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MERRIMACK VALLEY FLOOD CONTROL

ANALYSIS
OF
DESIGN

FLOOD PROTECTION

MERRIMACK RIVER
LOWELL MASS

1941

CORPS OF ENGINEERS, U. S. ARMY

U.S. ENGINEER OFFICE BOSTON, MASS.

ANALYSIS OF DESIGN

FLOOD PROTECTION - LOWELL, MASSACHUSETTS

Table of Contents

Paragraph	Subject	Page
	I. INTRODUCTION	
A.	Authorization	I-1
B.	Description of Project	I-1
C.	Local Cooperation	I-2
	PLATES	
I-1	Project Location and Index	
	II FOUNDATION AND MATERIALS INVESTIGATION	
A.	Field Exploration of Foundations and Existing Dikes	II-1
B.	Exploration for Borrow Materials	II-2
C.	Method and Extent of Laboratory Tests and Investigations	II-3
	TABLES	
II-A	Water Content Determinations	
II-B	Specific Gravity Determinations	
II-C	Natural Density Determinations	
II-D	Permeability Tests on Foundation Materials	
II-E	Permeability Tests on Impervious Materials	
II-F	Permeability Tests on Fill Materials	
II-G	Soil Characteristics	

PLATES

II-1 Plan of Exploration for Borrow Materials

II-2 Shear Tests on Impervious Borrow Material

II-3 Typical Stress-Deformation Curves from Shear Tests

II-4 Shear Tests on Ash Fill

II-5 Results of Compaction Tests on Impervious Material

III. DESIGN OF DIKE EMBANKMENTS AND FOUNDATIONS

Paragraph		Page
A.	Basis of Design	III-1
B.	Geological Features and Characteristics of Foundation Materials	III-2
C.	Availability and Characteristics of Borrow Materials	III-3
D.	Dike Embankment Sections	III-6
E.	Economy of Construction	III-7
F.	Stationl Analysis	III-7
G.	Seepage Analysis	III-9
H.	Drainage Well Analysis	III-10
I.	Filter Analysis	III-10
J.	Miscellaneous Analysis	III-11

PLATES

III-1 Geological Profile Along Center-Line of Dike, Lakeview Section

III-2 Geological Profile Along Center-Line of Dike, Rosemont Section

III-3 Typical Grain Size Curves of Foundation Materials, Lakeview Section

III-4 Typical Grain Size Curves of Foundation Materials, Rosemont Section

PLATES (contd.)

III-5 Range of Grain Size Gradation of Impervious Borrow Material

III-6 Typical Grain Size Curves, Sand and Gravel Pits Nos. 1 & 2

III-7 Results of Stability Analysis

III-8 Details of Stability Analysis

III-9 Flow Net and Seepage Analysis, Sta. 0+00 to 8+83

III-10 Flow Net and Seepage Analysis, Sta. 8+83 to 36+50

III-11 Flow Net and Seepage Analysis, Sta. 9+61 to 15+35

III-12 Flow Net and Seepage Analysis, Sta. 0+20 to 8+49

III-13 Computation for Total Quantity of Seepage

III-14 Computation for Loss of Head In Drainage-Well

III-15 Filter Analysis

IV. DRAINAGE AREAS AND PUMPING CAPACITIES

Paragraph		Page
A.	Drainage Area Characteristics	IV-1
B.	Seepage	IV-2
C.	Pumping Capacities	IV-2

PLATES

IV-1 Drainage Area

V. MECHANICAL EQUIPMENT FOR PUMPING STATIONS

A.	Selection of Power	V-1
B.	West Street Pumping Station	V-1
C.	Beaver Street Pumping Station	V-4

PLATES

V-1 West Street Pumping Station
General Arrangement of Equipment No. 1

V-2 West Street Pumping Station
General Arrangement of Equipment No. 2

V-3 Beaver Street Pumping Station
General Arrangement of Equipment

VI. STRUCTURAL DESIGN

Paragraph		Page
A.	General Standards	VI-1
B.	Unit Weights	VI-1
C.	Load Assumptions	VI-1
D.	Structural Steel	VI-1
E.	Concrete Design Standards	VI-1
F.	Structure Analysis	VI-

TYPICAL COMPUTATIONS

1-5	Sheet Piling Wall - Beaver Street Area
6-7	End Monolith - Basement
8	Storage Shed
9-16	Stop-Log Section

VII. CONSTRUCTION PROCEDURE

A.	Sequence of Operations	VII-1
B.	Field Control for Construction of Dikes	VII-1
C.	Drainage Wells	VII-2
D.	Concrete Construction	VII-2
E.	Structural Steel Construction	VII-3
F.	Equipment Furnished by the Government	VII-3

VIII. SUMMARY OF COST

Paragraph		Page
A.	Estimated Cost	VIII-1

ANALYSIS OF DESIGN

FLOOD PROTECTION - LOWELL, MASS.

I. INTRODUCTION

A. AUTHORIZATION.- The project for flood protection at Lowell, Mass., as described herein, is proposed as an element of the comprehensive plan for flood control reservoirs and related flood control works for the Merrimack River Basin authorized by the Flood Control Acts approved June 22, 1936, (Public No. 738, 74th Congress) and June 28, 1938, (Public No. 761, 75th Congress).

B. DESCRIPTION OF PROJECT.- Flood protection will be provided at the sections indicated on Plate I-1. Briefly, the work will consist of the construction of two (2) composite concrete and steel sheet piling flood walls; improvement to two (2) existing spoil embankments; construction of two (2) pumping stations; and minor appurtenant structures. The flood walls will be designed to act as cantilever diaphragms. Improvement to the dikes will consist chiefly of placing an impervious blanket on the river side of the existing spoil embankments and protecting the blanket with a top cover. The West Street pumping station will consist of a reinforced concrete substructure and a one-story superstructure, of a structural steel and brick, with glass block panels serving as windows. This station will contain three (3) 42-inch vertical propeller-type pumps operating from

a wet sump, the pumps being driven through right-angle gear units by gasoline engines; and a 16-inch electric driven vertical volute-type bottom-suction and horizontal-discharge pump placed in a dry sump with the wet sump serving as a suction chamber. The heating equipment will be located in an intermediate chamber over the wet sump. The intake chamber will contain racks, a gate for keeping the wet well dry when no pumping is required, and a gate for closing off the conduit from backflow during high river stages when pumping is prosecuted. The engine room will contain a 7-1/2-ton crane for handling equipment. The Beaver Street Pumping station will likewise have a reinforced concrete substructure, one-story structural steel and brick superstructure, with glass block panels serving as windows. There will be two (2) 24-inch vertical volute-type bottom-suction and horizontal-discharge pumps set in a dry well and driven through right-angle gear units by gasoline engines on the engine room floor. The intake chamber will serve as a suction chamber for the pumps and will contain trash racks and a backflow sluice gate at the mouth of the conduit. The engine room will contain a 4-ton crane.

C. LOCAL COOPERATION.- The City of Lowell will, without cost to the Government; (1) furnish all lands, easements, and rights-of-way necessary for prosecution of the work, including the impervious borrow area; (2) construct interceptor sewers to the pumping stations; (3) remove all buildings within the work limits; (4) maintain and operate the project without expense to the United States.

II. FOUNDATION AND MATERIALS INVESTIGATION

A. FIELD EXPLORATION OF FOUNDATIONS AND EXISTING DIKE.-

(a) Extent.- Foundation exploration at the dike site included drilling in overburden and rock, and excavation and sampling of test pits. The location and extent of the exploration at the two sections are shown on sheets 3, 4 and 33 of the contract drawings.

(b) Drill Holes.- Holes were drilled in the overburden using a 3-inch casing and a 2-inch sampling tube and in rock using a 1-11/16-inch diamond or borts bit. Soil samples were taken at 5-foot intervals or at changes in material. Each soil sample was obtained by first washing out material to the bottom of the casing, then driving the sampling spoon into undisturbed material below the bottom of the casing. A total of six (6) holes were drilled at the Lakeview Section and four (4) holes at the Rosemont Section, with a total footage of 295 feet in overburden and 10.3 feet in rock.

(c) Test Pits.- Test pits were excavated and sampled at the two sections. Wherever possible, the test pits were extended by augering after they had been excavated to an approximate depth of six (6) feet. Bag and constant volume samples were taken, the former at intervals of two (2) feet or at changes in material. A total of four (4) pits at the Lakeview Section and five (5) pits at the Rosemont Section were excavated and sampled with a total footage of 75.2 feet.

B. EXPLORATION FOR BORROW MATERIALS.- (a) Extent.- Exploration to locate sources of suitable borrow materials for the construction of an impervious blanket and for backfill in drainage trenches included test pit excavation and sampling and auger boring conducted as described in paragraph II-A(c). Prior to the selection of the final borrow areas, all possible sources of materials in the vicinity were explored. The locations of the explorations are shown on Plate II-1.

(b) Impervious Material.- Explorations to locate a suitable source of impervious material consisted of reconnaissance within a 10-mile radius, one (1) auger boring and the excavation and sampling of eleven (11) test pits. The selected impervious borrow area shown on Plate II-1 contains an extensive supply of suitable impervious material.

(c) Bank-Run Sand and Gravel Backfill.- Explorations to determine a suitable source of bank-run sand and gravel for backfill consisted in examining and sampling the faces of two sand and gravel pits near the dike site. The locations of these two pits are shown on Plate II-1. A definite borrow area will not be selected, as any source which yields material falling within the limits stated in the specifications will be suitable.

(d) Screened Gravel Backfill.- Exploration was conducted to locate sources of screened sand and gravel for backfill. A screening plant is located in each of the pits described in paragraph II-B(c) and shown on Plate II-1.

C. METHODS AND EXTENT OF LABORATORY TESTS AND INVESTIGATIONS.-

(a) Classification.- All soil samples from test pits, auger borings and drill holes were classified using the M.I.T. scale of classification and recorded in the report of each hole. The grain size distributions of a sufficient number of samples were determined by sieve and hydrometer methods to serve as a guide in the classification of all samples. Geological descriptions of the overburden at each drill hole were prepared after studying the laboratory logs, field logs, and soil samples. The bedrock at one drill hole, D10, was classified by a geologist after examing the rock cores and field logs.

(b) Water Content.- Water content determinations were made on six (6) samples from the impervious borrow area, the results of which are tabulated in Table II-A.

(c) Specific Gravity.- Four (4) specific gravity determinations were made on samples from test pits, excavated in the dike foundation, impervious borrow area, and test pit T16p, the results of which determinations are tabulated in Table II-B.

(d) Density.- (1) The natural density of material from test pit T20p, excavated into the foundation of the Beaver Street pumping station, Rosemont Section, was determined by testing three (3) constant volume samples. The results of these tests are tabulated in Table II-C.

(2) The density to which the impervious borrow material can be economically compacted in the embankment has not been determined by field tests using full size construction equipment. During

the initial stages of placing the impervious material, its moisture content will be adjusted until the greatest density using the specified equipment is obtained. It is believed that the binder material (material of grain sizes less than 1/4 inch) can be economically compacted to a unit dry weight of 126 pounds per cubic foot. (See paragraph II-C(g).)

(e) Permeability.- (1) Permeability tests were made on twenty-two (22) remolded specimens of foundation materials, the results of which are tabulated in Table II-D. All tests were corrected to a temperature of 10 degrees Centigrade. The tests on all samples were made with the materials compacted to densities assumed equivalent to those for the materials after construction or for materials existing in the undisturbed foundation.

(2) Remolded specimens of impervious material taken from test pits excavated in the borrow area were tested by three different methods: (a) One test was performed in a consolidation device. The material was compacted, in a moist state, to a density of approximately 130 pounds per cubic foot. The specimen was saturated and tested by forcing water from the bottom upward through the specimen under pressure. (b) Thirteen (13) tests were performed on lightly compacted specimens contained in 2-inch diameter plastic tubes. Each specimen was placed either moist or dry, saturated under vacuum, and then tested using a downward-flow, falling-head of de-aired water.

(e) Two (2) tests were performed on relatively dense specimens contained in 2-inch diameter plastic tubes. Each specimen was placed dry, saturated by capillary upward flow of de-aired water, and then tested using a downward-flow, falling-head of de-aired water. The results of these three sets of tests are tabulated in Table II-E. For purposes of design, a range for the value of the coefficient of permeability of 1×10^{-5} to 1×10^{-7} has been selected as being the best possible to obtain based upon a consideration of all influencing factors.

(3) Remolded specimens of seven (7) selected samples from test pits and drill holes in the existing spoil dike, earth and ash fill were tested. Table II-F is a tabulation of these test results.

(f) Shearing Strength.- (1) The shearing strengths of representative samples of impervious material and ash fill material were determined by testing in the triaxial compression device. All tests were performed using the general procedure described in the publication: "Notes on Soil Testing for Engineering Purposes," by A. Casagrande and R. E. Fadum, Graduate School of Engineering, Harvard University, January 1940.

(2) The results of five (5) shearing strength tests performed on a representative sample of impervious material are shown on Plate II-2. A typical stress-deformation curve is plotted on Plate II-3, together with pertinent data for a single test.

(3) The results of three (3) shearing strength tests performed on a representative sample of loose ash fill are shown on

Plate II-4. A typical stress-deformation curve is plotted on Plate II-3, together with pertinent data for a single test.

(g) Laboratory Compaction.- Laboratory compaction tests using three different amounts of compaction were performed on two (2) samples of impervious material from which all stones retained on the 1/4-inch sieve had been removed. The procedure of testing for a specific amount of compaction is the same as that described in the publication: "Notes on Soil Testing for Engineering Purposes," by A. Casagrande and R. M. Fadum, Graduate School of Engineering, Harvard University, January 1940. The three amounts of compaction used in these tests were 10, 25, and 40 standard blows of the tamper. The results of the tests are shown on Plate II-5. Based upon experience with similar materials, it is believed that the maximum dry density that can be obtained using the specified equipment will be 126 pounds per cubic foot. This density will probably be most economically obtained at a water content of about 10 percent dry weight, as obtained from Plate II-5.

(h) Miscellaneous.- Since the embankment and its foundation contain no materials which would tend to consolidate appreciably due to a change in stress or expand or contract upon exposure to varying moisture conditions, no consolidation or expansion tests were performed.

(i) Summary.- Average values, or ranges of values, for the soil characteristics of each different type of soil in the dike embankment and its foundation, including the foundation of the sheet pile sections, are tabulated in Table II-G. The values listed in this

table were obtained from the soil tests described in this section or, where no tests were performed, they have been assumed using, as a guide, the experience and judgment resulting from testing similar materials.

III. DESIGN OF DIKE EMBANKMENTS AND FOUNDATIONS

A. BASIS OF DESIGN.- (a) General.- The design of the dike embankment and its foundation for both the Rosemont and Lakeview Sections involved: (1) A geological analysis of foundation conditions; (2) A study of the physical characteristics of foundation materials and available embankment materials; (3) An economic study to determine the most advantageous use of structure excavation and borrow materials; and (4) An analysis of the structural stability of the embankment designed using the preceding components. The methods and extent of investigations to determine the physical characteristics of the foundation and embankment materials have been described in Section II. There follows a description of the results of those investigations which pertain to the design, a discussion of the choice and economy of the embankment sections, and the analysis demonstrating that the section is satisfactory for the following criteria:

(1) The slopes and density of the embankment and its foundation must be such that no structural failure, either by flow or shear slide, can occur in the embankment or its foundation.

(2) Seepage must be so controlled that no detrimental uplift pressures or transportation of material can occur.

(3) The design freeboard must not be jeopardized by appreciable embankment settlement.

B. GEOLOGICAL FEATURES AND CHARACTERISTICS OF FOUNDATION MATERIALS.-

(a) Description of Site. - The dike site is located in the City of Lowell, partly upon the north bank of the Merrimack River and partly upon the north bank of Beaver Brook, extending from Bridge Street to the end of Beshman Street, as shown on Plate I-1. The dike is divided into two sections, designated as Rosemont and Lakeview. Separating these two sections is an extensive deposit of ash and earth fill, at an average elevation of 80 feet above mean sea level.

(b) Bedrock.- Bedrock was encountered in only one drill hole, D10, and no rock outcrops were discovered adjacent to the dike site. The bedrock as evidenced by the single drill hole core is quartzite and schist intruded by diorite-gabbro. Bedrock in general is believed to be at depths less than 50 feet, elevation 20, over the entire site. Plates III-1 and III-2 are geological profiles of the two sections, Lakeview and Rosemont, showing the actual and assumed bedrock profile.

(c) Overburden.- (1) There are two predominate types of overburden at the site: (a) Earth and ash fill; and (b) Natural soil. PlatesIII-1 and III-2 are geological profiles showing the boundary between these two types together with subdivisions of each.

(a). The earth and ash fill may be divided into two components: (1) The dumped fill, consisting principally of ashes; and (2) the spoil dike (constructed during channel improvement operations in 1938), consisting principally of earth and rock.

(b). The natural soil is divided geologically into three types: (1) A deposit of glacial till overlying bedrock which was encountered at the western ends of both the Rosemont and Lakeview sections; (2) A deposit of clayey silt, probably a lake deposit, encountered at the eastern ends of both the Rosemont and Lakeview sections; and (3) Deposits of find sands and sand and gravel, probably river or outwash deposits, encountered over the remaining areas and overlying the above two deposits. The classification of the overburden encountered in each drill hole and test pit in the foundation is shown on sheets 3, 4 and 33 of the contract drawings. Plates III-3 and III-4 show typical grain size gradations of the natural soils encountered.

(2) From a study of the results of the tests described in Section II, the geology of the deposits, and all records of exploration, it is concluded that the values of the physical characteristics for the several materials, as tabulated in Table II-G, are the best representative averages or ranges for design purposes. No shear tests were made on the natural foundation deposits. The assumed shearing strengths of materials in these deposits are based upon experience with similar materials.

C. AVAILABILITY AND CHARACTERISTICS OF BORROW MATERIALS.-

(a) Random Fill.- (1) Material for the random fill portions of the embankment will be obtained principally from, (a) excavation from the face of the existing spoil dike for the compacted impervious blanket section, and from (b) the operation of leveling to grade the crest of

the existing spoil dike. Should the excavated material prove insufficient, additional material for random fill will be obtained from areas to be selected by the contracting officer. Ash fill or mixed ash and earth fill are considered unsuitable for random fill to be placed on slopes. ~~Excess excavation of this latter material will be wasted.~~

(2) The physical characteristics of the random fill material will depend principally upon the various types of materials which will be utilized in that portion of the embankment, and to a lesser degree upon the density which will result from the specified compaction. Depending then upon these two factors, the physical characteristics of random fill may vary within wide limits. The limits tabulated in Table II-C are considered to be the best that it is possible to obtain at the present time based upon experience.

(b) Impervious Fill.- (1) The material for the compacted impervious fill section of the dike embankment will be obtained from the location shown on Plate II-1. The material is a well-graded silty sand with varying small amounts of gravel. The range of the grain size gradation for the materials encountered in exploration is shown on Plate III-5.

(2) The borrow area deposit is in general of considerable size and probably extends to bedrock at unknown depths. It is believed that the deposit contains only a minor quantity of stones and boulders greater than 3 inches in size. A definite area for borrow will be laid out and surveyed prior to commencement of work

(3) Based upon the test results described in Section II and upon experience with similar materials, it is concluded that the impervious material will possess the physical characteristics tabulated in Table II-6 when compacted in the manner specified. The optimum moisture content at which maximum density can be obtained using the specified compaction effort is believed to be within the tabulated range. However, during the initial stages of compaction an adjustment of the water content may be made to obtain the maximum density.

(c) Bank-Run Sand and Gravel.- Bank-run sand and gravel for drainage trench backfill may be obtained from any source which yields clean, well-graded material fulfilling the following gradation specifications:

Total Passing	Percent by Weight
2" Sieve	80 to 100
1/4" Sieve	50 to 80
No. 50 Sieve	5 to 20

It is believed that bank-run material from either of the two sand and gravel pits, whose locations are shown on Plate II-1, will satisfy these specifications as evidenced by the grain size gradation curves plotted on Plate III-6.

(d) Screened Gravel.- Screened gravel for drainage trench backfill may be obtained from any source which yields clean, washed material fulfilling the following gradation specifications:

Total Passing	Percent by Weight
2" Sieve	97 to 100
1" Sieve	40 to 70
1/4" Sieve	0 to 10

D. DIKE EMBANKMENT SECTIONS.- (a) Selected Section.- Prior to the submittal of the Definite Project Report dated November 1940, the general location and characteristics of available borrow materials, and the foundation conditions at the site, had been determined by field and laboratory investigations. As the design progressed to its final stage, the sections shown on sheets 3, 4, and 33 of the contract drawings were developed. The chosen sections utilize economical borrow materials and the existing spoil dike constructed in 1938. The essential features of the embankment are described in the following paragraphs.

(b) Impervious Features.- The dike embankment will contain an impervious riverside blanket 3 feet in thickness extending from the crest to the depth shown on sheets 3, 4 and 33 of the contract drawings. Along most of the dike embankment in the Rosemont Section and along the entire dike embankment in the Lakeview Section this impervious blanket penetrates into natural foundation deposits. Due to the great depth of the artificial fill at the eastern end of the Rosemont Section it is impractical to extend the impervious blanket to the depth of natural deposits.

(c) Drainage Features.- At the landside toe toe of the dike embankment, at the locations shown on sheets 3, 4 and 33 of the contract drawings, there will be constructed a drainage trench containing a tile drain pipe and back filled with screened gravel and/or bank-run sand

and gravel. Two-inch drainage wells on fifteen foot centers extending to a depth of five (5) feet into the most pervious foundation strata encountered in exploration empty into the drainage trench along certain sections of the embankment as shown on sheets 4 and 33 of the contract drawings. These wells are provided for the purpose of draining the more pervious foundation strata to prevent the possibility of foundation failure by piping. For purposes of observation of the seepage pressures in the more pervious foundation strata three additional drainage wells in each of the two sections, Rosemont and Lakeview, will be installed as observation wells.

(d) Slope Protection.- Slope protection will be provided by the compacted random fill (and a six inch layer of seeded topsoil) placed over the compacted impervious blanket.

E. ECONOMY OF CONSTRUCTION.- The dike embankment will utilize to the fullest extent the existing spoil dike and all suitable materials excavated therefrom. The impervious borrow area shown on Plate I-1 contains the nearest suitable deposit of impervious material. Bank-run sand and gravel and screened gravel may be obtained (from various commercial companies) within a 3-1/2 mile radius.

F. STATICAL ANALYSIS.- (a) Method.- (1) Following the precedent of the Swedish Geotechnical Committee's study on slope failures in artificial cuts, the stability ratio against shear failure of the dike embankment and its foundation was determined by investigating the forces tending to cause movement (driving forces) and those producing potential resistance to movement on several circular sliding surfaces of weakness

selected by systematic trial. This method investigates only the possibility of a shear failure. In the analysis, the driving forces include the rotating effect of the weight of the soil mass and water above the surface of failure and also the forces generated by water pressure. The forces producing potential resistance consist of the shearing strength generated along the sliding surface. The ratio of the potential resisting force to the driving force is termed the stability ratio. A sufficient number of potential planes were analysed to determine the position of the plane having the least stability ratio. The stability ratio of the weakest plane is termed the least stability ratio for the section analysed. A least stability ratio of unity therefore, indicated equality of driving and potential resisting forces and implies that the embankment is on the verge of failure, while a least stability ratio greater than unity indicates that the structure possesses reserve strength.

(2) Analysis was made of the maximum section of the dike embankment taken at Station 20/65. In the analysis, the embankment and its foundation were considered to be integral and sliding surfaces were passed through foundation and embankment sections without discrimination. Only the severest conditions tending to produce a failure from a sudden lowering of the water surface on the riverside of the dike embankment were considered in the analysis. The landside slope was not analysed since this slope is far less apt to fail than the riverside slope. The soil characteristics used in the analysis were obtained from the results of tests described in Section II and tabulated in Table II-3.

(b) Results.- The results of the investigations of three (3) potential failure planes through the riverside portion of the dike embankment and its foundation are shown on Plate III-7. The least stability ratio, considering the embankment and its foundation as a unit, and for the condition existing immediately after a sudden drawdown (which, from previous experience, is known to produce the least stability ratio) is 1.52. The method of analysis in detail is shown on Plate III-8.

G. SEEPAGE ANALYSIS.- (a) To study the effect of steady seepage through and beneath the dike embankment and beneath the sheet pile sections and to determine the maximum quantity of seepage, graphical flow nets using selected maximum coefficients of permeability from test data tabulated in Table II-C were prepared. The flow nets, Plates III-9, III-10, III-11 and III-12 were drawn for sections at four different stations for the headwater and tail water conditions shown on the individual plates.

(b) The flow nets indicate that, barring unusual discontinuities in the foundation strata, the seepage will be well controlled by the drainage trench and drainage wells with the possibility of piping reduced to a minimum. Computations for the total seepage through both the Rosemont and Lakeview Sections are shown on Plate III-13. It is believed that the total actual seepage through each section for conditions of steady seepage at maximum head may be less than that computed by a factor of 1/2, considering that the actual coefficients of permeability may be less, by the same factor, than those used for computing the total seepage.

Below is tabulated a range for the predicted maximum seepage, one value being that computed, the other, 50% of the value computed.

Section	Predicted Range of Seepage under Maximum Head in Cubic Feet per Minute
Rosemont	1.3 to 0.7
Lakeview	3.8 to 1.9
	5.1 to 2.6

H. DRAINAGE WELL ANALYSIS.- The drainage wells consist of standard 2-inch diameter well-points installed on 15 foot centers as shown on sheets 3, 4 and 33 of the contract drawings. A rational design of the size and spacing of these drainage wells has not been attempted. However, the computations shown on Plate III-14 indicate that the drainage wells as designed will discharge all seepage passing through the more pervious foundation strata with a loss in head of only 0.05 feet. Referring to the flow net, Plate III-10, this loss is insignificant compared with the loss in head between adjacent potential drops. Hence, it is concluded that these wells will provide the effective drainage of seepage illustrated by the flow net, Plate III-10.

I. FILTER ANALYSIS.- A filter analysis was made to determine whether or not the bank run sand and gravel, used for backfill in the drainage trench against natural foundation sands, will act as a satisfactory filter to permit the rapid discharge of seepage yet preclude the possibility of clogging through transportation of fines. The details of the filter analysis are shown on Plate III-15. The method used is that developed by K. von Terzaghi, described in the paper, "An Experimental

Investigation of Protective Filters" by G. E. Bertram, published in 1940 by the Graduate School of Engineering, Harvard University. The analysis shown on Plate III-15 indicates that the bank run sand and gravel described in Paragraph III-C (e) is entirely satisfactory as a filter.

J. MISCELLANEOUS ANALYSIS.- An analysis of the settlement of the dike embankment is not required since no conditions will exist which will cause an appreciable settlement. An analysis of dike embankment failure by flow slide has not been made. It is believed that the fine sand deposits which occur in certain areas beneath the dike are sufficiently dense as evidenced by the density values tabulated on Table II-C, and the imposed embankment stresses sufficiently small to preclude the possibility of failure by flow slide.

IV. DRAINAGE AREAS AND PUMPING CAPACITIES

A. DRAINAGE AREA CHARACTERISTICS.- (a) Lakeview Section.- The drainage area in this section consists of 860 acres of urban land of varying degrees of development as follows (see Plate IV-1):

Area No.	Area (acres)	Degree of Development
A	250	Undeveloped, grass and brush cover. Drains principally into Area B, partly into C.
B	125	Undeveloped, swampy meadow. Drains into Area D sewer at southwest corner of Area B.
C	115	Limited residential development, sewered.
D	190	Residential development, sewered.
E	40	Undeveloped, no sewers.
F	140	Thikly settled, sewered.
Total	860	About 52% of total area is sewered.

The highest point of the drainage area is Elev. 260 M.S.L. at the northern extremity of Area A. The watershed slopes gently to the Merrimack River and has a long axis about 2 miles in length in a north-south direction. There are 6 combined storm and sanitary sewer outfalls in the Lakeview section, as follows:

Type of Sewer	Location	Computed Capacity Flowing Full (c.f.s.)
60" brick circular	West Street	180
48" brick circular	Lakeview Avenue	90
16" cast iron	Fulton Street	4
39" x 26" brick oval	Broughton Avenue	35
10" cast iron	Front Street	3
18" Akron circle	Front Street	8
		320

Although the existing sewer system may be extended over the area in the future, it is unlikely that the trunk sewer will be increased in size.

(b) Rosemont Section. - The drainage area in this section consists of 70 acres of residential development, all of which is served by combined storm and sanitary sewers (see Area G on Plate IV-1). There is one main trunk sewer on Beaver Street, a brick sewer 37" x 25", and 4 small sewers from 12" to 15" in diameter serving short streets and emptying into Beaver Brook. The computed capacity flowing full is 55 c.f.s.

B. SEEPAGE.- The seepage flow through the dikes and flood walls will be small and, consequently, should not contribute appreciable quantities of flow to the total run-off.

C. PUMPING CAPACITIES.- (a) Lakeview Section.- Three 42-inch pumps were agreed upon after a conference including a manufacturer's agent, the consulting engineer for the pumping stations, and members of the Department, as best suited to handle the requisite capacity of the West Street pumping station. In addition, primarily for pumping during periods of low sewer flow, there will be provided one 16-inch pump. The four pumps will have a total design capacity of 340 c.f.s. at maximum pumping head and about 380 c.f.s for river stage at or below maximum permissible water surface elevation of 54 in the pump sump. This exceeds the existing full sewer capacity and is equivalent to a peak discharge of from .40 to .44 c.f.s. per acre for the entire 860

acres tributary area. Diversion of about 250 acres can be made at the upper end of the area. Should further development of the area overtax the present sewer capacities, or the pumping capacity provided, such diversion or its equivalent could be made. The remaining tributary area of 610 acres could then contribute between .5 and .6 c.f.s. per acre without overtaxing the pumping capacity. Thus, the three 42-inch pumps and the one 16-inch pump are considered as providing adequate pumping capacity for the Lakeview area.

