MERRIMACK VALLEY FLOOD CONTROL

ANALYSIS OF DESIGN
FOR
BENNINGTON DAM

BENNINGTON
NEW HAMPSHIRE

1946

CORPS OF ENGINEERS, U.S. ARMY.

U.S. ENGINEER OFFICE BOSTON, MASS.

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SECTION 1. PERTINENT DATA

The following tabulation summarizes data which are pertinent to the design of the initial and ultimate developments of Bennington Dam:

1. Location. -

The dam is to be located on the Contoocook River, one half (1/2) mile south of the Town of Bennington, N. H., approximately 18 miles northeast of Keene, New Hampshire, and approximately at river mile 147 above the mouth of the Merrimack River.

2. Drainage Area. -

Net Drainage Area	142 Sq. Mi.
Gross Drainage Area	186 Sq. Mi.

3. Meteorological Data. -

Average Annual Rainfall	39.88 inches
Max. 24-hour precipitation in 24 hours. (Concord, N.H.)	5.97 inches

4. Flood Data. -

5 November 1927	5,500 c.f.s.(Est.)
19 March 1936	13,600 c.f.s.
21 September 1938	15,400 c.f.s.

5. Hydrological Data. -

Average Yearly Flow	300 c.f.s.
Average Annual Flood	3,100 c.f.s

6. Reservoir Data. -

	Initial	Ultimate
Flood Control Storage	60,000 acre ft.	50,000 acre ft.
Conservation Storage	1,500 acre ft.	40,000 acre ft.
Conservation Storage Elevation	678.2	699.5
Length of Conservation Pool	3.8 miles	9.2 miles
Area of Conservation Pool	300 acres	3,300 acres
Elevation of Spillway Crest	705.0	712.0
Reservoir Area at Maximum Surcharge	5,020 acres	5,660 acres
Surcharge Storage Capacity	52,000 acre ft.	56,000 acre ft.
Surcharge Storage Utilized	5.2 inches	5.7 inches

7. Dam. -

Type -	Rolled Earth Fill
Total Length -	3535 feet

	Initial	Ultimate
Elevation Top of Dam	724.0	730.0
Maximum Height Earth Embankment	64.0 Ft.	70.0 Ft.
Principal Top Width of Embankment	25.0 Ft.	25.0 Ft.
Freeboard Provided	7.2 Ft.	6.9 Ft.

8. Spillway. -

Type	-	Concrete Ogee Section
Total Length	-	300 feet

Elevation Spillway Lip	705.0	712.0
Maximum Surcharge	11.8	11.1
Coefficient of discharge at maximum discharge	3.8	3.8
Spillway Discharge at maximum discharge	45,900 c.f.s.	42,200 c.f.s.

9. Outlet Hydraulics. -

Number of Conduits	6, of which 2 will be spares
Size of Conduit and Gate	4' x 6' high
Type of Gate	Sluice Gates, hydraulically operated
Number of gates per Conduit	2, of which one is an upstream emergency gate and the other a service gate.

10. Designed Discharge Through Outlets. - (Combined for 4 Conduits)

Initial Development	3,680 c.f.s.
Ultimate Development	4,040 c.f.s.

11. Down Stream Channel Capacity. - 4,000 c.f.s.

12. Spillway Design Flood. -

Based on 24-hour Duration as follows:

Total Rainfall	17.4 inches
Infiltration Rate	0.05 inches per hour
Total Run-off	16.3 inches
Peak Inflow to Reservoir	77,600 c.f.s.
Peak Spillway Discharge:	
Initial	45,900 c.f.s.
Ultimate	42,200 c.f.s.
Maximum Water Surface Elev.	
Initial	716.8
Ultimate	723.1
Maximum Fetch	
Initial	5.5 miles
Ultimate	5.5 miles
Wind Velocity	
Initial	80 miles per hour
Ultimate	80 miles per hour

13. Tailwater Elevations. -

	Initial	Ultimate
Pool with Maximum Spillway Discharge Elev.	676.0 (Min.)	675.4 (Min.)
Pool at 4 Outlets Discharge Elev.	667.3	667.6

14. Demonstration Floods (Reservoir Design Floods).-

	Initial 1936	Initial 1938	Ultimate 1936
Natural Peak at Bennington	13,600 cfs	15,400 cfs	13,600 cfs
Reservoir Inflow Allowing for Valley Storage	16,500 cfs	18,000 cfs	16,500 cfs
Reservoir Inflow Reduced by West Peterboro Reservoir	14,500 cfs	14,700 cfs	14,500 cfs
Vol. Bennington Hydrograph	12.0 "	7.4"	12.0"
Duration of Flood in Days	13	6	13
Effective Controlled Reservoir Discharge During Flood	1,800 cfs	1,760 cfs	2,000 cfs
Max. Reservoir Discharge During Emptying	4,000 cfs	4,000 cfs	4,000 cfs
Max. Water Surface Elev.	706.0	702.1	712.9
Flood Control Storage Utilized	64,000 a.f.	49,000 a.f.	54,000 a.f.

15. Cofferdams. -

Upstream Cofferdam-elevation - 685.0
Downstream Cofferdam-elevation - 670.0

16. Miscellaneous Design Features. -

Bellmouth Intake Elliptical
Maximum Discharge Velocities at Portal
Initial 40 f/sec.
Ultimate 44 f/sec.
Floor Transition Sections at intake Flat Parabolic
Metal Conduit Liner 16 feet-8 inches long
Gate Operation:

Reservoir Stage	Gates Open	Total Discharge in cfs	Storage Acre Ft.
678.2	1	400	0
680	2	920	1,000
682.0	3	1,530	3,000
684.0	4	2,240	5,000

(2 additional gates as spares)

Stilling Basin:
Length 104 feet
Baffles 2 rows, 5 feet high

17. Quantities - Principal Items of Work. -

Total Volume of Earth Dam

Compacted Impervious Material	150,000 c.y.
Compacted Pervious Material	174,000 c.y.
Compacted Random Material	204,000 c.y.
Semi-compacted Fill	100,900 c.y.
Hand-compacted Fill	6,000 c.y.
Selected Gravel and Gravel Backing	78,000 c.y.
Screened Gravel	4,100 c.y.
Sand	800 c.y.
Dumped Rock Fill	85,700 c.y.
Cobble Facing	7,500 c.y.
Derrick Stone	5,200 c.y.
Concrete	50,725 c.y.
Porous Concrete	1,800 c.y.
Total	868,725 c.y.

Excavation Items:

Stripping	92,000 c.y.
Common Excavation	496,000 c.y.
Trench Excavation	2,800 c.y.
Borrow Excavation	310,400 c.y.
Rock Excavation	53,100 c.y.
Total ...	954,300 c.y.

Reinforcing Steel	387,500 lbs.
Miscellaneous Metal	38,000 lbs.
Corrugated Metal Pipe	3,200 l.f.

Equipment Building

Tile Gages
Piezometers
Wells
Manhole
Monorail Hoist
Gasoline Electric Standby Unit
Oil Pressure System
Electric Power, Lighting and Telephone Systems
Roadway
Seeded Topsoil - 82,000 s.f.
Highway Guard Rail
Water Supply Well
Spillway Relief Wells
Maintenance Building
Operators' Quarters
Log Boom
Water Level Recorder
Slide Gates

SECTION II. INTRODUCTION

2-01. AUTHORITY.- The Bennington Reservoir was approved by the Chief of Engineers 10 December 1943 and is a part of the comprehensive plan for flood control of the Merrimack River Basin as authorized by the Flood Control Act approved 22 June 1936 and amended by the Flood Control Act approved 28 June 1938.

(a) Flood Control Act.- (1) Section 5 of the Flood Control Act, approved 22 June 1936 (Public No. 738--74th Congress) is as follows:-

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

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

(2) Section 4 of the Flood Control Act, approved 28 June 1938, (Public No. 761--75th Congress, 3rd Session) is as follows:-

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

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

(b) Present Plan.- The Bennington Reservoir was not included in the original comprehensive plan of reservoirs and related flood control works, but as a result of extensive investigations by and in accordance with the recommendations of the Board of Engineers for Rivers and Harbors, was approved by the Chief of Engineers in the 10th Indorsement, dated 10 December 1943, by letter from the Chief of Engineers to the Resident Member, Board of Engineers for Rivers and Harbors, dated 6 December 1941, subject: "Reservoir Plans for the Contoocook Basin, New Hampshire," File No. 7402 (Merrimack River, Hopkinton-Everett Res.) 41, and was substituted together with Beards Brook Reservoir for the Hopkinton-Everett Reservoir.

2-02. DESCRIPTION OF PROJECT.- (a) Location.- The Bennington Dam is located on the Contoocook River one-half (1/2) mile south of the town of Bennington, New Hampshire.

(b) Initial Development.- In the proposed initial flood control development of the Bennington site the dam consists of a gravity ogee type concrete spillway, concrete abutments and earth-filled embankment sections. The concrete abutments are adjacent to the spillway and consist of gravity sections 97'-6" long which connect with the earth embankments. The embankment sections have an aggregate length of 3255 feet, a maximum height of 64 feet and are constructed of rolled earth fill with a central impervious core. The embankment is protected by an upstream rock blanket and downstream rockfilled drainage toe. The top width of the dam at Elevation 724. is 25 feet with an average slope of 1 on 2-1/2 on the upstream side and 1 on 2-1/4 on the downstream side. The top width and side slopes of the ultimate embankment will be similar to that of the initial construction. The stilling basin is composed of a concrete mat with reinforced concrete baffles, end sill and concrete gravity-type guide walls. It is proposed to construct the dam initially for flood control only, and provisions are made in the design of the initial stage to enable raising the embankment, spillway and abutments.

(c) Ultimate Development.- In view of the favorable results of an analysis of this project for future conservation storage in addition to flood control storage, provision has been made in the initial design for an ultimate addition of seven (7) feet to the spillway and six (6) feet to the embankment and abutments. The ultimate construction will accommodate an additional 30,000 acre feet of storage.

(d) Spillway Structures.- The spillway structure has a crest length of 300 feet at Elevation 705. and a maximum height of 49 feet. Six (6) gated conduits, 4' wide by 6' high, are centrally located in the spillway for control of the discharge. Access is gained to the gate chambers from the equipment building which is located on the downstream berm on the east side of the stilling basin by means of a passageway incorporated with the stilling basin gravity wall, thence through the spillway section to the various gate chambers, thence to an adit on the west side of the spillway. This plan provides an emergency exit and

a convenient method of access from one side of the dam to the other. A detailed analysis of design is given in Section VIII of this Analysis of Design.

(e) Equipment Building.- The equipment building is located alongside the spillway as shown on Plate A-2. The building is of brick masonry with limestone trim and has a concrete basement section as shown on Plate A-20. Access to the passageway through the spillway is gained from the heater room in the basement of the building.

(f) Maintenance Building.- A maintenance building constructed of concrete block and having a pitched roof with asphalt shingles and provided to house a shop, tool room, oil and paint storeroom, general storage space, toilet, shower, dressing room, and heater room is constructed about 200 feet east of the equipment building adjacent to the access road leading to the equipment building and the spillway. (See Plate A-2). A three-car garage with shop space is attached to one end of the building. During the construction of the dam this building will be used as a temporary field office. (See Plate A-21.)

(g) Access Roads.- Two access roads are provided. The access road on the east end of the dam leads from a proposed highway to the maintenance building, equipment building and the spillway. The access road on the westerly end of the dam leads off the existing U. S. Highway Route 202 to the top of the embankment. (See Plate A-2). Posts with chains are provided to prevent use of either access road by the general public.

(h) Downstream Terrace and Dike.- The downstream terrace is constructed of random pervious materials rolled into a semi-compacted mat as noted in the specifications. The dumped rock dike is constructed along the edge of the discharge channel and the area between the dike and the embankment is filled with random materials, compacted, as noted above, to form the downstream terrace.

(i) Estimated Cost.- The estimated cost for construction of the dam over a period of two (2) years is $5,777,000, which includes the reservoir and construction costs.

2-03. DEPARTURES FROM DEFINITE PROJECT REPORT.- (a) Changes Recommended by New England Division Office.- (1) Elimination of Trash Racks and Addition of Log Boom.- In the first indorsement dated 30 April 1945 to the letter of transmittal of the Definite Project Report, Division Engineer to the Chief of Engineers, the following statements were made with reference to eliminating the trash racks and substituting of log boom therefor:

"It is suggested that trash racks be eliminated at the entrance to the outlet conduits. It should be noted that in order to maintain a minimum flow in the river to compensate for the loss of the Powder Mill

Dam, located upstream from the proposed Huntington Dam, then the outlets will normally be submerged. Freshets would be expected to further submerge the outlets. Under these conditions the trash racks would not afford sufficient protection to justify their installation. It is believed that a log boom located in the pool upstream of the outlets would be more effective than trash racks."
The above recommendations were concurred with by the Chief of Engineers and incorporated in the final design.

(c) Elimination of Operating Platform.- An Operating Platform was to be provided on the upstream face of the spillway section as a working area when placing stop logs upstream of the outlet control gates. The operating level of the platform is indicated at Elevation 677.0. Under the ultimate stage of construction with conservation pool level at Elevation 699.5 (40,000 acre feet), the working level would be submerged by 12.5 feet and would serve no useful purpose. Under normal operation, three gates would be closed. Repairs to any one of the operating gates (downstream gate) can be effected, without resort to the use of stop logs, by closing the upstream gate. In the event it becomes necessary to repair the upstream gate on the side walls of the conduit, the work could be carried on during low water stage in the reservoir by the use of either a cofferdam or wooden bulkhead placed upstream of the entrance to the conduit in question. The stop log recesses and operating platform have been eliminated in accordance with recommendations made by the Division Engineer and concurred with by the Chief of Engineers.

(d) Changes Recommended by the Chief of Engineers.- In the second indorsement dated [illegible] to the letter of transmittal of the Definite Project Report, Chief of Engineers to the Division Engineer, the following recommendations were made, as quoted below:

"Drainage should be provided for the terrace at Elevation 670."

"Stilling basin baffle blocks should be placed with the vertical face downstream."

"The air vent for the downstream sluice gate should be eliminated."

The above recommendations were complied with in the final design.

(e) Changes Made by the District Office.- (1) Addition of West Access Road.- West Access Road, leading from the existing U. S. Highway [illegible] to the top of the embankment, is provided in the final design. It provides the only means by which an inspection or maintenance car can get to the top of the embankment west of the spillway and is considered necessary for [illegible].

(2) Addition of Field Office.- Inasmuch as there is no [illegible] site which is available for use as a field office during construction, it is necessary to construct such a building; and since a [illegible] building will be required as a permanent structure at the [illegible] the field office [illegible] designed that it could be converted [illegible] without alterations.

(3) Cobble Blanket on Downstream Face.- The downstream slope above the elevation of the top of the rock fill on the downstream slope is protected against surface erosion by a one-foot thick layer of cobbles which may be obtained by collecting oversize stones from impervious and random sections, from raking the surface of the selected pervious section of the embankment during construction, and from other sources to be determined by the contractor. It was the intent of the original design that this blanket be sand, gravel and cobbles. This change from sand, gravel and cobbles, to cobbles only was influenced by the fact that there would be a large quantity of cobbles available from culling the fill and from the borrow areas and that the cobbles would afford more permanent protection for the side slopes than sand, gravel and cobbles.

(4) Elimination of Removable Block on Top of Spillway.- The removable block section shown on the original drawings of the spillway and originally considered a convenient means of keying the ultimate section of the spillway with the initial section, is eliminated. When it is decided to reconstruct the dam to the ultimate height, it is believed that it will be simpler to cut a keyway in the concrete than it would be to attempt to remove a definite preformed section of concrete. It is also believed that a newly cut section will afford better bond between the ultimate section and the initial structure, and that since there will not be any reinforcing steel in the concrete section to be cut, the keyway can be neatly constructed.

(5) Elimination of Seepage Drain Pipes from Spillway.- The seepage drain pipes shown on the original drawings and intended to provide escape through the spillway for any water entering the joint between the original concrete spillway and the concrete segment added for the ultimate is eliminated because it is believed that the head of water would always be the same both inside and outside the joint; and should the head on the outside recede, the head in the joint would also recede though at a slightly slower rate due to the resistance offered by wall friction in the joint. It is also believed that no harm would be done to the structure with water standing in the joint. In view of this supposition it is further believed that the seepage pipes would not provide any practical use.

(6) Modification of Equipment Building.- After a more detailed study of the layout and facilities of the Equipment Building the following additions and changes were made. (See Appendix IV of the Definite Project Report, Paragraph g):

a. The framework of structural steel in the walls is eliminated.

b. Space for a repair shop is provided on the first floor.

c. Storeroom is changed from basement to the first floor.

d. Two (2) fuel oil tanks are provided in heater room.

e. Gasoline storage tank for standby generator is buried in ground outside building.

f. Septic tank is provided for waste.

B.

SECTION III. SELECTION OF SITE AND TYPE OF DAM

3-01. INTRODUCTION. - The available limits within which the dam could be located are included within the entire reach of the Contoocook River between the Monadnock Power Dam and a point approximately 1,200 feet upstream from the Powder Mill Dam. The area within these limits was explored for the most suitable site by the usual methods of damsite explorations described in other sections of this design analysis. Following the completion of the subsurface exploration, numerous engineering studies and comparative estimates of various arrangements of structures were made. With consideration given to the initial and the ultimate developments described in the Definite Project Report, the most suitable and economical location and design were selected.

3-02. SELECTION OF DAM SITE.- The number of possible damsite locations included in the investigations was reduced considerably due to the fact that the underlying strata of the upstream area was considered too pervious for the foundation of heavy masonry structures. The down stream area was considered unsatisfactory since a dam constructed in this area would encroach upon the town of Bennington and decrease the efficiency of the Monadnock Power Dam and the diversion canal, both of which are owned and used by the Monadnock Paper Mills. The site selected as most suitable, giving due consideration to the reasons set forth above and the value of the site selected as a damsite, is shown on Plate No. A-2.

3-03. SELECTION OF TYPE OF DAM.- (a) Further foundation exploration was authorized as soon as the damsite was selected. This exploration revealed the surface deposits to be gravelly and silty sand to a depth of ten (10) to fifteen (15) feet. Beneath these surface deposits the soil in the east bank is a compact glacial till which is continuous to bed rock and provides an excellent foundation for the abutment and spillway structure. This till extends under the river bed and tapers out in the west bank where the till is underlain by variable sands and silt as shown on the geological profiles on Plates Nos. VI-1 and VI-2. Except for a small area where till exists as a thin layer near the ground surface, the west bank is composed entirely of sand and silt. Bed rock varies in depth between fifty (50) feet on the east bank to more than one hundred fifty (150) feet on the west bank below the ground surface.

(b) In view of the heterogeneous foundation it was decided that an earth fill embankment was the most economical section. The section of dam selected has a central impervious core flanked by random fill and blanketed with pervious fill on the upstream and downstream faces, as shown on Plates Nos. A-14 and A-15. The upstream face and downstream toe are further protected by a dumped rock blanket. The downstream face is protected by a cobble facing. With the exception of rock, all materials for the construction of the dam are available within the immediate vicinity of the dam. Rock is available at a borrow area approximately 3 miles from the dam site.

(c) In order to utilize the strata of impervious till which is underlain with rock on the east bank, the spillway location was moved downstream and the embankment was turned to meet it. This arrangement of the spillway and discharge [illegible] with [illegible] walls; the spillway discharge can be immediately directed back to the natural stream bed. This feature eliminates a sharp bend in the channel and further encroachment on the village of Bennington.

(d) A number of studies and estimates were made of the cost of cantilever walls for the stilling basin as compared with gravity walls and of a hollow type spillway section as compared with a gravity type. The estimates revealed that the cost of the gravity type structures was equal to or less than the thin slab type structures. Furthermore, in view of the severe climatic conditions to which these structures will be subjected, gravity type structures were considered the more suitable of the two types and were therefore selected. The impervious strata of till which goes down to bedrock in the spillway area provides an excellent foundation for the gravity type structures.

(e) Concrete gravity type abutment sections are provided on each end of the spillway. These sections extend into the embankment sections and are founded on the underlying impervious glacial till. A considerable saving is effected by the use of abutment sections in lieu of high retaining walls along each end of the spillway. A number of estimates were made to compare the costs for the various types of wall designs and abutment sections. The proposed design was selected as it is deemed the most satisfactory and the least costly of any of the designs considered.

(f) The stilling basin consists of a reinforced concrete mat with reinforced concrete baffles and end sill. The bottom of the mat is composed of a porous concrete layer one (1) foot thick. This feature reduces the uplift by allowing free drainage to the perforated collector pipes located under the mat.

3-04. TRANSPORTATION.- (a) Railroad.- A branch line of the Boston and Maine Railroad, originating at Nashua, New Hampshire, and running northwesterly through the villages of Greenfield, Hancock and Bennington and extending as far north as Hillsboro, passes through the proposed reservoir basin. Studies made by the Boston and Maine Railroad and the Boston District Office of a possible relocation of the existing railroad line at the damsite disclosed that relocation of this section of the railroad would be impractical and too costly. Consequently, it was decided to eliminate the line from Greenfield to Bennington and to rehabilitate the abandoned line between West Henniker and Hillsboro; in that way servicing the Town of Bennington from the Concord branch. It is anticipated that the rehabilitation of the [illegible] railroad and removal of the railroad passing through the [illegible] will be completed during the first construction season and the [illegible] of the embankment can be completed during the second construction season.

(b) Highways.- (1) One main first class highway, U. S. Highway Route No. 202, and several second and third class roads, which are located within the limits of the proposed reservoir, [illegible]

Plate A-1, will require either relocation or raising.

(2) Route No. 202 which is located along the westerly side of the river between Peterboro and Bennington is subject to inundation for a total length of approximately three (3) miles. It is proposed to relocate the affected portions of the road on higher ground with an elevation adquate to meet the requirements for the ultimate development.

(3) It is proposed to relocate the second class surfaced highway, which is now located on the easterly side of the river within the proposed dam site, onto high ground east of its present location. The main access road to the operating buildings of the dam will connect with the relocated road and provide access to the dam site from the larger villages in the vicinity. Other sections of this road which are subject to inundation during flood stages in the initial development will be raised to a minimum elevation of 708. When the ultimate development is constructed, these roads will be raised to elevation 715.0 or relocated, whichever is determined to be the most economical.

(4) Relocation of Second-Class Road.- For the initial stage it is proposed to raise the second-class road, located within the limits of the reservoir, to elevation 695.0 to provide access between the villages of Greenfield and Hancock during periods of high water other than extreme flood flows. It is also proposed to replace an existing bridge with one of higher elevation in order to obtain clearance for the extreme flood flows. For the ultimate stage it is proposed to relocate the road on higher ground downstream from its present location and to construct a new bridge.

(6) Communication Lines.- A trunk telephone line in the area will require relocation. Other communication and power lines serving only the reservoir basin will be removed as they will not be required after the dam is constructed.

(7) Conclusions.- The problem of relocation of railroads, roadways and utilities do not present any great difficulties. It is believed that the solutions discussed above are the most economical and feasible.

SECTION IV. HYDROLOGY

4-01. INTRODUCTION.- The purpose of this section is to present a resume of the hydrological studies made in order to develop the design of the Bennington Dam and Reservoir for both the initial and future developments. The scope of this section includes summaries of pertinent features, precipitation and run off, record floods, channel capacity, reservoir areas and capacities and spillway design flood. Detailed discussions of the hydrological features of the project are included in Appendix I - Hydology of the Definite Project Report on Bennington Reservoir, April 1945. Pursuance of the studies of the above features resulted in the determination of the required height of the dam for both the initial and ultimate development in order to provide protection against overtopping during the worst possible flood conditions.

4-02. PRECIPITATION AND RUN-OFF.- (a) Rainfall.- The average annual rainfall in the Contoocook River Basin above the Bennington Dam Site is 39.88 inches. This figure is the average figure obtained from records of six U. S. Weather Bureau Stations located in adjacent localities. These stations have records ranging from twelve years to sixty-six years in length. The records of the nearest well-equipped Weather Bureau Station, which is located at Concord, New Hampshire, show that over a period of seventy-four years the greatest 24-hour precipitation amounted to 5.97 inches. There is no significant seasonal variation in precipitation as the annual rainfall is fairly well distributed throughout all the months of the year. Generally, short-period rainfall intensities occur more frequently in the late spring and summer while excessive rainfalls of longer duration occur more frequently in the late summer and early fall.

(b) Run-off.- Daily flows have been estimated at the dam site for the period from September 1917 to the present time, by prorating the records of adjacent U. S. Geological Survey streamgaging stations to the 186 square mile drainage area of the dam site. Discharge hydrographs for the period from September 1917 to September 1944 are shown on Plates A-3 and A-4. The average yearly flow indicated by this record is approximately 300 c.f.s. The average annual flood for this record is estimated to be approximately 3,100 c.f.s.

4-03. FLOODS OF RECORD.- Utilizing observed flood stages at the dam site and comparative concurrent flow records of adjacent gaging stations, the instantaneous peaks of the floods of November 5, 1927, March 19, 1936 and September 21, 1938 were estimated to be 5,500, 13,600 and 15,400 c.f.s., respectively.

4-04. CHANNEL CAPACITY.- The channel capacity in the Contoocook River below the Bennington Dam site limits the maximum allowable discharge from the outlet conduits to approximately 4,000 c.f.s. This rate is determined from computations of the capacity of downstream dams and from field observation of actual flood conditions at the site. Due to

[illegible] below the damsite discharges [illegible] inundation of downstream [illegible] such flooding will not [illegible] property damage.

4-05. RESERVOIR.- Area and capacity curves for the reservoir for both initial and ultimate development are shown on Plate I-3 of Appendix I "Water Power" of the Feasibility Project Report.

(a) Initial Development.- (1) Conservation.- The existing conservation pool, created by the Powder Mill Dam, has a storage capacity of approximately 1,500 acre feet at elevation 678.2. This pool, which covers an area of approximately 300 acres and has a main valley length of about 3.2 miles and a maximum width of about 1/4 mile will be maintained for operation of the downstream mills in the town of Bennington, New Hampshire. The operating [illegible] and control equipment of the Powder Mill Dam will be removed during construction operations of the Bennington Dam and future conservation storage will be provided through [illegible] flow regulation by means of pick operation at Bennington Dam.

(2) Flood Control.- A net storage of 60,000 acre feet, which is equal to six (6) inches of runoff from the 186 square miles drainage area above the dam site, will be available for flood regulation between the conservation pool level of elevation 678.2 and the spillway lip elevation of 705.0. This storage will have the effect of reducing a flow equivalent to the March 1936 flood at the Bennington site from 15,600 c.f.s. to 1,600 c.f.s. during the original pooling period. This storage will also reduce a peak flow equivalent to the September 1938 flood from 15,400 c.f.s. to 1,600 c.f.s. during the original pooling period.

(b) Ultimate Development. - (1) Conservation.- Provision has been made in the design of the dam to permit the future raising of the spillway crest to elevation 712.0 in the event it is subsequently decided to develop the Peterboro Reservoir for multiple-purpose use. This elevation will provide 90,000 acre feet of storage, of which 40,000 acre feet will be allocated to conservation storage and the remainder, or 50,000 acre feet, will be retained for flood control. The maximum conservation pool level will be elevation 699.5. The ultimate pool surface will cover an area of approximately 3,300 acres and will have a main valley length of [illegible] miles and a maximum width of approximately 1.2 miles.

(2) Flood Control.- A net storage of 50,000 acre feet, which is equal to six (6) inches of runoff from the 186 square mile drainage area will be available for flood regulation between the ultimate conservation pool level of 699.5 and the ultimate spillway lip elevation of 712.0. Less flood control storage is required in the ultimate development because of the additional storage at West Peterboro Reservoir and since the 30-foot head provided on the outlets by the conservation pool results in a greater initial discharge.

4-06. RESERVOIR DESIGN FLOOD. - Two (2) severe floods of record occurring in March 1936 and September 1938 were used as reservoir design flood to check the storage, the design discharge and the proposed method

of the operation of the dam. The 1936 flood was practically a three-peak flood of thirteen (13) days duration and had a large run-off volume. The 1938 flood was a one-peak flood of six (6) days duration and had a higher discharge than the 1936 flood, but had less total volume. With the gates operated as discussed in Section V, the maximum stage resulting from routing the 1936 flood through the reservoir was elevation 706.0, and in the 1938 flood the maximum stage was 702.1.

4-07. SPILLWAY DESIGN FLOOD. - (a) Construction.- The spillway design flood was constructed on the basis of unit hydrographs and maximum possible precipitation. An inflow unit graph, computed from the reservoir inflow hydrograph of the September 1938 storm, was modified to represent an inflow graph applicable to rates of run-off expected during the high rainfall intensities. The modification was made with the use of empirical formulae developed by Franklin F. Snyder and treated in his article on "Synthetic Unit Graphs", which was published in the Transactions of the American Geophysical Union, Part 1, 1938. The maximum possible storm rainfall to be expected over this area as determined by the Hydro-meteorological Section of the Weather Bureau was 17.4 inches in a 24-hour period. After deducting the infiltration losses a run-off of 16.30 inches remained. With a base flow of 5 c.f.s. per square mile, the resulting spillway design flood hydrograph has a peak of 77,600 c.f.s.

(b) Routing.- In order to allow for all contingencies and also provide a safety factor commensurate with the economic magnitude of the project, it was decided to consider that all the flood control was utilized by previous run-off and that all outlets were rendered inoperative through conduit obstructions or mechanical gate failures. Thus the flood was routed through the spillway surcharge storage beginning at elevation 705.0, in the case of the initial development and at elevation 712.0, in the case of the ultimate development. Consideration was not given to the effect of proposed upstream reservoirs as the magnitude of this effect on the peak of the outflow hydrograph is practically negligible. The spillway design flood was routed through the 300 foot spillway surcharge storage in the initial development resulting in a peak outflow of 45,900 c.f.s. and a maximum pool elevation of 716.8. Routing the spillway design flood through the 300 foot spillway surcharge storage in the ultimate development gave a peak outflow of 42,200 at a maximum pool elevation of 723.1.

(c) Freeboard Requirements.- (1) Initial Development.- The wave height was computed by means of the Stevenson-Molitor formula:

$$H = .017\sqrt{VF} + 2.5 - \sqrt[4]{F}$$

Where H = wave height in feet
V = wind velocity in miles per hour = 80
F = fetch in statute miles = 5.5

Solving the above formula, the wave height equalled 4.53 feet. The wave rideup was computed as 1.4 times H, the total height of wave from crest to trough. The formula of the Lorentz Zuderzee Commission of Holland, published in 1923 and stated below, was used in computing the wind set-up.

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

where

s = wind setup in feet

V = velocity of wind in miles per hour = 80

F = fetch in statute miles = 5.5

D = depth in feet = 40

A = angle at wind fetch = 0

Solving the above formula, the wind set-up equalled 1.1 feet. The freeboard required for the spillway design flood in the initial development was computed to be 7.5 feet by modifying the wave height by the wave rideup factor and adding it to the wind set-up. The top elevation of the dam was then determined as follows:

Elevation crest of spillway	705.0
Maximum head on spillway	11.8
Freeboard requirement	7.5
	724.3
Adopted elevation for top of dam	724.0

(2) Ultimate Development.- The same formulae were used in computing the freeboard for the ultimate development as were used in computing the freeboard for the initial development. The only difference introduced in the computation was in the value of the depth, "D", which increased by seven (7) feet. This decreased the wind setup by 0.2 of a foot, giving a figure of 7.3 feet for the required freeboard. The top elevation of the dam in the ultimate development was determined as follows:

Elevation of crest of spillway	712.0
Maximum head on spillway	11.1
Freeboard requirement	7.3
	730.4
Adopted elevation for top of dam	730.0

(3) Adopted Elevations. - The adopted elevations for the top of dam have been determined by rounding off the computed elevations to the nearest lower foot. This is considered justifiable in view of the extreme improbability of all the adverse criteria occurring simultaneously, i.e., maximum possible storm occurring with flood control reservoir full and all six outlets inoperative followed by a gale of hurricane proportions striking the maximum pool level at the exact angle of maximum fetch at the instant of maximum outflow. The adopted elevations in effect establish the freeboard of the initial development as 7.2 feet and of the ultimate development as 6.9 feet.

4-08. COFFERDAMS DESIGN FLOOD. - The diversion of the river during construction will be accomplished by means of a cofferdam downstream from

Powder Mill Dam. Inspection of a composite hydrograph indicated that storm discharges in excess of 5,000 c.f.s. at Bennington occurred only four times in the period 1917 to 1944 and two of these floods occurred during early spring run-offs. The routing of two of these storms (6 April 1923 and 24 June 1944) through the six open conduits of the dam gave pool elevations of 681.0 and 683.8 respectively. On this basis the cofferdam elevation was established at 685.0, allowing 1.2 feet of freeboard for the greater storm. This height is considered adequate to prevent overtopping, considering the severity of this particular flood. The top of the downstream cofferdam was established at elevation 670.0 for a discharge through the six conduits of 3,300 c.f.s. This height provides a freeboard of 1.0 feet if the flashboards are not removed from the downstream Monadnock Power Dam and 2.9 feet of freeboard if the flashboards are removed or fail.

5-01. INTRODUCTION.- The purpose of this section is to present the various hydraulic criteria for the design of the dam and its pertinent structures and to indicate the formulae and coefficients employed and assumptions made in the computations. The scope of this section includes discussions on the basis of design, criteria for the initial and ultimate developments, conduit structures, reservoir conditions, gating facilities, spillway and stilling basin features, and cofferdams. A bibliography of the reference books referred to in the design and computations is given at the end of this section.

5-02. GENERAL.- (a) Basis of Design.- The design of the dydraulic features of the Bennington Dam are based on theoretical analysis supplemented by results of hydraulic model studies for similar structures under comparable conditions. Established precedent has been followed in all instances where possible. The design considers all contingencies and is as conservative as possible within reasonable economic limits. A bibliography at the end of this section lists references and special publications used in connection with the design.

(b) Influence of Site Conditions.- The design of the conduits and overflow section is influenced by the location of the dam and prior use of the river. The flood control dam is to be located about one-half mile south of the Town of Bennington, New Hampshire, and about 600 feet downstream from the Powder Mill Dam of the Monadnock Paper Company. This dam is used at present to maintain a pool for storage and stream regulation. The crest of its spillway is elevation 675.0 with flashboards maintained at elevation 678.2. The control equipment of this small dam is to be removed upon completion of the flood control dam and the regulatory functions of the Powder Mill Dam will be taken over by the flood control structure. About 2,700 feet downstream from the proposed location of the flood control dam is a small dam and power generating station of the Monadnock Paper Company, referred to herein as the Monadnock Power Dam. This dam forms the tailwater control for the proposed flood control structure. The crest of its spillway is elevation 663.5 with flashboards maintained to elevation 665.5.

(c) Criteria for Initial Development.- The criteria for initial hydraulic requirements are as follows: (1) The present conservation pool at Elevation 678.2 shall be maintained and regulation of normal river flows equivalent to the present regulation by means of the Powder Mill Dam shall be obtained.

(2) In the event the river inflow exceeds the downstream channel capacity, the reservoir outflow shall be limited to the channel capacity, and all excess shall be stored in the flood control pool. The outflow regulation shall be accomplished by gate operation.

(3) The spillway shall safely pass a flood discharge resulting from the most severe storm meteorologically possible in this area, the storm being accompanied by winds of hurricane force during the maximum reservoir stage.

(d) Criteria for Ultimate Development.- The criteria for ultimate development of the overflow section are as follows:

(1) A conservation pool at Elevation 699.5 shall be maintained for downstream flow regulation to provide optimum benefit to downstream water users.

(2) Flood regulation and passage of severe floods over the spillway shall be accomplished as specified for the initial scheme.

5-03. OUTLETS.- (a) Hydraulic Criteria.- The criteria governing the hydraulic design of the outlets are as follows:

(1) The outlets shall readily discharge normal daily flows with a minimum of gate operation.

(2) The outlets shall discharge, without partial gate openings, approximately 4,000 c.f.s., with reservoir stage at initial and ultimate spillway crests for the control of flows during flood periods and for emptying the reservoir following floods.

(3) The design of the outlets shall provide flexibility for possible changes in operating requirements.

(4) The outlets shall distribute the discharge over a considerable width of the stilling basin in order to assist in the formation of the hydraulic jump and to dissipate the energy in the high velocity jets.

(5) The outlet design shall provide additional conduits for emergency use in case other conduits become inoperative and for use in shortening the emptying period following a full reservoir.

(b) Design of the Conduits.- (1) General Layout.- Studies made in designing the conduits indicated that six rectangular conduits, four feet in width and six feet high, spaced uniformly across the face of the spillway, will meet all the above criteria. These conduits are placed in the spillway overflow section in order to utilize a common stilling basin layout which effects a definite economy in construction. The use of a large number of relatively small conduits distributed across the spillway was adopted to spread the conduit discharge uniformly across the floor of the stilling basin and to permit greater flexibility in operation. Each conduit consists of a bell mouth entrance; a gate chamber containing hydraulically operated emergency gates; a straight tunnel section; and a portal. The transaction between the tunnel and the portal consists of a vertical downward curve which gradually converges to the portal. The portal has flaring sidewalls and a depressed roof, and its area is

about 94 per cent of the conduit area. The vertical curve is utilized to make the flow discharge tangent to the toe of the spillway and to prevent the jet from leaving the apron. The flaring portal walls and depressed roof is utilized to produce an initial spread to the jet which will tend to increase as the discharge enters the stilling basin. The slight contraction at the portal is utilized to assure that the hydraulic control will remain at the portal for full conduit discharges and to prevent the occurrence of slight negative pressures near the roof.

(2) Invert Elevation.- The inverts of the conduits are elevation 667.0 in order to satisfy reservoir and tailwater grades. This elevation completely submerges the mouth of the conduit below the level of the conservation pool, and yet maintains the portal invert above the tailwater stage. This feature is considered desirable as it eliminates difficulties that may result from ice conditions which are objectionable in this latitude. The selected elevation allows complete drawdown of the conservation pool and also provides a short drop from the outlet portals to the tailwater. This feature is advantageous as it aids in spreading the jet before it submerges into the stilling basin.

(c) Construction of Rating Curve.-The discharge rating curve shown on Plate V-3 was computed for a single conduit as set forth in the following paragraphs.

(1) Low Reservoir Stages.- The lower portion of this curve, up to a discharge of approximately 340 c.f.s., was computed with the assumption that the hydraulic control was located at the end of the bell mouth entrance section. The discharge for flow at the critical depth was computed from the formula $Q = A\sqrt{\frac{gA}{W}}$ in which

Q = discharge in c.f.s.
A = area of the wetted cross section
g = acceleration of gravity
W = surface width of wetted cross section

Values of the constants "A" and "W" were determined for assumed depths and the discharge computed. The reservoir elevation corresponding to the computed discharge was determined by adding to the assumed water depth, the velocity head for average velocity of flow with minimum energy and an additional one-tenth of the velocity head to allow for minor upstream friction and turbulence losses. The average velocity of flow with minimum energy was determined from the computed discharge.

(2) High Reservoir Stages.- The upper portion of the discharge rating curve was computed with the assumption that the hydraulic control was located at the portal restriction. In this section of the curve, the conduit will flow full and the discharge will act as a supported jet flaring over the apron into the stilling basin. The

discharge is computed as the product of the area of the portal and the mean velocity at right angles to this portal section. To determine the amount of effective head available to produce flow for the latter condition, the losses in the conduit were estimated in terms of portal velocity head. Friction losses were estimated from the Chezy formula with the "C" coefficient computed by means of the Kutter formula with an "n" coefficient of 0.013 for the concrete and steel lining. Losses caused by curvature in the conduit were computed by applying a percentage of the 90° bend loss for a bend ratio (radius of bend to diameter of pipe) of approximately ten, as discussed in subparagraph (3) below. Other losses such as entrance, gate slot and convergence were computed by conventional methods. The coefficient "k" of estimated losses (Loss = $k \frac{v^2}{2g}$) in terms of velocity head in the portal section are listed below:

Entrance	.044
Entrance (friction)	.002
Gate Slots	.044
Conduit passage (friction)	.084
Portal transition (friction)	.046
Bend loss	.113
Contraction loss	.020
Portal velocity head	1.000
	1.353

(3) Discussion of Bend Losses.- A study was made to evaluate the head loss caused by the introduction of the vertical curve in the conduit at the downstream end. Most of the empirical data on the subject has been obtained from laboratory tests on relatively short radii pipe fittings about six inches in diameter. The data indicates that a minimum amount of turbulence exists when the ratio of the diameter of the conduit to the radii of the bend is approximately five; this phenomenon is attributed to the synchronization of the turbulence waves formed in the upstream area of the bend with those formed in the downstream area. This conclusion is based on data obtained from velocity traverses not only in the region of the bend itself but also some distance below the bend. Since tests indicate that an appreciable part of bend losses occur in the straight section of the conduit immediately downstream from the bend and since the jet in this instance leaves the confines of the conduit immediately downstream from the bend, the use of data based on such tests is probably conservative. A qualitative study of bend losses was made during the model studies of the power tunnel at Fort Peck Dam but because of the varying approach conditions no relationship was established between the head loss and the degree of curvature of the bend. It was, therefore, decided to use the method described on pages 118 to 120 of Creager & Justin's "Hydro-Electric Handbook" as an acceptable conventional method of computing the bend loss although the limitations mentioned above still apply. The coefficient "K" of losses in terms of velocity head (Loss = $k \frac{v^2}{2g}$) was determined by use of charts published in the above noted handbook. The ratio of the radius of the bend to the effective

diameter of the conduit is approximately ten. Referring to Figure 67, Page 119, Creager and Justin's handbook, a bend loss coefficient of 0.42 for a 90° bend was observed. Inasmuch as the actual bend under discussion is approximately 13-1/2 degrees, the loss coefficient 0.42 is reduced by a factor of 0.27 (Figure 68, page 120, Creager and Justin) to 0.113.

(d) Design Features.- (1) Bell Mouth Intake.- In designing the curves of the bell mouth intake of the conduit, reference was made to the "Report of Model Tests of Conduit Intakes", U. S. Engineer Office, Providence, R. I. These tests were of a qualitative nature designed primarily to determine the best intake shape to eliminate negative pressures. Since the maximum conduit velocities at Bennington will be approximately 40 ft. per sec., this factor is of importance. It was also considered desirable to shorten the bell mouth entrance section as much as possible (8 ft. over-all) to permit the use of a reasonably long tangent upstream from the first gate slot, thereby obtaining flow approximately laminar and diminishing the effect of entrance turbulence at the gate slot. An elliptical-shaped entrance profile was indicated as well adapted for this particular structure. The roof of the entrance portal was formed in the shape of an ellipse whose major axis was twice the minor axis. The side walls were similar except that the major axis was made 1.25 times the minor axis. The floor of the entrance section presented a slightly different problem since the floor of the approach channel is only 1.5 feet lower than the sill of the entrance section. A flat parabolic curve was selected for this transition. A comparison of this section with the sections shown for the sluicing conduits of the John Martin Dam, Arkansas River* on Plate 25 of the model study for that project indicates that this design is relatively conservative. A similar comparison made with the entrance section for the flood control sluices of the Wolf Creek Dam* shown on Plates 6 and 8 of the model study report therefor confirms the adequacy of the design.

(2) Gates and Gate Passages.- Two gates, an upstream emergency gate and a service gate, are provided in each conduit. The gates are 4' x 6' in size and are hydraulically operated. A 16 ft. 8 in. length of metallic conduit liner is provided to protect the concrete against possible cavitation due to turbulence originating at the gate slots. The conduit liner begins at a point 1 foot 8 inches upstream from the emergency gate and extends well beyond the service gate section. Vents are provided for the service gates to prevent the formation of negative pressures during partial gate opening. It was not considered necessary to provide vents for the emergency gates because of the infrequency of their operation.

(e) Proposed Operation.- Normal operation of the outlets is based on the maintenance of a conservation pool at Elevation 678.2 which is the same elevation as the pond at the existing Powder Mill Dam located just upstream from the proposed dam. The Powder Mill Dam, which provides flow regulation for the three dams in the Town of Bennington, New Hampshire, will be abandoned upon completion of the flood control dam. Normally,

*See Bibliography on page 5-11.

all gates in the proposed Bennington Dam will be closed except for regulating the required daily flow for the Bennington Power dams. Should a storm occur, on the Bennington Reservoir drainage area and a freshet or flood flow is produced thus causing the reservoir level to rise above 678.2, the gates will be opened in the following sequence:

Reservoir Stage	Gates Open	Storage in Acre Feet	Total Discharge in C.F.S.
678.2	1	0	400
680.0	2	1,000	920
682.0	3	3,000	1,530
684.0	4	5,000	2,240

The two additional conduits will assist in emptying the reservoir following a flood storage accumulation period and will also provide additional gating facilities in the event there is any mechanical failure of any of the other gates. Instructions in the Operating Manual will specify that alternate gates be opened, until the fourth gate is opened, to distribute the discharge over the stilling basin. In the initial development, with the reservoir stage at spillway crest elevation 705.0, the discharge capacity of four conduits is approximately 3,630 c.f.s. In the ultimate development, with reservoir stage at the spillway lip elevation 712.0, the discharge capacity is approximately 3,960 c.f.s. The maximum channel capacity in the river immediately below the dam site is approximately 4,000 c.f.s.

5-04. SPILLWAY.- (a) Hydraulic Requirements for Spillway.-(1) Initial Development.- The spillway is designed to safely discharge a peak flow of 45,900 c.f.s. from the maximum pool elevation of 716.8 to the floor of the stilling basin, Elevation 656.0 without developing negative pressures on the underside of the nappe at any point. A smooth transition spillway section was designed to avoid impact and fatigue stresses in the structure. The spillway crest is at Elevation 705.0.

(2) Ultimate Development.- The spillway is designed to safely discharge a peak flow of 42,200 c.f.s. from the maximum pool elevation of 723.2 to the floor of the stilling basin at Elevation 656.0 without developing appreciable negative pressures on the underside of the nappe at any point and making the transition smoothly so as to avoid impact or fatigue stresses in the structure. The spillway crest is at Elevation 712.0. The initial and ultimate spillways are designed so that the lower spillway can be raised economically and with the least amount of modifications to the basic structure.

(b) Discharge Capacity.- The spillway rating curve was computed by the weir formula:

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

Where Q = the discharge in c.f.s.
C = the coefficient of discharge
L = the spillway length in feet
H = the head in feet on the spillway

The effect of the velocity of approach was negligible and was omitted in the computations. A curve of "C" values plotted as abscissae against "H"

values as ordinates was constructed beginning with 3.0 at zero head and gradually approaching 3.8 at a design head of 10.4 feet. Although higher coefficients have been obtained in model studies, it was considered that 3.8 is a more conservative value for design purposes. In preparing this curve, reference was made to similar graphs shown on page 133 of Creager & Justin's "Hydro-Electric Handbook" and to graphs on page 341 of C. V. Davis' "Handbook of Applied Hydraulics." Reference was also made to U.S.G.S. Water Supply Paper #200. A study of spillway length with respect to the total cost of the structure indicated that a 300-foot spillway was the most economical length. This length is also desirable because it will distribute the discharge in the stilling basin over a large area, thereby lowering the total hydrostatic head on the impervious glacial till foundation. From this basic data, the rating curves shown on Plate V-1 were computed.

(c) Shape of Spillway Nappe.-(1) Initial Development.- The equation of the overfall section of the spillway was computed by means of the following exponential equation recommended in Circular Letter No. 3281 dated September 1944:

$$x^{1.85} = 2H_c^{0.85}\, y$$

where x = horizontal distance from the crest line to the point
Y = vertical distance from the spillway to the point
H_c = design head on ogee crest line

The nappe of the spillway was based on this formula from the crest at Elevation 705.0 to Elevation 682.0, the selected point of reverse curvature. From this point to the floor of the stilling basin, the transition is made by means of a circular curve with a radius of 52.39 feet. The upstream nappe of the spillway was also computed as recommended in the above circular letter, as follows:

Radius = 0.5 H_c to point 0.175 H_c from crest line
Radius = 0.2 H_c to point 0.282 H_c from crest line

The results of these computations are shown graphically in Plate V-1.

(2) Ultimate Development.- In order to fit an exponential curve from the ultimate spillway lip at Elevation 712.0 tangent to the initial spillway face it was necessary to deviate slightly from the exponential function mentioned in the previous paragraph. A very close approximation to the theoretical curve was obtained by shifting the ultimate crest line upstream seven feet and computing an exponential curve which meets the initial spillway face in a point of common tangency, which was selected to conform to a construction joint at Elevation 694.78. The equation of this curve becomes $X^{1.578} = 8.182Y$ with the point of origin at the new spillway crest line. The question of slight negative pressures that may tend to develop due to the difference between the exact exponential curve and the adapted is purely academic in view of the extreme improbability of

the occurrence of such a storm run-off and the very short interval of time that the maximum head can be sustained. Check computations were made to show the depth of the nappe for the maximum flood discharge over both the initial and the ultimate spillway surfaces. The empirical values (listed on pg. 209 of Creager & Justin's Handbook) of the upper and lower nappes for a head of one foot on the crest of the spillway were multiplied by the respective design heads of 11 and 12 feet for the ultimate and the initial spillways. The results of these computations are shown graphically on Plate V-1.

5-05. HYDRAULIC REQUIREMENTS FOR STILLING BASIN.- (a) Capacities (1) Initial Development.- The stilling basin is designed to dissipate the energy of the spillway discharges for all ranges in flow up to and including the spillway design discharge of 45,900 c.f.s. for the initial development. The dissipation of the kinetic energy will be accomplished by means of a hydraulic jump within the stilling basin for all possible ranges in tailwater conditions. The stilling basin will reduce discharge velocities to prevent downstream scour and erosion.

(2) Ultimate Development.- In the ultimate development the spillway basin is designed to meet the above requirements for an upper limiting condition of 42,200 c.f.s. discharging over the spillway crest at Elevation 712.0

(b) Tailwater Discharge Rating Curve.- The first step in the design of the stilling basin was the determination of tailwater rating curve for Bennington Dam covering a discharge range up to approximately 50,000 c.f.s. This curve was based on backwater computations from the Monadnock Power Company dam which is located approximately 1/2 mile downstream from the Bennington stilling basin. The basic curve was computed with the assumption that the Monadnock Power Company dam remains intact as the control for the complete range of discharges and that the flashboards have been either removed or destroyed. Inasmuch as the Monadnock Power Company dam is constructed of concrete on a rock outcrop in a narrow section of the valley, this assumption is quite reasonable. The Monadnock Power Company dam has a spillway length of 172 feet and a crest elevation of 663.5. The pool elevation is normally raised by means of flashboards to elevation 665.5. The abutment walls were raised following the 1938 flood to Elevation 673.2. The amount of water used by a relatively small turbine in the generating station has been neglected in this study. The total computed discharge capacity of the spillway, assuming no abutment overtopping, is 18,000 c.f.s. which is in excess of the maximum flow of record. In order to obtain the limiting range of tailwater conditions obtainable at the Bennington Dam site, a second series of backwater computations were made for discharges assuming that both dam and abutments of the Monadnock Power Company dam failed completely and that the hydraulic control remained at the same point. The latter assumption is tenable because of the rock at the site and the rapid drop in the channel downstream from the Power Dam. The results of the computations are illustrated graphically on Plate V-2.

(c) Depth Required for Hydraulic Jump.-(1) Initial Development.- The tailwater depth required to dissipate the energy in the 45,900 c.f.s.

falling from the spillway crest (Elevation 705.0) to the lip of the spillway bucket (Elevation 656.) is computed by means of the following hydraulic jump equations:

$$D_2 = -\frac{D_1}{2} + \sqrt{\frac{2V_1^2 D_1}{g} + \frac{D_1^2}{4}}$$

where D_1 = depth before jump

D_2 = depth after jump

V_1 = average velocity in the jet before the jump

g = acceleration due to gravity

It was found from a preliminary study made of theoretical tailwater required in the intial and ultimate developments that the initial development with its greater spillway discharge of 45,900 c.f.s. furnished one limiting condition. For simplicity and over-all economy of design, the spillway crest width of 300 feet was maintained as far as the stilling basin end-sill. By means of a series of reasonable approximations, a stilling basin floor elevation of 656.0 was found to satisfy all limiting conditions when the modification discussed below was applied. The discharge per foot of spillway width at the peak of the spillway design flood in the initial development was computed at 153.c.f.s. The velocity and depth of the horizontal jet leaving the spillway bucket and entering the stilling basin at this elevation was computed at 61.3 feet per second and 2.50 feet respectively. Entering these figures into the above formula, the necessary depth of tailwater for the occurrence of a free-forming jump was computed at 22.95 feet. Model and prototype studies indicate that the hydraulic jump will form at 85 - 90% of the theoretical depth when baffle piers and end sills are incorporated in the stilling basin design.* Since the tailwater elevation corresponding to the discharge of 45,900 c.f.s. for the most unfavorable condition, namely, with the Monadnock Power Dam removed completely, would produce a tailwater elevation of 676.0 and since this provides a tailwater depth of 20 feet or 87% of the theoretical depth needed, the design is considered adequate.

(2) Ultimate Development.- In the case of the ultimate development, the velocity and depth of the jet leaving the spillway bucket and entering the stilling basin were computed at 64.5 ft. per sec. and 2.1[illegible] feet respectively. The theoretical tailwater depth needed for this condition, computed by the above formula, is 22.[illegible] feet. Since the tailwater elevation corresponding to a discharge of 42,200 c.f.s. for the most unfavorable condition, namely, with the Monadnock Power Dam completely destroyed, would produce a tailwater elevation of 675.4 and since this provides a tailwater depth of 19.4 feet or 85% of the theoretical depth needed, the design is considered adequate. It should be noted that complete destruction of the Monadnock Power Dam is very improbable, and that the tailwater will likely be above this extreme condition.

* See Bibliography

(d) Length of Stilling Basin.- The length of the stilling basin has been designed somewhat greater than ordinarily recommended when based on a constant multiplied by the depth of flow after the jump, or when the length and depth are related to the Froude number. The length of the stilling basin, which is 104 feet, is slightly more than four (4) times the depth of flow. The additional length the stilling basin is desired because of (1) the erodable nature of the glacial till foundation material, (2) the fact that formation of an hydraulic jump during conduit discharges cannot be mathematically verified, and (3) the alignment of the spillway with respect to the embankment section and the downstream terrace makes it expedient to be conservative in the design criteria. The selected length of the basin assures that all turbulence and wave action will be leveled out before the water discharges into the natural stream bed.

(e) Baffles and Sill. Two rows of baffles, 5 feet high, were incorporated in the stilling basin design as well as an end sill 5 feet high to deflect any high velocity currents from the channel floor. The spacing, size, and relative position of the baffle piers and end sill conform with recommendations made by the U.S. Waterways Experiment Station, Vicksburg, Mississippi, whose comments were requested while the design of the structure was under consideration.

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Weir Experiemnts, Coefficients and Formulas	U.S.G.S. Water Supply Paper #200

Model Study Reports from the U. S. Waterways Experiment Station:

Hydraulic Characteristics of Power Tunnel, Fort Peck Dam

Spillway and Regulating Sluices for Wolf Creek Dam

Spillway and Stilling Basin for John Martin Dam

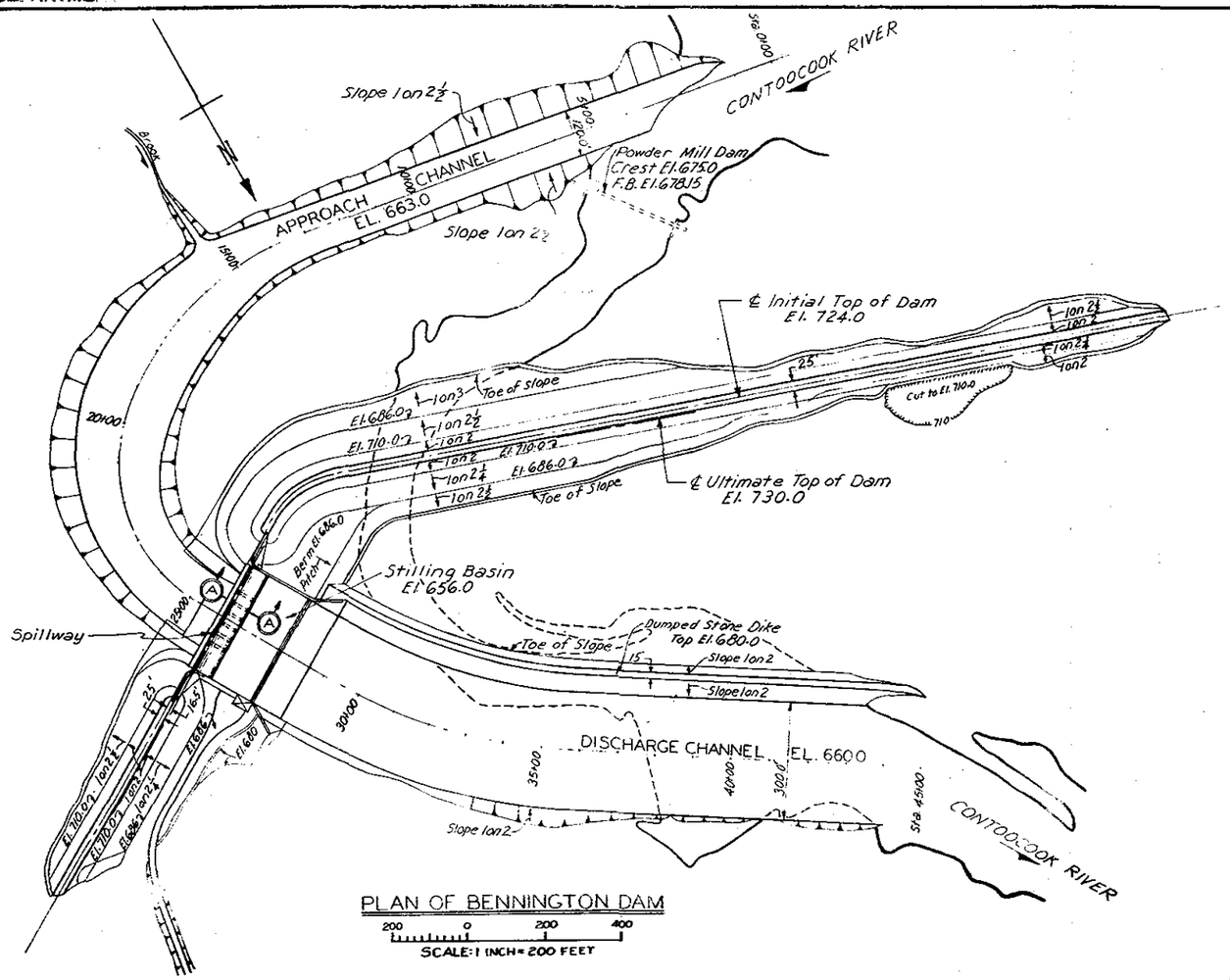
The SAF Stilling Basin

Spillway and Stilling Basin, Wappapello Dam

Spillway and Stilling Basin, Canton Dam

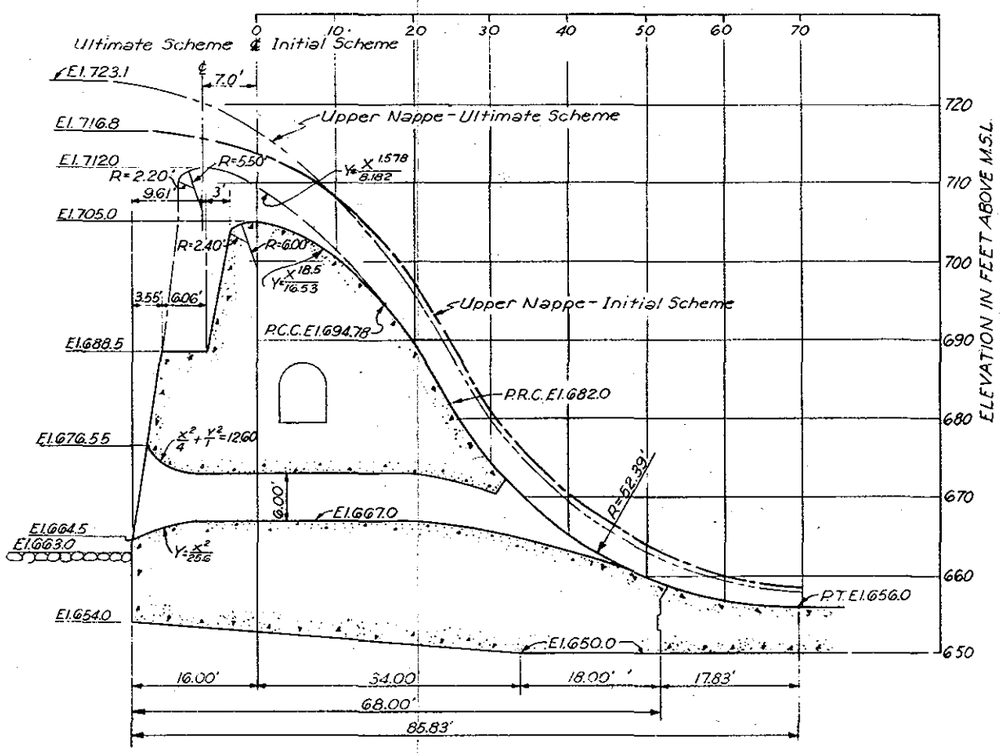
Spillway and Stilling Basin, Loyalhanna Dam

Spillway and Stilling Basin, Dale Hollow Dam



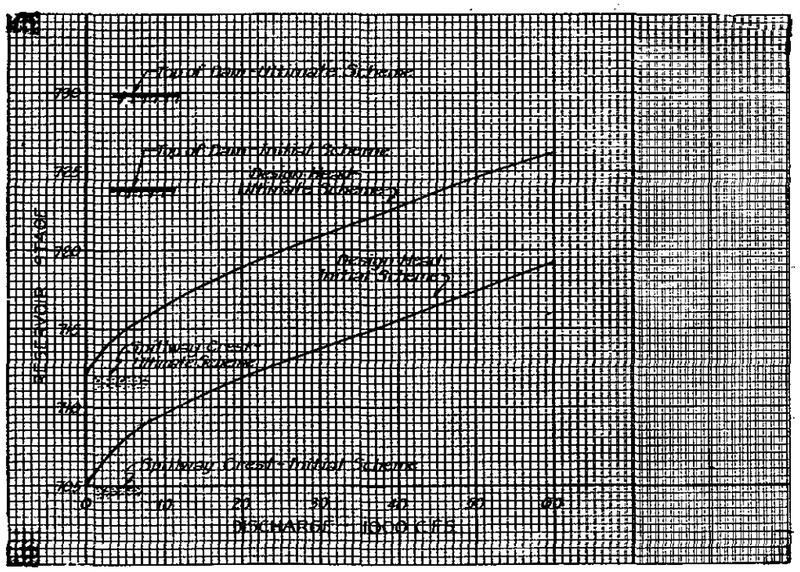
PLAN OF BENNINGTON DAM

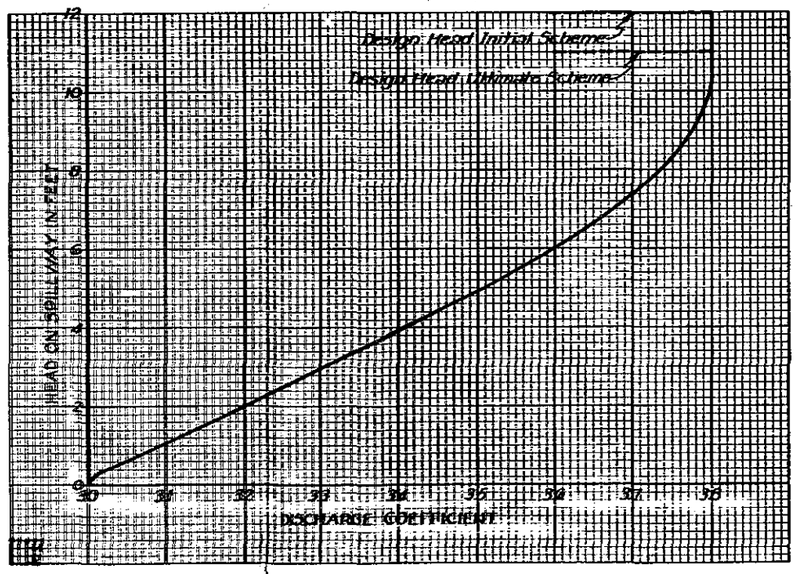
SCALE: 1 INCH = 200 FEET



SECTION A-A

SCALE: 1 INCH = 10 FEET



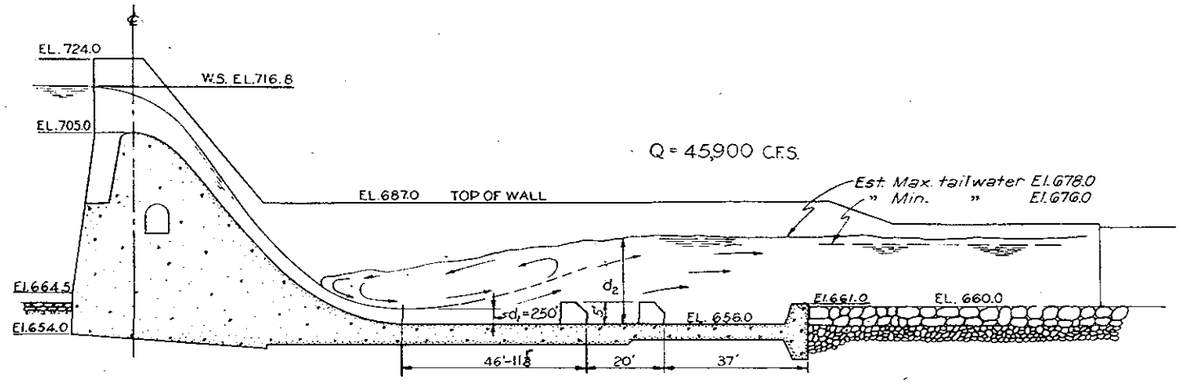


Note:
For further dimensional details refer to Contract Drawings.
For explanation of computations involved refer to text.

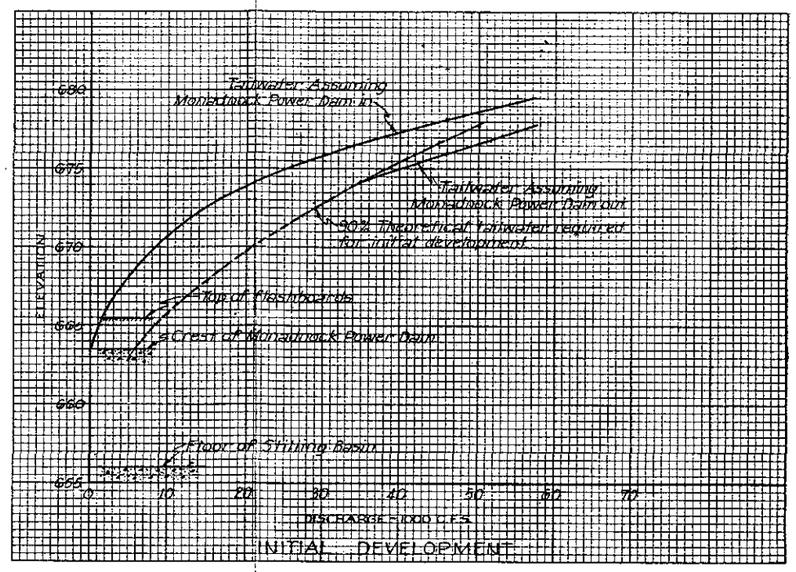
MERRIMACK VALLEY FLOOD CONTROL

BENNINGTON DAM
CONTOOCOOK RIVER

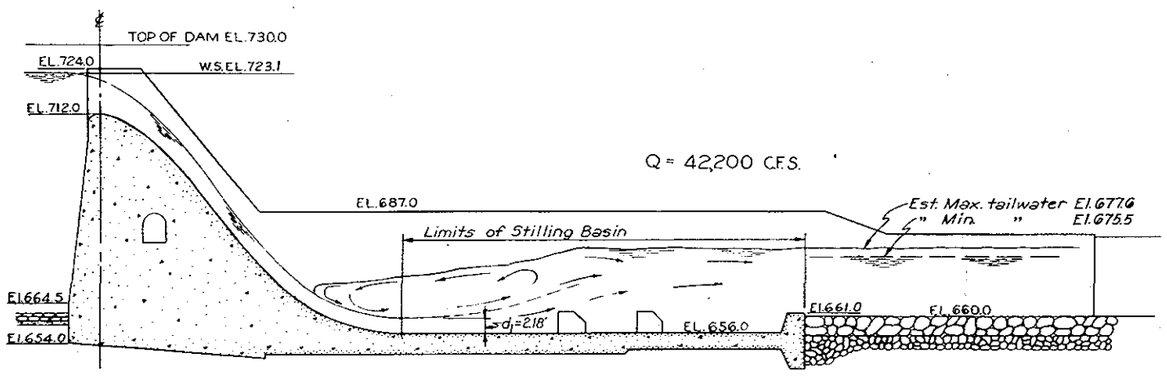
SPILLWAY STUDIES



SECTION THRU ℄ OF STILLING BASIN (INITIAL WEIR)

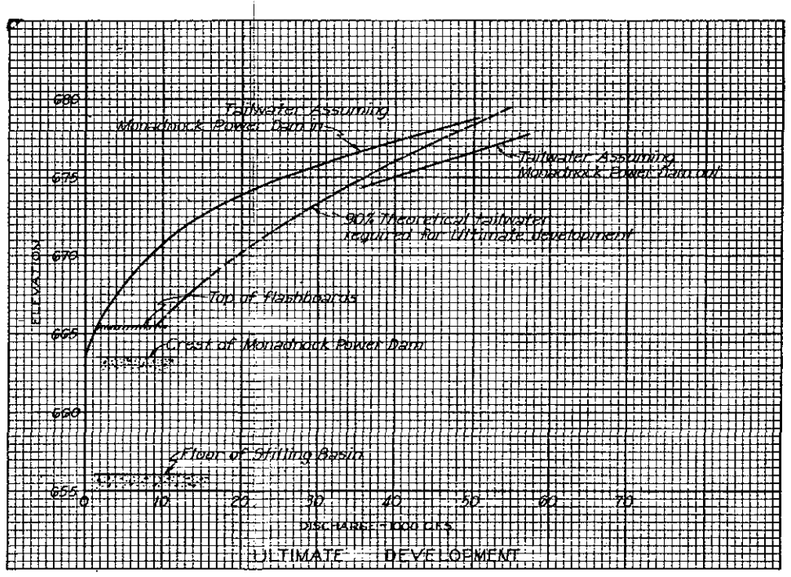


INITIAL DEVELOPMENT



SECTION THRU ℄ OF STILLING BASIN (ULTIMATE WEIR)

SCALE: 1" = 20'



ULTIMATE DEVELOPMENT

FORMULA FOR HYDRAULIC JUMP $d_2 = -\frac{d_1}{2} + \sqrt{\frac{2 v_1^2 d_1}{g} + \frac{d_1^2}{4}}$

d_1 = Depth above hydraulic jump (in feet)

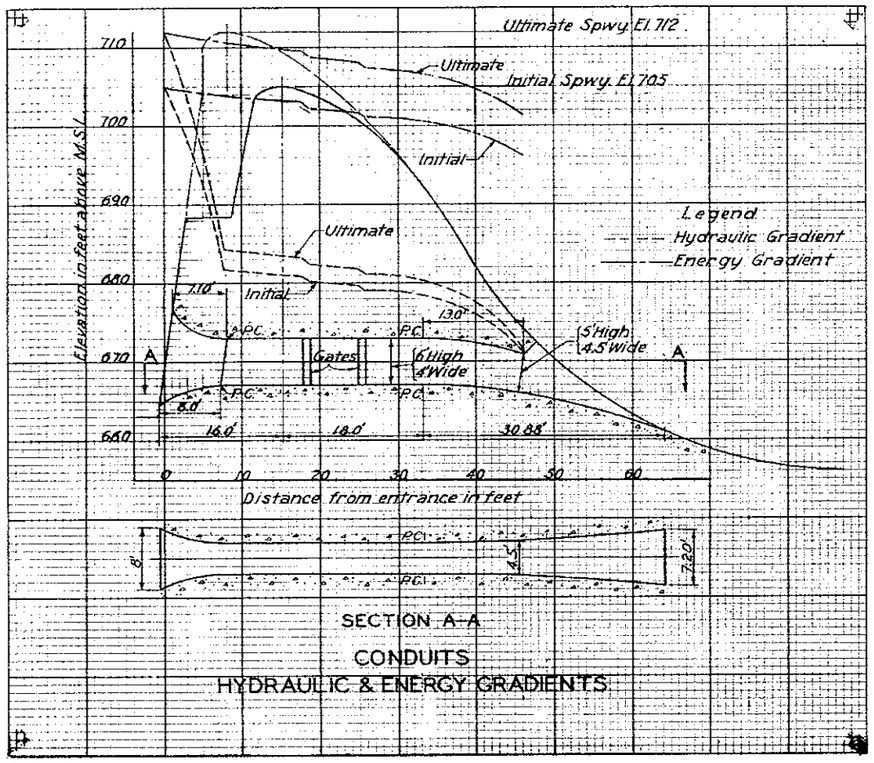
d_2 = Depth below " " "

g = Acceleration due to gravity

Note:
For further dimensional details refer to Contract Drawings ~~Nos.~~
For explanation of computations involved refer to text.

