HOUSATONIC RIVER FLOOD CONTROL

HALL MEADOW BROOK
DAM & RESERVOIR

HALL MEADOW BROOK

(UPPER NAUGATUCK RIVER, ABOVE TORRINGTON)

CONNECTICUT

DESIGN MEMORANDUM NO. VI

DETAILED DESIGN OF STRUCTURES

U.S. Army Engineer Division, New England

Corps of Engineers Waltham, Mass.

JUNE 1960

U. S. ARMY ENGINEER DIVISION, NEW ENGLAND
CORPS OF ENGINEERS
424 TRAPELO ROAD
WALTHAM 54, MASS.

ADDRESS REPLY TO:
DIVISION ENGINEER

REFER TO FILE NO. NEDGW

14 June 1960

SUBJECT: Hall Meadow Brook Dam and Reservoir, Hall Meadow Brook, Housatonic River Basin, Connecticut - Design Memorandum No. VI - Detailed Design of Structures

TO: Chief of Engineers
Department of the Army
Washington, D. C.
ATTENTION: ENGCW-E

There are submitted for review and approval ten (10) copies of Design Memorandum No. VI - Detailed Design of Structures, for the Hall Meadow Brook Dam and Reservoir, Hall Meadow Brook, Housatonic River Basin, Connecticut, in accordance with EM 1110-2-1150.

FOR THE DIVISION ENGINEER:

J. C. Dingwall
for JOHN WM. LESLIE
Chief, Engineering Division

Incl.
Des. Memo No. VI-
Detailed Design of Structures
(10 cys)

FLOOD CONTROL PROJECT

HALL MEADOW BROOK DAM AND RESERVOIR

HALL MEADOW BROOK

HOUSATONIC RIVER BASIN

CONNECTICUT

DESIGN MEMORANDA INDEX

Number	Title	Submission Date	Approved
I	Hydrology & Hydraulic Analysis		
	Preliminary	11 Mar 1960*	6 April 1960
	Final	9 May 1960	31 May 1960
II	Site Geology	9 May 1960	31 May 1960
III	General Design	15 Apr 1960	25 May 1960
IV	Concrete Materials	1 Apr 1960	13 Apr 1960
V	Embankment & Foundations		
VI	Detailed Design of Structures	14 June 1960	

* Initial submission in draft to secure approval of spillway design flood and outlet requirements

HALL MEADOW DAM AND RESERVOIR

HALL MEADOW BROOK

HOUSATONIC RIVER BASIN

CONNECTICUT

DESIGN MEMORANDUM NO. VI

DETAILED DESIGN OF STRUCTURES

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U. S. ARMY ENGINEER DIVISION, NEW ENGLAND
OFFICE OF THE DIVISION ENGINEER
WALTHAM 54, MASS.

FLOOD CONTROL PROJECT

HALL MEADOW BROOK DAM AND RESERVOIR

HALL MEADOW BROOK
HOUSATONIC RIVER BASIN
CONNECTICUT

DESIGN MEMORANDUM NO. VI

DETAILED DESIGN OF STRUCTURES

June 1960

A. INTRODUCTION

1. Purpose. - The purpose of this memorandum is to facilitate the review by higher authority of the structural design of the various features of the project. The basic criteria, typical design computations, and other data pertinent to the design are presented herein.

2. Scope. - This memorandum covers the following structures: outlet works, the main spillway, lining and retaining walls, the Reuben Hart diversion spillway and auxiliary spillway and the modifications to the existing Reuben Hart spillway.

3. Previous Reports. - No previous report on the structural design for these structures has been submitted. The latest previous description of the proposed structures and improvement is set forth as part of the recommended project plan in the Design Memorandum No. III - General Design, submitted on 15 April 1960 and approved on 25 May 1960.

4. Location of Project. - The Hall Meadow Brook Reservoir Project is located within the City of Torrington and the Town of Goshen. The reservoir is formed by a dam located on Hall Meadow Brook, 0.4 mile above its confluence with Hart Brook to form the West Branch of the Naugatuck River and about 5 miles above Torrington, a dike across a saddle east of the dam and a spillway in the West abutment of the dike; and the existing Reuben Hart Dam on Hart Brook west of Hall Meadow Brook and a diversion canal with a spillway connecting the Reuben Hart Reservoir with the Hall Meadow Brook Reservoir. All structures are in the city of Torrington.

5. Description. - a. General. - A description of each of the principal elements of the proposed plan of improvement for the Hall Meadow Brook Reservoir Project is presented in the following paragraphs. See General Plan, Plate No. 1.

b. Dam. - The dam to be constructed on Hall Meadow Brook is composed of a rock and rolled earth fill embankment 1200 feet in length and having a maximum height of 73 feet above stream bed. The top elevation is 917 feet above m.s.l. to provide for a 14.1 foot spillway surcharge and 4.9 foot freeboard. The top width of the dam is 20 feet, with a 14-foot gravel surface roadway. Highway guard rails will not be provided since access will be limited to official use only. The upstream dam slope is 1 on 2 from top of dam to elevation 898.0, 1 on 2.5 from elevation 898.0 to elevation 864.0 where a 10-foot berm is provided for access to the inlet structure and 1 on 2 below elevation 864.0. The downstream slope is 1 on 2. The embankments consist of an upstream and downstream zone of quarry-run type rock on gravel bedding. A large rock zone is provided on the downstream toe which will be carried to rock surface. A rock filter zone is provided below elevation 875.0 as a transition between the gravel bedding and the downstream rock fill. The earth section of the embankment consists of compacted silty sand (sandy till) with either an upstream more impervious zone or a downstream pervious zone. Two schemes for controlling the seepage through the foundation are presently being considered; one scheme consists of an upstream impervious blanket and a 5-foot deep foundation cut-off, and the other scheme consists of a foundation cut-off to rock with grout curtain into rock between Station 2+30± to Station 12+10±. Only nominal stripping is required under the embankment. Access to top of dam is from the relocated Route 72 on the right abutment. Materials to be used in the embankment are obtained from required excavations and from nearby borrow areas. Final embankment sections will be presented in the Design Memorandum on Embankments and Foundations.

c. Outlet Works. - The outlet works are located on the right bank of the brook under the dam and founded on rock. It consists of an inlet channel, an inlet structure, a conduit, and an outlet channel.

(1) The inlet channel is excavated in earth and is 170 feet long with a bottom width of 12 feet and side slopes of 1 on 2. The invert elevation varies from elevation 850 to elevation 849.

(2) The inlet structure founded on rock contains a reinforced concrete control weir with crest at elevation 859. Stoplogs are provided to control and operate the permanent pool. A 3' x 4' manually operated gate is also provided upstream of the structure, for the flow of the stream during construction and before a permanent

pool is established and to operate and dewater the permanent pool. Trash rack will also be provided.

(3) The 48-inch diameter conduit constructed on rock under the dam has a total length (including transition) of approximately 313 feet with an upstream invert at elevation 842 and an invert at the outlet portal at elevation 841. The conduit is reinforced concrete. A grout ring is provided in line of the foundation cut-off and three seepage collars will also be provided.

(4) The outlet channel excavated in rock and earth is 14 feet wide and approximately 210 feet long having a .5 of 1 percent slope from elevation 840 to the brook. Downstream of the conduit portal for a length of 25 feet a reinforced concrete structure is provided for the transition. A stilling basin is not considered necessary. See Plate Nos. 2, 3 and 4.

d. Dike. - The dike is to be constructed on a saddle east of the dam and consists of a rolled fill embankment with a length of 1,350 feet and a maximum height of 47 feet. The top elevation is 917 feet, m.s.l., which provides for a 14.1 foot surcharge and a 4.9 foot freeboard. The top width of the dike is 12 feet with an 8-foot gravel surface roadway. Highway guard railings will not be provided. The reservoir and landside slopes are 1 on 2.5. The slopes will be protected by rock slope protection overlying gravel bedding. The main embankment will consist of compacted silty sand with either an upstream more impervious zone or a downstream pervious zone or inclined pervious wick. Neither a foundation nor an exploratory trench is considered necessary at this time. Slightly more than nominal stripping is required under the embankment at the lower ground elevation. A waste ballast has been provided on the landside toe to aid and control foundation seepage and improve the stability at the embankment. The right abutment will be separated from the spillway channel by a concrete retaining wall. Materials to be used in the embankment are obtained from required excavations and from nearby borrow areas. Final embankment sections will be presented in the Design Memorandum on Embankments and Foundations.

e. Spillway. - The spillway is located adjacent to the right abutment of the dike and separated from the embankment by a concrete retaining wall. The spillway is an uncontrolled, fixed-crest, trapezoidal weir with a crest length of 100 feet at elevation 898.0 feet, m.s.l. The weir will be designed as a gravity ogee section founded on rock and, if required, an upstream grout curtain will be provided under the weir to minimize uplift. The structure has a maximum discharge capacity of 19,200 c.f.s. (the outflow for the spillway design flood)under the design surcharge of 14.1 feet. Flood discharges over the structure will occur infrequently and no improvement is planned in the valley immediately below the spillway discharge channel. Flowage easement will be acquired for a distance of 3/4 mile.

The spillway approach channel excavated in rock and earth is about 450 feet long. It has a maximum invert elevation of 893 at the weir. The bottom elevation slopes down from the weir at 1 percent grade for about 200 feet where the slope changes to .5 of 1 percent. The spillway chute or discharge channel excavated in rock and earth is about 1320 long. It slopes down at a 6 percent grade from invert elevation 892 to invert elevation 866 and thence the slope changes to .5 of 1 percent. The discharge channel bottom width varies from 97.5 feet at the weir bucket to 75 feet in a distance of about 412 feet. The excavated materials from the spillway are used in the proposed embankments as rock and random fills. Excavation operations will proceed at a rate that will allow the excavated materials to be placed in the embankments with minimum stockpiling. See Plate No. 5.

f. Reuben Hart Diversion. - The diversion canal from existing Reuben Hart water supply reservoir is excavated in rock and earth and consists of an approach channel and auxiliary spillway, a spillway, a discharge channel and modification to the existing Reuben Hart spillway.

(1) The approach channel is about 300 feet long and 122.5 feet wide with a bottom elevation of 906.0 feet, m.s.l. The north wall is wholly contained in rock. The south wall is contained by a retaining wall which ties on the westerly side into the existing retaining wall of the Reuben Hart Reservoir and on the easterly side it ties into a concrete cap located on top of the rock excavation face, which will be lined with concrete to minimize loss of water through the rock joints. The top of the wall and cap is set at elevation 914.5 feet, m.s.l. and together with the modified Reuben Hart spillway will act as an auxiliary spillway to pass flows higher than the standard project flood. The overall length of the auxiliary spillway is about 207 feet.

(2) The spillway is an uncontrolled, fixed-crest, trapezoidal weir with a crest length of 125 feet at elevation 911.0 feet, m.s.l. (the same as the existing Reuben Hart spillway). The weir will be designed as a gravity ogee section founded on rock and, if required, anchors and an upstream grout curtain will be provided. The structure has the discharge capacity of 2900 c.f.s. for the standard project flood under the design head of 3.5 feet, and has a maximum discharge capacity of 6100 c.f.s. for the spillway design flood under the design surcharge of 5.4 feet.

(3) The discharge channel varies from a width of 122.5 feet and a bottom elevation of 905.0 feet, m.s.l., to a width of 40.0 feet and a bottom elevation of 898.0 feet, m.s.l., in an

average length of 310 feet. From this point, the channel has a constant width of 40 feet and a slope of 2 feet in 100 feet for an approximate length of 330 feet and thence the channel is excavated to rock and follows the natural rock slope to elevation 858.0 feet, m.s.l. and thence discharges onto the permanent pool.

(4) The existing concrete weir of the Reuben Hart spillway is 55 feet wide with a crest elevation of 911.0 feet m.s.l. It will be modified by raising the crest to elevation 914.5 feet, m.s.l. At this crest elevation it will act as an auxiliary spillway to pass flows larger than the standard project flood.

(5) Excavation operations for the diversion canal will proceed at a rate that will allow the excavated materials to be placed in the permanent structures with minimum stockpiling. The relocated Route 72 crosses the canal and this will necessitate the construction of an 80-foot span bridge. See Plate Nos. 6, 7 and 9.

g. Existing Reuben Hart Dam. - The existing Reuben Hart Dam constructed in 1932 is owned and operated by the Torrington Water Company; is an earth fill dam about 1,000 feet long with a maximum height of 55 feet above the stream bed. The dam has a top width of 15 feet with a top elevation of 917.0 feet above m.s.l. and upstream and downstream slopes of 1 on 2. The upstream slope is protected with rock riprap and the downstream slope is protected with seeded topsoil. The embankment is homogeneous with a central concrete core wall to rock having a top elevation of 908.0 feet above m.s.l. At the left abutment the embankment is separated from the spillway concrete weir and the discharge channel by a concrete retaining wall. The uncontrolled concrete weir founded on rock is 55 feet long and has a crest elevation of 911.0 feet above m.s.l. After the floods of 1955, an additional 28 feet of spillway has been provided to the left of the concrete weir and separated from the latter by a concrete retaining wall. This addition is a broad crest type spillway constructed of rock blocks and having a crest elevation which varies from elevation 911.7 to 912.7 feet above m.s.l.

h. Administrative Facilities and Utilities. - No administrative facilities or utilities are provided. Maintenance and operation will be accomplished by the State of Connecticut which is expected to use a small mobile group operating from a separate headquarters.

i. Access Roads. - The site is located on State Route 72, which will be relocated. The reconstructed Route 72 will be adjacent to the right end of the dam and will serve as the main access road. Access will be limited for official use only.

Access to the reservoir area will be via existing Route 72 which will be provided with access to the relocated Route 72.

B. HYDROLOGY

6. General. - The Design Memorandum No. 1, Hydrology and Hydraulic Analysis, includes the basic data and hydrological requirements for the main spillway, outlet works and the Reuben Hart Diversion. A preliminary report was submitted on 11 March 1960 and approved 6 April 1960. A final report was submitted on 9 May and approved 31 May 1960. A summary of the hydrology criteria is given below.

7. Spillway Design Flood. - The spillway design flood was derived from the estimated probable maximum precipitation over the watershed with the effect of storage at North Pond and Reuben Hart Reservoir taken into account. The probable maximum precipitation over the basin amounts to 24.0 inches in 24 hours with 19.5 inches occurring in a 6-hour period. Infiltration, surface detention and other losses are assumed at the rate of 0.05 inches per hour, resulting in a total rainfall excess of 22.8 inches.

a. Reuben Hart Reservoir. - The adopted spillway design flood with a peak inflow of 7800 c.f.s. was constructed by applying the rainfall excess to the adopted unit hydrographs for the areas above the Reuben Hart Dam. Routing the spillway design flood through the Reuben Hart Reservoir, assuming the reservoir initially full to spillway crest elevation of 911.0 ft. m.s.l., results in a maximum surcharge elevation of 916.4 ft. m.s.l. and a maximum discharge of 6,100 c.f.s. to Hall Meadow Brook Reservoir and 1650 c.f.s. to Hart Brook.

b. Hall Meadow Brook Reservoir. - The adopted spillway design flood with a peak inflow of 26,600 c.f.s. was developed by applying the rainfall excess to the adopted unit hydrograph for the net drainage area above the Hall Meadow Brook Dam, and adding the outflow from Reuben Hart Reservoir as described above. Routing the spillway design flood through the reservoir, assuming the reservoir initially has 6 inches of storage utilized from an antecedent flood and that the outlet is inoperative, results in a maximum surcharge elevation of 912.1 ft. m.s.l. and a maximum discharge of 19,200 c.f.s.

8. Flood Control Outlet. - A 48-inch diameter ungated outlet is provided on the right bank of the brook through the dam to pass the normal flow of the brook and to limit the maximum discharge to approximately the channel capacity downstream. With the pool at spillway crest, the resulting discharge is 450 c.f.s. At maximum surcharge, the discharge is 510 c.f.s. The estimated channel capacity downstream of the dam is 400 c.f.s., which will not be exceeded until the pool rises above 890.0 ft. m.s.l., which represents about 71 percent of the flood control capacity.

Upon completion of current work on the river channels through Torrington, Connecticut, the channel capacities will be 12,600 c.f.s. on the Lower West Branch and 19,500 c.f.s. on the Naugatuck River below the confluence. The standard project flood at these two locations as modified by Hall Meadow Brook Reservoir and the Reuben Hart Diversion with an ungated 48-inch outlet will be 12,400 c.f.s. and 19,300 c.f.s. respectively. The discharge capacity of the selected outlet will satisfy diversion requirements using an upstream cofferdam with top elevation 880 feet, m.s.l.

9. Freeboard. - A freeboard of 4.9 feet above the maximum surcharge pool of 912.1 feet, m.s.l. is provided resulting in a top of dam elevation of 917.0 feet, m.s.l.

C. HYDRAULIC DESIGN

10. General. - The hydraulic design of the spillway and outlet works is discussed in Design Memorandum No. 1 - Hydrology and Hydraulic Analysis. Data pertinent to the design of the main spillway, the outlet works and the Reuben Hart diversion spillway and auxiliary spillway are given below.

11. Main Spillway. - The main spillway selected for the Hall Meadow Brook Dam is an ogee section with the top portion of the upstream face inclined at an angle of 45° with the radius of the upstream portion of the ogee crest 5.44 feet and the equation of the coordinates of the downstream section $y = 0.0817 x^{1.768}$. To facilitate the construction of the weir a tangent of 1.34 feet in length was used to separate the parabolic curve from the bucket curvature which has a 15-foot radius. The toe of the weir is set at an elevation of 1 foot above the proposed channel grade to avoid the possibility of the uneven rock excavation interfering with the flow. It was concluded that a spillway length of 100 feet provided the most practical layout. Assuming the reservoir initially has 6 inches of storage utilized from an antecedent flood and neglecting the relatively small discharge through the outlet, the reservoir pool for the selected spillway length was 912.1 with a corresponding discharge of 19,200 c.f.s.

12. Outlet Conduit. - An ungated circular outlet conduit with a diameter of 48 inches was adopted. The conduit has a length (including transition) of about 315 feet. To insure that the conduit will rest on sound rock, the invert elevation has been set at 842.0 feet, m.s.l. through the transition at the upstream end and then will have a slope of one foot to the portal end.

13. Intake. - A drop structure at the entrance to the conduit, with a crest elevation of 859.0, will maintain a permanent pool to mitigate fish and wildlife losses. To permit flexibility in the final

adopted permanent pool elevation, the weir will include 6 stop-log sections, each 5 feet in length. Drainage of the permanent pool will be accomplished with a hand-operated sluice 3' x 4' located in the upstream face of the weir with a gate invert at elevation 850.0 m.s.l. The vertical curve to support the jet at the intake was determined from the following formula:

$$\frac{X^2}{D^2} + \frac{Y^2}{\left(\frac{2D}{3}\right)^2} = 1$$

where D is equal to 4 feet. Since the jet will be suppressed in the horizontal direction by the walls of the weir, a simple circular curve with a radius of 22.0 feet was used instead of an elliptical shape. A transition section 10 feet in length will provide fillets from a 4' square section to a circular section with a 4-foot diameter.

With the sluice gate closed and stop logs for the permanent pool at elevation 860.0, the weir remains the hydraulic control until the flow exceeds about 270 c.f.s. At that time the weir will become submerged with the pool elevation about 862.0 m.s.l. The conduit itself will flow under pressure when the discharge exceeds about 80 c.f.s. With the sluice gate open, the control will not shift to the weir until the discharge exceeds about 180 c.f.s.

14. Trash Racks. - A metal trash rack with a maximum opening of about 4 square feet will span the weir portion of the intake and also provide access to the hand-operated sluice at the end of the weir. The average velocities through unobstructed openings with the pool at spillway crest will be 2.5 feet per second.

15. Outlet Stilling Basin. - From preliminary geologic investigations, it is considered that the bedrock in which the outlet channel is located is capable of withstanding the maximum velocity of 35 feet per second which would occur with the pool at spillway crest. Since a stilling basin is not considered necessary, the transition from the portal end of the circular conduit to the 14' trapezoidal discharge channel will be a 25-foot reinforced concrete apron to support and spread the discharge.

16. Reuben Hart Diversion. - The diversion from the existing Reuben Hart water supply reservoir will be accomplished by a new spillway located upstream of the present easterly abutment and a diversion canal into the Reuben Hart reservoir. In addition the existing spillway will be modified and incorporated into a new auxiliary spillway for floods exceeding the standard project flood.

17. Diversion Spillway. - The diversion spillway is an uncontrolled ogee weir with a 45° upstream face and a crest elevation of 911.0 (elevation of existing Reuben Hart spillway). The radius of the upstream portion of the ogee crest is 2.50 feet and the equation of the coordinates of the downstream section is $Y = .143\ x^{1.779}$. A tangent 3.84 feet in length was used to connect the ogee section to the bucket with a 5' radius. The elevation at the toe was set 1 foot above the channel grade to avoid the possibility of interference to flows from the uneven rock. The selected length of 125 feet will permit the discharge of a standard project flood with a head of 3.5 feet, thereby providing a freeboard of 2.5 feet on the existing dam. With spillway design flood conditions, the diversion spillway will discharge 6,100 c.f.s. into the Hall Meadow Reservoir and the auxiliary spillway will discharge 1,650 c.f.s. into the present spillway discharge channel at Reuben Hart Dam. The Reuben Hart pool will be elevation 916.4 feet which will leave a freeboard of 0.6 feet on the existing dam.

18. Auxiliary Spillway. - The south wall of the approach channel to the diversion weir constitutes an auxiliary spillway. The wall is an extension of the Reuben Hart spillway which will be raised 3.5 feet to elevation 914.5, the same top grade of the wall. The length of the auxiliary overflow section, including the Reuben Hart spillway will be 207 feet. For purposes of hydraulic analyses, the effective length of the spillway was reduced to 200 feet to allow for the angle of approaching flow. The top of the wall will be curved but because of its low design head, the curvature will not conform to an ogee section.

D. GEOLOGY

19. General. - A general discussion of geologic conditions and a partial record of subsurface investigations is presented in Design Memorandum No. 2, Site Geology, submitted on 9 May 1960 and approved 31 May 1960.

20. Bedrock Conditions. - The bedrock at the site is comprised of gneisses and schists generally hard, with a steeply dipping highly distorted foliation. Extensive horizontal jointing closely spaced near the surface becoming more widely spaced with depth forms a major rock structure control throughout the area. A secondary control is formed by a steeply dipping, randomly spaced joint set whose strike varies from N 30°W to N 80°E. The depth and degree of horizontal jointing varies throughout the site but is generally most intense to depths of 10 to 15 feet below the natural rock surface.

21. Foundation Conditions. - The design of all footings for concrete structures have been established at depths generally below the zone of most prominent jointing and weathering. Intersecting joint planes will govern to a large degree the shape of rock excavations, depending on the relative orientation of the rock structure and the structural excavations. In areas of extensive cuts, it is considered practical to excavate the rock to slopes of 4 on 1 after removal of loose and blocky surface rock. Jointing and associated weathering observed in borings and outcrops adjacent to the stream channel will prohibit the excavation of rock slopes to close tolerances in the outlet works and will require that the foundation for the conduit be placed at or near stream grade to obtain a suitable foundation for anchoring of the upstream control weir structure.

The extensive and apparent continuous nature of the horizontal jointing, dipping 5 to 10°, requires that consideration be given to the relatively high degree of water transmissibility of the rock to depths of 15 to 25 feet below the rock surface. Measured rates of flow during pressure testing of the rock indicates values as great as 20 gpm at 0 pounds of pressure. In view of the apparent seepage along the horizontal planes relieved only by randomly spaced vertical joints, the structures will be designed for maximum uplift with drainage to be provided by steeply inclined drain holes which will intersect the horizontal joint system. Where stability consideration necessitates the investigation of shearing resistance to horizontal movement, a sliding friction of rock on rock should be used as the unit shearing strength of the material with an assumed angle for sliding friction of 25°. In the diversion canal upstream of the weir, anticipated water losses during storage will be controlled by use of a grout curtain and an impervious membrane. (See Plate No. 9). It is considered that the stop or packer method of grouting would be most effective in sealing the narrow ridge between the diversion canal and the existing Reuben Hart Spillway Channel. The grouting would be conducted prior to construction of the weir and walls, and would include the existing Reuben Hart weir, downstream of which considerable water losses occur during periods of high water. The extent of grouting will be generally governed by the depth of horizontal jointing which would be to shallow depths of 15 to 20 feet in the vicinity of the dam and to a depth of 0.5 of the ultimate static head on the spillway structure, or a minimum depth of 15 feet. Water losses would be further prevented by construction of a spray-on-type asphalt gunnite membrane in an area upstream from the auxiliary spillway weir as shown on Plate 9. This method is relatively inexpensive and can be rapidly constructed over the irregular rock surface. If undesirable seepage is observed during the first year of operation, the membrane could be later extended over the entire approach channel during the annual period of lowest reservoir level.

The close horizontal jointing of the rock requires that anchors be steeply inclined to intersect a maximum number of horizontal joint planes.

E. CONCRETE MATERIALS

22. Concrete Materials. - Concrete materials are covered in detail in Design Memorandum No. IV - Concrete Materials, submitted 1 April 1960 and approved 13 April 1960.

F. STRUCTURAL DESIGN

23. Purpose. - This section of the design memorandum presents the design criteria, basic data and assumptions used in the structural design of the appurtenant structures. A brief description of of the structures with loading conditions and assumptions used is included to show the design procedure. Typical computations are included in the Appendix showing the maximum conditions for the critical structures. Additional computations following the same procedure will be made wherever warranted by a change in loading or a reduction in section.

24. Scope. - The structural design of the spillway weir, spillway lining and retaining walls, inlet structure, conduit, outlet structure, Reuben Hart Diversion spillway weir and lining and modifications to the existing portions of the Reuben Hart Dam are included herein.

25. Design Criteria. -

a. General. - All working stresses conform to those specified in the Engineering Manual EM 1110-1-2101, "Working Stresses for Structural Design", dated 6 January 1958. Loading conditions, design assumptions and other design criteria are based on the following applicable parts in the Engineer Manual for Civil Works: Standard Practice for Concrete (Part CXX, October 1953); Gravity Dam Design (EM 1110-2-2200, Sept. 1958); Structural Design of Spillways and Outlet Works (Part CXXIV, December 1952) and Retaining Walls (issued as Part X, Chapter 9, dated July 1945). Accepted engineering practice has been employed in cases where the Engineering Manual for Civil Works does not apply.

b. Concrete. - The following table lists the concrete and reinforced concrete stresses used in the design of structures. In each case, the Civil Works Manual exposure classification A (applicable to structures subject to moderately severe weather exposure) has been used.

Flexure -	Lbs. per Sq. In.
Extreme fiber stresses in compression(except conduit)	1,050
Extreme fiber stresses in compression (conduit)	1,350

Flexure - (cont'd)	Lbs. per Sq. In.
Extreme fiber stresses in tension (plain concrete)	60
Shear - (v)	
Beams - no web reinforcement	90
Beams with properly designed web reinforcement	240
Footings - at critical section	75
Bond - (u) Deformed bars -	
Top bars	210
All others	300
Bearing - (fc)	
Load on entire area	750
Load on one-third area or less-maximum permissible	1,125
Modular Ratio - (n)	10

c. Reinforcement. - (1) Grade and Working Stresses. - All reinforcement in the structures, including temperature and shrinkage reinforcement was designed for the working stresses of new billet steel, intermediate grade, deformed bars which is 20,000 p.s.i. in flexural tension. The reinforcement will conform to the requirements of Federal Specification QQ-S-632, Type II, Grade C and to ASTM A-305-56T.

(2) Spacing. - The clear distance between parallel bars will not be less than 1½ times the diameter of round bars except that in no case will the clear distance between parallel bars be less than 1 inch, or 1½ times the maximum size of the coarse aggregate.

(3) Minimum Cover for the Main Reinforcement. - The minimum cover from main steel reinforcement to surface was maintained at 4" in all cases.

