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FOUNDATION AND SOILS INVESTIGATION

BAYOU COCODRIE DRAINAGE STRUCTURE

TECHNICAL MEMORANDUM NO. 3-289

WATERWAYS EXPERIMENT STATION

VICKSBURG, MISSISSIPPI

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FOUNDATION AND SOILS INVESTIGATION

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PART I: INTRODUCTION

Location and Description of Project

1. The Bayou Cocodrie drainage structure will be constructed through the Tensas-Cocodrie Levee at Sta 624+29 on the east bank of Bayou Cocodrie about three miles west of Shaw, Louisiana. The site lies between a reach of Bayou Cocodrie and Grand Cut-Off Bayou; the Red River is about 2000 ft west and south of the site.

2. The proposed structure consists of a five-barrel (each 10 ft by 15 ft) reinforced-concrete box culvert, approximately 200 ft in length, having a 66-ft intake basin at the upstream end, a gate operating tower and stilling basin at the downstream end, and the necessary approach and outlet channels. The approach and outlet channels will be protected by 18 in. of riprap overlying a gravel blanket for distances of 136 and 143 ft, respectively. Flow through the culvert will be controlled by five mechanically operated, vertical-lift gates at the outlet end. (See plates 1 and 2.)

3. In the general vicinity of the site the natural ground surface elevation is about 40.0 ft msl; the lowest elevation of the bed of Bayou Cocodrie is about 11.0 and that of the Red River about -48.0. The invert elevation of the culvert is 13.0 at the intake and 10.0 at the outlet. The invert of the stilling basin is 0.0. The levee will have a crown

elevation of 56.0 which may be ultimately raised to elevation 64 by means of a wall on top of the levee.

4. During excavation and construction operations the site will be protected by an earth cofferdam with a crown elevation of 52.0. The center line of the proposed cofferdam is shown on plate 1. A levee will be constructed across the bayou to the west bank, and a channel routing the bayou through the structure will be excavated.

Geology of Site

5. The site of the proposed Bayou Cocodrie control structure lies near the southern edge of Dismal Swamp, a large floodbasin which extends northward to Deer Park, Louisiana. North, east, and west of Dismal Swamp meander belt ridges of the present and several former courses of the Mississippi River rise from 10 to 20 ft or more above the lowland. Southwest of the site, the lowland continues in a series of lakes and marshes, traversed by the natural levee ridges of the Red River, Bayou Natchitoches, Bayou Cocodrie, and several smaller bayous. The Red River has occupied its present course near the site for several hundred years. While the present and the reconstructed former courses of the Mississippi River pass east and west of the proposed site, it is believed that a Mississippi course older than any of these may have occupied the present basin prior to the inception of its backswamp role. There is also a likelihood that the Black River may have preceded the Red River in the present Red River course, and that either of these streams may have used Grand Cut-Off Bayou east of the site. Grand Cut-Off Bayou was active until the construction of the Mississippi River levee system.

PART II: FIELD EXPLORATION

Borings

6. Seven borings, of which four were undisturbed sample borings, were made during the period January-March 1948. Locations of these borings are shown on plates 1 and 3; the boring logs are also shown on plate 3. All the borings were carried from 5 to 10 ft into the sand stratum found at a depth of approximately 70 ft below the natural ground surface. Due to high water at the time of drilling, some of the borings were shifted from the planned locations. Borings 1 and 5 were not drilled and their scheduled locations are not shown on these plates.

General Soil Conditions

7. All borings indicated relatively uniform stratification which is generalized as follows:

- a. Natural ground surface (approximate elev 40) to elev 28. A highly stratified zone of silts and silty clays, reddish brown in color, having an average water content of about 30 per cent. These soils are termed Red River deposits in this report.

- b. Elev 28 to approximate elev -10. A stratum of highly plastic, jointed, blue-gray clays with a clay content (< 0.005 mm) averaging about 80 per cent, a liquid limit of 91 per cent, plastic limit of 29 per cent, and plasticity index of 62 per cent. The upper 18 ft of these clays had an average water content of about 55 per cent; a middle zone (6 ft), an average water content of 45 per cent; and the lower 14 ft, an average water content of 50 per cent.

- c. Elev -10 to elev -30. A stratified zone of gray sandy and clayey silts, silty sands, silty clays, and clays.

- d. Below elev -30. Clean uniform fine to medium sands and gravelly sands. The depth of this sand stratum is unknown but indications from available geological information are that it is 100 ft or more in thickness. As the bed of the Red River reaches depths as low as elev -48, this stratum may be considered as being connected to the river. This is confirmed by observations of piezometers installed in this stratum at the site.

Piezometers

8. After completion of the borings, six piezometers were installed in March 1948 at the landside edge of the levee crown in the vicinity of Sta 627 and 628. Three of these piezometers (2, 3, and 4) were installed for the purpose of observing the pore pressures induced in the clay foundation by the levee which was completed in the fall of 1947. These piezometers consisted of a porous stone tube 1 ft long and 1 in. ID, surrounded by Ottawa sand. The porous tip was connected to 3/8-in. ID Saran tubing which was carried inside a 3/4-in. galvanized pipe to about one foot above ground surface. The water level in the piezometer was determined by an electrical sounding device. Of the three remaining piezometers (Nos. 5, 1-S, and 2-S) two were placed in the heterogeneous strata between elev -10 and -30 and one in the sands found below elev -30, to determine the effect of river stages on the hydrostatic pressures in these strata. Piezometer 5 was of the porous stone type described above. Piezometers 1-S and 2-S consisted of 1-1/4-in. galvanized pipe with 5-ft sand filters beneath the open lower ends. All piezometers were filled with water at the time of installation. Locations of the piezometers are shown on plates 1 and 4.

9. In October 1948, two additional piezometers with porous stone tips (Nos. 6 and 7) were installed in the clay strata between elev 28 and -10 landward and riverward of the levee at locations where the hydrostatic

pressures in the clay would not reflect the influence of the levee load. The purpose of these piezometers was to obtain the elevation of the natural ground-water table.

