CONNECTICUT RIVER FLOOD CONTROL

OTTER BROOK
DAM & RESERVOIR

OTTER BROOK, NEW HAMPSHIRE

DESIGN MEMORANDUM NO. 8

DETAILED DESIGN MEMORANDUM
FOR SPILLWAY WEIR, OUTLET WORKS
AND MISCELLANEOUS STRUCTURES

Corps of Engineers, U.S. Army - Office of the Division Engineer

New England Division - Boston, Mass.

FEBRUARY 1956

CORPS OF ENGINEERS, U. S. ARMY
OFFICE OF THE DIVISION ENGINEER
NEW ENGLAND DIVISION
857 COMMONWEALTH AVENUE
BOSTON 15, MASS.

Address Reply to:
Division Engineer

Refer to File No.
NEDGW

15 February 1956

SUBJECT: Submission of Design Memoranda, Otter Brook Dam and Reservoir, Connecticut River Basin, New Hampshire

TO: Chief of Engineers
Department of the Army
Washington 25, D. C.
ATTENTION: ENGWE

There are submitted herewith, for review and approval, 10 copies each of the Design Memoranda entitled "Concrete Materials" and "Detailed Design Memorandum for Spillway Weir, Outlet Works and Miscellaneous Structures," (Design Memoranda Nos. 4 and 8, respectively) for Otter Brook Dam and Reservoir, Connecticut River Basin, New Hampshire, together with one set of full-size drawings for Design Memorandum No. 8, all in accordance with Paragraph 4214.12 of Orders and Regulations.

FOR THE DIVISION ENGINEER:

J. R. Thompson

J. R. THOMPSON
Lt. Colonel, Corps of Engineers
Executive Officer

3 Incls.
1. Design Memo No. 4 -(Copies 1 to 10, incl.)
2. " " " 8 - " " " " "
3. Full-sized drawings for Design Memo No. 8

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FLOOD CONTROL PROJECT

OTTER BROOK DAM AND RESERVOIR

OTTER BROOK

CONNECTICUT RIVER BASIN

NEW HAMPSHIRE

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1	Hydrology and Hydraulic Analysis	15 Feb 1955 * 30 June 1955	7 Sept 1955
2	Real Estate Memorandum	4 May 1955	13 July 1955
3	Geology and Soils	27 May 1955 ** 30 June 1955	17 Aug 1955
4	Concrete Materials	15 Feb 1956	
5	General Design Memorandum	27 May 1955 ** 30 June 1955	8 Sept 1955
6	Relocations		
7	Embankment Design	27 May 1955 ** 30 June 1955	13 Sept 1955
8	Detailed Design Memorandum for Spillway Weir, Outlet Works and Miscellaneous Structures	15 Feb 1956	
9	Reservoir Management		

* Initial submission in Draft to secure approval of spillway design flood and outlet requirements, approved 17 March 1955.

** Submission of Preliminary Draft for discussion and review.

OTTER BROOK DAM AND RESERVOIR

CONNECTICUT RIVER BASIN

NEW HAMPSHIRE

FLOOD CONTROL PROJECT

DESIGN MEMORANDUM NO. 8

GENERAL DESIGN MEMORANDUM

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OTTER BROOK DAM AND RESERVOIR
OTTER BROOK
CONNECTICUT RIVER BASIN
NEW HAMPSHIRE

A. PERTINENT DATA

a. Purpose — Flood Control

b. Location of Dam

State	New Hampshire
County	Cheshire
City and Town	Keene and Roxbury
River	Otter Brook, 2.4 miles above its confluence with The Branch, 4.9 miles above the Ashuelot River and 31.4 miles from the Connecticut River.
Distance from Keene	East northeast 2.4 airline miles

c. Drainage Areas

Otter Brook at Gaging Station	42.3 sq. mi.
Otter Brook at Dam site	47 " "
Otter Brook at The Branch	54.5 " "
The Branch at the Ashuelot R.	100 " "
Ashuelot River at Connecticut R.	421

d. Stream Flow

Record of U.S.G.S. gaging station on Otter Brook Oct. 1923 to Sept. 1953

	Average Rates	
	C.F.S.	C.F.S. Per sq.mi.
Average Annual	71.0	1.67
Maximum year (1938)	110	2.60
Minimum year (1930)	39.4	0.93
Maximum month (March, 1936)	440	10.4
Minimum month (Oct. 1949)	2.88	.068
Maximum day (Sept 21, 1938)	4040	95.5
Minimum day (Sept 9, 10, 12, 30, 1954)	1.3	.031

e. Maximum Floods of Record (Peaks exceeding 3,000 c.f.s.)

(Otter Brook at Gaging Station)

Sept. 21, 1938	6,130 c.f.s.
Mar. 18, 1936	3,580 "
Nov. 26, 1950	3,540 "
Nov. 4, 1927	3,180 "
Apr. 12, 1934	3,020 "

f. Spillway Design Flood Data

Peak inflow, full reservoir, c.f.s.	38,000
Total volume of rainfall, inches	24.8
Infiltration rate, inches per hour	0.067
Total volume of runoff, acre-feet	53,150
Total volume of runoff, inches	21.2
Duration of flood, days	4.0
Reservoir stage at start of flood, feet, m.s.l.	781.0
Gates	Closed, inoperative

g. Reservoir

City and Town affected — Keene and Roxbury, N.H.

Pool	Elev.	Area in Acres	Net Storage Acre-Feet	Net Storage Inches of Runoff
Permanent Conservation	701	70	700	0.3
Flood Control	781	375	17,600	7.0
Maximum Surcharge	797.1	444	6,500	2.6
Maximum Wave Fetch	(at elev. 797.0)		1.67 miles	

h. Dam

Type	Rolled fill, earth
Length, feet	1,288
Maximum height above stream bed, feet	133
Slopes of Embankment	
Upstream, above elevation 683,	1 on $2\frac{1}{2}$
below elevation 683,	1 on 6
Downstream	1 on $2\frac{1}{2}$

h. Dam (Continued)

Elevations, mean sea level datum	
Top of dam	802.0
Streambed at downstream toe	665
Streambed at center line	669
Top width	25 feet
Freeboard above spillway design Flood height	4.9 feet

i. Spillway Weir

Type	Uncontrolled, ogee
Crest length, feet	145
Elevation of crest, m.s.l.	781.0
Maximum head, feet from reservoir surface	16.1
Peak discharge, c.f.s.	34,500

j. Outlet Works

Intake Structure

Intake channel width, feet	24
Tower height, feet	136.5
Tower diameter, feet	20
Length (bellmouth entrance to end of transition), feet	48.5

Gates - Service

Type	Hydraulic slide
Number and size	3-4'6" x 2'6"
Elevation gate sills, m.s.l.	683.0

Gates - Emergency: None

Capacity of Outlets

	1 Gate	2 Gates	3 Gates
Reservoir at elevation 709.5, c.f.s.			800
Reservoir at spillway crest	775	1390	1500
Reservoir at elevation 797.1, c.f.s.	840	1500	1620

Conduit

Type - Conduit on rock foundation - Boston Horseshoe Section	
Inside diameter, feet	6.0
Maximum concrete thickness, feet	2.5
Length, feet	541.5

j. Outlet Works (Continued)

Stilling Basin

Elevation at exit portal, m.s.l.	677.0
Elevation at stilling basin floor, m.s.l.	658.0
Apron length, feet	60
Stilling basin length, feet	38
Width, feet	25

Service Bridge

Type	Deck, Warren steel truss
Number of spans	2
Length of spans, feet	95
Roadway width, feet	10
Design load	H-15

k. Concrete Quantities (approximate)

Spillway weir	900 c.y.
Intake structure (intake, tower, and transition)	1,800 c.y.
Conduit	1,300 c.y.
Stilling basin	500 c.y.
Service bridge	240 c.y.
Concrete backfill	1,400 c.y.
Total	6,140 c.y.

CORPS OF ENGINEERS, U. S. ARMY
OFFICE OF THE DIVISION ENGINEER
NEW ENGLAND DIVISION
BOSTON 15, MASS.

FLOOD CONTROL PROJECT

OTTER BROOK DAM AND RESERVOIR
OTTER BROOK
CONNECTICUT RIVER BASIN
NEW HAMPSHIRE

DESIGN MEMORANDUM NO. 8

DETAILED DESIGN MEMORANDUM FOR SPILLWAY
WEIR, OUTLET WORKS AND MISCELLANEOUS STRUCTURES

15 February 1956

B. INTRODUCTION

1. Purpose. - The purpose of this memorandum is to facilitate the review by higher authority of the designs, plans and specifications for the Otter Brook Spillway Weir, Outlet Works and Miscellaneous Structures. The basic criteria, typical design computations and other data pertinent to the design are presented herein.

2. Scope. - This memorandum covers the following structures: The spillway weir; the outlet works including the intake, gate tower and superstructure, transition, conduit, stilling basin and service bridge; operator's quarters; utility building; fire fighting facilities; utilities; parking areas and roadways; and log boom. Design of the earth embankment is covered in Design Memorandum No. 7 (Embankment Design).

3. Construction Time and Schedule. - The construction of the dam and appurtenances will be accomplished under a continuant contract to be awarded in August 1956. This contract will extend through the balance of the 1956 construction season and two additional full seasons, ending late in Calendar Year 1958. Channel closure and final diversion are scheduled for the Spring of 1957. The preceding construction schedule is contingent upon a $100,000 supplemental appropriation requested for Fiscal Year 1956, which will be used to initiate highway relocation, as well as the availability of other necessary funds. The highway relocation will replace Branch Road which will be blocked by the cofferdam. During the 1956 construction season, the intake walls and slab, lower part of the gate tower, transition, conduit, stilling basin and utility building, which will serve as a field office, will be constructed. The upstream portions of the outlet works will be constructed first to permit early placement of embankment cofferdam fill to

elevation 711.0 around the base of the intake structure. During the 1947 construction season, the upper part of the gate tower, tower superstructure, dam embankment and service bridge will be completed. Also during the 1957 construction season, the log boom and fire fighting facilities will be constructed. A separate contract for repairs to the operator's quarters will be awarded early in 1958 after the needs have been determined by inspection. Work on the parking areas and roadways will be interspersed over the entire contract period permitting the contractor to rough grade and use these roads during construction. Construction of the spillway weir. installation and testing of gate operating equipment, topsoiling, final grading, seeding and paving will not be completed until the closing months of the contract. The construction schedule has been accelerated since the submission of Design Memorandum No. 5 (General Design Memorandum) to take advantage of allotments now anticipated in Fiscal Years 1956 and 1957.

4. Previous Reports. - No previous detailed report on the spillway weir, outlet works and miscellaneous structures has been submitted. The latest previous description of the proposed structures and improvements is set forth as the recommended project plan in Design Memorandum No. 5 (General Design Memorandum). Minor differences between the structures described herein and those in the recommended project plan are covered in the descriptions of the structures.

C. HYDROLOGY

5. General. - Design Memorandum No. 1 (Hydrology and Hydraulic Analysis) includes the basic data and hydrological determinations for the spillway, outlet works and diversion requirements. A summary of the applicable results is given below.

6. Spillway Weir. - By applying the spillway design flood, which is derived from an estimate of the maximum possible precipitation over the drainage area to an initially full reservoir, the resulting maximum spillway discharge is 34,500 c.f.s. The accompanying maximum water surface elevation within the reservoir is 797.1, equivalent to 16.1 feet of surcharge.

7. Outlet Works. - A 6-foot diameter horseshoe-shaped conduit, controlled by three gates, each 2'-6" x 4'-6", with a discharge capacity of approximately 1500 c.f.s., was found adequate to pass the normal flow of the stream with only a minor encroachment on the reservoir storage capacity, to pass discharges required for operation of the reservoir during floods and to permit draining of the reservoir within a reasonable time after a flood. The estimated channel capacity of Otter Brook below the dam site is approximately 1,000 c.f.s., which will be the normal maximum discharge of the outlet works.

8. Diversion. - The construction schedule requires that the stream be diverted through the outlet following the 1957 spring runoff. It was determined that a 10-year flood for the construction season, routed through the reservoir storage and the 6-foot diameter horseshoe conduit, produces a maximum discharge of about 800 c.f.s. During such a flood the maximum pool elevation is 709.5, and an elevation of 711.0 has been selected for the top of the embankment cofferdam.

D. HYDRAULIC DESIGN

9. General. - The hydraulic design of the spillway weir and outlet works is shown in Design Memorandum No. 1 (Hydrology and Hydraulic Analysis). A summary of the results which are pertinent to the design of the spillway weir and outlet works are given below.

10. Spillway Weir. - The shape of the weir is in accordance with Part CXVI Chapter 3 of the Engineering Manual. Control dimensions are 3.19 feet and 7.22 feet radii respectively for the curvature of the upstream portion of the ogee crest, the formula $Y = 0.0492\ X^{1.83}$ for the coordinates of the downstream profile and a bucket with a radius of 20 feet. The approach channel elevation at the upstream face of the weir is 776, the crest is at elevation 781.0 and the toe elevation at the end of curvature is at elevation 775.1, tangent to the downstream chute slope of 0.0566. With a design discharge of 34,500 c.f.s., the maximum headwater for structural design purposes is at elevation 796.8 which is the energy gradient at the weir. The maximum tailwater is at elevation 783.6 which is the maximum water surface at the downstream toe. Details of the weir shape are shown on Plate No. 8-3.

11. Outlet Works. - a. Intake. - The conduit intake, besides serving as the entrance to the three 2'-6" by 4'-6" gate passages, is designed to facilitate maintenance of a conservation pool at elevation 701. A concrete weir, containing five stop log openings, having a total effective length of 34'-4", is provided at the entrance to the center gate for this purpose. The stop log openings will permit adjustment for variations in flow and possible raising or lowering of the conservation pool three feet from elevation 701. There is also a 6-inch low flow outlet through the conservation weir just above the floor elevation which serves as a source of cool water from the bottom of the reservoir during the summer months.

b. Approach Channel. - The walls of the approach channel provide a 24-foot approach width to the conservation weir and side gate passages. Beyond the conservation weir, the flow to the side gate passages is suppressed between the weir and side walls, thus eliminating the need of elliptical side curves in the conduit entrance. The intake roof curve of each of the three passageways is based on the equation $\frac{X^2}{D^2} + \frac{Y}{(2/3D)^2} = 1$ where D = 4'-6."

The intake floor is level at elevation 683.0.

c. Gate Section and Transition. - The gate passages beyond the entrance ellipses provide a gradual transition to the 2'-6" x 4'-6" areas at the three gates. Continuing beyond the gate section, the passage areas are held constant, with a combined cross-sectional area of 33.75 square feet, to the dividing pier noses. From the ends of the piers to the typical conduit section, a distance of 14.75 feet, a smooth transition decreases the area to 29.85 square feet. The crown of each gate passage is level with the crown of the conduit at elevation 687.5. The bottom of each passage is level at elevation 683.0 to the center line of the gate

tower. Beyond this point, a 26-foot long vertical transition lowers the invert to elevation 681.5, varying the height from 4.5 feet in the gate passageways to the 6-foot conduit diameter.

d. Air Vents. - Three 6" x 2'-6" air vents with 18-inch diameter risers located downstream from the gates tap a 24-inch diameter header at elevation 710, which is connected to the main 24-inch diameter riser. With the air vent designed for a maximum velocity of 150 feet per second, the pressure drop in the conduit, or head loss in the vents, equals 1.2 feet of water.

e. Hydrostatic Pressures in Gate Passages. - Pressure gradients in the gate passages have been computed for use in the structural design of the walls and slabs. Since structural criteria permits an increase of 33-1/3 percent in the normal allowable working stresses when the reservoir is at maximum surcharge level, it was found that the governing design condition occurs when the reservoir is at spillway crest elevation and normal allowable working stresses are used. Pressure gradients have, therefore, been based on a reservoir at spillway crest elevation. Pressure gradients were determined in the following manner. Upstream of a closed gate the pressure gradient is the static pool elevation. In an open gate passage, the pressure gradient is based on the velocity head due to the discharge for the appropriate number of open gates as shown on Plate No. 1-26 in Design Memorandum No. 1 (Hydrology and Hydraulic Analysis). Downstream of a closed gate, the pressure gradient is taken as being at the invert of the gate passage. The following table shows the resulting pressure gradients for the various combinations of gate openings.

Elevation of Pressure Gradient

	Upstream of gates		Downstream of gates	
No. of gates open	Open Passages	Closed Passages	Open Passages	Closed Passages
-	-	781	-	-
1	707.5	781	707.5	683.0
2	722.1	781	722.1	683.0
3	750.5	-	750.5	-

f. Conduit. - A 6-foot diameter reinforced concrete horseshoe conduit of constant cross section, conducts the flow from the transition to the exit portal at the head of the stilling basin. The conduit has an area of 29.85 square feet, a length of 541.5 feet and a constant slope of 0.833 percent.

g. Stilling Basin. - A stilling basin, 35 feet in length located downstream of a parabolic apron, is provided to produce a hydraulic jump for a range of flows up to a maximum of 1,670 c.f.s. At this flow a tailwater depth of 0.78 D_2 is provided. The apron slope follows the trajectory shape given by the expression $Y = 0.00833\ X + 0.00513\ X^2$ for a horizontal distance of 60.0 feet to the elevation of

the stilling basin floor at elevation 658.0. The stilling basin side walls have a 4 on 1 slope and are tangent to the horseshoe shape at the exit portal. Below the point of tangency, the sidewalls at the portal are curved to conform to the shape of the sides of the conduit. This curvature is gradually eliminated over a 4-foot transition beyond which the side walls have a 4 on 1 slope for their entire height. At the outlet portal the apron is shaped to conform to the bottom curvature of the horseshoe conduit. This curvature is gradually reduced over a distance of 15 feet so that the intersection of the apron and side walls conforms to the trajectory based on the equation $Y = .00805X^2$. In plan, the apron flares from the exit base width of 4.9 feet to the stilling basin floor width of 25 feet. The stilling basin floor is 35 feet long and will be provided with two staggered rows of baffles 2.5 feet high. The end sill is four feet above the stilling basin floor at elevation 622.0 with the top of the stilling basin wall at elevation 674.0, providing a minimum of 3.7 feet of freeboard. For a discharge of 1,670 c.f.s., the portal velocity of 56.0 feet per second is stilled to 7.2 feet per second at the end sill while for a discharge of 1,000 c.f.s., the corresponding velocities are 33.5 and 5.5 feet per second.

E. GEOLOGY

12. General. - A detailed description of the geology of the general region, the geology of the dam site, surficial and subsurface investigations and foundation conditions is contained in Design Memorandum No. 3 (Geology and Soils).

13. Spillway Weir Foundation. - a. Weir Description. - A description of the spillway weir crest shape and a general description of the weir respectively, are given in paragraphs 10 and 20 of this memorandum. Details of the weir shape are also shown on Plate No. 8-3.

b. Explorations. - Four test borings and four test pits were made in the immediate vicinity of the weir; test borings FD-20, FD-19, FD-18 and FD-17; and test pits FTT-1, FT-2, FT-5 and FT-6. The locations of these explorations are shown on Plate No. I-1 of Design Memorandum No. 3 (Geology and Soils). The base of the weir at elevation 771.0 is located well below the bedrock surface and at least five feet below the level of 90-100 percent rock core recovery from the test borings. The bedrock is dark grey, coarse-grained and well foliated. Weathering has occurred along occasional joints and foliation planes in the bedrock foundation of the weir. This weathering is generally insignificant and consists mainly of slight oxidation and staining of the rock in zones ranging from 1/8 to 1/2 inch thick which extend along both sides of the thin films or crusts of clay-like material which usually fill the joints and foliation planes. The rock cores from borings FD-18 and FD-19 show a few minor fractured zones. One such fractured zone encountered in boring FD-19, approximately 50 feet downstream from the right end of the weir, appears to be about 5 feet in thickness. There is no reason to assume from existing data, however, that this zone extends under the weir foundation.

c. Anchor Bars. - Under maximum surcharge loading, the concrete weir section, by itself is unstable. Consequently, the weir is anchored to the rock by means of #11 steel bars extending 9 feet into the foundation. As the stability

of the weir is dependent upon the mass of rock which the anchor bars engage, a further evaluation of joint weathering and fractured zones will be made if these conditions appear in the excavation during construction. If it is found necessary, longer anchor bars will be used.

14. Outlet Works Foundation. - The outlet works are entirely on bedrock which is adequate for support of the design loads. Seepage through the bedrock under the embankment should not present a major problem, and in general the effectiveness of grouting is questionable because of the clay-like fillings which occur in weathered fissures in the rock. Blasting for the conduit excavation may, however, loosen the adjacent rock and a grout curtain is therefore provided for the conduit at the center line of dam. The grout will be placed from 2 or 3 holes starting in the rock berm on each side of the conduit and extending below the base of the conduit.

F. CONCRETE

15. Concrete Materials. - Concrete materials are covered in detail in Design Memorandum No. 4 (Concrete Materials), which will be submitted for approval 15 February 1956.

G. STRUCTURAL DESIGN

16. Purpose. - This section of the design memorandum presents the design criteria, basic data and assumptions used in the design of the spillway weir, outlet works and miscellaneous structures. A brief description of the structures with loading conditions and assumptions used is included to show the design procedure. Typical computations are included in Appendix A, 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.

17. Scope. - The structural design of the spillway weir, outlet works intake, gate tower, conduit, stilling basin, and service bridge is outlined herein.

18. Design Criteria. - a. General. - All working stresses conform to those specified in the Engineering Manual for Civil Works, Part CXXI, Chapter I, "Stresses and Criteria for Structural Design," dated May 1953. Loading conditions, design assumptions and other design criteria are based on the following applicable parts of the Engineering Manual for Civil Works issued by the Office of the Chief of Engineers: Standard Practice for Concrete (Part CXX, October 1953); Gravity Dam Design (Part CXXII, October 1952); Structural Design of Spillways and Outlet Works (Part CXXIV, December 1952); Retaining Walls (issued as Part X, Chapter 9, dated July 1945) and Design of Miscellaneous Structures (Part CXXIX, Chapter 2, Conduits, Culverts and Pipes, June 1948). Accepted engineering practice has been employed in cases which the Engineering Manual for Civil Works does not cover.

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

Description	Unit Stress (p.s.i.)
Flexure - f'_c = 3,000 p.s.i.	
Extreme fiber stresses in compression	1,050
Shear - (v)	
Beams - no web reinforcement	90
Beams with properly designed web reinforcement	360
Flat slab at critical section	90
Footings - at critical section	90
Bond - (u)	
Deformed bars	
Top bars	210
In two-way footings (except top bars)	240
All others	300
Bearing - (f_c)	
Load on entire area	750
Load on one-third area or less - (maximum permissible)	1,125
When loaded area is greater than one-third, the stress coefficient should vary lineally between the above values.	
Modular Ratio - (n) = 10	

c. Reinforcement. - (1) Grade and Working Stresses. All reinforcement in the structures, including temperature and shrinkage reinforcement, is designed for the working stresses of new billet steel, intermediate grade, deformed bars with an allowable stress of 18,000 p.s.i. in flexural tension. The reinforcement will conform to the requirements of Federal Specification QQ-B-71a and to ASTM A-305-50T.

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

(3) Minimum Cover for Main Reinforcement. -

Structure	Minimum Cover (Inches)
Outlet works below El. 704, except Conservation weir	4
Intake tower above El. 704, bridge pier and abutment	3
Conservation weir	3
Bridge deck	1½
Superstructure and stairways	1½
Interior floor slabs	3/4
Interior girders and columns	1½
Interior floor beams	1

In some instances the concrete covering of stirrups, spacer rods and similar secondary reinforcement will be reduced by the diameter of such rods. However, in flat slabs, all reinforcement will have 3/4" minimum cover.

(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 will be avoided in the design.

(5) Temperature and Shrinkage Reinforcement. - Temperature and shrinkage reinforcement will be provided in slabs and walls where the main reinforcement extends in only one direction. This reinforcement, based on deformed bars, will provide for a minimum ratio of steel area to concrete area (bd) of 0.002 with a maximum spacing between bars of 18 inches.

d. Structural Steel. - The structural steel will conform to the Specifications for the Design, Fabrication and Erection of Structural Steel for Buildings, issued by the American Institute of Steel Construction. The allowable design working stresses conform to those given in the Engineering Manual for Civil Works using a basic stress of 18,000 p.s.i. (yield point 33,000 p.s.i. minimum) which is recommended for bridge structures and highway bridges.

e. Increase in Normal Working Stresses. - Normal allowable working stresses will be increased 33-1/3 percent where earthquake loads govern. Normal working stresses for the gate tower will be increased 33-1/3 percent when spillway surcharge loading is used. The superimposed load on the conduit is such that the allowable stresses will be increased as follows:

f_c = 1,800 p.s.i.
v = 180 p.s.i.
f_s = 20,000 p.s.i.
u = 350 p.s.i.

19. Basic Data and Assumptions. -

a. Controlling Elevations (m.s.l.)

Top of dam	802.0
Spillway design flood pool	797.1
Maximum water surface upstream of spillway weir	796.8
Spillway crest	781.0
Conservation pool	701.0
Gate sill	683.0
Conduit invert at exit portal	677.0
Stilling basin floor	658.0

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

Material	UNIT WEIGHT, pcf Saturated	Moist	Dry	Submerged
Rock Fill	135	115	115	72
Gravel Bedding and Pervious Fill	141	130	125	78
Impervious Fill	143	140	126	81
Foundation Till	143	140	126	81
Foundation Sand	128	115	106	66
Foundation Rock			160	
Concrete (plain and reinforced)			150	
Steel			490	

(2) Live Loads. - The following live loads have been 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.
Floor Slabs in Gate House	270 lbs. per sq. ft.
Floor Beams in Gate House	270 lbs. per sq. ft. plus heaviest equipment load where applicable
Design load on stairs	75 lbs. per sq. ft.
Service bridge	H-15 loading

c. External Water Pressure. - In cases where hydrostatic pressure affects the design of a structure, it has been assumed to act over the entire area in question under the full head. Specific uplift assumptions for each structure are given in the section where the structure is treated separately.

d. Earth Pressure. - Earth pressures used against the gate tower, intake structure and outlet works retaining walls have been determined in general in accordance with Part X Structural Design, Chapter 9 Retaining Walls. Inasmuch as these structures are relatively rigid, the "at rest" pressures have been used in some cases and will be discussed in the description of each structure where used. Pressures against the conduit due to embankment fill have been determined by the methods outlined in Part CXXIX, Chapter 2, Conduits, Culverts, and Pipes, using Case III.

e. Earthquake Forces. - A review of the earthquake history of New England reveals that shocks with an estimated intensity of VIII or greater on the Rosi-Forel scale have been felt in Canada and New York and also along the Eastern Seaboard. The Keene, New Hampshire area, lying between these regions, felt some of the effects of these shocks with reduced intensity, but otherwise has been free of serious earthquakes. Using the acceleration values which correspond to the Rosi-Forel ratings, as given in Engineering for Dams, Volume II, by Hinds, Creager and Justin, page 280, it is deduced that the area around Keene, New Hampshire, has never experienced a shock equal to 0.05g since Colonial times when the first records were made. A value of 0.05g is therefore believed to be conservative, especially for reinforced concrete structures which are well adapted to resist earthquake shocks. Therefore, this value has been used in the stability analysis of the intake tower with the reservoir empty and pool at spillway crest elevation.

f. Ice Pressure. - Horizontal forces due to the expansion of ice have been assumed at 10,000 pounds per lineal foot, acting 1 foot below the conservation pool elevation at points where the pool is in contact with the intake structure. Ice pressure has not been considered in the design of the spillway and stilling basin walls.

g. Wind Pressure. - A wind pressure of 30 pounds per square foot has been used in the design of the intake tower and service bridge.

h. Wave Pressure. - Wave pressure on the intake tower is negligible and has been disregarded in the design.

i. Frost Protection. - On the basis of temperature records at Keene, New Hampshire, 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 6 feet above foundation level has been used for structures founded on earth. The rock fill slope protection is not considered as effective frost protective cover for the service bridge pier and abutment which are founded on the dam embankment.

j. Location of Resultant. - The structures are designed so the resultant of the horizontal and vertical loads is held within the middle third except with earthquake loading where it is held within the middle half.

k. Factor of Safety Against Sliding. - For the spillway weir, a factor of safety against sliding of 4 has been held based on shear-friction of the concrete on rock. A minimum factor of safety of 1.45 has been mintained for the gate tower except on earthquake loading where it is 1.15.

20. Spillway Weir. - a. Description. - The spillway weir is located in a natural saddle in the right abutment which will be deepened and widened to form the spillway channel. It consists of a low, concrete ogee section, 145 feet long at spillway crest elevation 781.0 which is 10 feet above the bedrock foundation. To maintain stability against overturning and sliding, #11 anchor bars will be grouted into bedrock. Details of the weir shape are described in paragraph 10 and are also shown on Plate No. 8-3. The concrete weir will terminate at the rock sides of the spillway channel without end walls or facing. In the event the rock is found to be badly fractured or weathered when the channel is excavated, a reinforced concrete lining will be placed at one or both ends of the weir.

b. Stability Analysis. - The critical loading was found to be with the water at maximum surcharge level of 796.8 and tailwater at El. 783.6. Uplift considered was due to full head over 100% of the base area. The hydrostatic lateral pressure on the downstream face was taken at 60% of head over the projected area. With this loading condition at the juncture of concrete and rock, the resultant falls outside of the base and it is necessary to utilize some of the rock underlying the weir as acting with the weir. This is accomplished by using anchor bars embedded 9 feet in the rock and considering the engaged 8 ft. block of rock acting with the weir. Analyzed on this basis, with the stability computed about a plane 8 feet below the bottom of concrete, the bearing is then distributed over nearly the full base area.

c. Anchors. - In order to anchor the concrete weir firmly to the underlying rock, it is necessary to use #11 bars grouted into the rock to a depth of 9 feet at an angle of 30 degrees with the vertical. The spacing of the anchor bars will be 6'-0" in each direction.

d. Sliding. - It was determined that there is enough resistance in the rock to overcome sliding if the 8 feet of rock to which the concrete is anchored is considered acting with the weir. Only the rock directly below the base of concrete was considered as resisting the sliding forces.

21. Outlet Works Intake. - a. Description. - The intake structure just upstream of the intake tower is partially embedded in rock and consists of three channels. The outside walls above the rock surface are gravity type while below the rock surface there will be a 2'-0" thick concrete lining anchored to the rock. The dividing walls between channels, together with the upstream closure section form the control weir for maintaining the conservation pool.

b. Design. - The gravity side walls were investigated for loading conditions with the reservoir full and with a rapid drawdown. Under the latter condition the fill behind the wall was considered saturated but exerting no hydrostatic pressure. In determining the lateral soil pressure, "at rest," coefficients

were used, inasmuch as the structure will be very rigid. If active pressure coefficients were to be used, no appreciable reduction in the section could be made. The gravity section of the walls will be backed by a 3-foot layer of pervious fill containing a 6-inch, V.C. horizontal drain adjacent to the landside base. Rock anchors are to be provided for the concrete lining in rock in order to overcome the possibility of hydrostatic pressure that could exceed the downward weight of the walls and base when a rapid drawdown occurs and hydrostatic pressure remains due to water remaining in the earth dam.

22. Gate Structure. - a. Description. - The gate structure consists of a superstructure, tower and gate section.

b. Superstructure. - The superstructure is octagonal in shape with a flat roof and integral cast in place parapets. A flat roof has been substituted in place of the peaked roof shown in Design Memorandum No. 5. (General Design Memorandum) to permit conventional waterproofing. It also will allow the use of a downspout to prevent rain water from staining the walls. The roof will be a 6-inch concrete shell while sidewalls and parapets will be of concrete, 12 inches in maximum thickness. Architectural treatment is held to a minimum in the interest of economy. Floor slab and beam live loadings were determined as an equivalent uniform live load giving the same moment as that obtained by standing a gate upright on the critical span. Access to the floors below will be by means of a long radius curved concrete stairway of conventional design.

c. Tower. - The tower is octagonal in shape with a circular inside shaft 16 feet in diameter. It has been designed as a continuous horizontal ring. The maximum loading for the lower portion will be with reservoir at spillway crest elevation and the inside of the tower empty. The upper portion of the tower will be governed by loading resulting from the pool at spillway surcharge and design stresses increased by 33-1/3 percent.

d. Gate Chamber. - The gate chamber is 25 feet in diameter with walls designed vertically spanning from conduit roof level to floor at El. 712.63. The conduit walls, roof, and base slabs are of necessity of such massive proportions that only nominal stresses will be obtained under loading conditions of alternate gates opened or closed.

e. Stability. - The gate tower, as a whole, was analyzed for the following conditions of loading:

Case I - Reservoir empty, wind and earth downstream face.

