THAMES RIVER FLOOD CONTROL

WEST THOMPSON

DAM & RESERVOIR

QUINEBAUG RIVER, CONNECTICUT

DESIGN MEMORANDUM NO. 8

DETAILED DESIGN OF STRUCTURES

U.S. Army Engineer Division, New England
Corps of Engineers Waltham, Mass.

DECEMBER 1962

17

U. S. ARMY ENGINEER DIVISION, NEW ENGLAND
CORPS OF ENGINEERS
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ADDRESS REPLY TO:
DIVISION ENGINEER

REFER TO FILE NO.

NEDGW

7 December 1962

SUBJECT: West Thompson Dam and Reservoir, Quinebaug River, Thames River Basin, Connecticut, Design Memorandum No. 8 - Detailed Design of Structures

TO: Chief of Engineers
ATTN: ENGCW-EZ
Department of the Army
Washington 25, D.C.

In accordance with EM 1110-2-1150, there is submitted Design Memorandum No. 8 - Detailed Design of Structures, for the West Thompson Dam and Reservoir, Quinebaug River, Thames River Basin, Connecticut.

FOR THE DIVISION ENGINEER:

John Wm Leslie

JOHN WM. LESLIE
Chief, Engineering Division

Incl (10 cys)
Design Memo No. 8
Detailed Design of Structures

FLOOD CONTROL PROJECT

WEST THOMPSON DAM

QUINEBAUG RIVER

THAMES RIVER BASIN
CONNECTICUT

DESIGN MEMORANDA INDEX

Design Memo No.	Title	Submission Date	Approved
1	Hydrology and Hydraulic Analysis		
	Preliminary	10 May 1962*	22 Jun 1962
	Final	20 Aug 1962	27 Sep 1962
2	Geology		
3	Real Estate		
4	Relocations	31 Oct 1962	
5	General Design	24 Aug 1962	28 Sep 1962
6	Embankment		
7	Concrete Materials		
8	Detailed Design of Structures	7 Dec 1962	
9A	Reservoir Management (Preliminary)		
9B	Reservoir Management (Final)		

* Initial submission in draft to secure approval of spillway design flood and top of dam.

WEST THOMPSON DAM AND RESERVOIR
QUINEBAUG RIVER
THAMES RIVER BASIN
CONNECTICUT

DESIGN MEMORANDUM NO. 8

DETAILED DESIGN OF STRUCTURES

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

FLOOD CONTROL PROJECT

WEST THOMPSON DAM AND RESERVOIR

QUINEBAUG RIVER

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DESIGN MEMORANDUM NO. 8

DETAILED DESIGN OF STRUCTURES

DECEMBER 1962

A. INTRODUCTION

1. Purpose. - The purpose of this memorandum is to facilitate the review by higher authority of the structural, mechanical, and electrical designs of the various features of the West Thompson Dam and Reservoir. The basic criteria, typical design computations and data pertinent to the design are presented herein.

2. Scope. - This memorandum covers the following structures:

Spillway weir and retaining walls; intake structure and cut-and-cover conduit; operating house; gates and machinery, and electrical service.

3. Previous Reports and Revisions. - Detailed reports on the structural, mechanical and electrical design for the spillway, outlet works and miscellaneous structures have not been previously submitted. The latest previous description of the proposed structures and improvements is contained in the recommended project plan in Design Memorandum No. 2 - General Design Memorandum, submitted 24 August 1962 and approved 28 September 1962 and as amended in 2nd Indorsement to OCE, dated 26 October 1962.

B. HYDROLOGY

4. General. - Design Memorandum No. 1 - Hydrology and Hydraulic Analysis includes detailed hydrologic data and hydraulic requirements for the spillway and outlet structures. A summary of the hydrological criteria is given below.

5. Reservoir Storage Capacity. - West Thompson Reservoir will contain 25,600 acre-feet of flood control storage, equivalent to 6.5 inches of runoff from the net drainage area of 74 square miles. In addition, a small permanent pool of 1,200 acre-feet, equivalent to 0.3 inches of runoff, will be provided for recreational uses.

6. Spillway Design Flood. - The composite spillway design flood inflow hydrograph to West Thompson Reservoir has a peak discharge of 85,000 c.f.s., of which 82,000 c.f.s. is produced by the 74-square mile local area below Westville Dam. The spillway design flood was routed through the surcharge storage above the 320 foot L-shaped side channel spillway assuming the outlet works operable and the gates fully open at time of maximum surcharge. The flood routing resulted in a maximum pool elevation of 356.5 feet, m.s.l., a surcharge depth of 14 feet and peak outflow of 63,000, c.f.s. The outlet discharge at maximum pool stage would be about 5,000 c.f.s.

7. Top of Dam Elevation. - The top of dam elevation was determined from the following data: elevation of spillway crest, 342.5 feet, m.s.l., maximum spillway surcharge 14.0 feet; and minimum freeboard 5.0 feet. The selected top of dam elevation is 361.5 feet, m.s.l.

8. Outlet Works. -

a. Flood Control. - The channel capacity of the Quinebaug River through the City of Putnam, Connecticut, the principal damage center about two miles downstream of West Thompson dam site, is estimated to be 7,500 c.f.s. During regulation periods, the total flow at Putnam will consist of uncontrolled runoff from the local drainage area supplemented whenever appropriate by regulated outflow from the five upstream reservoirs. The normal maximum regulated release from West Thompson Reservoir will range between 3,000 and 4,000 c.f.s. depending upon the residual local runoff at downstream index points. A 12-foot diameter, horseshoe-shaped conduit with three 5' x 9' gates was found to be adequate to maintain a flow of about 3,500 c.f.s. with a pool at an elevation that represents 20 percent of the flood control storage. The maximum discharge will be about 5,100 c.f.s. with the pool at spillway crest.

b. Recreation. - A concrete control weir with stop-logs will be constructed upstream of the center flood control gate which will automatically maintain the level of the recreation pool. The combined length of the five stop-log openings is 30 feet which will provide adequate discharge capacity for normal summertime flow. The stop-logs will permit minor adjustment of the permanent recreation pool level.

9. Diversion. - The construction schedule requires that the Quinebaug River be diverted through the flood control conduit during the second construction season. A 10-year frequency flood was selected to determine the required height of the cofferdam. The flood hydrograph was routed through the reservoir storage and resulted in a maximum pool stage of 310 feet m.s.l., and maximum discharge of 2,600 c.f.s. Elevation 312, m.s.l., has been selected for the top of the cofferdam.

C. HYDRAULIC DESIGN

10. General. - The hydraulic design of the spillway and outlet works is discussed in detail in Design Memorandum No. 1 - Hydrology and Hydraulic Analysis. Data pertinent to the design of the spillway and outlet works are given in the following paragraphs.

11. Spillway. - The thickness of overburden and location of the rock formation in the right abutment together with the natural topography at the dam site led to the selection of an L-shaped, side channel spillway. Economic studies led to the adoption of a 320-foot spillway crest length and the corresponding maximum surcharge of 14 feet. The shape of the ogee crest was designed for a surcharge of 11 feet which is 79 percent of the expected maximum head. The crest of the weir conforms to conventional parabolic curves, terminating in a tangent having a slope of 3 on 2 at elevation 326.75 feet, m.s.l.

12. Spillway Discharge Channel. - A 60-foot channel width was selected so as to fully utilize the limited hydraulic head available and to minimize the cost of the highway bridge. The spillway channel will have an invert slope of 8.3 percent through the overflow section, 3.9 percent for 180 feet below the spillway and 1.0 percent for the remaining 645 feet to where the flow returns to the river. The relatively high tailwater and rock topography obviate the need for a stilling basin.

13. Outlet Works. -

a. Intake. - The walls of the approach channel will terminate in elliptical side curves conforming to the equation $\frac{X^2}{56.25} + \frac{Y^2}{6.25} = 1$, and the intake roof will be shaped to conform to the equation $\frac{X^2}{182.25} + \frac{Y^2}{20.25} = 1$.

b. Gates and Transition. - A 12-foot diameter horseshoe-shaped conduit with three 5' x 9' sluice gates will be provided for flood regulation. A 42-foot long transition will be provided between the three gates which have a total cross section area of 135 and the conduit which has an area of 119.2 square feet. The invert of the intake and transition sections will be level at elevation 292.0 feet, m.s.l. The sidewalls of the transition will converge about 1 in 6.5 and the roof line will rise on a slope of 1 in 14.

c. Air Vent. - With the reservoir pool at spillway crest, elevation 342.5 feet, m.s.l., the maximum air demand per gate will be about 165 c.f.s. Assuming a maximum velocity of 150 feet per second, the air vent will require a cross sectional area of 1.1 square feet per gate. The air vents to the individual gates will be 15 inches in diameter, shaped to prevent cavitation and will be combined in a common riser 27 inches in diameter, providing about 3.3 square feet. Entrance to the air vent will be in the freeboard range above maximum surcharge (elevation 356.5 feet, m.s.l.) The junction of the individual gate air vents with the common riser will be at approximately elevation 307 feet, m.s.l., in order to prevent any backflow of water from one gate passage to another.

d. Hydrostatic Pressure in Gate Passages. - Pressure gradients in the gate passages have been computed for use in the structural design of the walls and slabs. Since the structural criteria permits an increase of 33-1/3 per cent in the normal working stresses when the reservoir is at maximum surcharge level, it was found that the governing design condition occurs when the reservoir is at maximum surcharge with the increased working stresses being used. Pressure gradients with conduit discharge are based on the reservoir at spillway crest elevation. Critical points on the pressure gradients are noted below:

(1) Upstream of a closed gate, the hydrostatic pressure is equal to the static pool elevation with reservoir at maximum surcharge.

(2) Downstream of a closed gate, the hydrostatic head is equal to the invert elevation of the gate passage.

(3) In an open gate passage, the pressure gradient is dependent upon the velocity which varies with the cross sectional area of the passage for the appropriate number of open gates.

The following table shows the elevation of the pressure gradient for various conditions of discharge in the gate passages:

Elevation of Pressure Gradients

	Upstream of Gates		Downstream of Gates	
Number of Gates Open	Open Passage	Closed Passage	Open Passage	Closed Passage
0	-	356.5	-	292.0
1	300.7	356.5	300.3	292.0
2	301.3	356.5	300.9	292.0
3	314.9	-	314.7	-

e. Stilling Basin. - There are no provisions for a stilling basin as geological investigation has determined that bedrock is capable of withstanding maximum discharge velocities.

D. GEOLOGY

14. General. - A detailed description of geologic conditions at the site and a record of foundation exploration will be presented in Design Memorandum No. 2 - Geology. Geologic conditions will also be discussed in Design Memorandum No. 6 - Embankment.

15. Bedrock Structure. - The bedrock is largely granitized schist and gneiss, generally hard with a highly distorted foliation dip. The trend of the structural foliation is nearly parallel to the spillway alignment generally N 50° to 60° W with a westerly dip of from 30° to 50°. Local variations of this structural orientation occur in areas of intense granitic intrusion. The major joint system as measured at limited bedrock exposures on the right abutment is N 15° to 20°W with an easterly dip from 45° to 55°.

16. Foundation Conditions. - Explorations made in the vicinity of the concrete structures on the right abutment indicate a highly variable condition of weathering. The design of all footings have been established at depths generally below the zone of most intense jointing and weathering. Intersecting joint planes and structural foliation will govern to a large degree the shape of structural rock excavations. Line drilling will be effective in reducing overbreak where necessary to minimize concrete

quantities or control the shape of rock cuts such as for the bridge piers and outlet works. In areas of open cut excavation it is considered practical to excavate the rock to slopes of 4 on 1 after removal of loose and blocky surface rock. Jointing and associated weathering will necessitate the removal of 5 to 10 feet of rock to obtain a sound footing in portions of the spillway weir. In some sections excavations will necessarily be made beyond design slopes where prominent joints or planes of foliation are coincident with and behind the design slopes.

High water losses during drilling and extreme difficulty in installing pressure-testing equipment coupled with highly erratic pressure-testing results, indicates the necessity for grouting to reduce seepage beneath structures. A grout curtain should be established to a depth of 0.6 of the maximum hydrostatic head. It is estimated that properly spaced and oriented relief drains in rock could reduce hydrostatic pressures by about 40 percent. Where concrete structures on rock require the investigation of shearing resistance to horizontal movement a coefficient of friction of 0.6 is recommended. In the area of the side channel spillway where anchoring is required to stabilize structures, the anchors should be inclined to the east at an angle of about 50° to the horizontal. The dry unit weight of the rock for design use if 170 pounds per cubic foot.

E. CONCRETE

17. Concrete Materials. - Concrete materials will be covered in detail in Design Memorandum No. 7 - Concrete Materials.

F. STRUCTURAL DESIGN

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

19. Scope. - The structural design including stability investigations of the spillway weir, lining, intake structure, conduit, service and access bridges and access bridge abutments are included herein.

20. Design Criteria. -

a. General. - All working stresses conform to those specified in the Engineering Manual EM 1110-1-2101, "Working Stresses for Structural Design", dated 6 January 1958. Loading conditions, design assumptions and other design criteria are based on the following applicable parts, in the 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 (EM 1110-2-2200, 25 September 58); Structural Design of Spillways and Outlet Works (Part CXXIV, October 1956); Retaining Walls (EM 1110-2-2502, 29 May 1961) and Design of Miscellaneous Structures (Part CXXIX, Chapter 2, Conduits, Culverts and Pipes, June 1948). Accepted engineering practice has been employed in cases where the Engineering Manual for Civil Works does not apply.

b. Concrete. - The following table lists the concrete and reinforced concrete stresses used in the design of structures.

Flexure	Lbs. per sq. in.
Extreme fiber stresses in compression (except conduit and bridges)	1,050
Extreme fiber stresses in compression (conduit)	1,350
Extreme fiber stresses in compression (bridges)	1,200
Extreme fiber stresses in tension (plain concrete)	60
Shear - (v)	
Beams - no web reinforcement	90
Beams with properly designed web reinforcement	240
Footings - at critical section	75
Tower Sections at points of contraflexure (in accordance with the recommendations for Box Culverts - by Univ. of Illinois)	

Bond - (u) Deformed bars	Lbs. per sq. in.
Top bars	210
All others	300

Bearing - (fc)	
Load on entire area	750
Load on one-third area or less-maximum permissable	1,125

Modular Ratio - (n)

c. Reinforcement. -

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

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

(3) Minimum Cover for the Main Reinforcement. -

Structure	Minimum Cover (Inches)
Lower portion of gate tower	4
Middle portion of gate tower and bridge abutments	3
Bridge decks	1-1/2
Tower superstructure and stairways	1-1/2
Interior floor slabs	3/4
Interior girders and beams	1-1/2
Conduit	4

The concrete covering of stirrups, spacer rods and similar secondary reinforcement may be reduced by the diameter of such rods.

(4) Splices. - All splices will be lapped 24 diameters to develop by bond, the total working strength of the bars. Splices in the main reinforcement at points of maximum moment have been avoided in the design.

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

d. Structural Steel. - Structural steel is designed for the working stresses of ordinary bridge and building steel (yield point 33,000 p.s.i. minimum) which conforms to the specifications for the Design, Fabrication and Erection of Structural Steel for Buildings, issued by the American Institute of Steel Construction. Basic working stress is 20,000 p.s.i. except for bridge steel which is 18,000 p.s.i.

e. Increase in Normal Working Stresses. - Allowable working stresses are increased 33-1/3% where wind load or earthquake forces govern.

21. Basic Data and Assumptions. -

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

Top of dam	361.5
Spillway crest	342.5
Maximum water surface just upstream at spillway weir	356.5
Elevation of conservation weir	303.0

b. Loads. -

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

MATERIAL	UNIT WEIGHT (lbs/cu. ft.)			
	Dry	Saturated	Moist	Submerged
Rockfill	120	-	-	78

MATERIAL	UNIT WEIGHT (lbs/cu. ft.)			
	Dry	Saturated	Moist	Submerged
Compact Pervious Fill & Gravel Bedding	135	147	142	85
Compact Impervious Fill	130	145	140	83
Concrete (Plain & 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.
Intake Tower Operating Room floor slabs	500 lbs. per sq. ft.
Intake Tower Operating Room floor beams	150 lbs. per sq. ft. plus gate leaf load

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 available. Specific uplift assumptions for each structure are described as each structure is treated separately.

d. Earth Pressure. - Earth pressures used against the gate tower, intake structure and bridge abutment have been determined in general in accordance with EM 1110-2-2502, 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 methods outlined in Part CXXIX, Chapter 2, Conduits, Culverts and Pipes, using Condition II.

e. Earthquake Forces. - Structures where earthquake is a factor such as the gate tower have been investigated for an earthquake shock of 0.05g. Stability investigations of the gate tower considered earthquake forces applied with the pool at spillway crest elevation by Savages virtual mass method.

f. Ice Pressure. - Horizontal forces due to the expansion of ice have been considered in the design of the conservation pool weir and a force of 6,000 # per linear foot has been applied to the top of the weir.

g. Wind Pressure. - A wind pressure of 30 pounds per square foot has been used in the design of the gate tower. Wind load for the service bridge and access bridge will be figured at 50 lbs. per sq. ft., over the projected area.

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

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

j. Location of Resultant. - In the design of structures, the resultant of the horizontal and vertical loads has in general been within the middle third except on earthquake loading where it is 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.

l. Allowable Soil Bearing. - In the design of the abutment for the service bridge an allowable soil bearing value of 3 tons per sq. ft. was used.

22. Spillway Weir and Lining. -

a. Description. - The side channel spillway has a low ogee crest 320 feet long. A 1'-0" thick lining anchored to rock will be utilized between the spillway section and the channel bed. Contraction joints will be provided at approximately 25 to 30 foot spacing.

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

Case I. - Reservoir empty, dead load of weir and wind load on downstream face.

Case II. - Reservoir to spillway crest,no tailwater, uplift varying from full headwater at the heel to 0 at the toe.

Case III. - Reservoir at maximum surcharge elevation of 356.5,no tailwater, uplift varying from maximum headwater at heel to 0 at toe.

Case IV. - Same as Case III, except tailwater considered at elevation 349.5, uplift varying from maximum headwater at heel to maximum tailwater at toe.

Case IIIA. - Same as Case III, but considering rock acting with the concrete weir.

Case IVA. - Same as Case IV, but considering rock acting with the concrete weir.

c. Design. - Cases III & IV study the effect of a high and low tailwater elevation. Under Case III & IV loading, the resultant force falls outside the middle half and it became necessary to utilize rock anchors extending 10 feet into rock. Under Cases IIIA & IVA, loading, considering 3.5 feet of rock on the upstream face acting with the weir, the resultant falls at the mid third point. The maximum bearing pressure was found to be 2240 lbs. per sq. ft. under Case I loading.

d. Concrete Lining. - Rock anchors holding the 1'-0" thick lining to the rock face were figured for a head of water to the top of lining reduced by 50% because of drains into the rock. Maximum design net head was 20 feet and required #11 anchors sunk 10 feet into rock. The concrete lining was designed under two way action spanning between the rock anchors.

23. Gate Tower. -

a. Description. - The gate tower has a gate section containing three 5' x 9' hydraulically operated gates. The lower portion of the tower is 21'-6" x 30'-0" inside dimensions. At the operating floor it is 26'-4" x 31'-10" inside dimensions. On the upstream edge of the tower there is a projection containing concrete trash piers and stop log slots. The tower is founded on rock and will be approximately 100 feet high from top of roof to the bottom of the foundation.

b. Stability. - The tower was analyzed for the following conditions of loading:

Case I. - Construction condition with no earth and wind load.

Case II. - Normal operating condition with pool at spillway crest elevation of 342.5 and gates closed.

Case III. - Permanent pool condition with water at elevation 305, gates closed and ice load applied.

Case IV. - Flood discharge condition with pool at maximum surcharge elevation of 356.5 and gates closed.

Case V. - Case I, but with earquake load applied.

Case VI. - Case II, but with earquake load applied.

Maximum bearing pressure of 8100 lbs. per sq. ft. occurs under Case V with earquake east to west. The resultant is within the middle third for all conditions of loading.

c. Superstructure. - The roof is slab and beam construction and will be designed for a 40 lbs. per sq. ft. live load plus a light weight fill for drainage. The operating room floor is also slab and beam construction using a live load of 500 lbs. per sq. ft. as determined from the gate load for the slabs and 150 lbs. per sq. ft. plus gate concentration load for the beams.

d. Tower Section. - A typical section thru the tower at elevation 310.0 was designed using earth hydrostatic water pressure on the outside. Concrete Stairway and platforms inside the tower will be designed for a 100 lbs. per sq. ft. live load.

e. Base Section. - The base portion of the tower will be analyzed as a continuous vertical frame with the top of the walls fixed at the massive concrete in the gate chamber floor.

f. Intake Projection. - The roof of the intake projection was designed for a five foot differential head. The sidewalls and base slab were designed as a continuous frame with 50 percent of the lateral load on the sidewalls being distributed in each direction.

g. Concrete Trash Bars. - The trash bars will be designed for a head of water equal to 50 percent of the head to the platform level.

24. Conservation Weir. - Directly upstream of the intake tower is a small conservation weir which regulates the conservation pool. This structure will be designed as a U section except for the end wall which will be designed lateraly to the side walls. An ice thrust of 6000 lbs. per linear foot will be applied at the level of the weir.

25. Conduit. -

a. Description. - The conduit will be a 12'-0" horseshoe type section approximately 200 feet long. Construction joints will be spaced approximately every 20 feet. The conduit will be totally in rock cut for its full length and a load factor of 1.0 has been used in computing the weight of fill on the roof. As the rock appears to be of good quality, the roof has been computed as an arch with springing points at mid height. The lower half of the conduit has been computed for hydrostatic head only assuming the sidewalls anchored at the springing point of the arch above.

b. Loading. - The arch roof was designed for the following two conditions of loading:

Case I. - Rapid drawdown from spillway crest with 100% of weight of earth saturated to spillway crest and moist above. Horizontal loading taken at 50% of the above.

Case II. - Water at spillway crest elevation with 100% of weight of earth submerged to spillway crest and saturated above. Lateral pressures taken at hydrostatic pressure plus 50% of weight submerged to crest plus 50% of weight saturated above.

26. Conduit Outlet. - The outlet sidewalls and apron will be 1'-0" thick slabs anchored to rock. The design will be similar to the lining at the spillway. Drains drilled into the rock to relieve hydrostatic flow will be considered as reducing the head by 50 percent. There is also a gravity type headwall to retain the fill of the dam. The headwall will be designed in accordance with EM 1110-2-2502, titled Retaining Walls.

27. Service Bridge. - The service bridge is a single 60 foot span bridge supported on the gate tower and an abutment at the top of the dam. The bridge deck of reinforced concrete will be carried on two welded plate girders. The clear distance between curbs is 10 feet and the bridge was designed for H20 loading. Girders are designed for composite action using welded steel shear connectors. The abutment at the top of the dam is a Tee shaped section designed for unbalanced earth loading. Provisions will be made to allow for some vertical and horizontal movement after the abutment and bridge are constructed. The maximum bearing pressure is 4,600lbs. per sq. ft. which is under the allowable value of 5,000 lbs. per sq. ft.

28. Access Bridge. - The access bridge is a single 115.0'span having 5 welded steel girders at 8'-0" c.to c. The deck will be of

reinforced concrete 30'-0" between curbs and 37'-8" out to out of concrete. The bridge has been designed for an H20-S16-44 loading in accordance with the AASHO. Railing for this bridge as well as the service bridge will be of aluminum.

G. MECHANICAL DESIGN

29. General. - Gates, hydraulic operating equipment, crane, and diesel electric standby unit will be housed in the operating tower. General arrangement of the mechanical equipment is shown on Plate No. 8-6.

a. Gates. - The flow through each conduit will be controlled by a 5'-0" x 9'-0" cast-iron hydraulically operated slide-type service gate. The gates are designed to withstand a hydrostatic head of 65 feet. The normal maximum operating pressure of the hydraulic system will be approximately 1000 p.s.i.

b. 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 tank will be installed beneath each pump to act as a reservoir to insure an adequate supply.

c. 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. A shear pin is provided to protect the hanger from damage if the oil pressure is applied without releasing the gate hanger lock.

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

e. Crane. - A five ton bridge crane with hand chain propelled bridge and hand chain propelled trolley, with motor operated hoist will be installed in the operating room. The hoist will be used to service the gates and other equipment and will have sufficient hook travel to reach to the bottom of the service well where it can pick up the gates.

f. Heating. - A warm air furnace located in the operating room will maintain a 50 degree temperature in the room.

g. Diesel Electric Standby Unit. - A diesel driven electric generating unit will be located in the operating room. This unit will furnish emergency service for the hydraulic pump and the lighting system.

H. ELECTRICAL DESIGN

30. Electric Service. - Electric service will be obtained from the Connecticut Light and Power Company. Secondary service for the flood control tower and the utility building will be 120/208 volts, 3 phase, 4 wire, 60 cycles. Service for the operator's quarters will be 120/208 volts, single phase, 3 wire, 60 cycles and will be separately metered.

31. Secondary Distribution System. - The operator's quarters will be served overhead. The flood control tower and the utility building will be served underground from a common transformer bank.

32. Emergency Diesel Engine - Generator. - A non-automatic, battery started, diesel engine-generator unit will be installed in the flood control tower together with a control panel for generator control and a 3 PDT switch to transfer the flood control tower and utility building to either the utility company's service or to the emergency generator.

33. Interior Wiring Systems. - Conductors in the tower and portions of the utility building (heater room and vault), will have heat and moisture resisting type insulation and will be installed in steel conduit. Wiring in the remaining portions of the utility building and in the operator's quarters will be nonmetallic sheathed cable.

34. Communication System. - Ducts or conduits and pull boxes will be installed for use by the local telephone company in providing service to the tower. The local telephone company will provide service to the utility building and operator's quarters.

35. Lightning Protection. - Air terminals will be located at the top of the tower and will be connected to down leads on the exterior of the structure. The down leads will be connected to ground rods placed in undisturbed earth near the base of the tower.

36. Grounding System. - The electrical grounding system for equipment in the tower will be connected to ground rods placed in undistrubed earth near the base of the tower. The grounding system

in the tower, utility building and the operator's quarters will be connected to the water supply service. The neutral of the 120/208v secondary system will be connected to the grounding system of each building.

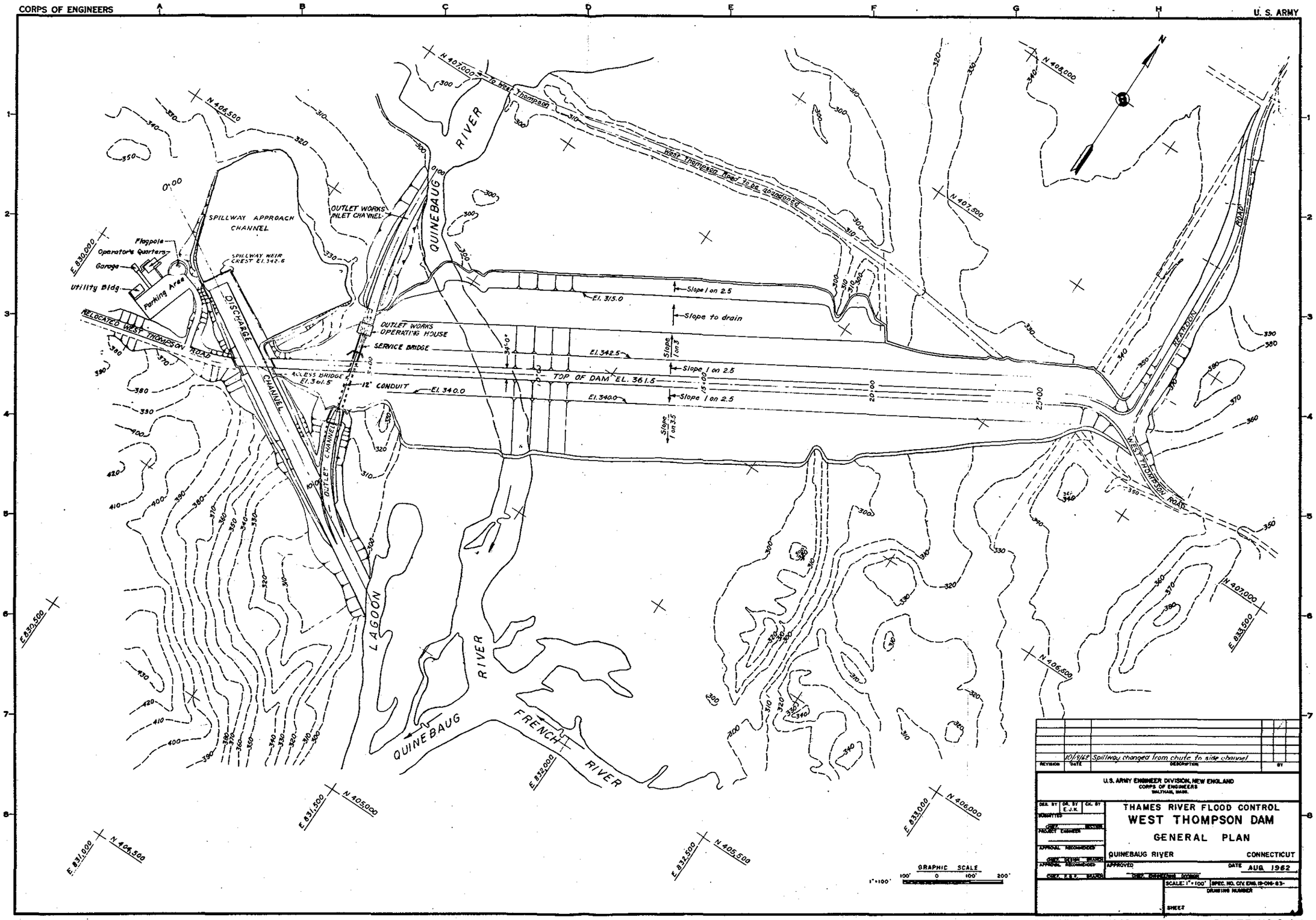
37. Electrical Load Data. -

Flood Control Tower	Connected	Estimated Maximum Demand
2 Gate pump motors @ 10 hp	20 hp	10. kva
1 Crane @ 10 hp	10 hp	-
Lighting, recept. & misc.	6 kva	4 kva
Utility Building	14 kva	8 kva
		22 kva

Since the operator's quarters will not be connected to the emergency generator, its load need not be considered.

The largest motor the emergency generator will be required to start is the 10 hp gate pump motor. Basing the generator size on the nearest standard size to 4 times the kva of the largest motor (4 x 10 = 40 kva), a 37.5 kva, 30 kw generator will be used.

CORPS OF ENGINEERS U. S. ARMY



REVISION	DATE	DESCRIPTION	BY
	10/9/62	Spillway changed from chute to side channel	

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

DES. BY — DR. BY E.J.K. — CK. BY

SUBMITTED — CHIEF, SECTION — PROJECT ENGINEER

APPROVAL RECOMMENDED — CHIEF, DESIGN BRANCH

APPROVAL RECOMMENDED — CHIEF, F.&S. BRANCH

THAMES RIVER FLOOD CONTROL
WEST THOMPSON DAM
GENERAL PLAN

QUINEBAUG RIVER — CONNECTICUT

APPROVED — CHIEF, ENGINEERING DIVISION — DATE AUG. 1962

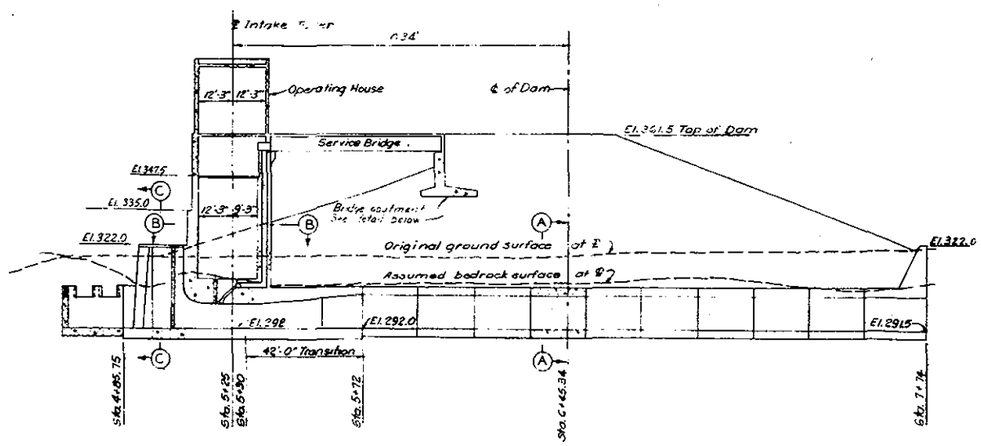
SCALE: 1" = 100' — SPEC. NO. CIV. ENG. 19-ON-63-

DRAWING NUMBER

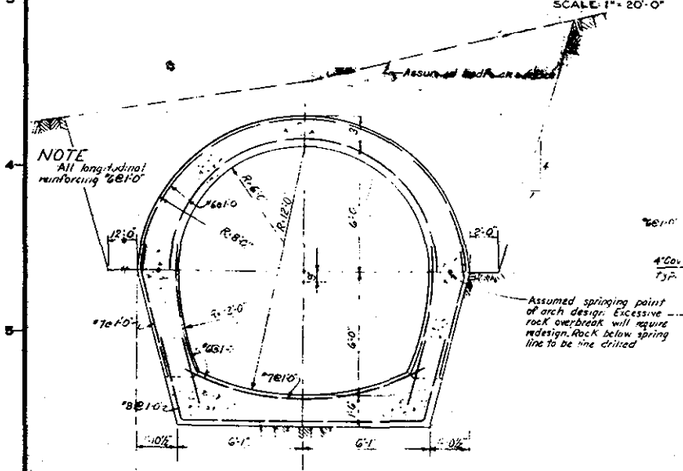
SHEET

REVISED OCT. 1962 PLATE NO. 8-1

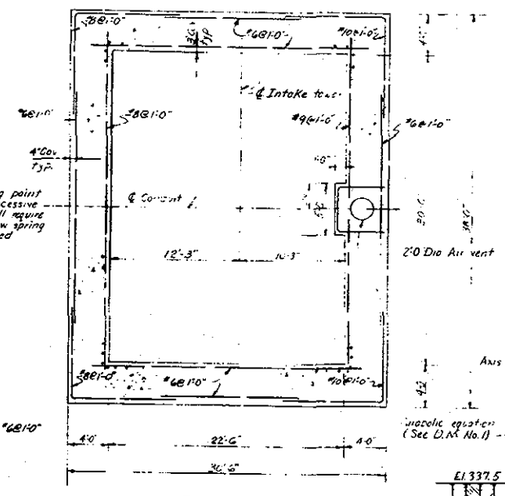
CORPS OF ENGINEERS U. S. ARMY



PROFILE ALONG ℄ OUTLET WORKS
SCALE 1" = 20'-0"



SECTION A-A
SCALE 1/4" = 1'-0"



SECTION B-B
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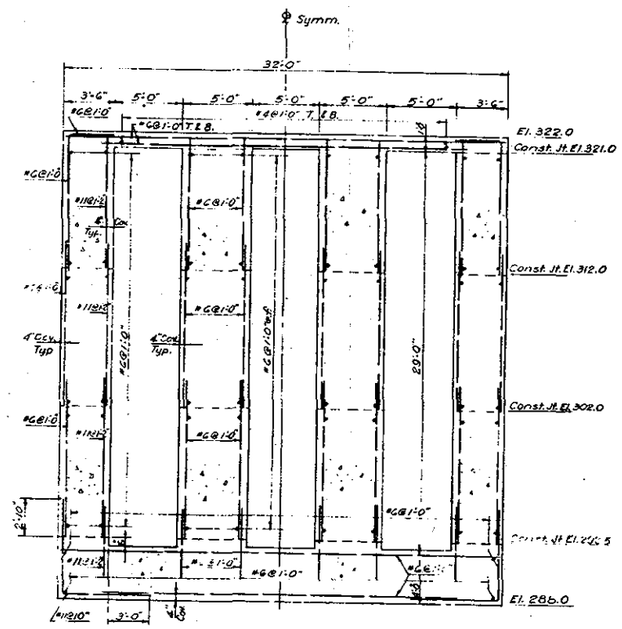
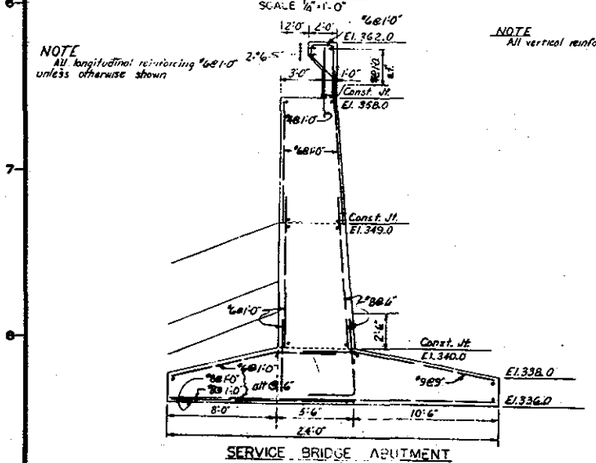
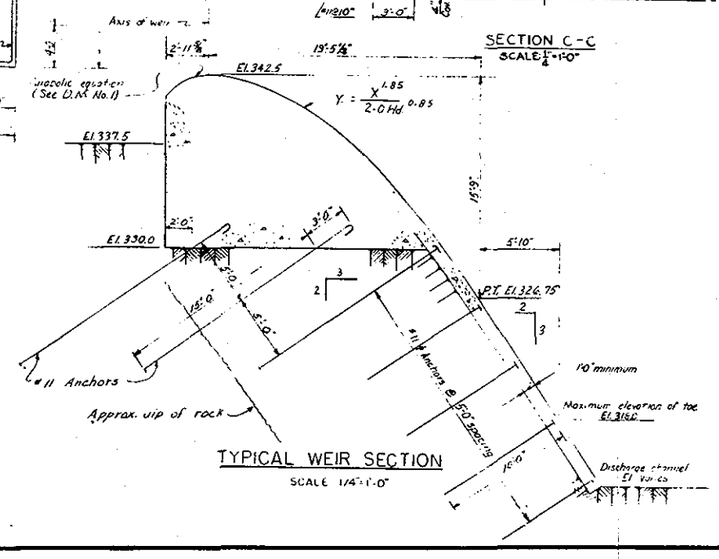


SECTION C-C
SCALE 1/4" = 1'-0"



SERVICE BRIDGE ABUTMENT
SCALE 1/4" = 1'-0"



TYPICAL WEIR SECTION
SCALE 1/4" = 1'-0"