(4) Splices. - All splices will be lapped 30 diameters to develop by bond, the total working strength of the bars. Splices in the main reinforcement at points of maximum moment have been avoided in the design.

(5) Temperature and Shrinkage Reinforcement. - Temperature and shrinkage reinforcement will be provided where the main reinforcement extends in only one direction. Such reinforcement will provide for a ratio of steel area to concrete area (bd) of 0.002 with a minimum of .0012 in each face up to a maximum of #6 bars @ 12" cc.

d. Structural Steel. - Structural steel was designed for the working stresses of ordinary bridge and building steel (yield point 33,000 p.s.i. minimum) which conforms to the specifications for the Design, Fabrication and Erection of Structural Steel for Buildings, issued by the American Institute of Steel Construction. Allowable design working stresses conform to those given in the Engineering Manual for Civil Works using a basic stress of 20,000 p.s.i.

e. Increase in Normal Working Stresses. - No increases in normal allowable stresses were used.

26. Basic Data and Assumptions. -

a. Controlling Elevations of Dam and Appurtenant Structures (m.s.l.) -

(1) Main Structure -

Top of Dam and Dike	917.0
Spillway Crest	898.0
Maximum water surface just upstream at spillway weir	912.1
Maximum tailwater elevation at spillway weir	900.0
Conduit invert	842.0
Conduit outlet	841.0

(2) Reuben Hart Diversion -

Main Weir	911.0
Auxiliary Weir	914.5
Modified Existing Weir	914.5
Maximum water surface upstream of weir	916.4

b. Loads. - (1) Dead Loads. - The following unit weights for materials were used:

MATERIAL	UNIT WEIGHT (lbs/cu. ft.)			
	Dry	Saturated	Moist	Submerged
Rockfill	115	135	115	72
Gravel Bedding & Pervious Fill	125	141	130	78
Impervious Fill	130	145	140	82
Concrete (Plain & Reinforced)	150			
Steel	490			

(2) Live Loads. - The following live loads were used:

Water	62.5 lbs. per cu. ft.
Wind	30 lbs. per sq. ft.
Equipment	as furnished by manufacturer
Snow	40 lbs. per sq. ft.

c. External Water Pressure. - Triangular distribution of the water pressure in the reservoir pool on the spillway and abutments was used. Tailwater pressure was taken at 60% of full value for the spillway section.

d. Internal Water Pressure. - Uplift pressure under the dam was assumed effective on 100% of the area of the base, varying uniformly from tailwater head at the toe to full headwater at the heel for the main spillway. The uplift pressure for the Reuben Hart Diversion spillways was assumed as full headwater pressure across the entire base.

e. Earth Pressure. - Earth pressures were determined in general in accordance with Part X, Structural Design, Chapter 9, Retaining Walls. "At rest" pressures were used in all cases.

f. Earthquake Forces. - Because of the small size of the structures involved, earthquake forces are not a factor and were disregarded.

g. Ice Pressure. - Horizontal forces due to ice pressure were included in the design of the Reuben Hart auxiliary weir as well as the conservation weir.

h. Frost Protection. - On the basis of temperature records and frost penetration depth curves derived by the Arctic Construction and Frost Effects Laboratory of the Corps of Engineers, a minimum frost protective cover of 4 feet above foundation level will be used for any structures founded on earth.

27. Main Spillway Weir and Lining. -

a. Description. - The main spillway weir is an ogee-shaped concrete overflow section 100 feet long with crest elevation at 898.0 m.s.l. (See Plate No. 5). Contraction joints will be provided at a 25.0 ft. spacing. A short section of concrete lining will be anchored to the rock on the West Side and a section of lining with a small gravity section above the rock surface will be employed on the East Side. Vertical contraction joints will be employed in the lining at a spacing not over 30 feet.

b. Spillway Stability. - The following loading conditions were used in the design:

Case I. - Reservoir at maximum surcharge and tailwater levels, full uplift over 100% of the base and hydrostatic lateral pressure on the downstream face taken at 60%.

Case II. - Same as Case I, except 10 feet of rock beneath weir included as acting with the weir.

Case III. - Water at spillway crest and uplift over 100% of base area varying from full head upstream to 0 downstream.

c. Spillway Design. - Maximum bearing under Case III was found to be only 893 lbs. per sq. ft. Under Case I loading it was found that the co-efficient of sliding friction on the base was 1.97 and that the resultant fell within the middle third of the base. As a result of the high friction co-efficient it was decided to employ rock anchors that would engage 10 feet of rock to act with the base. Under this condition of loading (Case II) the co-efficient of friction was reduced to 0.93. However, considering the restraining effect that can be obtained from the lateral resistance of rock below the base elevation of 891.0 there is an ample factor of safety against sliding.

d. Spillway Lining. - Rock anchors holding the 1'-0" thick lining to the rock face were figured for a head of water to the top of lining reduced by 50% because of drains into the rock. Design net head was determined to be 8 feet and requires #11 anchors sunk 10 feet into rock and spaced 6'-0" by 6'-0". The concrete lining was designed under two-way action spanning between the rock anchors.

e. Gravity Wall Above Spillway Lining. - The gravity wall section above the rock surface was designed for a rapid draw down condition assuming moist soil and no uplift.

28. Reuben Hart Diversion. - Structures included are a new ogee shaped spillway section 125'-0" long with crest elevation at 911.0 m.s.l., an auxiliary spillway consisting of existing spillway raised to a crest elevation of 914.5 and a new semi-gravity section joining the new spillway and the existing spillway. The semi-gravity section wall will have a top elevation of 914.5, the same as the auxiliary spillway and is approximately 150 ft. long. Contraction joints in the new weir will be spaced at approximately 31 feet center to center and in the new semi-gravity wall at approximately 21 feet center to center. (See Plate Nos. 6 and 7.)

a. New Spillway Section. -

(1) Stability. - The spillway was analyzed for the following conditions of loading:

Case I. - Construction condition, dead load of structure only.

Case II. - Normal operating condition with reservoir at crest elevation, no tailwater and full uplift over 100% of the base.

Case III. - Surcharge elevation of 914.5 and tailwater at 907.0 full uplift over 100% of the base.

Case IIIA.- Surcharge elevation at 916.4 and tailwater at elevation 908.7 full uplift over 100% of the base.

(2) Spillway Design. - Bearing pressure under all conditions was found to be low and the resultant within the mid third of the base. In order to obtain a satisfactory factor of safety against sliding it was found necessary to engage 10 feet of rock to act with the concrete.

b. Modification to Existing Spillway. -

(1) Stability. - Additional concrete added to the existing section was considered to make it act as a combined unit. The combined section was analyzed for the same loading conditions as the new spillway section except as follows:

Case II. - Ice thrust of 7,500 lbs. per linear foot applied.

Case III. - Surcharge elevation at 916.4 and no tailwater. Full uplift over the base to headwater elevation.

(2) Spillway Design. - The ice thrust application required the addition of the bucket downstream in order to obtain a satisfactory stability condition with the resultant within the base.

c. Semi-Gravity Sections. - The semi-gravity sections were analyzed for the same conditions of loading as the Modified Spillway Section. It was necessary to design the concrete section as a cantilever from the rock surface. Tension anchors will be provided anchoring the concrete and reinforcing steel will be provided in the upstream face to resist tension.

29. Inlet Structure. - The Inlet Structure is a small rectangular concrete structure 12'-0" by 21'-0" with invert at elevation 842.0, crest of weir at 859.0 and top of stop-logs at elevation 862.0. The structure was designed as a vertical U section with the top supported by beams and struts. The structure was designed for a loading condition with water to elevation 862.0 on the outside with inside dry. It was also investigated for an ice thrust of 10,000 lbs. per linear foot. Due to the confining area around the structure a higher value for ice pressure was used here than on the structures in the Reuben Hart Diversion. (See Plate No. 3)

30. Conduit. -

a. Description. - The conduit will be a circular section 4'-0" inside diameter and 313 feet long. Contraction joints will be provided at 31 foot intervals.

b. Loading. - The conduit was investigated for the following two loading conditions.

Case I. - Rapid drawdown from spillway crest with earth saturated to elevation 898.0 and moist from elevation 898.0 to top of dam. Vertical load taken as the rectangular weight above the conduit roof and horizontal load as 50% of the vertical.

Case II. - Water at spillway crest, earth submerged to elevation 898 and moist above. Vertical load taken as the rectangular weight above conduit roof and horizontal as 50% of submerged and moist weights plus hydrostatic pressure to elevation 898.0.

c. Conduit Design. - Conduit section required was found to be 1'-3" thick and steel reinforcement consists of #6 rods at 1'-0" spacing except at invert where spacing was decreased to 6" because of bond requirements.

31. Outlet Structure. - The outlet structure consists of gravity type side walls with a small channel slab anchored to rock. Design of the walls was based on accepted practice. (See Plate No. 4)

32. <u>Sluice Gate</u>. - A cast iron sluice gate 3'-0" wide x 4'-0" high will be installed on the upstream wall of the weir structure with the invert at Elevation 850.00. A floor stand with hand crank for manual operation of the gate will be mounted on the weir deck at Elevation 862.00. (See Plate No. 8).

This gate will be used for the flow of the stream during construction and before a permanent pool is established and to operate and dewater the permanent pool.

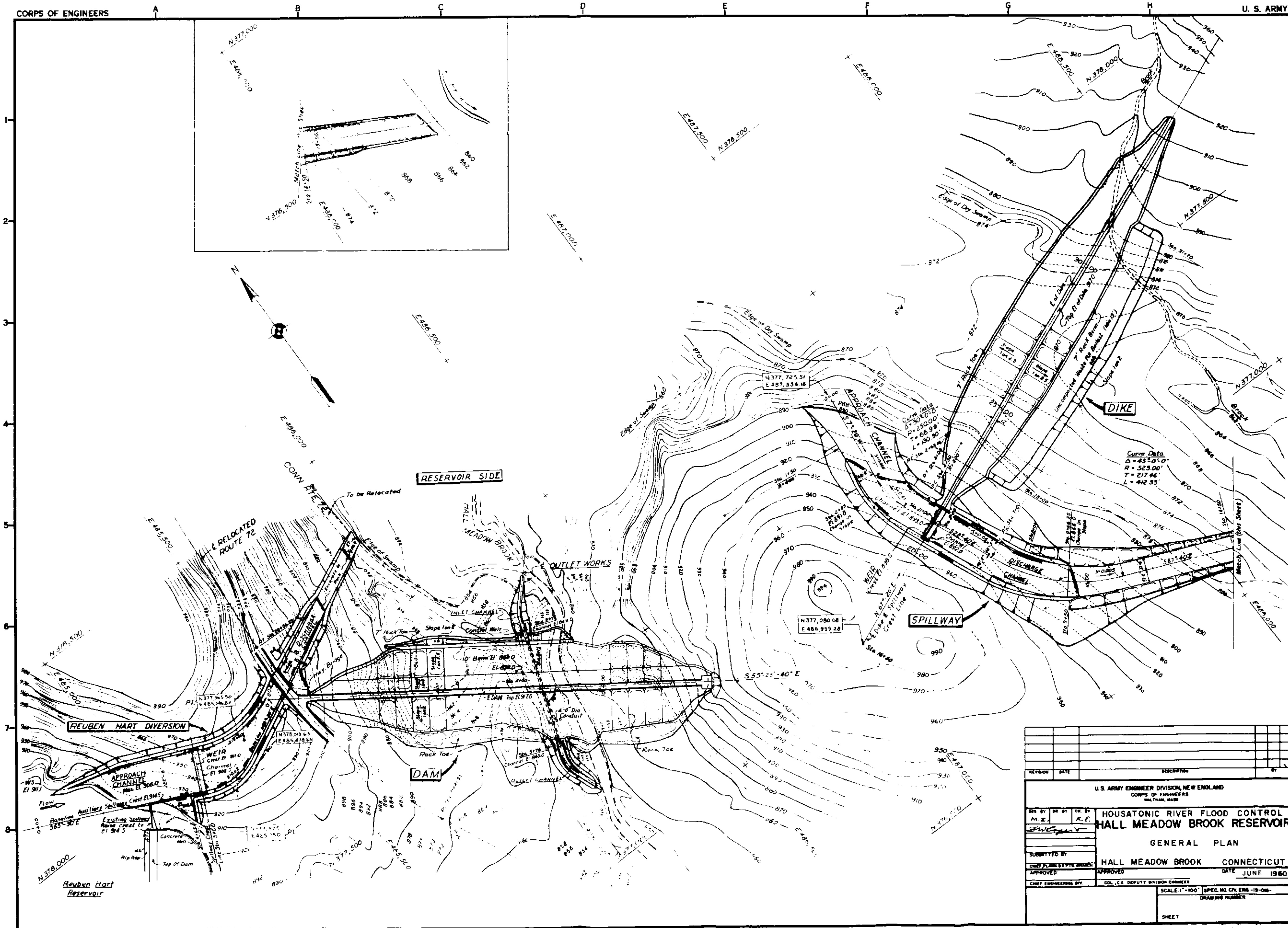

CORPS OF ENGINEERS
U. S. ARMY
RESERVOIR SIDE
DIKE
SPILLWAY
DAM
OUTLET WORKS
INLET CHANNEL
OUTLET CHANNEL
APPROACH CHANNEL
DISCHARGE CHANNEL
REUBEN HART DIVERSION
RELOCATED ROUTE 72
CONN RTE 72
HALL MEADOW BROOK
Reuben Hart Reservoir
U.S. ARMY ENGINEER DIVISION, NEW ENGLAND
CORPS OF ENGINEERS
HOUSATONIC RIVER FLOOD CONTROL
HALL MEADOW BROOK RESERVOIR
GENERAL PLAN
HALL MEADOW BROOK CONNECTICUT
DATE JUNE 1960
SCALE: 1" = 100'

PLATE NO VI - 1

CORPS OF ENGINEERS

U. S. ARMY

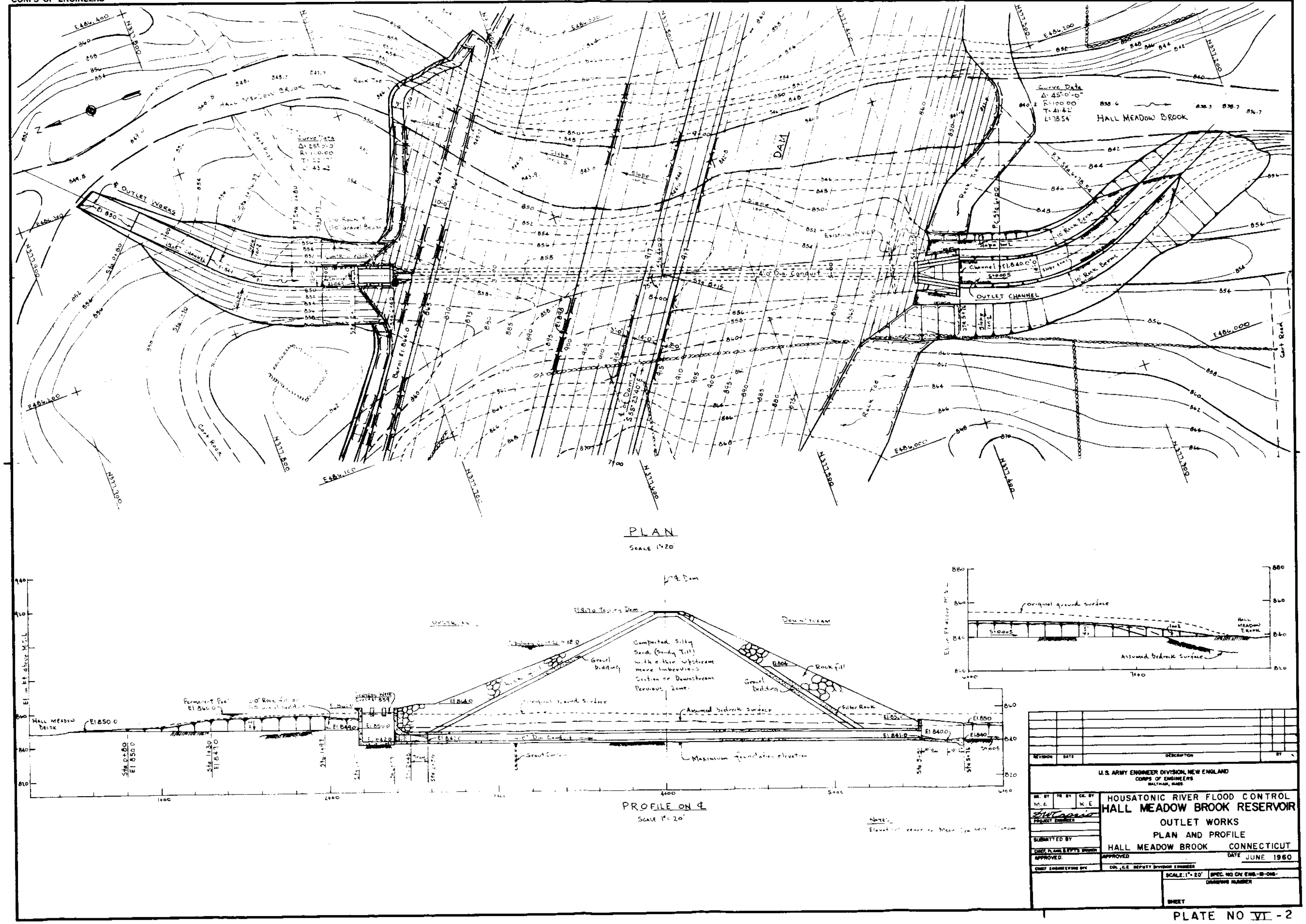
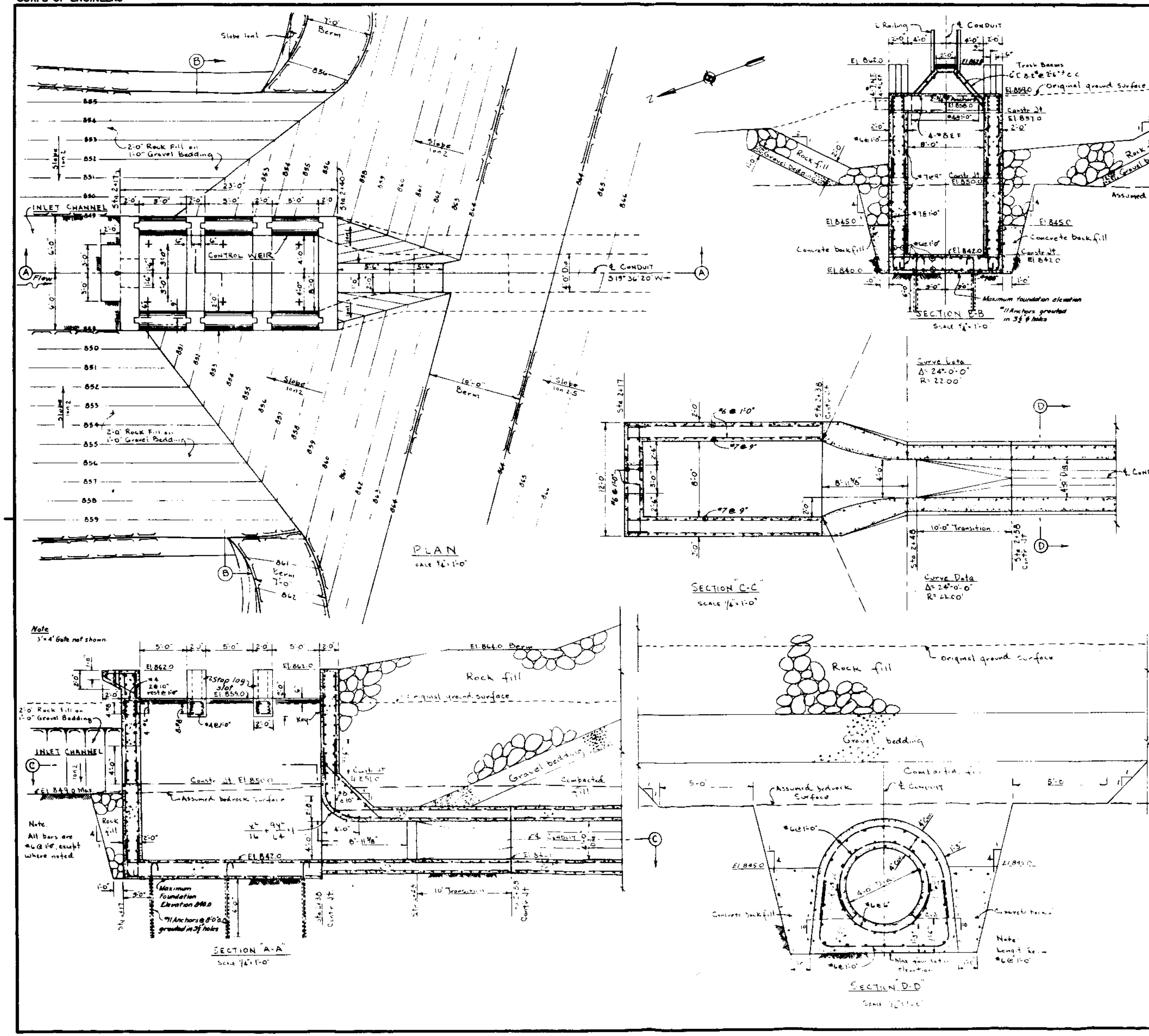


PLATE NO VI - 2



REVISION	DATE	DESCRIPTION	BY

U.S. ARMY ENGINEER DIVISION, NEW ENGLAND
CORPS OF ENGINEERS
WALTHAM, MASS.

DR. BY K.E. | TR. BY M.Z. | CK. BY M.Z.

HOUSATONIC RIVER FLOOD CONTROL
HALL MEADOW BROOK RESERVOIR
OUTLET WORKS
INLET STRUCTURE

HALL MEADOW BROOK CONNECTICUT

PROJECT ENGINEER
SUBMITTED BY
CHIEF, PLANS & SPECS. BRANCH
APPROVED:
CHIEF, ENGINEERING DIV.
APPROVED
COL., C.E. DEPUTY DIVISION ENGINEER
DATE JUNE 1960
SCALE: AS SHOWN | SPEC. NO. CIV. ENG.-19-016-
DRAWING NUMBER
SHEET