(b) Rosemont Section.- In like manner, the selection of two 24-inch pumps was recommended to provide for the pumping capacity of the Beaver Street pumping station to take care of the Rosemont area. These will provide a total design capacity of from 65 to 80 c.f.s. (depending on the head). This will be equivalent to a run-off of about 1 c.f.s. per acre for the 70 acres of tributary area.

V. MECHANICAL EQUIPMENT FOR PUMPING STATIONS

A. SELECTION OF POWER.- Investigation of available electric power at the sites showed an inadequacy of the requisite facilities. The installation of required new power lines and equipment involved considerable expense which, in view of the infrequent period of operation, was not justifiable. This fact, coupled with the high standby charges, the expense of which the City of Lowell preferred to avoid if possible, indicated the adoption of gasoline-engine drives for the pumps.

B. WEST STREET PUMPING STATION.- (a) Pumps.- As indicated in Paragraph IV-C(a), three 42-inch propeller type pumps were selected. Each of the pumps will deliver not less than 45,000 gallons per minute at a total dynamic head of 24 feet and not less than 52,000 gallons per minute at a total dynamic head of 16 feet. The selected dynamic head of 24 feet represents the maximum static pumping head plus a conservative allowance for all losses in head. In addition to the 42-inch pumps, one 16-inch mixed-flow type of pump was selected having a capacity of 7,000 g.p.m. and 8,000 g.p.m. for the respective heads mentioned above. The 16-inch pump will pump low sewer flows during high river stages and will act as an auxiliary unit during use of the 42-inch pumps if required.

(b) Right-Angle Gear Units.- The gear units will be of the self-contained type designed for transmitting the power from the horizontal engine shaft through spiral bevel gears to the vertical pump shafts. The units will have a cast iron and structural steel

housing and will have a service factor of not less than 1.25 times the power required to drive the pumps under any condition of head from zero to 24 feet.

(c) Gasoline Engines.- The gasoline engines will be of the heavy-duty, 4-cycle, internal combustion, stationary type capable of continuously driving the pumps at their rated speed under any head condition developed. Each engine shall operate at a governed speed not exceeding 1200 r.p.m. when driving the pump at its rated speed. At this speed the rating of the engine shall be such that when driving the gear unit and pump at the 16-foot total head conditions, the horsepower required shall not be more than 80 percent of that which the engine is capable of delivering with auxiliaries attached and with fuel as specified as shown by an actual test curve. Also when driving the gear unit and pump at the 24-foot total head condition, the horsepower shown on the actual test curve shall not be exceeded. The engines will be mounted on concrete bases and directly connected through flexible couplings to the right-angle gear units.

(d) Crane.- A 7-1/2-ton overhead crane will be installed in the engine room as an aid for servicing equipment. The crane will be of standard construction and will be hand-operated by chains from the engine room floor.

(e) Sluice Gates.- A motor-operated seating pressure sluice gate will be located at the entrance to the pump sump. Normally, this gate will be closed, being opened only at period when the pumps

are to be operated. A second motor-operated sluice gate, likewise a seating pressure gate, will be located in the gravity discharge conduit to prevent backflow from high river stages. Normally, this gate will be open to permit the gravity flow of the sewer discharges to the river.

(f) Water System.- The city water supply will be connected to the station to supply cooling water for the gasoline engines and for station service. The sump pump will have a connection so that, in times of emergency, cooling water may be supplied through it.

(g) Gasoline System.- Gasoline will be stored in a 3500-gallon tank buried underground adjacent to the pumping station. Service to each engine will be through individual lines direct to the engines from the tank. Drip pans will be provided on each engine connecting to a common header returning to the tank. Gasoline piping shall be copper pipe outside of the building and copper tubing inside the station, except that where the tubing is embedded it shall be placed in wrought iron sleeves.

(h) Sump Pump.- For purposes of drying the sump after periods of operation, a motor-operated sump pump of 50 g.p.m. will be provided.

(i) Valves.- Flap valves willbe provided at the end of each of the 42-inch discharge lines to prevent backflow. Gate valves will also be inserted in the discharge lines, since the discharge pipe is considerably below the maximum flood stage. The 16-inch pump will be provided with a gate valve on the inlet

line and gate and check valves on the discharge line.

(j) Heating and Ventilating System.- The heating system shall be of the two-pipe gravity type consisting essentially of an oil-burning boiler unit, a 500-gallon fuel oil storage tank, and two steam-supplied unit heaters located in the engine room. The oil burner will be of the rotary type with electric ignition.

The ventilating system shall consist essentially of an electric-driven fan located on the roof for venting the engine room, an air duct leading from the pump sump through the engine room to a stationary ventilator on the roof, and a blower located on the engine room floor with an intake duct connecting with the pump sump and a discharge duct extending through the wall of the station for venting the pump sump.

(k) Switchboard.- The switchboard will be a self-supporting, safety, steel enclosed, dead-front type switchboard with removable cover plates in the front and designed for setting against the wall. The switchboard shall provide electric power control for the entire pumping station.

C. BEAVER STREET PUMPING STATION.- (a) Pumps.- The Beaver Street Pumping Station will have two 24-inch vertical mixed-flow volute pumps, each one delivering not less than 12,500 gallons per minute at a total dynamic pumping head of 24 feet and not less than 16,000 gallons per minute at a total head of 14 feet. The two units in conjunction will match the design capacity of the sewers at the maximum design head of 24 feet. Two units are provided in order to allow flexibility of operation for low flows.

(b) Right-Angle Gear Units.- The gear units will be of the self-contained type designed for transmitting the power from the horizontal engine shaft through spiral bevel gears to the vertical pump shafts. The units will have a cast iron and structural steel housing and will have a service factor of not less than 1.25 times the power required to drive the pumps under any condition of head from zero to 24 feet.

(c) Gasoline Engines.- The gasoline engines will be of the heavy-duty, 4-cycle, internal combustion, stationary type capable of continuously driving the pumps at their rated speed under any head condition developed. For engine rating requirements see Paragraph V-B(c). The engines will be mounted on concrete bases and directly connected through flexible couplings to the right-angle gear units.

(d) Cranes.- The cranes for the Beaver Street Pumping Station will have a capacity of not less than 4 tons.

(e) Sluice Gate.- The Beaver Street station will be equipped with only one sluice gate, seating pressure gate at the entrance to the gravity discharge conduit for the purpose of preventing back-flow during high river stages when the pumps are operating. This gate will have a hand-operated hand stand.

(f) Water System.- The city water supply will be connected to the station to supply cooling water for the gasoline engines and for station service. The sump pump will have a connection so as to provide emergency cooling water if necessary. Water will be secured from a connection to the outlet discharge.

(g) Gasoline System.- Gasoline will be stored in a 900-gallon tank buried underground adjacent to the pumping station. Service to each engine will be through individual lines direct to the engines from the tank. Drip pans will be provided on each engine connecting to a common header returning to the tank. Gasoline piping shall be copper pipe outside of the building and copper tubing inside the station, except that where the tubing is embedded it shall be placed in wrought iron sleeves.

(h) Sump Pump.- For purposes of drying the sump after periods of operation, a motor-operated sump pump of 30 g.p.m. will be provided.

(i) Valves.- The two pumps will be provided with gate valves in the inlet lines, closed in normal position to prevent water entering from the inlet chamber; and gate valves and check valves in the discharge line for purposes of preventing backflow and positive shut-off for servicing the pump when the river stage is above elevation 56.0 M.S.L.

(j) Heating and Ventilating Systems.- The heating system will consist essentially of an oil-burning heating furnace of the warm-air type, with built-in electric-driven blower, and a 180-gallon underground fuel oil storage tank.

The ventilating system will consist essentially of two adjustable louvres set in the station wall and a stationary ventilator on the roof of the building.

VI. STRUCTURAL DESIGN

A. GENERAL STANDARDS.- The flood walls, pumping stations and appurtenant structures were designed in accordance with standard engineering practice. Consideration was given to economy in design, simplicity of construction, pleasing appearance and durability.

B. UNIT WEIGHTS.- In the design of various structures the following unit weights were used:

	Lbs. per Cu. Ft.
Concrete	150
Water	62.5
Earth, moist	115

C. LOAD ASSUMPTIONS.- (a) Lateral earth pressures used in the design of walls were determined by the Rankine formula. For saturated soils an equivalent liquid pressure of 81.5 pounds per square feet per foot of depth was assumed. For design of flood walls a net passive pressure of 279 pounds per square foot per foot of depth was assumed.

(b) Uplift was considered as acting over the full base varying from 100% maximum headwater to 100% of the accompanying tailwater.

D. STRUCTURAL STEEL.- All structural steel was designed in accordance with the Standard Specifications of the American Institute for Steel Construction.

E. CONCRETE DESIGN STANDARDS.- (a) Standard Specifications.- The design for reinforced concrete was governed by the "Recommended Practice and Standard Specifications for Concrete and Reinforced Concrete", American Society of Civil Engineers, June, 1940.

(b) Class of Concrete.- All concrete used will be Class A concrete, required to have an average 28-day compressive strength of 3000 lbs. per square inch and a 1-inch maximum size of aggregate.

(c) Working Stresses.- The allowable working stresses for all Government designed structures are based on a concrete having a minimum 28-day compressive stress of 2500 lbs. per square inch and are shown in the following table:

TABLE VI-1 - ALLOWABLE WORKING STRESSES FOR CONCRETE REGION

Type of Stress	Stress (lbs. per square inch)
Flexure:	
Extreme fiber stress in compression	1125
Shear:	
No web reinforcement	50
No web reinforcement and special anchorage	75
Web reinforcement	150
Web reinforcement and special anchorage	100
Bond: Deformed bars only	
No special anchorage	125
With special anchorage	167
Axial Compression:	
In columns with lateral ties	375
Bearing:	
On full area	625
Reinforcing Steel:	
Tension - Main steel	18,000
Tension - Web reinforcement	16,000
Minimum lap	40 diameters

Ratio of modulus of elasticity of concrete to steel - 1/12. The design of the pumping stations was executed by Weston and Sampson, Consulting Engineers.

(d) Anchorage, Spacing and Covering of Reinforcement.- Adequate embedment beyond face of supports, laps of continuous steel and continuity

of reinforcing in rigid frames was provided. The spacing of bars and the protective covering of concrete was maintained equal to or greater than the minimum allowable as specified in Section VIII of the contract specifications.

(e) Temperature Steel.- Expansion and construction joints were located so as to limit pours to such lengths that shrinkage stresses and cracks will be minimized. Longitudinal reinforcement for volume changes due to shrinkage and temperature change is provided to an amount of 0.50 sq. in. area per foot of height.

F. STRUCTURE ANALYSIS.- (a) Flood Walls.- These cantilever walls are composed of steel sheetpiling capped by a reinforced concrete stem enclosing the piles from a point approximately four feet below the ground to their upper ends, forming from there up the complete wall.

The loading condition for the flood walls was assumed to be water to the top of the wall on the river side and at ground surface on the land side.

Depths of penetration and bending moments were determined from the method outlined in the publication, "Steel H-Beams and Steel Sheet Piling used as Cantilevers in Soil", published by the Carnegie-Illinois Steel Corporation, Pittsburgh, Pa. The "Simplified Load" method was used, neglecting wall friction of the soil. The penetration thus determined is approximately 1.5 times the clear height above the ground. At the Lakeview Section, however, the piling was carried approximately 17 feet, in general, below the required penetration depth to penetrate an impervious strata and form a seepage cut-off.

In computing passive pressure for submerged earth, a coefficient

of 1.5 was applied to the theoretical value as test information indicates that additional resistance results due to the undisturbed condition and natural compaction of the soil. The coefficient assumed is not excessive and is compatible with values in general use.

The reinforcement in the wall was proportioned to resist the full moment at a point two feet below the ground with bars being cut off systematically at points where the moment reduction permitted. The reinforcing will be direct-connected to the sheetpiling by "L" bars passing through holes burned in the piling 6 inches above the wall base.

Monoliths 1-5 of the Rosemont flood wall will be reinforced concrete cantilever monoliths without sheetpiling, with the reinforcing designed for the maximum moment occurring.

(h) Pumping Stations.- The pumping stations for this project were designed and prepared by the consulting engineering firm of Weston and Sampson, Boston, Massachusetts. The general design of the stations is adapted from those constructed by the Providence Engineer District.

(1) West Street Pumping Station.- (a) Design.- (1). Superstructure.- Structural steel columns in the walls of the superstructure take up the direct roof loads as well as wind loads on the superstructure. Sidewall columns also carry crane brackets which support the crane runway for a 7-1/2-ton crane. These columns are designed to carry full live and dead load from the roof; dead load, live load, and impact from the crane; bending due to concentrically applied loading, and bending due to wind load on the building. A pin-ended condition was assumed at the base.

Allowable stress in columns are figured from the formula $P/A = \frac{18000}{1 + \frac{l^2}{18000}}$ with a maximum allowable stress of 15,000 pounds per square inch for dead load plus live load and a maximum allowable stress of 20,000 pounds per square inch for combined dead, live, and wind loads; l/r limited not to exceed 120.

The roof slab is of reinforced concrete designed to carry the full dead load (including 40 pounds snow load) plus a live load of 30 pounds per square foot of roof surface. The roof beams are of structural steel encased in concrete fireproofing. Roof loads are carried through these beams to the wall columns. These beams, together with the wall columns, form portal frames which take up wind loads and crane thrusts. The end connections are designed to take up all such horizontal loads.

(2). Substructure.- The substructure is of reinforced concrete consisting of a wet sump 41'-6" x 25'-0", serving the three 42-inch pumps; a dry well compartment housing the 16-inch pump; inlet chamber; discharge chamber; gravity inlet and discharge conduits; and a boiler room located on an intermediate floor between the sump and engine room floors.

The base slab was designed as a simply supported beam using as the loading condition full uplift, resulting from maximum river stage, over the entire base, less the weight of the base slab. The exterior walls of the wet sump were designed as simply supported beams using a system of horizontal walers and cross-beams at Elevation 58.5 in order to decrease the thickness of the walls as well as to distribute any

difference in thrust to the transverse walls. For the walls a loading condition of maximum hydrostatic head combined with saturated earth to Elevation 72.5, maximum river stage, was assumed. All interior walls, beams and slabs were designed as simply supported beams using loading conditions consistent with maximum applied loads.

(b) Architecture.- The pumping station superstructure will be a flat-roofed, brick and glass-block structure built around a structural steel framing 59'-0" x 29'-0" overall. The 12.5-inch thick brick walls, capped with a cast stone coping, extend above the roof slab forming a parapet around the roof. The roof slab will be 6 inches thick, covered with a cinder concrete fill to drain. Exhaust mufflers for the gasoline engine will be placed on the roof. There will be pilasters, both external and internal, serving as fireproof column enclosures, for the structural steel columns. The engine room floor will be an 8-inch reinforced concrete slab. A hand-operated traveling crane of 7-1/2 tons lifting capacity will operate the full length of the building and will be used for installing and moving pumps and machinery. Access for the crane hoist to the pump room will be had through openings, normally covered with removable floor plates, in the operating floor. Heat will be supplied by a steam heating system having an oil-burning unit in the boiler room and two unit heaters in the engine room.

Glass block panels are used as their appearance blends well with the design and because of the amount of light they afford. Adjustable louvres in the walls and a motor-operated exhaust ventilator on the roof supply the ventilation for the operating room.

Entry to the building is supplied by two doors: the main

entrance, 7'-10" x 10'-0", consisting of two leaves of hollow steel construction, leading directly to the operating floor and being sufficiently large to provide clearance for movement of the mechanical equipment; a service entrance at the east end of the building, which is also used for reaching the motor-operated sluice gate hoist stands.

(2) Beaver Street Pumping Station.- (a) Design.- (1). Superstructure.- This station parallels the design of the West Street Pumping Station, making allowances for the fact that it is a smaller station and has a 4-1/2-ton crane.

(2). Substructure.- The substructure of the Beaver Street Pumping Station is of reinforced concrete consisting of a dry sump 24'-0" x 28'-0", housing the two 24-inch volute pumps; inlet chamber; and the gravity discharge conduit.

The base slab of the sump was designed as a two-way slab, using as the loading condition full uplift, resulting from maximum river stage over the entire base less the weight of the slab. The exterior walls were designed as simply supported beams in which 3/4 of the load was distributed to the walls acting as vertical beams and 1/4 the load with the walls acting as horizontal beams. The criterion of loading was assumed as maximum hydrostatic head combined with saturated earth to Elevation 75.0, maximum river stage at this location. The west wall of this station acts as an integral part of the flood wall. All interior walls, beams and slabs were designed as simply supported beams using loading conditions consistent with maximum applied loads. Steady beams will be provided for the pump shafts.

(b) Architecture.- The general architecture of the

Beaver Street Pumping Station will closely parallel that of the West Street Pumping Station. It is, of course, a smaller station having overall dimensions of 26'-0" x 28'-0". There will be no external pilasters, the fireproofing encasements in this instance being on the interior only. The roof treatment will simulate the West Street Pumping Station, excepting that the reinforced slab will be 5 inches. The engine room floor will be an 8-inch reinforced concrete slab. The crane will be of 4-ton capacity. Heat for the building will be supplied by a warm-air type oil-burning furnace located in the operating room.

Glass block panels will likewise be used in the Beaver Street Pumping Station as will adjustable louvres and an exhaust ventilator on the roof.

There will be one door providing entrance. This entrance will be 6'-0" x 9'-0", consisting of two leaves of hollow steel construction leading directly to the operating floor.

(c) Stop-log Bulkhead.- This structure is designed essentially as a cantilever section. Water pressure against the stop-logs is transmitted to the base through two reinforced concrete end piers and two intermediate removable structural steel A-frames. Each steel frame is connected by an anchor and thrust casting to a grillage beam embedded in the concrete base slab.

The base slab is designed as a continuous beam spanning between the grillage beams; the structural members as columns subjected to combined axial loading and bending; piers as cantilever stems; and the stop-logs as simply supported beams.

The loading assumption was water to the top of the structure on the river side and to the top of the slab on the land side.

The sheetpile wall is continued under the bulkhead as a seepage cut-off.

(d) Storage Shed.- The storage shed is a thin-walled structure keyed to the flood wall which forms one of the walls of the building. The roof is designed as a fixed-ended beam with a load assumption of full dead load and 30 pounds live load. The floor slab is a uniformly supported slab resting on the ground. An 5-inch wall is provided to carry the roof load plus any lateral wind load. All reinforcing steel is single-layer steel.

VII. CONSTRUCTION PROCEDURE

A. SEQUENCE OF OPERATIONS.- It is expected that the flood walls, dikes, pumping stations and appurtenant works will be completed in 300 calendar days after the date of receipt by the contractor of notice to proceed with advertisement for bids scheduled for late in June, 1941. Due to the state of national emergency existent and the resultant delay in securing materials due to priorities, a definite program of construction is not proposed. However, it shall be incumbent upon the contractor to complete the shell of the pumping stations at the earliest possible date so as to allow passage of sewage upon completion of the interceptors by local interests.

B. FIELD CONTROL FOR CONSTRUCTION OF DIKES.- (a) Classification of Materials.- The character and classification of materials available and requirements of the material for various sections of embankment have been previously described (Sections II and III). Detailed descriptions of the requirements are given in Sections V and VI of the contract specifications. Sufficient field and laboratory tests have been made of the available materials from excavation and designated borrow areas to establish the satisfactory character and quantity of material for construction. Control of the character of the materials to be placed in the various sections will be secured by field inspection and tests.

(b) Methods of Placing and Rolling.- A detailed description of the placing and rolling of materials is given in Section V of the contract specifications. The thickness before compaction of each layer

of material shall not be more than 10 inches, except where rock is allowed in the random section. The contractor will be required to maintain the proper amount of moisture in the material for optimum compaction. Compaction equipment has been specified, and each layer will be required to be rolled with 4 complete passes of the tractor treads.

(c) Tests for Compaction.- Samples of all embankment materials will be taken before and after placement and compaction. Results of tests on these samples will be used to make corrections and adjustments of methods and materials necessary to secure the desired compaction in the embankment.

C. DRAINAGE WELLS will be installed in the immediate vicinity of the dikes, as shown on Sheets 3, 4 and 33 of the contract drawings, to intercept seepage flowing through the dike. Observation wells will be installed to note the rate of seepage and general location of the same.

D. CONCRETE CONSTRUCTION.- Requirements and instructions for the composition, control and placing of concrete are given in Section VIII of the contract specifications. All concrete will be Class "A" as designated in the specifications and will have an average compressive strength of not less than 3400 pounds per square inch in accordance with a standard 28-day test.

The facilities of the Central Concrete Laboratory, U.S.M.A., West Point, N.Y., will be utilized for laboratory control. In the field, the necessary facilities and inspection forces will be maintained

to inspect concrete operations, make field and acceptance tests, and prepare samples and specimens for shipment to the Central Concrete Laboratory.

E. STRUCTURAL STEEL CONSTRUCTION.- All structural steel construction and placing shall be such as to insure secure anchorage and reliable alignment. Rigid supervision and inspection will be maintained of structural steel construction and placing in the field.

F. EQUIPMENT FURNISHED BY THE GOVERNMENT.- Bids for the furnishing of equipment for the pumping stations, as directed in paragraph 1-13 of the contract specifications, will be opened as follows: (1) West Street Pumping Station - June 12, 1941, with 220 days from date of notice to proceed for completion of the contract; (2) Beaver Street Pumping Station - June 16, 1941, with 180 days from date of notice to proceed for completion of the contract.

VIII. ESTIMATED COST

The cost of floodwalls, dikes, pumping stations (including pumping equipment), and appurtenant structures to be constructed as flood protection at Lowell, Massachusetts, based on preliminary design, was estimated as follows:

Floodwalls and appurtenant structures	$ 95,000
Dikes	95,000
Pumping Stations (including equipment) . . .	235,000
Total (including engineering, contingencies and overhead)	$425,000

TABLE II-A

WATER CONTENT DETERMINATIONS

Impervious Borrow Material from T17p

Sample No.	Sample Depth	Water Content in %
1 (w.c.)	2.5'	12.4
1 (w.c.)	2.5'	11.7
2 (w.c.)	4.0'	14.4
2 (w.c.)	4.0'	14.9
3 (w.c.)	5.5'	14.4
3 (w.c.)	5.5'	13.4

TABLE II-B

SPECIFIC GRAVITY DETERMINATIONS

Material	Hole	Sample	Depth	Specific Gravity
Ash Fill	T13p	1	0-6.0'	2.25
Impervious Borrow	representative mixed sample			2.69 (two tests)
Impervious Borrow	T16p	1	4.0-6.0	2.68

TABLE II-C

NATURAL DENSITY DETERMINATIONS

Material	Hole	Sample	Depth	Void Ratio	Relative Density
Sand Fill	T20p	1	1.6'-2.1'	0.837	29 %
	T20p	2	3.9'-4.4'	0.936	37 %
Fine Sand	T20p	4	6.5'-7.0'	0.726	83 %

TABLE II-D
Permeability Tests on Foundation Materials
(Remolded Samples)

Hole No.	Sample No.	Sample Depth	Void Ratio	Coefficient of Permeability in 10^{-4} cm/sec.
D1	3	9.0'	0.746	12.2
D1	6A	15.3'	0.438	32.6
D1	7A	18.4'	0.413	6.0
D1	9	24.0'	0.628	2.07
D2	5A	13.5'	0.705	180
D2	6	16.5'	0.461	8.5
D2	11	30.0'	0.686	0.34
D3	3	18.3'	0.651	9.49
D3	6	25.6'	0.365	6.5
D4	3	7.4'	0.478	152
D4	5	12.6	0.683	10.3
D5	3	7.5'	0.720	19.3
D5	7	18.0'	0.410	7.65
D5	11	27.5'	0.839	0.21
D6	5A	17.0'	0.596	3.19
D7	6	17.5'	—	14.6
D8	5A	29.5'	—	0.86 m
D8	6	35.0'	—	0.96 m
D8	6	35.0'	—	8.9
D8	9A	49.5'	—	0.30
D8	9	50.0'	—	0.09 m
D9	6	30.0'	—	0.03 m

Note: (1) Unless otherwise noted, all tests were run on materials placed dry and saturated under vacuum.

(2) Notation, m, after a value indicates a material placed moist and saturated under vacuum.

TABLE II-E

PERMEABILITY TESTS ON IMPERVIOUS MATERIALS

(Remolded Samples)

Hole No.	Sample No.	Sample Depth	Void Ratio	Coefficient of Permeability in 10^{-4} cm/sec.
T7p	3	2.0-4.1	0.445	0.91
T7p	3	2.0-4.1	--	0.76 (placed dry, saturated by capillarity)
T7p	4	4.1-6.0	--	0.59
$T8_t$	5	5.2-5.7	0.443	0.61
$T8_t$	5	5.2-5.7	--	0.22 (placed dry, saturated by capillarity)
$T8_t$	6	5.7-7.0	--	0.27
T16p	1	4.0-6.0	--	1.65m
T16p	2	4.0-6.0	--	2.20 m
T17p	2	2.0-3.0	--	2.6 m
T17p	2	2.0-3.0	--	0.06 m (not evacuated)
T17p	2	2.0-3.0	--	5.6
T17p	3	3.0-4.5	0.488	0.82
T17p	4	4.5-6.0	--	0.35
T18p	2	2.0-4.0	--	1.51
T18p	3	4.0-6.0	--	1.36

Mixed material from the following pits:				Coef. of Permeability in cm/sec.
$T7_p$	3	2.0-4.1		8.2×10^{-8} (placed moist and very compact in consolidation device, not evacuated)
		4.1-6.0		
$T8_t$	5	5.2-5.7		
	6	5.7-7.0		10.0×10^{-8} (placed moist and very compact in 2" tube, not evacuated)
$T17_p$	3	2.0-3.0	One Sample	
	4	3.0-4.5		
T18p	2	2.0-4.0		3.0×10^{-6} (placed as above, but evacuated to saturate)
	3	4.0-6.0		

(See note, Table II-D)

TABLE II-F

PERMEABILITY TESTS ON FILL MATERIALS
(Remolded Samples)

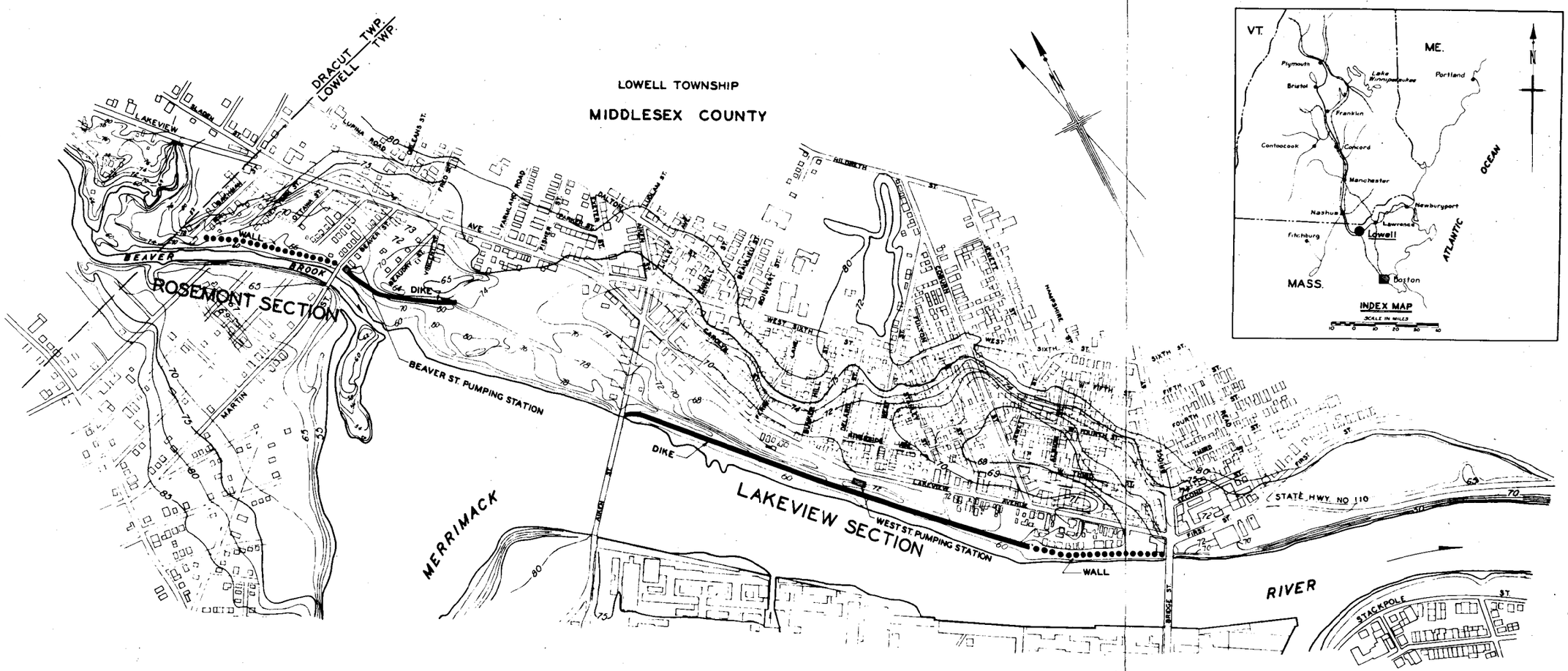
Hole No.	Sample No.	Sample Depth	Void Ratio	Coefficient of Permeability in 10^{-4} cm/sec.
D6	2	5.0'	0.553	43.1
D7	2A	4.6	0.894	25.4
D8	1	5.0'	—	2.7 m
D8	2	10.0'	—	10.1 m
D8	4	25.0'	—	0.17 m
T13p	1	0.0'-6.0'	—	7.5 m
T14p	1	1.0'-2.0'	—	10.4 m