REVISION	DATE	DESCRIPTION	BY

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

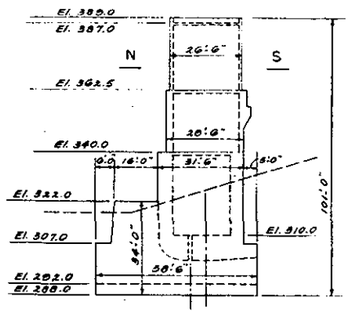
DES. BY C.M.T. | DR. BY A.J.Z. | CK. BY
SUBMITTED
CHIEF, STRUCTURAL BRANCH
PROJECT ENGINEER
APPROVAL RECOMMENDED
CHIEF, DESIGN BRANCH
APPROVAL RECOMMENDED
CHIEF, A.E.D. BRANCH

THAMES RIVER FLOOD CONTROL
WEST THOMPSON DAM
OUTLET WORKS AND SPILLWAY WEIR
STRUCTURAL - PROFILE AND SECTIONS
QUINEBAUG RIVER CONNECTICUT

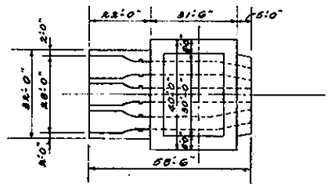
APPROVED DATE
CHIEF, ENGINEERING DIVISION

SCALE | SPEC. NO. CIV. ENG. 19-016-
DRAWING NUMBER
SHEET

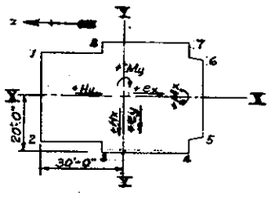
PLATE NO. 8-2



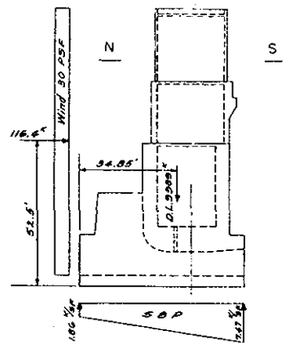
ELEVATION – INTAKE TOWER



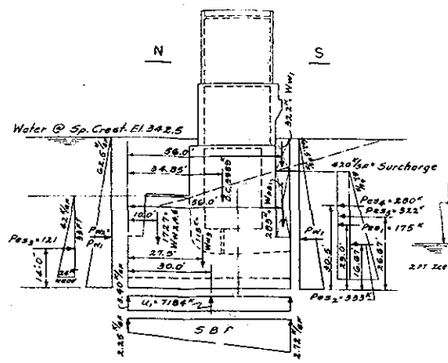
PLAN AT EL. 310



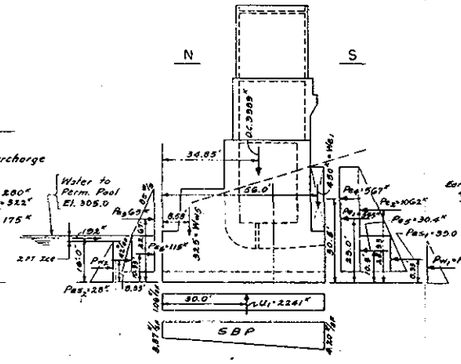
PLAN OF BASE



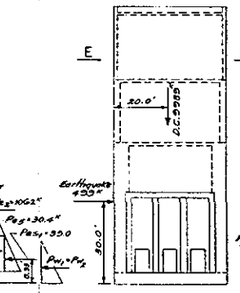
CASE I A

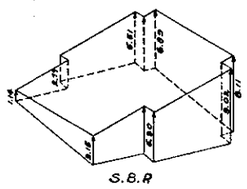


CASE II

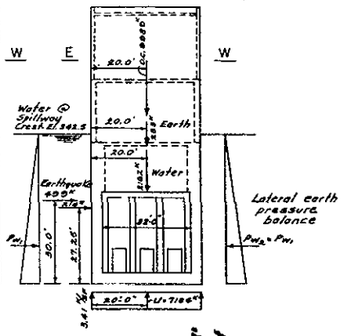


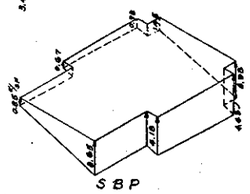
CASE III B



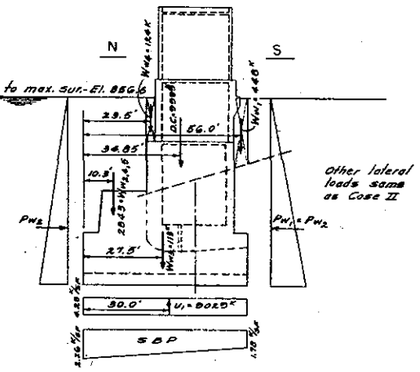


CASE V B





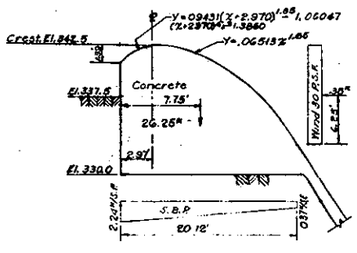
CASE VI B



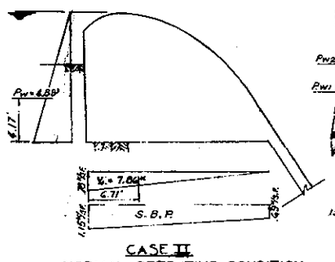
CASE IV

SUMMARY OF BASE PRESSURES:– ALL POINTS ARE COMPRESSION BEARING

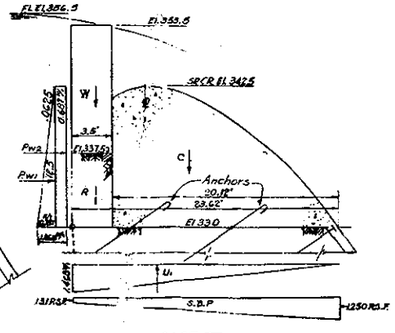
	CASE	ΣV K	HORIZ.K ΣHy	ΣHx	MOM.K ΣMy	ΣMx	e ft. ex	ey	BASE PRESSURES K/SQ.FT. 1	2	3	4	5	6	7	8	UPLIFT KSF
NORMAL STRESS	I A	5989	116	0	5340	0	5.39	0	1.85	1.85	3.97	6.98	7.47	7.47	6.99	3.97	0
	II	5250	989	0	4460	0	0.85	0	2.25	2.25	2.42	2.68	2.72	2.72	2.68	2.42	3.40
	III B	5523	1929	0	3240	0	0.38	0	3.87	3.87	4.00	4.18	4.21	4.21	4.18	4.00	1.06
INCREASED	IV	4252	989	0	4630	0	-1.09	0	2.26	2.26	2.08	1.82	1.78	1.78	1.82	2.08	4.28
	V B	6989	0	499	48440	15000	4.85	1.50	1.14	3.15	5.30	8.02	8.11	6.29	5.51	2.78	0
	VI B	5250	989	713	4460	20800	0.85	3.97	0.85	3.65	4.18	4.43	3.98	1.45	0.93	0.68	3.41



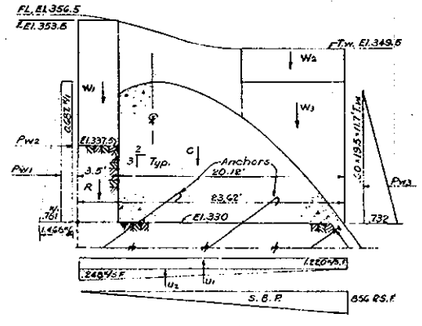
CASE I
CONST. CONDITION



CASE II
NORMAL OPERATING CONDITION
WATER AT SPILLWAY CREST
NO TAILWATER



CASE III A
FLOOD DISCHARGE CONDITION
WITH ROCK ANCHORS
WATER AT EL. 356.5
NO TAILWATER



CASE IV A
FLOOD DISCHARGE CONDITION
WITH TAILWATER TO EL. 349.5
BASE OF CONC. WEIR TO EL. 330.

REVISION	DATE	DESCRIPTION	BY

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

DES. BY C.M.T. | DR. BY C.M.A. | CK. BY

SUBMITTED
CHIEF, ENGINEERING DIVISION
PROJECT ENGINEER

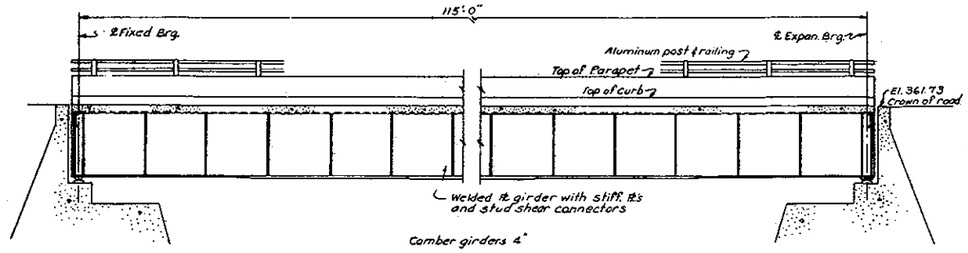
APPROVAL RECOMMENDED
CHIEF, DESIGN BRANCH

APPROVAL RECOMMENDED
CHIEF, F.E.D. BRANCH

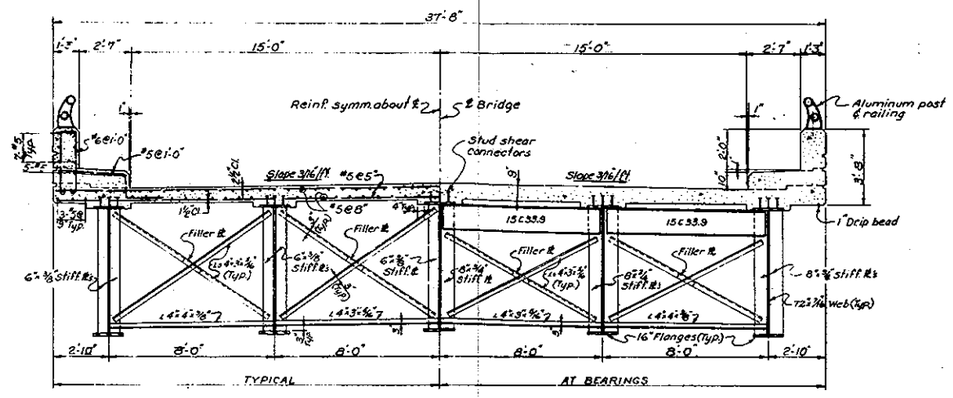
THAMES RIVER FLOOD CONTROL
WEST THOMPSON DAM
OUTLET WORKS AND SPILLWAY WEIR
STABILITY DIAGRAMS
QUINEBAUG RIVER CONNECTICUT

APPROVED DATE
CHIEF, ENGINEERING DIVISION

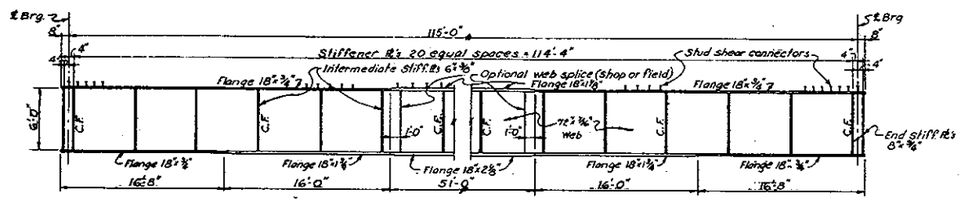
SCALE | SPEC. NO. CIV ENG 19-016-
DRAWING NUMBER
SHEET



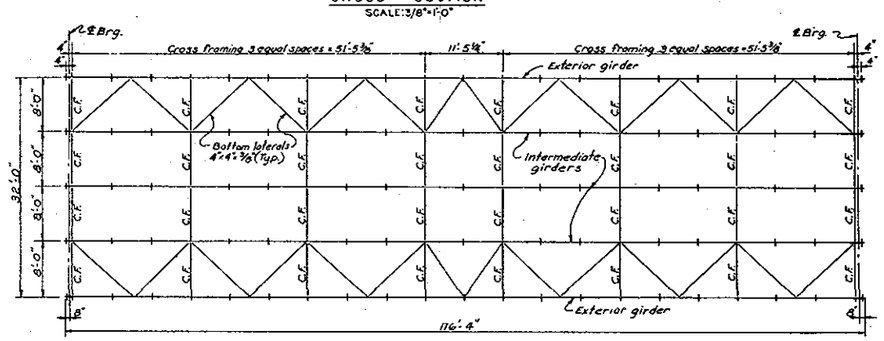
SECTION ALONG ℄ BRIDGE
SCALE: 3/16" = 1'-0"



CROSS SECTION
SCALE: 3/8" = 1'-0"

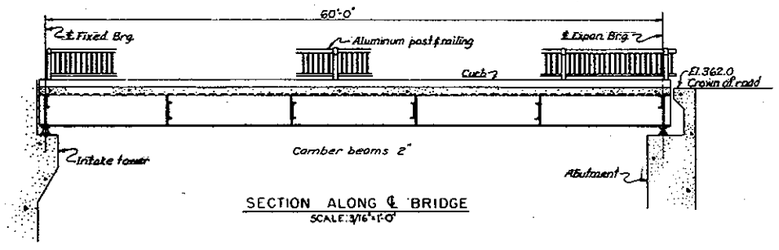


INTERMEDIATE GIRDER ELEVATION
SCALE: 3/16" = 1'-0"

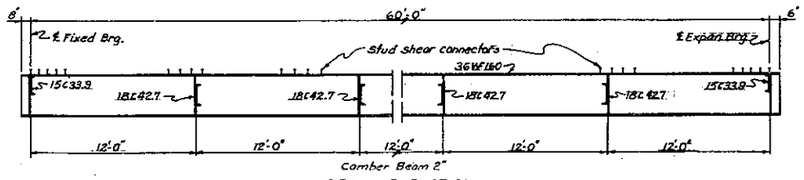


FRAMING PLAN
SCALE: 1/8" = 1'-0"

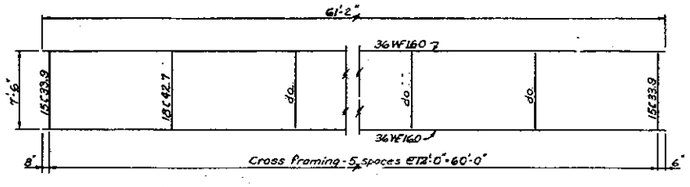
ACCESS BRIDGE DETAILS



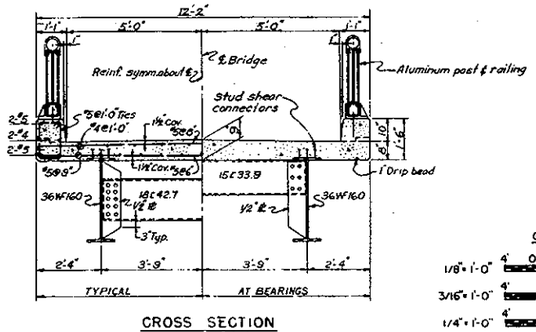
SECTION ALONG ℄ BRIDGE
SCALE: 3/16" = 1'-0"



BEAM ELEVATION
SCALE: 1/4" = 1'-0"



FRAMING PLAN
SCALE: 3/16" = 1'-0"



CROSS SECTION
SCALE: 1/2" = 1'-0"

GRAPHIC SCALES

SERVICE BRIDGE DETAILS

REVISION	DATE	DESCRIPTION	BY

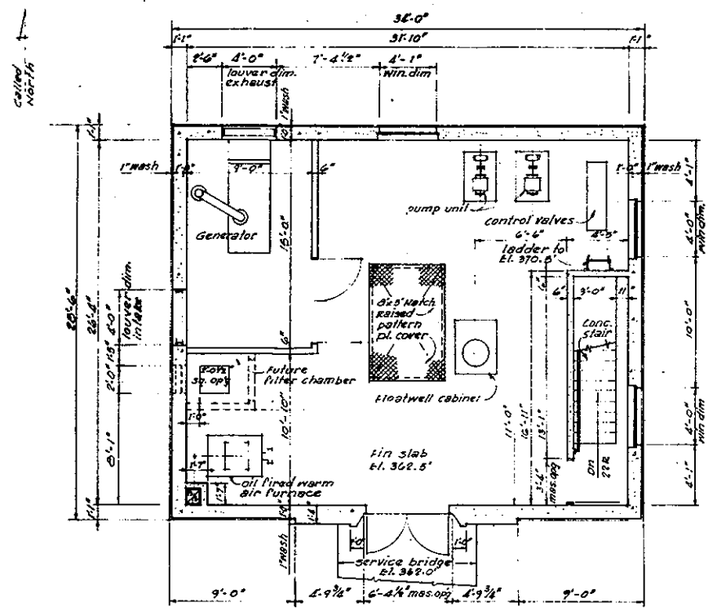
U.S. ARMY ENGINEER DIVISION, NEW ENGLAND
CORPS OF ENGINEERS
WALTHAM, MASS.

THAMES RIVER FLOOD CONTROL
WEST THOMPSON DAM
ACCESS AND SERVICE BRIDGES
STRUCTURAL
PLANS, ELEVATIONS AND SECTIONS
QUINEBAUG RIVER CONNECTICUT

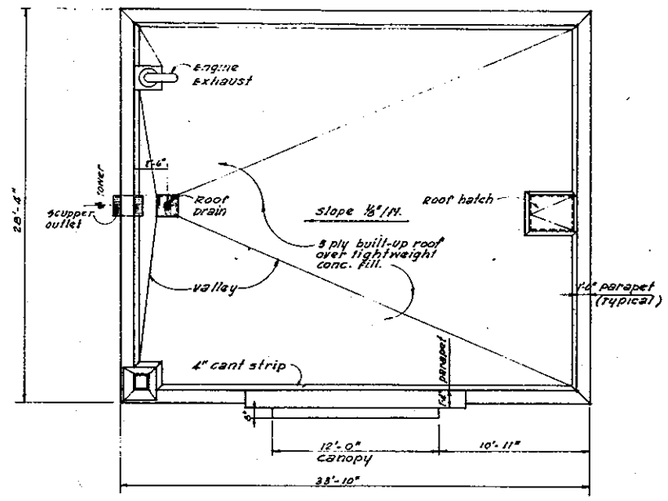
DES. BY J.P.H. DR. BY J.P.H. CK. BY
SUBMITTED
CHIEF, STRUCT. SECTION
PROJECT ENGINEER
APPROVAL RECOMMENDED
CHIEF, DESIGN BRANCH
APPROVAL RECOMMENDED
CHIEF, F.&S. BRANCH
APPROVED DATE
CHIEF, ENGINEERING DIVISION
SCALE SPEC. NO. CIV. ENG. 19-016
DRAWING NUMBER
SHEET

PLATE NO. 8-4

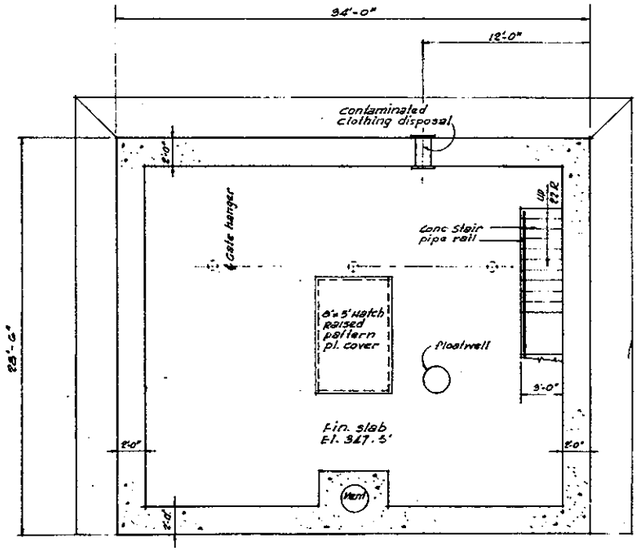
CORPS OF ENGINEERS — U. S. ARMY



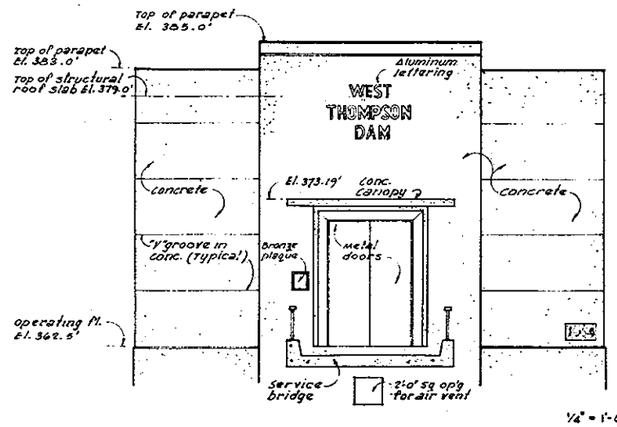
OPERATING ROOM PLAN



ROOF PLAN



BASEMENT PLAN



SOUTH ELEVATION

GRAPHIC SCALE 1/4" = 1'-0"

REVISION	DATE	DESCRIPTION	BY

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

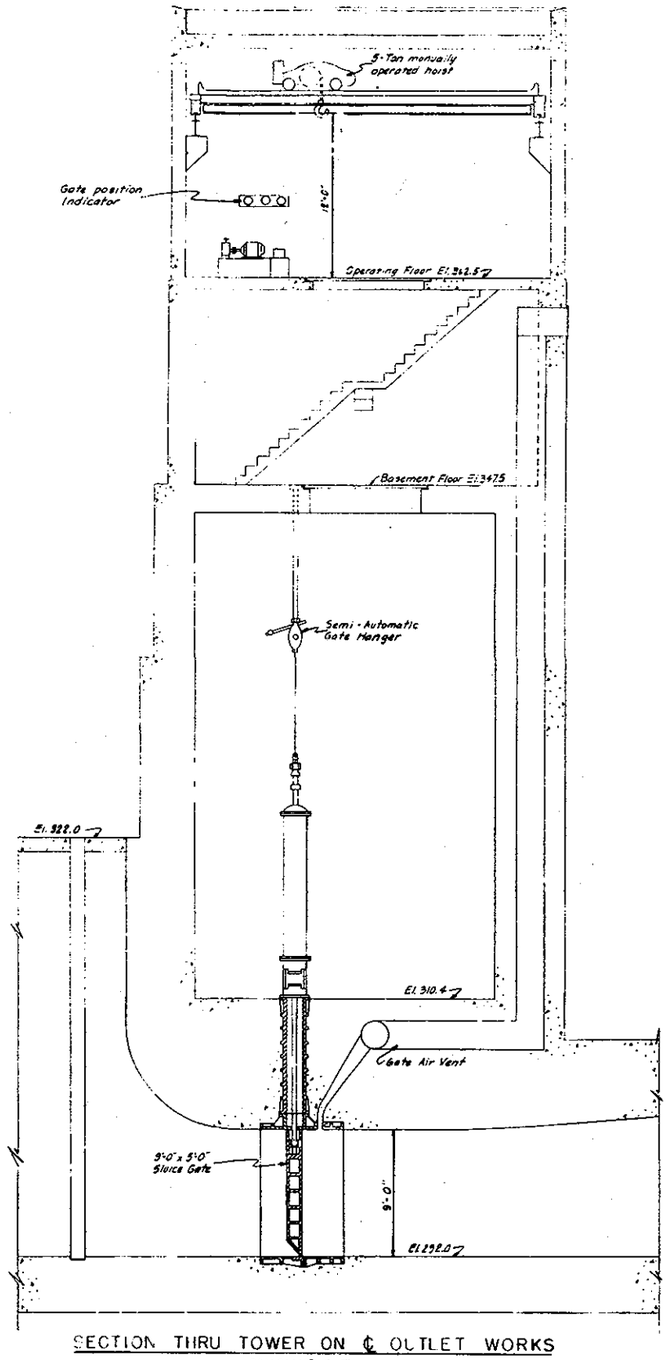
THAMES RIVER FLOOD CONTROL
WEST THOMPSON DAM
OPERATING TOWER
ARCHITECTURAL
PLANS AND ELEVATION
QUINEBAUG RIVER — CONNECTICUT

DES. BY | CH. BY | CK. BY
SUBMITTED
CHIEF, ARCH. SECTION
PROJECT ENGINEER
APPROVAL RECOMMENDED
CHIEF, DESIGN BRANCH
APPROVAL RECOMMENDED
CHIEF, P.E.D. BRANCH
APPROVED — DATE
CHIEF, ENGINEERING DIVISION

SCALE: 1/4" = 1'-0" | SPEC. NO. CIV. ENG. 19-016
DRAWING NUMBER
SHEET 1

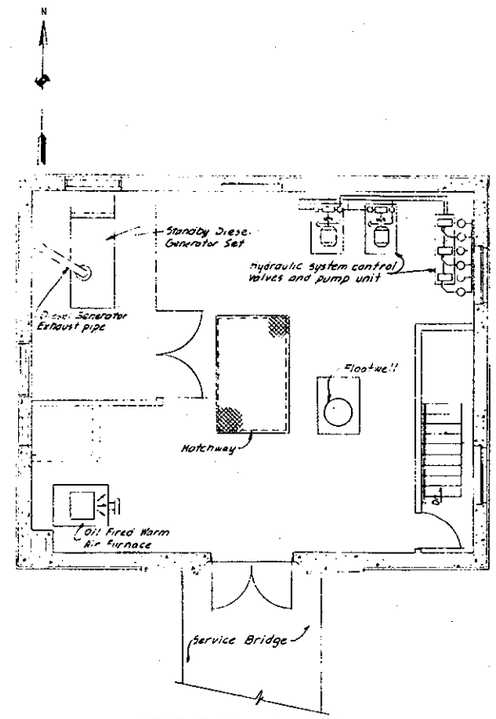
PLATE NO. 8-5

CORPS OF ENGINEERS U. S. ARMY



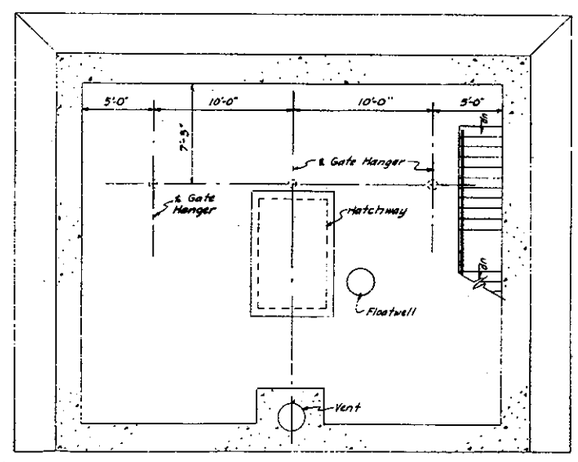
SECTION THRU TOWER ON ℄ OUTLET WORKS

SCALE: 1/4" = 1'-0"



OPERATING ROOM PLAN

SCALE: 1/4" = 1'-0"



BASEMENT PLAN

SCALE: 1/4" = 1'-0"

GRAPHIC SCALE

1/4" = 1'-0" 4' 0 4' 8'

REVISION	DATE	DESCRIPTION	BY

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

DES. BY J.W.F. | DR. BY R.P.C. | CK. BY

SUBMITTED

CHIEF, DESIGN BRANCH

PROJECT ENGINEER

APPROVAL RECOMMENDED

CHIEF, DESIGN BRANCH

APPROVAL RECOMMENDED

CHIEF, R.E.D. BRANCH

THAMES RIVER FLOOD CONTROL
WEST THOMPSON DAM
OPERATING TOWER
MECHANICAL
PLANS AND SECTION
QUINEBAUG RIVER CONNECTICUT

APPROVED DATE

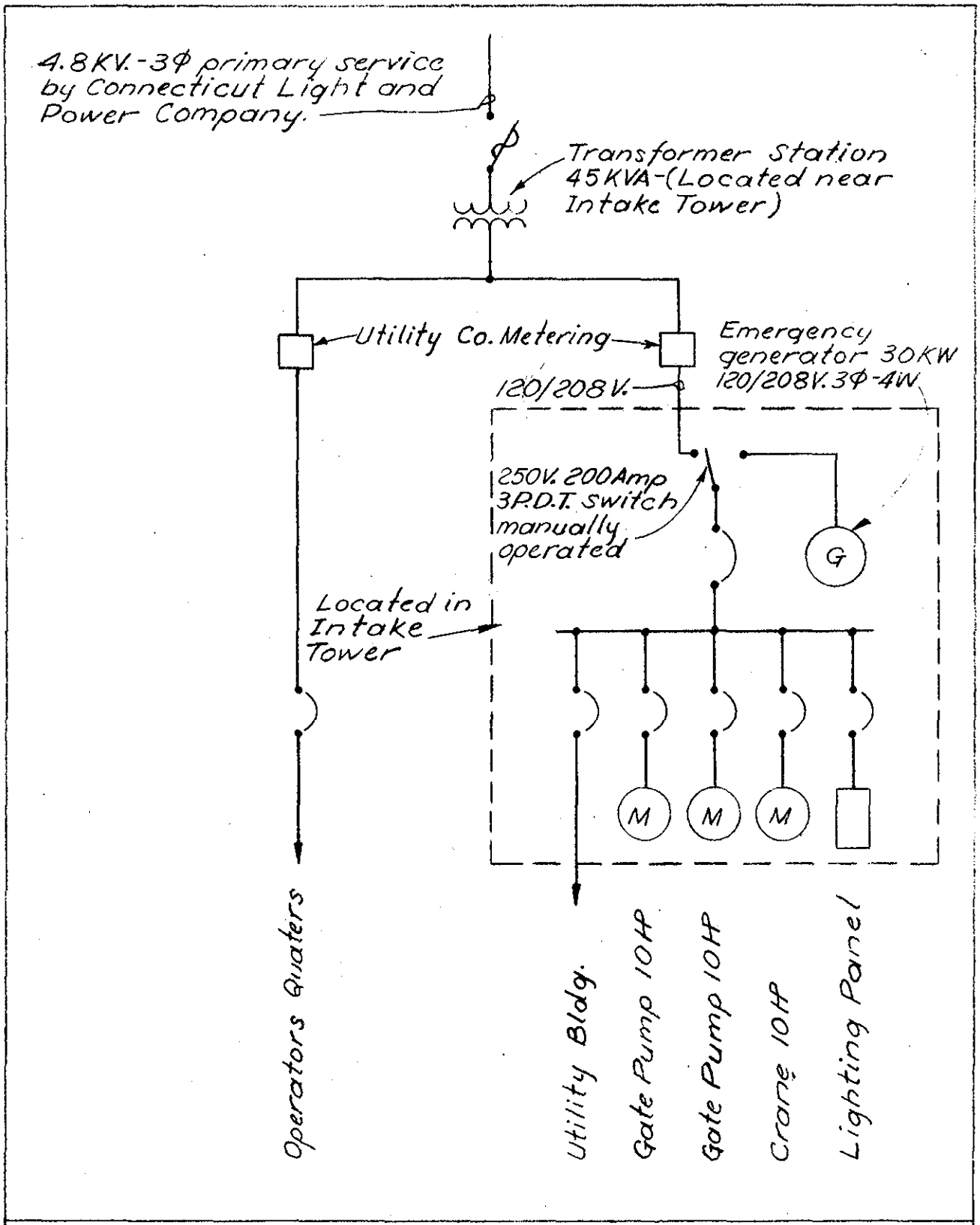
CHIEF, ENGINEERING DIVISION

SCALE: 1/4" = 1'-0" SPEC. NO. CIV. ENG. 19-016-

DRAWING NUMBER

SHEET

PLATE NO. 8-6



THAMES RIVER FLOOD CONTROL

WEST THOMPSON RESERVOIR

ELECTRICAL DISTRIBUTION

ONE LINE DIAGRAM

QUINEBAUG RIVER CONNECTICUT

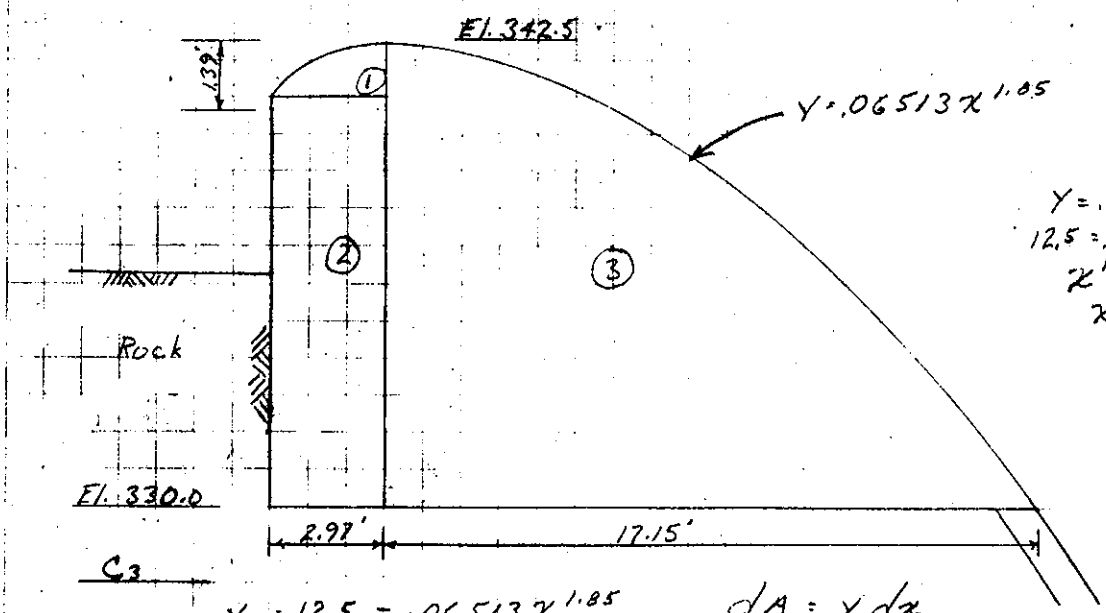
PLATE NO. 8-7

WEST THOMPSON DAM

INDEX FOR STRUCTURAL COMPUTATIONS

	Item	Page
1.	Spillway Weir Stability Analysis	DM 1-DM 6
2.	Spillway Lining	DM 7
3.	Intake Structure, Stability	DM 8-DM 14
4.	Intake Structure, Structural Design	DM 15-DM 27
5.	Conduit, Structural Design	DM 28-DM 37
6.	Service Bridge Design	DM 38-DM 45
7.	Service Bridge Abutment	DM 46-DM 51
8.	Access Bridge	DM 52-DM 63

SUBJECT West Thompson Dam
COMPUTATION Spillway Weir - Stability
COMPUTED BY JWF CHECKED BY CMB DATE



$Y = .06513X^{1.85}$
$12.5 = .06513X^{1.85}$
$X^{1.85} = 191.92$
$X = 17.15'$

$Y = 12.5 - .06513X^{1.85}$ $\quad dA = Y\,dx$

$dA = 12.5\,dx - .06513X^{1.85}\,dx$

$$AREA_3 = \int_0^{17.15} 12.5\,dx - \int_0^{17.15} .06513X^{1.85}\,dx =$$

$$A_3 = \left[12.5X\right]_0^{17.15} - \left[\frac{.06513X^{2.85}}{2.85}\right]_0^{17.15} = 12.5(17.15) - \frac{.06513(17.15)^{2.85}}{2.85} =$$

$A_3 = 214.38 - 75.21 = 139.17\,\square'$

$dM_Y = X\,dA = XY\,dx$

$$M_{Y_3} = \int_0^{17.15} 12.5X\,dx - \int_0^{17.15} .06513X^{2.85}\,dx =$$

$$M_{Y_3} = \left[\frac{12.5X^2}{2}\right]_0^{17.15} - \left[\frac{.06513X^{3.85}}{3.85}\right]_0^{17.15} = \frac{12.5(17.15)^2}{2} - \frac{.06513(17.15)^{3.85}}{3.85} =$$

$M_{Y_3} = 1{,}838.25 - 954.83 = 883.42$

$\bar{X}_3 = M_Y/A = 883.42/139.17 = 6.35'$

$$M_{X_3} = \int \frac{Y^2\,dx}{2} = \int_0^{17.15} (12.5 - .06513X^{1.85})^2\,dx/2$$

$$M_{X_3} = \int_0^{17.15} \left(12.5^2 - 2\times 12.5\times .06513X^{1.85} + .06513^2X^{2(1.85)}\right)\frac{dx}{2}$$

$$M_{X_3} = \int_0^{17.15} \frac{12.5^2\,dx}{2} - \int_0^{17.15} 12.5(.06513)X^{1.85}\,dx + \int_0^{17.15} \frac{.06513^2}{2}X^{3.70}\,dx$$

SUBJECT West Thompson Dam
COMPUTATION Spillway Weir - Stability
COMPUTED BY JWF CHECKED BY Cmt DATE

$$M_{x_3} = \left[\frac{12.5^2}{2}x\right]_0^{17.15} - \left[\frac{12.5(.06513)}{2.85}x^{2.85}\right]_0^{17.15} + \left[\frac{.06513^2\,x^{4.70}}{2\times 4.70}\right]_0^{17.15}$$

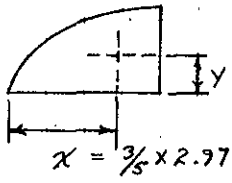
$$M_{x_3} = \frac{12.5^2(17.15)}{2} - \frac{12.5(.06513)(17.15)^{2.85}}{2.85} + \frac{.06513^2(17.15)^{4.70}}{2\times 4.70} =$$

$$M_{x_3} = 1339.8 - 940.2 + 287 = 686.6$$

$$\bar{Y}_3 = \frac{Mx}{A} = \frac{686.6}{139.17} = 4.93'$$

C1

Approx C.G.