Case II - Reservoir to spillway crest, all gates open.

Case Ia-Same as Case I plus earthquake forces acting on downstream side.

Case IIa-Same as Case II plus earthquake forces acting on downstream side.

Case III - Reservoir to El 797.1, all gates open.

The resultant force was found to fall within the middle third in Cases I, II and IIa, and within the middle half in Case Ia. Maximum bearing under Case Ia was found to be 9,050 lbs. per square foot with "at rest" earth pressure used in all cases. It is evident that the resultant will be within the middle half for Case III which will be investigated in the final design.

23. Conduit. - a. Description. - The conduit consists of a transition section 26 feet in length leading from the three 2'-6" x 4'-6" rectangular conduits to a 6'-0" horseshoe shaped conduit. The 6'-0" horshoe conduit extends from the end of the transition section to the stilling basin and is on rock for the entire length. The maximum conduit cross-section of the center line of dam is shown on Plate No. 8-3, and design computations for this section are included in Appendix A. Where loading conditions permit, the conduit thickness is reduced from 2'-6" to 2'-0" and 1'-6" respectively.

b. Design Loading. - Due to the depth of fill over the conduit and the proximity of the natural rock surface to the elevation of the top of conduit, the design was based on Case III as outlined in the Engineering Manual, Part CXXIX, Chapter 2. The following loading conditions govern the design of the conduit:

(1) Upstream of Sta. 7 + 55. -

Case I. - Pool at El. 701 after drawdown from spillway crest El. 781. For vertical load use 150% of submerged earth weight and 100% weight of water to El. 701 plus 150% of weight of earth saturated above El. 701. For horizontal load use 50% in above assumptions where 150% appears.

Case II. - Pool at El. 797.1 before seepage develops. For vertical load use 150% of submerged earth weight and 100% weight of water to El. 701 plus 150% of weight of moist earth above El. 701 plus 100% of weight of water from surface of dam to El. 797.1. For horizontal load substitute 50% in above assumptions where 150% appears.

Case III. - Pool at El. 797.1 with previous development of steady seepage. For vertical load use 150% weight of submerged earth + 100% of weight of water from conduit to El. 797.1. For horizontal load substitute 50% in above assumption where 150% appears.

(2) Downstream of Sta. 7 + 55. -

Case I. - Pool at El. 701 after drawdown from spillway crest El. 781. For vertical load use 150% of submerged earth weight and 100% of weight of water to El. 701 plus 150% of weight of saturated earth to seepage line plus 150% of weight of moist earth from seepage line to surface of dam. For horizontal loads substitute 50% in above assumptions where 150% appears.

Case II. - Pool at El. 781.0. For vertical load use 150% of weight of saturated earth to seepage line plus 150% of weight of moist earth from seepage line to surface of dam. For horizontal load use 50% instead of 150% in the above assumption.

24. Stilling Basin. - The stilling basin walls and base slab will be subjected to the same type of loading conditions as at the conduit intake and design will be based on the same premises.

25. Service Bridge. - The service bridge consists of two 95-foot span deck Warren steel trusses with verticals. The bridge trusses, abutments and center pier are of conventional design following accepted practice. Investigations were also made for a bridge using built up and rolled steel girders in several span lengths. The adopted design was found to be the most economical.

26. Housing Facilities. - a. General. - Housing facilities for the damtender will be provided at the site because of the following considerations.

(1) The gates will have to be operated on short notice.
(2) Rainfall and reservoir stages must be observed frequently.
(3) Communication will be facilitated during emergencies.
(4) To safeguard Government property.

b. Description. - An existing 1½ story wood frame, seven (7) room dwelling in good condition is located about 230 feet from the downstream toe of the dam and about 140 feet from the stilling basin, (See Plate No. 8-1). The main house is 24 x 28 feet in plan with a double dormer on the east roof. There is also a one-story ell, 16 x 22 feet in plan and an open porch, 16 x 14½ feet. It has clapboard siding and a full basement under the main house and ell. Installed facilities include hot water, oil heat and water obtained from a well in the yard. Based on an inspection of the exterior, it is considered that with some minor repairs, painting and installation of hardwood flooring, the house may be made entirely adequate for use as an operator's quarters at considerably less cost than would be necessary for a new dwelling.

c. Acquisition. - This property is located immediately adjacent to the dam and in the work area. Under the project plan, direct access from the property to Keene (westerly along Peg Shop Road) will be cut off by the spillway channel which will not be bridged. Construction of the dam in the immediate vicinity will reduce the properties' value as a "retreat," which use it now serves, and it is deemed in the best interest of the Government to acquire this real estate as a part of the land acquisition for the project. The remaining egress will be across Government property as the 47-year old bridge over Otter Brook will be abandoned to avoid its maintenance. Following an inspection of the premises and determination of the modifications, a separate contract will be let for the necessary repairs.

27. Administration Facilities. - a. Description. - Administration facilities consist of a combination utility building and garage 32 feet wide by 69 feet long, located as shown on Plate No. 8 - 1. The building is similar to other constructed

at flood control dams in the New England Division. It is provided with a 12'x12' office, 12'x8' stock room, 12'x12' storage room, 8'x10' fireproof storage room, toilet facilities, 10'x10' heater room, 20'x31' work shop and 24'x31' two bay garage. The work shop has been widened four (4) feet in order to accommodate the standby generator and the overall length of the building is now 69 feet instead of 65 feet as given in Design Memorandum No. 5. The building is equipped with an oil fired not water heating system.

b. Field Office. - Specifications will require that the building be ready for occupancy within 90 days after award of the contract in order that it may serve as the Government field office during construction. However, because it is located in the vicinity of the stilling basin, the contractor will be permitted to omit features not essential for occupancy until rock excavation in the area is completed.

28. Fire Fighting Facilities. - a. Description. - A separate gravity water supply system for fire fighting will be installed for protection of Government buildings and equipment. For location and layout of this system, see Plate No. 8-1, General Plan. The system consists of a ground-water-fed-storage pond situated high in the left abutment in the borrow pit area about 600 feet downstream of the dam center line, an intake and control structure with a 6-inch gate valve and a 6-inch pipe line 840 feet long leading to a hydrant between the operator's quarters and the utility building in the building area. The hydrant is inclosed in a hose house measuring 6'x6' in plan. The gravity system was selected in favor of installing a fire pump adjacent to the stilling basin, as described in paragraph 39-C of Design Memorandum No. 5, on the basis of cost. The gravity system has a lower first cost as well as lower operation and maintenance costs.

b. Design. - The design of the fire fighting system is based upon a fire stream demand of 250 g.p.m. with a 50 p.s.i. hydrant pressure or a 500 g.p.m. demand with 10 p.s.i. hydrant pressure which can be increased by a fire department pumper. The first criterion is controlling in respect to required head which has been computed at 129 feet. The storage pond dimensions provide 5 feet of freeboard, 4 feet for possible ice thickness, 3 feet for fire water storage and 2 feet for dead storage. Under the most adverse ice conditions, the fire storage volume available will furnish 500 g.p.m. for 1 hour and 28 minutes.

c. Alternate System. - Specific approval of the Contracting Officer will be required before construction of the intake and control structure at the storage pond is initiated. This will permit observation of ground water conditions in the borrow and storage pond area during dry weather. If the ground water supply is insufficient, a concrete storage tank with a capacity of 8,600 gallons (250 g.p.m. for 34 minutes) will be constructed at substantially the same location in lieu of the storage pond.

29. Access Road and Parking Areas. - A single access road, 1,630 feet long, leads from Branch Road (an existing highway) through the building area to a rotary and flagpole on the west abutment at the top of the dam. The maximum grade on this road does not exceed 10%. The road across the top of the dam leads from the

rotary to an intersection with the relocated highway on the east abutment. Both the access road and the road on the dam have an 18-foot pavement with 3'-6" gravel shoulders. The road has a 3-inch bituminous surface on a 12-inch gravel base. A paved parking area and driveway are provided in the building area, and a gravel surfaced public overlook parking area is provided adjacent to the dam and the relocated highway. In addition to the above access roads and parking areas, the contract specifications will require that a haul road be provided to the base of the intake structure and left in good condition for future Government use. For the location of the access road and parking areas, see Plate No. 8-1.

30. Log Boom. - The log boom is located across the reservoir about 800 feet upstream of the dam center line. It is anchored by means of concrete "dead-men" located on opposite banks at maximum water surface elevation 797.0. The log boom will collect trash and floating debris, and no trash racks will be installed on the outlet works intake. If, following construction, trash racks are found necessary, they will be installed in the stop log slots that are provided.

H. MECHANICAL DESIGN

31. Gates. - The flow through each conduit will be controlled by a 2'-6"x4'-6" cast iron hydraulically operated slide type service gate. This type of gate was selected in that it is particularly adapted for the intended use. They are of simple construction, less subject to damage from careless handling or from closing against obstructions; all of which will result in low maintenance costs. The gates are designed to withstand a hydrostatic head of 112 feet. The gate leaf, frame, bonnet and conduit lining will be of cast iron; bearing strips and seals will be of bronze. The hydraulic cylinder will be a forging bored and honed to the proper diameter with flanges integrally forged on each end. The normal maximum operating pressure will be about 250 p.s.i. with pressure relief valves set at 300 p.s.i.

32. Gate Control. - Each of the gates will be controlled by a four-way balanced piston valve located adjacent to the pumps. A motor-driven high pressure rotary pump will supply oil to the gate operating cylinders and a spare unit will be provided to insure continuous operation in the event of failure of the service pump. The entire system will operate full of oil at all times and a small oil storage tank will be installed near the pumps to act as a reservoir to insure an adequate supply.

33. Gate Hangers. - Locking of the gates at the fully open position will be accomplished by a semi-automatic gate hanger mounted on the ceiling of the gate chamber. This hanger automatically holds the gate open until manually released either from the gate chamber or by remote control from the pump room. A shear pin is provided to protect the hangar from damage if the oil pressure is applied without releasing the gate hanger lock.

34. Gate Position Indicators. - Gate position indicators will be provided at both the gate chamber and pump room floor levels.

35. Crane. - A 5-ton capacity electrical hoist, hand travel, monorail hoist will be installed in the operating house. The hoist will be used to service the operating house equipment and will have sufficient hook travel to reach to the bottom of the service well where it can pick up the gates.

36. Gate Chamber Drain. - A sump pit will be constructed in the base of the gate chamber with a gravity drain leading to the conduit. The drain will discharge downstream from the gates at the downstream end of one of the dividing piers, near the roof of the conduit. It is expected negative pressure at this location will provide a draining action even with the conduit flowing full. Provision will be made to permit installation of a motor operated sump pump of 30 g.p.m. capacity at the base of the sump pit. The pump will be installed to drain the gate chamber if necessary.

37. Heating System. - A heating system will be provided for the control house to facilitate maintenance operations during freezing weather. The system will be designed for an outside temperature of -10° F. and an inside temperature of 40° F. The heating system will be of the forced warm air type, consisting of an oil-fired furnace blower, duct system and oil burner of the pressure atomizing gun type with electric ignition.

38. Standby Gasoline Electric Generator Unit. - A gasoline engine driven generator will be provided for emergency operation of the oil pump and lighting system in the event of failure of commercial power. The unit will consist of a 60-horse power four-cylinder gasoline engine direct connected to a 480-volt, 3-phase, 60-cycle, 37.5 KVA .8 P.F. 30 KW generator mounted on a common base. The standby unit will be of ample capacity to operate the oil pump motor.

39. Computations. - Typical mechanical design computations are included in Appendix A.

I. ELECTRICAL DESIGN

40. Electrical Service. - Electric service will be obtained from a local utility line presently located on Peg Shop Road. The line is 2400V single phase 2-wire and will be changed to three phase by adding the third wire. A terminal pole will be installed for supplying necessary 480V, 3 phase, 3 wire, 60 cycle electric service to all Government facilities. The incoming utility supply voltage may be made 4160V, 3 wire instead of 2400V 3 wire as stated in Design Memorandum No. 5, to give better line voltage regulation. Also, the service voltage has been changed to 480V, 3 wire, as stated in Design Memorandum No. 5 to give better voltage regulation at the Gate House.

41. Emergency Source of Power. - A 37.5 KVA 30 KW 3 phase 60 cycle 480V 3 wire generator driven by a 60 B.H.P. gasoline engine will be installed in the Utility Building. When permitted by the operator, through positioning of a "Hand-Off-Auto" selector switch, the engine will start automatically on failure of normal utility supply voltage.

42. Distribution. - a. Power Circuits. - Utility secondary metering and a service entrance switch will be installed at the Utility Building. A 3-pole double throw switch with automatic transfer to automatically start the engine generator, manual return to normal utility service, will be installed in the Utility Building for emergency service to all Government facilities. Circuit breakers will be installed in the Utility Building for duplicate power and lighting feeders to the Gate House. The feeders will be run in 2E-2T underground duct from the Utility Building to the foot of the dam, up the downstream slope of the dam, within and near the top of the impervious section, thence in exposed steel conduit supported underneath the Service Bridge to the Operating Room Floor of the Gate House, a total distance of approximately 700 feet.

b. Lighting Circuits. - Distribution panels for miscellaneous small power and lighting will be installed in the Utility Building and the Gate House.

c. Emergency Lighting Circuits. - Self contained battery-charger-light units will be installed at four locations and connected to normally energized 120V receptacle circuits.

(1) Gate House - Gate floor level
(2) Gate House - Under Operating floor for stair illumination
(3) Gate House - Entrance floor level
(4) Utility Building - At standby unit

43. Communication. - Provision will be made for installation of communication facilities by N.E.T. & T. Co. as follows:

a. Space on incoming utility line for joint construction.
b. Phone in Utility Building.
c. Conduit in duct line from Utility Building to Gate House for phone at entrance floor level.

d. Conduit from terminal utility pole to operator's quarters for phone at this location.

44. Interior and Exterior Lighting. - 120V single phase lighting and receptacle circuits will be installed as required in accordance with the National Electrical Code. 120V will be obtained from 480V - 120/240V single phase dry type transformers.

45. Motors. - 440V 3 phase motors will be installed for driving the hydraulic pumps for gate operation and for the hoist in the gate house. Conduit will be provided for possible future use for the sump pump at 440 volts.

46. 110V single phase motors will be installed for all other facilities such as the oil-fired space heaters in the gate house and Utility Building, etc. Motor and control enclosures will be suitable for conditions encountered.

47. Control. - Gate control by hydraulic valves and pump motor control by push buttons will be located at the pump room floor level. Indicators will be installed for remote indication of gate position at pump room floor level of the Gate House. Electric hoist control will be located at the entrance floor level of the Gate House. The sump pump if installed will have float control. Magnetic starters will be full voltage type except starters for Gate Pump motors, which will be reduced voltage type.

48. Heating. - Small auxiliary electric heaters will be installed as required in the control switchgear and motors in the Gate House. All other heating, including the operator's quarters will be by oil-fired heating equipment.

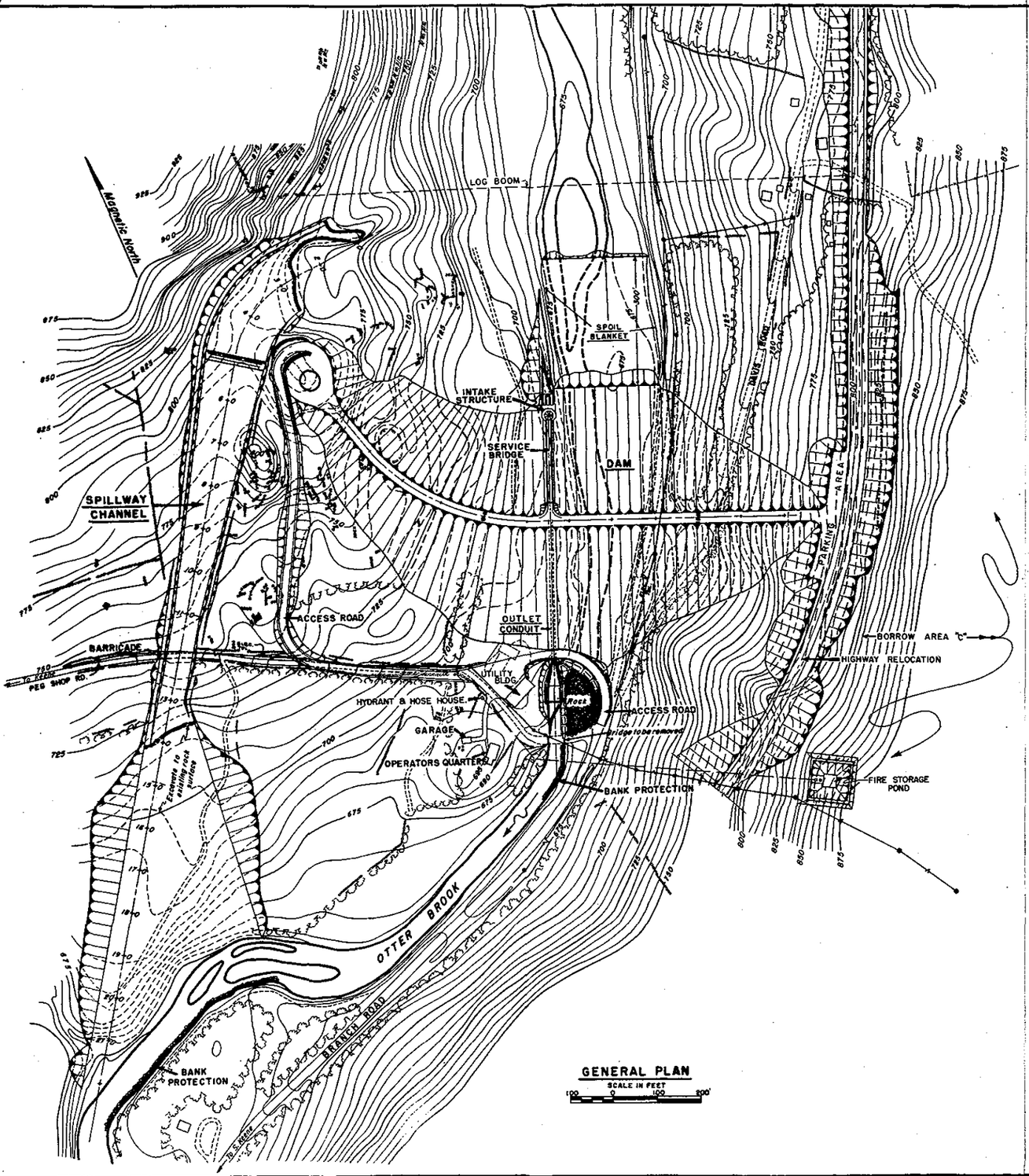
49. Grounding System. - The electric system distribution will consist of 3 phase 3 wire 480V and will be ungrounded. All conduit and equipment will be grounded in accordance with the National Electrical Code. A solid ground will be obtained at the Utility Building and operator's quarters water piping and again at the Gate House by connection to the intake pipe for the float well. The middle leg of the lighting transformers will be connected to these grounds.

50. Conduit and Cables. - All interior wiring will be in rigid steel conduit. Underground ducts will be non-metallic except for pole risers which will be rigid steel, galvanized and coated with protective compound. Type RH-RW dual rated 600V copper wire will be used generally in interior work in the operator's quarters. Type RHW will be used in the Utility Building and the Well Pump House, if one is installed. Type RR - Style USE will be used for the main power feeders to the Gate House and Type RHW will be used generally for power and control wiring in the Gate House.

51. Electrical load. -	Connected	Phase	Estimated Coincident Demand
Operator's Quarters			
Range	5KW		
Lighting, Ref., etc.	3KW		
	8KW	1	4KW
Well House (if installed)			
Pump, etc.	3KW	3	3KW

Electrical load (Cont.)	Connected	Phase	Estimated Coincident Demand
Garage	None		
Utility Bldg.			
Shop	3KW		
Receptacles	3KW		
	6KW	1	2KW
Gate House			
Floodlights	2KW		
Lighting	2KW		
Receptacles	1KW		
Heaters, etc	1KW	1	
Sump Pump (if installed)	½KW	3	
Auxiliary Hoist	1KW	3	7KW*
Pump Motors (2-5 HP)	8KW	3	
	15½KW		
(Locked Rotor Starting @ 6 KVA/HP = 30 KVA for either pump motor) (Use Reduced Voltage Starting)			
TOTALS	32½KW		16KW

* 1 Pump Motor only running.



NOTES

Elevations refer to Mean Sea Level Datum.
Existing power and telephone lines shown on this sheet to be relocated as required.

REVISION	DATE	DESCRIPTION	BY

CORPS OF ENGINEERS, U. S. ARMY
OFFICE OF THE DIVISION ENGINEER
NEW ENGLAND DIVISION
BOSTON, MASS.

DR. BY	TR. BY	CK. BY
J. W. L.	S. F. C.	G.W.H.

PROJECT ENGINEER

SUBMITTED BY: CHIEF, PLANN. & REPTS. BRANCH

CHIEF, ENGINEERING DIVISION

CONNECTICUT RIVER FLOOD CONTROL

OTTER BROOK DAM

GENERAL PLAN

OTTER BROOK, NEW HAMPSHIRE

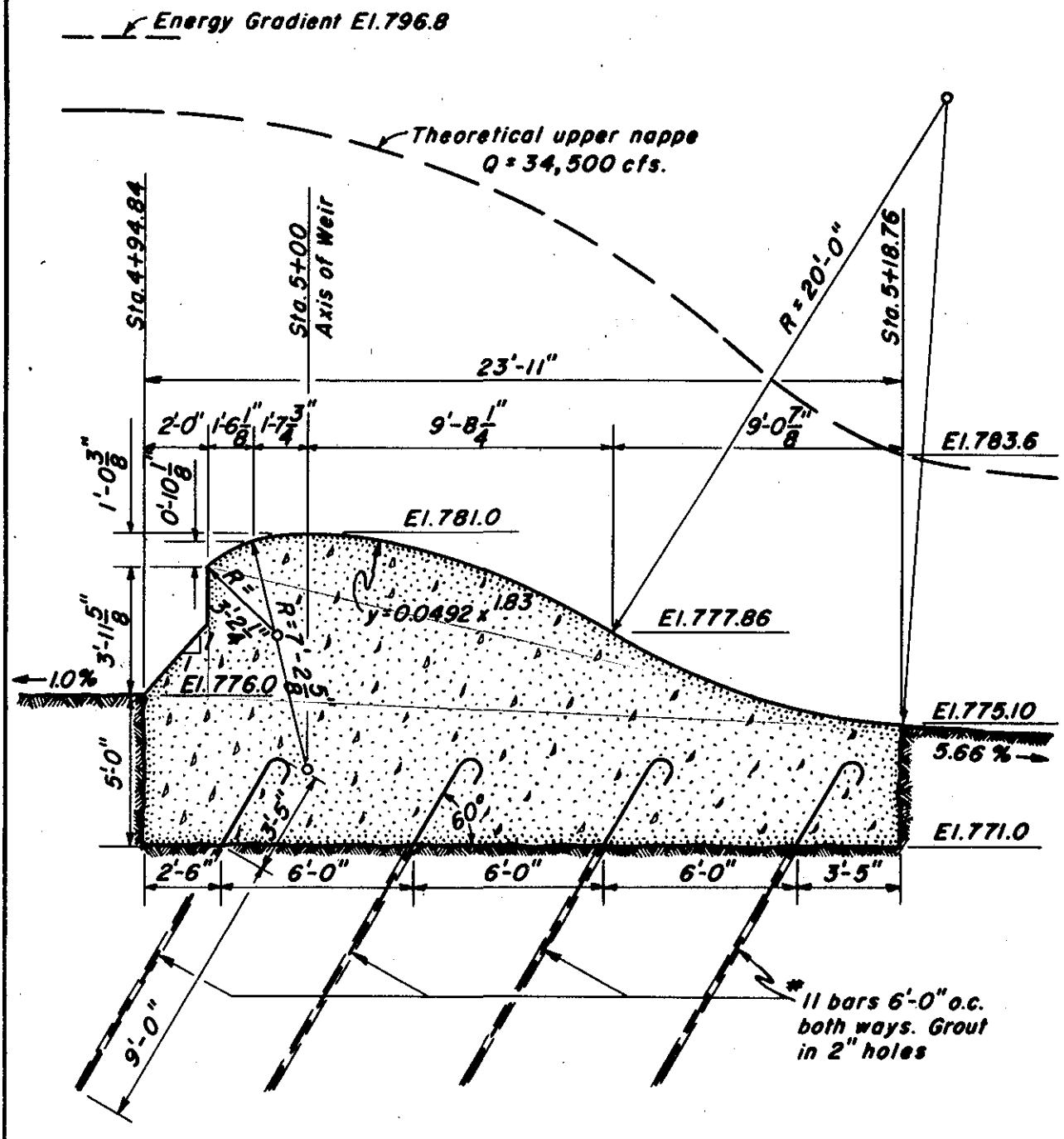
DATE FEB. 1956

APPROVED: LT. COL., C.E., FOR THE DIVISION ENGINEER

SCALE: 1" = 100'

DRAWING NUMBER CT-1-4072

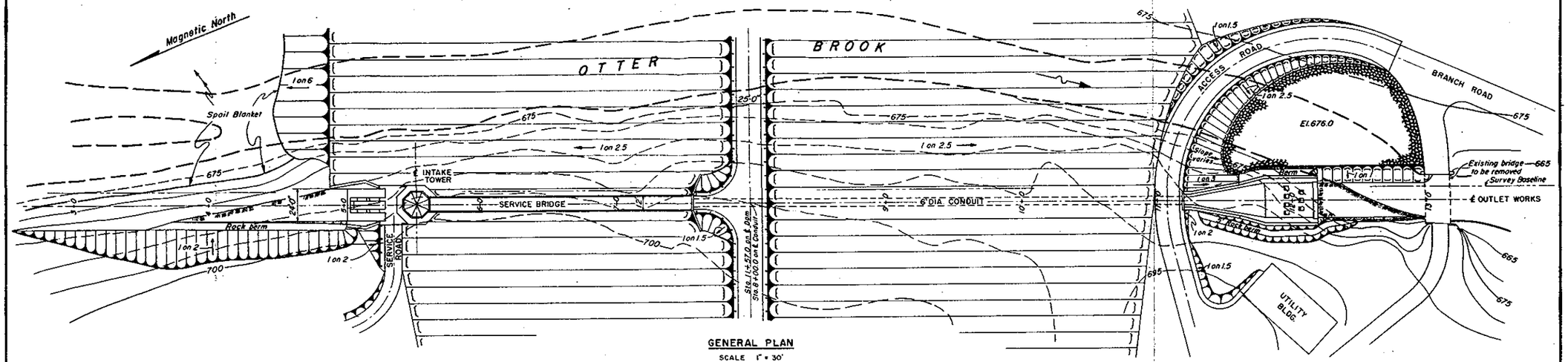
SHEET 1 OF 1



SECTION THRU SPILLWAY WEIR

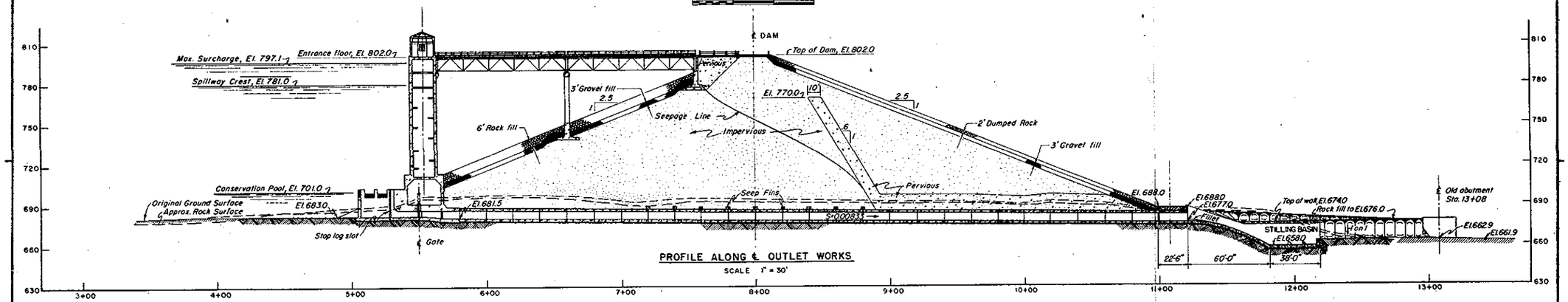
SCALE 1" = 5'-0"

CONNECTICUT RIVER FLOOD CONTROL
OTTER BROOK DAM
SPILLWAY WEIR
FEBRUARY 1956
OTTER BROOK, NEW HAMPSHIRE
NEW ENGLAND DIVISION BOSTON, MASS.



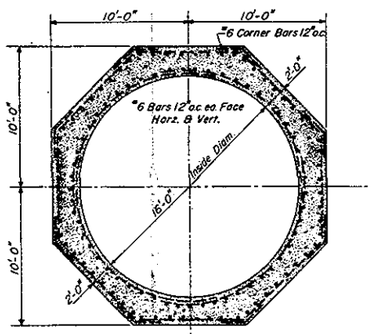
GENERAL PLAN

SCALE 1" = 30'



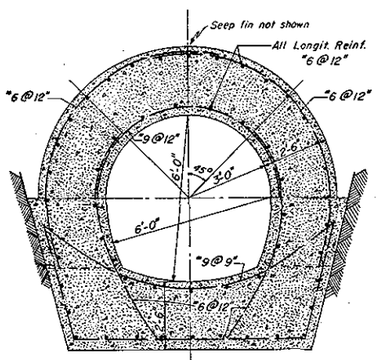
PROFILE ALONG ℄ OUTLET WORKS

SCALE 1" = 30'



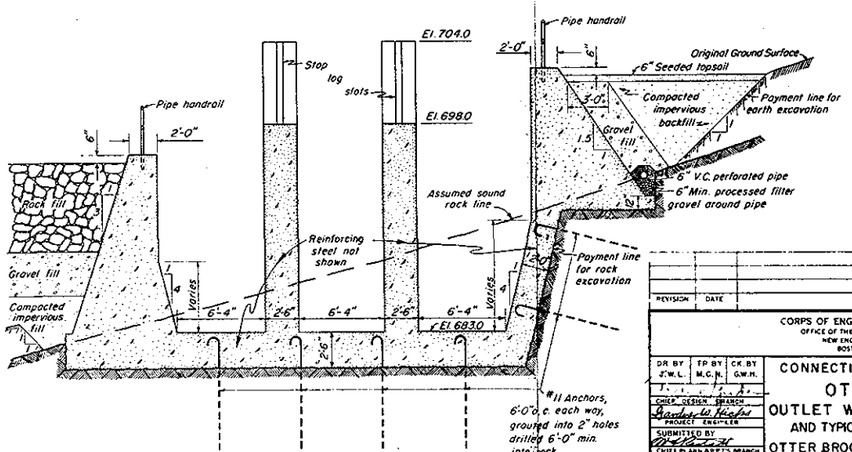
SECTION THRU INTAKE TOWER AT EL. 714.60

SCALE 1/4" = 1'-0"



CONDUIT SECTION AT ℄ DAM

SCALE 1/2" = 1'-0"



SECTION THROUGH INTAKE

SCALE 1/4" = 1'-0"

REVISION	DATE	DESCRIPTION	BY

CORPS OF ENGINEERS, U. S. ARMY
OFFICE OF THE DIVISION ENGINEER
NEW ENGLAND DIVISION
BOSTON, MASS.

