragraphs describe the characteristics of the materials and deposits of each area, as determined from the exploration and investigations described in Section V, which influence the design of the dam embankment and dam embankment foundation.

(b) Pervious Material. - (1) The material for the compacted pervious fill sections of the dam embankment will be obtained from Borrow Area B, and a minor quantity of suitable material will be obtained from structure excavation. The borrow area deposit is, in general, quite homogeneous and contains a silty sand and gravel till. For design pur-

poses it has been assumed that 25 percent of the material in the deposit consists of stones and boulders larger than 6 inches in size which must be removed before the material is compacted. The material in nature is very compact and the finer phase occurs at a unit dry weight of 119 pounds per cubic foot and has a moisture content of approximately 5 percent by dry weight. The grain size curve of the material tested by the Taylor-Fidler Method is shown on Figure VI-6. The variation of permeability with void ratio is shown on Figure VI-8.

(2) A typical grain size curve of the material with the stones and boulders greater than 6 inches removed, is shown on Figure VI-7. When compacted, the pervious material will have the values of the characteristics tabulated on Figure VI-2 which have been used for design purposes. The results of the critical void ratio and consolidation tests on representative samples are shown on Figures VI-8, 9 and 10.

(c) Impervious Material. - (1) The impervious fill for the dam embankment will be obtained from Borrow Area A. The material in the deposit is a very silty sand till containing some clay and gravel sizes. The deposit contains a small percentage of stones and boulders over 6 inches in size which will be removed before compaction. The suitable material is, in general, from 10 to 15 feet in depth overlying a sandy gravel. In nature, the finer phase of the suitable material has a unit dry weight of 124 pounds per cubic foot and a moisture content of 10 percent based on the dry weight.

(2) A typical grain size curve of the material with the stones greater than 6 inches removed is shown on Figure VI-7. When compacted, the material will have the values of the characteristics which are used for design purposes and are tabulated on Figure VI-2. The results of shearing strength tests are shown on Figure VI-12. The grain size curve of the sample tested is shown on Figure VI-6. The optimum moisture content of the binder material (material passing the 1/4 inch sieve) for efficient compaction to any unit weight can be obtained from the curve on Figure VI-13. It is concluded from the laboratory tests that the material compacted to any density will not expand due to an increase of the moisture content.

(d) Filter Materials. - Filter materials will be obtained from the sandy gravel deposits in Borrow Area C. The gravel for the filter against the downstream rock toe will be processed and will be graded according to the specification for concrete coarse aggregate with maximum size of 2 inches. The finer phase of filter material against the downstream rock toe will have the gradation stated below for sand and gravel backing.

(e) Sand and Gravel for Backing and Drainage Blankets. - The material for sand and gravel backing and drainage blankets will be obtained from the deposits in Borrow Area C. The material for sand and gravel backing will be bank-run and contain at least 40 percent by dry weight of stones passing the 6-inch mesh screen and retained on the 1/4 inch sieve and of the material passing the 1/4 inch sieve, not more than 5 percent by dry weight will pass the No. 200 mesh sieve. Drainage blanket material will be a well graded sand or bank-run gravelly sand of which the material passing the 1/4-inch sieve not more than 5 percent shall pass the 200 mesh sieve. Excess material from coarse aggregate

processing may be used providing it passes this specification. All stones greater than 6 inches will be removed from materials for the backing and blanket. The coefficient of permeability when compacted will be approximately 100 x 10^{-4} cm. per sec.

(f) Rock. - Material for the rock fill sections of the embankment will be obtained from required rock excavation and by culling of the material from Borrow Area B. The bedrock to be excavated and the boulders in the deposit of Borrow Area B are composed mainly of sound granite and granitic gneiss.

6-04. Dam Embankment Sections. (a) Sections Considered. - (1) Prior to the submission of the Definite Project Report dated September 15, 1939, the general location and characteristics of available borrow materials and the general foundation conditions had been determined by field and laboratory investigations. Sections were subsequently chosen to satisfy the foundation conditions and to utilize the impervious and pervious borrow materials described in paragraph 6-03. Prior to the selection of the final section, the three sections shown on Figure VI-14 were considered.

(2) Section No. 1 was submitted with the Definite Project Report. Provisions as shown were provided for the heightening of the section for the future ultimate development. A top width of 71 ft. 9 inches was provided to prevent the necessity of disturbing the dumped rock on the upstream slope of the initial section, in the event that the embankment be heightened. The major portions of the embankment consisted of impervious and pervious materials described in paragraph 6-03.

(3) At the Design Conference on November 10, 1939, (see Appendix C), Section No. 2 was presented with the impervious section centered in the initial section. Provisions were made to heighten the embankment by adding all material on the downstream slope and sloping the impervious section. Less material was required for this section for initial development than for that of Section No. 1, resulting in a lower initial cost. To minimize the quantity of pervious material described in paragraph 6-03 (b) which contains numerous boulders and will be relatively expensive to excavate, random material was incorporated in a large part in Section No. 2. The material for the random fill sections was a uniform fine to medium sand, free from boulders, which can be obtained adjacent to the site.

(4) At the Design Conference, November 10, 1939, several changes and recommendations in the design of Section No. 2 were made from which Section No. 3 was developed. Due to the gradation it was decided to utilize pervious material in place of random material in the shell sections.

(b) Selected Section. - As the design progressed to its final stage, the section shown on Sheet 8 of the contract drawings was developed. This section is essentially the same as Section No. 3 (Figure VI-14) except that a large upstream rock fill section is incorporated to utilize all rock available from Borrow Area B and from structure excavations. The embankment may be heightened for the ultimate

development as described in Section I. The essential features of the embankment will be as described in the following paragraphs.

(c) Impervious Features. - The dam embankment for the initial development will contain a central compacted impervious fill section of material described in paragraph 6-03 (c). Since the overburden at the site is shallow, as described in paragraph 6-02 (d), the impervious fill will extend to the bedrock for the full length of the embankment. The bedrock will be thoroughly grouted to a depth of 20 to 50 feet, as shown on Sheet 9 of the contract drawings, to prevent seepage through the joints described in paragraph 6-02 (c). Impervious fill will encase a part of the non-overflow concrete section at the east end of the embankment. The non-overflow section will be constructed with lateral walls as shown on Sheet 14 of the contract drawings to provide a proper seal.

(d) Pervious Features. - The compacted impervious fill section will be flanked with compacted pervious fill sections composed of material described in paragraph 6-03 (b). A large upstream rock fill section will be provided to protect the embankment during drawdown of the reservoir. The upstream sections will rest directly on the foundation overburden material described in paragraph 6-02 (d). Between the rock fill and compacted pervious fill sections, there will be a layer of sand and gravel backing of material described in paragraph 6-03 (e). The downstream shell section will rest on the drainage blanket described in the following paragraph.

(e) Drainage Features. - To control any seepage through the bedrock which might not be prevented by grouting, a downstream drainage blanket will be constructed of sand and gravel or excess sand from aggregate processing described in paragraph 6-03 (e). The blanket will also lower the line of seepage in the downstream compacted pervious fill section. The blanket will extend approximately the full length of the embankment as shown on Sheet 7 of the contract drawings. The drainage blanket joins the downstream rock toe which serves as a lateral drain and protects the toe of the embankment against erosion and excessive seepage pressures. A two course filter of materials described in paragraph 6-03 (d) will be placed on surfaces of the rock toe in contact with the foundation and pervious section.

(f) Slope Protection. - The slopes of the dam embankment will be protected against erosion by dumped rock on the upstream slope and by seeded topsoil on the downstream slope. Protection is not required on the slopes of the abutments due to their natural features.

6-05. Location of Centerline. The location of the centerline was shifted three times during the design. The initial centerline passed through drill holes D9 and D10 shown on Sheet 3 of the contract drawings. Most of the drill hole exploration was located in reference to this centerline. During the course of design, the left end of the centerline was shifted upstream about 100 feet, passing approximately through drill holes D9 and D2. This centerline was used to determine the relative costs of an all concrete dam and an earth embankment dam as discussed in Section II. For the final selected centerline shown on Sheet 3 of the

contract drawings, the position of the left end was shifted further upstream about 40 feet. The final location is superior for the spillway structure, while the embankment yardage is approximately the same as for the previous location. The final centerline does not cross the fault zone described in paragraph 6-02 (c).

6-06. Economy of Construction. The embankment will be composed of suitable materials from required structure excavation and Borrow Areas A, B and C. As described in paragraph 6-03 (a), the borrow areas contain the most economical suitable materials for the embankment. The pervious material from Borrow Area B will be difficult to excavate due to the numerous boulders. It was deemed advisable to use this material as described in paragraph 6-04 (a). Provision has been made to utilize all of the stone and boulders from the Borrow Area B as well as from structure excavation.

6-07. Statical Analysis. (a) General. - (1) Following the precedent of the Swedish Geotechnical Committee's study on slope failures in artificial cuts, the stability ratio against shear failure of the dam embankment and dam embankment foundation was determined by investigating the forces tending to cause movement (driving forces) and those producing potential resistance to movement on several circular sliding surfaces of weakness which were selected by systematic trial. This method investigates only the possibility of a shear failure. The analysis of stability to insure against flow slide failure is described in paragraph 6-08. 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 shearing strength generated along the sliding surface. The ratio of the potential resisting force and the driving force is called the stability ratio. A sufficient number of potential planes were analyzed to determine the position of the plane having the least stability ratio. The stability ratio of the weakest plane is called the "minimum" stability ratio for the section analyzed. A "minimum" stability ratio of unity therefore, 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) Analysis was made of the assumed section for the ultimate development which includes the section designed for the initial development. It is known by experience that the initial section will have a minimum stability ratio, by the method used, equal to or greater than the minimum stability ratio of the assumed maximum section for the ultimate development. In the analysis the embankment and foundation were considered to be integral and sliding surfaces were allowed to pass through foundation and embankment sections without discrimination. The soil characteristics used in the analyses of the various sections were obtained from the results of tests described in Section V, tabulated on Figure VI-2.

(b) Upstream Slope. - The results of the investigations of several potential failure planes in the upstream section of the dam embankment are shown on Figure VI-15. The minimum stability ratio,

considering the embankment and foundation as a whole, and for the conditions existing immediately after a sudden drawdown (which, from previous experience, are known to produce the least stability) is 1.42. The method of analysis in detail is shown on Figures VI-16 and VI-17. The section within the potential sliding surface was divided into vertical slices and the driving and potential resisting forces determined for each slice. The stability ratio was determined by summing up each set of forces for the entire length of the sliding surface. The usual approximation was made that the soil forces on either side of each slice balance each other. It is assumed that above low water level for power storage the rock fill and sand and gravel backing sections will drain during drawdown and that the compacted pervious material is saturated. The water pressure at any point is equal to the vertical height of water above that point. These conditions were assumed since drawdown will occur in a relatively few days and the time required to drain completely the compacted pervious fill section is 2540 days according to the computations shown on Figure VI-18. The same assumptions were made for the compacted impervious fill section.

(c) Downstream Slope. - The results of the analysis of the downstream section of the dam embankment for the ultimate development are shown on Figure VI-15. The minimum stability ratio against shear failure for the embankment and foundation is 2.19. The same method described for the upstream slope was used except that the determination of hydrostatic pressures was made from a flow net drawn for the condition of steady seepage.

(d) Foundation and Abutments. - Stability of the foundation against shear failure was determined in conjunction with the analyses of the upstream and downstream slopes as described above. No problem exists on either abutment or at the transition section between the concrete and earth embankment.

6-08. Flow Failure Analysis. (a) General. - A flow failure is a movement permitted by the rapid liquefaction of a loose saturated mass of cohesionless material. A mass of cohesionless material can liquefy only when it is saturated and when drainage and permeability do not permit a rapid change in void ratio during a disturbance. The disturbance may be caused by an internal subsidence, an earthquake, a shear failure, a large shock or similar phenomena. Liquefaction may be prevented by compacting the mass to or below the critical void ratio. The critical void ratio is dependent upon the stress and various other conditions and, until more is known about these factors, the results of laboratory tests must be considered as only indicative of the true values. Values of the critical void ratio have been determined for assumed stress conditions using methods developed at Harvard University and the Massachusetts Institute of Technology as described in paragraph 5-04 (h). For correlation of the test data with embankment and foundation conditions in the following analyses it is assumed that the greatest minor principal stress that can occur in the foundation due to any disturbance is equal to one half the maximum embankment pressure and in the embankment is equal to the total embankment pressure at the bottom of the section.

(b) Embankment Section. - The only part of the dam embankment which has characteristics that might permit a flow failure is the upstream compacted pervious fill. A large mass of the downstream compacted pervious fill will not become saturated. Other cohesionless materials to be placed in the embankment are very pervious and are placed where drainage conditions are very favorable. The maximum embankment load on the upstream compacted pervious fill will be 4.2 tons per square foot (assuming an average unit weight of 142 pounds per cubic foot). The critical void ratio for a minor principal stress of 4.2 tons per square foot, as determined by the Taylor-Fidler method, the results of which are shown on Figure VI-8, is 0.37. The critical void ratio by the Harvard method for a minor principal stress of 4.2 tons per square foot will be slightly greater than 0.35, since the results shown on Figure VI-9 are for a minor principal stress of 5.27 tons per square foot. To produce void ratios of 0.37 and 0.35 after construction, the material must be compacted to void ratios of 0.39 and 0.38, respectively, according to the curves shown on Figure VI-10. The material will be compacted to a void ratio of 0.38 as described in paragraph 6-03 (b); hence, according to the results of either set of tests, a flow failure cannot occur in material compacted to this void ratio.

(c) Embankment Foundation. - Since the portion of the dam embankment downstream from the core will be placed on rock, except near the right abutment where stress intensities are inappreciable, the only part of the dam embankment foundation that need be considered is upstream from the core. The permeability characteristics and drainage conditions of the foundation material upstream from the core are such that a flow slide can occur if the material is not sufficiently dense. According to the assumption in paragraph 6-08 (a), the maximum minor principal stress that can develop is 2.1 tons per square foot. The critical void ratio for this minor principal stress is 0.82 according to the test results shown on Figure VI-4. The average natural void ratio of the foundation material as shown on Figure VI-2 is 0.80. This void ratio, due to a maximum embankment load of 4.2 tons per square foot, will be reduced to 0.74 according to the curves on Figure VI-5. Since this void ratio is lower than the critical void ratio it is concluded that a flow failure cannot occur in the foundation.

6-09. Seepage Analysis. (a) Seepage. - The seepage through the dam embankment for the initial development was analyzed by means of flow nets to determine the seepage pressures and the quantity of seepage. In this analysis, bedrock was assumed to be impervious and the permeability values given on Figure VI-2 were used. The flow net for the maximum section of the initial development, constructed by the principles described by Dr. Arthur Casagrande of Harvard University in the Journal of New England Water Works Association, June 1937, is shown on Figure VI-19. The flow nets indicate that the seepage pressures will be negligible and that for the conditions assumed piping could not occur. The seepage per foot of embankment at maximum section of the initial development is computed, as shown on Figure VI-19, at 0.00002 cubic foot per minute. The total seepage through the initial dam embankment is computed at 0.006 cubic foot per minute. Any seepage that might develop through the rock foundation will not alter the flow nets since it will discharge into the drainage blanket or downstream from the toe. There

is no unusual seepage problem at the right abutment or at the transition between the earth embankment and the concrete section.

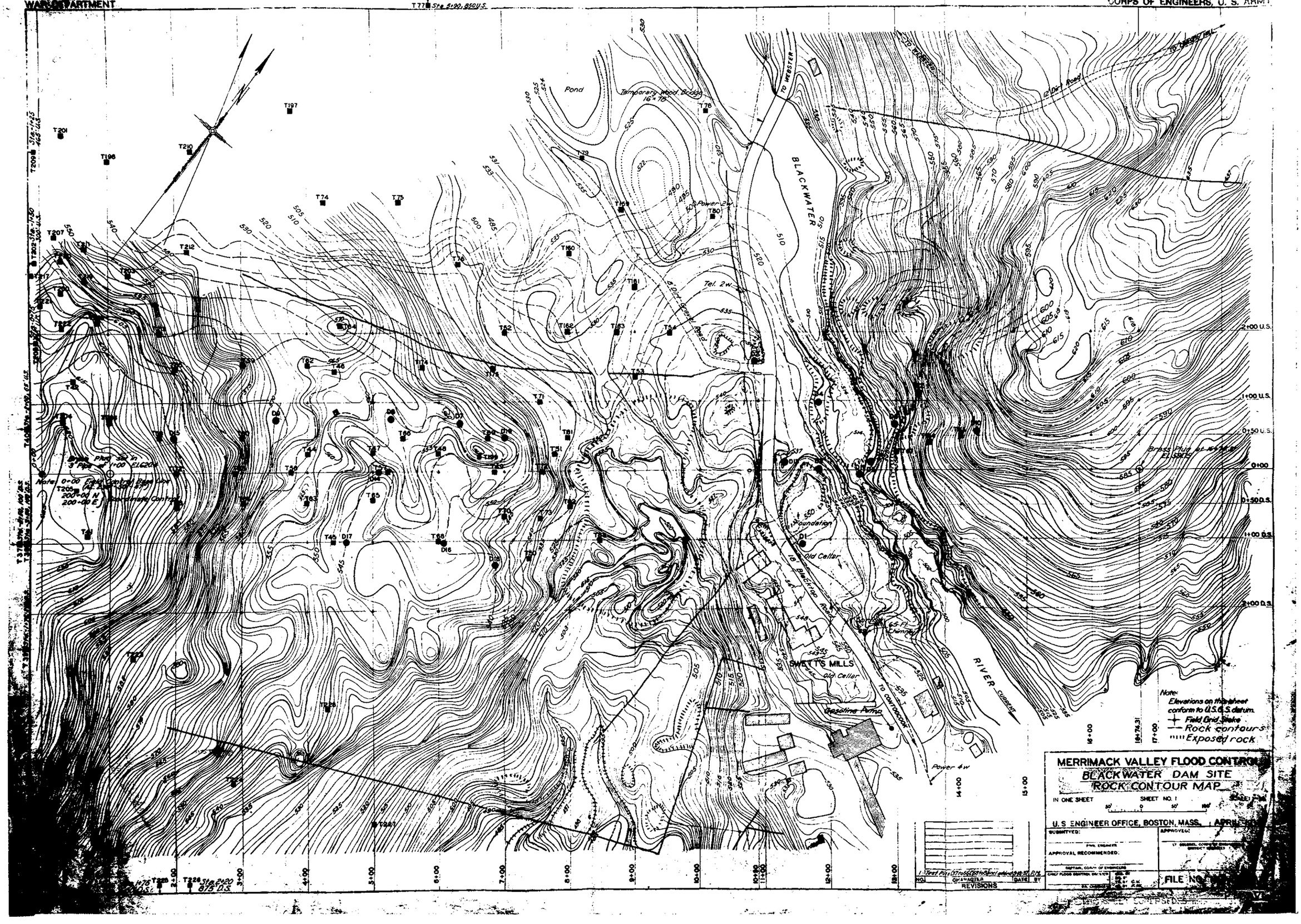
(b) Filters. - (1) The results of the filter test described in paragraph 5-04 (e) indicate that pervious embankment material will not be transported into sand filter material (material passing the 1/4 inch sieve) as described in paragraph 6-03 (d) with a hydraulic gradient as high as 30. The results of this test are considered representative for the pervious embankment material adjoining sand and gravel backing, sand and gravel filter and blanket materials. As shown by the values on Figure VI-2 the permeability of either the blanket, sand and gravel backing and filter materials is more than 10 times greater than the permeability of the compacted pervious material. It is concluded from the test and ratio of the permeabilities that sand filter or sand and gravel backing and filter and blanket material will serve satisfactorily as a filter placed next to the compacted pervious fill.

(2) The gradation of the gravel filter material adjacent to the rock toe as described in paragraph 6-03 (d) was chosen from the design studies described in the Analysis of Design for Franklin Falls Dam. The gravel filter material will be of sizes less than 2 inches which will insure ample safety in this phase. A gravel layer between the upstream rock fill and sand and gravel backing is not considered necessary since the sand and gravel backing will contain at least 40 percent stones greater than 1/4-inch.

6-10. Miscellaneous Analyses. -Expansion tests, as described in paragraph 5-04 (j) indicate that no expansion will occur in the compacted impervious fill due to an increase in moisture content of the material. Settlement analyses will be made after the exact construction program is determined. No provisions are made in the design for settlement of the embankment after construction since it is believed that the settlement will be negligible. If appreciable settlement does occur after construction, the embankment will be heightened as a maintenance operation.

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY



SUMMARY OF SOIL TEST DATA - MAIN DAM EMBANKMENT

	Upstream Foundation Material Natural Characteristics	Embankment Materials - Compacted Characteristics				
		Impervious	Pervious	Sand Filter	Gravel Filter	Sand and Gravel Backing
Dry Weight Binder Material lbs. per cu.ft.	94 Av.	120	122	111	111	111
Total Wet Weight lbs.per cu.ft.	---	140	144	---	---	131
Optimum Water Content Percent dry weight	---	13	10	---	---	---
Void Ratio, e Binder Material	Range 0.58-0.91 Av. 0.80	0.40	0.38	0.50	0.50	0.50
Max. Void Ratio, e_{max}	1.00	---	0.60	---	---	---
Min. Void Ratio, e_{min}	0.59	---	0.32	---	---	---
Degree of Compaction Percent	49	---	79	---	---	---
Coef. of Permeability, k 10^{-4} cm.per sec.	60	0.001	2	100	---	100
Angle of Internal Friction, ϕ-Degrees	34	27	38	36	36	36
Cohesion, C Tons per sq. ft.	0	0.10	0	0	0	0
Specific Gravity, s	2.71	2.70	2.70	---	---	---