CORPS OF ENGINEERS U. S. ARMY

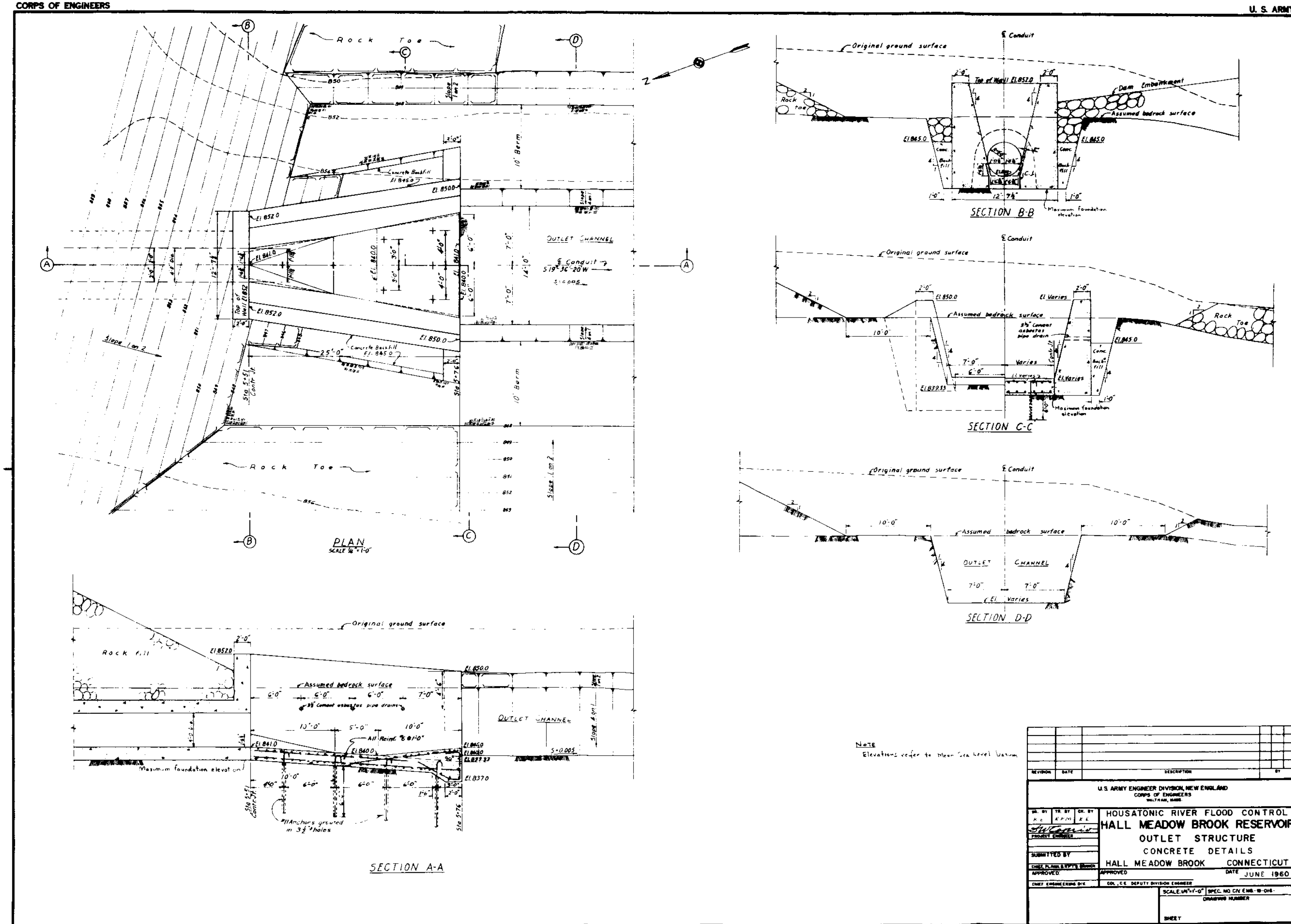


PLATE NO VI - 4

CORPS OF ENGINEERS

U. S. ARMY

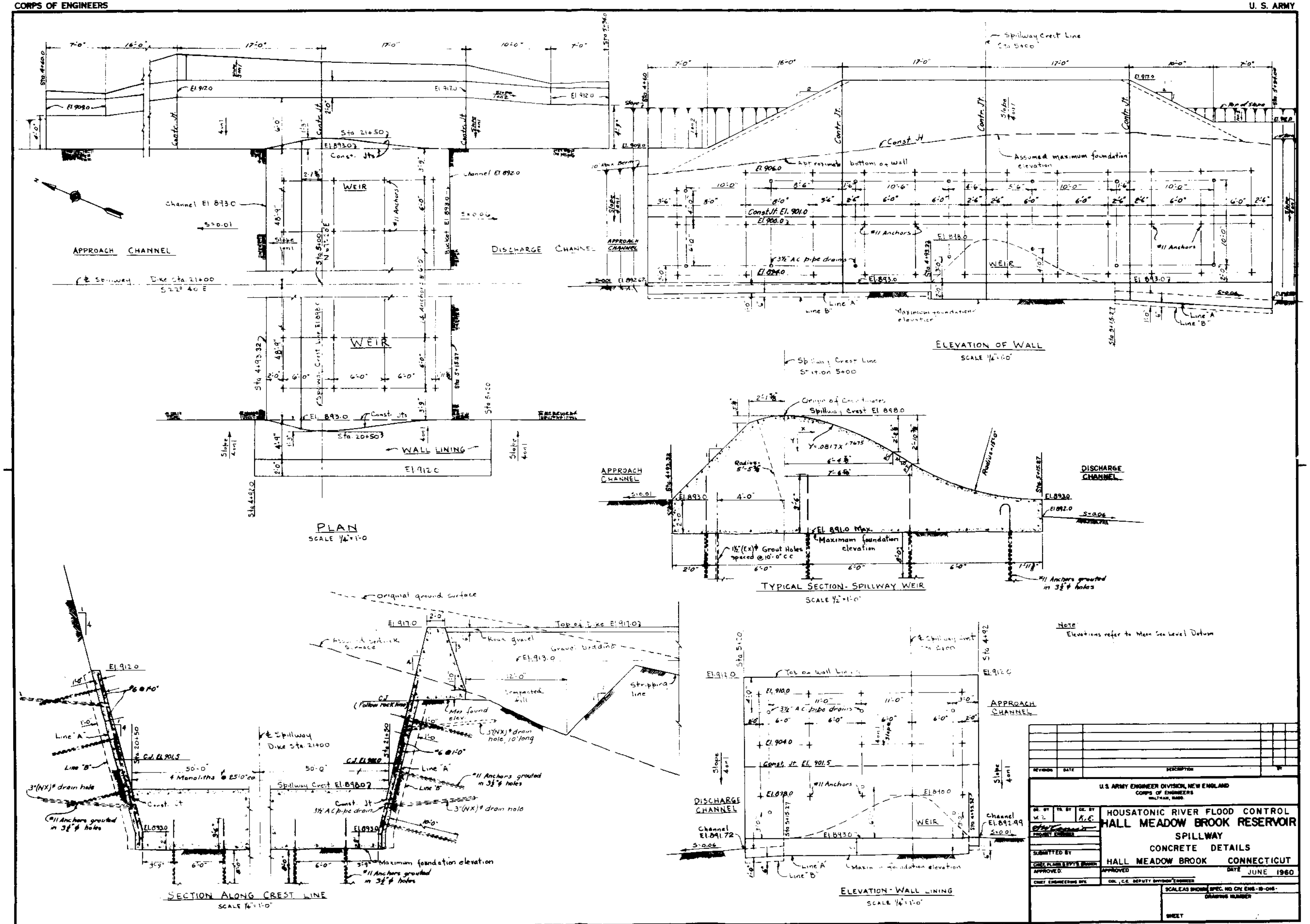
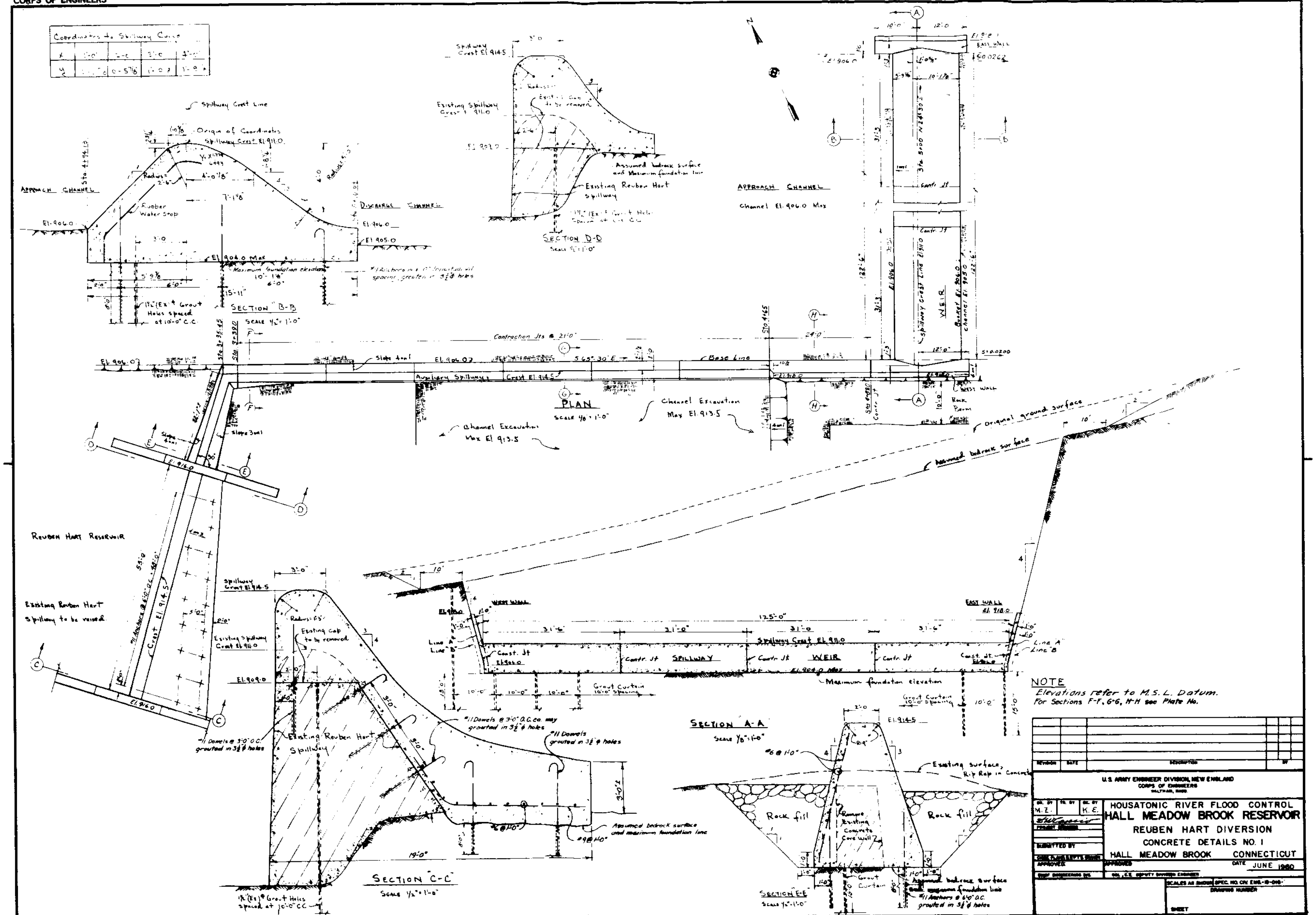
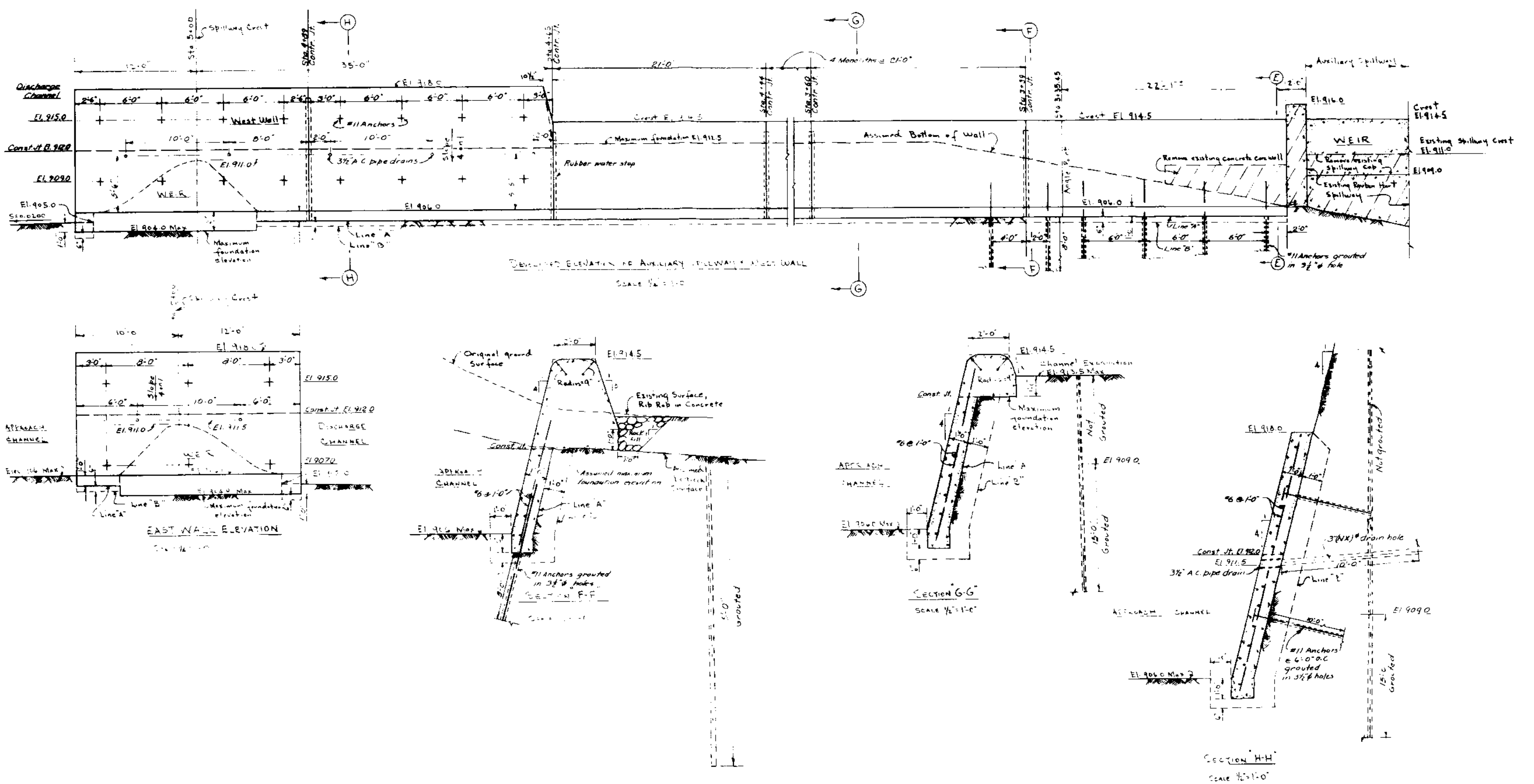


PLATE NO VI - 5


Coordinates to Spillway Curve
Spillway Crest Line
Origin of Coordinates
Spillway Crest El. 911.0
APPROACH CHANNEL
DISCHARGE CHANNEL
Rubber Water Stop
El. 906.0
El. 905.0
El. 904.0 Max
Maximum foundation elevation
1½"(Ex)# Grout Holes spaced at 10'-0" C.C.
SECTION "B-B"
Scale ½" = 1'-0"
Spillway Crest El. 914.5
Existing Spillway Crest El. 911.0
Existing Cap to be removed
Assumed bedrock surface and Maximum foundation line
Existing Reuben Hart Spillway
SECTION D-D
APPROACH CHANNEL
Channel El. 906.0 Max
WEIR
Contraction Jts @ 21'-0"
Slope 4 on 1
Auxiliary Spillway Crest El. 914.5
Base Line
PLAN
Scale 1/8" = 1'-0"
Channel Excavation Max El. 913.5
Original ground surface
Assumed bedrock surface
REUBEN HART RESERVOIR
Existing Reuben Hart Spillway to be raised
Crest El. 914.5
WEST WALL
EAST WALL
El. 918.0
SPILLWAY
Spillway Crest El. 911.0
El. 904.0 Max
Maximum foundation elevation
Grout Curtain 10'-0" Spacing
Line "A"
Line "B"
Const. Jt
Contr. Jt
SECTION "A-A"
Scale 1/8" = 1'-0"
#11 Dowels @ 3'-0" O.C. grouted in 3⅛" holes
#11 Dowels grouted in 3⅛" holes
Assumed bedrock surface and maximum foundation line
SECTION "C-C"
Scale ½" = 1'-0"
1½"(Ex)# Grout Holes spaced at 10'-0" C.C.
Existing surface, Rip Rap in Concrete
Rock fill
Remove Existing Concrete Core wall
Grout Curtain
Assumed bedrock surface
#11 Anchors @ 6'-0" O.C. grouted in 3⅛" holes
SECTION E-E
NOTE
Elevations refer to M.S.L. Datum.
For Sections F-F, G-G, H-H see Plate No.
U.S. ARMY ENGINEER DIVISION, NEW ENGLAND
CORPS OF ENGINEERS
WALTHAM, MASS.
HOUSATONIC RIVER FLOOD CONTROL
HALL MEADOW BROOK RESERVOIR
REUBEN HART DIVERSION
CONCRETE DETAILS NO. I
HALL MEADOW BROOK CONNECTICUT
DATE JUNE 1960



NOTE

Elevations refer to M.S.L. Datum.

For Section E-E see Plate No.

REVISION	DATE	DESCRIPTION	BY

U.S. ARMY ENGINEER DIVISION, NEW ENGLAND
CORPS OF ENGINEERS
WALTHAM, MASS.

DR. BY M.Z. TR. BY CK. BY K.E.

PROJECT ENGINEER

SUBMITTED BY

CHIEF, PLAN'G & ENG'G BRANCH

APPROVED: CHIEF ENGINEERING DIV.

HOUSATONIC RIVER FLOOD CONTROL
HALL MEADOW BROOK RESERVOIR
REUBEN HART DIVERSION
CONCRETE DETAILS NO. 2
HALL MEADOW BROOK CONNECTICUT

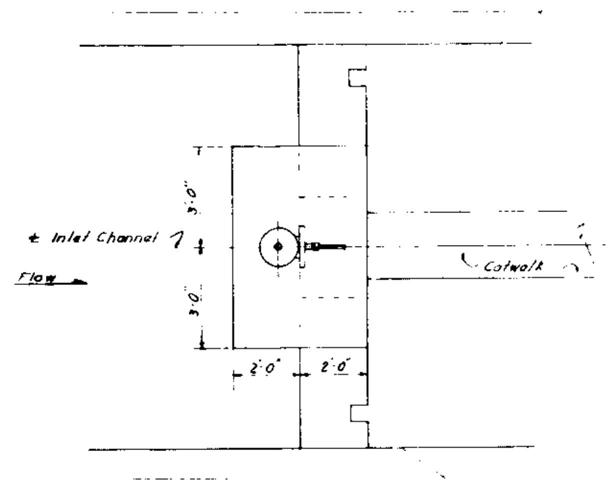
APPROVED: COL., C.E. DEPUTY DIVISION ENGINEER DATE JUNE 1950

SCALE AS SHOWN SPEC. NO. CIV. ENG.-19-016-

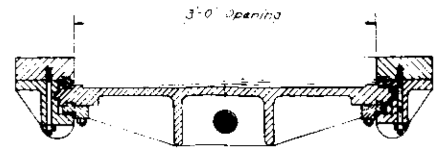
DRAWING NUMBER

SHEET

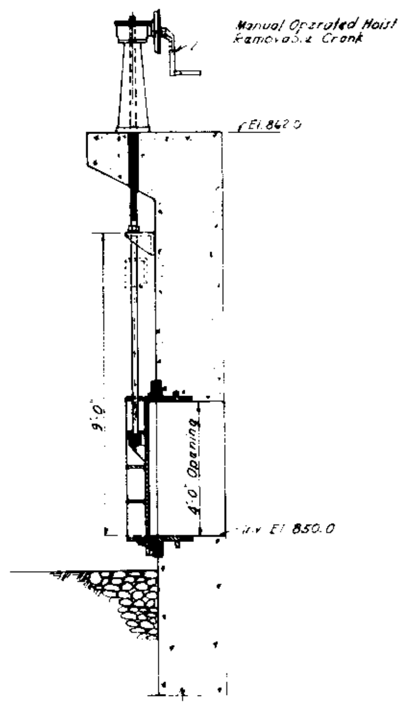
CORPS OF ENGINEERS U. S. ARMY



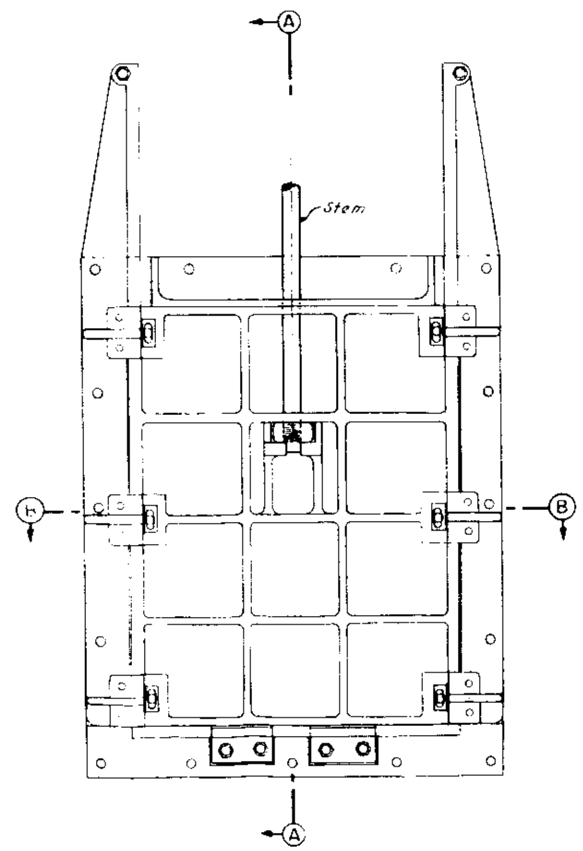
PART PLAN
SCALE 1/2"=1'-0"



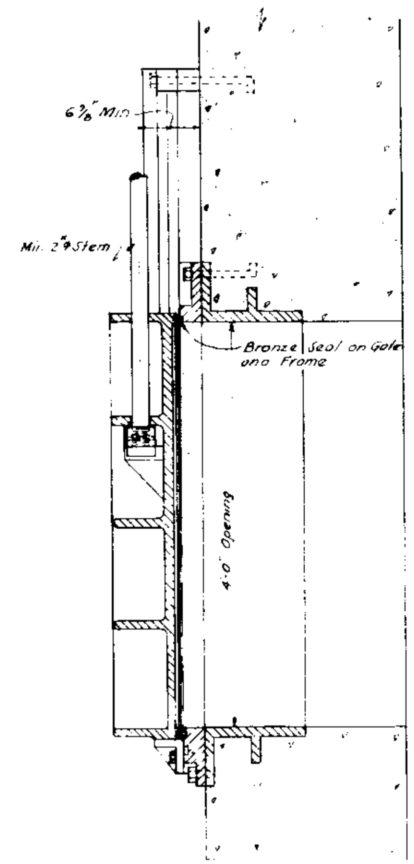
SECTION B-B
SCALE 1 1/2"=1'-0"



GATE ASSEMBLY
SCALE 1/2"=1'-0"



UPSTREAM ELEVATION
SCALE 1 1/2"=1'-0"



SECTION A-A
SCALE 1 1/2"=1'-0"

GRAPHIC SCALES
1/2"=1'-0"
1 1/2"=1'-0"

REVISION	DATE	DESCRIPTION	BY

U.S. ARMY ENGINEER DIVISION, NEW ENGLAND
CORPS OF ENGINEERS
WALTHAM, MASS.

DES. BY L.H.P. | DR. BY J.P.K. | CK. BY

SUBMITTED — CHIEF, MECH. SECTION

PROJECT ENGINEER

APPROVAL RECOMMENDED — CHIEF, DESIGN BRANCH

APPROVAL RECOMMENDED — CHIEF, P & R BRANCH

HOUSATONIC RIVER FLOOD CONTROL

HALL MEADOW BROOK RESERVOIR

SLUICE GATE

PLAN, ELEVATIONS & SECTIONS

HALL MEADOW BROOK CONNECTICUT

APPROVED — CHIEF, ENGINEERING DIVISION DATE JUNE 1960

SCALE AS SHOWN | SPEC. NO. ENG.-19-016-

DRAWING NUMBER

SHEET

PLATE NO. VI - 8

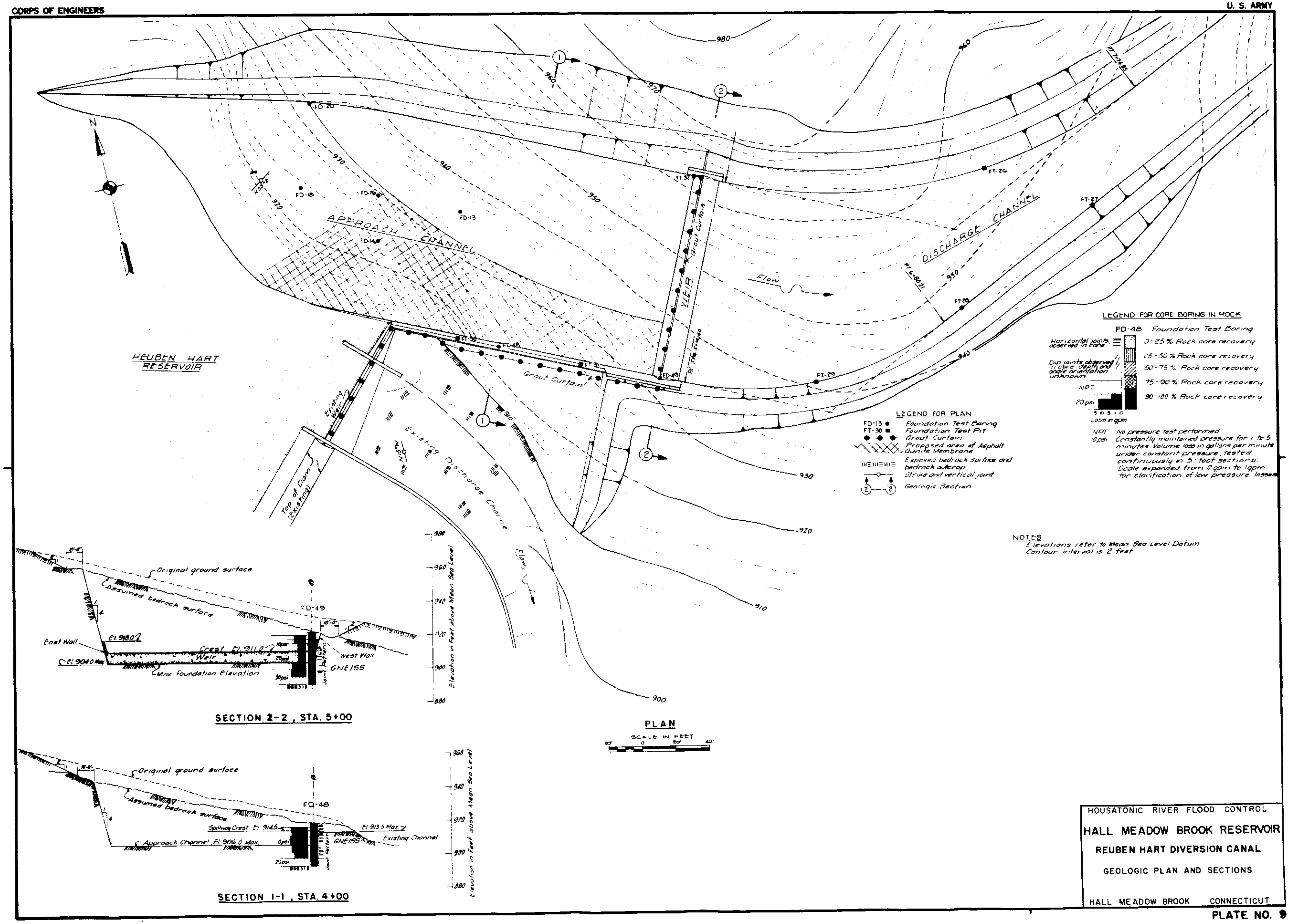

REUBEN HART RESERVOIR
APPROACH CHANNEL
DISCHARGE CHANNEL
WEIR
Grout Curtain
Existing Weir
Top of Dam (Existing)
Existing Discharge Channel
Flow
FD-13
FD-18
FD-20
FD-48
FD-49
FT-26
FT-27
FT-28
FT-29
FT-30
FT-31
FT-32
PLAN
SCALE IN FEET
LEGEND FOR PLAN
FD-13 Foundation Test Boring
FT-30 Foundation Test Pit
Grout Curtain
Proposed area of Asphalt Gunite Membrane
Exposed bedrock surface and bedrock outcrop
Strike and vertical joint
Geologic Section
LEGEND FOR CORE BORING IN ROCK
FD-48 Foundation Test Boring
Horizontal joints observed in core
Dip joints observed in core, depth and angle orientation unknown
0-25 % Rock core recovery
25-50 % Rock core recovery
50-75 % Rock core recovery
75-90 % Rock core recovery
90-100 % Rock core recovery
NPT
20 psi
Loss in gpm
NPT No pressure test performed
10 psi Constantly maintained pressure for 1 to 5 minutes. Volume loss in gallons per minute under constant pressure, tested continuously in 5-foot sections. Scale expanded from 0 gpm to 1 gpm for clarification of low pressure losses
NOTES
Elevations refer to Mean Sea Level Datum
Contour interval is 2 feet
Original ground surface
Assumed bedrock surface
East Wall
West Wall
Crest El 911.0
Weir
Max Foundation Elevation
GNEISS
Joint Pattern
Elevation in Feet above Mean Sea Level
SECTION 2-2, STA. 5+00
Spillway Crest El 914.5
Approach Channel El 906.0 Max.
Existing Channel
SECTION 1-1, STA. 4+00
HOUSATONIC RIVER FLOOD CONTROL
HALL MEADOW BROOK RESERVOIR
REUBEN HART DIVERSION CANAL
GEOLOGIC PLAN AND SECTIONS
HALL MEADOW BROOK CONNECTICUT

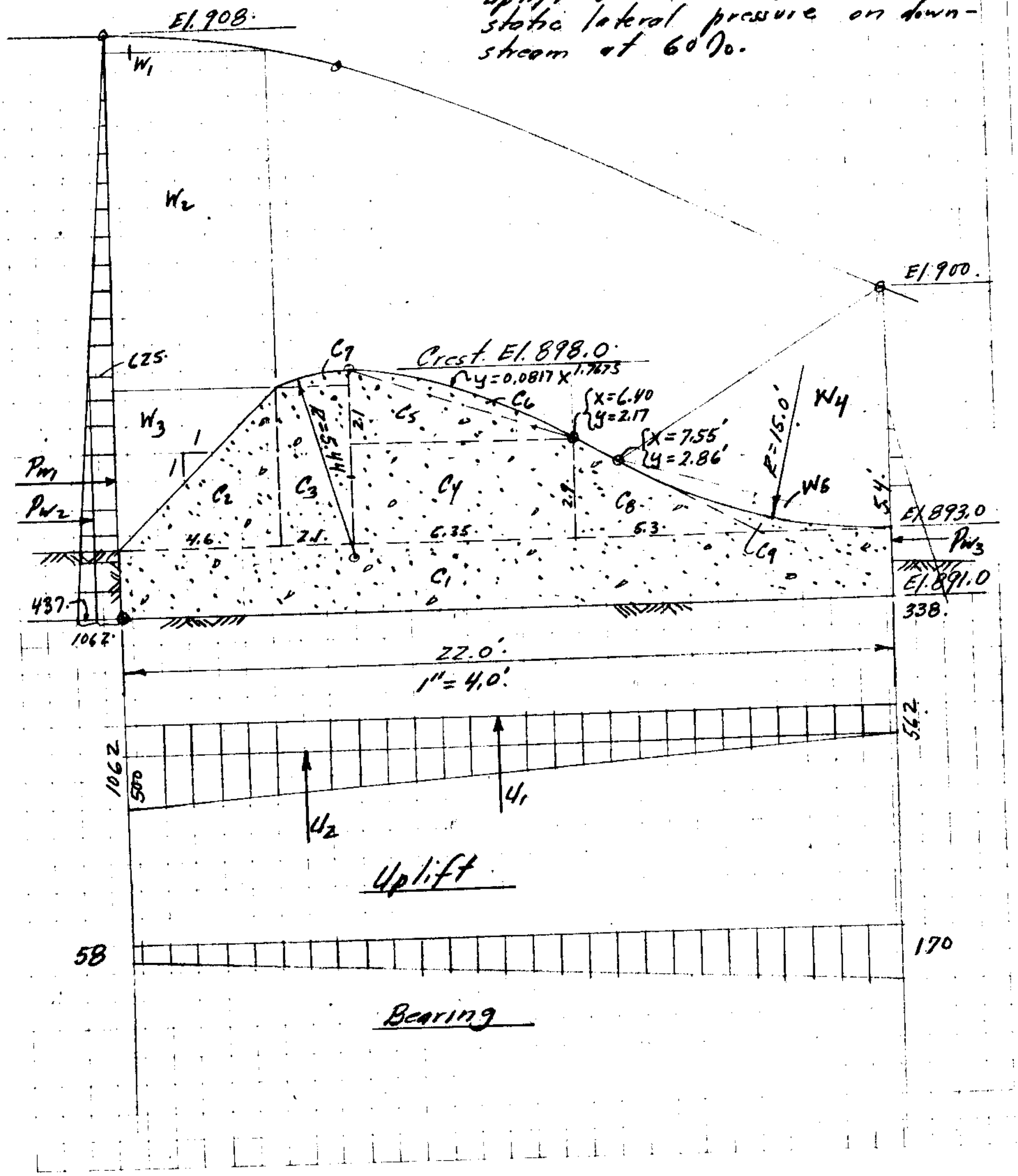
PLATE NO. 9

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U. S. ARMY

SUBJECT Hall Meadow Brook Dam
COMPUTATION Spillway Section
COMPUTED BY JSM CHECKED BY GFH DATE May 2, 1960

Case I - Reservoir at max. surcharge level; tailwater at El. 900.0; full uplift over 100% of base; hydrostatic lateral pressure on downstream at 60%.