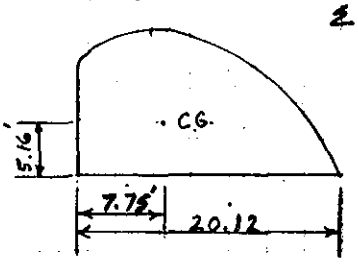
$A_1 = \frac{2}{3}\times 1.39\times 2.97 = 2.75\ ft^2$

Concrete Only

Item	Factors	Weight	H-Arm	H-Mom	V-Arm	Mom
C1	.15 x 2.75	0.42	1.78	0.75	11.63	4.9
C2	.15 x 2.97 x 11.11	4.95	1.49	7.38	5.56	27.6
C3	.15 x 139.17	20.88	9.32	196.-	4.93	102.9
	ΣV =	26.25	ΣMom_H =	203.13	ΣMom_V =	135.4

$$\frac{\Sigma M_H}{\Sigma V} = \frac{203.13}{26.25} = 7.75' = H \qquad \frac{\Sigma M}{\Sigma V} = \frac{135.4}{26.25} = 5.16' = V$$

within mid 1/3



$e = 10.06 - 7.75 = 2.31$

$$f = \frac{26.25}{20.12}\left(1 \pm \frac{6\times 2.31}{20.12}\right)$$

$f_{max} = 1305(1.690) = 2205$ PSF
$f_{min} = 1305(0.310) = 404$ PSF

CASE I - Construction Condition

C		26.25 ↓		+203.13
Wind	0.03 x 12.5 x 1	.38 ←	6.25'	- 2.38
				200.75

$$\frac{\Sigma H}{\Sigma V} = \frac{.38}{26.25} = .015 \text{ ok.}$$

$$\frac{\Sigma M}{\Sigma V} = \frac{200.75}{26.25} = 7.65 > 6.71 < 13.42 \text{ ok}$$

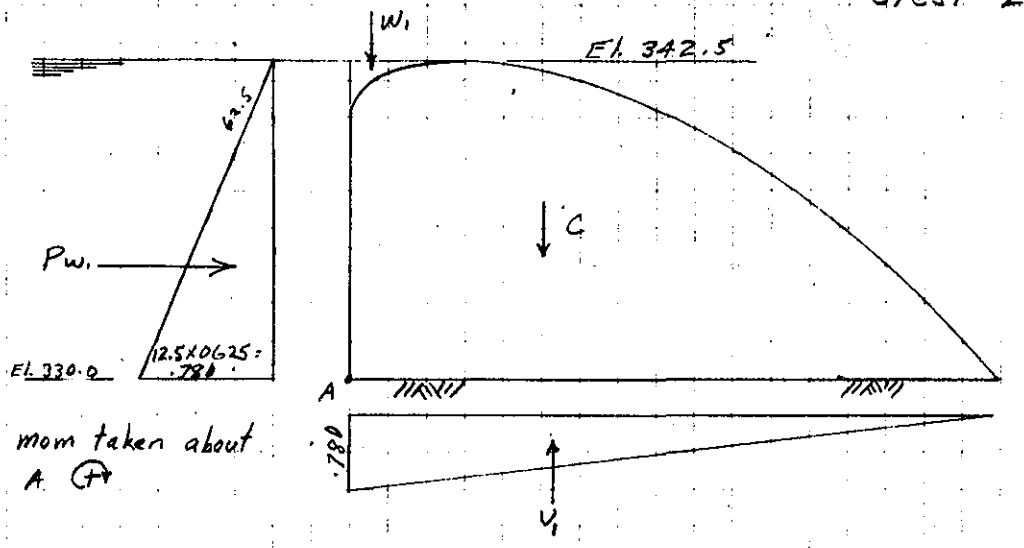
$e = 10.06 - 7.65 = 2.41$ H

$$f = 1305\left(1 \pm \frac{6\times 2.41}{20.12}\right)$$

$f_{max} = 1305 \times 1.720 = 2240$ PSF
$f_{min} = 1305 \times .280 = 365$ PSF

SUBJECT West Thompson Dam
COMPUTATION Spillway Weir - Stability
COMPUTED BY WWF CHECKED BY Cmt DATE

CASE II - Normal Operating Condition - Pool at Spillway Crest Elevation



mom taken about A ⊕

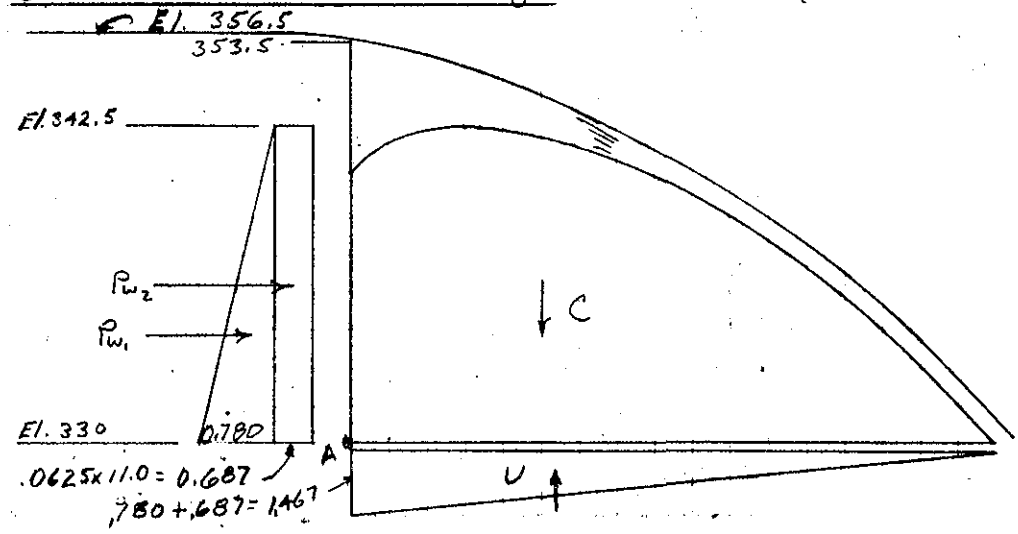
C			26.25 ↓		+ 203.13
W1	.0625 (1.39 x 2.97 − 2.78)		0.08 ↓	7/10 x 2.97 = 2.08	+ .17
V1	.780 x 20.12 x ½		7.86 ↑	20.12/3 = 6.71	− 52.75
		ΣV =	18.47 ↓		
Pw1	0.781 x 12.5 x ½		4.88 →	12.5/3 = 4.17	+ 20.35
		ΣH =	4.88 →	ΣM =	+170.90

$\frac{\Sigma H}{\Sigma V} = \frac{4.88}{18.47} = .26$ OK

$\frac{\Sigma M}{\Sigma V} = \frac{170.90}{18.47} = 9.23 > 6.71 < 13.42$ O.K. $e = 10.06 - 9.23 = 0.83$ ft.

$f = \frac{18.47}{20.12}\left(1 \pm \frac{6 \times 0.83}{20.12}\right) = 0.918\,(1 \pm .248)$ $f_{max} = 1,148$ f_{min} 690

CASE III - Flood Discharge Condition (No Tail water)



SUBJECT West Thompson Dam
COMPUTATION Spillway Weir - Stability
COMPUTED BY JWF CHECKED BY Cmt DATE

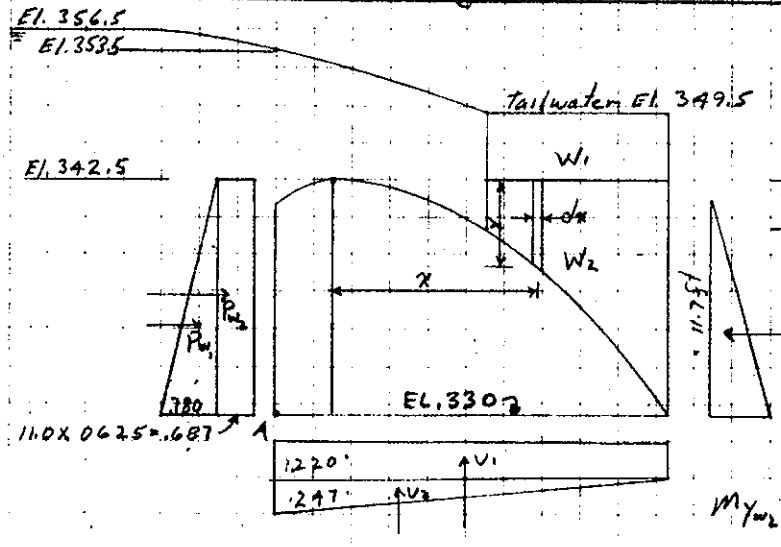
Case III (Cont'd)

C		26.25↓		+203.13
V	1.467 x 20.12 x ½	14.72↑	6.71	- 99.—
	ΣV =	11.53↓		
P_{w1}		4.88 →		+ 20.35
P_{w2}	0.687 x 12.5	8.60 →	6.25	+ 53.75
	ΣH =	13.48	ΣM =	+178.23

$\frac{\Sigma H}{\Sigma V} = \frac{13.48}{11.53} = 1.17$ too high $\quad \frac{\Sigma M}{\Sigma V} = \frac{178.23}{11.53} = 15.40 > 13.4$ N.G.

MUST ADD ROCK w/ ANCHORS SEE CASE III A

Case IV (flood discharge condition with tailwater)



$dA_{w_2} = \int_8^{17.15} y\,dx = \int_8^{17.15} (.06513x^{1.85})\,dx =$

$\left[\frac{.06513x^{2.85}}{2.85}\right]_{8.0}^{17.15} = \left[.0229(17.15)^{2.85}\right] - \left[.0229(8.0)^{2.85}\right] = 75.92 - 8.58 = 66.84$

$dm_{y_{w_2}} = x\,dA = x\,y\,dx$

$M_{y_{w_2}} = \int_8^{17.15} x(.06513x^{1.85})\,dx =$

$\left[\frac{.06513x^{3.85}}{3.85}\right]_8^{17.15} = \frac{.06513(17.15)^{3.85}}{3.85} - \frac{.06513(8)^{3.85}}{3.85}$

$M_{y_{w_2}} = 917 \quad \bar{x} = 917/66.84 = 13.72 + 2.97 = 16.69$ from pt A

C		26.25↓		+203.13
W1	.0625 x 9.15 x 7	4.00↓	15.54	+ 62.16
W2	.0625 x 66.84	4.18↓	16.69	+ 69.76
V1	1.220 x 20.12	24.55↑	10.06	- 246.97
V2	0.247 x 20.12 x ½	2.48↑	6.71	- 16.64
	ΣV =	7.40↓		
P_{w1}	.780 x 12.5 x ½	4.88 →	4.17	+ 20.35
P_{w2}	.687 x 12.5	8.60 →	6.25	+ 53.75
P_{w3}	.732 x 11.7 x ½	4.28 ←	3.90	- 16.69
	ΣH =	9.20 →		+ 128.85

$\Sigma H/\Sigma V = \frac{9.20}{7.40} = 1.24$ too high $\quad \Sigma M/\Sigma V = 128.85/7.40 = 17.41$ outside mid ½ N.G.

MUST ADD ROCK w/ ANCHORS SEE CASE IV A

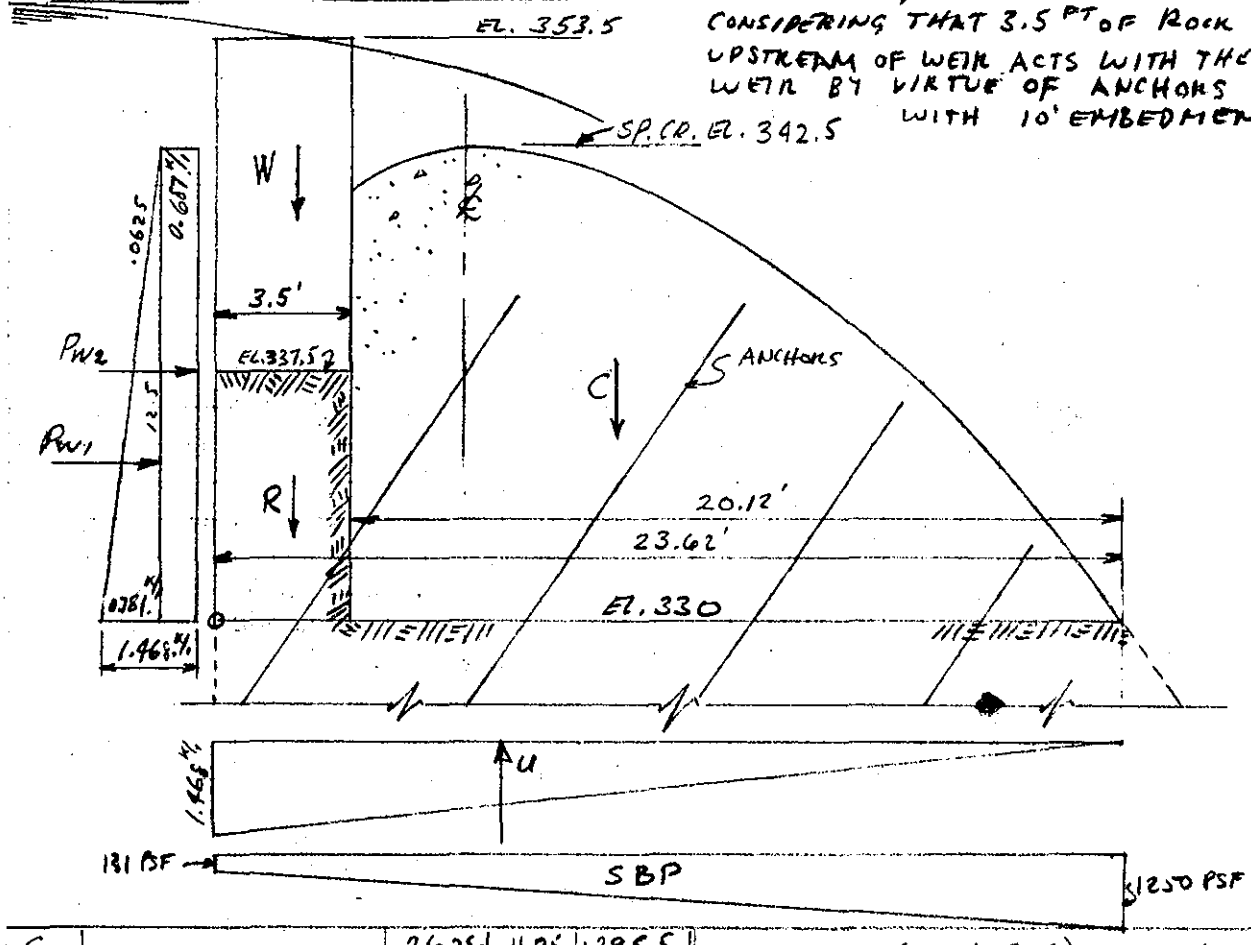
NED FORM 223
27 Sept 49

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

SUBJECT: WEST THOMPSON DAM
COMPUTATION: SPILLWAY WEIR - STABILITY
COMPUTED BY: GMB CHECKED BY: JWF DATE: 11/13/62

CASE III A: — FLOOD DISCHARGE CONDITION WITH ROCK ANCHORS
FL. EL. 356.5 NO TAILWATER, CHECK STABILITY AT ELEV. 330.

CONSIDERING THAT 3.5 FT OF ROCK UPSTREAM OF WEIR ACTS WITH THE WEIR BY VIRTUE OF ANCHORS WITH 10' EMBEDMENT.



C		26.25↓	11.25	+295.5
R	.15 x 7.5 x 3.5	3.94↓	1.75	+ 6.9
W	.0625 x 16 x 3.5	3.50↓	1.75	+ 6.1
U	1.468 x 23.62/2	17.34↑	7.88	- 136.8
	ΣV	16.35↓		
P_{w2}	0.687 x 12.5	8.6→	6.25	+ 53.7
P_{w1}	0.781 x 12.5/2	4.9→	4.17	+ 20.4
	ΣH	13.5→	ΣM	+245.8

$\Sigma H/\Sigma V = \frac{13.5}{16.35} = 0.82$

$\Sigma M/\Sigma V = \frac{245.8}{16.35} = 15.0' > 7.88' < 15.76'$ ok

$e = 15.0 - 23.62/2 = 3.19'$ RIGHT OF ℄ (11.81)

$f = \frac{16.35}{23.62}\left(1 \pm \frac{6 \times 3.19}{23.62}\right) = 0.692(1 \pm .81)$

$f_{MAX} = 692(1.81) = 1250$ #/SF.

$f_{MIN} = 692(0.19) = 131$ #/SF.

$S_{(S-f)} = \frac{.65(16350) + 90(23.62)144}{13500} = 23.5 > 4.0$ ok

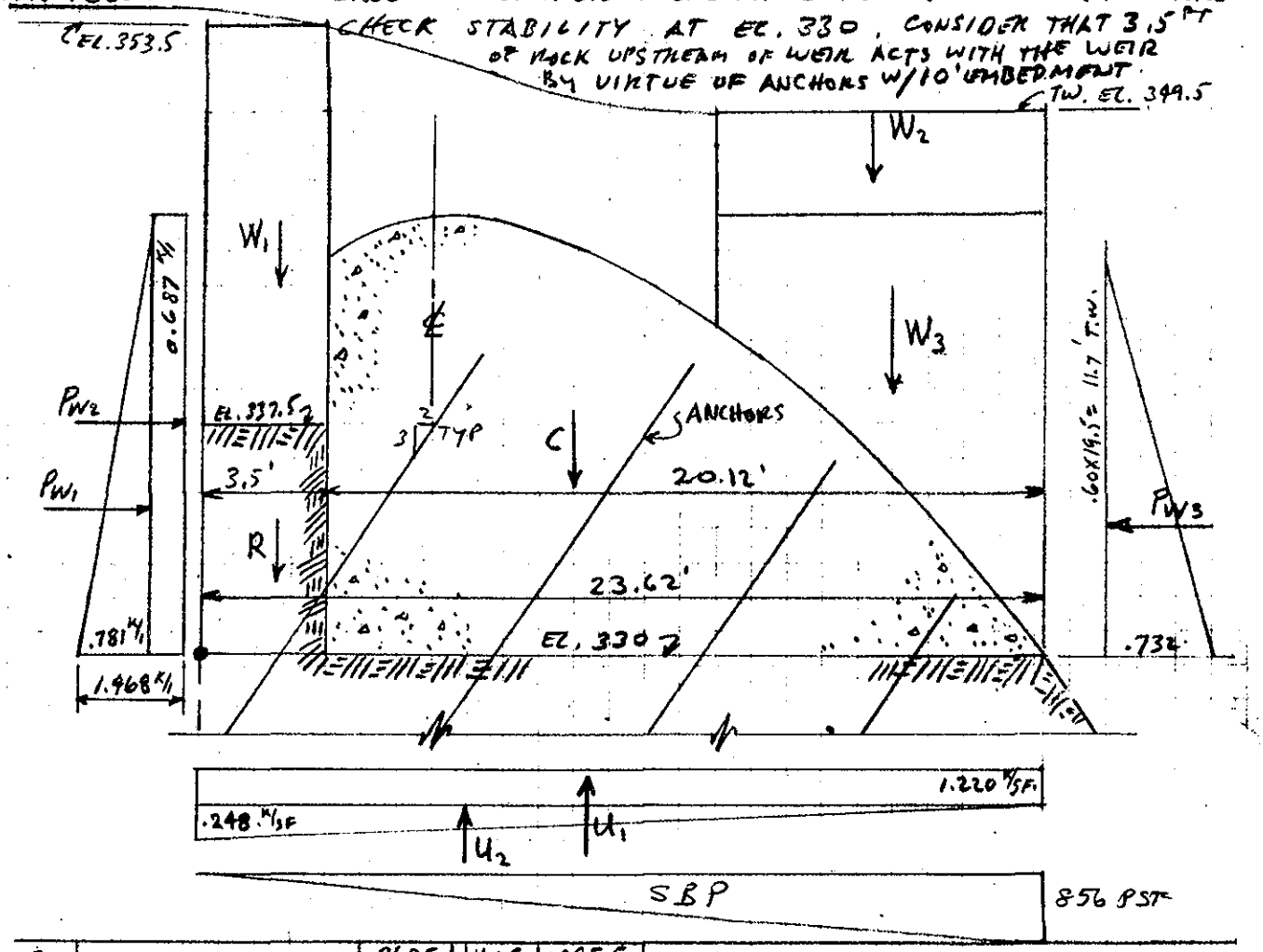
(10630, 307,000)

NED FORM 223
27 Sept 49

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

SUBJECT: WEST THOMPSON DAM
COMPUTATION: SPILLWAY WEIR - STABILITY
COMPUTED BY: Cmt CHECKED BY: JWF DATE: 11/14/62

CASE IV A :- FLOOD DISCHARGE CONDITION WITH TAILWATER TO EL. 349.5
BASE OF CONCRETE WEIR AT EL. 330. USE ROCK ANCHORS
CHECK STABILITY AT EL. 330. CONSIDER THAT 3.5 FT OF ROCK UPSTREAM OF WEIR ACTS WITH THE WEIR BY VIRTUE OF ANCHORS W/10' EMBEDMENT.



C		26.25 ↓	11.25	+295.5
R	.15 x 7.5 x 3.5	3.94 ↓	1.75	+ 6.9
W1	.0625 x 16.0 x 3.5	3.50 ↓	1.75	+ 6.1
W2	.0625 x 9.15 x 7	4.00 ↓	19.04	+ 76.0
W3	.0625 x 66.84	4.18 ↓	20.19	+ 84.5
U1	1.220 x 23.62	28.80 ↑	11.81	- 341.0
U2	0.248 x 23.62/2	2.93 ↑	7.88	- 23.1
	ΣV =	10.14 ↓		
Pw1	.781 x 12.5 x 1/2	4.88 →	4.17	+ 20.4
Pw2	.687 x 12.5	8.60 →	6.25	+ 53.7
Pw3	.732 x 11.7 x 1/2	4.28 ←	3.90	- 16.7
		ΣH = 9.20 →		ΣM = +162.3

$f_{MAX} = \frac{2(10140)}{23.62} = 856$ PSF.

$f_{MIN} = 0$ PSF.

$S_{(s-f)} = \frac{.65(10140) + 90(23.62)144}{9200} = 34.1 > 4.0$ ok
(6600 · 307000)

$\Sigma H/\Sigma V = \frac{9.2}{10.14} = 0.91$.

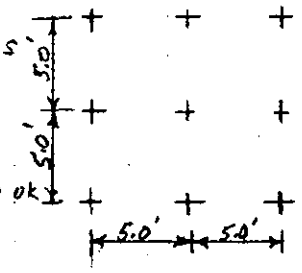
$\frac{\Sigma M}{\Sigma V} = \frac{162.3}{10.14} = 16.0' > 7.88 \approx 15.76'$
CLOSE ENOUGH AT 1/3 PT.

SUBJECT West Thompson Dam
COMPUTATION Spillway Weir - LINING.
COMPUTED BY I.W.F. CHECKED BY Cmt DATE

Lining at Spillway Weir

head reduced 50% because of drains

For 40' head $w = 40(62.5)\cdot.50 = 1{,}250\ Lb/ft^2$



Try #11 anchors @ 5'-0" c.c. each way

$1.56 \times 20{,}000 = 31{,}200$ $\quad 5 \times 5 \times 1{,}250 = 31{,}200$ ok

Design as two way slab.

$-M = .033(1{,}250)(5)^2 = 1030\ ft\text{-}Lb$

$V = \frac{wS}{3} = \frac{1{,}250\ (5.0)}{3}(1) = 2080\ Lb/ft.$

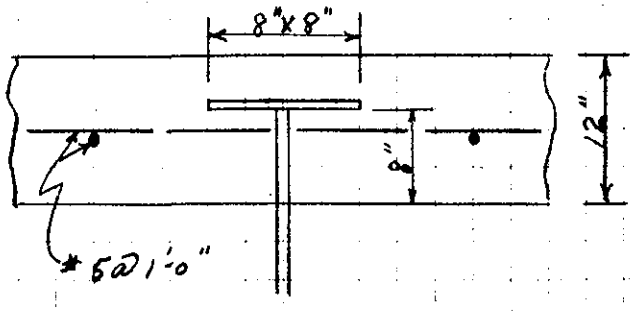
$M = \frac{2080\ (5.0)^2}{10} = 5{,}200\ ft.\text{-}Lb.$

assume a 2.5' wide beam

req'd $d = \sqrt{\frac{5{,}200\ (12)}{160\ (2.5 \times 12)}} = \sqrt{13} = 3.6"$ OK have 6"

$A_s = \frac{5.2\ (12)}{20 \times .885 \times 6 \times 2.5} = 0.235\ in^2$

temp $A_s = .002 \times 12 \times 12 = 0.288\ in^2$ use #5 @ 12" each way
$A_s = 0.31\ in^2$



Punching Shear

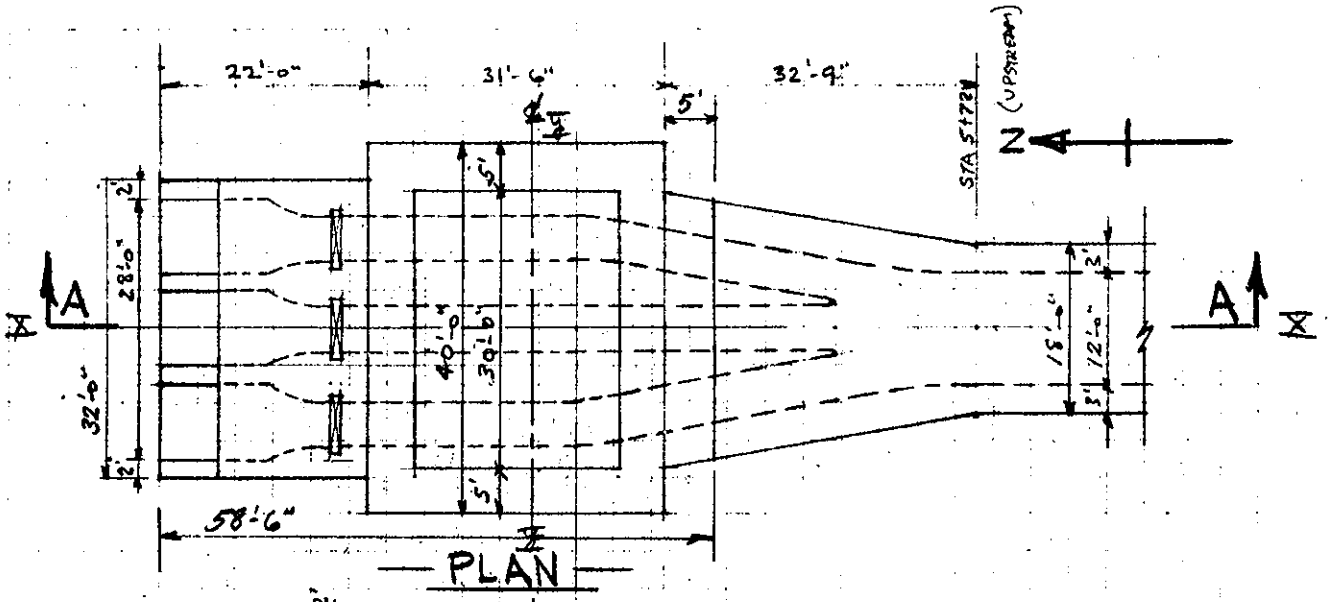
$V = \frac{31{,}200}{4 \times 8 \times 8} = 122\ p.s.i.$ ok

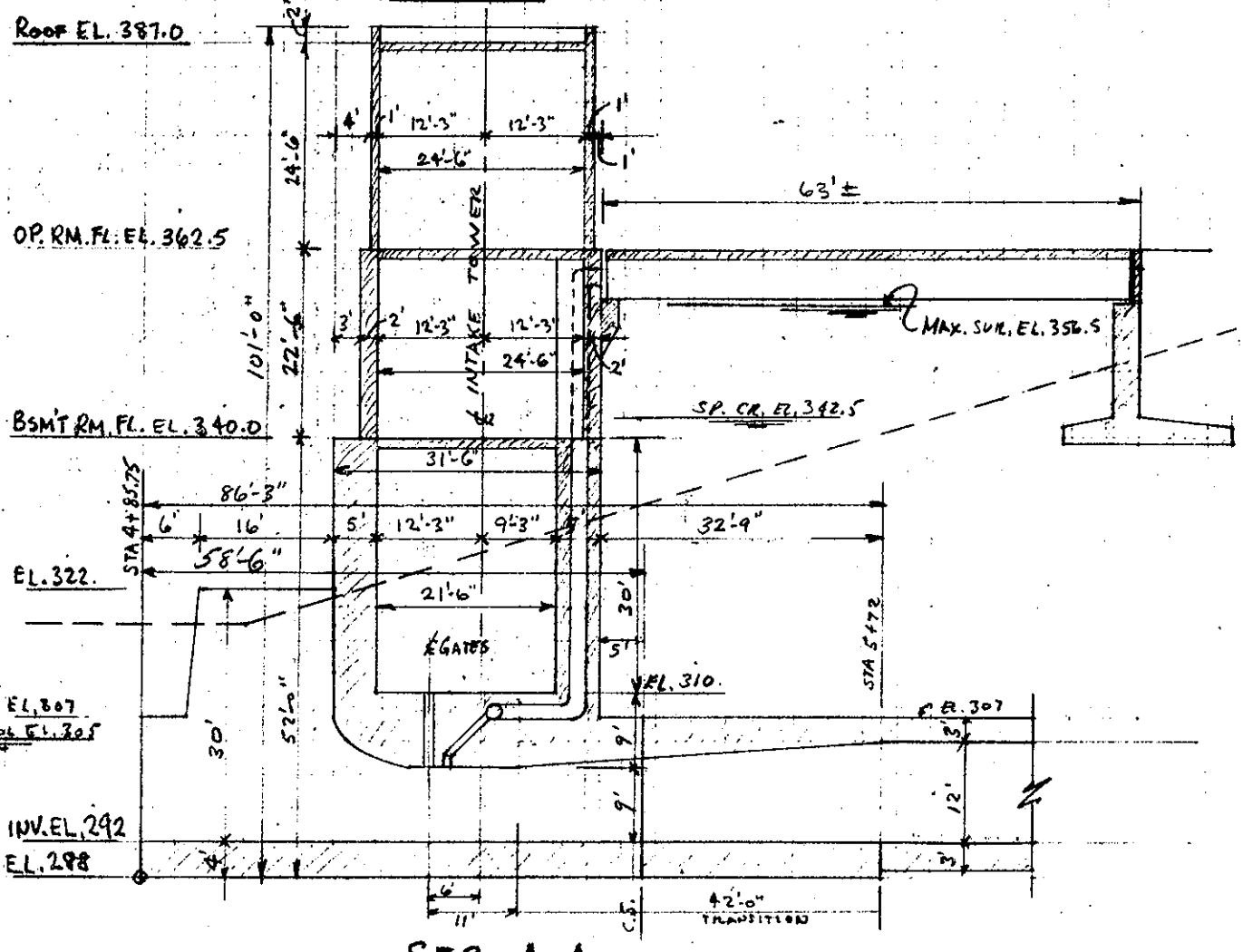
Allow. $0.2 f'c = 600\ PSI$

NED FORM 223
27 Sept 49

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

SUBJECT: WEST THOMPSON RES.
COMPUTATION: INTAKE STRUCTURE – SKETCH
COMPUTED BY: Cmt CHECKED BY: R.A.K. DATE: 8/10/62





NED FORM 223
27 Sept 49

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

SUBJECT: WEST THOMPSON RES.
COMPUTATION: INTAKE STRUCTURE
COMPUTED BY: [illegible] CHECKED BY: R.A.L. DATE: 8/13/62

LOADING CONDITIONS: — REF. EM - PART CXXIV OCT '56

I - CONSTRUCTION CONDITION; NO EARTH, DRY, 30 PSF WIND
II - NORMAL OPERATING CONDITION; POOL TO SP. CR. EL. 342.5, GATES CLOSED, ICE (3000 PSF)
III - PERMANENT POOL CONDITION; PERM. POOL @ EL. 305, ICE, GATES CLOSED
IV - FLOOD DISCHARGE CONDITION; POOL @ MAX. SUR. EL. 356.5, GATES CLOSED, NO ICE
V - CASE I W/ EARTHQUAKE; 0.05 G., NO WIND
VI - CASE II W/ EARTHQUAKE; 0.05 G., NO WIND

OVERTURNING & SLIDING CRITERIA: —

CASE	RESULTANT TO FALL WITHIN	SLIDING ΣH/ΣV
I	MID 1/3	2.65
II	" 1/3	"
III	" 1/3	"
IV	" 1/3	"
V	BASE WIDTH	
VI	" "	0.85

SOIL CHARACTERISTICS (EST.) PCF

MAT.	DRY	MOIST	SAT.	SUB.	φ
ROCK	120	—	140	78	
COMPACT. PERVIOUS	135	142	147	85	30
COMPACT. IMPERVIOUS	130	140	145	85	30.5
CONCRETE	150				

K @ REST = 0.5

STRESSES: —		NORMAL	PSI	INCREASED	PSI
CONCRETE	f'_c	=	3000	:	3000
	f_c	=	1050	:	1350
	a	=	1.44	:	2.0
	j	=	0.885	:	0.889
	K	=	160	:	200
	v	=	90	:	90
	u	=	210 T.B. 300	:	210 T.B. 300
REINF.	f_s	=	20000	:	27000

LIVE LOADS: —

GATES (3) = LEAF 3(14±) + STEEL & FRAME 3(10±) = 42+30 : 72^K
CRANE CAP. : 10^K±
BRIDGE D.L. REACTION (63' SPAN) = 62^K
BRIDGE L.L. REACTION " AASHO - H20-44 APPENDIX "A" NO IMPACT W/IMP 57.5 = 45.2^K
FLOOR L.L. = 250 PSF
ROOF L.L. : 40 PSF

NED FORM 223
27 Sept 49

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

SUBJECT: WEST THOMPSON RES.
COMPUTATION: INTAKE STRUCTURE — STABILITY
COMPUTED BY: Cant CHECKED BY: R.A.K. DATE: 8/14/62

CONCRETE ONLY: — + GATES, CRANE & BRIDGE D.C.

ITEM	FACTORS	V^K	ARM'	M_H^{IK}	ARM'	M_V^{IK}
ROOF	.15 x 29.5 x 30 x 1	110.2	39.25	+ 4327.3	98.5	+ 10855
OP. RM. FL.	.15 x 24.5 x 30 x 1.25	137.8	39.25	+ 5409.1	74.0	+ 10197
BSMT FL.	.15 x 21.5 x 30 x 1.25	120.9	37.75	+ 4565.4	51.5	+ 6190
1FT WALLS	.15 x 40 x 26.5 x 2 x 1	318.0	39.25	+ 12481.5	87.75	+ 27905
"	.15 x 24.5 x 26.5 x 2 x 1	194.8	39.25	+ 7644.9	87.75	+ 17094
2FT WALLS	.15 x 40 x 22.5 x 2 x 2	540.0	39.25	+ 21,195.0	63.25	+ 34155
"	.15 x 24.5 x 22.5 x 2 x 2	330.7	39.25	+ 12,981.9	63.25	+ 20917
5FT WALLS	.15 x 40 x 30 x 2 x 5	1800.0	37.75	+ 67,950.0	37.0	+ 66600
"	.15 x 21.5 x 30 x 2 x 5	967.5	37.75	+ 36,523.1	37.0	+ 35798
BASE @ TOWER	.15 x 40 x 31.5 x 22	4158.0	37.75	+ 156,964.5	11.0	+ 45738
TRASH PLATFORM	.15 x 32 x 16 x 1	76.8	14.0	+ 1075.2	33.5	+ 2573
TRASH WALLS	.15 x 4 x 2 x 16 x 29	556.8	14.0	+ 7795.2	18.5	+ 10300
"	.15 x 4 x 2 x 6 x 15	108.0	3.0	+ 324.0	11.5	+ 1242
"	.15 x 3 x 3 x 8 x 29	313.2	18.0	+ 5638.8	18.5	+ 5794
TRASH BANKS	.15 x 3 x 1.5 x 29 x 3.5	68.5	6.5	+ 445.3	18.5	+ 1267
TRASH WALLS Δ	.15 x 4 x 14 x 1.12/2 x 2	9.4	5.63	+ 53.0	23.67	+ 222
TRASH BASE	.15 x 4 x 32 x 22	422.4	11.0	+ 4646.4	2.0	+ 845
TRANSITION	.15 x 19 x 5 x 29	413.2	56.0	+ 23142.0	9.5	+ 3925
DEDUCT CONDUITS	.15 x 5 x 37 x 9 x 3	749.2	40.5	− 30342.6	8.5	− 6368
	.15 x 4 x 7½ x 5 x 3	31.5	24.3	− 765.4	14.33	− 451
	.15 x 3 x 5 x 1.0½ x 14.5	16.6	53.7	− 893.5	13.33	− 221
AIR VENT	.15 x π x 1² x 11	5.2	46.0	− 239.4	20.0	− 104
	.15 x π x 51	24.0	51.0	− 1225.7	45.5	− 1092
GATE HOLE	.15 x 4 x 7 x 1 x 3	28.4	33.25	− 942.6	17.5	− 497
NEW ENT. TRANS.	.15 x 21.5 x 3 x 5	48.4	50.0	− 2418.7	67.75	+ 3637
HANDRAIL	.15 x 2 x 3 x 12	10.8	54.5	+ 588.6	68.5	+ 740
	.15 x 2 x 3 x 4½ x 12	6.2	54.5	+ 337.4	65.83	+ 408
SLAB OVR D.C. SERV.	.15 x 5 x 8 x 2 x 1	12.0	39.25	− 471.0	63.2	− 759
BRIDGE		62.0	54.5	+ 3379.0	70% FACTOR	+ 1447
3 GATES		72.0	33.25	+ 2394.0	17.5	+ 1224
CRANE		10.0	39.25	+ 392.5	97.0	+ 970
	2470 C.Y. ΣV	9988.7	ΣM_H	+348,105.6	ΣM_V	+ 299,951

$\Sigma M_H/\Sigma V = \frac{348,105.6}{9988.7} = 34.85 > 19.5 < 39.0$ ok

C.G. VERT. $= \frac{\Sigma M_V}{\Sigma V} = \frac{299,951}{9988.7} = 30.02$ FROM BASE

$e = 34.85 - 30.0 = 4.85$

$SBP_{DWNSTM} = 4.736 + \frac{9989(4.85)28.5}{560726} = +7.198$ K/SF (2.462)

$SBP_{UPSTM} = 4.736 - \frac{9989(4.85)30}{560726} = +2.144$ K/SF (2.592)

CONDITION I_A: — CONSTRUCT. COND.