FIG. VI - 2

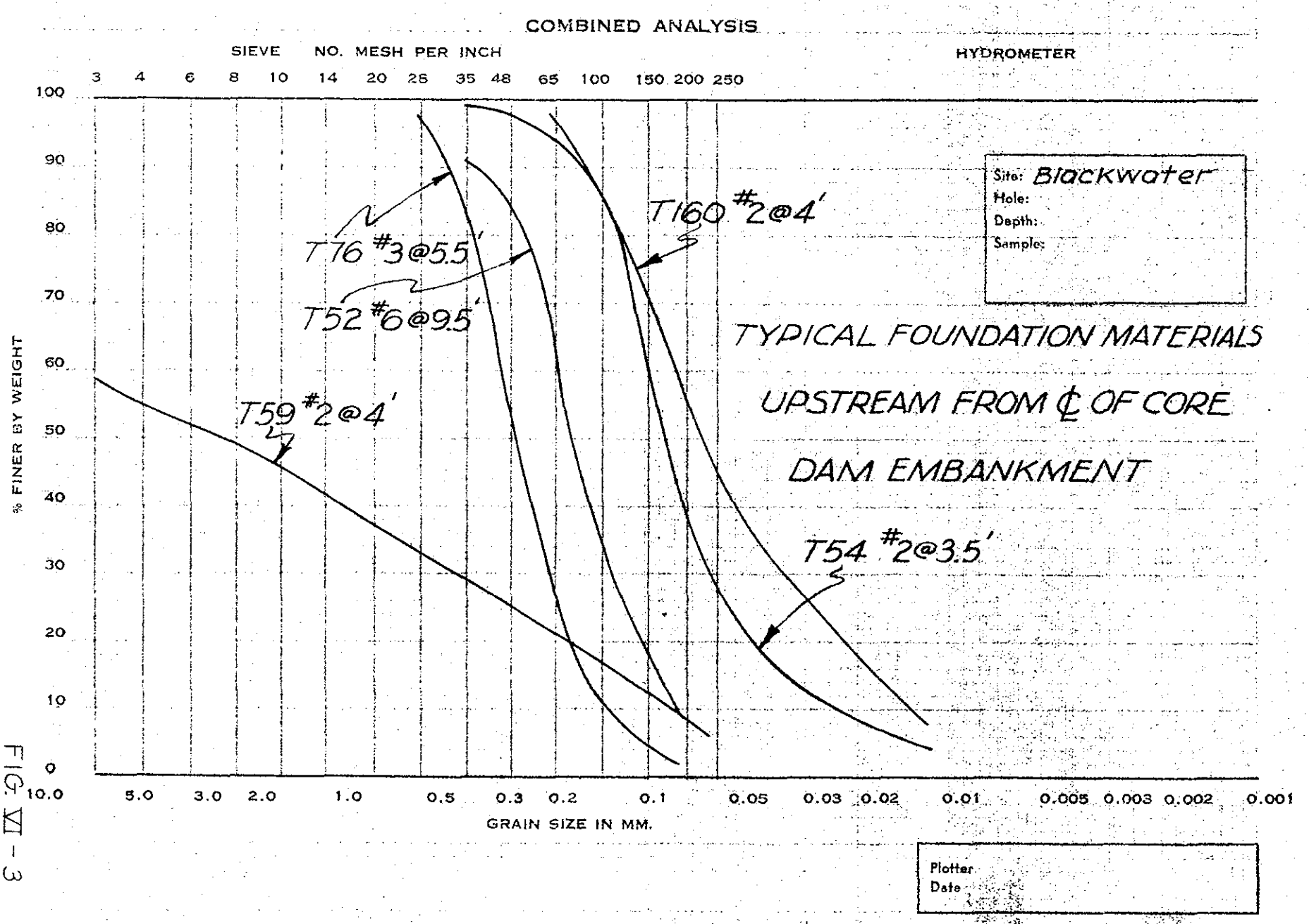
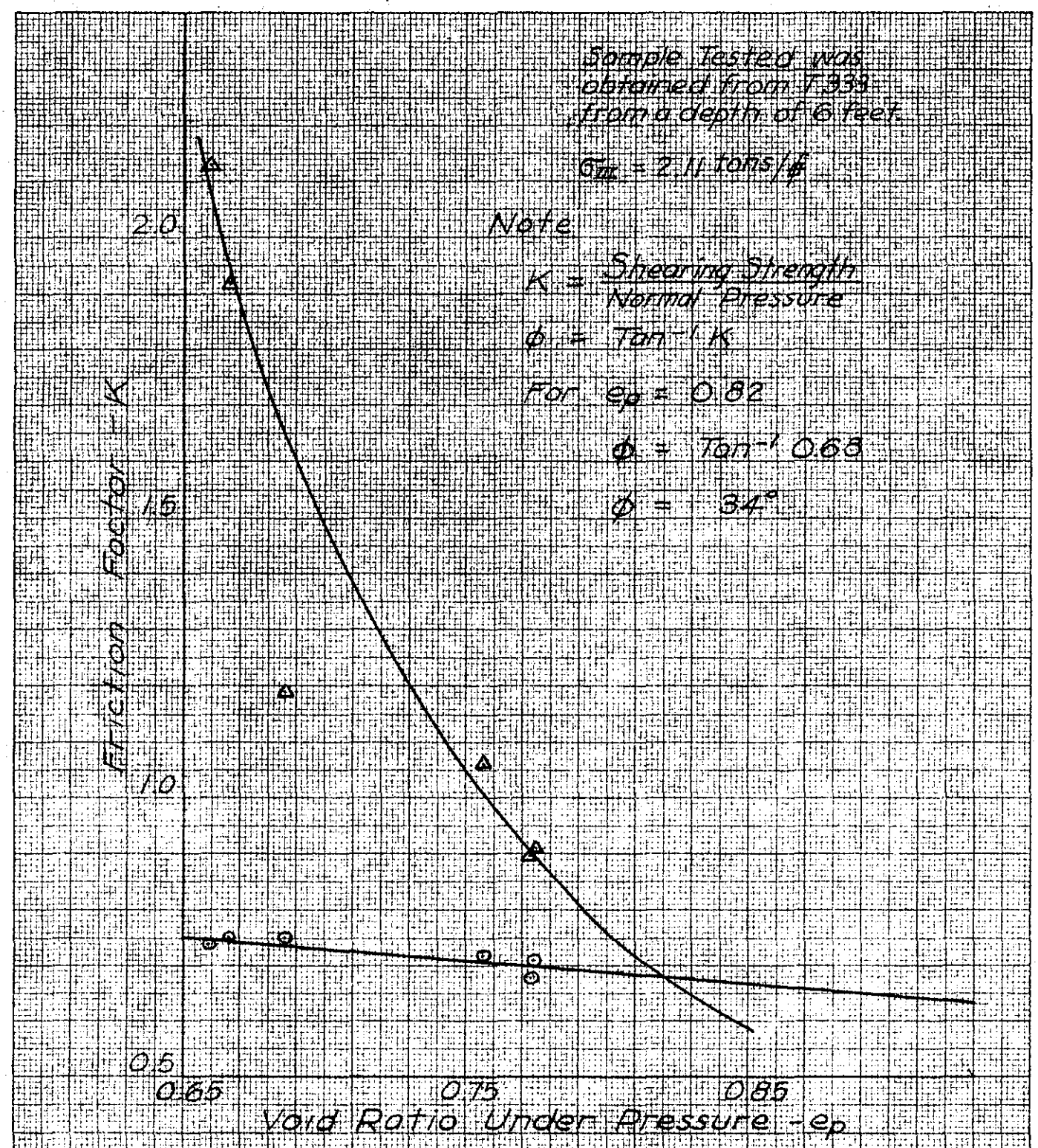

COMBINED ANALYSIS
SIEVE NO. MESH PER INCH
HYDROMETER
3 4 6 8 10 14 20 28 35 48 65 100 150 200 250
% FINER BY WEIGHT
100 90 80 70 60 50 40 30 20 10 0
10.0 5.0 3.0 2.0 1.0 0.5 0.3 0.2 0.1 0.05 0.03 0.02 0.01 0.005 0.003 0.002 0.001
GRAIN SIZE IN MM.
T16 #3@5.5'
T52 #6@9.5'
T59 #2@4'
T160 #2@4'
T54 #2@3.5'
Site: Blackwater
Hole:
Depth:
Sample:
TYPICAL FOUNDATION MATERIALS
UPSTREAM FROM ℄ OF CORE
DAM EMBANKMENT
Plotter
Date

FIG. VI - 3



CRITICAL VOID RATIO AND SHEARING STRENGTH TEST RESULTS
UPSTREAM FOUNDATION MATERIAL
DAM EMBANKMENT

FIG. VI - 4

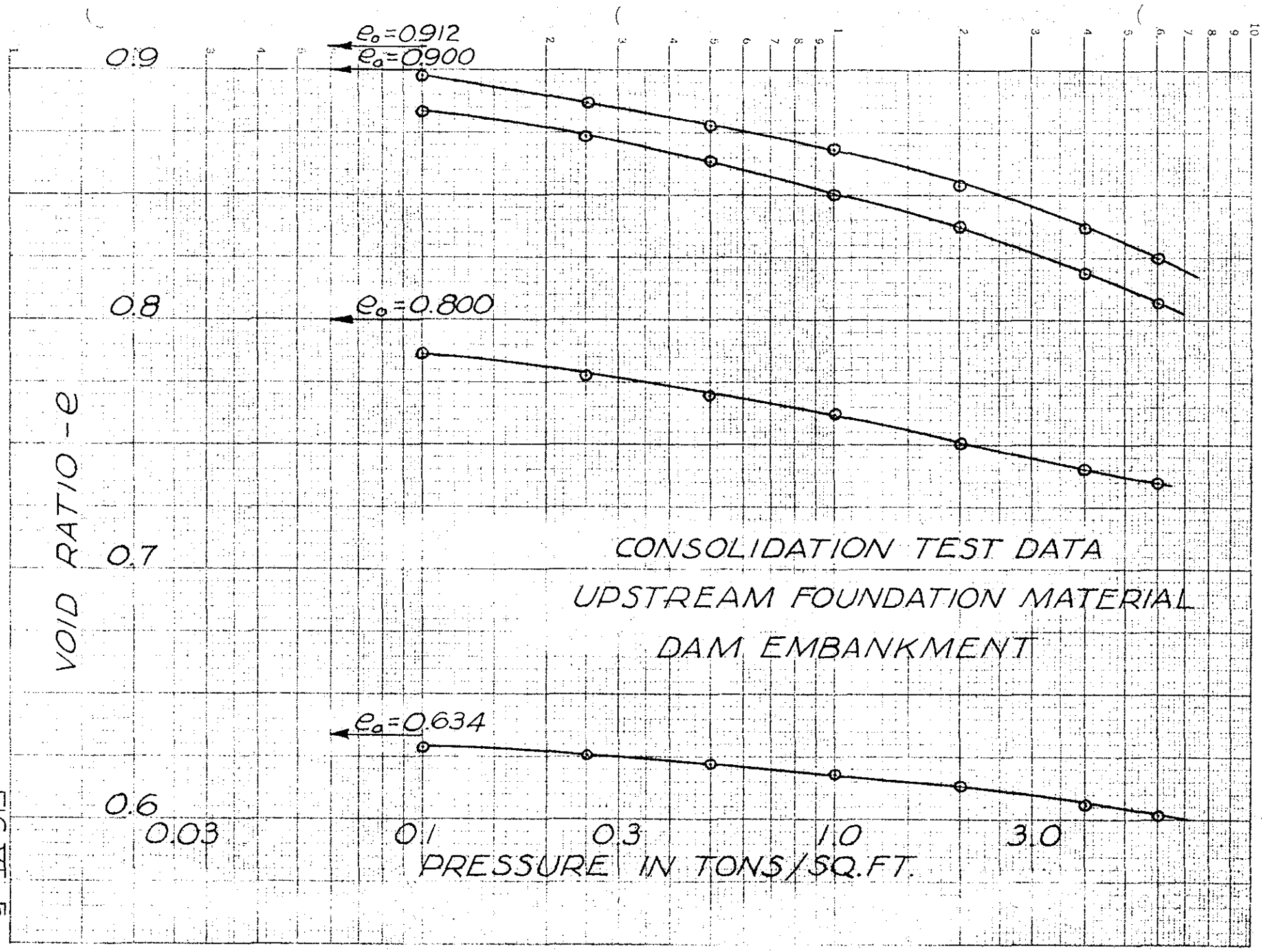

e_o=0.912
e_o=0.900
e_o=0.800
e_o=0.634
0.9
0.8
0.7
0.6
VOID RATIO - e
0.03
0.1
0.3
1.0
3.0
PRESSURE IN TONS/SQ.FT.
CONSOLIDATION TEST DATA
UPSTREAM FOUNDATION MATERIAL
DAM EMBANKMENT

FIG. VI-5

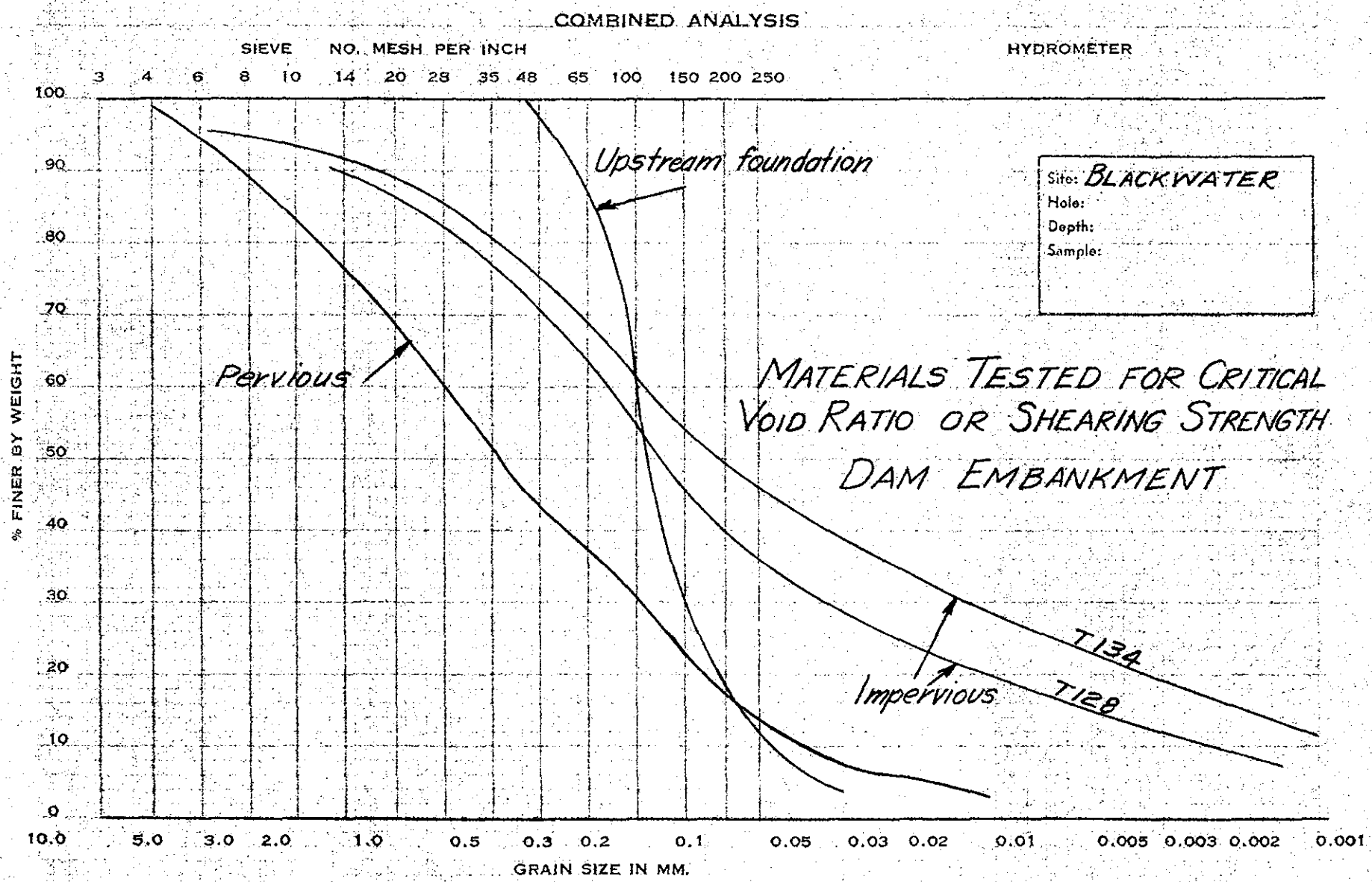

COMBINED ANALYSIS
SIEVE NO. MESH PER INCH
HYDROMETER
3 4 6 8 10 14 20 28 35 48 65 100 150 200 250
100 90 80 70 60 50 40 30 20 10 0
% FINER BY WEIGHT
10.0 5.0 3.0 2.0 1.0 0.5 0.3 0.2 0.1 0.05 0.03 0.02 0.01 0.005 0.003 0.002 0.001
GRAIN SIZE IN MM.
Upstream foundation
Pervious
Impervious
T134
T128
Site: BLACKWATER
Hole:
Depth:
Sample:
MATERIALS TESTED FOR CRITICAL VOID RATIO OR SHEARING STRENGTH
DAM EMBANKMENT
Plotter GPC
Date 1/3/40

FIG. VI-6

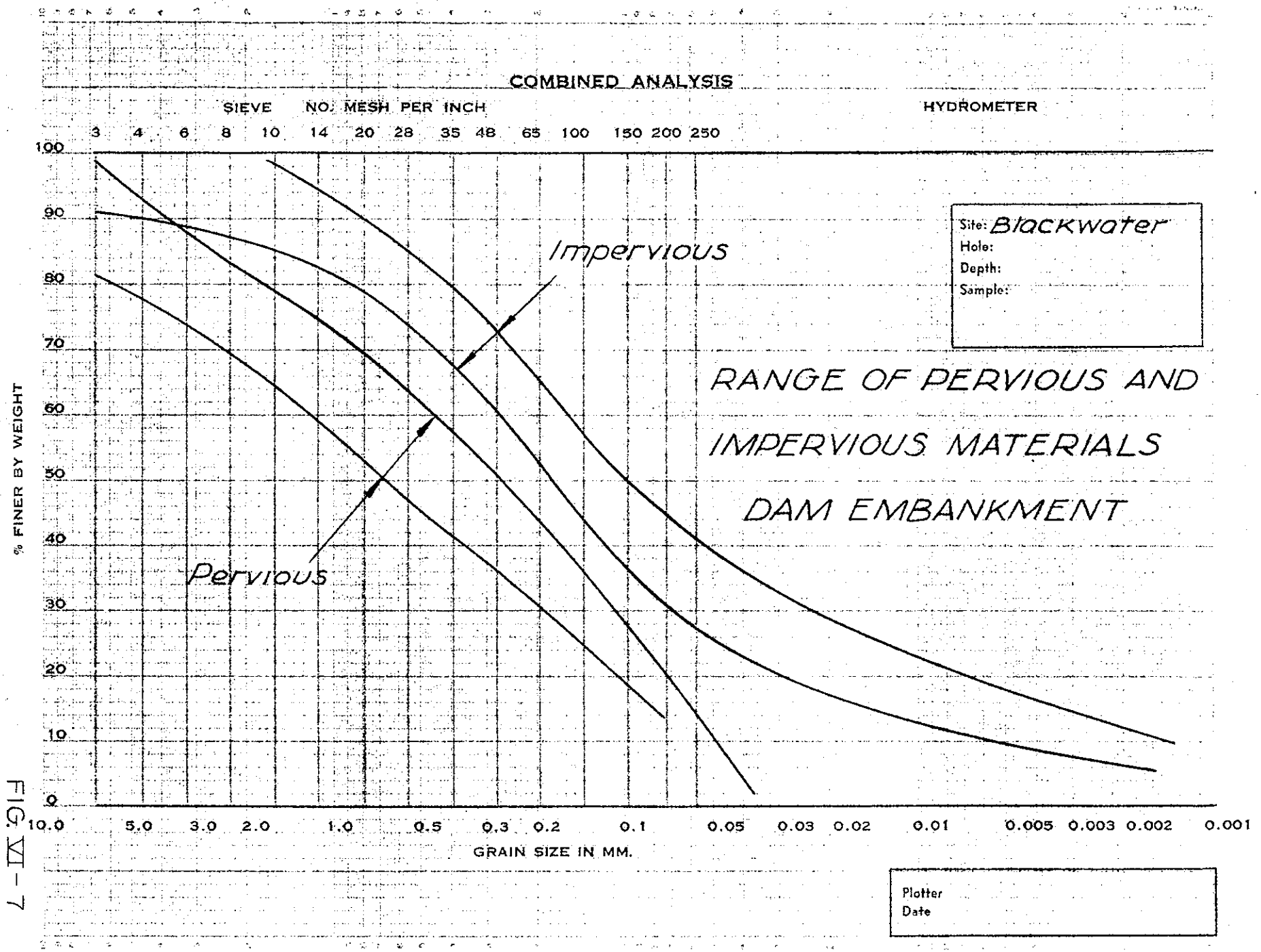
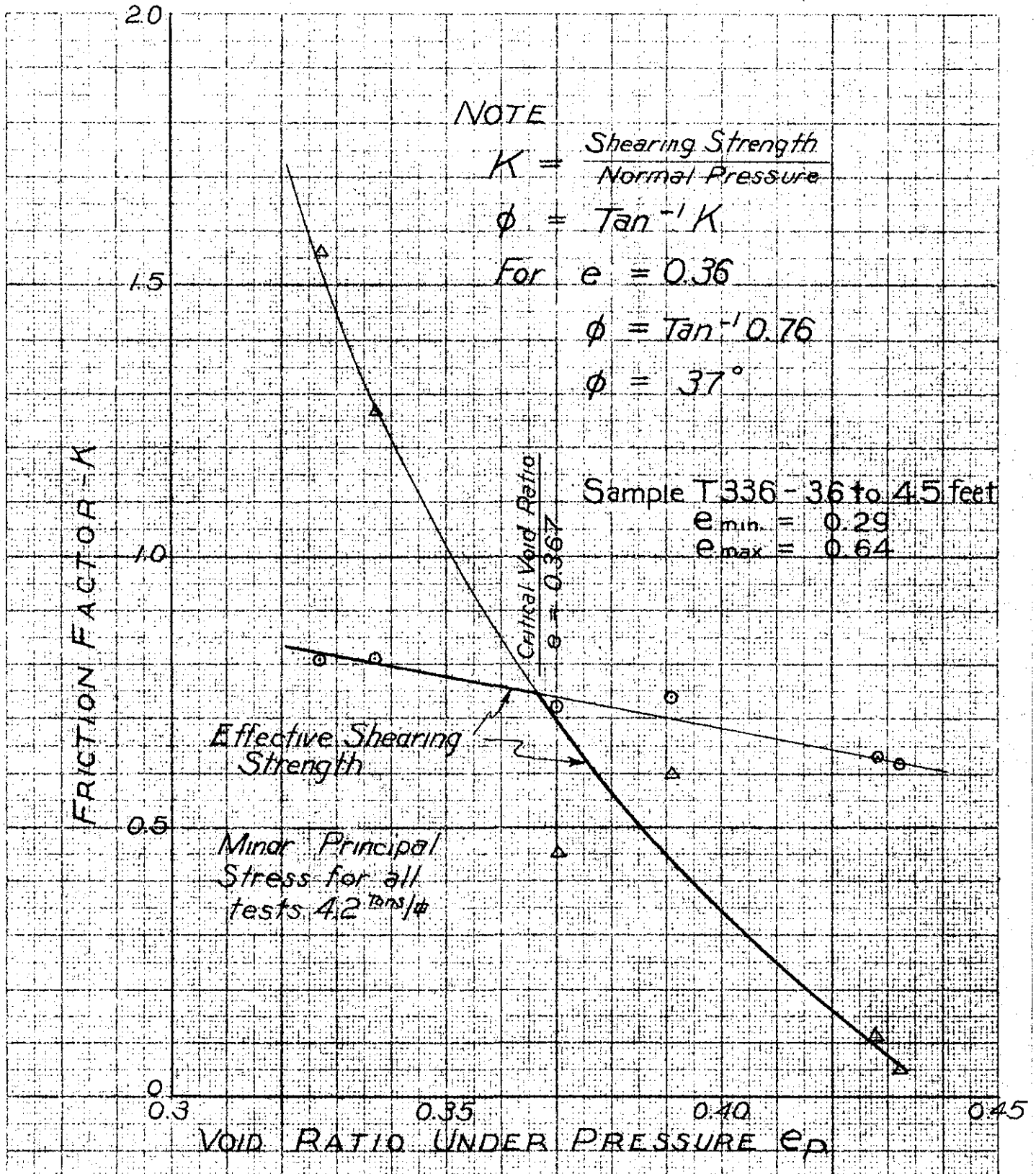

COMBINED ANALYSIS
SIEVE NO. MESH PER INCH
HYDROMETER
3 4 6 8 10 14 20 28 35 48 65 100 150 200 250
100
90
80
70
60
50
40
30
20
10
0
% FINER BY WEIGHT
10.0 5.0 3.0 2.0 1.0 0.5 0.3 0.2 0.1 0.05 0.03 0.02 0.01 0.005 0.003 0.002 0.001
GRAIN SIZE IN MM.
Impervious
Pervious
Site: Blackwater
Hole:
Depth:
Sample:
RANGE OF PERVIOUS AND
IMPERVIOUS MATERIALS
DAM EMBANKMENT
Plotter
Date

