T.S.B.

CONNECTICUT RIVER FLOOD CONTROL PROJECT

CHICOPEE, MASS.

CONNECTICUT & CHICOPEE RIVERS, MASSACHUSETTS

ANALYSIS OF DESIGN
FOR
LOCAL PROTECTION WORKS

ITEM C. 3a - HIRED LABOR

DIKE WEST OF B. & M. R.R.

DECEMBER 1939

CORPS OF ENGINEERS, U. S. ARMY

U.S. ENGINEER OFFICE PROVIDENCE, R.I.

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DIKE WEST OF BOSTON & MAINE RAILROAD

CHICOPEE, MASS.

CORPS OF ENGINEERS, U. S. ARMY

UNITED STATES ENGINEER OFFICE, PROVIDENCE, RHODE ISLAND

CHICOPEE DIKE

C.3a. DIKE WEST OF B. & M. RR.

ANALYSIS OF DESIGN

CONTENTS

CHICOPEE DIKE

PERTINENT DATA

Location. - Connecticut River, Chicopee, Massachusetts.

Area Protected.................7.8 acres.

Limits of Dike Design...Station 0+00 to 16+28.93

Elevations. - Above mean sea level.

Top of wall	70.6
Top of dike	72.6

Embankment. - Station 5+65 to 16+28.93.

Total length of dike	1063.93 ft.
Total impervious fill	14,800 cu. yds.
Total random fill	30,800 " "
Total sheet steel piling	22,114 " "
Total riprap, hand-placed	1,360 " "
Total rock fill	1,500 " "
Total gravel bedding	1,790 " "
Total topsoil	2,230 " "

Wall and Stop-Log. - Station 0+00 to 5+80.19.

Total length of wall	580.19 ft.
Total concrete	1,168 cu. yds.
Total concrete piling	4,112 lin. ft.

I. INTRODUCTION

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A. AUTHORIZATION AND PAST REPORTS. - The dike on the south bank of Chicopee River, west of the Boston and Maine Railroad, (Item C.3a) is a part of the Chicopee dike project, which is authorized under the Flood Control Act approved June 28, 1938. It is a part of the Connecticut River Flood Control Plan recommended by the District Engineer in "Report of Survey and Comprehensive Plan for Flood Control in the Connecticut River Valley," dated March 20, 1937, approved by the Chief of Engineers November 29, 1937, and published as House Document No. 455, 75th Congress, 2nd session.

B. BRIEF DESCRIPTION OF DIKE AND APPURTENANT STRUCTURES. - The "Fiscal Year 1940 Unit", Item C.3a, covered by this analysis will consist of a concrete flood wall 580± feet long between Stations 0+00 and 5+80.19, and an earth dike 1064± feet long between Stations 5+65 and 16+28.93. There is one secondary road ramp over the dike, and a stop-log structure at the Boston and Maine Railroad. The work will include a sump and discharge conduit for a pumping station. The pumps will be located inside the plant of the Moore Drop Forge Company and will be provided by this firm.

II. SELECTION OF SITE

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A. GENERAL LOCATION. - The "Fiscal Year 1940 Unit", Item C.3a, is located in the western portion of the City of Chicopee on the south bank of the Chicopee River, west of the Boston and Maine Railroad, and approximately 1,200 feet from the eastern edge of the Connecticut River channel in its normal stage height. The protection works begin at the Boston and Maine Railroad on the south bank of the Chicopee River and proceed in westerly and southerly directions along the line of an existing dike until it reaches the branch line of the Boston and Maine Railroad. The dike crosses this railroad and follows it, in a southeasterly direction to the main line of the railroad, at a point about 800 feet south of the starting point.

B. ALINEMENT. - The alinement of the dike and flood wall was determined with regard to the topography, protection afforded and by economic studies.

III. GEOLOGICAL INVESTIGATION

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A. NATURE OF VALLEY. - The Chicopee River Valley, east of the confluence of the Chicopee and Connecticut Rivers, is entrenched in thick unconsolidated formations of glacial sediments. At its mouth the Chicopee River is entrenched in the flood plain deposits of the Connecticut River. The dike site under consideration is located in this stretch. About 1/3 mile upstream of the site the river has eroded to bedrock and exposed slightly inclined shale strata. At this point the rock surface dips steeply toward the deeply buried rock valley of the Connecticut River.

B. METHOD AND EXTENT OF EXPLORATIONS. - Subsurface explorations were accomplished by means of bore holes. Soil samples of 1-1/2-inch diameter were obtained at intervals, in some cases continuously, by means of standard sampling equipment. One large boring was completed for the purpose of obtaining 4-7/8-inch diameter undisturbed samples. At some bore holes, attempts were made to obtain information on the compactness of foundation strata by noting the number of blows required to drive the sampling spoon a distance of one foot. The location and records of foundation explorations are shown on Plate No. 2 subtitled "Subsurface Explorations".

C. SITE. - The upper portion of the foundation is a fill varying in thickness from about 8 feet to 21 feet. Fronting on the Chicopee River, this fill contacts pervious sand and gravel (Class 5), which in turn contacts impervious silt and clay at about river level. The section of dike fronting on the Connecticut River is likewise underlaid by the same pervious strata, but here it occurs at a slightly greater depth, and is blanketed by flood plain deposits of fine sand, silt, and gravel (Classes 6, 7, 8, 9, and 10). The impervious silt and clay formation, mentioned above, underlies the whole area, and is part of the very extensive glacial lake deposits of the Connecticut Valley in this vicinity. These materials occur in alternating thin bands of silt and clay (Classes 10 and 12C) and are more correctly called varved clays. All test data made on foundation samples are indicated on the geologic section, Plate No. 3.

D. NATURE OF EXCAVATIONS. - Excavations for the wall footings will be made in a fill material. Those for the earth embankment exploration and cut-off trench will be made in natural deposits chiefly of sand and silt.

E. SUBSURFACE SEEPAGE. - Seepage through the dike is reduced to a small quantity by use of a sheet pile cut-off under the concrete wall along the Chicopee River and by the presence of a natural silt blanket on the foreshore and under the fill along the earth dike section. Assuming a maximum flood level of elevation 70.6 (top of concrete wall), the seepage has been estimated at 130 gal./min. for the entire dike from Stations 0+00 to 16+28.93. Plate Nos. 10 and 11 show the sections considered in estimating seepage. They also show relative values of coefficients of permeability. Results of seepage computation are summarized in Table No. 1, on the following page.

TABLE NO. 1

ESTIMATED TOTAL SEEPAGE

Dike West of B.&M. RR.
Chicopee, Mass.

Section	Length of Section L ft.	Seepage per ft. q cu. ft./min./ft.	Basis of Estimate	Total Seepage for Section Q = q x L cu. ft./min.
Concrete Wall Sta. 0+00 - Sta. 6+00	600	0.01	Flow Net	6.0
Earth Dike Sta. 6+00 - Sta. 8+40	240	0.015	Flow Net	3.5
Sheet Pile Section at Stop-Log Structure Sta. 8+40 - Sta. 9+40	100	0.01	Estimated	1.0
Earth Dike Sta. 9+40 - Sta. 16+40 Natural silt cover much greater here.	700	0.01	Estimated	7.0

Total Seepage, Sta. 0+00 - Sta. 16+40: 17.5 cu.ft./min.
or
130 gals./min.

IV. HYDRAULIC DESIGN

IV. HYDRAULIC DESIGN

A. DESIGN FLOOD. - The design flood on which the dike grade is based is the maximum predicted flood reduced by the 20 reservoirs included in the approved Comprehensive Plan of reservoirs for the Connecticut River. The determination of the maximum predicted flood is discussed in Appendix 1 of "The Report of Survey and Comprehensive Plan for the Connecticut River" dated March 20, 1937. It has a peak discharge at Chicopee of 312,000 c.f.s., approximately 15 per cent greater than the maximum flood of record. The following table lists the adopted grades for the top of the earth dike.

DESIGN GRADES

Location	Dike Type	Dike Station	Dike Grade
B. & M. Railroad on Chicopee River	Conc. Wall	0+00	70.6
East of Moore Drop Forging Company	" "	5+65	70.6
" " " " " "	Earth	5+65	72.6
South " " " " "	"	16+29	72.6

B. FREEBOARD. - The freeboard for the earth dike is 5 feet and for the concrete wall 3 feet, as recommended by the Board of Engineers for Rivers and Harbors.

C. LOCAL CONDITIONS. - During flood periods the Connecticut River, in addition to its main channel, flows over the local flood plain. This is augmented by flood waters from the Chicopee River. The ground elevations of this flood plain vary from approximately 60.0 to 66.0 mean sea level. During the major flood in 1936, the elevation of the flood water was about 70.0 mean sea level, which overtopped an existing dike by about 5 feet. The effect of the proposed dikes in raising the water surface elevation during floods will be negligible since the dikes encroach but slightly on the flood plain.

D. PUMPING REQUIREMENTS. - The drainage area tributary to the local protection works is 9 acres. The area consists of the factory buildings of the Moore Drop Forge Company, railroad tracks, and undeveloped land. A 5-year, 2-hour storm of 1.88 inches total rainfall with a runoff coefficient of 0.6 was adopted. The resulting runoff which would require pumping at time of high river stages is 5 second-feet. Two pumps are proposed each capable of discharging about 75 per cent of this amount, or 7.5 second-feet total. These pumps will be installed and operated by the Moore Drop Forge Company.

V. LABORATORY AND FIELD INVESTIGATION OF SOILS

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A. CLASSIFICATION OF MATERIALS. - Soils, based on grain sizes, have been classified into 16 classes and are shown graphically on Plate No. 5 and described in Table No. 2 on the following page. Soils of uniform grain size are designated by even numbers, soils of variable grain size by odd numbers and grain sizes of materials follow the M.I.T. classification except that the size demarcation between silt and coarse clay is not 0.002 mm. but varies from 0.006 mm. to 0.0006 mm.

B. GRAIN SIZE ANALYSIS. - Grain size curves of samples have been obtained from sieve and hydrometer analysis and the soil classified. Sedimentary units of soil were grouped and drawn up as shown on Plate No. 3 titled "Geologic Section."

C. WATER CONTENT AND VOID RATIO. - The water content and void ratio of the materials in their natural states from the proposed dike foundations and borrow areas have been determined.

D. PERMEABILITIES. - Permeabilities for the various classes of foundation and embankment materials are shown on Plate Nos. 10 and 11. For embankment materials these values were determined by tests on samples from borrow area. For materials in the foundation permeabilities were estimated from examination of samples and experience with similar materials.

E. SHEAR AND COHESION. - Direct shear tests have been made on materials from the proposed dike foundations and on borrow area materials. With a 20-foot layer of granular material between dike and underlying silt, stability is not a problem.

PROVIDENCE SOIL CLASSIFICATION
U. S. ENGINEER OFFICE
PROVIDENCE, R. I.

TABLE NO. 2

CLASS	DESCRIPTION OF MATERIAL
1	Graded from Gravel to Coarse Sand. - Contains little medium sand.
2	Coarse to Medium Sand. - Contains little gravel and fine sand.
3	Graded from Gravel to Medium Sand. - Contains little fine sand.
4	Medium to Fine Sand. - Contains little coarse sand and coarse silt.
5	Graded from Gravel to Fine Sand. - Contains little coarse silt.
6	Fine Sand to Coarse Silt. - Contains little medium sand and medium silt.
7	Graded from Gravel to Coarse Silt. - Contains little medium silt.
8	Coarse to Medium Silt. - Contains little fine sand and fine silt.
9	Graded from Gravel to Medium Silt. - Contains little fine silt.
10	Medium to Fine Silt. - Contains little coarse silt and coarse clay. Possesses behavior characteristics of silt.
10 C	Medium Silt to Coarse Clay. - Contains little coarse silt and medium clay. Possesses behavior characteristics of clay.
11	Graded from Gravel or Coarse Sand to Fine Silt. - Contains little coarse clay.
12	Fine Silt to Clay. - Contains little medium silt and fine clay (colloids). Possesses behavior characteristics of silt.
12 C	Clay. - Contains little silt. Possesses behavior characteristics of clay.
13	Graded from Coarse Sand to Clay. - Contains little fine clay (colloids). Possesses behavior characteristics of silt.
13 C	Clay. - Graded from sand to fine clay (colloids). Possesses behavior characteristics of clay.

F. COMPACTION. - Compaction tests based on the Proctor analysis procedure have been made on impervious embankment materials. Typical results are shown on Plates Nos. 7 and 8.

G. COMPRESSIBILITY. - Undisturbed samples have been taken from one 6-inch bore hole but consolidation tests are not yet complete. Preliminary results show the compressible layer of alternately banded silt and clay similar to that on the opposite or north bank of the river where dike settlements of 3 to 6 inches were computed. As the layer of varved silt is thinner on the south bank, it is expected that settlements for this dike will not exceed 6 inches. There will be no lateral movement of the foundation under the protection works.

H. OTHER TESTS. - Other tests include Atterberg limits, extraction for solubility and specific gravity.

J. BORROW AREA. - Location of suitable materials within economic haul has been a major difficulty. Pervious material near the dike is at a prohibitive depth and below the water table. Impervious material on the foreshore is either too wet for economic compaction or too pervious.

Of the various borrow areas explored and used in previous dike construction on the north side of the Chicopee River, Borrow Area "B" has been selected as most suitable. This is the nearest area to the site of the present work (0.9 mile haul to center of operations). Acceptable materials are available in adequate quantity. The area is located at a sufficient elevation above the river to be unaffected by floods.

In general, three different types of materials are available in this area:

(1) Random impervious in the upper portion of the pit, a very

compact glacial deposit graded from gravel to fine silt, Classes 7, 9, and 11-9. This material has a high angle of internal friction ($\phi = 40°$) and is reasonably impervious ($k = 0.1$ to 5×10^{-4} cm/sec). Because of the high shearing resistance, this material will be used as a riverbank fill along the concrete wall next to the river where the slope is steepest (1 on 1-3/4). Since a sheet pile cut-off is used in this section, permeability is not a major factor, (see Plate No. 9).

(2) Random pervious in lower 20 to 40 feet of the pit, layers of coarse to fine sand with some gravel lenses, Classes 2, 4, 6, 6-4, and 5. This material will make up the bulk of the earth portion of the dike with the more pervious portion being placed next to the landside toe, see Plate Nos. 10 and 11.

(3) Impervious in upper 10 feet on east side of borrow area, medium to fine silt, Classes 8, 10, and 8-10. This material will be used in the impervious blanket on the earth dike section, (see Plate Nos. 10 and 11).

Plate No. 6 shows the range in grain size for these materials. Compaction characteristics are shown in Plate Nos. 7 and 8, Table No. 3, on the following page, summarizes data on this borrow area.

These different materials can be obtained from the one borrow area by operating the pit with benches at two levels and stationing a trained soils inspector at the pit to route materials to the proper location in the dike.

TABLE NO. 3

BORROW MATERIALS AVAILABLE

Borrow Area "B" - Granby Road
Dike West of B.&M. RR.
Chicopee, Mass.

Type of Material and Intended Use	Occurrence	Volume Available Cu.Yds.	Volume Required Cu.Yds.	Test Data Indicating Suitability			
				Classification	Water Content Natural	Water Content Optimum	Permeability $k \times 10^{-4}$ cm./sec.
Random Impervious Blanket along concrete wall	Upper 5 to 20 ft. of pit.	6,000 to 10,000	4,600	7, 9 and 11-9	3 - 10%	9%	0.1 - 5
Random Pervious Earth Dike Section	Lower 20 to 40 ft. of pit.	Over 23,000	23,000	2, 4, 5, 6 and 6-4	4 - 10%	None	100 - 200
Impervious Blanket on Earth Dike Section.	Upper 10 ft. East side of borrow area	Over 14,000	13,800	8, 10, and 8-10	15-25%*	20% *	0.01 - 1.0*

*Estimated values.
Investigations not complete

VI. DIKE DESIGN CRITERIA AND GENERAL DESIGN

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A. DIKE DESIGN CRITERIA. - The Chicopee dike embankment section calls for a large section of random material to be covered on the river-side by a blanket of impervious material. General design criteria for the Chicopee Dike include safety, stability and reduction of seepage and are as follows:

(1) The crest of the dike is at such a grade that overtopping at design flood and by wave action is eliminated.

(2) a. The slopes of the dike are such that they will be stable under all conditions.

b. For the Chicopee Dike sections between 10 to 20 feet in height, a water-side slope of 1 on 3 and a land-side slope of 1 on 2-1/2 are required. The slopes for dike sections greater than 20 feet in height are 1 on 3 for both sides. After a study of the flow of flood waters in both Chicopee and Connecticut Rivers, 12-inch hand-placed rip-rap over a 6-inch gravel bed was required over critical areas to prevent scour.

(3) a. The line of saturation is well within the land-side toe.

b. The dike is designed for a fairly low line of saturation and analyzing the Chicopee dike sections, the flood water will seep very slowly through the impervious blanket of low permeability to the random section where, because of a somewhat greater permeability, together with the influence of the toe drain, the line of saturation will drop below the land-side toe.

(4) Seepage and run-off behind the dike are collected by the

toe drain and existing catch basins. Such seepage through the foundation or dike as may occur will have a very low velocity and will be collected by the toe drain. The toe drain is connected to a manhole which also collects the surface water from existing catch basins. This manhole will be connected to a sump and pumping plant to be built by local interests.

(5) There is no possibility for free passage of water from riverside to landside. All pipes, conduits, etc. are removed from under the dike except one water main which has been in place for many years and is well below our cut-off trench.

(6) No material soluble in water is used in any part of the dike and soils laboratory tests have been made of borrow materials for solubility. (See Paragraph V. H.)

(7) The foundation under the dike is sufficiently stable to resist stresses due to the embankment load. (See Paragraph V. G.)

(8) Where the dike crosses an existing road, a ramp of adequate width and proper grade is provided for traffic over the ramp. The ramp, which is nearly perpendicular to the center line of the dike, is built of random material with cross-sectional slopes of 1 on 1-1/2 on the landside of the dike and, because of the saturated condition of the riverside ramp immediately following floods, with cross-sectional slopes of 1 on 2.

B. GENERAL DESIGN. - The design criteria for the riverbank fill and seepage cut-off between Stations 0+00 to 4+09± are as follows:

(1) a. The slope of the riverbank fill shall be stable under all conditions.

b. The slope of 1 on 1-3/4 is required. The angle of internal friction of the random material is 40° (see Paragraph V-J-1) under the most adverse condition.

(2) a. The riverbank fill shall encroach upon the river channel as little as possible.

b. The steepest slope of the riverbank fill including a factor of safety is 1 on 1-3/4 which extends into the present river channel approximately 13 feet.

(3) a. The riverbank fill shall be protected against scour.

b. The riverbank fill is to have 12 inches of riprap with a bed of 6 inches of gravel. The riverbank was severely scoured during the flood of September 1938 and approximately 20 feet of bank was lost. The bank erosion extended in one area right up to the facade of a brick industrial building. The riprap is considered sufficiently heavy to resist this scouring action. An added factor of safety is secured with the use of steel sheet piling cut-off which is to be driven sufficiently deep so that if the riprap and fill should be washed away, the steel sheet piling, which is an integral part of the wall, would act as a water-tight revetment or bulkhead.

(4) a. An adequate seepage path under the concrete wall on bearing piles shall be developed.

b. To guard against roofing under the base of the wall and subsequent piping, steel sheet piling is to be driven through the pervious strata 3 feet into the impervious clay. The protected area is subject to extremely heavy vibrations from the many powerful hammers of the Moore Drop Forging Company plants. These vibrations can be felt several hundred feet from the buildings and at one point the concrete wall is less than 25 feet from the hammers. Under these conditions, roofing may be expected. During the floods of March 1936 and September 1938 and before the existing dike was overtopped, many sand boils appeared, although the existing dike is only five feet high and the flood waters rose rapidly. These sand boils indicated a porous foundation. Economic studies of several riverbank sections including an impervious material blanket and a steel sheet piling cut-off through the river bottom gravel and stones, showed that the steel sheet piling cut-off as an integral part of the wall was the most economical. In addition the impact from the vibration would make the contact planes between earth and concrete unstable and create a definite seepage path. A heavy blanket of impervious material should have a slope of at least 1 on 2 and would extend so far into the river as to obstruct free flow in the river channel. The cinder fill and pickling baths for the products of the plants create a slight acid condition in the soil. This is being counteracted by the introduction of a lime trench at the top of the piling.

VII. STRUCTURAL DESIGN

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A. GENERAL. - The reinforced concrete construction in the Chicopee Dike, Item C.3a, includes three typical flood walls, two end cut-off walls, a railroad stop-log structure, and a drainage sump. The walls run from Station 0+00 to Station 5+80.19, and the stop-log structure permits the Boston and Maine Railroad trace to pass through the earth dike at Station 9+02.68.

The typical flood walls are of the cantilever type and the first runs from Station 0+12.5 to Station 1+81.0. It has a stem height of 8'-6" above the base. The second runs from Station 1+81.0 to Station 4+21.0 and has a stem height of 10'-0" above the base. The last runs from Station 4+21.0 to Station 5+49.0 and has a stem height of 11'-6" above the base. The concrete flood walls are located along the edge of a steep riverbank composed of recently placed cinders and with a slope of approximately 1 on 1-1/2. The foundation is subjected to constant vibration from the operations of the many hammers in the forging plant. Due to the compressibility of the foundation and the steepness of the riverbank finished slope, which will be 1 on 1.75, bearing piles are necessary. Wood piles are considered unsuitable because the permanent water table is well below the top of the piles. Concrete bearing piles are therefore proposed.

One cut-off wall runs from Station 5+49.0 to 5+80.19 and is a typical 8'-6" flood wall with three seep fins which are buried in the earth dike. The other cut-off wall runs from Station 0+00 to Station 0+12.5 and consists of a small wall with a step in the base. It is buried in the earth dike but has no seep fins as it is to be joined by later construction.

The stop-log structure consists of two dike retaining walls placed parallel to the track. The clear distance between them is 19'-0". The stems are 10'-11" high above the base in the middle and taper down on each side with the earth dike. Three seep fins are provided on the dike side of each wall to prevent excessive seepage or piping along the face of the concrete. Each wall has a vertical groove in the center of the trackside face. These grooves hold stop-logs 18'-10" long, which form a barrier to the water in time of flood. A concrete cap for the sheet piling forms a sill upon which the logs rest for about 4'-0" on each end. In the center, the sill drops about 4 inches below the base of rail to avoid impact of the rail on concrete when a train crosses. In time of flood this small slot around the tracks is to be blocked with sand bags.

For complete details concerning the walls see Plate No. 17.

For complete details concerning the stop-log structure see Plate No. 16.

B. SPECIFICATIONS FOR STRUCTURAL DESIGN.

(1) General. - The structural design of the flood wall has been executed, in general, in accordance with standard practice. The specifications which follow cover the conditions affecting the design for stability and for reinforced concrete.

(2) Unit Weights. - The following unit weights for materials were assumed in the structural design:

Water	62.5	pounds	per	cubic	foot
Dry earth	100	"	"	"	"
Saturated earth	125	"	"	"	"
Concrete	150	"	"	"	"
Steel	490	"	"	"	"
Timber	50	"	"	"	"

(3) Earth Pressures. - In computing active earth pressures, equivalent fluid pressures computed by use of Rankine's formula were used. They are as follows:

Earth, dry, equivalent liquid loading = 35 pounds per cu. ft.
Earth, saturated, " " " = 80 " " " "

In computing passive resistances, Rankine's formula was used with the angle ϕ = 34 degrees.

(4) Hydrostatic Uplift. - With the river up, hydrostatic uplift on the base is assumed as 100% on the riverside of the sheet pile and on the landside of the sheet pile it tapers from 50%, the difference between headwater and tailwater, to tailwater. With the river down, a uniform uplift from tailwater is assumed over the entire base.

(5) Overturning. - The resultant of all external loads, including hydrostatic uplift, but excluding base pressure, shall fall inside the outer pile.

(6) Sliding. - The total horizontal forces due to external loads shall not exceed the resistance available from passive pressures and shear on the concrete piles.

(7) Bearing. - The total bearing per pile, after the uplift has been subtracted, shall not exceed 30 tons.

(8) Frost Cover. - All footing bases shall be at least 4 feet beneath the surface of the ground to avoid heaving by frost action.

(9) Path of Creep. - When using a steel sheet pile cut-off for a concrete wall on bearing piles provided with a filter, the minimum path of creep shall be five times the difference in elevation of headwater and tailwater. When, however, the steel sheet piling length as determined by the head penetrates a clay or similar impervious stratum,

the minimum depth of penetration into the impervious material shall be 3 feet. The path of creep is defined as the perimeter of the structure lying below and between the earth surfaces on two sides of the wall.

(10) Reinforced Concrete. - In general, the design of the reinforced concrete was in accordance with the recommendations of the Joint Committee and the American Concrete Institute. Specifically, the working stresses are as follows:

a. Ultimate Strength. - The allowable working stresses in concrete are based on an average ultimate compressive strength of 3,400 pounds per square inch in 28 days.

b. Flexure. - Extreme fiber stress in compression = 800 pounds per square inch.

c. Shear.

Without special anchorage = 60 pounds per square inch.
With special anchorage = 90 " " " "

d. Bond.

Without special anchorage = 100 pounds per square inch
With special anchorage = 200 " " " "

e. Embedment.

Minimum embedment to develop bond = 40 diameters.

f. Ratio of Moduli of Elasticity.

$E_s/E_c = n = 12$

g. Protective Concrete Covering.

In lower face of footings = 4 inches
Other than in lower face of footings = 3 inches

h. Temperature Steel. - Minimum steel in any exposed face is 5/8"Ø bars spaced one foot on centers.

(11) Reinforcing Steel. - The steel assumed to be used is new billet steel, intermediate grade, deformed bars. The effective cross-sectional areas are taken as net, and the working stress used is as follows:

Tension, main steel = 18,000 pounds per square inch.

(12) Structural Steel. - The design of the steel structures has been governed by the Standard Specifications for Steel Construction of the American Institute of Steel Construction. Maximum allowable unit working stresses are as follows:

a. Flexure (tension or compression) = 18,000 pounds per square inch.

b. Shear = 12,000 pounds per square inch.

(13) Timber. - The structural timber to be used is select White Oak, surfaced four sides, and creosoted. The maximum allowable working stresses used are high, due to intermittent use and to the probability of support by sandbags. They are as follows:

a. Flexure (tension or compression) 1,750 pounds per square inch.

b. Shear (parallel to grain) 156 pounds per square inch.

c. Bearing (perpendicular to grain) 265 pounds per square inch.

(14) Concrete Piles. - The concrete piles shall hold an axial load of 30 tons. For further specifications see Plate No. 17.

C. BASIC ASSUMPTIONS FOR DESIGN. -

(1) Loadings. - In general, each member of a structure is designed to resist the most unfavorable combination of loadings. The assumed river high water is Elevation 72.6, and tailwater is assumed at Elevation 60.0. The wall is built to Elevation 70.6 with the idea that

future flashboards or sandbags may raise it to the design grade.

The critical case for design of the flood walls, is when the river is up to Elevation 72.6, and the tailwater is at Elevation 60.0. The critical case for the stop-log is when the river has receded, leaving a saturated dike to be retained.

(2) Flood Walls. - The stem and base of the walls were designed as simple cantilever beams fixed at the intersection of the stem and the base. The principal load on the stem is the horizontal pressure of the water. The principal load on the base is the difference between the weights of earth, concrete, or water acting down, and the uplift and base pressure acting up.

(3) Stop-Log Structure. - The walls were designed with the stem and base acting as simple cantilever beams about the junction of the stem and base. The principal load on the stem is the lateral pressure of the saturated earth dike. The principal load on the base is the difference between the weights of earth and concrete acting down, and bearing and uplift acting up. The three seep fins on each wall will act as counterforts, but the stop-log is structurally safe without them.

(4) Stop-Logs. - The load on the timber stop-logs is due to the head of the water. They are designed as simple beams supported at each end.

VIII. CONSTRUCTION PROCEDURE

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A. FIELD OPERATIONS. - Since this is a Hired Labor project, the work must be completed by June 30, 1940. Assuming construction will commence on December 15, 1939, it is contemplated that the work will be completed by June 30, 1940.

The following tabulation presents a proposed time limit of operations:

CONSTRUCTION PROGRAM

Designation	Quantity	Time limit of operations (Calendar Yr.) (1939-40)	No. of working days	Daily rate of Construction
Preparation of Site	-	Dec.15-Dec.30	10	-
Excavation	8,300 c.y.	Dec.20-Jan.31	25	332 c.y.
Placing Steel Sheet Piling	22,114 s.f.	Dec.26-Feb.25	40	555 s.f.
Concrete Piling	4,152 l.f.	Jan.20-Apr.15	55	76 l.f.
Placing Embankment	45,600 c.y.	Apr.1-June 15	30	1,520 c.y.
Concrete in Walls and Stop-log	1,150 c.y.	Apr.1-June 15	45	25 c.y.
Riprap	1,360 c.y.	May 1-June 15	35	40 c.y.
Placing of Topsoil	2,230 c.y.	May 20-June 15	15	159 c.y.
Placing of Gravel Bedding	1,790 c.y.	Apr.15-June 15	45	40 c.y.

B. INSPECTION AND TESTS. - The usual field inspection of all portions of the construction work will be made. Progress reports, including log of work accomplished will be kept.

Field and laboratory tests of embankment materials, concrete and other materials will be made in order to control the quality of the work.