ITEM	FACTOR	V^K	A'	M_H^{IK}
CONC.		9989 ↓		+ 348106.
	ΣV_{I_A}	9989 ↓		
N→S WIND	.03 x 97 x 40	116.4 →	48.5	+ 5645
	ΣH_{I_A}	116.4	ΣM_{I_A}	+ 353,751

$\Sigma H/\Sigma V = \frac{116.4}{9989} = .01 < .65$ ok

$\frac{\Sigma M}{\Sigma V} = \frac{353751}{9989} = 35.41 > 19.5 < 39.0$ ok

$SBP_{DWST\ I_A} = \frac{9989}{2109} + \frac{9989(5.41)28.5}{560726} = 4.736 + 2.736 = 7.472$ K/SF (23.5 = " +2.256 = 6.992)

$SBP_{UPST\ I_A} = 4.736 - \frac{9989(5.41)30}{560726} = 4.736 - 2.840 = 1.896$ K/SF (8 = " − 0.761 = 3.968)

CONDITION V_A: — EARTHQUAKE + I (N→S)

ITEM	FACTOR	V^K	A'	M_H^{IK}
CONC		9989 ↓		+ 348106.
	ΣV_{V_A}	9989 ↓		
EARTHQUAKE	.05 x 9989	499 →	30.0	+ 14993
	ΣH_{V_A} 499		ΣM_{V_A}	+ 363,099

$\Sigma H/\Sigma V = \frac{499}{9989} = 0.05 < .65$ ok

$\Sigma M/\Sigma V = \frac{363099}{9989} = 36.35 > 19.5 < 39.0$ ok

$SBP_{DWST} = 4.736 + \frac{9989(6.35)28.5}{560726} = 4.736 + 3.218 = 7.95$ K/SF (21.5 = " + 2.652 = 7.388)

$SBP_{UPST} = 4.736 - \frac{9989(6.35)30.5}{560726} = 4.736 - 3.383 = 1.353$ K/SF (8 = " − 0.902 = 3.834)

C.G. OF BASE

	22 x 32	704	11.0	7740
	40 x 31.5	1260	37.75	47500
	29 x 5	45	18.0	8120
	A =	2109		63360
	$\bar{X}$ =	30.0'		2109

MOM. OF INERTIA I_y @ C.G.

	32(22)³/12	28395
	40(31.5)³/12	104186
	29(5)³/12	302
	704(11)²	85194
	1260(7.75)²	75679
	45(26)²	98020
	I_y =	560726 FT⁴

MOM. OF INERTIA @ I_x

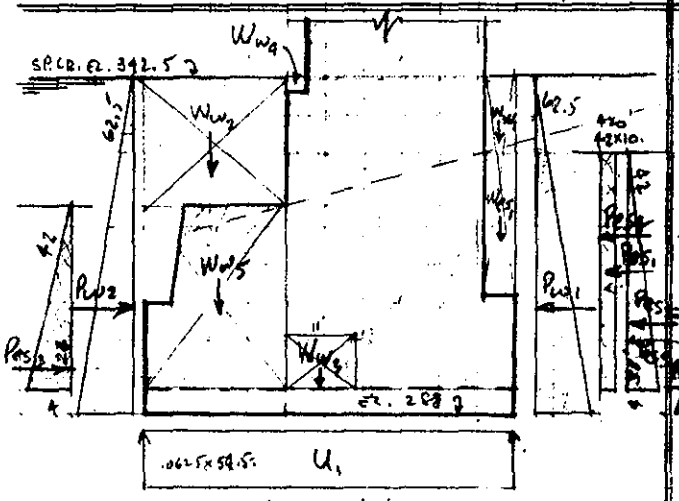
	22(32)³/12	60075
	31.5(40)³/12	168000
	5(29)³/12	10162
	I_x =	238237 FT⁴

NED FORM 223
27 Sept 49

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

SUBJECT: WEST THOMPSON RES.
COMPUTATION: INTAKE STRUCTURE — STABILITY
COMPUTED BY: CMB CHECKED BY: R.A.K. DATE: 8/17/62

CONDITION II :— WATER TO EL. 342.5



EQ. SUR. DWST = $\frac{1}{4}h$ = 42/4 = +10'±
EQ. SUR. UPST. = $\frac{h}{6(1.16)} = \frac{33}{6(1.16)}$ = −5'±, 33 − 5 = 28'
ICE DOES NOT AFFECT STABILITY.

ITEM	FACTORS	V^K	A'	M'^K
Conc.		9989 ↓		+348106
W_{es1}	.085 x 23 x 29 x 5	283 ↓	56.0	+ 15875
W_{w1}	.0625 x 35.5 x 29 x 5	322 ↓	56.0	+ 18016
W_{w2}	.0625 x 20.5 x 22 x 32	902 ↓	11.0	+ 9922
W_{w3}	.0625 x (11/2) x 11 x 5 x 3	113 ↓	27.5	+ 3120
W_{w4}	.0625 x 2.5 x 40 x 3	19 ↓	23.5	+ 440
W_{w5}	.0625 x 29 x 8 x 3 x 11	478 ↓	5.5	+ 2631
"	.0625 x 15.0 x 4 x 6 x 2	45 ↓	3.0	+ 135
"	.0625 x 6.5 x 2 x 29 x 8	283 ↓	15.0	+ 4241
U_1	.0625 x 54.5 x 2109	7184 ↑	30.0	− 215513
	ΣV_{II}	5250 ↓		
P_{es1}	.042 x 38 x 10 x 11	175 ←	23.0	− 4038
P_{es2}	.042 x 38 x 38/2 x 11	333 ←	16.67	− 5560
P_{es3}	.042 x 30 x 24/2 x 8	121 →	14.0	+ 1693
P_{es4}	.42 x 23 x 29	280 ←	30.5	− 8544
P_{es5}	.042 x 23 x 23/2 x 29	322 ←	26.67	− 8585
$P_{w1} = P_{w2}$	→ ←			
	ΣH_{II}	989 ←	ΣM_{II}	+161,939

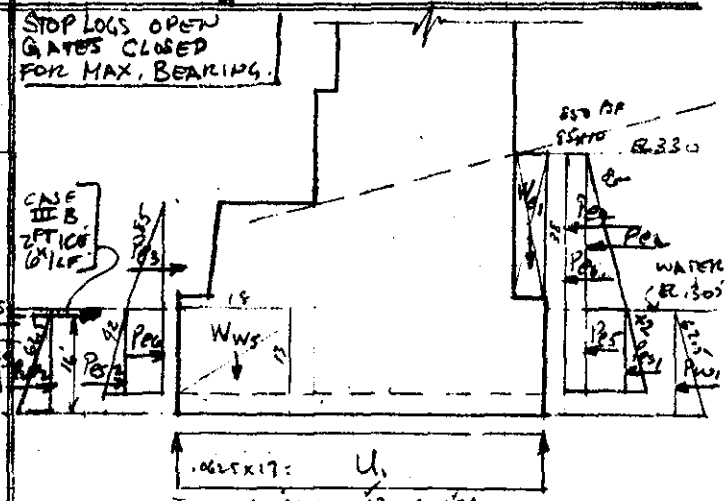
$\Sigma H/\Sigma V_{II} = \frac{989}{5250} = 0.19$ ok
$\Sigma M/\Sigma V = \frac{161939}{5250} = 30.85$ ok e = 30.85 − 30.0 = 0.85'

$SBP_{II\ DWNSTM} = \frac{5250}{2109} + \frac{5250(0.85)28.5}{560726} = 2.489 + .227 = 2.716$ K/SF

$SBP_{II\ UPST} = \frac{5250}{2109} - \frac{5250(0.85)30}{560726} = 2.489 - .239 = 2.250$ K/SF
23.5 : " + .187 = 2.676
8 : " − .064 = 2.425

CONDITION III_A :— WATER TO EL. 305, NO ICE

STOP LOGS OPEN
GATES CLOSED
FOR MAX. BEARING.



SBP @ 23.5 III_A : 4.041 + .043 = 4.084
III_B : " + .156 = 4.197
SBP @ 8 III_A : " − .015 = 4.026
III_B : " − .086 = 3.995

ITEM	FACTORS	V^K	A'	M'^K
Conc.	-	9989 ↓		+348106
W_{e1}	.155 x 23 x 29 x 5	450 ↓	56.0	+ 25212
W_{w5}	.0625 x 13 x 8 x 11 x 3	214 ↓	5.5	+ 1180
"	.0625 x 13 x 6.5 x 3 x 7	111 ↓	14.5	+ 1608
U_1	.0625 x 17 x 2109	2241 ↑	30.0	− 67224
	ΣV_{III_A}	8523 ↓		
P_{e1}	.85 x 38 x 11	355 ←	23.0	− 8172
P_{e2}	.085 x 25 x 25/2 x 40	1062 ←	25.33	− 26913
P_{e3}	.085 x 11/2 x 17 x 8	63 →	22.67	+ 1491
P_{e4}	.85 x 23 x 29	567 ←	30.5	− 17292
P_{e5}	.085 x 13 x 25 x 11	304 ←	10.5	− 3191
P_{e6}	.085 x 13 x 13 x 8	115 →	10.33	+ 1187
P_{es1}	.042 x 13/2 x 11	39 ←	8.33	− 325
P_{es2}	.042 x 13²/2 x 8	28 →	8.33	+ 237
$P_{w1} = P_{w2}$	→ ←			
CONDITION III_A	ΣH_{III_A}	2121 ←	ΣM_{III_A}	+255,854
ICE EFF.	6.0 x 32.0	192 →	16.0	+ 3072
CONDITION III_B w/ICE	ΣH_{III_B}	1929 ←	ΣM_{III_B}	+258,726

$\Sigma H/\Sigma V_{III_A} = \frac{2121}{8523} = 0.25$ ok $\Sigma H/\Sigma V_{III_B} = \frac{1929}{8523} = 0.23$ ok
$\Sigma M/\Sigma V_{III_A} = \frac{255,854}{8523} = 30.12$ ok $\Sigma M/\Sigma V_{III_B} = \frac{258,726}{8523} = 30.38$ ok
$e_{III_A} = 30.12 - 30.0 = 0.12$ $e_{III_B} = 30.38 - 30.0 = 0.38$

$SBP_{DWNST\ III_A} = \frac{8523}{2109} + \frac{8523(.12)28.5}{560726} = 4.041 + .052 = 4.093$ K/SF
III_B = " + $\frac{8523(.38)28.5}{560726}$ = 4.041 + .165 = 4.206 K/SF

$SBP_{UPST\ III_A}$ = " − $\frac{8523(.12)30}{560726}$ = 4.041 − .055 = 3.986 K/SF
III_B = " − $\frac{8523(.38)30}{560726}$ = 4.041 − .173 = 3.868 K/SF

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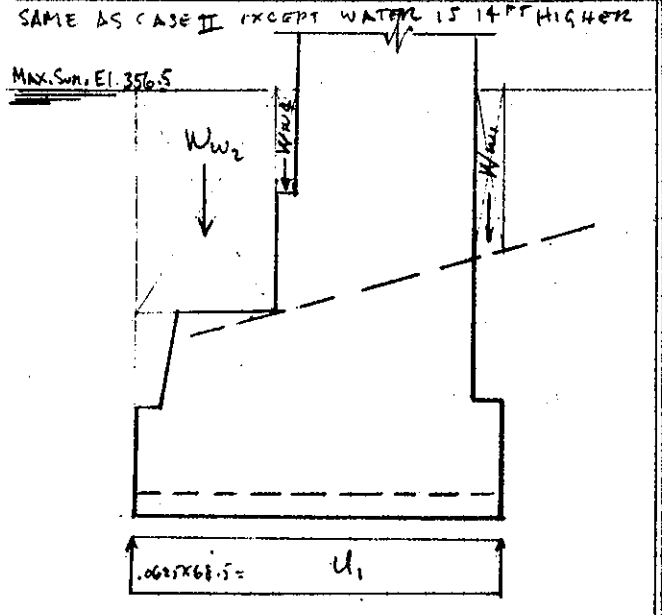
NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U. S. ARMY

PAGE DM 12

SUBJECT: WEST THOMPSON RES.
COMPUTATION: INTAKE STRUCTURE — STABILITY
COMPUTED BY: Cmt CHECKED BY: R.A.K. DATE: 8/20/62

CONDITION IV :— FLOOD DISCHARGE, WATER TO EL. 356.5

SAME AS CASE II EXCEPT WATER IS 14 FT HIGHER



ITEM	FACTORS	V^k	A'	M^{1k}
CONC.		9989 ↓		+348106
W_{ps_1}		283 ↓		+ 15875
W_{w_1}	.0625×49.5×29×5	448 ↓	56.0	+ 25121
W_{w_2}	.0625×34.5×22×32	1518 ↓	11.0	+ 16698
W_{w_3}		113 ↓		+ 3120
W_{w_4}	.0625×16.5×40×3	124 ↓	23.5	+ 2908
W_{w_5}		478 ↓		+ 2631
"		45 ↓		+ 135
"		283 ↓		+ 4241
U_1	.0625×68.5×2109	9029 ↑	30.0	−270875
	ΣV_{IV}	4252 ↓		− 4038
				− 5560
				+ 1693
				− 8544
				− 8585
	ΣH_{IV}	989 ←	ΣM_{IV}	+122,926

$\Sigma H_{IV}/\Sigma V = \frac{989}{4252} = 0.23$ ok

$\Sigma M_{IV}/\Sigma V = \frac{122,926}{4252} = 28.91$ ok

$e = 30.00 - 28.91 = 1.09$ Upstream

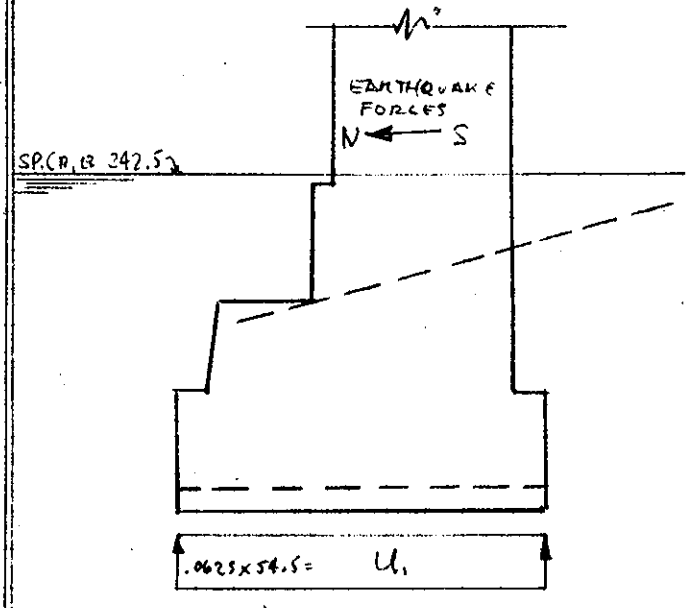
SBP_{IV} DWNSTM $= \frac{4252}{2109} - \frac{4252(1.09)28.5}{560726} = 2.016 - .236 = 1.780$ k/sf

23.5 = −.194 = 1.822

SBP_{IV} UPSTM $= \frac{4252}{2109} + \frac{4252(1.09)30}{560726} = 2.016 + .248 = 2.264$ k/sf

δ = +.066 = 2.082

CONDITION VI_A :— CASE II W/ EARTHQ'KE S→N



ITEM	FACTORS	V^k	A'	M^{1k}
ΣV_{II}		5250 ↓	ΣM_{II}	+161,939
	ΣV_{VI_A}	5250 ↓		
.05G.		499 ←		− 14993
HYDRODYNAMIC FORCE BY VIRTUAL MASS METHOD ASSUME CYLINDER OF WATER OF 40 FT DIA.				
DISP. $H_{20} = \pi(20)^2 54.5(.0625)(.05)$		214 ←	27.25	− 5832
	ΣH_{II}	989 ←		
	ΣH_{VI_A}	1702 ←	ΣM_{VI_A}	+141114

$\Sigma H_{VI_A}/\Sigma V = \frac{1702}{5250} = .32$ ok

$\Sigma M_{VI_A}/\Sigma V = \frac{141114}{5250} = 26.88$ ok $e = 30 - 26.88 = 3.12$ ← or ℄

SBP_{VI_A} DWNSTM $= \frac{5250}{2109} - \frac{5250(3.12)(23.5)}{560726}$ (.832) $= 1.657$ k/sf

SBP_{VI_A} UPSTM $= 2.489 + \frac{5250(3.12)30}{560726}$ (.876) $= 3.365$ k/sf

23.5 = 2.489 − .686 = 1.803

δ = 2.489 + .234 = 2.723

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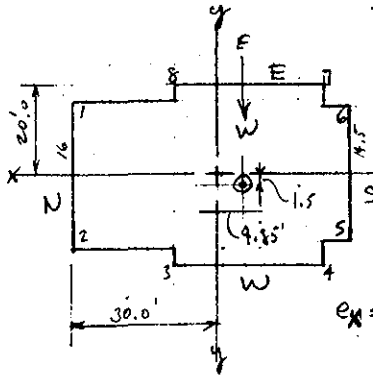
NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U. S. ARMY

SUBJECT: WEST THOMPSON RES.
COMPUTATION: INTAKE STRUCTURE — STABILITY
COMPUTED BY: [illegible] CHECKED BY: R.A.K. DATE: 8/21/62

CONDITION V_B: — EARTHQUAKE E→W + CASE I



$\Sigma M_x = 14993^{'K}$
$\Sigma H_x = 499^K$
$\Sigma V = 9989^K$
$e_y = \frac{14993}{9989} = 1.50'$

$\Sigma M_y = 348106^{'K}$
$\Sigma H_y = 0$
$e_x = \frac{348106}{9989} - 30 = 34.85 - 30 = 4.85'$

$SBP = \frac{P}{A} \pm \frac{M_y \bar{x}}{I_y} \pm \frac{M_x \bar{y}}{I_x}$

REV.	W→E	E→W
SBP_2	$SBP_1 = \frac{9989}{2109} - \frac{9989(4.85)30}{560726} - \frac{9989(1.5)16}{238237}$ (4.736, 2.592, 1.006)	$= +1.138$ K/SF
SBP_1	$SBP_2 = 4.736 - 2.592 + 1.006$	$= +3.150$ K/SF
SBP_8	$SBP_3 = 4.736 - \frac{9989(4.85)8}{560726} + \frac{9989(1.5)20}{238237}$ (.691, 1.258)	$= +5.303$ K/SF
SBP_7	$SBP_4 = 4.736 + \frac{9989(4.85)23.5}{560726} + 1.258$ (2.030)	$= +8.024$ K/SF
SBP_6	$SBP_5 = 4.736 + \frac{9989(4.85)28.5}{560726} + \frac{9989(1.5)14.5}{238237}$ (2.462, 0.912)	$= +8.110$ K/SF
SBP_5	$SBP_6 = 4.736 + 2.462 - 0.912$	$= +6.286$ K/SF
SBP_4	$SBP_7 = 4.736 + 2.030 - 1.258$	$= +5.508$ K/SF
SBP_3	$SBP_8 = 4.736 - .691 - 1.258$	$= +2.787$ K/SF

CONDITION I_B: — CONST. COND, WIND S→N

CONC S→N WIND	ΣV_{I_B}	9989 ↓		+348106
	.03×40×82 ΣH_{I_B}	98.4 ←	60.0	− 5904
			ΣM_{I_B}	+342202

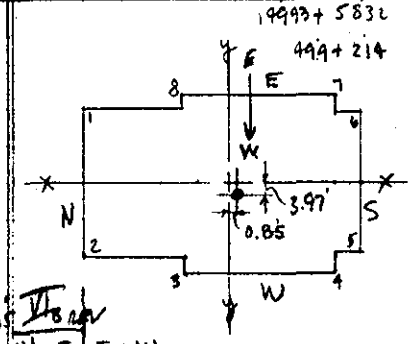
$\Sigma M/\Sigma V = \frac{342202}{9989} = 34.26 - 30 = 4.26'$ RT OF ℄

SBP_{I_B} DWNSTM $= 4.736 + \frac{9989(4.26)28.5}{560726}$ (2.163) $= +6.899$ K/SF

SBP_{I_B} UPSTM $= 4.736 - \frac{9989(4.26)30}{560726}$ (2.277) $= +2.459$ K/SF

23.5 = " + 1.783 = 6.519
8 = " − 0.607 = 4.129

CONDITION VI_B: — EARTHQUAKE E→W + CASE II



$\Sigma M_x = 20825^{'K}$
$\Sigma H_x = 713^K$
$\Sigma V = 5250^K$
$e_y = \frac{20825}{5250} = 3.97'$

$\Sigma M_y = 161,939^{'K}$
$\Sigma H_y = 989^K$
$e_x = \frac{161,939}{5250} - 30 = 30.85 - 30 = 0.85$

VI_B REV. W→E	E→W
SBP_2	$SBP_1 = \frac{5250}{2109} - \frac{5250(0.85)30}{560726} - \frac{5250(3.97)16}{238237}$ (.239, 1.400) $= +0.850$ K/SF
SBP_1	$SBP_2 = 2.489 - 0.239 + 1.400$ $= +3.650$ K/SF
SBP_8	$SBP_3 = 2.489 - \frac{5250(0.85)8}{560726} + \frac{5250(3.97)20}{238237}$ (.064, 1.75) $= +4.175$ K/SF
SBP_7	$SBP_4 = 2.489 + 0.187 + 1.75$ $= +4.426$ K/SF
SBP_6	$SBP_5 = 2.489 + 0.287 + 1.268$ $= +3.984$ K/SF
SBP_5	$SBP_6 = 2.489 + 0.527 - 1.268$ $= +1.748$ K/SF
SBP_4	$SBP_7 = 2.489 + 0.187 - 1.75$ $= +0.926$ K/SF
SBP_3	$SBP_8 = 2.489 - 0.064 - 1.75$ $= +0.675$ K/SF

CONDITION V_C: — EARTHQK S→N + CASE I

ΣV_{I_C}	9989 ↓		+348106
ΣH_{I_C}	499 ←		− 14993
		ΣM_{I_C}	+333113

$\Sigma M/\Sigma V_{I_C} = \frac{333113}{9989} = 33.35 - 30 = 3.35'$ RT OF ℄

SBP_{I_C} DWNSTM $= 4.736 + \frac{9989(3.35)23.5}{560726}$ (1.701) $= +6.437$ K/SF
23.5 = 4.736 + 1.402 = 6.138

SBP_{I_C} UPSTM $= 4.736 - \frac{9989(3.35)30}{560726}$ (1.790) $= +2.946$ K/SF
8 = 4.736 − 477 = 4.259

CONDITION VI_C: — EARTHQK N→S + CASE II

ΣV_{VI_C}	5250 ↓		+167878
	989 ←		+ 14993
	499 →		+ 5832
	214 →		
ΣH_{VI_C}	276 ←	ΣM_{VI_C}	+188303

$\Sigma M/\Sigma V_{VI_C} = \frac{188303}{5250} = 35.87 - 30 = 5.87'$ RT OF ℄

SBP_{VI_C} DWNSTM $= 2.489 + \frac{5250(5.87)23.5}{560726}$ (1.560) $= +4.055$ K/SF

SBP_{VI_C} UPSTM $= 2.489 - \frac{5250(5.87)30}{560726}$ (1.649) $= +0.840$ K/SF

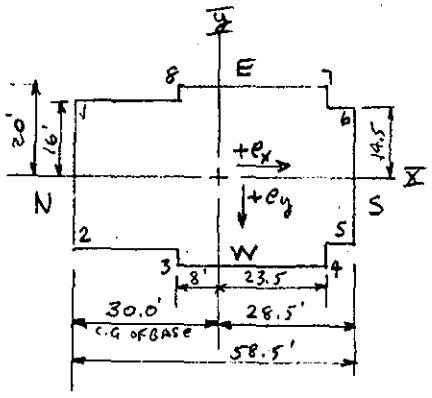
NED FORM 223
27 Sept 49

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

SUBJECT: WEST THOMPSON RES.
COMPUTATION: INTAKE STRUCTURE — STABILITY
COMPUTED BY: Cmt CHECKED BY: R.A.K. DATE: 8/22/62

SUMMARY OF LOADING CONDITIONS: — ALL PT'S ARE COMPRESSION BEARING

	CASE	$\Sigma V_{\downarrow}^{K}$	ΣH_y^{K}	ΣH_x^{K}	ΣM_y^{1K}	ΣM_x^{1K}	ECCENTRICITY e_x'	ECCENTRICITY e_y'	SBP KSF 1	2	3	4	5	6	7	8	UPLIFT KSF ↑
NORMAL STRESSES	I A	9989	116 →	0	353751	0	5.39	0	1.856	1.856	3.968	6.992	7.472	7.472	6.992	3.968	0
	I B	9989	98 ←	0	342202	0	4.26	0	2.459	2.459	4.179	6.519	6.899	6.899	6.519	4.129	0
	II	5250	989 ←	0	161939	0	0.85	0	2.250 (5.361)	2.250 (5.361)	2.825 (5.753)	2.678	2.716	2.716	2.678	2.425 (5.753)	3.406
	III A	8523	2121 ←	0	255854	0	0.12	0	3.986	3.986	4.026	4.084	4.093	4.093	4.084	4.026	1.063
	III B	8523	1929 ←	0	258926	0	0.38	0	3.868	3.868	3.995	4.177	4.206	4.206	4.177	3.995	1.063
INCREASED STRESSES	IV	4252	989 ←	0	122926		-1.09	0	2.264	2.264	2.082	1.822	1.780	1.780	1.822	2.082	4.281
	V A	9989	499 →		363099		6.33	0	1.353	1.353	3.834	7.386	7.950	7.950	7.386	3.834	0
	V B	9989	0	499 ↓	348106	14993	4.85	1.50	1.138	3.150	5.303	8.024	8.110	6.256	5.508	2.787	0
	V B REV	9989	0	499 ↑	348106	14993	4.85	1.50	3.150	1.138	2.787	5.508	6.256	8.110	8.024	5.303	0
	V C	9989	499 ←	0	333113	0	3.35	0	2.946	2.946	4.259	6.138	6.437	6.437	6.138	4.259	0
	VI A	5250	1702 ←	0	148114	0	-3.12	0	3.365 (7.073)	3.365 (7.073)	2.723	1.803	1.657	1.657	1.803	2.723	3.406
	VI B	5250	989 ←	713 ↓	161939	20825	0.85	3.97	0.850	3.650	4.175 (7.503)	4.426 (8.063)	3.984	1.448	0.926	0.675	3.406
	VI B REV	5250	989 ←	713 ↑	161939	20825	0.85	3.97	3.650	0.850	0.675	0.926	1.448	3.984	4.426 (8.063)	4.175 (7.503)	3.406
	VI C	5250	276 ←	0	188303	0	5.87	0	0.840	0.840	–		4.055	4.055	–		3.406



PLAN OF BASE

ALL RESULTANTS FALL WITHIN MID 1/3 ok
MAX. SLIDING FACTOR $(\Sigma H/\Sigma V) = 0.32 < .65$ ok

NED FORM 223
27 Sept 49

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

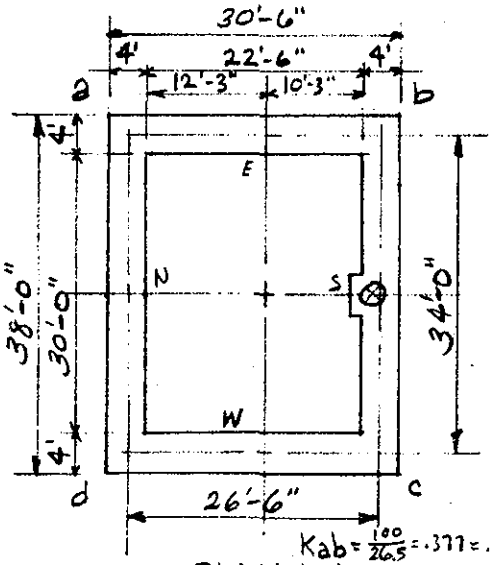
SUBJECT: WEST THOMPSON RES.
COMPUTATION: INTAKE STRUCTURE — DESIGN
COMPUTED BY: Cmt CHECKED BY: R.A.K. DATE: 8/22/62

SCHEDULE OF DESIGN PROCEDURES: —

SEC.	ITEM	ELEMENT	DESIGN METHOD	TEST LOADING	STRESS CRITERIA	PAGE
WALLS	1.	5FT WALLS BETWEEN EL. 310 & 340	1-1 = HORIZ. BOX	a. II w/ICE	NORMAL	
				b. III	"	
				c. IV	INCREASED	
			1-2 = VERTICAL COLUMN w/ BENDING	d. V A	"	
				e. V B	"	
				f. VI A	"	
				g. VI B	"	
				h. DETERMINE PRIN. STRESS @ 4-SIDES		
	2.	2FT WALLS BETWEEN EL. 340 & 362.5	2-1: HORIZ. BOX	a. II w/ICE + BRIDGE REACTION	NORMAL	
				b. IV NO ICE "	INCREASED	
			2-2 = VERTICAL COLUMN w/ BENDING	c. V A "	"	
				d. V B "	"	
				e. VI A "	"	
				f. VI B "	"	
				g. DETERMINE PRIN. STRESS @ 4-SIDES		
	3.	1FT WALLS BETWEEN EL 362.5 & 387.	3-1: HORIZ. BOX	a. I A	NORMAL	
			3-2 = VERTICAL COLUMN w/ BENDING	b. V A	INCREASED	
				c. V B	"	
				d. VI A	"	
				e. VI B	"	
				f. DETERMINE PRIN. STRESS @ 4-SIDES		
BASE	4.	4FT BASE SLAB UNDER TOWER @ EL. 292	ONE-WAY CONTINUOUS SLAB OVER 4-SUPPORTS	a. I A	NORMAL	
				b. II	"	
				c. IV	INCREASED	
				d. V A	"	
				e. V B	"	
				f. VI A	"	
				g. VI B	"	
	5.	22FT CANTILEVERED BASE @ NORTH FACE	5-1 = CANTILEVER N-S	a. I B S→N	NORMAL	
			5-2 = ONE WAY CONTINUOUS SLAB OVER 4-SUP.	b. IV	INCREASED	
				c. V c	"	
				d VI A	"	
	6.	5FT CANTILEVERED BASE @ SOUTH FACE	6-1 = CANTILEVER N-S	a. I A	NORMAL	
			6-2 = ONE-WAY CONTINUOUS SLAB OVER 4-SUP.	b. IV	INCREASED	
				c. V A	"	
				d VI c N→S	"	
FLOORS	7.	ROOF SLAB @ EL 387	SLAB + BEAM	40 PSF L.L.	NORMAL	
	8.	OPERATING RM. FL. EL. 362.5	SLAB & BEAM	500 PSF	"	
	9.	BASEMENT RM. FL. EL. 340	"	GATE LOAD	"	
	10.	GATE RM. FL. EL. 310	1-WAY CONTINUOUS SLAB OVER 4-SUPPORTS	"	"	
	11.	SERVICE BRIDGE HAUNCH	CANTILEVER	BRIDGE REACTION	"	
	12.	CRANE				

SUBJECT: WEST THOMPSON DAM
COMPUTATION: INTAKE STRUCTURE – DESIGN
COMPUTED BY: Cmt CHECKED BY: DATE: 11/14/62

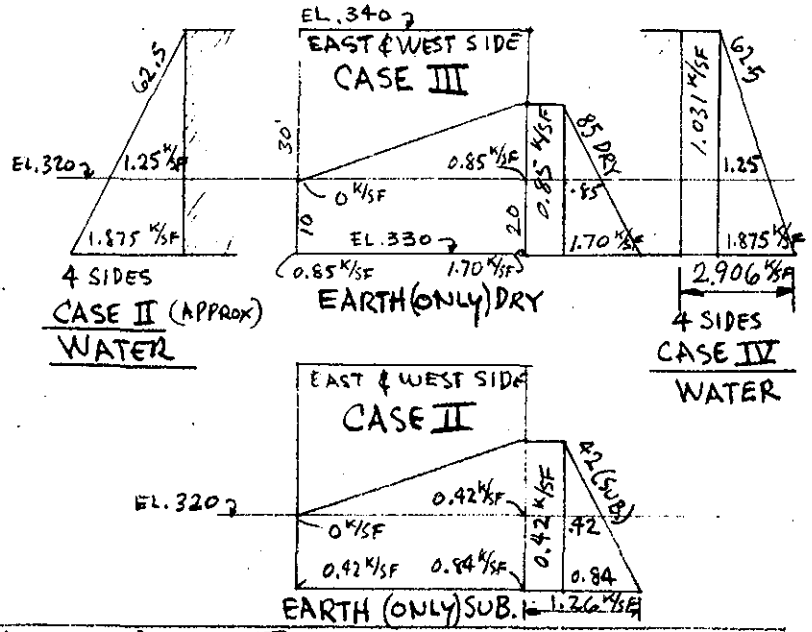
(1.) DESIGN OF 5FT WALLS @ EL. 322.0 : —



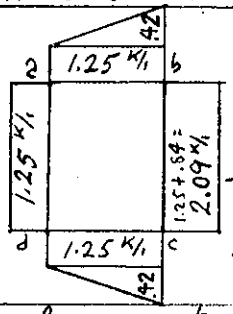
$K_{ab} = \frac{100}{26.5} = .377 = .562$

PLAN $K_{ad} = \frac{100}{34.0} = \frac{.294}{.671} = .438$

SKETCH OF LD'G: — USE LD'G @ EL. 320.0 (NO ICE)



CASE II @ EL. 320 : WATER & SUB. EARTH, NO ICE



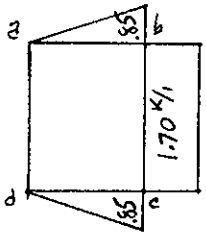
$-M^F_{ad,da} = 1.25(34)^2/12 = 120^{'K}$

$-M^F_{ab,dc} = \frac{1.25(26.5)^2}{12} + \frac{.42(26.5)^2}{30} = 83$ (73 + 10)

$-M^F_{ba,cd} = 73 + 15 = 88$

$-M^F_{bc,cb} = 2.09(34)^2/12 = 201$

CASE III @ EL. 320 : – DRY EARTH ONLY



$-M^F_{ad,da} = \quad = 0$

$-M^F_{ab,dc} = 2(10) = 20$

$-M^F_{ba,cd} = 2(15) = 30$

$-M^F_{bc,cb} = 1.70(34)^2/12 = 164$

	a		b		c		d		a		b		c		d	
	.562	.562	.438	.438	.562	.562	.438	.438								
FEM	-83	+88	-201	+201	-88	+83	-120	+120	-20	+30	-164	+164	-30	+20	0	0
	-21	+63	+50	-50	-63	+21	+16	-16	+11	+75	+59	-59	-75	-11	-9	+9
	+32	-11	-25	+25	+11	-32	-8	+8	+38	+6	-30	+30	-6	-38	+5	-5
	-22	+20	+16	-16	-20	+22	+18	-18	-18	+13	+11	-11	-13	+18	+15	-15
	+10	-11	-8	+8	+11	-10	-9	+9	+6	-9	-5	+5	+9	-6	-7	+7
	-10	+11	+8	-8	-11	+10	+9	-9	-7	+8	+6	-6	-8	+7	+6	-6
ΣM	-94	+160	-160	+160	-160	+94	-94	+94	+10	+123	-123	+123	-123	-10	+10	-10
ΔV	↓2.5	↑2.5	0	0	↑2.5	↓2.5	0	0	↓5.0	↑5.0	0	0	↑5.0	↓5.0	0	0
SBV	16.5 1.9	16.5 3.7	35.5	35.5	16.5 3.7	16.5 1.9	21.3	21.3	3.8	7.5	29.7	29.7	7.5	3.8	0	0
V	↑15.9	↑22.7	↑35.5	↑35.5	↑22.7	↑15.9	↑21.3	↑21.3	↓1.2	↑12.5	↑29.7	↑29.7	↑12.5	↓1.2	0	0
N	21.3		22.7		21.3		15.9		0		12.5		0		1.2 (TEN)	
+M	+2		+141		+2		+86		0		+123		0		+10	

$15.9 - 1.25x - \frac{.42x^2}{26.5(2)} = 0, \; x = 11.7'$

$+M_{ab} = 15.9(11.7) - \frac{1.25(11.7)^2}{2} - \frac{.42(11.7)^3}{26.5(6)} - 94 = +2$

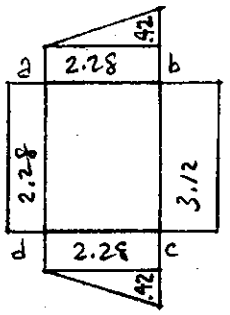
(186, 86, 4)

SUBJECT: WEST THOMPSON DAM

COMPUTATION: INTAKE STRUCTURE - DESIGN

COMPUTED BY: CMB CHECKED BY: DATE: 11/19/62

CASE IV :— @ EL. 320. WATER @ FLOOD DISCHARGE & SUB. EARTH.



$$M^F_{ab,dc} = \overset{133}{2.28(26.5)^2/12} + \frac{\overset{10}{.42(26.5)^2}}{30} = 143$$

$$M^F_{ba,cd} = \overset{133}{2.28(26.5)^2/12} + \frac{\overset{15}{.42(26.5)^2}}{20} = 148$$

$$M^F_{ad,da} = 2.28(34)^2/12 = 220$$

$$M^F_{bc,cb} = 3.12(34)^2/12 = 300$$

	a .562	.562 b	.438	.438 c	.562	.562 d	.438	.438 a
	-143	+148	-300	+300	-148	+143	-220	+220
	-43	+85	+67	-67	-85	+43	+34	-34
	+43	-22	-34	+34	+22	-43	-17	+17
	-34	+31	+25	-25	-31	+34	+26	-26
	+15	-17	-12	+12	+17	-15	-13	+13
	-16	+16	+13	-13	-16	+16	+12	-12
ΣM	-178	+241	-241	+241	-241	+178	-178	+178
ΔV	2.4	2.4	0	0	2.4	2.4	0	0
SBV	30.2 / 1.9	30.2 / 3.7	53	53	30.2 / 3.7	30.2 / 1.9	39	39
V	29.7	36.3	53	53	36.3	29.7	39	39
N	39		36.3		39.0		29.7	
+M	+11		+209		+11		+152	

$$29.7 - 2.28x - \frac{.42x^2}{26.5(2)} = 0, \quad x = 12.5'$$

$$+M_{ab} = \underset{372}{29.7(12.5)} - \underset{178}{2.28(12.5)^2/2} - \underset{5}{\frac{.42(12.5)^3}{26.5(6)}} - 178 = +11$$

NED FORM 223
27 Sept 49

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

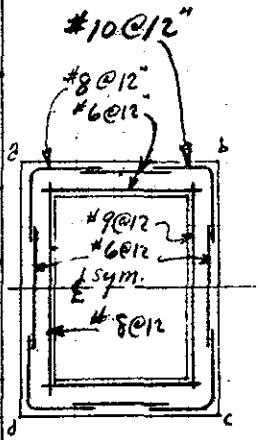
SUBJECT: WEST THOMPSON RES.
COMPUTATION: INTAKE STRUCTURE — DESIGN OF 4 FT WALLS @ EL. 322
COMPUTED BY: Cmt
CHECKED BY: R.A.K.
DATE: 9/25/62
PAGE: DM 17

1-1 :- DESIGN OF 5 FT WALLS (CONT.) — REDUCTION FACTOR FOR INCREASED STRESSES = 1.44/2.00 = 0.72

SUMMARY M, V, N

$v = V/12(.875)43.5 = V^k/.462$; $\Sigma o = V^k/1.55$; $A_s = M^{\#}/62.6$

JT.			I ICE	II NO ICE	III	IV	SELECT M,V,N	V FACE	ADJ N	V/3/4	FINAL M,V,N	d'/12	Ms	d REQ	t	d	d'	d''	v	Σo	As	N/20	As REQ	As GIVEN
A	AD	M	-250	-94	+10	-128 (.72)	-250			+49	-93 (.72)	-56	-149.	30.5	48"	43.5"	4.5"	19.5"						#8 @
		V	52.5	21.3	0	39	52.5	44.8											74.5	2.98	2.38	1.72	0.66	12
		N	39.8	15.9	1.2(T)	29.7	39.8		4.7		34.4													
	AB	M	+250	-94	+10	-128	-250			+36	-102 (.72)	-71	-173	32.9	"	"	"	"	53.5	2.14	2.76	2.18	0.58	"
		V	39.8	15.9	1.2	29.7	39.8	24.7																
		N	52.5	21.3	0	39	52.5		4.6		43.6													
	+AB	M	+13	+2	NEG.	+11 (.72) +8	+25			-20.1	+0.5	-	+0	0	"	"	"	"	-	-	0	-	0	#6 @
		N	52.5	21.3	0	39	0	-	-		43.6													12
B	BA	M	-250	-160	-123	-174 (.72)	-211			+45	-141 (.72)	-71	-212.	36.4	"	"	"	"	67.8	2.71	3.38	2.18	1.20	#10
		V	39.8	22.7	12.5	36.3	27	31.3																@
		N	52.5	21.3	0	39	0		-		43.6													12
	BC	M	-250	-160	-123	-174	-211			+66.4	-126 (.72)	-69	-194	34.8	"	"	"	"	103	4.05	3.10	2.08	1.02	#10
		V	52.5	35.5	29.7	53	45	46.8											790					@
		N	39.8	22.7	12.5	36.3	27		5.4		41.7								*					12"
	+BC	M	+209	+141	+123	+150 (.72)	+211			-33.3	-127 (.72)	+66	+195	34.4	"	"	"	"	-	-	3.11	2.08	1.03	#9 @
		N	39.8	22.7	12.5	36.3	27		5.4		41.7													12
C	CB	M	-250	-160	-123	-174	-211								"	"	"	"					1.02	#10
		V	52.5	35.5	29.7	53	45	46.8	SAME AS (BC)															@
		N	39.8	22.7	12.5	36.3	27																	12"
C	CD	M	-250	-160	-123	-174	-211								"	"	"	"					1.20	#10
		V	39.8	22.7	12.5	36.3	27		SAME AS (BA)															@
		N	52.5	21.3	0	39	0																	12"
	+CD	M	+13	+2	NEG.	+8	+25		SAME AS (+AB)						"	"	"	"					0	#6 @
		N	52.5	21.3	0	39	0																	12
D	DC	M	-250	-94	+10	-128	-250								"	"	"	"					0.58	#8
		V	39.8	15.9	1.2	29.7	39.8		SAME AS (AB)															@
		N	52.5	21.3	0	39	52.5																	12
D	DA	M	+250	-94	+10	-128	+250								"	"	"	"					0.66	#8
		V	52.5	21.3	0	39	52.5		SAME AS (AD)															@
		N	39.8	15.9	1.2(T)	29.7	39.8																	12
	+DA	M	+209	+86	+10	+115 (.72) +110	+209			-25	+127 (.72) +91	+56	+147	30.4	"	"	"	"	-	-	2.37	1.72	0.65	#8 @
		N	39.8	15.9	1.2(T)	29.7	39.8	-	4.7		34.4													12



SKETCH of REINF. FOR 4 FT WALLS @ EL. 322

* TO BE CHECKED BY UNIV. ILLINOIS STUDY ON BOX CULVERTS

NED FORM 223
27 Sept 49

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

SUBJECT: WEST THOMPSON RES.
COMPUTATION: INTAKE TOWER – DESIGN OF WALLS BETWEEN EL. 310 & 340
COMPUTED BY: Gmt CHECKED BY: RAK DATE: 9/19/62

1-2 :– FLEXURAL FORCES ON 5' WALLS DUE TO EARTHQUAKE FORCES.