FIG. XI - 7



CRITICAL VOID RATIO AND
SHEARING STRENGTH TEST RESULTS
TAYLOR - FIDLER METHOD
PERVIOUS MATERIAL
DAM EMBANKMENT

FIG. VI-8

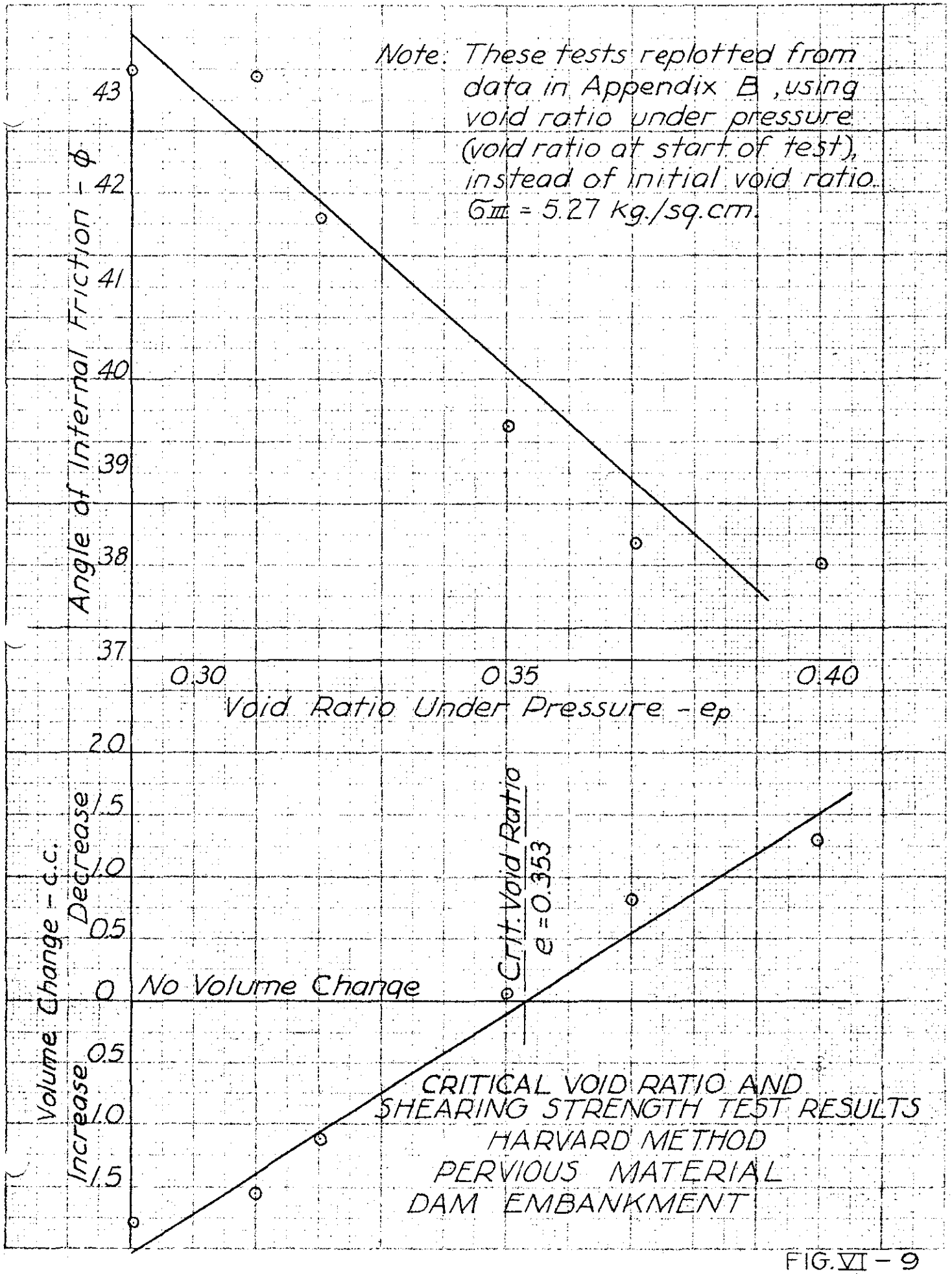
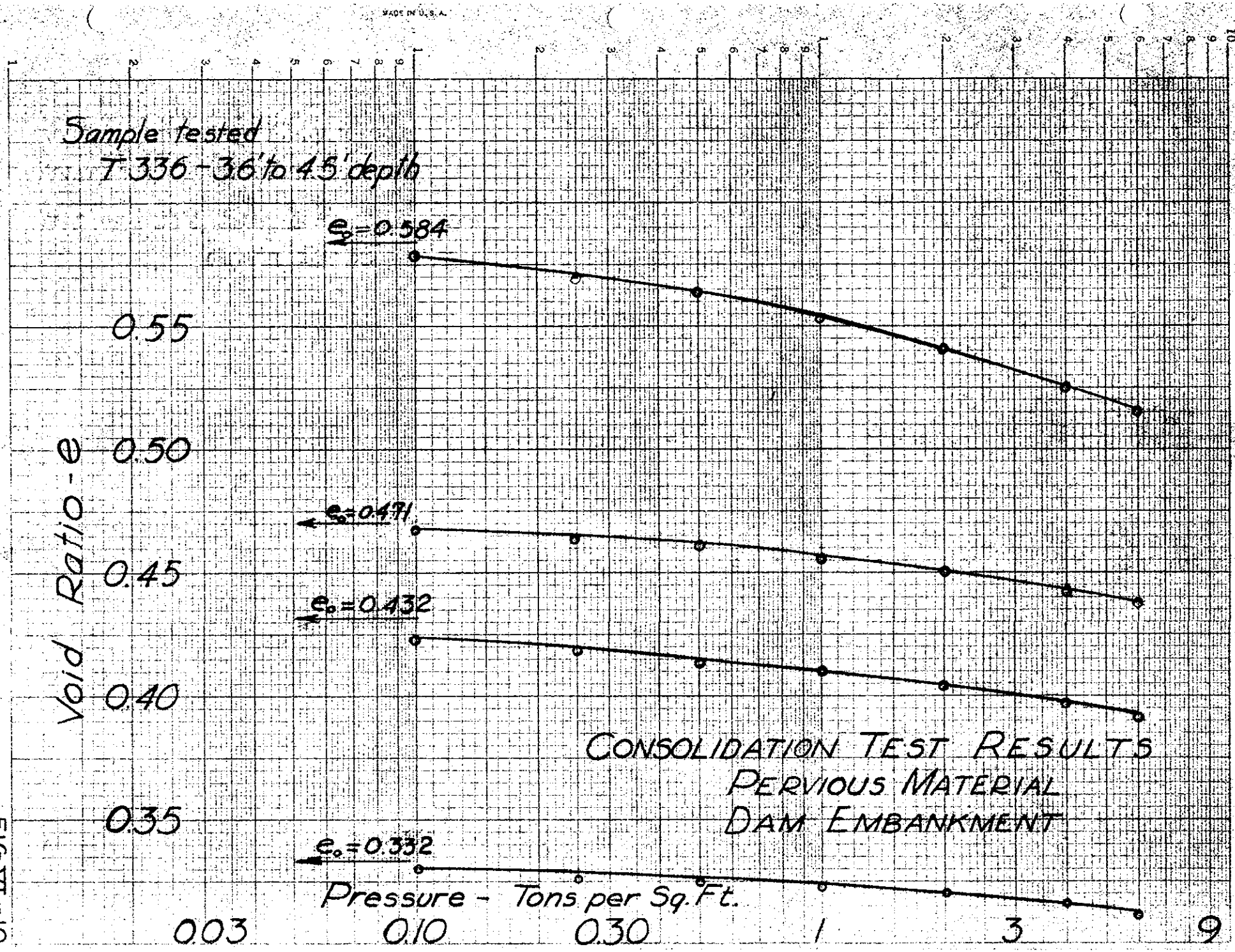

Note: These tests replotted from data in Appendix B, using void ratio under pressure (void ratio at start of test), instead of initial void ratio.
σ_{III} = 5.27 kg./sq.cm.
Angle of Internal Friction - ϕ
43
42
41
40
39
38
37
0.30
0.35
0.40
Void Ratio Under Pressure - e_p
Volume Change - c.c.
Decrease
Increase
2.0
1.5
1.0
0.5
0
0.5
1.0
1.5
No Volume Change
Crit. Void Ratio
e = 0.353
CRITICAL VOID RATIO AND
SHEARING STRENGTH TEST RESULTS
HARVARD METHOD
PERVIOUS MATERIAL
DAM EMBANKMENT

FIG. VI - 9



CONSOLIDATION TEST RESULTS
PERVIOUS MATERIAL
DAM EMBANKMENT

FIG. VI-10

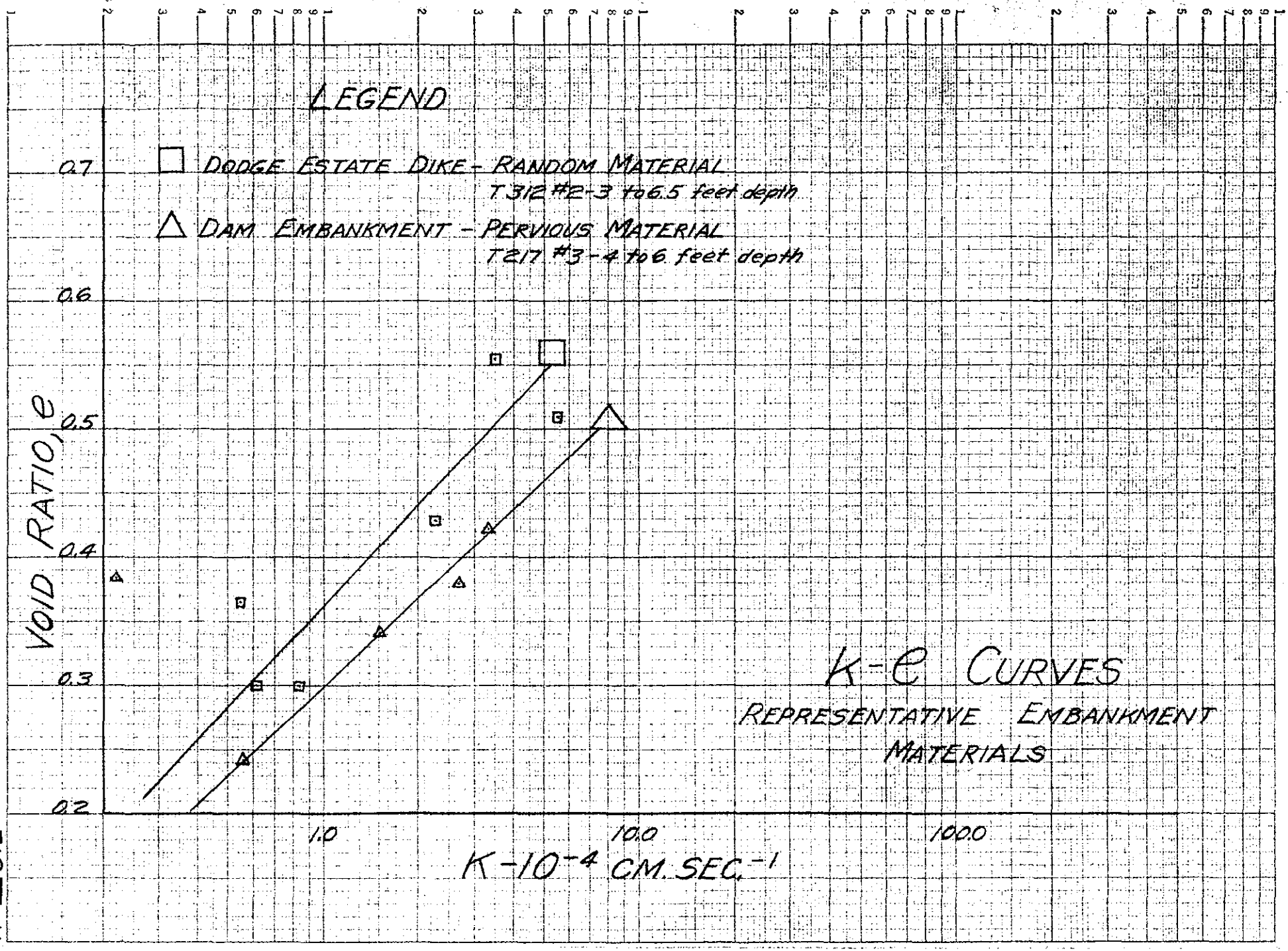

LEGEND
DODGE ESTATE DIKE - RANDOM MATERIAL
T312 #2-3 to 6.5 feet depth
DAM EMBANKMENT - PERVIOUS MATERIAL
T217 #3-4 to 6 feet depth
VOID RATIO, e
0.7
0.6
0.5
0.4
0.3
0.2
1.0
10.0
100.0
K-10^-4 CM. SEC.^-1
K-e CURVES
REPRESENTATIVE EMBANKMENT MATERIALS

FIG. VI-11

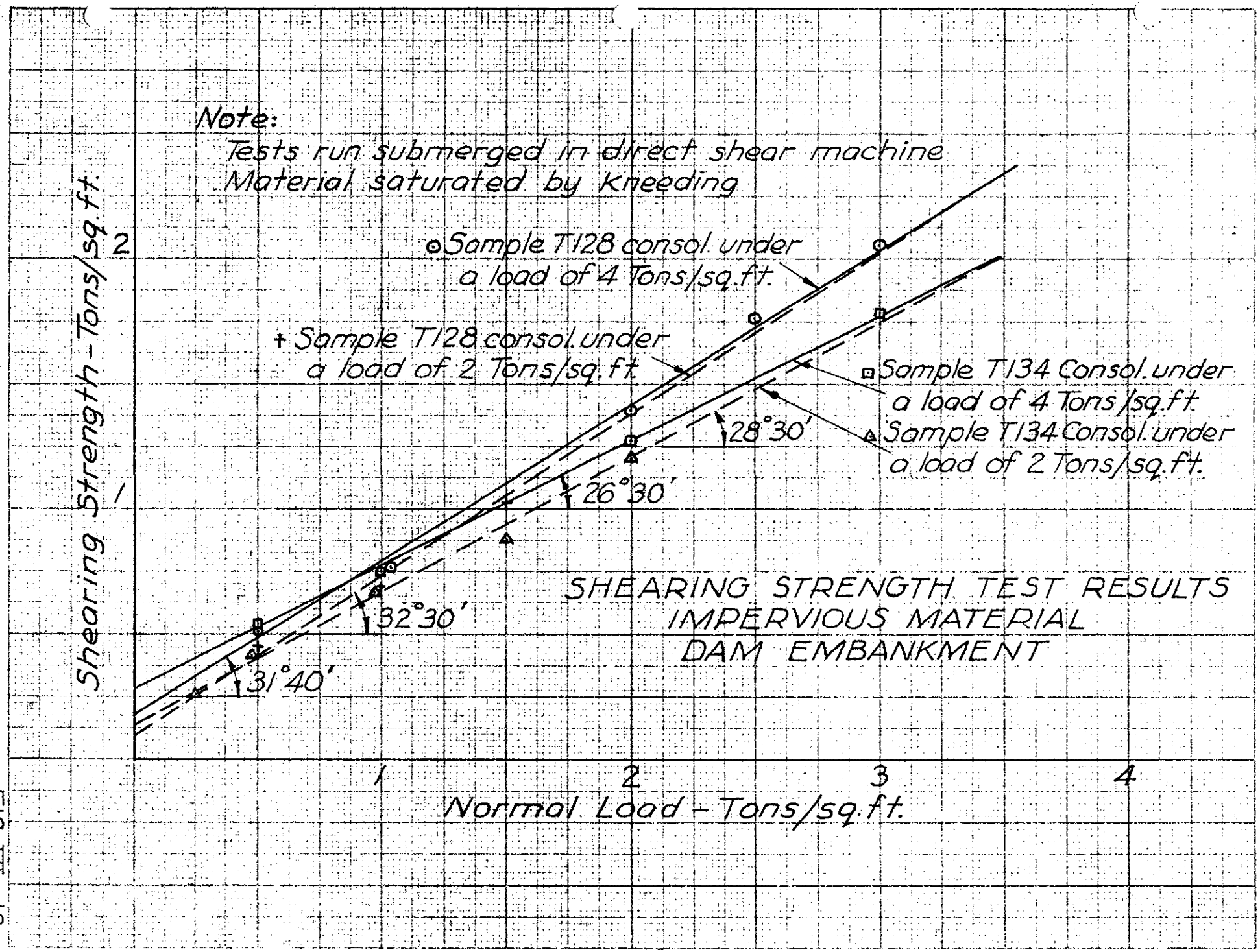

Note:
Tests run submerged in direct shear machine
Material saturated by kneeding
Sample T128 consol. under a load of 4 Tons/sq.ft.
Sample T128 consol. under a load of 2 Tons/sq.ft.
Sample T134 Consol. under a load of 4 Tons/sq.ft.
Sample T134 Consol. under a load of 2 Tons/sq.ft.
28°30'
26°30'
32°30'
31°40'
SHEARING STRENGTH TEST RESULTS
IMPERVIOUS MATERIAL
DAM EMBANKMENT
Shearing Strength - Tons/sq.ft.
Normal Load - Tons/sq.ft.
1
2
3
4

FIG. VI - 12

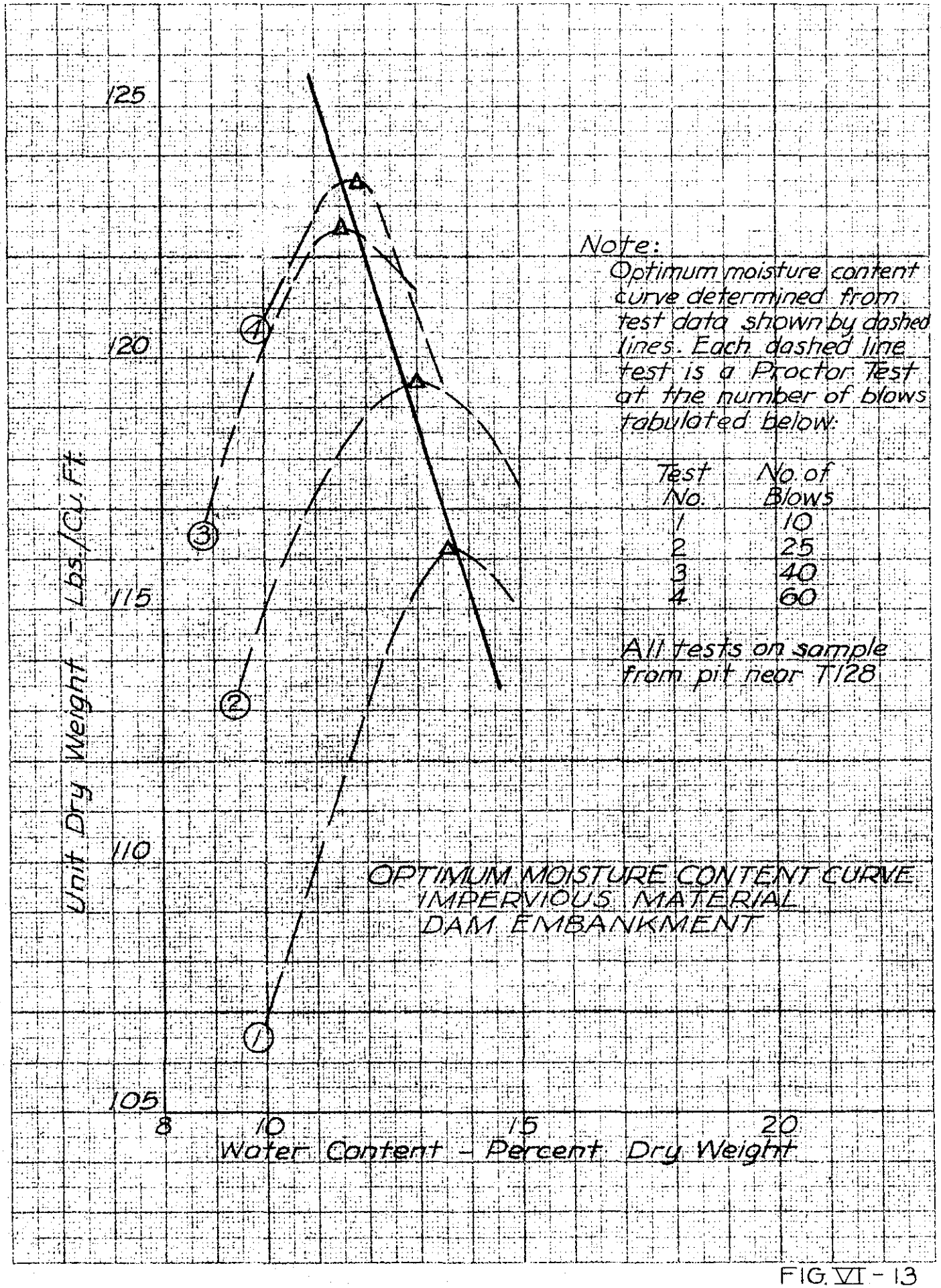
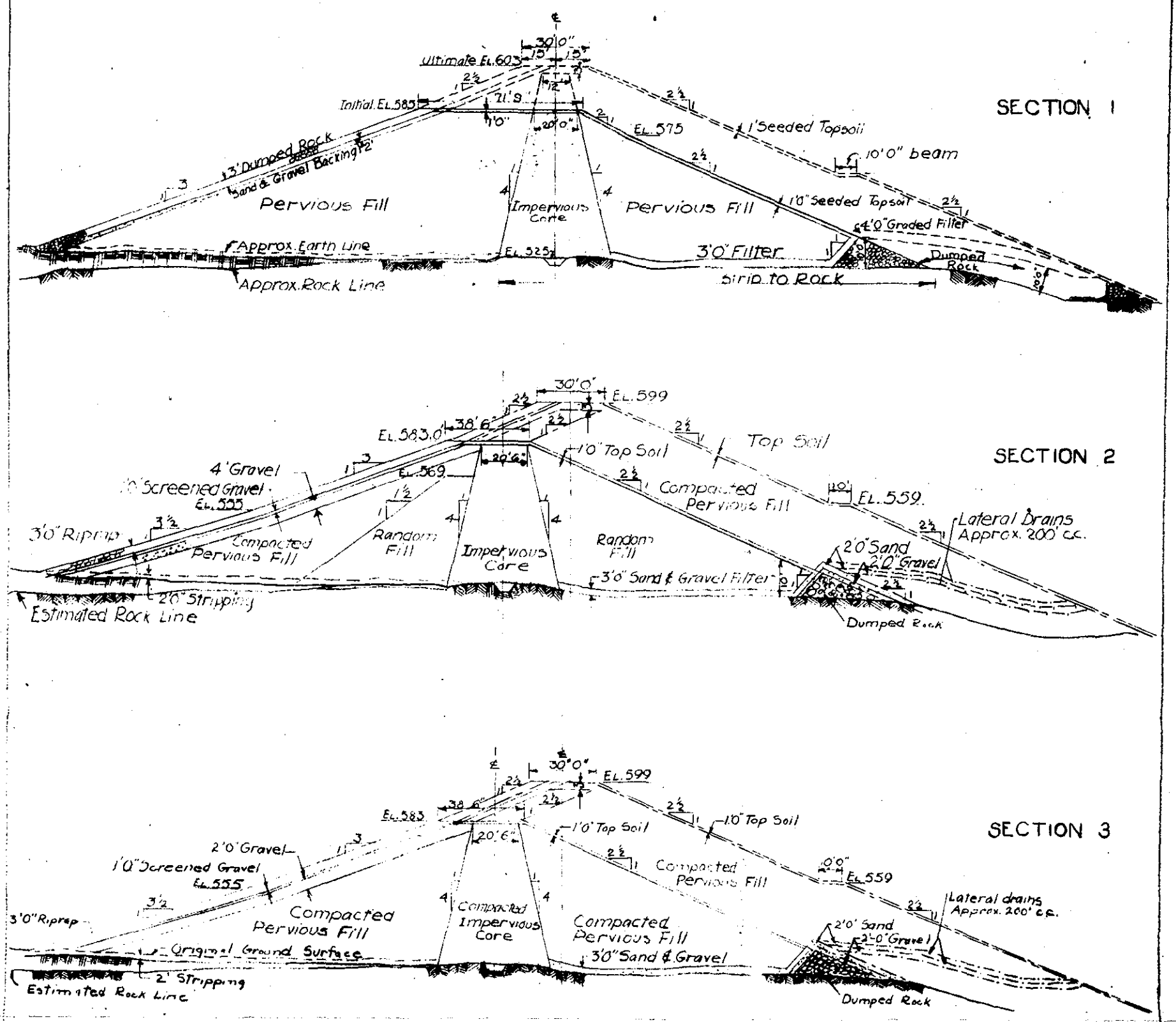
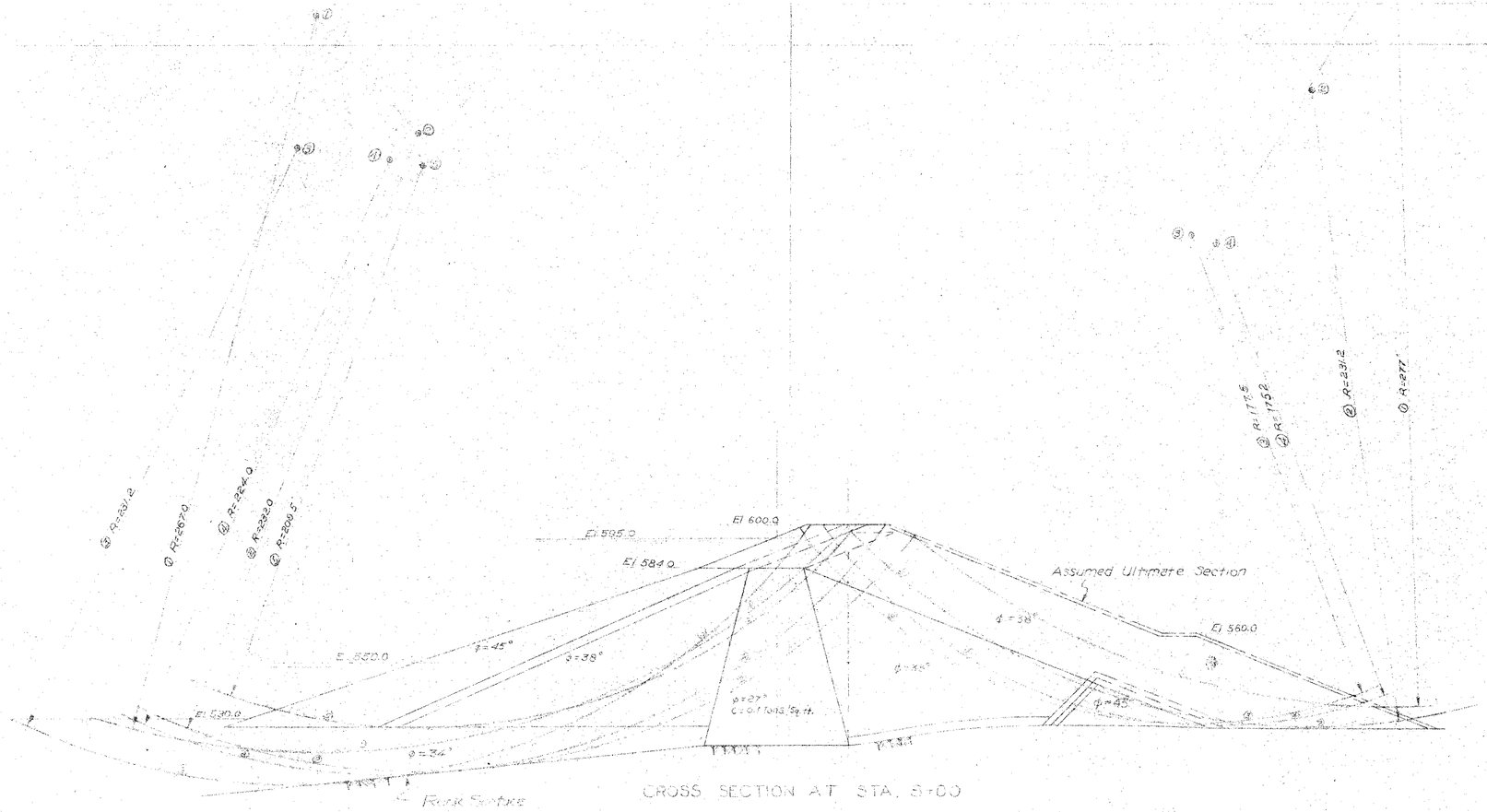