IX. SUMMARY OF COSTS

IX. SUMMARY OF COSTS

The total estimated construction cost of the Chicopee Dike, Item C.3a is $150,000, including 10% for contingencies and 15% for engineering and overhead, and is distributed as follows:

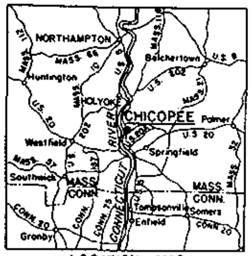
a.	Embankment	$41,776	
b.	Reinforced concrete	56,111	
c.	Drainage	4,150	
d.	Steel sheet piling	28,149	
e.	Riprap, hand-placed	10,387	
f.	Rock fill	5,257	
g.	Miscellaneous	4,170	
	Total		$150,000

X. PLATES

X. LIST OF PLATES

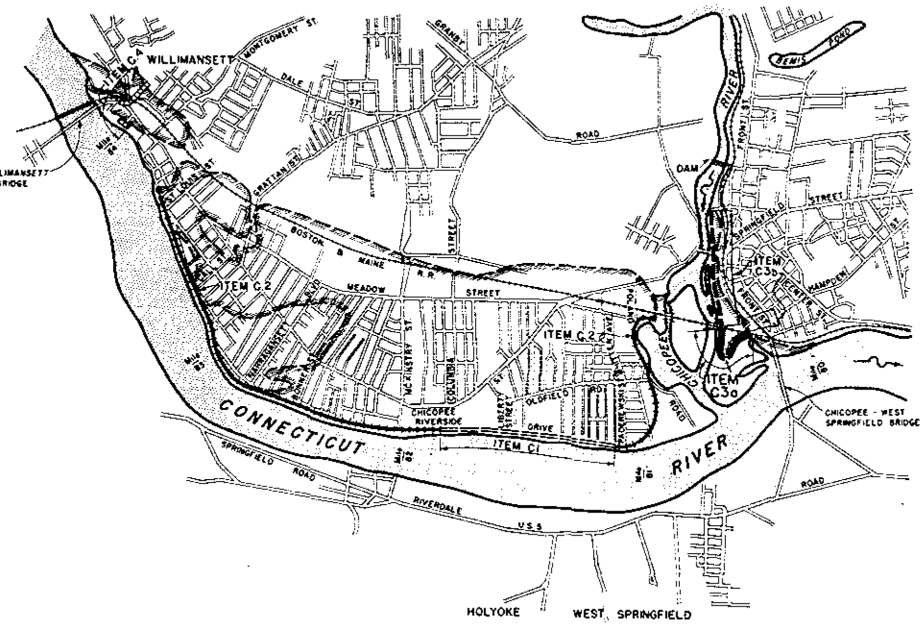
WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY



LOCATION MAP

SCALE 1" = 8 MILES



VICINITY MAP

SCALE 1" = 1500'

LEGEND

- Item C.1 Initial Fiscal Year 1939 Unit, Dike
- Item C.2 Fiscal Year 1939 Section, Dike
- Item C.3a Fiscal Year 1940 Unit, West of B.&M. R.R. South Bank Chicopee River
- Item C.3b Future Construction, Fiscal Year 1940 Unit, South Bank Chicopee River
- Item C.4 Fiscal Year 1940 Section Willimansett Dike
- Overflow Limits, March 1936 Flood

KEY	DATE	REVISION (Indicated by △)	REV.BY	CK.BY	AP.BY

CONNECTICUT RIVER FLOOD CONTROL

CHICOPEE DIKE

FISCAL YEAR 1940 UNIT

PROJECT LOCATION AND INDEX

CONNECTICUT & CHICOPEE RIVERS — MASSACHUSETTS

IN 21 SHEETS — SCALE: 1 IN = 1500 FT. (1500' 0 1500' 3000') — SHEET NO. 1

U. S. ENGINEER OFFICE, PROVIDENCE, R. I. — NOV. 1939

SUBMITTED: [signature] SENIOR ENGINEER, HEAD, DESIGN SECTION
APPROVAL RECOMMENDED: [signature] PRINCIPAL ENGINEER, CHIEF OF ENGINEERING DIV.
APPROVED: [signature] COL. CORPS OF ENGINEERS, DISTRICT ENGINEER

DESIGNED: [signature] ASST. ENGINEER
DRAWN W.S.R. TRACED W.L. CHECKED #75

FILE NO. CT-4-1995

C.3a

(1)

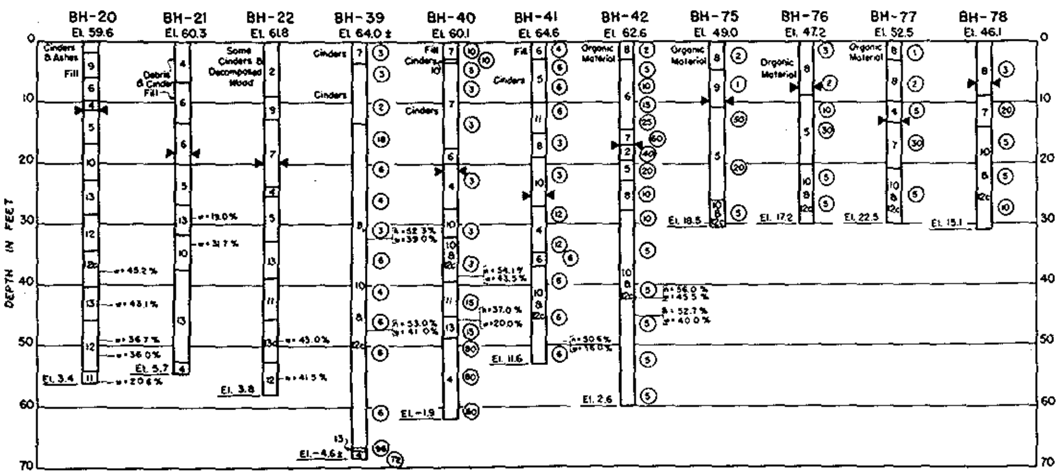
PLATE NO. I

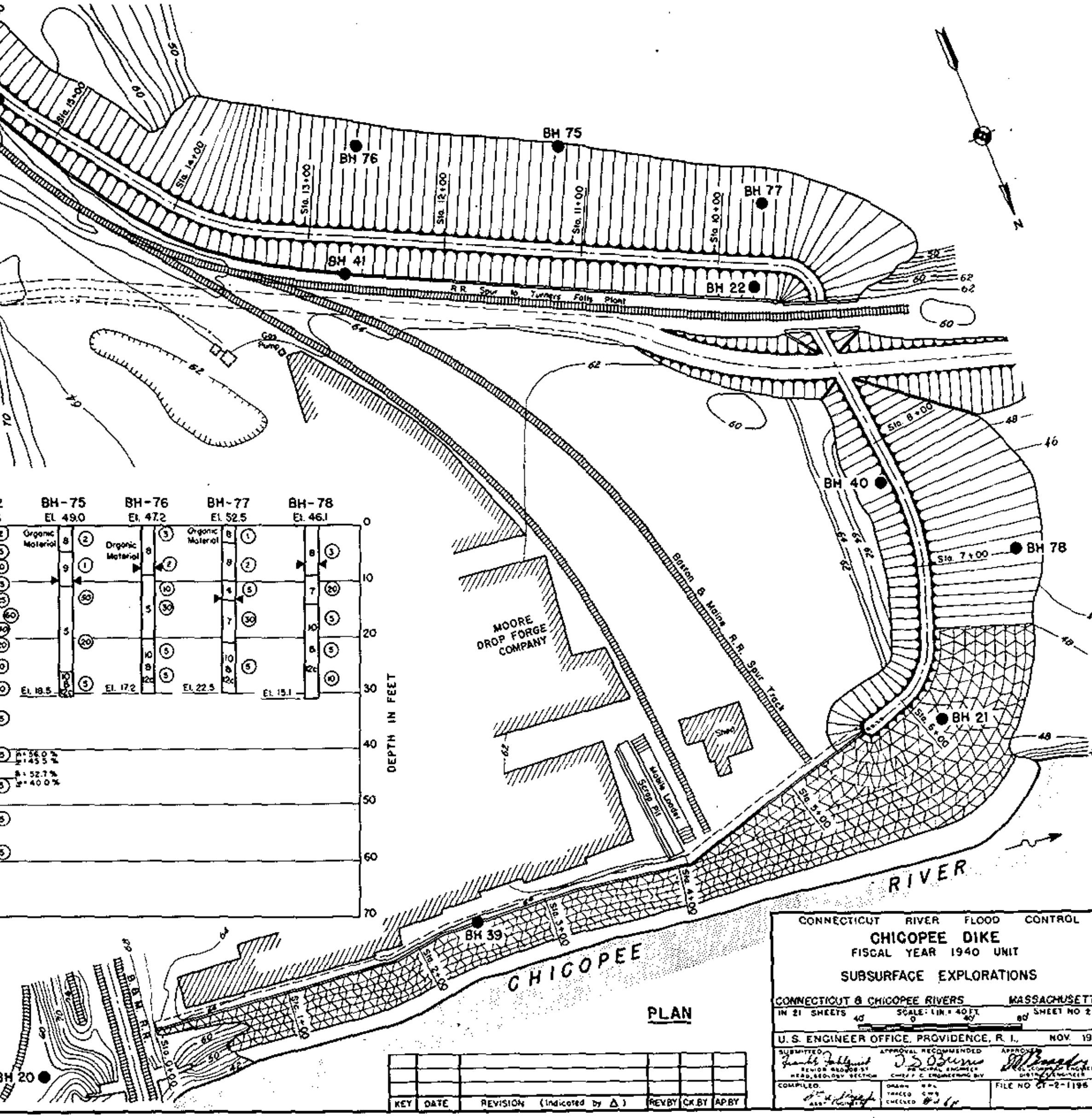
WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY

DESCRIPTION OF NUMERICAL CLASSES

- [1] Graded from Gravel to Coarse Sand – Contains little medium sand.
- [2] Coarse to Medium Sand – Contains little gravel and fine sand.
- [3] Graded from Gravel to Medium Sand – Contains little fine sand.
- [4] Medium to Fine Sand – Contains little coarse sand and coarse silt.
- [5] Graded from Gravel to Fine Sand – Contains little coarse silt.
- [6] Fine Sand to Coarse Silt – Contains little medium sand and medium silt.
- [7] Graded from Gravel to Coarse Silt – Contains little medium silt.
- [8] Coarse to Medium Silt – Contains little fine sand and fine silt.
- [9] Graded from Gravel to Medium Silt – Contains little fine silt.
- [10] Medium to Fine Silt – Contains little coarse silt and coarse clay. Possesses behavior characteristics of silt.
- [10C] Medium Silt to Coarse Clay – Contains little coarse silt and medium clay. Possesses behavior characteristics of clay.
- [11] Graded from Gravel or Coarse Sand to Fine Silt – Contains little coarse clay.
- [12] Fine Silt to Clay – Contains little medium silt and fine clay (colloids). Possesses behavior characteristics of silt.
- [12C] Clay – Contains little silt. Possesses behavior characteristics of clay.
- [13] Graded from Coarse Sand to Clay – Contains little fine clay (colloids). Possesses behavior characteristics of silt.
- [13C] Clay – Graded from sand to fine clay (colloids). Possesses behavior characteristics of clay.





NOTES

Elevations refer to M.S.L. datum.

Numbers in circles indicate number of blows required to drive 2" O.D. sample spoon one foot. Spoon driven by 300 pound hammer dropped 18 inches.

Classes 8, 10 and 12C indicated in records of bore holes occur in thin alternating bands.

LEGEND

- BH ● Drive sample bore hole
- ▸◂ Water table at time of exploration
- n Porosity = Volume of voids / Total volume
- w Natural water content in percent of dry weight (at time of exploration).

KEY	DATE	REVISION (Indicated by △)	REV.BY	CK.BY	AP.BY

CONNECTICUT RIVER FLOOD CONTROL

CHICOPEE DIKE

FISCAL YEAR 1940 UNIT

SUBSURFACE EXPLORATIONS

CONNECTICUT & CHICOPEE RIVERS — MASSACHUSETTS

IN 21 SHEETS — SCALE: 1 IN. = 40 FT. — SHEET NO 2

U. S. ENGINEER OFFICE, PROVIDENCE, R. I. — NOV. 1939

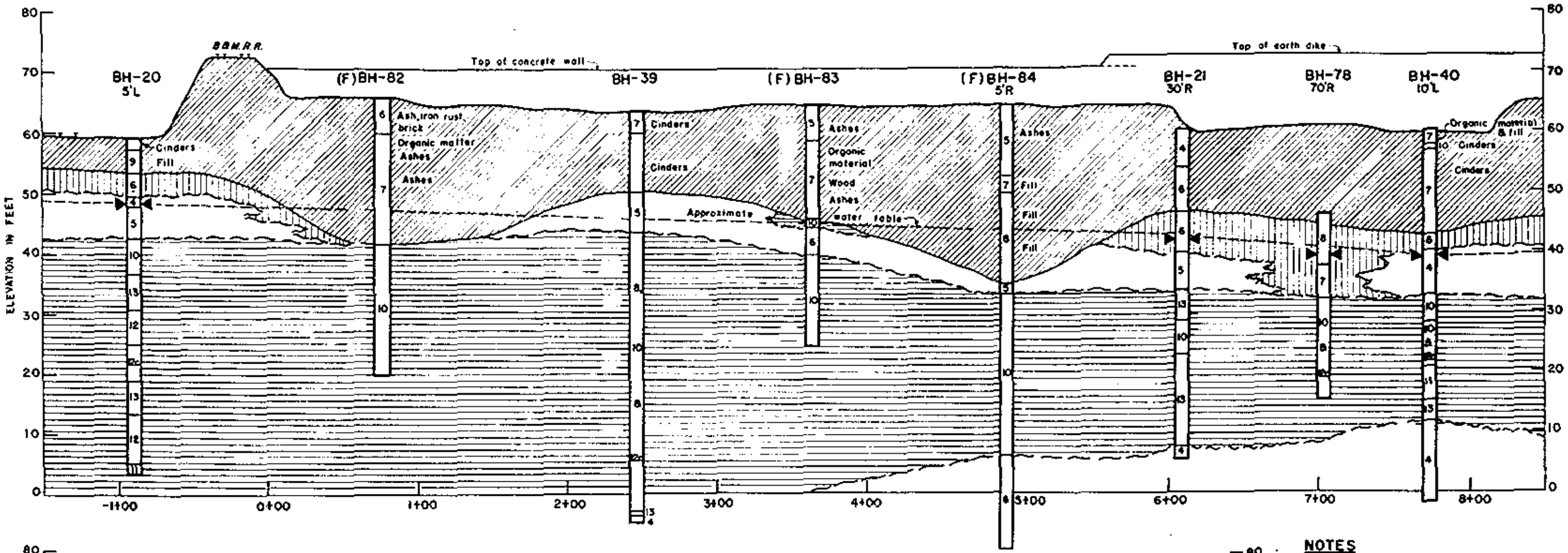
SUBMITTED: [signature] Senior Geologist, Head, Geology Section — APPROVAL RECOMMENDED: [signature] Principal Engineer, Chief F. C. Engineering Div. — APPROVED: [signature] Col. Corps of Engineers, District Engineer

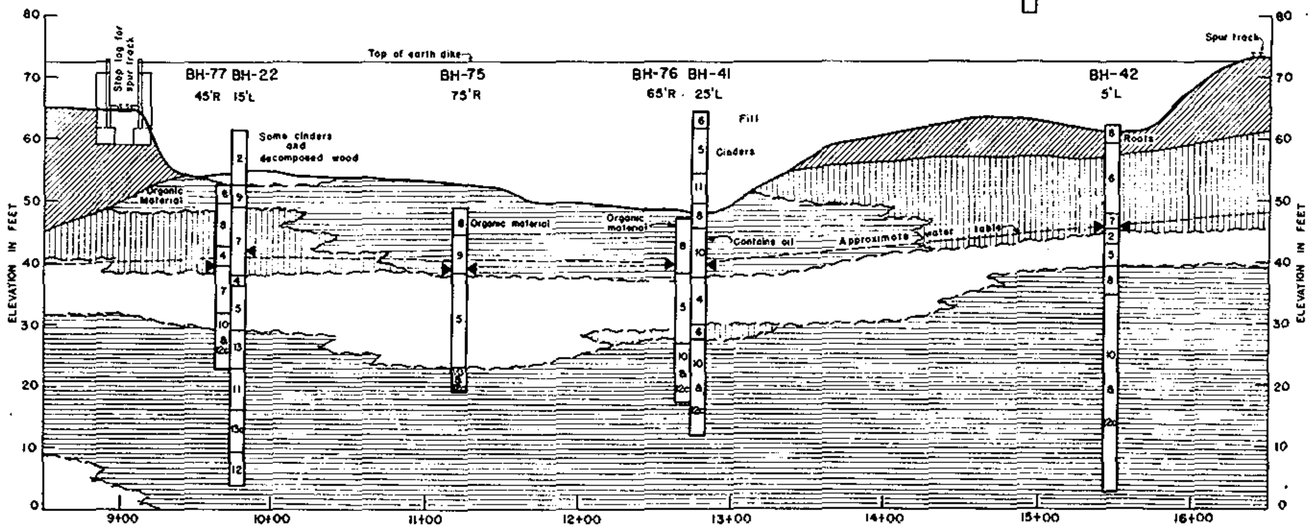
COMPILED: [signature] Asst. Engineer — DRAWN — TRACED CWS — CHECKED — FILE NO CT-2-1196

PLATE NO. 2

WAR DEPARTMENT

CORPS OF ENGINEERS, U.S. ARMY





NOTES

Elevations refer to Mean Sea Level Datum.
Stationing shown is that along ℄ dike.
R indicates river side of ℄ dike.
L indicates land side of ℄ dike.
For description of numerical classes see Table No 5

Classes 8, 10, & 12c indicated in records of bore holes occur in thin alternating bands.
F indicates materials classed by visual inspection.

LEGEND

Artificial fill
Impervious formation
Pervious formation
Moderately impervious formation

BH *Drive sample bore hole*
▶| |◀ *Water table at time of exploration*

KEY	DATE	REVISION (Indicated by Δ)	REV.BY	CK BY	AP BY

CONNECTICUT RIVER FLOOD CONTROL
CHICOPEE DIKE
WILLIMANSETT SECTION DIKE
GEOLOGIC SECTION

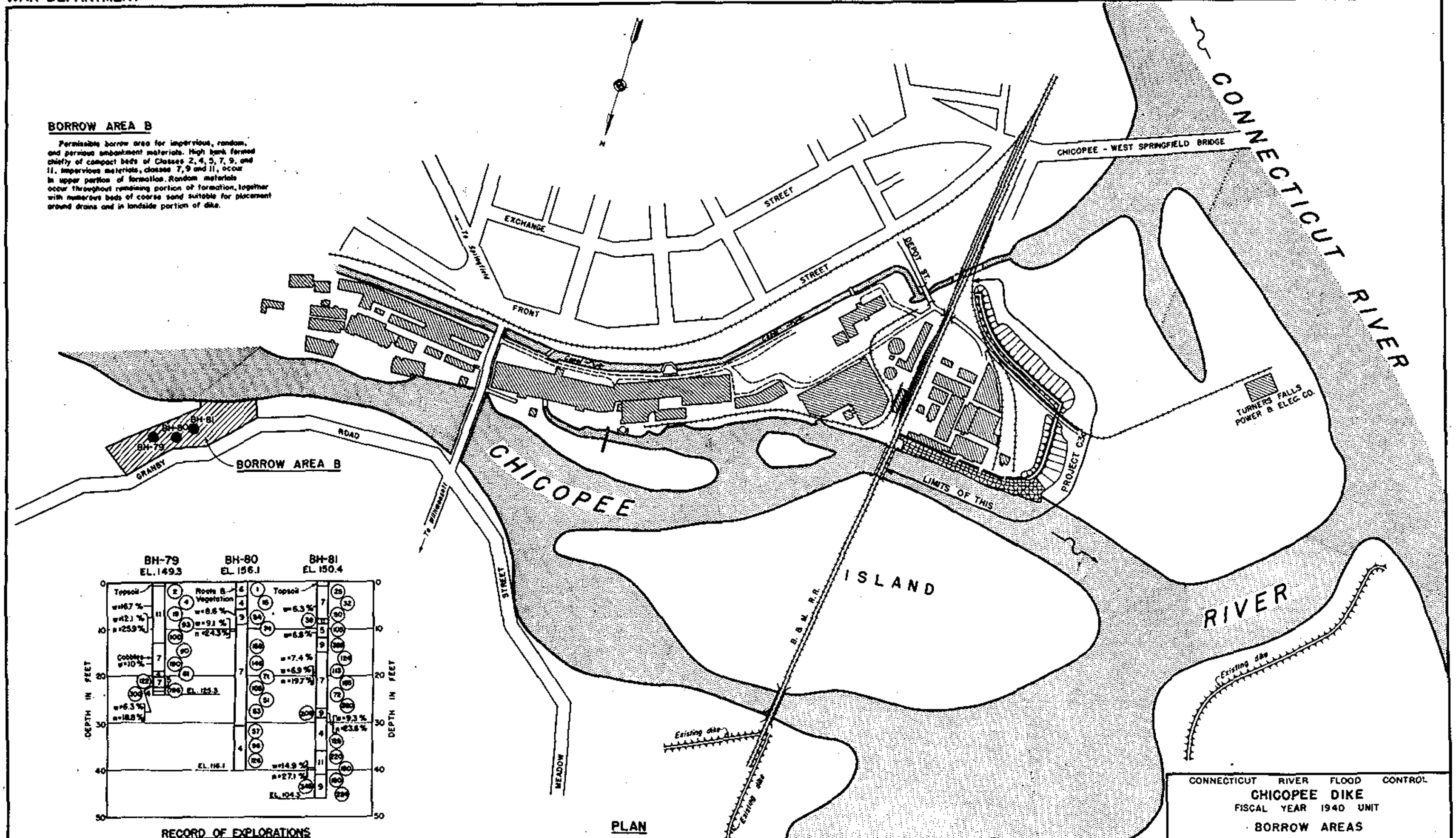
CONNECTICUT & CHICOPEE RIVERS — MASSACHUSETTS
IN 1 SHEET — HOR. SCALE: 1 IN. = 40 FT. VERT. SCALE: 1 IN. = 10 FT. — SHEET NO. 1
U S ENGINEER OFFICE, PROVIDENCE, R. I., DEC. 1939
COMPILED: — DRAWN: — TRACED: — CHECKED: — FILE NO. CT.-2-1203

PLATE NO. 3

C.3a

BORROW AREA B

Permissible borrow area for impervious, random, and pervious embankment materials. High bank formed chiefly of compact beds of Classes 2, 4, 5, 7, 9, and 11. Impervious materials, classes 7, 9 and 11, occur in upper portion of formation. Random materials occur throughout remaining portion of formation, together with numerous beds of coarse sand suitable for placement around drains and in landside portion of dike.



NOTES

For description of numerical classes and general notes see Sheet No. 2

w – Natural water content in per cent of dry weight (at time of exploration).

n – Porosity = $\frac{\text{Volume of voids}}{\text{Total volume}}$

Borrow Area B has been further explored with auger borings and test pits. The logs, samples, and test data for these explorations are available for inspection at the U.S. Engineer Area Office at West Springfield, Mass.

KEY	DATE	REVISION (Indicated by △)	REV. BY	CK. BY	AP. BY

CONNECTICUT RIVER FLOOD CONTROL

CHICOPEE DIKE

FISCAL YEAR 1940 UNIT

BORROW AREAS

CONNECTICUT & CHICOPEE RIVERS — MASSACHUSETTS

IN 21 SHEETS — SCALE: 1 IN. = 200 FT. — SHEET NO. 3

U. S. ENGINEER OFFICE, PROVIDENCE, R. I. — NOV. 1939

SUBMITTED: Senior Geologist, Head, Geology Section
APPROVAL RECOMMENDED: Principal Engineer, Chief F. C. Engineering Div.
APPROVED: Col., Corps of Engineers, District Engineer

COMPILED: Asst. Engineer
DRAWN W.E.L.
TRACED W.E.L.
CHECKED

FILE NO. CT-2-1197

PLATE NO. 4

PROVIDENCE DISTRICT SOIL CLASSIFICATION

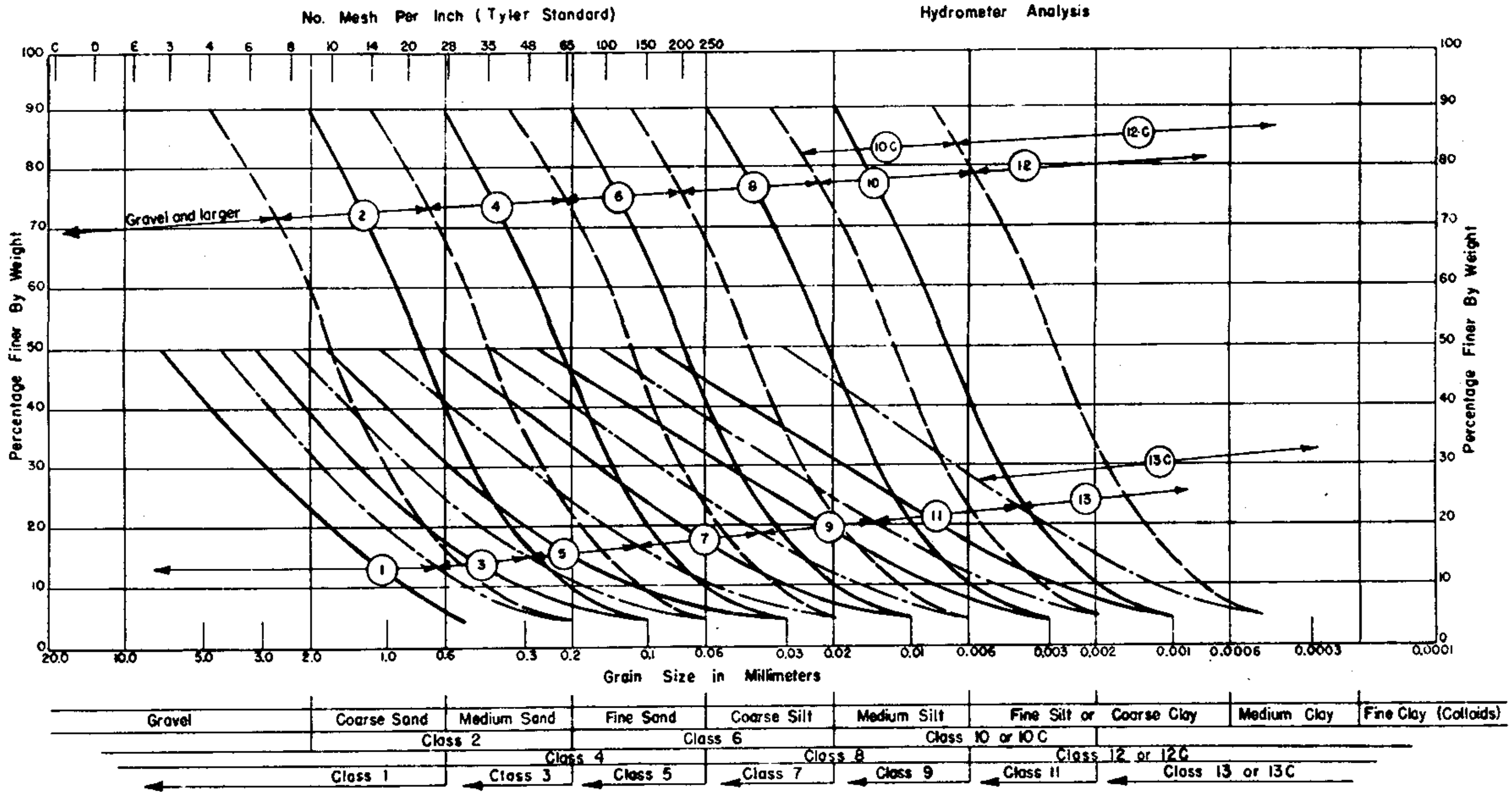
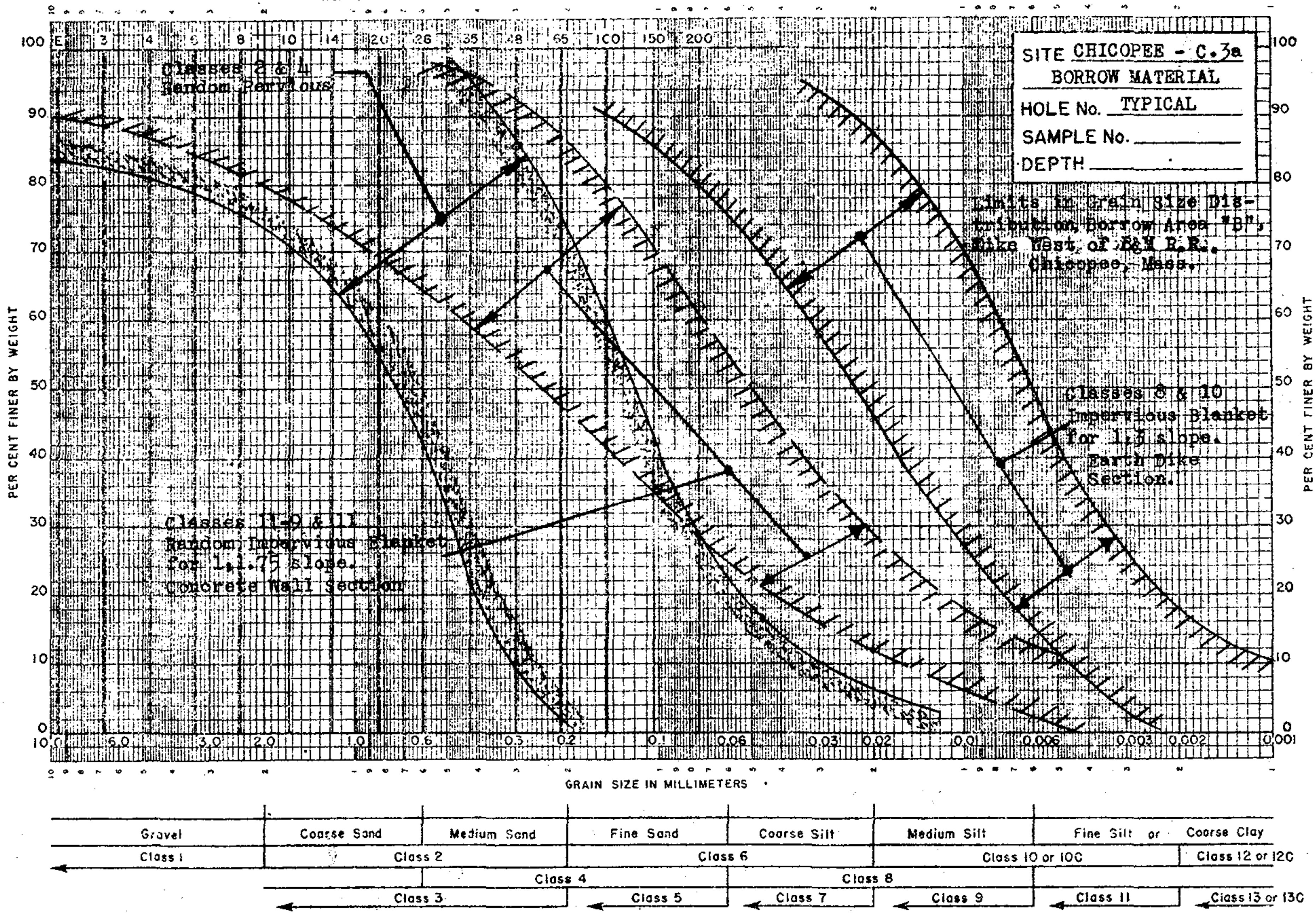


DIAGRAM SHOWING LIMITS OF SOIL CLASSES

PLATE NO. 5

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY



SOILS LABORATORY

MECHANICAL ANALYSIS

PROVIDENCE, R.I.