CASE VI A :– ERTHQK S→N + CASE II
TAKE ERTHQK FORCES AT EL. 322 (TOP OF PLATFORM)

ΣV	A	My	HYDRODYNAMIC FORCE
110.2	64.5	7107.9	$\pi(20)^2 20.5(.0625).05 = 80.5^K$
137.8	40.0	5512.0	× 10.25'
120.9	17.5	2115.7	825.2 'K
318.0	53.75	17092.5	4136.3 'K
194.8	53.75	10470.5	FLEX. MOM 4961.5 'K
540.0	29.25	15795.0	
330.7	29.25	9673.0	A = 40×2×5 = 400
1080.0	9.0	9720.0	21.5×2×5 = 215
583.2	9.0	5249	615 #
3415.6	24.22	82735.6	$S_x = \frac{31.5(40)^3 - 21.5(30)^3}{6(40)} = 5981.25$ FT³
.05	C.G.		
170.8	×24.22	4136.3	$S_y = \frac{40(31.5)^3 - 30(21.5)^3}{6(31.5)} = 5037.47$ FT³

$$f = \frac{P}{A} \pm \frac{M_y}{S_y}$$

$$f = \frac{3415.6}{615} \pm \frac{4961.5}{5037.47} = 5.554 \pm .985$$

$f_{MAX} = 5.554 + .985 = 6.539$ KSF } ALL COMP.
$f_{MIN} = 5.549 - .985 = 4.669$ KSF }

CASE VI B REV – EARTHQK W→E + CASE II
TAKE ERTHQK FORCES @ EL. 322.

$$f = \frac{P}{A} \pm \frac{M_x}{S_x}$$

$$f = 5.554 \pm \frac{4961.5}{5981.25} = 5.554 \pm .830$$

$f_{MAX} = 5.554 + .830 = 6.380$ KSF } ALL COMP. ok
$f_{MIN} = 5.554 - .830 = 4.724$ KSF } 44.30 PSI

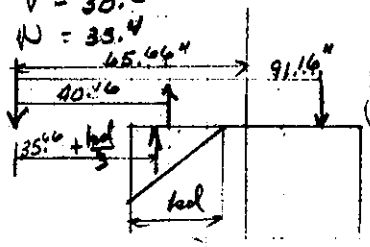
SHEAR = 170.8 + 80.5 = 251.3^K

$$v_v = \frac{251300(1.5)}{615(144)} = 4.26 \text{ psi}$$

HORIZ. STRESSES @ B MEMBER BA
FOR CONDITION II NO ICE

M = 250 − 56.3 = 193.7 'K/'
V = 30.6
N = 35.4

$$e = \frac{M}{N} = \frac{193.7(12)}{35.4} = 65.66''$$



$n A_s = 10(2) = 20$
$(n-1)A_s' = 9(0.44) = 3.96$

$$\frac{kd}{f_c}\left[(16 f_c kd)(35.66 + \frac{kd}{3}) + (40.16)(3.96)(f_c)(\frac{kd - 4.5}{kd}) - (91.16)(20.0)(f_c)(\frac{55.5 - kd}{kd})\right]$$

$2kd^3 + 213.96kd^2 + 159.03kd - 715.65 - 101187 + 1823.2kd$
$kd^3 + 106.98kd^2 + 991.12kd - 50,951.32$
$kd = 16.68$

$$12 f_c(16.68/2 + 3.96\left(\frac{16.68 - 4.5}{16.68}\right)f_c - 20\left(\frac{55.5 - 16.68}{16.68}\right)f_c = 35.4$$

$100.08 f_c + 2.89 f_c - 46.55 f_c = 35400$

$f_s = 20(627.44)\frac{38.82}{16.68} = 14,592$ psi, $f_c = 627.44$

$f_s' = 627.44\left(\frac{12.18}{16.68}\right) = 4578$ psi

PRINCIPAL STRESSES IN COMPRESSION AREA

MAX. SHEAR (DIAG. COMP.) $= \sqrt{\left(\frac{627.44 - 44.3}{2}\right)^2 + 52.0^2} = 287$ PSI

MAX NORMAL STRESS = 335.87 + 287 = 623 PSI comp. ok

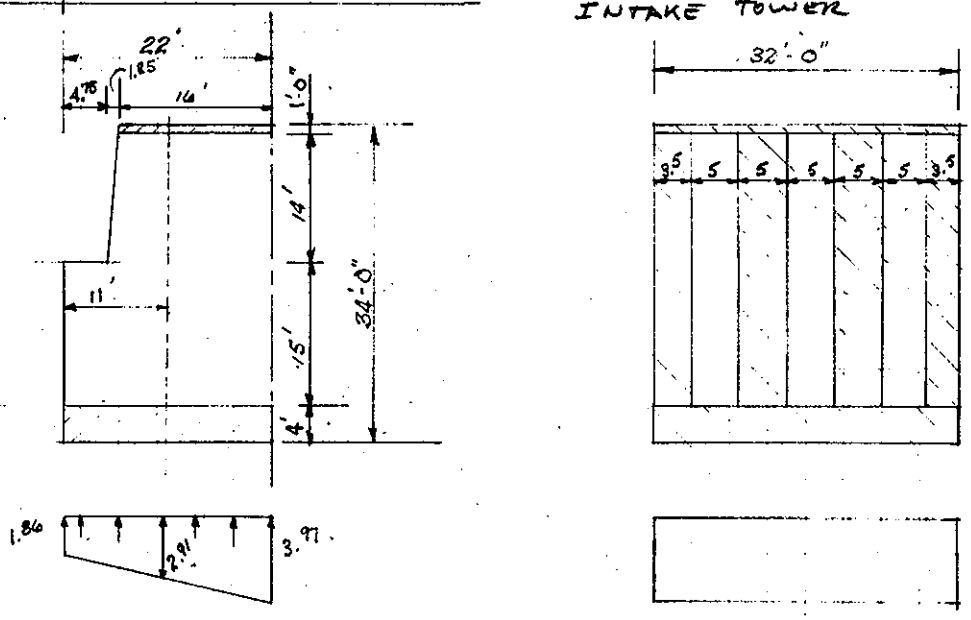
MIN " " = 335.87 − 287 = 49 PSI comp. ok

NED FORM 223
27 Sept 49

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

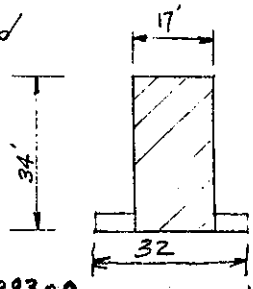
SUBJECT: WEST THOMPSON RES.
COMPUTATION: INTAKE TOWER ~ DESIGN OF FORWARD INTAKE WALLS
COMPUTED BY: R.A.K. CHECKED BY: CMT DATE: Oct. 1962

CASE Ia. CONSTRUCTION CONDITION :- CANTILEVERED FROM FACE OF INTAKE TOWER



No.	FORCE	↓	↑	→	←	Arm	M↻	M↺
Trash Platform	16 x 32 x 1.0 x .150	76.8				8.0		614.4
Trash Walls	.15 x 4 x 2 x 16 x 29	556.8				8.0		4,454.4
"	.15 x 4 x 2 x 6 x 15	108.0				19.0		2052.0
"	.15 x 3 x 3 x 8 x 29	313.2				4.0		1252.8
Trash Bars	.15 x 3 x 1.5 x 29 x 3.5	68.5				15.5		1061.9
Trash Walls	.15 x 4 x 14 x 1.12/2 x 2	9.4				16.37		154.0
Trash Base	.15 x 4 x 32 x 22	422.4				11.0		4,646.4
		1555.1						14,235.9
Bearing Pressure	1.86 x 32.0 x 22.0		1309.4			11.0	14,403.8	
"	2.11 x 22 x 32.0 x 1/2		742.7			7.33	5,444.1	
		1555.1	2052.1		—		19,847.9	14,235.9
Σ			497.0				5,612	

Assumed T-Beam



$M = 5,612.0$, $V = 497$ $29.3^{K/'}$

$KF = 98(5210) = 510,579$ ok

$A_s = \frac{5612.0}{1.57 \times 403 \times 32} = 0.28^{in^2}$. #6@12 BOT.

$\Sigma_o = \frac{29300}{300 \times .885 \times 403} = .27$

$v = \frac{29300}{12 \times j \times 403} = 6.8^{PSI}$

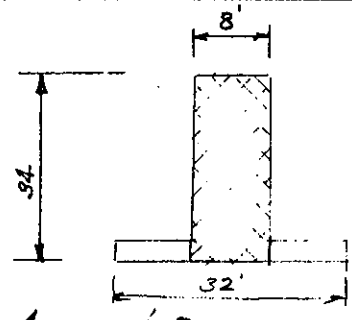
NED FORM 223
27 Sept 49

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

SUBJECT: West Thompson Reservoir
COMPUTATION: Intake Tower ~ Design of Forward Intake Walls
COMPUTED BY: R.A.K. CHECKED BY: Cmt DATE: Oct. 1962

Case Ia. (Consider only the two foot wall section) @ 11'

No.	Force	↓	↑	→	←	Arm	↻	↺
Trash Platform	.15 x 32.° x 1.° x 5.0	24.°				2.5		60.°
Trash Walls	.15 x 4.° x 2.° x 5.° x 29.°	174.°				2.5		435.°
"	.15 x 4 x 2.° x 6.° x 15	108.°				8.°		864.°
Trash bars	.15 x 3 x 1.5 x 29 x 3.5	68.5				4.5		308.2
Trash Walls &	.15 x 4 x 14.° x 1.12/2 x 2.°	9.4				5.37		50.5
Trash Base	.15 x 4.° x 32.° x 11.°	211.2				5.5		1161.6
		595.1						2879.3
Bearing Pressure	1.86 x 11.° x 32.°		654.7			5.5	3,601.0	
"	1.05 x 11.° x 32.° x 1/2		184.8			3.67	678.2	
		595.1	839.5		—		4,279.2	2,879.3
			244.4				1399.9	



Assumed Beam

$M = 1399.9^{K}$, $V = 244.4^{K}$ or $30.55^{K/'}$

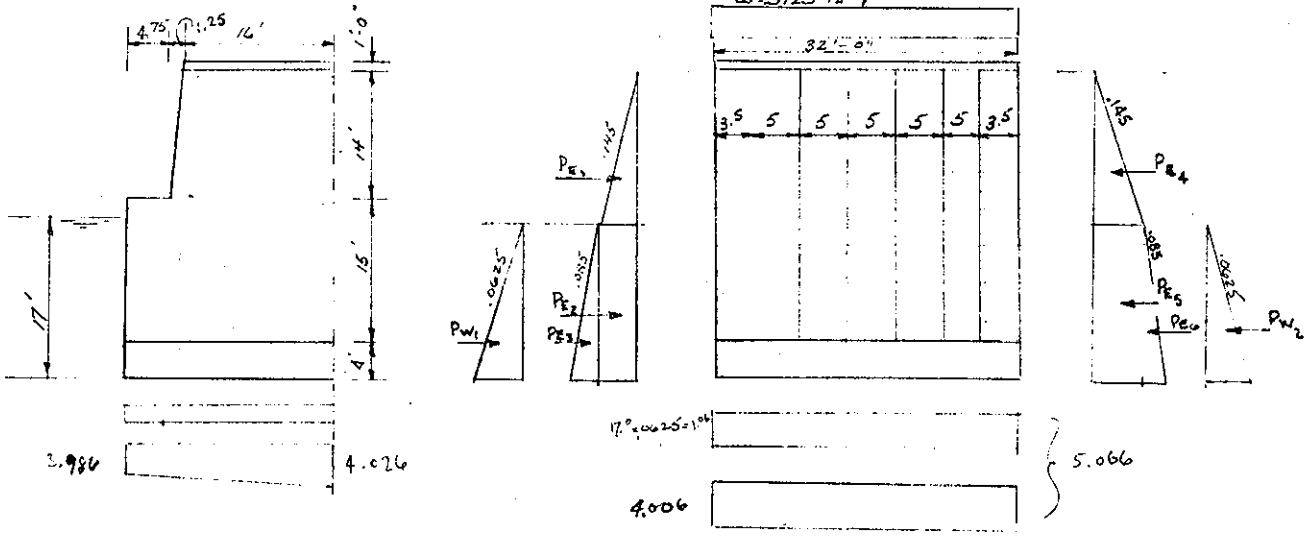
$KF = 98(5210) = 510,579$ ok

$A_s = \frac{1399.9}{1.57 \times 403 \times 32} = 0.07$

$v = \frac{30550}{12(j)403} = 7.1$ PSI

$\Sigma o = \frac{30,550}{210 \times j \times 403} = 0.4$

Case IIIA – Permanent Pool Condition (Taken ± face intake tower)



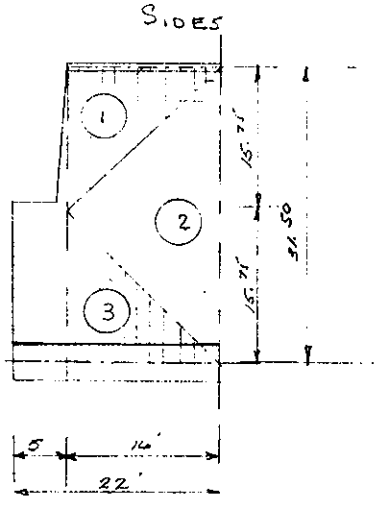
NED FORM 223
27 Sept 49

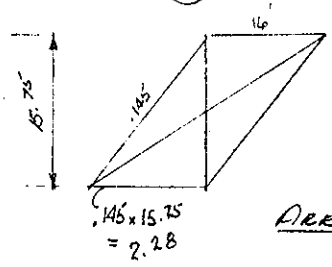
NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U. S. ARMY

SUBJECT West Thompson Res.
COMPUTATION Intake Tower ~ Design of Forward Intake Walls
COMPUTED BY R.A.K CHECKED BY Cmt DATE Oct. 1962

Consider the area of outside walls to be 16' × 31'-6' and the load will be distributed as shown below. —

SIDES





FORCE DIAGRAM

Area triangle = $2.28 \times \frac{15.75}{2}$
= 18.0 k'

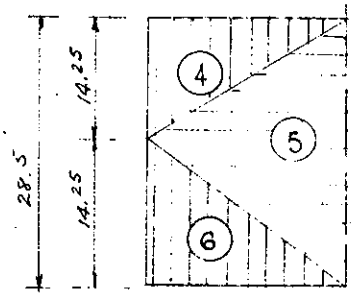
Volume = $\frac{1}{3} \times 18.0 \times 16.0$
= 96.0 k'

AREA DISTRIBUTED

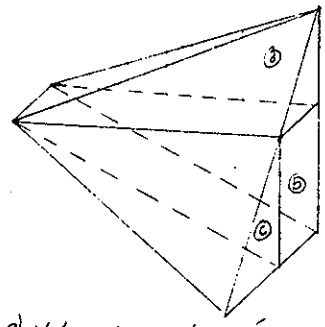
$\frac{1}{2} \times 16 \times 15.75 = 126.0$ □'

$\therefore \frac{96.0}{126.0} = 0.76$ k/'

BASE & TOP



AREA 4 + AREA 6 = AREA 5



a) Vol. = $\frac{1}{3} \times .145 \times 15.75 \times 16.0 \times 15.75$
= 191.8

b) Vol. = $\frac{1}{2} \times .145 \times 15.75 \times 15.75 \times 16.0$
= 287.8

c) Vol. = $\frac{1}{3} \times .145 \times 15.75 \times 15.75 \times \frac{1}{2} \times 16.0$
= 95.9

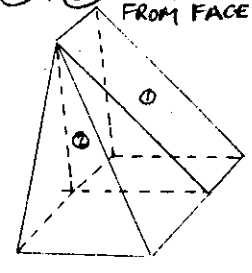
total = 575.5 k

AREA DISTRIBUTED = $31.5 \times 16.0 \times \frac{1}{2} = 252.0$

$\frac{575.5}{252.0} = 2.28$ k/SF

AREA 1 + 3 + 4 + 6 GO INTO FRAME ANALYSIS
AREA 2 + 5 CANTILEVER FROM FACE

3



∴ for Simplicity Assume
1 Top takes 96 k = 8%
2 Side take 575.5 k = 50%
3 Bottom 479.6 k = 42%
1151.1 k

1) Vol. = $\frac{1}{2} \times .145 \times 15.75 \times 15.75 \times 16.0$ = 287.8

2) Vol. = $\frac{1}{3} \times .145 \times 15.75 \times 15.75 \times 16.0$ = 191.8
total = 479.6 k

Area dist. = $15.75 \times \frac{1}{2} \times 16.0 = 126$

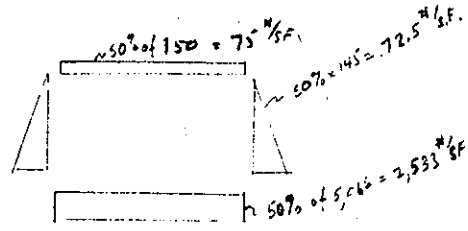
$\therefore \frac{479.6^k}{126} = 3.80$ k/SF

total = 4.5

LOADS FOR FRAME ANALYSIS: —

Assume Intake tower wall to take 50% & rest 50%

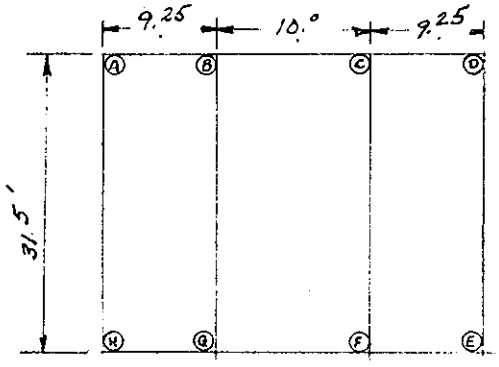
NEW LOADING DIAGRAM



NED FORM 223
27 Sept 49

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

SUBJECT: WEST THOMPSON RES.
COMPUTATION: TRASH RACK ~ TRANSVERSE SECTION @ FACE OF INTAKE TOWER
COMPUTED BY: R.A.K. CHECKED BY: Cmt DATE: Oct. 1962



$K = \frac{I}{L}$, where b & $\frac{1}{12}$ are common factors use $\frac{t^3}{L}$

Member: A-B, C-D $= \frac{\bar{1}^3}{9.25} = 0.1081$

B-C $= \frac{\bar{1}^3}{10.0} = 0.1000$

A-H, D-E $= \frac{3.\bar{5}^3}{31.5} = 1.3611$

B-G, C-F $= \frac{5.0^3}{31.5} = 3.9682$

H-G, F-E, $= \frac{4.\bar{0}^3}{9.25} = 6.9189$

G-F $= \frac{4.\bar{0}^3}{10.0} = 6.4000$

$\frac{K}{\Sigma K} =$

① A-B, D-C = $.1081/.1081 + 1.3611 = .07$
A-H, D-E = $1.361/1.4692 = .93$

② B-A, C-D = $.1081/.1081 + .1000 + 3.9682 = .03$
B-C, C-B = $.1000/4.1763 = .02$
B-G, C-F = $3.9682/4.1763 = .95$

③ H-A, E-D = $1.3611/1.3611 + 6.9189 = .16$
H-G, E-F = $6.9189/8.28 = .84$

④ G-H, F-E = $6.9189/6.9189 + 3.9682 + 6.400 = .40$
G-B, F-C = $3.9682/17.2871 = .23$
G-F, F-G = $6.4000/17.2871 = .37$

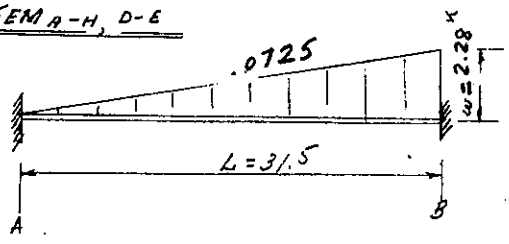
FEM A-B, B-A, C-D, D-C $= .075 \times \frac{9.\bar{2}5^2}{12} = \underline{0.54}^{K'}$

FEM B-C, C-B $= .075 \times \frac{10.\bar{0}^2}{12} = 0.63^{K'}$

FEM H-G, G-H, F-E, E-F $= 2.533 \times \frac{9.\bar{2}5^2}{12} = \underline{18.0}^{K'}$

FEM G-F, F-G $= 2.533 \times \frac{10.\bar{0}^2}{12} = \underline{21.1}^{K'}$

FEM A-H, D-E



For Simplicity analize as shown

$C_{AB} = \frac{1}{30} w L^2 = \frac{1}{30} \times 2.28 \times 31.\bar{5}^2 = \underline{75.5}^{K'}$

FEM H-A, E-D

$C_{BA} = \frac{1}{20} \times 2.28 \times 31.\bar{5}^2 = \underline{113.}^{K'}$

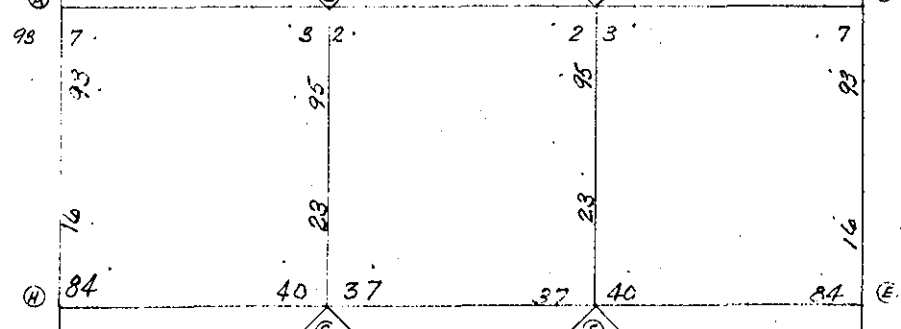
NED FORM 223
27 Sept 49

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

SUBJECT West Thompson Res.
COMPUTATION Trash Rack Walls – Transverse Section @ Face of Intake Tower
COMPUTED BY R.A.K. CHECKED BY Cmt DATE Oct. 1962

Case III_A (cont.) Permanent Pool Condition

Shear	8.12	1.30	-0.60	-0.36	0.38	0.38	+0.36	-0.60	1.30	8.12
	-3.88	+0.95	-0.95	-0.36	0	0	+0.36	-0.95	+0.95	-3.88
	12.-	0.35	0.35	0	0.38	0.38	0	0.35	0.35	12.-
Moment	+6.5	-6.5	-2.3	+2.7	-0.5	+0.5	-2.7	+2.3	+6.5	-6.5
	-2.6	-0.2	+0.2	+4.7	+0.1	-0.1	-4.7	-0.2	+0.2	+2.6
	+2.8	0	-0.3	-4.7	0	0	+4.7	+0.3	0	-2.8
	-7.0	-0.5	+0.	+3.-	+0.	-0.	-3.-	-0.	+0.5	+7.0
	+7.5	0.	-2.7	-0.4	0	0	+0.4	+2.7	0	-7.5
	-69.7	-5.3	0	+0.1	0	0	-0.1	0	+5.3	+69.7
	+75.5	-0.54	+0.54	0	-0.63	+0.63	0	-0.54	+0.54	-75.5



	-113.-	+18.0	-18.0	0	+21.1	-21.1	0	+18.0	-18.0	+113.-
	+15.-	+80.0	-1.2	-0.8	-1.1	+1.1	+0.8	+1.2	-80.0	-15.-
	-34.8	-0.6	+40.-	+0.	+0.6	-0.6	-0	-40.-	+0.6	+34.8
	+5.7	+29.7	-16.2	-9.4	-15.0	+15.0	+9.4	+16.2	-29.7	-5.7
	-3.5	-8.1	+14.8	+1.5	+7.5	-7.5	-1.5	-14.8	+8.1	+3.5
	+1.9	+9.7	-9.5	-5.5	-8.8	+8.8	+5.5	+9.5	-9.7	-1.9
Mom.	-128.7	+128.7	+9.9	-14.2	+4.3	-4.3	+14.2	-9.9	-128.7	+128.7
	24.-	11.7	11.7	0	12.7	12.7	0	11.7	11.7	24.-
	+3.88	+17.33	-17.33	+0.36	0	0	-0.36	-17.33	+17.33	3.88
Shear	+27.88	29.03	-5.63	+0.36	12.7	12.7	+0.36	-5.63	29.03	27.88

"Take average width of outside wall to be 2'-9"

Design

Member A–B

$M = 6.5$, $V = 1.30$, $N = 8.12$ reduced $M = 6.5 - \frac{1}{3} \times 1.12 \times 3.5 = 5.19$ (1.31)

$M_s = M + N \times \frac{d''}{12} = 5.19 + \frac{8.12}{1.01} \times \frac{1.5}{12} = 6.20$ K'

$d = \sqrt{\frac{6.2}{.16}} = 6.2 < 7.5$ ∴ No Comp. A_s req.

$A_s = \frac{12 M_s}{f_s j d} - \frac{N}{f_s} = \frac{6.2}{1.44 \times 7.5} - \frac{8.12}{20.0} = 0.57 - 0.40 = 0.17$ in² Min A_s

SUBJECT: West Thompson Res.

COMPUTATION: Trash Rack Walls ~ Transverse Section @ Face of Intake Tower:

COMPUTED BY: R.A.K. CHECKED BY: Cmt DATE: Oct. 1962

Member B-C

$N = 8.12^k$

$M = 0.5$, $+M = 0.44$, $V = 0.38$, reduced $M = 0.5 - \frac{1}{3} \times \overset{0.97}{0.29} \times 5.0 = 0.03^{k'}$

$A_s = \frac{0.03}{1.44 \times 7.5} = 0$ MIN A_s

Member A-H

$M = 6.5$, $V = 8.12$, $N = 1.30$ reduced $M_{face} = 6.5 - \frac{1}{3} \times \overset{2.6}{7.83} \times 1.0 = 3.9$

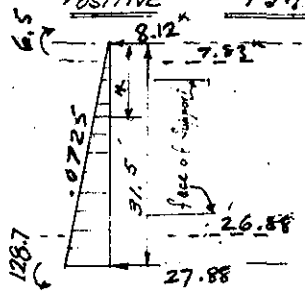
$M_s = 3.9 + 1.30 \times \frac{12.0}{12} = 5.20$

$A_s = \frac{5.2}{1.44 \times 28.5} - \frac{1.30}{20.0} = 0.127 - 0.065 = 0.06^{in^2}$ MIN A_s

$d = \sqrt{\frac{5.2}{.16}} = 5.7^{in} < 28.5$ No Comp. A_s req.

$v = \frac{7830}{12 \times j \times 28.5} = 26$ $\quad \Sigma o = \frac{7830}{210 \times j \times 28.5} = 1.5^{in^2}$

Positive $+M$



Point of zero shear $= (\frac{x^2}{2} \cdot .0725) - 8.12 = 0$ $\therefore x = 15.0'$

$+M = 8.12 \times 15.0 - 6.5 - \frac{15.0^2}{2} \times .0725 \times \frac{15.0}{3} = 122 - 6.5 - 40.8$

$= 74.7^{'k}$

Reduction Factor $= \overset{1.30}{\frac{7.83}{6} \times 1.0} + [\overset{17.92}{26.88} \times \overset{16.67}{\frac{4.0}{6}} - \overset{1.30}{\frac{7.83}{6} \times \frac{4.0}{6}}] \frac{\overset{7.92}{15.0}}{31.5} = 9.22$

$\therefore +M = 74.7 - 9.2 = 65.5^{'k}$ $\quad d = \sqrt{\frac{73.0}{.160}} = 21.4 < 28.5$ $\therefore$ No Comp. Steel req'd.

$N = 1.30 + 15.0 \times 2.75 \times .150 = 7.50^k$

$M_s = 65.5 + 7.5 \times \frac{12.0}{12} = 73.0$

$A_s = \frac{73}{1.44 \times 28.5} - \frac{7.5}{20.0} = 1.78 - 0.38 = \underline{1.40^{in^2}}$ USE #11@12

Moment @ H-A

$M = 128.7$, $N = 1.30 + \overset{13.0}{31.5} \times 2.75 \times .150 = 14.3$, $V = 27.88$

reduced $M = 128.7 - \frac{1}{3} \times \overset{35.9}{26.88} \times 4.0 = 92.8^{'k}$

$M_s = 92.8 + 14.3 \times \frac{12}{12} = 107.1$

$A_s = \frac{107.1}{1.44 \times 28.5} - \frac{14.3}{20.0} = 2.62 - 0.72 = \underline{1.90^{in^2}}$ USE #11@10"

$v = \frac{\overset{54.0}{25580}}{12 \times j \times 28.5} = 85$ $\quad \Sigma o = \frac{25580}{210 \times j \times 28.5} = 4.9$

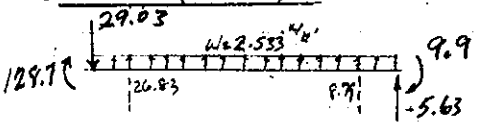
$d = \sqrt{\frac{107.1}{.160}} = 25.9 < 28.5$ $\therefore$ No Comp. steel req'd.

NED FORM 223
27 Sept 49

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

SUBJECT: West Thompson Res.
COMPUTATION: Intake Tower – Design of Forward Intake Walls
COMPUTED BY: R.A.K. CHECKED BY: Cmt DATE: Oct. 1962

Member H-G



$M = 128.7$, $N = 27.88$, $V = 29.03$

reduced $M = 128.7 - \frac{1}{3} \times 27.33 \times 2.75^2 = 103.7^{k'}$

$M_s = 103.7 + \frac{27.88}{45.3} \times \frac{19.5}{12} = 149.0$

$A_s = \frac{149.0}{1.44 \times 43.5} - \frac{27.88}{20.0} = 2.38 - 1.39 = 0.99 \text{ in}^2$

$d = \sqrt{\frac{149.0}{.160}} = 32.5 < 43.5''$

$v = \frac{26,830}{12 \times j \times 43.5} = 58$

$\Sigma o = \frac{26,830}{300 \times j \times 43.5} = 2.33 \text{ in}^2$

∴ No Comp. Steel req'd.

G – H

$M = 9.9$, $N = 27.88$, $V = -5.63$

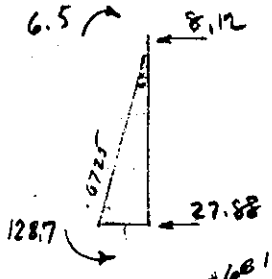
$M_s = 9.9 + \frac{27.88}{45.1} \times \frac{19.5}{12} = 55.0$

$A_s = \frac{55.0}{1.44 \times 43.5} - \frac{27.88}{20.0} = 0.88 - 1.39 = 0$ MIN A_s

Member H-A

Resist. Mom w/ #6@12 = $(.72 \times 1.16)\, 1.44 (47.5 \times 2.5) - 14.3 = 33.2^{'k}$

point where moment = $33.2^{'k}$



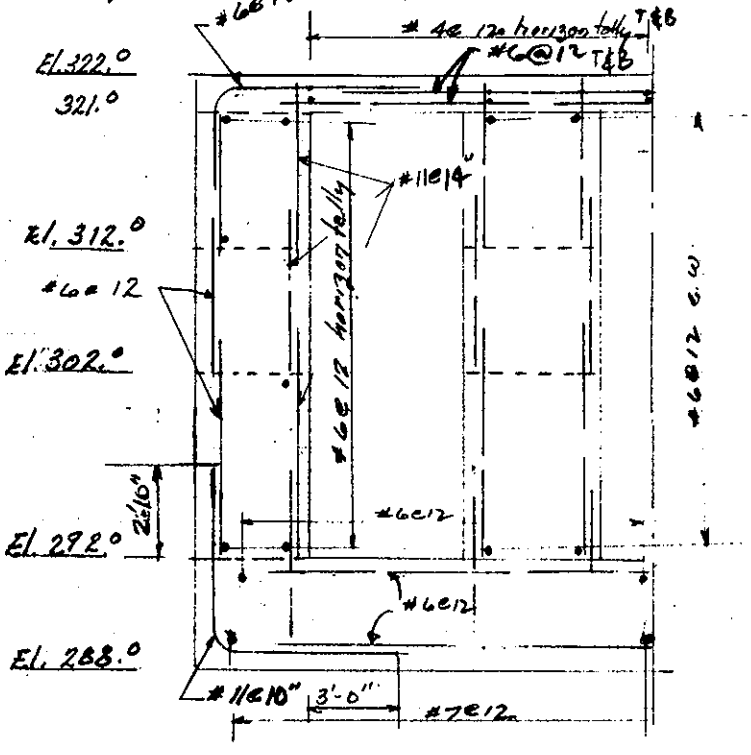
$27.88(x) - 128.7 - 2.284 x^2/2 + \frac{.0725}{6} x^2 \left(2.284 - [31.5 - x]\right) = -33.2$

$x^3 - 123.726 x^2 + 2307.310 x - 7903.448 = 0$

∴ $x = 4.44'$

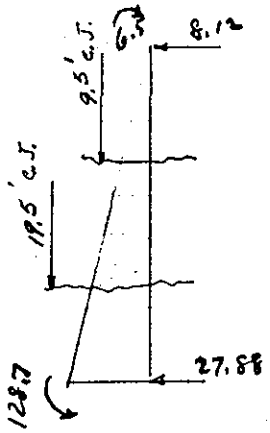
∴ Extend bars 4'-6" beyond ℄ = 4.5 – 2.0 = 2.5' < 24d

∴ USE 24d = 2'-10"



SUBJECT West Thompson Res.
COMPUTATION Intake Tower - Design of Forward Intake Walls
COMPUTED BY R.A.K. CHECKED BY CMT DATE Oct. 1961

Member H-A - Outside Walls



Moment @ c.J. @ El. 312.0

$= 8.12 \times 9.5 - 6.5 - \frac{9.5^2}{2} \times .0725 \times \frac{9.5}{3} = 77.0 - 6.5 - 10.3 = +60.2^{k'}$

$N = 1.30 + 9.5 \times \overset{3.92}{2.75} \times .150 = 5.22^k$

$M_s = 60.2 + 5.22 \times \frac{12}{12} = 67.42^{k'}$

$A_s = \frac{67.42}{1.44 \times 28.5} - \frac{5.22}{20.0} = 1.64 - .26 = \underline{1.38}\ in^2$

Moment @ c.J. @ El. 302.0

$= 8.12 \times 19.5 - 6.5 - \frac{19.5^2}{2} \times .0725 \times \frac{19.5}{3} = 158.0 - 6.5 - 89.5 = 62.$

$N = 1.30 + 19.5 \times \overset{9.05}{2.75} \times .150 = 9.35^{in}$

$M_s = 62.0 + 9.35 \times \frac{12}{12} = 71.35^{'k}$

$A_s = \frac{71.35}{1.44 \times 28.5} - \frac{9.35}{20} = 1.74 - 0.47 = \underline{1.27}\ in^2$

Member AB: Top Slab - Designed for 5 ft Differential Head.