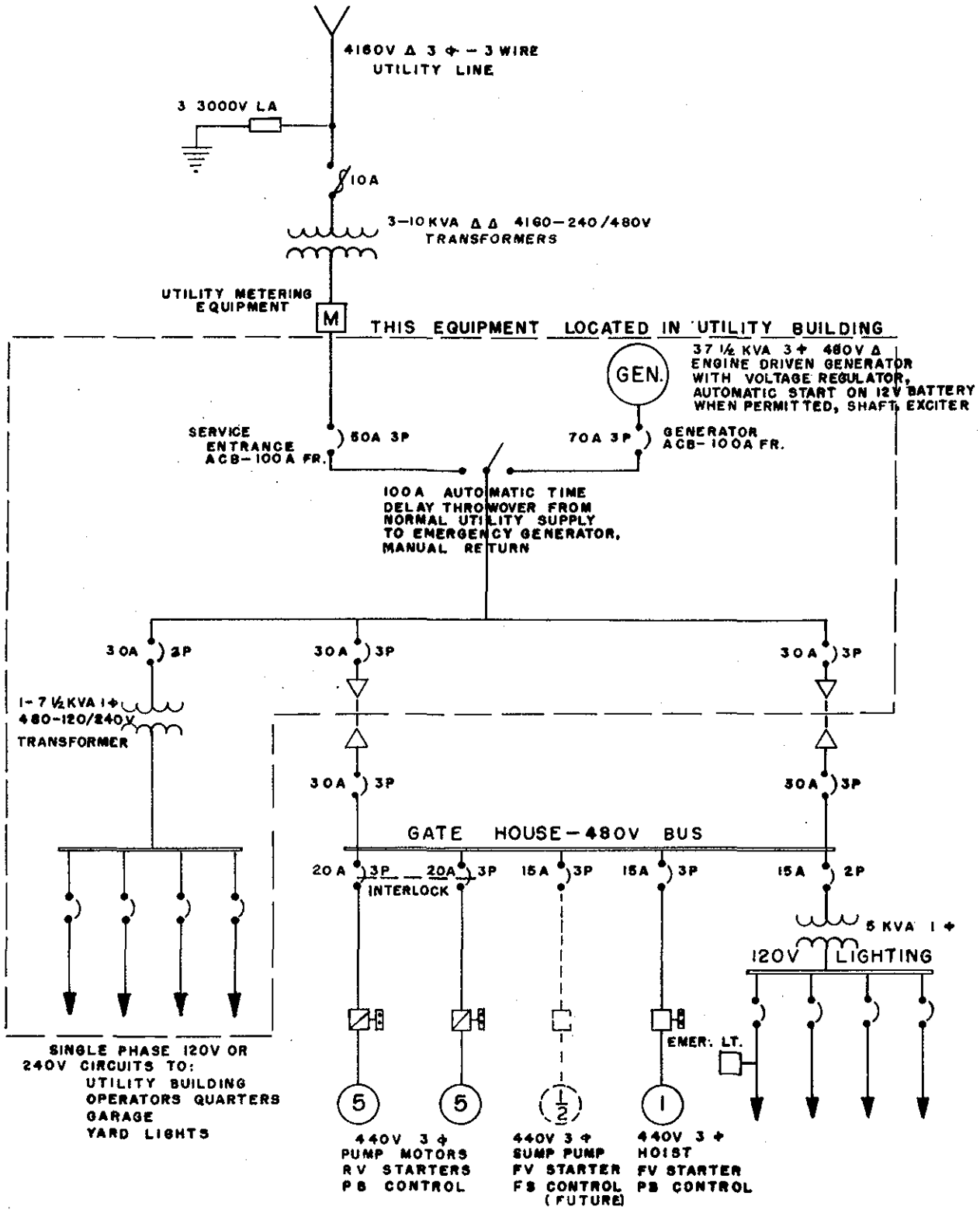
DR. BY	TR. BY	CK. BY
J.W.L.	M.C.H.	G.W.H.

CHIEF, DESIGN BRANCH
PROJECT ENGINEER
SUBMITTED BY
CHIEF, PLANN. & RPTS. BRANCH
APPROVED BY
CHIEF, ENGINEERING DIV.

CONNECTICUT RIVER FLOOD CONTROL
OTTER BROOK DAM
OUTLET WORKS, PLAN AND PROFILE
AND TYPICAL CONCRETE DESIGN SECTIONS
OTTER BROOK, NEW HAMPSHIRE

DATE FEB. 1956
APPROVED BY
LT. COL., C.E. FOR THE DIVISION ENGINEER

SCALE: AS SHOWN
DRAWING NUMBER
CT-1-4073
SHEET 1 OF 1



CONNECTICUT RIVER FLOOD CONTROL

OTTER BROOK DAM

ELECTRICAL DISTRIBUTION

PROPOSED ONE-LINE DIAGRAM

OTTER BROOK NEW HAMPSHIRE

PLATE NO. 8.4

APPENDIX A - TYPICAL DESIGN COMPUTATIONS

INDEX TO SHEETS

Title		Page No.
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3.	Intake Tower - Stability	A-15
4.	Intake Tower - Section at El. 714.60	A-25
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7.	Gate Loading	A-57
8.	Hydraulic Cylinder	A-58
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APPENDIX A - TYPICAL DESIGN COMPUTATIONS

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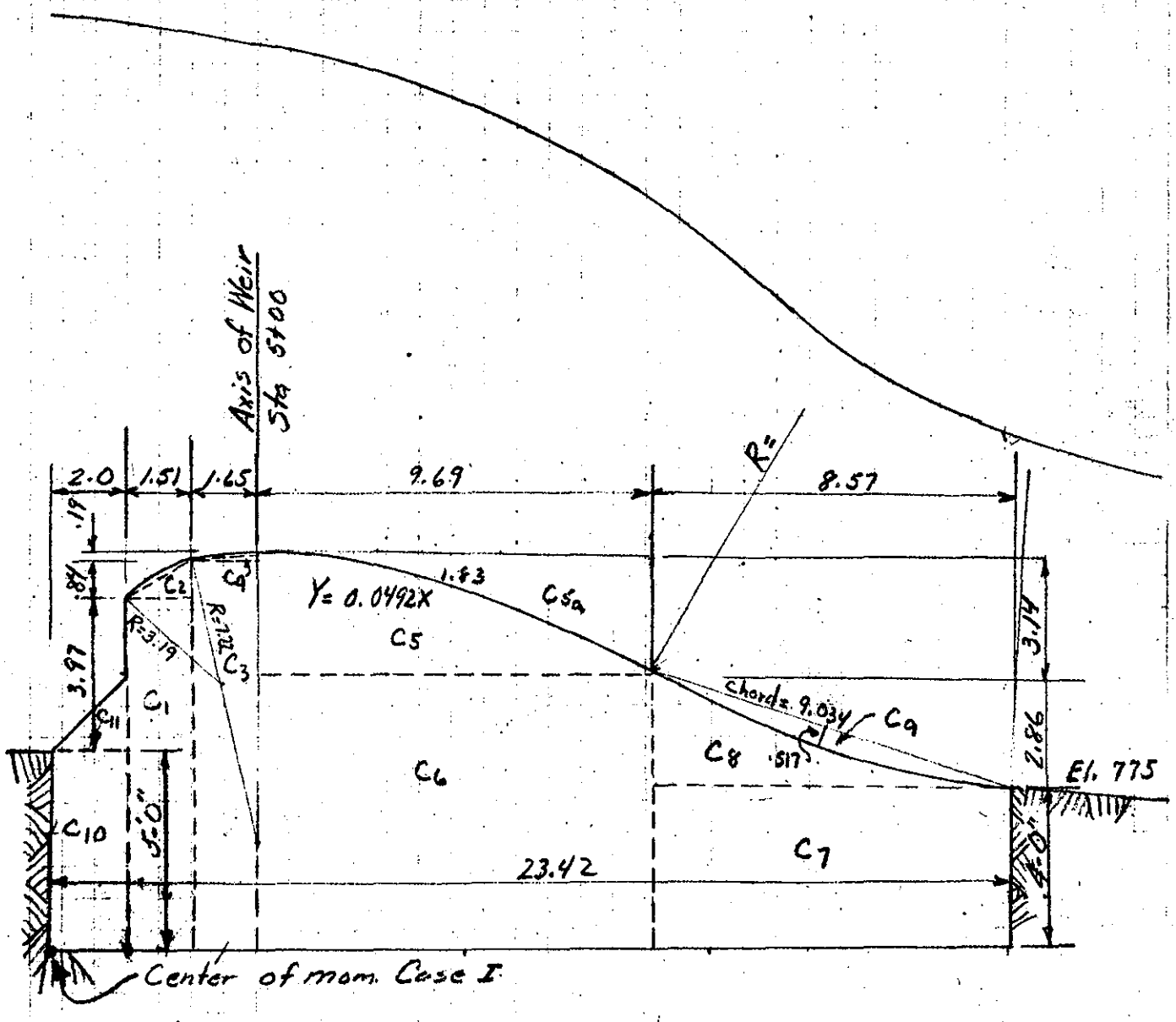
NED FORM 223
27 Sept 49

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

PAGE A-1

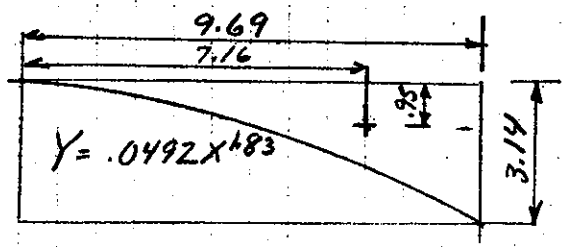
SUBJECT: Otter Brook Dam
COMPUTATION: Spillway Section
COMPUTED BY: H.E.W. CHECKED BY: DATE: 11/19/55



ARMY-NED-BOSTON

SUBJECT: Otter Brook Dam
COMPUTATION: Spillway Section
COMPUTED BY: H.E.W. CHECKED BY: DATE: 11/19/55

Find area and centroid of C1



$$Y = .0492x^{1.83} = \frac{x^{1.83}}{20.325}$$

$$\text{Area} = \int_0^{9.69} \frac{x^{1.83}}{20.325}\,dx = \left[\frac{x^{2.83}}{57.52}\right]_0^{9.69} = \frac{9.69^{2.83}}{57.52} = 10.75\ \square'$$

$$M_y \int_0^{9.69} \frac{x^{2.83}}{20.325}\,dx = \left[\frac{x^{3.83}}{77.85}\right]^{9.69} = \frac{9.69^{3.83}}{77.85} = 76.975$$

$$x = \frac{76.975}{10.75} = 7.16'$$

$$M_x = \frac{1}{2}\int_0^{9.69} \frac{x^{3.66}}{413.11}\,dx = \int_0^{9.69} \frac{x^{4.66}}{2 \times 413.11 \times 4.66} = \frac{9.69^{4.66}}{3850.11} = 10.25$$

$$y = \frac{10.25}{10.75} = .95$$

Spillway to be investigated for the following loading conditions

Case I - Reservoir empty wind load on downstream face

Case II - Reservoir to spillway crest, no tailwater

Case III - Reservoir at max. surcharge level tailwater at El. 783.6, full uplift over 100% of base, hydrostatic lateral pressure on downstream at 60%

Case IIIa - Same as Case III with effect of anchors to rock included.

NED FORM 223
27 Sept 49

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

SUBJECT Otter Brook Dam
COMPUTATION Spillway Section
COMPUTED BY H.E.W CHECKED BY RJM DATE 12/55

Stability Case I Loading

Item					Force	Arm	Moment
C_1	1.51 x 8.97	=	13.54	x .150	2.032	2.755	5.597
C_2	.84 x 1.51 x .5	=	0.63	x .150	0.095	3.007	0.286
C_3	9.81 x 1.65	=	16.19	x .150	2.428	4.335	10.525
C_4	1.65 x .19 x .5	=	.16	x .150	0.024	4.610	0.111
C_5	3.14 x 9.69	=	30.43	x .150	4.565	10.005	45.673
- C_{5a}		=	-10.75	x .150	-1.613	12.320	- 19.872
C_6	6.86 x 9.69	=	66.47	x .150	9.971	10.005	99.760
C_7	8.57 x 4.0		34.28	x .150	5.142	19.135	98.392
C_8	8.57 x 2.86 x .5	=	12.26	x .150	1.839	17.707	32.563
- C_9			- 3.12	x .150	- 0.468	19.135	- 8.955
C_{10}	2 x 5		10.00	x .150	1.500	1.00	1.500
C_{11}	2 x 2 x .5		2.00	x .150	.300	1.33	.399
			172.09		25.815		265.979

$$R = \frac{265979}{25.815} = 10.30$$

Add wind load

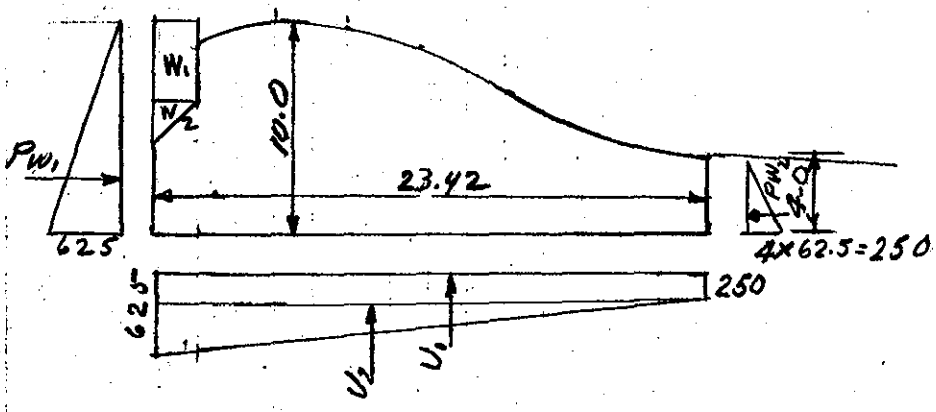
	Force	Arm	Mom
Concrete	25.815 ↓		265979
Wind = 30 x 6. = 180#	180 ←	7.0	1260
			264719

$$R = \frac{264.719}{25.815} = 10.25' \qquad e = 1.46$$

$$\text{Bearing} = \frac{25815}{23.42}\left(1 \pm \frac{6 \times 1.46}{23.42}\right) = \begin{array}{l} 1510\ \#/\square' \text{ at heel} \\ 690\ \#/\square' \text{ at toe} \end{array}$$

SUBJECT Otter Brook Dam
COMPUTATION Spillway Section
COMPUTED BY H.E.W. CHECKED BY RPM DATE 12/55

Case II Loading



Item		Force Horiz	Force Vert.	Arm	Moment
Conc.			+25.815		265.979
W_1	2 x 3 x 62.5		.375	1.00	.375
U_1	250 x 23.42		- 5.855	11.71	- 68.562
U_2	375 x 23.42 x .5		- 4.391	7.81	- 34.296
P_{w1}	625 x 10.0 x .5	+3.125		3.33	+ 10.406
P_{w2}	250 x 4 x .5	- .500		1.33	- .665
		+2.625	+15.944		173.237

$$R = \frac{173.237}{15.944} = 10.87 \qquad e = 0.84$$

$$\text{Bearing} = \frac{15.944}{23.42}\left(1 \pm \frac{6 \times 0.84}{23.42}\right) = \begin{array}{l} 830^{\#} \text{ at heel} \\ 530^{\#} \text{ at toe} \end{array}$$

$$\frac{\Sigma H}{\Sigma V} = \frac{2.625}{15.944} = .16 < .65$$

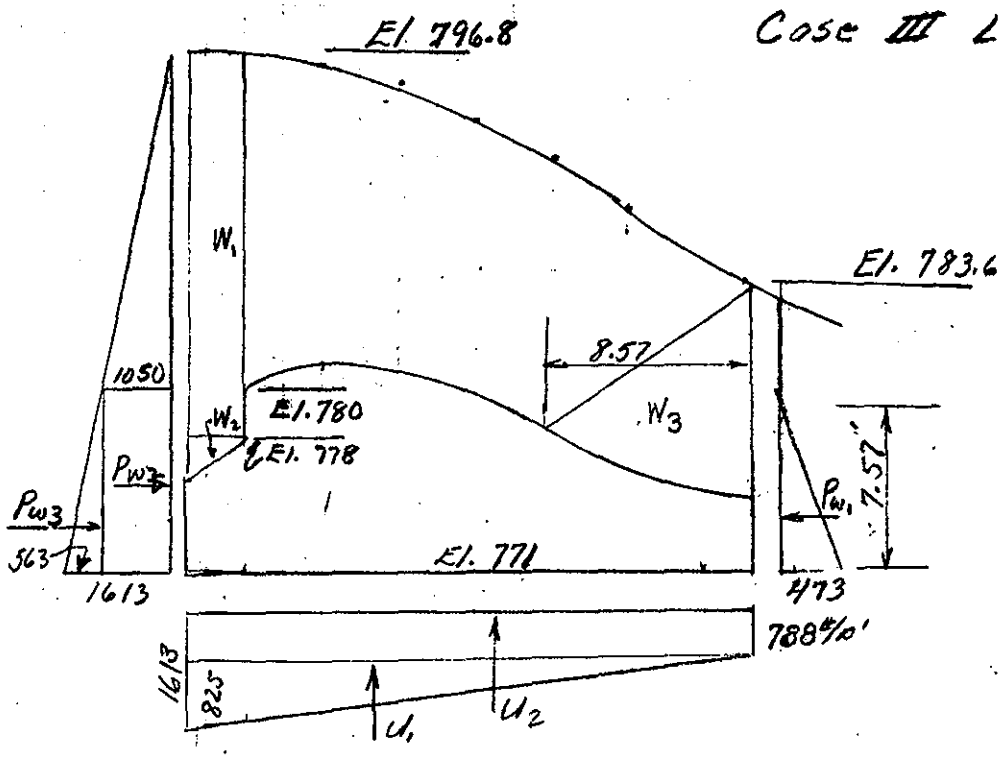
Factor of safety = 4.0

NED FORM 223
27 Sept 49

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

SUBJECT Otter Brook Dam
COMPUTATION Spillway Section
COMPUTED BY H.E.W. CHECKED BY [illegible] DATE 11/22/55

Case III Loading



Lateral pressure due to tailwater to be taken at 60% of tailwater head (¶ 2.03 Chapt 2 Part CXXII)

$= 12.6 \times 62.5 \times .6 = 473$

Uplift varies from full headwater to full tailwater values

Item		Force Hor. #	Force Vert. #	Arm ft.	Moment ft#
Conc			25 815	10.3	266000
U_1	825 × 23.58 × .5		−9,730	7.86	−76500
U_2	788 × 23.58		−18550	11.8	−219000
P_{W1}	473 × 7.57 × .5	−1790		2.52	− 4500
P_{W2}	1050 × 9.0	+9450		4.50	+ 42 525
P_{W3}	563 × 9. × .5	+2533		3.00	+ 7599
W_1	2 × 18.8 × 62.5		2350	1.00	+ 2350
W_2	2 × 2 × .5 × 62.5		125	.67	83
W_3	8.57 × 8.6 × .5 × 62.5		2303	20.6	+47500
		+10,193	+2313		+66 057

$\frac{\Sigma M}{\Sigma V} = \frac{66\,057}{2313} = 28.5'$ – outside base – use anchors to rock – see Case IIIa

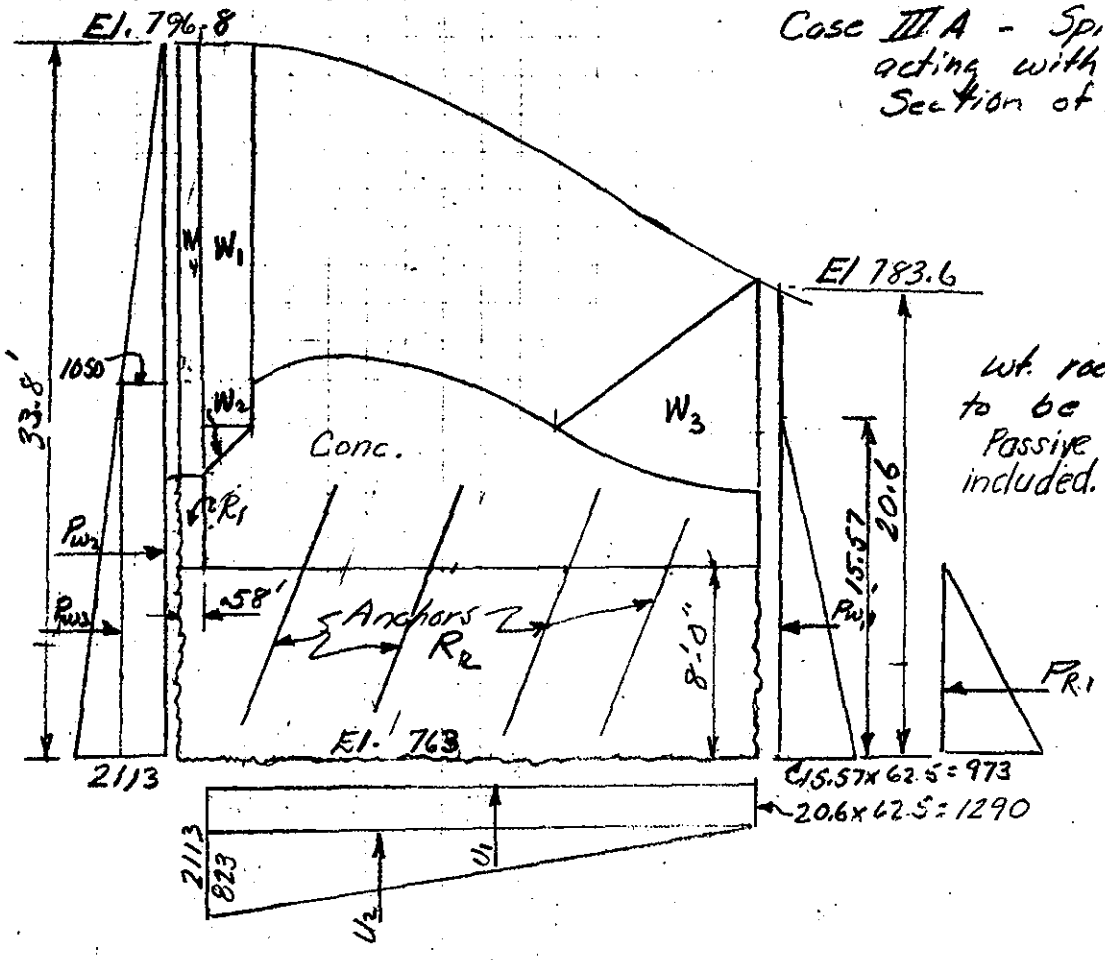
ARMY-NED-BOSTON

NED FORM 223
27 Sept 49

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

SUBJECT: Otter Brook Dam
COMPUTATION: Spillway
COMPUTED BY: HEW CHECKED BY: RSM DATE: 12/6/55

Case III A - Spillway considered acting with an 8'-0" Section of rock



wt. rock assumed to be 160 #/cu.ft
Passive resistance included. (P_{R1})

Item		Force Horiz	Force Vert	Arm	Moment
Conc			25.815	10.88	280,867
R_1	5 x .58 x 160		464	.29	135
R_2	8 x 24.00 x 160		30,720	12.00	368,640
U_1	1290 x 24.		-31,000	12.00	-372,000
U_2	823 x 24.00 x .5		-9,860	8.00	-78,900
P_{w1}	973 x 15.57 x .5	-7575		5.19	-39,314
P_{w2}	1050 x 17	+17850		8.5	+151,725
P_{w3}	1063 x 17 x .5	+9036		5.67	+51,234
W_1	2 x 18.8 x 62.5		2350	1.58	+3,713
W_2	2 x 2 x .5 x 62.5		125	1.25	156
W_3	8.57 x 8.6 x .5 x 62.5		2303	21.14	48,685
W_4	20.8 x .58 x 62.5		754	.29	219
		-19311			
P_{R1}		+19311		2.67	-51,560
			21,671	19.3	+363600

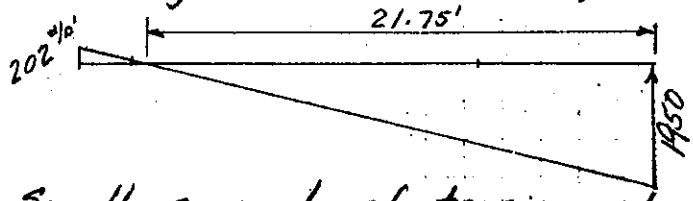
ARMY-NED-BOSTON

SUBJECT Otter Brook Dam
COMPUTATION Spillway
COMPUTED BY HEW CHECKED BY RMV DATE 12/6/55

Case III A

$$\frac{\Sigma M}{\Sigma V} = \frac{363600}{21671} = 16.75' \qquad \frac{2}{3} \times 24 = 16'$$

Bearing Pressure Diagram

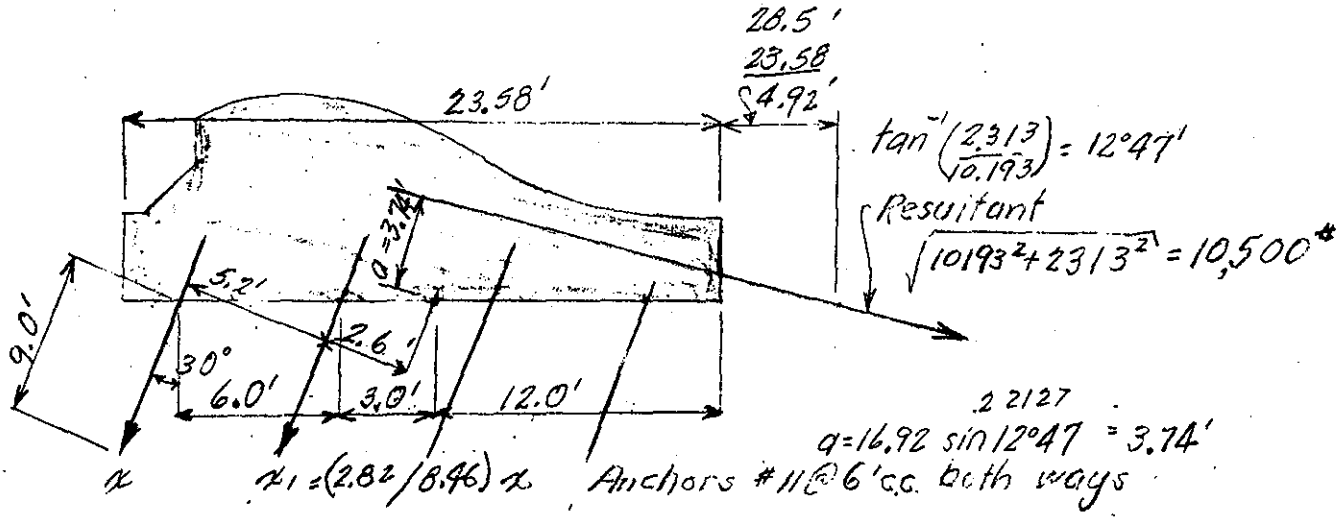


$$\text{Bear } P = \frac{21671 \times 2}{21.75} = 1950 \text{ \#/□'}$$

Small amount of tension at heel is negligible as quantity of rock considered acting with concrete is conservative

SUBJECT Otter Brook Dam
COMPUTATION Spillway Section
COMPUTED BY [illegible] CHECKED BY DATE 12/6/55

Determination of tension on anchors



Assume tension in anchors sufficient to bring resultant to mid-point of base. Anchors x and x_1 only would be stressed, others would provide added safety.

$M_{mid\ pt.}$: $7.8x + \frac{2.6^2}{7.8}x = 3.74 \times 10500$

$x(7.8 + .87) = 39300$

$x = 4540^{\#}/1$ Assuming anchors 6' c.c. both ways, pull per anchor $= 6 \times 4540 = 27240^{\#}$ ea. (outside line)

Using #11 anchors and 50% effective bond,

$u = 27240/4.43 \times 108 = 57^{\#}/\square'' < .50 \times 300$ OK

$f_t = 27240/1.56 = 17{,}400^{\#}/\square''$ OK

} Upstream line of anchors.

