WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY

FOUNDATION INVESTIGATION; LOCK SITE NO. 2, PROPOSED PEARL RIVER IMPROVEMENT

Technical Memorandum No. 131-1

of the

U. S. WATERWAYS EXPERIMENT STATION

Vicksburg, Mississippi

December 1, 1938

M.R.C.12M.8-19-38

WAR DEPARTMENT
MISSISSIPPI RIVER COMMISSION
U. S. WATERWAYS EXPERIMENT STATION
P. O. BOX 80
VICKSBURG, MISSISSIPPI

IN REPLY REFER TO FILE
NO._____

December 1, 1938

Subject: Submittal of report on foundation investigation; Lock Site No. 2, Proposed Pearl River Improvement.

To: The District Engineer, U. S. Engineer Office, Mobile, Ala.

Herewith is Technical Memorandum No. 131-1, constituting a complete report on an investigation conducted at this Station of the foundation of Lock No. 2 of the Proposed Pearl River Improvement. Included in this report are tables and figures summarizing all data obtained during the investigation. The basic data themselves have been retained in the files of this Station. This report supersedes all previous reports made of the investigation in question.

Paul W. Thompson
1st Lieut., Corps of Engineers
Director

TABLE OF CONTENTS

FOUNDATION INVESTIGATION; LOCK SITE NO. 2

PROPOSED PEARL RIVER IMPROVEMENT

* * * * * * * * * * * * * * * * *

PART I: THE PROBLEM AND THE PURPOSE

Introductory

1. Introduction. This memorandum constitutes a report of an investigation of the foundation of Lock No. 2 of the proposed Pearl River Improvement Survey. The investigation was conducted in the Soil Mechanics Division of the U. S. Waterways Experiment Station for the Mobile Engineer District.

2. Authority. Authority for this investigation is contained in a 2nd Indorsement to a letter dated April 2, 1938, from the District Engineer, Mobile, Alabama, to the Chief of Engineers, Washington, D. C., Subject: "Special Laboratory Investigations".

The Problem

3. General. A major problem in connection with the design of important structures is the determination of the stability of the foundation medium considering various types of foundations. In recent years, it has become increasingly apparent that the successful solution of this problem requires a knowledge of the stress-strength characteristics of the foundation medium, together with an understanding of the stability requirements of the structure. Thus, a comprehensive investigation of the foundation was considered desirable in connection with the design of the locks for the proposed improvement of Pearl River in the Mobile Engineer District.

4. General Description of the Lock. The proposed lock under consideration in this investigation is to consist of three principal parts: the upper and lower gate bays, and the lock chamber. The lock chamber is to be formed by anchored sheet pile bulkheads which transmit little load to the foundation. The gate bays, however, are to consist of large concrete monoliths, which will support the gates and appurtenant accessories.

5. Specific. Differential settlements of the monoliths could prevent the proper operation and closure of the miter gates; hence, the stability of the foundation medium beneath them is of major importance. The specific problem of this investigation was the determination and comparison of the stability of the foundation medium for various types of foundations.

The Purpose

6. The purpose of this investigation was to solve the problem stated above, and thus to provide data from which a safe, economical foundation design could be made.

Personnel

7. The investigation described in this memorandum was conducted in the Soil Mechanics Division of the U. S. Waterways Experiment Station. Chief of this Division is Spencer J. Buchanan, Engineer, who supervised the execution of the study and edited this memorandum. Exploration was conducted by the field forces of the Mobile Engineer District, who were instructed in the technique of sampling by William J. Rowland, Surveyman, of this Station. Project Engineers for the investigation were: Otto H. Meyer, Assistant Engineer, who analyzed the stresses in the foundation medium; Joseph B. Eustis, Junior Engineer, who prepared the testing schedule; and Victor K. Wagner, Jr., Assistant Engineering Aide, who investigated the stability of the foundation medium, and prepared this memorandum. The tests made in connection with the project were supervised by Frederick A. Harris, Assistant Engineering Aide.

PART II: EXPLORATION, TESTS, AND THE FOUNDATION MEDIUM

Exploration

8. Preliminary. In order to determine the general nature of the subsoil and the extent of the detailed exploration required, the vicinity of the proposed site was first explored by preliminary borings, three inches in diameter. The locations of these borings are shown by Figure 1, and their logs by Figure 2. The samples obtained from these borings were forwarded to the U. S. Waterways Experiment Station where they were examined and tested. A report on the results of the preliminary exploration incorporating the data obtained was prepared and transmitted to the District Engineer in a letter from this Station dated January 21, 1938, Subject: "Report on Results of the Preliminary Exploration of the Subsoil in the Vicinity of the Proposed Lock No. 2 of the Pearl River Project". This report formed the basis for the detailed exploration, which was planned jointly by representatives of the Mobile Engineer District, and this Station.

9. Detailed. Upon completion of the preliminary work, the detailed exploration was undertaken, and borings were made as shown by Figure 1. The center of the lock chamber was tentatively assumed at Station 647+73.03, and both 3-inch and 6-inch borings were made north and south of this point on four ranges perpendicular to the centerline of the proposed canal. The distance between the ranges was approximately 294 feet, roughly the length of the lock chamber; thus, providing data for investigating the foundation conditions at three possible positions of the lock

along the centerline. General samples were obtained from the 3-inch borings, while both general and undisturbed samples were obtained from the 6-inch borings. The logs of these borings are shown by Figures 2-5, inclusive. All samples were forwarded to the U. S. Waterways Experiment Station for testing.

Tests

10. Tests Performed. The procedure followed in testing the samples has been described in a manual of this Station, "Laboratory Procedure in Testing Soils and Sediment". A copy of this manual is on file at the Mobile District Office, therefore, a description of the details of these tests is not included in this memorandum. An outline of the tests performed is as follows:

a. Visual classification: Samples from all borings.

b. Water content: Samples from all borings.

c. Mechanical analysis: Samples from Borings Nos. 1 to 5, and 11 to 18, inclusive, and from Borings Nos. 37 and 38.

d. Consolidation: Samples from Borings Nos. 15, 16, and 37.

e. Quick shear: Samples from Borings Nos. 15, 16, and 37.

11. Tests Results. The results of the mechanical analysis, visual classification and water content tests of all the samples have been incorporated in the logs of borings, Figures Nos. 2 to 5, inclusive. The mechanical analysis diagrams for samples from the 6-inch borings are shown by Figures Nos. 6 to 24, inclusive. The results of consolidation tests of undisturbed samples from Borings Nos. 15, 16, and 37, are shown by Figures Nos. 25 to 34-a, inclusive. The results of the quick shear tests of the undisturbed samples are given in condensed form in Table I, on the following page.

TABLE I: RESULTS OF QUICK SHEAR TESTS

Boring No.	Sample No.	Elevation of Sample	Shear Strength Tons per Sq. Ft.
15	19	-23.1	0.45
15	21	-36.7	0.47
15	24	-55.8	0.45
15	25	-61.1	0.45
16	16	-26.7	0.67
37	13	-20.9	0.65
37	14	-28.4	1.00
37	16	-40.4	1.15

The Foundation Medium

12. Description. As shown by Figures Nos. 1-5, inclusive, the foundation medium under the explored sites of the proposed lock consists of strata of cohesive material varying in thickness from about 12 to 60 feet, overlain by approximately 55 feet of sand and gravel and underlain by sand. The bed of cohesive material reaches its minimum thickness between Stations 649+19.7 and 652+13.03. Experience has shown that deposits of sand and gravel such as those existing at the site of the proposed structure are relatively incompressible. The consolidation of such materials takes place rapidly; hence, any settlement due to consolidation of this material will occur during the construction period. For this reason, only the strata of cohesive mate-

rials received special attention in this investigation. Examination of the samples of the cohesive materials, coupled with the results of the consolidation tests indicates that it has been consolidated at some time in its history to a degree in excess of that which could have been accomplished under the conditions existing at present. This advanced state of consolidation could have been caused either by the weight of a large overburden, or by drying caused by exposure to the atmosphere; probably the latter.

PART III: ANALYSIS OF STRESSES IN FOUNDATION MEDIUM

Basic Considerations

13. Spread Foundations. The basic problem of this investigation, as mentioned in Par. 5, was the determination and comparison of the stability of the foundation medium for various types of foundations. The most economical method of supporting the lock, would be on a spread foundation, and in the event that the stability requirements of the structure are complied with, this method may be the most desirable. Upon completion of the exploration and tests, it was apparent that conditions at the site are favorable for the use of this type of foundation, for as mentioned in Par. 12 above, the strata of cohesive material in which the majority of the consolidation would occur have been well consolidated in the past. In addition, advantage can be taken of the overlying strata of sand and gravel to distribute the load on the cohesive material below. Consequently, the first portion of the analysis was conducted considering the structure supported on a spread foundation.

14. Bearing Piles. The minimum amount of settlement will occur if the structure is supported on piles driven through the clay, since the load will then be transmitted directly to the underlying sands which are virtually incompressible. However, the highly consolidated nature of the clay may make penetration by the piles difficult, and it is conceivable that the piles may be driven to refusal without complete penetration. No means other than an actual test using a full scale pile is known by which the possible depth of penetration can be determined. Therefore,

it was considered advisable to compute the stress assuming bearing piles driven only to the top of the clay. For this assumed condition, the stress on the clay will be a maximum, and the settlement computed from this stress will be representative of the maximum that can occur when bearing piles are used.

15. Initial Data. The basic data from which the foundation stresses were computed were furnished by the District Engineer. These data are shown by the advance design sheets described as follows:

a. The general plan, showing the layout of the lock, canal and levees with the accompanying pertinent data regarding elevations of various features of the lock, levee, and canal.

b. The preliminary design sheets, showing the dimensions of the structures, unit weights of materials, and the forces applied to the structure under the various loading conditions assumed in the design.

16. Loading Conditions. Five conditions of loading were assumed by the Mobile District in the analysis of stresses in the lock structure. These conditions were as follows:

Case I - Normal Condition: Ordinary high water, gate sealed.

Case II - Normal Condition: Low water, gate sealed.

Case III - Unwatered Condition: Ordinary high water.

Case IV - Construction Condition: Saturation at Elev. 40.0 (ground water).

Case V - Extreme high water condition.

Of the above, Case V was considered for the upper gate bay only; the other conditions were considered for both bays. The analysis of the foundation stresses was made for all conditions.

Analysis of Stresses Considering Spread Foundations

17. Method of Analysis. The stress at any point in an elastic foundation, resulting from any system of superimposed loads may be computed by the theory of elasticity using the equation of Boussinesq. The development of the general equation is too detailed and complex to be included in this memorandum. However, various investigators have developed relatively simple solutions for certain special cases. One of these, which was used in this investigation, was developed by Nathan M. Newmark.* Newmark computed the stress at a unit depth below the corner of a rectangle subjected to a unit uniform load, for various dimensions of the rectangle, and tabulated these values in the publication mentioned in the footnote. For convenience, and ease of interpolation, the data obtained by Newmark are presented graphically by Figure 35. Obviously, for any depth other than unity, the scale may be modified to give a unit depth by dividing each side of the rectangle by the original value of the depth. The total stress at a point is the sum of the stresses resulting from all of the neighboring elements of load on the foundation.

18. Foundation Loading. The load on the foundation medium was computed from the unit weights of the material, the dimensions of the monoliths, the quantities of excavation and fill, and the various design loading conditions. In order to make possible the computation of stresses by Newmark's method, it was necessary to reduce all loadings to equivalent uniform loadings on rectangular areas. For this purpose, the actual applied

*"Simplified Computation of Vertical Pressures in Elastic Foundations", Circular No. 24, Engineering Experiment Station, University of Illinois.

loads were replaced by equivalent uniform loads acting on rectangular areas so that the total stress on the foundation medium would be the same for both cases.

19. Effect of Sheet Piling. The presence of the sheet pile cut-off walls at each of the monoliths will affect to some extent, the transmission of stress through the foundation medium. The proportion of incident stress transmitted to the medium on the far side of the sheet pile wall is a function of the stiffness of the piles, the slack in the interlocks, the modulus of elasticity of the soil, and probably other factors. No methods are known by which the amount of stress transmitted may be determined. However, it is believed that the presence of the sheet piling causes some concentration of the stress under the structure, and if the monoliths were completely surrounded by a perfectly rigid sheet pile wall extending down to the top of the clay, the entire weight of the monoliths would be concentrated on this stratum. This is the most adverse condition conceivable for the case of the spread foundation. The actual stress developed in the foundation medium will lie between that existing when the confining effect of the sheet piles is not considered and that existing when the weight of the structure is considered as fully concentrated on the compressible stratum.

20. Computation of Stresses. The change in stresses in the foundation medium for all loading conditions were computed at a number of points at two elevations (-15 and -40), approximately the top and bottom of the compressible stratum. The locations of the points are given by their rectangular coordinates using as the origin the intersection of the center-

line of the upstream pintles with the centerline of the lock chamber. The change in stress under the structure is given by Table II for the case in which the effect of the sheet piling was not considered. The change in stress on horizontal sections beneath the structure at Elevations -15 and -40, Case II loading, with the effect of the sheet piling neglected is shown by Figures 36 and 37.

TABLE II
CHANGE IN STRESS IN FOUNDATION MEDIUM FOR SPREAD FOUNDATION NEGLECTING EFFECT OF SHEET PILING

COORDINATES OF POINT	DESIGN ASSUMPTION - CASE									
	I		II		III		IV		V	
	NET STRESS CHANGE IN LBS. PER SQ. FT. AT ELEVATION									
	-15	-40	-15	-40	-15	-40	-15	-40	-15	-40
UPPER GATE BAY										
-53, 32.5	-64	-94	15	-41	138	13	110	5	-60	-95
-53,-32.5	-97	-305	-20	-252	103	-198	75	-206	-95	-306
-19, 32.5	346	236	580	330	846	403	824	415	326	231
-19,-32.5	278	184	512	278	778	351	756	363	258	179
-19, 63.5	1075	707	1165	768	1179	775	1241	802	1098	716
-19,-63.5	917	542	1007	[illegible]03	1021	610	1083	637	940	551
24, 32.5	203	264	250	[illegible]99	172	289	170	289	178	256
24,-32.5	-43	21	4	56	-74	46	-76	46	-68	13
24, 63.5	1457	1011	1449	1027	1262	966	1266	981	1501	1016
24,-63.5	979	597	971	613	784	552	788	567	1023	602
LOWER GATE BAY										
317.5, 32.5	-370	-283	-178	-147	-35	-133	139	-8		
317.5,-32.5	-555	-472	-363	-338	-220	-322	-46	-197		
317.5, 66.5	1120	617	1107	670	1011	652	1152	739		
317.5,-66.5	834	210	821	263	725	245	866	332		
362, 32.5	-896	-565	-110	-310	-444	-405	118	-166		
362,-32.5	-931	-619	-145	-364	-479	-459	83	-220		
391.5, 32.5	-1303	-1057	-1110	-969	-1349	-1051	-1015	-891		
391.5,-32.5	-1321	-1114	-1128	-1026	-1367	-1108	-1033	-948		
391.5, 66.5	-632	-594	-956	-650	-899	-650	-875	-574		
391.5,-66.5	-766	-761	-1090	-817	-1033	-817	-1009	-741		

Analysis of Stresses Considering Bearing Piles

21. Foundation Stresses. It is not possible to arrive at an exact estimate of the stresses existing in the foundation medium when the structure is supported on bearing piles driven to the top of the clay because of the virtual impossibility of ascertaining the proportion of the total load transmitted to the soil by skin friction along the piles. For this reason, it was assumed that the total load on the piles is transmitted to their points, and there applied to the foundation medium. This condition is analogous to the most adverse condition considered in Par. 19 above for a spread foundation, for in both cases the total load is assumed transmitted directly to the compressible stratum.

PART IV: STRENGTH OF THE FOUNDATION MEDIUM

Preliminary

22. Critical conditions. In determining the stability of a foundation medium, the possibility of two types of failures must be considered; first, failure caused by excessive settlement resulting from consolidation of the compressible strata in the foundation medium; and second, failure in shear, with the material flowing laterally from beneath the structure. Concerning the former type of failure, it may be mentioned that consolidation can take place only by the expulsion of water from the voids of the compressible strata, and hence, is a slow process requiring continuous application of load. Therefore, the critical condition for settlement is Case II, normal condition; low water-gate sealed. Although the high water periods of the Pearl River are of long duration, they will be excluded from the canal by the levees, and hence, will not be effective at the lock. Only floods in the tributaries acting as feeders to the canal above the lock will be effective, and they are of such short duration as to warrant no consideration. For the latter-mentioned type of failure, the loading condition producing the greatest lateral stress gradient will be critical. This was found to be Case I, the normal condition; ordinary high water, gate sealed.

Settlement

23. Theoretical Considerations. As indicated by Figures 36 and 37, upon completion of the lock, the net change in stress in some portions of the foundation medium is negative. This is due to the fact that the net

stress change is the algebraic sum of the positive stress exerted by the weight of the gate bays and fill, and the negative stress resulting from the excavation for the lock. In the construction of the lock, however, the excavation of the portion of the lock chamber and canal adjacent to the monoliths probably would be completed before the placement of concrete, and upon the completion of this excavation, the change in stress over the entire foundation medium would be negative. If the excavation were left open long enough, the underlying material would expand under the influence of this negative stress to a degree of consolidation corresponding to the reduced overburden. Upon application of the loads from the structure, the material would reconsolidate, as indicated by the void-ratio pressure diagram, to a degree corresponding to the final stress. The exact relation between the time of rebound observed in the test and the time of rebound in nature is as yet imperfectly understood, and therefore, the proportion of the theoretical rebound which will take place during construction is not known. It was believed advisable; however, to adopt the conservative assumption that all rebound is completed before the application of loads produced by the structure; This is the more unfavorable situation, and the effect of interruption of construction operations by high water is provided for. The magnitude of the settlement of a structure resulting from consolidation of the compressible material in the foundation medium is computed by the formula:

$$\text{Settlement} = \frac{e_1 - e_2}{1 + e_1} t$$

in which e_1 is the void-ratio prior to consolidation, e_2, the void-ratio

after consolidation, and t, is the thickness of the stratum in question. The total settlement is the sum of the increments resulting from the consolidation of the compressible strata.

24. Analysis of Settlement. The settlement of the structure was estimated considering both the spread and pile foundations, with the lock in two of the three positions mentioned in Par. 9. The first location considered was with the center of the lock chamber at Station 647+73.03. In this position, the foundation media under the upper and lower gate bays is represented in cross sections by the logs of Borings Nos. 16 and 15, respectively. Since the thickness of the cohesive material at Boring No. 15 is about 42 feet greater than at Boring No. 16, a second position of the lock was considered with the center of the chamber at Station 650+66.4. At this location, the foundation media under the upper and lower gate bays is represented in cross sections by the logs of Borings Nos. 16 and 37, respectively. A third position of the lock was considered with the center of the chamber at Station 644+79.69, but as may be noted by reference to Figures 2 to 5, inclusive, the thickness of the compressible strata at this position is greater than at the two previously mentioned. Therefore, the settlement obviously would be greater so that further analysis was not made. The results of the settlement analysis are given by Table III on the following page, in which the coordinates shown refer to the x and y distances from an origin taken at the intersection of the centerline of the upstream pintles with the centerline of the lock chamber.

TABLE III
RESULTS OF SETTLEMENT ANALYSIS
ALL SETTLEMENTS IN FEET

Center of Lock Chamber of Station 647+73.03

Coordinates of Point	Ultimate Settlement for Spread Foundation. Effect of Sheet Piling Neglected.	Ultimate Settlement for Bearing Piles Driven to Top of Clay, and for Spread Foundation Assuming Full Confinement of Stress by Sheet Piling
	Upper Bay	
-53, 32.5	0.01	0.07
-53,-32.5	0.01	0.07
-19, 32.5	0.02	0.05
-19,-32.5	0.02	0.05
-19, 63.5	0.05	0.08
-19,-63.5	0.04	0.07
24, 32.5	0.02	0.12
24,-32.5	0.02	0.12
24, 63.5	0.06	0.16
24,-63.5	0.04	0.14
	Lower Bay	
317.5, 32.5	0.02	0.13
317.5,-32.5	0.02	0.13
317.5, 66.5	0.04	0.19
317.5,-66.5	0.03	0.19
362, 32.5	0.02	0.43
362,-32.5	0.03	0.43
391.5, 32.5	0.01	0.13
391.5,-32.5	0.01	0.13
391.5, 66.5	0.01	0.01
391.5,-66.5	0.01	0.01

(Cont'd.)

TABLE III
(Cont'd.)

RESULTS OF SETTLEMENT ANALYSIS
ALL SETTLEMENT IN FEET

Center of Lock Chamber at Station 650+66.40

Coordinates of Point	Ultimate Settlement for Spread Foundation. Effect of Sheet Piling Neglected.	Ultimate Settlement for Bearing Piles Driven to Top of Clay, and for Spread Foundation Assuming Full Confinement of Stress by Sheet Piling.
	Upper Bay	
-53, 32.5	0.01	0.07
-53,-32.5	0.01	0.07
-19, 32.5	0.02	0.05
-19,-32.5	0.02	0.05
-19, 63.5	0.05	0.08
-19,-63.5	0.03	0.08
24, 32.5	0.02	0.11
24,-32.5	0.02	0.11
24, 63.5	0.05	0.16
24,-63.5	0.04	0.14
	Lower Bay	
317.5, 32.5	0.02	0.07
317.5,-32.5	0.02	0.07
317.5, 66.5	0.03	0.10
317.5,-66.5	0.01	0.09
362, 32.5	0.03	0.16
362,-32.5	0.03	0.16
391.5, 32.5	0.01	0.07
391.5,-32.5	0.01	0.07
391.5, 66.5	0.01	0.01
391.5,-66.5	0.01	0.01

In interpreting the values of settlement given in Table III, it should be borne in mind that the values given for the case in which bearing piles are used may be exceeded, due to the disturbance of the compressible material by the driving operation. This fact has been given consideration by several investigators,* but an estimate of the amount of this disturbance, and its effect toward increasing the settlement was not considered practicable.

25. Time of Settlement. The time required for the consolidation of a stratum of material in nature may be ascertained from the results of a consolidation test of a specimen of the material. The ratio of the time scale in nature to the time scale in the test is equal to the ratio between the square of the thickness of the stratum in question to the square of the thickness of the test specimen. The time-settlement curves for the upper and lower bays at each position of the lock are shown by Figures 38 to 40, inclusive. It should be noted that Figure 39 represents the time-settlement relationship for the upper bay when the center of the lock chamber is at Station 647+73.03, and for the lower bay when the center of the lock is at Station 650+66.40.

Lateral Flow

26. Security against failure in shear. Shear failures of foundation media are characterized by lateral flow of the material from beneath the structure. This flow results from excessive shearing stresses caused by superimposed loads. The tendency toward lateral flow existing in any

*See for instance, "The Structure of Clay and its Importance in Foundation Engineering", by Arthur Casagrande, Journal of the Boston Society of Civil Engineers, Vol. XIX, No. 4, April, 1932.

stratum of material may be expressed in terms of "active lateral pressure", which is the magnitude of the resultant force obtained by subtracting the shearing resistance of the material from the shearing force created by the loads. Naturally, when the shearing stress created by the loads becomes less than the shearing resistance of the material, no active pressure is produced, and instead the material offers a "passive resistance" to the active pressure transmitted to it by adjoining material. Before lateral flow can take place, the total active lateral pressure due to the applied load and the overburden must be greater than the passive resistance of the overburden, coupled with the passive resistance caused by the shearing strength of the soil in which no active pressure is created. The total passive resistance of the overburden depends on its shear strength and weight, since during flow the entire mass of soil above the critical stratum must be lifted. While lateral flow of cohesionless material is possible, loads much higher than those existing under the lock are required to produce this condition and hence, the stability of the cohesive material only was determined in this investigation. Preliminary computations on basis of the considerations outlined above indicated that the maximum active force exerted by the cohesive material is approximately 60 pounds per square foot, while the passive resistance of the overburden is about 3500 pounds per square foot. It can be seen that no possibility of foundation failure by lateral flow exists. The above-mentioned computations neglected the effect of the sheet pile cut-off walls which would confine a large portion of the material even if a tendency toward lateral flow did exist.

PART V: CONCLUSIONS AND DISCUSSION OF RESULTS

27. Strength of foundation medium. The strength of the foundation medium has been ascertained for the two locations of the lock, considering the two critical conditions; namely, failure by excessive settlement resulting from extensive consolidation of the compressible strata, or failure in shear resulting in lateral flow of this material from beneath the structure. The limits within which this strength has been ascertained are given in the body of the report. Conclusions regarding the two critical conditions are summarized in the following subparagraphs.

a. Settlement to be experienced considering spread foundation. The settlement to be experienced by the lock has been estimated for each of the two locations given in Par. 24 and the results are shown in Table III. The results, as explained in Pars. 19 and 21, are based on the following assumptions:

(1) No confinement of the stress in the portion of the foundation medium, which overlies the compressible materials, will be effected by the sheet piles under the monoliths. (Results in Column 2 of Table III.)

(2) Full confinement of the stress by the sheet piles under the monoliths, which provides for no distribution of this stress by the material overlying the compressible portion of the foundation medium. This condition is analogous to that for bearing piles driven only to the top of the compressible strata described above. (Results in Column 3 of Table III.)

No method of analysis is known to be available by which it is possible to narrow the limits established by the assumptions given above. However, consideration of several features of sheet piles which, it is believed, affect their restraining ability are worthy of mention here. The sheet piles will be approximately 40 ft. long (base of monoliths, El. +13 or +15; top of impervious strata in which seal will be made, El. -15, and assuming a 10-ft. penetration); hence, they cannot be expected to be perfectly rigid as assumed in (2) above. The

slack in the joints of the sheet piles, estimated to be 1/8 to 1/4 inch, is believed to be sufficient to allow sufficient strain to develop for the transmission of stress. Further, the materials lying between the base of the monoliths and the compressible strata are sand and gravel, both of which are relatively incompressible, and thus only a slight lateral deformation of the sheet piling would be required in order for the stress to be distributed through this medium in a normal manner. Consideration of the features outlined above indicates that it may be concluded that the proposed lock structure will experience settlements closely approximating those given in Column 1 of Table III in the event spread foundations are used.

b. Settlement to be experienced considering use of bearing piles. The estimate of the settlement to be experienced by the proposed lock at either of the two locations given in Par. 24 considering a bearing pile foundation is given for two conditions; namely, piles driven only to the top of the compressible strata; and piles driven completely through those materials into the deep, underlying sand. For the first of the above situations (piles to top of clay), the maximum settlement, which it is believed will occur, is that shown in Column 3 of Table III. For the second situation, (piles through the clay), it is believed there will be no settlement.

c. Stability of foundation medium insofar as shear strength is concerned. It may be concluded from the information shown in Par. 27 that the foundation medium, considering the center of the lock at either Station 647+73.03 or Station 650+66.40, is exceedingly stable, and that no danger of failure by lateral flow exists.

28. Location of the lock. It is believed that the proposed lock may best be located with the center of the chamber at Station 650+66.40. In this position, the thickness of the strata of clay under the upper and lower gate bays would be less than at any of the other sites explored, and consequently the probable settlement would be less. In this position, there is ample cohesive material to form an impervious seal for the bottom of the sheet pile cut-off walls.

END OF MEMORANDUM PROPER
(Figures follow)

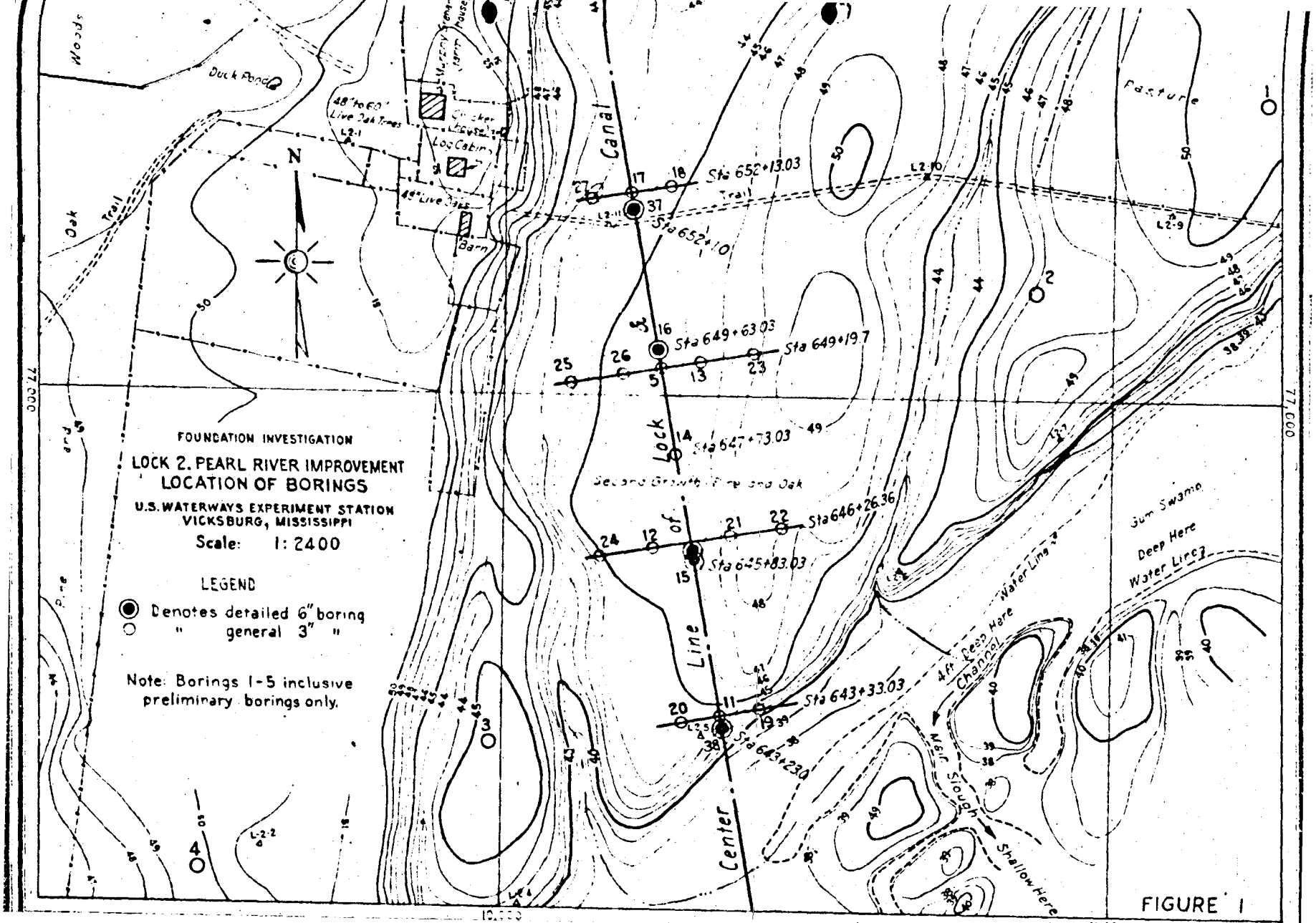
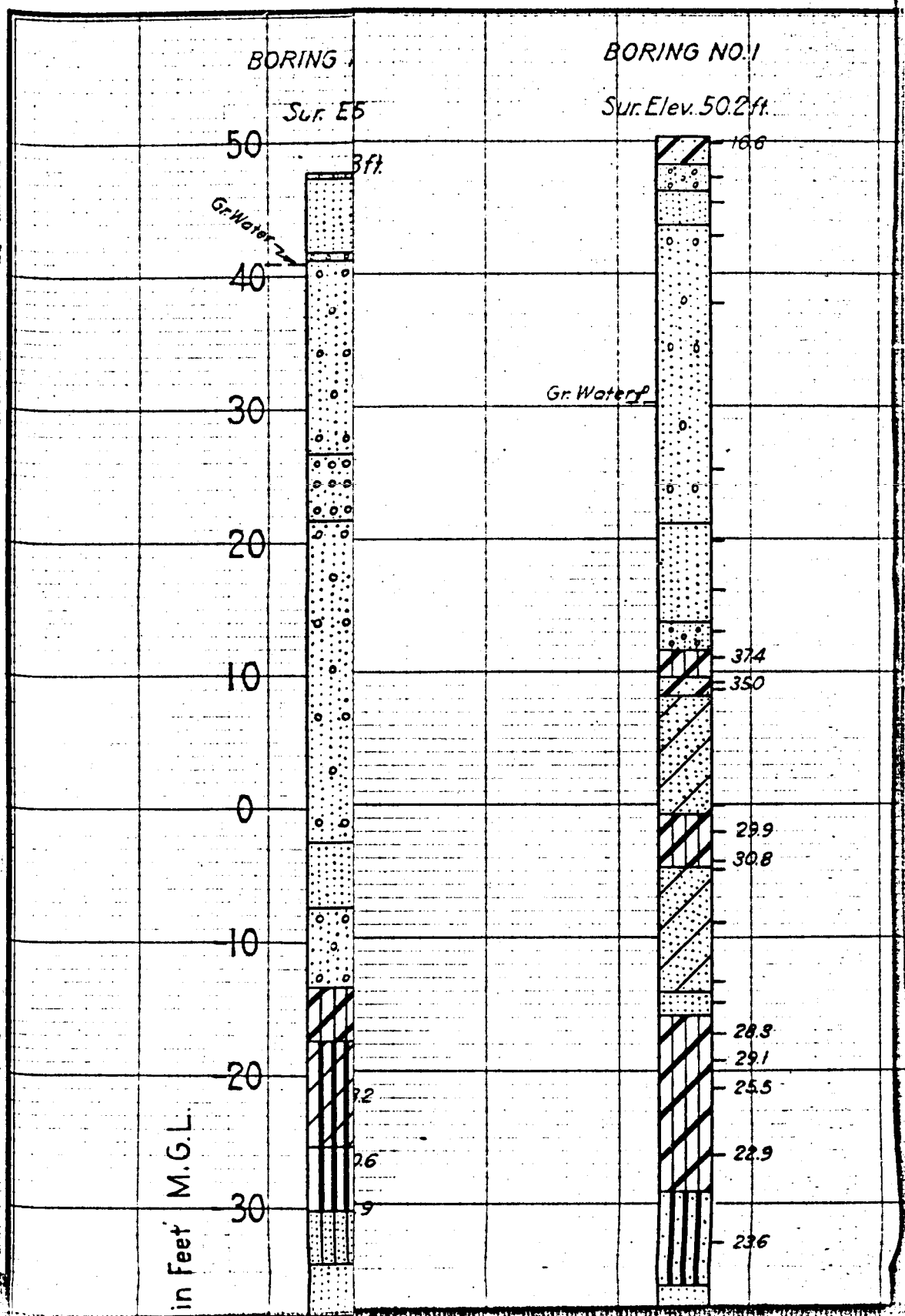
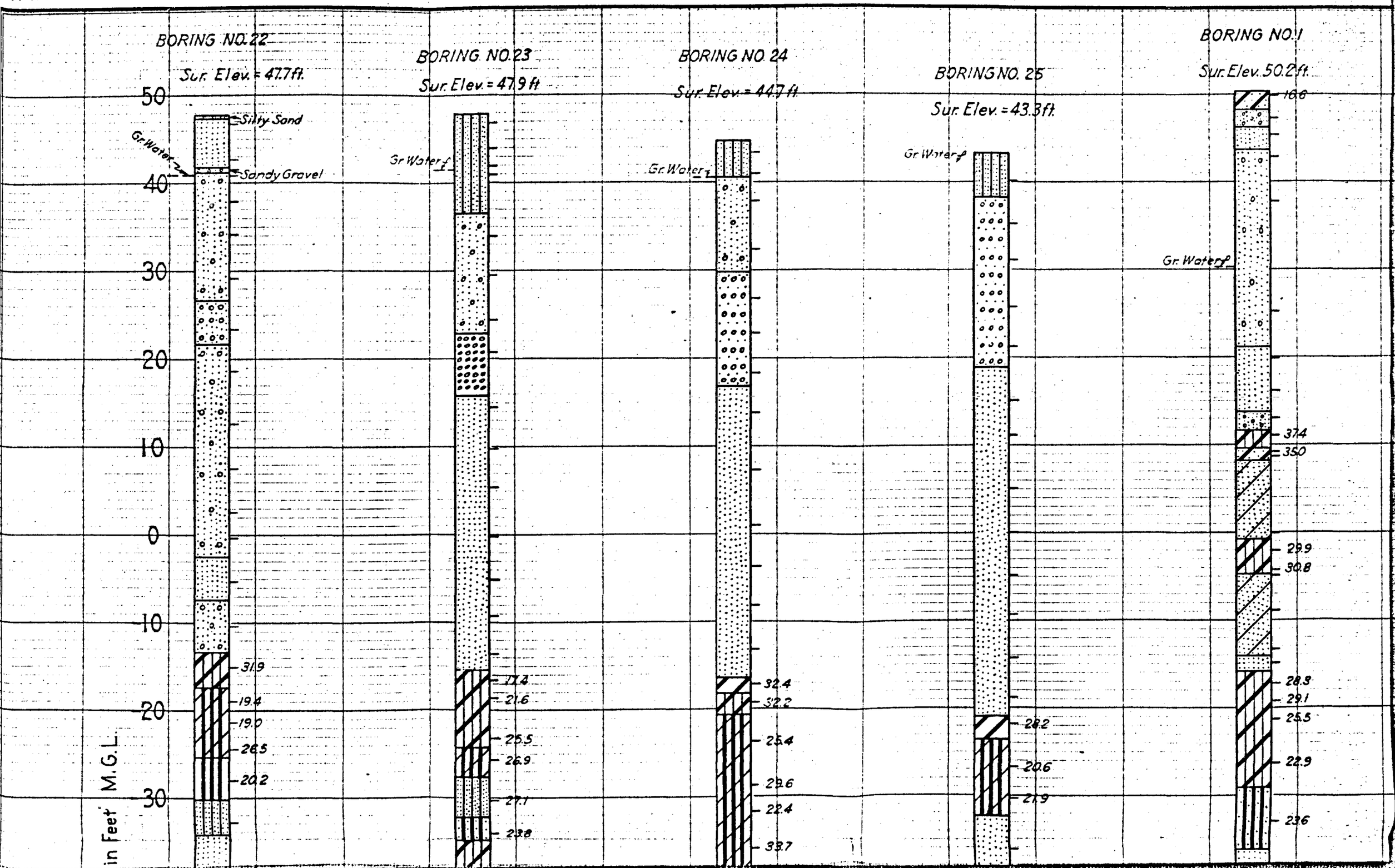

