WATER RESOURCES DEVELOPMENT PROJECT

SAXONVILLE LOCAL PROTECTION

SUDBURY RIVER

MERRIMACK RIVER BASIN

FRAMINGHAM, MASSACHUSETTS

DESIGN MEMORANDUM NO. 5

DESIGN OF STRUCTURES

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
WALTHAM, MASS.

SEPTEMBER 1974

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
424 TRAPELO ROAD
WALTHAM, MASSACHUSETTS 02154

REPLY TO
ATTENTION OF:

NEDED-E

19 September 1974

SUBJECT: Saxonville Local Protection, Sudbury River, Merrimack River Basin, Framingham, Massachusetts, Design Memorandum No. 5 - Design of Structures

HQDA (DAEN-CWE-B)
WASH DC 20314

In accordance with ER1110-2-1150, there is submitted for review and approval, DM No. 5, Design of Structures, for the Saxonville Local Protection Project.

FOR THE DIVISION ENGINEER:

John Wm. Leslie

JOHN WM. LESLIE
Chief, Engineering Division

Incl (10 Cys)
as

WATER RESOURCES DEVELOPMENT PROJECT

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DESIGN MEMORANDA INDEX

No.	Title	Anticipated Submission Date	Date Submitted	Date Approved
1	Hydrologic Analysis		12 Dec 72	23 Feb 73
2	General Design - Phase I - Plan Formulation		30 Apr 73	27 Aug 73
2	General Design - Phase II		26 Jul 74	5 Sep 74
3	Concrete Materials	Dec 74		
4	Embankments and Foundations	Dec 74		
5	Design of Structures		19 Sep 74	

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DESIGN MEMORANDUM NO. 5
DESIGN OF STRUCTURES

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WATER RESOURCES DEVELOPMENT PROJECT

SAXONVILLE LOCAL PROTECTION
SUDBURY RIVER
MERRIMACK RIVER BASIN
FRAMINGHAM, MASSACHUSETTS

A. PERTINENT DATA

PURPOSE	Flood Control
LOCATION	
State	Massachusetts
County	Middlesex
Town	Framingham
Village	Saxonville, about 20 miles west of Boston
River	Sudbury, about 15 miles upstream of its mouth at the Concord River
River Basin	Merrimack

RECORD FLOODS

Date	Peak Discharge* (cfs)	Flood Elevation* (msl)
March 1936	2,050	125.5
July 1938	1,800	125.1
September 1954	2,850	127.6
August 1955	4,400	129.6
March 1968	2,100	127.0

* At Concord Street Bridge

STANDARD PROJECT FLOOD

	Drainage Area (sq. mi.)	Design Discharge (cfs)
Sudbury River at Concord Street Bridge	86	10,000
Sudbury River at confluence w/Cochituate Brook	106	11,900

AREAS ON LEFT BANK OF SUDBURY RIVER

Inundated by 1955 flood of record, acres	22
Subject to Standard Project Flood inundation, acres	29
Protection against 1955 flood, acres	16
Protection against Standard Project Flood, acres	23
Type of area protected	Industrial, commercial, public and residential

FLOOD WALLS

At Saxonville Pond	Gravity Type, Concrete
Top elevation	154.2 msl
Length	170 feet
Maximum height	10.2 feet
Stations 0+00 to 2+00	L-Type, reinforced concrete, anchored base
Top elevation	137.0 msl
Maximum height	21.5 feet
Maximum base width	20 feet
Stations 2+00 to 3+20	T-Type, reinforced concrete
Top elevation	137.0 msl
Maximum height	27.5 feet
Maximum base width	26.8 feet
Stations 3+20 to 6+50	T-Type, reinforced concrete
Top elevation	137.0 msl
Maximum height	31.0 feet
Maximum base width	27.5 feet
Stations 6+50 to 6+80	I-Type, reinforced concrete and steel sheet piling
Top elevation	137.0 msl
Maximum height above grade	10 feet
Stations 18+83 to 19+20	I-Type, reinforced concrete and steel sheet piling
Top elevation	136.5 msl
Maximum height above grade	10 feet
Stations 19+20 to 22+00	T-Type, reinforced concrete
Top elevation	136.5 msl
Maximum height	26.0 feet
Maximum base width	19.5 feet

FLOOD WALLS

Stations 22+75 to 24+10	I-Type, reinforced concrete and steel sheet piling
Top elevation	136.0 msl
Maximum height above grade	10 feet
At Danforth Street	Gravity-type concrete
Top elevation	135.0 msl, Max.
Length	40 feet
Maximum height	8.5 feet

DIKE

Type	Earthfill with rock slope protection
Top elevation	Varies 137.0 to 134.5 msl
Top width	12 feet
Maximum height	23 feet
Slopes	Riverside 1 on 2.5; Landside 1 on 2
Total length	2,500

PUMPING STATION

Substructure	Reinforced concrete
Size	24'-9"x17'-5"
Pumps	2 Axial flow pumps
Pump capacity, each	10,500 GPM, 17' static head
Engines	Diesel
Gravity discharge line	4'-0" x 4'-0" Reinforced concrete

CONCORD STREET GATE

Location	Concord Street
Type	Steel Swing Miter
Size opening	55 feet
Operating mechanism	Hand operated

B. INTRODUCTION

1. PURPOSE. The purpose of this memorandum is to present for review and approval the basic design criteria for the structures to be included in the Saxonville Local Protection Project, Framingham, Massachusetts. The basic criteria, typical design computations, and other pertinent data are presented.

2. SCOPE. This memorandum covers the design of the concrete floodwalls, the street gate structure and the pumping station.

3. PREVIOUS REPORTS. A detailed report on the design of the structural features for this project has not been previously submitted. The latest previous description of the proposed structures is contained in the recommended project plan in Design Memorandum No. 2, "Phase II, General Design," submitted 26 July 1974. Design Memorandum No. 2 also contains the detailed geological studies for the project and a description of the foundation materials upon which these structures will be founded. The basic hydrology and hydraulic design criteria for this project is contained in Design Memorandum No. 1, "Hydraulic Analysis," submitted 12 December 1972 and approved 23 February 1973.

4. LOCATION. The project area is located along the left bank of the Sudbury River in the village of Saxonville, Framingham, Massachusetts. The protection works extend from the Saxonville Pond Dam at Central Street to the Danforth Street Bridge, a total distance of about 3,800 feet. A general plan of the project area is shown on Plate 5-1.

5. DESCRIPTION OF PROJECT. The proposed protection plan requires about 2,500 feet of earth dikes, 1,000 feet of channel realignment, 1,340 feet of concrete floodwalls, a street gate structure, a concrete intake structure, a pumping station and interior drainage. A minimum freeboard of 3 feet for earth dikes and 2 feet for concrete floodwalls has been provided in the design of the proposed project. Plans and elevation of the project are shown on Plates 5-2 through 5-4.

C. FOUNDATIONS

6. FOUNDATION CONDITIONS.

a. General. In general, the project will be built through an area of artificial fills up to 10 feet thick and underlain in places by organic silts as thick as 4 feet, across an area of marshy materials involving minor stripping and, lastly, across the existing river bed itself which is alluvium up to 8 feet thick. All of these materials overlie glacial materials, largely glacial lake deposits, up to 40 feet thick; the principal exception is the area north of station 0+50, where

glacial till is at the surface and bedrock is very shallow. Bedrock outcrops upstream at the base of the existing Saxonville Pond Dam and the bedrock is chloritic schist.

b. Concrete Structures. Concrete floodwalls will include an L-Wall from Station 0+00 to about Station 2+00 founded on bedrock, a T-wall from Station 2+00 to Station 6+50 founded on bedrock, glacial till and silty gravel, and a T-wall between Station 19+20 and Concord St. founded on sandy silt. The I-wall from Station 22+75 to about Station 24+10 will be constructed through approximately 7 feet of manmade fill into a loose silty fine sand and fine sandy silt. The concrete gate structure for Concord Street will be founded on a sandy silt foundation about 12 feet below the street surface. The foundations for the pumping station and sluice gate structures will be founded on loose to moderately compact sandy silts at depths of 5 feet and 10 feet below the ground surface, respectively.

c. Foundation Design. Detailed foundation design studies will be presented in Design Memorandum No. 4, "Embankments and Foundations."

D. CONCRETE

7. GENERAL. Concrete for this project will be supplied from nearby commercial sources that were previously found acceptable for use in Civil Works Projects. Concrete materials will be discussed in detail in Design Memorandum No. 3, "Concrete Materials."

E. ARCHITECTURAL DESIGN

8. ARCHITECTURAL DESIGN. Architectural design of structures and facilities required for this project will be based upon the application of aesthetic criteria to the functional needs and to provide a harmonious relationship with the suburban environment. Although the immediate environs of the project are primarily commercial and industrial, Concord Street is a "gateway" access to the village for residents. The area between the earth dike and the east side of Concord Street is also programmed for approximately 75 low-rise apartments, town houses, or similar housing under redevelopment authority auspices. The property adjacent to the east end of the vehicular gate at Concord Street is a well maintained residence with landscaped grounds extending to the river edge. The existing bridge across the river is a concrete structure with panelled concrete railing. Landscaping and other visual amenities which provide additional aesthetic enhancement to the project are described in Design Memorandum No. 2, Phase 2, "General Design."

From the public standpoint, the principal structure of this project is the flood wall with vehicular gate structure at Concord Avenue. The top of the wall is approximately ten feet above the road level and, for the most part, within unobstructed view. Primary aesthetic consideration

will be to provide simple mass and texture relationships which are commensurate with the functional requirements. Exposed concrete finish of those elements will be Class A. Use of textured form liners and feature strips will be utilized to provide a formal configuration on the river side of the wall and adjacent to the commercial property at the west side of the gate. In addition, existing gas light standards along the present parking lot, at the location of the floodwall, will be relocated and uniformly spaced along the wall to coincide with the wall pattern. Monolith blocks of concrete extending thirty inches above grade will be installed between the light standards to provide protection for the wall. The wall face adjoining the private residence will have an informal pattern of projections and texture to create a harmonious relationship with the landscaping and human scale. See Plate 5-12.

A small pumping station will be located within a "commercial" zoned area. The locus of the structure places it in an undeveloped area of level ground with sparse vegetation. The facades will be designed with functionally required apertures, light colored brick faced masonry walls, and concrete foundations and trim. See Plate 5-8.

Concrete finish on the river face of the flood wall adjacent to the industrial building will be Class C.

All exposed surfaces of concrete not otherwise described will have a Class B finish.

Provisions for the handicapped are not required in connection with the architectural design. Exterior work in areas of public access or use will reflect required criteria for handicapped persons as appropriate.

F. STRUCTURAL DESIGN

9. GENERAL. This section presents the design criteria, basic data, assumptions, and the procedures used in the structural design of concrete flood walls, steel sheet pile flood walls, steel flood gate with supporting structure, and the pumping station. Typical computations are included in Appendix A showing the stability, maximum loading conditions, and typical designs for critical sections. Typical design sections and graphical presentation of loading conditions are shown on Plates 5-5, 5-6, 5-7, and 5-9.

10. DESIGN CRITERIA.

a. General. All working stresses conform to those specified in the working stresses for structural design EM 1110-1-2101, dated

1 November 1963. General loading conditions, design assumptions and other design criteria are based on the following applicable parts of the Engineering Manual for Civil Works: Standard Practice For Concrete (EM 1110-2-2000, Nov 1971), Retaining Walls (EM 1110-2-2502, dated May 1961), Wall Design (EM 1110-2-2501, Change 3, dated 18 June 1962), Structural Design of Pumping Station (EM 1110-2-3104, dated 9 June 1958), and Details of Reinforcement - Hydraulic Structures, (EM 1110-2-2103, dated 21 May 1971).

b. Concrete. Concrete working stresses are in general accordance with ACI Standard Building Code Requirements for Reinforced Concrete (ACI-318-63), using concrete with a minimum ultimate compressive strength of 3,000 lbs. per square inch.

Flexure	(Extreme fiber stresses in compression)	- P.S.I.
	Roof and floor systems of Pumping Station	- 1350
	All other concrete	- 1050

c. Reinforcement.

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

(2) Spacing. The clear distance between parallel bars will not be less than 1-1/2 times the nominal diameter of the 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 Principal Reinforcement.

Item	Min. Cover (inches)
Flood Walls	
Bottom of flood wall base slabs	6"
All other surfaces	4"
Pumping Station	
Interior slabs	1-1/2"
Interior girders and beams	1-1/2"
Concrete exposed to water	4"
Bottom of base slab	6"
Concrete above grade (interior)	2"

(4) Splices. All splices shall be lapped splices in accordance with the ACI Building Code. Tension splices will be staggered longitudinally so that no more than half the bars are spliced at any section within the required lap length.

(5) Temperature and Shrinkage Reinforcement. Temperature and shrinkage reinforcement is provided where the main reinforcement extends in only one direction. Such reinforcement will provide for a ratio of steel area to concrete area of .001 at each face with a minimum of No. 4 bars at 12-inch centers in thin sections and a maximum of No. 6 bars at 12 inches in thick sections. A minimum ratio of steel area to concrete area of .0025 is provided for the roof slab of the pump house.

d. Structural Steel. Structural steel working stresses are in general accordance with the requirements of EM 1110-1-2101, Working Stresses for Structural Design using Column B which is for hydraulic structures. Steel conforming to the requirements of ASTM A-36 will be used for the framing members and skin plates of the vehicular gate.

e. Cast Steel. Hinge shoe castings for the Concord Street gate will be made of cast steel conforming to QQS-681d, Class 70-36 having a minimum yield strength of 36,000 p.s.i. and a working strength of 18,000 p.s.i.

f. Bushings and Pins. Hinge bushings will be corrosion resistant self aligning ball type. Hinge pins will be of corrosion resistant steel (SS-410 condition "T", minimum yield 80,000 p.s.i.).

g. Increase in Working Stresses. Allowable working stresses have not been increased for any loading conditions.

h. Waterstops. Rubber or polyvinylchloride waterstops will be used in all concrete contraction and expansion joints.

11. BASIC DATA AND ASSUMPTIONS

a. Loads.

(1) Dead Loads

Concrete	150	p.c.f.
Steel	490	p.c.f.
Soil, saturated	125	p.c.f.
Soil, submerged	62	p.c.f.
Rock, channel protection (in place)	140	p.c.f.
Soil, saturated (compacted)	140	p.c.f.
Soil, submerged (compacted)	78	p.c.f.

(2) Live Loads

Water	62.5	p.c.f.
Wind	50	p.s.f.
Pump house roof slab	30	p.s.f.
Pump house operating floor slab	340	p.s.f.
Pump house operating floor beams	100	p.s.f. plus equipment loads
Highway	A.A.S.H.O.	HS-20

b. External Water Pressure. Hydrostatic pressure under maximum flood condition is taken as full pressure on riverside of flood walls and varying from full head at bottom of riverside base to zero at ground surface landside by the creep method. For the pump house, hydrostatic pressure is taken as the head to ponding elevation uniformly distributed over the base slab area.

c. Earth Pressure. Earth pressures were determined in general accordance with EM 1110-2-2502, Retaining Walls, dated 29 May 1961.

d. Earthquake Forces. An earthquake force of 0.1 times the gravity loads (Moderate Damage - Zone 2) will be considered in the design of the masonry wall reinforcing for the pump house superstructure.

e. Frost Protection. All structures will have a minimum of 4'-6" cover for frost protection.

f. Uplift Factor of Safety. The empty pumping station has a factor of safety against uplift =1.8.

g. Location of Resultants. In the investigations for stability of walls, the resultant of the horizontal and vertical forces has been held within the middle half of the base when flood is at the top of the wall and within the middle third of the base when flood is within three feet of the top of the wall.

h. Sliding. Sliding of concrete flood walls will be resisted either by friction on the base slab when it is within allowable limits or by passive pressure. In either case, a factor of safety of 1.5 against sliding is provided for in design. Sliding is not a consideration in design of pumping station.

12. PREPARATION OF DESIGN COMPUTATIONS. Extensive use of computer programming has been used in preparing computations. The mathatron desk-top computer was used to analyze the structures when appropriate programs were available.

13. DESIGN OF STRUCTURES.

a. General. The structures required as part of this project include an "L" shaped flood wall approximately 200 feet in length, two section of T-type flood walls, 450 feet and 280 feet in length, three sections of concrete capped I-type sheet pile walls totaling 200 feet in length, and two relatively low concrete gravity walls. Miscellaneous structures include an intake structure at Saxonville Pond, a vehicular flood gate at Concord Street, and a pumping station. Consideration was given to the protection of existing building foundations adjacent to the flood walls during construction.

b. L-Type Flood Wall.

(1) General. The L-type wall to be constructed from Station 0+00 to Station 2+00 will provide the protection adjacent and parallel to Mill Building No. 3 in the Saxonville Industrial Park, with the landside face of stem located three feet from the building line. The space between the building and wall will be filled with gravel and an underdrain connected to the pumping station will be installed in this fill to intercept seepage. The wall base will be founded on bedrock which lies approximately four feet below the existing stream bed. Rock anchors will be utilized to economically meet stability requirements for flood loading. From available data, the bottom of foundation of the existing building appears to be located approximately 2.5 feet above and three feet horizontally from the bottom of the proposed wall base. Shorter than normal lengths of monolith will be specified for this wall to minimize the length of excavation open adjacent to the building at any one time.

(2) Loading Conditions.

(a) Loading I - Water surface to top of wall riverside. 100% uplift on riverside bottom of base varying uniformly to an uplift landside assuming water at the basement floor of the building. Resultant of forces to fall within the mid-half of the base.

(b) Loading II - Same as Loading I, except water surface three feet below the top of the wall and the resultant of forces to fall within the middle-third of the base.

(c) Loading III - Channel empty and active earth with parking surcharge behind wall considering building removed, 50 lbs. per square foot wind applied from landside.

(3) Typical Design Section. Design computations are included in Appendix A for the typical wall section. A detail of the typical wall section and loading conditions are shown on Plate 5-6. In the

design, no landside passive resistance was assumed. An adequate factor of safety against sliding is available considering only the static frictional resistance of the concrete base on the bedrock. Bearing pressures are well within allowable values. Drilled in anchors will be provided to engage the rock mass assumed in the stability analysis.

c. T-Type Flood Walls.

(1) General. T-type flood walls will be utilized for approximately 730 feet of protection. Two design sections have been developed for the reach of wall extending from Station 2+00 to Station 6+50. From Station 2+00 to Station 3+20, the wall will have a flat base founded on bedrock. The downstream portion of this reach will be founded on sand and gravel soil utilizing a sloping base to resist horizontal forces. One typical sloped base design section suffices for the reach of wall extending from Station 19+20 downstream to the vehicular gate. This wall will be founded on a sandy silt soil.

(2) Loading Conditions.

(a) Loading I - Water surface to top of wall riverside. Uplift is assumed varying from full head at bottom of base slab riverside to zero at the ground surface landside by method of creep. Resultant to fall within the middle-half of base.

(b) Loading II - Same as Loading I, except water surface 3 feet below the top of the wall and the resultant of force to fall within the middle-third of the base.

(c) Loading III - Channel empty, 50 lbs. per square foot wind load applied from landside. A 2'-0 earth surcharge on landside in addition to the wind load is applied to allow for vehicular parking along the wall. Resultant of forces to fall within middle-third of base under this loading condition.

(3) Typical Design Sections. Computations are included in Appendix A, for the T-wall design sections previously noted. The typical "T" wall design sections are at Stations 2+80, 5+50, and 20+00. Graphical presentations of the loading conditions and typical design sections are shown on Plate 5-5. Special monoliths at angle points and at point where the corner of an existing building is within base of wall will be checked during preparation of contract documents.

d. I-Type Flood Walls.

(1) General. The two upstream sections of the I-type wall, Station 6+50 to 6+80 and Station 18+85 to 19+20, serve as transitions

between the juncture of the T-type flood walls and earth dike. The section adjacent and downstream of the vehicular gate is used to decrease the height of earth dike; thereby, minimizing encroachment on adjacent property. In all instances, the height of protection provided by the I-type wall is limited to ten feet. The walls will consist of PZ-32 steel sheet piling driven through the dike fills to a required penetration. The portion of wall extending above grade and the top 5'-6" portion below grade will be constructed of reinforced concrete. The piling will be embedded 4 feet into the concrete section. The bottom of sheeting will be determined by seepage consideration in addition to stability requirements where the wall joins adjacent structures.

(2) Loading. The critical loading used in design of the I-walls assumes water at the top of the wall riverside and at the ground surface landside. Ground surface is assumed at the same level on both sides of the wall.

(3) Typical Design Section. The design computations for the I-wall downstream of the vehicular gate are included in Appendix A. This wall has the maximum protection height of 10 feet and the least favorable soil conditions encountered at any of the three I-wall locations. Similar procedures will be followed in the design of I-walls for the other locations and for lesser heights of protection. Plate 5-6 of this memorandum shows details of this maximum design section and assumed loading.

e. Concrete Gravity Walls.

(1) General. Low concrete gravity walls will be utilized at two locations. One wall will be constructed at the upstream end of the project adjacent to Saxonville Pond east of the existing dam. It will extend from an existing wall and run parallel with Central Street to the intake structure and thence from the intake structure to Water Street. The layout and connection to the intake structure will be determined in final design. The top of the wall will be at Elevation 154.2, approximately 5'-9" above grade. The second wall is located at the downstream end of the project adjacent to the end of the dike at Danforth Street. The top of the wall is set at Elevation 135.0, 6 inches above the top of the abutting dike. The bottom of wall is set at Elevation 126.5, approximately 4'-6" below existing sidewalk grade. The town of Framingham has recently awarded a contract for the design of a relocated Danforth Street Bridge. It is anticipated that the bridge design and layout will eliminate the requirements for this wall by the time construction documents are prepared.

(2) Loading Condition. The critical loading for the gravity walls is when flood is at top of the protection waterside and at the

ground surface landside. For this loading, the resultant falls within the middle-third of the base.

(3) Typical Design Sections. Typical design calculations have been included in Appendix A. Plate 5-6 shows the assumed wall loading.

f. Concord Street Gate.

(1) General. The street gate will be a miter type steel gate and will close a 55' clear opening between abutment walls at Concord Street during flood conditions. When closed the gate will have a base angle of 30° with a line normal to the center line of opening. When folded back into the abutment, the skin plate will be on the exposed side, thereby presenting a more pleasing appearance. A concrete monolith containing a base slab, sill and gate housing will support the gates and serve as the flood wall in this reach of the river. The design of the structure is similar to the T-type walls described herein. Soil conditions at the location of the gate supporting structure require that it be designed as a monolith to eliminate the effects of expected differential settlements. Because the structure will be viewed by the traveling public from nearby residences and establishments, exposed surfaces of concrete abutments will be architecturally treated. Surfaces of flood walls adjacent to the gate structure facing on existing residence and business establishments will be similarly treated. Plate 5-7 shows details and loading of the gate and abutment monoliths.

(2) Elevations and Dimensions. The following critical elevations and dimensions are established for the vehicular gate.

Elevation of top of sill	127.3
Elevation of top of gate	136.0
Elevation of bottom of base slab	115.0
Height of gate leaf	8.7

(3) Loading Conditions.

(a) Loading I - Storm condition with water to the top of closed gate, uplift varying from full head at bottom of base riverside by creep method to zero at ground surface landside and the resultant to fall within mid-half of base.

(b) Loading II - Storm condition with water three feet from top of the gate, uplift as in Loading I and resultant to fall within mid-third of base.

(c) Loading III - Water down, gates in pockets unjacked, and HS-20 highway loading in two lanes over base slab.

(d) Loading IV - Water down, gate closed, 50 lbs. per square foot wind load from landside.

(4) Concrete Abutment Design. Stability computations for the gate monolith and the various conditions of load are included in Appendix A. The base slab has been divided into a series of four 6-foot wide strips for analysis by computer program "Effram" -Analysis of Plane Frames on Elastic Foundations, Program No. 713, DOF 7110. The analysis allows a more economical design of base slab than would result from assuming a rigid foundation. An example of one strip checked for several conditions of loading has been included in the appendix. The other strips will be investigated for the various loading cases in determining the reinforcing of the base slab. The gate pocket monoliths extending to the riverside from the gate monolith will be designed as free standing flood walls and have not been included with the gate monolith in stability analysis. A contraction joint separates the gate monolith from the pocket monoliths.

(5) Steel Gate Design. The skin plate is designed to span continuously over the supporting 16" wide flange members. A36 steel, painted, is used for the skin plate and framing members. The skin plate is 5/16" thick. The top and bottom horizontal members are designed for the skin plate reactions plus the axial loads imposed by miter action. The axial loads are then taken out directly to the concrete abutment through the hinge fittings. The intermediate horizontal member spanning between vertical end posts carries only skin plate reactions. Torsional stress effects in the gate leafs were investigated and found to be of no consequence; however, diagonal bracing of gate leaf is provided to eliminate fabrication deformation and to provide torsional strength.

g. Pumping Station.

(1) General. The station is located in back of the protection line. The roof, operating floor and portions below grade will be constructed of reinforced concrete. The walls above grade will be constructed of 4" face brick backed by 8-inch thick concrete masonry units. The outisde dimensions of the superstructure will measure 24'-9" x 17'-5". The intake and gravity discharge chambers will be 4'-0" wide. Flood discharge will be by two 18-inch pipes carried over the dike. Plate 5-9 shows details of the pumping station. Typical design sections are included in Appendix A.

(2) Roof Slab and Superstructure. The roof slab will be a 5-inch structural slab of uniform thickness overlain with fill concrete to provide slope for proper drainage. A 5'-0" x 11'-0" opening with

operable hatch will be provided in the roof slab to facilitate removal of equipment from the operating floor. This will negate the necessity of providing a crane and attendant framing within the building. The roof slab will be supported on concrete spandrel beams and by interior beams framing the hatch opening. The beams in turn will be supported by the concrete masonry bearing walls.

(3) Operating Floor. The operating floor is a reinforced concrete slab and beam construction. Beams form the opening at each of the pump locations. The slab is designed for a live load of 340 lbs. per square foot which allows for the possibility of equipment being placed on the floor in other than its design location. The beams are designed for a minimum live load of 100 lbs. per square foot plus equipment loads and pump thrust.

(4) Stability. Because the structure is located behind the protection line, not subject to flood loading, the stability analysis consists of a buoyancy check considering the station to be empty and the water outside at a ponding elevation of 121.0. A factor of safety of 1.8 against uplift exists under such conditions. A maximum bearing pressure under the entire base slab area of 1,200 lbs. per square foot results from the assumption of maximum loading within the structure and minimum groundwater condition outside. During the preparation of contract documents, various conditions of unbalanced loading within the structure will be investigated.

(5) Substructure. The exterior substructure walls are designed as vertical slabs fixed at the base slab and laterally restrained at the operating floor slab. The walls separating the intake and gravity discharge chambers from the pump chamber are designed as restrained at the operating floor and fixed at the base with a full water load in the discharge and intake chambers, with the pumping chamber empty. The base slab is designed as a two way slab restrained by the walls resting on it.

G. MECHANICAL DESIGN

14. GENERAL. Plate 5-10 shows the general arrangement of pumps and other mechanical equipment, and Appendix B shows pump computations and curves. Electric service is not considered adequately reliable at the station for electric motor drive.

15. PUMPING EQUIPMENT. The pumps will be driven by diesel engines through right angle gear units. The pumps will be vertical, fixed blade, propeller or mixed-flow type of standard construction using steel columns and discharge elbows; cast iron, cast steel or steel

plate bowls and suction bells; cast steel or bronze impellers; stainless steel pump shafts; and bronze bearings. Each pump shall be provided with its own individual electric motor driven centralized pressure lubrication system. All pumps will be capable of operation against all river stages up to the top of the dike.

Trash racks are not required as all inlets to the drains leading to the sump are adequately screened.

Two pumps will be provided with each pump having a capacity of 23.3 c.f.s. (10,500 gpm), two-thirds of the required station capacity, against the standard project flood with suction sump at high water elevation. Each pump will discharge over the top of the dike. Two sluice gates, one located in front of each pump, will seal off the pump sump except when opened for activation of the station. Gates will be sized for an entrance velocity of approximately 2.5 f.p.s. with all pumps operating. A sluice gate will be provided on the river side of the dike to close off the gravity flow conduit.

16. SLUICE GATES. All sluice gates at the pumping station will be seating pressure type of standard cast iron construction with bronze seals. Electric motor-operated floorstands will be installed for operation of each gate. Floorstand will also be provided with a hand wheel for manual operation.

17. SUMP PUMP. An electric motor-operated wet pit type sump pump will be provided in the pumping station for dewatering and for handling any leakage into the sump.

18. HEATING. The station will be equipped with an oil-fired warm air furnace thermostatically controlled to maintain inside temperature at 55°F. In addition, a humidistat will override the thermostat to operate the heater as necessary to maintain the relative humidity at not higher than 50%.

19. VENTILATION. A small fan with duct work will be provided for ventilation of the sump. Another fan with duct work will provide air for combustion and cooling for the air cooled diesel operated emergency generator. The pump diesel engines will be radiator cooled with each radiator connected to exterior wall louver by duct work to prevent short circuiting of cooling air. Inlet louvers will be provided for air supply to the radiators.

20. WATER LEVEL INDICATORS. Indicators for both the station sump and the river water levels will be provided in the station. The sump water level indicator will also incorporate controls to provide for automatic shutdown of pump diesel engines at a predetermined sump low water level. An alarm will also be sounded at shutdown.

H. ELECTRICAL DESIGN

21. ELECTRIC SERVICE. Electric service will be obtained from the Boston Edison Company at 120/208V, 3-phase, 4-wire. The service will be extended from existing lines on Watson Place overhead approximately 450 feet along a new access road to the pumping station.

22. INTERIOR ELECTRICAL FEATURES.

a. Wiring System. All conductors will have heat and moisture resistant type insulation and will be installed in rigid steel conduit. Layout of the electrical system is shown on Plate 5-11.

b. Communication System. Conduit will be installed for use by the New ngland Telephone Company to provide telephone service.

c. Emergency Lighting. A battery operated emergency lighting unit will be provided.

23. ELECTRIC LOAD DATA.

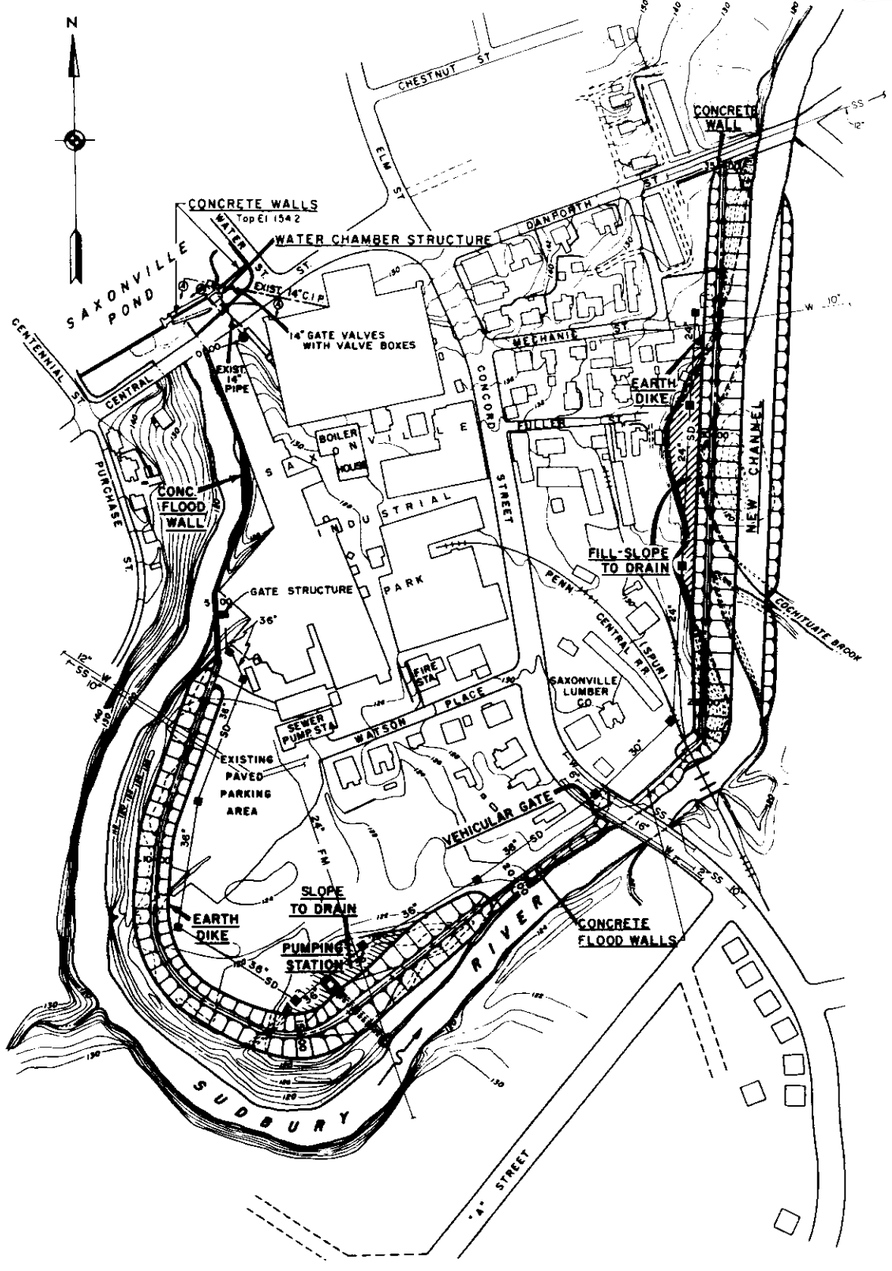
Item	Connected Load	Demand Load
Furnace	2@ 1/3 HP	2@ 1/3
Generator Fan	1/8 HP	1/8
Sump Exhaust Fan	1/3 HP	
Grease Unit	1/3 HP	1/3
Sump Pump	1 HP	
Floor Stand	2@ 1 HP	1
Sluice Gate	2 HP	2
Lighting	2.6 KW	2.6
Receptacles	1.2 KW	
TOTAL LOADS	10.2 KVa	6.7 KVa

A 10KW, 3 phase, 4 wire 120/208, 60 hertz diesel engine generator set is proposed to handle emergency demand loads.

I. CORROSION PROTECTION

24. CORROSION PROTECTION. The steel sheet piles near Stations 6+50, 19+20 and east of Concord Street are the only metal materials embedded in earth. These piles are driven completely in undisturbed earth which consists principally of sandy fine silt, silty fine sand and gravel with cobbles.

The pH of a soil sample in boring FD-2 was 7.8. As these foundation materials are considered non-corrosive, further tests were not considered warranted. The sheet piling will not be painted or otherwise protected. This action conforms to instructions contained in Guide Specification CE-1409.

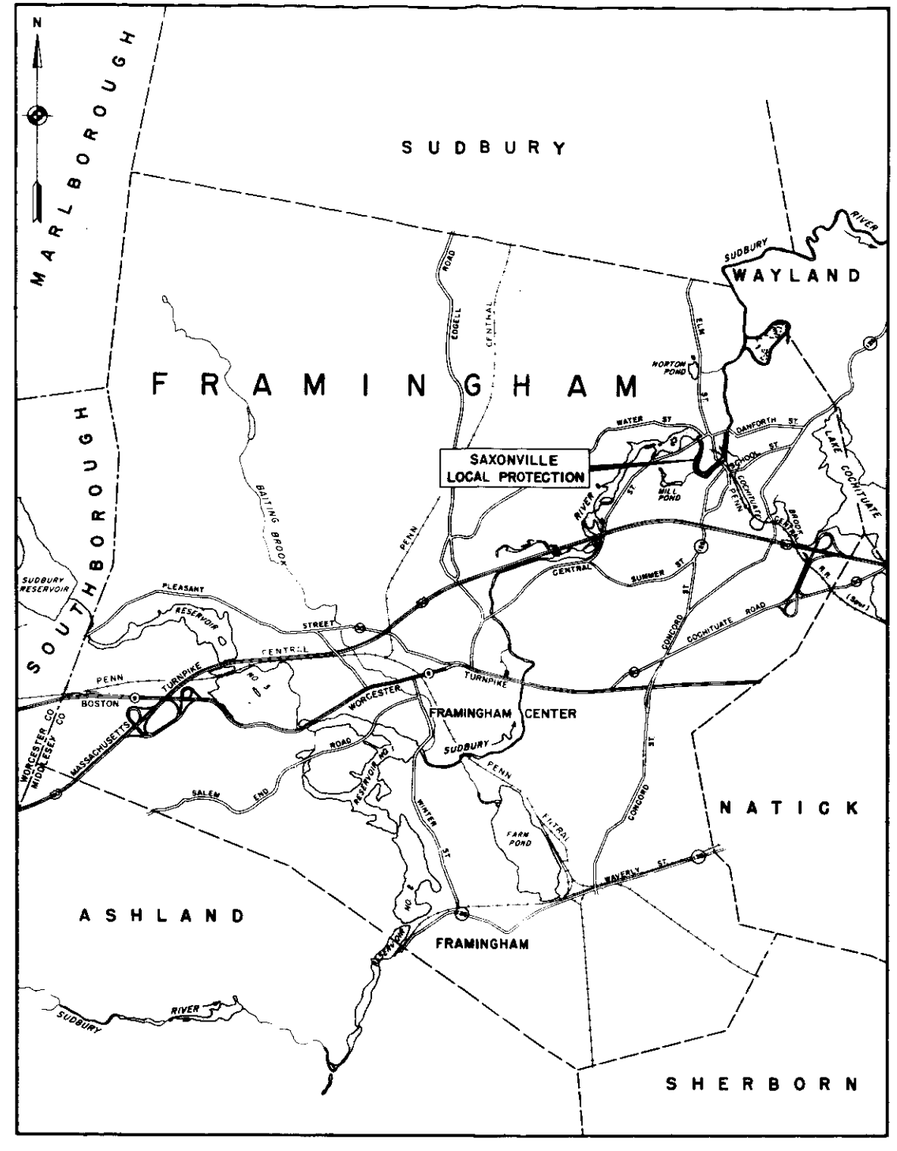


GENERAL PLAN

SCALE IN FEET

1" = 100'-0" 100' 0 100 200

LEGEND – UTILITIES		
NEW	EXISTING	DESCRIPTION
—SD—		STORM DRAIN
■		DRAIN INLET
	—W—	WATER LINE
	—SS—	SANITARY LINE
	—G—	GAS LINE
	—FM—	FORCE MAIN



VICINITY MAP

SCALE IN FEET

1000 0 1000 3000 5000

GRAPHIC SCALES

1" = 10'-0" 10' 0 10' 20'

1" = 5'-0" 5' 0 5' 10'

NOTES:

1. Topography shown is from survey by the Corps of Engineers with two-foot contour interval.
2. Elevations are in feet and refer to mean sea level datum

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS

DR. BY	TR. BY	CK. BY
J B M	SDL	

WATER RESOURCES DEVELOPMENT PROJECT
FRAMINGHAM, MASSACHUSETTS

SAXONVILLE LOCAL PROTECTION

GENERAL PLAN AND VICINITY MAP

SUDBURY RIVER MASSACHUSETTS

PROJECT ENGINEER

CHIEF, PLANS & REPORTS BRANCH

APPROVED DATE

CHIEF ENGINEERING DIVISION

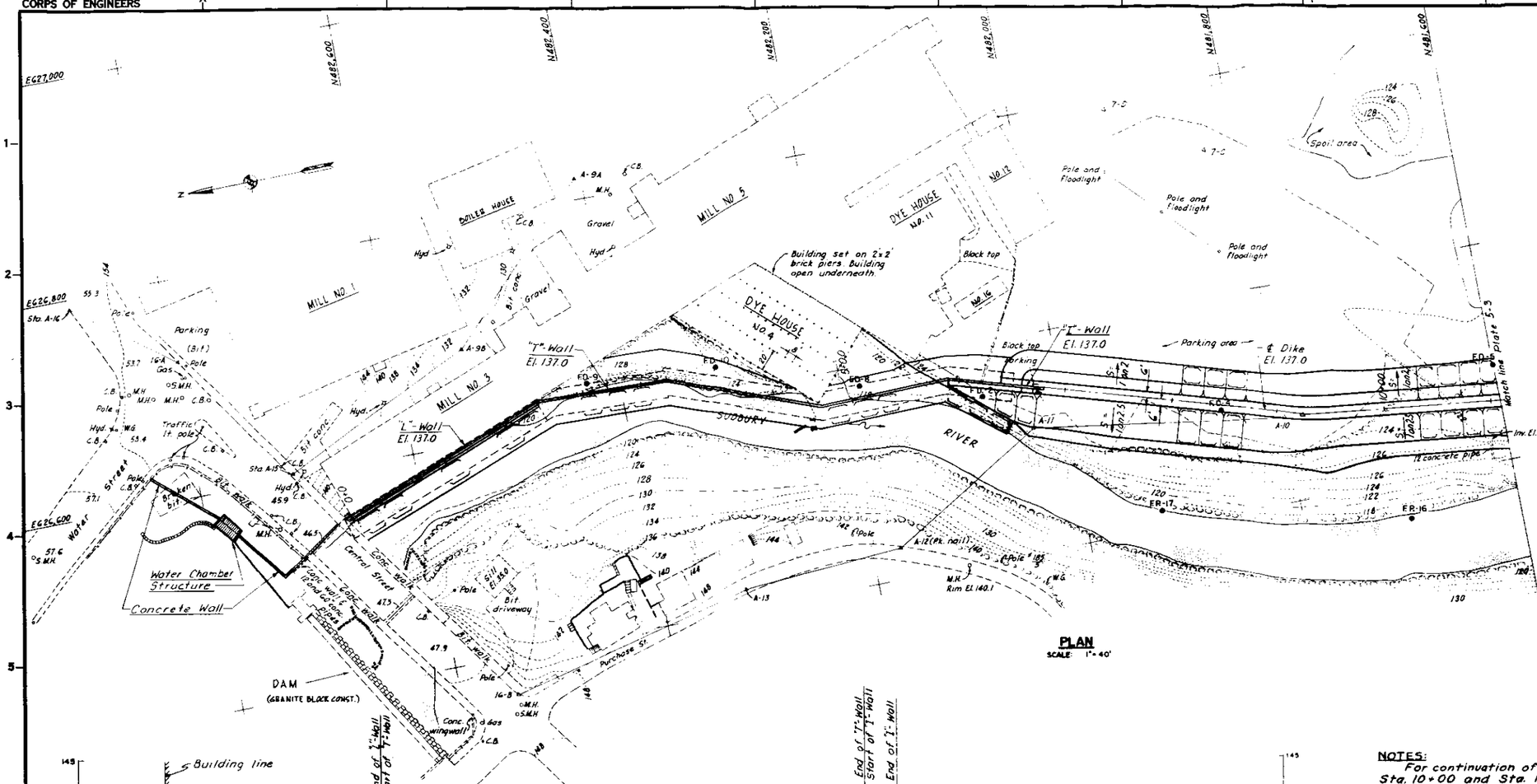
SCALE AS SHOWN

DRAWING NUMBER

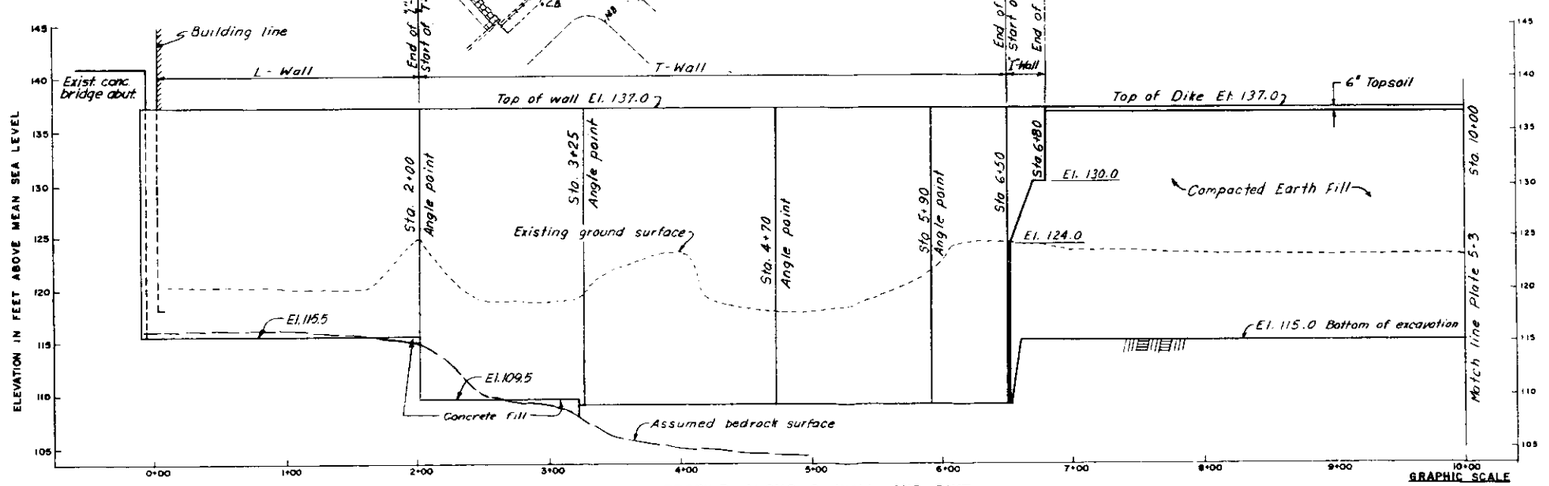
SHEET

CORPS OF ENGINEERS

U. S. ARMY



PLAN
SCALE: 1" = 40'



PROFILE ALONG ℄ WALL AND DIKE

SCALE: HORIZ. 1" = 40'
VERT. 1" = 5'

NOTES:
For continuation of profile between Sta. 10+00 and Sta. 11+08±, see Plate 5-3.