Check:

	Forces		Moments
Σ	↓ 2313#		66 057'#
Anchors			
x: 4540 × .866	↓ 3920	3.0	11 760
1510 × .866	↓ 1310	9.0	11 790
Σ	7543	12.0	89 607'#

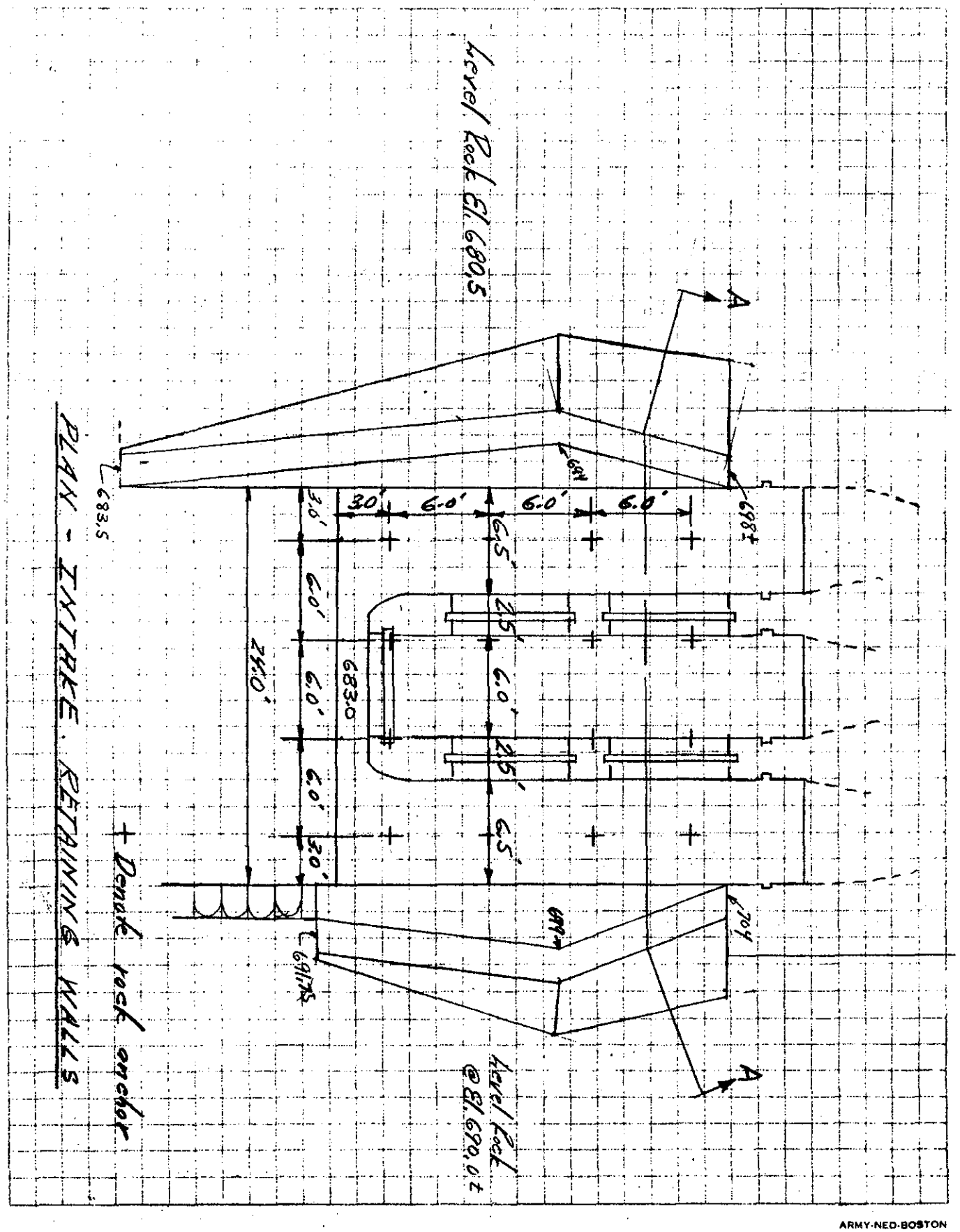
$f = \Sigma H/\Sigma V = 2313/10193 = .228 < .65$ - consider shear included, below.

$$(S_{s-f}) = \frac{f\Sigma V + rsA}{\Sigma H} = \frac{\overset{535}{.228 \times 2313} + \overset{127000}{0.5 \times 75(23.58 \times 144)}}{10193} = 12.5 > 4 \text{ OK.}$$

SUBJECT OTTER BROOK DAM

COMPUTATION Retaining Walls @ Intake

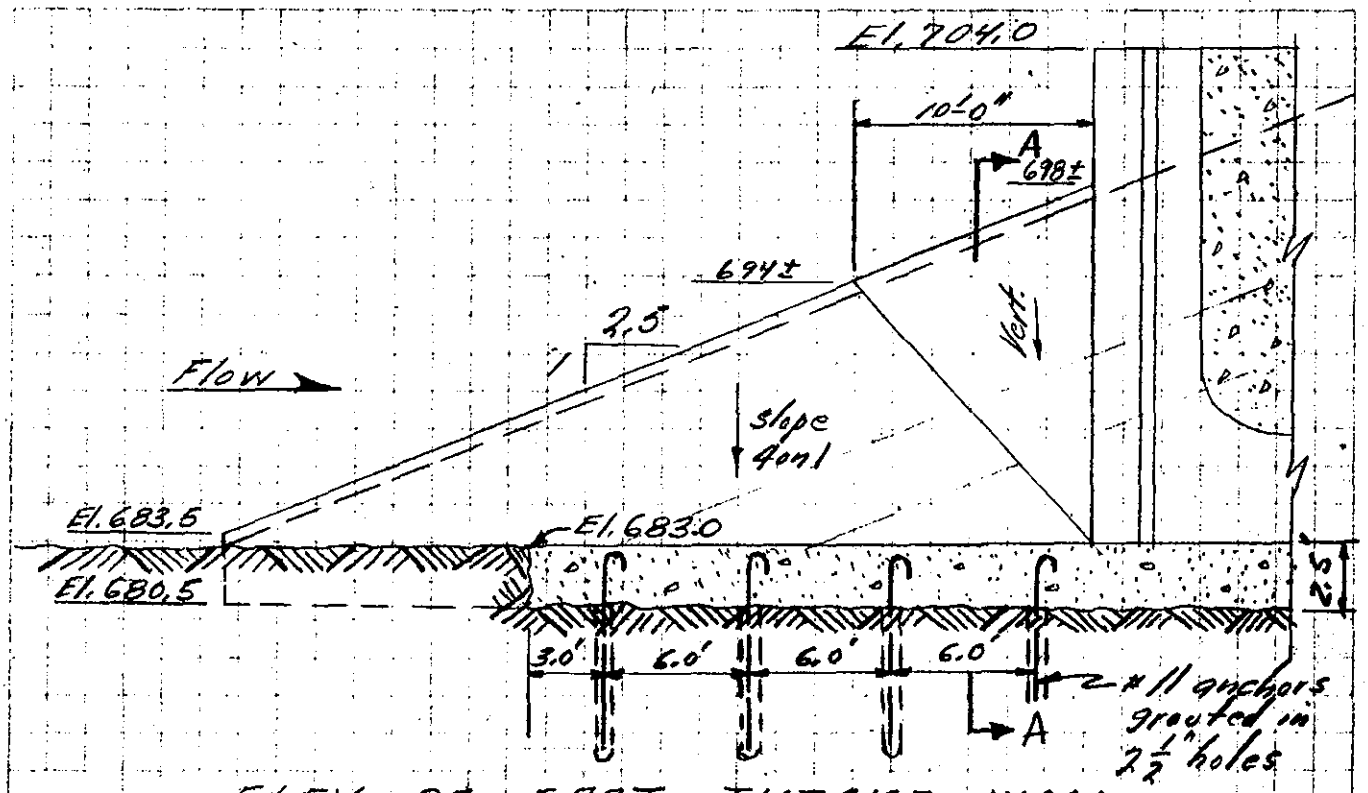
COMPUTED BY JSM CHECKED BY RJM DATE Nov. 1955



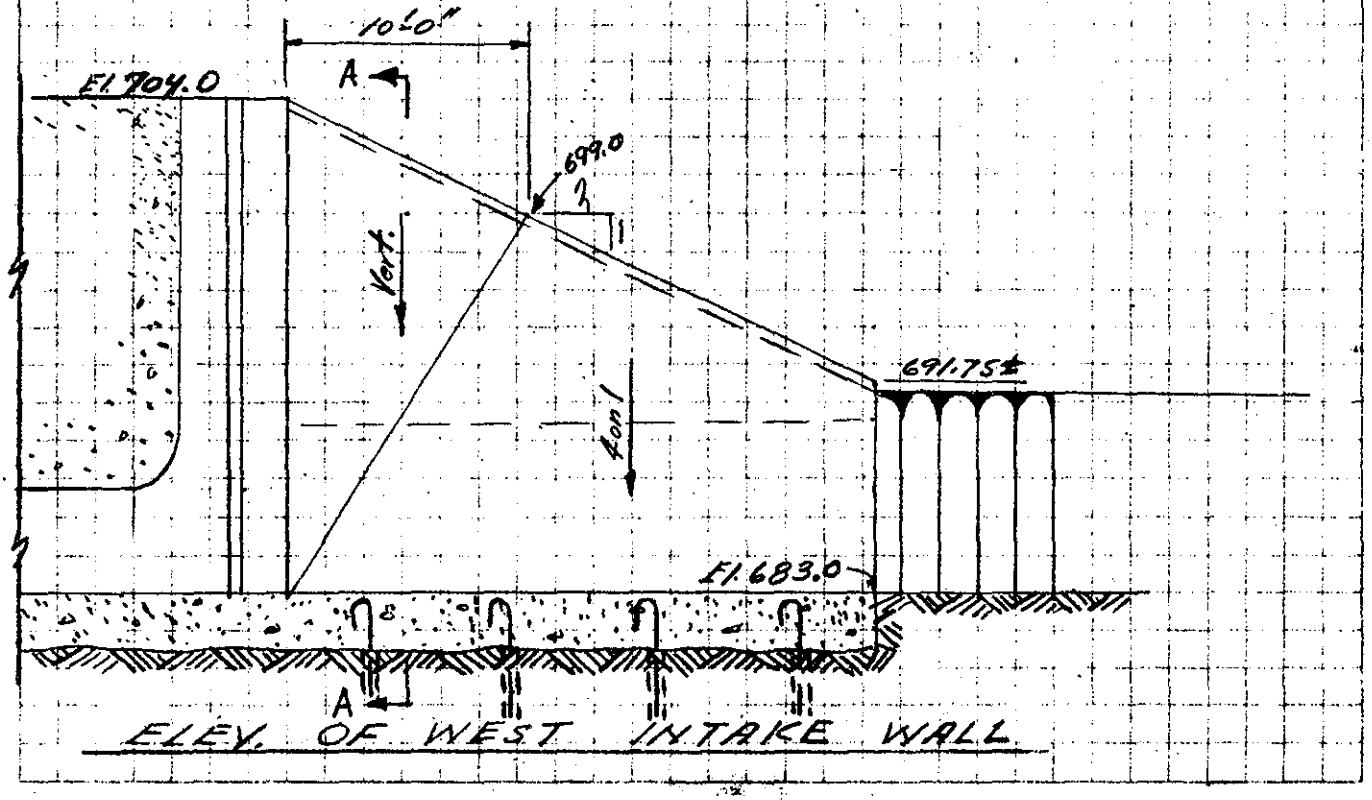
NED FORM 223
27 Sept 49

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

SUBJECT OTTER BROOK DAM
COMPUTATION Retaining Walls @ Intake
COMPUTED BY JSM CHECKED BY RM DATE Nov. 1955

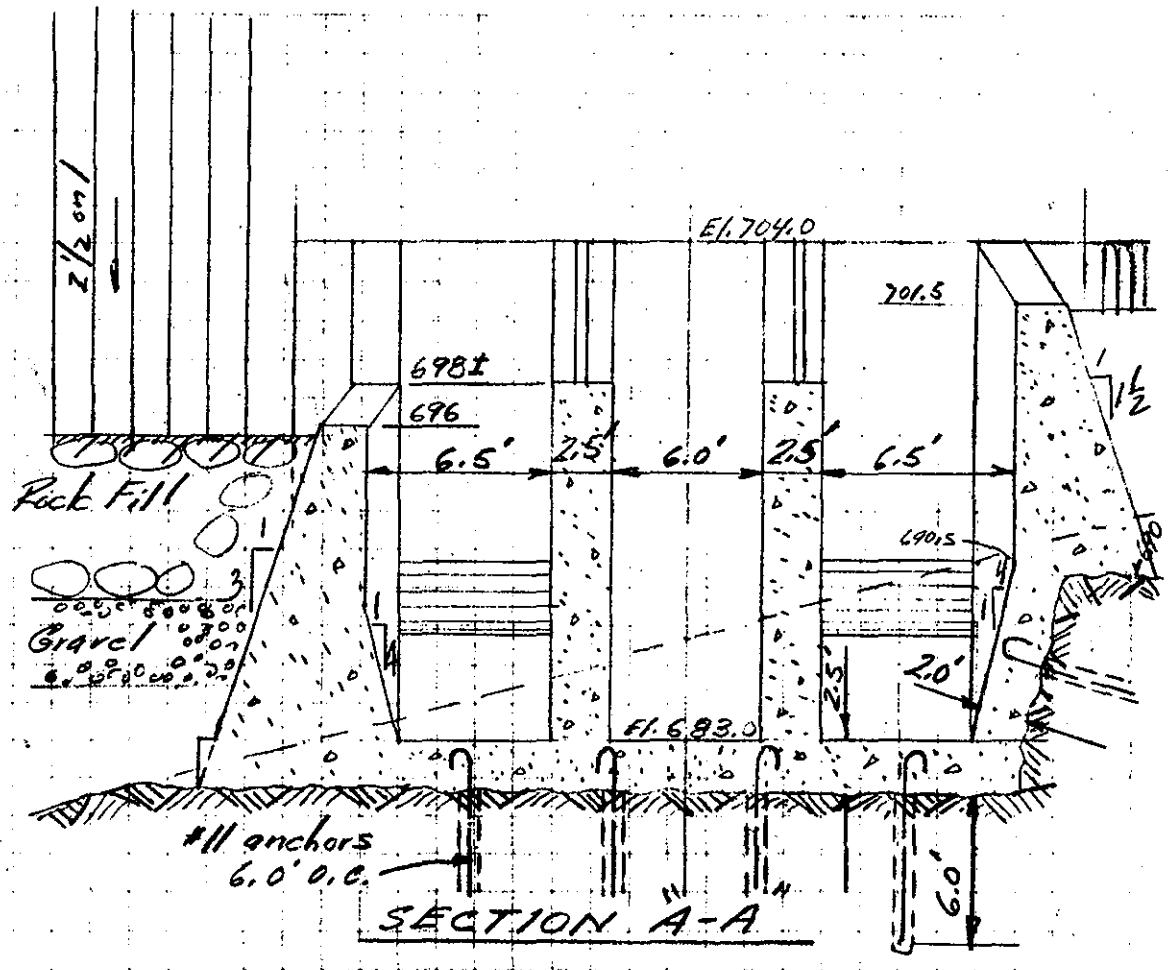


ELEV. OF EAST INTAKE WALL
Scale: 1/8" = 1'-0"



ELEV. OF WEST INTAKE WALL

SUBJECT OTTER BROOK DAM
COMPUTATION Retaining Walls @ Intake
COMPUTED BY JSM CHECKED BY RSM DATE Nov. 1955



SUBJECT: OTTER BROOK DAM
COMPUTATION: Retaining Walls @ Intake
COMPUTED BY: JSM CHECKED BY: RBW DATE: Nov. 1955

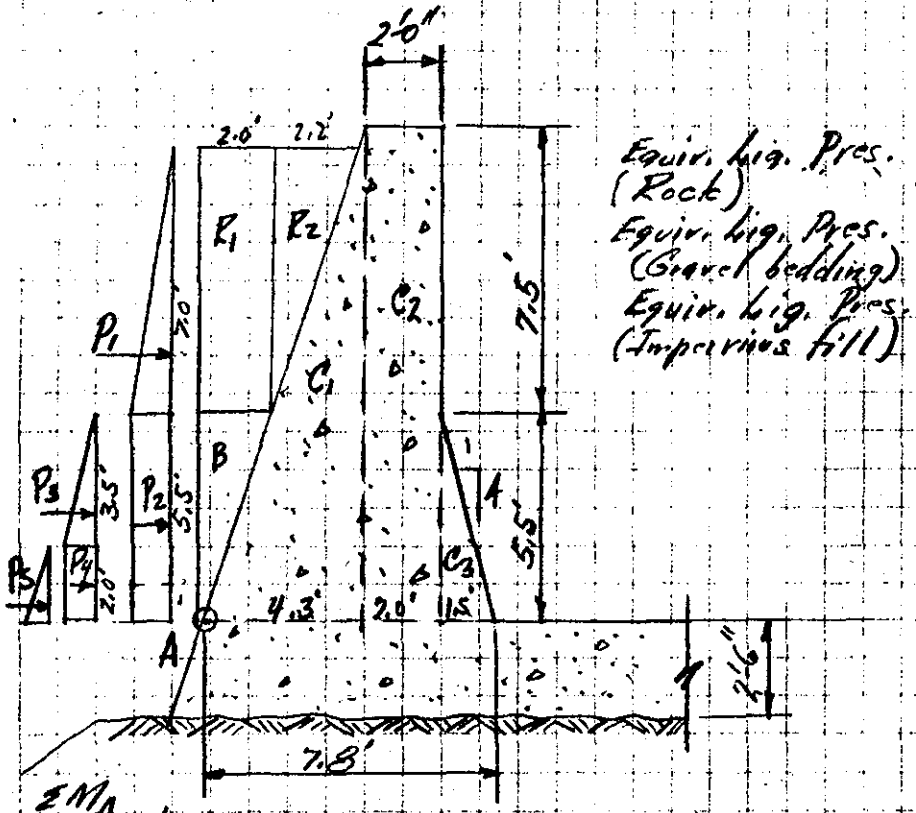
East Wall @ Sect. "A-A":

Case I:

Complete submergence - Full hydrostatic pressure (cancelled out) plus "at rest" pressure based on γ_m' (Uplift in concrete omitted - produces no change in position of resultant).

For rock & gravel bedding, $k_o = 0.45$

For impervious fill, $k_o = 0.70$



Equiv. Liq. Pres. (Rock) $= 0.45 \times 72 = 32.4$ #/□'

Equiv. Liq. Pres. (Gravel bedding) $= 0.45 \times 78 = 35$

Equiv. Liq. Pres. (Impervious fill) $= 0.70 \times 81 = 56.7$

ΣM_A

$C_1 = \frac{1}{2} \times 4.3 \times 13.0 \times 150$	=	4,200	×	2.87	=	12,050
$C_2 = 2.0 \times 13.0 \times 150$	=	3,900	×	5.30	=	20,700
$C_3 = \frac{1}{2} \times 1.5 \times 5.5 \times 150$	=	618	×	6.80	=	4,220
$R_1 = 2.0 \times 7.0 \times 72$	=	1,008	×	1.00	=	1,008
$R_2 = \frac{1}{2} \times 2.2 \times 7.0 \times 72$	=	555	×	2.73	=	1,515
$B = \frac{1}{2} \times 2.0 \times 5.5 \times 78$	=	429	×	0.67	=	288
$P_1 = \frac{1}{2} \times 32.4 \times 7.0^2$	=	795	×	7.83	=	6,230
$P_2 = 32.4 \times 7.0 \times 5.5$	=	1,250	×	2.75	=	3,430
$P_3 = \frac{1}{2} \times 35 \times 3.5^2$	=	214	×	3.17	=	678
$P_4 = 35 \times 3.5 \times 2.0$	=	245	×	1.0	=	245
$P_5 = \frac{1}{2} \times 56.7 \times 2.0^2$	=	113	×	0.67	=	76
						50,440

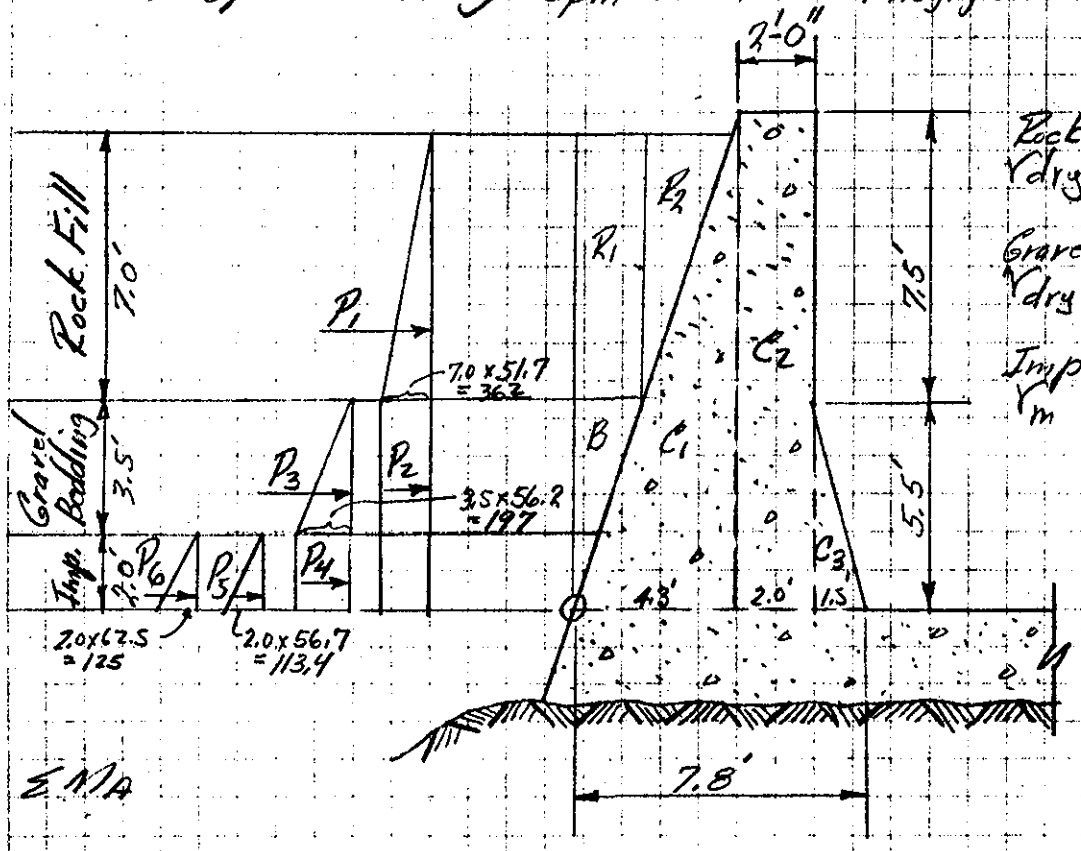
$$\frac{\Sigma M}{\Sigma V} = \frac{50,440}{10,710} = 4.7' \text{ 3rd Pt.} = 5.2'$$

SUBJECT: OTTER BROOK DAM
COMPUTATION: Retaining Walls @ Intake
COMPUTED BY: JSM CHECKED BY: RJM DATE: Nov. 1955

East Wall @ Sect. 'A-A' (Cont.)

Case II:

Rapid Drawdown - Rock and bedding "at rest" using γ_{dry} plus impervious fill "at rest" using γ_m plus hydrostatic pressure for water up to bottom of bedding. Uplift in concrete of negligible effect.



Rock γ_{dry} = 0.45 x 115 = 51.7

Gravel γ_{dry} = 0.45 x 125 = 56.2

Imp. γ_m = 0.70 x 81 = 56.7

ΣM_A

$C_1 - C_3$ =		=	8,718	x —	=	36,970
R_1	= 2.0 x 7.0 x 115	=	1,610	x 1.00	=	1,610
R_2	= 1/2 x 2.2 x 7.0 x 115	=	887	x 2.73	=	2,420
B	= 1/2 x 2.0 x 5.5 x 125	=	688	x 0.67	=	461
P_1	= 1/2 x 362 x 7.0	=	1,270	x 7.83	=	9,950
P_2	= 362 x 5.5	=	1,990	x 2.75	=	5,480
P_3	= 1/2 x 197 x 3.5	=	345	x 3.17	=	1,090
P_4	= 197 x 2.0	=	394	x 1.00	=	394
P_5	= 1/2 x 113.4 x 2.0	=	113	x 0.67	=	76
P_6	= 1/2 x 125 x 2.0	=	125	x 0.67	=	84
						58,535

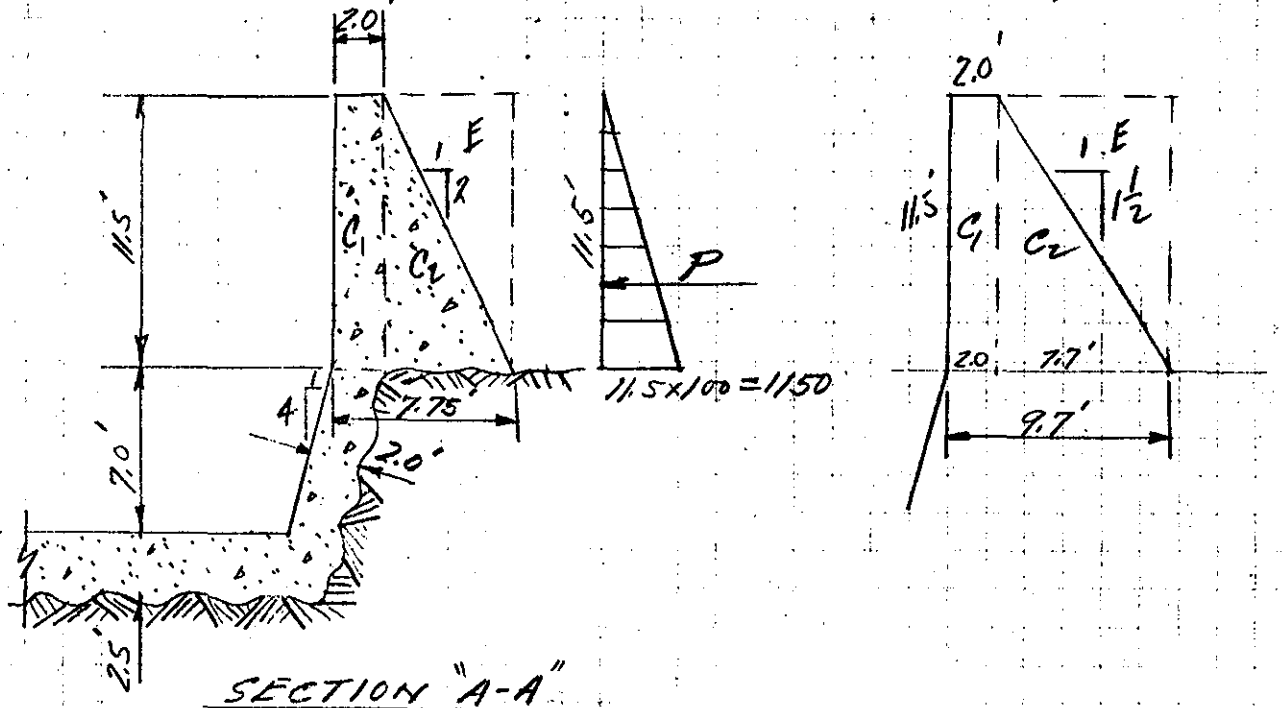
$\frac{\Sigma M}{\Sigma V} = \frac{58,535}{11,903} = \frac{4.9'}{5}$ 3rd Pt. = 5.2' ok

SUBJECT OTTER BROOK DAM
COMPUTATION Retaining Walls @ Intake
COMPUTED BY JSM CHECKED BY GBW DATE Dec. 1955

West Wall @ Sect. "A-A":

Design assumption: Three foot vertical column of gravel with a pipe drain. "At rest" pressure of saturated impervious but no hydrostatic pressure.

Equiv. liq. Press. = 0.70 × 143 = 100.1 say 100 #/□'



SECTION "A-A"

$C_1 = 2.0 \times 11.5 \times 150 = 3{,}450 \times 6.75 = 23{,}288$
$C_2 = \frac{1}{2} \times 5.75 \times 11.5 \times 150 = 4{,}950 \times 3.83 = 18{,}959$
$E = \frac{1}{2} \times 5.75 \times 11.5 \times 143 = 4{,}730 \times 1.92 = 9{,}082$
$\qquad 13{,}030$

$P = \frac{1}{2} \times 1150 \times 11.5 = 6{,}620 \times 3.83 = 25{,}355$
$\qquad 76{,}684$

$\frac{\Sigma M}{\Sigma V} = \frac{76{,}684}{13{,}030} = 5.9'$ 3rd Pt. = 5.2' Try 1½ on 1 slope

$C_1 = 2.0 \times 11.5 \times 150 = 3{,}450 \times 8.7 = 30{,}015$
$C_2 = \frac{1}{2} \times 7.7 \times 11.5 \times 150 = 6{,}630 \times 5.2 = 34{,}476$
$E = \frac{1}{2} \times 7.7 \times 11.5 \times 143 = 6{,}340 \times 2.57 = 16{,}294$
$\qquad 16{,}420$
$P = \frac{1}{2} \times 1150 \times 11.5 = 6{,}620 \times 3.83 = 25{,}355$
$\qquad 106{,}140$

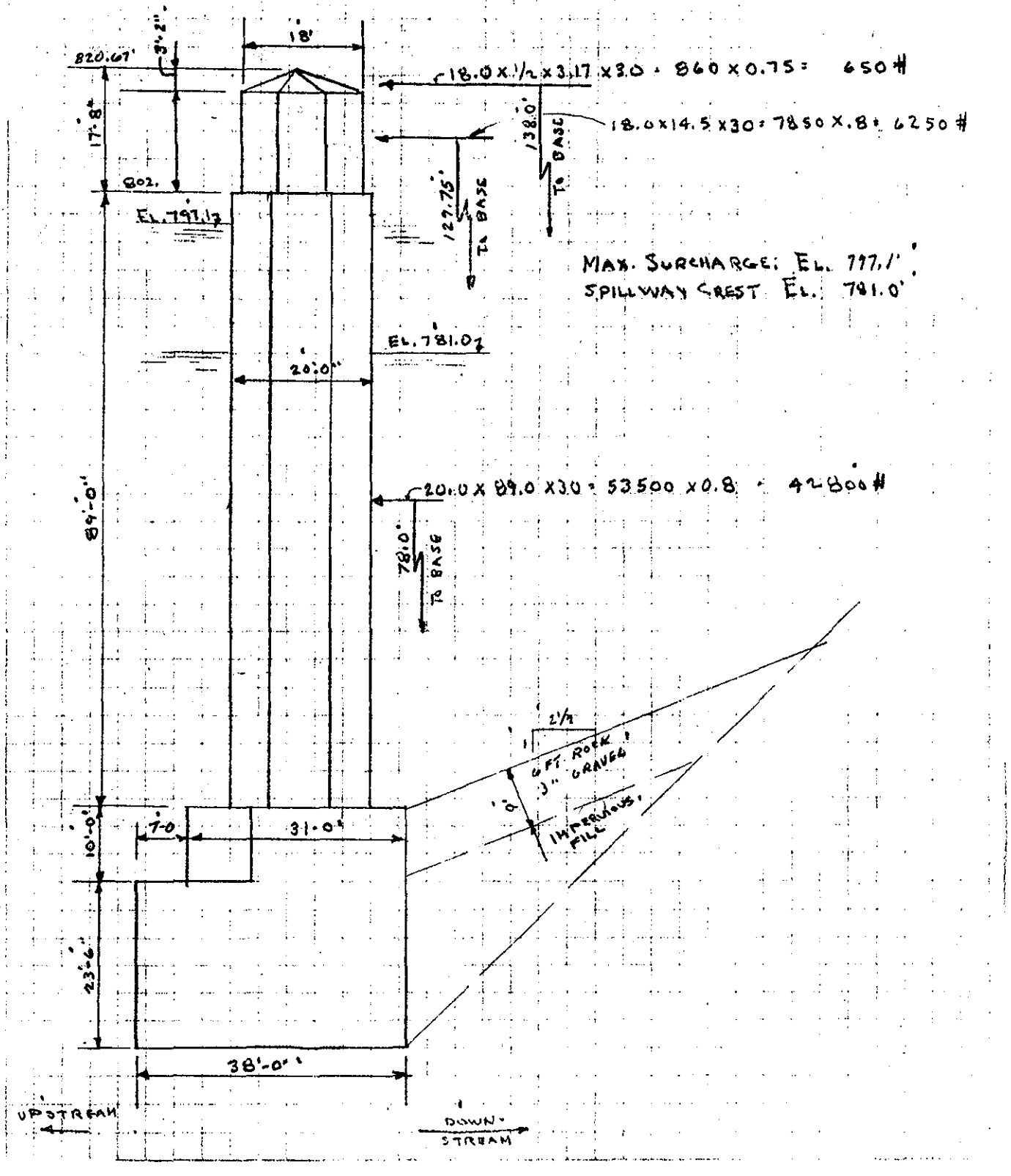
$\Sigma M / \Sigma V = 106{,}140 / 16{,}420 = 6.5'$ 3rd Pt. = 6.5' ok use

NED FORM 223
27 Sept 49

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

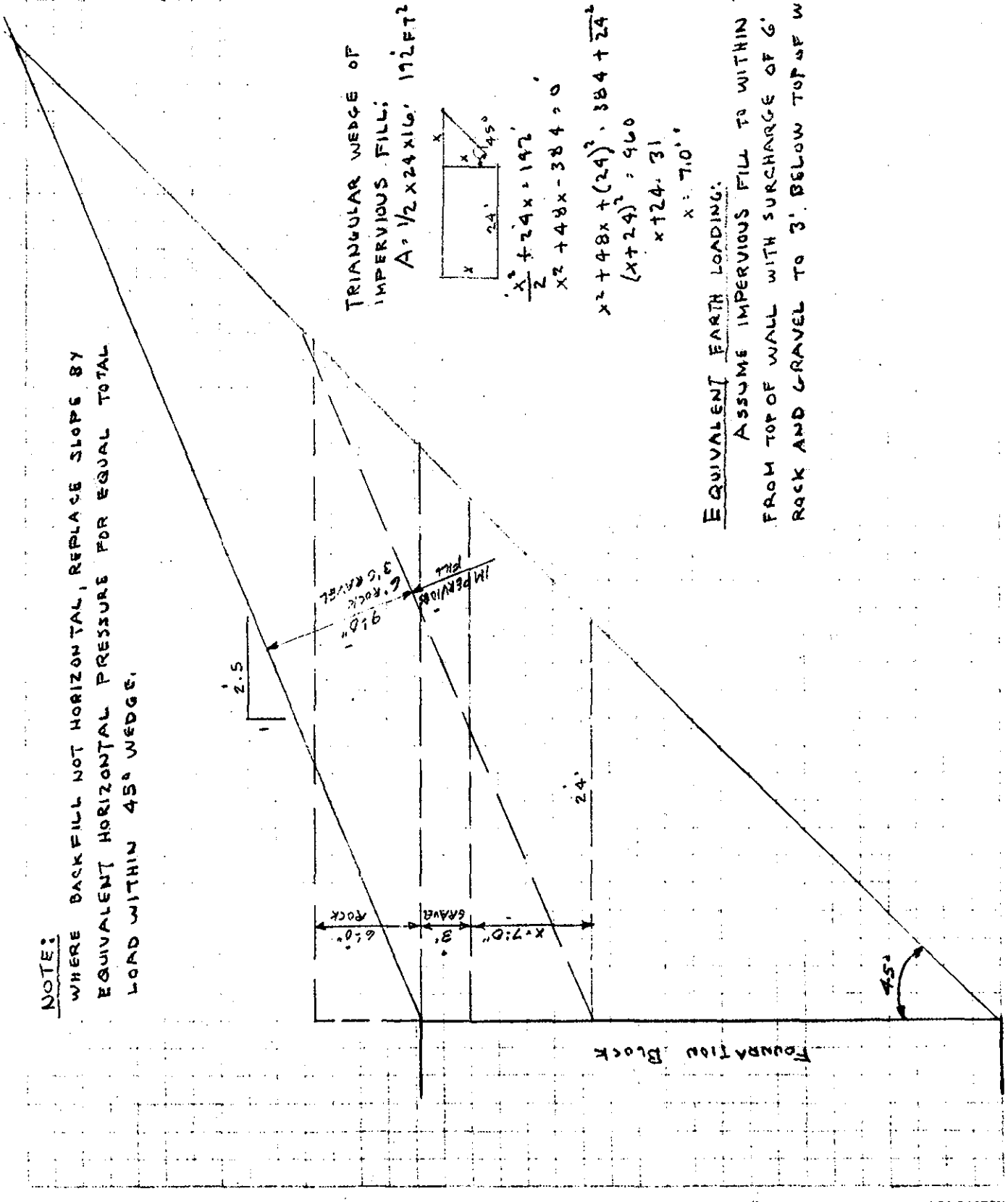
SUBJECT: OTTER BROOK DAM
COMPUTATION: INTAKE TOWER - STABILITY
COMPUTED BY: FREDERICK CHECKED BY: EPR DATE: 11-29-55

CASE I. NO WATER IN RESERVOIR (RAPID DRAWDOWN). WIND IN UPSTREAM DIRECTION WITH EARTH AGAINST DOWNSTREAM FACE OF STRUCTURE.