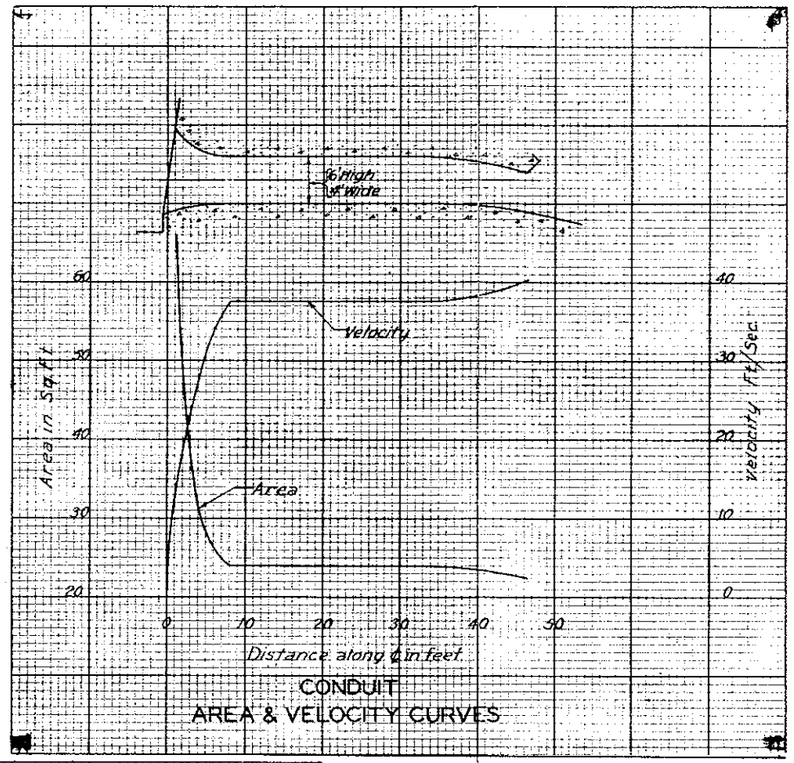
MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON DAM
CONTOOCOOK RIVER

STILLING BASIN STUDIES

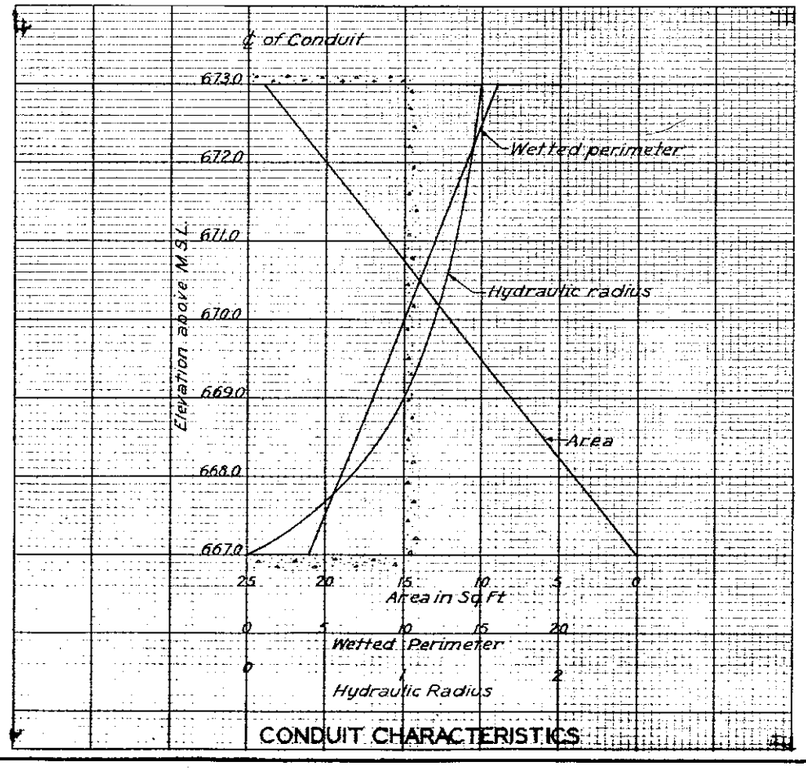


SECTION A-A

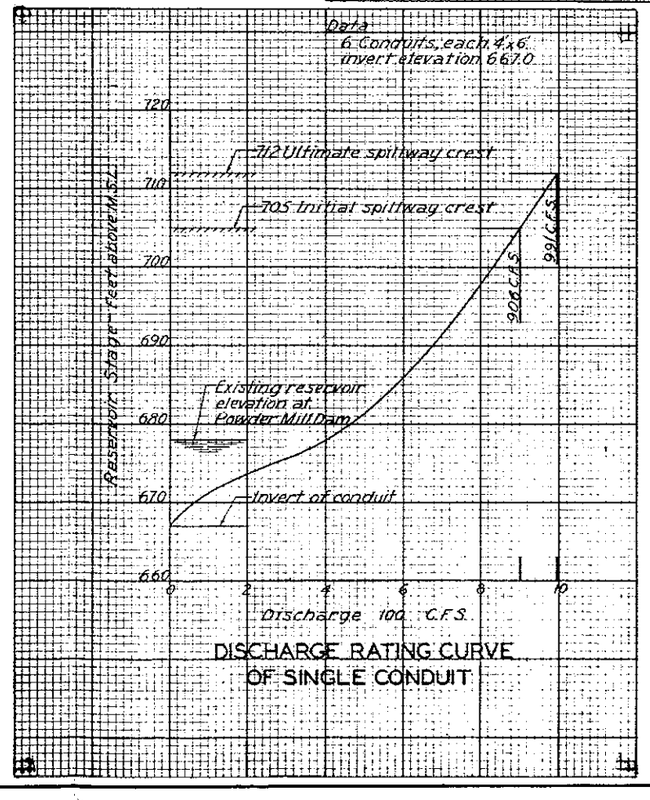
CONDUITS
HYDRAULIC & ENERGY GRADIENTS



CONDUIT
AREA & VELOCITY CURVES



CONDUIT CHARACTERISTICS



DISCHARGE RATING CURVE
OF SINGLE CONDUIT

Note:
For further dimensional details refer to Contract Drawings.
For explanation of computations involved refer to text.

MERRIMACK VALLEY FLOOD CONTROL

BENNINGTON DAM
CONTOOCOOK RIVER

OUTLET STUDIES

PLATE V-

SECTION VI

GEOLOGICAL AND SOILS INVESTIGATIONS

6-01. INTRODUCTION.-Geological and soils investigations were made to determine subsurface conditions at the dam site, characteristics of soils encountered and availability of the construction materials. Methods employed included reconnaissance of the site and area, seismic exploration for location of bedrock, drilling in overburden and rock, excavation and sampling of test pits and trenches, special pressure and pumping tests, and a thorough laboratory testing program. The location, extent and results of the exploration at the site are shown on Plates. A-5 to A-13 inclusive. Complete reports of drill holes, test pits and seismic investigations are on file in the Boston District Office. All soil samples and rock cores are stored in the Boston District Soils Laboratory.

6-02. GENERAL TOPOGRAPHY AND GEOLOGY OF AREA.-(a) Contoocook Valley. The rocks of the Contoocook Valley are metamorphic and igneous of the Paleozoic age. The metamorphic rocks are schists and gneisses which were deposited originally as flat beds below the water surface. A period of diastrophism which occurred subsequent to this deposition resulted in the crumbling and folding of the layers until they appeared at every attitude. Large amounts of magma were injected into the metamorphosed rocks, where relatively slow cooling occurred and granite or related granitic rocks formed. A period of regional uplift followed and streams began to gradually erode the overlying metamorphic rocks. As the drainage pattern developed, the streams cut downward through the metamorphic rocks and exposed large areas of granite. The granite was not a durable rock and has since been affected in local areas by weathering to depths as great as 50 feet. At the beginning of the Pleistocene period the Contoocook Valley had been developed, and the river was flowing northward in a bedrock channel. During the Pleistocene period, the valley was covered by a portion of the continental glacier that occupied Canada and the northern part of the United States. The glacier modified existing topography by erosion and deposition. Erosion by the glacier consisted of bevelling the northern slopes of hills and steepening their southern slopes. In addition, it scoured and widened north-south valleys and tended to deepen and widen saddles transverse to its line of motion. Deposition by the glacier caused the greatest change in topography. Glacial deposits are of two types: the unsorted, unstratified glacial till deposited directly from the ice, and the sorted gravels, sands, silts and clays that are washed from the ice and deposited in water. Glacial till may consist of a heterogeneous mixture of clay, silt, sand, gravel, cobbles and boulders; or some of these sizes may be lacking. In general, the till of the Contoocook Valley does not have material as fine as the clay sizes in any appreciable amount. Most of the till may be broken down into two classes: (1) sandy till which has a predominance of sand sizes; and (2) silty till which has a predominance of silt sizes. Glacial till is highly stable, capable of furnishing good support, and is practically impervious. It was deposited in elliptical shaped hills known as drumlins, and as blankets which were spread across the valleys, partially filling them and disorganizing the drainage system. In many cases inter-

stream divides were buried by glacial deposits and streams now pass from one old drainage basin to another. Where streams still flow within the old valley, they are seldom in the old channel; therefore, when a stream has a rock floor, there may be a buried channel nearby. The sorted materials consist of gravels, sands, silts and clays, or, as in the case of a modified glacial drift, combinations of these sizes. These materials have been washed from the ice, sorted or partially sorted and deposited at some point where the velocity of the water slackens. The Contoocook Valley has many of these materials in the form of terrace deposits, eskers, and delta deposits. An extensive blanket of sediments was deposited on the floor of the valley during the recession of the glacier when the northward flowing Contoocook River was dammed by the ice front and a large glacial lake formed. When the ice front retreated northward the lake was drained into the Merrimack River Valley and erosion of the present Contoocook channel began.

(b) Vicinity of the Dam Site.- In the vicinity of the site of the Bennington Dam, the river flows through extensive shallow surface deposits of unconsolidated sediments which overlie deposits of glacial till in some areas, as described in the following paragraphs. Bedrock is deeply buried throughout the vicinity except where it outcrops in the floor of the river approximately 3/4 mile downstream from the dam. The western wall of the present valley is composed of a valley terrace deposit and the eastern wall is composed of glacial till. Esker and kame deposits are common throughout the area.

(c) Source of Data.-Information pertaining to historical and existing geology of the Contoocook Valley has been abstracted from reports on the Bennington site presented to the Boston District Office by the following geologists: Mr. Sidney Paige, Geologist, North Atlantic Division Office, Mr. Charles P. Berry, Consulting Geologist, Columbia University, and Mr. Irving B. Crosby, Consulting Geologist, Boston.

6-03. METHOD AND SCOPE OF GEOLOGICAL AND SOILS FIELD INVESTIGATION.- (a) General. Reconnaissance was conducted in the general vicinity selected for the project location. Topography and geography were studied and a superficial examination was made to investigate the principal geological formations. Primary consideration was given to gathering information necessary for the proper development of a satisfactory foundation and to locating sources of suitable construction materials. Fieldwork included seismic exploration, drilling and sampling of holes in overburden and rock, excavation and sampling of test pits and trenches, installation and reading of observation wells, and employment of pressure and pumping tests.

(b) Seismic Exploration.- Under the supervision of Mr. E. R. Shepard, Office of the Chief of Engineers, seismic explorations were conducted at twenty-one locations within the project area (For those in immediate vicinity of dam site and borrow area, see Plates A-5 and A-11) Tables of information obtained from these investigations are shown on Plates A-10 and A-13. Some indication of the general character of the overburden was obtained by wave velocities above the rock surface.

Eighteen (18) seismic lines were fired at the proposed site to determine conditions in the foundation and three (3) at the proposed site for impervious borrow material.

(c) Drill Holes.- Foundation conditions were investigated by drilling eighty (80) holes with a total footage of 4,783.9 feet. Sites of pervious and impervious borrow materials were explored by drilling thirty-one(31) holes with a total footage of 1093.8. Holes were drilled in the overburden and rock using a 3-inch casing in the overburden and a 1-5/8 inch diamond bit in the rock to obtain bedrock cores. Samples of soils encountered were taken with a 2-inch drive sampler. Preliminary drill holes in the borrow areas were advanced using a 2-inch casing, and sampling materials with a 1-inch drive sampler. Bedrock coring was not done for borrow exploration.

(d) Shallow Test Pits and Trenches. In order to inspect the soils in the foundation and borrow areas in their natural condition and to obtain samples of the undisturbed materials, shallow test pits and trenches were excavated. Foundation conditions were explored by means of eighteen (18) test pits with a total depth of 117 feet and by means of three (3) test trenches with a total depth of 45.8 feet. Exploration of borrow areas included fourteen (14) pits with a total depth of 159.0 feet and five (5) trenches with a total depth of 84.5 feet. After test pits had been excavated to an approximate depth of 6 feet, they were extended, wherever possible, by auger borings. Bag, jar, constant volume and chunk samples were taken. Field tests for natural density were performed in the pits located in the pervious borrow area. Where required, undisturbed samples, one cubic foot in size, were taken.

(3) Deep Test Pits and Trenches.-To supplement the shallower surface explorations, a test pit 24.8 feet deep was excavated in the spillway foundation area and the pervious borrow area "B" was explored with four (4) deep pits with a total depth of 77.5 feet and two (2) trenches with a total depth of 61.7 feet. The spillway foundation pit, T60p, was sheathed with timber during excavation. Plate VI - 3 shows a sketch of the pit, a graph of the gradation range of silty till encountered and a tabulation of results of tests of samples taken. A total of forty-five (45) samples, which included waxed chunks, cubic-foot-size undisturbed samples and 50-lb. bags were taken. A power crane with 1/2 cy. clamshell bucket was employed to excavate the deep holes required to obtain a complete exploration of the pervious borrow area. These holes are shown on Plate A-11 as T84t, T86p to T89p, inclusive. Cobbles and small boulders encountered were weighed in the field to determine the percentage of larger stones available for future crushing for concrete aggregates. One-hundred pound bag samples were taken of each layer encountered.

(f) Observation Wells. Eighteen (18) observation wells were installed in drill holes within the dam site area to obtain detailed knowledge of ground water conditions (Plate A-5). Each observation well consists of a 1-1/4" pipe, perforated at the bottom, extending slightly above ground surface. Sand and gravel has been placed around the perforated section to prevent the wells from being plugged by movement of the

fine material. Each well point was placed in the most pervious zone encountered in the drilling of the hole. Five (5) of the wells are located on the western side of the river and have their points exposed to the pervious layer than underlies the till blanket. One well is near the outlet of the proposed discharge channel. The remaining twelve (12) wells are in the spillway area and have been installed in the weathered granite which underlies the till. The wells in the spillway area have hydrostatic heads at or above the ground surface. Readings on the earlier wells were at first taken daily, and then weekly. Since fluctuations were not great for short intervals, monthly readings are being continued in 1946.

Observations to date are summarized in the following tabulation:

Observation Well	Ground Surface Elevation	Elevation of Water in Obser. Wells	
		Minimum	Maximum
West Abutment:			
D 8	695.1	684.6	698.8
D11	671.7	673.5	678.8
D18	671.9	672.9	673.9
D19	681.6	675.6	690.0
D57 Spillway	700.3	687.7	693.6
D38	672.8	667.9	669.7
D39	673.8	673.6	674.9
D42	673.2	673.9	674.6
D43	677.6	674.3	675.9
D45	673.3	670.0	671.5
D66	673.7	673.2	675.3
D67	673.4	670.4	675.3
D75	670.6	669.5	671.2
D76	669.4	667.1	667.5
D77	667.5	666.5	670.0
D78	674.0	671.7	674.5
D79	675.7	673.4	675.6
Discharge Channel			
D46	667.7	667.1	669.9

(c) Pressure Tests in Bedrock.- Two(2) drill holes, D66 and D67, located in the spillway area, indicated the presence of a zone of highly weathered rock immediately beneath the till body. Drill hole D66 penetrated a total of 40 feet of rock and D67 penetrated 47 feet of rock. In each of these holes bedrock was tested for the total leakage and then 5-foot sections of the rock were tested to determine the zones of greatest leakage. [illegible] flow of water under a pressure of approximately 70 lbs. per sq. inch. Drill hole D66 had approximately ten (10) feet of weathered rock which was too soft to allow testing. Except for a large seam near the bottom of the hole which accounted for a flow of ten (10) gallons per minute, the permeability of the rock was relatively high. In drill hole D67 there were approximately twenty (20) feet of decomposed rock in which the pressure testing equipment could not be sealed, but four feet of moderately fractured rock with a total loss of 5 gallons per minute water 70 lbs. per sq. inch pressure, and 17 feet of [illegible] unfractured rock. Pressure applied to the water in drill hole D67 caused the water level in drill hole D66 to rise, thus indicating

network of interconnected fractures within a localized area.

(h) Pumping Tests.- (1) Preliminary.- Small scale pumping tests were first conducted at drill holes D66 and D67, located on the spillway area a distance of 150 feet apart, to investigate the perviousness of decomposed rock and to determine the feasibility of lowering the hydrostatic head during construction of the spillway. A four-hour pumping test at drill hole D67 at an average flow of 3.5 gallons per minute lowered the water level in drill hole D66 a distance of 16 inches. Sixteen hours after the cessation of pumping the water level in drill hole D66 had risen only 9 inches. Two days later, when the water level had returned to normal, drill Hole D66 was again pumped and observations taken at drill hole D67. Four hours of pumping lowered the water level in drill hole D67 a distance of 20 inches. One hour after the pumping ceased the water in drill hole D67 had risen 5 inches.

(2) Final.-In order to obtain more complete information on the behavior of the ground water in the weathered rock zone, a large scale pumping test was conducted during the period 15-20 October 1945. In preparation for this test ten (10) additional observation wells were installed. These observation wells were installed in drill holes D38, D39, D42, D43, and D45, which were previously driven for foundation investigation, and which were extended to penetrate the bedrock. In addition, drill holes D75 to D79, inclusive, were drilled into rock and observation wells installed therein. In order to avoid drilling the 24.8 foot depth of test pit T60, observation well D75 and pumped well D80 were both installed in the bottom of the test pit. Drill hole D75 was drilled first to serve a. as a pilot hole for the pumped well and b. as an observation well during pumping. D80 was driven with a 6-inch chopping bit employing an 8-inch casing through the overburden and sealing the casing against the bedrock. A 6-inch hole was chopped a depth of 30 feet into bedrock. Pumping started on 15 October at the rate of 5 gallons per minute. This rate was maintained continuously until 17 October when the rate was increased to 10 gallons per minute. It was soon found that the increased rate could not be sustained continuously, and on 20 October, which was the last day of the test period, the rate had dropped to an average of 9.4 gallons per minute. Contours showing the water pressure heads before the test and at the end of the pumping period are shown on Plate VI - 4. Recovery of water levels after the test was relatively slow. Some holes recovered to an elevation of 5 feet below normal level after 1-1/2 days and did not return to normal elevation until after more than a week.

6-04. GEOLOGY OF SITE.- (a) General.-Principal conditions encountered with reference to the geology of the dam site are discussed in the following sub-paragraphs. A plan of foundation exploration showing the location of drill holes, seismic lines, and contours of till and bedrock is shown on Plate A-15. Geological profiles are shown on Plates VI-2. Graphic logs of individual explorations are shown on Plates A-6 to A-10 inclusive. A summary of information obtained from seismic investigation is shown on Plate A-10.

(b) East Abutment.- The east abutment of the dam and the embankment foundation as far as station 8 + 38 is composed of a compact glacial till that extends to bedrock which is located at depths of approximately 50 feet below the surface. The till varies from sandy till

near the surface to silty till which forms the major portion of the body. Overlying the till is a cover of variable silty sand and gravel. Cobbles and boulders occur occasionally in the till and frequently in the surface deposits. These conditions prevail within the embankment area and the adjacent downstream area. Upstream from the embankment area a variable thickness of sand deposit is found overlying the till.

(c) Spillway Area.- The spillway area extends from station 8 + 38 to station 11 + 38 along the centerline of the dam. The principal soil in this area is a compact, sandy to silty till which extends from a depth of approximately 10 feet below the surface to bedrock located at depths from 30 to 60 feet. The bedrock is a porphyritic granite and has a very badly weathered capping that is much more pervious than the overlying till. This weathered zone varies in depth from 5 to 30 feet or more and grades into fresh, moderately fractured granite. The top of the till body appears slightly weathered and is shown as a thin layer on the geological profiles. On top of the till is a compact layer of variable silty sand and gravel and which contains cobbles and boulders. The ground water table is very close to the ground surface in this area and the hydrostatic head in the bedrock under the till is slightly above the ground surface. Conditions in the immediate upstream and downstream areas are essentially the same.

(d) River Channel Area. The river channel area from station 11 + 33 to station 22 + 00 is composed of surface deposits of material which varies between silty and gravelly sand averaging 20 feet in thickness, overlying an extensive body of sandy to silty till which, in turn, overlies weathered granite located at depths in excess of 50 feet below the surface. The surface of the till drops in both the upstream and downstream directions.

(e) Drainage Well Area. The section of the embankment foundation from station 22 + 00 to 30 + 00 has 10 to 15 feet of variable silty and gravelly sand. Beneath these sediments, the body of sandy to silty till splits into two sections: The upper section tapers out laterally and disappears near station 32 + 00; the lower section dips sharply toward the west and disappears against the rock floor. The zone between the till blankets is occupied by sand and silt strata ranging in permeability from pervious to semi-pervious. Bedrock in this area is approximately 100 feet below the surface. As shown on Profile B of Plate VI-1, the upper section of till tapers out upstream permitting ready access of water to the underlying more pervious soils. Downstream from the dam, the pervious sub-stratum is cut off by contact of the upper till layer upon bedrock.

(f) From station 30 + 00 to 37 + 00 the embankment is underlain by large terrace deposits of sand and silt which extend from the ground surface to bedrock at depths ranging from 100 to 150 feet. Materials in this terrace possess gradations ranging from loose, medium sand at the surface to moderately compact silt at considerable depth. The abutment beyond the dam is composed of soil of the same nature.

6-05. FOUNDATION PROBLEMS.-Insofar as their physical properties are concerned, no serious problem is presented by the nature of the soils and

rock encountered. Overburden is loose to very compact; soft silts or peats were not encountered; and rock, although weathered at the surface, is satisfactory since structures are not to be founded directly thereon. However, there are present at two (2) locations subsurface water pressures which will influence the construction procedure and design.

(a) Spillway Foundation.- At the site of the spillway foundation, the glacial till upon which the concrete spillway structure will rest is underlain by a weathered zone of rock containing water under a slight artesian head. At the northwest corner of the spillway, approximately 10 feet of till will remain over this weathered rock after excavating to a depth of about 17 feet below present ground surface for the spillway structure foundation. Should the hydrostatic head in the rock not be lowered, the reduction of load on the rock during excavation would allow the remaining till to be subjected to uplift forces 1.2 times greater than its weight, and there would be the possibility of a blow-up and softening of the till through upward seepage which is always present. Pumping tests to determine the behavior of the artesian water in the weathered rock have been discussed in Paragraph 6-03(h). Proposed treatment for this condition is discussed in the design of the spillway foundation in Paragraph 7-03(c).

(b) Drainage Well Area.- At the west portion of the main embankment, between Stations 22 + 00 and 30 +00, as discussed in paragraph 6-04(e), the glacial till tapers out upstream allowing free access of water from the reservoir to the more pervious sands beneath the upper till layer beneath and downstream from the embankment. This seepage will not be relieved naturally because the layers of till are sealed against bedrock downstream from the dam. This situation allows the possibility of high pressure heads being developed under the upper layer of till when the water level in the reservoir rises during flood stages. The use of drainage wells to relieve this potential pressure is described in Paragraph 7-03 (b).

6-06. SOURCES OF CONSTRUCTION MATERIALS:-(a) General.- Satisfactory sources of construction materials are readily available in the immediate vicinity of the dam site. The Contocook Valley has large amounts of glacio-fluvial material in the form of eskers, kames, and terrace deposits. These deposits vary in the amount of sorting they have received; hence, variations in grading and degree of permeability are available. Also available in the immediate vicinity of the site are massive deposits of glacial till which are in general overlain by a variable thickness of pervious sands, silty sands, and gravelly sands, with occasional boulders concentrated generally on the surface. The general locations of borrow areas are shown on Plate A-1 . It is proposed that the Government will furnish to the Contractor borrow areas A, B-1, B-2, and C. All other borrow required will be furnished from sources to be obtained by the Contractor. Plate A-11 is a plan of borrow exploration. Plates A-12 and A-13 show the logs of the results of exploration.

(b) Impervious Borrow.- The most suitable locally available sources of impervious borrow are the massive deposits of glacial till. Five (5) different areas were investigated as possible sources of impervious borrow. The most economical deposit and the one selected as a source

of impervious borrow for the embankment is area A located approximately 2,000 feet northeast of the spillway location. This area is a continuation of the deposit that forms the east abutment of the dam. Borrow Area A is situated on a side hill and has a rather irregular topography, and is cut by a small creek which flows intermittently. The material in the deposit is a well-graded gravel, sand, and silt, with some clay sizes and occasional boulders. The thickness of the material suitable for impervious borrow ranges from 10 feet to more than 25 feet. Ground water observations taken during drilling operations in the till indicate that the water table during the fall and winter is at a depth of about 5 feet. Overlying the till is a layer of material approximately 15 feet in thickness. This material is classified as random borrow and is used in the embankment as described in the following sub-paragraph.

(c) Random Borrow.- The random borrow consists of approximately the top 15 feet of semi-impervious to pervious soil which overlies the impervious borrow area described in the preceding paragraph. The material consists of variable sands and sandy till and contains occasional cobbles and boulders. Approximately one foot of topsoil is required to be stripped. Excavation of this material is required to obtain the impervious borrow.

(d) Pervious Borrow.- There are two principal types of deposits which are suitable for pervious fill in the embankment: (1) The chain of eskers which lie principally on the east side of the river and extend for about a mile starting in the vicinity of the Powder Mill Dam; and (2) the extensive terrace deposits which form the western edge of the valley. The eskers are approximately 25 feet in height and consist generally of well graded clean sand, gravel, and cobbles, with pockets and lenses of clean sand and sandy gravel. The terrace deposits consist of fairly clean sand, graded from fine to coarse and with gravelly phases. Both the eskers and the terrace deposits have been worked as borrow pits for local roads and other construction. Both deposits are close to the dam site, and explorations for pervious borrow have been confined to these two deposits. Ground water in the pervious deposits is below the proposed depth of excavation. Stripping from one to two feet of sandy topsoil is required to expose usable material. The eskers shown on Plate A-11 are designated as borrow areas B-1 and B-2 which are expected to be the source of coarser graded materials and concrete aggregates. The terrace deposit shown as Borrow Area C is much finer in gradation than the deposits in B-1 or B-2 and is primarily included because of the economy of having a borrow source on each side of the river.

(e) Structure Excavation.- More than half of the embankment materials required will come from structure excavation. A small portion of impervious material is available from excavation of the silty till for the spillway foundation. The coarser surface deposits encountered, particularly in the approach channel excavation, will be suitable for pervious fill, and the remainder of excavation required can be used for random and semi-compacted fill. Oversize stones and boulders encountered are expected to furnish the bulk of riprap stone.

(f) Selected Gravels.- Gravel backing for use under riprap is available from Borrow Area B without processing by using the coarsest ranges of bank run material. The selected gravel required for the drainage

features may be obtained from the pervious borrow sources by selecting the coarser range of pervious borrow materials available.

(g) Processed Aggregates.- It is considered economical to establish a plant at the site to process aggregates from material excavated from the pervious borrow area. Because of the substantial proportion of oversize material available, crushing or oversize material is proposed. The suitability of aggregates processed from the eskers was checked by the North Atlantic Division Central Concrete Laboratory from samples submitted for complete analysis. Results of tests show the material will be satisfactory if properly processed and washed. Experience of this office and other Federal and State agencies also indicate that clean glacial gravel deposits of the type encountered here are satisfactory sources of concrete aggregate.

(h) Rock for Slope Protection and Dumped Fill.- Rock for the cobble facing of the downstream embankment slope may be obtained by culling the larger sizes of rocks encountered during the placing of fill materials. For the purpose of collecting oversize stones, raking of the embankment surface during construction by means of power equipment may be a satisfactory procedure. Required derrick stone and riprap are available from two sources: a. The separation and use of small boulders between one-half cubic foot and one cubic yard in size from structure excavation, and borrow excavation to be augmented by collecting surface boulders within the project area and from stone walls in the vicinity; b. The establishment by the Contractor of a rock quarry at Bell Ledges. It is estimated that two-thirds of the total rock required may be obtained by separation and collection of boulders. Almost without exception these boulders are granite, and are sound and unweathered, except on exposed sides where shallow surface weathering may be observed. The large exposures of bedrock at Bell Ledges consist of porphyritic granite. The formation at this location is suitable for quarrying of sound rock without the removal of an appreciable amount of overburden. The construction of approximately one mile of access road to the site is required. The haul distance is three miles, all of which is downhill.

6-07. LABORATORY TESTS AND INVESTIGATIONS. - (a) General.- To determine the characteristics of foundation and embankment materials required for design and construction, a comprehensive program of laboratory tests was followed, using the soil samples, rock cores and data obtained from the field exploration. Work was accomplished in the District Soils Laboratory. The methods used are described briefly in the following paragraphs. Results of tests are shown in summary form on Plates VI-5, VI-6, and VI-7. A summary table of soil properties is shown on Figure 4. Plate VI-6. All soil samples from the field explorations were classified and recorded in the report of each hole. Geological descriptions of the overburden at each drill hole were prepared after a study of the laboratory logs, field logs and soil samples. The bedrock at each drill hole was classified by a geologist after examining the rock cores and field logs.

(b) Mechanical Analysis.- Mechanical analysis of selected representative samples were made using a standard sieve analysis with a minimum sieve of 100 meshes per inch and an hydrometer analysis of all sizes passing that sieve. The results of these tests were used as a guide in the classification of materials. Resulting gradation ranges of the soils

are shown in Plate VI-5 as Figures 1 to 5, inclusive.

(c) Water Content.- The natural water content of principal materials was determined from samples obtained in the field and transported to the laboratory in paraffined jars. Some values were determined in connection with the testing of waxed chunk samples for density. Results incorporated in Plate VI-6, Figure 4, are reported in terms of percentage of oven dry weight.

(d) Specific Gravity.- The specific gravity of principal materials was obtained by the water displacement method (A.A.S.H.O. T100-38). Results are incorporated in Figure 4, Plate VI-6.

(e) Density.-(1) Natural.- Density of soils having cohesion was determined from undisturbed paraffined chunk samples. Density of cohesionless soils was determined in the field by the sand displacement method. Results are expressed in terms of unit dry weight. Void ratios were calculated from the determined specific gravities. Results are incorporated in Figure 4, Plave VI-6.

(2) Compacted.- Compaction characteristics of the binder portion of the silty till have been determined by a. the Modified Proctor Test using a ten-pound hammer with an 18-inch drop and 5 layers of soil, and b. the Standard Proctor Test. Material passing the No. 4 sieve is considered binder. Compaction characteristics for the binder portion of cohesionless soils included a. the determination of minimum dry density by placing soil in a container without vibration or impact, and b. the determination of maximum dry density obtained by the "Providence District" method of vibrating the soil confined in a cylinder under constant pressure of a spring. The densities to which the fill materials can be economically compacted in the dam embankments have not been determined by field tests using actual construction equipment. Results of the Standard and Modified Proctor density tests are shown on Plate VI-5, Figure 6.

(f) Permeability. Permeability of materials was determined by using de-aired water in a falling head type apparatus with a plastic, transparent permeameter following the general recommendations of G.D. Bertram in a publication of Harvard University Graduate School of Engineering entitled, "An Experimental Investigation of Protective Filters." All test results were corrected to a temperature of 10° Centigrade. Samples tested were binder materials in the remolded state. Tests were made at three or more void ratios in order to determine possible range of permeability. Several samples of the silty till under the spillway area were tested in the undisturbed condition. No consistent difference was found between permeability in the horizontal and vertical directions for this material. Results of permeability tests are shown on Figure 7, Plate VI-5.

(g) Shear Strength.- (1) Test Procedure.- A comprehensive shear testing program was undertaken using the triaxial compression type of shear apparatus. Samples tested were 2.8" in diameter and 6" or 8" high. Three types of tests were performed:

a. Compacting and testing remolded specimens in the dry state.

b. Compacting remolded specimens in the dry state, saturating, applying the minor principal stress, and testing immediately.

c. From cubic foot undisturbed samples, specimens were cut, saturated, subjected to the minor principal stress, and tested immediately.

(2) Spillway Foundation Material.-Both remolded and undisturbed samples of the silty till from test pit T60p were tested as described above; these samples represent the soil upon which the spillway will be founded. In addition to these tests, several undisturbed specimens of silty till from test pit T60p were tested for unconfined compressive strength. Results are shown on Figure 1, Plate VI-6.

(3) Embankment Foundation Materials.- Samples of the gravelly and silty sand encountered as embankment foundation were tested only in the remolded condition. The ranges of shearing test results are shown on Figure 2 of Plate VI-6.

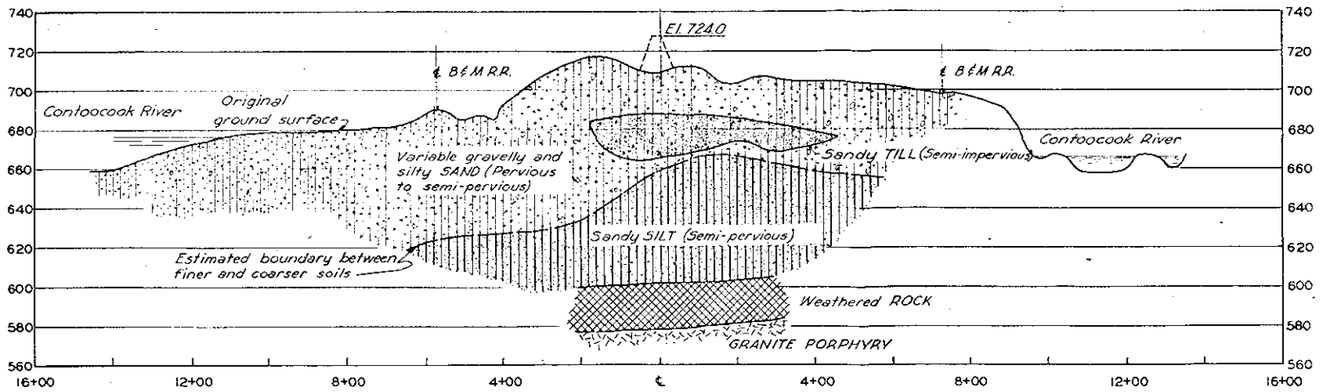
(4) Embankment Materials.- Tests on remolded samples were used to determine the shear characteristics of the embankment materials. Resulting ranges of shear strength are shown on Figures 1,2 and 3, Plate VI-6. Impervious fill characteristics are shown by the curves for remolded material in Figure 1. Random fill is represented by the results in Figure 2. The selected pervious fill shear strength is shown by Figure 3 and the higher strength ranges of Figure 2.

(h) Critical Void Ratio.- To determine the critical void ratio, a series of triaxial compression tests was run upon a sample from Borrow Area "C" which represents the finer allowable range of pervious borrow material encountered. Lateral pressure was held constant throughout the test and the volume change corresponding to maximum shearing strength was recorded. Results are given on Plate VI-7.

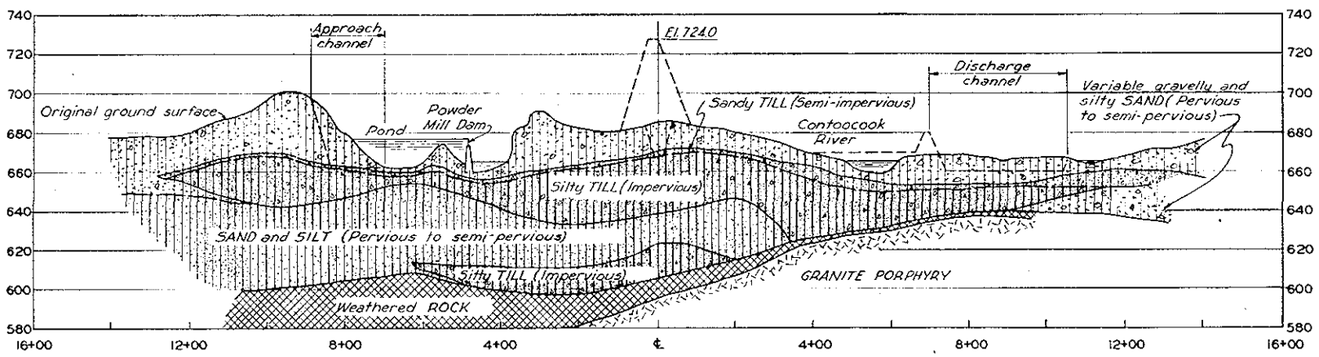
(i) Consolidation.- A series of three consolidation tests were run upon remolded samples of the silty till in order to determine the settlement characteristics of the impervious section of the embankment. Two samples from Borrow Area "A" and one sample from test pit T60p in the spillway foundation were tested. Results of the test on the spillway foundation material are shown on Figure 5, Plate VI-6.

(j) Expansion.- Expansion tests were not made on the impervious material for the dam embankment. Results of tests performed previously in the District on similar material indicated that no detrimental expansion will occur due to addition of moisture, even though the material is highly compacted.

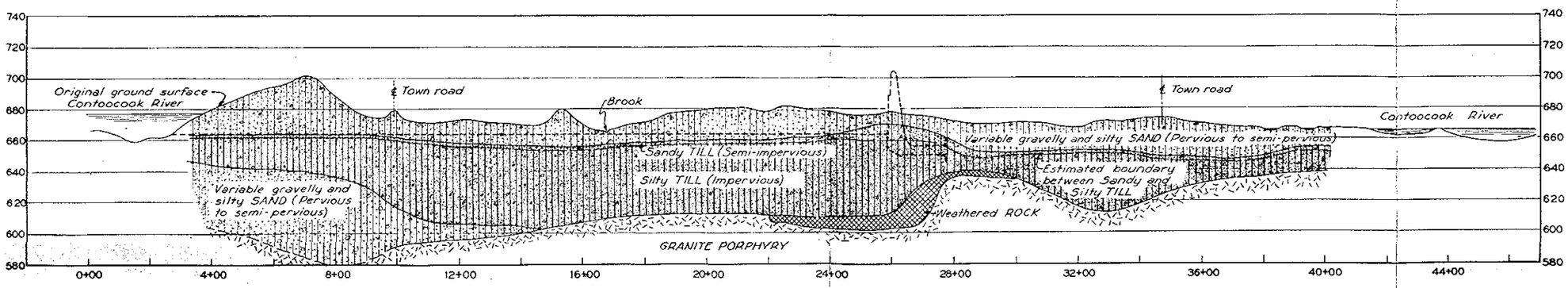
(k) Optimum Moisture.- The optimum moisture contents of the several embankment materials were determined in connection with the compaction tests described in paragraph 6-07(e). Results of these tests are included in Figure 4, Plate VI-6.



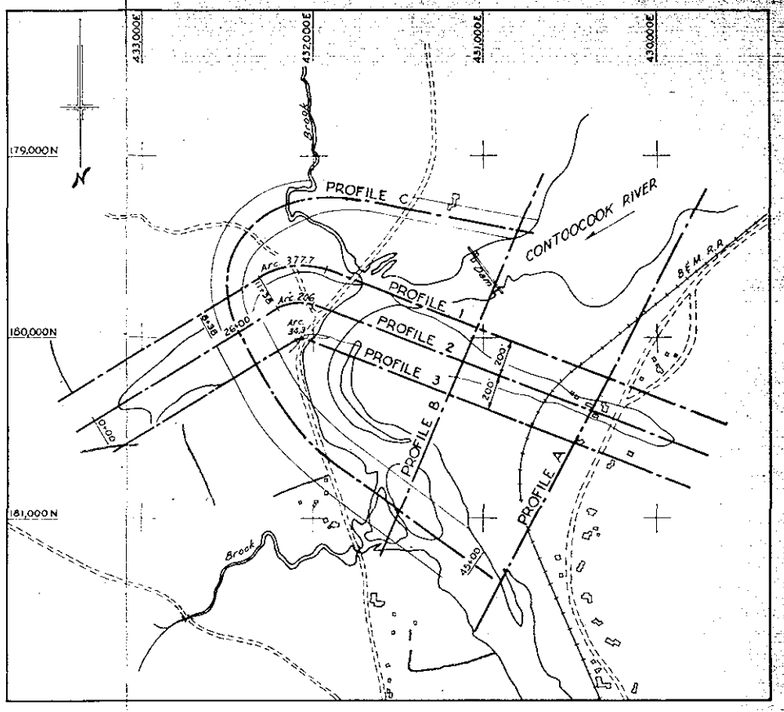
PROFILE A



PROFILE B



PROFILE C



KEY PLAN

500 0 500 1000
SCALE IN FEET

NOTES

Elevations are based on U.S.C.&G.S. Datum of Mean Sea Level.

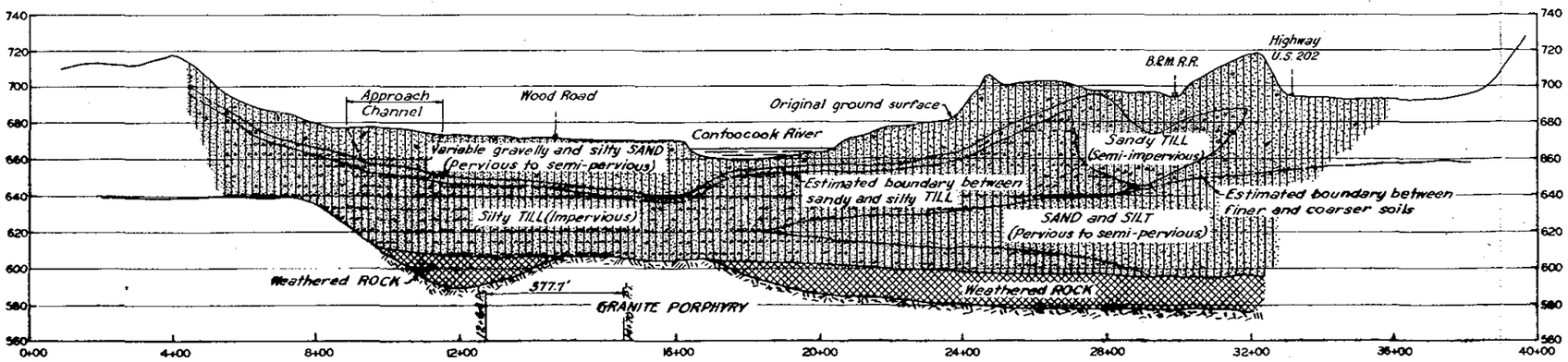
Boulders were encountered in majority of explorations and are generally scattered over the ground surface east of railroad.

Ground water encountered approximately at ground surface between Sta. 8+00 and 24+00.

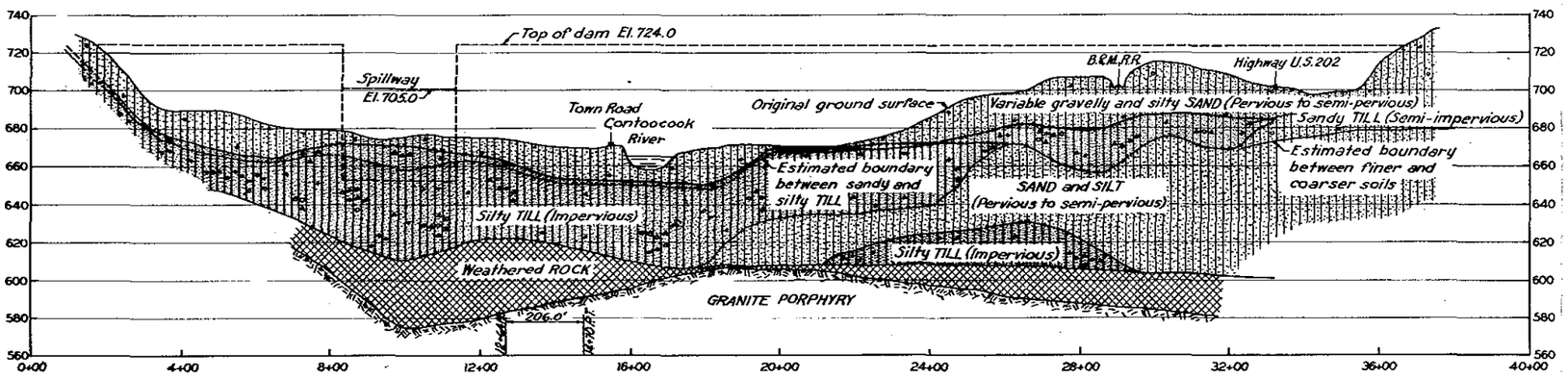
MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON DAM
CONTOOCOOK RIVER

GEOLOGICAL PROFILES NO. 1

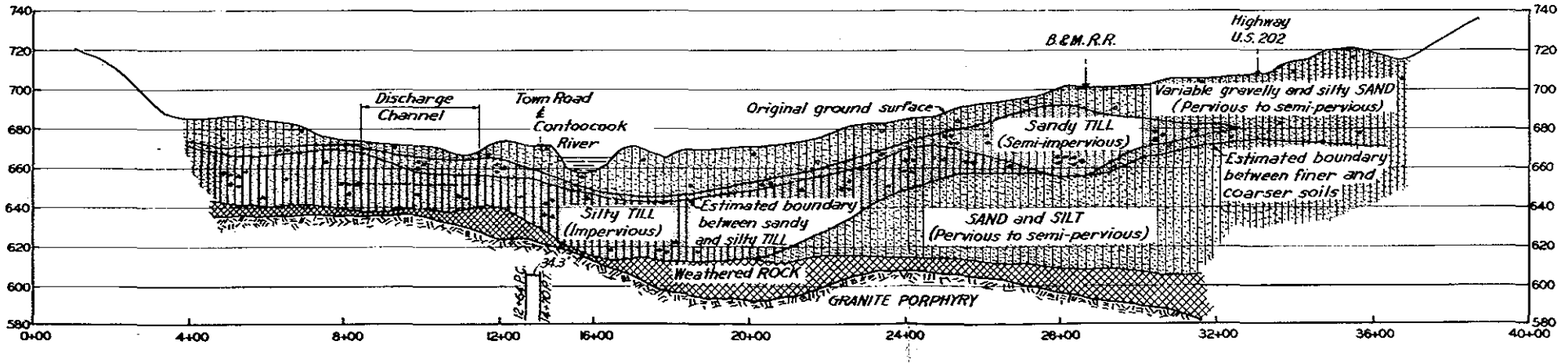
U.S. ENGINEER OFFICE, BOSTON, MASS. FEB. 1946



PROFILE 1-200 FT UPSTREAM FROM ℄ OF DAM



PROFILE 2-℄ OF DAM



PROFILE 3-200 FT DOWNSTREAM FROM ℄ OF DAM

NOTES

Elevations are based on U.S.C. & G.S. Datum of Mean Sea Level.

Stationing on Profiles 1 and 3 corresponds to stationing on ℄ of dam (Profile 2).

Boulders were encountered in majority of explorations and are generally scattered over the ground surface east of railroad.

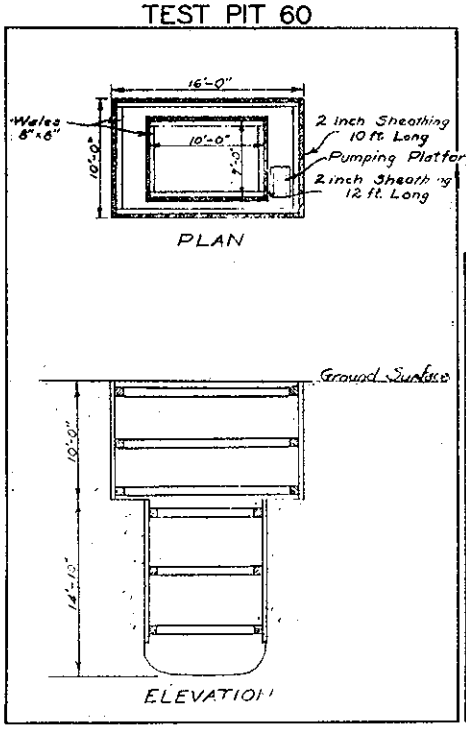
Ground water encountered approximately at ground surface between Sta. 8+00 and 24+00.

MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON DAM
CONTOOCOOK RIVER

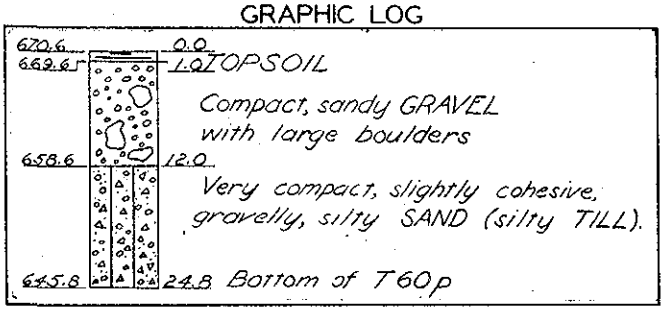
GEOLOGICAL PROFILES NO. 2

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

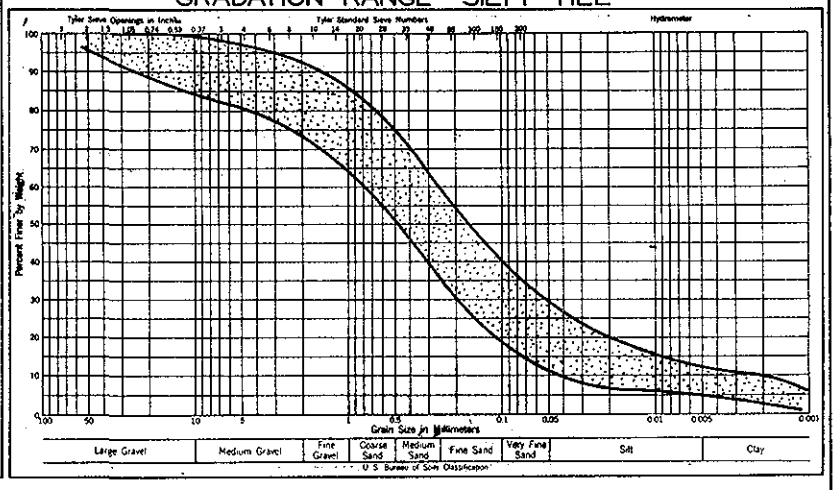
TEST PIT 60



GRAPHIC LOG



GRADATION RANGE - SILTY TILL



SUMMARY OF TEST DATA

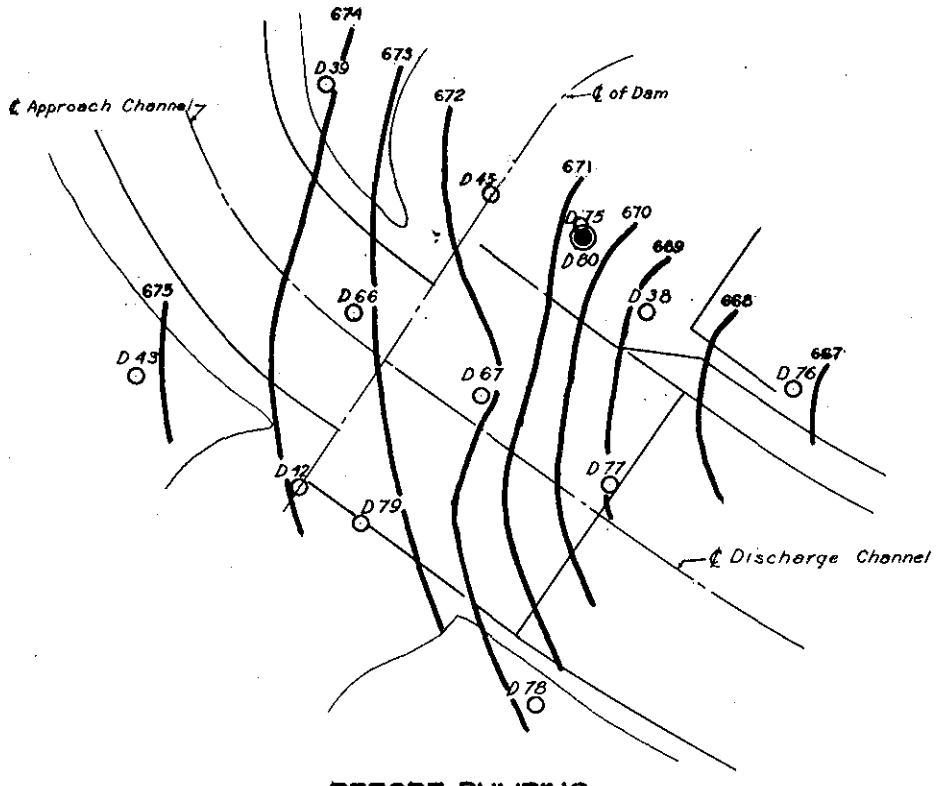
SAMPLE NO.	DEPTH ft.	WATER CONTENT w % dry wt.	DRY DENSITY d p. c. f.	SPECIFIC GRAVITY	COMPACTION CHARACTERISTICS: MAX. DRY DENSITY	OPT. WATER CONTENT w % dry wt.	TOTAL SAMPLE MAX. DRY DENSITY p. c. f.	PERMEABILITY 10^{-4} cm/sec: UNDISTURBED	REMOLDED	SHEAR STRENGTH: REMOLDED DRY	REMOLDED SATURATED	UNDISTURBED
1	15.0-16.0							0.04				*
2	17.0-18.0							0.04				*
3	" "											
4	19.0-20.0	7.4	137.9									
5	" "	5.3	143.6	2.68				0.34				*
6	20.0-21.0	7.6	135.6									
7	" "	5.7										
8	" "	8.3	143.5									
9	" "	9.7	132.1									
10	" "	8.1	133.9									
11	" "	10.2	126.9									
12	" "	8.6	131.9	2.71								
13	" "	8.7	134.6									
14	" "	9.5	132.1									
15	" "	8.3	134.9									
16	" "	7.7	135.8	2.69								
17	" "	7.5	138.9	2.71								
18	" "											
19	22.5-23.0)			2.69)	130.8)	7.5)	132.8)		0.39)	41°-45°)	36°-39°)	
20	21.5-22.5)											
21	" "		134.6									
22	" "											
23	" "		140.9									
24	" "		139.2									
25	" "		136.8									
26	" "	1.0	132.4									*
27	22.5-23.5											
28	" "		136.0									
29	" "		135.8									
30	" "		137.3									
31	" "		138.8									
32	" "		137.4									
33	" "		140.6									
34	" "		140.2		135		140					
35	24.0-24.8	11.5	108.6	2.7								
36	" "		136.2									
37	" "		134.2									
38	" "	7.6	135.2	2.66								
39	" "	7.7	135.8	2.75								
40	" "		134.9									
41	" "	8.1	134.7	2.68								
42	" "	9.9	128.5	2.70								
43	" "		134.5									
44	" "	7.5		2.60								
45	" "	17.8	131.1	2.62								
AVERAGES		8.4	135.1	2.68								

* See Fig. 1, Plate VI - 6

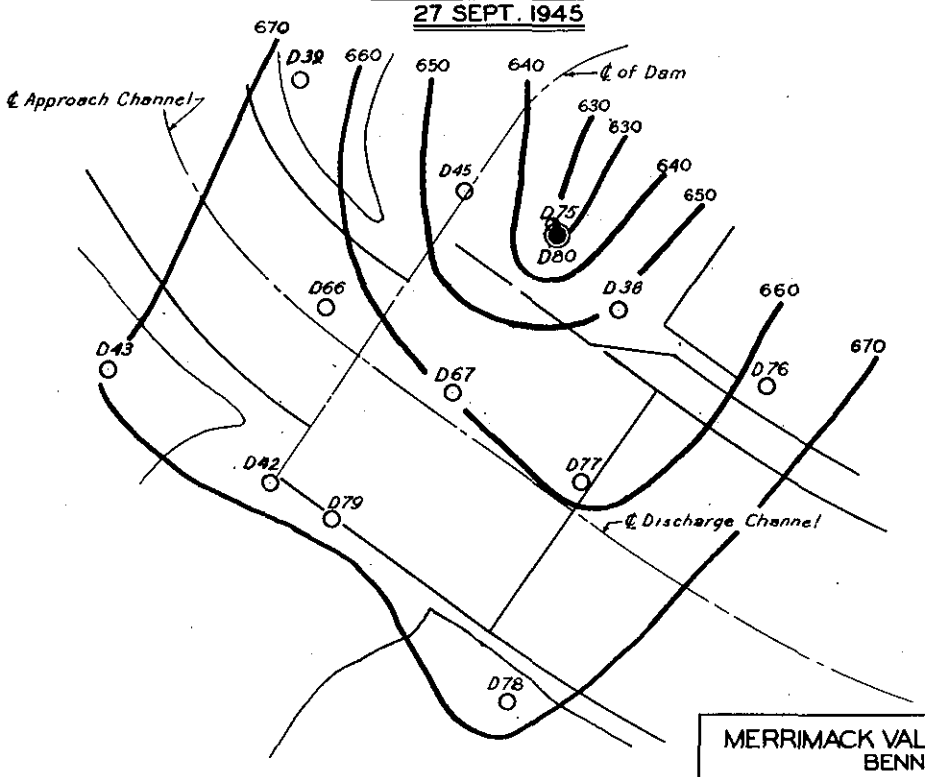
MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON DAM
CONTOOCOOK RIVER

DEEP TEST PIT T60p

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



BEFORE PUMPING
27 SEPT. 1945



AFTER PUMPING
20 OCT. 1945

Legend

Pumped well

Observation well

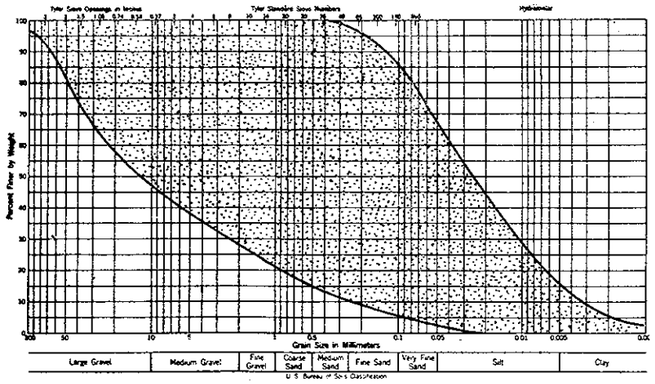
Light lines are structure outlines

MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON DAM
CONTOOCOOK RIVER

PUMPING TEST WATER DRAWDOWN CONTOURS

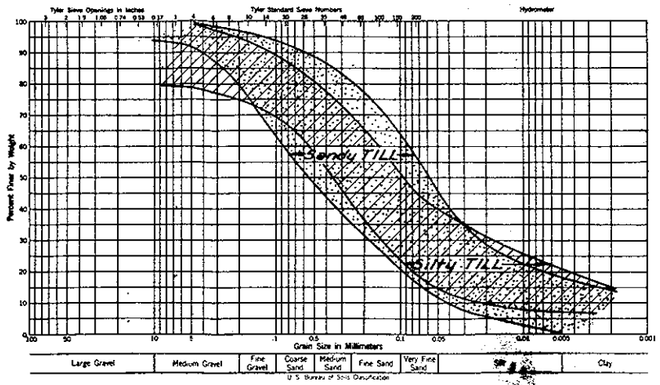
100 0 100 200
SCALE IN FEET

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



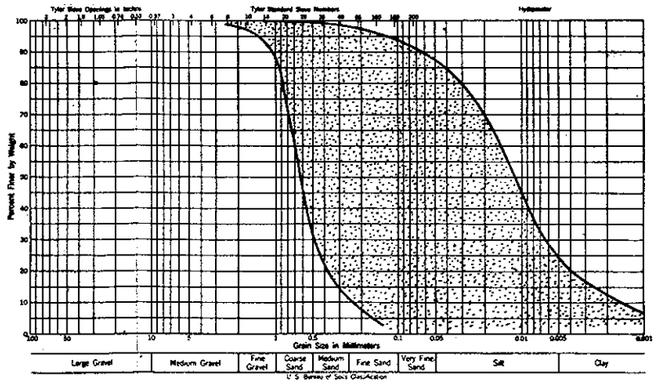
UNCONSOLIDATED SURFACE DEPOSITS

FIG. 1



SANDY AND SILTY TILL

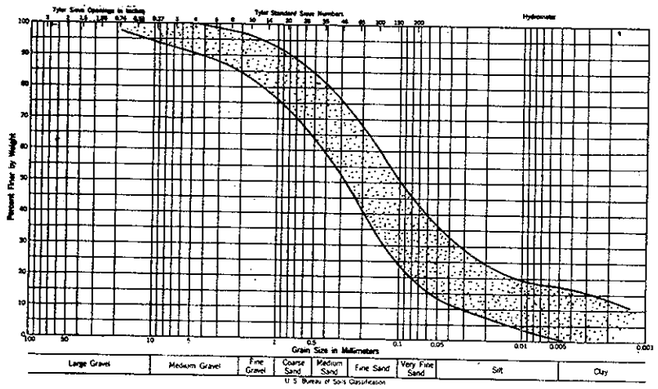
FIG. 2



SILT AND SAND UNDERLYING WEST ABUTMENT

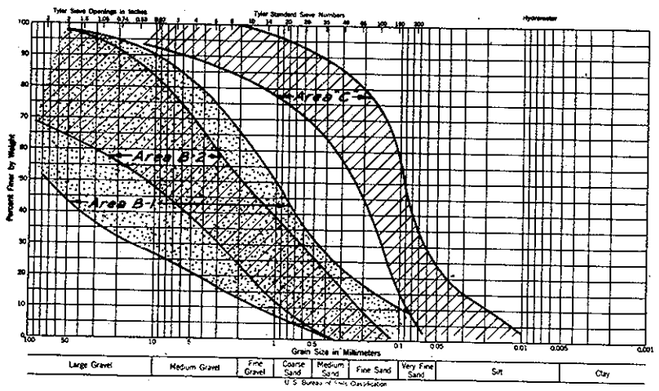
FIG. 3

GRADATION RANGES OF RANDOM AND FOUNDATION MATERIALS



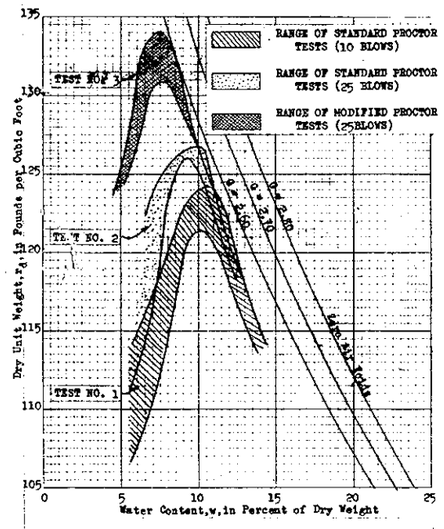
IMPERVIOUS

FIG. 4



PERVIOUS

FIG. 5



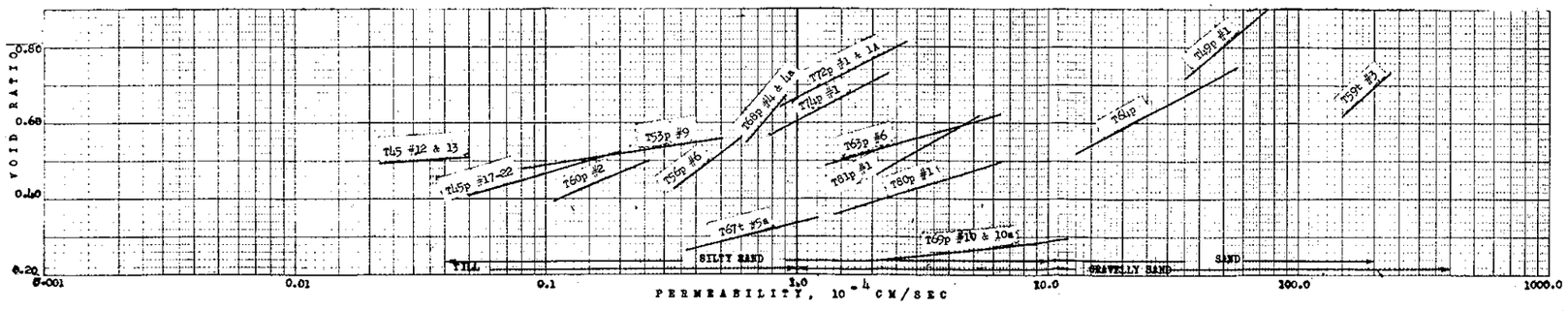
SUPPLEMENTAL DATA

ITEM	UNIT	TEST NO. 1	TEST NO. 2	TEST NO. 3
MAX. DENSITY, < #4 SIEVE	p.c.f.	121.5 to 124.3	126.2 to 126.8	130.7 to 134.0
OPT. WATER CONTENT, < #4 SIEVE	%	10.6 to 10.7	9.3 to 10.0	8.3 to 7.3
SPECIFIC GRAVITY .	—	2.70	2.70	2.70

RANGE OF MOISTURE-DENSITY PLOTS FOR COMPACTED IMPERVIOUS FILL

FIG. 6

GRADATION RANGES OF BORROW MATERIALS



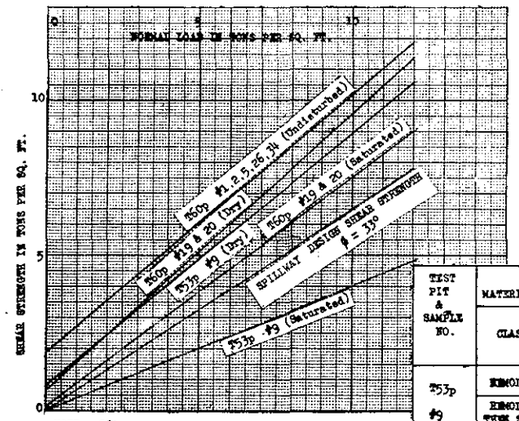
PERMEABILITY-VOID RATIO PLOT FOR REPRESENTATIVE SAMPLES OF PRINCIPAL FOUNDATION AND EMBANKMENT MATERIALS

FIG. 7

MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON DAM
CONTOOCOOK RIVER

SOIL DATA SUMMARY
NO. 1

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

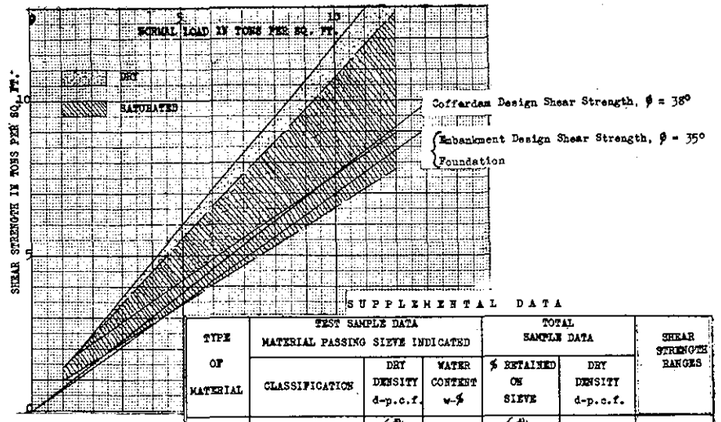


NOTE: T60p represents material encountered in spillway excavation
T53p represents material encountered in impervious borrow area

SUPPLEMENTAL DATA

TEST PIT & SAMPLE NO.	TEST SAMPLE DATA MATERIAL PASSING 3/4" SIEVE			TOTAL SAMPLE DATA	
	CLASSIFICATION	DRY DENSITY p.c.f.	WATER CONTENT %	% RETAINED ON 3/4" SIEVE	DRY DENSITY p.c.f.
T53p #9	REMOLDED DRY	119.7	--	2.0	120.2
	REMOLDED DRY, THEN SATURATED	122.8	11.5 to 13.3	2.0	123.5
T60p #19&20	REMOLDED DRY	124.3	--	11.1	127.6
	REMOLDED DRY, THEN SATURATED	125.9	9.6 to 11.4	11.1	129.0
T60p #1,2,5 26,34	UNDISTURBED	--	5.0 to 7.3	4.9	--

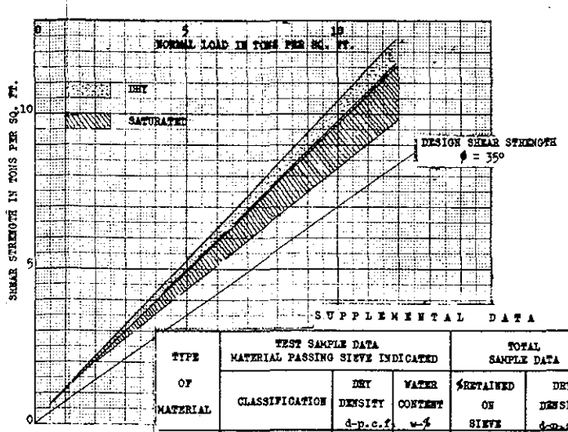
SILTY TILL (FOUNDATION & BORROW)
(IMPERVIOUS)
FIG. 1



SUPPLEMENTAL DATA

TYPE OF MATERIAL	TEST SAMPLE DATA MATERIAL PASSING SIEVE INDICATED			TOTAL SAMPLE DATA		SHEAR STRENGTH RANGES
	CLASSIFICATION	DRY DENSITY d-p.c.f.	WATER CONTENT w-%	% RETAINED ON SIEVE	DRY DENSITY d-p.c.f.	
VARIABLE GRAVELLY AND SILTY SAND	REMOLDED DRY	< #4 99.57	--	< #4 2.0	100.4	34° to 49°
		< 3/8" 148.8	--	< 3/8" 30.0	156.4	
	REMOLDED DRY, THEN SATURATED	< #4 94.76	27.3	< #4 2.0	96.7	32° to 45°
		< 3/8" 150.0	8.0	< 3/8" 30.0	157.4	

GRAVELLY AND SILTY SAND
(SEMI-PERVIOUS)
FIG. 2



SUPPLEMENTAL DATA

TYPE OF MATERIAL	TEST SAMPLE DATA MATERIAL PASSING SIEVE INDICATED			TOTAL SAMPLE DATA		SHEAR STRENGTH RANGES
	CLASSIFICATION	DRY DENSITY d-p.c.f.	WATER CONTENT w-%	% RETAINED ON SIEVE	DRY DENSITY d-p.c.f.	
GRAVELLY SAND	REMOLDED DRY	< #4 110.0	--	< #4 0.4	110.0	44° to 46°
		< 3/4" 135.6	--	< 3/4" 22.4	141.6	
	REMOLDED DRY, THEN SATURATED	< #4 110.0	20.1	< #4 0.4	110.0	37° to 45°
		< 3/4" 136.3	8.3	< 3/4" 22.4	142.2	

SAND AND GRAVEL
(PERVIOUS)
FIG. 3

ENVELOPES OF MOHR CIRCLES FOR REPRESENTATIVE SAMPLES OF MATERIAL IN UNCONSOLIDATED QUICK SHEAR TESTS

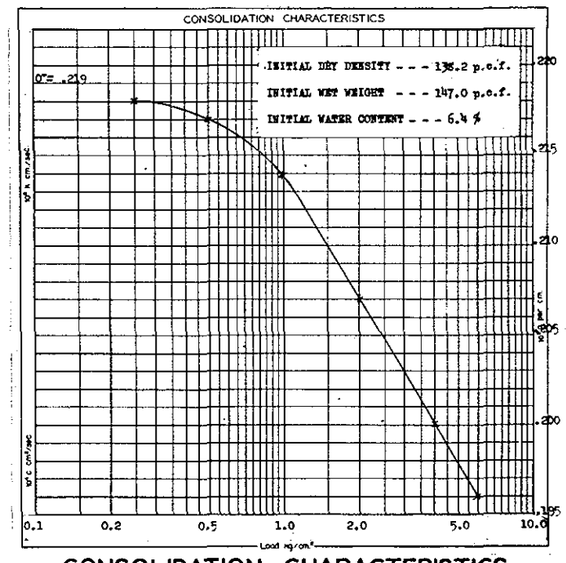
IDENTIFICATION			NATURAL PROPERTIES						REMOLDED PROPERTIES					
									COMPACTION CHARACTERISTICS					
									LABORATORY TEST DATA (a)					
TYPE	OCCURRENCE	CLASSIFICATION	WATER CONTENT w % dry wt.	DRY DENSITY d p.c.f.	SPECIFIC GRAVITY	VOID RATIO e from cols. (E) & (F)	PERMEABILITY k 10^{-4}cm/sec	SHEAR STRENGTH φ deg.	MIN. DRY DENSITY p.c.f.	MAX. DRY DENSITY p.c.f.	OPT. WATER CONTENT w % dry wt.	TOTAL SAMPLE MAX. DRY DENSITY p.c.f.	PERMEABILITY k 10^{-4}cm/sec	SHEAR STRENGTH φ deg.
A	B	C	D	E	F	G	H	I	J	K	L	M	N	O
Foundation Materials	Unconsolidated Surface Deposits	Variable gravelly and silty SAND	21.2(b) (21) 3.5 - 37.4	100.2 (24) 76 - 137	2.68 (7) 2.63 - 2.71	0.67 (24) 0.22 - 1.20	0.5 - 50.0 est. from col.N	35° - 50° est. from Fig. 2	105.7 (c) (2) 90 - 121	119.6 (d) (8) 104 - 135	12.4 (e) (6) 8.0 - 17.1	123.5 (8) 105 - 160	0.5 - 50.0 (6) See Plate VI-5 Fig 7	See Fig 2
	Upper zone of TILL	Weathered brown sandy TILL	9.4 (5) 8.2 - 10.7	128.0 (6) 116 - 131	2.76 (2) 2.75 - 2.76	0.35 (6) 0.31 - 0.48	0.05-0.60 est. from col.N	36° - 41° est. from Fig. 1	-----	132.4 (f) (1) See Plate VI-5 Fig 6	8.2 (1) See Plate VI-5 Fig 6	135.8 (1)	0.05 - 0.60 (2) See Plate VI-5 Fig 7	36° - 41° est. from Fig. 1
	Principal TILL body	Grey compact silty TILL	9.6 (28) 1.1 - 22.5	132.0 (22) 121 - 143	2.71 (8) 2.66 - 2.75	0.28 (22) 0.18 - 0.39	0.02-0.60 est. from col.N	22° - 42° est. from Fig.1	-----	134.0 (f) (3) See Plate VI-5 Fig 6	7.8 (3) See Plate VI-5 Fig 6	137.8 (3)	0.02 - 0.60 (3) See Plate VI-5 Fig 7	See Fig 1
	West Abutment	SAND and SILT	6.7 (8) 3.2 - 14.2	99.8 (10) 75 - 134	2.73 (4) 2.72 -2.73	0.71 (10) 0.27 - 1.27	1 - 100 est. from col.N	37° - 46° est. from col.O	-----	112.0 (f) (1)	12.2 (1)	112.5 (1)	1.5 - 200 (3) See Plate VI-5 Fig 7	Dry 43° - 45° Sat. 37° - 40°
Borrow Materials	Impervious Fill	Silty TILL	11.6 (3) 10.8-12.5	125.3 (3) 123-127	2.71 (2) 2.70-2.71	0.35 (3) 0.33 - 0.37	0.1 - 1.0 est. from col.N	22° - 42° est. from Fig.1	-----	134.0 (f) (1) See Plate VI-5 Fig 6	7.3 (1) See Plate VI-5 Fig 6	135.9 (1)	0.08 - 0.50 (3) See Plate VI-5 Fig 7	See Fig 1
	Random Fill	Silty SAND and sandy TILL	19.1 (13) 5.6 - 37.4	105.4 (15) 69 - 134	2.70 (8) 2.66 - 2.72	0.60 (15) 0.26 - 1.44	1.0 - 50.0 est. from col.N	35° - 50° est. from Fig 2	-----	120.2 (f) (10) 104 - 135	12.4 (5) 8.0 - 17	124.0 (10) 108 - 154	1.0 - 33.0 (4) See Plate VI-5 Fig 7	See Fig 2
	Pervious Fill	GRAVEL and SAND	8.0 (33) 1.3 - 27.5	110.6 (33) 75 - 150	2.69 (16) 2.64-2.76	0.52 (33) 0.12 - 1.24	10 - 100 est. from col.N	37° - 45° est. from Fig 3	104.0 (10) 92.0-114.5	126.6 (d) (10) 105.0-137.6	-----	133.8 (10) 105.0-149.8	10 - 100 (3) See Plate VI-5 Fig 7	See Fig 3

NOTES

(a) Laboratory test data for material passing No. 4 mesh sieve
(b) Figures in ( ) under average data indicate number of tests. Figures below ( ) indicate range of test results
(c) Minimum density obtained by placing dry material in container without compaction or vibration.
(d) Maximum density obtained by Vibrated Density Test.
(e) Total densities are based on laboratory compaction test results adjusted for coarser fraction of sample not included in test specimens.
(f) Modified Proctor Tests.

