CONNECTICUT RIVER FLOOD CONTROL PROJECT

CLAREMONT DAM

SUGAR RIVER, NEW HAMPSHIRE

ANALYSIS OF DESIGN

WAR DEPARTMENT CORPS OF ENGINEERS U. S. ARMY
U. S. ENGINEER OFFICE PROVIDENCE, R. I.

JANUARY 1945

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CLAREMONT DAM

ANALYSIS OF DESIGN

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I. PERTINENT DATA

I. PERTINENT DATA

Drainage area

Net (below Lake Sunapee)	199 sq. miles
Gross (including Lake Sunapee)	245 sq. miles

Elevations

Top of dam	El. 650 m.s.l.
Spillway crest	El. 630 m.s.l.
Pool at maximum flood	El. 645 m.s.l.
Low water in stream at dam site	El. 523 m.s.l.
Streambed at dam site	El. 522 m.s.l.

Hydraulic data

Reservoir capacity

Elevation	Area of reservoir in acres	Capacity in acre feet	Corresponding storage in inches of runoff from the gross drainage area
Below El. 630.0, spillway crest	1420	78,300	6.0
Total below El. 650, top of dam	1670	108,300	8.3
Total below El. 645 (surcharge for spillway design discharge)	1610	101,000	7.7

Years of hydrographs	15 years (1929 to 1943 incl.)
Maximum flow, peak (estimated) Flood of March, 1936	13,400 c.f.s.
Maximum flow, 24 hour mean, Flood of March 1936	11,200 c.f.s.
Estimated peak inflow, spillway design flood	92,000 c.f.s.
Estimated peak outflow, spillway design flood	87,000 c.f.s.
Estimated volume, spillway design flood	14.7 inches

Estimated volume, outlet design flood	9.08 inches
Estimated peak inflow, outlet design flood	25,300 c.f.s.
Approximate tailwater elevation at maximum flood (Pool at El. 645)	El. 546.2 m.s.l.
Approximate tailwater elevation outlet discharge of 10,300 c.f.s. (Pool at El. 630)	El. 534.0 m.s.l.
Outlet capacity	
Pool at El. 630	10,200 c.f.s.
Spillway capacity	
Pool at El. 650.0 (Top of dam)	137,800 c.f.s.
Pool at El. 645.0 (Maximum flood elevation)	87,000 c.f.s.
Embankment	
Maximum height above streambed	128 feet
Total length on top	2,600 feet
Top width of dam	30 feet
Total fill in embankment	3,426,000 c.y.
Total impervious core and cut-off trench	510,000 c.y.
Total random fill	790,000 c.y.
Total pervious fill	1,720,000 c.y.
Total dumped rock	262,000 c.y.
Total gravel bedding	94,000 c.y.
Total topsoil	50,000 c.y.
Outlet works	
Shape of tunnel	Circular
Diameter of tunnel	16 feet
Minimum thickness of concrete lining	1 foot
Length of tunnel, portal to portal	931± feet

II. INTRODUCTION

A. AUTHORIZATION. - The Claremont Dam is one of 20 dams included in the Comprehensive Plan for flood control in the Connecticut River Basin as reported in House Document No. 455, 75th Congress, 2d Session. This report was authorized by (1) Section 6 of River and Harbor Act approved August 30, 1935, Public No. 409, 74th Congress, (2) Resolution dated March 27, 1936 by the Committee on Flood Control of the House of Representatives (3) Section 6 of the Flood Control Act approved June 22, 1936, Public No. 738, 74th Congress, (4) Public No. 334, 73d Congress, H. R. 8562, approved June 13, 1934, and (5) Instructions from the Office of the Chief of Engineers dated August 8, 1936. The general comprehensive plan for flood control in the Connecticut River Basin as set forth in House Document No. 455, was approved by Congress in the Flood Control Act of June 28, 1938, Public No. 761, 75th Congress. This plan was modified to include the works recommended by the Chief of Engineers in House Documents No. 653 and No. 724, 76th Congress, 3d Session, which were approved by Congress in the Flood Control Act of August 18, 1941, Public Law 228, 77th Congress. The Claremont Reservoir is one of the 20 reservoirs included in the approved plan as set forth in House Document No. 724.

B. EFFECT OF CLAREMONT RESERVOIR IN REDUCING FLOODS. - The Claremont reservoir will provide almost complete flood protection for the communities of Claremont and West Claremont on the Sugar River, and will have a considerable flood-reducing effect on the Connecticut River below its confluence with the Sugar River. In this respect the effect of the reservoir is estimated as follows:

Location	Drainage Area (square miles)	% Reduction of peak discharge
Bellows Falls	5,387	6.6
Vernon	6,240	5.0
Holyoke	8,284	3.8
Springfield	9,587	2.8
Hartford	10,643	2.6

The effect of the reservoir will be as shown above in the case of a general rain augmented by melting snow. The reservoir will effect greater reductions than shown above in the case of concentrated rainfall over its watershed and less rainfall over other parts of the Connecticut River watershed. A reverse distribution of rainfall will cause the reservoir to be less effective in its reduction of Connecticut River peak discharges.

C. CONSERVATION STORAGE. - Studies to determine the ratio of cost versus benefits for conservation storage combined with flood control storage, indicate that storage for this purpose is not feasible at this site.

D. RECREATIONAL FACILITIES. - Various positions of the reservoir area, particularly the flat area in the valley bottom downstream of the dam and between the existing State Highway Route No. 11 and Sugar River, possess excellent possibilities for possible future development as recreational areas for the public. No construction directly related to the development of a recreational area has been included in the contract work.

E. GENERAL BRIEF DESCRIPTION OF THE DAM. - The dam will be an earth-fill structure having a maximum height of 128 feet from the river bed to the top of the dam and a total length of approximately 2600 feet. The embankment will be placed by the rolled-fill method. The

upstream face of the embankment will be riprapped with dumped rock; the downstream face will be topsoiled and seeded except for the toe which will be dumped rock.

The reservoir outlet will consist of a gate-controlled tunnel excavated in the rock of the left abutment and discharging into the river well below the downstream toe of the dam. The gates will be housed in a concrete intake tower located at the upstream end of the tunnel. The spillway weir will be located in the right abutment about 900 feet from the center line of the dam. The weir will be founded on rock and the south end of the weir will be separated from the earth abutment by a concrete retaining wall. The spillway discharge will be carried in a channel excavated in rock down the slope of the abutment to the valley floor at a point remote from the downstream toe of the dam. Across the valley a pilot and drainage channel will be excavated in the overburden to induce erosion along the proper alignment and discourage a tendency for the spillway discharge to meander.

III. SELECTION OF SITE

III. SELECTION OF SITE

A. GENERAL LOCATION. - The Claremont Dam site is located on the Sugar River, New Hampshire, a tributary of the Connecticut River, having a total drainage area of 269 square miles at its confluence with the Connecticut. The area to be controlled by the reservoir contains 245 square miles. The site selected is located on the Sugar River about one mile southeast of Claremont, Sullivan County, New Hampshire, and about 7.1 miles above its confluence with the Connecticut River. (See Plates Nos. 1 and 2).

B. SITES CONSIDERED. - Several sites above and below the site finally selected were investigated. All of these sites were eliminated principally because of unfavorable geologic conditions and the difficulty of securing sufficient storage capacity. The exact alinement was chosen so that Flat-rock Brook can be economically diverted without danger of damage to the dam embankment. From the consideration of reservoir capacity, geologic conditions and construction costs the site selected is considered the most feasible site for flood control purposes.

C. SELECTION OF SITE. - A thorough investigation of the river valley including map studies, field reconnaissance, reservoir and site surveys and geologic investigations led to the selection of the site. Possible dam sites at this point are limited by topography to a reach of the river of about 3,000 feet. With soft varved silt in the valley forming a major foundation problem at this point, borings were taken over the whole 3000 feet of this reach and show this silt nearly 100 feet thick downstream decreasing to 40 feet thick upstream of the centerline selected. The site selected is at the extreme upstream reach of possible sites in this reach and has the least average depth of soft varved silt of all the possible sites in this reach,

which is considered a real advantage. It is satisfactory from the viewpoint of economy, safety and utility, and was found adaptable to a design of dam and appurtenant structures, the cost of which has a favorable ratio to the benefits derived.

D. SELECTION OF TYPE OF DAM. - A study of the geologic conditions including determination of the surface of bed rock resulted in the adoption of an earth-fill dam as being the most practicable type for the site. Available embankment materials are well separated into types in natural state with impervious glacial till in spillway excavation in right abutment, and pervious sands and gravels in river terrace upstream from dam on left side of river. Such natural segregation of materials is well adapted to a rolled fill type of dam. Rock lies close to the surface on the left abutment, but the deep deposit of overburden which occurs in the river bed and on the right abutment, makes the cost of a concrete dam prohibitive. No saving in embankment volume would result from the use of a rock-fill dam owing to the unstable nature of the foundation. Hence a rock-fill dam would be infeasible. A hydraulic-fill dam was given consideration, but owing to inherent drawbacks in the construction method and the objectionable stresses induced in the foundation by this method it will not be used. The materials available from excavations and borrow areas are found to be suitable for the rolled-fill method of embankment construction, and it was therefore adopted. The upstream slope of the embankment will be protected with dumped riprap. Heavy rock fills will be constructed at both upstream and downstream toes. Due to an insufficiency in the quantity of rock from excavations, the downstream slope will be topsoiled and seeded and provision will be made to carry the surface run-off through a system of underground drains to an outfall

well below the downstream toe. Provision has been made to decrease the amount of topsoiling and seeding on the downstream slope, if rock becomes available in greater quantities than are anticipated.

E. TREATMENT OF RAILROAD. - The single track line of the Concord and Claremont Railroad owned by the Northern Railroad Company and leased and operated by the Boston & Maine Railroad passes through the damsite and will require relocation or abandonment. This railroad extends from Concord, New Hampshire to Claremont Junction, New Hampshire. The minimum section considered for abandonment is the portion lying between Claremont and Newport, a distance of approximately 12 miles.

The contract drawings are based upon the premise that the railroad will be abandoned before construction of the dam commences. In the event it is decided to relocate the railroad, this will not affect the design of the dam and its appurtenances as none of the several possible alignments for the relocation involve a change in any of the structures. Rock excavation for a short section of the highway relocation is included in the contract work and will be affected by the alinement of the railroad, if relocated.

F. BROOK DIVERSION. - A small stream known as Flatrock Brook enters the Sugar River at a point within the area of the dam embankment. The drainage area of the brook is approximately 250 acres. Several schemes were investigated to divert the stream away from the dam and dispose of its flow. The most feasible was the position indicated on Plate No. 58 and consists of a channel excavated in rock and discharging into the outlet channel of the dam.

IV. GEOLOGICAL INVESTIGATIONS

IV. GEOLOGICAL INVESTIGATION

A. NATURE OF VALLEY. - The Sugar River Valley wherein the reservoir will be formed is located in the foothills of the Green Mountain on the north and the Unity Mountain mass on the south. The broad summit of the Unity Mountain, approximately 1300 feet above the valley bottom, is situated 2.5 miles south of the river. That of the Green Mountain, situated at a similar distance away but to the north reaches a height of about 1500 feet above the river. The valley is made up essentially of two parts. A geologically ancient and more extensive valley than that which is now expressed in the topography dates back in origin to pre-glacial time. A second and younger valley which above Claremont is superimposed upon the older, is formed in glacial sediments. The pre-glacial rock valley is deeply buried and ledge rock is only infrequently exposed at river level at localities where the present stream is directed against the lower slopes of the main hillside. About 1.5 miles below the site the river flows over schist ledges for a considerable distance, its course being north of west to the Connecticut River. Downstream of Claremont this stretch of valley is geologically younger than the completely buried and blocked pre-glacial valley which underlies Claremont on the south. This buried valley is directed in a south of west course toward the Connecticut River.

The sediments overlying bedrock are of glacial origin. A ground moraine or glacial till of chiefly ice deposited mixed materials of fine and coarse texture occurs on bedrock in the higher elevations. Similar sediments also occur in the lower elevations but here they are buried beneath glacial stream deposited materials. Glacial lake sediments occur beneath the valley floor. These extend westerly for several miles where

they merge with the more extensive lake deposits of the Connecticut valley. Bedrock is comprised of steeply inclined schistose formations and granite.

B. METHOD AND EXTENT OF EXPLORATIONS.

1. Extent. - Subsurface explorations were accomplished by core borings, test pits and auger borings. These investigations were made (1) to locate and sample rock beneath the abutments and in the spillway area, (2) to determine the character, thickness, and quality of overburden in these areas, (3) to locate the upstream and downstream portals of the stream control tunnel and foundations for pier and wall for access bridge to gate house, (4) to determine ground water conditions, and (5) to investigate borrow areas. Additional explorations involving special sampling techniques and procedures were conducted in the foundation area. A summary of these investigations is given in Table 1, "Summary of Subsurface Investigations" (Page 12). The location of each exploration is shown on Plate No. 4, "Plan of Subsurface Explorations", and Plate No. 5, "Plan of Borrow Areas", and the records on Plate Nos. 6, 7, and 8.

2. Cased borings. - All overburden drilling was done using wash boring and dry sampling methods. Frequent soil samples were obtained by means of suitable sampling spoons. Special precautions were taken to assure that samples were undisturbed by wash water. Numerous undisturbed samples of diameters varying from 2" to 4-7/8" were obtained by the special Providence U.S.E.D. methods and techniques. A large number of the borings, after completion, were equipped as observation wells and fluctuations of the water table recorded over several months. Rock cores were obtained in more than one half of the total number of borings. Pressure and seepage tests were conducted in a few selected borings.

Table 1 - Summary of Subsurface Investigations

Exploration Method	Exploration by	Number of Exploration	Lin. ft. explored Overburden	Lin. ft. explored Rock	Purpose of exploration
Borings (Preliminary)	Contract	8	416.0	103.5	Preliminary borings to outline foundation conditions for definite project report. Accomplished 1936.
Borings	U.S.E.D. (Hired labor)	88*	5357.5	1360.2	To determine foundation conditions, and to locate bedrock for spillway location, and stream control tunnel.
Borings (Borrow Area)	U.S.E.D.	12	569.6	- -	To investigate pervious borrow area in depth and develop lateral extension of impervious materials on east side of spillway approach channel.
Test pits (Foundation)	U.S.E.D.	12	- -	- -	To investigate coarse granular deposits occurring beneath flood plain silts throughout foundation area.
Test pits & Auger borings (Borrow Area)	U.S.E.D.	78**	628.9	- -	To investigate both pervious and impervious borrow areas.

*Includes 13 borings for obtaining special undisturbed samples.

**Includes three deep test pits in right abutment.

3. Test pits and auger borings. - Numerous shallow test pits and auger borings ranging in depth from 3.0 to 15.5 feet were excavated in the pervious borrow area. Two test pits each 17 feet deep were excavated in the impervious borrow-spillway area.

4. Other investigations. - Several sites were investigated in the stretch of valley extending from the finally selected site to about 0.7 miles downstream. Explorations indicate that these alternative sites compared to that finally selected do not have any advantage in foundation conditions and in addition would require considerably more rock excavations for spillway channel.

C. SITE.

1. Right abutment. - The right end of the dam will abut against a hill of compact glacial till extending about 140 feet above the valley bottom. At the southerly end of this hill, in the vicinity of the dam, the bedrock surface is deeply buried. Approximately 900 feet north of the centerline of dam the bedrock surface emerges to form a rock ridge throug out which numerous ledges of a steeply inclined schistose rock occur. The spillway weir and discharge channel will be located at the southern end of this rock ridge, the spillway approach channel being located in till overburden. The glacial sediments most prominently developed are designated as Classes 7, 9, and 11 with occasional strata of Classes 8 and 10. (See description of classes in Table 14, "Providence Soil Classification" on Page 48). The formation contains numerous boulders and is very compact. It will form an impervious abutment as demonstrated by the compact nature of the material and the high water table. The geologic relations are shown on Plate No. 9 "Record of Subsurface Explorations - Profiles and Sections." Rock contours are shown on Plate No. 10.

2. Left abutment. - Bedrock is situated at shallow depth throughout the left abutment providing opportunity for keying the impervious embankment section to rock, and for location of the stream control tunnel and gate tower. The overburden is comprised of a glacial till. Steeply inclined weathered schist is exposed in the brook bed situated about 400 feet downstream of the center line. Further downstream the bedrock surface is deeply buried beneath glacial till.

3. Valley section between abutments. - The overburden beneath the flood plain of the valley is made up of three differing groups of sediment. Directly beneath the flood plain fine sands and silts (Classes 6 and 8) occur relatively pervious loose sands and gravels (Classes 2, 3, 4, 5, and 7) from 15 feet to 20 feet thick. Below this sand and gravel formation is a glacial lake deposit comprised chiefly of soft, cohesionless and interstratified silt and rock flour (Classes 8, 10 and 12). Some of these extremely fine textured materials are laminated and varved forming thin beds having cohesion to a slight degree, but the cohesionless characteristic is by far more prominent. The thickest section of this glacial lake deposit, approximately 95 feet, occurs beneath the downstream toe. Throughout other portions of the foundation the thickness ranges from 30 feet to 50 feet. A massive and compact formation of bedded and non-bedded sediments occurs between the bedrock surface and the soft silt and rock flour deposits. These materials are represented by a variety of textures, some having been deposited by earlier glacial stream or lake action and others as glacial till. All are well consolidated or compacted.

D. NATURE OF EXCAVATIONS.

1. Overburden excavations. - Overburden excavations will be re-

quired for (1) the tunnel intake and approach channels, (2) the spillway approach channel, (3) the cut-off trench and (4) pervious borrow. Construction of the cut-off trench to provide for keying of the impervious embankment section to the underlying glacial lake deposit throughout the flood plain of the valley will require excavation of the sand and gravel formation occurring beneath the flood plain silts and fine sands. About one half of the excavation required will be made below the water table. In the higher ground of both abutments excavations for the cut-off trench will be made in well compacted and relatively impervious glacial till (Classes 7, 9 and 11).

The glacial till overburden of the spillway approach channel consists predominantly of compact and well graded granular materials (Classes 7, 9 and 11) ranging in particle sizes from rock flour to gravel. Numerous cobbles and boulders also occur. These excavations will be carried to a maximum of about 50 feet in depth. All materials excavated from the spillway channel will be used in the impervious or the random impervious embankment sections.

Excavations in the pervious borrow areas will be made chiefly in stratified sand and sand and gravel deposits (Classes 1, 2, 3, 4, 5). Stratifications or beds containing much finer textured materials (Classes 6, 7, 8, 9) also occur, but in minor amount.

2. Rock excavation. - a. General. - Rock excavations will be required for construction of the tunnel, foundations of the gate tower, and the spillway discharge channel. Rock stripping will also be required over the spillway weir foundation and for the embankment cut-off at rock. The rock formation is a fine grained and laminated phyllite, called also phyllitic schist. It is steeply inclined and has a well developed slatey cleavage.

X-ray and petrographic examinations indicate the rock is comprised chiefly of very fine grained quartz and sericite (white) mica. The average strike of bedding, which also is parallel to the slatey cleavage, is N 30° E (magnetic). The tunnel, throughout a considerable portion of its length, will extend across the strike or direction of the schistose strata. In excavation of such a rock formation, this fact and the steep inclined altitude of beds is a desirable condition both as to rock stability and breakage. A system of joints or fractures occur at nearly right angles to the slatey cleavage. Where these occur adjacent to the excavation lines there may be difficulty in holding to the precise line of excavation. During tunnel construction thin rock slabs may become loosened and fall into the excavated space. This behavior however will not be serious, and if encountered at all will be confined to narrow zones where steeply inclined cleavage planes are more pronounced and the rock more easily shattered by blasting operations. The plans provide for installation of steel liner plates, if this condition is encountered.

Due to its slatey character the rock is badly split and weathered. This condition is more prevalent in the upper zone of bedrock, down to a depth ranging between 5 feet and 10 feet. Weathering to greater depth in localized areas is expected. For excavations in the spillway discharge channel it is planned to operate in such a manner as will produce suitable sizes of rock for placement as riprap and dumped rock on the embankment. In order to reduce to a minimum the splitting and shattering of foundation rock beneath the weir, embankment key wall in left abutment, and gate tower structure, the specifications provide that rock stripping be done as much as possible by hand methods, using pneumatic tools. A minimum of blasting by mud capping will be allowed.

b. Excavation methods. - Construction of Claremont Dam involves over 250,000 cubic yards of rock excavation in a formation much different than any so far encountered on Connecticut Valley flood control projects. Whereas, at previously completed projects, constructions have been made in soft and decayed schist formations, hard blocky granites, and steeply inclined quartzite and mica schist, at Claremont, constructions are proposed in a slatey schist or phyllitic schist formation. The problem of controlling rock overbreak, preserving the wall rock of excavations from shattering, and at the same time producing a rock spoil that will be suitable for use as dumped rock and rip-rap, is considered an important one. Four possible rock excavation methods in open cut have been studied both as to practicability and costs. These four methods are briefly outlined.

(1) The first method consists of ordinary rock excavation with sloping sides and 10 foot berms at 30 foot vertical intervals. This method would allow the contractor unrestricted operations, and would produce a considerable quantity of rock overbreak, and much shattering of the wall rock. In addition, the blasted rock available for rock fills would be comprised of shattered slabs and smaller thin slabby pieces and flakes.

(2) The second method consists of ordinary rock excavation (prescribed operations) with sloping sides and 10 foot berms at 30 foot vertical intervals. The contractor's operations are prescribed in that he would be required in blasting to operate in such a manner that shattering of the wall rock is reduced to a minimum and at the same time a rock spoil is produced which would be suitable for rock fills. The specifications and plans provide this as one alternative method of open cut excavation.

(3) The third and fourth methods consist of rock excavation with vertical sides prepared by close drilling and broaching or by means of the wire saw. Before any blasting is done in proximity to the side walls of the excavation, a complete severance of the rock to be blasted from the wall rock to be left in place is required. This rock cutting may be accomplished by either pneumatic means - close drilling and broaching - or by using the wire saw. By providing an initial severance of the rock, and prescribed blasting operations as in (2), a wall rock that is free of shattering effects will be left in place. In anticipation of this condition the sides of open cut excavations have been drawn to be vertical which, compared to rock cuts with sloping sides and berms, contributes considerable reduction in the amount of required rock excavation. Although the unit costs of rock excavations by these methods is greater, the total costs are less than those for excavation methods outlined in (1) and (2). The specifications and plans provide for these two as alternative methods of open cut excavations.

(4) The wire saw method of cutting rock, although perhaps not so well known in engineering construction, is a well known method in the quarrying industry, particularly in the quarrying of slate and the softer building stones, such as Indiana limestone. The method is adaptable to only a limited variety of rock formations of which the rock at the Claremont Damsite is included. A sawing test conducted on a large block of rock from the site indicates the speed of cutting will be comparable to cutting speeds obtained in Pennsylvania slate quarries. The method requires drilling of 36-inch diameter calyx holes, located approximately 60 feet apart, along the sides of the cut. These holes are used to accommodate posts equipped with 20-inch sheaves or wheels through which is threaded an endless 3-strand wire

rope. This cable, powered by an electric motor, drags an abrasive on or through the rock, cutting the rock along a line about 3/8-inches wide. The cuts are made in a size approximating 60 ft. x 12 ft. The wire saw method of excavating in slate quarries is illustrated in Photo No. 5.

E. SUBSURFACE LEAKAGE. - Seepage and pressure testing of borings located in the left abutment and in the spillway weir area indicate potentialities for considerable water circulation. This condition which is more prominent in the upper zone of bedrock, from the bedrock surface to a depth of about 20 feet, will require careful stripping of weathered rock throughout the area delimited by the central impervious core and grouting of bedrock beneath a concrete group cap or cut-off extending along the center line of dam in the left abutment. Careful stripping and grouting of the rock foundation at the site of the weir will also be required. These two measures will provide an effective barrier to water circulation through the left abutment and beneath the spillway weir. Leakage or seepage in the right abutment and in the embankment foundation will be controlled by an earth cut-off, this cut-off being constructed to contact the compact and relatively impervious glacial till in the abutment and the glacial lake deposits beneath the valley bottom.

V. HYDROLOGY AND HYDRAULICS

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A. CLIMATE.

1. Temperatures. - Long, cold winters and short summers are characteristic of the Sugar River basin. Freezing temperatures begin about the end of October and end about the middle of April. Over 160 days with minimum temperatures below freezing may be expected each year, and 15 to 20 days are usually below 0° F.

The summer season, when the maximum daily temperatures are 70° F. or higher, lasts from late May to early September. The maximum temperature rarely rises to 100° F. but average daily maximum temperatures are commonly 20° to 24° F. higher than the average daily minimum temperatures.

In the spring, temperatures above freezing sufficient to cause flood runoff from melting snow may begin at any time during the last 20 days of March. Snow melt is likely to continue, sometimes intermittently, until early May for an average of about five weeks.

The nearest station for which temperature records are available is at Hanover, New Hampshire, approximately 20 miles northwest of the center of the area but somewhat lower in elevation. Over a period of 59 years the temperature at Hanover has ranged from a minimum of minus 40° to a maximum of 101° F. The mean annual temperature is 43.6° F. Mean monthly temperatures are shown in the following table.

TABLE 2. MEAN MONTHLY TEMPERATURES
AT HANOVER, NEW HAMPSHIRE

Month	Mean temp.	Month	Mean temp.	Month	Mean temp.
Jan.	17.7	May	54.9	Sept.	58.8
Feb.	18.7	June	63.6	Oct.	47.4
March	29.1	July	68.7	Nov.	34.5
April	42.5	Aug.	65.9	Dec.	21.7

2. Precipitation. - a. Mean rainfall. - The mean annual precipitation at Newport, New Hampshire, for 15 years of record from 1929 to 1943 is 37.70 inches. This is the only station within the Sugar River watershed with a record of sufficient length to be of value. A list of other stations in the vicinity of the watershed is shown on Plate No. 11 with the corresponding periods of record, and the location of these stations is shown on Plate No. 12. The mean precipitation and mean snowfall (see subparagraph c below) are shown in the following Table 3 for Newport and, in comparison, for Hanover, which has a record much longer than any other station in the region.

TABLE 3. MEAN PRECIPITATION AND SNOWFALL AT NEWPORT AND HANOVER, NEW HAMPSHIRE

Station	NEWPORT, N. H.		HANOVER, N. H.	
Period of record, years	15	13	102	59
	Mean precipitation, Inches	Mean snowfall (unmelted), Inches	Mean precipitation, Inches	Mean snowfall (unmelted), Inches
Jan.	2.96	16.4	2.65	17.1
Feb.	2.20	12.9	2.29	18.4
March	3.52	16.1	2.62	12.2
April	3.37	7.7	2.60	5.2
May	3.11	0.0	3.09	0.0
June	3.65	0.0	3.43	0.0
July	3.64	0.0	3.57	0.0
August	3.22	0.0	3.48	0.0
Sept.	3.47	0.0	3.23	0.0
Oct.	2.67	0.1	3.08	0.2
Nov.	3.21	5.0	2.89	5.7
Dec.	2.68	8.5	2.58	12.5
YEAR	37.70	66.7	35.51	71.3

b. Storms. - Three types of storms are characteristic of this area: (1) continental cyclonic disturbances, (2) hurricanes, and (3) thunderstorms. Continental storms may be of the stationary frontal type or rapidly moving intense cyclones. They are not limited to any season or month but follow one another at more or less regular intervals and with varying intensities throughout the year. This area is so situated that it comes under the influence of most of the cyclonic disturbances which affect the United States. The normal path of hurricanes is to the south and east of New England, but hurricanes may be deflected over this area by continental cyclonic conditions. They are most likely to occur during the summer and autumn months. Thunderstorms may be of local origin, or they may be of the frontal type associated with continental storms during the summer months. A definite combination of meteorological conditions is recognized as being responsible for most of the great flood-producing storms of the eastern United States. They are (1) a persistent high-pressure area over the western Atlantic Ocean, (2) another high-pressure area over the central and northern interior of the continent, and (3) a low-pressure trough between these "highs" including one or more moving centers. The storm of 17-21 September 1938, which is the maximum storm of record in this general region, had these characteristic conditions. Its duration was 98 hours and the maximum average depth of rainfall over an area of 245 square miles was 15.5 inches.

c. Snowfall. - The mean monthly and annual snowfall at Newport and Hanover are shown in Table 3. The maximum snowfall at Newport during any winter season for the period of record is 89.6 inches in 1939-40. Prevailing low temperatures during the winter months usually permit a major

portion of the snowfall to remain on the ground until the spring thaw. Occasionally, however, a realtively short period of melting temperatures occurs during the winter and causes minor flood runoff.

B. RUNOFF.

1. Records. - Stream gaging records have been collected by the U. S. Geological Survey at West Claremont on the Sugar River from October 1928 to date. This gaging station has a drainage area of 269 square miles and is located just downstream from Redwater Brook. Records are obtained by means of a water-stage recorder. This is the only gaging station on the river. Pertinent data from the record of the station are shown in the following Table 4.

TABLE 4. SUMMARY OF STREAM GAGING RECORD FOR SUGAR RIVER AT WEST CLAREMONT, NEW HAMPSHIRE (Adjusted for storage in Sunapee Lake)

Period	Runoff in inches			Mean runoff as % of rainfall
	Mean	Maximum	Minimum	
January	1.30	3.17	0.33	44%
February	0.90	1.88	0.31	41%
March	2.80	11.76	0.49	80%
April	5.86	10.04	2.30	167%
May	2.64	7.05	0.76	85%
June	1.20	3.25	0.21	34%
July	0.65	2.06	0.12	18%
August	0.35	1.07	0.12	11%
September	0.61	5.78	0.04	18%
October	0.46	1.56	0.12	19%
November	1.07	2.72	0.40	36%
December	1.22	4.02	0.44	45%
YEAR	19.06	26.45	9.97	50%

2. Floods. - a. Historical floods. - Through field investigation and research, historical knowledge of several of the most severe floods on the Sugar River was obtained. Prior to October 1928, no definite information is available on flood heights or discharges. The known dates of such floods are:

February 1824	April 1869
February 1839	October 1869
January 1841	April 1870
April 1843	March 1913
March 1859	November 1927

b. Floods of record. - The maximum discharge at West Claremont during the period of record is 14,000 cubic feet per second in March 1936. The September 1938 storm resulted in a peak discharge of 13,100 c.f.s. at this station. These discharges indicate respective runoff rates of 52 and 49 c.f.s. per square mile.

c. Frequency and magnitude. - The following Table 5, based upon an analysis of a 15-year record at the West Claremont gaging station, shows the number of occasions on which floods of different magnitudes have been equaled or exceeded and the maximum daily discharge of record for each calendar month. The instantaneous peak discharge for each occurrence was obviously greater than the daily average used in preparation of the table, but the data on instantaneous peaks are not readily available.

TABLE 5. FLOOD OCCURRENCES - SUGAR RIVER
AT WEST CLAREMONT, NEW HAMPSHIRE
October 1928 to September 1943

Month	Maximum daily average discharge in thousand cubic feet per second							Maximum daily discharge c.f.s.
	1	2	3	4	5	6	10	
	Number of times equaled or exceeded in record							
Jan.	3	1	1	1	0	0	0	4460
Feb.	4	0	0	0	0	0	0	1940
March	14	8	3	2	2	1	1	11200
April	36	19	10	4	3	2	0	6430
May	16	4	2	2	1	1	0	6160
June	8	3	2	1	0	0	0	4040
July	5	1	0	0	0	0	0	2960
August	0	0	0	0	0	0	0	725
Sept.	1	1	1	1	1	1	0	8720
Oct.	1	0	0	0	0	0	0	1250
Nov.	7	1	0	0	0	0	0	2980
Dec.	6	2	1	0	0	0	0	3520
TOTAL	101	40	20	11	7	5	1	----

d. Flood characteristics. - As seen in the foregoing Table 5, floods are most likely to occur during the months of March, April, and May when melting snow is a contributing factor, but they may occur in any month of the year. High river stages are likely to prevail for several weeks during the spring months, and floods at this time of year are accordingly of greater volume for any given peak than those which occur during summer and autumn. Higher peak rates of discharge are more likely to occur as a result of summer and autumn storms.

e. Watershed characteristics. - (1) General. - The total drainage area of the Sugar River is 274 square miles, of which 245 are above the Claremont Dam site. The basin has a maximum length from east to west of 21 miles and from north to south of 25 miles. The watershed is hilly and contains many lakes, ponds, and swampy areas, all of which tend to reduce the intensity of flood runoff. The total area of lakes and ponds is estimated to be 13.7 square miles. Sunapee Lake, with a surface area of 6.6 square miles and a drainage area of 46 square miles at the outlet, is the principal lake and is sufficiently large to necessitate considering it separately in all unit hydrograph and design flood studies. Elevations range from 290 feet at the mouth to over 2700 feet at mountain peaks on both the northwest and the southeast borders of the watershed. The slope of the stream is steep, averaging approximately 29 feet per mile from Sunapee Lake to the dam site and 33 feet per mile from the dam site to the mouth. Except for the town of Claremont below the dam site and for Newport and Sunapee above it, population is very sparse and scattered.

(2) *Infiltration rates.* - *For the second and major* portion of the storm of September 1938, estimates indicate that the infiltration rate from 17 to 21 September was approximately 0.06 inch per hour. During flood-producing storms in the winter and spring, infiltration is considered negligible. For design studies of maximum summer and autumn storms, 0.05 inch per hour is considered the maximum safe rate to allow for infiltration.

C. AREA AND CAPACITY OF RESERVOIR.

Area and capacity curves were determined by planimetering the contours on a map prepared from an aerial survey. Ten-foot contours were used on a scale of 510 feet to the inch. The resulting curves are shown on Plate

No. 14. The capacity data in tabular form follow in Table 6.

TABLE 6. CAPACITY OF CLAREMONT RESERVOIR

Elevation, feet	Capacity, acre-feet	Elevation, feet	Capacity, acre-feet
520	0	590	30700
530	100	600	40900
540	1100	610	52300
550	3500	620	64700
560	7700	630	78300
570	13800	640	92900
580	21600		

D. COMPUTED SPILLWAY FLOOD.

1. Maximum possible rainfall. - a. General. - The computed spillway flood, as defined in Engineer Bulletin R & H No.9, 1938, is "the flood computed by means of the forecast worst storm, the highest runoff factors and the worst runoff-producing combinations for the drainage area in question". The maximum possible precipitation as recorded in Hydrometeorological Report No. 1 was adopted as the basis for the forecast worst storms, it being stated in the Report that these data are applicable to the Connecticut River Basin above Northampton, Massachusetts. Both the maximum possible rainfall for summer and autumn and the maximum possible rainfall plus snow melt in winter and spring were studied to determine which condition might produce the maximum rate of flood discharge. It was found that the high rates of rainfall possible only during the summer and autumn would produce a higher rate of flood discharge than the more moderate winter and spring rains combined with snow melt, even though the volume of runoff for the winter and spring conditions is greater. The requirements of "highest

runoff factors and worst runoff-producing combinations" were met by allowing a maximum infiltration rate of 0.05 inch per hour during the summer and autumn and no infiltration during the winter and spring. Also the precipitation was arranged according to the storm pattern which would produce the maximum rate of runoff, viz., with the maximum intensities occurring near the middle of the storm period. Studies of storm rainfall in New England have shown that such a storm pattern is likely to occur in this area.

b. Summer and autumn. - The maximum possible rainfall in 48 hours, as interpolated from Table VIII, page 84 of Hydrometeorological Report No. 1, is 16.6 inches for a drainage area of 245 square miles and 16.9 inches for 199 square miles. For determining runoff at the Claremont Dam site, the worst conditions of rainfall occur from the forecast worst storm of 16.9 inches in 48 hours over the net drainage area of 199 square miles (excluding Sunapee Lake and hereinafter referred to only as the net drainage area), and a synchronous storm over the Sunapee Lake drainage area of 46 square miles, such that the total rainfall is equal to the forecast worst storm of 16.6 inches in 48 hours for the gross drainage area of 245 square miles. The forecast worst storms for net and gross drainage areas and the resulting forecast worst storm for the Sunapee Lake drainage area are shown in Table 7. Deduction of infiltration at 0.05 inch per hour gives a maximum possible rainfall-excess of 14.7 inches in 36 hours.

TABLE 7. - FORECAST WORST STORM WITHOUT SNOW

(Infiltration estimated at 0.05 inch per hour)

6-hour periods	Net D. A. (199 sq. mi.)		Sunapee D. A. (46 sq. mi.)		Total D. A. (245 sq. mi.)	
	Precipitation, inches	Runoff, inches	Precipitation, inches	Runoff, inches	Precipitation, inches	Runoff, inches
1	0.10	0.00	0.10	0.00	0.10	0.00
2	0.25	0.00	0.25	0.00	0.25	0.00
3	2.00	1.70	2.00	1.70	2.00	1.70
4	9.10	8.80	7.50	7.20	8.80	8.50
5	4.00	3.70	4.00	3.70	4.00	3.70
6	1.10	0.80	1.10	0.80	1.10	0.80
7	0.25	0.00	0.25	0.00	0.25	0.00
8	0.10	0.00	0.10	0.00	0.10	0.00
TOTALS	16.90	15.00	15.30	13.40	16.60	14.70

c. Winter and spring. - The maximum possible rainfall plus snow melt in 48 hours, as interpolated from Table VII, page 80 of Hydrometeorological Report No. 1, is 23.3 inches for a drainage area of 245 square miles and 24.3 inches for 199 square miles. For determining runoff at the Claremont Dam site, an analysis of the worst conditions of rainfall and snow melt was made as described in subparagraph b. Forecast worst storms for winter and spring are shown in Table 8 of this analysis. The rainfall-excess is equal to the sum of rainfall and snow melt, or 23.3 inches in 48 hours.

TABLE 8. FORECAST WORST STORM WITH SNOW RELEASE
(No infiltration assumed)

6-hour periods	Net D. A. (199 sq. mi.) Runoff, inches	Sunapee D. A. (46 sq. mi.) Runoff, inches	Total D. A. (245 sq. mi.) Runoff, inches
1	2.0	1.6	1.8
2	2.4	1.9	2.3
3	3.3	2.6	3.1
4	5.9	4.6	5.7
5	3.9	3.1	3.8
6	2.6	2.1	2.6
7	2.2	1.7	2.2
8	2.0	1.5	1.8
Totals	24.3	19.1	23.3

2. Unit hydrographs. - The unit hydrographs adopted for deriving the computed spillway flood are those approved by the Office, Chief of Engineers (Subject: Hydrology report - Claremont Dam, 3rd Ind., 17 January 1941), and are shown in the following Table 9 and plotted on Plate No. 15.

TABLE 9. ADOPTED UNIT HYDROGRAPHS

Time, hours	Net D. A. (199 sq. mi.) discharge, c.f.s.	Sunapee D.A. (46 sq. mi.) discharge c.f.s.	Time, hours	Net D. A. (199 sq. mi.) discharge, c.f.s.
0	00	0	63	580
3	200	580	66	520
6	700	1280	69	460
9	1500	1620	72	400
12	2900	1380	75	340
15	4300	940	78	300
18	4700	670	81	260
21	4400	550	84	220
24	3700	470	87	180
27	3000	420	90	150
30	2400	370	93	110
33	1900	330	96	80
36	1600	290	99	50
39	1400	250	102	30
42	1250	210	105	0
45	1100	170		
48	1000	140		
51	900	110		
54	800	70		
57	720	40		
60	650	0		

3. Computed spillway flood hydrograph. - The natural computed spillway flood at the dam site was determined by adding the routed outflow of the flood at Sunapee Lake to the flood evaluated by applying the adopted unit hydrograph for the net drainage area to the forecast worst storm for that area (see Tables 7 and 9), and adding a base flow of 400 c.f.s. The Sunapee routed outflow results from the flood evaluated by applying the adopted unit hydrograph for the Sunapee drainage area to the forecast worst storm for that area, and routed through the surcharge storage above the maximum operating stage of Sunapee Lake, and through the natural valley storage between Sunapee Lake and the dam site. The resulting flood at the dam site is shown in the following Table 10 and plotted on Plate No. 16.

TABLE 10. COMPUTED SPILLWAY FLOOD
AT CLAREMONT DAM SITE

Time, hours	Discharge, c.f.s.	Time, hours	Discharge, c.f.s.	Time, hours	Discharge, c.f.s.
0	400	45	27670	90	6220
3	740	48	24590	93	5510
6	1590	51	21910	96	4910
9	4990	54	20020	99	4230
12	12090	57	18180	102	3690
15	22650	60	16600	105	3140
18	37900	63	15180	108	2730
21	53230	66	13860	111	2220
24	61540	69	12670	114	2080
27	63930	72	11550	117	1890
30	59700	75	10500	120	1820
33	52950	78	9520	123	1750
36	45090	81	8500	126	1700
39	37620	84	7710		
42	32050	87	6920		

E. SPILLWAY DESIGN FLOOD.

1. General. - The spillway design flood is a flood having the same volume as the computed spillway flood but a peak rate of discharge 50% greater. This percentage represents a factor of safety with respect to rate of discharge to cover any rate greater than that of the computed spillway flood which could possibly result from non-uniform distribution of the maximum rainfall on the watershed. To compute the spillway design flood at the dam site, the same rainfall-excess is used as for the computed spillway flood, viz., 14.7 inches in 36 hours plus the same base flow of 400 c.f.s. The method of computation differs from that used for the computed spillway flood, however, in that a distorted 6-hour unit hydrograph or distribution graph is applied to the two maximum 6-hour periods of rainfall-excess. This distorted unit hydrograph represents a distribution of runoff at the dam site that might occur if one inch of rainfall-excess were concentrated over the widest portion of the watershed. Its shape is determined by peaking the normal 6-hour unit hydrograph sufficiently to make the peak rate of discharge of the spillway design flood 50% greater than for the computed spillway flood.

2. Peaked unit hydrographs. - The peaked unit hydrographs adopted for deriving the spillway design flood are those approved by the Office, Chief of Engineers (Subject: Hydrology report - Claremont Dam, 3rd Ind., 17 January 1941), and are shown in the following Table 11 and plotted on Plate No. 15.

TABLE 11. PEAKED UNIT HYDROGRAPHS

Time, hours	Net D. A. (199 sq. mi.) discharge, c.f.s.	Sunapee (46 sq. mi.) discharge, c.f.s.	Time, hours	Net D. A. (199 sq. mi.) discharge, c.f.s.	Sunapee (46 sq. mi.) discharge, c.f.s.
0	0	0	36	1080	190
3	850	920	39	930	160
6	2400	2130	42	780	140
9	5550	2050	45	620	120
12	8200	1000	48	520	90
15	6650	700	51	400	60
18	4180	540	54	310	40
21	2980	450	57	200	20
24	2290	370	60	120	0
27	1850	300	63	70	
30	1520	250	66	0	
33	1300	220			

3. Spillway design flood hydrograph. - The natural spillway design flood at the dam site has a peak discharge of 92,000 c.f.s. The discharge rates are shown in the following Table 12 and are plotted on Plate No. 16.

TABLE 12. SPILLWAY DESIGN FLOOD
AT CLAREMONT DAM SITE

Time, hours	Discharge, c.f.s.	Time, hours	Discharge, c.f.s.	Time, hours	Discharge, c.f.s.
0	400	42	25060	84	3840
3	740	45	21960	87	3620
6	1790	48	19080	90	3420
9	11030	51	16350	93	3210
12	27550	54	14350	96	3030
15	61200	57	12260	99	2840
18	92000	60	10710	102	2680
21	89600	63	8960	105	2490
24	77320	66	7590	108	2420
27	60930	69	6460	111	2290
30	46270	72	5260	114	2160
33	38580	75	4760	117	2040
36	33180	78	4280	120	1920
39	29080	81	4060		

4. Investigation of winter storm. - A natural spillway design flood for the forecast worst storm with snow cover was similarly computed from the runoff given in Table 8. The maximum spillway discharge resulting from this flood is less than that for the spillway design flood computed from the forecast worst storm without snow. Consequently, the flood determined from the summer storm governs the determination of the size of spillway.

F. OUTLET DESIGN FLOOD.

1. General. - The outlet design flood is a hypothetical flood

greater than any of record but of probable though very rare occurrence. A rainfall-excess of 9.08 inches on 199 square miles, or 9.00 inches on 245 square miles, in 60 hours was adopted as the basis for computing the outlet design flood. A storm pattern having maximum intensity at the middle of the storm period was assumed. The flood has a probable freuency of 100 years, based upon volume frequency studies for the Connecticut River Basin.

2. Outlet design flood hydrograph. - Use of the average rainfall by 6-hour periods and the adopted 6-hour unit hydrographs, described in connection with the computed spillway flood, plus a base flow of 2 c.f.s. per square mile produced the hydrograph of the outlet design flood at the dam site, shown on Plate No. 16. The peak rate of discharge is 26,200 c.f.s.

(See Page 37 for Table 13)

TABLE 13. OUTLET DESIGN FLOOD
AT CLAREMONT DAM SITE

Time, hours	Discharge, c.f.s.	Time, hours	Discharge, c.f.s.	Time, hours	Discharge, c.f.s.
0	490	60	23050	120	3030
3	530	63	21300	126	2500
6	620	66	19340	132	2100
9	960	69	17300	138	1840
12	1610	72	15370	144	1670
15	2480	75	13550	150	1560
18	4060	78	12060	156	1510
21	5910	81	10740	162	1470
24	8050	84	9690	168	1430
27	10560	87	8750	174	1380
30	13210	90	7980	180	1350
33	16080	93	7250	186	1320
36	18910	96	6600	192	1250
39	21650	99	6000	198	1170
42	23780	102	5470	204	1060
45	25410	105	4950	210	930
48	26100	108	4510	216	770
51	26220	111	4060	222	630
54	25590	114	3700	228	550
57	24550	117	3320	234	500

G. VALLEY STORAGE.

1. General. - Valley storage is defined as the volume of natural storage occupied by a stream flowing in its natural channel, including both the normal channel and the overbank or flood channel. That part of the natural storage which is above the low-water surface of the stream is effective in regulating the flow, being alternately filled and drained by the rising and falling stages of the stream. For any reach of a stream, this effective valley storage acts very much like reservoir storage. It stores water as the stage rises, thus reducing the rates of outflow from the reach below the rates of inflow. Conversely, it releases water as the stage falls, thereby increasing the rates of outflow above the rates of inflow.

2. Computation of effective valley storage. - Effective valley storage within the reservoir basin was computed for various rates of discharge at the dam site up to that of the spillway design flood. This was accomplished by determining the surface profile for each rate of discharge and computing the volume under each profile by the end-area method. The profile of the May 1940 flood was drawn from high-water marks, and profiles for three design floods were determined by means of the tailwater rating curve and approximate rating curves constructed at several points by backwater studies. The floods used were the outlet design flood, the computed spillway flood, and the spillway design flood. The results are expressed on Plate No. 17 in a form suitable for use in flood routing. Valley storage is plotted against elevation for each of the flood profiles investigated. The total valley storage below spillway crest is plotted against discharge at the dam site on Plate No. 18 for use in reverse-routing as stated below.

3. Use of valley storage in flood routing. - The effective storage capacity of a reservoir to impound the waters of any given flood is the difference between the total volume behind the dam, up to the level of the flow line, and the volume which would be occupied by valley storage in the same reach of river if the dam did not exist at all. The larger the flood, the greater is the valley storage and consequently the smaller is the net or effective reservoir storage. In routing the design floods through the reservoir, valley storage was taken into account by one of two methods as follows:

a. For routing the outlet design flood or other floods that will utilize only the storage below the spillway crest, it was found most convenient and accurate to route the flood "at the dam site" through net reservoir storage, obtained by deducting the valley storage curve from the capacity curve. The condition is then as if two reservoirs were in series, the upper containing only the valley storage and the lower containing only the net reservoir storage. The flood hydrograph "at the dam site" represents the flood as it would pass from the upper into the lower reservoir. This method truly represents the conditions that prevail when a flood is routed through a reservoir which is empty at the beginning of the flood.

b. For routing the spillway design flood through surcharge storage, the method found most accurate is to reverse-route the design flood "at the dam site" through valley storage below the spillway crest, and to route the resulting inflow flood through net surcharge storage.

H. SPILLWAY HYDRAULICS.

1. Criteria for spillway design. - The spillway must be adequate

to pass the spillway design flood with no possibility of overtopping the dam under the following conditions:

a. The reservoir filled to spillway crest at the beginning of the spillway design flood.

b. The outlet gates closed at the beginning of the spillway design flood and remaining closed for its entire duration.

c. The maximum possible wind and wave action concurrent with the maximum spillway discharge.

2. Spillway discharge coefficient. - The spillway for the Claremont Dam will be a concrete overflow type with an ogee-shaped crest designed to fit the underside of the nappe when the spillway is operating at maximum design surcharge and with a coefficient of discharge of 3.90. Discharge over the spillway was computed by means of the formula

$$Q = CLH^{3/2}$$

in which Q = discharge in cubic feet per second,

C = coefficient of discharge,

L = length of spillway crest in feet,

H = head on spillway crest in feet.

The head is measured to the surface elevation of the pool and therefore includes consideration of the velocity of approach. The economic maximum surcharge was determined (see subparagraph 4) to be 15 feet. For heads less than 15 feet the coefficient of discharge was reduced by ratios, published by recognized authorities from 3.90 to a minimum of 3.09 at zero head. The rating curve for the adopted spillway and the curve showing variations in the coefficient of discharge are shown on Plate No. 19.

3. Determination of inflow flood. - As stated in Paragraph

G3b, the spillway design flood "at the dam site" was reverse-routed through total valley storage below spillway crest in order to obtain the hydrograph representing the inflow of the spillway design flood to a full reservoir. The inflow flood has a peak discharge of 92,500 c.f.s. and is plotted on Plate No. 16.

4. Surcharge-length and discharge-length relations. - The spillway design inflow flood was routed through net surcharge storage for various lengths of spillway to determine the maximum surcharge and discharge corresponding to each length. The results are plotted on Plate No. 20. Cost studies and physical considerations resulted in the determination of 15 feet to be the economic maximum surcharge. The surcharge storage, for a surcharge of 15 feet, is 22,200 acre-feet, equivalent to 1.7 inches of runoff from the gross drainage area. Head loss due to friction in the approach channel at maximum surcharge was computed at 0.13 foot. A surcharge of 14.85 feet was found to result from a crest length of 395 feet, and this length was adopted for the final design.

5. Freeboard. - Freeboard is defined as the difference in elevation between the maximum surcharge elevation and the top of the dam. Its function is to avoid any possibility of the dam being overtopped by wind or wave action when the spillway is operating at maximum surcharge. No allowance is made for frost action, since the spillway design flood can be expected only during the late spring, summer, or autumn months.

a. Maximum wave height. - Maximum wave height was estimated to be 3.45 feet by the formula

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

in which h = vertical wave height in feet,

V = wind velocity, taken as 60 miles per hour,

F = fetch, taken as 3 miles.

b. Ride-up. - Wave height plus ride-up was computed as 1.4 times the wave height alone or 4.83 feet.

c. Set-up. - Wind set-up was computed to be 0.33 foot by the formula

$$S = \frac{V^2 F \cos A}{800D}$$

in which S = set-up in feet,

V & F = same as for wave height,

D = mean depth of water, taken as 115 feet,

A = angle between wind and fetch, taken as zero.

d. Selected freeboard. - The reservoir will extend southeast from the dam, while the prevailing winds are from the northwest and west. Therefore, maximum wave heights and wind set-up are less likely to occur at Claremont Dam than at a dam where the maximum fetch toward the dam parallels the direction of prevailing winds. The allowance for wind set-up was therefore considered negligible, and the adopted freeboard above maximum surcharge is equal to 5.0 feet.

I. OUTLET HYDRAULICS.

1. Criteria for outlet design. - The capacity of the outlet must be adequate to permit the following operations:

a. Partial release of the runoff from the outlet design flood in such manner as to obtain the greatest flood reduction at downstream damage centers by utilizing all the storage but not permitting the pool elevation to exceed the spillway crest.

b. Passing minor floods which do not produce damage, without utilizing more than a minor portion of the storage capacity of the reservoir.

c. Passing the streamflow during construction, up to a size of flood having an average recurrence interval of 10 years for the construction season, without permitting the pool elevation to rise above a level which can be practicably and economically provided for during construction.

2. Retarding basin discharge. - If an outlet has just sufficient capacity to pass the outlet design flood with the maximum pool elevation at the spillway crest, the discharge capacity at crest elevation is defined as the retarding basin discharge. It is determined by trial routing of the outlet design flood "at the dam site" through net reservoir storage and various assumed outlet capacities until the capacity is found which will just produce a maximum pool elevation at the spillway crest. The retarding basin discharge as computed by this method for Claremont Dam is equal to 5,600 cubic feet per second.

3. Design discharge. - The retarding basin discharge multiplied by a flexibility factor gives the design discharge. The outlet will consist of a 16-foot tunnel controlled by three 6-foot by 12-foot gates. The discharge capacity of such an outlet with the pool at spillway crest elevation is 10,200 cubic feet per second. The flexibility factor for the outlet is therefore equal to 1.8. This arrangement of gates satisfies the condition that the outlet must be capable of releasing 80 percent of the design discharge with one gate closed. The outlet capacity with only two gates operating and the pool at spillway crest elevation will be 9400 cubic feet per second.

4. Control of the outlet design flood. - a. With all gates open, the 16-foot tunnel will pass the outlet design flood with a maximum pool

elevation of 616.2 feet and a maximum discharge rate of 9470 c.f.s. This flow is in excess of the safe channel capacity through the town of Claremont. By regulating the gates to permit a maximum discharge of 9000 c.f.s., thereby incurring only very minor damage in Claremont, the outlet design flood can be passed with a maximum pool elevation of 616.7 feet. Under these conditions, the time required to drain the pool after the occurrence of the peak inflow rate will be 7.5 days.

b. With one gate remaining closed, the tunnel will pass the outlet design flood with a maximum pool elevation of 618.8 feet and a maximum discharge rate of 8730 c.f.s. The pool can be drained in 8.25 days after the occurrence of the peak rate of inflow.

c. By regulating the gates to permit a maximum discharge of 9000 c.f.s. two outlet design floods, with their inflow peaks spaced eight days apart, can be passed with maximum pool elevations of 616.7 feet for the first flood and 619.7 feet for the second flood. The time required to drain the pool will be 7.75 days after the inflow peak of the second flood.

5. Stream diversion during construction. - The flood of May 1937 had a magnitude corresponding to an average recurrence interval of ten years for the period May to October inclusive, and was therefore adopted as a construction flood. The outlet selected for Claremont Dam would have passed this flood with a maximum pool elevation of 550 feet, or 25 feet above the sill of the gates. The peak rate of inflow of the construction flood is 5200 cubic feet per second, and the maximum discharge through the outlet would be 3650 cubic feet per second. The resulting tailwater elevation, determined as described in the following subparagraph, would be 528.7 feet above mean sea level.

6. Tailwater rating curve. - The tailwater rating curve has been developed for a point approximately 800 feet below the center line of the dam. The low-water range was determined by comparison of stage relations at the dam site with those at the West Claremont gaging station for the period of the September 1938 flood. The high-water curve was obtained by means of backwater computations based upon measured valley cross sections. The curves are shown on Plate No. 21.

VI. LABORATORY AND FIELD INVESTIGATIONS OF SOILS FOR EMBANKMENT AND FOUNDATIONS

VI. LABORATORY AND FIELD INVESTIGATIONS OF SOILS FOR EMBANKMENT AND FOUNDATIONS

A. SCOPE. - Soils from excavations and borrow areas for embankment construction are covered in this section. Major soils problem at this site concerns a deposit of soft varved silt in valley foundation which is discussed in detail in Appendix A, entitled "Foundation Treatment with Drain Wells and Relief Well". For continuity bulk of soil properties of foundation silt are covered in Appendix A in conjunction with analysis of the problem and are only briefly mentioned in this section.

B. CLASSIFICATION OF MATERIALS.

1. The Providence District has adopted a convenient system of soil classification having standardized terms. In this classification, soils are described by name plus a grain-size number which latter is a function of position and shape of grain-size curve. Grain-size numbers are divided into 16 classes as shown graphically on Plate No. 22 and described in Table 14.

2. The particular features of this system are: (1) it recognizes that accumulations of sediments in nature may be either slight mixtures of various grain sizes relatively uniform in this respect, of intergradations of these types, (2) it affords a means of rapidly estimating relative permeabilities, (3) it aids in showing more accurately the distribution and geologic relations of the overburden, especially of those portions composed of non-uniformly graded materials, and (4) it contributes to a more reliable classification of soils in the field prior to laboratory confirmation.

3. Even numbers are used to designate materials of uniform

texture. Odd numbers designate materials of variable texture. The sizes for gravel, sand, silt and clay are the same as those in the M.I.T. classification with the single exception that size demarcation between fine silt and coarse clay is not rigidly held at 0.002 mm. but is allowed to vary from 0.006 mm. to 0.0006 mm. This flexibility is introduced in recognition of the fact that natural depositions of fine-textured sediments, such as glacial silt (rock flour) behave differently from clays despite similarity of grain-size distribution.

4. In compilation of geologic sections for study purposes, grain-size numbers often are used alone as an abbreviated notation in substitution for a more complete description of the soil as recorded in logs of the explorations. Subscript "$_v$" following a grain-size number indicates visual classification. Use of double class number as (4-2) indicates that the coarser part of the grain-size curve lies in Class 4 range and the finer lies in the Class 2 range - in this case a very uniform sand. In contrast, Class 2-4 represents a much more variably graded sand. In using double class numbers, even numbers are never used in combination with odd numbers.