Pore pressure in the clay foundation

10. Piezometer observations are shown on plate 4 together with the stages of the Red River at Bayou Cocodrie. It will be noted that the piezometers in the clay foundation beneath the levee indicate that stages of the Red River have little effect on the hydrostatic pressures in the clay. It thus appears that the excess pore pressures in the clay foundation, due to incomplete consolidation under the levee load, might be represented by the difference between the piezometer readings and some relatively fixed, normal ground-water table. The few readings on piezometers 6 and 7, which are away from the influence of the levee load, appear somewhat affected by the river stages. The lowest water levels of these two piezometers were about elev 28. This elevation also is the approximate depth of soil oxidation, which would indicate that it is probably the minimum water table reached in the clay foundation.

11. Observations made on settlement plates beneath the levee shortly after construction indicated that from 25 to 40 per cent of the total expected settlement of the foundation had already occurred in the vicinity of the piezometer installation. In computing the degree of consolidation from the piezometer observations, the excess pore pressures in the foundation were assumed to be equal to the difference between the piezometric head shown by the piezometers and an assumed water table at elev 28. Based on observations of piezometers 2 and 3, it is estimated

that the clay foundation was about 58 per cent consolidated under the levee load on 1 July 1948, and about 64 per cent consolidated on 1 January 1949. In computations of settlement of the proposed structure, it was assumed that the foundation is 50 per cent consolidated under the existing levee load.

Hydrostatic pressures beneath the clay foundation

12. Piezometer 1-S in the sands below elev -30 closely followed the Red River stages. Of the two piezometers in the stratified zone between elev -10 and -30, piezometer 5 also showed close agreement with river stages with the greatest variance occurring during the period of lowest stages. The performance of piezometer 2-S appears questionable, and the piezometer is believed to be partially clogged. On the basis of the observations of piezometers 1-S and 5, it is concluded that the piezometric head at the bottom of the clay stratum (elev -10) is approximately equal to the water level of the Red River.

PART III: LABORATORY TESTS

13. Visual classification and natural water content determinations were made on all samples except sands. Classification tests (mechanical analyses and Atterberg limits) were made on selected samples. Consolidation tests and rebound tests were performed on undisturbed specimens of clays and silty clays. A comprehensive program of shear strength tests was carried out, principally on undisturbed samples from the clay stratum. A few triaxial compression tests were made, but most of the shear tests were of the unconfined compression type. Permeability tests were performed on remolded samples of the sands below elev -30. Compaction tests were made on soil samples from the Red River deposits and strength tests were performed on the compacted specimens.

Classification Tests

14. Data on the grain-size distribution and plasticity characteristics are shown on plate 5. Classification data from consolidation test specimens are contained in table 1. Data pertaining to samples on which shear strength tests were performed are shown on plate 6 and in table 2.

Consolidation and Rebound Tests

15. Attention is directed to plate 6 and table 1. Six consolidation tests were performed on undisturbed specimens of the foundation soils: four on clay specimens taken between elev 28 and -10, and two on silty clay specimens taken between elev -10 and -30. The average compression index, C_c, of the clays was 0.50 indicating a high degree of

compressibility, while that of the silty clays was 0.30.

16. The time required for 50 per cent consolidation of the 1.2-in.-thick clay specimens drained top and bottom ranged from 13 to 160 min for the various load increments and averaged about 40 min for the load range involved in the field; that of the silty clays averaged about 8 min. As will be discussed later, a much faster rate of consolidation of the foundation under the existing levee was indicated by piezometers and settlement plates installed in the field. It is believed that there may have been unavoidable smearing and sealing of the surfaces of these highly jointed clays in preparing the test specimens, which greatly reduced the rate of consolidation.

17. Because of the possibility of the structure excavation remaining open for sufficient time to allow considerable rebound of the foundation clays, the rebound of loaded laboratory specimens was observed. Four undisturbed specimens of the clays between elev +10 and -10 were first consolidated under pressure equal to the in situ pressures beneath the levee center line and then allowed to rebound by removing the load in decrements. The average swelling index (C_s) of these clays was about 0.05. The time for 50 per cent rebound showed a greater variation with load than the consolidation tests, being from 9 to 210 min under the various loads, with an average time of 80 min for the average load of 0.24 ton per sq ft expected after excavation.

Shear Strength Tests on Undisturbed Foundation Soils

18. The shear strength testing program consisted primarily of unconfined compression tests. A few quick triaxial compression tests were

performed in addition to triaxial testing of single specimens, some of which were tested undrained with the lateral pressure, σ_3, equal to the total in situ pressure of soil plus water (QT-total), and others first consolidated under the effective in situ pressure of buoyed soil and then sheared undrained (Q_cT-eff). Shear strength test results and pertinent data on the soils tested are shown in table 2. The results of the unconfined compression tests are plotted against depth on plate 7, together with natural water contents of test specimens and classification data. The following tabulation gives the average results of the unconfined compression and quick triaxial tests on the clays and silty clays:

Test	Boring and Location	Number of Specimens	Average Cohesion ton/sq ft
Unconfined compression	No. 2 -- Structure center line	20	0.33
	No. 7 -- 300 ft west of structure center line	12	0.36
	No. 8 -- 300 ft east of structure center line	8	0.34
	Total	40	0.34
Quick triaxial	No. 2 -- Structure center line	3 $\phi = 2^{\circ}$	0.30

Note: Cohesion = one-half unconfined compressive strength.

The average shear strength of all the single triaxial test specimens, QT-total and Q_cT-eff, was 0.45 ton per sq ft, assuming the shear strength equals $\left(\frac{\sigma_1 - \sigma_3}{2}\right)$ (sin 60°). Based on the shear strength test results, a shear strength of 650 lb per sq ft was used for the foundation clays in all of the stability analyses.