FOUNDATION INVESTIGATION
LOCK 2, PEARL RIVER IMPROVEMENT
LOCATION OF BORINGS
U.S. WATERWAYS EXPERIMENT STATION
VICKSBURG, MISSISSIPPI
Scale: 1: 2400
LEGEND
Denotes detailed 6" boring
" general 3" "
Note: Borings 1-5 inclusive preliminary borings only.
N
Canal
Lock
Center
Line
Sta 652+13.03
Sta 652+10
Sta 649+63.03
Sta 649+19.7
Sta 647+73.03
Sta 646+26.36
Sta 645+83.03
Sta 643+33.03
Sta 643+23.0
Duck Pond
Woods
Oak
Trail
Pine
Pasture
Barn
Log Cabin
Cracker House
Nursery Greenhouse
48" to 60" Live Oak Trees
48" Live Oak
Second Growth Pine and Oak
Gum Swamp
Deep Here
Water Line
4ft Deep Here
Channel
Near Slough
Shallow Here
77,000
10,000

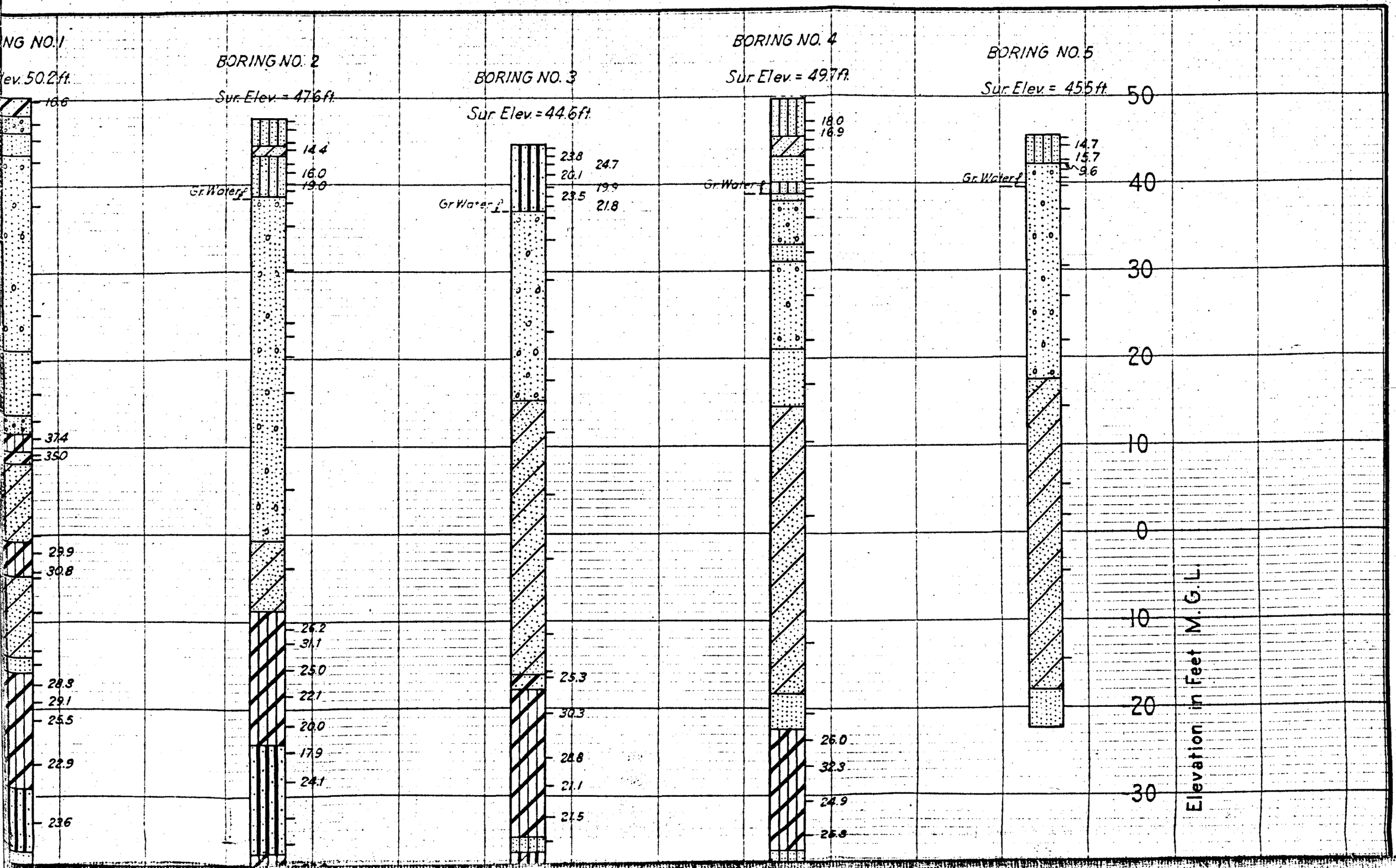
FIGURE 1

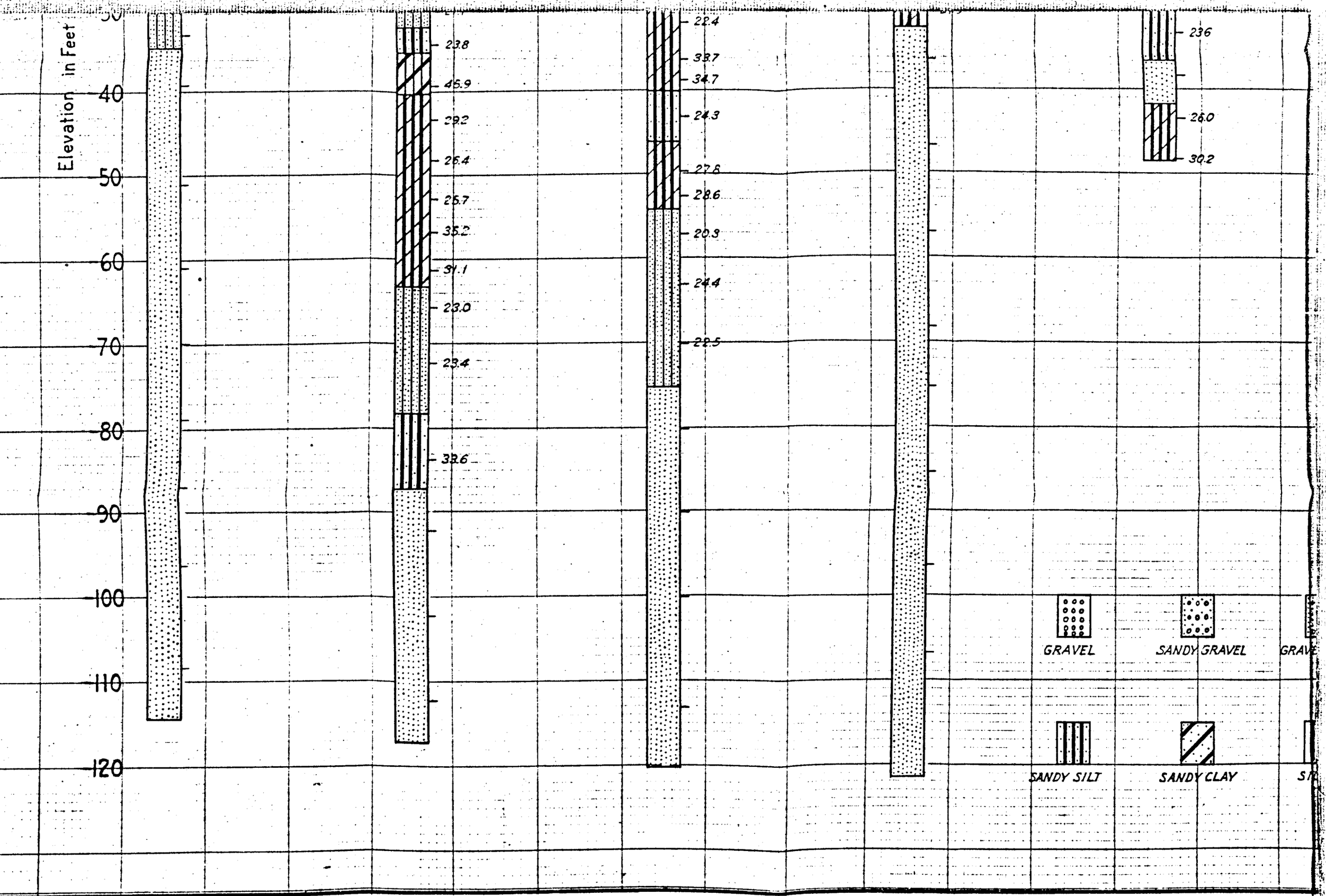
WAR DEPARTMENT

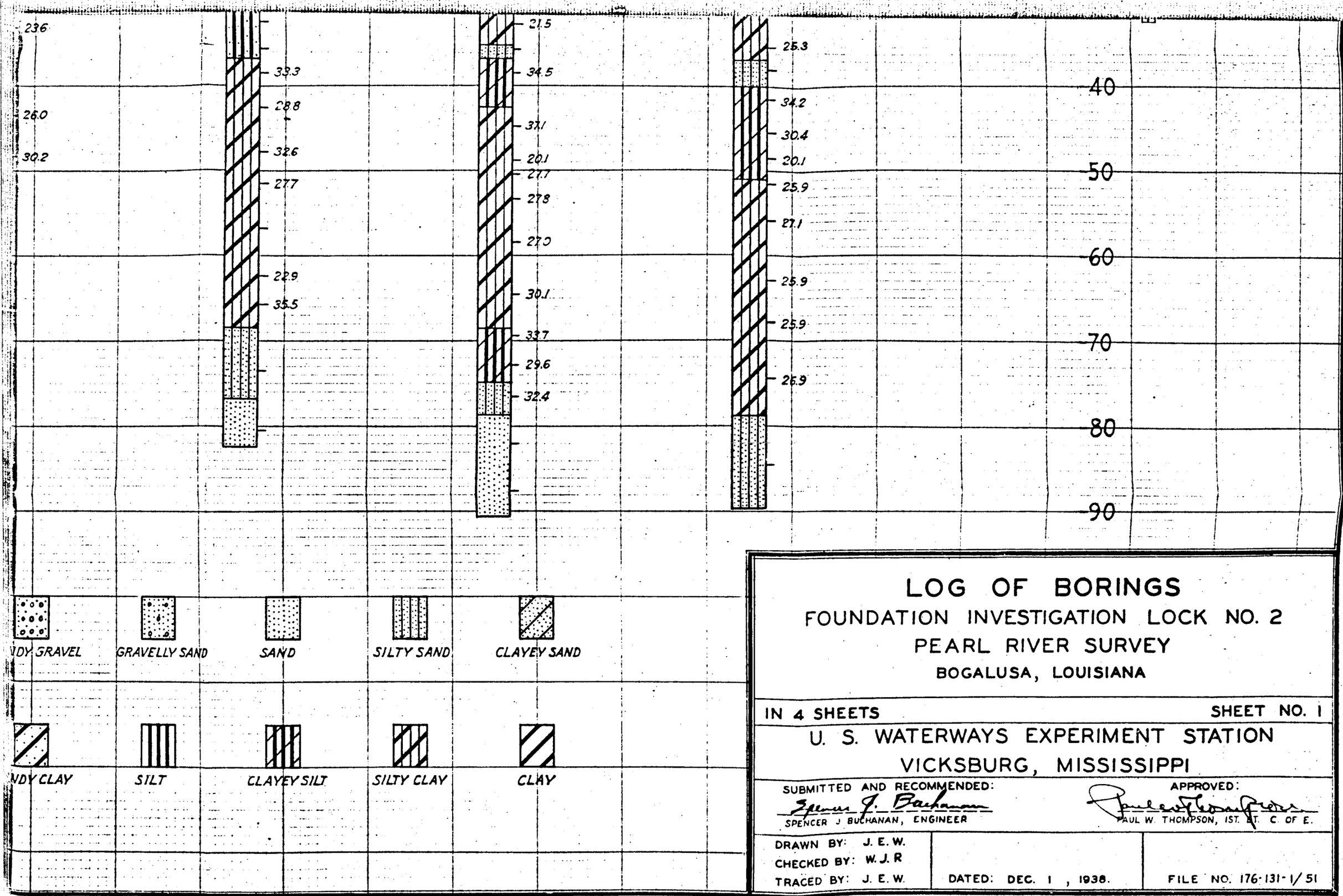


WAR DEPARTMENT




NG NO. 1
ev. 50.2 ft.
16.6
37.4
35.0
29.9
30.8
28.3
29.1
25.5
22.9
23.6
BORING NO. 2
Sur. Elev. = 47.6 ft.
14.4
16.0
19.0
Gr. Water
26.2
31.1
25.0
22.1
20.0
17.9
24.1
BORING NO. 3
Sur. Elev. = 44.6 ft.
23.8
20.1
23.5
24.7
19.9
21.8
Gr. Water
25.3
30.3
28.8
21.1
21.5
BORING NO. 4
Sur. Elev. = 49.7 ft.
18.0
16.9
Gr. Water
26.0
32.3
24.9
25.8
BORING NO. 5
Sur. Elev. = 45.5 ft.
14.7
15.7
9.6
Gr. Water
50
40
30
20
10
0
10
20
30
Elevation in Feet M.G.L.


Elevation in Feet
30
40
50
60
70
80
90
100
110
120
23.8
45.9
29.2
26.4
25.7
35.2
31.1
23.0
23.4
33.6
22.4
33.7
34.7
24.3
27.8
28.6
20.3
24.4
22.5
23.6
26.0
30.2
GRAVEL
SANDY GRAVEL
SANDY SILT
SANDY CLAY



LOG OF BORINGS

FOUNDATION INVESTIGATION LOCK NO. 2
PEARL RIVER SURVEY
BOGALUSA, LOUISIANA

IN 4 SHEETS — SHEET NO. 1

U. S. WATERWAYS EXPERIMENT STATION
VICKSBURG, MISSISSIPPI

SUBMITTED AND RECOMMENDED:
Spencer J. Buchanan
SPENCER J. BUCHANAN, ENGINEER

APPROVED:
Paul W. Thompson
PAUL W. THOMPSON, 1ST. LT. C. OF E.

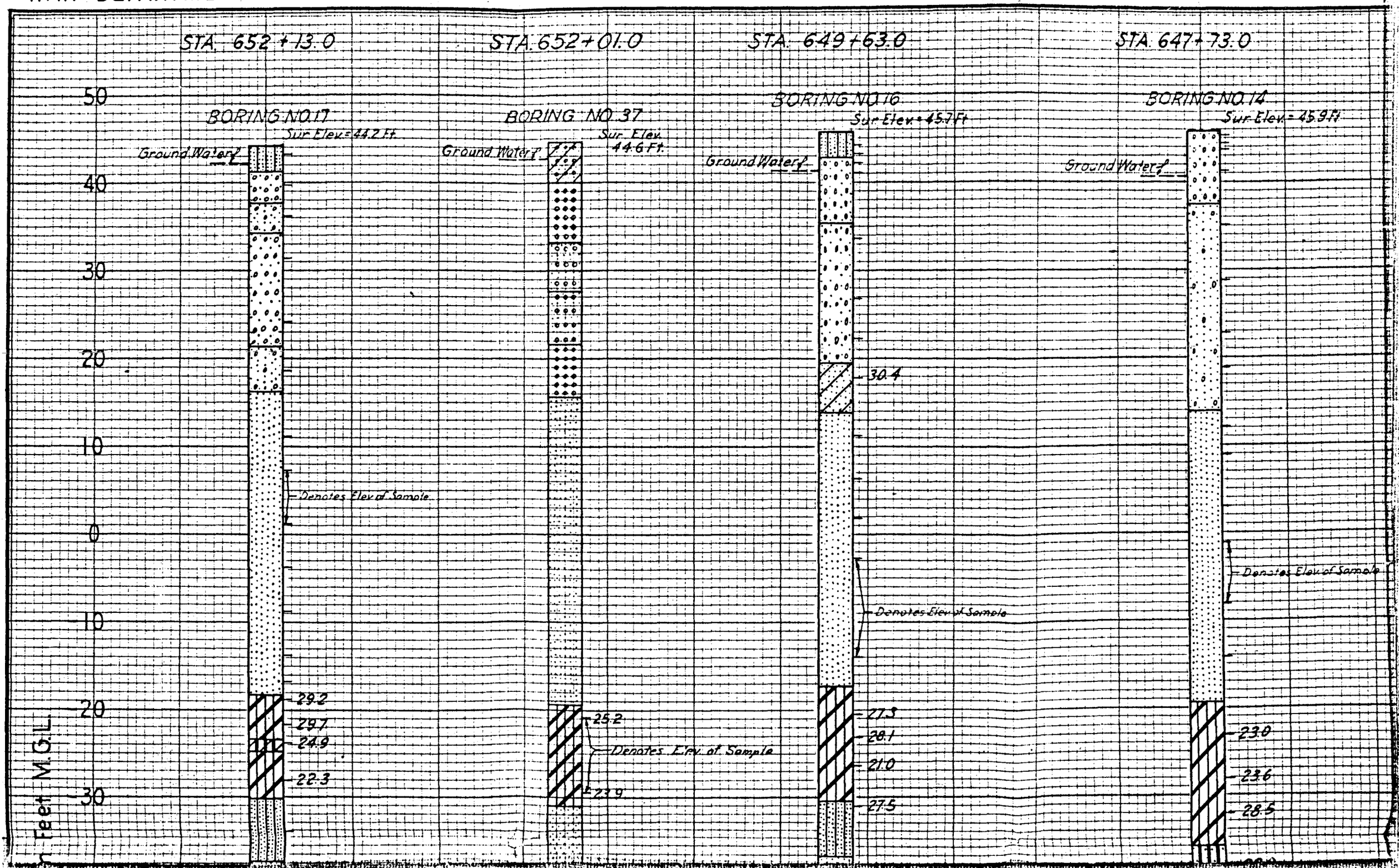
DRAWN BY: J. E. W.
CHECKED BY: W. J. R
TRACED BY: J. E. W.

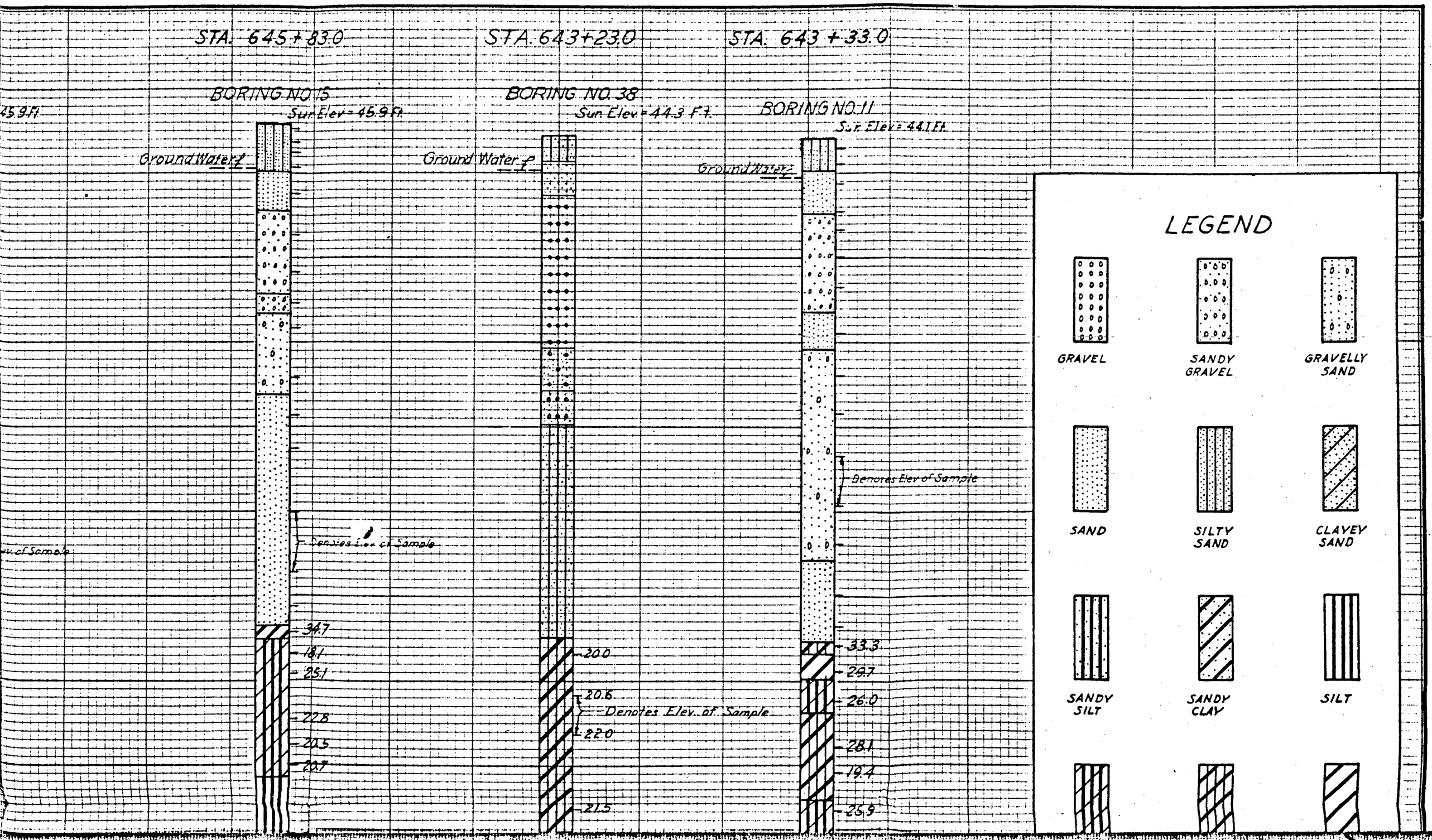
DATED: DEC. 1, 1938.

FILE NO. 176-131-1/51

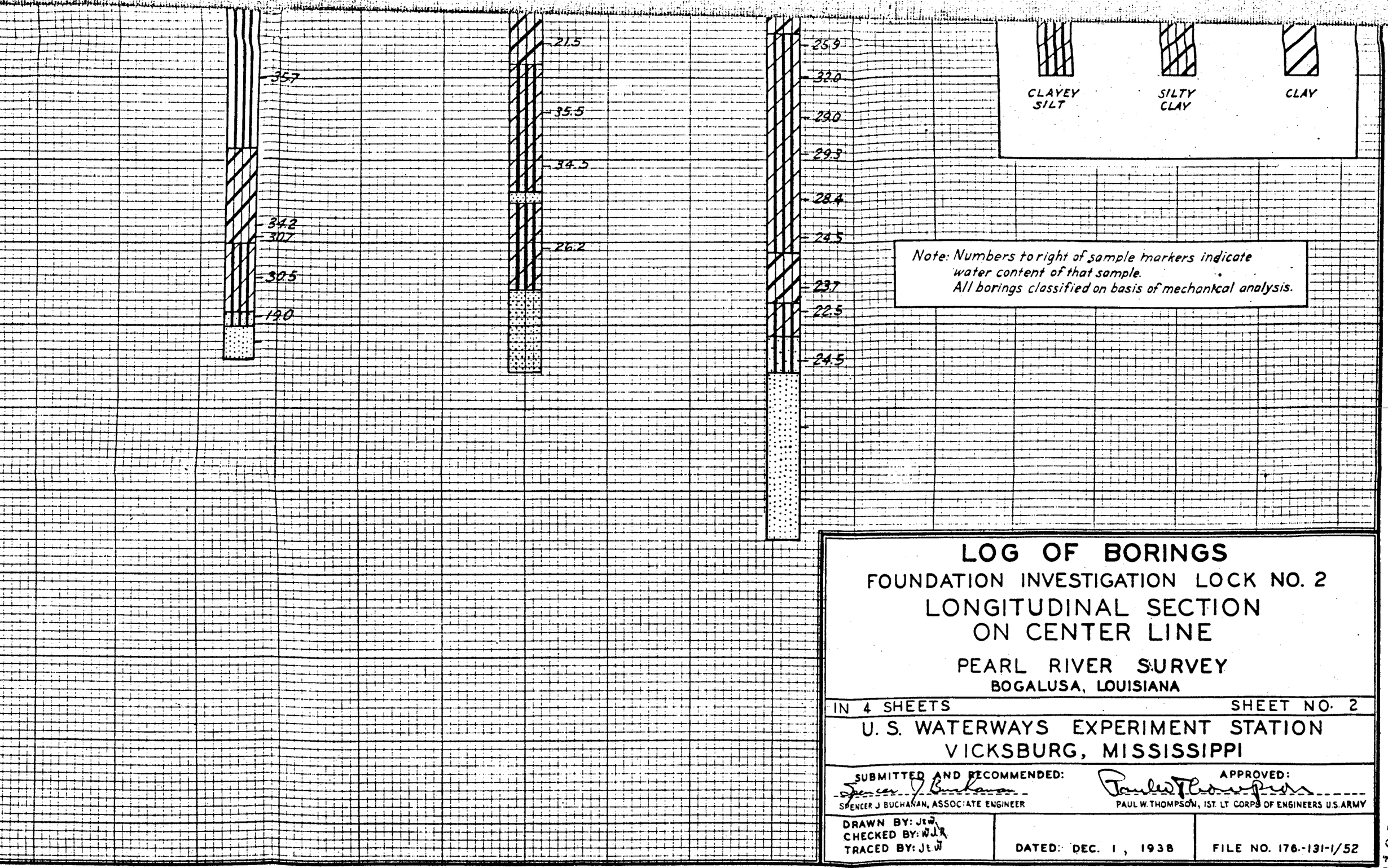
FIGURE 2

WAR DEPARTMENT




STA. 645+83.0
BORING NO. 15
Sur. Elev = 45.9 Ft.
45.9 Ft.
Ground Water
Denotes Elev. of Sample
34.7
18.1
25.1
22.8
20.5
20.7
STA. 643+23.0
BORING NO. 38
Sur. Elev = 44.3 Ft.
Ground Water
20.0
20.6
Denotes Elev. of Sample
22.0
21.5
STA. 643+33.0
BORING NO. 11
Sur. Elev = 44.1 Ft.
Ground Water
Denotes Elev. of Sample
33.3
29.7
26.0
28.1
19.4
25.9
LEGEND
GRAVEL
SANDY GRAVEL
GRAVELLY SAND
SAND
SILTY SAND
CLAYEY SAND
SANDY SILT
SANDY CLAY
SILT

Elevation Fee
40
50
60
70
80
90
100
110
120
23.8
28.5
22.0
23.2
26.2
27.3
24.8
24.9
30.2



LOG OF BORINGS

FOUNDATION INVESTIGATION LOCK NO. 2

LONGITUDINAL SECTION ON CENTER LINE

PEARL RIVER SURVEY
BOGALUSA, LOUISIANA

IN 4 SHEETS — SHEET NO. 2

U. S. WATERWAYS EXPERIMENT STATION
VICKSBURG, MISSISSIPPI

SUBMITTED AND RECOMMENDED: Spencer J Buchanan
SPENCER J BUCHANAN, ASSOCIATE ENGINEER

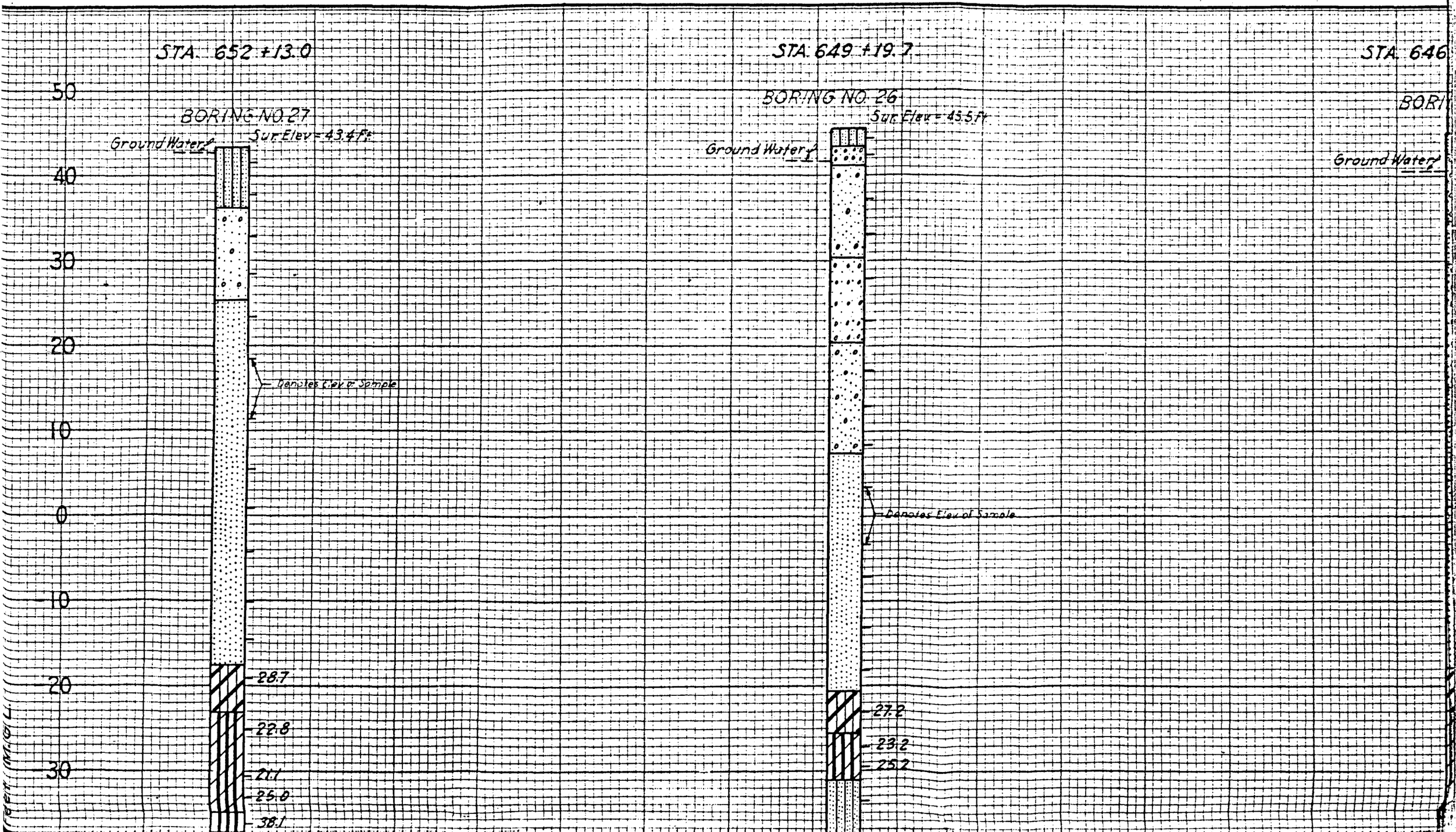
APPROVED: Paul W Thompson
PAUL W. THOMPSON, 1ST. LT CORPS OF ENGINEERS U.S. ARMY