C. GRAIN-SIZE ANALYSIS. - Grain-size curves of samples were obtained by means of sieve and hydrometer analyses. The materials were carefully classified, and their occurrence in the dam foundation is shown on Plate No. 23. Typical gradation curves of soils proposed for the various embankment types are shown on Plate No. 24.

D. WATER CONTENT AND VOID RATIO.

1. Water contents and void ratios were determined on 2-inch, 3-inch, and 5-inch undisturbed cylinder samples of interstratified silt

PROVIDENCE DISTRICT SOIL CLASSIFICATION

CLASS	DESCRIPTION OF MATERIAL
1	Graded from Gravel to Coarse Sand. – Contains little medium sand.
2	Coarse to Medium Sand. – Contains little gravel and fine sand.
3	Graded from Gravel to Medium Sand. – Contains little fine sand.
4	Medium to Fine Sand. – Contains little coarse sand and coarse silt.
5	Graded from Gravel to Fine Sand. – Contains little coarse silt.
6	Fine Sand to Coarse Silt. – Contains little medium sand and medium silt.
7	Graded from Gravel to Coarse Silt. – Contains little medium silt.
8	Coarse to Medium Silt. – Contains little fine sand and fine silt.
9	Graded from Gravel to Medium Silt. – Contains little fine silt.
10	Medium to Fine Silt. – Contains little coarse silt and coarse clay. Possesses behavior characteristics of silt.
10C	Medium Silt to Coarse Clay. – Contains little coarse silt and medium clay. Possesses behavior characteristics of clay.
11	Graded from Gravel or Coarse Sand to Fine Silt. – Contains little coarse clay.
12	Fine Silt to Clay. – Contains little medium silt and fine clay (colloids). Possesses behavior characteristics of silt.
12 C	Clay. – Contains little silt. Possesses behavior characteristics of clay.
13	Graded from Coarse Sand to Clay. – Contains little fine clay (colloids). Possesses behavior characteristics of silt.
13C	Clay. – Graded from sand to fine clay (colloids). Possesses behavior characteristics of clay.

occurring in valley foundation of dam. Typical results are shown on Plate No. 25.

2. Water contents and void ratios were also determined for the glacial till in right abutment, Borrow Area "B". This till was sampled by driving a 2-1/2 inch I.D. spoon, which often yields samples of reasonable natural void ratio in cohesive materials. The best of these soil cores were paraffined in the field to maintain moisture content, and tested in the District Laboratory for void ratio and water content. Although such paraffined cores are not true undisturbed samples, they are believed to give reasonably close approximations to natural conditions, and are so quoted in logs of the holes. Typical results are shown on Plate No. 23.

E. COMPACTION.

1. Many compaction tests were run on representative samples of soils proposed for embankment use. For impervious materials, an impact test was used, the standard Proctor Compaction Test. Typical compaction curves are shown on Plates Nos. 26 and 27, and compaction properties of borrow materials are summarized in Table 15 (Page 51), entitled "Summary of Materials Available and Required".

2. For cohesionless materials a value termed feasible maximum density, was determined by means of the Providence Vibrated Density Test in which a vertical load of 800 pounds is maintained by a calibrated spring on a sample in a 6-inch diameter cylinder, and compaction obtained by means of vibrating side of cylinder with a hammer (60 - 100 blows) until further compaction is negligible. The advantage of this test is that with cohesionless materials it produces compaction results more reasonably equal

to those easily obtained in the field. Further advantages of this type of test are that it does not break down soil grains as occurs from an impact type test and it is performed more quickly than the impact type. Results of compaction tests are summarized on Table 15, entitled "Summary of Materials Available and Required".

3. The Providence District has found it desirable to compute degree of compaction of pervious soils (sands and gravels) in relation to maximum and minimum densities of these soils. Degree of compaction is expressed as "Density Ratio", which is defined as follows:

$$\text{Density Ratio} = D_r = \frac{d_n - d_0}{d_{100v} - d_0}$$

Where d_n = dry density, pcf, in natural ground or in the embankment

d_0 = dry density, pcf, loosest state from laboratory test for minimum density, placed dry

d_{100v} = dry density, pcf, densest state from Providence Vibrated Density Test

Thus, for d_n = 110 pcf, d_0 = 100 pcf, d_{100v} = 120 pcf, Density Ratio = D_r = 0.50. On basis of percentage of compacted test density, this corresponds to $\frac{110}{120} \times 100 = 92\%$ compaction for this particular example.

F. PERMEABILITY.

1. Permeability tests were run on remolded samples using de-aired water. Results on pervious materials are expressed as permeability-void ratio curves. Results on remolded impervious materials have generally been expressed as permeabilities at or near compacted state. All permeabilities are expressed as coefficient of permeability, k, at 10°C (50°F)

TABLE NO. 15

SUMMARY OF MATERIALS AVAILABLE AND REQUIRED

THIS TABLE IS BASED ON QUANTITIES UNDER SCHEDULE "A"

ESTIMATE NO. 3
JANUARY 1945

TYPE	MATERIALS REQUIRED: EMBANKMENT MEASURE CU YDS	SHRINKAGE AND LOSS FACTOR (A)	EXCAVATION MEASURE CU YDS	MATERIALS AVAILABLE - EXCAVATION MEASURE: SOURCE	EST. TOTAL AMOUNT USABLE CU YDS	SOIL PROPERTIES: TYPE	CLASS	K X 10^{-4} CM./SEC.	ϕ	COMPACTION CHARACTERISTICS: OPT. %	DCW-PCF	PROV. VIBRATED DENSITY PCF	ESTIMATED MINIMUM TO EXCAVATE: TOTAL CU YDS	USABLE CU YDS	WASTE CY YDS	REMARKS
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)
IMPERVIOUS (M)	510,000	1.00	510,000	SPILLWAY CHANNEL & CHUTE, REQUIRED EXCAVATION	330,000	TILL	9, 11, 13	0.007-0.036	27°-40°	9.5-14.9	120-132	-	330,000(E)	330,000	0	STRIPPING INCLUDED IN (RM) MATERIAL WHICH OVERLIES (M).
				SPILLWAY CHANNEL EXTENSION	200,000/	DO	DO	DO	DO	DO	DO		DO (B)	DO	0	AVAILABLE IF NEEDED.
				TUNNEL INLET AND OUTLET CHANNELS	180,000	DO	DO	0.07 -0.104	27°-32°	7.4-10.4	125-135	-	180,000(E)	180,000	0	DO
					710,000/									510,000		
RANDOM IMPERVIOUS (RM)	294,000	0.95	310,000	FLATTENING SLOPE, RIGHT ABUTMENT - UPSTREAM SIDE.	80,000	TILL	7, 9	0.05 - 0.5	27°-32°	7.4-10.4	125-135	-	80,000(E)	80,000	48,000 (STRIPPING)	
				SPILLWAY CHANNEL & CHUTE, REQUIRED EXCAVATION	140,000-	DO	DO	DO	DO	DO	DO	-	160,000(E)	140,000	20,000 (STRIPPING)	
				SPILLWAY CHANNEL EXTENSION	150,000/	DO	DO	DO	DO	DO	DO	-	DO (B)	DO	DO (STRIPPING)	AVAILABLE IF NEEDED
				TUNNEL INLET AND OUTLET CHANNELS	100,000	DO	DO	DO	40°	±0.0%	±25	-	100,000(E)	100,000	0	STRIPPING INCLUDED IN (RP) MATERIAL.
				DO	10,000	SILT	8, 10	-	-	-	-	-	10,000(E)	0	10,000	SILT PROBABLY TOO WET TO COMPACT READILY IN DAM EMBANKMENT
	16,000	0.80	20,000	BORROW AREA "A"	80,000	SILTY GRAVEL	<-3	0.1-1.0	33°-40°	-	-	-	35,000	30,000	5,000 (STRIPPING)	
	116,000(1)	1.0	116,000(1)	ROCK EXCAVATION	116,000/(1)	SOFT ROCK FINES	-	-	-	-	-	-		116,000/(1)	55,000	(1) FROM ROCK EXCAVATION - SOFT MATERIAL USABLE AS SOIL - SEE REMARKS UNDER DUMPED ROCK. MAXIMUM AMOUNT AVAILABLE - USE (RP) IF ACTUAL QUANTITY FALLS SHORT.
	426,000(E)		446,000		656,000/									446,000-		
RANDOM PERVIOUS (RP)	366,000(E)	0.85	430,000	CUT-OFF TRENCH	53,000	GRAVELLY SANDS: SANDY GRAVELS: MED. TO FINE SANDS	2,3,4,5,6	1.0-10.0	35°	-	-	115-120	53,000(E)	53,000	0	INCLUDES EXCAVATION FOR TOE TRENCHES, AND DIVERSION CHANNEL. STRIPPING INCLUDED UNDER TOPSOIL.
				BORROW AREA "A"	1,600,000	DO	1-5, 6-4, 4-2, 1-3, 4	1.0-10.0-	33°-40°	-	-	115-120	393,000(B)	358,000	35,000 (STRIPPING)	
				OUTLET WORKS	15,000								25,000(E)	15,000	10,000 (STRIPPING)	
				SPILLWAY CHANNEL CHUTE	4,000								6,000(E)	4,000	2,000 (STRIPPING)	
					1,672,000									450,000	47,000	
PERVIOUS (P)	1,145,000	0.85	1,350,000	BORROW AREA "A"	585,000	CLEAN SAND	2,4,4-2	25 - 300	34°-39°	-	-	115-125	480,000(B)	450,000	30,000 (STRIPPING)	
					625,000	SANDY GRAVEL	1,3,1-3	5 - 110	36°-44°	-	-	140-150	525,000(B)	490,000	35,000 (STRIPPING)	
					1,150,000	GRAVELLY SAND	2,3,7-3, 5-3	40 - 160	32°-40°	-	-	117-148	435,000(B)	410,000	25,000 (STRIPPING)	
					2,360,000									1,350,000	90,000	
					- 660,000	DEDUCTED FOR USE AS BEST PERVIOUS										
					1,700,000											
BEST PERVIOUS (P)	572,000	0.85	670,000	BORROW AREA "A"	1,150,000 (SEE PERVIOUS FOR OTHER USE OF THIS MATL)	GRAVELLY SAND	2,3,7-3 5-3	40 - 160	32°-40°	-	-	117-148	710,000(B)	670,000	40,000 (STRIPPING)	
GRAVEL BEDDING	94,000	0.85	110,000	BORROW AREA "A"	AMPLE FROM COARSE SANDY GRAVEL ABOVE	COARSE SANDY GRAVEL	1,3,1-3	4 - 110	36°-44°	-	-	140-151	118,000(B)	110,000	8,000 (STRIPPING)	
DUMPED ROCK	262,000	1.3 (BULKING FACTOR)	202,000	INLET CHANNEL	32,000	VERY FINE GRAINED QUARTZ MICA SCHIST WITH VERY STRONG CLEAVAGE	-	-	-	-	-	-	344,000	202,000(C) (HARD FOR DUMPED ROCK)	(10,000) (TUNNEL EXCAVATION)	APPROX. EST. DISPOSITION OF ROCK EXCAVATION. 344,000 CY TOTAL. EXCAVATION MEASURE 202,000 CY HARD ROCK FOR DUMPED ROCK AND RIPRAP. NOT AT PRESENT DETERMINABLE WHETHER AVAILABLE. 142,000 CY EXCESS, EXCAV. MEASURE. 10,000 CY TUNNEL EXCAV. - WASTE. 132,000 ASSUME BULK SUITABLE TO ROLL IN RANDOM IMPERVIOUS. 116,000(1) USED IN RANDOM IMPERVIOUS. 16,000 CY WASTE 10,000 CY TUNNEL WASTE 26,000 CY TOTAL WASTE
				OUTLET CHANNEL	20,000									116,000/(C)(1) (SOFT, ROLLED IN AS SOIL)	(16,000) (NOT SUITABLE TO ROLL AS SOIL)	
				TUNNEL	10,000									318,000	26,000	
				SPILLWAY APPROACH CHANNEL	45,000											
				(C) SPILLWAY CHUTE	192,000											
				HIGHWAY	39,000											
				OTHERS	4,000											
					344,000											
TOPSOIL	50,000	0.80	62,000	DAM STRIPPING	72,000	TOPSOIL	-	-	-	-	-	-	72,000(E)	62,000	10,000	
ACCESS ROAD FILL	33,000	0.85	39,000	BORROW AREA "A"	AMPLE	GRAVELLY SAND	3,7-3, 5-3	39 - 170	32°-40°	-	-	117-148	39,000(B)	33,000	6,000	
BANKRUN GRAVEL BASE	7,000	0.85	8,000	BORROW AREA "A"	AMPLE	SANDY GRAVEL	1,2, 1-2, 1-5, 3-5	4 - 110	36°-44°	-	-	140-151	8,000(B)	7,000	1,000	
SAND FILL FOR DRAIN WELLS	1,800	0.70	2,500	BORROW AREA "A"	AMPLE	SAND	4-2, 9-5	100 - 300	34°-39°	-	-	115-125	2,500(B)	1,800	700	
CONCRETE AGGREGATE	10,000 5,000	0.70 0.70	15,000 7,000	BORROW AREA "A"	AMPLE	SANDY GRAVEL CLEAN SAND	-	-	-	-	-	-	-	-	-	CONCRETE AGGREGATE NOT PAID UNDER EXCAVATION - INCLUDED IN CONCRETE ITEM.

TOTAL EARTH BORROW (B) 2,745,500 CY
TOTAL EARTH EXCAVATION (E) 1,014,000 CY
TOTAL 3,759,500 CY
LESS WASTE 255,700 CY
USABLE 3,503,800 CY

TOTAL: 255,700 — DOES NOT INCLUDE ROCK WASTE.

GENERAL NOTES:

(A) SHRINKAGE AND LOSS FACTOR = $\frac{\text{EMBANKMENT MEASURE}}{\text{EXCAVATION MEASURE}}$

(B) QUANTITIES FOLLOWED BY LETTER "B" ARE BORROW SOILS, AND QUANTITIES FOLLOWED BY LETTER "E" ARE SOILS FROM EXCAVATION.

(C) ROCK EXCAVATION MAY NOT YIELD REQUIRED AMOUNT OF HARD ROCK. PROBABLY DETERMINABLE ONLY DURING CONSTRUCTION AND DEPENDENT ON EXCAVATION METHODS USED.

(D) SOFT ROCK USABLE AS SOIL, AND PROBABLY A MAXIMUM. ANY DECREASE WILL CORRESPONDINGLY INCREASE BORROW REQUIRED.

(E) TOTAL VOLUME OF RANDOM SECTION = 792,000 CY. RANDOM IMPERVIOUS AND RANDOM PERVIOUS QUANTITIES OBTAINED BY USE OF MEAN SLOPE BETWEEN LIMITS OF RANDOM SECTION.

COEFFICIENT OF PERMEABILITY "K" GIVEN IN UNITS X 10^{-4} CM./SEC.

ϕ - ANGLE OF INTERNAL FRICTION.

*OPT - OPTIMUM WATER CONTENT - IMPACT COMPACTION TEST.

DCW - DRY COMPACTED WEIGHT AT OPTIMUM - IMPACT COMPACTION TEST.

assumed average temperature of ground water.

2. Results for Borrow Areas "A" and "B" are indicated graphically on Plates Nos. 28, 29 and 30. Borrow Area "A" is major source of pervious and random pervious materials. Spillway channel excavation, also designated as Borrow Area "B", is major source of impervious and random impervious materials.

3. General expected permeability for different sections of dam embankment is indicated in a Generalized Section, Plate No. 31, and also on Table 15 (Page 51), entitled "Summary of Materials Available and Required". Typical k-e (permeability vs. void ratio) curves for different types of soils are shown on Plate No. 28.

4. Visual permeability estimates are made during soil classification to assign materials in following permeability groups for estimated density in natural deposit:

Permeability Group	Coefficient of Permeability, k In units cm/sec x 10^{-4}
Impervious (M)	<0.1
Random Impervious (RM)	0.1 - 1.0
Random Pervious (RP)	1.0 - 10
Pervious (P)	10 - 500
Very Pervious (VP)	>500

This is an absolute classification (used for all District sites) and has proven very helpful in studying seepage problems. Permeability grouping of absolute classification does not necessarily correspond to usage grouping at a particular site, where object of latter is to secure a permeability ratio of at least one to ten between different components

of embankment section using available soils. As an example, a soil in pervious group (absolute classification at semi-loose natural density) may be in random pervious group when compacted, and because of availability be actually used in pervious section of embankment at a particular site where soils forming adjoining random section of embankment, are considerably more impervious, (RM) and (M) groups.

G. DIRECT SHEAR.

1. Direct shear tests were run on remolded samples of pervious and impervious materials at compacted density to simulate conditions in the embankment. Because of size of shear box, it was necessary to screen out materials larger than #10 Tyler sieve size before testing. Results obtained therefore are for fraction finer than #10 sieve, which procedure gives conservative results. Pervious samples were tested at approximately 6% moisture content with full drainage permitted. Impervious samples were tested at optimum water content with full drainage permitted, and both types of soils were run after full consolidation and as "slow" tests at 0.06 inches per minute rate of strain. Average direct shear curves for proposed pervious embankment materials are shown on Plate No. 32. Average direct shear curves for proposed impervious embankment materials are shown on Plate No. 33. Points shown on curves at normal loads of 1, 2, and 3 tons/ sq. ft. were obtained by averaging such points from many tests run on such materials. Indicated cohesion for pervious materials is not true cohesion and is not used in computations of stability. Results are summarized on Plate No. 31 and on Table 15, entitled "Summary of Materials Available and Required".

2. General values for impervious material, $\phi = 30°$ and c =

0.4 - 0.5 tsf are considered reasonable since soil is a cohesive glacial till. For pervious materials, Plate No. 32 shows straight line graph values of $\phi = 33° - 37°$ and small cohesion values of 0.1 - 0.2 tsf. Actually these materials are cohesionless and indicated cohesion is probably due to inherent difficulties of direct shear test.

3. Shear strength of soft silt in the dam foundation is difficult to determine since the material consists of interstratified deposits of fine sand, silt, and coarse clay, sufficiently cohesionless as to be very difficult to handle in undisturbed state. Study of this soil is covered in Appendix A.

H. TRIAXIAL COMPRESSION TESTS.

1. Two series of triaxial compression tests were run to investigate shear and critical void ratio on sands which will comprise bulk of the pervious material. A typical sample of medium sand from Borrow Area "A", Class 4-2, sample BT-89, B5, and a typical sample of gravelly sand from Borrow Area "A", Class 7-3, sample BT-80, B3, were used for these two series. Gradation curves for these samples are shown on Plate No. 34. The tests were run by constant volume method and gave following results where $e_{critical}$ is the equal strength void ratio defined by Taylor (1):

(1) D. W. Taylor, "A Comparison of Results of Direct Shear and Cylindrical Compression Tests", A.S.T.M. Proceedings, 1939.

Lateral Pressure, σ_3 psi	$e_{critical}$	$D_{critical}$ pcf	D_r
Sample BT-89,B5	Medium Sand	Class 4-2	
15	0.698	98.9	0.41
45	0.606	103.9	0.63
110	0.514	111.0	0.85
Sample BT-80,B3	Gravelly Sand	Class 7-3	
15	0.506	112.0	0.25
45	0.419	118.8	0.57
110	0.371	122.9	0.74

Mohr's circles on Plate No. 35 show $\phi = 34°$ for the medium sand and $\phi = 36°$ for the gravelly sand, both being cohesionless materials.

2. A special series was also run on sample of coarse gravel from Borrow Area "A", sample BT-61,B7, Class 1. Mechanical analyses of original sample and fractions therefrom used in this series are shown on Plate No. 36. As there was excessive coarse aggregate in original sample for size of available apparatus, five different size fractions were prepared by screening out aggregate coarser than a limiting sieve. From this series properties of original sample were determined by extrapolation. Gradations of the five fractions can also be considered to represent five different soils. Except for one series at constant volume, bulk of tests were run at constant lateral pressure, using 2.8-inch diameter specimens for finer fractions and 4-inch diameter specimens for coarser fractions.

a. Results are given in detail in Special Test Report #1, Claremont Dam "Triaxial Tests on Coarse Gravel for Critical Void Ratio and Shear Strength" (copies in libraries of Office, Chief of

Engineers and of U. S. Waterways Experiment Station) and brief abstracts are given here for minimum and maximum lateral pressures (σ_3) used:

FRACTION	UNIF. COEF. C_u	CRITICAL VOID RATIO, CORRESPONDING DRY DENSITY - PCF AND DENSITY RATIO, D_r						ϕ AT	$e_{critical}$
		σ_3 = 8 PSI			σ_3 = 45 PSI			σ_3 = 8 PSI	σ_3 = 45 PSI
		e_{crit}	d_{crit}	D_r	e_{crit}	d_{crit}	D_r		
-#4	7.36	0.546	110.6	0.122	0.380	123.9	0.683	38°-57'	40°-00'
					at 30 psi	at 30 psi			at 30 psi
-3/8"	11.9	0.468	116.9	0.188	0.370	125.3	0.519	36°-54'	40°-51'
-1/2"	13.8	0.414	120.9	0.191	0.328	128.7	0.496	40°-08'	42°-05'
-1"	21.2	0.345	127.6	0.380	0.271	135.0	0.665	41°-05'	45°-00'
-2"	30.0	0.326	130.4	0.310	0.254	137.8	0.629	40°-06'	48°-21'
Original Sample by Extrapolation	39.4	0.294	133.6	0.371	0.230	140.6	0.657	44°-05'	49°-48'

b. As with other cohesionless sands and gravels, critical void ratio decreases with increase in lateral pressure, as indicated on Plate No. 37, indicating a need for greater compaction in the lower part of the embankment where loads are greater. However, all critical densities are well below anticipated embankment densities, D_r = 0.80 to 0.85, so it will not be difficult to compact to d_{crit}. No danger of a flow slide is anticipated. Plate No. 38 shows relation of e_{crit} and D_r as a band for different size fractions (other tests on sands and gravels have found this relationship as a simple straight line).

c. Variation of ϕ with e_{crit} indicated on Plate No. 39 shows considerable increase of ϕ with lower e, or greater density, being most marked for gravel samples with greater amount of coarse aggregate.

Mohr circles on Plates Nos. 40 and 41 at e_{crit} indicate conservative values for ϕ, as no difficulty is anticipated in compacting above d_{crit}. It is generally anticipated to compact to D_r = 0.75 to 0.80 for sands and 0.80 to 0.85 for gravels with only normal rolling.

I. CONSOLIDATION. - Compressibility and permeability of soft silt deposit underlying valley section of dam were investigated by means of consolidation tests on 4-7/8-inch undisturbed samples. Results of these tests together with further studies of this problem of soft silt foundation are covered in Appendix A.

J. OTHER TESTS.

1. Other miscellaneous tests were also run such as specific gravity, magnesium sulphate soundness on rock fragments, and a field rolling experiment placing soft fractured rock into earthen embankment.

2. The soundness test was conducted on rock from an exploratory quarry designated as Q-1, located in right abutment. This rock is identified geologically as thinly bedded, siliceous sericite schist with strong slaty cleavage. Procedure followed was essentially that described for soundness test of concrete aggregate in Volume II of the "Portland Cement Concrete Manual" issued by the Central Concrete Laboratory, North Atlantic Division, Mt. Vernon, New York, April 1940. A loss of 21.6% was shown after 5 complete cycles of 16 hours immersion, 6 hours in a drying oven, and 2 hours cooling.

3. Rock excavation in Spillway Channel will produce considerable quantity of decomposed mica schist. This material is flat in shape, and the smaller sizes which would be unsuitable for riprap or dumped rock sections can be satisfactorily rolled into the random impervious sections.

Experimental rolling tests were conducted in November 1940 by transporting soft, broken rock from quarry Q-1 to Surry Mt. Dam and rolling it into random section of that dam. The material was readily compacted by tractors and sheepsfoot rollers and accordingly has been considered satisfactory for use in rolled fill at Claremont.

K. SOURCES OF EMBANKMENT MATERIALS.

1. Spillway Excavation (Borrow Area "B").

a. Excavation for Spillway Channel (designated also as Borrow Area "B") passes through the right abutment containing a very compact deposit of cohesive glacial till, Classes 9, 11 and 13. This material exists at natural water content of approximately 11%, practically optimum for compaction, and only a moderate number of boulders were encountered in the borings. This is a very favorable source of excellent impervious material, and full requirements for embankment can be obtained by widening approach channel if amount from minimum excavation is not quite sufficient. Gradation range for this material is shown on Plate No. 42.

b. Upper 5 to 10 feet of right abutment consists of slightly more sandy and less compact till, Classes 7 and 9, which comprises weathered layer, and is slightly less impervious than parent till. Natural water content of this is about 10% and is close to optimum compaction requirements. Material is very suitable for random impervious fill. To prevent a slide on reservoir drawdown it is planned to flatten present steep natural slope of right abutment at channel entrance, and this excavation will provide largely random impervious material of the weathered zone. Gradation range for this material is shown on Plate No. 43.

c. The spillway chute across flood plain will furnish a small amount of material - sands and gravels, Classes 2, 3, 4, 5 and some sandy silt, Class 6, from upper flood plain deposit. Since this will be obtained as full face excavation, it is expected to be largely a source of random pervious.

d. Excavation of schist rock with strong cleavage is expected to give large amounts of soft weathered rock which will break up under rolling and compact with only few voids. As its suitability has been shown in field rolling experiments, it is planned to use this soft rock as random impervious in dam.

2. Borrow Area "A". - An extensive river terrace of sand and gravel exists on left river bank, 1/2 to 1-1/2 miles upstream of the damsite, in the reservoir. (See Plate No. 5, "Plan of Borrow Areas".) This terrace varies in width from a few hundred feet to 4,000 feet and will permit excavations up to a 25-foot cut before reaching water table. Generally coarse gravels, Classes 1, 1-3, 3 are found near the river, changing to pervious sands (both uniform and gravelly), Classes 2, 4, 7-3, and 5-3, farther back. Finer sands and sandy silts, Classes 6 and 8, underlie the gravelly sand, and appear at the surface in the area of Spring Farm Brook. In the portion of the borrow area west of Quabbinnight Brook, random material, Classes 5 and 6-4, silty gravel and silty sand, lies over pervious and was probably carried down by surface wash from nearly bare hillside above.

a. The larger part of material is sand and gravelly sand, which are also the most pervious of materials present. Ample gravel is also available, and consists of well-graded gravel which com-

pacts easily to high densities. The area contains nearly twice as much pervious materials as is needed for the dam, approximately distributed between soil types as follows:

BORROW AREA "A" MATERIALS AVAILABLE		CY	CY
Pervious	Sandy Gravel	625,000	
	Gravelly Sand	1,150,000	
	Sand	585,000	
	TOTAL	2,360,000	2,360,000
Random Pervious	Sandy Gravel	375,000	
	Silty Gravel	35,000	
	Silty Sand	1,185,000	
	TOTAL	1,595,000	1,595,000
Random Impervious			155,000
	TOTAL		4,110,000

b. Plates Nos. 44 and 45 show ranges in gradation for pervious and random pervious materials in this area. Properties of materials are given in Table 15 (Page 51) entitled "Summary of Materials Available and Required", and results of permeability tests are summarized on Plate No. 30.

c. Distribution of materials is quite favorable, allowing the obtaining of several different types of material at the same time, with only slight selection, and in order needed for the dam. This area will furnish source of select pervious, pervious, gravel for bedding and roads, concrete aggregate, and also random pervious as a by-product.

3. Other excavations.

a. Excavation for tunnel inlet and outlet channels contains both sand and gravel, Classes 4, 4-2, 1-3, of the terrace, as well as glacial till, Classes 7, 9, 11, and 13 of the hillsides. The excavation will provide largely impervious and random impervious materials with some random pervious. In the outlet channel there is also some uniform silt, Class 10, which probably will be too wet to compact in the dam, and if so, will be wasted.

b. The cut-off trench has deposits of varying depths of coarse silt and fine sand, Classes 6 and 4, underlain by deposits of coarser sands and gravels, Classes 2, 4, 9-7, 1 and 1-5, which form fluvial flood plain deposits. This material is largely suitable for random pervious fill.

ials with some random pervious. In the outlet channel there is also some uniform silt, Class 10, which probably will be too wet to compact in the dam, and if so, will be wasted.

b. The cut-off trench has deposits of varying depths

c. Site of dam is largely open fields, and stripping under main dam will be about 12 inches in depth except near downstream toe where it will be about 3 feet to make more definite contact with pervious layers in zone of seepage emergence. Stripping under dam and also over spillway channel excavation will provide ample source of topsoil.

4. Usage estimate. - Quantities of materials required, compared with materials available, and their physical characteristics are summarized in Table 15 (Page 51), entitled "Summary of Materials Available and Required". This table gives conservative estimates of shrinkage and loss factors and represents a preliminary plan for usage of available soils.

VII. DESIGN CRITERIA

VII. DESIGN CRITERIA

The Claremont Dam is designed for safety and stability during all conditions of run-off from the drainage area. Discussions of the various loading, stability and safety requirement conditions to which the dam will be subjected and its ability to resist these conditions are given in the various sections of this analysis. These demonstrate that the dam meets the following design criteria:

(1) The spillway capacity is so great that there is no danger of overtopping the dam even with the outlet closed and the reservoir full when the spillway flood occurs. (See Section V, Paragraph H).

(2) The freeboard is so great that there is no danger of overtopping the dam by waves. (See Section V, Paragraph H.5).

(3) The upstream and downstream slopes of the dam are such that with the materials used in construction they will be stable under all conditions. (See Section IX, Paragraph G).

(4) The line of saturation is well within the downstream toe. (See Section IX, Paragraph F).

(5) Water which passes through and under the dam will, when it comes to the surface, have a velocity so small that it is incapable of moving any of the material of which the dam or its foundation is composed. (See Section IX, Paragraph B).

(6) There is no possibility for the free passage of water from the upstream to the downstream face. (See Section IX, Paragraph E).

(7) No material soluble in water is used in any part of the dam.

(8) The foundation is sufficiently stable to resist undue stresses caused by the embankment load. (See Section IX, Paragraph C).

VIII. HYDRAULIC DESIGN

VIII. HYDRAULIC DESIGN

A. SPILLWAY. - The spillway is located in a saddle near the right abutment of the dam and comprises an approach channel, a weir and a chute.

1. Approach channel. - The approach channel will be of sufficient size to insure maximum velocities not exceeding 6.5 ft. per sec. upstream of the crest drop down. Riprap will be placed on the side of the channel nearest the dam.

2. Weir. - The weir will be set in a rock cut. It will be constructed of concrete and will have an ogee crest. In plan, the weir will be an arc with a radius of 375 feet and a length of 395 feet at crest elevation 630.

3. Chute. - In the first 400 ft., the bottom of the chute will converge from a width of 395 feet to a width of 100 feet. The bottom width will remain constant until the excavation leaves rock at about 1,000 feet below the weir. From the end of the rock cut to the Sugar River, a distance of about 1,000 ft. a pilot channel 10 feet wide will be excavated in earth. The bottom profile of the chute is designed to maintain flow at super-critical velocity with a freeboard of 5 feet in rock. For the last 200 feet of rock cut and for the entire length of the pilot channel the slope will be flattened to approximately .006.

4. Design computations. - The flow over the weir crest was computed by the formula

$$Q = CLH^{3/2}$$

in which Q = discharge in c.f.s.

L = crest length in feet

H = head measured from spillway crest to water surface in the pool

C = discharge coefficient

The value of C for the design surcharge of 15 feet was taken as 3.8. The length was determined by the requirement that the weir must pass the design discharge (86,600 c.f.s.) at the design surcharge. The construction of the spillway rating curve over the entire range of discharges is discussed in Section V, "Hydrology and Hydraulics".

Flow in the unlined rock cut spillway chute was computed on the basis of the Manning formula, $v = \frac{1.486}{n} R^{2/3} S^{1/2}$, in which the roughness coefficient, n, was taken as .045.

B. OUTLET. - The outlet works comprise a concrete lined gate controlled tunnel through the left abutment of the dam, an unlined stilling basin excavated in rock and an outlet channel excavated in earth.

1. Intake. - a. The trash racks will be constructed of structural steel shapes and will be supported by the intake walls and two piers. The gross area will be 1400 square feet. A raking platform will be provided.

b. The entrance to the conduit will have a level floor and straight vertical walls. The vertical curve of the roof will be a quarter ellipse with major and minor axes 1.5 and 1.0 times the tunnel diameter, respectively.

c. The bottom of the intake tower will consist of three passages, in each of which will be slots for one service gate and one emergency gate. Three 6.0 ft. by 12.0 ft. service gates and one 6.0 ft. by 12.0 ft. emergency gate will be provided.

2. Conduit. - a. The conduit floor will be level from the intake to the beginning of the circular tunnel section. The invert of the circular section will be set on a slope of approximately .003.

b. The upper transition will extend 60 ft. from the end of the gate section to the beginning of the circular section. The ends of the dividing piers will be approximately 30 feet downstream from the end of the gate section. In the first 30 feet of the transition the gross section will change from a rectangle 28 feet wide by 12 feet high with 2 dividing piers to a rectangle 19 feet wide by 14 feet high. In the remaining 30 feet of the transition, the section will change to a circle with a diameter of 16 feet.

c. The lower transition will occupy the last 30 feet of the conduit. The change will be from the 16 feet diameter circle to a trapezoidal section having side slopes of 10 vertical to 1 horizontal.

3. Operating house. - The gate operating machinery will be housed in the top of the intake structure. Individual hoists will be provided for each of the service gates. A travelling crane will handle the emergency gate. A plate girder service bridge will give access to the operating house.

4. Design. - a. Capacity. - (1) The economics of stream diversion during construction was the governing factor in the determination of the tunnel size.

(2) The following basic formulae were used to estimate losses of head:

(a) Loss through trash racks: $2.0 \frac{V^2}{2g}$ in which v is the velocity just upstream of the racks.

(b) Loss through the bell mouth entrance: $.1 \frac{v^2}{2g}$ in which v is the velocity in the reduced section.

(c) Loss due to turbulence caused by the gate slots: $.03 \frac{V^2}{2g}$ in which V is the velocity past the slots.

(d) Transition loss due to enlargement of section $.2\left(\frac{V1^2}{2g} - \frac{V2^2}{2g}\right)$ in which V1 and V2 are the velocities before and after enlargement, respectively.

(e) Transition loss due to contraction of section $.1\left(\frac{V2^2}{2g} - \frac{V1^2}{2g}\right)$ in which V1 and V2 are the velocities before and after contraction, respectively.

(f) Loss due to curvature: $.25\sqrt{\frac{d}{90}}\,\frac{V^2}{2g}$ in which d is the amount of deflection in degrees and v is the velocity in the curved section.

(g) Loss due to friction: $.00493\,\frac{L}{R^{4/3}}\,\frac{V^2}{2g}$ in which L and R are, respectively, the length and mean hydraulic radius of the reach and V is the mean velocity in the reach. This expression is derived from the Manning formula, $v = \frac{1.486}{n} R^{2/3} S^{1/2}$, in which S is the loss of energy due to friction per foot of length and n is a coefficient of roughness taken as .013.

(3) The following tabulation gives a summary of head losses and capacities when the pool is at spillway crest elevation. The total head is measured from spillway crest (El.630) to top of tunnel at the exit portal (El.538).

SUMMARY OF HEAD LOSSES AND CAPACITIES

Item	All gates open: Feet	All gates open: Fraction of exit vel. head	Two gates open: Feet	Two gates open: Fraction of exit vel. head
Friction	32.0	.791	31.2	.915
Curvature	8.2	.204	7.0	.204
Trash racks	1.7	.041	2.7	.081
Bellmouth entrance	3.5	.087	6.6	.195
Gate slots	1.0	.026	2.0	.058
Transition	5.3	.130	8.4	.247
Total loss	51.7	1.279	57.9	1.700
Exit velocity head	40.3	1.000	34.1	1.000
Total head - ft.	92.0	2.279	92.0	2.700
Exit velocity - f.p.s.	50.9		46.8	
Tunnel area - s.f.	201.		201.	
Discharge - c.f.s.	10,200		9,400	

(4) The hydraulic and energy gradients for the conduit discharging with the pool at spillway crest elevation are shown on Plate No. 46.

b. Rating curves. - (1) All gates open. - (a) For discharges less than approximately 3,000 c.f.s. the control will be at the beginning of the circular tunnel section. The expression $Q = a\sqrt{\frac{ag}{b}}$ was used to compute the depth-discharge relationships. In this formula

a = area of the water section

b = top width of the water section

g = acceleration due to gravity

Pool elevations were estimated by adding 1.33 times the velocity head to the water surface elevation at the control section. The factor 0.33 is the estimated loss of head between the pool and the control section as a fraction of the velocity head in the control section.

(b) At discharges greater than 3,000 c.f.s. the control is the tunnel exit. The hydraulic gradient at this section was assumed to be at the roof. The pool elevations were estimated by adding 2.28 times the velocity head at the exit to the elevation of the roof at the exit. The estimated losses are 1.28 times the exit velocity head.

(c) Since the control shifts from the beginning of the circular section to the exit at some discharge between 2,000 c.f.s. and 3,000 c.f.s. the two curves were joined by a common tangent to give the entire rating curve.

(2) Two gates open. - The following table summarizes the computation methods used for the approximate ranges of discharge indicated.

Q in c.f.s.	Control Section	Flow formula used	Estimated loss from pool to control section
0 - 700	beginning of circular section	$Q = a\sqrt{\frac{ag}{b}}$	.52 x vel. head in control
700-1500	downstream ends of dividing piers	$Q = 5.67bD^{3/2}$	.27 x vel. head in control
over 1500	exit portal	$Q = av$	1.70 x vel. head in control

The three curves were merged to give the entire rating curve.

(3) One gate open. - (a) The control is at the ends of the dividing piers for all discharges.

(b) Formulas and estimated losses for approximate ranges of discharge are tabulated on following page.

Q in c.f.s.	Flow formula used	Estimated losses from pool to control section
0 - 1,000	$Q = 5.67bD^{3/2}$	.41 x vel. head in control
over 1,000	$Q = av$	.30 x vel. head in control

(4) The computed rating curves are plotted on Plate No. 47. Curves of hydraulic elements used as a guide in determining the control sections are plotted on Plates Nos. 48, 49 and 50.

c. Following is a summary of estimated performance.

Condition	Performance
Pool at El. 630 - all gates open	Discharges 10,200 c.f.s.
Pool at El. 630 - one gate closed	Discharges 9,400 c.f.s. (80% of full capacity = 8,200 c.f.s.
Pool at El. 550 (25 ft. above intake invert)	Discharges 3,650 c.f.s. (construction flood)

d. Stilling basin. - The unlined stilling basin is so proportioned that the jump will form within its limits at all discharges up to the design discharge (10,200 c.f.s.). The maximum velocity at the tunnel exit is approximately 51 ft. per sec., and at the end of the stilling basin, 8 ft. per sec. The derivation of the tailwater rating curve upon which the stilling basin computations were based is discussed in Section V, "Hydrology and Hydraulics".

e. Outlet channel. - The channel downstream of the stilling basin will be excavated in earth. It will be 75 feet wide at the bottom and will have side slopes of 1 vertical to 2 horizontal. The maximum velocity will be approximately 8 ft. per sec.

C. FREEBOARD. - The top of the dam will be at elevation 650 thereby providing a freeboard of 5 feet above the maximum pool elevation. Determination of the amount of freeboard is fully discussed in Section V, "Hydrology and Hydraulics".

D. MODEL STUDIES. - No model studies of the spillway will be made since no unusual features are incorporated in the design. Model studies of the outlet works will be made to determine the behavior of the transition section and stilling basin, and to determine the effect of the curvature of the tunnel on flow through the stilling basin.

E. METHODS USED TO REDUCE ERODING VELOCITIES.

1. Spillway. - No special construction for reduction of erosion is contemplated. It is expected that gradual erosion along the line of the pilot channel will accompany small discharges over the weir, and that the eroded material will be deposited near the mouth of the pilot channel and possibly in the main stream channel. In the case of the spillway design discharge, the damage to the river bank and to downstream communities would occur regardless of special velocity reduction measures that might be taken at the dam site. The toe of the dam is sufficiently far removed from the spillway chute to be safe at all discharges. Since the outlet will pass all but extremely rare floods, it is not economically justifiable to provide special means to reduce velocity of spillway discharge for the purpose of reducing maintenance at the mouth of the outlet channel.

2. Outlet channel. - An unlined stilling basin excavated in rock will reduce maximum velocities in the outlet channel to approximately 8 feet per second.

IX. EMBANKMENT AND FOUNDATION DESIGN

IX. EMBANKMENT AND FOUNDATION DESIGN

A. MATERIALS AVAILABLE. - A general discussion of sources of embankment material has been covered in Section VI, Paragraph K and is summarized in Table 15 (Page 51) entitled "Summary of Materials Available and Required." An ample supply of pervious and random pervious materials is available in Borrow Area "A". Excavation of tunnel inlet and outlet channels and spillway channel will provide material required for impervious core and also random impervious material for the random dam section. Materials are reasonably well separated in the borrow areas, so that no difficulty in selection is anticipated to obtain desired material.

B. ECONOMY OF CONSTRUCTION. - Alternate methods of embankment construction were considered with a viewpoint of materials available. The adopted rolled fill method represents the most economical and practical use of materials available in the area.

C. DISTRIBUTION OF MATERIALS IN EMBANKMENT.

1. General. - The essential components of the dam consist of a central impervious core with a cut-off trench to rock or impervious soils in the foundation and abutments, outer pervious shells, and transition zones of random fill between the shells and the central core. The lower portion of the pervious shells will consist of more pervious materials and extends part way under the random section. The upstream dam slope and downstream slope in zone of seepage emergence are protected by dumped rock which rests upon a bedding course to protect the pervious shell from being carried into the rock by seepage and wave wash. The downstream dam slopes above the zone of seepage emergence are topsoiled,

with a drainage system of catch basins and drain pipes to take care of surface run-off. Soils will be placed in all sections with the more impervious placed towards the dam centerline and the more pervious placed towards the dam slopes to effect desirable transition in permeability. Generalized Section, Plate No. 31, shows makeup of dam and Plate No. 24 illustrates gradation of typical embankment soils.

2. Horizontal pervious drains. - a. Horizontal pervious blanket drains are formed at the base of the dam by a 12 ft. layer of more pervious soil which extends part way under the random sections to contact innermost line of drain wells noted in Appendix A. This blanket will be formed of coarse sand and gravelly sand which is the more pervious of materials available at this site.

b. The downstream horizontal pervious layer will function to collect seepage from the random section insuring that line of seepage will remain far inside the downstream slope of dam. This pervious drain is designed to collect water expelled from the foundation silt through the drain wells. This horizontal pervious layer will also serve to collect any under seepage in the upper pervious flood plain deposit of sand and gravels which passes through the cut-off trench.

c. The upstream horizontal drain will function to direct seepage within the upstream portion of the dam during drawdown of the reservoir pool in a more nearly downward direction rather than a semi-horizontal direction which would occur provided the drain was not present. A slide in the upstream slope of a dam during reservoir drawdown is caused partly by the horizontal component of the seepage forces which tends to push the dam embankment upstream. The action of the drain in

changing direction of seepage tends to convert an overturning force to a restraining force and thus improve stability, which is a major problem at this site.

3. General filter requirements. - a. Impervious and random impervious materials available are nearly all variably graded tills and modified tills, Classes 9, 11, 13 for impervious and Classes 7 and 9 for random impervious. This will provide filter action against movement of the random and impervious materials into voids of the upper foundation deposit of sand and gravel. In event any particularly open gravels are encountered in this upper foundation deposit, such will be particularly protected for filter action by covering it with definitely variably graded soil containing appreciable coarse aggregate. The sand and gravelly sand composing 12 foot pervious blanket will act as an excellent filter between upper flood plain deposits and embankment above of pervious and random pervious fills.

b. The pervious sands of the shells will be protected against washing into the dumped rock by a 2 ft. layer of gravel bedding which will be selected bank-run gravel meeting the grain size requirements shown on Plate No. 51. Well-graded gravel occurs in large quantities in Borrow Area "A", and the more pervious of this will be concentrated at outer faces of pervious shells to increase effective thickness of the transition from dumped rock to sand of the pervious shells.

D. HEIGHT AND TOP WIDTH OF DAM. - The dam has a freeboard of 5 feet above the maximum surcharge of 15 feet as determined in Section V, Paragraph H.4 of this Analysis. It is shown in Section V, Paragraph H.5, that the maximum wave height is 3.45 feet and that the

run-up of waves on the 1 on 2-1/2 slope may reach 1.38 feet additional, or a total of 4.83 feet. The riprap protected slope and the top width of 30 feet, consisting of a bituminous macadam surface and strips of stabilized gravel and hand-placed riprap along the edges, will effectively protect against any possible damage from spray or wave action, thus satisfying criterion 2 of Section VII.

E. FOUNDATION TREATMENT.

1. Foundation conditions. - a. Plate No. 23 shows foundation conditions with a deep deposit of soft silt in the valley furnishing a major problem of stability.

b. In general, the foundation under the valley section of the dam is composed of 15-20 ft of pervious flood plain deposit over soft varved silt (silt, lean clay and fine sand interstratified), varying up to 100 ft in depth. This soft silt overlies compact glacial till which in turn generally overlies interstratified silt similar in grain size characteristics to the upper deposit, but extremely compact. Probably the lower silt deposit was formed in a glacial lake and was compacted by a re-advance of the glacier, while the upper deposit was formed in a post-glacial lake.

c. In the bottom of the ancient bedrock valley are found deposits of compact, pervious sands and gravels. The extent of this deep pervious deposit has not been explored completely away from the dam site, and it is not known whether this buried pervious has an outlet downstream.

d. The right abutment is composed of compact, impervious, glacial till while the left abutment is rock with a relatively shallow overburden of till.

2. Treatment to improve stability. - a. The soft varved silt foundation of low shear strength presents major problem of stability at this site. Treatment has been extensively studied, and the design adopted involves four major features to reduce stress in the foundation and to accelerate rate shear strength increases as the dam load is added:

(1) Flat slopes to reduce applied stress.

(2) Extended construction period of three seasons to aid foundation silt in adjusting itself to gradual load application.

(3) Drain wells through silt to accelerate removal of water forced out of silt by consolidation, and thus accelerate gain in shear strength to accommodate applied loads.

(4) Relief wells to tap buried pervious deposit and avoid possibility of flotation decreasing shear strength in foundation deposits at downstream toe.

b. Details of treatment are shown on inclosed plates. Slopes vary from 1:2-1/2 to 1:6 upstream and 1:2-1/2 to 1:5 downstream as shown on Plate No. 56 (Section at Station 15+00). Locations of drain wells are shown on Plate No. 55 (Embankment Details No. 1). Locations of relief wells tapping the buried pervious deposit are shown on Plate No. 55 (Embankment Details No. 1).

c. Analysis of stability and effect of drain wells is quite complex and is presented in detail in Appendix "A".

3. Cut-off treatment. - Foundation will be stripped of organic and unsuitable material, including topsoil, from the whole area

occupied by the dam. This material is from one to two feet deep on the floor of the valley.

a. Across the valley floor a cut-off trench will be constructed with a 20 ft bottom width, through pervious sands and gravels and carried to impervious silt below. This cut-off trench goes 17 ft below water table and will require some water control to work in dry and avoid undue loosening of silt.

b. Exploration trenches will be constructed from the valley floor up the right and left abutments and will in general be constructed as a cut-off through the lesser impervious weathered till forming surface soil. Any loose or disintegrated rock which may be encountered in the left abutment below 15 foot surcharge will be removed and a concrete grout cap installed. The rock will then be grouted to close off any fissures and crevices. It is expected, however, rock will only be encountered near the river bank.

c. This treatment will result in a complete cut-off to impervious material for entire length of dam. Except for the deep pervious layer, no other pervious channels through the foundation are shown by borings. This deep pervious layer is being treated by relief wells to relieve possible seepage pressures.

F. SETTLEMENT.

1. Settlement of the dam over the soft silt deposit will be considerable, varying up to 3-1/2 ft in the vicinity of Stations 15+00 to 16+00. Rate of settlement will depend upon acceleration of consolidation by drain wells which is analyzed in Appendix A. Settlements have been estimated by computation from results of consolidation tests.

Plate No. 52 indicates total estimated settlement along centerline of dam, also estimated settlement at Station 14+00 where the deposit of soft silt is the deepest at the toe of the downstream slope.

2. Because of deep deposit of silt under downstream toe there will be considerable settlement of toe drain as well as embankment there, Plate No. 53 indicates magnitude of this settlement which will be compensated for by giving proper camber to drain pipe so that it will settle to desired grade.

3. In order to check actual settlement during construction season, settlement gages will be installed. From observations obtained by end of second construction season, the total settlement of dam will be recomputed. As necessary the top of dam will be raised during third construction season, so that it will settle to desired grade after construction is completed.

G. SEEPAGE. - 1. As cut-off is complete, seepage will be very small at this site. It will be composed of seepage through the dam, through the silt foundation (particularly sand varves) between the lines of drain wells at each side of centerline and through the deeply buried pervious layer to relief wells.

2. For computation of maximum seepage through dam, random and pervious sections were considered as having infinite permeability. Flow net for seepage through core and cut-off with 15 foot surcharge is shown on Plate No. 54. This method of computation is quite conservative as random impervious material has the effect of enlarging the core. Computation of seepage from this flow net gives 22.3×10^{-4} cfs/sta. Considering length of dam = 2,600 feet and allowing for reduced height on abutments, seepage through the dam totals 0.040 cfs.

3. Quantity of flow of water between drain wells is computed by formula given by Muskat in "Flow of Homogeneous Fluids," Page 529.

$$\text{Flow per well} - Q = \frac{\pi k_H \; HD}{\frac{2\pi d}{a} + \ln \frac{a}{2\pi r_w}}$$

The formula represents flow from one well, in an infinite line of output wells, to flow into a corresponding well of an infinite line of input wells. Symbols in formula have following significance and values:

k_H = horiz. permeability of material between wells = $.02 \times 10^{-4}$ cm/sec.

H = total head loss between two lines of wells = 99 ft.

a = Distance between drain wells in line drive = 60 ft.

2d = Distance between the two lines of wells = 224 ft.

r_w = Radius of drain well = 1/2 ft.

D = Depth of well (assumed as 50 ft for average).

ln = Natural logarithm, base e.

A line of 18 output wells 112 feet upstream from the centerline and an equal line of 18 input wells 112 feet downstream from the centerline, for horizontal permeability of $k_H = .02 \times 10^{-4}$ cm/sec., gives Q = 0.002 cfs of flow from output line to input line.

4. The bulk of seepage through buried pervious layer will be tapped by five relief wells. Computations for this are in Appendix A, and estimated maximum seepage is 0.4 cfs for 5 wells or .08 cfs per well.

5. In summary, total seepage through the dam and varved silt amounts to 0.042 cfs at toe drain, a negligible amount. Possible seepage through the deeply buried pervious layer is estimated as a maximum of 0.4 cfs which is taken care of by a separate drainage system from five relief wells.

H. LINE OF SATURATION. - Plate No. 31 shows position of the line of saturation estimated from seepage computations. Even for rare occurrence of a maximum surcharge of 15 feet with downstream tailwater, line of saturation is far within dam slopes. The 12 foot horizontal drainage blanket draws down the line of seepage so that even with tailwater at Elevation 546, practically all of the flow will be carried through the pervious drain, and the discharge will come out in the downstream rock toe.

I. STABILITY OF SLOPES. - 1. Stability of dam slopes is dependent on rate of gain of shear strength in foundation silt, and this feature is covered in detail in Appendix "A".

a. For collecting surface runoff the downstream slope of the dam is provided with a system of berms, gutters and drain pipes.

2. Slopes of the spillway entrance channel will be excavated in glacial till and flattened to a 1:3 slope below the spillway crest, elevation 630, to withstand rapid drawdown. Above a berm at elevation 630, the slope is steeper at 1:2. These slopes have not been analyzed in detail but design is based on a fairly complete stability analysis of a similar problem of spillway channel slopes in glacial till covered in analysis of design for Union Village Dam, July 1944.

a. Between the entrance to the spillway channel and the upstream slope of the dam, natural slope of the abutment hillside is relatively steep, maximum of 1:1 with an average slope to the 620 contour of 1:2. This condition is satisfactory for dry exposure, but there is considered to be a danger of a slide on rapid drawdown. To remedy this, and since material can be used in lieu of other impervious borrow, this slope is being flattened to 1:3.

3. Fill for the access road to the intake tower will be constructed of free draining materials which, with 1:2 slope, will insure stability. Other soil slopes of the tunnel intake channel are relatively flat and low and so present no problem in stability.

a. Slopes of the tunnel outlet channel involve deeper excavation but do not present any serious problem since there will only be drawdown from tailwater, a relatively mild condition of flow drawdown. Deposits of uniform silt are indicated in borings on the centerline of the tunnel outlet channel near the downstream portal, and if these deposits extend to proposed finished slopes, some changes may be required during construction to improve stability. Present design of 1:2 slopes with 5 foot berms at 20 foot differences in elevation is considered stable and reasonable with information at present available. As excavation proceeds details of slopes can be changed should conditions prove different from those disclosed by centerline borings. Further protection of slopes from erosion by ground water is given by pervious wedges, with base drainage, installed under 12 inch topsoil dressing as shown on Plate No. 60 (Miscellaneous Drainage Details). These pervious wedges are installed on the uphill sides of the channel only to control high ground water fed down from the hillside above.

X. STRUCTURAL DESIGN

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A. SPECIFICATIONS FOR STRUCTURAL DESIGN.

1. General. - The structural design has been carried out in general accordance with standard practice. The specifications contained in the following paragraphs cover the general conditions affecting the design of concrete, reinforced concrete and structural steel. Structural design computation will be found in Appendix B. The structures included in the structural design computations are the intake structure, gate shaft, tunnel, tunnel transition, operating house, outlet portal, spillway weir and retaining wall. No structural design computations for the trash racks and service bridge are included in Appendix B as the size and loading conditions of these structures are the same as that used in Knightville Dam. (Reference may be made to Knightville Dam Analysis of Design, Appendix A, 1938.) Ordinarily stresses due to computed loadings have been held low and a liberal use of reinforcement steel has been resorted to in order that unsightly cracks and fissures due to temperature and uncertain loading conditions might be avoided. Expansion and contraction joints have been provided and structural members have been designed for stability and durability.

2. Loads. - a. Dead loads. - Dead loads comprise the weight of the structures, hoists, gate guides, etc., and do not include cranes, gates or any movable equipment. The following unit weights for material have been used:

Item	Unit Weight lbs. per cu. ft.
Concrete, plain and reinforced	150
Brick	120
Steel	490
Dry earth	100
Rock	165

b. Live loads. - Live loads as used in computations were as follows:

Item or Material	Weight
Water	62.5 lbs. per cu. ft.
Uplift	Variable, see specific section
Wind	20 lbs. per sq. ft.
Equipment	As furnished by manufacturers
Snow	40 lbs. per sq. ft.
Design load on operating floor slab	1000 lbs. per sq. ft.
Design load on operating floor beams	100 lbs. per sq. ft. of floor area, plus maximum concentration from equipment
Design load on stairs	75 lbs. per sq. ft.
Design load on gratings	100 lbs. per sq. ft.

c. Earth pressures. - Rankine's formula is used in computing horizontal earth pressures for dry earth where drainage is present. Where the earth is saturated and no drainage provided, an equivalent liquid pressure of 80 pounds per square foot is used.

3. Structural steel. - The design of structural steel was carried out in accordance with the standard specifications for Steel Construction for Buildings of the American Institute of Steel Construction.

4. Concrete and reinforced concrete. - In general, the concrete design has been governed by the report of "The Joint Committee on Standard Specifications for Concrete and Reinforced Concrete" on "Recommended Practice and Standard Specifications for Concrete and Reinforced Concrete" submitted in 1940. Where any requirements set forth in this report are not in accordance with the specifications mentioned above, the requirements noted below have governed the design.

a. Modulus of elasticity. - In the design of reinforced concrete structures, the modulus of elasticity of concrete with a required 28-day strength of 3000 pounds per square inch was taken as 1/12th that of steel.

b. Coefficient of expansion. - The coefficient of expansion of steel and concrete is assumed at 0.00006 parts per degree Fahrenheit.

c. Allowable working stresses. - (1) General assumptions. - The allowable stresses in concrete as used in design are listed below and are based on an ultimate compressive strength in 28 days of 3000 pounds per square inch.

(a) Flexure. - (fc)	Lbs. per sq. in.
Extreme fiber stresses in compression	800
Extreme fiber stresses in compression adjacent to supports of continuous or fixed beams or rigid frames	900
(b) Shear. - (v)	
Beams with no web reinforcement and without special anchorage of longitudinal steel	60

	Lbs. per sq. in.
Beams with no web reinforcement but with special anchorage of longitudinal steel	90
Beams with properly design web reinforcement but without special anchorage of longitudinal steel	120
Beams with properly designed web reinforcement and with special anchorage of longitudinal steel	180
Footings where longitudinal bars have special anchorage	60
Footings where longitudinal bars have no special anchorage	40

(c) Bond. - (u)

In beams and slabs and one-way footings	100
Where special anchorage is provided	200
In two-way footings	75

Note: These stresses are for deformed bars only. Plain bars are not to be used.