Compaction and Shear Strength Tests on Remolded Soils

19. Compaction tests using standard Proctor equipment, but with the number of blows reduced from 25 per layer to 15, were run on four mixes of the Red River deposits found in the upper 12 ft of the natural ground. It was expected that the mixes would represent backfill consisting of these materials. The lesser number of compaction blows was used because compaction of the backfill will be limited to that obtained by crawler-type tractors. A few molds were compacted with 8 blows per layer. One-inch-diameter unconfined compression tests were performed on all compaction molds. Compaction curves and variation of cohesive strength versus molding water content are shown on plate 8. The following tabulation gives the average compaction and strength characteristics of the Red River deposits:

Grain-size distribution	Sand 13%; silt 49%; clay 38%.	
Atterberg limits	LL = 50; PL = 18; PI = 32.	
Natural water content	32%.	
15-blow compaction	23% optimum water content, 98 lb per cu ft optimum dry density.	
	Molding Water Content	Cohesion t/sq ft
Unconfined compression tests	22%	1.01
	24%	0.79
	26%	0.58
	28%	0.41
	30%	0.20

Two consolidated-quick triaxial tests on specimens molded at about optimum water content gave an average shear strength of $\phi = 10^{\circ}$ and c = 0.43 ton per sq ft. It was assumed in stability analyses involving the reconstructed levee over the structure that the embankment material would be placed at water contents somewhat drier than the average natural water contents, and that the embankment will have a shear strength of 650 lb per sq ft. (The levee and backfill will be placed in 12-in. layers and compacted by three passes of a crawler-type tractor.)

Permeability of Sands Below Elev -30

20. Laboratory permeability tests on remolded sand samples taken between elev -33 and -44 gave the following values adjusted to a void ratio of 0.60:

Sample	Coefficient of Permeability
Medium to fine sand 6 per cent finer than 0.05 mm	$k = 77 \times 10^{-4}$ cm/sec
Gravelly sand 2 per cent finer than 0.05 mm	$k = 300 \times 10^{-4}$ cm/sec
Sand with some gravel 5 per cent finer than 0.05 mm	$k = 85 \times 10^{-4}$ cm/sec

The coefficients of permeability, as computed for 8 sand samples on the basis of effective size (D_{10}) using Hazen's equation* for uniform clean sands with C = 100, gave values ranging from 225 to 900 x 10^{-4} cm/sec, with an average of 450 x 10^{-4} cm/sec. Since the fines probably occur in nature as thin layers between clean sands rather than distributed throughout the sand mass as in the laboratory test specimens, it is believed

* $k = C(D_{10})^2$ in which D_{10} is the effective size in cm and $C(\frac{1}{\text{cm/sec}})$ varies from 100 to 150.

that the in situ horizontal permeability of these sands will be about 500×10^{-4} cm/per sec.

PART IV: STABILITY ANALYSES

21. Stability analyses were made on the excavation slopes for the structure, channel slopes, cofferdam, training walls, the completed structure with its levee surcharge, and the levee closure over Bayou Cocodrie. The following design values were used in the analyses:

a. The maximum depth of potential failure surfaces at elev -10, the bottom of the thick foundation clay stratum.

b. Unit weights, saturated or wet:

	lb/cu ft
Existing levee, cofferdam, levee over bayou . . .	110
Foundation soils	110
Compacted levee on structure	125

c. Shear strengths:

	lb/sq ft
Existing levee (for shallow slides)	500
Compacted levee over structure	650
Cofferdam and levee over bayou	400
Clay foundation soils above elev -10	650
Sandy and silty soils below elev -10	$\phi = 25°$ $c = 100$

d. In analyses of the cofferdam in the existing borrow pit and of the levee over the bayou, it was assumed that the surfaces of the borrow pit and bayou bed were planes of weakness having a shear strength of 250 lb per sq ft.

Channel Slopes

22. Stability analyses by Taylor's method* were made on the approach and outlet channel slopes. Factors of safety against sliding are shown on plate 1. The factors of safety ranged from 1.24 to 1.51, excepting the value of 1.09 for the 1 on 1-1/2 slopes of the pilot channel to the river

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* "Stability of Earth Slopes." Boston Society of Civil Engineers, July 1937.

which is considered acceptable, since natural flattening of the pilot channel slopes is anticipated.

Excavation Slopes

23. The factors of safety of the excavation slopes for the structure are shown on plate 9. They ranged from 1.26 to 1.73 for the various cuts. The analyses were made generally by Taylor's method of circular arc analysis, except for the maximum cut through the existing levee which was analyzed also by the method of planes. This analysis is shown on plate 10. The lowest factor of safety, 1.26, was obtained for the maximum cut by the method of planes, with the assumption of uplift beneath the bottom of the clay foundation at elev -10. A piezometric pressure equivalent to elev 18 (the maximum allowable; see paragraph 34) was applied at elev -10, with excavation in the dry to elev 8.

Cofferdam

24. The work area will be ringed with an earth cofferdam having its crown at elev 52. The center line of the proposed cofferdam is shown on plate 1. Stability analyses were made by the circular arc method and the method of planes. Analyses shown on plate 11 were made at three locations of the cofferdam: (1) landside of the existing borrow pit, (2) in the existing borrow pit, and (3) riverside of the structure excavation. The factors of safety for the proposed slopes and distances from excavation slopes ranged from 1.25 to 1.62, except for the cofferdam at the landside edge of the borrow pit which had a factor of safety of 1.16 by the method of planes and 1.25 by the circular arc method. Proposed cofferdam slopes

are 1 on 3 above elev 40 (approximate natural ground surface). Where the cofferdam crosses the existing borrow pit, which has a minimum surface elevation of 20, the cofferdam slopes below elev 40 are 1 on 10 to elev 28 and 1 on 4 from elev 28 to the ground surface or bottom of the landside excavation. Where founded on natural ground the cofferdam toe should be a minimum of 75 ft from the 1-on-4 excavation slopes, 50 ft from the 1-on-5 slopes, and 50 ft from the landside slope of the existing borrow pit.

Structure and Levee

25. Analyses were made, by the circular arc method and method of planes, of the stability of the structure with its levee surcharge against sliding in the clay foundation either to the riverside or landside. In these analyses the shear strength through the barrels was assumed to be zero, and the end restraint provided by the retaining walls and base slabs of the approach and stilling basin was neglected. It was evident that sliding would occur unless the barrels could provide some resistance against sliding. Therefore, the base of the barrel sections was designed as a monolithic slab with sufficient reinforcing steel to take enough horizontal tensile stress (135,000 lb per ft width of structure) to provide a reasonable factor of safety. Analyses of the structure and levee are shown on plate 12. The factors of safety against sliding riverward, taking into account the restraint of the barrel base, were 1.30 by the method of planes and 1.53 by the circular arc method. Factors of safety against sliding landward were somewhat greater.