L.L. $= .0625 \times 5 = .3125$
D.L. $= .150 \times 1 = .15$
$0.4625\ k/\#$

$SBM = .463(8)^2/8 = 3.7^{'k/'}$

$d_{REQ} = \sqrt{\frac{3.7}{.16}} = \sqrt{23.1} = 4.8'' < 7.5''$ ok

$A_s = \frac{3.7}{1.44(7.5)} = 0.34\ \#/'$ use #6 @ 12 T&B

$V = .463(8)/2 = 1.85^{k/'}$

$v = \frac{1850}{12(.88)7.5} = 24.$ PSI ok

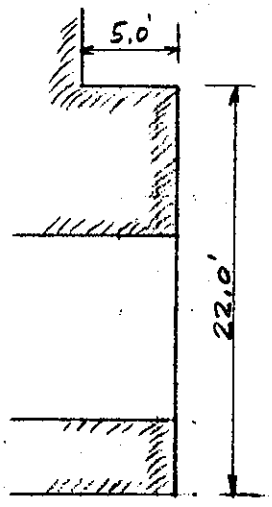
$\Sigma_o = \frac{1850}{210(.88)7.5} = 1.35^{k/'}$

NED FORM 223
27 Sept 49

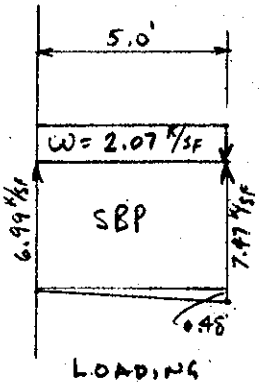
NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U.S. ARMY

SUBJECT: WEST THOMPSON RES. - INTAKE STRUCTURE
COMPUTATION: DESIGN OF DOWNSTREAM TOE
COMPUTED BY: Cmb CHECKED BY: DATE: 10/22/62

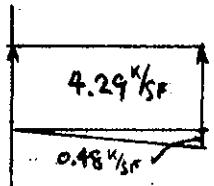
CASE IA GOVERNS FOR BEARING PRES.



ELEV. OF TOE



LOADING



NET LOADING.

WT. OF TOE = 413.2 K ↓
DEDUCT HOLES = 3(5)5(10).15 = 113.2 K ↑
300 K ↓

$w = 300/5\times29 = 2.07$ K/SF.

@ DOWNSTREAM FACE OF TOWER

$M = \overset{53.6}{4.29(5.0)^2/2} + \overset{4.0}{.48\left(\frac{5}{2}\right)\frac{2}{3}(5)} = 57.6$ 'K/'

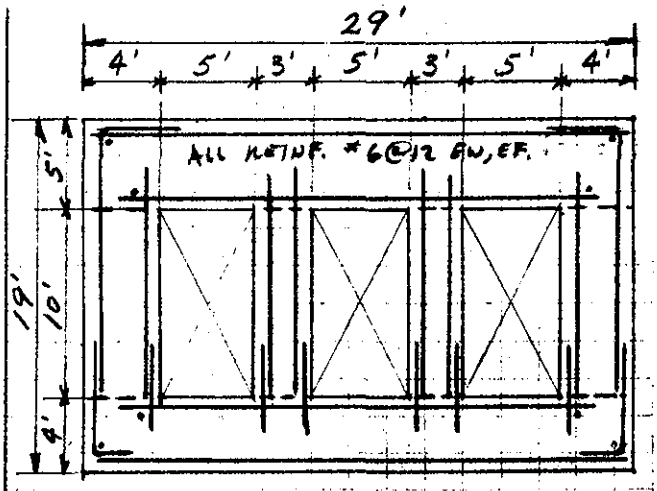
$d_R = \sqrt{\frac{57.6(1000)}{160}} = \sqrt{360} = 19''$, $d_{G,W} = 22(12) - 4.5 = 259.5''$

$A_s = \frac{57.6}{1.44(259.5)} = 0.15$ #/' USE MIN A_s

#6 @ 12

$V = \overset{21.5}{4.29(5)} + \overset{1.2}{.48\,5/2} = 22.7$ K/'

$v = \frac{22700(29)/14}{12(.87)259.5} = 17.4$ PSI ok



BASE: - CASE IA

$SBM_{BASE} = (\overset{4.53}{4.29 + .24})\,5^2/8 = 14.2$ 'K/'

$d_R = \sqrt{\frac{14.2}{.16}} = \sqrt{88.5} = 9.4''$, $d_G = 48 - 4.5 = 43.5''$

$A_s = \frac{14.2}{1.44(43.5)} = 0.22$ #/' USE MIN A_s

$V = 4.53\left(\frac{5}{2}\right) = 11.35$ K/'

$v = \frac{11350}{12(.87)43.5} = 25$ PSI ok

TOP SLAB: - CASE IV

$w = 5(.15) + \frac{731}{5\times29} = .75 + 5.05 = 5.80$ K/SF

$V = 5.80(5)/2 = 14.5$ K/'

$M = 5.80(5)^2/8 = 18.1$ 'K/'

$d_R = \sqrt{\frac{18.1}{.160}} = \sqrt{113} = 10.6''$, $d_G = 60 - 4.5 = 55.5''$

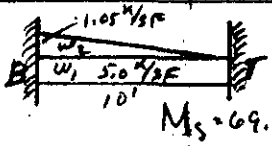
$A_s = \frac{18.1}{1.44(55.5)} = 0.23$ USE MIN A_s

$v = \frac{14500}{12(.87)55.5} = 25$ PSI ok

WALLS: - CASE IV

420 ▯
1176 ▯ △ 420
3406 ▯ △ 625
5002 ▯ 10' △ 1045
w_1 w_2

$W_1 = 5.0(10) = 50.0$ K
$W_2 = 1.05(10/2) = 5.2$ K



$V_{1B} = 25.0$ K $V_{1F} = 25.0$
$V_{2B} = 3.5$ $V_{2F} = 1.7$
$V_B = 28.5$ K $V_F = 26.7$ K

$SBM_1 = 50(10/2) = 62.5$ 'K/' $N = 5.8(6.5) = 37.7$ K/'
$.128(5.2)10 = \frac{6.6}{69.1}$ 'K/' $d_G = 48 - 4.5 = 43.5''$

$M_s = 69.1 + \frac{19.5}{12}(37.7) = 130$ 'K $d_R = \sqrt{\frac{130}{.16}} = \sqrt{814} = 28.5''$ ok

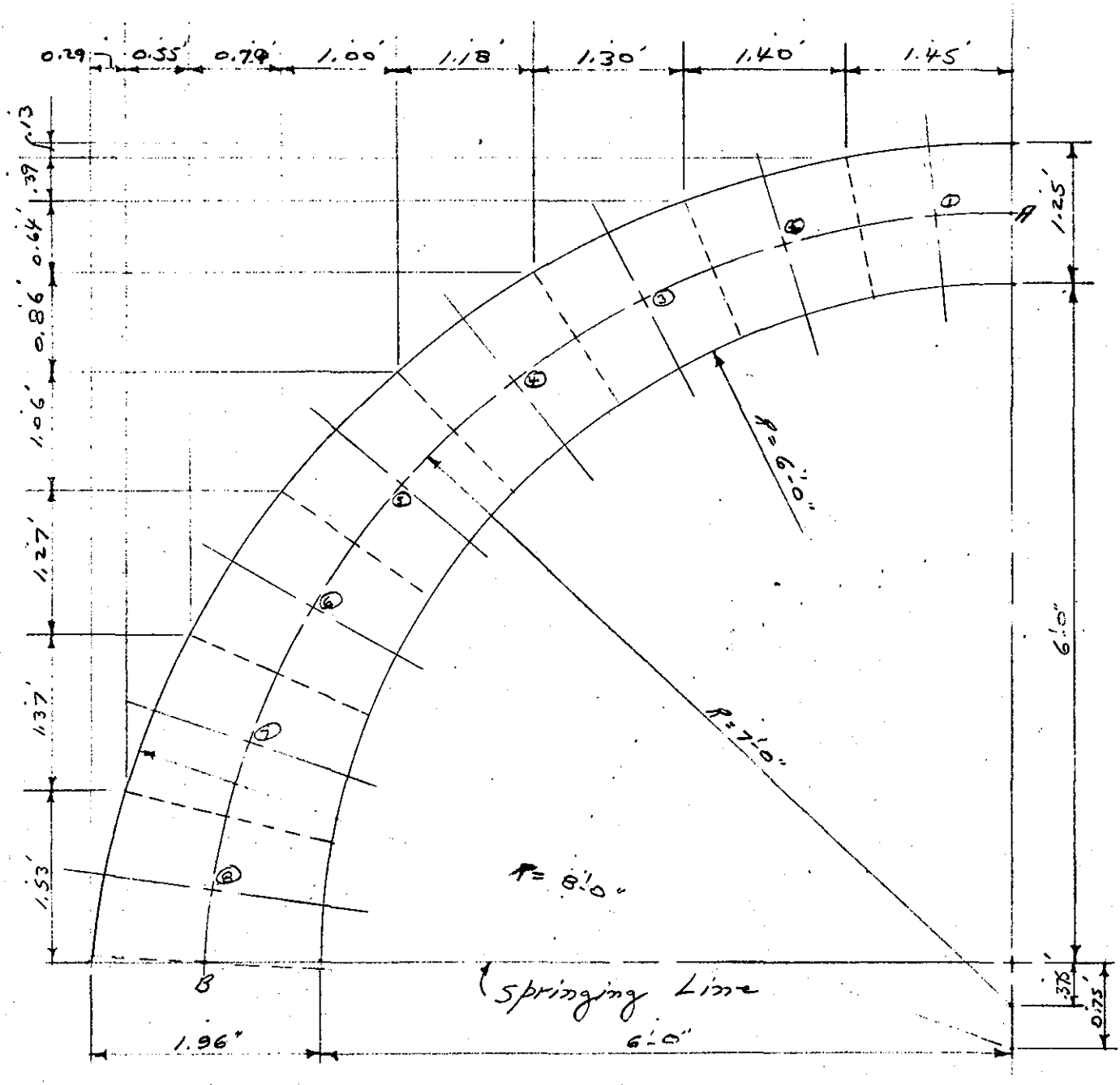
$A_s = \frac{130}{1.44(43.5)} - \frac{37.7}{20} = 2.07 - 1.88 = 0.19$ USE MIN A_s

MIN A_s = #6@12 E.W., E.F.

NED FORM 223
27 Sept 49

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

SUBJECT WEST THOMPSON - CONDUIT
COMPUTATION SECTION AT ℄ DAM
COMPUTED BY R.N.W. CHECKED BY R.A.K. DATE 16 OCT. 1962



Scale: 3/4" = 1'-0"

NOTE: SHOULD ROCK EXCAVATION BE SUCH THAT ASSUMED SPRINGING POINTS CAN NOT BE MAINTAINED, THE CONDUIT MUST BE RE-DESIGNED FOR A NEW SPRINGING POINT OR AS A COMPLETE RING.

SUBJECT WEST THOMPSON - CONDUIT
COMPUTATION SECTION AT ℄ DAM
COMPUTED BY R.N.W. CHECKED BY JWF DATE 7 AUG. 62

Conduit Load Factors:

Horiz. = 0.5
Vert. = 1.0

Assume lateral earth load acts on conduit from top of cond. to middle (voussoirs 1-6) - below this point only hydrostatic forces act.

Est. Unit wts. - earth fill

Saturated — 145 pcf
Moist — 140
Submerged - 83

Amount of fill over conduit

$= 362 - 292.8 - 14 = 55.2'$ say $56'$

Case I - Rapid drawdown condition - Pool at Elev. 305 after drawdown from Spillway Crest at Elev. 342.5

Vert. load $= 1.0[19(140) + 37(145)] = \boxed{8025^{\#}}$.

Horiz. load $= .5(8025) = \boxed{4013^{\#}}$.

Case II - Pool at Elev. 356.5 after development of the steady seepage condition — earth submerged below Elev. 356.5

Vert. load $= 1.0[56(140) + 83(50.5)] + 62.4(50.5) = \boxed{8113^{\#}}$.

Horiz. load $= .5(4962) + 3151 = \boxed{5632^{\#}}$.

For design memo, design for Case I

NED FORM 223
27 Sept 49

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

SUBJECT WEST THOMPSON – CONDUIT
COMPUTATION SECTION AT ℄ DAM
COMPUTED BY R.N.W. CHECKED BY R.A.K. DATE 18 OCT. 1962

SECT.	x	y_1	d_s	d	$a = \frac{d_s}{d^3}$	ay_1	y	ay^2
A		6.63		1.25			+1.61	
1	-0.66	6.59	1.33	1.25	.68095	4.48746	+1.57	1.67847
2	-1.96	6.34	1.33	1.28	.63419	4.02076	+1.32	1.10501
3	-3.20	5.85	1.33	1.34	.55276	3.23365	+0.83	0.38080
4	-4.31	5.14	1.33	1.42	.46450	2.38753	+0.12	0.00669
5	-5.27	4.23	1.33	1.52	.37872	1.60199	-0.79	0.23636
6	-6.05	3.15	1.33	1.64	.30152	0.94979	-1.87	1.05439
7	-6.60	1.95	1.33	1.78	.23582	0.45985	-3.07	2.22258
8	-6.92	0.66	1.33	1.92	.18790	0.12401	-4.36	3.57190
B	-6.98	—		1.96			-5.02	
Σ					3.43636	17.26504		10.25620

$$\frac{\Sigma a y_1}{\Sigma a} = \frac{172.65}{3.436} = +5.024$$

Vertical Loads

↓ Abs

SECT.	Load KSF	Horiz. Proj.	Vert. Load	wt. Vousoir	Total Load	ΣV	$X_n - X_{n-1}$	M_{SV}
A						11.89	.66	232.49
1	8.03	1.45	11.64	0.25	11.89	11.89	1.30	224.65
2	8.07	1.40	11.30	0.26	11.56	23.45	1.24	209.19
3	8.15	1.30	10.60	0.27	10.87	34.32	1.11	180.11
4	8.26	1.18	9.75	0.28	10.03	44.35	.96	142.02
5	8.39	1.00	8.39	0.30	8.69	53.04	.78	99.44
6	8.56	0.79	6.76	0.33	7.09	60.13	.55	58.07
7	8.76	0.55	4.82	0.36	5.18	65.31	.32	25.00
8	8.97	0.29	2.60	0.38	2.98	68.29	.06	4.10
B						68.29	—	0
Σ					68.29			

Vertical Loads

1. 8.025 + .145 (.065)
2. 8.025 + .145 (.325)
3. 8.025 + .145 (.84)
4. 8.025 + .145 (1.59)
5. 8.025 + .145 (2.55)
6. 8.025 + .145 (3.715)
7. 8.025 + .145 (5.035)
8. 8.025 + .145 (6.485)

SECT.	$M_{SV}\,a$	$M_{SV}\,a\,y$	$H \times y$	$M_i = M + Hy$	$M_v = M_{SV} - M_i$	V	H
A	—	—	+66.24	+215.53	+16.96	0	41.14
1	152.98	240.17	+64.59	+213.88	+10.77	11.89	"
2	132.67	175.12	+54.30	+203.59	+5.60	23.45	"
3	99.56	82.63	+34.15	+183.44	-3.33	34.32	"
4	65.97	7.92	+4.94	+154.23	-12.21	44.35	"
5	37.66	-29.75	-32.50	+116.79	-17.35	53.04	"
6	17.51	-32.74	-76.93	+72.36	-14.29	60.13	"
7	5.90	-18.10	-126.30	+22.99	+2.01	65.31	"
8	0.77	-3.36	-179.37	-30.08	+34.18	68.29	"
B	—	—	-206.52	-57.23	+57.23	68.29	"
Σ	513.02	421.89	—	—	—	—	—

$$M = \frac{\Sigma m a}{\Sigma a} = \frac{513.02}{3.44}$$

$$M = +149.29$$

$$H = \frac{\Sigma m a y}{\Sigma a y^2} = \frac{421.89}{10.26}$$

$$H = +41.14$$

SUBJECT WEST THOMPSON CONDUIT

COMPUTATION SECTION AT ℄ DAM

COMPUTED BY R.N.W. CHECKED BY R.A.K. DATE 19 OCT. 62

Horizontal Loads

load at crown = 4013 #

① 4.013 + .1037(.065) = 4.02
② " + .1037(.325) = 4.05
③ " + " (.84) = 4.10
④ " + " (1.59) = 4.18
⑤ 4.013 + .1037(2.55) = 4.28
⑥ " + " (3.715) = 4.40
⑦ " + " (5.035) = 4.54
⑧ " + " (6.485) = 4.69

SECT.	Load KSF	Vert. Proj.	Horiz. Load	ΣH	$y_n - y_{n-1}$	M_{SH}	$M_{SH} \cdot a$	$M_{SH}\, ay$	$H \times y$	$M_c = M + Hy$	$M_H = M_S - M_c$	H_H
A	—	—	—	-31.84	.04	-140.60	—	—	-39.46	-136.25	-4.35	+7.33
1	4.02	0.13	0.52	-31.84	0.25	-139.33	-94.88	-148.96	-38.48	-135.27	-4.06	+6.81
2	4.05	0.39	1.58	-31.32	0.49	-131.37	-83.31	-109.97	-32.35	-129.14	-2.23	+5.23
3	4.10	0.64	2.62	-29.74	0.71	-116.02	-64.13	-53.23	-20.34	-117.13	+1.11	+2.61
4	4.18	0.86	3.59	-27.12	0.91	-94.90	-44.08	-5.29	-2.94	-99.73	+4.83	-0.98
5	4.28	1.06	4.54	-23.53	1.08	-70.23	-26.60	+21.01	+19.36	-77.43	+7.20	-5.52
6	4.40	1.27	5.59	-18.99	1.20	-44.81	-13.51	+25.27	+45.83	-50.96	+6.15	-11.11
7	4.54	1.37	6.22	-13.40	1.29	-22.02	-5.19	+15.94	+75.25	-21.54	-0.48	-17.33
8	4.69	1.53	7.18	-7.18	0.66	-4.74	-0.89	+3.88	+106.86	+10.07	-14.81	-24.51
B	—	—	—		—	—			+123.04	+26.25	-26.25	-24.51
Σ	—	—	—		—	—	-332.59	-251.35	—	—	—	—

$$M = -\frac{332.59}{3.44} = -96.79 \qquad H = -\frac{251.35}{10.26} = -24.51$$

Summary of Moments, thrusts, etc.

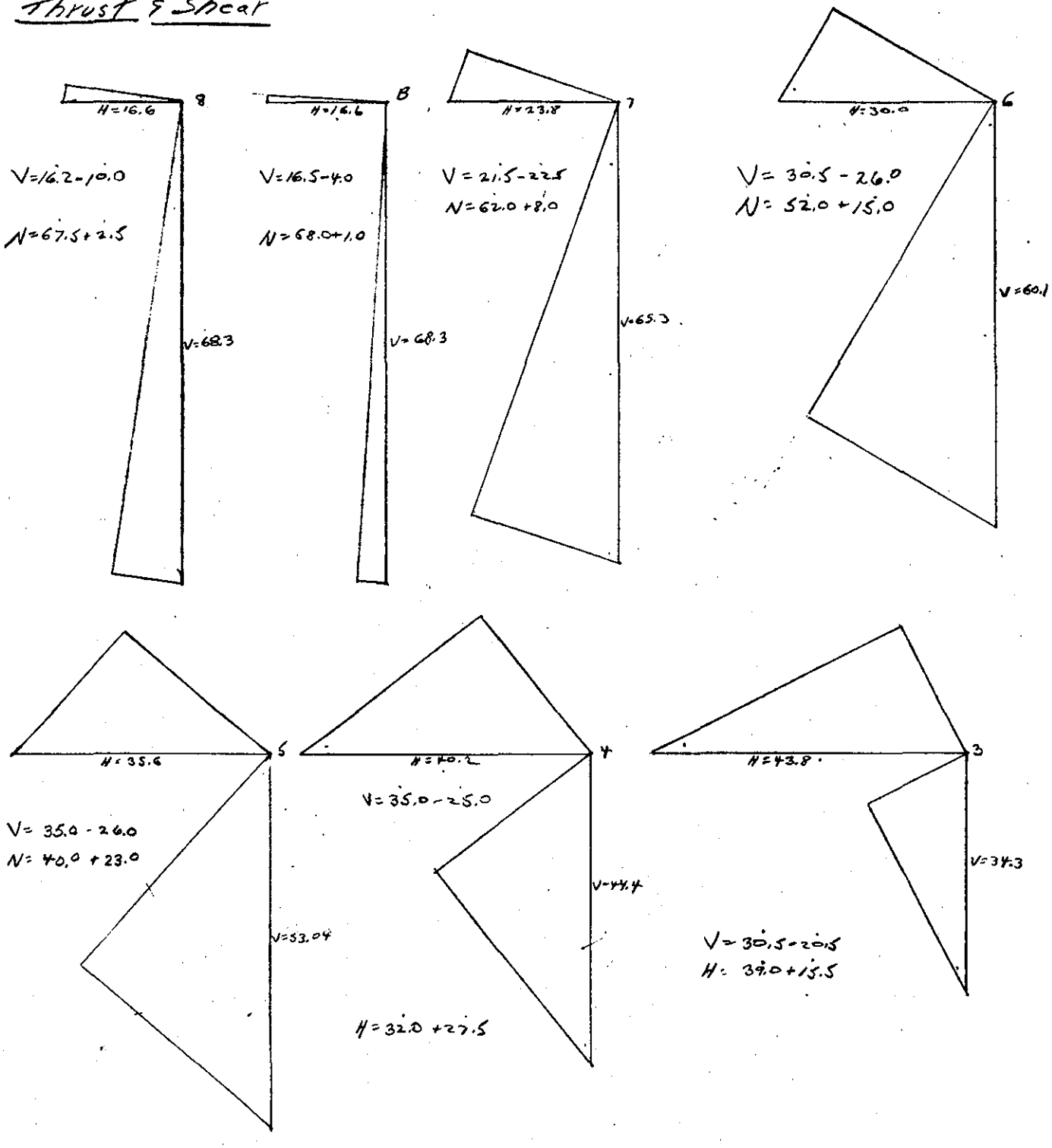
SECT.	M_V	M_H	H_V	H_H	H	V	M	N	V
A	+16.96	-4.35	41.14	+7.33	48.47	0	+12.61	48.47	0
1	+10.77	-4.06	"	+6.81	47.95	11.89	+6.71	49.0	7.5
2	+5.60	-2.23	"	+5.23	46.37	23.45	+3.37	51.2	9.5
3	-3.33	+1.11	"	+2.61	43.75	34.32	-2.22	54.5	10.0
4	-12.21	+4.83	"	-0.98	40.16	44.35	-7.38	59.5	10.0
5	-17.35	+7.20	"	-5.52	35.62	53.04	-10.15	63.0	9.0
6	-14.29	+6.15	"	-11.11	30.03	60.13	-8.14	67.0	4.5
7	+2.01	-0.48	"	-17.33	23.81	65.31	+1.53	70.0	1.0
8	+34.18	-14.81	"	-24.51	16.63	68.29	+19.37	70.0	6.2
B	+57.23	-26.25	"	-24.51	16.63	68.29	+30.98	69.0	12.5

NED FORM 223
27 Sept 49

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

SUBJECT WEST THOMPSON CONDUIT
COMPUTATION SECTION AT ℄ DAM
COMPUTED BY RNW CHECKED BY R.A.K. DATE 22 OCT. 62

Thrust & Shear

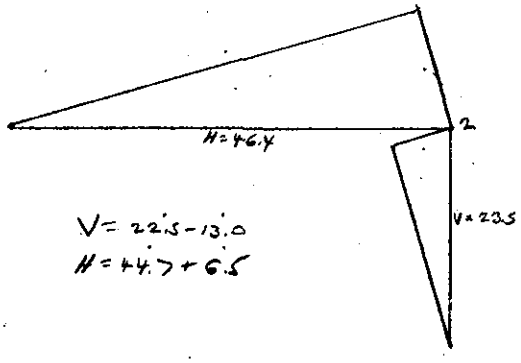


NED FORM 223
27 Sept 49

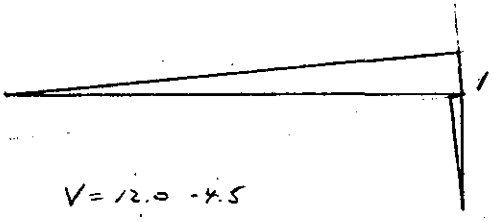
NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U. S. ARMY

SUBJECT WEST THOMPSON CONDUIT
COMPUTATION SECTION AT ℄ DAM
COMPUTED BY F.N.W. CHECKED BY R.A.K. DATE 22 OCT. 62

Thrust & Shear



V= 22.5-13.0
H=44.7+6.5



V= 12.0 -4.5
H= 47.8 + 1.2

pt A – N = 48.5"
V= 0

SUBJECT WEST THOMPSON CONDUIT
COMPUTATION SECTION AT ℄ DAM
COMPUTED BY R.N.W. CHECKED BY R.A.K. DATE 24 OCT. 62

Point A

$t = 15''$
$d = 10.5''$

$f_c' = 3000$ psi
$f_c = 1350$ psi
$K = 236$

$M = 12.61$
$N = 48.47$

$S = \frac{12(15)^2}{6} = 450$

$f = \frac{P}{A} \pm \frac{M}{S} = \frac{48470}{12(15)} \pm \frac{12(12610)}{450} = 269 \pm 336 = 605$ psi comp.
67 psi tens.

check section as eccentrically loaded column

$e' = \frac{12(12.61)}{48.47} = 3.12$ $\qquad \frac{e'}{t} = \frac{3.12}{15} = 0.208$

Reference: M.C. Design Handbook

$\frac{1000\,N}{f_c\,bt} = \frac{1000(48.47)}{1350(12)(15)} = 0.199$

$\frac{1000\,Ne'}{f_c\,bt} = 0.199(.208) = 0.041$

$\therefore p = 0$ no steel required

Point B

$t = 23.5$
$d = 19.0$
$d'' = 7.25$

$M = 30.98$
$N = 69.0$
$V = 12.5$

$S = \frac{12(23.5)^2}{6} = 1105$

$f = \frac{69000}{12(23.5)} \pm \frac{12(30980)}{1105} = 245 \pm 336 = 581$ psi comp.
91 psi tens.
ok with min reinf

$e' = \frac{12(30.98)}{69} = 5.39$ $\qquad \frac{e'}{t} = \frac{5.39}{23.5} = 0.229$

$\frac{1000\,N}{f_c\,bt} = \frac{1000(69)}{1350(12)(23.5)} = 0.181$ $\qquad \frac{1000\,Ne'}{f_c\,bt} = .181(.229) = .041$

no steel required

$v = \frac{3(12500)}{2(12)(23.5)} = 66$ psi

Voussoir #3 $\quad V = 10'' \quad t = 16''$

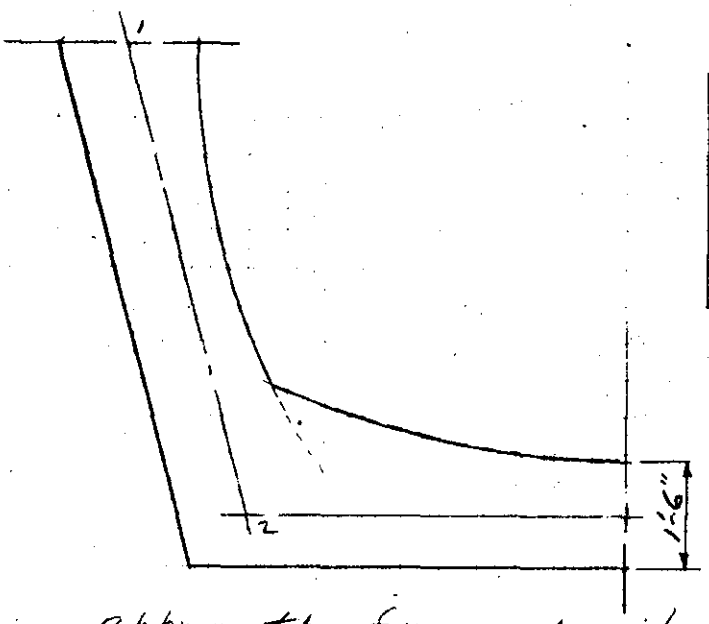
$v = \frac{3(10000)}{2(12)(16)} = 78$ psi ok

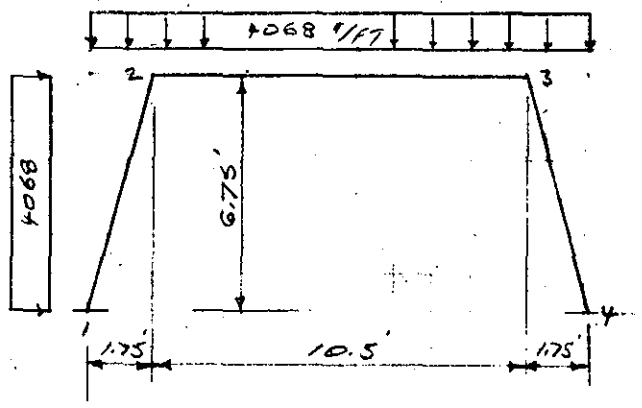
SUBJECT WEST THOMPSON CONDUIT
COMPUTATION SECTION AT ℄ DAM
COMPUTED BY R.A.W. CHECKED BY R.A.K. DATE 29 OCT. 62

Conduit below springing line

Consider the half conduit as a frame fixed at the springing line and subjected only to hydrostatic head.
max. head = 356.5' - 291.3 = 65.2'

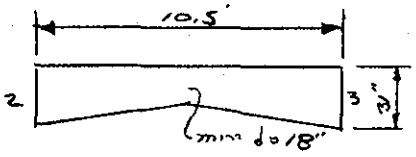
$w = 62.4(65.2) = 4068$ #/FT²





Idealized Frame and Loading

Approx the frame by idealizing the loading, frame shape & members as shown. Use as reference PCA pamphlet "Concrete Beams & Columns with Variable Moment of Inertia"

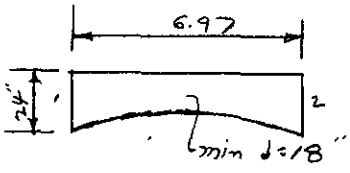


$a = 0.5 \qquad b = \left(\frac{1.5}{2.58}\right)^3 = 0.20$

from chart 8-11: $K_2 = K_3 = 9.5 \qquad K = \frac{9.5(1.5)^3}{10.5} = 3.05$

from chart 8.2 $c = 0.65$

" " 8.3 $f = .099 \qquad fem = .099(10.5)^2(4.068) = 44.4$ K-FT



$a = 0.5 \qquad b = \left(\frac{1.5}{2}\right)^3 = .42$

$K_1 = K_2 = 6.3 \qquad K = \frac{6.3(1.5)^3}{6.97} = 3.05$

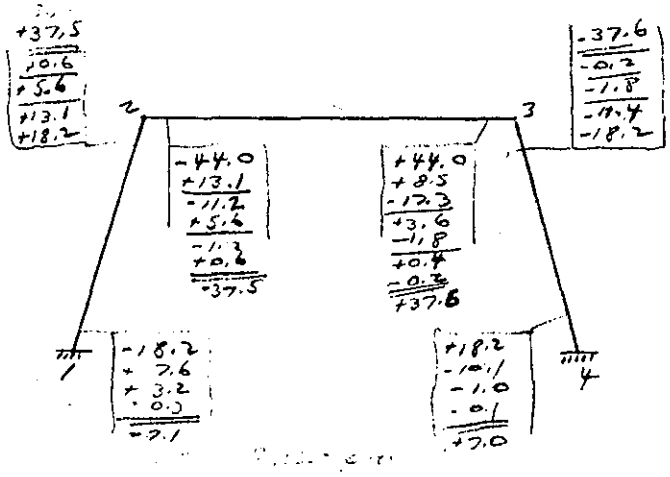
$c = 0.58 \qquad f = .092$

$fem = .092(6.97)^2(4.068) = 18.2$ K-FT

SUBJECT WEST THOMPSON CONDUIT

COMPUTATION SECTION AT ℄ DAM

COMPUTED BY R.N.W. CHECKED BY R.A.K. DATE 30 OCT. 62



$+M_{2\text{-}3} = 4.068(10.5)^2\frac{1}{8} - 37.5 = 18.5$ K-FT

Member 1-2

approx. $N = 4.068\left(\frac{10.5}{2}\right) = 21.4^k$

pt. 2 $M_s = 37.5 + \frac{7.5}{12}(21.4) = 50.9$ K-FT

$A_s = \frac{12(50.9)}{20(.866)(19.5)} - \frac{21.4}{20} = 1.81 - 1.07 = 0.74$ use #8 @ 1'-0"

Member 2-3

approx. $N = 4.068(6.75)\frac{1}{2} = 13.7^k$

pt. 2 $M_s = 37.5 + \frac{11}{12}(13.7) = 50.1$

$A_s = \frac{12(50.1)}{20(.866)(26.5)} - \frac{13.7}{20} = 1.31 - 0.68 = 0.63$ IN² use #7 @ 1'-0" outside

mid span $M_s = 18.5 + \frac{4.5}{12}(13.7) = 23.6$ K-FT

$A_s = \frac{12(23.6)}{20(.866)(13.5)} - \frac{13.7}{20} = 1.21 - 0.68 = 0.53$ IN² use #7 @ 1'-0" inside

$\varepsilon_o = \frac{5.25(4.068)}{300(.866)(26.5)} = 3.10$ use #8 @ 1'-0" at point 2

Member 1-2

pt. 2 $\varepsilon_o = \frac{4.068(6.75)\frac{1}{2}}{300(.866)(\ldots)} = 2.91$ #7 @ 1'-0" OK

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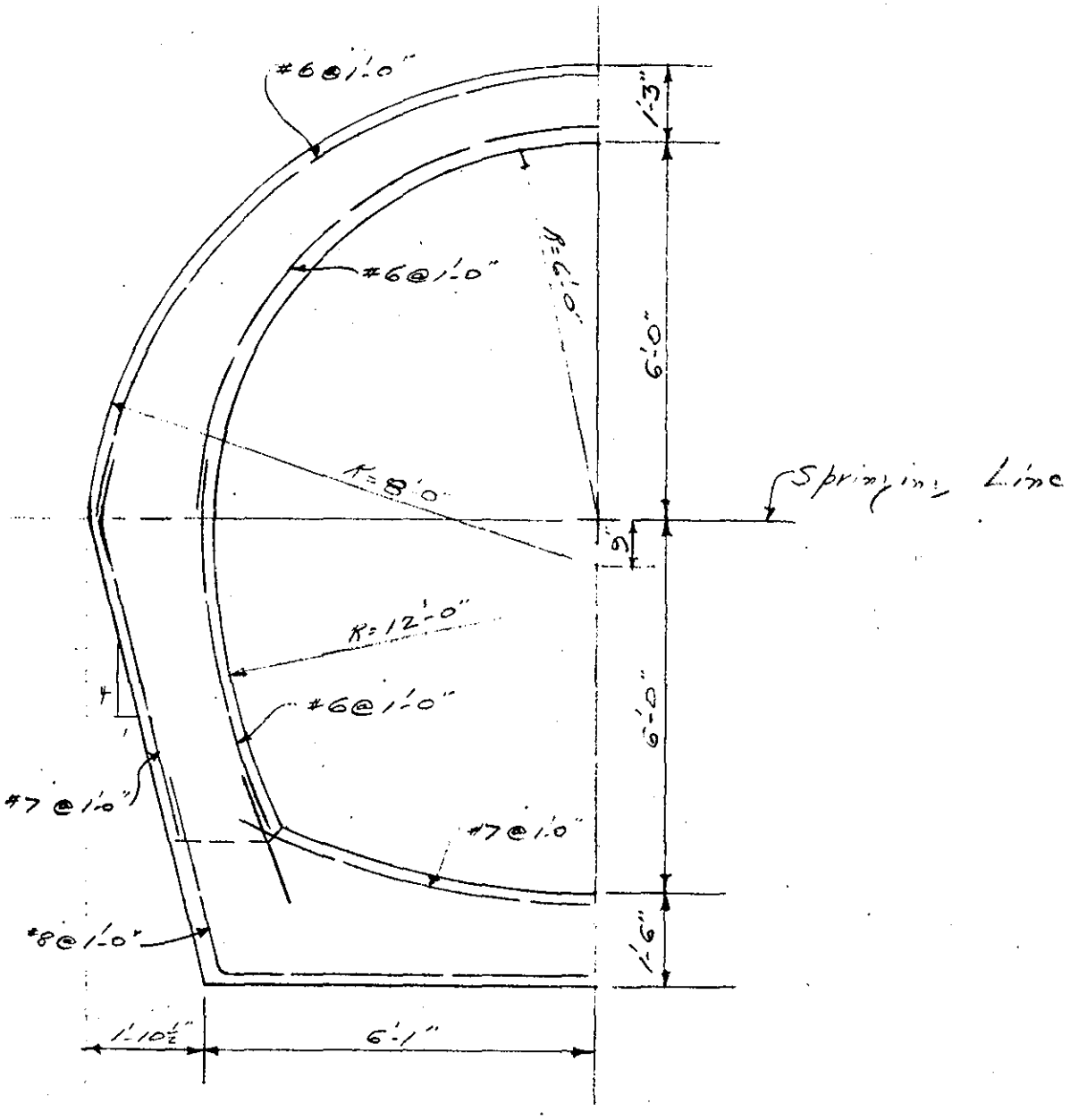
SUBJECT WEST THOMPSON CONDUIT
COMPUTATION SECTION BT ℄ DAM
COMPUTED BY R.N.W. CHECKED BY DATE 30 Oct 52



Scale: 3/8" = 1'-0"

SUBJECT West Thompson Dam
COMPUTATION Service Bridge
COMPUTED BY GER. CHECKED BY JWF DATE 8/2/62

Slab Design

Stringers @ 7'-6" c.c.

$Impact = \frac{50}{7.5+125} = 0.38$ Use 30% max.

Wheel load $= 1.3 \times 16. = 20.8^k$.