125
120
115
110
105
8
10
15
20
Unit Dry Weight - Lbs./Cu. Ft.
Water Content - Percent Dry Weight
1
2
3
4
Note:
Optimum moisture content curve determined from test data shown by dashed lines. Each dashed line test is a Proctor Test at the number of blows tabulated below:
Test No. | No. of Blows
1 | 10
2 | 25
3 | 40
4 | 60
All tests on sample from pit near T128
OPTIMUM MOISTURE CONTENT CURVE
IMPERVIOUS MATERIAL
DAM EMBANKMENT

FIG. VI - 13



BLACKWATER DAM EMBANKMENT
MAJOR SECTIONS CONSIDERED

FIG. VI - 14



CROSS SECTION AT STA. 5+00

Stability Ratios for Sliding Surfaces Shown

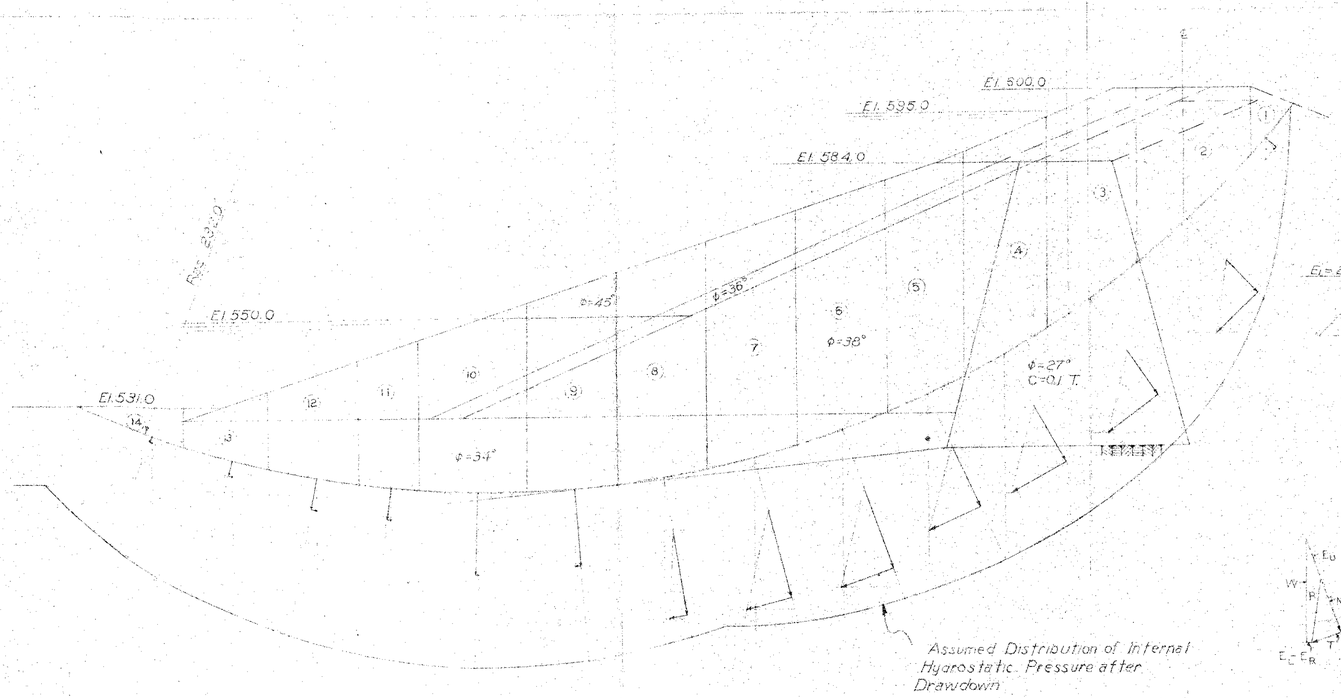
Sliding Surfaces	UPSTREAM DRAWDOWN	Sliding Surfaces	DOWNSTREAM STEADY SEEPAGE
1	1.48	1	2.19
2	1.42	2	2.29
3	1.76	3	3.11
4	1.45	4	2.54
5	1.46	5	
Minimum Stability Ratio		Minimum Stability Ratio	
2	1.42	1	2.19

MERRIMACK VALLEY FLOOD CONTROL
BLACKWATER DAM
BLACKWATER RIVER
RESULTS OF STABILITY ANALYSIS
MAIN EMBANKMENT

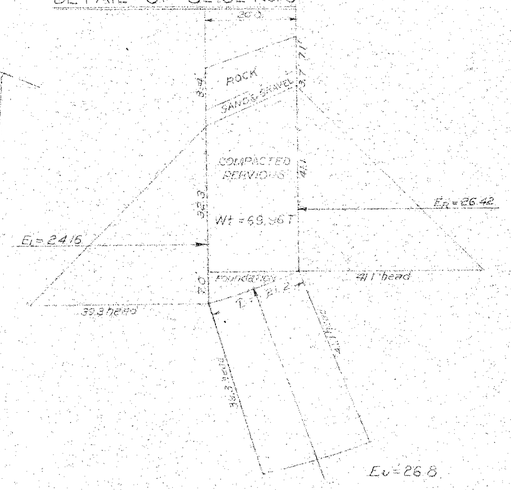
U.S. ENGINEER OFFICE – BOSTON, MASS.

Scale 1" = 30' DEC. 1939

FIG. VI-15



DETAIL OF SLICE NO. 6



GRAPHICAL DETERMINATION OF FORCES

W = TOTAL WGHT OF SECTION
E_U = TOTAL UPLIFT (HYDROSTATIC)
E_L & E_R = HYDROSTATIC PRESSURES ON SIDES
R = RESULTANT OF E_U, W, E_L + E_R
N = NORMAL COMPONENT OF R
T = TANGENTIAL COMPONENT OF R
L = LENGTH OF ARC OF SLIDING SURFACE
c = COHESION IN T/FT²
φ = APPARENT ANGLE OF INTERNAL FRICTION

MAXIMUM SECTION

COMPUTATIONS FOR SLICE NO. 6

1. WEIGHT OF ROCK
   ROCK - DRY = 105 #/FT³ = 0.0525 T/FT³
   8.2 x 20.0 x 0.0525 = 8.61 T
2. WEIGHT OF COMPACTED PERVIOUS
   PERVIOUS - SAT. = 144 #/FT³ = 0.072 T/FT³
   36.8 x 20 x 0.072 = 53.0 T
3. WEIGHT OF SAND AND GRAVEL
   SAND & GRAVEL - DRY = 111 #/FT³ = 0.0555 T/FT³
   3.7 x 20 x 0.0555 = 4.11 T
4. WEIGHT OF PERVIOUS FOUNDATION
   FOUNDATION - DRY = 121 #/FT³ = 0.0605 T/FT³
   7.0 x 20 x ½ x 0.0605 = 4.24 T
4. E_R - HYDROSTATIC PRESSURE ON RIGHT
   41.1² x ½ x 0.0313 = 26.42 T
5. E_L - HYDROSTATIC PRESSURE ON LEFT
   39.3² x ½ x 0.0313 = 24.16 T
6. E_U - HYDROSTATIC UPLIFT PRESSURE
   40.5 x 21.2 x 0.0313 = 26.88 T
7. FROM FORCE POLYGON - N = 38.2 Tons T = 24.8 Tons

SUMMATION FOR ENTIRE SURFACE

$$M.S.R. = \frac{\text{RESISTING FORCES}}{\text{SLIDING FORCES}} = \frac{\Sigma N \tan \Phi + \Sigma L c}{\Sigma T} = \frac{223.97}{158.2} = 1.42$$

MERRIMACK VALLEY FLOOD CONTROL
BLACKWATER DAM
BLACKWATER RIVER

DETAILS OF STABILITY ANALYSIS

DAM EMBANKMENT
UPSTREAM SLOPE - DRAWDOWN CONDITION
U.S. ENGINEER OFFICE - BOSTON, MASS.

SCALE:- 1" = 20' / 1" = 40 T

DEC, 1939

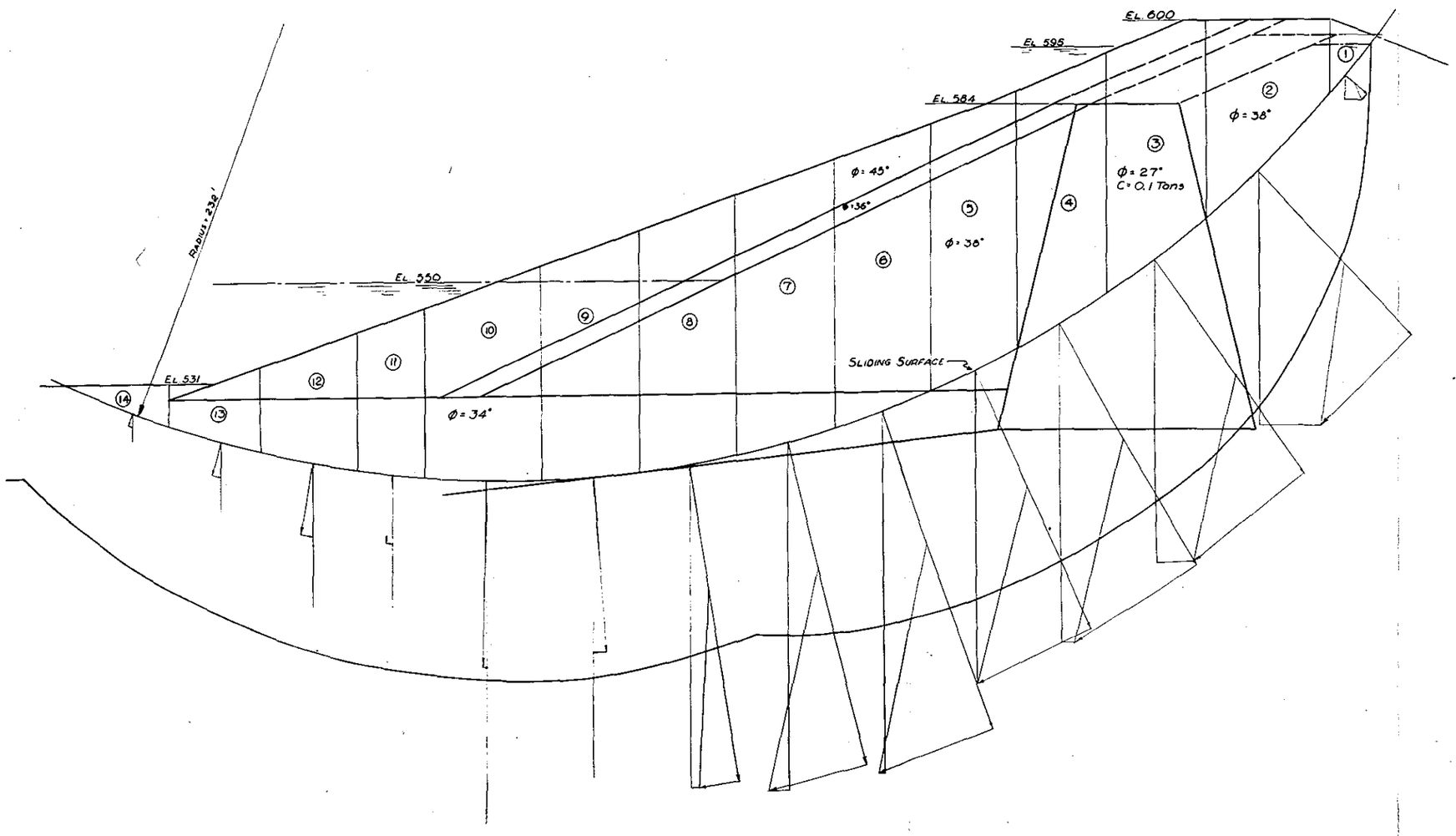
FIG. VI - 16

Section	Compacted Impervious (Wsat=160#/c.f.=0.0700 T/c.f.) Volume	δ	Weight	Compacted Pervious (Wsat=144#/c.f.=0.0722 T/c.f.) Volume	δ	Weight	Sand & Gravel (Wsat=111#/c.f.=0.0533 T/c.f.) Volume	δ	Weight	Rock (Wsat=103#/c.f.=0.0525 T/c.f.) Volume	δ	Weight	Pervious Foundation (Wsat=135#/c.f.=0.0675 T/c.f.) Volume	δ	Weight	Pervious Foundation (Wsat=121#/c.f.=0.0605 T/c.f.) Volume	δ	Weight	Total
1		0.0700		40·9½·679 18·2·½·162	0.0722	4.8					0.0533						0.0605		4.8
2				162		47.1	30·120·36		1.99				32·80·½·1024 (32·80·½·1024)		0.81				49.90
3	36·20·720		50.4				32·20·64		3.55				32·20·640		3.03				52.98
4	40.5·18.8·734		52.78				32·186·6.51		3.02				32·183·981		5.15				61.55
5				412·173·713		51.4	32·173·606		3.30				62·173·1071		5.61				60.37
6				368·20·736		53.0	37·20·740		4.11				82·20·1640		8.61	7·20·½·70		4.24	69.96
7				28·20·560		40.3	36·20·720		3.99				100·20·2120		11.12	98·20·1960		11.86	67.27
8				19·20·380		27.4	32·77·1461		1.37	32·14·448		2.94	(82·20·1640) 238		12.5	14·20·280		16.95	62.28
9				10·20·200		14.4				33·20·66		4.32	67·20·134		7.04	130·29·½·1885 (130·29·½·1885)		12.81	58.19
10				66·12.8·½·427		3.8				32·18.2·582		3.81	32·9·½·14.4		0.75	160·24·½·1920 (160·24·½·1920)		32.9	65.62
11																120·13.5·½·202.5		13.80	26.08
12																95·20·½·1900		12.82	28.31
13																186·6.2·½·576		3.89	13.66
14																			5.44
15																			5.44

Section	Hydrostatic Uplift: Eu = ½(H1+H2)/c	δ	Eu	Lateral Hydrostatic Pressures: ½M²	δ	EL	ER = ½M²	δ	ER	EL − ER
1	92·13·½·597	0.0313	.87	½(148)²	.0095	3.42				3.42
2	228·352·803		35.1	½(320)²	512	16.02	½(148)²	.0095	3.42	12.60
3	358·25·895		28.11	½(388)²	752.7	23.55	½(32)²	512	16.02	7.53
4	403·25·1.867		26.14	½(410)²	840	26.29	½(388)²	752.7	23.55	2.74
5	412·192·792		24.8	½(410)²	840	26.42	½(41)²	840	26.29	0.13
6	403·212·858		26.88	½(503)²	772	24.16	½(410)²	840	26.42	−1.26
7	378·206·780		24.6	½(355.7)²	637	19.94	½(503)²	772	24.16	−4.22
8	36·204·734		22.97	½(377.4)²	695	21.75	½(355.7)²	637	19.94	1.81
9	38.5·20·770		24.10							
10	38·24.4·927		29.01							
11	302·13.6·411		12.86							
12	221·20.5·453		14.18							
13	112·19.2·215		6.73							
14	82·23·½·94.5		2.9							
15										

Φ Degrees	Tan Φ	Normal N	N·Tan Φ	Cohesion C	L	L·C	Tangential +T	Tangential −T	ΣT
38	0.781	38	29.7				29.7		
38	0.781	19.7	15.40				15.3		
27	0.510	240	12.23	0.10	250	25	28.4		
27	0.510	280	14.30	0.10	216	22	28.3		
38	0.781	30.1	23.51				23.6		
24	0.674	38.2	25.74				24.8		
—	—	39.2	26.32				20.7		
—	—	38.9	26.22				8.1		
—	—	34.0	22.92				2.8		
—	—	36.8	24.81					0.8	
—	—	13.3	8.96					1.4	
—	—	14.1	9.50					2.6	
—	—	6.8	4.57					1.9	
—	—	2.4	1.6					0.8	
						47	165.7	7.5	158.2 TOTALS

Forces: Resisting ΣN Tan Φ	Resisting Σ L·C	Driving ΣT	Stability Ratio (ΣN Tan Φ + Σ L·C) / ΣT
2192.5	4.7	158.2	14.2



MERRIMACK VALLEY FLOOD CONTROL
BLACKWATER RIVER
BLACKWATER DAM

DETAILS OF STABILITY ANALYSIS
UPSTREAM SLOPE SAMPLE COMPUTATIONS
MAIN DAM EMBANKMENT

SCALE
1 ½ 0 1 2 3 FEET / TONS

IN SHEETS SHEET NO SCALE 1" = FEET
SCALE 1" = TONS

U. S. ENGINEER OFFICE 1939

FIG. VI-17

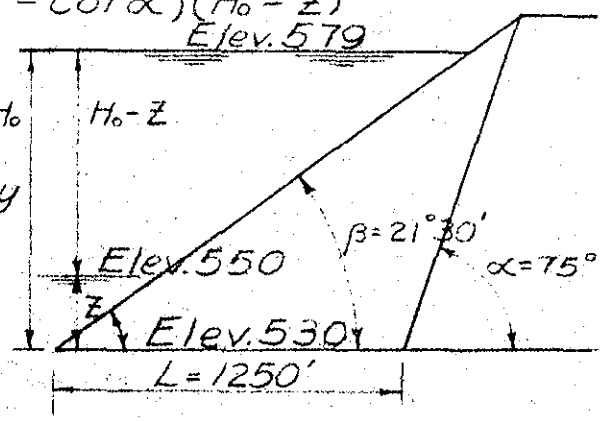
Subject: Blackwater Dam Embankment
Computation: Drainage of Compacted Pervious During Drawdown
Computed by J.C.M. Checked by J.J.F. Date: Dec. 26, 1933

Computation of time required to drain upstream
COMPACTED PERVIOUS FILL SECTION

Formula derived for Design Analysis of
Arkport Dam, Binghamton District:

$$t \cdot \frac{K}{n} \cdot \sin\beta = L \ln \frac{H_0}{Z} - (\cot\beta - \cot\alpha)(H_0 - Z)$$

Where:–
t = time
K = coefficient of permeability
n = porosity
H_0 = initial reservoir depth
Z = final reservoir depth
L, β, α as shown in figure



$$t \frac{0.0118}{0.25} \times 0.367 = 1250 \ln \frac{40}{20} = [2.539 - 0.268]29$$

$$0.0173t = 1250 \times 0.896 - [2.271]29$$

$$= 1120 - 66 = 1054$$

$$t = 1054/0.0173 = 61{,}000 \text{ hours}$$

$$t = 61{,}000/24 = 2540 \text{ Days}$$

FIG VI-18

SEEPAGE PER FOOT OF LENGTH

$q = \bar{k} h \frac{n_F}{n_E}$

$\bar{k} = 1 \times 10^{-7}$ cm/sec $= \frac{60}{30.5} \times 10^{-7}$ ft/min

$h = 49$ feet

$n_F = 9$

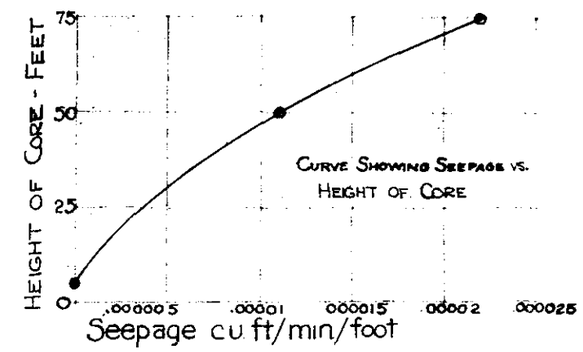
$n_E = 4$

$q = \left(\frac{60}{30.5}\right)(10^{-7})(49)\left(\frac{9}{4}\right) = 2.2 \times 10^{-5}$ cu. ft/min/ft.

Note: $\bar{k}$ = Average coefficient of permeability selected from test data and is equal to $\sqrt{k_{max} \cdot k_{min}}$

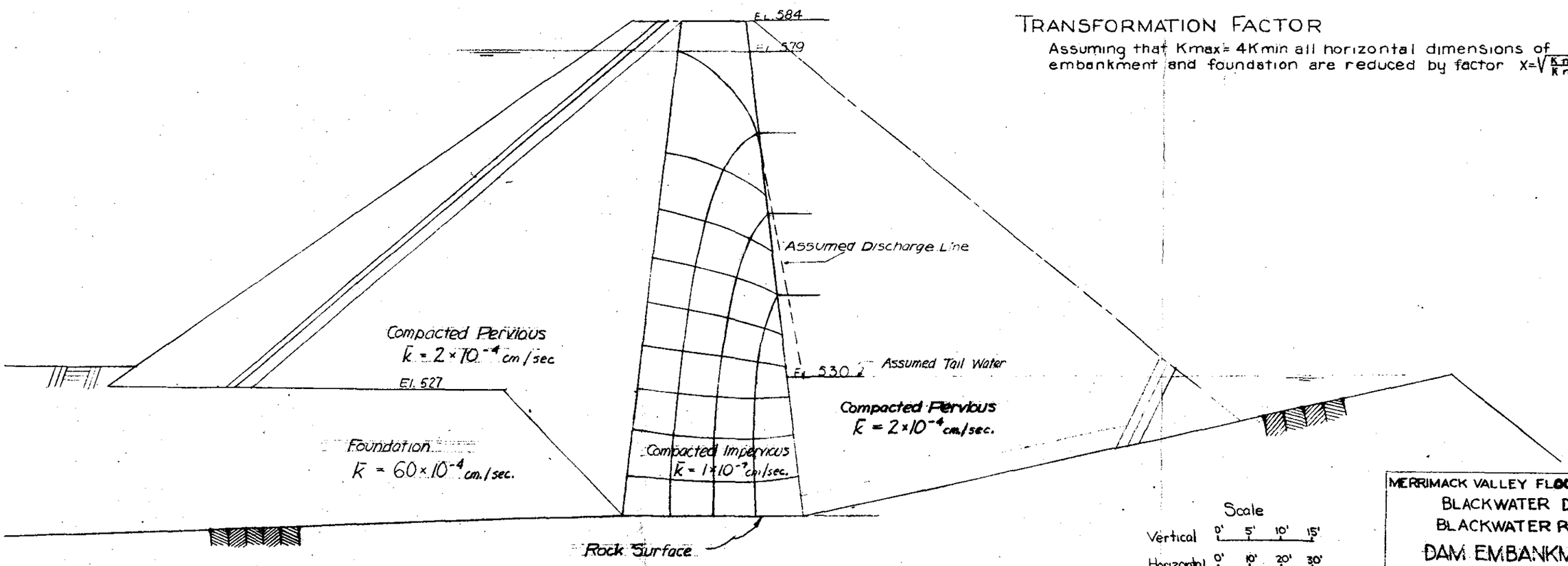
TOTAL SEEPAGE

Station	Length Feet	Avg. Ht. of Core Feet	Seepage Cu.ft/min/ft.	Total Seepage Cu ft./min.
2+00 – 3+00	100	10.	.0000005	.00005
3+00 – 4+00	100	19.	.0000019	.00019
4+00 – 5+00	100	27.5	.0000039	.00039
5+00 – 6+00	100	30.5	.0000047	.00047
6+00 – 7+00	100	38.5	.0000071	.00071
7+00 – 8+00	100	52.0	.0000117	.00117
8+00 – 9+00	100	60.0	.0000150	.00150
9+00 – 10+00	100	55.0	.0000130	.00130
			Total	.00578



TRANSFORMATION FACTOR

Assuming that $K_{max} = 4K_{min}$ all horizontal dimensions of embankment and foundation are reduced by factor $X = \sqrt{\frac{K_{max}}{K_{min}}} = 2$.