Retaining Walls

Bearing capacity

26. Analyses of the bearing capacities of the footings of the wing and approach walls were made. The clay foundation beneath the footings was assumed to have a shear strength of 650 lb per sq ft. Based on Terzaghi's equations for the bearing capacity beneath shallow footings, the bearing capacity of the clays with the footing based on the ground surface is 3700 lb per sq ft for general shear and 2470 lb per sq ft for local shear. Since in all cases the computed heel pressures of the footings were equal to or greater than the toe pressures, an average footing pressure was used to determine the factors of safety with respect to bearing capacity. Average foot pressures of the intake approach wing walls and of the stilling basin wing walls were 2555 and 2370 lb per sq ft, respectively; that of the intake approach walls 1350 lb per sq ft. The factors of safety ranged from 1.77 to 3.01 assuming general shear and 1.28 to 2.10 assuming local shear.

Stability against sliding

27. The intake approach walls were analyzed also for stability against sliding along the base of the footing and for stability of the wall and backfill against base failure in the clay foundation. The factor of safety against sliding along the base of the footing, assuming active earth pressure of the sand backfill, was about 1.50 with a resistance of 20 lb per sq in. being provided by the approach slab. The factor of safety against deep sliding of the wall and backfill, as shown on plate 12, was 1.34 by the method of planes allowing a resistance of 100 lb

per sq in. provided by the approach slab. By the circular arc method the factor of safety was 1.33 against deep sliding with no thrust against the slab. The factor of safety against a base failure, using Terzaghi's equation 56 (10) "Theoretical Soil Mechanics," for base failure of a vertical bank, was 1.42.

Levee over Bayou Cocodrie

28. The levee crossing the bayou will have a maximum height of 42 ft, as the bed of the bayou is at about elev 15, and gross levee grade about elev 57. Analyses were made of the levee section by the method of planes. For sliding along the bottom of the foundation clays (elev -10), the minimum factor of safety for the recommended section was 1.32. For sliding along the bed of the bayou, which was assumed to be a horizontal plane of weakness, the factor of safety was 1.26. These analyses are shown on plate 10.

PART V: OTHER DESIGN CONSIDERATIONS

Settlement of the Structure

29. If there is no appreciable rebound of the clay foundation due to excavation, no settlement will occur in the approach or stilling-basin sections, since the loads imposed by the structure will be less than the weight of the excavated material.

30. Beneath the barrels, some settlement can be expected, its magnitude being affected by the degree of rebound of the foundation soils during and after excavation. In the analyses of settlement beneath the barrels, an average consolidation curve based on samples 27 and 34 (boring 2, plate 6) was used for the clay foundation extending from the base of the barrels to elev -10. An average curve based on the consolidation curves of samples 46 and 50, boring 2, was used for the heterogeneous zone between elev -10 and -30, using a weighted stratum thickness based on the proportions of clays, silty clays and clay silts in this zone.

31. It was assumed that the existing effective pressures in the clay foundation are equal to the natural overburden with the water table at elev 28 plus 50 per cent of the existing levee load. It was further assumed that construction of the structure will lower the water table in the vicinity to the base of the barrels.

32. Assuming that no rebound will occur in the clay foundation, it is estimated that a 7-in. settlement will occur beneath the center of the barrels. The virgin compression curves were used in making this computation. It was then assumed that the clay foundation will rebound after excavation, and that after rebounding the foundation is reconsolidated

under the structure load. Under these assumptions it is estimated that the foundation will settle 12 in. beneath the center of the barrels. The amount of rebound which will actually occur is indeterminate, since it depends upon the length of time the excavation remains open and whether or not the excavation is flooded. Therefore, it was deemed advisable to allow for the maximum rebound, and the base slab was designed with a camber of 12 in. beneath the center of the levee.

Drainage and Uplift Control

33. In order to prevent any uplift beneath the approach and stilling-basin slabs, a sand blanket with perforated (1/4-in. holes) collector pipes has been included. These collector pipes will discharge horizontally through the side walls. Drains have also been provided behind the retaining walls to drain the sand backfill. The drain outlets in the stilling basin were put near the toe of the steps where the water pressure in the stilling basin will be a minimum during periods of high discharge. The foundation and backfill drainage systems are shown on plate 13 with details of collector outlets shown on plate 14. No pressure relief in the deep underlying sands or silts is considered necessary after construction. The maximum differential head in a landward direction is 14 ft of water and 23 ft in the riverward direction. This latter condition is created only when the Red River is low and Bayou Cocodrie is high. As Bayou Cocodrie does not penetrate to the underlying pervious strata, little excess hydrostatic pressure is expected in the deep sands when this stream is high and the Red River is low.

Control of Subsurface Hydrostatic Pressures During Construction

34. Piezometers installed in the semipervious soil below elev -10 and in the sands below elev -30 indicate that the hydrostatic head at these elevations is dependent upon the stage of the Red River. The excavation for the structure will be deep enough so that the pressures at these elevations would exert an upward force greater than the weight of the overlying impervious stratum for certain stages of the Red River. As a consequence it will be necessary to relieve the hydrostatic pressure in the more pervious soils found below elev -10 so that the effective weight of the overlying soil is never exceeded. Observations of the pressures in the strata below elev -10 should be made during the construction operations by means of piezometers in the excavation area. Computations using submerged weights and net heads indicate that the piezometric head acting at elev -10 should not exceed elev +18 when excavating for the culverts, or elev -4 when excavating for the stilling basin. It is considered necessary to reduce the hydrostatic pressure in the sands in order to reduce satisfactorily the pressures in the silty soils between elev -10 and -30. Since the stage of the Red River varies considerably throughout the year, the necessity for relieving hydrostatic pressures in the pervious strata also varies. During the last half of the calendar year the river stage is almost always less than elev 30 and often less than elev 20. Therefore, if excavation for the culverts were made during this period it might not be necessary to relieve the pressures in the sand, provided the Red River was not above elev 18. However, under any conditions, the pressure in the area of the stilling basin must be relieved when excavation

is made in that area. It must be assumed that the excavation will remain open and not be flooded even during very high stages of the Red River, since the site will be protected by a cofferdam. Consequently, some pressure relief facilities will be required to relieve the pressure beneath the entire excavation area when the river rises, even though they may not be required except around the stilling-basin area at the time excavation is made.