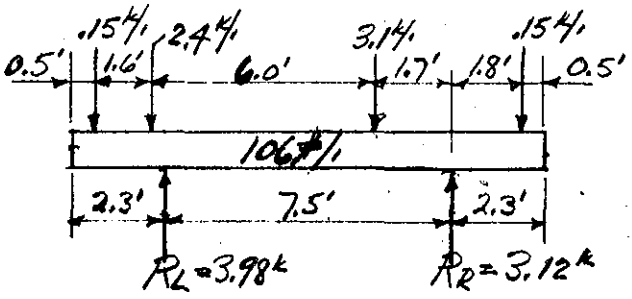
H20 Loading

1961 AASHO Specs.
$f'_c = 3000.$
$f_c = 1200.$
$f_s = 20000.$ - reinf.
$f_s = 18000.$ - steel girders

Distribution — $S = 7.5'$ $E = .4 \times 7.5 + 3.75 = 6.75'$

$\frac{20.8}{6.75} = 3.1^k/'$ $\frac{16.}{6.75} = 2.4^k/'$

Curb $- 150 \times .83 \times 1.08 = 135.$ -
Rail $= 15.$
$150.\ \#/'$



$8\frac{1}{2}$" slab avg. $= 106.\ \#/\square'$

$R_L = .15 + 6.1 \times .106 = 0.80$
$3.1 \times 1.7/7.5 = 0.71$
$2.4 \times 7.7/7.5 = 2.47$
3.98^k

$R_R = 1.6 + 2.4 + 3.1 - 3.98 = 3.12^k$.

$+M = \overset{5.3}{3.12 \times 1.7} - \overset{.52}{.15 \times 3.5} - \overset{.85}{.106 \times 4 \times 2} = 4.0^{'k}$

Per '61 AASHO — $+M = \frac{9.5}{32} \times 20.8 = 6.2^{'k}$

Dead ld. $+M = \overset{3.0}{.80 \times 3.75} - \overset{.84}{.15 \times 5.6} - \overset{1.94}{.106 \times 6.1 \times 3} = +0.2^{'k}$

Total $+M = 6.2 + .2 = 6.4^{'k}$.

Slab is 9" at ℄ & 8" at ends.

$d = \sqrt{\frac{6400}{147}} = 5.7" + 2.0"(cov.) + .7"(wear) = 8.4" < 8.7"$ ok.

$+A_s = \frac{6.4 \times 12}{20 \times .875 \times 6.8} = .64$ — #5 @ 6" bot. $= .62\ \square"$ ok.

For -M see P. 2

SUBJECT West Thompson Dam
COMPUTATION Service Bridge
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Slab design (cont.)

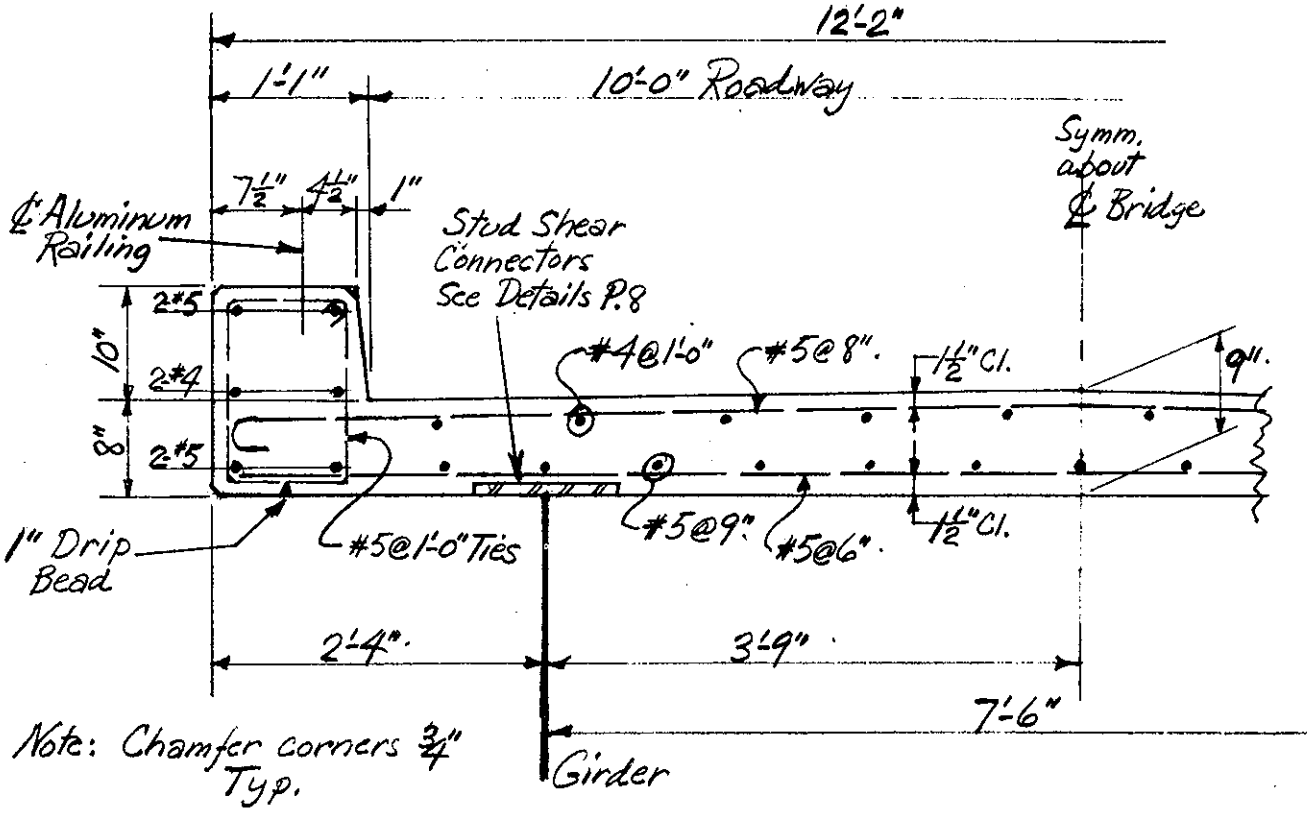
– M at girders: .24 × 1.2 = .29
.15 × 1.8 = .27
3.10 × 0.3 = .94
V = 3.49k M = 1.50 ik

#5@8 Top – $u = \frac{3490}{2.9 \times 7 \times 6.1} = 225 < 300$. ok.

∴ Use #5@8 Top bars ←

Longtd. As : $A_s = \frac{220}{\sqrt{7.5}} = 80\%$ Use 67% max. ✓.

.67 × .62 = .41 □" #5@9 bot = .41 parallel to traffic. ✓
#4@12 top — " " "



Note: Chamfer corners ¾" Typ.

Bridge Cross Section
¾" = 1'-0"

SUBJECT West Thompson Dam
COMPUTATION Service Bridge
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Design of Composite Beams

7'-6" c.c. — 60'-0" span c.c. bearings

Dead Load

Slab – 106 × 6.08 = 650.
Curb + rail = 150.
Beam – = 160.
Bracing etc. = 40.
1.0 k/l.

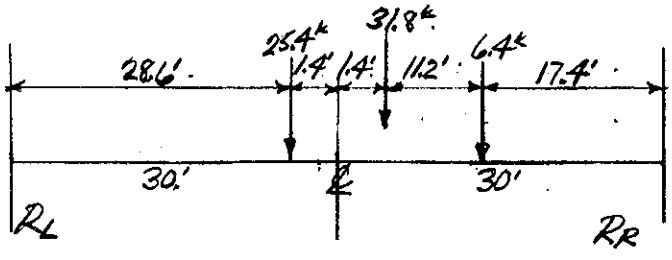
$M = \frac{1}{8} \times 1.0 \times 60^2 = 450.^{1k}$

Live Load

$\text{Impact} = \frac{50}{60+125} = .27$
185

Max. load = $\frac{1.7+7.7}{7.5}$ = 1.25 × wheel load on one bm.
(See P. 1)

Rear wheel = 16 × 1.25 × 1.27 = 25.4^k
Front " = 4 × " " = 6.4
} 31.8^k



$R_L = \frac{31.8 \times 28.6}{60.} = 15.2^k$

$+M = 15.2 \times 28.6 = \underline{435.}^{1k}$ max.

By Lane Loading – From AASHO-Appendix A – 10' lane ld = 558.^{1k}

$+M = \frac{558}{2} \times \frac{7.5}{7.0} \times 1.27 = 380.^{1k}$ – 435.^{1k} is max.

∴ Truck loading governs.

Dead ld. M = 450.^{1k}

Live ld. M = 435.^{1k}

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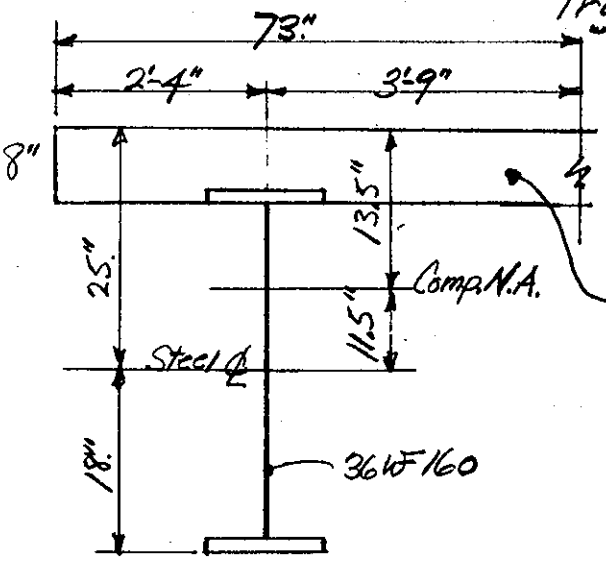
Beam (cont.)

Try 36WF 160
Composite section.



Beam depth

$\frac{62 \times 12}{25} = 30"$ 36WF ok.

Trans. Conc. $A = \frac{(73 \times 8) - 11}{10} = \underline{\underline{57.}}\ \square"$

$57 \times 4 = 228.$
$47.1 \times 25 = 1180.$
$104.1 \qquad 1408./104.1 = 13.5"$

$7.3 \times \frac{8^3}{12} = 300.$
$57 \times 9.5^2 = 5100.$
$47.1 \times 11.5^2 = 6250.$
$9750.$
$I(n=10) = 21400.$

$f_{DL} = \frac{12 \times 450.}{541.} = 10.0\ ^k/\square"$ top & bot.

$f_{LL} = \frac{12 \times 435 \times 6.5 \times 29.5}{21400} = 1.6$ top, 7.2 bot.

f_s top flg $= 10.0 + 1.6 = 11.6\ ^k/\square"$ ok.
f_s bottom flg $= 10.0 + 7.2 = 17.2\ ^k/\square"$

$f_{Conc.} = \frac{12 \times 435 \times 13.5}{21400 \times 10} = 330.\ ^\#/\square"$ ok.

Deflections

Dead ld. $\Delta = \frac{5}{384} \times \frac{1.0 \times \overset{13.0 \times 10^6}{60^4} \times 1728}{29000 \times 9740.} = 1.04"$ Camber 2"

Live load

Lane ld $= 640 \times \frac{1}{2} \times 1.27 = 410.\ ^\#/'$
$18 \times$ " " $= 11.5^k$

$\frac{12^3}{48} = \frac{36}{1}$

$\Delta = \frac{22.5 \times .41 \times \overset{13.0 \times 10^6}{60^4}}{29000 \times 21400} + \frac{36 \times 11.5 \times \overset{21.6 \times 10^4}{60^3}}{29000 \times 21400} = .195 + .145 = .34"$

Allow $= \frac{60 \times 12}{800} = .90"$ Actual $= \frac{.34}{60 \times 12} = \frac{1}{2120}$ ok.

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Beam (cont.)

Bearing at supports

Live load —

By truck (see P.3) $= 25.4 + 6.4 \times \frac{46}{60} = 30.3^k$. ← max.

By lane load $= 410^{\#/'}$ (P.4) $\times 30' + 26^k \times \frac{1}{2} \times 1.27 = 12.3 + 16.5 = 28.8^k$

Dead load $= 1.0 \times 31' = 31.^k$.

Total Dead + live $= 31 + 30.3 = 61.3^k$. ←

Shear $v = \frac{61300}{32.25 \times .65} = 2920.\#/\square''$ – allow $= \frac{3}{4} \times 11000 = 8250.$ ok.

Web crippling $= \frac{61300}{.69(7 + 1.9)} = 10000.\#/\square'' <$ allow. $= 24000 \times \frac{18}{20} = 21600.$ ok.
6.15

∴ End stiffeners not required.

Dead load beam stresses before concrete has set

$f_{DL} = \frac{12 \times 450.}{541.} = 10.0^k/\square''$ top & bot. flgs.

Bms. braced by 18Es @ 12'± c.c. ∴ $b = 12'' \times 2 = 24.''$ if both bms. moved laterally in same direction.

$\frac{ld}{bt} = \frac{60 \times 12 \times 30}{24 \times 1} = 1080.$ $f_b = \frac{12000000}{1080} = 11.1^k/\square'' > 10.0$ ok.

Load at Supports

Dead = 2 × 31	=	62.0
Live	=	45.0 (no imp.)
Shoes	=	1.0
		108.0 k

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Design of welded studs - Pg 114-117

$\frac{3}{4}$"ϕ x 4" high - $\frac{H}{d} = \frac{4}{.75} = 5.3 > 4.2$ ✓

$Q_{ue} = 330 \times .75^2\sqrt{3000} = 10200.^{\#}$ per stud ✓

Safety Factor of 4 - Resist. $Q = \frac{10200}{4} = 2550.^{\#}$ per stud. ✓

At support

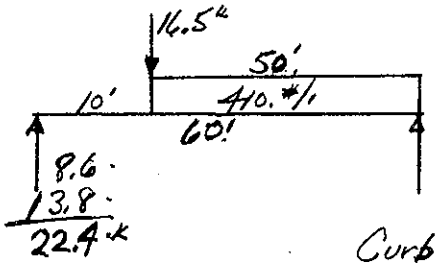
Live ld. $V = 30.4^k$ (P.5)
Curb $= 4.6$
$35.^k$

Assume curb load possibly added after concrete has set.
$= 150 \times 30 = 4.5^k$

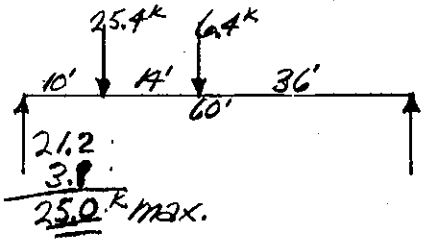
$S = \frac{35000 \times 57 \times 9.5}{21400.} = 890.^{\#}/''$ ✓ 3 Studs - Spacing $= \frac{3 \times 2550}{890} = 8.6''$ ✓

At 10' from support

Lane loading



Truck loading



Curb $= 4.5 - .15 \times 10' = 3.0^k$ Total $= 25.0 + 3.0 = 28.0^k$.

3 Studs - Spacing $= 8.6 \times \frac{35.}{28.0} = 10.7''$ ✓

At 20' from support

Lane

$V = .41 \times 40 \times \frac{20}{60} + 16.5 \times \frac{40}{60}$

$= 5.5 + 11.0 = 16.5^k$

Truck

$V = 25.4 \times \frac{40}{60} + 6.4 \times \frac{26}{60} = 17.0 + 2.8 = 19.8^k$ ✓

Curb $= 4.5 - .15 \times 20' = 1.5$. Total $= 19.8 + 1.5 = 21.3^k$

Spacing $= 8.6 \times \frac{35}{21.3} = 14.1''$ ✓

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At ℄ of span

Lane

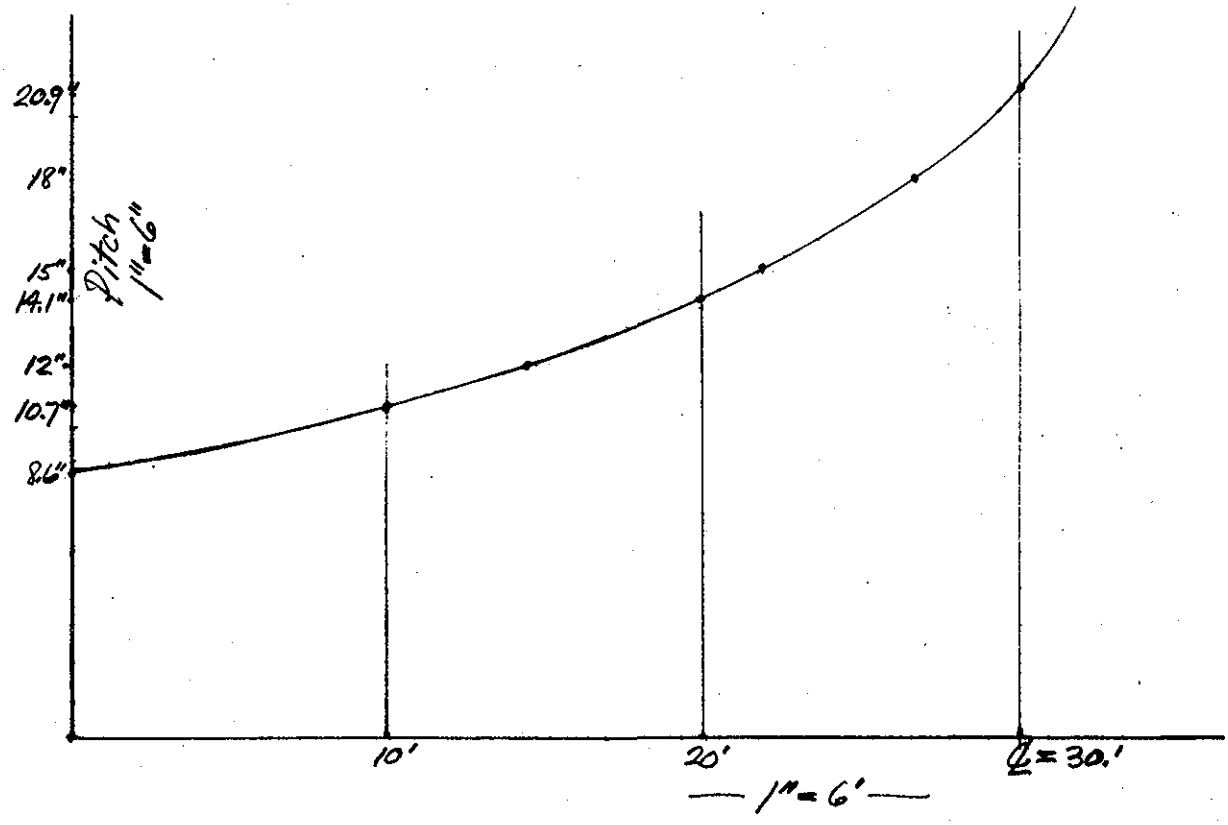
$V = 41 \times 30' \times \frac{15.0}{60} + 16.5 \times \frac{30}{60}$

$= 3.1 + 8.3 = 11.4^k$

Truck

$V = 25.4 \times \frac{1}{2} + 6.4 \times \frac{16}{60} = 12.7 + 1.7 = 14.4^k$

Spacing $= 8.6 \times \frac{35}{14.4} = 20.9''$



Use following pitch — 3-¾"φ x 4" high studs per line.

15@8" = 10'-0", 9@10" = 7'-6", 6@12" = 6'-0", 4@15" = 5'-0", 1@18"

Total = 30'-0" to ℄ of span.

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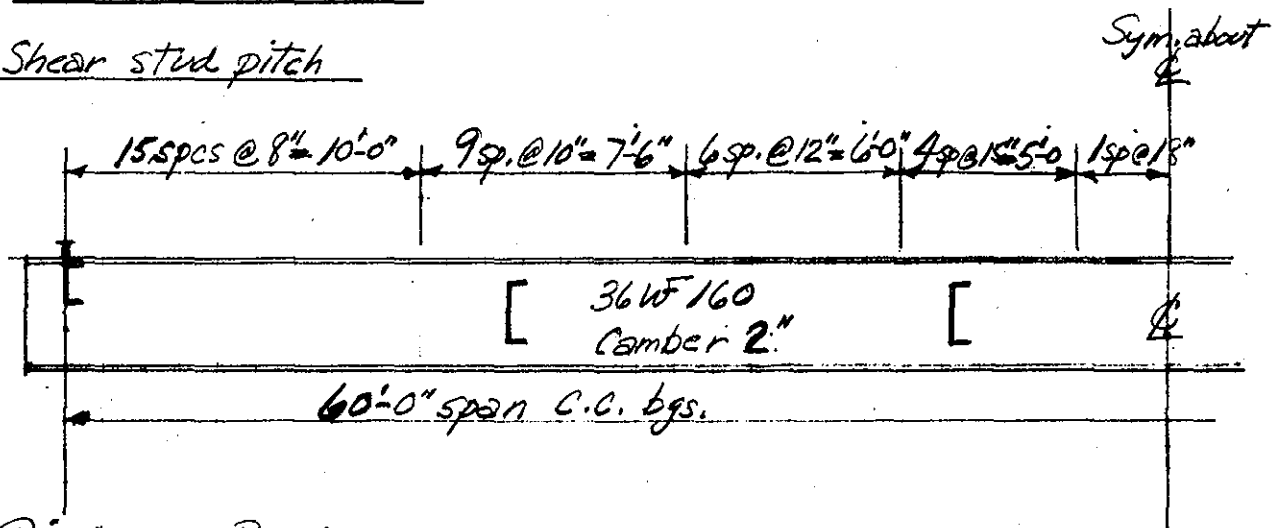
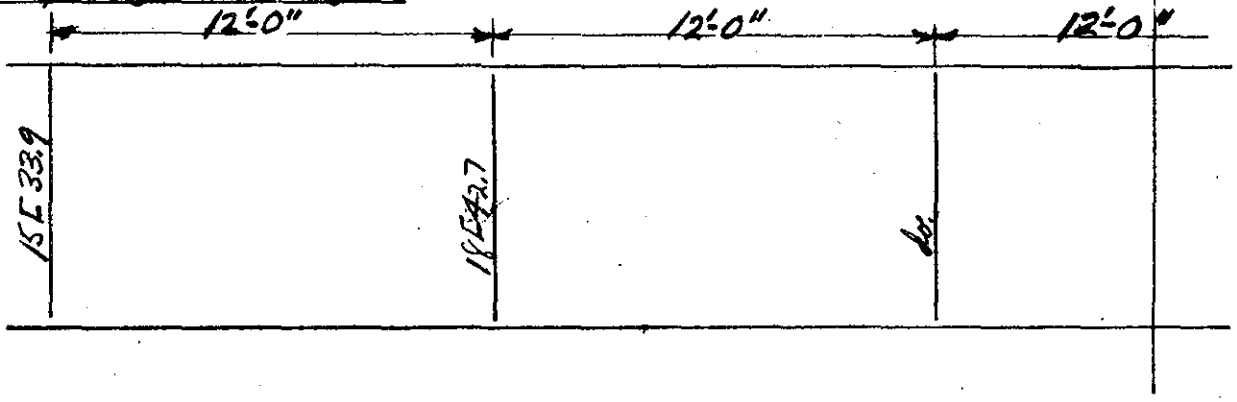
Beam Details

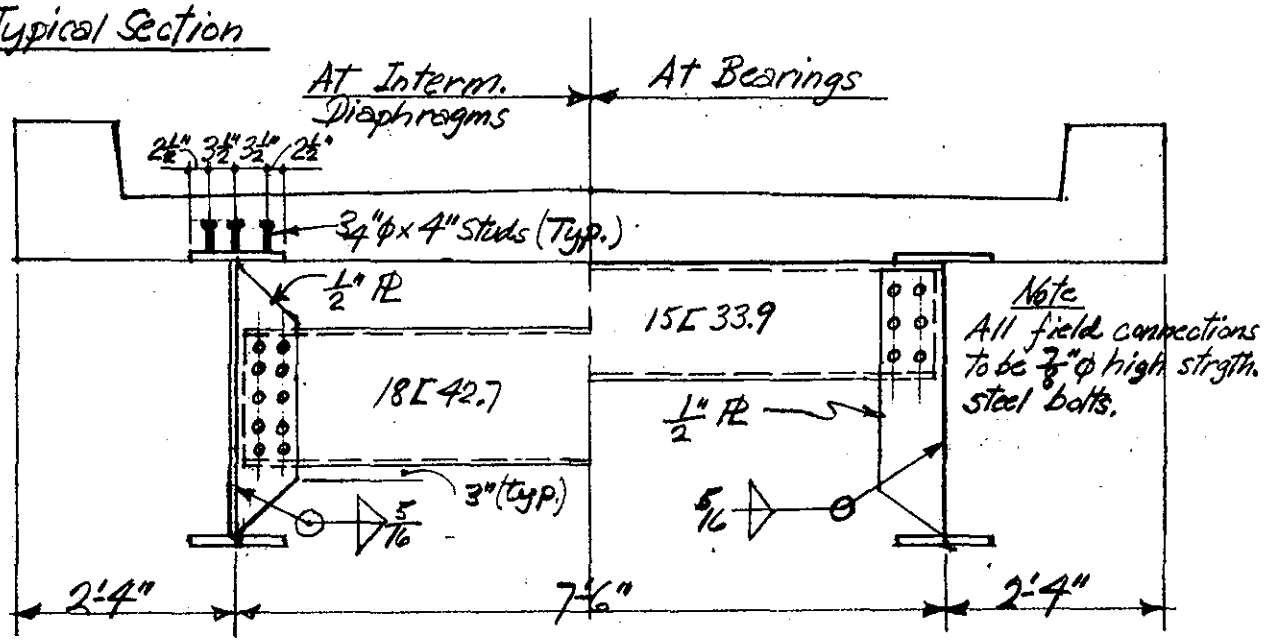
Shear stud pitch



Diaphragm Bracing



Typical Section



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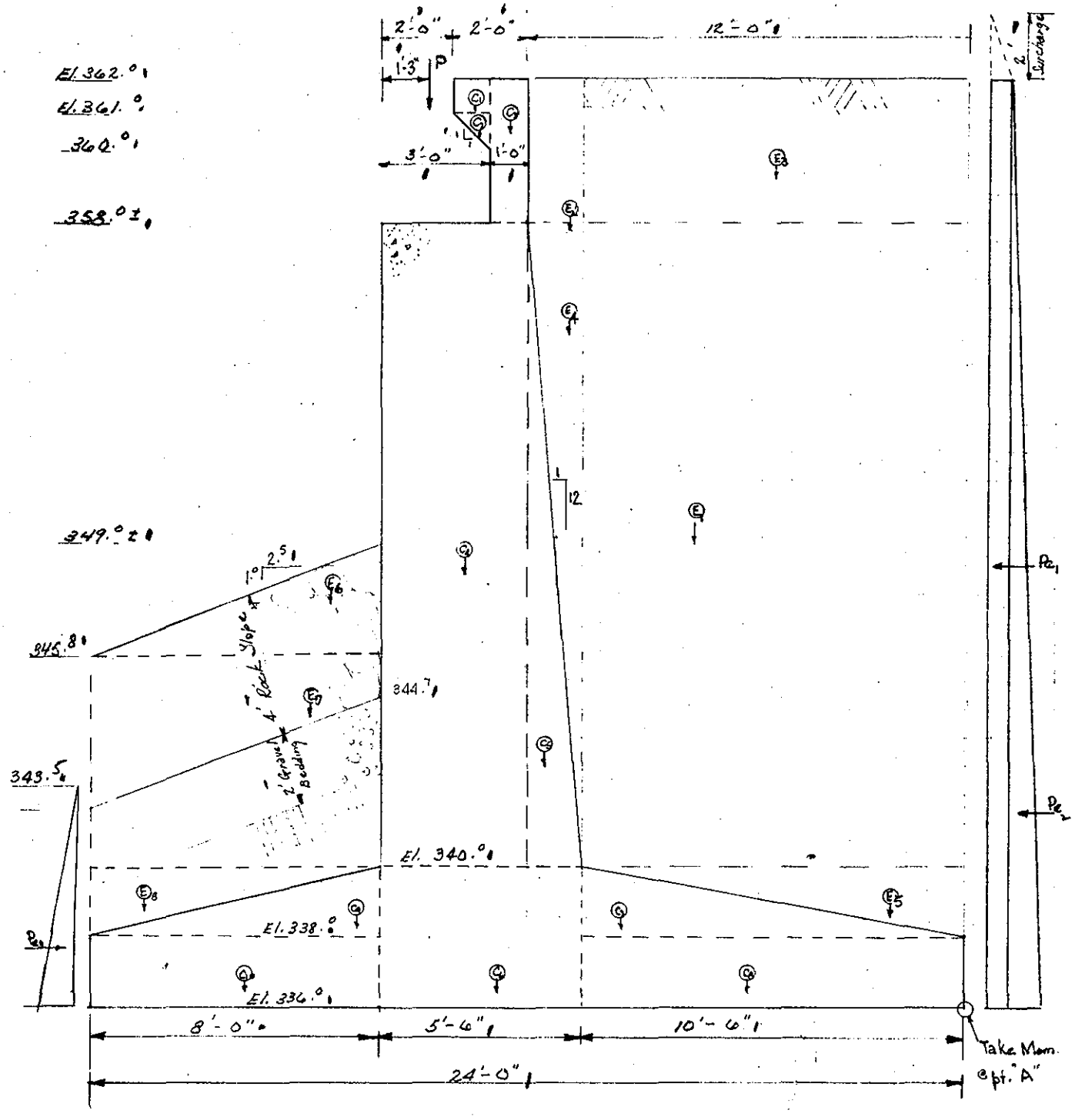
SUBJECT: WEST THOMPSON DAM
COMPUTATION: SERVICE BRIDGE ABUTMENT
COMPUTED BY: R.A.K. CHECKED BY: JER. DATE: NOV 1962



SCALE 1" = 4'-0"

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

SUBJECT: WEST THOMPSON DAM
COMPUTATION: SERVICE BRIDGE ABUTMENT
COMPUTED BY: R.A.K. CHECKED BY: JER. DATE: Nov. 1962

CASE I

Use moist soil, no uplift, water at perm. pool elev. of 306.0, Use 2' live load surcharge, at rest pressure with the resultant falling in mid 1/4 (1/2), and bridge load.

Data: From H. Baker – Soils Lab.

Soil – $\phi = 30.0°$ $\gamma_{dry} = 130$ #/cf $\gamma_{moist} = 140$ $\gamma_{sat.} = 145$ $\gamma_{sub} = 83$
$K_{active} = 0.33$ $K_{at\ rest} = 0.5$ Allow soil pr. = 5000. #/□'

from "Design of Masonry Structures and Foundations" by Williams
with sloping fill in front, the height of an equivilant depth of fill is

$$h' = \frac{1}{3} \times \frac{h \tan S}{\tan S + \tan \varphi} = \frac{1}{3} \times \frac{8.7' \times \frac{1.0}{2.5}}{\frac{1.0}{2.5} + \tan 30.} = 1.2'$$

$\therefore$ Equiv. $H = 8.7' - 1.2' = 7.5'$

Analize 1' Section

Symbol	Factors	↓	↑	→	←	Arm	M	M
C_1	.150 × 1.0 × 1.0	.15				13.5		2.0
C_2	.150 × 1.0 × 1.0 × 1/2	.08				13.33		1.0
C_3	.150 × 1.0 × 4.0	.60				12.5		7.5
C_4	.150 × 4.0 × 18.0	10.80				14.0		151.2
C_5	.150 × 1.5 × 18.0 × 1/2	2.025				11.5		23.3
C_6	.150 × 5.6 × 4.0	3.36				13.25		44.5
C_7	.150 × 10.5 × 2.0 × 1/2	1.575				7.0		11.0
C_8	.150 × 10.5 × 2.0	3.15				5.25		16.5
C_9	.150 × 8.0 × 2.0 × 1/2	1.20				18.67		22.4
C_{10}	.150 × 8.0 × 2.0	2.4				20.0		48.0
E_1	.140 × 10.5 × 18.0	26.46				5.25		138.9
E_2	.140 × 1.5 × 4.0	0.84				11.25		9.5
E_3	.140 × 10.5 × 4.0	5.88				5.25		30.9
E_4	.140 × 1.5 × 18.0 × 1/2	1.89				11.0		20.8
E_5	.140 × 10.5 × 2.0 × 1/2	1.47				3.5		5.1
E_6	.140 × 8.0 × 3.2 × 1/2	1.792				18.67		33.5
E_7	.140 × 8.0 × 5.8	6.496				20.0		129.9
E_8	.140 × 8.0 × 2.0 × 1/2	1.12				21.33		23.9
Pa_1	.140 × 0.5 × 2.0 × 26.0				3.64	13.0		47.3
Pa_2	.140 × 0.5 × 26.0² × 1/2				23.66	8.67		205.1
Pa_3	.140 × 0.5 × 7.5² × 1/2			1.97		2.5	4.9	
Bridge	108.0 k/16 wide	6.75				14.75		99.6
		78.04		1.97	27.3 25.33		4.9	1071.9 1067.0

$\frac{\Sigma M}{\Sigma V} = \frac{1067.0}{78.04} = 13.67'$ from point "A" toward bridge. > 8.0 < 16.0 $\therefore$ in mid 1/3

SUBJECT: West Thompson Dam
COMPUTATION: Service Bridge Abutment
COMPUTED BY: R.A.K. CHECKED BY: GER. DATE: Nov. 1962

Case I (continued)

$$\frac{\Sigma H}{\Sigma V} = \frac{25.33}{78.04} = 0.32 < 0.58 \quad \therefore \text{ok}$$

$$\text{Bearing Pressure} = \frac{\Sigma V}{BL}\left(1 \pm \frac{6e}{L}\right) = \frac{78.04}{24'}\left(1 \pm 6 \times \frac{1.67}{24'}\right) = 3.25\begin{pmatrix}1.4175\\0.5825\end{pmatrix} = \begin{matrix}4.6\ ^k/_{\square'}\\1.9\ ^k/_{\square'}\end{matrix}$$

$\therefore$ B.P. = 4.6 k/□' @ toe & 1.9 k/□' @ heel

Case IA

Same as case I, but no bridge load

$$\frac{\Sigma M}{\Sigma V} = \frac{967.4}{71.29} = 13.57, \text{ ok} \qquad \frac{\Sigma H}{\Sigma V} = \frac{25.33}{71.29} = 0.36 \text{ ok}$$

$$\text{Bearing Pressure} = \frac{71.29}{24'}\left(1 \pm 6 \times \frac{1.57}{24'}\right) = 2.97\begin{pmatrix}1.3925\\0.6075\end{pmatrix} = \begin{matrix}4.1\ ^k/_{\square'} \text{ @ toe}\\1.8\ ^k/_{\square'} \text{ @ heel}\end{matrix}$$

Case II

Moist soil, no uplift, water at perm. pool elev. of 305'.0 use 2' live load surcharge, active pressure with the resultant falling in mid 1/3, and bridge load.

Symbol	Factors	↓	↑	→	←	Arm ∢	M ↻	M ↺
C	Concrete	25.34						327.4
E	Earth	45.95						392.5
P_{e_1}	.140 × 0.33 × 2.0 × 26.0				2.4	13.0		31.2
P_{e_2}	.140 × 0.33 × 26.0² × 1/2				15.6	8.67		135.4
P_{e_3}	.140 × 0.33 × 7.5² × 1/2			1.3		2.5	3.3	
Bridge		6.75				14.75		99.6
		78.04			16.7			982.8

$$\frac{\Sigma M}{\Sigma V} = \frac{982.8}{78.04} = 12.59 > 8.0 < 16.0 \quad \therefore \text{ok} \qquad \frac{\Sigma H}{\Sigma V} = \frac{16.7}{78.04} = .21 < .58 \quad \therefore \text{ok}$$

$$\text{Bearing Pressure} = \frac{78.04}{24.0}\left(1 \pm 6 \times \frac{0.59}{24.0}\right) = 3.25\begin{pmatrix}1.1475\\0.825\end{pmatrix} = \begin{matrix}3.7\ ^k/_{\square'}\\2.7\ ^k/_{\square'}\end{matrix}$$

.8525 2.8

Case IIa

Same as case II, but no bridge load

$$\frac{\Sigma M}{\Sigma V} = \frac{883.2}{71.29} = 12.39 \qquad \frac{\Sigma H}{\Sigma V} = \frac{16.7}{71.3} = .23$$

$$\text{Bearing Pressure} = 2.97\left(1 \pm 6 \times \frac{0.39}{24}\right) = \begin{matrix}3.3\ ^k/_{\square'} \text{ @ toe}\\2.7\ ^k/_{\square'} \text{ @ heel}\end{matrix}$$

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SUBJECT West Thompson Dam
COMPUTATION Service Bridge Abutment
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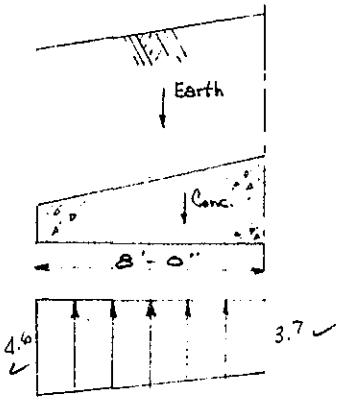
Steel Design

Toe @ Stem



Case I

$$M_{@stem} = 3.7 \times 8.0 \times \frac{8.0}{2} + 0.9 \times \frac{8.0}{2} \times 8.0 \times \frac{2}{3} - 1.8 \times \frac{8.0}{3} - 6.5 \times 4.0$$
$$- 1.1 \times 8.0 \times \frac{2}{3} - 1.2 \times \frac{8.0}{3} - 2.4 \times 4.0$$
$$= 118.4 + 19.2 - 4.8 - 26.0 - 5.9 - 3.2 - 9.6$$
$$= \underline{88.1^{k'}}$$

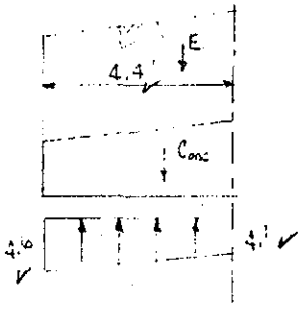
$$d = \sqrt{\frac{88,100}{160}} = 23.5^{in} < 43.5 \quad \therefore \text{ok}$$

$$V = 29.6 + 3.6 - 1.8 - 6.5 - 1.1 - 1.2 - 2.4 = 20.2^{k}$$

$$v = \frac{20,200^{\#}}{12 \times j \times 43.5} = 43 < 60 \qquad \Sigma o = \frac{20,200}{300 \times j \times 43.5} = 1.75^{in^2}$$

$$A_s = \frac{88.1}{1.44 \times 43.5} = 1.4^{in^2} \quad \therefore \text{ Use \#8 @ 6 on bottom} \quad A_s = 1.58, \ \Sigma o = 6.3$$

Check Shear @ 3'-0" Distance Out

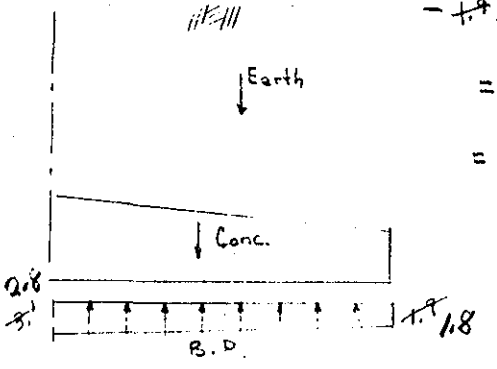


$$4.1 \times 4.4 + 0.5 \times \frac{4.4}{2} - .140 \times 1.8 \times \frac{4.4}{2} - .140 \times 5.8 \times 4.4 - .140 \times 2.0 \times \frac{4.4}{2}$$
$$- 1.1 \times .150 \times \frac{4.4}{2} - 2.0 \times 4.4 \times .15 = 18.04 + 1.1 - 0.5 - 3.6 - 0.6 - .4 - 1.3$$
$$= 12.74^{k}$$

$$\therefore v = \frac{12,740^{\#}}{12 \times j \times 32.5} = 37 < 60$$

Heel @ Stem (Case IA)

$$M_{@stem} = 5.88 \times \frac{10.5}{2} + 26.46 \times \frac{10.5}{2} + 1.47 \times 10.5 \times \frac{2}{3} + 1.575 \times \frac{10.5}{3} + 3.15 \times \frac{10.5}{2}$$
$$- \overset{1.8}{\cancel{1.9}} \times 10.5 \times \frac{10.5}{2} - \overset{1.0}{\cancel{1.2}} \times \frac{10.5}{2} \times \frac{10.5}{3}$$
$$= 30.9 + 138.9 + 10.3 + 5.5 + 16.5 - \overset{100}{\cancel{104.7}} - \overset{18.4}{\cancel{22.0}}$$
$$= \cancel{75.4^{k}} \ 84.1^{k}$$



$$d = \sqrt{\frac{4000}{400}} = \overset{23.0"}{\cancel{21.8}} < 43.5$$

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Heel @ Stem (continued)

$V = 5.88 + 26.46 + 1.47 + 1.58 + 3.15 - \cancel{19.75}\ 18.9 - \cancel{6.8}\ 5.2 = \cancel{42.29}\ 44.4^k$

$v = \frac{\cancel{12,290}\ 14400}{12 \times j \times 43.5} = \cancel{27}\ 31.$

$\Sigma o = \frac{\cancel{12,290}\ 14400}{210 \times j \times 43.5} = \cancel{1.52}\ 1.8$

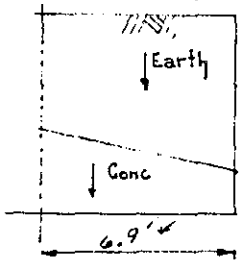
$A_s = \frac{\cancel{75.4}\ 84}{1.44 \times 43.5} = \cancel{1.2}\ 1.34$ in²

Steel Design

Use ~~#7 @ 6~~ #9 @ 9 on top " "

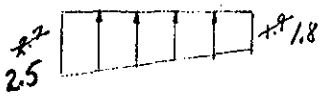
$A_s = \cancel{1.20}\ 1.33$
$\Sigma o = \cancel{5.5}\ 4.7$

Check Shear @ "d" Distance Out.