DRAWN BY: JEW CHECKED BY: WJR TRACED BY: JEW	DATED: DEC. 1, 1938	FILE NO. 176.-131-1/52

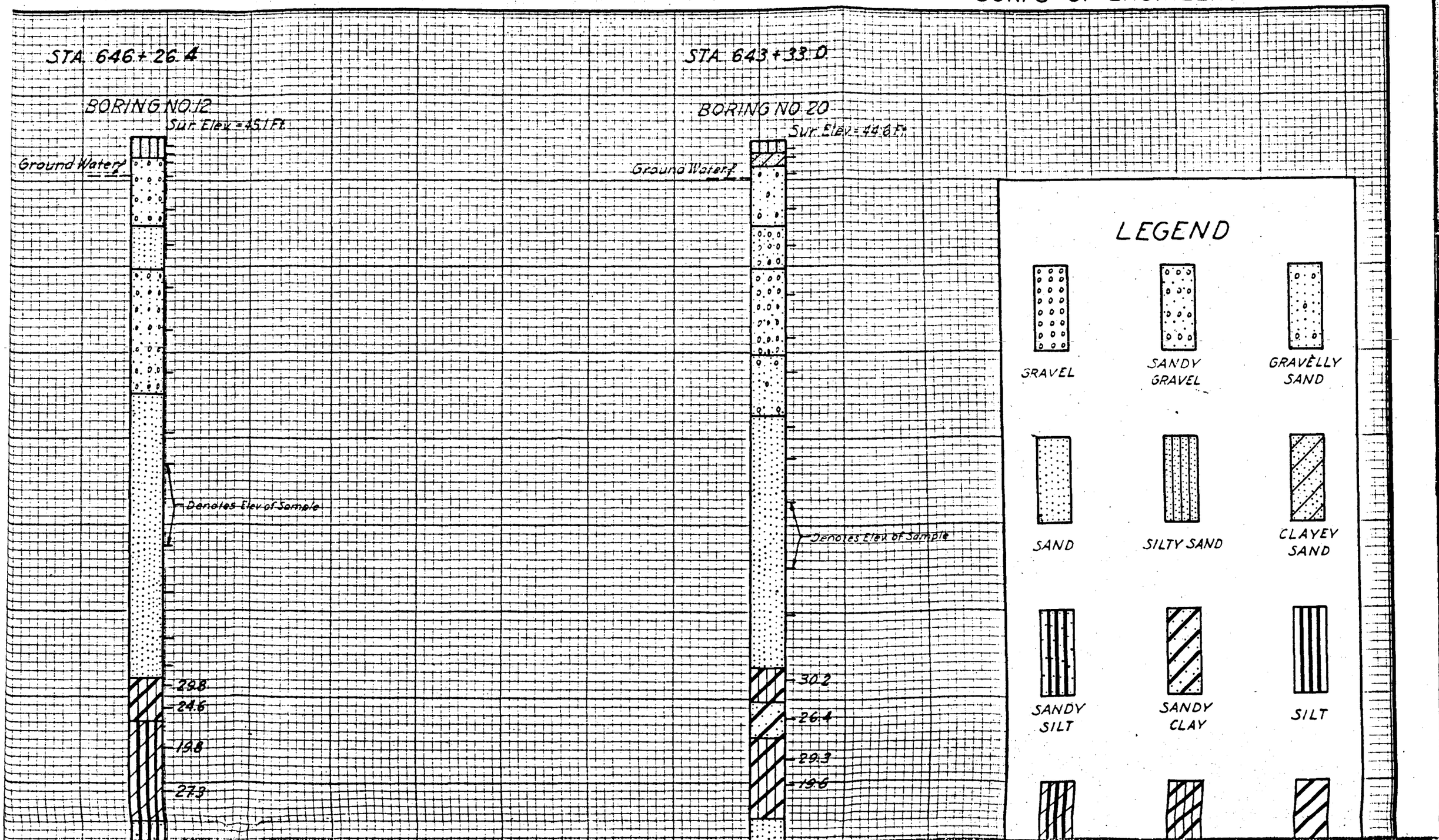
31642

FIGURE 3

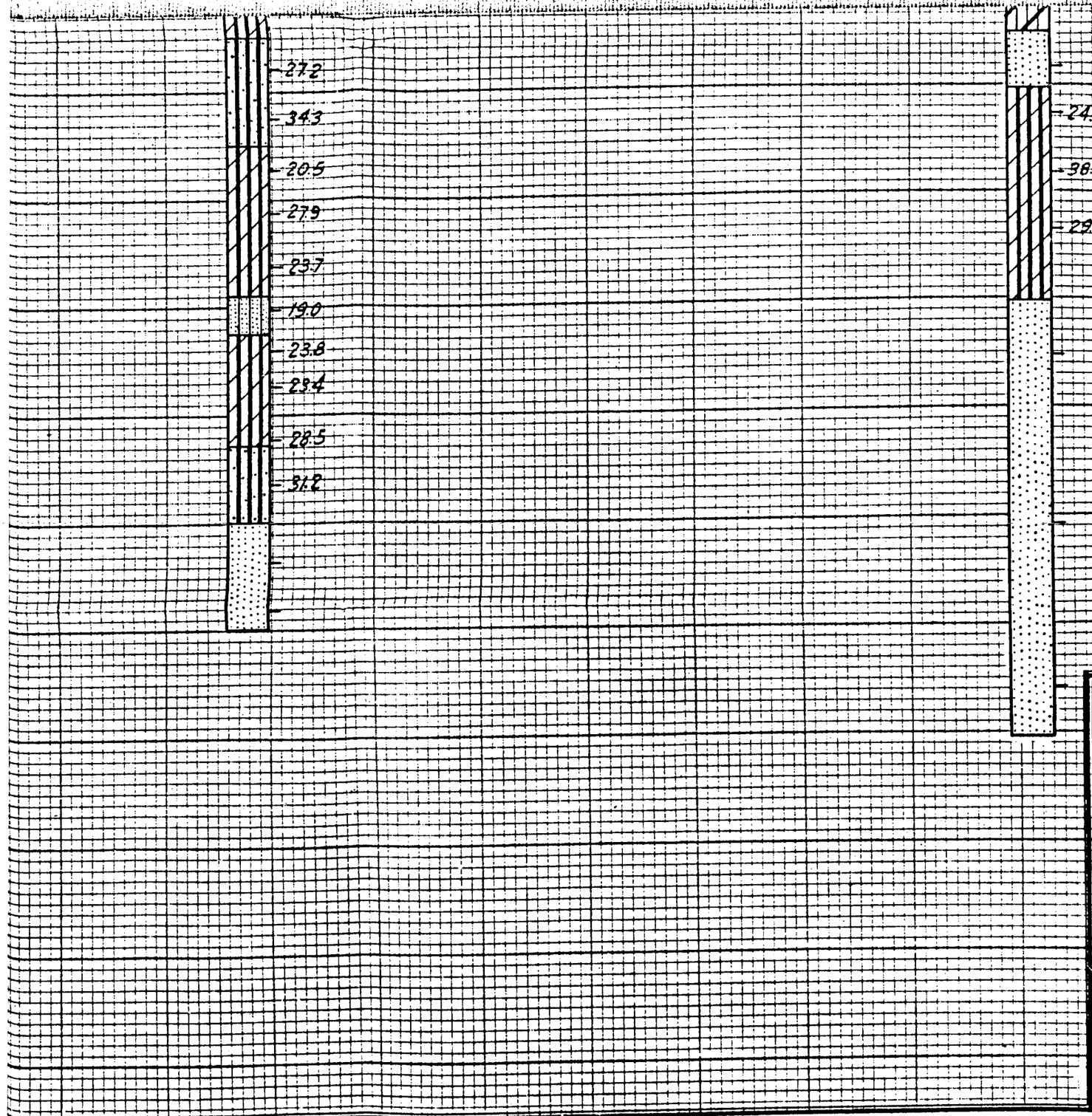
WAR DEPARTMENT

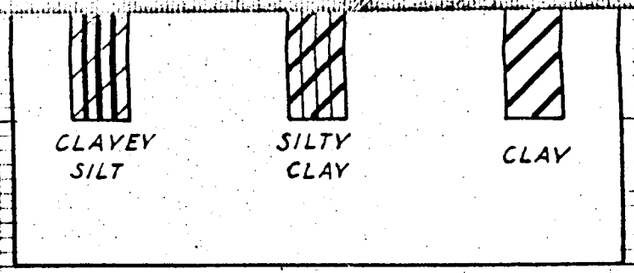


CORPS OF ENGINEERS U. S. ARMY



Elevation feet
30
40
50
60
70
80
90
100
110
120
21.1
25.0
38.1
21.1





Note: Numbers to right of sample markers indicate water content of that sample.
Boring 12 classified on basis of mechanical analysis of the samples. Borings 20, 26, and 27 classified by visual inspection of samples.

LOG OF BORINGS

FOUNDATION INVESTIGATION LOCK NO. 2

LONGITUDINAL SECTION WEST OF CENTER LINE

PEARL RIVER SURVEY
BOGALUSA, LOUISIANA

IN 4 SHEETS — SHEET NO. 3

U. S. WATERWAYS EXPERIMENT STATION
VICKSBURG, MISSISSIPPI

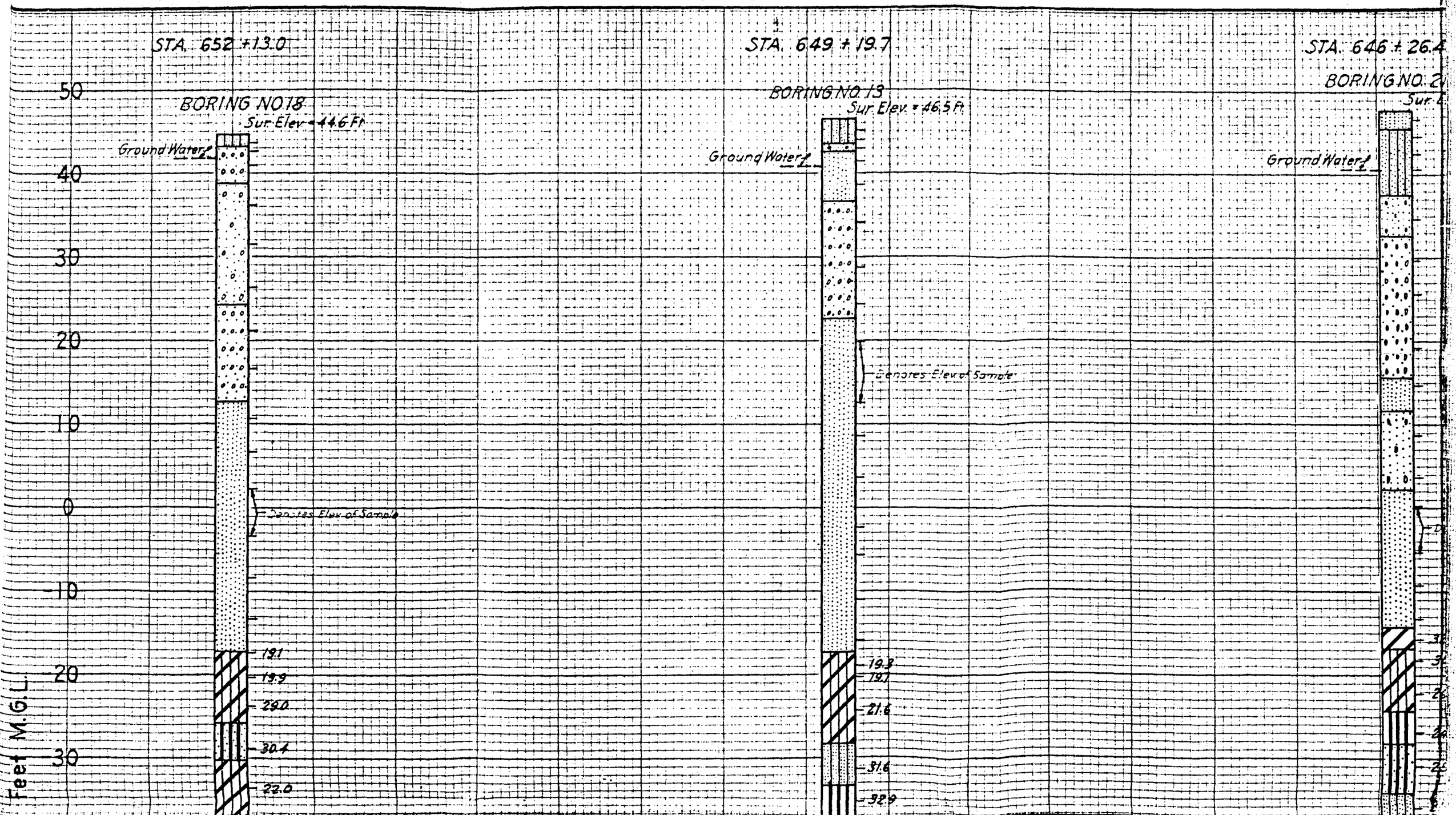
SUBMITTED AND RECOMMENDED: Spencer J. Buchanan
SPENCER J BUCHANAN, ASSOCIATE ENGINEER

APPROVED: Paul W Thompson
PAUL W THOMPSON, 1ST. LT CORPS OF ENGINEERS U.S.ARMY

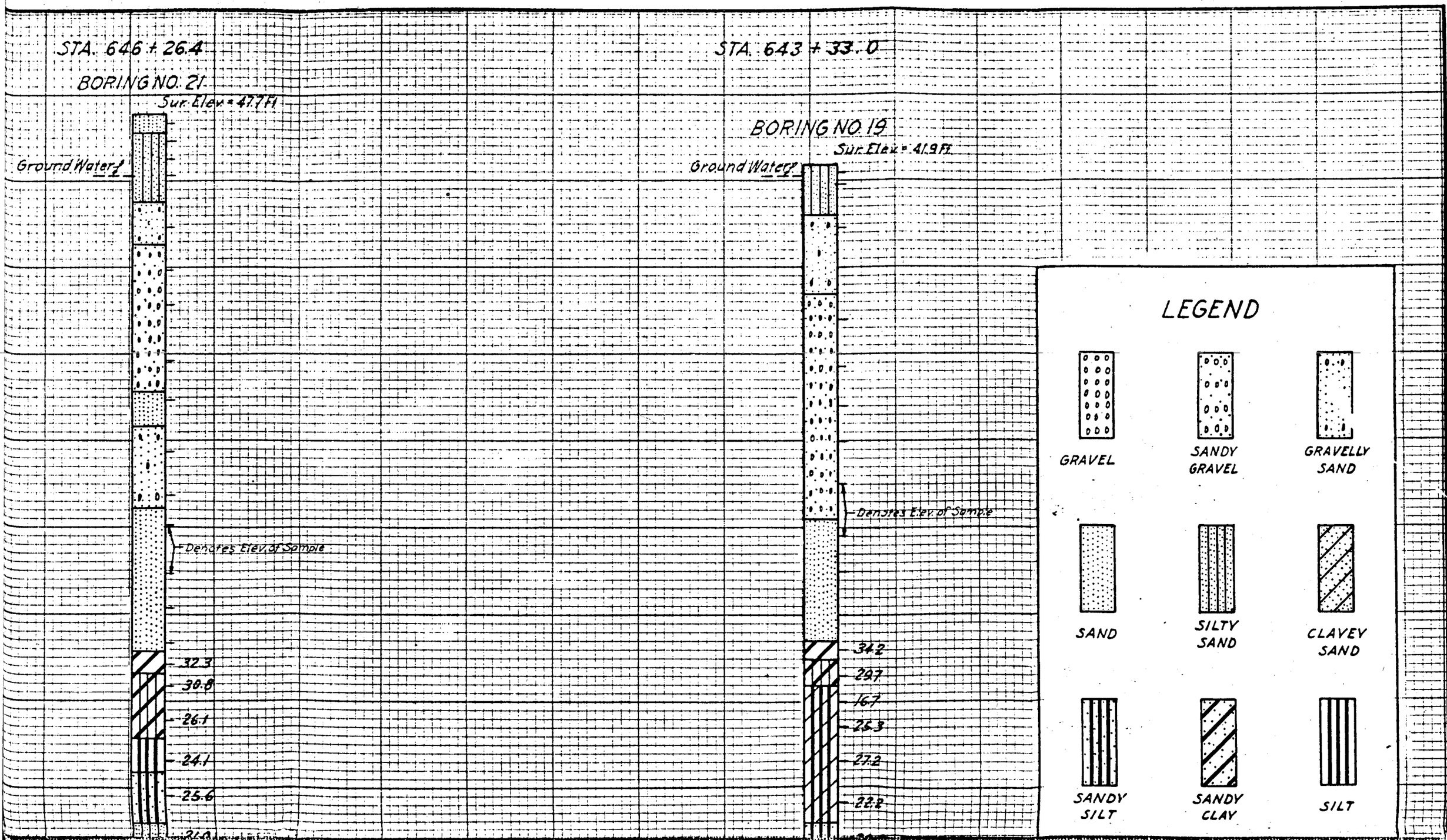
DRAWN BY: JEW CHECKED BY: W J R TRACED BY: JEW	DATED DEC. 1, 1938	FILE NO. 176.-131-1/53

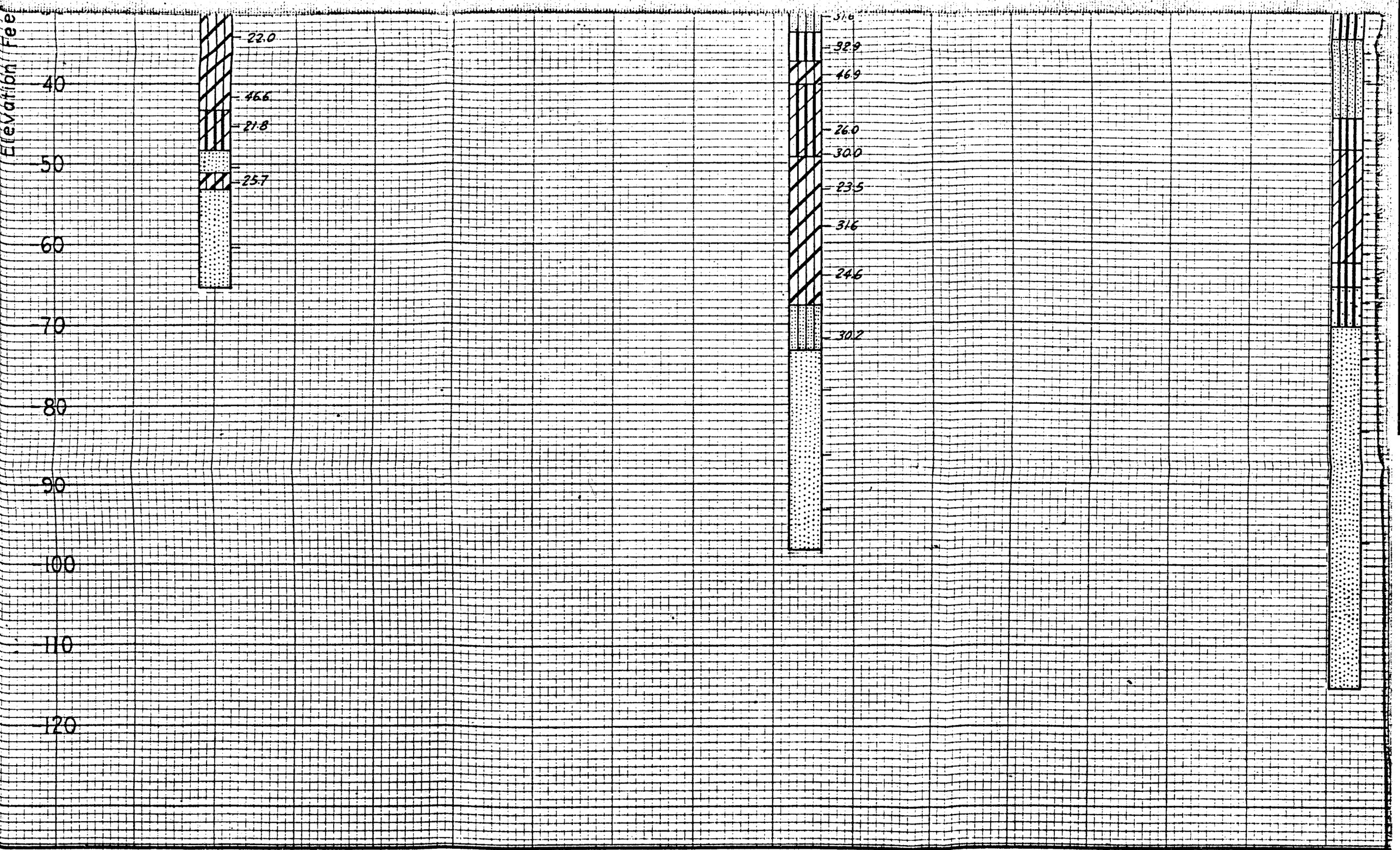
FIGURE 4

WAR DEPARTMENT



CORPS OF ENGINEERS U.S. ARMY




Elevation Feet
40
50
60
70
80
90
100
110
120
22.0
46.6
21.8
25.7
31.6
32.9
46.9
26.0
30.0
23.5
31.6
24.6
30.2

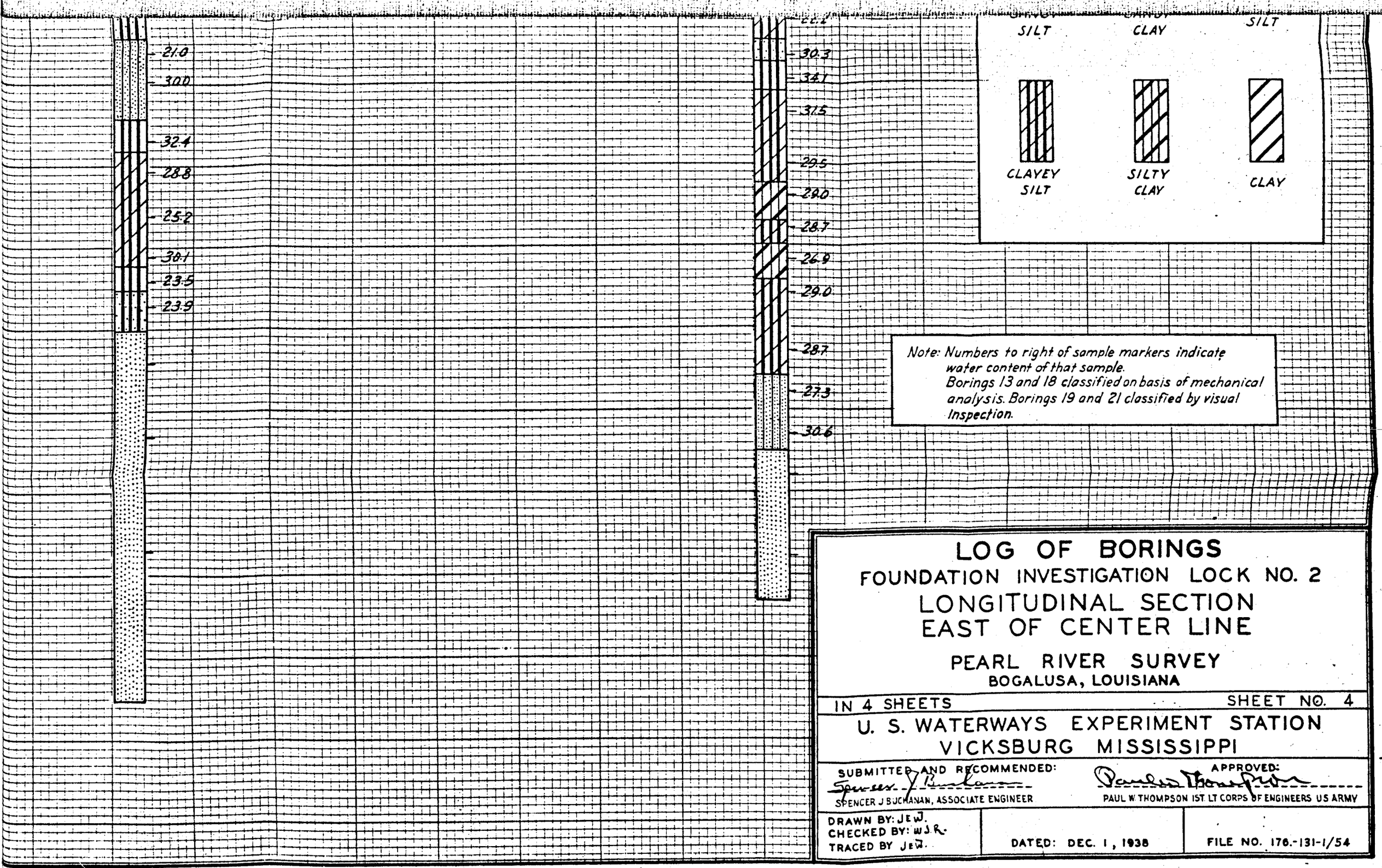

21.0
30.0
32.4
28.8
25.2
30.1
23.5
23.9
30.3
34.1
31.5
29.5
29.0
28.7
26.9
29.0
28.7
27.3
30.6
SILT
CLAY
SILT
CLAYEY SILT
SILTY CLAY
CLAY
Note: Numbers to right of sample markers indicate water content of that sample.
Borings 13 and 18 classified on basis of mechanical analysis. Borings 19 and 21 classified by visual inspection.
LOG OF BORINGS
FOUNDATION INVESTIGATION LOCK NO. 2
LONGITUDINAL SECTION
EAST OF CENTER LINE
PEARL RIVER SURVEY
BOGALUSA, LOUISIANA
IN 4 SHEETS
SHEET NO. 4
U. S. WATERWAYS EXPERIMENT STATION
VICKSBURG MISSISSIPPI
SUBMITTED AND RECOMMENDED:
SPENCER J BUCHANAN, ASSOCIATE ENGINEER
APPROVED:
PAUL W. THOMPSON 1ST LT CORPS OF ENGINEERS US ARMY
DRAWN BY: J.E.W.
CHECKED BY: W.J.R.
TRACED BY J.E.W.
DATED: DEC. 1, 1938
FILE NO. 176.-131-1/54