SUBJECT: OTTER BROOK DAM
COMPUTATION: INTAKE TOWER - STABILITY
COMPUTED BY: FREDERICK CHECKED BY: EPR DATE: 11-29-55

CASE I.

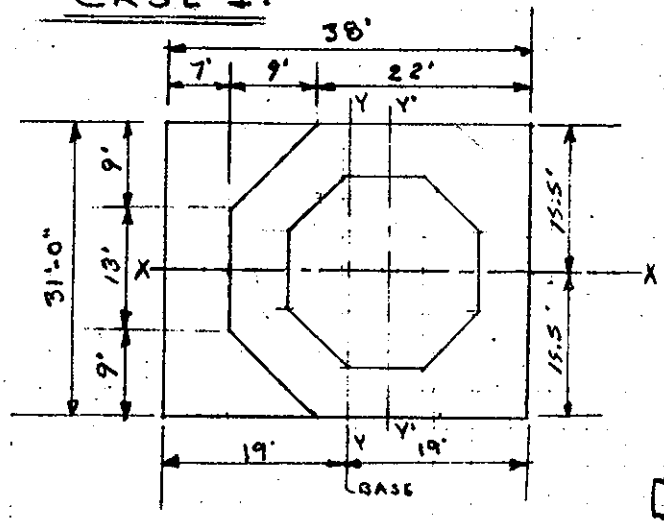


NED FORM 223
27 Sept 49

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

SUBJECT: OTTER BROOK DAM
COMPUTATION: INTAKE TOWER - STABILITY
COMPUTED BY: FREDERICK CHECKED BY: EDR DATE: 11-29-55

CASE I.



C.G. TOP SECT. BASE BLOCK: (LEFT END)

	A	x	Ax
22 x 31 =	682	x 27 =	18400
9 x 13 =	117	x 11.5 =	1346
2 x $\overline{9}^2$ x 1/2	81	x 13 =	1050
	880		20796

C.G. = 20796/880 = 23.6' (Y'-Y' AXIS)

[NEGLECT BRIDGE LOAD - WORST CONDITION]

DEAD LOAD SUPERSTRUCTURE:

ROOF	$\overline{18}^2 \times \pi/4 \times 1 \times 150$	=	38200 #
WALLS	1.0 x 14.5 x 150 x 8 x 7	=	122000 #
EL. 802 FLOOR	$\overline{16}^2 \times \pi/4 \times 300$	=	60000
OPER.	$\overline{16}^2 \times \pi/4 \times 150$	=	30000
HEATER	" " "	=	30000
LANDINGS	$[(\overline{16}^2 \times \pi/4) - (9 \times 14.5)] 150 \times 4$	=	42600
WALLS	$(\overline{20}^2 - \overline{16}^2 \times \pi/4 - \overline{6}^2 \times 2) 150 \times 89$	=	1,710000.
STAIRS	$(12 \times 2.5 \times 1) \times 150 \times 8$	=	36000.
	$(10 \times 3 \times 1)(150 \times 2)$	=	9000.
	TOTAL	=	2,077,800. #

DEAD LOAD TOP SECTION FOUNDATION BL'K

EL. 712.63 FL'R.	$\overline{16}^2 \times \pi/4 \times 1.5 \times 150$	=	45300. #
WALLS TO TOP OF FDN. BL'K	$(\overline{31}^2 - \overline{25}^2 \times \pi/4 - \overline{9}^2)\ 150 \times 10$	=	585000
HAUNCHES	$\overline{4.5}^2 \times 1/2 \times 22.0 \times 150 \times \pi$	=	105000
STAIRS	22 x 2.5 x 1 x 150	=	8250
SLAB ABOVE HAUNCHES	3 x 4.5 x 20.5 x π x 150	=	131000
	TOTAL		874.550

DEAD LOAD BOTTOM SECT. FOUNDATION BLOCK

TOTAL 38 x 31 x 23.5 = 27700 FT³
LESS: $\overline{25}^2 - \pi/4 \times 12$ = 5900
CONDUITS 3(2.5 x 4.5 x 22) = 745
3(4.25 x 4.25 x 17) = 1030
7675 FT²

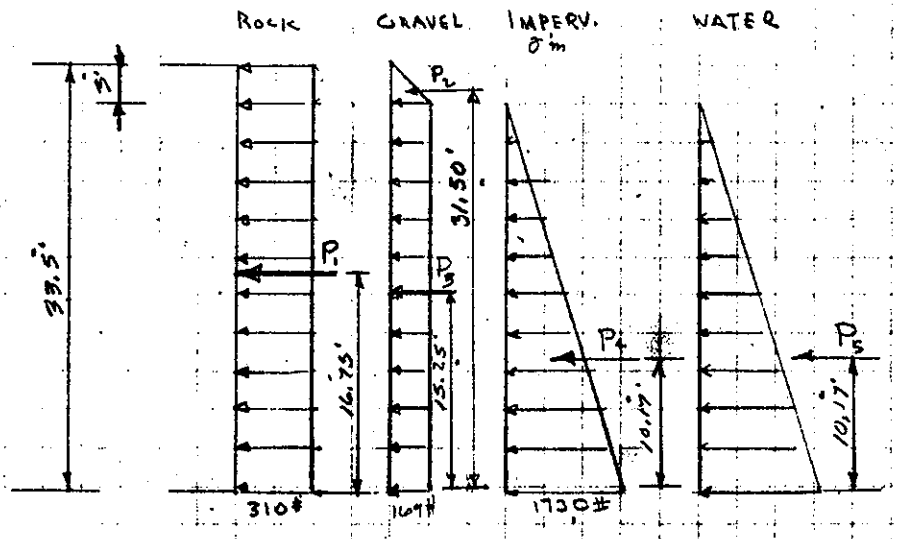
(27700 - 7675)(150) = 20080 x 150 = 3,010,000 #

SUBJECT OTTER BROOK DAM
COMPUTATION INTAKE TOWER - STABILITY
COMPUTED BY FREDERICK CHECKED BY EPR DATE 11-29-55

CASE I.

EARTH LATERAL LOADS (RAPID DRAWDOWN IN RESERVOIR):

ROCK & BEDDING AT REST USING GAMMA DRY PLUS IMPERVIOUS FILL AT REST USING GAMMA'm, PLUS HYDROSTATIC PRESSURE FOR WATER UP TO BOTTOM OF BEDDING.



ROCK DRY 115 x 6 = 690# x .45 = 310#
P_1 = 310 x 33.5 x 31 = 322000 #

GRAVEL 125 x 3 = 375 x .45 = 169#
P_2 = 3 x 169 x 1/2 x 31 = 7850 #
P_3 = 30.5 x 169 x 31 = 160,000 #

IMPERVIOUS γ'm
30.5 x 81 = 2470# x .70 = 1730 #
P_4 = 30.5 x 1730 x 1/2 x 31 = 817000 #

WATER 30.5 x 62.4 = 1900 #
P_5 = 1900 x 30.5 x 1/2 x 31 = 898000 #

NED FORM 223
27 Sept 49

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

SUBJECT: OTTER BROOK DAM
COMPUTATION: INTAKE TOWER - STABILITY
COMPUTED BY: FREDERICK CHECKED BY: EPR DATE: 11-29-55

CASE I.

FORCES ACTING		↓ K	↑ K	→ K	← K	ARM FT	MOMENTS ↶ K'	MOMENTS ↷ K'
DL	SUPERSTRUCTURE	2079.				22.50'		46800.
"	TOP SECT. FDN BL'K.	875.				23.60'		20650.
"	BOT. " " "	3010.				19.00'		57200.
WIND	ROOF				0.65	138.0'	89.80'	
"	TOP SECT.				6.25	129.75	810.00'	
"	TOWER				42.80	78.00'	3340.00'	
P_1					327.00'	16.75'	5400.00'	
P_2					7.85	31.50'	247.00'	
P_3					160.00	15.25'	2440.	
P_4					817.0'	10.17'	8310.	
P_5					898.0'	10.17'	9140.	
TOTAL		5964.			2256.		29777.	124650
NET		5964 ↓		2256. ←			94873 ↺	

LOCATION OF RESULTANT: 94873'/5964 = 15.85' (MIDDLE 1/3 = 12.67')

SLIDING: 2256/5964 = .384 < .67 OK

e = 19.0 - 15.85 = 3.15'

$I_{Y-Y\,(BASE)}$ = 31 x 38³/12 = 142000 FT⁴ A = 38 X 31 = 1178 FT²

BEARING @ UPSTREAM SIDE = $\frac{5964}{1178} + \frac{5964 \times 3.15 \times 19}{142000}$ = 5.05 + 2.52 = 7.57 K/FT

BEARING @ DOWNSTREAM SIDE = 5.05 - 2.52 = 2.53 K/FT²

SLIDING FACTOR OF SAFETY:

.67 X 5964 = 4000K

$\frac{4000}{2256}$ = 1.77

FACTOR OF SAFETY = 1.77

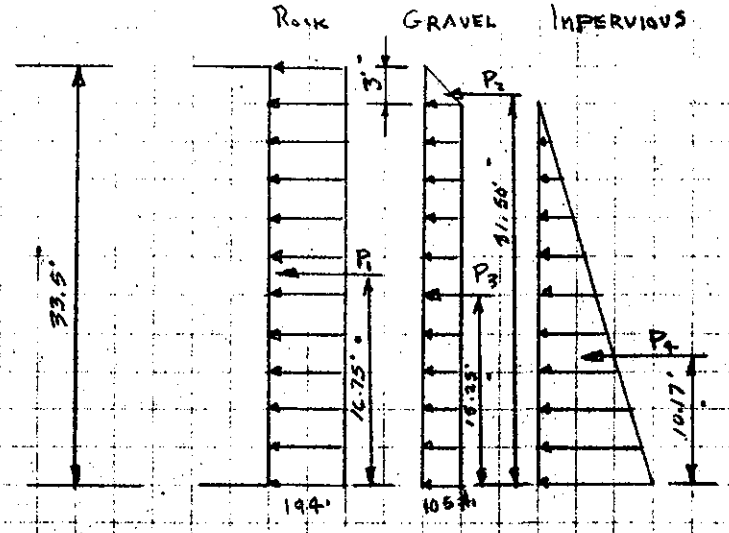
SUBJECT: OTTER BROOK DAM
COMPUTATION: INTAKE TOWER - STABILITY
COMPUTED BY: FREDERICK CHECKED BY: EPR DATE: 11-29-55

CASE II. RESERVOIR TO SPILLWAY CREST. ALL GATES OPEN.

UPLIFT 101.5 x .0625 x (38 x 31) = 7460.0 KIPS

EARTH - LATERAL LOADS

COMPLETE SUBMERGENCE. FULL HYDROSTATIC PRESSURE CANCELLED OUT. USE AT REST PRESSURES BASED ON GAMMA'm.



ROCK: 72 x 6 = 432# x .45 = 194#
P_1 = 194 x 33.5 x 31 = 201500 #'

GRAVEL: 78 x 3 = 234# x .45 = 105#
P_2 = 3 x 105 x 1/2 x 31 = 4900 #'
P_3 = 30.5 x 105 x 31 = 99400 #'

IMPERVIOUS: 30.50 x 81 = 2470 x .70 = 1734#
P_4 = 30.5 x 1734 x 1/2 x 31 = 817000 #'

BRIDGE LOAD
2 x 46K = 92.0K'

WATER LOAD
W_1 = 31 x 7 x 78 x 62.5 = 1,048,000 #

OCTAGONAL AREAS:
A = 1/4 x 8 x $\overline{13}^2$ CTH 180°/8 = 338 x 2.41 = 815 FT²
A = 1/4 x 8 x 8.33 CTH 180/8 = 138.8 x 2.41 = 334
481 FT²

W_3 = 481 x 68 x 62.5 = 1,940,000 #
W_4 = $\overline{9}^2$ x 68 x 62.5 = 344000 #'
W_2 = $\overline{9}^2$ x 78 x 62.5 = 396000 #

SUBJECT OTTER BROOK DAM
COMPUTATION INTAKE TOWER - STABILITY
COMPUTED BY FREDERICK CHECKED BY EPR DATE 11-29-55

CASE II.

FORCES ACTING		↓ K	↑ K	→ K	← K	ARM	MOMENT ↺ K'	MOMENT ↻ K'
DL	SUPERSTRUCTURE	2079.				22.50		468.00.
"	TOP SECT. FOU. BL'K	875.				23.60		20650.
"	BOT. " " "	3010.				19.00		57200.
W_1		1048.				3.50		3670.
W_2		396.				10.00		3960.
W_3		1.940.				22.50		43700.
W_4		344.				35.00		12020.
CONDUIT	1775 x 62.5	111.				17.75		1975.
P_1					201.5	16.75	3370.	
P_2					4.9	31.50	154.	
P_3					99.4	15.25	1510.	
P_4					817.0	10.17	8300.	
BRIDGE		92.				33.50		3080.
UPLIFT			7460.			19.00	141800.	
TOTAL		9895.	7460.		1123.		155134.	193055.
NET		2435 ↓		← 1123.			37921 ↻	

LOCATION OF RESULTANT: 37921/2435 = 15.60' WITHIN MID 1/3 - OK

SLIDING: 1123/2435 = .46 < .67 OK

e = 19.0 - 15.60 = 3.40'

BEARING @ UPSTREAM SIDE = $\frac{2435}{1178} + \frac{2435 \times 3.40 \times 19}{142000}$ = 2.06 + 1.11 = 3.17 K/FT²

BEARING @ DOWNSTREAM SIDE = " - " = 2.06 - 1.11 = 0.95 K/FT²

SLIDING FACTOR OF SAFETY:

.67 x 2435 = 1630 K

$\frac{1630}{1123}$ = 1.45

SLIDING FACTOR OF SAFETY = 1.45

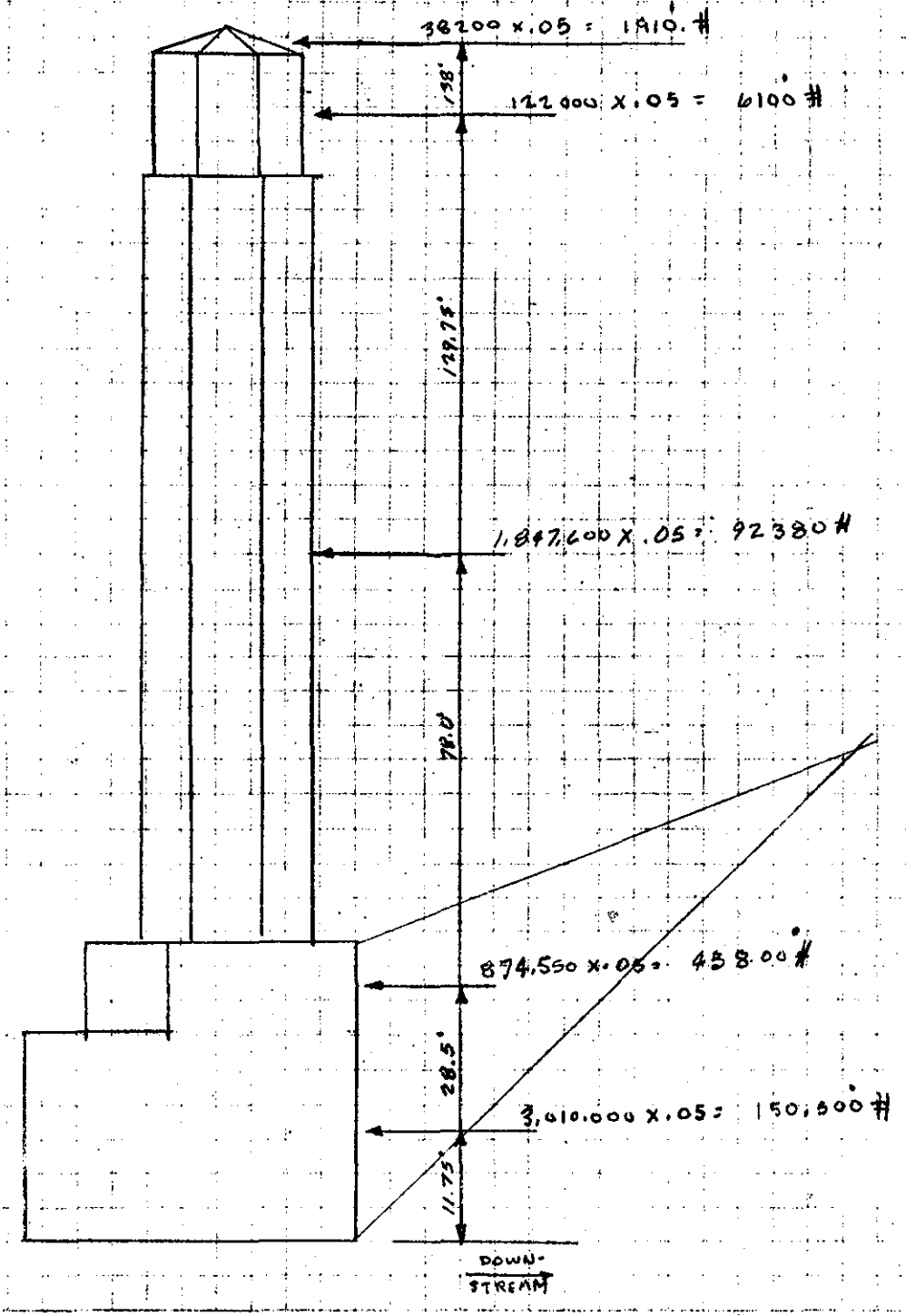
NED FORM 223
27 Sept 49

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

SUBJECT: OTTER BROOK DAM
COMPUTATION: INTAKE TOWER - STABILITY
COMPUTED BY: FREDERICK CHECKED BY: EPR DATE: 11-30-55

CASE IIa SAME AS CASE II PLUS EARTHQUAKE FORCES ACTING ON DOWNSTREAM SIDE.

$P_e = \alpha W$ $\alpha = .05$



SUBJECT: OTTER BROOK DAM
COMPUTATION: INTAKE TOWER – STABILITY
COMPUTED BY: FREDERICK CHECKED BY: EPR DATE: 11-30-55

CASE II a.

MOMENTS (EARTHQUAKE) ↶ K'

1.91	X 138	=	264.00 K'
6.1	X 129.75	=	789.
92.38	X 78.0	=	7170.
43.80	X 28.5	=	1250.
150.50	X 11.75	=	1770.
294.69			11243.0 K'

37921
− 11243
$\Sigma M = 26678$ K'

1123
295
$\Sigma H = 1418$ K

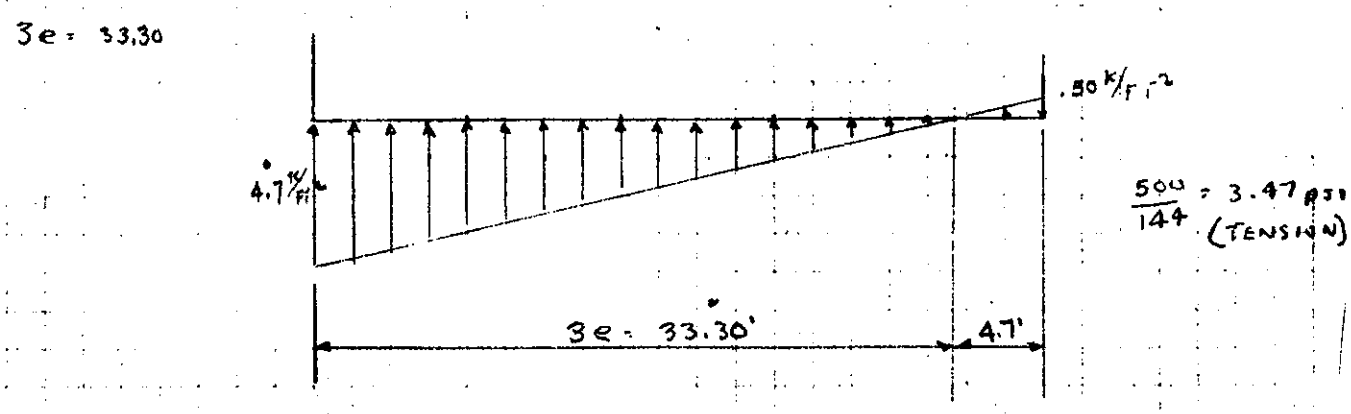
LOCATION OF RESULTANT: 26678/2435 = 11.10 FT OK, WITHIN 50% OF BASE WIDTH.

SLIDING: 1418/2435 = .58 OK

$e = 19.0 - 11.10 = 7.90'$

BEARING @ UP-STREAM SIDE = $\frac{2435}{1178} + \frac{2435 \times 7.9 \times 19}{142000} = 2.06 + 2.58 = 4.64$ K/FT²

BEARING @ DOWN STREAM SIDE = " − " = 2.06 − 2.58 = −.48 K/FT²



SLIDING FACTOR OF SAFETY:
.67 X 2435 = 1630
1630/1418 = 1.15
SLIDING FACTOR OF SAFETY = 1.15

SUBJECT OTTER BROOK DAM
COMPUTATION INTAKE TOWER - STABILITY
COMPUTED BY FREDERICK CHECKED BY EPR DATE 11-30-55

CASE Ia. SAME AS CASE I PLUS EARTHQUAKE FORCES ACTING ON DOWNSTREAM SIDE.

	LOAD	MOM
EARTHQUAKE	← 295	11243 K' ↶
H =	2256	
ΣH	→ 2551 K'	
ΣV = 5862		948787
		ΣM = 836307 K'

LOCATION OF RESULTANT: 83630/5964 = 14.0' (WITHIN MID 1/2 - OK)

SLIDING: 2551/5964 = .43 < .67 OK

e = 19.00 - 14.0 = 5.0'

BEARING @ UPSTREAM SIDE = 5964/1178 + 5964 x 5.0 x 19 / 142000 = 5.05 + 4.0 = 9.05 K/FT²

BEARING @ DOWNSTREAM SIDE " — " = 5.05 - 4.0 = 1.05 K/FT²

SLIDING FACTOR OF SAFETY:

.67 x 5964 = 4000

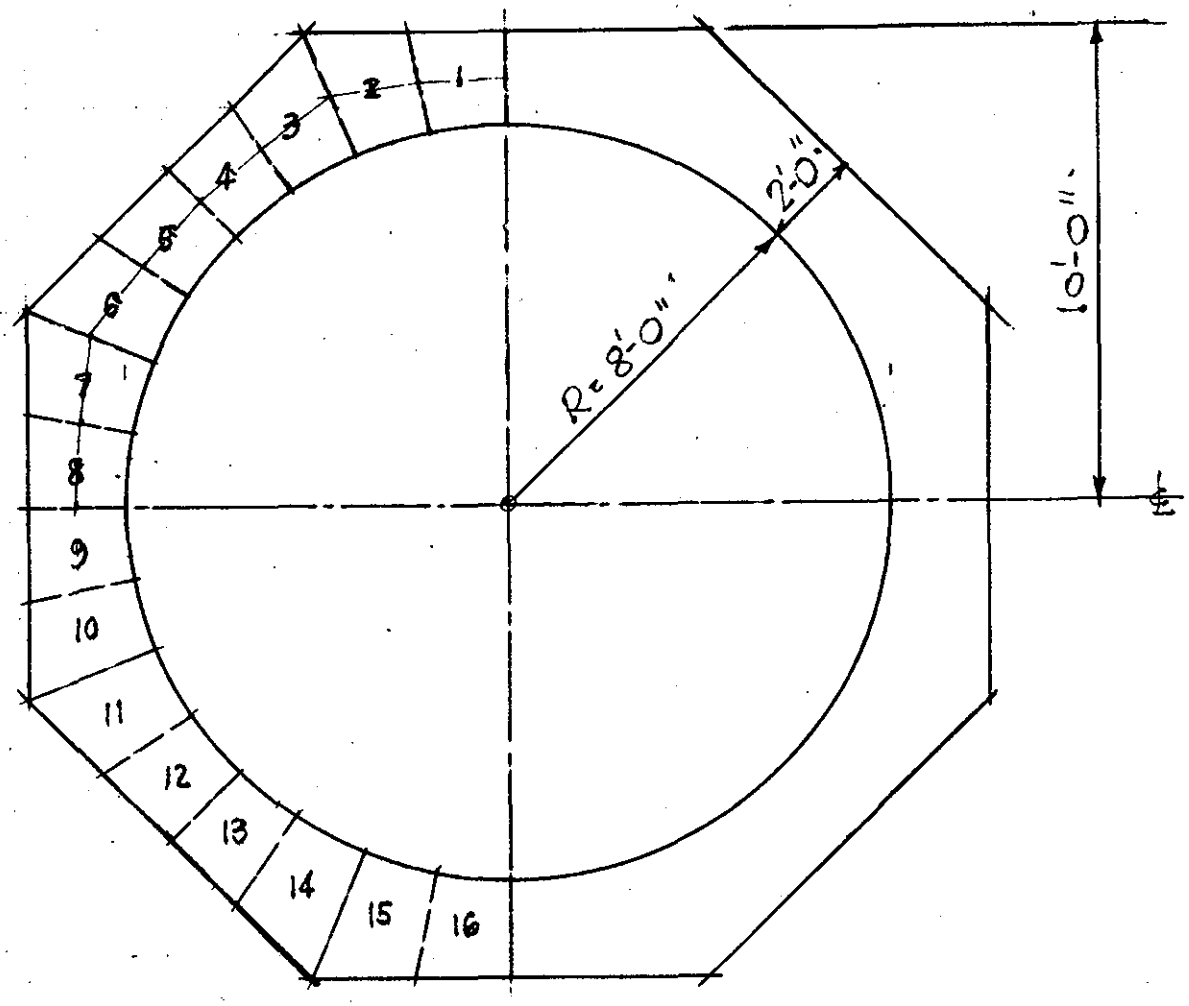
4000/2551 = 1.57

SLIDING FACTOR OF SAFETY = 1.57

NED FORM 223
27 Sept 49

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

SUBJECT Otter Brook Dam - Keene, N.H.
COMPUTATION Intake Tower - Section at El. 715.0
COMPUTED BY EPR CHECKED BY HEN DATE 7-27-55



Horizontal Section thru Wall above El. 714.6

Max. surcharge El. 797.8

Bottom section El. 714.6'

82.4 ft. max. head @ 62.5#/cu.ft. 5150#/s.f.

Spillway crest El. ~81.0'
El. 714.6
66.4' × 62.5# = 4150#/s.f.

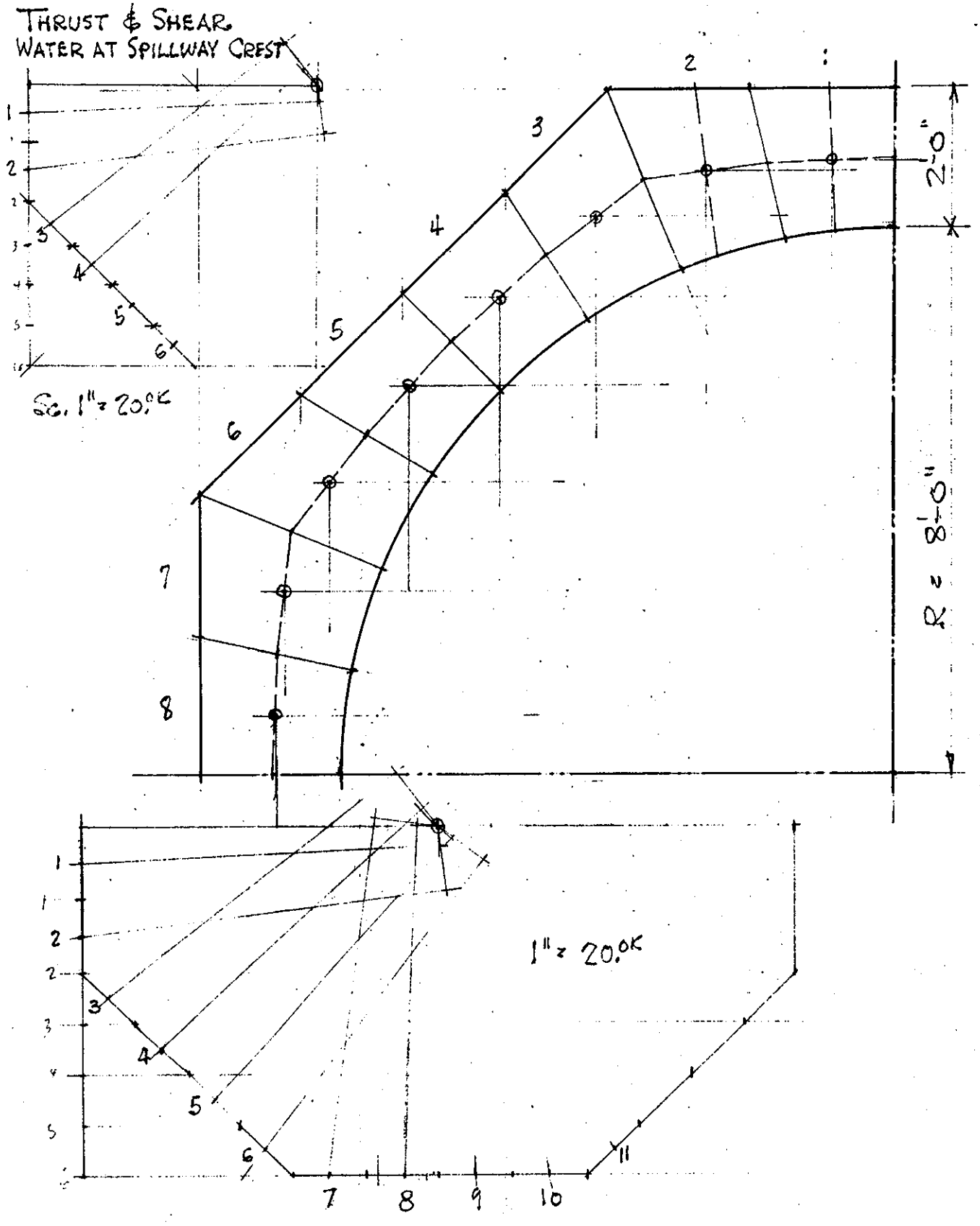
$\frac{5150}{4150} = 1.24$ The stresses with max. surcharge will not exceed the allowable 33% increase

ARMY-NED-BOSTON

NED FORM 223
27 Sept 49

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

SUBJECT Otter Brook Dam - Keene, N.H.
COMPUTATION Intake Tower - Section at El. 715.0
COMPUTED BY EPR CHECKED BY JSM. DATE 10-18-55



NED FORM 223
27 Sept 49

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

SUBJECT Otter Brook Dam
COMPUTATION Intake Tower Walls - Section at El. 715.0
COMPUTED BY EPR CHECKED BY DATE