(See note, table II-D)

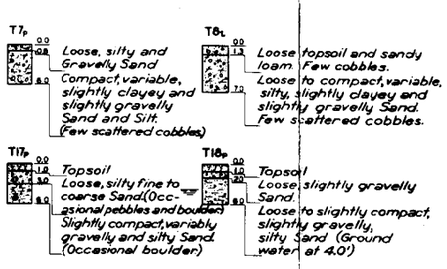
TABLE II-G

SOIL CHARACTERISTICS -- LOWELL DIKE

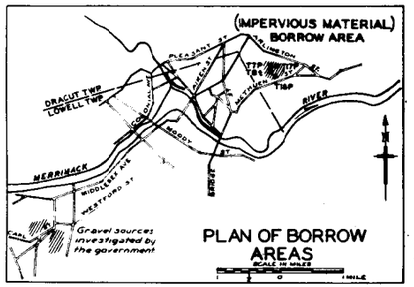
Soil Characteristics	Natural Foundation Materials		Artificial Foundation Materials		Embankment Materials			
	Fine Sand	Silty and Gravelly Sand	Ash Fill	Existing Dike	Compacted Impervious	Random Fill	Gravel Backfill	Sand and Gravel Backfill
Unit Dry Weight of Binder Material, #/CF	98	105	66	99	126	120	102	102
Void Ratio, e, of Binder Material	0.72	0.60	1.40	0.70	0.33	0.40	0.65	0.65
Maximum Void Ratio, $e_{max.}$	1.20	—	—	1.04	—	—	—	—
Minimum Void Ratio, $e_{min.}$	0.60	—	—	0.59	—	—	—	—
Degree of Compaction, %	80	—	30	76	—	—	—	—
Optimum Water Content, % Dry Weight	—	—	—	—	10	8—10	—	—
Coefficient of Permeability k, 10^{-4} cm./sec.	10-30	20-200	4-40	10-100	0.001-0.1	10-100	—	100
Angle of Internal Friction, ϕ, degrees	32	36	39	33	32	32	36	34
Cohesion, c	0	0	0	0	0	0	0	0
Specific Gravity, s	2.70	2.70	2.25	2.70	2.69	2.70	2.70	2.70



DRAWING NO.	TITLE	SHEET NO.	DRAWING NO.	TITLE	SHEET NO.	DRAWING NO.	TITLE	SHEET NO.
	GENERAL			LAKEVIEW SECTION			ROSEMONT SECTION	
50/1	Project Location and Index	1		WEST STREET PUMPING STATION (CONT.)			FLOODWALLS AND DIKES	
50/2	Hydrographs	2	52/13	Steel Reinforcement No. 8	18	53/1	Plan, Profile and Sections	33
			52/14	Steel Reinforcement No. 9	19	53/2	Floodwall - Storage Shed	34
			52/15	Steel Reinforcement No. 10	20	53/3	Beaver St. Bulkhead - Masonry & Reinforcing	35
	LAKEVIEW SECTION		52/16	Structural Steel Framing	21	53/4	Beaver St. Bulkhead - Metal Details	36
			52/17	Miscellaneous Steel No. 1	22			
	FLOODWALLS AND DIKES		52/18	Miscellaneous Steel No. 2	23			
51/1	Plan, Profile and Sections No. 1	3	52/19	Miscellaneous Steel No. 3	24		BEAVER STREET PUMPING STATION	
51/2	Plan, Profile and Sections No. 2	4	52/20	Plumbing and Heating	25	54/1	General Plan and Outlet Details	37
51/3	Detail of Floodwall	5	52/21	Gasoline and Fuel Oil Piping	26	54/2	Architectural - Plans and Elevations	38
			52/22	Electric Power and Lighting No. 1	27	54/3	Architectural - Sections and Details	39
			52/23	Electric Power and Lighting No. 2	28	54/4	Architectural - Miscellaneous Details	40
	WEST STREET PUMPING STATION		52/24	Reinforcing Schedule No. 1	29	54/5	Steel Reinforcement No. 1	41
52/1	General Plan and Diversion Conduit	6	52/25	Reinforcing Schedule No. 2	30	54/6	Steel Reinforcement No. 2	42
52/2	Layout Plan and Outlet Details	7	52/26	General Arrangement of Equipment No. 1	31	54/7	Steel Reinforcement No. 3	43
52/3	Architectural - Plans and Elevations	8	52/27	General Arrangement of Equipment No. 2	32	54/8	Steel Reinforcement No. 4	44
52/4	Architectural - Elevations and Details	9				54/9	Steel Reinforcement No. 5	45
52/5	Architectural - Sections and Details	10				54/10	Steel Reinforcement No. 6	46
52/6	Steel Reinforcement No. 1	11				54/11	Miscellaneous Steel No. 1	47
52/7	Steel Reinforcement No. 2	12				54/12	Miscellaneous Steel No. 2	48
52/8	Steel Reinforcement No. 3	13				54/13	Plumbing and Gasoline Piping	49
52/9	Steel Reinforcement No. 4	14				54/14	Electric Power and Lighting	50
52/10	Steel Reinforcement No. 5	15				54/15	Reinforcing Schedule	51
52/11	Steel Reinforcement No. 6	16				54/16	General Arrangement of Equipment	52
52/12	Steel Reinforcement No. 7	17						



LOG OF TEST PITS
IMPERVIOUS MATERIAL
BORROW AREA



Note:
Elevations on all drawings refer to U.S.G.S. datum

MERRIMACK VALLEY FLOOD CONTROL
FLOOD PROTECTION - LOWELL, MASS.
PROJECT LOCATION AND INDEX

IN 52 SHEETS — SHEET NO. 1 — SCALE: 1" = 300'

U.S. ENGINEER OFFICE, BOSTON, MASS.

APPROVAL RECOMMENDED: CHIEF, DEFENSE AND FLOOD CONTROL SECTION

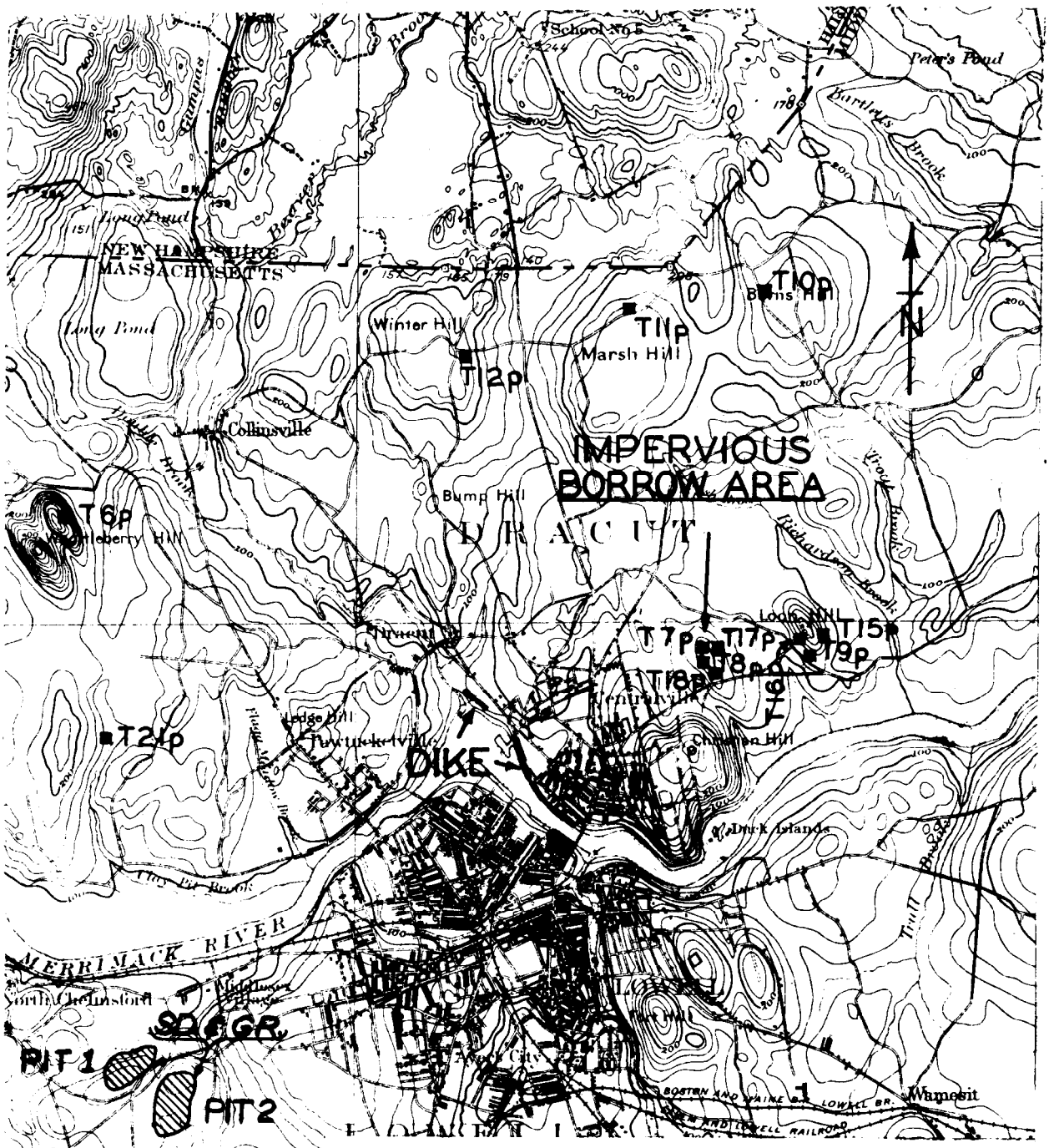
APPROVED: LT. COL. CORPS OF ENGINEERS, DISTRICT ENGINEER

SUBMITTED: CHIEF, DESIGN SECTION

ASS'T ENGINEER

FILE NO. M 54-50/1

PLATE I-1



LOCAL PROTECTION- LOWELL, MASS.

PLAN OF EXPLORATION FOR BORROW MATERIALS

1 ½ 0 1 MILES

SCALE

MARCH 1941

PLATE II-1

No 339 10 10X10 PER INCH THE FREDERICK POST CO. CHICAGO, ILL.

LOWELL DIKE
SHEAR TESTS ON IMPERVIOUS BORROW MATERIAL

Method of Testing:

Apparatus: Triaxial Compression
Sample Diameter: 3.5 cm. } Approximately
" Height: 10.0 cm. }
Tests Run:
Quickly
Fully Saturated
Free Drainage
At Constant Lateral Pressure

Material:

Tests 1, 2, 3, & 5 were performed on the composite sample from Test Holes T7p, T8c, T12p, T18p, which was previously subjected to a 10-blow compaction test. Test 4 was on a sample from T8c, and is not truly representative.

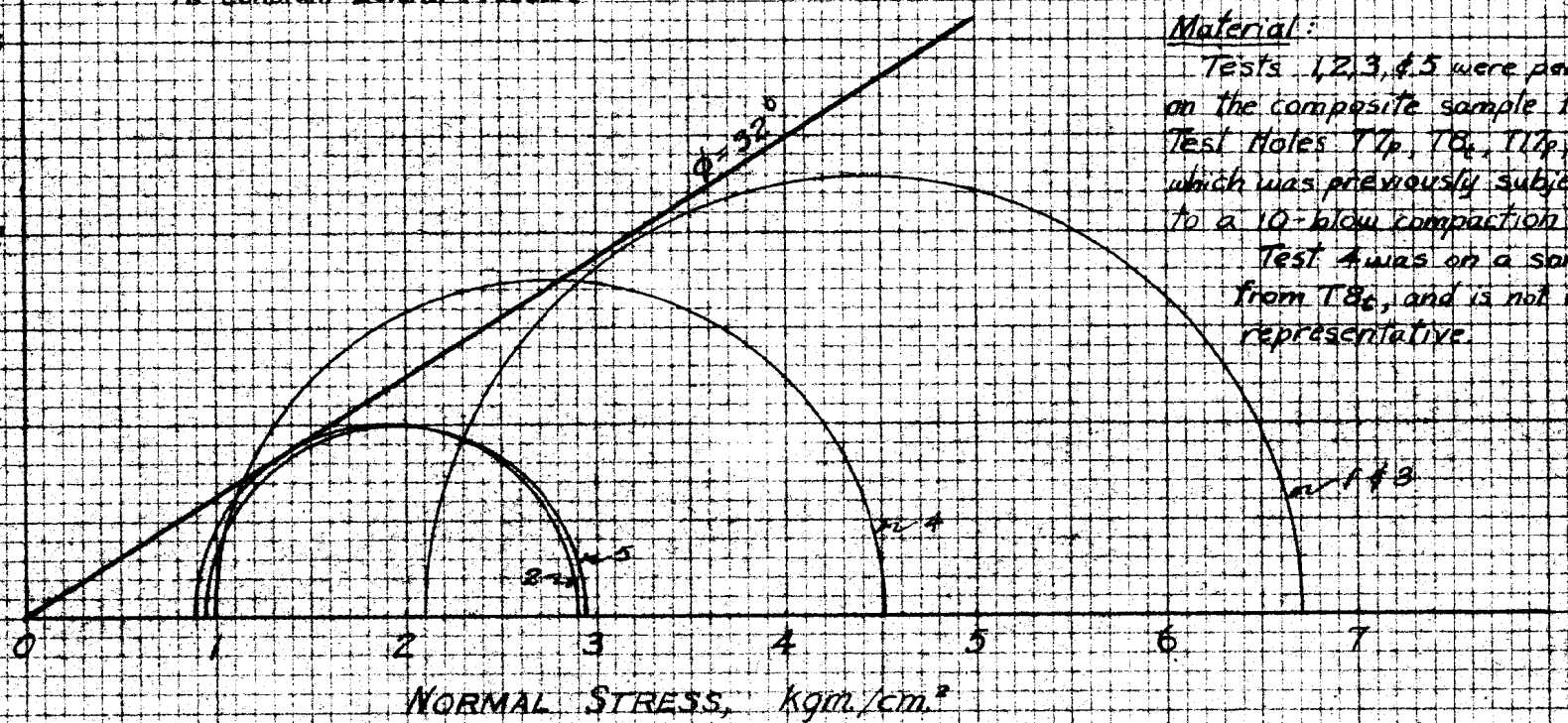


PLATE II-2

No. 330-10 10X10 PER INCH THE FREDERICK POST CO., CHICAGO, ILL.

LOWELL DIKE
SHEAR TESTS
Typical Stress - Deformation Curves

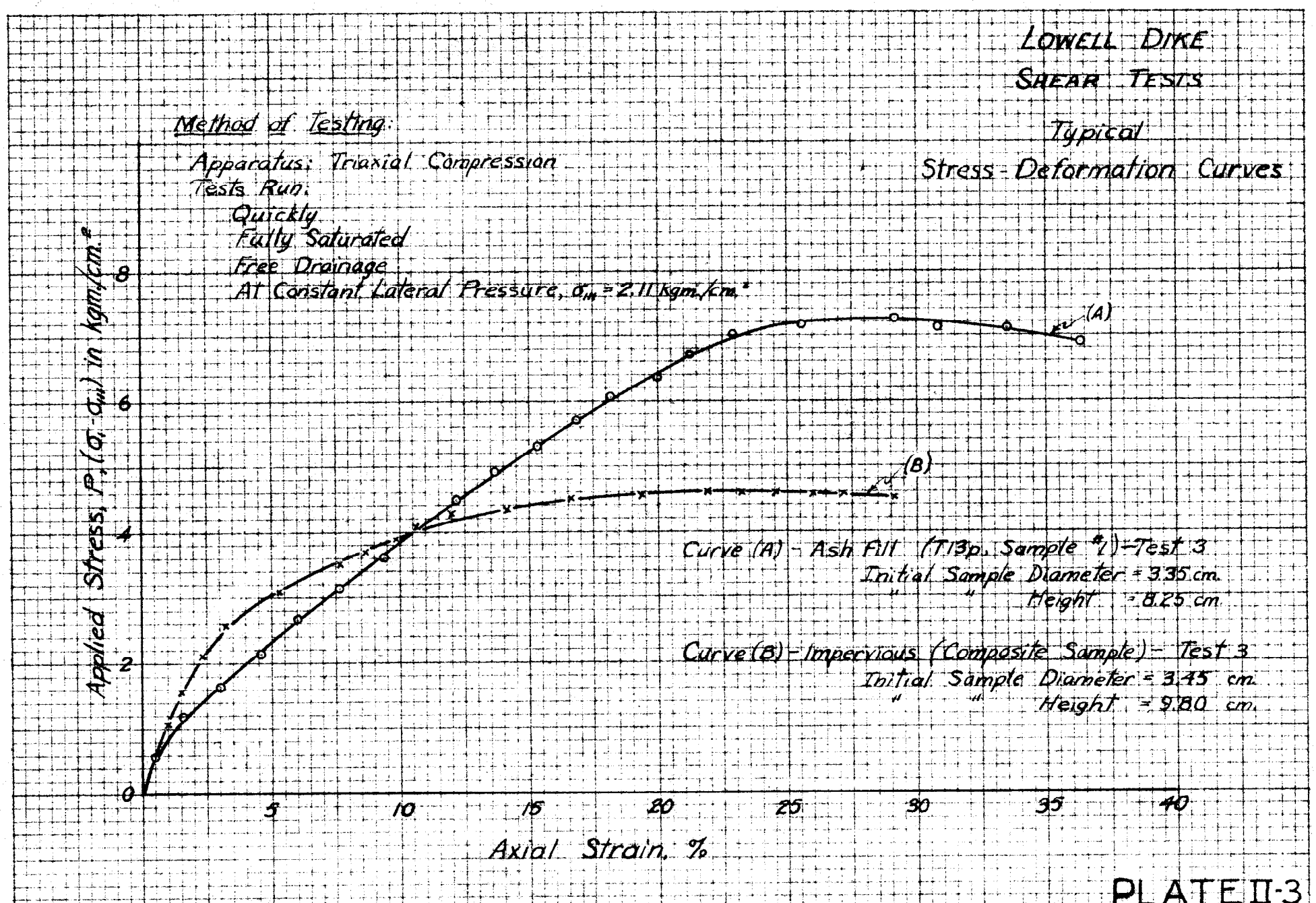


PLATE II-3

NO. 330 10 10X10 PER INCH THE FREDERICK POST CO. CHICAGO, ILL.

LOWELL DIKE
SHEAR TESTS ON ASH FILL

Material: Sample #1
Test Pit T13p

Material was tested as follows:

Apparatus: Triaxial Compression
Sample Diameter: 3.5 cm. } Approximately
" Height: 10.0 cm. }
Tests Run:
Quickly
Fully Saturated
Free Drainage
At Constant Lateral Pressure

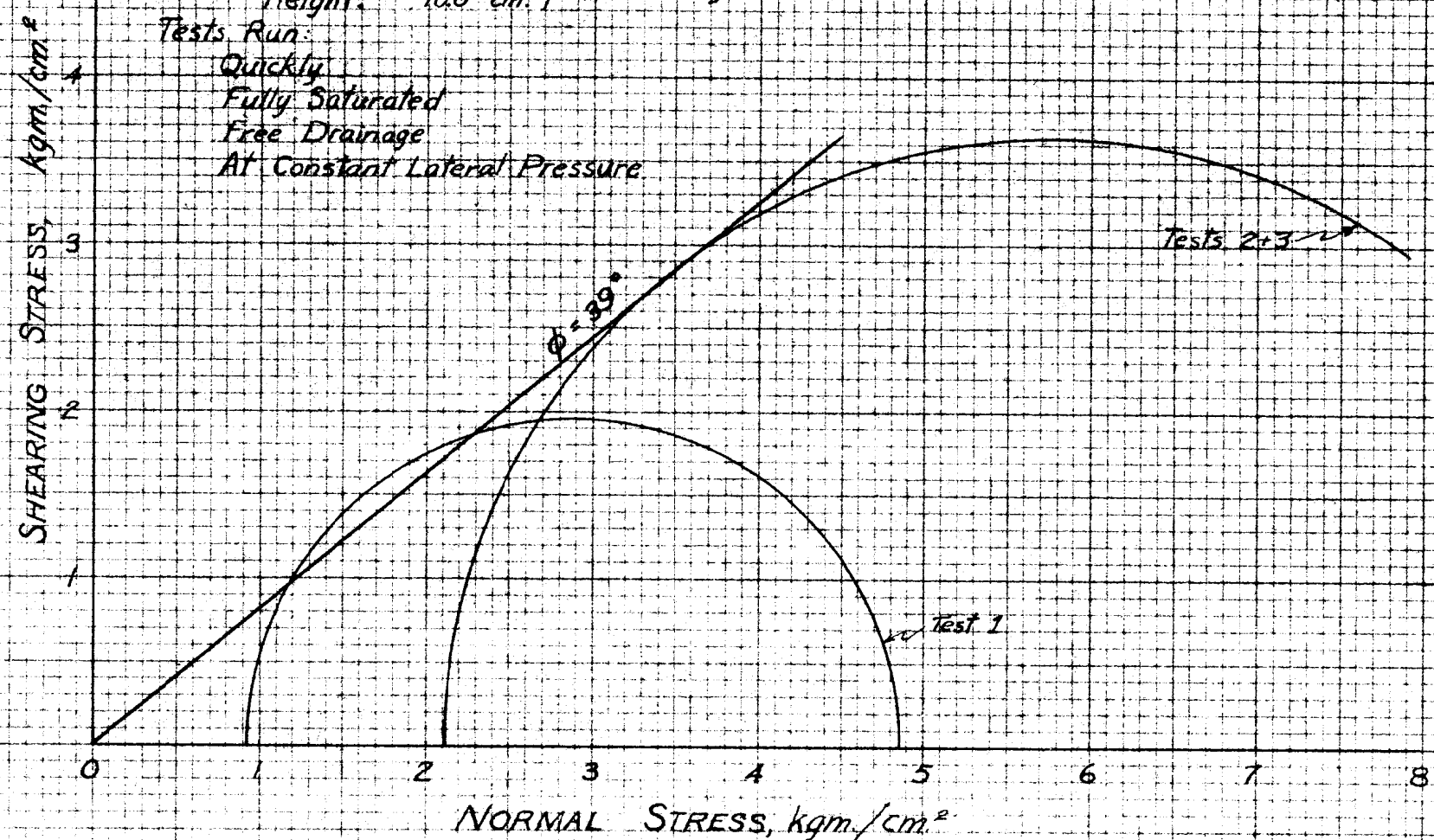
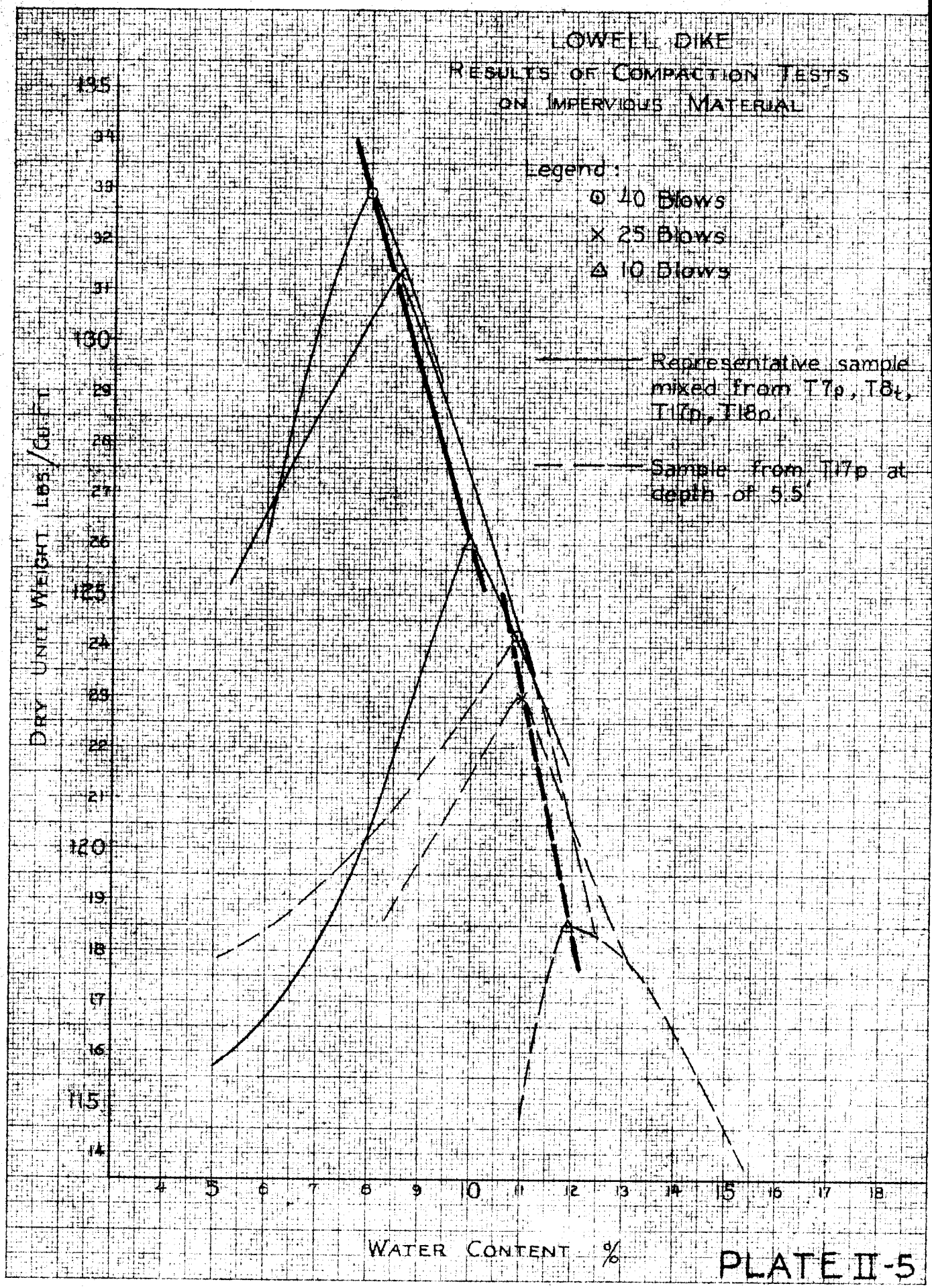
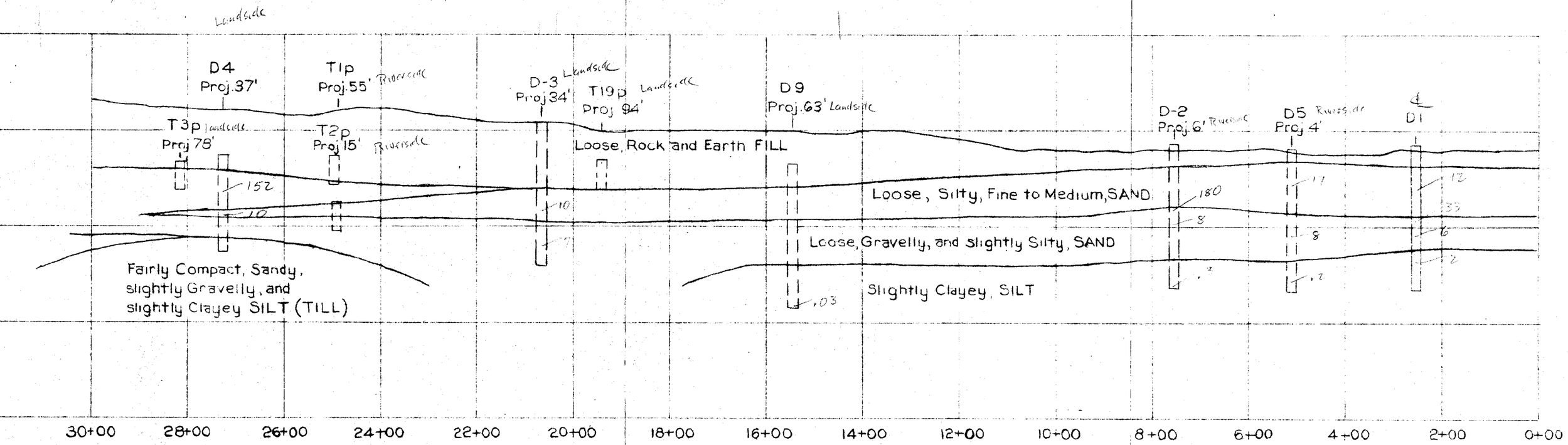


PLATE II-4


LOWELL DIKE
RESULTS OF COMPACTION TESTS
ON IMPERVIOUS MATERIAL
Legend:
⊙ 40 Blows
× 25 Blows
△ 10 Blows
Representative sample mixed from T7p, T8t, T17p, T18p.
Sample from T17p at depth of 5.5'
DRY UNIT WEIGHT LBS./CU.FT.
WATER CONTENT %
PLATE II-5



LOCAL PROTECTION - LOWELL, MASS.