FIGURE 5

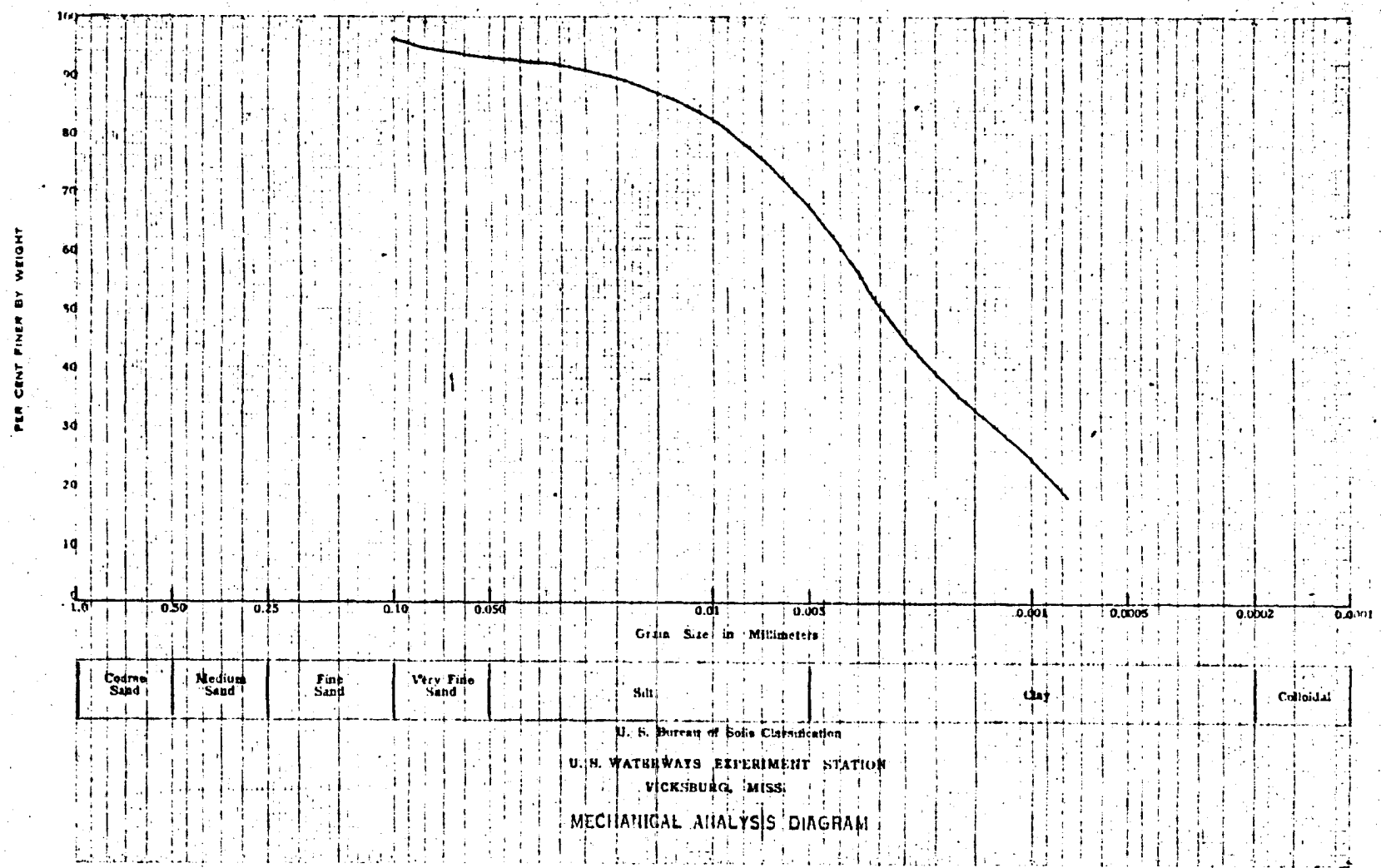


Figure 6

Sample No. 16 - Detailed Boring No. 15

Location of Sample Elev. -14.6. Lock Site 2

Remarks Pearl River Project

Date Sample Taken

Date Sample Tested 3-2-38

Specific Gravity of Sample 2.73

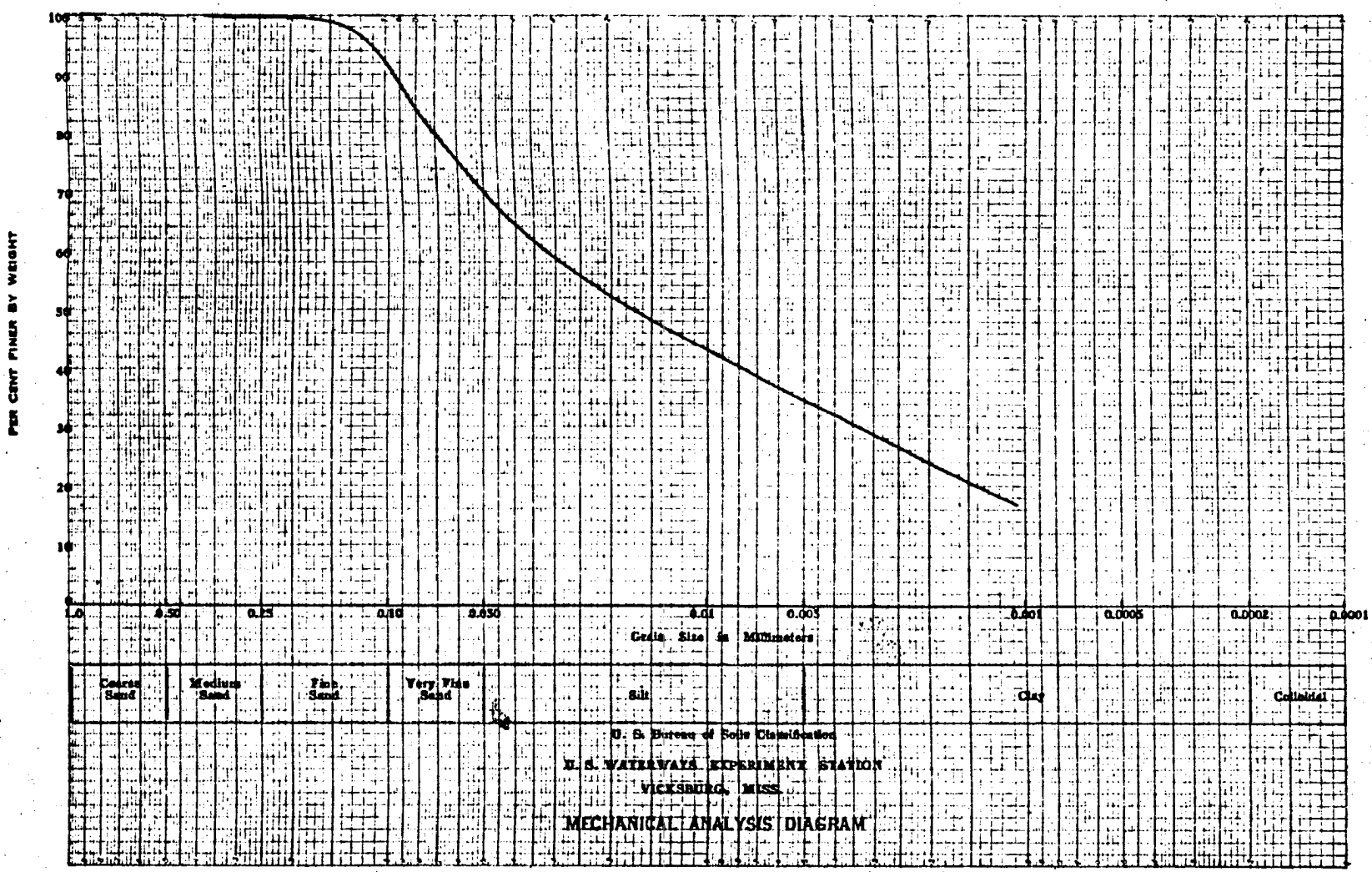


Figure 7

Sample No. 17 - Detailed Boring No. 15

Location of Sample Elev. -16.7. Lock Site 2

Remarks Pearl River Project

Date Sample Taken

Date Sample Tested 2-28-38

Specific Gravity of Sample 2.72

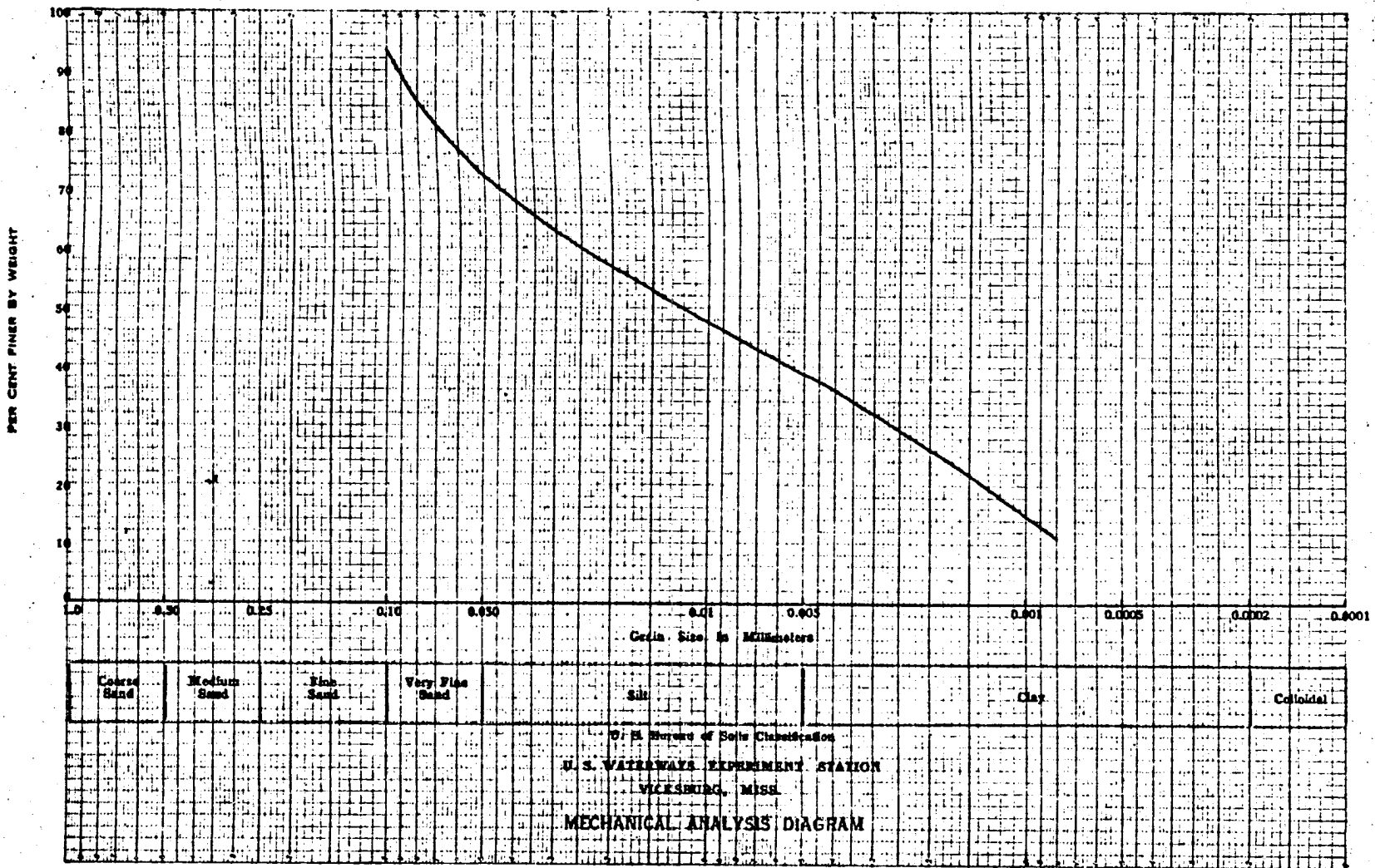


Figure 8

Sample No. 18 - Detailed Boring No. 15

Location of Sample Elev. -20.1. Lock Site 2

Remarks Pearl River Project

Date Sample Taken

Date Sample Tested 3-3-38

Specific Gravity of Sample 2.70

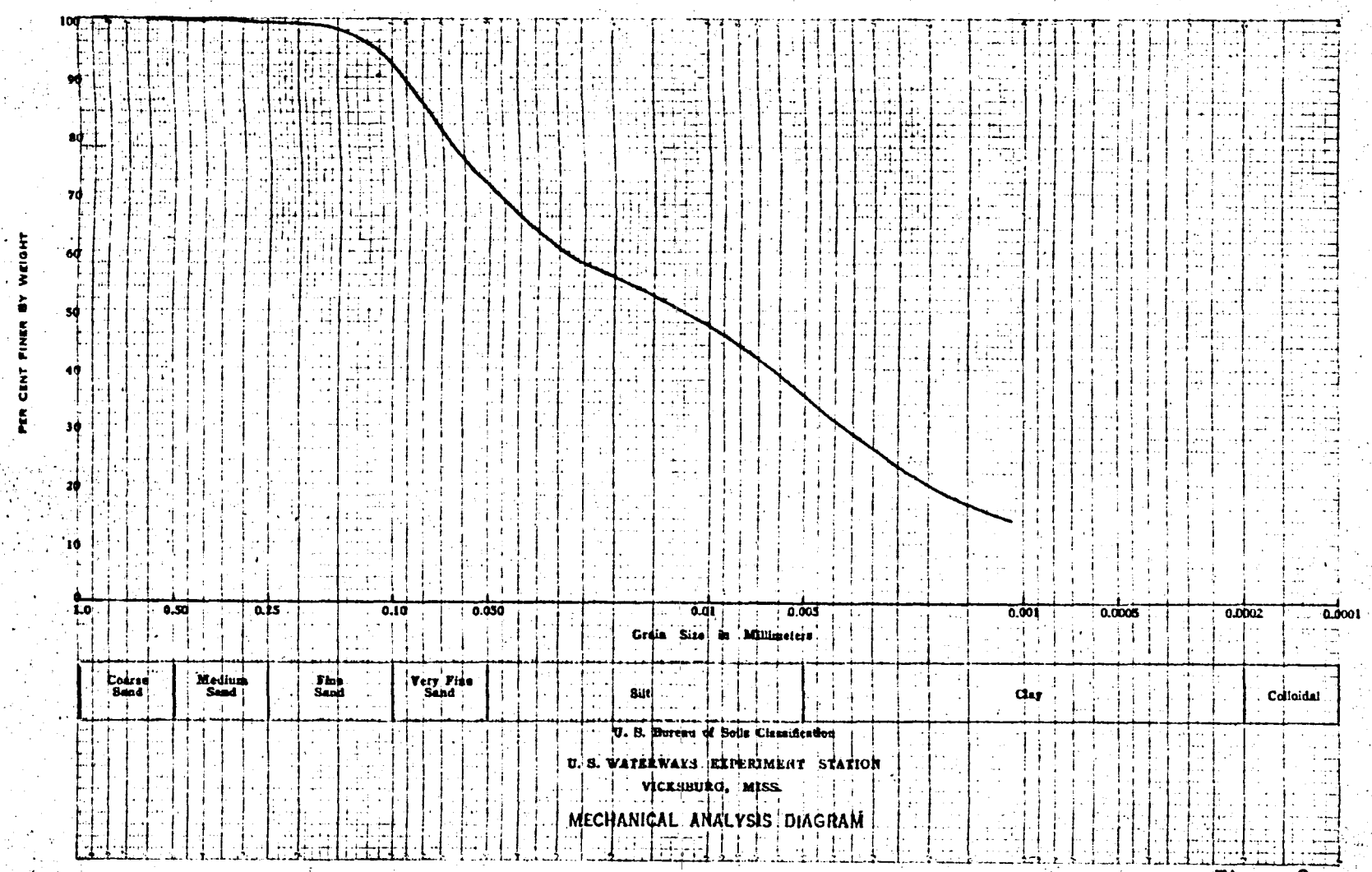


Figure 9

Sample No. 19 - Detailed Boring No. 15

Location of Sample Elev. -23.1. Lock Site 2

Remarks Pearl River Project

Date Sample Taken

Date Sample Tested 2-28-38

Specific Gravity of Sample 2.70

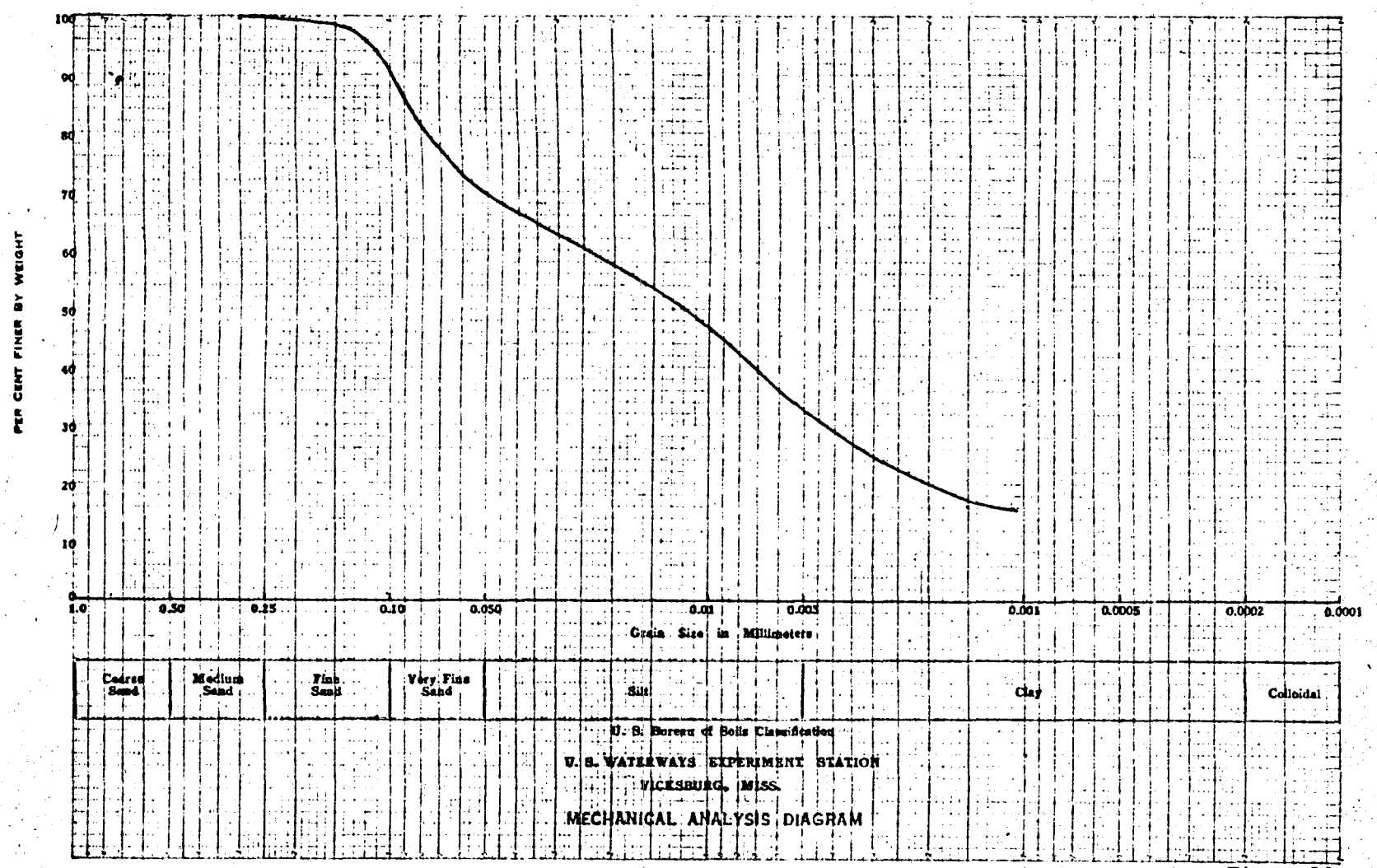


Figure 10

Sample No. 20 - Detailed Boring No. 15

Location of Sample Elev. -30.1. Lock Site 2

Remarks Pearl River Project

Date Sample Taken

Date Sample Tested 2-28-38

Specific Gravity of Sample 2.68

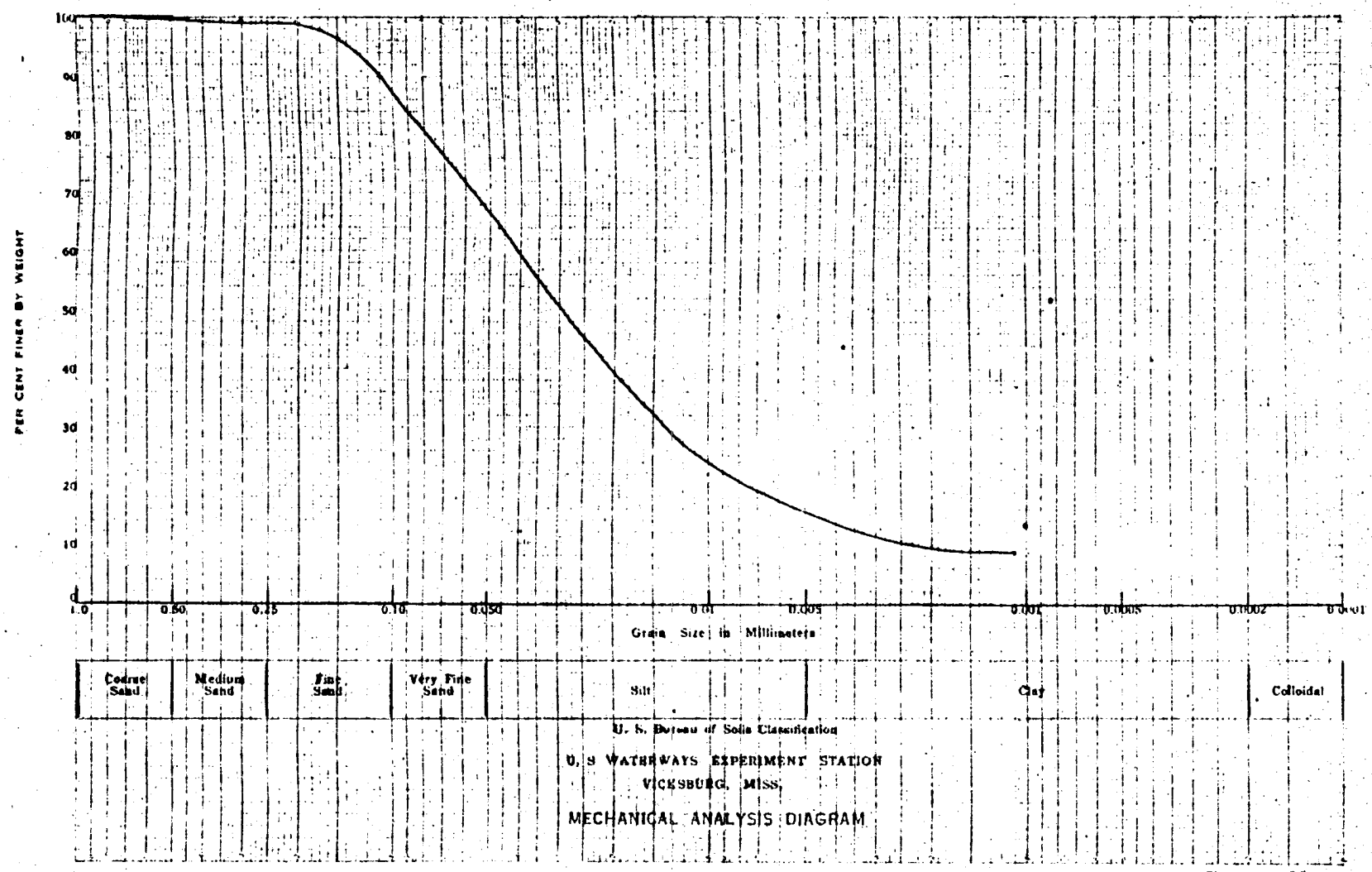


Figure 11

Sample No. 21 - Detailed Boring No.15

Location of Sample Elev. -36.7. Lock Site 2

Remarks Pearl River Project

Date Sample Taken

Date Sample Tested 2-28-33

Specific Gravity of Sample 2.69

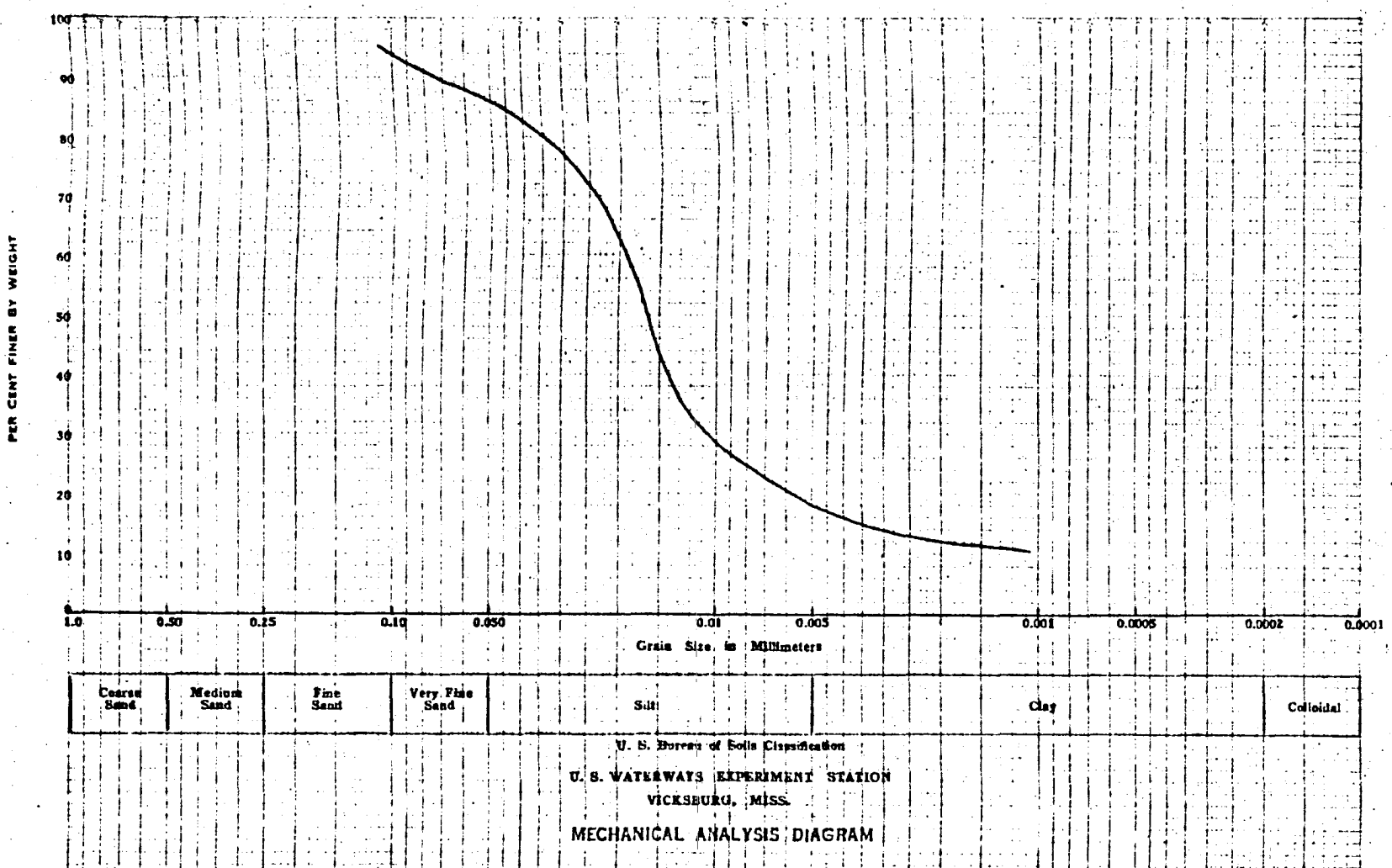


Figure 12

Sample No. 22 - Detailed Boring No. 15

Location of Sample Elev. -42.7. Lock Site 2

Remarks Pearl River Project

Date Sample Taken

Date Sample Tested 2-28-38

Specific Gravity of Sample 2.72

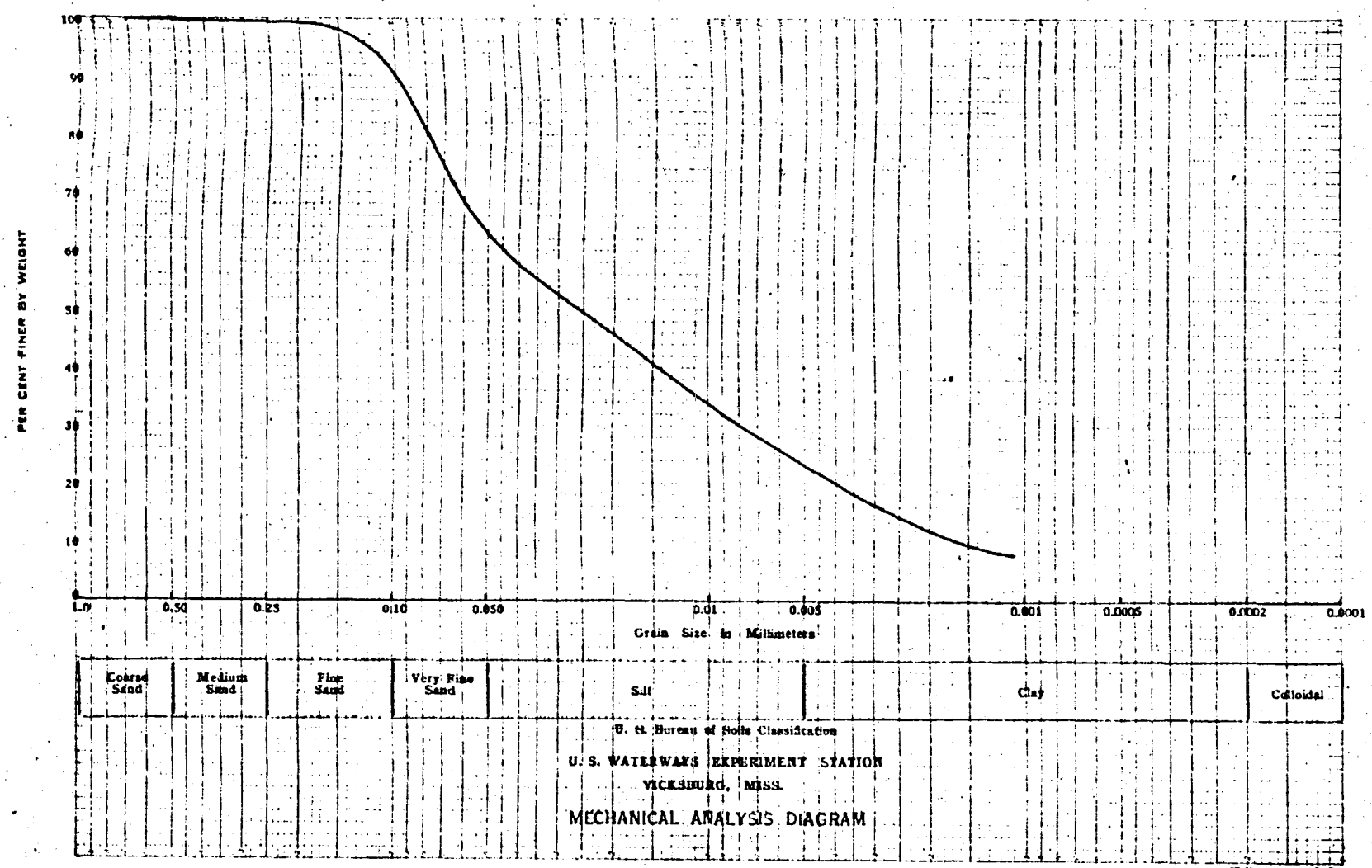


Figure 13

Sample No. 23 - Detailed Boring No. 15

Location of Sample Elev. -46.9. Lock Site 2

Remarks Pearl River Project

Date Sample Taken

Date Sample Tested 2-28-38

Specific Gravity of Sample 2.70

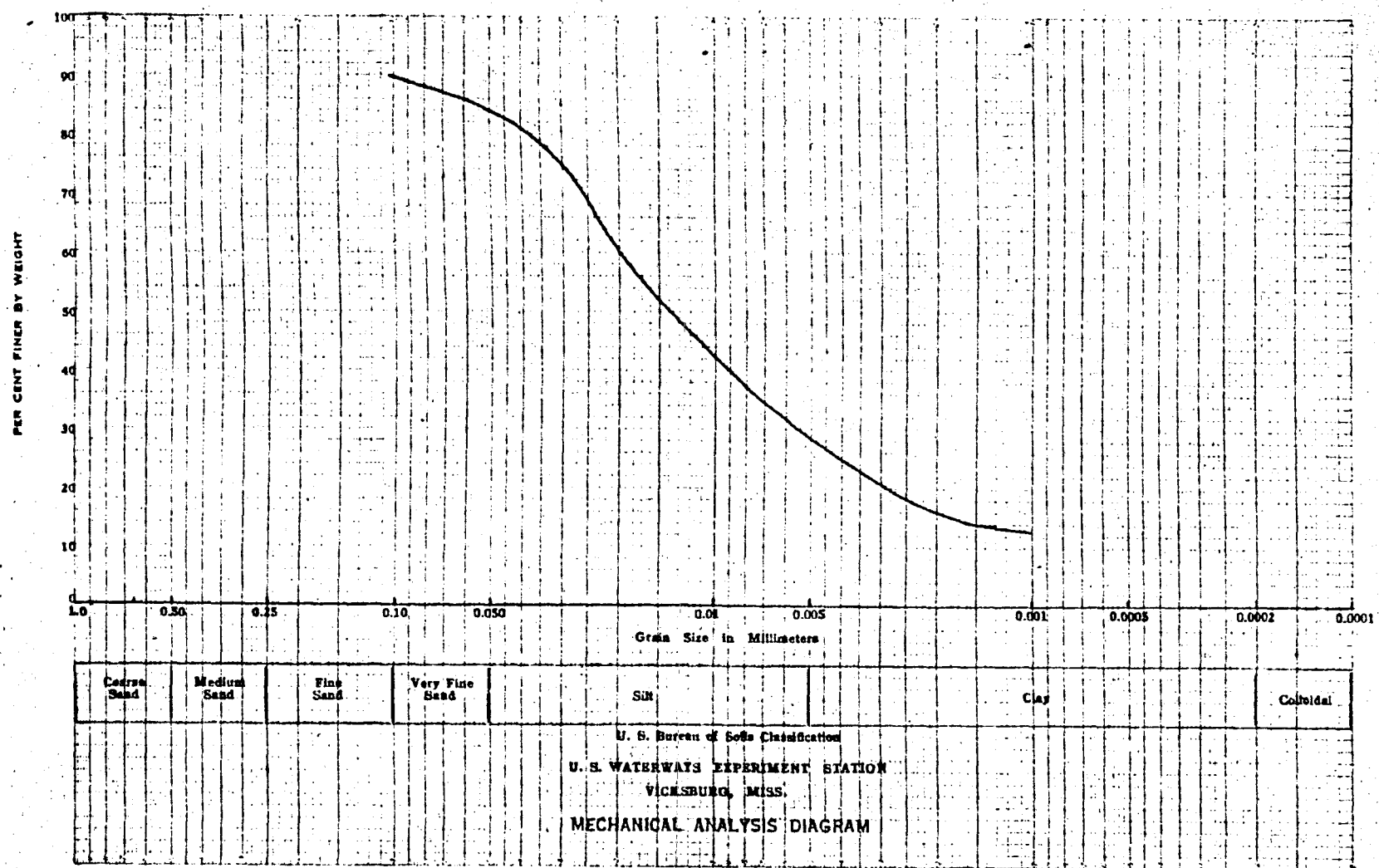


Figure 14

Sample No. 24 - Detailed Boring No. 15

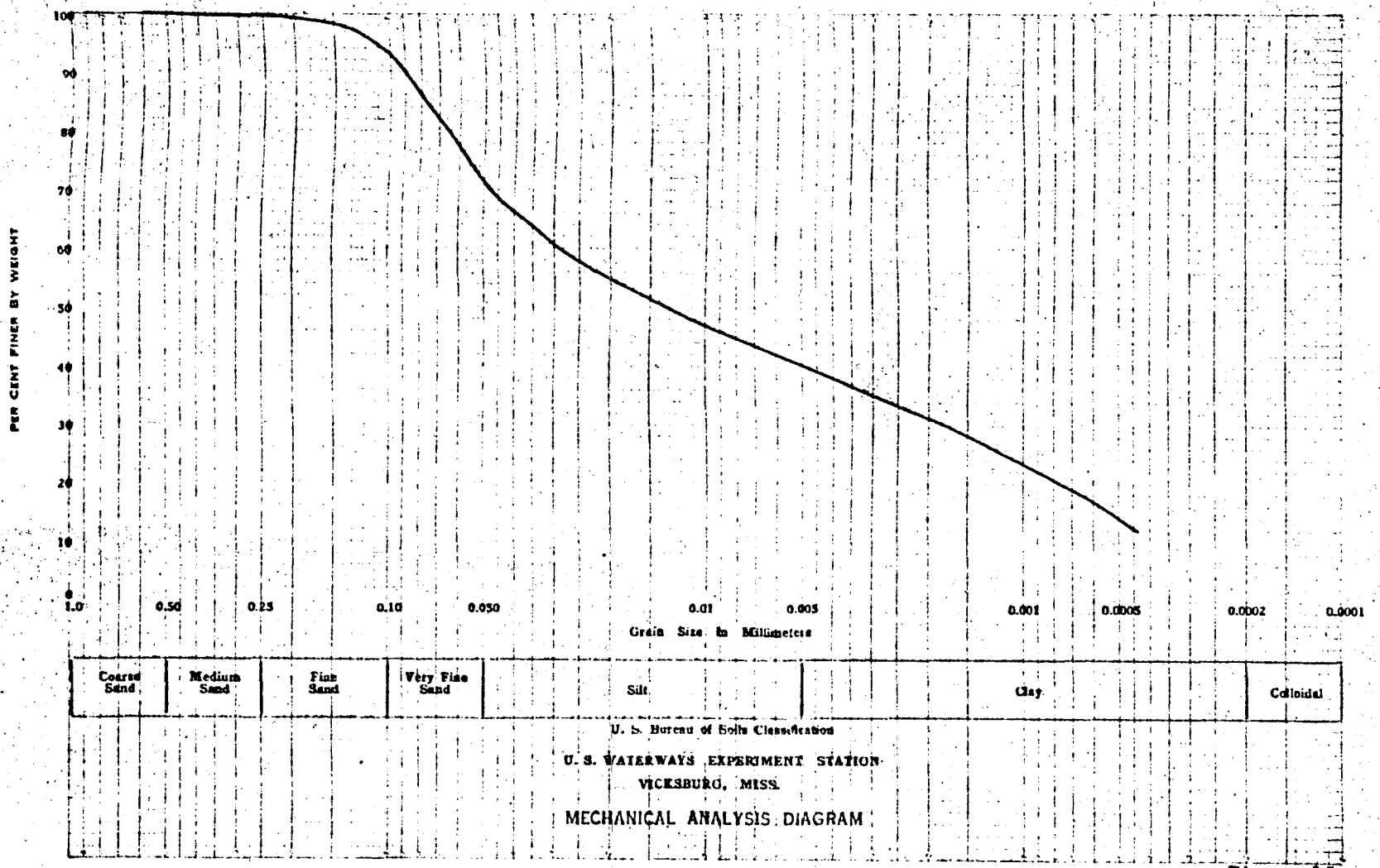
Location of Sample Elev. -55.8. Lock Site 2.