REVISION	DATE	DESCRIPTION	BY

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

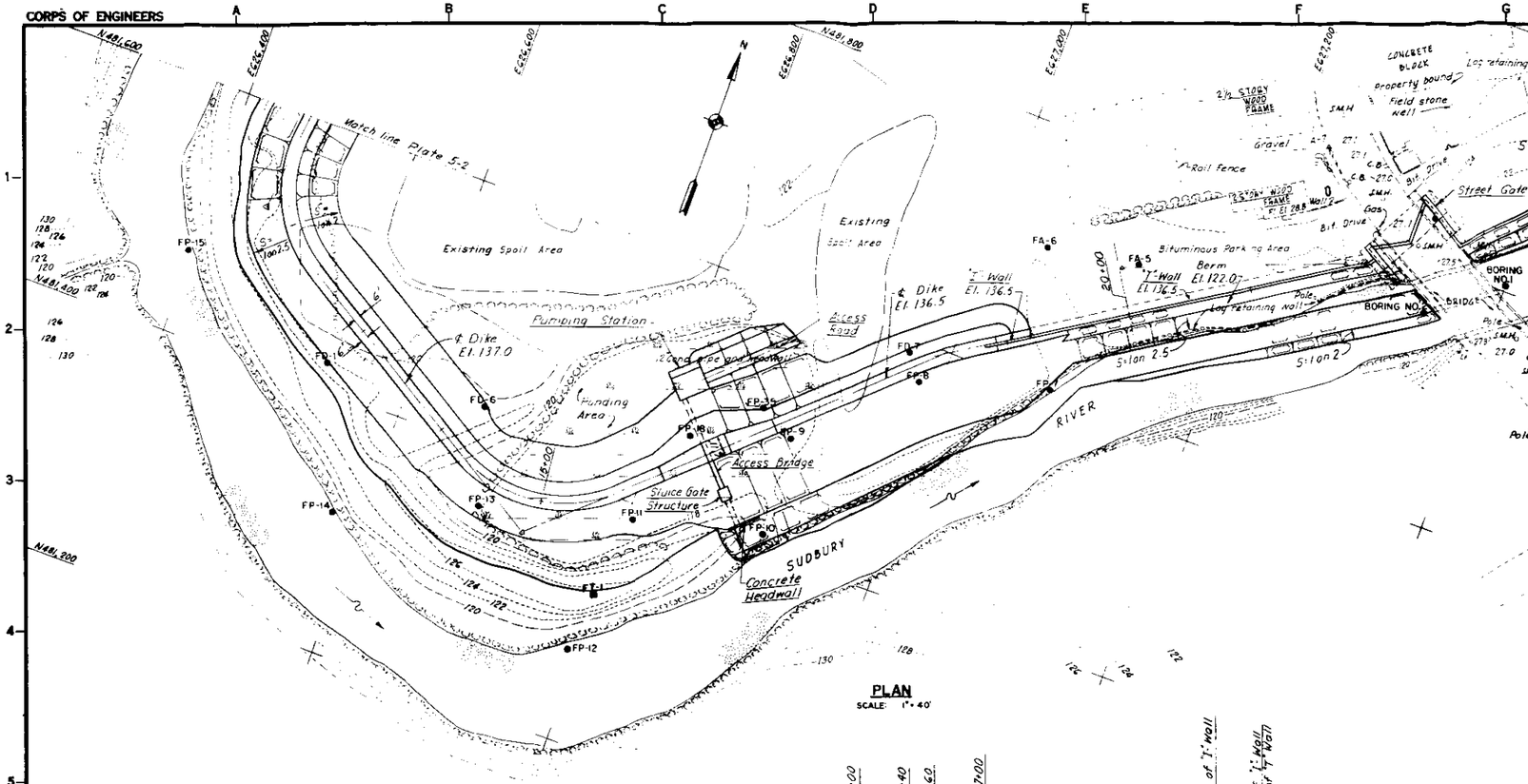
WATER RESOURCES DEVELOPMENT PROJECT
FRAMINGHAM, MASSACHUSETTS

SAXONVILLE LOCAL PROTECTION
WALLS AND DIKES
PLAN AND PROFILE NO. 1

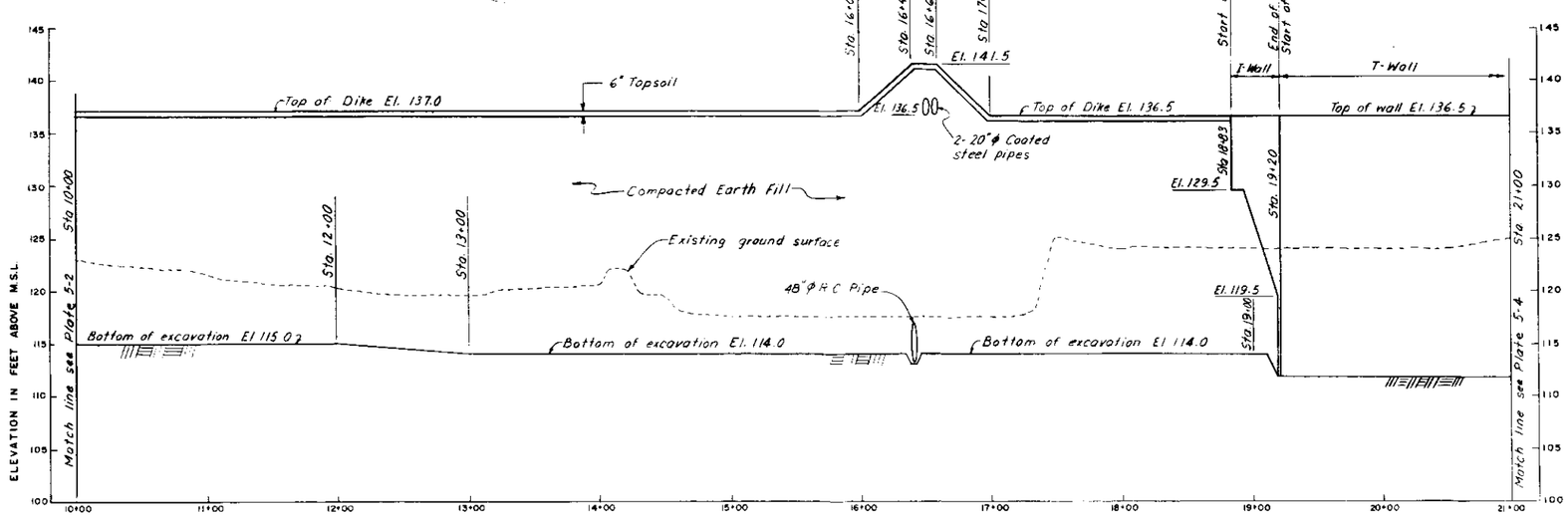
SUDBURY RIVER MASSACHUSETTS

SCALE | SPEC. NO. CIV. ENG. 19-696-
DRAWING NUMBER
SHEET

CORPS OF ENGINEERS — U. S. ARMY



PLAN
SCALE: 1" = 40'



PROFILE ALONG ℄ WALL AND DIKE
SCALE: HORIZ 1" = 40' VERT 1" = 5'

NOTES:
For continuation of profile between Sta. 21+00 and Sta. 24+95±, see Plate 5-4

GRAPHIC SCALE
1" = 40'-0" 40' 0 40' 80'
1" = 5'-0" 5' 0 5' 10'

REVISION	DATE	DESCRIPTION	BY

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

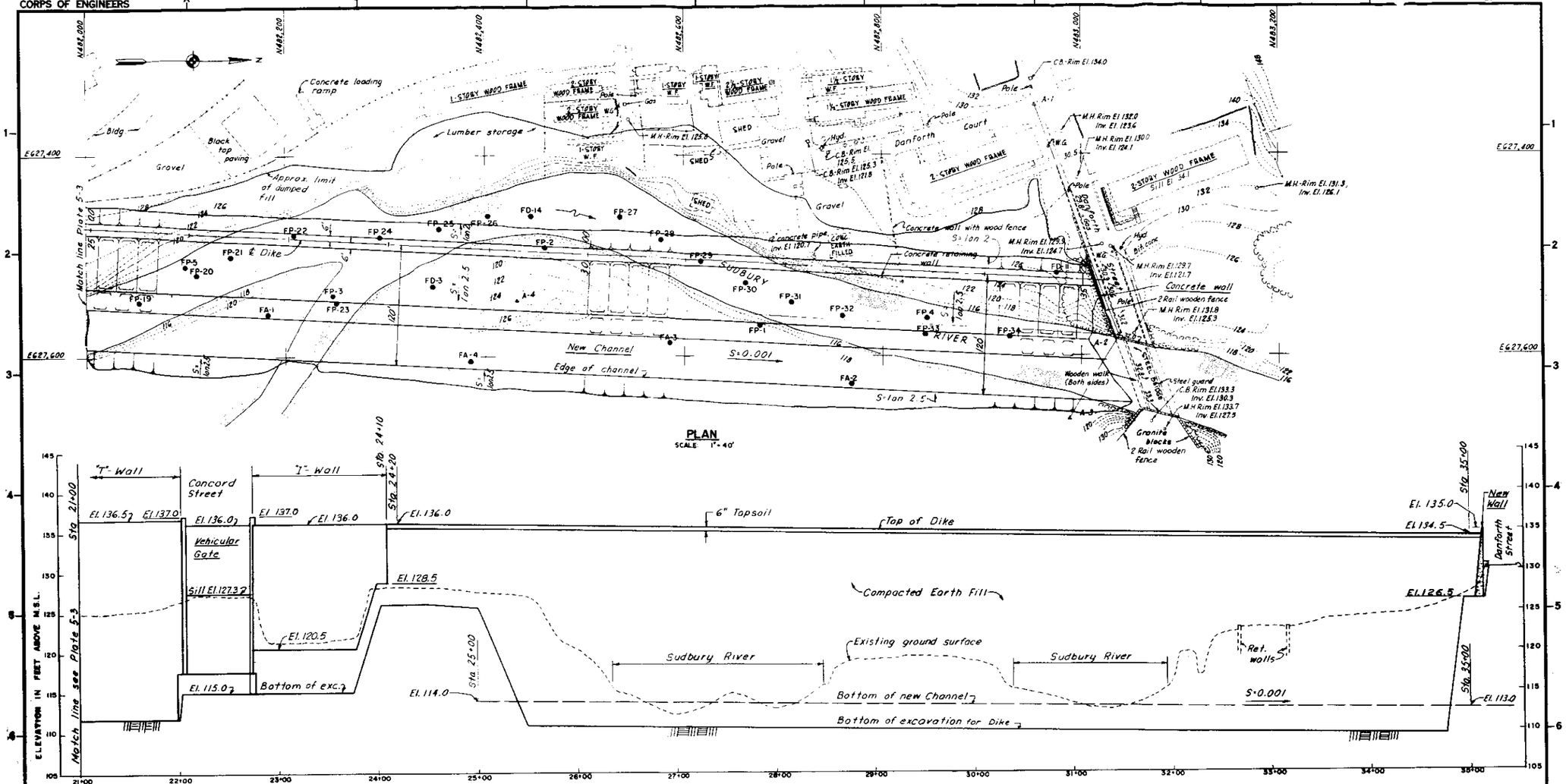
DES. BY | DR. BY | CK. BY
SUBMITTED: CHIEF, SECTION
REVIEWED: PROJECT ENGINEER
APPROVED: CHIEF, BRANCH

WATER RESOURCES DEVELOPMENT PROJECT
FRAMINGHAM, MASSACHUSETTS
SAXONVILLE LOCAL PROTECTION
WALLS AND DIKES
PLAN AND PROFILE NO. 2
SUDBURY RIVER — MASSACHUSETTS
APPROVED: CHIEF, ENGINEERING DIVISION — DATE
SCALE | SPEC. NO. | DRAWING NUMBER | SHEET

PLATE 5-3

CORPS OF ENGINEERS

U. S. ARMY



GRAPHIC SCALE

1" = 40'-0" 40' 0 40' 80'

1" = 5'-0" 5' 0 5' 10'

REVISION	DATE	DESCRIPTION	BY

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

WATER RESOURCES DEVELOPMENT PROJECT
FRAMINGHAM, MASSACHUSETTS

SAXONVILLE LOCAL PROTECTION
WALLS AND DIKES
PLAN AND PROFILE NO. 3

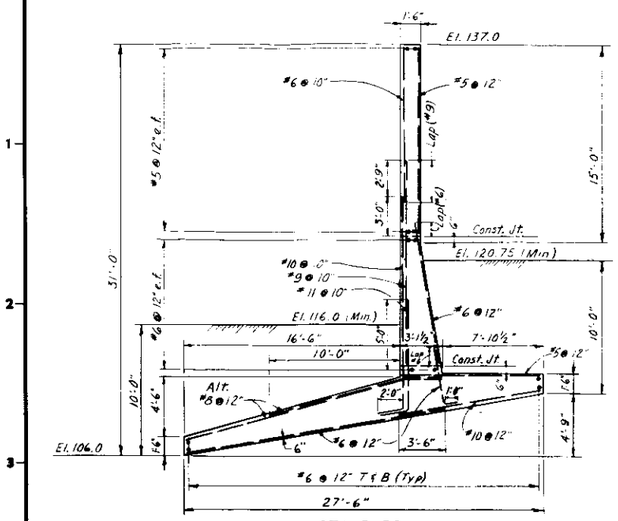
SUDBURY RIVER — MASSACHUSETTS

SUBMITTED: CHIEF, SECTION
REVIEWED: PROJECT ENGINEER
APPROVED: CHIEF, BRANCH
APPROVED: CHIEF, ENGINEERING DIVISION
DATE

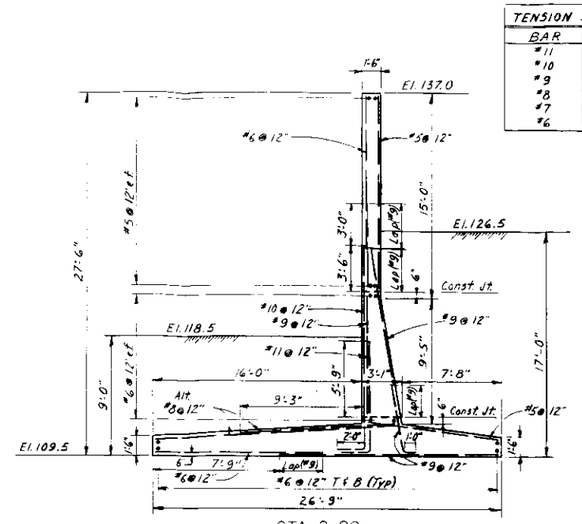
SCALE
SPEC. NO.
DRAWING NUMBER
SHEET

PLATE 5-4

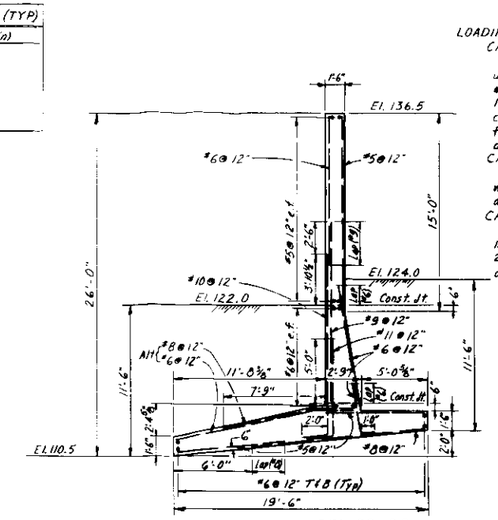
TENSION SPLICES (TYP)	
BAR	LAP (in)
#11	60
#10	48
#9	39
#8	30
#7	24
#6	18



STA 5+50
(APPLIES STA 3+20± TO 6+50±)
SCALE 1/4"=1'-0"



STA 2+80
(APPLIES STA 2+00± TO 3+20±)
SCALE 1/4"=1'-0"



STA 20+00
(APPLIES STA 19+20± TO 22+00±)
SCALE 1/4"=1'-0"

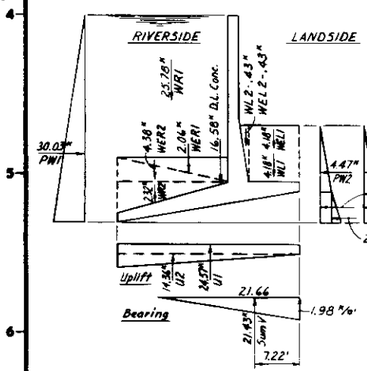
LOADING CONDITIONS

CASE I
Water to top of wall, uplift varying from full head at riverside heel to 0 at landside ground surface by creep method. Resultant to fall in middle half. Normal design stresses.

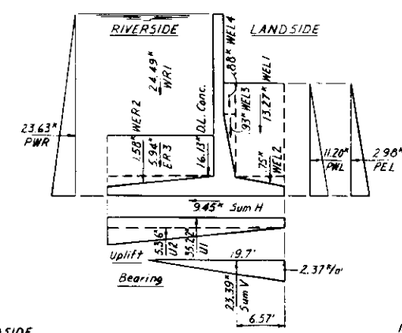
CASE II
Same as Case I except water down 3'-0" and resultant to fall in the middle third.

CASE III
Water down, 50 #/□' wind load applied to landside and 2'-0" equivalent surcharge applied on landside.

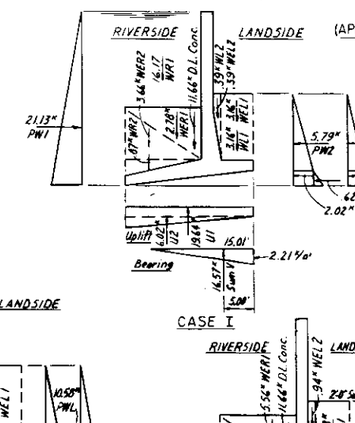
NOTES:
Cover for steel reinforcement shall be 4" except as noted.
All walls are on Left Bank.



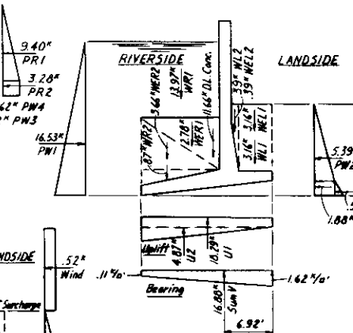
CASE I



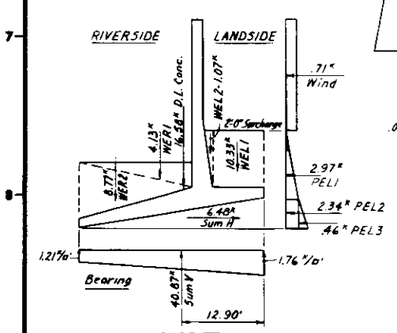
CASE I



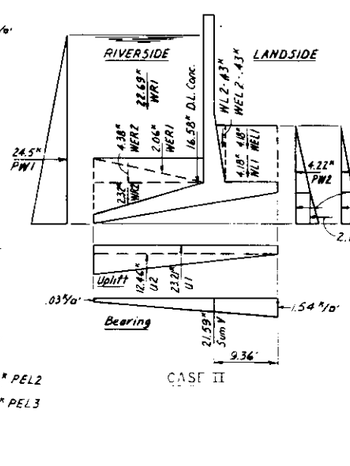
CASE I



CASE II



CASE III



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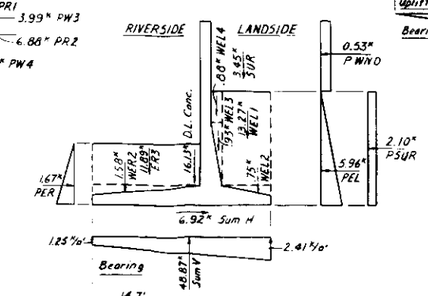
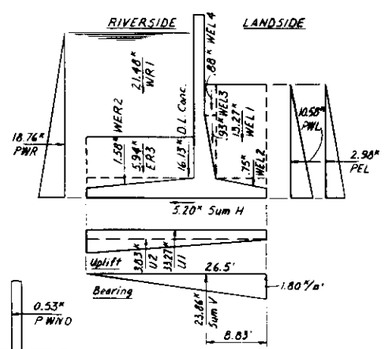
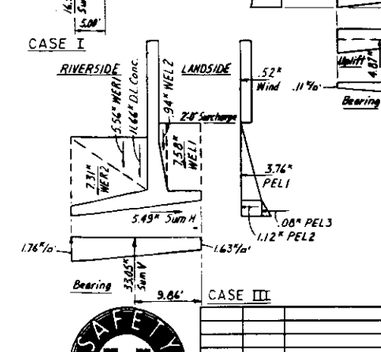


CASE III



CASE II



CASE III

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

REVISION	DATE	DESCRIPTION	BY

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

DES. BY RLH | DR. BY HDP | CK. BY

SUBMITTED:
CHIEF, SECTION
APPROVAL RECOMMENDED:
CHIEF, TECH. ENG. BRANCH
REVIEWED:
PROJECT ENGINEER
APPROVAL RECOMMENDED:
CHIEF BRANCH

WATER RESOURCES DEVELOPMENT PROJECT
FRAMINGHAM, MASSACHUSETTS

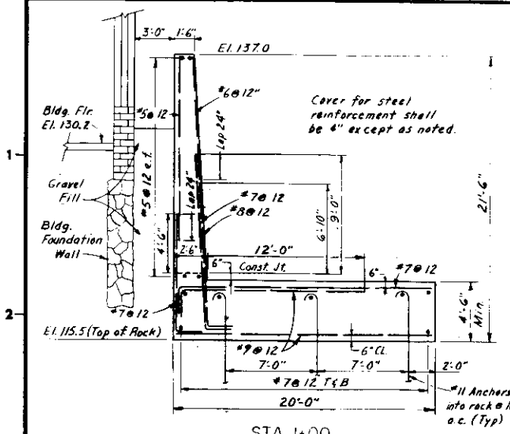
SAXONVILLE LOCAL PROTECTION
FLOOD WALLS
LOADING AND STRUCTURAL DETAILS NO. 1

SUDBURY RIVER MASSACHUSETTS

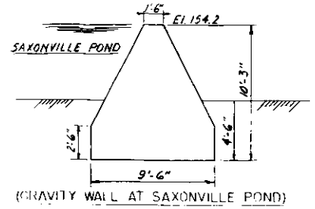
APPROVED DATE
CHIEF, ENGINEERING BRANCH

SCALE | SPEC. NO. | DRAWING NUMBER | SHEET

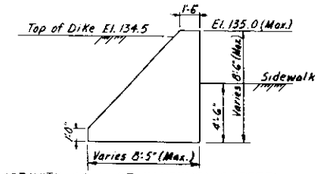
CORPS OF ENGINEERS U. S. ARMY



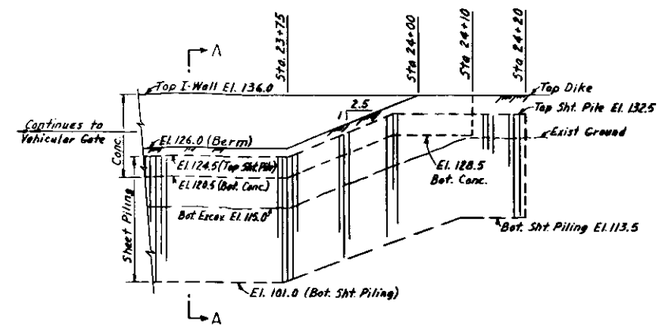
STA 1+00
(APPLIES STA 0+00 TO 2+00)
SCALE 1/4"=1'-0"



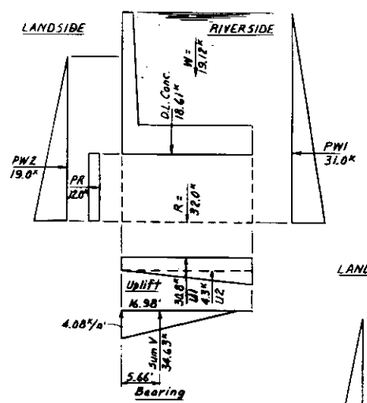
(GRAVITY WALL AT SAXONVILLE POND)
SCALE 1/4"=1'-0"



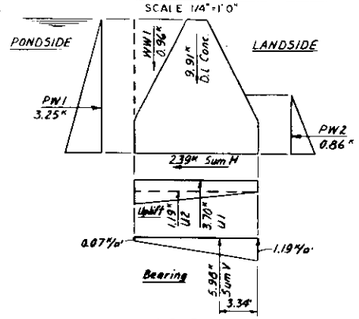
(GRAVITY WALL AT DANFORTH STREET)
SCALE 1/4"=1'-0"



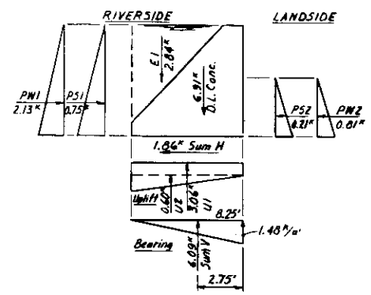
ELEVATION I-WALL TO DIKE TRANSITION
(STATION 23+75 TO STA 24+20)
SCALE 1"=10'-0"



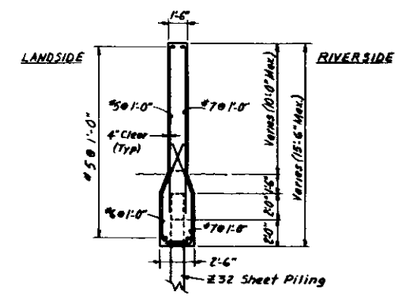
CASE I



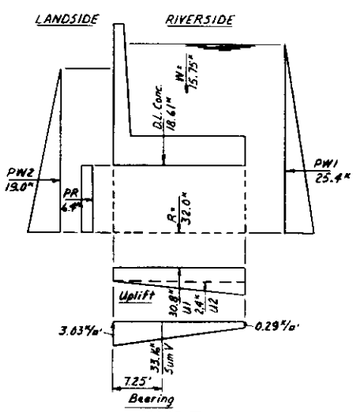
LOADING



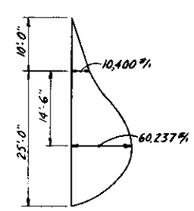
LOADING



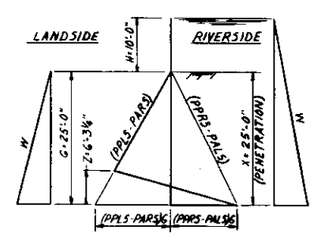
SECTION A-A
(TYPICAL I-WALL)
SCALE 1/4"=1'-0"



CASE II



MOMENT DIAGRAM
(I-WALL)



LOADING
(I WALL)

NOTE:
For Loading Conditions see 5b.

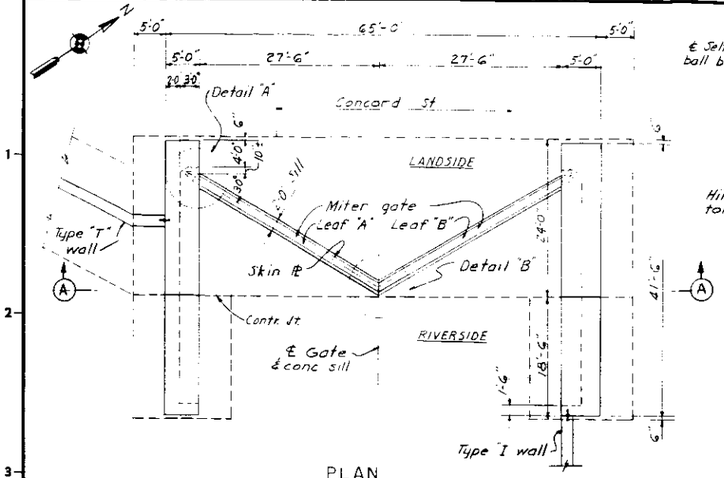
GRAPHIC SCALES
1/4" 1' 0" 4' 0 4' 8'

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

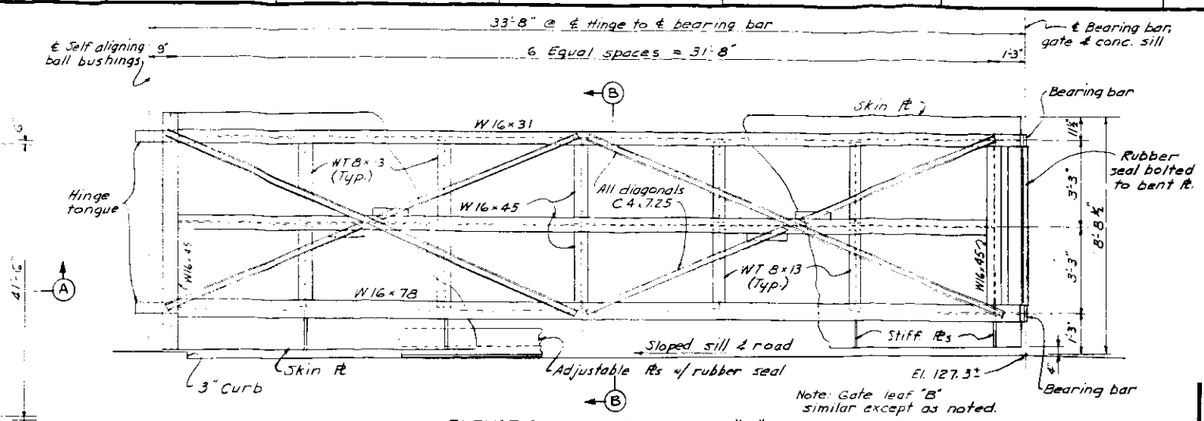
WATER RESOURCES DEVELOPMENT PROJECT
FRAMINGHAM, MASSACHUSETTS
SAXONVILLE LOCAL PROTECTION
FLOOD WALLS
LOADING AND STRUCTURAL DETAILS NO. 2
SUDBURY RIVER MASSACHUSETTS

SCALE SPEC. NO. SHEET

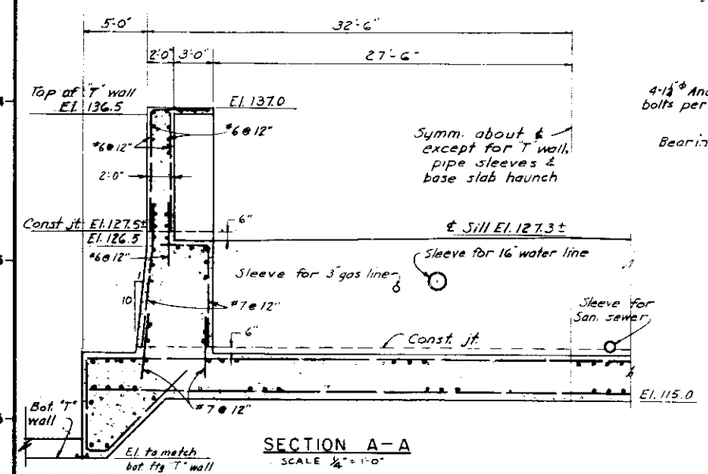
PLATE 5-6



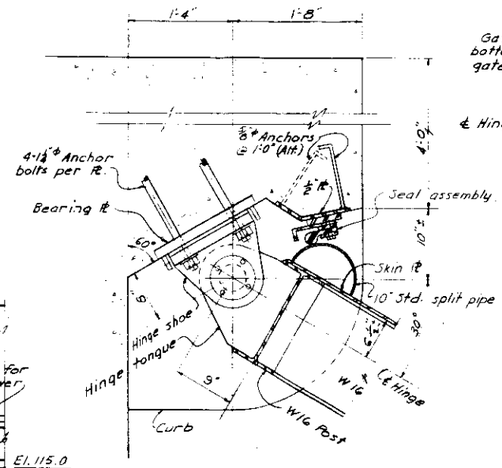
PLAN
SCALE 1/8" = 1'-0"



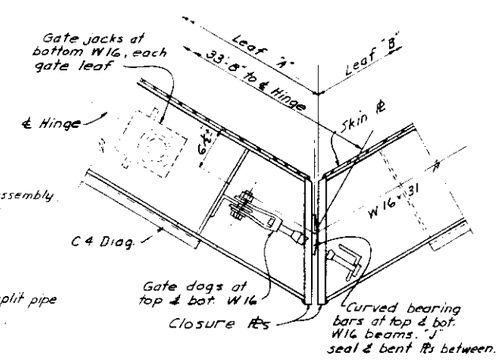
ELEVATION — GATE LEAF "A"
SCALE 1/2" = 1'-0"



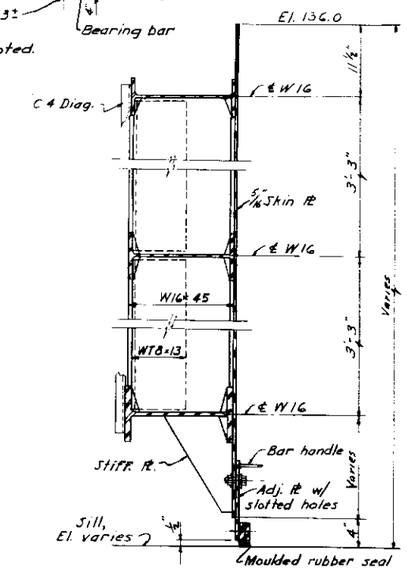
SECTION A–A
SCALE 1/4" = 1'-0"



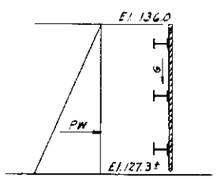
DETAIL "A"
SCALE 1 1/2" = 1'-0"



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

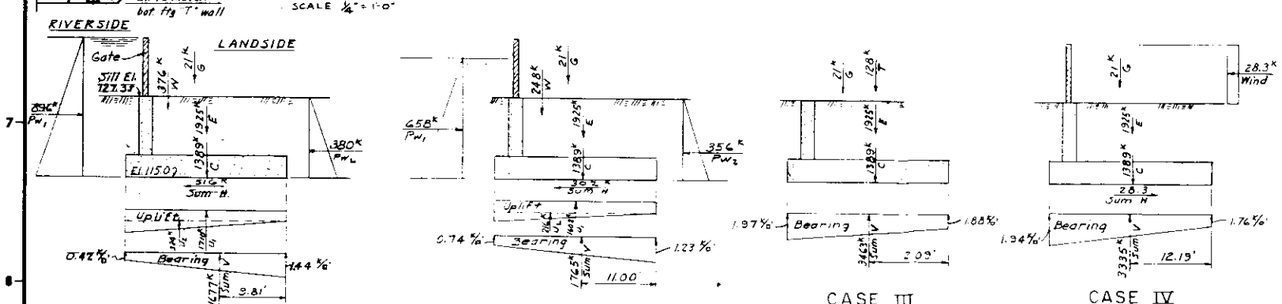


SECTION B–B
SCALE 1 1/2" = 1'-0"



GATE LOADING DIAGRAM

All steel to conform to ASTM A-36



CASE I
Gate closed, water to top of gate, uplift varying from full head at bottom of base on riverside to 0 at landside ground surface by method of creep. Resultant to fall within middle half of base slab.

CASE II
Same as Case I except water 3' down from top of gate and resultant to fall within middle ⅓ of base slab.

CASE III
Gate open in pocket, 2 HS-20 trucks on roadway above base slab, no water

CASE IV
Gate closed, not jacked, no water, 50 #/□' wind applied, landside.

FOUNDATION LOADING DIAGRAMS

GRAPHIC SCALES

REVISION	DATE	DESCRIPTION	BY

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

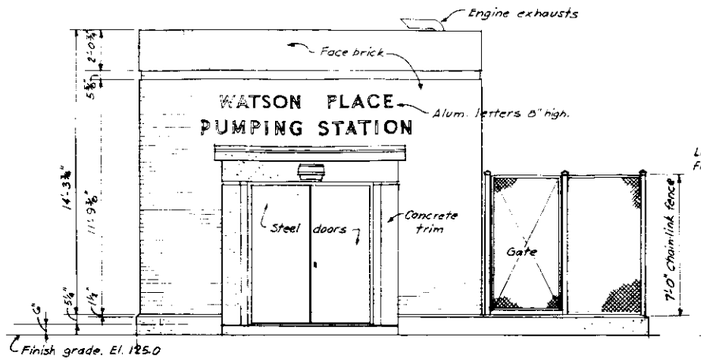
WATER RESOURCES DEVELOPMENT PROJECT
FRAMINGHAM, MASSACHUSETTS

SAXONVILLE LOCAL PROTECTION
CONCORD STREET GATE
STRUCTURAL DETAILS
SUDBURY RIVER MASSACHUSETTS

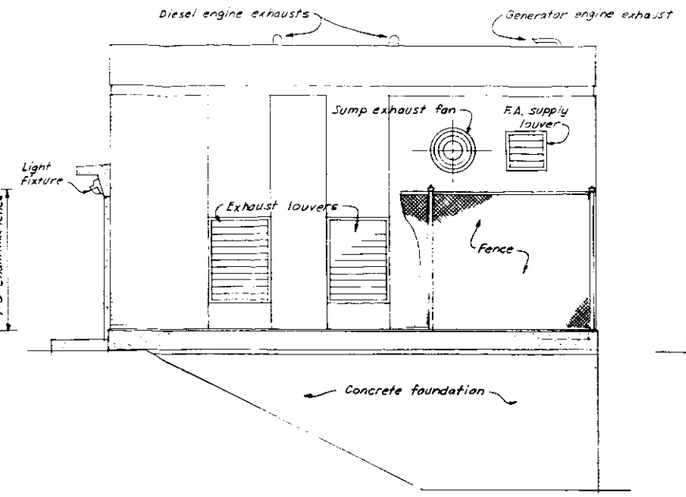
APPROVED DATE
CHIEF, ENGINEERING DIVISION

SCALE AS SHOWN SPEC. NO.
DRAWING NUMBER
SHEET

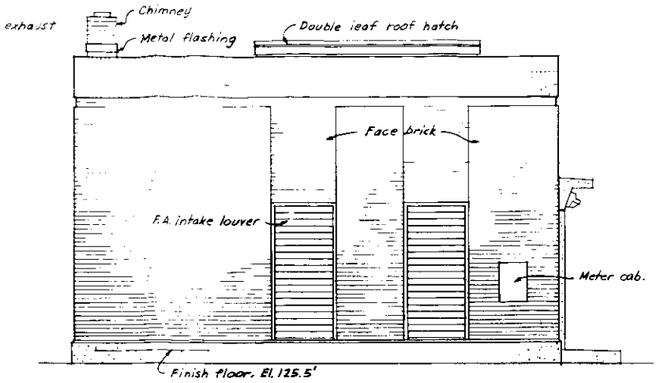
CORPS OF ENGINEERS U. S. ARMY



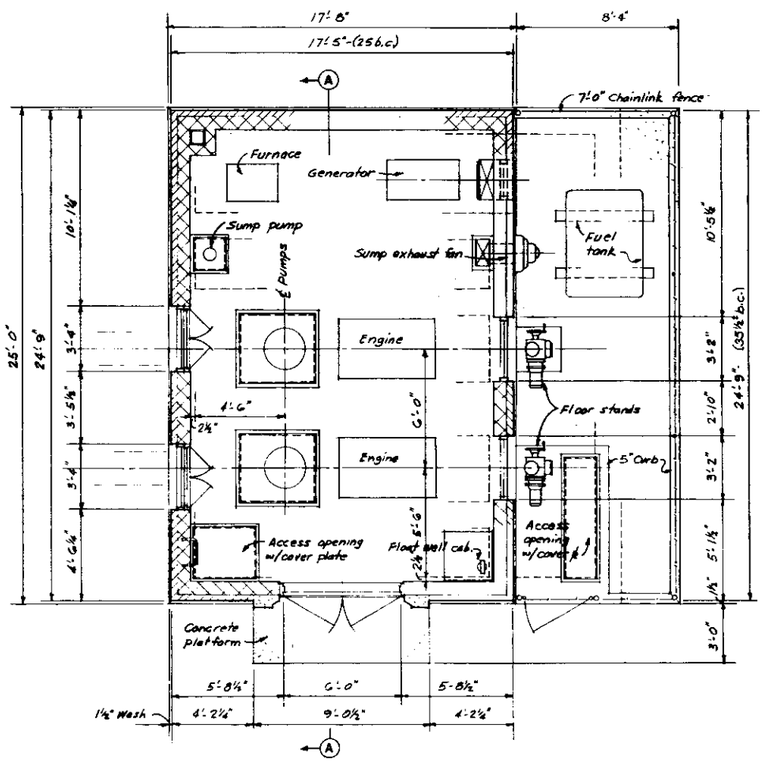
NORTH ELEVATION



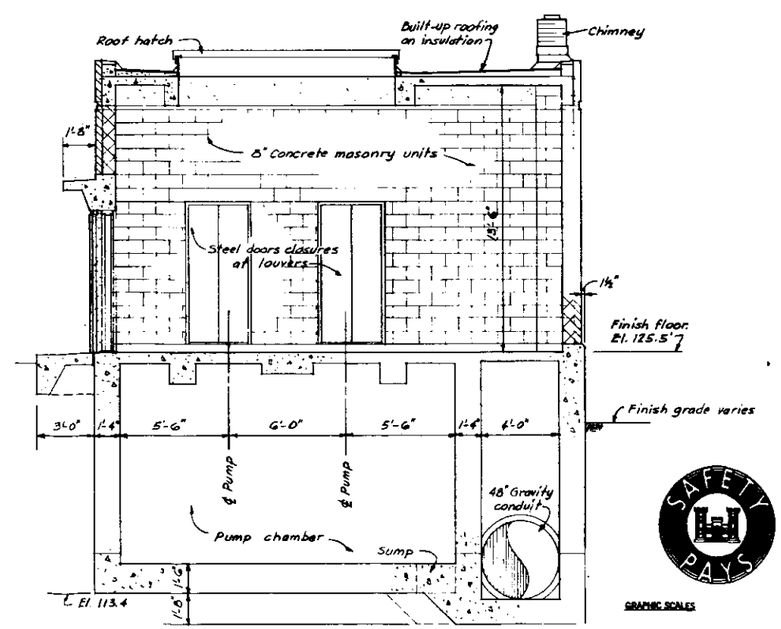
WEST ELEVATION



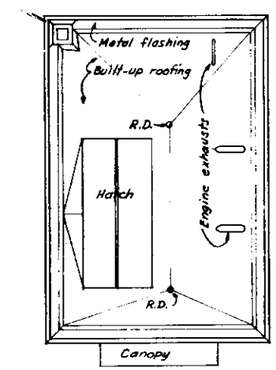
EAST ELEVATION



FLOOR PLAN



SECTION A-A



ROOF PLAN
SCALE 1/4"=1'-0"

GRAPHIC SCALES

1/4"=1'-0"

3/8"=1'-0"

SAFETY PAYS

REVISION	DATE	DESCRIPTION	BY

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

WATER RESOURCES DEVELOPMENT PROJECT
FRAMINGHAM, MASSACHUSETTS

SAXONVILLE LOCAL PROTECTION
PUMPING STATION - ARCHITECTURAL
PLANS, ELEVATIONS AND SECTION

SUDBURY RIVER MASSACHUSETTS

APPROVED DATE

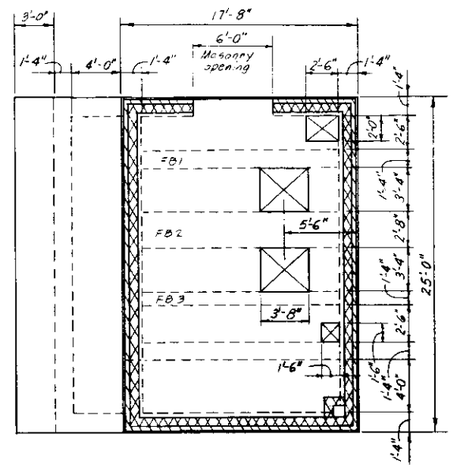
SCALE 3/8"=1'-0" SPEC. NO.

DRAWING NUMBER

SHEET

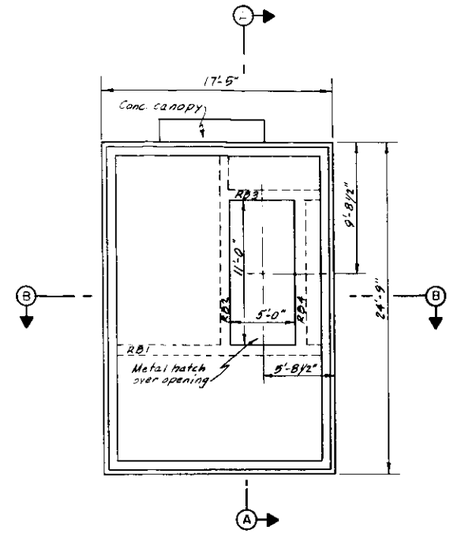
PLATE 5-8

CORPS OF ENGINEERS U. S. ARMY



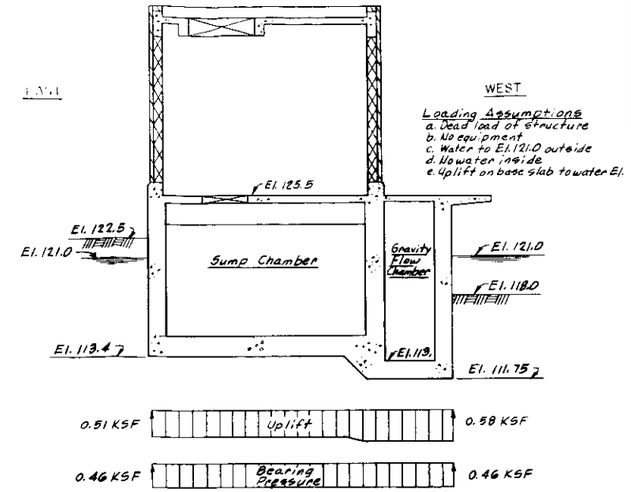
OPERATING FLOOR PLAN

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

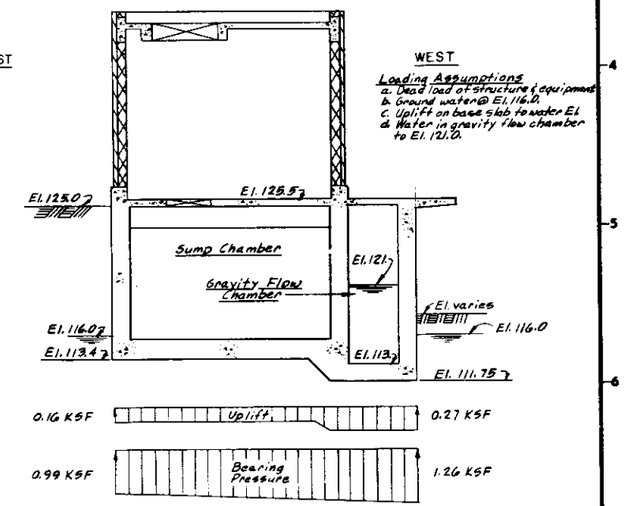


ROOF PLAN

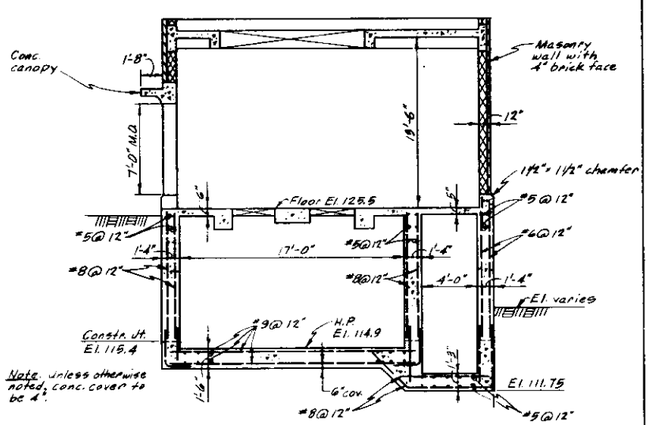
SCALE: 1/4" = 1'-0"



CASE 1 – STABILITY

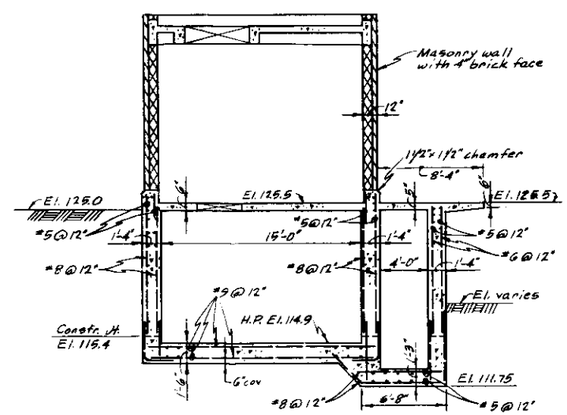


CASE 2-STABILITY



SECTION A-A

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



SECTION B-B

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

GRAPHIC SCALES

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

REVISION	DATE	DESCRIPTION	BY

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

DES. BY | DR. BY | CK. BY

SUBMITTED:
CHIEF, SECTION
APPROVAL RECOMMENDED:
CHIEF, TECH. ENG. BRANCH
REVIEWED:
PROJECT ENGINEER
APPROVAL RECOMMENDED:
CHIEF BRANCH

WATER RESOURCES DEVELOPMENT PROJECT
FRAMINGHAM, MASSACHUSETTS

SAXONVILLE LOCAL PROTECTION
PUMPING STATION
STRUCTURAL DETAILS

SUDBURY RIVER MASSACHUSETTS

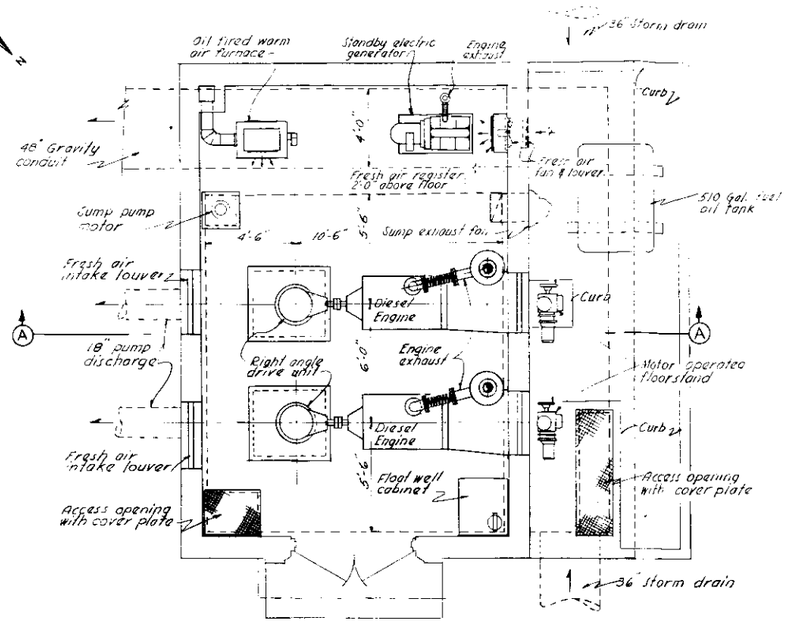
APPROVED: CHIEF, ENGINEERING DIVISION DATE

SCALE: AS SHOWN | SPEC. NO.

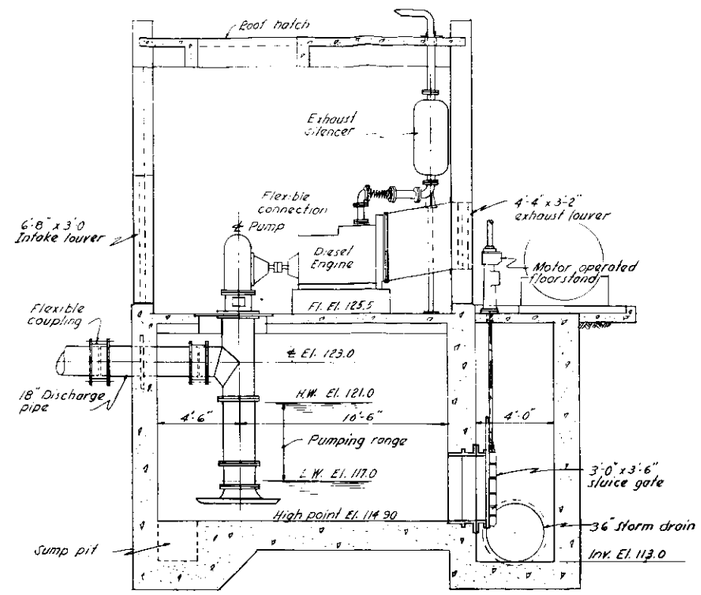
DRAWING NUMBER

SHEET

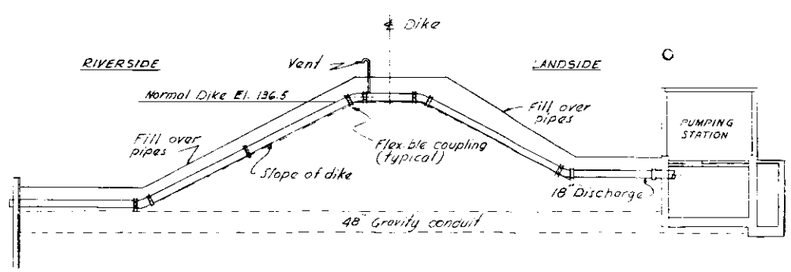
PLATE 5-9



FLOOR PLAN
SCALE 3/8" = 1'-0"



SECTION A-A
SCALE 3/8" = 1'-0"



SECTION AT DIKE
SCALE 1" = 10'

SAFETY PAYS

GRAPHIC SCALES

3/8" = 1'-0" 4' 2' 0' 4'

1" = 1'-0" 10' 0' 10' 20'

REVISION	DATE	DESCRIPTION	BY

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

DES. BY JTS | DR. BY JPV | CK. BY

SUBMITTED:
CHIEF, SECTION
APPROVAL RECOMMENDED:
CHIEF, TECH. ENG. BRANCH
REVIEWED:
PROJECT ENGINEER
APPROVAL RECOMMENDED:
CHIEF BRANCH

WATER RESOURCES DEVELOPMENT PROJECT
FRAMINGHAM, MASSACHUSETTS

SAXONVILLE LOCAL PROTECTION

PUMPING STATION - MECHANICAL

PLAN AND SECTIONS

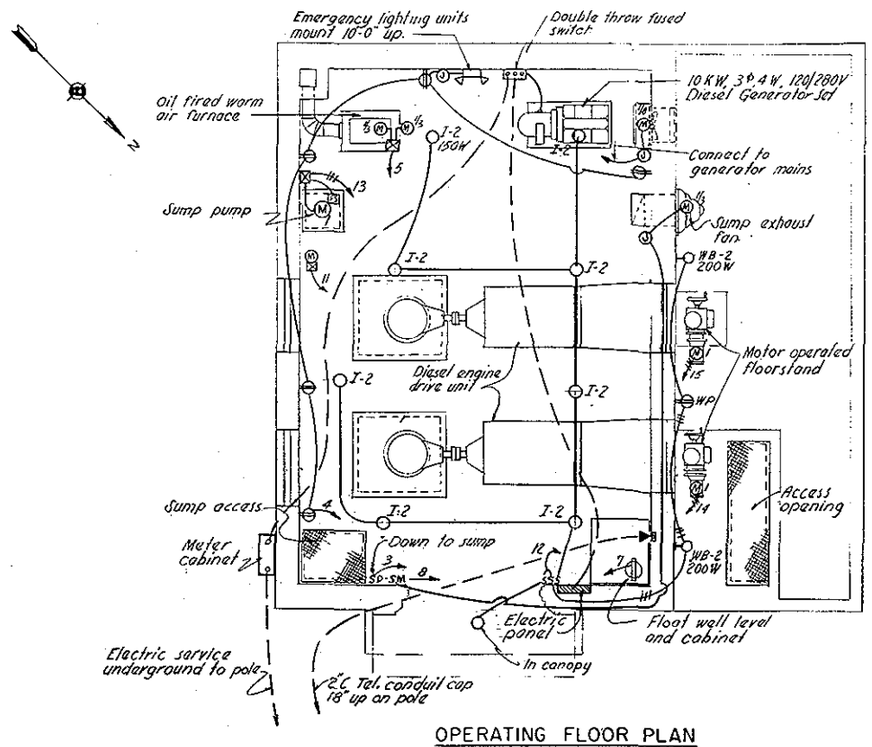
SUDBURY RIVER MASSACHUSETTS

APPROVED DATE
CHIEF, ENGINEERING DIVISION

SCALE | SPEC. NO.
DRAWING NUMBER
SHEET

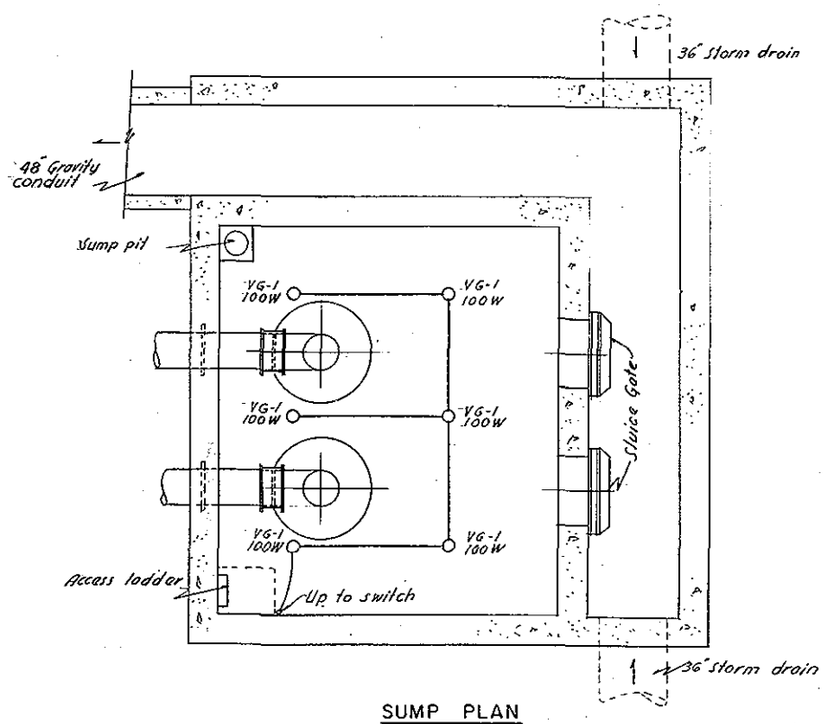
PLATE 5-10

CORPS OF ENGINEERS — U. S. ARMY



OPERATING FLOOR PLAN

SCALE 3/8" = 1'-0"



SUMP PLAN

SCALE 3/8" = 1'-0"

PANEL				
120/208V		3Φ 50AMP MAIN BREAKER		4 WIRE
CIRCUIT NUMBER	BREAKER SIZE AMPS	NUMBER OF POLES	SERVES	CONNECTED LOAD KVA
1	20	1	Lighting	1.2
2	20	1	Spare	—
3	20	1	Furnace	2 @ 1/3
4	20	1	Receptacles	1.0
5	20	1	Sump lighting	.8
6	20	1	Float well receptacle	.2
7	20	1	Spare	—
8	20	1	Sump exhaust fan	1/3
9	20	1	Spare	—
10	20	1	Spare	—
11	20	1	Grease Unit	1/3
12	20	1	Exterior lighting	.6
13	15	3	Sump pump	1
14	15	3	Floor stand No. 1	1
15	15	3	Floor stand No. 2	1
16	20	3	Sluice gate	2

LEGEND	
SYMBOL	DESCRIPTION
○	Ceiling outlet
⊢○	Wall outlet
ⓓ	Junction box
⊸⊖	Duplex convenience outlet - WP indicates weatherproof
Ⓜ 1/3	Motor - HP as indicated
	Emergency lighting unit
	Panel
⊠	Controller
◄	Telephone outlet, 1'-0" up
——	Branch circuit concealed in ceiling or wall
— —	Branch circuit in floor
——►	Home run to panelboard. Note: Any circuit without further designation indicates a two-wire circuit. For a greater number of wires indicate as follows —#— (3 wires) —##— (4 wires) etc.
S	Single pole switch, 4'-0" up
FS	Float switch

SAFETY PAYS

GRAPHIC SCALES

3/8" = 1'-0" 4' 2' 0 4'

REVISION	DATE	DESCRIPTION	BY

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

DES. BY LFF | DR. BY JPV | CK. BY

SUBMITTED:

CHIEF, SECTION

APPROVAL RECOMMENDED:

CHIEF, ELEC. ENG. BRANCH

REVIEWED:

PROJECT ENGINEER

APPROVAL RECOMMENDED:

CHIEF BRANCH

WATER RESOURCES DEVELOPMENT PROJECT
FRAMINGHAM, MASSACHUSETTS

SAXONVILLE LOCAL PROTECTION
PUMPING STATION - ELECTRICAL
PLANS AND PANEL SCHEDULE

SUDBURY RIVER — MASSACHUSETTS

APPROVED — DATE

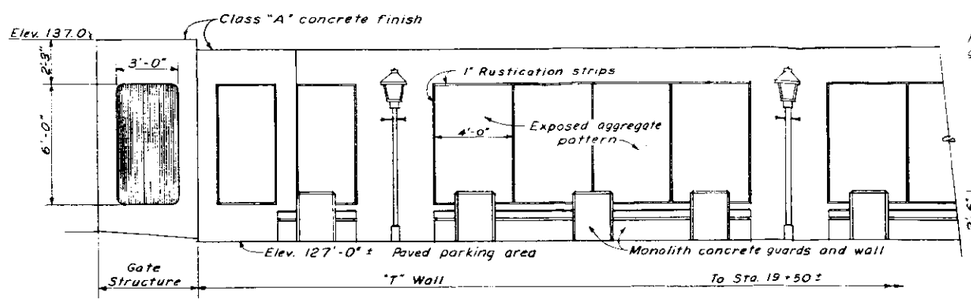
CHIEF, ENGINEERING DIVISION

SCALE | SPEC. NO.