SUMMARY OF PROPERTIES OF MATERIALS ENCOUNTERED

FIG. 4



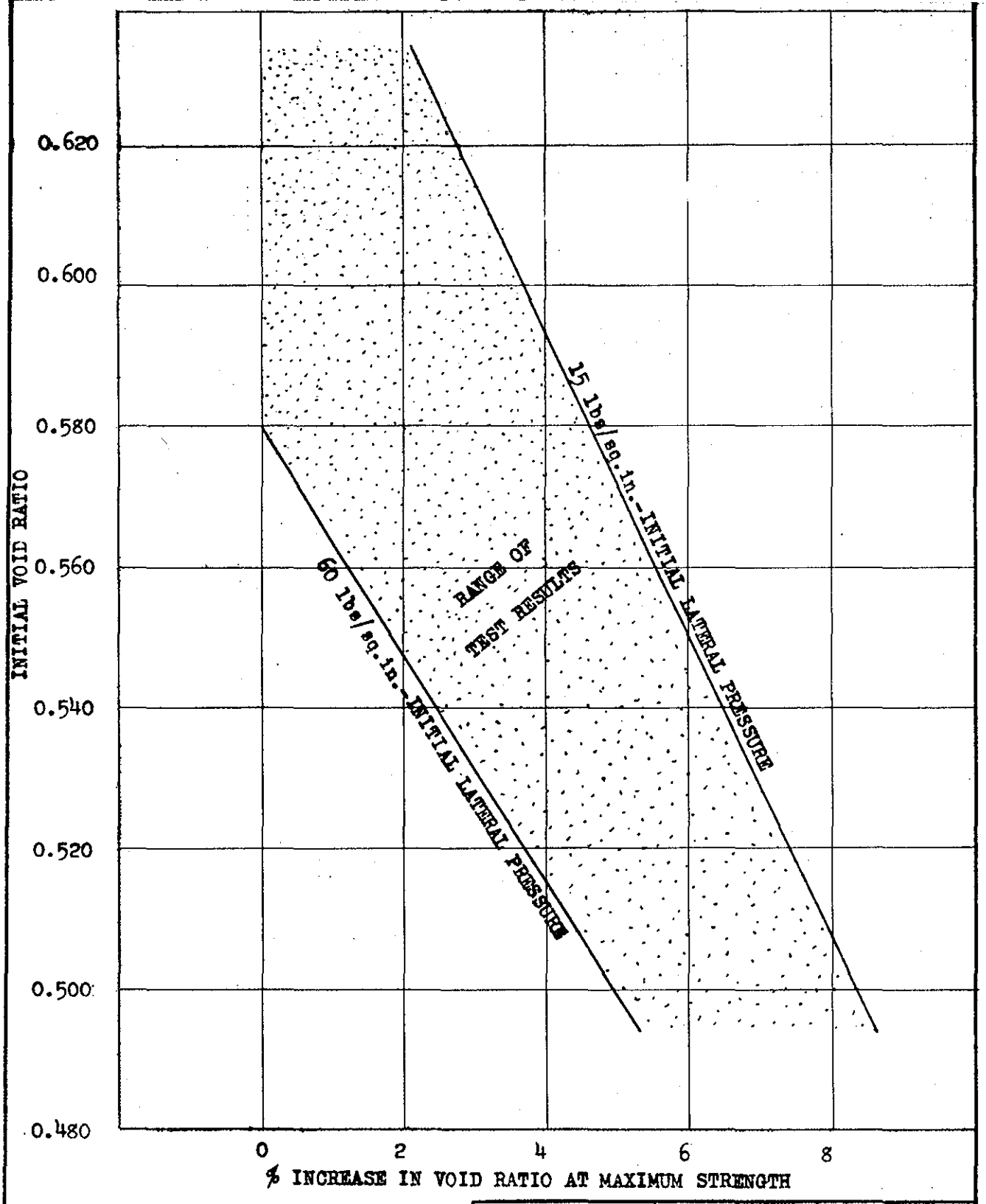
CONSOLIDATION CHARACTERISTICS
FOR SILTY TILL
FIG. 5

MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON DAM
CONTOOCOOK RIVER

SOIL DATA SUMMARY
NO. 2

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

PLATE VI-6



NOTE:
Tests run on Sample #1, T81p (Finer limit of pervious borrow Area C — See Fig. 5, Plate VI - 5)

MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON RESERVOIR
CONTOOCOOK RIVER

CRITICAL VOID RATIO TESTS

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

PLATE VI-7

SECTION VII

DESIGN OF FOUNDATIONS AND EMBANKMENT

7-01. INTRODUCTION.- The design of the foundation and embankment of the dam included studies of foundation conditions and materials that are available for the construction of the dam. The design also included the choice of a section which (1) utilizes economically the materials that are available for the construction of the embankment and (2) is safe under any condition. This section includes a description of the results of those investigations which are pertinent to the design, a discussion of the choice and economy of the embankment section, and the analysis demonstrating that the section is satisfactory for the design criteria detailed below.

7-02. DESIGN CRITERIA.- The following design criteria shall be satisfied:

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

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

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

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

(e) Settlement of the spillway shall not be detrimental to the design height or structure safety.

(f) The slopes shall be adequately protected against surface erosion.

7-03. DESIGN FEATURES.- (a) Embankment Section.- (1) Selected Section.- Details of the sections selected are shown on Plates A-14 and A-15. The main embankment section includes a central core of compacted impervious material, bounded on each side by a transition section of compacted random fill, which, in turn, is covered with an outer zone of compacted pervious fill. The upstream slope is surfaced with dumped rock fill on a gravel bedding; the downstream slope is protected with a cobble facing. A toe drain constructed of selected gravel and covered with a dumped rock fill is provided at the toe of the embankment.

(2) Impervious Features.- The compacted impervious section of the embankment consists of a central core which ties into the till at the east abutment and into the till for a large portion of the valley which the dam crosses. On the west side of the valley, where no till occurs in the foundation and where till is located too far below the surface for economical contact, the depth of that portion of the central

impervious core below the stripping line is reduced and an impervious blanket is added under the upstream portion of the embankment. The blanket extends slightly beyond the upstream toe of the dam and, in the future, should there be any need for more positive control of seepage, the blanket may be economically extended upstream.

(3) Random Features.- The compacted random sections of the embankment have been included in the design to provide a transition between the impervious core and the pervious section of the dam. Dimensions of these random sections have been chosen to utilize the estimated quantity of suitable material from structure excavation at the dam site and of required random excavation in the impervious borrow area.

(4) Pervious Features.- The compacted pervious sections of the dam have been designed to provide adequate stability for the upstream section during rapid drawdown of water elevation. The sections have also been designed to maintain the maximum line of seepage in the downstream sections well below ground surface for conditions of sustained high upstream pool elevation. The selected gravel toe drain is designed to collect water that percolates through and beneath the embankment, to blanket the foundation, and to prevent any uplift or movement of foundation soil. A perforated tile pipe collects and carries off excess water from the selected gravel. The drain details are shown on Plate A-16.

(5) Slope Protection.- The upstream slope of the embankment is protected from wave action and surface erosion by a 5-foot layer of dumped rock fill laid on a 2-foot layer of gravel backing, as indicated on Plates A-14 and A-15. The downstream slope up to 7 feet above maximum tail water elevation is protected by a similar layer of dumped rock laid directly on selected gravel. The slope above the elevation of the top of the rock fill on the downstream slope is protected against surface erosion by a one-foot layer of 3 to 8 inch cobbles, which may be obtained by collecting oversize stones from impervious and random sections and from raking the surface of the selected pervious section of the embankment during construction.

(b) Relief Well System.- As described in par. 6-04 (e), the embankment foundation between station 22+00 and 30+00 has a pervious substratum connected upstream with the reservoir but does not have a natural outlet downstream. A drainage well system has been designed to relieve the hydrostatic head in this stratum. Each well placed at one hundred (100) foot intervals in the system will consist of an eighteen (18) inch diameter hole into which a 6-inch diameter well pipe will be installed. Protective backfill of concrete sand will be provided around the well pipe. The discharge from these wells will be collected and emptied into the discharge channel. Details of the installation are shown on Plate A-16.

(c) Spillway Foundation.- (1) Location.- The location of the combined spillway and outlet structure at the site indicated on the drawings was selected after considerable subsurface exploration work had been carried on in the general vicinity of the structure. A location could not be provided where the bedrock is at an elevation sufficiently high to provide for economical construction. Several locations were

studied for a spillway founded on overburden. From the standpoint of minimizing the settlement of the spillway structure, it was considered necessary to found the spillway on the compact glacial till at a location where the till extended unbroken to rock and where the depth to rock was at a minimum. The location chosen is the only location in the vicinity which meets those criteria and which is suitable from a hydraulic standpoint.

(2) Design Details.- In order to form a working surface upon the top of the till excavation and in order to prevent the top of the excavation from becoming loosened, approximately one foot of concrete will be placed as soon after excavation as practicable. The surface of this concrete will be left rough. The downstream toe and the stilling basin slabs will be founded on a drainage filter which will consist of a layer of sand laid upon the till. The layer of sand will be covered by a layer of screened gravel. A system of open joint pipe will be laid in the gravel layer as shown on Plate A-19. A layer of porous concrete approximately one foot thick will be placed upon the filter to provide protection for the drainage system and a durable working surface, and to facilitate seepage entering the filter layer. The concrete slabs will be placed directly on the porous concrete layer.

(3) Pumping Wells.- The necessity for reduction of hydrostatic head in the bedrock during construction operations is of prime importance. For this reason, the design specifications require the contractor to maintain the area in a dry condition during the construction of the spillway and stilling basin.

(d) Approach and Discharge Channels.- The approach and discharge channels have been designed to pass the required quantities of water. The side slopes, which vary between 1 on 2 and 1 on 2-1/2, have been designed for stability in the event of a rapid drawdown of the water surface in the channel. Where appreciable water velocities occur, riprap protection over a gravel filter has been provided. (See Plate A-2) The thickness and size of riprap has been increased in the zone unusually high and erratic water velocities immediately downstream from the stilling basin. The details of slopes and slope protection are illustrated on Plate A-15.

(e) Upstream Cofferdam.- The cofferdam consists of an earth embankment with a 20-foot top width. The cofferdams will be approximately 25 feet high in the river section and approximately 15 feet high in the land sections (Plate VII - 4). The upstream side slopes of the cofferdams are 1 on 2-1/2 and the downstream side slopes are 1 on 2. The embankment is constructed utilizing selected stripping and random materials from structure excavation. A downstream toe drain of coarse bank run gravel is provided to prevent erosion by water seeping through or under the structure.

(f) Downstream Dike.- Superimposed on the west side of the spillway discharge channel is a dumped rock fill dike, as shown in Plate A-2. This dike acts to guide the spillway discharge away from the main embankment under high discharge conditions. The toe of the dam embankment is thus protected from the possibility of scouring and erosion. The dike slope of 1 on 2 for each face is flat enough to

render the structure safe against possible wave and scouring action of high water.

7-04. AVAILABILITY AND CHARACTER OF MATERIALS.- (a) General.- Sources of construction materials have been comprehensively discussed in paragraph 6-06. The following subparagraphs serve principally to define the gradations and supplement information previously furnished. A materials summary table is given to show the sources of various quantities incorporated in the design.

(b) Compacted Impervious Fill.- Most of this material will come from Borrow Area "A." The till excavated from the spillway site will also be available. This soil is a silty till, consisting of an impervious, well graded gravel, sand and silt, with some clay sizes. Impervious material will conform to the following gradation:

Sieve Size (Tyler)	Diameter, mm.	Percent Finer by Weight
6"		100
No. 10		75 - 95
No. 48		45 - 75
No. 200		20 - 45
	0.005	5 - 20

(c) Compacted Pervious Fill.- Compacted pervious material will be available from the pervious borrow areas and from the coarser structure excavation material encountered. Material will range from a graded sand to a sand and gravel, in accordance with the following table:

Sieve Size (Tyler)	Percent Finer by Weight
6"	100
1"	60 - 100
1/4"	40 - 100
No. 10	25 - 95
No. 48	10 - 70
No. 200	0 - 20

(d) Compacted Random Fill.- Generally speaking, compacted random fill will be composed of materials not complying with the gradation required for either impervious or pervious fill. It will fall between the coarser limit of pervious fill and the finer limit of impervious fill. Structure excavation is expected to provide this material as required.

(e) Gravel Backing.- This material is a sandy gravel conforming to the following gradation:

Sieve Size (Tyler)	Percent Finer by Weight
6"	100
1"	35 - 85
1/4"	20 - 60
No. 10	10 - 35
No. 48	0 - 15
No. 200	0 - 5

(f) Selected Gravel.- This material is available from the coarser range of pervious borrow materials to be found in borrow areas "B-1" or "B-2" and will have the following gradation:

Sieve Size (Tyler)	Percent Finer by Weight
6"	100
1"	60 - 85
1/4"	40 - 70
No. 10	25 - 55
No. 48	10 - 30
No. 200	0 - 5

(g) Processed Aggregates.- Processed aggregates may be obtained at the option of the contractor by establishing and operating a processing plant at Borrow Area "B-1" or from an established processing plant.

(h) Rock for Slope Protection and Dumped Fill.- Rock is available from the oversize obtained from stripping, excavation and from the Bell Ledges which is located about 3 miles from the job site.

(i) Materials Summary.- A summary of (1) required construction materials with their sources indicated, and (2) materials available from required excavations and from selected borrow areas with the disposition of the materials indicated is shown in the following tabulation. Quantities indicated include allowance for shrinkage and settlement.

TABLE I

CONSTRUCTION MATERIALS REQUIRED FOR DAM

Item	Quantity - C.Y. Embankment Measure	Quantity - C.Y. Excavation Measure	SOURCE Excavation Measure - C.Y.	
Compacted Impervious Fill	150,000	172,500	37,000	Structure Excavation
			135,500	Impervious Borrow
Compacted Random Fill	204,000	234,500	200,000	Structure Excavation
			34,500	Random Borrow
Compacted Pervious Fill	174,000	200,000	100,000	Structure Excavation
			100,000	Pervious Borrow
Semi-compacted Fill	100,900	116,000	19,300	Random Borrow
			96,700	Structure Excavation
Hand-compacted Fill	6,000	6,900	6,900	Structure Excavation
Selected Gravel	44,000	50,600	50,600	Special Pervious Borrow
Gravel Backing	34,000	39,100	39,100	Special Pervious Borrow
Gravel Base for Roadway	5,000	6,000	6,000	Special Pervious Borrow
Sand	800	1,000	1,000	Special Pervious Borrow

Item	Quantity - C.Y. Embankment Measure	Quantity - C.Y. Excavation Measure	SOURCE Excavation Measure - C.Y.	
Screened Gravel	4,100	4,700	4,700	Special Pervious Borrow
Cobble Facing	7,500	7,500	7,500	Special Pervious Borrow
Processed Concrete Aggregate	65,000	130,000	130,000	Special Pervious Borrow
Dumped Rock Fill	85,700	68,500	10,500 58,000	Rock Quarry Oversize from Structure & Borrow Exc.
Derrick Stone	5,200	4,200	4,200	Rock Quarry

TABLE II

MATERIALS AVAILABLE FROM EXCAVATION AND BORROW

Item	Quantity - C.Y. Excavation Measure	Disposition - C.Y. Excavation Measure
Common Excavation	496,800	40,000 Spoil 16,200 Oversize usable for dumped rock fill 37,000 Compacted Impervious Fill 200,000 Compacted Random Fill 100,000 Compacted Pervious Fill 96,700 Semi-compacted Fill 6,900 Hand-compacted Fill
Borrow - Impervious and Random	199,300	5,000 Spoil 5,000 Oversize usable for dumped rock fill 135,500 Compacted Impervious Fill 34,500 Compacted Random Fill 19,300 Semi-compacted Fill
Borrow - Pervious	111,100	6,100 Spoil 5,000 Oversize for dumped rock 100,000 Compacted Pervious Fill
Borrow - Special and Processed Sand and Gravel (from Pervious Borrow Areas)	238,900	50,600 Selected Gravel 39,100 Gravel Backing 6,000 Gravel Base for Roadway 1,000 Sand 4,700 Screened Gravel 7,500 Cobble Facing 130,000 Gravel for processed concrete aggregate
Stripping Damsite	92,000	Upstream Cofferdam, Spoil and Seeded Top-soil

Item	Quantity - C.Y. Excavation Measure	Disposition - C.Y. Excavation Measure
Rock Excavation	53,100	21,300 Waste and Spoil 31,800 Process for dumped rock fill

7-05. DESIGN STUDIES.- (a) General.- The dam is designed on the basis of previous practice, which has proven sound. The following paragraphs are devoted to the study of those design phases which require special analysis and which sound engineering practice required to be checked in justification of the final design.

(b) Stability Analysis.- (1) Method.- Following the precedent of the Swidish Geotechnical Committee's study on slope failures in artificial cuts, the stability ratio against shear failure of the dam embankment and its foundation was determined by investigating the forces tending to cause movement (driving forces) and those forces producing potential resistance to movement on several circular sliding surfaces of weakness selected by systematic trial. This method investigates only the possibility of a shear failure. In the analysis, the driving forces include the rotating effect of the weight of the soil mass and water above the surface of failure and the forces generated by seeping water. The force producing potential resistance consists of the shearing strength generated along the sliding surface. The ratio of the potential resisting force to the driving force is called the stability ratio. A sufficient number of potential planes were analyzed to determine the position of the plane having the least stability ratio. The stability ratio of the weakest plane is called the least stability ratio for the section analyzed. A least stability ratio of unity, therefore, indicates equality of driving and potential resisting forces and implies that the embankment is on the verge of failure, while a least stability ratio of greater than unity indicates that the structure possesses reserve strength.

(2) Design Shear Strength.- Design shear strengths to be used were selected on the basis of previous experience with similar materials, and were influenced by consideration of the consolidation and seepage factors associated with each analysis. The tests actually run on these materials, as shown in Figures 1, 2, and 3, Plate VI-6, served to check the assumed values. The design shear strength has been plotted on the figures to offer a comparison with test results. It is seen from Figure 1 that the results of tests on silty till from Test pit T60p, representing the spillway foundation material, show that the design value of 33° for the angle of friction is safe. The low test value of shear strength for the saturated impervious borrow material, as represented by the curve for Test pit T53p, #9, would affect only the analysis of embankment stability, and is not critical. The gravelly and silty sand encountered in embankment foundation random fill and pervious fill showed by test that the design value of 35° for the angle of friction was logical. A higher value of 38° was allowed for the upstream cofferdam design because of the coarser nature of material composing it, and its smaller cross section allowing a more rapid drainage of the slope.

(3) [illegible] section for ultimate development, which includes the section designed for the initial development. It is known by experience that the initial section will have a minimum stability ratio, by the method used, equal to or greater than the minimum stability ratio of the assumed maximum section for the ultimate development. In the analysis, the embankment and its foundation were considered to be intact and sliding surfaces were allowed to pass through foundation and embankment sections without discrimination. The soil characteristics used in the analysis of the various sections are contained on Plate VI-6. The results of the investigations of several potential failure planes in the upstream section of the embankment are shown on Plate VII-1. The minimum stability ratio is 1.06 for the conditions existing immediately after a sudden drawdown of upstream pool. Analyses were made using the method of slices. The usual approximation was made that the soil forces on either side of each slice balance each other. It is assumed that the rock slope protection, the gravel backing, the compacted previous, and compacted random sections will drain as rapidly as the pool is drawn down, and that the compacted impervious section will not drain and is saturated. Assuming that the compacted random section does not drain, the stability ratio decreases to 1.1. The results of the analysis of the downstream section of the dam embankment for the slope to level grounds are shown on Plate VII-1. The minimum stability ratio is 1.38 for conditions of steady seepage.

(4) Spillway Foundation.- Analysis of the stability of the spillway foundation is included in paragraph 7-09(a).

(5) Spillway Excavation Slope.- Analysis was made of the slopes proposed for the excavation of the spillway and outlet structure (Plate VII-1). Three potential failure surfaces were analyzed to show a minimum stability ratio of 1.55. A condition of sudden breakdown in the excavation was seen as well as the [illegible] rock [illegible] from the overlying surface deposits and some rock [illegible] bulk. These slopes are neither satisfactory.

(6) Upstream Cofferdam.- The upstream slope of the main section of the cofferdam was investigated for stability under sudden drawdown of water from the maximum elevation of 660 to elevation 647 (Plate VII - 1). [illegible] stability ratio of 1.10. Since the cofferdam will [illegible] drawdown conditions [illegible] quite conservative, this stability ratio is believed satisfactory.

(7) Flow Failure Analysis.- Flow failure is defined as the liquefaction and movement of mass. A loose, saturated, cohesionless material [illegible] the possibility of flow failure [illegible] that it is highly [illegible] in connection with this project. Either the [illegible] retai[illegible] void [illegible] sampl[illegible] Thus, [illegible] the [illegible] the [illegible]

material would tend to cause tension in the water and increased intergranular pressure. Since the finer range of pervious borrow materials is typical of random materials, the test results are considered representative of the materials conceivably subject to flow failure conditions.

(d) Seepage Analysis.- (1) Embankment.- The seepage beneath and through the earth embankment has been studied by flow nets, Plate VII - 2. Based upon these flow nets, the total seepage through and beneath the embankment, including spillway section, is estimated at one-half cubic foot per second for maximum head for the ultimate section. Of this total, a negligible portion will be discharged beneath the spillway into the collector system and 0.2 c.f.s. through the drainage well system. The greater portion of the remaining seepage will occur beneath the western portion of the embankment in the section containing the upstream impervious blanket.

(2) Spillway.- Seepage beneath the spillway structure is collected by the drainage filter beneath the downstream toe and stilling basin. The seepage water is collected by the open joint drains which discharge into the stilling basin through wells in the side walls. The total quantity of seepage which will be discharged into this system is estimated at 0.0001 c.f.s. for ultimate maximum upstream pool height based upon the flow nets on Plate VII - 1. The maximum hydraulic gradient for flow into the drainage filter is approximately 0.85, which is considered sufficiently less than the critical maximum of 1.0 to be amply safe, assuming no unusual local discontinuities.

(3) Water Drawdown in Spillway Foundation During Construction.- Although the contractor will be responsible for the control of water during construction, it was considered desirable to provide a check on any proposed method by making an analysis of the effect of pumping on the drawdown of water in the spillway foundation. The results of the pumping test described in paragraph 6-03 were used to provide data for making this analysis. The use of three wells, each pumping 10 gallons per minute, shows a probable drawdown of water from Elevation 672.0 to 640.5 near the center of the excavation area. The head of water at the northwest corner, the most critical area of excavation, has a drawdown from Elevation 668.2 to 642.0. The maximum probable level during pumping was found to be below Elevation 648, which compares favorably with the general spillway foundation excavation elevation of 650. Detailed analysis of drawdown conditions is shown on Plate VII - 5.

(4) Upstream Cofferdam.- Analysis of quantity of seepage both through and under the cofferdam in the river section and on the land section has been determined by construction of flow nets for these two cases (Plate VII - 4). The total quantity of seepage that may be expected if the maximum upstream water surface is maintained will be approximately 0.02 c.f.s. From the proportions of the flow net, the required resistance to uplift or flotation at the downstream toe of the cofferdam is assured with a maximum discharge gradient of approximately 0.67 in comparison with a maximum allowable gradient of 1.0.

(e) Settlement Analysis.- (1) Embankment.- Based upon the experience gained as a result of settlement observations on the completed Franklin Falls Dam, the settlement of the compacted embankment at its maximum section is expected to be from two to six inches due to foundation consolidation, which will occur simultaneously with construction, and one to four inches due to consolidation of the compacted impervious section under its own weight, the latter settlement will occur gradually over an extended number of years. Computations based upon the consolidation curve shown on Plate VI - 6 indicate that the embankment will settle from two to five inches under its own weight. This checks with the estimated settlements based on the settlement taking place on Franklin Falls Dam. In order to provide for these settlements and to insure that the dam crest has a level appearance, an allowance of one foot camber has been made in the maximum section.

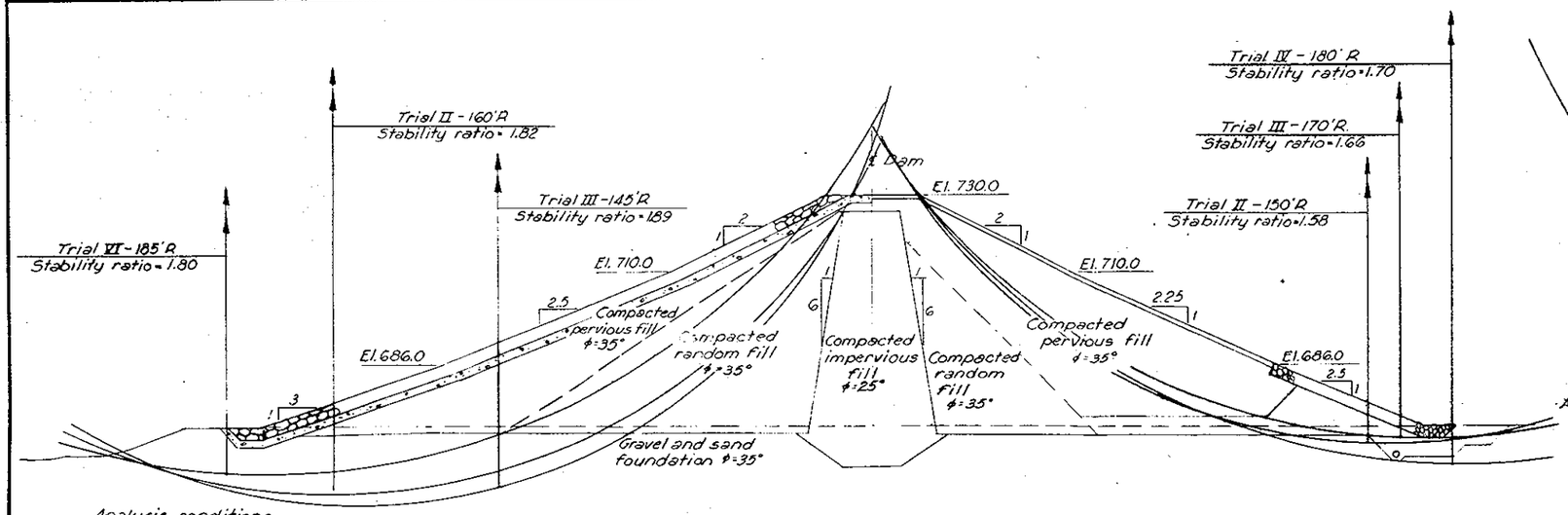
(2) Spillway Foundation.- The spillway foundation of glacial till has been highly preconsolidated to pressures indicated by the geology of the region to be more than 100 tons per square foot. Hence, the spillway settlement will be of the order of one-fifth as great as that computed on the basis of the consolidation characteristics shown on Plate VI-6, in which the pressure-void ratio curve for remolded material is shown. Based on a computed settlement of $1\frac{1}{2}$ inches for remolded till, it is estimated that the spillway foundation settlement will be $\frac{1}{4}$ to $\frac{1}{2}$ inch. The construction procedure of quickly placing a concrete surface on the freshly excavated till foundation, discussed in paragraph 7-03(c), is necessary to minimize the remolding of the excavation surface and hence minimize spillway settlement.

(f) Filter Analysis.- In all sections of the embankment and its foundation through which water passes, a study has been made to insure that the gradation of adjoining soils is such that the finer sizes of one soil will not be transported by the seeping water into the voids of an adjoining soil. In areas where seepage water is collected, the gradations of adjacent soils fulfill the above criteria, and, in addition, the soils become progressively several times more pervious in the direction of discharge. The criteria used for these analyses is contained in Chapter XXI of the Military Engineering Manual, OCE. The determination of the general ranges of materials encountered is illustrated by Plate VII - 3.

7-06. ECONOMY OF CONSTRUCTION.- Economy in the design of the structure has been given careful consideration. The use of an earth dam in preference to that of concrete was dictated by the lack of a suitable foundation for economical concrete dam construction. The sections chosen have been designed to use the impervious, random, and pervious materials encountered in proper proportions to minimize the amount of waste and spoil resulting. All borrow materials required and a source for processing concrete aggregates were found within three (3) miles of the site. It is anticipated that since materials from the structure excavation may be used in the proposed dam and since additional materials required are so readily available, a corresponding savings in the total cost of the project will result.

7-07. CONCLUSIONS.- The foregoing analysis and studies of the proposed design have shown that the structure will satisfy all of the

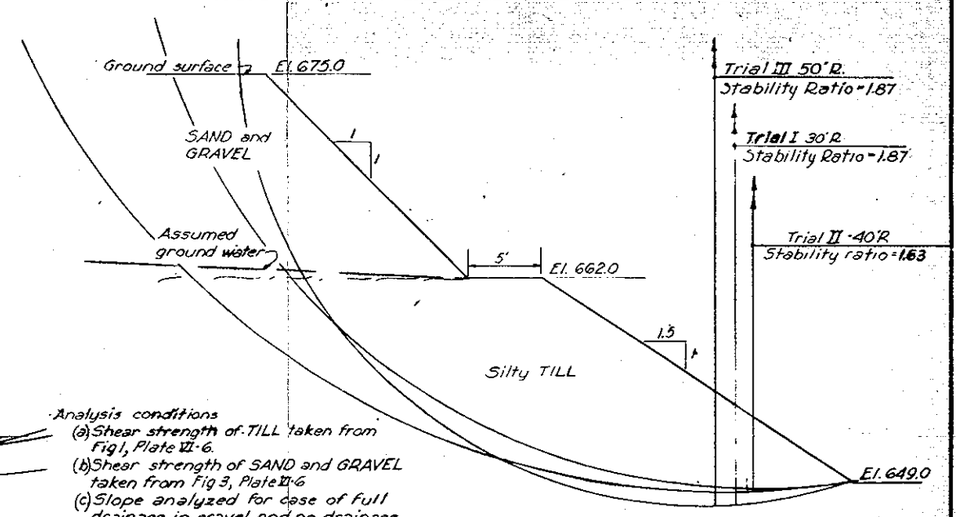
design criteria, and that the sections chosen are well adapted to economical construction.



Analysis conditions
(a) Shear strength of all materials represented by an angle of internal friction of 35°.

(b) Upstream slope analyzed for case of sudden drawdown of pool from El. 717 to El. 667 with full drainage of compacted pervious and random sections.
(c) Downstream slope analyzed for case of sudden drawdown of pool from El. 678 to El. 667 with full drainage of compacted pervious and random sections.

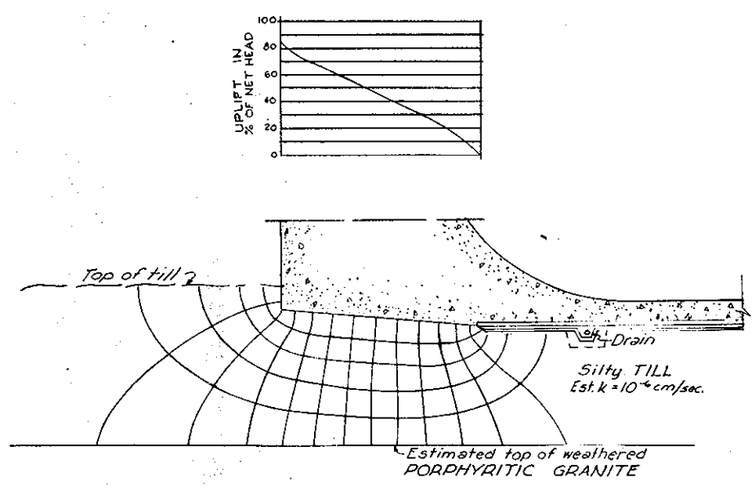
TYPICAL ULTIMATE EMBANKMENT SECTION



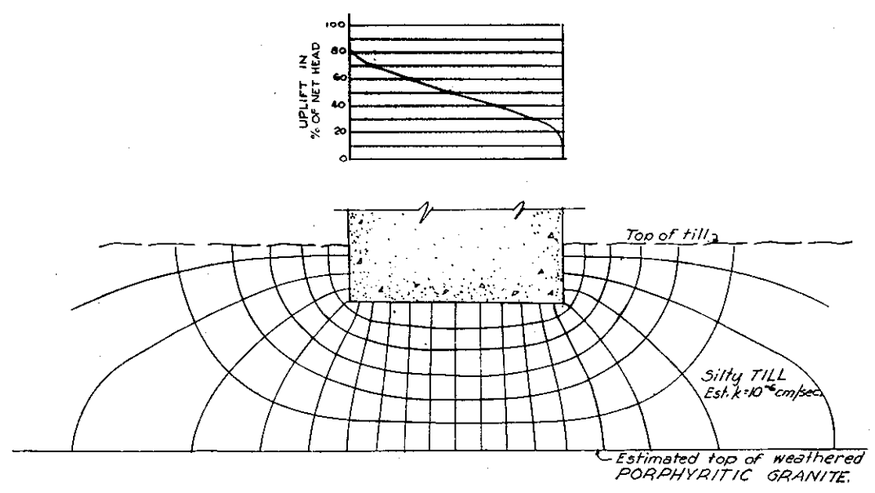
Analysis conditions
(a) Shear strength of TILL taken from Fig 1, Plate VI-6.
(b) Shear strength of SAND and GRAVEL taken from Fig 3, Plate VI-6
(c) Slope analyzed for case of full drainage in gravel and no drainage in till.

SLOPES FOR SPILLWAY EXCAVATION
Scale: 1" = 5'

STABILITY ANALYSES



OVERFLOW SECTION



NON OVERFLOW SECTION

FLOW NETS AND UPLIFT DIAGRAMS FOR SPILLWAY

NOTE:
Elevations based on U.S.C.&G.S. Datum of Mean Sea Level.

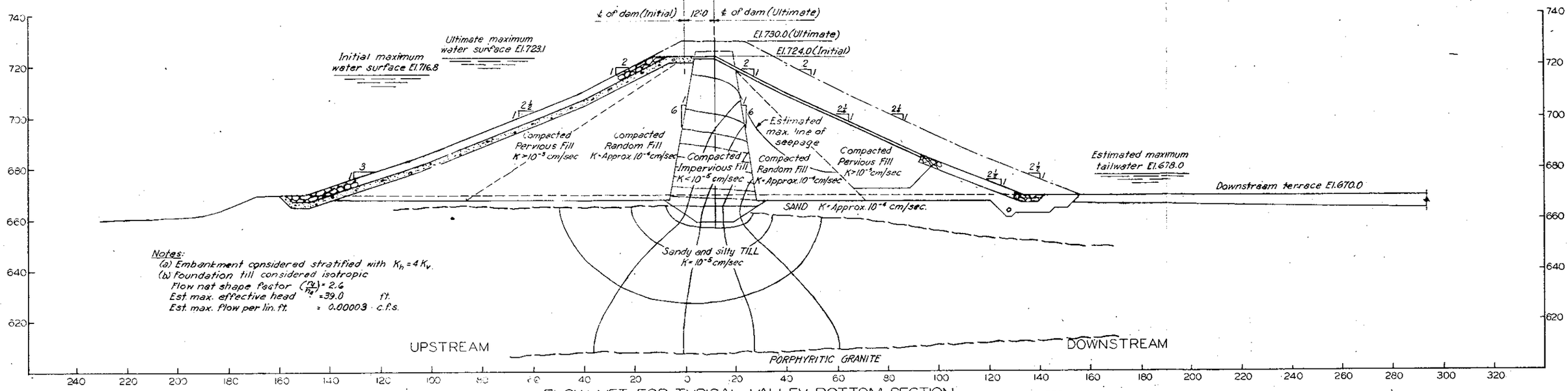
MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON DAM
CONTOOCOOK RIVER

STABILITY AND SEEPAGE ANALYSES

20 10 0 20 40
SCALE IN FEET

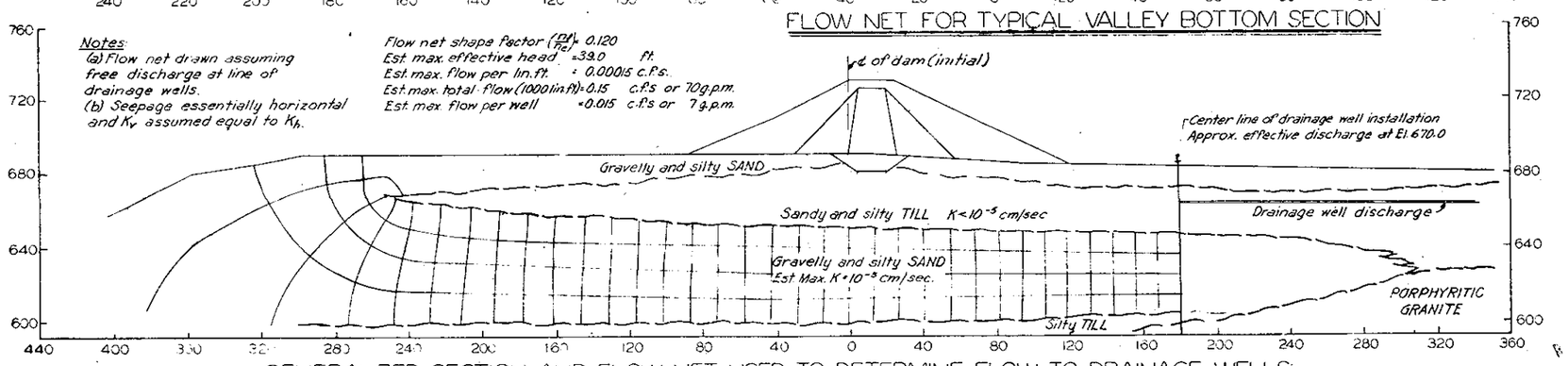
U.S. ENGINEER OFFICE, BOSTON, MASS.

THIS DRAWING REDUCED TO ONE-HALF THE ORIGINAL SCALE



Notes:
(a) Embankment considered stratified with $K_h = 4 K_v$.
(b) Foundation till considered isotropic
Flow net shape factor $(\frac{n_f}{n_d}) = 2.6$
Est. max. effective head = 39.0 ft.
Est. max. flow per lin. ft. = 0.00003 c.f.s.

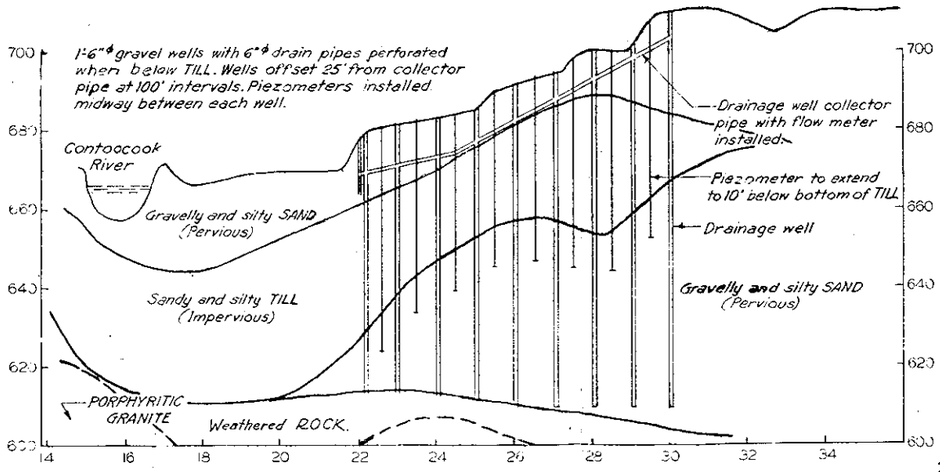
FLOW NET FOR TYPICAL VALLEY BOTTOM SECTION



Notes:
(a) Flow net drawn assuming free discharge at line of drainage wells.
(b) Seepage essentially horizontal and K_v assumed equal to K_h.

Flow net shape factor $(\frac{n_f}{n_d}) = 0.120$
Est. max. effective head = 39.0 ft.
Est. max. flow per lin. ft. = 0.00015 c.f.s.
Est. max. total flow (1000 lin. ft) = 0.15 c.f.s. or 70 g.p.m.
Est. max. flow per well = 0.015 c.f.s. or 7 g.p.m.

GENERALIZED SECTION AND FLOW NET USED TO DETERMINE FLOW TO DRAINAGE WELLS



PORTION OF GEOLOGICAL PROFILE 200 FT. D.S. FROM ℄ OF DAM SHOWING PROPOSED DRAINAGE WELL INSTALLATION

THIS DRAWING REDUCED TO ONE-HALF THE ORIGINAL SCALE

NOTE:
Elevations are based on U.S.C. & G.S. Datum of Mean Sea Level.

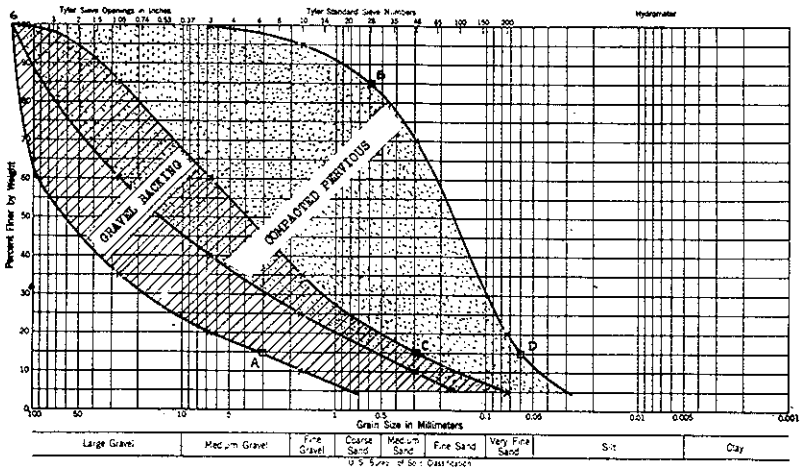
MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON DAM
CONTOOCOOK RIVER

SEEPAGE ANALYSIS

SCALE AS SHOWN

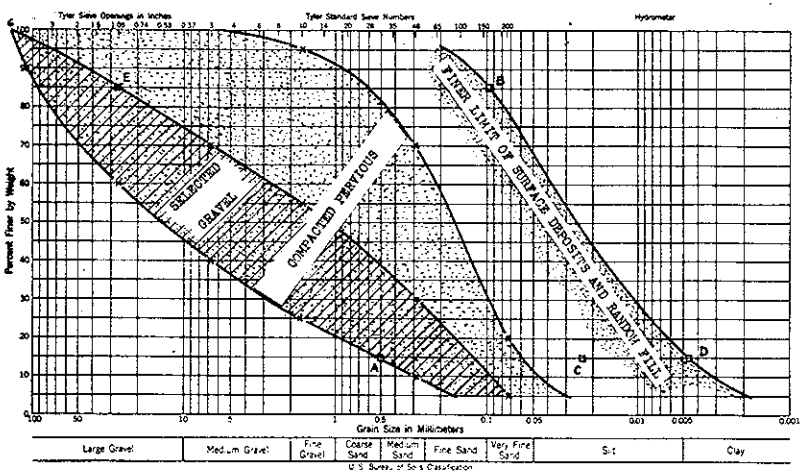
U.S. ENGINEER OFFICE, BOSTON, MASS

FEB. 1946



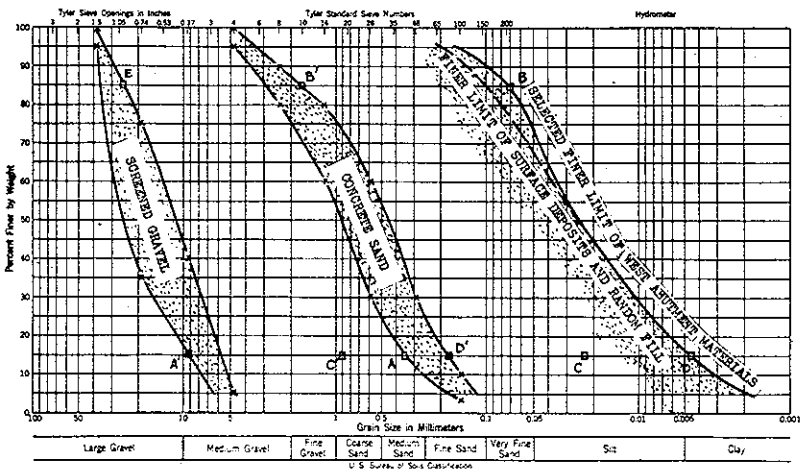
GRAVEL BACKING

FILTER ACTION AGAINST COMPACTED PERVIOUS



COMPACTED PERVIOUS FILL AND SELECTED GRAVEL

FILTER ACTION AGAINST RANDOM FILL AND SURFACE DEPOSITS



COMBINED 3/4" AND 1-1/2" SCREENED GRAVEL

FILTER ACTION AGAINST CONCRETE SAND

CONCRETE SAND

FILTER ACTION AGAINST SURFACE DEPOSITS AND MATERIAL UNDERLYING WEST ABUTMENT

CRITERIA FOR FILTER DESIGN

1. Using a standard gradation curve, a filter layer must meet the following criteria:

a. The 15 percent size (A or A') of the filter must be less than 5 times the 85 percent size (B or B') of the adjoining finer material to be protected.

b. The 15 percent size (C or C') of the filter must be greater than 5 times the 15 percent size (D or D') of the adjoining finer material to be protected.

2. The 85 percent size (E) of filter layer adjoining an open joint, or perforated pipe, should be greater than 2 times the width of slot or diameter of opening.

3. The coefficient of permeability of a filter layer should be not less than 10 times that of the adjoining finer material to be protected.

NOTE:
x symbolizes percentages for gradation control.
□ symbolizes filter control points.

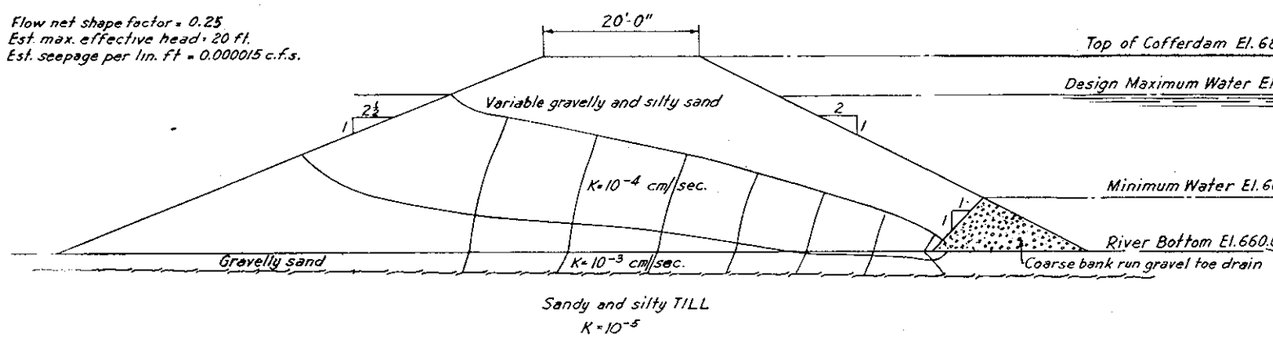
MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON DAM
CONTOOCOOK RIVER

FILTER ANALYSES

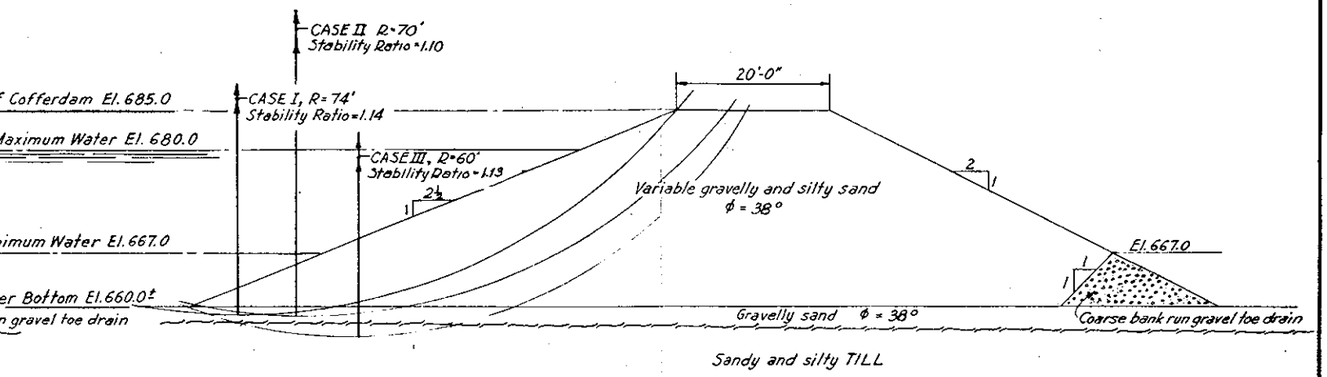
U.S. ENGINEER OFFICE, BOSTON, MASS

FEB. 1946

PLATE VII-3

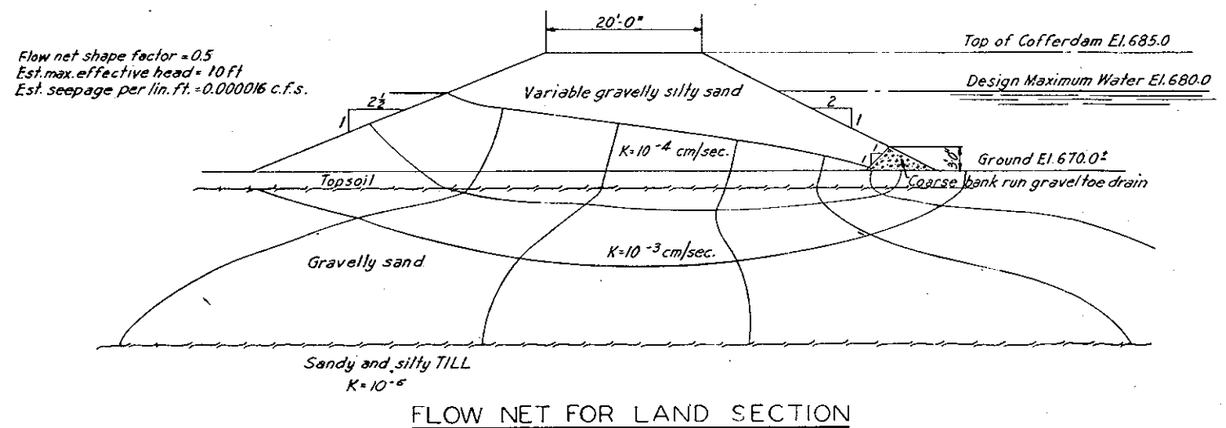


FLOW NET FOR RIVER SECTION



STABILITY ANALYSIS FOR RIVER SECTION

Analysis conditions
(a) Shear strength of TILL taken from Fig 4, Plate VI-6.
(b) Shear strength of gravelly and silty SAND taken from Fig 2, Plate VI-6.
(c) Slope analyzed for case of full drawdown from maximum water.



FLOW NET FOR LAND SECTION

Elevations are based on U.S.C. & G.S. datum Mean Sea Level.

MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON DAM
CONTOOCOOK RIVER

STABILITY AND SEEPAGE ANALYSES
FOR UPSTREAM COFFERDAM

10' 0' 10' 20'
SCALE IN FEET

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

THIS DRAWING REDUCED TO ONE HALF THE ORIGINAL SCALE

WAR DEPARTMENT
U. S. ENGINEER OFFICE, BOSTON, MASS.
Page 1

Subject: ANALYSIS OF WATER DRAWDOWN IN SPILLWAY FOUNDATION
Computation: DURING CONSTRUCTION - BENNINGTON DAM
Computed by: P.J.L. Checked by: J.G. Date: 30 March '46

FORMULA: $H - Z = \frac{Q}{2\pi mk}\left(\log_e R - \frac{1}{n}\log_e X_1 X_2 \ldots X_n\right)$*

H = original height of artesian water table (in feet)

Z = height of water table at dist. X from well being pumped

Q = flow from all wells (g.p.m.) $Q/7.48$ = c.f./min.

m = thickness of water bearing strata

k = coefficient of permeability

R = radius of influence

n = no. of wells pumping

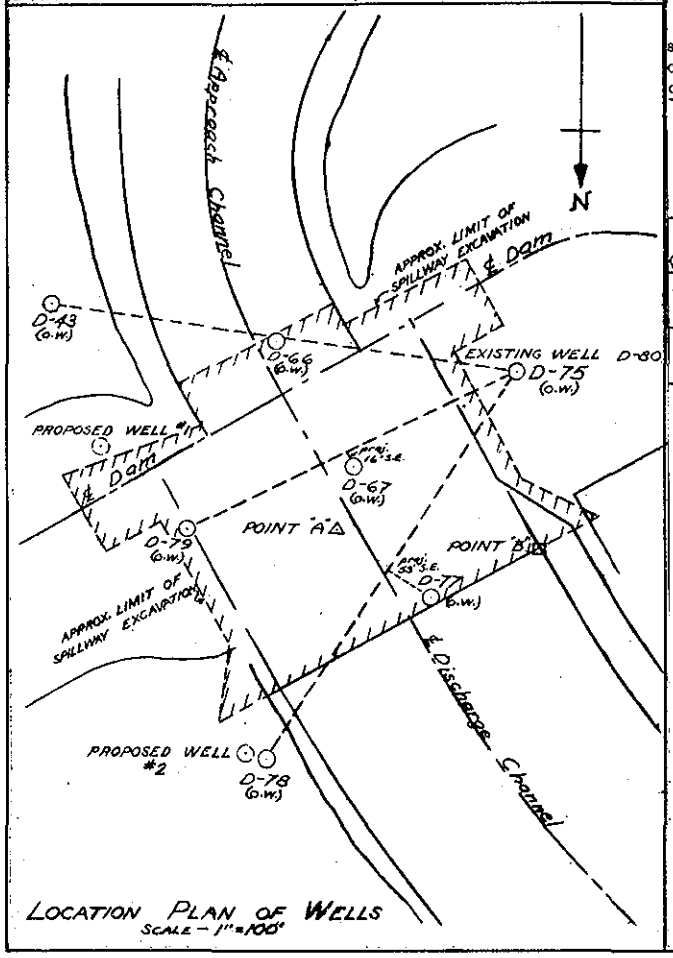
X = dist. of point under study from well being pumped

Note: Use R = 1000 ft. $\log_e R = 6.91$

I. COMPUTATIONS FOR mk VALUES
BASED ON PUMPING TEST OF 15-20 OCTOBER 1945

Well	Q (g.p.m.)	H-Z (ft.)	X (ft.)	$\log_e X$	$\log_e R - \log_e X$	mk (ft.²/min.)
D-67	5	7.97	195	5.27	1.64	0.0219
	10	12.62				0.0276
D-79	5	3.90	375	5.93	0.98	0.0267
	10	6.80				0.0307
D-66	5	4.28	252	5.53	1.38	0.0343
	10	8.23				0.0357
D-43	5	3.55	490	6.20	0.71	0.0213
	10	6.40				0.0236
D-77	5	7.50	245	5.50	1.41	0.0200
	10	11.70				0.0256
D-78	5	4.15	480	6.18	0.73	0.0187
	10	6.75				0.0230

*Formula #109 - Grundwasserabsenkung bei Fundierungsarbeiten von Dr.-Ing. Wilhelm Kurielais



LOCATION PLAN OF WELLS
SCALE - 1" = 100'

WAR DEPARTMENT
U. S. ENGINEER OFFICE, BOSTON, MASS.
Page 2

Subject: ANALYSIS OF WATER DRAWDOWN IN SPILLWAY FOUNDATION
Computation: DURING CONSTRUCTION - BENNINGTON DAM
Computed by: P.J.L. Checked by: J.G. Date: 30 March '46

COMPUTED VALUES OF mk

	PUMPING AT 5 g.p.m.	PUMPING AT 10 g.p.m.
AVERAGE	0.0238	0.0277
MAXIMUM	0.0343	0.0357
MINIMUM	0.0187	0.0230

II. COMPUTATIONS FOR AMOUNT OF DRAWDOWN*

POINT		Q (g.p.m.)	mk (cf²/min.)	$X_1 X_2 X_3$ (ft.)	$\log_e 1000$	$\log_e X_1 X_2 \ldots X_n$	H-Z (ft.)
A	Max.	30	0.0230	255×255×255	6.91	5.54	38.0
	Min.	"	0.0357	"	"	"	24.5
	Ave.	"	0.0277	"	"	"	31.5
B	Max.	"	0.0230	190×370×460	"	5.77	31.6
	Min.	"	0.0357	"	"	"	20.8
	Ave.	"	0.0277	"	"	"	26.2

*COMPUTATIONS BASED ON:-
EXISTING WELL D-80
PROPOSED WELL #1
PROPOSED WELL #2
} PUMPING RATE - 10 G.P.M. each

WAR DEPARTMENT
U. S. ENGINEER OFFICE, BOSTON, MASS.
Page 3

Subject: ANALYSIS OF WATER DRAWDOWN IN SPILLWAY FOUNDATION
Computation: DURING CONSTRUCTION - BENNINGTON DAM
Computed by: P.J.L. Checked by: J.G. Date: 30 March '46

DRAWDOWN RESULTS (THREE WELLS PUMPING)
RATE OF PUMPING - 10 G.P.M. EACH

POINT	ELEV. OF WATER HEAD BEFORE PUMPING	MAXIMUM DRAWDOWN	MINIMUM DRAWDOWN	AVERAGE DRAWDOWN	ELEVATION OF WATER HEAD AFTER PUMPING		
					MAXIMUM	MINIMUM	AVERAGE
A	671.5	38.0	24.5	31.5	647.0	633.5	640.0
B	668.2	31.6	20.8	26.2	647.4	636.6	642.0

MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON DAM
CONTOOCOOK RIVER

WATER DRAWDOWN IN SPILLWAY FOUNDATION

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

PLATE VII-5

SECTION VIII. STRUCTURAL DESIGN

8-01. GENERAL.-This structural analysis includes the design criteria for all masonry structures in the spillway area and the Equipment and Maintenance Buildings. All masonry structures are designed for the proposed ultimate construction. Provision is made in the design of the initial structure to facilitate the construction of the ultimate structure. The structures have been designed in accordance with accepted engineering practice. Consideration has been given in the design to simplicity of construction and durability. The latter is an important factor as the winters in Bennington are severe. Satisfactory hydraulic action and pleasing appearance were other factors considered in the design of the structures. The stability analysis of the masonry structures is shown on Plate VIII-1.

8-02. DESIGN CRITERIA.-(a) Code.-In general, the concrete and reinforced concrete design has been governed by the recommendations of the 1940 Joint Committee Report, "Recommended Practice and Standard Specifications for Concrete and Reinforced Concrete", and Chapter XII and Part X, Chapter I of the Engineering Manual for Civil Works. The structural steel design is in accordance with the American Institute of Steel Construction, "Specification for the Design, Fabrication and Erection of Structural Steel for Buildings", and Part X, Chapter I of the Engineering Manual for Civil Works.

(b) Assumed Loads.- In order to ascertain the extreme conditions of loading, the hydraulic structures have been analyzed assuming (1) the reservoir empty and (2) full surcharge storage and maximum tailwater. The forces acting on these structures consist of the dead weight of the structure, hydrostatic pressure (horizontal, vertical and uplift), and vertical and lateral earth pressures. In addition to this loading, the spillway and non-overflow structures were investigated for an additional loading as a result of an earthquake with a full and empty reservoir. The Equipment building was analyzed for the dead loads of the structure, snow load, the equipment installed or stored therein and live load. The Maintenance Building was designed in accordance with standard practices for wood frame construction.

(c) Uplift Pressures.- Uplift pressures were determined from the flow net diagrams which were prepared for the spillway and abutment sections, as indicated on Plate No. VII-1. Saturation levels for the wall sections were assumed consistent with the location, backfill and drainage conditions immediately after a sudden drawdown. Uplift pressures were considered as acting over 100% of the base area.

(d) Lateral Earth Pressures.-The lateral earth pressures used in the design were determined by the formula $h = cw$, where "h" is the unit lateral pressure, "w" is the unit weight of earth, and "c" is a factor determined by the formula "$c = \frac{1-\sin \phi}{1+\sin \phi}$" in which "$\phi$" is the laboratory-determined angle of internal friction.

(e) Concrete.-(1) Class of Concrete.-Five (5) classes of concrete are used in the construction of the masonry structures. The

B

[illegible] six (6) inches [illegible] three-quarters inch (3/4") in order to obtain the [illegible] durability [illegible] economy. [illegible] aggregate gradations and locations in [illegible] structure [illegible] follows:

Class 1 - Six-inch (6") aggregate will be used in the West Discharge Channel Wall to Elevation 674 East Discharge Wall to Elevation 678, Abutment Sections and Gated Spillway Monoliths to Elevation 662 and in the ungated monoliths, Stilling Basin Floor Slab and End Sill.

Class 2-Three-inch (3") aggregate will be used in the Stilling Basin Baffles, West Discharge Channel Wall above Elevation 674, East Discharge Channel Wall above Elevation 678 Gated Spillway Monoliths above Elevation 662 and in Stilling Basin Baffles.

Class 3 - 1-1/2" aggregate will be used in the Equipment Building Piers, Platforms and Slabs on Embankment, Tile Gage Sections.

Class 4 - 3/4" aggregate will be used in the Equipment Building substructure, Adit Walls, Roof Slab Building.

Class 5 - Porous Concrete will be used under the Stilling Basin Floor Slab, Discharge Channel Walls and downstream toe of Spillway.

(2) Compressive Strength of Concrete.- The design of all concrete structures is based on concrete having a 28-day strength of 3,000 pounds per square inch and a ratio of the modulus of elasticity of steel to concrete equal to 10/1.

8.03. STRUCTURE ANALYSIS.-(a) Spillway.-(1) The spillway structure is a wide-base gravity type ogee section founded on glacial till and is designed to withstand the forces that will be applied to the ultimately constructed section. The gated and ungated monoliths of the spillway have been analyzed for the two worst conditions, namely, with reservoir full and with the reservoir empty. The stability analysis assuming the reservoir full is based on a water level in the reservoir equivalent to the extreme high water resulting from the spillway design flood and corresponding tailwater. The uplift pressures have been determined from the flow net diagrams on Plate VII-1 and are equal to the full effective head applied to 100% of the base. The resultant falls within the middle third for both conditions of loading with a maximum base pressure of 8,530 pounds which is within the allowable load limit for the foundation.

(2) In addition to the above analysis, both sections have been investigated with earthquake loadings added to the normal loadings for the reservoir full and empty. The computed resultant for the reservoir empty falls within the middle third and for the reservoir full the resultant falls just outside the third point for both the gated and ungated sections. Due to the remoteness of any possible earthquake shock in this location at the same time that the reservoir is full, the sections were not increased to provide for the resultant to fall within the middle third of the base.

(3) The shearing strength of the till used in computing the sliding resistance of the spillway sections was obtained from laboratory tests. The results of these tests are shown on Plate No. VI-6. The resulting factor of safety against sliding is 1.81 for the gated section and 1.93 for the ungated section.

(4) The spillway has been dived into ten (10) monoliths of thirty (30) feet each. In accordance with the recommendations contained in the Engineering Manual for Civil Works, keys have not been provided in the construction and contraction joints.

(5) The base of the spillway is sloped to allow the free drainage of any seepage water in the foundation prior to the placing of the base course of concrete. A one-foot layer of porous concrete will be placed under the downstream toe, which, with the gravel drain, aids in the collection of any foundation seepage and allows free drainage to the collector pipes under the stilling basin. A twelve-inch (12") concrete mat is provided over the entire foundation area of the spillway immediately after the excavation is completed to prevent the foundation material from becoming soft and to facilitate the movement of equipment during the placing of the remainder of the concrete.

(b) Abutment Sections.- (1) The abutment or non-overflow sections are wide base gravity type sections and are founded on glacial till. These structures are designed on the basis of the proposed ultimate construction. Provision has been made in the initial construction for the future addition of six (6) feet in height to formulate the ultimate project. The section analyzed for stability is the deepest section which is located adjacent to the spillway. The analysis of the section is based on the maximum water level reached during a spillway design flood with maximum tailwater. The uplift pressures have been determined from the flow net diagrams shown on Plate No. VII-1, and are equal to the full effective head applied to 100% of the base. Under these loadings the resultant falls within the middle third of the base and gives a maximum foundation pressure of 8,000 pounds per square foot. The computed resultant for the section, with the reservoir empty, falls within the middle third of the base and gives a maximum base pressure of 9,100 pounds per square foot which is within the allowable load limit for the glacial till foundation. The factor of safety against sliding is 2.1 which factor is well above the permissable minimum of 1.5. With the additional earthquake loads applied, the resultant falls within the middle third of the base for the reservoir empty and just outside the middle third for the reservoir full. As in the case of the spillway, the section was not increased to bring the resultant back within the middle third with the reservoir full.

(2) The base of the abutment section is sloped to allow the free drainage of any seepage water in the foundation prior to the placing of the one-foot base course of concrete. The abutment section is divided into monoliths with lengths of approximately thirty-five (35) feet.

(3) A one-foot mat of concrete is provided for the base as soon as possible after excavation is completed to prevent the foundation from becoming soft and to facilitate the pouring of the remainder of the concrete.

(c) Stilling Basin Walls.- (1) A gravity-type concrete section was selected for the discharge channel walls because of its greater durability and approximately equal cost to reinforced counterfort walls. The stability analysis for the wall section, as shown on Plate No. VIII-1 is based on the lateral pressure of the earth retained to berm height and normal tail water occurring shortly after a design flood in which the saturation line would be 9.5 feet above normal tail water in the embankment fill. The uplift pressure used is based on 100% of the tail water head applied at the toe and increasing uniformly to 100% of the hydrostatic head at the heel, based on the saturation line, and is applied to 100% of the base. Under these conditions, the resultant is held within the middle third of the base and the factor of safety against sliding is based on the shearing strength of till obtained from the curve on Plate VI-6.

(2) To decrease the uplift and lateral pressure on the spillway walls immediately after drawdown, to allow free drainage of the embankment and to stabilize the foundation immediately after excavation, one foot of porous concrete has been provided as a base course for the walls, except at the well sections. The bases of the walls are sloped to allow for the drainage of any seepage water prior to the placement of the porous concrete. The maximum length of a wall monolith is forty-five (45) feet.

(d) Stilling Basin Slab.-(1) The stilling basin slab is a reinforced concrete slab with a one-foot base course of porous concrete founded on a mat of screened gravel and sand for drainage and the reduction of uplift. The design of the slab is based on a slab thickness computed to withstand the uplift and impact pressures to which it is subjected with an additional increment added for cavitation and spalling.

(2) In order to reduce the uplift pressures, the stilling basin is drained by a six-inch (6") layer of sand and a six-inch (6") layer of screened gravel founded on till. A one-foot (1) layer of porous concrete is provided over the filter as a base for the structural slab. Three (3) perforated tile pipes serving as collectors are located transversely at the third points under the slab and are connected with the sand and gravel layer. The drains then connect with the wells provided in the retaining walls. The wells discharge into the stilling basin thereby providing a system for relieving any excess head. The slab is divided into monoliths thirty (30) feet wide. The monoliths vary from thirty-five (35) feet to forty-five (45) feet in length. Conforming to the opinion of the Board of Consultants, weep holes were not provided in the slab.

(e) Equipment House.-(1) The Equipment House consists of a superstructure above berm height and a basement. The Equipment House is located on the east wall of the discharge channel. The structure is supported entirely by means of piers on one monolith of the discharge channel wall. This section of wall has been analyzed separately for the additional weight of the structure assuming the same hydrostatic and dry loading conditions used in the wall design.

(2) The superstructure houses the standby generators, switch board, oil pumps, toilet facilities and a wash room and is equipped

with a monorail and chain hoist for handling a gate or other heavy equipment. The roof is a reinforced concrete slab which is supported by reinforced concrete beams which transmit the load to the twelve-inch face brick side walls. The architecture of the structure is simple in order to have the building blend in with the surrounding massive concrete structures and the simple design of other structures in the vicinity. The roof is designed to withstand a snow load of 50 pounds per square foot in addition to the dead load. The floor slab is designed to withstand a live load of 450 pounds per square foot in addition to the dead load. All interior partitions are 8" brick walls. The chain hoist and monorail have a rated lifting capacity of 2 tons.

(3) The substructure or basement consists of a reinforced concrete floor slab and walls and contains the electrical transformers, heater and oil storage tanks. The passageway to the gate operating chambers in the east discharge channel wall enters the basement and facilitates the removal of the gates or other equipment from the operating chambers to the work shop of the Equipment Building. The basement floor slab is designed to withstand the live load of equipment superimposed on it and the dead weight of the floor slab.

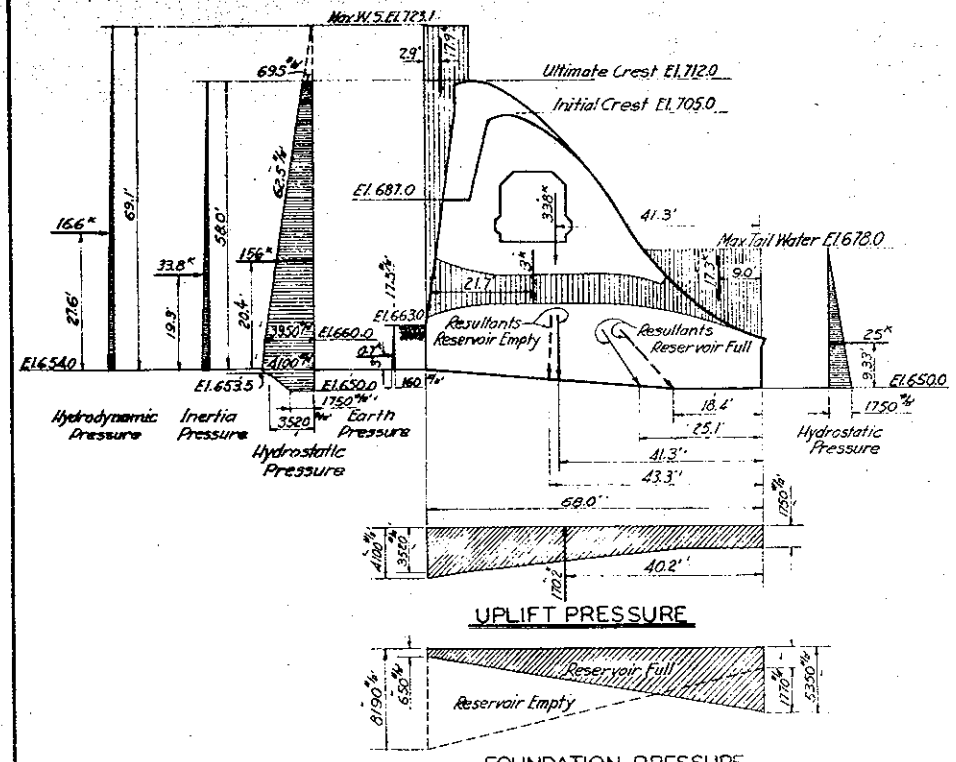
(f) Maintenance Building.- (1) The Maintenance Building is designed to conform with the prevailing type of architecture of houses in the locality and with the dam operator's proposed quarters. The structure is primarily laid out for a permanent maintenance and storage building. Because there is no suitable structure available in the vicintiy for a field office, it is proposed to utilize the building as a temporary field office during the construction period.

(2) The building consists of concrete block walls, wood roof with asphalt shingles, concrete foundation walls and floor slab, and clapboard siding on the ends above the eave line.

(3) The interior partition walls around the heater room and the oil and paint storage room and the partition wall between the three-stall garage and the shop and storage portion of the building are constructed of concrete block. The remainder of the interior finish, including the temporary partitions, constructed in the building while it is in use as a field office is gypsum board, except that the partitions for the toilet and shower room are constructed of cement-asbestos board.

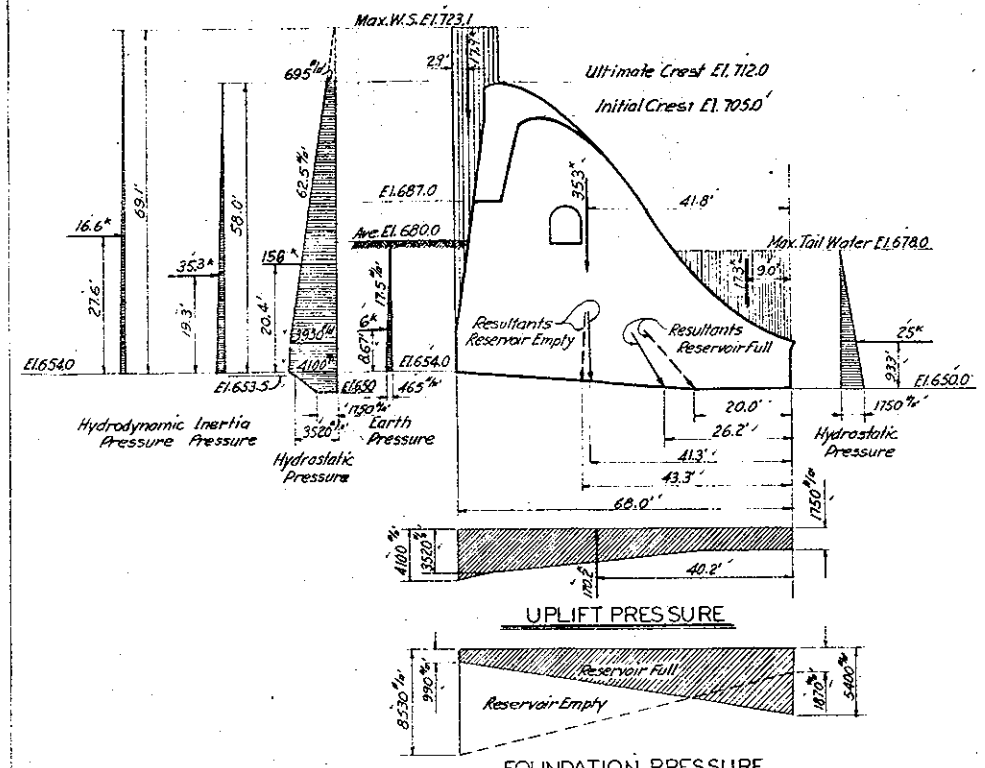
8-04. ACCESS ROADS.-(a) Main Access Road.- The main access road is a first-class bituminous paved road, 18' wide, and is located on the easterly side of the river and connects the proposed relocated second-class surfaced highway with the berm on the downstream side of the embankment. This road serves the Equipment Building, Maintenance Building and operator's quarters and provides an entrance to the dam site from the larger villages in the vicinity.