SUBJECT Hall Meadow Brook Dam
COMPUTATION Spillway Section
COMPUTED BY JSM CHECKED BY GEH DATE May 2, 1960

C_1	= (2.0)(22.0)(150).	=	6,600.	× 11.0.	=	72,500.	
C_2	= ½(4.6)(4.6)(150).	=	1,582.	× 3.07.	=	4,860.	
C_3	= (2.1)(4.6)(150).	=	1,450.	× 5.65.	=	8,180.	
C_4	= (2.9)(6.35)(150).	=	2,760.	× 9.875.	=	27,300.	
C_5	= ½(6.35)(2.1)(150).	=	1,000.	× 8.82.	=	8,820.	
C_6	= ½(6.7)(0.4)(150).	=	201.	× 10.00.	=	2,010.	
C_7	= ½(2.1)(0.6)(150).	=	95.	× 6.00.	=	570.	
C_8	= ½(5.3)(2.9)(150).	=	1,150.	× 14.82.	=	17,100.	
C_9	= ½(4.6)(0.5)(150).	=	172.	× 18.50.	=	3,190.	144,530.
W_1	= ½(4.6)(0.6)(62.5).	=	86.	× 1.53.	=	132.	
W_2	= (4.6)(9.8)(62.5).	=	2,820.	× 2.30.	=	6,500.	
W_3	= ½(4.6)(4.6)(62.5).	=	660.	× 1.53.	=	1,010.	
W_4	= ½(8.0)(6.7)(62.5).	=	1,680.	× 19.50.	=	32,800.	
W_5	= ½(8.0)(0.5)(62.5).	=	125.	× 18.20.	=	2,270.	
			20,381. ↓			187,242. ↻	

Neg. 45

P_{W_1}	= (625)(7.0).	=	4375.	× 3.50.	=	15,350.
P_{W_2}	= ½(437)(7.0).	=	1,530.	× 2.33.	=	3,560.
			5,905 →			18,910 ↻

P_{W_3}	= ½(338)(5.4).	=	913. ←	× 1.80.	=	1,650. ↺

U_1	= (562)(22.0).	=	12,360.	× 11.00.	=	136,000.
U_2	= ½(500)(22.0).	=	5,500.	× 7.33.	=	40,300.
			17,860. ↑			176,300. ↺

$\Sigma V = 2{,}521.$ $\Sigma M = 28{,}202.$

$$\frac{\Sigma M}{\Sigma V} = \frac{28{,}202.}{2{,}521.} = 11.18' \quad e = 0.18'$$

(5)

$$\frac{\Sigma H}{\Sigma V} = \frac{4{,}992}{2{,}521} = 1.97 > 0.65$$ Anchor concrete to rock

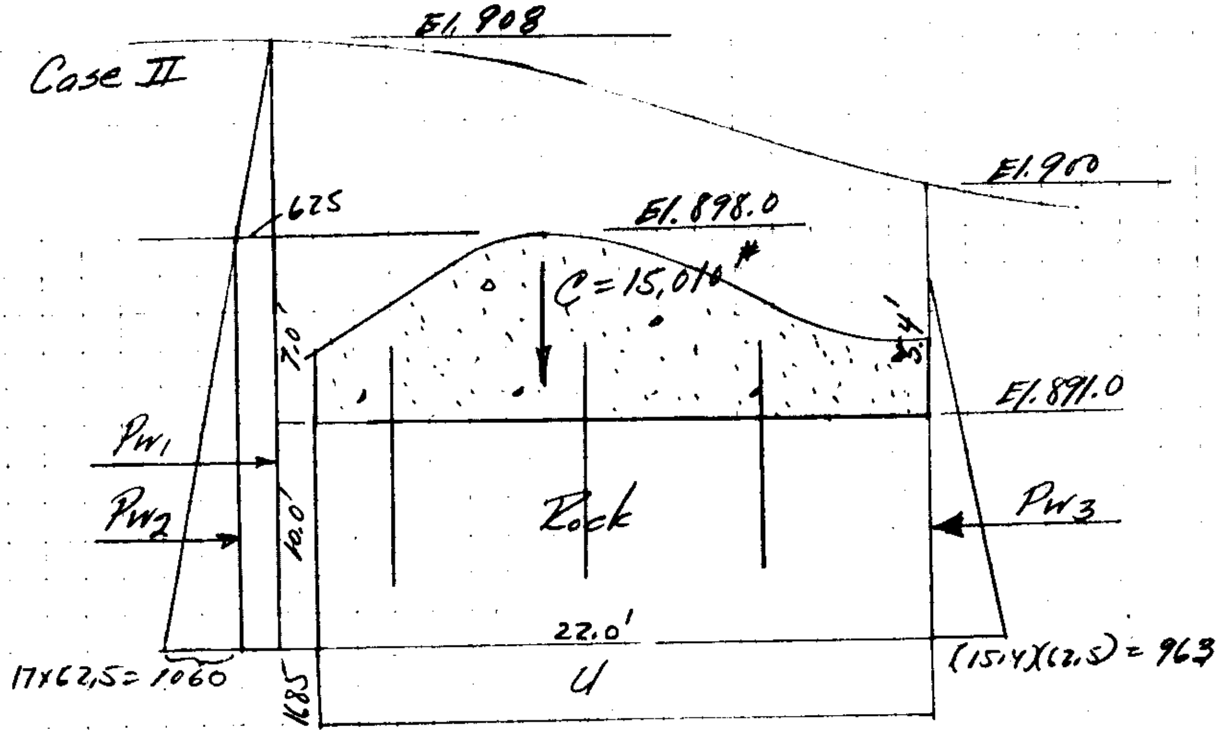
Engage 10'0" section of rock and check coeff. of sliding at bot. of rock. Due to seams in rock assume 100% of surcharge head uplift over the entire base.

From soils lab. allow $\frac{\Sigma H}{\Sigma V} = 0.45$.

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U. S. ARMY

SUBJECT Hall Meadow Brook Dam
COMPUTATION Spillway Section
COMPUTED BY JSM CHECKED BY GFH DATE [illegible]



$P_{w_1} = (62.5)(17.0) = 10,620$

$P_{w_2} = \frac{1}{2}(1060)(17.0) = 9,000$

$\quad = 19,620 \rightarrow$

$P_{w_3} = \frac{1}{2}(963)(15.4) = 7,420 \leftarrow$

$\Sigma H = 12,200 \rightarrow$

$C = 15,010$

$Rock = (10)(22.0)(160) = 35,200$

$\quad = 50,210 \downarrow$

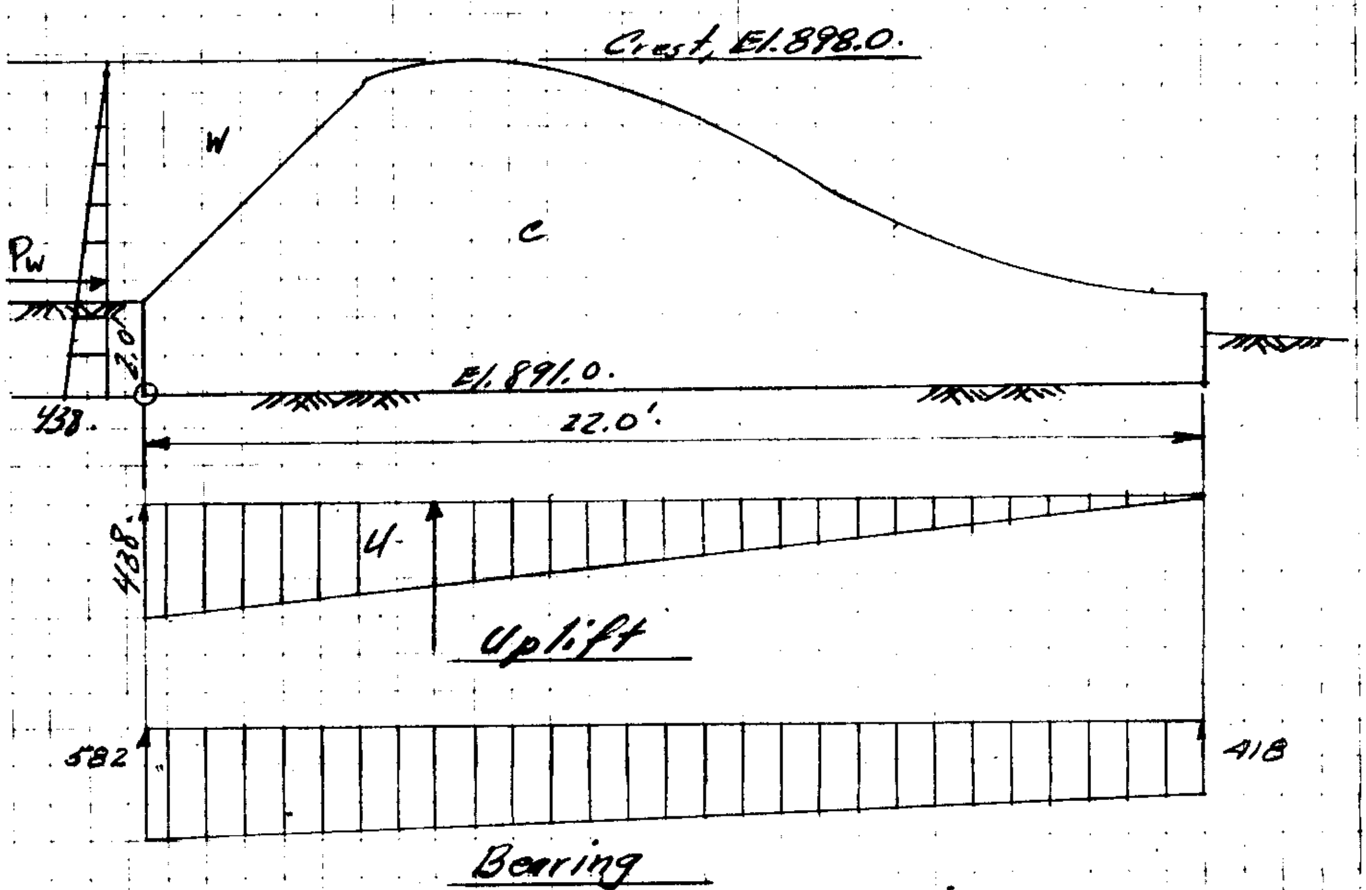
$U = (1685)(22.0) = 37,100 \uparrow$

$\quad = 13,110 \downarrow$

$\frac{\Sigma H}{\Sigma V} = \frac{12,200}{13,110} = 0.93 > 0.45$ therefore the unbalanced horiz. force of 12.2^k will be taken by lateral resistance of the rock below El. 891.0

SUBJECT Hall Meadow Brook Dam
COMPUTATION Spillway Section
COMPUTED BY JSM CHECKED BY GFH DATE May 3, 1960

Case III: Water at spillway crest elevation, uplift varying from full uplift @ heel to zero at toe.



C = = 15,010. = 144,530.

W = 1/2(5.0)(5.0)(62.5). = 780. × 1.67. = 1,300.

Pw = 1/2(438)(7.0). = 1,530. × 2.33 = 3,570.

149,400.

U = 1/2(438)(22.0). = 4,820. × 7.33. = 35,300.

ΣM = 114,100

$\frac{\Sigma M}{\Sigma V} = \frac{114,100}{10,970.} = 10.40'$ e = 0.6'

$\text{Bearing} = \frac{10,970}{22.0} \pm \frac{(6)(10,970)(0.6)}{22.0^2}$

= 500 ± 82 = 582 & 418

SUBJECT Hall Meadow Brook Dam

COMPUTATION Spillway Lining

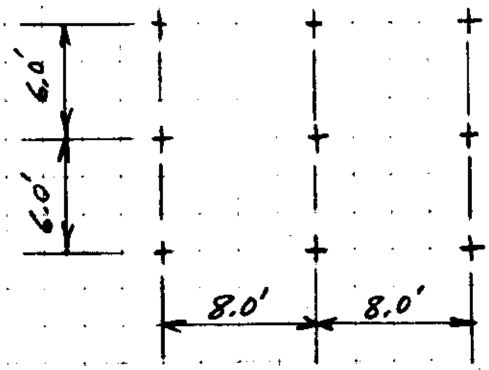
COMPUTED BY JSM CHECKED BY EPG DATE May 13, 1960

Design of Lining at Spillway Weir:

Assume hydrostatic pressure at surface of rock. Reduce head 50% due to drains

For 16' head $w = (16)(62.5)(0.5) = 500$ #/□'

Try #11 anchors, 6'-0" c.c. Vert. & 8'-0" c.c. Horiz.



Design as 2-way slab $m = 6.0/8.0 = 0.75$

$-M = (0.05)(500)(\overline{6.0}^2) = 900$ '#.

Supporting beam between anchors

For the long span, equiv. $w = \frac{(500)(6.0)}{3} \times \frac{3-\overline{0.75}^2}{2}$

$= (1000)(1.22) = 1220$ #/'

Assuming a 2'-0" wide beam

$M = \frac{(1220)(\overline{8.0}^2)}{10} = 7800$ '#.

Req'd $d = \sqrt{\frac{7800 \times 12}{160 \times 24}} = 24.4 = 4.9$" have 6".

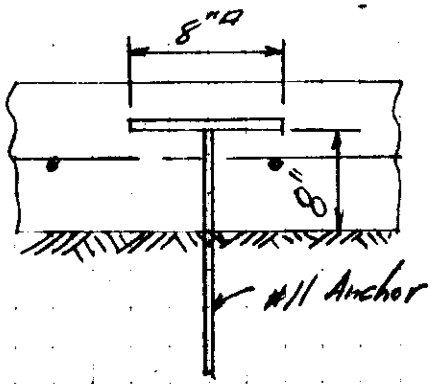
$A_s = \frac{(7800)(12)}{(20,000)(7/8)(6)(2)} = \underline{0.45}$ □"/' Use #6 @ 12" ea. way.

Pull on anchor $= (6.0)(8.0)(500) = 24,000$ #.

Allow load on #11 anchor $= (1.56)(20,000) = 31,200$ #.

For 18' head use #11 anchors on a 6'-0" □ grid

SUBJECT Hall Meadow Brook Dam
COMPUTATION Spillway Lining
COMPUTED BY JSM CHECKED BY EPG DATE May 13, 1960



Check punching shear

$$v = \frac{24,000}{(8)(8)(4)} = 94 \text{ \#/□"} \text{ OK.}$$

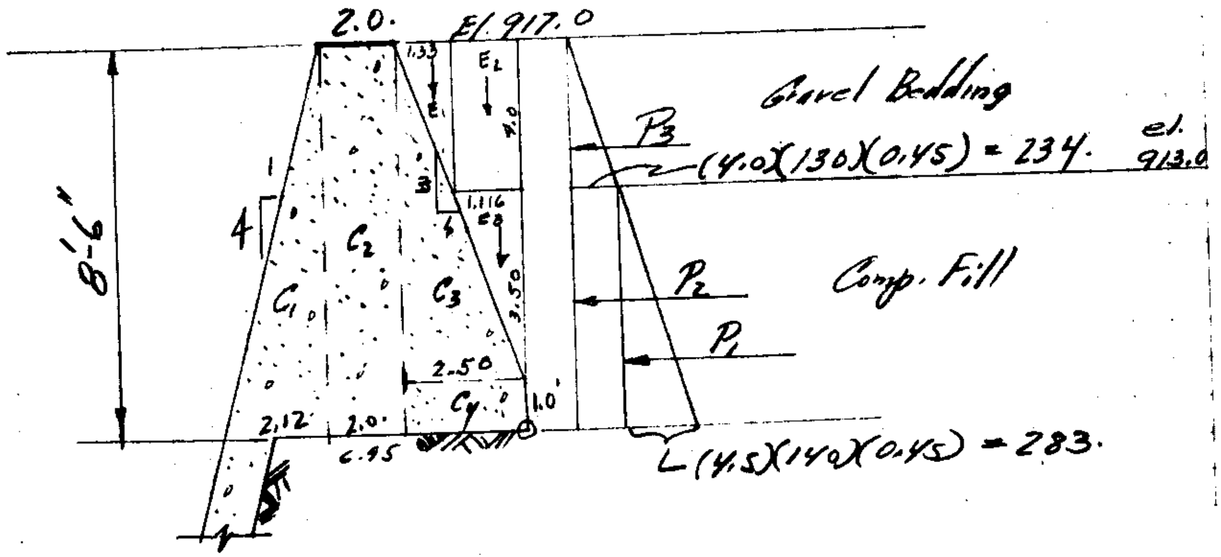
NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U. S. ARMY

SUBJECT Hell Meadow Brook Dam
COMPUTATION Retaining Wall @ Spillway
COMPUTED BY JSM CHECKED BY GFH DATE May 4, 1960

Section @ Spillway Crest Line:

Assume rapid drawdown, moist soil, no uplift:



$E_1 = 130 \times 1.33 \times 4.0 \times 1/2$	= 346 × 1.61	=	557
$E_2 = 130 \times 1.116 \times 4.0$	= 580 × 0.558	=	323
$E_3 = 140 \times 1.116 \times 3.5 \times 1/2$	= 273 × 0.372	=	102
$C_1 = 1/2(2.12)(8.5)(150)$	= 1,350 × 5.54	=	7,500
$C_2 = (2.0)(8.5)(150)$	= 2,550 × 3.83	=	9,760
$C_3 = 1/2(2.50)(7.5)(150)$	= 1,405 × 1.67	=	
$C_4 = (1.0)(2.50)(150)$	= 375 × 1.25	=	597
	ΣV = 6,879		18,839

$P_1 = 1/2(283)(4.5)$	= 636 × 1.50	=	953
$P_2 = (234)(4.5)$	= 1,052 × 2.25	=	2,360
$P_3 = 1/2(234)(4.0)$	= 468 × 5.83	=	2,730
	ΣH = 2,156		6,043

$\Sigma M = 24,982$

$$\frac{\Sigma M}{\Sigma V} = \frac{24,982}{6,879} = 3.63$$ within mid 1/3 O.K.

$$\frac{\Sigma H}{\Sigma V} = \frac{2156}{6879} = 0.318 < 0.45$$ O.K. for sliding

SUBJECT HALL MEADOW BROOK RESERVOIR REUBEN HART DIVERSION

COMPUTATION STABILITY ANALYSIS - New Weir Section

COMPUTED BY E.P.G. CHECKED BY GFH DATE MAY 23 1960

CONDITIONS TO BE INVESTIGATED FOR THE VARIOUS SECTIONS.

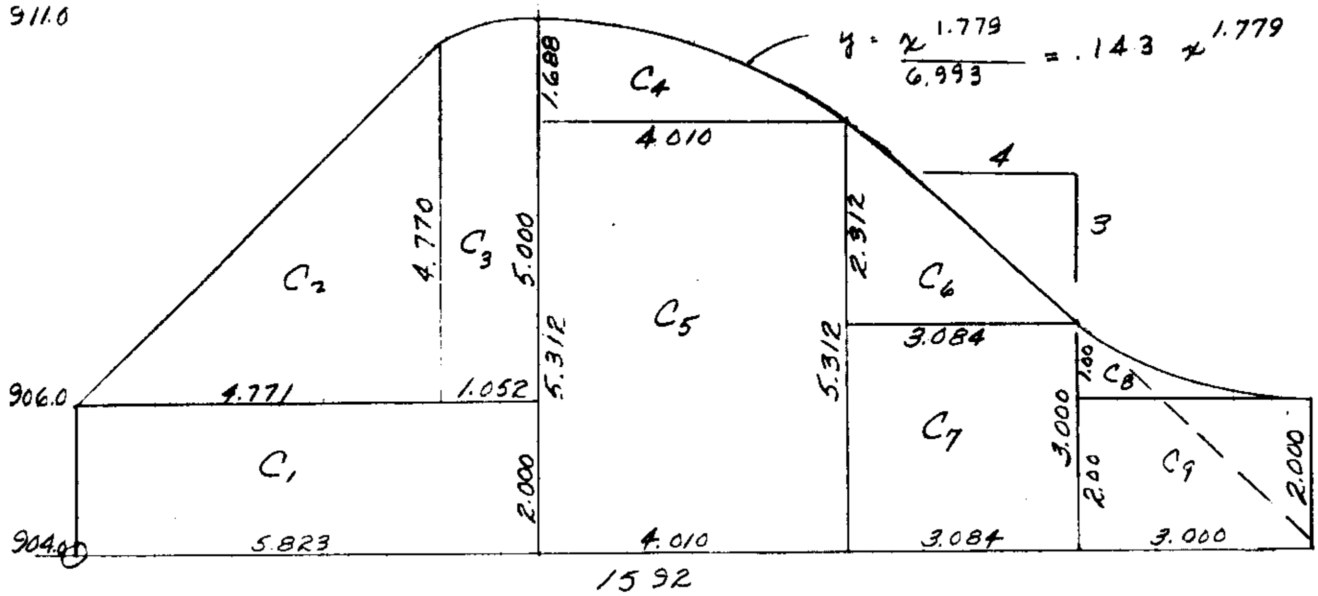
CASE I CONSTRUCTION CONDITION
NOTE: THIS CONDITION NOT CRITICAL BY INSPECTION

CASE II NORMAL OPERATING CONDITION
RESERVOIR AT WEIR CREST - ELEV. 911.0
NO TAILWATER
FULL UPLIFT (CONSTANT FROM HEEL TO TOE)
ICE PRESSURE (WHERE APPLICABLE USE 7.5 KIPS)

CASE III FLOOD DISCHARGE CONDITION
POOL AT MAXIMUM FLOOD ELEVATION 916.4
TAILWATER AT 60% WHERE APPLICABLE
FULL UPLIFT (CONSTANT FROM HEEL TO TOE)

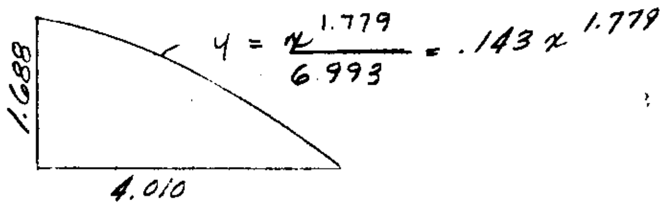
NOTE: EARTHQUAKE LOADINGS ARE NOT CRITICAL FOR THESE SECTIONS.

SUBJECT Hall Meadow Brook Reservoir Reuben Hart Diversion
COMPUTATION Stability Analysis Weir Section
COMPUTED BY E.P.G. CHECKED BY GFH DATE April 14 1960



Nappe Properties of C-4

$$y = 1.69 - 0.143\, x^{1.779}$$
$$dA = 1.69\, dx - 0.143\, x^{1.779} dx$$



$$y = 1.69 - .143 x^{1.779}$$
$$dA = 1.69\, dx - .143\, x^{1.779} dx$$
$$A = \int_0^{4.010} 1.69\, dx - \int_0^{4.010} .143\, x^{1.779} dx$$
$$A = \left[1.69 x\right]^{4.010} - \left[\frac{.143\, x^{2.779}}{2.779}\right]^{4.010}$$
$$A = 1.69(4.010) - \frac{.143(4.010)^{2.779}}{2.779} =$$
$$A = 6.76 - 2.47 = 4.29 \text{ ⌀}.$$

SUBJECT: HALL MEADOW BROOK RESERVOIR — REUBEN HART DIVERSION
COMPUTATION: STABILITY ANALYSIS WEIR SECTION
COMPUTED BY: E.R.G. CHECKED BY: GFH DATE: APRIL 14, 1960

$$dM_y = x\,dA = x\,y\,dx$$

$$M_y = \int_0^{4.010} 1.69x\,dx - \int_0^{4.010} .143\,x^{2.779}\,dx$$

$$= \left[\frac{1.69\,x^2}{2}\right]_0^{4.010} - \left[\frac{.143\,x^{3.779}}{3.779}\right]_0^{4.010}$$

$$= \frac{1.69}{2}(4.010)^2 - \frac{.143\,(4.010)^{3.779}}{3.779}$$

$$= 13.5 - 7.2 = 6.3$$

$$\bar{x} = \frac{M_y}{A} = \frac{6.3}{4.29} = 1.47'$$

$$M_x = \int y^2 \frac{dx}{2} = \int_0^{4.010} \left(1.69 - .143x^{1.779}\right)^2 \frac{dx}{2}$$

$$M_x = \int_0^{4.010} \left[1.69^2 - 2\times1.69 \times .143x^{1.779} + .143^2 x^{2(1.779)}\right] \frac{dx}{2}$$

$$M_x = \int_0^{4.010} \frac{1.69^2}{2}dx - \int_0^{4.010} 1.69(.143)x^{1.779}dx + \int_0^{4.010} \frac{.143^2}{2} x^{3.558}\,dx$$

$$M_x = \left[\frac{1.69^2 x}{2}\right]_0^{4.010} - \left[\frac{1.69\,(.143\,x^{2.779})}{2.779}\right]_0^{4.010} + \left[\frac{.143^2 x^{4.558}}{2\times4.558}\right]_0^{4.010}$$

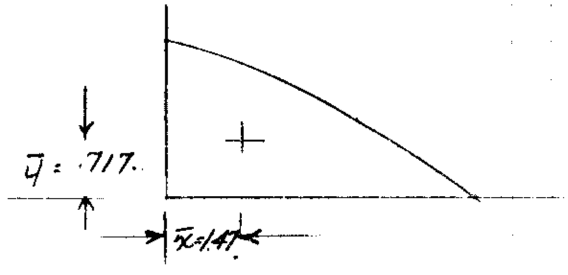
$$M_x = \frac{1.69^2}{2}(4.010) - \frac{1.69(.143)(4.010)^{2.779}}{2.779} + \frac{.143^2(4.010)^{4.558}}{2\times4\times4.558}$$

$$= 5.72 - .087(4.010)^{2.779} + .0023\,(4.010)^{4.558}$$

$$= 5.72 - 4.1 + 1.455$$

$$= 3.075$$

$$y = \frac{M_x}{A} = \frac{3.075}{4.29} = .717$$



SUBJECT Hall Meadow Brook Reservoir Reuben Hart Diversion

COMPUTATION

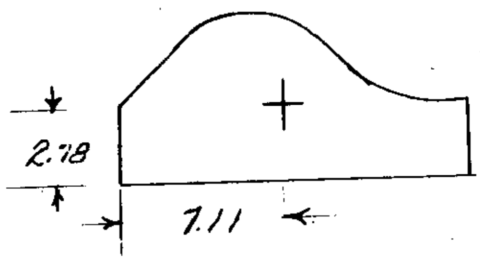
COMPUTED BY E. P. G. CHECKED BY GEH DATE April 14 1960

Concrete Only

Item	Factors	Wgt	H. Arm	H. Mom.	V. Arm	C.G. Mom.
C_1	.15 x 2.00 x 5.82	1.75	2.91	5.10	1.00	1.75
C_2	.15 x 4.77 x 4.77 x .5	1.70	3.12	5.30	3.59	6.10
C_3	.15 x 4.88 x 1.05	.77	5.30	4.08	4.44	3.42
C_4	.15 x 4.29	0.64	7.29	4.66	5.72	3.66
C_5	.15 x 5.31 x 4.01	3.20	7.82	25.00	2.66	8.51
C_6	.15 x 2.31 x 3.08 x ½	0.53	10.86	5.79	1.50	80
C_7	.15 x 3.00 x 3.08	1.37	11.37	12.85	3.77	5.16
C_8	.15 x 1.00 x 3.00 x ½	0.22	13.92	3.13	2.33	.51
C_9	.15 x 2.00 x 3.00	0.90	14.42	12.98	1.00	.90
		ΣV 11.08		ΣH_M = 78.89		ΣM_{CG} 30.81

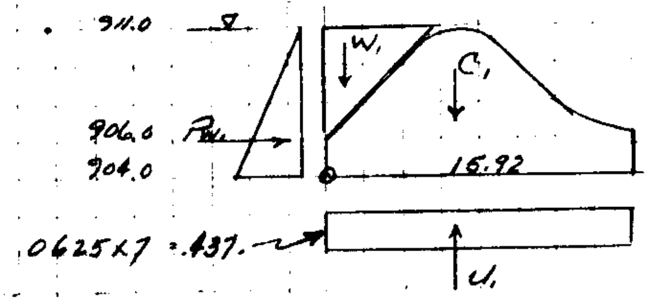
$$\frac{\Sigma M_H}{\Sigma V} = \frac{78.89}{11.08} = 7.11$$

$$\frac{\Sigma M_{CG}}{\Sigma V} = \frac{30.81}{11.08} = 2.78$$



SUBJECT HALL MEADOW BROOK RESERVOIR REUBEN HART DIVERSION
COMPUTATION STABILITY ANALYSIS WEIR SECTION
COMPUTED BY EPG CHECKED BY GFH DATE MAY 23 1960

CASE II NORMAL OPERATING CONDITION
NOTE: ICE PRESSURE NOT APPLICABLE AS THERE IS A CONSTANT FLOW OVER THIS WEIR.