S.L. FORM NO. 50

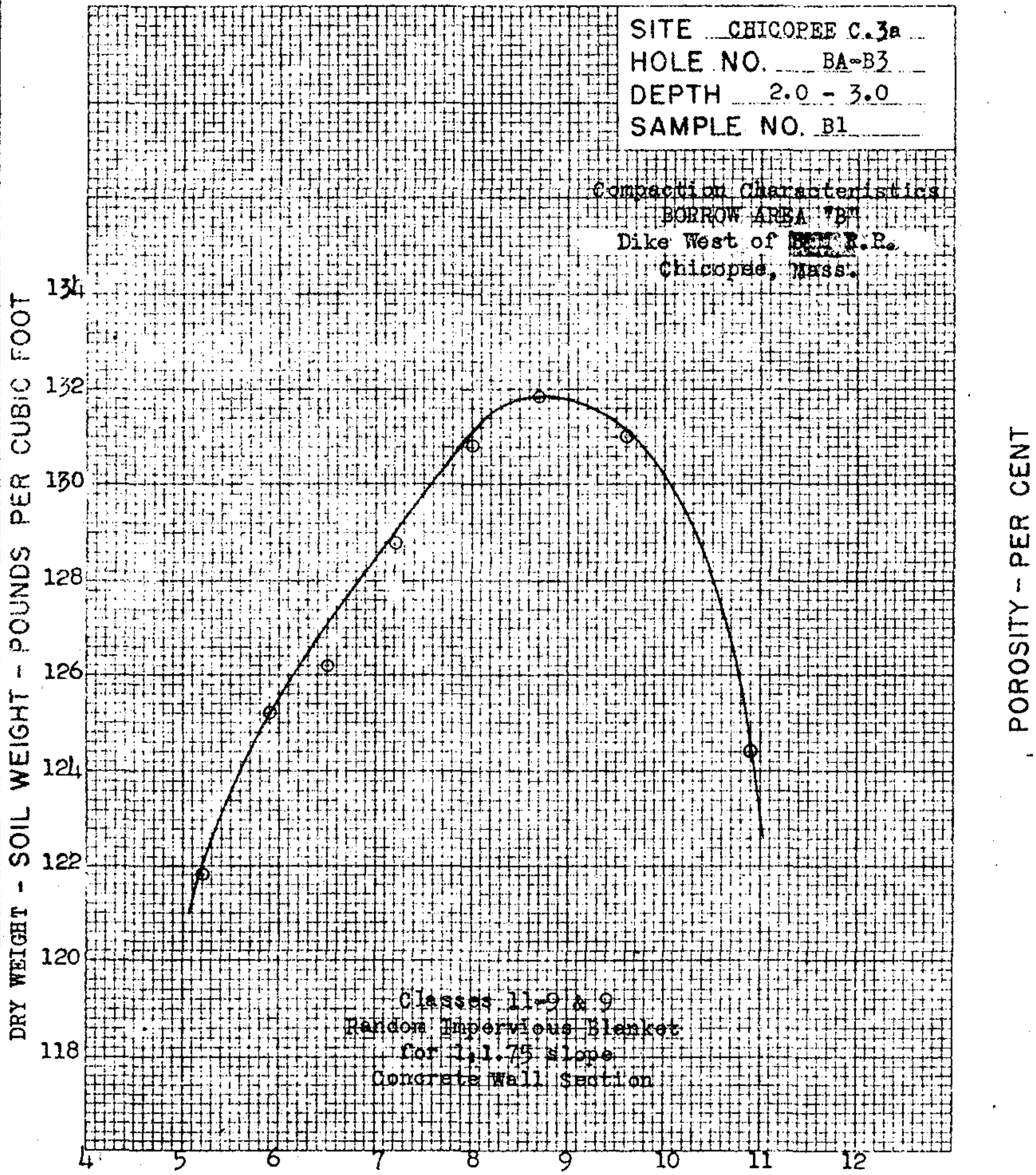
U. S. GOVERNMENT PRINTING OFFICE 140825

PLATE NO. 6

WAR DEPARTMENT | CORPS OF ENGINEERS, U. S. ARMY

COMPACTION CHARACTERISTICS

SITE CHICOPEE C.3a
HOLE NO. BA-B3
DEPTH 2.0 - 3.0
SAMPLE NO. B1



WATER CONTENT - PER CENT

Class 11-9

MATERIAL SCREENED OUT

Minimum Size, mm. ________

Per Cent by weight None

No. Blows/Layer 25

Area of Tamper, sq. in. 3.14

Weight of Tamper, lbs. 5.5

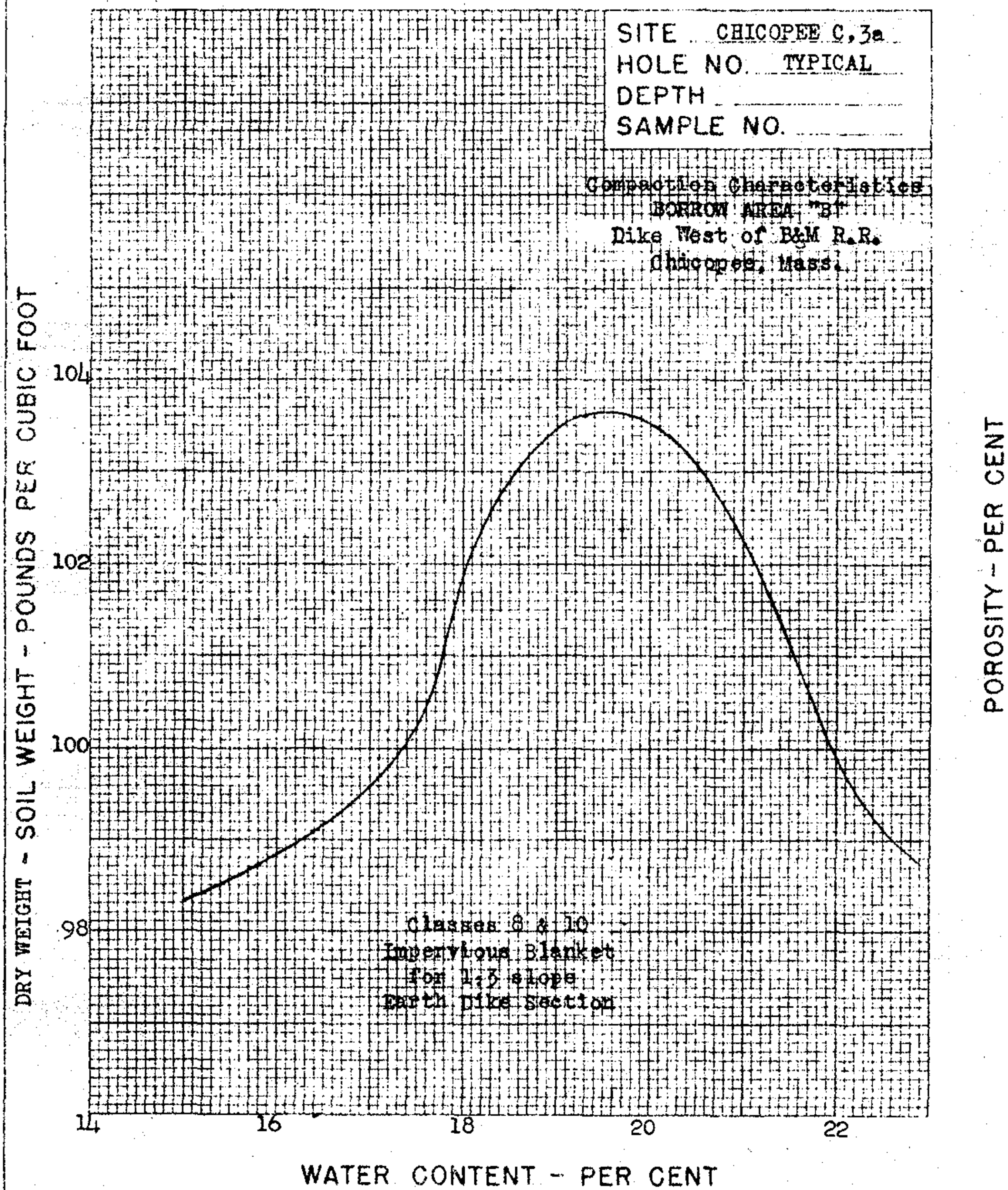
Fall of Tamper, in. 12

PLATE NO. 7

SOILS LABORATORY | PROVIDENCE, R. I.

WAR DEPARTMENT — CORPS OF ENGINEERS, U. S. ARMY

COMPACTION CHARACTERISTICS



Class 8-10

MATERIAL SCREENED OUT

Minimum Size, mm.

Per Cent by weight None

No. Blows/Layer 25

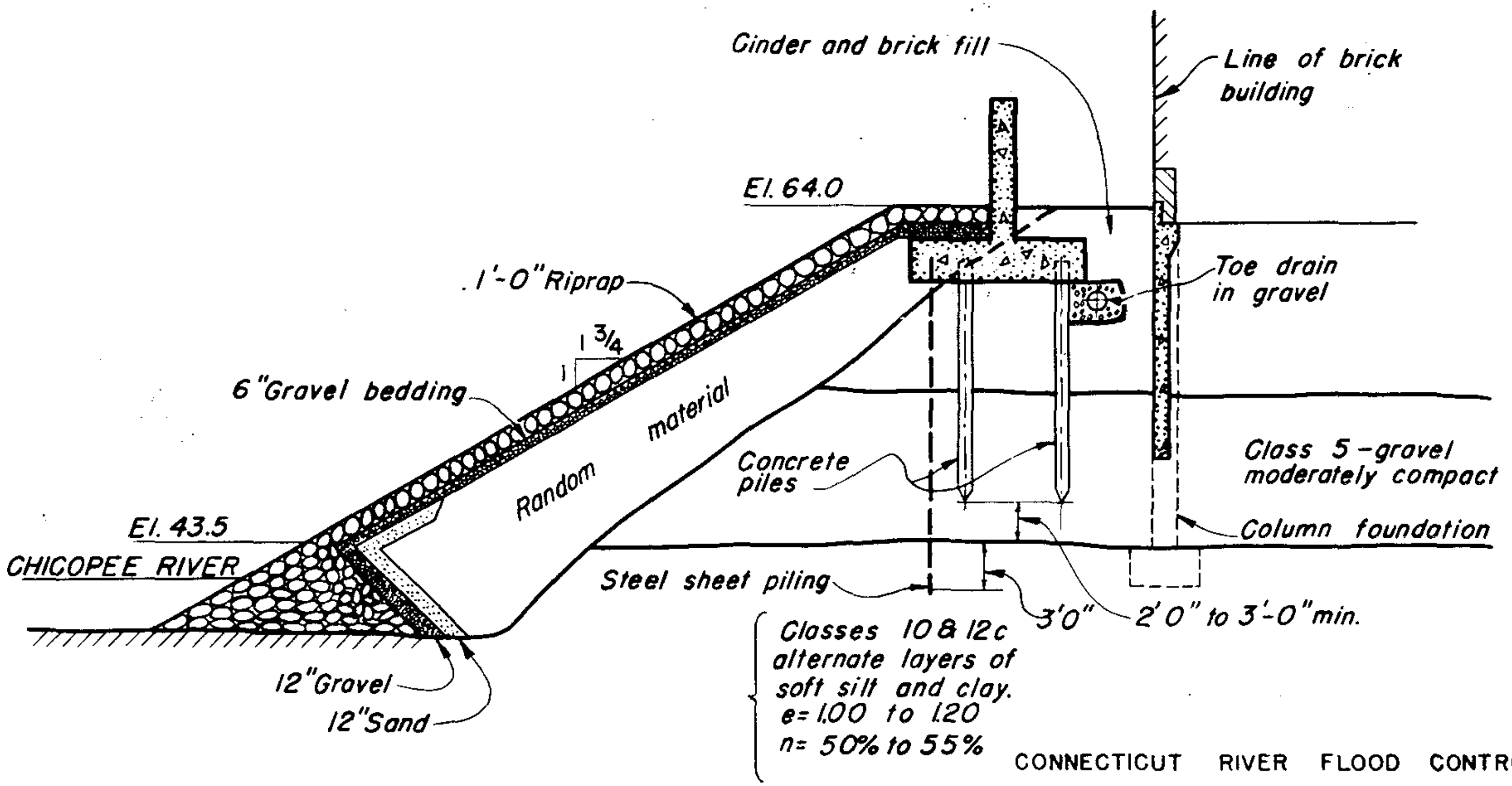
Area of Tamper, sq. in. 3.14

Weight of Tamper, lbs. 5.5

Fall of Tamper, in. 12

PLATE NO. 8

SOILS LABORATORY — PROVIDENCE, R. I.



SCALE: 1" = 10'

CONNECTICUT RIVER FLOOD CONTROL

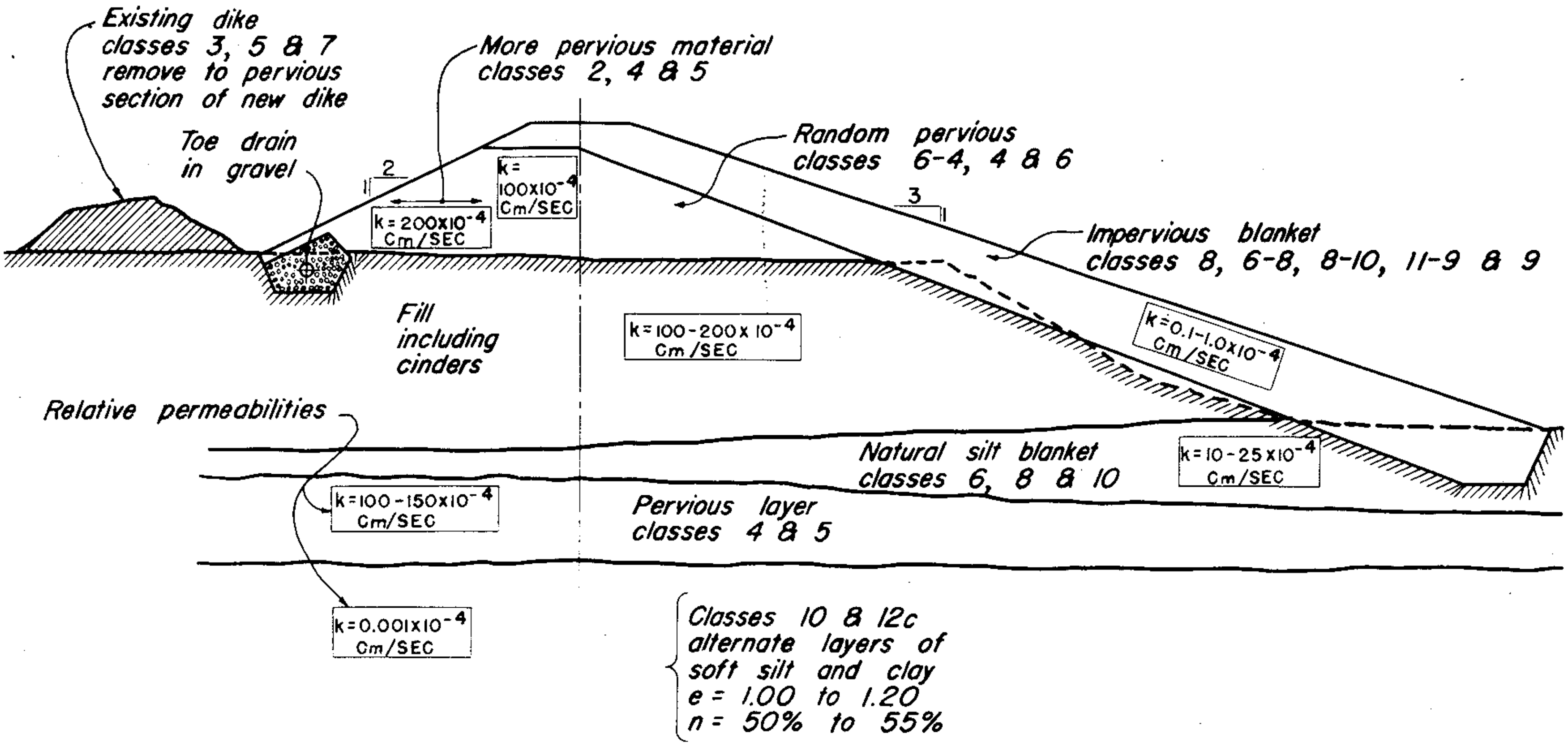
GEOLOGIC SECTION
AT CONCRETE WALL
STATION 1 + 00

CONNECTICUT & CHICOPEE RIVERS MASS.

U.S. ENGINEER OFFICE PROVIDENCE, R. I.

FILE NO. CT-2-1200

PLATE NO. 9



SCALE: 1" = 15'

CONNECTICUT RIVER FLOOD CONTROL

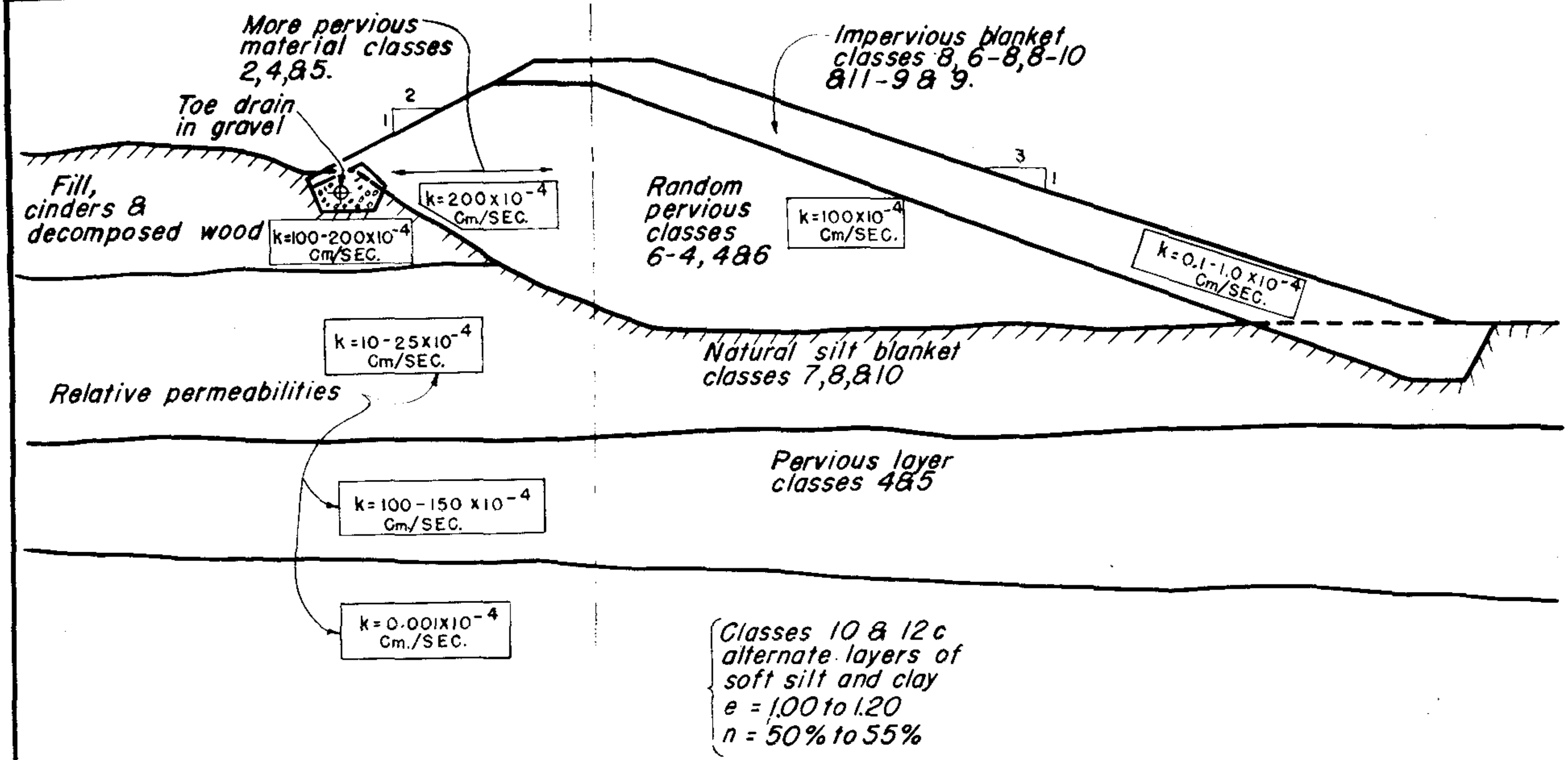
GEOLOGIC SECTION AT DIKE STATION 7+00

CONNECTICUT & CHICOPEE RIVERS MASS.

U.S. ENGINEER OFFICE PROVIDENCE, R.I.

FILE NO. CT-2-1201

PLATE NO. 10



SCALE: 1" = 15'

CONNECTICUT RIVER FLOOD CONTROL

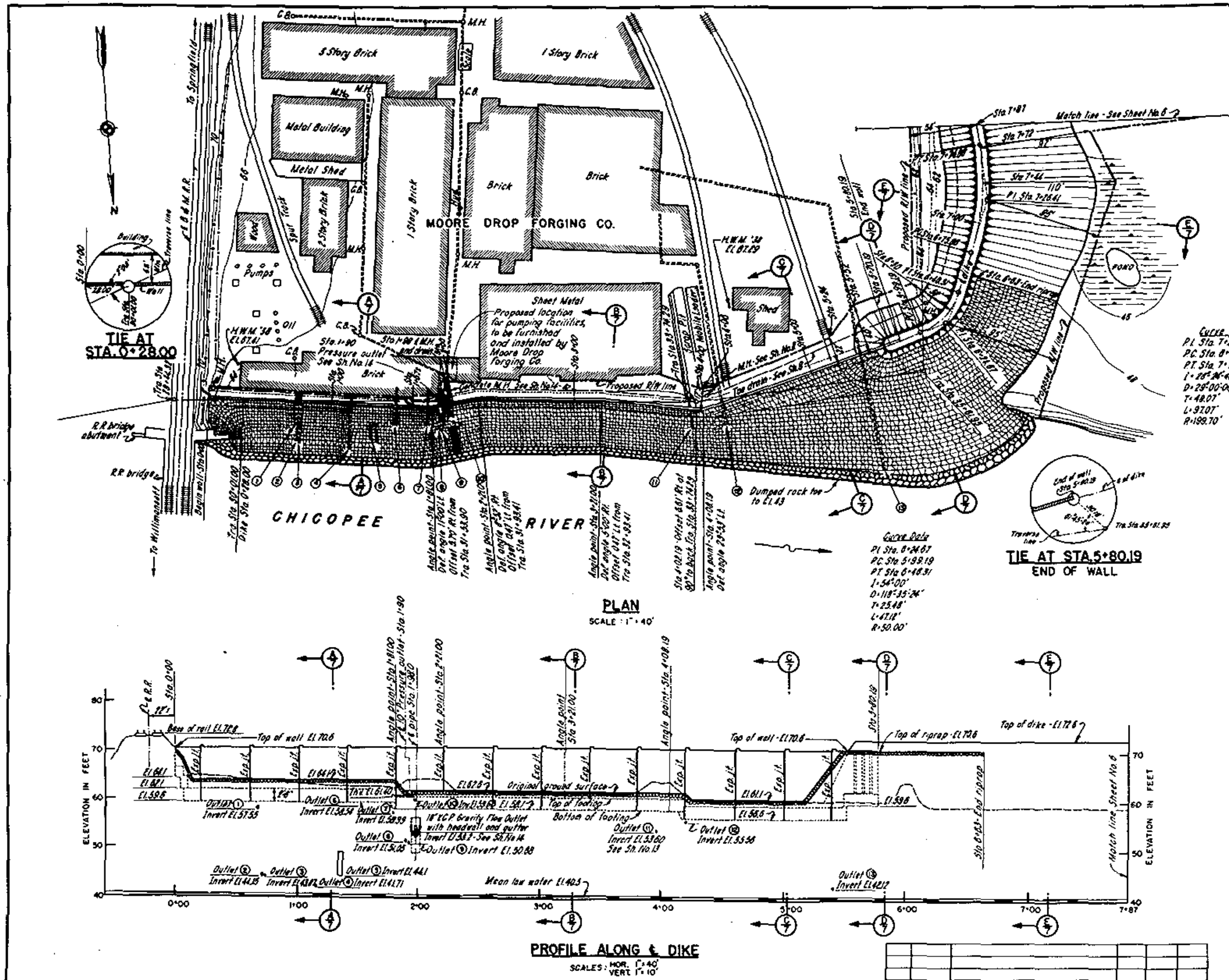
GEOLOGIC SECTION AT DIKE STATION 12+00

CONNECTICUT & CHICOPEE RIVERS MASS.

U.S. ENGINEER OFFICE PROVIDENCE, R.I.

FILE NO. CT.-2-1202

PLATE NO. 11



SCHEDULE OF OPENINGS TO BE ABANDONED			
OUTLET NO.	STA.	DESCRIPTION	APPROX. EL. OF INVERT
①	0+67	5" C.I. pipe	57.55
②	0+70	6" C.I. pipe	44.15
③	0+73	4" C.I. pipe	43.87
④	1+15	8" C.I. pipe	41.71
⑤	1+25	Concrete water intake	44.1
⑥	1+52	4" Galv. pipe	58.54
⑦	1+79	4" Galv. pipe	58.99
⑧	1+92	8" V.C. inside 10" C.I.	51.05
⑨	1+93	5" Iron pipe	50.88
⑩	1+96	1" Steel pipe	59.68
⑪	3+97	2" Iron pipe	53.60
⑫	4+22	4" V.C. pipe	53.56
⑬	5+41	6" C.I. pipe	42.12

Note:
Pipes to be removed or steel sheet piling to be driven through pipes.

NOTES

All elevations refer to Mean Sea Level Datum.
Riverside face of flood wall is on ℄ dike.
For concrete wall details see Sheets Nos. 9 and 10.
Numbers in circles refer to schedule of openings through bank.
For drainage profile see Sheet No. 8.

CONNECTICUT RIVER FLOOD CONTROL
CHICOPEE DIKE
FISCAL YEAR 1940 UNIT
PLAN AND PROFILE NO. 1
CONNECTICUT & CHICOPEE RIVERS — MASSACHUSETTS
IN 21 SHEETS — SCALE: 1 IN. = 40 FT. — SHEET NO. 5
U. S. ENGINEER OFFICE, PROVIDENCE, R. I., NOV. 1939
SUBMITTED: Senior Engineer, Head, Design Section — APPROVAL RECOMMENDED: Principal Engineer, Chief F. C. Engineering Div. — APPROVED: Lt. Col., Corps of Engineers, District Engineer
DESIGNED: Asst. Engineer — DRAWN — TRACED — CHECKED — FILE NO. CT-4-1997

C. 3a

5

PLATE NO. 12

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY

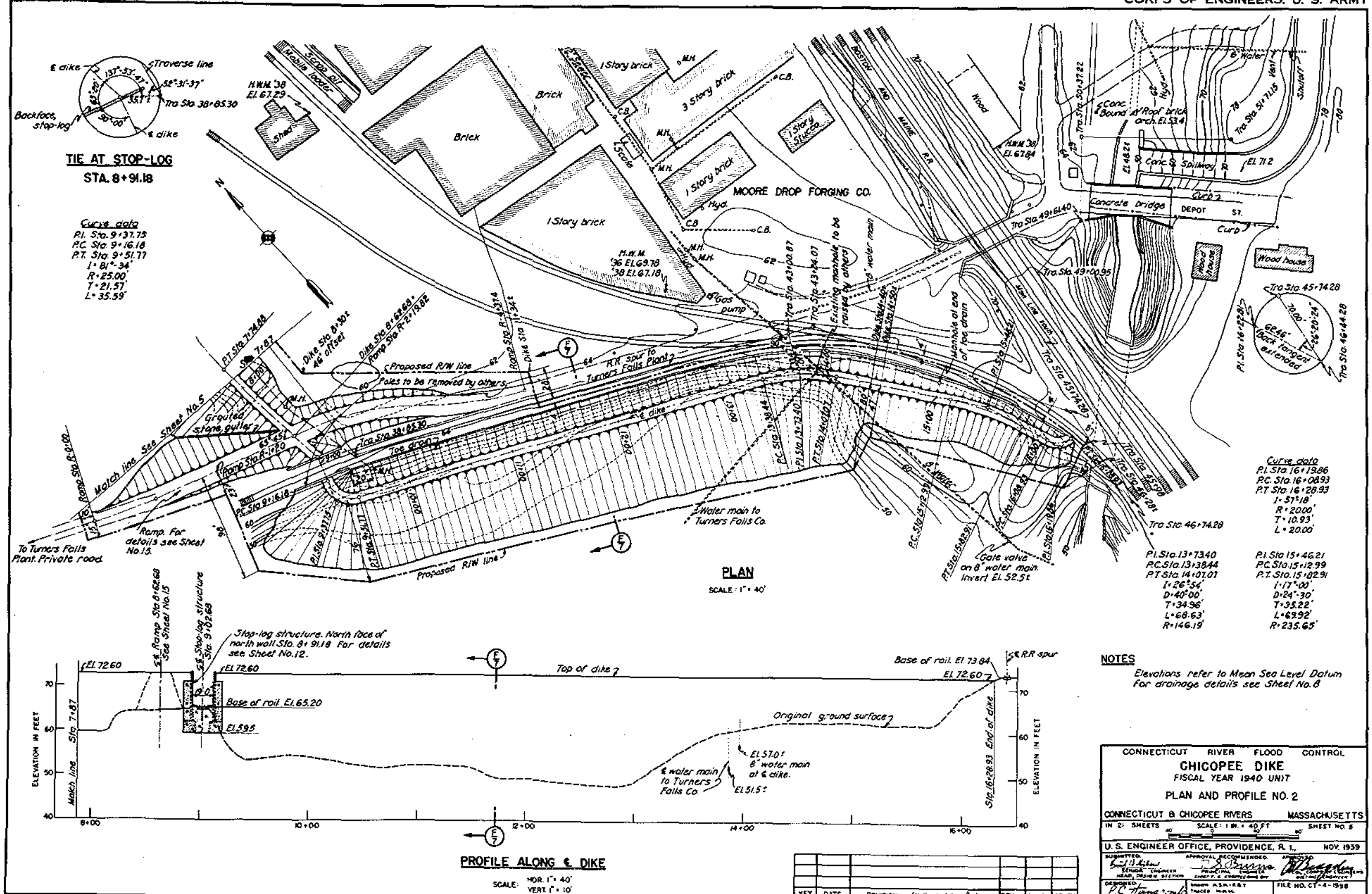
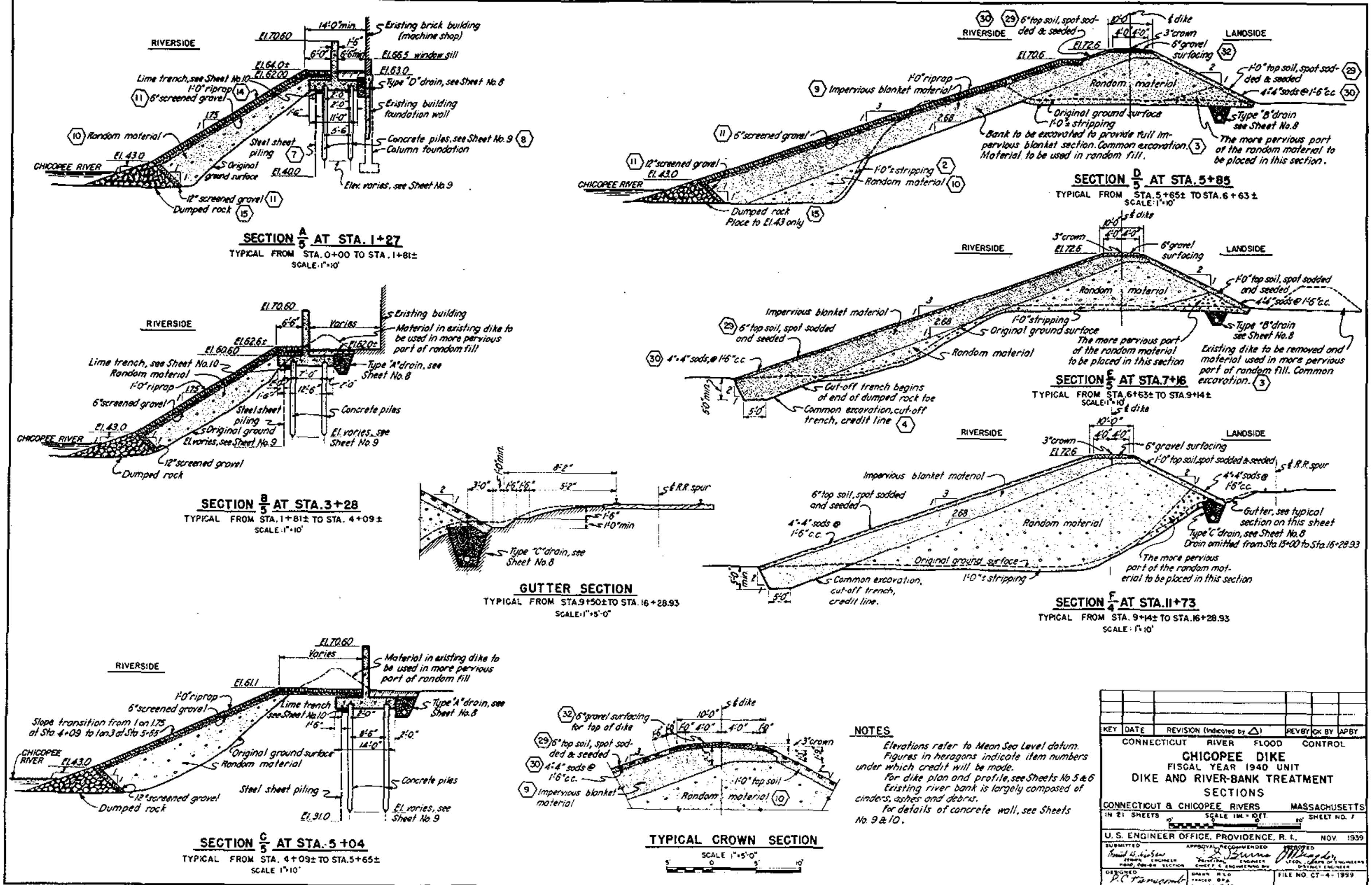


PLATE NO. 13

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY



NOTES

Elevations refer to Mean Sea Level datum.
Figures in hexagons indicate item numbers under which credit will be made.
For dike plan and profile, see Sheets No. 5 & 6.
Existing river bank is largely composed of cinders, ashes and debris.
For details of concrete wall, see Sheets No. 9 & 10.