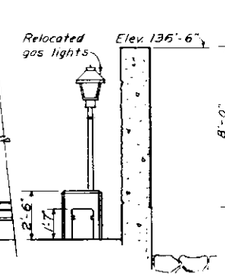
DRAWING NUMBER

SHEET

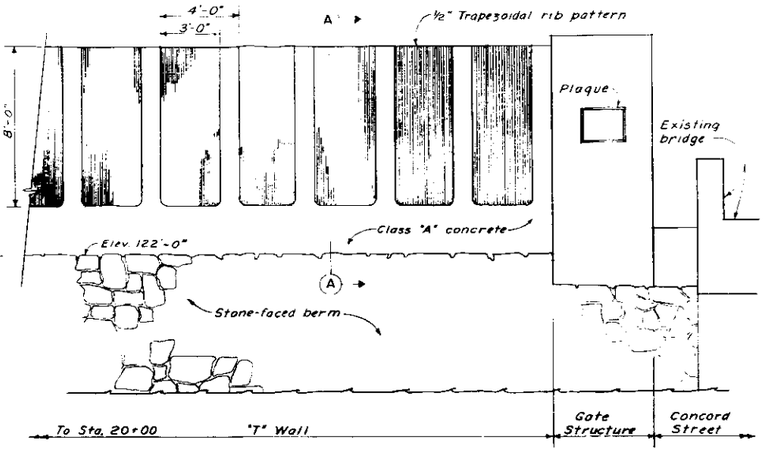
PLATE 5-11



LANDSIDE ELEVATION WEST OF CONCORD STREET

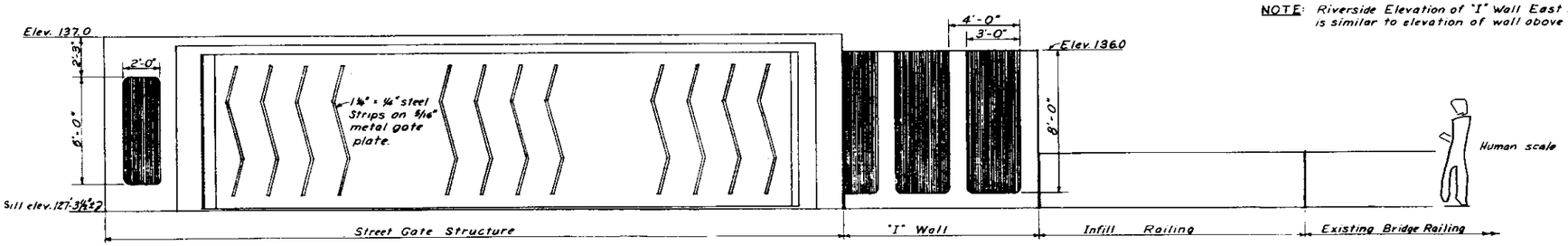


SECTION A-A

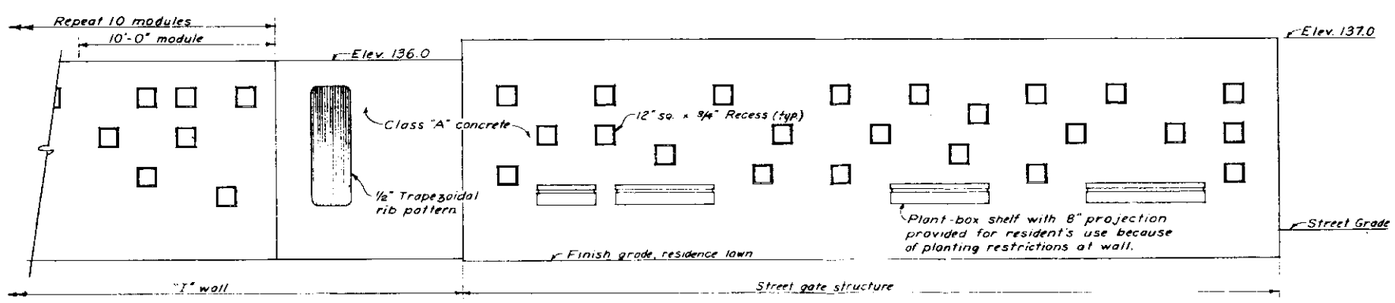


RIVERSIDE ELEVATION WEST OF CONCORD STREET

NOTE: Riverside Elevation of "I" Wall East of Concord Street is similar to elevation of wall above



CONCORD STREET GATE LOOKING EAST



LANDSIDE ELEVATION EAST OF CONCORD STREET

GRAPHIC SCALE

3/8" 1'-0" 4' 2' 0 2' 4'

SAFETY PAYS

REVISION	DATE	DESCRIPTION	BY

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

WATER RESOURCES DEVELOPMENT PROJECT
FRAMINGHAM, MASSACHUSETTS

SAXONVILLE LOCAL PROTECTION
ARCHITECTURAL TREATMENT OF WALLS AND STREET GATE

SUDBURY RIVER MASSACHUSETTS

DESIGNED BY
CHIEF, DESIGN BRANCH
APPROVAL RECOMMENDED
CHIEF, TECH. ENG. BRANCH
CHIEF, ENGINEERING DIVISION
PROJECT ENGINEER
APPROVAL RECOMMENDED
CHIEF, BRANCH

APPROVED
DATE
CHIEF, ENGINEERING DIVISION

SCALE 3/8"=1'-0" SPEC. NO.
DRAWING NUMBER
SHEET

APPENDIX A

TYPICAL DESIGN COMPUTATIONS

INDEX TO SHEETS

TITLE	PAGE NO.
L- TYPE FLOOD WALL, STATION 1+00	A1 - A5
TEE TYPE FLOOD WALL, STATION 2+80	A6 - A11
TEE TYPE FLOOD WALL, STATION 5+50	A12 - A17
GRAVITY WALL AT SAXONVILLE POND	A18 - A19
GRAVITY WALL AT DANFORTH STREET	A20 - A21
I - TYPE FLOOD WALL - TYPICAL	A22 - A25
CONCORD STREET GATE	A26 - A60
PUMPING STATION	A61 - A81

27 Sept 49

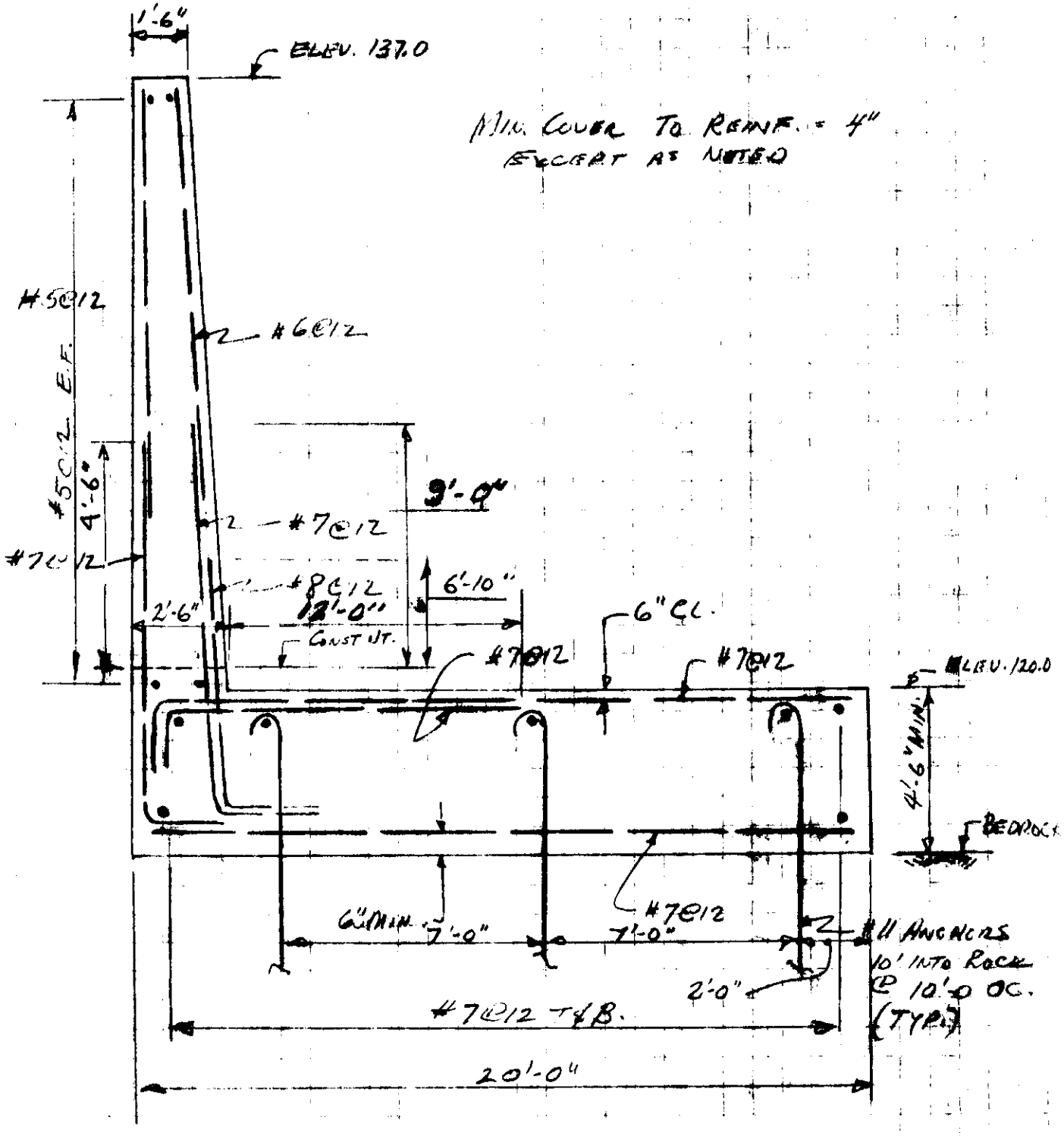
CORPS OF ENGINEERS, U. S. ARMY

PAGE A1

SUBJECT Saxonville Local Protection

COMPUTATION L Wall - Reinf. Details

COMPUTED BY RLS CHECKED BY JRF DATE Aug 1974



"L" WALL REINF.

N.T.S.

27 Sept 49

CORPS OF ENGINEERS, U. S. ARMY

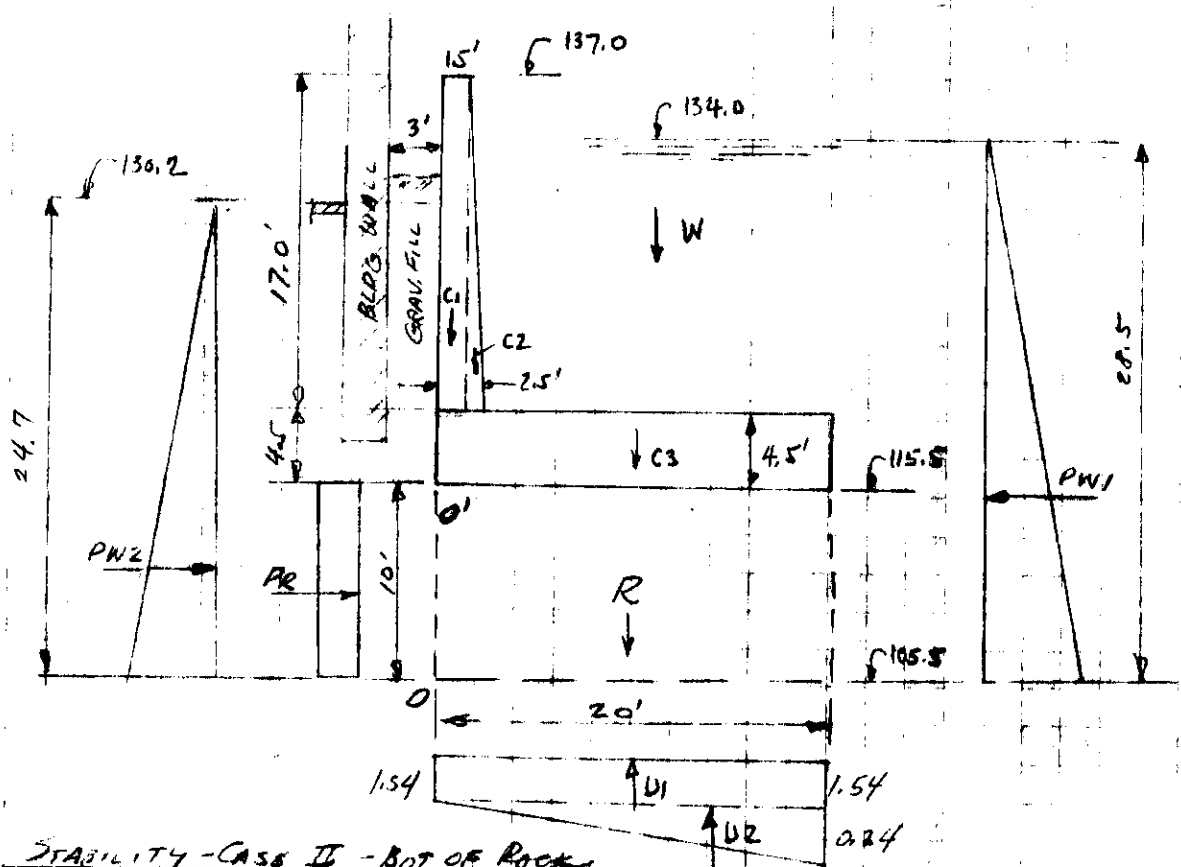
PAGE A2

SUBJECT: SAXONVILLE LOCAL PROTECTION

COMPUTATION: L-WALL - STABILITY ANALYSIS (UTILIZING ROCK ANCHORS)

COMPUTED BY: R. HARRINGTON CHECKED BY: JRF DATE: AUG 1974

CASE II
- ENGAGE 10' OF ROCK.
- ASSUME BASE WIDTH: 20'
- ASSUME TAIL WATER @ EL. FLOOR (130.2)



STABILITY - CASE II - BOT OF ROCK

SEC	FACTOR	↓	↑	→	←	ARM	↻	↺
C1	1.5 × 17 × 0.15	3.83				0.75	2.9	
C2	1.0 × 17 × ½ × 0.15	1.28				1.83	2.3	
C3	4.5 × 20 × 0.15	13.50				10.00	135.0	
W	18 × 14 × .0625	15.75				11.00	173.3	
PW1	.0625 × 28.5² × ½				25.4	9.50		241.3
PW2	.0625 × 24.7 × ½			19.0		8.23	156.4	
U1	1.54 × 20		30.8			10.00		308.0
U2	0.24 × 20 × ½		2.4			13.33		32.0
R	10 × 20 × 0.160	32.0				10.00	320.0	
PR	TO BALANCE EH			6.4		5.08	32.0	
		66.36	33.2	25.4	25.4		821.9	581.3
	Σ	33.16					240.6	

$\Sigma M/\Sigma V = \frac{240.6}{33.16} = 7.25'$ WITHIN MID 1/3 ok. $e = 10 - 7.25 = 2.75'$

$B.P. = \frac{33.16}{20} \pm \frac{91.2 \times 6}{20^2} = 3.03, 0.29$ $M = 2.75 \times 33.16 = 91.2$

SUBJECT JAXONVILLE LOCAL PROTECTION

COMPUTATION L WALL STABILITY ANALYSIS (UTILIZING ROCK ANCHORS)

COMPUTED BY R. HARRINGTON CHECKED BY JRF DATE AUG. 1974

CASE I - WATER RIVERSIDE TO EL. 137.0
TAIL WATER AT ELEV. 130.2
@ BOT. OF ROCK.

SEC	FACTOR	↓	↑	→	←	ARM @	↻	↺
C1	SEE CASE II	3.83					2.9	
C2	do	1.28					2.3	
C3	do	13.50					135.0	
W	18 x 17 x .0625	19.12				11.00	210.3	
PW1	.0625 x 31.5² x ½				31.0	10.5		325.5
PW2				19.0			156.4	
U1			30.8					308.0
U2	.43 x 20 x ½		4.3			13.33		57.3
R		32.00					320.0	
PR				12.0		5.00	60.0	
		69.73	35.1	31.0	31.0		886.9	690.8
	Σ	34.63					196.1	

ΣM/ΣV = 196.1 / 34.63 = 5.66 INSIDE M.O. ½ OK

e = 10 - 5.66 = 4.34' M = 34.63 x 4.34 = 150.3'K

B.P. = 2 x 34.63 / 3 x 5.66 = 4.08 K/SF

CASE I AT BOT. OF CONCRETE

						ARM @ O'		
C1	SEE CASE I - BOT. ROCK	3.83					2.9	
C2		1.28					2.3	
C3		13.50					135.0	
W		19.12					210.3	
Pw1	.0625 x 21.5² x ½				14.44	7.17		103.5
Pw2	.0625 x 14.7² x ½			6.75		4.90	33.1	
U1	0.92 x 20		18.4			10.00		184.0
U2	0.42 x 20 x ½		4.2			13.33		56.0
		37.73	22.6	6.75	14.44		383.6	343.5
	Σ	15.13			7.69		40.1	

ΣH/ΣV = 7.69 / 15.13 = 0.51 F.S. = 0.70 / 0.51 = 1.37 SAY OK

ΣM/ΣV = 40.1 / 15.13 = 2.65' e = 10 - 2.65 = 7.35'
M = 15.13 x 7.35 = 111.2'K

$$P = \frac{15.13}{20} \pm \frac{111.2 \times 6}{20^2} = 2.43, -0.91$$

PROVIDE ANCHORS TO ROCK

27 Sept 49

CORPS OF ENGINEERS, U. S. ARMY

SUBJECT: SAKONNET LOCAL PROTECTION

COMPUTATION: L-WALL - STABILITY ANALYSIS

COMPUTED BY: _____ CHECKED BY: JRF DATE: AUG. 1974

CASE II @ BOT OF CONC.

SEC	FACTOR	↓	↑	→	←	HEADED	↻	↺
C1		3.63					2.9	
C2		1.28					2.3	
C3		13.50					135.0	
W		15.75					173.3	
Pw1	.0625×18.5²×½				10.70	6.17		66.0
Pw2	.0625 14.7² × ½			6.75		4.90	33.1	
U1	0.92 × 20		18.4			10.00		184.0
U2	0.24 × 20 × ½		2.4			13.33		32.0
		34.36	20.8	6.75	10.70		346.6	282.0
	Σ	13.56			3.95		64.6	

$e = M/\Sigma V$ $3.95/13.56 = 0.29$ $e = 10 - 4.76 = 5.24$

$\Sigma M/\Sigma V = \frac{64.6}{13.56} = 4.76'$ $M = 5.24 \times 13.56 = 71.$

$20/3 = 6.67$ OUTSIDE MID 1/3

$BP = 13.56/20 \pm \frac{71 \times 6}{20^2} = 0.68 \pm 1.07 = 1.75, -0.39$

IF 2.5 K/' FORCE ↓ WAVE ADDED 18' FROM O'

$\Sigma V = 13.56 + 2.5 = 16.06^k$

$\Sigma M = 64.6 + 2.5 \times 18 = 109.6^{'k}$

$\Sigma M/\Sigma V = 109.6/16.06 = 6.82$ INSIDE MID 1/3 OK.

CASE I AT BOT OF CONC. (SEE SHT. A5)

ASSUME 2.5 K/' FORCE ↓ ADDED 18' FROM O'

$\Sigma V = 15.13 + 2.5 = 17.65$

$\Sigma M = 40.1 + 2.5 \times 18 = 85.0^k$

$\Sigma M/\Sigma V = 85.0/17.65 = 4.82' = 5.0$ AT MID 1/2 OK

ASSUME #11 REINF. BAR ANCHORS. - $A_s = 1.56 \text{ in}^2$

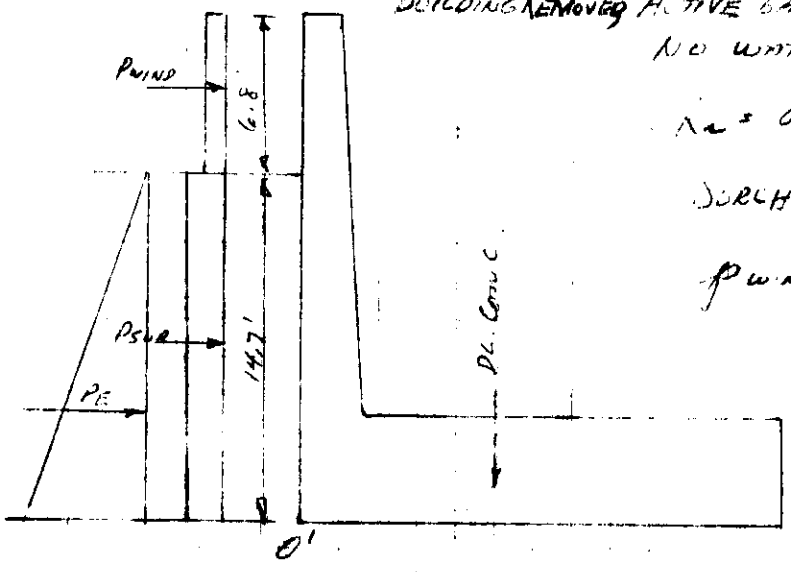
IF $f_{s\,ALLOW} = 20$ KSI BAR SPACING $= \frac{20 \times 1.56}{2.5} = 12.48''$ SAY 10" MAX SPAC.

SUBJECT Cofferdam Lock Protection

COMPUTATION Wall Stability Analysis

COMPUTED BY RLH CHECKED BY JRF DATE AUG. 1974

CHECK LOADING II @ BASE OF CONC.



BUILDING REMOVED ACTIVE EARTH, PARKING SURCHARGE (2'), WIND
NO WATER

$K_a = 0.33$ $\gamma_{EAT} = 125\#/ft^3$

SURCH = 2' OF EARTH.

$p_{WIND} = 50\#/SF$

SPC.	FACTOR	↓	→	ARM 0	↻
C	SEE A4	18.61			140.2
P_{WIND}	.050 × 6.8		.34	18.10	6.2
P_{SUR}	0.33 × 0.125 × 2 × 14.7		1.21	7.35	8.9
P_E	0.33 × 0.125 × 14.7² × 0.5		4.46	4.90	21.9
	Σ	18.61	6.01		177.2

$\Sigma H/\Sigma V = 6.01/18.61 = 0.32$

$\Sigma M/\Sigma V = 177.2/18.61 = 9.52$ $e = 10.0 - 9.52 = 0.48$ LANDSIDE

$M = 0.48 \times 18.61 = 8.9$

$BP = 18.61/20 \pm \frac{8.9 \times 6}{20^2} = 0.93 \pm 0.13$

B.P. = 1.06 LANDSIDE, 0.80 RIVERSIDE

ROCK ANCHORS

ALL ANCHORS – 3 @ 10'-0" O.C. ALONG WALL BASE
TO ENGAGE ROCK ASSUMED IN STABILITY ANALYSIS
(REAR ANCHOR IN TENSION SEE PREV. SHT.).

STRESS IN ANCHORS

MAX $\Sigma H = 7.69^{K}/'$ CASE I

$f_v = \frac{7.69 \times 10}{3 \times 1.56} = 16.43$ KSI OK (THIS NEGLECTS STATIC REACTION BOTH CONC. & ROCK)

27 Sept 49

CORPS OF ENGINEERS, U. S. ARMY

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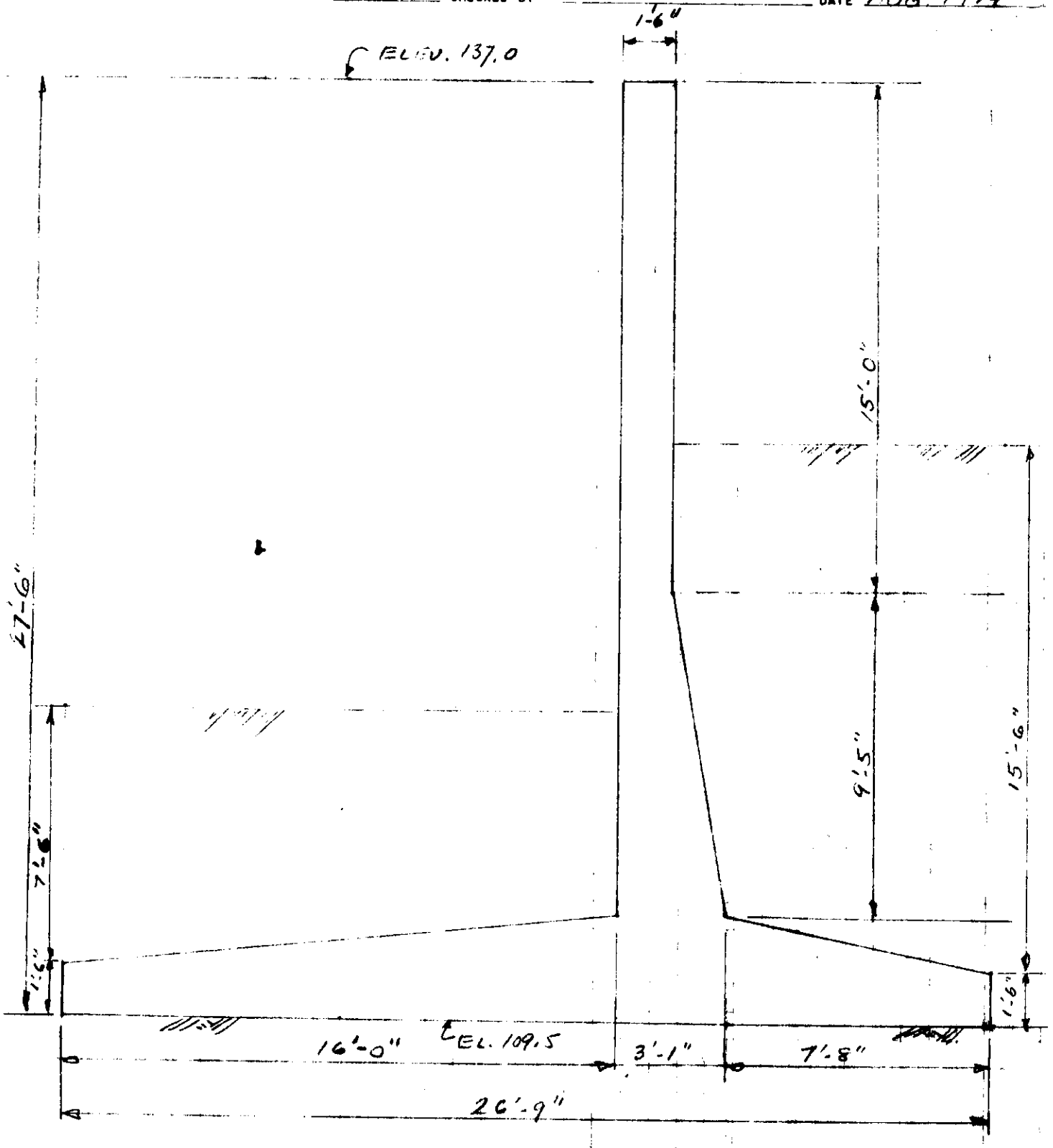
SUBJECT DANVILLE LOCAL PROTECTION

COMPUTATION TEE TYPE FLOODWALL - STA. 2+80 (STA 2+0 to 3+20)

COMPUTED BY RLH CHECKED BY JHC DATE AUG. 1974



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

```
LOADING CONDITIONS:- #1--WATER TO TOP OF WALL
                     #2--WATER TO 3 FT BELOW TOP OF WALL
                     #3--NO WATER,50PSF WIND ON LANDSIDE
                         3 FT SURCHARGE (LS)

ALL UNITS  POUND--FEET

INSTRUCTIONS:-------
              1. SET MAGNITUDE 8,  DECIMAL 2
              2. CONTROL MODE = AUTO,  PROGRAM MODE = AUTO
              3. SWITCH,ANSWERS ONLY ON,AUTOMATIC TAB
              4. READER&PUNCH@AUTO
              5. TO CLEAR: RESET MEMORY,((=

ENTER FOLLOWING ITEMS:----
PROJECT NAME                          =SAXONVILLE
STATION                               =2+80
DATE                                  =AUGUST 1974
HEIGHT OF WALL (FROM BOT. OF BASE)    =27.5
WEIGHT OF CONCRETE                    =150
   "   "  WATER                       =62.5
   "   "  EARTH (SUB)                 =62.5
   "   "  EARTH (DRY)                 =125
COEFF. FRICTION                       =.65
LATERAL ACTIVE EARTH PRESSURE  KA     =.33
HEIGHT OF EARTH RIVERSIDE             =9
   "   "    "   LANDSIDE             =17


          ..........&02

DIMENSIONS(FEET):----

WIDTH,BASE                =      26.73
  "  ,STEM@TOP            =       1.5
  "  ,STEM @ TOP OF BASE=         3.07
  "  ,BASE @ HEEL & TOE =         1.5
  "  ,BASE @ STEM         =       3.07
HEIGHT,STEM                =      24.42
  "   ,STEM TAPER          =       9.42
  "   ,STEM STRAIGHT       =      15
HEEL TO RIVERSIDE STEM     =      16.03
TOE TO LANDSIDE STEM       =       7.62

CONCRETE ONLY:------
                          LOAD#   MOMENT#FT
C1   =                  5496.42    92275.94
C2   =                  6014.37    80383.92
C3   =                  1890.23    20210.81
C4   =                   898.16    19445.33
C5   =                  1111.22    20071.11
C6   =                   723.97    12723.26
CONC=                  16134.41   245110.38

CUBIC YARDS/LF OF WALL=                          3.98
```

SUBJECT SAVANNAH LOCAL PROTECTION

COMPUTATION TEE TYPE FLOOD WALL - STA. 2+40

COMPUTED BY R.L.H. CHECKED BY JAC DATE AUG. 1974

CONDITION #1:------

CREEP RATIO= 4.16

INSTRUCTIONS:-------SET DECIMAL 1

	+VERT#	-VERT#	+HORIZ#	-HORIZ#	+MOMENT#FT	-MOMENT#FT
CONC=	16134.4				245110.3	
WR1 =	24487.1				196366.4	
WER2=	1575.1				8421.1	
ER3 =	5942.7				47656.1	
WEL1=	13268.3				304113.1	
WEL2=	748.4				18105.7	
WEL3=	926.0				17210.9	
WEL4=	883.9				16197.1	
U1 =		35220.5				470732.9
U2 =		5361.3				47770.2
PWR =			23632.8		216634.1	
PWL =				11199.7		63465.0
PEL =				2980.3		16888.4
PP =					&41	.0
SUMS=	53966.2	40581.8	23632.8	14180.0	1069815.3	598856.7
SUMV=	23384.4#					
SUMH=			9452.7#			
SUMM=					470958.6#FT	
H/V =	.4					
M/V =	20.1FT					
SBPL=	2205.0PSF					
SBPR=	-455.4PSF					

IF SBPR IS NEGATIVE, THEN SBPR=0 & SBPL= 2365.3PSF

BRGL= 19.7FT

SUBJECT DANVILLE LOCAL PROTECTION

COMPUTATION TEE TYPE FLOOD WALL - STA 2+80

COMPUTED BY RLH CHECKED BY JAC DATE AUG 1974

CONDITION #2 W/ ACTIVE PRES. :--------

CREEP RATIO= 5.8

INSTRUCTIONS:-----SET MAG 8, DEC 1

	+VERT#	-VERT#	+HORIZ#	-HORIZ#	+MOMENT#FT	-MOMENT#FT
CONC=	16134.4	..	..	..	245110.3	
WR1 =	21479.9	..	..	..	172251.2	
WER2=	1575.1	..	..	..	8421.1	
ER3 =	5942.7	..	..	..	47656.1	
WEL1=	13268.3	..	..	..	304113.1	
WEL2=	748.4	..	..	..	18105.7	
WEL3=	926.0	..	..	..	17210.9	
WEL4=	883.9	..	..	..	16197.1	
U1 =	..	33272.1	..	..	..	444692.3
U2 =	..	3829.5	..	..	..	34121.6
PWR =	..	..	18757.8	..	153188.8	
PWL =	..	..	..	10580.1	..	59954.1
PEL =	..	..	..	2980.3	..	16888.4
PP =	..	..	..		..&41	.0
SUMS=	60959.0	37101.6	18757.8	13560.4	982254.8	555656.6
SUMV=	23857.4#					
SUMH=	..	..	5197.3#			
SUMM=	..	..	..	..	426598.2#FT	

H/V = .2

M/V = 17.8FT

SBPL= 1797.2PSF

SBPR= -12.1PSF

IF SBPR IS NEGATIVE, THEN SBPR=0 & SBPL= 1797.2PSF

BRGL= 26.5FT

CONDITION #3 :-------

NO CREEP

	+VERT#	-VERT#	+HORIZ#	-HORIZ#	+MOMENT#FT	-MOMENT#FT
CONC=	16134.4	..	..	..	245110.3	
WER2=	1575.1	..	..	..	8421.1	
ER3 =	11885.5	..	..	..	95312.3	
WEL1=	13268.3	..	..	..	304113.1	
WEL2=	748.4	..	..	..	18105.7	
WEL3=	926.0	..	..	..	17210.9	
WEL4=	883.9	..	..	..	16197.1	
PEL =	..	..	..	5960.6	..	33776.8
PER =	..	..	1670.6	..	5011.8	
PWND=	..	..	..	525.0	..	11681.2
PSUR=	..	..	..	2103.7	..	17881.8
SUR =	3447.0	..	..	..	76299.4	
PP =	..	..	..		..&41	.0
SUMS=	48869.0		1670.6	8589.3	785782.2	63340.0
SUMV=	48869.0#					
SUMH=	..	..	-6918.7#			
SUMM=	..	..	..	..	722442.2#FT	

H/V = -.1

M/V = 14.7FT

SBPL= 2410.0PSF

SBPR= 1246.3PSF

IF SBPR IS NEGATIVE, THEN SBPR=0 & SBPL= 2726.9PSF

BRGL= 35.8FT

27 Sept 49

CORPS OF ENGINEERS, U.S. ARMY

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SUBJECT SAXONVILLE LOCAL PROTECTION

COMPUTATION TEE TYPE FLOODWALL - STA 2+80

COMPUTED BY RLH CHECKED BY JAC DATE AUG 1974

STEM DESIGN: CASE I

X MEASURED FROM TOP OF STEM @ 1/5 PT'S, REINF @R.S.FACE, ~~3.5~~ 4" MIN COV.

X FT	SHEAR#	MOMENT#FT	D REQD"	D GIV"	SHEAR,PSI	REINFX10
-24.4	12948.8	130998.1	29.3	33.3	32.3	27.2
-19.5	10185.2	74261.1	22.1	23.5	35.9	21.8
-14.6	6655.0	32774.4	14.6	14.5	38.2	15.6
-9.7	2983.1	9715.6	7.9	14.5	17.1	4.6
-4.8	745.5	1213.9	2.8	14.5	4.2	.5

TOE DESIGN:

X MEASURED FROM L.S. TOE @ 1/3 PT'S, REINF @ BOT FACE, ~~4.5~~ 6" MIN COV.

-7.6	-14774.0	-36050.4	15.4	32.3	38.0	7.7
-5.0	-9139.6	-17217.6	10.6	26.0	29.2	4.5
-2.5	-4213.9	-4600.5	5.5	19.7	17.7	1.6

HEEL DESIGN:

X MEASURED FROM R.S. HEEL @ 1/3 PT'S, REINF @ TOP FACE, ~~3.5~~ 4" MIN COV.

-16.0	6936.7	-74237.6	22.0	33.3	17.3	15.4
-10.6	6578.5	-36801.4	15.5	27.0	20.2	9.4
-5.3	3454.9	-9007.7	7.6	20.7	13.8	3.0

STEM DESIGN: CASE III

X MEASURED FROM TOP OF WALL @ 1/3 PT'S,REINF@L.S.FACE,~~3.5~~ 4"MIN COV.

X FT	SHEAR#	MOMENT#FT	D REQD"	D GIV"	SHEAR,PSI	REINFX10
-24.4	-5524.3	-39208.9	16.0	33.3	13.8	8.1
-16.2	-1828.7	-9265.1	7.8	17.0	8.9	3.7
-8.1	-406.8	-1655.6	3.3	14.5	2.3	.7

TOE DESIGN:

X MEASURED FROM L.S. TOE @ 1/3 PT'S, REINF@TOP FACE IF +M, ~~3.5~~ 4"MIN COV.

-7.6	2384.7	7290.5	6.9	33.3	5.9	1.5
-5.0	1275.3	2707.3	4.2	27.0	3.9	.6
-2.5	480.4	543.4	1.8	20.7	1.9	.1

HEEL DESIGN:

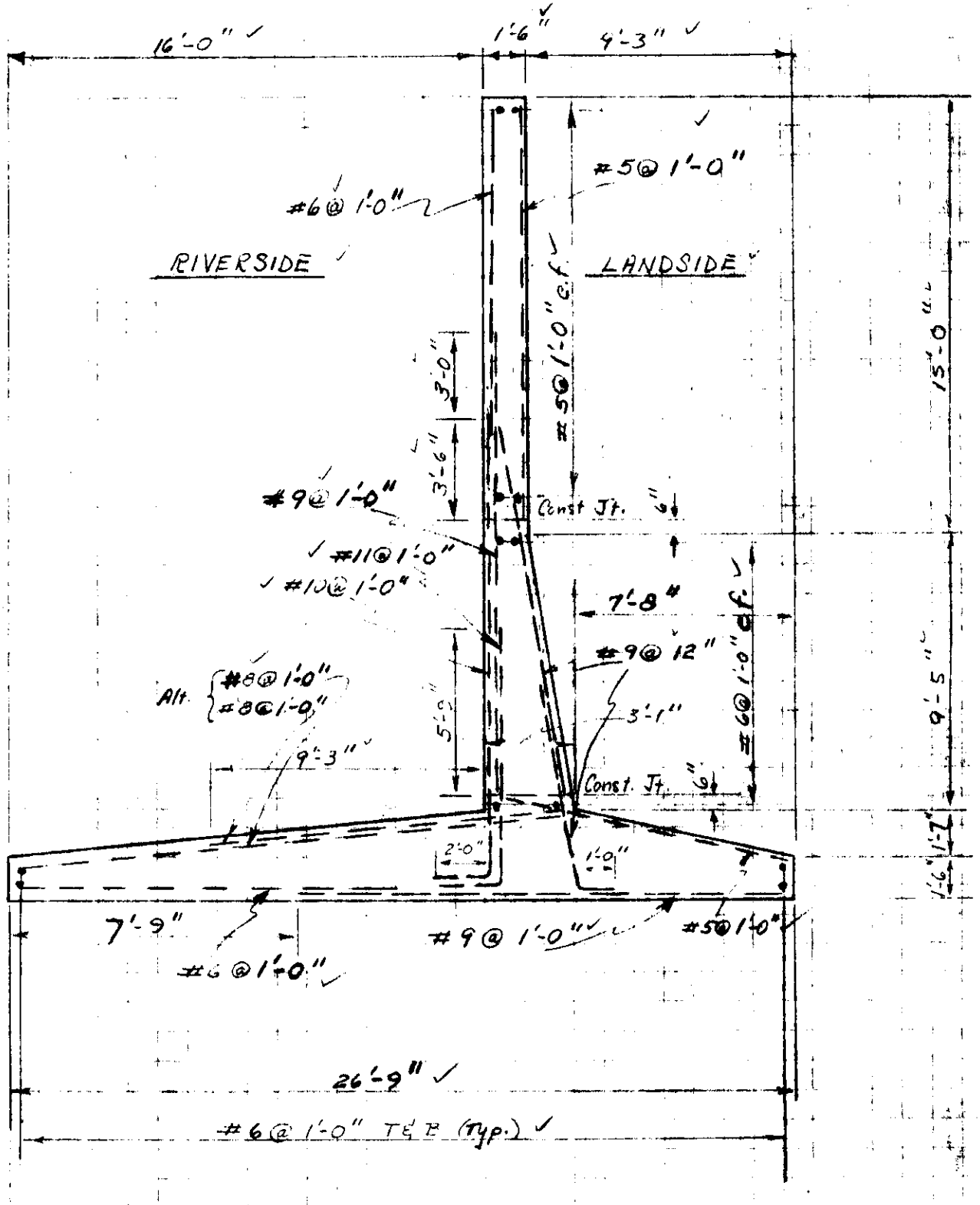
X MEASURED FROM R.S. HEEL @ 1/3 PT'S, REINF@BOT FACE IF +M, ~~4.5~~ 6"MIN COV

-16.0	-6628.1	39025.9	16.0	32.3	17.0	8.3
-10.6	-3243.8	13155.8	9.3	26.0	10.3	3.5
-5.3	-1034.3	2241.0	3.8	19.7	4.3	.7

SUBJECT SAXONVILLE LOCAL PROTECTION

COMPUTATION TEE WALL @ STA 2+80

COMPUTED BY JAC CHECKED BY RLH. DATE AUG 74



27 Sept 49

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

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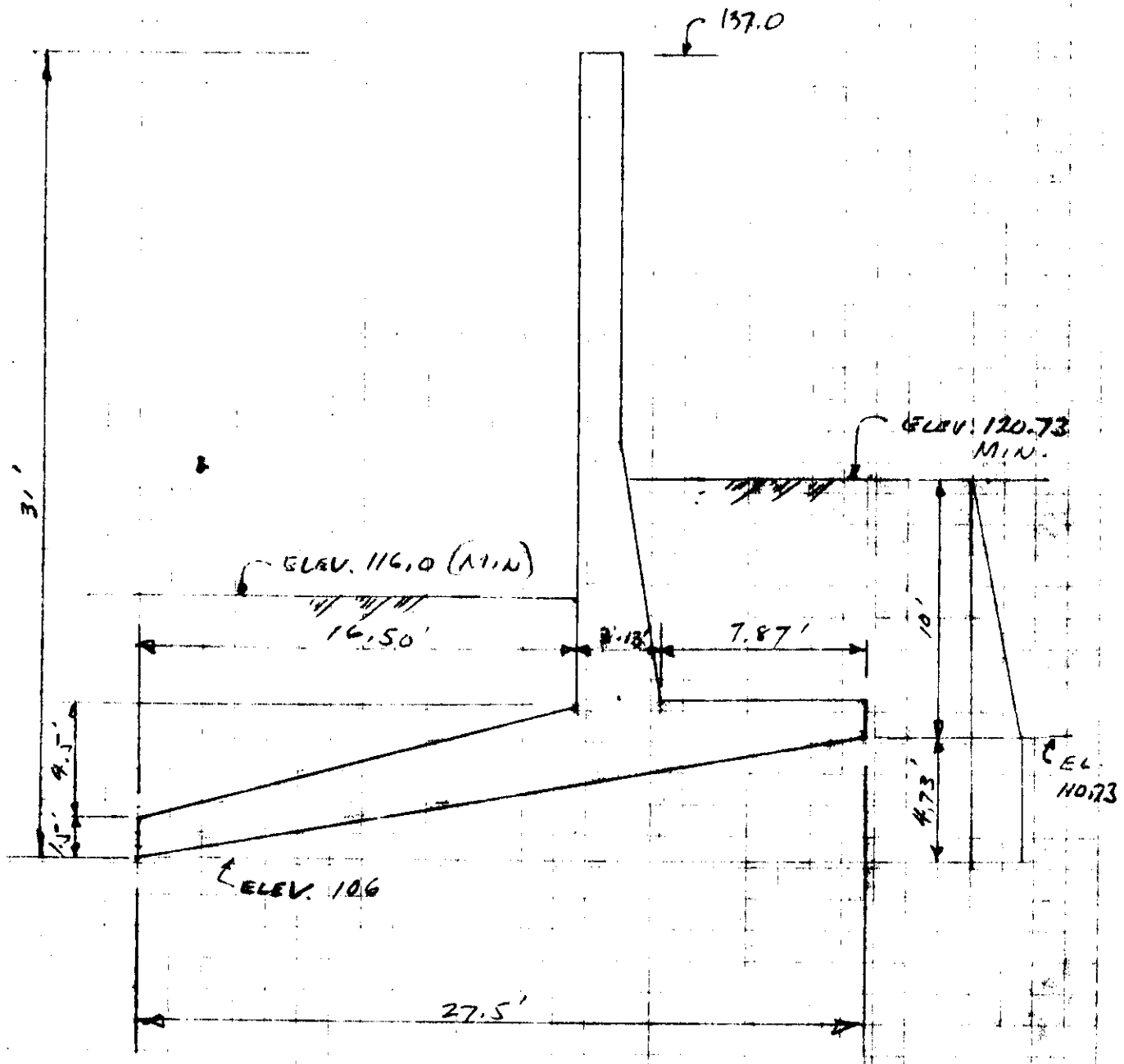
SUBJECT SAXONVILLE LOCAL PROTECTION
COMPUTATION TEE TYPE FLOOD WALL - STA 5+50 (STA 3+20 to 6+50
COMPUTED BY RLH. CHECKED BY JAC DATE AUG 1974



SCALE: 1" = 6'

SUBJECT SAXONVILLE LOCAL PROTECTION

COMPUTATION TEE TYPE FLOOD WALL - STA. 5+50

COMPUTED BY RLH CHECKED BY JAC DATE AUG. 1974

```
LOADING CONDITIONS:- #1 WATER TO TOP OF WALL
                     #2 WATER 3 FT BELOW TOP OF WALL
                     #3 WATER DOWN 50PSF WIND LOAD ON LANDSIDE
ALL UNITS IN POUNDS AND FEET
INSTRUCTIONS:-----
                     #1 SET MAGNITUDE 7, DECIMAL 2
                     #2 CONTROL MODE = AUTO, PROGRAM MODE = AUTO
                     #3 SWITCH, ANSWERS ONLY ON, AUTOMATIC TAB
                     #4 READER & PUNCH @ AUTO
                     #5 TO CLEAR: RESET MEMORY, ((=
ENTER FOLLOWING ITEMS:
PROJECT ——— SAXONVILLE LOCAL PROTECTION
STATION ——— 5+50
HEIGHT OF WALL (FROM BOT. OF BASE)=31
WEIGHT OF CONCRETE =150
   "   "  WATER =62.5
   "   "   EARTH (SUB) =62.5
LATERAL ACTIVE EARTH PRESSURE  KA=.33
HEIGHT OF RIVER SIDE HORIZ. PROJECTION OF BASE=4.5
  "    "  EARTH RIVERSIDE =8.5
  "    "    "   RIVERSIDE STEM=4.0
  "    "    "   LANDSIDE =8.5          ..........&02
         ..........&02
         ..........&02
         ..........&02
         ..........&02
         ..........&02
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DIMENSIONS (FEET):------
WIDTH BASE                        =        27.5
HORIZ. PROJ OF BASE               =        4.5
WIDTH OF STEM @ TOP               =         1.5
  "   "   "   @  "  OF BASE       =         3.12
  "   "  BASE @ HEEL & TOE        =         1.5
HEIGHT OF STEM RIVERSIDE          =        25.00
  "    "   "   LANDSIDE           =        24.77
  "    "   "   TAPER              =         9.77
  "    "   "   STRAIGHT           =        15.00
  "    "  BASE SLOPE              =         4.72
THICKNESS OF BASE AT R. STEM      =         3.16
    "     "   "   "  L. STEM =               2.85
HEEL TO RIVERSIDE STEM            =        16.50
TOE  "  LANDSIDE   "              =         7.87
```

SUBJECT SAXONVILLE LOCAL PROTECTION

COMPUTATION TEE TYPE FLOOD WALL - STA 5+50

COMPUTED BY RLH CHECKED BY JAC DATE AUG 1974

CONCRETE ONLY:----

	LOAD#	MOMENT#FT
C1 =	5575.00	57143.75
C2 =	1195.06	10703.91
C3 =	2475.00	13612.50
C4 =	1558.33	11427.77
C5 =	3712.50	71465.62
C6 =	2062.50	34031.25
CONC=	16578.39	198384.82

CUBIC YARDS/LF OF WALL = 4.09

CONDITION #1:----

CREEP RATIO = 2.32

	+VERT#	-VERT#	+HORIZ#	-HORIZ#	+MOMENT#FT	-MOMENT#FT
CONC=	16578.39				198384.82	
WEL1=	4181.13				16453.53	
WEL2=	432.87				3641.98	
WER1=	2062.50				34031.25	
WER2=	4382.81				96421.87	
WR1 =	25781.25				496289.06	
WR2 =	2320.31				51046.87	
WL1 =	4181.13				16453.53	
WL2 =	432.87				3641.98	
UI =		24568.92				337822.67
U2 =		14356.16				263196.34
PW1 =			30031.25			168508.68
PW2 =				4467.07	14890.25	
PW3 =				4218.90		9961.30
PW4 =				2465.19		7760.81
PR1 =				9709.74	32365.83	
PR2 =				9170.31		21652.14
SUMS=	60353.27	38925.08	30031.25	30031.25	963621.02	808901.95
SUMV=	21428.19					
SUMH=			-.00			
SUMM=					154719.06	

M/V = 7.22

SBL= 1889.30PSF

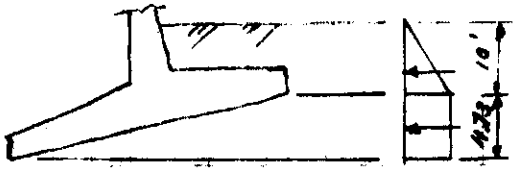
SBR= -330.89PSF

IF SBR IS NEGATIVE THEN SBR =0 & SBL = 1978.49PSF

BEARING LENGTH= 21.66FT.

CASE I - PASSIVE RESISTANCE (PR) CHECK.