(d) Bearing. - (fc)

Where a concrete member has an area at lease twice the area of bearing	475

(e) Axial compression. -

In short columns with lateral ties where the slenderness ratio is not greater than 11	500

(f) Steel stresses. -

Tension.....18,000

Web reinforcement...16,000

Compression (nfc)... 9,000

(g) Combined stresses. - In structures where secondary stresses develop from wind loads, impact, vibration, or temperature,

these allowable unit stresses may be increased not more than 50 percent over the working stresses given in the preceding paragraphs, provided the section so formed is not less than that required by the dead and live loads alone. However, in reinforcing steel, no combination of forces may develop a stress of more than 28,000 pounds per square inch.

(h) Protective concrete covering. -

Type of structure	Minimum Thickness in inches
Slabs	1-1/2 inches
Beams	2 "
Heavy structures, reinforced against temperature and shrinkage stresses, exposed to water	4 "
Bottom of slabs bearing on rock	4 "

(2) Special assumptions for hydrostatic heads. - The assumed allowable stresses due to hydrostatic loads under different conditions are given below:

(a) For structures 10 feet or more in rock 50 per cent of hydrostatic head to spillway crest.

(b) For structures above rock full hydrostatic head to spillway crest.

(c) For hydrostatic head with water at surcharge level an increase of 50 percent in allowable steel stress was assumed.

B. OUTLET WORKS AND APPURTENANT STRUCTURES. -

1. General description. - The outlet works will consist of an intake structure with trash racks, a gate tower, a tunnel transition section, and an outlet portal and apron from which the tunnel discharge is carried back into the river. For plan and sections of outlet works,

see Plate No. 58. The operating house, containing gates and operating equipment will be located directly above the gate tower. All the structures will be firmly founded in or upon solid rock and will be of reinforced concrete except the operating house which will be of brick with structural steel columns, supporting the crane rails and roof. For a more detailed description and calculation of all pertinent stresses see Appendix B. The more general design criteria, main loads and stresses will be found in the following paragraphs.

2. Intake Structure. - a. Approach channel walls. - On the upstream side of the trash rack supports the approach channel walls will consist of a 12-inch concrete lining anchored with 1-inch diameter rods, to the sides of the 4 on 1 slope of the rock cut. The lining will be reinforced with 3/4 inch diameter bars, 18 inches on centers both ways for temperature. Adequate drains will be provided behind and through the lining. The floor will be 2 feet thick, reinforced with 3/4-inch diameter rods, 1 foot on centers both ways, and adequately anchored.

b. Trash racks and supports. - There will be three panels of trash racks, each composed of 5 sections. Each section will be 8 feet 10 inches wide by 10 feet 3 inches high, securely bound together by a welded channel frame. The bars will be flat rolled steel, round edge 6 inches by 1 inch thick and spaced approximately 13 inches apart. All members are designed for a head of 30 feet above the top of the rack, assuming the racks to be 50 percent blocked. No design computation for the trash racks is included in Appendix B as the size of the racks and loading conditions are the same as for Knightville Dam. The trash racks will be supported by the side walls and two piers. A raking platform above will serve as access to clean or remove the racks.

c. Center piers. - The lower portions of the piers were designed to take the differential hydrostatic pressure by considering it as a framework which transferred the load directly into the base slab and by cantilever action into the walls of the gate tower. The load on the top portion of the piers was transferred directly to the walls of the gate tower by means of cantilever action.

3. Intake tower. - a. The intake tower will be located at the upstream end of the tunnel and is founded in solid rock. The total height of the tower from the invert of the tunnel to the operating floor will be 125.5 feet. The tower will project approximately 47 feet into rock. To effect a reasonable saving in concrete the thickness of the walls have been decreased in steps toward the top. For practical reasons a minimum thickness of 24 inches was used for the upper part of the walls. Horizontal construction joints are provided approximately every 10 feet. Each service gate well within the tower will have a steel stairway to permit safe and easy access.

b. The tower above the roof of the tunnel was designed horizontally as a continuous frame. Because of the varying moment of inertia within many of the individual members it was decided to utilize the method of "column analogy" to determine the stiffness, fixed end moments, and carry-over factors for these members. On finding the stiffness and carry-over factors the assumption was made that the variation in moment of inertia was gradual, sudden changes in section at sharp corners being discounted. Corner pilasters were not taken into account because it was considered that their length and consequent flexibility would have little effect on the frame stiffness and also because of the

by means of a concrete stairway. Access from the upper basement to lower basement will be by means of a steel stairway.

C. SPILLWAY WEIR. -

1. Analysis of the stability of the spillway weir as shown on Pages 250-252 of Appendix B has been based on the following criteria and assumptions:

a. Uplift assumption for sound rock foundations satisfactorily grouted shall be full reservoir head on the heel varying uniformly to zero head or tailwater head on the toe with an effective uplift area of 50% of the total base area.

b. Uplift assumption for a horizontal plane through the masonry shall be full reservoir head on the upstream corner varying uniformly to zero head or positive tailwater head on the downstream corner with the effective uplift area equal to 50% of the total area.

c. When the total computed heel pressure is equal to the computed uplift pressure, the uplift pressure for sound rock foundation or horizontal planes through masonry shall be applied to total area of base or horizontal plane.

d. Sliding factor for weir without specially designed shear keys or cutoffs or stepped foundation shall not exceed 0.7.

e. Foundation pressures shall not be excessive.

f. Factor of safety against overturning shall be not less than 1.5.

To provide additional resistance to sliding, an upstream cutoff or shear key will be provided. Moreover, a keyed construction joint should be used at the top of the upstream cutoff, since it is

considered essential to provide a positive mechanical means of transmitting shear across a plane of potential weakness caused by the abrupt change in cross section where the relatively narrow cutoff trench joins the massive concrete in the weir.

The top monolith in the weir will have a minimum height of 8 feet to insure stability of the upper part of the weir under all conditions.

A downstream cutoff will be provided to protect the toe from underminimg due to excessive scour.

2. Stability computations as shown on Pages 253-255 of Appendix B were also made to show the section required with uplift assumed to act on 67% of the base. A sloping upstream face giving a base 2 feet wider than the weir with vertical upstream face would be required. Since it is considered that an uplift assumption of 50% is sufficiently conservative for the Claremont Spillway weir, the vertical upstream face has been adopted.

D. DESCRIPTION OF OTHER STRUCTURES.

1. Retaining wall. - There will be a retaining wall at the south end of the spillway weir. This retaining wall will be founded on rock and will be of the gravity type. Gravel drains are provided at the rear of the wall so that uplift was neglected and the wall designed for the thrust of dry earth on the back side.

2. Service bridge. - The service bridge will connect the operating house with the roadway which connects the top of dam and provides access for pedestrians and for vehicles transporting gates and other equipment to the operating house. The bridge will consist of two

equal spans of 70.0 feet from center to center of bearings. The intermediate pier and bridge abutment will be of concrete, moderately reinforced for bending and temperature, and will be founded on rock.

Bridge seats will be provided on the wall of the operating house and on the bridge abutment. The straight chord plate girder design was selected as being the most economical type of structure. Each span will consist of two plate girders, 5 feet deep and 70 feet long, center to center of bearings. The floor will be a reinforced concrete slab, supported on the girders, which will be 7 feet-6 inches on centers. The slab will project as a cantilever over the girders on either side, providing a roadway 12 feet wide between 1 foot-6 inch wide curbs. The railing will consist of two horizontal rails of 2-1/2 inch square tubing with channels welded to the tubing and slotted for vertical square bars spaced approximately 5 inches on centers. Two intermediate 4 inch square posts will be placed between each pair of concrete posts.

The design conforms in all respects with the specification of the American Association of State Highway Officials, second edition, 1935. Standard H-15 loading was assumed. No special loading was considered in transporting a gate across the bridge as the combined load of one gate and truck will not exceed that of a standard H-15 loading. Maximum stresses, both tensile and compression, will be below 18,000 pounds per square inch. No design computation for the service bridge is included in Appendix B as the width, span length and loading conditions of the bridge are the same as that used for the design of the Knightville Dam bridge.

3. Access roads. - Access to the dam will be from the left abutment and will be provided by Unity Road, a portion of which will be relocated by the State of New Hampshire. Access to the spillway, raking platform, and river valley downstream of the dam, will be provided by paved roads, adequate drainage being provided by ditches and reinforced concrete pipe culverts where required. The road on top of the dam and the access roads will be paved with 2-1/2 inches of bituminous macadam.

E. ARCHITECTURE.

1. Intake tower superstructure. - a. The intake tower superstructure will be located on top of gate tower and will be of modern design, 34'-7" x 47'-7" outside dimensions, with a flat roof, brick and glass block exterior with cast stone trim. The roof slab will be covered with four ply built-up roofing over a cinder concrete fill, sloped to drain toward two scuppers and exterior copper leaders located on the east side of the building. The roof is enclosed by a brick parapet wall approximately 3'-0" high. Access to the roof will be had by a steel ladder located in the interior of the building and an aluminum cover roof scuttle with compensating hinges.

b. The standby unit at the operating floor elevation 650.5, will be enclosed by removable metal wall sections and steel roof deck. Wired glass will be used for glazing the partition. The area enclosing the standby unit will be heated by a warm air furnace located in the heater room at floor elevation 640.5.

2. Buildings. - a. A six-room Cape Cod type frame house and a two car frame garage, of standard construction, will be provided for the operating attendant.

b. A combination utility building and two car garage of frame construction, with concrete between studs, will be provided, with facilities for storing tools, maintenance equipment and sanitary provisions for public use.

3. Entrance gates and wall. - a. There will be two driveway gateways, 24'-0" wide, with piers 11'-0" high and one pedestrian gate 6'-0" wide with piers 8'-4" high. The masonry for piers of driveway gates and pedestrian gate will be of dressed random coursed ashlar. The wall will be 5'-0" high, 2'-0" wide and of coarse, mortar rubble masonry. All stone will be obtained from local excavations. The driveway gate piers will have a reinforced concrete core. Gates will be constructed of copper bearing steel.

b. The driveway gate piers will each have an electric lighting fixture and the pedestrian gate one electric lighting fixture.

XI. MECHANICAL AND ELECTRICAL EQUIPMENT

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A. GATES. - 1. The flow through the tunnel will be controlled by three caterpillar-type self-closing service gates 6 feet wide by 12 feet high in the clear. The gates will operate in vertical wells extending the full height of the gate structure and will be controlled from the intake tower superstructure located on top of the structure. Emergency gate wells will be located directly upstream from the service gates. One emergency gate of the same type as the service gates will be provided. It will be operated during periods when any service gate is inoperative. This type of gate was selected, as it is particularly adapted to the intended use. It is self-closing under all conditions of head, requires less power to raise because of lower friction coefficients, has no sliding contact between seals, bottom seal may be adjusted to insure a tight gate, and, in general, is of simple design easy to maintain.

2. The body of the emergency and service gates will be constructed of structural steel with no member less than 1/2 inch in thickness or stressed greater than 10,000 pounds per square inch in tension under full load conditions. Seals will be bronze, and all moving parts of corrosion resisting hardened steel. Unit stresses used in the design of steel castings will not exceed one-sixth of the ultimate strength of the steel used. A factor of safety of 15 will be used for castings subject to severe shock or vibration, and a factor of safety of 10 will be used for castings subject to mild shock.

3. Cast iron conduit linings will be embedded in concrete between the emergency and service gate frames and also to a point ap-

proximately 12 feet below the service gate frames. The linings will protect the concrete from cavitation and erosion. Air vents will be provided in the ceiling of the service gate conduit linings, directly downstream from the gate frames. The vents from the gates will extend to a common header from which two shafts will extend upward to openings just below the operating house floor. Each vent shaft will have an area of 7 square feet.

B. HOISTS. - Individual electric-powered stationary cable hoists of 70-ton capacity will be provided to raise each service gate at a speed of approximately one foot per minute. The hoists will have push-button control and be equipped with mechanical and electrical safety devices. Hand cranks will be provided for use in the event of power failure.

C. CRANE. - A 60-ton capacity crane will be installed to operate the emergency gate in any gate well. It will be equipped with an electric powered hoist and hand operated bridge and trolley travel. The travel of the bridge and trolley is so arranged that the crane hook can be moved over any gate well. This will facilitate the making of repairs to service gates, hoists, etc. The crane hoist will have a lifting speed of approximately one foot per minute. The capacity of 60-ton was determined by consideration of the fact that the probability of ever having to raise the emergency gate against the maximum hydrostatic head is most remote, but should it ever become necessary, the inherent additional capacity of a 60-ton crane will be sufficient to do so.

D. STANDBY GASOLINE ELECTRIC GENERATOR UNIT. - A gasoline engine-driven generator will be provided for emergency operation of the gate hoists and lighting system in the event of failure of commercial power.

The unit will consist of a 70 horsepower four-cylinder gasoline engine direct connected to a 240 volt, 3 phase, 25 KW generator mounted on a common base. The standby unit will be of ample capacity to start and operate either the crane or one gate hoist.

E. LIGHTING AND POWER SYSTEM. - The lighting system will provide for the illumination of the operating house and three 1,000 watt outdoor flood lights. The system will be 120 volts supplied by a transformer so connected that it will be energized by either commercial power or the standby unit. Lights will be controlled from a lighting panel board set in the wall of the operating house. The power system will be 240 volt 3 phase 60 cycles, operating from either commercial current or the standby unit. A panel board will be provided to distribute the power to the various items of equipment. All equipment will be protected by appropriate safety and protective devices.

XII. CONSTRUCTION PROCEDURE

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A. SEQUENCE OF OPERATIONS. - The total period of time, and the sequence of operations as outlined below were chosen to permit the most economical employment of men and equipment, to enable utilization of the flood control installations as early as practicable, and to keep construction and interest costs to the Government at a minimum. Embankment construction will commence as soon as possible at the start of the project and, so far as possible, will continue at a uniform rate throughout each working season. This will result in a gradual compaction which will induce a stabilized condition in the soft foundation and avoid overloading which could cause failure by shearing. In order to accelerate consolidation of silt foundation, drain wells will be installed in the first construction season prior to placing of any embankment load on valley section. Piezometers will also be installed by the Government at this time to provide a record of variation in excess pressures. Relief wells for release of seepage pressures in deep pervious deposits will be installed during the last construction season in order to minimize distortion from possible lateral movements in the silt foundation. The procedure outlined will permit the coordination of operations so that most of the materials excavated may be placed directly in final position. The method of closure planned is one which is least conducive to failure by overtopping of the incompleted embankment. The highway and railroad on the northeast bank will be permitted to remain in service for the longest period practicable.

It is assumed that the construction period will extend over approximately three working seasons, based on the assumption that the

contract award will be made in May or June. It is expected that the construction will be completed within 915 calendar days. No work will be permitted on the embankment fill during freezing weather. No concrete work will be permitted between November 15 and March 15 of any season except with adequate cold weather protection.

The major items of work involved in the construction and the items of work planned for each construction season follows:

1. Major items of work. - a. The major items of work and estimated quantities for each item follows:

Stripping -	132,000 cu. yds.
Earth excavation and borrow - - - - - - - - - - - - - -	3,668,000 cu. yds.
Rock excavation, open cut, sloping sides	
Under Schedule A (see subparagraph b below)- - -	334,000 cu. yds.
Under Schedule B (see subparagraph b below)- - -	51,000 cu. yds.
Rock excavation, open cut, vertical sides (by close drilling and broaching method or by wire saw method)	
Under Schedule B - - - - - - - - - - - - - - - - - -	260,000 cu. yds.
Rock excavation, tunnel - - - - - - - - - - - - - - - -	10,000 cu. yds.
Earth embankment (including gravel bedding) - - - - - -	3,114,000 cu. yds.
Topsoil -	50,000 cu. yds.
Dumped rock -	262,000 cu. yds.
Concrete -	12,000 cu. yds.

b. The difference between Schedules A and B lies in the method of excavating rock, open cut. Under Schedule A, open cut rock excavation in the intake channel, outlet channel and spillway channel will, in general, be performed with the sides of the cut sloped

one horizontal to four vertical. Under Schedule B, open cut rock excavation in the intake channel, outlet channel and spillway channel will, in general, be performed with vertical sides by means of close drilling and broaching method or by the wire saw method. (See Section IV, Paragraph D.2b).

2. First season. - During the first construction season it is planned to:

(a) clear and strip the dam site except for the river bed and abutments.

(b) install foundation drains.

(c) excavate and backfill the toe trench and cut-off trench except in the river bed and abutments.

(d) excavate the river relocation channel.

(e) complete a considerable portion of the excavation for the outlet works.

(f) commence the excavation for the spillway.

(g) place 1,000,000 cubic yards of the earth embankment to Elevation 565, approximately, in the area between the railroad and the river relocation channel. (See Plate No. 55).

(h) excavate the rock portion of the relocated highway No. 11 on the right abutment.

(i) construct the garage and utility building.

(j) construct a portion of the access roads to subgrade.

(k) commence landscaping operations.

The State of New Hampshire will complete the highway

detour on the left abutment, including the connecting portion of the relocated highway, by the middle of the first season (See Plate No. 3). All work for this road construction will be performed by the State except that rock excavation as stated in (h) above will be performed by the United States. The relocated highway will be completed by the State of New Hampshire by the end of the first season.

Between seasons only such construction as will not be affected by freezing and cold weather will take place.

3. Second season. - During the second construction season it is planned to:

(a) strip the right abutment.

(b) excavate and backfill the cut-off trench on the right abutment.

(c) complete the excavation for the outlet works and the construction of the intake channel, outlet channel, stilling basin, tunnel, downstream portal and the intake tower substructure.

(d) close off the railroad and the highway on the right abutment and place an additional 1,000,000 cubic yards of earth to bring the embankment up to Elevation 621, approximately, (see Plate No. 55).

(e) continue the earth and rock excavation in the spillway.

(f) construct the service bridge pier and abutment.

(g) construct the operator's dwelling.

(h) continue construction of the access roads.

(i) continue landscaping operations.

4. Third season. - During the third construction season it is planned to:

(a) construct the upstream and downstream diversion cofferdams and divert the river flow through the outlet works.

(b) strip the riverbed and left abutment.

(c) excavate the cut-off trench, construct the grout cap and perform the grouting operations.

(d) complete the closure section and top of the embankment including the approach embankment to the service bridge.

(e) complete the spillway excavation and construct the retaining wall and spillway weir (placing of the concrete in the spillway weir will be retarded so as to keep a minimum clearance of 10 feet between the top of the spillway weir construction and the top of the dam construction).

(f) construct the intake tower superstructure.

(g) construct the service bridge.

(h) complete the rotaries, parking areas, access roads, grading, landscaping, cleanup operations and all other work required by the plans and specifications for the completion of the entire project.

(i) perform all other work necessary to complete the entire project.

(j) clean up the site.

B. DESCRIPTION OF VARIOUS OPERATIONS.

1. Clearing. - Clearing operations are divided into clearing of the dam site incidental to construction, and clearing and removing structures, trees, and debris within the reservoir area. Clearing of the

dam site involves removal of wood and brush and structures within the area of the foundation of the dam, outlet, spillway, and access roads. Clearing of the reservoir area is especially suited for separate contract; it is not urgent in connection with the construction of the dam and involves settlement of questions pertaining to rights-of-way and other negotiations with local authorities.

2. River diversion. - River diversion at Claremont Dam will consist of two separate operations (see Plate No. 55). First, relocation of a portion of the existing river bed through a 50-foot wide channel excavated at the start of the project; and second, diversion through the tunnel, at the beginning of the third construction season, after the outlet works have been sufficiently completed to carry the river flow.

a. Relocation channel. - A channel, 50 feet in width, with an approximate slope of 0.003 will be excavated for the purpose of straightening the river at the dam site, and thus permitting a greater quantity of embankment to be placed northeast of the closure section.

The maximum flood of record, March 1936, had an estimated peak discharge at the dam site of 13,400 c.f.s. at a river stage of about 535.3 m.s.l. A flood of this magnitude would pass through the proposed closure gap at an average velocity of about 9 feet per second. Excessive erosion to the partially constructed embankment along the bank of the relocation channel, during flood stage, is not anticipated for two reasons: (1) a 100-foot berm is provided between the edge of the channel and the toe of the partially constructed embankment thus providing a greater cross sectional area for a given flow; (2) the channel is so curved that the tendency is for erosion to occur on the bank of

the channel opposite the embankment. (See Plate No. 55).

b. Diversion through tunnel. - The capacity of the outlet tunnel is such that it will pass (1) a flood of 10-year frequency as determined for the construction season from May 1st to November 1st; and (2) the outlet design flood when the pool is at spillway crest elevation. A study of the hydrographs covering a continuous period of 15 years shows that a flood of 10 years' frequency, as determined for the construction season, is equivalent to a flow of 5200 c.f.s. Using a rate of flow of 5200 c.f.s. and a 16-foot diameter tunnel, the required elevation of the main cofferdam was computed to be 550.0 m.s.l. Allowance is made for ponding in the reservoir. Hydraulic computations also reveal that the outlet design flood requires that the outlet be capable of discharging 6200 c.f.s. at spillway crest elevation or 3032 c.f.s. at the main cofferdam elevation of 550.0 m.s.l. Thus the determining factor for outlet capacity and cofferdam height is the requirement that the outlet be capable of passing a stream flow equivalent to a flood of 10 years' frequency.

The first operation in diverting the river flow through the tunnel will be to construct upstream and downstream diversion cofferdams to Elevation 532± m.s.l. The function of the upstream cofferdam is to protect the beginning of the construction of the main cofferdam against normal stream flow. The first stage of the closure section, which is in fact the main cofferdam, will then be placed to a top elevation of 550± m.s.l. The other stages of the closure section will be constructed as described in the following paragraphs.

3. Embankment construction. - The embankment will be constructed by the rolled-fill method. Drain wells will be installed in the foundation before placing embankment. Approximately 1,000,000 cubic yards of embankment material will be placed during each construction season in accordance with the scheme outlined on Plate No. 55. Dumped rock and impervious materials will be obtained from spillway and outlet works excavations. Gravel and pervious fill will be obtained from the borrow area.

In the first construction season the portion of the embankment in the area between the railroad and the river relocation will be brought up to Elevation 565. In the second construction season this portion of the embankment will be extended to the right abutment and brought up to Elevation 621. In the third construction season the diversion cofferdams will be constructed, the river flow will be diverted through the outlet works, the closure section will be placed and the entire embankment will be completed.

The closure section will be constructed in stages. This will be accomplished by placing a series of berms parallel to the embankment centerline. The berms will be of sufficient width to permit turning and movement of the equipment. Each stage will be constructed as rapidly as is consistent with safety and the requirements for rolled-fill construction as outlined in the specifications. Plate No. 57 may be used as a guide to the procedure. The use of modern construction equipment and standard methods of construction are contemplated throughout. The rolled fill will be placed by trucks or crawler wagons and

rolled by sheeps-foot rollers for the impervious sections and by tractors for the pervious sections. Settlement gages will be used to determine the settlement of the foundation. For cross-sections of the embankment see Plates Nos. 56 and 57.

4. Outlet works, structures, gates, etc. - By the beginning of the third construction season the outlet works will be sufficiently completed so that the river flow can be diverted through the tunnel. Excavation in the intake and outlet channels may commence at the start of the first season, but excavations at the intake structure and tunnel must be deferred until about the middle of the first season when the highway detour and the connecting portion of the highway relocation on the left abutment will be completed. Upon completion of the excavation the concrete will be placed in the intake, intake tower, tunnel, and outlet portal. The semi-steel tunnel linings section and the gate guides and guard angles in the transition will be placed simultaneously with the concrete. The gates will be placed in closed position to insure a correct fit and left in this position while the concrete is being poured around the linings. Immediately before the diversion of the river, the gates will be removed to permit completion of the intake tower and operating house.

No special cofferdam protection is considered necessary for the excavation and construction of the tunnel and intake structure, provided that the material adjacent to the river is left intact until such time as there is no longer need of protection from high water.

C. LABORATORY AND FIELD CONTROL OF EARTHWORK.

1. Field soils laboratory. - For design purposes many tests

have been performed by the District Soils Laboratory on soil samples from foundation and prospective borrow sources, both in disturbed and undisturbed form. For control of construction and to obtain as-built records of embankment construction and foundation treatment, a field soils laboratory will be established at the site to classify materials from excavation and to aid the placement and control the construction of earthwork in the dam, spillway channel, and other miscellaneous items.

a. The field laboratory will be staffed by trained soils inspectors and equipped to perform the following tests: (1) grain size and classification of materials, (2) water contents of borrow materials, (3) determination of embankment densities, (4) laboratory determination of compaction characteristics of borrow materials, (5) approximate permeability determinations, and (6) general soil sampling by shallow explorations.

b. The field soils laboratory will conduct further soils explorations and testing: to aid in planning usage of materials, to control selection of soils and compaction results and to record actual conditions encountered in event of later claims. Reports will be prepared to record composition of earthwork as constructed, compaction obtained, as-built records of foundation treatment and readings of observation devices.

2. Types of earthwork. - Embankment will consist of the following types of materials, locations of which are shown on Plate No. 56 (Section at Station 15+00) and in generalized form in Generalized Section, Plate No. 31. Selection of soils from available materials and control of

compaction will be based on tests by the field soils laboratory.

a. Pervious fill. - Bulk of material will be obtained as sands and gravels from Borrow Area "A". The sands are more abundant and generally more pervious than the well-graded gravels. Best pervious (gravelly sands Classes 2, 3, 7-3, 5-3) will be placed at the base of the embankment to form horizontal pervious blanket drains 12 feet thick. The more pervious of the coarser gravels (Classes 1, 3) will be concentrated at outer faces behind gravel bedding. Elsewhere more pervious remaining material (largely sands, Classes 2, 4-2) will be placed at outer parts of pervious shells and adjacent to zones of best pervious, and lesser pervious materials (Classes 3, 1-3, 4, sands and gravels) will be placed adjacent to the random section.

b. Impervious fill. - All of this material will be obtained from spillway excavation (Borrow Area "B") and outlet works excavation. Excavation in spillway entrance channel may be widened if necessary to furnish required volume. The material will consist largely of cohesive glacial tills (Classes 9, 11, 13). More variably graded materials, containing some coarse aggregate, will be placed against any open porous materials in cut-off trench or foundation to obtain filter action.

c. Random fill. -

(1) Material will be obtained as a by-product in excavation of impervious from Area "B" and pervious from Area "A", and also from miscellaneous excavations as intake and outlet channels, cut-off trench, spillway chute and toe trenches. It will consist of two general types: random impervious (till, Classes 7 and 9) from weathered

or upper soil of Area "B", and random pervious (silty gravels, fine sands and silty sands) from excavations and from Area "A".

(2) Considerable amount of soft weathered rock is anticipated from excavations in the spillway channel and intake and outlet channels, which will not be suitable for dumped rock and which breaks up under rolling to form essentially a soil compacting without excess voids. Suitable material will be used in random sections and is expected to be largely a random impervious material. The amount is dependent considerably on methods employed in excavating rock and is not determinable in advance; however, the random section is considered sufficiently large to accommodate a very substantial volume of such soft rock. In the use of soft rock, care will be exercised to avoid rock pockets and excess voids, and where necessary for filter action, other random material will be placed between the compacted soft rock and the impervious core.

(3) Random material will be placed with more impervious material (random impervious) adjacent to impervious core and more pervious (random pervious) adjacent to pervious shells. Any more open gravels in the foundation will be covered with variably graded materials containing some coarse aggregate to provide filter action.

(4) The random section is amply large to accommodate the volume of random which it is estimated will be obtained in excavations and as a by-product in borrowing pervious and impervious. It is fairly probable that random obtained will be somewhat less, so that pervious will be substituted for random in outer part of random sections. If it should occur that more random is actually encountered, then limits of the random section will be increased slightly, to be determined later during construction.

d. Rock toes and dumped rock. - For location in embankment see Plate No. 31. In general, no laboratory tests will be required for these materials. Although use of boulders and cobbles from borrow pits as rock fill will be permitted, the major portion will come from structure excavation in rock. The rock will be dumped in place with the larger rocks at the outer faces and the smaller rocks and spalls used to fill in and adjacent to the embankment.

e. Gravel bedding. - The bedding for dumped rock on dam embankment will be obtained as coarse bank-run gravel from Borrow Area "A" where an ample supply exists in the portion nearer the river. The material must meet grain-size requirement shown on Plate No. 24 to provide satisfactory filter action.

f. Roads.

(1) Access roads will in general be built over till or other frost-heaving materials except for roads on top of dam, to intake tower, and some portions of relocated state highway which will be in rock cut. Except where roads are built on pervious fills or in rock cuts, a 12-inch compacted pervious subbase will be placed, followed by a 12-inch bank-run gravel base which must meet grain size gradation shown on Plate No. 51. If particularly bad frost action conditions are encountered at some spots, depth of pervious subbase will be increased as determined at that time. In rock cuts the 12-inch compacted pervious subbase is omitted, and 12-inches of bank-run gravel base only is used between rock surface and bituminous macadam surfacing.

(2) The road surfaces will be 2-1/2 inches of penetration macadam, using crushed stone with asphalt cement for bitu-

minous material. Samples of aggregate will be sent to the District Soils Laboratory to check suitability for use with asphalt, and also to check aggregate durability. Samples of bituminous materials will be taken and sent to the District Soils Laboratory for testing.

(3) In addition to the 12-inch subbase described in (1) above, pervious wedges will be constructed around culvert pipes passing under road pavements, where road is constructed over frost-heaving materials, in order to avoid differential heaving from frost action. Pervious wedges will be 2 feet deep at the culvert for a distance of 3 feet on each side of the culvert and tapering to zero depth in twenty feet.

g. Topsoil. - When stockpiles have been well established, topsoil will be sampled by field forces and sent via the District Soils Laboratory to an agronomist for testing to determine fertilizer requirements and any minor variations in seed mixture needed. Topsoil under site of sawmill, containing considerable sawdust, will be wasted and not used on any slopes requiring topsoil dressing.

D. PLACEMENT OF TYPES OF MATERIALS.

1. Method of placing and compacting. - After stripping has been completed, the foundation will be prepared by plowing followed by rolling to make ready for application of rolled fill. The cut-off trench will be filled with impervious material placed in the dry. The impervious, random and pervious sections of the embankment will be brought up to a crown running parallel with centerline of dam, having lateral slopes of 2 percent minimum and 5 percent maximum grade toward the outer edges of the embankment to permit rain runoff to drain readily. This slope will

be maintained until completion of embankment.

a. Wetting. - Embankment materials will be excavated in the dry. If the water table is high and natural water content of soil in place is above that required for optimum compaction, it will be necessary to lower the water table before excavation and place material on the dam and allow it to dry out to proper water content. Before spreading and rolling, in case the previous layer has dried out or cracked on the surface, it will be dampened to proper moisture content and, if necessary, harrowed to insure adequate bond. For impervious and random impervious materials, the water content will be controlled approximately at optimum for ease of compaction. Pervious materials will be thoroughly wetted to facilitate compaction by substantial saturation.

b. Spreading. - When two or more different materials are being transported onto the embankment, they will be placed systematically so that in any area of the section there will be the required proportions of the materials. After dumping, the materials will be bulldozed or otherwise spread in layers of the required thickness and, if the water content is considered correct, it will be rolled immediately. If the preceeding layer of material is too smooth to insure proper bond, it will be roughened or loosened by harrowing.

c. Compaction. - When the moisture content and condition of the spread layer are satisfactory, it will be compacted by equipment in satisfactory working condition. Old or inefficient equipment will not be permitted.

(1) Compacting of impervious and random impervious fill will be done by sheepsfoot rollers of satisfactory weight pulled by

crawler type tractor using 6 inch layers. Compaction of pervious fill will be done be means of crawler type tractors aided by liberal application of water using 9 inch layers.

(2) Each square foot of embankment will be compacted by sufficient number of passes of the compacting equipment to obtain at least 97% of the compacted test density as determined by laboratory tests:

For soils possessing a definite optimum water content, (impervious and random impervious)	Proctor Compaction Test - A.S.T.M. D698-42T, or equivalent test.
For cohesionless soils not possessing such optimum water content (pervious and random pervious)	Providence Vibrated Density Test (under 800 lb. load in 6 in. cyl. with 60-100 hammer blows) or equivalent test.

(3) Samples will be frequently tested in the project field laboratory before and after compaction, and corrections, adjustments or modification of methods will be made as necessary to secure desired degree of compaction.

E. CONCRETE CONSTRUCTION. - The concrete will be composed of cement, fine aggregate, coarse aggregate and water so proportioned and mixed as to produce a plastic, workable mixture. The different mixes of concrete will be used throughout, except under special conditions. On the drawings and in the specifications, these types are designated as Class "A" and Class "B". The minimum average compressive strength for Class "A" concrete will be 3,400 pounds per square inch, and for Class "B" concrete, 3,000 pounds per square inch, in accordance with a standard 28-day test. Concrete aggregates, formerly available at commercial pits within a 20-mile limit, have been tested by the Central Concrete Testing

Laboratory at Mount Vernon, New York, and found to be of suitable quality. These pits have shut down on account of the war but are expected to reopen. Aggregates from other nearby commercial sources are under investigation. Aggregates available in Borrow Area "A" are being tested and, if found suitable, may be used at the contractor's option.

1. Laboratory control. - A small concrete testing laboratory will be set up at the site. The tests performed here will supplement those made at the Central Laboratory. The laboratory will be used principally to control the quality of concrete during construction. Facilities will be available for testing the grading of aggregates, designing concrete mixtures, mixing of trial concrete batches for the purpose of developing actual relations between compressive strength and water-cement ratio, controlling workability of concrete by slump tests, and casting of concrete cylinders for compressive strength tests.

a. Cement. - Cement will be tested by a recognized testing laboratory and results of these tests will be known before the cement is used. True Portland cement of a well-known and acceptable brand will be used throughout.

b. Fine aggregate. - Natural sand will be used as fine aggregate, which will be subject to careful, thorough analysis, and tests made on mortar specimens for compressive strength.

c. Coarse aggregate. - Washed gravel or crushed stone of required sizes will be used as coarse aggregate. It must consist of hard, tough and durable particles free from adherent coating and must be free from vegetable matter. Only a small amount of soft, friable, thin or elongated particles will be allowed. The aggregate will be subject

to freezing and thawing tests and to careful, thorough analysis, including magnesium sulphate test for soundness.

d. Water. - The amount of water used per bag of cement for each batch of concrete will be predetermined; in general, it will be the minimum amount necessary to produce a plastic mixture of the strength specified. Slump tests will be required in accordance with specifications.

2. Field control. - a. Storage. - The concrete components will be stored separately before mixing. The cement will be stored in a thoroughly dry, weather-tight and properly ventilated building. The fine and coarse aggregates will be stored in such a manner that inclusion of foreign material will be avoided.

b. Mixing. - The exact proportion of all materials in the concrete will be predetermined. The mixing will be done in approved mechanical mixers of a rotating drum type, and there must be adequate facilities for accurate measurement and control of each of the materials used in the concrete. Mixing will be done in batches of sizes as directed and samples will be taken for slump tests and compressive strength tests. Inspectors will at all times supervise and inspect the mixing procedure.

c. Placing. - Concrete will be placed before initial set has occurred. All concrete will be placed upon clean, damp surfaces. Rock surfaces will be free from loose material or other matter interfering with a satisfactory bond. Mass concrete will be placed monolithic. Mechanical vibration will be applied, and forking or hand-spading will be applied adjacent to forms and on exposed surfaces in order to insure smooth, even surfaces. Locations of vertical and horizontal construction joints, as well as contraction and expansion joints, are indicated on the

drawings. The locations of construction joints are tentative only and may be changed to suit conditions in the field. Before placing the concrete, all reinforcing steel will be inspected and pouring of the concrete will be supervised and directed by Government inspectors. Adequate precautions will be taken if concrete is to be placed in cold or hot weather.

F. STRUCTURAL STEEL CONSTRUCTION. - Structural steel construction other than reinforcement and anchor bars consists of (1) girders in steel service bridge; (2) structural steel in operating house; (3) trash racks; (4) miscellaneous angles and flat bars; and (5) steel stairways in gate wells.

1. Service bridge. - The service bridge consists of two equal spans, each made up of two approximately 70-foot long by 5-foot deep plate girders with a reinforced concrete slab. The weight of one girder is approximately 8.5 tons. Each girder may be fabricated in one piece and erected as a unit without use of falsework.

2. Intake tower. - The structural steel in the intake tower will consist of steel columns supporting crane rails and roof beams. Erection of the steel frame will take place before the brick masonry is constructed. Each column section below the crane rail will be 22-1/2 inches and above the crane rail 8 inches and will be fabricated and erected as a unit. The beams supporting the roof are rolled sections 24 inches by 100 pounds and will be incased in concrete.

3. Trash racks. - The trash racks will be shop riveted, bolted and welded in sections 8'-10" length and 10'-3" height. The racks will not be placed until such time that the embankment construction has pro-

gressed above Elevation 550.0 m.s.l. in the closure section. Each trash rack section weighs approximately 4000 pounds. U-bolts have been placed in the concrete of the raking platform and in the walls of the gate tower for use in future removals and replacements.

4. Miscellaneous frames, angles and steel bars. - Miscellaneous structural steel, such as metal door frames, guard angles and flat steel bars embedded in concrete, metal sleeves for conduits, gratings and grilles, etc., will be erected and placed as indicated on the drawings and at such time as required.

5. Steel stairways. - The steel stairs in the service gate wells will consist of structural steel channels supporting stair treads, railings, and landings. The stair stringers will be anchored to the concrete wall at each landing. Each flight of stairs may be shop fabricated in one piece and bolted to supports anchored in the concrete wall. The stair treads and landings will be of steel grating with abrasive nosing. Stair railing will be wrought iron pipe with welded and bolted connections.

G. MISCELLANEOUS CONSTRUCTION. - Miscellaneous construction items, including riprapping of earth slopes other than dumped rock in the embankment, construction of the access roads, operator's dwelling, garage and utility building, erection of boundary fencing and handrails, and performance of landscaping work, will be done at such times as required.

XIII. SUMMARY OF COST

XIII. SUMMARY OF COST

The total construction cost of the Claremont Dam, excluding clearing of reservoir, has been estimated to be $3,735,000, including 10 percent for contingencies and 15 percent for engineering and overhead. The detailed estimate of cost is given in Table 16 and is based on unit prices of prewar level. This amount has been distributed as follows:

(1)	Embankment - - - - - - - - -	$3,122,000
(2)	Concrete features - - - - -	303,000
(3)	Steel construction - - - -	150,000
(4)	Miscellaneous - - - - - - -	160,000

(1) The embankment item consists of all earth fill for the dam with cut-off and exploration trenches, grouting in left abutment, dumped rock, gravel bedding, rock toes, downstream slope drainage structures, topsoiling, seeding and sodding of downstream slope, bituminous macadam road with guard rails on top of the dam, settlement gages, and all excavated materials used in the embankment and in the approach embankment to the service bridge.

(2) The concrete features included consist of intake structure, intake tower below and including the floor of the superstructure, tunnel lining, downstream portal, service bridge concrete slab, retaining wall, spillway weir, miscellaneous concrete structures and all steel reinforcement.

(3) The steel construction included consists of steel girders for service bridge, gates and machinery, and trash racks.

(4) Miscellaneous items included are operator's dwelling, garage, utility building, intake tower superstructure (exclusive of concrete

floors included under Item 2), access roads, parking areas, road drainage, river diversion, clearing and grubbing of dam site, landscaping miscellaneous steel, iron and metal work, pipes and all other items not included under Items (1), (2) and (3).

TABLE 16. ESTIMATE OF COST (1)

Item	Designation	Unit	Quantity	Unit Price	Amount
1	Diversion and Care of River	Job	-		$15,000
2	Clearing and Grubbing	Job	-		2,000
3	Common Stripping	Cu.Yd.	132,000	0.40	52,800
4	Common Excavation - General	Cu.Yd.	968,000	0.40	387,200
5	Common Excavation - B orrow	Cu.Yd.	2,745,000	0.30	823,500
6	Common Excavation - Trench	Cu.Yd.	500	1.00	500
7	Rock Excavation - Open Cut (Sloping Sides)	Cu.Yd.	51,000(2)	1.30	66,300
8	Rock Excavation - Open Cut (Vertical Sides by Close Drilling and Broaching)	Cu.Yd.	260,000	1.55	(3)
9	Rock Excavation - Open Cut (Vertical Sides by Wire Saw Method)	Cu.Yd.	260,000	1.45	377,000
10	Mucking of Overbreak - Open Cut at $0.75 per Cubic Yard	Cu.Yd.	3,600(4)	0.75	2,700

(1) This estimate of cost is based on quantities and unit prices under Contract Schedule B. (See Section XII, Paragraph A.1 b).

(2) Under Schedule A, this quantity is 334,000.

(3) Item 8 is an alternate to Item 9.

(4) Under Schedule A, this quantity is 17,000.

ESTIMATE OF COST (continued)

Item	Designation	Unit	Quantity	Unit Price	Amount
11	Rock Excavation Foundation	Cu.Yd.	300	3.00	$ 900
12	Close Drilling and Broaching	Sq.Ft.	16,000	1.00	(5)
13	Rock Excavation - Tunnel	Cu.Yd.	10,000	10.00	100,000
14	Mucking of Overbreak - Tunnel at $2.00 Per Cubic Yard	Cu.Yd.	500	2.00	1,000
15	Steel Lining Support - Tunnel	Lbs.	19,300	0.07	1,351
16	Dry Packing	Cu.Yd.	150	2.00	300
17	Impervious Fill	Cu.Yd.	510,000	0.10	51,000
18	Random Fill	Cu.Yd.	790,000	0.10	79,000
19	Pervious Fill	Cu.Yd.	1,720,000	0.10	172,000
20	Rolling for Each Additional Two Trips - For Impervious and Random Materials	Square (100 sq.ft.)	200,000	0.01	2,000
21	Rolling for Each Additional Two Trips - For Pervious Materials	Square (100 sq.ft.)	180,000	0.01	1,800
22	Miscellaneous Fill	Cu.Yd.	14,000	0.30	4,200
23	Gravel Bedding - Bank Run	Cu.Yd.	94,000	0.75	70,500
24	Bank Run Gravel Base	Cu.Yd.	7,000	1.00	7,000
25	Compacted Backfill	Cu.Yd.	10,000	0.25	2,500
26	Semi-compacted Backfill	Cu.Yd.	1,000	0.25	250
27	Embankment - Roads and Drives	Cu.Yd.	33,000	0.30	9,900
28	Dumped Rock	Cu.Yd.	262,000	0.50	131,000
29	Hand Placed Riprap - 12" and 15"	Cu.Yd.	500	3.00	1,500
30	2'-0" Riprap	Cu.Yd.	1,300	3.00	3,900
31	Gravel-Filled Drains	Lin.Ft.	700	1.00	700
32	Core Drilling in Rock or Concrete	Lin.Ft.	800	3.00	2,400

(5) This item is for Schedule A only.

ESTIMATE OF COST (continued)

Item	Designation	Unit	Quantity	Unit Price	Amount
33	Ordinary Drilling in Rock or Concrete	Lin.Ft.	2,200	0.75	$1,650
34	Pressure Grouting	Cu.Ft.	3,000	1.00	3,000
35	36-Inch Standard Strength Reinforced Concrete Sewer Pipe with Mortar or Mastic Joints	Lin.Ft.	750	6.00	4,500
36	Reinforced Concrete Culvert Pipe - 12-Inch Standard Strength with Mortar or Mastic Joints	Lin.Ft.	150	1.20	180
37	Reinforced Concrete Culvert Pipe - 15-Inch Standard Strength with Mortar or Mastic Joints	Lin.Ft.	90	1.65	148.50
38	Reinforced Concrete Culvert Pipe - 21-Inch Extra Strength with Mortar or Mastic Joints	Lin.Ft.	300	3.00	900
39	Reinforced Concrete Culvert Pipe - 30-Inch Extra Strength with Mortar or Mastic Joints	Lin.Ft.	370	5.00	1,850
40	Reinforced Concrete Culvert Pipe - 36-Inch Extra Strength with Mortar or Mastic Joints	Lin.Ft.	100	7.00	700
41	Standard Strength Vitrified Clay Pipe - 4-Inch - Closed Joint	Lin.Ft.	270	0.35	94.50
42	Standard Strength Vitrified Clay Pipe - 4-Inch - Open Joint	Lin.Ft.	1,030	0.30	309
43	Standard Strength Vitrified Clay Pipe - 6-Inch - Closed Joint	Lin.Ft.	420	0.40	168
44	Standard Strength Vitrified Clay Pipe - 6-Inch - Open Joint	Lin.Ft.	20	0.45	9

ESTIMATE OF COST (continued)

Item	Designation	Unit	Quantity	Unit Price	Amount
45	Standard Strength Vitrified Clay Pipe - 8-Inch - Closed Joint	Lin.Ft.	1,700	0.65	$ 1,105
46	Standard Strength Vitrified Clay Pipe - 10-Inch - Closed Joint	Lin.Ft.	610	0.80	488
47	Standard Strength Vitrified Clay Pipe - 12-Inch - Closed Joint	Lin.Ft.	120	1.00	120
48	Extra Strength Vitrified Clay Pipe - 4-Inch - Closed Joint	Lin.Ft.	50	0.40	20
49	Extra Strength Vitrified Clay Pipe - 8-Inch - Closed Joint	Lin.Ft.	130	0.75	97.50
50	Extra Strength Vitrified Clay Pipe - 8-Inch - Open Joint	Lin.Ft.	700	0.70	490
51	Extra Strength Vitrified Clay Pipe - 10-Inch - Closed Joint	Lin.Ft.	200	0.90	180
52	Portland Cement	Bbl.	15,000	2.50	37,500
53	Steel Reinforcement.	Lb.	600,000	0.05	30,000
54	Class "A" Concrete - Tunnel Lining	Cu.Yd.	3,000	12.00	36,000
55	Class "A" Concrete - Intake Tower and Downstream Portal	Cu.Yd.	4,400	15.00	66,000
56	Class "A" Concrete - Roads and Bridge Structures	Cu.Yd.	440	20.00	8,800
57	Class "B" Concrete - Spillway Weir and Miscellaneous Structures	Cu.Yd.	3,600	15.00	54,000
58	Concrete Backfill at $5.00 Per Cubic Yard	Cu.Yd.	600	5.00	3,000

ESTIMATE OF COST (continued)

Item	Designation	Unit	Quantity	Unit Price	Amount
59	Miscellaneous Structural Steel	Lb.	84,000	0.10	$ 8,400
60	Miscellaneous Iron and Steel	Lb.	18,000	0.15	2,700
61	Miscellaneous Wrought Iron Pipe	Lb.	4,700	0.20	940
62	Miscellaneous Black Steel Pipe	Lb.	6,500	0.10	650
63	Miscellaneous Brass and Bronze	Lb.	150	1.00	150
64	Copper Water Stops	Lb.	2,850	0.50	1,425
65	Metal Stairs	Job	-		5,000
66	Settlement Gages	Job	-		2,000
67	Structural Steel in Service Bridge	Lb.	87,000	0.10	8,700
68	Intake Tower Superstructure	Job	-		16,500
69	Gates and Accessories	Job	-		85,000
70	Traveling Crane	Job	-		14,000
71	Gasoline-Electric Standby Unit	Job	-		6,000
72	Heating System for Intake Tower Superstructure	Job	-		600
73	Electric Light and Power System	Job	-		7,000
74	Exterior Electrical Work	Job	-		3,500
75	Operator's Dwelling	Job	-		9,800
76	Garage	Job	-		850
77	Utility Building	Job	-		7,500
78	Well Water Supply	Job			400
79	Water Supply Lines	Lin.Ft.	800	1.00	800
80	Relief Wells, 125 Feet Deep	Each	5	800.00	4,000

2400 x 1000 x 2/27 = 180,000 cy. 180,000 x .45 = 80,000

ESTIMATE OF COST (continued)

Item	Designation	Unit	Quantity	Unit Price	Amount
81	Relief Wells, Each Additional Vertical Foot	Lin.Ft.	150	5.00	$ 750
82	Drain Wells	Lin.Ft.	13,000	3.00	39,000
83	Septic Tank	Job	-		200
84	Brick Manholes and Brick Catch Basins	Each	20	100.00	2,000
85	Concrete Manholes	Each	4	300.00	1,200
86	Concrete Drop Inlets	Each	4	50.00	200
87	Bituminous Macadam Pavement	Sq.Yd.	13,650	0.65	8,872.50
88	Bituminous Material	Gal.	34,200	0.15	5,130
89	Asphalt Paved Gutters and Chutes	Sq.Yd.	3,300	3.00	9,900
90	Highway Guard Rail	Lin.Ft.	7,150	0.90	6,435
91	Cable Hand Rail	Lin.Ft.	910	0.80	728
92	Boundary Fencing	Lin.Ft.	5,000	0.75	3,750
93	Utility Gates	Each	3	50.00	150
94	Entrance Gates and Wall	Job	-		4,000
95	Tile Gages	Job	-		700
96	Flagpole and Base	Job	-		1,000
97	Topsoil	Cu.Yd.	50,000	1.00	50,000
98	Sodding	Sq.Yd.	1,900	0.14	266
99	Fertilizing and Seeding - Treatment "A"	Acre	14	300.00	4,200
100	Fertilizing and Seeding - Treatment"B"	Acre	31	200.00	6,200
					$2,952,107
	Contingencies 10%				295,211
					$3,247,318
	Engineering and overhead 15%				487,682
	TOTAL CONSTRUCTION COST FOR CONTRACT SCHEDULE B				$3,735,000

XIV. INDEX OF PLATES AND PHOTOGRAPHS

XIV. INDEX OF PLATES AND PHOTOGRAPHS

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4	Plan of Subsurface Explorations
5	Plan of Borrow Areas
6	Record of Subsurface Explorations No. 1
7	Record of Subsurface Explorations No. 2
8	Record of Subsurface Explorations No. 3
9	Record of Subsurface Explorations - Profiles and Sections
10	Rock Contours
11	Periods of Record of Rainfall Stations
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14	Area and Capacity Curves
15	Adopted Unit Hydrographs
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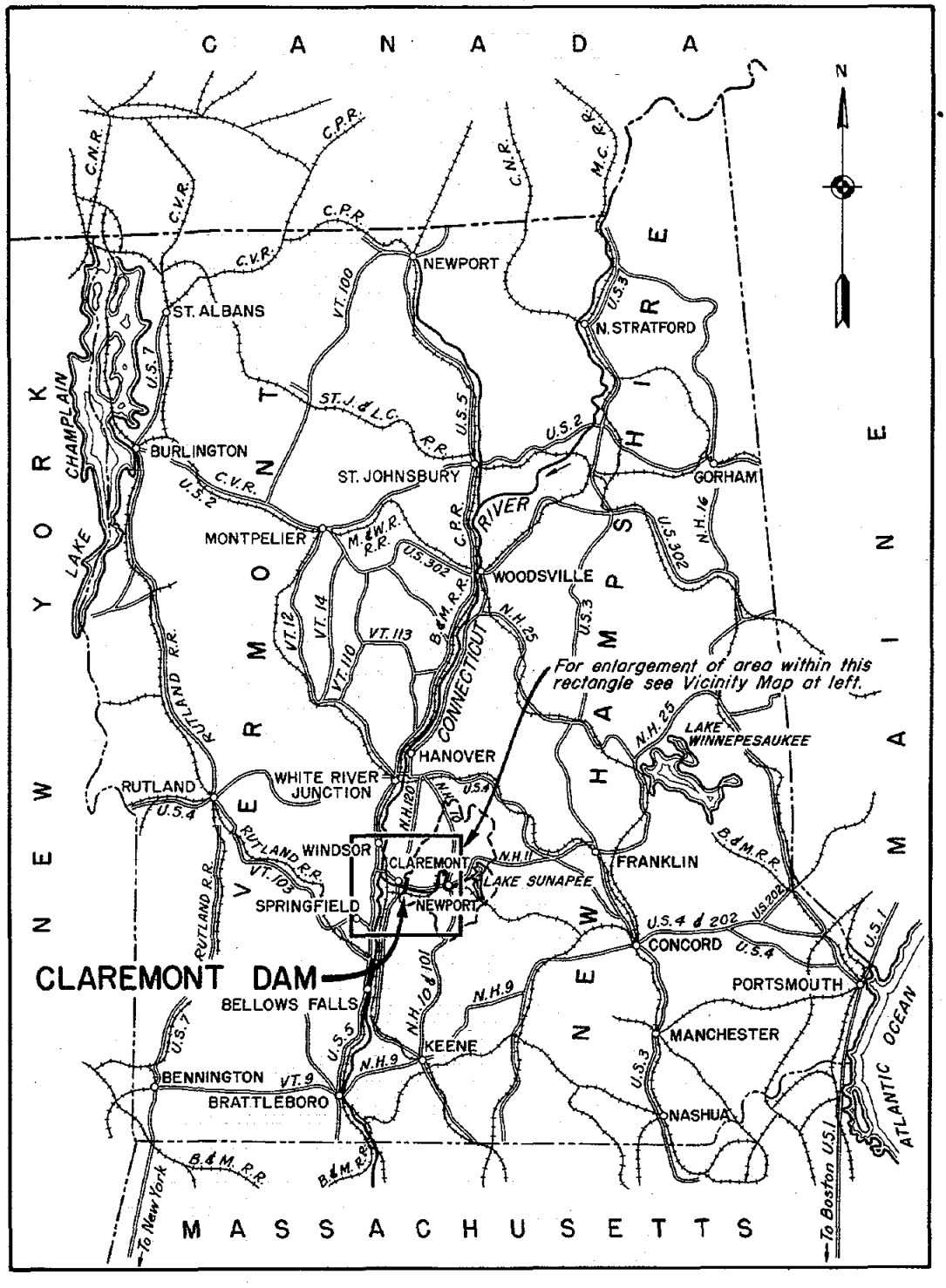
1	Aerial Photograph (Oblique) of Site with Structures Indicated Graphically
2	Aerial Photograph (Vertical) of Dam Site
3	View from Northeast to Southwest Abutment
4	View from Southwest to Northeast Abutment
5	Wire Saw Method of Excavating in Slate Quarry



LOCATION MAP

SCALE

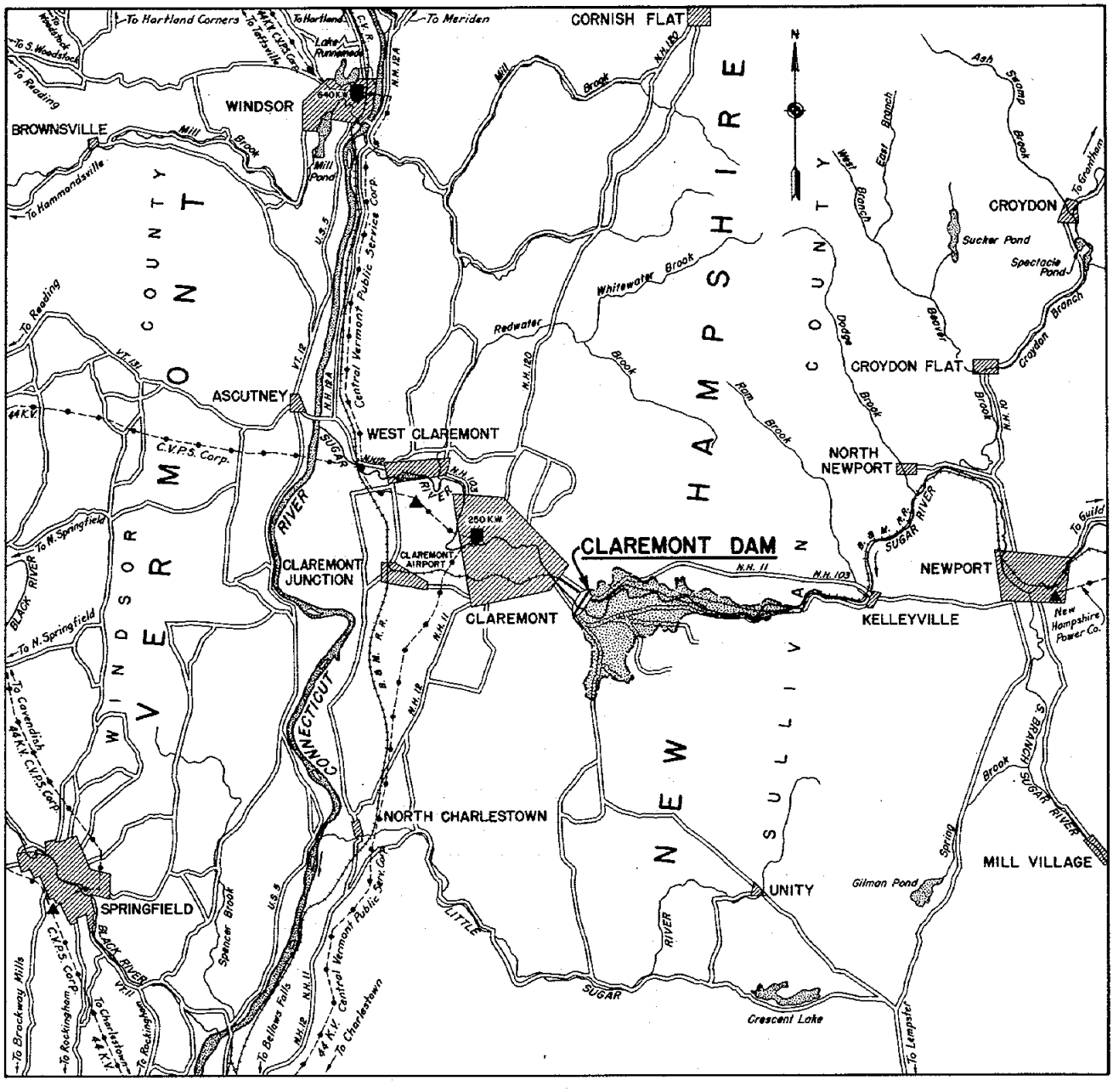
10 0 10 20 30 MI.