Remarks Pearl River Project

Date Sample Taken

Date Sample Tested 2-28-38

Specific Gravity of Sample 2.71



U. S. WATERWAYS EXPERIMENT STATION
VICKSBURG, MISS.

MECHANICAL ANALYSIS DIAGRAM

Figure 15

Sample No. 25 - Detailed Boring No. 15
Location of Sample Elev. -61.1, Lock Site No. 2
Remarks Pearl River Project

Date Sample Taken
Date Sample Tested 2-28-38
Specific Gravity of Sample 2.71

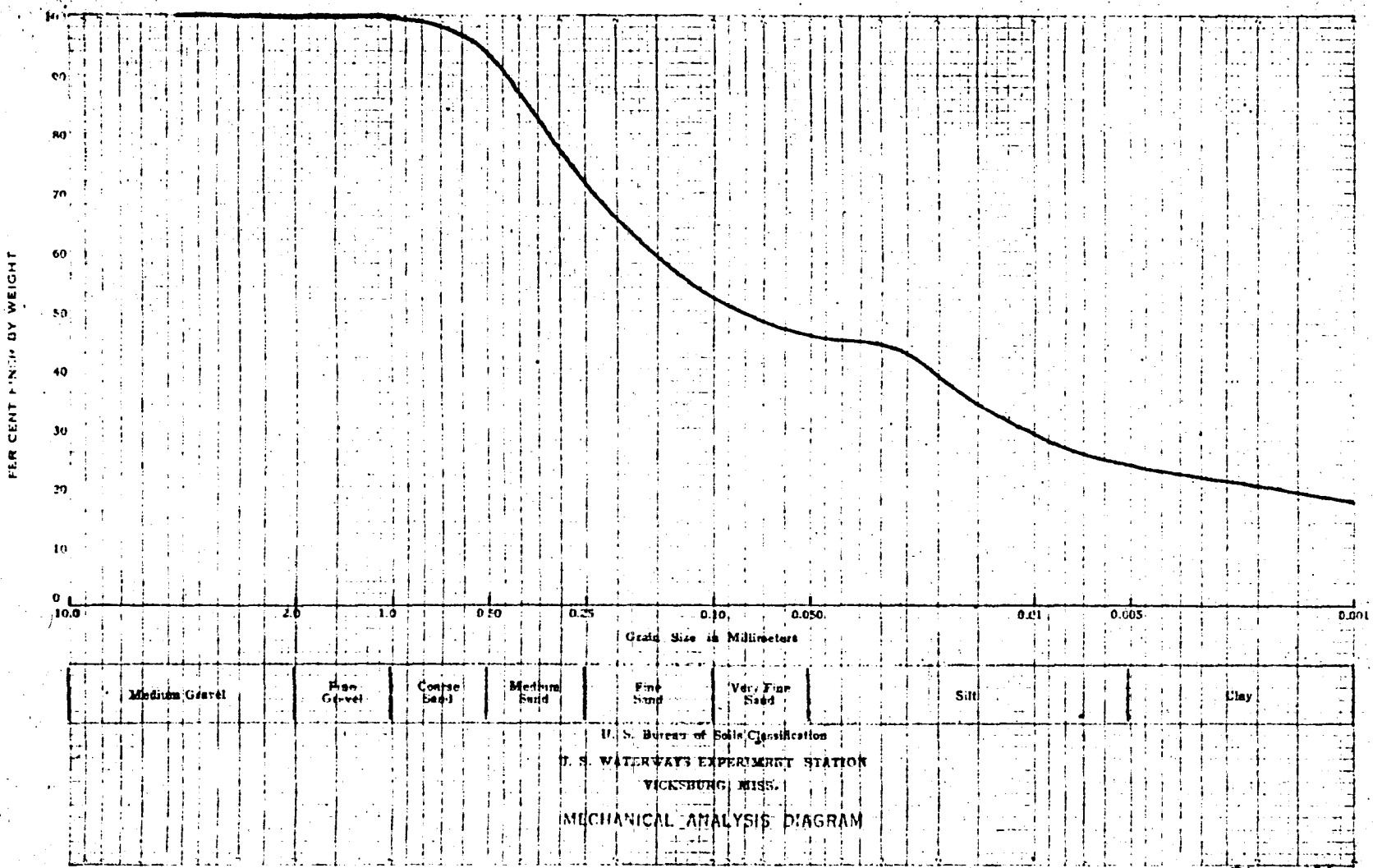


Figure 16

Sample No. 1 - Detailed Boring No. 16

Location of Sample Elev. +44.1 M.G.L. Lock Site No. 2

Remarks Pearl River Project

Date Sample Taken 2-23-38

Date Sample Tested 3-3-38

Specific Gravity of Sample 2.68

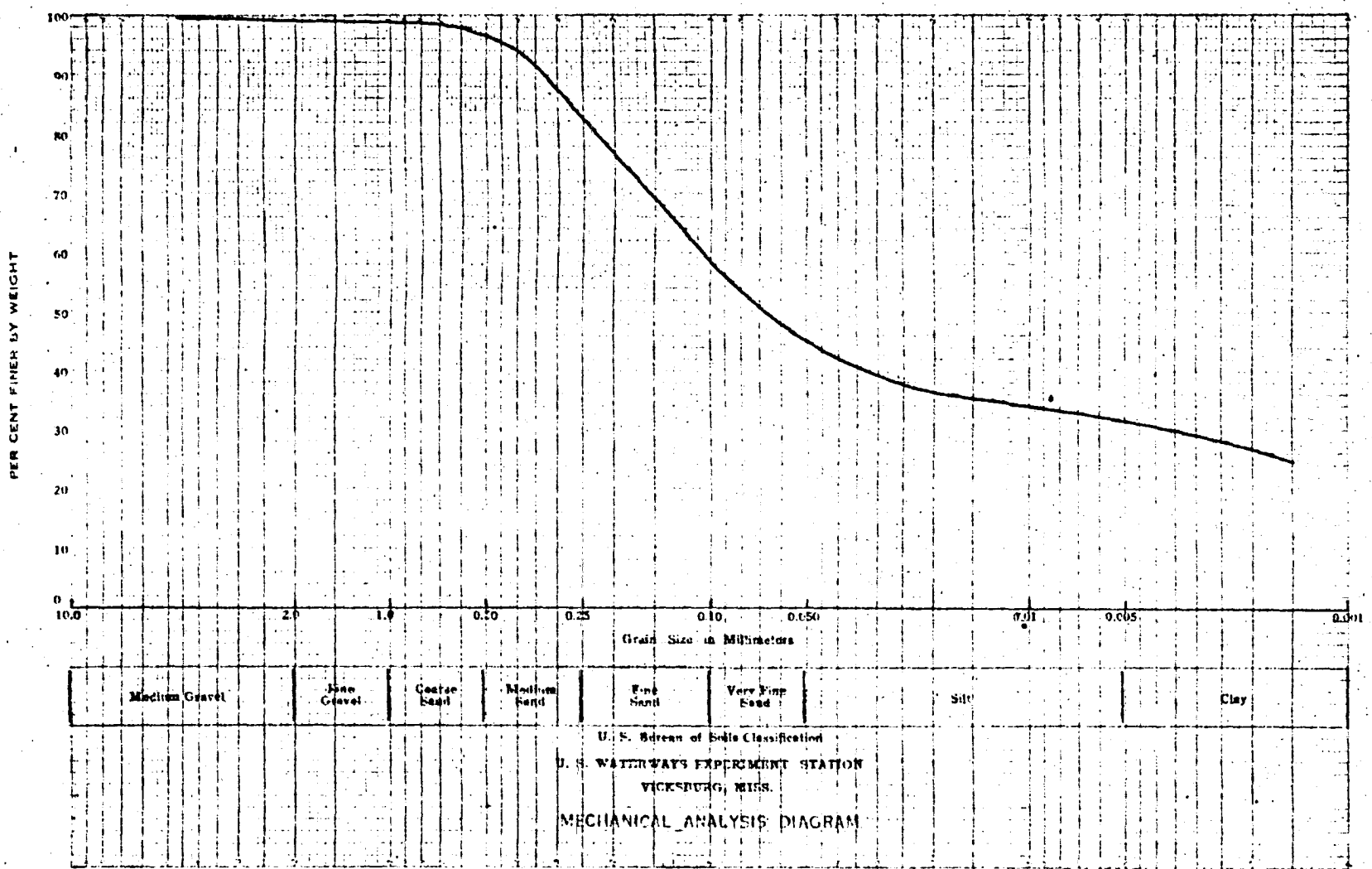


Figure 17

Sample No. 8 - Detailed Boring No. 16

Location of Sample Elev. +17.7 M.G.L. Lock Site No. 2

Remarks Pearl River Project

Date Sample Taken 2-23-38

Date Sample Tested 3-4-38

Specific Gravity of Sample 2.69

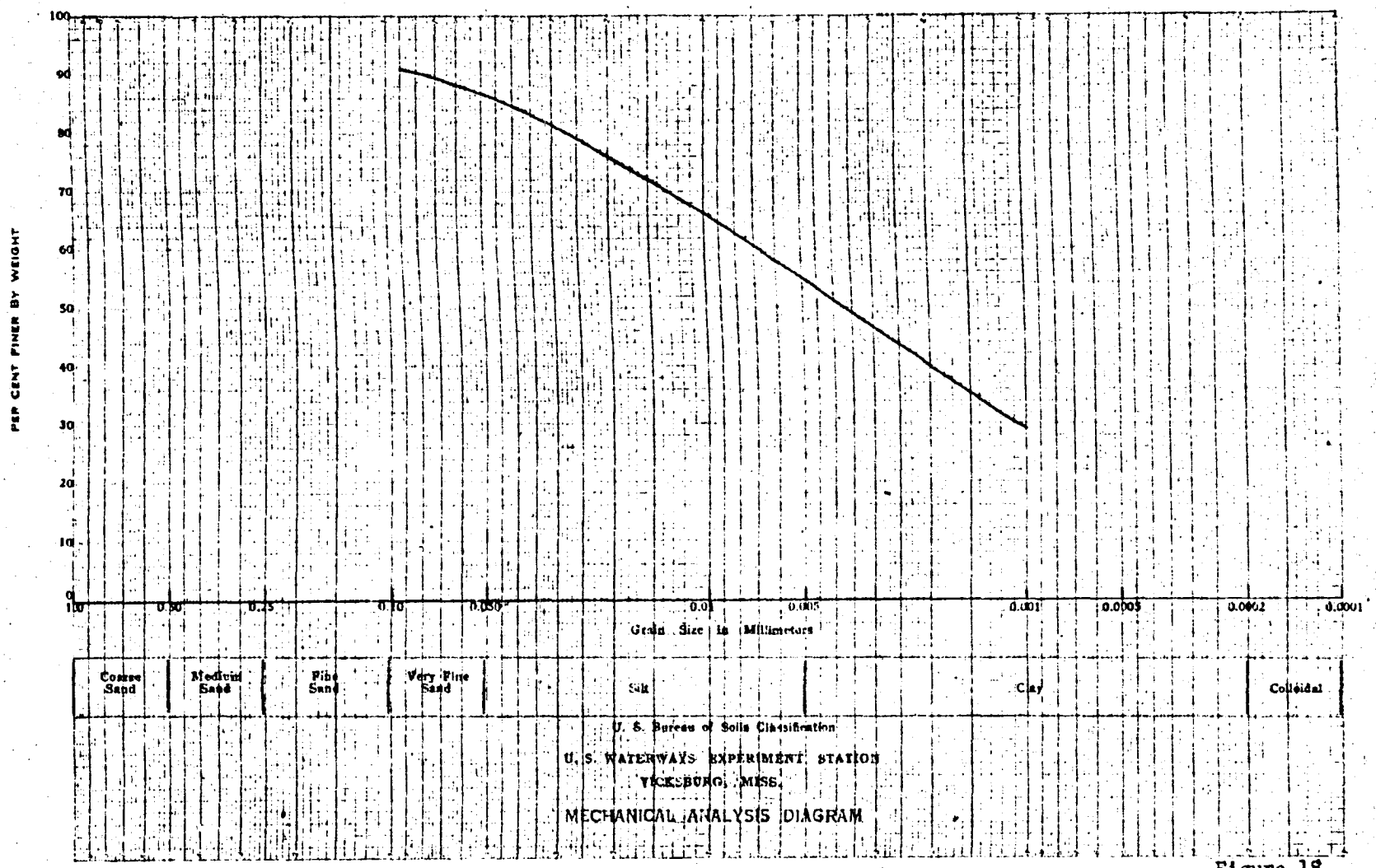


Figure 18

Sample No. 14 - Detailed Boring No. 16

Location of Sample Elev. -20.8 M.G.L. Lock Site No. 2

Remarks Pearl River Project

Date Sample Taken 2-25-38

Date Sample Tested 3-8-38

Specific Gravity of Sample 2.72

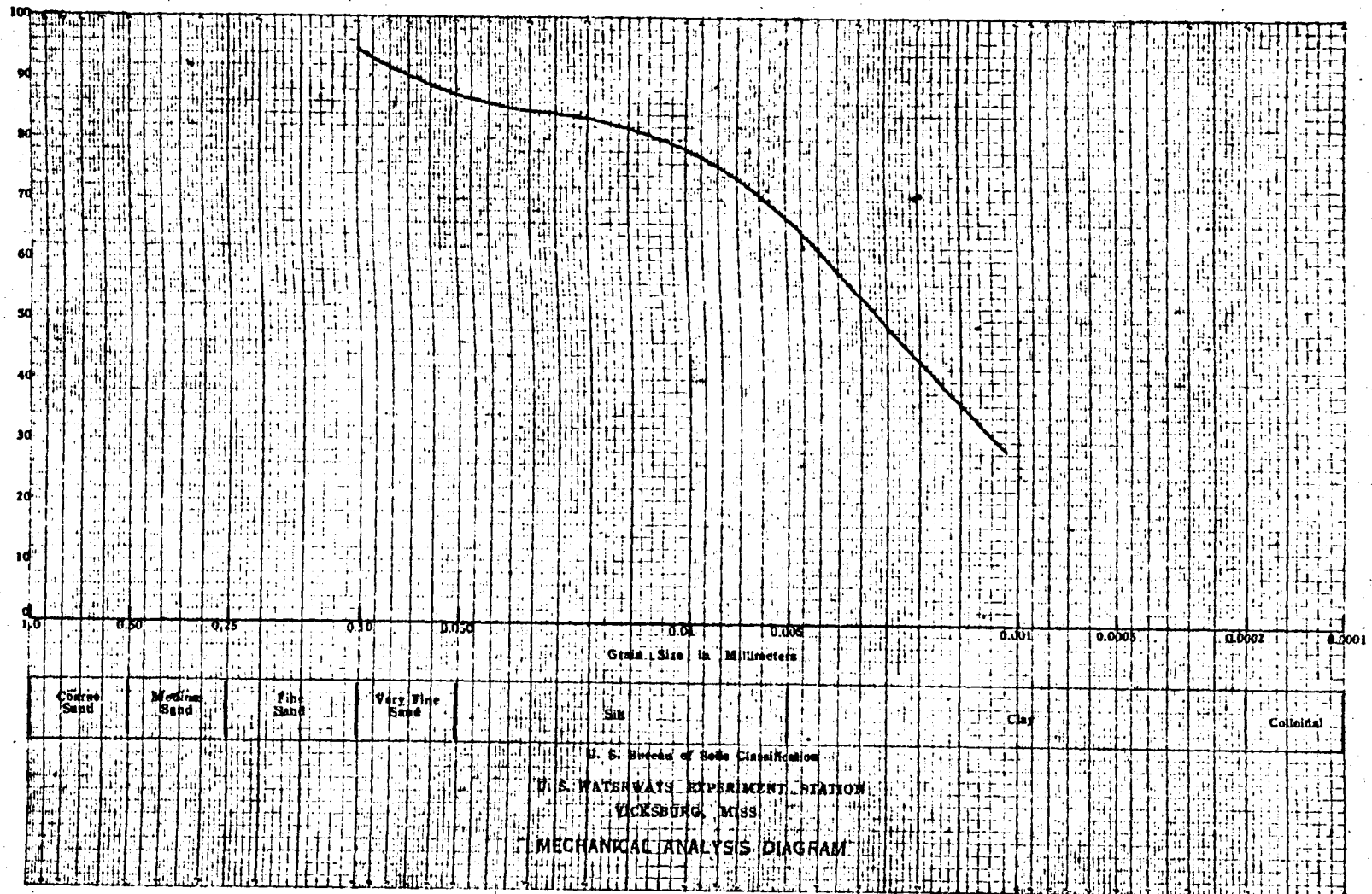


Figure 19

Sample No. 15 - Detailed Boring No. 16

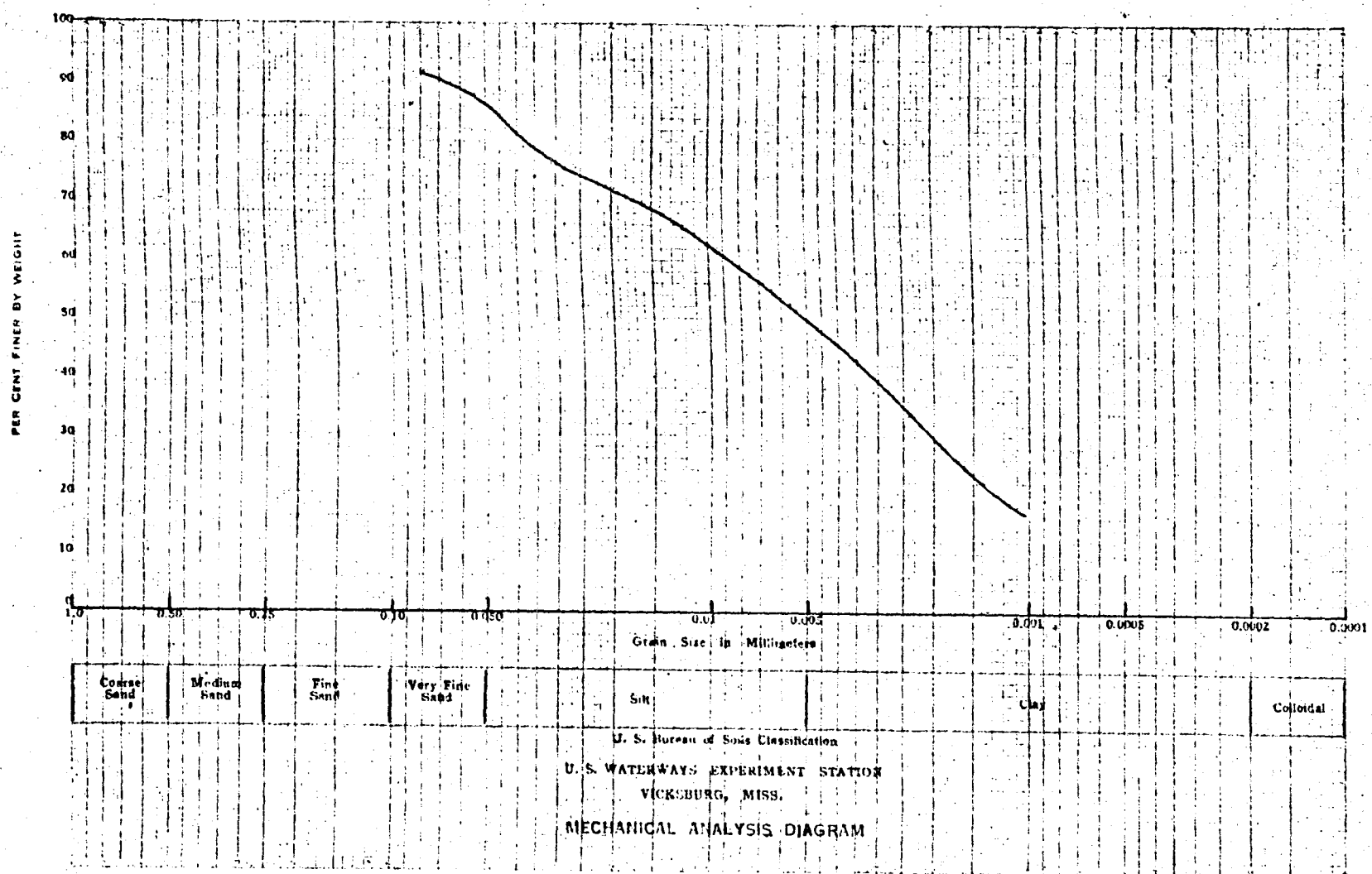
Location of Sample Elev. -23.3. Lock Site No. 2

Remarks Pearl River Project

Date Sample Taken 2-23-38

Date Sample Tested 3-8-38

Specific Gravity of Sample 2.75



U. S. WATERWAYS EXPERIMENT STATION
VICKSBURG, MISS.

MECHANICAL ANALYSIS DIAGRAM

Figure 20

Sample No. 16 - Detailed Boring No. 16
Location of Sample Elev. -26.7. Lock Site No. 2
Remarks Pearl River Project

Date Sample Taken 2-23-38
Date Sample Tested 3-8-38
Specific Gravity of Sample 2.71

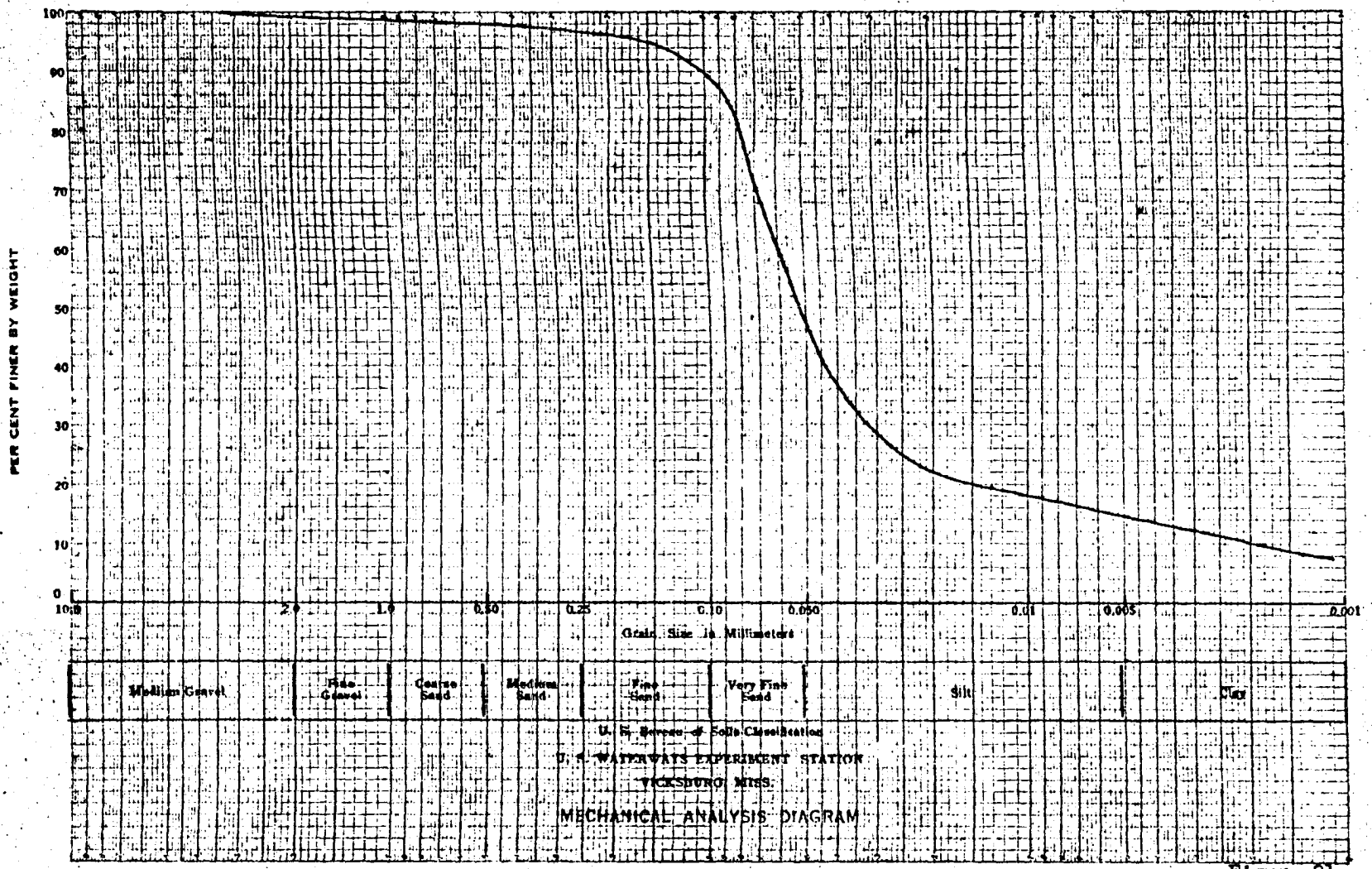


Figure 21

Sample No. 17 - Detailed Boring No. 16

Location of Sample Elev. -31.3. Lock Site No. 2

Remarks Pearl River Project

Date Sample Taken 2-23-38

Date Sample Tested 3-8-38

Specific Gravity of Sample 2.68

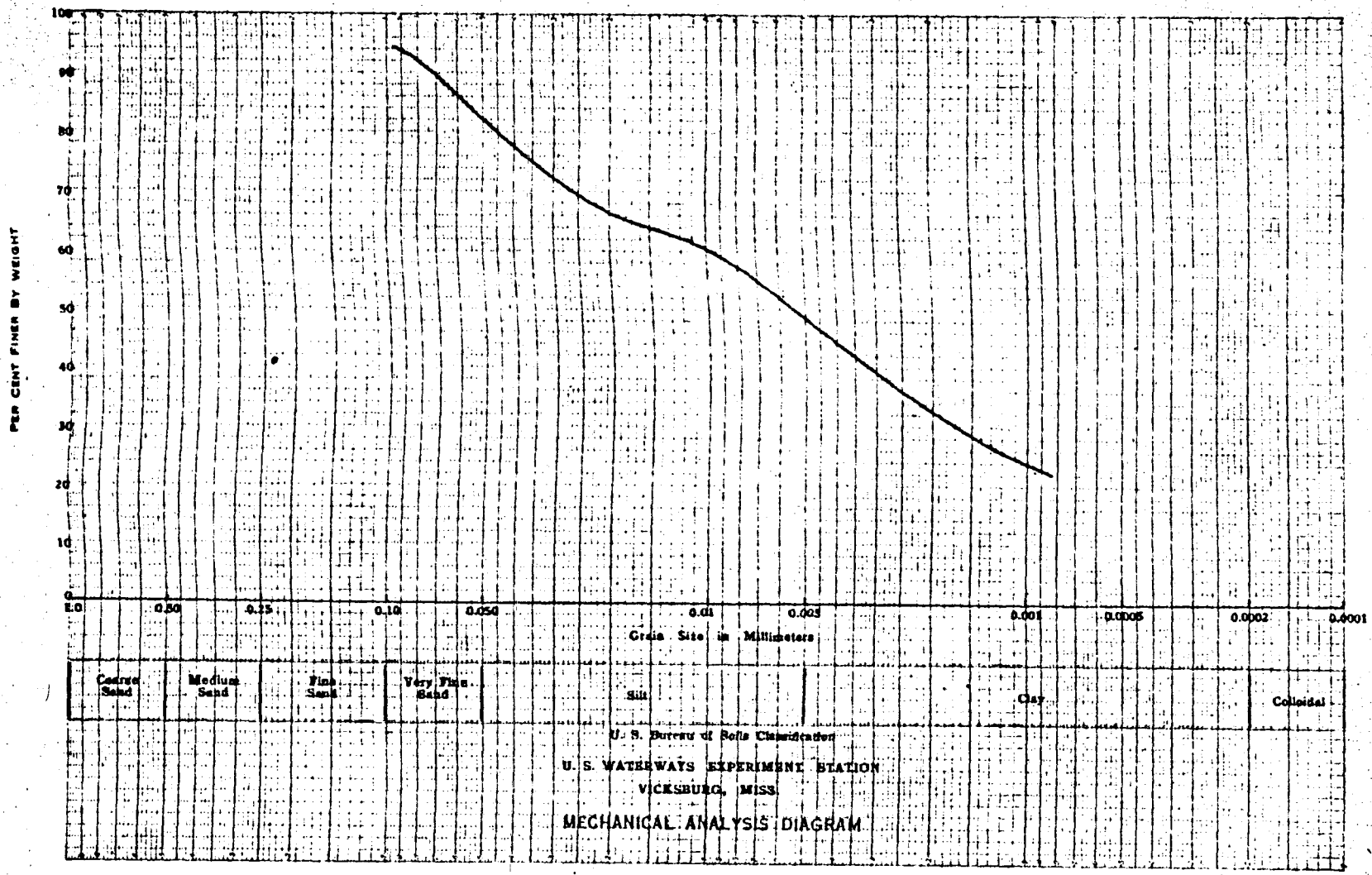


Figure 22

Sample No. 13 - Detailed Boring No. 37

Location of Sample Elev. 20.9 M.G.L. Lock Site No. 2

Remarks Pearl River Project

Date Sample Taken 5-18-38

Date Sample Tested 6-6-38

Specific Gravity of Sample 2.71

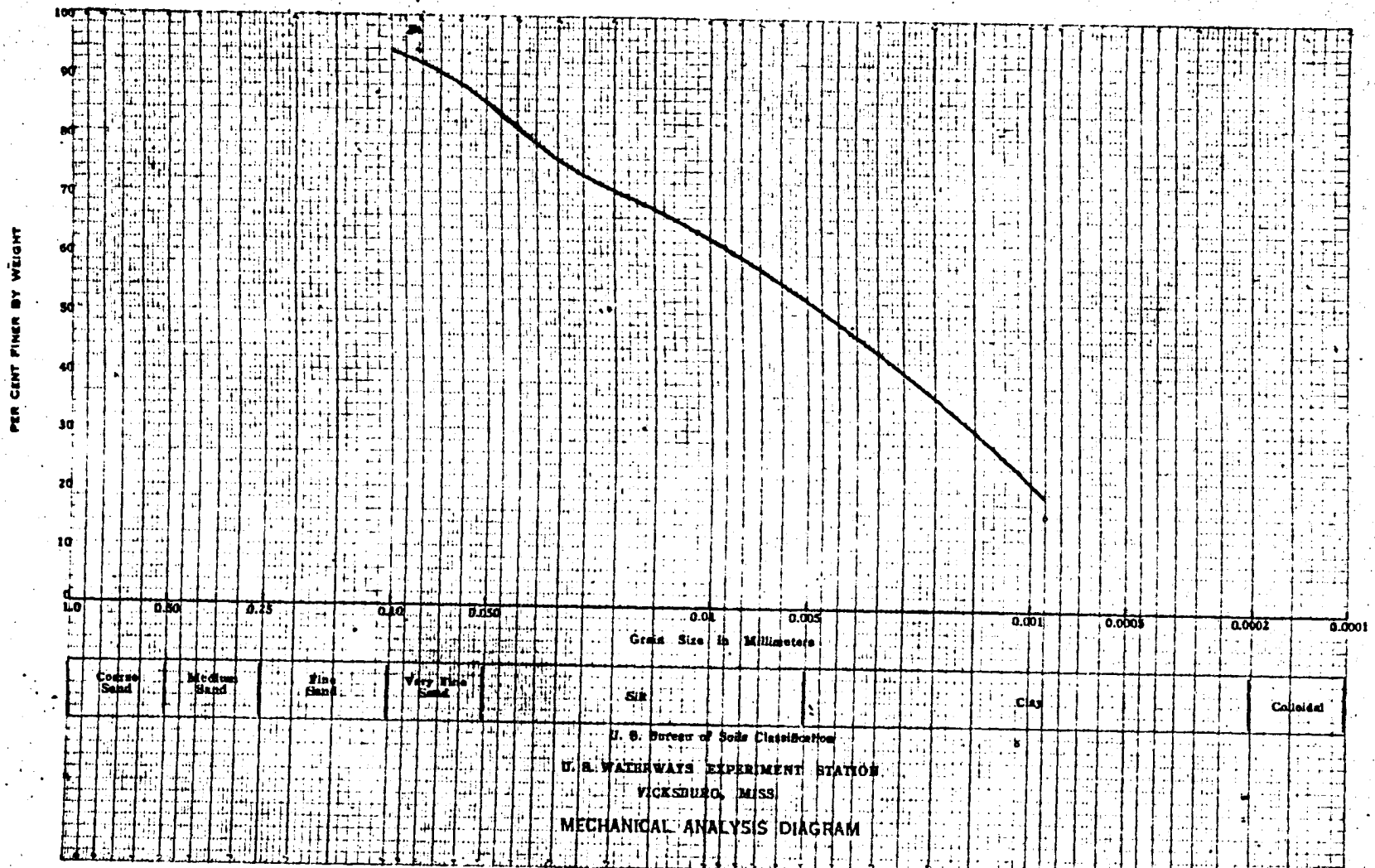


Figure 23

Sample No. 14 - Detailed Boring No. 37

Location of Sample Elev. -28.4 M.G.L. Lock Site No. 2

Remarks Pearl River Project

Date Sample Taken 5-18-38

Date Sample Tested 6-6-38

Specific Gravity of Sample 2.71

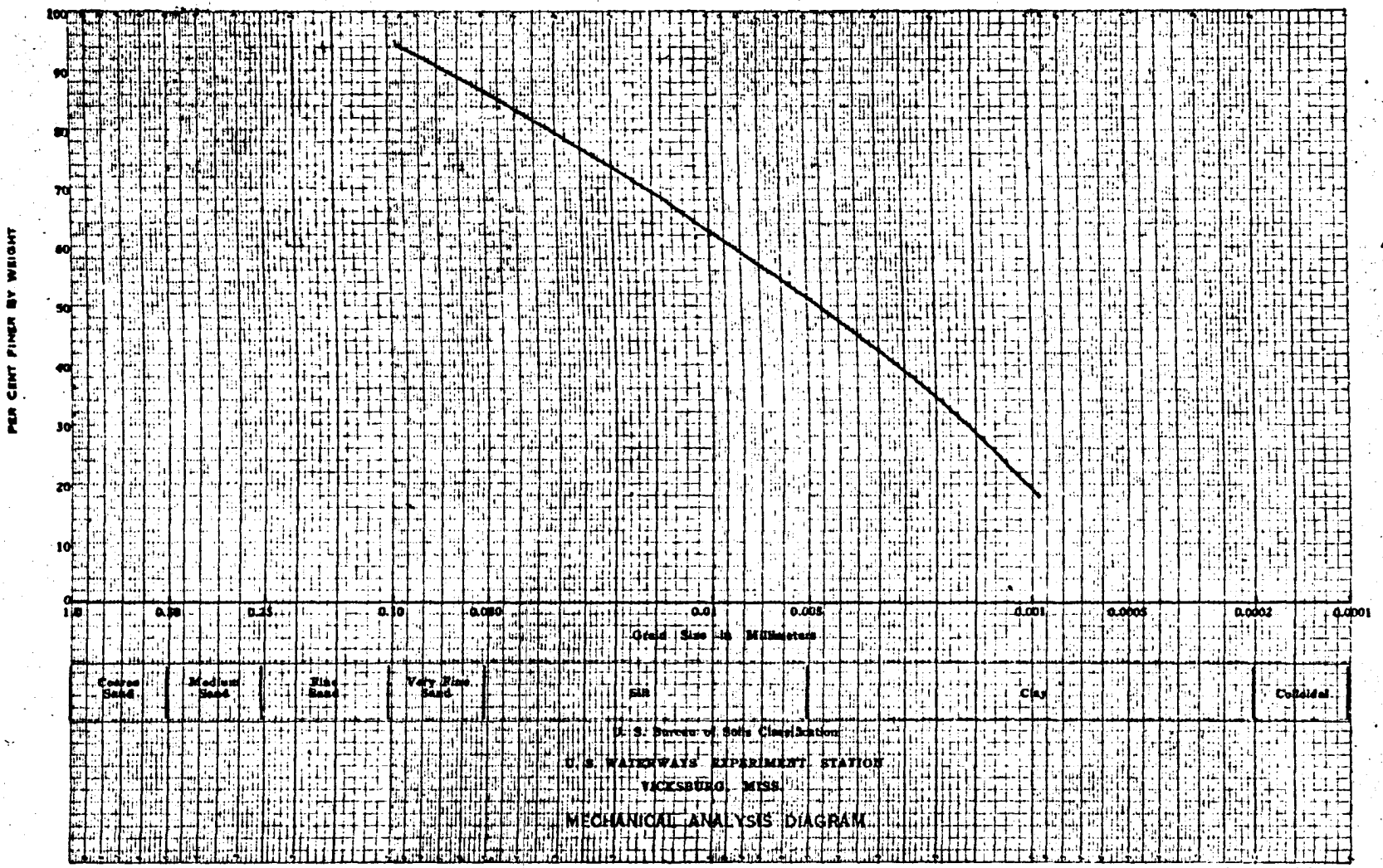


Figure 24

Sample No. 16 - Detailed Boring No. 37

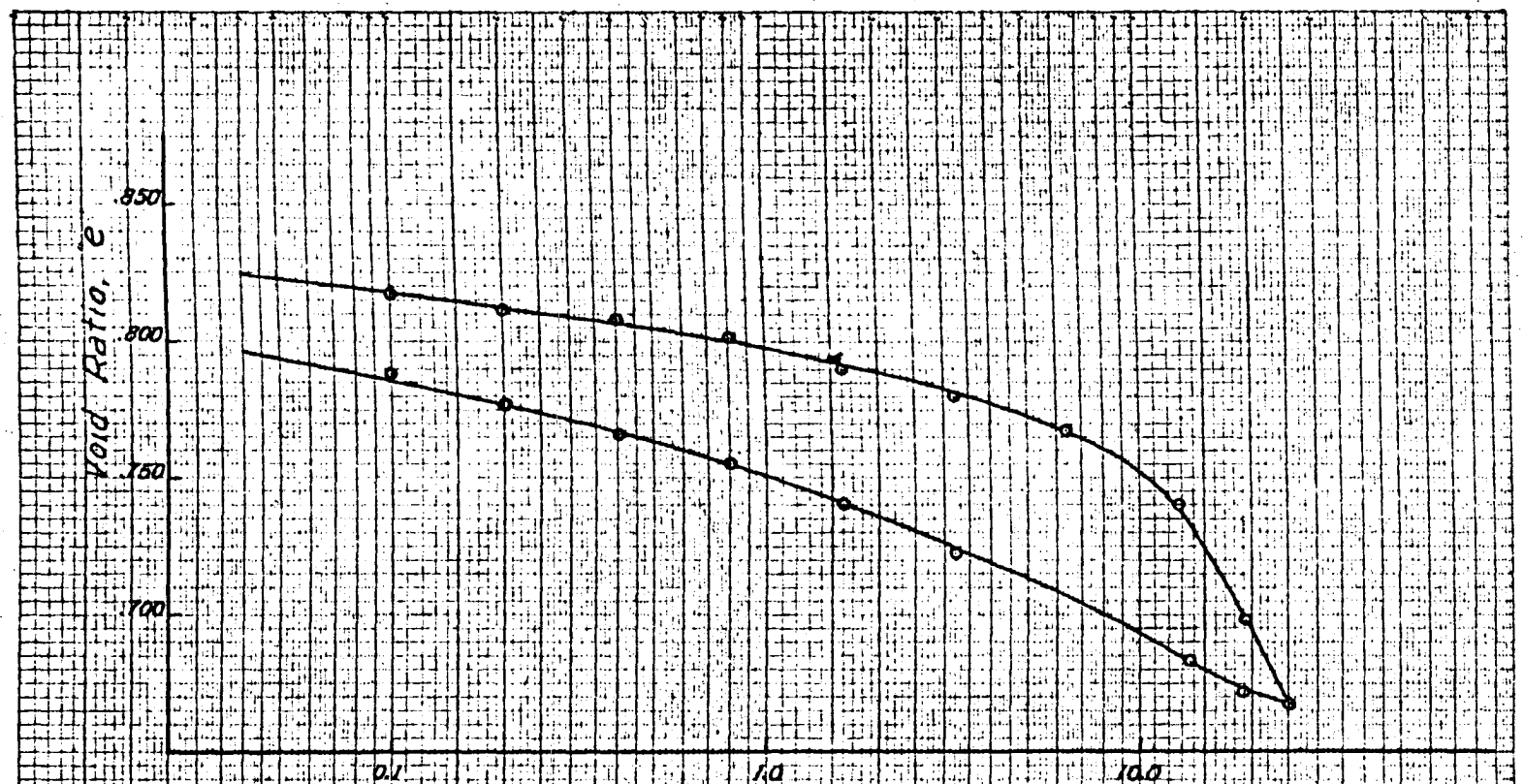
Location of Sample Elev. 40.4 M.G.L., Lock Site No. 2