PR REQD = 9710 + 9170 = 18,880#



K_P REQD × .0625 × 10²/2 + K_{PR} × .0625 × 10 × 4.73 = 18.8

K_P REQD = 3.10 ≱ 3.0 AVAIL. OK

SUBJECT SAXONVILLE LOCAL PROTECTION
COMPUTATION TEE TYPE FLOOD WALL - STA 5+50
COMPUTED BY RLH CHECKED BY JAC DATE AUG. 1974

CONDITION #2:

CREEP RATIO = 2.85

	+VERT#	-VERT#	+HORIZ#	-HORIZ#	+MOMENT#FT	-MOMENT#FT
CONC=	16578.39				198384.82	
WEL1=	4181.13				16453.53	
WEL2=	432.87				3641.98	
WER1=	2062.50				34031.25	
WER2=	4382.81				96421.87	
WR1 =	22687.50				436734.38	
WR2 =	2320.31				51046.87	
WL1 =	4181.13				16453.53	
WL2 =	432.87				3641.98	
UI =		23208.52				319117.19
U2 =		12458.23				228401.04
PW1 =			24500.00			112972.22
PW2 =				4219.73	14065.77	
PW3 =				3985.30		9409.74
PW4 =				2139.29		6734.81
PR1 =				7280.06	24266.86	
PR2 =				6875.61		16234.08
SUMS=	57259.52	35666.76	24500.00	24499.99	895142.89	692869.08
SUMV=	21592.76					
SUMH=			.00			
SUMM=					202273.80	
M/V =	9.36					
SBL=	1535.94PSF					
SBR=	34.43PSF					

IF SBR IS NEGATIVE THEN SBR =0 & SBL = 1536.68PSF
BEARING LENGTH= 28.10FT.

CONDITION #3:-----
WEIGHT OF EARTH(SAT.)=125
HEIGHT OF EARTH LANDSIDE PLUS SURCHARGE=10.5

NO CREEP

	+VERT#	-VERT#	+HORIZ#	-HORIZ#	+MOMENT#FT	-MOMENT#FT
CONC=	16578.39				198384.82	
WEL1=	10329.86				40649.91	
WEL2=	1069.44				8997.85	
WER1=	4125.00				68062.50	
WER2=	8765.62				192843.75	
WIND=				713.88	13663.04	
PEL1=				2970.00	11880.00	
PEL2=				2337.50		5519.09
PEL3=				459.92		1447.91
SUMS=	40868.32			6481.31	534481.88	6967.00
SUMV=	40868.32					
SUMH=				-6481.31		
SUMM=					527514.87	
H/V =	-.15					
M/V =	12.90					
SBL =	1759.24PSF					
SBR =	1213.00PSF					

SLIDING

TAN SLOPE : 4.73/27.5 = 0.172
ΣH/V = 0.15
ΣH/V ON SL. = 0.322

φ = 30°
TAN φ = 0.577
0.577/0.322 = 1.79 > 1.5

SUBJECT SAXONVILLE LOCAL PROTECTION
COMPUTATION TEE TYPE FLOODWALL - STA 5+50
COMPUTED BY RLH CHECKED BY JHC DATE AUG 1974

CASE 1 LOADING - STEM

ENTER FOLLOWING ITEMS:----
HEIGHT OF EARTH R.S. =4.0
HEIGHT OF WATER R.S. =25.0
HEIGHT OF EARTH L.S. =8.5
WEIGHT OF EARTH =62.5

	DIST	D IN	V LBS	M FT LBS	AS SQ IN	V #/SQ IN	
0 =		33.05 ~~34.05~~	18951.17	160869.36	~~3.18~~	3.28	42.05
1/5=	5.00	23.05 ~~24.05~~	12373.67	83185.95	~~2.32~~	2.42	
2/5=	10.00	13.50 ~~14.50~~	7031.25	35156.25	~~1.68~~	1.75	
3/5=	15.00	13.50 ~~14.50~~	3125.00	10416.66	~~.48~~	0.52	

CASE 3 LOADING - STEM
ENTER FOLLOWING
WEIGHT OF EARTH =125
HEIGHT OF EARTH L.S.=10.5
HEIGHT OF " R.S.=4.0
HEIGHT OF WIND LOAD =16.5

0 =		33.05 ~~34.05~~	2768.90	21337.42	~~.42~~	.43
1/5=	5.00	23.05 ~~24.05~~	1448.90	10837.57	~~.30~~	.31
2/5=	10.00	13.50 ~~14.50~~	825.00	5568.75	~~.25~~	.27
3/5=	15.00	13.50 ~~14.50~~	500.00	2500.00	~~.11~~	.12

CASE I LOADING LANDSIDE BASE
ENTER FOLLOWING ITEMS
LENGTH OF BEARING =21.66
BEARING =1978.49
HEIGHT OF WATER =31
" " EARTH =8.5
WEIGHT OF EARTH =62.5

	DIST	D IN	V #	M FT LBS	AS SQ IN	V #/SQ IN	
STEM=		27.71 ~~29.71~~	-10018.97	-42641.40	~~.96~~	1.03	-31.53
1/3=	2.62	22.31 ~~24.31~~	-7223.95	-19904.29	~~.55~~	-0.60	
2/3=	5.24	16.90 ~~18.90~~	-3884.29	-5214.21	~~.18~~	-0.20	

CASE 3 LOADING LANDSIDE BASE
ENTER FOLLOWING ITEMS:
WEIGHT OF EARTH =125
HEIGHT OF EARTH =8.5
BEARING P (HEEL) =1213.00
BEARING P (TOE) =1759.24

STEM=		27.71 ~~29.71~~	-2299.82	-10903.64	~~.84~~	-0.26

CASE I LOADING RIVERSIDE BASE

STEM=		33.50 ~~34.50~~	8331.43	75883.82	~~1.48~~	1.52	22.59
1/3=	5.50	26.83 ~~27.83~~	6763.86	38161.23	~~.90~~	0.95	
2/3=	11.00	20.16 ~~21.16~~	3931.53	10547.92	~~.33~~	0.35	

CASE 3 LOADING RIVERSIDE BASE

STEM=		32.50 ~~34.50~~	-4052.76	-18909.75	~~.36~~	-0.38
1/3=	5.50	25.83 ~~27.83~~	-1193.61	-11221.87	~~.27~~	-0.29

CORPS OF ENGINEERS, U.S. ARMY

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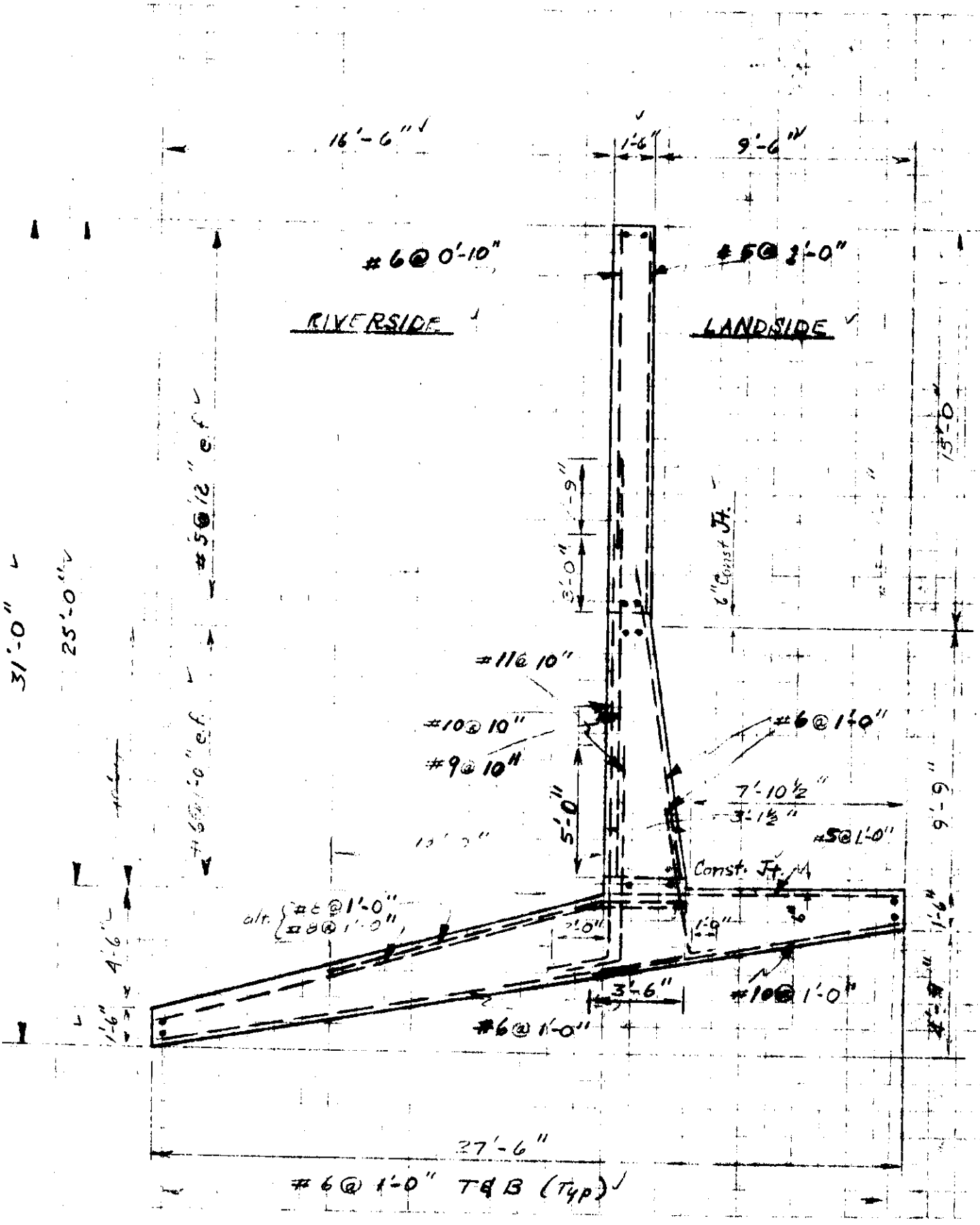
SUBJECT SAXONVILLE LOCAL PROTECTION

COMPUTATION TEE WALL @ STA 5+50

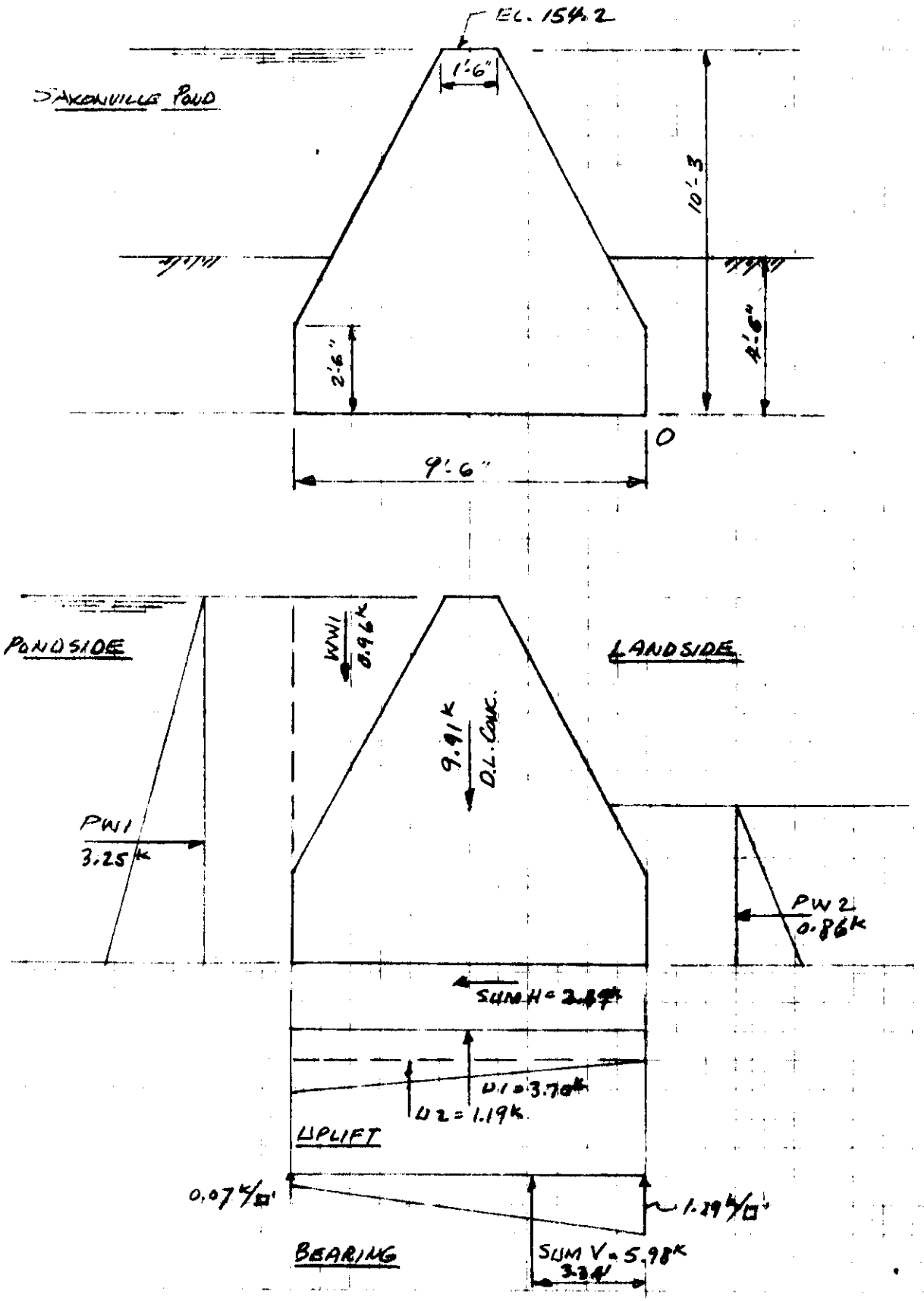
COMPUTED BY J.A.C. CHECKED BY JAC DATE AUG 74



SUBJECT SAXONVILLE LOCAL PROTECTION

COMPUTATION GRAVITY WALL @ SAXONVILLE POND - STABILITY

COMPUTED BY R. HARRINGTON CHECKED BY JRF DATE AUG. 1974



SUBJECT SAXONVILLE LOCAL PROTECTION

COMPUTATION CONCRETE GRAVITY WALL @ SAXONVILLE POND

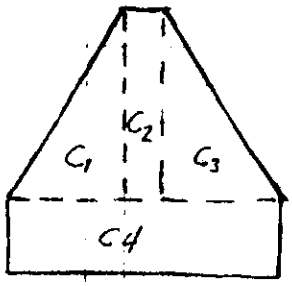
COMPUTED BY R.L.H. CHECKED BY JRB DATE AUG. 1974

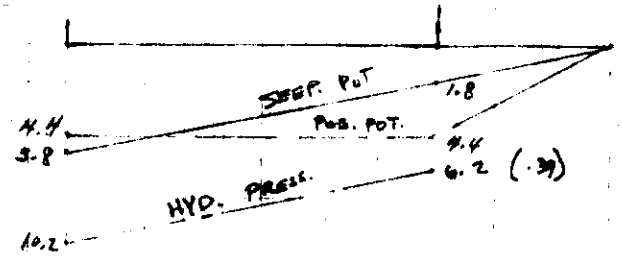
ASSUMPTIONS: $\gamma_{SAT} = 125\ \#/ft^3$

$\gamma_{SUB} = 62.5\ \#/ft^3$

$K_a = 0.33$

$\phi = 30°, \tan\phi = 0.577$





STABILITY @ LANDSIDE TOE (O) POND TO TOP OF WALL

LD	FACTOR	↓	↑	→	←	ARM @ O	↻	↺
C_1	0.150 × 4.0 × 7.7 × .5	2.31				6.83		15.77
C_2	0.150 × 1.5 × 7.7	1.73				4.75		8.22
C_3	0.150 × 4.0 × 7.7 × .5	2.31				2.67		6.19
C_4	0.150 × 2.5 × 9.5	3.56				4.75		16.91
W_{W1}	0.0625 × 7.7 × 4.0 × 0.5	0.96				8.17		7.84
P_{W1}	0.64 × 10.2 × 0.5			3.25		3.40	11.05	
P_{W2}	0.39 × 4.4 × 0.5				.86	1.47		1.26
U1	0.39 × 9.5		3.70			4.75	17.58	
U2	0.25 × 9.5 × 0.5		1.19			6.34	7.54	
		10.87	4.89	3.25	.86		36.17	56.17
	Σ	5.98		2.39				20.00

$\Sigma H/\Sigma V = 2.39/5.98 = 0.39$ $\quad \frac{0.577}{0.39} = 1.48 > 1.50$

$\Sigma M/\Sigma V = 20/5.98 = \underline{3.34}$ $\quad 9.5/3 = 3.17$ INSIDE MIDDLE ⅓ OK

$e = 4.75 - 3.34 = 1.41$

$$B.P. = \frac{5.98}{9.5} \pm \frac{5.98 \times 1.41 \times 6}{9.5^2} = 0.63 \pm 0.56$$

B.P. TOE = 1.19 #/□'

B.P. HEEL = 0.07 #/□'

C.R. = 2.39'

NED FORM 223
27 Sept 49

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

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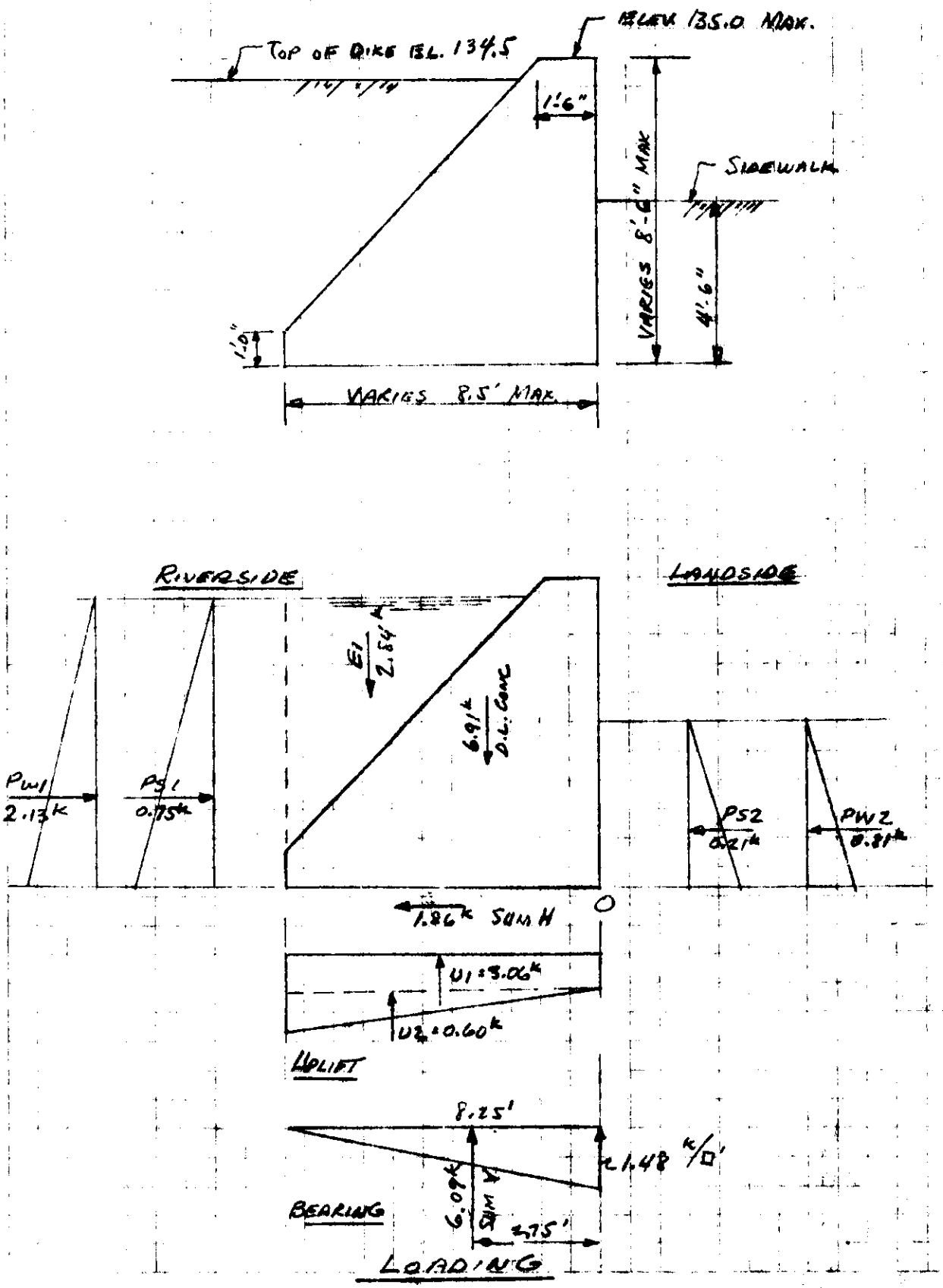
SUBJECT SAXONVILLE LOCAL PROTECTION
COMPUTATION GRAVITY WALL @ DANFORTH ST.
COMPUTED BY R.L.H. CHECKED BY JRF DATE AUG 1974



SUBJECT SAXONVILLE LOCAL PROTECTION

COMPUTATION GRAVITY WALL @ DANFORTH ST

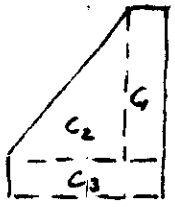
COMPUTED BY R.L.H. CHECKED BY JRF DATE AUG. 1974

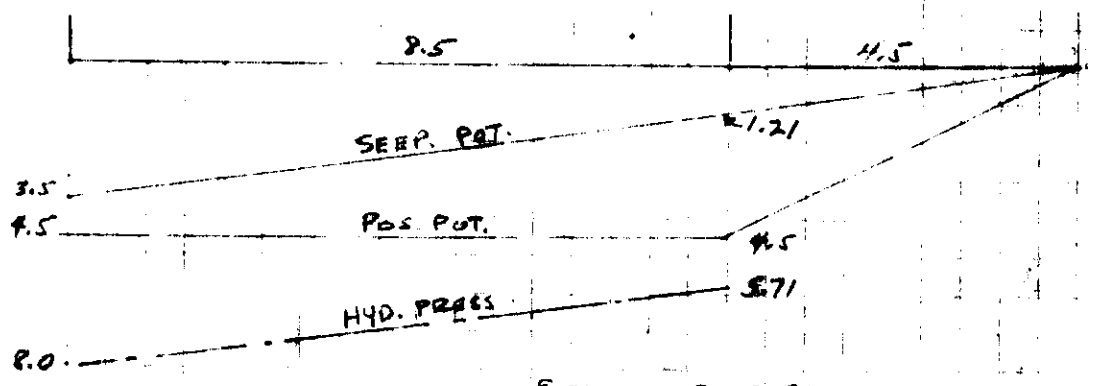
ASSUMPTIONS: $\gamma_{SAT} = 125\ \#/FT^3$

$\gamma_{SUB} = 62.5\ \#/FT^3$

$K_a = 0.33$

$\Phi = 30°$, $\tan\Phi = 0.577$





$8.0 \times .0625 = 0.50$

$5.71 \times .0625 = 0.36$

STABILITY @ O (BASE WIDTH = 8.5') WATER TO TOP OF DIKE

LD.	FACTOR	↓	↑	→	←	ARM @ O	↷	↶
C1	7.5 × 1.5 × 0.15	1.69				0.75		1.27
C2	7.0 × 7.5 × 0.5 × 0.15	3.94				2.83		15.09
C3	8.5 × 1.0 × 0.15	1.28				4.25		5.44
E1	7.0 × 6.5 × 0.5 × 0.125	2.84				6.33		17.98
U1	0.36 × 8.5		3.06			4.25	13.00	
U2	0.14 × 8.5 × 0.5		0.60			5.67	3.40	
Pw1	0.50 × 8.5 × 0.5			2.13		2.83	6.03	
PS1	0.33 × .0625 × 8.5² × 0.5			0.75		2.83	2.12	
Pw2	0.36 × 4.5 × 0.5				0.81	1.50		1.22
PS2	0.33 × .0625 × 4.5² × 0.5				0.21	1.50		0.31
		9.75	3.66	2.88	1.02		24.55	41.31
	Σ	6.09		1.86				16.76

$\Sigma H / \Sigma V = \frac{1.86}{6.09} = 0.31$ $\quad \frac{0.577}{0.31} = 1.86 > 1.5$ OK

$\Sigma M / \Sigma V = \frac{16.76}{6.09} = 2.75$ $\quad 8.5/3 = 2.83$ OK.

$e = 4.25 - 2.75 = 1.5'$

$B.P._{MAX} = \frac{2 \times 6.09}{3 \times 2.75} = 1.48^k/S.F.$

LENGTH BEG = 2.75 × 3 = 8.25

SUBJECT SAXONVILLE

COMPUTATION I-Wall STA. 18+85 - 19+20

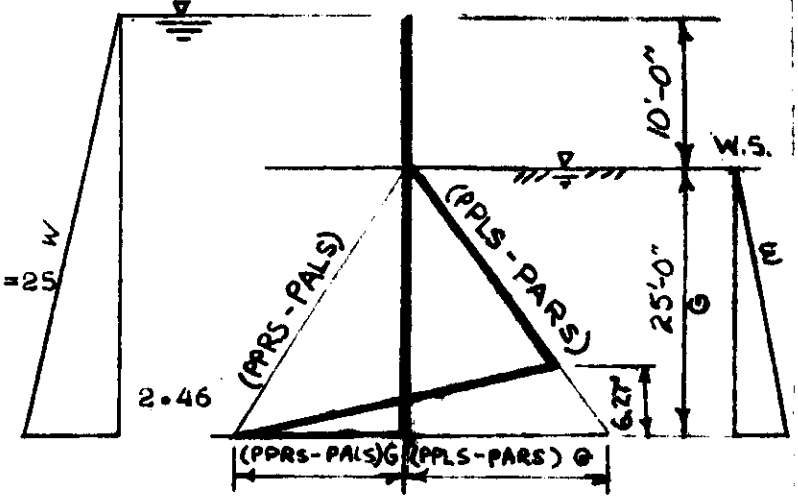
COMPUTED BY JRF CHECKED BY RLH DATE AUG. 1974

I-WALL STABILITY
FIND MINIMUM PENETRATION (X)
FIND Z DISTANCE
GIVEN WEIGHT OF SOIL,WATER,KP,KA,HEIGHT OF WALL ABOVE GRADE & T.W.ELEV.
BY C.M.TERZIAN 8JULY66

LOADING:-------RIVERSIDE WATER TO TOP OF WALL
LANDSIDE WATER AT ANY HEIGHT
GRADE EQUAL BOTH SIDES
DRY AND SUBMERGED PASSIVE PRESSURE CONSIDERED DUE TO T.W.

ALL UNITS POUND-FEET

WGT OF EARTH(SAT) =125
KA =.41
KP =2.46
WGT OF WATER =62.5
WALL HGT ABOVE GRADE =10
L.S. WATER TO TOP WALL=10
GUESS PENETRATION (X),(>ACTUAL)=25



X, Z, KP = 23.75 5.96 2.46

FIND KP AND Z WHEN PENETRATION (X) IS GIVEN (MUST BE > MINIMUM)
LOADING SAME AS ABOVE

GIVEN PENETRATION (X)=25

X, Z, KP = 25 6.27 2.33

FIND POINT OF ZERO SHEAR(DIST.DOWN FROM GRADE) =XO FT
FIND MAXIMUM MOMENT =M FT-LBS
FIND SECTION MODULUS FOR FS=20000PSI =SM CU-IN

XO, M, SM = 14.50 60237.49 36.14

PPLD= 292.12 < 307.5 = 125 x 2.46 OK
PPRS= 146.06 < 153.7 = 62.5 x 2.46 OK
PALD= 51.25
PALS= 25.62

USE PZ-32 (S.M. = 38.3 in.^{3})
WITH 25'-0" PENETRATION

END

SUBJECT SAXONVILLE LOCAL PROTECTION
COMPUTATION I-TYPE WALL DESIGN OF TYPICAL SECTION
COMPUTED BY R. HILLINGTON CHECKED BY JRF DATE AUG. 1974

SHEET PILE SELECTION

MAX MOM. = 60,237 '# f_s ALLOW = 20,000 #/in²

$S_{REQD} = \frac{60,237 \times 12}{20,000} = 36.14$ in³

USE - PZ 32 S PROVIDED = 38.3 in³

REINFORCEMENT

VERT. REINF, RIVERSIDE
DESIGN FOR MOMENT 2' BELOW BERM LEVEL

@ PILE ENCASEMENT

$M = \frac{.0625 \times 12^3}{6} = 18.00$ 'k

$d = 30'' - 4.5'' = 25.5''$

A_s REQD $= \frac{18}{1.44 \times 25.5} = 0.49$ in²

@ CHANGE IN WIDTH (DESIGN FOR 10' HEIGHT)

$M = \frac{.0625 \times 10^3}{6} = 10.4$ 'k

$d = 18'' - 4.5'' = 13.5''$

A_s REQD $= \frac{10.4}{1.44 \times 13.5} = 0.53$ in² $V = .0625 \times 10^2 \times \frac{1}{2} = 3.13$ k/'

#7 @ 12 $A_s = 0.60$ in² $4 = \frac{3130}{2.75 \times \frac{7}{8} \times 13.5} = 96.3$ < [illegible]

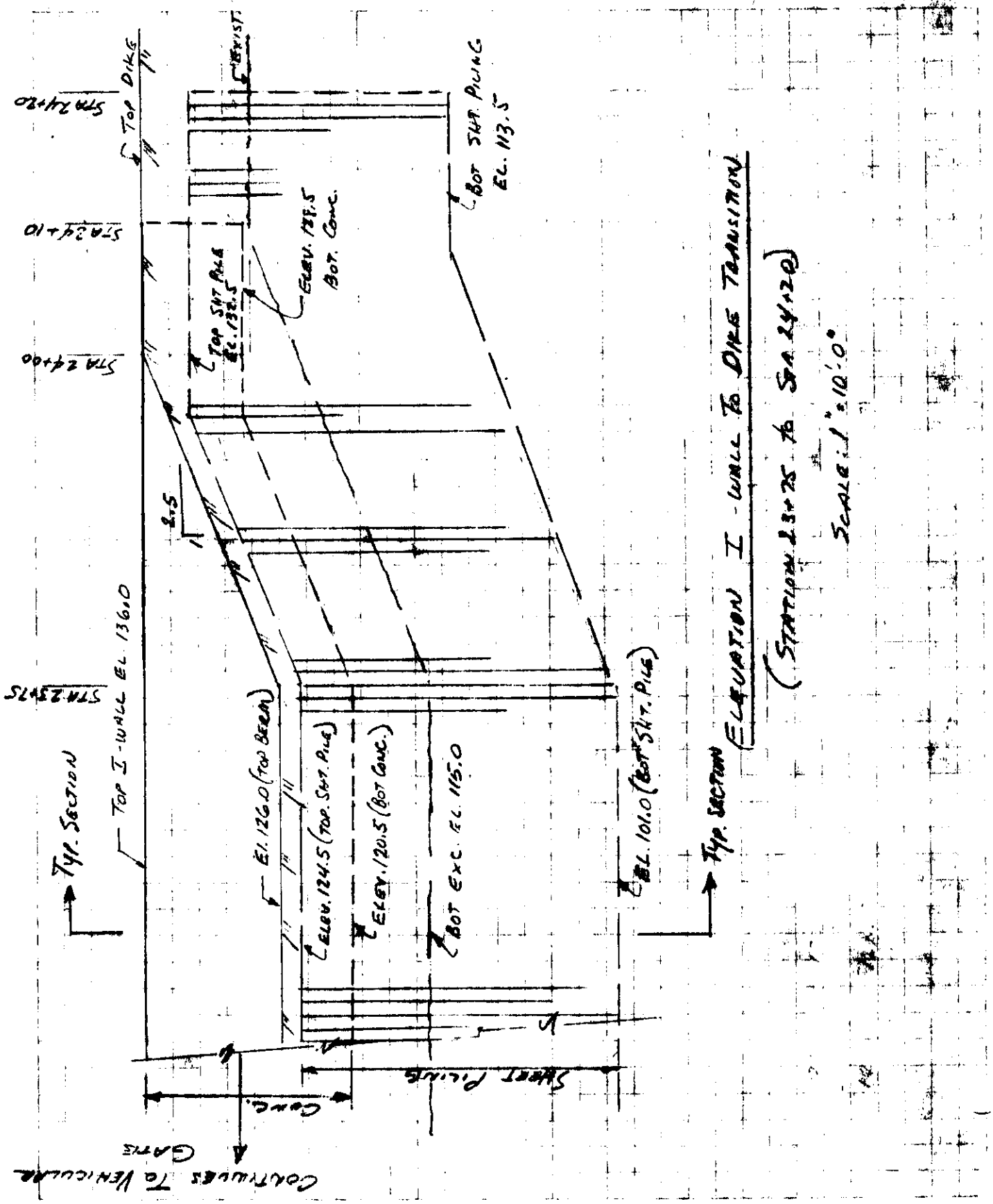
USE - #7 @ 12" RIVERSIDE VERTICAL

USE - #6 @ 12 BELOW GR. & #5 @ 12 ABOVE GR. LANDSIDE VERT.

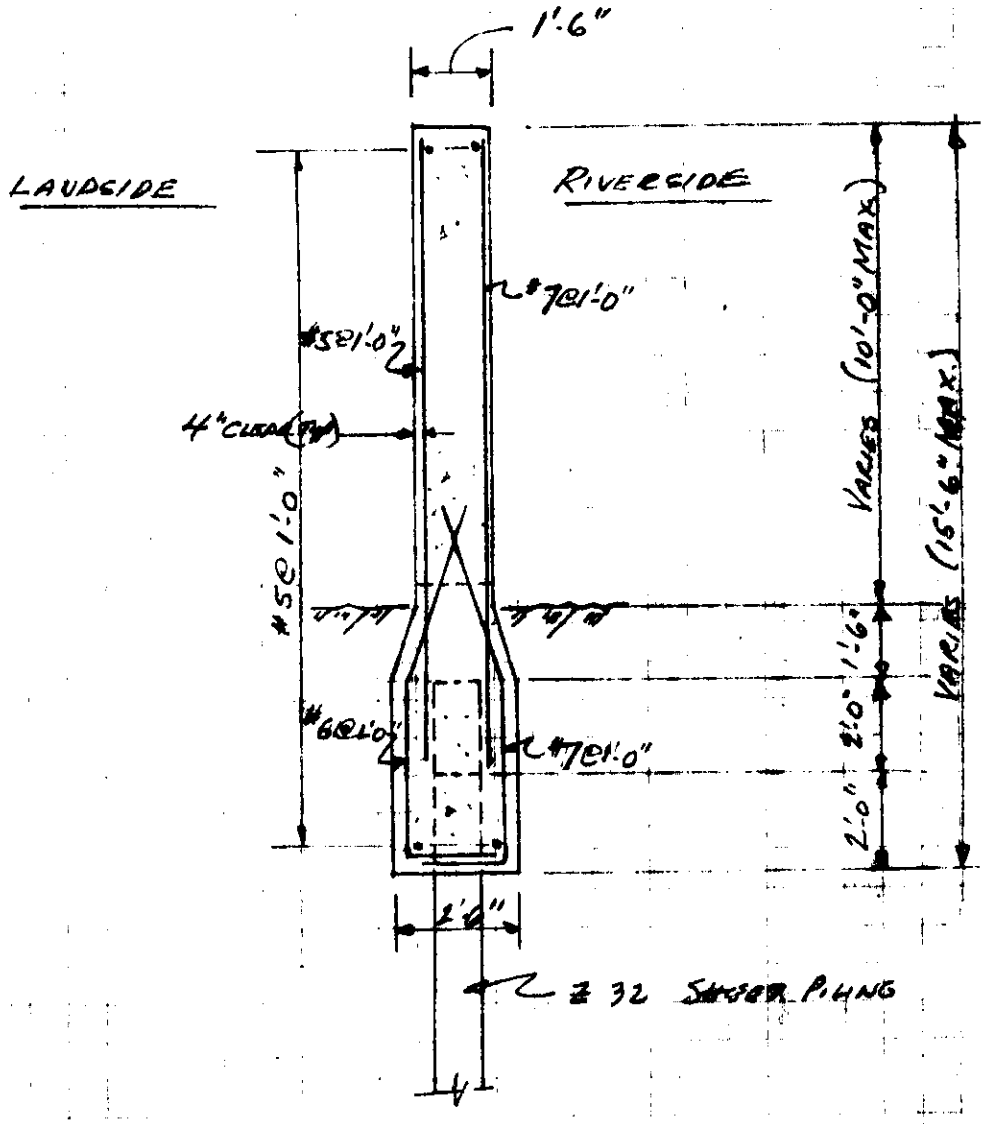
HORIZONTAL REINF.

$.001 \times 18'' \times 12'' = 0.22$ USE - #5 @ 12 EACH FACE HORIZ.

SUBJECT SAVANNAH LOCAL PROTECTION
COMPUTATION I-WALL ELEVATION
COMPUTED BY R. HARRINGTON CHECKED BY JRF DATE AUG. 1971



SUBJECT SAXONVILLE LOCAL PROTECTION
COMPUTATION "I" WALL - TYPICAL SECTION
COMPUTED BY R. HARRINGTON CHECKED BY JRF DATE AUG 1974



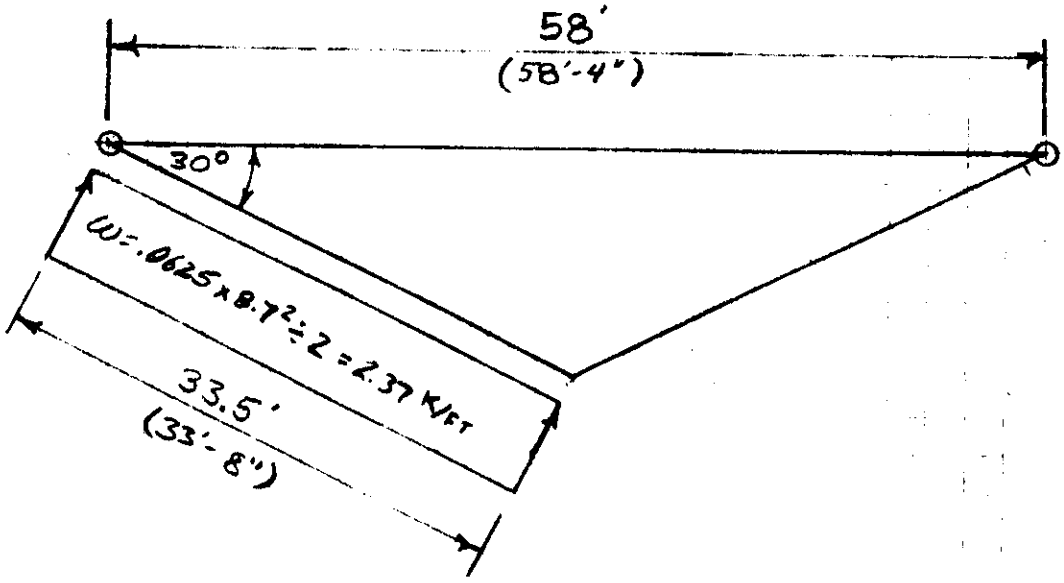
TYPICAL "I" WALL

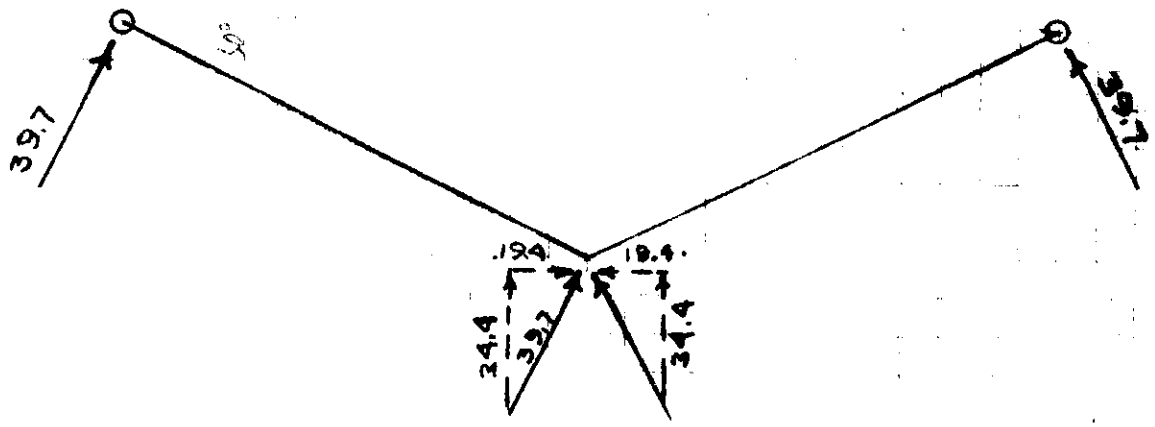
SCALE: 1/4" = 1'-0"

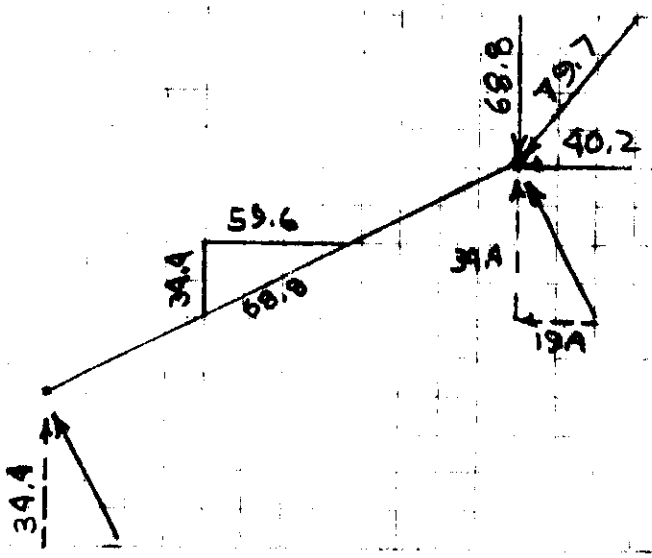
SUBJECT SAXONVILLE

COMPUTATION STREET GATE - STEEL GATE DESIGN (1)

COMPUTED BY JRF CHECKED BY TAJ DATE July 1974





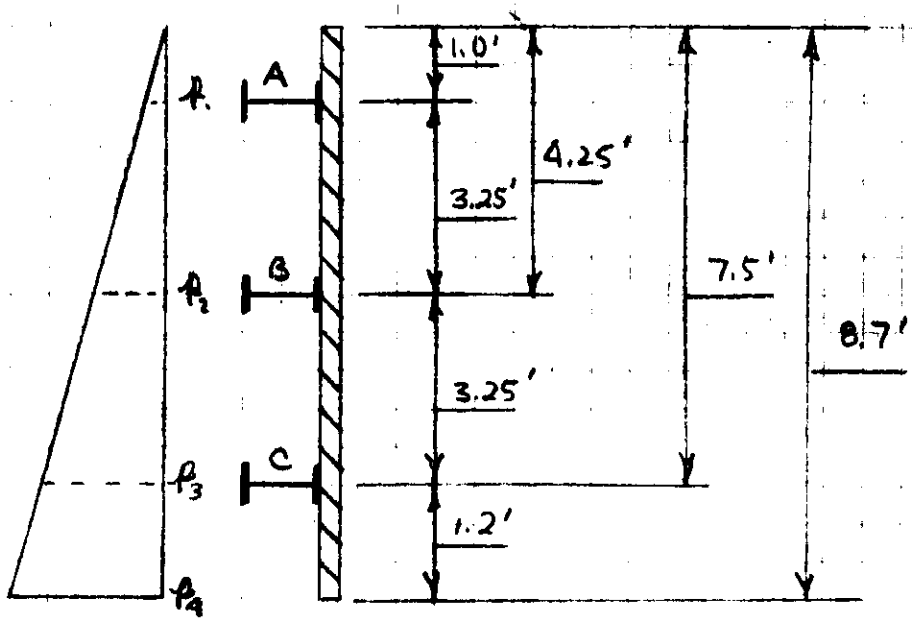


SUBJECT SAXONVILLE

COMPUTATION Street Gate - STEEL GATE DESIGN (2)

COMPUTED BY JRF CHECKED BY DATE July 1974

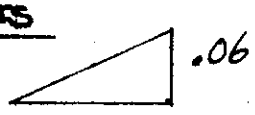
TRY EQUAL SPACING BETWEEN BEAMS A & C



p_1 = .0625 × 1.0 = .06 K/FT²
p_2 = .0625 × 4.25 = .27 " "
p_3 = .0625 × 7.50 = .47 " "
p_4 = .0625 × 8.70 = .54 " "

FIXED END MOMENTS

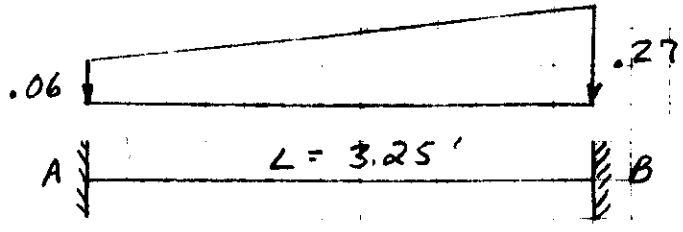
FEMA = ½ × .06 × 1 × ⅓



FEMA = .01 K·ft.

FEMAB = $\frac{.06 \times 3.25^2}{12} + \frac{.21 \times 3.25^2}{30}$

FEMAB = .13 K·FT



FEMBA = $\frac{.06 \times 3.25^2}{12} + \frac{.21 \times 3.25^2}{20}$

FEMBA = .16 K·FT

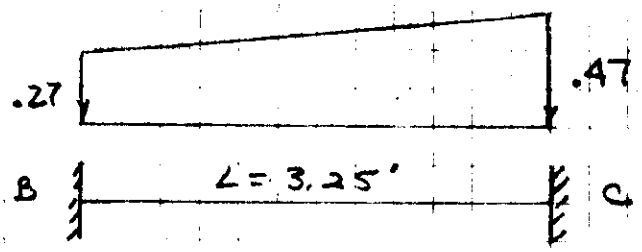
SUBJECT Saxonville

COMPUTATION Street Gate - STEEL GATE DESIGN (3)

COMPUTED BY JRF CHECKED BY TKH DATE JULY 1974

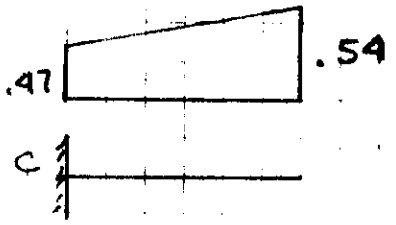
$FEM_{BC}: \frac{.27 \times 3.25^2}{12} + \frac{.20 \times 3.25^2}{30}$

$FEM_{BC} = .31$ K·FT



$FEM_{CB}: \frac{.27 \times 3.25^2}{12} + \frac{.20 \times 3.25^2}{20}$

$FEM_{CB} = .34$ K·FT



$FEM_C = \frac{1.2^2}{6}(2 \times .54 + .47)$

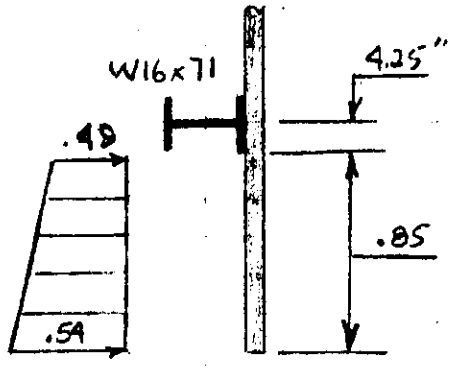
$FEM_C = .37$ K·FT

	A		B		C	
	0	1	.50	.50	1	0
	.01	-.13	+.16	-.31	+.34	-.37
		+.12	+.075	+.075	+.03	
		+.04	+.06	+.015	+.04	
		-.04	-.03	-.015	-.04	
MOMENTS (K·FT)	-.01	.01	+.265	-.265	+.37	-.37
REACTIONS (K)	.03	.13	.41	.51	.69	.61
BEAM LOAD (K)	.16		.92		1.30	

SUBJECT SAXONVILLE

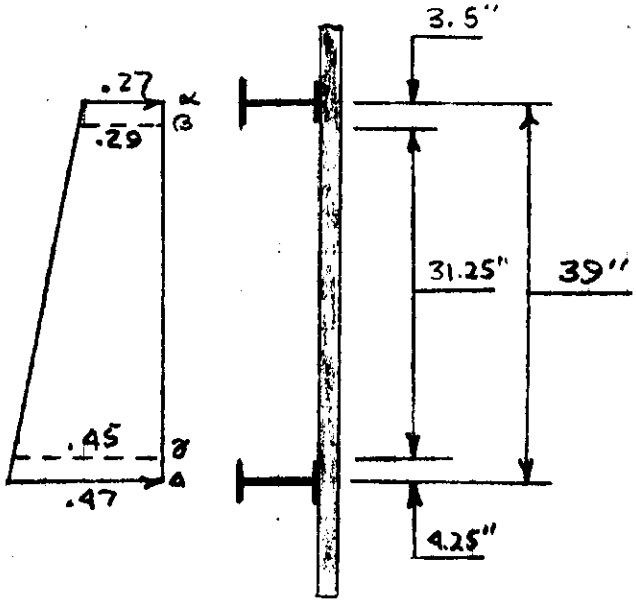
COMPUTATION Street Gate - STEEL GATE DESIGN (4)

COMPUTED BY JRF CHECKED BY JKH DATE July 1974



TAKE MOMENT AT END OF FLANGE OF BOTTOM MEMBER

$$M = \frac{.85^2}{6}(2 \times .54 + .49) = .19 \text{ K}\cdot\text{FT}$$



$$FEM\alpha = .27 \times \frac{3.25^2}{12} + \frac{.20 \times 3.25^2}{30}$$

$$FEM\alpha = .31^{K'}$$

$$FEM\beta = .29 \times \frac{2.60^2}{12} + \frac{.16 \times 2.60^2}{30}$$

$$FEM\beta = .20^{K'}$$

$$FEM\gamma = \frac{.29 \times 2.6^2}{12} + \frac{.16 \times 2.6^2}{20}$$

$$FEM\gamma = .22^{K'}$$

$$FEM\Delta = \frac{.27 \times 3.25^2}{12} + \frac{.20 \times 3.25}{20}$$

$$FEM\Delta = .34^{K'}$$

DESIGN SKIN PL

1. USING MOMENT DISTR. W/ MOMENT @ WEB

$M_{max} = .37^{K'}$

$S_{REQ} = \frac{.37 \times 12}{18} = .247$ $\quad S_{PL} = 2t^2$

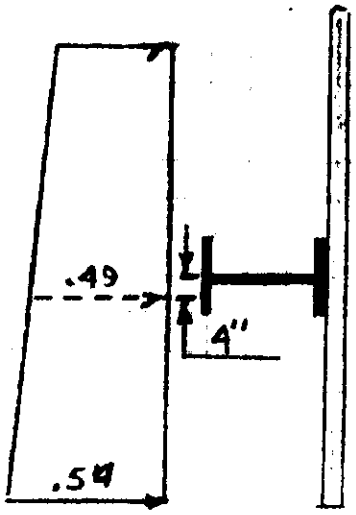
$S_{PL} = S_{REQ}$

$2t^2 = .247$ $\quad F_b = \frac{.37 \times 12}{.25} = 17.8$ KSI

$t = .35$ in. ∴ USE 3/8" PL A36 $t = .375''$

2. USING MOMENT DISTRIBUTION W/ 1/3 Va REDUCTION

$\frac{b_f}{12} = \frac{8.5}{12} = .71$ FT.



$M = .37 - \frac{1}{3}(.69)(.71)$

$M = .21^{K \cdot FT}$ (UPPER PART OF FLANGE)

$M = \frac{.85^2}{6}(2 \times .54 + .49)$

$M = .19^{K \cdot FT}$ (LOWER PART OF FLANGE)

$S_{REQ} = \frac{.21 \times 12}{18} = .14$

$2t^2 = .14$ $\quad F_b = \frac{.21 \times 12}{.195} = 12.9$ KSI

$t = .26$ in ∴ USE 5/16" PL A36 $t = .31$ in.