LEGEND

HIGHWAYS

RAILROADS

WATERSHED AREA



VICINITY MAP

SCALE

0 — 2 MI.

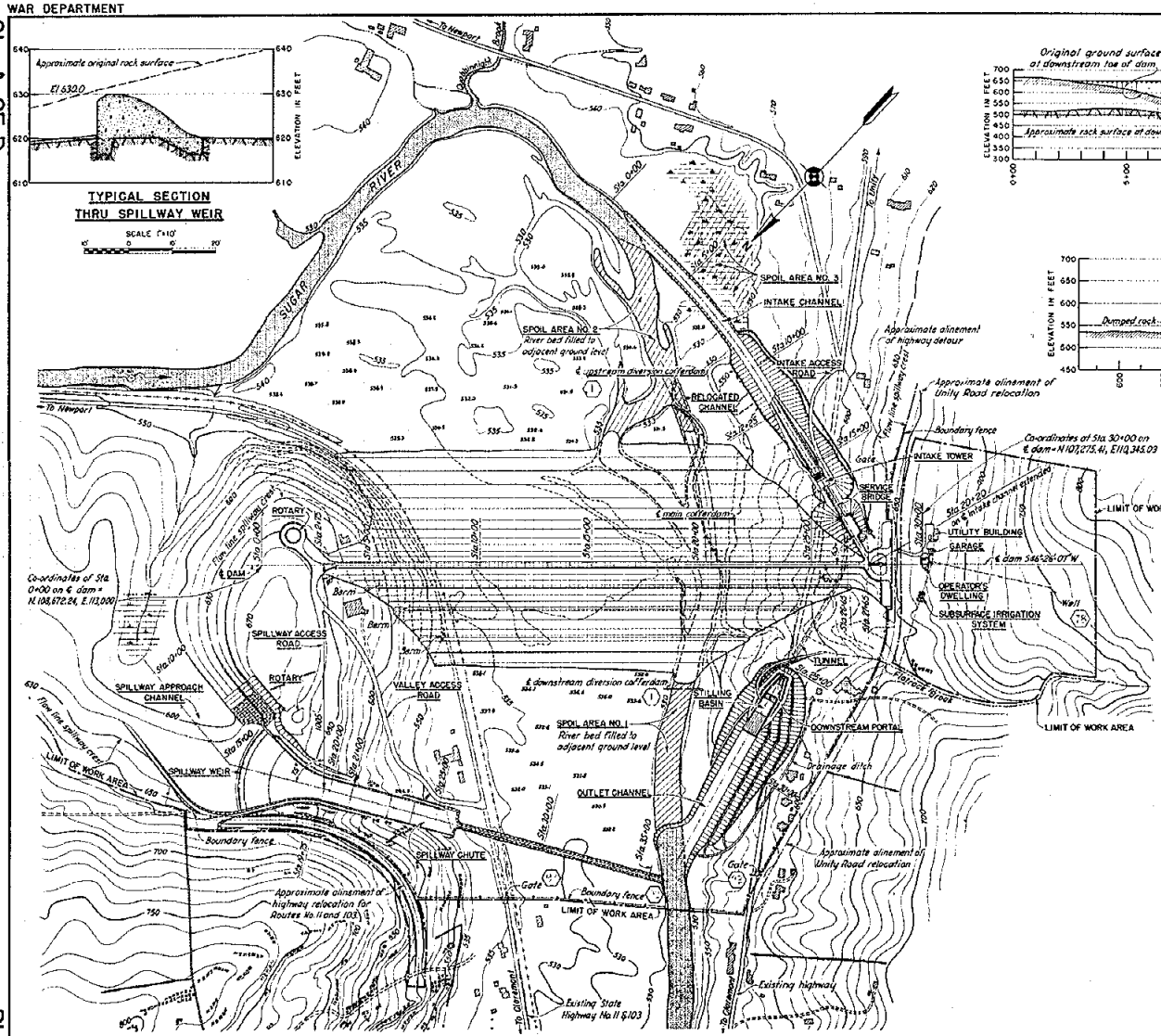
LEGEND

- HIGHWAYS
- RAILROADS
- ELECTRIC POWER LINES
- MAXIMUM RESERVOIR FLOW LINE
- ELECTRIC SUBSTATION
- HYDROELECTRIC GENERATING PLANT
- HYDROELECTRIC AND FUEL GENERATING PLANT

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY

CL A. OF D.

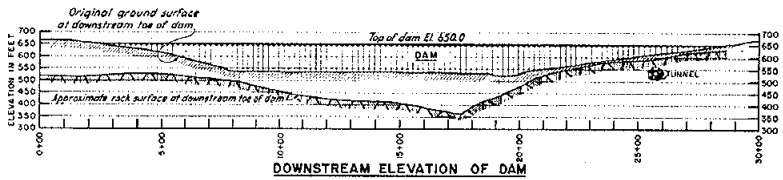


PLAN

SCALE 1" = 200'

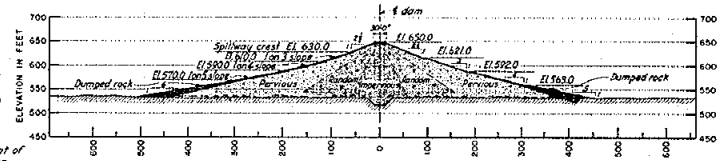
TYPICAL SECTION THRU SPILLWAY WEIR

SCALE 1" = 10'



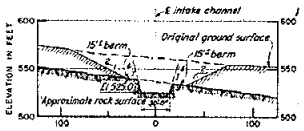
DOWNSTREAM ELEVATION OF DAM

SCALE 1" = 200'



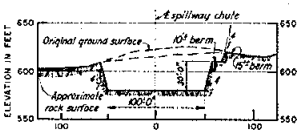
SECTION THRU DAM AT STATION 15+00

SCALE 1" = 100'



SECTION THRU INTAKE CHANNEL AT STATION 12+25

SCALE 1" = 50'



SECTION THRU SPILLWAY CHUTE AT STATION 21+00

SCALE 1" = 50'

NOTES

Elevations refer to Mean Sea Level Datum.
Contour interval is ten feet except as shown.
Spoil area.
For location of existing buildings to be removed see Sheet No. 4.

KEY	DATE	REVISION (Indicated by △)	REV BY	CK. BY	AP BY

CONNECTICUT RIVER FLOOD CONTROL

CLAREMONT DAM

GENERAL PLAN

SUGAR RIVER — NEW HAMPSHIRE

IN 94 SHEETS — SCALE: 1 IN. = 200 FT. — SHEET NO. 2

U.S. ENGINEER OFFICE, PROVIDENCE, R.I., JAN. 1945

SUBMITTED: C. D. Lewis, SENIOR ENGINEER, PROJECT ENGINEER — APPROVAL RECOMMENDED: J. L. Morris, HEAD ENGINEER, CHIEF, ENGINEERING DIV. — APPROVED: M. E. Tuttle, COL., CORPS OF ENGINEERS, DISTRICT ENGINEER

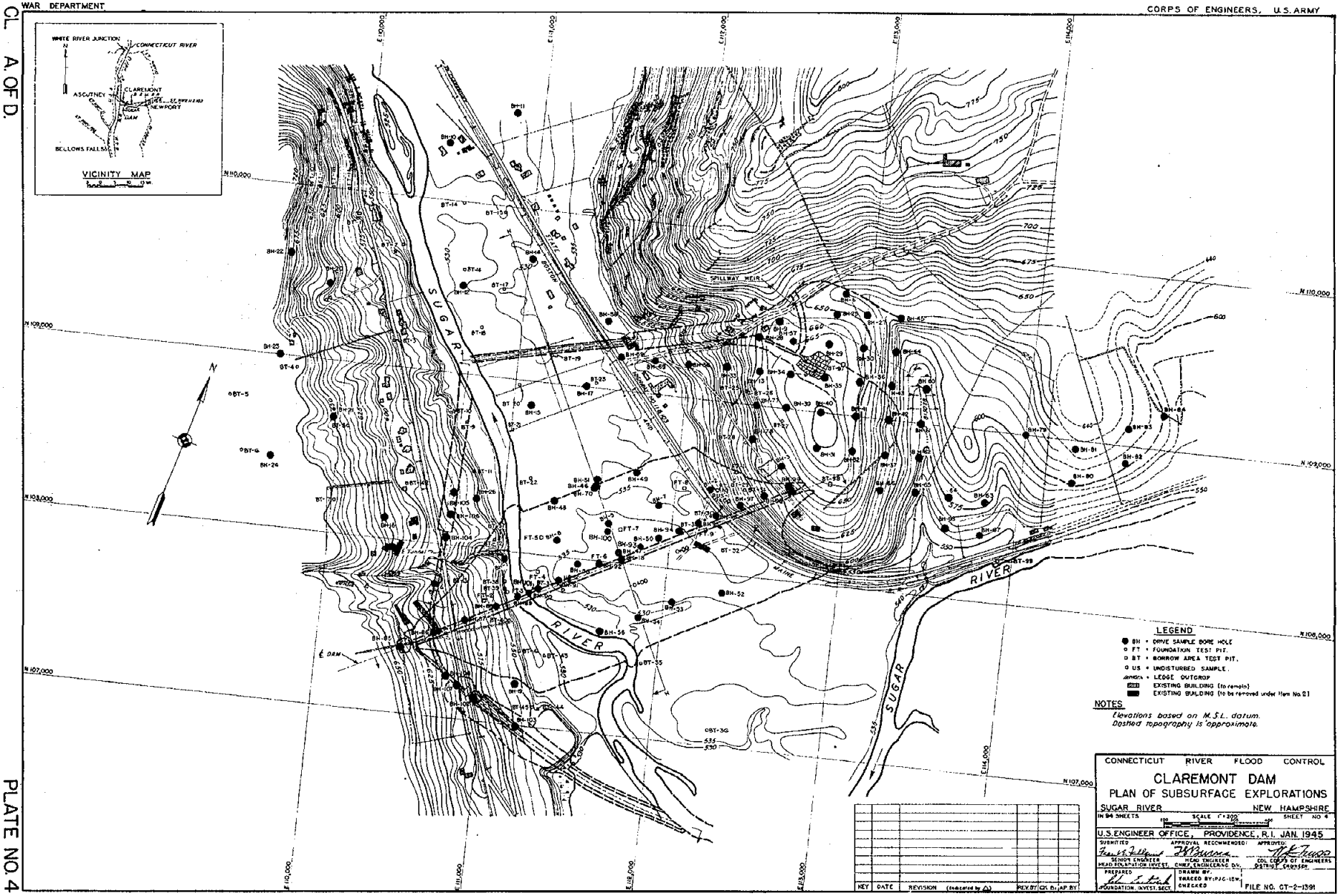
PREPARED: A. C. Neidke, PROJECT ENGR. NO. 2 — DRAWN: J.B.M. TRACED: K.A.V. CHECKED: [illegible]

FILE NO. CT-1-1779

PLATE NO. 3

WAR DEPARTMENT

CORPS OF ENGINEERS, U.S. ARMY

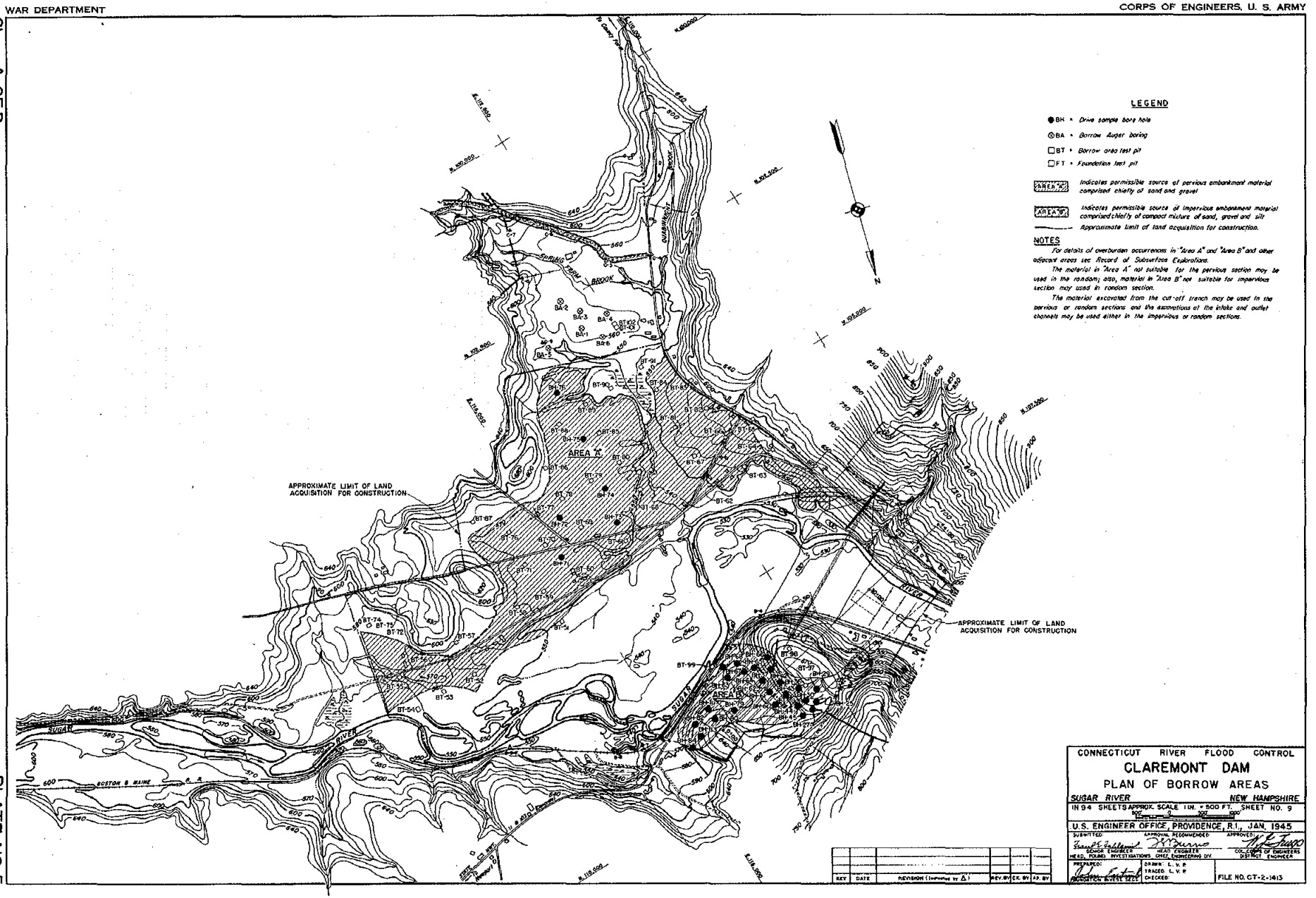


CL. A. OF D.

PLATE NO. 4

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY

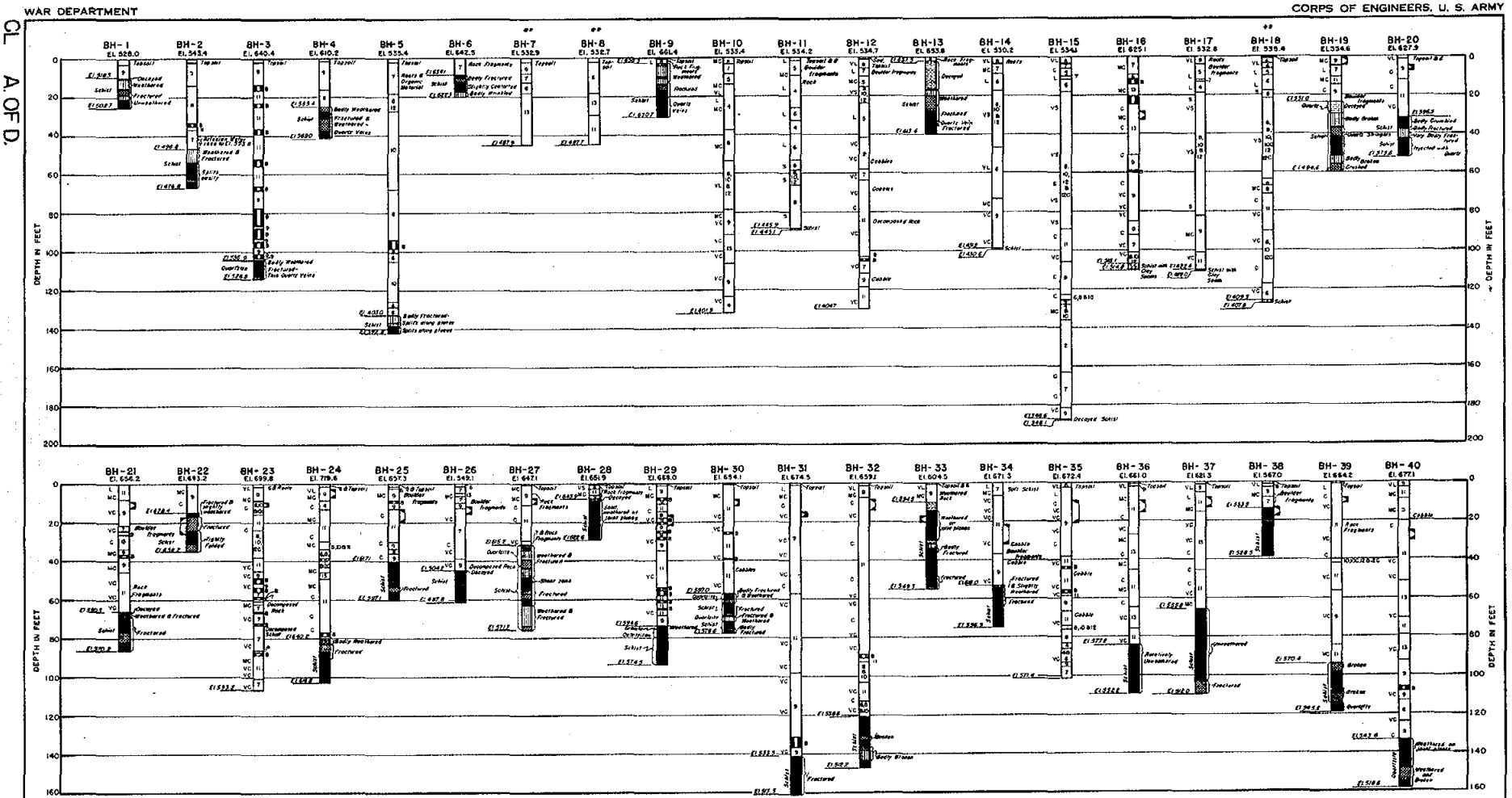


CL A. OF D.

PLATE NO. 5

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY



DESCRIPTION OF NUMERICAL SOIL CLASSES

1. Graded from Gravel to Coarse Sand - Contains little medium sand.
2. Coarse to Medium Sand - Contains little gravel and fine sand.
3. Graded from Gravel to Medium Sand - Contains little fine sand.
4. Medium to Fine Sand - Contains little coarse sand and coarse silt.
5. Graded from Gravel to Fine Sand - Contains little coarse silt.
6. Fine Sand to Coarse Silt - Contains little medium sand and medium silt.
7. Graded from Gravel to Coarse Silt - Contains little medium silt.
8. Coarse to Medium Silt - Contains little fine sand and fine silt.
9. Graded from Gravel to Medium Silt - Contains little fine silt.
10. Medium to Fine Silt - Contains little coarse silt and coarse clay. Possesses behavior characteristics of silt.
10C. Medium Silt to Coarse Clay - Contains little coarse silt and medium clay. Possesses behavior characteristics of clay.
11. Graded from Gravel or Coarse Sand to Fine Silt - Contains little coarse clay.
12. Fine Silt to Clay - Contains little medium silt and fine clay (colloids). Possesses behavior characteristics of silt.
12C. Clay - Contains little silt. Possesses behavior characteristics of clay.
13. Graded from Coarse Sand to Clay - Contains little fine clay (colloids). Possesses behavior characteristics of silt.
13C. Clay - Graded from sand to fine clay (colloids). Possesses behavior characteristics of clay.

LEGEND

- ** - Undisturbed soil samples were obtained from this bore hole.
- VL - Very loose overburden. VS - Very soft overburden. L - Loose overburden.
- S - Soft overburden. MC - Medium compact overburden.
- C - Compact overburden. VC - Very compact overburden.
- BH - Drive sample bore hole.
- Maximum height of water table during period of observation.
- Minimum height of water table during period of observation.
- B - Boulder.
- Rock Core Recovery < 15%.
- Rock Core Recovery 15% - 50%.
- Rock Core Recovery 50% - 75%.
- Rock Core Recovery > 75%.

NOTES

Elevations refer to Mean Sea Level Datum.
For location of bore holes see Sheet No. 4.
Overburden compactness was determined by the number of blows required to drive the sample spoon one foot.
Bedrock is composed of steeply inclined beds of very fine grained quartz sericite schist or phyllite with some quartz intrusions. Due to the strongly developed slaty cleavage the rock breaks into large and small slab-like masses and smaller sizes of flat, elongated, and flakey pieces.
Classes 6, 8, 10, 10c, 12, and 12c when shown in one stratum usually occur in thin, interstratified layers.
The logs, samples, and test results pertaining to the above investigations may be inspected at the United States Engineer Office, Providence, R. I.

KEY	DATE	REVISION (Indicated by △)	REV. BY	CK. BY	AP. BY

CONNECTICUT RIVER FLOOD CONTROL

CLAREMONT DAM

RECORD OF SUBSURFACE EXPLORATIONS NO. 1

SUGAR RIVER — NEW HAMPSHIRE

IN 94 SHEETS — SCALE AS SHOWN — SHEET NO. 5

U. S. ENGINEER OFFICE, PROVIDENCE, R.I., JAN. 1945

SUBMITTED: SENIOR ENGINEER, HEAD (CONSTRUCTION) INVESTIGATIONS — APPROVED RECOMMENDED: HEAD ENGINEER, CHIEF, ENGINEERING DIV. — APPROVED: COL. CORPS OF ENGINEERS, DISTRICT ENGINEER

PREPARED: CONSTRUCTION SUPERVISOR — DRAWN: R.R.L. TRACED: A.J.H. CHECKED

FILE NO. CT-2-1409

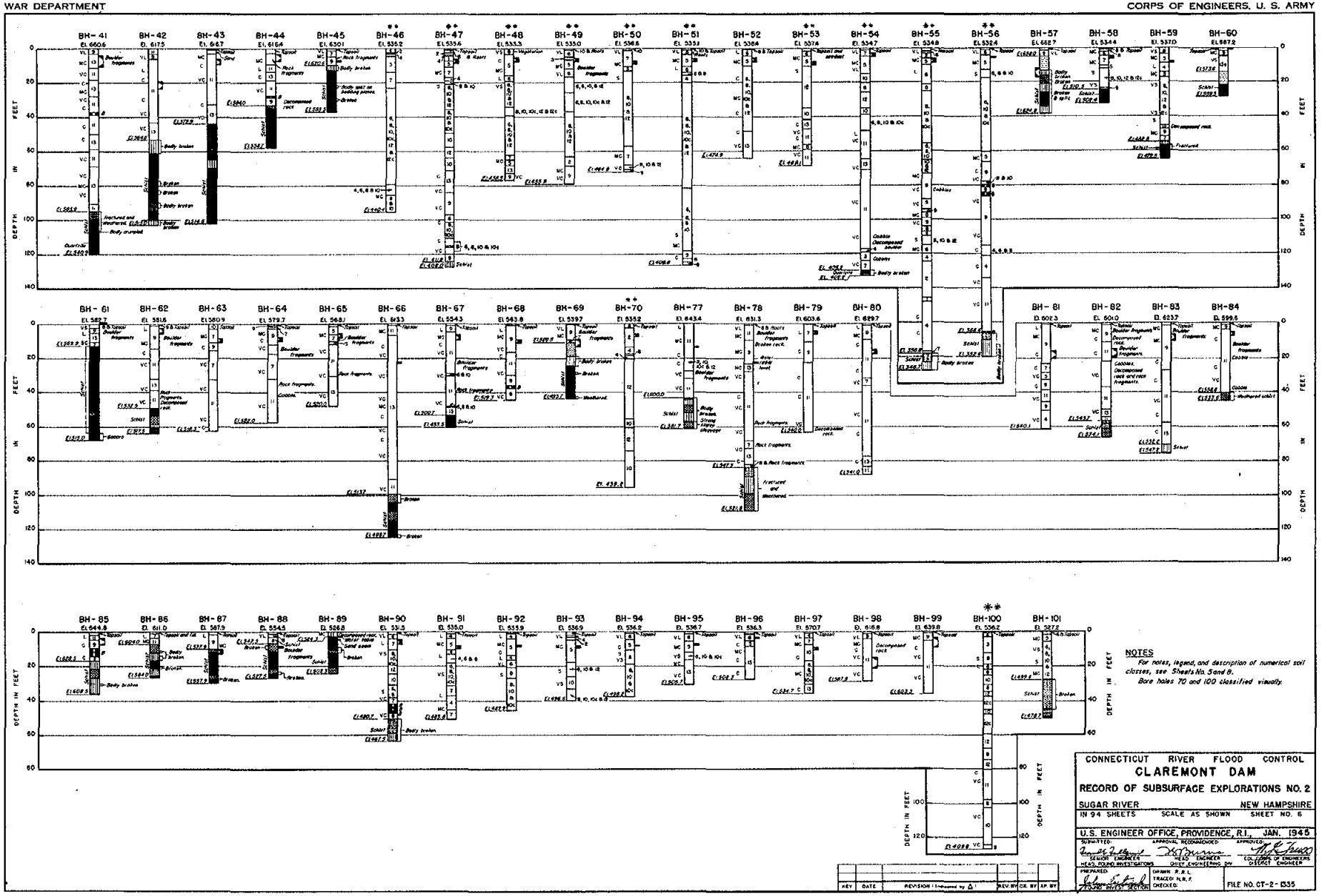
CL. A. OF D.

PLATE NO. 6

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY

CL A. OF D.



NOTES

For notes, legend, and description of numerical soil classes, see Sheets No. 5 and 8.

Bore holes 70 and 100 classified visually.

KEY	DATE	REVISION	REV. BY	CK. BY	AP. BY

CONNECTICUT RIVER FLOOD CONTROL

CLAREMONT DAM

RECORD OF SUBSURFACE EXPLORATIONS NO. 2

SUGAR RIVER — NEW HAMPSHIRE

IN 94 SHEETS — SCALE AS SHOWN — SHEET NO. 6

U.S. ENGINEER OFFICE, PROVIDENCE, R.I., JAN. 1945

SUBMITTED: SENIOR ENGINEER, HEAD, FOUND. INVESTIGATIONS

APPROVAL RECOMMENDED: HEAD ENGINEER, CHIEF ENGINEERING DIV.

APPROVED: COL. CORPS OF ENGINEERS, DISTRICT ENGINEER

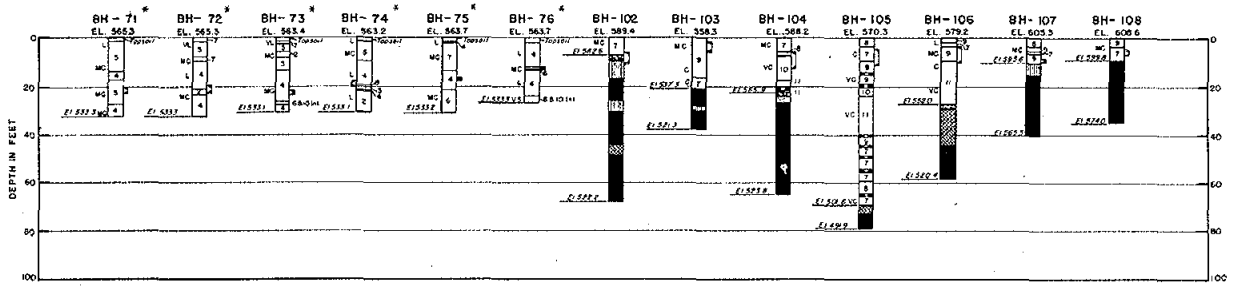
PREPARED: DRAWN: R. R. L. TRACED: H. R. F. CHECKED:

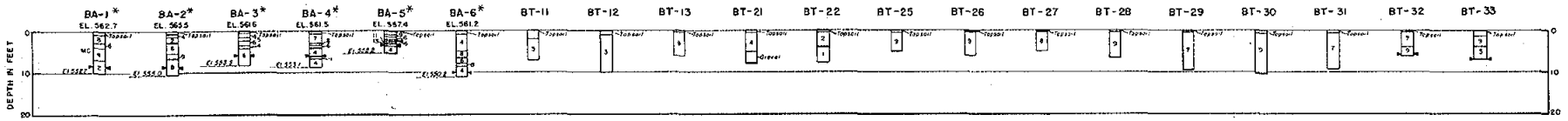
FILE NO. CT-2-1535

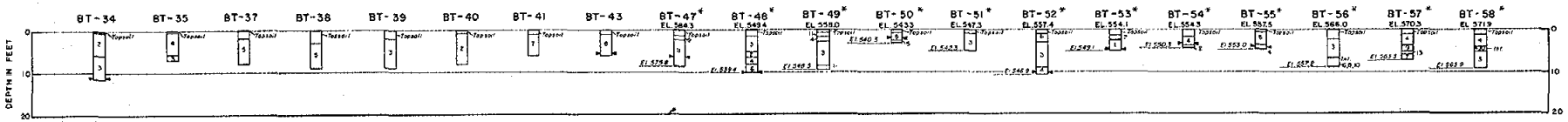
PLATE NO. 7

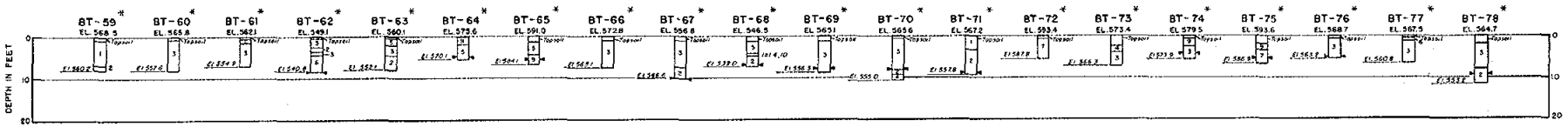
WAR DEPARTMENT

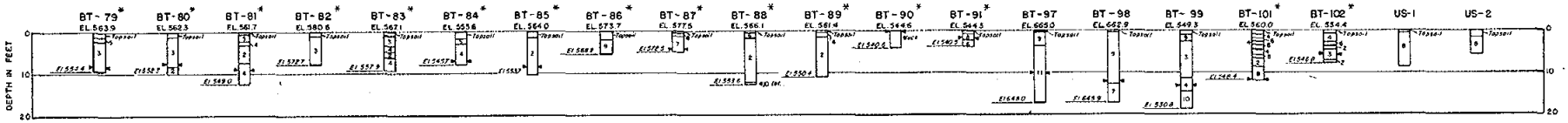
CORPS OF ENGINEERS, U. S. ARMY

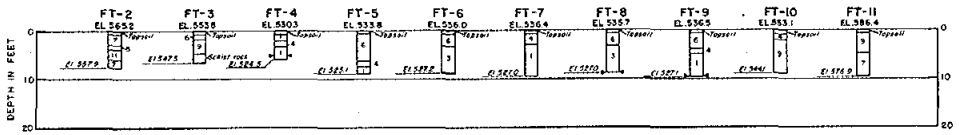












NOTES

For notes, legend, and description of numerical soil classes, see Sheets No. 4, 5 and 9.

Bore holes 102 thru 108 classified visually.

** Located in Pervious Borrow Area.*

KEY	DATE	REVISION (Indicated by △)	REV BY	CK BY	AP BY

CONNECTICUT RIVER FLOOD CONTROL

CLAREMONT DAM

RECORD OF SUBSURFACE EXPLORATIONS NO. 3

SUGAR RIVER — NEW HAMPSHIRE

IN 94 SHEETS — SCALE AS SHOWN — SHEET NO. 7

U. S. ENGINEER OFFICE, PROVIDENCE, R.I., JAN. 1945

SUBMITTED: SENIOR ENGINEER, HEAD, FOUNDATION INVEST. — APPROVAL RECOMMENDED: HEAD ENGINEER, CHIEF, ENGINEERING DIV. — APPROVED: COL., CORPS OF ENGINEERS, DISTRICT ENGINEER

PREPARED: FOUNDATION INVESTIGATION — DRAWN: V.E. TRACED: A.J.M. CHECKED:

FILE NO. CT-2-1411

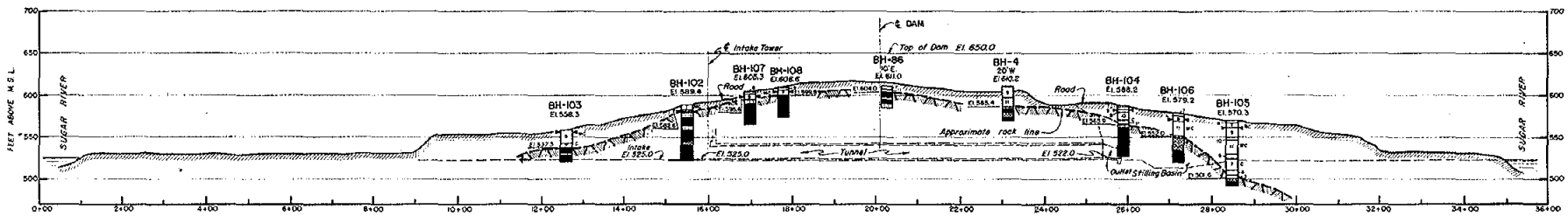
CL A. OF D.

PLATE NO. 8

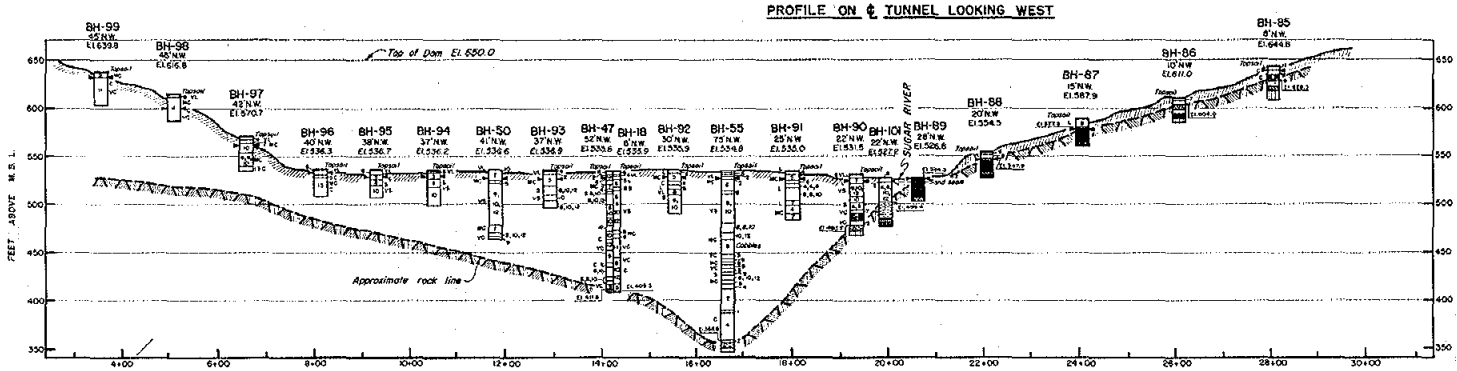
WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY

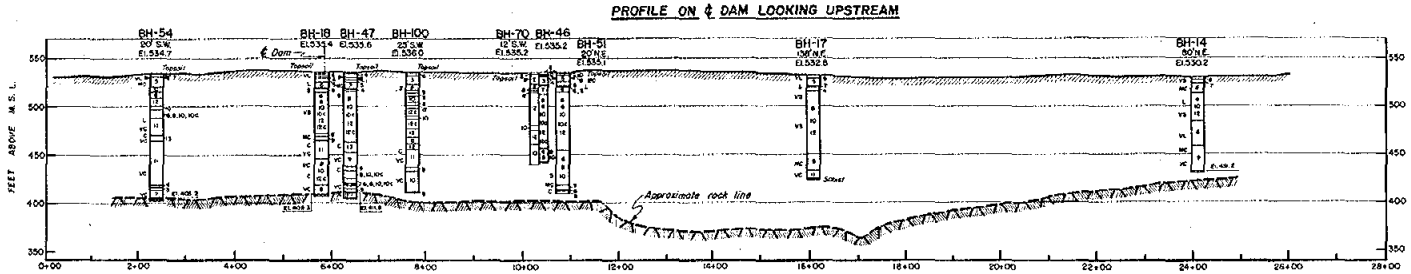
CL A. OF D.



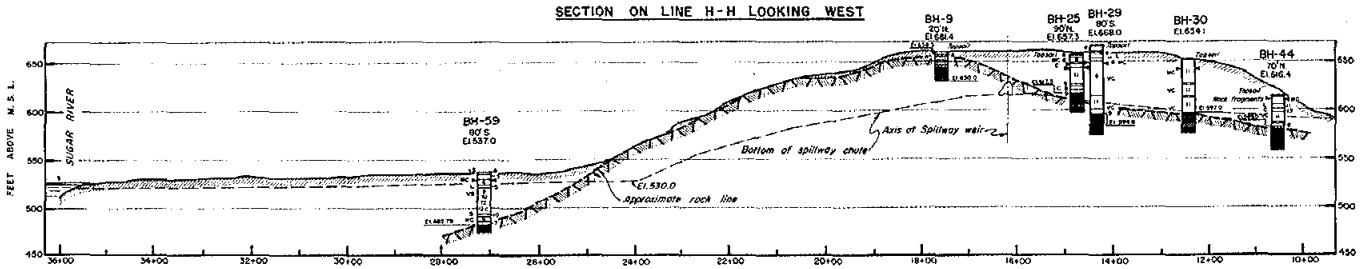
PROFILE ON ℄ TUNNEL LOOKING WEST



PROFILE ON ℄ DAM LOOKING UPSTREAM



SECTION ON LINE H-H LOOKING WEST



PROFILE ON ℄ SPILLWAY CHANNEL LOOKING NORTH

LEGEND

VL - Very loose overburden.
VS - Very soft overburden
L - Loose overburden.
S - Soft overburden
MC - Medium compact overburden.
C - Compact overburden.
VC - Very compact overburden.
BH - Drive sample bore hole.

Average height of water table during period of observation.
Boulder
Rock core recovery 15%.
Rock core recovery 16% to 50%.
Rock core recovery 51% to 75%.
Rock core recovery greater than 75%.

NOTES:

For description of numerical soil classes see Sheet No. 5
For location of profiles, sections, and bore holes, see Sheet No. 4
For log of bore holes, see Sheets No. 5, 6 and 7
Bedrock is composed of steeply inclined beds of very fine-grained quartz sericite schist or phyllite with some quartz intrusions. Due to the strongly developed slaty cleavage the rock breaks into large and small slab-like masses and smaller sizes of flat, elongated, and flaky pieces.

KEY	DATE	REVISION (Indicated by △)	REVDR	CK BY	AP BY

CONNECTICUT RIVER FLOOD CONTROL

CLAREMONT DAM

RECORD OF SUBSURFACE EXPLORATIONS
PROFILES AND SECTIONS

SUGAR RIVER — NEW HAMPSHIRE

IN 94 SHEETS — SCALE: HOR. 1 IN. = 100 FT. VERT. 1 IN. = 50 FT. — SHEET NO. 8

U. S. ENGINEER OFFICE, PROVIDENCE, R.I., JAN. 1945

SUBMITTED: SENIOR ENGINEER, HEAD, FOUNDATION INVEST. — APPROVAL RECOMMENDED: HEAD ENGINEER, CHIEF, ENGINEERING DIV. — APPROVED: COL. CORPS OF ENGINEERS, DISTRICT ENGINEER

PREPARED: FOUNDATION INVEST. SECTION — DRAWN: J.E. & A.B.C.W. TRACED: G.H.R. CHECKED

FILE NO. CT-2-1412

PLATE NO. 9

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY

CL A. OF D.

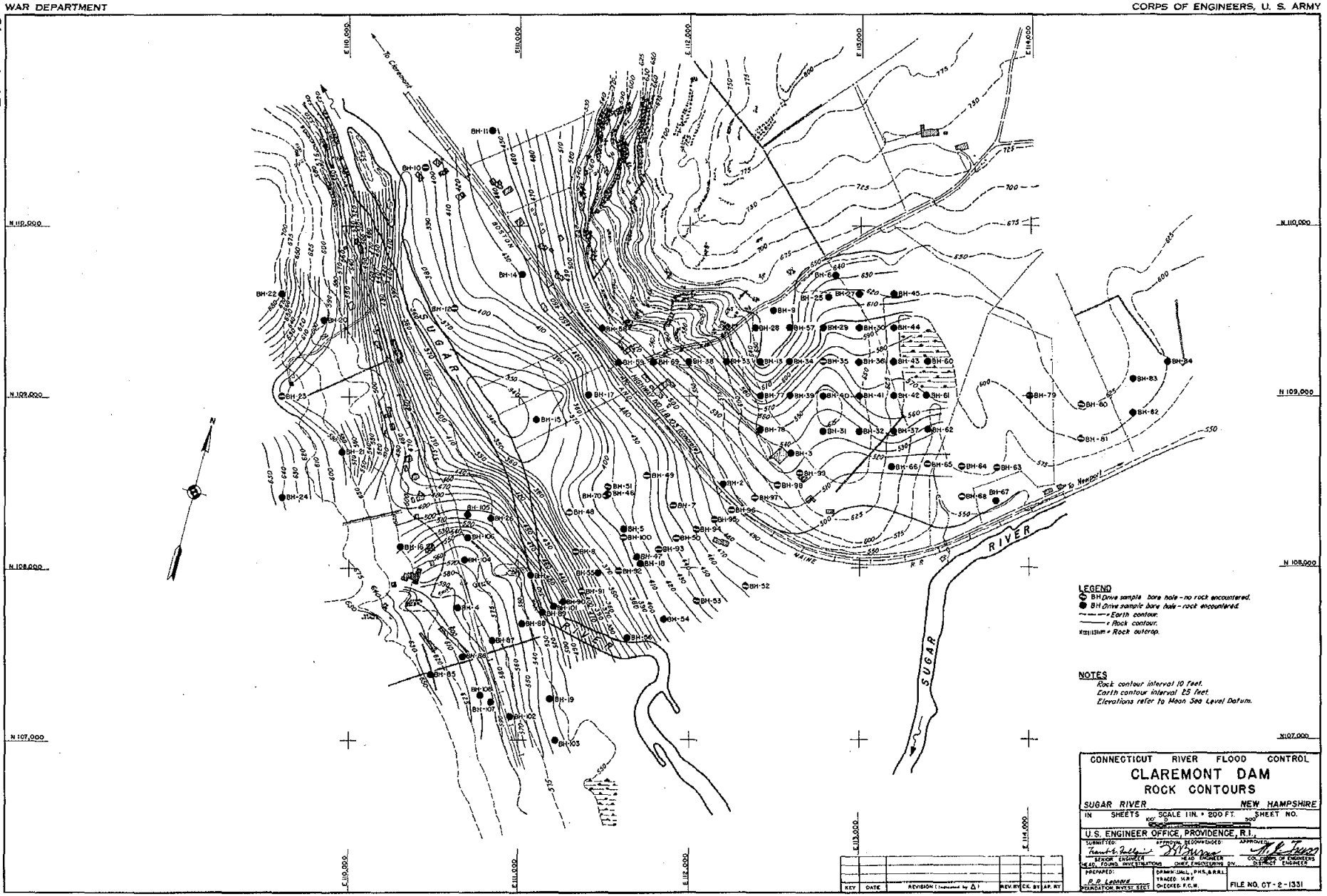
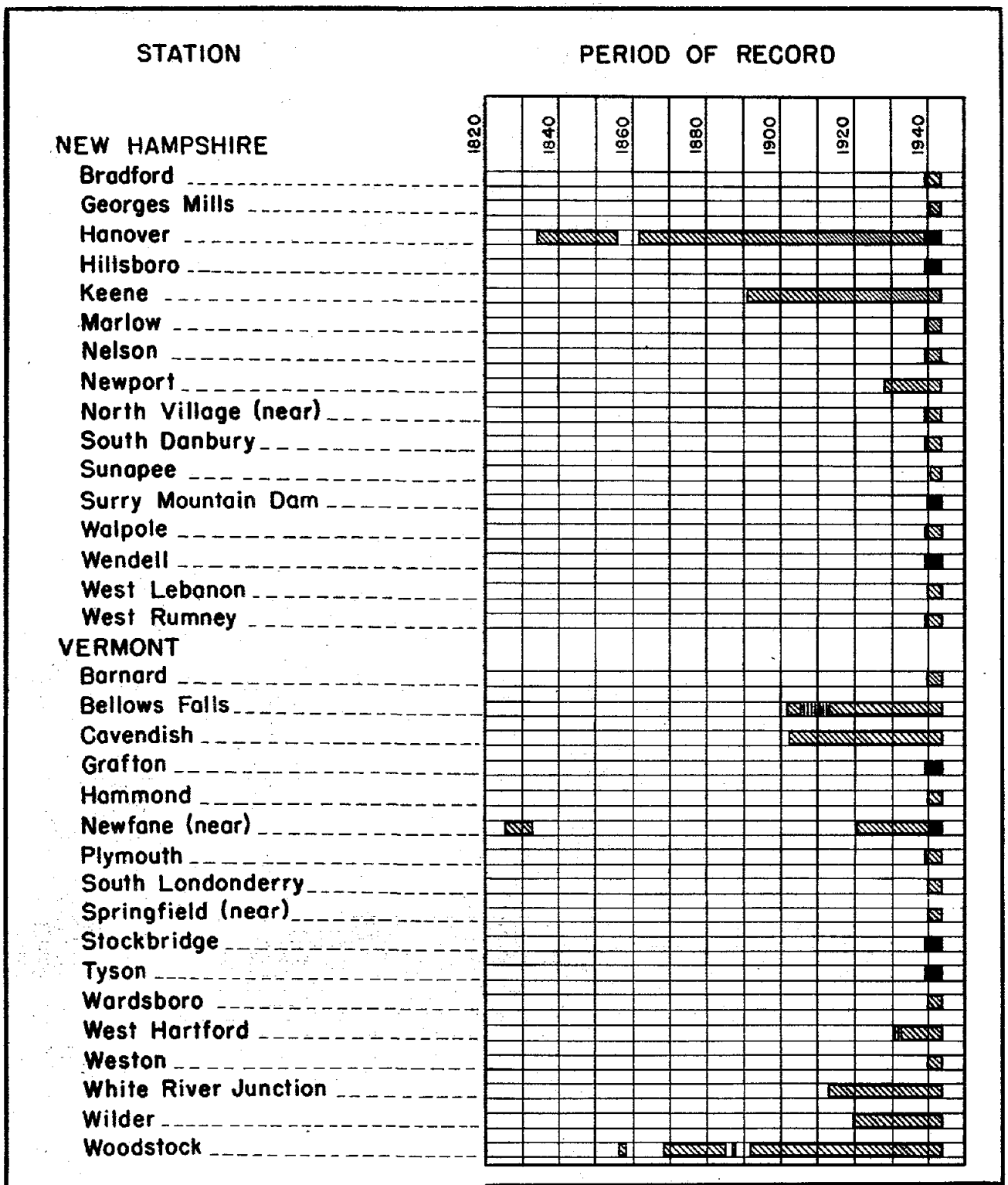


PLATE NO. 10



LEGEND

Non-recording gage record

Recording gage record

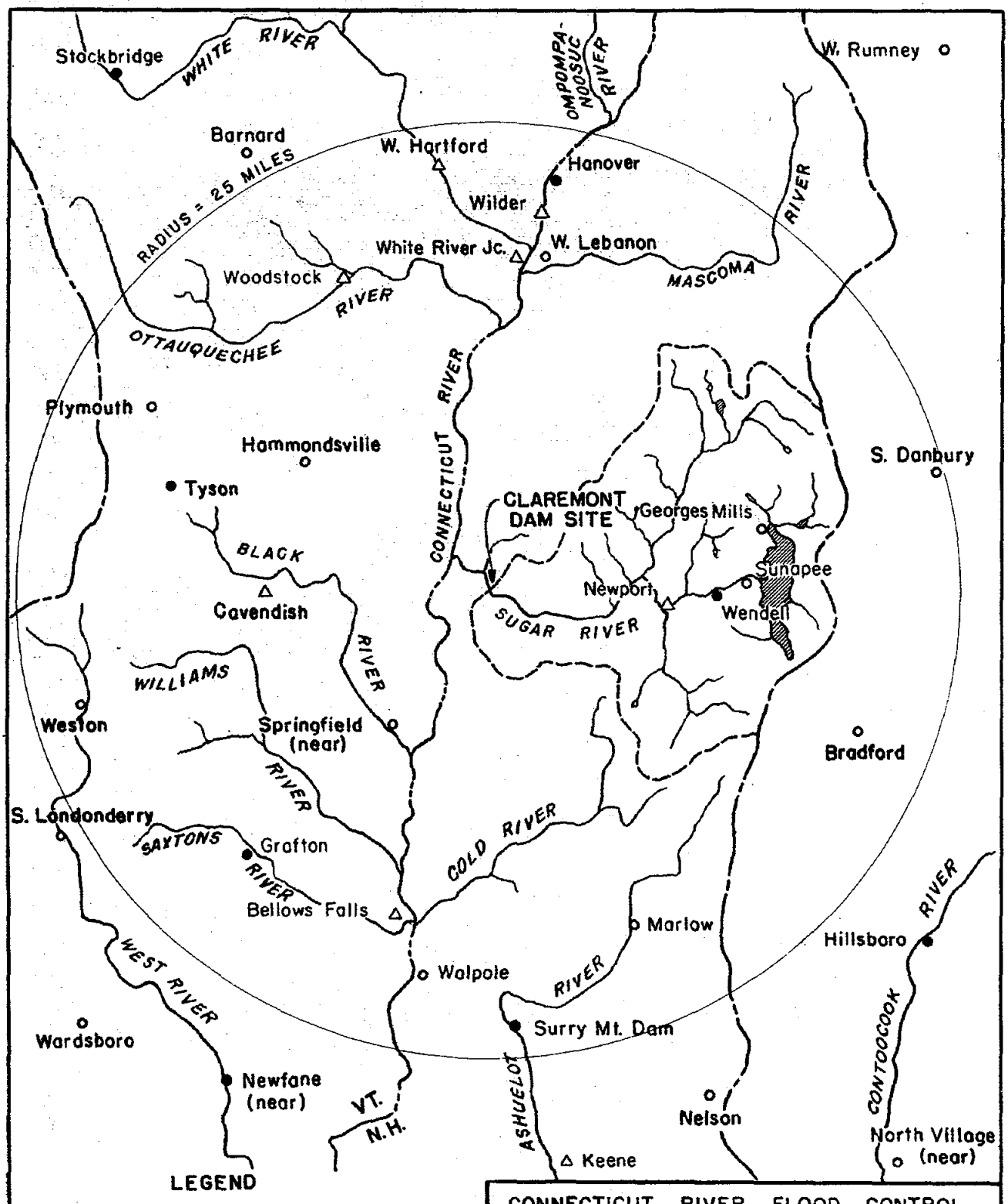
NOTE: Vertical divisions represent the end of the year indicated.

CONNECTICUT RIVER FLOOD CONTROL

PERIODS OF RECORD OF RAINFALL STATIONS IN VICINITY OF CLAREMONT WATERSHED

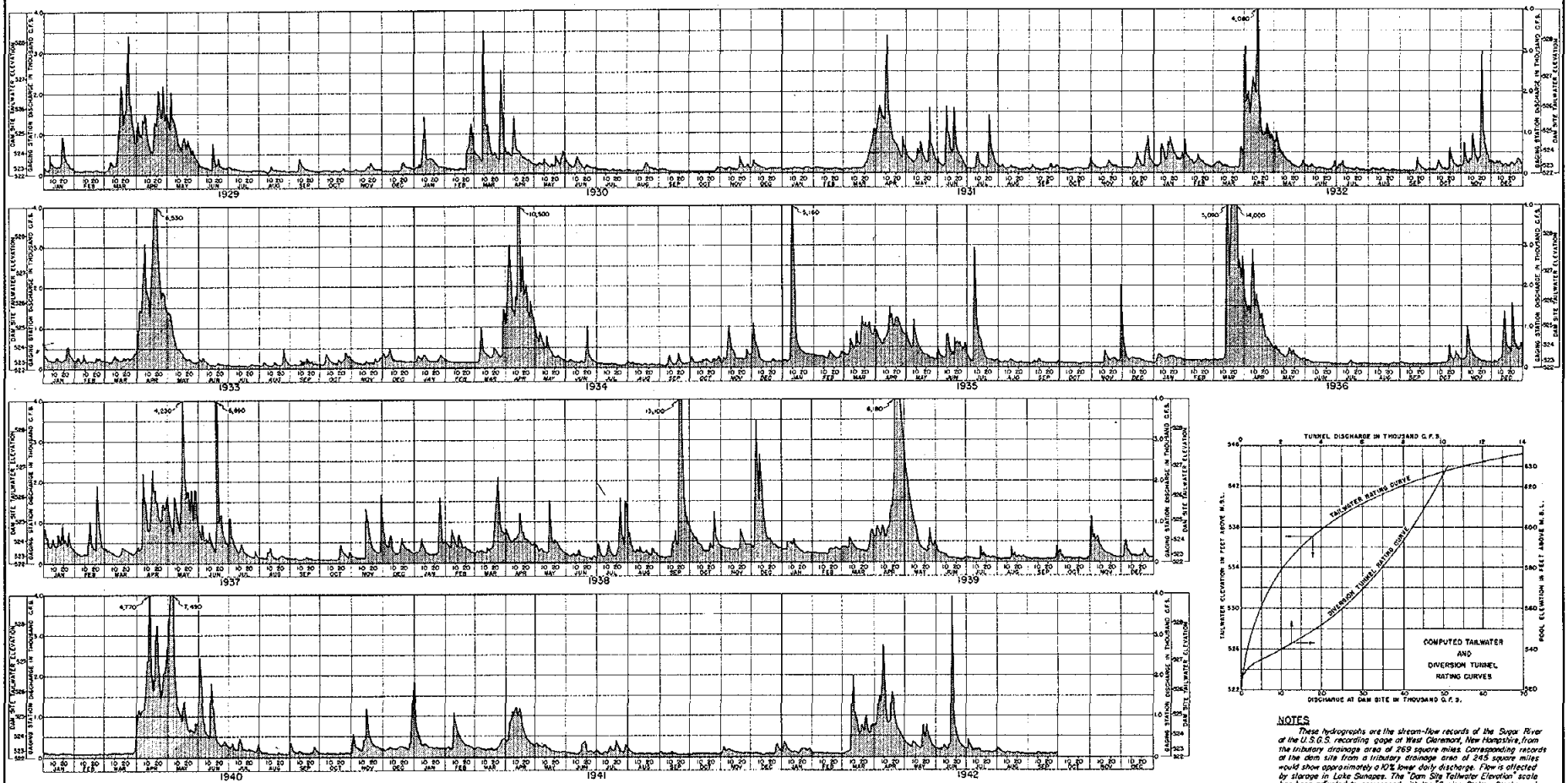
U. S. ENGINEER OFFICE PROVIDENCE, R. I.

JULY 1944



CL A. OF D.

PLATE NO. 12



NOTES

These hydrographs are the stream-flow records of the Sugar River at the U.S.G.S. recording gage at West Claremont, New Hampshire, from the tributary drainage area of 269 square miles. Corresponding records at the dam site from a tributary drainage area of 245 square miles would show approximately a 10% lower daily discharge. Flow is affected by storage in Lake Sunapee. The "Dam Site Tailwater Elevation" scale has been adjusted to correspond with the "Gaging Station Discharge in Thousand C.F.S." as plotted. Example: 3300 C.F.S. as read from the hydrograph corresponds to tailwater elevation 528 and 3000 C.F.S. at the dam site as shown on the tailwater rating curve.

NOTES (cont'd)

The data contained hereon is not intended as a representation or warranty but is furnished for information only.

It is expressly understood that the Government does not guarantee any part thereof and will not be responsible for any deduction, interpretation, or conclusion made by the contractor therefrom.