KEY	DATE	REVISION (Indicated by △)	REV. BY	CK. BY	APP. BY

CONNECTICUT RIVER FLOOD CONTROL

CHICOPEE DIKE

FISCAL YEAR 1940 UNIT

DIKE AND RIVER-BANK TREATMENT SECTIONS

CONNECTICUT & CHICOPEE RIVERS — MASSACHUSETTS

IN 21 SHEETS — SCALE: 1 IN. = 10 FT. — SHEET NO. 7

U. S. ENGINEER OFFICE, PROVIDENCE, R. I. — NOV. 1939

FILE NO. CT-4-1999

PLATE NO. 14

C.3a

7

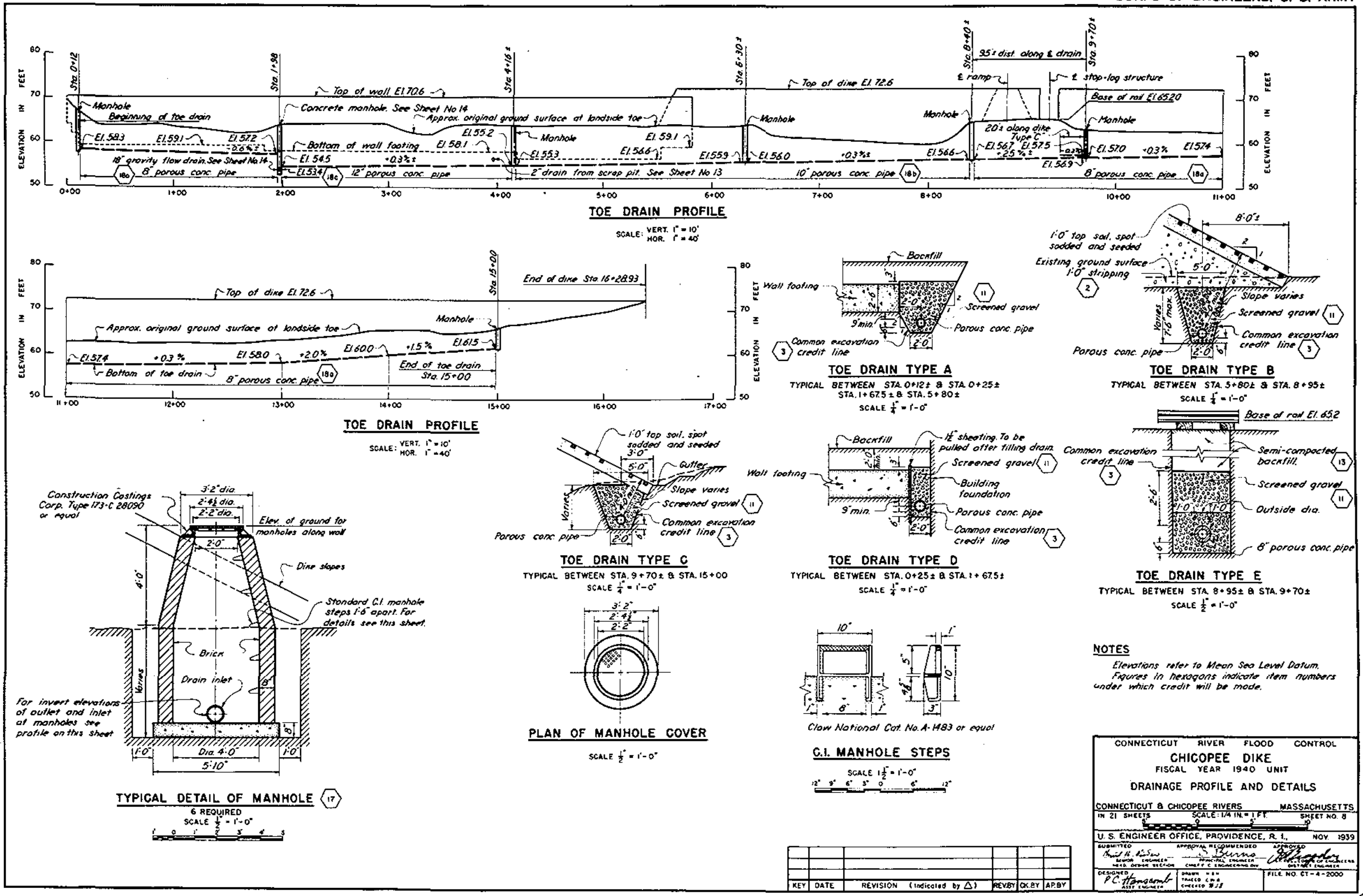
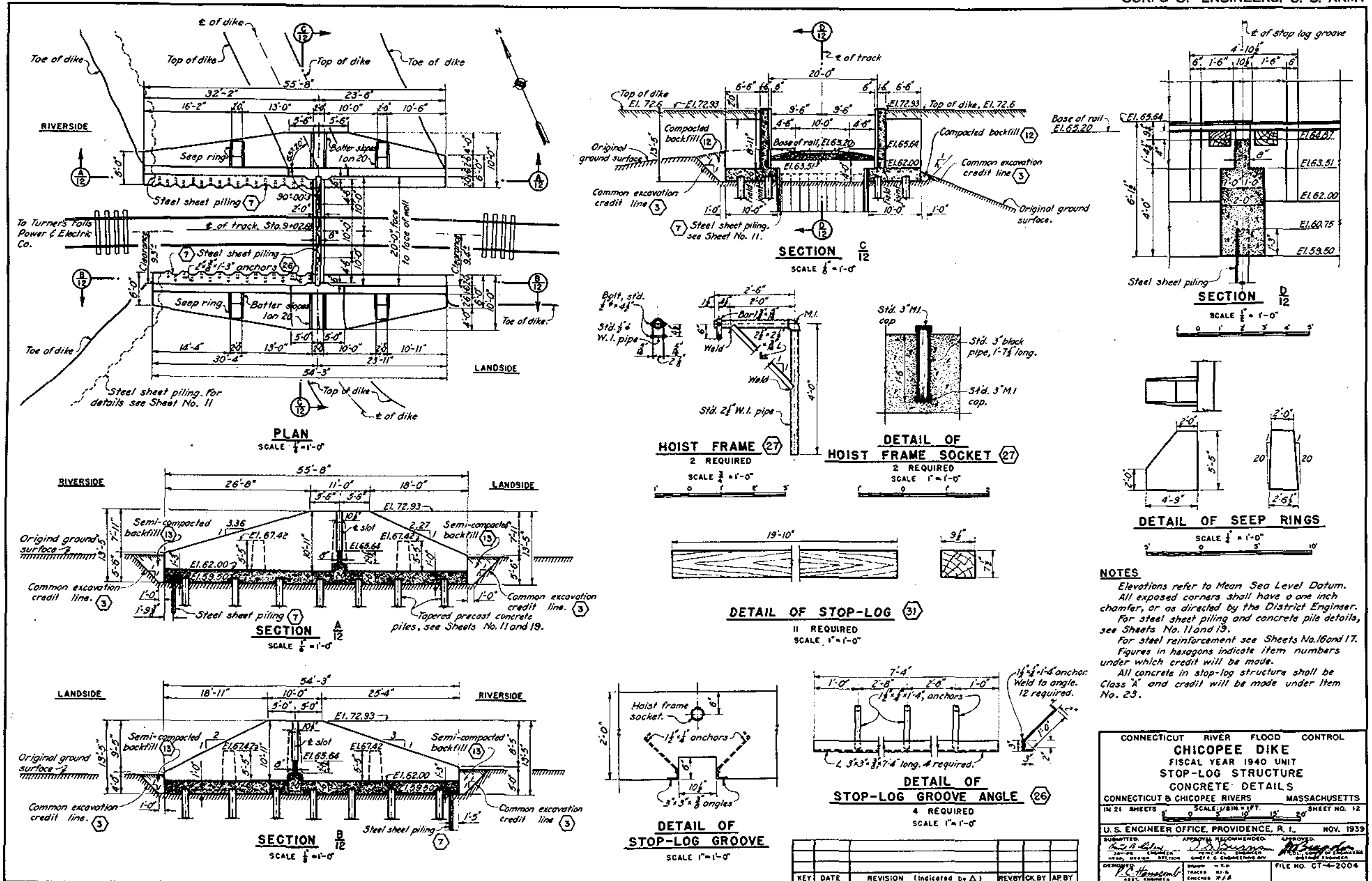

TOE DRAIN PROFILE
SCALE: VERT. 1" = 10' HOR. 1" = 40'
ELEVATION IN FEET
Sta. 0+12
Sta. 1+98
Sta. 4+16±
Sta. 6+30±
Sta. 8+40±
Sta. 9+70±
Manhole
Beginning of toe drain
Top of wall El. 70.6
Concrete manhole. See Sheet No 14
Approx. original ground surface at landside toe
Top of dike El. 72.6
35± dist. along ℄ drain
℄ ramp
℄ stop-log structure
Base of rail El. 65.20
20± along dike Type "C"
Bottom of wall footing
18" gravity flow drain. See Sheet No. 14
8" porous conc. pipe
12" porous conc. pipe
2" drain from scrap pit. See Sheet No 13
10" porous conc. pipe
El. 58.3
El. 59.1
El. 57.2
El. 54.5
El. 53.4
El. 55.2
El. 58.1
El. 55.3
El. 59.1
El. 56.6
El. 55.9
El. 56.0
El. 56.6
El. 56.7
El. 57.5
El. 56.9
El. 57.0
El. 57.4
+0.3%±
+0.3%
+2.5%±
TOE DRAIN PROFILE
SCALE: VERT. 1" = 10' HOR. 1" = 40'
Sta. 15+00
End of dike Sta. 16+28.93
Top of dike El. 72.6
Approx. original ground surface at landside toe
Manhole
El. 57.4
+0.3%
El. 58.0
+2.0%
El. 60.0
+1.5%
El. 61.5
Bottom of toe drain
8" porous conc. pipe
End of toe drain Sta. 15+00
Backfill
Wall footing
Screened gravel
Porous conc. pipe
Common excavation credit line
9" min.
TOE DRAIN TYPE A
TYPICAL BETWEEN STA. 0+12± & STA. 0+25± STA. 1+67.5± & STA. 5+80±
SCALE 1/4" = 1'-0"
8'-0"±
1'-0" top soil, spot sodded and seeded
Existing ground surface
1'-0" stripping
Slope varies
Screened gravel
Common excavation credit line
Porous conc. pipe
TOE DRAIN TYPE B
TYPICAL BETWEEN STA. 5+80± & STA. 8+95±
SCALE 1/4" = 1'-0"
1'-0" top soil, spot sodded and seeded
Gutter
Slope varies
Screened gravel
Common excavation credit line
Porous conc. pipe
TOE DRAIN TYPE C
TYPICAL BETWEEN STA. 9+70± & STA. 15+00
SCALE 1/4" = 1'-0"
Backfill
1 1/2" sheeting. To be pulled after filling drain.
Screened gravel
Wall footing
Building foundation
Porous conc. pipe
Common excavation credit line
TOE DRAIN TYPE D
TYPICAL BETWEEN STA. 0+25± & STA. 1+67.5±
SCALE 1/4" = 1'-0"
Base of rail El. 65.2
Common excavation credit line
Semi-compacted backfill.
Screened gravel
Outside dia.
8" porous conc. pipe
TOE DRAIN TYPE E
TYPICAL BETWEEN STA. 8+95± & STA. 9+70±
SCALE 1/2" = 1'-0"
Construction Castings Corp. Type 173-C 28090 or equal
3'-2" dia.
2'-4 1/2" dia.
2'-2" dia.
Elev. of ground for manholes along wall
Dike slopes
Standard C.I. manhole steps 1'-6" apart. For details see this sheet.
Brick
Drain inlet
For invert elevations of outlet and inlet at manholes see profile on this sheet
Dia. 4'-0"
5'-10"
TYPICAL DETAIL OF MANHOLE
6 REQUIRED
SCALE 1/2" = 1'-0"
PLAN OF MANHOLE COVER
SCALE 1/2" = 1'-0"
Claw National Cat. No. A-1483 or equal
C.I. MANHOLE STEPS
SCALE 1 1/2" = 1'-0"
NOTES
Elevations refer to Mean Sea Level Datum.
Figures in hexagons indicate item numbers under which credit will be made.
KEY DATE REVISION (Indicated by △) REV.BY CK.BY AP.BY
CONNECTICUT RIVER FLOOD CONTROL
CHICOPEE DIKE
FISCAL YEAR 1940 UNIT
DRAINAGE PROFILE AND DETAILS
CONNECTICUT & CHICOPEE RIVERS MASSACHUSETTS
IN 21 SHEETS SCALE: 1/4 IN. = 1 FT. SHEET NO. 8
U. S. ENGINEER OFFICE, PROVIDENCE, R. I., NOV. 1939
FILE NO. CT-4-2000

PLATE NO. 15

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY



C.3a

(12)

PLATE NO. 16

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY

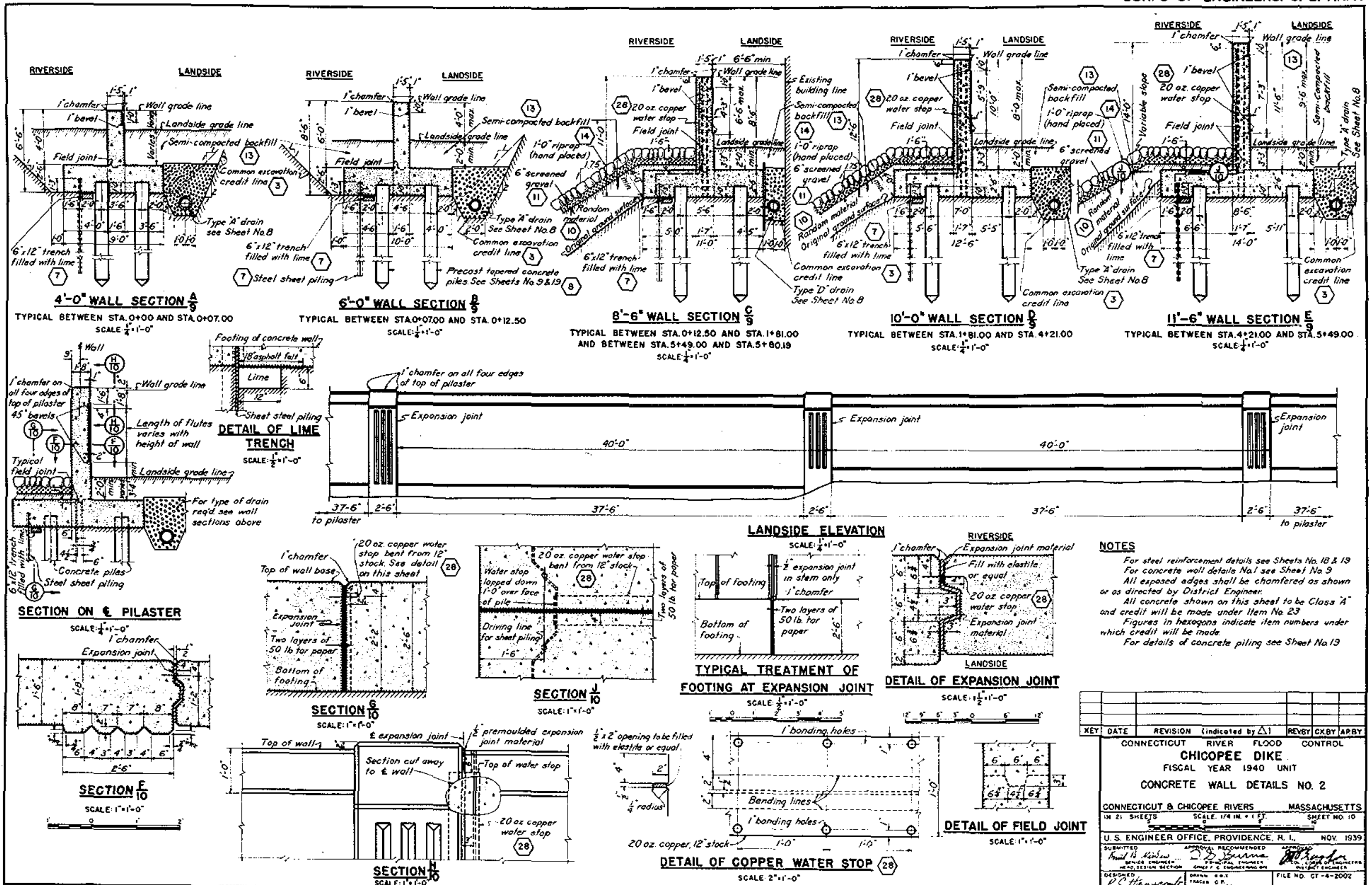
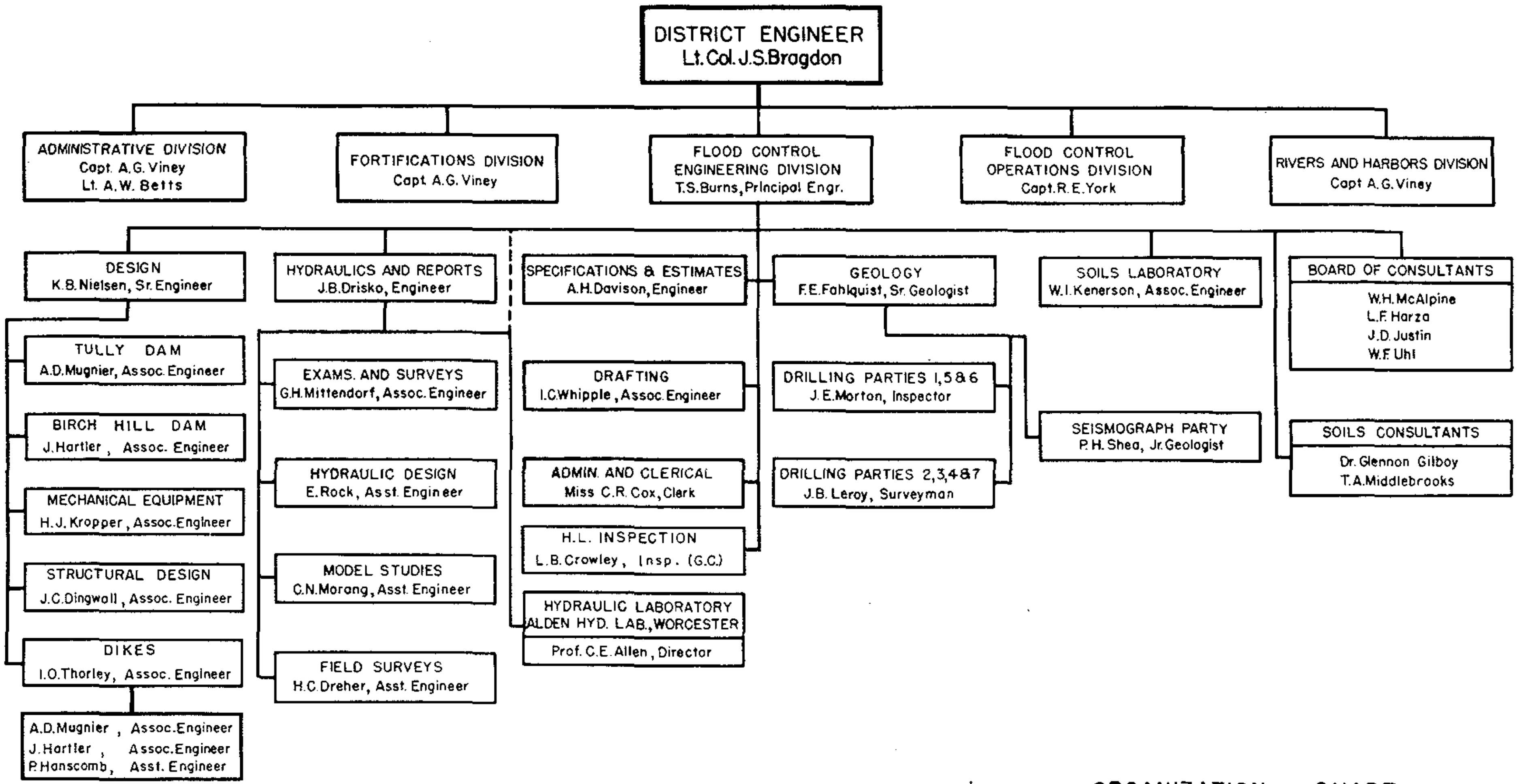


PLATE NO. 17

C.3a



ORGANIZATION CHART
FLOOD CONTROL
ENGINEERING DIVISION
U.S. ENGINEER OFFICE, PROVIDENCE, R.I.
DECEMBER 1939

PLATE NO. 18

CONNECTICUT RIVER FLOOD CONTROL PROJECT

CHICOPEE, MASS.

CONNECTICUT & CHICOPEE RIVERS, MASSACHUSETTS

ANALYSIS OF DESIGN
FOR
LOCAL PROTECTION WORKS

ITEM C. 3a - HIRED LABOR
DIKE WEST OF B. & M. R. R.

APPENDIX A

DECEMBER 1939

CORPS OF ENGINEERS, U. S. ARMY

U. S. ENGINEER OFFICE PROVIDENCE, R. I.

WAR DEPARTMENT

U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

Page 1

ject Chicopee Mass C.3a

omputation Stop Log on Piles

omputed by W.L.J. Checked by ______ Date 10/11/39

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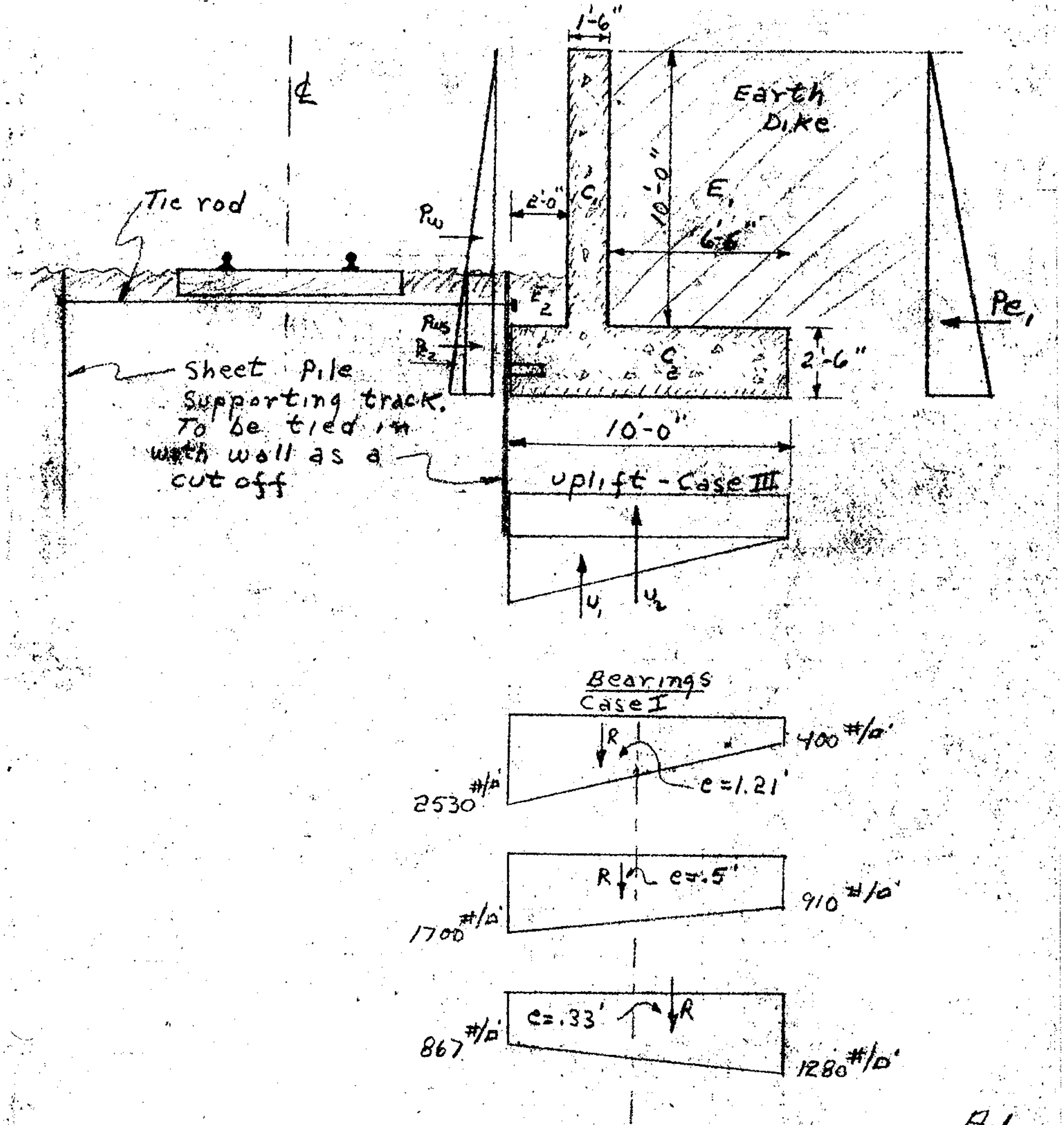

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ect Chicopee Mass C.3a

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Stability

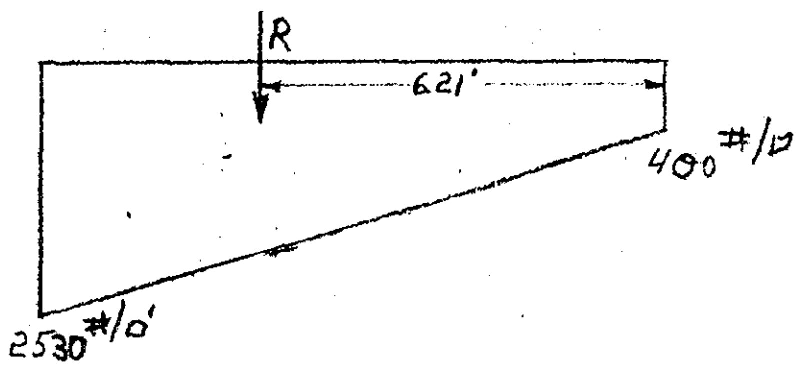
Case I Saturated Dike, no uplift.
(It is assumed that the river has receded, leaving the dike damp, but no uplift.)

Forces	↓	↑	→	←		↻	↺
C_1 10×1.5×150	2250				7.25		16,300
C_2 10×2.5×150	3750				5.0		18,750
E_1 10×6.5×125	8130				3.25		26,300
E_2 2.5×2×125	625				9.0		5,620
Pe_1 $12.5^2 \times \frac{1}{2} \times 80$				6250	4.16		26,000
Pe_2 $4.5^2 \times \frac{1}{2} \times 80$			810		1.5	1210	
	14,755		810	6250		1210	92,970
	ΣV = 14,755		ΣH = 5440			ΣM = 91,760	

$$\frac{\Sigma M}{\Sigma V} = \frac{91,760}{14,755} = 6.21' \quad \text{O.K.}$$

$$\text{Eccentricity} = 6.21 - 5 = 1.21'$$

$$\text{Bearing} = \frac{14,755}{10}\left(1 \pm \frac{6 \times 1.21}{10}\right) = \begin{matrix} 2530 \ \#/\square' \\ 400 \ \#/\square' \end{matrix}$$



WAR DEPARTMENT

U. S. ENGINEER OFFICE, PROVIDENCE, R. I. Page 3

Subject: Chicopee, Mass C.3a

Computation: Stop log on Piles

Computed by: W. S. J. Checked by: RSM Date: 10/13/39

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Stability

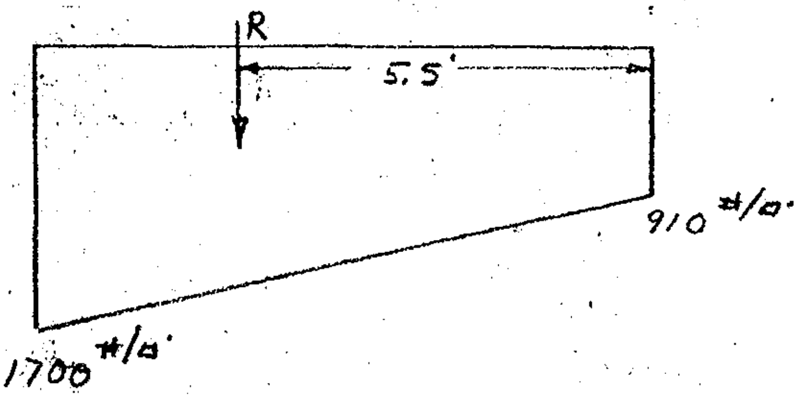
Case II - River Down, Dike Dry

Forces	↓	↑	→	←		↷	↶
C_1	2250						16,300
C_2	3750						18,750
E_2 2.5×2×100	500				9.0		4,500
E_1 10×6.5×100	6500				3.25		21,100
P_e $12.5^2 \times \frac{1}{2} \times 35$				2730	4.16		11,350
P_{e_2}' $4.5^2 \times \frac{1}{2} \times 35$			354		1.5	530	
	13,000		354	2730		530	72,000
	ΣV=13000#		ΣH=2376			ΣM=71,470	

$$\frac{\Sigma M}{\Sigma V} = \frac{71,470}{13,000} = 5.5' \quad \text{O.K.}$$

$$e = 5.5 - 5.0 = 0.5'$$

$$\text{Bearing} = \frac{13,000}{10}\left(1 \pm \frac{6 \times .5}{10}\right) = \begin{matrix} 1700 \ \#/\square' \\ 910 \ \#/\square' \end{matrix}$$



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U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

Project: Chicopee, Mass C.3a

Computation: Stop log on Piles

Computed by: W. S. Jr. Checked by: Date: 10/13/39

Stability

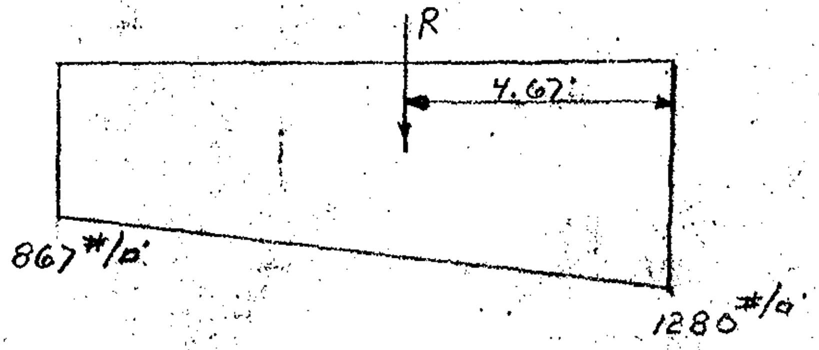
Case III - River Up, Dike Saturated, line of saturation (for uplift) assumed 4.5' above base.

Forces	↓	↑	→	←		↻	↺
C_1	2250						16,300
C_2	3750						18,750
E_1	8130						26,300
E_2	625						5,620
Pe_1 $12.5^2 \times \frac{1}{2} \times 80$				6250	4.16		26,000
P_w $8^2 \times \frac{1}{2} \times 62.5$			2000		7.17	14,350	
P_{ws} $8 \times 62.5 \times 4.5$			2250		2.25	5,060	
Pe_4 $4.5^2 \times \frac{1}{2} \times 80$			810		1.5	1210	
U_1 $8.0 \times \frac{1}{2} \times 62.5 \times 10 \times \frac{1}{2}$		1250			6.67	8350	
U_2 $4.5 \times 62.5 \times 10$		2800			5.0	14,000	
	14,755	4050	5060	6250		42,970	92,970
	$\Sigma V = 10,705$		$\Sigma H = 1190$			$\Sigma M = 50,000$	

$$\frac{\Sigma M}{\Sigma V} = \frac{50,000}{10,705} = 4.67' \quad \text{O.K.}$$

$$e = \frac{10}{2} - 4.67 = .33'$$

$$\text{Bearing} = \frac{10,705}{10}\left(1 \pm \frac{6 \times .33}{10}\right) = \begin{matrix} 1280\ \#/\square' \\ 867\ \#/\square' \end{matrix}$$



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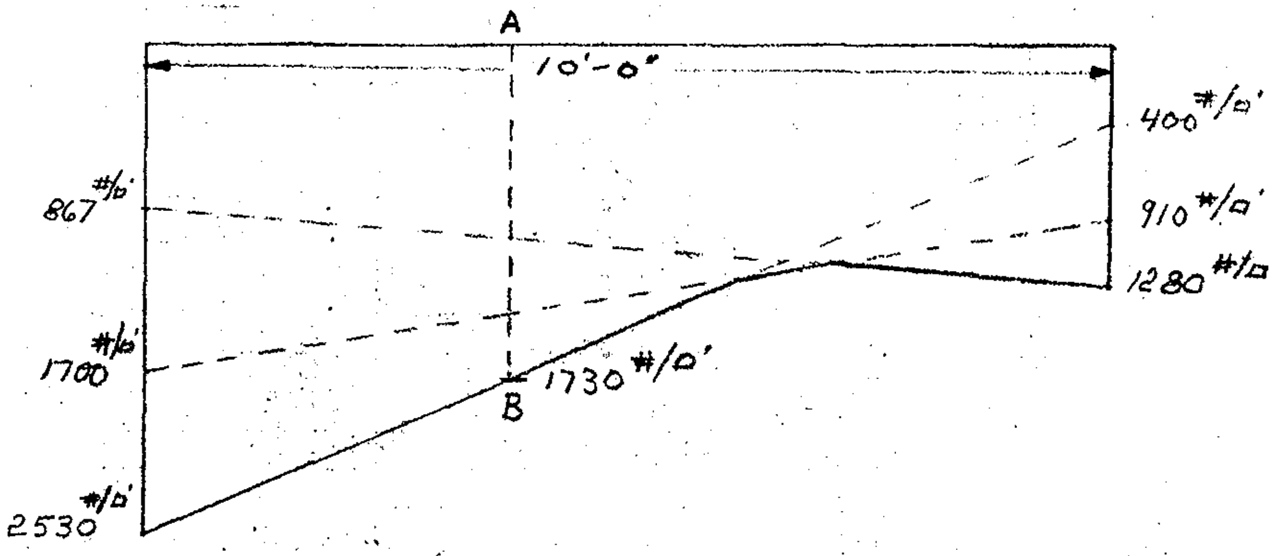
Max Bearing Diagram
(Combine Case I, II, & III.)