35. In order not to restrict the contractor unduly, it is considered advisable to permit him to develop the pressure relief system and to decide upon the elevations at which the facilities are to be installed subject to the approval of the contracting officer. In granting such latitude to the contractor, it should be required that he provide adequate means to flood the excavation should he not be able to reduce excessive uplift pressures caused by a sudden river rise.

36. Several methods of relieving hydrostatic pressures in the underlying sands might be used satisfactorily. One method would be a well point system which would maintain the piezometric head in the stilling-basin area below elev -4, and in the remainder of the excavation below elev 18. This might require either a one- or two-stage system, depending upon the time of year the excavation was made. If the contractor so desires, he should be permitted to place a well point system in narrow trenches in order to get it as deep as possible. A second method would be to install pressure relief wells and allow them to flow freely into the excavation where the water could be suitably collected and pumped out. This would be satisfactory in the barrel and approach-channel area, but a well point system is essential in the stilling-basin area to provide more

positive control of pressures in the sand. A third possible method would be a system of wells using deep well pumps. Such wells could be installed at suitable spacings around the perimeter of the excavation, either prior to excavation or at a certain stage of excavation. The number of wells could be increased as the excavation is carried deeper.

37. When all construction below the base of the barrels has been completed, and the stilling basin flooded, the pressure in the deep sand in the stilling-basin area may be permitted to rise to elev 18. Upon completion of all construction work below elev 20, it will be possible to flood the structure and discontinue the operation of the pressure relief system, provided the water level in the structure is maintained at not more than 12 ft below the piezometric head in the deep sands.

Backfill and Levee Material

38. To reduce the lateral earth pressures against the intake and outlet training walls, a wedge of sand should be provided next to the walls as shown on plate 9, with a drain at the base of the wedge as shown on plate 14. The pervious sand wedge should be covered with an 18-in. layer of impervious soil at the surface. The remainder of the backfill up to natural ground surface (approximately at elev 40) should be composed of Red River deposits excavated from the upper 12 to 15 ft of the natural ground, with compaction of 12-in. (loose) lifts by three passes of a crawler-type tractor. The driest material should be used closest to the walls and sides of the conduit. Above elev 40, the levee should be reconstructed of material removed from the existing levee, or of material from the upper 12 to 15 ft of Red River deposits, and compacted in the

same manner as described above. Excavated material which is to be used for backfill and levee reconstruction should be utilized as far as possible in the cofferdam construction above elev 40 (natural ground surface). Any of the materials available from the excavation can be used in the construction of the levee crossing the bayou.

PART VI: ENGINEERING MEASUREMENT DEVICES

Temporary Devices

39. Temporary measurement devices which are recommended for observation during construction are:

a. Piezometers. Several piezometers should be installed within the excavation area to determine the hydrostatic pressures in the foundation below elev -10, the bottom of the clay. These piezometers will insure safe excavation and construction operations, as they will indicate the need for further pressure relief or, if necessary, flooding of the excavation. Installation and observations should preferably be made by Government personnel.

b. Lateral movement markers. Stakes should be placed at elev 45 on the edge of the 1-on-25 berm of the excavation on the levee center line for the purpose of revealing lateral movement during excavation. If indications of excessive lateral movement are observed the excavation slopes would have to be flattened.

Permanent Devices

40. Recommended permanent measurement devices consist of piezometers, settlement plates, and reference markers. Proposed locations of these devices are shown on plate 2. Details of piezometers and settlement plates are shown on plate 14. The reference markers consist of brass carriage bolts set in various concrete members of the structure. Piezometers 1, 2, 5 and 6, located beneath the approach and stilling-basin slabs, are to be used for observing hydrostatic pressures developed in strata beneath the clay foundation soils. Piezometers 3, 4, 7 and 8 are for the purpose of observing hydrostatic pressures in the sand backfill behind the walls. Observation of the settlement plates installed on top of the barrels beneath the levee surcharge, together with observations of the

reference markers on the intake head wall and gate tower, will provide data on settlement of the structure. The reference markers on the intake approach wall and footing, the stilling-basin wall, and the stilling-basin wing wall, will show the horizontal and vertical movements of the walls.

PART VII: CONCLUSIONS AND RECOMMENDATIONS

Excavation Slopes and Embankment Sections

41. The excavation slopes shown on plate 9 and the embankment sections shown in plan on plate 1 are recommended. Recommended cofferdam slopes and minimum distances from excavation slopes are given in paragraph 24.

Structure

42. Because of the weak clay foundation, it was necessary to design the base of the barrels as a monolithic reinforced slab in order to provide a satisfactory factor of safety against sliding of the structure with its levee surcharge. Expected settlement of the barrels beneath the center line of the levee was computed to be from 7 to 12 in. depending upon the amount of rebound of the clay foundation after excavation. Little or no settlement is expected at the head walls. The culvert barrel should be constructed with a camber approximately equal to the maximum expected differential settlement of 12 in. of the structure as shown on plate 2.

Drainage and Uplift Control

43. During excavation and construction operations, relief of hydrostatic pressures in the sandy soils beneath the clay foundation should be provided as described in paragraph 34. After construction no pressure relief measures in these strata are considered necessary. Recommended permanent drainage and uplift control measures consist of drainage of the

sand backfill behind the walls, and sand blankets with collector pipe beneath the stilling-basin and approach slabs.

Backfill and Levee Materials

44. A clean sand wedge should be placed next to the retaining walls as shown on plate 9. The remainder of the backfill up to elev 40 should consist of the reddish brown soils excavated from the upper 12 to 15 ft of the natural ground. Above elev 40, the levee should be reconstructed with these soils and/or the material excavated from the existing levee. The levee crossing the bayou may be constructed with any of the excavated soils.

Compaction

45. Since it is not considered practicable to dry to any appreciable extent the clay-type soils available for backfill and levee reconstruction, it is recommended that compaction of these items be accomplished by three passes of a crawler-type tractor at the existing water content. In order to dry out the material as much as possible before use, it is recommended that the material specified for backfill and levee reconstruction be used in the cofferdam construction above elev 40 to the greatest extent possible. The levee section over the bayou and the cofferdam were designed on the assumption of dragline construction with little compaction by moving equipment.