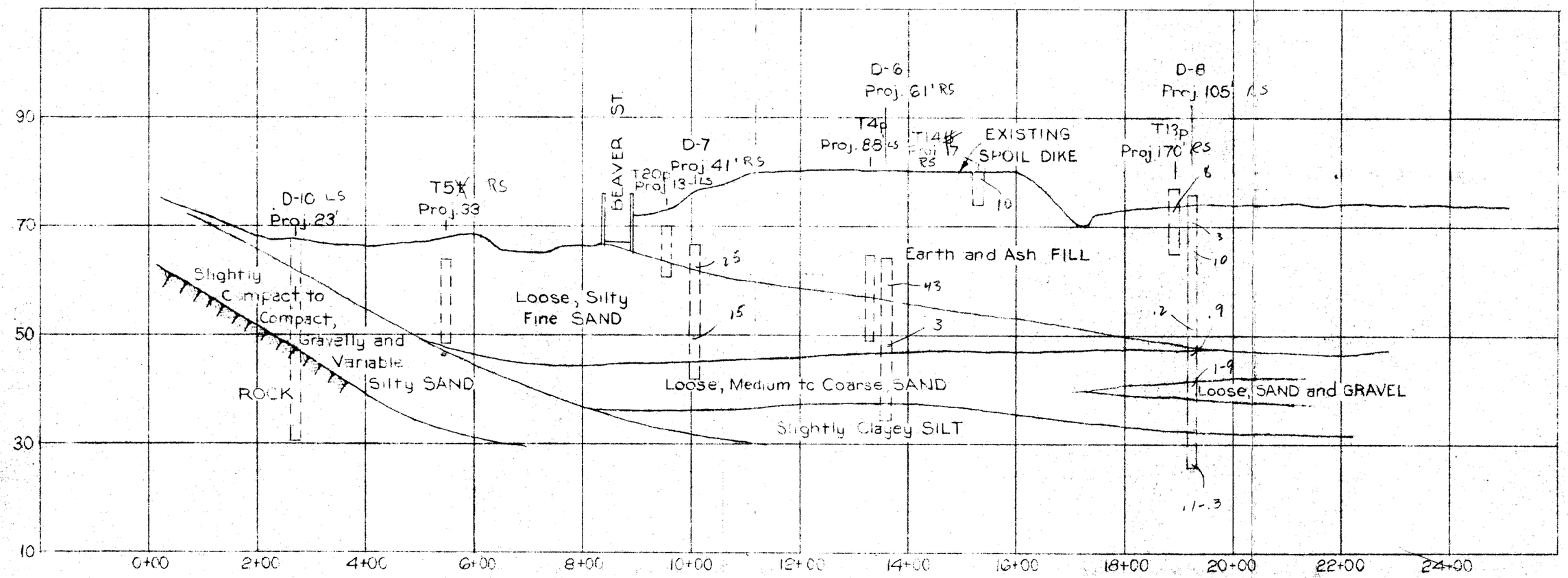
LAKEVIEW SECTION

GEOLOGICAL PROFILE ALONG ℄ OF DIKE

SCALE - HOR 1" = 200', VERT. 1" = 20'

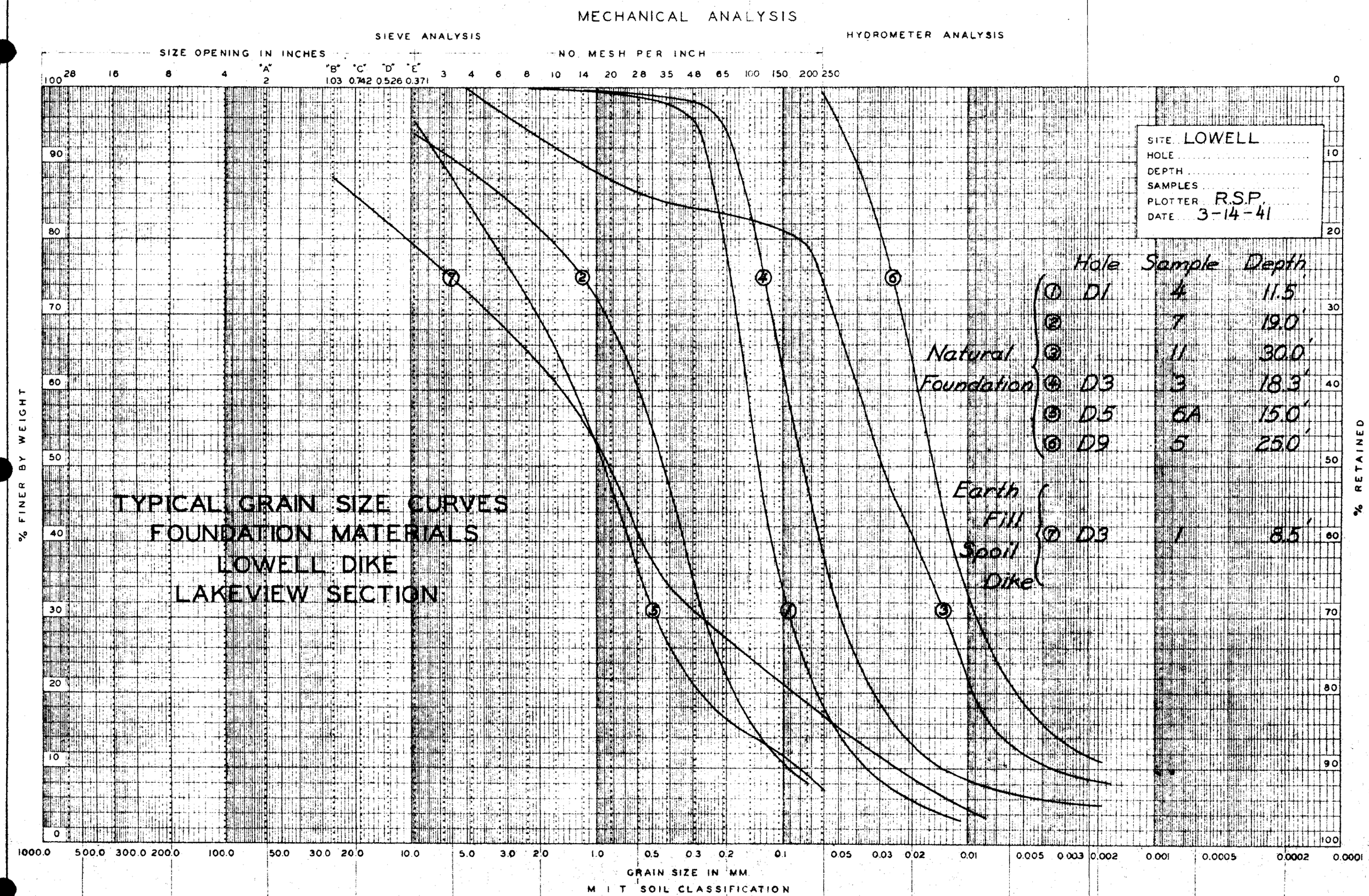
MAR, 1941

PLATE III-1



LOCAL PROTECTION-LOWELL, MASS.
ROSEMONT SECTION
GEOLOGICAL PROFILE
ALONG ℄ OF DIKE
SCALE- HOR. 1" = 200' VERT. 1" = 20'
MAR., 1941

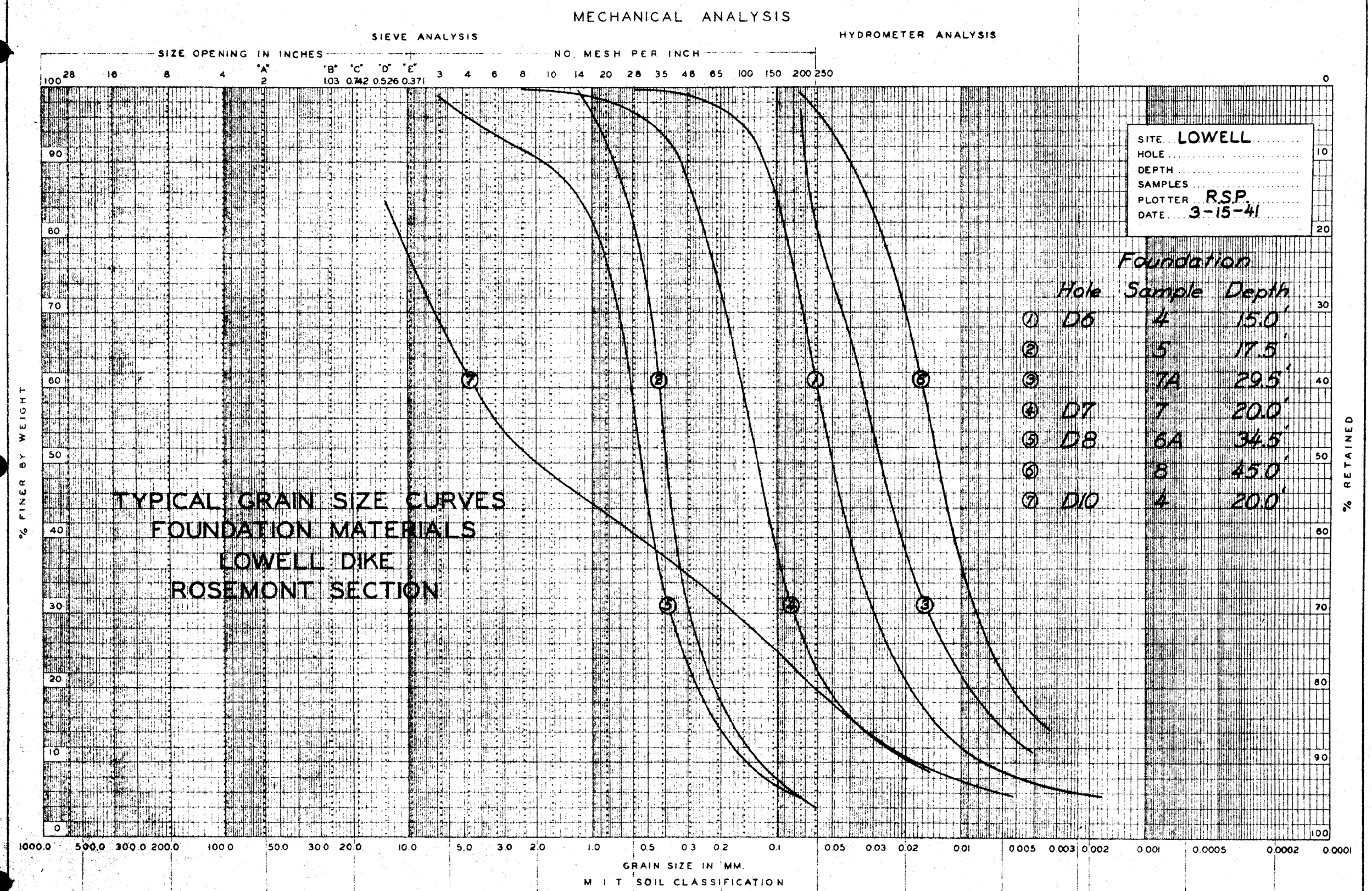
PLATE III-2



* These sizes not included in original M.I.T. Classification

PLATE III-3

SL-5.00



* These sizes not included in original M.I.T. Classification

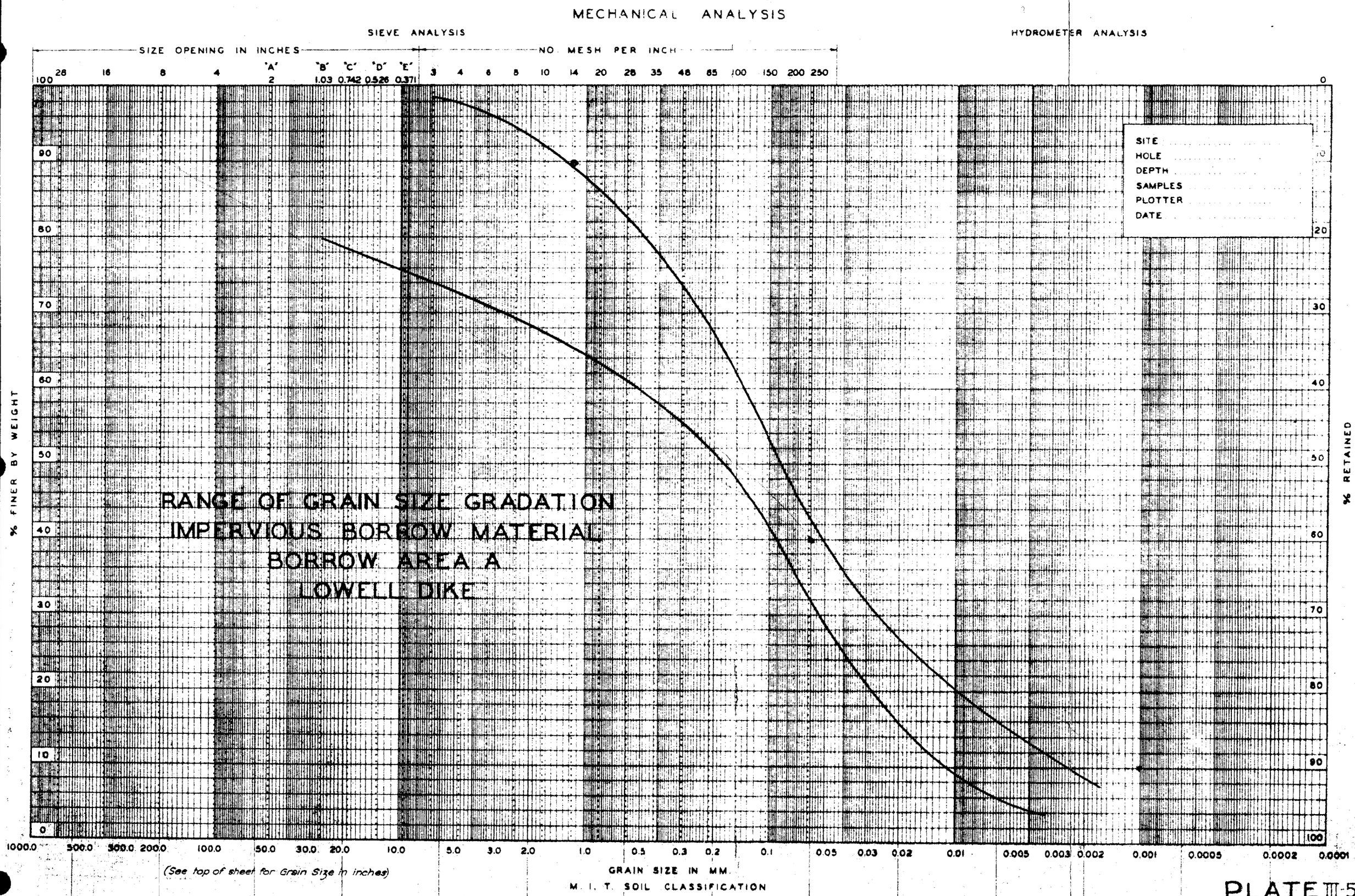
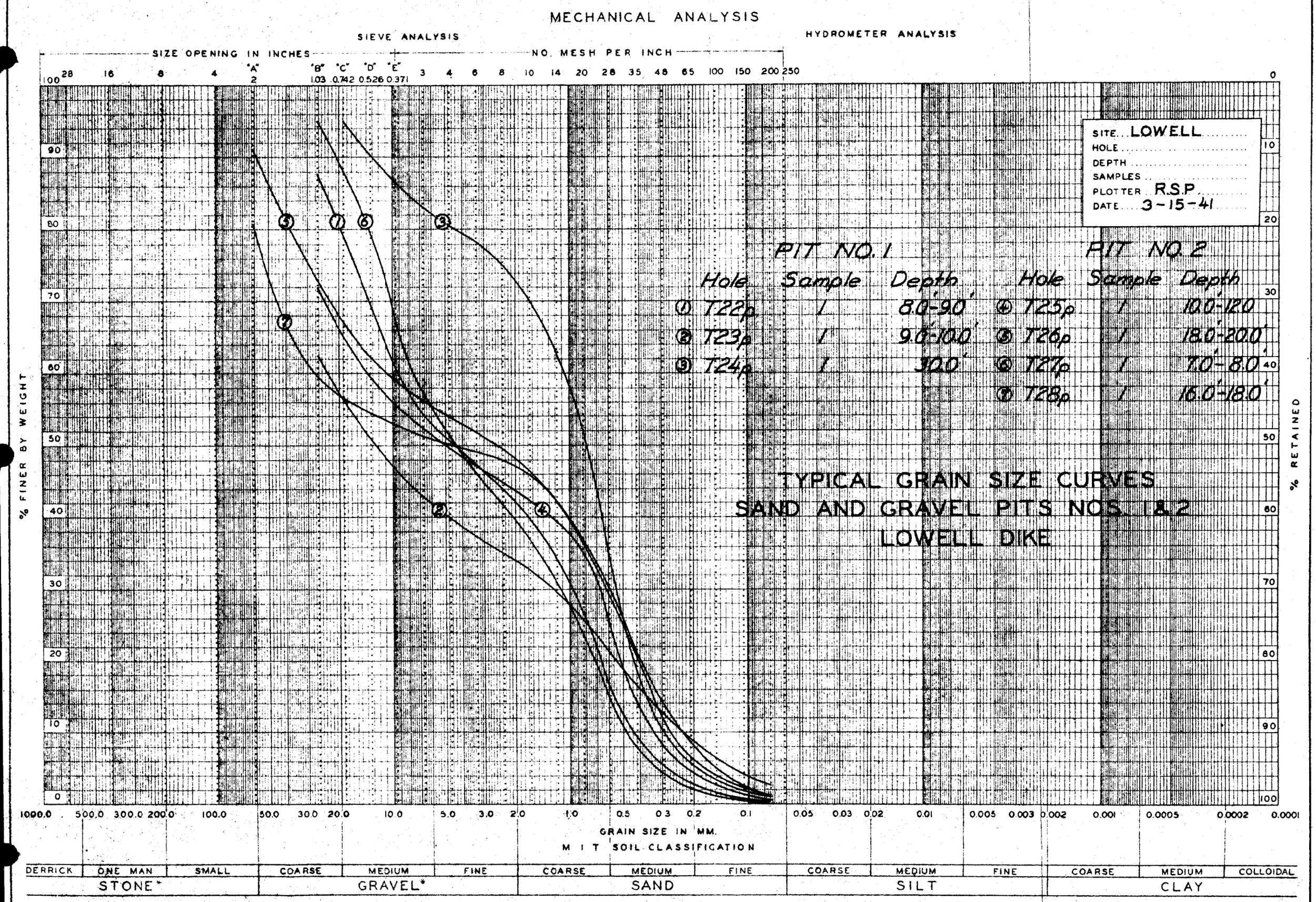


PLATE III-5

* These sizes not included in original M.I.T. Classification



* These sizes not included in original M.I.T. Classification

PLATE III-6

SL-5.00

	Material "B" Moist – 140 #/c.f. = 0.07 Tons			Material "A" Sat – 138 #/c.f. = 0.069 Tons			Material "A" Moist – 126 #/c.f. = 0.063 Tons			Material "C" Sat. – 130 #/c.f. = .065 Tons		
	Volume	δ	Weight	Volume	δ	Weight	Volume	δ	Weight	Volume	δ	Weight
1	17.5	0.07	1.22				15.3	0.063	0.96			
2	26.6		1.86	18.5	0.069	1.28	27.5		1.73			
3	9.7		.68	48.0		3.32	10.5		0.66			
4				59.7		4.12	4.1		0.26			
5				27.2		1.88				16.8	0.065	1.09
6				16.2		1.12				36.3		2.36
7				12.5		0.86				20.1		1.31

Areas of Sections Measured with Planimeter

	Volume	δ	Weight	Volume	δ	Weight	Volume	δ	Weight	Volume	δ	Weight	Total
1													2.18
2													4.87
3													4.66
4													4.38
5													2.97
6													3.48
7													2.17

	Hydrostatic Uplift E_U	δ	E_U	Lateral Hydrostatic Pressures	δ	E_L		δ	E_R	$E_L - E_R$
1	0			0		0	0		0	0
2	½(6.7+7.1)[illegible]	.0313	1.59	½6.7² = 22.4	.0313	0.70	0		0	0.70
3	½(8.2+6.7)[illegible]		1.92	½8.2² = 33.6		1.05	½6.7² = 22.4	.0313	0.70	0.35
4	½(7.8+8.2)[illegible]		1.91	½7.8² = 30.5		0.95	½8.2² = 33.6		1.05	0.10
5	½(6.5+7.8)6.[illegible] = 43.6		1.36	½6.5² = 21.1		0.66	½7.8² = 30.5		0.95	0.29
6	½(4.8+6.5)9.[illegible] = 51.4		1.6	½4.8² = 11.5		0.36	½6.5² = 21.1		0.66	0.30
7	½(0+4.8)(6.5-3.0)		0.94	0		0	½4.8² = 11.5		0.36	0.36

	Φ Degrees	Tan Φ	Normal N	N Tan Φ	Cohesion C	L	τC	Tangential +T	Tangential −T	ΣT
1	32	0.625	1.64	1.02	0	10.8	0	1.45		
2			2.98	1.86		8.2		1.90		
3			2.59	1.62		7.6		1.28		
4			2.36	1.48		6.1		0.96		
5			1.56	0.97		9.1		0.50		
6			1.89	1.18				0.0		
7			1.27	0.79					0.2	
Totals								6.09	0.2	5.89

Forces Resisting ΣN Tan Φ	Forces Resisting Σ τC	Forces Driving ΣT	Stability Factor ΣN Tan Φ / ΣT
8.92	0.0	5.89	1.52

Computation for Slice No. 3

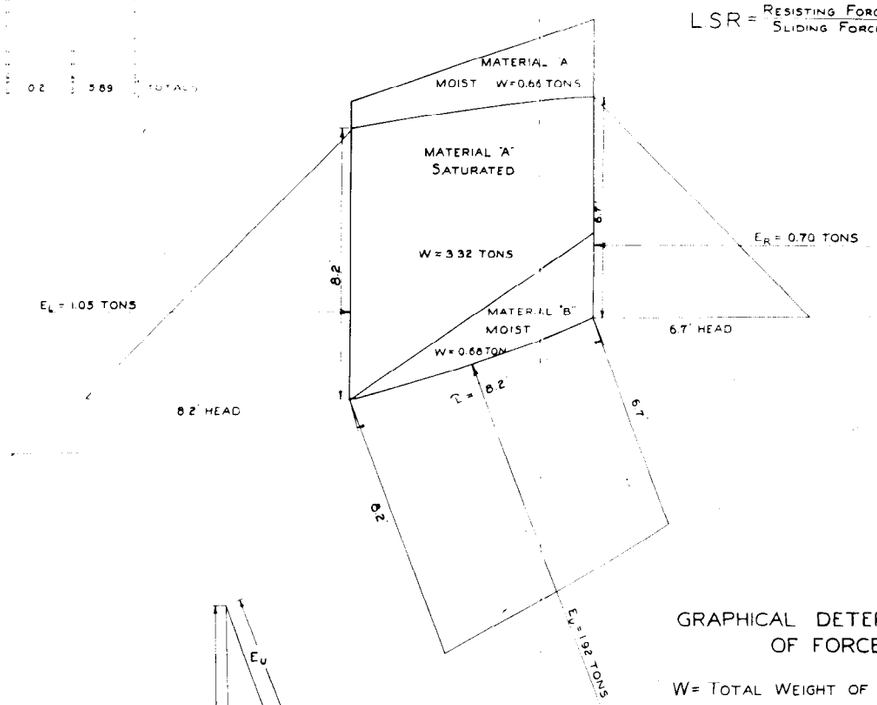
1. Weight of Material "A"
   Moist – 126 #/c.f. × 0.063 tons/c.f.
   10.5 × 0.063 = 0.66 tons
2. Weight of Material "A"
   Saturated 138 #/c.f. = 0.069 tons/c.f.
   48.0 × 0.069 = 3.32 tons
3. Weight of Material "B"
   Moist 140 #/c.f. = 0.07 tons/c.f.
   9.7 × 0.07 = 0.68 tons
4. E_U = Hydrostatic Uplift Pressure
   ½ (8.2 + 6.7) 8.2 × .0313 = 1.92 tons
5. E_L = Hydrostatic Pressure on Left
   ½ 8.2² × .0313 = 1.05 tons
6. E_R = Hydrostatic Pressure on Right
   ½ 6.7² × .0313 = 0.70 tons
7. W = Total Weight = 4.66 tons
8. From Force Polygon N = 2.59 tons T = 1.28 tons

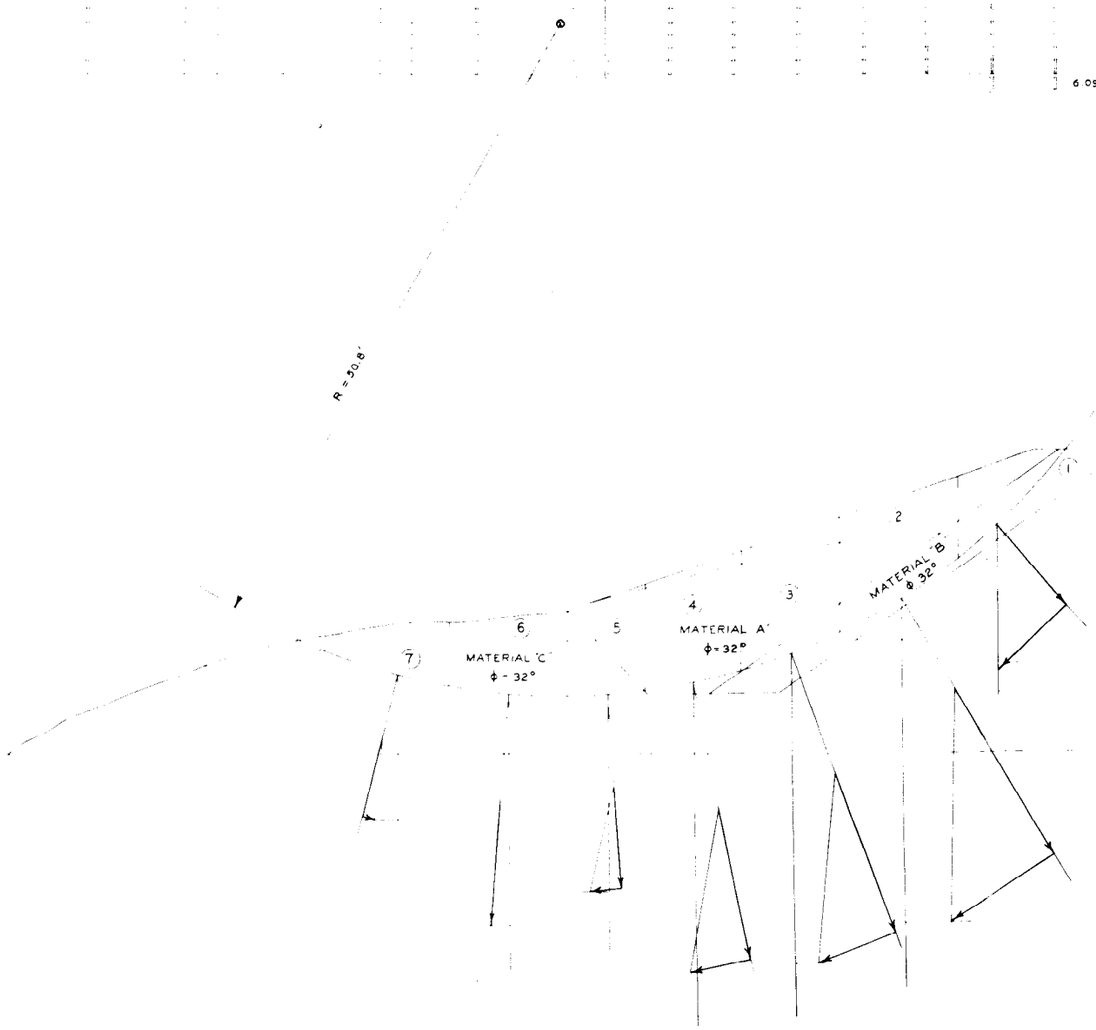
Summation for Entire Surface

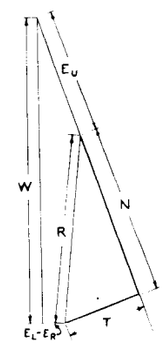
$$LSR = \frac{\text{Resisting Forces}}{\text{Sliding Forces}} = \frac{\Sigma N \tan \phi}{\Sigma T} = \frac{8.92}{5.89} = 1.52$$

Detail of Slice No 3

Scale 0 1 2 3 4 5 Feet







Graphical Determination of Forces

W = Total Weight of Section
E_U = Total Uplift (Hydrostatic)
$E_L - E_R$ = Hydrostatic Pressure on Sides
R = Resultant of E_U, W, E & E_R
N = Normal Component of R
T = Tangential Component of R
τ = Length of Arc of Sliding Surface

Merrimack Valley Flood Control
Lowell Dike
Details of Stability Analysis
Sta. 20+65

Scale 5 0 5 Feet
0 Tons

No. Sheets — Sheet No. — Scale Feet / Tons

U. S. Engineer Office 1939

TRANSFORMATION FACTOR

Assuming that $K_{max} = 4K_{min}$ all horizontal dimensions are reduced by factor, $x = \sqrt{\frac{K_{max}}{K_{min}}} = 2$