Remarks Pearl River Project

Date Sample Taken 5-18-38

Date Sample Tested 6-6-38

Specific Gravity of Sample 2.72



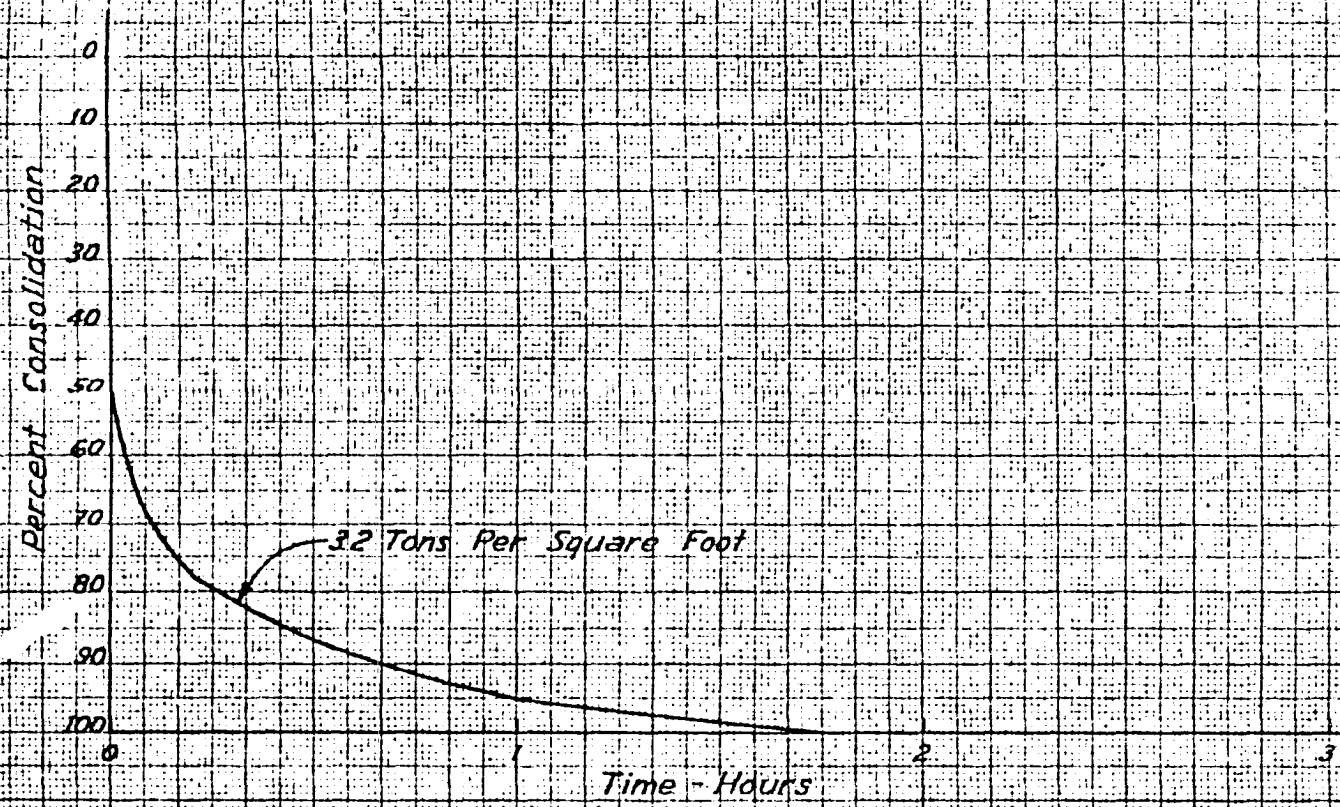
FOUNDATION MATERIAL

PROPOSED LOCK 2, PEARL RIVER IMPROVEMENT

HOLE 15, SAMPLE 16, ELEVATION -14.6 FT. M.G.L. STA. 645+83.0

VOID RATIO - PRESSURE DIAGRAM

FIGURE 25



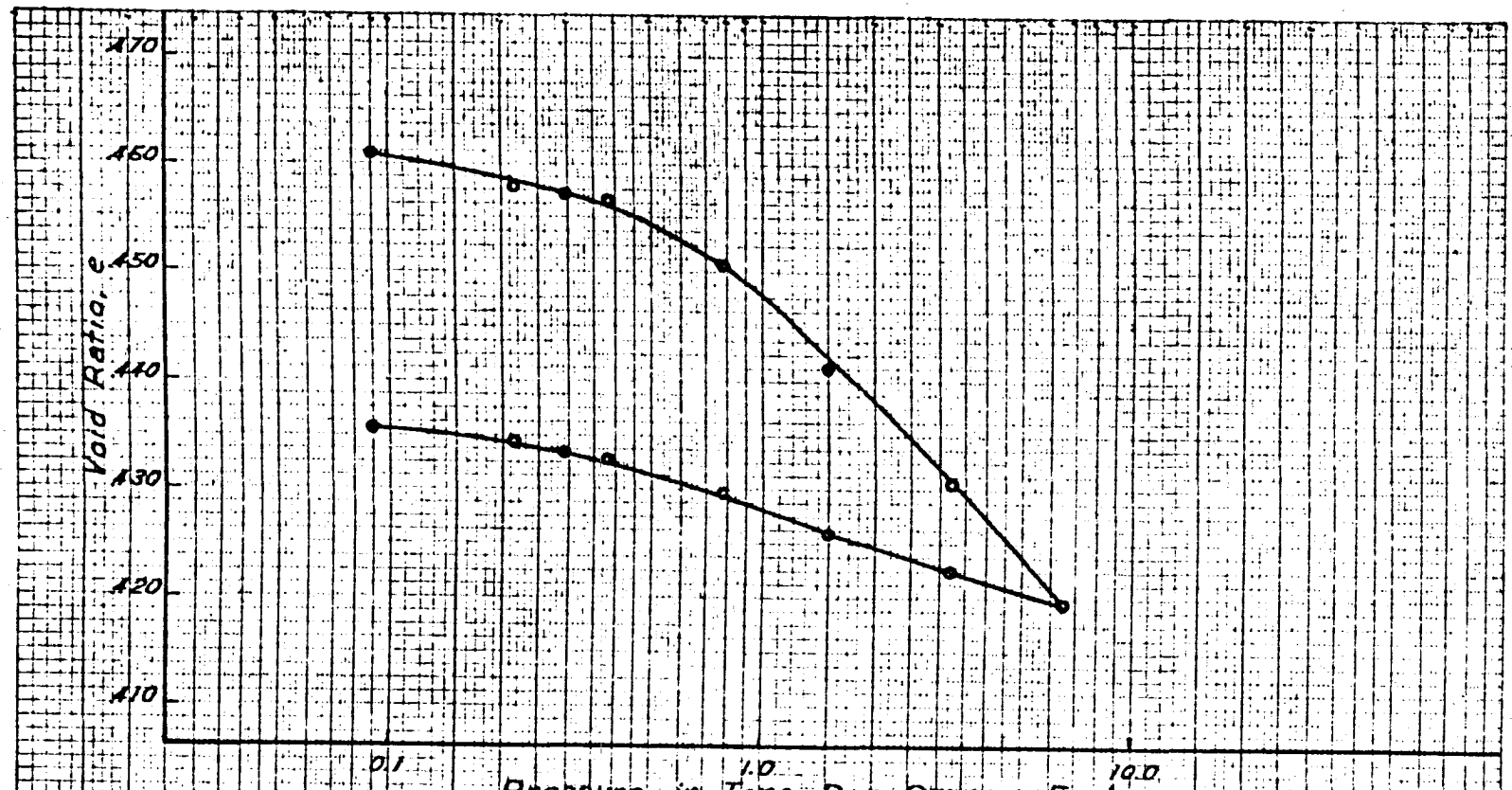
FOUNDATION MATERIAL

PROPOSED LOCK 2, PEARL RIVER IMPROVEMENT

HOLE NO. 15, SAMPLE NO. 16, ELEVATION -14.6 STATION 645+83.0

TIME CONSOLIDATION DIAGRAM

FIGURE 25 a



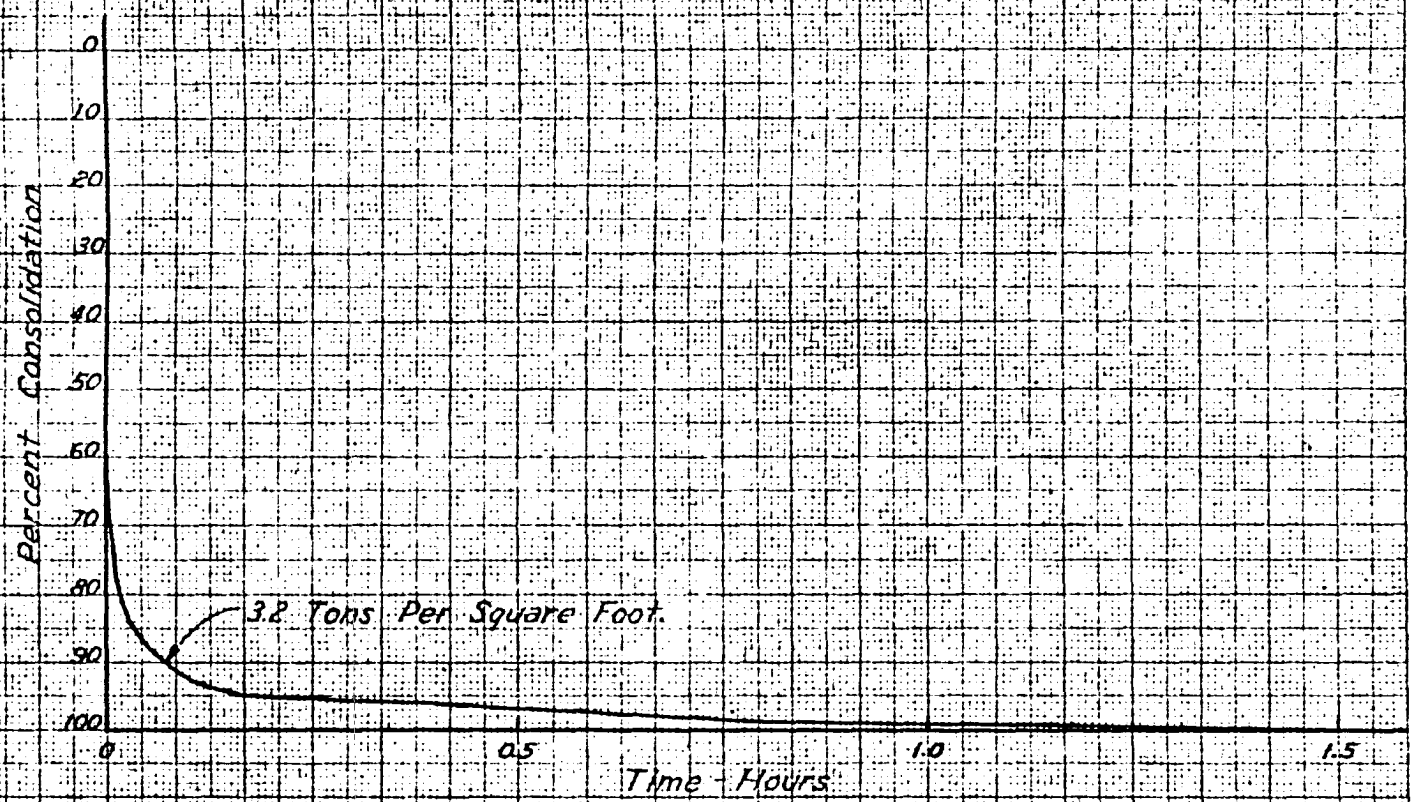
FOUNDATION MATERIAL

PROPOSED LOCK 2, PEARL RIVER IMPROVEMENT

HOLE 15, SAMPLE 17, ELEVATION -16.7 FT. M. G. L. STA. 645+88.0

VOID RATIO - PRESSURE DIAGRAM

FIGURE 26



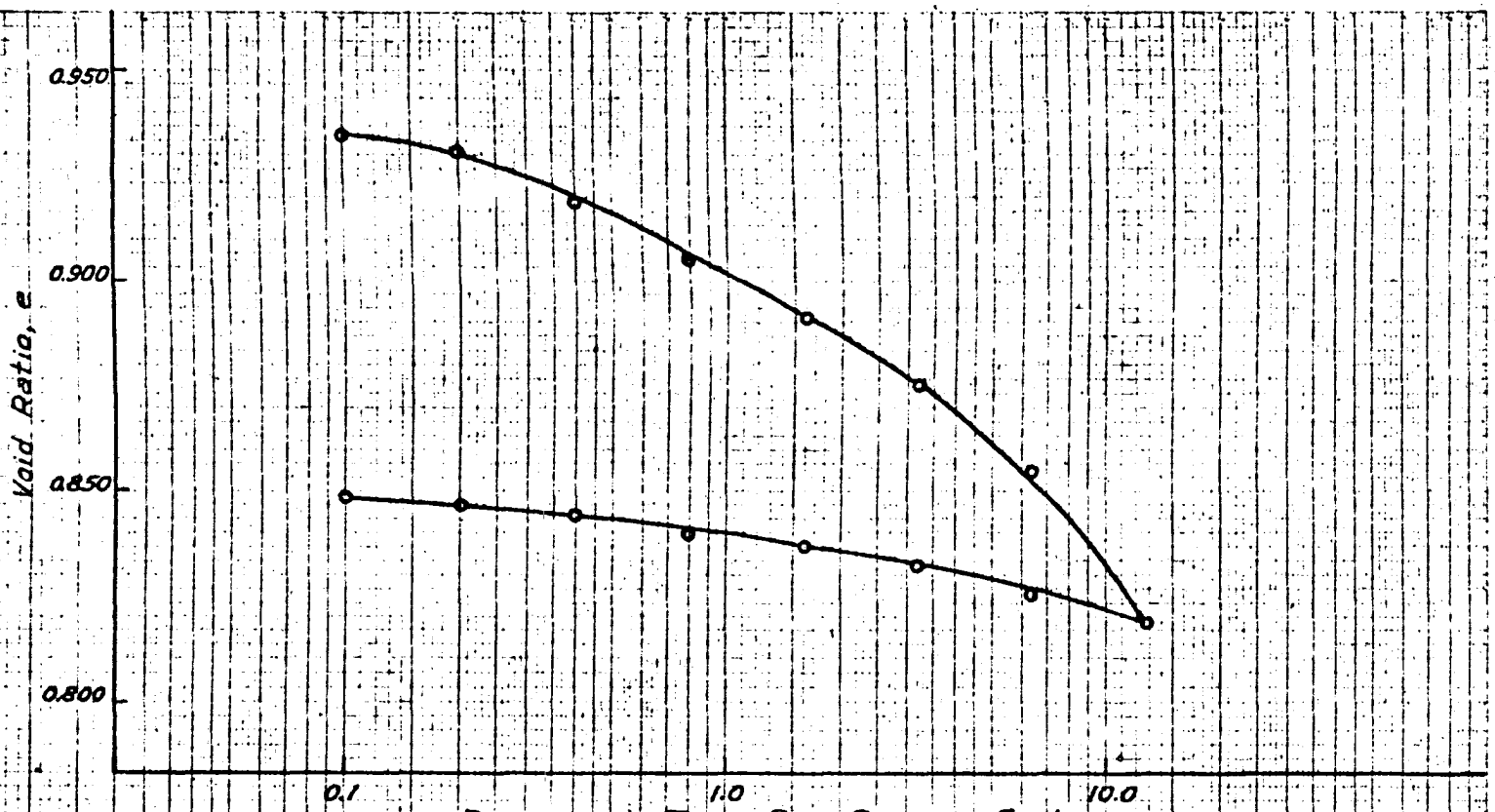
FOUNDATION MATERIAL

PROPOSED LOCK 2, PEARL RIVER IMPROVEMENT

HOLE NO. 15, SAMPLE NO. 17, ELEVATION -16.7, STATION 645+83.0

TIME CONSOLIDATION DIAGRAM

FIGURE 26 a



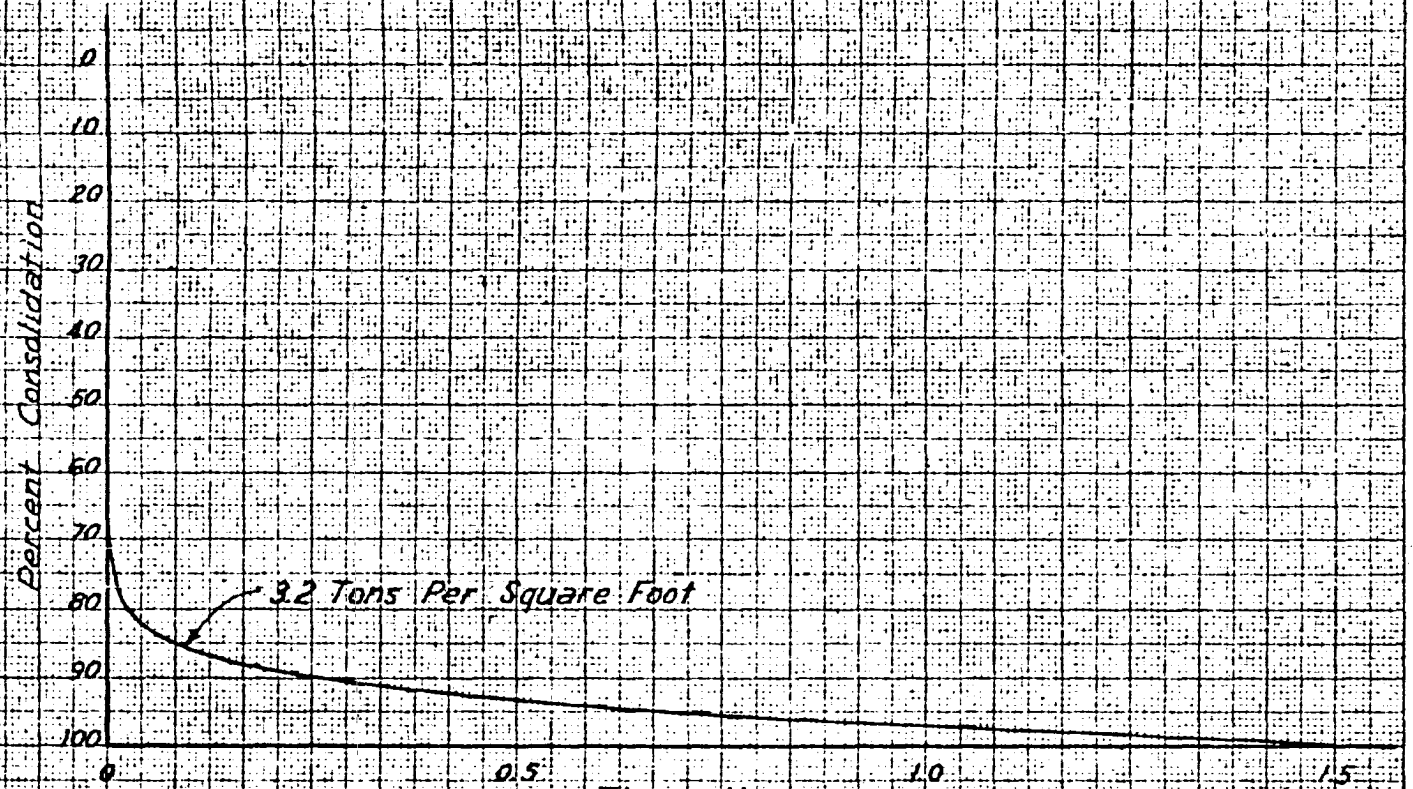
FOUNDATION MATERIAL

PROPOSED LOCK 2, PEARL RIVER IMPROVEMENT

HOLE 15, SAMPLE 22, ELEVATION -427 FT. M.G.L. STA. 645+83.0

VOID RATIO - PRESSURE DIAGRAM

FIGURE 27



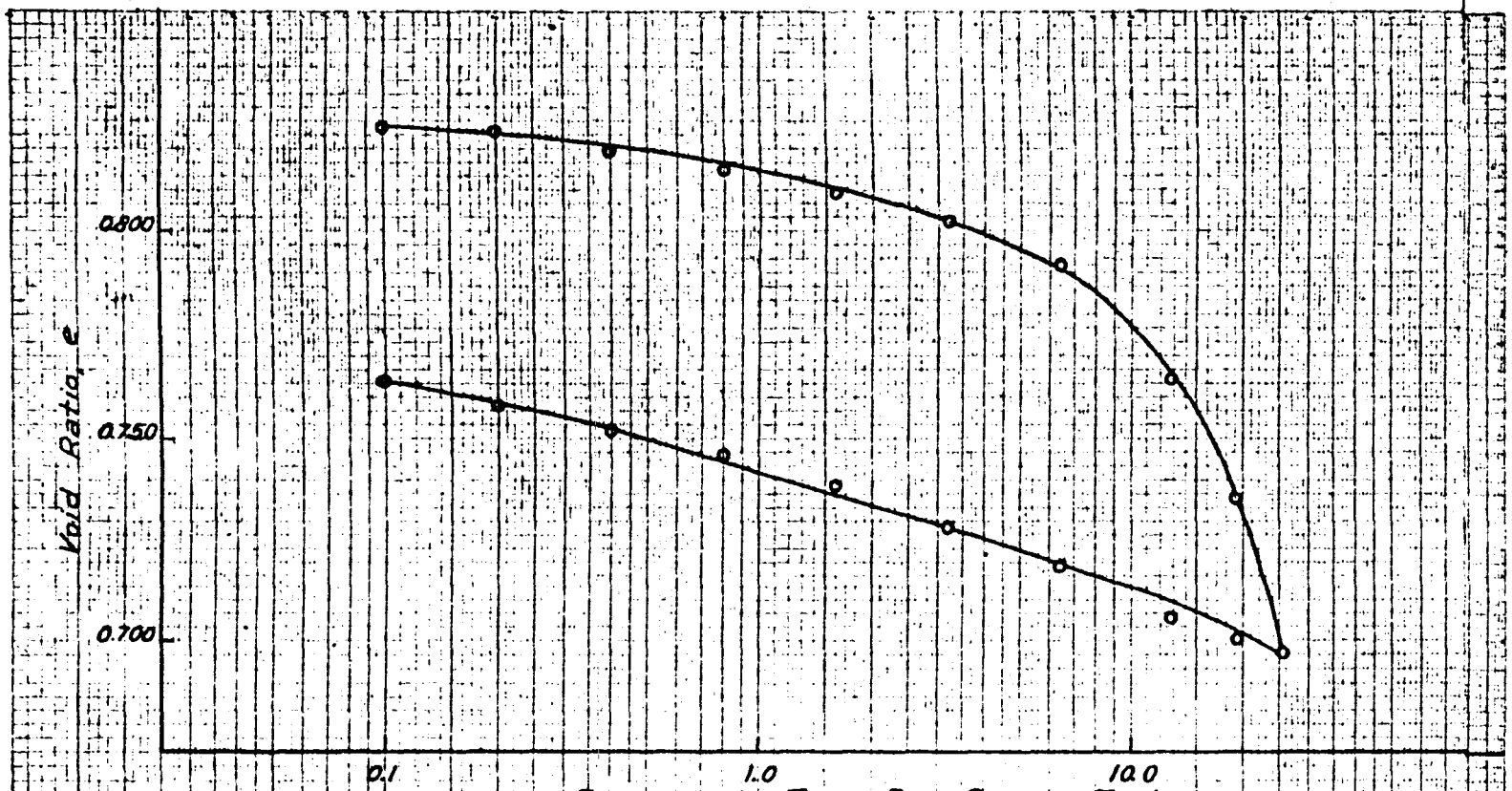
FOUNDATION MATERIAL

PROPOSED LOCK 2, PEARL RIVER IMPROVEMENT

HOLE NO. 15, SAMPLE NO. 22, ELEVATION -42.7, STATION 645+83.0

TIME CONSOLIDATION DIAGRAM

FIGURE 27.a



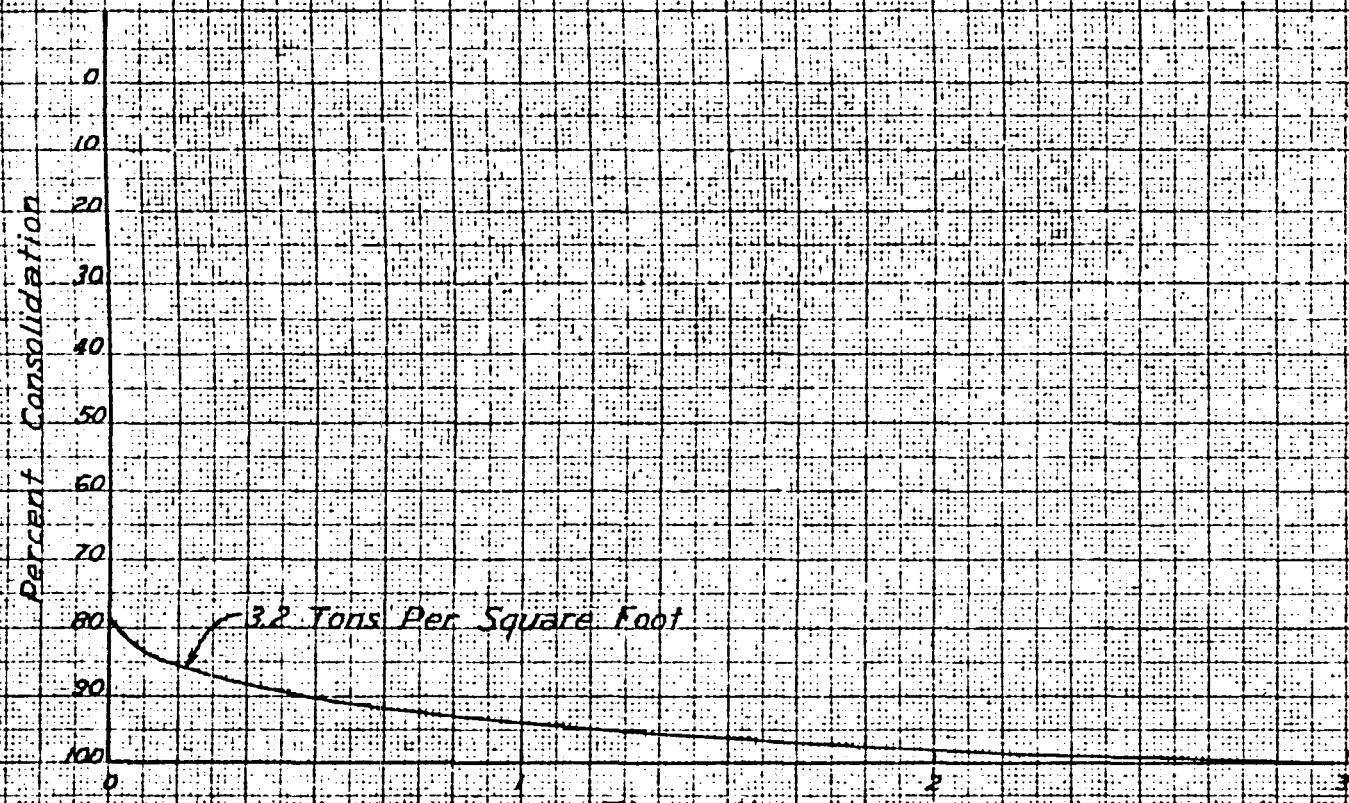
FOUNDATION MATERIAL

PROPOSED LOCK 2, PEARL RIVER IMPROVEMENT

HOLE 15 SAMPLE 24 ELEVATION -55.8 FT. M.G.L. STA. 645+830

VOID RATIO-PRESSURE DIAGRAM

FIGURE 28



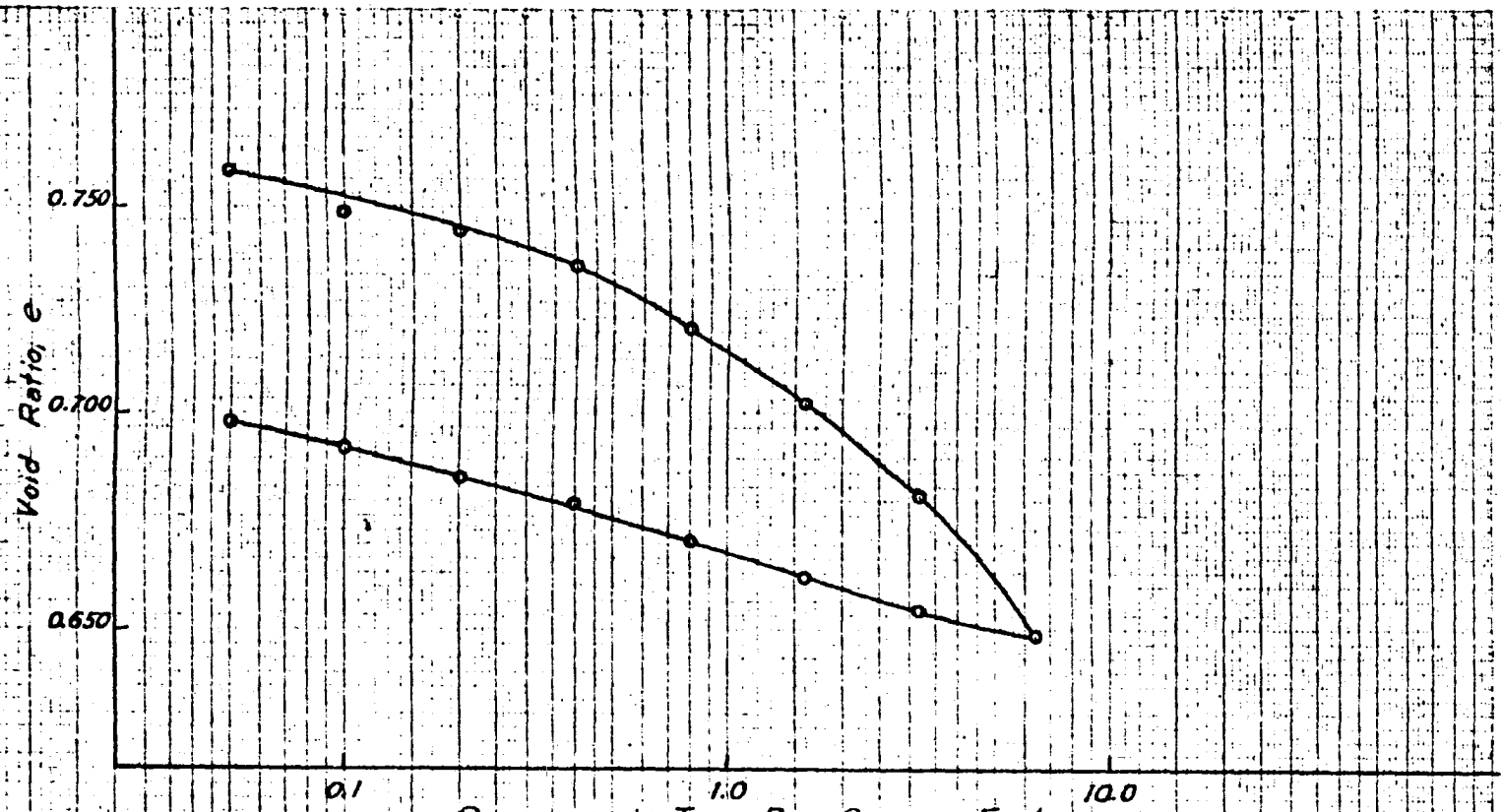
FOUNDATION MATERIAL

PROPOSED LOCK 2, PEARL RIVER IMPROVEMENT

HOLE NO. 15 SAMPLE NO. 24, ELEVATION -55.8 STATION 645+83.0

TIME CONSOLIDATION DIAGRAM

FIGURE 28a



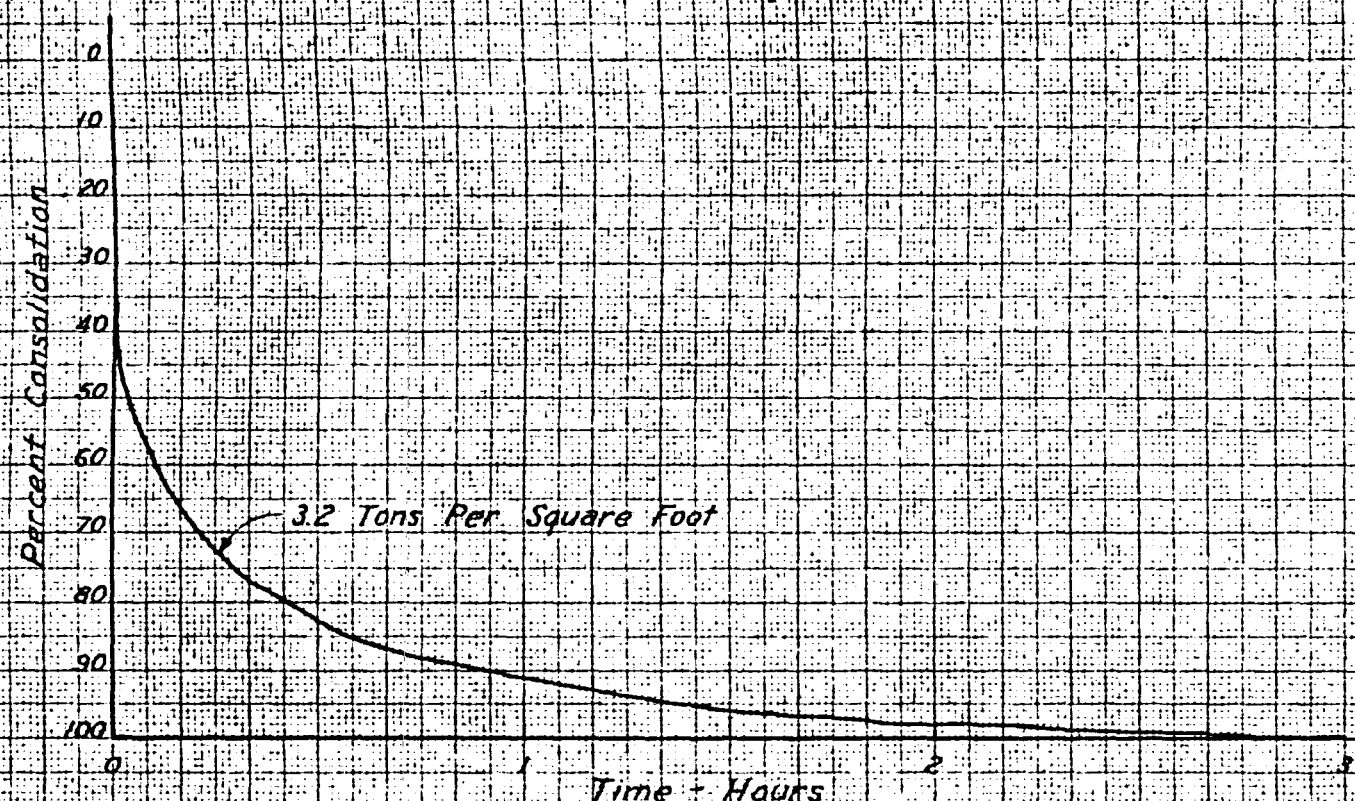
FOUNDATION MATERIAL

PROPOSED LOCK 2, PEARL RIVER IMPROVEMENT