SUBJECT SAXONVILLE

COMPUTATION Street Gate - STEEL GATE DESIGN (7)

COMPUTED BY JRF CHECKED BY TKH DATE JULY 1974

4. USING FIXED-END MOMENTS @ ENDS OF FLANGES

USE FEM γ = .22 K·FT [PG. A(10)]

$S_{REQ} = \frac{.22 \times 12}{18} = .15$

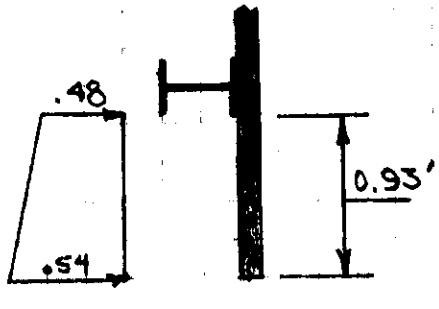
$2t^2 = .15$ $f_b = \frac{.22 \times 12}{.195} = 13.5$ KSI

$t = .27$ in. ∴ USE 5/16" PL (A36) $t = .31$ in. $S = .195$

CONCLUSIONS

USE 5/16" PL

IF FRAME MEMBERS ARE LOCATED ONE INCH HIGHER ON THE LEAF (THE 1" IS FOR CLEARANCE CONSIDERATION AT THE BOTTOM OF THE LEAF) CHECK CANTILEVER MOMENT



$M = \frac{.93^2}{6}(2 \times .54 + .48)$

$M = .22$ K·FT

$f_b = \frac{.22 \times 12}{.195}$

$f_b = 13.5$ KSI

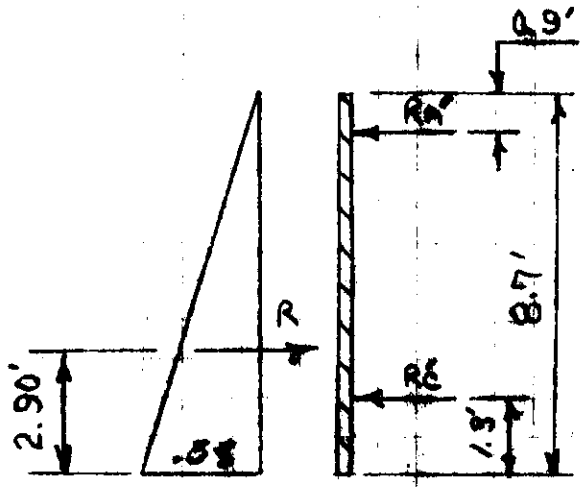
∴ THE 5/16" PL IS OK

SUBJECT SAXONVILLE
COMPUTATION STREET GATE - STEEL GATE DESIGN
COMPUTED BY JRF CHECKED BY TKM DATE JULY 19[illegible]

FRAMING DESIGN

AXIAL LOADS ACTING ON THE GATE WILL BE TAKEN BY THE TOP & BOTTOM MEMBERS. THE AXIAL LOADS WILL BE TAKEN DIRECTLY BY THE CONCRETE SINCE THE HINGES WILL BE LOCATED AT THE TOP AND BOTTOM MEMBERS.

AXIAL LOADS



$\Sigma M_C = 0$

$6.5\,R'_A = \frac{1}{2} \times 8.7 \times 0.55\,(2.9 - 1.3)$

$R'_A = 0.59^{K/'}$

$R'_C = \frac{1}{2} \times 8.7 \times 0.55 - 0.59$

$R'_C = 1.80^{K/'}$

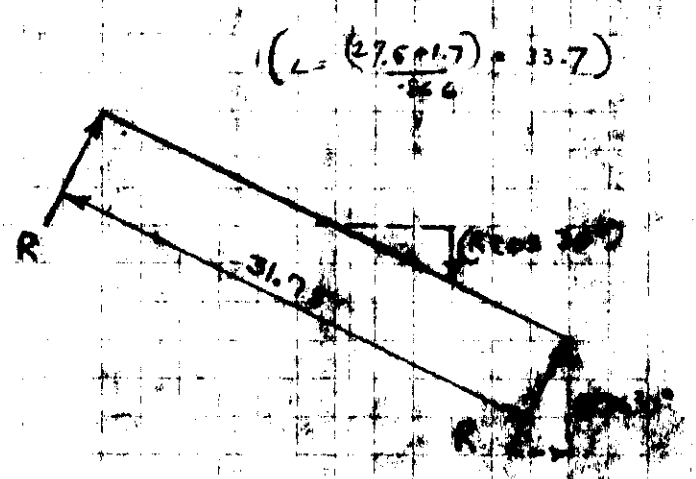
$R_A = 0.59 \times \frac{31.75}{2} = 9.4^K$ ✓

$R_C = 1.80 \times \frac{31.75}{2} = 28.6^K$ ✓

$P = R \cos 30^\circ \div \sin 30^\circ$

$P_A = (9.4)(.866) \div (0.500) = 17.6^K$ (16.3)

$P_C = (28.6)(.866) \div (0.500) = 50.4^K$ (49.5)



$F_u = .055$ K/in (ONE SIDE)

TOTAL FORCE $= \sqrt{55^2 + 50^2}$

TOTAL FORCE = 74.3 lb/in < 2400 lb/in ∴ WELDS OK

CHECK HORIZONTAL BEAMS w/ 5/16" PL

BEAM C

OK FOR COMBINED SHEAR ON PL see pg (9)

Combined Axial Load & BENDING

$r = \sqrt{\frac{1247}{26.65}} = 6.84$

$l/r = \frac{32.5 \times 12}{6.84} = 57.0$ ∴ $F_a = 14.7$

$\frac{f_a}{F_a} = \frac{1.9}{14.7} = .13$

$f_b = \frac{171.6 \times 12}{126.5} = 16.3$

$.13 + \frac{16.3}{18} = 1.04$ ∴ N.G.

TRY W 16 x 78

A =	23.0	d =	16.32
I =	1050	b_f =	8.59
S =	128	d/A_f =	2.17

$b = 8.59 + 2 \times 15.8 \times .3125$
$b = 8.59 + 9.88$
$b = 18.47$

$AR = 18.47 \times .3125 = 5.77$

$\Sigma A = 23.0 + 5.8 = 28.8$

$\Sigma M_{o-o} = 0 + 5.77\,(8.16 + .3125 \div 2) = 48.0$

$\frac{\Sigma M}{\Sigma A} = \frac{48.0}{28.8} = 1.67$

$I = 1050 + 23 \times 1.67^2 + 5.77 \times 6.65^2 = 1369$

$S_{TOP} = \frac{1369}{6.80} = 201.3$ $\qquad S_{BOT} = \frac{1369}{9.83} = 139.3$

COMBINED SHEAR ON R

$f_{bH} = \frac{171.6 \times 12}{201.3} = 10.2$

$f_a = \frac{50.4}{28.8} = 1.8$

$f_H = 8.4$ KSI

$f_{bv} = \frac{.22 \times 12}{.195} = 13.5$

MAX $v = \frac{13.5 + 8.4}{2} = 10.9 < 12.0$ ∴ OK

COMBINED BENDING & AXIAL LOAD

$r = \sqrt{\frac{1369}{28.8}} = 6.89$

$l/r = \frac{32.5 \times 12}{6.89} = 56.6$ ∴ $F_a = 14.7$

$f_b = \frac{171.6 \times 12}{139.3} = 14.8$ $\qquad \frac{1.8}{14.7} + \frac{14.8}{1.8} = .94 < 1.0$ ∴ USE W16x78 FOR BEAM C

SUBJECT SAXONVILLE
COMPUTATION STREET GATE - STEEL GATE DESIGN (17)
COMPUTED BY JRF CHECKED BY TKH DATE JULY 1974

VERTICAL BRACING

$P = \frac{9.5}{6} = 1.58^k$

TRY WT 8x13 $A = 3.84$ in.^{2} $r = 1.12$ in.

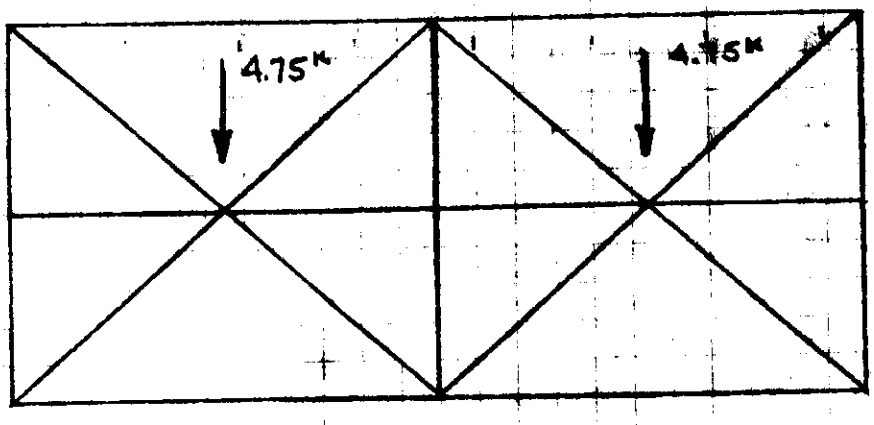
$l = 39$ in.

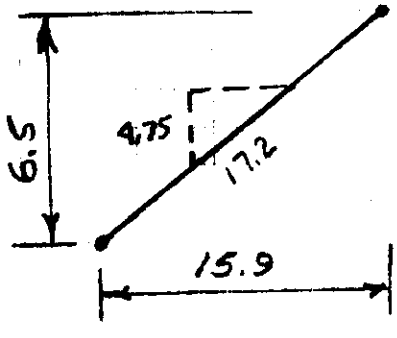
$l/r = \frac{39}{1.12} = 34.8$ $\therefore F_A = 16.3$ ksi

$F_a = \frac{1.58}{3.84} = .41$ ksi < 16.3 $\therefore$ WT 8x13 OK

DIAGONAL BRACING

l/r (MAX.) = 300 (TENSION)





$P = 4.75 \times \frac{17.2}{6.5} = 12.6^k$

$l = 17.2' \div 2 = 8.6'$

$r_{REQ} = \frac{8.6}{300} = .34$

C3x5 $r = .41$
C4x7.25 $r = .45$

SUBJECT SAKONVILLE

COMPUTATION STREET GATE - STEEL GATE DESIGN (19)

COMPUTED BY JRF CHECKED BY TKH DATE JULY 1974

Mom. @ 3/4 PT.

$.75 \times 31.75 = 23.8'$

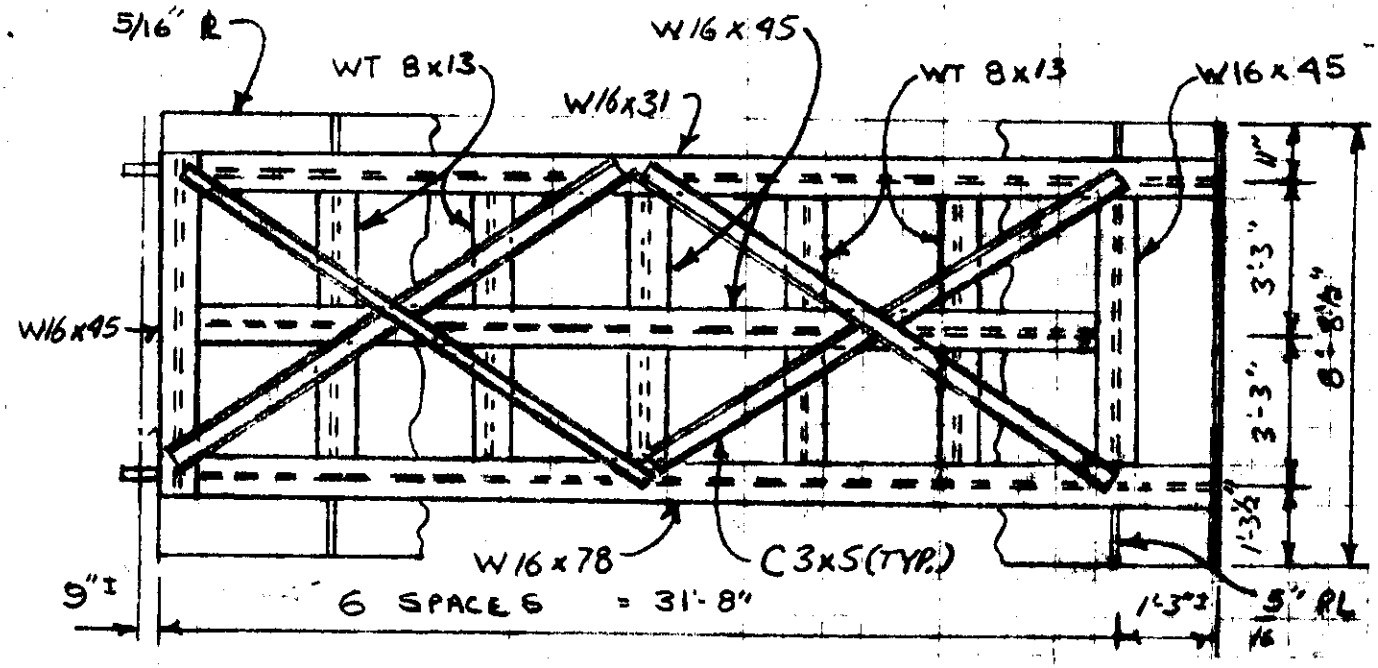
$$w = \frac{9.5^K}{31.75'} = 0.30 \text{ K/FT}$$

$$M = .30 \times \frac{23.8^2}{2} = 85.0 \text{ K·FT.}$$

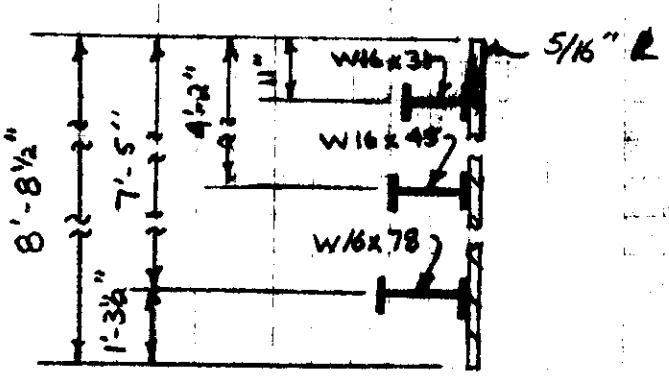
BOTTOM BEAM (C)

$$f = \frac{85.0 \times 12 \times 81.1}{72572} = 0.44 \text{ KSI NEGL.}$$

TOTAL STRESS @ OUTER FLANGE $= 14.8 + .4 = 15.2 < F_b$ ∴ OK



SECTIONS - STREET GATE
N.T.S.

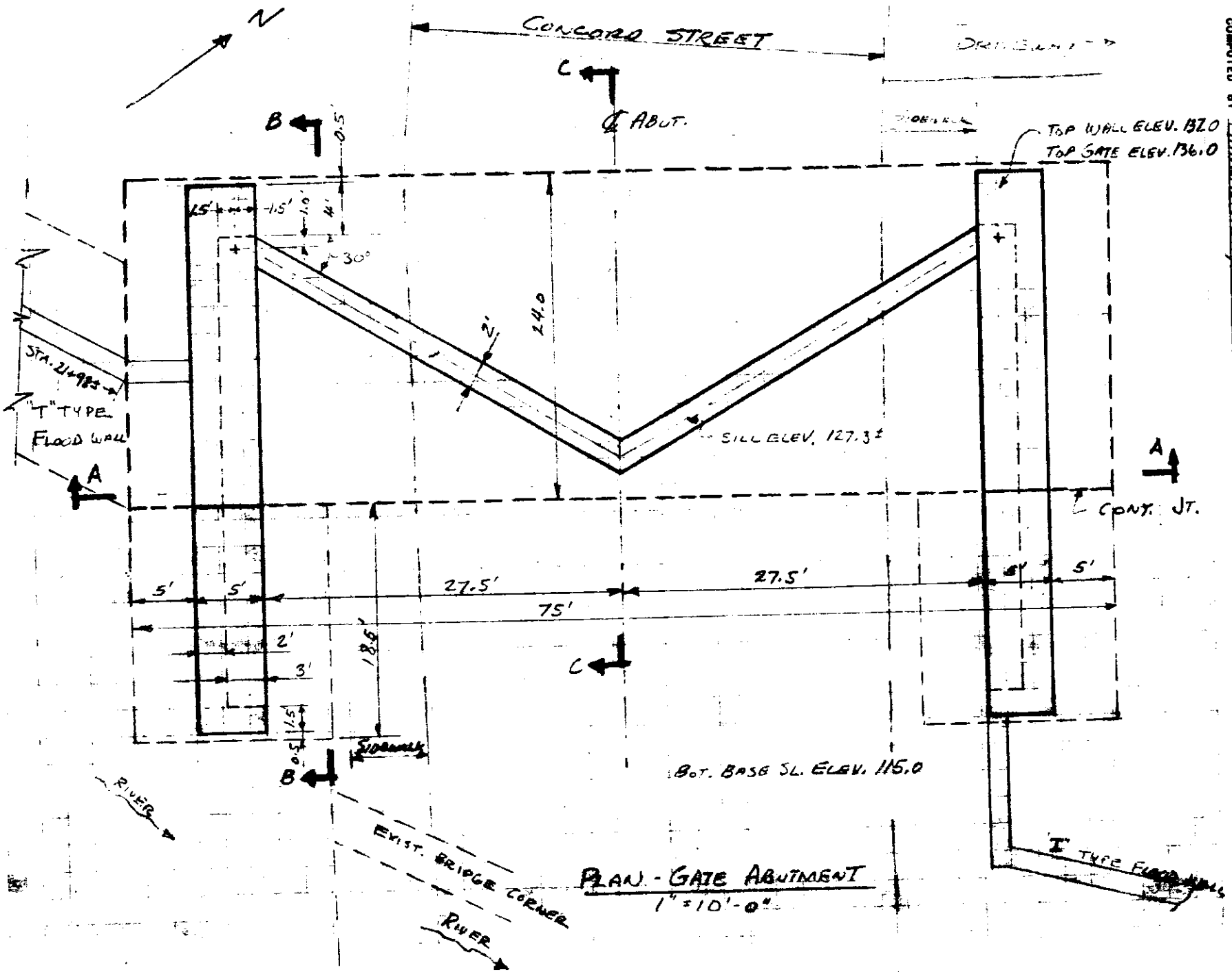


CORPS OF ENGINEERS, U.S. ARMY

SUBJECT Saxonville Locks Protection

COMPUTATION Lighter Gate - Sta 22+50± - Abutment

COMPUTED BY Harrington, R.L. CHECKED BY JRF DATE July 1974



Plan - Gate Abutment
1" = 10'-0"

27 Sept 49

CORPS OF ENGINEERS, U.S. ARMY

SUBJECT Sandy Lake Lock & Dam Protection

COMPUTATION Highway Gate - Sta 22+50± - Abutment

COMPUTED BY Harrington, R.C. CHECKED BY JRF DATE July 1974

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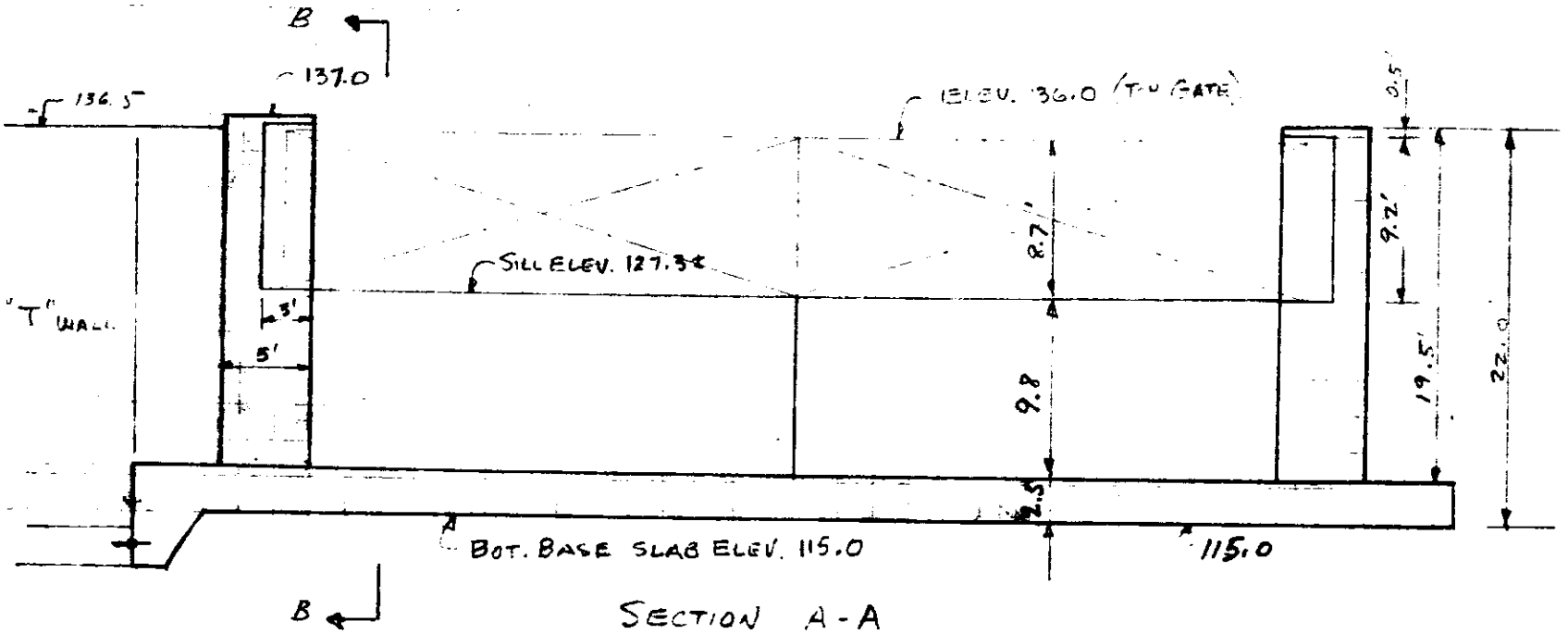
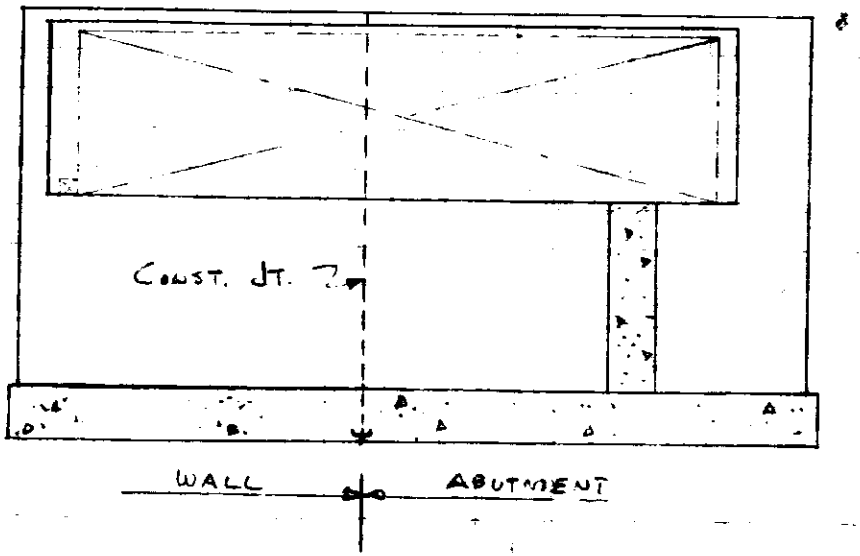


SECTION A-A

1" = 10'



SECTION B-B

SUBJECT SAXONVILLE LOCAL PROTECTION
COMPUTATION VEHICULAR GATE - STA 22+50± - ABUTMENT STABILITY
COMPUTED BY HARRINGTON, R.L. CHECKED BY JRF DATE JULY 1974

ASSUMPTIONS:

COMPACTED EARTH γ_{SAT} - 140 #/FT³ WT. CONC = 150 #/FT³
" " γ_{SUB} - 78 #/FT³ WT. WATER = 62.5 #/FT³
" " γ_{DRY} - 120 #/FT³ WT. STEEL = 490 #/FT³

ϕ = 25° TAN ϕ = 0.466

K_{ACTIVE} = 0.33 $K_{AT\ REST}$ = 0.50

FOUNDATION SOIL - SANDY SILT

ALLOWABLE BEARING PRESSURE = 1500 #/SF NET
MODULUS OF SUBSOIL REACTION = 150 #/IN³.

LOADING CONDITIONS

CASE I

STORM CONDITION - WATER TO TOP OF GATE (ELEV. 136.0). UPLIFT VARIES FROM FULL HEAD RIVERSIDE BASE TO ZERO AT GROUND SURFACE LANDSIDE BY "PATH OF CREEP METHOD." RESULTANT TO FALL WITHIN MID 1/2 OF BASE SLAB.

CASE II

STORM CONDITION - WATER 3' DOWN FROM TOP OF GATE (ELEV. 133.0) UPLIFT SAME AS CASE I. RESULTANT TO FALL WITHIN MID 1/3 OF BASE SLAB.

CASE III

TRAFFIC CONDITION - WITH 2-LANES H20-16 GATE OPEN, NO WATER

CASE IV

WIND LOADING - WATER DOWN, GATES CLOSED BUT NOT JACKED, FILL IN PLACE, 50 #/FT² WIND LOAD FROM LANDSIDE.

CASE V

NORMAL CONDITION - GATES IN POCKETS, (a) JACKED, (b) NOT JACKED (NOT CRITICAL)

SUBJECT SAXONVILLE LOCAL PROTECTION

COMPUTATION VEHICULAR GATE - STA 22+50± - ABUTMENT STABILITY

COMPUTED BY HARRINGTON, R.L. CHECKED BY J DATE JULY 1974

DESIGN STRESSES (REINFORCED CONC.)

$f_c' = 3000$ #/in²
$f_c = 1050$ #/in²
$f_s = 20{,}000$ #/in²

NOMENCLATURE

W_W = WEIGHT OF WATER
C_1 = WEIGHT OF CONCRETE
E_1 = WEIGHT OF EARTH
P_W = HORIZ. WATER PRESSURE
P_E = HORIZ EARTH PRESSURE
P_R = PASSIVE RESISTANCE
U = UPLIFT
G = WEIGHT OF GATE
W = WIND
W_E = WEIGHT OF EARTH

LOADS ON ABUTMENT

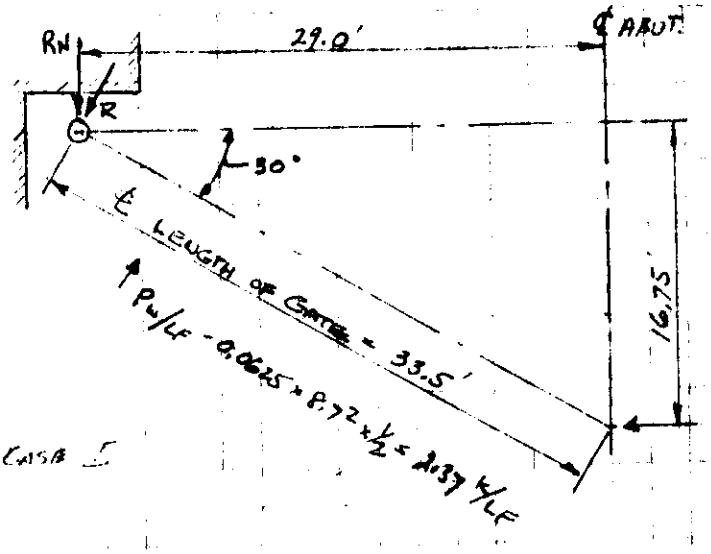
1. CONCRETE
2. GATES
3. EARTH
4. WATER (GRAVITY)
5. WATER (LATERAL)
6. UPLIFT
7. WIND
8. TRAFFIC SURCHARGE

STABILITY ANALYSIS - ONLY THE PORTION OF ABUTMENT WITH FULL BASE SEAL WILL BE CONSIDERED AS RESISTING GATE LOADS. THE PORTION OF WALLS EXTENDING RIVERSIDE WILL BE SEPARATED FROM THE MAIN MONOLITH BY A CONTRACTION JOINT. THESE SECTIONS OF WALL WILL BE CHECKED AS SEPARATE MONOLITHS.

SUBJECT JACKSONVILLE LOCAL PROTECTION

COMPUTATION VEHICULAR GATE STA. 22+50 ± - ABUTMENT STABILITY

COMPUTED BY HARRINGTON, R.L. CHECKED BY JRF DATE JULY 1974

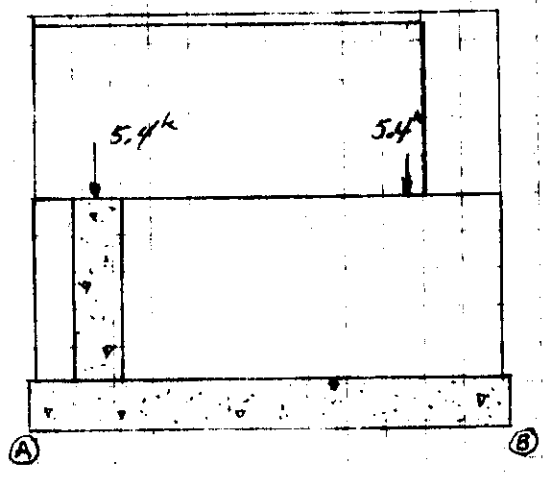


LENGTH GATE = $\frac{29.0}{0.866} = 33.49$

$R = 33.5 \times 2.37 = 79.4^k$
$R_N = 79.4 \times 0.866 = 68.8^k$ } CASE 5

WEIGHT OF GATE - FROM J. FAY. = 10.4 K/LEAF. , 20.8 K - 2 LEAFS.

GATE CLOSED
JACKED



SECTION C-C

27 Sept 49

CORPS OF ENGINEERS, U.S. ARMY

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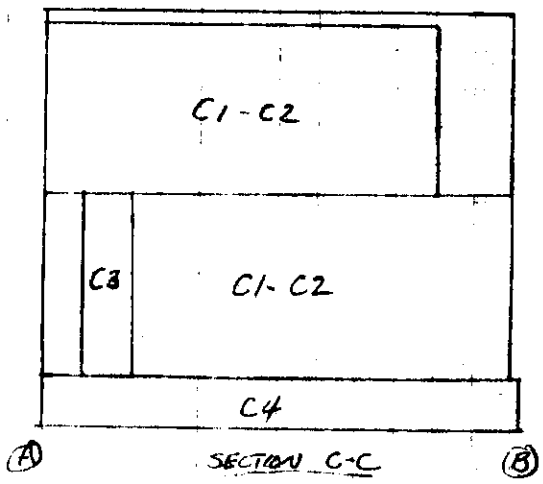
SUBJECT SAVANNAH LOCAL PROTECTION

COMPUTATION VEHICULAR GATE STA 22+50 ± ABUTMENT STABILITY

COMPUTED BY HARRINGTON, R.L. CHECKED BY JRF DATE JULY 1974

STABILITY @ TOE OF ABUTMENT (B)

1. CONCRETE LOADS.



SEC.	FACTOR	↓	↑	→	←	ARMS	↷	↶
C1	0.15 × 23.5 × 5.0 × 19.5 × 2	687.4				12.25		8421
C2	0.15 × 19.5 × 3.0 × 9.2 × 2		161.5			14.25	2301	
C3	0.15 × 32.0 × 2.0 × 9.8 × 2	188.2				13.87		2610
C4	0.15 × 75.0 × 24.0 × 2.5	675.0				12.00		8100
Σ		1550.6	161.5				2301	19131
		1389.1						16830

2. GATES

a. IN POCKETS NOT JACKED

G	10.4 × 2	20.8				21.75		452.4

b. IN POCKETS JACKED

G	5.2 × 2	10.8				5.0		54.0

c. CLOSED

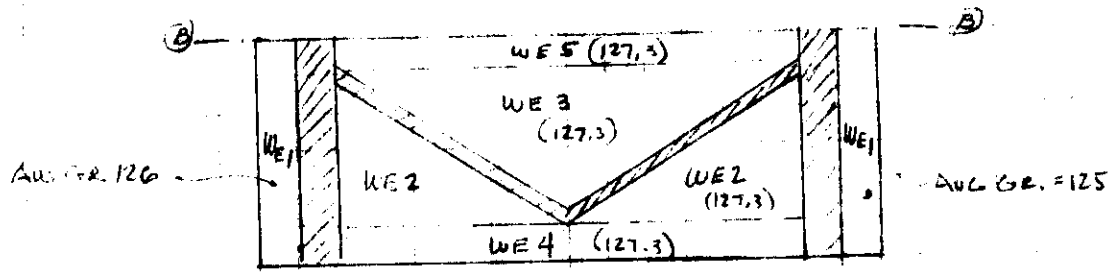
G	10.4 × 2	20.8				13.87		288.5

SUBJECT SACRAMENTO LOCAL PROTECTION

COMPUTATION VEHICULAR GATE STA 22+50± ABUTMENT STABILITY

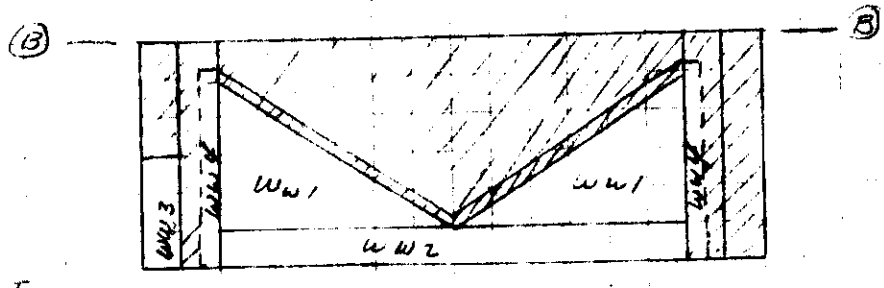
COMPUTED BY HARRINGTON, R.L. CHECKED BY JRF DATE JULY 1954

3. EARTH (GRAVITY LOADS ONLY - LATERAL ACTIVE PRESSURES BALANCE @ B-B)



SEC	FACTOR	↓	↑	→	←	ARM @ B	↷	↶
WE1	0.14 × 24 × 5.0 × 8.0 × 2	268.8				12.0		3226
WE2	0.14 × 16 × 27.5 × 9.8	603.7				17.5		10565
WE3	0.14 × 55 × 15.9 × 9.8 × ½	599.9				9.8		5879
WE4	0.14 × 55 × 1.5 × 9.8	113.2				23.25		2632
WE5	0.14 × 55 × 4.5 × 9.8	339.6				2.25		764
Σ		1925.2						23066

4. WATER (GRAVITY LOADS)



CASE I

WW1	0.0625 × 16 × 27.5 × 8.7	239.3			17.5	4188
WW2	0.0625 × 55 × 1.5 × 8.7	44.9			23.25	1044
WW3	0.0625 × 9 × 5 × 10.0	28.1			19.5	548
WW4	0.0625 × 19.5 × 3 × 8.7 × 2	63.6			14.25	906
Σ		375.9				6686

CASE II

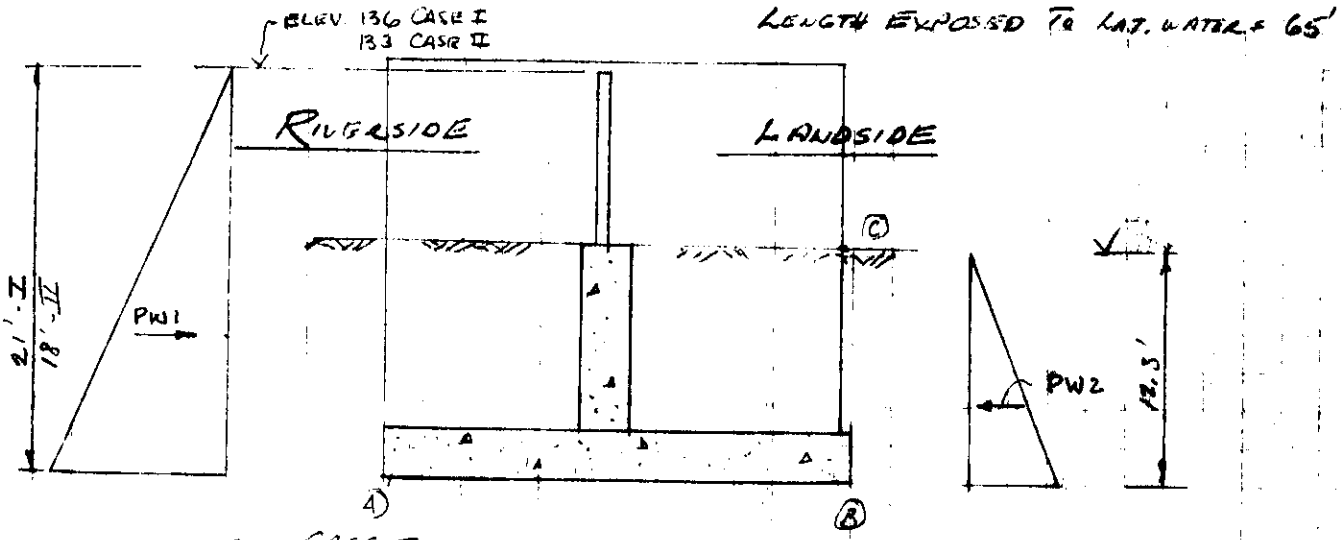
WW1	0.0625 × 16 × 27.5 × 5.7	156.8			17.5	2744
WW2	0.0625 × 55 × 1.5 × 5.7	29.4			23.25	684
WW3	0.0625 × 9 × 5 × 7.0	19.7			19.5	384
WW4	0.0625 × 19.5 × 3 × 5.7 × 2	41.7			14.25	594
Σ		247.6				4406

SUBJECT Saxonville Local Protection

COMPUTATION Vehicle Gate: Sta 22+50 Abutment Stability

COMPUTED BY Harrington, R.L. CHECKED BY JRF DATE July 1974

5. LATERAL WATER



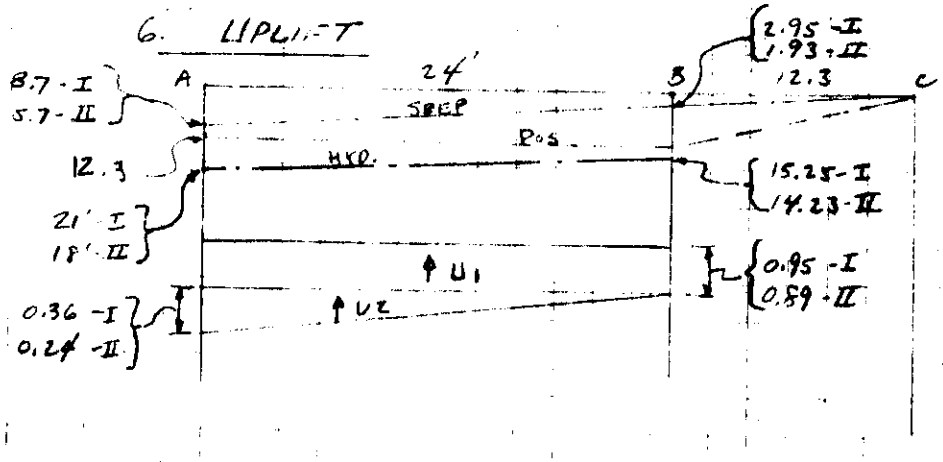
a. CASE I

Sym	FACTOR	↓	↑	→	←	ARM @ B	↻	↺
P_{W1}	$.0625 \times 21^2 \times \frac{1}{2} \times 65'$			895.8		7.0	6271	
P_{W2}	$0.95 \times 12.3 \times \frac{1}{2} \times 65$				377.8	4.1		1557
Σ				516.0			4717	

b. CASE II

P_{W1}	$.0625 \times 18^2 \times \frac{1}{2} \times 65$			658.1		6.0	3949	
P_{W2}	$0.89 \times 12.8' \times \frac{1}{2} \times 65$				355.8	4.1		1458
Σ				302.3			2490	

6. UPLIFT



$C.R.-I = \frac{36.3}{8.7} = 4.17 > 3$

$C.R.-II = \frac{36.3}{5.7} = > 3.0$

SUBJECT: SAXONVILLE LOCAL PROTECTION
COMPUTATION: VEHICULAR GATE STA 22+50 ABUTMENT STABILITY
COMPUTED BY: HARRINGTON, R.L. CHECKED BY: JRF DATE: JULY 1974

6. UPLIFT (CONTD)

a. CASE I

SEC	FACTOR	↓	↑	→	←	ARM @ B	↷	↶
U_1	0.95 × 24 × 75		1710			12.0	20520	
U_2	0.36 × 24 × ½ × 75		324			16.0	5184	
Σ			2034				25704	

b. CASE II

U_1	0.89 × 24 × 75		1602			12.0	19224	
U_2	0.24 × 24 × ½ × 75		216			16.0	3456	
Σ			1818				22680	

C. GROUNDWATER (NON FLOOD CONDITION) G.W @ ELEV. 118.0 (ASSUMED)

U_1	.0625 × 3 × 24 × 75		337			12.0	4044	
Σ			337				4044	

7. WIND LOADING

F_W	.050 × 8.7 × 65				28.3	16.65		471

~~8. TRAFFIC LOADING (EQUIV. 3' EARTH SURCH. LAND SIDE)~~
~~ASSUME 2-10 LOADS φ = 0.35 × 1.40 × 3 = 0.140~~

ΣF_E	0.140 × 12.3 × 20				34.4	6.15		211

8. TRAFFIC LOADING - 2 LANES HS-20 OVER BACK SLAB (SEE SHT 11A)

	4 × 16	64				5.0		320
	4 × 16	64				19.0		1216
ΣT		128						1536

SUBJECT: SAXONVILLE LOCAL PROTECTION

COMPUTATION: VEHICULAR GATE STA 22+50± - ABUTMENT STABILITY

COMPUTED BY: HARRINGTON, R.L. CHECKED BY: JRF DATE: JULY 1974

LOADING CASE I - FULL FLOOD

LOAD	FACTOR	↓	↑	→	←	ARM@B	↷	↶
C	SEE SHT No. 6	1389						16830
G	" " "	21						308
E	" " No. 7	1925						23066
W	" " "	376						6606
Pw	" " No. 8			516			4314	
U	" " No. 9		2034				25704	
Σ		3711	2034	516			30418	46870
		1677						16452

$\Sigma M/\Sigma V = \frac{16452}{1677} = 9.81'$ (WITHIN MID ⅓ OK) $e = 12 - 9.81 = 2.19'$

BEARING PRESSURE $= \frac{1677}{24 \times 75}\left(1 \pm \frac{6 \times 2.19}{24}\right) = 1.44$ K/SF LAND SIDE, 0.32 K/SF RIVER SIDE

F.S. SLIDING $= \frac{.466 \times 1677}{516} = 1.51$ OK

LOADING CASE II - FLOOD 3' FROM TOP

C	SEE SHT No. 6	1389						16830
G	"	21						308
E	7	1925						23066
W	"	248						4404
Pw	8			302			2490	
U	9		1818				22680	
		3583	1818	302			25170	44590
		1765						19420

$\Sigma M/\Sigma V = \frac{19420}{1765} = 11.00$ (WITHIN MID ⅓ OK) $e = 12 - 11.0 = 1.00'$

BEARING PRESSURE $= \frac{1765}{24 \times 75}\left(1 \pm \frac{6 \times 1.00}{24}\right) = 1.22$ LAND SIDE, 0.72 RIVER SIDE

SUBJECT SAVANNAH LOCAL PROTECTION

COMPUTATION VEHICULAR GATE STA 22+50 : ABUTMENT STABILITY

COMPUTED BY HARRINGTON, R.L. CHECKED BY JRB DATE JULY 1954

LOADING CASE III - TRAFFIC CONDITION (SEE SHT. 11A)

SEC	FACTOR	↓	↑	→	←			↺
C	SEE SHT. No 6	1389						16830
G	" " " "	21						452
E	" " " 7	1925						23066
T	" " " 9	128						1536
		3463						41884

$\Sigma M/\Sigma V = \frac{41884}{3463} = 12.09$ $e = 0.09$

BEARING PRESSURE $= \frac{3463}{24 \times 75}\left(1 \pm \frac{6 \times 0.09}{24}\right) = 1.97$ RIVERSIDE, 1.88 LANDSIDE

LOADING CASE IV - WIND LOADING - GATES CLOSED, NO WATER

C	SEE SHT. No. 6	1389						16830
G	"	21						289
E	7	1925						23066
WIND					28.3			471
		3335			28.3			40656

$\Sigma M/\Sigma V = \frac{40656}{3335} = 12.19$ $e = 0.19$

$B.P. = \frac{3335}{24 \times 75}\left(1 \pm \frac{6 \times 0.19}{24}\right) =$ 1.94 KSF RIVER SIDE, 1.76 KSF LAND SIDE

SUMMARY STABILITY ANALYSIS

LOADING CASE	e	B.P. LANDSIDE	B.P. RIVERSIDE
I	2.18	1.44 KSF	0.42 KSF
II	1.00	1.23	0.74
III	0.16	1.88	1.97
IV	0.19	1.76	1.94

27 Sept 49

CORPS OF ENGINEERS, U. S. ARMY

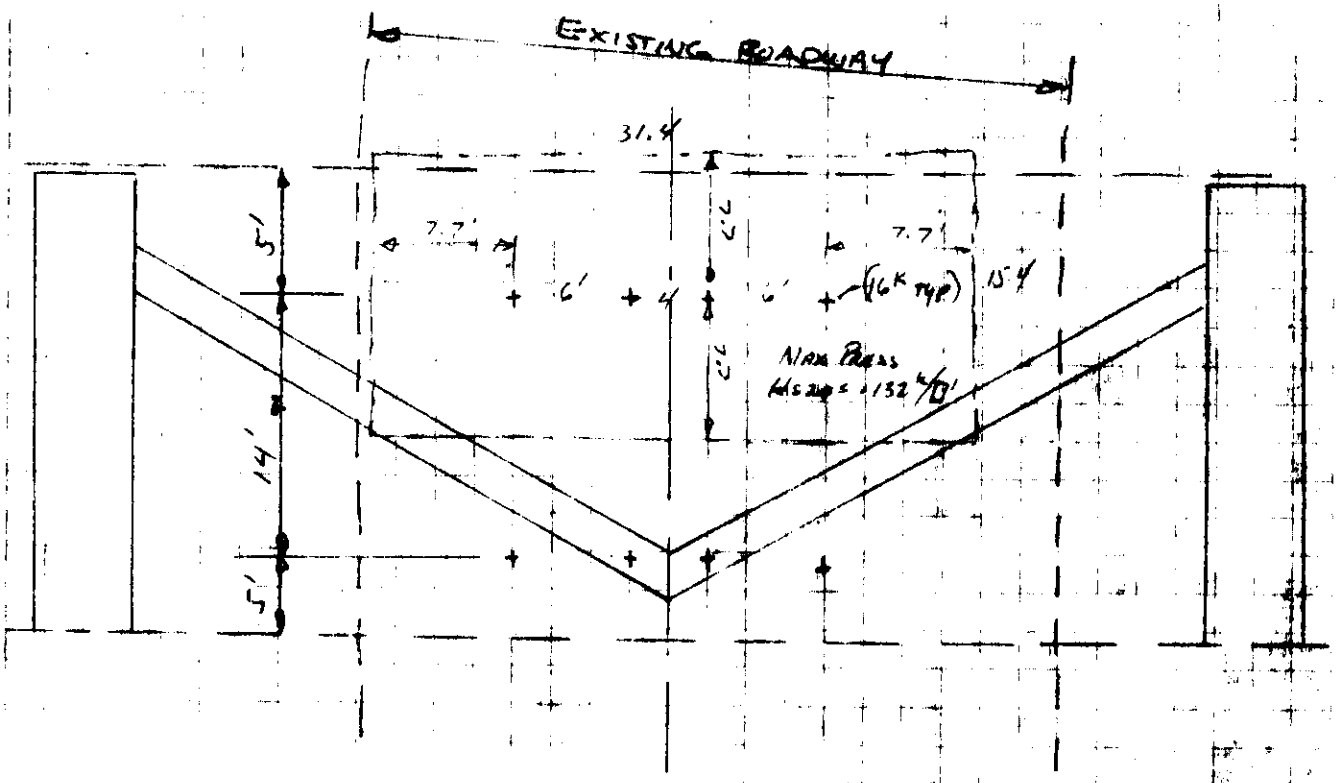
PAGE [illegible]

SUBJECT SAXONVILLE LOCAL PROTECTION

COMPUTATION VEHICULAR GATE STA 22+50± ABUTMENT STABILITY

COMPUTED BY RLH CHECKED BY JRF DATE JULY 19[illegible]

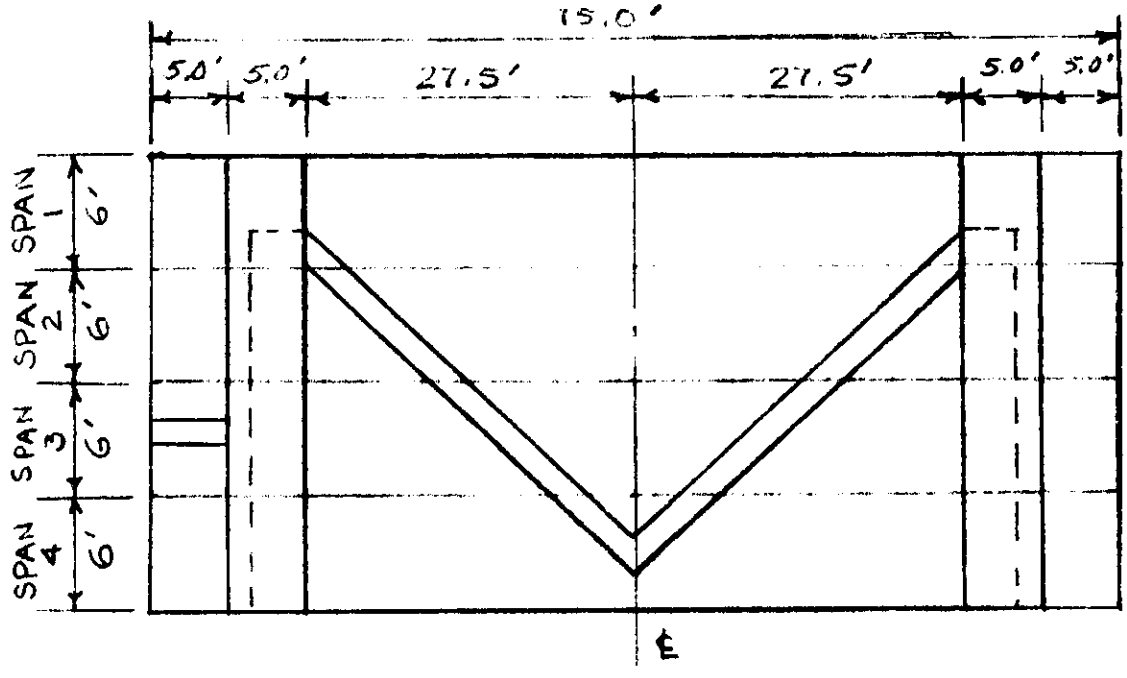
CASE III - 2 LANES OF HS-20 LOADING ON GATE MONOLITH BASE



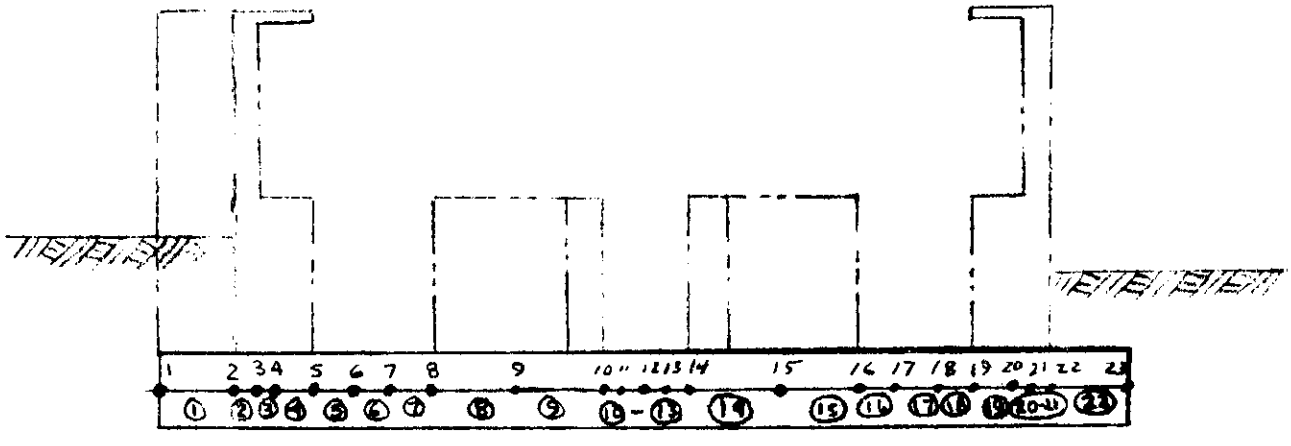
SUBJECT SAXONVILLE

COMPUTATION STREET GATE - BASE SLAB ANALYSIS

COMPUTED BY JRF CHECKED BY DATE AUG 1974



PLAN
N.T.S.