(b) West Access Road.-A 12 foot gravel access road with three (3)foot shoulders is provided on the westerly side of the dam for the access of maintenance equipment and personnel to the top of the earth embankment. This road leads off the present U.S.Highway Route No. 202 at the toe of the embankment and extends along the toe to the top of the embankment. This road will not be open to the public and is to be used exclusively for maintenance operations.



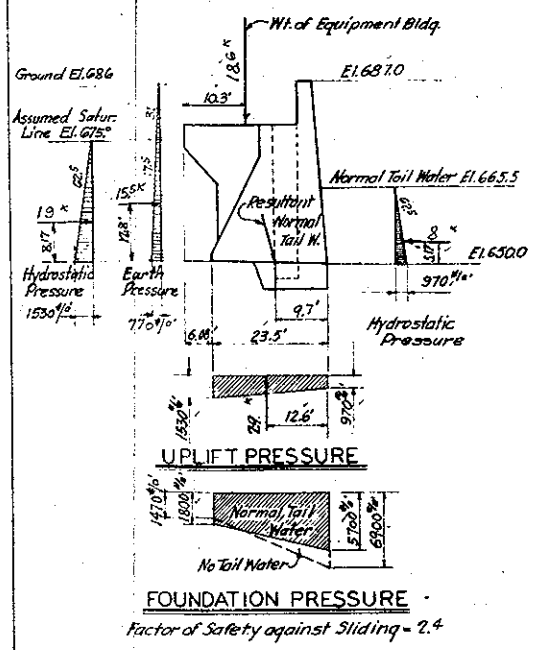
Factor of Safety against Sliding = 1.81

GATED SECTION



Factor of Safety against Sliding = 1.93

UNGATED SECTION



Factor of Safety against Sliding = 2.4

MONOLITH WITH EQUIPMENT BLDG.

DESIGN DATA

Scale of pressures:
Horizontal pressures: 1" = 6000 pounds per sq. ft.
Foundation & uplift pressures: 1" = 6000 pounds per sq. ft.

Reservoir Full:
Ultimate Development Max. W.S. El. 723.1
Initial Development Max. W.S. El. 716.8
Maximum Tail Water El. 678.0

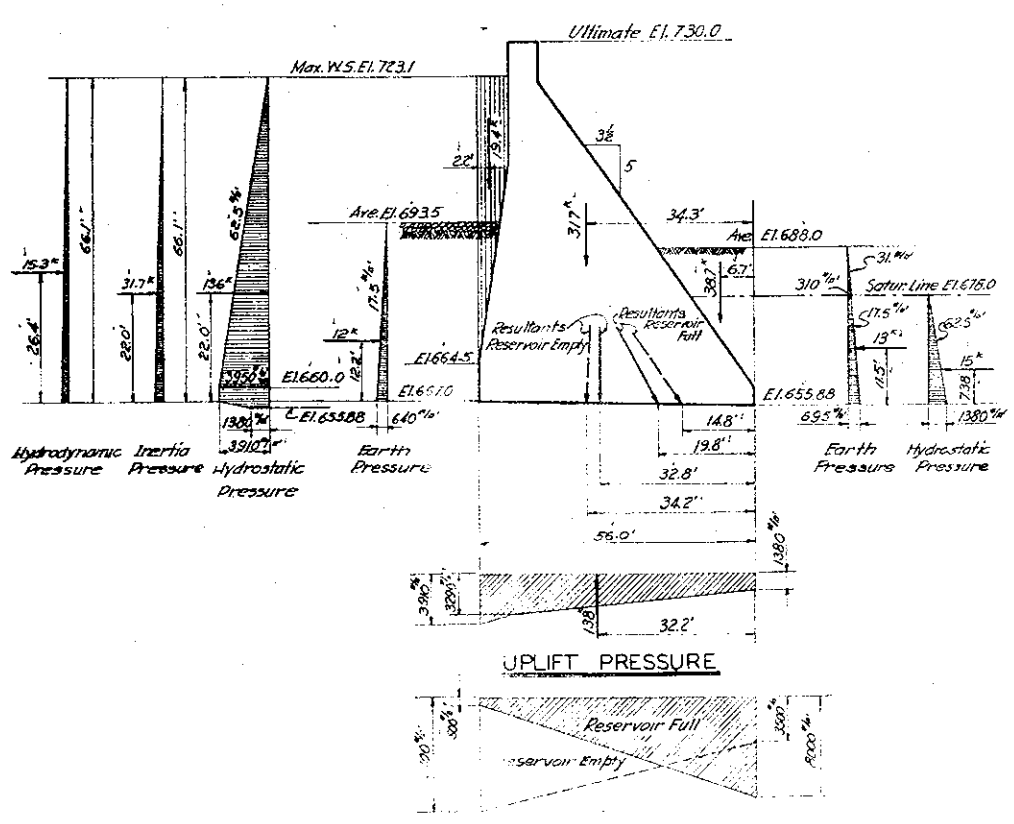
Weight of Material:
Weight of concrete = 150 pounds per cu. ft.
Weight of water = 62.5 pounds per cu. ft.
Weight of earth (dry) = 125 pounds per cu. ft.
Weight of earth (submerged) = 80 pounds per cu. ft.
Horizontal pressure of earth (dry) = 31 pounds per sq. ft. of depth.
Horizontal Pressure of earth (submerged) = 80 pounds per sq. ft of depth.

Pressures and Loads indicated are average pressures and loads per foot of sections and are proportioned to the length of Monolith on which they act.

Spillway Uplift:
Uplift pressures have been determined from the flow net study and are equal to the full effective head applied to 100% of the base area.

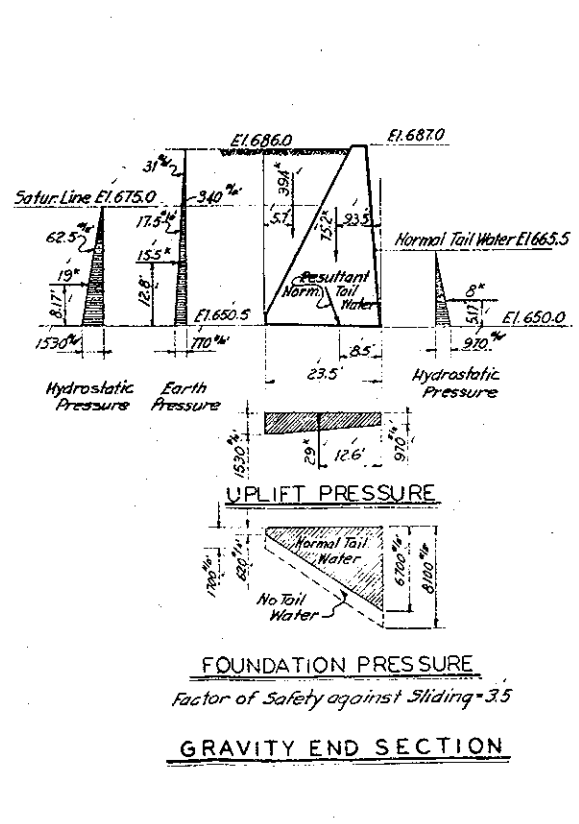
Spillway Sliding:
Foundation pressures & sliding factors do not include forces due to earthquake.
Earthquake forces have been applied acting downstream for full Reservoir condition and upstream for empty Reservoir condition.
The shearing strength of Till used in computing resistance to sliding was obtained from diagram, "Shear Strength of Till" see Stability Analysis Sheet G1.

—— Resultant with Earthquake.
—— Resultant without Earthquake.



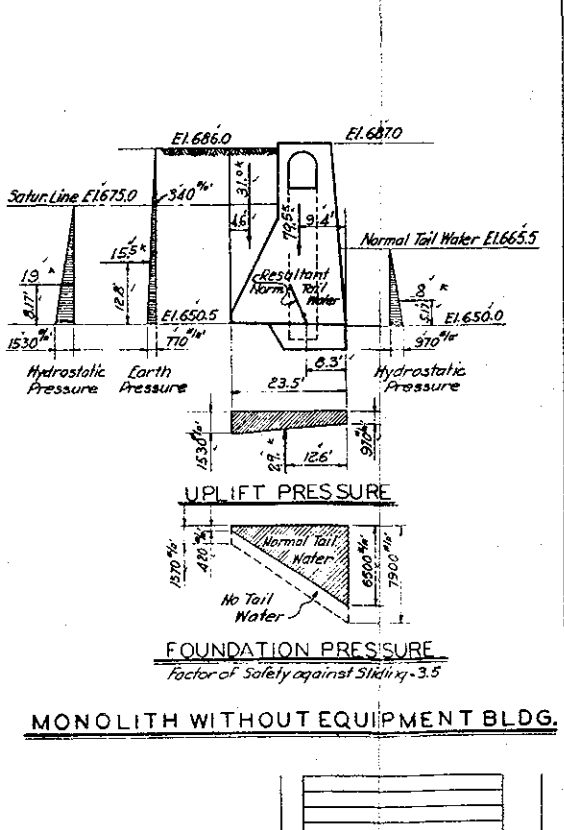
Factor of Safety against Sliding = 2.1

GRAVITY NON OVERFLOW SECTION



Factor of Safety against Sliding = 3.5

GRAVITY END SECTION



Factor of Safety against Sliding = 3.5

MONOLITH WITHOUT EQUIPMENT BLDG.

NO.	CHARACTER	DATE	BY
	REVISIONS		

MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON DAM
CONTOOCOOK RIVER
STABILITY ANALYSIS
MASONRY STRUCTURES

IN SHEETS — SHEET NO. — SCALE: AS NOTED (10' 0' 10' 20')

U. S. ENGINEER OFFICE, BOSTON, MASS.

APPROVAL RECOMMENDED: John E. Allen, Chief Engineering Division
APPROVED: Colonel, Corps of Engineers, District Engineer
PREPARED: Wm Hall
CHAS. T. MAIN, INC.
DES. BY W.A.B. DR. BY D.A.B. TR. BY E.A.M. CK. BY R.A.C.
FILE NO. M19-12/32

PLATE VIII-1

SECTION IX, OPERATING EQUIPMENT

9-01. INTRODUCTION.-The purpose of this section is to outline the basis for the selection of the various items of equipment which are specified in the construction plans and specifications and to briefly describe the features of the equipment selected.

9-02. GATES IN OUTLET WORKS.-A gravity type ogee section spillway with conduits incorporated in the spillway section was selected as the most economical method of providing an outlet works consisting of a spillway weir and a control works. This type of construction limits the selection of the type of control gate to a hydraulically operated slide gate. Inasmuch as the proposed structure is a low head dam with a maximum head of 45 feet, the simplest form of slide gate, consisting of a cast iron leaf and frame and combination bronze slide and seal, is recommended. Because the winters in the vicinity of the damsite are severe, oil, in preference to water, was selected as the operating medium for the hydraulic rams. There are no innovations in the design selected. Design features are practically identical with the design features of the gates now installed in the Blackwater Dam which was constructed during 1940 under the supervision of the U. S. Engineer Office, Boston, Mass. The gates installed in the Blackwater Dam have proven to be entirely satisfactory. Allowable working stresses adopted in the selection of the gates are considered very conservative. See Appendix "B".

9-03. OIL PRESSURE SYSTEM.- (a) Description.- The oil pressure system is designed to deliver approximately 10 g.p.m. of oil to the rams at a maximum operating pressure of 500 p.s.i. The system includes dual positive displacement pumps driven by constant speed electric motors, a combination pump mounting base, an expansion tank reservoir, pump selection valves, pneumatic shock absorber, pressure control by-pass valve, pressure and return mains, ram control valves and branch oil lines to rams.

(b) Design Features.- Rotary positive displacement type pumps were selected in lieu of reciprocating type because of the greater simplicity of design and lower installation cost of the rotary type pump. The expansion reservoir tank is incorporated in the pump mounting base rather than as a separate unit. This provision decreases the amount of exposed piping and provides a neater layout. Lubricated tapered plug type valves were selected because this type provides quick quarter turn operation and minimizes wire drawing. Piping specified is standard weight seamless steel. The exterior of the piping is painted to prevent corrosion and to present a neat appearance. Pipe fittings of forged steel were selected for strength and tightness of joints. Because of the small sized piping required (1-1/2" max.) and in order to provide an economical installation, screw fittings are to be used. In order to provide easy methods of assembly and removal of subassemblies for maintenance operations, unions will be provided in the most advantageous locations.

9-04. EQUIPMENT HOIST.- A two-ton spur gear hoist, trolley mounted, will be installed on a fixed monorail over the basement hatch in the equipment building and will be used for the installation and maintenance of the equipment below the equipment room floor. Wherever it may be necessary to remove slide gates or hydraulic rams, the hoist may be trans-

ferred from the trolley to the gate chamber [illegible] be suspended from the gate hanger located on the ceiling of the gate chamber.

9-05. STAND-BY GENERATOR SET.-A 208-volt-3 phase, 60-cycle, four wire generator of 25 KW capacity, direct connected to a gasoline engine is selected to furnish emergency power and lighting. The capacity of the generator is sufficient to handle the starting load of the pump motor and full lighting load simultaneously. The generator will be driven at 1,800 r.p.m. This speed is considered satisfactory for industrial type gasoline engines since the requirements for operations under load conditions will be infrequent and of short duration. The engine will be equipped with single magnetic ignition and electric starting. A generator will not be required as equipment on the engine as the engine will not be run for long enough periods to keep the storage battery charged. The battery will be charged by a separate battery charger fed from the electric power and lighting system.

9-06. ELECTRICAL WORK.-(a) The power supply for the dam and associated buildings is taken from a 3-phase, 60-cycle, 2400-volt delta-connected overhead distribution circuit of the Public Service Company of New Hampshire. The 2400-volt primary line is brought from the overhead line underground a distance of several hundred feet along the access road to the transformer vault in the basement of the equipment building at the transformer vault where the voltage is transformed to 120/208 through three single-phase oi-insulated distribution transformers.

(b) A 120/208 volt, 3-phase, 4-wire, gasoline driven engine generator set (see paragraph 9-05) is provided to carry the load in the event of commercial power failures caused by lightning, sleet or other events. Power is distributed throughout the structures at 120/208 volts 4-wire, 3-phase. Conductors are of sufficient size to comply with the National Electrical Code requirements for carrying capacity and to insure circuit voltage regulation consistent with good practice.

(c) Spare capacity in duct banks, extra conduit sleeves in the equipment room and elsewhere, and spare circuits on the switchboard and panels are incorporated to provide for any unforeseen increase in electrical requirements at a later date.

SECTION X. CONSTRUCTION PROCEDURE

10-01. GENERAL.-It is proposed to construct the initial stage dam in two construction seasons extending from April through October. All construction work will be performed by contractual forces. It is not considered possible to continue construction during the winter following the first construction season due to the severity of the weather in Dennington. It is anticipated that the relocation and raising of roads and other facilities affected by the reservoir will be accomplished during the second season of construction. This program of road relocation will not affect the construction procedure as traffic can be detoured on other roads until the roads to be relocated and raised are completed. The railroad abandonment would not be completed until some time during the second season of construction and because of this the contractor will not be able to construct that portion of the west embankment where the railroad is located until the railroad work is completed.

10-02. SCHEDULE OF OPERATIONS.- The sequence of construction operations proposed for the two-year construction period is presented graphically in the accompanying Plate No. X-1 and described in the following paragraphs:

(a) First Season.-

(1) Construct the Maintenance Building as a temporary field office.

(2) Construct the west embankment from Sta. 20+00 to Sta. 37+00 to Elevation 724.

(3) Construct the east embankment to Elevation 686.

(4) Excavate area for Spillway, Stilling Basin, Abutment sections and Discharge Channel Walls.

(5) Construct the Spillway and Abutment sections to Elevation 679.5.

(6) Construct the Discharge Channel Walls complete.

(7) Excavate the Spillway Approach Channel except for a portion at the river bank to serve as a cofferdam.

(8) Excavate the Spillway Discharge Channel.

(b) Second Season.-

(1) Complete the Spillway to Elevation 705. and the Abutment sections to Elevation 724.

(2) Construct the upstream and downstream cofferdams.

(3) Excavate the temporary upstream diversion channel.

(4) Divert and construct the west embankment section in the upper bed between Sta. 11+[illegible] to Sta. 20+00 to Elevation 72+.

(5) Complete the east embankment section to Elevation 72+.

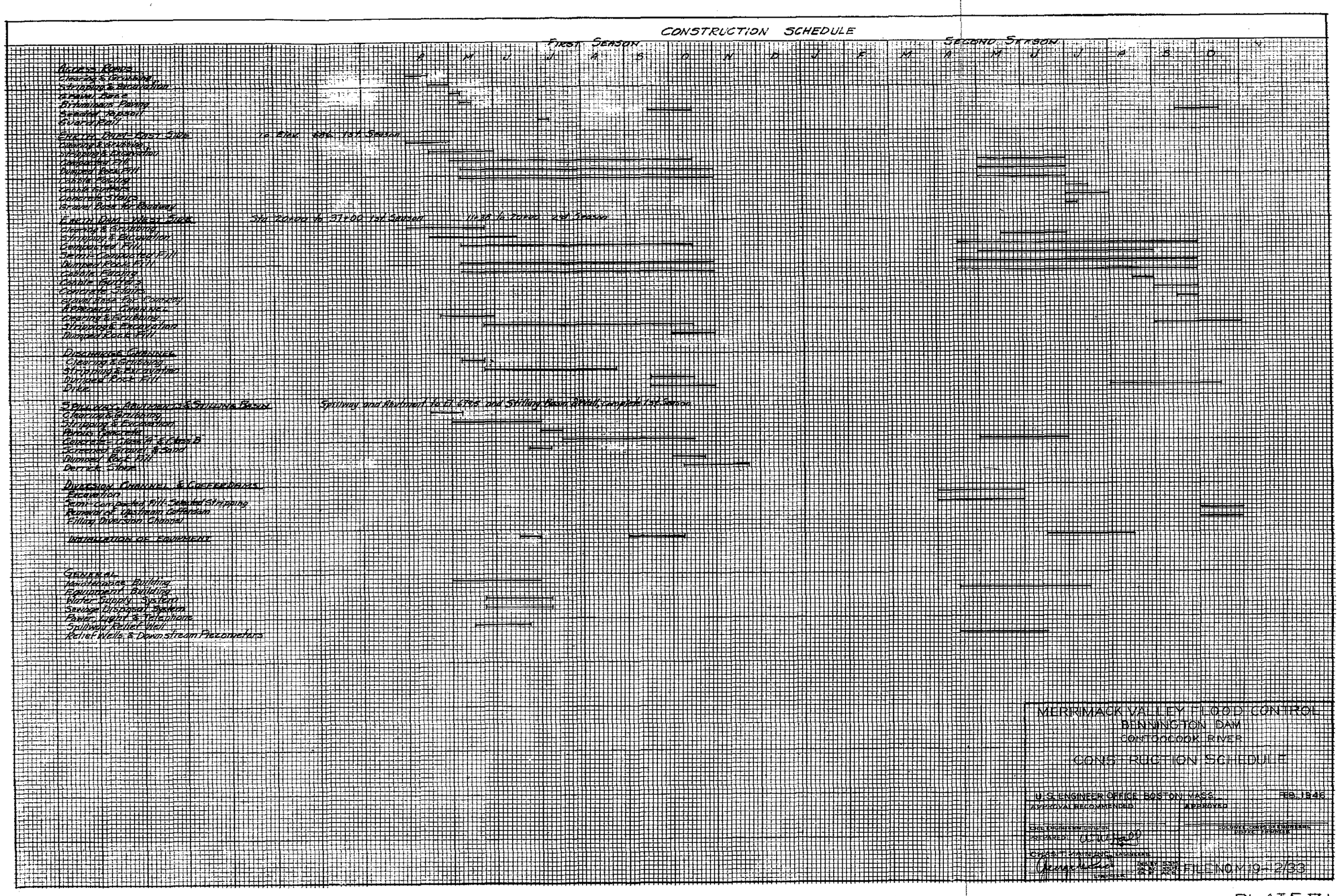
(6) Construct the downstream Terrace and Stone Dike.

(7) Complete the Main and West Access Roads.

(8) Construct the Equipment House.

(9) Remove cofferdams and dress spoil areas.

(10) Final cleanup



MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON DAM
CONTOOCOOK RIVER

CONSTRUCTION SCHEDULE

U.S. ENGINEER OFFICE, BOSTON, MASS. FEB. 1948
APPROVAL RECOMMENDED: APPROVED:
CHIEF, ENGINEERING DIVISION
PREPARED: CHIEF, CONSTRUCTION DIVISION
CHIEF, PLANNING BRANCH
CHIEF, DESIGN BRANCH
CHIEF, CONSTRUCTION BRANCH
FILE NO. Y19-2/33

PLATE X-1

SECTION XI. SUMMARY OF COST

11-01. ESTIMATED COSTS. The cost of the proposed Bennington Dam and appurtenances, based upon final design, is estimated as follows:

ITEM	DESIGNATION	QUANTITY	UNIT	UNIT COST	TOTAL COST
1.	Reservoir Clearing	1	Job	L.S.	$5,000
2.	Removal of Existing Structures	1	Job	L.S.	4,000
3.	Diversion and Care of River and Dewatering Site	1	Job	L.S.	70,000
4.	Clearing and Grubbing	50	Acre	400.00	20,000
5.	Stripping	92,000	Cu.Yd.	.50	46,000
6.	Excavation - Common	496,800	Cu.Yd.	.60	298,080
7.	Excavation - Trench - to 6-foot depth	2,300	Cu.Yd.	2.00	4,600
8.	Excavation - Trench Greater than 6-foot depth	500	Cu.Yd.	3.50	1,750
9.	Excavation - Borrow - Random & Impervious	199,300	Cu.Yd.	.55	109,615
10.	Excavation-Borrow-Pervious	111,100	Cu.Yd.	.55	61,105
11.	Excavation - Rock	53,100	Cu.Yd.	3.50	185,850
12.	Placing and Compacting impervious fill	150,000	Cu.Yd.	.18	27,000
13.	Placing and Compacting Random Fill	204,000	Cu.Yd.	.18	36,720
14.	Placing and Compacting Pervious Fill	174,000	Cu.Yd.	.15	26,100
15.	Semi-Compacted Fill	100,900	Cu.Yd.	.10	10,090

ITEM	DESIGNATION	QUAN-TITY	UNIT	UNIT COST	TOTAL COST
16.	Additional Rolling	20,000	Square	$ .10	$ 2,000
17.	Hand Compacted Fill	6,000	Cu.Yd.	1.00	6,000
18.	Selected Gravel	44,000	Cu.Yd.	1.50	66,000
19.	Screened Gravel	4,100	Cu.Yd.	4.00	16,400
20.	Gravel Backing	34,000	Cu.Yd.	3.00	102,000
21.	Sand	800	Cu.Yd.	3.00	2,400
22.	Dumped Rock Fill	85,700	Cu.Yd.	4.00	342,800
23.	Cobble Facing	7,500	Cu.Yd.	3.00	22,500
24.	Derrick Stone	5,200	Cu.Yd.	8.00	41,600
25.	Concrete in Abutments, Spillway and Discharge Channel Walls	43,300	Cu.Yd.	18.00	779,400
26.	Concrete in Stilling Basin Floor Slab and Baffles	7,250	Cu.Yd.	13.00	94,250
27.	Concrete - Miscellaneous	175	Cu.Yd.	30.00	5,250
28.	Porous Concrete	1,800	Cu.Yd.	10.00	18,000
29.	Portland Cement	72,500	Bbl.	3.50	253,750
30.	Steel, Reinforcement	387,500	Lb.	.09	34,875
31.	Copper Water Stops	4,100	Lb.	1.25	5,125
32.	Miscellaneous Metals	1	Job	L.S.	9,500
33.	Tile Gages	1	Job	L.S.	600
34a.	Bituminous Coated Perferated Corrugated Metal Pipe - 10-inch	900	Lin.Ft.	1.40	1,260
34b.	Bituminous Coated Perforated Corrugated Metal Pipe 12-inch	2,300	Lin.Ft.	1.50	3,450

ITEM	DESIGNATION	QUAN-TITY	UNIT	UNIT COST	TOTAL COST
35.	Spillway, Stilling Basin and Embankment Piezometers and Settlement Gages	1	Job	$ L.S.	$ 3,500
36.	Downstream Piezometers	500	Lin.Ft.	5.00	2,500
37.	Embankment Relief Walls	800	Lin.Ft.	25.00	20,000
38a.	Relief Wall Collector Pipe - 6-inch concrete	225	Lin.Ft.	2.00	450
38b.	Relief Wall Collector Pipe - 12-inch R.C.	1,400	Lin.Ft.	2.50	3,500
39.	Cement Asbestos Pipe 6-inch	35	Lin.Ft.	2.00	70
40.	Manhole	1	Job	L.S.	350
41.	Monorail Hoist	1	Job	L.S.	350
42.	Gasoline-Electric Stand-by Unit	1	Job	L.S.	3,000
43.	Installation of Government furnished equipment	1	Job	L.S.	10,000
44.	Oil Pressure System	1	Job	L.S.	10,000
45.	Electric Power, Lighting and Telephone Systems	1	Job	L.S.	23,000
46.	Gravel Base for Roadway	5,000	Cu. Yd.	1.60	8,000
47.	Bituminous Paving	3,630	Sq.Yd.	2.00	7,260
48.	Cobble Gutters	550	Sq. Yd.	3.00	1,650
49.	Seeded Top Soil	2	Acre	1,700.00	3,400
50.	Highway Guard Rail	326	Lin.Ft.	1.50	489
51.	Gate Posts, Drop Inlet and Culvert Pipe	1	Job	L.S.	1,500
52.	Water Supply Well	60	Lin.Ft.	10.00	600
53.	Log Boom	1	Job	L.S.	4,000

ITEM	DESIGNATION	QUANTITY	UNIT	UNIT COST	COST
54.	Equipment Building Superstructure	1	Job	L.S.	$ 19,000
55.	Maintenance Building	1	Job	L.S.	25,000
56.	Gates	1	Job	L.S.	75,000
57.	Pump and Storage Tank in Maintenance Building	1	Job	L.S.	1,000
58.	Operator's Quarters	1	Job	L.S.	18,000
59.	Recording Equipment in Equipment House				1,500
				TOTAL	$2,956,189
	Contingencies 10%				295,611
					$3,251,800
	District and Division Overhead (6% of $3,251,800)				195,100
	Inspection, Supervision and Engineering during Construction				317,300
	District and Division Office Overhead Hired Labor Operation of $317,300 @ 12%				38,100
	TOTAL CONSTRUCTION COST				$3,802,300

XII. CONFERENCE NOTES

12-01. GENERAL.- During the progress of the design, two meetings of the Board of Consultants were held in the Boston District Office and the selection of structures and features of design were discussed in detail and decisions made. Several inspection trips were made to the proposed site by the consultants in order to obtain a complete picture of actual field conditions prior to the conferences. Informal discussions were held at various times with Mr. W. H. McAlpine and Dr. A. Casagrande on various features of design.

12-02. RESULTS OF BOARD OF CONSULTANTS CONFERENCES.- (a) First Conference.- The first conference was held on 1 September 1944 with Dr. A. Casagrande, Harvard University, Cambridge, Mass., and Mr. W. F. Uhl, Chas. T. Main, Inc., Boston, Mass., consultants, and Mr. W. H. McAlpine, Office, Chief of Engineers, Washington, D. C. attending.

(1) General.- The following major decisions and modifications were agreed upon after a review of the preliminary plans:

a. It was agreed by the consultants that the location selected for the dam and spillway is the best possible location and that additional subsurface explorations should be made in order to obtain more complete information on the foundation.

b. An outlet for power should not be installed.

c. Dr. Casagrande recommended that a series of wells would be sufficient to relieve the hydraulic head in the pervious sand strata in the foundation on the west bank.

(2) Earth Embankment.- a. The Board of Consultants agreed to the following: a top width of twenty-five (25) feet for embankment; a top width of fifteen (15) feet for the core; the top of the core to be four (4) feet below the top of the embankment section, and side slopes of the core to be six on one (6 on 1).

b. Recommended provision of one (1) foot of gravel under the three-foot (3') layer of riprap on the upstream slope.

c. Recommended provision of a covering of gravel or small quarry waste in lieu of topsoil.

d. Recommended provision of riprap for the full length of the side slope along the discharge channel on the embankment side.

e. Recommended use of derrick stone as riprap on the level portion of the downstream terrace in the area adjacent to the spillway. In order to prevent erosion of the terrace, a stone breakwater to be provided at the top of the slope of the spillway discharge channel on the downstream terrace.

(3) Spillway.- a. It was generally agreed that either a gravity or hollow type of spillway may be employed, whichever is

[illegible] to be the [illegible]. The final decision was left to this office.

[illegible]. Pore lining was definitely recommended by Mr. McAlpin and Mr. [illegible] in order to [illegible] and may come to face.

[illegible]. Dr. Casagrande recommended that an impervious material under the riprap be provided at the entrance to the spillway and [illegible].

[illegible]. It was recommended that a minimum thickness of [illegible] for vertical walls of the spillway retaining wall. The bearing pressure of 6[illegible] tons per square foot was considered safe.

(c) A second conference was held on 14-15 December 1944 for a final review of the approved design. The conference was attended by Mr. John D. Dorian, Philadelphia, Pa., in addition to the group noted in subparagraph (a) above. The following modifications and revisions were agreed upon and incorporated in the final design of the [illegible] contained in the District's Project Report on Shenango Reservoir, dated [illegible] 1944:

(1) General.- a. The general arrangement, type and location of structures was considered satisfactory.

b. Recommended further exploration, including continuation of pressure tests of rock and observation of water level ranges.

(2) Earth Embankment.- a. Drain pipe in toe drain to be lowered.

b. Very pervious blanket under the random fill section to be omitted.

c. Drainage blanket to be constructed of selected sand and gravel.

d. Downstream toe to be protected with three (3) feet of hoped riprap placed on one (1) foot of screened gravel backing.

e. Gravel facing on downstream face of dam to be rake: gravel, screened gravel or stones culled from pervious fill.

f. In region where highly pervious till on west abutment opens upstream, an impervious blanket to be carried to upstream face of embankment to enable upstream extension when dam is raised to ultimate height.

g. The addition of two (2) wells on west end of downstream well drainage system to overlap beginning of upstream blanket and locating well closer to downstream toe of dam is recommended.

(3) Spillway.- a. The spillway design flood to be recomputed on basis of new data and recommendations from the Office of the Chief of Engineers.

b. Cut-off wall under spillway to be omitted and the foundation to be placed in a straight plane sloping to the toe. The first foot of concrete to be placed in one layer on the till foundation immediately following excavation. Underdrainage to begin at downstream third point. Upstream face to be backfilled with compacted till below elevation 660. Site to be dewatered by sloping to ditches and pumping.

c. Lengths of monoliths and construction pours of spillway approved. No keys to be provided in construction or contraction joints, except between stilling basin slabs where keys are to be special, similar to Arkabutla Dam.

d. Absorptive form lining to be used on all exposed concrete faces.

e. Air-entraining cement to be used in all concrete.

f. Filters under stilling basin and downstream portion of spillway to be six (6) inches sand, six (6) inches gravel and twelve (12) inches porous concrete. Lateral pipes, piezometers and manholes approved.

g. Protection below end sill of stilling basin to consist of five (5) feet of derrick stone on eighteen (18) inches of riprap over eighteen (18) inches of screened gravel and to extend sixty (60) feet downstream. Surface of stone to be reasonably smooth and one (1) foot lower than top of end sill.

h. Type of gates, stop-log provisions and trash bars approved. Gates subject to slight enlargement to satisfy OCE requirements for additional capacity. Dam to be equipped with simple stop-log gate and device for handling.

i. Provision for removal of portion of spillway crest to facilitate future raising approved.

j. Economy of counterfort retaining walls vs gravity retaining wall to be refigured. Gravity walls recommended if cost differential is not large.

k. Suitability of gravel for use in concrete to be tested.

(c) The final design of the dam has been modified to incorporate suggestions and recommendations made by the New England Division and the Office, Chief of Engineers, in the 1st and 2nd Indorsements to letter to the Chief of Engineers, through the Division Engineer, New England Division, Boston, Mass., dated 30 April 1945, subject: "Definite Project Report for Bennington Reservoir, Contoocook River, New Hampshire".

ANALYSIS OF DESIGN

BENNINGTON DAM

APPENDIX A

SELECTED CONTRACT DRAWINGS

FOREWORD

The drawings comprising Appendix A are selected from the drawings prepared for issuance with the contract specifications and are referred to in the various Sections of the Analysis of Design.

ANALYSIS OF DESIGN

BENNINGTON DAM

APPENDIX A

SELECTED CONTRACT DRAWINGS

INDEX OF PLATES

MERRIMACK VALLEY FLOOD CONTROL

BENNINGTON DAM

CONTOOCOOK RIVER

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A-4	Hydrographs No. 2
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A-6	Record of Foundation Exploration No. 1
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A-10	Record of Foundation Exploration No. 5
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A-16	Embankment Drainage
A-17	Masonry Structures - General Plan and Elevations
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A-19	Stilling Basin - Plan and Sections
A-20	Equipment Building - Plans and Sections
A-21	Maintenance Building - Plans & Sections
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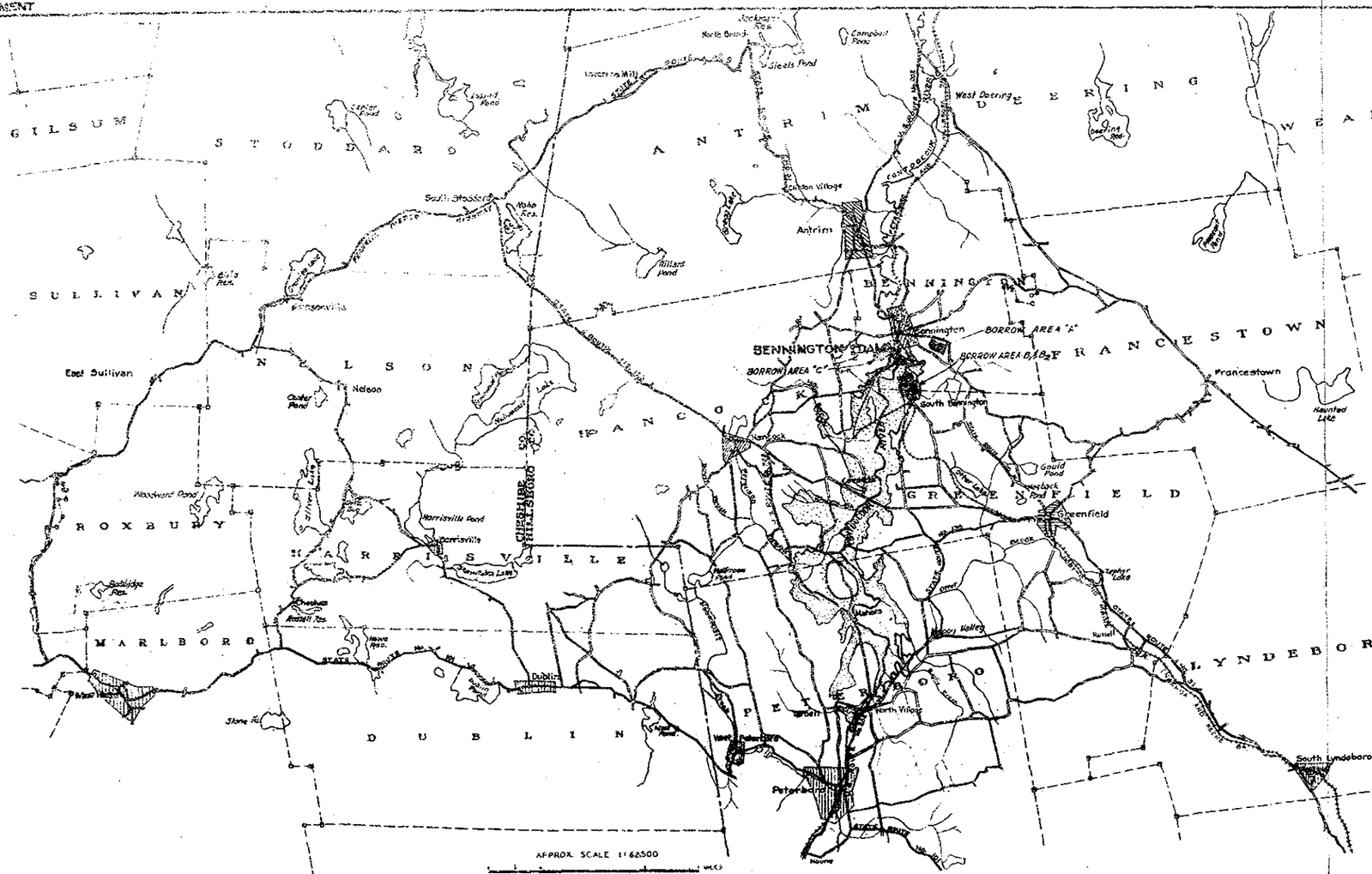

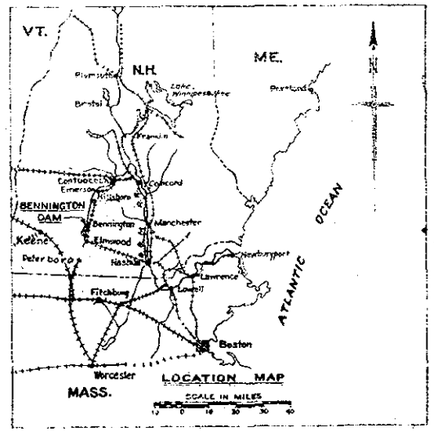

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	GENERAL	
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50/2	General Plan and Sections	2
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NO.	CHARACTER	DATE	BY
	REVISIONS		

MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON DAM
CONTOOCOOK RIVER

PROJECT LOCATION PLAN AND INDEX

IN 66 SHEETS — SHEET NO. 1 — SCALE AS SHOWN

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

APPROVAL RECOMMENDED: Chief, Foundation Division

APPROVED: Colonel, Corps of Engineers, District Engineer

PREPARED: Chief, Foundation Section

OFFICE, T. MAINTENANCE: Chief, Design Section

FILE NO. M19-58/1

PLATE A- 1

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY

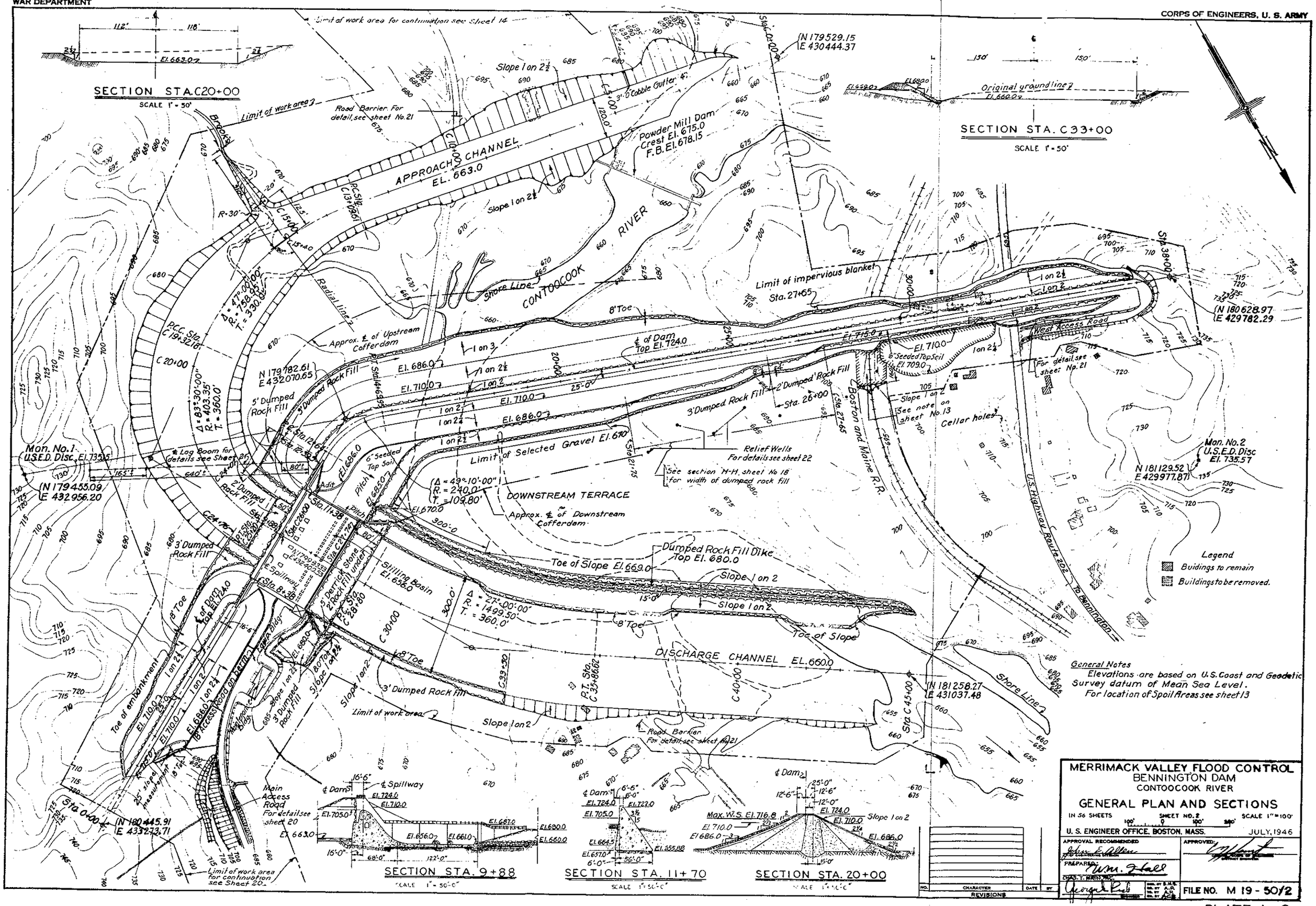
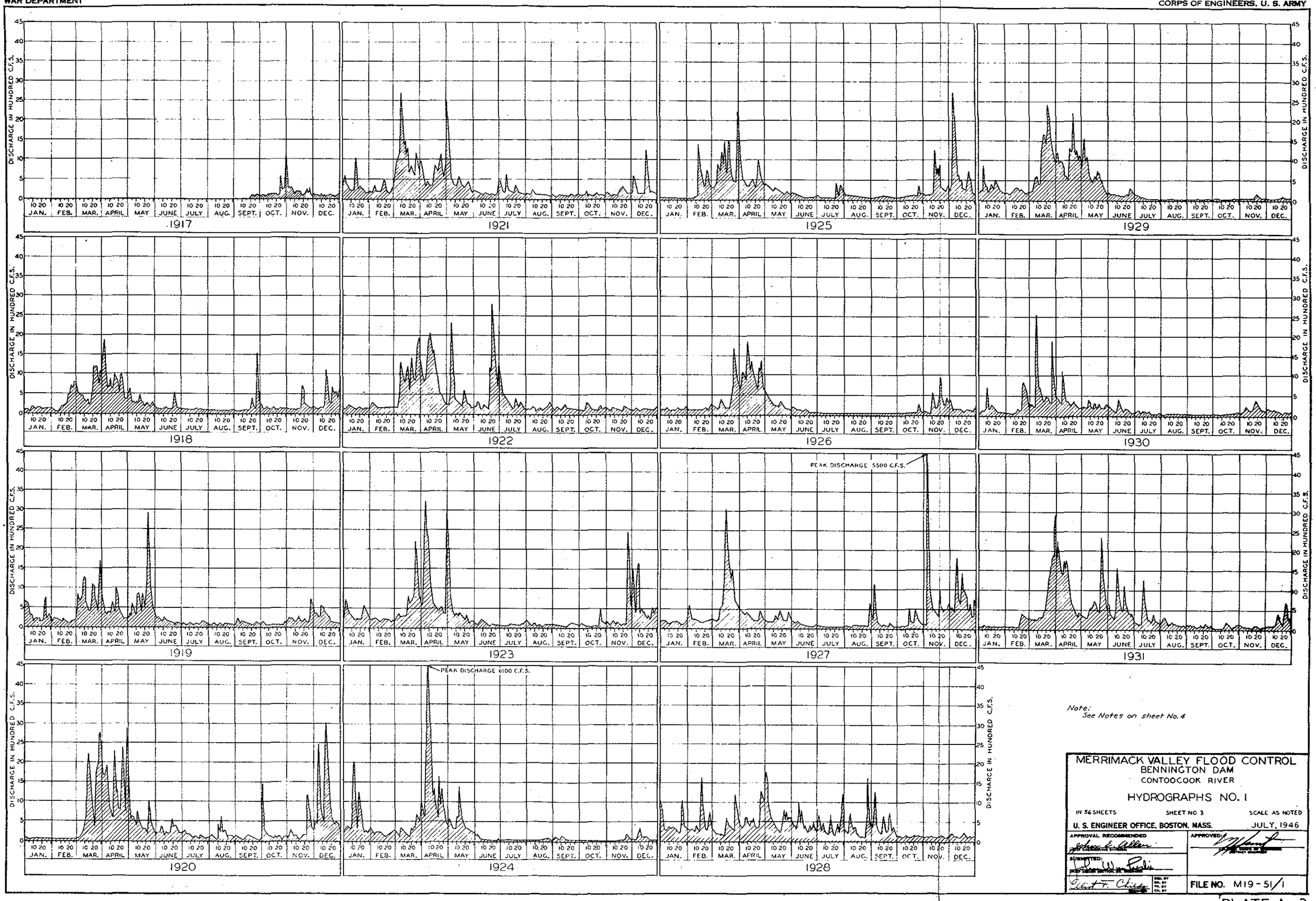


PLATE A-2



Note:
See Notes on sheet No. 4

MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON DAM
CONTOOCOOK RIVER

HYDROGRAPHS NO. 1

IN 56 SHEETS | SHEET NO 3 | SCALE AS NOTED

U. S. ENGINEER OFFICE, BOSTON, MASS. | JULY, 1946

APPROVAL RECOMMENDED: John E. Allen, CHIEF, PLANNING DIVISION

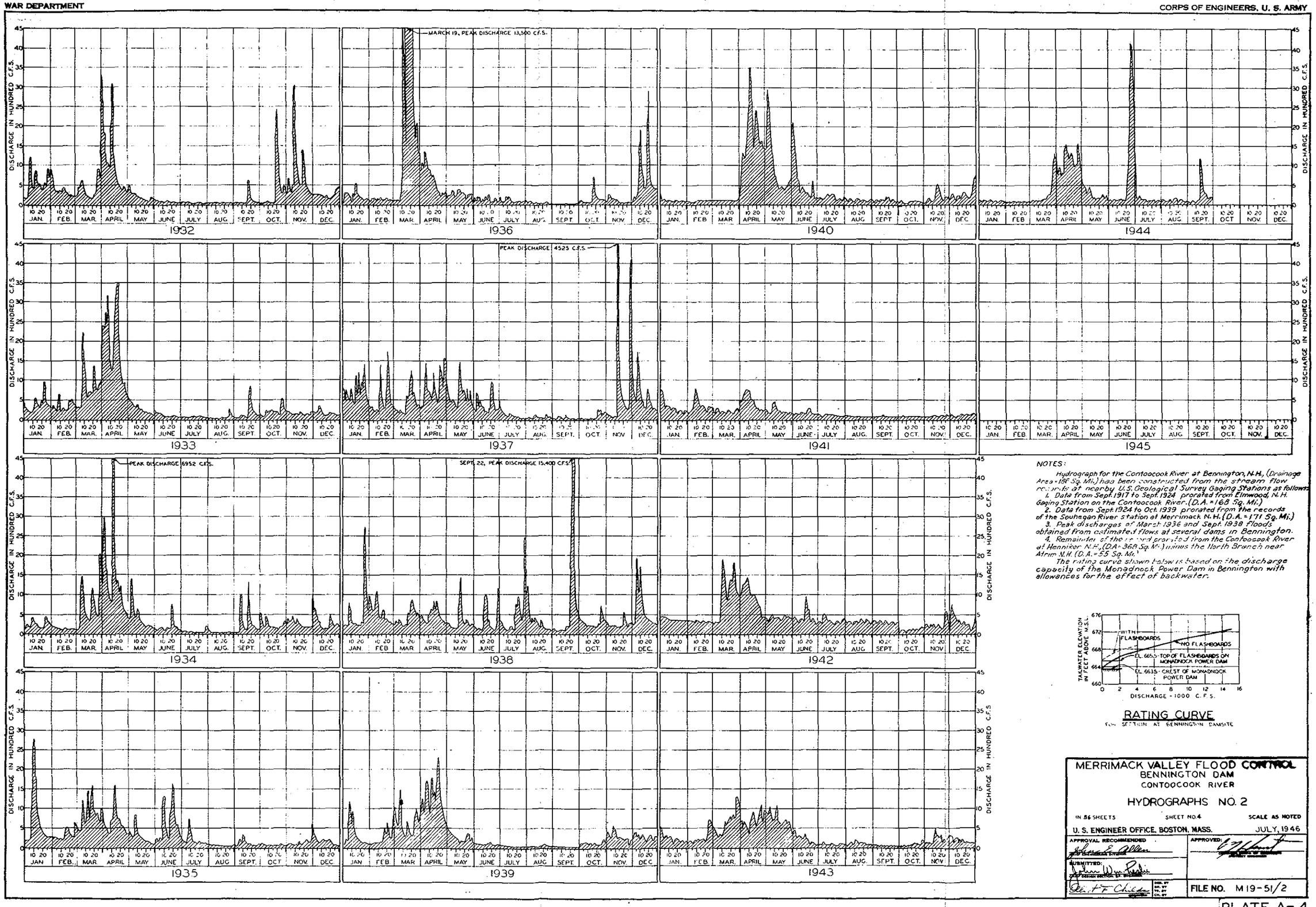
APPROVED: [illegible], COLONEL, CORPS OF ENGINEERS, DISTRICT ENGINEER

SUBMITTED: [illegible], CHIEF, DESIGN SECTION, ENG. DIVISION

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

FILE NO. M19-51/1

PLATE A-3



NOTES:

Hydrograph for the Contoocook River at Bennington, N.H., (Drainage Area = 186 Sq. Mi.) has been constructed from the stream flow records at nearby U.S. Geological Survey Gaging Stations as follows:

1. Data from Sept. 1917 to Sept. 1924 prorated from Elmwood, N.H. Gaging Station on the Contoocook River. (D.A. = 168 Sq. Mi.)
2. Data from Sept. 1924 to Oct. 1939 prorated from the records of the Souhegan River station at Merrimack N.H. (D.A. = 171 Sq. Mi.)
3. Peak discharges of March 1936 and Sept. 1938 floods obtained from estimated flows at several dams in Bennington.
4. Remainder of the record prorated from the Contoocook River at Henniker N.H. (D.A. = 368 Sq. Mi.) minus the North Branch near Atrim N.H. (D.A. = 55 Sq. Mi.)

The rating curve shown below is based on the discharge capacity of the Monadnock Power Dam in Bennington with allowances for the effect of backwater.

RATING CURVE

FOR SECTION AT BENNINGTON DAMSITE

MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON DAM
CONTOOCOOK RIVER

HYDROGRAPHS NO. 2

IN 56 SHEETS — SHEET NO. 4 — SCALE AS NOTED

U. S. ENGINEER OFFICE, BOSTON, MASS. — JULY, 1946

APPROVAL RECOMMENDED: C. Allen, Chief, Design Branch

APPROVED: C. H. Lent, Colonel, Corps of Engineers, District Engineer

SUBMITTED: Arthur W. Parker, Chief, Design Section, Ed. Br.

Geo. F. F. Childs, Designed

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

FILE NO. M 19-51/2

PLATE A-4

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY

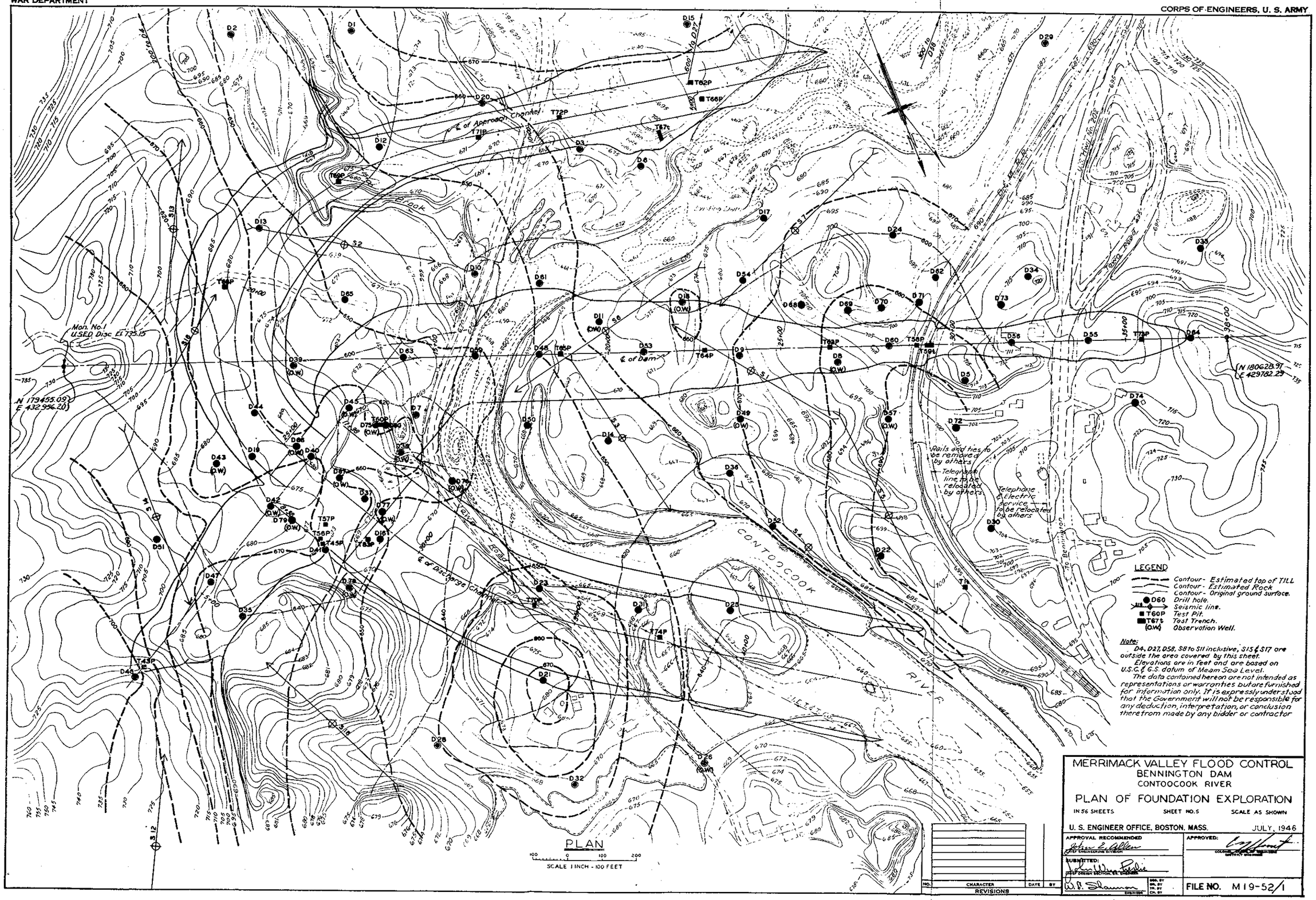
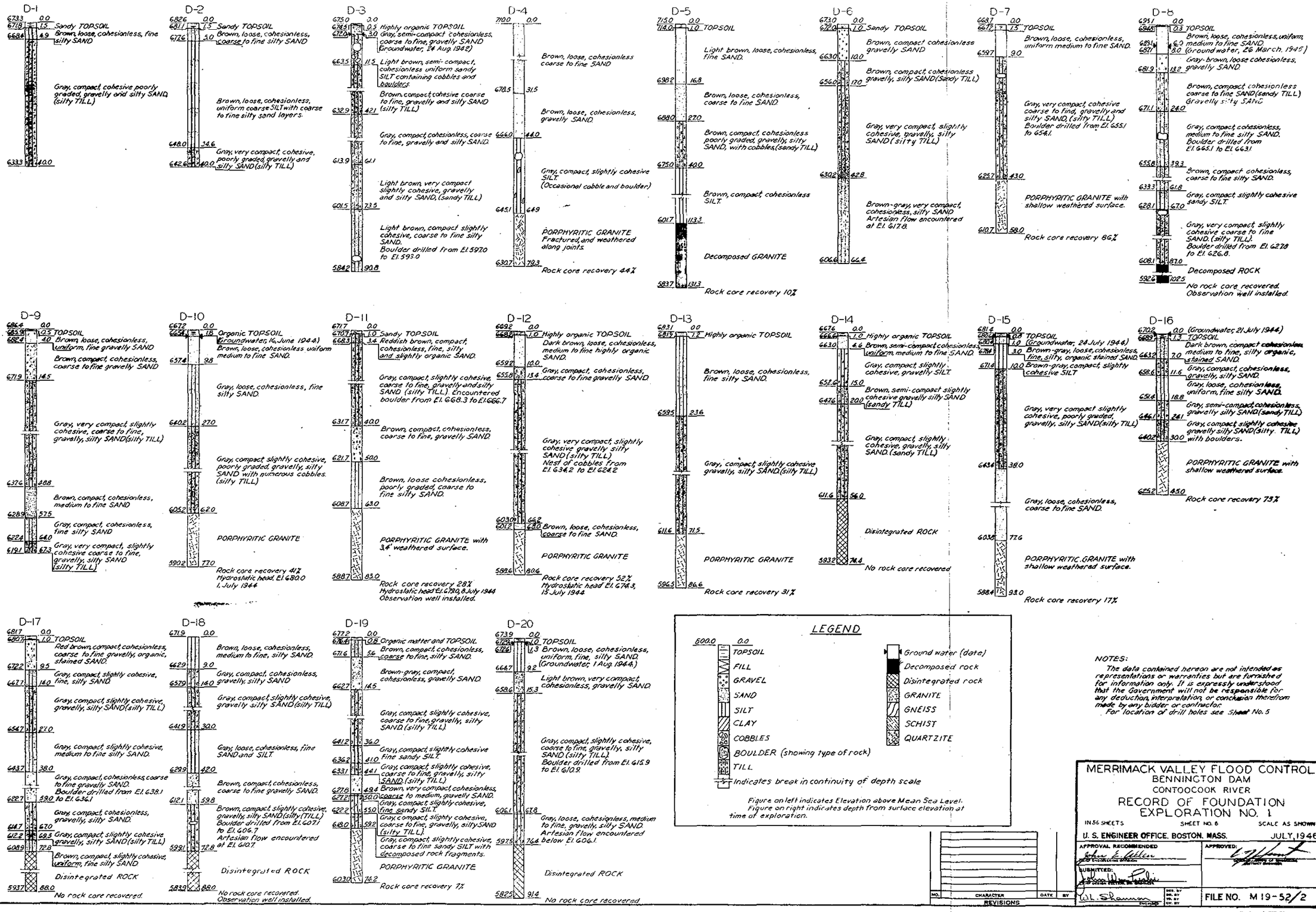


PLATE A-5



NOTES:

The data contained hereon are not intended as representations or warranties but are furnished for information only. It is expressly understood that the Government will not be responsible for any deduction, interpretation or conclusion therefrom made by any bidder or contractor.

For location of drill holes see Sheet No. 5

NO.	CHARACTER	DATE	BY
	REVISIONS		

MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON DAM
CONTOOCOOK RIVER
RECORD OF FOUNDATION EXPLORATION NO. 1

IN 56 SHEETS — SHEET NO. 6 — SCALE AS SHOWN

U. S. ENGINEER OFFICE, BOSTON, MASS. — JULY, 1946

APPROVAL RECOMMENDED: John E. Miller, Chief Engineering Division

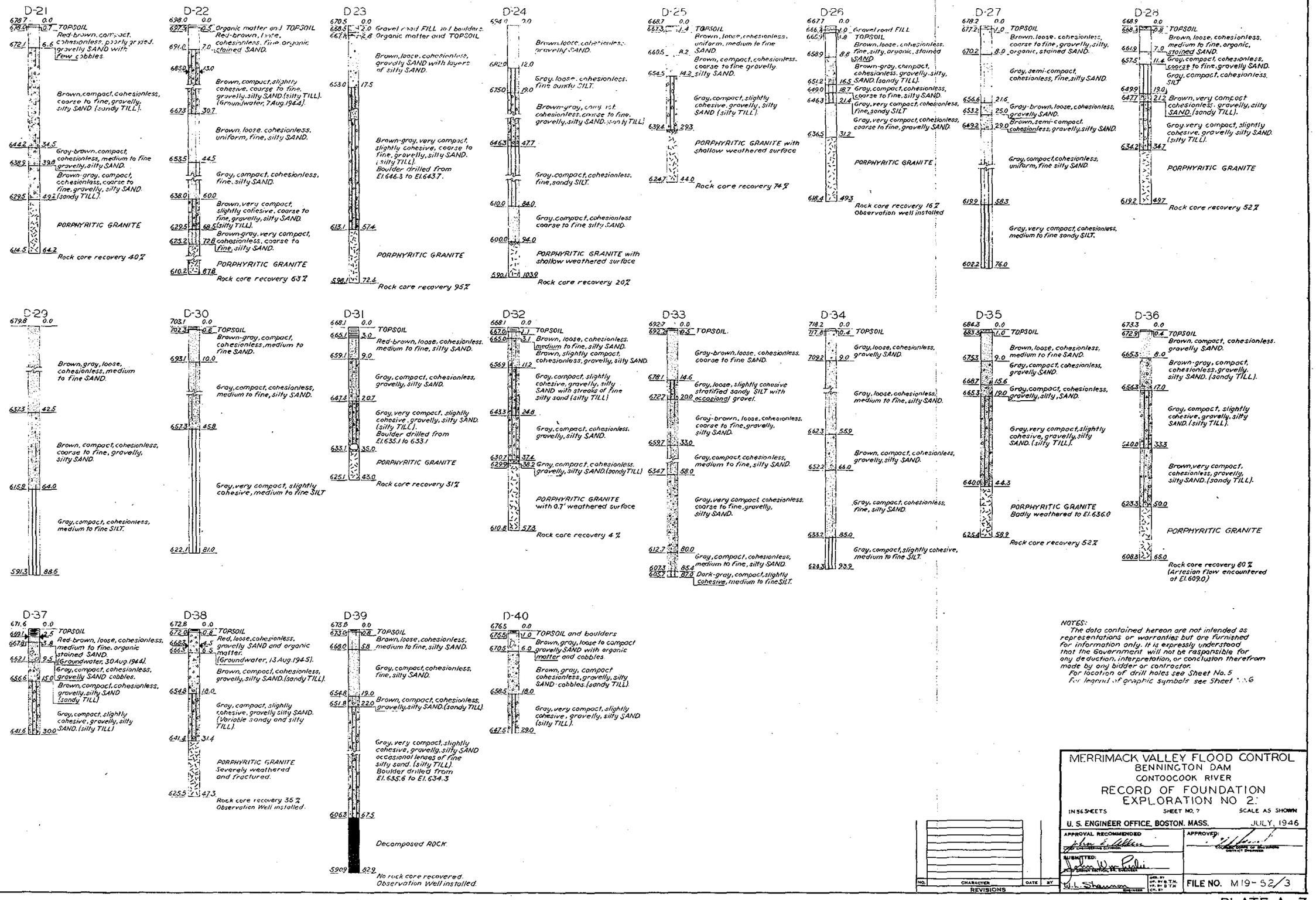
APPROVED: Colonel, Corps of Engineers, District Engineer

SUBMITTED: Chief Design Section, Div. Branch

W.L. Shannon, Engineer

DES. BY / DR. BY / CK. BY / CH. BY

FILE NO. M 19-52/2



NOTES:
The data contained hereon are not intended as representations or warranties but are furnished for information only. It is expressly understood that the Government will not be responsible for any deduction, interpretation, or conclusion therefrom made by any bidder or contractor.
For location of drill holes see Sheet No. 5
For legend of graphic symbols see Sheet No. 6

NO.	CHARACTER	DATE	BY

REVISIONS

MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON DAM
CONTOOCOOK RIVER
RECORD OF FOUNDATION EXPLORATION NO 2

IN 56 SHEETS — SHEET NO. 7 — SCALE AS SHOWN

U. S. ENGINEER OFFICE, BOSTON, MASS. — JULY, 1946

APPROVAL RECOMMENDED: John E. Allen, CHIEF ENGINEERING DIVISION

APPROVED: COLONEL CORPS OF ENGINEERS, DISTRICT ENGINEER

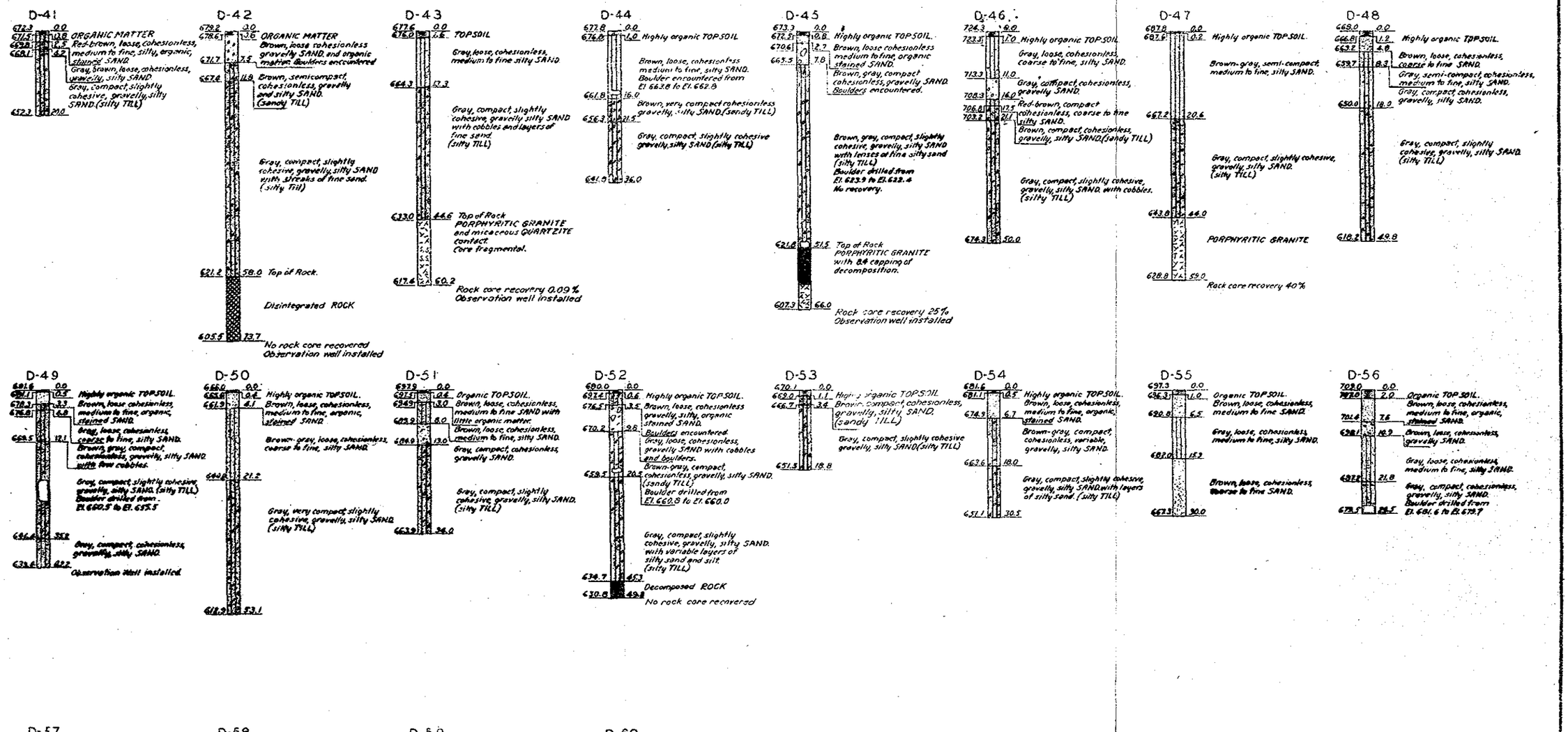
SUBMITTED: John W. Rollins, CHIEF DESIGN SECTION, SR. ENGINEER

W. L. Shannon, ENGINEER

DES. BY
DR. BY G.T.N.
TR. BY G.T.N.
CK. BY

FILE NO. M19-52/3

PLATE A-7



NOTES:
The data contained hereon are not intended as representations or warranties but are furnished for information only. It is expressly understood that the Government will not be responsible for any deduction, interpretation or conclusion therefrom made by any bidder or contractor.
For location of drill holes see Sheet No. 5
For legend of graphic symbols see Sheet No. 6

MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON DAM
CONTOOCOOK RIVER

RECORD OF FOUNDATION EXPLORATION NO. 3

IN 56 SHEETS — SHEET NO. 8 — SCALE, AS SHOWN

U. S. ENGINEER OFFICE, BOSTON, MASS. — JULY, 1946

APPROVAL RECOMMENDED — Chief Engineering Division

APPROVED — Chief, Engineering Division

SUBMITTED — Chief Design Branch, Eng. Division

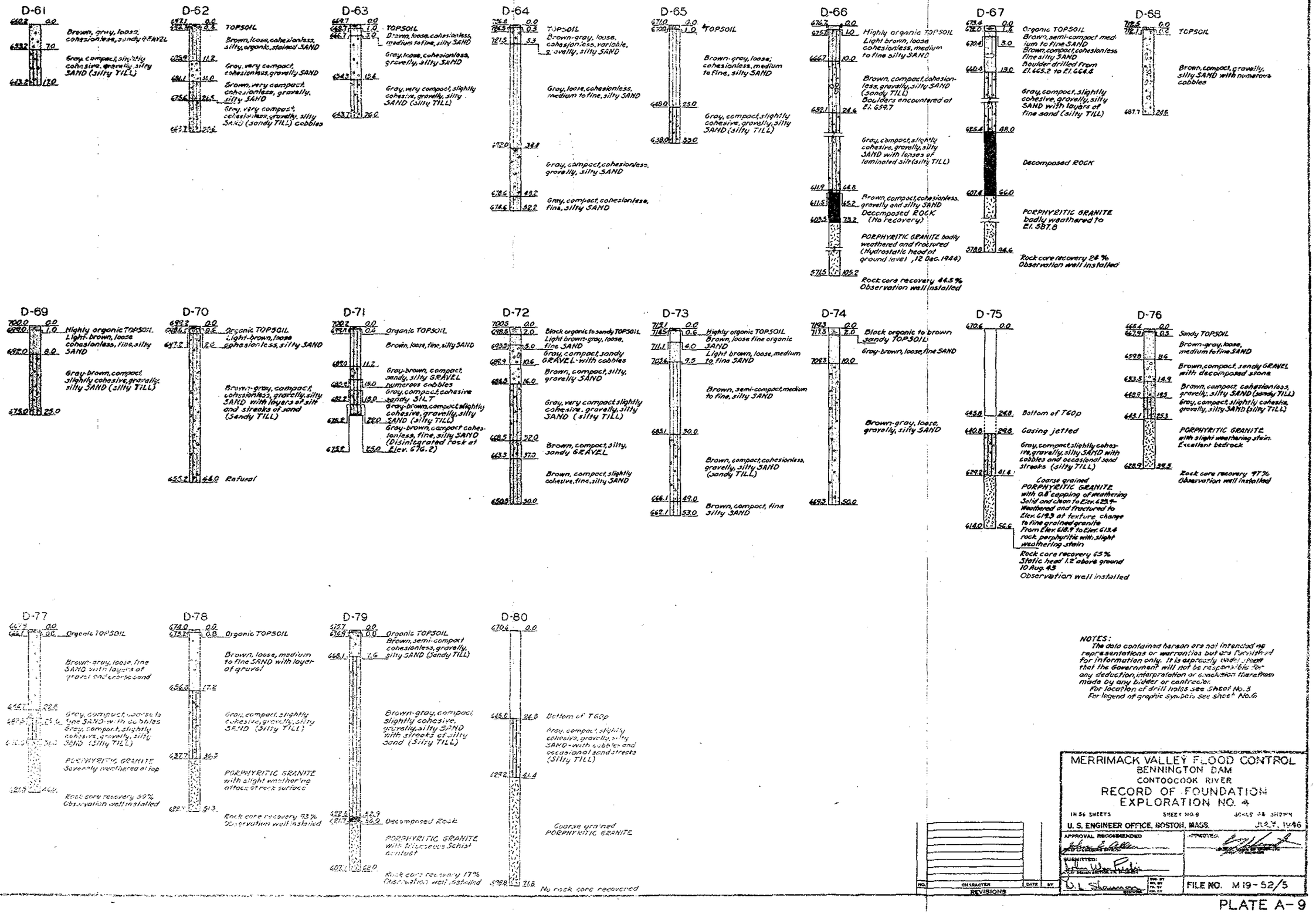
W. L. Shannon — Designed by

FILE NO. M19-52/4

NO.	CHARACTER	DATE	BY

REVISIONS

PLATE A-8



NOTES:
The data contained hereon are not intended as representations or warranties but are furnished for information only. It is expressly understood that the Government will not be responsible for any deduction, interpretation or conclusion therefrom made by any bidder or contractor.
For location of drill holes see Sheet No. 5
For legend of graphic symbols see sheet No. 6

NO.	CHARACTER	DATE	BY
	REVISIONS		

MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON DAM
CONTOOCOOK RIVER
RECORD OF FOUNDATION EXPLORATION NO. 4

IN 56 SHEETS — SHEET NO. 9 — SCALE AS SHOWN

U. S. ENGINEER OFFICE, BOSTON, MASS. — JULY, 1946

APPROVAL RECOMMENDED: John L. Cullen
APPROVED: [signature]
SUBMITTED: John W. Fuller, Chief Design Section
U. L. Sherman, Designer
DES. BY / DR. BY / CK. BY / CH. BY

FILE NO. M 19-52/5

PLATE A-9

T1t

0.0 Brown, sandy TOPSOIL
2.0 Light brown, uniform, fine to medium SAND fairly loose dry.
9.8

T43p

0.0 TOPSOIL
2.0 Compact gravelly SAND with boulders
5.0 compact SAND and GRAVEL
9.0

T45p

0.0 Organic TOPSOIL with boulders
1.3 Brown fine SAND and SILT with boulders.
3.0 Brown, compact, cohesionless gravelly, silty SAND (sandy TILL)
3.7 Gray, compact, slightly cohesive, gravelly, silty
6.0 SAND (silty TILL)

T56p

0.0 TOPSOIL
1.2 Brown, compact, fine, silty SAND.
3.0 Brown, compact, slightly cohesive, gravelly, silty, SAND containing cobbles and boulders (sandy TILL) (side of pit shows 6'0" diameter boulder)
4.5 Blue gray, semi-compact, slightly cohesive, gravelly silty SAND, containing cobbles
6.0 and boulders (silty TILL)

T57p

0.0 HUMUS and TOPSOIL
1.0 Brown, compact, silty, SAND (Boulders encountered
3.0 in sides and bottom of pit)

T58p

0.0 Yellow-gray, compact uniform SAND
1.0 Yellow-brown, compact variable, gravelly SAND containing cobbles (boulder area
3.0 encountered at 3.0')

T59t

0.0 TOPSOIL
0.5 Yellow-gray, compact, uniform SAND
20.0

T60p

0.0 Organic TOPSOIL
1.0 Compact gravel with large boulders
12.0 Gray, very compact, slightly cohesive, gravelly, silty SAND (silty TILL) with boulders
24.8

T62p

0.0 Highly organic TOPSOIL
0.6 Gray brown loose coarse to fine SAND
3.0 Water-abandoned.

T63p

0.0 Highly organic TOPSOIL
0.2 Brown-gray, semi-compact medium to fine SAND
6.0 Gray, loose SAND & GRAVEL
7.5 Refusal.

T64p

0.0 Highly organic TOPSOIL
0.5 Light brown, semi-compact medium to fine SAND.
2.5 Water.

T65p

0.0 Sandy TOPSOIL
0.5 Brown, semi-compact medium to fine SAND
5.0

T66p

0.0 Organic TOPSOIL
0.5 Light brown, semi-compact, medium to fine SAND.
1.5 Gray, semi-compact, medium to fine, silty SAND.
2.5 Gray, compact, SAND & GRAVEL.
7.5 Refusal-boulder.