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Water at Spillway Crest

	GIVEN				DERIVED			Vert. Lds.		Horz. Lds.		Co-ord. Diff.	
1	2	3	4	5	6	7	8	9	10	11	12	13	14
No	d_s	d	x	y	$a = \frac{d_s}{d^3}$	ay	ay^2	V ⊗	ΣV	h ⊗	Σh	$x_n-(x_{n-1})$	$y_n-(y_{n-1})$
1	1.85	2.08	- 0.92	+ 8.92	0.206	+1.835	+16.350	+ 8.60	+ 8.60	0	0	- 1.79	- 0.13
2	1.79	2.45	- 2.71	+ 8.79	0.122	+1.072	+ 9.410	+ 8.60	+17.20	0	0	- 1.58	- 0.65
3	1.79	2.45	- 4.29	+ 8.14	0.122	+0.993	+ 8.100	+ 6.05	+23.25	+ 6.05	+ 6.05	- 1.40	- 1.14
4	1.85	2.08	- 5.69	+ 7.00	0.206	+1.442	+10.100	+ 6.05	+29.30	+ 6.05	+ 12.10	- 1.31	- 1.31
5	1.85	2.08	- 7.00	+ 5.69	0.206	+1.172	+ 6.670	+ 6.05	+35.35	+ 6.05	+ 18.15	- 1.14	- 1.40
6	1.79	2.45	- 8.14	+ 4.29	0.122	+0.524	+ 2.250	+ 6.05	+41.40	+ 6.05	+ 24.20	-0.65	- 1.58
7	1.79	2.45	- 8.79	+ 2.71	0.122	+0.330	+ 0.894	0	+41.40	+ 8.60	+ 32.80	- 0.13	- 1.79
8	1.85	2.08	- 8.92	+ 0.92	0.206	+0.190	+ 0.177	0	+41.40	+ 8.60	+ 41.40	0	- 1.84
9			- 8.92	- 0.92	0.206	-0.190	+ 0.177	0	+41.40	+ 8.60	+ 50.00	+ 0.13	- 1.79
10			- 8.79	- 2.71	0.122	- 0.330	+ 0.894	0	+41.40	+ 8.60	+ 58.60	+ 0.65	- 1.58
11			- 8.14	- 4.29	0.122	- 0.524	+ 2.250	- 6.05	+35.35	+ 6.05	+ 64.65	+ 1.14	- 1.40
12			- 7.00	- 5.69	0.206	- 1.172	+ 6.670	- 6.05	+ 29.30	+ 6.05	+ 70.70	+ 1.31	- 1.31
13			- 5.69	- 7.00	0.206	- 1.442	+10.100	- 6.05	+ 23.25	+ 6.05	+ 76.75	+ 1.40	- 1.14
14			- 4.29	- 8.14	0.122	- 0.993	+ 8.100	- 6.05	+ 17.20	+ 6.05	+ 82.80	+ 1.58	- 0.65
15			- 2.71	- 8.79	0.122	- 1.072	+ 9.410	- 8.60	+ 8.60	0	+ 82.80	+ 1.79	- 0.13
16			- 0.92	- 8.92	0.206	- 1.835	+ 16.350	- 8.60	0	0	+ 82.80	-	-
					2.624		+107.902						

ARMY-NED-BOSTON

NED FORM 223
27 Sept 49

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

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SUBJECT Otter Brook Dam
COMPUTATION Intake Tower Walls - Section at El. 1150
COMPUTED BY _____ CHECKED BY _____ DATE _____

Cantilever Moments

	15	16	17	18	19	20	21	22	23	24
	Vert.	Horz.	m 15+16	ma	may	m+M	H·y	Mom. 20+21	Thrust	Shear
1	- 2.13	0	- 2.13	- 0.438	- 3.907	+ 373.937	- 369.484	+ 4.453	42.°	2.5
2	- 17.52	0	- 17.52	- 2.137	- 18.784	+ 358.547	- 364.099	- 5.552	43.°	7.°
3	- 44.70	0	- 44.70	- 5.453	- 44.387	+ 331.367	- 337.175	- 5.808	43.°	7.°
4	- 77.25	- 6.897	- 84.15	- 17.335	- 121.345	+ 291.917	- 289.954	+ 1.963	42.°	2.5
5	- 115.73	- 22.748	- 138.48	- 28.527	- 162.319	+ 237.587	- 235.691	+ 1.896	42.°	2.5
6	- 156.13	- 48.158	- 204.29	- 24.923	- 106.920	+ 171.777	- 177.700	- 5.923	43.°	7.°
7	- 183.04	- 86.394	- 269.43	- 32.870	- 89.078	+ 106.637	- 112.254	- 5.617	43.°	7.°
8	- 188.42	- 145.106	- 333.53	- 68.707	- 63.210	+ 42.537	- 38.119	+ 4.418	42.°	2.5
9	- 188.42	- 221.282	- 409.70	- 84.398	+ 77.546	- 33.633	+ 38.119	+ 4.486	42.°	2.5
10	- 183.04	- 310.782	- 493.82	- 60.246	+ 163.267	- 117.753	+ 112.254	- 5.499	43.°	7.°
11	- 156.13	- 403.370	- 559.50	- 68.259	+ 292.831	- 183.433	+ 177.700	- 5.733	43.°	7.°
12	- 115.73	- 493.88	- 609.61	- 125.579	+ 714.545	- 233.543	+ 235.691	+ 2.148	42.°	2.5
13	- 77.25	- 586.497	- 663.75	- 136.733	+ 957.131	- 287.683	+ 289.954	+ 2.271	42.°	2.5
14	- 44.70	- 673.992	- 718.69	- 87.680	+ 713.715	- 342.623	+ 337.175	- 5.448	43.°	7.°
15	- 17.52	- 727.812	- 745.33	- 90.930	+ 799.275	- 369.263	+ 364.099	- 5.164	43.°	7.°
16	- 2.13	- 738.576	- 740.71	- 152.586	+ 1361.067	- 364.643	+ 369.484	+ 4.841	42.°	2.5
				- 986.801	- 609.950					
					+ 5079.477					
					+ 4469.527					

8.0

$$M = -\frac{\Sigma ma}{\Sigma a} = -\frac{-986.801}{2.624} = +\underline{376.067}$$

$$H = -\frac{\Sigma may}{\Sigma ay^2} = -\frac{+4469.527}{107.902} = -\underline{41.422}$$

ARMY-NED-BOSTON

SUBJECT: Otter Brook Dam
COMPUTATION: Intake Tower Walls - Section at El. 715.0
COMPUTED BY: EPR CHECKED BY: JSM DATE:

③ Max. M = 5800'# N = 43000# Shear = 7000#

b = 12" t = 2.45×12 = 29.5" d = 26" $K = \frac{5800}{(26)^2} = 8.6$

$M_s = 5800 + \left(\frac{11.25}{12} \times 43000\right) = 5800 + 41300 = 47100'\#$

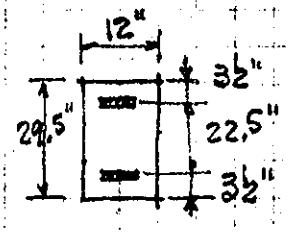
$K = 47100/(26)^2 = 70$

$A_s = \frac{12 \times 47100}{18000 \times j \times 26} - \frac{43000}{18000} = 1.38 - 2.39$

Use min. $A_s = 0.0025\,bd = 0.0025 \times 12 \times 26 = 0.78\,\square''$ (.39)

Use #6 @ 12" Horz. e.f.

⑨ M = 4486'# N = 42000# b = 12" t = 25" d = 21.5"



#6 @ 12" e.f. $A_s = 0.44\,\square''$ n = 10

$A = (12 \times 29.5) + (2 \times 0.44 \times 10) = 354 + 8.8 = 362.8\,\square''$

$\frac{12x^2}{2} + (9 \times 0.44)(x - 3.5) = (10 \times 0.44)(26 - x)$ (3.96)

$6x^2 + 3.96x - 13.9 = 114 - 4.4x$

$6x^2 + 8.36x - 127.9 = 0$

$x^2 + 1.39x - 21.3 = 0$

$x = \frac{-1.39 \pm \sqrt{1.93 + 85}}{2} = \frac{-1.39 \pm 9.3}{2} = \frac{7.9}{2} = 3.95"$

$I_c = 12 \times \overline{3.95}^3 \times \frac{1}{3} = 248$

$I_s = 3.96 \times (0.45)^2 = -$

$I_s = 4.4 \times (22.05)^2 = 2140$

$2388\ in^4$

$f_c = \frac{43000}{363} + \frac{12 \times 5800 \times 3.95}{2388} = 119 + 115 = 234\ \#/\square''$

$f_s = 10\left(119 - \frac{12 \times 5800 \times 22}{2388}\right) = 10(119 - 639) = 5200\ \#/\square''$

Use #6 @ 12" Horz. e.f.

SUBJECT Otter Brook Dam.
COMPUTATION Conduit Section at ℄ of Dam
COMPUTED BY H.E.W. CHECKED BY DATE 11/4/55

Determination of max loading

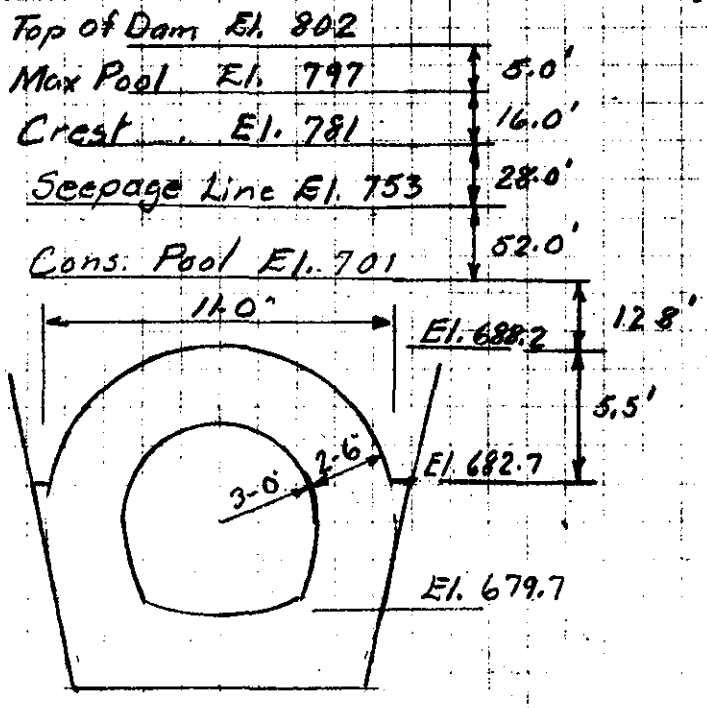


Table of Unit Weights (From N.E.D soils Lab.)

Material	Unit Weight, pcf. γ			
	Sat.	Moist	Dry	Submerged $\gamma'm$
Rock Fill	135	115	115	72
Gravel Bedding + Pervious Fill	141	130	125	78
Impervious Fill	143	140	126	81
Foundation Fill	143	140	126	81
Foundation Sand	128	115	106	66

SUBJECT Otter Brook Dam
COMPUTATION Conduit Section at ℄ of Dam
COMPUTED BY H.E.W. CHECKED BY DATE 11/5/55

Determination of max loading (cont.)

Case I Loading — Pool at El. 701 after rapid drawdown from El. 781

Vert. Load — 150% of V_{sub} + 100% V_w up to El 701, plus 150% of V_{sat} to seepage line, plus V_{moist} to top.

$1.50 \times 12.8 \times 81 + 12.8 \times 62.5 + 1.5 \times 143 \times 52 + 1.5 \times 140 \times 49$
$1555 + 800 + 11,155 + 10,290 = 23800$ #/'.

Horiz Load
Substitute 50% in above assumptions where 150% appears

El. 688.2

$.50 \times 12.8 \times 81 + 12.8 \times 62.5 + .5 \times 143 \times 52 + .50 \times 140 \times 49$
$520 + 800 + 3,720 + 3,430 = 8470$ #/'

El. 682.7

$.50 \times 18.3 \times 81 + 18.3 \times 62.5 + .5 \times 143 \times 52 + .50 \times 140 \times 49$
$740 + 1140 + 3720 + 3430 = 9030$ #/'

Case II Loading - Pool at El 781

Vert. Load
150% of V_{sat} to seepage line plus 150% of V_{moist} to top.

$1.5 \times 64.8 \times 143 + 1.5 \times 49 \times 140 = 13,900 + 10290 = 24,190$

Horiz Load
Subst. 50% in above assumptions where 150% appears.

El. 688.2
$.5 \times 64.8 \times 143 + .5 \times 49 \times 140 = 4630 + 3430 = 8060$#

El. 682.7

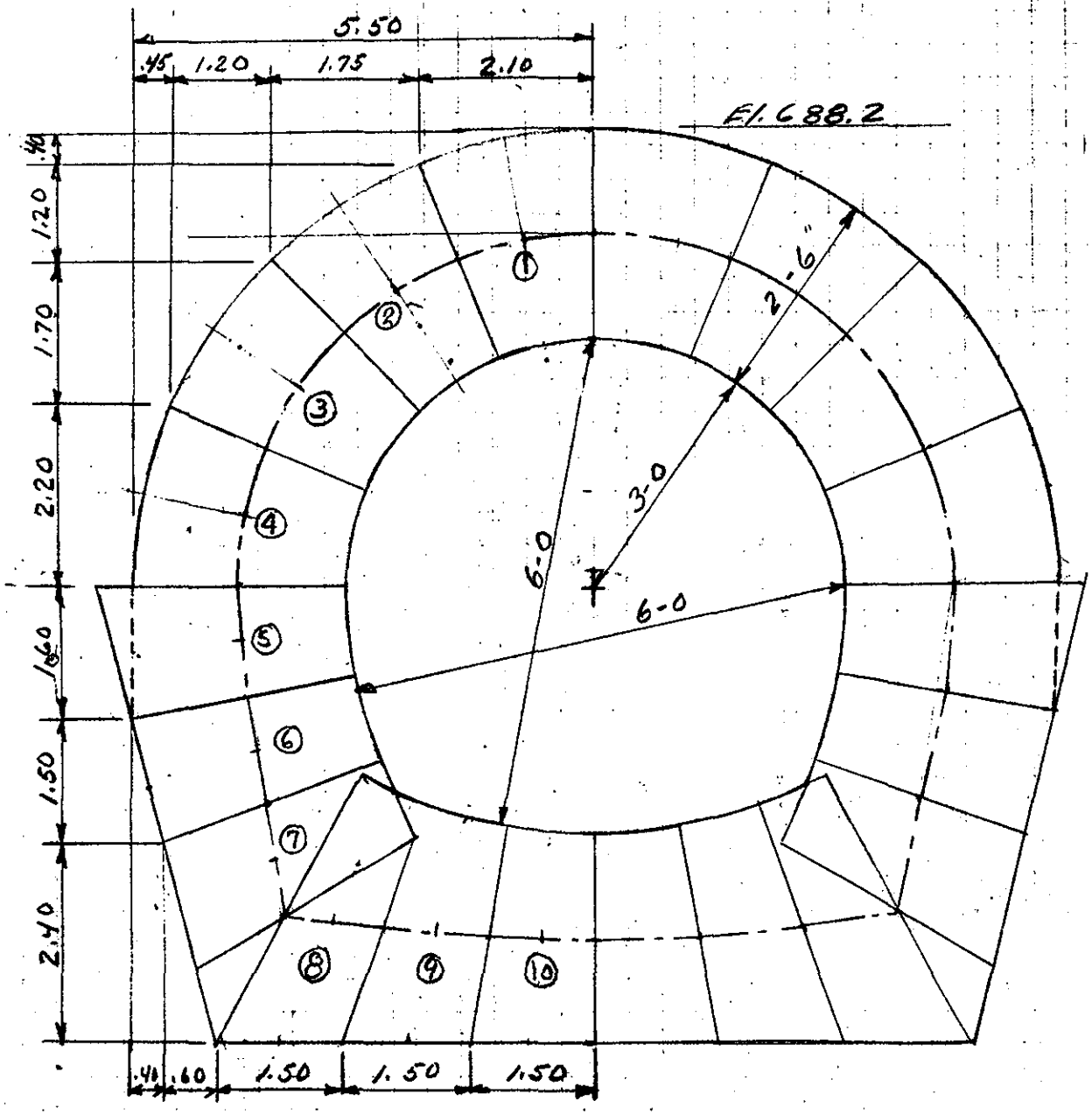
$.5 \times 70.3 \times 143 + .5 \times 49 \times 140 = 5030 + 3430 = 8460$

NED FORM 223
27 Sept 49

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

SUBJECT Otter Brook Dam
COMPUTATION Conduit Section at ℄ of Dam
COMPUTED BY H.E.W. CHECKED BY RSM DATE 11/7/55

Conduit at ℄ of Dam to be designed for Case II Loading



ARMY-NED-BOSTON

Given					Derived				Vertical Load							
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	
No.	d_s	d	x	y_1	a	ay_1	y	ay^2	Intensity of Vert. Load	Horiz. Proj. Vous.	Vert Load	Weight of Vous.	Total Vert. Load	Adj. Vert. Load	ΣV	
1	1.67	2.50	-0.80	-0.10	0.106	-0.011	+3.98	1.679	24.21	2.10	+50.84	0.63	+51.47	51.47	51.47	Table
2	1.67	2.50	-2.35	-0.70	0.106	-0.074	+3.38	1.211	24.37	1.75	+42.65	0.63	+43.28	+43.28	94.75	Cont'd
3	1.67	2.50	-3.55	-1.80	0.106	-0.191	+2.28	0.551	24.69	1.20	+29.63	0.63	+30.26	+30.26	125.01	on
4	1.67	2.50	-4.15	-3.35	0.106	-0.355	+0.73	0.056	25.11	0.45	+11.30	0.63	+11.93	+11.93	136.94	Next
5	1.33	2.60	-4.20	-4.85	0.076	-0.369	-0.77	0.045	0.	—	—	0.52	+0.52	+0.52	137.46	Sheet
6	1.33	2.70	-4.00	-6.20	0.068	-0.422	-2.12	0.306	-4.53	0.40	-1.81	0.54	-1.27	-19.55	117.91	
7	1.33	2.90	-3.80	-7.50	0.055	-0.413	-3.42	0.643	-4.63	0.60	-2.78	0.58	-2.20	-29.62	88.29	
8	1.23	3.30	-3.10	-8.25	0.034	-0.281	-4.17	0.591	-4.74	1.50	-7.11	0.61	-6.50	-75.06	13.23	
9	1.23	2.80	-1.90	-8.40	0.056	-0.470	-4.32	1.045	-4.74	1.50	-7.11	0.52	-6.59	-6.59	6.64	
10	1.23	2.55	-0.65	-8.50	0.074	-0.629	-4.42	1.446	-4.74	1.50	-7.11	0.47	-6.64	-6.64	—	
					0.787	-3.215		7.593					+114.26	0		

$$\bar{y} = \frac{-3.215}{0.787} = 4.08$$

Intensity of Vert. Loads:

No. 1 = 24.190 + (1.50)(0.10)(143) = 24.21 k/a'
2 = 24.190 + (1.50)(0.85)(143) = 24.37
3 = 24.190 + (1.50)(2.35)(143) = 24.69
4 = 24.190 + (1.50)(4.30)(143) = 25.11
5 = = 0
6 = (64.8 + 7.8) 62.5 = 4.53
7 = (64.8 + 9.25) " = 4.63
8 = (64.8 + 11.0) " = 4.74
9 = do = do
10 = do = do

The vert. load of 114.26 k will be distributed proportionately among vous. 6, 7 & 8

No. 6 = 0.4/2.5 (114.26) = -18.28
7 = 0.6/2.5 (") = -27.42
8 = 1.5/2.5 (") = -68.56
-114.26

NED FORM 223
27 Sept 49

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

SUBJECT: OTTER BROOK DAM
COMPUTATION: Cantilever Section @ ℄ Dam
COMPUTED BY: LSM
CHECKED BY: GAM
DATE: Nov. 1955
PAGE A-34

Horiz. Load					Diff. in Coord		Cantilever Moment									
17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33
No.	Intensity of Hor Load	Vert. Proj. of Vous	Horiz. Load	ΣH	$x_n - x_{n-1}$	$y_n - y_{n-1}$	Vert.	Horiz.	m	ma	may	m+M	H×y	Mom.	Thrust	Shear
1	8.07	0.40	+3.23	3.23	-1.55	-0.60	-9.96	-0.24	-10.20	-1.08	-4.30	+294.27	-220.37	+73.90	57.8	15.0
2	8.13	1.20	+9.76	12.99	-1.20	-1.10	-89.74	-2.18	-91.92	-9.74	-32.92	+212.55	-187.16	+25.40	79.5	35.5
3	8.24	1.70	+14.01	27.00	-0.60	-1.50	-203.44	-16.47	-219.91	-23.31	-53.15	+84.56	-126.24	-41.68	111.0	32.5
4	8.37	2.20	+18.41	45.41	-0.05	-1.50	-278.45	-56.97	-335.42	-35.55	-25.95	-30.95	-40.42	-71.37	132.5	7.9
5	4.40	1.60	+7.04	52.45	+0.20	-1.35	-285.36	-125.09	-410.39	-31.19	+24.02	-105.92	+42.63	-63.29	136.0	18.0
6	4.54	1.50	+6.81	59.26	+0.20	-1.30	-257.81	-195.90	-453.71	-30.85	+65.40	-149.24	+117.38	-31.86	126.0	20.8
7	4.66	2.40	+11.18	70.44	+0.70	-0.75	-234.23	-272.94	-507.19	-27.89	+95.38	-202.70	+189.37	-13.33	103.5	8.0
8	0	–	–	70.44	+1.20	-0.75	-172.43	-325.77	-498.20	-16.94	+70.64	-193.73	+230.89	+37.16	20.0	49.0
9	0	–	–	70.44	+1.25	-0.10	-156.55	-336.39	-492.89	-27.60	+119.23	-188.42	+239.20	+50.78	15.7	9.0
10	0	–	–	70.44	–	–	-148.25	-343.38	-491.63	-36.38	+160.80	-187.16	+244.74	+57.58	15.2	2.7
										-240.53	+419.15					

$$M = -\frac{\Sigma ma}{a} = -\frac{-240.53}{0.787} = +304.47.$$

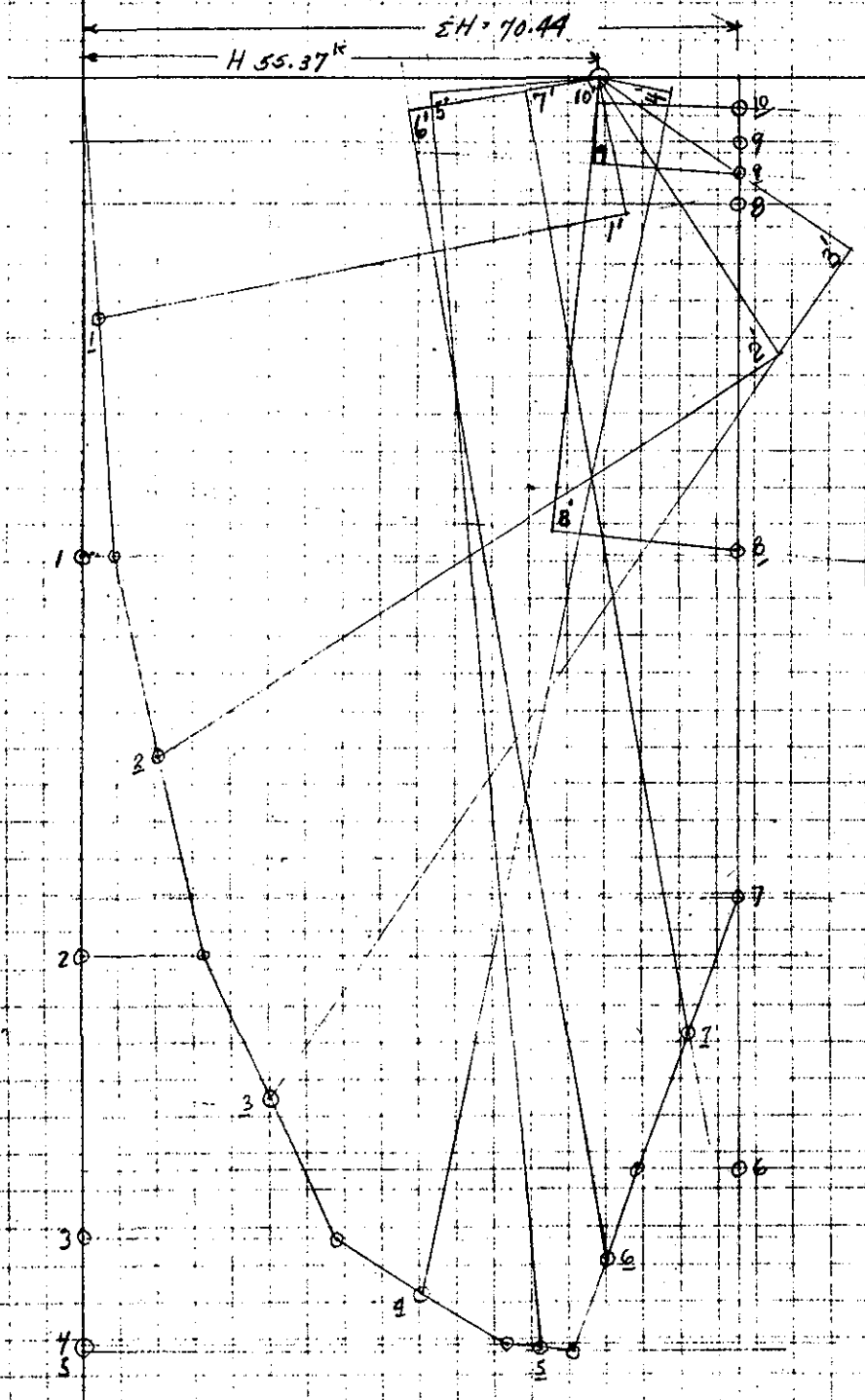
$$H = -\frac{\Sigma may}{\Sigma ay^2} = -\frac{419.15}{7.57} = -55.37.$$

ARMY-NED-BOSTON

SUBJECT OTTER BROOK DAM

COMPUTATION Conduit Section @ E Dam

COMPUTED BY JSM CHECKED BY RSM DATE Nov. 1955



	Thrust	Shear
1	57.8	15.0
2	79.5	35.5
3	111.0	32.5
4	132.5	7.9
5	136.0	18.0
6	126.0	20.8
7	103.5	8.0
8	20.0	49.0
9	15.7	9.0
10	15.2	2.7

Force Diagram

Scale: 1" = 20^K

SUBJECT OTTER BROOK DAM

COMPUTATION Conduit Section @ ℄ Dam

COMPUTED BY JSM CHECKED BY RBM DATE Nov. 1955

Design of Reinf.:

$\frac{H_f}{H_c} = \frac{109}{114} = 0.96 > 0.75$ ∴ Use stresses allowed for Condition III.

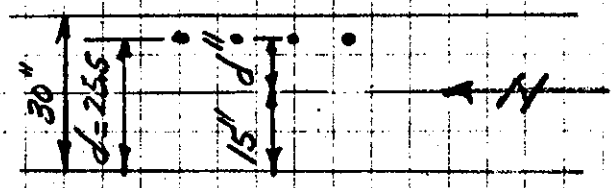
Concrete:

$f_c' = 4{,}000$ #/□"

$f_c = 0.45 f_c' = 1800$ #/□" $K = 359$, $j = 0.842$

$v = 0.045 f_c' = 180$ #/□"

Steel $f_s = 20{,}000$ #/□"



Vous. No. 1

Mom. = +73.90^{1k}, Thrust = 57.8^{k}, Shear = 15.0^{k}

For axial comp.

$M_s = M + \frac{d''}{12} N = 73{,}900 + \frac{10.5}{12} 57{,}800 = 124{,}400^{1\#}$

Req'd $d_M = \sqrt{\frac{124400}{359} = 385} = 18.7''$ have $25.5''$

$v = \frac{15{,}000}{12 \times 0.842 \times 25.5} = 58$ #/□" OK

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

$= \frac{12 \times 124{,}400}{20{,}000 \times 0.842 \times 25.5} - \frac{57{,}800}{20{,}000}$

$= 3.5 - 2.9 = \frac{0.6}{3}$ □"/'

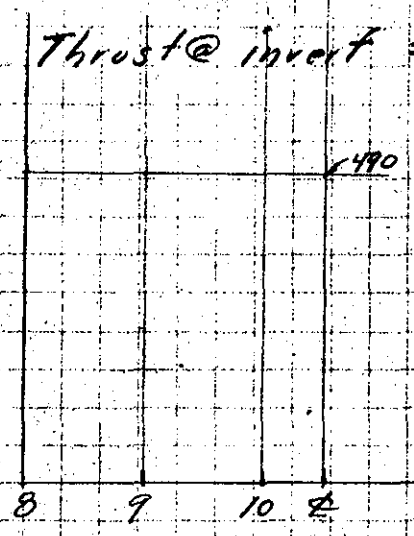
Vous. No. 2

SUBJECT OTTER BROOK DAM
COMPUTATION Conduit Section @ ℄ Dam
COMPUTED BY JSM CHECKED BY RSM DATE Nov. 1955

M @ crown $= m + M + Hy$
$= 0 + 304.47 + (-55.37)(4.08)$
$= +78.47^{1k}$
Thrust @ " $= 55.37^{k}$ ✓

M @ Invert $= -490 + 304.47 + (-55.37)(4.42) = 58.5^{1k}$ ✓

Thrust @ invert $= \Sigma H - H = 70.44 - 55.37 = 15.07^{k}$ ✓



M_s @ crown $= 78{,}470 + \frac{10.5}{12} 55370$
$= 126{,}870^{1\#}$

$A_s = \frac{12 \times 126{,}870}{20{,}000 \times 0.842 \times 25.5} - \frac{55{,}370}{20{,}000}$
$= 3.56 - 2.76 = \underline{0.8}^{\square''/ft}$

Use #9 @ 12"

Vous. No. 4:

$M = -71.37^{1k}$, Thrust $= 132.5^{k}$, Shear 7.9^{k}

$M_s = 71{,}370 + \frac{10.5}{12} 132{,}500 = 174{,}500^{1\#}$ $d = \sqrt{\frac{174500}{359}} = 22''$ reqd.