CONNECTICUT RIVER FLOOD CONTROL

CLAREMONT DAM
HYDROGRAPHS

SUGAR RIVER — NEW HAMPSHIRE

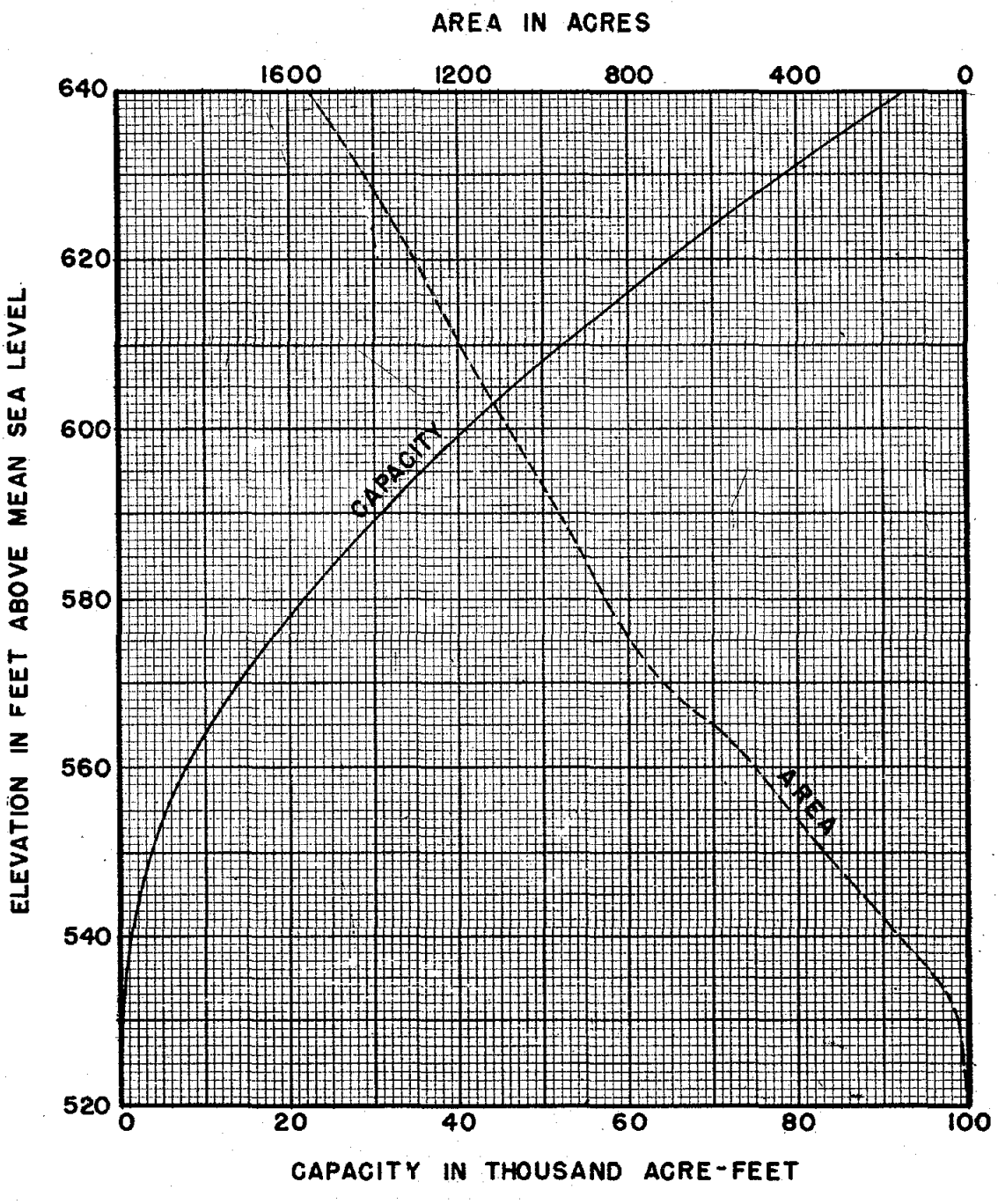
IN 94 SHEETS — SCALE AS SHOWN — SHEET NO. 3

U.S. ENGINEER OFFICE, PROVIDENCE, R.I., JAN. 1945

SUBMITTED:	APPROVAL RECOMMENDED:	APPROVED:
SENIOR ENGINEER	HEAD ENGINEER	COL. CORPS OF ENGINEERS, DISTRICT ENGINEER
HEAD, HYDRAULICS SECTION	CHIEF, ENGINEERING DIV.	
PREPARED: HYDRAULICS SECTION	DRAWN WGL; TRACED C.E.B.; CHECKED	FILE NO. CT-3-1245

CL A. OF D.

PLATE NO. 13



CONNECTICUT RIVER FLOOD CONTROL

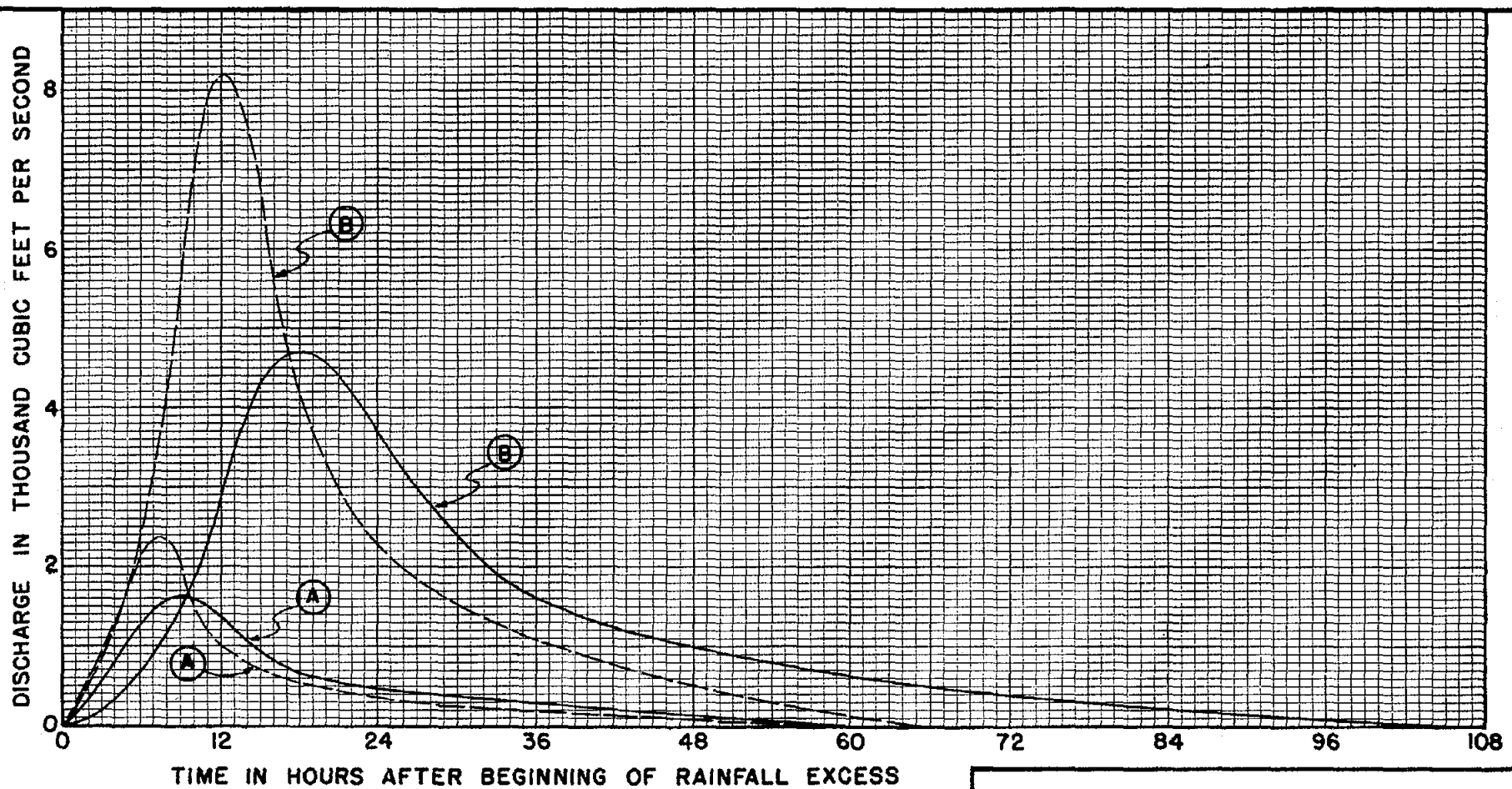
CLAREMONT RESERVOIR

AREA AND CAPACITY CURVES

SUGAR RIVER NEW HAMPSHIRE

U. S. ENGINEER OFFICE PROVIDENCE, R. I.

MARCH 1944



LEGEND

——— ADOPTED UNIT HYDROGRAPHS

– – – – PEAKED UNIT HYDROGRAPHS

(A) UNIT HYDROGRAPHS FOR SUNAPEE LAKE DRAINAGE AREA (46 SQUARE MILES).

(B) UNIT HYDROGRAPHS FOR NET DRAINAGE AREA (199 SQUARE MILES).

CONNECTICUT RIVER FLOOD CONTROL

CLAREMONT DAM

ADOPTED UNIT HYDROGRAPHS

SUGAR RIVER NEW HAMPSHIRE

U. S. ENGINEER OFFICE PROVIDENCE, R. I.

MAY 1944

CL A. OF D.

PLATE NO. 15

CONNECTICUT RIVER FLOOD CONTROL

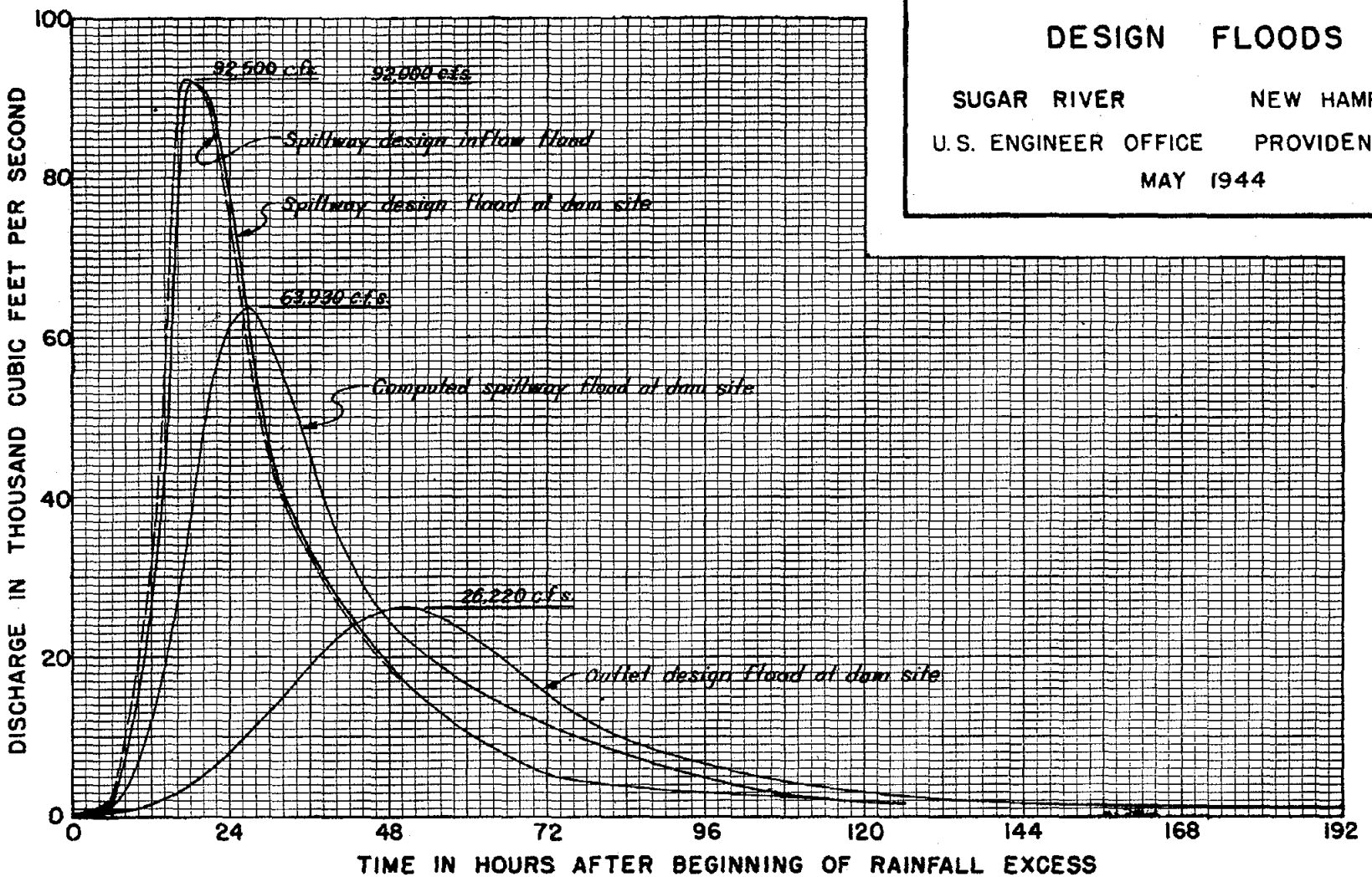
CLAREMONT DAM

DESIGN FLOODS

SUGAR RIVER NEW HAMPSHIRE

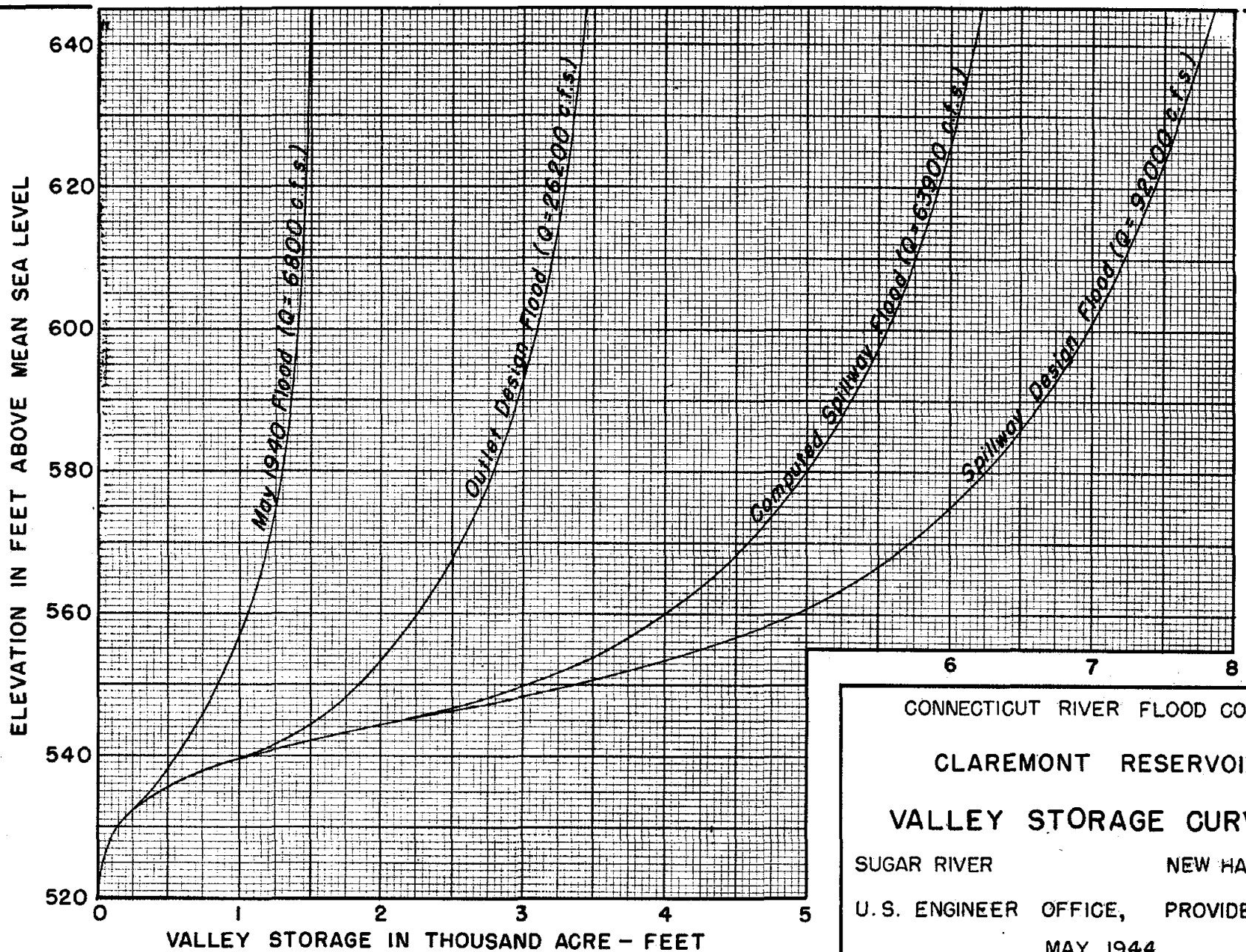
U. S. ENGINEER OFFICE PROVIDENCE, R. I.

MAY 1944



CL. A. OF D.

PLATE NO. 16



CONNECTICUT RIVER FLOOD CONTROL

CLAREMONT RESERVOIR

VALLEY STORAGE CURVES

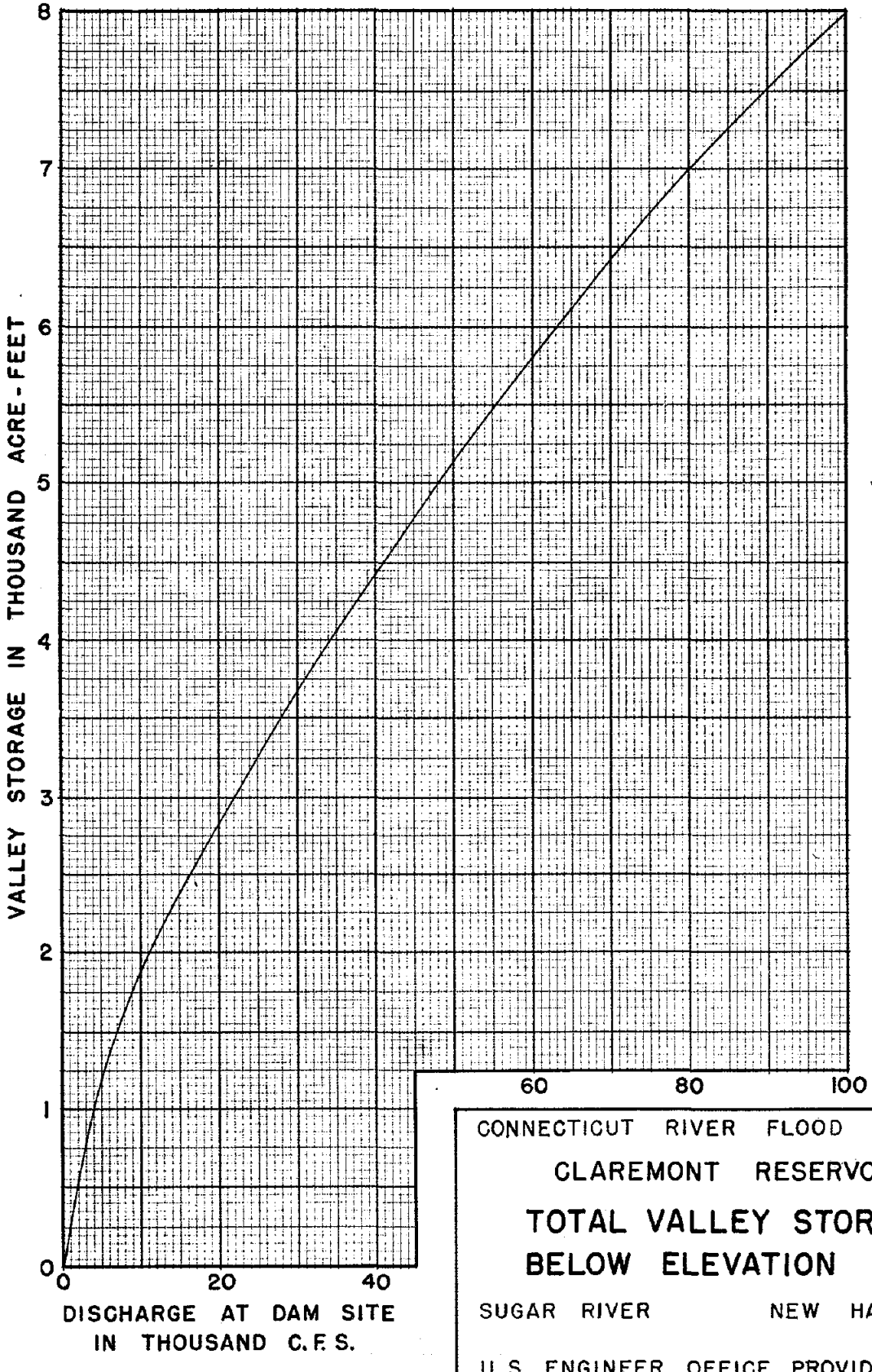
SUGAR RIVER NEW HAMPSHIRE

U.S. ENGINEER OFFICE, PROVIDENCE, R.I.

MAY 1944

CL A. OF D.

PLATE NO. 17



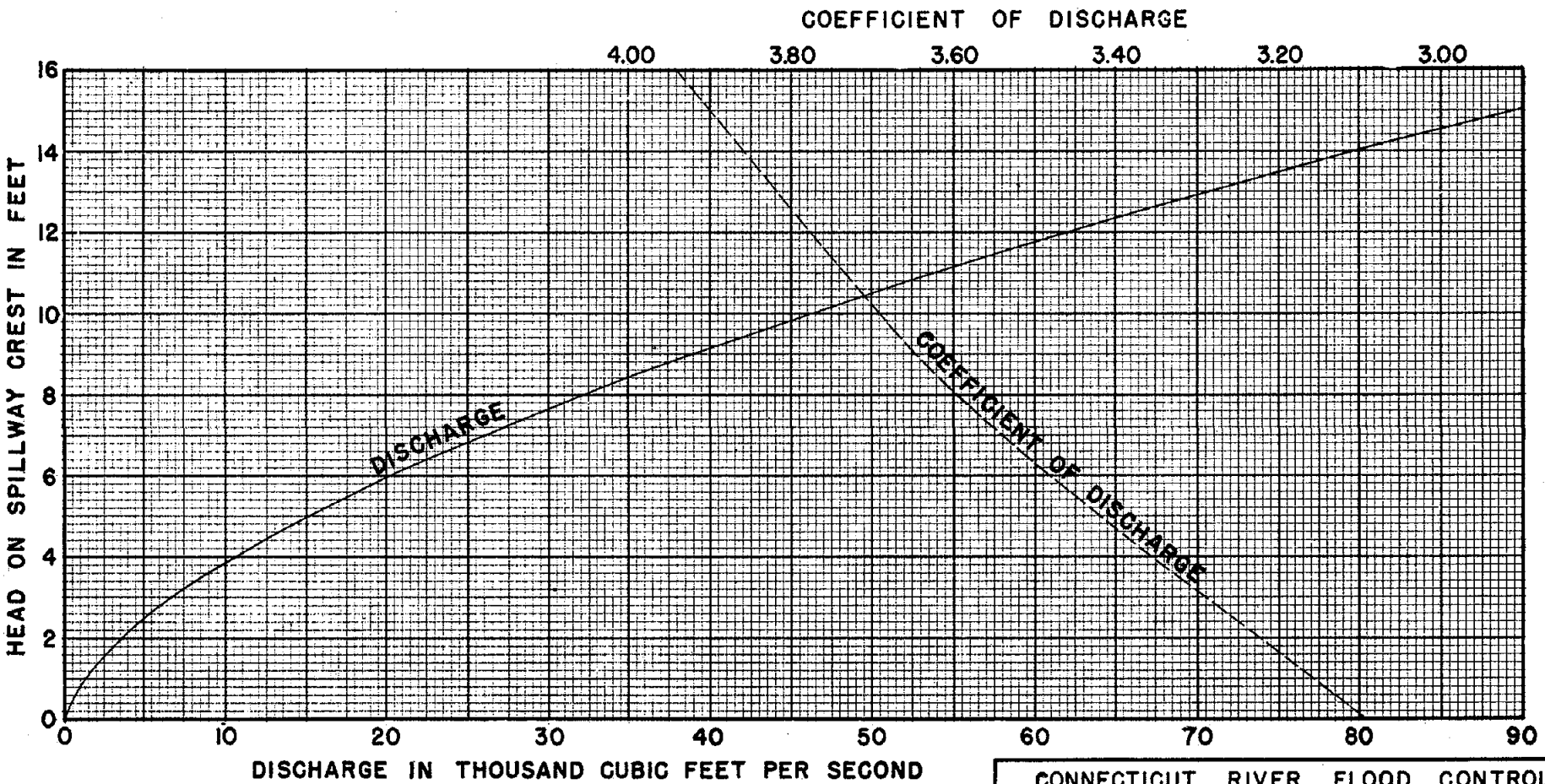
CONNECTICUT RIVER FLOOD CONTROL

CLAREMONT RESERVOIR

TOTAL VALLEY STORAGE BELOW ELEVATION 630

SUGAR RIVER NEW HAMPSHIRE

U. S. ENGINEER OFFICE PROVIDENCE, R. I.
MAY 1944



SPILLWAY CREST LENGTH = 395 FEET

CONNECTICUT RIVER FLOOD CONTROL

CLAREMONT DAM

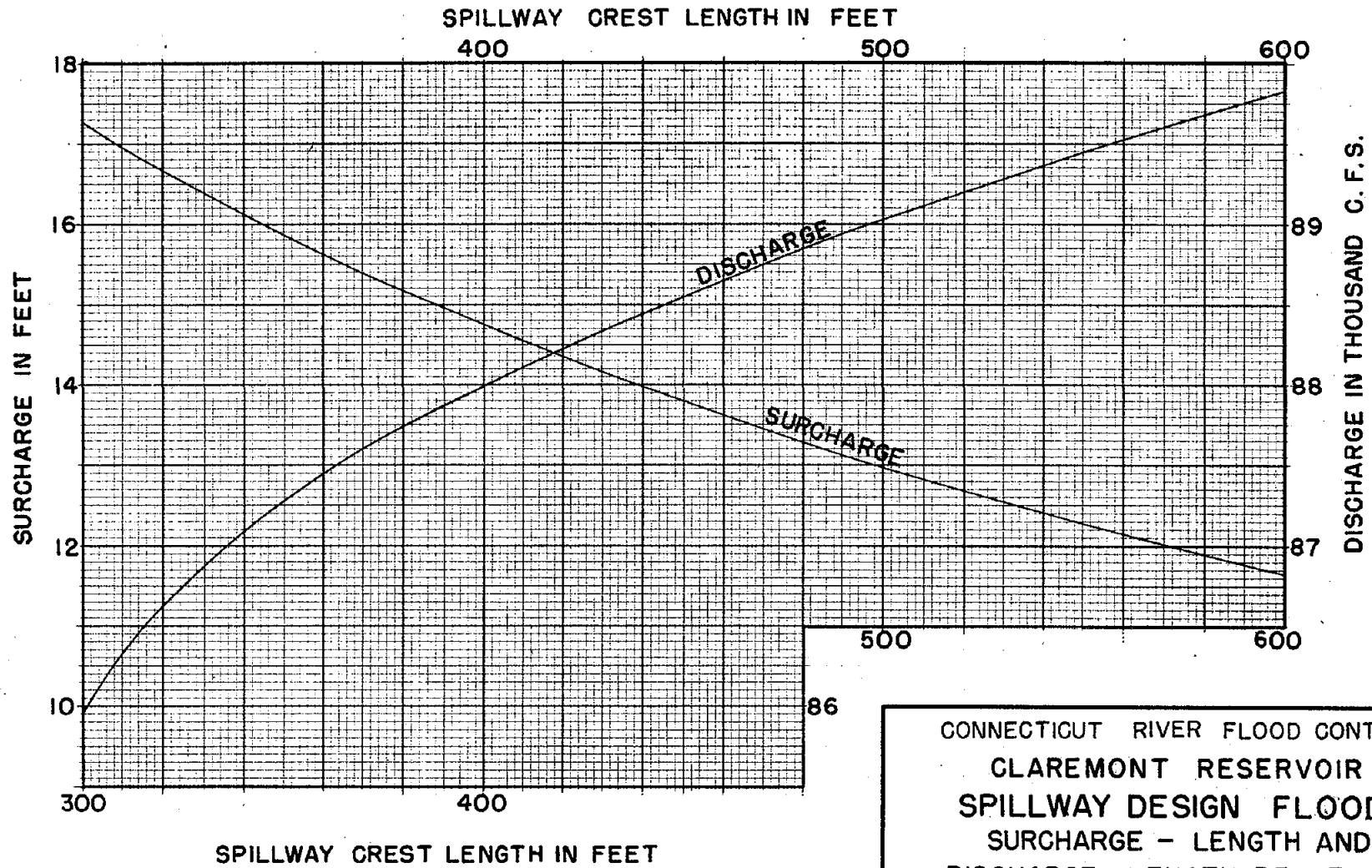
SPILLWAY RATING CURVE

SUGAR RIVER NEW HAMPSHIRE

U. S. ENGINEER OFFICE PROVIDENCE, R. I.
MAY 1944

CL A. OF D.

PLATE NO. 19



CONNECTICUT RIVER FLOOD CONTROL

CLAREMONT RESERVOIR

SPILLWAY DESIGN FLOOD

SURCHARGE - LENGTH AND

DISCHARGE - LENGTH RELATIONS

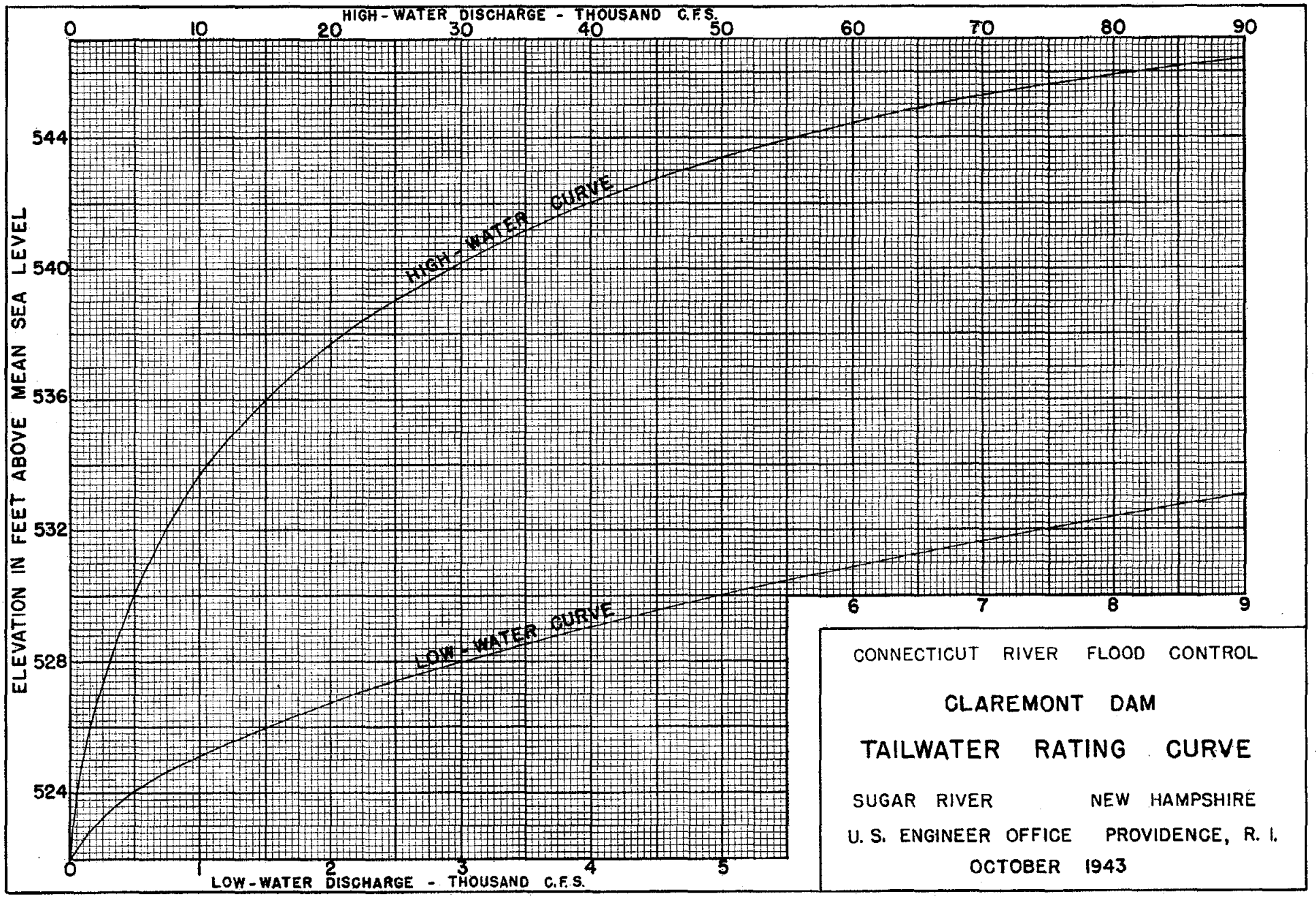
SUGAR RIVER NEW HAMPSHIRE

U.S. ENGINEER OFFICE PROVIDENCE, R.I.

MAY 1944

CL A. OF D.

PLATE NO. 20



CONNECTICUT RIVER FLOOD CONTROL

CLAREMONT DAM

TAILWATER RATING CURVE

SUGAR RIVER NEW HAMPSHIRE

U. S. ENGINEER OFFICE PROVIDENCE, R. I.

OCTOBER 1943

CL A. OF D.

PLATE NO. 21

PROVIDENCE DISTRICT SOIL CLASSIFICATION

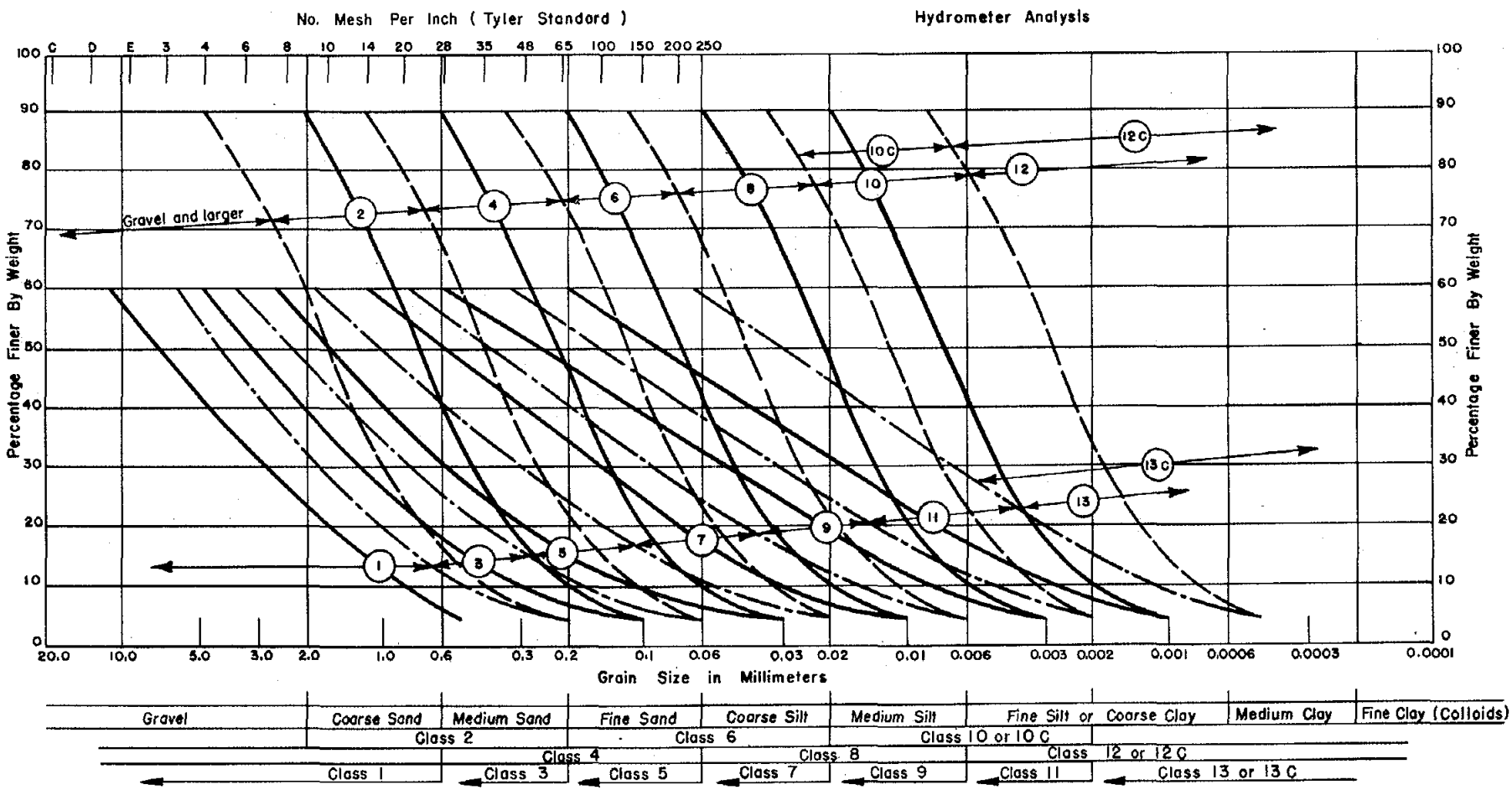
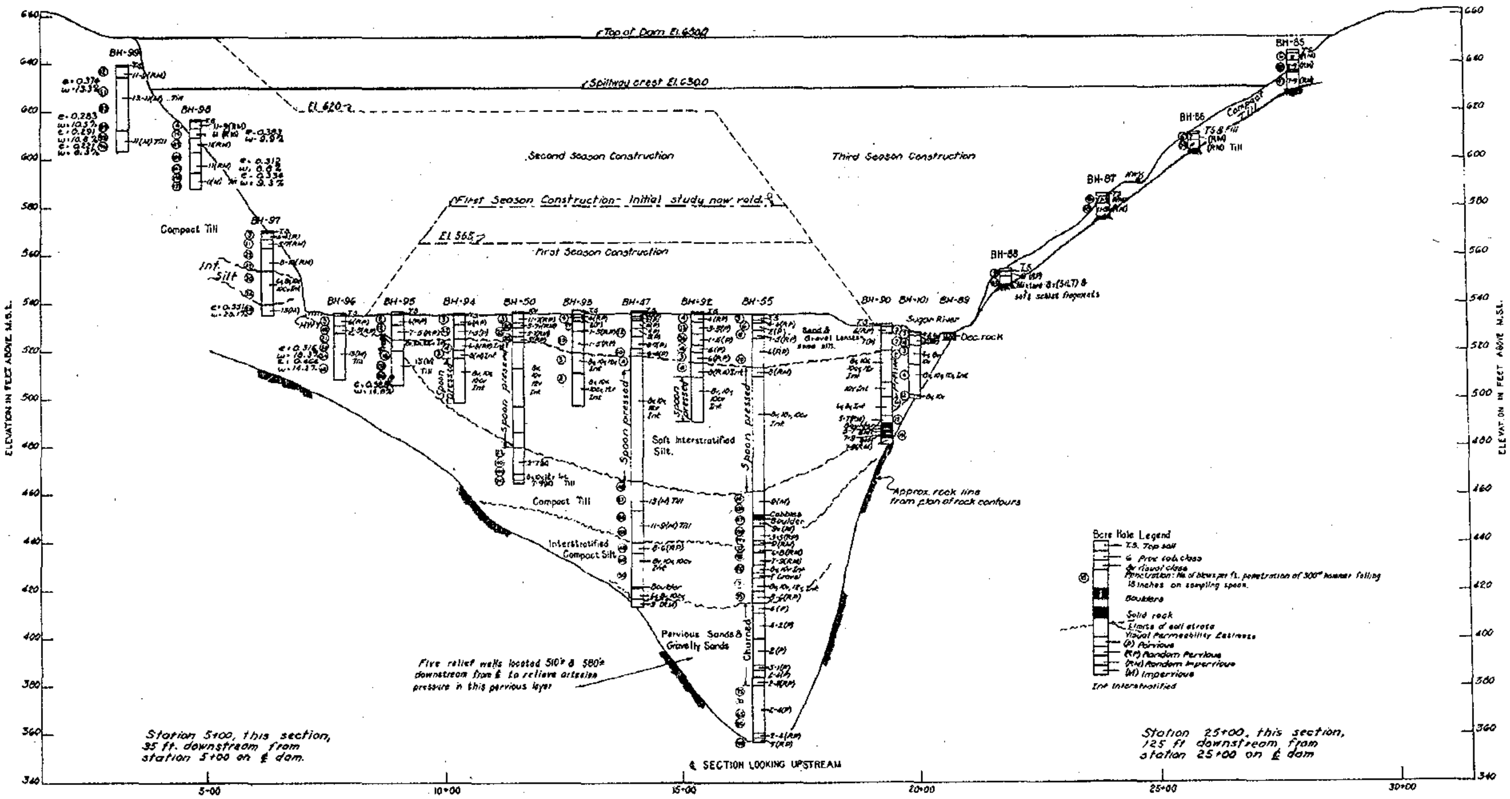


DIAGRAM SHOWING LIMITS OF SOIL CLASSES

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY

CL A. OF D.



Note:
Section not parallel to center line of dam.
Horizontal stations not to indicated scale.

KEY	DATE	REVISION (indicated by △)	REV. BY	CK. BY	AP. BY

CONNECTICUT RIVER FLOOD CONTROL

CLAREMONT DAM

SOIL PROFILE ON APPROXIMATE ℄ OF DAM

SUGAR RIVER — NEW HAMPSHIRE

IN SHEETS — SCALES: HOR. 1" = 100' VERT. 1" = 20' — SHEET NO.

U.S. ENGINEER OFFICE, PROVIDENCE, R.I.

SUBMITTED: ENGINEER, HEAD, SOILS LABORATORY

SOILS LABORATORY STUDY

PREPARED: SOILS LABORATORY — DRAWN: — TRACED: — CHECKED:

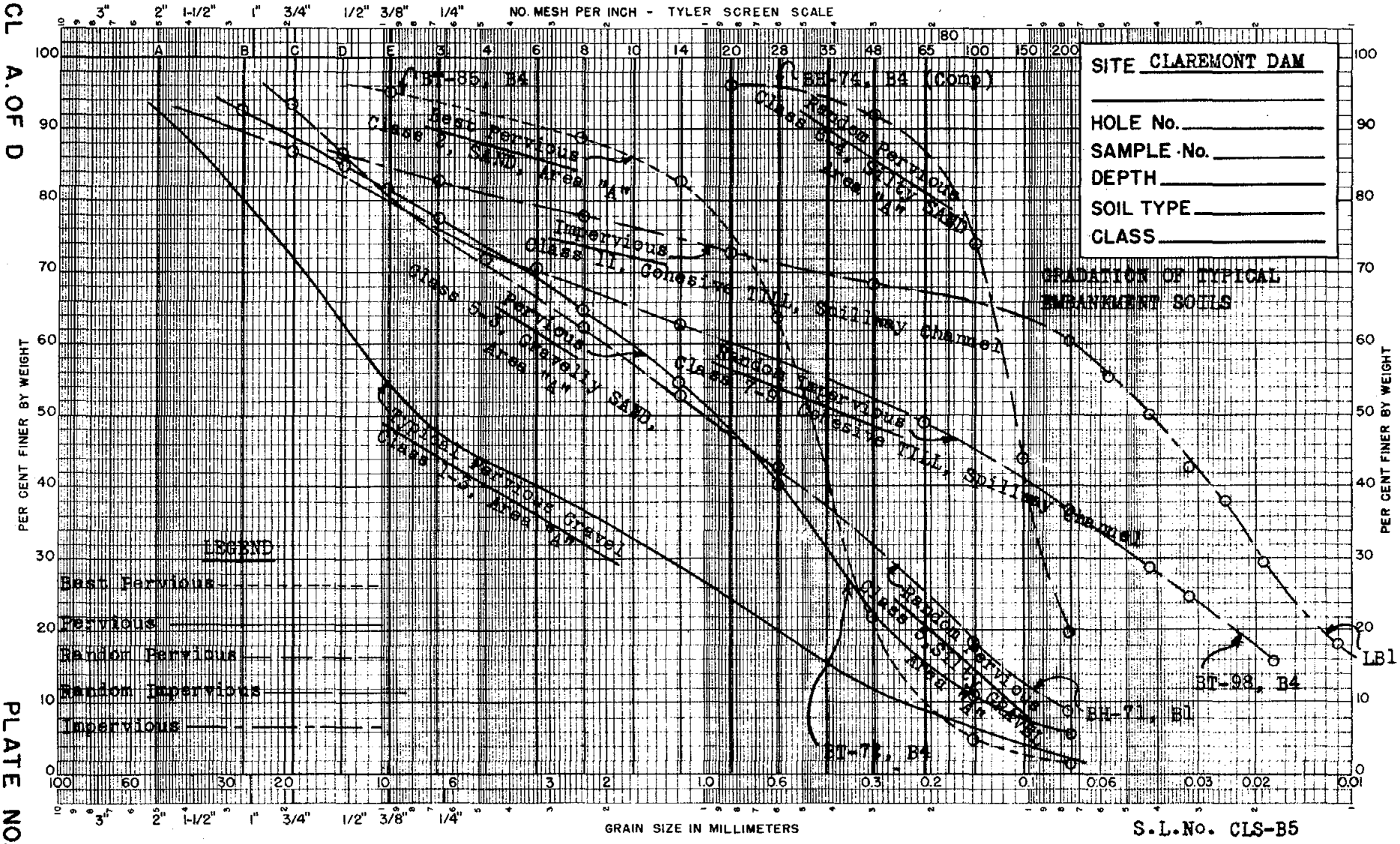
SL. NO. CLS - A29

FILE NO.

PLATE NO. 23

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY



S.L.No. CLS-B5

Gravel	Coarse Sand	Medium Sand	Fine Sand	Coarse Silt	Medium Silt
Class 1	Class 2		Class 6		
		Class 4		Class 8	
	Class 3		Class 5	Class 7	Class 9

SOILS LABORATORY, ENGINEERING DIVISION

MECHANICAL ANALYSIS

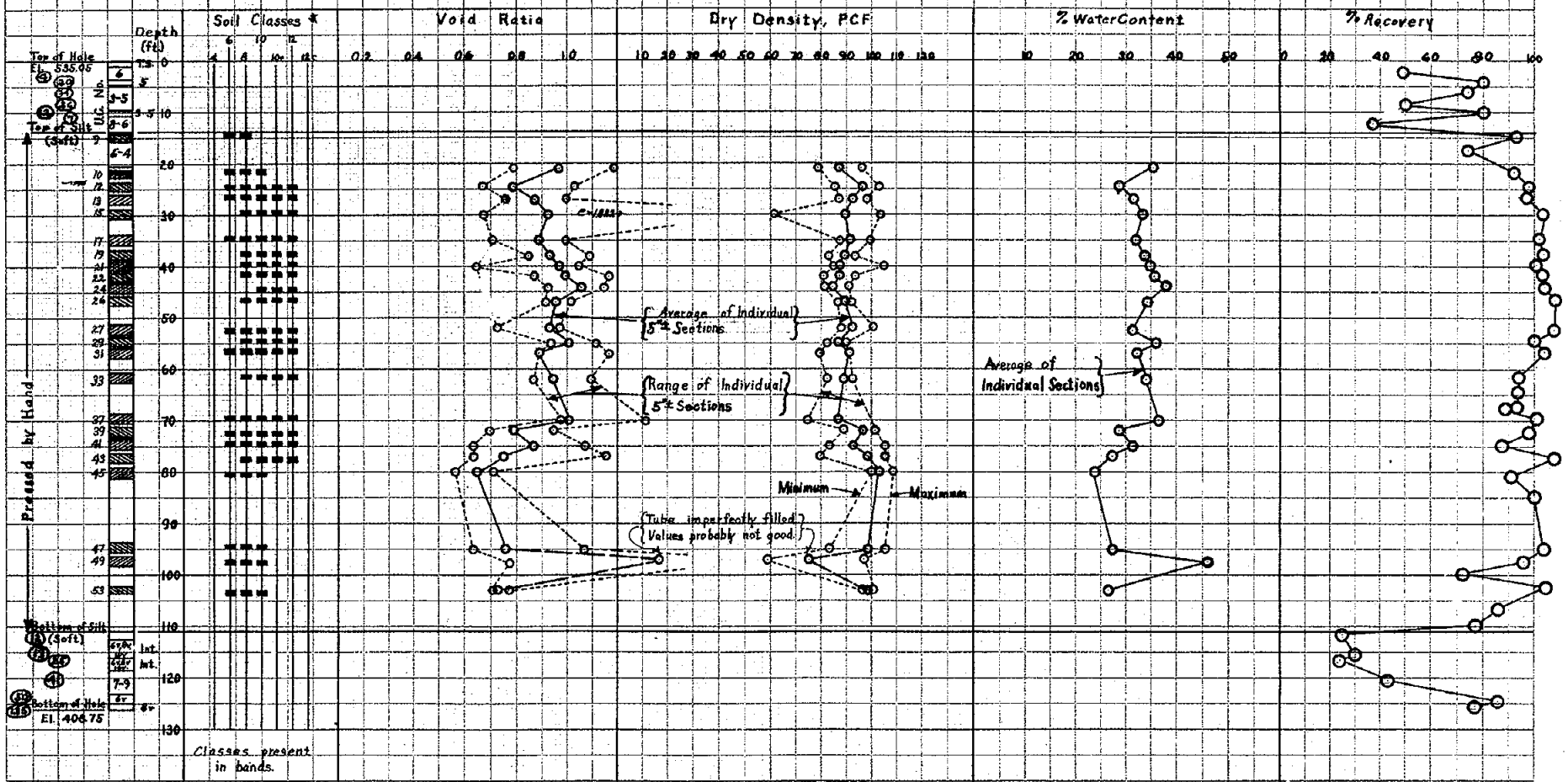
PROVIDENCE, R.I.

CL A. OF D

PLATE NO. 24

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY



(4) Penetration in blows per foot of 18" fall of 300# hammer.

* Visual Classifications

Undisturbed sample in soft silt stratum

Indicates classifications of stratified materials occurring in silt strata

Figures in graphic log represent soil classifications occurring above and below soft silt stratum

See detailed laboratory log for void ratios of individual sections of samples.

SUMMARY SHEET
BH-51
2" SHELBY TUBE SAMPLING
CLAREMONT DAM

TYPICAL BORING WITH PROPERTIES OF FOUNDATION SILT

Water contents computed from void ratios for S%=100. Water contents and void ratios are overall values for 5"+ sections.

S.L.No. CLS-A31

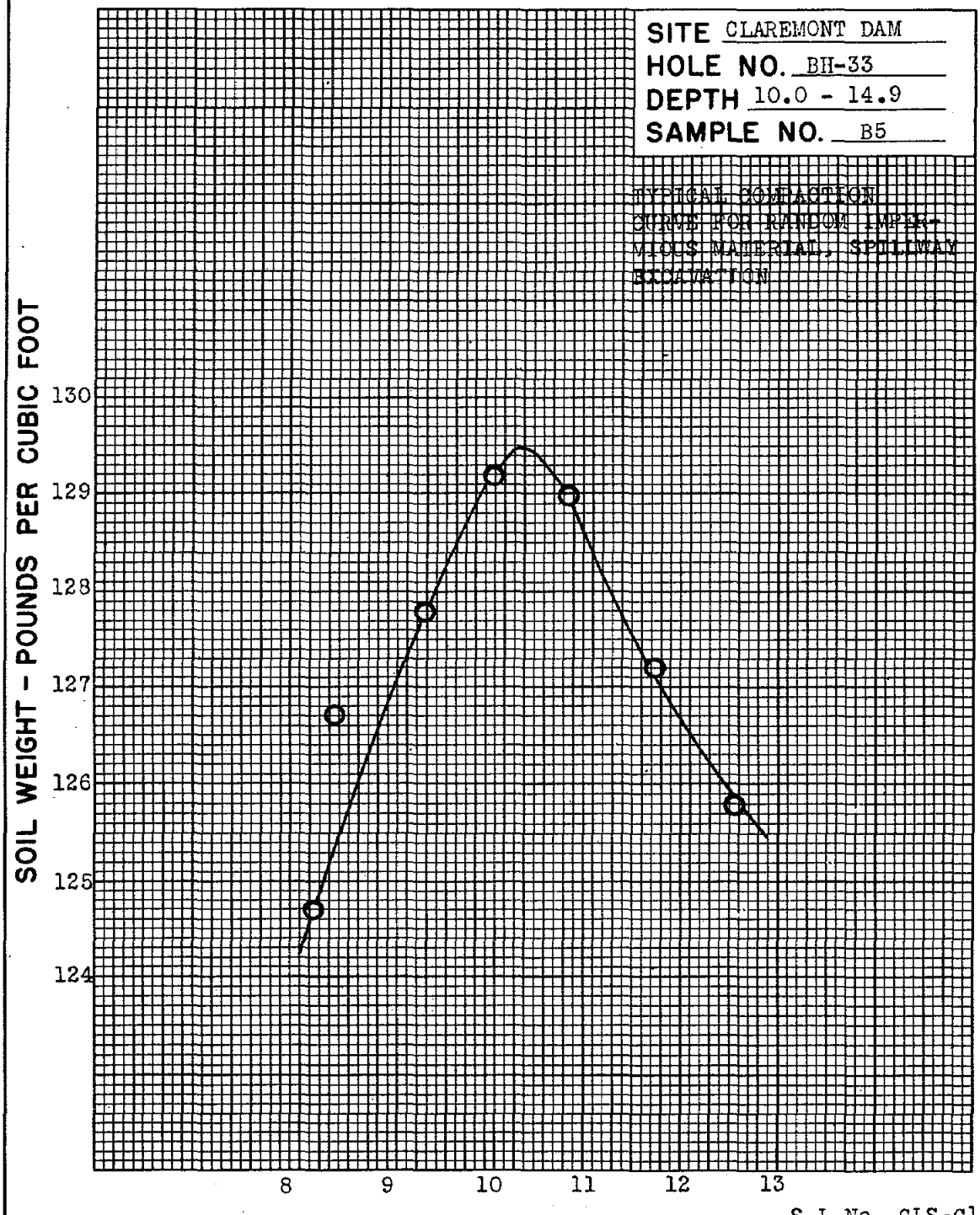
ENGINEERING DIVISION - SOILS LABORATORY

PROVIDENCE, R. I.

CL A. OF D.