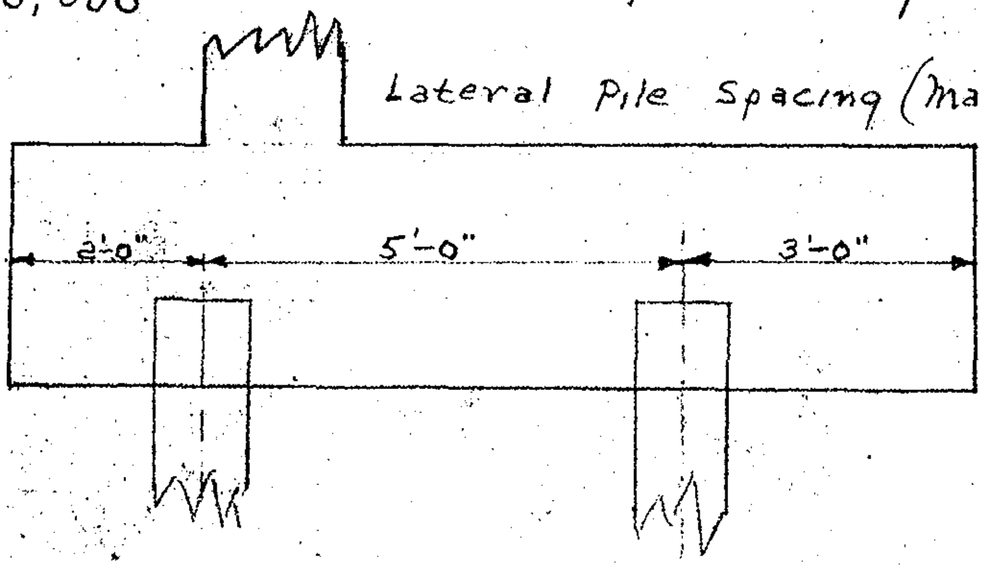
By planimeter Area = 8.1 □" $\frac{8.1}{2} = 4.05$ □"

Line A-B divides diagram into two equal parts, each equal to 8000# bearing, or 4.05 □".

Assume a 14" concrete pile, 30 Ton load.

$\frac{60,000}{8,000} = 7.5'$ c.c. (Spacing along wall of center piles)

Lateral Pile Spacing (Max Section)



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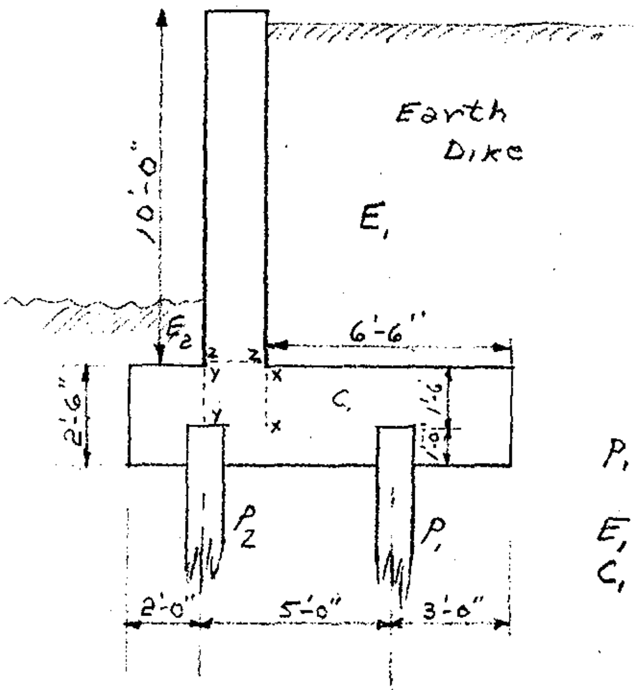
Design of Base
(maximum Section)



Punching Shear

$$\frac{60,000}{18 \times 14 \times 4} = \frac{60,000}{1000} = 60 \text{ \#/□"} \text{ O.K}$$

moment at x-x (Case I)

$46,000 = P_1$ $E_1 = 125$ #/cu.ft

P_1 ↑ $46,000 \times 3.5 = 161,000$ '#

E_1 $6.5 \times 10 \times 125 \times 7 = 56,600 \times 3.25 = 184,000$ '#

C_1 $6.5 \times 2.5 \times 150 \times 7 = 17,100 \times 3.25 = 55,500$ '#

73,700#↓ 239,500'#

$\Sigma V = 27,700$# $\Sigma M = 75,500$ '#

Assume moment is taken by 7'-0" of base. (Pile spacing = 7'-0" cc)

$$\frac{75,500}{7} = 10,800 \text{ '\#} \qquad \frac{27,700}{7} = 3950 \text{ \#}$$

$$A_s = \frac{10,800 \times 12}{18000 \times .884 \times 14.5} = \frac{130,000}{230,000} = .565 \text{ □"}$$

Use 7/8"φ @ 1'-0"

$$\text{Bond} = \frac{3950}{2.75 \times .884 \times 14.5} = 113 \text{ \#/□"} \text{ O.K.}$$

$$\text{Shear} = \frac{3950}{12 \times .884 \times 14.5} = 26 \text{ \#/□"} \text{ O.K.}$$

Run 7/8 φ @ 1'-0" thru top of slab.

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Design of Base
(continued)

Moment at x-x (Case III)

$49000 = P_1$ $E_1 = 125\ \#/\text{cu.ft.}$

P_1		$49000^{\#} \times 3.5' = 172000'^{\#}$
U_1 $160 \times \frac{1}{2} \times 6.5 \times 7$		$3650^{\#} \times 4.4 = 16000'^{\#}$
U_2 $280 \times 6.5 \times 7$		$12700^{\#} \times 3.25 = 41,200'^{\#}$
	↑ $65,350^{\#}$	$229,200'^{\#}$ ↶
$E_1 + C_1$	↓ $73,700^{\#}$	$239,500'^{\#}$ ↷
	$\Sigma V = 8,350^{\#}$ ↓	$\Sigma M = 10,300'^{\#}$ ↷

$\frac{7}{8}\phi$ @ 1'-0" O.K.

Shear & Bond O.K.

Consider Base as a Beam from Pile to pile, along the wall.

Max Bearing $= 2530\ \#/\square'$

$$\frac{wl^2}{10} = \frac{2530 \times 7^2}{10} = 12,400'^{\#}$$

$$\frac{wl}{2} = \frac{2530 \times 7}{2} = 8,850^{\#}$$

$$A_s = \frac{12,400 \times 12}{18000 \times .884 \times 14.5} = .65\ \square''$$ use $\frac{7}{8}$" ϕ @ 1'-0" cc

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Design of Base (Continued)

Moment at x-x (Case II)

50,000 = P_1 E_1 = 100

P_1 ↑ 50,000 × 3.5 = 175,000 '#

E_1 6.5 × 10 × 100 × 7 = 45,500 × 3.25 = 148,000 '#
C_1 17,100 55,500 '#
↓ 62,600 # 203,500

ΣV = 12,600 # ΣM = 28,500 '#

7/8 ϕ @ 1'-0" c.c.

Shear O.K. Bond O.K.

Design of stem (Dike side)

$10^2 \times \frac{1}{2} \times 80 = 4000^{\#} \times 3.33 = 13{,}300\ '^{\#}$

$A_s = \frac{13{,}300 \times 12}{18000 \times .884 \times 14.5} = .7\ \square''$ Use 3/4" ϕ @ 1'-0" c.c.
5/8 ϕ @ 1'-0" c.c.

$\text{Shear} = \frac{4000}{12 \times .884 \times 14.5} = 26\ \#/\square''$

Use 5/8" ϕ @ 1'-0" c.c. in R.R. side of stem.

Longitudinal stem steel is 5/8 ϕ @ 1'-0" c.c. in both faces

Project: Chicopee Mass - C-3a Stop-Log
Computation: Stability of Average Section
Computed by: R.H.M. Checked by: Date: 10/13/39

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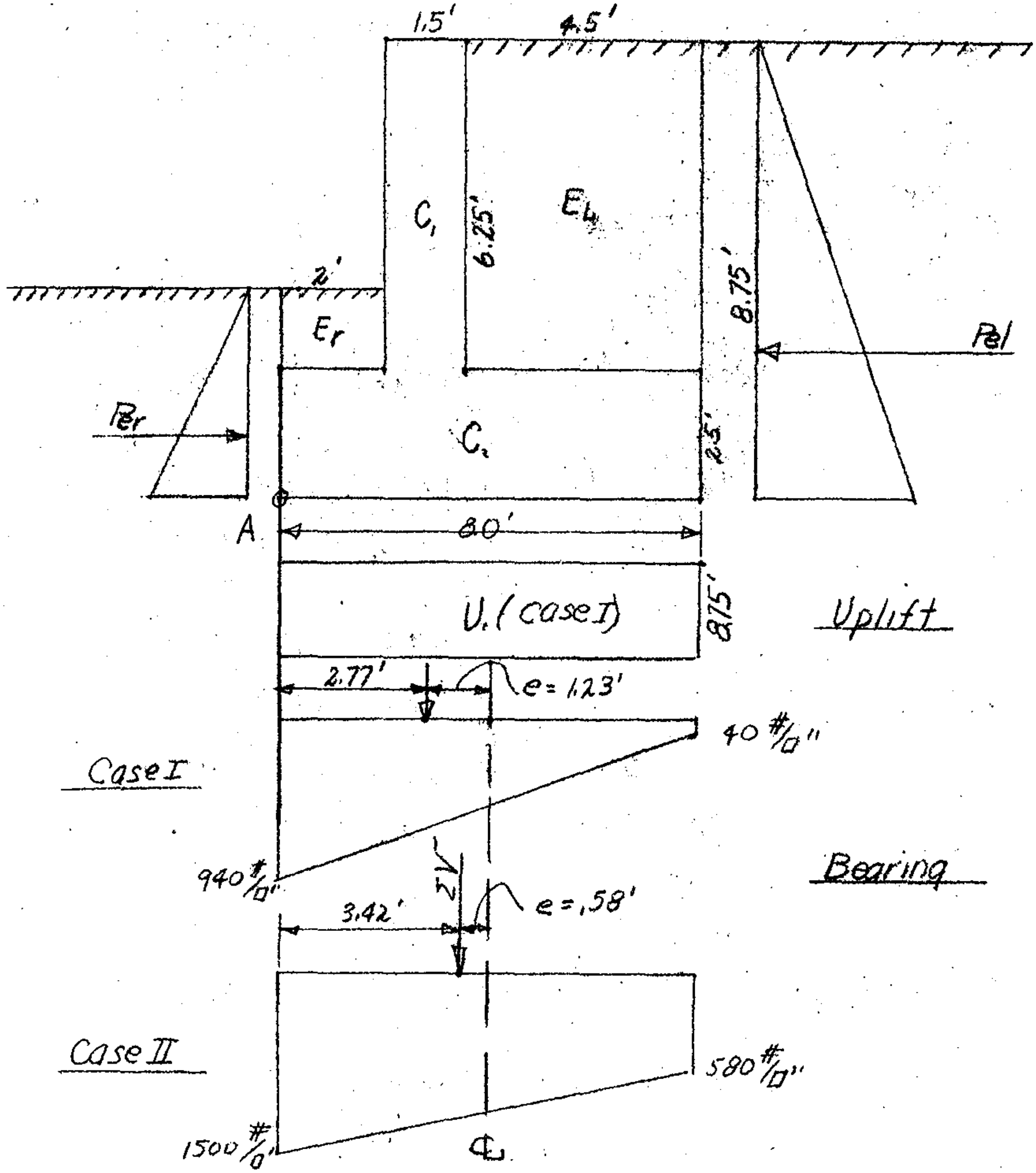
Case I River down - Saturated dike



...t Chicopee Mass - C-3a Stop Log
...nputation Stability of Average Section
...nputed by R.H.M. Checked by Date 10/13/39

Case I - River down - Saturated dike (80 #/□" lateral pressure)

Forces Acting		↓	↑	→	←	Arm	Moments ↷	Moments ↶
C_l	1.5×6.25×150	1410				2.75	3880	
C_r	2.5×8.0×150	3000				4.00	12000	
E_l	4.5×6.25×125	3520				5.75	20240	
E_r	1.5×2.0×125	380				1.00	380	
U.	8.75×6.25×8		4380			4.00		17520
P_el	$8.75^2 \times \frac{1}{2} \times 80$				3060	2.92		8940
P_er	$4^2 \times \frac{1}{2} \times 80$			640		1.33	850	
		8310	4380	640	3060		37350	26460
		ΣV = 3930 # ↓		ΣH = 2420 # ←			ΣM = 10890 '# ↷	

$$\frac{\Sigma M}{\Sigma V} = \frac{10890}{3930} = 2.77' \quad \frac{1}{3}\text{ Base} = 2.67 \quad e = 4 - 2.77 = 1.23'$$

O.K.

$$B.P. = \frac{3930}{8}\left(1 \pm \frac{6 \times 1.23}{8}\right) = 490 \begin{cases} 1.92 = 940 \text{ \#/□"} \\ .08 = 40 \text{ \#/□"} \end{cases}$$

Case II No Uplift - 80 #/□" lateral pressure from dike

ΣV = 8310 # ↓ ΣM = 28410 '# ↷

$$\frac{\Sigma M}{\Sigma V} = \frac{28410}{8310} = 3.42' \quad \frac{1}{3}\text{ Base} = 2.67' \; ; \; e = 4 - 3.42 = .58'$$

$$B.P. = \frac{8310}{8}\left(1 \pm \frac{6 \times .58}{8}\right) = 1040 \begin{cases} 1.44 = 1500 \text{ \#/□"} \\ .56 = 580 \text{ \#/□"} \end{cases}$$

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ct Chicopee Mass C-3a Stop Log

mputation Stability of Average Section

mputed by R.H.M. Checked by ______ Date 10/16/39

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Case III River down dry earth in dike

Forces Acting		↓	↑	→	←	Arm	Moments ↻	Moments ↺
C_1		1410				2.75	3880	
C_2		3000				4.00	12000	
E_L	9.5x6.25x100	2810				5.75	16160	
E_r	1.5x2.0x100					1.00	300	
P_{el}	$875^2 \times \frac{1}{2} \times 35$				1340	2.92		3910
P_{er}	$4^2 \times \frac{1}{2} \times 35$			280		1.33	370	
		7220	0	280	1340		32710	3910
		ΣV = 7220 # ↓		ΣH = 1060 # ←			ΣM = 28800 '# ↻	

$$\frac{\Sigma M}{\Sigma V} = \frac{28800}{7220} = 3.99' \text{ (call it } 4.00'\text{)} ; \quad \frac{1}{3} \text{ Base} = 2.67' ; \quad e = 4 - 3.99 = .01$$

$$B.P. = \frac{7220}{8} = 900 \,\#/\square'$$ equally distributed over entire base

Note Case II governs the spacing of the piles in the riverside; Case III governs the pile spacing in the dike side.

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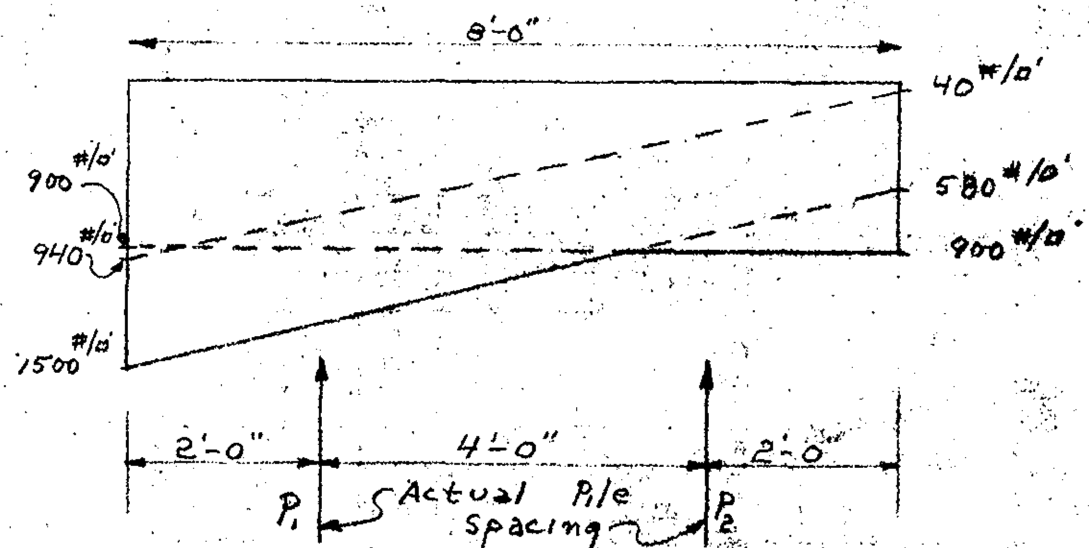
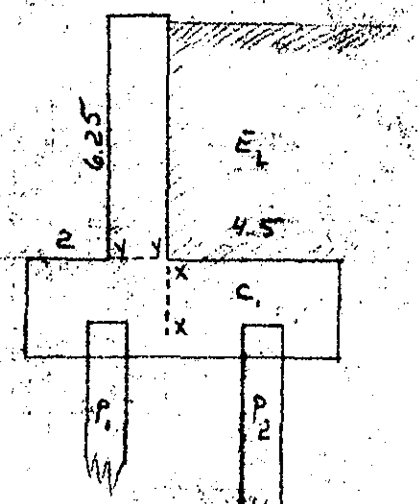
Maximum Bearing



Longitudinal pile spacing = 8'-0" c.c.

Max. load on P_1 = 40,000# = 20 tons

" " " P_2 = 28,800# = 14.4 tons.



Mom. at X-X

P_2		28,800 × 2.5 = 72,000'#
E_L	27,600 × 2.25 = 62,000'#	
C	13,500 × 2.25 = 30,000'#	
	41,100	92,000'#

ΣV = 12,300 ΣM = 20,000'#

V & M taken by 8'-0"

$\frac{12,300}{8}$ = 1510# shear OK

$\frac{20,000}{8}$ = 2500'#

$\frac{5}{8}$"ϕ @ 1'-0" OK

Top & bottom transverse steel

A.12

Mom at y-y (stem)

$6.25^2 \times \frac{1}{2} \times 80 \times \frac{6.25}{3} = 3260'^{\#}$

$\frac{5}{8}\phi$ @ 1'-0" cc. O.K.

Consider base as a beam

Max Bearing = 1500 #/□'

$\frac{wl^2}{10} = \frac{1500 \times 64}{10} = 9600'^{\#}$

$A_s = \frac{9600 \times 12}{18000 \times .884 \times 14.5} = .5^{\square''}$ $\frac{7}{8}\phi$ @ 1'-0" cc

Use $\frac{7}{8}\phi$ for all longitudinal base steel.

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Page 1

Subject Chicopee Mass C-3a

Computation Wall on Piles at Moore Drop Forge Plant

Computed by W. S. Jr. Checked by .... Date July 29, 1939

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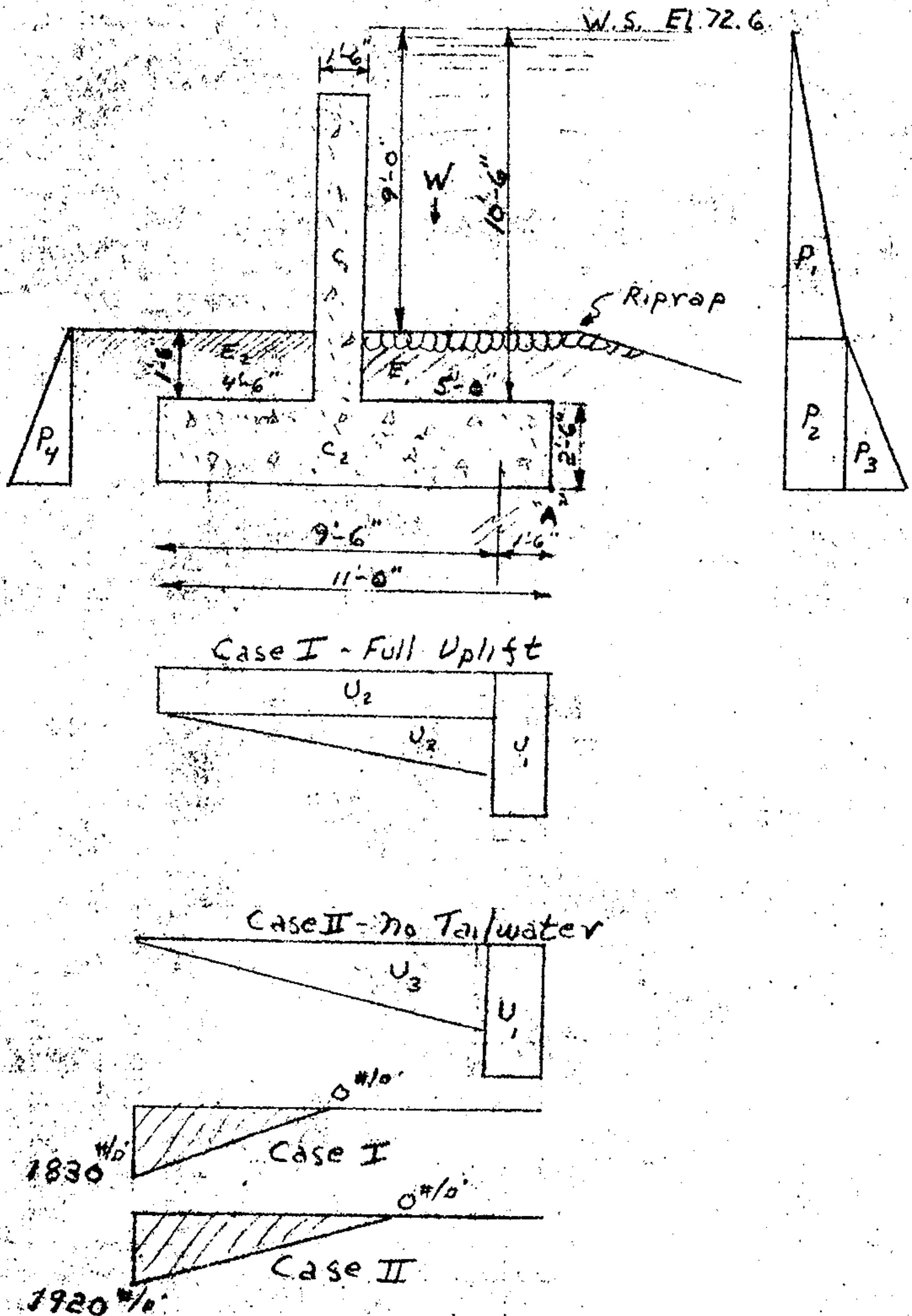

B1

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...ject Chicopee, Mass. C.3a

...putation Wall on Piles

...mputed by W.S.J. Checked by PSM Date

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Case I Full Uplift

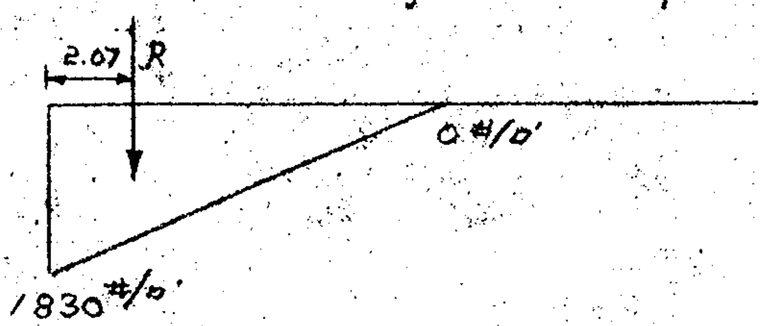
moments at "A"

Forces	↓	↑	→	←		↻	↺
C_1 8.5×1.5×150	1900				5.75		10,950
C_2 11×2.5×150	4130				5.5		22,800
E_1 1.5×5×125	940				2.5		2,350
E_2 1.5×4.5×125	843				8.75		7,390
W 9×5.0×62.5	2810				2.5		7,030
P_1 9^2×½×62.5				2535	7.0		17,750
P_2 9×62.5×4.0				2250	2.0		4,500
P_3 4^2×½×80				640	1.33		850
P_4 4^2×½×80			640		1.33	850	
U_1 13×62.5×1.5		1220			.75	920	
U_2 4×62.5×9.5		2380			6.25	14,900	
U_3 9×62.5×½×9.5×½		1340			4.66	6240	
	10,623	4940	640	5425		22,910	73,620
	ΣV = 5,683#		ΣH = 4785			ΣM = 50,710#	

$$\frac{\Sigma M}{\Sigma V} = \frac{50,710}{5,683} = 8.92' \qquad 11 - 8.92 = 2.08' \quad \text{O.K.}$$

$$\text{Bearing} = \frac{5683}{3 \times 2.07} \begin{cases} \times 2 = 1830 \ \#/\square' \\ \times 0 = 0 \ \#/\square' \end{cases}$$

Unbalanced horizontal force = 4785 #/ft.



st Chicopee Mass. C.3a
mputation Wall on Piles
mputed by W.S.J. Checked by RSM Date 10/2/39

Case II No Tailwater

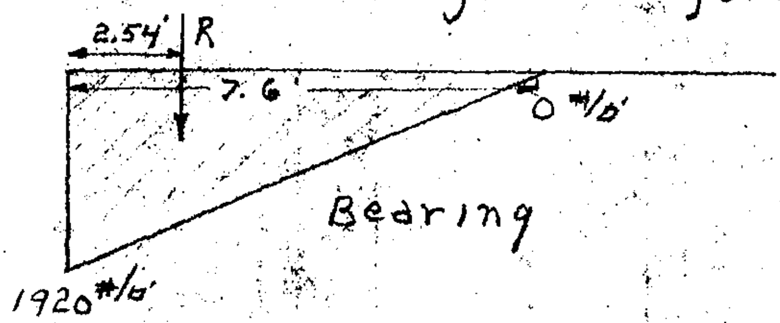
Moments at "A"

Forces	↓	↑	→	←		↻	↺
C_1	1900						10,950
C_2	4130						22,800
E_1	940						2,350
E_2 1.5×4.5×100	675				8.75		5,900
W	2810						7,030
P_1				2535			17,750
P_2				2250			4,500
P_3				640			850
P_4 $4^2 \times \frac{1}{2} \times 35$			280		1.33	370	
U_1		1220				920	
U_2		—				—	
U_3 $13 \times \frac{1}{2} \times 62.5 \times 9.5 \times \frac{1}{2}$		1930			4.66	9000	
	10,455	3150	280	5425		10,290	72,130
	ΣV = 7305		H = 5145			ΣM = 61,840	

$$\frac{\Sigma M}{\Sigma V} = \frac{61,840}{7305} = 8.46' \qquad 11 - 8.46 = 2.54' \quad \text{O.K.}$$

$$\text{Bearing} = \frac{7305}{3 \times 2.54} \begin{cases} \times 2 = 1920 \ \#/\square' \\ \times 0 = 0 \ \#/\square' \end{cases}$$

Unbalanced horizontal force = 5,145 #/ft.



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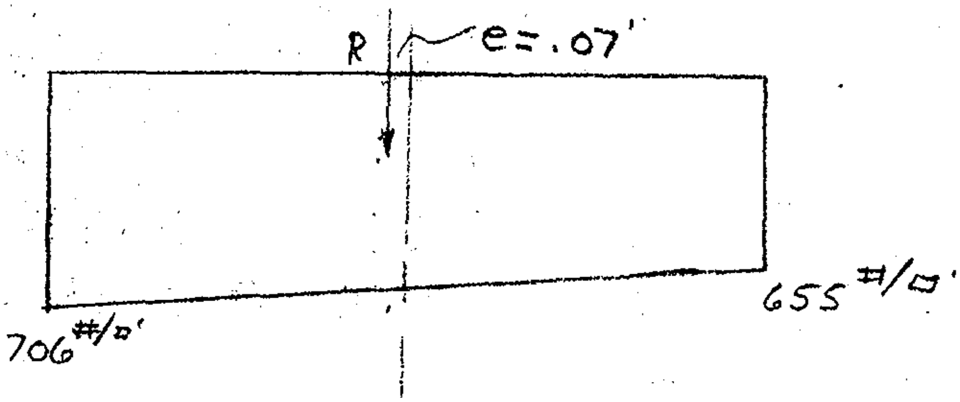
Case III River Down

Moments at A.

Forces	↓	↑	→	←		↻	↺
C_1	1900						10,950
C_2	4130						22,800
E_1 1.5 × 5 × 100	750				2.5		1,875
E_2 1.5 × 4.5 × 100	675				8.75		5,900
	7455						41,525

$\frac{\Sigma M}{\Sigma V} = 5.57'$ $e = 5.5 - 5.57 = -.07'$

$\text{Bearing} = \frac{7455}{11}\left(1 \pm \frac{6 \times .07}{11}\right) = 680 \begin{cases} \times 1.038 = 706 \text{ \#/□'} \\ \times .962 = 655 \text{ \#/□'} \end{cases}$



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t Chicopee, Mass C.3a

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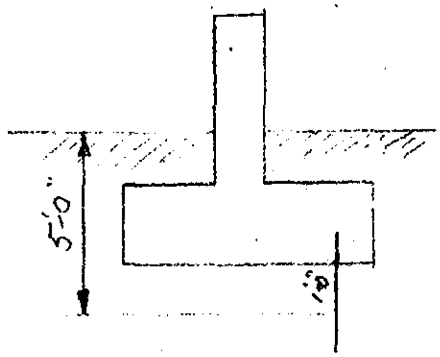
Resistance Against Lateral Loads.



Assume 1'-0" of sheet pile acts as a key.

Wt of cinder fill = 70 #/cu ft ±

"ϕ" of cinder fill = 34° ±

$$\text{Passive Resistance} = \frac{wh^2}{2}\left(\tan^2\left(45^\circ + \frac{\phi}{2}\right)\right)$$

$$" \quad " = \frac{70 \times 5^2}{2}(3.49) = 3{,}060 \text{ \#/ft} \rightarrow$$

Lateral load = 5145

Passive Resist. = 3060

2085 #/ft ← (unbalanced)

Piles are 8'-0" c. to c. (See page #6)

8 × 2085 = 16,700#

Two piles per row $\frac{16700}{2}$ = 8,350 #/pile

Shear per Pile

$$\frac{8350}{14 \times 14} = 42 \text{ \#/□"} \quad \text{O.K.}$$

B.S.

WAR DEPARTMENT

U. S. ENGINEER OFFICE, PROVIDENCE, R. I. **Page** 6

...t Chicopee Mass. C.3a

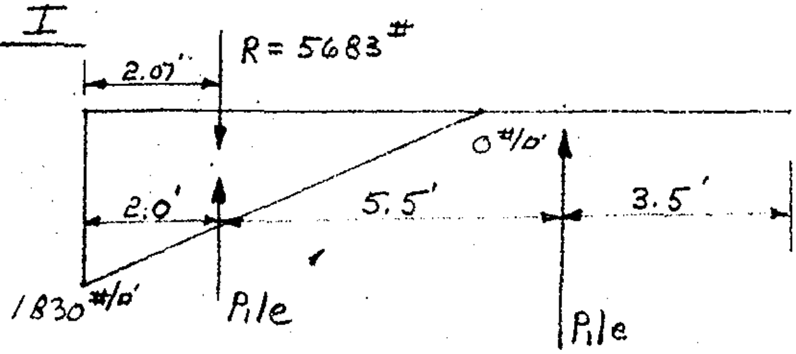
...mputation Wall on Piles

...mputed by W.S.J. **Checked by** RSM **Date** 10/9/39

U. S. GOVERNMENT PRINTING OFFICE 3—10528

Pile Spacing

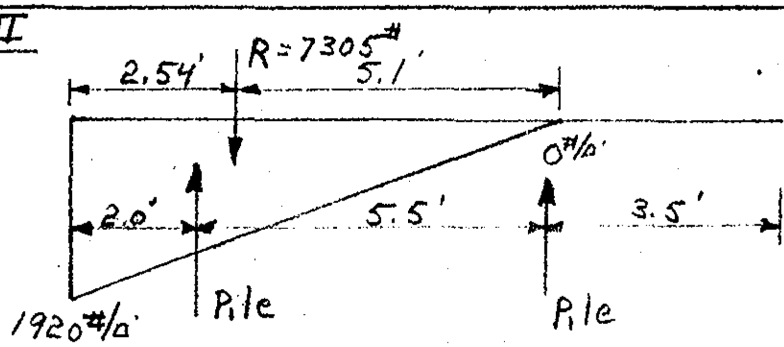
Case I



Rear pile carries entire load

$$\frac{60,000}{5683} = 10.5' \text{ c. to c. } (\perp \text{ to paper})$$

Case II

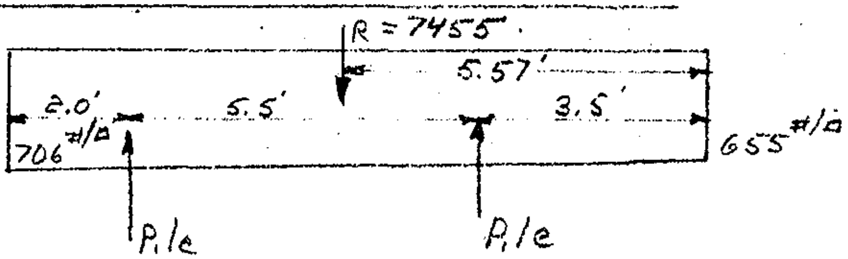


$$\frac{5.1 \times 7305}{5.5} = 6770\#/ \text{ on rear pile}$$

535#/ on front pile (neglect)

$$\frac{60,000}{6770} = 8.8' \text{ c. to c. } (\perp \text{ to paper})$$

Case III



5.57
3.50
2.07

$$\frac{7455 \times 2.07}{5.5} = 2800\#/ \text{ on rear pile}$$

4655#/ on front pile

B6

WAR DEPARTMENT

U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

Page 7

Subject Chicopee, Mass C. 3a.