FLOW NET FOR TRANSFORMED MAXIMUM SECTION
STA. 8+50

MERRIMACK VALLEY FLOOD CONTROL
BLACKWATER DAM
BLACKWATER RIVER
DAM EMBANKMENT
FLOW NET & SEEPAGE ANALYSIS
U.S. ENGINEER OFFICE-BOSTON, MASS.
DEC. 1939

FIG. VI-19

VII DESIGN OF DODGE ESTATE DIKE

7-01. Characteristics of Site. (a) Description. - The Dodge Estate Dike will be located across a low saddle about 3/4 mile west of the dam site. The dike will be divided into two rolled-earth embankments by a low hill near the middle of the saddle. The walls of the saddle and the slopes of the low hill are moderately steep. (See Sheet 12 of the contract drawings.)

(b) Overburden. - The characteristics of the overburden were determined by the investigations described in Section V. The overburden over the entire site is a moderately compact well graded silty sand till overlying bedrock to a depth of about 50 feet. The range of the grain size distributions of materials sampled from the foundation is shown on Figure VII-1. The values of the characteristics of the overburden shown on Figure VII-2 are considered representative and are based on the similarity of the material to the natural deposit of Borrow Area B and the investigations described in Section V.

7-02. Description of Dike Section. - The details of the dike section are shown on Sheet 12 of the contract drawings. The major part of the section is composed of compacted fill obtained from Borrow Area D. A small cutoff trench is extended into the foundation overburden. A downstream bank run gravel toe is incorporated where the height of the section is greater than 10 feet above the stripped ground surface. The material for the toe will be obtained from Borrow Area C. The maximum height of the dike for the initial development is 18 feet.

7-03. Analysis of Section. (a) Material for Compacted Fill. - The fill material is a well graded silty sand till similar to the pervious material for the dam embankment. The range of grain size gradation of the materials sampled from the deposit in which the borrow area is located is shown on Figure VII-1. The values of the characteristics of the material, as determined by the investigations described in Section V and based on the similarity of this material to the pervious material for the dam embankment, are shown on Figure VII-2. These values are considered representative.

(b) Material for Toe. - The toe material is similar to that used for the sand and gravel drainage blanket for the dam embankment as described in paragraph 6-03(e).

(c) Stability and Settlement. - Because of the low height of the embankment and the satisfactory characteristics of the materials involved, no mathematical stability or settlement analysis of the dike and dike foundation was made. From experience, it is concluded that the embankment will satisfy criteria (1), (2) and (4) stated in paragraph 6-01.

(d) Seepage. - To insure that seepage through the dike and dike foundation is satisfactorily controlled, the downstream toe of pervious material is incorporated. The material in the toe will satisfactorily perform as a filter material for the fill and foundation materials since the gradations and permeabilities are similar to those described in paragraphs 6-03(b) and 6-03(e) for which the conclusions in paragraph 6-09(b) were made. A cutoff into the foundation of random fill material is provided to insure a uniform and compact section in the upper few feet of the foundation. From a study of the precautionary measures adopted, the characteristics of the materials and the size of the embankment, it is concluded that criterion (3) stated in paragraph 6-01 is satisfied.

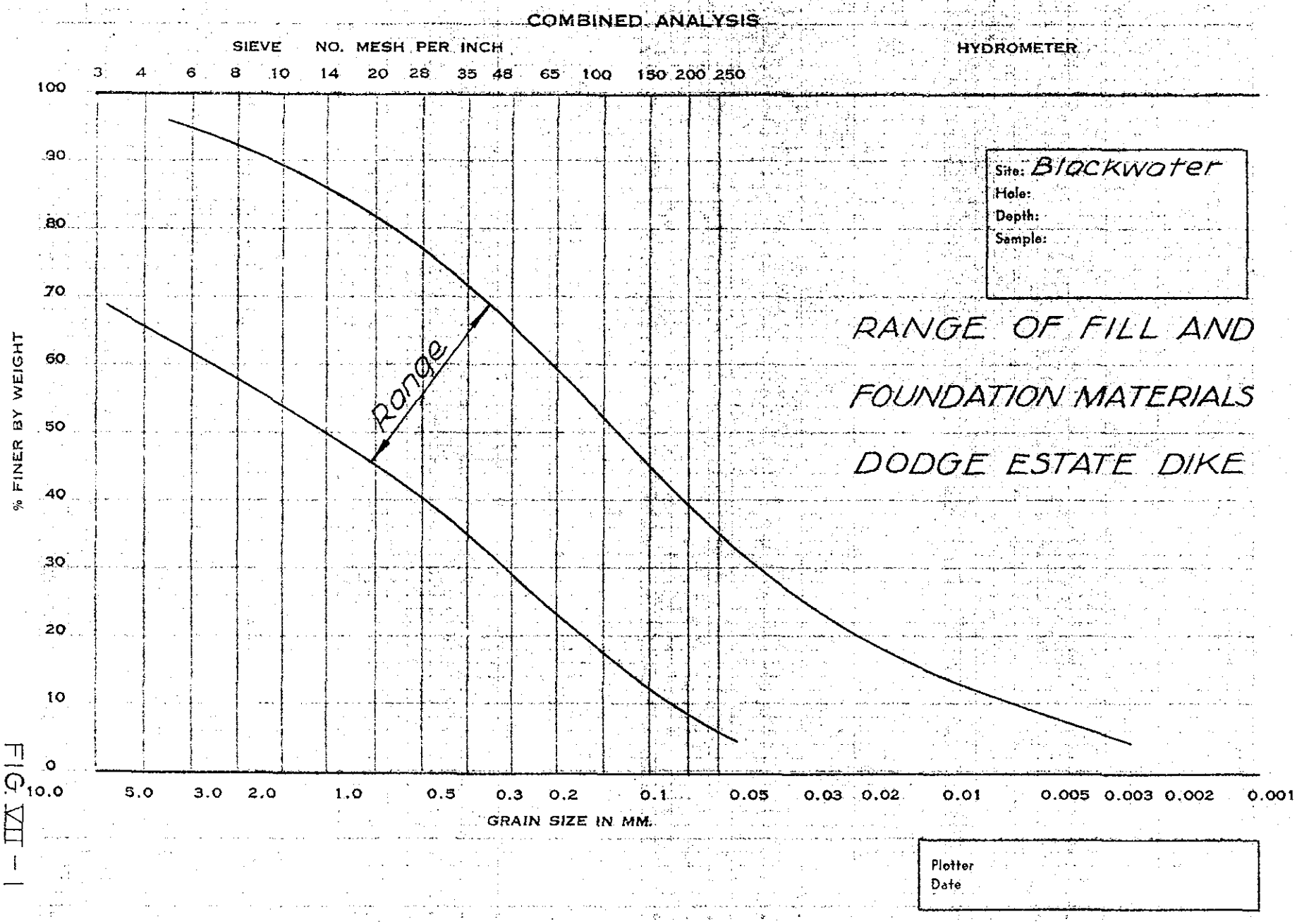

COMBINED ANALYSIS
SIEVE NO. MESH PER INCH
HYDROMETER
3 4 6 8 10 14 20 28 35 48 65 100 150 200 250
% FINER BY WEIGHT
100 90 80 70 60 50 40 30 20 10 0
10.0 5.0 3.0 2.0 1.0 0.5 0.3 0.2 0.1 0.05 0.03 0.02 0.01 0.005 0.003 0.002 0.001
GRAIN SIZE IN MM.
Range
Site: Blackwater
Hole:
Depth:
Sample:
RANGE OF FILL AND
FOUNDATION MATERIALS
DODGE ESTATE DIKE
Plotter
Date

FIG. XIII - 1

SUMMARY OF SOIL TEST DATA - DODGE ESTATE DIKE

	Foundation Material Natural Characteristics	Embankment Materials - Compacted Characteristics		
		Fill	Gravel	Sand
Dry Weight Binder Material lbs. per cu.ft.	119	120	111	111
Total Wet Weight lbs.per cu.ft.	132	142	---	---
Optimum Water Content Percent Dry Weight	---	10	---	---
Void Ratio, e Binder Material	0.42	0.40	0.50	0.50
Max. Void Ratio, e_{max}	0.60 Av.	0.60 Av.	---	---
Min. Void Ratio, e_{min}	0.32 Av.	0.32 Av.	---	---
Degree of Compaction Percent	64	71	---	---
Coef. of Permeability, k 10^{-4} cm.per sec.	1	1 Av.	---	100
Angle of Internal Friction Ø-Degrees	36	38	36	36
Cohesion C Tons per sq. ft.	0	0	0	0
Specific Gravity, s	2.70	2.70	---	---

FIG. VII-2

VIII DESIGN OF LITTLE HILL DIKE

8-01. General. The general procedure of design for the Little Hill Dike followed that for the dam embankment and dam embankment foundation as described in Section VI. The dike is designed to satisfy the criteria stated in paragraph 6-01 and to permit future heightening for the ultimate development.

8-02. Geological Features and Characteristics of Foundation Materials. (a) General. - The geological features and characteristics of the foundation materials, with particular emphasis upon those characteristics influencing the proper design of the dike and dike foundation, are summarized in the following paragraphs.

(b) Description of Site. - The Little Hill Dike will be located in a low saddle between Bean and Little Hills. The dike will be divided into three parts by low rock knolls rising above the saddle floor. The walls of the saddle have moderately steep slopes. Bedrock is generally shallow over the entire site with the exception of under the east abutment. Bedrock from the saddle bottom near the east abutment to the west abutment in general follows the topography, outcropping on the low knolls, and is buried to a depth of about 5 to 15 feet in the region between the knolls. (See Sheet 11 of the contract drawings.)

(c) Bedrock. - The bedrock over the site is a gneissic granite. The classification of the bedrock at each drill hole is shown on Sheet 4 of the contract drawings. The rock, except the top few feet, is structurally sound, but contains seams and joint cracks. Some of the joints are open, as disclosed by the pressure pumping test data at drill hole D26. The loss of water under a pressure of 37 pounds per square inch exceeded 8 gallons per minute in a section 2 feet thick. No open joints were crossed by drill hole D22, as disclosed by the pressure pumping data.

(d) Overburden. - (1) The overburden in the saddle floor and on the west abutment is extremely variable, consisting of a deposit of silty sand to gravelly till with pockets of numerous large and small boulders. In the east abutment and in the saddle floor where the rock dips downward under the abutment, the overburden is an impervious, very silty sand and gravel till. This till is blanketed to a depth of about 6 feet with a slightly silty sand till. The impervious till may or may not be sealed to the bedrock below the saddle floor. Information from drill hole D27 discloses that there may be a thin layer of pervious gravel between the impervious till and bedrock.

(2) The values of the characteristics of the overburden in the saddle floor shown on Figure VIII-1 based on the investigations described in Section V are considered representative for design purposes. The characteristics of the impervious till and blanket till in the east abutment may be considered the same as those for the compacted impervious fill and compacted pervious fill respectively,

shown on Figure VIII-1. The characteristics of probable gravel between the till and bedrock were not determined, but it is believed the gravel is very pervious. Typical grain size curves of foundation and west abutment samples taken during the exploration are shown on Figure VIII-2.

8-03. Availability and Characteristics of Dike Materials. (a) General. - Most of the material for the dike will be obtained from structure excavation and from Borrow Areas H-1 and H-2. The borrow areas contain the most economical materials in the vicinity of the site suitable for a safe design. The following paragraphs describe those characteristics of the material in each borrow area pertinent to the design of the dike and its foundation.

(b) Pervious Material. - The material for the compacted pervious fill section of the dike is variable, ranging from well graded silty sand to sandy gravel of which less than 25 percent by weight will pass the No. 200 mesh sieve. The material will be obtained from a blanket deposit varying from 3 to 15 feet in depth in Borrow Area H-1 and from structure excavation. The material in nature contains a few boulders larger than 6 inches in size which will be removed prior to compaction. Typical grain size curves of the material are shown on Figure VIII-3. A critical void ratio test was made on a sample which was considered to have the least favorable critical void ratio. The results of the test and a grain size curve of the sample tested are shown on Figures VIII-4 and VIII-5, respectively. The results of the consolidation test are shown on Figure VIII-6. The results of critical void ratio and consolidation tests described in paragraph 6-08(b) for the pervious material for the dam embankment are considered representative for the major part of the material for the compacted pervious section of the dike. Since the material is exceedingly variable, no representative permeability-void ratio curve may be drawn. The characteristics used for design purposes are shown on Figure VIII-1.

(c) Impervious Material. - The material for the compacted impervious fill section of the dike will be obtained from Borrow Area H-1. The impervious material lies directly below the pervious material described in subparagraph (b) and extends in general to bedrock. This material is a compact, very silty, sand and gravel till containing a small percentage of clay sizes. The deposit contains a few stones and boulders over the 6-inch size which will be removed before compaction. The range of grain size gradation of material with stones larger than 6 inches removed is shown on Figure VIII-7. When compacted, the material will have, in general, the values of the characteristics tabulated on Figure VIII-1 which are used for design purposes. The results of the shearing strength tests and a grain size curve of the sample tested are shown on Figures VIII-8 and VIII-5. The optimum moisture content of the binder material (material passing the 1/4-inch sieve) for compaction to any unit weight may be obtained from the curve on Figure VIII-9. The laboratory tests indicate that the material will not expand upon saturation. It is believed, based upon the similarity of this material to the impervious material for the dam embankment, that the binder material will be compacted to a unit dry weight of binder material of 120 pounds per cubic foot.

(d) Sand and Gravel for Backing, Filters and Drainage Toe. - The sand and gravel for backing under the upstream rock blanket and downstream drainage toe and filter materials for the drainage blanket will be obtained from Borrow Area H-2. These materials will be bank-run sand and gravel containing at least 25 percent by dry weight passing the 6-inch sieve and of the material passing the 1/4-inch sieve, not more than 5 percent will pass the No. 200 mesh sieve.

(e) Gravel for Drainage Blanket. - The gravel for the drainage blanket, if required, at the transition zone downstream from the center line, will be obtained by processing material from Borrow Area H-2 or from Borrow Area C. The gravel will be graded according to the specifications for concrete coarse aggregate with a maximum size of 2 inches.

(f) Rock. - Rock and riprap will be obtained from areas selected by the contractor. Suitable rock can be obtained from various outcrops in the vicinity, from numerous stone walls, and by culling in the borrow areas. In general, the bedrock is a sound gneissic granite and the stones in walls and borrow areas are of variable igneous and metamorphic origin. No rock shall be used which is not satisfactorily sound. Rock suitable for the upstream blanket will not pass a 6-inch sieve and the maximum dimension will be limited to 2 feet.

8-04. Dike Sections. (a) Sections Considered. - The development of the dike section progressed as information in regard to the foundation and borrow materials became available. It was desired to use the most economical materials and to obtain a stable section. The dimensions required for heightening of the dike for the ultimate development were also considered. Prior to the selection of the final section, the two shown on Figure VIII-10 were considered.

(1) Section No. 1 was submitted at the Design Conference, November 10, 1939. The section was composed of two materials; impervious material as described in paragraph 8-03(c) and pervious material to be obtained from the deposit in the flats upstream from the center line. The material in the deposit in the flats is variable, ranging from fine to medium sand to slightly gravelly fine to coarse sand. It was not planned to use the pervious material described in paragraph 8-03(d) since the extent of the deposit was not known. The top width was 25 feet to permit future heightening of the dike.

(2) At the Design Conference, November 10, 1939, it was decided not to use the pervious material from the deposit in the flat due to its lack of gradation, but to make a homogeneous embankment of the impervious material. The section with the recommended changes is shown as Section No. 2.

(b) Selected Section. - After it was determined by exploration that a considerable deposit of well graded pervious material as described in paragraph 8-03(b) overlies the desired impervious material in Borrow Area H-2, the section shown on Sheet 11 of the contract drawings was selected. The dike may be heightened by placing material on the downstream face, not requiring the disturbance of the upstream rock fill. The essential features of the dike will be as described in the following paragraphs (for details see Sheet 11 of the contract drawings).

(c) Impervious Features. - The dike will contain a compacted impervious fill section of material described in paragraph 8-03(c). The compacted impervious fill will be sealed to bedrock between station 0+40 and approximately station 11+30 since the overburden is shallow. From approximately station 11+30 to station 14+00, the impervious fill will be sealed to the impervious abutment material described in paragraph 8-02(d). It is planned to grout the bedrock to depths shown on Sheet 11 of the contract drawings but the actual depths of grouting will be determined in the field.

(d) Pervious Features. - The upstream face of the compacted impervious fill section will be blanketed with 6 feet of pervious material consisting of sand and gravel and rock as described in paragraphs 8-03(d) and (f), respectively. At the base of the downstream portion of the dike, there will be incorporated a section of pervious material described in paragraph 8-03(b). This compacted pervious fill section will lower the line of seepage in the downstream portion and utilize available borrow material. The foundation material downstream from the cutoff trench is, in general, quite pervious.

(e) Drainage Features. - (1) The drainage features consist of a downstream bank-run sand and gravel toe and, if necessary, a drainage blanket between approximately stations 11+00 and 12+00. The downstream sand and gravel toe to be constructed of material described in paragraph 8-03(d) will drain the compacted pervious fill section of the dike and the foundation material. Any seepage through the bedrock will pass into the foundation material either under the dike or downstream from the toe. The toe extends into the foundation material to relieve any seepage pressure at that critical point.

(2) As discussed in paragraph 8-02(d), it is not known if a gravel layer lies between the bedrock and the impervious till where bedrock dips. During construction, exploration will be made to determine if the till properly seals to rock. If a layer of gravel exists between the till and bedrock, a drainage blanket as shown on Sheet 11 of the contract drawings will be incorporated. The drainage blanket will properly control the seepage through the foundation. It will have a width of 100 feet and be located over the transition point. It will be constructed of materials described in paragraphs 8-03(d) and (e).

(f) Slope Protection. - The slopes of the dike will be protected against erosion by the rock on the upstream slope and by seeded topsoil on the downstream slope. The downstream slope of the gravel toe will be protected by riprap. No provisions are made to protect the abutment slopes since natural conditions are deemed satisfactory.

8-05. Economy of Construction. The dike will be composed of borrow material adjacent to the site and from the structure excavations. If screened gravel is required for the downstream drainage blanket, it will be obtained from Borrow Area H-1 or from Borrow Area C, depending upon the yardage required and the relative cost. Since rock must be quarried or gathered in the vicinity, the upstream rock fill section has been reduced to the minimum deemed necessary.

8-06. Statical Analysis. The upstream and downstream slopes were analyzed by the method and procedure used for the dam embankment (see paragraph 6-07) using the values given on Figure VIII-1. In the upstream section, it was assumed that only the sand and gravel backing and rock fill drain freely during drawdown. The section analyzed was the assumed maximum section of the ultimate development. The results of the analyses are shown on Figure VIII-11. The minimum stability ratios for the upstream and downstream slopes, including the foundation, are 1.43 and 1.97, respectively. Analyses were not made for either abutment since no problems exist.

8-07. Flow Failure Analysis. (a) Due to the character of the dike and foundation materials and the drainage conditions, the only portion which warrants study of flow failure is the compacted pervious fill section. Theoretically, as shown by the flow net discussed in paragraph 8-08, this section will not become saturated, but it is believed that very adverse weather conditions might produce saturation. If the material in this section is sufficiently compacted, no flow failure can occur. (See paragraph 6-08(a).)

(b) A definite critical void ratio cannot be determined for all the material to be placed since it is quite variable as described in paragraph 8-03(b). According to the results shown on Figures VIII-4 and VI-8, which are considered as representative of the range of materials, it is believed that the critical void ratio under pressure will range between 0.37 and 0.50. Assuming the maximum pressure at the base of the dike for the section for ultimate development to be 3.6 tons per square foot, the consolidation curves on Figures VI-10 and VIII-6 show that materials compacted to a void ratio of 0.35 and 0.47 will produce the above stated critical void ratio values. Using the maximum and minimum void ratios shown on Figures VI-10 and VIII-6, the degrees of compaction corresponding to void ratios of 0.37 and 0.50 of the two materials are 77 and 70 percent, respectively.

(c) It is concluded that if the material is compacted to a degree of compaction greater than 77 percent, no flow failure can occur. Experience with similar cohesionless materials indicates that the material can be economically compacted to a degree of compaction of 80 which will be used for a control criterion.

8-08. Seepage Analysis. (a) Seepage. - The seepage through the dike was analyzed similarly to that for the dam embankment (see paragraph 6-09) using the values given on Figure VIII-1. The flow nets for the maximum designed section of the initial development sealed to bedrock and to impervious till are shown on Figure VIII-12. The flow nets indicate that the seepage pressures will be negligible and that for the conditions assumed, piping cannot occur. Any seepage that might develop through the bedrock where the compacted impervious fill section is sealed to rock will not alter the flow net since it will discharge into the foundation overburden under the embankment or downstream from the toe. No analysis was made at the transition zone from bedrock to till since the conditions are not known. It is believed that the drainage blanket, if incorporated, will control seepage for any conditions. The total seepage through the dike and foundation will depend upon the

conditions at the foundation transition zone. Assuming till to be sealed to bedrock and bedrock impervious, the total estimated seepage for full reservoir is 0.03 cubic foot per minute. No seepage problem occurs in the abutments.

(b) Filters. - All dike and foundation materials, except the sand and gravel backing placed on impervious material on the upstream slope, are similar in gradation to those of the dam embankment and, therefore, the filters as designed are considered satisfactory similarly as described in paragraph 6-09(b). The results of the filter test using impervious material and sand and gravel backing, as described in paragraph 5-04(e), indicate that no material will be transported with a hydraulic gradient as high as 30. Since such a gradient is improbable, it is concluded that the sand and gravel backing will serve satisfactorily as filter material next to the compacted impervious fill section.

8-09. Miscellaneous Analyses. Expansion tests, as described in paragraph 5-04(d), indicate that no expansion will occur in the compacted impervious fill section due to an increase in moisture content of the material. Settlement analyses will be made after the exact construction program is determined. No provisions are made in the design for settlement of the embankment after construction since it is believed that the settlement will be inappreciable. If appreciable settlement does occur after construction, the embankment will be heightened as a maintenance operation.