T67t

0.0 Organic TOPSOIL
0.5 Gray-brown, semi-compact, silty SAND & GRAVEL.
22.0

T68p

0.0 Organic TOPSOIL
0.2 Brown, loose, fine SAND with organic material.
1.2 Gray compact, fine SAND
5.0 Brown, loose, SAND & GRAVEL
6.0 with cobbles.
In water-stopped

T69p

0.0 organic TOPSOIL
0.9 Light brown, loose, medium to fine, SAND with cobbles.
2.4 Brown loose SAND & GRAVEL
3.6 Gray, compact, gravelly SAND
5.0 Brown, semi-compact SAND &
8.0 GRAVEL with cobbles and boulders.
Refusal on boulder.

T70p

0.0 Highly organic TOPSOIL
1.6 Light brown, semi-compact, medium to fine SAND.
3.5 Gray, compact, gravelly, silty
6.0 SAND with boulders.
Refusal on boulder.

T71p

0.0 Highly organic TOPSOIL
1.4 Brown-gray, compact, medium to fine SAND (water saturation at 3.4)
6.0 Water

T72p

0.0 Highly organic TOPSOIL
0.8 Light brown, compact, medium to fine SAND.
5.0 Water.

T73p

0.0 Highly organic TOPSOIL
0.1 Brown, semi-compact, medium to fine SAND
22.0 Refusal on cobble

T74p

0.0 Organic TOPSOIL
1.6 Light brown, semi-compact, medium to fine SAND
7.0 Refusal on boulder

T83p

0.0 Sandy TOPSOIL
0.5 Gray, semi-compact stratified coarse to fine SAND
3.5 Water.

SEISMIC DATA

SHOT	SURFACE EL.	APPROX. ROCK EL.
S1	684.7	619.0
S2	666.7	585.0
S3	668.4	622.0
S4	682.0	636.0
S5	698.3	629.0
S6	671.3	607.0
S7	694.6	613.0
S8	690.9	570.0
S9	731.3	Deeper than 574.0
S10	675.7	649.0
S11	674.2	651.0
S12	724.4	675.0
S13	698.0	649.0
S14	692.2	644.0
S15	675.9	Deeper than 616.0
S16	682.5	Deeper than 623.0
S17	687.3	647.0
S18	674.3	652.0

NOTES:
The data contained hereon are not intended as representations or warranties but are furnished for information only. It is expressly understood that the Government will not be responsible for any deduction, interpretation or conclusion therefrom made by any bidder or contractor.
For location of test pits and seismic lines see Sheet No. 5
For legend of graphic symbols see Sheet No. 6

NO.	CHARACTER	DATE	BY

REVISIONS

MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON DAM
CONTOOCOOK RIVER
RECORD OF FOUNDATION EXPLORATION NO. 5

IN 56 SHEETS SHEET NO. 10 SCALE AS SHOWN
U. S. ENGINEER OFFICE, BOSTON, MASS. JULY, 1946

APPROVAL RECOMMENDED: John E. Allen, CHIEF ENGINEERING DIVISION
APPROVED: DISTRICT ENGINEER
SUBMITTED: CHIEF GEOLOGY SECTION, ENGINEERING DIVISION
DES. BY, DR. BY, TR. BY, CK. BY
FILE NO. M19-52/6

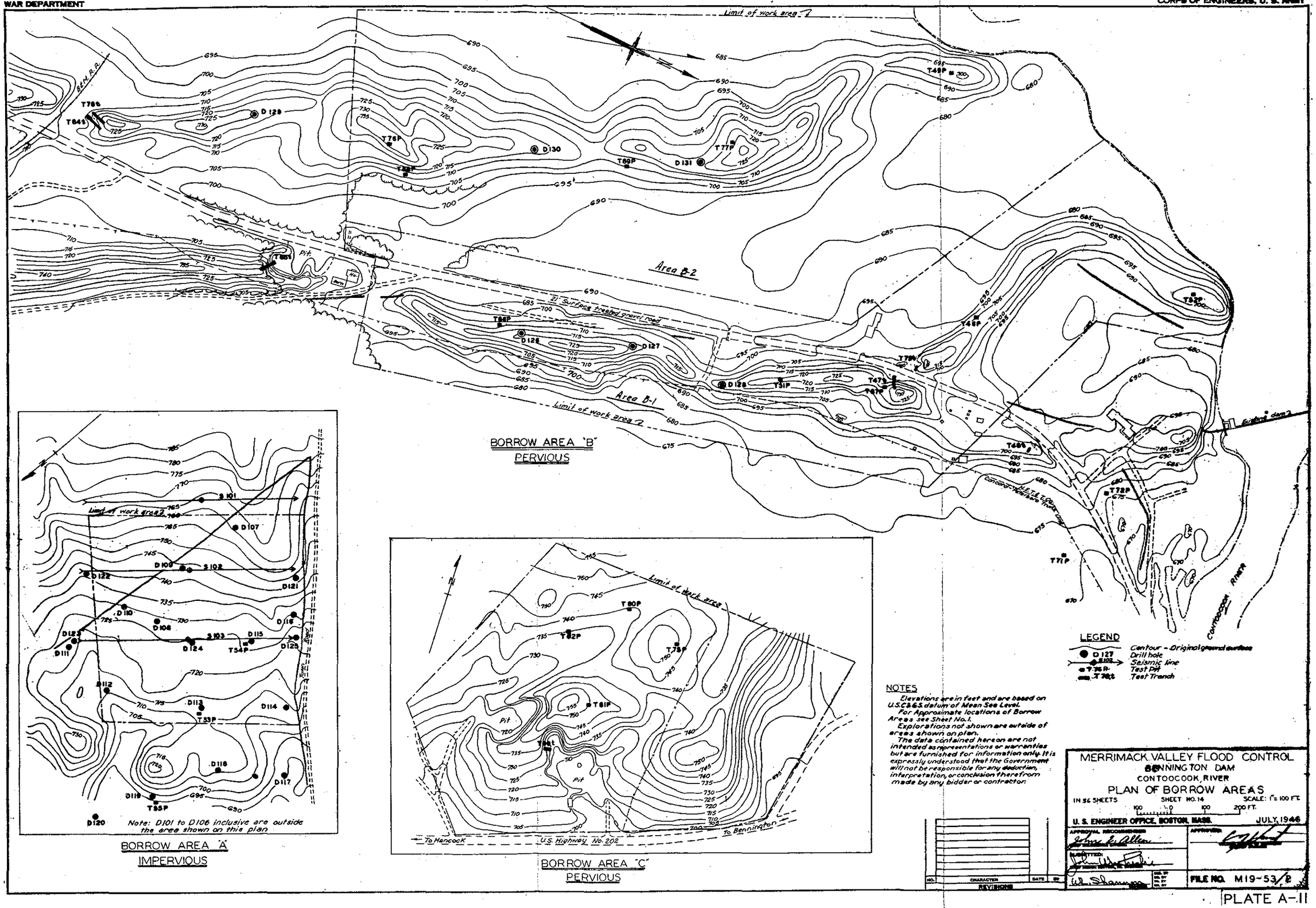

Limit of work area
B&M R.R.
Pit
Area B-2
Surface treated gravel road
Area B-1
Limit of work area
BORROW AREA "B"
PERVIOUS
Concord-Hancock-Bennington Truck Line
Existing dam
CONTOOCOOK RIVER
Limit of work area
Pit
To Hancock
U.S. Highway No. 202
To Bennington
BORROW AREA "C"
PERVIOUS
Note: D101 to D106 inclusive are outside the area shown on this plan
BORROW AREA "A"
IMPERVIOUS
LEGEND
Contour - Original ground surface
Drill hole
Seismic line
Test Pit
Test Trench
NOTES
Elevations are in feet and are based on U.S.C.&G.S. datum of Mean Sea Level.
For Approximate locations of Borrow Areas see Sheet No. 1.
Explorations not shown are outside of areas shown on plan.
The data contained hereon are not intended as representations or warranties but are furnished for information only. It is expressly understood that the Government will not be responsible for any deduction, interpretation, or conclusion therefrom made by any bidder or contractor.
MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON DAM
CONTOOCOOK RIVER
PLAN OF BORROW AREAS
IN 56 SHEETS
SHEET NO. 14
SCALE: 1" = 100 FT.
U. S. ENGINEER OFFICE, BOSTON, MASS.
JULY, 1946
FILE NO. M19-53/2
REVISIONS

PLATE A-11

D-101

0.0 Brown, semi-compact cohesionless, coarse to fine SAND - with pebbles

18.4 Refusal

D-102

0.0 TOPSOIL

1.0 Brown, compact, gravelly SAND

20.0

D-103

0.0 TOPSOIL

3.0 Light brown, loose, fine, silty SAND

7.5 Brown, semi-compact, gravelly SAND

10.5 Brown, compact, gravelly, silty SAND

15.0 Brown, compact, cohesionless, fine SAND

20.0

D-104

0.0 TOPSOIL

0.5 Brown, semi-compact, cohesionless, fine SAND with cobbles

4.5 Brown, semi-compact, fine, silty SAND

17.0 Brown, compact, fine SAND

27.0 Brown, compact, gravelly SAND

28.0 Refusal

D-105

0.0 TOPSOIL

1.0 Brown, semi-compact, cohesionless, medium to fine SAND

20.0 Brown, compact, cohesionless, coarse to fine SAND

25.0 Brown, compact, gravelly, silty SAND - with cobbles

28.5

D-106

0.0 TOPSOIL

0.5 Brown, semi-compact, fine, silty SAND - with cobbles

4.5 Brown, semi-compact fine SAND

13.0 Brown, compact, gravelly SAND

16.5 Gray, compact, slightly cohesive, gravelly, silty SAND (Silty TILL)

21.3 Refusal

D-107

0.0 TOPSOIL

1.0 Brown, compact, cohesionless, medium to fine SAND

10.8 Brown, compact, cohesionless, coarse to fine, silty SAND

11.2 Refusal

D-108

0.0 TOPSOIL

1.0 Brown, semi-compact, cohesionless, medium to fine, silty SAND

4.5 Brown-gray, compact, slightly cohesive, gravelly, silty SAND (Silty TILL)

18.5

D-109

0.0 TOPSOIL (Boulder area)

1.0 Brown, compact cohesionless gravelly, silty SAND

9.5 Brown, compact, gravelly SAND. Cobbles and boulders

15.0 Refusal

D-110

0.0 TOPSOIL (Boulder area)

1.5 Brown, compact, gravelly SAND - with cobbles

10.0 Brown, compact, gravelly, silty SAND - cobbles

20.2 Gray, compact, slightly cohesive, gravelly, silty SAND - with layers of gravelly sand (Silty TILL)

25.0

D-111

0.0 TOPSOIL (Boulder area)

0.5 Brown, compact, cohesionless, coarse to fine, silty SAND

7.5 Brown, compact, gravelly SAND

16.0 Brown-gray, compact, slightly cohesive, gravelly silty SAND (Silty TILL)

25.0

D-112

0.0 TOPSOIL (Boulder area)

1.0 Brown, compact, gravelly SAND - with cobbles

7.5 Brown-gray, compact, slightly cohesive, gravelly, silty SAND (Silty TILL)

24.5

D-113

0.0 TOPSOIL (Boulder area)

0.5 Brown, semi-compact, cohesionless, medium to fine SAND - cobbles

5.0 Brown, compact, gravelly SAND

6.6 Gray, compact, slightly cohesive, gravelly, silty SAND (Silty TILL)

35.0

D-114

0.0 TOPSOIL (Boulder area)

5.0 Brown, semi-compact, cohesionless, coarse to fine, gravelly SAND

9.0 Gray, compact, slightly cohesive, gravelly, silty SAND (Silty TILL)

43.0

D-115

0.0 TOPSOIL (Occasional boulder)

1.0 Brown, compact, gravelly SAND

15.0 Gray, compact, slightly cohesive, gravelly, silty SAND (Silty TILL)

36.0 Refusal

D-116

0.0 TOPSOIL

0.5 Brown, loose, gravelly SAND

5.0 Brown, loose SAND & GRAVEL

8.0 Brown, semi-compact, gravelly SAND - with cobbles

16.0 Gray, compact, slightly cohesive, gravelly, silty SAND (Silty TILL)

25.0

D-117

0.0 TOPSOIL

0.5 Brown, semi-compact, coarse to fine SAND

10.0 Gray, compact, slightly cohesive, gravelly, silty SAND (Silty TILL)

16.3

D-118

0.0 TOPSOIL

0.5 Brown, semi-compact, coarse to fine SAND

10.0 Brown, very compact, gravelly, silty SAND

20.0 Gray, very compact, slightly cohesive, gravelly, silty SAND (Silty TILL)

30.0

D-119

0.0 TOPSOIL

0.5 Brown, compact, gravelly SAND

8.0 Gray, compact, slightly cohesive, gravelly, silty SAND (Silty TILL)

25.0

D-120

0.0 Organic TOPSOIL

1.0 Brown, semi-compact, gravelly SAND

5.0 Gray, compact, slightly cohesive, gravelly, silty SAND (Silty TILL)

25.0

D-121

0.0 Sandy TOPSOIL

0.5 Light brown, semi-compact, fine SAND

8.5 Brown-gray, compact, gravelly, silty SAND - with cobbles and boulders

16.5 (Ground water 11 June 1945)

14.0 Gray, compact, slightly cohesive, gravelly, silty SAND (Silty TILL)

24.3 PORPHYRITIC GRANITE Slightly weathered with horizontal mechanical fracturing

99.5 Rock core recovery 45%

D-122

0.0 Sandy TOPSOIL

0.8 Brown-gray, compact, gravelly SAND

13.5 Gray, compact, slightly cohesive, gravelly, silty SAND (Silty TILL)

60.0 Observation well installed

D-123

0.0 Sandy TOPSOIL

0.8 Light-brown, loose, gravelly SAND

11.5 Brown-gray, compact, slightly cohesive, gravelly silty SAND - with sand streaks and occasional boulders (Silty TILL)

50.0 Observation well installed

D-124

0.0 Highly organic TOPSOIL with boulders

0.9 Brown, semi-compact, gravelly SAND

10.0 Brown-gray, compact, slightly cohesive, gravelly, silty SAND (Silty TILL)

50.0

D-125

0.0 Organic TOPSOIL

0.9 Brown, loose, gravelly SAND

12.2 Gray, compact, slightly cohesive, gravelly, silty SAND (Silty TILL)

50.0

NOTES:

The data contained hereon are not intended as representations or warranties but are furnished for information only. It is expressly understood that the Government will not be responsible for any deduction, interpretation or conclusion therefrom made by any bidder or contractor.

For location of drill holes see Sheet No. 14

For legend of graphic symbols see Sheet No. 6

NO.	CHARACTER OF REVISIONS	DATE	BY

MERRIMACK VALLEY FLOOD CONTROL

BENNINGTON DAM

CONTOOCOOK RIVER

RECORD OF BORROW EXPLORATION NO. 1

IN 56 SHEETS — SHEET NO. 15 — SCALE AS SHOWN

U. S. ENGINEER OFFICE, BOSTON, MASS. — JULY, 1946

APPROVAL RECOMMENDED: John E. Allen, Chief, Design Branch

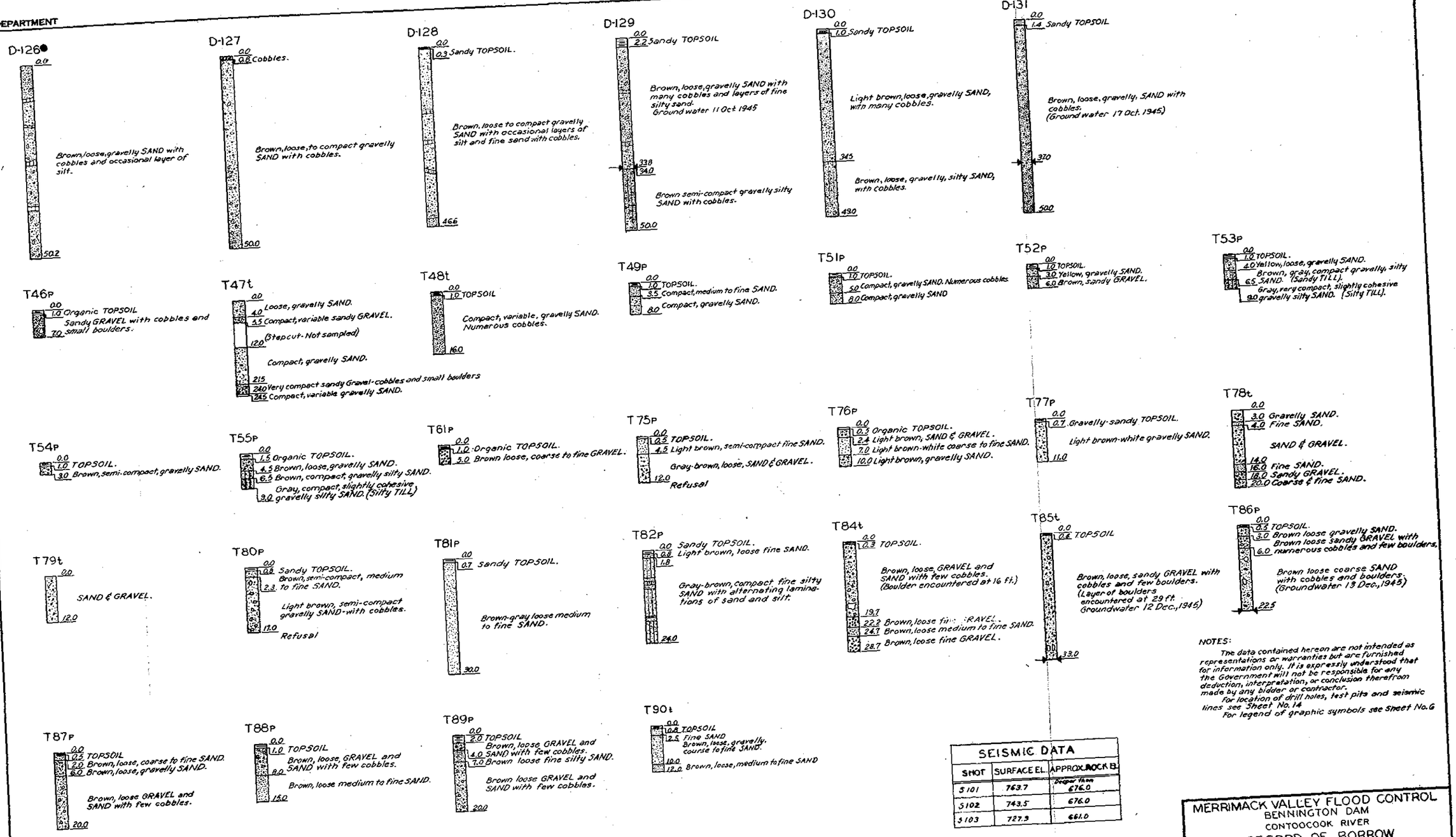
APPROVED: [illegible], Colonel, Corps of Engineers, District Engineer

SUBMITTED: [illegible], Chief, Soils Mechanics Section

W. L. Shannon, Chief, Soils Mechanics Section

FILE NO. M19-53/3

WAR DEPARTMENT



NOTES:

The data contained hereon are not intended as representations or warranties but are furnished for information only. It is expressly understood that the Government will not be responsible for any deduction, interpretation, or conclusion therefrom made by any bidder or contractor.

For location of drill holes, test pits and seismic lines see Sheet No. 14

For legend of graphic symbols see Sheet No. 6

SEISMIC DATA

SHOT	SURFACE EL.	APPROX. ROCK EL.
		Dodge Tank
S101	763.7	676.0
S102	743.5	676.0
S103	727.3	661.0

NO.	CHARACTER	DATE	BY
	REVISIONS		

MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON DAM
CONTOOCOOK RIVER

RECORD OF BORROW EXPLORATION NO. 2

SHEET NO. 16 IN 56 SHEETS

SCALE AS SHOWN

JULY, 1946

U. S. ENGINEER OFFICE, BOSTON, MASS.

APPROVAL RECOMMENDED: John E. Allen

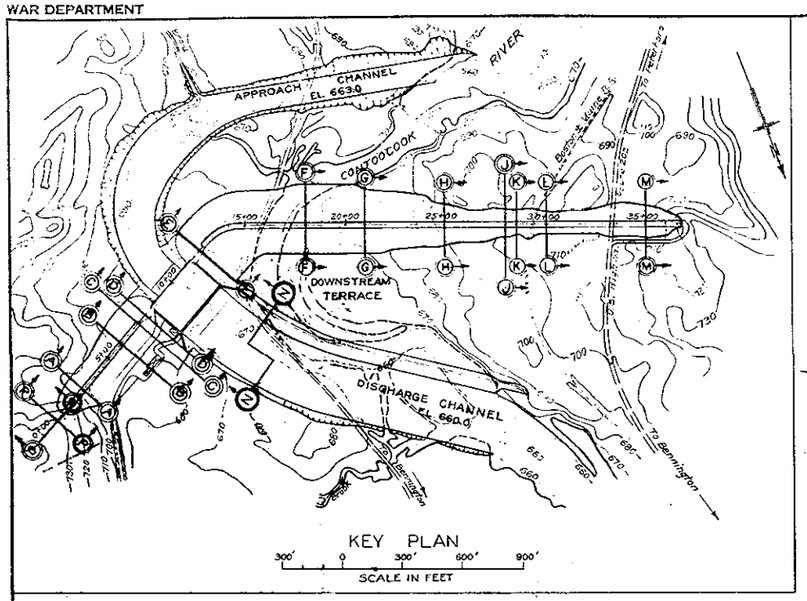
APPROVED:

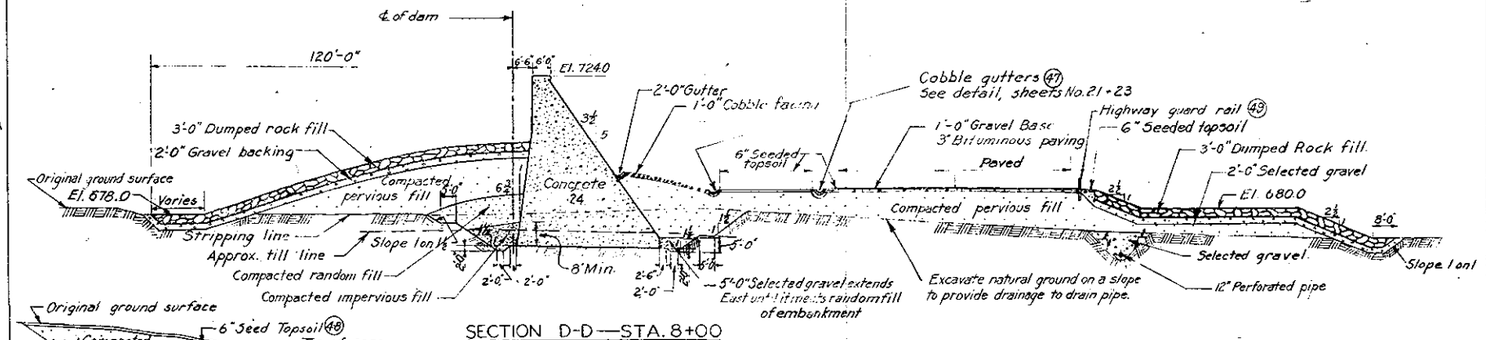
SUBMITTED:

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

FILE NO. M19-53/4

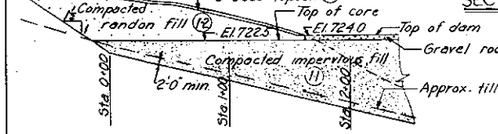
PLATE A-13





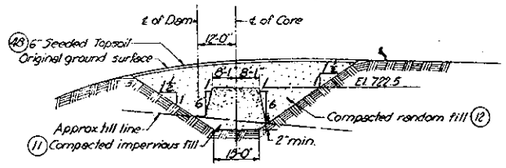
SECTION D-D — STA. 8+00

SCALE: 1" = 20'-0"



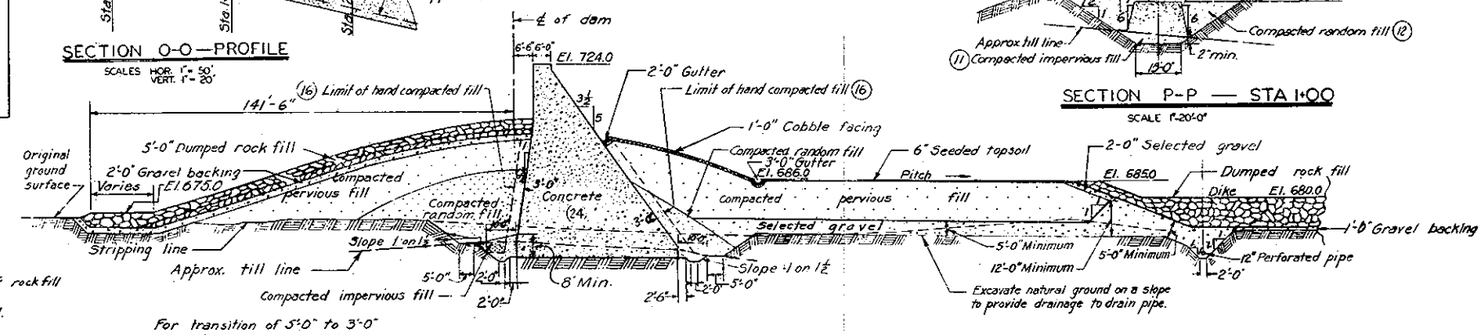
SECTION O-O — PROFILE

SCALES HOR. 1" = 50' VERT. 1" = 20'



SECTION P-P — STA 1+00

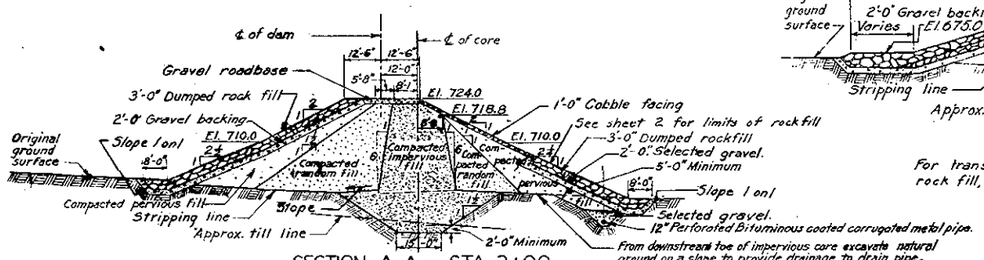
SCALE 1"=20'-0"



For transition of 5'-0" to 3'-0" rock fill, see sheet No. 23.

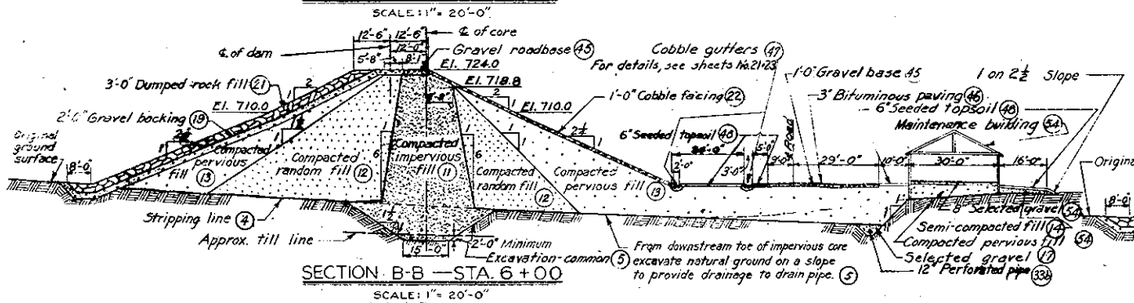
SECTION E-E — STA. 12+00

SCALE: 1" = 20'-0"



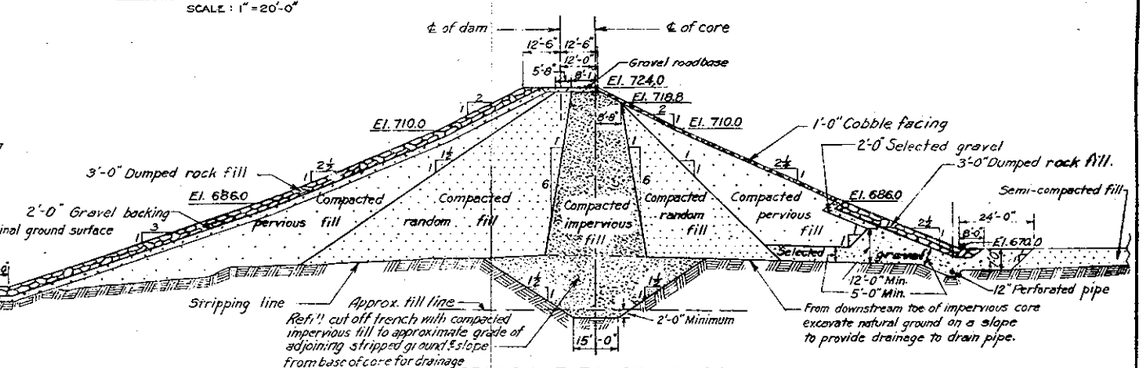
SECTION A-A — STA. 3+00

SCALE: 1" = 20'-0"



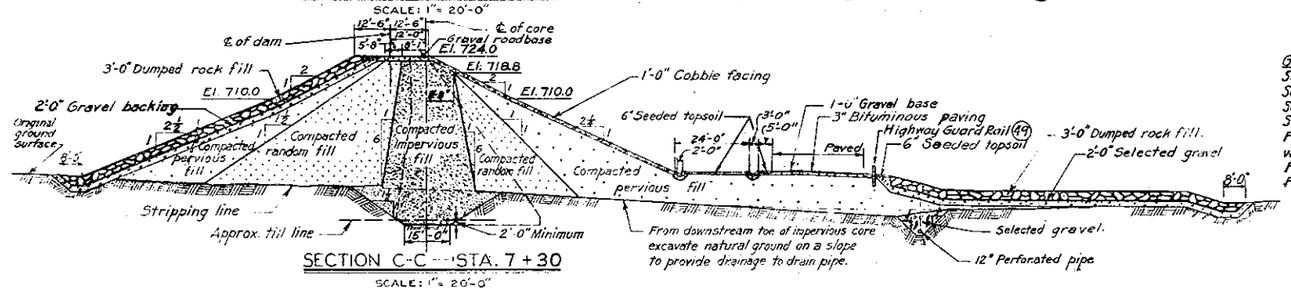
SECTION B-B — STA. 6+00

SCALE: 1" = 20'-0"



SECTION F-F — STA. 18+00

SCALE: 1" = 20'-0"



SECTION C-C — STA. 7+30

SCALE: 1" = 20'-0"

General Notes

See sheet 18 for additional sections.
See sheet 13 for camber details.
See sheet 13 for estimated quantities of earthwork.
See sheet 18 for detail of top of dam.
Figures in circles indicate item number under which payment will [illegible]
For limits of embankment Sections see sheet 2
For location of toe drain see sheet 22

NO.	CHARACTER	DATE	BY
	REVISIONS		

MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON DAM
CONTOOCOOK RIVER

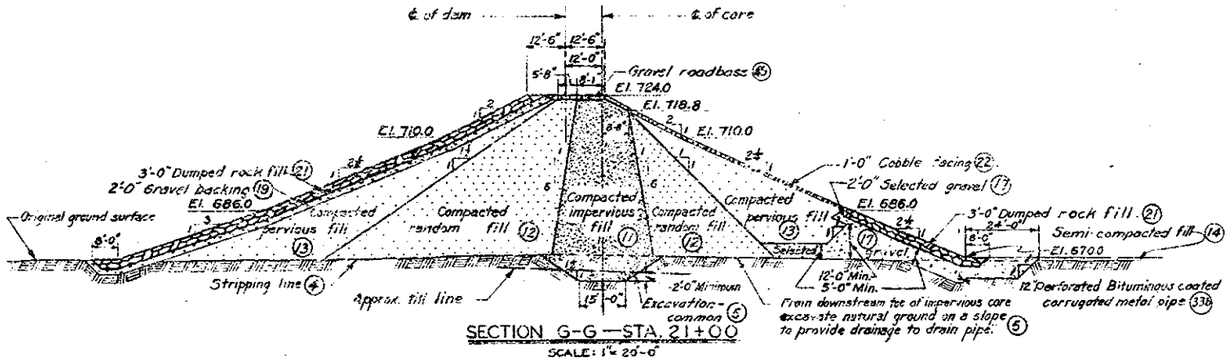
EMBANKMENT SECTIONS NO. 1

IN 56 SHEETS — SHEET NO. 17 — SCALE: 1" = 20'-0"

U. S. ENGINEER OFFICE, BOSTON, MASS. — JULY 1946

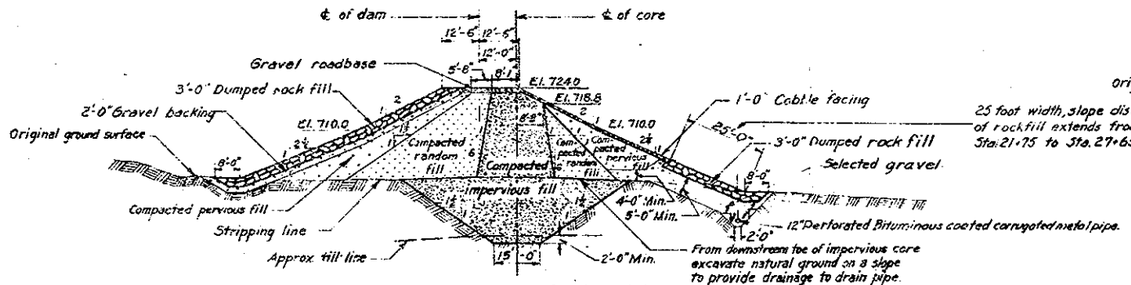
APPROVAL RECOMMENDED: John E. Allen, Chief, Design Branch
APPROVED: [illegible], Colonel, Corps of Engineers, District Engineer
PREPARED: [illegible]
CHIEF, MAIN DIV.: [illegible]
DR. BY J.S.M. — TR. BY G.W.T. — TR. BY J.W.T. — CK. BY A.C.O.
FILE NO. M19-53/5

PLATE A-14



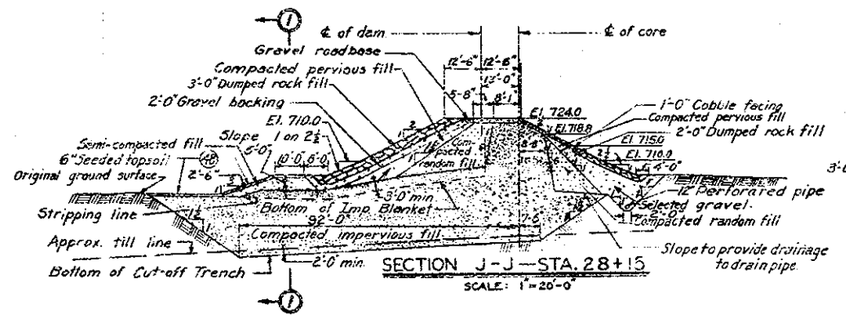
SECTION G-G — STA. 21+00

SCALE: 1" = 20'-0"



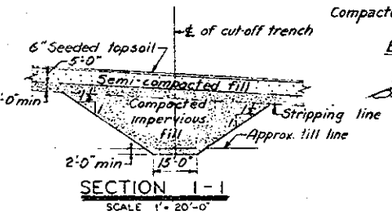
SECTION H-H — STA. 25+00

SCALE: 1" = 20'-0"



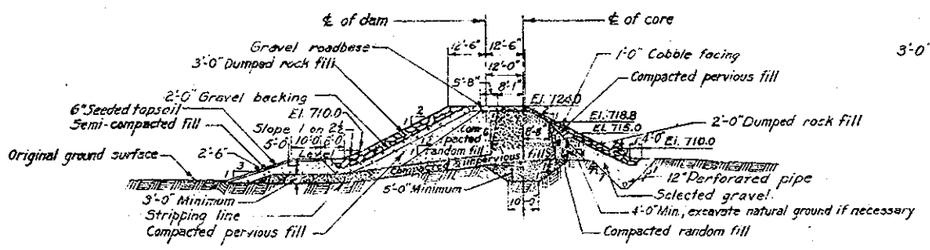
SECTION J-J — STA. 28+15

SCALE: 1" = 20'-0"



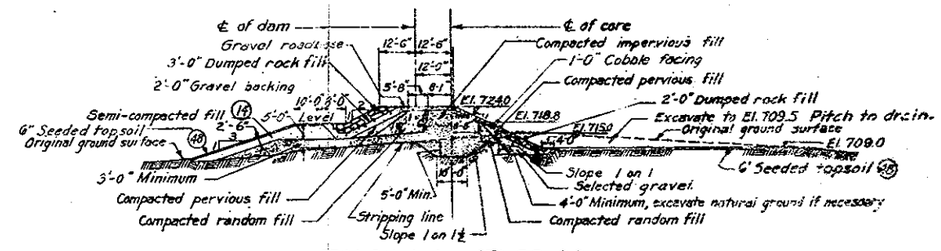
SECTION I-I

SCALE 1" = 20'-0"



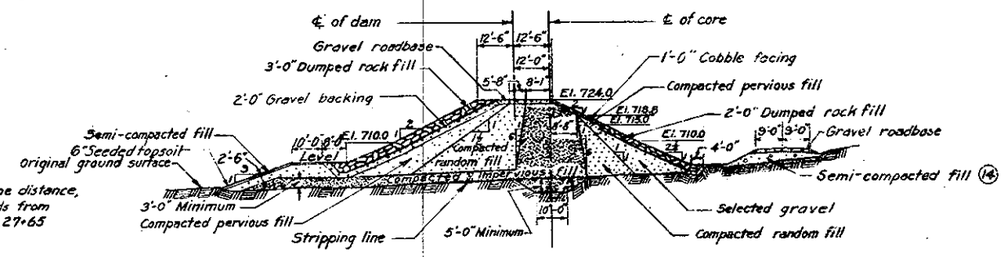
SECTION K-K — STA. 28+60

SCALE: 1" = 20'-0"



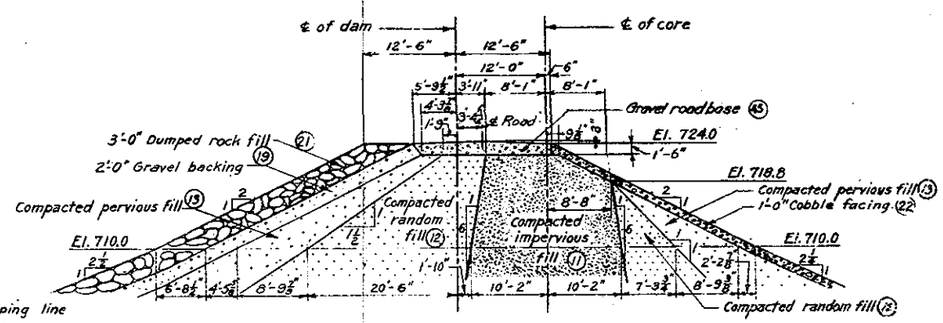
SECTION L-L — STA. 30+00

SCALE: 1" = 20'-0"



SECTION M-M — STA. 35+00

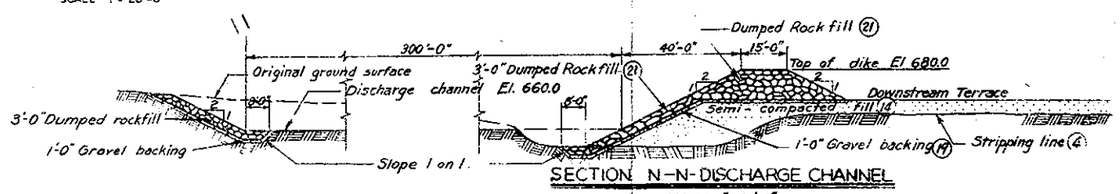
SCALE: 1" = 20'-0"



DETAIL - TOP OF DAM

SCALE: 1" = 8'-0"

Note: Top of dam is to be cambered along axis as shown on sheet 13



SECTION N-N-DISCHARGE CHANNEL

SCALE: 1" = 20'-0"

General Notes

For limits of embankments sections see sheet 2 & 19

For location of toe drain see sheet 22

Figures in circles indicate item number under which payment will be made.

For estimated quantities of earth work see sheet 13

NO.	CHARACTER	DATE	BY
	REVISIONS		

MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON DAM
CONTOOCOOK RIVER

EMBANKMENT SECTIONS NO. 2

IN 56 SHEETS — SHEET NO. 18 — SCALE: 1" = 20'-0"

U. S. ENGINEER OFFICE, BOSTON, MASS. — JULY, 1946

APPROVAL RECOMMENDED: John E. Allen — CHIEF, ENGINEERING DIVISION

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

PREPARED: W W Hall

CHAS. T. MAIN, INC.

FILE NO. M19-53/6

PLATE A-15

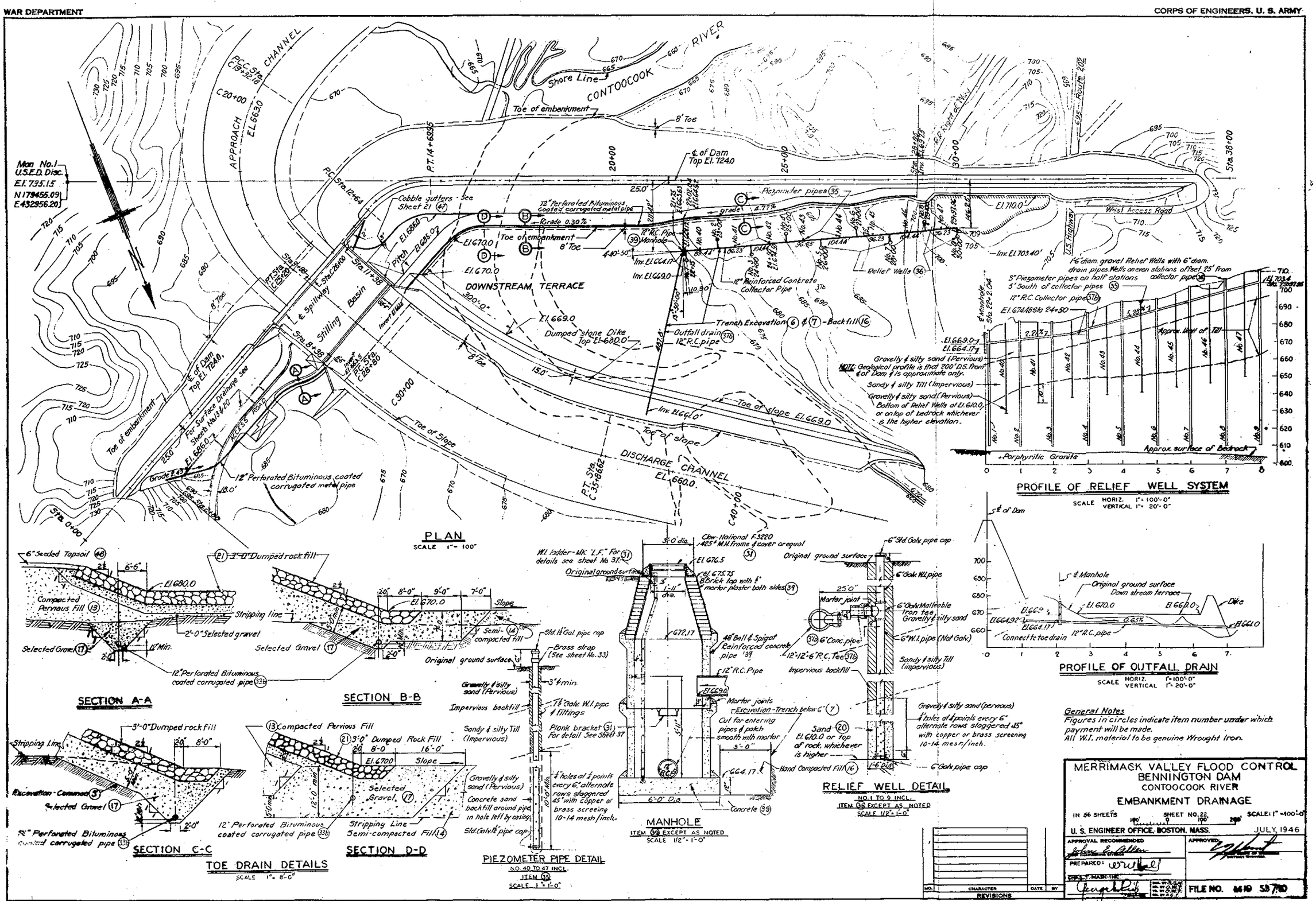
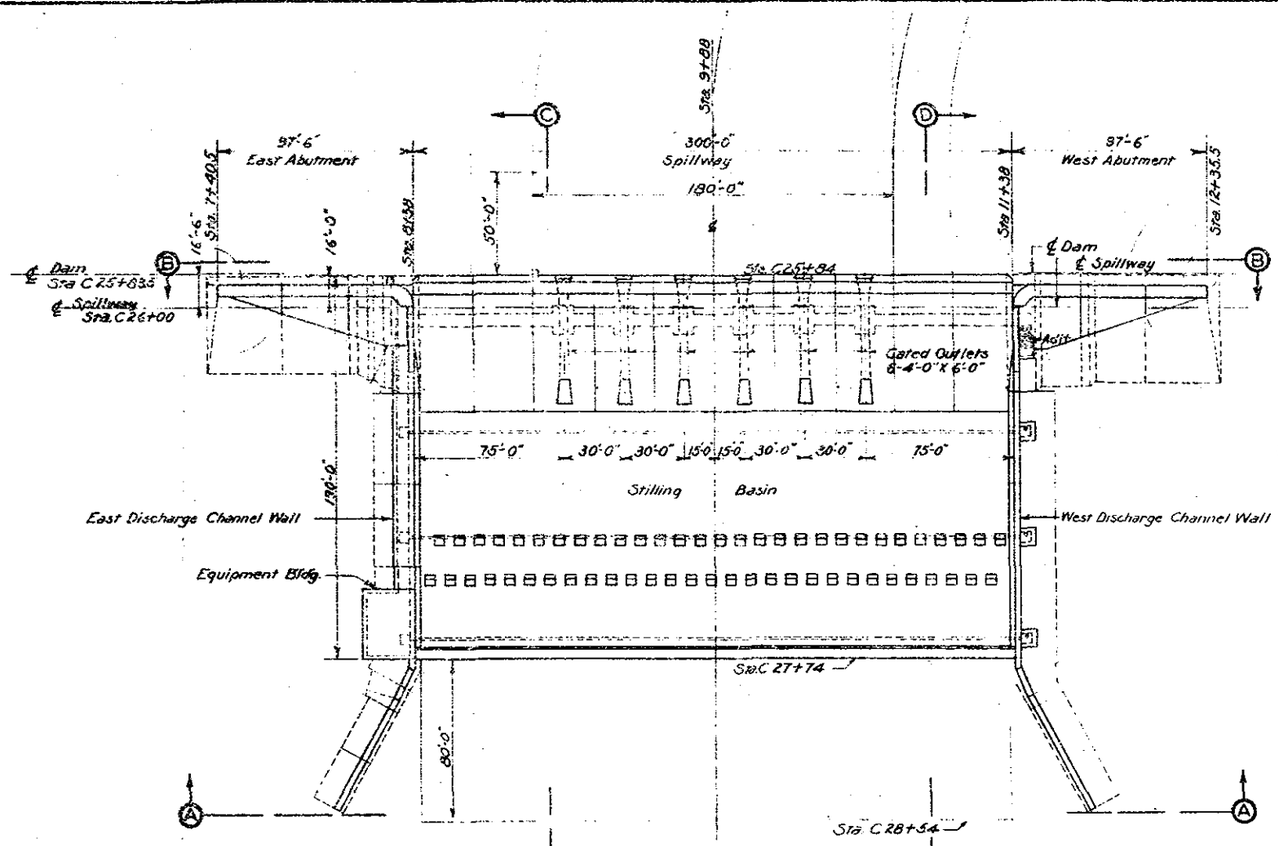
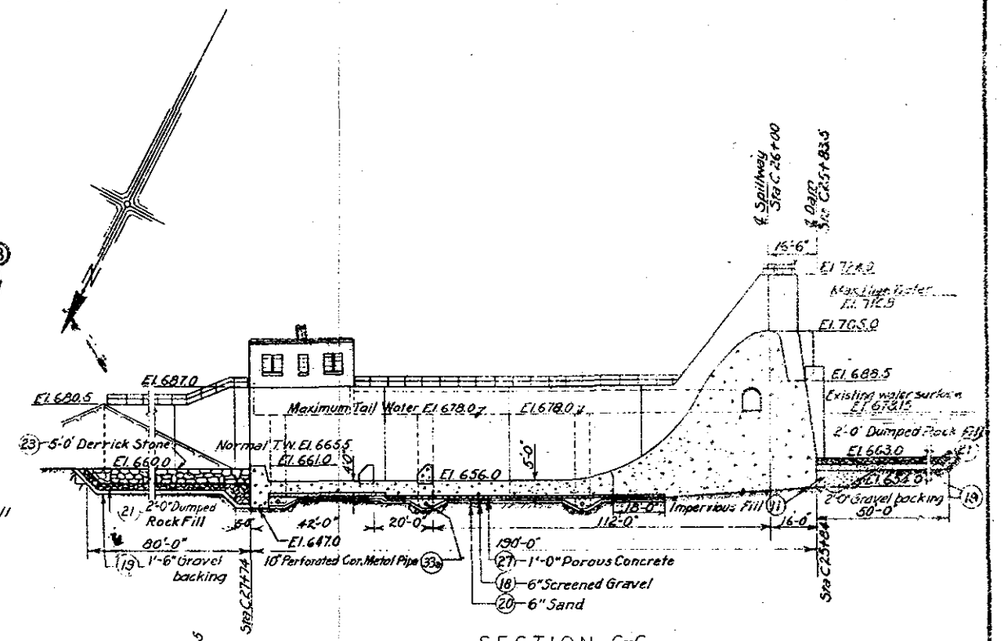

PLAN
SCALE 1" = 100'
CONTOOCOOK RIVER
Shore Line
APPROACH CHANNEL
DISCHARGE CHANNEL
DOWNSTREAM TERRACE
Stilling Basin
℄ Spillway
℄ of Dam Top El. 724.0
Toe of embankment
Toe of slope
Toe of slope El. 669.0
Dumped stone Dike Top El. 680.0
Trench Excavation (6) & (7) - Backfill (16)
Outfall drain (37b) 12" R.C. pipe
12" Reinforced Concrete Collector Pipe
Relief Wells (36)
Piezometer pipes (35)
Cobble gutters - See Sheet 21 (47)
12" Perforated Bituminous Coated corrugated metal pipe
Grade 0.30%
Grade 4.77%
Manhole
Man No. 1 U.S.E.D. Disc. El. 735.15 N 179455.09 E 432956.20
West Access Road
U.S. Highway Route 202
Access Road
For Surface Drainage see Sheets No. 3 & 20
Grade 2.43
Inv. El. 703.40'
Inv. El. 669.0'
Inv. El. 664.17
Inv. El. 660.0'
Sta. 38+00
Sta. 0+00
Sta. 8+36
Sta. 11+38
P.C. Sta. C 19+32.8
P.T. Sta. C 35+86.62
P.C. Sta. C 26+80
P.T. 14+69.95
P.C. Sta. 12+64
R.R. Right of Way
Gravelly & silty sand (Pervious)
Sandy & silty Till (Impervious)
Gravelly & silty sand (Pervious)
Bottom of Relief Wells at El. 610.0, or on top of bedrock whichever is the higher elevation.
NOTE: Geological profile is that 200' D.S. from ℄ of Dam & is approximate only.
Approx. surface of Bedrock
Porphyritic Granite
Approx. limit of Till
PROFILE OF RELIEF WELL SYSTEM
SCALE HORIZ. 1" = 100'-0" VERTICAL 1" = 20'-0"
PROFILE OF OUTFALL DRAIN
SCALE HORIZ. 1" = 100'-0" VERTICAL 1" = 20'-0"
Connect to toe drain
Original ground surface
Down stream terrace
Dike
SECTION A-A
SECTION B-B
SECTION C-C
SECTION D-D
TOE DRAIN DETAILS
SCALE 1" = 8'-0"
Compacted Pervious Fill (13)
Selected Gravel (17)
Stripping line
Semi-compacted Fill (14)
12" Perforated Bituminous coated corrugated pipe (33b)
Excavation - Common (3)
PIEZOMETER PIPE DETAIL
NO. 40 TO 47 INCL.
ITEM (35)
SCALE 1" = 1'-0"
MANHOLE
ITEM (39) EXCEPT AS NOTED
SCALE 1/2" = 1'-0"
RELIEF WELL DETAIL
NO. 1 TO 9 INCL.
ITEM (38) EXCEPT AS NOTED
SCALE 1/2" = 1'-0"
Clow-National F-3220 .425" M.H. frame & cover or equal
Brick top with 6" mortar plaster both sides (39)
48" Ball & Spigot Reinforced concrete pipe (39)
Cut for entering pipes & pointed smooth with mortar
Sand (20) El. 610.0 or top of rock, whichever is higher
Hand Compacted Fill (16)
Impervious backfill
Gravelly & silty sand (pervious)
6" Std Galv. pipe cap
6" Galv. W.I. pipe
6" Galv. Malleable Iron Tee
6" W.I. pipe (Not Galv.)
12" R.C. Tee (37b)
6" Conc. pipe (37b)
General Notes
Figures in circles indicate item number under which payment will be made.
All W.I. material to be genuine Wrought iron.
MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON DAM
CONTOOCOOK RIVER
EMBANKMENT DRAINAGE
IN 56 SHEETS
SHEET NO. 22
SCALE: 1" = 100'-0"
U. S. ENGINEER OFFICE, BOSTON, MASS.
JULY, 1946
APPROVAL RECOMMENDED
APPROVED
PREPARED:
CHKD.
FILE NO. 4-19 58/70
REVISIONS
NO. CHARACTER DATE BY

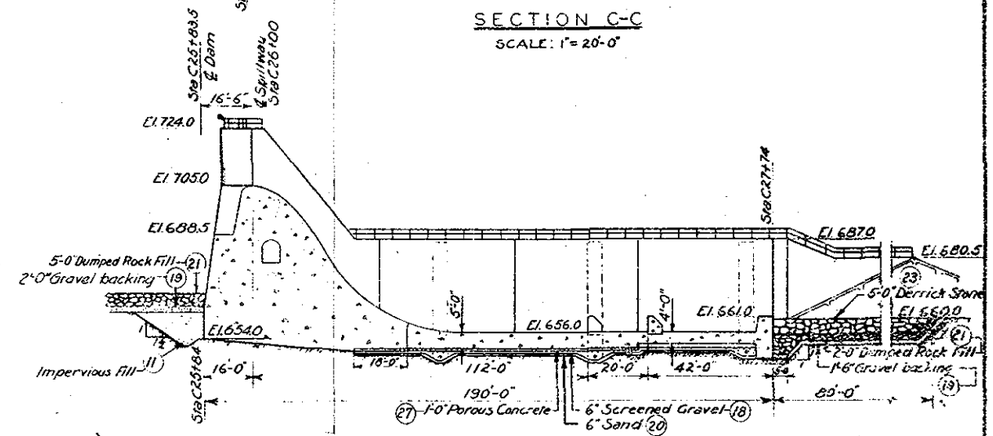
PLATE A-16



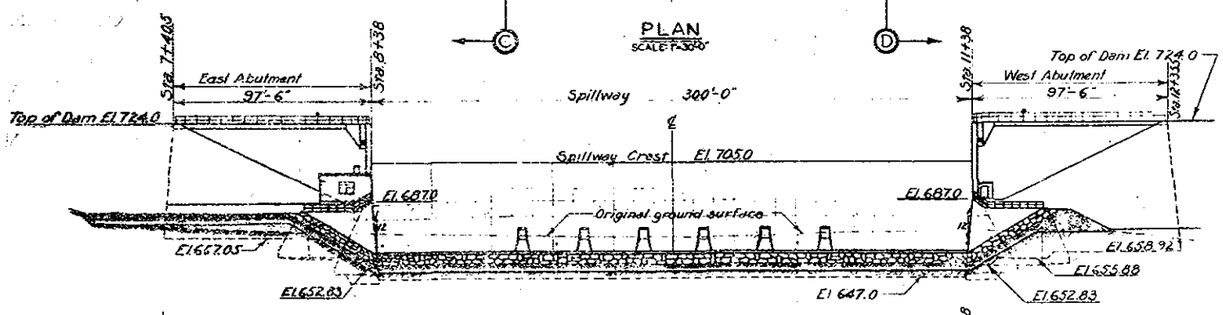
PLAN
SCALE: 1" = 30'-0"



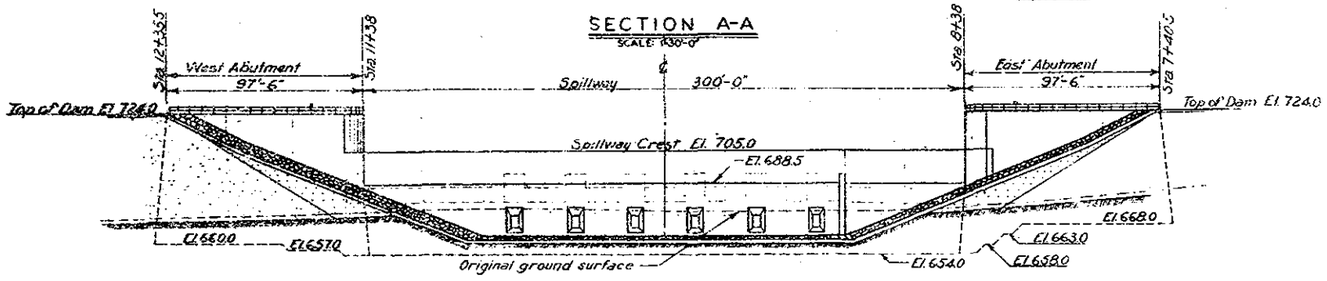
SECTION C-C
SCALE: 1" = 20'-0"



SECTION D-D
SCALE: 1" = 20'-0"



SECTION A-A
SCALE: 1" = 30'-0"



SECTION B-B
SCALE: 1" = 30'-0"

General Notes

For Abutments see Sheets 25, 26 and 27
For Discharge Channel Walls see Sheets 28 and 29
For Spillway see Sheets 30 and 31
For Stilling Basin see Sheet 32
Figures in circles indicate item number under which payment will be made.

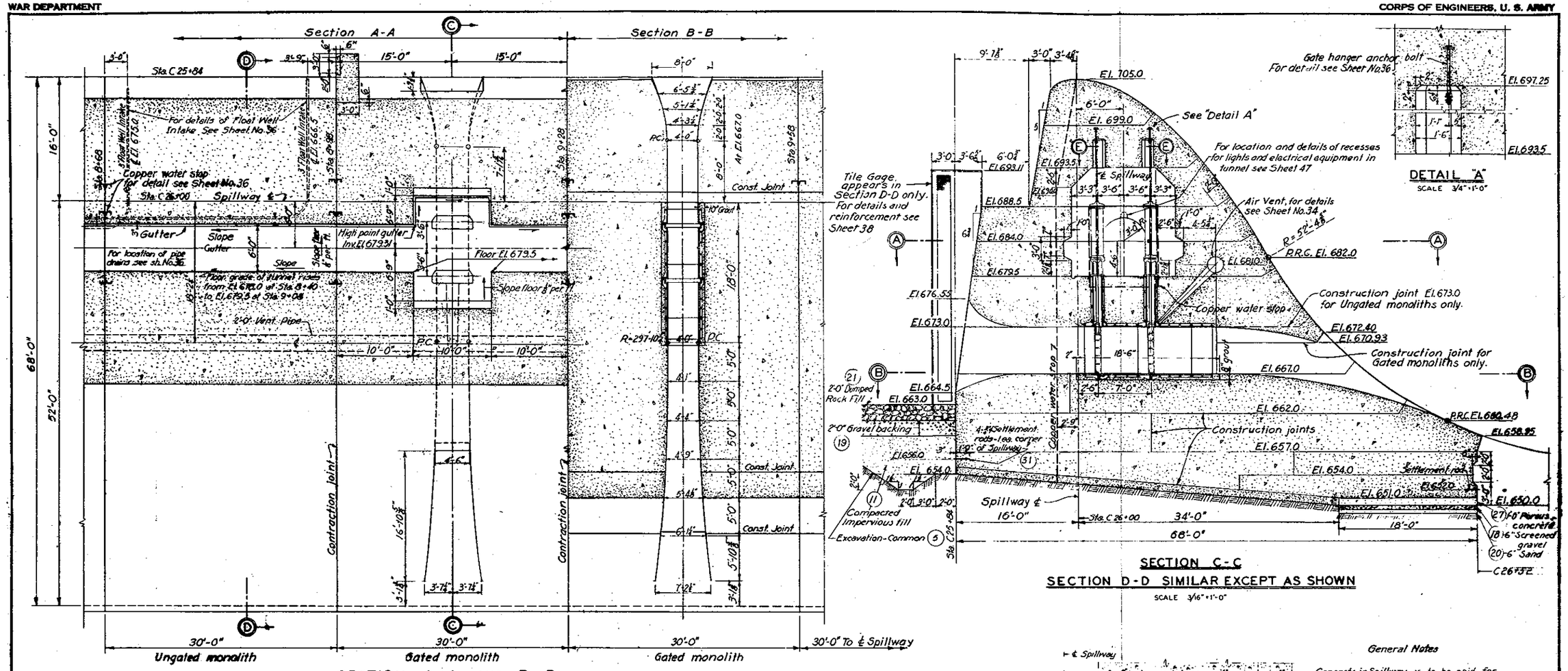
MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON DAM
CONTOOCOOK RIVER
MASONRY STRUCTURES
GENERAL PLAN AND ELEVATIONS
IN 55 SHEETS — SHEET NO 24 — SCALE: 1" = 30'-0"

U. S. ENGINEER OFFICE, BOSTON, MASS. — JULY 1946

APPROVAL RECOMMENDED: Edward Fuller
APPROVED: C. P. Hunt
PREPARED: W. W. Hall
CHAS. T. MAIN INC.
DES. BY L.E.P. DR. BY A.C.C. TR. BY A.C.C. CK. BY P.J.T.
FILE NO. M 19-54/1

NO.	CHARACTER	DATE	BY
	REVISIONS		

PLATE A-17



General Notes

Concrete in Spillway is to be paid for under item (24).
Figures in circles indicate item number under which payment will be made.
All exposed edges and joints to be formed to a 1"-45° except for joints in downstream face of Spillway.
All external horizontal surfaces to have slope of 1/8" per ft.
For detail of Spillway reinforcing see Sheet 31
For reinforcing schedule see Sheet 50
For detail of passageway drainage see Sheet 36
For location and detail of spillway piezometer well see Sheet 33
Conduit liner, gates and equipment furnished by gov't. to be erected under item (42)
All contraction joints shall be painted with asphalt emulsion 12" wide from outside edges and around openings.

NO.	CHARACTER	DATE	BY
	REVISIONS		

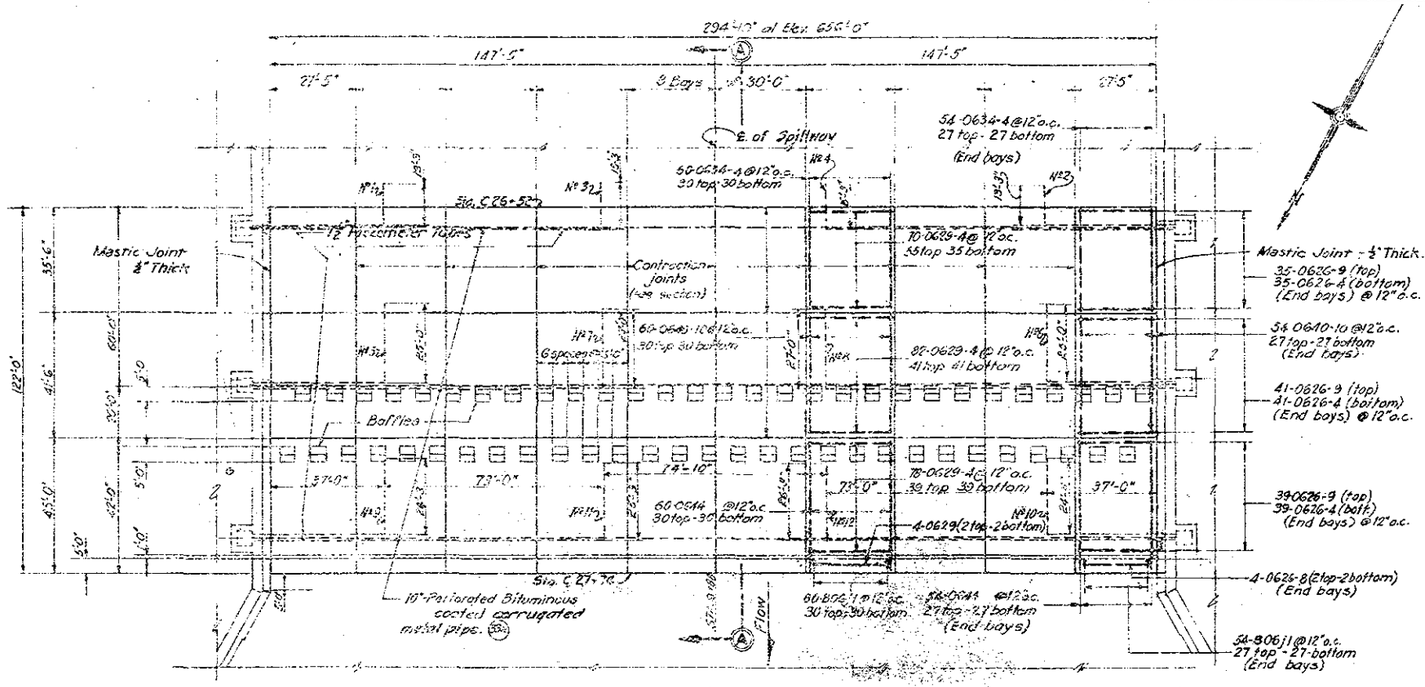
MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON DAM
CONTOOCOOK RIVER
SPILLWAY SECTIONS

IN 56 SHEETS — SHEET NO 30 — SCALE 3/16" = 1'-0"

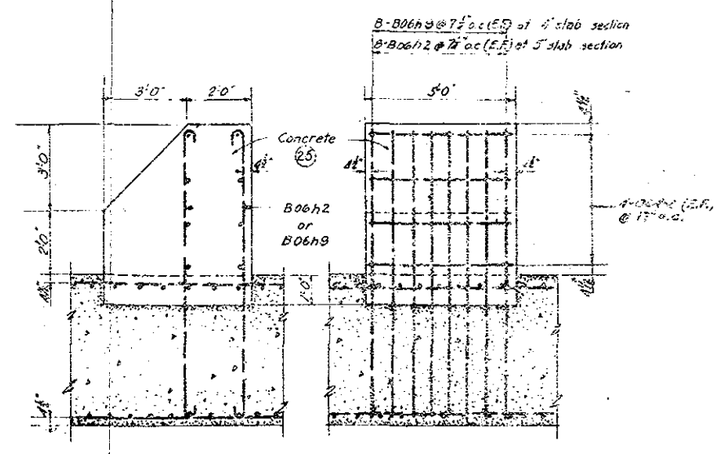
U. S. ENGINEER OFFICE, BOSTON, MASS. JULY 1946

APPROVAL RECOMMENDED: John E. Allen
PREPARED: W. W. Hall
CHAS. T. MAIN INC.
APPROVED:

FILE NO. M 19-56/7

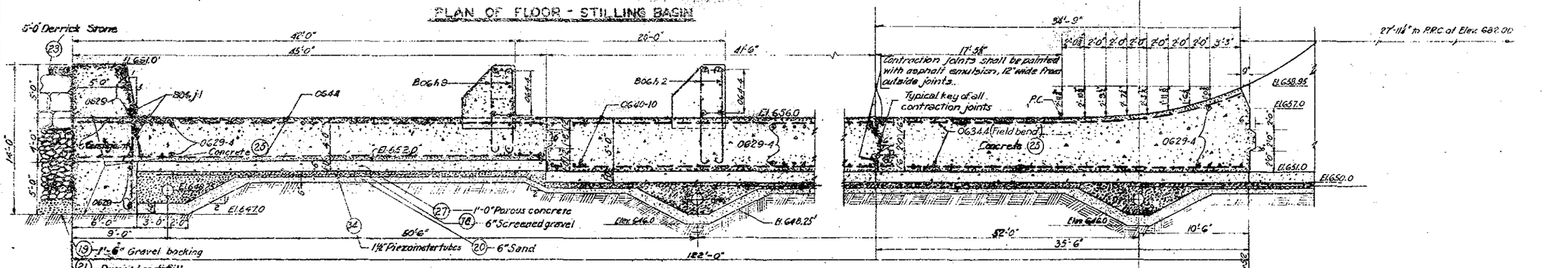


PLAN OF FLOOR - STILLING BASIN



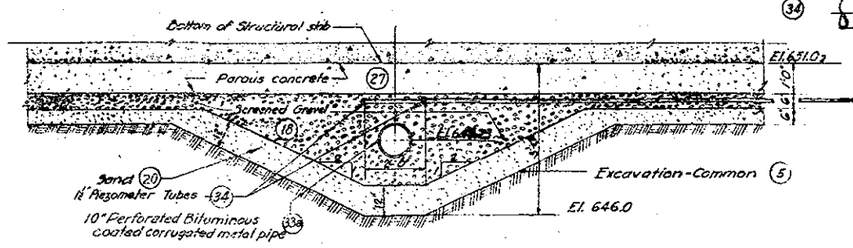
DETAIL OF BAFFLES

SCALE 1/2" = 1'-0"



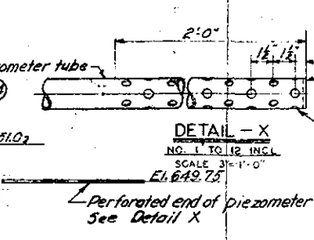
SECTION A-A

SCALE: ¼" = 1'-0"



TYPICAL DRAIN DETAIL

SCALE: ½" = 1'-0"



DETAIL - X

NO. 1 TO 12 INCL.

SCALE 3" = 1'-0"

General Notes:

Concrete in Stilling Basin slab, end sill and baffles is to be paid for under item (25)

For reinforcing schedule see Sheet 50

All reinforcing furnished, bent and installed to be paid for under item (29)

Figures in circles indicate item number under which payment will be made.

For detail of piezometer tubes in wells see Sheets 28, 29 & 35

All reinforcing shown on this sheet to have not less than 4" cover unless otherwise noted.

MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON DAM
CONTOOCOOK RIVER
STILLING BASIN
PLAN AND SECTIONS

IN 56 SHEETS — SHEET NO. 32 — SCALE 1" = 20'-0"

U. S. ENGINEER OFFICE, BOSTON, MASS. — JULY, 1946

APPROVAL RECOMMENDED: John L. Allen
CHIEF, DESIGN BRANCH, ENGINEERING DIVISION

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

DESIGNED: W. M. Hall

DRAWN: [illegible]

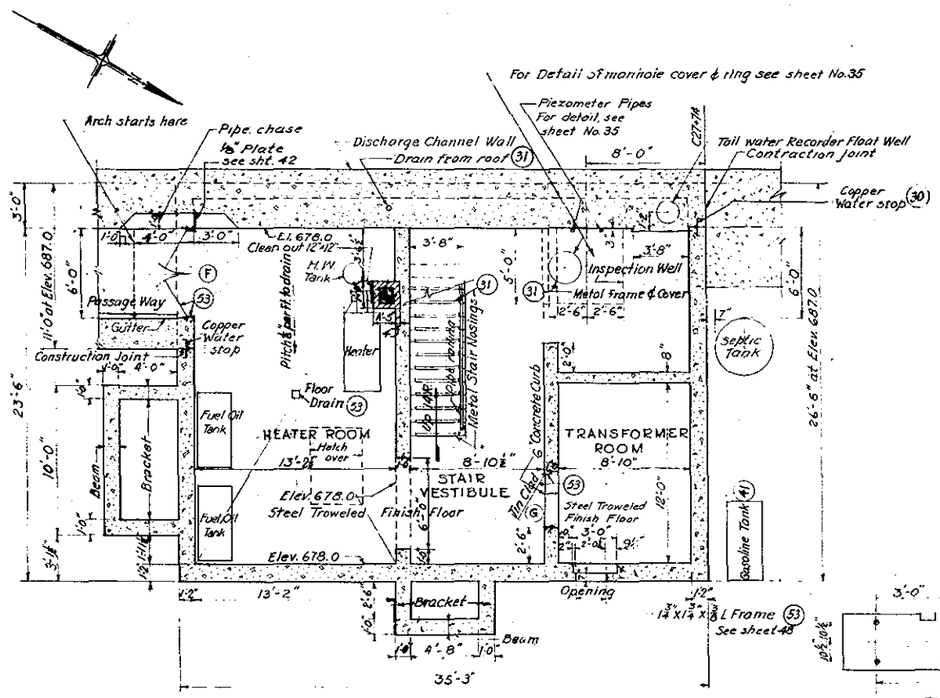
DES. BY [illegible]
DR. BY [illegible]
TR. BY [illegible]
CK. BY [illegible]

FILE NO. M19-34/9

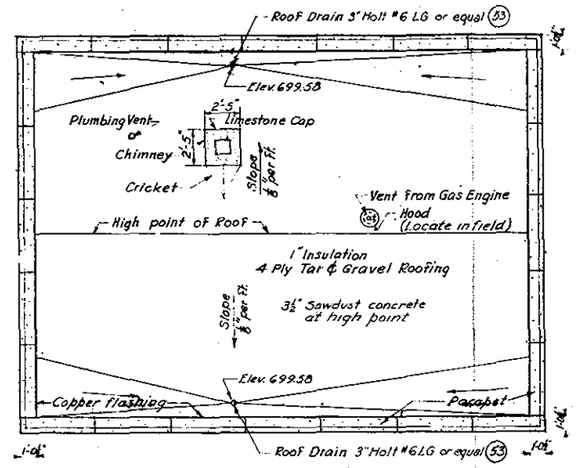
NO.	CHARACTER	DATE	BY

REVISIONS

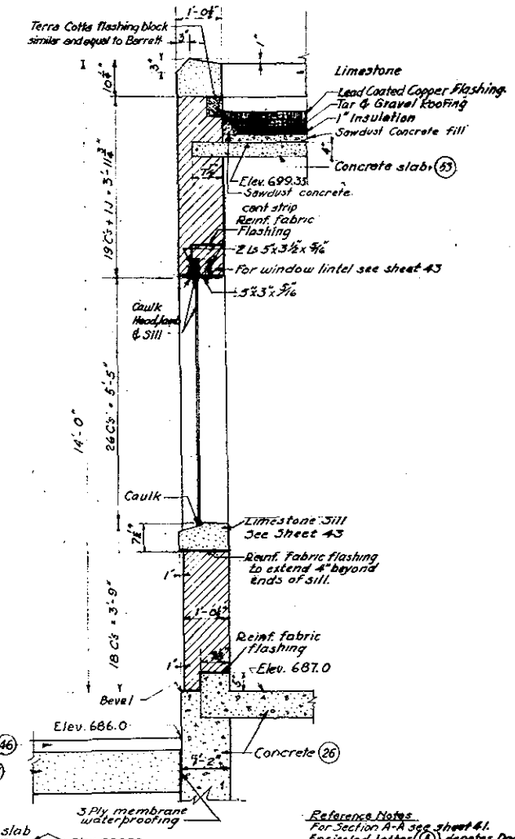
PLATE A-19



BASEMENT PLAN



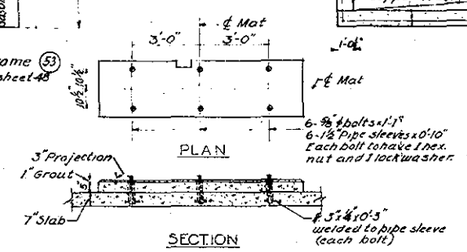
ROOF PLAN



SECTION C-C

SCALE 3/4" = 1'-0"

TYPICAL WALL SECTION

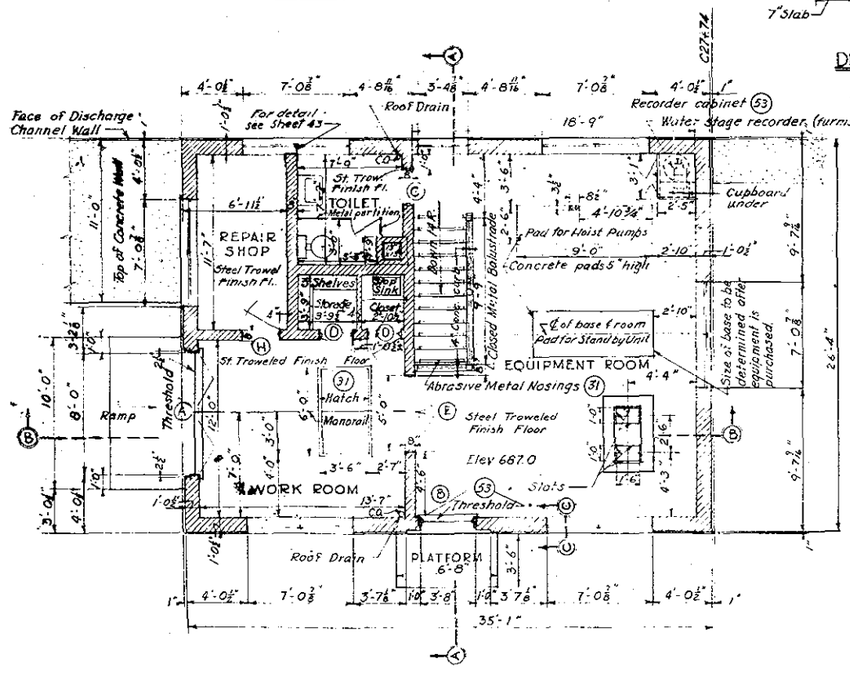


PLAN

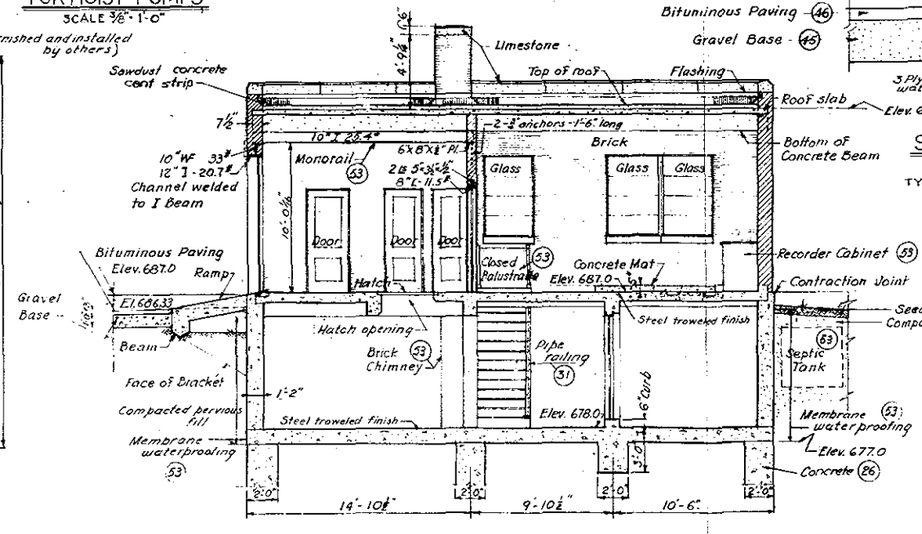
SECTION

DETAIL OF ANCHOR BOLTS FOR HOIST PUMPS

SCALE 3/8" = 1'-0"



GROUND FLOOR PLAN



LONGITUDINAL SECTION B-B

Bituminous Paving (46)

Gravel Base (45)

Reference Notes

For Section A-A see sheet 41.
Encircled letter (A) denotes Door Type, for details see Sheet 42.
For hatch cover details in Work Room see Sheet 42.
For details of sash, stairs, toilet compartment, recorder cabinet etc. see sheet 43.
For grading around building see sheet 13.

General Notes

The Equipment Building will be paid for as specified under item 53.
The electrical system complete will be paid for under item 46
The substructure will be paid for under the respective pay items as indicated

NO.	CHARACTER	DATE	BY
	REVISIONS		

MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON DAM
CONTOOCOOK RIVER
EQUIPMENT BUILDING
PLANS AND SECTIONS

IN 40 SHEETS SHEET 40 SCALE 1/4" = 1'-0"

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

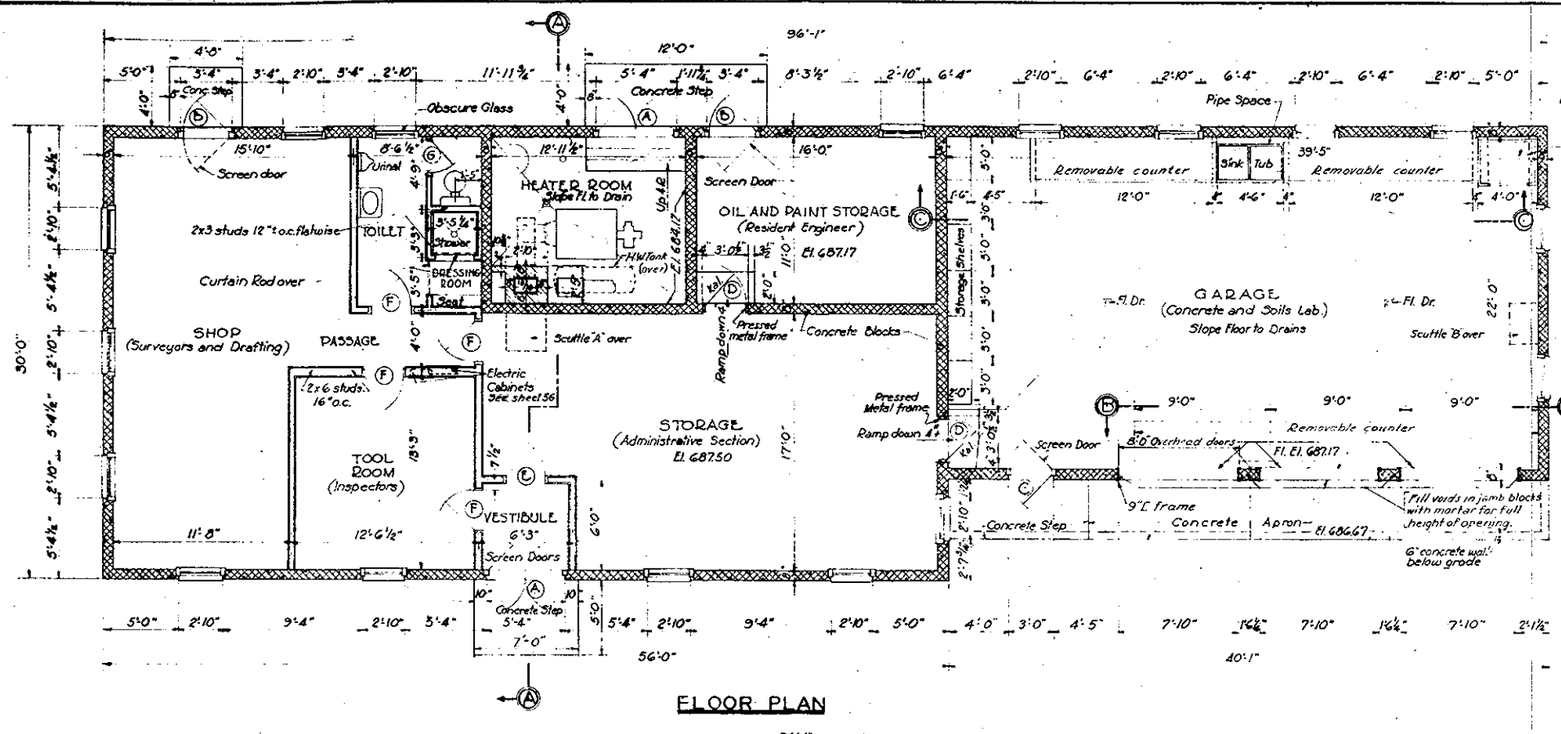
APPROVAL RECOMMENDED: John L. Allen, Chief, Engineering Division

PREPARED: W. W. Hall

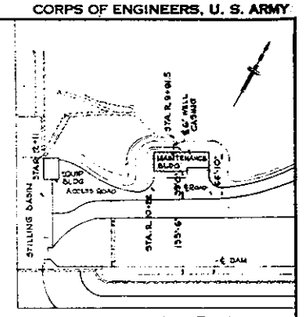
CHECKED: George Reid

APPROVED: [illegible], Colonel, Corps of Engineers, District Engineer

FILE NO. M 19-56/1



FLOOR PLAN



LOCATION PLAN

SCALE 1" = 100 FT.

General Notes:

The Maintenance Building complete as shown on Sheets 51-56 incl., including excavation, backfill inside the building walls, heating, plumbing and electrical work will be paid for under Item 54 as specified. Stripping, backfill outside the building walls, seeding, gravel walks, cement and bituminous paving, as shown or specified, will be paid for under their respective pay items.

All floor slabs, exterior steps and aprons to be reinforced with 6"x6" #8 gs wire mesh weighing 58 lbs. per 100 square feet, placed approximately in the centre of the slabs. Floor slabs to have steel troweled finish; steps, ramps, apron and exterior thresholds to have wood float finish.

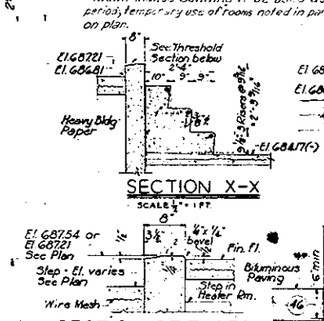
All interior stud partitions, except as shown, to be 2x4 studs 16" o.c. All sole pieces to be anchored to the concrete floor slabs with approved type metal anchors, at all corners, jambs of openings.

Both sides of all stud partitions, except in Toilet, to be covered with ½" plaster board full height with a wood chair rail 3'-0" above the floor. The Toilet Room is to have a ½" cement asbestos board dado 4'-0" high with plaster board above except in the shower where the cement asbestos board is to extend full height.

All ceilings to be ½" plaster board on 1"x3" wood strapping 16" o.c. nailed to the ceiling joists. Ceilings of the Heater Room and Oil and Paint Storage Rooms to have ⅛" plaster board batten strips between the upper side of the plaster board and the wood strapping at all joints of the plaster board.

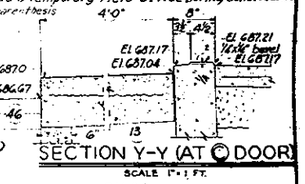
Cement will be paid for under Item 58.

Maintenance Building to be used as a Temporary Field Office during construction period; temporary use of rooms noted in parenthesis on plan.



SECTION X-X

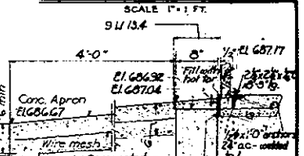
SCALE 1" = 1 FT.



SECTION Y-Y (AT ⓒ DOOR)

SCALE 1" = 1 FT.

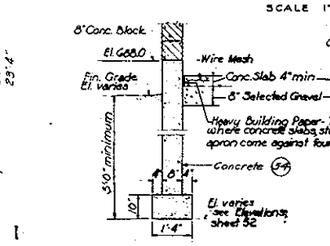
SECTION OF THRESHOLDS FOR Ⓐ AND Ⓑ DOORS



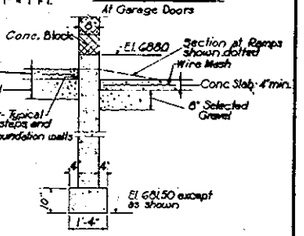
SECTION Z-Z

at Garage Doors

SCALE 1" = 1 FT.

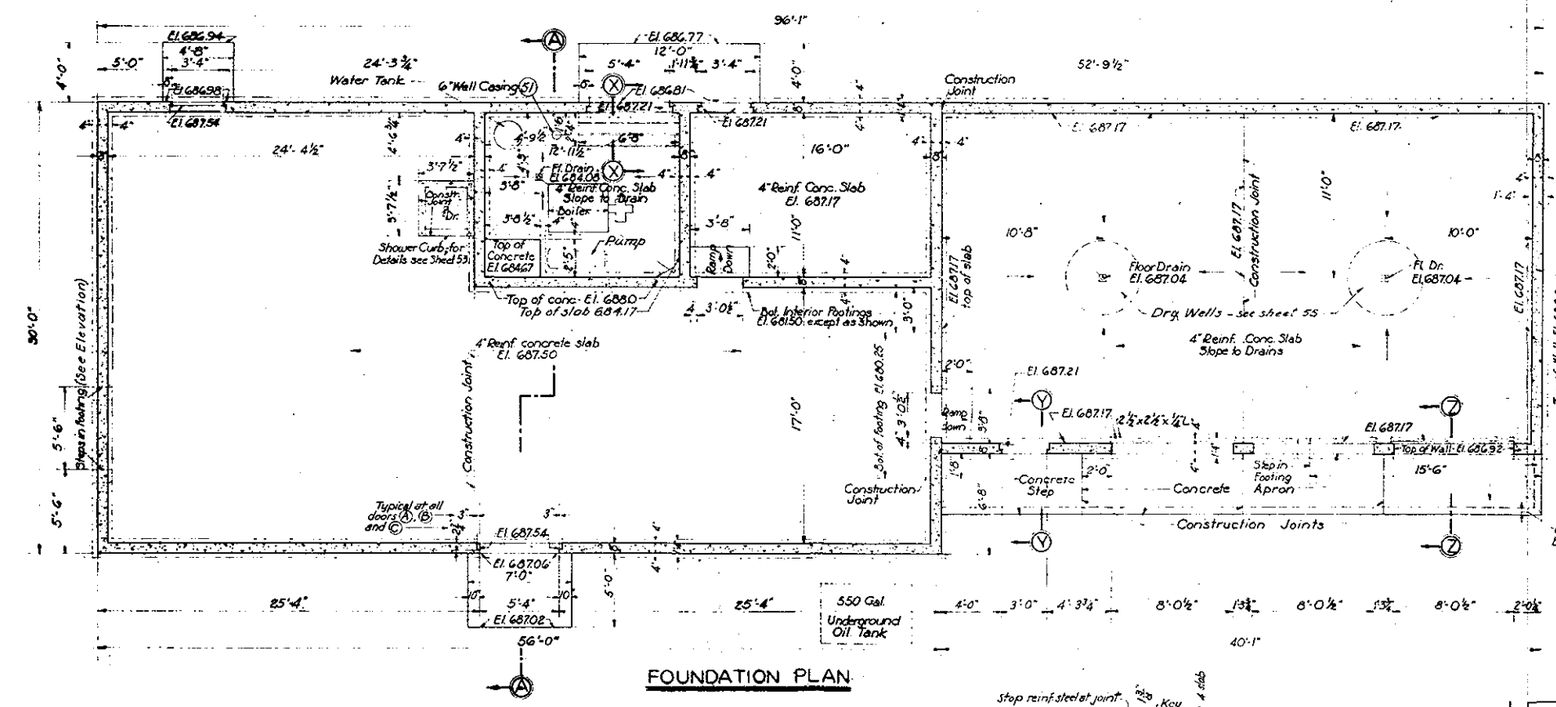


TYPICAL SECTION OF EXTERIOR FOUNDATION WALL

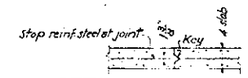


TYPICAL SECTION OF INTERIOR FOUNDATION WALL

SCALE ½" = 1 FT.



FOUNDATION PLAN



DETAIL OF CONSTRUCTION JOINTS IN SLABS

SCALE 1" = 1 FT.

Reference Sheets:

For Section A-A and Toilet and Shower Details, see Sheet 53.
For Door Schedule and Door Frame Details, see Sheet 53.
For Elevations B-B and C-C and Ladder and Scuttle Details, see Sheet 54.
For Grading around Building, see Sheet 15.

MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON DAM
CONTOOCOOK RIVER

MAINTENANCE BUILDING
PLANS & DETAILS

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

APPROVAL RECOMMENDED: Chief, Design Branch

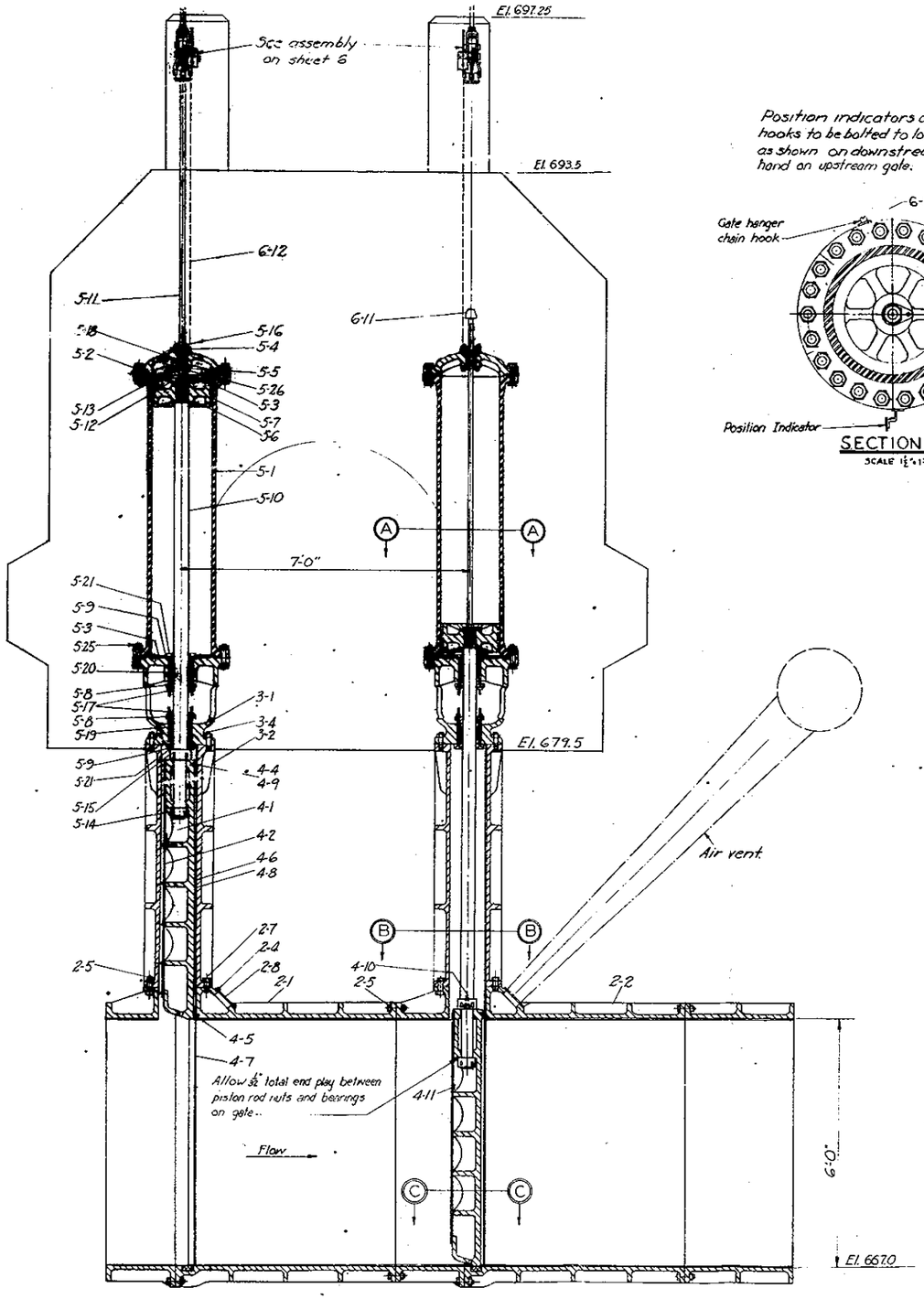
PREPARED: W. H. Hall, Charles T. Main, Inc.

APPROVED: Colonel, Corps of Engineers, District Engineer

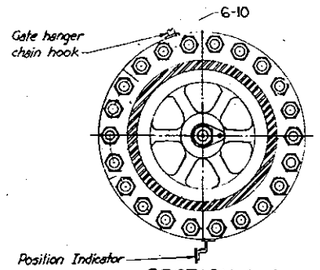
NO.	CHARACTER	DATE	BY
	REVISIONS		

FILE NO. M19-53/1

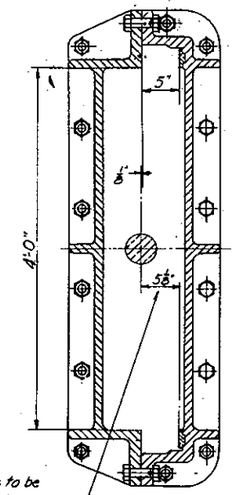
PLATE A-21



Position indicators and gate hanger chain hooks to be bolted to lower cylinder flanges as shown on downstream gate and opposite hand on upstream gate.

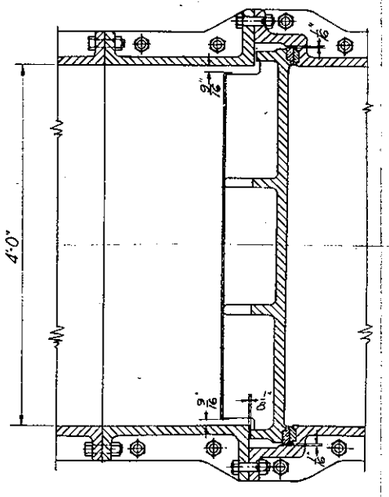


SECTION A-A
SCALE 1½"=1'-0"



Dimensions of parts to be carefully held to, so as to maintain this dimension

SECTION B-B
SCALE: 1½"=1'-0"



SECTION C-C
SCALE: 1½"=1'-0"

BILL OF MATERIAL FOR SIX CONDUITS

MARK	DESCRIPTION	MATERIAL	MAKE
2-1	Upstream conduit liner	Cast iron QQ-I-652	6
2-2	Downstream conduit liner	" " " " "	6
2-3	Fitted bolt	Bolt steel FF-B-571a	288
2-4	⅝"N.C. x 1" Cap screw	" " "	144
2-5	1"N.C. x 4" Machine bolt & hex. nut	" " "	1080
2-6	2" Solid square head pipe plug	Cast iron	288
2-7	1"N.C. x 3" Cap screw	Bolt steel FF-B-571a	48
2-8	Cover plate	Cast iron	6
3-1	Bonnet cover	Cast iron QQ-I-652	12
3-2	Bonnet	" " " " "	12
3-3	#10 Standard Taper Pin 4" long	Bolt Steel	24
3-4	Bonnet stud	Bolt Steel FF-B-571a	264
4-1	Gate	Cast iron QQ-I-652	12
4-2	Gate cover	Wrought iron A.S.T.M. A-162-39	12
4-3	Cover stud	" " A.S.T.M. A-152-39	588
4-4	Gate top seal	Bronze Class C	12
4-5	Liner top seal	" " D	12
4-6	Gate side seal	" " C	24
4-7	Liner side seal	" " D	24
4-8	Bonnet seal	" " D	24
4-9	Seal screw	" " D	936
4-10	Gate hook	"	24
4-11	⅜"N.C. x ½" Binding hd. mach. screw	"	60
4-12	⅜" Dia. Pin 18" long	Brass	936
5-1	Cylinder	Forged or welded steel	12
5-2	Cylinder head	Cast iron QQ-I-652	12
5-3	Cylinder gasket	Copper Q-Q-C-501a	48
5-4	Gland	Cast bronze Q-Q-B-691b	12
5-5	Packing seat	" " " "	12
5-6	Piston	Cast iron QQ-I-652	12
5-7	Piston ring	Manganese bronze QQ-B-726c	60
5-8	Gland	Cast bronze QQ-B-691b	24
5-9	Packing seat	" " " "	24
5-10	Piston rod	Aluminum bronze QQ-B-666	12
5-11	Piston rod extension	" " " "	12
5-12	Washer	Steel	12
5-13	Lock plate	"	12
5-14	Piston rod nut	Aluminum bronze QQ-B-671a	12
5-15	" " "	" " " "	12
5-16	Gland stud	Bronze	24
5-17	" "	"	96
5-18	Packing	Garlock Chevron 431 or equal	13
5-19	"	" " 431 " "	13
5-20	"	" " 431 " "	13
5-21	⅜"N.C. x 1¾" Fil. hd cap screw	Bronze	96
5-22	1½"N.C. Hex. jam nut	"	12
5-23	⅛" x 5½" cotter pin	"	24
5-24	⅜"N.C. x 1" Cap screw	Bolt steel	12
5-25	1¼"N.C. x 5¾" Mach. bolt & hex. nut	" "	528
*5-26	⅜"N.C. x 1¾" Cap screw	Bronze	36
6-1	Body	Steel	12
6-2	Hook	Cast steel QQ-S-681-b	24
6-3	Cam	" " " " " "	12
6-4	Hook pin	Bronze	12
6-5	Cam lever	Steel	12
6-6	Counterweight	Cast iron QQ-I-652	24
6-7	Safety stud	Red brass	24
6-8	Cam pin	Bronze	12
6-9	Stop stud	Bolt steel	24
6-10	Chain hook	Steel	12
6-11	Latch	"	12
6-12	Chain assembly	Bronze	12
6-13	Gage	Brass QQ-B-611a. & Steel	12
6-14	Clamp	Steel	12
6-15	Pointer	Brass QQ-B-611a	12
6-16	Pointer rod	Bolt steel	12
6-17	⅛" x 1½" Cotter pin	Bronze	24
6-18	#6 x 2" Taper pin	Bolt steel	24
6-19	#10-24 x ½" Mach. screw & hex. nut	Brass	168
6-20	⅜"N.C. x 1½" Mach. bolt & hex. nut.	Bolt steel	48
6-21	¾"N.C. Hex. Jam nut	Bronze	72
6-22	½"N.C. x ¾" Cap screw	Bolt steel	24
6-23	⅝" Spring lock washer	Bronze	24
6-24	½"N.C. x 3½" Mach. bolt & Hex. nut	Bolt steel	12
6-25	½"N.C. x ⅞" Cap screw	" "	24

MERRIMACK VALLEY FLOOD CONTROL
BENNINGTON DAM
CONTOOCOOK RIVER
4'-0" X 6'-0" SLIDE GATE ASSEMBLY

IN 6 SHEETS — SHEET NO. 1 — SCALE 3/4"=1'-0"

U. S. ENGINEER OFFICE, BOSTON, MASS. — JULY, 1946

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

PREPARED: W. W. Hall, Chas. T. Main, Inc.

APPROVED: Colonel, Corps of Engineers, District Engineer

DWN. BY J.H.S. / CK. BY D.K.M. / TR. BY D.K.M. / CH. BY ACB

FILE NO. M 19-60/1

NO.	CHARACTER	DATE	BY
	REVISIONS		

PLATE A-22

ANALYSIS OF DESIGN

BENNINGTON DAM

APPENDIX B

STRUCTURAL DESIGN COMPUTATIONS

FOREWORD

The computations comprising Appendix B are the computations made for the final structural design of the dam and its appurtenances.

ANALYSIS OF DESIGN

BENNINGTON DAM

APPENDIX B

STRUCTURAL DESIGN COMPUTATIONS

PLATES VIII-2 & VIII-3 Index

PLATES VIII-4 to VIII-85 (Inclusive) - Computations

PLATES VIII-56 & VIII-57 Gate & Hoist Computations

Plate VIII-2

CHAS. T. MAIN, INC. ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

Name: Bennington Dam
Subject: Design
Job No. US-16-15 Sheet No. 1-A
Date: Feb. 11 1946
Computed by:
Checked by:

Index

Description	Sheet No.
Index Sheets	1-A
" "	1-AA
Design Data	1-B
Shear Strength of Till diagram	1-C
Stability Analysis.	
Ultimate Ungated Section.	
" Sketch	1
" Computations	2
" Foundation Pressure	3
" Earthquake Stresses	4
" Location of Resultants	5
" Factor of Safety	6
Ultimate Gated Section.	
" Sketch	7
" Computations	8
" Earthquake Stresses	9
" Location of Resultants	10
" Foundation Pressure	11
" Factor of Safety	12
" Line of Resultants	12-A
Initial Gated Section.	
" Sketch	13
" Computations	14
" Foundation Pressure & Factor of Safety	15
Ultimate Gated Section.	
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" at El. 691	17
" at El. 679.5	18
" Computations	19
" at El. 667	20
" Computations	21
" Computations	22

Description	Sheet No.
Non-overflow Section.	
" Sketch	23
" Computations	24
" Earthquake Stresses	25
" Location of Resultants & Foundation Pressure	26
" Factor of Safety	27
" Line of Resultants	28
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" " at El. 678	30
" Computation	31
Stability Analysis.	
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" Sketch	32
" Foundation Pressure	33
" "	34
" Sketch	35
" Foundation Pressure	36
" "	37
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" "	40
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" Stilling Basin	43
" "	43-A
" "	43-B
" "	43-C
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Strength of Gate Leaf	45

Plate VIII-3

CHAS. T. MAIN, INC. ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

Name: Bennington Dam
Subject: Design
Job No. US-16-15 Sheet No. 1-AA
Date: Feb. 11 1946
Computed by: B.M.B.
Checked by:

Index

Description	Sheet No.
Equipment Bldg.	
Roof design, Plan.	46
" Slab	47
" Beam - RB1	48
" Lintel	49
Ground Floor, Plan	50
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" Beam B2	53
" Beam B3	54
" " stirrups	55
" Beam B5	56
" Beam B4	57
" Beams B5 & B6	57-A
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" Slab	58
Monolith with Equip. Bldg.	
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" Sketch	59
" Loading	60
" Beam - BR	61
" Header - BH	62
" Bracket - R3	63
" " " - R2	64
" Sketch of Bracket	65
" Location of Resultant of Loads in Equip. Bldg	66
Stability Analysis.	
Monolith with Equip. Bldg.	
" Sketch	67
" Computations	68
" Location of Resultant	69
" " "	70

Description	Sheet
Duct Line.	
Manholes.	
" Walls	71
" Slab	72

Plate VIII-4

WAR DEPARTMENT
U. S. ENGINEER OFFICE, BOSTON, MASS.

Page.....

Subject: BENNINGTON DAM - CONTOOCOOK RIVER
Computation: DESIGN DATA
Computed by..... Checked by..... Date.....

Working Stresses in Structural Materials

Concrete	f_c'	= 3000 lb. per sq. in.
Bending		= 1050 lb. per sq. in.
Shear		= 60 lb. per sq. in.
Bond		= 150 lb. per sq. in.
Steel Reinforcement	f_s	= 18000 lb. per sq. in.
	n	= 10
Structural Rolled Shapes	f	= 20000 lb. per sq. in.

Equipment Building Loads

Roof -	Live Load	= 50 lb. per sq. ft.
	Roofing	= 15 lb. per sq. ft.
Ground Floor -	Live Load	= 150 lb. per sq. ft.

Hydraulic Structures

All hydraulic structures are designed in accordance with working stresses recommended in Engineering Manual for Civil Works, Part 10, Chapter I

Plate VIII-5

CHAS. T. MAIN, INC., ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

Name: Bennington Dam
Subject: Design Data

Job No. US-16-15 Sheet No. 1-B
Date: Oct. 15 1945
Computed by: BJB
Checked by:

Reservoir Full	
Ultimate Development	Max. W.S. El. 723.1
Initial Development	" " El. 716.8
Maximum Tail Water	El. 678.0
Weight of Material	Pounds per cu. ft.
Weight of Concrete	150
Weight of Water	62.5
Weight of Earth (dry)	125
Weight of Earth (submerged)	80
Horizontal Pressure	Pounds per sq. ft. per ft. of depth.
Earth (dry)	31
Earth (saturated)	80

Pressures and Loads indicated are average pressures and loads per foot of sections and are proportioned to the length of monolith on wich they act.

Spillway Uplift.

Uplift pressures have been determined from the flow net study and are equal to the full effective head applied to 100% of the base area.

Spillway Sliding.

Foundation pressures and sliding factors do not include forces due to earthquake.

Earthquake forces have been applied acting downstream for full Reservoir condition and upstream for empty Reservoir condition.

The shearing strength of Till used in computing resistence to sliding were obtained from diagram "Shear Strength of Till."

CHAS. T. MAIN, INC. ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

Name: Bennington Dam
Subject: Shear Strength of Till

Plate VIII-6
Job No. US-16-15 Sheet No. 1-C
Date: Oct. 15 1945
Computed by: B.A.B.
Checked by:

Reference: Definite Project Report on Bennington Reservoir Plate II-11, Fig. 4 Line B A

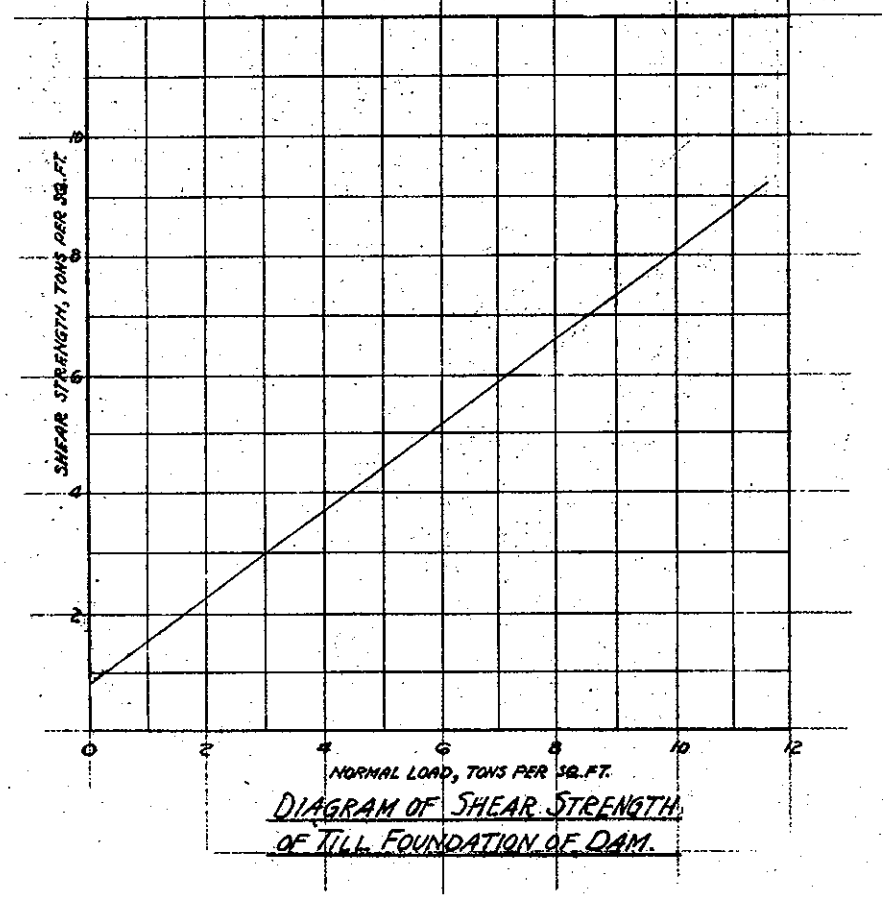


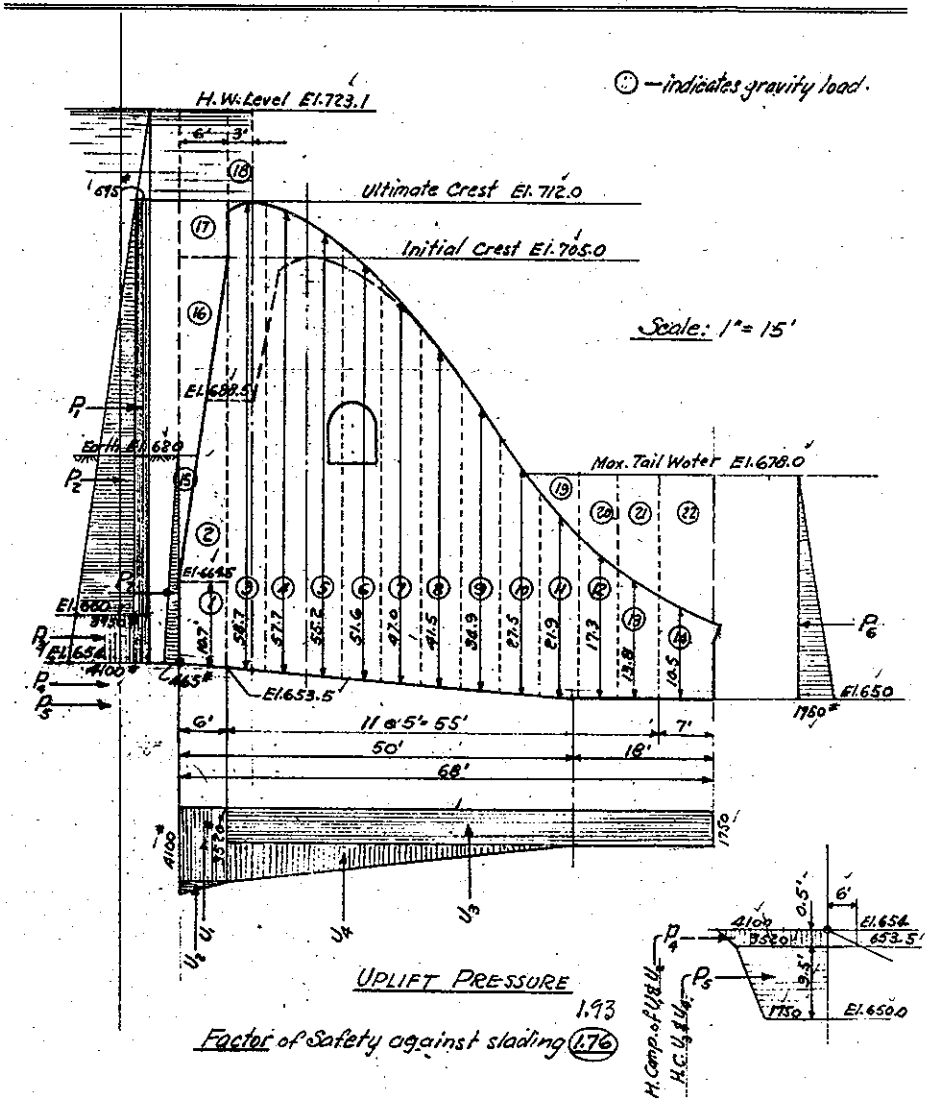
DIAGRAM OF SHEAR STRENGTH OF TILL FOUNDATION OF DAM.

CHAS. T. MAIN, INC. ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

Name: Bennington Dam
Subject: Ultimate Ungated Section

Plate VIII-7
Job No. US-16-15 Sheet No. 1
Date: Oct. 16 1945
Computed by: B.A.B.
Checked by: L.G.R.

Stability Analysis



⊙ — indicates gravity load.

Plate VIII-8

CHAS. T. MAIN, INC. ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

Name Bennington Dam
Subject Ultimate Ungated Section
Job No. US-16-13 Sheet No. 2
Date Oct. 16 1945
Computed by L. G. R.
Checked by B.A.B.

Stability Analysis

Description	Mark	Dimensions	Area	Wt. #/c.f.	Force K/ft.	Arm Ft.	Mom.
							7107
Concrete	①	10.7 × 6	64.2	150	9.6	3.0	29
	②	½ × 40.5 × 6	121.5	"	18.2	4.0	73
	③	58.7 × 5	293.5	"	44.0	8.5	374
	④	57.7 × 5	288.5	"	43.3	13.5	585
	⑤	55.2 × 5	276.0	"	41.4	18.5	767
	⑥	51.6 × 5	258.0	"	38.7	23.5	910
	⑦	47.0 × 5	235.0	"	35.2	28.5	1002
	⑧	41.5 × 5	207.5	"	31.1	33.5	1042
	⑨	34.9 × 5	174.5	"	26.2	38.5	1008
	⑩	27.5 × 5	137.5	"	20.6	43.5	896
	⑪	21.9 × 5	109.5	"	16.4	48.5	795
	⑫	17.3 × 5	86.5	"	13.0	53.5	695
	⑬	13.8 × 5	69.0	"	10.3	58.5	602
	⑭	10.5 × 7	73.5	"	11.0	64.5	710 657
					359.0	9488	9432
Voids		4.5 × 6	27.0	"	− 4.0	22.0	− 88
		π × 6²/8	14.1	"	− 2.1	22.0	− 46
					352.9	9354	9298
Earth	⑮	½ × 15.5 × 2.3	17.8	80	1.4	0.77	1
Water	⑯	½ × 40.5 × 6	121.5	62.5	7.6	2.0	15
	⑰	18.1 × 6	108.6	"	6.8	3.0	20
	⑱	11.1 × 3	33.3	"	2.1	7.5	16
	⑲	½ × 8.3 × 7.5	31.1	"	1.9	48.5	92
	⑳	10.7 × 5	53.5	"	3.3	53.5	177
	㉑	14.2 × 5	71.0	"	4.4	58.5	257
	㉒	17.5 × 7	122.5	"	7.7	64.5	496
					388.1	10928	10372
Uplift	U_1	3520 × 6			− 21.1	3.0	− 63
	U_2	½ × 580 × 6			− 1.7	2.0	− 3
	U_3	1750 × 62			− 108.5	37.0	− 4014
	U_4	½ × 1710 × 44			− 38.9	20.67	− 805
					ΣV = 217.9	55.42	5487

Plate VIII-9

CHAS. T. MAIN, INC. ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

Name Bennington Dam
Subject Ultimate Ungated Section
Job No. US-16-13 Sheet No. 3
Date Oct. 17 1945
Computed by L. G. R.
Checked by B.A.B.

Stability Analysis

Description	Mark	Dimensions	Area	Wt. #/c.f.	Force K/ft.	Arm Ft.	Mom
							5543
Total Vert. =					ΣV = 217.9		5487
Hor. Water	P_1	6.95 × 52			36.1	32.0	1155
	P_2	½ × 32.55 × 52			84.6	23.33	1975
	P_3	40.25 × 6			24.2	3.0	73
	P_4	3.810 × 0.5			1.9	0.25	—
	P_5	26.35 × 3.5			9.2	2.25	− 21
	P_6	½ × 17.50 × 28			− 24.5	5.33	− 130
Hor. Earth	P_7	½ × 4.65 × 26			6.0	8.67	52
					ΣH = 137.5		(8591)
							ΣM = 8647

$\Sigma V = 217.9 \qquad \Sigma H = 137.5$

(of Eq 654) $d = \frac{\Sigma M}{\Sigma V} = \frac{8647}{217.9} = 39.7'$

For Correction of "d" see Sh. 5
d = 41.8'
R falls 3.5' inside 3rd. pt.

$\frac{2}{3} \times 68 = \qquad = 45.3'$

e = 7.3' (see sh. 5)

Pressure on Base:

Reservoir Full.

$f = \frac{\Sigma V}{b}\left(1 \pm \frac{6 \times e}{b}\right) = \frac{217.9}{68}\left(1 \pm \frac{6 \times 7.8}{68}\right)$ } 5.40^k, 0.99^k

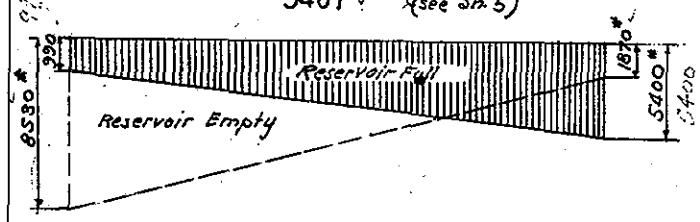
Reservoir Empty.

$\Sigma V = 354.3 \qquad \Sigma M = 9355'^k \qquad d = 26.7'$

$\Sigma H = 6.0 \qquad = 52 \qquad e = 7.3'$

$9407'^k$ (see Sh. 5)

$f = \frac{353.3}{68}\left(1 \pm \frac{6 \times 7.3}{68}\right)$ } 8.53^k, 1.87^k



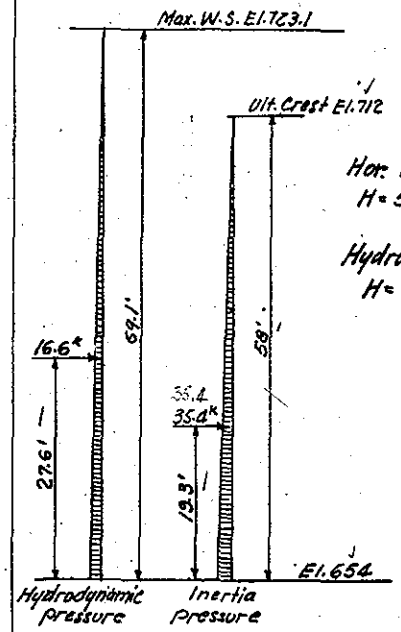
FOUNDATION PRESSURE.

CHAS. T. MAIN, INC. ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

Name: Bennington Dam
Subject: Ultimate Ungated Section

Plate VIII-10
Job No. US-16-18 Sheet No. 4
Date: Oct. 17 19 45
Computed by: B.A.B.
Checked by: L.G.R.

Stability Analysis.



Earthquake Stresses at Base.

(No vertical acceleration considered)

Gravity Wt. = 353.3 k.

Hor. Inertia Force = $\frac{1}{10}$ gravity = 0.1 × 353.3 = 35.4 k

H = 58' Arm = $\frac{1}{3}$H = $\frac{1}{3}$ × 58 = 19.3'

Hydrodynamic Force = 0.0347 (H)² = 16.6 k

H = 69' Arm = 0.4 H = 0.4 × 69 = 27.6'

Reservoir Full:

ΣV = 217.9 k		ΣM = 5643 ft k	
ΣH	= 137.5		3104
Hydrodynamic	= 16.6 @ 27.6		458
Inertia	= 35.4 @ 19.3		682
ΣH =	189.5		9787 ft k

(at Elev. 654) d = 44.8' 2/3 b = 45.3' ∴ Result. 0.5' outside 3rd pt. (see correction)

Reservoir Empty.

ΣV = 353.3 k		ΣM = 9355 ft k
→ ΣH	6.0	52
← Inertia	−35.3	−682
ΣH =	29.3	8725

d = 24.7' b/3 = 22.7 ∴ Result. 2' outside 3rd pt.

CHAS. T. MAIN, INC. ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

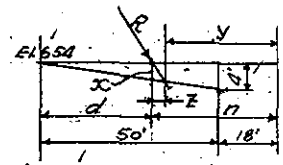
Name: Bennington Dam
Subject: Ultimate Ungated Section

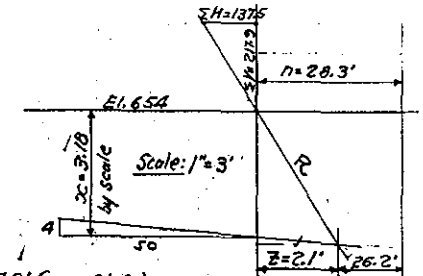
Plate VIII-11
Job No. US-16-18 Sheet No. 5
Date: Oct. 18 19 45
Computed by: B.A.B.
Checked by: L.G.R.

Stability Analysis.

Corrections for Location of Resultant due to Base Slope

Case I Reservoir Full (no earthquake considered)





d = 39.7' (see Sh. 3)
n = 28.3'
x = $\frac{4}{50}$ × 39.7 = 3.18'
z = 2.1'
y = 26.2' ∴ e = $\frac{68}{2}$ − 26.2 = 7.8' (see Sh. 3)

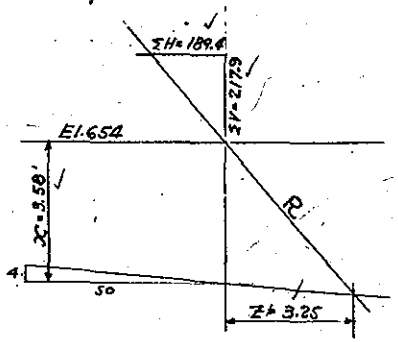
∴ y = 28.3 − 2.1 = 26.2'

Case II Reservoir Empty (no earthquake)

d = 26.7' Resultant considered as vertical. ∴ z = 0
n = y = 68 − 26.7 = 41.3' y = n
∴ e = $\frac{68}{2}$ − 26.7 = 7.3' (see Sh. 3)

Case III Reservoir Full – earthquake considered.

d = 44.8' (see Sh. 4)
n = 23.2'
x = $\frac{4}{50}$ × 44.8 = 3.58'
z = 3.25'
y = 23.2 − 3.25 = 19.95'



Case IV Reservoir Empty – earthquake.

d = 24.7 (see Sh. 4) Resultant considered as vertical ∴ z = 0
n = y = 43.3

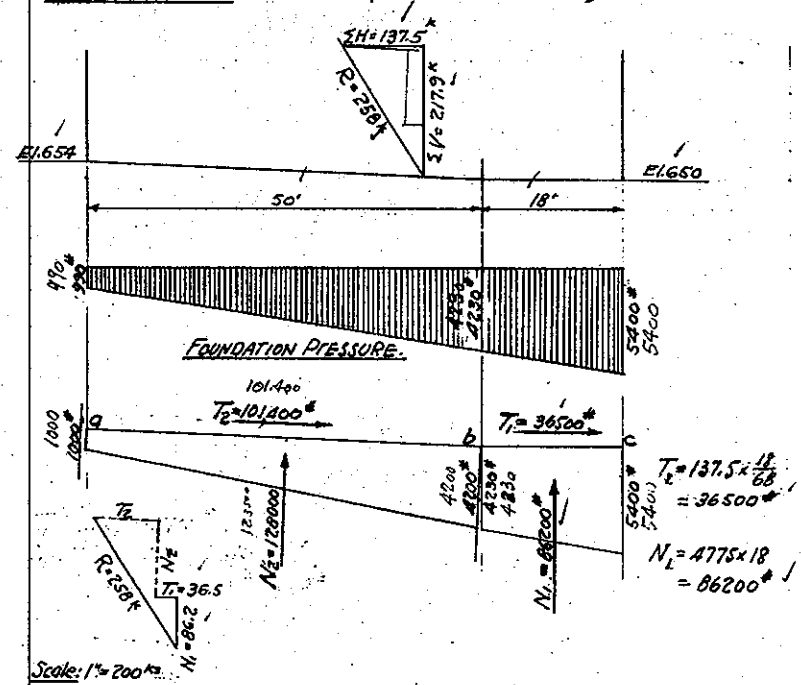
CHAS. T. MAIN, INC. ENG'RS.
201 DEVONSHIRE ST.
BOSTON, MASS.

Name: Bennington Dam
Subject: Ultimate Ungated Section
Stability Analysis.

Plate VIII-12
Job No. US-16-15 Sheet No. 6
Date: Oct. 18 1945
Computed by: B.A.B.
Checked by: L.G.R.

Factor of Safety against sliding.

Reservoir Full: (no earthquake considered)



Normal Force.

(ab) N_2 = 2600#/□' = 1.3 T/□' ∴ 1.3 T/□' × 50.2' = 88.4 tons

(bc) N_1 = 4790 = 2.4 ∴ 2.5 × 18' = 45

Shear Capacity of Soil.

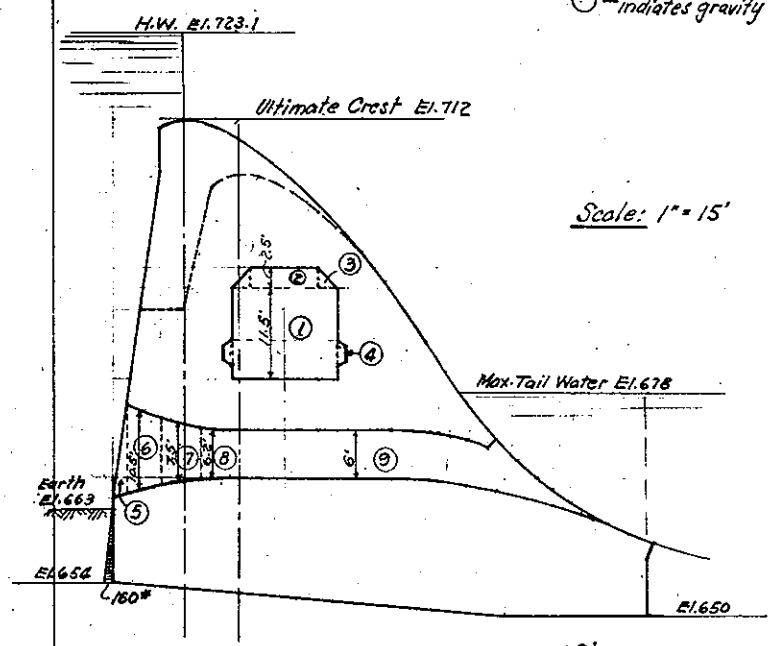
Total shearing Force: $T_1 + T_2$ = 137,900# = 69 tons. Total = 133.4 tons.

Factor of Safety: 133.4 / 69 = 1.93

CHAS. T. MAIN, INC. ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

Name: Bennington Dam
Subject: Ultimate Gated Section
Stability Analysis

Plate VIII-13
Job No. US-16-15 Sheet No. 7
Date: Oct. 19 1945
Computed by: B.A.B.
Checked by: L.G.R.



⊙ — indicates gravity loads

Scale: 1" = 15'

Factor of Safety against sliding 1.83 (1.81)

Note: General Dimensions, Elevations and Water & Uplift Pressure Diagrams same as for Ultimate Ungated Section, see Sh. 1

CHAS. T. MAIN, INC., ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

Name: Bennington Dam
Subject: Ultimate Gated Section

Plate VIII-1A
Job No. U.S.-16-15 Sheet No. 8
Date: Oct. 19 1945
Computed by: L.G.R.
Checked by: B.A.B.

Stability Analysis.

Description	Mark	Dimensions	Area	Wt. #/c.f.	Force K / Ft.	Arm Ft.	Mom
Concrete							9488
Gross	see Sh.		(see Sh. 2)		359.0		9432
Voids		4.5×6.0×20/30	18.0	150	− 2.7	22.9	− 60
		π×6²/8, ×20/30	9.4	"	− 1.4	22.0	− 31
	①	11.5×13.5×10/30	51.7	"	− 7.8	22.0	− 172
2549' →	②	2.5×11.0×10/30	9.2	"	− 1.4	22.0	− 31
	③	2×[1×2.5]×10/30	1.7	"	− 0.3	22.0	− 7
	④	2×[6.5×1]×5/30	2.2	"	− 0.3	22.0	− 7
	⑤	½×14×2×7.5/30	3.5	"	− 0.5	1.3	− 1
	⑥	10.5×4×6/30	8.4	"	− 1.3	4.0	− 5
	⑦	7.5×5×4.5/30	5.6	"	− 0.8	8.5	− 7
	⑧	6.2×5×4/30	4.1	"	− 0.6	13.5	− 8
	⑨	6×34.5×4/30	27.6	"	− 4.2	33.25	− 140
					337.7	9013	8963
Water			(See Sh. 2 ⑯ to ㉓)		33.8		1073
	⑤	½×14×2×7.5/30	3.5	62.5	0.2	1.3	−
	⑥	10.5×4×6/30	8.4	"	0.5	4.0	2
	⑦	7.5×5×4.5/30	5.6	"	0.3	8.5	3
	⑧	6.2×5×4/30	4.1	"	0.3	13.5	4
	⑨	6×34.5×4/30	27.6	"	1.7	33.25	56
					374.5	10152	10101
Uplift			(See Sh. 2)		− 170.2		− 4885
Total Vert.				ΣV =	204.3	5272	5216
Hor. Water			(See Sh. 3)		131.5		3052
Hor. Earth		½×160×9²			0.7	3.0	2
				ΣH =	132.2		3270
							8326

ΣV = 204.3 ΣH = 132.2 ΣM = 8326
(at EL. 654) d = ΣM/ΣV = 40.7 ⅔×68 = 45.4
R falls 2.5' inside 3rd pt.

For Correction of "d" see Sh. 10
d = 42.9'

CHAS. T. MAIN, INC. ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

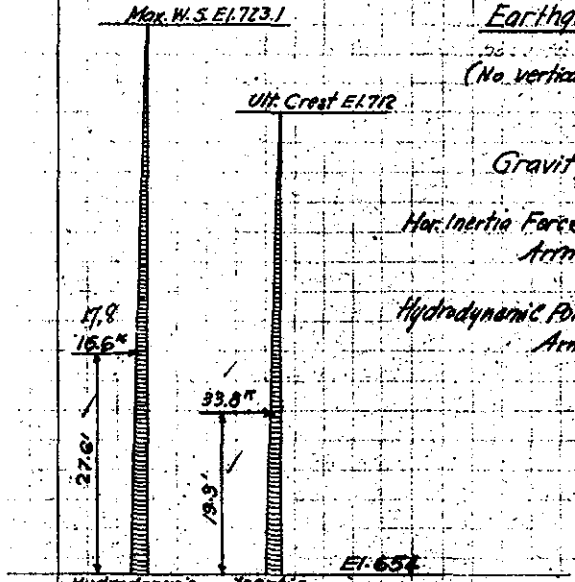
Name: Bennington Dam
Subject: Ultimate Gated Section

Plate VIII-1B
Job No. U.S.-16-15 Sheet No. 9
Date: Oct. 20 1945
Computed by: B.A.B.
Checked by: L.G.R.

Stability Analysis.

Earthquake Stresses at Base

(No vertical acceleration considered.)



Gravity Wt. = 337.7ᵏ

Hor. Inertia Force: 0.1×337.7 = 33.8ᵏ
Arm: ⅔×58 = 19.8'

Hydrodynamic Force: 0.10×337.7(17.8/19.9) = 16.6ᵏ
Arm: 0.6 H = 27.6'

Reservoir Full.

ΣV = 204.3ᵏ ΣM = 5272'ᵏ

Hor. Water & Earth	= 132.2		3054
Hydrodynamic	= 16.6 (17.8)	@ 27.6	458 492
Inertia	= 33.8	@ 19.8	652
	ΣH = 182.6ᵏ (183.8)		9436'ᵏ 9470

ΣM = 9436 (9470) ΣV = 204.3 d = 46.1 ⅔b = ⅔×68 = 45.4
(at El. 654) R is 0.7 outside 3rd pt.
See Sh. 10 for correction

Reservoir Empty.

ΣV = 337.7ᵏ ΣM = 9019'ᵏ
→ Hor. Earth = 0.7 2
← Inertia = −33.8 −652
ΣH = −33.1ᵏ 8369'ᵏ
ΣM = 8369 ΣV = 337.7 d = 24.9' ⅔×b = 22.7 − R is 2' inside 3rd pt.
(no correction is necessary)

CHAS. T. MAIN, INC. ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

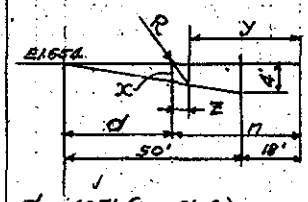
Name: Bennington Dam
Subject: Ultimate Gated Section

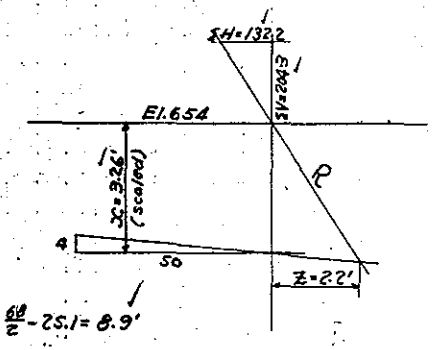
Plate VIII - 16
Job No. US-16-15 Sheet No. 10
Date: Oct. 22 1945
Computed by: B.A.B.
Checked by: J.G.R.

Stability Analysis.

Corrections for Location of Resultants due to Base Slope.

Case I Reservoir Full (no earthquake considered)



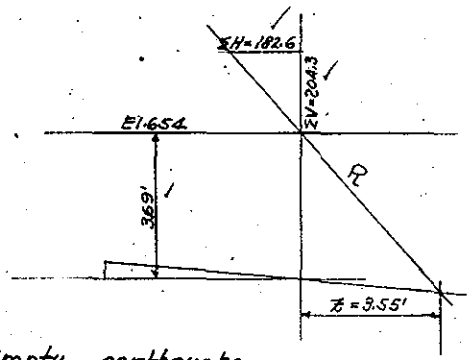


$d = 40.7'$ (see Sh. 8)
$n = 27.3'$
$x = \frac{4}{50} \times 40.7 = 3.26'$
$z = 2.2'$
$y = n - z = 27.3 - 2.2 = 25.1'$ $\therefore e = \frac{68}{2} - 25.1 = 8.9'$

Case II Reservoir Empty (no earthquake)
$\Sigma V = 337.7$ $\Sigma H = 0.7'$ $\Sigma M = 9021$ $d = 26.7$
$d = 26.7$ Resultant considered as vertical $\therefore z = 0$
$n = y = 68 - 26.7 = 41.3'$ $y = n$
$\therefore e = \frac{68}{2} - 26.7 = 7.3'$

Case III Reservoir Full - earthquake considered.

$d = 46.1'$ (see Sh. 9)
$n = 21.9'$
$x = \frac{4}{50} \times 46.1 = 3.69'$
$z = 3.55'$
$y = n - z = 21.9 - 3.55 = 18.35'$



Case IV Reservoir Empty - earthquake
$d = 24.7$ (see Sh. 9) Resultant considered as vertical $\therefore z = 0$
$n = y = 43.3'$

CHAS. T. MAIN, INC. ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

Name: Bennington Dam
Subject: Ultimate Gated Section.

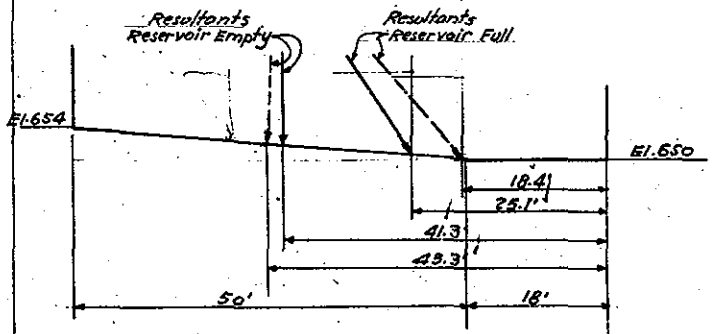
Plate VIII - 17
Job No. US-16-15 Sheet No. 11
Date: Oct. 22 1945
Computed by: B.A.B.
Checked by: J.G.R.

Stability Analysis.

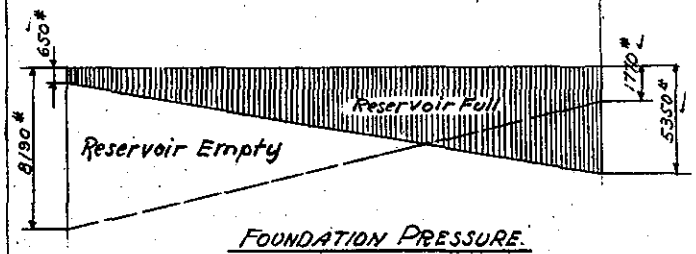
Pressure on Base.

Reservoir Full.
$e = 8.9'$ (see Sheet 4-B) $f = \frac{\Sigma V}{b}\left(1 \pm \frac{6 \times e}{b}\right) = \frac{204.3}{68'}\left(1 \pm \frac{6 \times 8.9}{68}\right)$ $\left.\begin{matrix} 5.35^k \\ 0.65 \end{matrix}\right\}$
$\Sigma V = 204.3^k$
$b = 68'$

Reservoir Empty.
$e = 7.3$ (see Sheet 4-B). $f = \frac{337.7}{68'}\left(1 \pm \frac{6 \times 7.3}{68'}\right)$ $\left.\begin{matrix} 8.19^k \\ 1.77 \end{matrix}\right\}$
$\Sigma V = 337.7^k$



LOCATION OF RESULTANTS.



FOUNDATION PRESSURE.

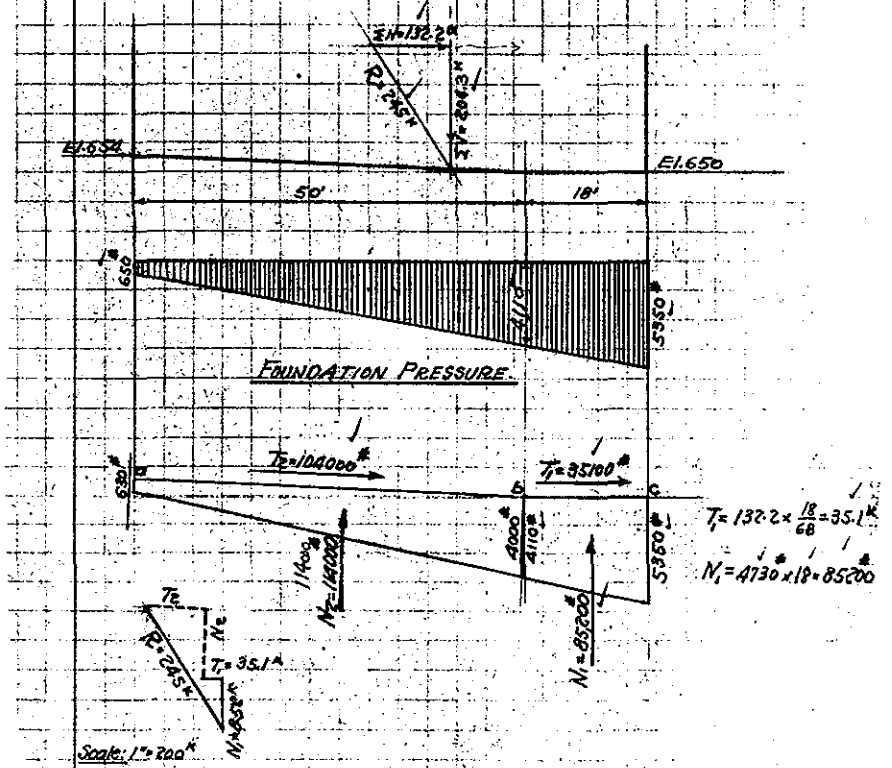
CHAS. T. MAIN, INC. ENG'RS.
201 DEVONSHIRE ST.
BOSTON, MASS.

Name Bennington Dam
Subject Ultimate Gated Section
Stability Analysis

Plate XIII - 1B
Job No. US-16-15 Sheet No. 12
Date Oct. 22 1945
Computed by B.A.B.
Checked by Y.G.R.

Factor of Safety against Sliding

Reservoir Full (no earthquake considered)



Normal Forces

(cb) $N_1 = 4730^{\#}/18' = 2.36^{T/\square'}$

(ba) $N_2 = 2280 = 1.14$ (2280, 1.14)

Shear Capacity of Soil.

$2.5^{T/\square'} \times 18 = 45$ tns.

$1.60 \times 50.2 = 80.5$ (80.5)

Total Shearing Force: $T_1 + T_2 = 139.1^{K} = 69.6$ tns. (125.5)

Total = 125.3 tns. (125.5)

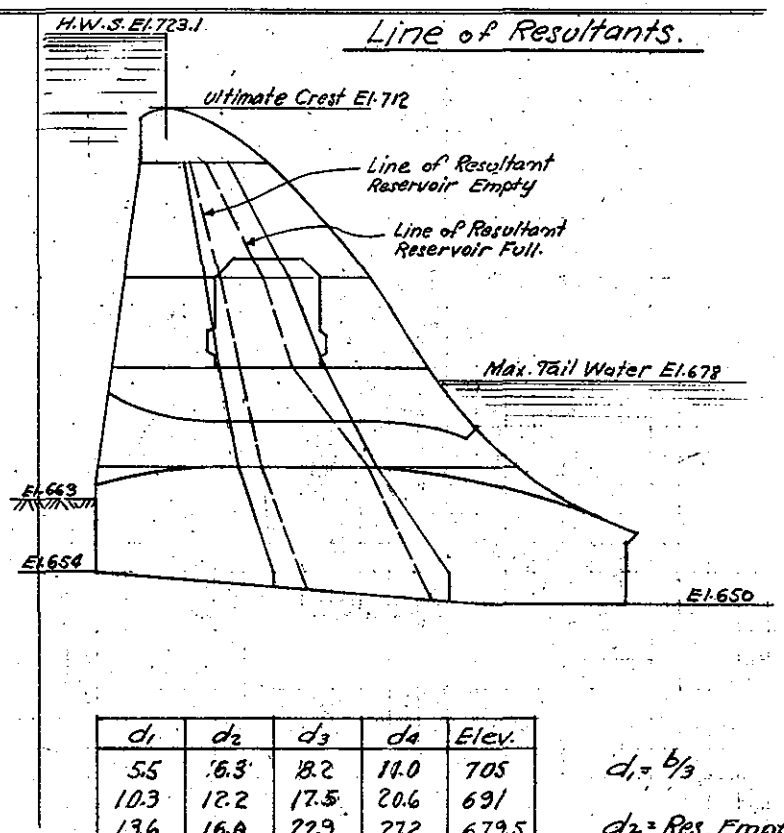
Factor of Safety: $\frac{125.5}{69.6} = 1.81$

CHAS. T. MAIN, INC., ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

Name Bennington Dam
Subject Ultimate Gated Section
Stability Analysis.

Plate XIII - 1B
Job No. US-16-15 Sheet No. 12-A
Date Oct. 23 1945
Computed by B.A.B.
Checked by Y.G.R.

Line of Resultants.



d_1	d_2	d_3	d_4	Elev.
5.5	6.3	8.2	11.0	705
10.3	12.2	17.5	20.6	691
13.6	16.0	22.9	27.2	679.5
18.0	20.8	34.8	36.0	667
22.7	26.7	42.9	45.3	Base

$d_1 = b/3$

d_2 = Res. Empty

d_3 = Res. Full

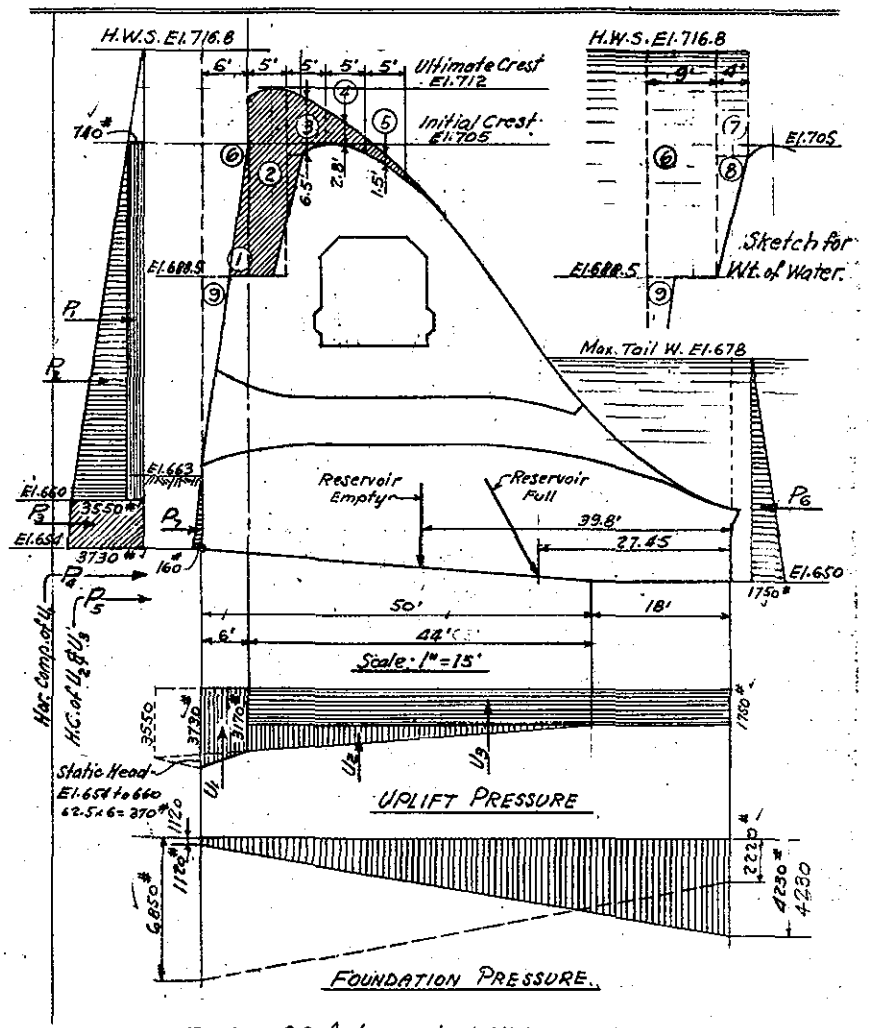
$d_4 = 2/3\, b$

CHAS. T. MAIN, INC. ENG'RS.
201 DEVONSHIRE ST.
BOSTON, MASS.