PLATE NO. 25

COMPACTION CHARACTERISTICS



S.L.No. CLS-C1

Class 5-7 (RM)

MATERIAL SCREENED OUT

Minimum Size, mm. -

Per Cent by weight -

e_c = 0.316

No. Blows/Layer 25

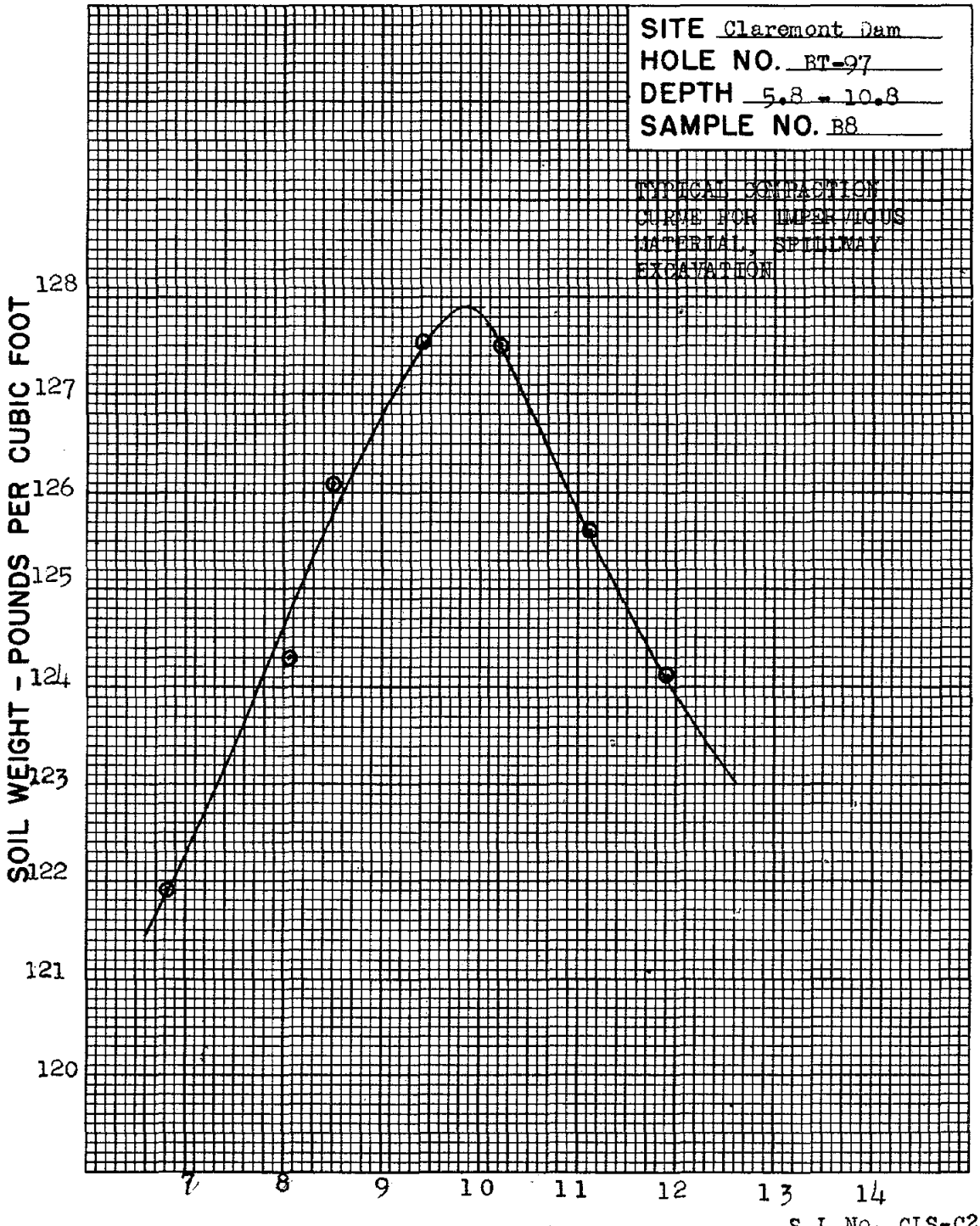
Area of Tamper, sq. in. 3.14

Weight of Tamper, lbs. 5.5

Fall of Tamper, in. 12

COMPACTION CHARACTERISTICS

SITE Claremont Dam
HOLE NO. BT-97
DEPTH 5.8 - 10.8
SAMPLE NO. B8



S.L.No. CLS-C2

Class 11 (M)

MATERIAL SCREENED OUT

Minimum Size, mm. 38.1
Per Cent by weight 6.5

No. Blows/Layer 25
Area of Tamper, sq. in. 3.14
Weight of Tamper, lbs. 5.5
Fall of Tamper, in. 12.0

U. S. GOVERNMENT PRINTING OFFICE : 1938—O-140027

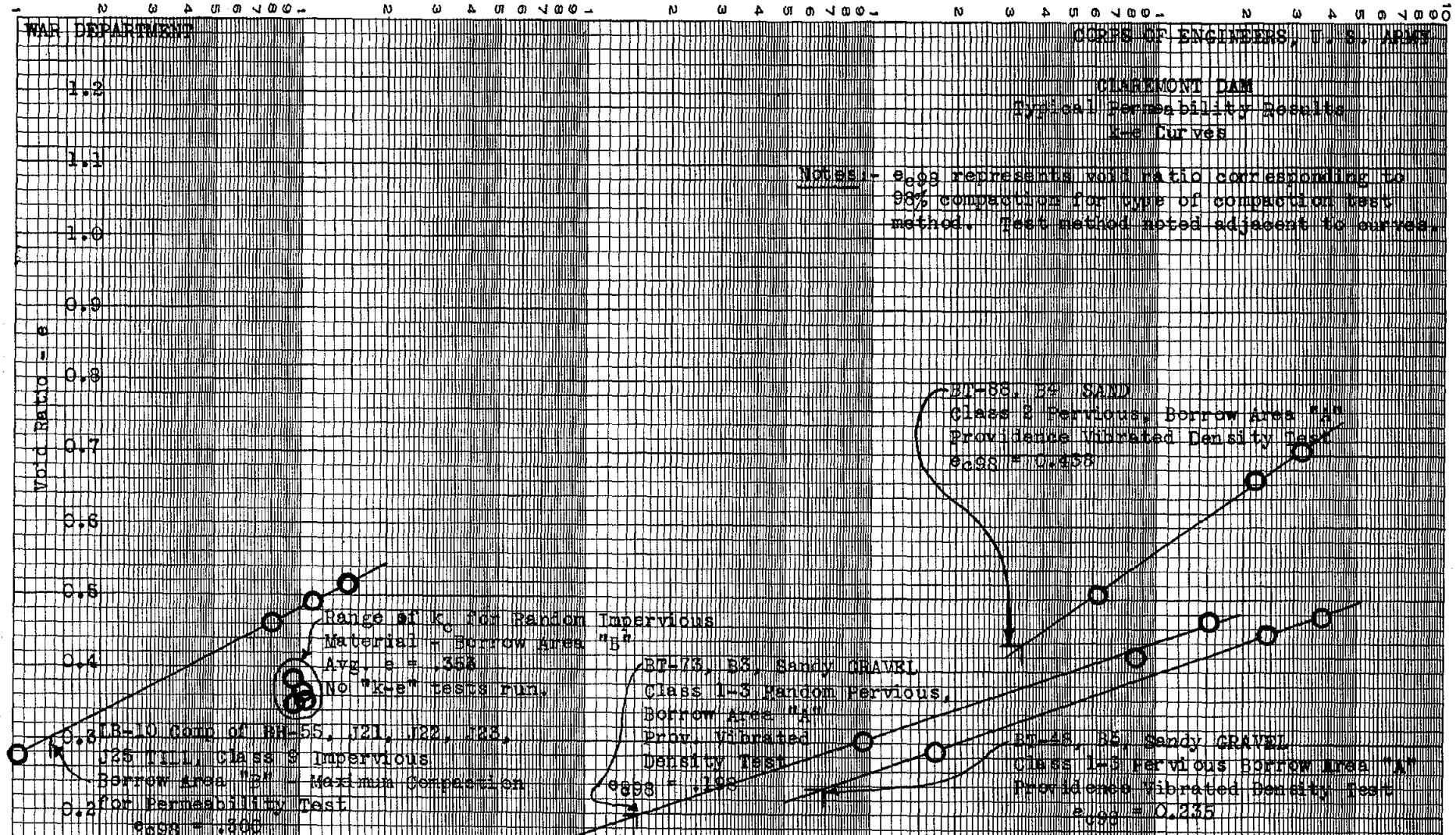
e at opt. w.c. = 0.344

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY

CLAREMONT DAM
Typical Permeability Results
k-e Curves

Notes:- e_{c98} represents void ratio corresponding to 98% compaction for type of compaction test method. Test method noted adjacent to curves.



S.L. No. CLS-58

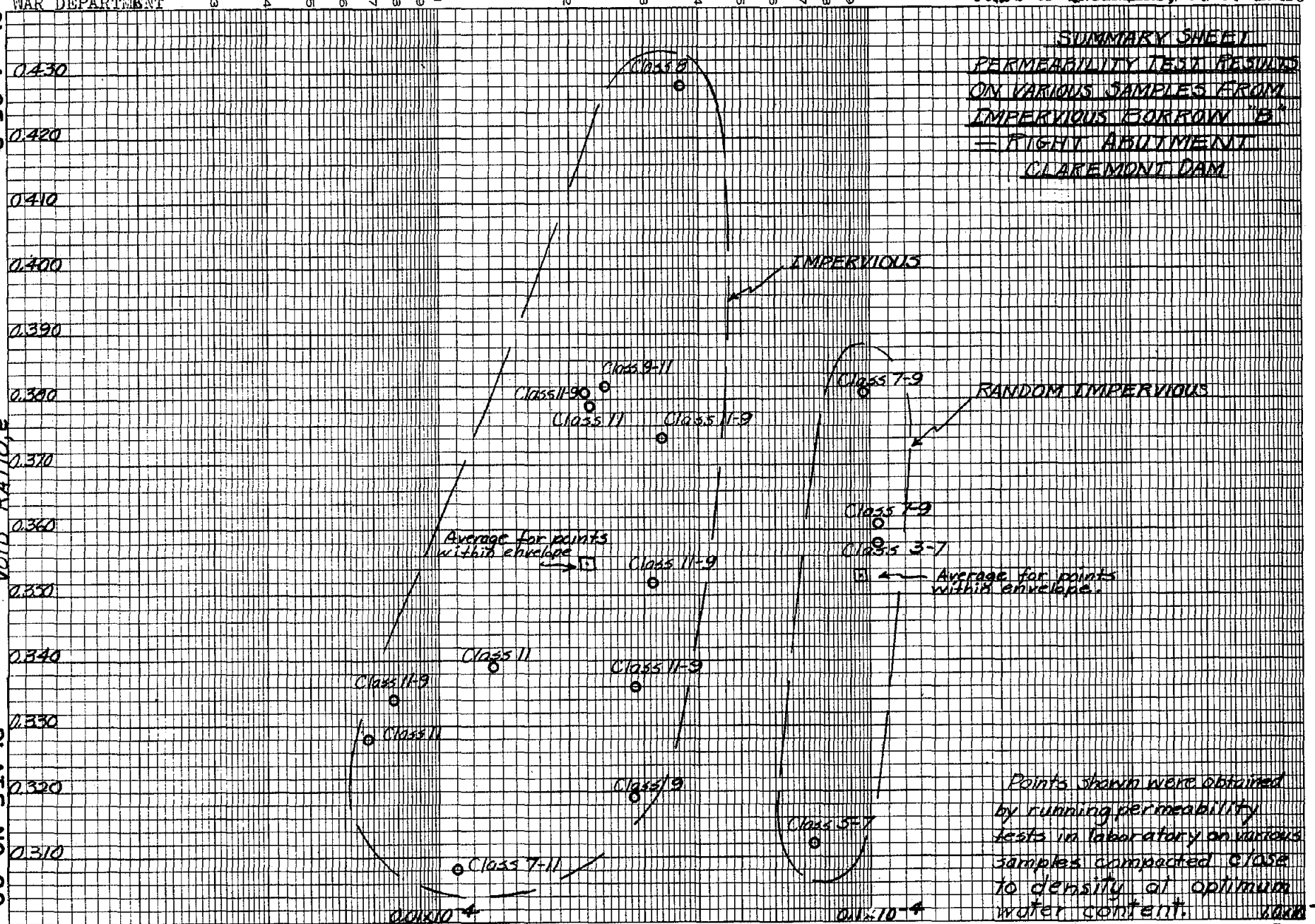
ENGINEERING DIVISION - SOILS LABORATORY

PROVIDENCE, R. I.

CL A. OF D

PLATE NO. 28

WAR DEPARTMENT — CORPS OF ENGINEERS, U. S. ARMY

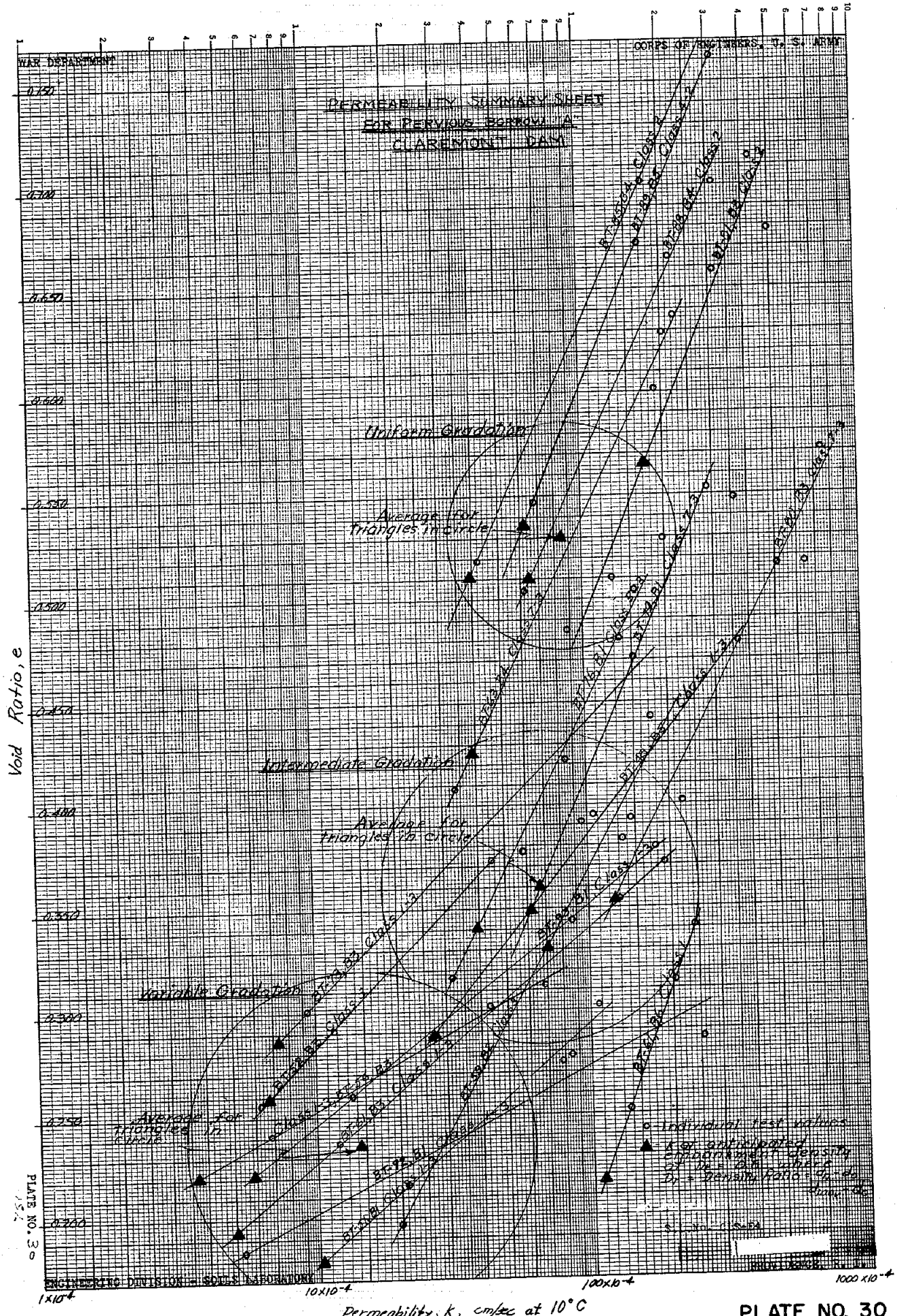


PERMEABILITY, k, cm/sec, at 10°C.

CL A OF D

PLATE NO. 29

ENGINEERING DIVISION - SOILS LABORATORY — S.L.No.CES-E3 — PROVIDENCE, R. I.



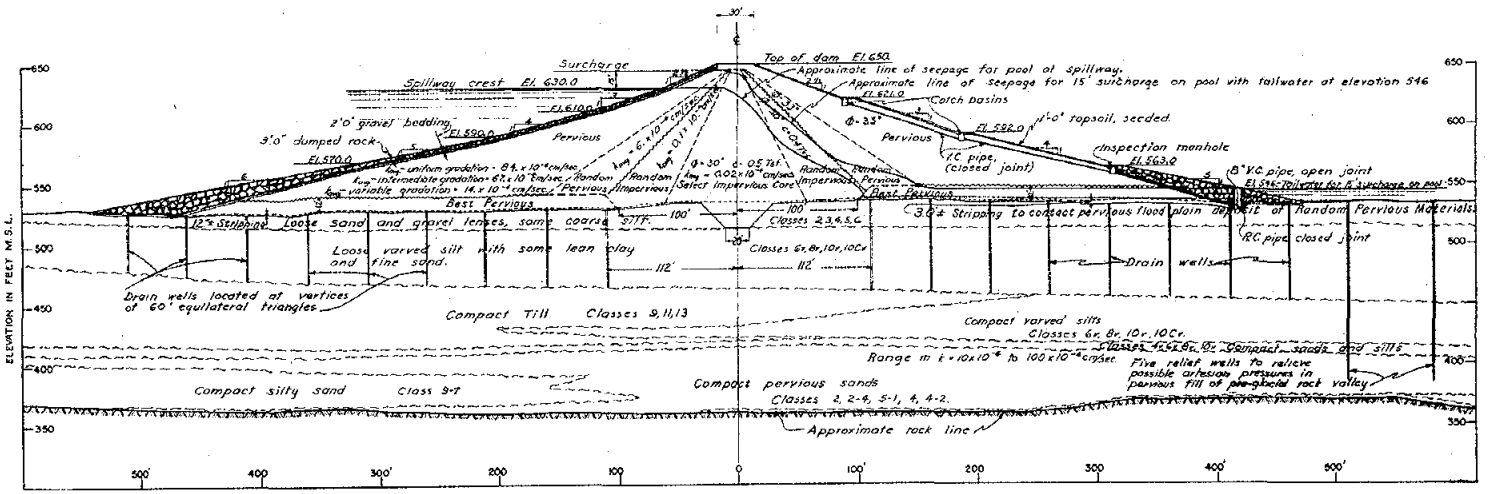
CL A. OF D.

PLATE NO. 30

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY

CL A. OF D.



CLAREMONT DAM
GENERALIZED SECTION

KEY	DATE	REVISION (indicated by △)	REVIEW	CK. BY	AP. BY

CONNECTICUT RIVER FLOOD CONTROL

CLAREMONT DAM
GENERALIZED SECTION

SUGAR RIVER — NEW HAMPSHIRE

IN 1 SHEETS — SCALE 1" = 40' — SHEET NO. 1

U.S. ENGINEER OFFICE, PROVIDENCE, R.I., JAN. 1945

SUBMITTED: ENGINEER, HEAD, SOILS LABORATORY

SOILS LABORATORY STUDY

PREPARED: SOILS LABORATORY

DRAWN: R.D.L. TRACED: CHECKED:

SL. NO. CLS - A 30
FILE NO.

PLATE NO. 31

SHEAR TEST

SUPPLEMENTARY DATA

Rate of strain _____

Consolidation _____

Shear Plane _____

Remarks *Average shear values, direct shear test.*

_____ Class _____

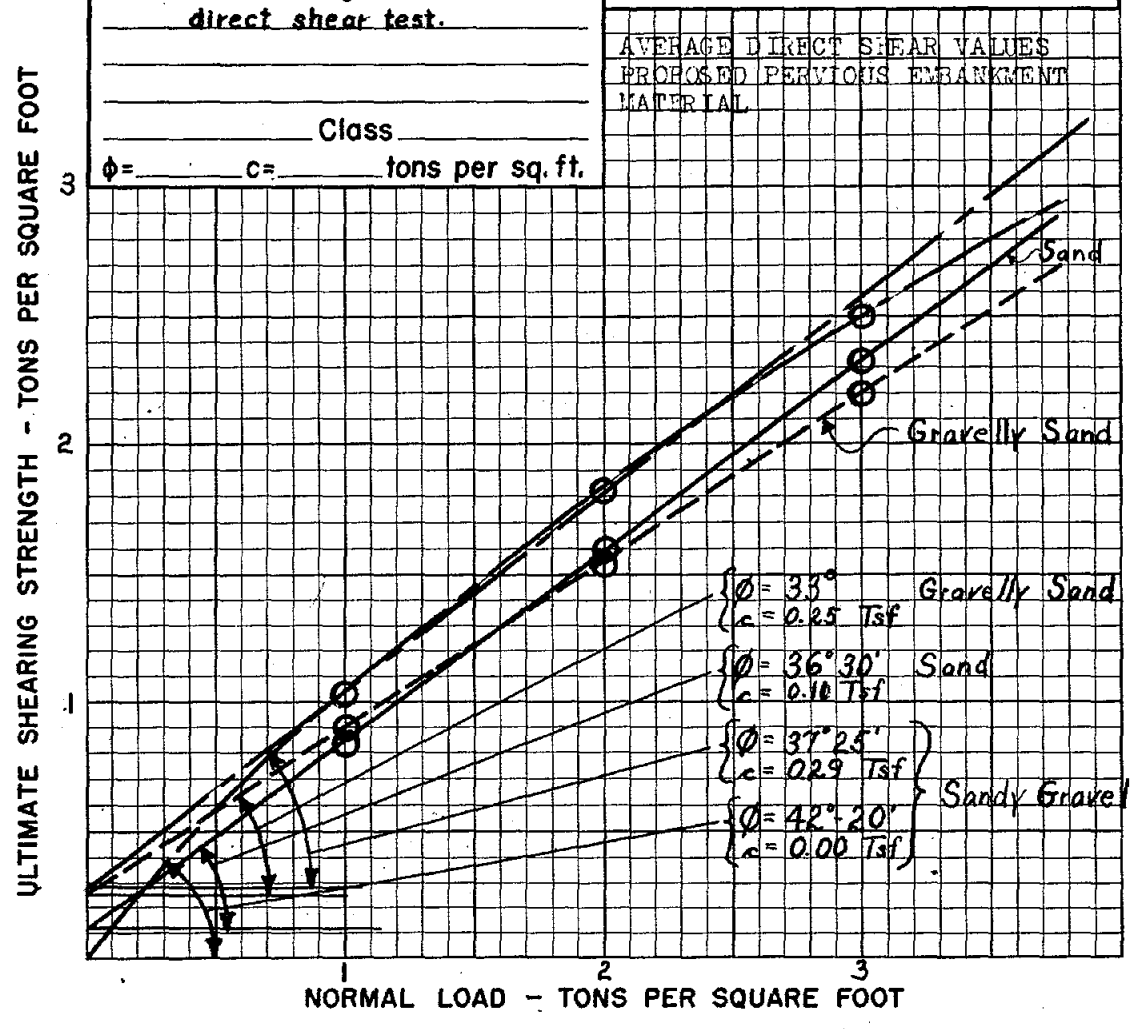
ϕ= _____ c= _____ tons per sq. ft.

SITE *CLAREMONT DAM BORROW AREA "A"*

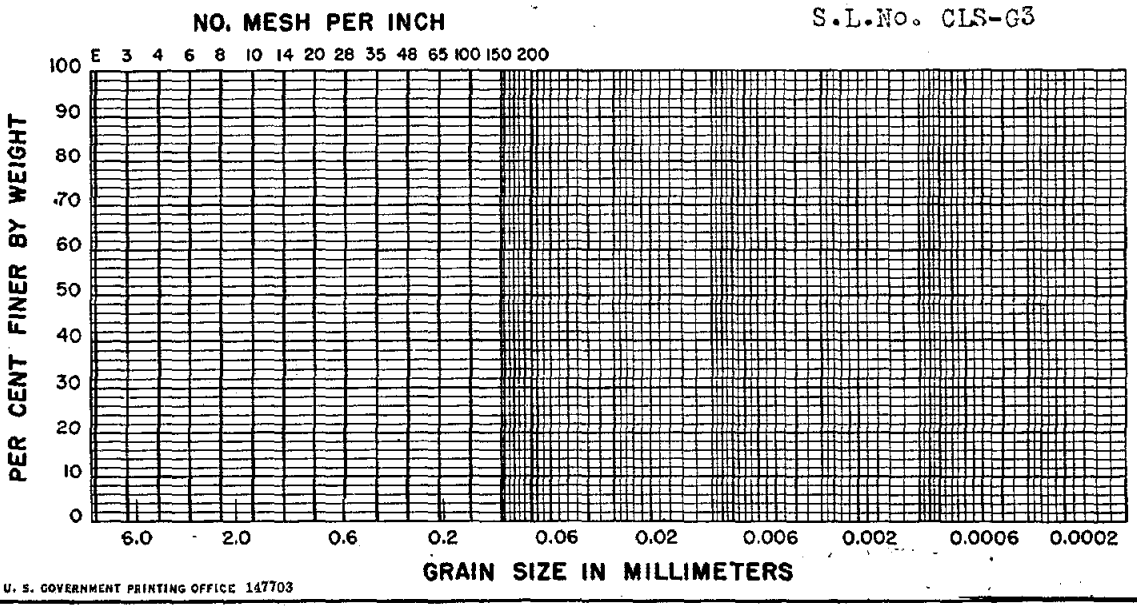
HOLE NO. _____

SAMPLE NO. _____

DEPTH _____



S.L.No. CLS-G3



U. S. GOVERNMENT PRINTING OFFICE 147703

SHEAR TEST

SUPPLEMENTARY DATA

Rate of strain _____

Consolidation _____

Shear Plane _____

Remarks *Average shear values, direct shear test.*

_____ Class _____

ϕ= _____ c= _____ tons per sq. ft.

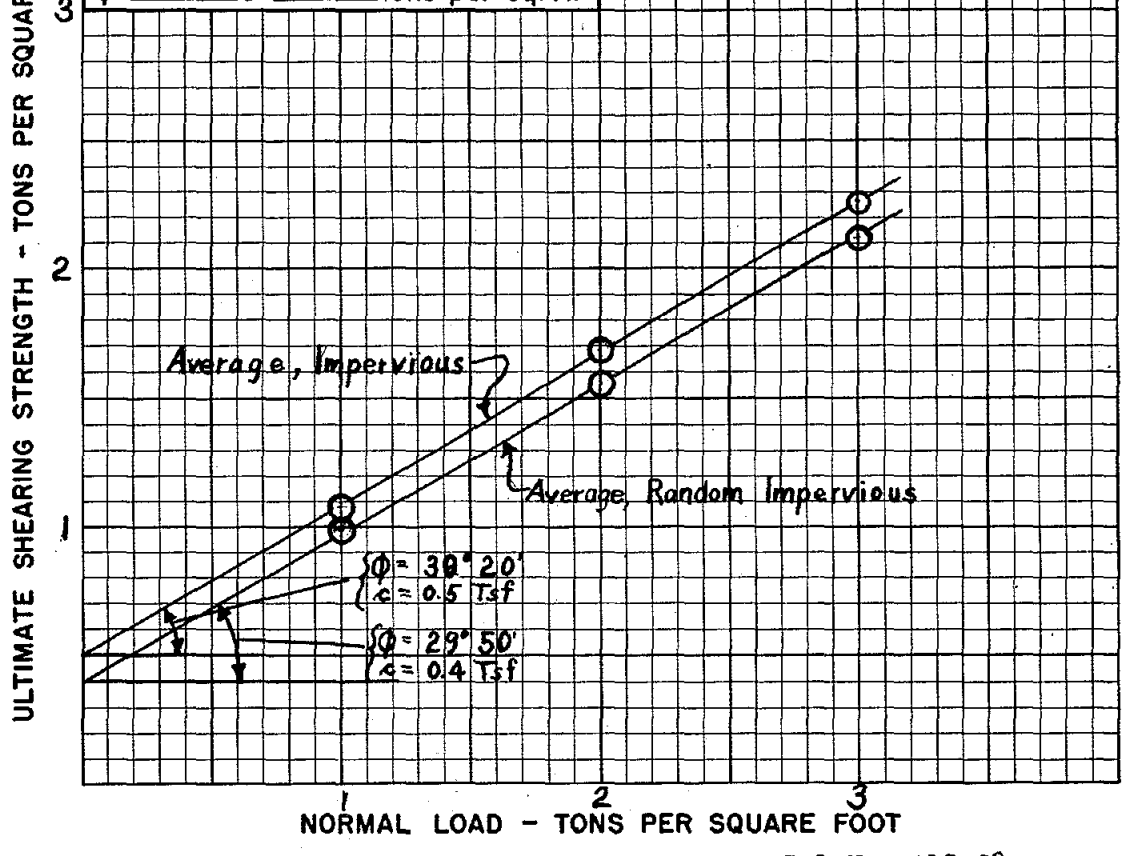
SITE CLAREMONT DAM
BORROW AREA "B"

HOLE NO. _____

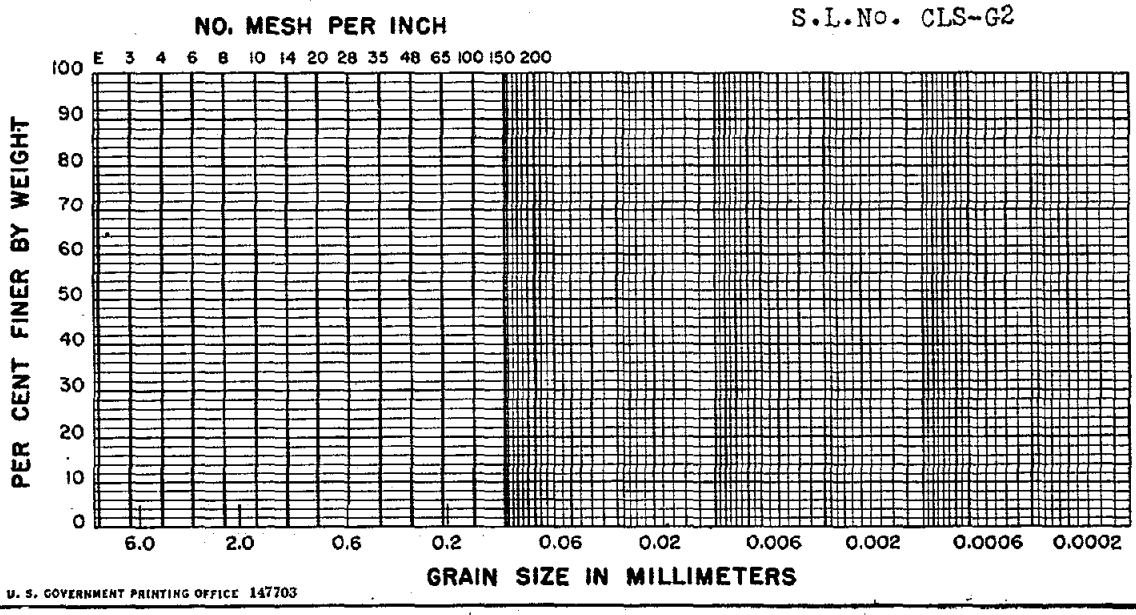
SAMPLE NO. _____

DEPTH _____

AVERAGE DIRECT SHEAR VALUES, PROPOSED IMPERVIOUS EMBANKMENT MATERIAL, BORROW AREA "B"



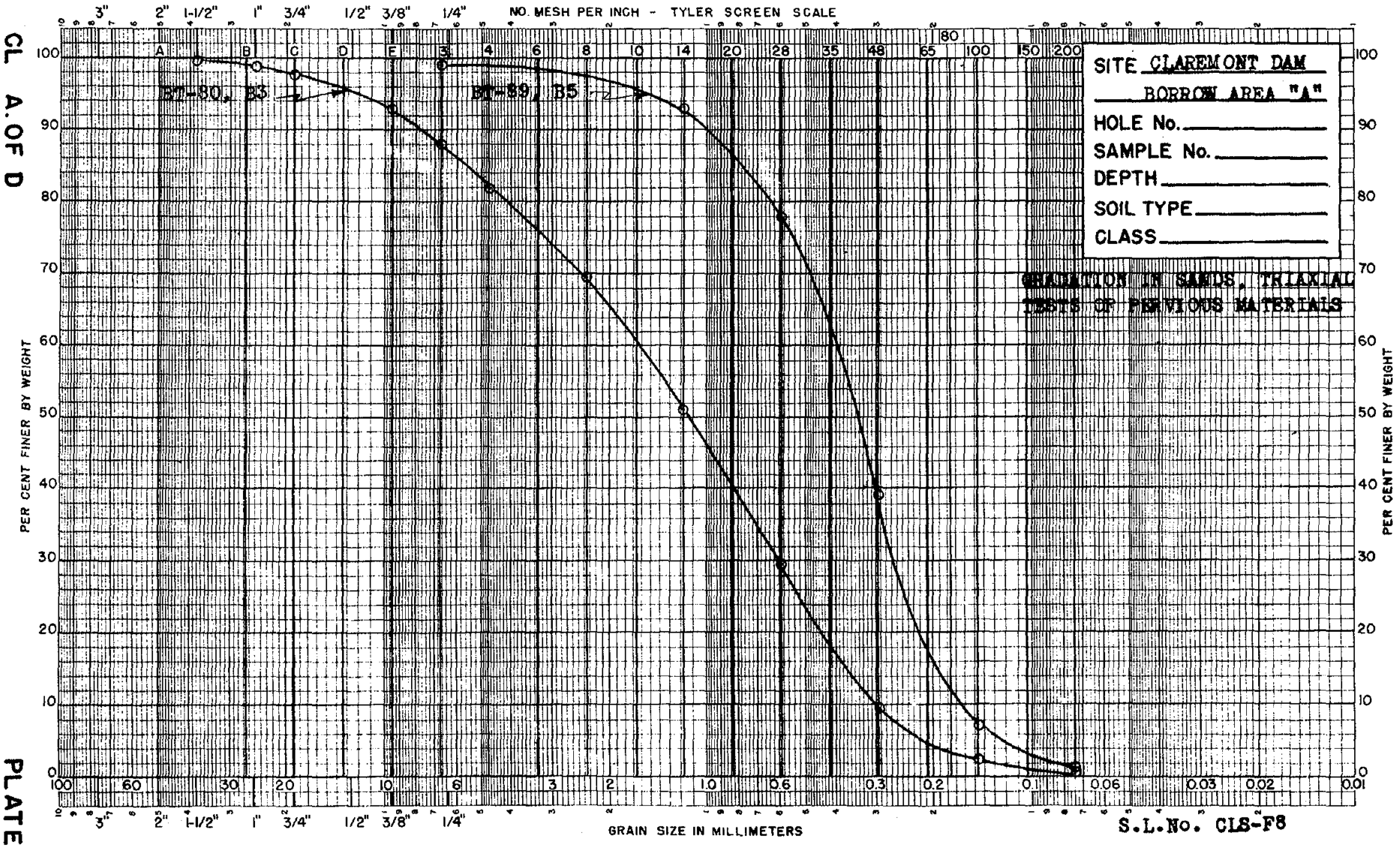
S.L.No. CLS-G2



U. S. GOVERNMENT PRINTING OFFICE 147703

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY



S.L.No. CLS-F8

Gravel	Coarse Sand	Medium Sand	Fine Sand	Coarse Silt	Medium Silt
Class 1	Class 2		Class 6		
		Class 4		Class 8	
	Class 3		Class 5	Class 7	Class 9

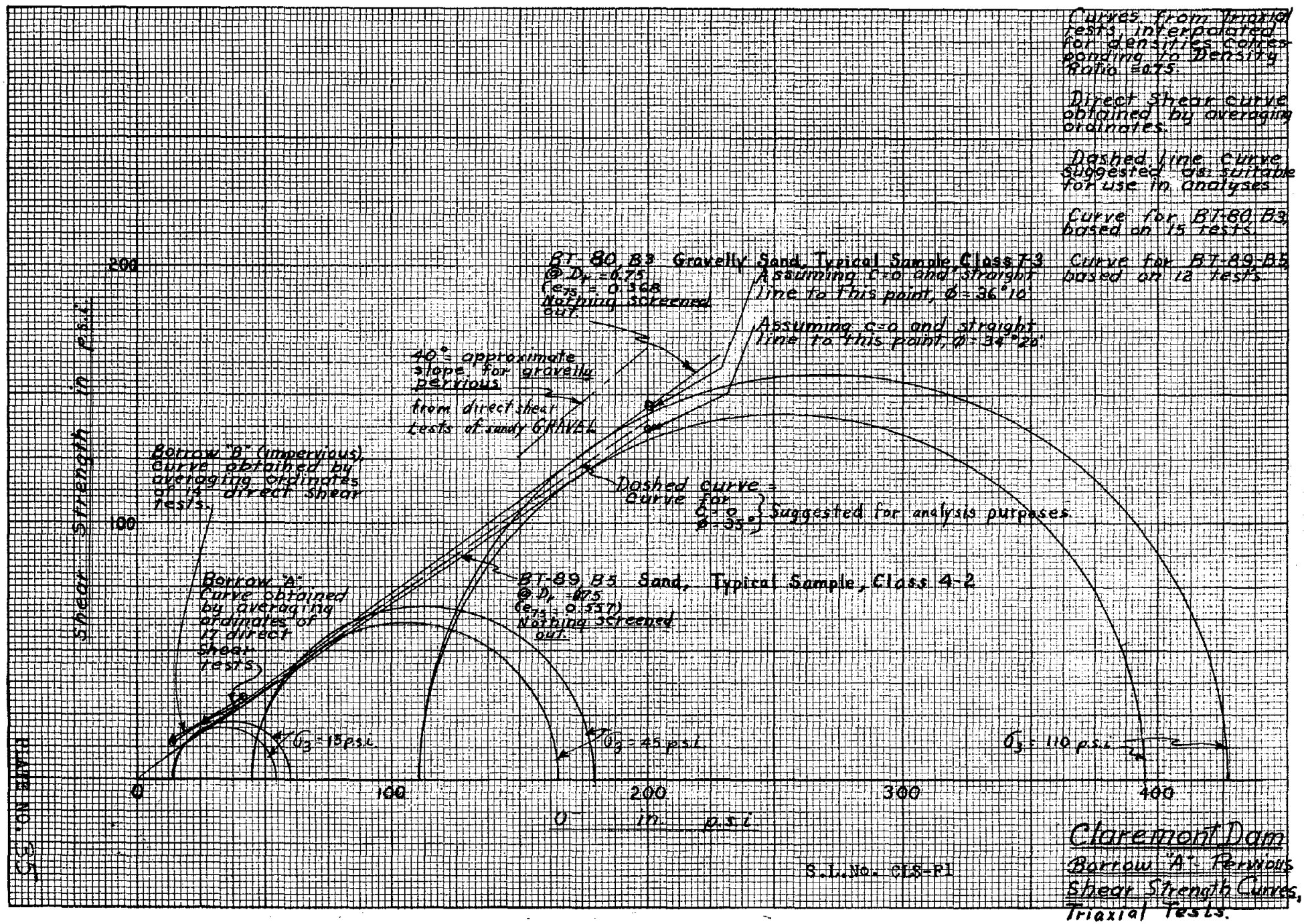
CL A. OF D

PLATE NO. 34

SOILS LABORATORY, ENGINEERING DIVISION

MECHANICAL ANALYSIS

PROVIDENCE, R.I.

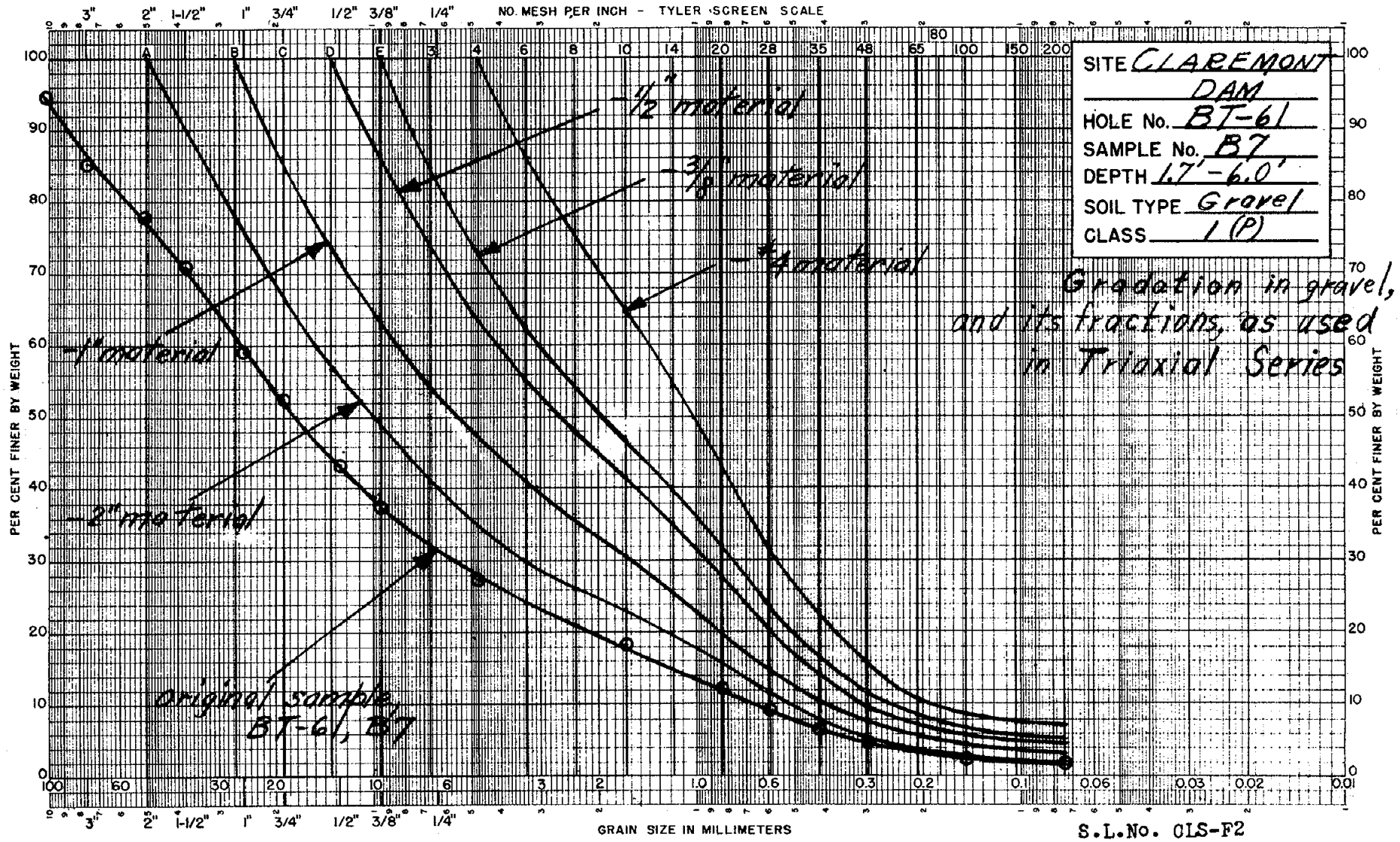


S.L.No. CLS-F1

Claremont Dam
Borrow "A" Pervious
Shear Strength Curves,
Triaxial Tests.

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY



S.L.No. CLS-F2

Gravel	Coarse Sand	Medium Sand	Fine Sand	Coarse Silt	Medium Silt
Class 1	Class 2		Class 6		
		Class 4		Class 8	
	Class 3	Class 5		Class 7	Class 9

SOILS LABORATORY, ENGINEERING DIVISION

MECHANICAL ANALYSIS

PROVIDENCE, R.I.

CL A. OF D.

PLATE NO. 36

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY

CLAREMONT DAM
SPECIAL TEST REPORT #1
TRIAXIAL TESTS - BT-61, B7

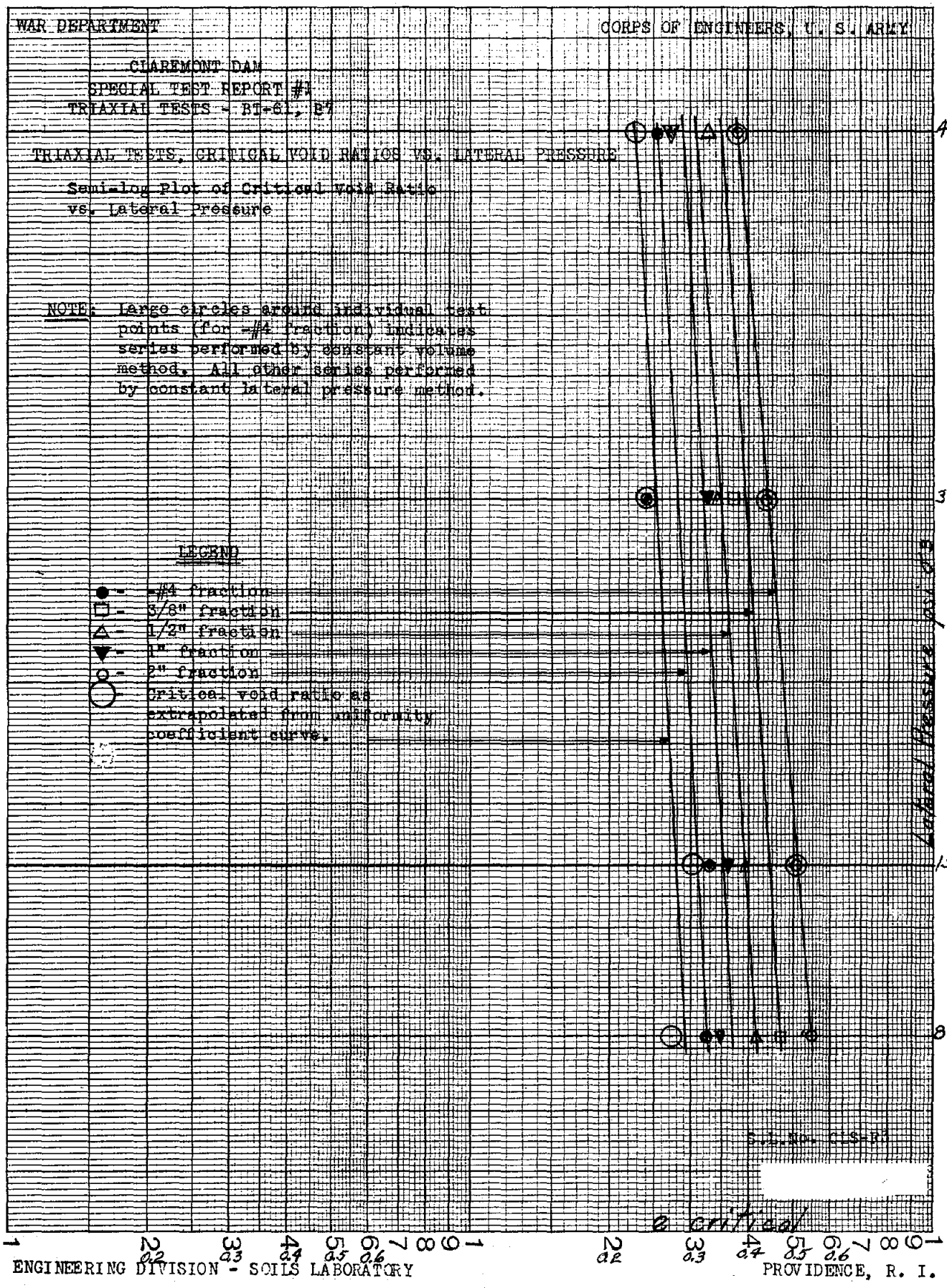
TRIAXIAL TESTS, CRITICAL VOID RATIOS VS. LATERAL PRESSURE

Semi-log Plot of Critical Void Ratio vs. Lateral Pressure

NOTE: Large circles around individual test points (for -#4 fraction) indicates series performed by constant volume method. All other series performed by constant lateral pressure method.

LEGEND

- ● - -#4 fraction
- □ - 3/8" fraction
- △ - 1/2" fraction
- ▼ - 1" fraction
- ○ - 2" fraction
- ◯ - Critical void ratio as extrapolated from uniformity coefficient curve.



S.L. No. C-S-R1

ENGINEERING DIVISION - SOILS LABORATORY

PROVIDENCE, R. I.

CL A.OF D.

PLATE NO. 37

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY

CHARMONT DAM

Special Test Report #1

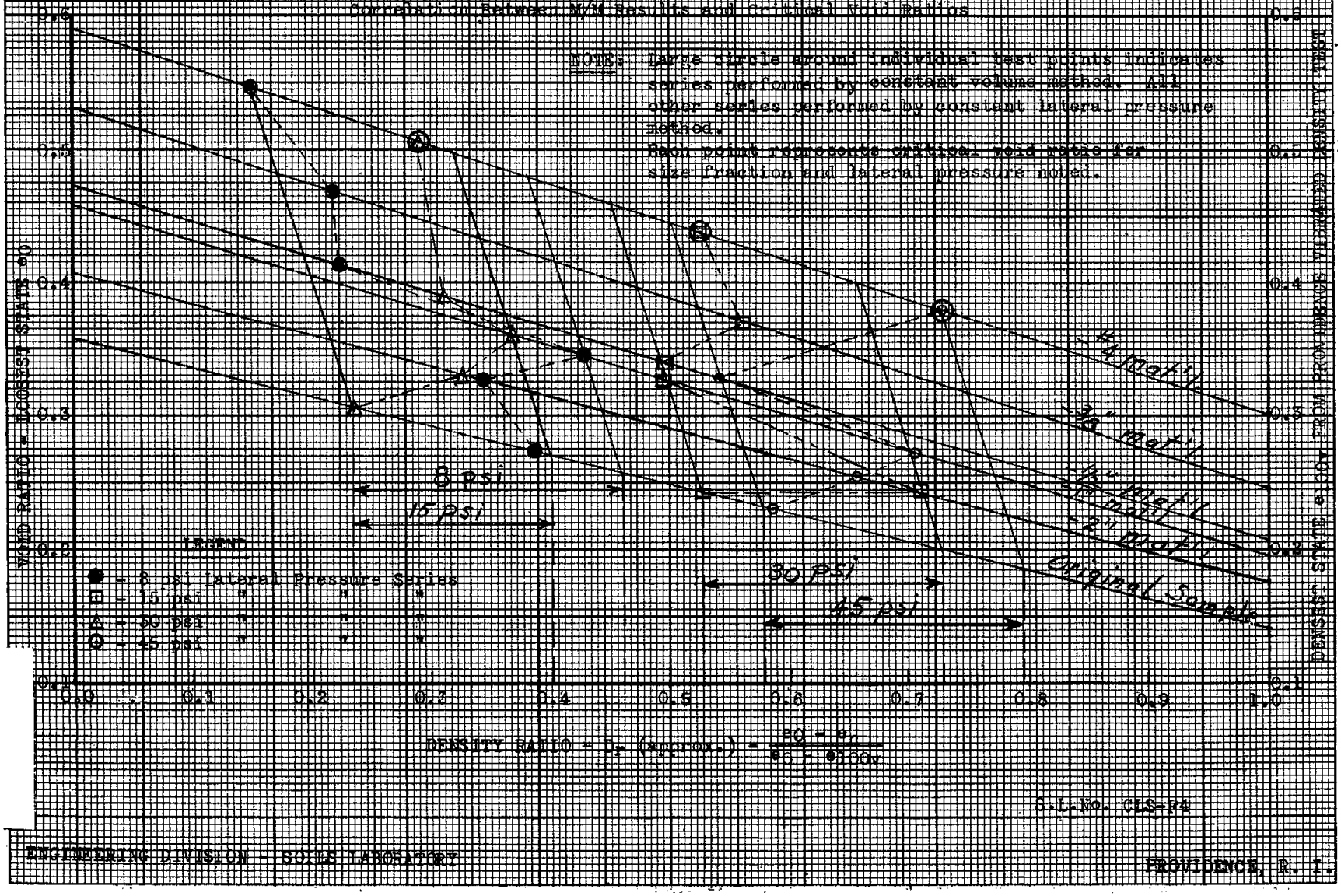
Triaxial Tests - B7-61, B7

TRIAXIAL TESTS, PLOT OF CRITICAL VOID RATIOS VS. COMPACTION DATA

Correlation Between M/M Results and Critical Void Ratios

NOTE: Large circle around individual test points indicates series performed by constant volume method. All other series performed by constant lateral pressure method.

Each point represents critical void ratio for size fraction and lateral pressure noted.



S.L. No. CLS-P4

ENGINEERING DIVISION - SOILS LABORATORY

PROVIDENCE, R. I.

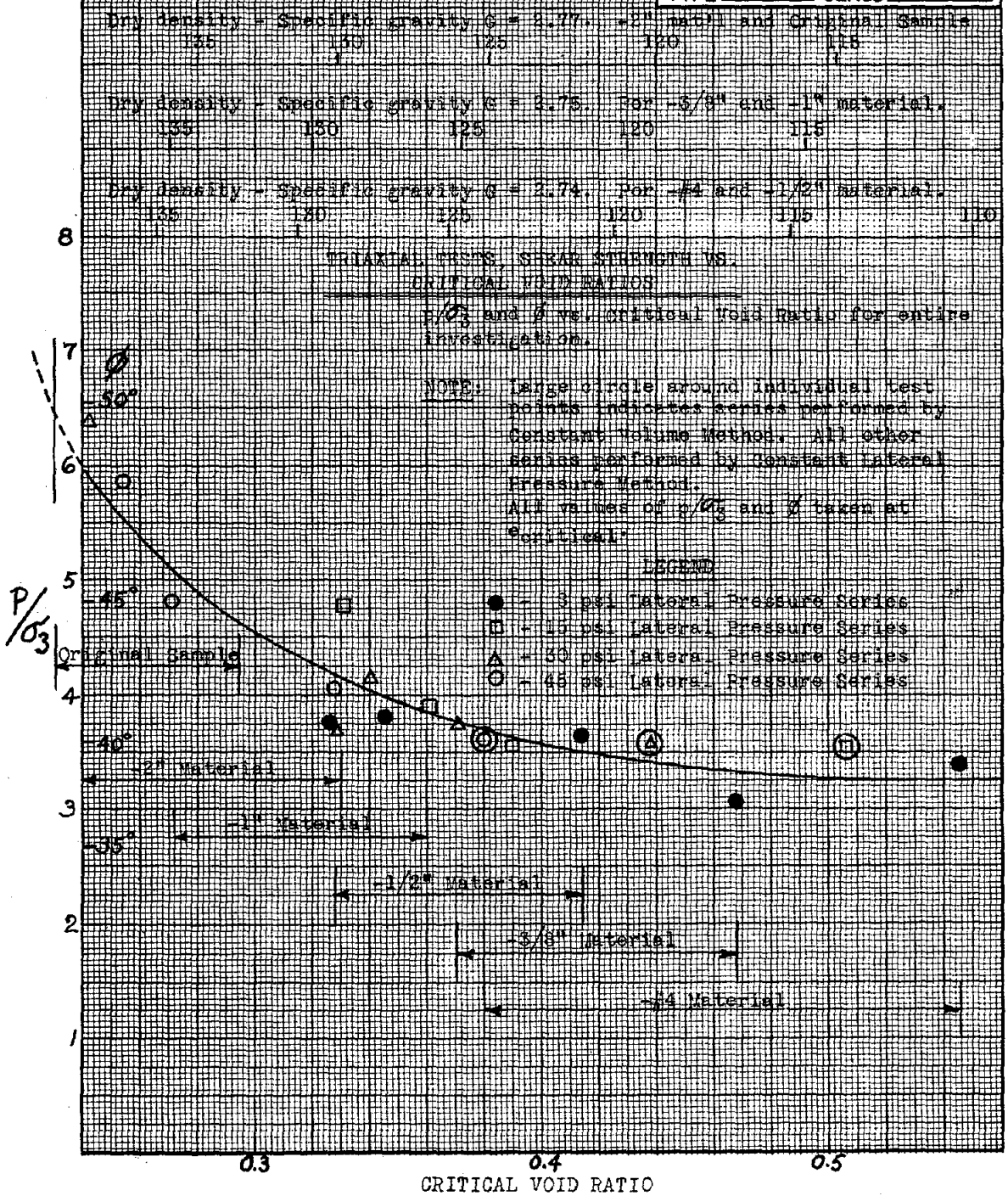
CL A. OF D.

PLATE NO. 38

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY

SUMMARY SHEET NO.	SITE CLAREMONT DAM
INITIAL $\sigma 3$ =	
METHOD - CONSTANT	EXPL. NO. BT-61
TYPE OF DRAINAGE	SAMPLE NO. B7
RATE OF STRAIN	DEPTH 1.7 - 6.0
	TYPE Gravel CLASS 1 (P)



TRIAXIAL COMPRESSION TEST

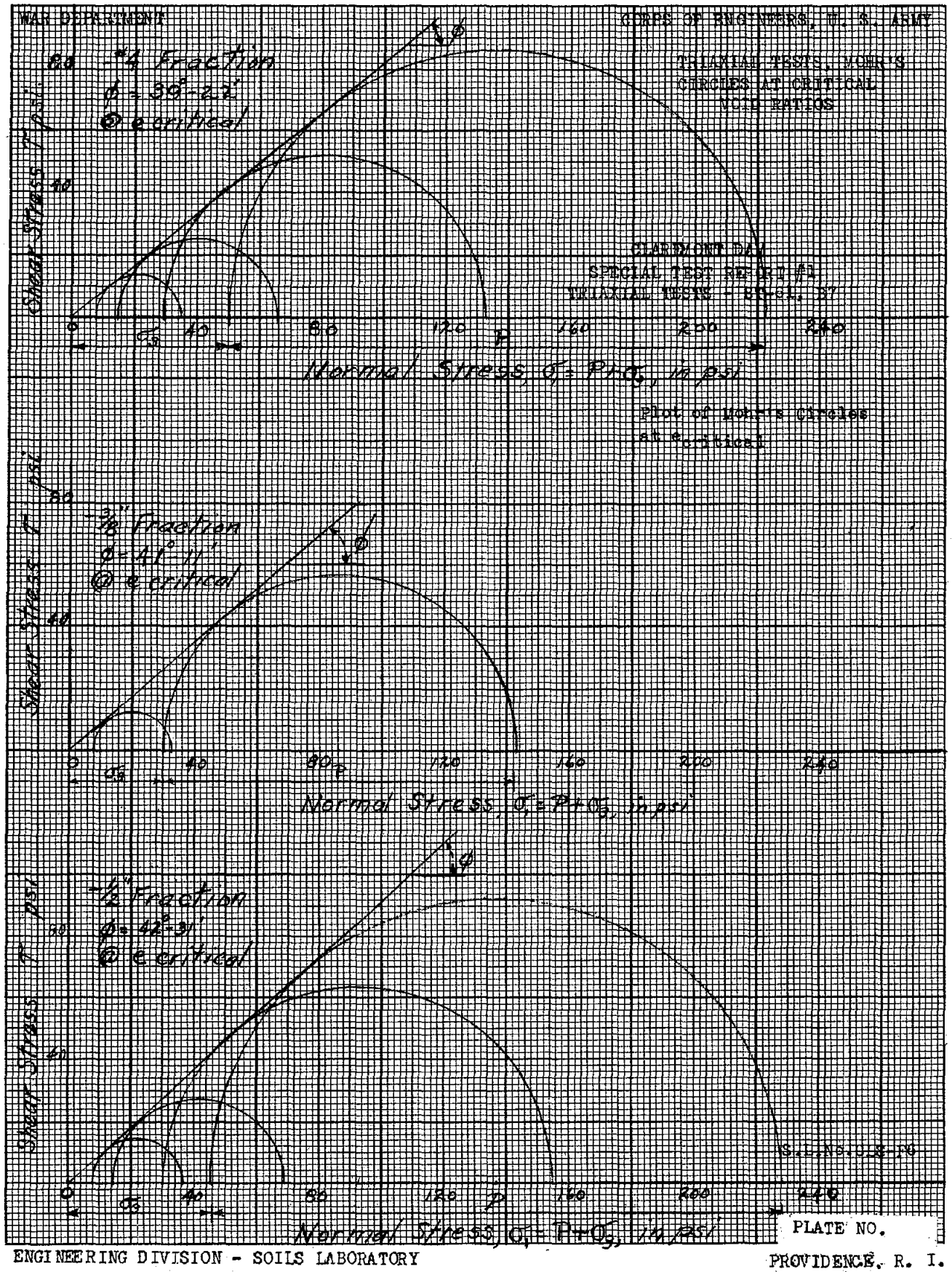
S.L.No. CLS -F5

ENGINEERING DIVISION - SOILS LABORATORY

PROVIDENCE, R.I.