SUMMARY OF SOIL TEST DATA -LITTLE HILL DIKE

	Foundation Materials Natural Characteristics			Embankment Materials Compacted Characteristics				
	Saddle Floor and Rt.Abut. Variable Till	Left Abut. Slightly Silty Sand Till	Left Abut. Very Silty Sand and Gravel Till	Impervious	Pervious	Sand Filter	Gravel Filter	Sand and Gravel Backing
Dry Weight Binder Material lbs. per cu.ft.	---	119	---	120	122	---	---	---
Total Wet Weight lbs. per cu.ft.	121	136	136	137	139	111	111	111
Optimum Water Content percent dry weight	---	---	---	12	10	---	---	---
Void Ratio, e Binder Material	---	0.42	---	0.40	0.39	0.50	0.50	0.50
Max. Void Ratio, e_{max}	---	0.64	---	---	0.64	---	---	---
Min. Void Ratio, e_{min}	---	0.33	---	---	0.33	---	---	---
Degree of Compaction Percent	---	71	---	---	81	---	---	---
Coef. of Permeability, k 10^{-4} cm.per sec.	1	1	0.005	0.005	10	100	---	100
Angle of Internal Friction, Ø-Degrees	34	35	34	34	35	36	36	36
Cohesion, C tons per sq. ft.	0	0	0	0.10	0	0	0	0
Specific Gravity, s	---	2.71	2.72	2.72	2.71	---	---	---

FIG. VIII-1

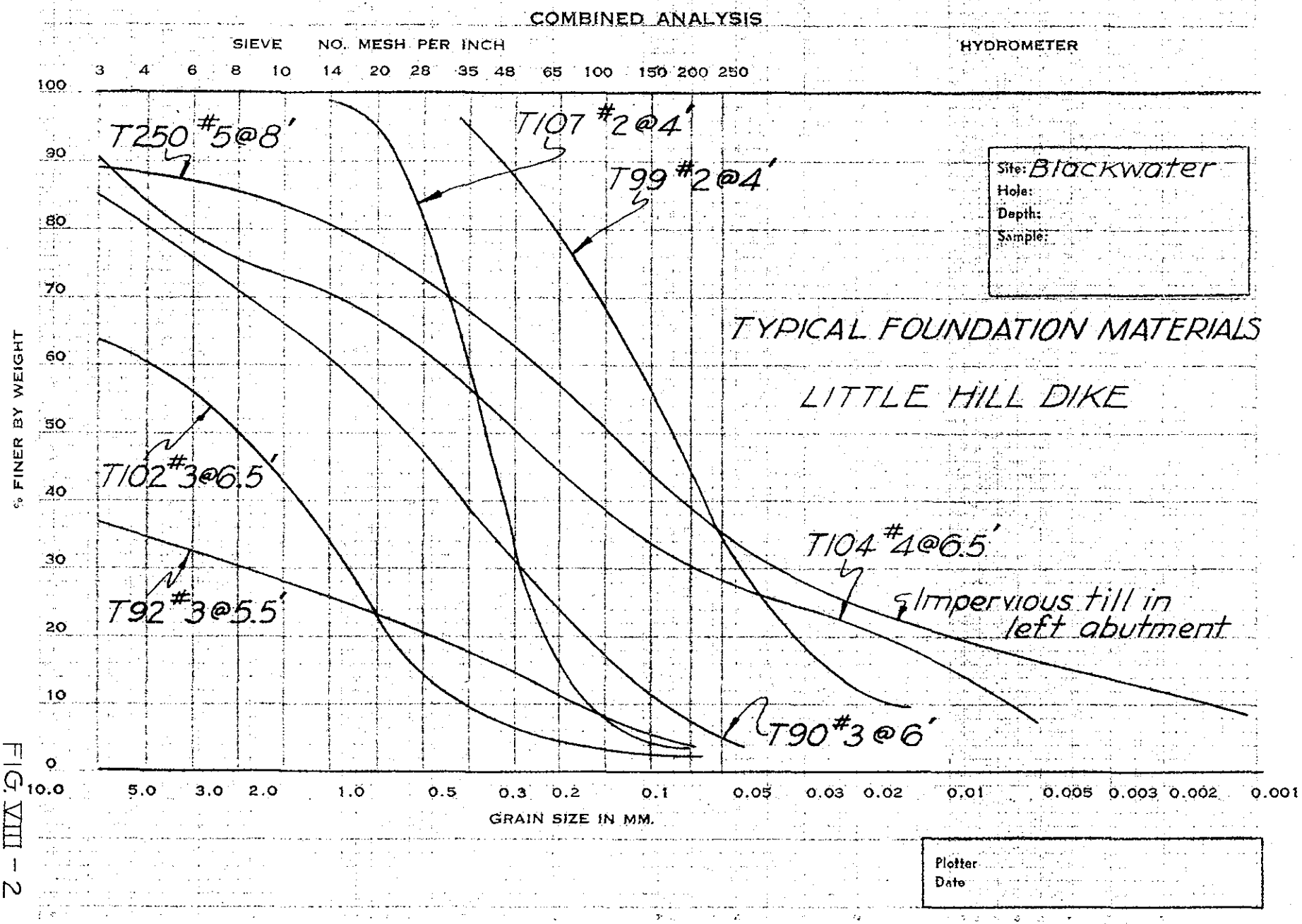

COMBINED ANALYSIS
SIEVE NO. MESH PER INCH
HYDROMETER
3 4 6 8 10 14 20 28 35 48 65 100 150 200 250
% FINER BY WEIGHT
100 90 80 70 60 50 40 30 20 10 0
10.0 5.0 3.0 2.0 1.0 0.5 0.3 0.2 0.1 0.05 0.03 0.02 0.01 0.005 0.003 0.002 0.001
GRAIN SIZE IN MM.
T250 #5@8'
T107 #2@4'
T99 #2@4'
T102 #3@6.5'
T92 #3@5.5'
T104 #4@6.5'
Impervious till in left abutment
T90 #3@6'
Site: Blackwater
Hole:
Depth:
Sample:
TYPICAL FOUNDATION MATERIALS
LITTLE HILL DIKE
Plotter
Date

FIG. VIII - 2

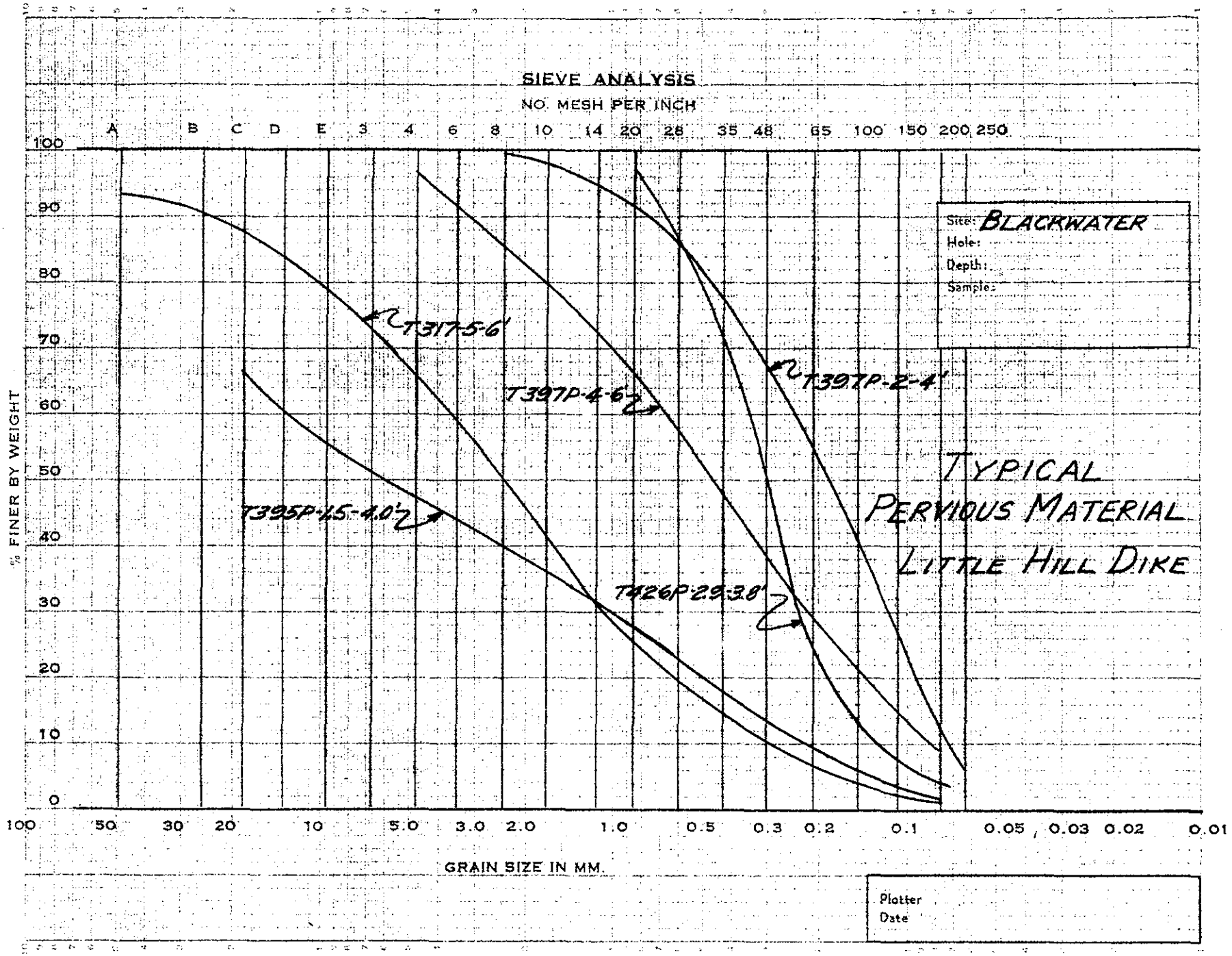
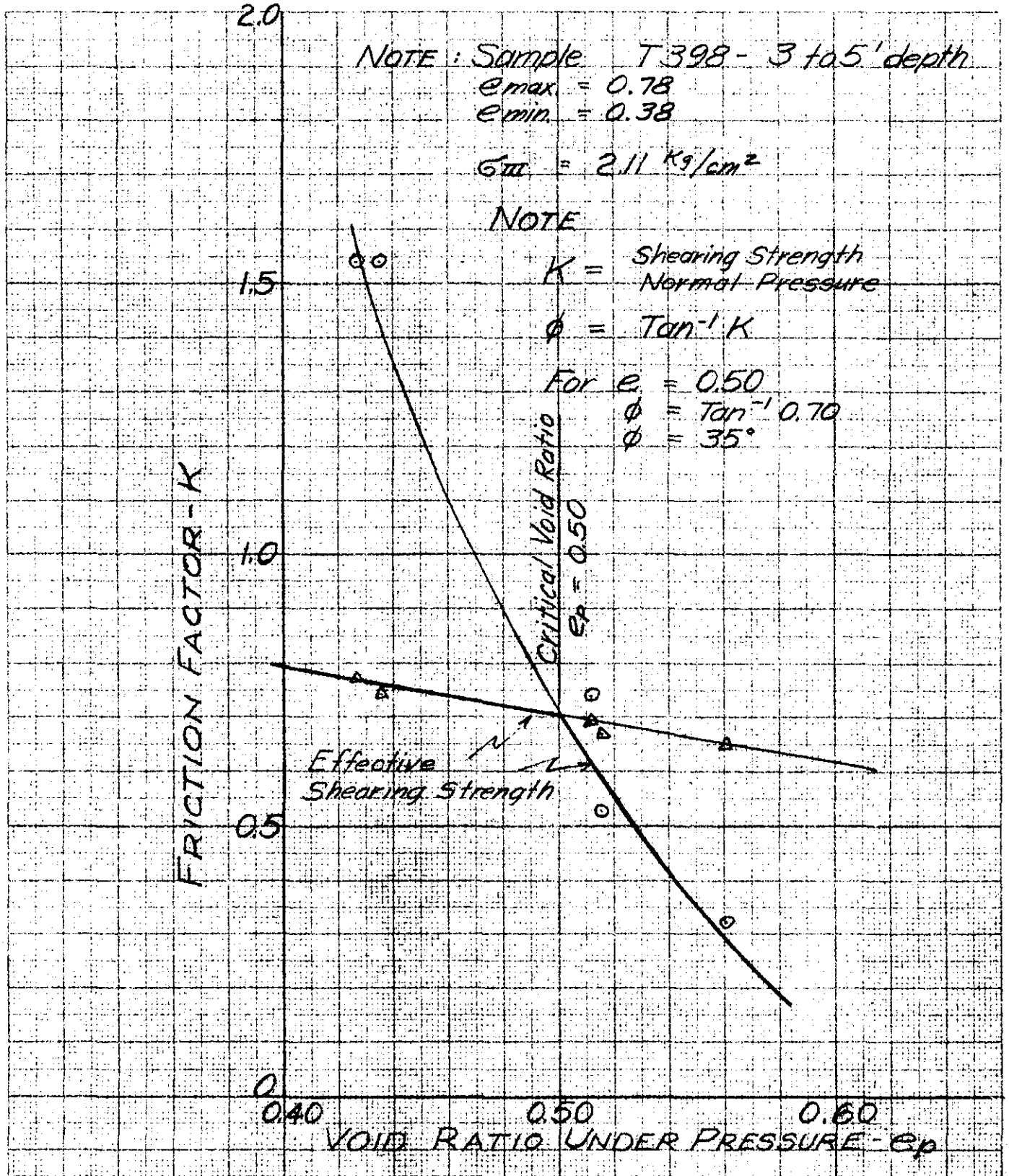

SIEVE ANALYSIS
NO. MESH PER INCH
A B C D E 3 4 6 8 10 14 20 28 35 48 65 100 150 200 250
% FINER BY WEIGHT
100 90 80 70 60 50 40 30 20 10 0
100 50 30 20 10 5.0 3.0 2.0 1.0 0.5 0.3 0.2 0.1 0.05 0.03 0.02 0.01
GRAIN SIZE IN MM.
T317-5-6'
T397P-4-6'
T397P-2-4'
T395P-1.5-4.0'
T426P-2.9-3.8'
Site: BLACKWATER
Hole:
Depth:
Sample:
TYPICAL
PERVIOUS MATERIAL
LITTLE HILL DIKE
Plotter
Date

FIG. VIII - 3



CRITICAL VOID RATIO AND SHEARING STRENGTH TEST RESULTS
PERVIOUS MATERIAL
LITTLE HILL DIKE

FIG. VIII-4

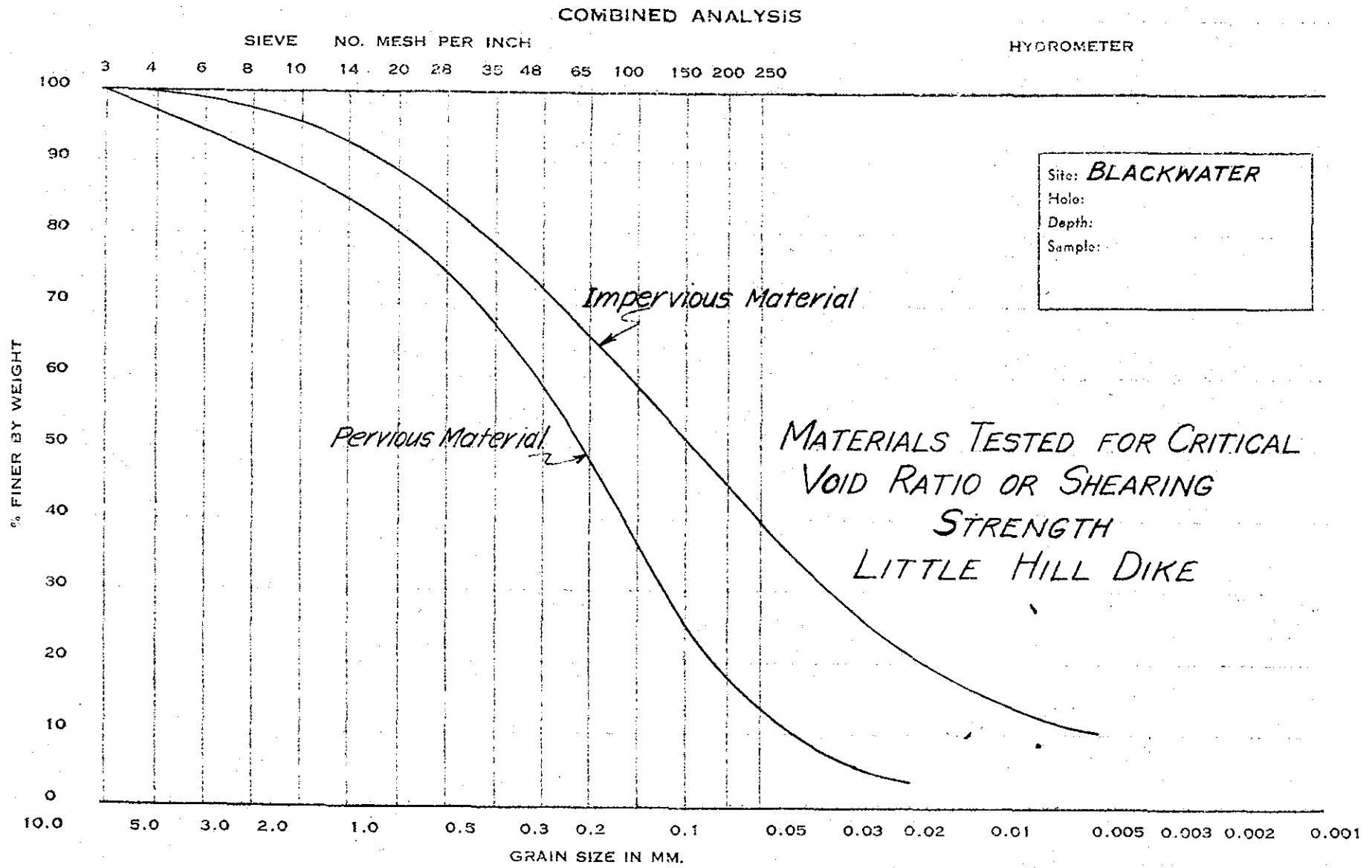
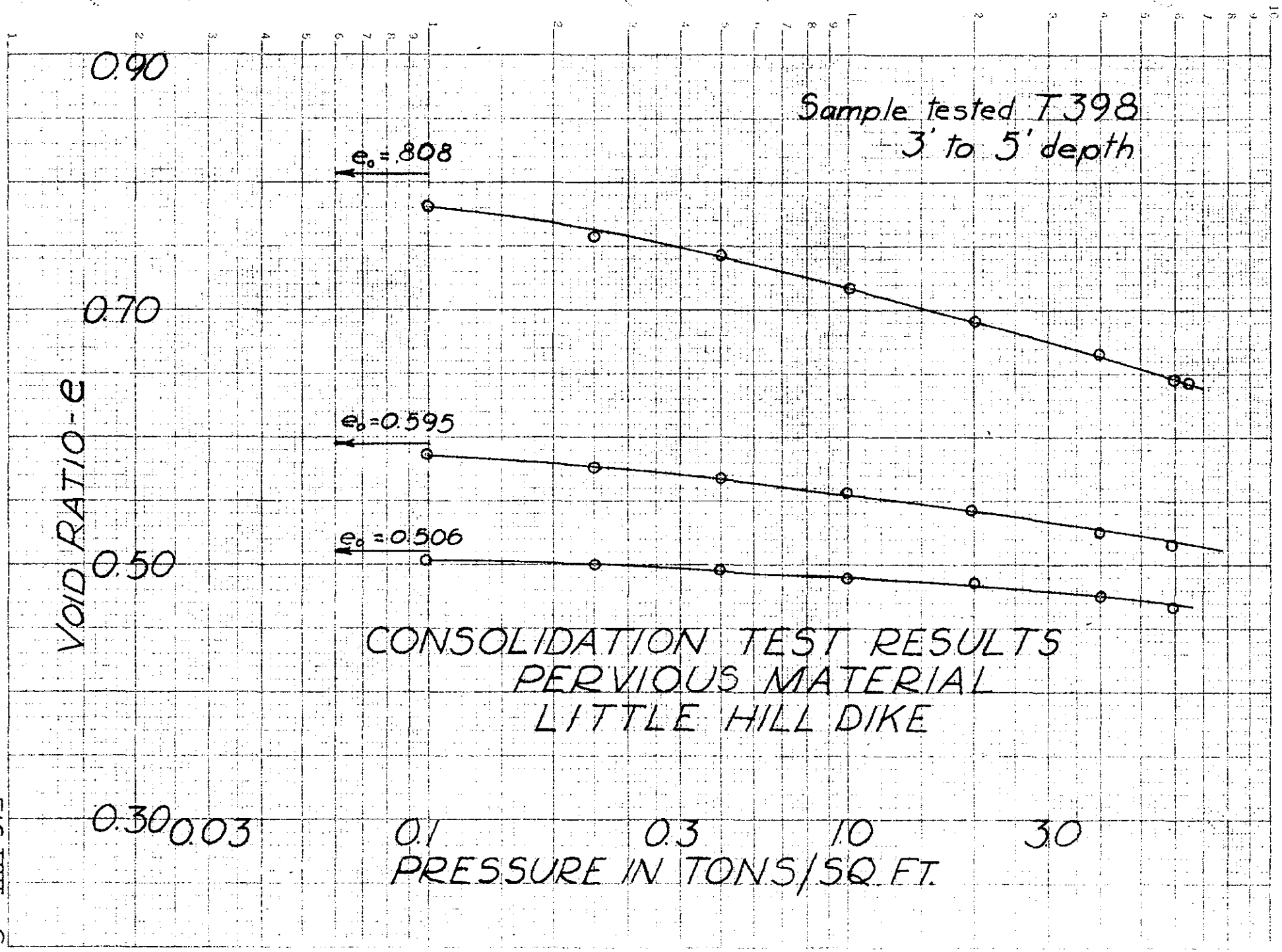

COMBINED ANALYSIS
SIEVE NO. MESH PER INCH
HYDROMETER
3 4 6 8 10 14 20 28 35 48 65 100 150 200 250
% FINER BY WEIGHT
100 90 80 70 60 50 40 30 20 10 0
10.0 5.0 3.0 2.0 1.0 0.5 0.3 0.2 0.1 0.05 0.03 0.02 0.01 0.005 0.003 0.002 0.001
GRAIN SIZE IN MM.
Impervious Material
Pervious Material
Site: BLACKWATER
Hole:
Depth:
Sample:
MATERIALS TESTED FOR CRITICAL VOID RATIO OR SHEARING STRENGTH
LITTLE HILL DIKE
Plotter GPC
Date 1/3/40

FIG. XIII - 5



CONSOLIDATION TEST RESULTS
PERVIOUS MATERIAL
LITTLE HILL DIKE

FIG. VIII - 6

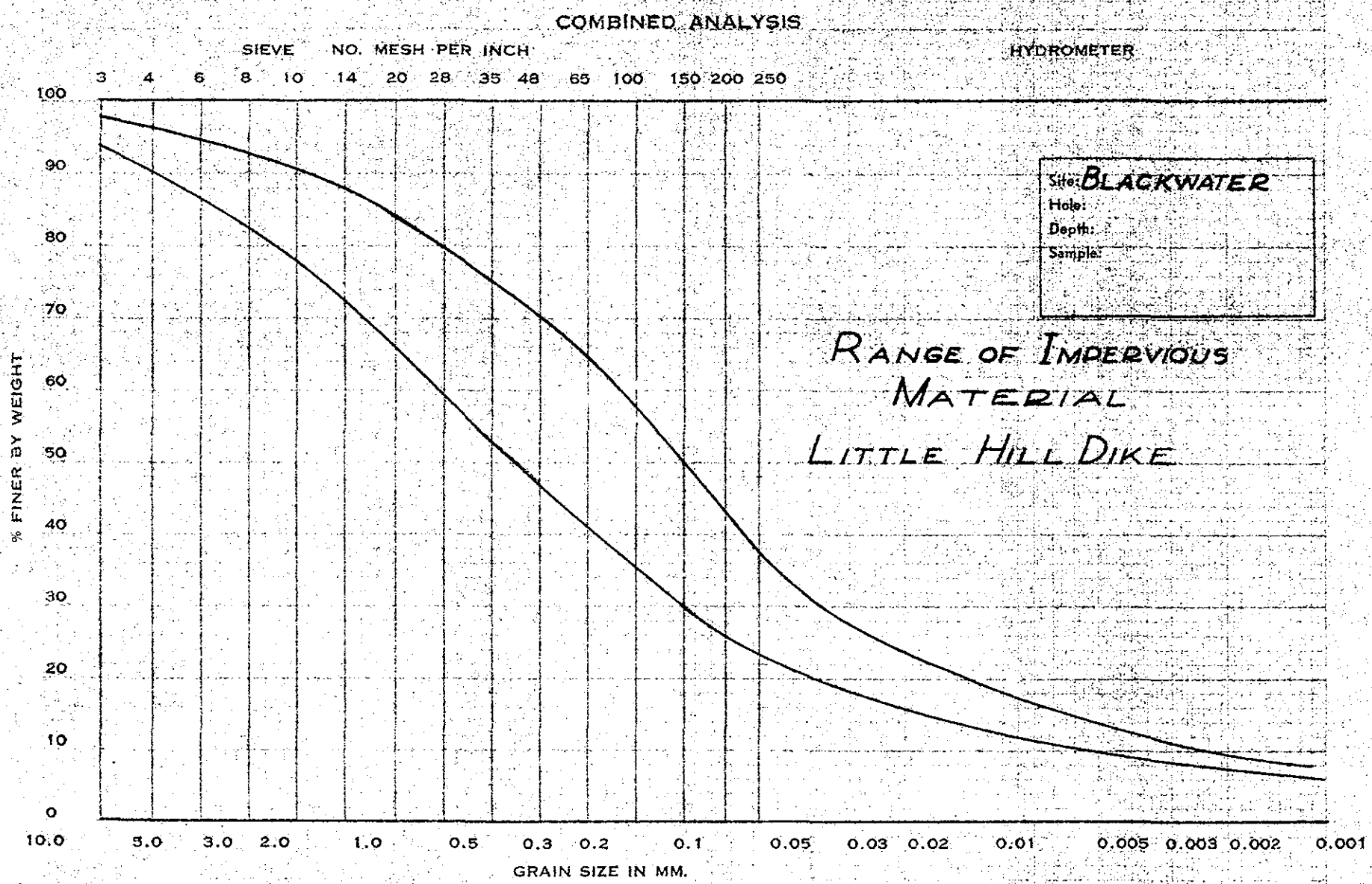

COMBINED ANALYSIS
SIEVE NO. MESH PER INCH
HYDROMETER
3 4 6 8 10 14 20 28 35 48 65 100 150 200 250
100
90
80
70
60
50
40
30
20
10
0
% FINER BY WEIGHT
10.0 5.0 3.0 2.0 1.0 0.5 0.3 0.2 0.1 0.05 0.03 0.02 0.01 0.005 0.003 0.002 0.001
GRAIN SIZE IN MM.
Site: BLACKWATER
Hole:
Depth:
Sample:
RANGE OF IMPERVIOUS MATERIAL
LITTLE HILL DIKE
Plotter
Date