Description		Vertical ↓	Vertical ↑	Horizontal →	Horizontal ←	Arm	Moment ↷	Moment ↶
C_1	5.00x5.00 (minor)	11.08					78.89	
W_1	.0625x4.77x4.77x½	.71				1.59	1.13	
U_1	.437 x 15.92		6.95			7.96		55.40
P_{W_1}	.0625x 7²x ½			1.53		2.33	3.56	
		11.79	6.95	1.53			83.58	55.40
		4.84		1.53			28.18	

$\frac{\Sigma H}{\Sigma V} = \frac{1.53}{4.84} = .316$

$\frac{\Sigma M}{\Sigma V} = \frac{28.18}{4.84} = 5.82$ $\qquad e = 7.96 - 5.82 = 2.14'$

$f = \frac{4.84}{15.92}\left(1 \pm \frac{6 \times 2.14}{15.92}\right) = .304(1 \pm .807)$

$f_{max} = .304(1.807) = 0.548$ k.s.f.

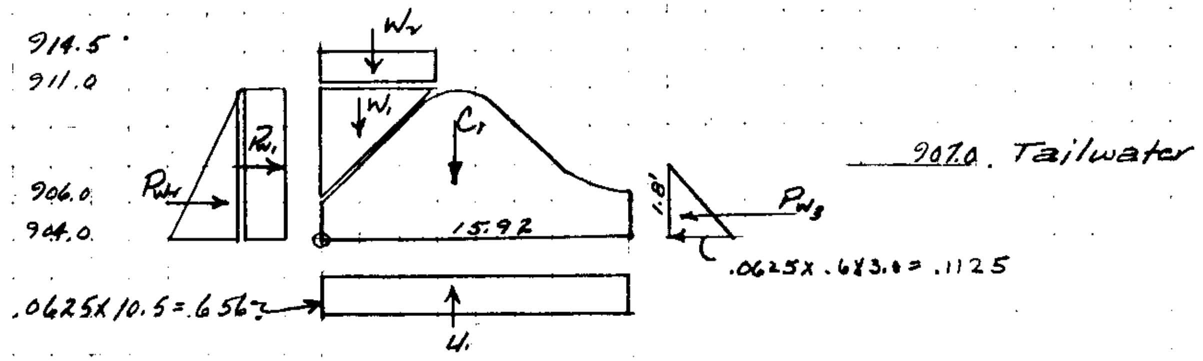
$f_{min} = .304(.193) = .059$ k.s.f.

SUBJECT: Hall Meadow Brook Reservoir — Reuben Hart Diversion

COMPUTATION: Stability Analysis Weir Section

COMPUTED BY: E.P.G. CHECKED BY: DATE: May 23, 1960

Case III Flood Condition. Crest Elev. 914.5



Description		Vertical ↓	Vertical ↑	Horizontal →	Horizontal ←	Arm	Moment ↻	Moment ↺
C_1		11.08					78.89	
W_1		.71				1.59	1.13	
W_2	.0625 × 4.77 × 3.5	1.04				2.38	2.48	
U_1	.656 × 15.92		10.45			7.96		83.20
P_{w1}	.0625 × 3.5 × 7.0			1.53		3.50	5.36	
P_{w2}	.0625 × 7 × 7 × ½			1.53		2.33	3.56	
P_{w3}	.1125 × 1.8 × ½				0.10	.6		.06
		12.83	10.45	3.06	.10		91.42	83.26
		2.38		2.96			8.16	

$\frac{\Sigma H}{\Sigma V} = \frac{2.95}{2.38} = 1.24$ High.

$\frac{\Sigma M}{\Sigma V} = \frac{8.16}{2.38} = 3.41 < 3.98 < 11.94$ check $e = 7.96 - 3.41 = 4.55$

$f = \frac{2.38}{15.92}\left(1 \pm \frac{6 \times 4.55}{15.92}\right) = .149\left(1 \pm 1.72\right)$

$f_{max} = .149 \times 2.72 = .405$ k.s.f.

$f_{min} = .149 \times -.72 = -.107$ k.s.f.

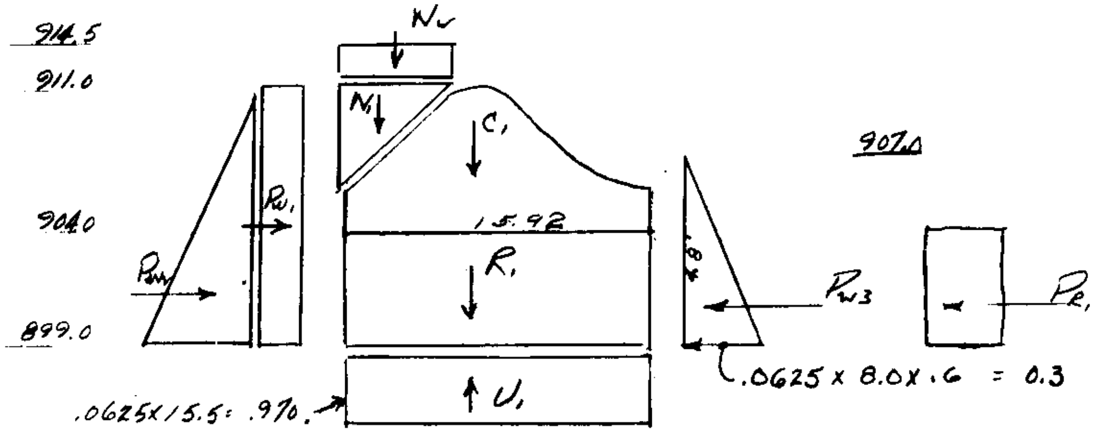
Sliding & location of resultant not satisfactory. Use section engaging rock with rock anchors.

SUBJECT HALL MEADOW BROOK RESERVOIR REUBEN HART DIVERSION

COMPUTATION STABILITY ANALYSIS WEIR SECTION

COMPUTED BY ______ CHECKED BY GFH DATE MAY 26 1960

CASE III FLOOD CONDITION CREST ELEV 914.5



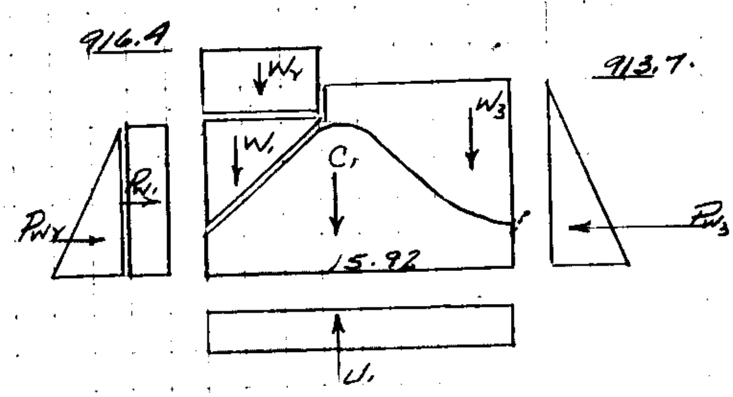
DESCRIPTION		VERTICAL ↓	VERTICAL ↑	HORIZONTAL →	HORIZONTAL ←	ARM	MOMENT →	MOMENT ←
C_1		11.08					78.89	
W_1		.71				1.59	1.13	
W_2	.0625 x 4.77 x 3.5	1.04				2.38	2.48	
R_1	.160 x 5 x 15.92	12.75				7.96	101.70	
U_1	.970 x 15.92		15.40			7.96		122.80
P_{W1}	.0625 x 3.5 x 12			2.62		6.00	15.72	
P_{WW}	.0625 x 12 x 12 x ½			4.50		4.00	18.00	
P_{W3}	0.3 x 4.8 x ½				.72	1.60		1.15
P_{R1}					6.40	2.50		16.00
		25.58	15.40	7.12	7.12		217.92	139.95
		10.18		—			77.97	

$\frac{\Sigma H}{\Sigma V} = \frac{0}{10.18} = 0$

$\frac{\Sigma M}{\Sigma V} = \frac{77.97}{10.18} = 7.66' > 3.98 < 11.94$ within mid 1/3 . OK.

SUBJECT: HALL MEADOW BROOK RESERVOIR — REUBEN HART DIVERSION
COMPUTATION: STABILITY ANALYSIS — WEIR SECTION
COMPUTED BY: E.P.G — CHECKED BY: GFH — DATE: MAY 23 1960

CASE III A FLOOD CONDITION — CREST ELEV 916.4



DESCRIPTION		VERTICAL ↓	VERTICAL ↑	HORIZONTAL →	HORIZONTAL ←	ARM	MOMENT →	MOMENT ←
C_1		11.08					78.89	
W_1		.71					1.13	
W_2	.0625 x 5.82 x 5.4	1.96				2.38	4.66	
W_3	.0625 x 10.10 x 7.7 x ½	2.43				13.55	32.93	
U_1	.775 x 15.92		12.35			7.96		98.30
P_{W1}	.0625 x 5.4 x 7			2.36		3.50	8.26	
P_{W2}	.0625 x 7.0^2 x ½			1.53		2.33	3.56	
P_{W3}	.364 x 9.7 x .6 x ½				1.06	1.94		2.06
		16.18	12.35	3.89	1.06		129.23	100.36
		3.83		2.83			28.87	

$\frac{\Sigma H}{\Sigma V} = \frac{2.83}{3.83} = .74$ HIGH — Rock anchors will take care of this.

$\frac{\Sigma M}{\Sigma V} = \frac{28.87}{3.83} = 7.53 > 5.31 < 10.62$ ok $e = 7.96 - 7.53 = .43$

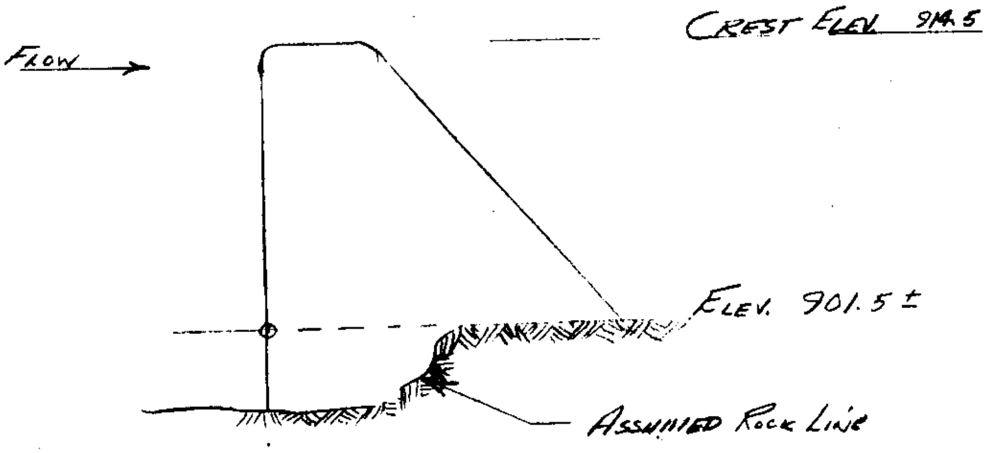
$f = \frac{3.83}{15.92}\left(1 \pm \frac{6 \times .43}{15.92}\right) = .241\left(1 \pm .16\right)$

$f_{max} = .241\,(1.16) = .280$ k.s.f.

$f_{min} = .241\,(.84) = .203$ k.s.f.

SUBJECT Hall Meadow Brook Reservoir
COMPUTATION Existing Reuben Hart Spillway with Modifications
COMPUTED BY E.P.G. CHECKED BY GFH DATE May 1960

Section C-C



Conditions to be Investigated

Case I: Construction Condition - no water in reservoir; no tailwater; wind load on downstream face. Modified section is stable by inspection.

Case II: Normal Operating Condition - pool @ el. 911 with ice pressure; no tailwater; full uplift over 100% of base.

Case III: Flood Discharge Condition — Reservoir at maximum flood pool elevation = 916.4; no tailwater; Full uplift over 100% of base.

Case IV: Construction Condition with Earthquake - Section is stable by inspection.

Case V: Normal Operating Condition with Earthquake Section is stable by inspection.

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION

CORPS OF ENGINEERS, U.S. ARMY

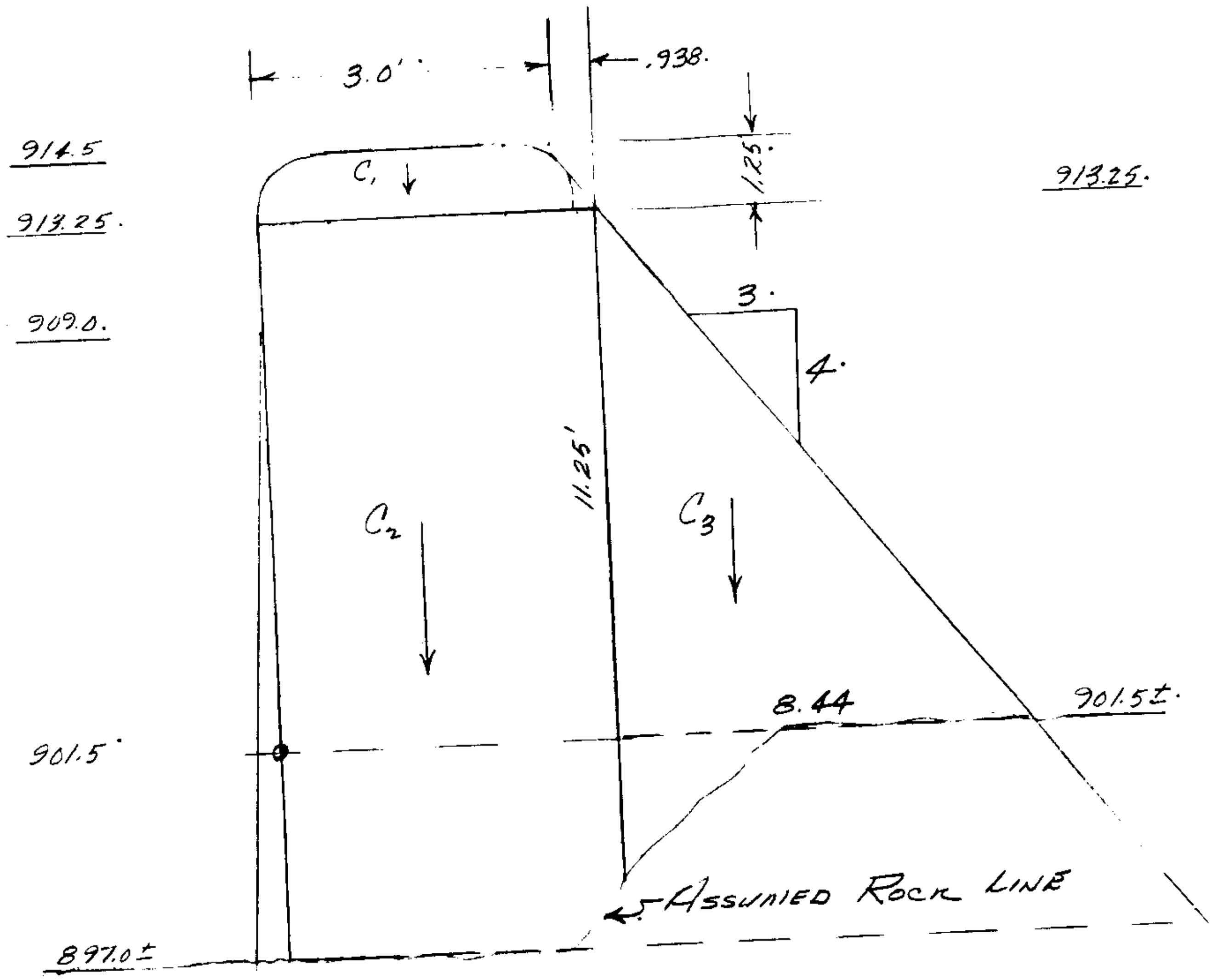
SUBJECT Hall Meadow Brook Reservoir

COMPUTATION Existing Reuben Hart Spillway with Modifications

DATE April 20 1960

COMPUTED BY E.P.G.

CHECKED BY GFH

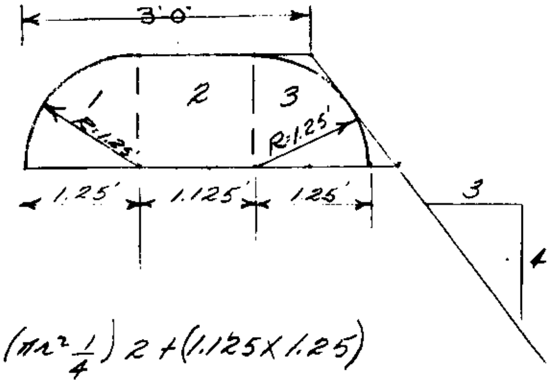


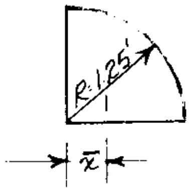
SUBJECT HALL MEADOW BROOK RESERVOIR

COMPUTATION EXISTING REUBEN HART SPILLWAY WITH MODIFICATIONS

COMPUTED BY E.P.G. CHECKED BY GFH DATE APRIL 23 1960

MODIFIED SPILLWAY CREST ONLY





$\bar{x} = \frac{4}{3}\frac{r}{\pi}$

$\bar{x} = \frac{4}{3} \times \frac{1.25}{3.14} = .53'$

$A: (\pi r^2 \frac{1}{4})\, 2 + (1.125 \times 1.25)$

$A: 3.14 \times \overline{1.25}^2 \times \frac{1}{4} \times 2 + 1.125 \times 1.25$

$A: 2.45 + 1.41 = 3.86$ s.f.

ITEM	FACTORS	WEIGHT	H. ARM	H. MOM.	V. ARM	C.G. MOM.
1	.15 × 1.22	.18	.72	.13	.53	.10
2	.15 × 1.41	.22	1.81	.40	.62	.14
3	.15 × 1.22	.18	2.90	.52	.53	.10
		.58		1.05		.34

$\frac{\Sigma Hm}{\Sigma V} = \frac{1.05}{.58} = 1.81'$

$\frac{\Sigma M_{CG}}{\Sigma V} = \frac{.34}{.58} = .59'$

CONCRETE ONLY

		WEIGHT	H. ARM	H. MOM.	V. ARM.	C.G. MOM.
C_1 CREST	.15 × 3.86	.58	1.81	1.05	12.34	7.16
C_2	.15 × 11.75 × 3.94	6.91	1.96	13.54	5.88	40.63
C_3	.15 × 11.25 × 8.44 × ½	7.12	6.73	47.92	5.75	40.94
		14.61		62.51		88.73

$\frac{\Sigma MH}{\Sigma V} = \frac{62.51}{14.61} = 4.28'$

$\frac{\Sigma M_{CG}}{\Sigma V} = \frac{88.73}{14.61} = 6.07'$ $\quad e = 6.18 - 4.28 = 1.90'$ O.K.

$f = \frac{14.61}{12.36}\left(1 \pm \frac{6 \times 1.90}{12.36}\right) = 1.18\,(1 \pm .925)$

$f_{MAX} = 1.18(1.925) = 2.27$ ksf

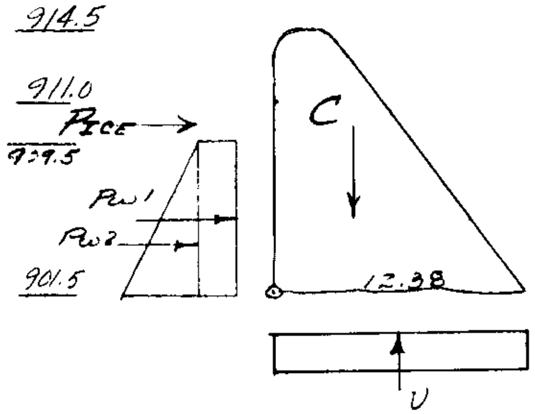
$f_{MIN} = 1.18(.075) = .085$ ksf

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U. S. ARMY

SUBJECT HALL MEADOW BROOK RESERVOIR
COMPUTATION EXISTING REUBEN HART SPILLWAY WITH MODIFICATIONS
COMPUTED BY E.P.G. CHECKED BY GFH DATE APRIL 23 1960

CASE II NORMAL OPERATING CONDITION WITH ICE WATER LEVEL 911.0



		Force-Kips	Arm	Moment
C		↓ 14.61		+ 62.51
U	.0625 x 9.5 x 12.38	↑ 7.74	6.19	− 47.90
Ice		→ 7.50	9.0	+ 67.50
Pw1	.0625 x 1.5 x 8.0	→ 0.75	4.0	+ 3.00
Pw2	.0625 x 8.0 x 4.0	→ 2.00	2.67	+ 5.33
		ΣV = 6.87	ΣM =	90.44
		ΣH: 10.25		

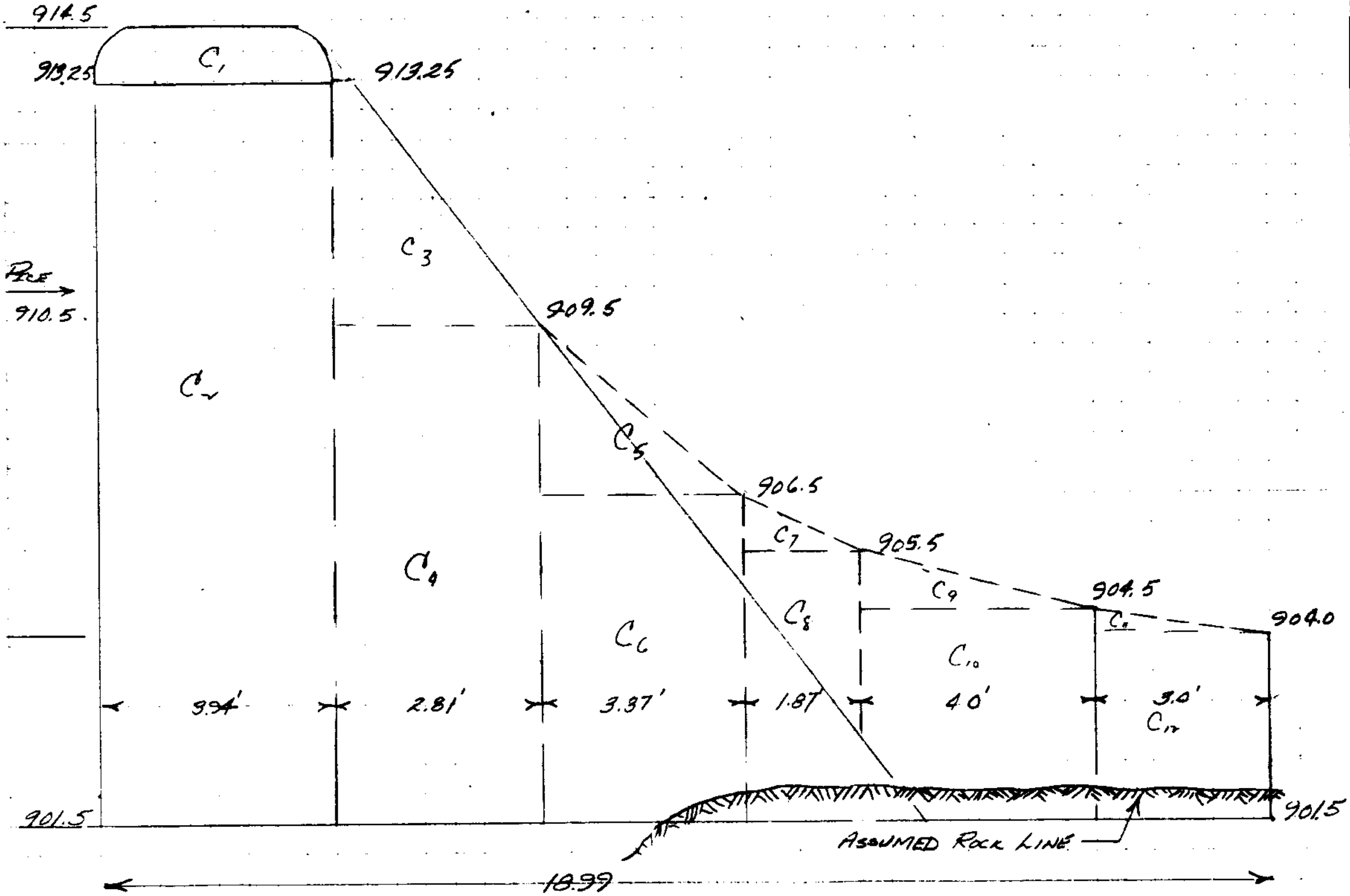
$\frac{\Sigma M}{\Sigma V} = \frac{90.44}{6.87} = 13.15'$ Unstable

See new section with bucket type spillway on page

FORM 223
Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U. S. ARMY

SUBJECT Hall Meadow Brook Reservoir
COMPUTATION Existing Reuben Hart Spillway
COMPUTED BY E.P.G. CHECKED BY GFH DATE May 13 1960



NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION

CORPS OF ENGINEERS, U. S. ARMY

SUBJECT HALL MEADOW BROOK RESERVOIR

COMPUTATION EXISTING REUBEN HART SPILLWAY WITH MODIFICATIONS

COMPUTED BY E.P.G. CHECKED BY GFH DATE MAY 12 1960

CONCRETE ONLY WITH BUCKET SPILLWAY

		WEIGHT	H ARM	H MOM.	V ARM	M.C.G.
C_1	.150 X 3.86	.58	1.81	1.05	12.34	7.16
C_2	.150 X 3.94 X 11.75	6.94	1.96	13.60	5.87	40.74
C_3	.150 X 2.81 X 3.75 X ½	.79	4.88	3.86	9.25	7.31
C_4	.150 X 7.81 X 8.0	3.37	5.34	18.00	4.00	13.48
C_5	.150 X 3.37 X 3.00 X ½	.76	7.87	5.98	6.00	4.56
C_6	.150 X 3.37 X 5.00	2.53	8.43	21.33	2.50	6.33
C_7	.150 X 1.87 X 1.0 X ½	.14	10.74	1.50	4.33	.61
C_8	.150 X 1.87 X 4.0	1.12	11.06	12.39	2.00	2.24
C_9	.150 X 4.0 X 1.0 X ½	.30	13.42	4.02	3.33	1.00
C_{10}	.150 X 4.0 X 3.0	1.80	14.09	25.36	1.50	2.70
C_{11}	.150 X 3.0 X 5 X ½	.11	17.09	1.88	2.67	.29
C_{12}	.150 X 3.0 X 2.5	1.13	17.59	19.88	1.25	1.41
		19.57		128.85		87.83