COMPUTER MODEL
SPAN 3

LOADING 1 - STORM CONDITION (STABILITY ANALYSIS)
CASE 1 - UNBALANCED LOAD DISTRIBUTED PROPORTIONALY TO THE APPLIED LOADS (NOT CRITICAL)
CASE 2 - UNBALANCED LOAD DISTRIBUTED AS A UNIFORM LOAD. (COMPUTER ANALYSIS TO FOLLOW)

SUBJECT SAXONVILLE
COMPUTATION STREET GATE - BASE SLAB ANALYSIS - LOADING 1 CASE 2
COMPUTED BY JRF CHECKED BY DATE AUG. 1974

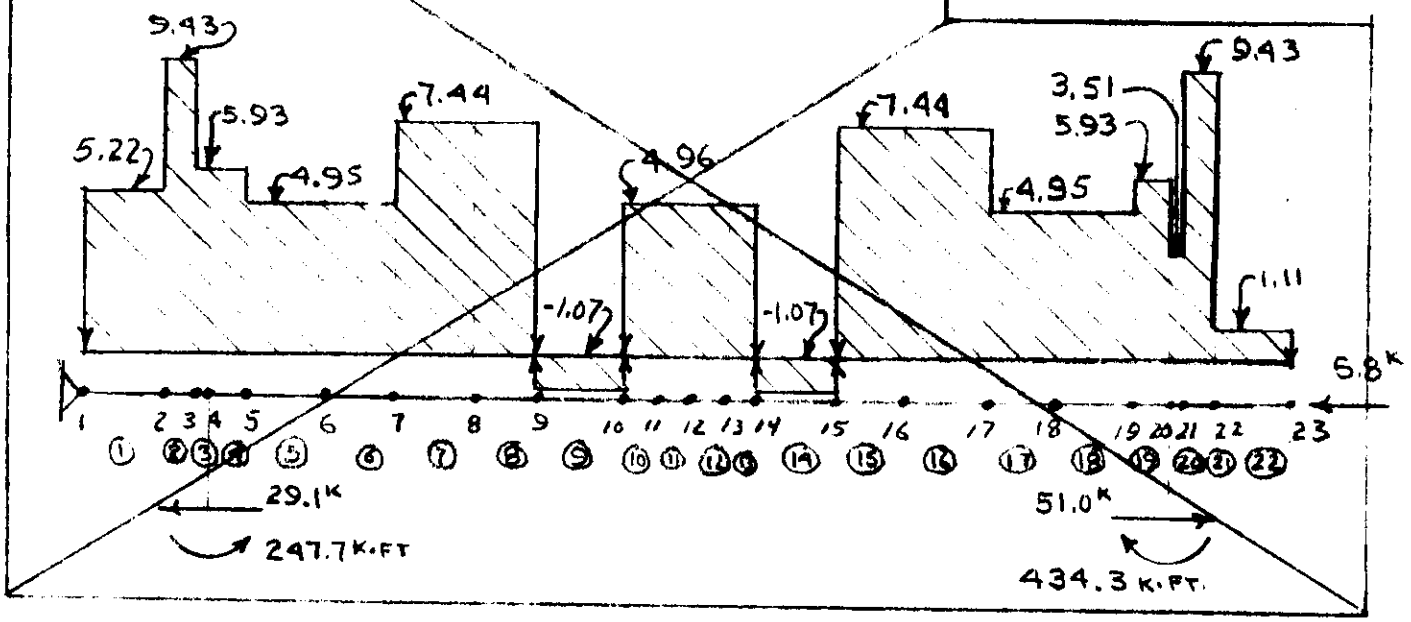
LOADING DIAGRAM

CASE 1

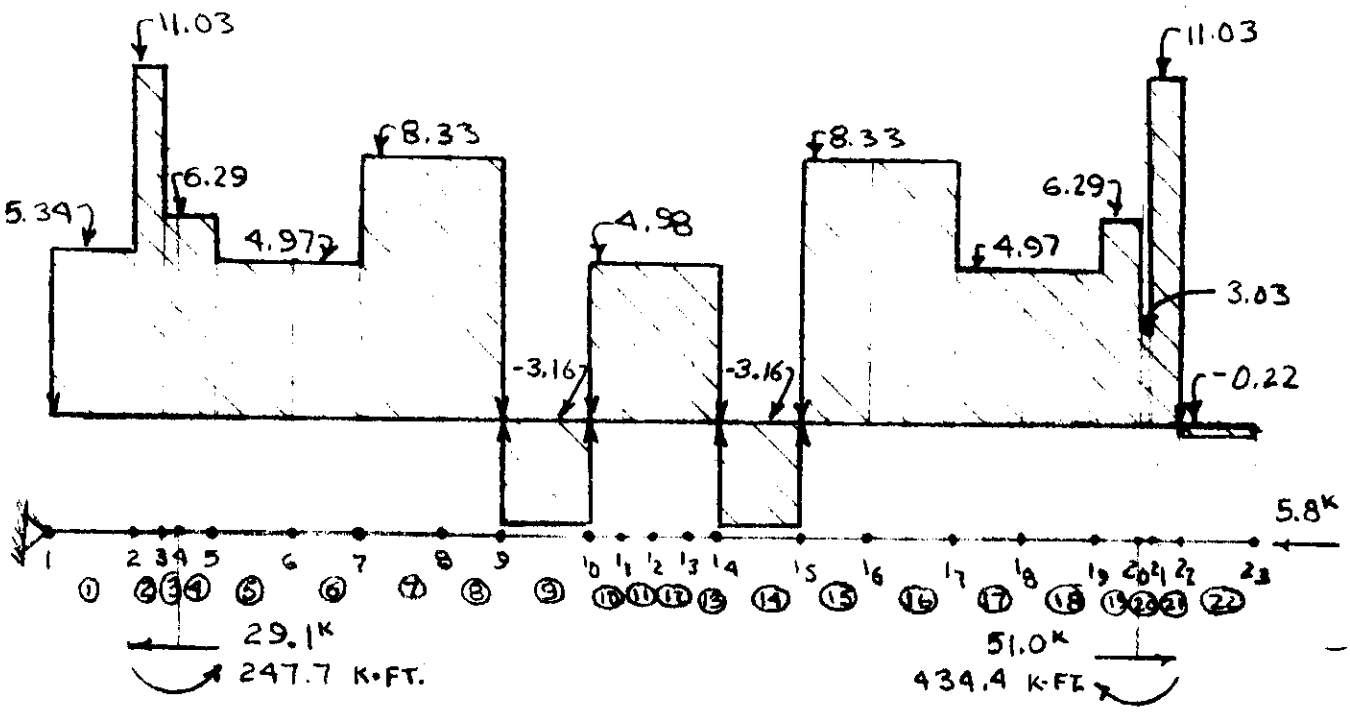
NOT CRITICAL

(n) — ELEMENT
n — NODE

LOADS
DEAD LOAD
EARTH
WATER
UPLIFT
UNBALANCED LD.



CASE 2



** DATE 08/21/74
** TIME 11:35:45
SECTION 1

SAXONVILLE, CONCORD STREET FLOOD GATE

SPAN 3 (6 FT. SECTION) BASE SLAB

LOADING 1- STORM CONDITION

NUMBER OF NODES = 23
NUMBER OF ELEMENTS = 22
NUMBER OF LOADINGS = 2

(RESTRAINT CODE* =1 SPECIFY RESTRAINT, =0 SPECIFY LOAD)

NODE	RESTRAINT CODE	X-COOR	Y-COOR	SPECIFIED DISPLACEMENTS X	Y	ROT
1	100	0.	0.	0.	0.	0.
2	0	5.00	0.	0.	0.	0.
3	0	7.00	0.	0.	0.	0.
4	0	7.50	0.	0.	0.	0.
5	0	10.00	0.	0.	0.	0.
6	0	15.00	0.	0.	0.	0.
7	0	19.00	0.	0.	0.	0.
8	0	24.20	0.	0.	0.	0.
9	0	28.20	0.	0.	0.	0.
10	0	33.50	0.	0.	0.	0.
11	0	35.50	0.	0.	0.	0.
12	0	37.50	0.	0.	0.	0.
13	0	39.50	0.	0.	0.	0.
14	0	41.50	0.	0.	0.	0.
15	0	46.30	0.	0.	0.	0.
16	0	50.30	0.	0.	0.	0.
17	0	56.00	0.	0.	0.	0.
18	0	60.00	0.	0.	0.	0.
19	0	65.00	0.	0.	0.	0.
20	0	67.50	0.	0.	0.	0.
21	0	68.00	0.	0.	0.	0.
22	0	70.00	0.	0.	0.	0.
23	0	75.00	0.	0.	0.	0.

1

** DATE 08/21/74
** TIME 11:35:45
** SECTION 1
** LOADING 2

CASE 2- UNBALANCED LOAD DISTRIBUTED UNIFORMLY

NUMBER OF NODAL POINT LOAD CARDS = 3

SPECIFIED NODAL POINT LOADS

NODE	FX	FY	MOM
4	-29.100	0.	247.700
20	51.000	0.	-434.300
23	-5.850	0.	0.

ELEMENT LOADS

LOAD TYPE * =1 CONCENTRATED LOAD
=2 UNIFORM DISTRIBUTED LOAD
=3 CONCENTRATED MOMENT
=4 TRIANGULAR DISTRIBUTED LOAD

DIST FR I * DISTANCE FROM NODE I, LOAD TYPES 1 AND 3 ONLY

END CODE * ONLY FOR LOAD TYPE 4 **
=0 NODE I WEIGHTED
=1 NODE J WEIGHTED

SUBJECT SAXONVILLE

COMPUTATION STREET GATE - BASE SLAB ANALYSIS, LOADING 1, CASE 2

COMPUTED BY JRF CHECKED BY DATE AUG. 1974

ELEMENT	NODE I	NODE J	LOAD TYPE	MAGNITUDE OF LOAD	DIST FR I /END CODE	DIRC-CODE X=2 / Y=1
1	1	2	2	-5.340	0.	1
2	2	3	2	-11.030	0.	1
3	3	4	2	-6.290	0.	1
4	4	5	2	-6.290	0.	1
5	5	6	2	-4.970	0.	1
6	6	7	2	-4.970	0.	1
7	7	8	2	-8.330	0.	1
8	8	9	2	-8.330	0.	1
9	9	10	2	3.160	0.	1
10	10	11	2	-4.980	0.	1
11	11	12	2	-4.980	0.	1
12	12	13	2	-4.980	0.	1
13	13	14	2	-4.980	0.	1
14	14	15	2	3.160	0.	1
15	15	16	2	-8.330	0.	1
16	16	17	2	-8.330	0.	1
17	17	18	2	-4.970	0.	1
18	18	19	2	-4.970	0.	1
19	19	20	2	-6.290	0.	1
20	20	21	2	-3.030	0.	1
21	21	22	2	-11.030	0.	1
22	22	23	2	.220	0.	1

1

** DATE 08/21/74
** TIME 11:35:45
** SECTION 1
** LOADING 2

ELEMENTS ON AN ELASTIC FOUNDATION

ELEMENT	SPRING I	PRESSURE J
1	.763077E+01	.686445E+01
2	.686445E+01	.655557E+01
3	.655557E+01	.647795E+01
4	.647795E+01	.610169E+01
5	.610169E+01	.542173E+01
6	.542173E+01	.494331E+01
7	.494331E+01	.439933E+01
8	.439988E+01	.404895E+01

9	.404395E+01	.371569E+01
10	.371569E+01	.363578E+01
11	.363578E+01	.358042E+01
12	.358042E+01	.354914E+01
13	.354914E+01	.354136E+01
14	.354136E+01	.363774E+01
15	.363774E+01	.380019E+01
16	.380019E+01	.408496E+01
17	.408496E+01	.435709E+01
18	.435709E+01	.478202E+01
19	.478202E+01	.503887E+01
20	.503887E+01	.509335E+01
21	.509335E+01	.530616E+01
22	.530616E+01	.532111E+01

NUMBER OF ITERATIONS = -1

ELEMENT	ZONE	LIMITS OF INTEGRATION (MAX=11)		
1	ODD	0.	,	5.000,
2	ODD	0.	,	2.000,
3	ODD	0.	,	.500,
4	ODD	0.	,	2.500,
5	ODD	0.	,	5.000,
6	ODD	0.	,	4.000,
7	ODD	0.	,	5.200,
8	ODD	0.	,	4.000,
9	ODD	0.	,	5.300,
10	ODD	0.	,	2.000,
11	ODD	0.	,	2.000,
12	ODD	0.	,	2.000,
13	ODD	0.	,	2.000,
14	ODD	0.	,	5.200,
15	ODD	0.	,	4.000,
16	ODD	0.	,	5.200,
17	ODD	0.	,	4.000,
18	ODD	0.	,	5.000,
19	ODD	0.	,	2.500,
20	ODD	0.	,	.500,
21	ODD	0.	,	2.000,
22	ODD	0.	,	5.000,

SUBJECT SAXONVILLE
COMPUTATION STREET GATE - BASE SLAB ANALYSIS, LOADING 1, CASE 2
COMPUTED BY JRF CHECKED BY DATE AUG. 1974

END ACTIONS WITH ELEMENT LOADS

ELEMENT	AXIAL I AXIAL J	SHEAR I SHEAR J	MOMENT I MOMENT J
1	-16.050	.000	.000
	16.050	-9.540	25.445
2	-16.050	9.557	-25.447
	16.050	-.392	35.995
3	-16.050	1.156	-35.813
	16.050	-1.011	36.605
4	-45.150	.720	211.122
	45.150	-.708	-209.139
5	-45.150	.711	209.139
	45.150	-4.629	-194.373
6	-45.150	4.630	194.373
	45.150	-5.461	-173.555
7	-45.150	5.460	173.556
	45.150	13.602	-193.498
8	-45.150	-13.601	193.502
	45.150	30.047	-280.330
9	-45.150	-30.047	280.330
	45.150	-7.201	-340.093
10	-45.150	7.207	340.097
	45.150	-4.586	-328.275
11	-45.150	4.591	328.279
	45.150	-1.842	-321.832
12	-45.150	1.845	321.834
	45.150	.994	-320.975
13	-45.150	-.991	320.974
	45.150	3.870	-325.834
14	-45.150	-3.870	325.826
	45.150	-31.833	-251.948
15	-45.150	31.834	251.948
	45.150	-13.369	-161.759
16	-45.150	13.369	161.760
	45.150	9.478	-152.284
17	-45.150	-9.478	152.284
	45.150	12.492	-196.586
18	-45.150	-12.491	196.585
	45.150	14.539	-265.045
19	-45.150	-14.541	265.044
	45.150	17.999	-305.849
20	5.850	-18.141	-128.358
	-5.850	17.406	119.471
21	5.850	-17.255	-119.621
	-5.850	28.920	73.372
22	5.850	-28.922	-73.377
	-5.850	.000	-.001

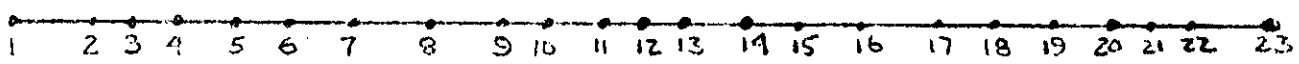
SUBJECT SAXONVILLE STREET GATE - BASE SLAB ANALYSIS, LOADING 1, CASE 2

COMPUTATION

COMPUTED BY JRF CHECKED BY DATE AUG. 1974

Summary Of Computer Results

SPAN #3 6 FT. SECTION



NODE		LOADING 1 ~~CASE 1~~	LOADING 1 CASE 2
1	P	-16.0	-16.0
	V	0.0	0.0
	M	0.0	0.0
2	P	-16.0	-16.0
	V	8.2	9.6
	M	21.9	25.4
3	P	-16.0	-16.0
	V	2.1	1.2
	M	32.3	35.8
4	P	-45.2	-45.2
	V	2.0	0.7
	M	-214.3	-211.1
5	P	-45.2	-45.2
	V	2.2	0.7
	M	-208.9	-209.1
6	P	-45.2	-45.2
	V	5.2	4.6
	M	-189.3	-194.9
7	P	-45.2	-45.2
	V	6.7	5.5
	M	-167.0	-173.6
8	P	-45.2	-45.2
	V	-8.9	-13.6
	M	-174.5	-193.5
9	P	-45.2	-45.2
	V	-21.6	-30.0
	M	-235.2	-280.3
10	P	-45.2	-45.2
	V	5.2	7.2
	M	-277.5	-340.1
11	P	-45.2	-45.2
	V	3.0	4.6
	M	-269.7	-328.3
12	P	-45.2	-45.2
	V	0.6	1.8
	M	-266.2	-321.8

NODE		LOADING 1 ~~CASE 1~~	LOADING 1 CASE 2
13	P	-45.2	-45.2
	V	-1.8	-1.0
	M	-267.4	-320.9
14	P	-45.2	-45.2
	V	-4.3	-3.9
	M	-273.4	-325.8
15	P	-45.2	-45.2
	V	21.4	31.8
	M	-228.3	-251.3
16	P	-45.2	-45.2
	V	7.2	15.4
	M	-171.4	-161.8
17	P	-45.2	-45.2
	V	-10.2	-9.5
	M	-179.9	-157.3
18	P	-45.2	-45.2
	V	-12.5	-12.5
	M	-225.6	-196.6
19	P	-45.2	-45.2
	V	-13.6	-14.5
	M	-291.8	-265.0
20	P	-45.2	-45.2
	V	-15.8	16.0
	M	-328.7	-305.8
21	P	5.8	5.8
	V	-15.2	-17.3
	M	97.9	119.6
22	P	5.8	5.8
	V	-23.3	-28.9
	M	59.3	+73.4
23	P	5.8	5.8
	V	0.0	0.0
	M	0.0	0.0

(+) AXIAL (P) ≡ COMPRESSION
(+) SHEAR (V) ≡ +Y
(+) MOM. (M) ≡ TENSION BOTTOM

SUBJECT SAXONVILLE
COMPUTATION STREET GATE - BASE SLAB ANALYSIS, LOADING 1, CASE 2
COMPUTED BY JRF CHECKED BY DATE AUG. 1974

SPAN # 3 (TYPICAL CALCULATION - NOT CONTROLLING CASE)
LOADING 1

MAXIMUM Moment (+) = 119.6 (NODE 21, CASE 2)
MAXIMUM Moment (-) = 305.8 (NODE 20, CASE 2)
MAXIMUM Shear = 31.8 (NODE 15, CASE 2)

b = 6 FT.

℄ BASE

d = 25.5"

P = -45.2 K
V = 1.8 K
M = -321.8 K·FT

$M_c = d^2 bK$
$M_c = (25.5)^2 \times 6 \times .152$
$M_c = 593.0$ K·FT. > 321.8

$M_s = M - \frac{Px}{12}$

$M_s = 321.8 - 45.2 \times \frac{10.5}{12}$

$M_s = 282.2$

$A_s = \frac{M_s}{ad} + \frac{P}{f_s}$

$A_s = \frac{282.2}{1.44 \times 25.5} + \frac{45.2}{20}$

$A_s = 9.95$ in² (6 FT. SECT.)

$A_s = 1.67$ in²/FT.

FACE OF ABUTMENTS

d = 25.5"

P = -45.2
V = -14.6
M = -265.8
} NODE 19

$M_c = 593.0$ K·FT.

$M_f = 305.8 - \frac{1}{3}(18.0)(5)$

$M_f = 275.8$ K·FT. ← USE

$M_s = 275.8 - 45.2 \times \frac{10.5}{12}$

$M_s = 236.2$ K·FT.

$A_s = \frac{236.2}{1.44 \times 25.5} + \frac{45.2}{20}$

$A_s = 8.69$ in² (6 FT. SECTION)

$A_s = 1.45$ in² (1 FT. SECTION)

MAXIMUM POSITIVE MOMENT (TYPICAL CALC. - NOT CONTROLLING CASE)

$d = 23.5''$

$P = 5.8$ K
$V = -17.3$ K
$M = 119.6$ K·FT

$M_c = 23.5^2 \times 6 \times .152$
$M_c = 503.6$ K·FT. > 119.6 ∴ OK

$M_s = 119.6 + 5.8 \times \frac{8.5}{12}$

$M_s = 123.7$

$A_s = \frac{123.7}{1.44 \times 8.5} - \frac{5.8}{20}$

$A_s = 9.82$ in² (6 FT. SECTION)

$A_s = 1.64$ in² (1 FT. SECTION)

MAXIMUM SHEAR STRESS

$V_{max} = 31.8$ K

$b = 6 \times 12 = 72''$

$d = 23.5''$

$v = \frac{31800}{72 \times 23.5}$

$v = 18.8$ psi < 60.0 psi ∴ OK

NED FORM 223
27 Sept 49

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

SUBJECT SAXONVILLE
COMPUTATION STREET GATE - BASE SLAB REINFORCING
COMPUTED BY JRF CHECKED BY DATE AUG. 1974

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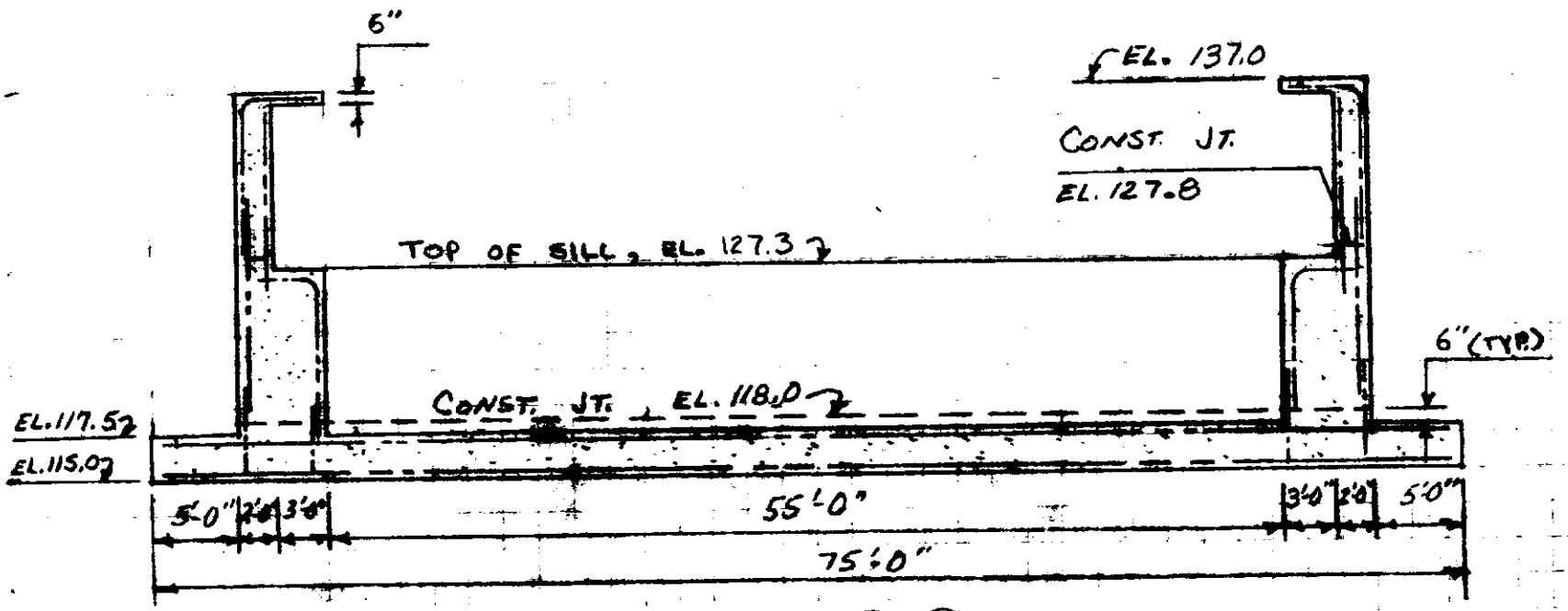
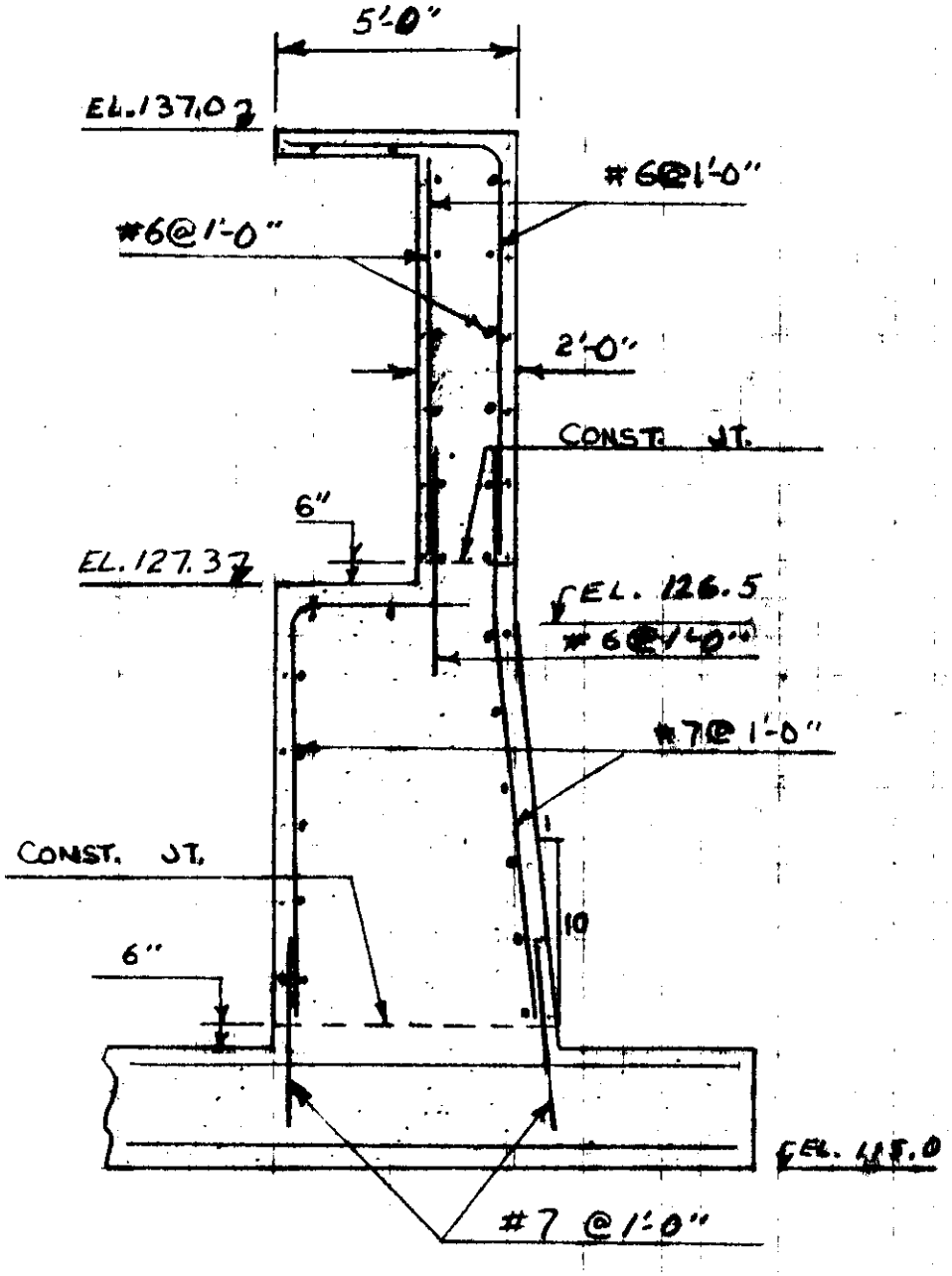
NOTE: TYPICAL ARRANGEMENT OF REINF. STEEL SHOWN.
SIZES TO BE DETERMINED IN DETAIL DESIGN



SECTION (A)-(A)
SCALE: 1" = 10'-0"

SUBJECT SAXONVILLE
COMPUTATION STREET GATE- WALL REINFORCING
COMPUTED BY VRF CHECKED BY DATE AUG. 1974

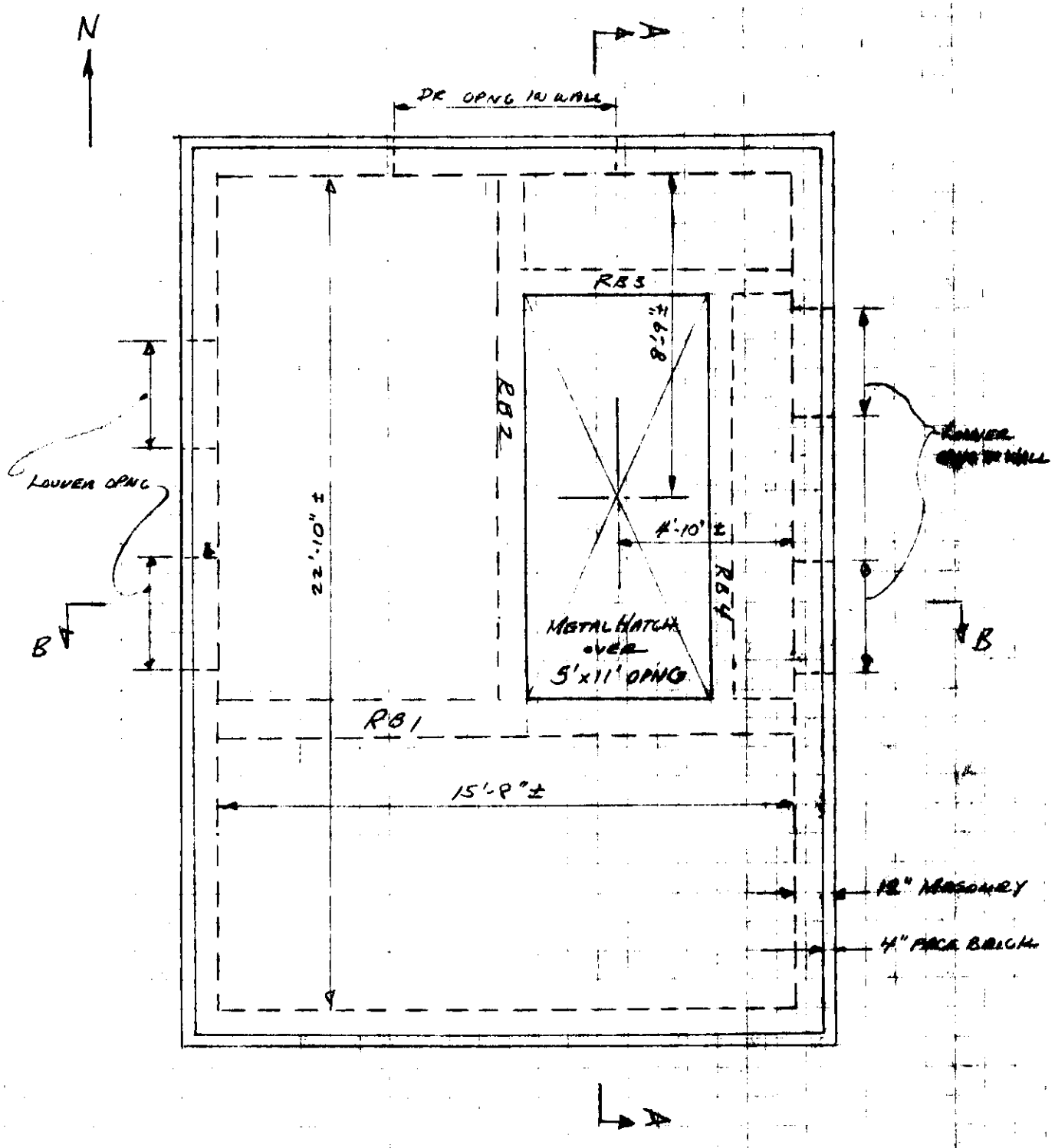


SECTION Ⓐ Ⓐ
SCALE: 1" = 4'-0"

SUBJECT SAXONVILLE LOCAL PROTECTION

COMPUTATION PUMPING STATION - STABILITY

COMPUTED BY HARRINGTON CHECKED BY JRF DATE JULY 1974



ROOF PLAN

1/4" = 1'-0"

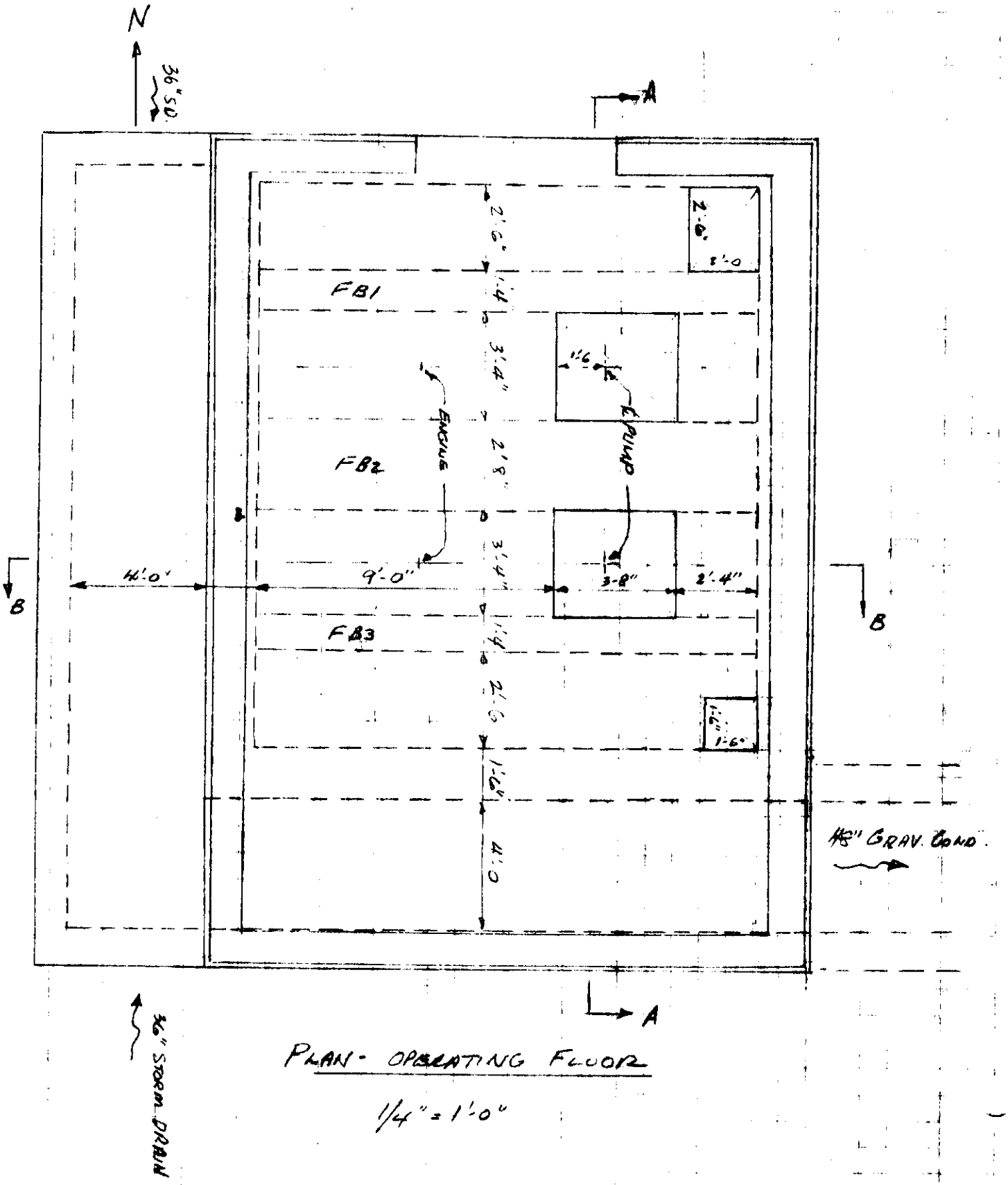
27 Sept 49

CORPS OF ENGINEERS, U. S. ARMY

SUBJECT SAXONVILLE LOCAL PROTECTION

COMPUTATION PUMPING STATION - PRELIMINARY

COMPUTED BY HARRINGTON CHECKED BY JRF DATE JULY 1974



PLAN - OPERATING FLOOR

1/4" = 1'-0"

27 Sept 49

CORPS OF ENGINEERS, U. S. ARMY

SUBJECT Saxonville Locks Pratt Section

COMPUTATION Pumping Station - Preliminary

COMPUTED BY [illegible] CHECKED BY JRF DATE July 1976

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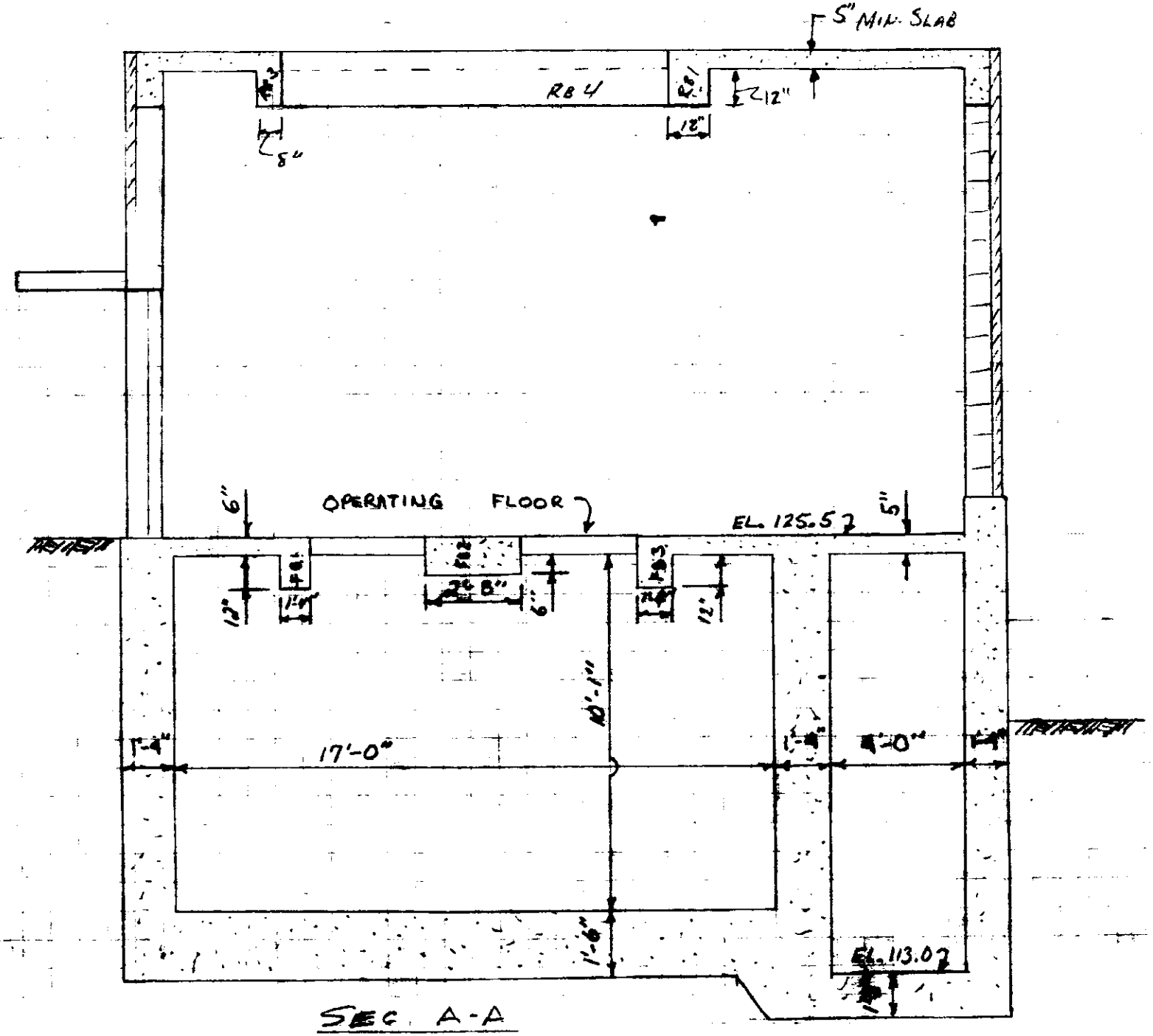
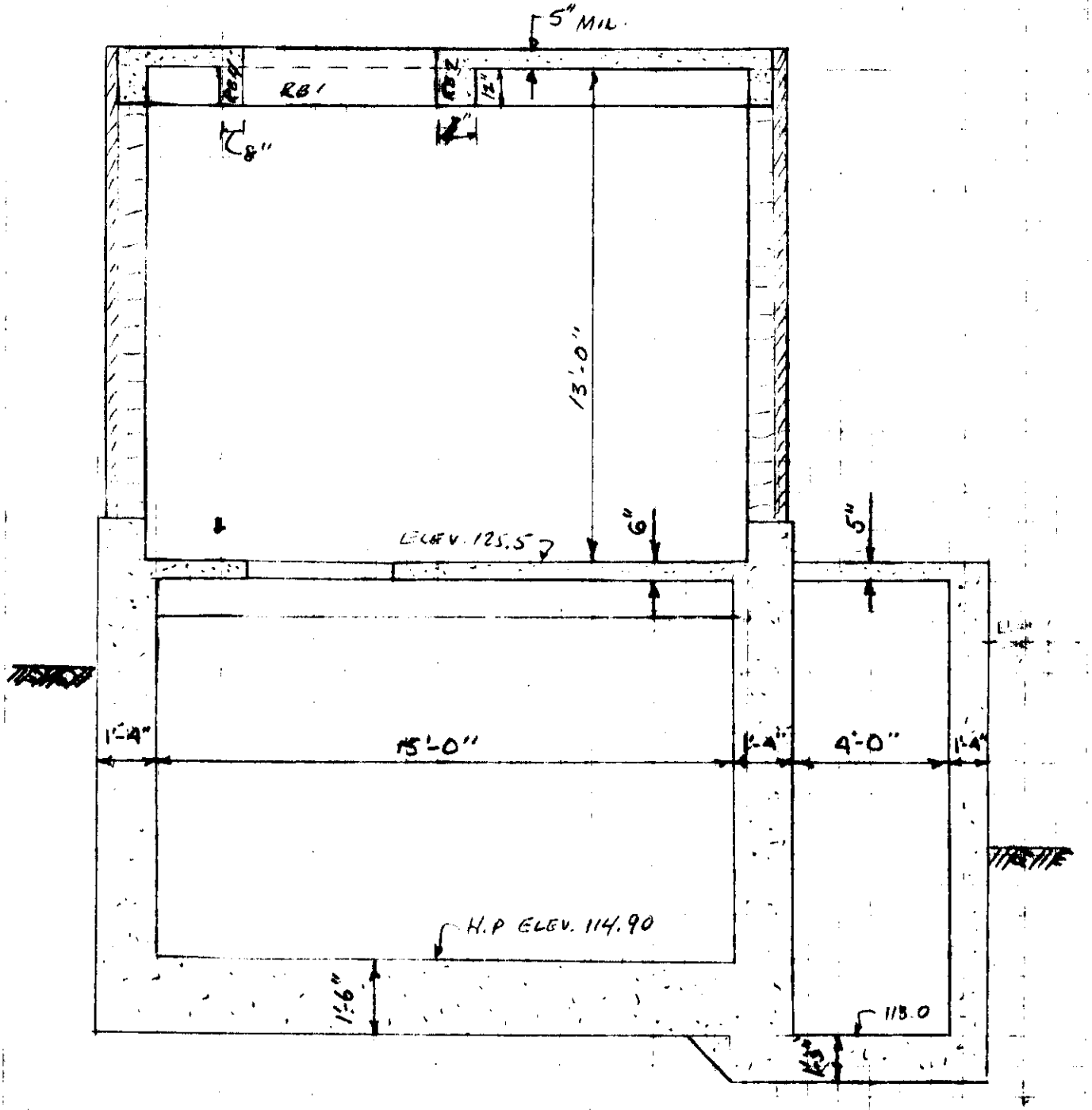


SEC. A-A

SUBJECT SAXONVILLE LOCAL PROTECTION

COMPUTATION PUMPING STATION - PRELIMINARY

COMPUTED BY HARRINGTON CHECKED BY JRF DATE July 1974



SECTION B-B

1/4" = 1'-0"

SUBJECT SAXONVILLE
COMPUTATION PUMPING STATION - ROOF
COMPUTED BY JRF CHECKED BY JAC DATE AUG. 1974

CRITERIA

CONCRETE $f'c = 3000$ psi
EXTREME FIBER STRESS IN COMPR., $f_c = 1350$ psi
Allowable $v = 60$ psi
$n = 9.2$

Min. COVER FOR REINF. = 3/4"

STEEL REINF. $f_s = 20{,}000$ psi

DESIGN

REF. ACI BUILDING CODE 318-63, Ch. 20, § 2002 & APPENDIX A, METHOD 2

Minimum thickness of slab, § 2002 (e)

(1) 3 1/2"
(2) $(2 \times 15.67 + 2 \times 7.50)\, 12/180 = 3.1$ in

MIN. THICKNESS = 3 1/2"

SUBJECT SAXONVILLE LOCAL PROTECTION

COMPUTATION PUMPING STA. PRELIM. DESIGN

COMPUTED BY RLH CHECKED BY JF DATE JULY 1974

ROOF MAX. SPAN = 8' ±

LOADS - ROOFING - 10 #/SF
CONC. EST. - 100
SNOW - 30
140 #/SF

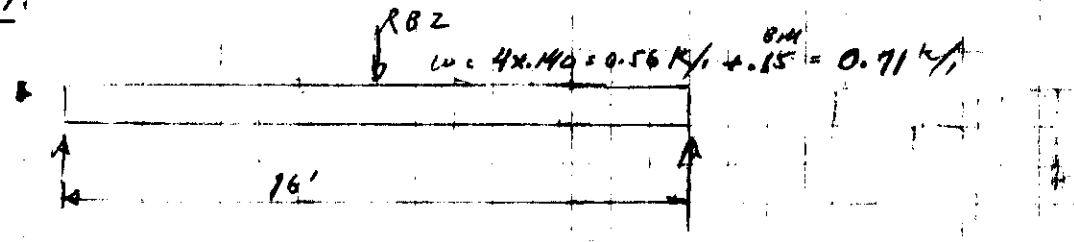
$M_{MAX} = \frac{wl^2}{8} = \frac{.140 \times 8^2}{8} = 1.12^{k/'}$ ✓

$d_{MIN} = \sqrt{\frac{1.12 \times 1000}{236}} = 2.17$ ✓

$T_{MIN} = 2.17 + 0.75 + 0.5 = 3.42$ SAY 5" MIN. d = 4" ±

$A_s = \frac{1.12}{1.44 \times 4} = 0.19$ - MIN REINF.

RB 1.



ASSUMES ALL AREA NORTH OF RB 1 CONNECTS TO RB1 VIA RB2

LOAD FROM RB2 = 8 × .140 + 0.150 = 1.27 K/FT

× 7.5' = 9.52 K ✓

$M_{RB1} = \frac{9.52 \times 16}{4} + \frac{0.71 \times 16^2}{8} = 38.1 + 22.7 = 60.8^{k}$ ✓

IF b = 12" $d_{MIN} = \sqrt{\frac{60.8 \times 1000}{236}} = 16.05$

$V = 0.71 \times 8 + 9.52/2 = 10.46$

$T_{MIN} = 16 + 1.5 + 0.5 = 18"$ SAY 18"

$A_s = \frac{60.8}{1.44 \times 16} = 2.63$ ✓ $v = \frac{10,460}{12 \times 7/8 \times 16} = 62$ PSI ✓

IF b = 18" $d_{MIN} = 13.1$ $T_{MIN} = 13.1 + 1.5 + .5 = 15.6$ SAY 16"

USE - 12" BM DROP BELOW BOT. OF SLAB.
MORE RIGID ANALYSIS WILL SHOW T = 12" OK.
b = 12"

SUBJECT SAXONVILLE

COMPUTATION PUMPING STATION - OPERATING FLOOR (2)

COMPUTED BY JRF CHECKED BY JAC DATE Aug. 1974

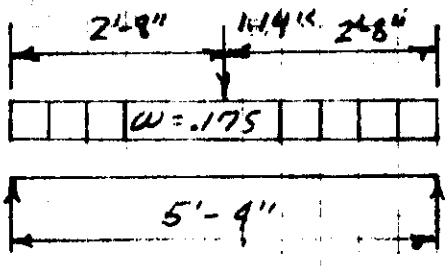
FLOOR SLAB DESIGN

SPAN 2 has the largest c.-c. distance ∴ USE SPAN 2 for the design.

$L_{c.c.} = 3'-4'' + \frac{(1'-4'')}{2} + \frac{(2'-8'')}{2} = 5'-4''$

ASSUME 6" SLAB ∴ $w_{DL} = \frac{6}{12} \times 150\ lb/ft^3 = .075\ K/FT^2$

$f'_c = 3000$ psi
$f_c = 1350$ psi



ENGINE	- 2.5K
GEAR BOX	- 0.8K
Hyd. THRUST	- 2.4K
	5.7K

$\frac{5.7^K}{5.0'} = 1.14^K$

$M_{MAX} = \frac{wl^2}{8} + \frac{Pl}{4}$ MIN. COVER = 1.5"

$M_{MAX.} = \frac{.175 \times 5.33^2}{8} + \frac{1.14 \times 5.33}{4}$

$M_{MAX} = 2.14$ K·FT.

$d = \sqrt{\frac{2.14}{.225}}$

$d = 3.08''$

6" Slab - 2" COVER, d = 4" > 3.08" ∴ USE 6" SLAB

$A_s = \frac{2.14}{1.45 \times 4} = 0.37\ in.^2$ USE #6@12"

$V = \frac{1.14}{2} + 2.67 \times .175 = 1.04^K$

$v = \frac{1040}{12 \times 4} = 21.7$ psi < 60 psi ∴ OK

SUBJECT SAXONVILLE

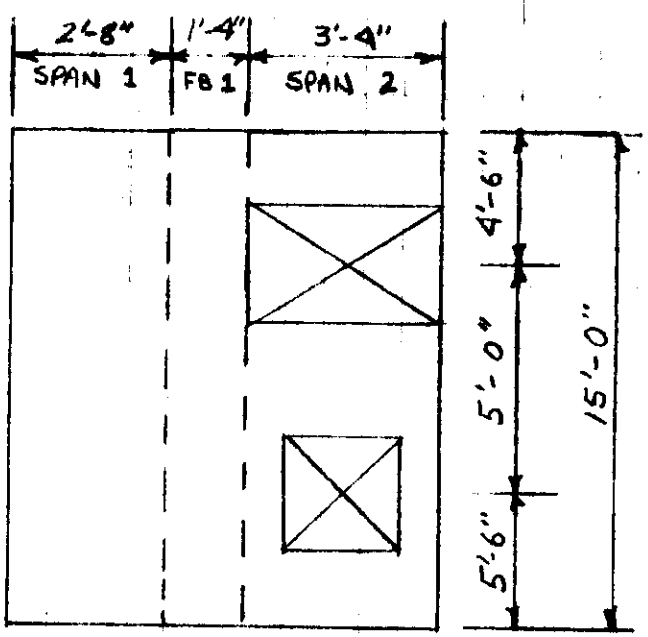
COMPUTATION PUMPING STATION - OPERATING FLOOR (3)

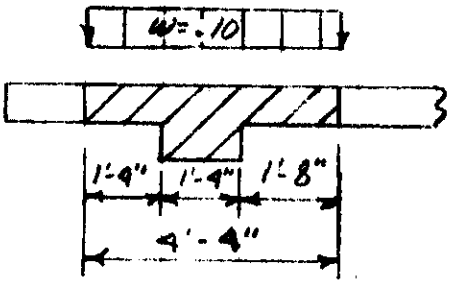
COMPUTED BY JRF CHECKED BY JAC DATE AUG. 1971

FB 2

LOADING 1 - PUMPS IN PLACE & OPERATING
LOADING 2 - PUMPS IN PLACE, ENGINE ON ℄ OF BEAM
LOADING 3 - PUMPS & ENGINE @ CENTER OF BEAM

EACH BEAM WILL TAKE ½ THE WT. OF THE Adjacent SLABS AND ½ THE LOAD OF THE PUMPS & ENGINES IN THERE OPERATING POSITION.