FIG. VIII - 7

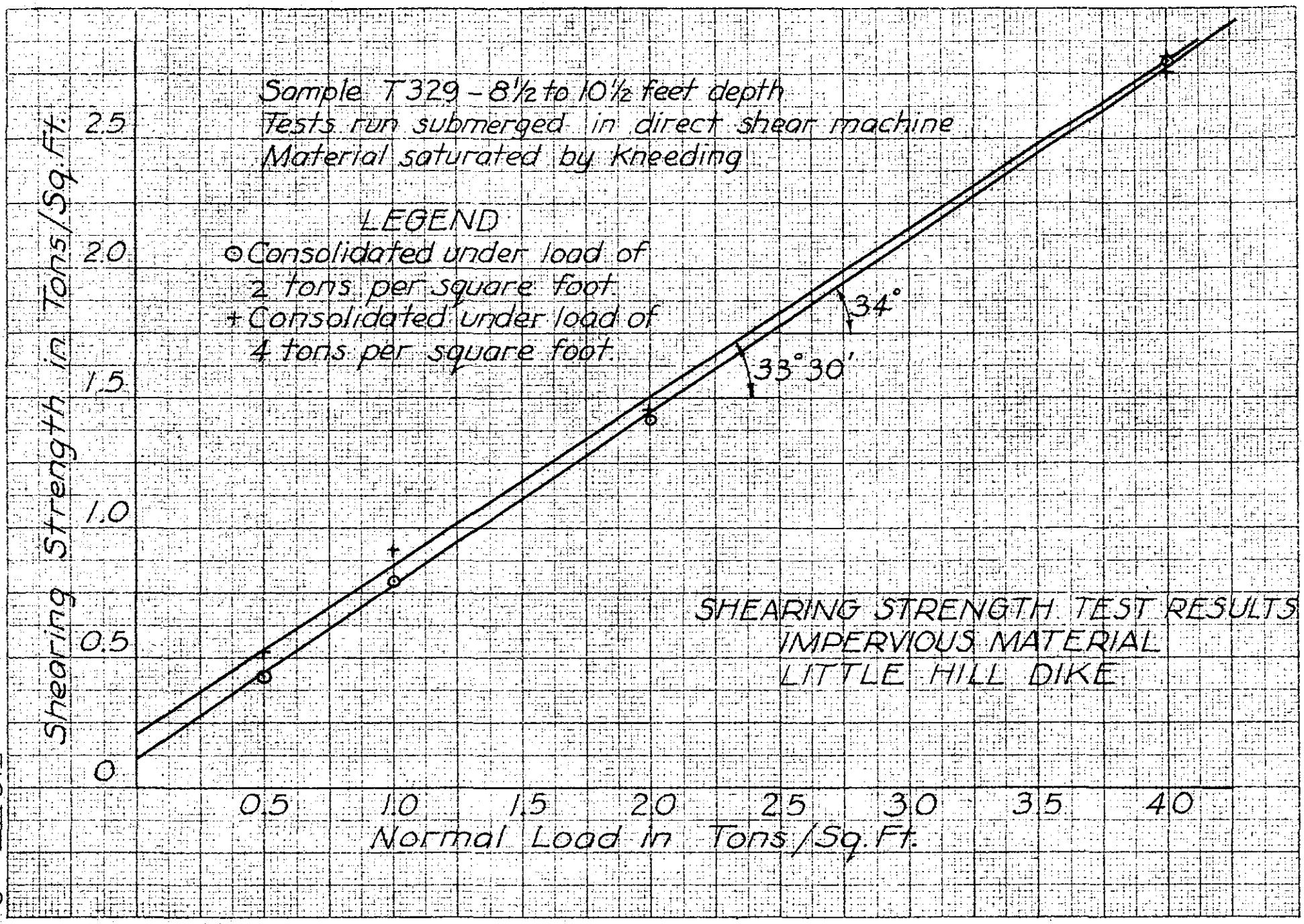

Sample T329 – 8½ to 10½ feet depth
Tests run submerged in direct shear machine
Material saturated by kneeding
LEGEND
⊙ Consolidated under load of 2 tons per square foot
+ Consolidated under load of 4 tons per square foot
34°
33° 30'
SHEARING STRENGTH TEST RESULTS
IMPERVIOUS MATERIAL
LITTLE HILL DIKE
Shearing Strength in Tons/Sq. Ft.
0
0.5
1.0
1.5
2.0
2.5
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
Normal Load in Tons/Sq. Ft.

FIG. VIII – 8

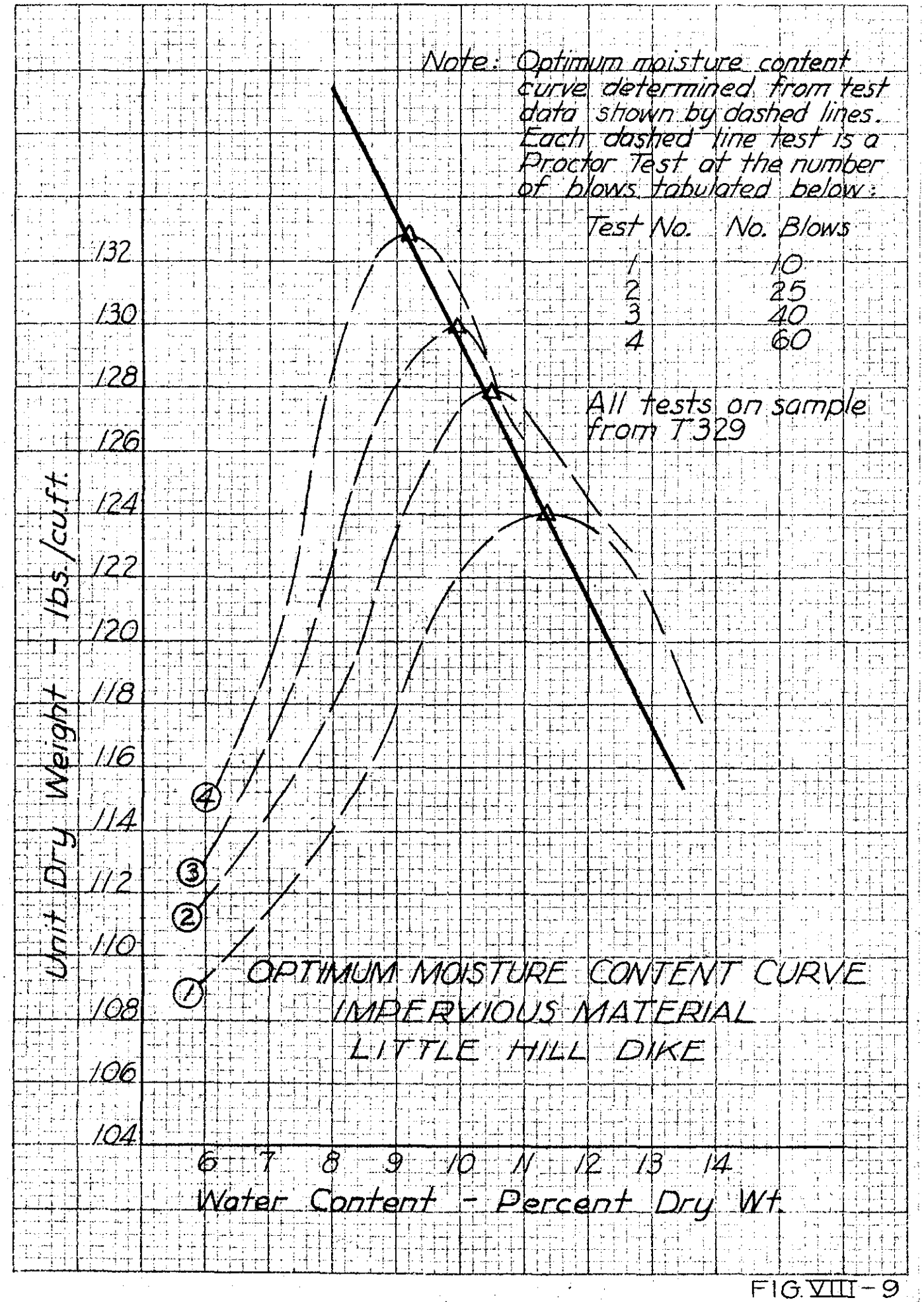
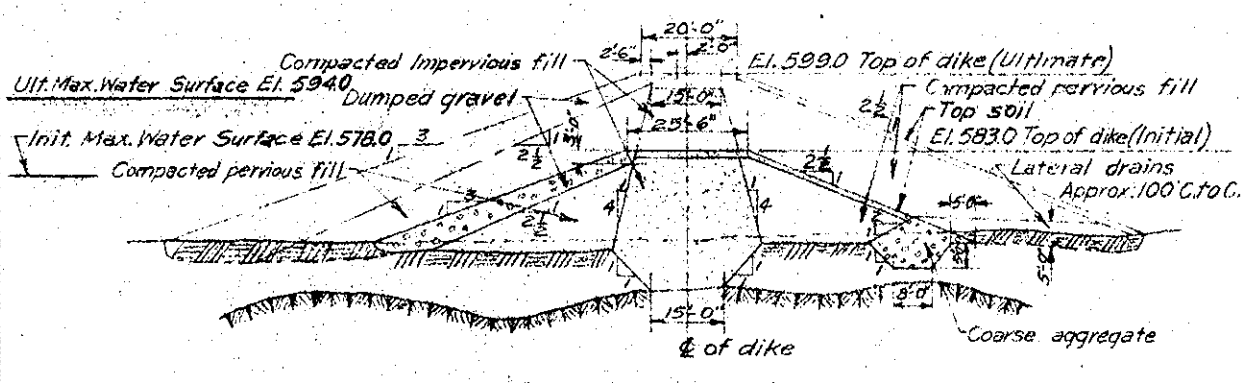
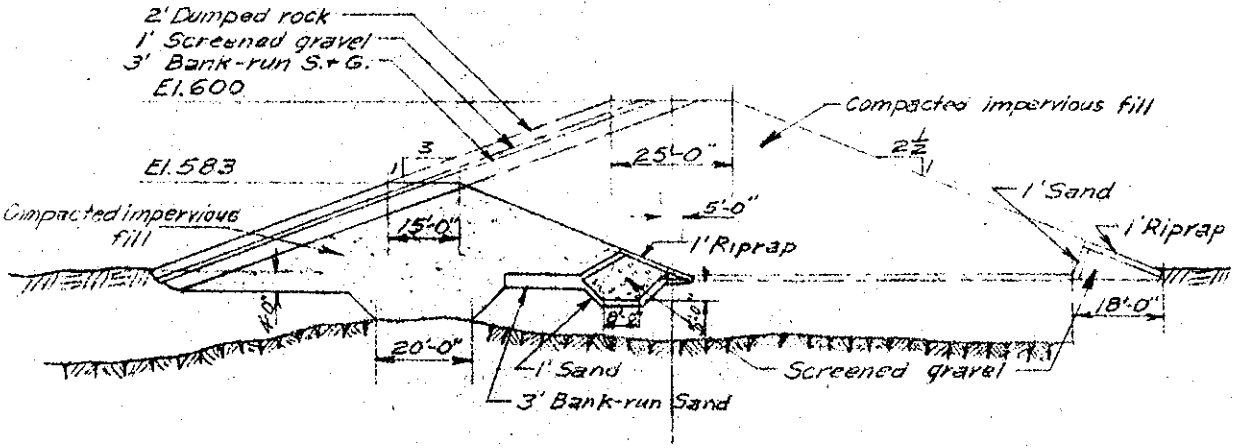

Note: Optimum moisture content curve determined from test data shown by dashed lines. Each dashed line test is a Proctor Test at the number of blows tabulated below:
Test No. No. Blows
1 10
2 25
3 40
4 60
All tests on sample from T329
OPTIMUM MOISTURE CONTENT CURVE
IMPERVIOUS MATERIAL
LITTLE HILL DIKE
Unit Dry Weight - lbs./cu.ft.
132
130
128
126
124
122
120
118
116
114
112
110
108
106
104
6 7 8 9 10 11 12 13 14
Water Content - Percent Dry Wt.

FIG. VIII - 9



SECTION NO. 1

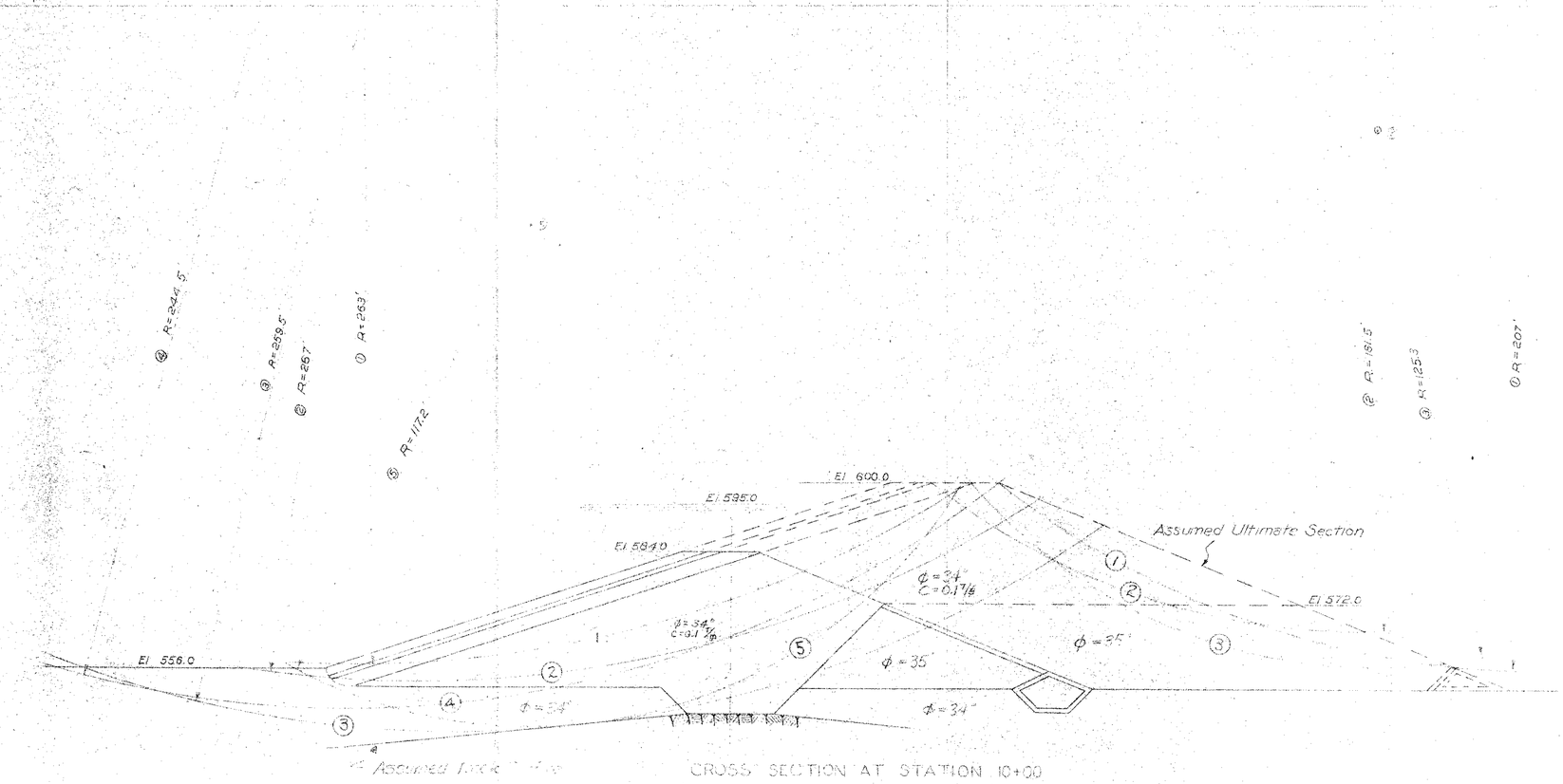


SECTION NO. 2

LITTLE HILL DIKE

Major Sections Considered

FIG. VIII - 10



Stability Ratios for Sliding Surfaces Shown

Sliding Surfaces	UPSTREAM DRAWDOWN	Sliding Surfaces	DOWNSTREAM STEADY SEEPAGE
1	2.10	1	2.03
2	1.95	2	2.17
3	1.77	3	1.97
4	1.90		
5	1.43		
Minimum Stability Ratio		Minimum Stability Ratio	
5	1.43	3	1.97

VERRYVACK VALLEY FLOOD CONTROL
BLACKWATER DAM
BLACKWATER RIVER

RESULTS OF STABILITY ANALYSIS

LITTLE HILL DIKE

U.S. ENGINEER OFFICE BOSTON, MASS.

Scale 1"=20' DEC. 1939

FIG VIII-11

SEEPAGE PER FOOT OF LENGTH

Cutoff Trench Sealed to Rock Surface X-X

$q = \bar{K} h \frac{n_F}{n_E}$

$\bar{K} = 5 \times 10^{-7}$ cm./sec. $= \frac{60}{30.5} \times 5 \times 10^{-7}$ feet/min

$h = 33$ feet

$n_F = 5.5$

$n_E = 6$

$q = \left(\frac{60}{30.5}\right)\left(5 \times 10^{-7}\right)(33)\left(\frac{5.5}{6}\right) = 2.98 \times 10^{-5}$ cubic feet/min/ft.

Cutoff Trench Sealed to Impervious Till at X-X and Rock Surface at Y-Y

$q = \bar{K} h \frac{n_F}{n_E}$

$\bar{K} = 5 \times 10^{-7}$ cm./sec $= \frac{60}{30.5} \times 5 \times 10^{-7}$ feet/min.

$h = 33$ feet

$n_F = 10.5$

$n_E = 6$

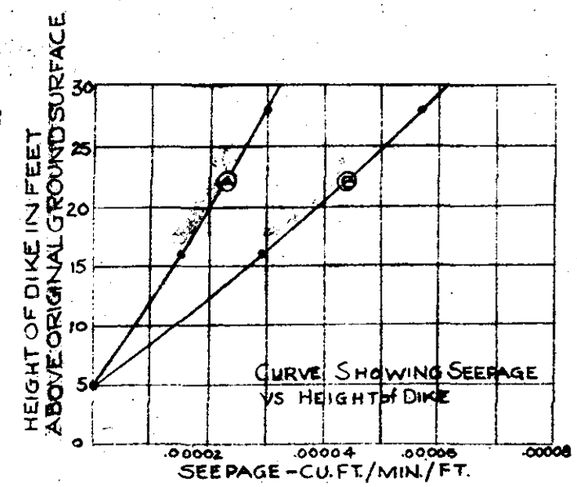
$q = \left(\frac{60}{30.5}\right)\left(5 \times 10^{-7}\right)(33)\left(\frac{10.5}{6}\right) = 5.70 \times 10^{-5}$ cubic feet/min./ft.

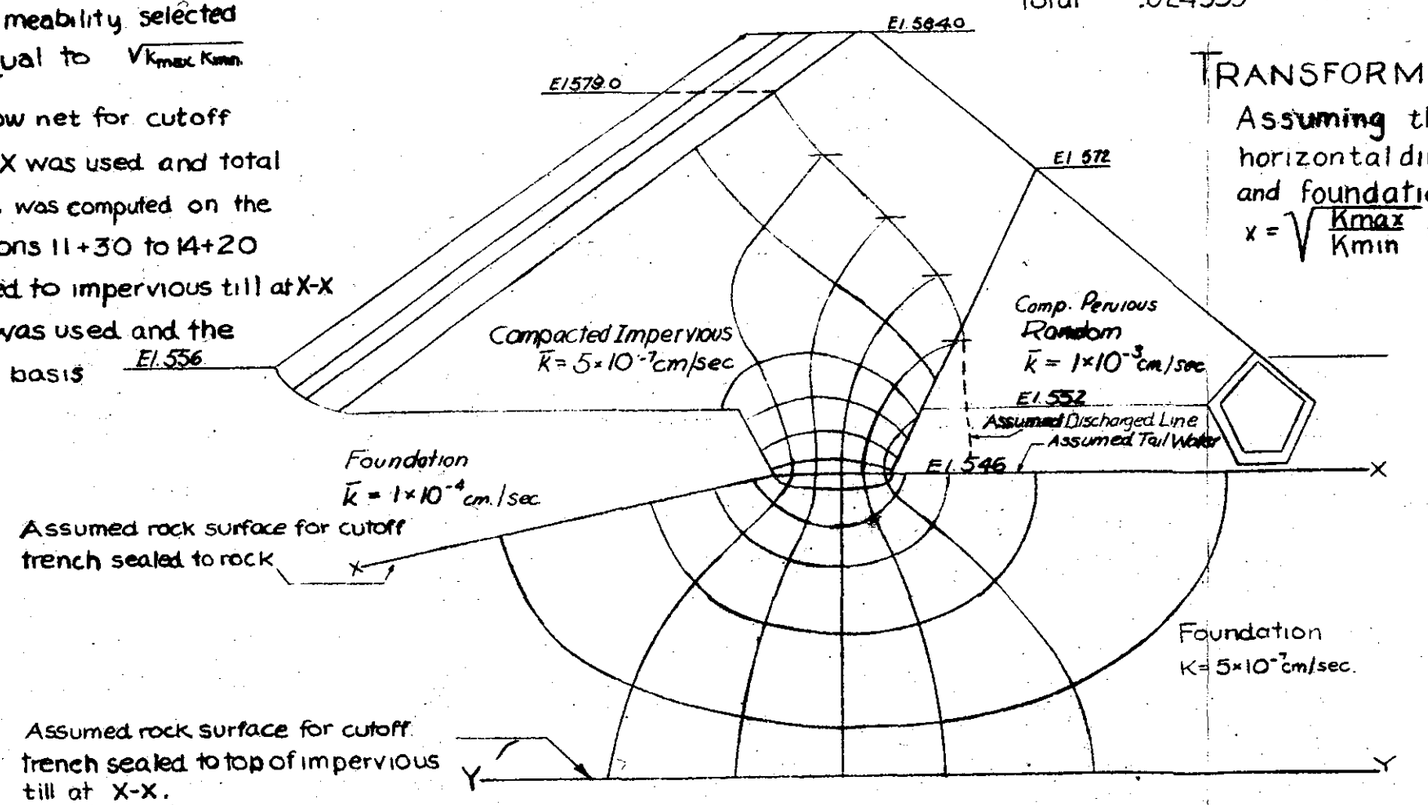
Note: $\bar{K}$ = Average coefficient of permeability selected from test data and is equal to $\sqrt{K_{max} K_{min}}$

Note: From stations 0+00 to 11+30 flow net for cutoff trench sealed to rock surface X-X was used and total seepage between these stations was computed on the basis of Curve Ⓐ. From stations 11+30 to 14+20 flow net for cutoff trench sealed to impervious till at X-X and with rock surface at Y-Y was used and the total seepage computed on the basis of Curve Ⓑ.

TOTAL SEEPAGE

Station	Length feet	Av. Ht. of Dike feet	Seepage cu ft/min/ft	Total Seepage cu.ft/min.
0+40 - 1+00	60	15	.0000110	.000660
1+00 - 2+00	100	15	.0000136	.001360
2+00 - 3+00	100	15	.0000136	.001360
3+00 - 3+60	60	11	.0000083	.000498
4+30 - 5+00	70	8	.0000043	.000301
5+00 - 5+20	20	7	.0000028	.000056
6+90 - 8+00	110	13	.0000110	.001210
8+00 - 9+00	100	20	.0000202	.002020
9+00 - 10+00	100	23	.0000240	.002400
10+00 - 11+00	100	24	.0000250	.002500
11+00 - 11+30	30	24	.0000250	.000750
11+30 - 13+00	170	27	.0000545	.009260
13+00 - 14+20	120	13	.0000215	.002580
			Total	.024955





TRANSFORMATION FACTOR

Assuming that $K_{max} = 4 K_{min}$ all horizontal dimensions of embankment and foundations are divided by factor $x = \sqrt{\frac{K_{max}}{K_{min}}} = 2$

Scale

Vertical 0' 5' 10' 15'

Horizontal 0' 10' 20' 30'

MERRIMACK VALLEY FLOOD CONTROL
BLACKWATER DAM
BLACKWATER RIVER
LITTLE HILL DIKE
FLOW NET & SEEPAGE ANALYSIS
U.S. ENGINEER OFFICE-BOSTON, MASS.
JANUARY 1940

FIG. VIII-12

IX STRUCTURAL DESIGN

9-01. General Standards. The various structures for the Blackwater Dam were designed in accordance with standard engineering practice. Consideration was given to economy in design and to simplicity of construction wherever possible. Satisfactory hydraulic action, pleasing appearance, and durability were other governing factors.