$\frac{\Sigma MH}{\Sigma V} = \frac{128.85}{19.57} = 6.58$

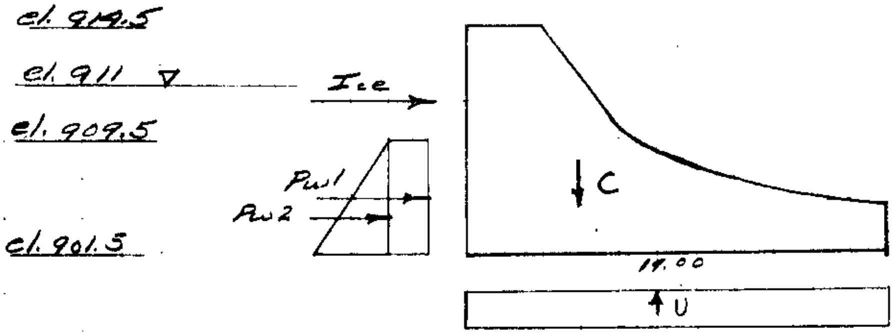
$\frac{\Sigma MCG}{\Sigma V} = \frac{87.83}{19.57} = 4.49$

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U. S. ARMY

SUBJECT Hall Meadow Brook Reservoir
COMPUTATION Existing Reuben Hart Spillway - Section C-C
COMPUTED BY E. P. G. CHECKED BY GFH DATE May 13 1960

Case II



Case II Normal Operating Condition with Ice

C		↓ 19.57		+ 128.85
U	.0625 x 9.5 x 19.0	↑ 11.33	9.50	− 108.00
P_{w1}	.0625 x 1.5 x 8.0	→ 0.75	4.0	+ 3.00
P_{w2}	.0625 x 8.0 x 4.0	→ 2.00	2.67	+ 5.33
Ice		→ 7.50	9.0	+ 67.50
	ΣV =	8.24	ΣM =	+ 96.68
	ΣH =	10.25		

$\frac{\Sigma H}{\Sigma V} = \frac{10.32}{8.24} = 1.25$

$\frac{\Sigma M}{\Sigma V} = \frac{96.68}{8.24} = 11.72$ within mid 1/3 $e = 9.50 - 11.72 = 2.22$

$$S_{s.f} = \frac{.65(\Sigma V) + .5 \times .4 \times 144\,(\text{Base})}{10.32}$$

$$= \frac{.65\,(8.24) + .5 \times .4 \times 144\,(18.99)}{10.32}$$

$$= \frac{5.35 + 545}{10.32} = 53.5 \quad \text{ok}$$

$$f = \frac{8.24}{19.0}\left(1 \pm \frac{6 \times 2.22}{19}\right) = .434\,(1 \pm .701)$$

$f_{max} = .434 \times 1.701 = 0.739$ ksf

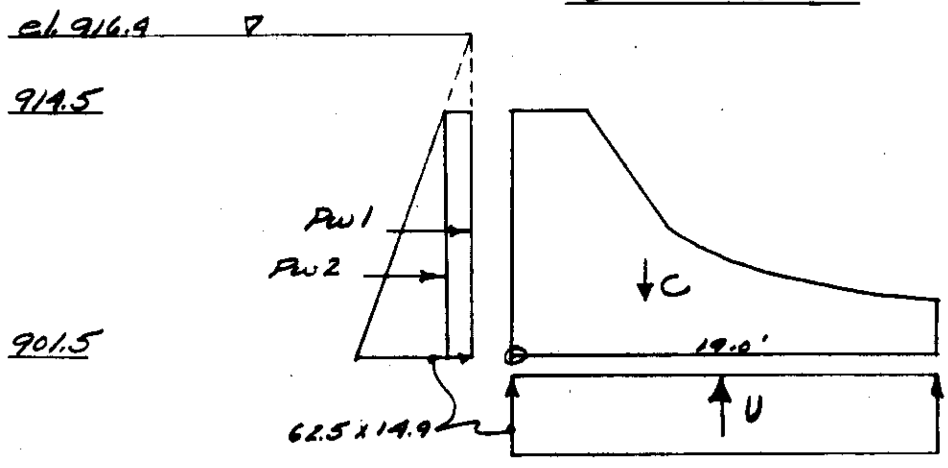
$f_{min} = .434 \times 0.299 = 0.130$ ksf

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U. S. ARMY

SUBJECT Hall Meadow Brook Reservoir - Reuben Hart Diversion
COMPUTATION Section C-C, - Stability
COMPUTED BY GFH CHECKED BY EPG DATE 24 May 1960

Case III



Unit	Factors	Force - Pounds			Arm	Moment	
		↓	↑	→	Ft	↷	↶
C		19,570				128,850	
U	62.5 x 14.9 x 19.0		17,700		9.5		168,000
Pw1	62.5 x 1.9 x 13.0			1542	6.5	10,020	
Pw2	62.5 x 13.0 x 6.5			5280	4.33	22,850	
		19570	17700	6,822		161,720	168,000
		17700					

ΣV = 1,870 ΣH

Resultant falls outside heel creating tension @ toe.

Design for tension @ toe:

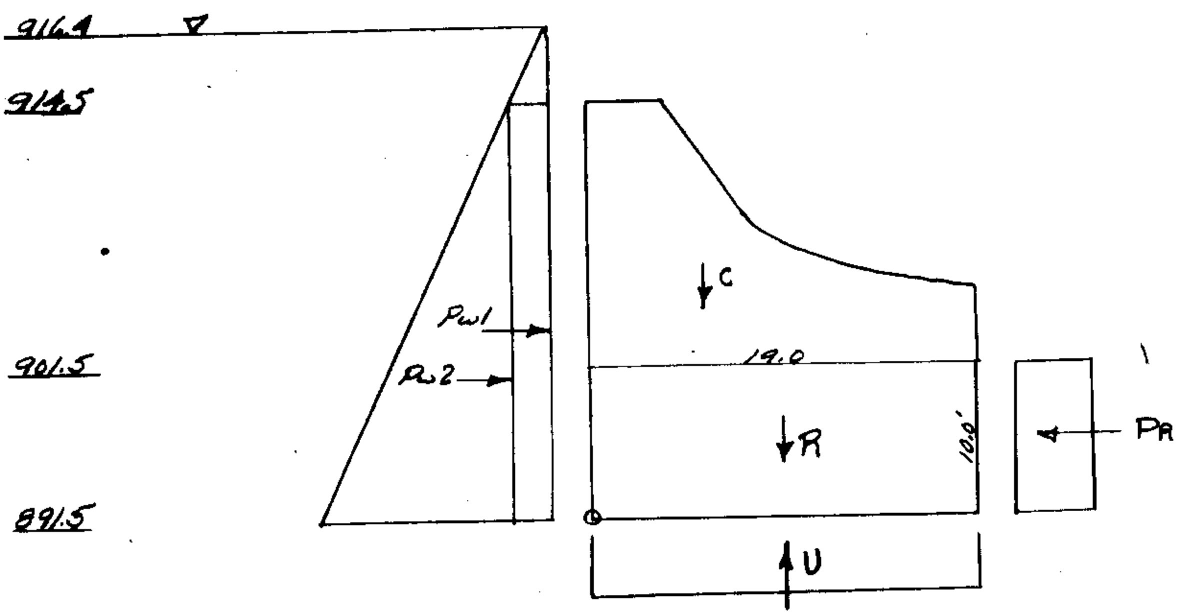
$\Sigma V = 1{,}870^{\#}$ $e = \dfrac{168{,}000 - 128{,}850}{1{,}870} = -20.9'$ from heel

$M = 32{,}870'^{\#}$ $M_s = (1{,}870 \times 38.9) - 32{,}870 = 39{,}830$

$A_s = \dfrac{39{,}830}{20{,}000 \times .866 \times 18} - \dfrac{1870}{20{,}000} = 0.128 - 0.093 = 0.035 \text{ in}^2$

$v = \dfrac{6822}{12 \times .866 \times 17 \times 12} = 3$ psi O.K.

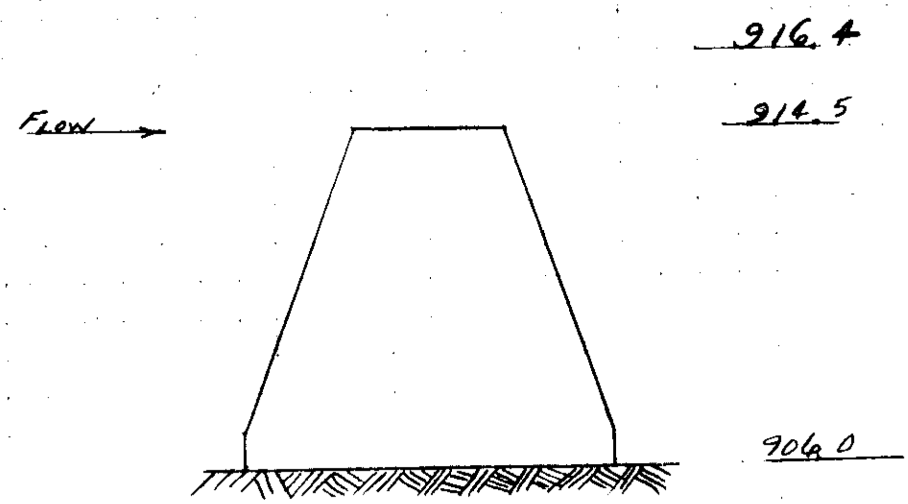
SUBJECT Hall Meadow Brook Reservoir - Reuben Hart Diversion
COMPUTATION Section C-C - Stability
COMPUTED BY GEH CHECKED BY ______ DATE 27 May 1960



Unit	Factors	Force - Pounds ↓	↑	→	Arm Ft	Moment ↷	↶
C		19570				128850	
R	160 x 10 x 19	30400			9.5	289000	
U	62.5 x 24.9 x 19.0		29600		9.5		281,000
Pw1	62.5 x 1.9 x 23.0			2730	11.5	31400	
Pw2	62.5 x 23² x 1/2			16540	7.66	126800	
PR				-19270	5		96,350
		49970		0		576,050	377,350
		29600		ΣH		377,350	
	ΣV =	20,370			ΣM =	198,700	

$$\frac{\Sigma M}{\Sigma V} = \frac{198700}{20370} = 9.75'$$ within mid 1/3 OK.

SUBJECT HALL MEADOW BROOK RESERVOIR
COMPUTATION AUXILIARY SPILLWAY SECTION E-E
COMPUTED BY E.P.G. CHECKED BY GFH DATE MAY 6 1960



CONDITIONS TO BE INVESTIGATED

CASE I CONSTRUCTION CONDITION
RESERVOIR DEWATERED
THIS CONDITION STABLE BY INSPECTION

CASE II NORMAL OPERATING CONDITION
RESERVOIR AT ELEV. 916.0
NO TAILWATER
100% UPLIFT ON ENTIRE BASE
ICE PRESSURE OF 7.5 KIPS

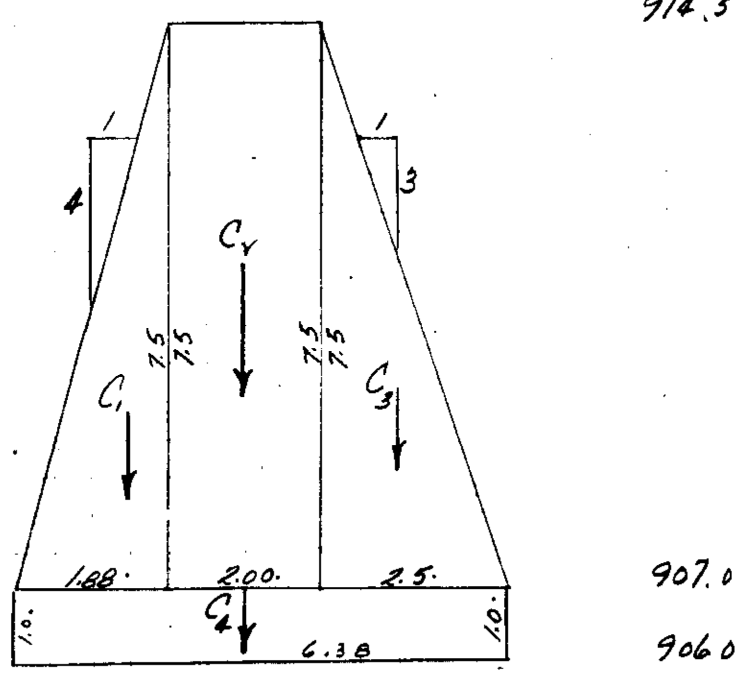
CASE III FLOOD DISCHARGE CONDITION
POOL AT MAXIMUM FLOOD ELEVATION 916.4
NO TAILWATER
100% UPLIFT ON ENTIRE BASE

CASE IV CONSTRUCTION CONDITION WITH EARTHQUAKE
FROM CASE I STABLE BY INSPECTION

CASE V NORMAL OPERATING CONDITION WITH EARTHQUAKE
FROM CASE II STABLE BY INSPECTION

SUBJECT Hall Meadow Brook Reservoir
COMPUTATION Auxiliary Spillway Section E-E
COMPUTED BY E.P.G. CHECKED BY GFH DATE May 10 1960

Concrete Only



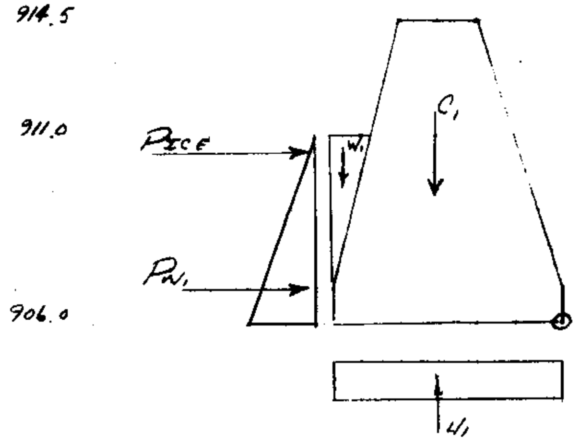
Description		Weight	H. Arm	H. Mom	V. Arm	M. C. G.
C_1	.150 X 1.88 X 7.5 X ½	↓ 1.06	1.26	1.34	3.50	3.71
C_V	.150 X 2.00 X 7.5	↓ 2.25	2.88	6.48	4.75	10.69
C_3	.150 X 2.50 X 7.5 X ½	↓ 1.41	4.71	6.64	3.50	4.94
C_4	.150 X 6.38 X 1.0	↓ .96	3.19	3.06	.5	.48
		5.68		17.52		19.82

$$\frac{\Sigma MH}{\Sigma V} = \frac{17.52}{5.68} = 3.08$$

$$\frac{\Sigma MCG}{\Sigma V} = \frac{19.82}{5.68} = 3.49$$

SUBJECT Hall Meadow Brook Reservoir Reuben Hart Diversion
COMPUTATION Auxiliary Spillway Section E-E
COMPUTED BY EPG CHECKED BY GFH DATE May 23 1960

Case II



Normal Operating Condition with Ice

Description		Vertical ↓	Vertical ↑	Horizontal →	Horizontal ←	Arm	Moment →	Moment ←
C		5.68				3.30		18.80
W_1	$.0625 \times 1.0 \times 4.0 \times \frac{1}{2}$	.13				6.05		.79
P_{W_1}	$.0625 \times 5^2 \times \frac{1}{2}$			.78		1.67	1.30	
P_{ICE}	7.5			7.50		4.50	33.70	
U_1	$.0625 \times 5.0 \times 6.38$		1.95			3.19	6.22	
		5.81	1.95	8.28	0.0		41.22	19.59
		3.86		8.28			21.63	

$\Sigma M \div \Sigma V = 21.63 \div 3.86 = 5.6'$ outside mid ½, therefore

$M = 35.00'^{k}$ Design for tension anchors & CANTILEVER SECTION FROM ROCK BASE

$M_N = 19.59 - 6.22 = 13.37'^{k}$

$N = 5.81 - 1.95 = 3.86^{k}$ EMBED ROCK ANCHORS 8' TO DEVELOP

$d = 6.38 - 0.50 = 5.88'$ FULL USE OF THE ROCK BASE.

$\frac{13.37}{3.86} = 3.46'$ $e = 5.88 - 3.46 = 2.42$

$M_S = 35.00 + (3.86 \times 2.42) = 44.35$

$A_S = \frac{44.35}{20 \times .866 \times 5.88} - \frac{3.86}{20} = 0.436 - 0.193 = 0.243\ in^2/FT.$

#11 Bars 6'0" o.c. $= 1.56\ in^2/6'$ $\frac{1.56}{6} = 0.26\ in^2/FT.$

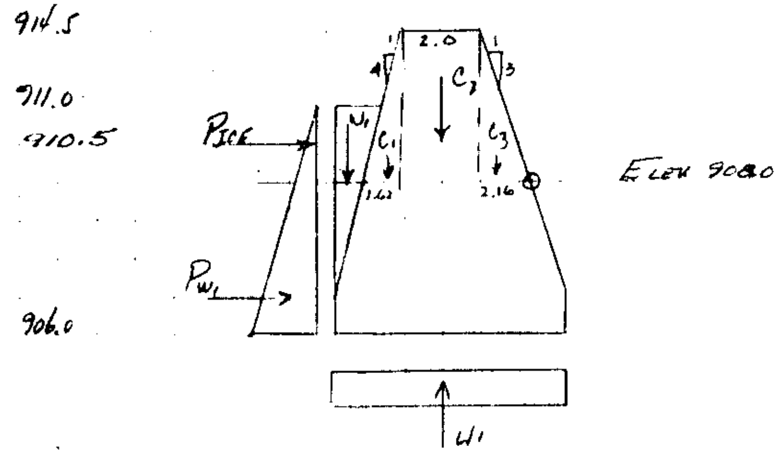
$v = \frac{8280}{12 \times .866 \times 5.88 \times 12} = 11\ psi$

SUBJECT HALL MEADOW BROOK RESERVOIR   REUBEN HART DIVERSION

COMPUTATION

COMPUTED BY ______ CHECKED BY GFH DATE MAY 23 1960

CASE II



NORMAL OPERATING CONDITION WITH ICE   SECTION AT ELEV 908.0

DESCRIPTION		VERTICAL ↓	VERTICAL ↑	HORIZONTAL →	HORIZONTAL ←	ARM	MOMENT →	MOMENT ←
C_1	.150 X 6.5 X 1.62 X ½	.79				4.70		3.71
C_2	.150 X 2.0 X 6.5	1.95				3.16		6.16
C_3	.150 X 6.5 X 2.16 X ½	1.05				1.44		1.51
P_{ice}	7.5			7.5		2.5	18.80	
		3.79		7.5	—		18.80	11.38
		3.79		7.5			7.31	

$M = 18.80'k$
$M_N = 11.38'k$
$N = 3.79$

$\frac{11.38}{3.79} = 3.15$ $e = 5.29 - 3.15 = 2.14'$

$M_S = 18.80 + (3.79 \times 2.14) = 26.91$ (8.11)

$A_S = \frac{26.91}{20 \times .866 \times 5.29} - \frac{3.79}{20} = .293 - .189 = 0.104 \text{ IN}^2/\text{FT}$

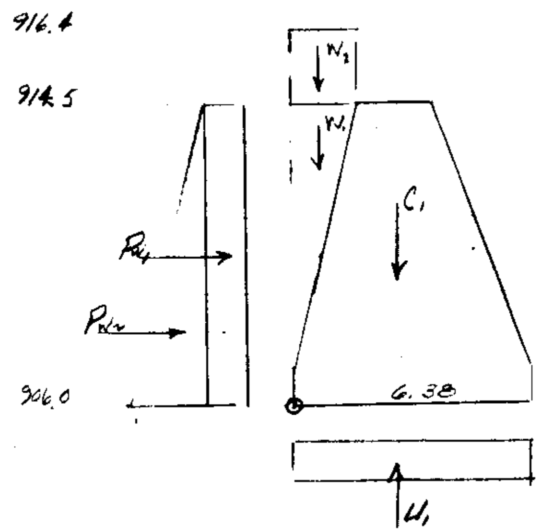
#8 AT 6'-0" O.C. = .79"/6' $\frac{.79}{6} = .013 \text{ IN}^2/\text{FT}$

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U. S. ARMY

SUBJECT Hall Meadow Brook Reservoir / Reuben Hart Diversion
COMPUTATION Stability Analysis Section E-E Auxiliary Spillway
COMPUTED BY E.P.G. CHECKED BY GFH DATE May 23, 1960

Case III



Description		Vertical ↓	Vertical ↑	Horizontal →	Horizontal ←	Arm	Moment →	Moment ←
C		5.68				3.08	17.49	
W_1	.0625×1.88×7.5×½	.44				.63	.28	
W_2	.0625×1.88×1.9	.22				.94	.21	
U_1	.0675×10.4×6.38		4.15			3.19		13.25
P_{W_1}	.0625×1.9×7.5			.89		3.75	3.34	
P_{W_2}	.0625×7.5×7.5×½			1.76		2.50	4.40	
		6.34	4.15	2.65	—		25.72	13.25
		2.19		2.65			12.47	

$\frac{\Sigma H}{\Sigma V} = \frac{2.65}{2.19} = 1.21$ High

$\frac{\Sigma M}{\Sigma V} = \frac{12.47}{2.19} = 5.70$ High e = 3.19 - 5.70 = -2.51 > 1.59

5.70 > 1.59 > 4.79 High

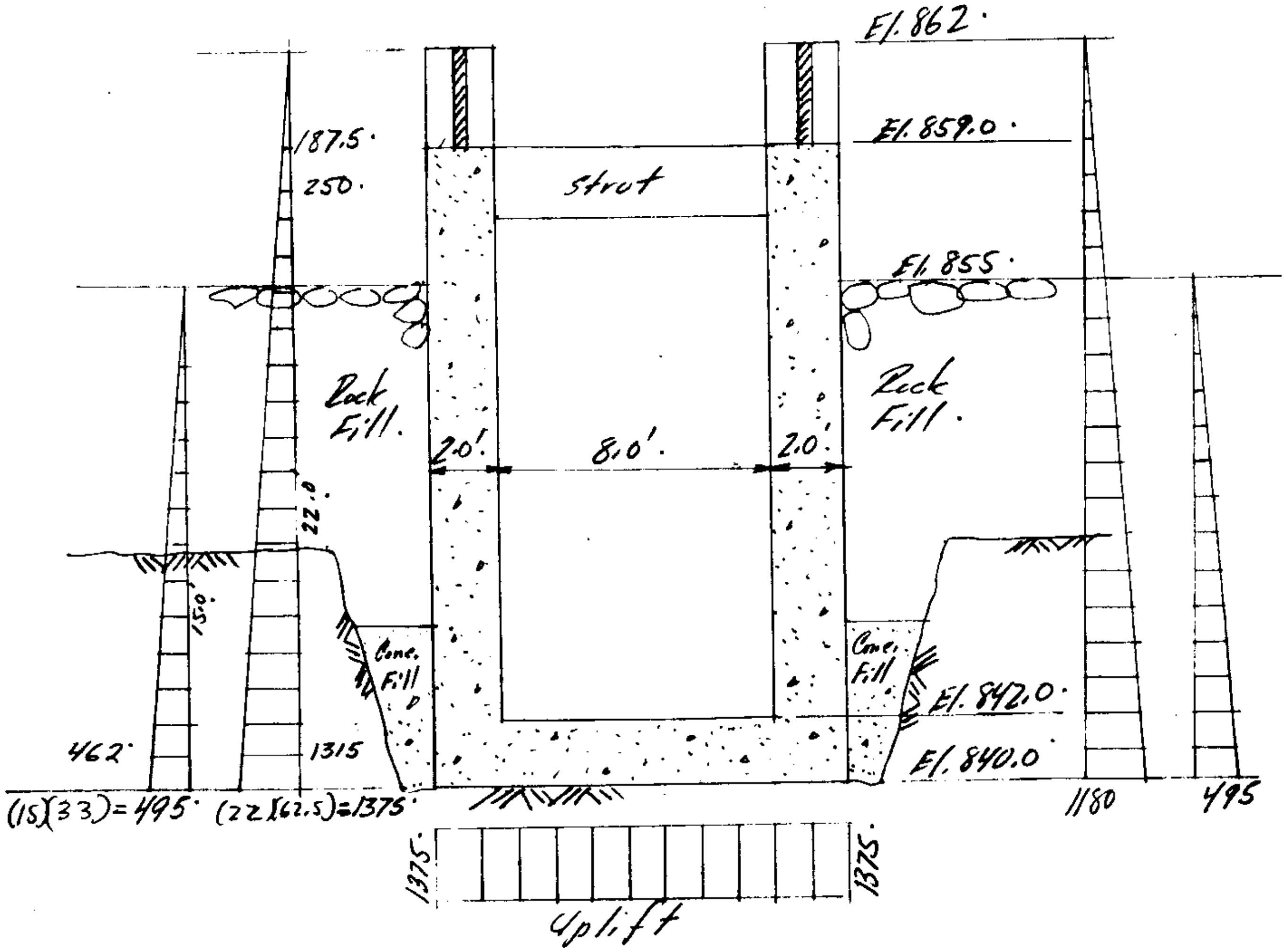
THE CONDITION WITH THE RESERVOIR AT ELEV. 911.0 & WITH ICE IS MORE CRITICAL THAN THE ABOVE CONDITION. THE SECTION HAS BEEN DESIGNED WITH REINFORCEMENT FOR THE MORE CRITICAL CONDITION.

SUBJECT Hall Meadow Brook Dam.
COMPUTATION Inlet Structure.
COMPUTED BY ISM. CHECKED BY C.C.C. DATE April 15, 1960

Transverse Section Thru Inlet Structure:

Design Assumption: Water to top of wall El. 862.0 with full uplift on base.

Equiv. liq. Pres. (Rock) = (0.45)(73) = 33 #/□' (S.b.)



NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION

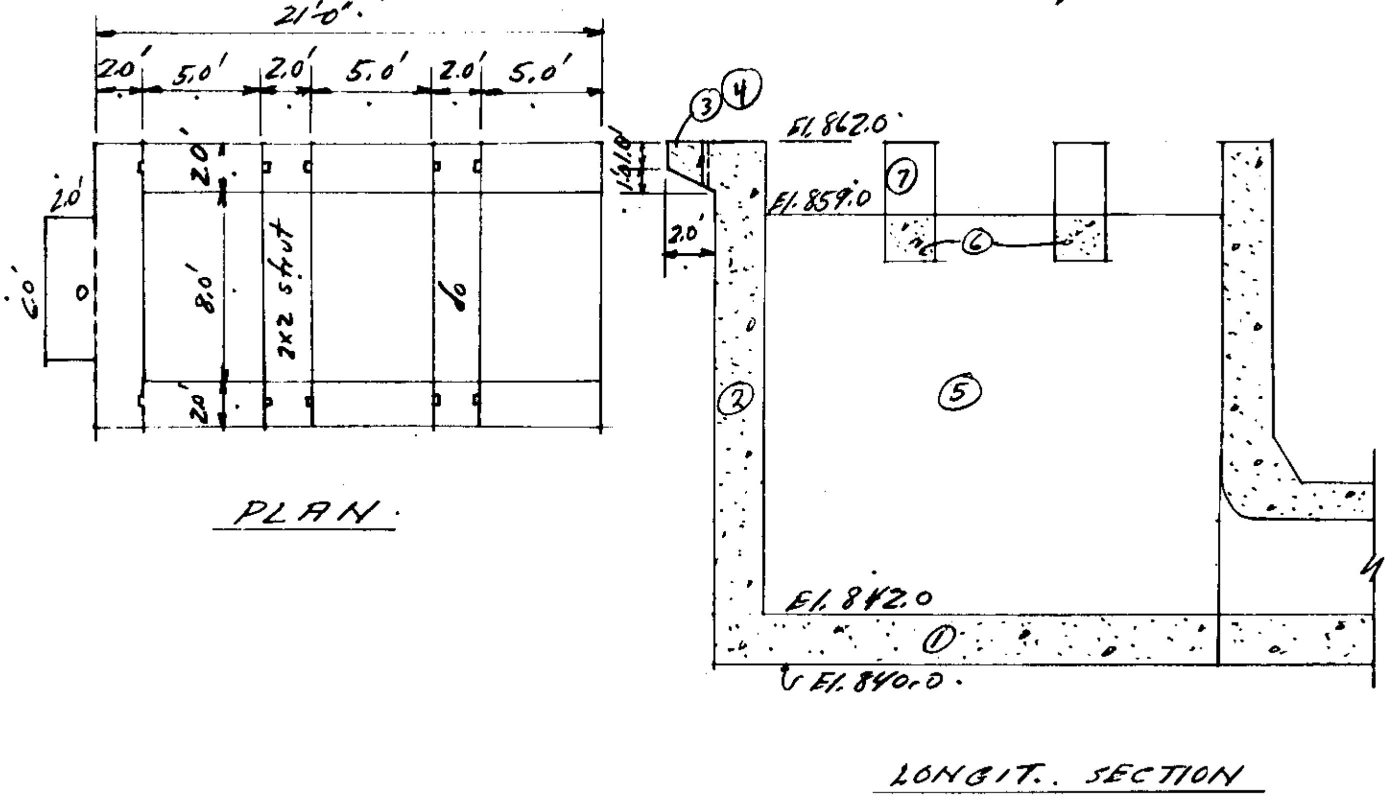
CORPS OF ENGINEERS, U. S. ARMY

SUBJECT Hall Meadow Brook Dam.