Engineering Measurement Devices

46. Installation of the temporary devices (reference stakes and piezometers) and the permanent devices (piezometers, reference markers, and settlement plates) described in paragraphs 39 and 40, is recommended.

Table 1

SUMMARY OF CONSOLIDATION AND REBOUND TEST DATA -- BORING 2

Sample No.	Elev msl	Classification	Mechanical Analysis % Sand	% Silt	% Clay	Atterberg Limits LL	PL	PI	Sp Gr G	Test Data w	γ_d Lb/Cu Ft	e	C_c	p_c T/Sq Ft	p_o T/Sq Ft	C_s
14	25.1	Clay, gray with organic matter	8	8	84	109	30	79	2.66	55	69	1.41	0.46	1.7	1.1	---
22	14.7	Clay, gray with concretions	5	13	82	96	30	66	2.65	65	61	1.70	0.58	0.8	1.3	---
26	9.9	Clay, gray	4	19	77	84	30	54	2.72	41	80	1.13	--	---	1.5	0.045
27	8.8	Clay, gray	8	18	74	80	28	52	2.67	49	73	1.29	0.49	1.2	1.5	---
32	2.7	Clay, gray	6	12	82	102	30	72	2.72	48	73	1.33	--	---	1.6	0.055
34	0.6	Clay, gray with concretions	4	17	79	88	29	59	2.70	50	73	1.31	0.45	1.8	1.7	---
37	-2.7	Clay, gray with concretions	3	10	87	104	32	72	2.70	45	76	1.21	--	---	1.7	0.060
40	-6.0	Clay, gray	6	16	78	93	31	62	2.72	40	81	1.10	--	---	1.8	0.045
46	-20.7	Silty clay, gray	8	38	54	51	25	26	2.68	41	81	1.06	0.34	0.9	2.2	---
50	-28.6	Silty clay, gray	11	48	41	51	24	27	2.72	36	86	0.98	0.27	0.8	2.4	---

NOTES: C_c = compression index
C_s = swelling index
p_c = preconsolidation pressure
p_o = existing in situ pressure (including 50 per cent of load imposed by existing levee)

Table 2

SHEAR STRENGTH TESTS ON UNDISTURBED SAMPLES

Boring	Sample No.	Sample Depth ft	Sample Elev msl	Classification	Mechanical Analysis % Sand	Mechanical Analysis % Silt	Mechanical Analysis % Clay	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI	Shear Strength Test Type	w	γ_d Lb/Cu Ft	σ_3 T/Sq Ft	ϕ^o	Shear Strength T/Sq Ft
No. 2	2	1.8	44.8	Clay, brown	6	15	79	--	--	--	1" UC	41	80	----	-	0.36
Sta 624+29	4	5.4	41.2	Silty clay, brown with roots	11	47	42	47	20	27	1" UC	29	91	----	-	0.38
80 ft					--	--	--	--	--	--	QT-total	44	77	0.32	-	0.14
Riverside					--	--	--	--	--	--	Q_cT-eff.	51	71	0.32	-	0.20
	7	10.8	35.8	Clay silt, brown	--	--	--	--	--	--	QT	28	93	----	4	0.35
	10	16.0	30.6	Silty clay, brown	6	41	53	58	22	36	1" UC	35	85	----	-	0.43
	14	21.5	25.1	Clay, gray with organic matter	8	8	84	109	30	79	1" UC	51	71	----	-	0.32
					--	--	--	--	--	--	QT-total	53	69	1.32	-	0.13
	15	23.5	23.1	Clay, gray	--	--	--	--	--	--	Q_cT-eff.	45	75	0.94	-	0.39
	16	24.7	21.9	Clay, gray	--	--	--	--	--	--	2.8" UC	48	71	----	-	0.30
	18	27.1	19.5	Clay, gray with concretions	4	6	90	--	--	--	QT	50	71	----	1	0.26
	19	28.3	18.3	Clay, gray	--	--	--	--	--	--	1" UC	62	63	----	-	0.23
	20	29.5	17.1	Clay, gray	--	--	--	--	--	--	1" UC	50	71	----	-	0.37
	21	30.7	15.9	Clay, gray	--	--	--	--	--	--	2.8" UC	48	63	----	-	0.20
	23	33.1	13.5	Clay, gray with concretions	--	--	--	--	--	--	1" UC	59	65	----	-	0.25
					--	--	--	--	--	--	Q_cT-eff.	49	65	1.20	-	0.11
	24	34.3	12.3	Clay, gray	--	--	--	--	--	--	1" UC	54	68	----	-	0.27
	25	35.5	11.1	Clay, gray	--	--	--	--	--	--	2.8" UC	50	73	----	-	0.17
	26	36.7	9.9	Clay, gray	4	19	77	84	30	54	1" UC	42	77	----	-	0.32
	28	39.1	7.5	Clay, gray	--	--	--	--	--	--	QT-total	40	82	2.26	-	0.35
					--	--	--	--	--	--	Q_cT-eff.	38	84	1.35	-	0.73
	29	40.3	6.3	Clay, gray	--	--	--	--	--	--	1" UC	42	80	----	-	0.27
	30	41.5	5.1	Clay, gray	--	--	--	--	--	--	1" UC	40	80	----	-	0.60
	31	42.6	4.0	Clay, gray	--	--	--	--	--	--	2.8" UC	47	74	----	-	0.25
	32	43.9	2.7	Clay, gray	6	12	82	102	30	72	1" UC	48	73	----	-	0.45
	34	46.0	0.6	Clay, gray with concretions	4	17	79	88	29	59	1" UC	45	75	----	-	0.45
	35	47.1	-0.5	Clay, gray	--	--	--	--	--	--	QT	46	75	----	2	0.34
	36	48.1	-1.5	Clay, gray with concretions	--	--	--	--	--	--	2.8" UC	47	73	----	-	0.19
	38	50.4	-3.8	Clay, gray with concretions	8	8	84	107	29	78	1" UC	52	71	----	-	0.43
					--	--	--	--	--	--	QT-total	46	76	2.90	-	0.64
					--	--	--	--	--	--	Q_cT-eff.	48	73	1.60	-	0.69
	42	55.8	-9.2	Silty clay, gray	--	--	--	--	--	--	1" UC	38	79	----	-	0.37
	48	71.8	-25.2	Clay, gray	--	--	--	--	--	--	1" UC	36	86	----	-	0.58

(Continued)

NOTES: UC = unconfined compression test; QT = quick triaxial compression test.
QT-total = single quick triaxial test with σ_3 = total in situ pressure (soil + water). $S = (\frac{\sigma_1 - \sigma_3}{2}) \sin 60^o$.
Q_cT-eff. = single consolidated-quick triaxial test with σ_3 = effective in situ pressure (buoyed weight of soil). $S = (\frac{\sigma_1 - \sigma_3}{2}) \sin 60^o$.