HOLE 15, SAMPLE 25, ELEVATION -61.1 FT. M. G. L., STA. 645+83.0

VOID RATIO - PRESSURE DIAGRAM

FIGURE 29



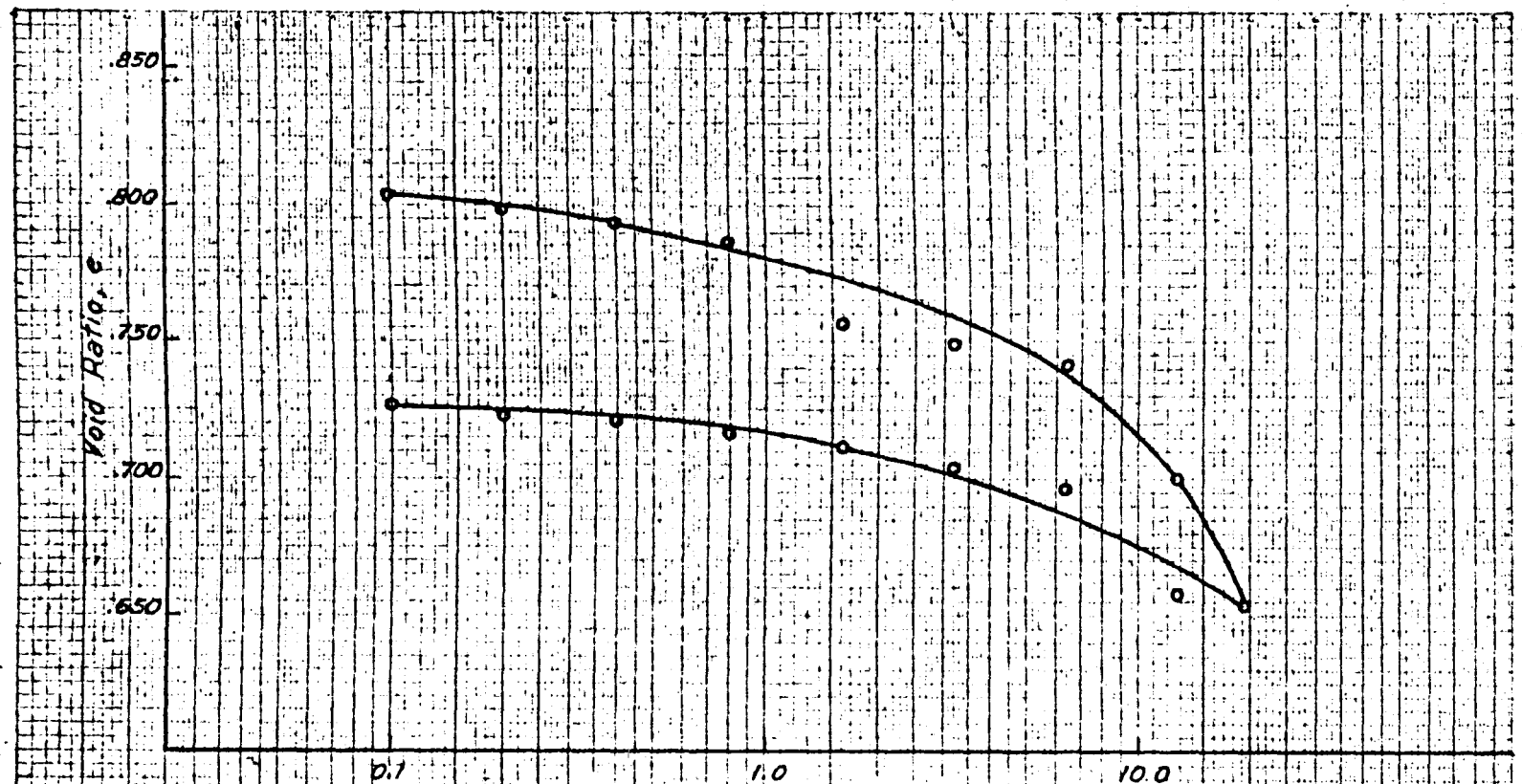
FOUNDATION MATERIAL

PROPOSED LOCK 2, PEARL RIVER IMPROVEMENT

HOLE NO. 15, SAMPLE NO. 25, ELEVATION -6.1, STATION 645+83.0

TIME CONSOLIDATION DIAGRAM

FIGURE 29 a



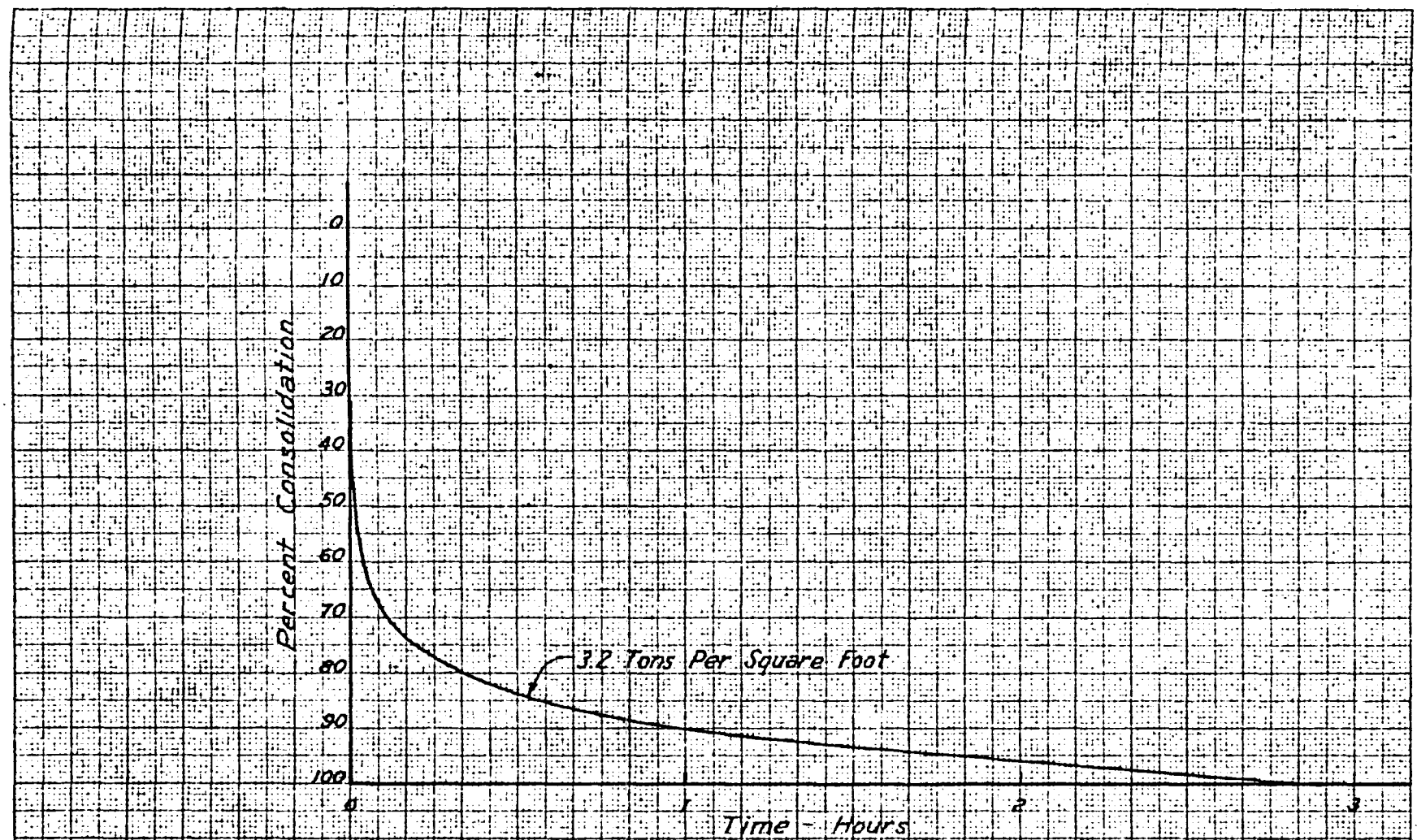
FOUNDATION MATERIAL

PROPOSED LOCK 2, PEARL RIVER IMPROVEMENT

HOLE 16, SAMPLE 15, ELEVATION -23.3 FT. M. G. L. STA. 649+63.0

VOID RATIO - PRESSURE DIAGRAM

FIGURE 30



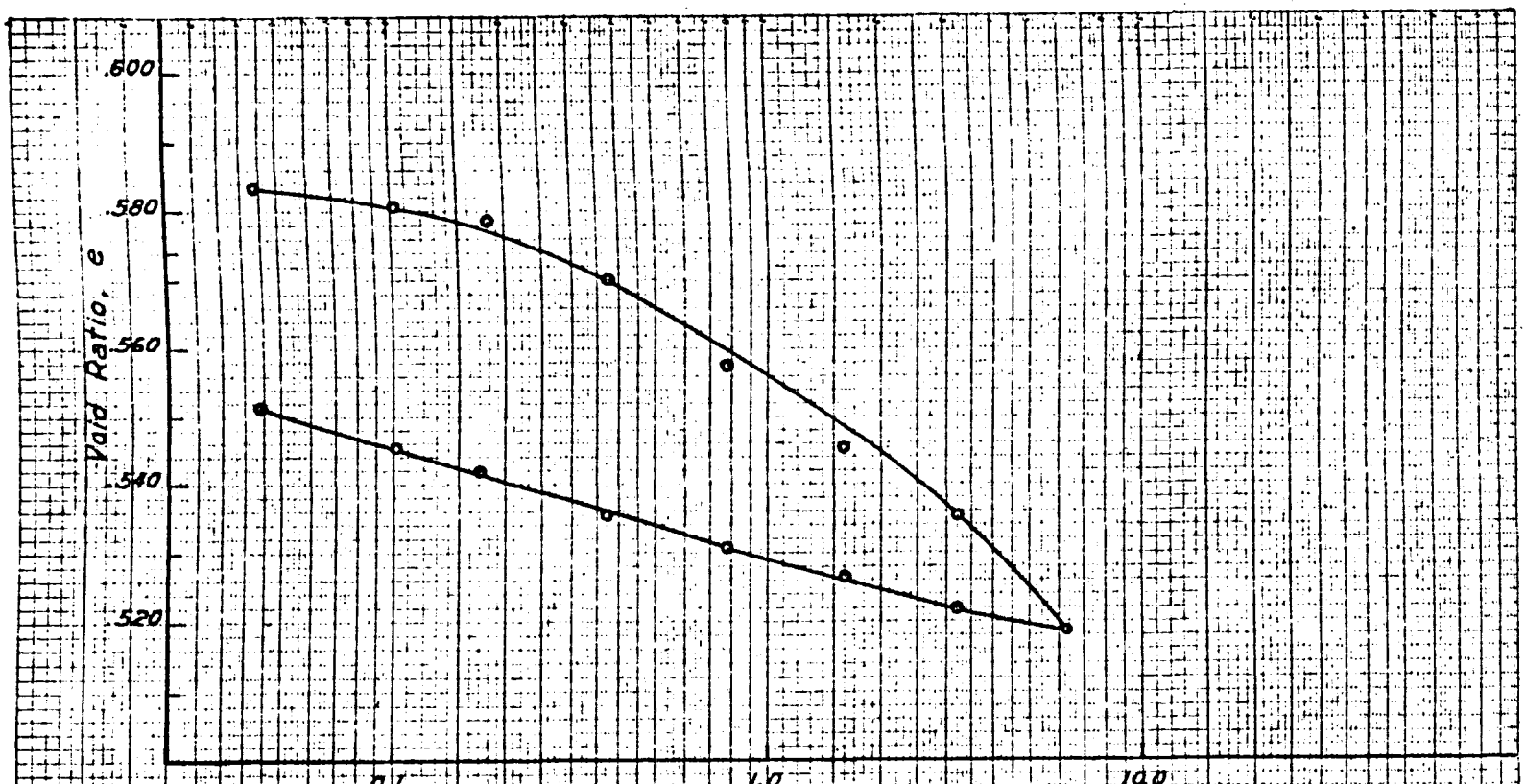
FOUNDATION MATERIAL

PROPOSED LOCK 2, PEARL RIVER IMPROVEMENT

HOLE NO. 14, SAMPLE NO. 15, ELEVATION -23.3, STATION 649+63.0

TIME CONSOLIDATION DIAGRAM

FIGURE 30 a



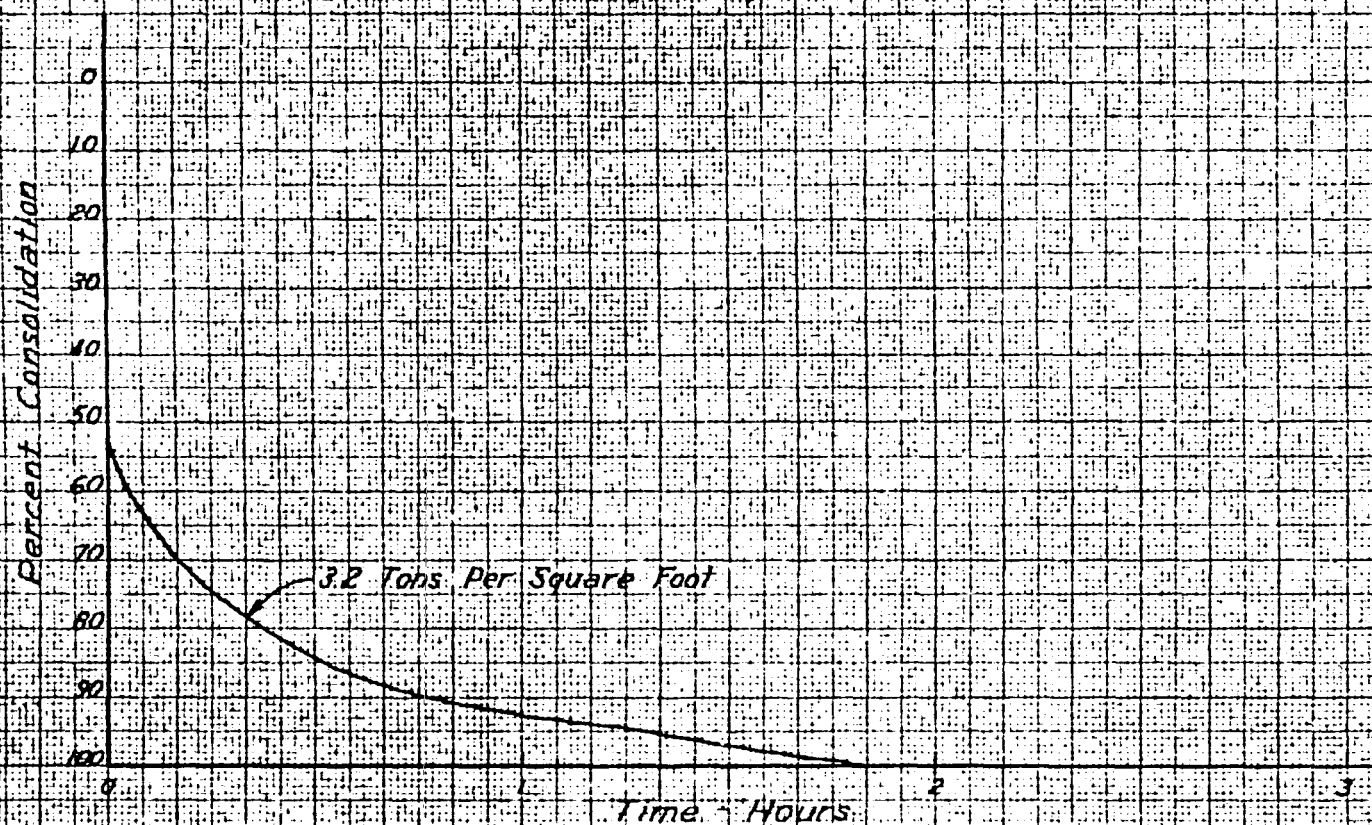
FOUNDATION MATERIAL

PROPOSED LOCK 2, PEARL RIVER IMPROVEMENT

HOLE 16, SAMPLE 16, ELEVATION -26.7 FT. M.G.L. STA 649+630

VOID RATIO - PRESSURE DIAGRAM

FIGURE 31



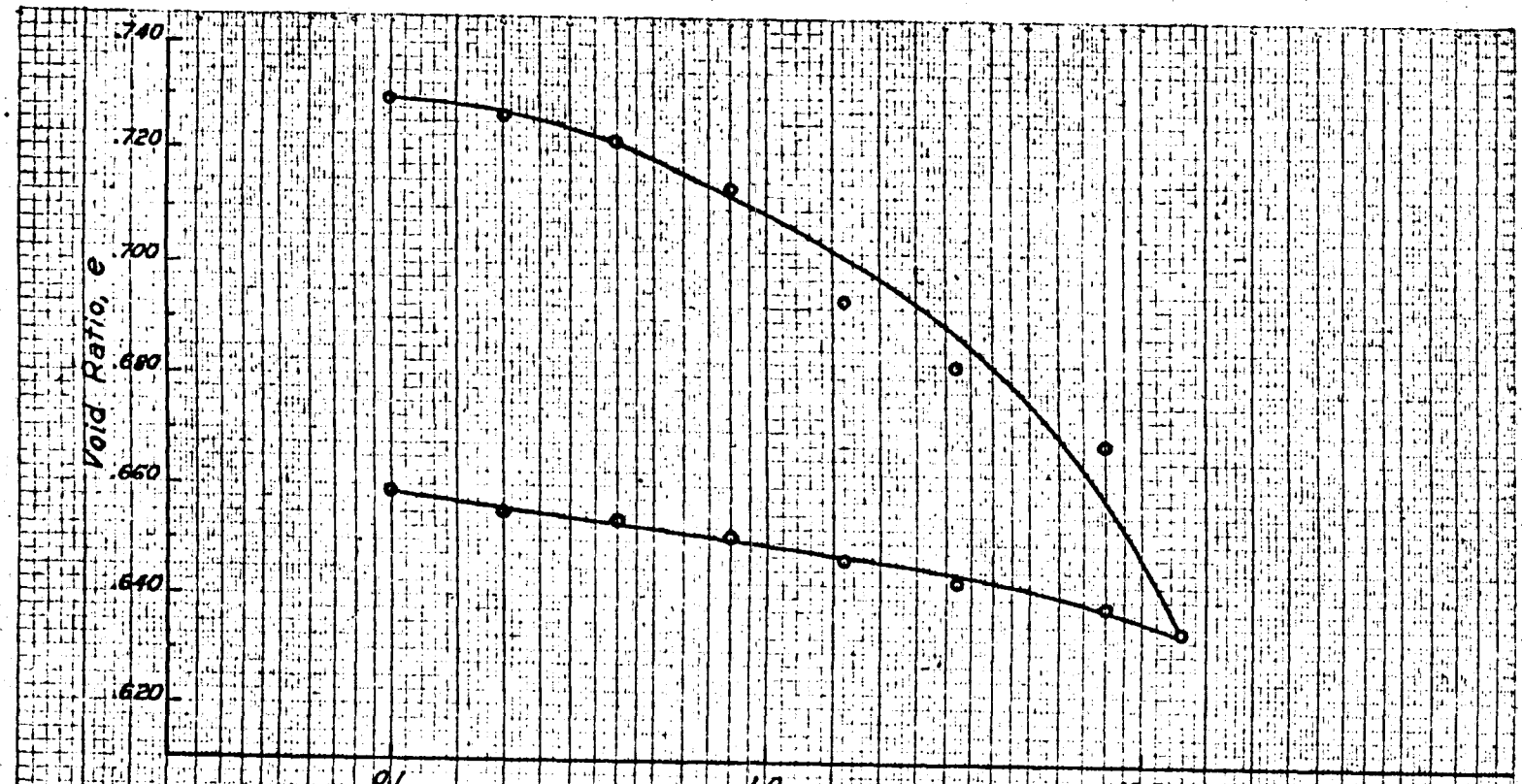
FOUNDATION MATERIAL

PROPOSED LOCK 2, PEARL RIVER IMPROVEMENT

HOLE NO. 16, SAMPLE NO. 16, ELEVATION -26.7, STATION 649+63.0

TIME CONSOLIDATION DIAGRAM

FIGURE 31a



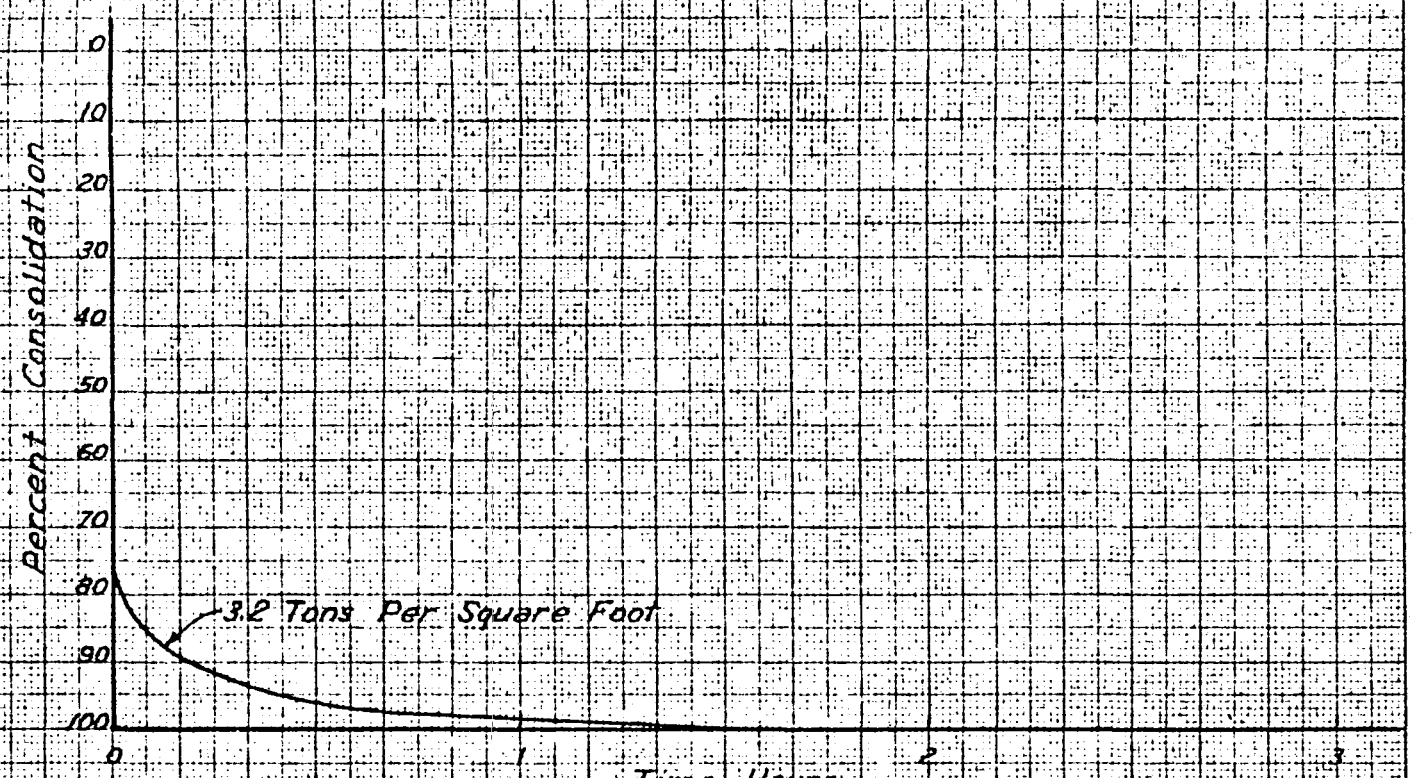
FOUNDATION MATERIAL

PROPOSED LOCK 2, PEARL RIVER IMPROVEMENT

HOLE 16, SAMPLE 17, ELEVATION -31.3 FT. M. G. L. STA. 549+63.0

VOID RATIO - PRESSURE DIAGRAM

FIGURE 32



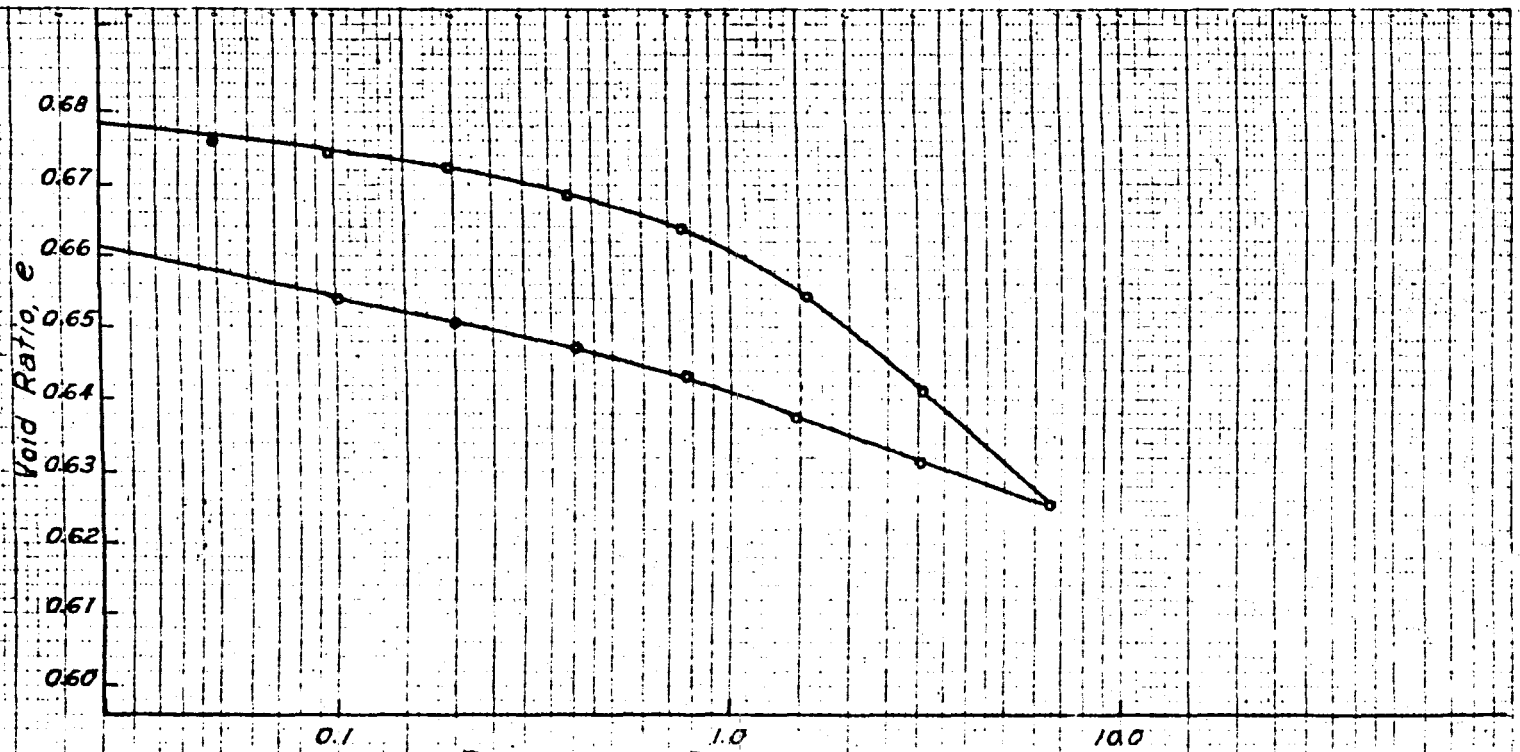
FOUNDATION MATERIAL

PROPOSED LOCK 2, PEARL RIVER IMPROVEMENT

HOLE 16, SAMPLE 17, ELEVATION -31.3 STATION 649+63.0

TIME CONSOLIDATION DIAGRAM

FIGURE 32 a



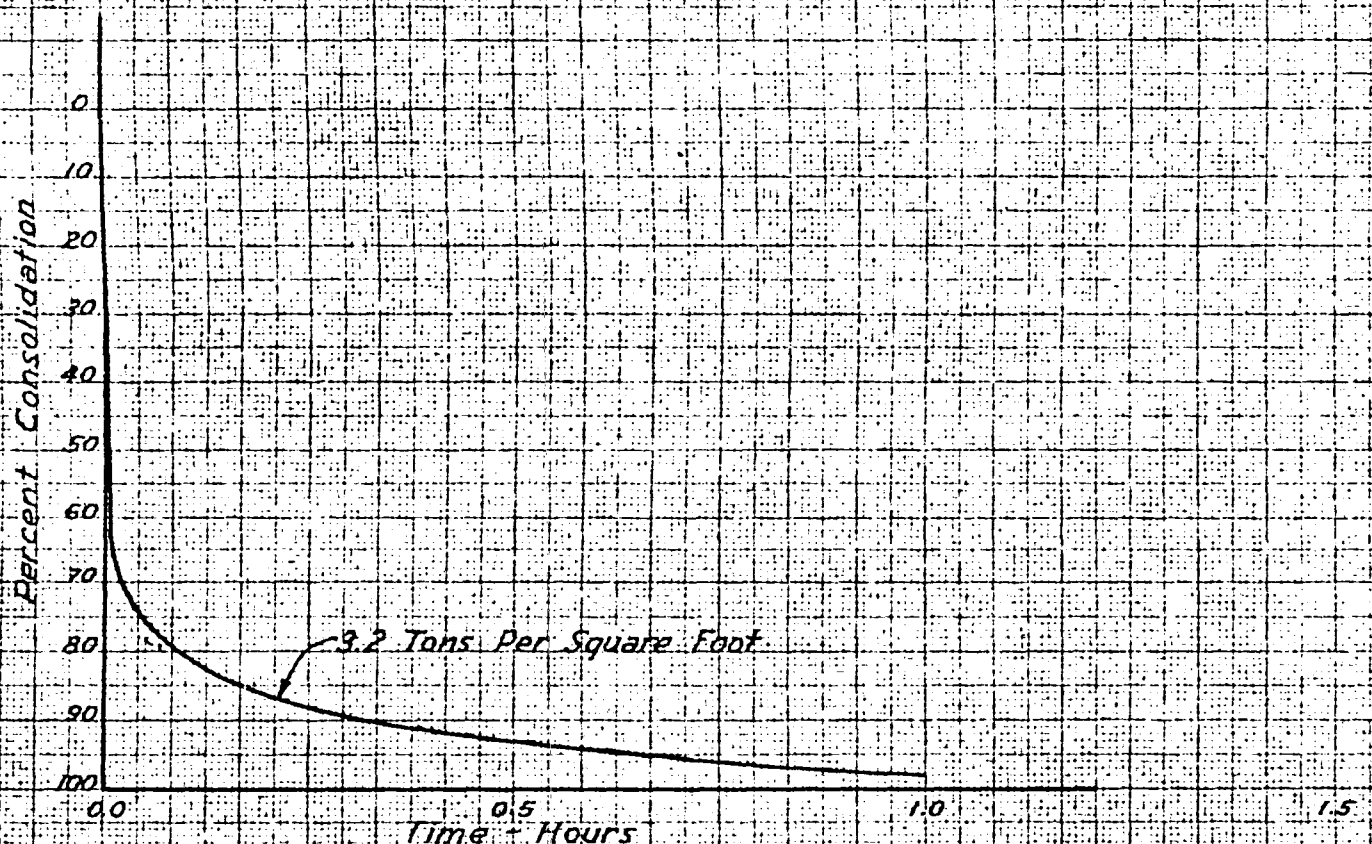
FOUNDATION MATERIAL

PROPOSED LOCK 2, PEARL RIVER IMPROVEMENT

HOLE 37 SAMPLE 13 ELEVATION -20.9 FT. M.G.L. STA. 652+01

VOID RATIO - PRESSURE DIAGRAM

FIGURE 33



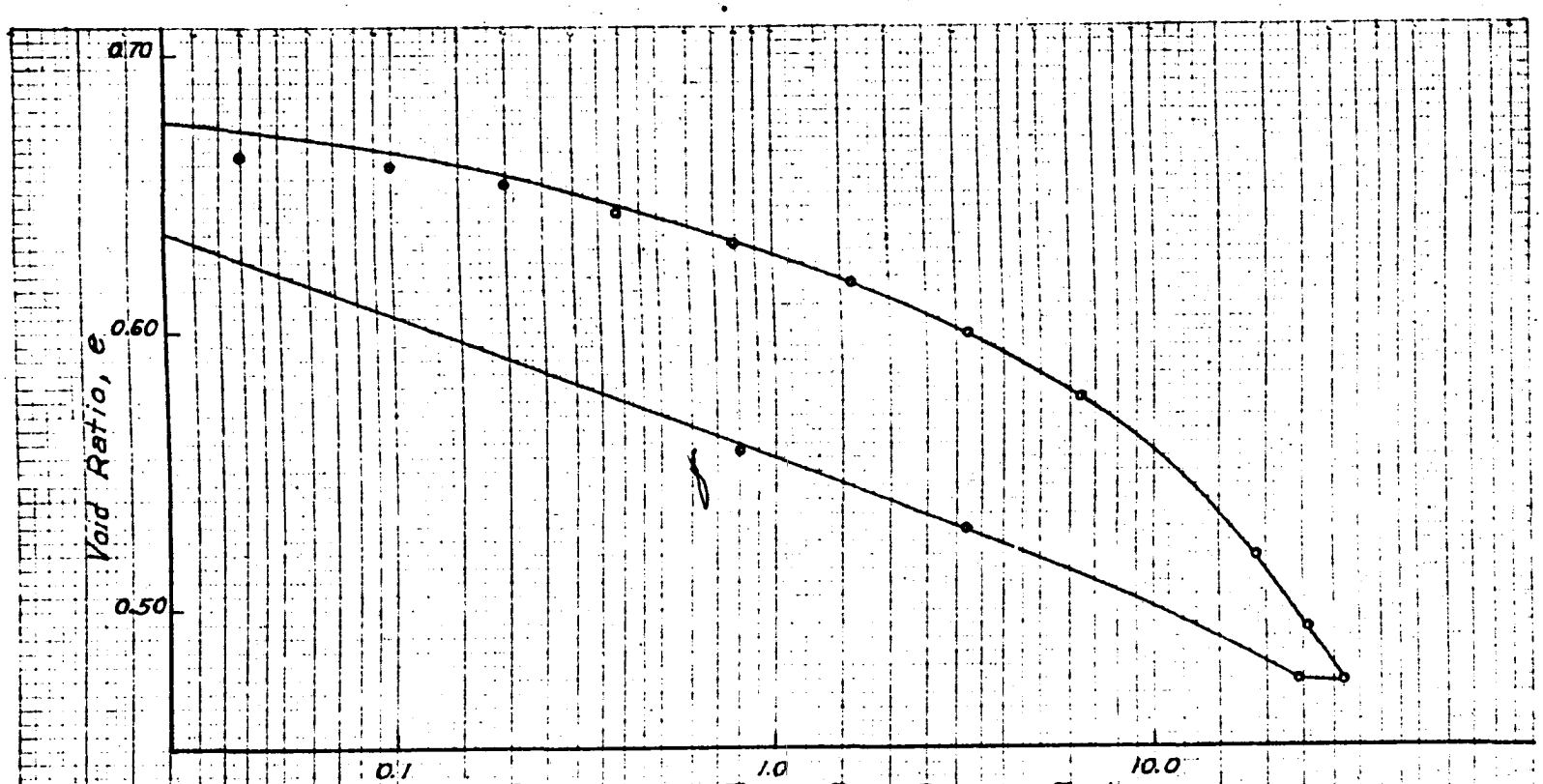
FOUNDATION MATERIAL

PROPOSED LOCK 2, PEARL RIVER IMPROVEMENT

HOLE 97 SAMPLE 13 ELEVATION -20.9 FT. M.G.L. STA. 652+01