COMPUTATION Inlet Structure.

COMPUTED BY JSM. CHECKED BY C.C.C. DATE April 15, 1960

Check intake structure for flotation, Sta. 2+17 to Sta. 2+38: Assume pool at El. 862.0 with full uplift on base.



①	(2.0)(21.0)(12.0)(.150)	= 75,700#
②	(2.0)(20.0)(12.0)(.150)	= 72,000
③	(2.0)(6.0)(1.0)(.150)	= 1,800
④	1/2(1.0)(2.0)(6.0)(.150)	= 900
⑤	(2.0)(19.0)(17.0)(.150) × 2	= 194,000
⑥	(2.0)(2.0)(8.0)(.150) × 2	= 9,600
⑦	(2.0)(2.0)(3.0)(.150) × 4	= 7,200
		361,200# ↓

Uplift

= (12.0)(21.0)(22.0)(62.5)

= 347,000# ↑

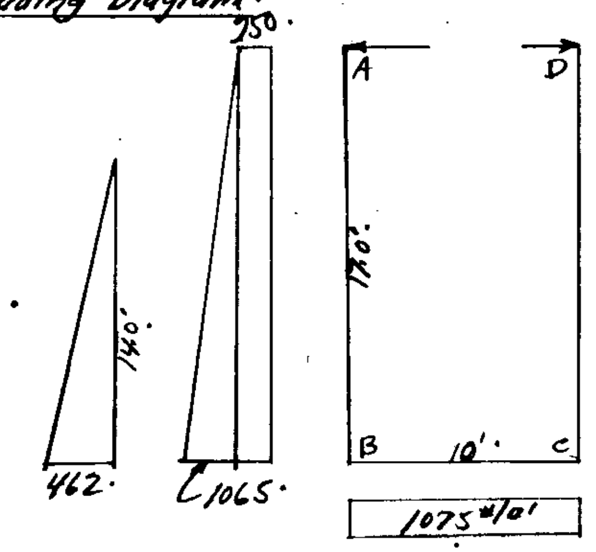
Ok with anchors.

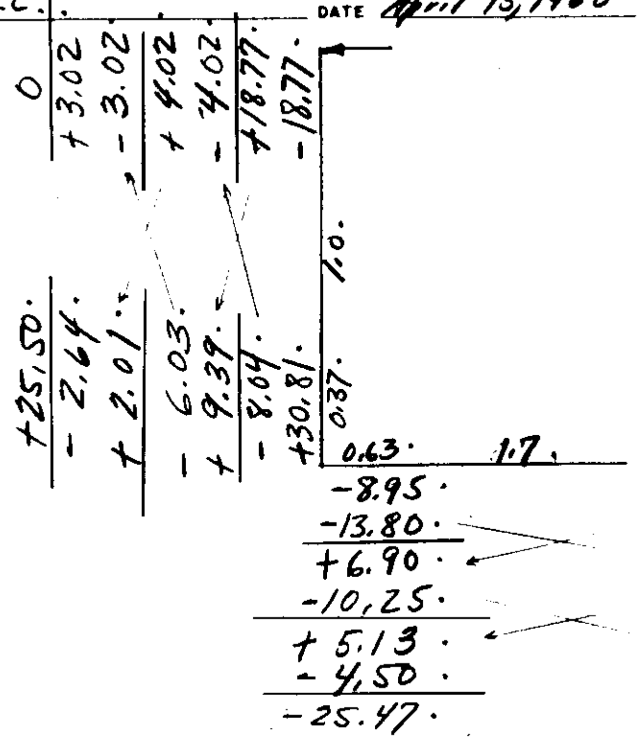
SUBJECT *Hall Meadow Brook Dam*

COMPUTATION *Inlet Structure.*

COMPUTED BY *ISM.* CHECKED BY *C.C.C.* DATE *April 15, 1960*

Loading Diagram:





F.E.M's:

$$AB = \frac{(250)(\overline{17.0}^2)}{12} + \frac{1}{15}\left(\frac{1}{2}\right)(1065)(17)(17) + \frac{\overline{0.823}^2(5-3\times0.823)}{30}\times\frac{1}{2}(462)(14)(14)$$

(2.53 written above $5-3\times0.823$)

$$= 6{,}000 + 10{,}250 + 2{,}520 = \underline{18.77}^{K}$$

$$BA = 6{,}000 + \frac{15}{10}(10{,}250) + \frac{0.823(10-8.23+3\times\overline{0.823}^2)}{30}\times\frac{1}{2}(462)(\overline{14}^2)$$

(3.80 and 2.03 written above; 4700, 26.1 written below)

$$= 6{,}000 + 15{,}400 + 9{,}410 = \underline{30.81}^{K}$$

(4.7 written below 9,410) [Error on safe side - Neglect.]

$$BC = CB = \frac{(1075)(\overline{10.0}^2)}{12} = \underline{8950}^{\#}$$

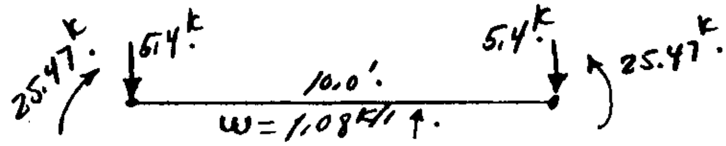
Stiff. factors:

$$BC = \frac{\overline{1.0}^3}{10} = 0.1 / 0.059 = 1.7$$

$$BA = \frac{\overline{1.0}^3}{17.0} = 0.059/0.059 = 1.0$$

SUBJECT Hall Meadow Brook Dam.
COMPUTATION Inlet Structure.
COMPUTED BY JSM. CHECKED BY C.C.C. DATE April 20, 1960

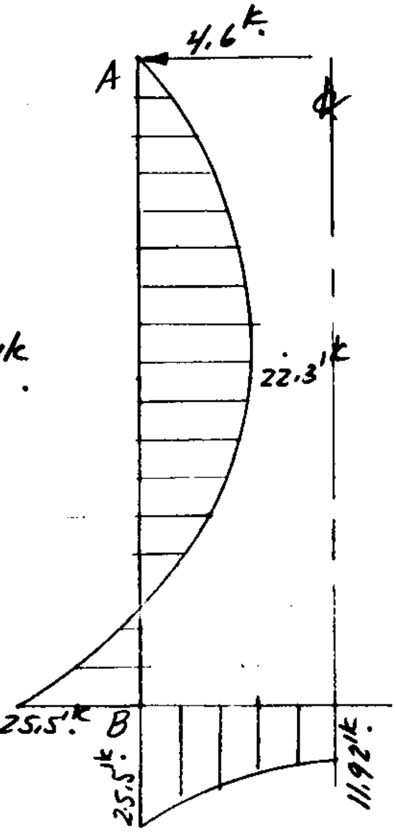
Member B-C:



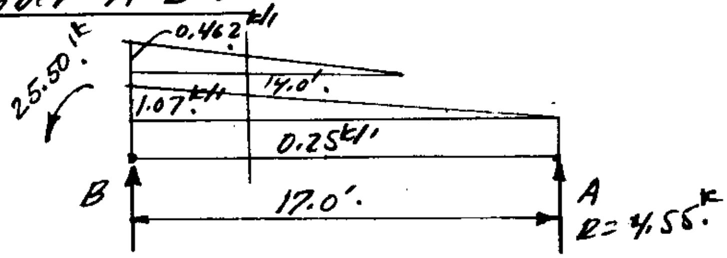
$R = (1.08)(5.0) = 5.40^k$

$M@℄ = (-5.4)(5.0) + (1.08)\left(\frac{5.0^2}{2}\right) + 25.47$

$= -27.0 + 13.45 + 25.47 = +\underline{11.92}'^k$



Member A-B:



Find R_A:

$(0.25)\left(\frac{17.0^2}{2}\right) + \frac{1}{2}(1.07)\left(\frac{17.0^2}{3}\right) + \frac{1}{2}(0.462)\left(\frac{14.0^2}{3}\right) - 25.50 - 17.0R_A = 0$

$36.2 + 51.6 + 15.1 - 25.50 - 17.0R_A = 0$

$R_A = \underline{4.55}^k$

M @ 5'0" From "B":

$= (0.25)\left(\frac{12.0^2}{2}\right) + \frac{1}{2}(0.766)\left(\frac{12.0^2}{3}\right) + \frac{1}{2}(0.297)\left(\frac{9.0^2}{3}\right) - (4.55)(12.0)$

$= 18.0 + 18.2 + 4.0 - 54.6 = \underline{-14.4}'^k$

M @ 8'6" From "B":

$= (0.25)\left(\frac{8.5^2}{2}\right) + \frac{1}{2}(0.54)\left(\frac{8.5^2}{3}\right) + \frac{1}{2}(0.18)\left(\frac{5.5^2}{3}\right) - (4.55)(8.5)$

$= 9.0 + 6.5 + 0.91 - 38.7 = \underline{-22.3}'^k$

SUBJECT Hall Meadow Brook Dam.
COMPUTATION Inlet Structure.
COMPUTED BY ISM CHECKED BY C.C.C. DATE April 21, 1960

Design of Members:

Allowable stresses:

$f'_c = 3,000$, $r = 90$, $K = 160$

$f_c = 1,050$ $u = 210$ (Top bars) $j = 0.885$

$f_s = 20,000$ $u = 300$ (all others)

Member B-C & B-A:
$-M = 25.5^{1k}$

$Req'd\ d_m = \sqrt{\frac{25.5}{0.160} = 160} = 12.7''$ have $24 - 4.5 = 19.5''$

$A_s = \frac{(25.5)(12)}{(20)(0.885)(19.5)} = \underline{0.89}^{\square''}$ #7@8" cut off 3'0"

M_{BA} @ 2'0" above base = 14.0^{1k}

$A_s = \frac{14}{25.5}(0.89) = 0.49^{\square''}$ #7@12" to El. 849.5 + 30D
#6@12" remainder

$+M_{BA} = 22.3^{1k}$

$A_s = \frac{(22.3)(12)}{(20)(0.885)(19.5)} = \underline{0.78}^{\square''}$ #7@9"

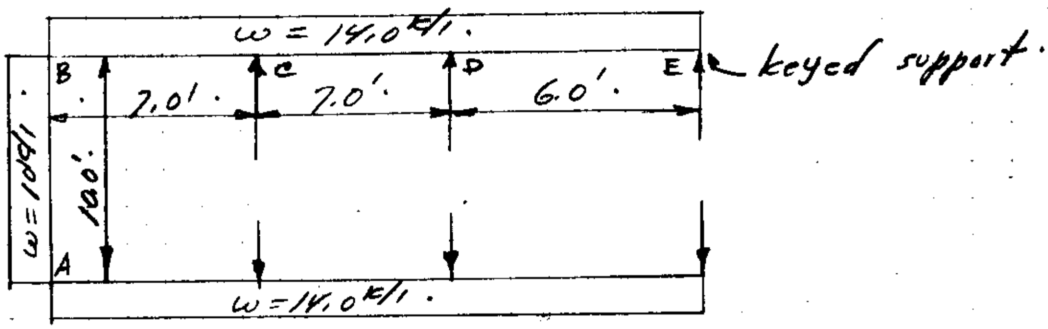
SUBJECT Hall Meadow Brook Dam.
COMPUTATION Inlet Structure.
COMPUTED BY JSM. CHECKED BY C.C.C. DATE April 23 1960

Design For Ice Pressure:

Assume pool at El. 859.0 with an ice pressure of 10k/l applied at El. 858.0.

Top two feet of structure to be designed as a horiz. frame.

w @ side walls = 10.0 + 14/17(4.6) = 10.0 + 3.8 = say 14 k/l



F.E.M's:

BC = (14.0)(7.0²)/12 = 57.2 ¹ᵏ

DE = (14.0)(6.0²)/12 = 42.0 ¹ᵏ

AB = (10.0)(10.0²)/12 = 83.3 ¹ᵏ

Stiff. Factors:

AB = 1/10 = 0.10 /0.1 = 1.0

BC = 1/7.0 = 0.143 /0.1 = 1.43

DE = 1/6.0 = 0.167 /0.1 = 1.67

SUBJECT Hall Meadow Brook Dam.
COMPUTATION Inlet Structure.
COMPUTED BY ISM. CHECKED BY C.C.C. DATE April 23, 1960

- 75.25
+ 1.60
- 1.10
+ 2.20
- 5.35
+10.70
-83.30

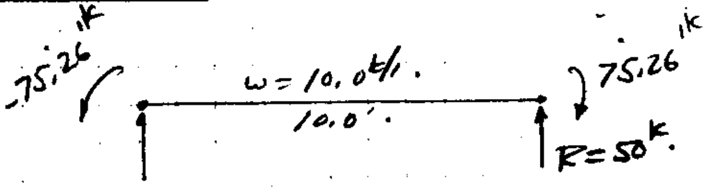
+M = 50.0

0.41

(1.0)

B 0.59	(1.43) 0.50	C 0.50	(1.43) 0.46	D 0.54	(1.67) 0 E
+57.20	-57.20	+57.20	-57.20	+42.00	-42.0
+15.40	0	0	+7.00	+8.20	+42.0
0	+7.70	+3.50	0	+21.00	+4.10
+3.16	-5.60	-5.60	-9.65	-11.35	-4.10
-2.80	+1.58	-4.83	-2.80	-2.05	-5.68
+2.30	+1.63	+1.63	+2.23	+2.62	+5.68
+75.26	-51.89	+51.90	-60.42	+60.42	0
52.4^k +M = 22.3^{1k}	45.6^k	48^k +M = 28.9^{1k}	50^k	52.2^k +M = 36.3^{1k}	31.8^k

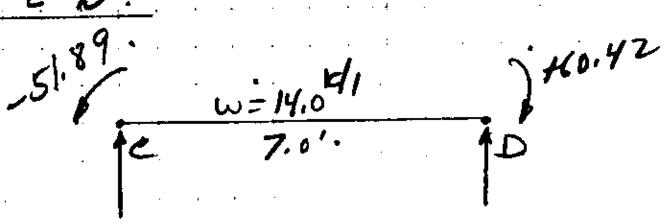
Member A-B:



$$+M = (10.0)\left(\frac{5.0^2}{2}\right) + 75.26 - (50)(5.0)$$

$$= 125.0 + 75.26 - 250.0 = \underline{-50.0}^{1k}$$

Member C-D:



R_D $$(14.0)\left(\frac{7.0^2}{2}\right) + 60.42 - 7.0D - 51.89 = 0$$

$$= 343 + 60.42 - 7.0D - 51.89 = 0$$

$$D = 351.53/7.0 = \underline{50.0}^{1k}$$

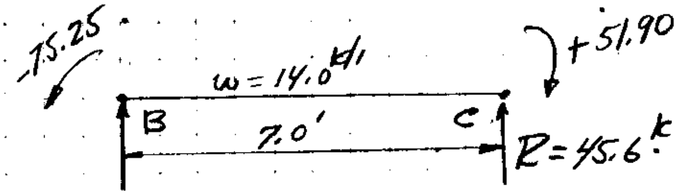
SUBJECT: Hall Meadow Brook Dam.
COMPUTATION: Inlet Structure.
COMPUTED BY: JSM. CHECKED BY: C.C.C. DATE: April 25, 1960

$X_0 = \frac{50.0}{14.0} = \underline{3.57'}$

$+M = (14.0)\left(\frac{3.57^2}{2}\right) + 60.42 - (50.0)(3.57)$

$= 189.3 + 60.42 - 178.5 = \underline{-28.78'^k}$

Member B-C:



R_C: $(14.0)\left(\frac{7.0^2}{2}\right) + 51.90 - 75.25 - 7.0C = 0$

$343 + 51.90 - 75.25 - 7.0C = 0$

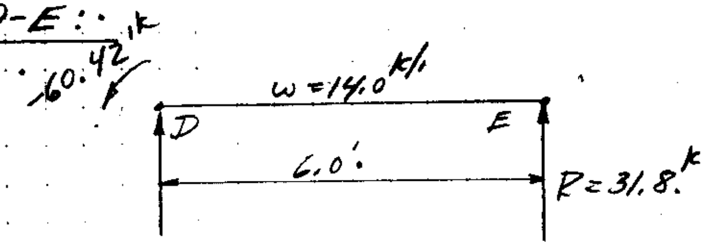
$C = \frac{319.65}{7.0} = \underline{45.6^k}$

$X_0 = 45.6/14.0 = 3.26'$

$+M = (14.0)\left(\frac{3.26^2}{2}\right) + 51.90 - (45.6)(3.26)$

$= 74.3 + 51.90 - 148.5 = \underline{-22.3'^k}$

Member D-E:



R_E: $(14.0)\left(\frac{6.0^2}{2}\right) - 60.42 - 6.0E = 0$

$252.0 - 60.42 - 6.0E = 0$

$E = 191.58/6.0 = \underline{31.8^k}$

$X_0 = 31.8/14 = 2.27'$

$+M = (14.0)\left(\frac{2.27^2}{2}\right) - (31.8)(2.27)$

$= 36.0 - 72.3 = \underline{-36.3'^k}$

SUBJECT Hall Meadow Brook Dam.
COMPUTATION Inlet Structure.
COMPUTED BY JSM. CHECKED BY C.C.C. DATE April 25, 1960

Design of Sections: Assume 24"x24" horiz. beam.

Member "A-B": $-M = 75.26'^{k}$, $+M = 50.0'^{k}$

For no ice pressure on side walls $-M = \frac{(10.0)(10.0^2)}{12} = \underline{83.3}'^{k}$

Req'd $d_M = \sqrt{\frac{83.3 \times 12}{0.160 \times 24} = 260} = \underline{16.15}"$ have $24 - 4.5 = 19.5"$

$-A_s = \frac{(83.3)(12)}{(20.0)(0.885)(19.5)} = \underline{2.9}^{\square"}$ 4-#8

$v = \frac{50.0}{(24)(0.885)(19.5)} = 121^{\#/\square"} > 90$ (Use stirrups)

Allowable $V = (90)(24)(0.885)(19.5) = 37.2^{k}$

$V' = 50.0 - 37.2 = 12.8^{k}$

For #4 stirrups

$s = \frac{(0.40)(20)(0.885)(19.5)}{12.8} = \underline{10.8}"$ Use 2@10", rest @ 12"

$+A_s = \frac{50}{83.3}(2.9) = \underline{1.74}^{\square"}$ 4-#6

Lower A-B

$w = 15 \times 62.5 = 937^{\#/'}$

$M = \frac{(937)(10.0^2)}{10} = 9{,}370'^{\#}$

$A_s = \frac{(9.37)(12)}{(20)(0.885)(19.5)} = \underline{0.33}^{\square"/'}$

#6 @ 12" o.c.

Member D-E:

$-M = 60.42'^{k}$, $+M = 36.3'^{k}$

$-A_s = \frac{(60.42)(12)}{(20)(0.885)(19.5)} = \underline{2.1}^{\square"}$ Use 4-#7.

$+A_s = 36.3/60.42(2.1) = 1.27^{\square"}$ Use 4-#6.

Design of Strut @ "D":

$R = 50 + 52.2 = 102.2^{k}$

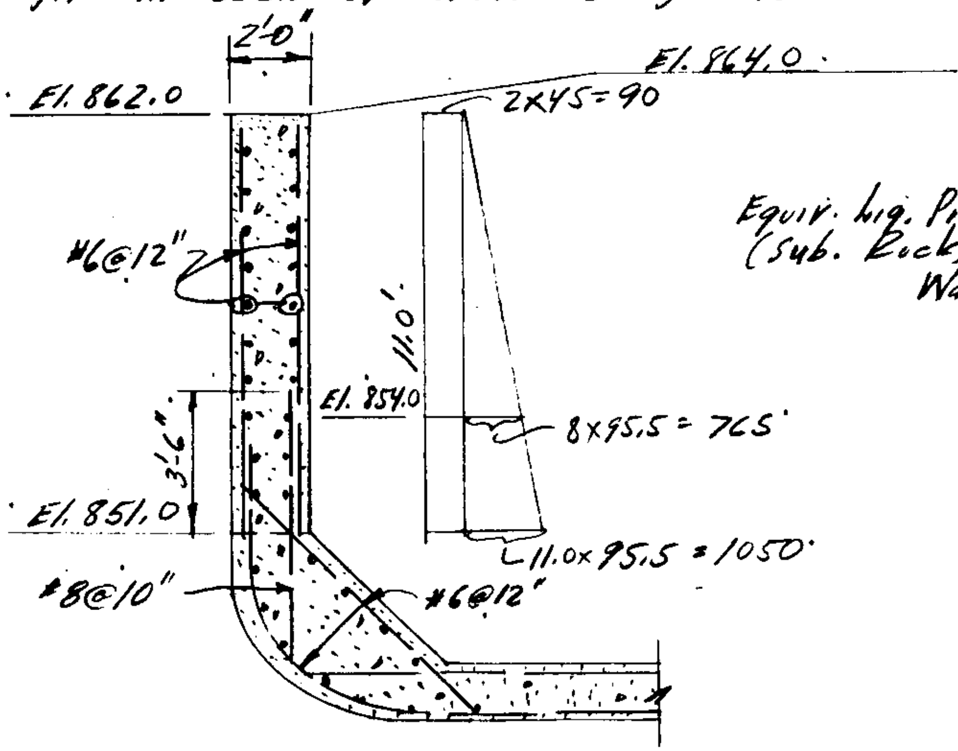
Min. reinf. $= (24)(24)(0.01) = 5.8^{\square"}$ Use 8-#8

Allowable $P = 0.80 \times 24.0^2(0.225 \times 3{,}000 + 16{,}000 \times 0.01) = \underline{384}^{k}$ (corrected: .675, 160; 480 struck out)

SUBJECT Hall Meadow Brook Dam.
COMPUTATION Inlet Structure.
COMPUTED BY ISM. CHECKED BY C.C.C. DATE May 4, 1960

Design of Vert. Wall @ Sta. 2+38 :

Assume water @ El. 862.0 with submerged rock fill in back of wall. Design as vert. cantilever.



Equiv. Liq. Pres. (Sub. Rock) $= 72 \times 0.45 = 33$

Water $= 62.5$

95.5

$$M = (90)(11.0)(5.5) + \tfrac{1}{2}(1050)\left(\frac{11.0^2}{3}\right)$$

$$= 5430 + 21{,}200 = 26{,}630$$

$$\text{Req'd } d_M = \sqrt{\frac{26{,}630}{160}} = \sqrt{167} = 12.9'' \quad \text{have } 24 - 4.5 = 19.5''$$

$$A_s = \frac{(26{,}630)(12)}{(20{,}000)(7/8)(19.5)} = \underline{0.94}^{\square''/'} \quad \#8 @ 10''$$

$$M @ El.\ 854.0 = (90)(8)(5.5) + \tfrac{1}{2}(765)\left(\frac{8.0^2}{3}\right) = 3960 + 8150 = \underline{12{,}110}^{'\#}$$

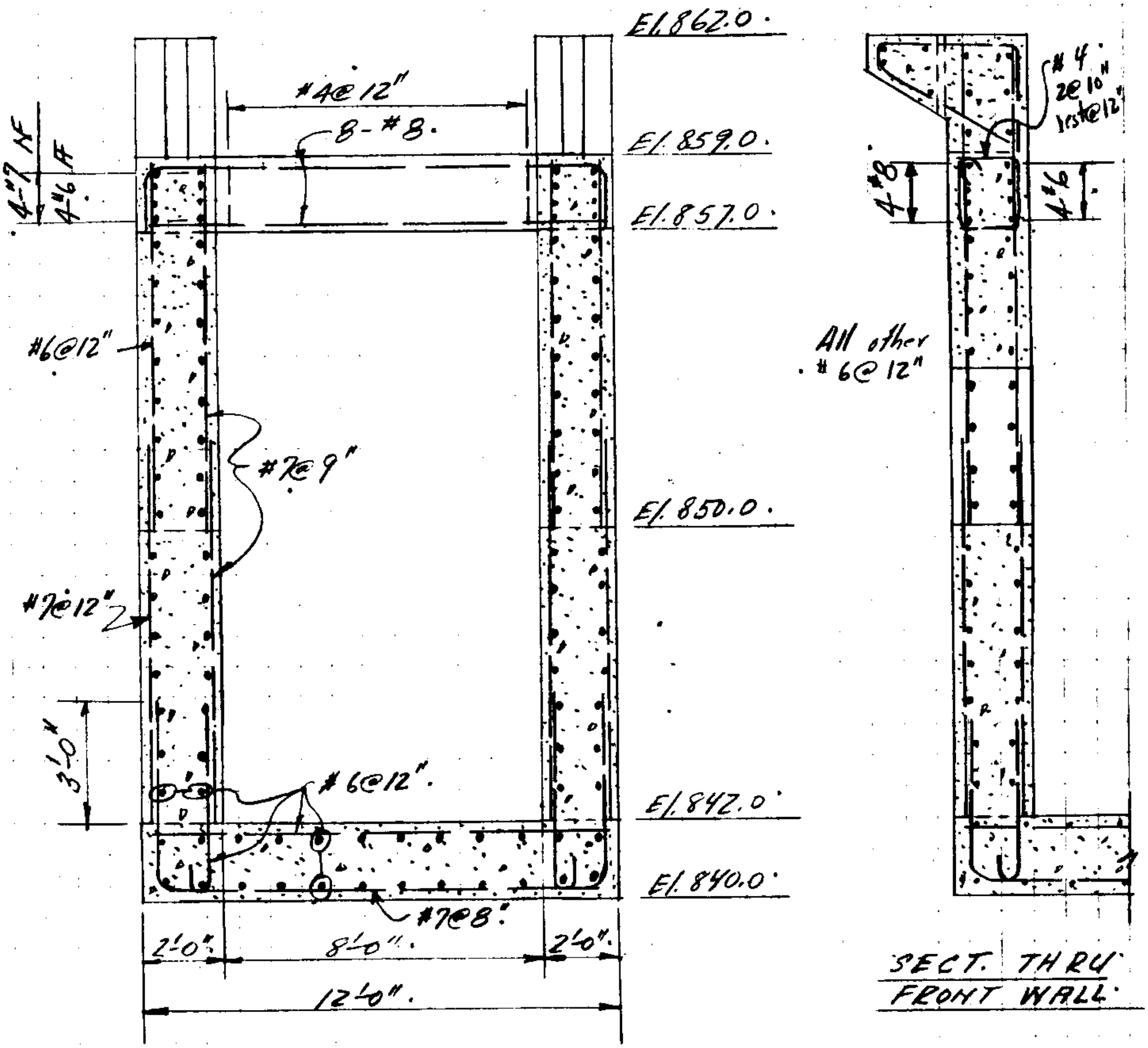
$$A_s = \frac{12.11}{26.63}(0.94) = \underline{0.43}^{\square''} \quad \#6 @ 12''$$

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U. S. ARMY

SUBJECT Hall Meadow Brook Dam.
COMPUTATION Inlet Structure.
COMPUTED BY JSM. CHECKED BY C.C.C. DATE April 26, 1960

Reinf. Details:.