Name: Bennington Dam
Subject: Initial Gated Section

Plate VIII-20
Job No. US-16-18 Sheet No. 13
Date: Oct. 23 19 45
Computed by: B.A.R.
Checked by: L.G.R.

Stability Analysis.



Factor of Safety against Sliding 2.2

CHAS. T. MAIN, INC. ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

Name: Bennington Dam
Subject: Initial Gated Section

Plate VIII-20
Job No. US-16-18 Sheet No. 14
Date: Oct. 23 19 45
Computed by: B.A.R.
Checked by: L.G.R.

Stability Analysis

Description	Mark	Dimensions	Area	Wt. #/c.f.	Force K/Ft.	Arm Ft.	Mom.
Concrete		(see Sh. 8)			337.7		9019
	①	½ × 16.5 × 2.44	20.0	150	− 3.0	5.19	− 17
	②	23.5 × 5	117.5	"	− 17.6	8.5	− 150
	③	6.5 × 5	32.5	"	− 4.9	13.5	− 66
	④	2.8 × 5	14.0	"	− 2.1	18.5	− 39
	⑤	1.5 × 5	7.5	"	− 1.1	23.5	− 26
					309.0		8721
Water		(see Sh.			20.0		1082
	⑥	28.3 × 9	254.4	62.5	15.9	4.5	72
	⑦	13.5 × 4	54.0	"	3.4	11.0	37
	⑧	½ × 14.8 × 4	29.6	"	1.9	10.33	20
	⑨	½ × 24 × 3.56	42.7	"	2.7	1.2	3
					352.9		8940
Uplift	U_1	3450 × 6			− 20.7	3.0	− 62
	U_2	½ × 1420 × 44			− 31.2	20.67	− 645
	U_3	1750 × 62			− 108.5	37.0	− 4014
Total Vert.					ΣV = 192.5		5219
					192.5		
Hor. Water	P_1	740 × 45			33.4	28.5	951
	P_2	½ × 2810 × 45			63.3	21.0	1330
	P_3	3640 × 6			22.0	3.0	66
	P_4	3450 × 0.5			1.7	0.25	—
	P_5	2469 × 3.5			8.6	2.25	19
	P_6	½ × 1750 × 28			− 24.5	5.33	− 130
Hor. Earth	P_7	½ × 160 × 9			0.7	3.0	2
					ΣH = 105.2		ΣM = 7457
					105.2		

$\Sigma V = 192.5^k$ (192.5) $\quad \Sigma H = 105.2^k$ (105.2) $\quad \Sigma M = 7457^{'k}$

(at El. 654) $d = \frac{7457}{192.5} = 38.7'$ (38.7)

$\frac{2}{3} \times 68 = 45.4'$

For correction of "d" see Sh. 15
d = 43.4
R falls 2' inside 3rd. pt.

CHAS. T. MAIN, INC., ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

Name Bennington Dam
Subject Initial Gated Section

Plate XIII-22
Job No. US-16-15 Sheet No. 15
Date Oct. 24 19 45
Computed by RAB
Checked by J. G. R.

Stability Analysis.

Correction for Location of Resultants due to Slope.

Reservoir Full.

$d = 38.7'$ (see Sh.) 38.7

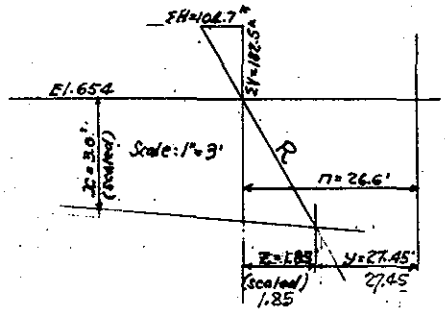
$n = 68 - 38.7 = 29.3'$ 29.3

$x = \frac{4}{50} \times 38.7 = 3.0'$ 3.1

$z = 1.85$ 1.85

$y = 29.3 - 1.85 = 27.45$
29.3 - 1.85 27.45

$\therefore e = \frac{68}{2} - 27.45 = 6.55'$
27.45 6.55



Reservoir Empty.

$\Sigma V = 309^k$; $\Sigma M = 8723'^k$ Resultant considered as vertical $\therefore z = 0$

$\Sigma H = 0.7^k$ (at El. 654) $d = \frac{8723}{309} = 28.2'$ $\therefore e = \frac{68}{2} - 28.2 = 5.8'$

$y = 68 - 28.2 = 39.8'$.

Pressure on Base.

Reservoir Full $e = 6.55'$ 6.55 $f = \frac{182.5}{68}\left(1 \pm \frac{6 \times 6.55}{68}\right)$ $\left.\begin{matrix} 4.23^k \\ 1.12^k \end{matrix}\right\}$ 4.23 1.12

$\Sigma V = 182.5^k$

Reservoir Empty $e = 5.8'$ $f = \frac{309}{68}\left(1 \pm \frac{6 \times 5.8}{68}\right)$ $\left.\begin{matrix} 6.85^k \\ 2.22 \end{matrix}\right\}$

$\Sigma V = 309^k$

Factor of Safety against Sliding.

Reservoir Full.

ave. Load: $\frac{182.5}{2 \times 68} = 1.34^{T/\square'}$

Shearing Capacity of Soil: $1.7^{T/\square'}$

ave. Shearing Force: $\frac{104.7}{2 \times 68} = 0.77^{T/\square'}$

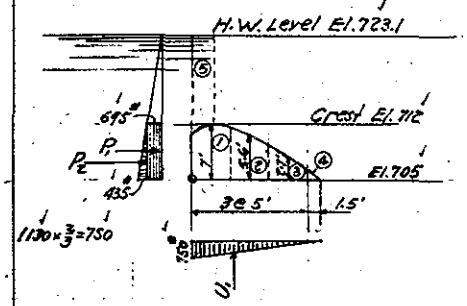
Factor: $\frac{1.7}{0.77} = 2.2$

CHAS. T. MAIN, INC. ENG'RS.
201 DEVONSHIRE ST.
BOSTON, MASS.

Name Bennington Dam
Subject Ultimate Gated Section
Elev. 705

Plate XIII-23
Job No. US-16-15 Sheet No. 16
Date Oct. 24 19 45
Computed by RAB
Checked by J. G. R.

Stability Analysis



Description	Mark	Dimensions	Area	Wt. #/c.f.	Force K/Ft.	Arm Ft.	Mom.
Concrete	①	7×5	35.0	150	5.3	2.5	13
	②	5.6×5	28.0	"	4.2	7.5	32
	③	2.7×5	13.5	"	2.2	12.5	28
	④	½×1.5×1	0.8	"	0.1	15.1	1
					11.8		74
Water	⑤	12×3	36	62.5	2.3	1.5	4
					14.1		78
Uplift	U₁	½×750×16.5			− 6.2	5.5	− 34
Total Vert.				ΣV =	7.9		44
Hor. Water	P₁	695×7			4.9	3.5	17
	P₂	½×435×7			1.5	2.33	4
				ΣH =	6.4		ΣM = 65

Reservoir Full.

$\Sigma V = 7.9^k$ $\Sigma M = 65'^k$

$d = \frac{\Sigma M}{\Sigma V} = \frac{65}{7.9} = 8.2'$

$\frac{1}{3} \times 16.5 = 5.5'$ Resultant falls 2.7' inside 3rd pt.

Reservoir Empty.

$\Sigma V = 11.8^k$; $\Sigma M = 74'^k$

$d = 6.28'$

$\frac{1}{3} \times b = 5.5'$ Resultant falls 0.78' inside 3rd. pt.

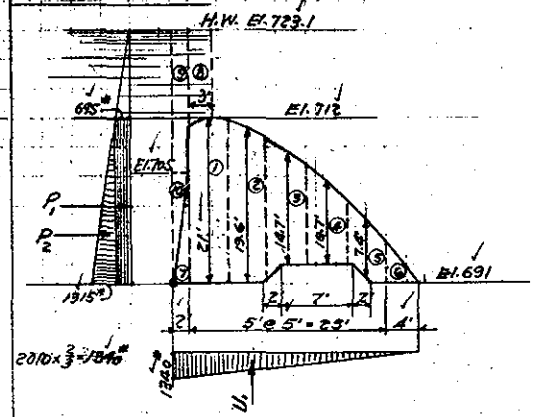
CHAS. T. MAIN, INC. ENG'RS.
201 DEVONSHIRE ST.
BOSTON, MASS.

Name Bennington Dam
Subject Ultimate gated Section
Elev. 691

Plate VIII-24
Job No. U.S-16-15 Sheet No. 17
Date Oct. 25 1945
Computed by B.A.B.
Checked by J.G.R.

Stability Analysis

Reservoir Full



Reservoir Empty.

$\Sigma V = 59.4^k$ $\Sigma M = 722^{'k}$
$d = 12.2'$ $1/3\,b = 10.3'$
Resultant falls 1.9' inside 3rd. pt.

Description	Mark	Dimensions	Area	Wt. #/c.f.	Force K/Ft.	Arm Ft.	Mom.
Concrete	①	21×5	105.0	150	15.8	4.5	71
	②	19.6×5	98.0	"	14.7	9.5	140
	③	14.7×5	73.5	"	11.0	14.5	160
	④	10.7×5	53.5	"	8.0	19.5	157
	⑤	7.4×5	37.0	"	5.6	24.5	137
	⑥	½×6×4	12.0	"	1.8	28.33 28.33	51
	⑦	½×14×2	14.0	"	2.1	1.33 1.33	3
Water	⑧	11.1×3	33.3	62.5	2.1	3.5 3.5	7
	⑨	18.1×2	36.2	"	2.3	1.0	2
	⑩	½×14×2	14.0	"	0.9	0.67 0.67	1
					64.7		729
Uplift		½×1340×31			20.8	10.33	215
Total Vert.					$\Sigma V = 43.9$		514
Hor. Water	P_1	695×21			14.6	10.5	153
	P_2	½×1315×21			13.8	7.0	97
					$\Sigma H = 28.4$		$\Sigma M = 764$

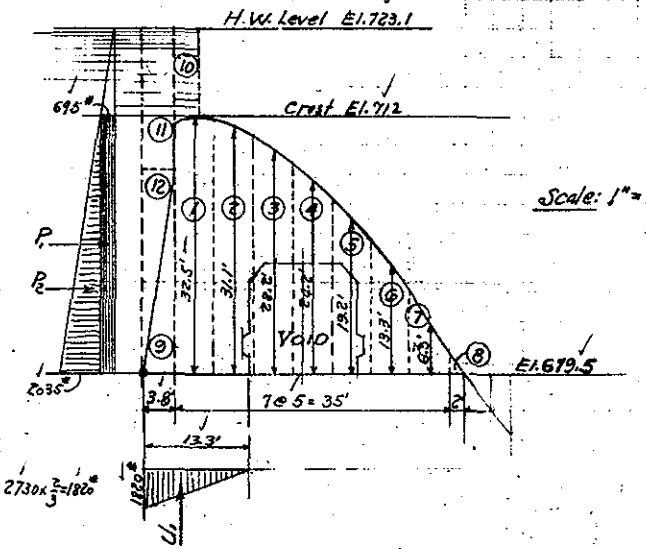
$\Sigma V = 43.9^k$ $\Sigma H = 28.4^k$ $\Sigma M = 769^{'k}$
$d = \frac{\Sigma M}{\Sigma V} = \frac{764}{43.9} = 17.5'$ $1/3 \times 31 = 10.3'$ Resultant Falls 7.2' inside third point

CHAS. T. MAIN, INC. ENG'RS.
201 DEVONSHIRE ST.
BOSTON, MASS.

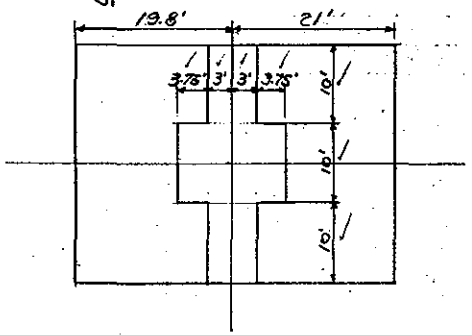
Name Bennington Dam
Subject Ultimate gated Section
Elev. 679.5

Plate VIII-25
Job No. U.S-16-15 Sheet No. 18
Date Oct. 25 1945
Computed by B.A.B.
Checked by J.G.R.

Stability Analysis.



Scale: 1" = 15'



Kern point inside middle third.

CHAS. T. MAIN, INC. ENG'RS.
201 DEVONSHIRE ST.
BOSTON, MASS.

Name: Bennington Dam
Subject: Ultimate gated Section
Elev. 679.5

Plate VIII-26
Job No. US-16-15 Sheet No. 19
Date Oct. 26 1945
Computed by RAB.
Checked by L.G.R.

Stability Analysis.

Discription	Mark	Dimensions	Area	Wt. #/c.f.	Force K/Ft.	Arm Ft.	Mom.
Concrete	1	32.5×5	162.5	150	24.4	6.3	153
	2	31.1×5	155.5	"	23.4	11.3	264
	3	28.2×5	141.0	"	21.2	16.3	345
	4	24.2×5	121.0	"	18.2	21.3	388
	5	19.2×5	96.0	"	14.4	26.3	380
	6	13.3×5	66.5	"	10.0	31.3	310
	7	6.3×5	31.5	"	4.7	36.3	171
	8	½×2×2	2.0	"	0.3	39.5	12
	9	½×25.5×3.8	48.5	"	7.3	2.54	18
					123.9		2041
Voids					− 2.7	19.78	
					− 1.4	"	
					− 7.8 (13.9)	"	
					− 1.4	"	
					− 0.6	"	− 272
					110.0		1769
Water	10	11.1×3	33.3	62.5	2.1	5.3	11
	11	18.1×3.8	68.8	"	4.3	1.9	8
	12	½×25.5×3.8	48.5	"	3.0	1.3	4
					119.4		1792
Uplift	U_1	½×1820×13.3			− 12.1	4.4	− 53
Total Vertical					ΣV = 107.3		1739
							1739
Hor. Water	P_1	695×32.5			22.6	16.25	368
	P_2	½×2035×32.5			33.0	10.8	356
					ΣH = 55.6		2463
							2423

Reservoir Full

ΣV = 107.3ᵏ ΣM = 2463

d = 22.9' ⅔×40.8 = 27.2' Resultant falls 4.3' inside 3rd. pt.

Reservoir Empty

ΣV = 110.0ᵏ ΣM = 1769'ᵏ

d = 16.0' ⅓×b = 13.6' Resultant falls 2.4' inside 3rd. pt.

CHAS. T. MAIN, INC. ENG'RS.
201 DEVONSHIRE ST.
BOSTON, MASS.

Name: Bennington Dam
Subject: Ultimate gated Section
Elev. 667

Plate VIII-28
Job No. US-16-15 Sheet No. 20
Date Oct. 26 1945
Computed by RAB.
Checked by L.G.R.

Stability Analysis.

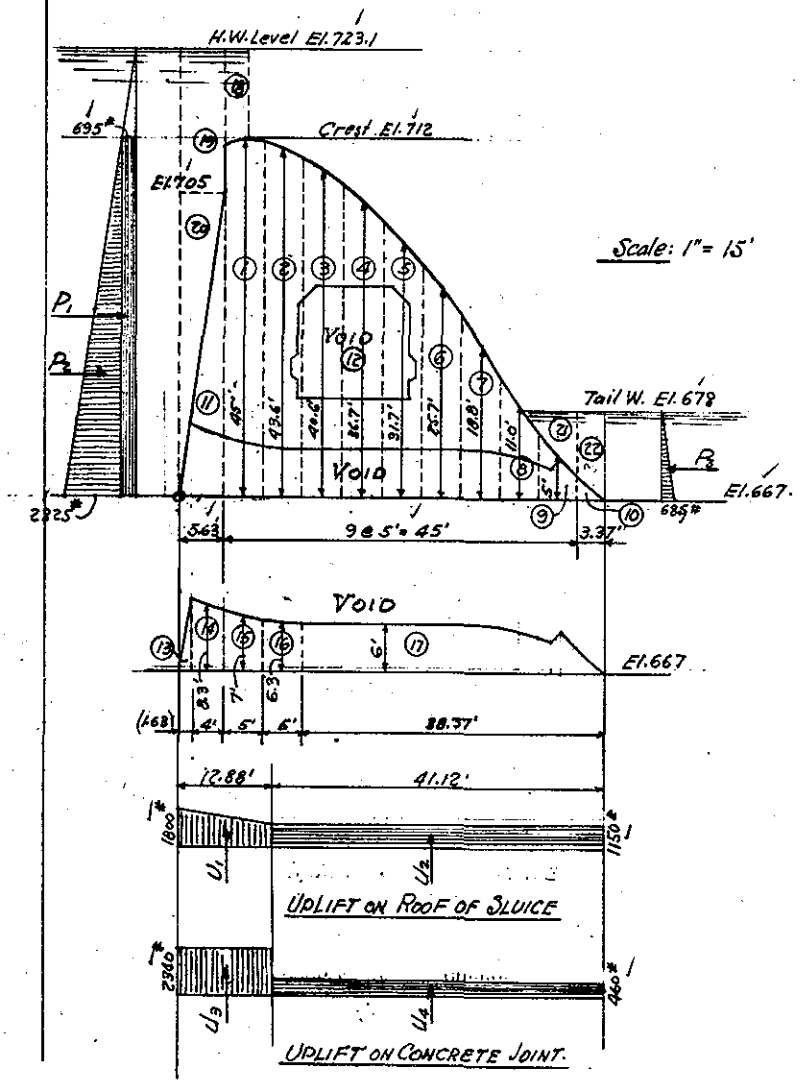


Plate VIII-28

CHAS. T. MAIN, INC. ENG'RS.
201 DEVONSHIRE ST.
BOSTON, MASS.

Name Bennington Dam
Subject Ultimate gated Section
Job No. US-16-15 Sheet No. 21
Date Oct. 27 1945
Computed by BAB.
Checked by J.G.R.

Elev. 667
Stability Analysis.

Description	Mark	Dimensions	Area	Wt. #/c.f.	Force K/Ft.	Arm Ft.	Mom.
Concrete	①	45×5	225.0	150	33.8	8.13	274
	②	43.6×5	216.0	"	32.4	13.13	426
	③	40.6×5	203.0	"	30.4	18.13	552
	④	36.7×5	183.5	"	27.5	23.13	635
	⑤	31.7×5	158.5	"	23.8	28.13	679
	⑥	25.7×5	128.5	"	19.3	33.13	638
	⑦	18.8×5	94.0	"	14.1	38.13	537
	⑧	11.0×5	55.0	"	8.2	43.13	354
	⑨	5×5	25.0	"	3.8	48.13	182
	⑩	½×3×3.37	5.1	"	0.7	51.75	36
	⑪	½×5.63×38	107.1	"	16.1	3.76	60
					210.1		4373
Voids	⑫	(see Sh. 13)	1	150	− 13.9	21.8	− 303
	⑬	½×8.5×1.63×7.9/30	1.7	"	− 0.3	11 (1.1)	− 3
	⑭	8.3×4×4/30	6.7	"	− 1.0	3.63	− 4
	⑮	7×5×4.9/30	5.2	"	− 0.8	7.13	− 5
	⑯	6.3×5×4/30	4.2	"	− 0.6	12.13	− 7
	⑰	6×36×4/30	28.8	"	− 4.3	33.63	− 145
					189.2		3906
					189.5		3891
Water	⑱	11.1×3	33.3	62.5	2.1	7.13	15
	⑲	18.1×5.63	102.0	"	6.4	2.81	18
	⑳	½×5.63×38	107.1	"	6.7	1.88	13
	㉑	½×7.6×8	30.4	"	1.9	48.20	92
	㉒	3.37×11	37.1	"	2.3	52.31	121
		1475		208.9	208.6	41.50	4165
Uplift	U_1	1475×12.88			− 19.0	6.44	− 123
	U_2	1150×41.22			− 47.2	33.44	− 1580
	U_3	2340×12.88			− 30.2	6.44	− 194
	U_4	1460×41.12			− 18.9	33.44	− 634
					93.3		1638
					93.5		1619

Plate VIII-29

CHAS. T. MAIN, INC. ENG'RS.
201 DEVONSHIRE ST.
BOSTON, MASS.

Name Bennington Dam
Subject Ultimate Gated Section
Job No. US-16-15 Sheet No. 22
Date Oct. 27 1945
Computed by BAB
Checked by J.G.R.

Elev. 667
Stability Analysis

Description	Mark	Dimensions	Area	Wt. #/c.f.	Force K/Ft.	Arm Fts.	Mom.
Total Vert. =					92.6		1637
				$\Sigma V =$	93.3		1634
Hor. Water	P_1	695×45			31.3	22.50	705
	P_2	½×2825×45		63.5	63.5	15.00	950
	P_3	½×685×11			− 3.8	3.67	− 14
				$\Sigma H =$	91.0	$\Sigma M =$	3275
					91.0		3260

Reservoir Full.

$\Sigma V = 93.3^k$ (93.6) $\Sigma M = 3275^{1k}$ (3260)

$d = \frac{\Sigma M}{\Sigma V} = \frac{3275}{93.3} = (34.8')$ 35.1'

2/3 × 54 = 36' Resultant falls (1.2) inside 3rd. pt. (10.3)

Reservoir Empty.

$\Sigma V = 189.2^k$ (189.5) $\Sigma M = 3906^{1k}$ (3891)

$d = \frac{\Sigma M}{\Sigma V} = (20.8')$ 20.6'

1/3 × 54 = 18' Resultant falls (2.8) inside 3rd. pt. 2.6

Plate VIII-30

CHAS. T. MAIN, INC. ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

Name: Bennington Dam
Subject: Non-overflow Section
Job No. U.S-16-18 Sheet No. 23
Date: Oct. 29 1945
Computed by: B.A.B.
Checked by: L.G.R.

Stability Analysis

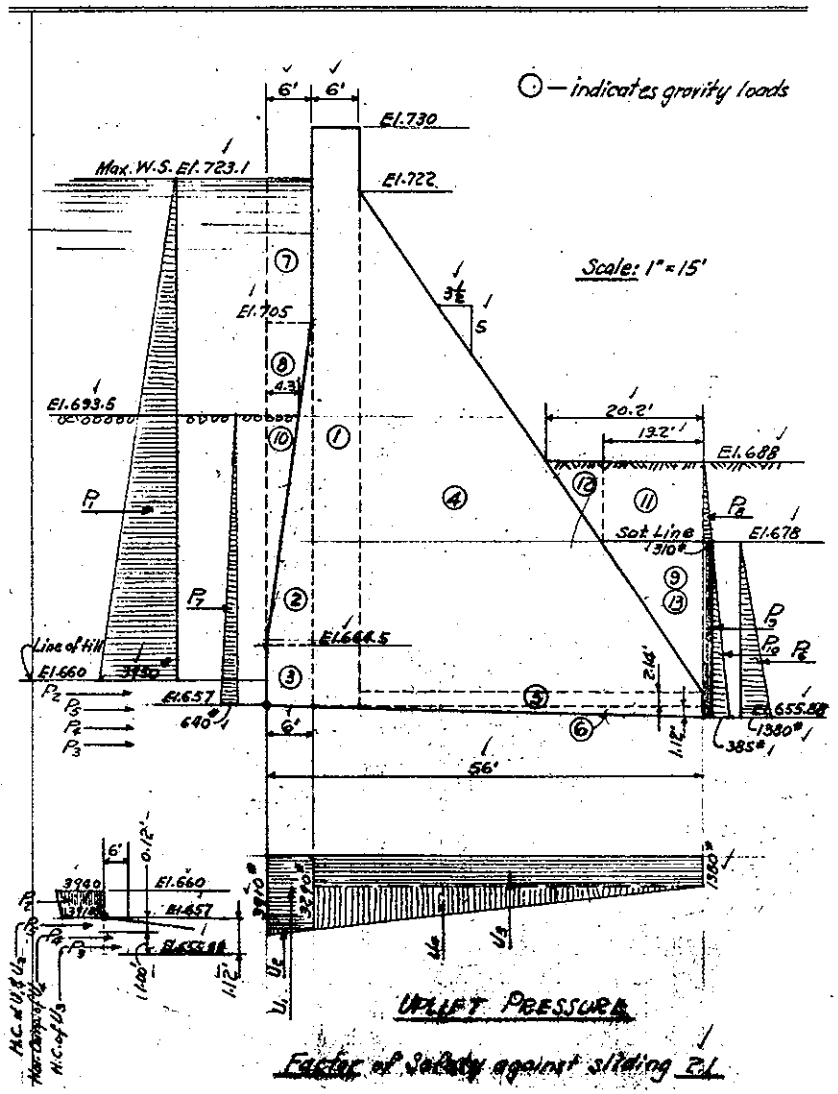


Plate VIII-31

CHAS. T. MAIN, INC, ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

Name: Bennington Dam
Subject: Non-overflow Section
Job No. US-16-18 Sheet No. 24
Date: Oct. 29 1945
Computed by: L.G.R.
Checked by: B.A.B.

Stability Analysis

Description	Mark	Dimensions	Area	Wt #/c.f.	Force K / Ft.	Arm Ft.	Mom.
Concr	①	73 × 6	438	150	65.7	9.0	591
	②	½ × 40.5 × 6	121.5	"	18.2	4.0	73
	③	7.5 × 6	45	"	6.7	3.0	20
	④	½ × 62.86 × 44	1383	"	207.5	26.67	5534
	⑤	2.14 × 44	94.2	"	14.1	34.0	480
	⑥	½ × 1.12 × 56	31.4	"	4.7	37.33	176
Water	⑦	18.1 × 6	108.6	62.5	6.8	3.0	20
	⑧	½ × 40.5 × 6	121.5	"	7.6	2.0	15
	⑨	½ × 18.86 × 13.2	124.5	"	7.8	51.6	402
Earth	⑩	½ × 29 × 4.3	62.3	80	5.0	1.43	7
	⑪	10 × 13.2	132	125	16.5	49.4	813
	⑫	½ × 10 × 7	35	125	4.4	40.47	178
	⑬	½ × 18.86 × 13.2	124.5	80	10.0	51.6	516
					375.0		8825
Uplift	U_1	3290 × 6			−19.7	3.0	−59
	U_2	½ × 620 × 6			−1.9	2.0	−4
	U_3	1380 × 50			−69.0	31.0	−2140
	U_4	½ × 1910 × 50			−47.7	22.67	−1083
					ΣV = 236.7		5539
Hor. Water	P_1 →	½ × 3940 × 63.1			124.3	24.03	2990
	P_2 →	3925 × 3			11.8	1.5	18
	P_5 →	3609 × 0.12			0.4	−0.06	−
	P_3 →	1380 × 1.0			1.4	−0.62	−1
	P_4 →	½ × 1910 × 1.0			1.0	−0.45	−
	P_6 ←	½ × 1380 × 22.12			−15.2	6.25	−95
Hor. Earth	P_7 →	½ × 640 × 36.5			11.7	12.17	142
	P_8 ←	½ × 310 × 10			−1.5	24.33	−37
	P_9 ←	310 × 22.12			−6.8	9.94	−68
	P_{10} ←	½ × 385 × 22.12			−4.3	6.25	−27
					ΣH = 122.9		ΣM = 8461

ΣV = 236.7^K ΣM = 8461'^K

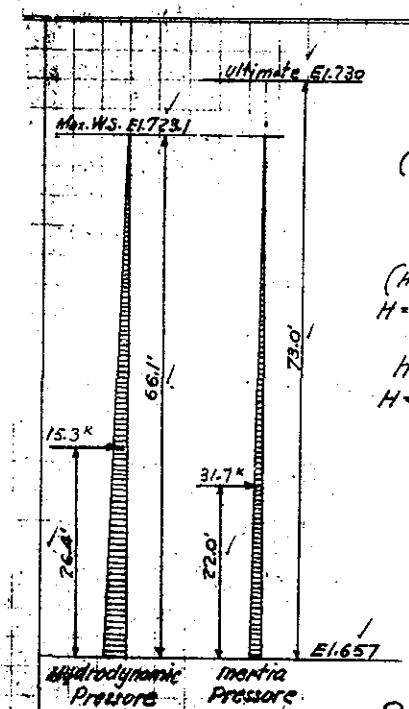
d = 35.7' ⅔ × 56 = 37.4' R falls 1.7' inside 3rd pt.

CHAS. T. MAIN, INC. ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

Name Bennington Dam
Subject Non-overflow Section.

Plate VIII - 32
Job No. US-16-15 Sheet No. 25
Date Oct. 29 1945
Computed by B.A.B.
Checked by J.G.R.

Stability Analysis.



Earthquake Stresses on Base.

(no vertical acceleration considered)

Gravity Wt.: 316.9^k
(Hor.) Inertia Force = 0.1 × 316.9 = 31.7^k
H = 66.1' Arm = ½ H = 22'

Hydrodynamic Force: 0.00347 × 66.1² = 15.3^k
H = 66.1' Arm = 0.4 × 66.1 = 26.4'

Reservoir Full.

ΣV = 236.7^k			6539
(Hor.) Water & Earth	=	122.8	2922
Hydrodynamic	=	15.3 @ 26.4	404
Inertia	=	31.7 @ 22	698
	ΣH =	169.8	9563

ΣM = 9563 ΣV = 236.7 (at El. 657) d = 40.5'

Reservoir Empty.

	ΣV = 352.8^k		ΣM = 8388^{'k}
(ΣH) ←	Hor. Earth	= -0.9	+10
←	Inertia	= -31.7	-698
		ΣH = -32.6^k	7700^{'k}

ΣM = 7700^{'k} ΣV = 352.8^k (at El. 657) d = 21.8'

CHAS. T. MAIN, INC. ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

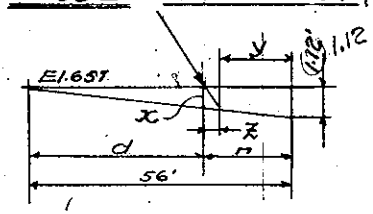
Name Bennington Dam
Subject Non-overflow Section

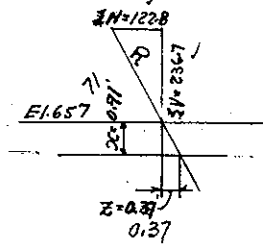
Plate VIII - 33
Job No. US-16-15 Sheet No. 26
Date Oct. 30 1945
Computed by B.A.B.
Checked by J.G.R.

Stability Analysis.

Corrections for Location of Resultants due to Base Slope.

Case I Reservoir Full (no earthquake considered)





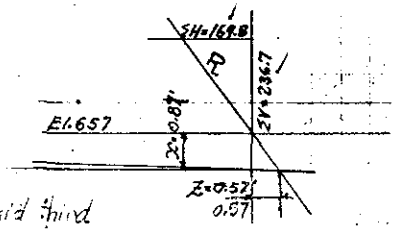
d = 35.7'
n = 56 - 35.7 = 20.3'
x = 1.12/56 × 35.7 = 0.71' 0.71
z = 0.37' 0.37 0.37 20.0
y = n - z = 20.3 - 0.37 = 19.93' ∴ e = 56/2 - 19.93 = 8.07'

Case II Reservoir Empty (no earthquake)

d = 23.8' Resultant considered as vertical ∴ z = 0
n = y = 56 - 23.8 = 32.8' ∴ e = 56/2 - 23.8 = 4.2'

Case III Reservoir Full - earthquake considered.

d = 40.5' (see Sh. 25)
n = 15.5' 0.8
x = 1.12/56 × 40.5 = 0.8'
z = 0.57' 0.57 14.93
y = 15.5 - 0.57 = 14.93' 13.07
∴ e = 56/2 - 14.93 = 13.07'
3.7% outside mid third



Case IV Reservoir Empty - earthquake

d = 21.8 (see Sh. 25) Resultant considered as vertical ∴ z = 0
y = n = 56 - 21.8 = 34.2'

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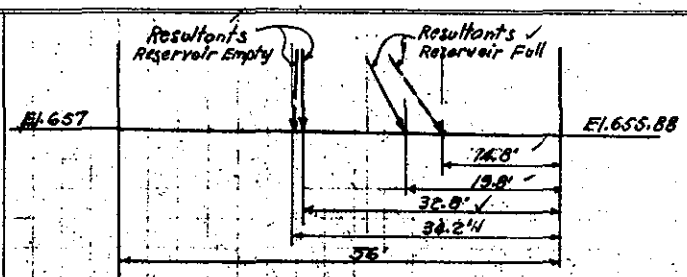
BOSTON, MASS.

Name Bennington Dam
Subject Non-overflow Section

Plate VIII - 34
Job No. US-16-15 Sheet No. 27
Date Oct. 30 19 45
Computed by B.A.B.
Checked by J.G.R.

Stability Analysis.



LOCATION OF RESULTANTS.

Pressure on Base:

Reservoir Full (no earthquake)

$e = 8.0'$ (see Sh. 26) $f = \frac{236.7}{56}\left(1 \pm \frac{6 \times 8.0}{56}\right)$ } 8.0^k, 0.5^k, .6

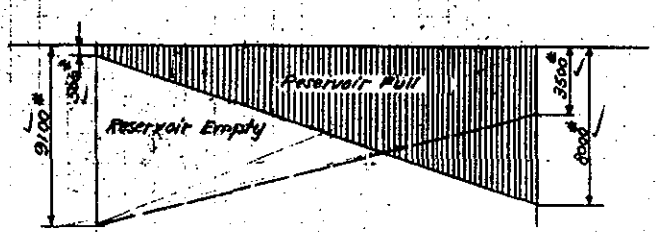
$\Sigma V = 236.7^k$

$b = 56'$

Reservoir Empty (no earthquake)

$e = 4.2'$ (see Sh. 26) $f = \frac{352.8}{56}\left(1 \pm \frac{6 \times 4.2}{56}\right)$ } 9.1^k, 3.5^k

$\Sigma V = 352.8$



FOUNDATION PRESSURE.

Factor of Safety against Sliding.

Reservoir Full - (no earthquake considered)

$\Sigma V = 236.7^k$ ave. load: $\frac{236.7}{56 \times 2} = 2.12$ tons/ft²; ave. shearing capacity of soil = 2.3 tons/ft²

$\Sigma H = 122.8^k$ ave. shearing force $= \frac{122.8}{56 \times 2} = 1.1$ tons/ft²

Factor: $\frac{2.3}{1.1} = 2.1$

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BOSTON, MASS.

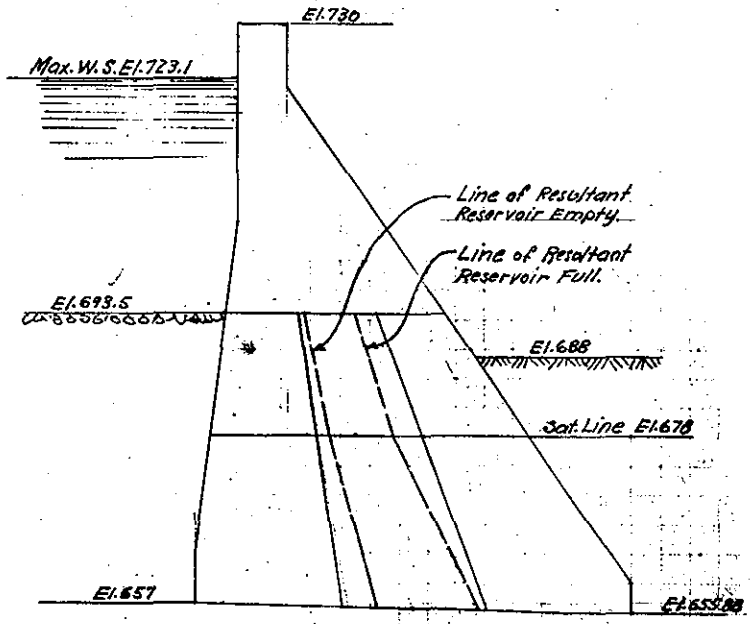
Name Bennington Dam
Subject Non-overflow Section

Plate VIII - 35
Job No. US-16-15 Sheet No. 28
Date Oct. 30 19 45
Computed by B.A.B.
Checked by J.G.R.

Stability Analysis.

Line of Resultants.

Scale: 1" = 15'



d_1	d_2	d_3	d_4	Elev.
9.22	9.98	16.30	18.44	693.5
13.60	15.32	23.20	27.20	678.0
18.67	23.20	36.20	37.34	657.0

$d_1 = b/3$
d_2 - Res. Empty
d_3 - Res. Full
d_4 - $2/3\,b$

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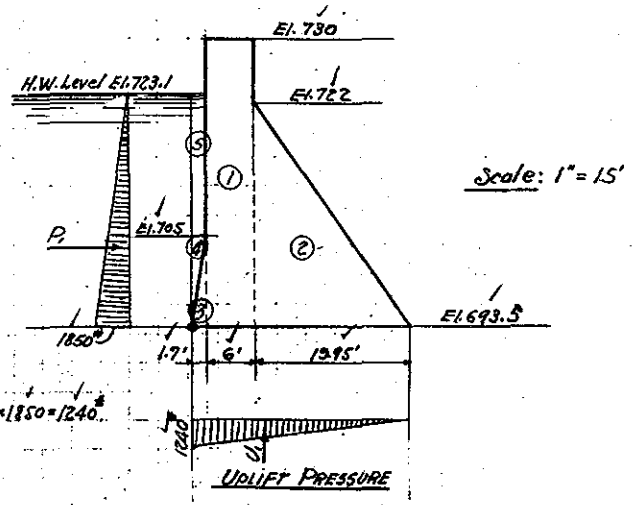
Name Bennington Dam
Subject Non-overflow Section.

Plate XIII - 36
Job No. U.S-16-15 Sheet No. 29
Date Oct. 31 1945
Computed by B.A.R.
Checked by J.G.R.

Elev. 693.5
Stability Analysis



Description	Mark	Dimensions	Area	Wt. #/c.f.	Force K/Ft.	Arm Ft.	Mom.
Concrete	①	6×36.5	219.0	150	32.8	4.7	154
	②	½×28.5×19.95	284.0	"	42.5	14.35	610
	③	½×1.5×1.7	9.8	"	1.5	1.13	2
Water	④	½×1.5×1.7	9.8	62.5	0.6	0.56	–
	⑤	1.7×18.1	30.8	"	2.9	0.85	2
					79.3		768
Uplift	U_1	½×1240×27.65			– 17.2	9.21	– 158
Total Vert.					ΣV = 62.1		610
Hor. Water	P_1	½×1850×36.5			ΣH = 33.8	12.17	411
							ΣM = 1021

ΣV = 62.1 ΣH = 33.8

$d = \frac{\Sigma M}{\Sigma V} = \frac{1021}{62.1} = 16.3'$ 2/3×27.65 = 18.44' Resultant falls 2.2' inside third point.

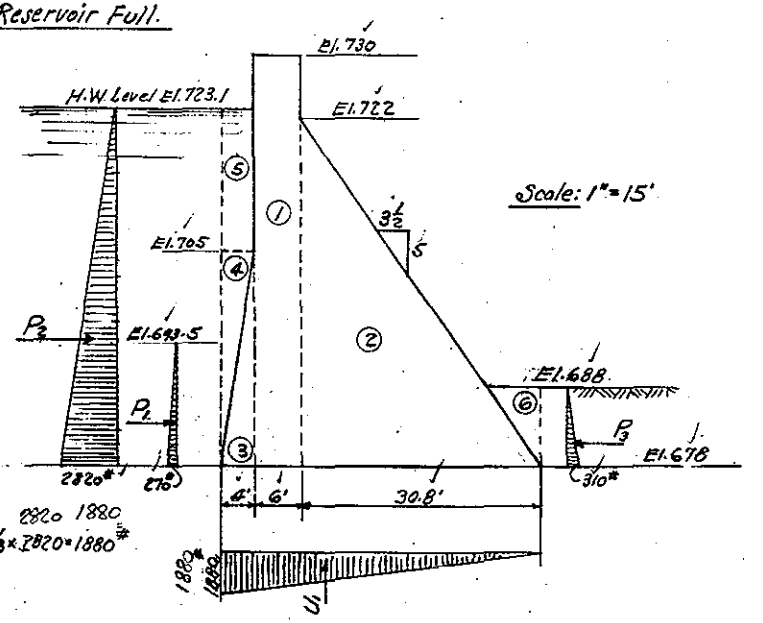
CHAS. T. MAIN, INC. ENG'RS.
201 DEVONSHIRE ST.
BOSTON, MASS.

Name Bennington Dam
Subject Non-overflow Section

Plate XIII - 37
Job No. U.S-16-15 Sheet No. 30
Date Oct. 31 1945
Computed by B.A.R.
Checked by J.G.R.

Elev. 678
Stability Analysis

Reservoir Full.



Description	Mark	Dimensions	Area	Wt. #/c.ft	Force K/Ft	Arm Ft.	Mom.
Concrete	①	6×52	312.0	150	47.7	7.0	334
	②	½×30.8×44	677.0	"	101.7	20.3	2063
	③	½×4×27	54.0	"	8.1	2.67	22
Water	④	½×4×27	54.0	62.5	3.4	1.33	5
	⑤	4×18.1	72.4	"	4.5	2.0	9
	⑥	½×7×10	35.0	"	2.2	38.47	85
					167.6		2518
Uplift	U_1	½×1880×40.8			– 38.8	13.6	– ~~525~~ 528
Total Vert.					ΣV = 129.0		1993 ~~1993~~
Hor. Water	P_1	½×270×15.5			2.1	5.2	11
	P_2	½×2820×45.1			63.5	15.03	955
	P_3	½×310×10			– 1.6	– 3.33	– 5
			2954	129			ΣM = ~~2954~~ 2955

ΣV = 129 (129.0) ΣH = 64 d = ~~2954~~ : 129 = 22.9' O.K.

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201 DEVONSHIRE ST.
BOSTON, MASS.

Name: Bennington Dam
Subject: Non-overflow Section

Plate VIII-3B
Job No. U.S.-16-15 Sheet No. 31
Date: Oct. 31 1945
Computed by: R.M.B.
Checked by: G.G.R.

Stability Analysis.

Location of Resultants.

Section at Elev. 693.5.
Reservoir Empty.

$\Sigma V = 76.8^k$ $\Sigma M = 766'^k$ (see Sheet 28)

$d = \frac{\Sigma M}{\Sigma V} = 9.98'$ $\quad 2/3\,b = 2/3 \times 27.65 = 18.44'$

Resultant falls 0.76 inside 3rd.pt.

Section at Elev. 678.
Reservoir Empty.

$\Sigma V = 157.5^k$ $\Sigma M = 2419'^k$ (see sheet 29)

$d = \frac{\Sigma M}{\Sigma V} = 15.32'$ $\quad 2/3\,b = 2/3 \times 40.8 = 27.2'$

Resultant falls 1.72' inside 3rd.pt.

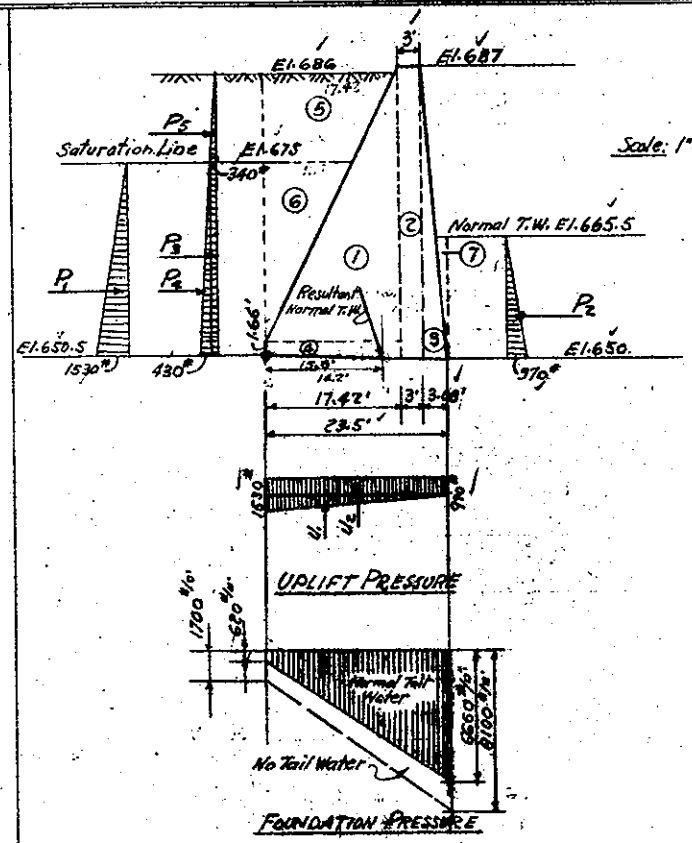
CHAS. T. MAIN, INC. ENG'RS.
201 DEVONSHIRE ST.
BOSTON, MASS.

Name: Bennington Dam
Subject: Discharge Channel Wall-West

Plate XIII-3B
Job No. U.S.-16-15 Sheet No. 32
Date: Nov. 1 1945
Computed by: R.O.B.
Checked by: E.A.H.

Stability Analysis.

Note: Treated as one foot Section



Scale: 1" = 15'

Factor of Safety against Sliding 3.5

SECTION B-B

Note: For Location of Sec. B-B see Sheet 35

CHAS. T. MAIN, INC. ENG'RS.
201 DEVONSHIRE ST.
BOSTON, MASS.

Name Bennington Dam
Subject Discharge Channel Wall - West
Job No. US-16-15 Sheet No. 33
Date Nov. 1 1945
Computed by B.A.B.
Checked by E.A.H.

Stability Analysis.

	Description	Mark	Dimensions Ft.	Area Sq. Ft.	Wt. #/c.f.	Force K/Ft.	Arm Ft.	Mom.
	Concrete	①	½ × 34.84 × 17.42	304.0	150	45.6	11.60	529
		②	37 × 3.3	111.0	"	16.6	18.92	316
		③	½ × 3.7 × 3.08	5.5	"	8.2	21.45	176
		④	1.85 × 17.42	32.2	"	4.8	8.71	42
(dry)	Earth	⑤	½(17.42+11.8) × 11	161.0	125	20.2	7.38	149
(submerged)	"	⑥	½ × 22.84 × 11.8	134.8	142.5 ×	19.2	3.93	76
	Water	⑦	½ × 15.5 × 1.29	10.0	62.5	0.6	23.07	14
						115.2		1302
	Uplift	U_1	½ × 560 × 23.5			− 6.6	7.83	− 52
		U_2	970 × 23.5			− 22.8	11.75	− 278
	Total	Vert.				ΣV = 85.8		972
	Hor. Water	P_1	½ × 1530 × 24.5			18.7	8.17	153
		P_2	½ × 970 × 15.5			− 7.5	4.67	− 35
	Hor. Earth	P_3	340 × 24.5			8.3	12.25	102
		P_4	½ × 430 × 24.5			5.3	8.17	44
		P_5	½ × 340 × 11			1.9	28.16	53
						ΣH = 26.7		ΣM = 1289

Normal Tail Water.

$\Sigma V = 85.8^K$ $\Sigma M = 1289'^K$

$d = \frac{\Sigma M}{\Sigma V} = \frac{1289}{85.8} = 15'$. $2/3 \times 23.5 = 15.7'$: Resultant falls 0.7' inside 3rd pt.

$\therefore e = 15.0 - \frac{23.5}{2} = 3.25'$

Pressure on Base.

$$f = \frac{85.8}{23.5}\left(1 \pm \frac{6 \times 3.25}{23.5}\right) \Big\}$$

$$f = 3.65(1 \pm 0.83) \Big\} \begin{matrix} 6.66^K \\ 0.62^K \end{matrix}$$

Factor of Safety against Sliding.

Reservoir Full.

ave. load: $\frac{85.8}{23.5 \times 2} = 1.82^T/\square'$ $\therefore$ Shearing Capacity of Soil = $2.0^T/\square'$

ave. shearing force: $\frac{26.7}{23.5 \times 2} = 0.57^T/\square'$ Factor: $\frac{2.0}{0.57} = 3.5$

CHAS. T. MAIN, INC. ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

Name Bennington Dam
Subject Discharge Channel Wall - West
Job No. US-16-15 Sheet No. 34
Date Nov. 1 1945
Computed by B.A.B.
Checked by E.A.H.

Stability Analysis

No Tail Water.

Description	Mark	Dimensions Ft.	Area Sq. Ft.	Wt. Cu. Ft.	Force K/Ft.	Arm Ft.	Mom. Ft./K
Concrete & Earth		see Sheet 33		ΣV =	114.6		1288
Hor. Water	P_1	see Sheet 32 & 33			18.7		153
Hor. Earth	P_3				8.3		102
	P_4				5.3		44
	P_5				1.9		53
				ΣH =	34.2	ΣM =	1640

$\Sigma V = 114.6^K$ $\Sigma M = 1640'^K$ $d = \frac{\Sigma M}{\Sigma V} = 14.3'$: R falls 1.4' inside 3rd pt.

$\therefore e = 14.3 - 11.75 = 2.55'$

Founda. Pressure: $f = \frac{114.6}{23.5}\left(1 \pm \frac{6 \times 2.55}{23.5}\right) \Big\}$

$$f = 4.88(1 \pm 0.65) \Big\} \begin{matrix} 8.05^K \\ 1.71^K \end{matrix}$$

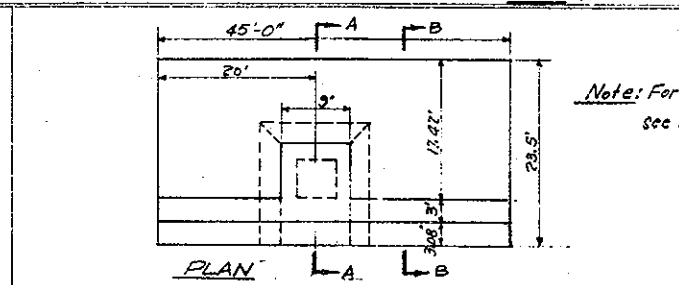
CHAS. T. MAIN, INC. ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

Name: Bennington Dam
Subject: Discharge Channel Wall - West

Plate VII - 4/2
Job No. US-16-73 Sheet No. 35
Date: Dec. 20 19 45
Computed by: R.J.B.
Checked by: E.A.H.

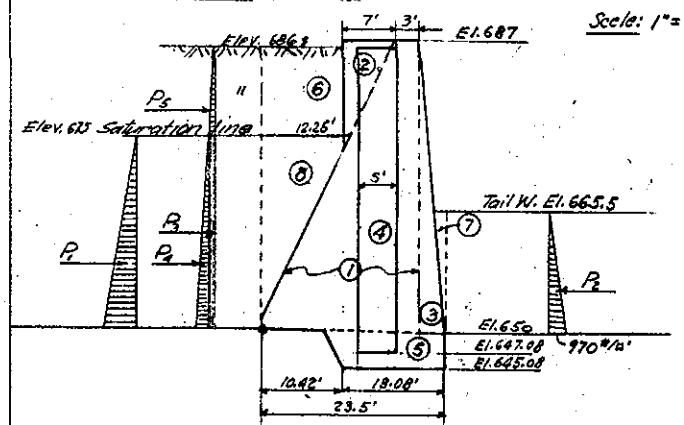
Stability Analysis

Note: Treated as Monolith.



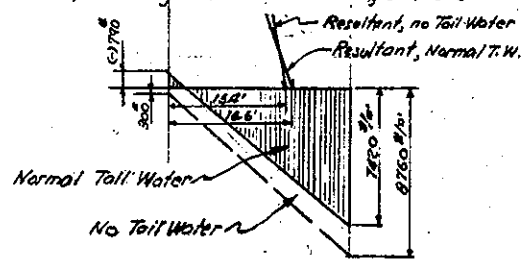
Note: For Section B-B see Sheet 32

Scale: 1" = 15'



SECTION A-A

For Uplift Diagram, see Sheet 32, Sect. B-B.



FOUNDATION PRESSURE.

CHAS. T. MAIN, INC., ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

Name: Bennington Dam
Subject: Discharge Channel Wall - West

Plate VII - 4/3
Job No. US-16-73 Sheet No. 9
Date: Dec. 20 19 45
Computed by: R.J.B.
Checked by: E.A.H.

Stability Analysis

No Tail Water.

Description		Dimensions Ft.	Volume Cu. Ft.	Wt. (Cu. Ft.)	Force Kips	Arm Ft.	Mom. Ft. K.
Concrete	①	$\frac{1}{2}(3+20.42)\times 37\times 45$	19500	150	2920.	13.50	39400
	②	$\frac{1}{2}\times 7\times 14\times 3$	442	"	66.	12.75	840
	③	$\frac{1}{2}\times 3.08\times 37\times 45$	2560	"	384.	22.48	8650
	⑤	$\frac{1}{2}\times 5\times(8+6+\sqrt{36})$	915.	"	137.	19.0	2600
		Gross			3507.		51490
Voids.	④	$5\times 5\times 39.92$	998	"	− 150.	14.92	− 2240
		Net			3357.		49250
Wt. of Earth	⑥	$(12.25+17.75)\times\frac{1}{2}\times 11\times 45$	7420	125	928.	7.57	7040
	⑧	$\frac{1}{2}\times 24.5\times 12.25\times 45$	6760	142.5 ×	964.	4.08	3940
		Gross Wt. of Earth					
Voids	⑦	$\frac{1}{2}\times 11\times 6.5\times 9$	320	125	− 40.	12.62	− 510
		Net Wt. of Earth					
		Total Vert.					
Hor. Earth	P_3	8.3 × 45			374.	12.25	4580
" "	P_4	5.3 × 45			238.	8.17	1945
" "	P_5	1.9 × 45			85.	28.16	2390
Hor. Water	P_1	18.7 × 45			842.	8.17	6880
		see Sheet 33		ΣH =	1539.		15795
						ΣM =	

$\Sigma V = 4787^k$ $\Sigma M = 73795'^k$ $d = \frac{\Sigma M}{\Sigma V} = 15.4'$ R falls 0.3' inside 3rd pt. ($\frac{2}{3}\times 23.5 = 15.7'$)

$\therefore e = 15.4 - 11.75 = 3.65'$

eccentricity about other axis is neglected → Foundn. Pressure: $f = \frac{4787}{23.5\times 45}\left(1 \pm \frac{6\times 3.65}{23.5}\right)$ (0.933, 0.933) → 0.76 k/□', 0.30 k/□'

Plate VIII - 47

CHAS. T. MAIN, INC., ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

Name: Bennington Dam
Subject: Discharge Channel Wall - West
Job No. 15-16-25 Sheet No. 37
Date: Dec. 20 1945
Computed by: B.A.B.
Checked by: E.A.H.

Stability Analysis

Normal Tail Water.

Description	Dimensions	Volume Cu. Ft.	Wt. (Cu. Ft.)	Force Kips	Arm Ft.	Mom. Ft. K.
Total Vert. from Sheet 36						
Wt. Water ⑦	$\frac{1}{2} \times 15.5 \times 1.29 \times 45$		62.5	28	23.07	650
Uplift from Sheet 33 (29.4×45)				−1322	(330×45)	−14850
Total Vert.			$\Sigma V =$			
Total Hor. from Sheet 36				1539		15795
Hor. Water P_2	$\frac{1}{2} \times 970 \times 15.5 \times 45$			−348	5.17	−1800
			$\Sigma H =$	1191		13995
					$\Sigma M =$	

$\Sigma V = 3493^k$ $\Sigma M = 57795^{'k}$ $d = \frac{\Sigma M}{\Sigma V} = 16.6'$. R falls 0.9' outside 3rd pt. ($\frac{2}{3} \times 23.5 = 15.7'$)

$\therefore e = 16.6 - 11.75 = 4.85'$.

Eccentricity about other axis is neglected → Founda. Pressure: $f = \frac{3.37}{3493}\left(1 \pm \frac{1.24}{\frac{6 \times 4.85}{23.5}}\right)$ (+) 7.42 k/□' (−) 0.79 k/□'

Plate VIII - 45

CHAS. T. MAIN, INC. ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

Name: Bennington Dam
Subject: Discharge Channel Wall - East
Job No. 15-16-15 Sheet No. 38
Date: Dec. 21 1945
Computed by: B.A.B.
Checked by: E.A.H.

Stability Analysis

Note: Treated as Monolith.

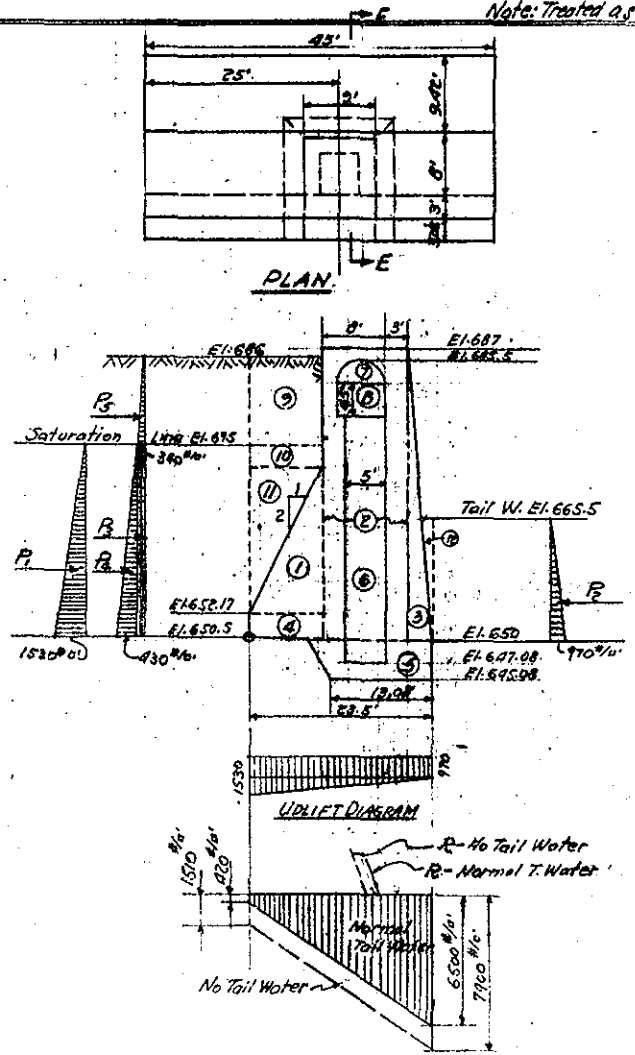


Plate VIII - 46

CHAS. T. MAIN, INC., ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

Name: Bennington Dam
Subject: Discharge Channel Wall - East
Job No. US-16-15 Sheet No. 32
Date: Dec. 21 1945
Computed by: R.A.B.
Checked by: C.A.H.

Stability Analysis

Moment about Toe Edge

Description	Dimensions	Volume Cu. Feet	Wt/Cu.ft.	Force Kips	Arm Feet	Moment Foot Kips
Concrete ①	½×9.42×[illegible]×45	3980	150	597	6.28	3750
②	11×37×45	18300	"	2740	14.92	40900
③	½×37×[illegible]×45	2560	"	384	21.45	8250
④	1.68×9.42×45	714	"	107	4.71	500
⑤	see ⑤ Sheet 36		"	137		2600
Gross				3965		56000
Voids						
⑥	5×5×31	775	"	- 116	14.92	- 1732
⑦	½×[illegible]×3²×45	634	"	- 95	14.42	- 1370
⑧	4.5×6×45	1226	"	- 182	14.42	- 2620
Total Voids				- 393		- 5722
Concrete Net				3572		50278
Wt. of Earth dry ⑨	9.42×11×45	4670	125	583	4.71	2750
" sat. ⑩	4×9.42×45	1695	142.5 ×	242	4.71	1140
" sat. ⑪	½×9.42×18.84×45	3980	142.5 ×	567	3.14	1780
Wt. Water ⑫	½×15.5×1.29×45	450	62.5	28	23.07	650
Gross Vert.				4992		56598
Uplift U_1 & U_2	see Sheet 32 & 33 (29.4²×45)			- 1320	(330^k × 45)	- 14850
Total Vert.			ΣV =	3672		41748
Hor. Water P_1	½×1530×24.5×45			842	8.17	6880
P_2	½×370×15.5×45			- 338	4.67	- 1580
Hor. Earth P_3	340×24.5×45			375	12.25	4590
" P_4	½×430×24.5×45			237	8.17	1940
" P_5	½×340×11×45			84	28.16	2360
			ΣH =	1200		14190
						55938

Normal Tail Water.

⅔×23.5 = 15.7'

$\Sigma V = 3672^k$ $\Sigma M = 55938^{'k}$ $d = \frac{\Sigma M}{\Sigma V} = 15.2'$ R falls 0.5' inside 3rd pt.

$\therefore e = 15.2 - \frac{23.5}{2} = 3.45'$

Eccentricity about other axis is neglected → Founda. Pressure: $f = \frac{3672}{23.5 \times 45}\left(1 \pm \frac{6 \times 3.45}{23.5}\right)$ 6.38 k/□' 0.92 k/□'

$f = 3.46\,(1 \pm 0.88)$

Plate VIII - 47

CHAS. T. MAIN, INC. ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

Name: Bennington Dam
Subject: Discharge Channel Wall - East
Job No. US-16-15 Sheet No. 40
Date: Dec. 22 1945
Computed by: R.A.B.
Checked by: C.A.H.

Stability Analysis

No Tail Water.

Description	Dimensions	Volume Cu. Ft.	Wt. Cu. Ft.	Force Kips	Arm Ft.	Mom. Ft. K.
Concrete Net	from Sheet 39			3572		50278
Wt. of Earth ⑨				583		2750
⑩				242		1140
⑪				567		1780
Total Vert.			ΣV =	4964		55948
Hor. Water P_1	from Sheet 39			842		6880
Hor. Earth P_3	from Sheet 39			375		4590
" P_4	from Sheet 39			237		1940
" P_5	from Sheet 39			84		2360
Total Hor.			ΣH =	1538		ΣM = 71718

$\Sigma V = 4964^k$ $\Sigma M = 71718^{'k}$ $d = \frac{\Sigma M}{\Sigma V} = 14.4'$ R falls 1.3' inside 3rd pt.

$\therefore e = 14.4' - 11.75 = 2.65'$

Eccentricity about other axis is neglected → Founda. Pressure: $f = \frac{4964}{23.5 \times 45}\left(1 \pm \frac{6 \times 2.65}{23.5}\right)$

$f = 4.7\,(1 \pm 0.68)$ 7.9 k/□' 1.51 k/□'

Factor of Safety against Sliding.

(for Data see Sheet 32) $\Sigma V = 3672^k$ $\Sigma H = 1200^k$

Area: 23.5'×45'

Ave. Load: $\frac{3672}{23.5 \times 45 \times 2} = 1.73$ Tons/□' ∴ (from Sheet 1-C) Shearing Capacity of Soil = 2.0 T/□'

Ave. Shearing Force: $\frac{1200}{23.5 \times 45 \times 2} = 0.57$ T/□'

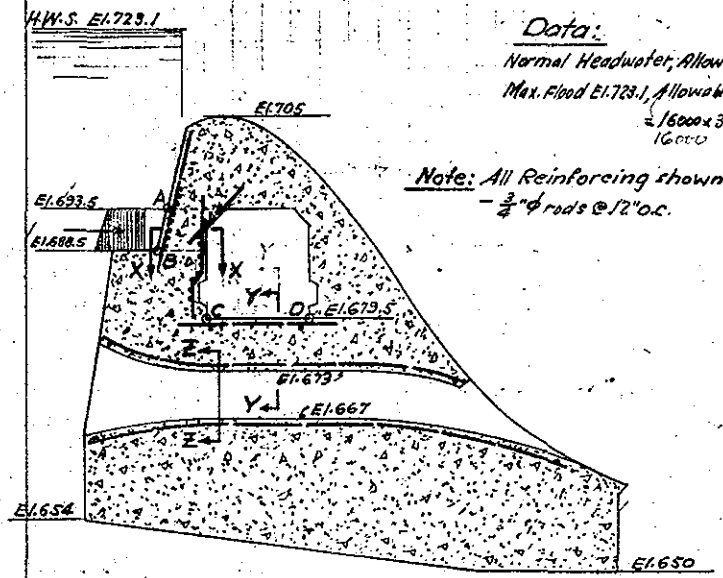
Factor: $\frac{2.0}{0.57} = 3.5$ O.K.

CHAS. T. MAIN, INC., ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

Name: Bennington Dam
Subject: Initial Gated Section

Plate VIII - 48
Job No. US-16-15 Sheet No. 41
Date: Nov. 7 19 45
Computed by: BAR
Checked by: GRR

Reinforcement.



Scale: 1" = 15'

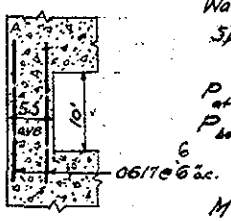
Data:

Normal Headwater, Allowable = 16000 (12000)

Max. Flood El. 723.1, Allowable = 16000 × 30% = 21000 (16000) (21000)

Note: All Reinforcing shown – $\frac{3}{4}$"φ rods @ 12" o.c.

Wall "A-B"



SEC. X-X

Wall designed as a beam with fixed ends.

Span - 10' $M = \frac{wl^2}{12}$

$P_{at\ top} = (723.1 - 693.5) \times 62.5 = 1850$ #/□'

$P_{bott.} = (723.1 - 688.5) \times 62.5 = 2160$

Load $= \frac{2160 + 1850}{2} \times 5' = 10000$ #/foot of beam

$M = \frac{10000 \times 10^2}{12} = 83300$ '#

d = use 60" $a_s = \frac{83300 \times 12}{21000 \times 7/8 \times 60} = 0.91$ □"

use min. $\frac{3}{4}$"φ @ 6" o.c.

$u = \frac{5000}{4.7 \times j \times 60} = 200$ #/□"

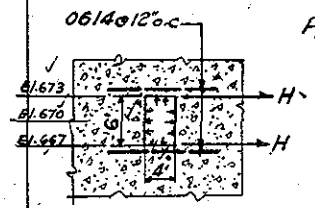
CHAS. T. MAIN, INC., ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

Name: Bennington Dam
Subject: Initial Gated Section

Plate VIII - 49
Job No. US-16-15 Sheet No. 42
Date: Nov. 2 19 45
Computed by: BAR
Checked by: GRR

Reinforcement.

Sluice.



SEC. Z-Z

Pressure (ave) $(723.1 - 670) \times 62.5 = 3320$ #/□'

$H = 3320 \times \frac{6}{2} = 9960$ #/foot

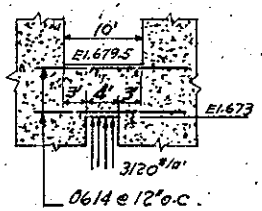
$f = 21000$

Req'd $A_s = \frac{9960}{21000} = 0.475$ □"/foot

use: $\frac{3}{4}$"φ 12" o.c. @ 0.44 □"

Top & Bott.

Slab "C-D"



SEC. Y-Y.

Pressure: $(723.1 - 673) \times 62.5 = 3120$ #/□'

Assume Span - 10' Slab with fixed ends.

" Reinforcing resists all pressure.

$M = m \times W \times L$

$W = 3.12 \times 4 = 12.48$ k $a = 0.3$

Coeff. $m = \frac{1 + 2a + 2a^2}{12} = \frac{1 + 0.6 + 0.18}{12} = 0.148$

$M = 0.148 \times 12.48 \times 10 = 18500$ '#

d = use 72" $a_s = \frac{18500 \times 12}{21000 \times 7/8 \times 72} = 0.17$ □"

use min. $\frac{3}{4}$"φ 12" o.c.

Top & Bott.

Plate VIII - 50

WAR DEPARTMENT
U. S. ENGINEER OFFICE, BOSTON, MASS.

Page.....

Subject: Bennington Reservoir
Computation: Stilling Basin
Computed by: EPP Checked by: Date: 8-26-46

M per ft. width of baffle = 46700'# resisted by 2'-0" of 5'-0" slab

$f_s = \frac{12 \times 46700 \times \frac{3}{8}}{\frac{24}{8} \times (60)^2} = 39$ #/□" unreinforced

With reinforcement d = 55"

As = 12 × 46700 / 18000 × j × 55 × 2 = 0.32□"

Req's 3/4"φ 16" o.c.

Plate VIII - 51

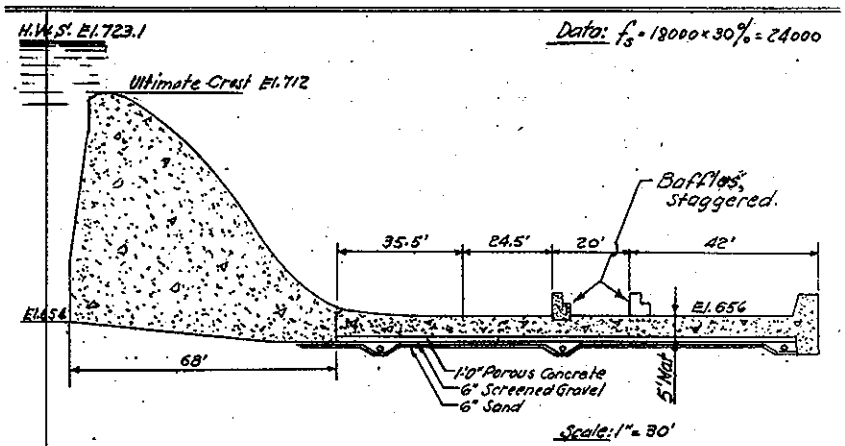
CHAS. T. MAIN, INC., ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

Name: Bennington Dam
Subject: Stilling Basin.

Job No. 45-16-15 Sheet No. 43
Date: Nov. 2 1945
Computed by: B.A.B.
Checked by: G.R.R.

Reinforcement.

Data: $f_s = 18000 \times 30\% = 24000$



Impulse Force on Baffles:

Spillway Discharge Coefficient "C" = 4.0 L = 1.0' H = 723.1 - 712 = 11.1'

$Q = C \times L \times H^{3/2} = 4 \times 1 \times 11^{3/2} = 146$ cfs/ft. width

$h = 723.1 - 656 = 67.1$

$v = \sqrt{2gh} = 8.02\sqrt{67} = 66$ f.p.s

Impulse Force $F \times t = M \times v$ (on basis of one second)

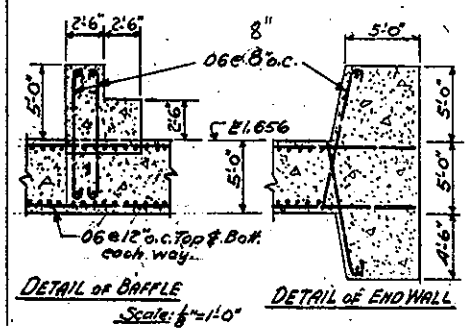
$F = \frac{146 \times 62.5}{32.2} \times 66 = 18700$ #/ width of spillway

(at bott. of baffle) $M = 18700 \times 2.5' = 46700$ '#/ft. width of baffle

$a_s = \frac{46700 \times 12}{18000 \times 7/8 \times 54} = 0.58$□"

In Baffle: use 3/4"φ 8" o.c.

In Mat: use 3/4"φ 8" o.c. each way.



DETAIL OF BAFFLE DETAIL OF END WALL

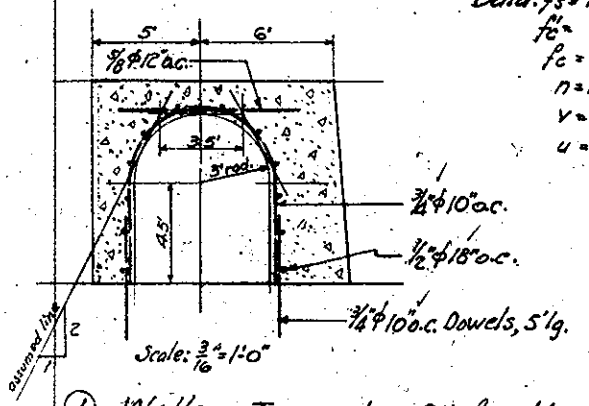
Scale: 1/8" = 1'-0"

CHAS. T. MAIN, INC., ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

Name: Bennington Dam
Subject: Discharge Channel Wall - East
Section C-C, Sheet 24
Investigation of Passageway Arch Roof

Plate VIII - 52
Job No. US-16-IS Sheet No. 43-A
Date: Dec. 24 1945
Computed by: RJB
Checked by: R

Reference: Dwg. Sheet 24



Data: $f_s = 18000$
$f'_c = 3000$, $K = 169$
$f_c = 1050$, $j = 0.88$
$n = 10$
$v = 60$
$u = 150$

① **Walls - Temperature Reinf. req'd.**

Min. Reinf. req'd = $0.0025\,b \times d$
$b = 12''$ d = use $(36'' - 3'') = 33''$

$a_s = 0.0025 \times 12 \times 33 = 0.99^{\square''}$/foot of wall
or $0.99 \div 2 = 0.5^{\square''}$ each face

use: 3/4"φ 10" o.c. ea. face
$a_s = 0.53^{\square''}$

use: 1/2"φ 18" for Horiz. Spacers.

② **Roof Slab.** - assume Temp. steel as active Reinforcing.

Span - use 3.5' (clear)
$d = 15''$
$a_s = 0.53^{\square''}$/foot

Resist. Moment: $\frac{w \times l^2 \times 12}{10} = a_s \times f_s \times j \times d$

$w = \frac{0.53 \times 18000 \times 0.88 \times 15 \times 10}{3.5^2 \times 12}$

Allowable (D+L) Load: 10000 #/□'
D.L. say 20×12 240
Allowable L.L. 9760 #/□'

Resist. Shear.
$V = v \times bjd = 60 \times 12 \times 0.88 \times 15 = 9500^{\#}$

Allowable (D+L) Load:
$9500 \div 3.5 = 2700$ (2.75 → 5400) #/□'
D.L. 240 #/□'
Allowable L.L. = ~~2460~~ 4960

Shear controls Allowable L. Load.