Computation Wall on Piles

Computed by W. S. J. Checked by RSM Date 10/9/39

U. S. GOVERNMENT PRINTING OFFICE 3—10528

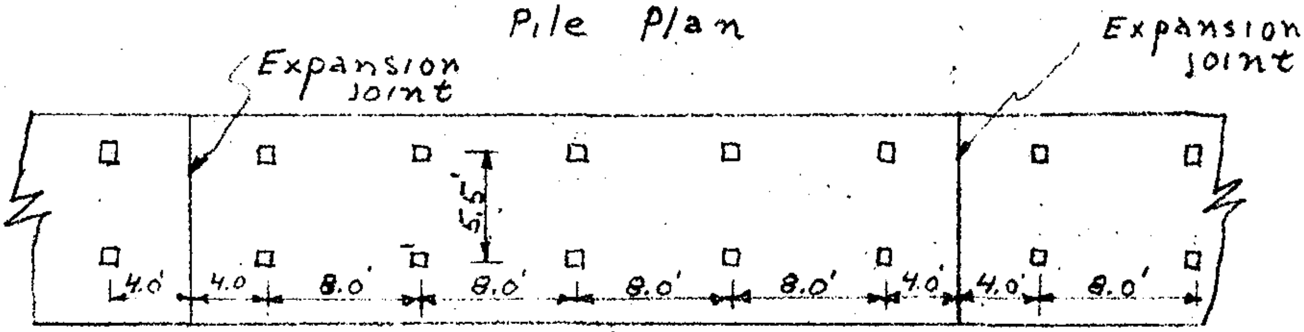
Pile Spacing

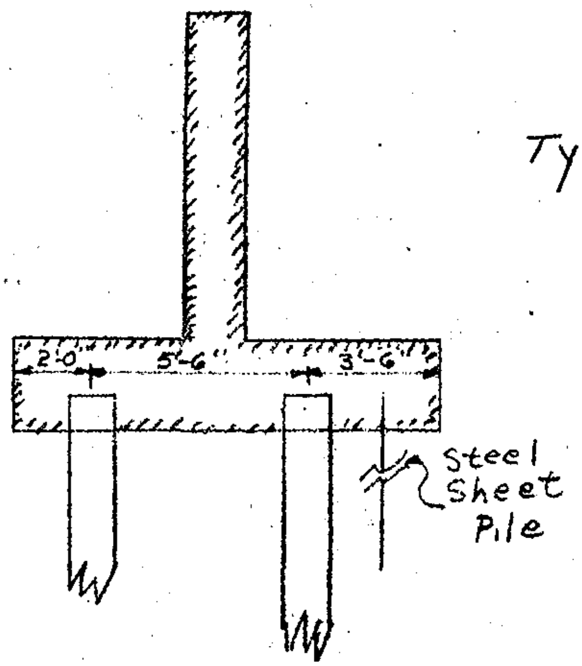
Case III (Cont'd)

$$\frac{60,000}{4655} = 12.8' \text{ c. to c.}$$

Case II Governs. — To permit equal spacing between expansion joints use 8'-0" c. to c.

Pile Plan





Typical Section at Piles

Subject: Chicopee Mass. C. 3a.
Computation: Wall on Piles
Computed by: W. S. J. Checked by: BMJ Date: 10/9/39

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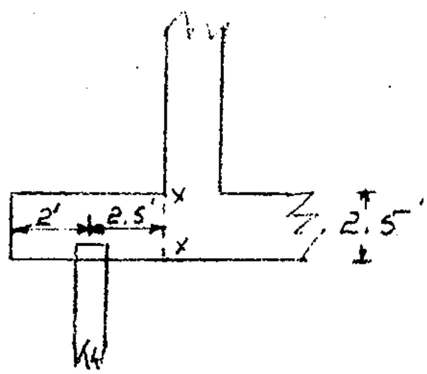
Design of Base

Landside Toe (Case II)

Cantilever action



$7305 \times 8 = 58{,}500\#/\text{Pile} \uparrow$

$58500 \times 2.5 = 146{,}000'\#$

$2.5 \times 4.5 \times 150 \times 8 = 13{,}500\# \downarrow$
$1.5 \times 4.5 \times 100 \times 8 = 5{,}400\# \downarrow$
$18{,}900\# \downarrow$

$18{,}900 \times 2.25 = 42{,}500'\#$

$\Sigma M = 146{,}000 - 42{,}500 = 103{,}500'\#$ moment x-x
$\Sigma V = 58{,}500 - 18{,}900 = 39{,}600'\#$ Shear at x-x

Assume that moment and shear is taken up by 8'-0" of wall.

$\frac{103{,}500}{8} = 12{,}900'\#$ $\sqrt{\frac{12900}{123}} = 10.2'' < 14.5''$ O.K.

$A_s = \frac{12{,}900 \times 12}{18000 \times .884 \times 14.5} = \frac{155{,}000}{230{,}000} = .674\,\square''$ use 1" ϕ @ 12" c.c. in bottom

$\text{Shear} = \frac{39{,}600}{8 \times 12 \times .884 \times 14.5} = \frac{39{,}600}{1220} = 32\#/\square''$

$\text{Bond} = \frac{4950}{3.14 \times .884 \times 14.5} = 120\#/\square''$ O.K.

Use $\frac{5}{8}''\phi$ @ 12" cc in top

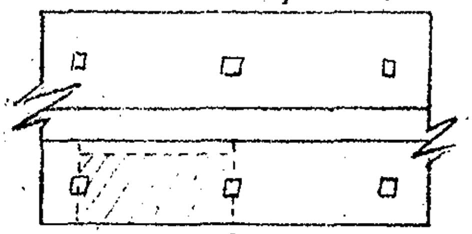
Subject Chicopee, Mass. C.3a.
Computation Wall on Piles
Computed by W. S. J. Checked by RSM Date 10/9/39

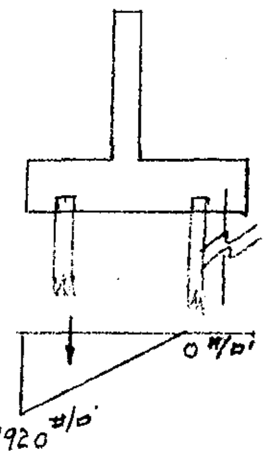
Design of Base

Landside Toe (Case II)

Beam action between pile rows.

Plan of Wall





Loading on beam

max load = 1920 #/'

$$\frac{Wl^2}{10} = \frac{1920 \times 8^2}{10} = 12,300'^{\#} = \text{moment}$$

$$\frac{Wl}{2} = \frac{1920 \times 8}{2} = 7,700^{\#} = \text{shear}$$

$$A_s = \frac{12300 \times 12}{18000 \times .884 \times 14.5} = \frac{147,500}{230,000} = .64^{\square''}$$

Use 1" ϕ top & bottom for the rear $4\frac{1}{2}$' of base

$$\text{Shear} = \frac{7700}{12 \times .884 \times 14.5} = 50^{\#/\square''} \quad \text{O.K.}$$

$$\text{Bond} = \frac{7700}{3.14 \times .884 \times 14.5} = 190^{\#/\square''} \quad \text{O.K.}$$

[Proje]ct: CHICOPEE, MASS. C.3a.
[Co]mputation: Wall on piles
[Co]mputed by: R.S.M. Checked by: Date: 10-17-39

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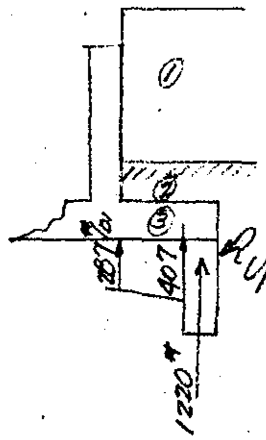
DESIGN OF BASE - Riverside toe.
CASE II

①	↓ 9 × 62.5 =	562 #/□'
②	↓ 1.5 × 125 =	188
③	↓ 2.5 × 150 =	375
	↓ Total	1125 #/□'



Bearing = 0

R Uplift

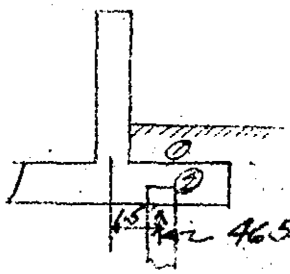
	V	arm	M
↓ 1125 × 5 =	5625 # ↓	2.5	14,000 '#
↑	1220 ↑	4.25	5,200
↑ 287 × 3.5 =	1000 ↑	1.75	1,750
↑ 120 × 3.5/2 =	210 ↑	2.4	500
Net total	3,195 # ↓		6,550 '# ↓

$A_s = \frac{6,550 \times 12}{18,000 \times .884 \times 14.5} = .34$ □"; use $\frac{3}{4}$"φ @ 1'-0" c-c.

$v = 3,195 / 12 \times 14.5 \times .884 = 20.8$ #/□"

CASE III

	V	arm	M
↑	4655	1.5	6980 '# ↑
↓ 1.5 × 100 × 5	750	2.5	1880 ↓
↓ 2.5 × 150 × 5	1880	2.5	4700 ↓
	2025 #		400 '# ↓



4655 #/lin. ft, from pg. 6

Stresses are negligible; use $\frac{5}{8}$"φ @ 1'-0"

BEAM ACTION BETWEEN PILE ROWS

$M = \frac{wl^2}{10} = \frac{655 + 678}{2} \times \frac{8^2}{10} = 4350$ '#

$V = \frac{wl}{2} = \frac{678 \times 8}{2} = 2720$ #

$A_s = \frac{4350 \times 12}{18000 \times .884 \times 14.5} = .22$ □", use $\frac{5}{8}$"φ @ 1'-0" c-c.

Shear = 2720 / 12 × 14.5 × .884 = 18 #/□"

Bond = $\frac{2720}{1.96 \times .884 \times 14.5} = 110$ #/□" OK

WAR DEPARTMENT

U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

Project: Chicopee, Mass. C.3a
Computation: Wall on Piles
Computed by: W.S.J. Checked by: RSM Date: 10/11/39

U. S. GOVERNMENT PRINTING OFFICE 3—10528

Design of Stem:

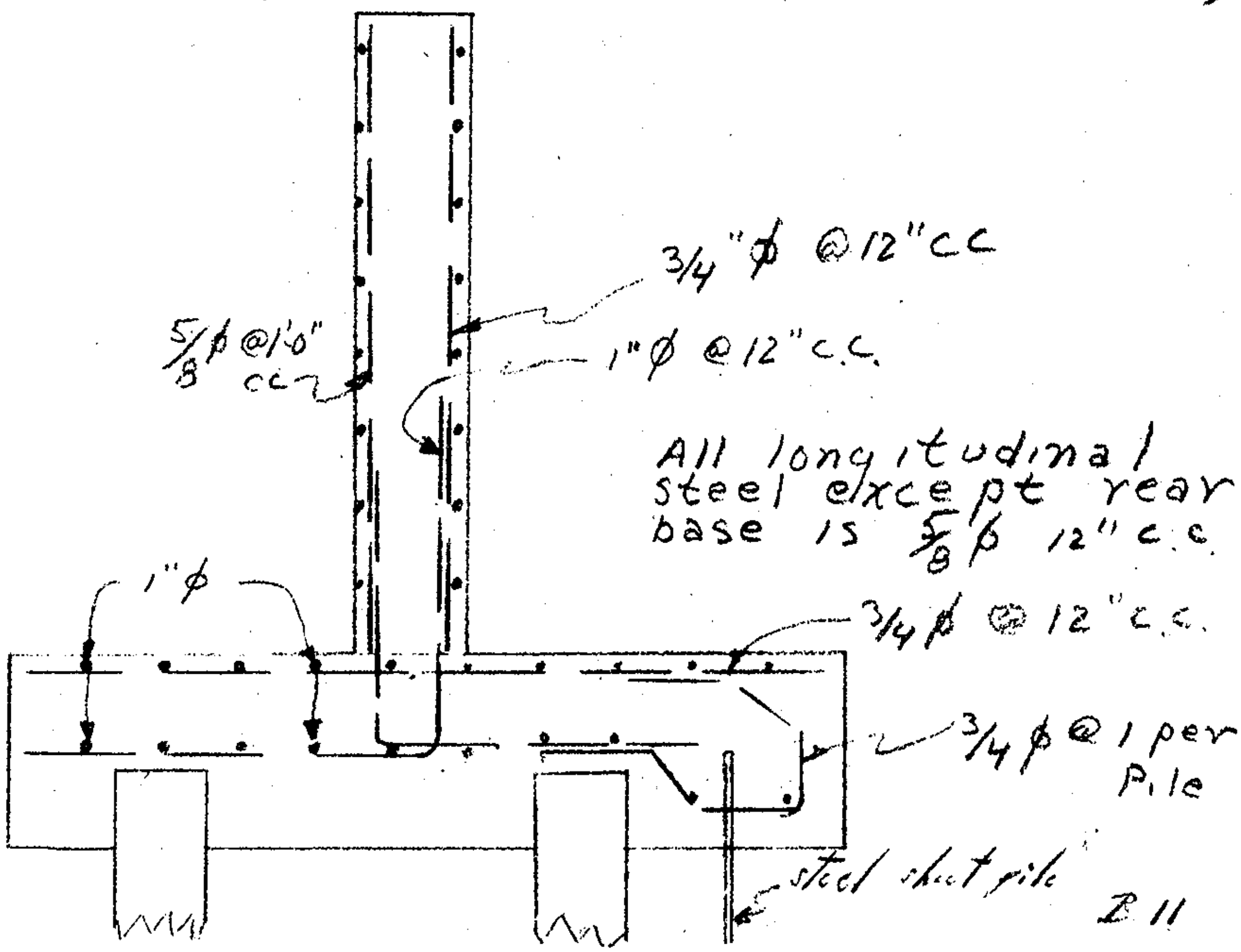
$10.5^2 \times \frac{1}{2} \times 62.5 = 3460^{\#} \times 3.5 = 12,000'^{\#}$

$A_s = \frac{12000 \times 12}{18000 \times .884 \times 14.5} = .63^{\square''}$ Use 1"ϕ @ 1'-0" cc

Shear $\frac{3460}{12 \times .884 \times 14.5} = 22^{\#/\square''}$ OK

Bond $\frac{3460}{3.14 \times .884 \times 14.5} = 86^{\#/\square''}$ OK.

Designed Section (see sheet no)

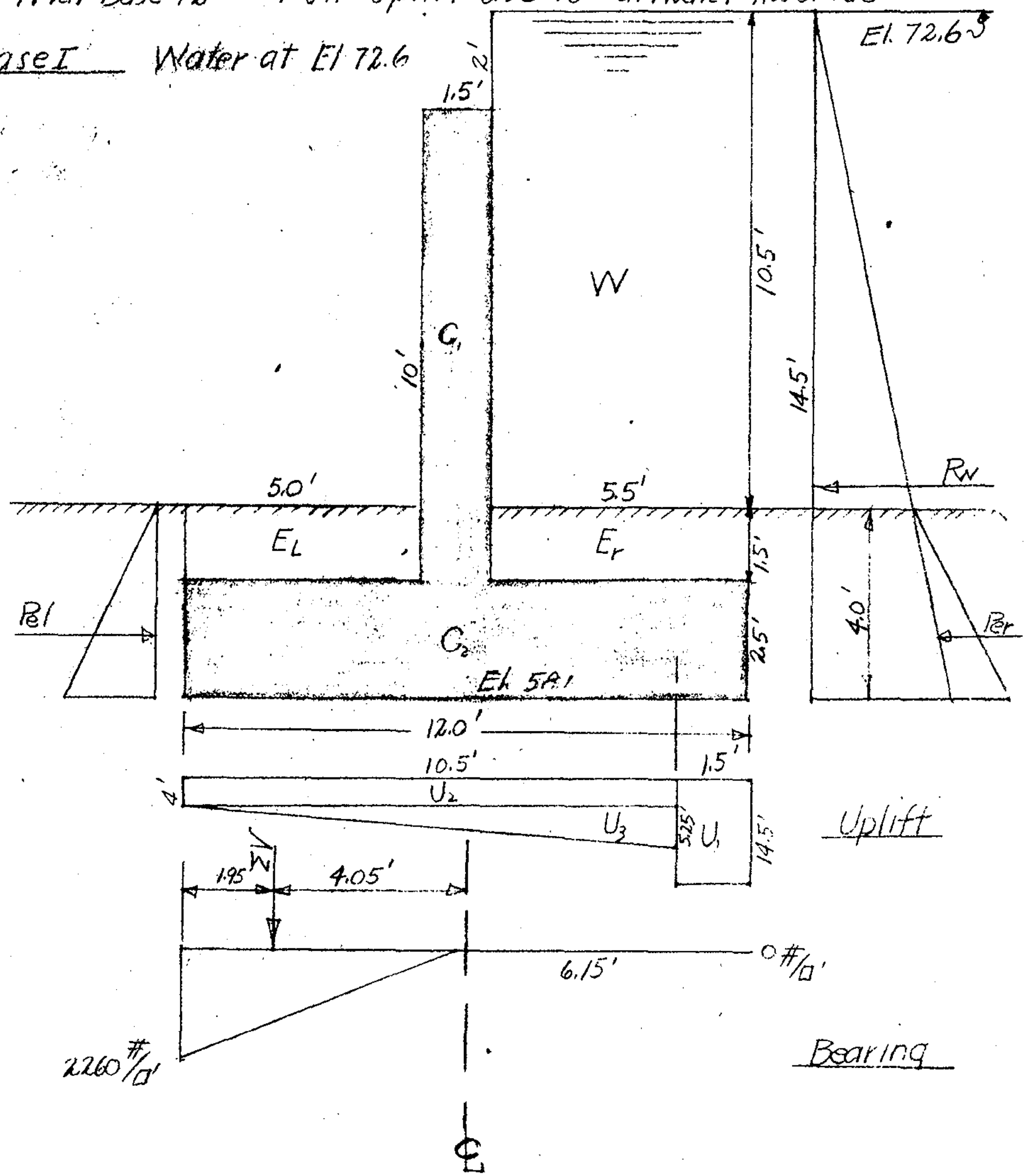


...ct Chicopee, Mass C-3a
...mputation Wall on Piles - 10' Stem (Above base) - Stability
...mputed by R.H.M. Checked by RSM Date 9/30/39

U. S. GOVERNMENT PRINTING OFFICE 3—10528

Trial Base 12' Full uplift due to tailwater Assumed

Case I Water at El 72.6



...t Chicopee, Mass. C.3a.
...putation Wall on Piles – 10 foot stem (Above base) - Stability
...puted by R.H.M. Checked by RSM Date 9/30/39

U. S. GOVERNMENT PRINTING OFFICE 3—10528

Trial Base 12' Full uplift due to tailwater assumed

Case I - Water at El. 72.6 - Full Uplift

Forces Acting		↓	↑	→	←	Arm	Moments ↻	Moments ↺
C_1	1.5x10x150	2250				5.75	12940	
C_2	2.5x12x150	4500				6.00	27000	
W	5.5x10.5x62.5	3610				9.25	33390	
Er	1.5x5.5x125	1030				9.25	9530	
E_L	1.5x5.0x125	940				2.50	2350	
U_1	14.5x1.5x62.5		1360			11.25		15300
U_2	4x10.5x62.5		2630			5.25		13810
U_3	$5.25\times\frac{10.5}{2}\times62.5$		1730			7.00		12110
P_w	$14.5^2\times\frac{1}{2}\times62.5$				6570	4.83		31730
P_{er}	$4^2\times\frac{1}{2}\times17.5$				140	1.33		190
P_{el}	$4^2\times\frac{1}{2}\times80$			640		1.33	850	
		12330	5720	640	6710		86060	73140
		ΣV = 6610 #↓		ΣH = 6070 ←			ΣM = 12920 '# ↻	

$$\frac{\Sigma M}{\Sigma V} = 1.95' \; ; \; \frac{1}{3}\text{Base} = 4' \; ; \; e = 6 - 1.95 = 4.05'$$

$$B.P. = \frac{6610}{3\times1.95}\times\begin{cases} 2 = 2\times1130 = 2260\ \#/\square'\ P.B.L. \\ 0 = 2\times0 = 0\ \#/\square'\ P.B.R \end{cases}$$

Note: The concrete piles on which the wall will rest will be designed to take care of the unbalanced horizontal force.

Project Chicopee; Mass. - C-39
Computation Wall on Piles - 10' Stem (Above base) - Stability
Computed by R.H.M. Checked by Date 10/4/39

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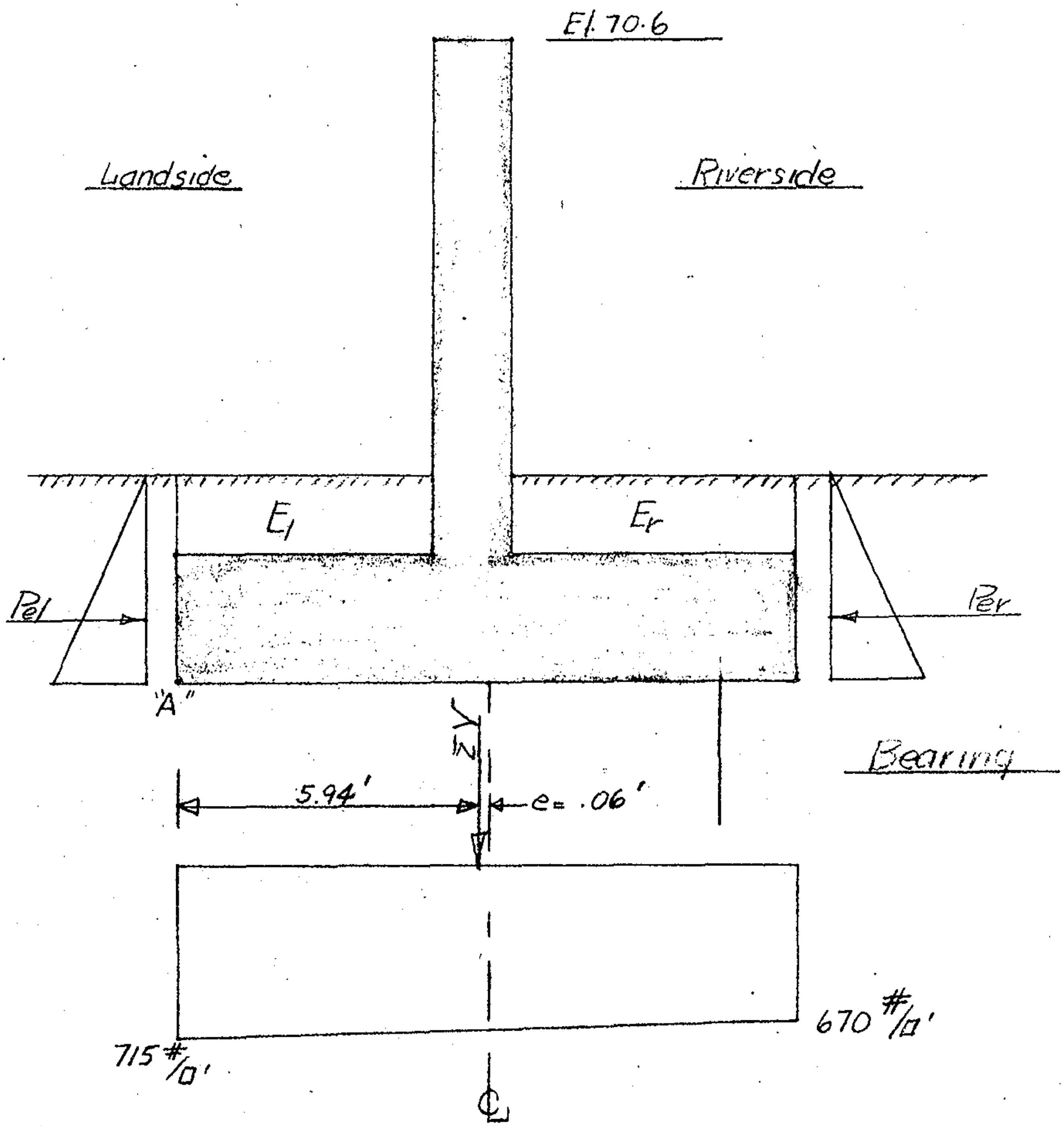
Case II - River down - no uplift



WAR DEPARTMENT

U. S. ENGINEER OFFICE, PROVIDENCE, R. I. Page 4

...t Chicopee - Mass C-3a

...mputation Wall on Piles - 10' Stem (Above base) - Stability

...mputed by R.H.M. Checked by ________ Date 10/4/39

U. S. GOVERNMENT PRINTING OFFICE 3—10528

Case II - River down - no uplift

Forces Acting		↓	↑	→	←	Arm	Moments ↷	Moments ↶
C_1		2250				5.75	12940	
C_2		4500				6.00	27000	
E_r	1.5x5.5x100	830				9.25	7680	
E_l	1.5x5.0x100	750				2.50	1880	
P_{er}	$4^2 \times \frac{1}{2} \times 35$				280	1.33		
P_{el}	$4^2 \times \frac{1}{2} \times 35$			280		1.33		
			0	280	280		51500	
		$\Sigma V = 8330$ # ↓		$\Sigma H = 0$			ΣM 49500 '# ↷	

$$\frac{\Sigma M}{\Sigma V} = \frac{49500}{8330} = 5.94' \; ; \; \tfrac{1}{3} \text{ Base} = 4' \quad \tfrac{2}{3} \text{ Base} = 8'$$

$$e = 6 - 5.94 = .06'$$

$$B.P = \frac{8330}{12}\left(1 \pm \frac{6 \times .06}{12}\right) = 694 \begin{cases} 1.03 = 715 \ \#/\square' \text{ P.B.L} \\ .97 = 670 \ \#/\square' \text{ P.B.R} \end{cases}$$

WAR DEPARTMENT

U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

Project: Chicopee : Mass. C-39

Computation: Wall on piles – 10' stem (above base) Stability

Computed by: R.H.M. Checked by: RSM Date: 10/5/39

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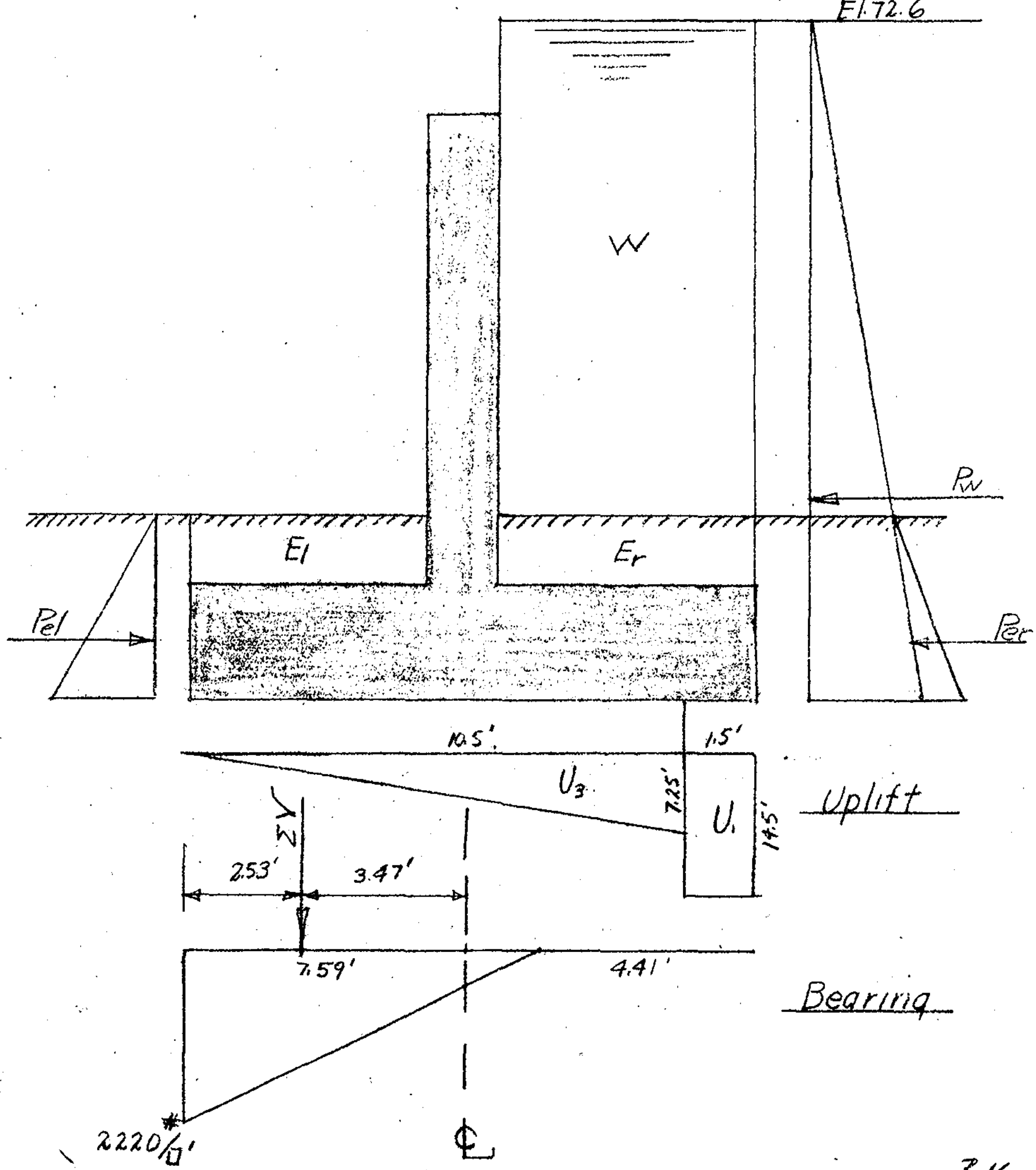
Case III – No tailwater – Condition of max. base pressure



...t Chicopee : Mass - C.3a
...nputation Wall on piles - 10' stem (above base) - Stability
...nputed by R.H.M Checked by RSM Date 10/4/39

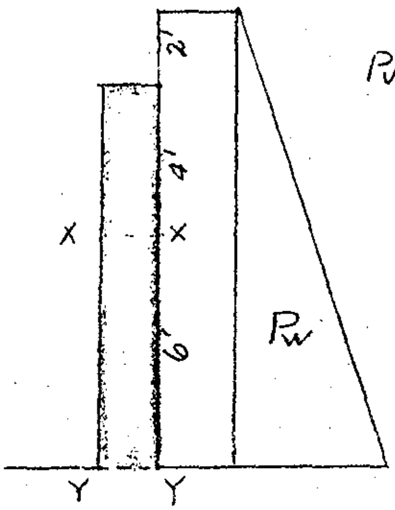
U.S. GOVERNMENT PRINTING OFFICE 3—10528

Case III - No tailwater - Condition of max. base pressure

Forces Acting		↓	↑	→	←	Arm	Moments ↻	Moments ↺
C_1		2250				5.75	12940	
C_2		4500				6.00	27000	
W		3610				9.25	33390	
Er		1030				9.25	9530	
El	1.5 x 5 x 100	750				2.50	1880	
U_1			1360			11.25		15300
U_3	$7.25 \times \frac{10.5}{2} \times 62.5$		2380			7.00		16660
P_w					6570	4.83		31730
P_{er}					140	1.33		190
P_{el}	$4^2 \times 1/2 \times 35$			280		1.33	380	
		12140	3740	280	6710		85120	63880
		ΣV = 8400# ↓		ΣH = 6430 # ←			ΣM = 21240 #↻	

$$\frac{\Sigma M}{\Sigma V} = \frac{21240}{8400} = 2.53' \quad e = 6 - 2.53 = 3.47'$$

$$B.P. = \frac{8400}{3 \times 2.53}\begin{pmatrix}2\\0\end{pmatrix} = 1110 \times \begin{cases}2 = 2220 \ \#/\square'' \\ 0 = 0 \ \#/\square''\end{cases}$$