Table 2 (Cont'd)

Boring	Sample No.	Sample Depth ft	Sample Elev msl	Classification	Mechanical Analysis % Sand	Mechanical Analysis % Silt	Mechanical Analysis % Clay	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI	Shear Strength Test Type	w	γ_d Lb/Cu Ft	σ_3 T/Sq Ft	ϕ^o	Shear Strength T/Sq Ft
No. 2 (Cont'd)	49	74.0	-27.4	Silty clay, gray	--	--	--	74	27	47	1" UC	39	81	----	-	0.70
					--	--	--	--	--	--	QT-total	42	79	4.30	-	0.63
					--	--	--	--	--	--	Q_cT-eff.	43	79	2.30	-	0.99
	51	76.4	-29.8	Silty clay, gray with sand lenses	--	--	--	--	--	--	QT	34	87	----	2	0.30
No. 7 Sta 621+29 85 ft Riverside	12	15.4	31.2	Silty clay, brown with organic matter	--	--	--	--	--	--	1" UC	33	87	----	-	0.30
	15	19.6	27.0	Clay, gray with organic matter	8	19	73	--	--	--	1" UC	48	72	----	-	0.35
	19	24.0	22.6	Clay, gray and brown	--	--	--	--	--	--	1" UC	57	65	----	-	0.24
	21	27.7	18.9	Clay, gray and brown	--	--	--	--	--	--	1" UC	54	67	----	-	0.38
	23	30.0	16.6	Clay, gray and brown	--	--	--	--	--	--	1" UC	46	75	----	-	0.38
	26	33.0	13.6	Clay, gray with concretions	4	10	86	--	--	--	1" UC	51	70	----	-	0.36
	28	35.5	11.1	Clay, gray	--	--	--	--	--	--	1" UC	51	70	----	-	0.32
	30	37.8	8.8	Clay, gray	--	--	--	--	--	--	1" UC	52	70	----	-	0.36
	33	41.3	5.3	Clay, gray	6	32	62	--	--	--	1" UC	40	80	----	-	0.46
	36	44.5	2.1	Clay, gray	--	--	--	--	--	--	1" UC	--	--	----	-	0.41
	39	48.0	-1.4	Clay, gray	4	17	79	--	--	--	1" UC	46	74	----	-	0.35
	41	50.4	-3.8	Clay, gray	--	--	--	--	--	--	1" UC	48	73	----	-	0.39
No. 8 Sta 627+29 80 ft Riverside	3	3.6	43.0	Clay, gray with organic matter	--	--	--	--	--	--	1" UC	40	78	----	-	0.31
	12	16.5	30.1	Clay, brown	--	--	--	--	--	--	1" UC	35	84	----	-	0.54
	15	20.2	26.4	Clay, gray with organic matter	7	7	86	--	--	--	1" UC	50	70	----	-	0.35
	21	27.0	19.6	Clay, gray with organic matter	--	--	--	--	--	--	1" UC	50	70	----	-	0.40
	27	33.9	12.7	Clay, gray with organic matter	7	10	83	--	--	--	1" UC	57	64	----	-	0.30
	36	43.8	2.8	Clay, gray	--	--	--	--	--	--	1" UC	37	81	----	-	0.21
	38	46.0	0.6	Clay, gray with organic matter	7	18	75	--	--	--	1" UC	44	74	----	-	0.39
	45	51.6	-5.0	Clay, gray	--	--	--	--	--	--	1" UC	49	71	----	-	0.19

See sheet 1 of table for notes.