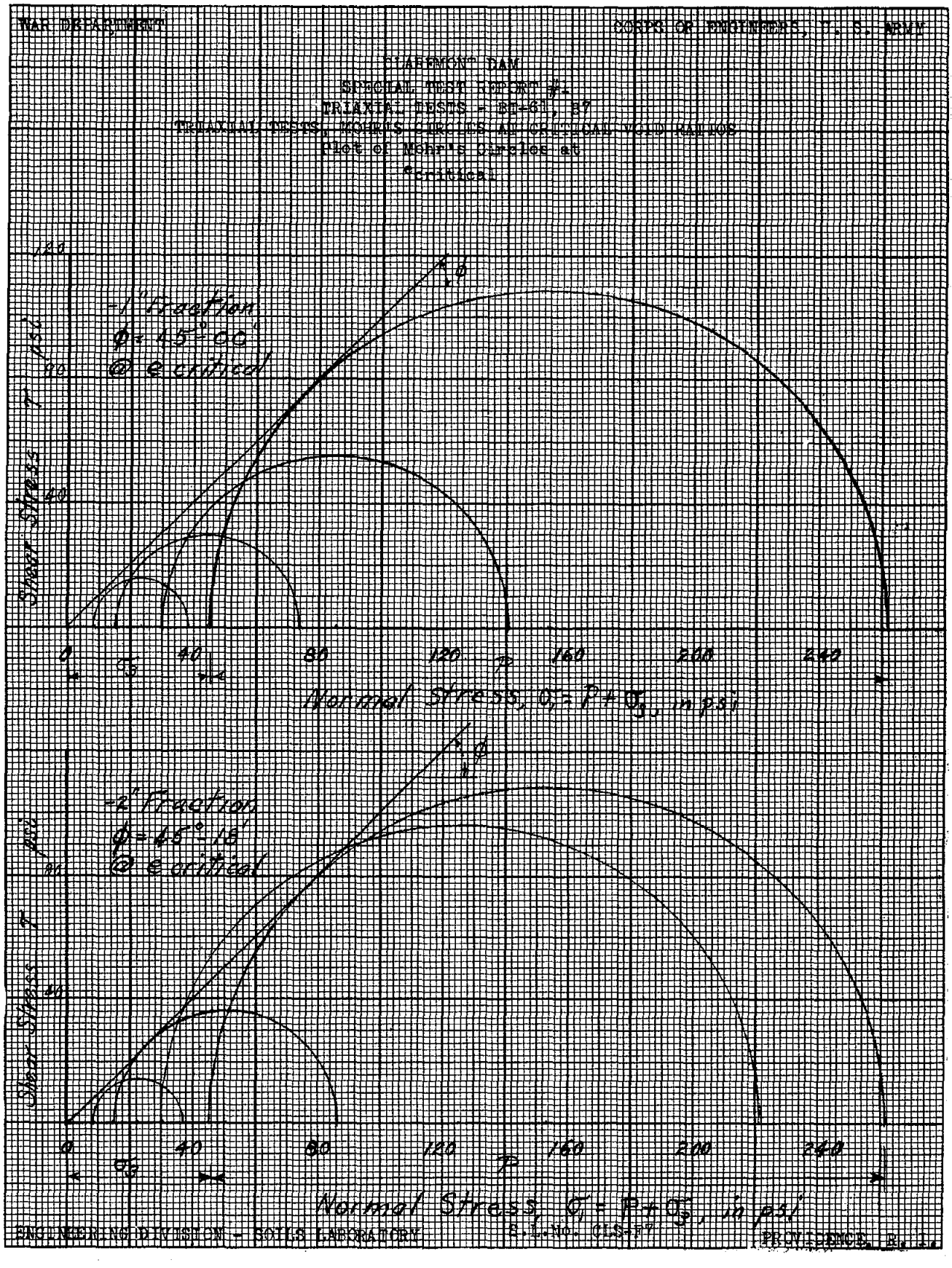
GL A. OF D.

PLATE NO. 39



ENGINEERING DIVISION - SOILS LABORATORY

PROVIDENCE, R. I.

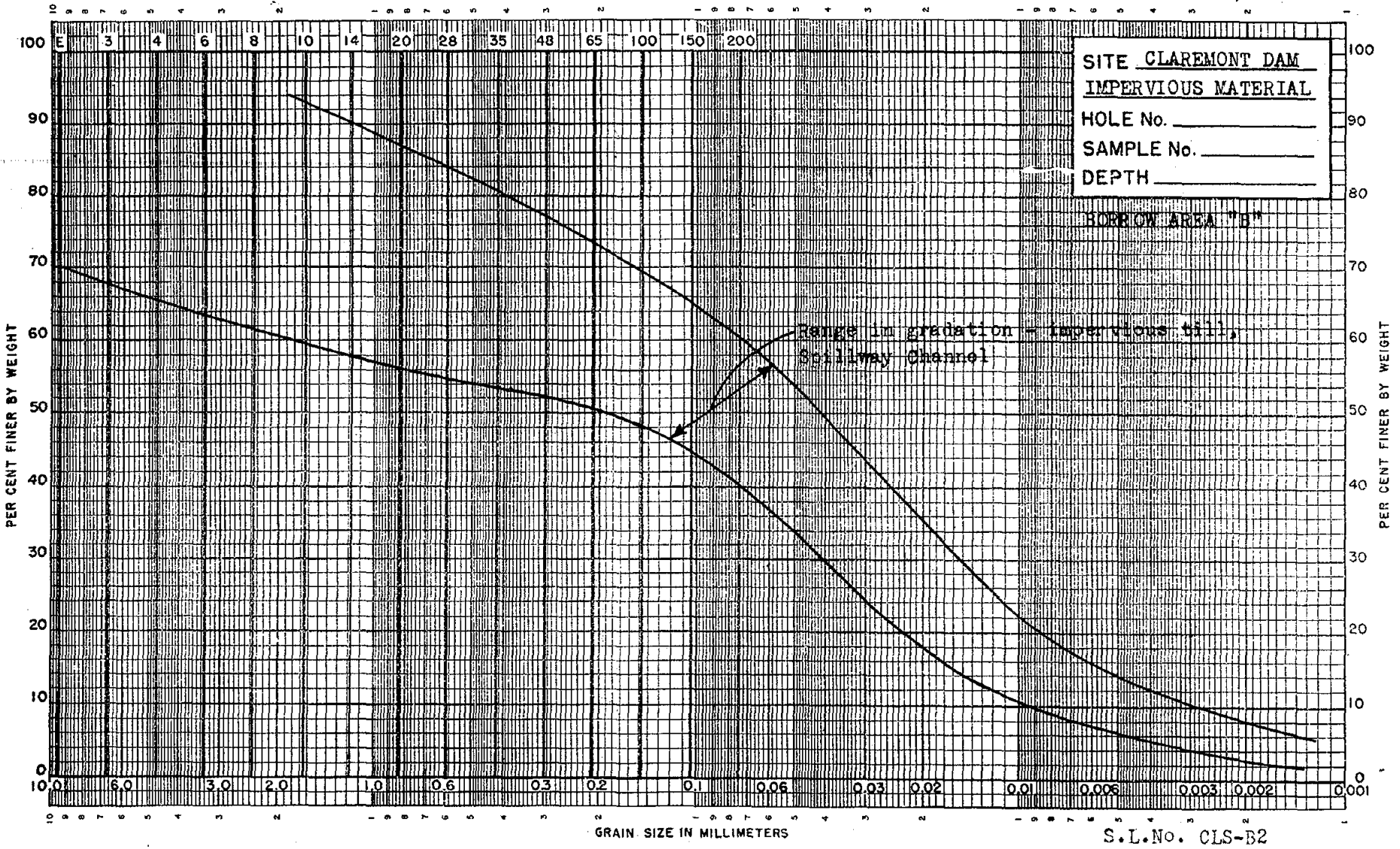


CL A. OF D.

PLATE NO. 41

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY



S.L.No. CLS-B2

Gravel	Coarse Sand	Medium Sand	Fine Sand	Coarse Silt	Medium Silt	Fine Silt or	Coarse Clay
Class 1	Class 2		Class 6		Class 10 or 10C		Class 12 or 12C
		Class 4		Class 8			
	Class 3		Class 5	Class 7	Class 9	Class 11	Class 13 or 13C

SOILS LABORATORY

MECHANICAL ANALYSIS

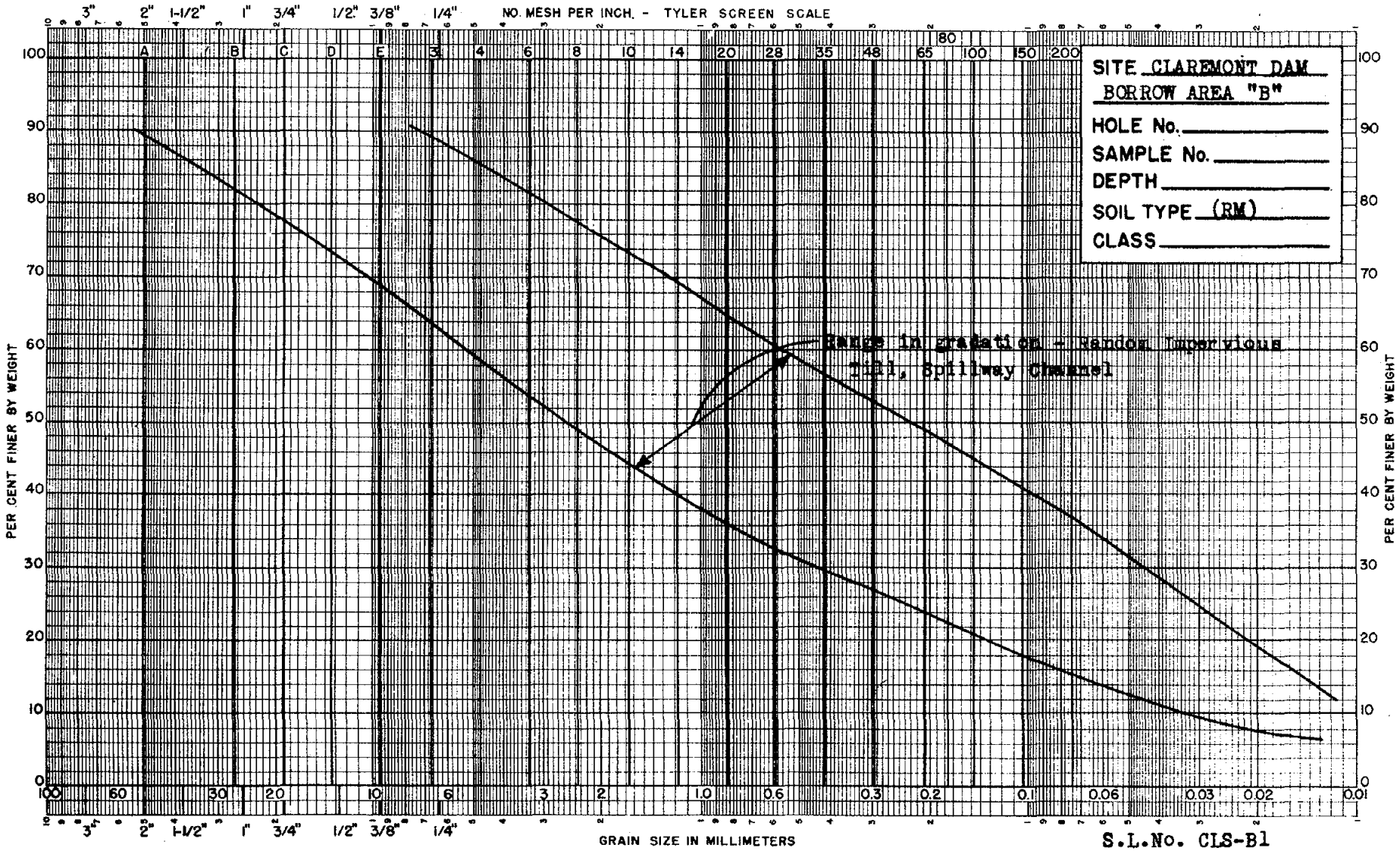
PROVIDENCE, R.I.

CL A. OF D.

PLATE NO. 42

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY



SOILS LABORATORY, ENGINEERING DIVISION

MECHANICAL ANALYSIS

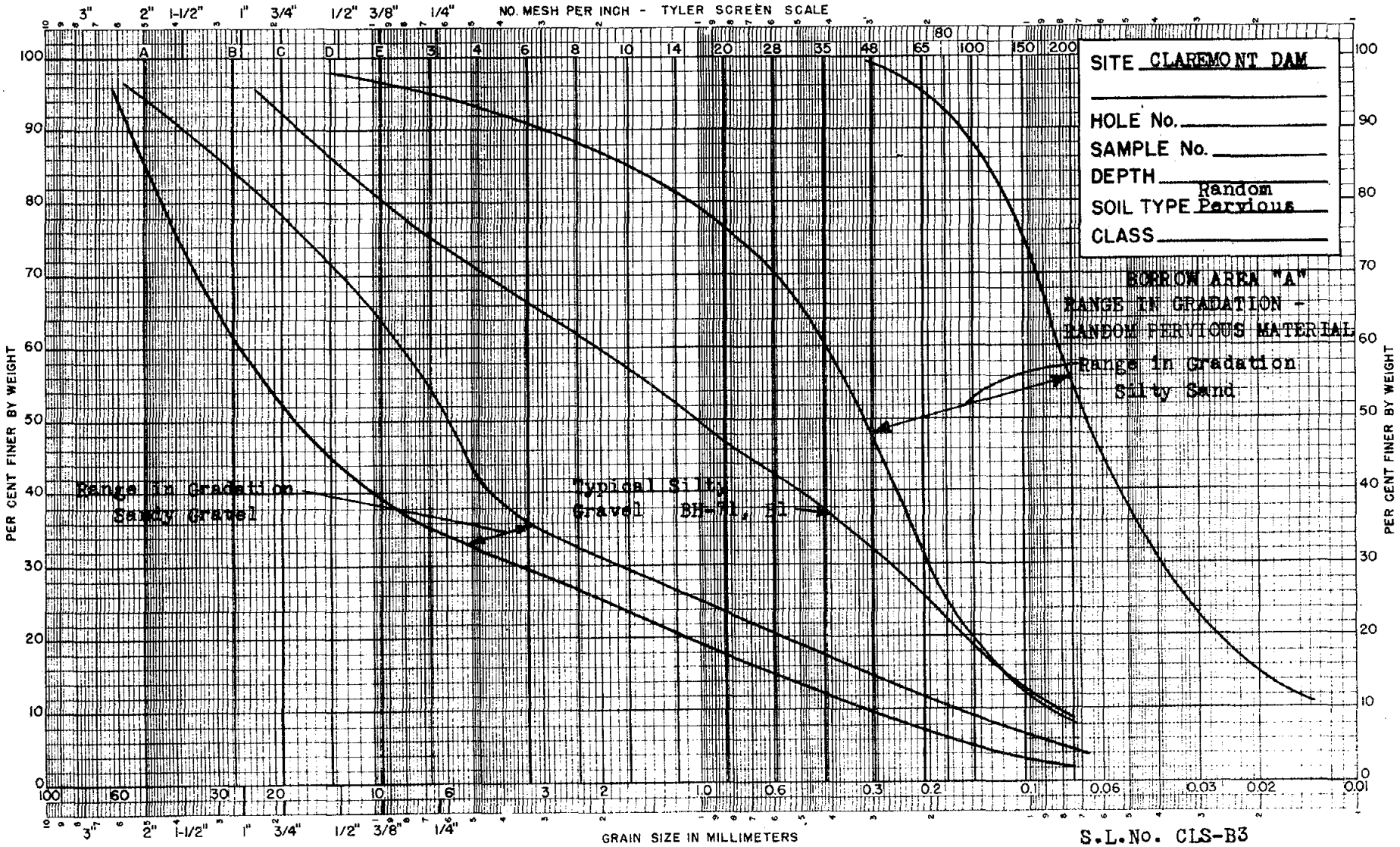
PROVIDENCE, R.I.

CL A.OF D..

PLATE NO. 43

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY



S.L.No. CLS-B3

Gravel	Coarse Sand	Medium Sand	Fine Sand	Coarse Silt	Medium Silt
Class 1	Class 2		Class 6		
		Class 4		Class 8	
	Class 3		Class 5	Class 7	Class 9

SOILS LABORATORY, ENGINEERING DIVISION

MECHANICAL ANALYSIS

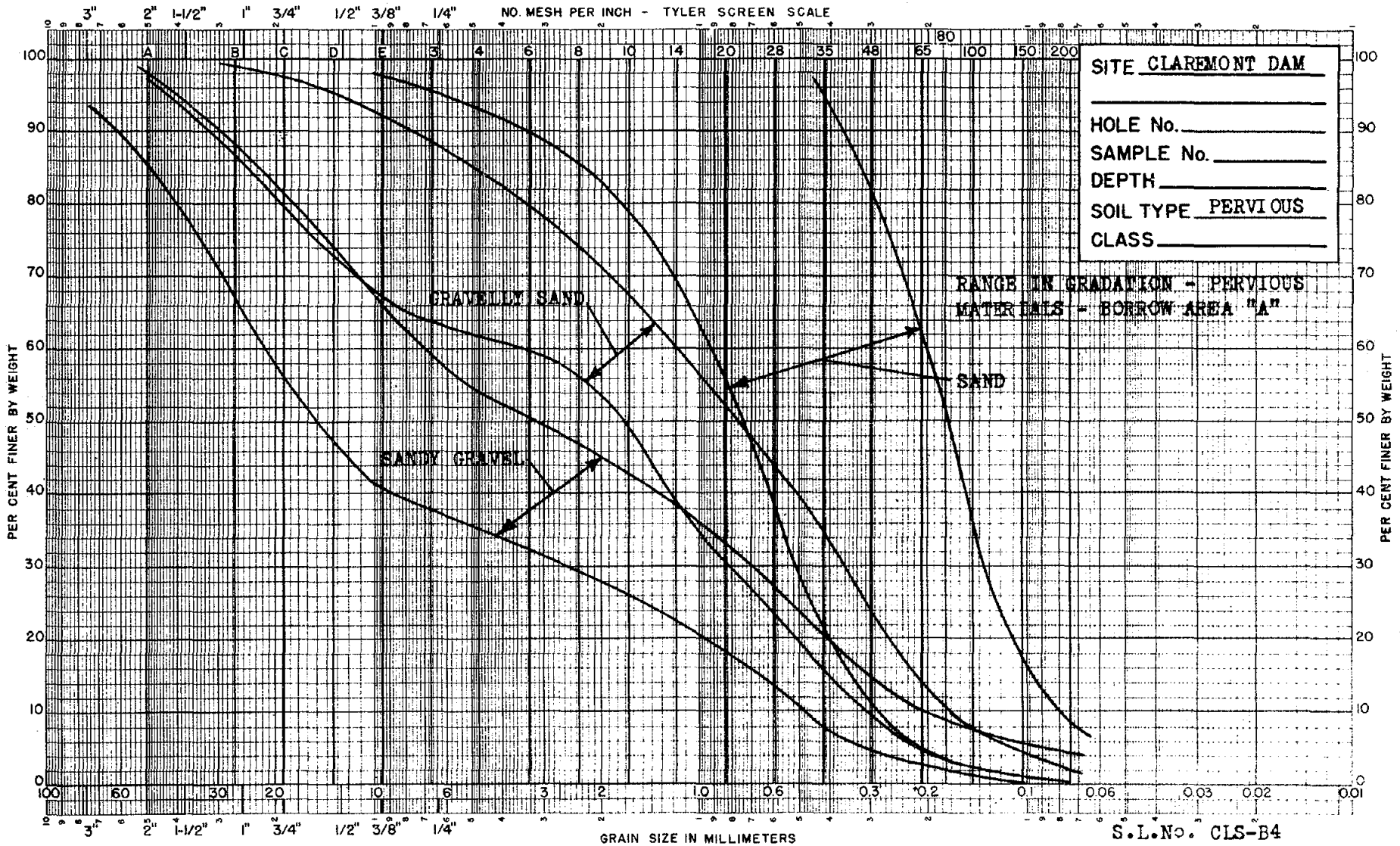
PROVIDENCE, R.I.

CL A. OF D.

PLATE NO. 44

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY



S.L.No. CLS-B4

Gravel	Coarse Sand	Medium Sand	Fine Sand	Coarse Silt	Medium Silt
Class 1	Class 2		Class 6		
		Class 4		Class 8	
	Class 3	Class 5		Class 7	Class 9

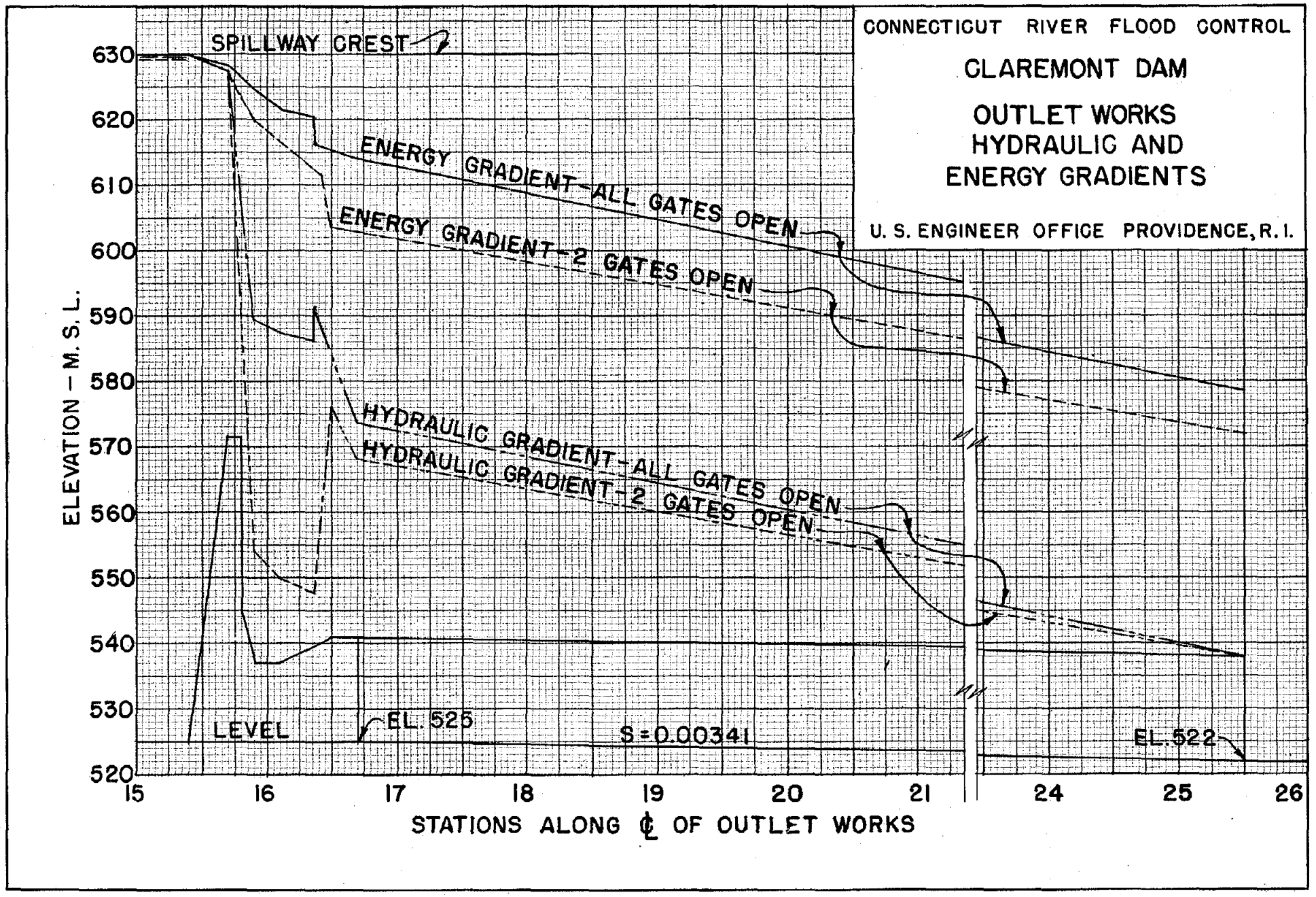
SOILS LABORATORY, ENGINEERING DIVISION

MECHANICAL ANALYSIS

PROVIDENCE, R. I.

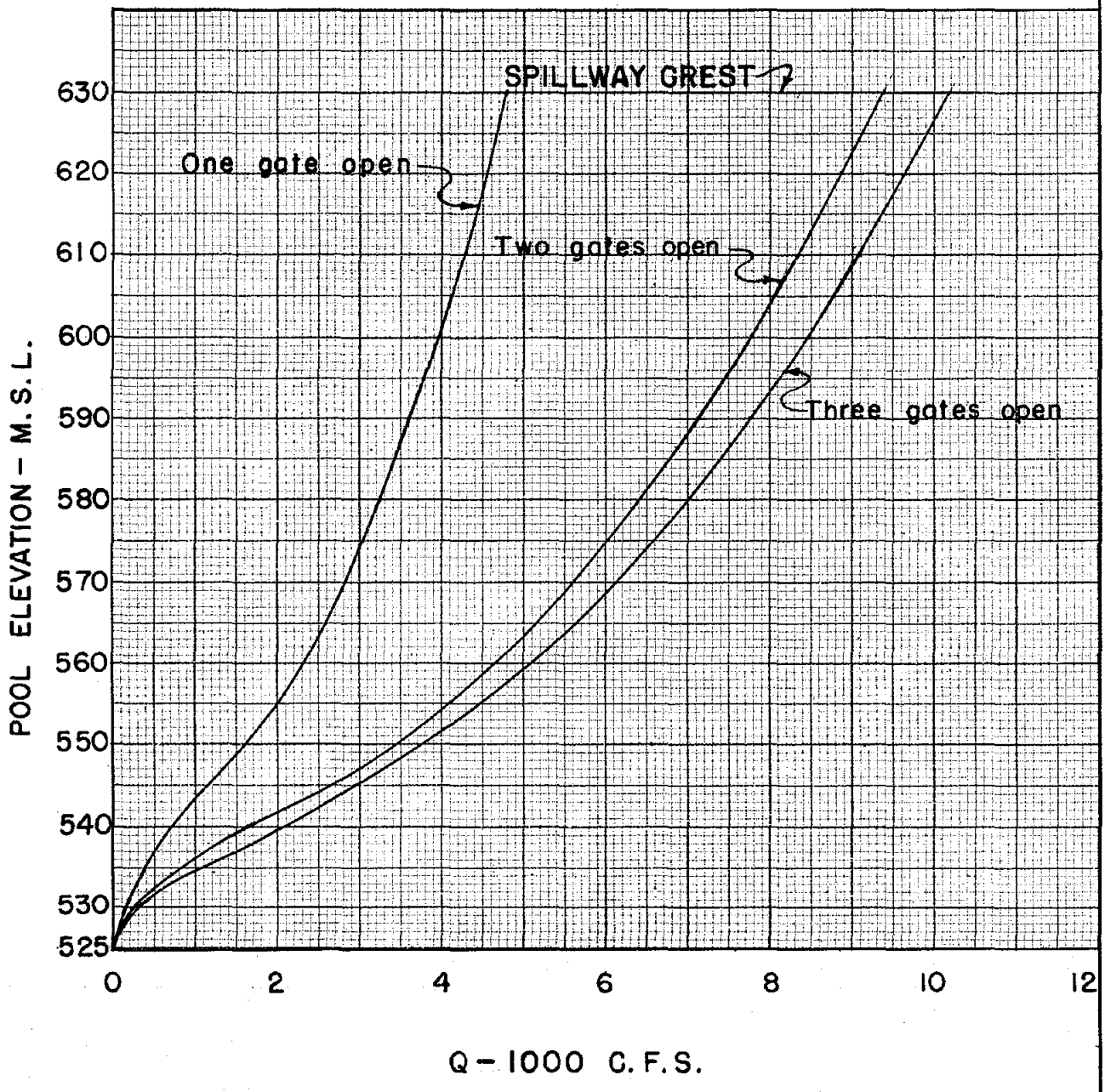
CL A. OF D.

PLATE NO. 45



CL A. OF D.

PLATE NO. 46

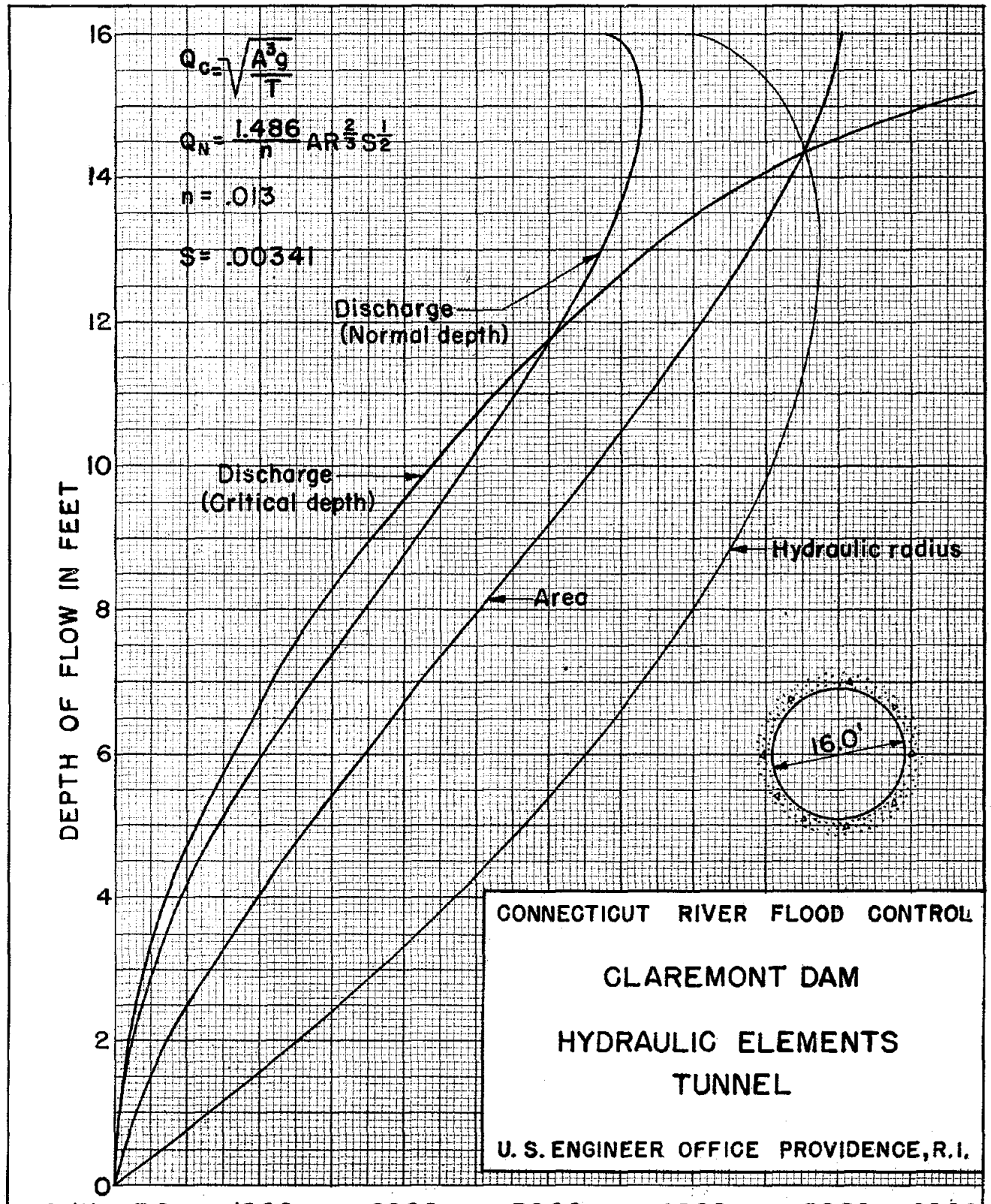


CONNECTICUT RIVER FLOOD CONTROL

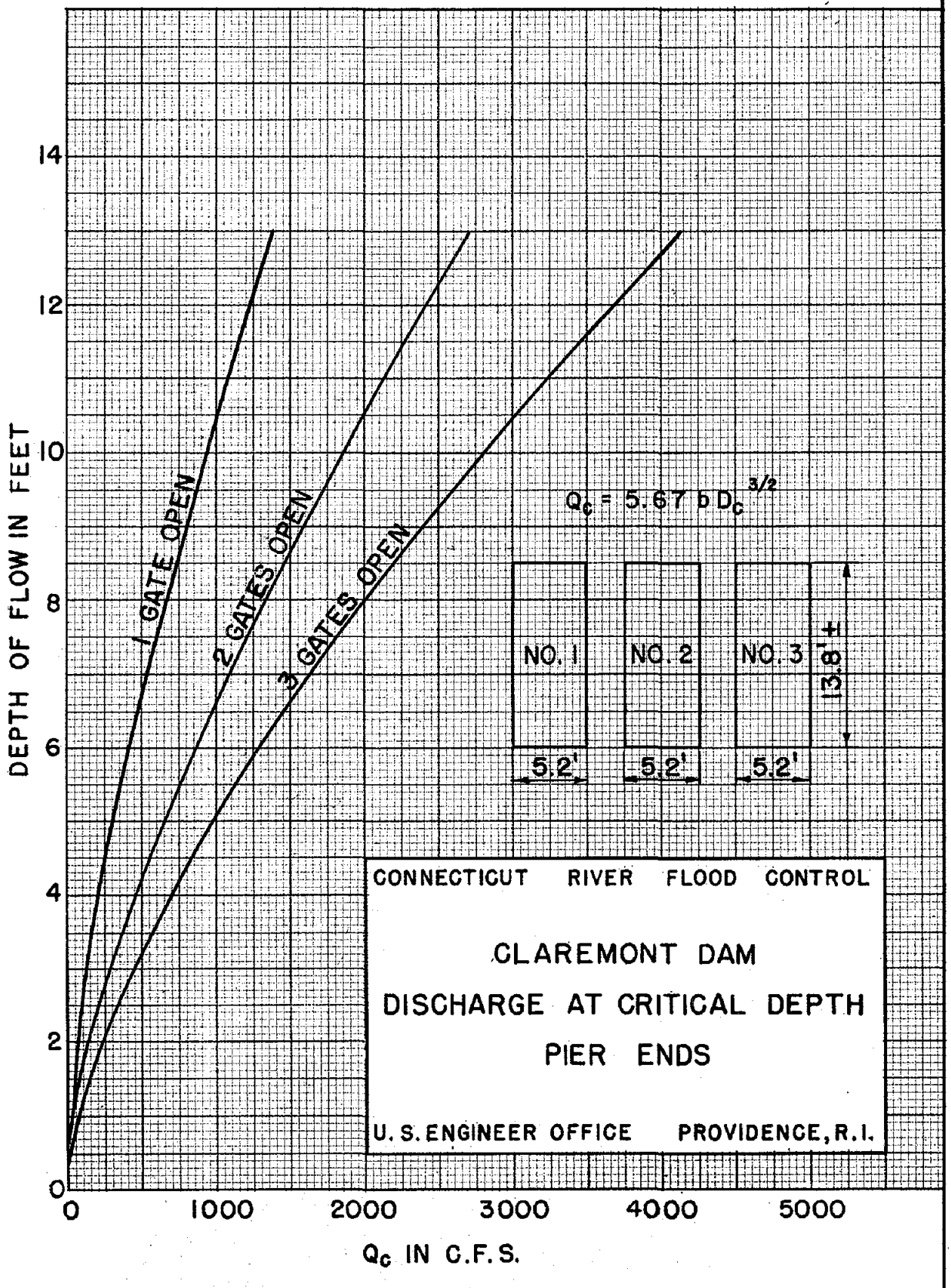
CLAREMONT DAM

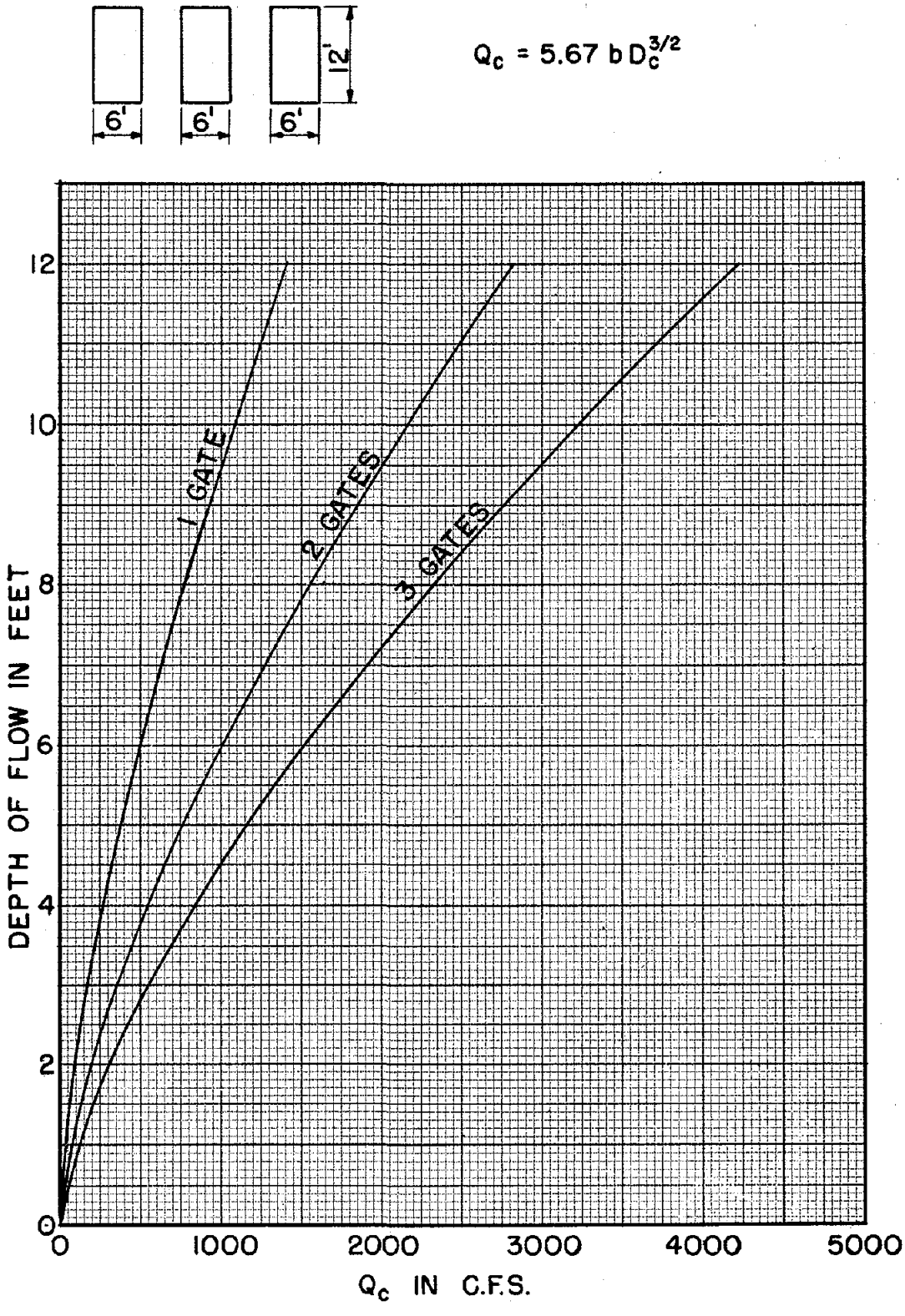
OUTLET RATING CURVES

U.S. ENGINEER OFFICE PROVIDENCE, R.I.



Q IN C.F.S.	1000	2000	3000	4000	5000	6000
R IN FT.	1	2	3	4	5	
A IN SQ. FT.	40	80	120	160	200	


DEPTH OF FLOW IN FEET
0
2
4
6
8
10
12
14
0
1000
2000
3000
4000
5000
Q_c IN C.F.S.
1 GATE OPEN
2 GATES OPEN
3 GATES OPEN
$Q_c = 5.67\ b\ D_c^{3/2}$
NO. 1
NO. 2
NO. 3
5.2'
5.2'
5.2'
13.8'+
CONNECTICUT RIVER FLOOD CONTROL
CLAREMONT DAM
DISCHARGE AT CRITICAL DEPTH
PIER ENDS
U. S. ENGINEER OFFICE PROVIDENCE, R.I.



CONNECTICUT RIVER FLOOD CONTROL

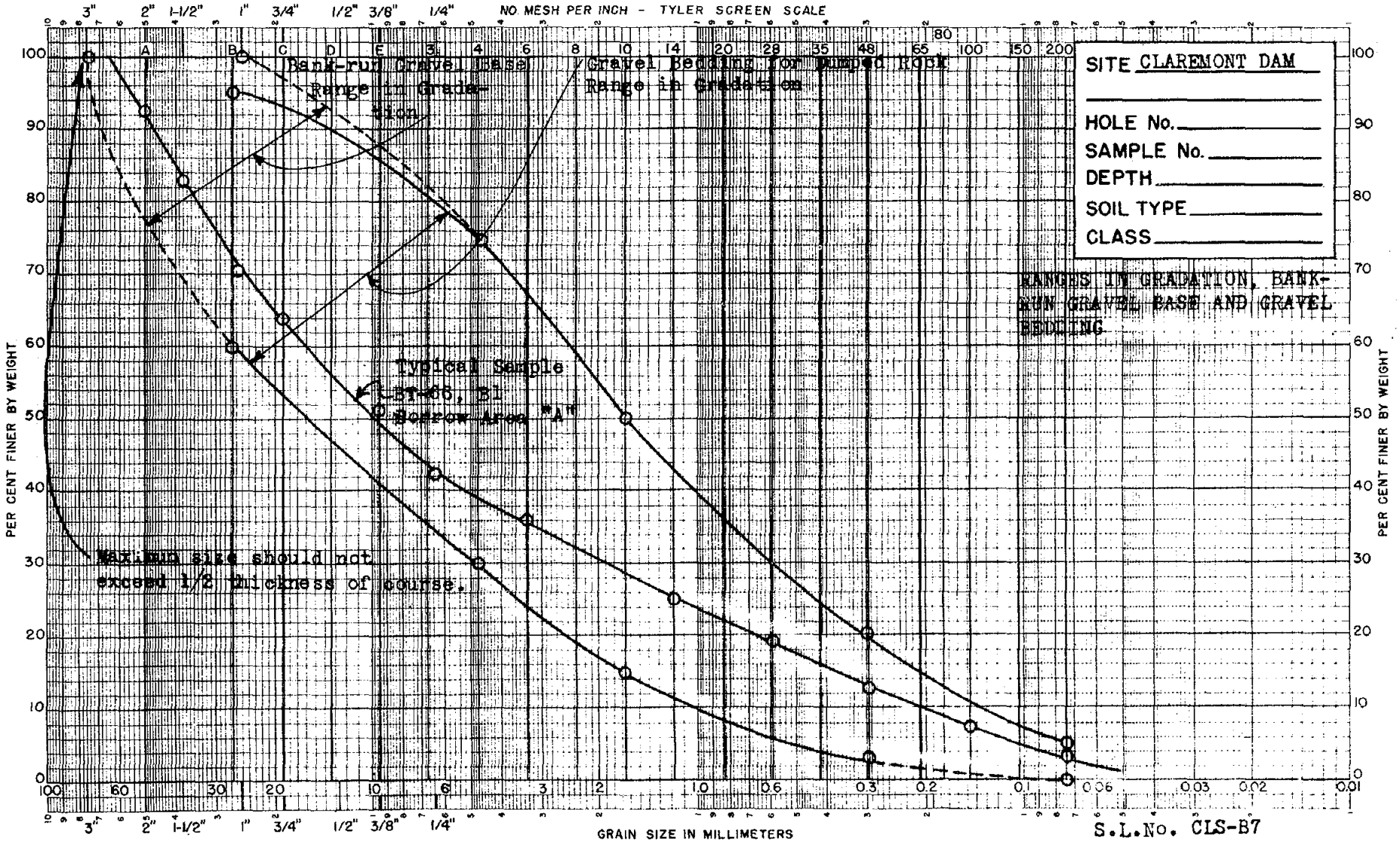
CLAREMONT DAM

DISCHARGE AT CRITICAL DEPTH
GATE SECTION

U. S. ENGINEER OFFICE PROVIDENCE, R. I.

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY



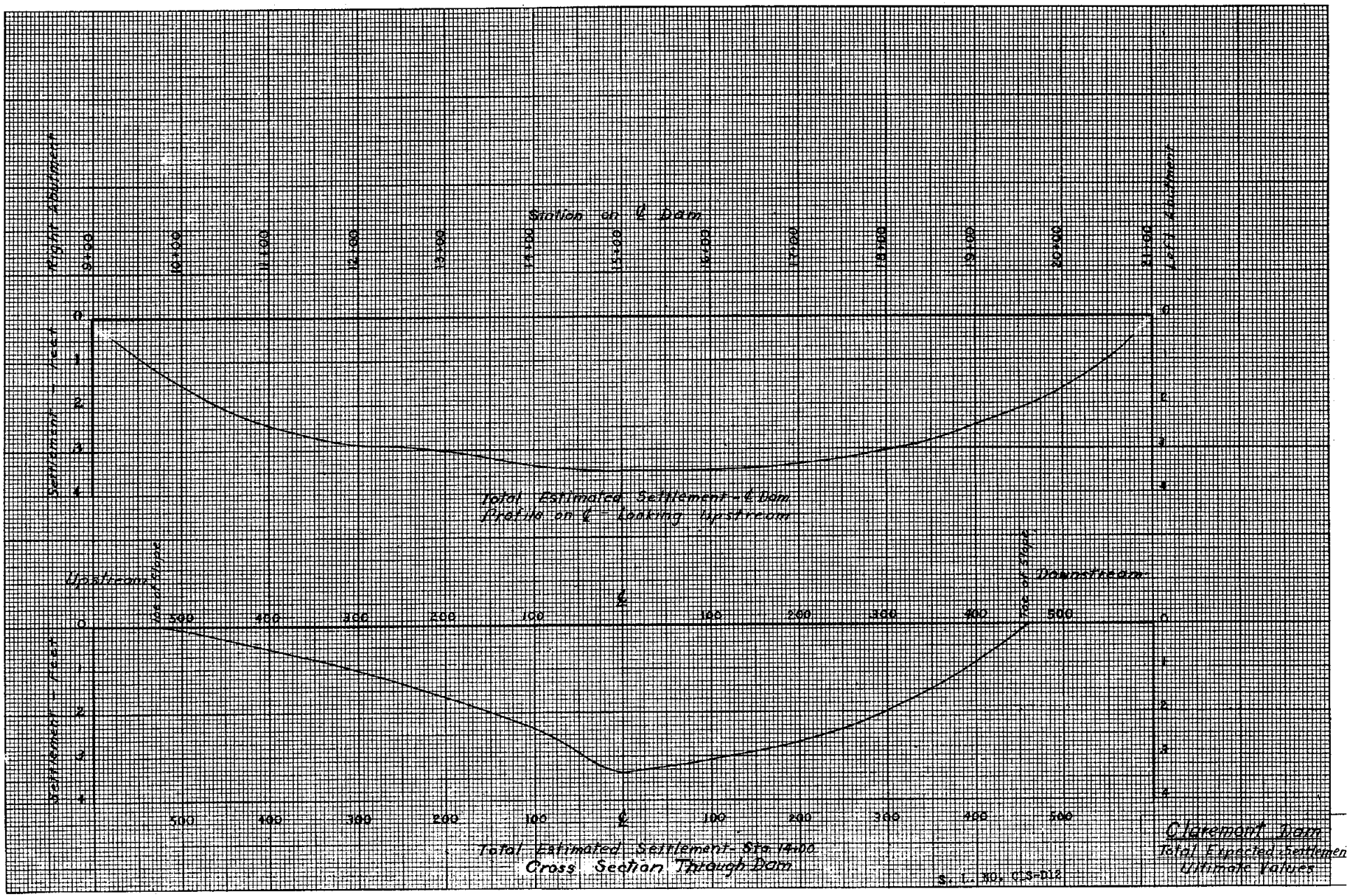
SOILS LABORATORY, ENGINEERING DIVISION

MECHANICAL ANALYSIS

PROVIDENCE, R. I.

CL A. OF D.

PLATE NO. 51



CL A. OF D.

PLATE NO. 52

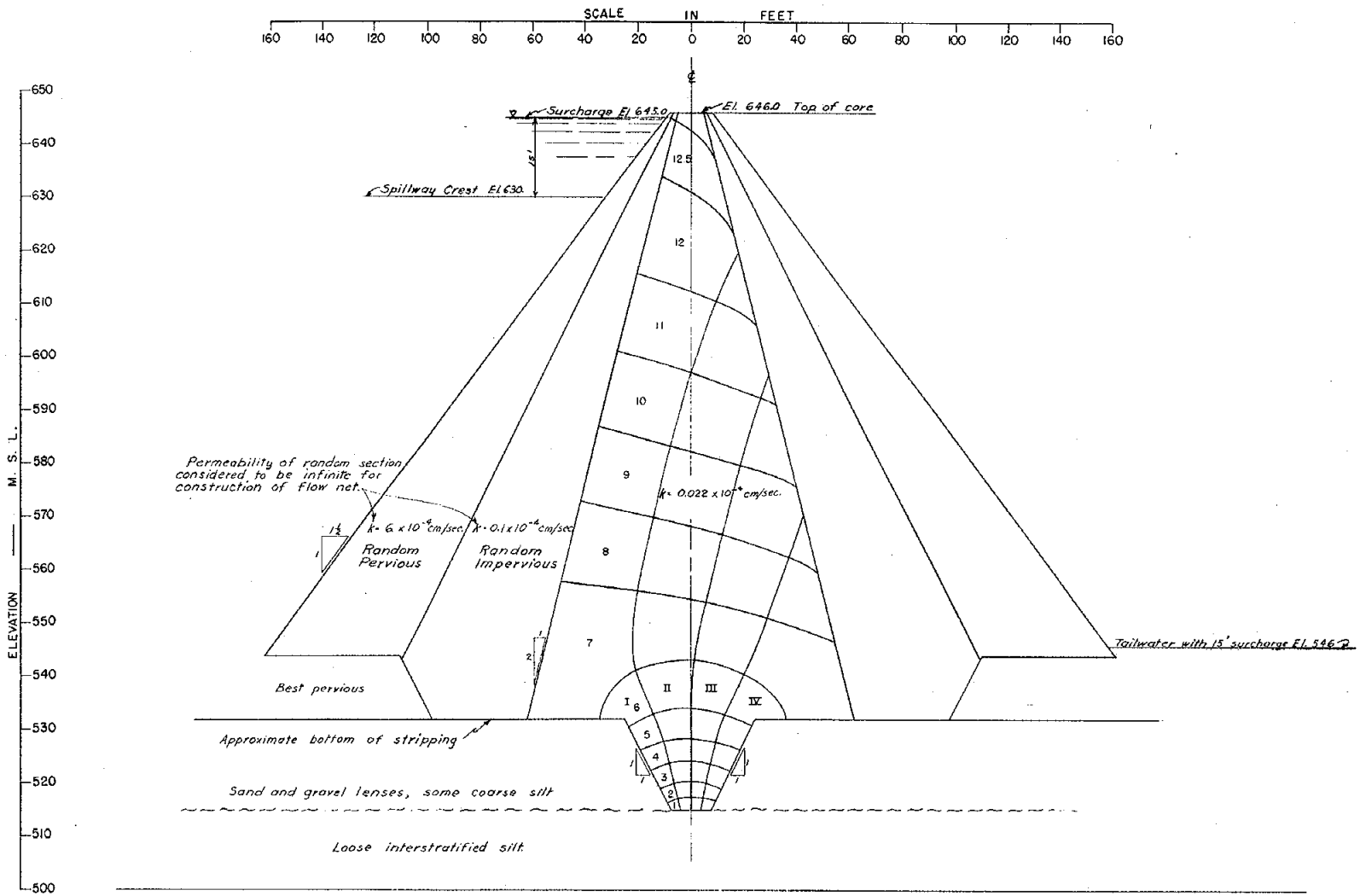
CLAREMONT DAM
Total Settlement at Downstream Toe Drain

2 October 1944

S.L. No. CLS-D5d

CL A. OF D.

PLATE NO. 53



SECTION AT STA. 14+00

$$Q = \bar{k}h\,\frac{n_F}{n_E} = 100\,\frac{0.022 \times 10^{-4}}{30.5}\,59\,\frac{12.5}{4} = 22.3 \times 10^{-4}\ \text{cfs/sta.}$$

$$\frac{k_h}{\bar{k}} = \sqrt{k_h k_v}$$

Transformation factor $= \sqrt{\frac{k_v}{k_h}} = \sqrt{\frac{1}{4}} = \frac{1}{2}$

(Used to reduce all horizontal dimensions)

CLAREMONT DAM
FLOW NET STUDY
STATION 14+00

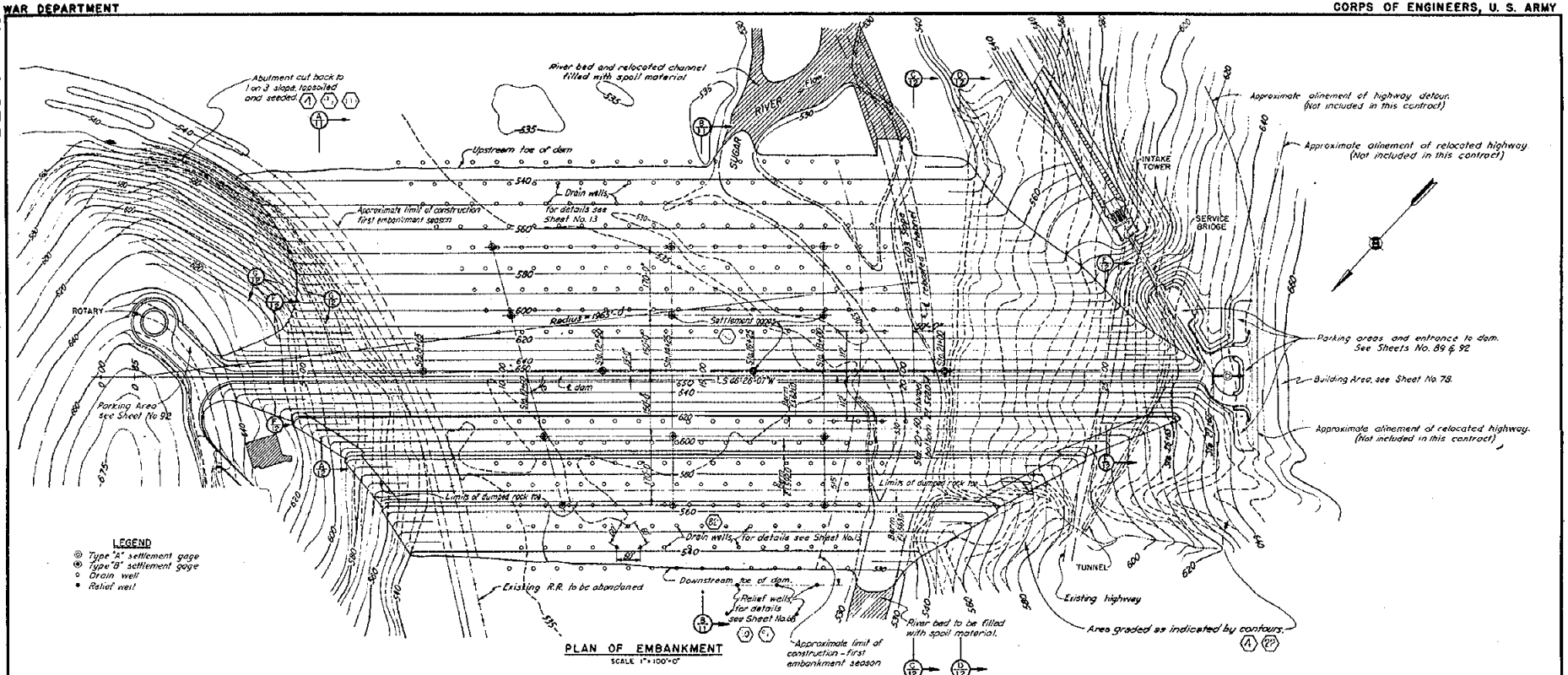
October 1944 SL-NO. CLS-E1

CL A. OF D.