$w_1 = .075 \times 1.33 = .10$ K/FT

$w_2 = .075 \times 1.67 = .13$ K/FT

$w_{FB1} = .150 \times 1.5' \times 1.33 = .30$ K/FT

$w_{D.L} = 0.53$ K/FT.

SUBJECT SAXONVILLE

COMPUTATION PUMPING STATION - OPERATING FLOOR (4)

COMPUTED BY URF CHECKED BY JAC DATE Aug. 19A

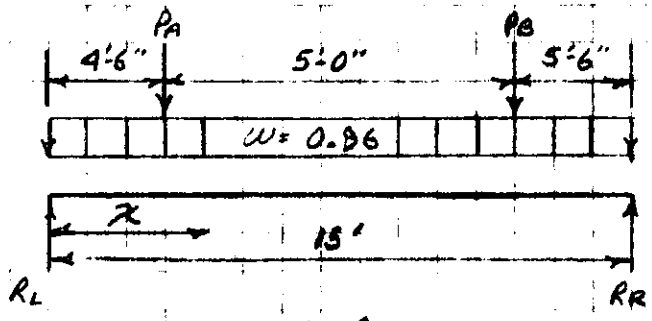
LOADING 1

$P_A = (3.0 - 2.4) \div 2 = 0.3^K$

$P_B = (3.3 + 2.4) \div 2 = 2.85^K$

$w = 0.53 + .10 \times 4.33$

$w = 0.96$



LOADING 1

$V = 0$ @ x

$R_L = (5.5 \times 2.85 + 10.5 \times 0.3 + \frac{15^2}{2} \times .096) \div 15$

$R_L = 8.455$

$R_R = (0.3 + 2.85 + .96 \times 15) - 8.455 = 9.095$

$x = (8.455 - 0.3) \div 0.96$

$x = 8.49'$

$M_x = 8.49 \times 8.455 - 0.96 \times \frac{8.49^2}{2} - 3.99 \times 0.3$

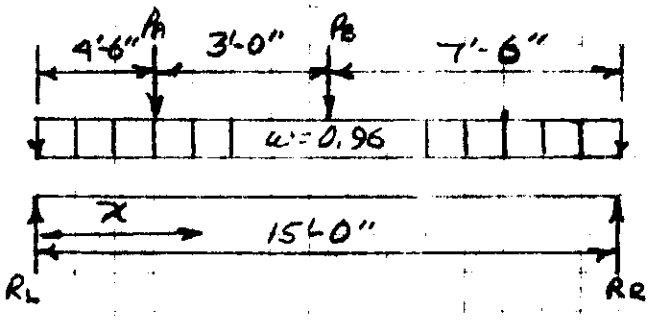
$M_x = 36.0$ K·FT.

LOADING 2

$P_A = \frac{3.0}{2} = 1.5^K$

$P_B = 3.3^K$

$R_L = \frac{15}{2} \times .96 + \frac{10.5 \times 1.5}{15} + \frac{7.5 \times 3.3}{15}$



LOADING 2

$R_L = 9.90^K$

$R_R = 15 \times .096 + 1.5 + 3.3 - 9.90 = 9.30^K$

$V = 0$ @ x

$x = 7.5$ ft

$M_x = 7.5 \times 9.3 - .96 \times \frac{7.5^2}{2}$

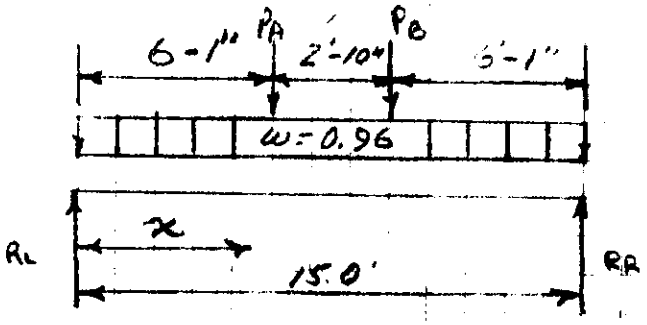
$M_x = 42.8$ K·FT.

SUBJECT SAXONVILLE

COMPUTATION PUMPING STATION - OPERATING FLOOR (5)

COMPUTED BY JRF CHECKED BY JAC DATE AUG 1974

LOADING 3



$P_A = 3.0^K$
$P_B = 3.3^K$

$R_L = (6.083 \times 3.3 + 8.92 \times 3.0 + 0.96 \times \frac{15^2}{2}) \div 15$

$R_L = 10.32^K$
$R_R = .96 \times 15 + 3.0 + 3.3 - 10.32$
$R_R = 10.38^K$

$V = 0 = x$

$x = (10.32 - 3.0) \div .96$
$x = 7.62'$

$M_x = 10.32 \times 7.62 - .096 \times \frac{7.62^2}{2} - 1.54 \times 3.0$

$M_x = 46.1$ K.FT.

$M_{max} = 46.1$ K.FT. (LOADING 3)

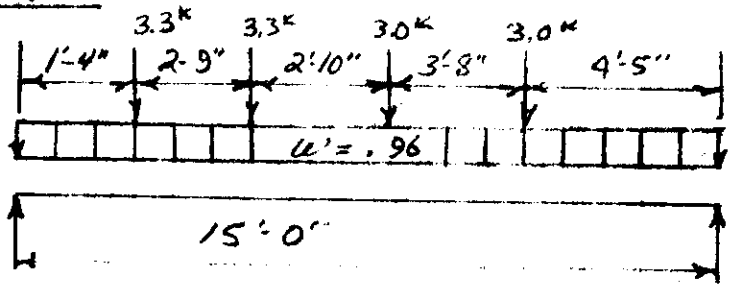
$f'c = 3000$
$Fc = 1350$

$d = \sqrt{\frac{46.1}{.225 \times 1.33}}$

$d = 12.41''$ ∴ USE $t = 15''$
$d = 13''$

$A_s = \frac{46.1}{1.45 \times 13} = 2.46 \text{ in}^2$

MAX V



$R_L = (4.42 \times 3.0 + 8.08 \times 3.0 + 10.92 \times 3.3 + 13.67 \times 3.3 + .96 \times \frac{15^2}{2}) \div 15$

$R_L = 15.11^K$

SUBJECT SAXONVILLE

COMPUTATION PUMPING STATION - OPERATING FLOOR (6)

COMPUTED BY JRF CHECKED BY JAC DATE Aug. 1974

$V_{MAX} = R_L = 15.11^{K}$

$v = \frac{15110}{16 \times 13}$

$v = 72.6$ psi > 60.0 psi $\therefore$ INCREASE d

$d_{REQ} = \frac{15110}{16 \times 60}$

$d_{REQ} = 15.7$ in.

$\therefore$ USE $t = 18''$, $d = 16''$ FOR FB1

$A_s = \frac{46.1}{1.45 \times 16}$

$A_s = 1.99$ in.^{2}

SUBJECT SAXONVILLE
COMPUTATION PUMPING STATION - STABILITY (5)
COMPUTED BY JRF CHECKED BY DATE AUG. 1974

ITEM	FORCE COMPUTATION	WT. (K)	X (FT.)	Y (FT)	Mx (K·FT)	My (K·FT)
ROOF						
SLAB	(.15)(15.0 x 22.33 x .42)	21.1	2.67	0	0	56.3
RB1	(.15)(15. x 1.0 x 1.0)	2.2	2.67	-4.50	-9.9	5.9
RB2	(.15)(15.33 x 1.0 x 0.67)	1.5	2.50	3.60	5.4	3.8
RB3	(.15)(7.3 x 1.0 x 0.67)	0.7	6.5 7.50	7.17	5.0	5.2
RB4	(.15)(11.0 x 1.0 x 0.67)	1.1	8.17	2.33	2.6	9.0
OPENING	(-.15)(11.0 x 5.0 x 0.42)	- 3.5	5.33	2.33	- 8.2	- 18.7
OPER. FLOOR						
SLAB	(.15)(20.33 x 22.33 x .5)	34.0	0	0	0	0
FB1	(.15)(15.0 x 1.0 x 1.67)	3.8	2.67	8.00	30.4	10.1
FB2	(.15)(15.0 x 0.5 2.67)	3.0	2.67	2.67	8.0	8.0
FB3	(.15)(15.0 x 1.5 x 1.0)	3.4	2.67	-2.50	- 8.5	9.1
OPENING -1	(-0.15)(2.0 x 2.5 x 0.5)	- 0.4	8.92	10.17	- 4.1	- 3.6
" -2	(-0.15)(3.33 x 3.67 x .5)	- 0.9	5.67	5.67	- 5.1	- 5.1
" -3	(-0.15)(3.33 x 3.67 x .5)	- 0.9	5.67	-0.33	0.3	- 5.1
" -4	(-0.15)(1.5 x 1.5 x .5)	- 0.2	9.50	-5.08	1.0	- 1.9
BASEMENT						
SLAB 1	(.15)(6.67 x 16.33 x 1.25)	20.4	2.67	-9.17	-187.1	54.5
" 2	(.15)(25.0 x 6.67 x 1.25)	31.3	-8.17	0	0	-255.7
" 3	(.15)(17.67 x 19.67 x 1.50)	78.2	2.67	2.67	208.8	208.8
WALLS						
BRICK N.	(.080)(17.67 x 14.0)	19.8	2.67	11.83	234.2	52.9
" E.	(.080)(22.33 x 14.0)	25.0	9.83	0	0	245.7
" W.	(.080)(22.33 x 14.0)	25.0	-5.50	0	0	-137.5
" S.	(.080)(17.67 x 14.0)	19.8	2.67	-11.83	-234.2	52.9
DOOR OPENING	(-0.080)(7 x 6)	- 3.4	2.67	11.83	- 40.2	-9.1
SUB TOTAL		281.0	—	—	-1.6	285.5

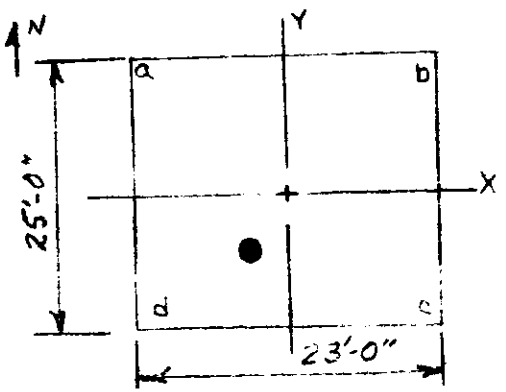
NED FORM 222
27 Sept 49

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

SUBJECT SAXONVILLE
COMPUTATION PUMPING STATION - STABILITY (6)
COMPUTED BY JRF CHECKED BY JAC DATE AUG. 1974

PAGE A 72

ITEM	FORCE COMPUTATION	WT. (K)	X (Ft)	Y (Ft)	Mx (K·Ft)	My (K·Ft)
WALLS (CONT.)						
SLUICE GATE 1	(-0.15) (3.5 x 3.0 x 1.33)	-2.1	-5.50	5.67	-11.90	+11.6
" " 2	(-0.15) (3.5 x 3.0 x 1.33)	-2.1	-5.50	-0.33	0.7	+11.6
LOUVRE E1	(-0.080) (4.75 x 3.5)	-1.3	9.83	5.67	-7.4	-12.8
" E2	(-0.080) (4.75 x 3.5)	-1.3	9.83	-0.33	0.4	-12.8
" W1	(-0.080) (3.75 x 3.0)	-0.9	-5.50	5.67	-5.1	5.0
" W2	(-0.080) (3.75 x 3.0)	-0.9	-5.50	-0.33	0.3	5.0
CONCRETE 1	(.15) (1.33 x 23.0 x 12.08)	55.4	0	-11.83	-655.4	0
" 2	(.15) (1.33 x 22.67 x 12.08)	54.6	-10.83	0	0	-591.3
" 3	(.15) (1.33 x 6.67 x 12.08)	16.1	-8.17	11.83	190.5	-131.5
" 4	(.15) (1.33 x 16.33 x 10.08)	32.8	3.33	11.83	388.0	109.2
" 5	(.15) (1.33 x 17.67 x 10.08)	34.2	8.17	2.33	279.7	279.4
" 6	(.15) (1.33 x 6.67 x 12.08)	16.1	8.17	-7.83	-126.1	131.5
" 7	(.15) (1.33 x 15.0 x 10.08)	30.2	2.67	-6.50	-196.3	80.6
" 8	(.15) (1.33 x 18.33 x 10.08)	36.9	-5.50	2.00	73.8	-202.9
SUB. TOTAL		267.7	—	—	-268.8	-317.4
SUB-TOTAL PG.		281.0	—	—	-1.6	285.5
TOTAL		548.7	—	—	-270.4	-31.9



$C_x = 11.5'$
$C_y = 12.5'$

$I_x = (1/12)(23)(25)^3 = 29948$
$I_y = (1/12)(25)(23)^3 = 25348$

SUBJECT SAXONVILLE
COMPUTATION PUMPING STATION - STABILITY (7)
COMPUTED BY JRF CHECKED BY JAC DATE AUG. 1974

$$BP = \frac{P}{A} \pm \frac{M_x C_y}{I_x} \pm \frac{M_y C_x}{I_y}$$

$$BP = \frac{548.7}{23 \times 25} \pm \frac{270.4 \times 12.5}{29948} \pm \frac{31.9 \times 11.5}{25348}$$

$BP = 0.95 \pm 0.11 \pm 0.01 =$

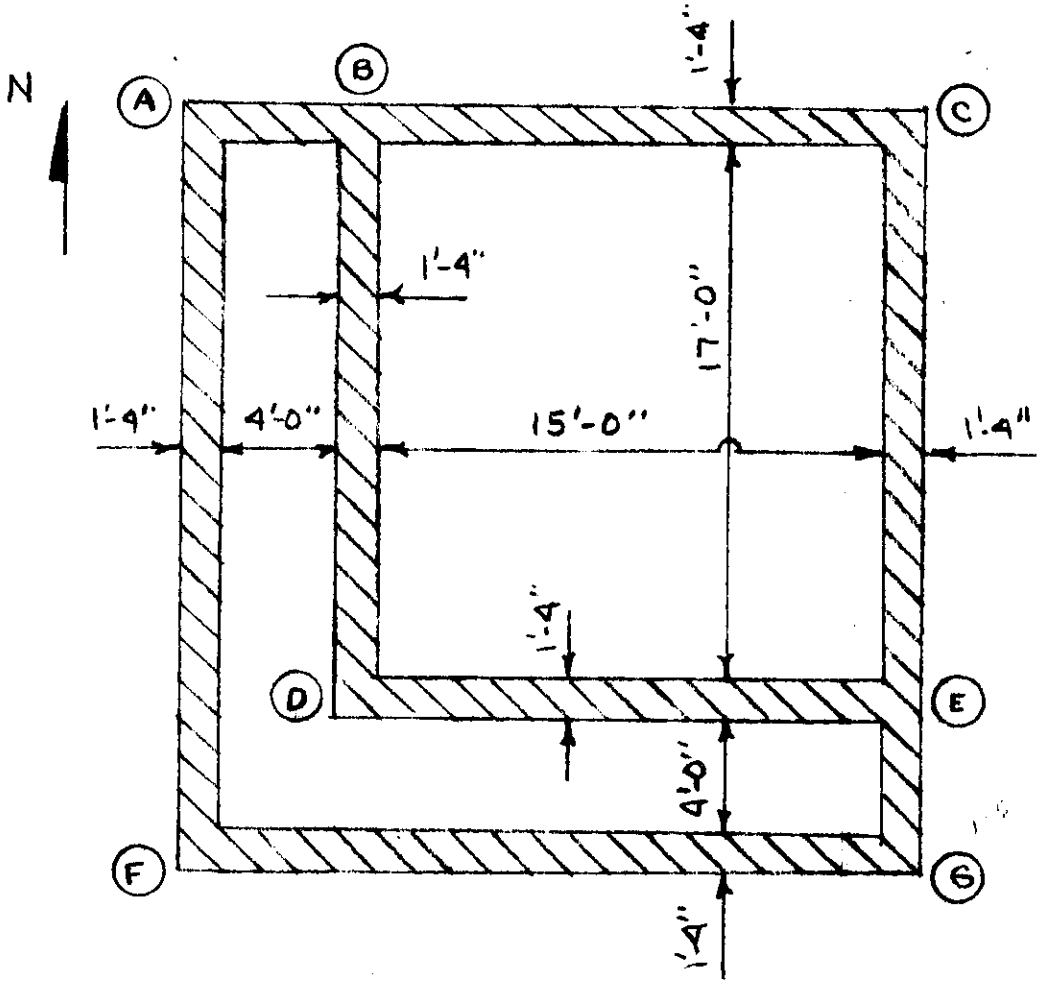
$BP_a = 0.95 - 0.11 + 0.01 = 0.85$ KSF

$BP_b = 0.95 - 0.11 - 0.01 = 0.83$ KSF

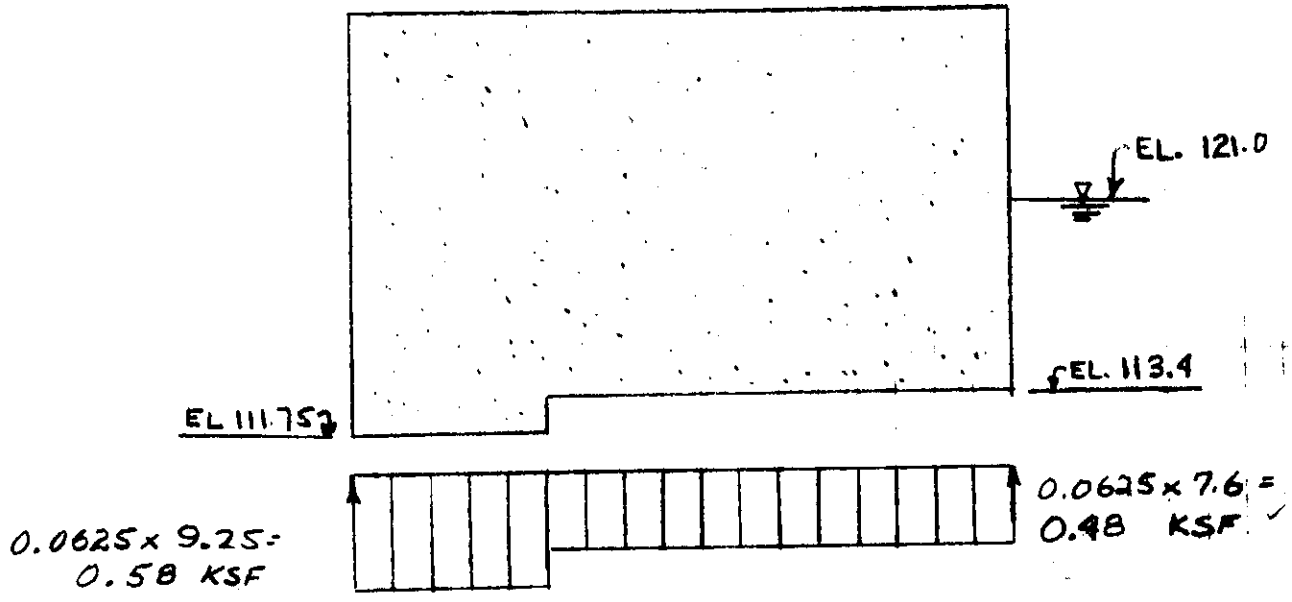
$BP_c = 0.95 + 0.11 - 0.01 = 1.05$ KSF

$BP_d = 0.95 + 0.11 + 0.01 = 1.07$ KSF

UPLIFT



SUBJECT SAXONVILLE
COMPUTATION PUMPING STATION - STABILITY (8)
COMPUTED BY JRF CHECKED BY FAC DATE AUG. 1974



ITEM	FORCE COMPUTATION	WT. (K)	X (ft)	Y (ft)	Mx (K·FT)	My (K·ft)
UPLIFT						
SLAB 1	0.58 × 7.50 × 16.33	-71.0	3.88	-8.75	621.3	-236.4
" 2	0.58 × 6.67 × 25.00	-96.7	-8.17	0	0	790.0
" 3	0.48 × 17.50 × 16.33	-137.2	3.33	3.75	-814.5	-456.9
TOTAL		304.9	—	—	106.8	96.7

$$F.S. = \frac{548.7}{304.9} = 1.80$$

BEARING PRESSURE w/ UPLIFT

$$BP = \frac{(548.7 - 304.9)}{23 \times 25} \pm \frac{(-270.4 + 106.8) \times 12.5}{2594B} \pm \frac{(-31.7 + 96.7) \times 11.5}{2534B}$$

$$BP_a = 0.42 - 0.07 - 0.03 = 0.32$$

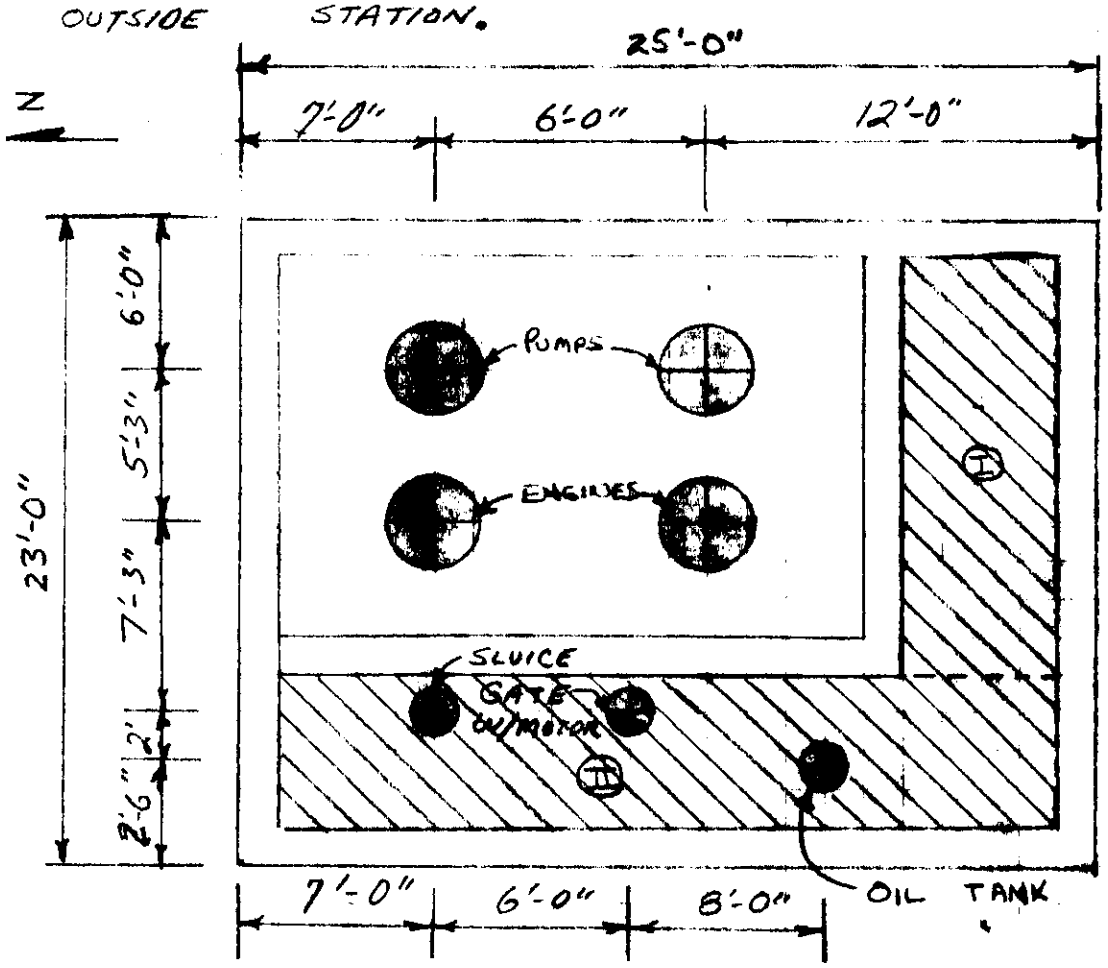
$$BP_b = 0.42 - 0.07 + 0.03 = 0.38$$

$$BP_c = 0.42 + 0.07 - 0.03 = 0.46$$

$$BP_d = 0.42 + 0.07 + 0.03 = 0.52$$

SUBJECT SAXONVILLE
COMPUTATION PUMPING STATION - STABILITY
COMPUTED BY URF CHECKED BY AC DATE AUG 1974

STABILITY CASE #2
PUMPING STATION W/ EQUIPMENT, WATER TO ELEV. 121.0 INSIDE STATION @ TO ELEV. 116 OUTSIDE STATION.



PUMPS - 3.0K (2)
ENGINE - 3.3K (2)
SLUICE GATE W/MOTOR - 1.3K (2)
OIL TANK - 4.0K (1)

WT BUILDING - ~~599~~.5K
546.2

Wt. of water

$W_{wI} = 4 \times 16.33 \times 8.0 \times 0.0625 = 32.7^K$

$W_{wII} = 4 \times 22.33 \times 8.0 \times 0.0625 = 44.6^K$

TOTAL WT. = 548.7 + 2(3.0 + 3.3 + 1.3) + 4.0 + 32.7 + 44.6
TOTAL WT. = 645.2

SUBJECT SAXONVILLE
COMPUTATION PUMPING STATION - STABILITY (13)
COMPUTED BY JAF CHECKED BY JAC DATE AUG. 1974

ITEM	FORCE COMPUTATION	WT. (K)	X (Ft.)	Y (Ft.)	Mx (K·Ft)	My (K·Ft)
BUILDING		548.7	-0.06	-0.49	-270.4	-31.9
PUMP (N)		3.0	5.67	+5.50	+16.50	17.01
" (S)		3.0	5.67	-0.50	+ 1.50	17.01
ENG. (N)		3.3	0.17	+5.50	+18.15	0.56
" (S)		3.3	0.17	+0.50	- 1.65	0.56
GATE (N)		1.3	-7.00	+5.50	+7.15	-9.10
GATE (S)		1.3	-7.00	-0.50	-0.65	-9.10
OIL TANK		4.0	-9.00	-8.5	-34.00	-36.00
WATER (N)		44.6	-8.17	0.0	0.00	-364.58
" (S)		32.7	2.00	-9.17	-299.86	65.40
UPLIFT						
SLAB 1	0.27 × 7.50 × 16.33	-33.1	3.33	-8.75	289.6	-110.2
" 2	0.27 × 6.67 × 25.00	-45.0	-8.17	0	0.00	367.6
" 3	0.16 × 17.50 × 16.33	-45.7	3.33	3.75	- 171.4	-1152.2
TOTAL		521.4	—	—	-448.1	-244.7

$$BP = \frac{521.4}{23 \times 25} \pm \frac{448.1 \times 12.5}{29948} \pm \frac{244.7 \times 11.5}{25348}$$

$BP_a = 0.91 - 0.19 + 0.11 = 0.83$ KSF

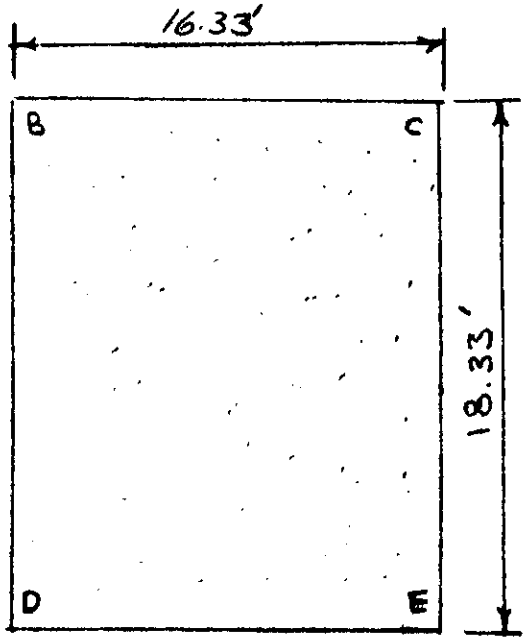
$BP_b = 0.91 - 0.19 - 0.11 = 0.61$ KSF

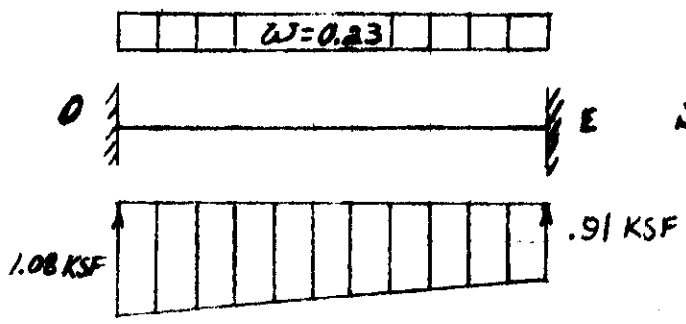
$BP_c = 0.91 + 0.19 - 0.11 = 0.99$ KSF

$BP_d = 0.91 + 0.19 + 0.11 = 1.21$ KSF

SUBJECT SAXONVILLE
COMPUTATION PUMPING STATION - BASE SLAB DESIGN (14)
COMPUTED BY JRF CHECKED BY JA DATE AUG. 1974

SLAB BCDE - DESIGN
LOADING 2





$R_D = 6.5^K$

$V = 6.5 - 1.5 \times .84$
$V = 5.24^K$

$v = \frac{5240}{12 \times 11.5}$

$v = 38.0 < 60.0 \quad \therefore$ OK

$u = \frac{5240.0}{3.1 \times .88 \times 11.5}$

$u = 167.0 < 264 \quad \therefore$ OK

$M(-) = \left[\frac{1}{12}(.68) + \frac{1}{20}(.17)\right] 16.33^2$

$M(-) = 17.4$ K·FT.

$M(+) = \frac{1}{14}(.77)(16.33)^2$

$M(+) = 14.7$ K·FT.

$\frac{SS}{LS} = \frac{16.83}{18.33} = 0.89$

@ FACE BD

$M(-) = 17.4 - \frac{1}{3}(6.5)(1.5)$

$M(-) = 14.2$ K·FT

S.S.
$As(-) = \frac{14.2}{1.44 \times 11.5}$

$As(-) = 0.86 \text{ in}^2$ - #9@12"

$As(+) = \frac{14.7}{1.44 \times 13.5}$

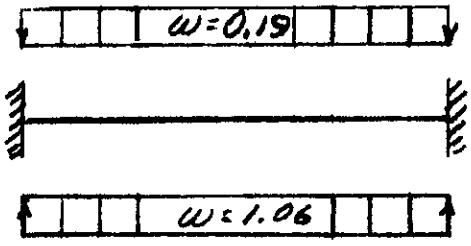
$As(+) = 0.76 \text{ in}^2$ - #8@12"

L.S.
$As(-) = 0.89 \times .86$
$As(-) = 0.77 \text{ in}^2$ - #8@12"

$As(+) = 0.89 \times 0.76$
$As(+) = 0.68 \text{ in}^2$ - #8@12"

SUBJECT SAXONVILLE
COMPUTATION PUMPING STATION - BASE SLAB DESIGN (10)
COMPUTED BY JRF CHECKED BY J.A.C. DATE AUG. 1974

SLAB ABDEGF
LOADING 1



$V = 0.87 \times \frac{6.67}{2} = 2.9^K$

$v = \frac{2900}{12 \times 8.5}$

$v = 28.4 < 60 \therefore OK$

$u = \frac{2900}{2.0 \times .88 \times 8.5}$

$u = 194 < 264 \therefore OK$

$M(-) = \frac{1}{12}(.87)(6.67)^2$

$M(-) = 3.2$ K·FT

$M(+) = \frac{1}{14}(.87)(6.67)^2$

$M(+) = 2.8$ K·FT

$As(-) = \frac{3.2}{1.44 \times 8.5}$

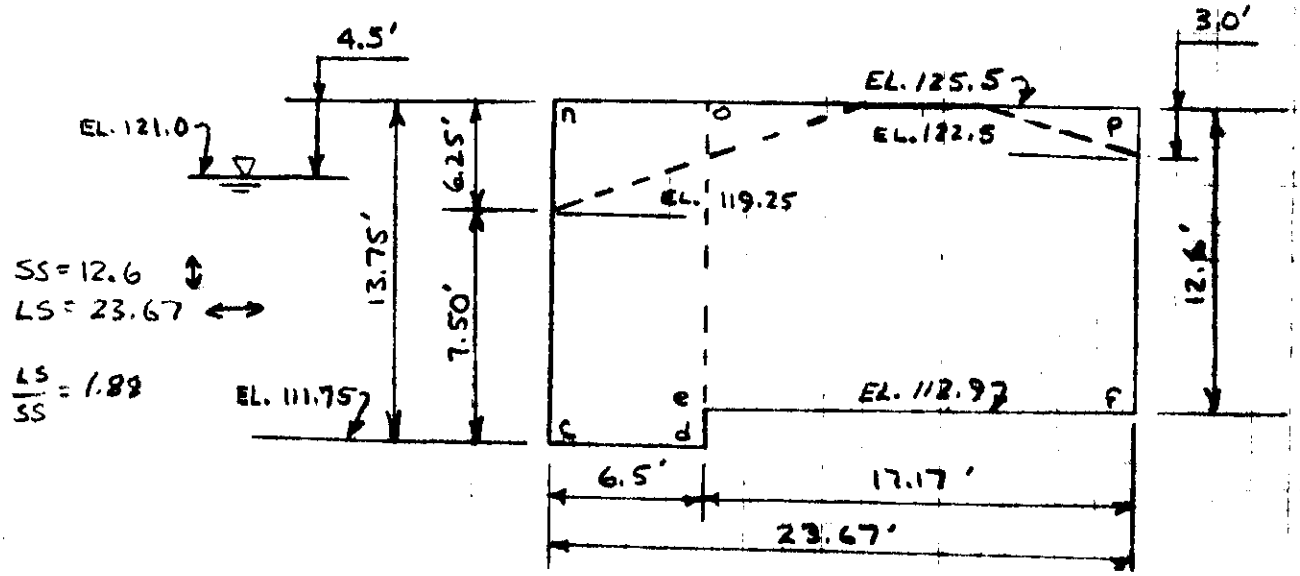
$As(-) = 0.26$ in.^{2} - #5@12"

$As(+) = \frac{2.8}{1.44 \times 10.5}$

$As(+) = 0.19$ in^2 - #4@12"

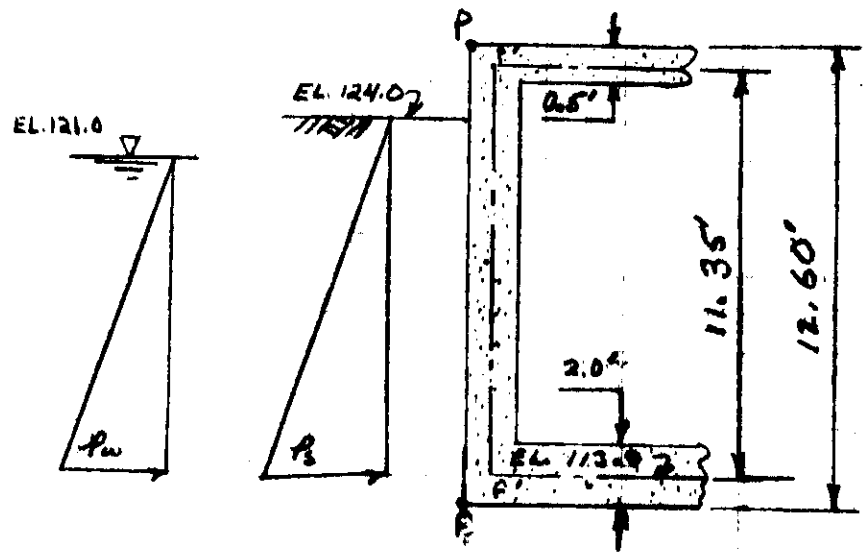
DESIGN OF WALLS

Wall cdeFpon



ASSUME ONE-WAY ACTION (VERT.), WITH WALL FIXED AT THE BOTTOM AND HINGED AT THE TOP

AVE. EL. = 124.0



$k = 0.33$
$\gamma_{WAT} = 62.5$ pcf
$\gamma_{SUB} = 62.5$ pcf

$p_w = 0.0625\,(121.0 - 113.9) = 0.44$ ksf

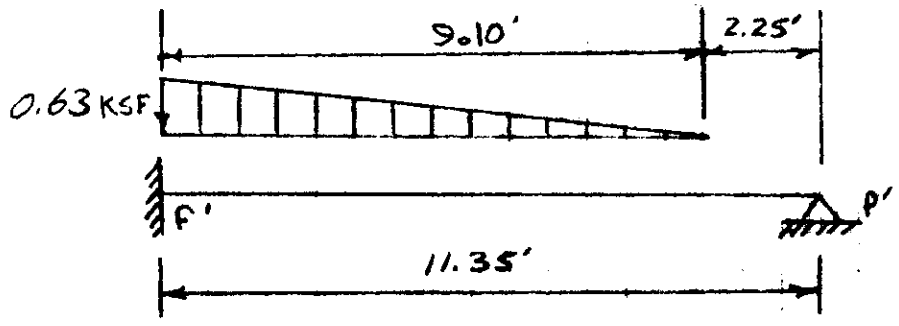
$p_s = .33 \times .0625\,(124.0 - 113.9) = 0.19$ ksf

$\Sigma p = 0.63$ ksf

SUBJECT SAXONVILLE

COMPUTATION PUMPING STATION - Wall DESIGN (4B)

COMPUTED BY JRF CHECKED BY VAC DATE AUG. 1979



M	4.32	0
Vs	2.10	.77
ΔM/L	+0.38	-.38
V	2.48 K	0.39 K

V = 0 5.61 ft. from p'

M(+) = 1.75 K·FT./FT.

MOMENT AT TOP OF BASE SLAB

$M(-) = 4.32 - \frac{1}{3}(2.48)(1.5)$

M(-) = 3.08 K·FT

$As(-) = \frac{3.08}{1.45 \times 17.5}$

As(-) = .18 in² USE # 4@12"

$As(+) = \frac{1.75}{1.45 \times 17.5}$

As(+) = 0.10 in.² USE # 4@12"

MIN. STEEL $= \frac{200}{fy}(b \times d) = \frac{200 \times 12}{40{,}000}(d) = .06d$

d(in.)	As(in.²)	USE
7.5	.45	# 6@12"
11.5	.69	# 8@12"
13.5	.81	# 8@12"

∴ 1'-6" walls USE #8@12

APPENDIX B

MECHANICAL DESIGN COMPUTATIONS

NED FORM 223
27 Sept 49

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

PAGE B-1

SUBJECT SAXONVILLE MASS LOCAL PROTECTION
COMPUTATION SAXONVILLE PUMPING STATION
COMPUTED BY JTS CHECKED BY DATE Aug 74

STATION DESIGN CAPACITY

35 cfs

Design flood elev.	138.5'
Top of dike	136.5'
Invert of discharge pipe	136.5'
Suction chamber high water (Pump Start)	121.0'
" " low " (Pump Stop)	117.0'

2 pumps (Each to handle 2/3 of total

PUMP DESIGN CONDITION POINT

$\frac{35 \times 2}{3} = 23.33$ cfs $= 10{,}500$ Gpm.

Diam. of disch. $= \sqrt{\frac{4 \times 23.33}{12\pi}} = 1.57$ ft $= 18.88$ in

Try 18" & 20" discharge pipe

Head @ Q = 10,500 gpm

		18"	20"
H.W Suction to top of disch. pipe	138.00 - 121	17.00	
" " " " " " "	138.17 - 121		17.17
Velocity head		2.74	1.80
Disch. pipe loss		6.32	3.68
Total Head		26.06'	22.65

SYSTEM LOSSES - 18" DISCHARGE PIPE

Gpm	Vel.	Vel.hd	Pipe H_L	Total
6,000	7.56	0.89	2.12	3.01
8,000	10.09	1.84	3.70	6.54
10,000	12.60	2.47	5.70	8.17
10,500	13.22	2.74	6.32	9.06
12,000	15.10	3.55	8.19	11.74
14,000	17.70	4.84	11.16	16.00
16,000	20.20	6.32	14.58	20.90

NED FORM 223
27 Sept 49

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

SUBJECT SAXONVILLE, MASS. LOCAL PROTECTION
COMPUTATION SAXONVILLE PUMPING STATION
COMPUTED BY JTC CHECKED BY DATE Aug, 74

SYSTEM LOSSES - 20" DISCHARGE PIPE

Gpm	Vel	Vel.Hd	Pipe hf	Total
6,000	6.13	0.58	1.22	1.80
8,000	8.17	1.04	2.15	3.19
10,000	10.20	1.62	3.34	4.96
10,500	10.72	1.80	3.68	5.48
12,000	12.30	2.33	4.76	7.09
14,000	14.30	3.18	6.44	9.62
16,000	16.30	4.15	8.42	12.57

NED FORM 223
27 Sept 49

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

SUBJECT SAXONVILLE, MASS. LOCAL PROTECTION
COMPUTATION SAXONVILLE PUMPING STATION
COMPUTED BY JTS CHECKED BY DATE Aug. 74

SUMMARY

Model Curves	Imp Diam in	RPM	Cont. HP	Max HP	Suct Hd Ft
PLATE 18					
18" Pipe Disch	16.23	1077	92	98	+1
20"	15.72	1113	81	87	+1
PLATE 19					
18" Pipe Disch.	17.24	1080	93	99	+1
20" " "	16.67	1117	80	86	+1
PLATE 20					
18" Pipe Disch.	18.80	1061	99	99	+2
20" " "	17.62	1132	81	87	+2
PLATE 21					
18" Pipe Disch.	14.61	1185	107	110	+2
20" " "	14.36	1205	101	103	+2

Computations and curves pages M-4 thru M-9 show that an axial flow pump with 16.23" impeller operating at 1077 RPM using an 18" discharge pipe satisfactorily meets requirements and is the best selection. Computations and curves are included to show head-capacity and horsepower requirements if 20" discharge pipe over dike is used. Although the pump horsepower requirements are less with the 20" discharge pipe the difference is such that the same size engine will suit the requirements of both conditions with 18" and 20" discharge piping. Therefor using 18" discharge piping is the more economical selection.

VALUES FROM CURVE - PLATE 18

	Qm	Hm	Pm
Max. Eff	11,250	31.0	109
	13,000	25.5	98
	14,250	20.0	88
	15,200	15.0	78
	16,050	10.0	68
	16,800	5.0	57
Values at cond. pt. 18"	12,850	26.1	99
" " " " 20"	13,700	22.6	93

ACTUAL MODEL RATIOS

18" Pump Discharge $\overline{MR}^2 = \frac{10,500}{12,850} = 0.817$; $MR = 0.904$

20" " " $\overline{MR}^2 = \frac{10,500}{13,700} = 0.766$; $MR = 0.875$

PROTOTYPE VALUES

	Qp	H	Pp
18" Pump Disch.			
	9,190	31.0	89
Cond. Pt.	10,500	26.1	81
Max. Eff.	10,620	25.5	80
	11,640	20.0	72
	12,420	15.0	64
	13,110	10.0	56
	13,730	5.0	46
20" Pump Disch.			
	8,620	31.0	84
Max. Eff	9,960	25.5	75
Cond. Pt.	10,500	22.6	71
	10,920	20.0	67
	11,650	15.0	60
	12,300	10.0	52
	12,880	5.0	44

18" DISCHARGE PIPE

$N = \frac{974}{0.904} = 1077$ rpm

Impeller diam. = 17.96 × 0.904 = 16.23 in

From plotted curve

Q = 10,500 gpm
P@ rated Q = 81 HP

BHP required by pump	= 81
+10%	= 89
Loss in reduct. gear +4%	= 3
BHP at engine coupling	= 92 HP (Continuous Rating)
P_{max} (Low Water)	= 85
+10%	= 94
Loss in reduct. gear +4%	= 4
BHP at engine coupling	= 98 HP (Maximum HP)

20" DISCHARGE PIPE

$N = \frac{974}{0.875} = 1113$ rpm

Impeller diam. = 17.96 × 0.875 = 15.72 in

From plotted curve

Q = 10,500 gpm
P@ rated Q = 71 HP

BHP required by pump	= 71
+10%	= 78
Loss in reduct. gear +4%	= 3
BHP at engine coupling	= 81 HP (Continuous Rating)
P_{max} (Low Water)	= 76
+10%	= 84
Loss in reduct. gear +4%	= 3
BHP at engine coupling	= 87 HP (Maximum HP)

SPECIFIC SPEED

18" DISCHARGE PIPE

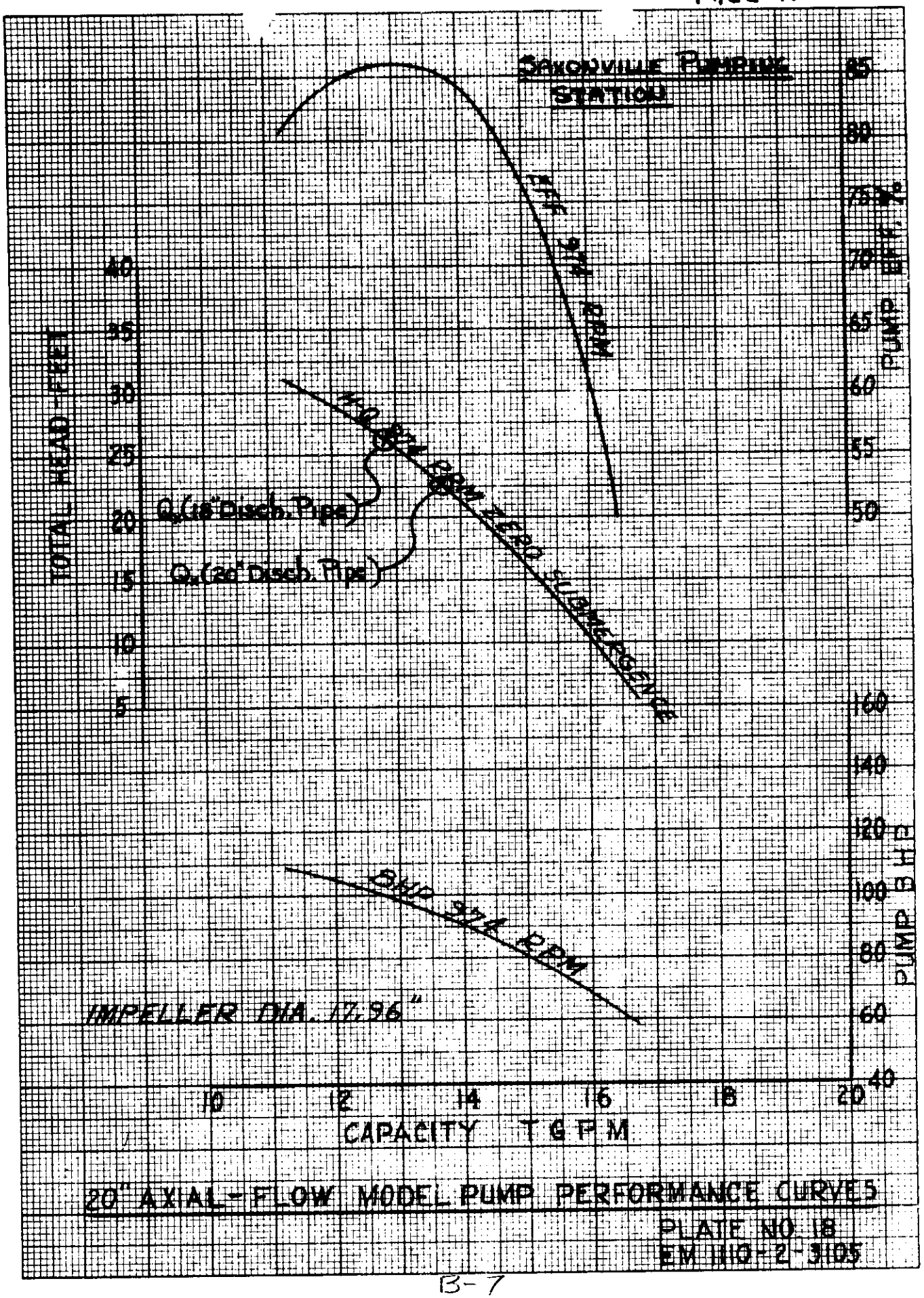
$N_s = \frac{1077\sqrt{10.620}}{25.674} = 9780$ rpm

Suction Hd = +1

20" DISCHARGE PIPE

$$N_s = \frac{1113\sqrt{9{,}960}}{25.5^{3/4}} = 9788 \text{ rpm}$$

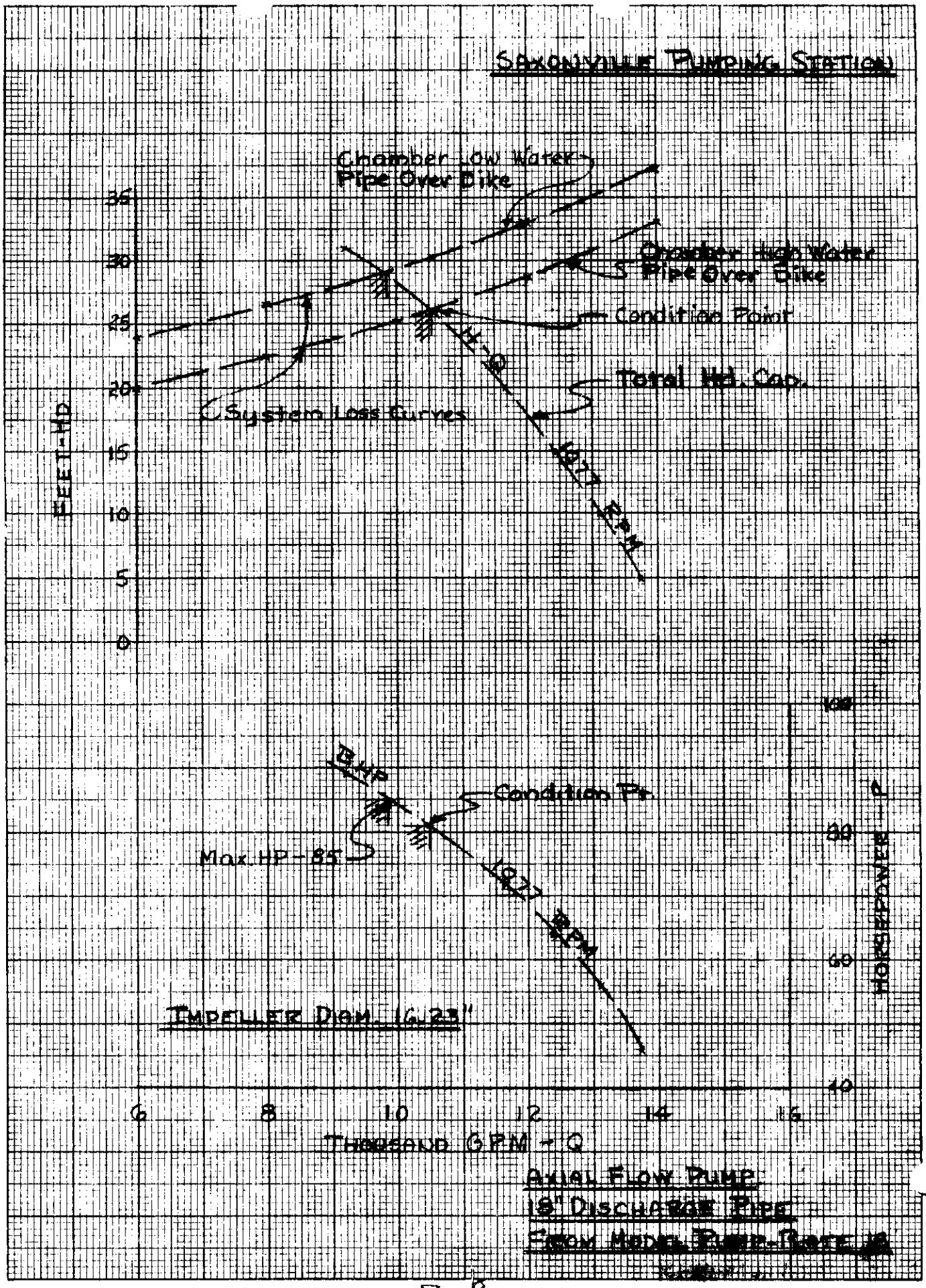
Suction Hd. = +1

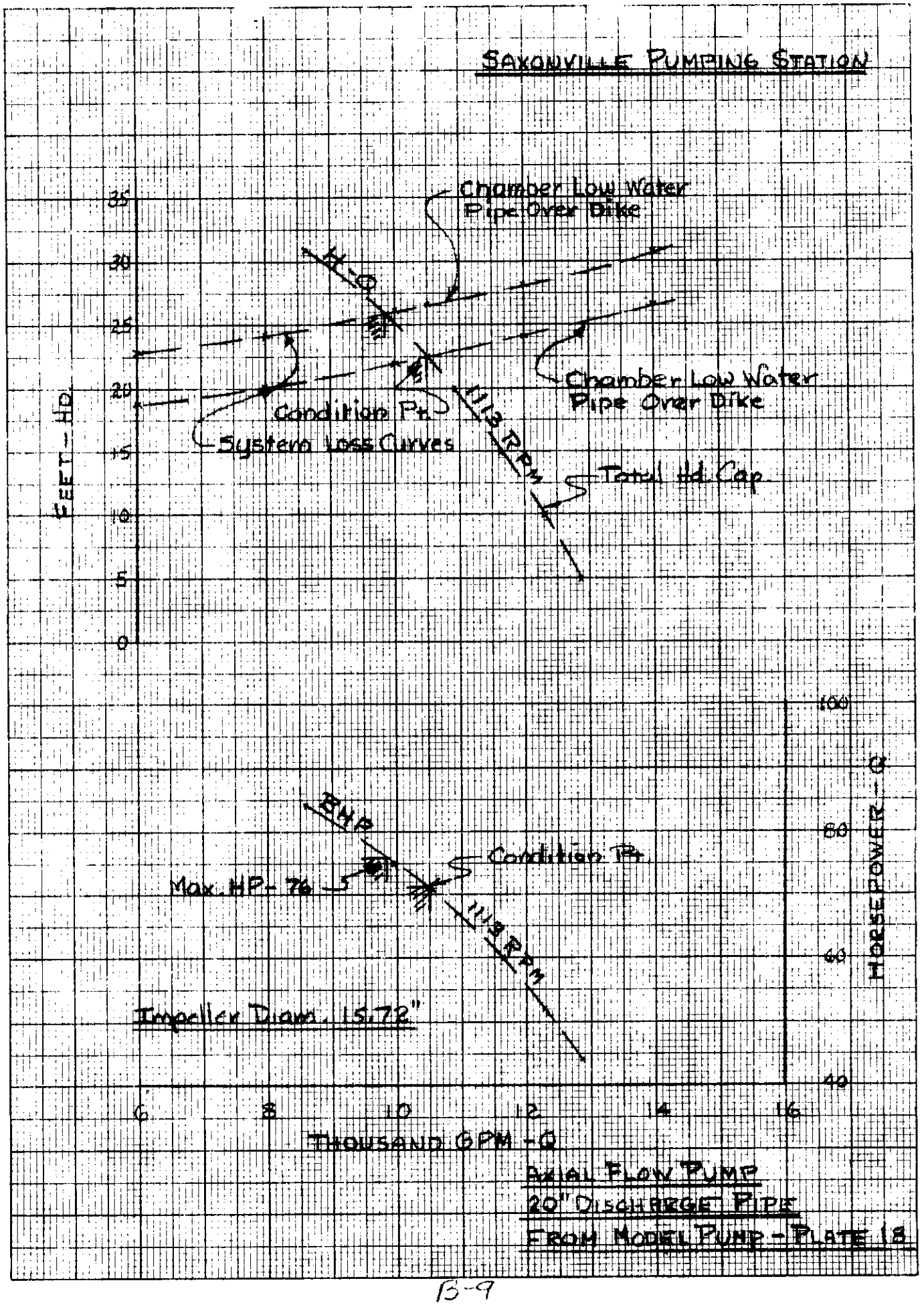


20" AXIAL-FLOW MODEL PUMP PERFORMANCE CURVES

PLATE NO. 18
EM 1110-2-3105

K+E 10 X 10 TO ½ INCH 46 1323
7 X 10 INCHES MADE IN U.S.A.
KEUFFEL & ESSER CO.