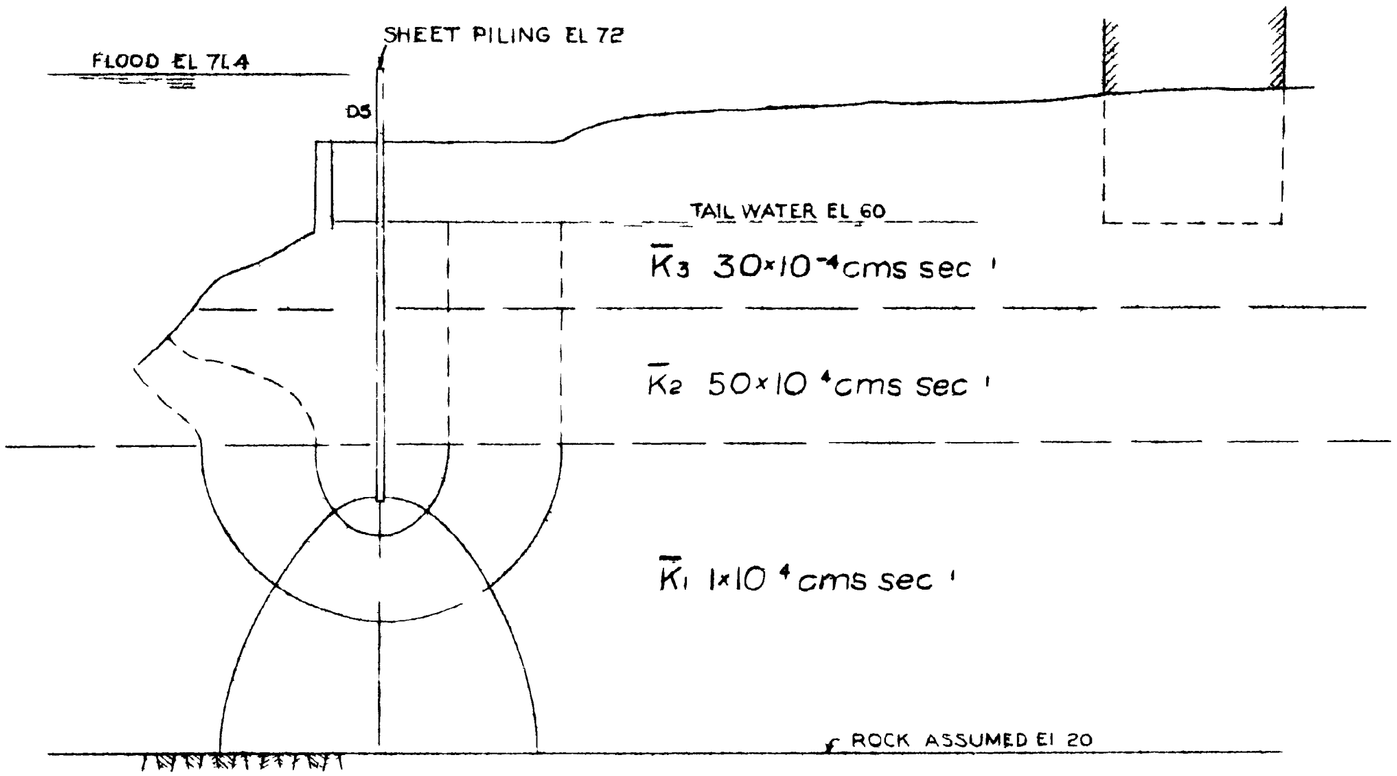
$$Q = \bar{K} \times i \times A \times t$$

$$= \bar{K} \times \frac{h}{N_E} \times N_F \times t$$

Q = quantity of Seepage per ft of Dike
$\bar{K}$ - coefficient of permeability, cms sec^{-1}
h = head, ft = 11.4
N_E = no of equipotential drops = 4.1
N_F = no of flow channels = 3
t = time = 1 min

$$Q = \frac{60}{305} \times 1 \times 10^{-4} \times \frac{11.4}{4.1} \times 3 \times 1$$

= 0.00164 cu ft/min /ft of dike



STA 5+10

Note $\bar{K}$ - average coefficient of permeability selected from test data and equal to $\sqrt{K_{max} \times K_{min}}$

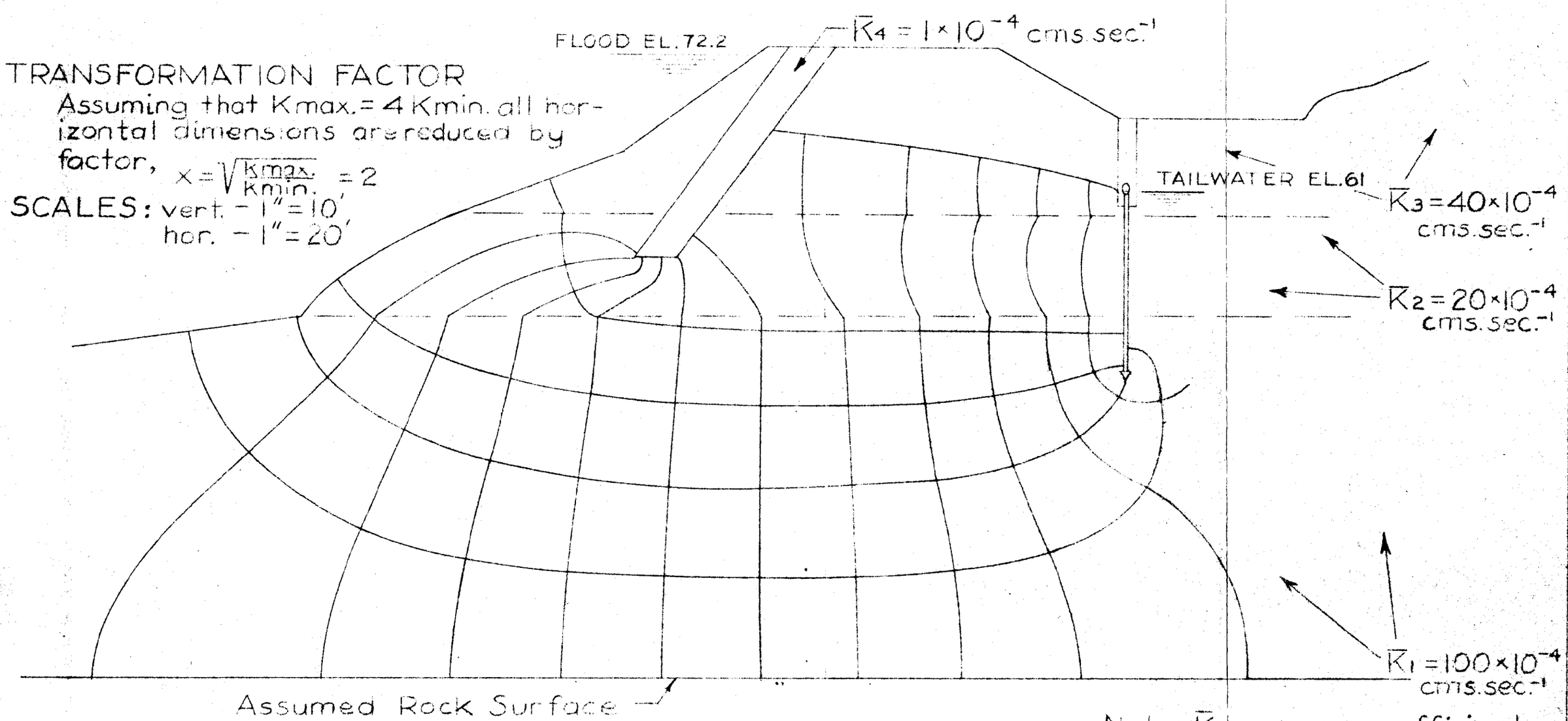
LOCAL PROTECTION LOWELL MASS
LAKEVIEW SECTION
STAS 0 00 TO 8+83
FLOW NET & SEEPAGE ANALYSIS
SCALE 1 = 20 HOR
1 10 VERT
MAR 1941

PLATE III

TRANSFORMATION FACTOR

Assuming that Kmax. = 4 Kmin. all horizontal dimensions are reduced by factor, $x = \sqrt{\frac{K_{max.}}{K_{min.}}} = 2$

SCALES: vert. – 1" = 10'
hor. – 1" = 20'



STA. 20+65

Note: $\bar{K}$ = average coefficient of permeability selected from test data and equal to $\sqrt{K_{max.} \times K_{min.}}$

$Q = \bar{K} \times i \times A \times t$

$= \bar{K} \times \frac{h}{N_E} \times N_F \times t$

Q = quantity of seepage per ft. of Dike
$\bar{K}$ = coefficient of permeability, cms. sec.$^{-1}$
h = head, ft. = 11.2
N_E = no. of equipotential drops = 12
N_F = no. of flow channels = 4.5
t = time = 1 min.

$$Q = \frac{60}{30.5} \times 100 \times 10^{-4} \times \frac{11.2}{12} \times 4.5 \times 1$$

= 0.0827 cu. ft./min./ft. of dike

LOCAL PROTECTION – LOWELL, MASS
LAKEVIEW SECTION
STAS. 8+83 TO 36+50
FLOW NET & SEEPAGE ANALYSIS
SCALE 1" = 20' HOR.
1" 10' VERT.

MAR. 1941

TRANSFORMATION FACTOR

Assuming that $K_{max} - 4\,K_{min}$ all horizontal dimensions are reduced by factor $x = \sqrt{\frac{K_{max}}{K_{min}}} - 2$

$Q\ \bar{K} \times L \times A \times t$

$\bar{K} \times \frac{h}{N_E} \times N_F \times t$

Q quantity of Seepage per ft of Dike

$\bar{K}$ coefficient of permeability cms sec^{-1}

h head ft = 13 8

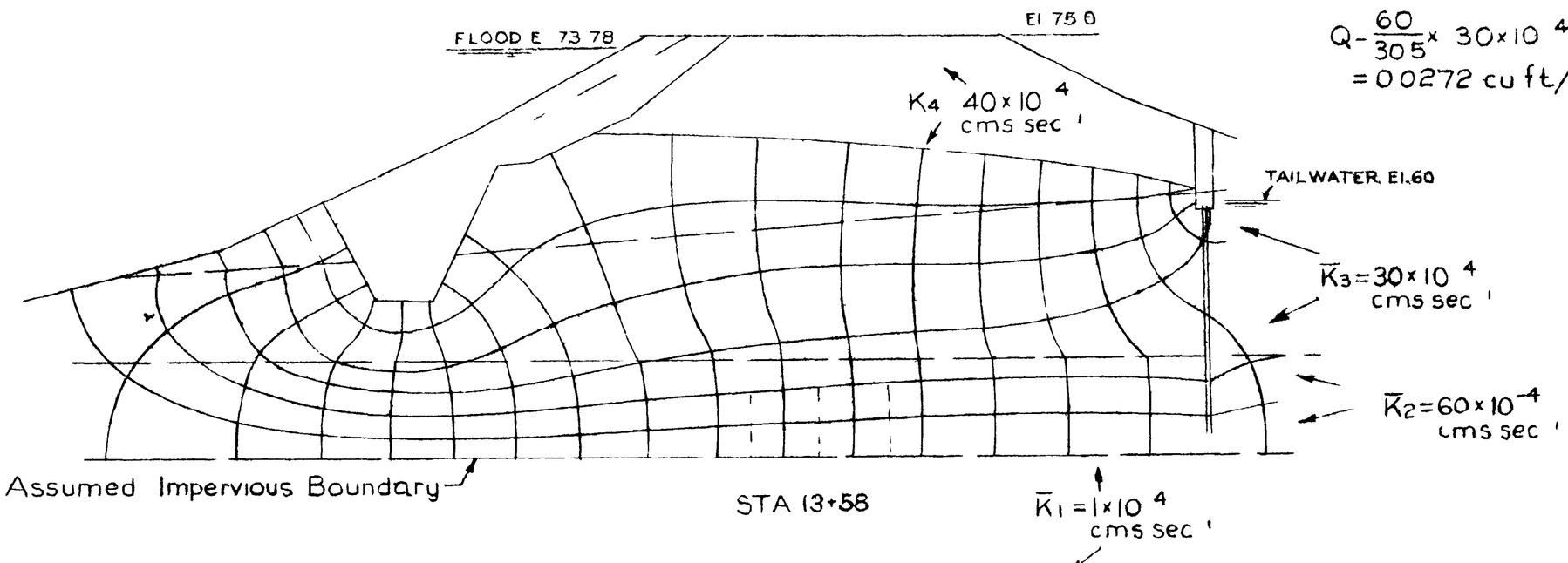
N_E no of equipotential drops = 18

N_F - no of flow channels = 6

t - time = 1 min

$Q - \frac{60}{305} \times 30 \times 10^{-4} \times \frac{13.8}{18} \times 6 \times 1$

= 0 0272 cu ft/min /ft of dike



Note $\bar{K}$ = average coefficient of permeability selected from test data and equal to $\sqrt{K_{max} \times K_{min}}$

LOCAL PROTECTION LOWELL, MASS
ROSEMONT SECTION
STAS. 9+61 TO 15+55
FLOW NET & SEEPAGE ANALYSIS
SCALE 1" = 20 HOR.
1" 10 VERT
MAR 1941

TRANSFORMATION FACTOR

Assuming that $\bar{K}max = 4Kmin$ all horizontal dimensions are reduced by factor $x = \sqrt{\frac{Kmax}{Kmin}} = 2$

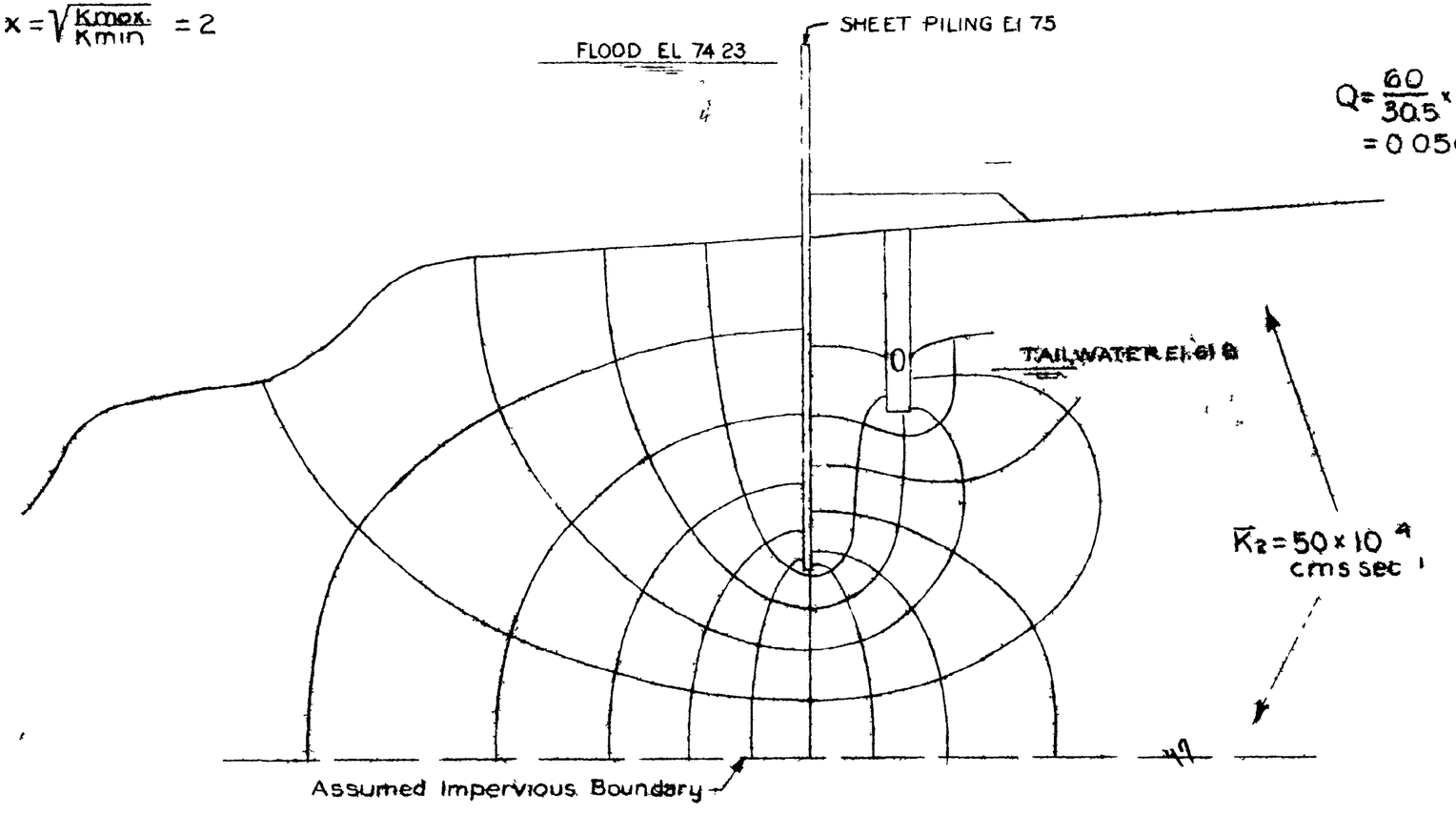
$Q\ \bar{K} \times L \times A \times t$

$= \bar{K} \times \frac{h}{N_E} \times N_F \times t$

Q - quantity of Seepage per ft of Dike
$\bar{K}$ = coefficient of permeability cms s
h = head, ft = 12 4
N_E no of equipotential drops = 12
N_F - no of flow channels - 5
t time = 1 min

$Q = \frac{60}{30.5} \times 50 \times 10^{-4} \times \frac{12.4}{12} \times 5 \times 1$

= 0 0508 cu ft /min /ft of dike



Note $\bar{K}$ = average coefficient of permeability selected from test data and equal to $\sqrt{Kmax \times Kmin}$

STA 5+48

LOCAL PROTECTION LOWELL MASS
ROSEMONT SECTION
STAS 0+20 TO 8+40
FLOW NET & SEEPAGE ANALYSIS
SCALE 1" = 10' HOR
1 = 5 VERT

MAR 1941

PLATE III-12

WAR DEPARTMENT

U. S. ENGINEER OFFICE, BOSTON, MASS.

Page

Subject LOWELL DIKE

Computation TOTAL QUANTITY OF SEEPAGE

Computed by M.M.M. Checked by VAA Date 3-12-41

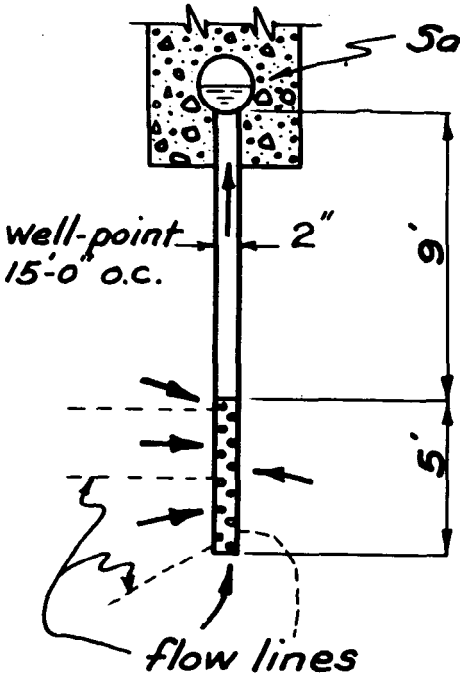
LAKEVIEW SECTION.-

Stationing	L in ft.	Flow Net	Q in cu.ft. per min.
0+00-8+83	945	Plate N Sta. 5+10	2
8+83-36+50	2705	Plate O Sta. 20+65	224
		Total=	226

ROSEMONT SECTION.-

Stationing	L in ft.	Flow Net	Q in cu.ft. per min.
40+00-50+00	1000	Plate — Sta. 48+10	15
9+61-15+55	630	Plate P Sta. 13+58	17
0+20-8+49	830	Plate Q Sta. 5+48	42
		Total=	74

Subject: LOWELL DIKE
Computation: LOSS OF HEAD IN DRAINAGE-WELL
Computed by: M.M.M. Checked by: [illegible] Date: 3-12-41



From Flow Net (Plate O):

$$Q = \frac{60}{30.5} \times K \times 10^{-4} \times \frac{h}{N_E} \times N_F \times L$$

$$= 1.968 \times 100 \times 10^{-4} \times \frac{12}{12} \times 4 \times 15$$

$= 1.2$ cu.ft. per min. = discharge into well.

Assume Standard 2-inch cast-iron pipe:—

i.d. = 2.067" Area = $\pi r^2 = 3.37$ in.^{2} = 0.0234 ft.^{2}

Velocity:—

$$v = \frac{Q}{A} = \frac{1.2}{0.0234} = 51.3 \text{ ft. per min} = 0.86 \text{ ft. per sec.}$$

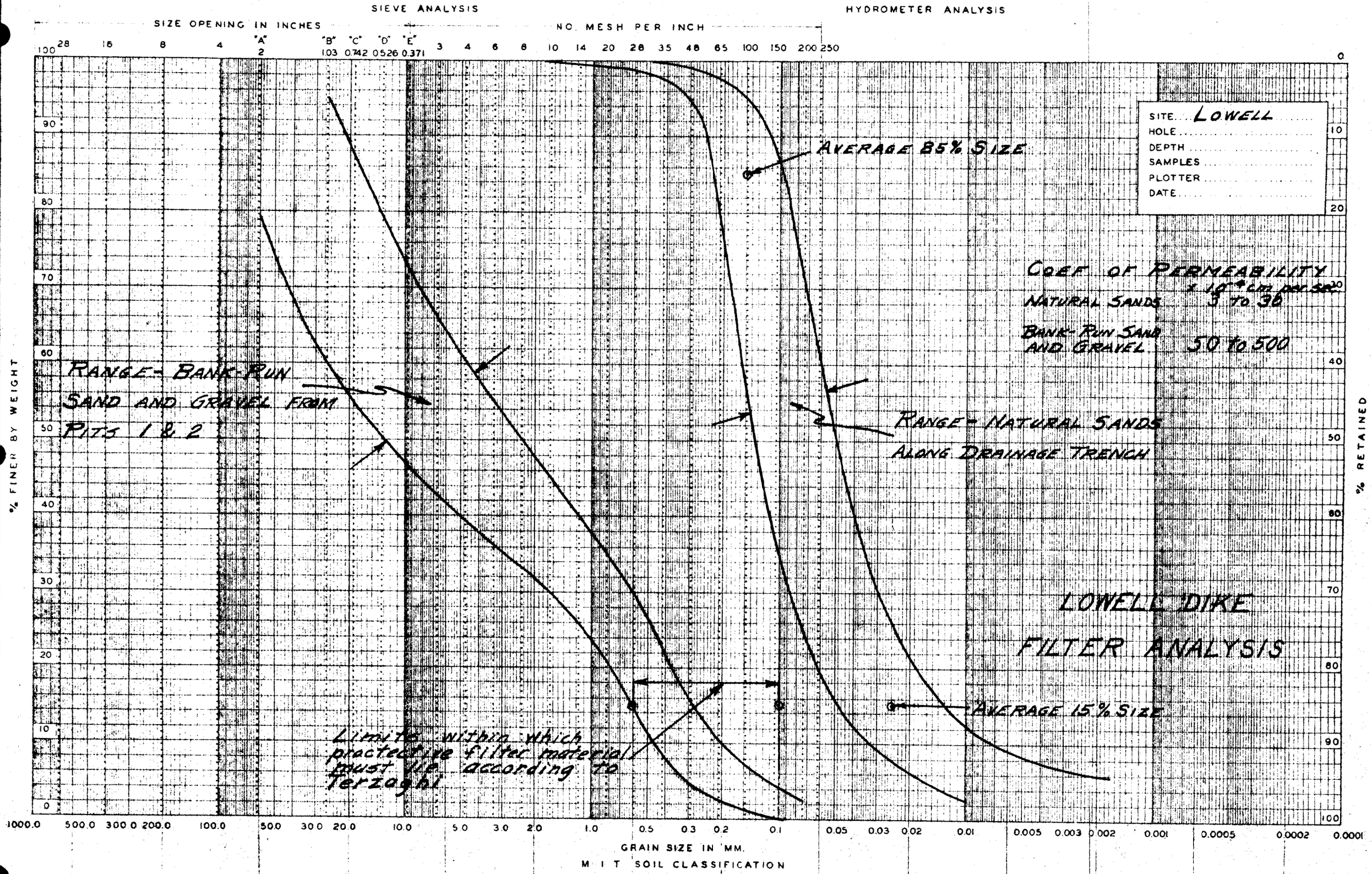
Loss of head due to friction and entrance:—

assume entrance loss equal to 20% friction loss:

$$120\% \left(f \frac{l}{d} \frac{v^2}{2g} \right) = 1.2 \left(0.04 \times \frac{14}{\frac{2.07}{12}} \times \frac{0.86^2}{64.4} \right)$$

$= \underline{0.045}$ ft., loss of head in well

MECHANICAL ANALYSIS



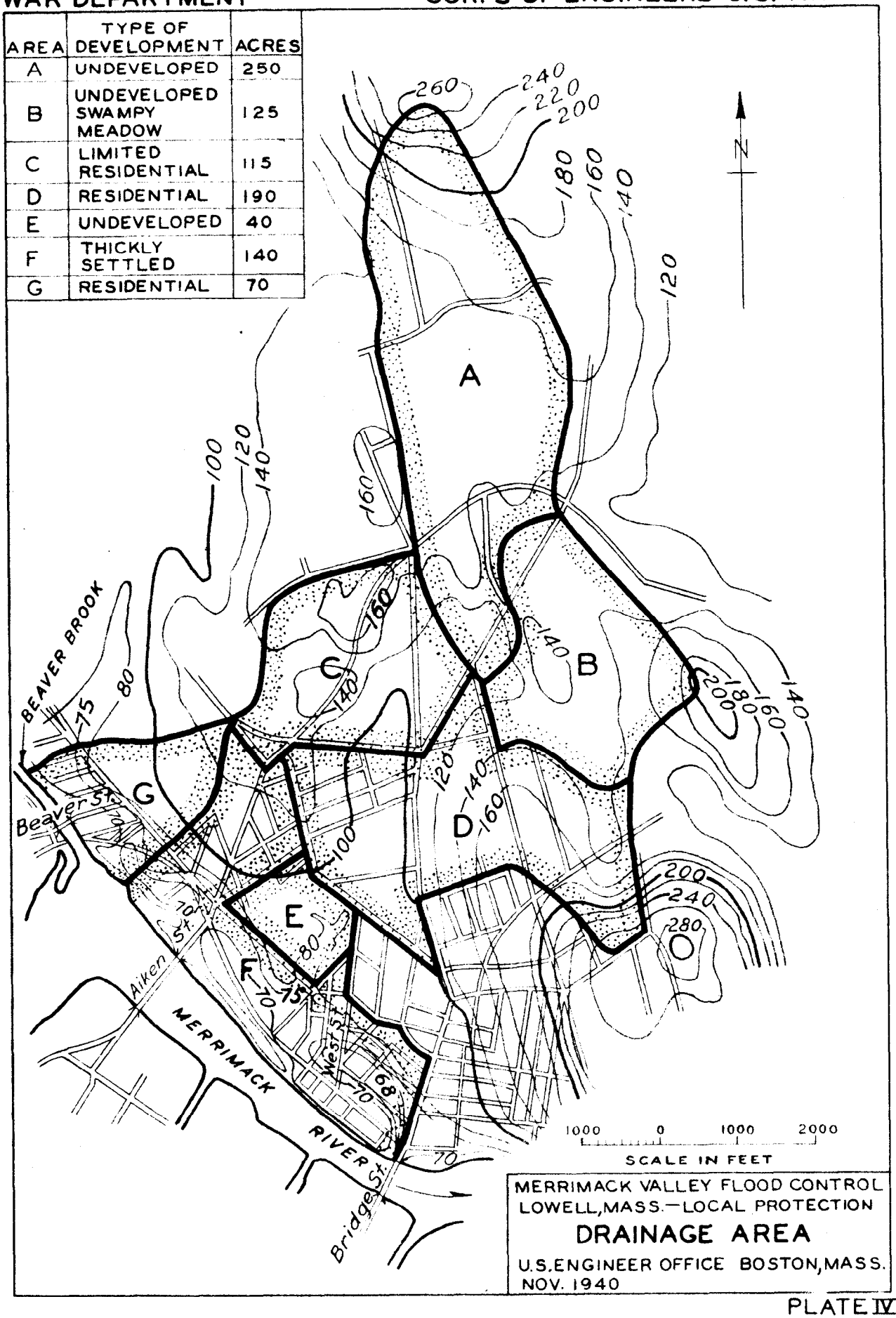
* These sizes not included in original M.I.T. classification

PLATE III-15

SL-5.00

WAR DEPARTMENT CORPS OF ENGINEERS U.S. ARMY

AREA	TYPE OF DEVELOPMENT	ACRES
A	UNDEVELOPED	250
B	UNDEVELOPED SWAMPY MEADOW	125
C	LIMITED RESIDENTIAL	115
D	RESIDENTIAL	190
E	UNDEVELOPED	40
F	THICKLY SETTLED	140
G	RESIDENTIAL	70