At sect X---X

P_w: $6^2 \times \frac{1}{2} \times 62.5 = 1130 \times 2 = 2260$ #'

$$A_s = \frac{12 \times 2260}{18000 \times .884 \times 14.5} = \frac{27120}{231000} = .12\ \square''$$

At Sect Y---Y (Neglect Per, Pel)

P_w: $12^2 \times \frac{1}{2} \times 62.5 = 4500 \times 4 = 18000$ #'

$$d = \sqrt{\frac{18000}{123}} = \sqrt{147} = 12.2'' \text{ OK}$$

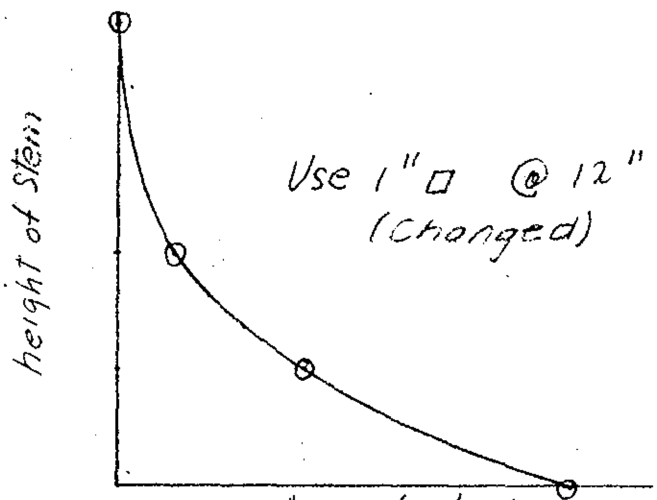
Use 18 - 3.5 = 14.5"

$$A_s = \frac{12 \times 18000}{18000 \times .884 \times 14.5} = \frac{216000}{231000} = .94\ \square''$$

Use 1" □ @ 12" c.c

$$\text{Shear} = \frac{4500}{12 \times .884 \times 14.5} = \frac{4500}{154} = 29.2\ \#/\square'' < 60\ \#/\square'' \text{ O.K.}$$

$$\text{Bond} = \frac{4500}{4 \times .884 \times 14.5} = \frac{4500}{51.3} = 87.7\ \#/\square'' < 150\ \#/\square''$$

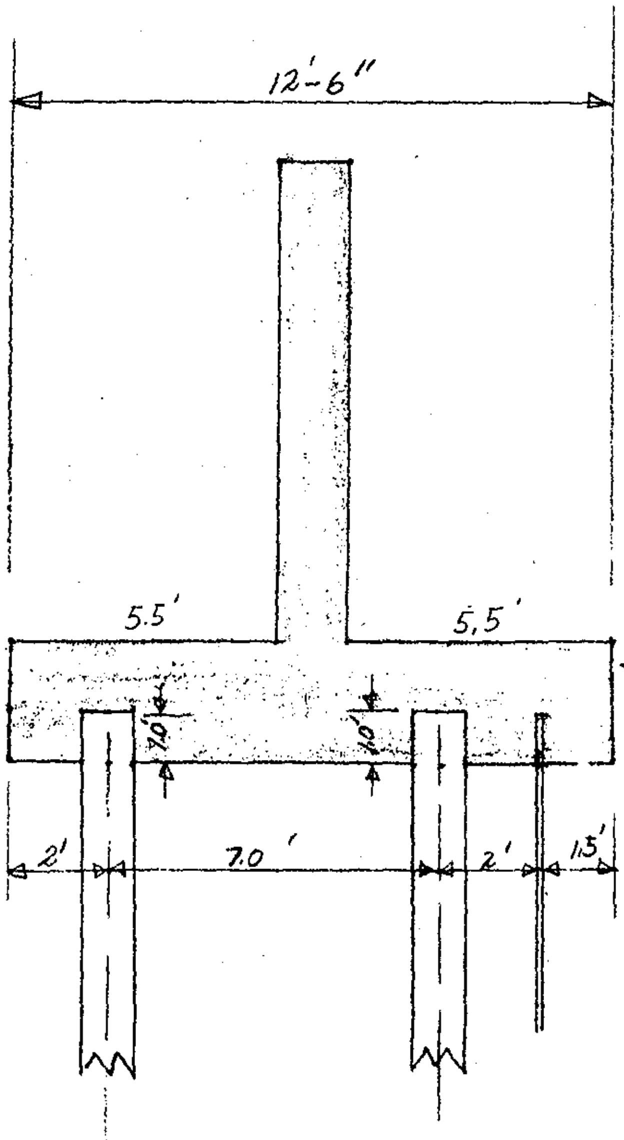


Use 1" □ @ 12" c.c.
(Changed)

II 18

Subject Chicopee - Mass. C-39
Computation Wall on piles - 10' stem (Above base) - Pile spacing
Computed by R.H.M. Checked by RSM Date 10/6/39

Note Piles to be driven in two rows; one two feet from landside end of base and the other two feet inside of the center line of the sheet piling. The two feet represents cover and is measured to the center of the piles. It was thought desirable to have the resultant fall a little inside of the land side pile (ie to fall between the two piles). So, in the case of this wall (10' stem) where the resultant in case I fell 1.95' from landside end of base, 6" was arbitrarily added to the land side slab. (See figure below)



ct Chicopee - Mass - C-3a
mputation Wall on piles - 10' stem (Above base) - Pile spacing
mputed by R.H.M. Checked by PSM Date 10/6/39

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Pile spacing (landside row of piles) (Case III governs)

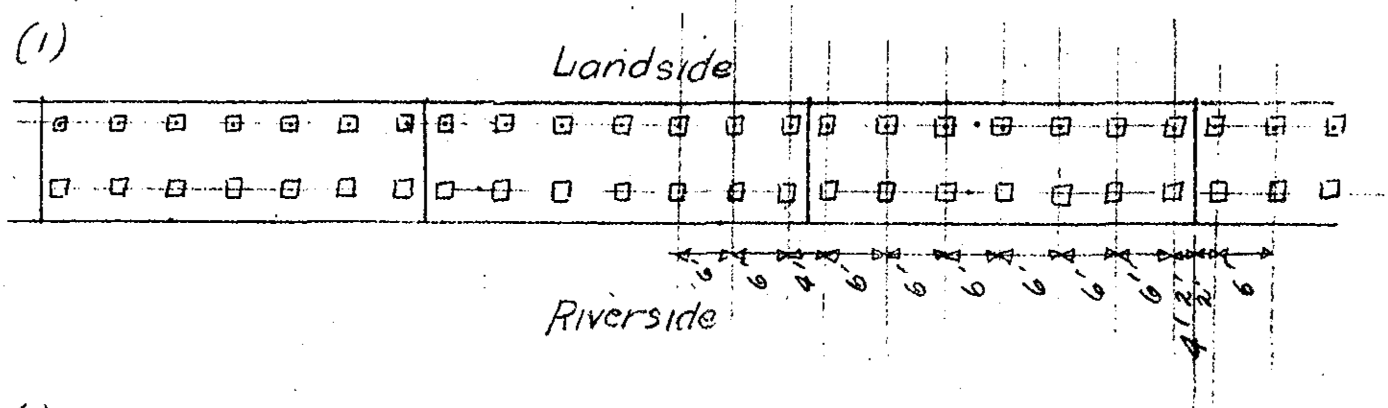
$$S = \frac{60,000}{8400} = 7.14'$$

Pile spacing (riverside row of piles) Case II governs.

$$S = \frac{60000}{\frac{4.44 \times 8330}{7}} = \frac{60,000 \times 7}{4.44 \times 8330} = \frac{420000}{36990} = 11.35'$$

Pile spacing will be taken at 6' (riverside and landside both). This is chosen because of (1) its easy adaptability to 40' lengths of wall (2) the necessity of balancing the sliding tendency largely by means of the piles.

(1)



(2) Unbalanced horizontal force — balanced by piles in shear

Horizontal force for 6' of wall = 6430 X 6 = 38580 #

Max. shearing resistance of piles = 2 X 14 X 14 X 60 =
= 120 X 196 = 23520 #

Horizontal force still unbalanced = 38580 - 23520 = 15,060 #

Max. available passive resistance: (1' of sheet pile included)

$$P.R = \frac{wh^2}{2}\left(\tan^2\left[45° + \frac{\phi}{2}\right]\right)$$

$$= \frac{70 \times 5^2}{2}(3.49) = 3060^{\#} \text{ or } 6 \times 3060 = 18360^{\#} > 15060^{\#} \text{ O.K.}$$

I 20

WAR DEPARTMENT

U. S. ENGINEER OFFICE, PROVIDENCE, R. I. — Page 10

Subject: Chicopee - Mass. - C-39

Computation: Wall on piles - 10' stem - Design of landside base steel

Computed by: R.H.M. Checked by: RSM Date: 10/6/39

Note

The base steel will be designed for two types of beam action

a) Cantilever out from stem over the six ft. of wall between piles.

b) Continuous beam action longitudinally between the piles.

a) Loads down

↓ E_1: 6 x 1.5 x 5 x 100 = 4500 x 2.5 = 11250 '#

↓ C_2: 6 x 2.5 x 5 x 150 = 11250 x 2.5 = 28130 '#

U_3: 3.45 x ½ x 5 x 62.5 x 6 = 3230 x 1.67 = 5390 '#

↑ Pile load. 60000 x 3.5 = 210,000 '#

176,000 #

$$d = \sqrt{\frac{29330}{123}} = \sqrt{238}$$

29330 #/ft of wall

15.5" ok Use 17"

$$A_s = \frac{12 \times 29330}{18000 \times .884 \times 17} = \frac{351960}{270300} = 1.30 \square''$$

Use staggered { 1" □ @ 12" c.c ; 5/8" φ @ 12" c.c }

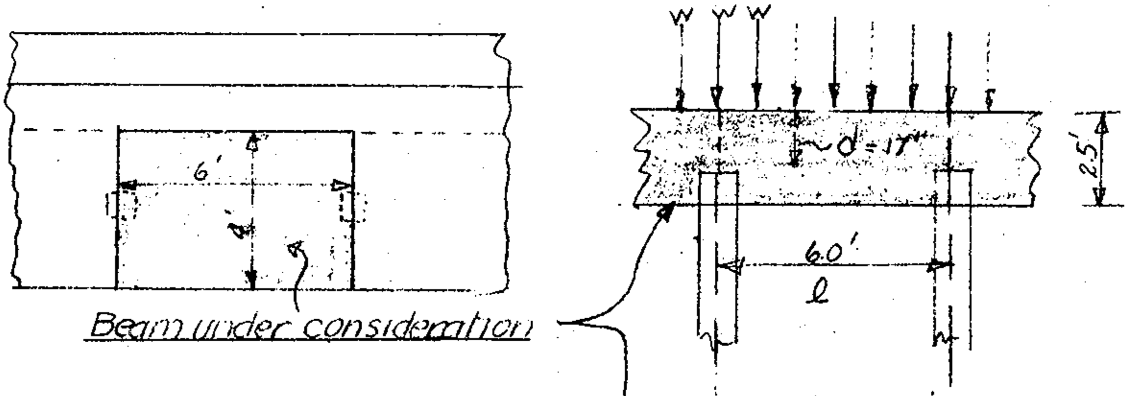
$$Shear = \frac{47480}{12 \times .884 \times 17 \times 6} = \frac{47480}{180.2 \times 6} = \frac{47480}{1081} = 43.9 \#/\square'' < 60 \#/\square'' \text{ ok}$$

$$Bond = \frac{47480}{5.96 \times .884 \times 17 \times 6} = \frac{47480}{537} = 88.4 \#/\square'' < 150 \#/\square'' \text{ ok}$$

Chicopee - Mass - C-3a
putation: Wall on piles : 10' stem - Design of landside base steel
puted by: R.H.M. Checked by: PSM Date: 10/7/39

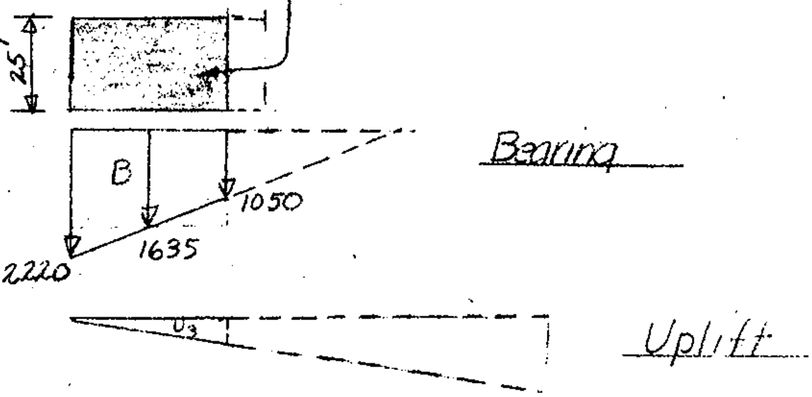
U. S. GOVERNMENT PRINTING OFFICE 3—10528

b) Continuous beam action $\left(\text{Max Mom} = \frac{wl^2}{10}\right)$



Load down (per ft)

B : 4X1635 = 6540#



Load up
U_3 : Neglect

$$\text{Max. moment} = \frac{wl^2}{10} = \frac{6540 \times 36}{10} = 23540 \text{ '\# (For 4' width of beam)}$$
or 5890 '# (For beam 1' wide)

$$d = \sqrt{\frac{5890}{123}} = \sqrt{47.9} = 7'' \quad \text{O.K} \quad \text{Use } 17''$$

$$A_s = \frac{5890 \times 12}{18000 \times .884 \times 17} = \frac{70680}{270{,}300} = .26 \square'' \quad \text{Use } \tfrac{3}{4}''\phi \text{ @ } 12'' \text{c.c}$$
$A_s = .44 \square''$ $\Sigma_0 = 2.36$

$$\text{Shear} = \frac{6540 \times 3}{12 \times .884 \times 17 \times 4} = \frac{4910}{180.2} = 27.2 \#/\square'' < 60 \#/\square'' \text{ O.K.}$$

$$\text{Bond} = \frac{4910}{2.36 \times .884 \times 17} = \frac{4910}{35.47} = 138\# < 150 \#/\square'' \text{ O.K.}$$

WAR DEPARTMENT

U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

Page 12

ct Chicopee - Mass - C-3a.

mputation Wall on piles - 10' stem - Design of Riverside Base Slab

mputed by R.H.M. Checked by RSM Date 10/7/39

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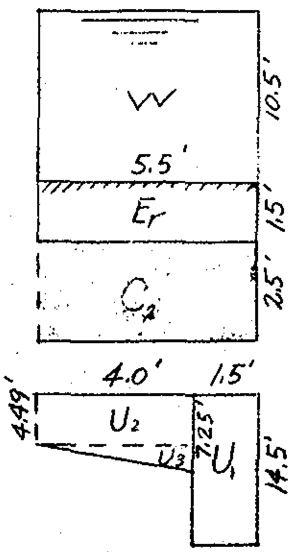
Note Riverside base slab designed as cantilever beam Case III governs for top steel.



Loads down (For one ft of wall)

W: 10.5 X 5.5 X 62.5 = 3610 X 2.75 = 9930

E_r: 1.5 X 5.5 X 125 = 1030 X 2.75 = 2830

C_2: 2.5 X 5.5 X 150 = 2060 X 2.75 = 5670

6700#↓ 18430 '#

Loads up

U_1: 14.5 X 1.5 X 62.5 = 1360 X 4.75 = 6460

U_2: 4.49 X 4.0 X 62.5 = 1120 X 2.00 = 2240

U_3: 2.76 X 4.0 X 62.5/2 = 350 X 2.67 = 920

Pile bearing (neglect)

2830#↑ 9620 '#

$\Sigma V = 3870$#↓, $\Sigma M = 8810$ '#

$$d = \sqrt{\frac{8810}{123}} = \sqrt{71.5} = 8.5'' \text{ OK} \quad \text{Use } 17''$$

$$A_s = \frac{8810 \times 12}{18000 \times .884 \times 17} = \frac{105800}{270300} = .39 \square'' \quad \tfrac{3}{4}''\phi @ 12'' \text{ c.c.}$$

$$\text{Shear} = \frac{3870}{12 \times .884 \times 17} = \frac{3870}{180.2} = 21.4 \#/\square'' < 60 \#/\square'' \text{ OK}$$

$$\text{Bond} = \frac{3870}{1.96 \times .884 \times 17} = \frac{3870}{29.45} = 131 \#/\square'' < 150 \#/\square'' \text{ O.K.}$$

P 33

...t Chicopee - Mass - C-39
...putation Wall on piles - 10' stem - Design of Riverside Base Steel
...puted by R.H.M. Checked by RSM Date 10/7/39

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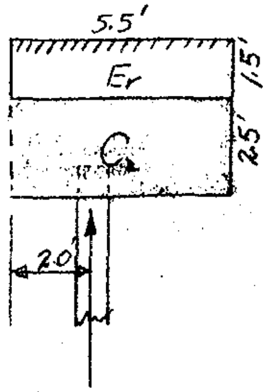
Bottom Steel - (Case II governs)



Loads down (For 6' of wall)

E_r: 1.5X5.5X100X6 = 4950 X 2.75 = 13160
C_2: 2.5X5.5X150X6 = 12 380 X 2.75 = 34050
17330 # 47210 '#

Load up

P: 31700 X 2 = 63400 '#

ΣV = 14370 # ΣM = 16190 '#

$P = \frac{8330 \times 6 \times 4.44}{7} = 31700$# or 2400 #/ft or 2700 '#/ft

$d = \sqrt{\frac{2700}{123}} = \sqrt{22} = 4.7''$ ok. Use 17''

$A_s = \frac{2700 \times 12}{18000 \times .884 \times 17} = \frac{32400}{270,300} = .12$ □" Use min. steel allowed 5/8"φ @ 12" c.c.

$Shear = \frac{2400}{12 \times .884 \times 17} = \frac{2400}{180.2} = 13.3$ #/□" ok.

$Bond = \frac{2400}{1.96 \times .884 \times 17} = \frac{2400}{29.5} = 81.3$ #/□" O.K.

(Note: 1.96 perimeter of a 5/8"φ bar which will probably be used at 12" c.c.)

WAR DEPARTMENT

U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

Page 14

ject Chicopee - Mass - C-3a

omputation Wall on piles - 10' stem - Steel Reinforcement

omputed by R.H.M Checked by ______ Date 10/9/39

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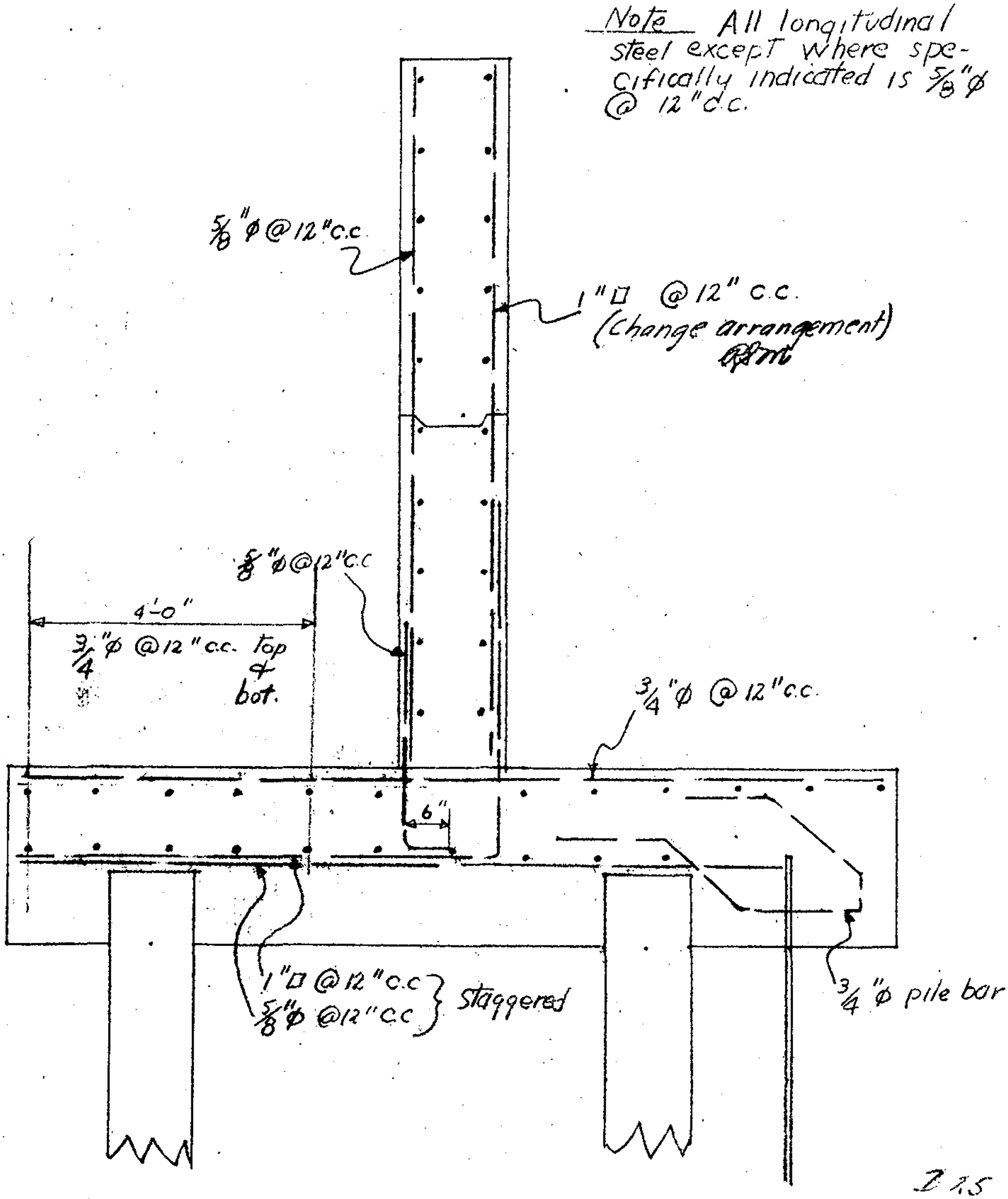


Z 15

WAR DEPARTMENT

U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

Subject: Chicopee - Mass · C3a

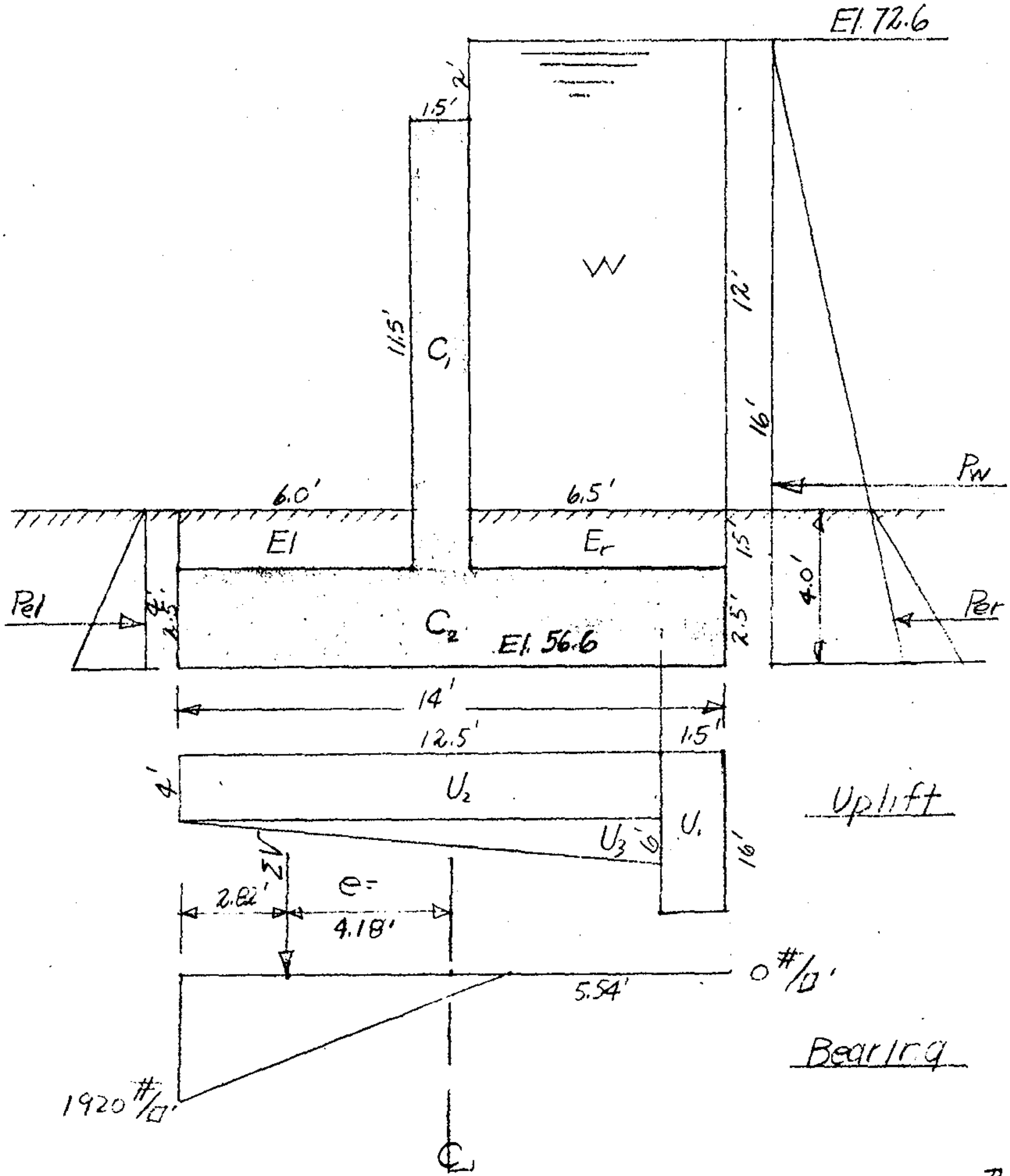
Computation: Wall on Piles - 11.5' stem Stability

Computed by: R.H.M. Checked by: RCM Date: 9/30/39

U. S. GOVERNMENT PRINTING OFFICE 3—10528

Trial Base 14' - Full uplift due to tailwater assumed

Case I



B26

WAR DEPARTMENT

U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

ject Chicopee - Mass. - C-3a
Computation Wall on piles - 11.5' stem (Above base) - Stability
Computed by ______ Checked by FSM Date 9/30/39

Trial Base 14.0' Case I - River at El. 72.6 - Tailwater

Forces Acting		↓	↑	→	←	Arm	Moments About "A" (↻)	Moments About "A" (↺)
C_1	1.5×11.5×150	2590				6.75	17480	
C_2	2.5×14×150	5250				7.00	36750	
W	12×6.5×62.5	4880				10.75	52460	
E_r	1.5×6.5×125	1220				10.75	13120	
E_L	1.5×6.0×125	1130				3.00	3390	
U_1	1.5×16×62.5		1500			13.25		19880
U_2	4×12.5×62.5		3130			6.25		19560
U_3	6×12.5×½×62.5		2340			8.34		19520
P_w	16²×½×62.5				8000	5.33		42640
P_{er}	4²×½×17.5				140	1.33		190
P_{el}	4²×½×80			640		1.33	850	
		15070	6970	640	8140		124050	101790
		ΣV = 8100 #↓		ΣH = 7500 #←			ΣM = 22260 '# ↻	

$$\frac{\Sigma M}{\Sigma V} = 2.82' ; \quad \frac{1}{3}\text{ Base} = 4.67' \quad ; \quad e = 7.00 - 2.82 = 4.18'$$

$$B.P. = \frac{8100}{3 \times 2.82} \begin{cases} 2 = 1920 \ \#/\square', & P.B.L. \\ 0 = 0 \ \#/\square', & P.B.R. \end{cases}$$