$A_s = \frac{12 \times 174{,}500}{20{,}000 \times 0.842 \times 25.5} - \frac{132{,}500}{20{,}000}$
$= 4.9^{\square''} - 6.6^{\square''}$ Use min. steel. #6 @ 12"

M @ Invert: $+M = 58.5^{1k}$, Thrust $= 15.07^{k}$

$M_s = 58{,}500 + \frac{10.5}{12} 15{,}070 = 71{,}700^{1\#}$

$A_s = \frac{12 \times 71{,}700}{20{,}000 \times 0.842 \times 25.5} - \frac{15{,}070}{20{,}000}$
$= 2.0 - 0.75 = \underline{1.25}^{\square''/ft}$ Use #10 @ 12" #9 @ 9 Used see P.9

At Vous. No. 2
$v = 35500 / 12 \times .842 \times 25.5 = 138\ \#/\square''$ (180 #/□" OK (Bond not a factor)

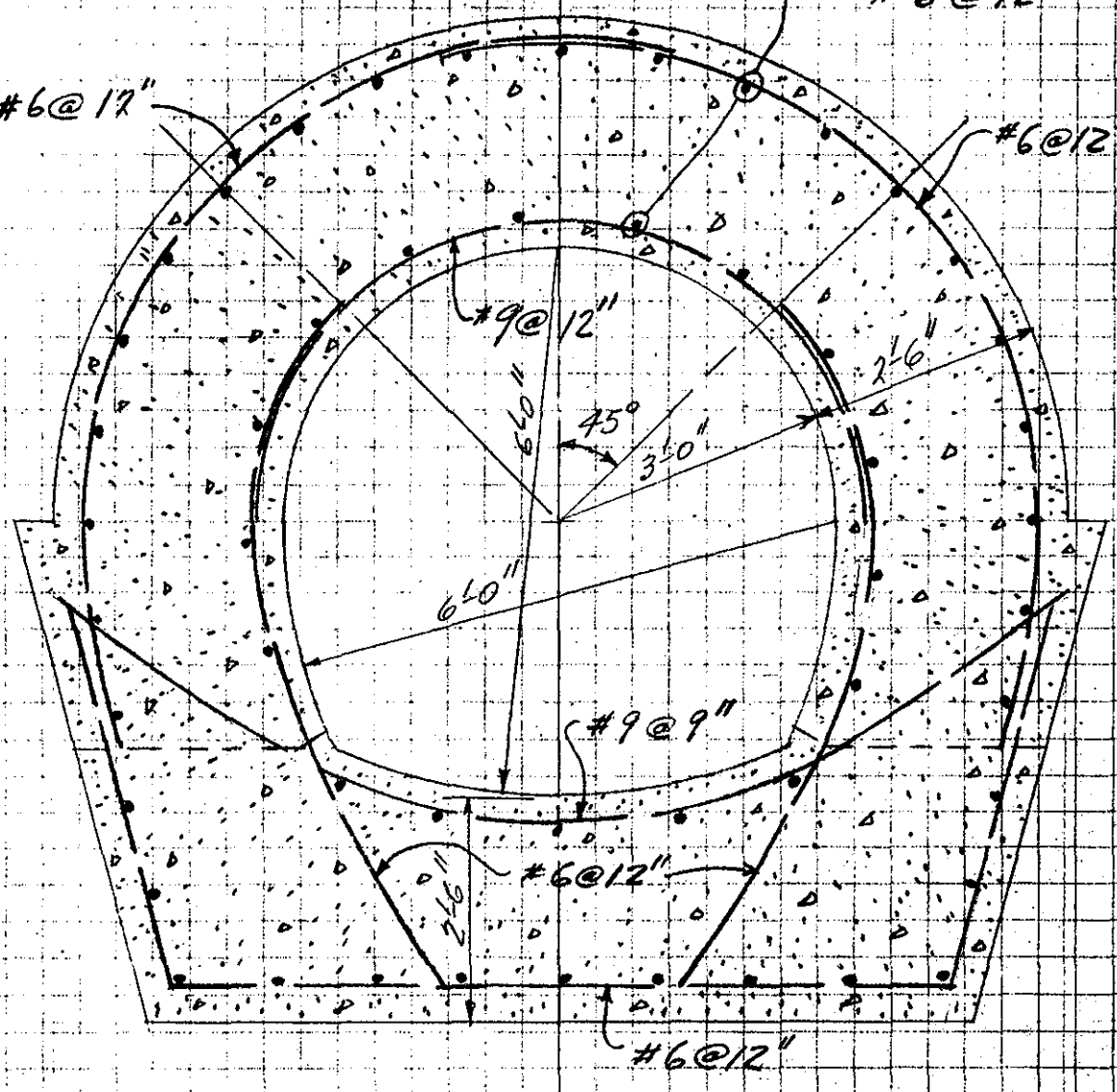
SUBJECT: OTTER BROOK DAM
COMPUTATION: Conduit Section @ ℄ Dam
COMPUTED BY: JSM CHECKED BY: RBM DATE: Nov. 1955

Max. shear @ No. 8 = 49.0 k

$$v = \frac{49{,}000}{12 \times 0.842 \times 32.5} = 150 \text{ psi} < 0K$$

$$\mu = \frac{49{,}000}{4.5 \times 0.842 \times 34.5} = 376 \text{ psi} > 350$$

$$\text{Reqd } \Sigma_o = \frac{49{,}000}{350 \times 0.842 \times 34.5} = 4.82 \quad \#9 @ 9''$$



Conduit Section @ ℄ Dam

SUBJECT Otter Brook Dam
COMPUTATION Service Bridge to Intake Tower
COMPUTED BY EPR CHECKED BY RMM DATE 10-31-55

Use H15 Truck load max. wheel load = 12000#

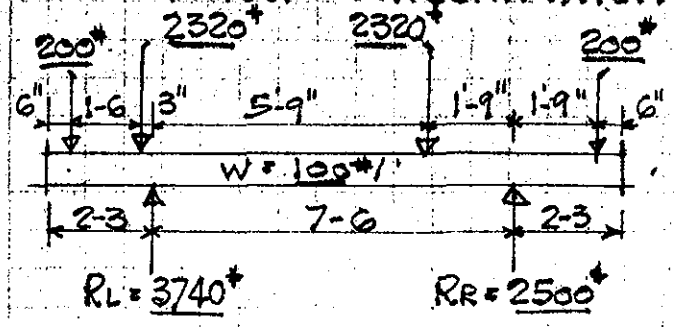
Slab span between supports = 7'-6".

LL impact = 50/7.5+125 = 50/132.5 = 0.37
Use max. 30%

Design wheel load of 1.3×12000 = 15600#

Width of slab over which wheel load may be distributed = 0.45 + 3.75
= (0.4×7.5) + 3.75 = 6.75'

Wheel concentration = 15600 ÷ 6.75 = 2320#



Curb + Railg = 200#
Assume 8" Sl = 100#

Truck located with wheel 12" from curb

RL = 600 + 200 = 800#
4640 { 2320 × 7.75/7.5 = 2400
2320 × 1.75/7.5 = 540
3740#

RR = 600 + 200 = 800
4640 − 2940 = 1700
2500#

+M = (1.75×2500) − (400×2.0) − (200×3.5)
= 4400 − 800 − 700 = 2900'#

−M = 2320×0.25 = 580
225×1.13 = 255
200×1.75 = 300 ok
V = 2745. 1135'#

DL +M = 3.75×800 = 3000
− 3 × 600 = −1800
− 5.5 × 200 = −1100
+100

t = 8" d = 6" M/bd² = 2900/(6)² = 81

+As = 12×2.9/18×j×6 = 0.37 □"

v = 2745/12×j×6 = 44#/□"

Min. −As = 0.005×12×6 = 0.36 □"

8" Slab #5 @ 8" o.c. t.&b. across bridge width;
#4 @ 12" o.c. t.&b. length of bridge

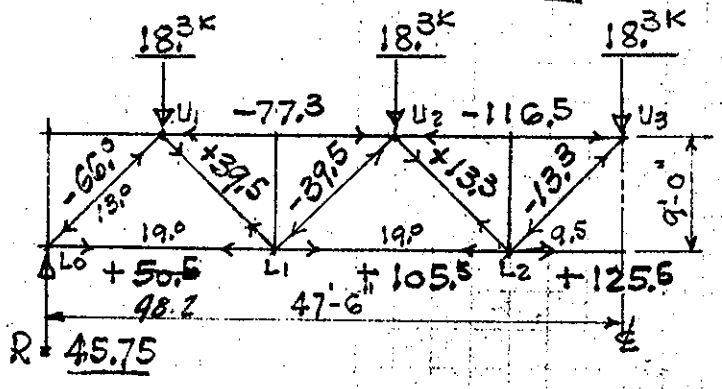
SUBJECT Otter Brook Dam
COMPUTATION Service Bridge
COMPUTED BY _____ CHECKED BY RSM DATE 10-31-55

Truss 7'-6" o.c. Span = 95'-0".

DL at panel points
19 × 960# = 18300#.

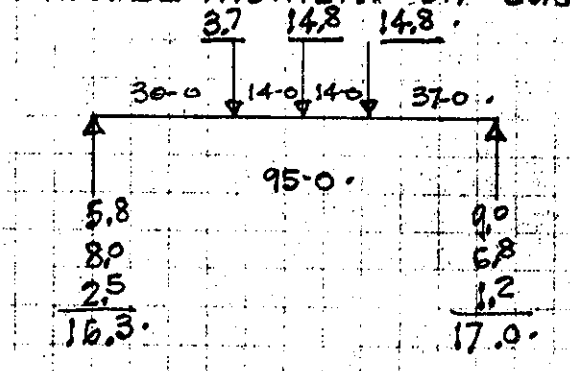
Slab Ld. = 800#
Truss T.C. = 50
BC = 60
Web = 30
Brcg = 20
960#/'.

R = 2.5 × 18.3 = 45.75K



U_1U_2 = (19.0 × 45.75) − (9.5 × 18.3) / 9.0 = −77.3.
U_2U_3 = (38.0 × 45.75) − 18.3 (9.5 + 28.5) / 9.0 = −116.5.
L_0L_1 = (9.5 × 45.75) / 9.0 = + ~~50.5~~ 48.2
L_1L_2 = (28.5 × 45.75) − (19.0 × 18.3) / 9.0 = + 105.5.
L_2L_3 = (47.5 × 45.75) − 18.3 (19.0 + 38.0) / 9.0 = + 125.5.
L_0U_1 = 45.75 × 13.0 / 9.0 = − 66.0.
U_1L_1 = (45.75 − 18.3) 13.0 / 9.0 = + 39.5.
L_1U_2 = Same = − 39.5.
U_2L_2 = (27.45 − 18.3) 13.0 / 9.0 = + 13.3.
L_2U_3 = Same = − 13.3.

Max. LL moment on each truss H15-S12 Loading

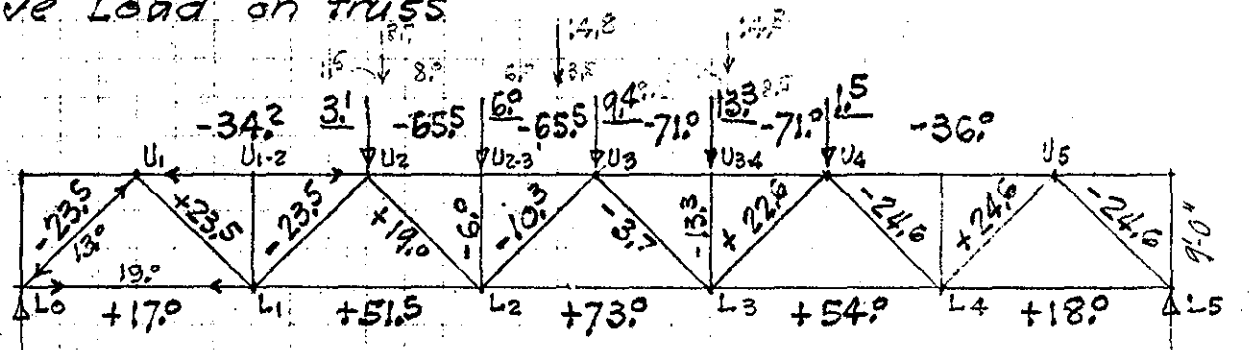


Impact = 50/(95 + 125) = 0.23.

M = (51.0 × 17.0) − (14.0 × 14.8)
= 867.0 − 207.0 = 659.0 K'

SUBJECT Otter Brook Dam
COMPUTATION Service Bridge
COMPUTED BY _____ CHECKED BY RJM DATE 11-1-55

Live Load on truss



R_L = 16.22 R_R = 17.08

R_L =	3.1 × 66.5 / 95	= 2.17	R_R =	3.10 − 2.17	= 0.93
	6.0 × 57.0 / 95	= 3.60		6.0 − 3.60	= 2.40
	9.4 × 47.5 / 95	= 4.70		9.4 − 4.7	= 4.70
	13.3 × 38.0 / 95	= 5.30		13.3 − 5.3	= 8.00
	1.5 × 28.5 / 95	= 0.45		1.5 − 0.45	= 1.05
	33.3	16.22			17.08

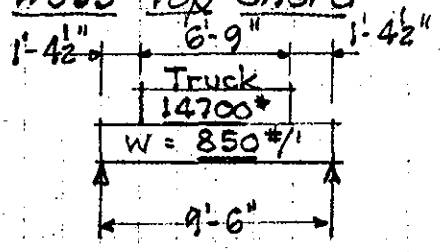
U_1U_2 = 19.0 × 16.22 / 9.0 = −34.2
U_2U_3 = (38.0 × 16.22) − (9.5 × 3.1) / 9.0 = −65.5
U_3U_4 = (57.0 × 16.22) − (9.5 × 9.4) − (19.0 × 6.0) − (28.5 × 3.1) / 9.0 = −71.0
= (38.0 × 17.08) − (9.5 × 1.5) / 9.0 = −71.0
U_4U_5 = (19.0 × 17.08 / 9.0 = −36.0
L_0L_1 = (9.5 × 16.22 / 9.0 = +17.0
L_1L_2 = 28.5 × 16.22 / 9.0 = +51.5
L_2L_3 = (47.5 × 16.22) − (9.5 × 6.0) − (19.0 × 3.1) / 9.0 = +73.0
= (47.5 × 17.08) − (9.5 × 13.3) − (19.0 × 1.5) / 9.0 = +73.0

L_0U_1 = 16.22 × 13 / 9 = 23.5
U_1L_1 = Same = 23.5
L_1U_2 = Same = 23.5
U_2L_2 = (16.22 − 3.1) 13 / 9 = 19.0
L_2U_3 = (13.12 − 6.0) 13 / 9 = 10.3
U_5L_5 = 17.08 × 13 / 9 = 24.6
L_4U_5 = Same = 24.6
U_4L_4 = Same = 24.6
L_3U_4 = (17.08 − 1.5) 13 / 9 = 22.6
U_3L_3 = (15.58 − 13.3) 13 / 9 = 3.73 OK

L_3L_4 = 28.5 × 17.08 / 9 = +54.0
L_4L_5 = 9.5 × 17.08 / 9 = +18.0

SUBJECT Otter Brook Dam
COMPUTATION Service Bridge
COMPUTED BY EPR CHECKED BY gfm DATE 11-1-55

Truss Top Chord



Max. stress DL = 116.5
LL = 71.0
187.5 C

Truck wheel ld. spread over 6'-9" (See Slab)
Ld + Impact 12000 × 1.23 = 14700#

W = Slab 800#
Chd 50
850#/'

$M = 0.85 \times \overline{9.5}^2/12 = 6.4$
$(1+0.28-0.04)\frac{1}{12} \times 14.7 \times 9.5 = 14.6$
21.0 k'

Try 2-12 JL 25

A = 2 × 7.32 = 14.64 □"
S = 2 × 23.9 = 47.8 in³
$I_{yy} = 2[4.5 + (7.32 \times 0.93)] = 21.6$ in⁴
restrained by slab
$r_{xx} = 4.43$ $l/r = 114/4.43 = 26$
$F_a = 16000 - \frac{0.485 l^2}{r^2} = 15680$

fa = 187500 ÷ 14.64 = 12800

fb = 12 × 21000 / 47.8 = 5250 Fb = 21000
$\frac{f_a}{F_a} + \frac{f_b}{F_b} = \frac{12800}{15680} + \frac{5250}{21000} = 0.815 + 0.25 = 1.065$

Truss Bottom Chord Max. stress = 125.5 + 73.0 = 198.5 K T

2 JL 8 × 6 × ½ A(net) = 13.5 − (2 × 0.5 × 1.0) = 12.5 □"
f = 198.500 ÷ 12.5 = 15.900 #/□"

Weight of Truss

TC	2 × 25# × 95.0	= 4750
BC	2 × 23# × 95.0	= 4400
End Diag.	2 × 30# × 13.0 × 2	= 1560
Inner Diag.	2 × 21# × 13.0 × 8	= 4400
" Vert	2 × 10# × 9.0 × 6	= 1080
		16190
Gusset		1610
		17800 or 9 Tons ea.

truss exclusive of bracing or bearings

NED FORM 223
27 Sept 49

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

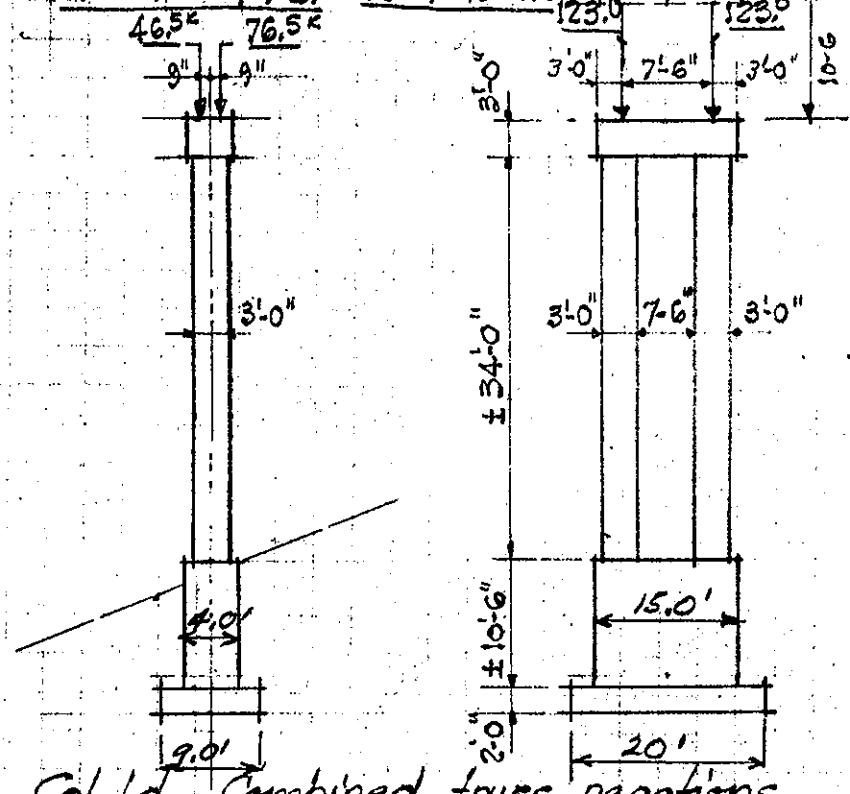
SUBJECT Otter Brook Dam
COMPUTATION Service Bridge
COMPUTED BY EPR CHECKED BY DATE Jan. 5, 1953

	D.L.S	Max.LL	Combined	Sections
L_0L_1	+ 48.2	+ 28.2	+ 76.4	Bott. Chord
L_1L_2	+105.5	+ 62.0	+ 167.5	
L_2L_3	+125.5	+ 73.0	+ 198.5	2 ⊥L 8 × 6 × 1/2
L_3L_4	+105.5	+ 62.0	+ 167.5	
L_4L_5	+ 48.2	+ 28.2	+ 76.4	
U_1U_2	- 77.3	- 49.4	- 126.7	Top Chord
U_2U_3	-116.5	- 72.2	- 188.7	
U_3U_4	-116.5	- 72.2	- 188.7	2-12 ⊥C 25
U_4U_5	- 77.3	- 49.4	- 126.7	
L_0U_1	- 66.0	- 38.5	- 104.5	2 ⊤ 6 × 6 × 3/8
U_1L_1	+ 39.5	+ 34.0	+ 73.5	2 ⊤ 5 × 3 1/2 × 3/8
$U_{1-2}L_1$	0	- 15.0	- 15.0	2 ⊤ 3 1/2 × 3 1/2 × 5/16
L_1U_2	- 39.5	- 28.3	- 67.8	2 ⊤ 5 × 3 1/2 × 3/8
U_2L_2	+ 13.3	+ 22.6	+ 35.9	2 ⊤ 4 × 3 × 3/8
$U_{2-3}L_2$	0	- 15.0	- 15.0	2 ⊤ 3 1/2 × 3 1/2 × 5/16
L_2U_3	- 13.3	- 14.5	- 27.8	2 ⊤ 4 × 3 × 5/16
U_3L_3	- 13.3	- 14.5	- 27.8	
$U_{3-4}L_3$	0	- 15.0	- 15.0	
L_3U_4	+ 13.3	+ 22.6	+ 35.9	
U_4L_4	- 39.5	- 28.3	- 67.8	
$U_{4-5}L_4$	0	- 15.0	- 15.0	
L_4U_5	+ 39.5	+ 34.0	+ 73.5	
U_5L_5	- 66.0	- 38.5	- 104.5	

End Posts
2 ⊥L 5 × 3 1/2 × 3/8

SUBJECT Otter Brook Dam
COMPUTATION Service Bridge
COMPUTED BY EPR CHECKED BY PSW DATE 11-1-55

Middle Pier - Vert. loads

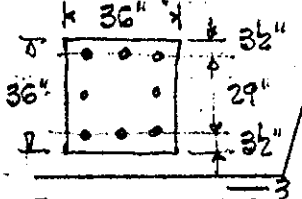


Wind on L.L. = 30°×200# = 6000#.
Wind Ld. on floor on trusses 300#×95° = 28500#.
360#.
3.0°×30#×34.0° = 3050#.

Col. Ld. Combined truss reactions = 123,000#.
Cap 3.0×150#×4.0×5.5 = 9,900
Col. (3)²×150#×34.0 = 46,000
178,900.#

M of unbalanced reactions = 0.75×30000 = 22,500'#

n = 10



Ac = 1296 □" As = 12.48 □" (4.68 □" ea. face)
A = 1296 + (9×12.48) = 1408 □"

$\frac{36x^2}{2} + (9\times 4.68)(x-3.5) = (10\times 4.68)(32.5-x)$

$x^2 + 4.95x - 92.5 = 0$

$x = \frac{-4.95 \pm \sqrt{24.5+370}}{2} = 7.45"$.

$I_c = 36\times 7.45^3\times\frac{1}{3} = 4960$
$I_s' = 9\times 4.68\times 3.95^2 = 655$
$I_s = 10\times 4.68\times 25.05^2 = 29400$
35015 in⁴. Void

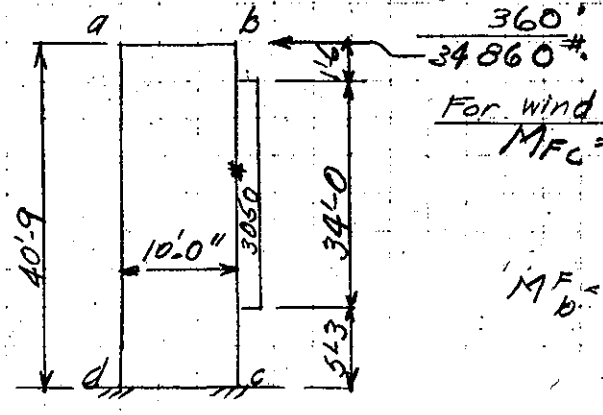
$f_c = \frac{178900}{1408} + \frac{12\times 22500\times 7.45}{35015} = 127 + 57 = 184$ #/□".

$f_s' = 9\left(127 + \frac{12\times 22500\times 3.95}{35015}\right) = 9(127+30) = 1410$ #/□".

$f_s = 10\left(127 - \frac{12\times 22500\times 25.05}{35015}\right) = 10(127-193) = -660$ #/□".

SUBJECT: Otter Brook Dam
COMPUTATION: Service bridge
COMPUTED BY: [illegible] CHECKED BY: EPR DATE: 11/16/55

Middle pier - transverse loads:



6000#
28500#
360#
34860#

ab 4'x3' $I = 4 \times 3^3/12 = 9$
$K = 9/10 = 0.9$

bc 3'x3' $I = 3^4/12 = 6.75$
$K = 6.75/40.75 = 0.17$

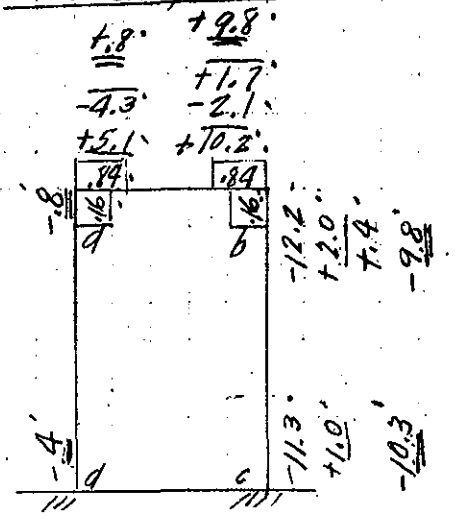
For wind on piers

$$M_{Fc} = \frac{90}{12 \times 40.75^2}\left[35.5^3(163-107) - 1.5^3(163-4)\right]$$
$$= 11.3^K$$

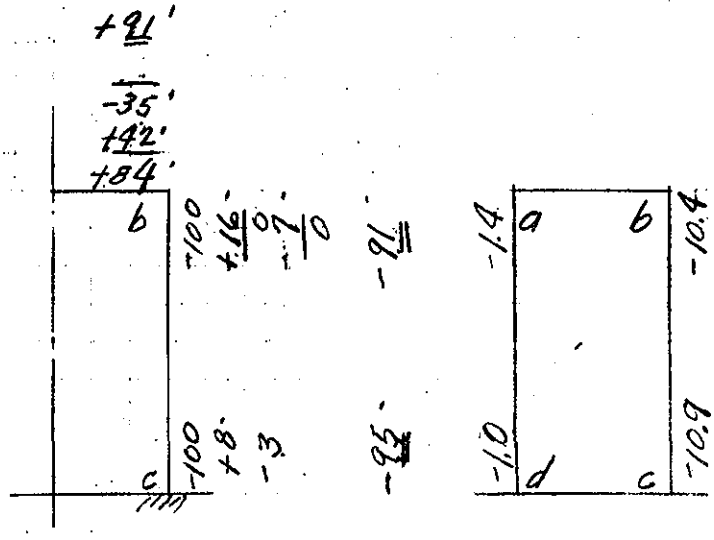
$$M_{Fb} = \frac{90}{12 \times 40.75^2}\left[39.25^3(163-117) - 5.25^3(163-16)\right]$$
$$= 12.2^K$$

Assume fixed at base Design by Cross & Morgan, pg. 109, 110.

Wind on piers



No sidesway Distribution



Sidesway:

$(10.3 + 9.8 + 0.8 + 0.4)/40.75 = .52^K$ $2(95+91)/40.75 = 9.2$

Correct by factor $\frac{.52}{9.2}$: at b: $\frac{.52}{9.2} \times 97 = .6$, same at c

Final moments

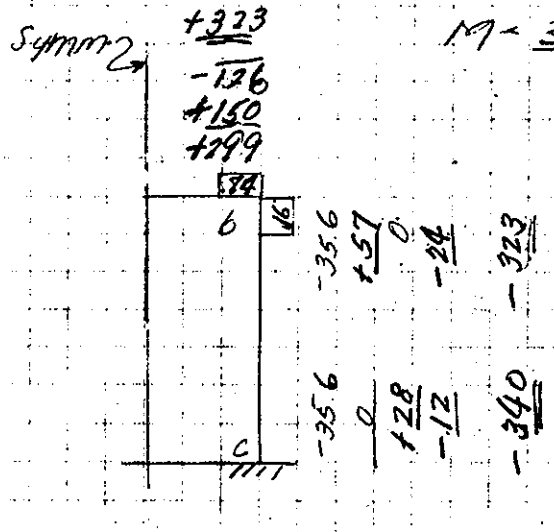
M_{da} $-.4 - .6 = -1.0$ M_{bc} $-9.8 - .6 = -10.4$

M_{cb} $= -10.3 - .6 = -10.9$ M_{ad} $-.8 - .6 = -1.4$

SUBJECT Otter Brook Dam
COMPUTATION Service Bridge
COMPUTED BY RJM CHECKED BY EPR DATE 11/16/55

Middle pier - transverse loads

Wind on superstructure



$M = \frac{34860}{2} \times \frac{40.75}{2} = 356'^K$

Apparent shear

$\frac{323+340}{40.75} \times 2 = 32.5^K$

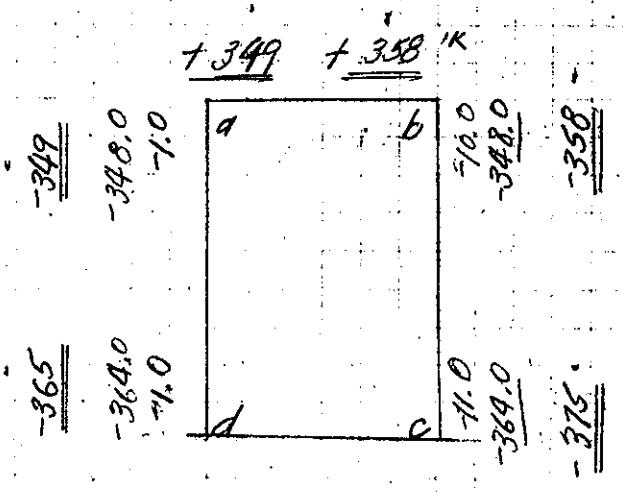
Actual shear 34.86^K

Adjust moments by factor of 34.86/32.5

$M_{cb} = M_{da} = \frac{34.86}{32.5} \times 340 = 364'^K$

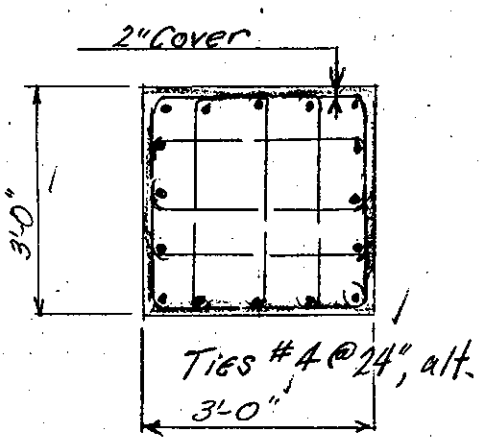
$M_{bc} = M_{ad} = \frac{34.86}{32.5} \times 323 = 348'^K$

Transverse loads - Total moments.



SUBJECT: Otter Brook Dam
COMPUTATION: Service Bridge
COMPUTED BY: [illegible] CHECKED BY: EPR DATE: 11/17/55

Midde pier - Design of columns
Design by Reinf. Conc. Design Handbook, Ex. 32, pg. 40



Reinf. 16-#11 = 25.0 □"

$g = 32/36 = .89$ $n = 10$, $p = .02$

$M = 375 + 22.5 = 397.5^{'K}$ max, $387.5^{'K}$ min.

$N = 179^K \pm \left(\frac{365+349+338+375}{10}\right) = 324^K$ max, 34^K min.

For max col. load.

$f_a = \frac{1.33 \times .8(.225 \times 3000 + 14000 \times .02)}{1 + 9 \times .02} = 870$ #/□"

$D = 4.8$

$e' = 397.5/324 = 1.23'$ $D\frac{e'}{t} = 4.8 \times \frac{1.23}{3.0} = 1.97$

$\frac{f_a}{f_o} = \frac{870}{3000} = .29$ $f_p = 1{,}140$ #/□" allowable (Table 34)

$Q = \frac{36 \times 36}{324000} \times 1{,}140 = 4.5$

For $\frac{e'}{t} = .41$, $Q = 4.5$, and $g = .89$, $np = .05$ (Diagram 30) - Use $p = .02$ - see below.

For min. col. load
$N = 34^K$; $M = 387.5^{'K}$

$e' = 387.5/34 = 11.4'$ Since $e > jd$, design by beam formula, Ex. 15 pg. 21. Neglect steel in sides of column.

$e = 137 + 15.5 = 152.5''$ $e/d = 152.5/33.5 = 4.55$

From Table 10, for $j = .88$, $i = 1.25$

$m = \frac{10 \times 7.8}{36 \times 33.5} \times 1.24 + \frac{9 \times 7.8}{36 \times 33.5} = .080 + .058 = .138$

$q = .080 + .058 \times \frac{2.5}{33.5} = .080 + .0044 = .0844$ From Table 11, $k = 0.30$

$\frac{1}{.30} \times \frac{9 \times 7.8}{1208} = .19$ $\frac{1}{.30} \times \frac{2.5}{33.5} = .25$ From Table 12, $z = .32$

For $z = .32$ and $k = 0.30$, $j = .90$ (Table 13) $i = 1.25$ (Table 10)

$f_s = \frac{34000}{.90 \times 7.8 \times 1.25} \times 4.55 = 17{,}600$ #/□" $- 14000 \times 1.33 = 18700$ #/□" allowable

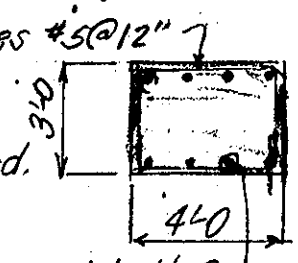
$f_c = \frac{17600}{10} \times \frac{.30}{1 - .30} = 760$ #/□" $- 540 \times 1.33 = 720$ #/□" allowable.

Note: $\frac{L}{D} = \frac{34}{3} = 11.3 > 10$. Reduction in allowable stresses is negligible.

SUBJECT Otter Brook Dam
COMPUTATION Service Bridge
COMPUTED BY [illegible] CHECKED BY EPR DATE 11/17/55

Design of Cap

Design for moment due to wind loads. Load of bridge (vert.) transmitted directly to columns.



$M = 358^{1k}$ wind load $+ (4 \times 3 \times .15) \times \overline{10}^2/12 = 15^{1k}$

$d = \sqrt{\frac{373\,000 \times 12}{48 \times 169 \times 1.33}} = \sqrt{415} = 20.4''$ reqd. - 33.5" supplied.

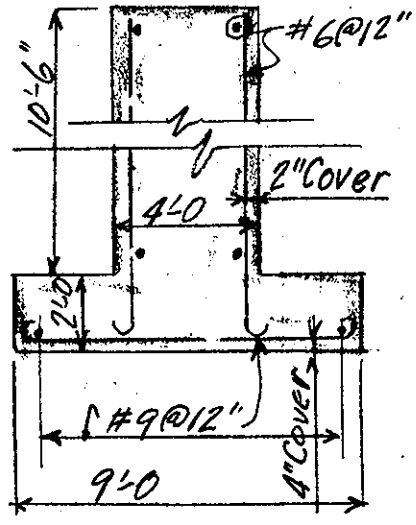
$\pm A_s = 373 \times 12/24 \times .88 \times 33.5 = 6.2\ \square''$ - Use 4 - #11, top and bott.