$.140 \times 6.9 \times 22.0 + .140 \times 6.9 \times 2.0 \times \frac{1}{2} + .150 \times 6.9 \times 1.3 \times \frac{1}{2} + .150 \times 2.0 \times 6.9$
$- \cancel{1.7}\ 1.8 \times 6.9 - \cancel{0.8}\ .7 \times 6.9 \times \frac{1}{2}$
$= 21.25 + 1.0 + 0.67 + 2.07 - \cancel{13.11}\ 12.4 - \cancel{2.76}\ 2.4 = \cancel{9.12}\ 10.2^k$

$v = \frac{\cancel{9,120}\ 10200}{12 \times j \times 34.5} = \cancel{24.8}\ 28 < 60$



Stem @ Base

$M_{@base} = .140 \times 0.5 \times 2.0 \times 22.0 \times 11.0 + .140 \times 0.5 \times \frac{22.0^2}{2} \times \frac{22.0}{3} - .140 \times 0.5 \times \frac{5.8^2}{2} \times \frac{5.8}{3}$
$= 33.88 + 124.23 - 2.28 = 155.83^{k'}$

$V = 3.08 + 16.94 - 1.18 = 18.84$, $N = .15 + .08 + .60 + 10.8 + 2.03 + 0.84 + 1.89 = 16.39^k$

$v = \frac{18,840^{\#}}{12 \times j \times 61.5} = 29 < 60$ $\Sigma o = \frac{18,840}{210 \times j \times 61.5} = 1.65$

$M_s = 155.83 + \frac{29.5}{12} \times 16.39 = 196.12^{k'}$

$A_s = \frac{196.12}{1.44 \times 61.5} - \frac{16.39}{20.0} = 2.21 - .82 = 1.40$ in² ∴ Use #8 @ 6 @ Slope $A_s = 1.58$ $\Sigma o = 6.3$

Stem @ Elev. 349.0

Moment $= .140 \times 0.5 \times 2.0 \times 13.0 \times \frac{13.0}{2} + .140 \times 0.5 \times \frac{13.0^2}{2} \times \frac{13.0}{3} = 11.83 + 25.63 = 37.46$

$N = .15 + .08 + .60 + .150 \times 4.0 \times 9.0 + 0.75 \times .150 \times \frac{9.0}{2} + 0.84 + .140 \times 0.75 \times \frac{9.0}{2} = 8.04$

$M_s = 37.46 + \frac{24''}{12} \times 8.04^k = 53.54^{k'}$, $V = 1.82 + 5.92 = 7.74^k$

$\Sigma o = \frac{7,740}{210 \times j \times 52.5} = 0.8$

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Stem @ Elev. 349.0

$A_s = \frac{53.54}{1.44 \times 52.5} - \frac{8.04}{20.0} = 0.71 - 0.40 = 0.31^{in^2}$ ✓

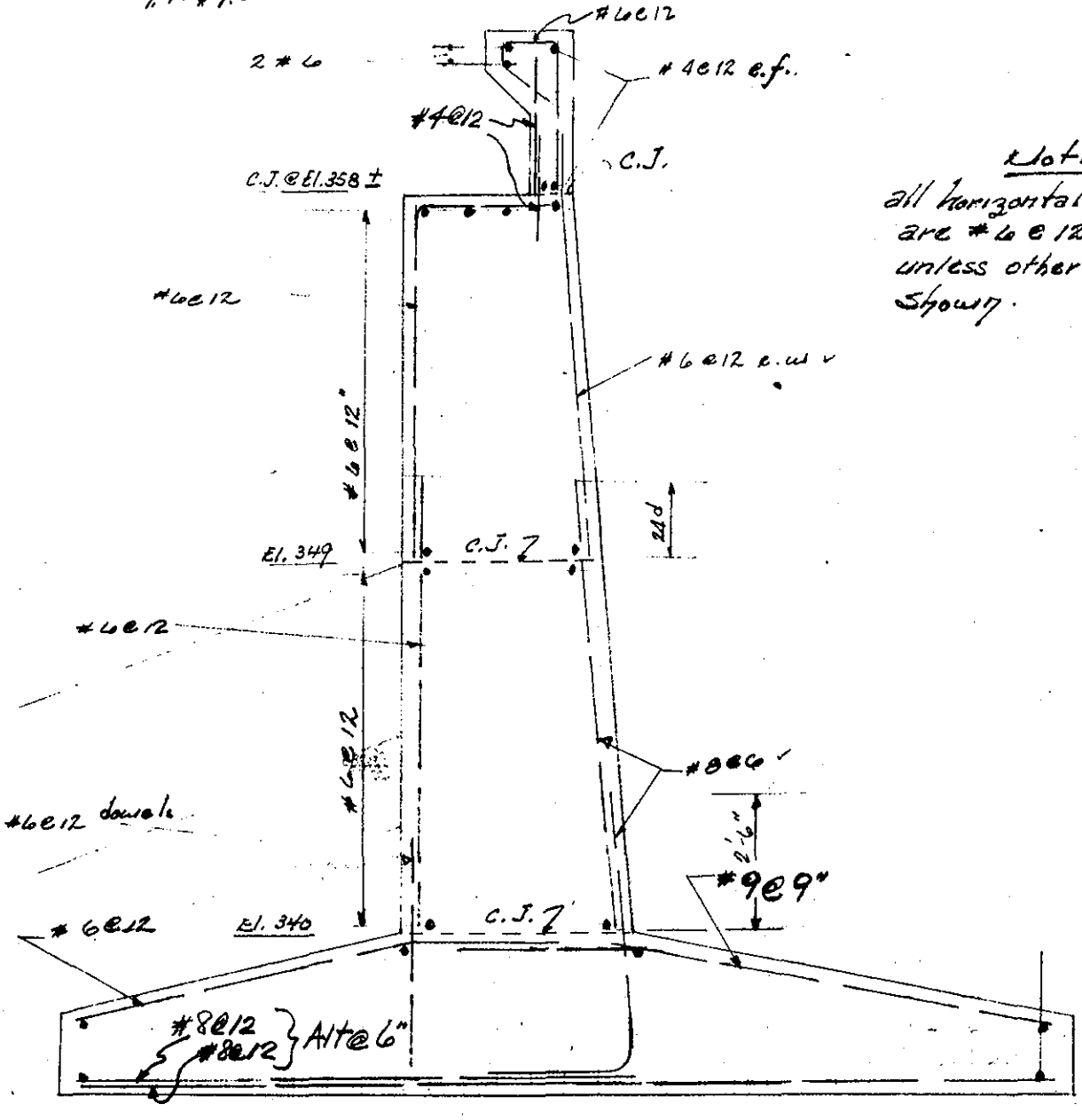
∴ Use #5@12 $A_s = 0.31$ ✓
$\Sigma_o = 2.0$

Const. Cond. @ Heel:
$\frac{25.4}{24} = 1.1$
$2.7 \times .15 = \frac{.4}{.7 \times 10.5^2/2} = 39^{k}$
$A_s = \frac{39}{1.44 \times 43.5} = .62$ – #8@12 ok

Stem @ Elev. 358.0± (Bridge Seat)

$Moment = .140 \times 0.5 \times 2.0 \times 4.0 \times \frac{4.0}{2} + .140 \times 0.5 \times \frac{4.0^2}{2} \times \frac{4.0}{3} = 1.12^{k'} + 0.19^{k'} = 1.31^{k'}$

$A_s = \frac{1.31}{1.44 \times 7.5} = 0.12$ ✓ ∴ Use #4@12"



Note
all horizontal bars are #6 @ 12 unless other wise shown.

SUBJECT: West Thompson
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Slab Design.

Loading H-20-S.16-44
$f_c = 3000$
$f_c = 1200$
$f_s = 20000$

Stringers @ 8'-0" c.c.
Impact = $\frac{50}{S+125}$ '61 AASHO P. 17 1.2.12.(C)

$\frac{50}{8+125} = 37.6 > 30$

Use 30%.

Wheel load = $1.3 \times 16 = 20.8^K$.

Spec. AASHO + following Conn. State Highway requirements
$7\frac{3}{4}$" min. slab thickness
25#/sq. ft. future paving load

Provide add. inch of slab for wearing surface.

Bending Moment (L.L.) '61 AASHO P 26 1.3.2.(C).
Case A - Main Reinf. Perpindicular to Traffic.

$.8\frac{(S+2)}{32}P_{20}$ S = effective span length.
$P_{20} = 16000$

$.8\left(\frac{8+2}{32}\right)(16000) = 4000$ ft. lbs without impact.

$4000 \times 1.3 = 5200$ ft. lbs/ft of width.

Bending Moment (D.L.)

$M = \frac{1}{10} wl^2 = \frac{137.5 \times 8^2}{10}$

$M = 880$ ft. lbs.

Dead Load
8" slab + 1" wearing surface 112.5#/□
Future paving allowance 25.0
137.5#/□

Total Moment

$M_{DL} + M_{LL} = 880 + 5200 = 6080$ ft. lbs. $6.08^{'K}$

$d = \sqrt{\frac{M}{kb}} = \sqrt{\frac{6080}{197}} = 5.66"$ Say 5.55 5.6 5.7"

1.5" cover.
.5" ½ bar.
1.0" wearing surface
8.7"

Use 9" slab.
d = 6".

Transverse steel.

$A_s = \frac{6080 \times 12}{20000 \times .875 \times 6} = .70$ sq. in.

Use #5 @ 5" Top & Bottom normal to ℄ bridge.

Longitudinal steel

$\frac{220}{\sqrt{8}} = 77.8\% > 67\%$

Use 67% of Main (transverse) Reinforcement. See AASHO P27, 1.3.2. E

$.70 \times .67 = .47$ sq. in.

Use #5 @ 8" Top & Bottom.

Shear & Bond Stresses - See AASHO, P 27 1.3.2.(F).

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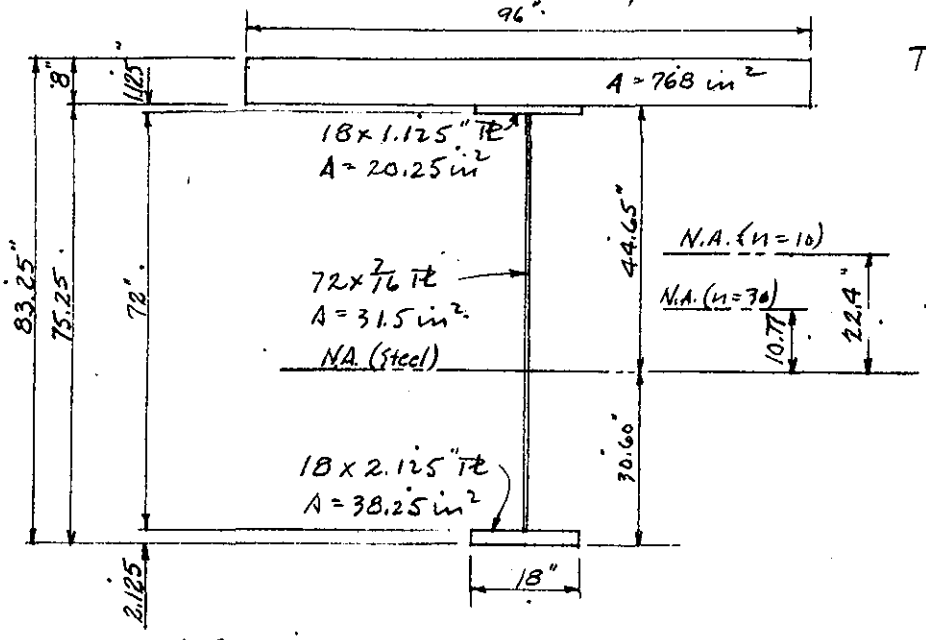
Intermediate Girder @ Midspan



Trial Design
Effective Pl. Width
1/4 Span = 115/4 = 28.75"
Dist. c to c beam = 8'-0"
12 x t of slab = 12x8 = 96"
Eff. W = 96"

Thickness of Web Pl
Assuming D = 72"
Not less than D/170 = .424
7/16" Pl.

A_{STEEL} = 90.00 in²

Steel Section

$$\frac{20.25 \times 74.69 + 31.5 \times 38.13 + 38.25 \times 1.06}{90} = \frac{1512.47 + 1201.10 + 40.55}{90} = 30.60"$$

$$I_{STEEL} = \overset{2.14}{4.03} + 20.25 \times 44.09^2 + 13{,}608 + 31.5 \times 7.53^2 + \overset{14.4}{17.09} + 38.25 \times 29.54^2$$

$$I_{STEEL} = \overset{2.14}{4.03} + 39{,}364.54 + 13{,}608 + 1786.08 + \overset{14.4}{17.09} + 33{,}377.39$$

I_{STEEL} = 88157 in⁴

Composite Section

n = 10

$$\frac{76.8 \times 48.65}{166.8} = 22.4"$$

$I_c = \frac{9.6 \times 8^3}{12}$ = ~~154~~ 410

76.8×26.25^2 = 52963

88157

90×22.4^2 = 45158

186432 in⁴

S_s = 186432/53 =

n = 30

$$\frac{25.6 \times 48.65}{115.6} = 10.77"$$

$I_c = \frac{3.2 \times 8^3}{12}$ = ~~54~~ 136

25.6×37.88^2 = 36,733

88,157

90×10.77^2 = 10,439

135,380 in⁴

Intermediate Girder @ Midspan.
Dead Load Stresses

Slab 8" + 1" wearing surface		900 #/ft
Girder 72 x 7/16 Pl (web)	107	
18 x 1 1/8 Pl (top flange)	68.9	
18 x 2 1/8 Pl (bot. flange)	130.2	
44 x 6 x 13.6/115 (½ stiffeners) = 31 Say	40	
	346.1	346.1

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Intermediate Girder @ Midspan (cont'd.)

Bracing $28 \times 20 \times \frac{7.2}{115} = 35$ Say

40
1286.1
Say 1290 #/ft.

$M_{DL} = \frac{1}{8} w l^2 = \frac{1.29 \times 115^2}{8} = 2133'^k = 25{,}596''^k$

$f_{sT} = \frac{25{,}596 \times 44.65}{88{,}157} = 12.96$ ksi

$f_{sB} = \frac{25596 \times 30.6}{88157} = 8.88$ ksi

Live Load Stresses

$M_{LL} = 1574'^k = 18{,}888''^k$ $M = 1.21 \times 1.45 \times \frac{1794}{2} = 1574'^k$

Impact Distribution 1.3.1.(B)(i)

$f_{cT} = \frac{18888 \times 30.25}{186432 \times 10} = .306$ ksi

$f_{sB} = \frac{18888 \times 53}{186432} = 5.37$ ksi

$f_{sT} = \frac{18888 \times 22.25}{186432} = 2.25$ ~~2.35~~ ksi

Impact
$I = \frac{50}{L+125} = \frac{50}{115+125}$
$= 21.0\% < 30\%$
Use 21%

Superimposed Dead Load Stresses

Sidewalks $2 \times 3.83 \times .875 \times 150/5 =$ 201
Future pavement $25 \times 8 =$ 200
$\frac{2 \times 2 \times 1.125 \times 150 \quad 40}{\text{Conc. rail + steel rail} \times 4}$ 105
3 (bins) 506 #/ft.

$M_{SDL} = \frac{1}{8} w l^2 = \frac{.506 \times 115^2}{8} = 836.48'^k = 10038''^k$

$f_{cT} = \frac{10038 \times 41.88}{135380 \times 30} = .104$ ksi

$f_{sB} = \frac{10038 \times 41.37}{135{,}380} = 3.07$ ksi

$f_{sT} = \frac{10{,}038 \times 33.88}{135{,}380} = 2.51$ ksi

Combined Stress

	f_{cT}	f_{sB}	f_{sT}
D.L. Stress		8.88	12.96
L.L. Stress	.306	5.37	~~2.35~~ 2.25
S.D.L. Stress	.104	3.07	2.51
	.41	17.32	17.82

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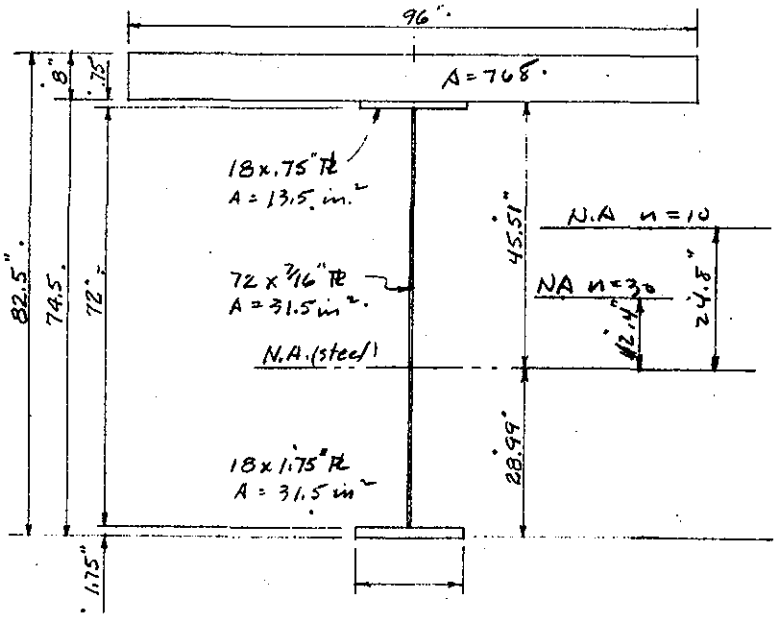
Assumed changes in sizes of flanges

Midspan	top	1.125"
	bottom	2.125"
Intermediate	top	0.750
	bottom	1.75
End	top	0.75
	bottom	0.75



$A_{STEEL} = 76.5 \text{ in}^2$

~~Assumed~~ intermediate section

Steel Section

$$\frac{13.5 \times 74.13 + 31.5 \times 37.75 + 31.5 \times .88}{76.5} = \frac{1000.76 + \overset{2217.61}{1189.13} + 27.72}{76.5} = 28.99"$$

$$I_{STEEL} = \frac{18 \times .75^3}{12} + 13.5 \times 45.13^2 + 136.08 + 31.5 \times 8.76^2 + \frac{18 \times 1.75^3}{12} + 31.5 \times \overset{28.11}{28.99}^2$$

$$= 1 + 27{,}496 + 136.08 + 24.17 + 8 + \overset{24{,}890}{26{,}473} = \overset{68{,}420}{70{,}003} \text{ in}^4$$

Composite Section, n = 10

$$\frac{76.8 \times 49.51}{153.3} = 24.80"$$

n = 30

$$\frac{25.6 \times 49.51}{102.1} = 12.4$$

I_c =

76.8×24.71^2 =	410 ~~154~~	25.6×37.11^2	136 ~~51~~
	46893		35255
	68,420 ~~70003~~		68,420 ~~70003~~
76.5×24.8^2	47051	76.5×12.4^2 =	11763
	164001 in⁴		117072 in⁴
	162,774		115,574

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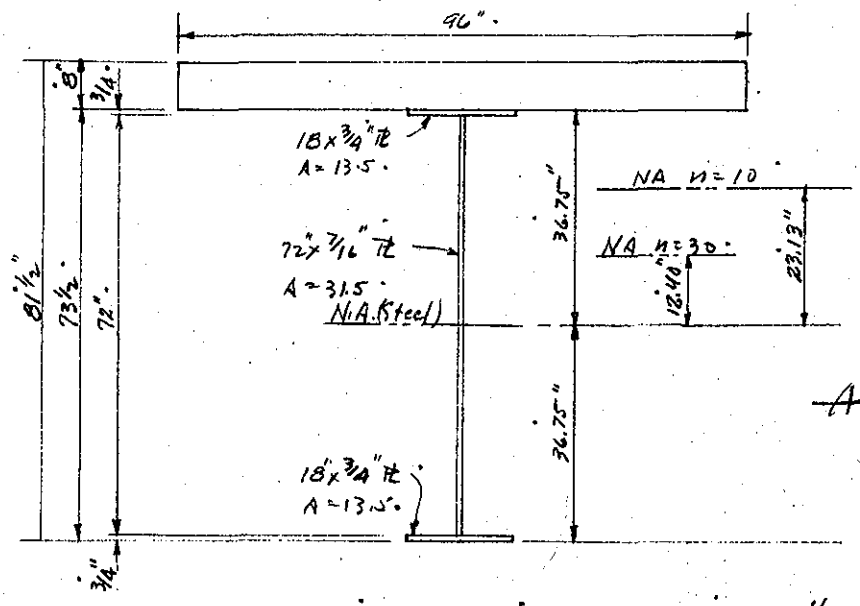

$A_{STEEL} = 58.5$

$I_o' =$

~~Assumed~~ end section

$I_{STEEL} = 35728 + 13608 = 49336 \text{ in}^4$

Composite Section

$n = 10$

$\frac{76.8 \times 40.75}{135.3} = 23.13''$

$I_c = \frac{9.6 \times 8^3}{12} =$ 410 / 154

$76.8 \times 17.62^2 =$ 23844

49336

$58.5 \times 23.13^2 =$ 31245

~~119825~~ in^4

104,835 ✓

$n = 30$

$\frac{25.6 \times 40.25}{84.1} = 12.40''$

136 / 51

$25.6 \times 28.35^2 =$ 20575

49336

$58.5 \times 12.4^2 =$ 8995

78957 in^4

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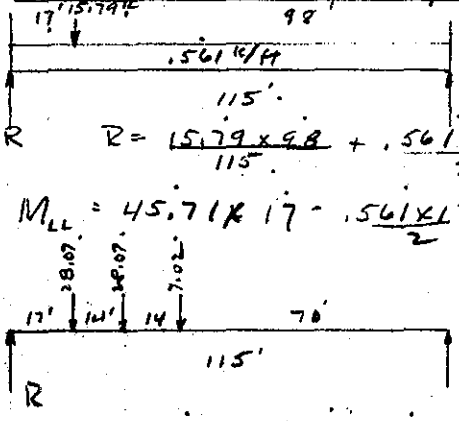
~~Tentative~~ location of change in flange thickness.
End Section 17' from end.

Dead Load Moment 17' from end.

$R_{DL} = 1.290 \times \frac{115}{2} = 74.175^{k}$

$M = 74.175 \times 17 - 1.29 \times \frac{17^2}{2} = 1260.98 - 186.41 = \underline{1074.57}^{'k}$

Live Load Moment 17' from end



$18 \times 1.21 \times 1.45/2 = 15.79^{k}$
$1.21 \times 1.45 \times .64/2 = .561^{k}$

$R = \frac{15.79 \times 98}{115} + \frac{.561 \times 115}{2} = 13.45 + 32.26 = 45.71^{k}$

$M_{LL} = 45.71 \times 17 - \frac{.561 \times 17^2}{2} = 777.07 - 81.06 = 696.01^{'k}$ Lane Loading

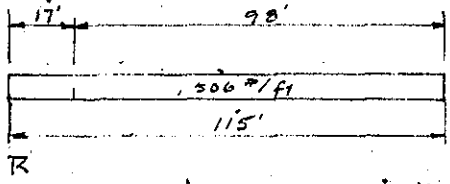
$8 \times 1.21 \times 1.45/2 = 7.02^{k}$
$32 \times 1.21 \times 1.45/2 = 28.07^{k}$

$R = \frac{28.07 \times 98 + 28.07 \times 84 + 7.02 \times 70}{115} = \frac{2750.86 + 2357.88 + 491.40}{115}$

$R = 48.70^{k}$

$M_{LL} = 48.70 \times 17 = 827.90 > 696.01$ Use $827.90^{'k}$

Superimposed Dead Load Moment 17' from end.



$R = \frac{.506 \times 115}{2} = 29.10^{k}$

$M_{SDL} = 29.10 \times 17 - .506 \times \frac{17^2}{2} = 494.70 - 73.12 = 421.58^{'k}$

Dead Load Stresses - End Section

$f_{STEB} = \frac{1074.57 \times 12 \times 36.75}{49336} = 9.61$ ksi.

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~~Terminus~~ location of change in flange thickness - End section -17' from end.

Live Load Stresses

$$f_c = \frac{827.90 \times 12 \times 21.62}{104835 \times 1.0} = \underline{0.20 \text{ ksi}}$$

$$f_{sB} = \frac{9934.8 \times 59.85}{104835} = \underline{5.67 \text{ ksi}}$$

$$f_{sT} = \frac{9934.8 \times 13.62}{104835} = \underline{1.29 \text{ ksi}}$$

SDL Stresses

$$f_c = \frac{421.58 \times 12 \times 32.35}{78957 \times 30} = \underline{0.07 \text{ ksi}}$$

$$f_{sB} = \frac{5058.96 \times 49.15}{78957} = \underline{3.15 \text{ ksi}}$$

$$f_{sT} = \frac{5058.96 \times 24.35}{78957} = \underline{1.56 \text{ ksi}}$$

Combined Stresses

	f_c	f_{sB}	f_{sT}
D.L. Stresses		9.61	9.61
L.L. Stresses	0.20	5.67	1.29
S.D.L. Stresses	0.07	3.15	1.56
	0.27 ksi	18.43 ksi. Less than 3% overstress O.K.	12.46 ksi

Change size of bottom flange from $\frac{3}{4}$" to $1\frac{3}{4}$" 16' from end. (or 1 ft. beyond 17')

SUBJECT West Thompson
COMPUTATION Access Bridge
COMPUTED BY DDM CHECKED BY LSP DATE

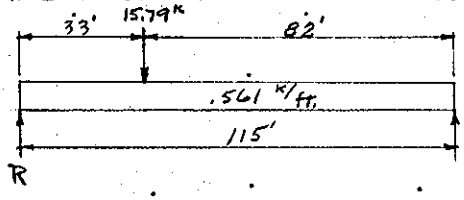
Tentative location of change in flange thickness.

Dead Load Moment 33' from end.

$$R_{DL} = 1.29 \times \frac{115}{2} = 74.175$$

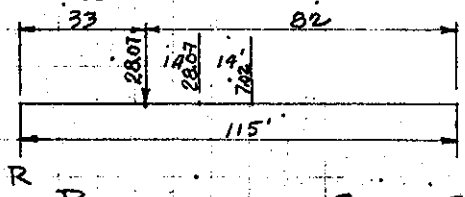
$$M = 74.175 \times 33 - 1.29 \times \frac{33^2}{2} = 2447.78 - 702.41 = \underline{1745.37}\ ^{'k}$$

Live Load Moment 33' from end.



$$R = 15.79 \times \frac{82}{115} + .561 \times \frac{115}{2} = 11.26 + 32.26 = 43.52^k$$

$$M_{LL} = 43.52 \times 33 - .561 \times \frac{33^2}{2} = 1436.16 - 305.46 = \underline{1130.70}\ ^{'k}$$

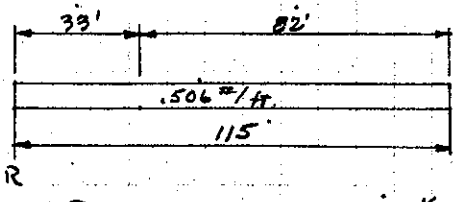


$$R = \frac{28.07 \times 82 + 28.07 \times 68 + 7.02 \times 54}{115} = \frac{2301.74 + 1908.76 + 379.08}{115}$$

$$R = 39.91^k$$

$$M_{LL} = 39.91 \times 33 = 1317.03 > 1130.70 \quad \text{Use } 1317.03\ ^{'k} \text{ for } M_{LL}$$

SDL Moment



$$R = .506 \times \frac{115}{2} = 29.10^k$$

$$M_{SDL} = 29.10 \times 33 - .506 \times \frac{33^2}{2} = 960.30 - 275.52 = 684.78\ ^{'k}$$

SUBJECT West Thompson
COMPUTATION Access Bridge
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Tentative location of change in flange thickness.

Dead Load Stresses - Intermediate Section

$$f_s^B = \frac{1745.37 \times 12 \times 28.99}{68420} = 8.87 \text{ ksi}$$

$$f_s^T = \frac{20944.44 \times 45.31}{68420} = 13.87 \text{ ksi}$$

Live Load stresses

$$f_s^B = \frac{1317.03 \times 12 \times 53.79}{162774} = 5.22 \text{ ksi}$$

$$f_s^T = \frac{15804.36 \times 20.71}{162774} = 2.01 \text{ ksi}$$

SDL Stresses

$$f_s^B = \frac{684.78 \times 12 \times 41.39}{115574} = 2.94 \text{ ksi}$$

$$f_s^T = \frac{8217.36 \times 33.11}{115574} = 2.35 \text{ ksi}$$

Combined Stresses

	f_s^B	f_s^T
D.L. Stresses	8.87	13.87
L.L. Stresses	5.22	2.01
S.D.L. Stresses	2.94	2.35
	17.03 ksi	18.23 ksi O.K. Less than 3% overstress.

Change size of bottom flange from $1\frac{3}{4}$" to $2\frac{1}{8}$" and top flange from $\frac{3}{4}$ to $1\frac{1}{8}$" 32' from end (8' beyond 33' figured).

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Stiffeners

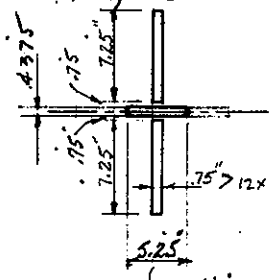
At supports.

Dead load (AB-6) 74.18 K.
Live load (AB-6) $28.07 + 28.07 \times \frac{101}{115} + \frac{7.02 \times 87}{115} =$ 58.03
S.D. Load (AB-6) 29.10 K.
161.31 Say 162 K

Web shear

$\frac{162}{72 \times .4375} = 5.14$ k.s.i. < 11 ksi. O.K.

Try 2 - 8 x 3/4" end stiffeners 8" < 12 times .75 (5.4375) (2.30) 1.6.17. AASHO



$A = 2(7.25 \times .75) + 5.25 \times .4375 = 13.175 \text{ in}^2$

$I_{xx} = 2\left(\frac{.75 \times 7.25^3}{12} + 5.4375 \times 4.5938^2\right) + 5.25 \times \frac{.4375^3}{12}$

$I_{xx} = 2(23.8174 + 114.7476) + .0366$

$I_{xx} = 277.13 + .04 = 277.17 \text{ in}^4$

(12 times web thickness. A.I.S.C.) $r = \sqrt{\frac{277.17}{13.175}} = 4.6"$ $\frac{l}{r} = \frac{.75 \times 72}{4.6} = 11.7$

Allowable $\frac{P}{A} = 17000 - .485(11.7)^2 = 17000 - 70 = 16.93$ ksi

Actual $\frac{P}{A} = \frac{162}{13.175} = 12.3$ ksi < 16.93 Use 4 - 8 x 3/4" stiffeners

Note: 4 stiffeners were used to provide a stiffener on each side of the anchor bolt.

Intermediate stiffeners. @ end

Try 6 x 3/8" stiffeners.. Max. width = 16 x 3/8 = 6" 1.6.80. AASHO

Web shear at support.

$\frac{162}{72 \times .4375} = 5.14$ ksi < 11 ksi

$d = \frac{12000}{\sqrt{s}}(t)$ where d = req. clear distance between stiffs.
t = thickness of web
s = av. unit shearing stress (1.6.80 AASHO)

$= \frac{12000 \times .4375}{\sqrt{4825}}$
5140

$d = 75.60$ (73.40) > 72" which is clear unsupported depth of web.

72" max. distance between stiffeners. (1.6.80. A.A.S.H.O.)

Use 6'-0" max. between pairs of intermediate stiffeners.

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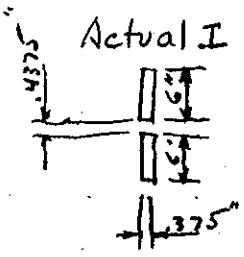
Intermediate stiffeners

Req. I

$I = \frac{d_o t^3 J}{10.92}$ See 1.6.80, AASHO $= \frac{72 \times .4375^3 \times 5}{10.92} = 2.76 \text{ in}^4$

$J = 25(D/d)^2 - 20$ but not less than 5

$J = 25(72/72)^2 - 20 = 5$



Actual I $= 2\left(\frac{.4375 \times 6^3}{12} + 2.25 \times 3.219^2\right)$ (.375)

$= 2(7.875 + 23.314) = 62.38 \text{ in}^4 > 2.76 \text{ in}^4$ O.K. (6.750; 60.128)

Shear connectors 1.9.5. AASHO

H = Height of stud $\frac{H}{d} = \frac{6}{.875} = 6.86 > 4.2$

d = diam. of stud

$f'_c = 3000$

Q_{uc} = critical load cap. of 1 shear connector

$Q_{uc} = 330\, d^2 \sqrt{f'_c} = 330 \times .875^2 \times 54.77 = 13838^{\#}$

With Safety Factor of 4 $\frac{13838^{\#}}{4} = 3459^{\#}$ resistance per $\frac{7}{8}$"ϕ stud.

At support.

Live load V = 58.03^k (AB-10)

S.D.L V = 29.10^k (AB-6) $S = \frac{Vm}{I}$ (1.9.5. A.A.S.H.O.)

Live Load

$S_1 = \frac{58.03 \times 76.8 \times 17.62}{104835} = .742$

S.D.L.

$S_2 = \frac{29.10 \times 25.6 \times 28.35}{78957} = .267$

$S = S_1 + S_2 = 1.009$ k/in

Spacing for 3 studs $\frac{3 \times 3459}{1009} = 10.28"$ Say 10"

At point 17' from support (Intermediate girder section)

Live load V = 48.7^k

SDL V = 20.50^k $29.10 - 17 \times .506 = 20.50$ (AB-6)

L.L.

$S_1 = \frac{48.7 \times 76.8 \times 24.71}{162,774} = .568$ ksi

S.D.L.

$\frac{20.50 \times 25.6 \times 37.11}{115,574} = .169$ ksi

$S_1 + S_2 = .568 + .169 = .737$

Spacing for 3 studs $\frac{3 \times 3459}{737} = 14.08"$

SUBJECT West Thompson
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Shear connectors
At point 33' from support
Live Load V = 43.52^k.
SDL V = 12.40^k. 29.10 − 33 × .506 = 12.40

$S_1 = \frac{43.52 \times 76.8 \times 26.25}{186432} = .471$ ksi (L.L.) $S_2 = \frac{12.40 \times 25.6 \times 37.88}{135380} = .089$ ksi (S.D.L.)

$S_1 + S_2 = .471 + .089 = .560$

Spacing for 3 studs $\frac{3 \times 3459}{560} = 18.53''$

Camber
Dead Load Deflection

Web 7/16 × 72" × 116'-4" =	12448#
Top flange 18 × 1 1/8 × 51'-0"	3514#
18 × 3/4 × 65'-4"	2999
Bottom flange 18 × 2 1/8 × 51'-0"	6829
18 × 1 3/4 × 32'-0"	3427
18 × 3/4 × 33'-4"	1530
Stiffeners 8 - 8 × 3/4 × 6'-0"	979
38 - 6 × 3/8 × 6'-0"	1744
Cross frames	2472
Longitudinal bracing	2156
	38098
Slab 9/12 × 8 × 150 × 115 =	103,500
	141598#

$w = \frac{142000}{115} = 1.235$ ^k/ft.

$\Delta = \frac{5 \times 1235 \times 115 \times 1728}{384 \times 88157 \times 29000000} = 1.90''$

Add 1"/100' of span 1.15
3.05" Use 4" camber.