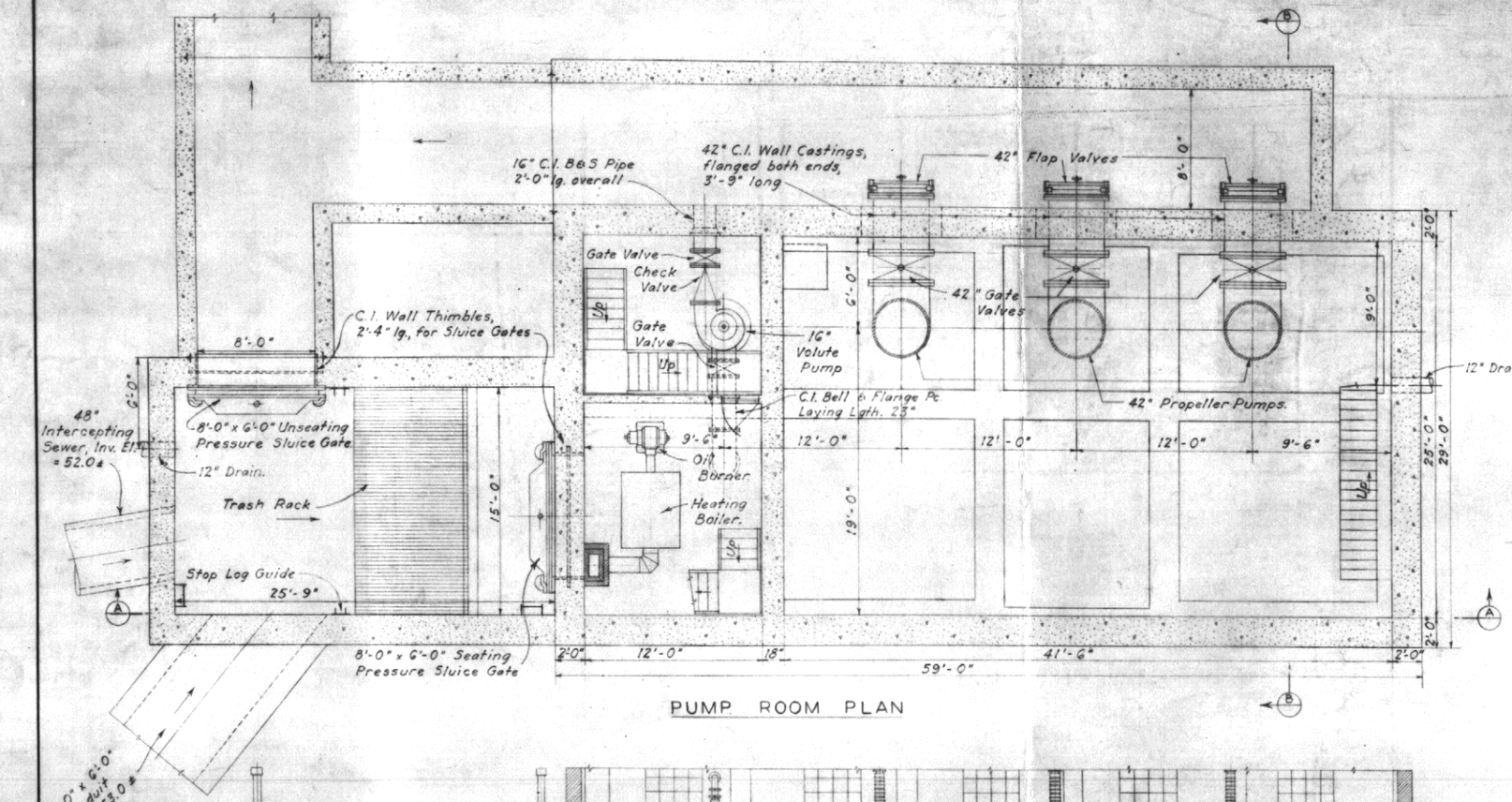


MERRIMACK VALLEY FLOOD CONTROL
LOWELL, MASS.—LOCAL PROTECTION

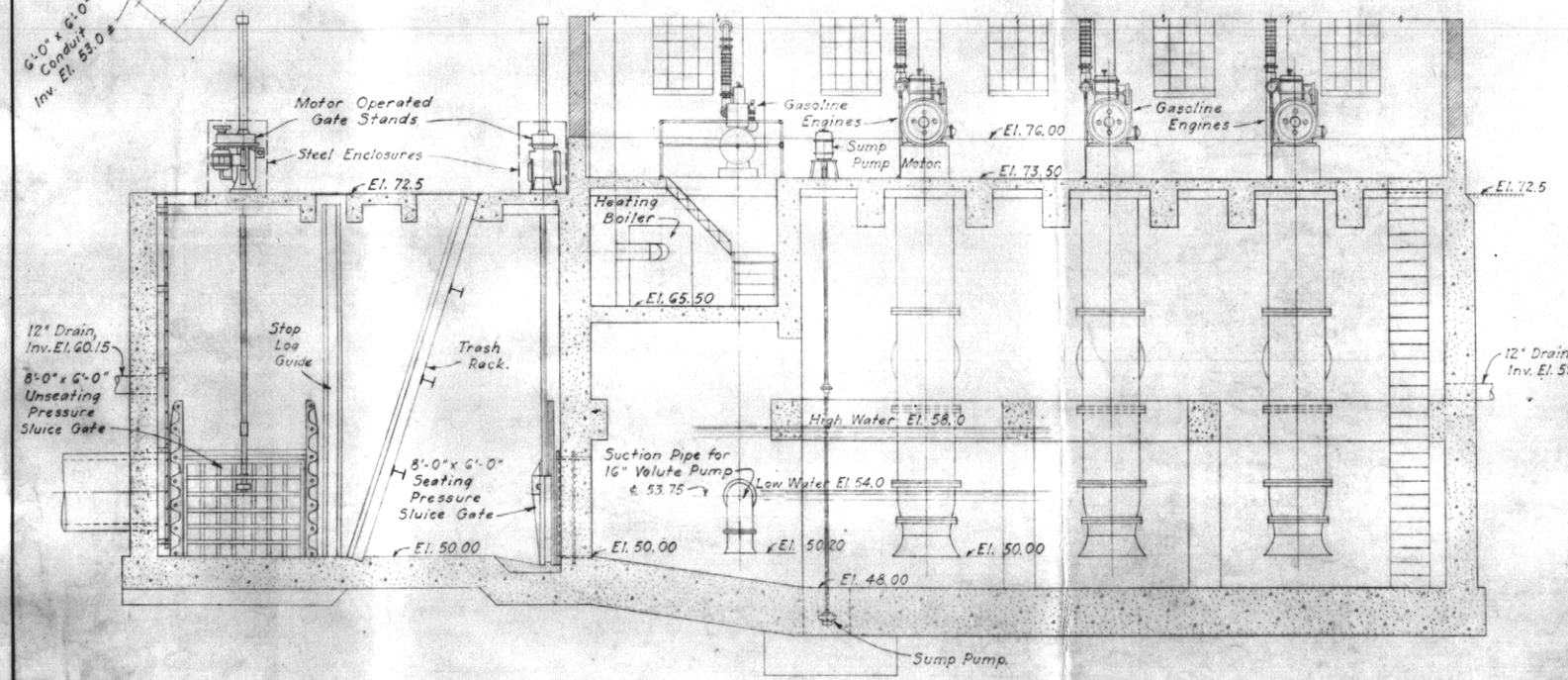
DRAINAGE AREA

U.S. ENGINEER OFFICE BOSTON, MASS.
NOV. 1940

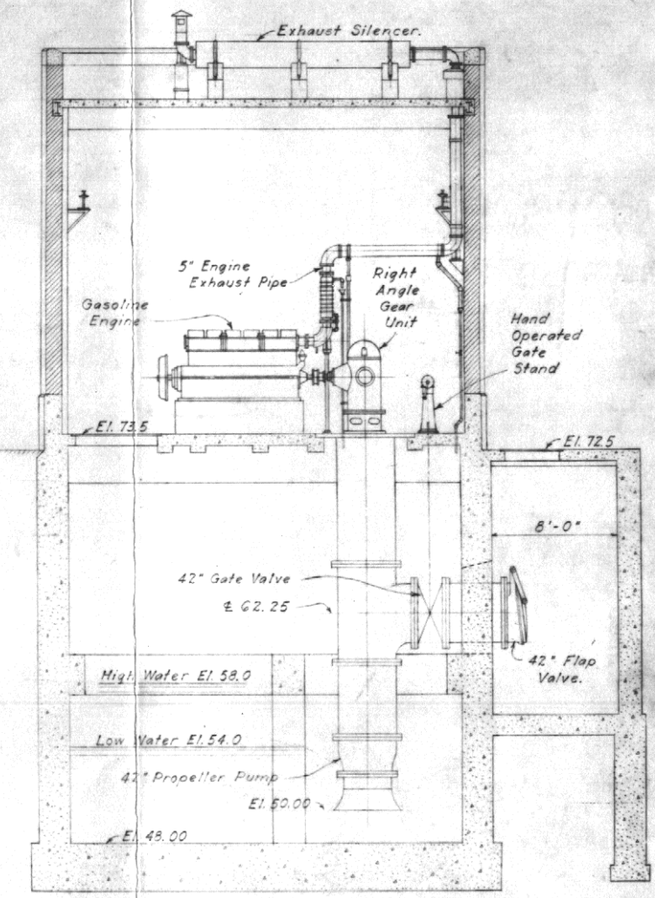
PLATE IV-1



PUMP ROOM PLAN



SECTION A



SECTION B

NO.	CHARACTER	DATE	BY
	REVISIONS		

MERRIMACK VALLEY FLOOD CONTROL
FLOOD PROTECTION – LOWELL, MASS.

WEST STREET PUMPING STATION
GENERAL ARRANGEMENT OF EQUIPMENT No. 1

IN 52 SHEETS SCALE: ¼ IN. = 1 FT. SHEET NO. 31

U.S. ENGINEER OFFICE, BOSTON, MASS.

APPROVAL RECOMMENDED: John E. Allen
CHIEF, DEFENSE AND FLOOD CONTROL SUBDIVISION

APPROVED: F. R. Gallagher
LT. COLONEL, CORPS OF ENGINEERS
DISTRICT ENGINEER

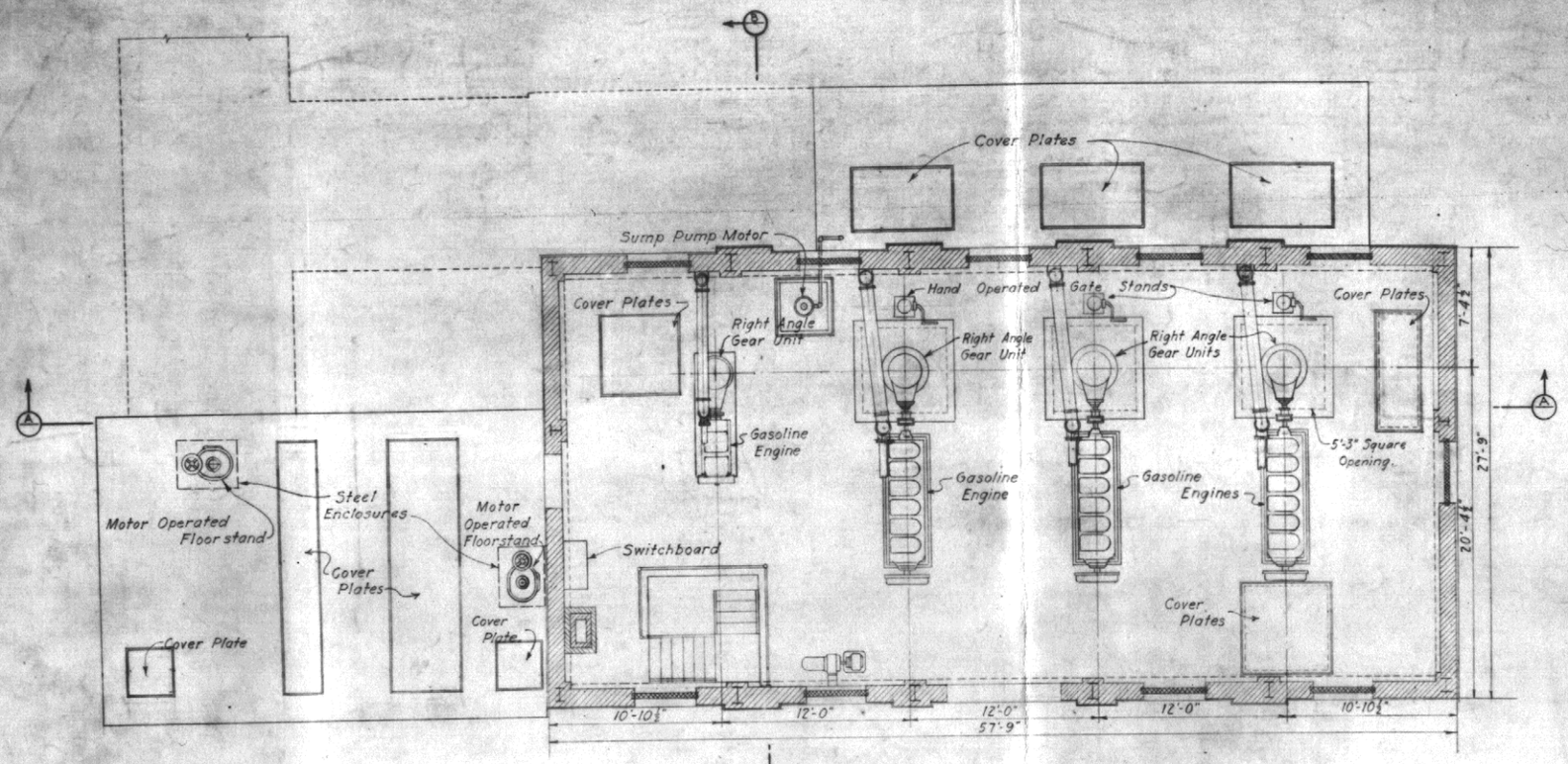
PREPARED BY
WESTON & SAMPSON
CONSULTING ENGINEERS, BOSTON, MASS.

FILE NO. M54-52/26

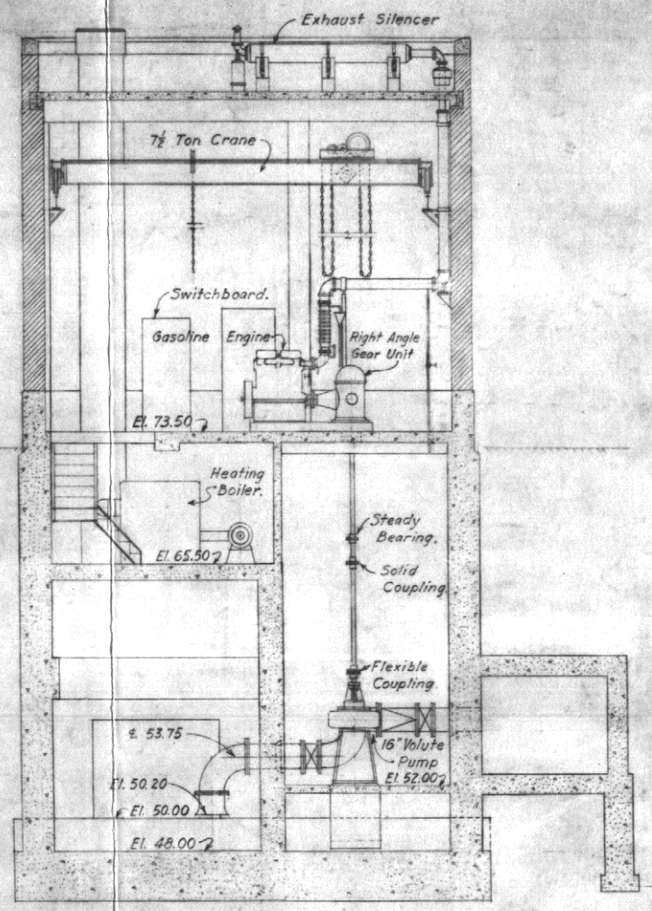
PLATE V-1

WAR DEPARTMENT

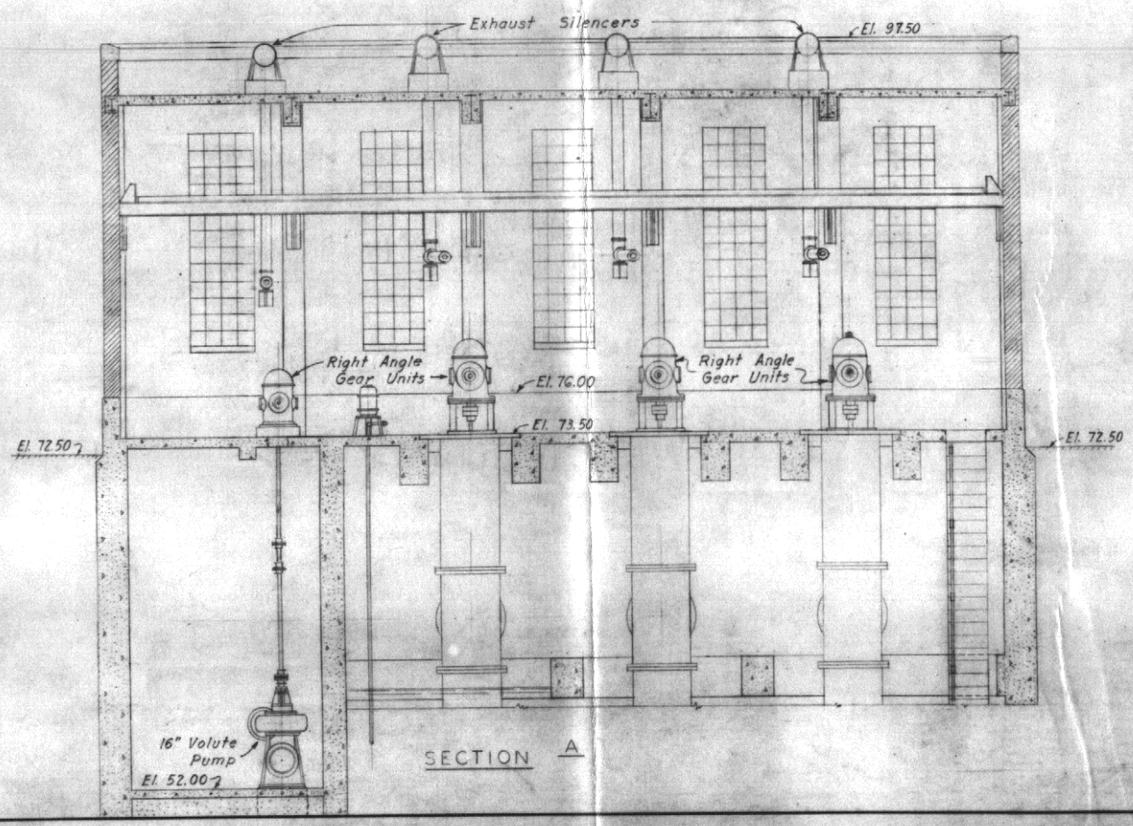
CORPS OF ENGINEERS, U.S. ARMY



ENGINE ROOM PLAN



SECTION B



SECTION A

MERRIMACK VALLEY FLOOD CONTROL
FLOOD PROTECTION – LOWELL, MASS.

WEST STREET PUMPING STATION
GENERAL ARRANGEMENT OF EQUIPMENT No. 2

IN 52 SHEETS SCALE: ¼ IN. = 1 FT. SHEET NO. 32

U.S. ENGINEER OFFICE, BOSTON, MASS.

APPROVAL RECOMMENDED
CHIEF, DEFENSE AND FLOOD CONTROL SUBDIVISION

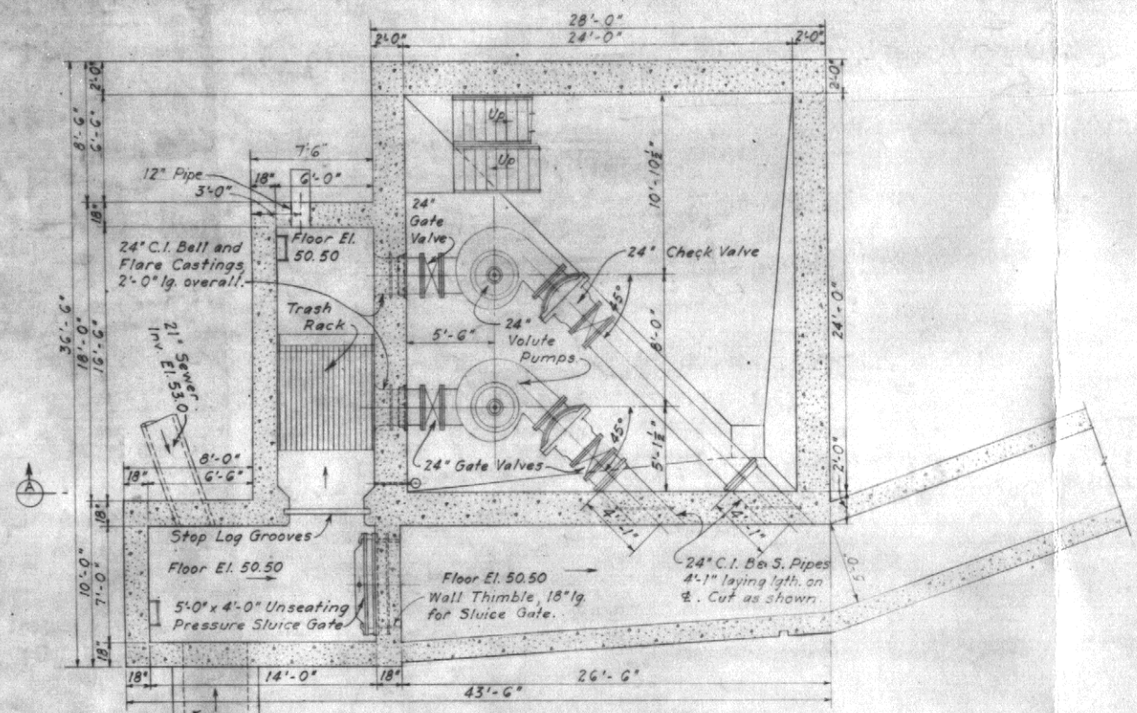
PREPARED BY
WESTON & SAMPSON
CONSULTING ENGINEERS, BOSTON, MASS.

APPROVED
LT. COLONEL, CORPS OF ENGINEERS
DISTRICT ENGINEER

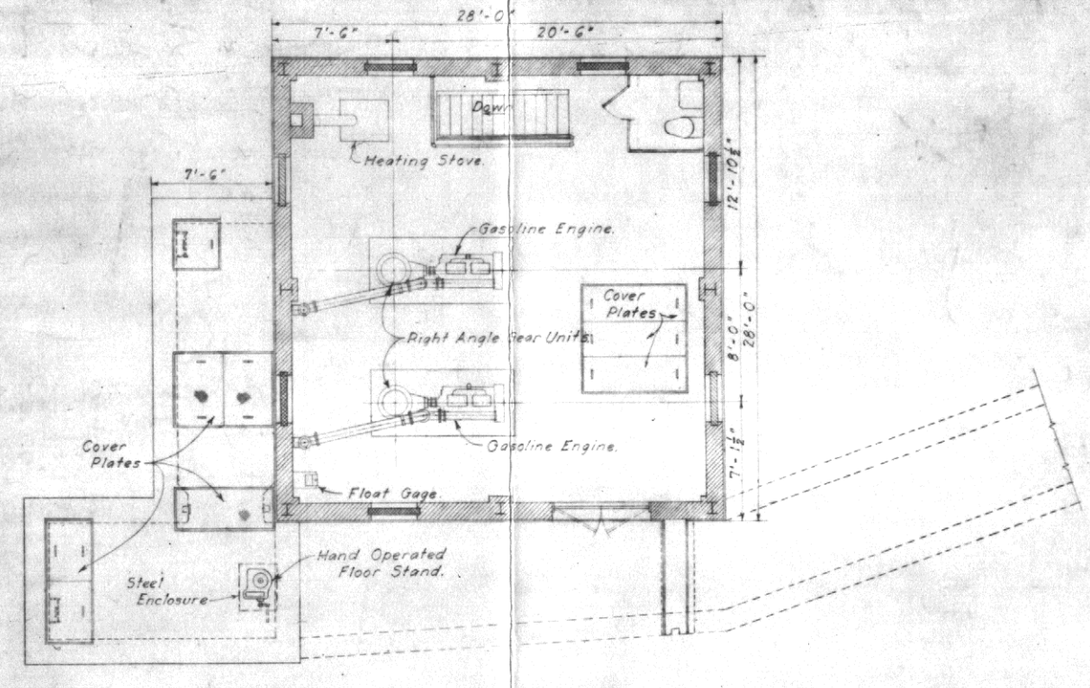
NO.	CHARACTER	DATE	BY
	REVISIONS		

FILE NO. M54-52/27

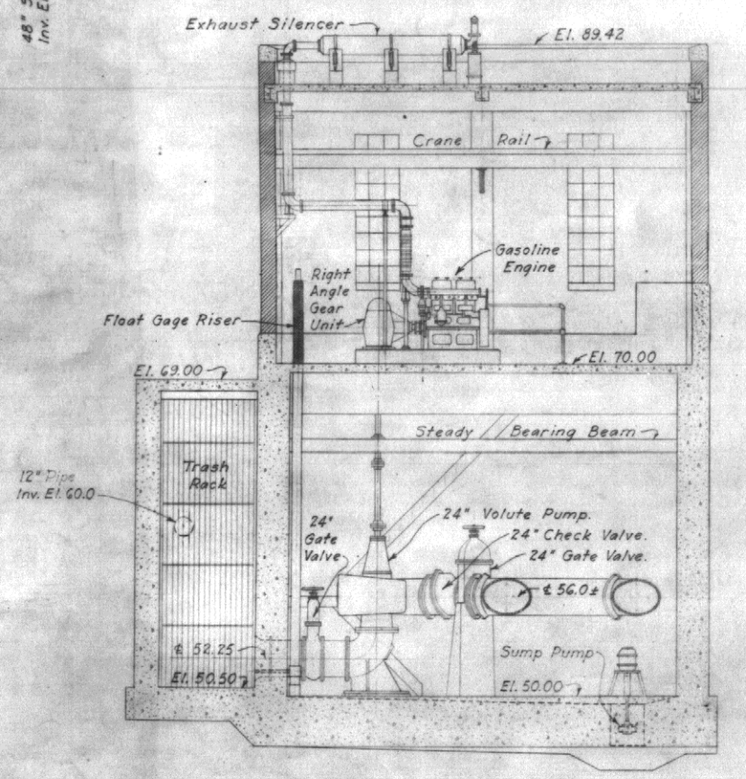
PLATE V-2



PUMP ROOM PLAN



ENGINE ROOM PLAN



SECTION A

MERRIMACK VALLEY FLOOD CONTROL
FLOOD PROTECTION – LOWELL, MASS.

BEAVER STREET PUMPING STATION
GENERAL ARRANGEMENT OF EQUIPMENT

IN 52 SHEETS SCALE: ¼ IN. = 1 FT. SHEET NO. 52

U.S. ENGINEER OFFICE, BOSTON, MASS.

APPROVAL RECOMMENDED
CHIEF, DEFENSE AND FLOOD CONTROL SUBDIVISION

APPROVED
LT. COLONEL, CORPS OF ENGINEERS
DISTRICT ENGINEER

PREPARED BY
WESTON & SAMPSON
CONSULTING ENGINEERS, BOSTON, MASS.