(continued)

CHAS. T. MAIN, INC., ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

Name: Bennington Dam
Subject: Discharge Channel Wall - East
SECTION C-C, Sheet 24
Investigation of Passageway Arch Roof

Plate VIII - 53
Job No. US-16-IS Sheet No. 43-B
Date: Dec. 24 1945
Computed by: RJB.
Checked by: R

(continued)

Check for Bond Stress.
Reinforcing - 3/4"φ 10" o.c. $\Sigma O = 2.8$
Allowable $V = 9500^{\#}$ d = use 15"
$\therefore u = \frac{9500}{2.8 \times 0.88 \times 15} = 256 > 150$

Assuming Temperature Reinf. as part of Main Reinforcing, additional rods are required, to develope full allowable shear.
$\Sigma O = \frac{9500}{0.88 \times 15 \times 150} = 4.8$ req'd.

Additional Req'd: $4.8 - 2.8 = 2.0$
use additional: 5/8"φ 12" o.c. $\Sigma O = 2.0$

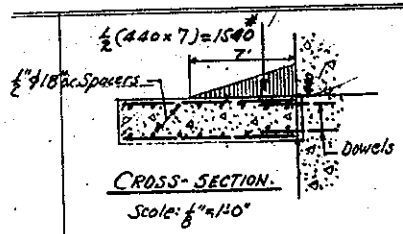
Answer.: use allowable L.L. - ~~2460~~ 4960 #/□'
" Reinforcing: Temperature Reinf. 3/4"φ 10" o.c.
Hor. Rods at Top 5/8"φ 12" o.c.

Plate VIII-54

CHAS. T. MAIN, INC., ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

Name: Bennington Dam
Subject: Abutments
Training Wall

Job No. US-16-15 Sheet No. 43-C
Date: Dec 26 1945
Computed by: RAB
Checked by: K



CROSS-SECTION.
Scale: ⅛"=1'-0"

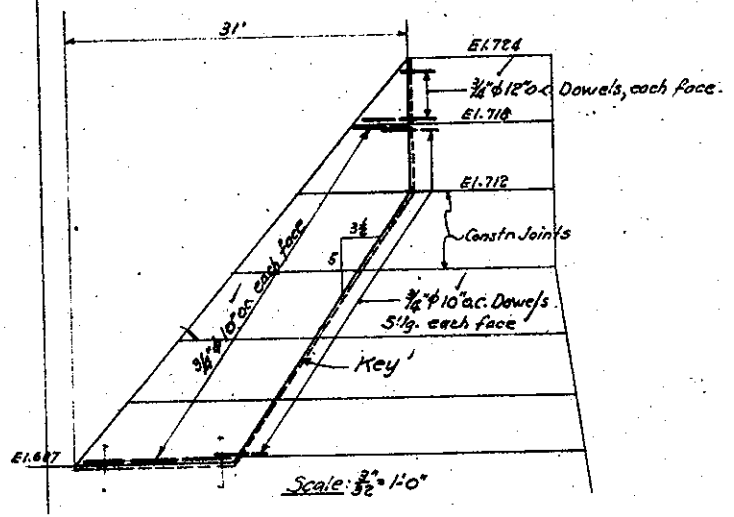
Data: $f_s = 18000$
$f_c' = 3000$
$f_c = 1050$
$n = 10$

Water static Head say, 7'
$b = 12"$ · d = we 33"

Min. Temp. Reinf.: $0.0025 \times bd$
$a_s = 0.0025 \times 12 \times 33 = 1^{\square"}$ / foot of Wall
or: $0.5^{\square"}$ each face

USE: ¾"φ 10" oc. ea. face
½"φ 18" o.c. spacers

Bend. Mom: $1540 \times 2.33 = 3600'\#$
Req'd $a_s = \frac{3.6}{1.39 \times 33} = 0.085 < 0.5$ req'd by Temp. Reinf.



Scale: 3/32" = 1'-0"

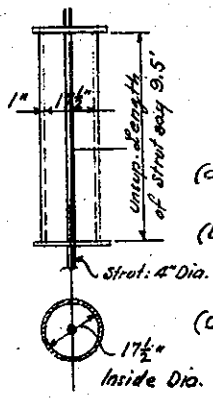
Plate VIII-55

CHAS. T. MAIN, INC., ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

Name: Bennington Dam
Subject: Miscellaneous.

Job No. US-16-15 Sheet No. 44
Date: Nov. 3 1945
Computed by: RAB.
Checked by: GRR

Strength of Operating Cylinder & Strut.



Data:
Allowable Load: 500#/□"
" " for Test: 1000#/□"

Area: $3.14 \times 17.5^2 \times \frac{1}{4} = 240^{\square"}$

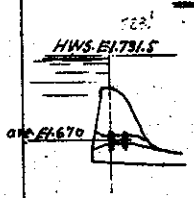
(a) Allowable Load: $500 \times 240 = 120{,}000$#

(b) Wall of Cylinder: 1" thick.
Actual $f = \frac{pr}{t} = 1000 \times 17.5 \times \frac{1}{2} = 8750\# < 18000$ O.K.

(c) Strut. Unsupp. Length $L = 9.5'$
Strut - 4"φ
$\therefore r = \frac{D}{4} = 1"$
$L/r = 9.5 \times 12 = 114$
$\therefore f_{allowable} = 10700$

Total Load allowable:
$10700 \times (3.14 \times 2^2) = 135000 > 120000$ O.K.

Hoist Capacity.



Data: Coefficient of Friction = 0.75
Gate (dimensions) = 4'×6'

Ave. Hydrostatic Pressure on Gate: $62.5 \times 53.1 = 3320$#/□'
Total Pressure: $3320 \times (4 \times 6) = 80000$#
(min. req'd) Hoist Capacity: $80000 \times 0.75 = 60000$#
Allowable Load = 120000
Factor: $\frac{120000}{60000} = 2$ O.K.

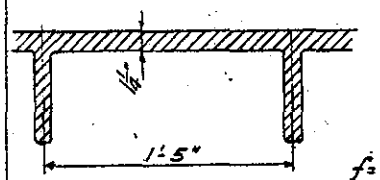
Plate VIII - 56

CHAS. T. MAIN, INC., ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

Name: Bennington Dam
Subject: Miscellaneous

Job No. 45-16-15 Sheet No. 45
Date: Nov. 3 1945
Computed by: RMB
Checked by: GRR

Strength of Gate Leaf.



Hydrostatic Pressure: 3320 #/□"

∴ 3320 ÷ 144 = 23 #/□"

$BM = 23 \times 17^2 \times \frac{1}{8} = 830$ "#'

f = 18000 Req'd S = 0.046 0.046

Actual $S = \frac{I}{c} = \frac{1.25^2}{6} = 0.26$ O.K.

$S = \frac{bd^2}{6} = \frac{1 \times 1.25^2}{6} = 0.26 \text{ in}^3$

Plate VIII - 57

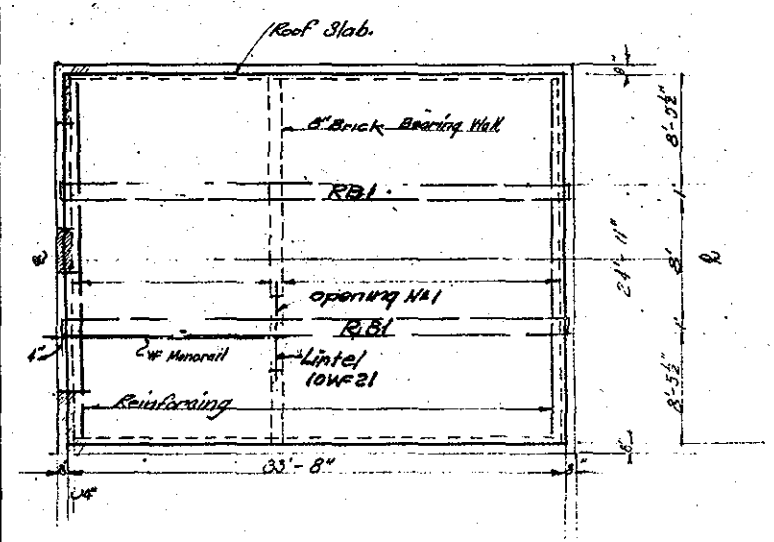
CHAS. T. MAIN, INC. ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

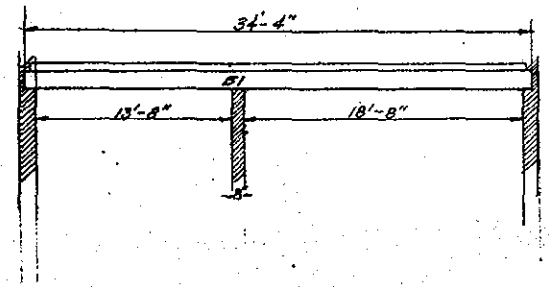
Name: Bennington Dam
Subject: Equipment Bldg.
Roof Design.

Job No. 45-16-15 Sheet No. 46
Date: January 12 1945
Computed by: Herzog
Checked by: RMB.

Roof Beam B1

Scale: 1/8" = 1'-0"





CHAS. T. MAIN, INC. ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

Name Bennington Dam
Subject Equipment Bldg.
Roof.

Job No. 115.16.15 Sheet No. 47
Date January 14 19 46
Computed by Herzog
Checked by BAB.

Slab Design.

Span 8.5'
Assum R = 4" d = 2.5

$\frac{wl^2}{10}$

① Loading:

D.L.	= 50
L.L	= 50
Roofing	= 15
	115#/□' ✓

② Moment:

$M_m = \frac{115 \times 8.5^2}{10} = 825^{1\#}$ ✓

③ Depth

$d = \sqrt{\frac{825}{169}} = 2.2''$ use: $d = 2.5''$

④ Reinforcing

$A_s = \frac{.825 \times 12}{18000 \times 2.5 \times .88} = .25^{\square''}$ ✓

use $\begin{cases} \frac{1}{2}''\phi \text{ @ } 10''\text{ o.c.} \\ \frac{3}{8}''\phi \text{ @ } 12''\text{ o.c.} \end{cases}$ spacers ✓

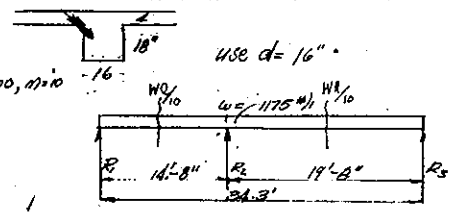
CHAS. T. MAIN, INC. ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

Name Bennington Dam
Subject Equipment Bldg.
Roof.

Job No. 115.16.15 Sheet No. 48
Date January 12 19 46
Computed by Herzog
Checked by BAB.

Roof Beam Mk. RB1 (B1 & B2 below)

Span = 34'-4"
Assume B1R = 16" x 18"
K = 169, J = .88, f_c = 3000, f_s = 18000, n = 10



Loading:

D.L Slab 4" @	50
L.L	50
Rfg	15
	115# x 8.25' = 950#
Beam Wt.	225#
	1175#/' ✓

Moment:

for d. $wl^2/8 = 1175 \times 19.2^2 \times \frac{1}{8} = 53{,}000^{1\#}$

$d = \sqrt{\frac{53{,}000 \times 12}{169 \times 16}} = 15.4''$ use $16.5''$

Reactions:

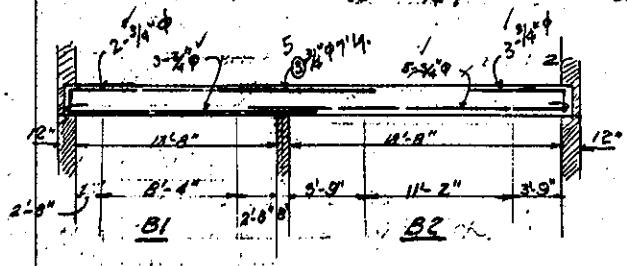
$1175 \times 7.3 \; 115\% = R_2$
$R_2 = 9150\#$ for left
$R_2 = 1175 \times 9.8 \times 115\% =$
$R_2 = 13{,}200\#$ for Right.
$R_1 = 8600\#$
$R_3 = 11{,}500\#$

Reinforcing:

Tension Steel. B1. $M = 1175 \times 14.67^2 \times \frac{1}{10} = 25200^{1\#}$ ✓ B2. $1175 \times 19.67^2 \times \frac{1}{10} = 45500^{1\#}$

$A_s = \frac{25200 \times 12}{18000 \times .88 \times 225} = 1.20^{\square''}$ ✓ Use: 3-¾"φ

$A_s = 2.22^{\square''}$ Use: 5-¾"φ ✓



For Stirrupping:

$\frac{V_m}{bdj} = \frac{9850}{16 \times 16.5 \times .88} = 45 < 60$ ✓

$= \frac{13200}{16 \times 16.5 \times .88} = 60.4$ ok.

no stirrups need except as ties use ⅜φ 18" o.c.

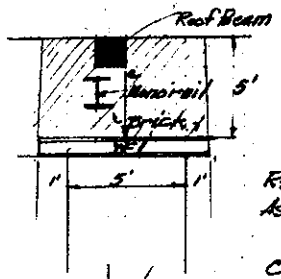
Plate XIII-60

CHAS. T. MAIN, INC. ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

Name: Bennington Dam — Job No. 115.16.15 Sheet No. 49
Subject: Equipment Bldg. — Date: January 19
Computed by: Herzog
Checked by: B.B.B.

Lintel to support Load from Monorail.

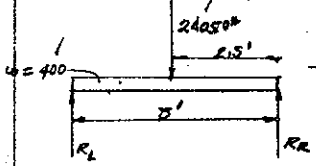
See sketch on sheet 15



c = concentrated load due to:
Reaction of roof beam +
Reaction of monorail x

R_2 - see sheet 16 = 23050#
Assume mon rail R = 1000#
24,050#

C = 24050



Loading due to brick uniform:
5 x 80 = 400#/ft

Reactions

$R_L = R_R = 25 \times 400 + 12025$ ~~13025#~~

Moment

M_m @ 2.5' = 2.5' x 13025 = 32 600
− $\frac{400}{2}$ 6.25' = 1 250
M_m = 31 350 ft#

Section Mod.

$Z = \frac{M}{S} = \frac{31\,350 \times 12}{20\,000} = 18.8$

use 10 WF 21

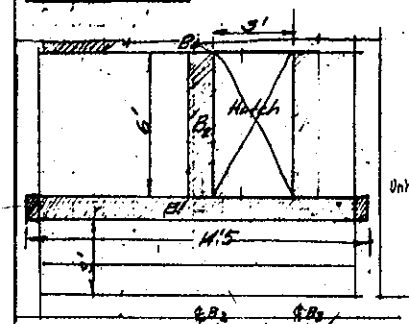
Plate XIII-61

CHAS. T. MAIN, INC. ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

Name: Bennington Dam — Job No. 115.16.15 Sheet No. 50
Subject: Equipment Bldg. — Date: Jan. 7 19 46.
Computed by: Herzog
Checked by: B.B.B.

Ground Floor.

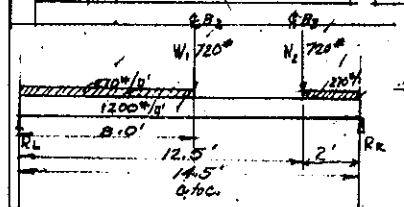
Beam 1 = B1 — Partially Fixed $Wl^2/10$



Span = 14.5' c.to.c.
Assume B_1 12" x 19"

Unif.
D.L. Slab: 7" @ 90 x 2.5' = 225#/ft over a
D.L. Slab: 7" @ 90 x 3.0' = 270#/ft of
L.L. 150#/ft² x 5.5' = 825#'s over a
Wt. of beam = 150 " "
Concentrated load 24013 = 720#
full length

D.L. 225
L.L. 825
Wt. of B. 150
1200#/ft



Loading Diagram.

D.L. 270
1470

① **Reactions:**

1200 x 7.25 =	8700#	17400
720 x 6.5/14.5 =	320	720
720 x 2.0/14.5 =	100	720
270 x 8 x 10.5/14.5 =	1560	2160
270 x 2 x 1.0/14.5 =	37	540
R_L =	10717#	21540
R_R =	10823#	V_{max}

② **Moment:** computed as for simple beam & decrease by ratio of 8/10 for reinf. at midspan.
Max Mom. at 7.45' from R_L
+ 10 717 x 7.45 = 79500
− 1470 x 7.45 x 3.7 = 40500
39000 ft#
$M_M = \frac{8}{10} \times 39000 = 31200$

③ $d = \sqrt{\frac{31200 \times 12}{169 \times 12}} = 13.6 < 17$ OK.

④ **Reinforcing:**

$A_s = \frac{31200 \times 12}{18000 \times .88 \times 17} = 1.39$ □"

use 3 - 3/4 ϕ Top & Bott.

Plate VIII -62

CHAS. T. MAIN, INC. ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

Name Bennington Dam
Subject Equipment Bldg.
Ground Floor

Job No. US 16/15 Sheet No. 51
Date 12.12.1945
Computed by R
Checked by RAB

Slab.

Main Floor 150# LL

L to R

14-4 +(1-0) + 8-11 +(1-0) + 10-0

14-4 / 1-1 / 13-3" Max Span.

DL.S = 100

LL = 150 / 250 × 13.3/10 = 4420 #

d² = 4420/169 = 26.2 d = 5+

A_s = 4420 × 12 / 18000 × .88 × 5.5 = .6.1

use 3/4" φ 9" o.c.

5/8" φ 6" o.c. d = 5.5 t = 7"

Spacer ½" φ 12" o.c.

v = 250 × 6.7 / 5.5 × 12 × .9 = 28 #/□" OK

Plate VIII -63

CHAS. T. MAIN, INC. ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

Name Bennington Dam
Subject Equipment Bldg.
Ground Floor

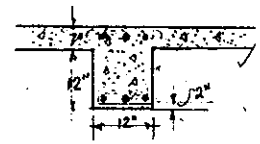
Job No. US16.75 Sheet No. 52
Date January 8 1945
Computed by Herzog
Checked by RAB

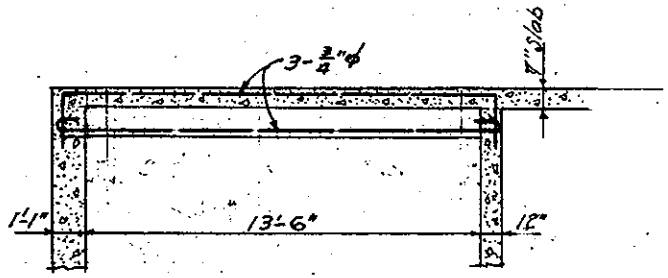
Beam B1 continued

Stirrups:

$\frac{V_m}{bdJ}$ = 10823 / 12 × 17 × .88 = v = 60.6 # / no stirrups requ'd. for Design.

Use tie stirrups ½" φ 18" o.c.





B1

Plate VIII - 64

CHAS. T. MAIN, INC. ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

Name Bennington Dam
Subject Equipment Bldg
Ground Floor

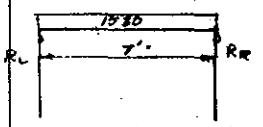
Job No. 48.16.25 Sheet No. 53
Date January 9 1945
Computed by Herzog
Checked by R.A.B.

Beam B2

$wl^2/10$

See sketch on sheet #51
Span: 7' c.to c. of Bl & Bl.
Assume B_2 = 12" x 19", d = 17".

Loading:
D.L. Slab 7" @ 90 x 4.5 = 405 #/' ✓
Wt. of beam = 150"
L.L. 150 x 6.5 = 975
1530 #/' full length.



① Reactions:
R_L = 1530 x 7.1/2 = 5350#
R_R = 5350#

② Moment: Partially fixed use $wl^2/10$ ✓

Max Moment = 1530 x 49 ÷ 10 = 7460'#

③ $d = \sqrt{\frac{7460 \times 12}{169 \times 12}}$ = 6.65" < 17 ok. ✓

④ Reinforcing:
$A_s = \frac{7460 \times 12}{18000 \times .88 \times 17}$ = 0.33 □"

- 2-½"φ Min. Reif. req'd: 0.005 x 12 x 19 = 1.13 □"

use - 2 ¾"φ Bott. ✓

Stirrups
$V_m = \frac{5350}{12 \times 17 \times .88}$ = 29.8 < 60 ok no stirrups req'd.
bdj

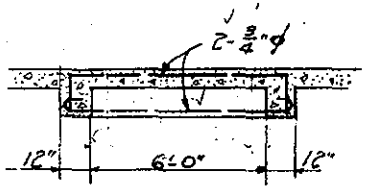


Plate VIII - 65

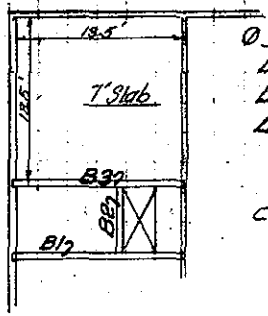
CHAS. T. MAIN, INC. ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

Name Bennington Dam
Subject Equipment Bldg
Ground Floor

Job No. 48.16.25 Sheet No. 54
Date January 22 1945
Computed by Herzog
Checked by R.A.B.

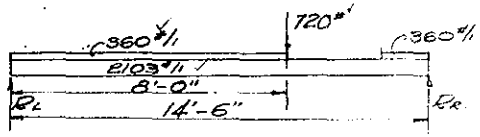
Beam-B3

Span = 14'-6" c.to c.
Assume B3 = 12" x 19", d = 17"



① Loading:
D.L. 7" Slab @ 90# x 6.25 = 563 #/' full length
D.L. 7" Slab @ 90 x 4.0 = 360 #/' 8' of len.
L.L. 150 x 10.25 = 1540 full length
Uniform = 2103 #/' full 2 only.

Concentrated 240 x 3 = 720#



② Reactions:

2103 x 7.25	=	15200#	30400
720 x 6.5/14.5	=	320	720
360 x 8 x 10.5/14.5	=	2610	2880
360 x 2 x 1'/14.5	=	49	720
R_L (max)	=	18179#	34720 #/'

③ Moment: computed as simple beam & reduced by ratio 8/10 for reinf. at midspan.
Max Mom @ 7.4' from R_L
18179 x 7.4/2 = 134500
- 2463 x 7.4 x 3.7 = 67500
Mm 67000
8/10 x 67000# = 53500'#

④ $d = \sqrt{\frac{53500}{169}}$ = 17.85
Use 17" o.k.

⑤ Reiforcing:

$A_s = \frac{53500 \times 12}{18000 \times .88 \times 17}$ = 2.38 □"

Use 4 - 7/8"φ Top & Bott. A_s = 2.4 □"

⑥ Stirrups: Assume: ½"φ A_s = .4 □
$\frac{18179}{12 \times 17 \times .88}$ = 101 > 60

$S_m = \frac{3}{2} \frac{A_s f_s jd}{V} = \frac{3 \times 0.4 \times 16 \times .88 \times 17}{2 \times 18179}$ = 8"

Plate VIII - 66

CHAS. T. MAIN, INC. ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

Name: Bennington Dam — Job No. US.16.TS — Sheet No. 55
Subject: Equipment Bldg. — Date: January 9, 19 45
Ground Floor — Computed by: Herzog
Checked by: B.A.B.

B3 Continued.

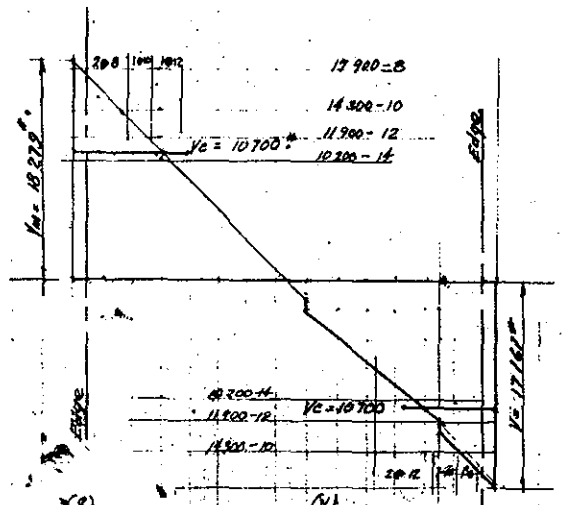
Scales: 1/4" = 1' (H.) 1" = 10 000# (V.)

$V_c = bdjv_c = 12 \times 17 \times 0.88 \times 60$
$V_c = 10{,}700^{\#}$.

Span = 14.5' c/c.

2463 x 8' = 19 704
18 279
1 425#

4.5 x 2103 = 9460



(3)	(Y)	
7.85	18 279	1.13
8.00	17 900	
10.00	14 300	
12.00	11 900	Vc = 10 700
14.00	10 200	

Spacing on left:
1/2"φ { 2 @ 8" / 1 @ 10" / 1 @ 12"

Spacing on right:
1/2"φ { 1 @ 8" / 1 @ 10" / 2 @ 12"

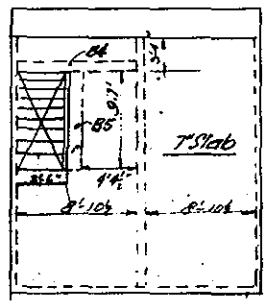
O.K.

Plate VIII - 67

CHAS. T. MAIN, INC. ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

Name: Bennington Dam — Job No. US.16.TS — Sheet No. 56
Subject: Equipment Bldg. — Date: January 22, 19 46
Ground Floor — Computed by: Herzog
Checked by: B.A.B.

Beam B5.



Data:
$f_c = 18\,000^{\#}/\square''$ $f' = 1050$
$f_c' = 3000^{\#}/\square''$ $j = 0.88$
$k = 169$ $V_c = 60$
Span approx: Clear = 9'-8" B5
Clear = 8'-10½" B4

Assume B5 12" x 7" d = 5.5"

Loading For Beam - B5
D.L. 7" Slab @ 90# x 3.17 = 284
L.L. 150 x 3.17 = 475
759#/'

Moment = 759 x 9.7² ÷ 10 = 7140'#

Reinforcing:
$A_s = \frac{7140 \times 12}{18000 \times .88 \times 5.5} = .98\square''$

$d = \sqrt{\frac{7140 \times 12}{169 \times 12}} = 6.5''$
use 5.5"

use 3-3/8"φ as additional reinf. in Slab. Top & Bott.

Notes - B4.
Use B5 for computing B4

CHAS. T. MAIN, INC. ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

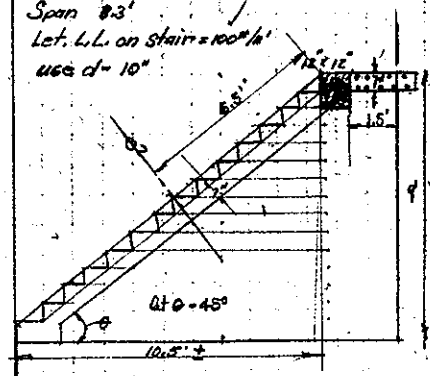
Name: Bennington Dam
Subject: Equipment Bldg.
Ground Floor.

Plate III - 68
Job No. US-16-IS Sheet No. 57
Date January 10 1945
Computed by Herzog
Checked by B.M.B.

Beam B4

For Data & Plan drawing see sheet 62

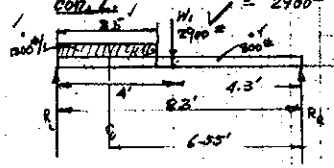
Span 8.3'
Let L.L. on stair = 100#/□'
use d = 10"



Loading:

D.L. Stair @ 100 x 6.5' = 650 @ 8.5'
L.L. Stair @ 100 x 6.5' = 650 @ 3.5'
uniform 1300#/□ @ 8.5'
Beam Wt 150# = 150# @ 8.3'
Live L. 150 = 150 @ 8.3'
Uniform load = 300#/□ @ 8.3'

Conc. L. = 2900# @ 4'



① Reactions:

Slab 300 x 4.2' = 1260# 2520
Stair 1300 x 3.5 x 6.55/8.3 = 3590# 4550
B5 - 2900 x 4.3/8.3 = 1500# 2900
R_M = 6350# 9970# ✓
R_R = 3620#

② Moment: computed as for simple beam & reduced by ratio of 8/10 for reinf. at midspan.

Max. Moment at 4' from R_L
+6350 x 4' = 25400 '#
- 1300 x 3.5 x 2.25 = 10200 '#
- 300 x 4 x 2 = 2400 '#
= -12600 '#
M_m = 8/10 x = 12800 = 10200 '#

$d = \sqrt{\frac{10200}{169}}$ = 7.8" ok
use 10".

③ Rein forcing

$a_s = \frac{10200 \times 12}{18000 \times 0.88 \times 10} = 0.772''^2$

use 2 - 3/4" ϕ T & B

4. Shear:

$\frac{V_m}{b\,j\,d} = \frac{6350}{12 \times .88 \times 10} = 60.2$ ok

CHAS. T. MAIN, INC. ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

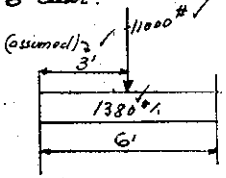
Name: Bennington Dam
Subject: Equipment Bldg.
Ground Floor

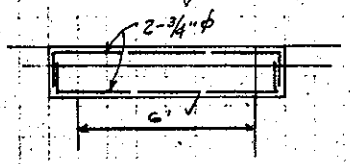
Plate III - 69
Job No. US-16-IS Sheet No. 57-A
Date Jan. 30 1946
Computed by R.M.B.
Checked by E.A.H.

Beam Mk. B5 12"x20" Span - 6' clear

Loading: Slab (D+L) @ 250#/□ x 4.5' = 1130'
Wt. Beam = 250'
uni. = 1380#/□

Reac. B1 - 11000# Conc.





$M = 1380 \times 6^2 \times \frac{1}{8} = 6200'$
$11000 \times 6 \times \frac{1}{4} = 16500'$
22700 '#

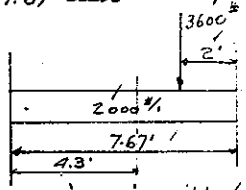
use d = 18"
$a_s = \frac{22.7}{1.29 \times 18} = 0.98''^2$

Use: 2 - 3/4" ϕ Top
2 - 3/4" ϕ Bot.

Beam Mk. B6 12"x20" Span 7.67' clear

Loading: Slab (D+L) @ 250 x 7 = 1750'
Wt. Beam = 250'
uni. = 2000#/□

Reac. B4 - 3600# Conc.



R_L = 2000 x 3.83 = 7660
3600 x 2/7.67 = 940
8600 #

Pt. max. Mom: 4.3' from left R

$M = 8600 \times 4.3 = 37000'$
$- 2000 \times 4.3 \times 2.15 = 19500'$
18500 '#

Use same Reinf. as B5
2 - 3/4" ϕ Top
2 - 3/4" ϕ Bot.

Plate VIII-70

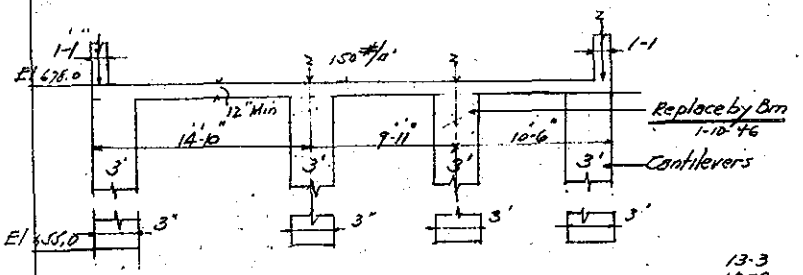
CHAS. T. MAIN, INC. ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

Name: Bennington Dam
Subject: Equip. Bldg
Job No. 11516/15 Sheet No. 58
Date 12-13 1945
Computed by R
Checked by R.W.B.

Basement Floor.

Slab.

Basement Fl. LL 150# Slab: 12" thick. 23'-0"



13-3
10-0
23-3
12-0

14-10 − (3-0 + 1-6) = 10'-4"

DL = 150

LL = 150 / 300 × 10.33/10 = 3200 # Use d = 7.5"

d = 3200/169 = 19+

As = 3200 × 1 / 18000 × .88 × 7.5 = .373

5/8" 10 oc.
or 1/2" φ 7 oc.

4" Cover

12 × 12 = 144/4 = .36 □"

5/8" φ 10 oc. d = 7.5 t = 12"

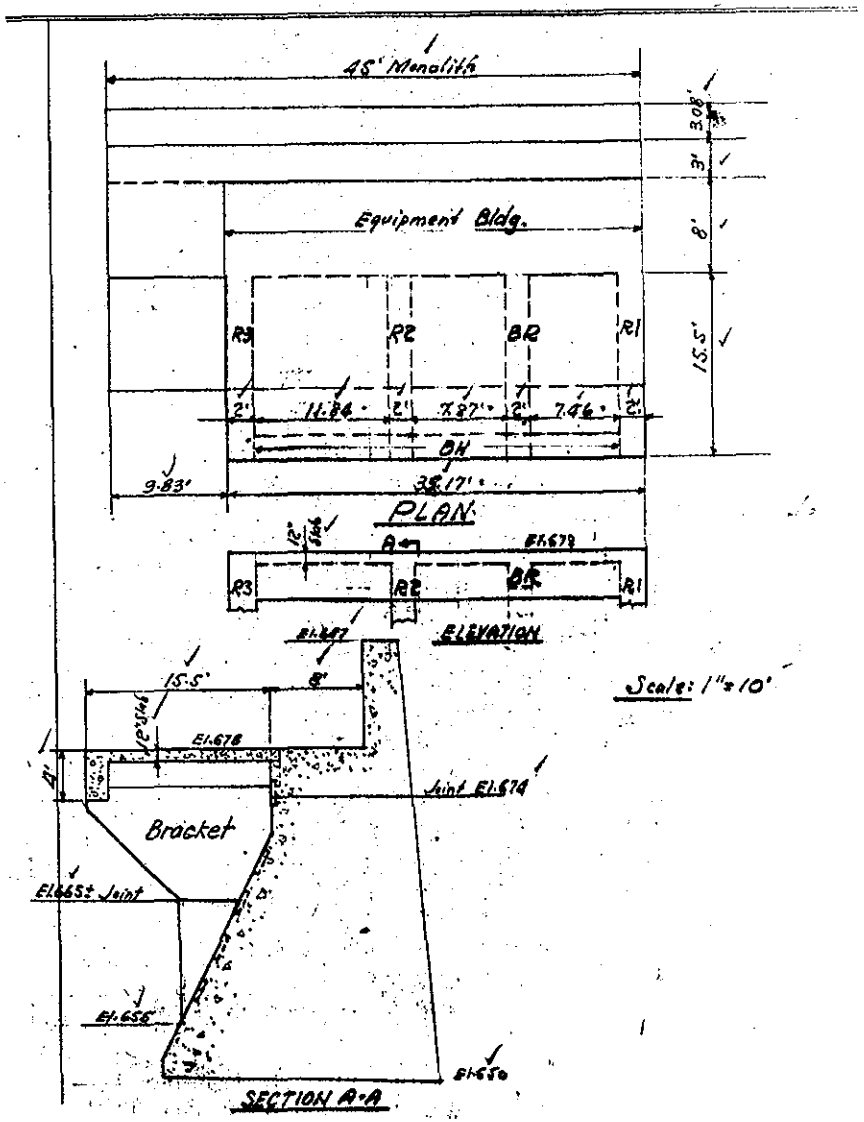
1/2" φ 12 oc Spacers

Plate VIII-71

CHAS. T. MAIN, INC. ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

Name: Bennington Dam
Subject: Equipment Bldg.
Job No. US-16-25 Sheet No. 59
Date Jan. 14 1946
Computed by R.W.B.
Checked by E.A.H.

Design of Brackets.



CHAS. T. MAIN, INC. ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

Name Bennington Dam — Job No. 15-16-15 — Sheet No. 60 — Plate VIII-72
Subject Equipment Bldg. — Date Jan. 15 1946
Computed by B.D.B. — Checked by E.A.H.

Design of Brackets.

Data: Wt. of a Rib: 55000#

Loading:

Roof: (D+L) @ 115 #/□' × area (34.25) × ½ = 49000#
Floor: " @ 250 × " (33.2 × 15.5) = 129000
Bsmt: " @ 300 × " (33.2 × 15.5) = 155000 — 1356000

Walls (Inside) 12" @ 150 × 8' × 9 = 10800
" " 150 × 14 × 9 = 18900
8" @ 100 × 8 × 9 = 7200
369900# ÷ (33.2 × 15.5) = say 700 #/□' ±

(a)

Weight of outside Walls carried by rib R_1 and R_3
13" Concrete @ 164# × 16' × 9' = 23600#
12" brick @ 120 × 16' × 14' = 27000
50600# - unif. spread over a Rib.

(b)

Weight of Front Outside Wall carried by Ribs & Beam.
13" Concrete @ 164 × 9 × 35.17 = 52000#
12" brick @ 120 × 14 × 35.17 = 59000
111,000# - Concentr. at end of Ribs (or Beam) proportionely.

(c)

Total Loads on Brackets and Beam.

(R3) Rf. + Fl. + Bsmt (700 #/□' × 15.5' × 7.92) = 86000#
Outside Wall 50000 } 136000# - unif. L.
Outside Wall of 111,000 × 7.92/35.17 = = 25000# - Conc. L. at end.
Header-Beam (3' × 2') × 150 = 900# × 5.92' = 3000# " " "

(R2) " — " 700 × 15.5 × 11.85 = } 129000# - unif. L.
Outside Wall of 111000 × 11.85/35.17 = = 37000# Con. L. at end.
Header Beam 900 × 11.85 = = 11000 " " "
Reaction from "Beam" BR = 53000 " " "

(R1) " — " 700 × 15.5 × 5.73 = 62000 }
Outside Wall = 50000 } 112000# - unif. L.
Outside Wall of 111000 × 5.73/35.17 = = 18000 - Con. L. at end
Header Beam 900 × 5.7 = = 3000 " " "
Reaction from "Beam" BR = 53000 " " "

CHAS. T. MAIN, INC. ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

Name Bennington Dam — Job No. 15-16-15 — Sheet No. 61 — Plate VIII-73
Subject Equipment Bldg. — Date Jan. 15 1946
Computed by B.D.B. — Checked by E.A.H.

Design of Brackets.

Beams.

BR 24" × 36" Span, use 15' c-c.

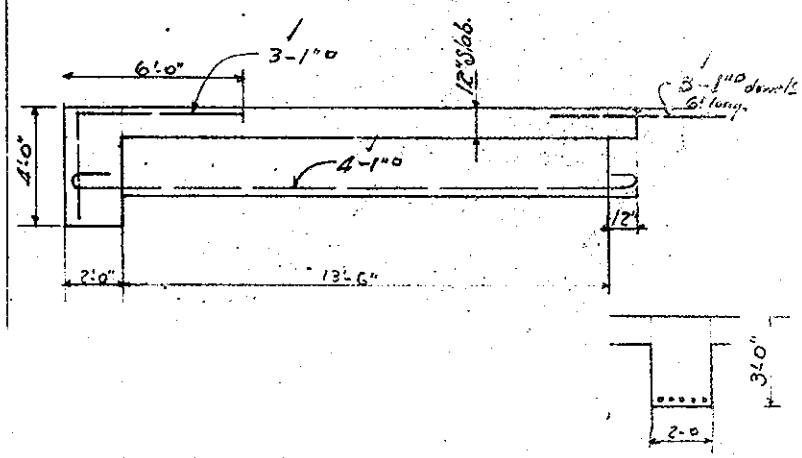
① Loading:
105000 ÷ 15.5 = 6800# / foot
Wt. Beam 450
7250#/unif.

② Moment: Coeff. 1/10
7250 × 15² × 1/10 = 163000#
$d = \sqrt{\frac{163000 \times 12}{169 \times 24}} = 22''$
use: d = 33"

④ Shear. Clear span 13.5'
V = 7250 × 6.75 = 48900# = $R_L = R_R$
$v = \frac{48900}{2 \times 0.88 \times 33} = 70$ #/□" > 60 O.K.

③ Reinforcing:
$a_s = \frac{163}{1.29 \times 33} = 3.8$ □"
Use: 4-1"□ Bars.

Special anchorage
No Web Reinf.



CHAS. T. MAIN, INC. ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

Name Bennington Dam — Plate VIII - 74
Subject Equipment Bldg. — Job No. US-16-75 Sheet No. 62
Date Jan. 15 1946
Computed by R.A.B.
Checked by E.A.H.

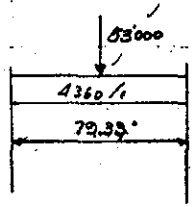
Design of Brackets
Beams.

Beam Mk. BH.

BH1 → 24" x 48" Span 17.33' clear
" 19.33' c-c.

Loading:
Outside Wall: 111000 ÷ 35.17 = 3160 #/foot of beam
Wt. Beam 1200
4360 #/foot - unif. L.

Conc. Load: Reaction from BR. = 53000# - conc. L.



① Moment Coeff. 1/10
4360 x 19.33² x 1/10 = 163000
53000 x 19.33 x 1/4 x 8/10 = 203000
366000 '#

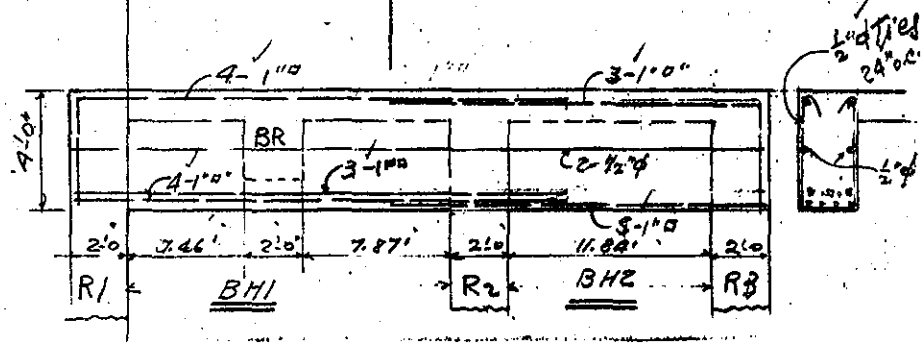
d = 45"

② Reinforcing:
a = 366 / 1.29 x 45 = 6.3 □"
use: 7-1" □

③ Shear.
4360 x 8.7 = 38000
53000 x 1/2 = 26500
64500 = V

v = 64500 / 24 x 0.88 x 45 = 68 O.K.

BH2
Min. Reinforcing req'd.:
b = 24" d = 45"
a_s = 0.005 x 24 x 45 = 5.4 □"
Use: 5-1" □ bott.
3-1" □ top.



CHAS. T. MAIN, INC. ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

Name Bennington Dam — Plate VIII - 75
Subject Equipment Bldg. — Job No. US-16-75 Sheet No. 63
Date Jan. 15 1946
Computed by R.A.B.
Checked by E.A.H.

Design of Brackets

Load on Beam Mk. BR. (continued see sheet 60)

BR
R.F. + Fl. + Bmt. (700 x 15.5 + 9.7) = 105000# - unif. L.
Outside Wall of 111000 x 9.7 / 35.17 = 30000# - Conc. L. at End.
Header Beam 900 x 9.7 = 9000 " "
Total 144000# $R_L = R_R$

Brkt. - Mk. R3

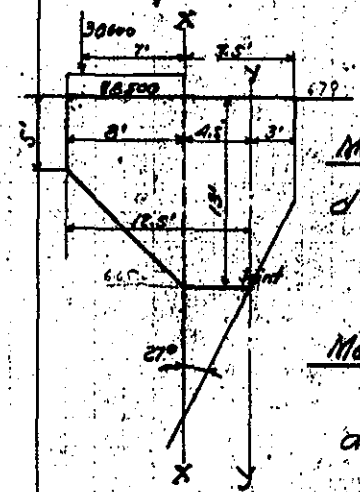
Shear Reinforcing

ΣV = Uni. L. 136000 + Front Wall C.L. 25000 + Header C.L. 5000 + Rib. Unif. 55000 = 221,000#

F = 221000 x Cos. 27°
= 221000 x 0.89 = 195000#

Dowels:
use: 1" φ, @ a = 0.79 □"
f_v = 16000

No. Req'd. = 195000 / 16000 x .79 = 16



Moment Reinforcing.

Loads: 25000 + 5000 = 30,000# Conc. L.
(136000 + 55000) x 8 / 15.5 = 98500# - unif.

Mom about X-X: 98500 x 4 = 394,000 '#
30000 x 7 = 210000
604000 '#

d = say 12.5" x 12 = 150"

a_s = 604 / 1.29 x 150 = 3.1 □" use: 3-1" □ (extended)

Mom. about Y-Y: 30000 x 11.5 = 345,000 '#
191000 x 4.75 = 906000
1,251,000 '#

d = say 150"

a_s = 1251 / 1.29 x 150 = 6.5 □" use: 7-1" □

Design of Brackets

Beams

B11

24" x 48" Depth 17.33" effective
19.33 c-c

Loading:

Outside Wall: 1100# = 25.17 = 3.16 kips/ft of beam
Wall Beam

Conc. Load: Reaction from B9 = 33000# - conc.
4200#/foot - uniformly L.

① Moment Coeff. $\frac{1}{10}$

4200 x 19.33² x $\frac{1}{10}$ = 169000'
43000 x 19.33 x $\frac{1}{10}$ = 83000
Total 252000

d = 45"

② Reinforcing:

$A_s = \frac{252}{1.29 \times 45}$ = 4.34

Use: 7-1"

③ Shear

4200 x 8.7' = 36500
33000 x $\frac{1}{2}$ = 16500
Total V = 52500

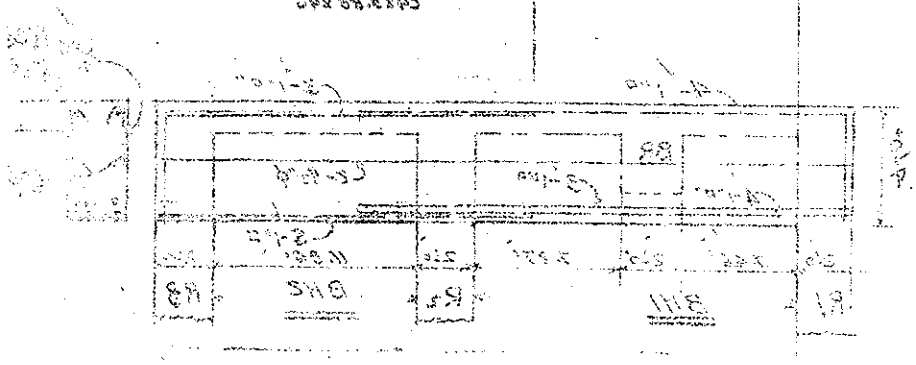
$v = \frac{52500}{24 \times 0.88 \times 45}$ = 68 O.K.

B12

Min. Reinforcing req'd:
b = 24" d = 45"

A_s = 0.005 x 24 x 45 = 5.4

Use: 5-1" bot.
3-1" top.



Design of Brackets

Beams

B12 (continued from sheet 60)

Outside Wall: 1100# x $\frac{8.7}{12.5}$ = 765#
Header Beam Dead 8.7' = $\frac{9000}{12.5}$ = 720#
Total 1485#

Shear - Reinforcing

ΣV = 1300# + 2500 + 5500 + 2500 = 22100#

Dowels:

use: 1" ϕ, A_s = 0.79"²
A_s = 1500

P_s = 22100 x cos 27°
= 21700 x 0.89 = 19300#

$A_s req'd = \frac{19300}{15000 \times .79}$ = 1.6

Moment Reinforcing

Load: 5500 x 5.5 = 30,000 ft-lbs.

$(1500 + 2500) \times \frac{5}{12.5}$ = 8500 - uniformly

Max. about X-X: 8500 x 4 = 34,000
30000 x 7 = 210000
244000

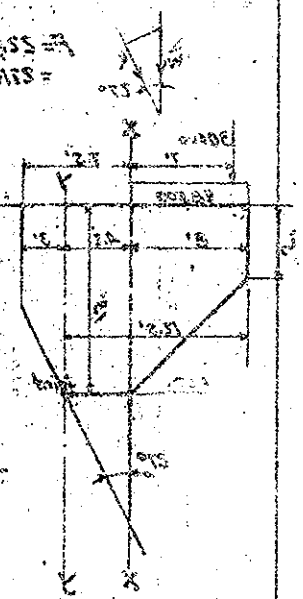
d = 19 (12.5/12 = 1.0)

$A_s = \frac{244}{1.29 \times 1.0}$ = 31 use: 3-1" (extension)

Max. about Y-Y: 30000 x 11.5 = 345000
1500 x 6.25 = 9300
354300

d = 19 - 1.0

$A_s = \frac{1251}{1.29 \times 1.0}$ = 6.5 use: 7-1"



CHAS. T. MAIN, INC. ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

Name: Bennington Dam — Job No. 15-16-15 — Sheet No. 64
Subject: Equipment Bldg. — Date: Jan. 15, 1946
Computed by: R.J.M.B.
Checked by:

Design of Brackets.

Brkt.-Mt. R2 — Shear Reinforcing.

$\Sigma V = \underset{\text{Unit. L.}}{134000} + \underset{\text{C.L.}}{37000} + \underset{\text{C.L.}}{11000} + \underset{\text{Rib Unit.}}{55000} + \underset{\text{C.L. Rec. Bl.}}{53000} = 285000^{\#}$

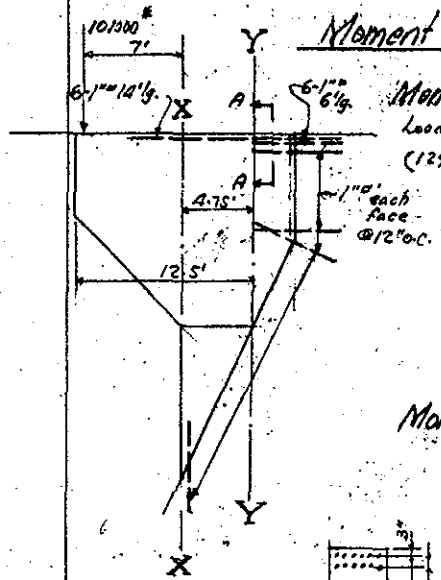
$F = 285000 \times \text{Cos. } 27^\circ$
$= 285000 \times 0.89 = 254000^{\#}$

Dowels:

use: 1" ϕ, a = 0.79 □"
$f_v = 16000$

No. req'd: $\frac{254000}{16000 \times 0.79} = 20$ rods

Moment Reinforcing.



Mom. about X-X:

Loads: 37000 + 11000 + 53000 = 101,000# C.L.
$(129000 + 55000) \times \frac{8}{15.5} = 95000$ - Uni. L.

$M = 95000 \times 4 = 380,000$
$101000 \times 7 = 707000$
$1,087,000'^{\#}$

$d = \text{say} = 150''$

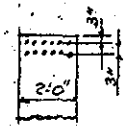
$a_s = \frac{1,087,000}{1.29 \times 150} = 5.6\,\square''$ req'd.
use: 6-1□"

Mom. about Y-Y:

$M = 101000 \times 11.5 = 1,160,000$
$184000 \times 4.75 = 875,000$
$2,035,000$

$d = \text{say } 12.5' = 150''$

$a_s = \frac{2035}{1.29 \times 150} = 10.6\,\square''$
use: 10-1"□



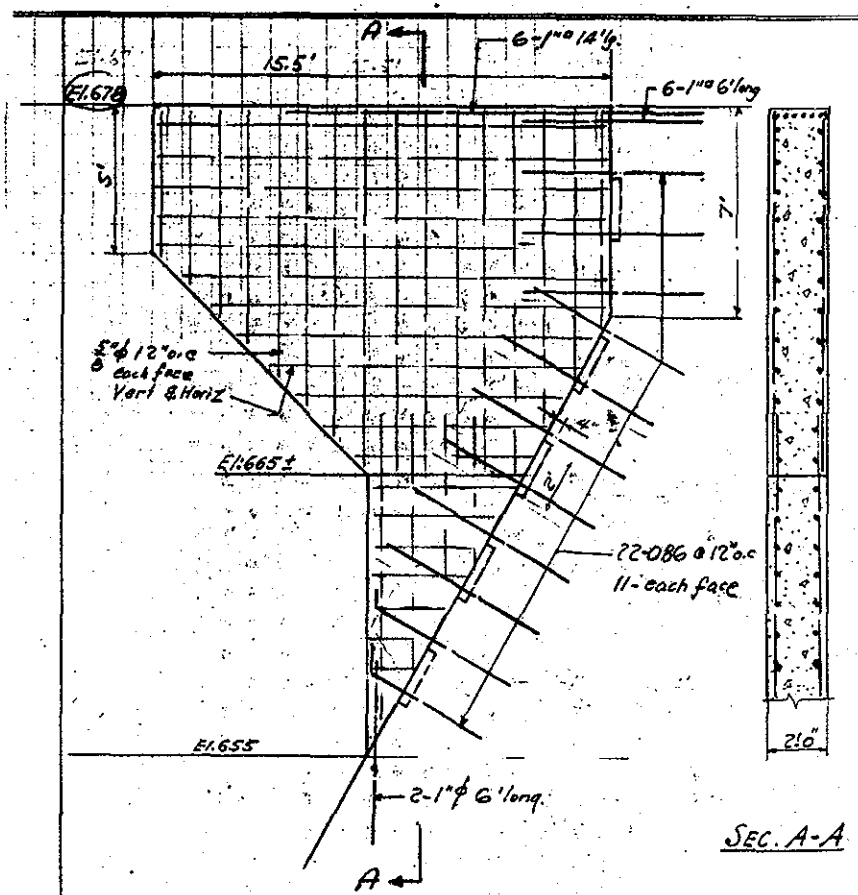
SEC. A-A

Note: Reinforcing for R1 and R3 - use same as R2.

CHAS. T. MAIN, INC. ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

Name: Bennington Dam — Job No. 15-16-15 — Sheet No. 65
Subject: Equipment Bldg. — Date: Jan. 15, 1946
Computed by: R.J.M.B.
Checked by:

Design of Brackets.



SEC. A-A

ELEVATION.

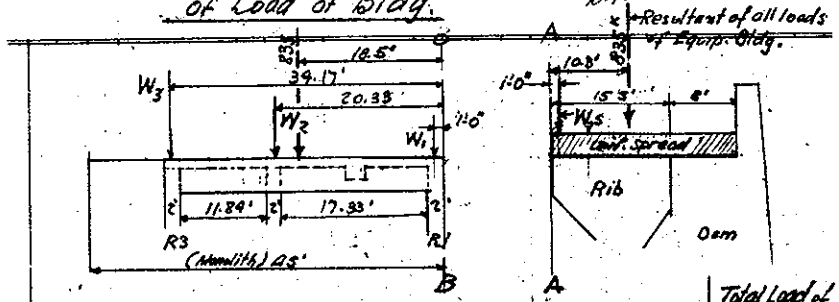
Scale: $\frac{1}{4}" = 1'-0"$

Plate VIII - 78

CHAS. T. MAIN, INC. ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

Name: Bennington Dam
Subject: Equipment Bldg.
Location of Resultant of Load of Bldg.

Job No. US-16-15 Sheet No. 66
Date: Jan. 16 1946
Computed by: B.&B.
Checked by: C.H.



Location of Resultant from Line A-A

	Conc. L.-W.	Uni. L.-W. Deduct Wt. of Rib		Total Load of Equip. Bldg. designated as Unif. spread.
Brkt.-Mk. R1.	94,000	167000 - 55000 = 112,000 × 23.5/15.5	=	170,000
" " R2	101,000	184000 - 55000 = 129000 × "	=	195,000
" " R3	30000	191000 - 55000 = 136000 × "	=	206,000
Total Conc. Load: →	205,000	Total Load of Unif. spread Loads, supported by Ribs	= 377,000	571,000#

Weight of 12" Brick Wall in back of Bldg. - 59000#

Conc. L.	205000 × 1'	=	205000
	571000 × 11.8"	=	6,750,000
	59000 × 26.82'	=	1,530,000
Weight of Bldg:	835000#		8,485,000

Distance of Resultant from Line A-A.
8,485,000 ÷ 835000 = 10.3'.

Location of Resultant from Line B-B.

R1 - W_1 = (74000 + 170000) × 1' = 244,000
R2 - W_2 = (101000 + 195000) × 20.33 = 6,050,000
R3 - W_3 = (30000 + 206000) × 39.17 = 8,100,000
776,000 14,394,000
Brick Wall = 59000 × 17.6 = 1,035,000
ΣV 835,000# 15,429,000'#

Distance of R from Line B-B = 18.5'.
∴ e = 18.5' - 39.17/2 = 0.9'

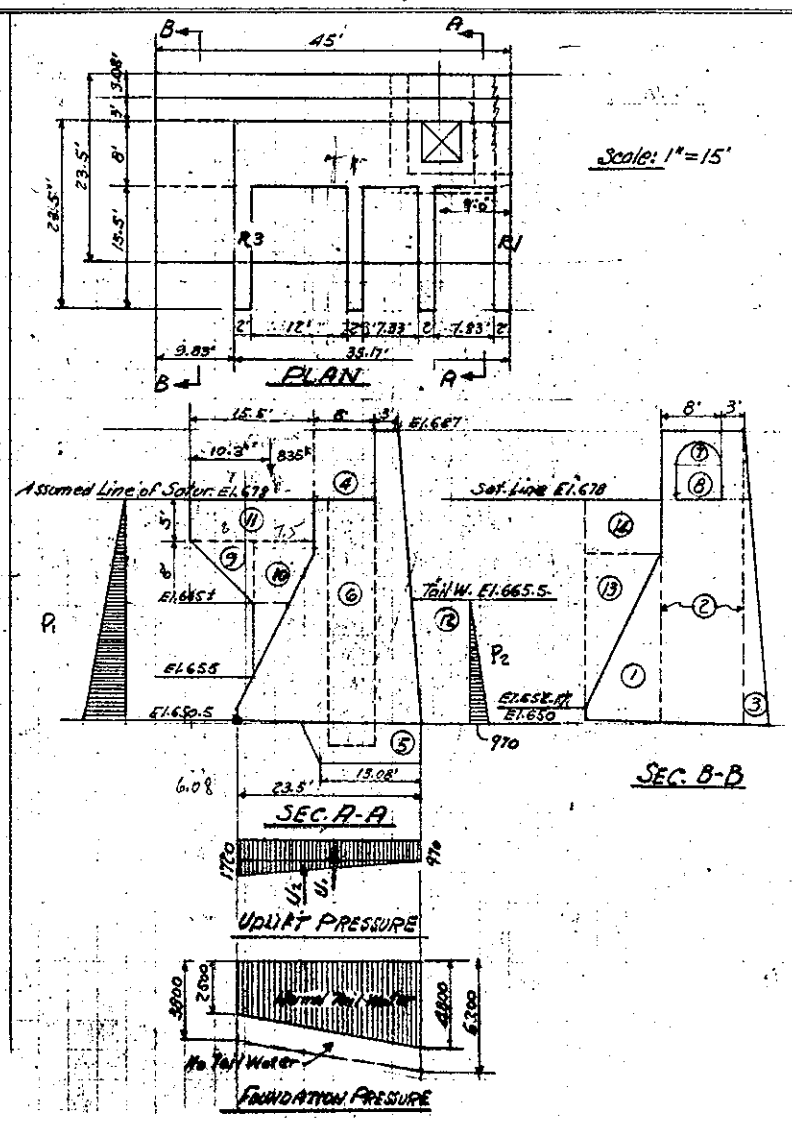
Location of R (Wt. of Bldg.) with respect to Monolith's Base: and eccentricity ∴ e = 45/2 - 18.5 = 4'

Plate VIII - 79

CHAS. T. MAIN, INC. ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

Name: Bennington Dam
Subject: Discharge Channel Wall - East Monolith with Equipment Bldg.
Stability Analysis

Job No. US-16-15 Sheet No. 67
Date: Jan. 17 1946
Computed by: B.&B.
Checked by: C.H.



WAR DEPARTMENT
U. S. ENGINEER OFFICE, BOSTON, MASS.

Subject: Bennington Dam
Computation: Monolith with Equipment Bldg
Computed by: EPR Checked by: Date: 12 April 1946

Investigation with assumed saturation line behind wall at El. 625.0
Earth surface at El. 686.0
See C.T. Malo computation sheet No. 68

	V		M
Concrete, Earth & Water	5282		54841
Uplift 970# × 23.5 × 45.0 =	−1030	×11.75 =	−12100
560# × 1/2 × 23.5 × 45.0 =	− 295	×7.88 =	− 2300
	−1325		
ΣV	3957		40441

$P_1 = \frac{1}{2} \times 62.5^{\#} \times \overline{84.0}^2 \times 45.0$ =	840.0	×8.17 =	6900
P_2 =	786.0	×12.8 =	8950
$P_3 = \frac{1}{2} \times 62.5 \times \overline{15.8}^2 \times 45.0$ =	− 338.0	×4.64 =	−1580
ΣH =	1292	ΣM =	54711

Location of Resultant

Normal Tailwater

$\frac{\Sigma M}{\Sigma V} = \frac{54711}{3957} = 13.8$ ft. from heel $e = 13.8 - 11.75 = 2.05'$

$f = \frac{3957}{23.5 \times 45}\left(1 \pm \frac{6 \times 2.05}{23.5}\right) = 3.74(1 \pm 0.52)$

$= 5.7^K$ (Toe) & 1.89^K (Heel)

Factor of Safety against Sliding

$\Sigma V = 3957^K$ Ave. Vert. Load $= 3957/23.5 \times 45 \times 2 = 1.87$ T/s.f.

Shear strength of soil = 1.7 T/s.f.

Ave. H = 1292/23.5 × 45 × 2 = 0.57 T/s.f.

Factor = 1.7/0.57 = 3.0

WAR DEPARTMENT
U. S. ENGINEER OFFICE, BOSTON, MASS.

Subject: Bennington Dam
Computation: Monolith with Equipment Building
Computed by: EPR Checked by: Date: 15 April 1946

No Tailwater

	V		M
Concrete	3518		47261
Earth	901		8410
Equipment Bldg	885		3520
Gross Vert.	5254		34191
Uplift −1530 × 1/2 × 23.5 × 45 =	− 810	×7.88 =	− 6350
ΣV	4444		47841

$P_1 = \frac{1}{2} \times 62.5^{\#} \times \overline{84.0}^2 \times 45$ =	840.0	×8.17 =	6900
P_2 =	786.0	×12.8 =	8950
ΣH =	1540.0	ΣM =	63691

Location of Resultant

$\frac{\Sigma M}{\Sigma V} = \frac{63691}{4444} = 14.35'$ from heel

$e = 14.35 - 11.75 = 2.55'$

$f = \frac{4444}{23.5 \times 45}\left(1 \pm \frac{6 \times 2.55}{23.5}\right) = 4.2(1 \pm 0.68)$

$= 6.9^K$ (Toe) & 1.47^K (Heel)

Factor of Safety against Sliding

$\Sigma V = 4444^K$ Ave. Vert. Load $= 4444/23.5 \times 45 \times 2 = 2.1$ T/s.f.

Shear strength of soil = 1.7 T/s.f.

Average H = 1540.0/23.5 × 45 × 2 = 0.73 T/s.f.

Factor = 1.7/0.73 = 2.4

Plate VIII - B2

CHAS. T. MAIN, INC. ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

Name: Bennington Dam — Job No. 45-16-15 Sheet No. 68
Subject: Discharge Channel Wall - East — Date: Jan. 17 19 46
Monolith with Equipment Bldg. — Computed by: B.A.B.
Stability Analysis — Checked by: E.A.H.
Moment abt. Toe Edge

Description		Dimensions	Volume Cu. Ft.	Wt. (Cu. Ft.)	Force Kips	Arm Ft.	Mom. Ft. K.
Concrete	①	$\frac{1}{2}\times 9.42\times 20.5\times 45$	4340	150	650	6.28	4080
	②	$11\times 37\times 45$	18800	"	2740	14.92	40900
	③	$\frac{1}{2}\times 3.08\times 37\times 45$	2560	"	384	21.45	8160
	⑤	see Sheet 36		"	137		2600
Gross:					3911		55740
Voids	④	$8\times 9\times 35.17$	2530	"	− 380	16.42	− 6250
	⑥	$5\times 30.92\times 5$	744	"	− 116	14.92	− 1730
	⑦	$\frac{1}{2}\times\pi\times 3^2\times 9.83$	139	"	− 21	14.42	− 300
	⑧	$4.5\times 6\times 9.83$	266	"	− 40	14.42	− 580
					− 557		− 8860
Concrete Net:					3354		46880
Ribs R_1-R_2-R_3	⑨	$\frac{1}{2}\times 8^2\times 2\times 3$	192	150	29	0.75	− 22
	⑩	$\frac{1}{2}\times 18\times 8\times 2\times 3$	432	"	65	4.42	286
	⑪	$5\times 15.5\times 2\times 3$	465	"	70	1.67	117
					164		+ 381
Total Concrete (Dam + Ribs)					3518		47260
Wt. Earth	⑬	assume (45'-6') $\frac{1}{2}\times 9.42\times 20.5\times 39$	3760	142.5	535	9.14	4890
	⑭	$9.42\times 7\times 39$	2570	"	366	4.71	1730
Wt. Water	⑫	$\frac{1}{2}\times 15.5\times 1.29\times 45$	450	62.5	28	23.07	650
Wt. Equip. Bldg. W = 835					835	4.22	+ 3520
Total ⑬⑭⑫ ΣW					1764		+ 7580
Gross Vert.					5282		54840
Uplift	U_1	$970\times 28.5\times 45$			− 1030	11.75	− 12100
	U_2	$\frac{1}{2}\times 750\times 23.5\times 45$			− 396	7.83	− 3110
Total Uplift					− 1426		− 15210
Total Vert.				ΣV =	3856		39630
Hor. Earth	P_1	$\frac{1}{2}\times 80\times 27.5^2\times 45$			1360	9.17	12350
Hor. Water	P_2	$\frac{1}{2}\times 62.5\times 18.8^2\times 45$			− 338	4.64	− 1580
				ΣH =	1022		10770
						ΣM =	50401

Plate VIII - B3

CHAS. T. MAIN, INC. ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

Name: Bennington Dam — Job No. 45-16-15 Sheet No. 69
Subject: Discharge Channel Wall - East — Date: January 28 19 45
Monolith with Equipment Bldg. — Computed by: Herzog
Stability Analysis — Checked by: B.A.B.

① About Toe edge: Location of Resultants.

From Sheet 60:

Reservoir Full

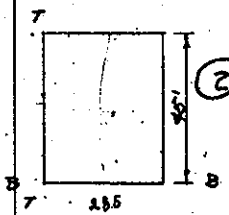
Total Moment, monolith + ribs + Bldg = 50401 'k

Total Vert. " " " = 3856

Resultant = $\frac{50401}{3856}$ = 13.1 — 11.8

$e_{T.T}$ = 1.3'

$\frac{11.8}{23.5}$ / 2



② About BB. **Investigation.**

Consider Monolith center of gravity = 22.5' on TT

Reservoir Full	V	Arm	M
Monolith	3354		75980
Earth	900	23.75	21400
Water	22	22.5	630
	4276		98010
Uplift	−1426	22.5	− 32000
	2850		66010
Ribs	176		3260
Eq. House.	835	18.5	15500
	3866		84270

R = $\frac{84270}{3866}$ = 22.0

e = 22.5 − 22.0 = 0.5'

Note: Eccentricity of Resultant about C of G. of base is such that investigation of foundation pressure about axis BB is not necessary.

CHAS. T. MAIN, INC. ENG'RS
201 DEVONSHIRE ST.
BOSTON, MASS.

Plate VIII - 84

Name: Bennington Dam — Job No. US-16-IS — Sheet No. 70
Subject: Discharge Channel Wall - East — Date: Jan. 18, 1946
Monolith with Equipment Bldg. — Computed by: B.A.R.
Stability Analysis — Checked by: E.A.H.
Mom. about Toe Edge.

Location of Resultants.

I. Normal Tail Water. (see Sheet 68).

$\Sigma V = 3856^{\#}$ $\Sigma M = 50401^{'\#}$ $2/3 \times 23.5 = 15.7'$

$d = \frac{\Sigma M}{\Sigma V} = 13'$ R falls 2.7' inside 3rd pt.

$\therefore e = 13' - \frac{23.5}{2} = 1.25'$

II. No Tail Water.

Total Vert. Forces (see Sheet 68) 5282^{k} − 28 = 5254^{k}; 54841 − 650 = 54191

Hor. Force add P, + 12350 = $66541^{'k}$

$\Sigma V = 5254^{k}$ $\Sigma M = 66540^{'k}$

$d = 12.7'$ R falls .3' inside 3rd pt.

$\therefore e = 12.7 - 11.75 = 0.95'$

Foundation Pressures.

I. Normal Tail Water.

$f = \frac{3856}{23.5 \times 45}\left(1 \pm \frac{6 \times 1.25}{23.5}\right)$

$3.66 \quad (1 \pm 0.32)$ } 4.8^{k}, 2.5^{k}

II. No Tail Water.

$f = \frac{5254}{23.5 \times 45}\left(1 \pm \frac{6 \times 0.35}{23.5}\right)$

$4.98 \quad (1 \pm 0.24)$ } 6.2^{k}, 3.8^{k}

Factor of Safety against Sliding:

$\Sigma V = 3856^{k}$

Ave. Vert. Load: $\frac{3856}{(23.5 \times 70)2} = 1.18$ T/□'

From diagram (sheet 1-C) Shear Strength of soil: 1.7 T/□'

Aver. Hor. Force: $\frac{1022}{(23.5 \times 70) \times 2} = 0.31$ T/□'

Factor: $\frac{1.7}{0.31} = 5.5$ OK.

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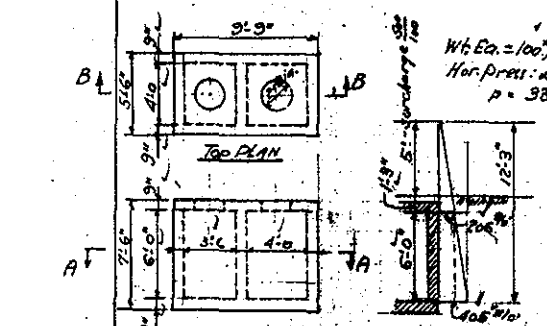
BOSTON, MASS.

Plate VIII - 85

Name: Bennington Dam — Job No. US-16-IS — Sheet No. 71
Subject: Duct Line — Date: Jan. 16, 1946
Computed by: B.A.R.
Checked by: E.A.H.

Manhole.
Reqd - 3



Data: Wheel Load - use 8 t.m.
Surcharge - use 300#
Stresses: $f_s = 18000$, $f'_c = 3000$, $f_c = 1050$, $n = 10$, $K = .169$, $j = 0.88$, $a = 12$

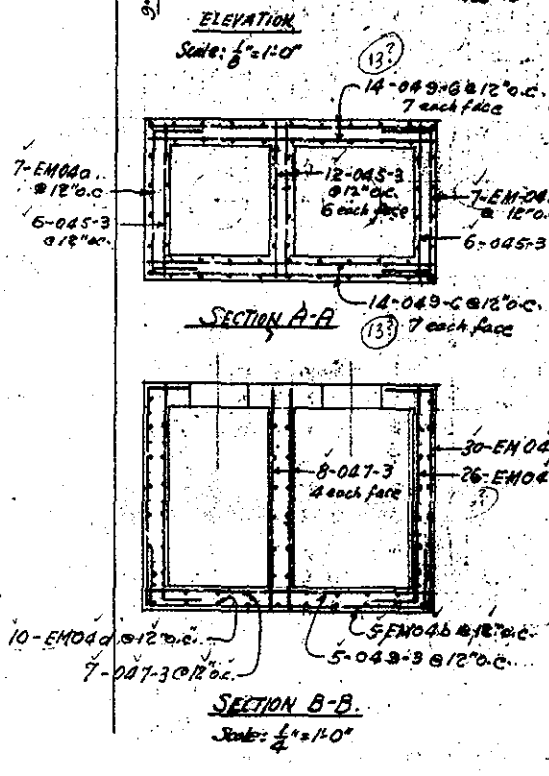
Wall Design.

End of Wall partially restrained.
Use Mom. Coeff. 1/10 for pos. & neg.

$M_{pos.}$ (max.) $= 206 \times 6^2 \times \frac{1}{10} = 740'$
$0.1283 \times (200 \times 6 \times \frac{6}{2}) \times 6 \times \frac{8}{10} = 370'$
(approx.) $1110'^{\#}$

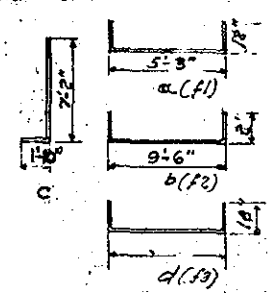
$d = \sqrt{\frac{1110}{169}} = 2.6''$ req'd.
use: $d = 7''$

$a_s = \frac{111}{1.29 \times 7} = 0.12''$



Reinforcing.

$a_{min} = 0.005 \times 12 \times 7 = 0.42''$ / foot
$\therefore$ 0.21" each face
use: ½"φ 12" o.c. each face Vert. & Hor.
use same Reinf. for all walls.



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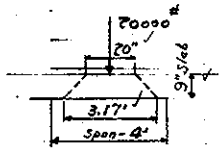
Name Bennington Dam
Subject Duct Line
Manhole
Req'd - 3

Plate VII - 86
Job No. US-16-25 Sheet No. 72
Date Jan. 16 1946
Computed by RAB.
Checked by E.A.H.

Top Slab Design.

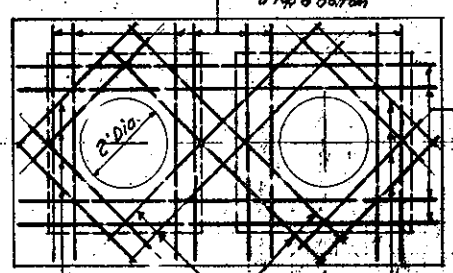


Loading: Wheel Load - 8 tons
Impact 25% - 2'
T. Load 10 tons
(spread over slab accordingly to specification) ∴ 1 ton per 1"

Assume Load uniformly distributed over 4' span ∴ Moment: $20000 \times 4 \times \frac{1}{10} = 8000$ '#

$d = \sqrt{\frac{8000}{169}} = 6.7''$ use: $d = 7''$

$a_s = \frac{8}{1.29 \times 7} = 0.89$ □" use: 2-3/4" ϕ.



PLAN
Scale: 1/4" = 1'0"

Bottom Slab.

Loads:
Wt. of Chamber $\left\{ (9.75 \times 7.25) \times 2 = 146^{\#} \right.$
Walls: $(9 \times 7.5) \times 2 = 120$
Slabs $(3.5 + 9) \times 4 \times 2 = 60$
326#

Wt. $(326 \times 0.75) \div 150 = 1.6$ cu ft
Wheel Load 20000
56000# ÷ (9.75 × 7.25)
Load on Base = 1050 #/□' uniform.

Mom.: $1000 \times 4^2 \times \frac{1}{10} = 1600$ '#

$d = \sqrt{\frac{1600}{169}} = 3''$ use $d = 7''$

$a_s = \frac{1.52}{1.29 \times 7} = 0.17$ □"
1/2" ϕ @ 12" o.k.