9-02. Unit Weights. In the design analysis of various structures the following unit weights were used:

	Lbs. per Cu. Ft.
Concrete, Class A and Class B	150
Water	62.5
Earth, dry	120 - 126
Rock, dry	110

9-03. Load Assumptions. (a) The assumed loads on major structures consisted of the weight of concrete, vertical and horizontal water pressures and uplift. Other pressures, such as ice, wave, wind and silt, were not considered. Earthquake pressures were investigated to determine their effect on the structures, but not considered as a rigid criterion of design. Uplift was considered as varying from 100 percent maximum headwater to 100 percent tailwater acting over 2/3 the base at foundation and 1/2 the base at sections within the concrete.

(b) Lateral earth pressures used in the design of walls were determined by either the Rankine or Coulomb formulas using physical constants of the backfill materials derived from laboratory tests. Saturation levels were assumed consistent with the location and backfill conditions.

9-04. Concrete Design Standards. (a) Standard Specifications. - The design for concrete and reinforced concrete structures was governed by the report of Committee 501 on "Reinforced Concrete Building Regulations and Specifications", Proceedings, Volume 7, 1936, of the American Concrete Institute.

(b) Class of Concrete. - Two classes of concrete were used. Class A concrete, required to have an average 28-day compressive strength of 3400 lbs. per square inch and a 3/4-inch maximum size of aggregate, was specified for the stairs and stair platforms, and for the walls and roof of the equipment room and stairwell shelter. Class B concrete, required to have an average 28-day strength of 3000 lbs. per square inch and a 2-inch maximum size of aggregate, was specified for the balance of the construction.

(c) Working Stresses. - The allowable working stresses for all reinforced concrete structures are shown in Table IX-1 and are based on a concrete having a minimum 28-day compressive strength of 2500 lbs. per square inch. Twenty-five percent higher stresses were allowed in parts

of structures where construction is temporary or where the maximum loads are developed from extreme design assumptions and are of short duration.

TABLE IX-1 - ALLOWABLE WORKING STRESSES FOR CONCRETE DESIGN

Type of Stress	Stress (lbs. per square inch)
Flexure:	
Extreme fiber stress in compression	1,000
Shear:	
No web reinforcement	50
No web reinforcement and special anchorage	75
Web reinforcement	150
Web reinforcement and special anchorage	300
Bond: Deformed bars only	
No special anchorage	125
With special anchorage	187
Axial Compression:	
In columns with lateral ties	385
Bearing:	
On full area	625
Reinforcing Steel:	
Tension - main steel	18,000
Tension - web reinforcement	16,000
Minimum lap	40 diameters

Ratio of modulus of elasticity of concrete to steel - 1/12

(d) Effective Spans. - For beams and slabs, the effective span length was taken as the center-to-center distance of supports in case of simply supported beams, or the clear distance between supports in cases of fixed ends for monolithic construction. In the case of short spans and thick supports, the effective spans were reduced to provide a design compatible with the probable behavior of the structure under load. All such modifications are indicated on the detailed computation sheets. (Appendix A).

(e) Anchorage, Spacing, and Covering of Reinforcement. - Adequate embedment beyond face of supports, laps of continuous steel and continuity of reinforcing in rigid frames was provided. The spacing of bars and the protective covering of concrete was maintained equal to or greater than the minimum allowable as specified in Section VIII of the contract specifications.

(f) Temperature Steel. - Expansion and construction joints were located so as to limit pours to such lengths that shrinkage stresses and cracks will be minimized. The maximum length or width of monoliths is 44 feet. Exposed surfaces of walls, except gravity structures, were provided with face reinforcement. Longitudinal steel and tension rings were provided around the conduits.

9-05. Structural Steel Standards. (a) Basis. - All structural

steel was designed in accordance with specifications of the American Institute of Steel Construction (1934).

(b) Allowable Stresses are shown in Table IX-2. In the design of trash bars and racks, 33-1/2 percent higher stresses were allowed for the extreme condition of the racks clogged under full water head.

TABLE IX-2 - ALLOWABLE WORKING STRESSES FOR STEEL DESIGN

TYPE OF STRESS	STRESS (Lbs. per Square Inch) A.I.S.C. (1934)
Tension (on net section)	18,000
Axial Compression	$\frac{18,000}{1+\frac{l^2}{18,000r^2}}$ Max. f = 15,000
Compression in Bending	$\frac{20,000}{1+\frac{l^2}{2,000b^2}}$
Combined Direct and Bending	$f = \frac{P}{A} \pm \frac{Mc}{I}$ sq. in.
Shear (Gross area, unstiffened webs)	$\frac{18,000}{1+\frac{h^2}{7200t^2}}$

9-06. Structure Analysis. (a) General. - (1) All concrete structures are founded on firm bedrock. Because of the structural soundness of the rock and also in view of the relatively low foundation pressures, no foundation grouting other than to reduce seepage is required. The grout curtain provided for this purpose extends the full length of the dam and consists of holes spaced 5 feet on centers, varying in depth from 20 to 50 feet.

(2) The several concrete structures were designed for conditions of loading which will occur with the ultimate development as shown on Figures I-2 and I-3 (maximum water surface Elevation 595.0 M.S.L., top of dam Elevation 600.0 M.S.L).

(b) Non-Overflow Section. - The non-overflow section is a full gravity section carrying a 13-foot roadway supported by piers. The initial construction, as shown on Figure I-2, has no roadway and is designed to give both a pleasing present appearance and adaptability to future construction. The structure was analyzed under two conditions of loading as shown on Figure IX-1, Case I, full hydrostatic pressure upstream, corresponding tailwater downstream, and Case II, reservoir empty. The section is keyed into the embankment core by three reinforced cut-off walls. The section is safe against overturning and sliding. The maximum foundation

pressure is 7.86 kips per square foot and the coefficient of sliding, 0.64.

(c) Penstock Intake. - (1) The penstock intake is a semi-gravity structure containing openings for stop logs, head gate, penstock, stairway, and galleries. The walls and slabs around the various openings are reinforced for the local conditions of loading and concentrations of stress. No reinforcing, however, is required for the stability of the penstock intake as a unit. The structure is designed for three conditions (see Figure IX-1): Case I, full water pressure with trash racks clogged and the openings in the structure unwatered; Case II, full water pressure with the penstock gate open; Case III, reservoir empty. The stress analysis is tri-dimensional, taking into consideration all the eccentricities of forces and the non-symmetries of sections. The maximum foundation pressure is 10.37 kips per square foot and the coefficient of sliding, 0.54.

(2) The walls of the stairwell, gatewell and stop log well are in general designed as fixed-ended horizontal beams. The base slab under the bellmouth was considered both as a fixed-ended beam and as a beam resting on three rack beams, and its reinforcing is designed to take care of both conditions. The rack beams are supported at the top by a horizontal concrete beam, which is designed as a fixed-ended beam.

(3) The penstock opening is reinforced to take the full inside water pressure without considering the weight of the overlying concrete. At the downstream end, a recessed collar is provided to serve as an anchor for the future penstock. This collar is designed to resist half of the force acting on the full area of the penstock under full hydrostatic head. The equipment room walls are designed in accordance with American Institute of Architects' File No. 4, and are dowelled to the intake structure. The roof of the equipment room is a slab resting on steel beams. In the initial stage, the penstock intake will be closed by a temporary bulkhead consisting of reinforced concrete beams placed in the gate recess.

(d) Spillway. - (1) The spillway is a gravity Ogee structure and is analyzed for two loading conditions (see Figure IX-1): Case I, full headwater and corresponding tailwater; and Case II, reservoir empty. The maximum foundation pressure is 8.18 kips per square foot and the coefficient of sliding, 0.62.

(2) The gate operating rooms, access galleries, and conduits are openings in solid masonry and cause concentration of stresses. No tests (model or photoelastic) were made. The stresses were evaluated according to the procedure outlined in the Bureau of Reclamation's Technical Memorandum No. 451 on "Gallery Stresses and Steel Reinforcing."

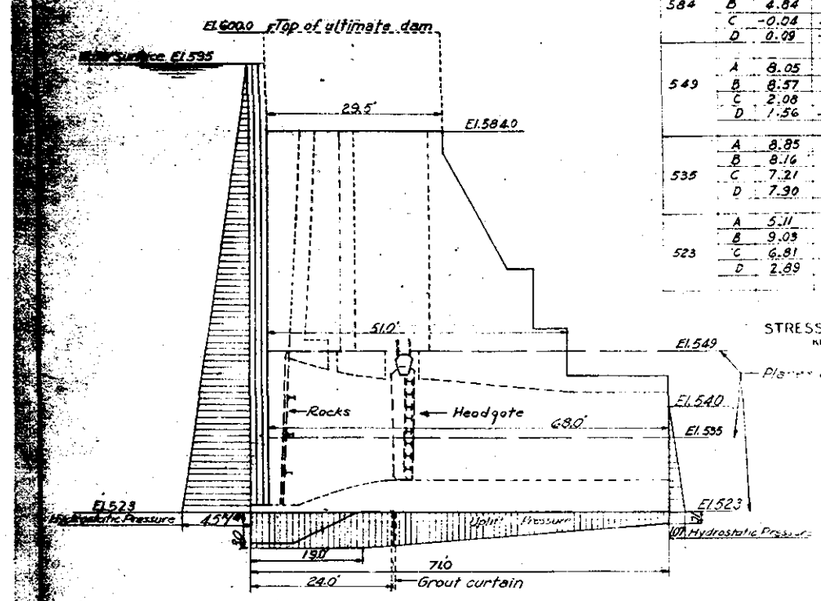
(e) Walls. - The spillway guide wall and west approach wall are in part a combination of concrete facing 3 feet thick and a gravity section. The facing sections are constructed integral with the gravity portion and are anchored to the rock with 1-1/4-inch bars spaced 10 feet on centers. The gravity sections are designed for full backfill loading

with saturation levels consistent with the fill and the location. Uplift on the base is assumed equal to full water pressure applied to 2/3 the base area. The east approach wall is entirely a facing section.

(f) Miscellaneous. - (1) The arrangement of construction and expansion joints was made as simple as possible, their location being governed by field requirements and structural consideration. Where water-tightness was a factor, copper water stops were inserted in the joints.

(2) Adequate provisions for grouting were made wherever required for the accurate alignment and for the erection of mechanical equipment and structural steel members.

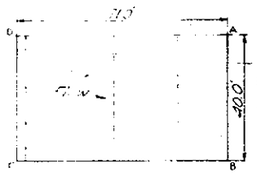
(3) Access to various points is afforded by means of stairways, galleries, ladders, and manholes, provided with substantial guards, railings, and cover plates.



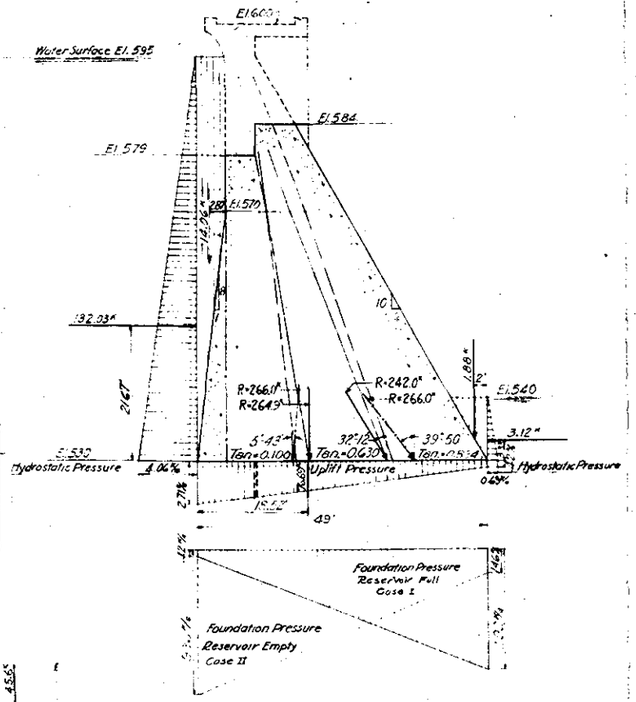
PENSTOCK INTAKE

CASE		I		II		III
		Gate Open		Rocks Clogged		Reservoir
Elevation	Point	No Uplift	With Uplift	No Uplift	With Uplift	Empty
584	A	4.97	4.61	4.61	4.42	2.15
	B	4.84	4.48	4.55	4.37	2.15
	C	-0.04	-0.37	-0.08	-0.26	2.36
	D	0.09	-0.24	-0.03	-0.20	2.36
549	A	8.05	7.65	7.96	7.48	1.41
	B	8.57	8.00	8.54	7.96	2.03
	C	2.08	0.29	1.18	-0.62	8.93
	D	1.56	-0.06	0.60	-1.10	8.31
535	A	8.85	5.50	6.40	9.17	-0.04
	B	8.16	5.33	5.94	8.84	1.02
	C	7.21	2.32	8.21	4.16	14.66
	D	7.90	2.49	8.67	4.49	13.24
523	A	5.11	5.51	5.60	6.06	0.13
	B	9.03	9.43	8.40	8.84	4.51
	C	6.81	4.37	5.16	2.70	10.37
	D	2.89	0.51	2.36	-0.08	5.99

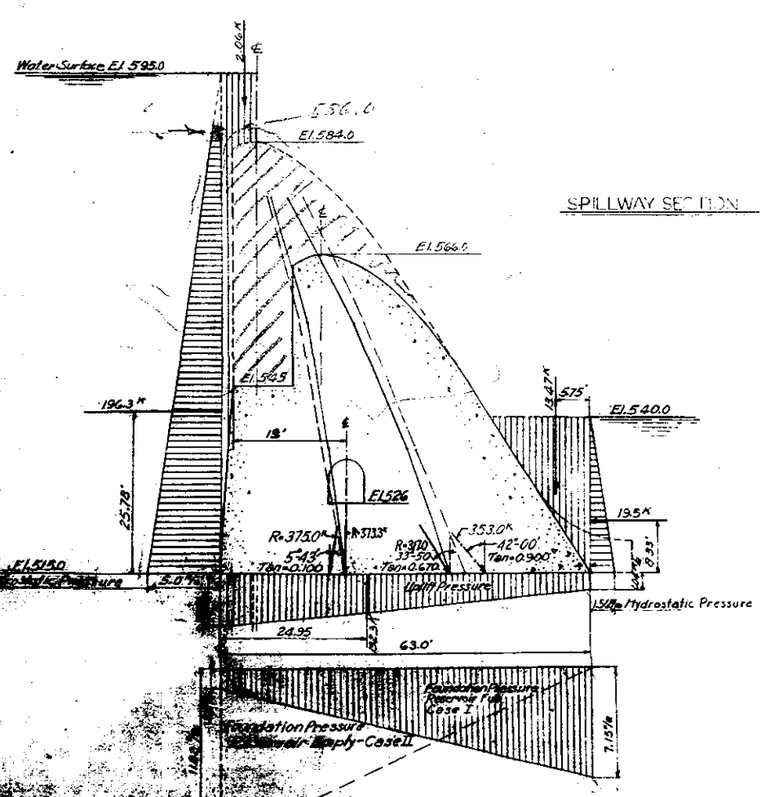
STRESS DISTRIBUTION
KIPS PER SQ FT.



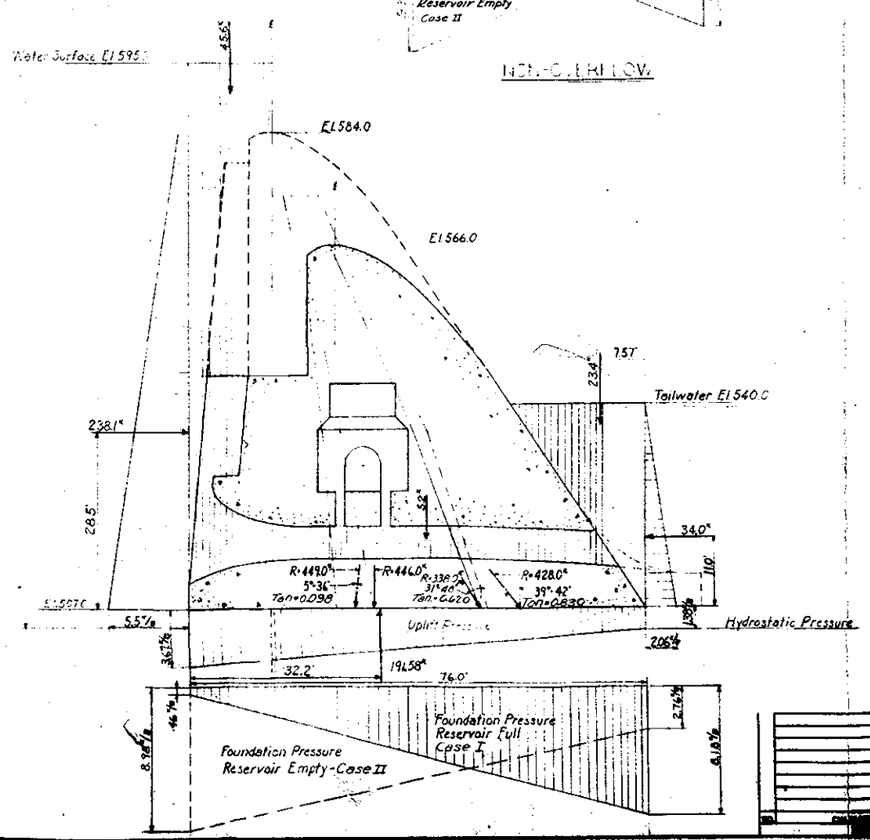
KEY PLAN



NON-OVERFLOW



SPILLWAY SECTION



BASIS OF ANALYSIS

A- Weights: Concrete 150#/cu ft
Water 62.5#/cu ft

B- Uplift.
For spillway and non-overflow section uplift pressure applied over ⅔ area at all sections with straight line variation of pressure from headwater at upstream face to tailwater at downstream face.
For penstock intake, uplift applied over ⅔ area at base and ½ area at intermediate sections with pressures based on head and tailwater and tapered to suit location of grout curtain and interior openings.

C- Earthquake loading:
Earthquake forces applied to full section of spillway and non-overflow section.
For Case I, inertia and hydrodynamic forces applied simultaneously with static water pressure and in same direction.
For Case II, inertia force applied in upstream direction.
Horizontal acceleration 0.1 g.
Vertical acceleration 0.
Hydrodynamic pressure 0.00347 $(H)^2$ kips/sq ft (arm 0.4H)

LEGEND

— — — Line of kern points
——— Line of resultants
Scale of pressure diagrams 1 inch = 4 kips.
Forces shown are in kips per linear foot of section.
—·—·— Resultant with earthquake
——— Resultant without earthquake

MERRIMACK VALLEY FLOOD CONTROL
BLACKWATER DAM
BLACKWATER RIVER
STABILITY ANALYSIS
MASONRY STRUCTURES

IN 1 SHEET — SCALE: 1"=10'

U. S. ENGINEER OFFICE, BOSTON, MASS. — JANUARY 1958

APPROVAL RECOMMENDED: [signature]
SUBMITTED: John E. Allen
[signature]

FILE NO.

X CONSTRUCTION PROCEDURE

10-01. Sequence of Operations. (a) General. - Except for clearing and removal of stumps over 6 inches in diameter at the dam and dike sites and in the borrow areas, which will be done by Government plant and hired labor, it is proposed to build the dam, dikes and appurtenant works by contract, with advertisement for bids scheduled for March 1940. In paragraph 1-13 of the specifications for construction of the dam and appurtenances, the order of work for contract operations has been outlined on the basis of a 1-1/2-year construction period. The earthwork distribution is shown on Sheet 6 of the contract drawings. The following paragraphs describe the sequence of operations for the 1-1/2-year period.

(b) 1940. - Clearing and removal of stumps over 6 inches in diameter will be done by Government plant and hired labor during the period February to May 1940. Assuming award of contract in May 1940, foundation areas will be stripped and prepared and borrow areas opened up. Concrete in the non-overflow section, the penstock intake, and west approach channel wall, totalling 11,000 cubic yards, will be initiated, diversion of the river effected, and concrete operations in the spillway started. The contract specifications require that the dam embankment and the dikes shall be kept sufficiently in advance of the concreting operations in the spillway so that with a discharge of 10,000 c.f.s. over the spillway as constructed, the river will not overtop the dam embankment or the dikes. Because of this requirement, earthwork and concrete operations will have to be closely coordinated. Construction of the dam embankment involves the placement of 135,000 cubic yards of earth fill and 70,000 cubic yards of rock fill; of the dikes, 70,000 cubic yards of earth fill; of the spillway, 10,000 cubic yards of concrete. If conditions prevent completion of the dam embankment and dikes, except for seeded topsoil, before freezing weather in the late fall of 1940, the requirement cited above concerning passage of 10,000 c.f.s. over the spillway without overtopping the dam embankment or dikes will allow the work to be carried safely over the winter of 1940-41. Conduit liners, gate bonnets, and covers will be installed integral with the spillway unless delivery is delayed, in which case recesses will be left for later installation.

(c) 1941. - The dam embankment, the dikes, the spillway, and discharge channel wall will be completed. Construction of the equipment room and the driveway will be started and carried to completion. All remaining items of work will be completed before June 30, 1941.

10-02. Field Control for Construction of Dam Embankment and Dikes. (a) Classification of Materials. - The character and classification of materials available and requirements of the material for various parts of the dam embankment and the dikes are given in Sections VI, VII and VIII. Detailed requirements are given in Sections

VI and VII of the contract specifications. Sufficient laboratory tests and explorations have been made, as outlined in Section V herein, of the available materials from excavation and designated borrow areas to establish the satisfactory character and quantity of material for construction. Control of the character of the materials to be placed in the various sections of the dam embankment and dikes will be accomplished by mechanical analyses in the field. A complete program of laboratory tests will be performed on samples selected from points systematically distributed throughout the embankment for a record of the materials used.

(b) Methods of Placing and Rolling. - A detailed description of the control to be exercised over the placement and rolling of materials is given in Section VI of the contract specifications. The maximum allowable thickness, before compaction, of each layer of material will be 6 inches for impervious material, 12 inches for pervious material, and 12 inches for the fill material for the Dodge Estate Dike. The contractor will be required to maintain the proper amount of moisture necessary for optimum compaction. Type, weight and speed of the compaction equipment have been specified, and each layer will be required to be rolled with 6 passes of the roller, with the provision that if sufficient compaction has not been attained with that amount of rolling, additional passes will be required and paid for at the contract price for additional rolling.

(c) Compaction. - To insure that the compacted material will have the characteristics used for design described in Sections VI, VII and VIII, the embankment materials will be compacted to required densities. Unless other conclusions are made based on additional laboratory tests prior to and during construction, the required densities, as obtained from the conclusions in Sections V to VIII inclusive, will be as follows (for material passing 1/4-inch sieve):

	Dry Weight (Lbs. Per Cu. Ft.)
Dam Embankment	
Pervious	123
Impervious	120
Dodge Estate Dike	
Fill	123
Little Hill Dike	
Pervious	*
Impervious	120

*Variable - degree of compaction 80 percent.

Samples of all embankment materials will be taken after placement and compaction. Results of tests on these samples will be used to make

corrections, adjustment and modifications of methods, materials, moisture content, and requirements for additional compaction in order to secure the required densities.

10-03. Dam Embankment Piezometers and Settlement Gages will be installed as shown on Sheet 10 of the contract drawings. The settlement gages will be flat plates connecting with gage pipes through the fill in such a manner as to permit reading during and after construction. Readings will be taken of the settlement gages periodically and checked with the computed settlement of each gage. The piezometer gages will be installed in the upstream pervious section. Readings of these gages will be taken during draw-down of the reservoir to provide data which will be useful in future designs of similar structures.

10-04. Grouting. The bedrock foundation of the spillway, non-overflow section, dam embankment, and Little Hill Dike will be grouted as shown on Sheets 9 and 11 of the contract drawings. The actual depths, spacing and direction of holes will be governed by the conditions encountered during the grouting operations.

10-05. Concrete Construction. The "Instructions Relative to Concrete Testing" contained in a letter from the Division Engineer to District Engineers, dated November 10, 1938 (F.C. NAD 1/29.1-A) will form the basis for laboratory and field control of concrete construction. As directed in this letter, the facilities of the Central Concrete Laboratory, U.S.M.A., West Point, N. Y., will be utilized for laboratory control. In the field, the necessary facilities and inspection forces will be maintained to inspect concrete operations, make field and acceptance tests, and prepare samples and specimens for shipment to the Central Concrete Laboratory.

10-06. Structural Steel Construction. Notarized certificates of tests certifying the compliance with specifications of materials entering into structural steel work will be accepted. At the discretion of the contracting officer, fabricated steel work and machinery will be inspected in the shop. Rigid supervision and inspection will be maintained of structural steel construction in the field.

XI SUMMARY OF COST

11-01. Estimated Cost. The cost of the proposed Blackwater Dam and appurtenances, based on preliminary design, and used only for analysis of economic justification, was estimated as follows:

Dam		
Embankment	$205,000	
Concrete Structures	385,000	
Miscellaneous	95,000	685,000
Dikes		55,000
Total (including engineering, contingencies and overhead)		740,000