PLATE NO. 54

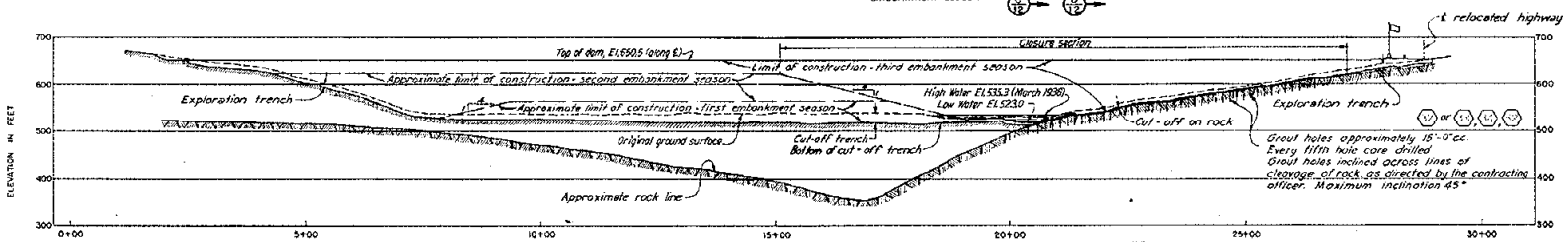
WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY



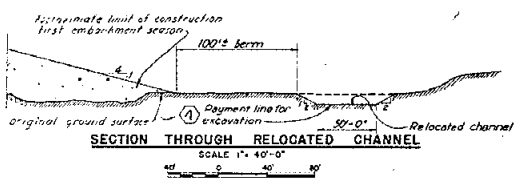
PLAN OF EMBANKMENT

SCALE 1" = 100'-0"



SECTION ON CENTER LINE OF DAM

SCALE 1" = 100'-0"



SECTION THROUGH RELOCATED CHANNEL

SCALE 1" = 40'-0"

NOTES

For downstream slope drainage system, see Sheets No. 13 and 14.

Elevations refer to Mean Sea Level Datum.

Contour interval five feet.

The cut-off trench, shall be excavated to satisfactory impervious material as directed by the contracting officer.

The longitudinal limits of the cut-off trench, exploration trench, and cut-off on rock will be determined in the field.

Figures in hexagons indicate item numbers under which payment will be made.

For details of settlement gages see Sheet No. 68.

The number and location of drain wells is subject to change.

KEY	DATE	REVISION (Indicated by △)	REV BY	CK BY	AP BY

CONNECTICUT RIVER FLOOD CONTROL

CLAREMONT DAM

EMBANKMENT DETAILS NO. 1

SUGAR RIVER — NEW HAMPSHIRE

IN 94 SHEETS — SCALE: 1 IN. = 100 FT. (0 100 200) — SHEET NO. 10

U. S. ENGINEER OFFICE, PROVIDENCE, R.I., JAN. 1945

SUBMITTED: G. P. Hussain, SENIOR ENGINEER, PROJECT ENGINEER

APPROVAL RECOMMENDED: J. E. Malone, HEAD ENGINEER, CHIEF ENGINEERING DIV.

APPROVED: Thos. F. Lanno, COL., CORPS OF ENGINEERS, DISTRICT ENGINEER

PREPARED: H. W. Hicks, PROJECT UNIT NO. 2

Drawn: C. M. V.; TRACED: G. W. Y., M. F.; CHECKED: D.P.

FILE NO. CT-1-1780

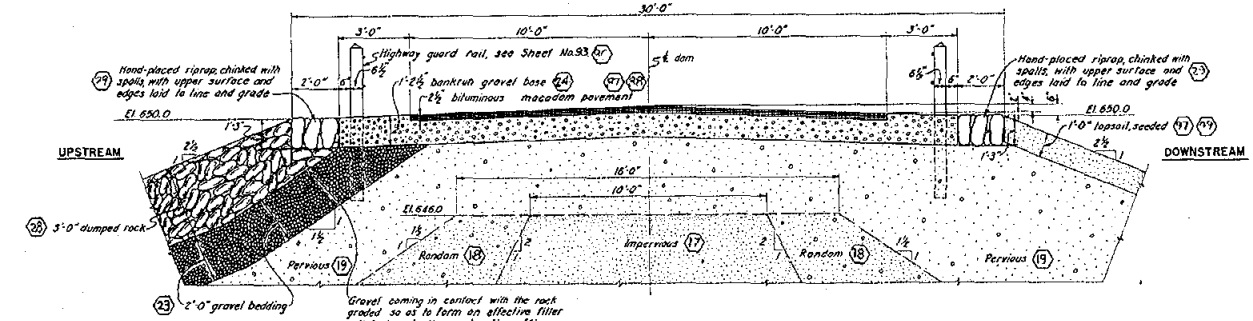
CL A. OF D.

PLATE NO. 55

WAR DEPARTMENT

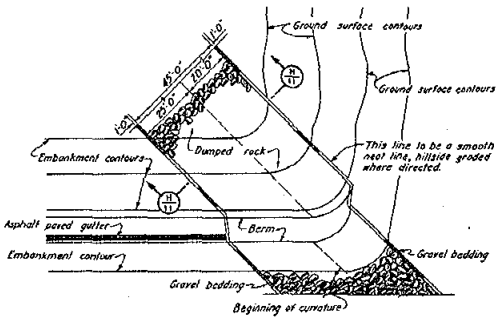
CORPS OF ENGINEERS, U.S. ARMY

CL A. OF D.



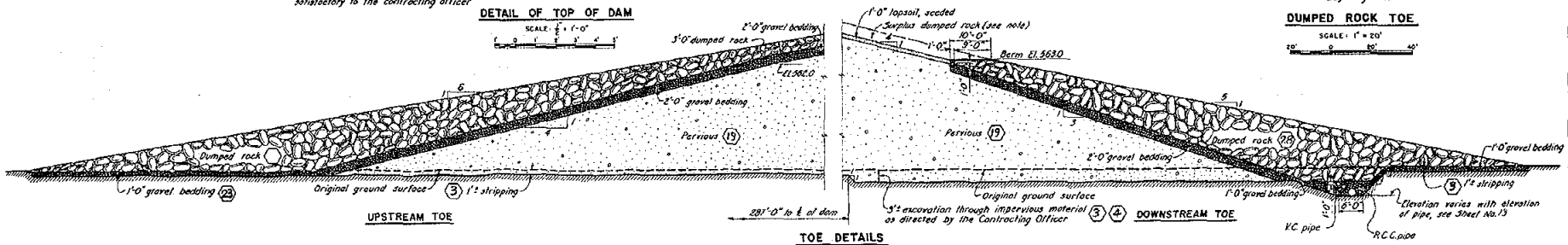
DETAIL OF TOP OF DAM

SCALE: ½" = 1'-0"



DUMPED ROCK TOE

SCALE: 1" = 20'

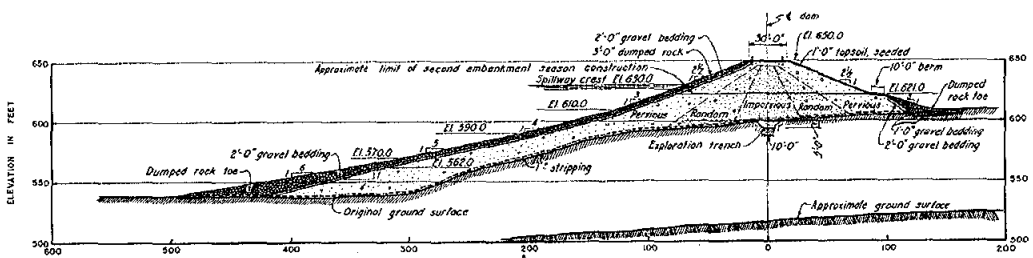


UPSTREAM TOE

DOWNSTREAM TOE

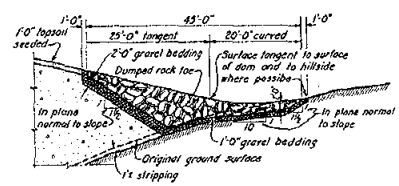
TOE DETAILS

SCALE: 1" = 10'



SECTION A/10 STA. 5+45

SCALE: 1" = 40'



SECTION H/11

SCALE: 1" = 10'

NOTES

Elevations refer to Mean Sea Level Datum

The cut-off trench shall be extended downward to satisfactory impervious material as directed by the Contracting Officer.

If available, surplus rock from excavations and borrow pits shall be dumped on the downstream surface of the dam between El. 563.0 and El. 598.0 in horizontal layers.

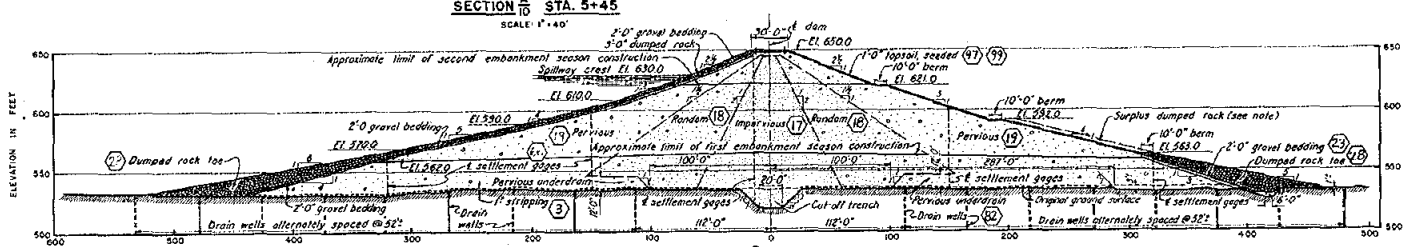
Figures in hexagons indicate item numbers under which payment will be made.

For location of sections see Sheet No. 10.

For downstream slope drainage system see Sheets No.13 and 14.

For location of settlement gages and drain wells see Sheet No. 10

Drain wells will be paid for under Item No. 66.



SECTION B/10 STA. 15+00

SCALE: 1" = 40'

KEY	DATE	REVISION (Indicated by △)	REV BY	CK BY	AP BY

CONNECTICUT RIVER FLOOD CONTROL

CLAREMONT DAM

EMBANKMENT DETAILS NO. 2

SUGAR RIVER — NEW HAMPSHIRE

IN 94 SHEETS — SCALE: 1 IN. = 40 FT. — SHEET NO. 11

U.S. ENGINEER OFFICE, PROVIDENCE, R.I., JAN. 1945

SUBMITTED: D. Hansen, SENIOR ENGINEER, PROJECT ENGINEER

APPROVAL RECOMMENDED: C. Tavano, HEAD ENGINEER, CHIEF ENGINEERING DIV.

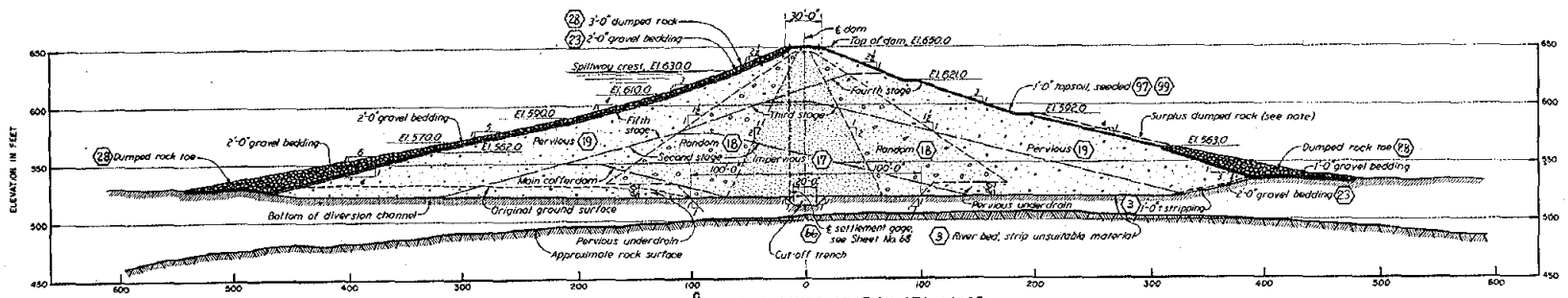
APPROVED: [signature], COL. CORPS OF ENGINEERS, DISTRICT ENGINEER

PREPARED: [signature], PROJECT UNIT NO. 8

DRAWN: C. W. V.; TRACED: B.C.; CHECKED: [illegible]

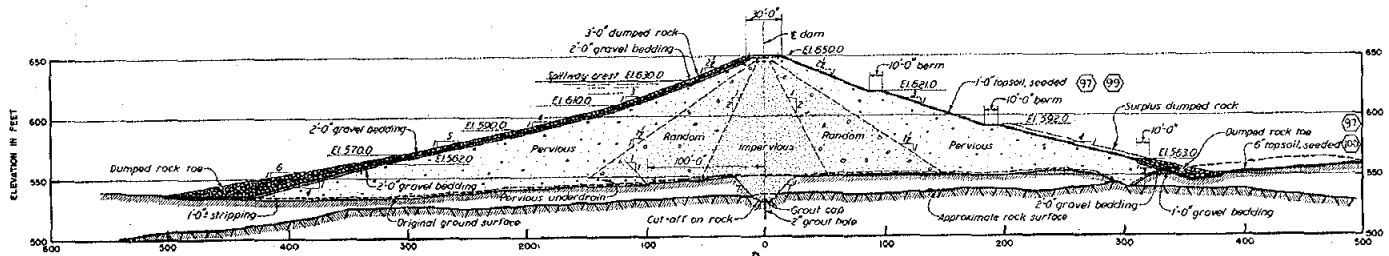
FILE NO. CT-1-1781

PLATE NO. 56



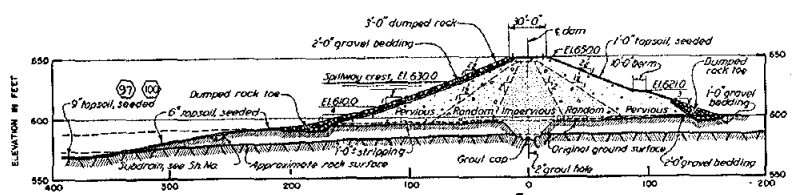
SECTION $\frac{C}{10}$ THRU CLOSURE SECTION STA. 20+25

SCALE 1" = 40'



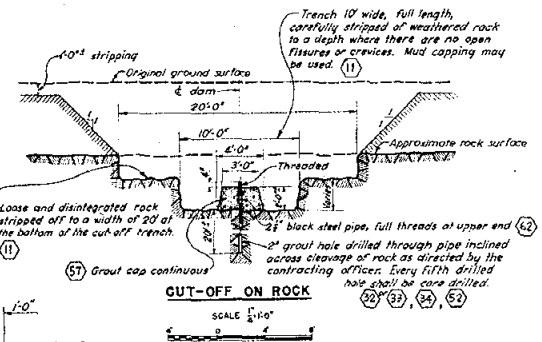
SECTION $\frac{D}{10}$ STA. 21+40

SCALE 1" = 40'



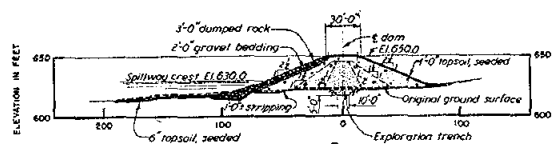
SECTION $\frac{E}{10}$ STA 25+00

SCALE 1" = 40'



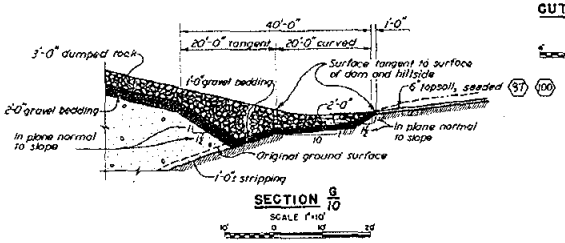
GUT-OFF ON ROCK

SCALE $\frac{1}{8}$" = 1'-0"



SECTION $\frac{F}{10}$ STA. 4+30

SCALE 1" = 40'



SECTION $\frac{G}{10}$

SCALE 1" = 10'

NOTES

If available, surplus rock from excavations and borrow pits shall be dumped on the downstream surface of the dam between El. 563.0 and El. 592.0.

For general notes applying to details on this sheet, see Sheets No. 10 and 11.

Stage construction indicated on Section $\frac{G}{10}$ shall be used throughout the closure section.

KEY	DATE	REVISION (Indicated by △)	REVBY	CK. BY	AP. BY

CONNECTICUT RIVER FLOOD CONTROL

CLAREMONT DAM

EMBANKMENT DETAILS NO. 3

SUGAR RIVER — NEW HAMPSHIRE

IN 94 SHEETS — SCALE: 1 IN. = 40 FT. — SHEET NO. 12

U. S. ENGINEER OFFICE, PROVIDENCE, R.I., JAN. 1945

SUBMITTED: D. D. Murphy, Resident Engineer, Project Engineer

APPROVAL RECOMMENDED: R. P. Burns, Chief, Engineering Div.

APPROVED: W. E. Guess, Colonel, C. of E., District Engineer

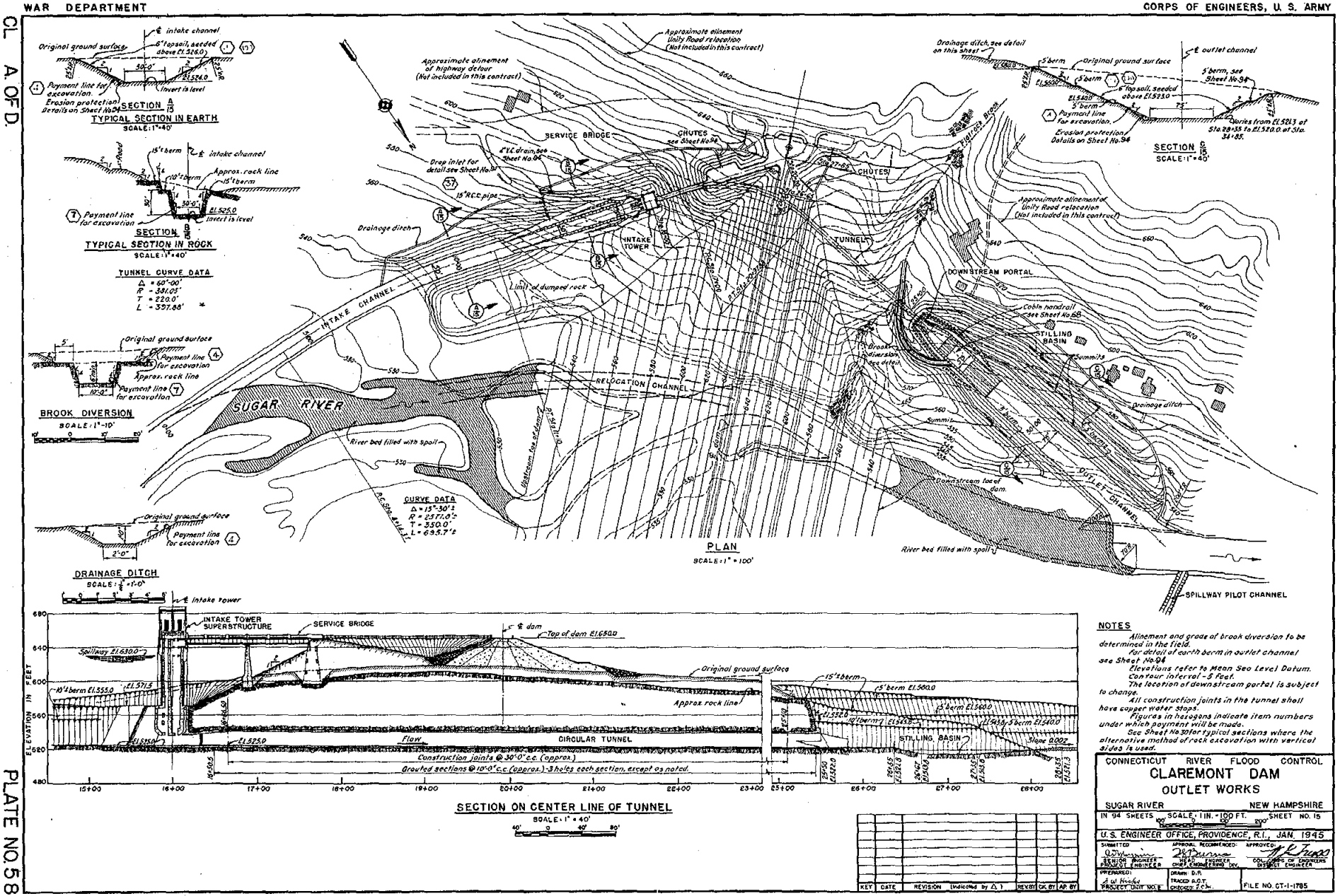
PREPARED: A. J. Maley, Project Unit No. 8

DRAWN: C.W.V. TRACED: J.A.C.-H.A.R. CHECKED: J.A.C.

FILE NO. CT-1-1782

CL A. OF D.

PLATE NO. 57



NOTES

Alinement and grade of brook diversion to be determined in the field.

For detail of earth berm in outlet channel see Sheet No. 94

Elevations refer to Mean Sea Level Datum.

Contour interval - 5 Feet.

The location of downstream portal is subject to change.

All construction joints in the tunnel shall have copper water stops.

Figures in hexagons indicate item numbers under which payment will be made.

See Sheet No. 30 for typical sections where the alternative method of rock excavation with vertical sides is used.

CONNECTICUT RIVER FLOOD CONTROL

CLAREMONT DAM

OUTLET WORKS

SUGAR RIVER — NEW HAMPSHIRE

IN 94 SHEETS — SCALE: 1 IN. = 100 FT. — SHEET NO. 15

U. S. ENGINEER OFFICE, PROVIDENCE, R.I., JAN. 1945

SUBMITTED: O. Williams, ASSISTANT ENGINEER — RESIDENT ENGINEER

APPROVAL RECOMMENDED: J. R. Burns, CHIEF ENGINEER, FLOOD CONTROL DIV.

APPROVED: J. R. Fuller, COLONEL, CORPS OF ENGINEERS, DISTRICT ENGINEER

PREPARED: A. W. Hicks, PROJECT UNIT NO. 2

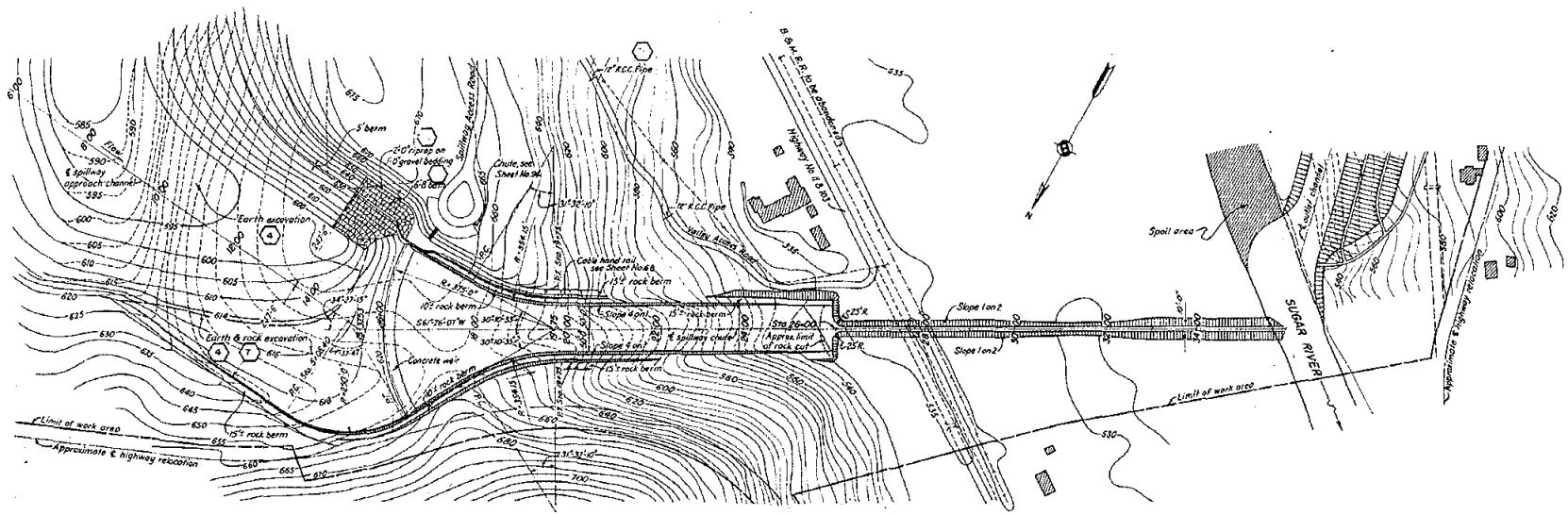
DRAWN: D.R. TRACED: R.D.T. CHECKED: F.C.M.

FILE NO. CT-1-1785

KEY	DATE	REVISION (Indicated by △)	REVIEW	CK. BY	AP. BY

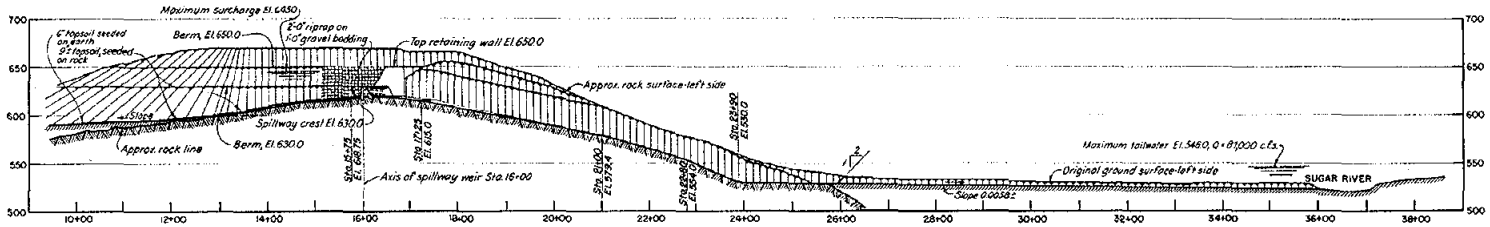
CL A. OF D.

PLATE NO. 58



PLAN

SCALE - 1" = 100'



PROFILE ON ℄ SPILLWAY CHANNEL

SCALE HORIZONTAL 1" = 100'
VERTICAL 1" = 50'

NOTES

Elevations refer to Mean Sea Level Datum
Contour interval is five feet and two feet
For details of spillway see Sheets No. 30, 31 and 32.
Figures in hexagons indicate item numbers under which payment will be made.

KEY	DATE	REVISION (Indicated by △)	REV BY	CK BY	AP BY

CONNECTICUT RIVER FLOOD CONTROL

CLAREMONT DAM

SPILLWAY

PLAN & PROFILE

SUGAR RIVER — NEW HAMPSHIRE

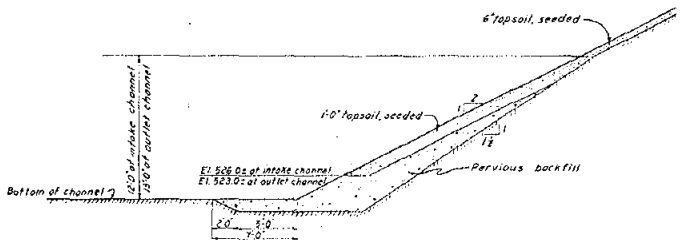
IN 94 SHEETS — SCALE 1IN. = 100 FT. — SHEET NO. 29

U.S. ENGINEER OFFICE, PROVIDENCE, R.I., JAN. 1945

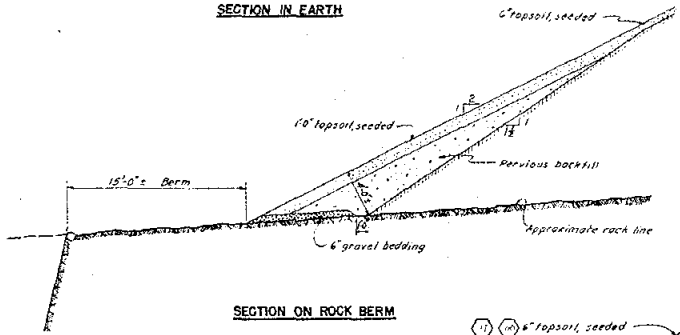
SUBMITTED: SENIOR ENGINEER, PROJECT ENGINEER — APPROVAL RECOMMENDED: HEAD ENGINEER, CHIEF, ENGINEERING DIVISION — APPROVED: COL. CORPS OF ENGINEERS, DISTRICT ENGINEER

PREPARED: PROJECT UNIT NO. 2 — DRAWN: C.W.V., G.H.H. — TRACED: H.V.R. — CHECKED: I.P.

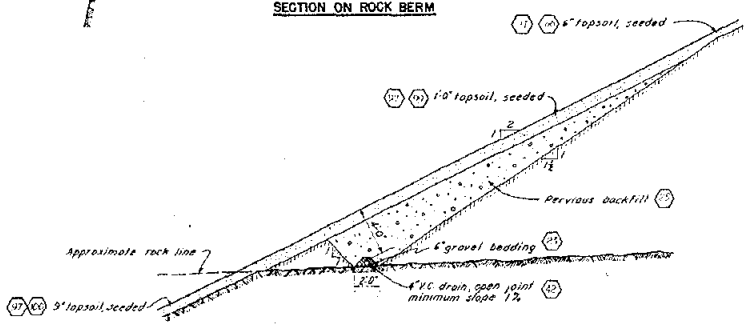
FILE NO. CT-1-1799



SECTION IN EARTH



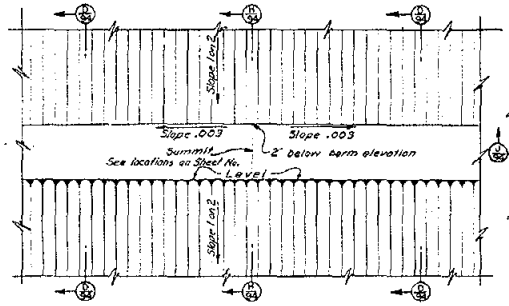
SECTION ON ROCK BERM



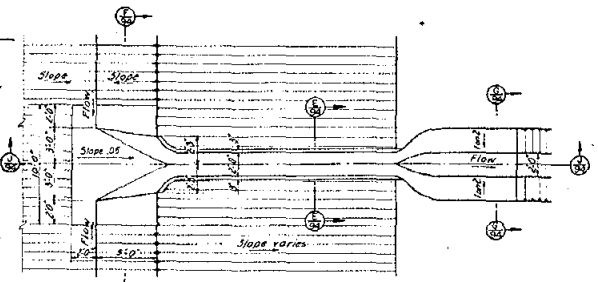
SECTION $\frac{B}{16}$

EROSION PREVENTION DETAILS

SCALE $\frac{1}{4}$" = 1'-0"

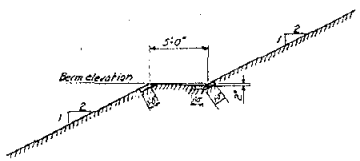


PLAN

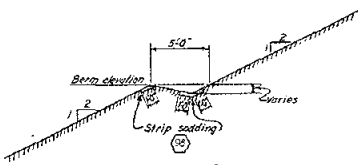


PLAN

SCALE $\frac{1}{4}$" = 1'-0"



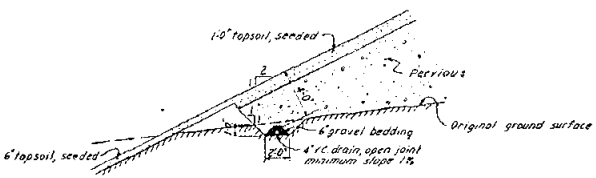
SECTION $\frac{N}{94}$



SECTION $\frac{D}{94}$

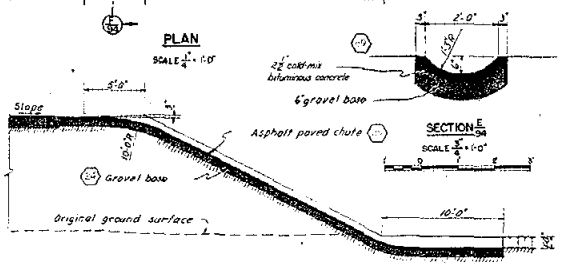
BERM AT OUTLET CHANNEL

SCALE $\frac{1}{4}$" = 1'-0"



SECTION $\frac{C}{16}$

SCALE $\frac{1}{4}$" = 1'-0"

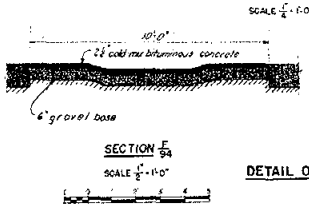


SECTION $\frac{J}{94}$

SCALE $\frac{1}{4}$" = 1'-0"

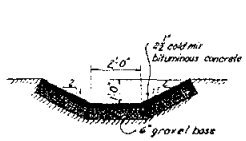
SECTION $\frac{E}{94}$

SCALE $\frac{3}{4}$" = 1'-0"



SECTION $\frac{F}{94}$

SCALE $\frac{1}{2}$" = 1'-0"



SECTION $\frac{G}{94}$

SCALE $\frac{1}{2}$" = 1'-0"

DETAIL OF CHUTE

NOTES:

Elevations refer to Mean Sea Level Datum.

Figures in hexagons indicate bid item numbers under which payment will be made.

For location of chutes see Sheets No. 15 and 92.

Erosion prevention treatment will be used on the uphill side of the intake and outlet channels where, in the opinion of the contracting officer, ground water will cause erosion or sloughing of the earth slopes.

KEY	DATE	REVISION (Indicated by △)	REC'D BY	CK. BY	AP. BY

CONNECTICUT RIVER FLOOD CONTROL

CLAREMONT DAM

MISCELLANEOUS DRAINAGE DETAILS

SUGAR RIVER — NEW HAMPSHIRE

IN 94 SHEETS — SCALE 1/4 IN. = 1 FT. — SHEET NO. 94

U.S. ENGINEER OFFICE, PROVIDENCE, R.I., JAN. 1945

SUBMITTED: G. D. Murphy, SENIOR ENGINEER, PROJECT ENGINEER

APPROVAL RECOMMENDED: J. B. Burns, HEAD ENGINEER, CHIEF ENGINEERING DIV.

APPROVED: J. B. Burns, COL., CORPS OF ENGINEERS, DISTRICT ENGINEER

PREPARED: Alf Hoko, PROJECT UNIT NO. 8

DRAWN: D.C. — O.P.

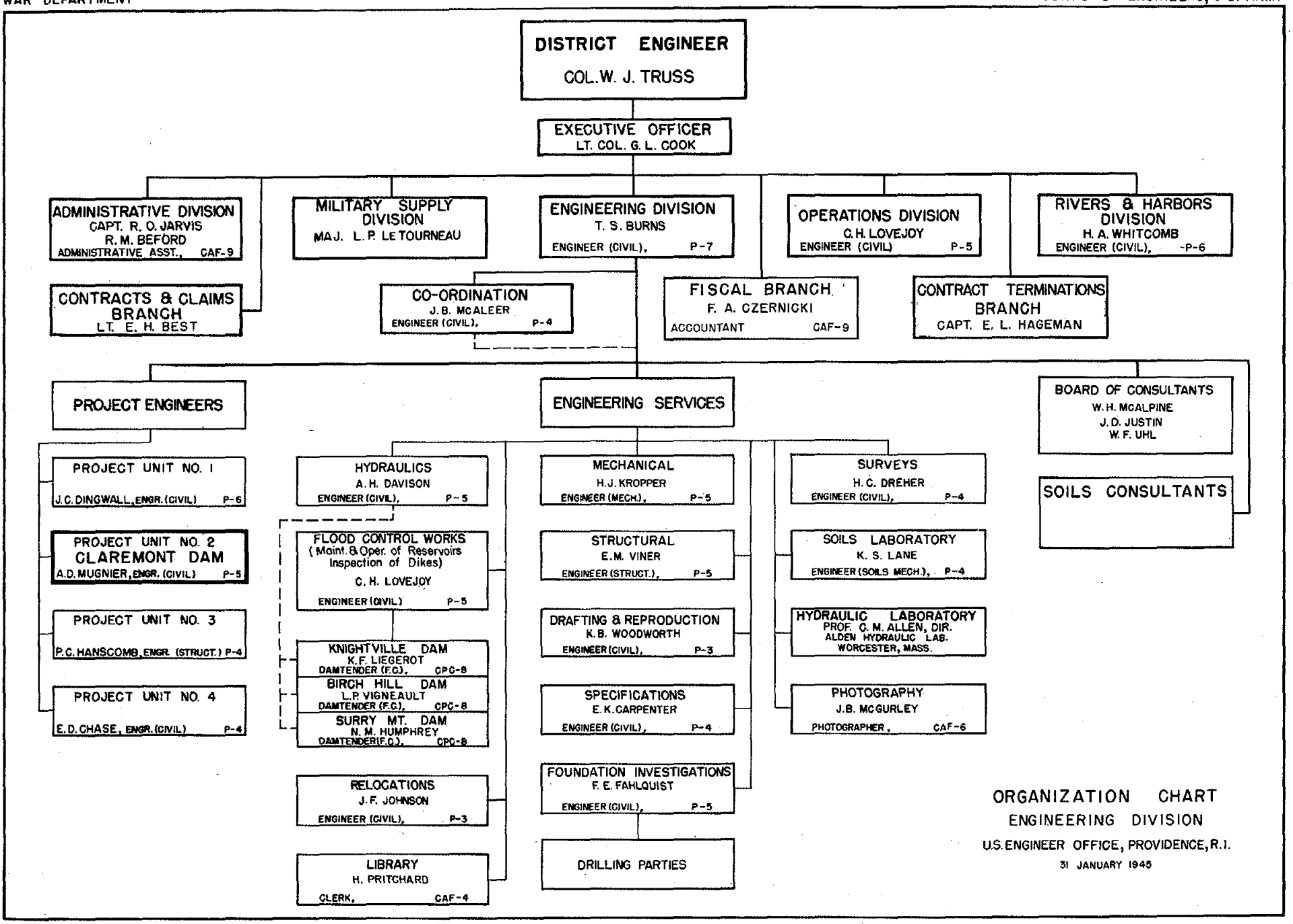
TRACED: R.J.O.

CHECKED:

FILE NO. CT-1-1864

WAR DEPARTMENT

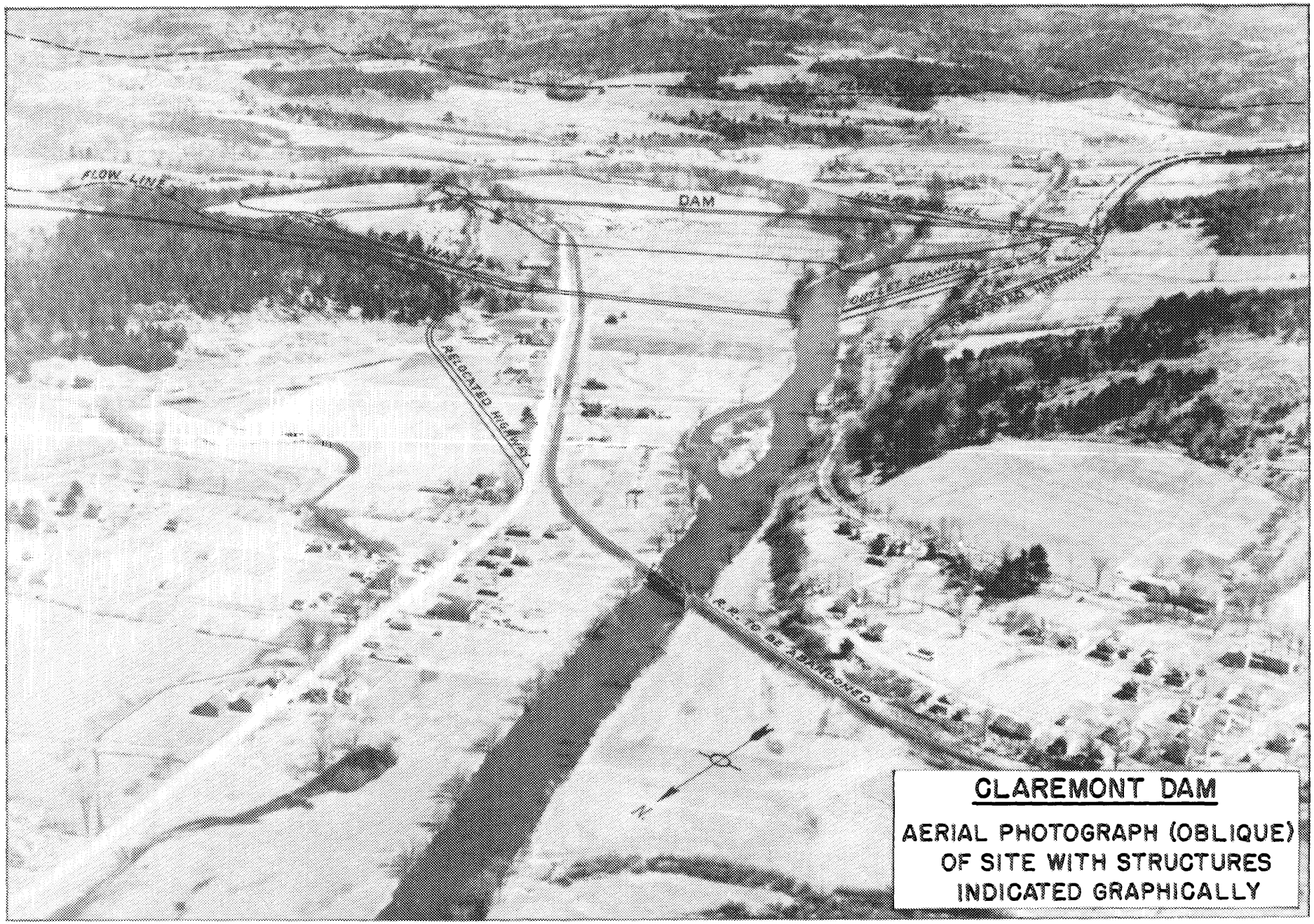
CORPS OF ENGINEERS, U.S. ARMY



ORGANIZATION CHART
ENGINEERING DIVISION
U.S. ENGINEER OFFICE, PROVIDENCE, R.I.
31 JANUARY 1945

CL A OF D

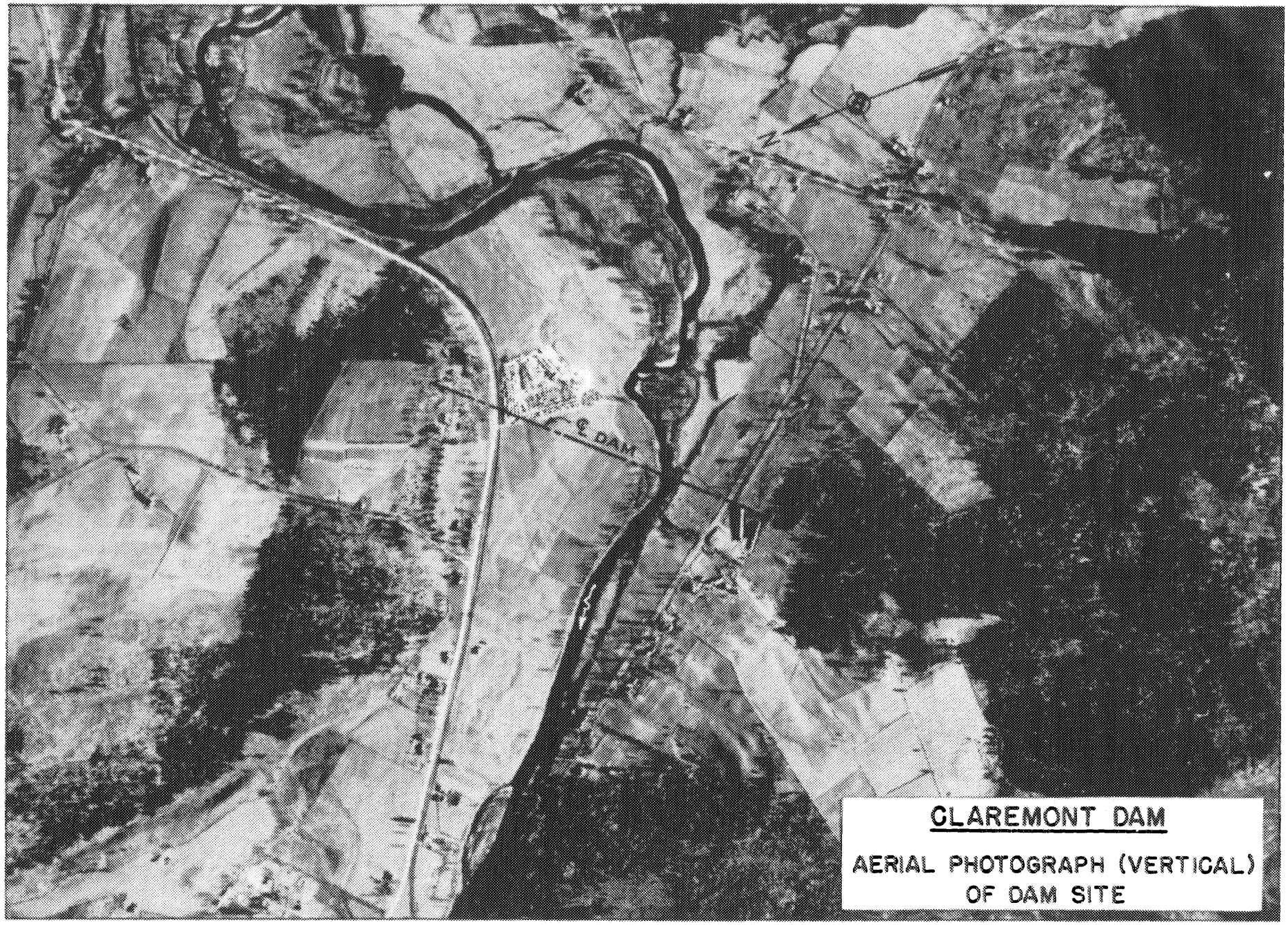
PLATE NO. 61



CLAREMONT DAM

AERIAL PHOTOGRAPH (OBLIQUE) OF SITE WITH STRUCTURES INDICATED GRAPHICALLY

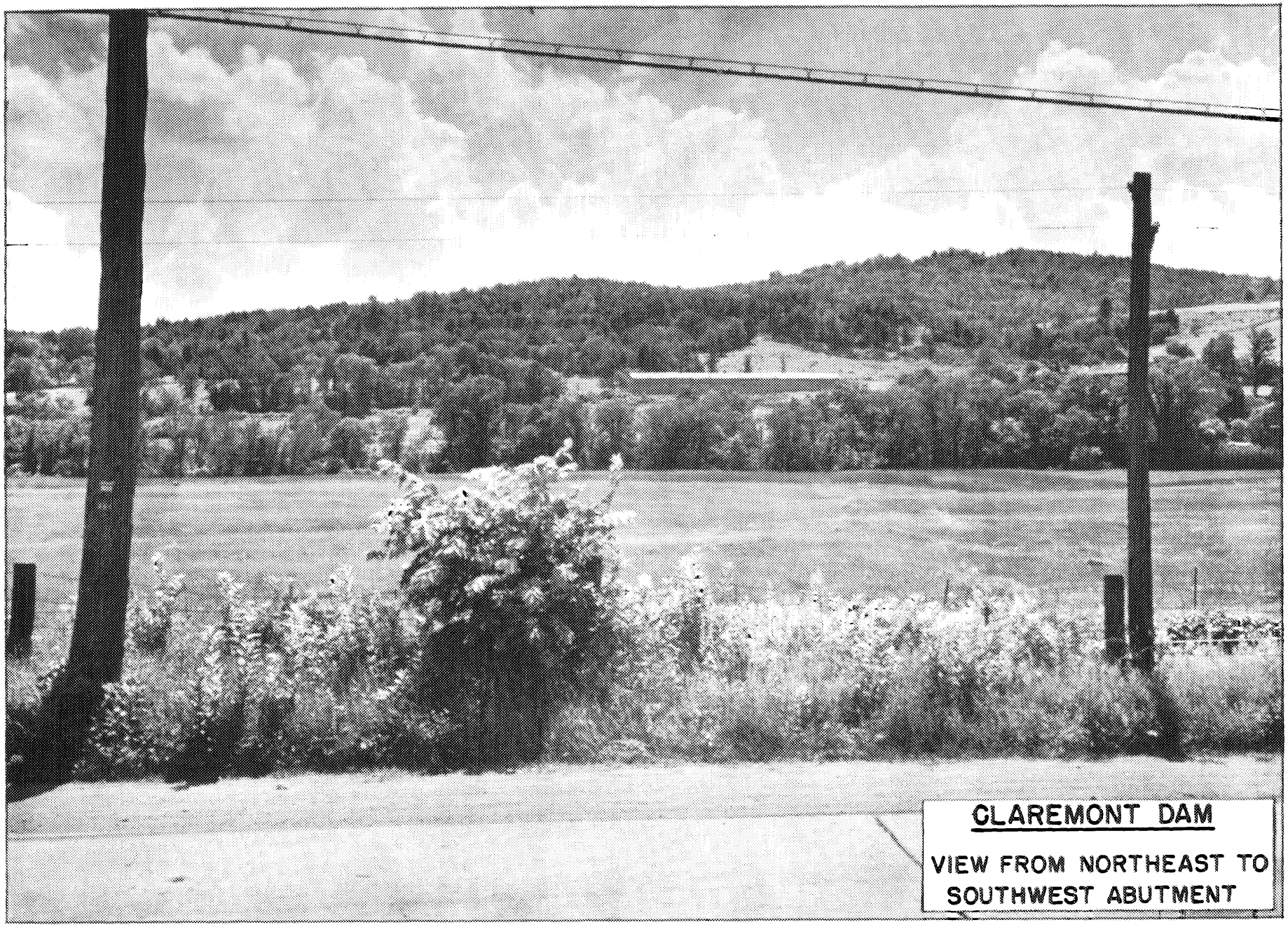
PHOTO NO. 1



CLAREMONT DAM

AERIAL PHOTOGRAPH (VERTICAL)
OF DAM SITE

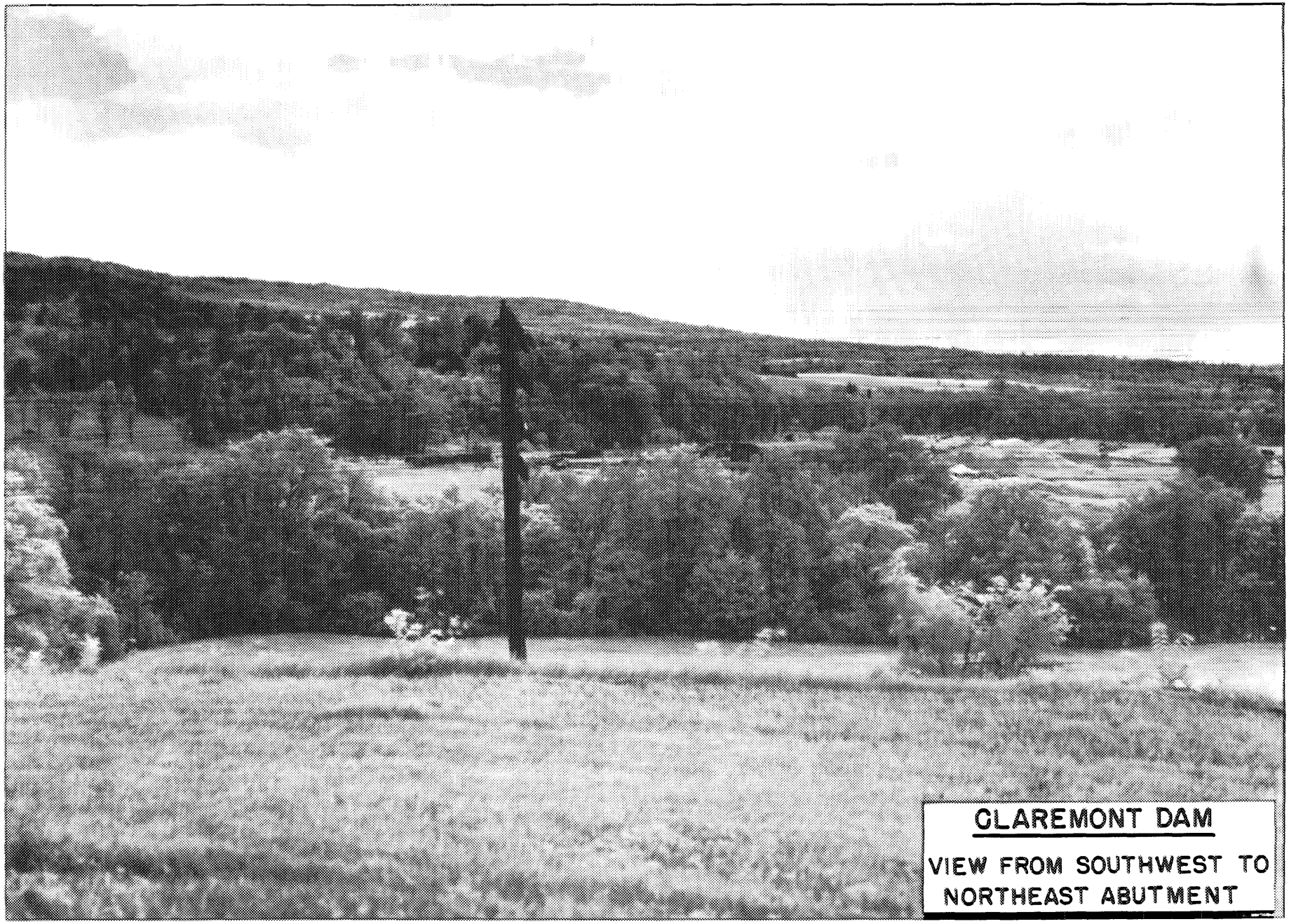
PHOTO NO. 2



CLAREMONT DAM

VIEW FROM NORTHEAST TO SOUTHWEST ABUTMENT

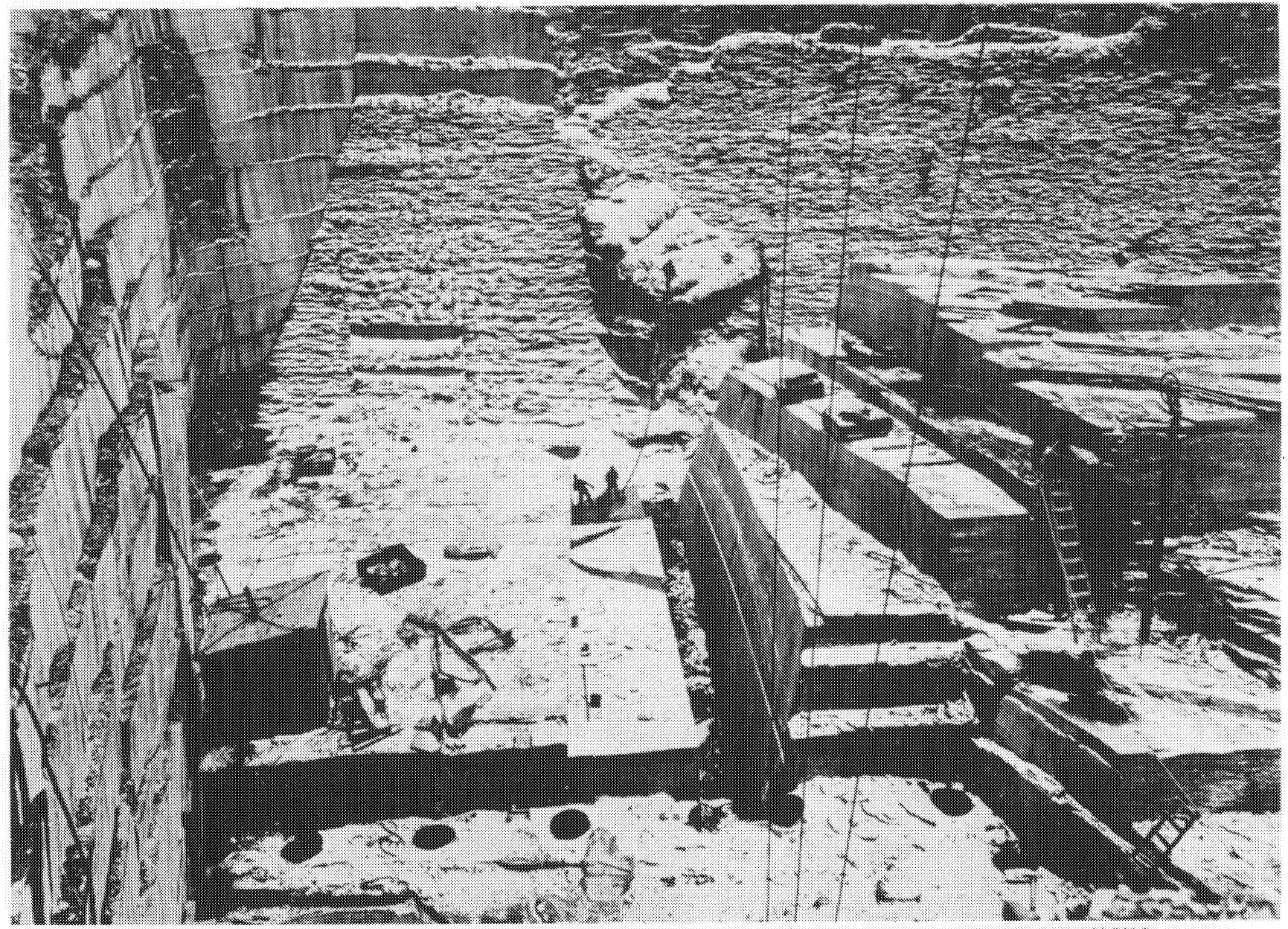
PHOTO NO. 3



CLAREMONT DAM

VIEW FROM SOUTHWEST TO NORTHEAST ABUTMENT

PHOTO NO. 4



WIRE SAW METHOD OF EXCAVATING IN SLATE QUARRY, PENNSYLVANIA

(Note smooth character of wall rock or rock faces left in place.)

PHOTO