TIME CONSOLIDATION DIAGRAM

FIGURE 33 a



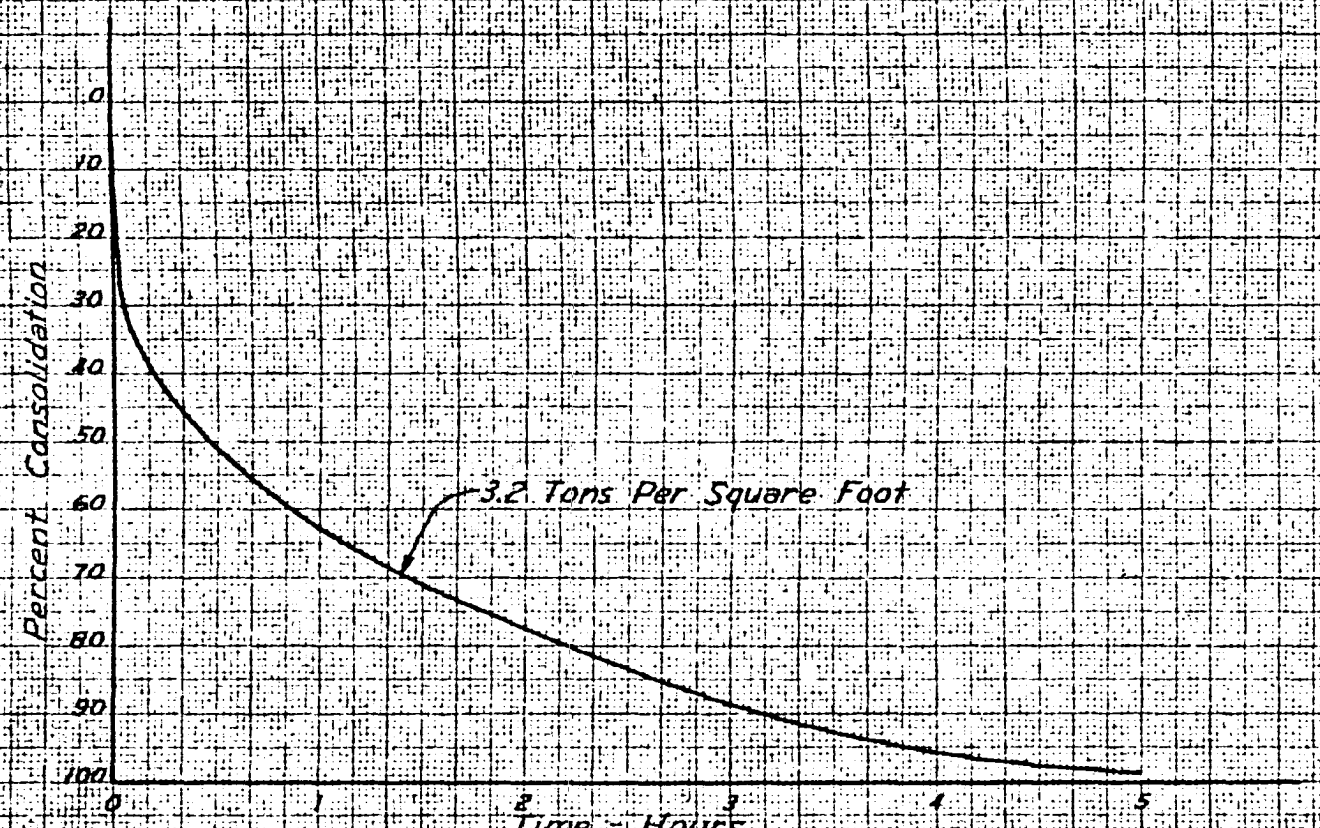
FOUNDATION MATERIAL

PROPOSED LOCK 2, PEARL RIVER IMPROVEMENT

HOLE 37, SAMPLE 16 ELEVATION -404 FT. M. G. L. STA. 652+01

VOID RATIO - PRESSURE DIAGRAM

FIGURE 34



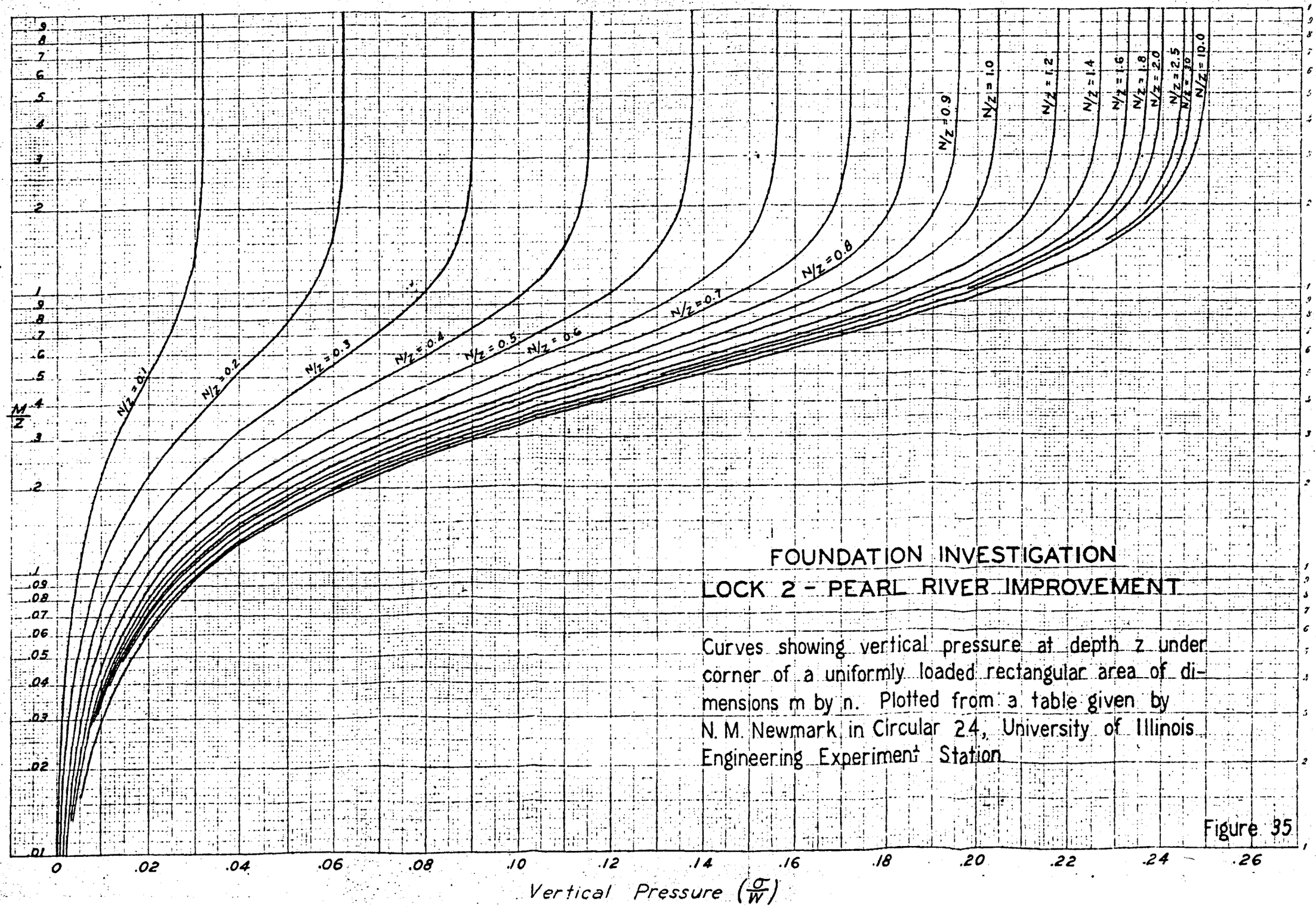
FOUNDATION MATERIAL

PROPOSED LOCK 2, PEARL RIVER IMPROVEMENT

HOLE 37 SAMPLE 16 ELEVATION -40.4 FT. M.G.L. STA. 652+01

TIME CONSOLIDATION DIAGRAM

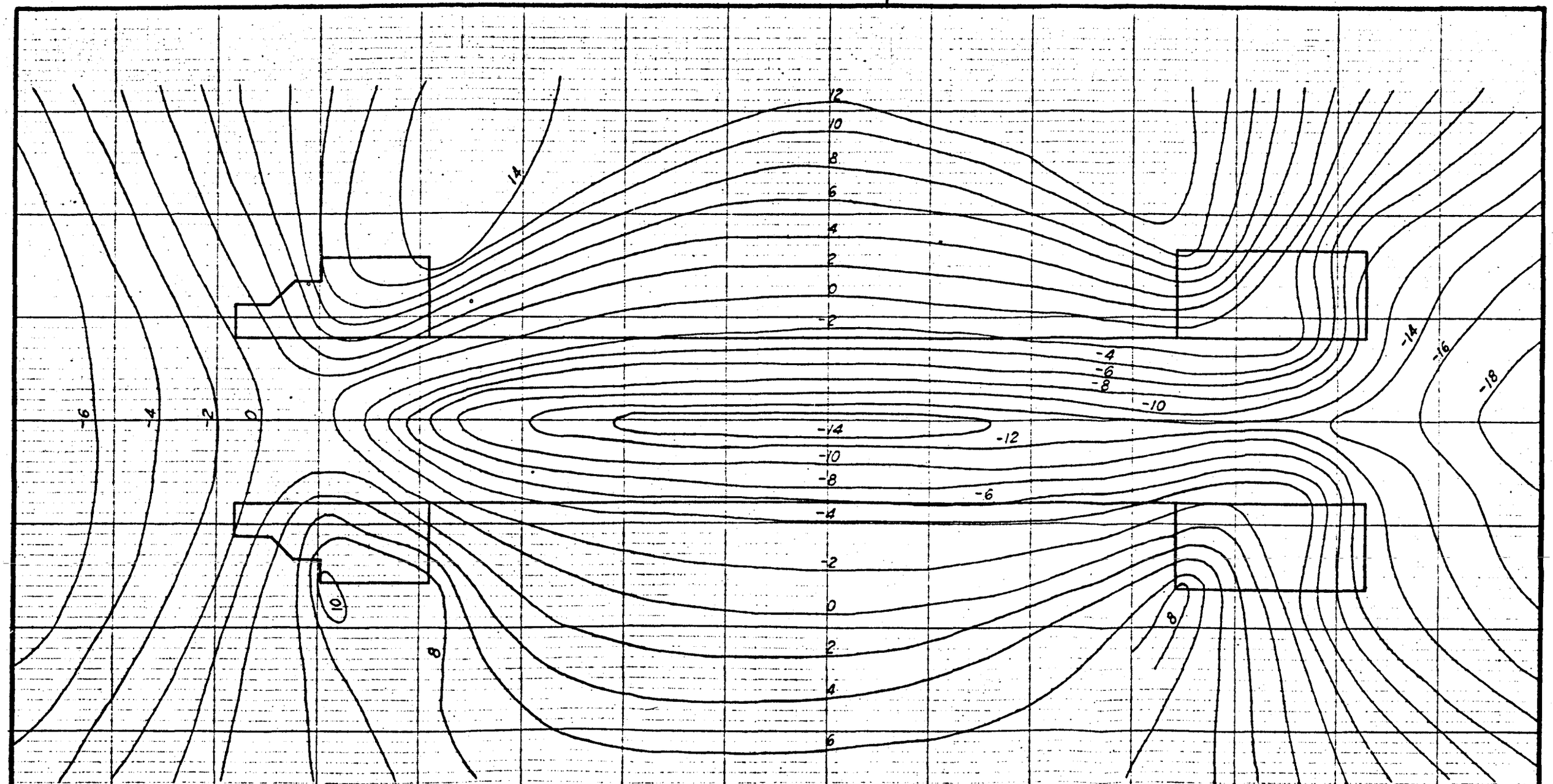
FIGURE 34 B



FOUNDATION INVESTIGATION
LOCK 2 - PEARL RIVER IMPROVEMENT

Curves showing vertical pressure at depth z under corner of a uniformly loaded rectangular area of dimensions m by n. Plotted from a table given by N. M. Newmark in Circular 24, University of Illinois Engineering Experiment Station.

Figure 35

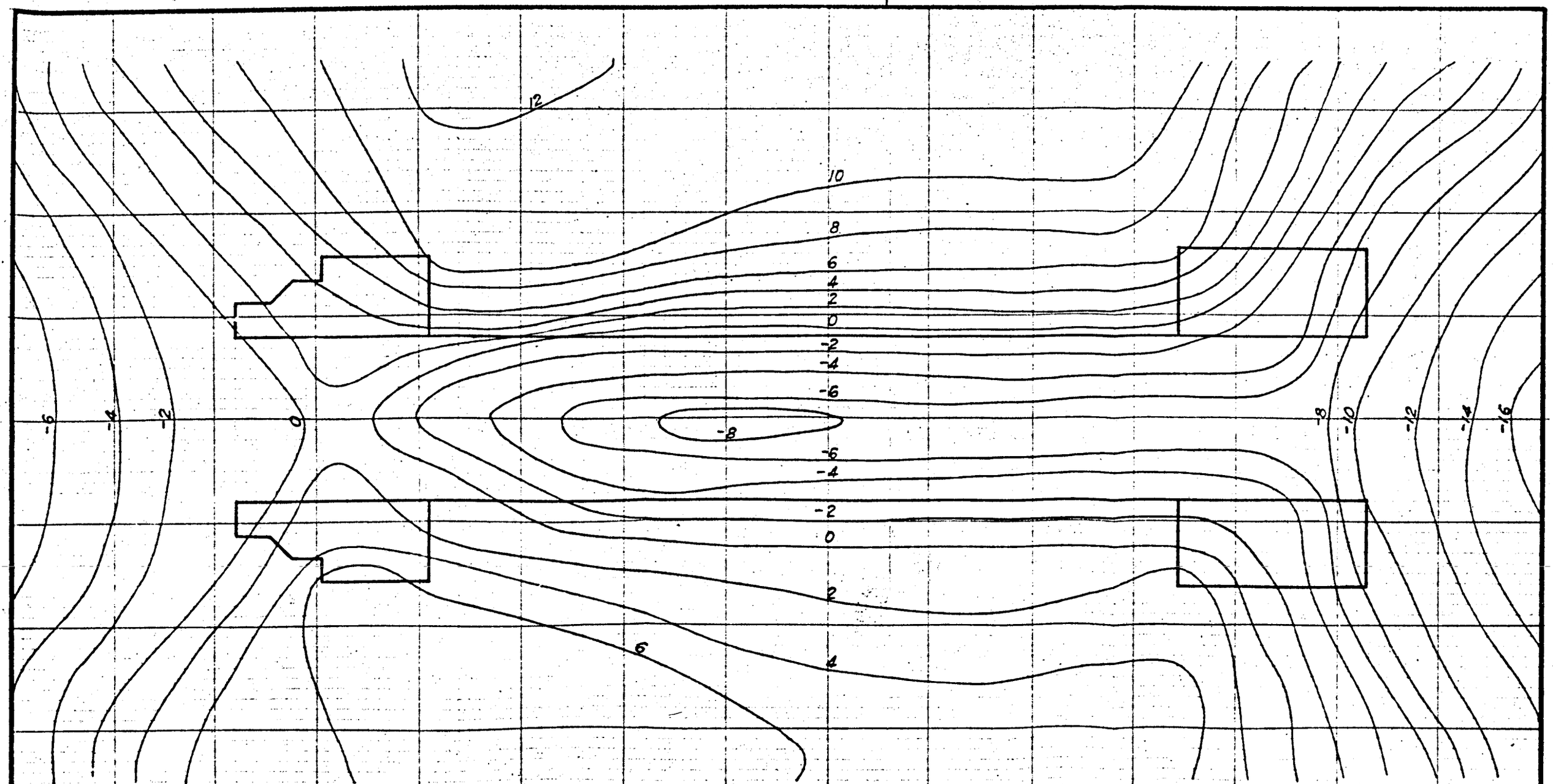


FOUNDATION INVESTIGATION
LOCK 2, PEARL RIVER IMPROVEMENT

Net change in foundation stress on horizontal section at elevation -15 for case II design assumption. Effect of sheet piling neglected.

SCALE 1"=40' STRESS IN HUNDREDS OF POUNDS

FIGURE 36



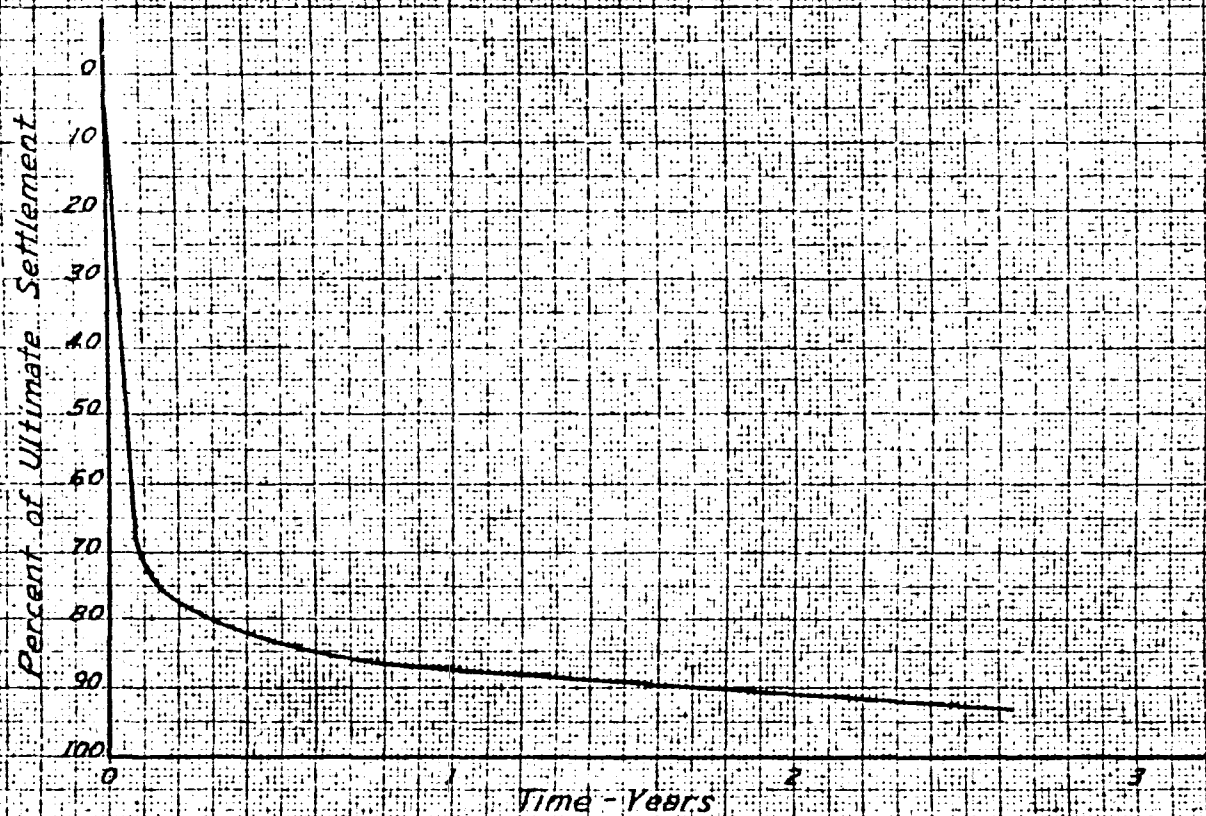
FOUNDATION INVESTIGATION

LOCK 2, PEARL RIVER IMPROVEMENT

Net change in foundation stress on horizontal section at elevation −40 for case II design assumption. Effect of sheet piling neglected.

SCALE 1"=40' STRESS IN HUNDREDS OF POUNDS

FIGURE 37



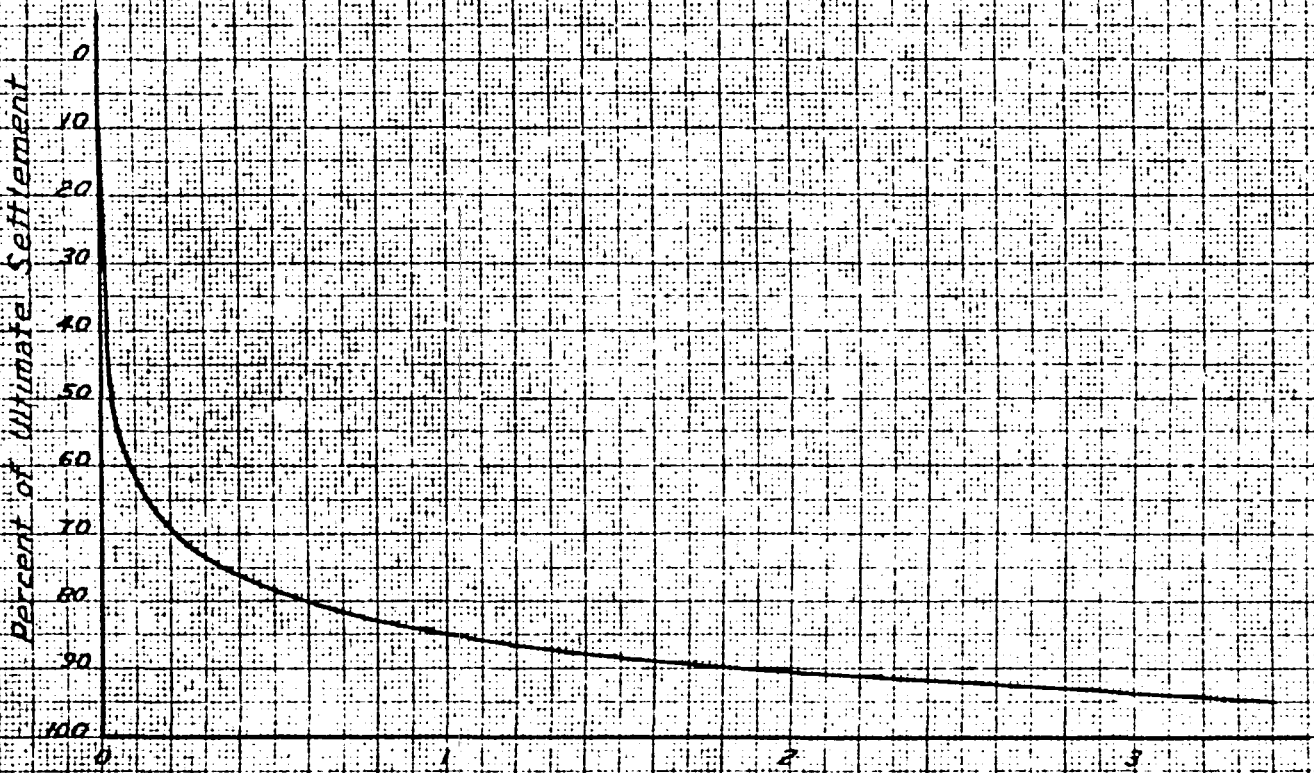
FOUNDATION INVESTIGATION

PROPOSED LOCK 2, PEARL RIVER IMPROVEMENT

LOAD AT HOLE 15 STATION 645+83

TIME SETTLEMENT DIAGRAM

FIGURE 38



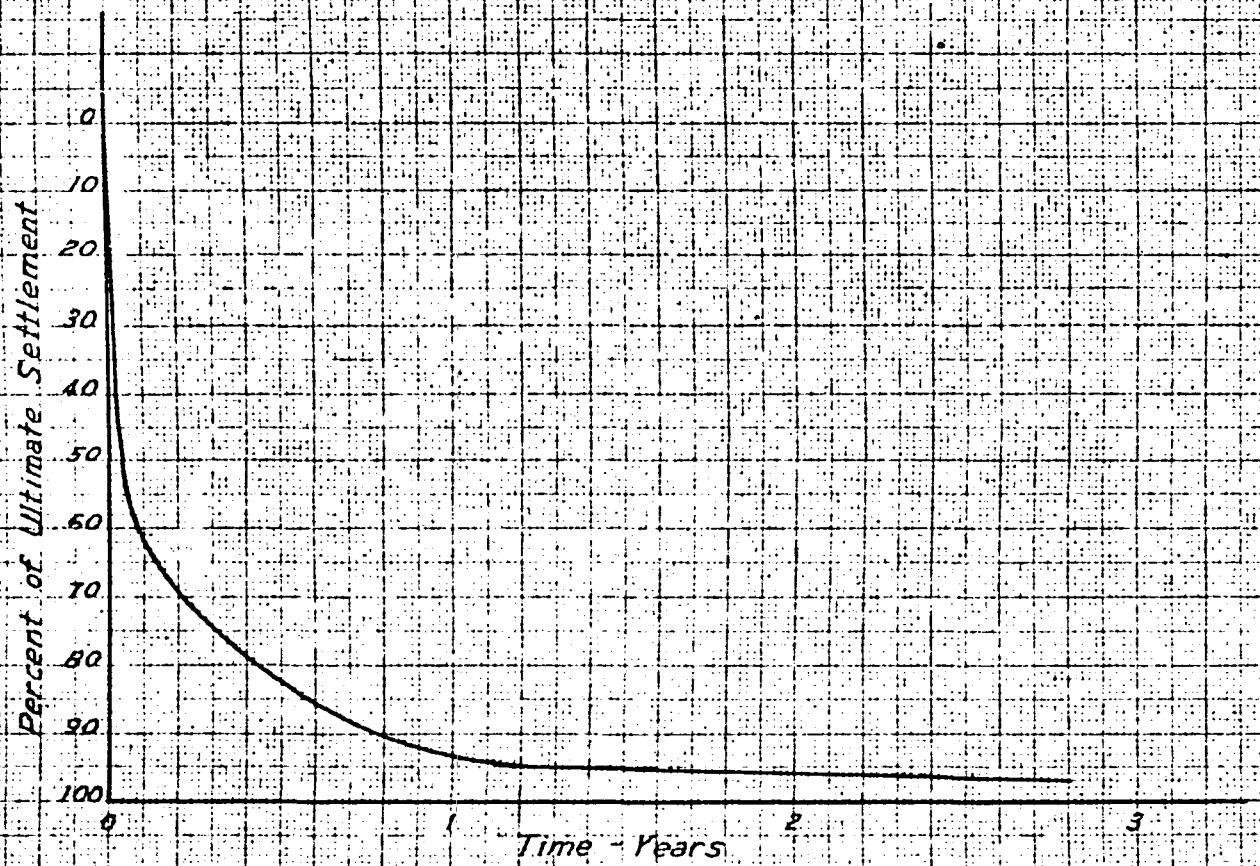
FOUNDATION INVESTIGATION

PROPOSED LOCK 2 PEARL RIVER IMPROVEMENT

LOAD AT HOLE 16 STATION 649+63

TIME SETTLEMENT DIAGRAM

FIGURE 39



FOUNDATION INVESTIGATION

PROPOSED LOCK 2, PEARL RIVER IMPROVEMENT

LOAD AT HOLE 37 STATION 652 + 01

TIME SETTLEMENT DIAGRAM

FIGURE 40