NO.	CHARACTER	DATE	BY

REVISIONS

FILE NO. M54-54/16

PLATE V-3

Subject: Lowell Local Protection
Computation: Sheet Piling Wall - Beaver St Area
Computed by: J.W.L. & V.H.K. Checked by: JWL Date: 2-28-41

Replace Loads above "A" by a single concentrated load

$562 \times 4.5 = 2580 \times 5.01 = 12700$
$562 \times 1.01 = 568 \times 1.33 = 755$
$3098\# \qquad 13455'\#$
$4.37'$

$\Sigma H = 0$

$$3098 - \frac{279X^2}{2} + \left[279X + 562 + 279(X + 2.01)\right]\frac{Z}{2} = 0$$

$$6196 - 279X^2 + [558X + 1124]Z = 0$$

$$2(279X + 562)Z = 279X^2 - 6196$$

$$Z = \frac{279X^2 - 6196}{2(279X + 562)} = \frac{279(X^2 - 22.2)}{2(2.79X + 562)} \quad \text{I}$$

$\Sigma M = 0$

$$3098(X + 4.37) - 279\frac{X^3}{6} + 2(279X + 562)\frac{Z^2}{6} = 0$$

$$18588(X + 4.37) - 279X^3 + 2(279X + 562)Z^2 = 0$$

$$Z^2 = \frac{279X^3 - 18588(X + 4.37)}{2(279X + 562)} \quad \text{II}$$

Square I and equate to II

$$\frac{(279)^2(X^2 - 22.2)^2}{(2)(279X + 562)^2} = \frac{279X^3 - 18588(X + 4.37)}{2(279X + 562)}$$

$$\frac{\cancel{(279)}(279)(X^2 - 22.2)^2}{\cancel{(2)(279X + 562)}(2)(279X + 562)} = \frac{\cancel{279}(X^3 - 66.5[X + 4.37]}{\cancel{2(279X + 562)}}$$

$$(X^2 - 22.2)^2 = (X^3 - 66.5X - 291)(2X + 4.03)$$

$$X^4 - 44.4X^2 + 490 = 2X^4 + 4.03X^3 - 133X^2 - 850X - 1176$$

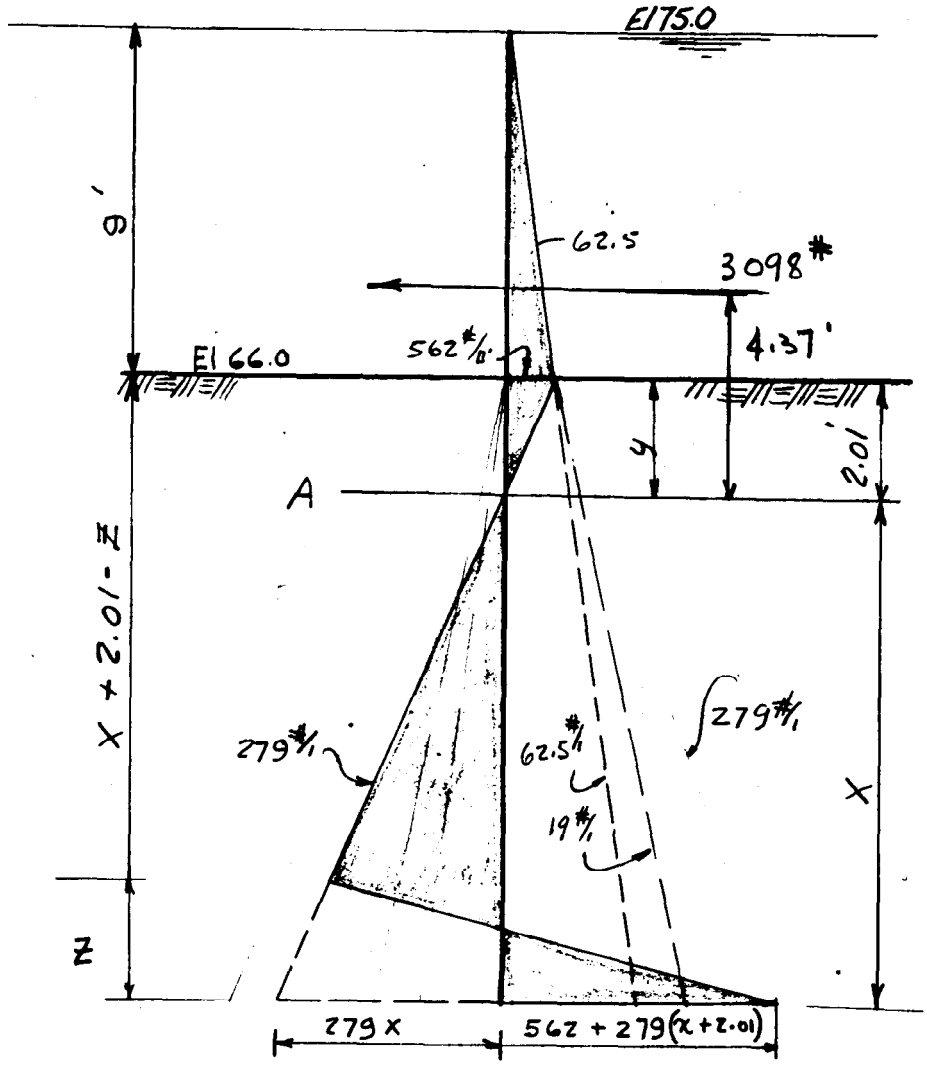
$$-X^4 - 4.03X^3 + 88.6X^2 + 850X + 1666 = 0$$

$$X^4 + 4.03X^3 - 88.6X^2 - 850X - 1666 = 0$$

$X = 11.38 \qquad Z = 4.02$

Use $X = 11.5'$

Subject: Lowell Local Protection
Computation: Sheet Piling Wall - Beaver St. Area
Computed by: J.W.L. & V.H.K. Checked by: JWL Date: 2-28-41



Assume: water at top of sheet piling due to possibility of wave action
multiply passive pressure by 1.5 (see Carnegie Steel Sheet Piling Handbook, page B6
$w = 100$ #/c.f. w (submerged) $= 61$ #/c.f.
$\phi = 32°$

$\frac{1-\sin\phi}{1+\sin\phi} = \frac{.47}{1.53} = .308$ $.308 \times 61 = 19$ #/'

$\frac{1+\sin\phi}{1-\sin\phi} = \frac{1.53}{.47} = 3.25$ $3.25 \times 61 \times 1.5 = 298$ #/'

net pp $= 298 - 19 = 279$ #/'

$\frac{562}{279} = 2.01' = y$

Subject Lowell Local Protection
Computation Sheet Piling - Beaver St Area
Computed by V.H.K. Checked by Date

Section modulus = $\frac{24000 \times 12}{16000}$ = 18.0 $in^3/ft.$

Use WZ 27 or equivalent (S = 19)

Required depth of penetration 11.5'
2.0'
13.5'

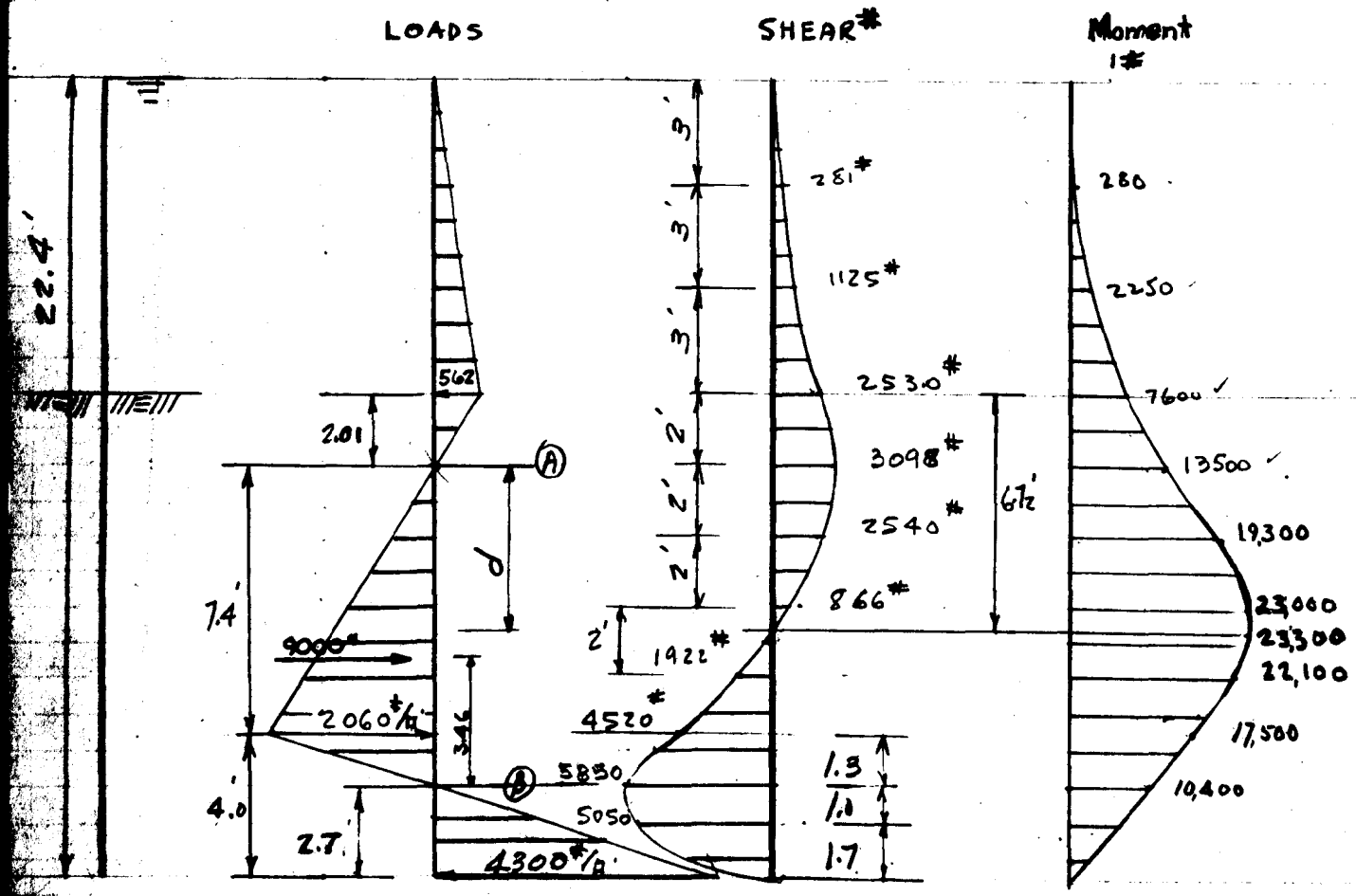
Total length of pile required 22.5'

In order to afford some factor of safety against sliding of the toe make total length 24'-0"

WAR DEPARTMENT
U. S. ENGINEER OFFICE, BOSTON, MASS.

Subject: Lowell Local Protection
Computation: Sheet Piling Wall - Beaver St Area
Computed by: J.W.L. & V.H.K. Checked by: JWL Date: 2-28-41

Page VI-3

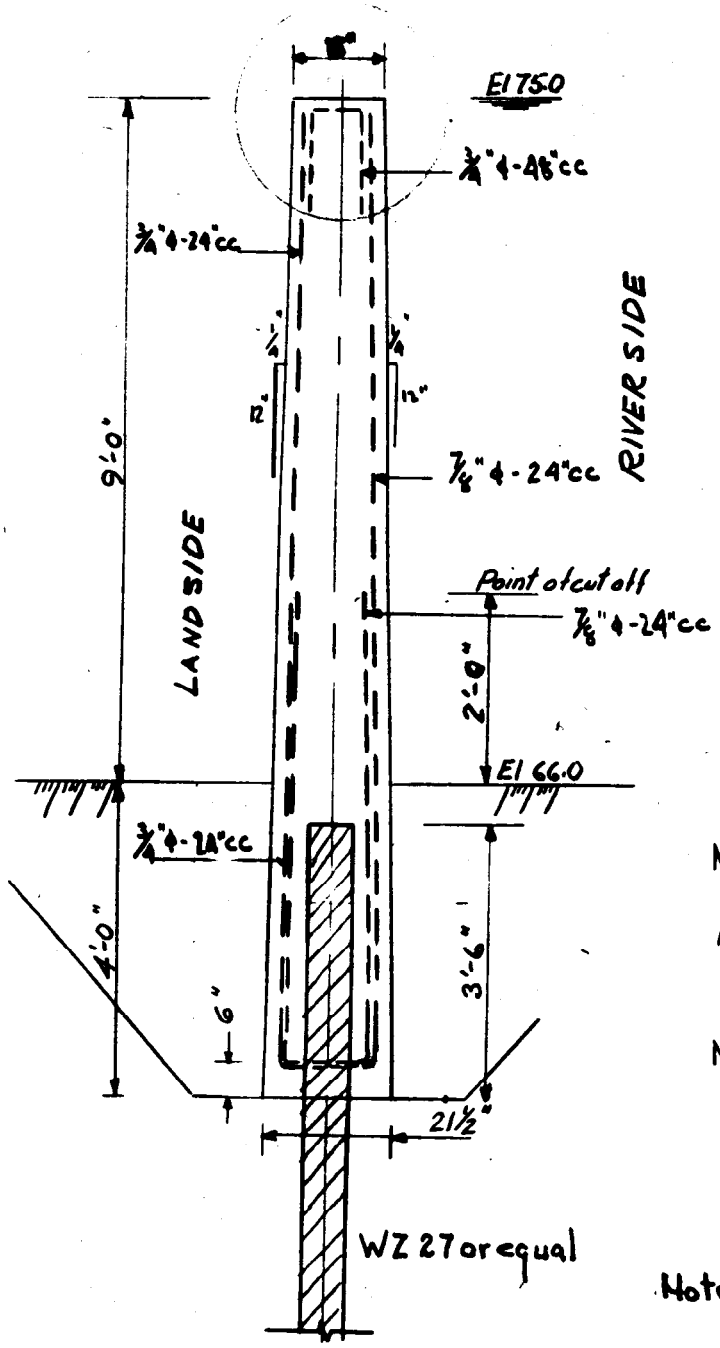


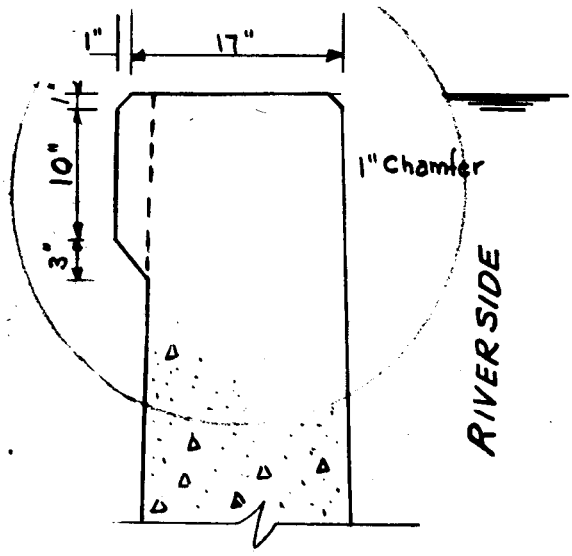
Max Moment say 24000 1#
Max Shear say 5800 #
Note - effect of friction on the pile has been neglected

13 x 279 = 3630 #/□' 7.4 x 279 = 2060 #/□' 3630 562 4192 #/□'	Max Shear A = 3098# B:- $3098 - \frac{6.06 \times 2060}{2} - \frac{2.67(2060)}{2}$ = 5850#	Zero Shear $3098 - \frac{279 d^2}{2} = 0$ d = 4.72 2 6.72'	3098 x 6.37 - 46.5(2)³ = 19,300 3098 x 8.37 - 46.5(4)³ = 23,000 3098 x 9.08 - 46.5(4.72)³ = 23,300 Max 3098 x 10.37 - 46.5(6)³ = 22,100 3098 x 11.77 - 46.5(7.4)³ = 17,500 check MAX M. 33670 - 4160 - 1780 - 4730 = 23000

Note
Rounding out dimensions causes slight unbalance in shear and moment

Subject: LOWELL LOCAL PROTECTION
Computation: SHEET PILING WALL - BEAVER ST. AREA
Computed by: JWA Checked by: V.H.K. Date: 2-28-41





DETAIL OF COPING & PIER

Moment 2'-0" below ground = 13,500'# (see pg 3)

$A_s = 13{,}500 \times 12 \div 18{,}000 \times .867 \times 17 = 0.61$ □

use 7/8"φ-12"cc (.6, 2.8)

Shear and bond O.K.

Moment 1'-0" above ground:

$62.5 \times \frac{8^3}{6} = 5340'\#$ ✓

$A_s = 5340 \times 12 \div 18{,}000 \times .867 \times 15.5 = 0.26$ □ ✓

use 7/8"φ-24"cc (land side)

Note: all horizontal steel ½"φ-10"cc ✓

SHEET PILING to be cut as indicated

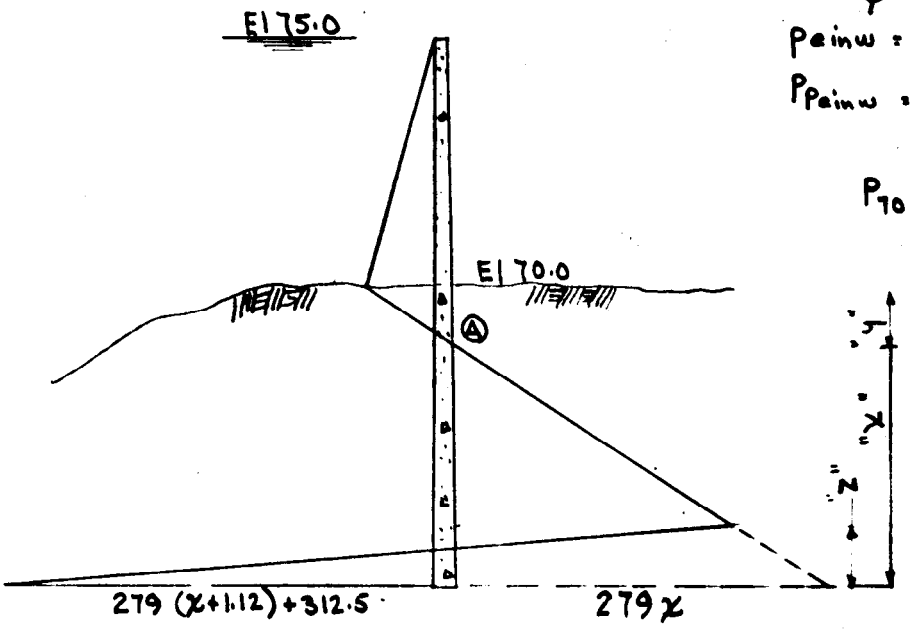
∴ Use WZ 27 (or equivalent)

Subject: LOWELL LOCAL PROTECTION
Computation: END MONOLITH (All concrete) - ROSEMONT
Computed by: P.H. Checked by: [illegible] Date: 3-3-41

W_E (dry) = 100#/c.f.
(sub) = 61#/c.f.
ϕ = 32°
P_{ainw} = 19#/□
P_{Painw} = 298#/□ } Net = 279#/□

P_{70} = 5 × 62.5 = 312.5#/□

distance to Ⓐ from ground = $\frac{312.5}{279}$ = 1.12
∴ y = 1.12



Replace loading above Ⓐ by a single force

312.5 × 5 × ½	= 782	× 2.79	= 2179
312.5 × 1.12 × ½	= 175	× 0.75	= 131
	957	2.41	2310

Required depth of penetration:

$\Sigma H = 0 \qquad 957 - 279\frac{x^2}{2} + [279x + 312.5 + 279(x+1.12)]\frac{z}{2} = 0$

$1914 - 279x^2 + (558x + 625)z = 0$

$z = \frac{279x^2 - 1914}{558x + 625} = \frac{x^2 - 6.86}{2x + 2.24}$

$\Sigma M_z = 0 \quad : \quad 957(x + 2.41) - 279\frac{x^3}{6} + (558x + 625)\frac{z^2}{6}$

subs. "z" & multiply by 6/279

$20.6(x + 2.41) - x^3 + \frac{(x^2 - 6.86)^2}{(2x + 2.24)} = 0$

$x^4 + 2.24x^3 - 27.5x^2 - 145.3x - 158.2 = 0 \qquad x = 6.34$

Subject: LOWELL LOCAL PROTECTION
Computation: END MONOLITH (All concrete) - ROSEMONT
Computed by: P. H. Checked by: [illegible] Date: 3-3-41

$$z = \frac{x^2 - 6.86}{2x + 2.24} = 2.24'$$

Point of zero shear: $957 - 279\left(\frac{d}{2}\right)^2 = 0$ $\quad d = 2.62'$, $d_{ground} = 3.74'$

Maximum moment: $957(2.41 + 2.62) - \frac{279}{6}(2.62)^3 = 3{,}976''^{\#}$

$A_s = 4000 \times 12 \div 18000 \times 0.87 \times 15.9 = 0.193$

use ½" φ-12" cc (0.196)
use ½" φ-10" cc temp.

For detail of coping see page VI-5

WAR DEPARTMENT

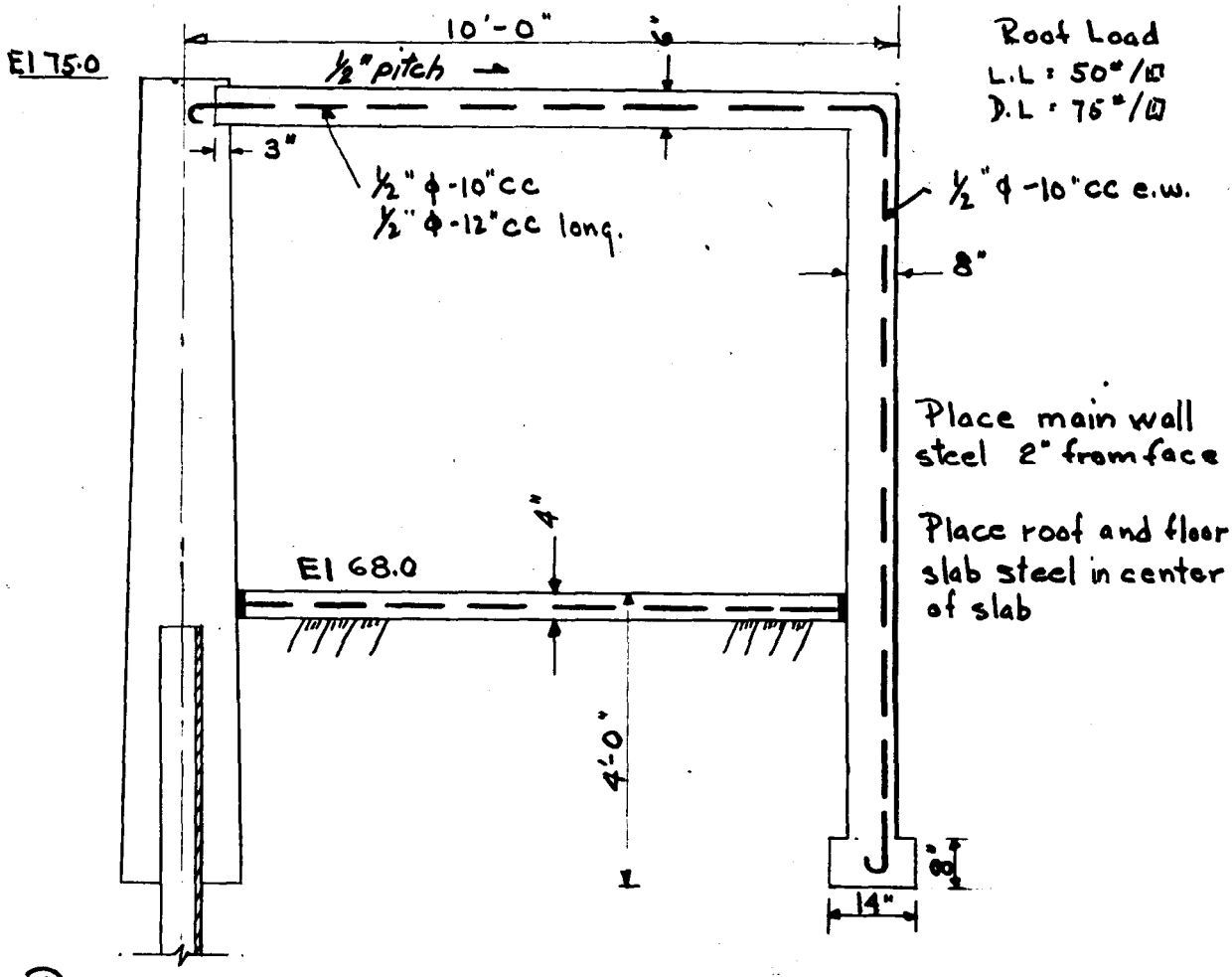
U. S. ENGINEER OFFICE, BOSTON, MASS.

Page

Subject: LOWELL LOCAL PROTECTION

Computation: STORAGE SHED

Computed by: JWH Checked by: V. H.K Date: 3-10-41



ROOF SLAB Assume 6" slab Span = 9.38'

$M = \frac{1}{12} \times 125 \times \overline{9.38}^2 \times 12 = 10{,}950$ in#

$A_s = 10{,}950 \div 18{,}000 \times 0.87 \times 3 = 0.233$ □/1 - use ½" ϕ-10" cc (0.235)
" ½" ϕ-12" cc long.

$V = 125 \times 4.69 = 586$# $v = 586 \div 12 \times 0.87 \times 3 = 18.7$ #/□ O.K.

$L_0 = 586 \div 188 \times 0.87 \times 3 = 1.20$" 1.89 supplied O.K.

$f_c = 2 \times 10{,}950 \div 0.4 \times 0.87 \times 12 \times 3^2 = 590$ #/□ O.K.

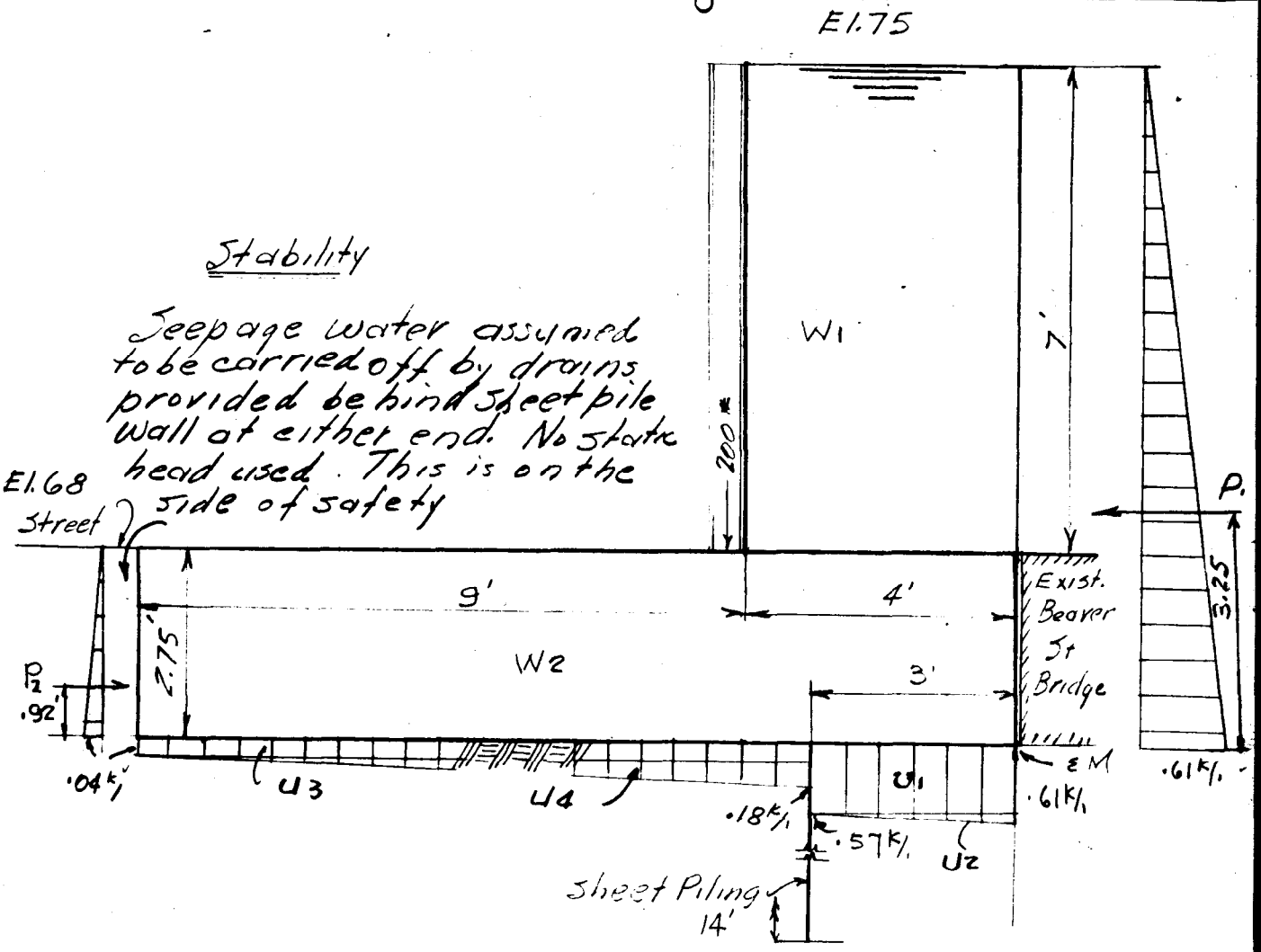
FLOOR SLAB Use uniformly supported slab resting on ground
t = 4" ½" ϕ-12" cc ea. way.

WALL

Use 8" wall with ½" ϕ-10" cc ea. way.

10" spacing on main steel of wall is used to tie into roof steel

Subject Lowell Local Protection
Computation Stop log section - Base Pressures
Computed by V.H.K Checked by JWL Date 3-4-41



Stability

Seepage water assumed to be carried off by drains provided behind sheet pile wall at either end. No static head used. This is on the side of safety

Assume water surface to top of wall
100% uplift
Friction on pile not taken into account because of the proximity of the pump house which may produce undesirable vibrations.