TYP. TRANSVERSE SECTION.
1/4" = 1'-0"

SECT. THRU FRONT WALL.

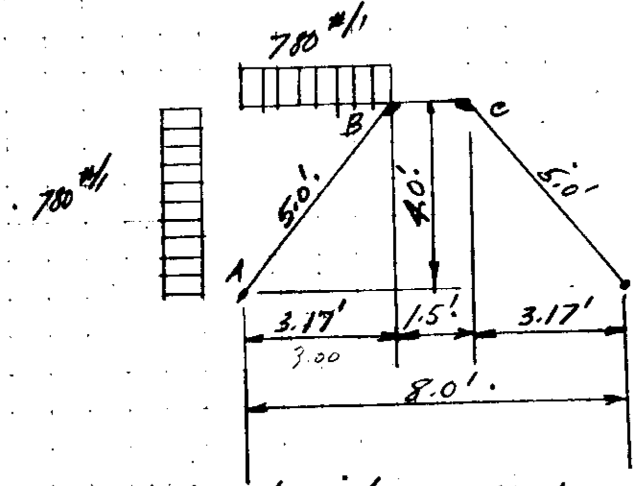
SUBJECT Hall Meadow Brook Dam.
COMPUTATION Inlet Structure.
COMPUTED BY JSM. CHECKED BY C.C.C. DATE April 27, 1960

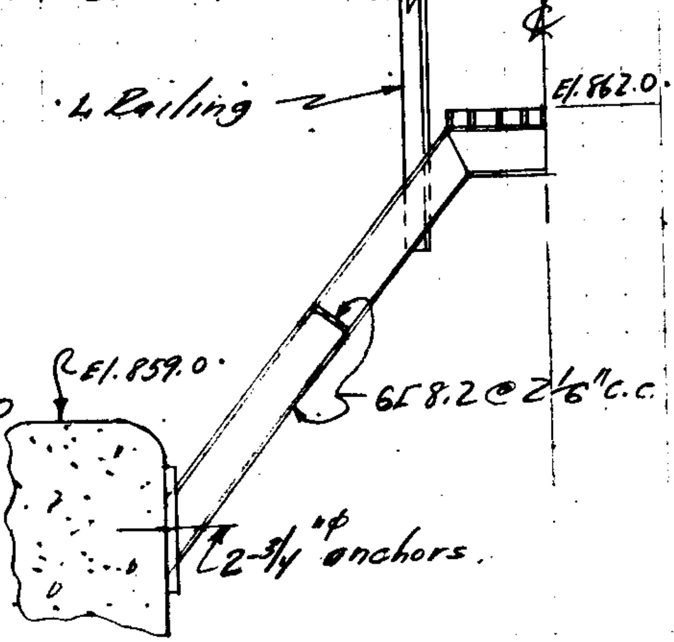
Design of Trash Beams:

Design for 5'0" differential head $w = 62.5 \times 5 = 312.5^{\#/\square'}$

with beams @ 2'6" o.c. $w = 312.5 \times 2.5 = 780^{\#/'}$

Case I - One side loaded:





From Kleinlogel: pp. 262-4.

Coeffs: $k = \frac{5.0}{1.5} = 3.33$ $\quad \beta = \frac{1.5}{8.0} = 0.19$

$\alpha = \frac{3.17}{8.0} = 0.396$ $\quad N = (2)(3.33) + 3 = 9.66$

For top loading

$$M_C = \frac{(780)(3.17^2)}{4}\left[-0.19 - \frac{3.33}{(2)(9.66)}\right] = -725^{\prime\#}$$

(-0.37)

$$V_D = (780)(3.17)\left(\frac{0.396}{2}\right) = 494^{\#}$$

For Side loading:

$$M_C = \frac{(780)(4.0^2)}{4}\left[-0.19 - \frac{3.33}{(2)(9.66)}\right] = -1155^{\prime\#}$$

(-0.37)

Total $M = \underline{1,880}^{\prime\#}$

$$V_D = \frac{(780)(4.0^2)}{(2)(8.0)} = 780^{\#}$$

Total $V = 1,274^{\#}$

Req'd $S = (1880)(12)/20,000 = \underline{1.13}^{\prime\prime 3}$ Use 6 [8.2#.

SUBJECT Hall Meadow Brook Dam
COMPUTATION Conduit Section @ ℄ Dam
COMPUTED BY JSM CHECKED BY EPG DATE April 11, 1960

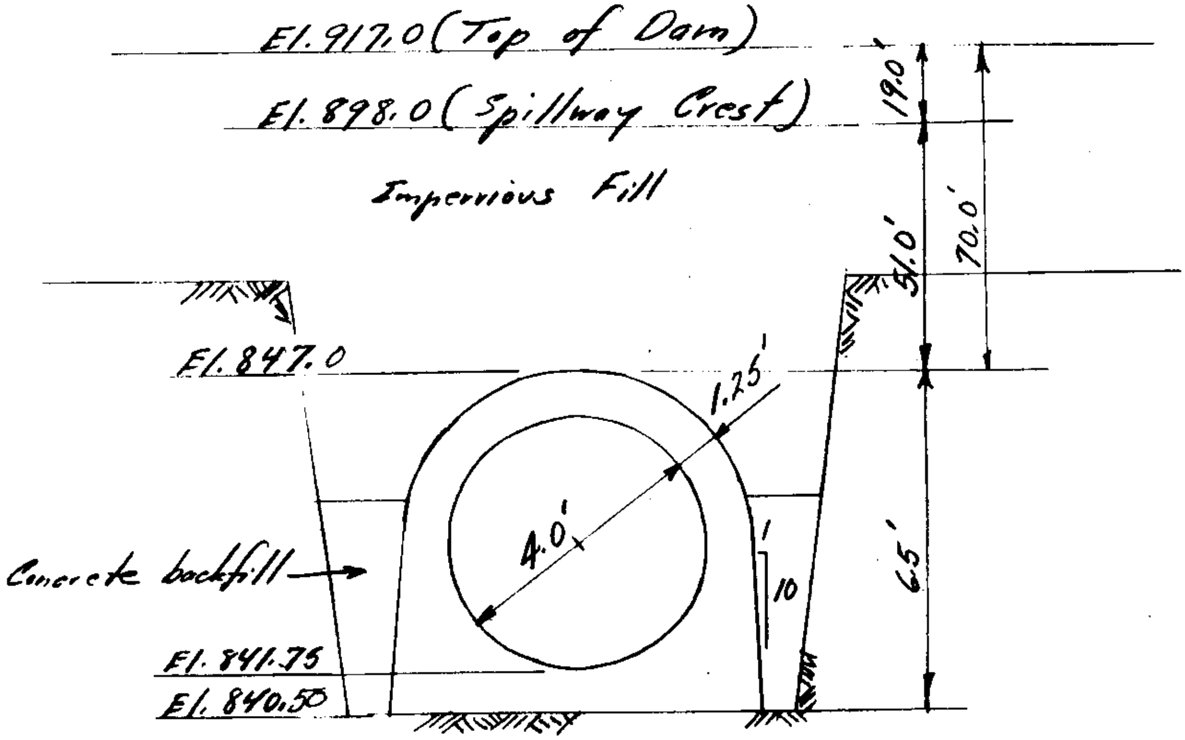


TABLE OF UNIT WEIGHTS (FROM N.E.D. SOILS LAB.)

	Material	UNIT WEIGHT (P.C.F.) Sat.	Moist	Dry	Submerged γ_m'	K_o
	Rock Fill	135	115	115	72	0.45
	Gravel Bedding & Pervious Fill	141	130	125	78	0.45
	Impervious Fill	145	140	130	82	0.70

Assumptions:

Case I - Rapid drawdown from spillway crest with 100% $\gamma_{sat.}$ to El. 898 + 100% γ_{moist}, El. 898 to top for vert. load. For horiz. load 50% of above to be used.

Case II - Water at spillway crest, El. 898.0 with

SUBJECT Hall Meadow Brook Dam
COMPUTATION Conduit Section @ ℄ Dam
COMPUTED BY ISM CHECKED BY EPC DATE April 20, 1960

100% $\gamma_{sub.}$ + Wt. of water to El. 898.0 + 100% γ_{moist}, El. 898.0 to top of dam for vert. load. For horiz. load use 50% $\gamma_{sub.}$ + hydrostatic head to El. 898.0 + 50% γ_{moist}, El. 898.0 to top of dam.

<u>Case I:</u>

Vert. Load = (1.0)(145)(70) = 10,150 #/' —
Horiz. " = (0.5)(145)(70) = 5,075 #/' —
{ Sat. & moist used as same weight. (Neglect error)

<u>Case II:</u>

Vert. Load = (1.0)(82.0)(51.0) + (62.5)(51.0) + (1.0)(145)(19.0)
= 4,180 + 3,190 + 2,750 = 10,120 #/'

Horiz. " = (0.5)(82.0)(51.0) + 3190 + (0.5)(145)(19.0)
= 2,090 + 3,190 + 1,375 = 6,655 #/'

<u>Design Section for Case I:</u>

Intensity of Vert. Load:	Intensity of Horiz. Load:
No. 1 = (145)(70.08) = 10.16	× 1/2 = 5.08
2 = (145)(70.58) = 10.23	" = 5.12
3 = (145)(71.46) = 10.36	" = 5.18
4 = (145)(72.62) = 10.53	" = 5.27
5 = (145)(73.61) = 10.67	" = 5.34
6 = (145)(74.38) = 10.79	" = 5.40
7 = (145)(75.67) = 10.97	" = 5.49

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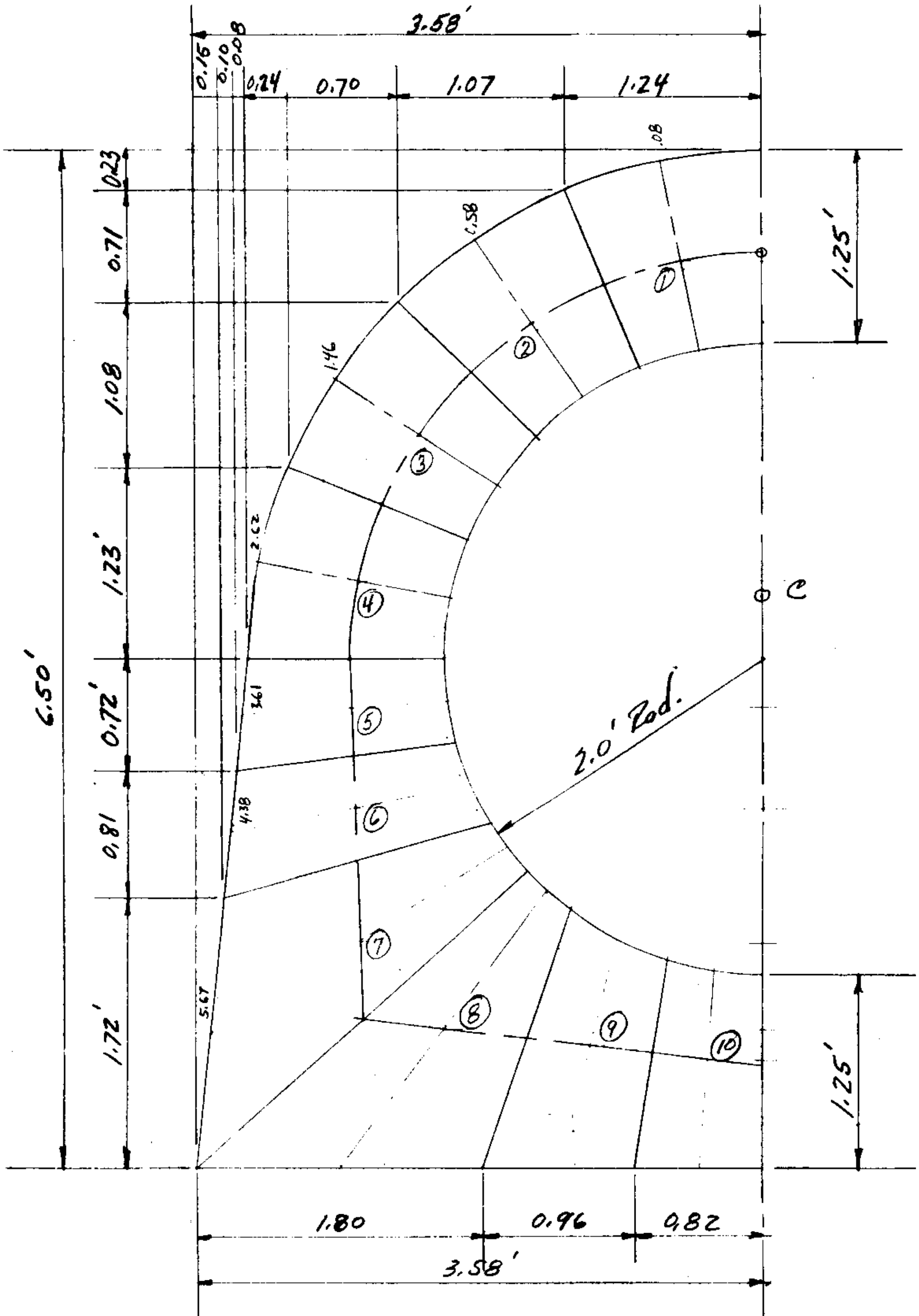
NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U. S. ARMY

SUBJECT Hall Meadow Brook Dam
COMPUTATION Conduit Section @ ℄ Dam
COMPUTED BY JSM CHECKED BY EPC DATE April 21, 1960



1" = 1'-0"

GIVEN					DERIVED											
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	
No.	d_s	d	x	y_1	$a=\frac{d_s}{d^3}$	ay_1	y	ay^2	INTEN. VERT. LOAD	HORIZ. PROJ. VOUS.	VERT. LOAD	WT. VOUS	TOTAL VERT. LOAD	ADJUST. VERT. LOAD	ΣV	
1	1.00	1.25	-0.50	-0.07	0.512	-0.036	+2.16	2.389	10.16	1.24	12.60	0.19	12.79	12.79	12.79	1
2	1.00	1.25	-1.45	-0.45	0.512	-0.230	+1.77	1.604	10.23	1.07	10.95	0.19	11.14	11.14	23.93	2
3	1.00	1.25	-2.18	-1.18	0.512	-0.604	+1.05	0.565	10.36	0.70	7.25	0.19	7.44	7.44	31.37	3
4	1.00	1.25	-2.55	-2.10	0.512	-1.075	+0.10	0.005	10.53	0.24	2.53	0.19	2.72	2.72	34.09	4
5	0.61	1.30	-2.60	-2.91	0.278	-0.809	-0.70	0.136	10.67	0.08	0.85	0.12	0.97	0.97	35.06	5
6	0.68	1.55	-2.58	-3.58	0.183	-0.655	-1.38	0.348	10.79	0.10	1.08	0.16	1.24	1.24	36.30	6
7	1.00	2.22	-2.53	-4.43	0.091	-0.403	-2.22	0.448	10.97	0.15	1.65	0.33	1.98	1.98	38.28	7
8	1.04	2.20	-2.00	-4.99	0.098	-0.489	-2.78	0.757	–	–	–	0.34	0.34	-38.59	-0.31	8
9	0.80	1.50	-1.09	-5.10	0.237	-1.209	-2.89	1.979	–	–	–	0.18	0.18	0.18	-0.13	9
10	0.70	1.28	-0.35	-5.18	0.334	-1.730	-2.98	2.966	–	–	–	0.13	0.13	0.13	0	10
					3.269	-7.24		11.197					38.93			

$$C = \frac{\Sigma a y_1}{\Sigma a} = \frac{-7.24}{3.269} = -2.215$$

The total vertical load of 38.93^k is assumed as taken entirely by Vous No. 8.

17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	22	33
No.	INTEN HORIZ LOAD	VERT. PROJ. VOUS	HORIZ. LOAD	ΣH	$X_n - X_{n-1}$	$Y_n - Y_{n-1}$	Vert.	Horiz.	m 24+25	ma	may	m+M	H×Y	Mom. 29+30	Thrust	Shear
1	5.08	0.23	1.17	1.17	-0.95	-0.39	-1.52	-0.06	-1.58	-0.81	-1.75	+51.29	-44.86	+6.43	21.0	2.0
2	5.12	0.71	3.64	4.81	-0.73	-0.72	-13.67	-0.52	-14.19	-7.27	-12.87	+38.68	-36.76	+1.92	25.0	5.4
3	5.18	1.08	5.60	10.41	-0.37	-0.95	-31.14	-3.98	-35.12	-17.98	-18.88	+17.75	-21.81	-4.06	30.2	4.7
4	5.27	1.23	6.49	16.90	-0.65	-0.80	-42.75	-13.87	-56.62	-28.99	-2.90	-3.75	-2.08	-5.83	33.5	0
5	5.34	0.72	3.84	20.74	+0.62	-0.68	-44.45	-27.39	-71.84	-19.97	+13.98	-18.97	+14.54	-4.43	34.7	3.0
6	5.40	0.81	4.38	25.12	+0.05	-0.84	-43.75	-41.40	-85.15	-15.58	+21.50	-32.28	+28.66	-3.62	35.7	1.2
7	5.49	1.72	9.43	34.65*	+0.53	-0.56	-41.93	-62.50	-104.43	-9.50	+21.09	-51.56	+46.11	-5.45	37.5	8.2
8	—	—	—	34.65	+0.91	-0.11	-21.64	-81.90	-103.54	-10.15	+28.21	-50.67	+57.74	+7.07	16.5	18.9
9	—	—	—	34.65	+0.74	-0.09	-21.92	-85.79	-107.71	-25.53	+73.77	-54.84	+60.03	+5.19	14.0	1.0
10	—	—	—	34.65	—	—	-22.02	-88.91	-110.93	-37.05	+110.41	-58.06	+61.89	+3.83	13.8	1.0
										-172.83	+232.56					

* NEGLECT ERROR

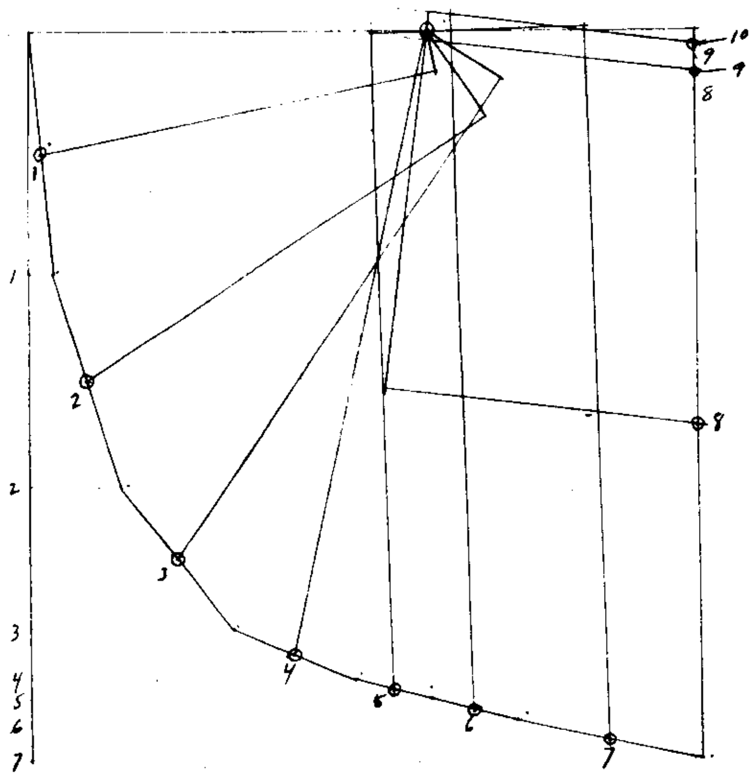
$$M = -\frac{\Sigma ma}{\Sigma a} = -\frac{172.83}{3.269} = +52.87$$

$$H = -\frac{\Sigma may}{\Sigma ay^2} = -\frac{232.56}{11.197} = -20.77$$

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U. S. ARMY

SUBJECT Hall Meadow Brook Dam
COMPUTATION Conduit Section @ ℄ Dam
COMPUTED BY JSM CHECKED BY EPG DATE April 21, 1960



FORCE DIAGRAM

1" = 10k

SUBJECT Hall Meadow Brook Dam
COMPUTATION Conduit Section @ ℄ Dam
COMPUTED BY JSM CHECKED BY EPG DATE April 22, 1960

Design of Section:

Allowable stresses

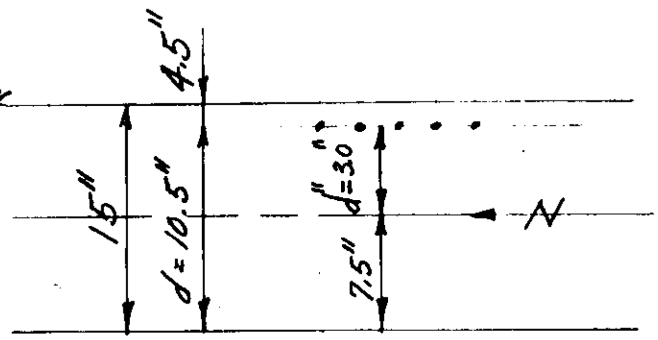
$f'_c = 3{,}000$ $n = 10$

$f_c = 1{,}350$ $K = 236$, $j = 0.866$, $k = 0.403$

$f_s = 20{,}000$ $\mu = 210$ (Top bars)

$v = 90$ $\mu = 300$ (all others)

Vous No. 1:



$M = 6.43^k$, $N = 21.0$, $V = 2.0^k$

$M_s = M + \frac{d''}{12} N$

$M_s = 6.43 + \frac{3.0}{12}(21.0)$

$= 6.43 + 5.25 = 11.68^{\prime k}$.

Req'd $d_m = \sqrt{\frac{11.68}{0.236} = 49.5} = 7.03$" have 10.5".

$A_s = \frac{12 M_s}{f_s j d} - \frac{N}{f_s}$.

$= \frac{(12)(11.68)}{(20)(0.866)(10.5)} - \frac{21.0}{20.0}$

$= 0.77 - 1.05$ Use min. steel #6 @ 1'0".

Vous No. 8:

$M = 7.1^{\prime k}$, $N = 16.5^k$, $V = 18.9^k$

$M_s = 7.1 + \frac{3.0}{12.0}(16.5)$

$= 7.1 + 4.1 = 11.2^{\prime k}$.

$A_s = \frac{(12)(11.2)}{(20)(0.866)(10.5)} - \frac{16.5}{20}$

$= 0.75 - 0.80$.

SUBJECT Hall Meadow Brook Dam
COMPUTATION Conduit Section @ ℄ Dam
COMPUTED BY ISM CHECKED BY EPG DATE April 22, 1960

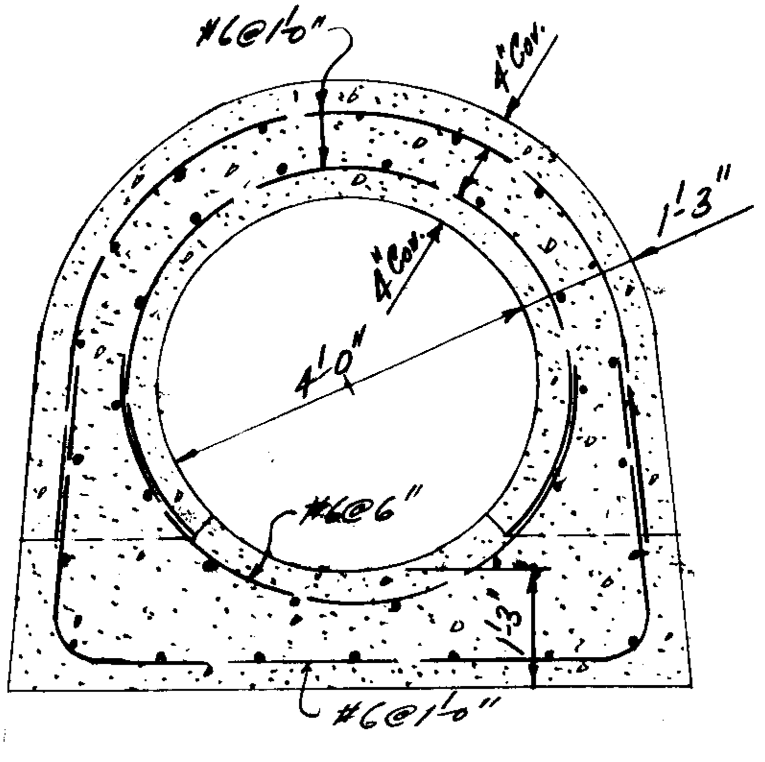
$v = \frac{18.9}{(12)(0.866)(22.0)} = \underline{82.8}^{\#/□''} < 90$

$u = \frac{18,900}{(2.4)(0.866)(22.0)} = 413^{\#/□''} > 210$

$Req'd\ \Sigma_0 = \frac{18,900}{(210)(0.866)(22.0)} = \underline{4.7}$ #6 @ 6″.

Vous No. 2:

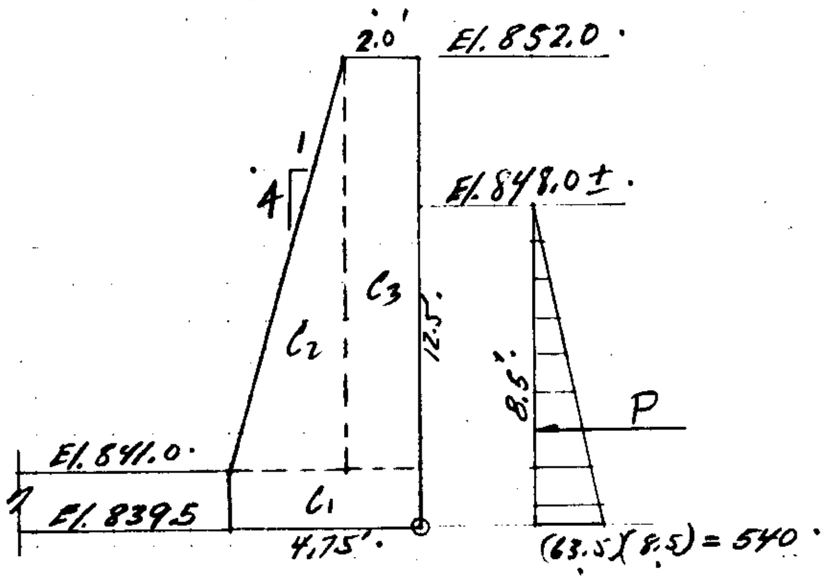
$Req'd\ \Sigma_0 = \frac{5,400}{(300)(0.866)(10.5)} = \underline{1.97}$ #6 @ 12″.



Longit. Reinf.
#6@1'-0"

SUBJECT Hall Meadow Brook Dam.
COMPUTATION Outlet Structure.
COMPUTED BY JSM. CHECKED BY C.C.C. DATE April 28, 1960

Design of outlet channel retaining wall.
Assume pervious backfill, saturated with no uplift.
Sat. wt. = 141 #/c.f. Assume "at rest" pressure $k_o = 0.45$.
Equiv. liq. pres. = (141)(0.45) = 63.5 #/'.



C_1	= (1.5)(4.75)(150)	= 1,070	× 2.375	= 2,540.
C_2	= 1/2(2.75)(11.0)(150)	= 2,270.	× 2.92.	= 6,640.
C_3	= (2.0)(11.0)(150)	= 3,300.	× 1.00.	= 3,300.
		6,640.		
P	= 1/2(540)(8.5)	= 2,290	× 2.83	= 6,500.
				18,980.

$\frac{\Sigma M}{\Sigma V} = \frac{18,980}{6,640} = 2.86'$ 3rd Pt. = 3.17'.