SAXONVILLE PUMPING STATION
Chamber Low Water
Pipe Over Dike
Chamber High Water
Pipe Over Dike
Condition Point
Total Hd. Cap.
System Loss Curves
FEET-HD
35
30
25
20
15
10
5
0
Condition Pt.
Max HP - 85
HORSEPOWER
100
80
60
40
IMPELLER DIAM. 16.23"
6
8
10
12
14
16
THOUSAND GPM - Q
AXIAL FLOW PUMP
18" DISCHARGE PIPE
FROM MODEL PUMP-PLATE 18


SAXONVILLE PUMPING STATION
Chamber Low Water Pipe Over Dike
Chamber Low Water Pipe Over Dike
Condition Pt.
System Loss Curves
1112 RPM
Total Hd. Cap
FEET - HD
35
30
25
20
15
10
5
0
BHP
Condition Pt.
Max. HP - 76
1118 RPM
Impeller Diam. 15.72"
100
80
60
40
HORSEPOWER - G
6
8
10
12
14
16
THOUSAND GPM - Q
AXIAL FLOW PUMP
20" DISCHARGE PIPE
FROM MODEL PUMP - PLATE 18

VALUES FROM CURVE - PLATE 19

	Q_m	H_m	P_m
	9,600	32.5	98
	10,350	30.0	95
Max. Eff.	11,700	25.0	85
	12,750	20.0	77
	13,550	15.0	69
	14,250	10.0	59
	14,800	5.0	48

Values at cond. pt. 18"	11,400	26.1	88
" " " " 20"	12,200	22.6	82

ACTUAL MODEL RATIOS

18" Pump Discharge $\overline{MR}^2 = \frac{10,500}{11,400} = 0.921$; MR = 0.960

20" " " $\overline{MR}^2 = \frac{10,500}{12,200} = 0.861$; MR = 0.928

PROTOTYPE VALUES

18" Pump Disch

	Q_p	H	P_p
	8,840	32.5	90
	9,530	30.0	87
Cond. Pt.	10,500	26.1	81
Max. Eff.	10,780	25.0	78
	11,740	20.0	71
	12,480	15.0	64
	13,120	10.0	54
	13,630	5.0	44

20" Pump. Disch.

	8,260	32.5	84
	8,910	30.0	82
Max. Eff.	10,070	25.0	73
Cond. Pt.	10,500	22.6	70
	10,970	20.0	66
	11,660	15.0	59
	12,260	10.0	51
	12,740	5.0	41

18" DISCHARGE PIPE

$N = \frac{1037}{0.960} = 1080$ rpm

Impeller diam = 17.96 × 0.960 = 17.24 in

From plotted curve

Q = 10,500 gpm		
P@ rated Q = 81 HP		
BHP required by pump	=	81
+10%	=	89
Loss in reduct. gear +4%	=	4
BHP at engine coupling	=	93 HP (Continuous Rating)
P_{max} (Low Water)	=	86
+10%	=	95
Loss in reduct. gear +4%	=	4
BHP at engine coupling	=	99 HP (Maximum HP)

20" DISCHARGE PIPE

$N = \frac{1037}{0.928} = 1117$ rpm

Impeller diam. = 17.96 × 0.928 = 16.67 in

From plotted curve

Q = 10,500 gpm		
P@ rated Q = 70 HP		
BHP required by pump	=	70
+10%	=	77
Loss in reduct. gear +4%	=	3
BHP at engine coupling	=	80 HP (Continuous Rating)
P_{max} (Low Water)	=	75
+10%	=	82
Loss in reduct. gear +4%	=	3
BHP at engine coupling	=	85 HP (Maximum HP)

SPECIFIC SPEEDS

18" DISCHARGE PIPE

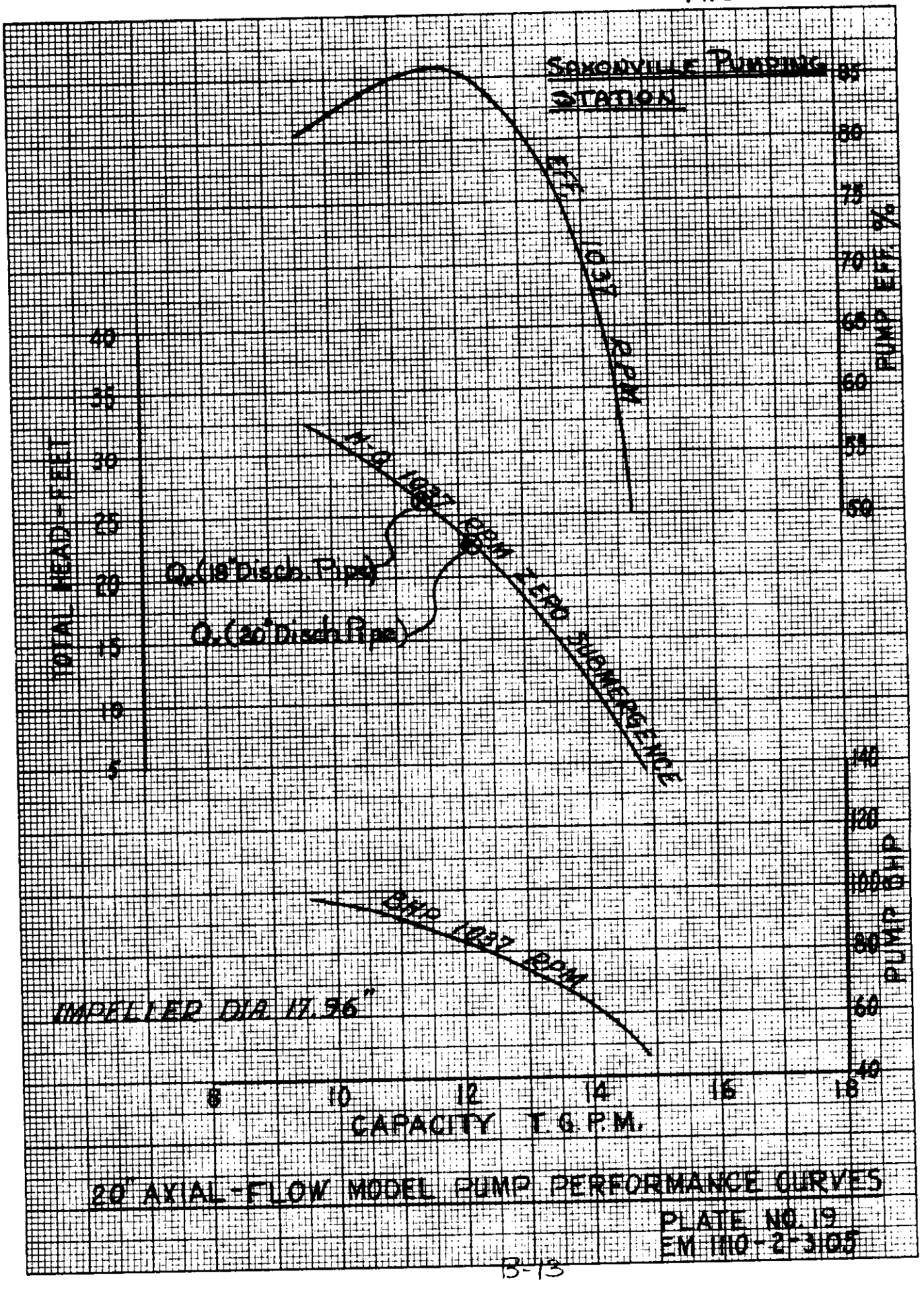
$N_s = \frac{1080\sqrt{10,780}}{25.0^{3/4}} = 10,029$ rpm

Suction Hd = +1

20" DISCHARGE PIPE

$$N_s = \frac{1117\sqrt{10{,}070}}{25.0^{3/4}} = 10{,}025 \text{ rpm}$$

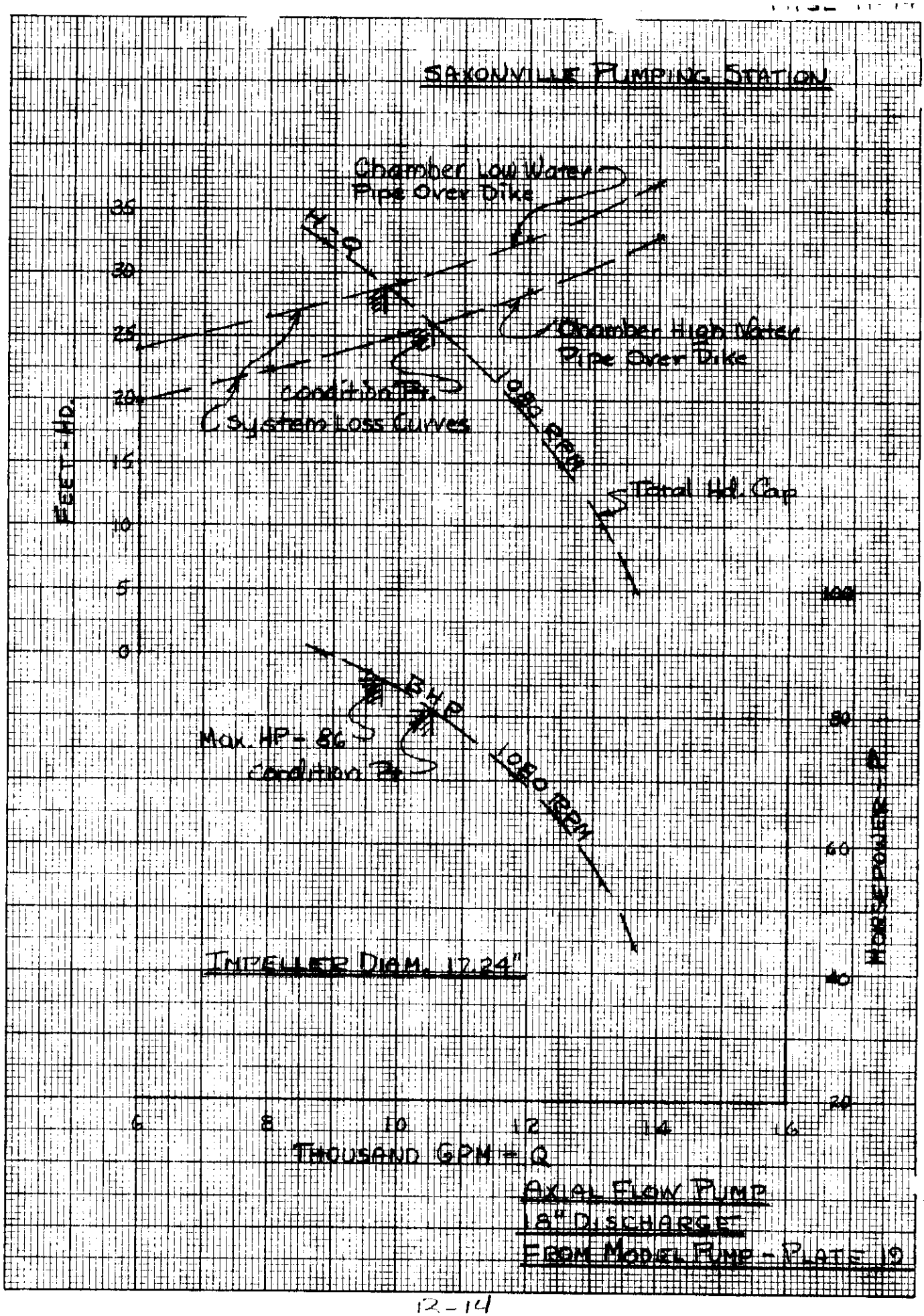
Suction Hd. = +1

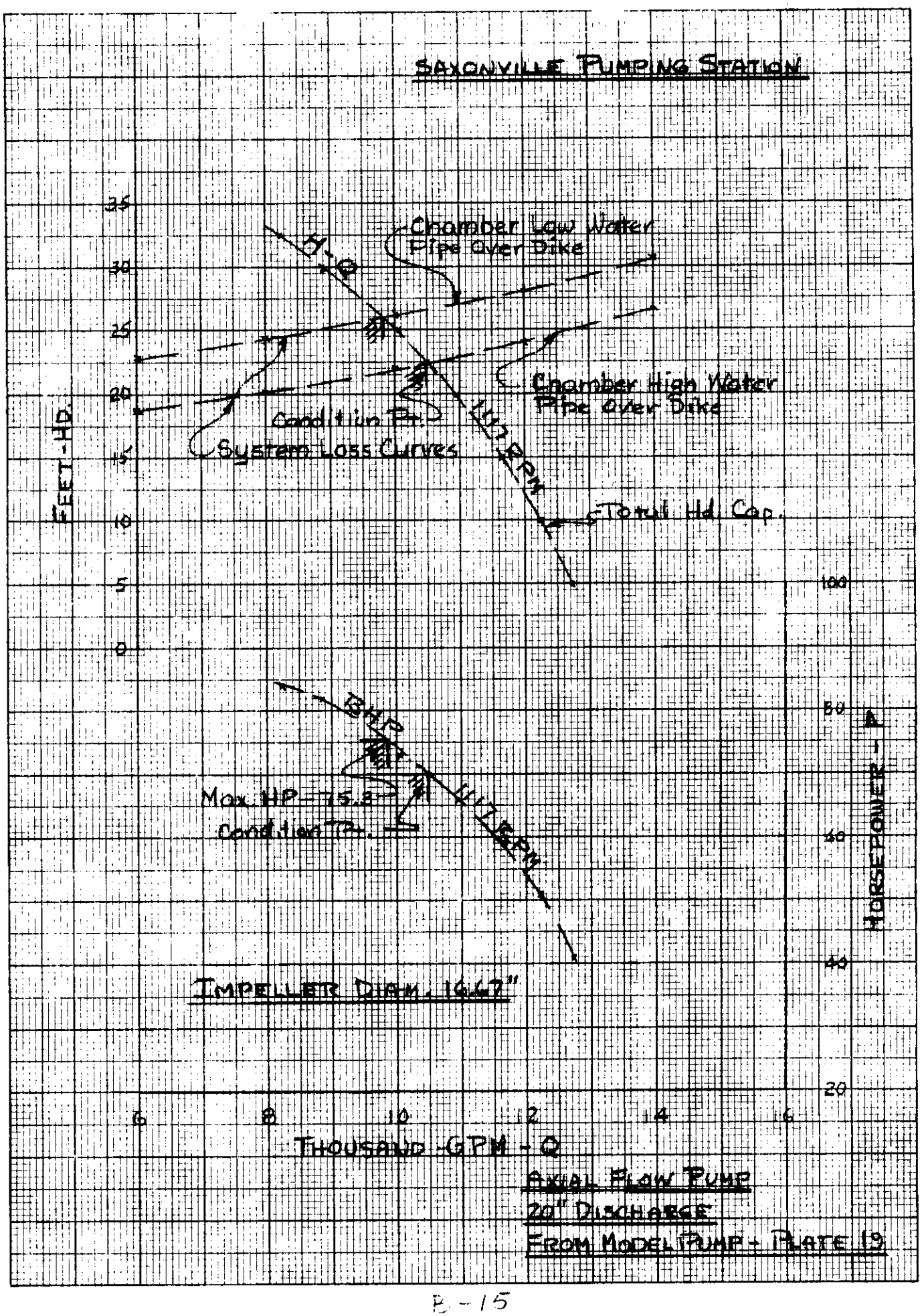


20" AXIAL-FLOW MODEL PUMP PERFORMANCE CURVES

PLATE NO. 19
EM 1110-2-3105

K+E 10 X 10 TO ½ INCH 46 1323
7 X 10 INCHES MADE IN U.S.A.
KEUFFEL & ESSER CO.


SAXONVILLE PUMPING STATION
Chamber Low Water Pipe Over Dike
Chamber High Water Pipe Over Dike
Condition Pt.
System Loss Curves
Total Hd. Cap
Max. HP - 86
Condition Pt.
IMPELLER DIAM. 17.24"
FEET - HD.
HORSE POWER
THOUSAND GPM = Q
AXIAL FLOW PUMP
18" DISCHARGE
FROM MODEL PUMP - PLATE 19



K+E 10 X 10 TO 1/2 INCH 46 1323
7 X 10 INCHES MADE IN U.S.A.
KEUFFEL & ESSER CO.

SUBJECT SAXONVILLE MASS. LOCAL PROTESTION
COMPUTATION SAXONVILLE PUMPING STATION
COMPUTED BY JTS CHECKED BY DATE Aug. 74

VALUES FROM CURVE - PLATE 20

	Q_m	H	P_m
Max. Eff.	10,000	29.0	106
	12,100	27.5	107
	14,600	23.5	105
	15,900	20.0	100
	17,500	15.0	87
	18,500	10.0	75
	19,000	7.5	65

Values at Cond. Pt. 18"	13,100	26.1	108
" " " " 20"	14,900	22.6	105

ACTUAL MODEL RATIOS

18" Pump Discharge $\overline{MR}^2 = \frac{10,500}{13,100} = 0.802$; MR = 0.895

20" " " $\overline{MR}^2 = \frac{10,500}{14,900} = 0.705$; MR = 0.839

PROTOTYPE VALUES

	Q_p	H	P_p
18" Pump Disch.			
	8,020	29.0	85
	9,700	27.5	86
Cond. Pt.	10,500	26.1	86
Max. Eff.	11,700	23.5	84
	12,740	20.0	80
	14,030	15.0	70
	14,830	10.0	60
	15,230	7.5	52
20" Pump Disch.			
	7,050	29.0	75
	8,530	27.5	75
Max. Eff.	10,290	23.5	74
Cond. Pt.	10,500	22.6	74
	11,200	20.0	70
	12,330	15.0	61
	13,040	10.0	53
	13,390	7.5	46

18" DISCHARGE PIPE

$N = \frac{950}{0.895} = 1061$ rpm

Pump size = 21 x 0.895 = 18.80 in

From plotted curve

Q = 10,500 gpm
P@ rated Q = 86 HP

BHP required by pump	=	86
+10 %	=	95
Loss in reduct. gear +4%	=	4
BHP at engine coupling	=	99 HP (Continuous Rating)
P_{max} (Low Water)	=	86
+10%	=	95
Loss in reduct. gear +4%	=	4
BHP at engine coupling	=	99 HP (Maximum H.P.)

20" DISCHARGE PIPE

$N = \frac{950}{0.839} = 1132$ rpm

Pump size = 21 x 0.839 = 17.62 in

From plotted curve

Q = 10,500 gpm
P@ rated Q = 74 HP

BHP required by pump	=	74
+10%	=	81
Loss in reduct. gear +4%	=	3
BHP at engine coupling	=	84 HP (Continuous Rating)
P_{max} (Low Water)	=	76
+10%	=	84
Loss in reduct. gear +4%	=	3
BHP at engine coupling	=	87 HP (Maximum HP)

SPECIFIC SPEED

18" DISCHARGE PIPE

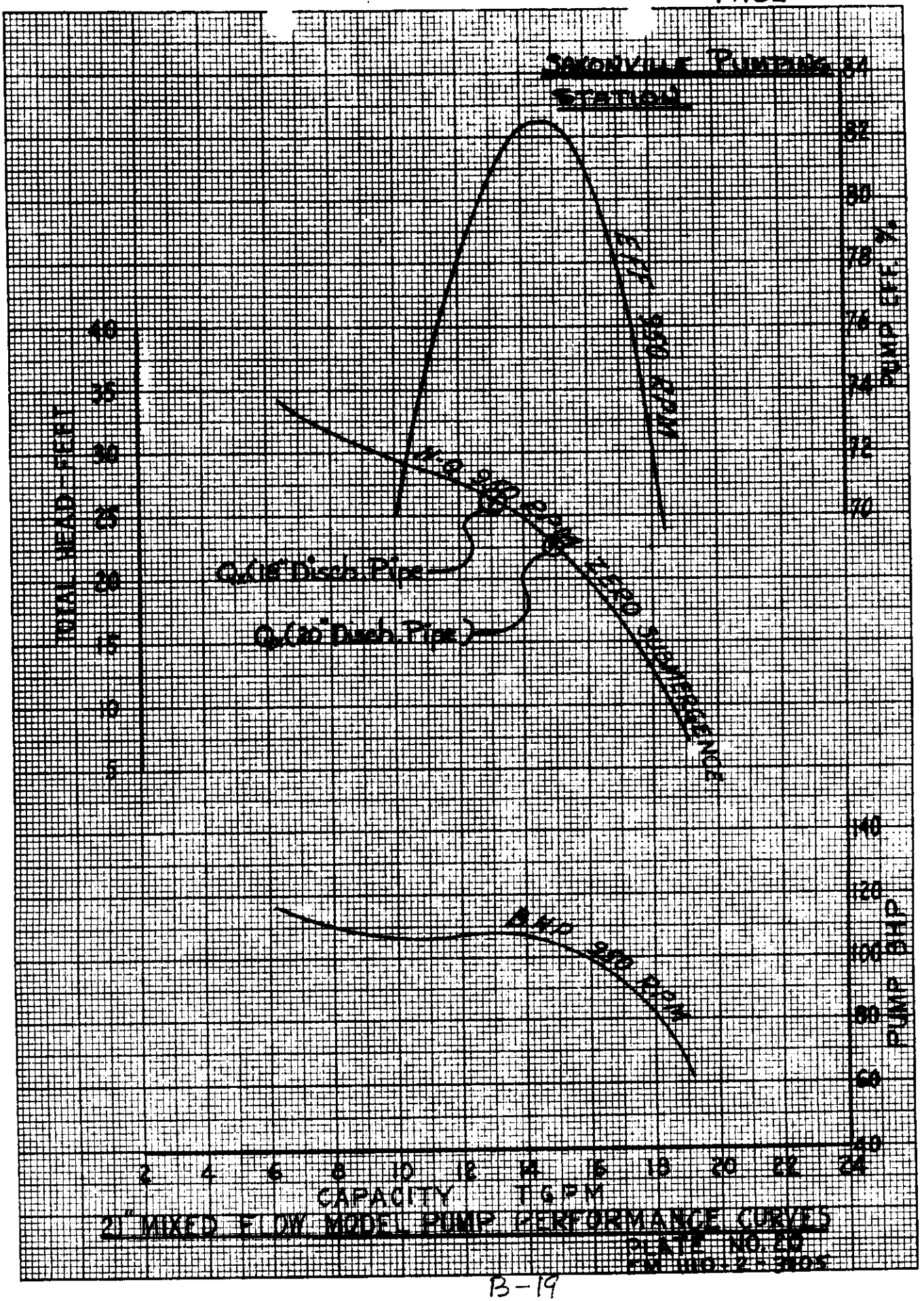
$N_S = \frac{1061\sqrt{11,700}}{23.5^{3/4}} = 10,752$ rpm

Suction Hd = +2

20" DISCHARGE PIPE

$$N_s = \frac{1132\sqrt{10,290}}{23.5^{3/4}} = 10,758 \text{ rpm}$$

Suction Hd = +2

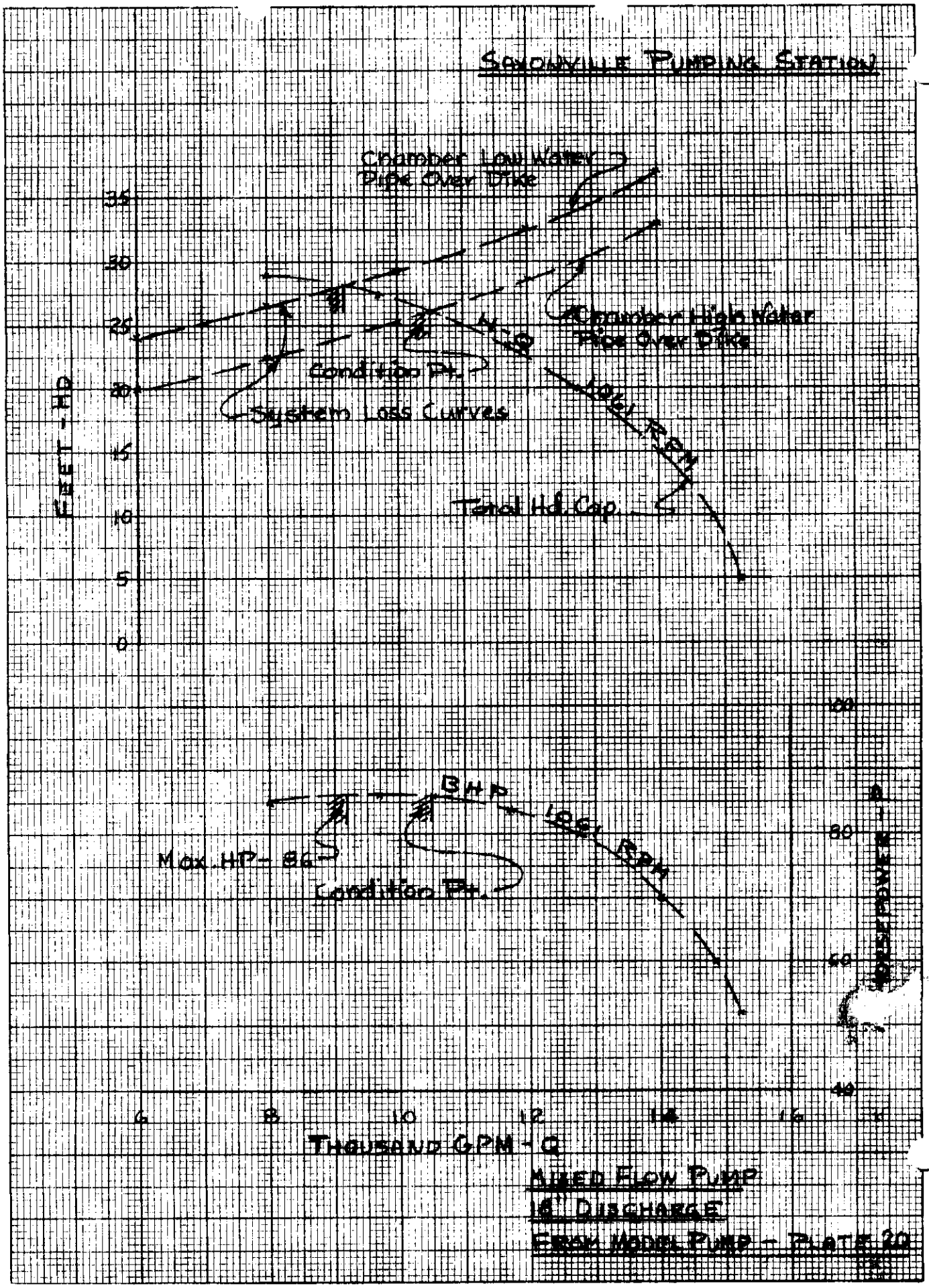


21" MIXED FLOW MODEL PUMP PERFORMANCE CURVES

PLATE NO. 20

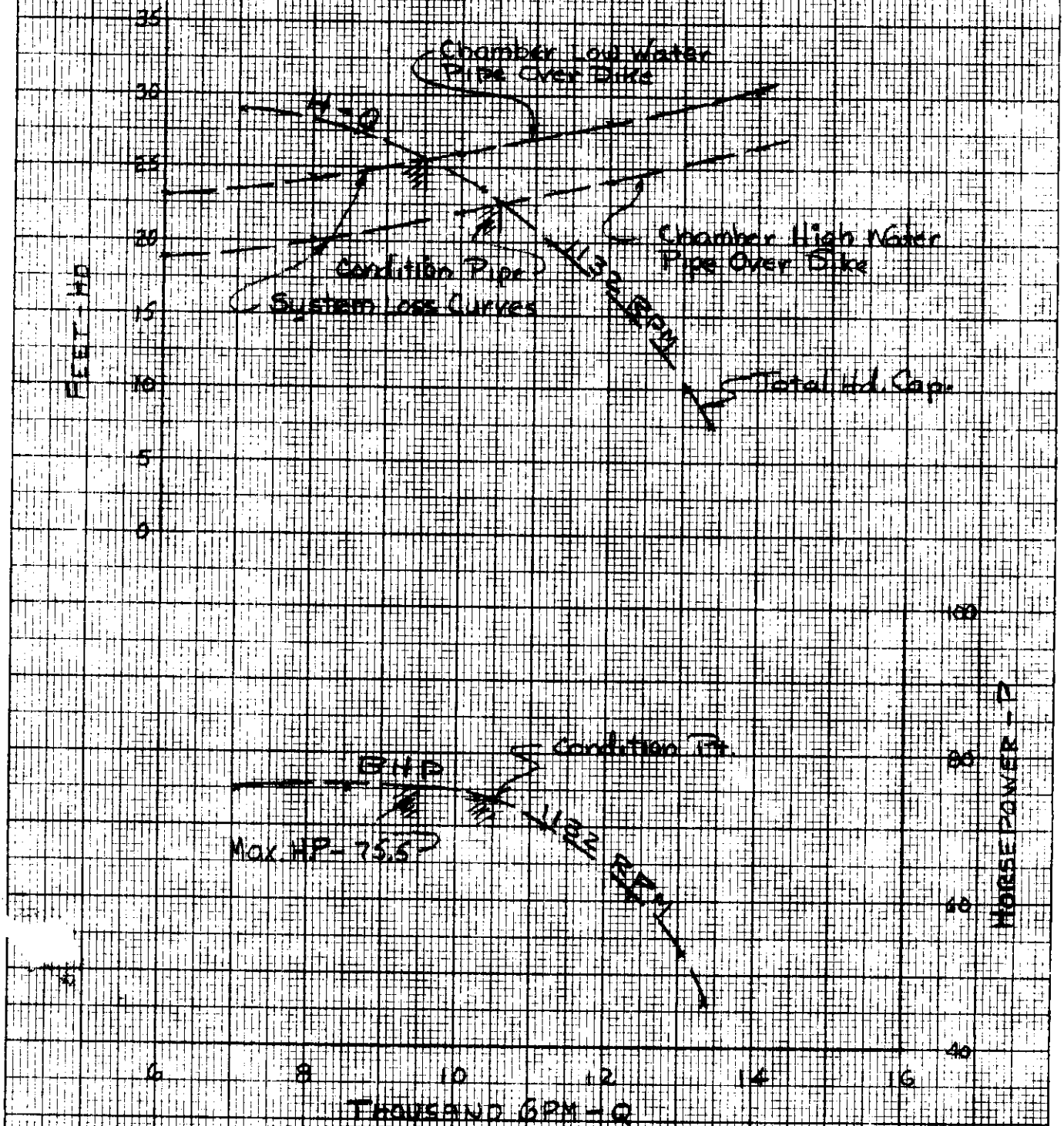
FM NO. 2-3105

K+E 10 X 10 TO ½ INCH 46 1323
7 X 10 INCHES MADE IN U.S.A.
KEUFFEL & ESSER CO.



MIXED FLOW PUMP
18" DISCHARGE
FROM MODEL PUMP - PLATE 20

SPRINGVILLE PUMPING STATION



MIXED FLOW PUMP
20" DISCHARGE
FROM MODEL PUMP - PLATE 20

KEUFFEL & ESSER CO

SUBJECT SAXONVILLE MASS. LOCAL PROTECTION

COMPUTATION SAXONVILLE PUMPING STATION

COMPUTED BY JTS CHECKED BY DATE Aug. 74

VALUES FROM CURVE - PLATE 21A

	Q_m	H_m	P_m
Max. Eff.	7,500	36.5	84.3
	8,450	30.0	81.0
	9,000	25.0	78.4
	9,350	20.0	75.0
	9,600	15.0	70.3
	9,800	10.0	64.0
	9,900	7.5	60.0
Values at cond. pt. 18"	8,880	26.1	79.2
" " " " 20"	9,180	22.6	76.8

ACTUAL MODEL RATIOS

18" Pump Discharge $\overline{MR}^2 = \frac{10,500}{8880} = 1.182$; MR = 1.087

20" " " $\overline{MR}^2 = \frac{10,500}{9,180} = 1.144$; MR = 1.069

PROTOTYPE VALUES

	Q_p	H_p	P_p
18" Pump Disch Max. Eff	8,870	36.5	99.7
	9,990	30.0	95.8
Cond. Pt.	10,500	26.1	93.6
	10,640	25.0	92.7
	11,060	20.0	88.7
	11,350	15.0	83.1
	11,590	10.0	75.7
	11,710	7.5	70.9
20" Pump Disch Max. Eff.	8,580	36.5	96.4
	9,660	30.0	92.6
	10,290	25.0	89.7
Cond. Pt.	10,500	22.6	87.8
	10,690	20.0	85.8
	10,980	15.0	80.4
	11,210	10.0	73.2
	11,320	7.5	68.6

VALUES FROM CURVE - PLATE 21A

	Q_m	H_m	P_m
Max. Eff.	7,500	36.5	84.3
	8,450	30.0	81.0
	9,000	25.0	78.4
	9,350	20.0	75.0
	9,600	15.0	70.3
	9,800	10.0	64.0
	9,900	7.5	60.0
Values at cond. pt. 18"	8,880	26.1	79.2
" " " " 20"	9,180	22.6	76.8

ACTUAL MODEL RATIOS

18" Pump Discharge $\overline{MR}^2 = \frac{10,500}{8880} = 1.182$; MR = 1.087

20" " " $\overline{MR}^2 = \frac{10,500}{9,180} = 1.144$; MR = 1.069

PROTOTYPE VALUES

	Q_p	H_p	P_p
18" Pump Disch Max. Eff	8,870	36.5	99.7
	9,990	30.0	95.8
Cond. Pt.	10,500	26.1	93.6
	10,640	25.0	92.7
	11,060	20.0	88.7
	11,350	15.0	83.1
	11,590	10.0	75.7
	11,710	7.5	70.9
20" Pump Disch. Max. Eff.	8,580	36.5	96.4
	9,660	30.0	92.6
	10,290	25.0	89.7
Cond. Pt.	10,500	22.6	87.8
	10,690	20.0	85.8
	10,980	15.0	80.4
	11,210	10.0	73.2
	11,320	7.5	68.6

18" DISCHARGE PIPE

$N = \frac{1288}{1.087} = 1185$ rpm

Impeller diam. = 13.438 × 1.087 = 14.61 in

From plotted curve

Q = 10,500 gpm	
P @ rated Q = 94 HP	
BHP required by pump	= 94
+ 10%	= 103
Loss in reduct. gear + 4%	= 4
BHP at engine coupling	= 107 HP (Continuous Rating)
P_{max} (Low Water)	= 96
+ 10%	= 106
Loss in reduct. gear + 4%	= 4
BHP at engine coupling	= 110 HP (Maximum Hp.)

20" DISCHARGE PIPE

$N = \frac{1288}{1.069} = 1205$ rpm

Impeller diam = 13.438 × 1.069 = 14.36 in

From plotted curve

Q = 10,500 gpm	
P @ rated Q = 88 HP	
BHP required by pump	= 88
+ 10%	= 97
Loss in reduct. gear + 4%	= 4
BHP at engine coupling	= 101 HP (Continuous Rating)
P_{max} (Low Water)	= 90
+ 10%	= 99
Loss in reduct gear + 4%	= 4
BHP at engine coupling	= 103 HP (Maximum HP.)

SPECIFIC SPEED

18" DISCHARGE PIPE

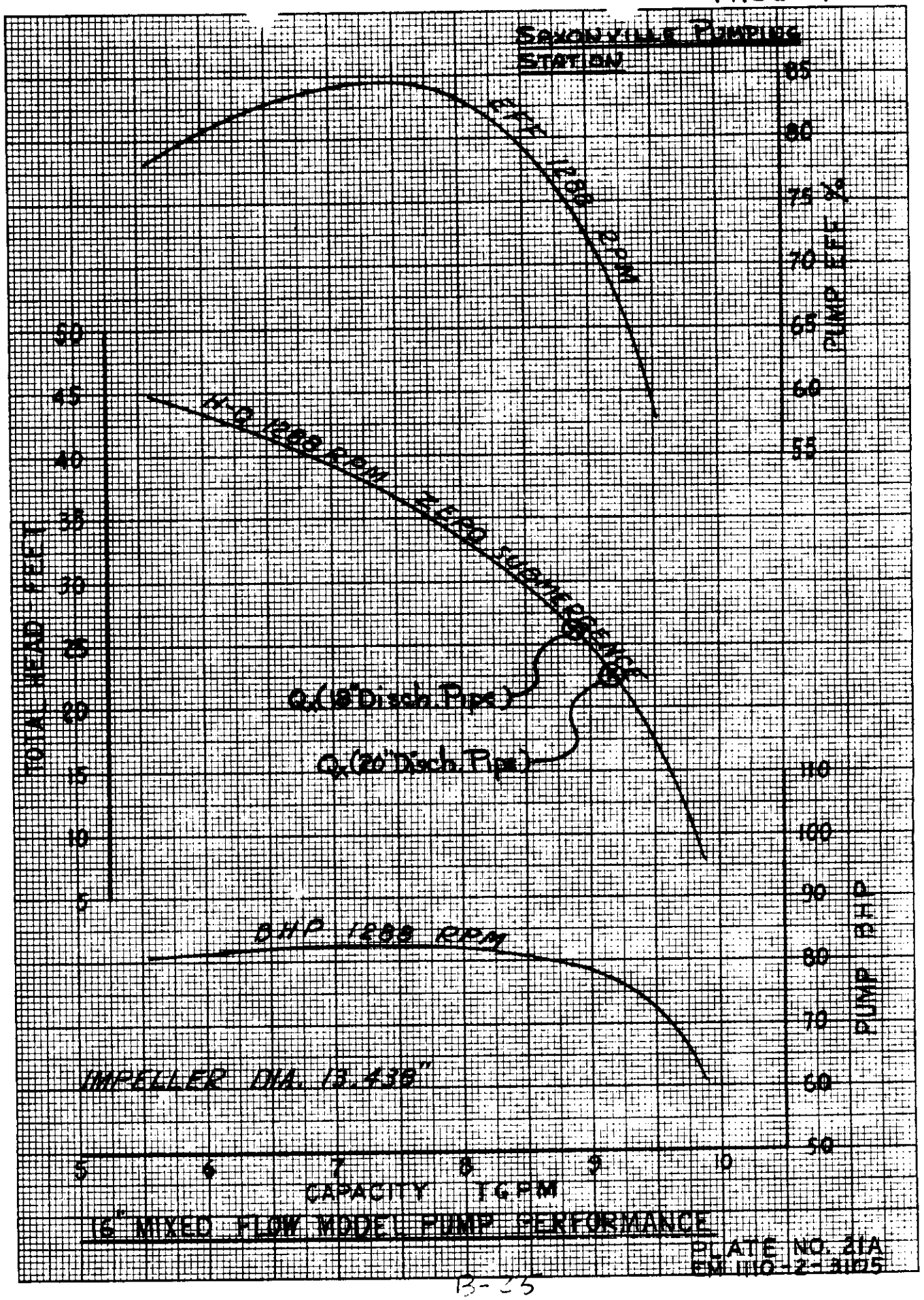
$N_s = \frac{1185\sqrt{8870}}{36.5^{3/4}} = 7{,}515$ rpm

Suction Hd = +2

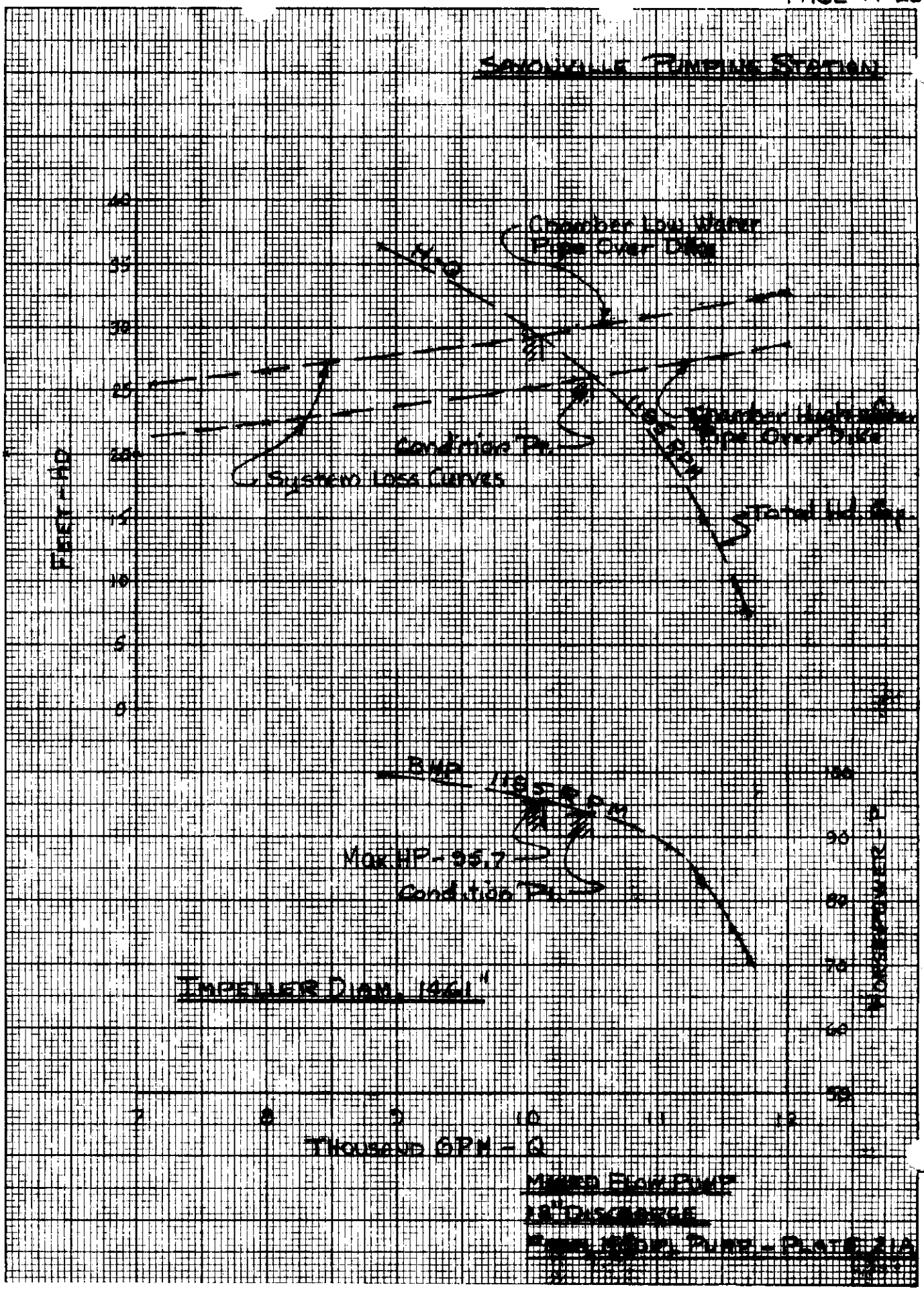
20" DISCHARGE PIPE

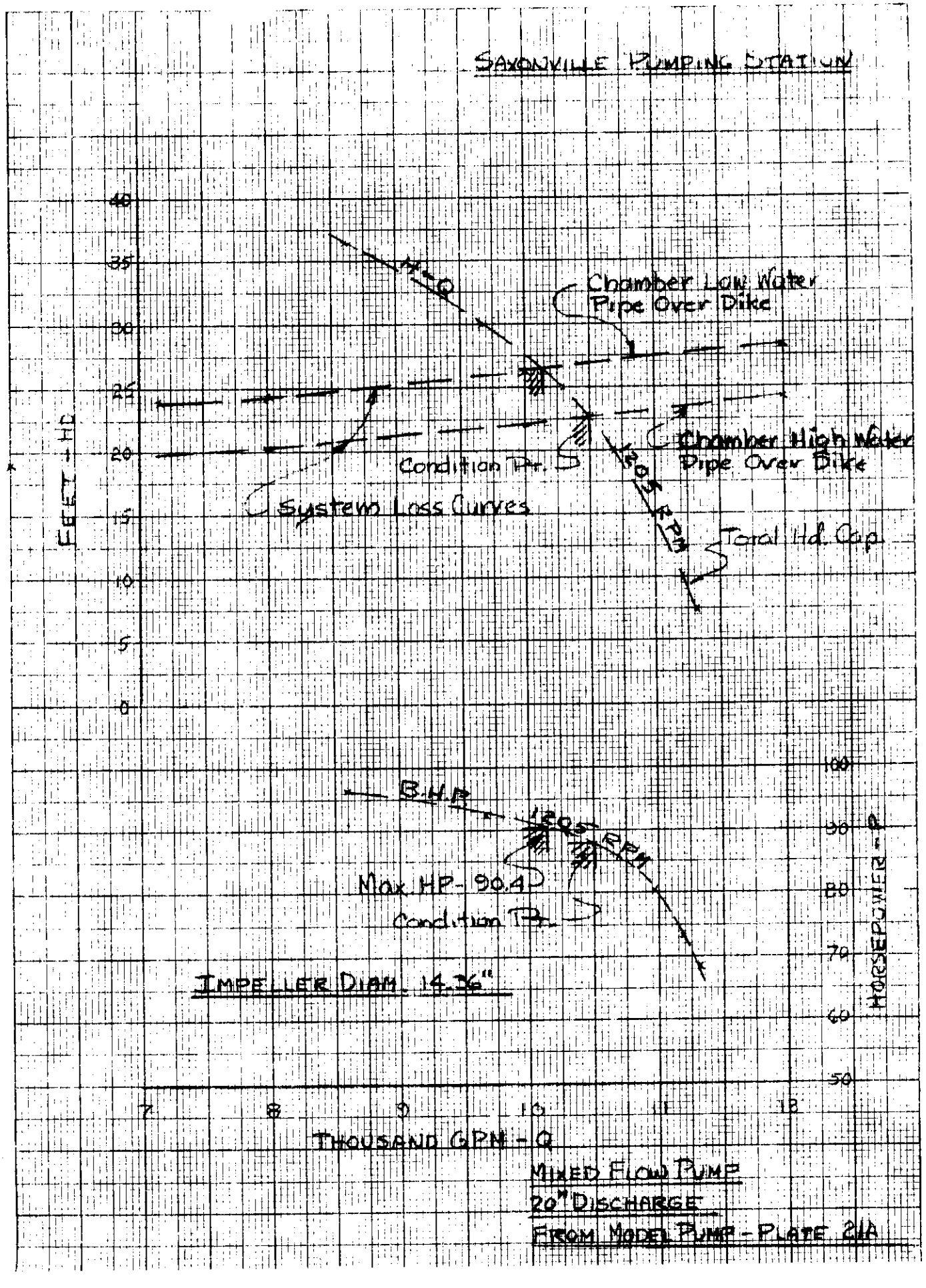
$$N_s = \frac{1205\sqrt{8580}}{36.5^{3/4}} = 7516 \text{ rpm}$$

Suction Hd = +2



K+E 10 X 10 TO ½ INCH 46 1323
7 X 10 INCHES MADE IN U.S.A.
KEUFFEL & ESSER CO.


SAVONVILLE PUMPING STATION
Chamber Low Water
Pipe Over Dike
Chamber High Water
Pipe Over Dike
Condition Pt.
System Loss Curves
Total Hd. Req.
BHP
Max HP - 95.7
Condition Pt.
IMPELLER DIAM. 14.61"
HEAD - FT.
HORSEPOWER
THOUSAND GPM - Q
MIXED FLOW PUMP
18" DISCHARGE
MODEL PUMP - PROTOTYPE



MIXED FLOW PUMP
20" DISCHARGE
FROM MODEL PUMP - PLATE 21A

KEUFFEL & ESSER CO

PUMP OPERATING CYCLE

Pipe Storage = 67,474 gallons
Sump " = 12,686 "

Total 80,160 "

Pump capacity = 10,500 gpm

Operating time (no inflow) = $\frac{80,160}{10,500}$ = 7.6 minutes

Min. Pump operating cylce (operating at full speed)
Inflow = 50% of pump capacity)

Time to empty = $\frac{80160}{5250}$ = 15.3 minutes

Time to refill = $\frac{80,160}{5250}$ = 15.3 "

Time between engine starts = 30.6 minutes

This is not to frequent starting of engine, however the pumps are diesel driven and may be operated at less than full speed so that the capacity approximately matches inflow, if desired to practically eliminate intermittent operation.