Uplift

U_1	$.57 \times 3 =$	1.71
U_2	$.04 \times 1.5 =$	.06
U_3	$.04 \times 10 =$	.40
U_4	$.14 \times 5$	.70
		2.87

$62.5 \times 9.75 = .61^k$

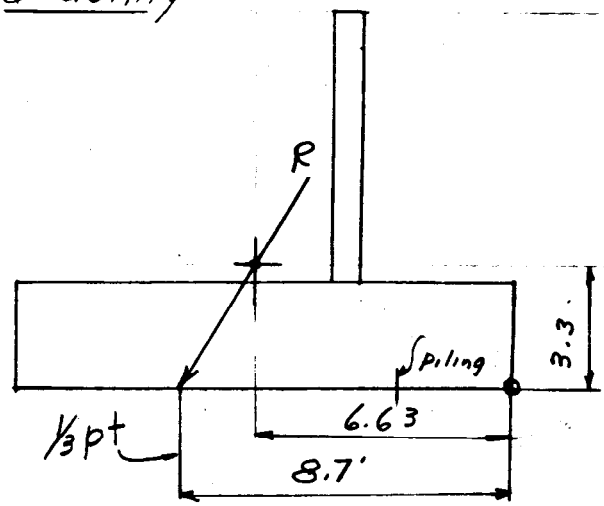
$.61 \times 9.75 \times \frac{1}{2} = 2.97^k$

$.02 \times 2.75 = .055$

Subject: Lowell Local Protection
Computation: Stop log section - Stability & Base Pressures
Computed by: V.H.K. Checked by: JWR Date: 3-4-41

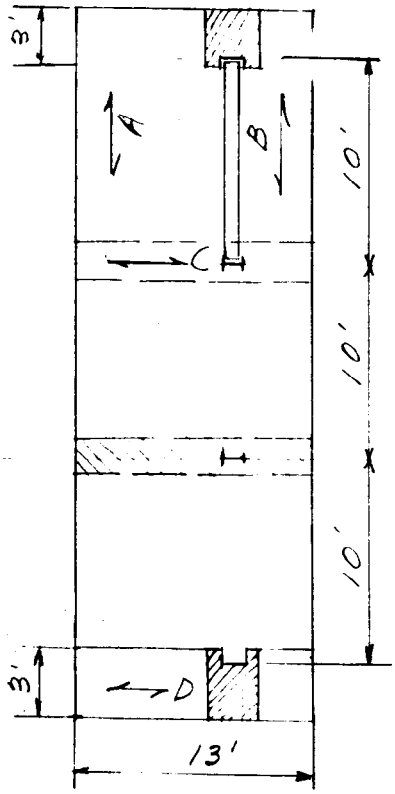
Stability

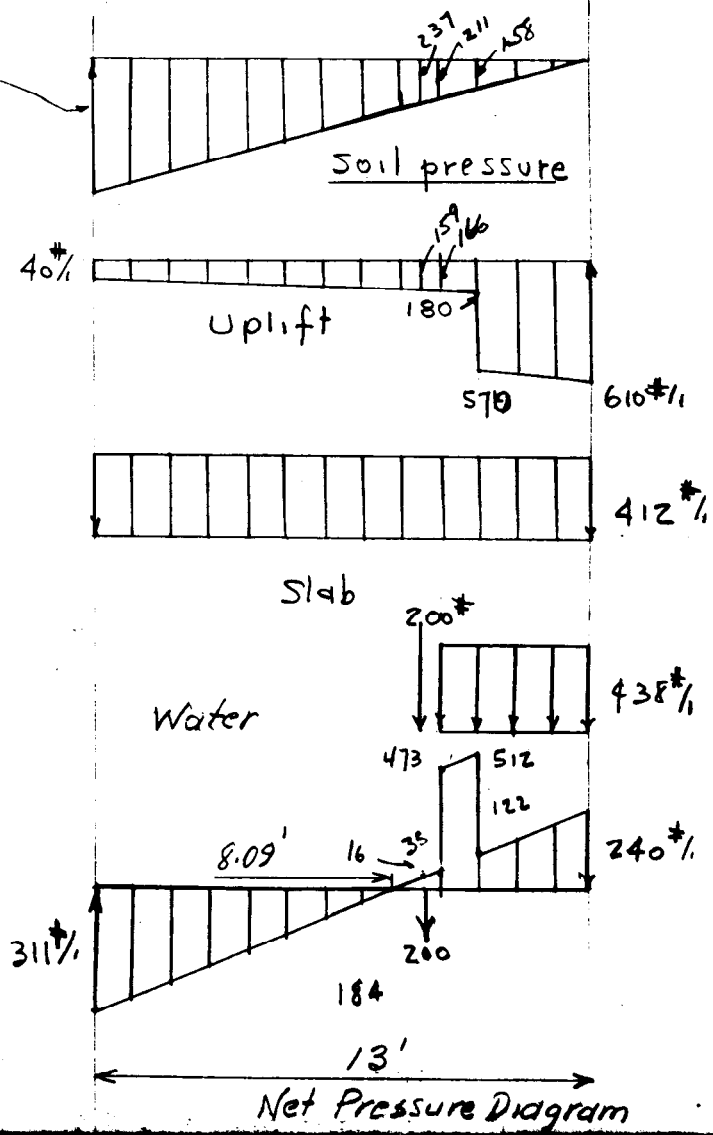
				F k	arm ft	ΣM 1k
W_1	7x4	28	62.5	1.75	2.0	3.5
W_2	13x2.75	35.8	150	5.37	6.5	35.0
				.20	4.5 (4.33)	.9
U_1	ΣV = 4.45 k			-1.71	1.5	-2.6
U_2				-.06	1.0	-.1
U_3				-.40	8.0	-3.2
U_4				-.70	6.33	-4.4
P_1	ΣH = 2.91			2.91	3.25	9.6
P_2				-.06	.92	-.1
	ΣM = 38.60					



$\frac{\Sigma M}{\Sigma V} = \frac{8.7'}{8.7'} = 1/3$ pt. OK

$f = \frac{2P}{A} = \frac{4.45 \times 2}{13} = .683\ ^{k}/_{\square} = 683\ ^{\#}/_{1}$

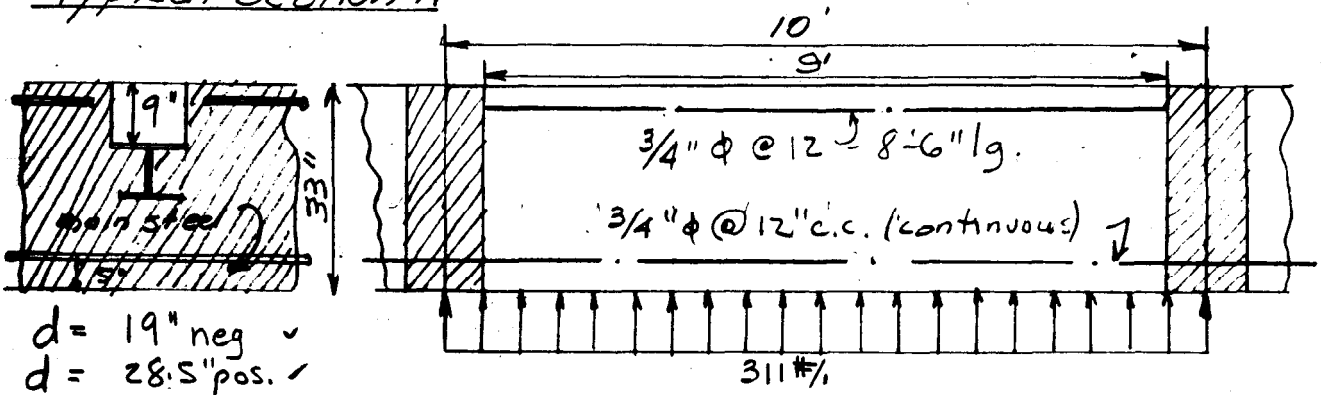




$311 \times 8.09 \times \frac{1}{2} = 1255$ ✓
$35 \times .91 \times \frac{1}{2} = 16$
$\frac{473+512}{2} = 200$
$\frac{122+240}{2} = 492$
543
1251 ✓

Subject: Lowell Local Protection
Computation: Stop Log Section
Computed by: V.H.K. Checked by: JWS Date: 3-4-41

Typical Section A



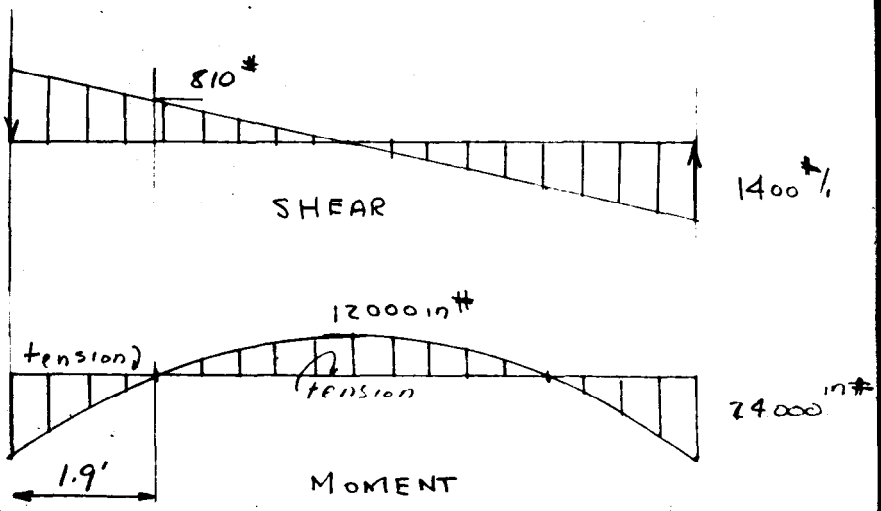
d = 19" neg ✓
d = 28.5" pos. ✓

V = 311 x 4.5 = 1400 #/' ✓

$M|_n = 311 \times \bar{9}^2 \times \frac{12}{12} = -24000$ in#

Mp = = 12000 in#

fs = 18000 #/□in ✓
fc = 1000 #/□" ✓
k = .4 v = 75 #/□" ✓
j = .867 u = 188 #/□" ✓
K = 173 ✓



$d = \sqrt{\frac{24000}{173 \times 12}} = 3.4$ in reqd. - no compression steel req'd

$d_v = \frac{1400}{12 \times .867 \times 75} = 1.8$ in shear O.K

$As_n = \frac{24000}{18000 \times .867 \times 19} = .081$ □in

$As_p = \frac{12000}{18000 \times .867 \times 28.5} = .027$ in

use 3/4" ϕ top + bottom @ 12" c.c

$\Sigma o_n = \frac{1400}{188 \times .867 \times 19} = .45$ in

$p = \frac{.88}{12 \times 30} = .00245$

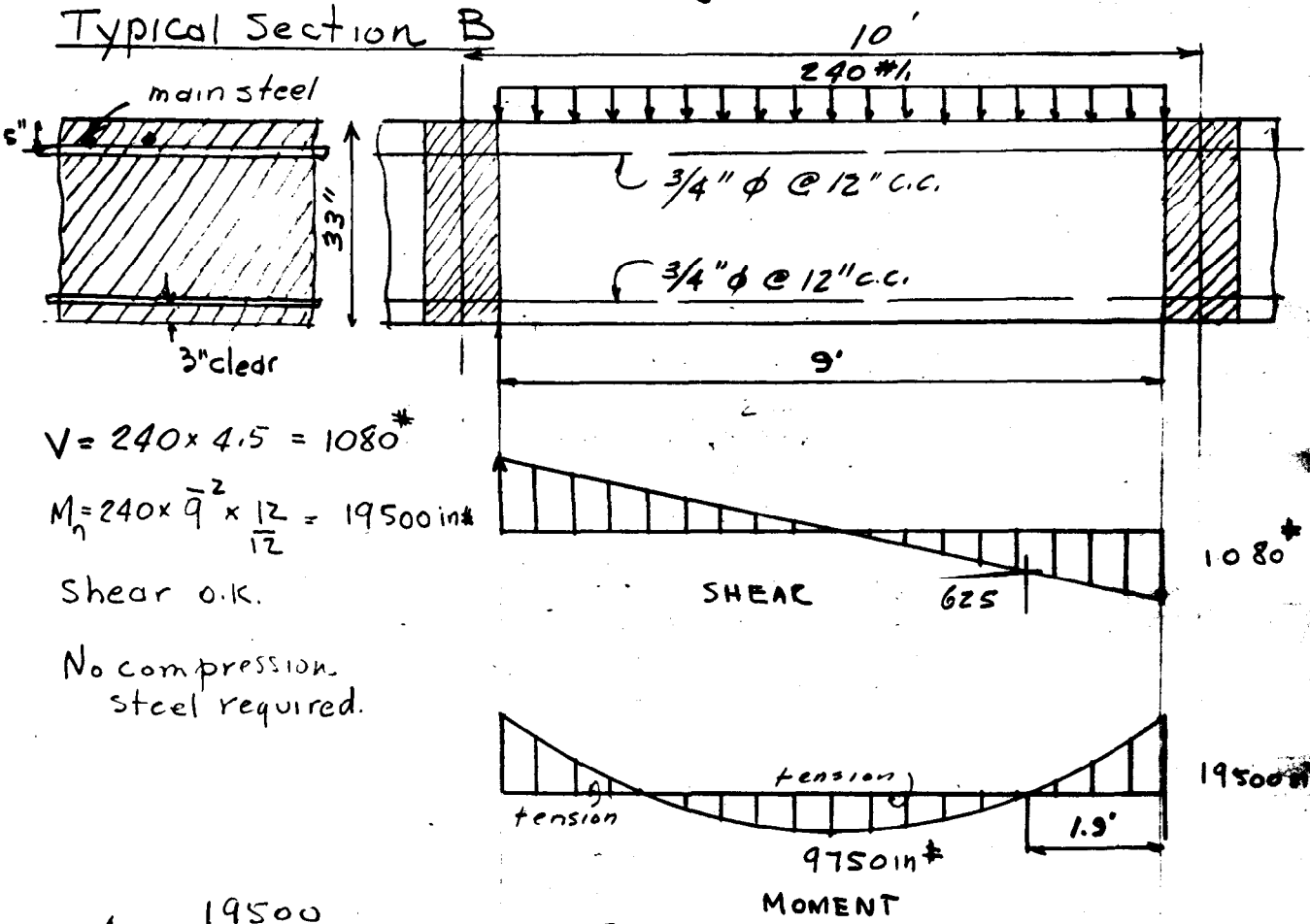
$\Sigma o_{p.s} = \frac{810}{188 \times .867 \times 19} = .26$ in

which is approx min required for shrink and temp. (.0025)

Slab is only partially confined at each end longitudinally

Subject: Lowell Local Protection
Computation: Stop log section
Computed by: V.H.R. Checked by: [illegible] Date: 3-7-41

Typical Section B



$V = 240 \times 4.5 = 1080^{\#}$

$M_n = 240 \times \overline{9}^2 \times \frac{12}{12} = 19500 \text{ in}^{\#}$

Shear o.k.

No compression steel required.

$$As_n = \frac{19500}{18000 \times .867 \times 28} = .045$$

$$As_p = \frac{9750}{18000 \times .867 \times 29.5} = .021$$

$$\Sigma o_n = \frac{1080}{188 \times .867 \times 28} = .237$$

$$\Sigma o_{pl} = \frac{625}{188 \times .867 \times 28} = .137$$

Use 3/4" ϕ ($A_s = .44$ $\Sigma o = 2.4$) top and bottom @ 12" c.c.

Subject: Lowell Local Protection
Computation: Stop log Section
Computed by: V. H. K. Checked by: [illegible] Date: 3-17-41

$7.75 \times 62.5 \times 10 = 4850$ #/'

$4850 \times \frac{1}{2} \times 7.75 = 18800$ #

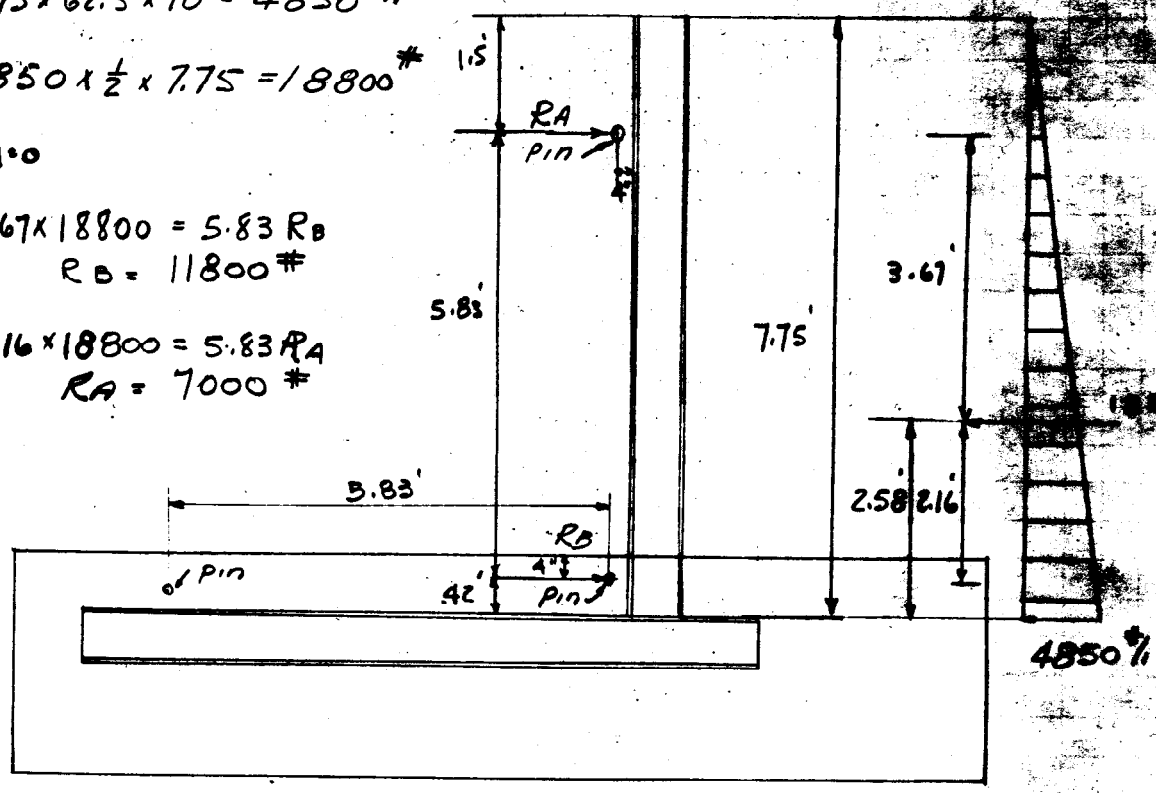
$\Sigma M = 0$

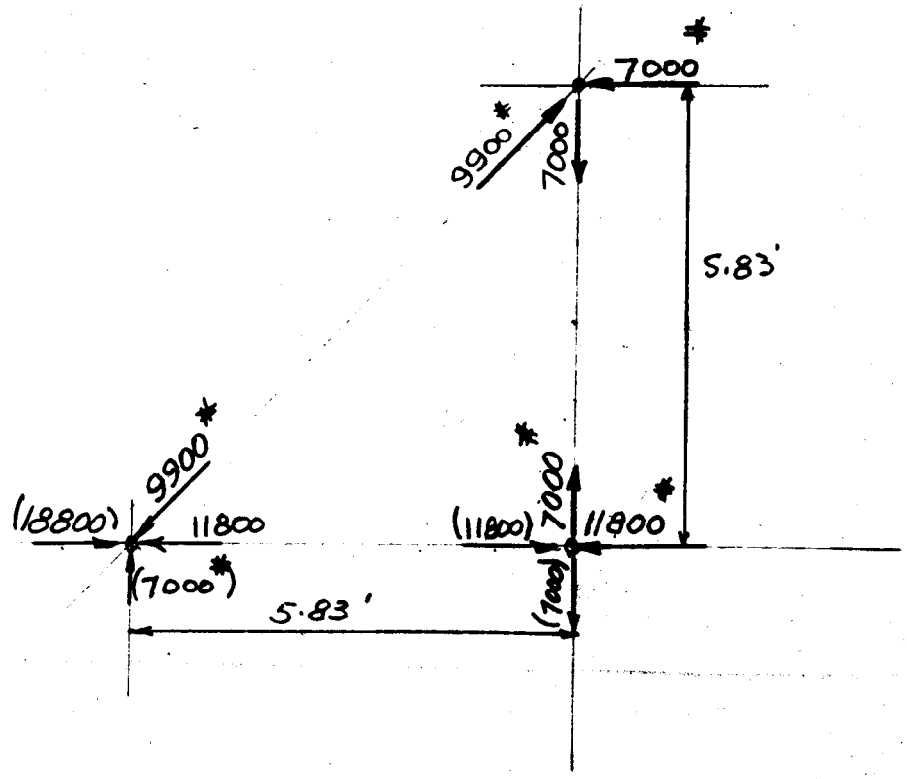
$3.67 \times 18800 = 5.83 R_B$

$R_B = 11800$ #

$2.16 \times 18800 = 5.83 R_A$

$R_A = 7000$ #





Subject: Lowell Local Protection
Computation: Stop Log Section
Computed by: V.H.K. Checked by: [illegible] Date: 3-17-41

Stop logs:-

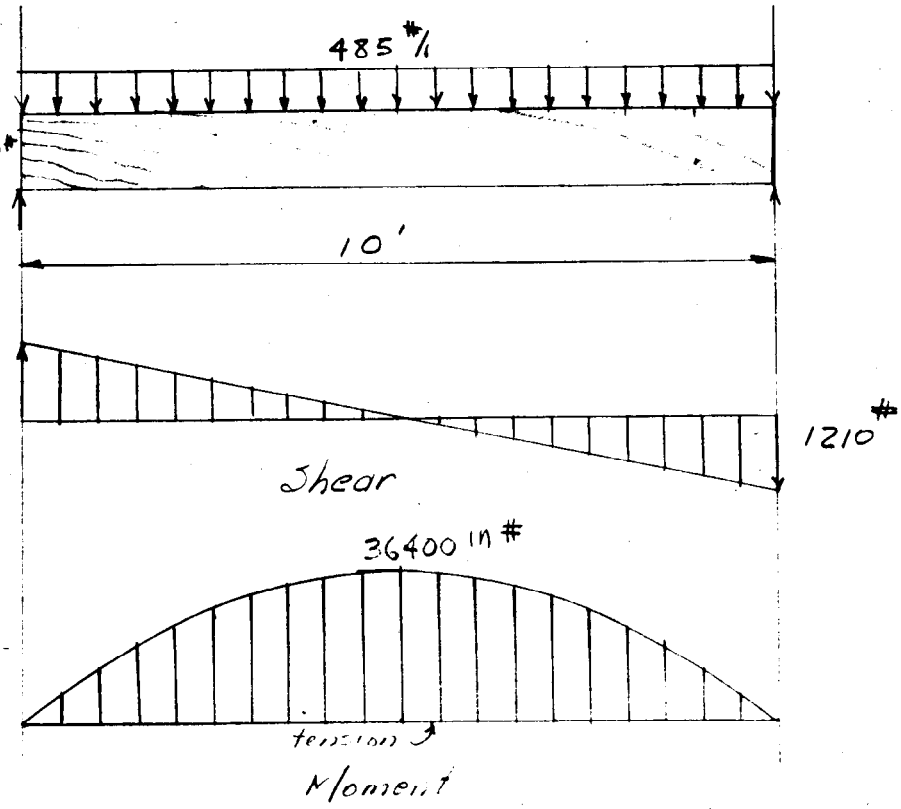
$62.5 \times 7.75 = 485\#/'$

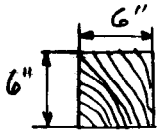
$M = \frac{485 \times (10)^2}{8} \times 12 = 72800$ in#

$V = 485 \times 5 = 2420\#$

$M = 36400$ in#/6" strip

$V = 1210\#/6"$ strip





Assume 6"x6" section

$I = \frac{(6)^4}{12} = 108 \text{ in}^4$

$Q = \frac{(6)^3}{8} = 27 \text{ in}^3$

$f = \frac{36400 \times 3}{108} = 1010\#/\square"$ (1200 allowed) OK.

$v = \frac{1210 \times 27}{6 \times 108} = 50\#/\square"$ (120 allowed) OK

$v_{max} = 75\#/\square"$

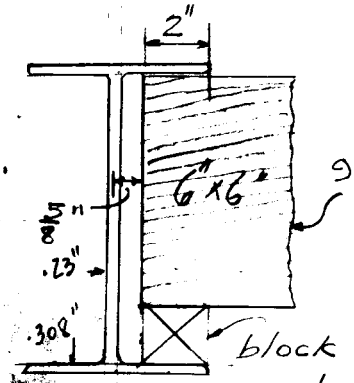
$y = \frac{5}{384}\frac{Wl^3}{EI} = \frac{5}{384} \times \frac{2420}{1,600,000} \times \frac{(120)^3}{108} = .315$ in.

$\frac{l}{400} = \frac{120}{400} = .30$ in

deflection O.K.

Deflection will be reduced by vertical rods

Bearing $\frac{1210}{6 \times 2} = 101\#/\square$ in O.K.



9'-10 3/4" long.

Approx. wght per beam

$\frac{1}{2} \times \frac{1}{2} \times 1 \times 60 \times 10 = 150\#$

A block is used for purposes of using alight stoplog giving case in manual transportation

B-81- 8 x 5 1/4 x 17#/'

Subject: Lowell Local Protection
Computation: Stop log Section
Computed by: V.H.K Checked by: JWL Date: 3-17-41

Vertical Member

$M = 16240'\#$
$P = 7000\#$

Try CB-81- $8'' \times 5\frac{1}{4}'' \times 17\#$

$S = 14.1 \quad r = 3.36 \quad c = 4'' \quad A = 5.0 \square in \quad I = 56.4$

$$f = \frac{P}{A} + \frac{Mc}{I} = \frac{7000}{5} + \frac{16240 \times 12 \times 4}{56.4}$$

$f = 1400 + 13900 = 15300\#/\square''$ tension O.K.

$f = \qquad = 12500\#/\square''$ comp. O.K.

$$f_{comp.} = \frac{18000}{1 + \frac{1}{18000}\left(\frac{L}{r}\right)^2} = 17500\#/\square in$$

$15000\#/\square in$ max

$\left(\frac{L}{r}\right) = \left(\frac{70}{3.36}\right) = 20$

least r
$\frac{L}{r} = 60$
$f = 15000\#/\square''$

Assuming shear taken only by web.

$$v = \frac{9820}{8 \times .23} = 5350\#/\square in \quad O.K.$$

Diagonal Member ~ 9900# direct stress compression

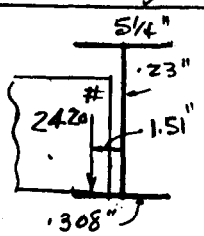
Try Light Beam CBLG $6'' \times 4'' - 12\#/'$ $A = 3.53 \square''$

$$\frac{P}{A} = \frac{9900}{3.53} = 2800\#/\square'' \text{ Comp. o.k.}$$

$$f_{comp.} = \frac{18000}{1.673} = 10750\#/\square''$$

(weight of member neglected)

least $r = .9$
$L = 5.83 \times 1.414 = 99''$
$\frac{L}{r} = \frac{99}{.9} = 110$
$\frac{(110)^2}{18000} = 0.673$



Per 6" strip

$M = 2420 \times 1.51 = 3650 \text{ in}\#$
$= 610 \text{ in}\#/in$

$V = 2420\#$
$= 403\#/in$ of strip

$I = \frac{6 \times (.308)^3}{12} = .0145$
$c = .154$

$$f = \frac{3650 \times .154}{.0145} = 38700\#/\square''$$

$$v = \frac{2420}{6 \times .308} = 1300\#/\square'' \text{ o.k.}$$

Use plates - stag. reduce & welded to

Subject: Lowell Local Protection
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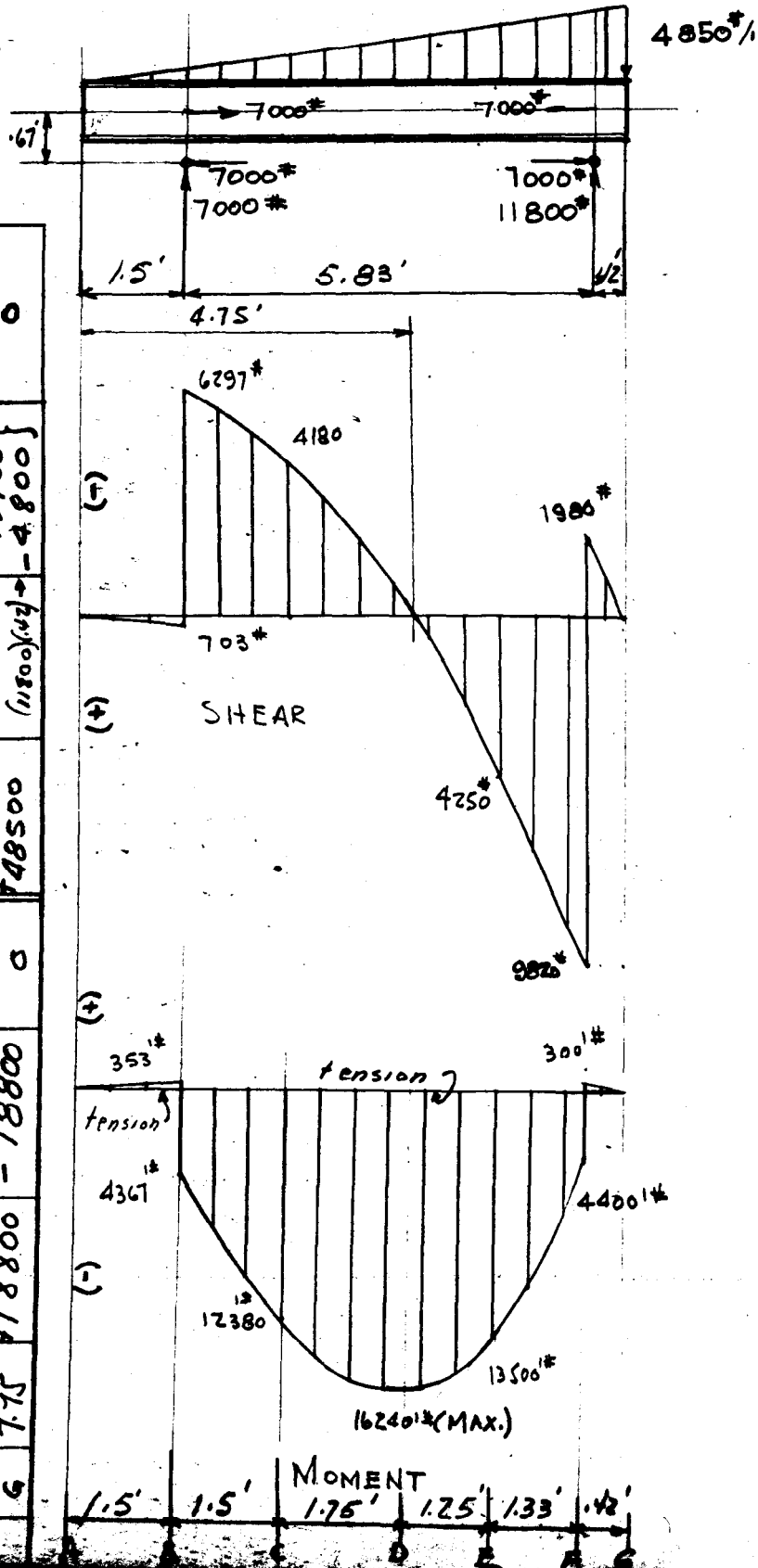
$$\frac{4850}{7.75} X \cdot \frac{X}{2} = 312.5 X^2$$

$$\frac{4850}{7.75} X \cdot \frac{X}{2} \cdot \frac{X}{3} = 104.1 X^3$$



	x	312.5X²	R	SHEAR #	104.1X³	7000x.67	7000(x-1.5)	Mom. #
B		+703		703	+353			+353
B'	1.5'	+703	-7000	-6297	+353	-4700		-4367
C	3.0'	+2820	-7000	-4180	+2820	-4700	-10500	-12380
D	4.75	+7000	-7000	0	+11160	-4700	-22,700	-16240
E	6.00	+11250	-7000	+4250	+22,700	-4700	-31,500	-13500
F		+16820	-7000	+9820	+41000	-4700	-40700	-4400
F'	7.33	+16820	-18800	-1980	+41000	0	-40700	+300
G	7.75	+18800	-18800	0	+48500	(11800)(.42)	-43700 / -4800	0