WAR DEPARTMENT

U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

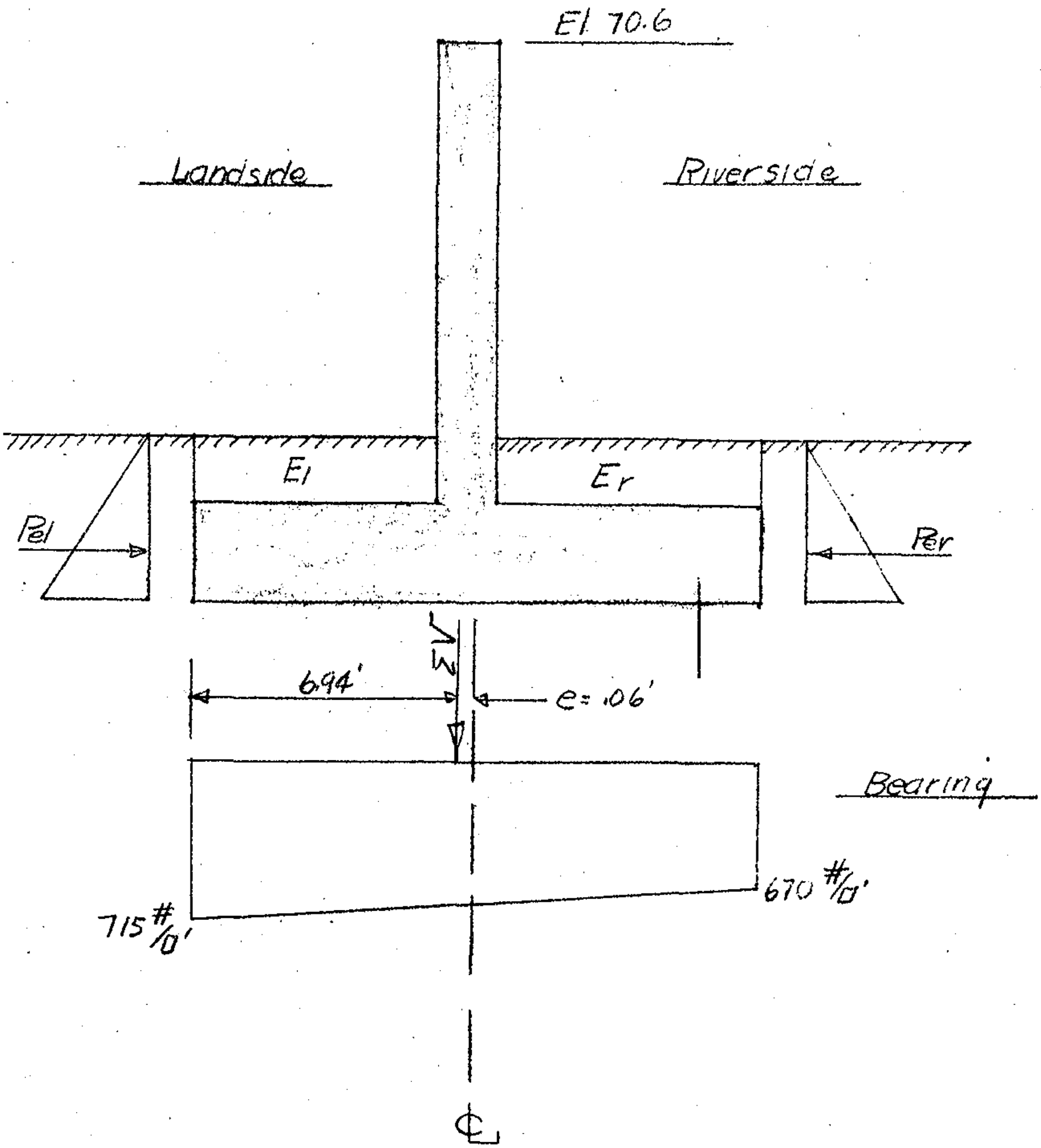
Subject Chicopee; Mass C-3a

Computation Wall on piles - 11.5' stem (Above base) - Stability

Computed by R.H.M. Checked by Date 10/4/39

U. S. GOVERNMENT PRINTING OFFICE 3—10528

Case II - Riverdown - no uplift



WAR DEPARTMENT

U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

Page 4

...t Chicopee - Mass. C-3a

...nputation Wall on piles - 11.5' Stem (Above base) - Stability

...nputed by R.H.M. Checked by .... Date 10/4/39

Case II - Riverdown - no uplift

Forces Acting		↓	↑	→	←	Arm	Moments about "A" ↻	Moments about "A" ↺
C_1		2590				6.75	17480	
C_2		5250				7.00	36750	
E_r	1.5×6.5×100	980				10.75	10540	
E_l	1.5×6.0×100	900				3.00	2700	
P_{er}	$4^2 \times \frac{1}{2} \times 35$				280	1.33		
P_{el}	$4^2 \times \frac{1}{2} \times 35$			280		1.33		
		9720	0	280	280		67470	
		ΣV = 9720 # ↓		ΣH = 0			ΣM = 67470 '# ↻	

$$\frac{\Sigma M}{\Sigma V} = \frac{67470}{9720} = 6.94' \; ; \; \tfrac{1}{3} \text{ Base} = 4.67 \; ; \; \tfrac{2}{3} \text{ Base} = 9.34'$$

$$e = 7 - 6.94 = .06'$$

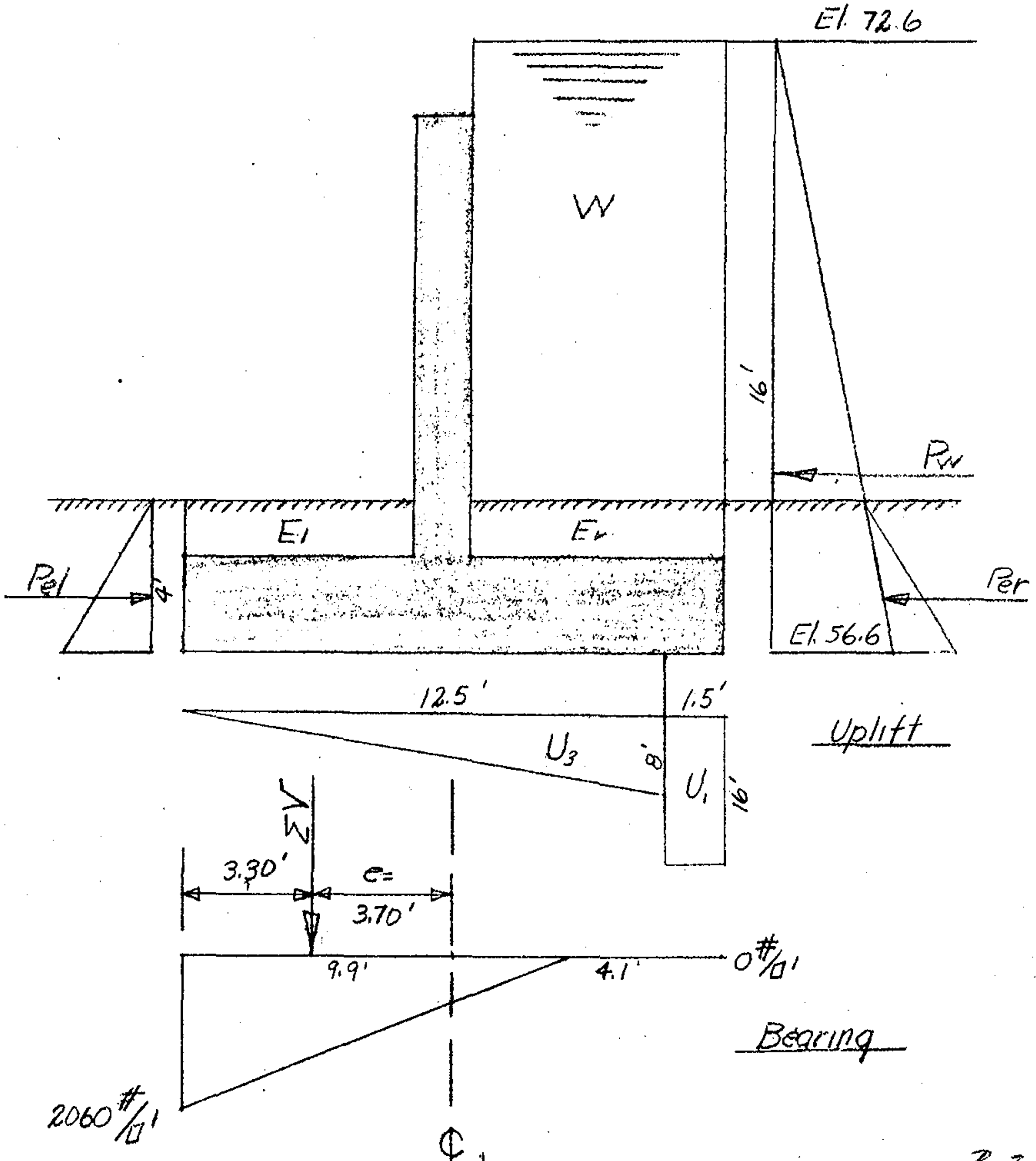
$$B.P. = \frac{9720}{14}\left(1 \pm \frac{6 \times .06}{14}\right) = 694 \begin{cases} 1.03 = 715 \ \#/\square' \ \text{P.B.L} \\ 0.97 = 670 \ \#/\square' \ \text{P.B.R.} \end{cases}$$

I 29

Subject: Chicopee – Mass C-39
Computation: Wall on piles – 11.5' stem (Above base) – Stability
Computed by: R.H.M. Checked by: FSM Date: 10/5/39

U.S. GOVERNMENT PRINTING OFFICE 3—10538

Case III River at El. 72.6 – No tailwater



B 30

WAR DEPARTMENT

U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

ject Chicopee - Mass. - C-39
Computation Wall on piles - 11.5' Stem (Above base) - Stability
Computed by R.H.M. Checked by RSM Date 10/5/39

Case III - River at El. 72.6 - No tailwater

Forces Acting		↓	↑	→	←	Arm	Moments about "A" ↷	Moments about "A" ↶
C_1		2590				6.75	17480	
C_2		5250				7.00	36750	
W		4880				10.75	52460	
E_r		1220				10.75	13120	
E_l	1.5×6.0×100	900				3.00	2700	
U_1			1500			13.25		19880
U_3	8×12.5×½×62.5		3130			8.34		26100
P_w					8000	5.33		42640
P_{er}					140	1.33		190
P_{el}	4²×½×35			280		1.33	370	
		14840	4630	280	8140		122880	88810
		ΣV=10210 #↓		ΣH=7860 #←			ΣM=34070 #↷	

$$\frac{\Sigma M}{\Sigma V} = \frac{34070}{10210} = 3.30' \quad \frac{1}{3}\text{Base} = 4.67 \;;\; e = 7.00 - 3.30 = 3.70'$$

$$B.P. = \frac{10210}{3 \times 3.30}\begin{pmatrix}2\\0\end{pmatrix} = 1030 \begin{cases} 2 = 2060\ \#/\square' & P.B.L \\ 0 = 0\ \#/\square' & P.B.R \end{cases}$$

WAR DEPARTMENT

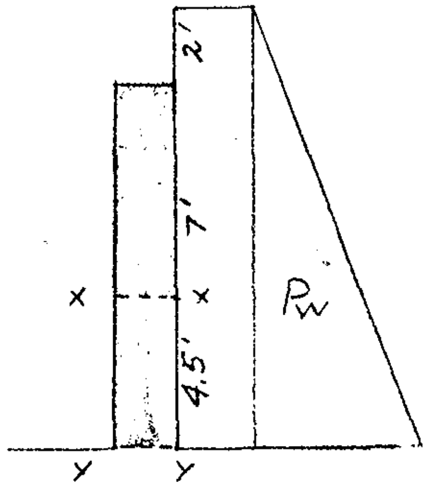
U. S. ENGINEER OFFICE, PROVIDENCE, R. I. Page 7

ct Chicopee - Mass C-3a

mputation Wall on piles 11.5' stem (Above base) Design of Stem Steel

mputed by R.H.M. Checked by RSM Date 10/4/39

U. S. GOVERNMENT PRINTING OFFICE 3—10528



At Sect X---X

$P_W: 9^2 \times \frac{1}{2} \times 62.5 = 2530 \times 3 = 7590$ '#

$A_s = \frac{12 \times 7590}{18000 \times .884 \times 14.5} = \frac{91080}{231000} = .39 \square''$

At sect Y---Y (Neglect Per, Pel)

$P_W: 13.5^2 \times \frac{1}{2} \times 62.5 = 5700 \times 4.5 = 25650$ '#

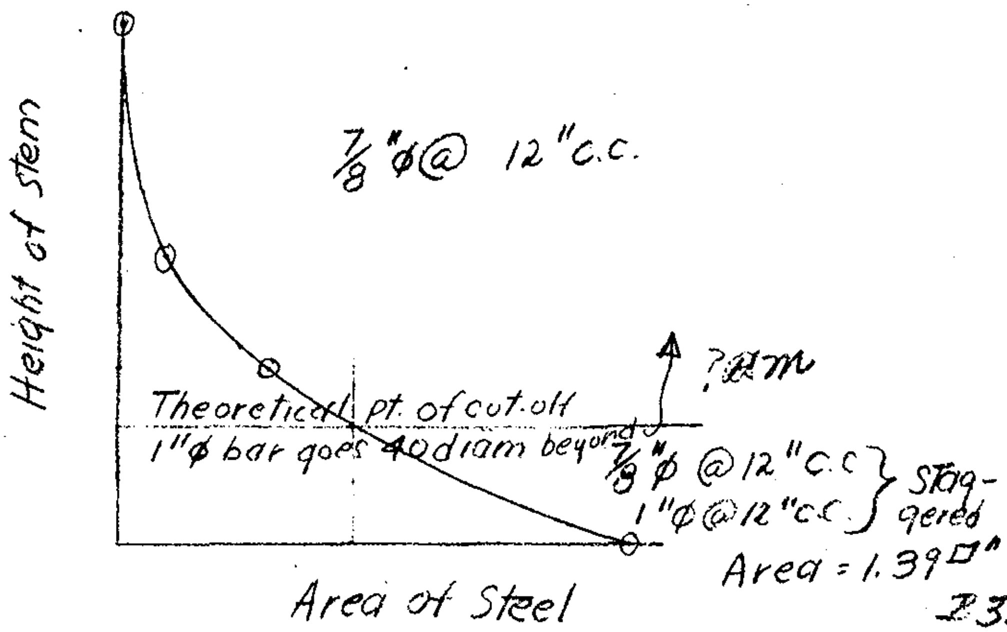
$d = \sqrt{\frac{25650}{123}} = \sqrt{209} = 14.5'$ OK

$A_s = \frac{12 \times 25650}{18000 \times .884 \times 14.5} = \frac{307800}{231000} = 1.33 \square''$

Use 7/8"φ @ 12" c.c & 1"φ @ 12" c.c staggered. $\Sigma_0 = 5.89$

$\text{Shear} = \frac{5700}{12 \times .884 \times 14.5} = \frac{5700}{154} = 37 \#/\square'' < 60 \#/\square''$ OK.

$\text{Bond} = \frac{5700}{5.89 \times .884 \times 14.5} = \frac{5700}{75.5} = 75.5 \#/\square'' < 150 \#/\square''$ OK.



Area = 1.39 □"

1.32

WAR DEPARTMENT

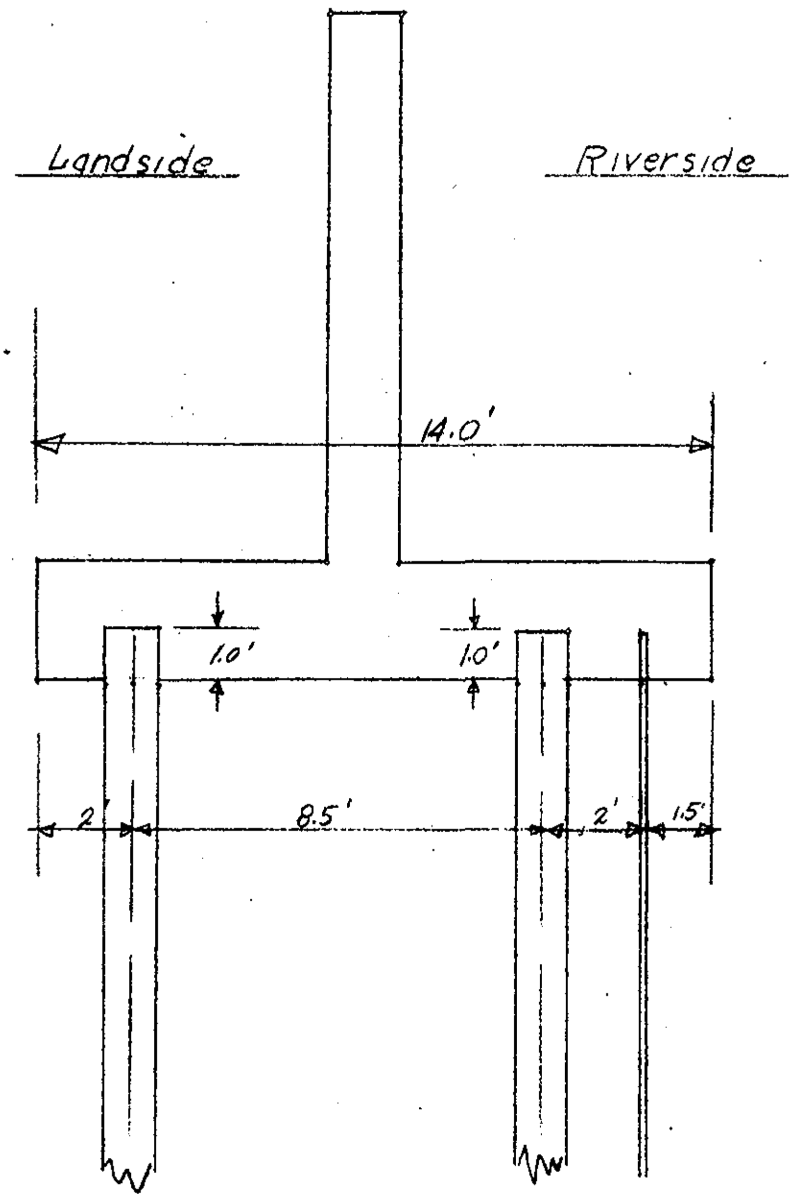
U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

ect Chicopee - Mass - C-30

omputation Wall on piles - 11.5' Stem - Pile spacing

omputed by R.H.M. Checked by RSM Date 10/6/39

U. S. GOVERNMENT PRINTING OFFICE 3—10528



Pile spacing (landside row of piles) (Case III governs)

$$S = \frac{60000 \times 8.5}{7.2 \times 10210} = \frac{510000}{73510} = 6.91'$$

Pile spacing (Riverside row of piles) (Case II governs)

$$S = \frac{60000 \times 8.5}{4.94 \times 9720} = \frac{510000}{48020} = 10.62'$$

Use 6'-0" spacing (same arrangement as for 10' wall)
(see page No. 9 of 10' wall computations)

Unbalanced horizontal force - to be balanced (in part) by piles in shear

Horizontal force for 6' of wall = 7860 × 6 = 47160#

Maximum shearing resistance of piles = 2×14×14×60 = 23520#

Max. available passive resistance: (1' of sheet pile included)

$$P.R. = \frac{wh^2}{2}\left(\tan^2\left[45° + \frac{\phi}{2}\right]\right) = \frac{70 \times 5^2}{2} \times 3.49 = 3060\ \#/ft$$

or 6 × 3060 = 18360#

Horizontal force still unbalanced = 47160 − 23520 − 18360
= 5280# or 880 #/ft

To overcome this we must add a key to increase P.R.

Try 1' key

$$P.R. = \frac{70 \times 36}{2} \times 3.49 = 4400\ \#/ft$$

4400 − 3060 = 1340 > 880 O.K.

WAR DEPARTMENT

U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

...ct Chicopee - Mass - C-39

...nputation Wall on piles 11.5' Stem - Design of landside base steel

...nputed by R.H.M. Checked by RM Date 10/9/39

U. S. GOVERNMENT PRINTING OFFICE 3—10538

Note: The base steel will be designed to take care of two kinds of beam action:
a) Cantilever action out from stem
b) Continuous beam action longitudinally between piles.

a) Loads down

E_1: 6 x 1.5 x 6 x 100 = 5400 x 3 = 16200

C_2: 6 x 2.5 x 6 x 150 = 13500 x 3 = 40500

18900 #↓ 56700 #

Loads up

U_3: 6 x 3.84 x ½ 6 x 62.5 = 4320 x 1.33 = 5750

Pile load: 60000 x 4 = 240,000

64320 #↑ 245,750 #

ΣV = 45420 # for 6' of wall or 7570 #/ft of wall. ↑

ΣM = 189100 # for 6' of wall or 31520 #/ft of wall '#

$$d = \sqrt{\frac{31520}{123}} = \sqrt{256} = 16'' \quad \text{OK. Use } 17''$$

$$A_s = \frac{12 \times 31520}{18000 \times .884 \times 17} = \frac{378240}{270300} = 1.40 \square''$$

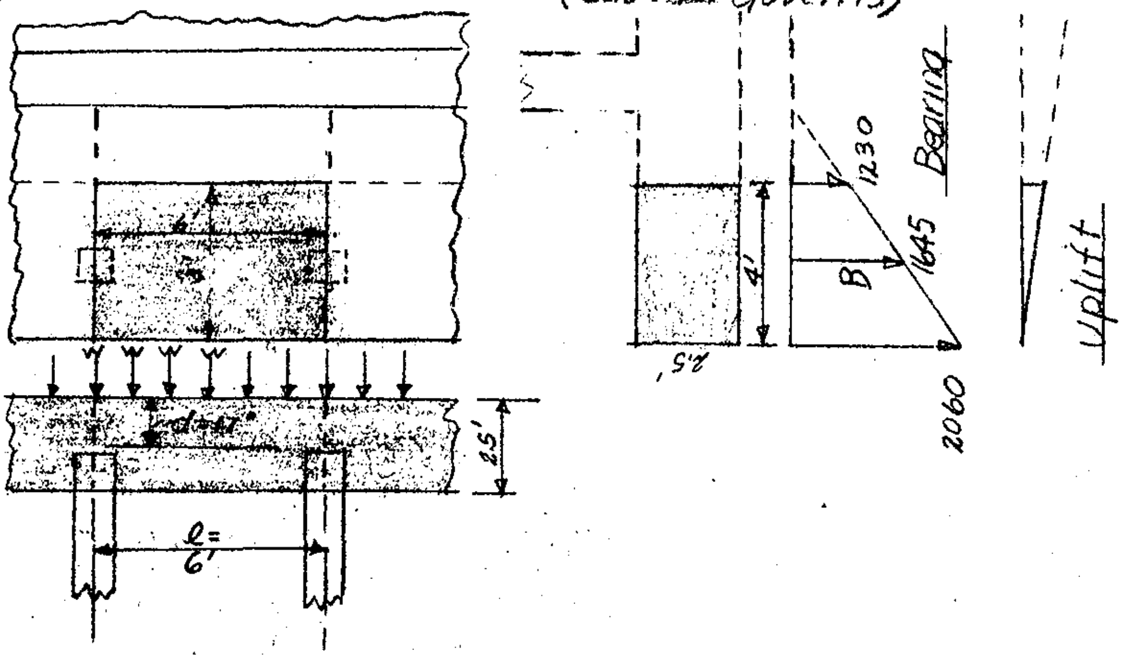
Use: (staggered)
7/8"φ @ 12" c.c. A_s =
1"φ @ 12" c.c. 1.39
Σ_o = 5.89

$$\text{Shear} = \frac{7570}{12 \times .884 \times 17} = \frac{7570}{180.2} = 42 \ \#/\square'' \ \text{ok.}$$

$$\text{Bond} = \frac{7570}{5.89 \times .884 \times 17} = \frac{7570}{88.51} = 85.5 \ \#/\square'' \ \text{ok.}$$

Project: Chicopee - Mass - C-3a
Computation: Wall on piles - 11.5' stem - Design of landside base steel
Computed by: R.H.M. Checked by: RSM Date: 10/9/39

b) Continuous beam action (Case III governs)



Load down (Per foot of wall)

B: 4 x 1645 = 6580#

Load up

U_3: Neglect

Max. moment $= \frac{wl^2}{10} = \frac{6580 \times 36}{10} = 23690$ '# (For 4' beam)
or 5920 '# (For beam 1' wide)

$d = \sqrt{\frac{5920}{123}} = \sqrt{48.1} = 7'$ OK Use 17"

$A_s = \frac{5920 \times 12}{18000 \times .884 \times 17} = \frac{71040}{270,300} = .26$ □" Use ¾"φ @ 12" c.c
$A_s = .44$□" $\Sigma_0 = 2.36$"

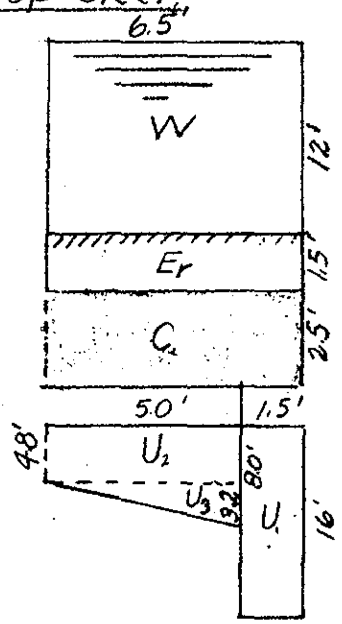
Shear $= \frac{6580 \times 3}{12 \times .884 \times 17 \times 4} = \frac{4940}{180.2} = 27.4$ #/□" OK.

Bond $= \frac{4940}{2.36 \times .884 \times 17} = \frac{4940}{35.47} = 139$ #/□"

...t Chicopee - Mass - C-3a
...putation Wall on piles - 11.5' Stem - Design of Riverside Base Slab
...puted by R.H.M. Checked by RSM Date 10/9/39

U.S. GOVERNMENT PRINTING OFFICE 3—10538

: Riverside base slab designed as cantilever beam (Case III governs for top steel.



Loads down (For one foot of wall)

W: 12x6.5x62.5 = 4880 × 3.25 = 15860
Er: 1.5x6.5x125 = 1220 × 3.25 = 3970
C1: 2.5x6.5x150 = 2440 × 3.25 = 7930
8540 #↓ 27760 #

Loads up

U1: 16x1.5x62.5 = 1500 × 5.75 = 8630
U2: 4.8x5x62.5 = 1500 × 2.50 = 3750
U3: 3.2x5x½x62.5 = 500 × 3.34 = 1670
Pile bearing (It is on safe side to neglect this)
3500 #↑ 14050 #

$\Sigma V = 5040$ #↓ $\Sigma M = 13710$ #

$d = \sqrt{\frac{13710}{123}} = \sqrt{111} = 10.6''$ OK Use 17"

$A_s = \frac{12 \times 13710}{18000 \times .884 \times 17} = \frac{164520}{270,300} = .61 \square''$ Use ⅝"φ @ 6" c.c
$\Sigma_o = 3.92''$

$Shear = \frac{5040}{12 \times .884 \times 17} = \frac{5040}{180.2} = 28 \#/\square''$ OK

$Bond = \frac{5040}{3.92 \times .884 \times 17} = \frac{5040}{58.91} = 85.6 \#/\square''$ OK

Bottom steel Case II governs

Use ⅞"φ @ 12" c.c (See page 10 of computations for wall with 10 foot stem.)

WAR DEPARTMENT

U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

Page 13

Subject Chicopee-Mass- C-39

Computation Wall on piles - 115' Stem - Steel reinforcement.

Computed by R.H.M. Checked by ______ Date 10/9/39

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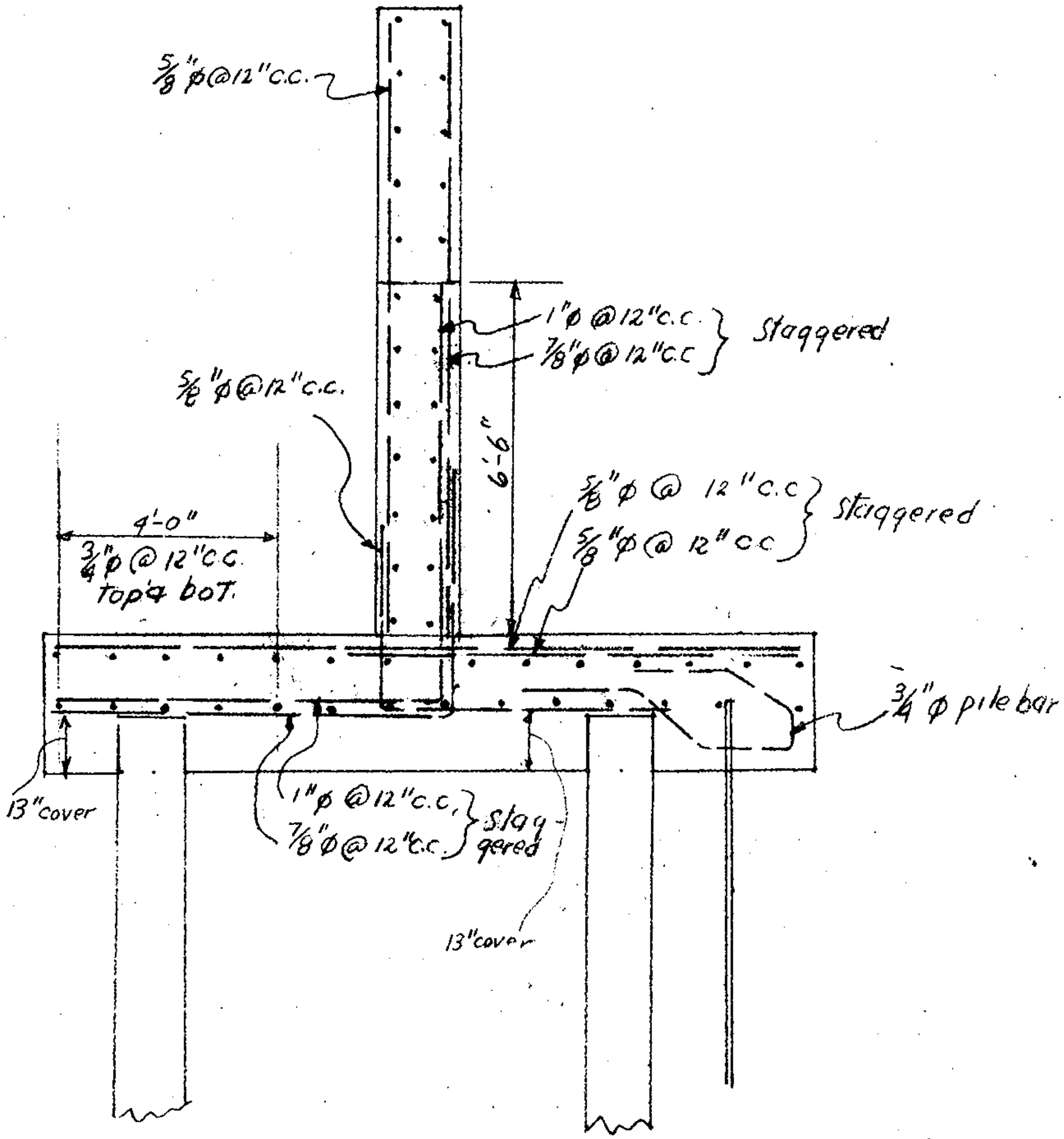
Note: All longitudinal steel, except where specifically indicated is 5/8"φ @ 12" c.c.



B 38

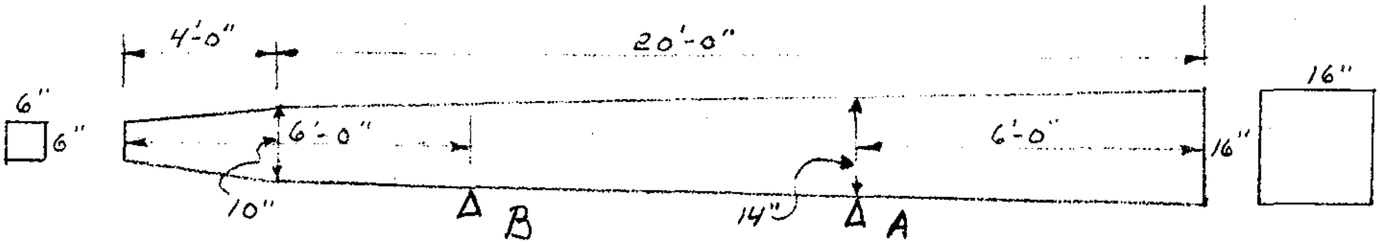
Project: Chicopee, Mass

Computation: Concrete Piles

Computed by: W. S. Jr. Checked by: ________ Date: 10/27/39

U. S. GOVERNMENT PRINTING OFFICE 3—10528

Design of Pile



Assume pile will be supported, at A and B. Maximum moment will be at A.

Mom at A

$$\frac{14}{12} \times 6 \times \frac{14}{12} \times 150 = 1230^{\#} \times 3 = 3700'^{\#}$$

$$2\left(\frac{2}{12} \times \frac{14}{12} \times \frac{1}{2} \times 6 \times 150\right) = 176^{\#} \times 4 = 700'^{\#}$$

$$1406^{\#} = V \qquad 4{,}400'^{\#} = M$$

$$d = \sqrt{\frac{4400}{123}} = 6" \qquad 14 - 2 = 12"d \quad \text{O.K.}$$

$$A_s = \frac{4400 \times 12}{18000 \times .884 \times 12} = \frac{53{,}000}{191{,}000} = .28^{\square"}$$

Shear OK

Assume pile is supported at each end maximum moment will be near the center.

wt of pile

$$20 \times \left(\frac{100 + 256}{2 \times 144}\right) \times 150 = 3720^{\#}$$

$$4 \times \left(\frac{36 + 100}{2 \times 144}\right) \times 150 = 294^{\#}$$

$$4014^{\#} = \text{weight}$$

$$\text{Average wt/ft} = \frac{4014}{24} = 167^{\#}/\text{ft.}$$

$$\text{Mom} = \frac{167 \times 24^2}{8} = 12{,}000'^{\#}$$

$$d = \sqrt{\frac{12{,}000}{123}} = \sqrt{97} = 9.8" \qquad 12.4 - 2 = 10.4"d \quad \text{OK}$$

WAR DEPARTMENT

U. S. ENGINEER OFFICE, PROVIDENCE, R. I.

Page ....

[Proj]ect Chicopee Mass

[C]omputation Concrete Piles

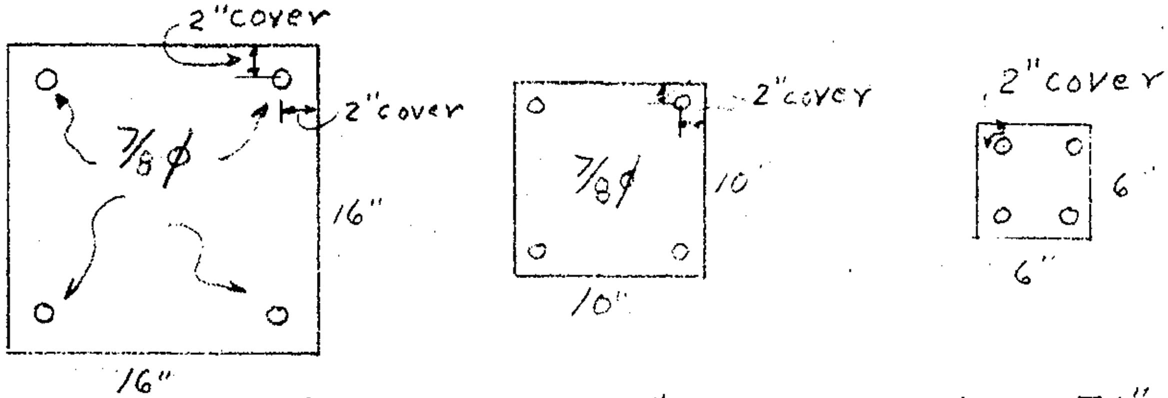
[C]omputed by W. S. J. **Checked by** .... **Date** 10/27/39

U. S. GOVERNMENT PRINTING OFFICE 3—10528

Design of Pile

$$A_s = \frac{12,000 \times 12}{18000 \times .884 \times 10.4} = \frac{144,000}{162,000} = .89^{\square''}$$

Shear O.K.



All cover is 2", All steel is 7/8" φ

By referring to fig 19 of "Concrete Piles" (Portland Cement Assoc.) this design can be checked. A uniform 16" pile 24'-0" long needs 4-7/8" φ.

C2