Footing

Loading

Bridge		246^k
Cap	$4 \times 3 \times 13.5 \times .15 =$	24
Cols.	$3 \times 3 \times 34 \times 2 \times .15 =$	92
Base	$4 \times 13.5 \times 10.5 \times .15 =$	85
Footing	$9 \times 20 \times 2 \times .15 =$	54
Earth	$[(9 \times 20) - (4 \times 13.5)] \times 9 \times .125 =$	141
		642^k

Moment	6×66	$= 396^{1k}$
	28.5×56.7	1615
	3.4×30	102
		2113^{1k}



$e = 2113/642 = 3.3'$

$p = \frac{642}{9 \times 20}\left(1 \pm \frac{6 \times 3.3}{20}\right) = 0$ to $7100\ \#/\square'$ with wind loading

$p = 642/9 \times 20 = 3550\ \#/\square'$ w/o wind loading

Base

Load/ft. $= 3.55 \times 9 - 4 \times 10.5 \times .15 - 5 \times .123 \times 9 = 19.7^k/'$ net

$M = 19.7 \times \overline{10}^2/12 = 165^{1k}$ for direct load.

$A_s = 165 \times 12/18 \times .88 \times 145 = .86\ \square''$ - Use temp. steel.

M for wind load will not require added steel.

Footing

2.5' projection $W = 7.1 - 2 \times .15 - 9 \times .13 = 5.6^k/\square'$

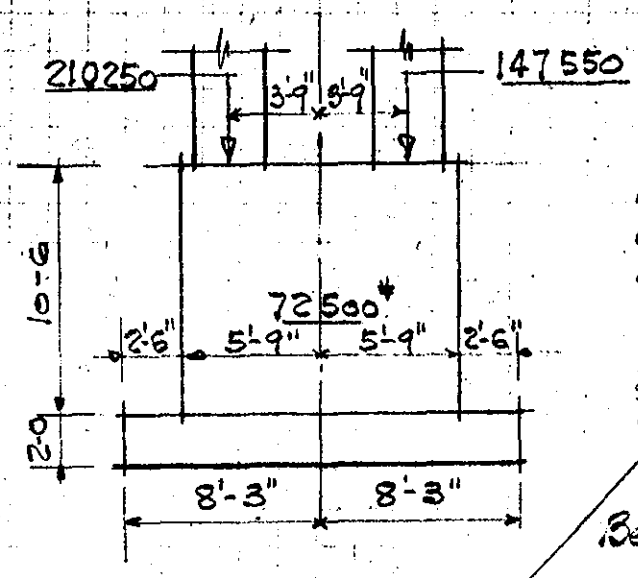
$M = 5.6 \times \overline{2.5}^2/2 = 17.5^{1k}$ $d = \sqrt{\frac{17,500}{169 \times 1.33}} = 8.8''$ reqd.

$A_s = 17.5 \times 12/24 \times .88 \times 19.5 = .5\ \square''$

Temp. steel $= 12 \times 24 \times .005 = 1.44\ \square''$ - Use #9 @12" - adequate.

SUBJECT Otter Brook Dam
COMPUTATION Service Bridge
COMPUTED BY _____ CHECKED BY PBM DATE 11-1-55

Middle Pier - Transverse Wind



Void

	147.55 × 4.5	=	665.0
Col's	210.25 × 12.0	=	2525.0
Base	72.8 × 8.25	=	597.0
Ftg.	44.5 × 8.25	=	367.5
Earth	132.0 × 8.25	=	1090.0
	606.80		5244.5

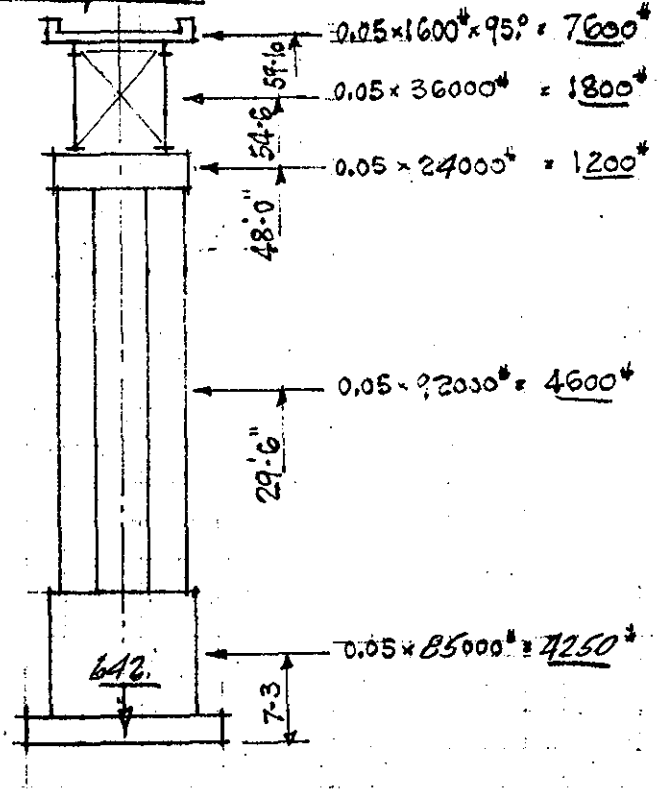
$\frac{\Sigma M}{\Sigma V} = \frac{5244.5}{606.8} = 8.66'$

$e = 8.66 - 8.25 = 0.41'$

$\text{Bearing} = \frac{606.8}{148.5} \pm \frac{606.8 \times 0.41 \times 6}{9 \times (16.5)^2}$

$= 40 \pm 610 = 4700$ b.s.f.
3480 b.s.f.

Earthquake



E.Q.		moment abt. base	
7.6	× 59.83	=	455.0
1.8	× 54.50	=	98.0
1.2	× 48.00	=	57.4
4.6	× 29.5	=	135.5
4.3	× 7.25	=	31.1
19.5			777.

$e = 777 \div 642 = 1.21'$

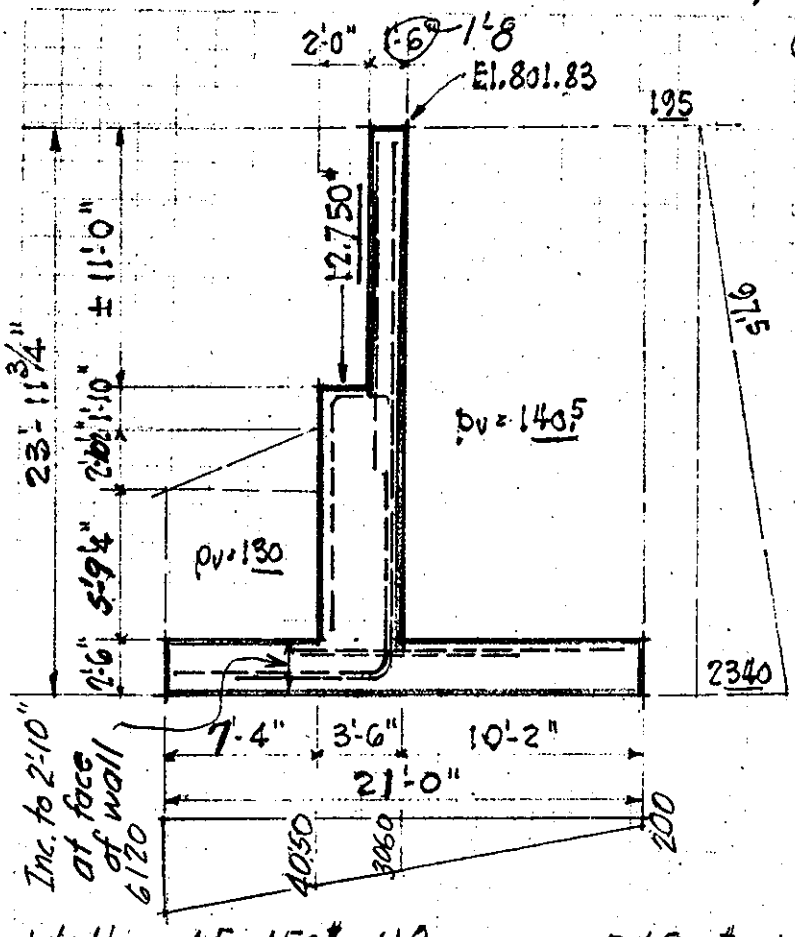
$\text{Bearing} = \frac{642}{9 \times 20}\left(1 \pm \frac{6 \times 1.21}{20}\right)$

= 4800#/□' to 2300#/□' OK

NED FORM 223
27 Sept 49

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

SUBJECT: Otter Brook Dam
COMPUTATION: Service Bridge Abutment
COMPUTED BY: E.P.R. CHECKED BY: [illegible] DATE: 11-3-55



CASE I No drains
Water trapped behind wall
Pervious fill allows drainage in front of wall

Pervious fill submerged
pv = 78 ph = 0.43 × 78 = 35
w = 62.5 water = 62.5
140.5# pw + ph = 97.5

Moist pervious fill front of wall pv = 130

2'-0" surcharge for L.L. on fill

Bridge reaction over 12'-0"
$= \frac{2 \times 76.500}{12} = 12750\#$

Wall	1.5 × 150# × 11.0	=	2480#	× 10.92	=	27,100'#
	3.5 × 150# × 10.48	=	5500#	× 11.92	=	65,600
Base	21.0 × 150# × 2.5	=	7850	× 10.5	=	82500
Earth	7.33 × 130# × 5.8	=	5550	× 17.3	=	96,000
	7.33 × 130 × ½ × 2.9	=	1380	× 16.1	=	22,300
	10.17 × 140.5# × 21.48	=	30700	× 5.08	=	156,200
Bridge		=	12750	× 12.67	=	161,500
		ΣM =	66210#			611,200'#

Active H	195# × 23.98	=	4680	× 11.99	= 56,200
	2340# × ½ × 23.98	=	28000	× 9.00	= 252,000
			32680		308,200
					919400'#
Passive H	(8.6)² × ½ × 59#	=	-2200	× 3.2	= -7,000
		ΣH =	30,480#		ΣM = 912,400'#

$\frac{\Sigma M}{\Sigma V} = \frac{912400}{66210} = 13.8$ e = 13.8 − 10.5 = 3.3' toward toe

Brg. $\frac{66210}{21} \pm \frac{66210 \times 3.3 \times 6}{(21)^2}$

$\frac{\Sigma H}{\Sigma V} = \frac{30480}{66210} = 0.46$ = 3160 ± 2960 = 6120 p.s.f. at toe, 200 p.s.f. at heel

ARMY-NED-BOSTON

NED FORM 223
27 Sept 49

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

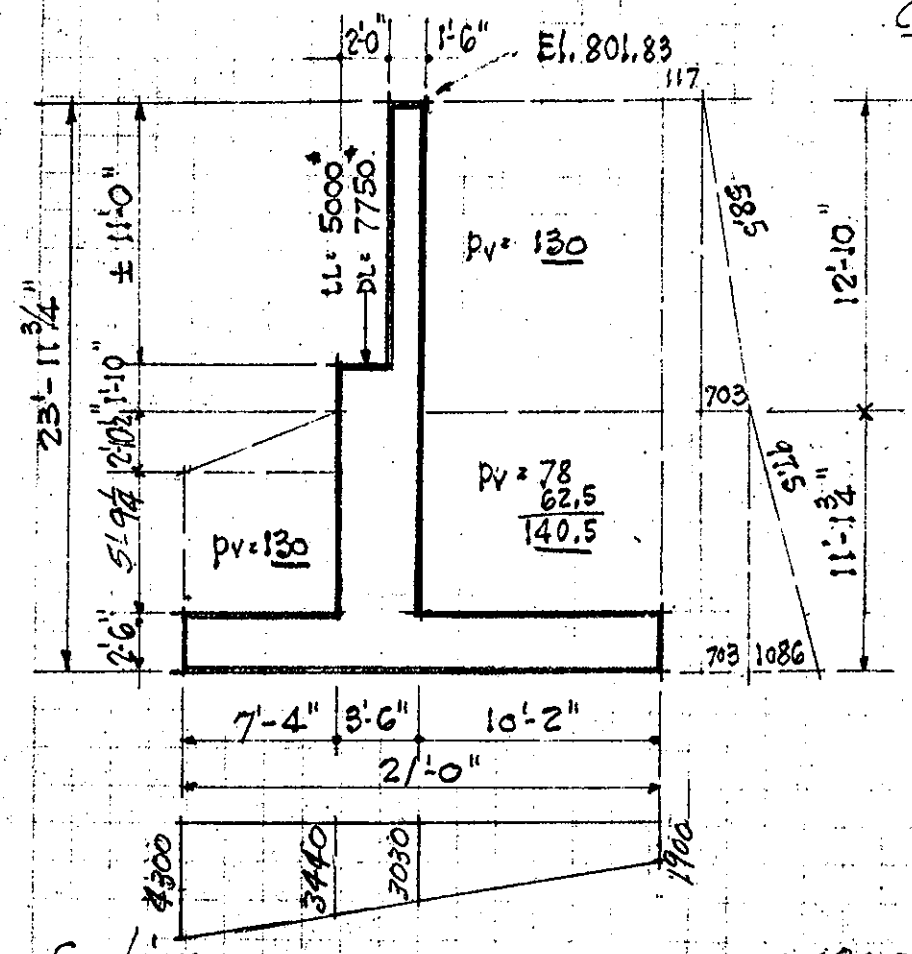
SUBJECT: Otter Brook Dam
COMPUTATION: Service Bridge Abutment
COMPUTED BY: EPR CHECKED BY: GM DATE: 11-3-55

CASE II Drains thru wall

Pervious Fill
Moist $p_v = 130^{\#}$
$p_h = 0.45 \times 130 = 58.5$

Submerged
$p_v = 140.5$
$p_h = 97.5$

2'-0" surcharge for L.L. on fill



Section		= 15830	—	175,200
Earth	$7.33 \times 130^{\#} \times 5.8$	= 5550	× 17.3 =	96,000
	$7.33 \times 130^{\#} \times 1/2 \times 2.9$	= 1380	× 16.1 =	22,300
	$10.17 \times 140.5^{\#} \times 8.64$	= 12400	× 5.08 =	63,000
	$10.17 \times 130 \times 12.83$	= 17,000	× 5.08 =	86,500
Bridge		= 12750	× 12.67 =	161,500
	ΣV	64910#		604,500'#

Active H	$117^{\#} \times 23.98$	= 2810#	× 11.99 =	33,700'#
	$703^{\#} \times 1/2 \times 12.83$	= 4520	× 16.0 =	72,000
	$703^{\#} \times 11.14$	= 7850	× 5.57 =	43,800
	$1086^{\#} \times 1/2 \times 11.14$	= 6050	× 4.2	25,500
		21230#		175,000
Passive H	$(8.6)^2 \times 1/2 \times 59^{\#}$	= −2200	× 3.2 =	− 7,000
	ΣH	= 19030#	ΣM =	772,500'#

$\frac{\Sigma M}{\Sigma V} = \frac{772500}{64,910} = 11.8'$ $e = 11.8 - 10.5 = 1.3'$ toward toe

ARMY-NED-BOSTON

SUBJECT Otter Brook Dam
COMPUTATION Service Bridge Abutment
COMPUTED BY EPR CHECKED BY PJM DATE 11-3-55

Case II (cont'd)

$$\text{Bearg} = \frac{64910}{21}(1 \pm 6 \times 1.3)$$

4300 p.s.f. at toe
&
1900 p.s.f. at heel

Stem above bridge seat

	V	M
$195 \times 11.0 =$	$2145^{\#} \times 5.5 =$	$11.800'^{\#}$
$\pi^2 \times 1/2 \times 97.5 =$	$5900 \times 4.1 =$	24200
	$8045^{\#}$	$36,000'^{\#}$

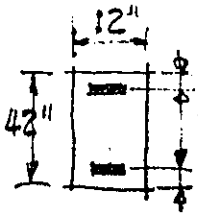
$t = 18''$ $d = 14''$ $M/bd^2 = 36030/14^2 = 169$
Inc. to 20" $As = 12 \times 36/18 \times 7/8 \times 15.5 = 1.75\ \square''$

$V = 8045/12 \times 1 \times 15.5 = 49\ \#/\square'' < 90$

below bridge seat

		M
$195^{\#} \times 21.48 =$	$4200 \times 10.74 =$	$45,000$
$(21.48)^2 \times 1/2 \times 97.5^{\#} =$	$22,500 \times 8.1 =$	183000
	$26,700^{\#}$	$228,000'^{\#}$
$(7.37)^2 \times 1/2 \times 59^{\#} =$	$-1,600 \times 2.8 =$	-4500
$H =$	$25,100^{\#}$	$223,500'^{\#}$

Bridge Wall		
	$12750^{\#} \times 0.75 =$	9600
	$2480 \times -1.0 =$	-2480
	5500	0
$N =$	$20730^{\#}$	$M = 7,120$
		$230,620'^{\#}$



$As' = 0.61\ \square''$ $n = 10$ $A = (12 \times 42) + 9(0.61 + 4.68) = 504 + 47.6 = 551.6\ \square''$

$As = 4.68\ \square''$ $\frac{12x^2}{2} + (9 \times 0.61)(x - 3.5) = (10 \times 4.68)(38 - x)$

$x^2 + 8.72x - 300 = 0$

$x = \frac{-8.72 \pm \sqrt{76 + 1200}}{2} = \frac{-8.72 \pm 35.72}{2} = 13.5''$

$Ic = 12 \times 13.5^3 \times 1/3 = 9850$
$Is' = 5.5 \times 10^2 = 550$
$Is = 46.8 \times 24.5^2 = 28000$
$38,400\ in^4$

SUBJECT Otter Brook Dam
COMPUTATION Service Bridge Abutment
COMPUTED BY EPR CHECKED BY RSM DATE 11-3-55

Stem below bridge seat (cont'd)

$$f_c = \frac{20730}{552} + \frac{12 \times 230{,}620 \times 13.5}{38400} = 38 + 972 = 1010 \text{ \#/□''}$$

$$f_s' = 9\left[38 + \left(972 \times \frac{10}{13.5}\right)\right] = 9\left[38 + 720\right] = 6800 \text{ \#/□''}$$

$$f_s = 10\left[38 - \left(972 \times \frac{24.5}{13.5}\right)\right] = 10\left[38 - 1738\right] = 17000 \text{ \#/□''}$$

Use #11 Vert. 4" i.f. of wall
#7 Vert. 12" o.f. of wall

#6 Horiz. 12" o.c. e.f.

Base Toe

	V		M
7.33 × 4050# =	29700#	× 3.67 =	109000'#
7.33 × ½ × 2070 =	7600	× 4.9 =	27200
	37300#		136200
less 2.5 × 150# × 7.33 =	-2750	× 3.7 =	-10200
	-5550	× 3.7 =	-20500
	-1380	× 2.5 =	-3500
	(-8680)		(-34200)
Σ	28620	M =	102000

t = 34" d = 29.5" $M/bd^2 = 102000/29.5^2 = 117 < 169$

As = 12 × 102 / 18 × j × 29.5 = 2.6 □''

u = 28620 / 13.29 × j × 29.5 = 83 #/□''

#11 @ 4" (As = 4.68)
Bottom

v = 28620 / 12 × j × 29.5 = 90 #/□''

Heel

	V		M
	30,700#	× 5.08 =	156,000'#
10.17 × 150# × 2.5 =	3,820	× 5.08 =	19,400
	34,520		175,400'#
10.17 × 200 =	2020	× 5.08 =	-10400
10.17 × ½ × 2860 =	14500	× 3.38 =	-49000
	-16520		-59400'#
	18,000	M =	116,000'#

As = 12 × 116 / 18 × j × 30.5 = 2.9 □'' #11 @ 6" Top (4.68)

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NEW ENGLAND DIVISION
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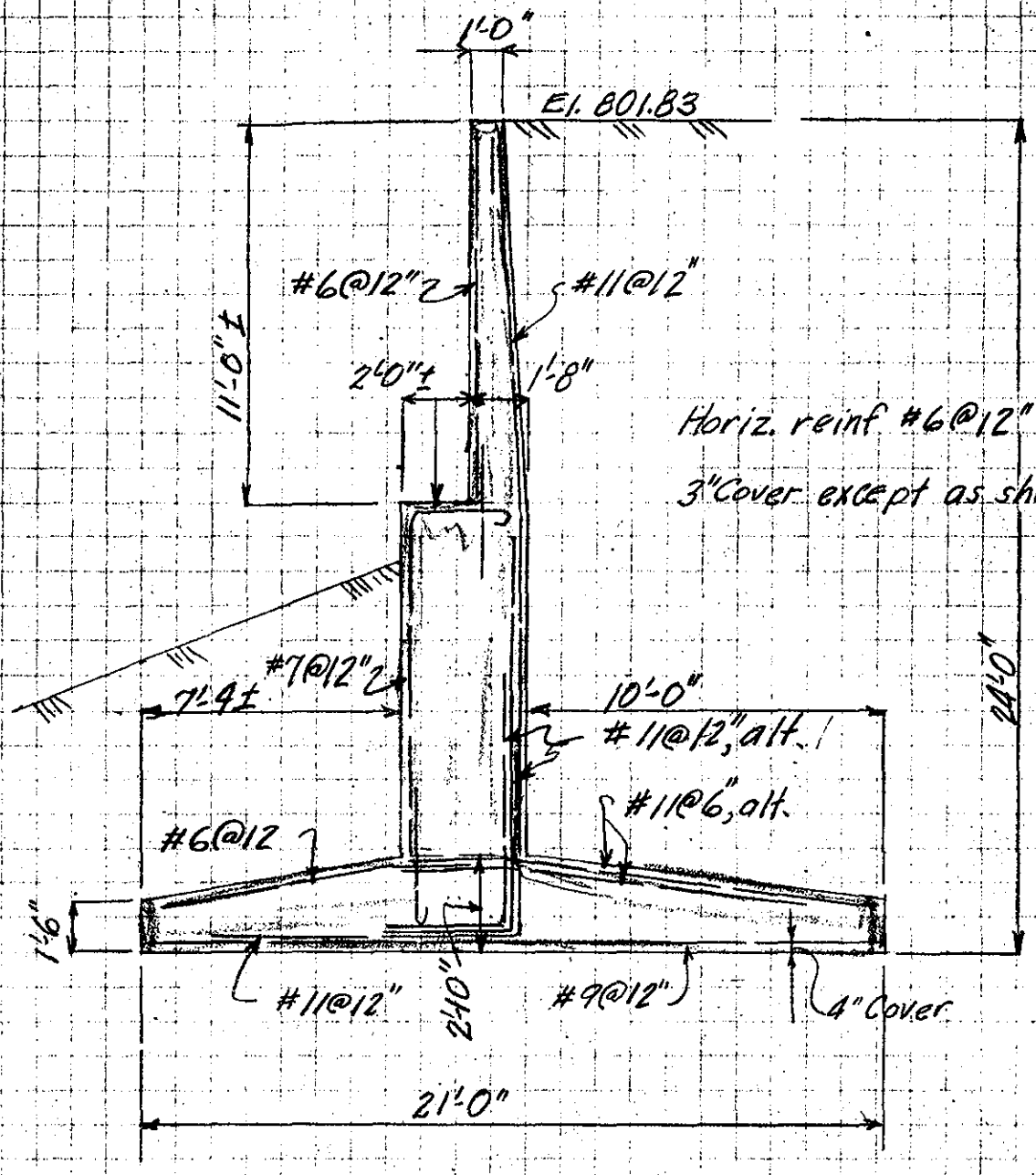
SUBJECT Otter Brook Dam
COMPUTATION Service Bridge Abutment
COMPUTED BY RFM CHECKED BY DATE 11/17/55



SUBJECT Otter Brook Dam
COMPUTATION Service Bridge Abutment
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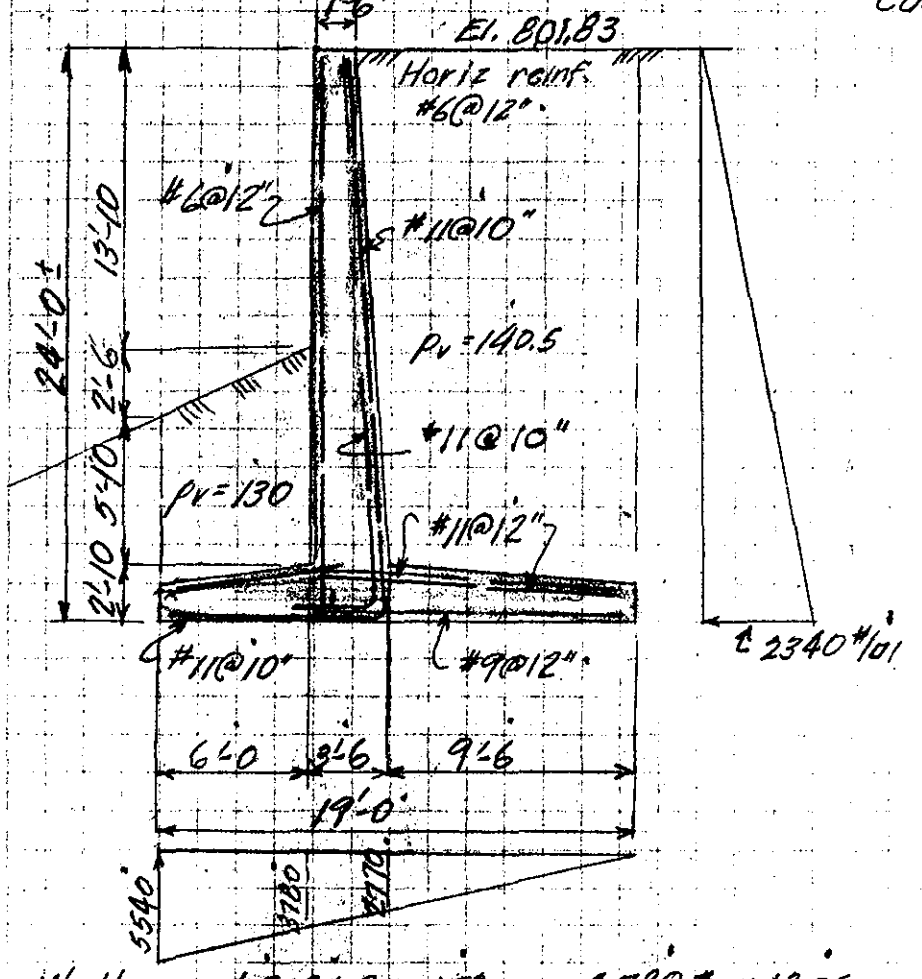
Wing walls - max. section

Case II



Wall	1.5×21.2×150 =	4780#	×12.25	58,500'#
	2×21.2×150/2 =	3180	×10.8	34,300
	2.8×19×150 =	8000	×9.5	76,000
Earth	19.5×21.2×140.5 =	28,200	×4.8	135,500
	2×21.2×140.5/2 =	2970	×10.2	30,300
	6×5.8×130	4520	16.0	72,300
	6×2.5×130/2	980	15.0	14,700
←H	2340×24/2	(28050)	9.0	252,500
→H	8.6²×59/2	(-2200)	3.2	− 7,000
		ΣV 52,630		ΣM 667,100

ΣM/ΣV = 667100/52630 = 12.7 e = 12.7 − 9.5 = 3.2

$p = \frac{52630}{19}\left(1 \pm \frac{6 \times 3.2}{19}\right) = 5540$ #/lf to 0

ΣH/ΣV = 25850/52630 = .49 < .65 OK

SUBJECT Otter Brook Dam
COMPUTATION Service Bridge Abutment
COMPUTED BY RPM CHECKED BY EPR DATE 11/18/55

Wingwalls

Stem

$M = (21.2^2 \times 97.5/2) = 21{,}900^{\#} \times 8.0 = 175{,}500^{\prime\#}$

$d = \sqrt{\frac{175500}{169}} = 32.3''$ reqd 38.5 supplied

$A_s = 175.5 \times 12/18 \times .88 \times 38.5 = 3.43$ - Use #11@5" = 3.7 □"

$v = 21900/12 \times .88 \times 38.5 = 54$ #/□"

Base - Toe

8.3 × .13 =	↓	1.08 K	5.8 × .13	↓	.75 K
2.8 × .15 =	↓	.42	2.8 × .15	↓	.42
	↑	3.78		↑	5.54
		2.28 K/□'		↑	4.37 K/□'

2.28 × 6 =	13.68 K	× 3	41 'K
2.09 × 6/2 =	6.27	× 4	25
	V = 19.95 K		M = 66 'K

$d = \sqrt{\frac{66000}{169}} = 19.5''$ reqd - 29.5" supplied

$v = 19950/12 \times .88 \times 29.5 = 64$ #/□"

$A_s = 66 \times 12/18 \times .88 \times 29.5 = 1.68$ □" - Use #11@10" = 1.85 □"

Base - Heel

.1405 × 21.2	↓	2.98	} 3.4 K/□' at toe
.15 × 2.8	↓	.42	
	↑	2.77	
	↓	.63 K/□' at stem	

.63 × 9.5 =	6.0 K	4.8	28.8 'K
2.77 × 9.5/2 =	13.2	6.3	84.2
	V = 19.2 K		M = 113.0 'K

$d = \sqrt{\frac{113000}{169}} = 26''$ reqd. - 30.5" supplied

$A_s = 113.0 \times 12/18 \times .88 \times 30.5 = 2.8$ □" - Use #11@6

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

SUBJECT OTTER BROOK DAM
COMPUTATION Gate Loading 4'-6"x2'-6" Gates
COMPUTED BY J. Perry CHECKED BY F.J.G DATE 7 Feb 56

Gate Opening 4'-6" x 2'-6"

Max. Design High Water Elevation	797.1
Elev. at Gate Sill	683.0
Hydrostatic Head at Gate Sill	114.1
Hydrostatic Head at Center of pressure	112.6
Total static load on Gate 4.5 x 2.5 x 62.5 x 112.6	79,400#
Friction load 60%	47,600#
Hydrostatic load on Hoist	47,600#
Weight of Gate (estimated)	3000#
" " Piston + Stem (estimated)	600#
Friction load (estimated)	800#
HOIST LOAD	52000#
25% OVERLOAD	13000
TOTAL HOIST LOAD	65000#

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

SUBJECT OTTER BROOK DAM.
COMPUTATION HYDRAULIC CYLINDER
COMPUTED BY F.J.G. CHECKED BY L. Perry DATE 7 FEB 56

ASSUMED CYLINDER DIA.	18" ID
CYLINDER AREA	255 SQ IN
HOIST LOAD	65000 LBS
LOAD PER SQ IN OF CYLINDER 65000/255	255 LBS
CYLINDER PISTON STROKE	4.5 FT.
CYLINDER VOLUME 255 x 54"	13770 CU IN
CYLINDER VOLUME PER FT OF STROKE	3060 CU. IN
CYLINDER VOLUME (GALS) PER FT OF STROKE	14 GALS.
RATE OF GATE LIFT (ASSUMED)	1 FT/MIN
PUMPING RATE	14 GPM.

SUBJECT OTTER BROOK DAM.

COMPUTATION GATE & CYLINDER.

COMPUTED BY F.J.G. CHECKED BY L. Perry DATE 7 FEB 56

DESIGN WORKING PRESSURE	255 PSI.
TEST PRESSURE	500 PSI.
HOOP STRESS $= f = \frac{Pr}{t} = \frac{500 \cdot 8}{1}$ (FOR 1" THICK WALL)	4000 PSI
UNSUPPORTED LENGTH OF STEM	6 FT.
GATE STEM DIA	3 IN.
RADIUS GYRATION $= r = \frac{DIA}{4} =$	.75
$l/r = 6 \times 12/.75$	96
$f_{ALLOWABLE} = 17000 - 0.485 \times \frac{L^2}{r^2} = 17000 - 0.485 \times \frac{5200}{.563}$	12,500#/□"
COLUMN LOADING ALLOWABLE $= 12,500 \times 3.14 \times \frac{3^2}{4}$	88,000#