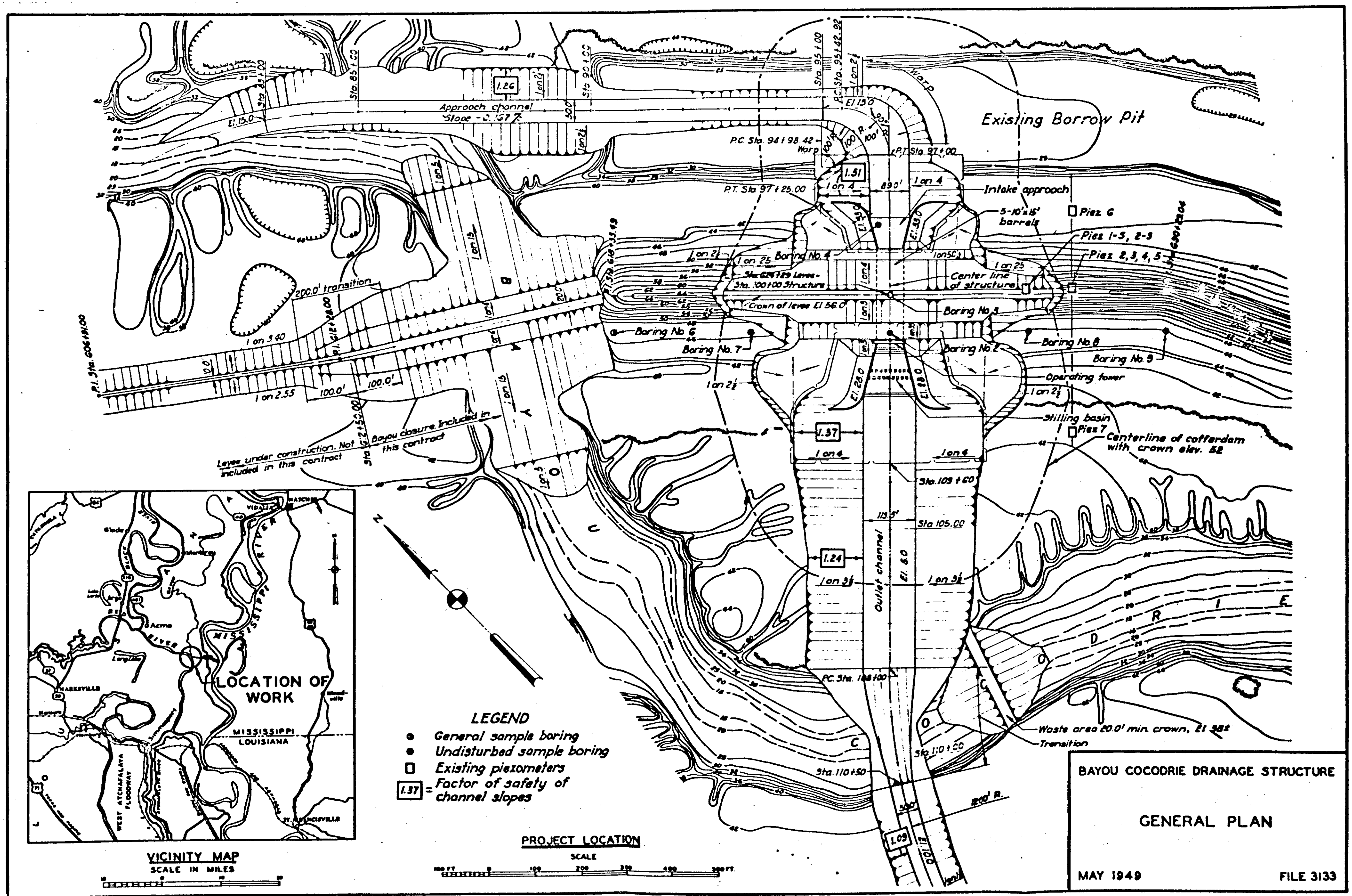
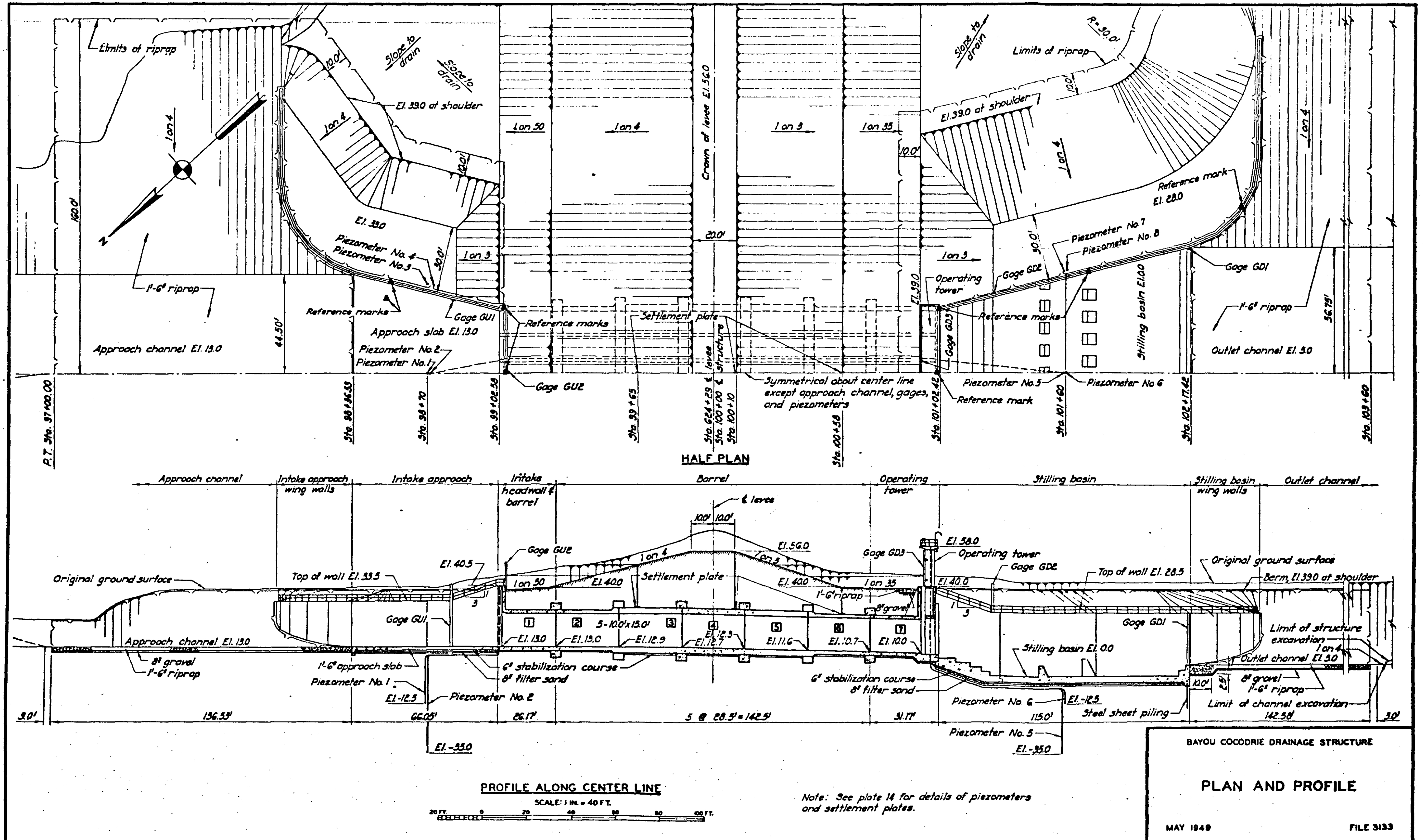

BAYOU COCODRIE DRAINAGE STRUCTURE
GENERAL PLAN
MAY 1949
FILE 3133
LEGEND
General sample boring
Undisturbed sample boring
Existing piezometers
1.37 = Factor of safety of channel slopes
VICINITY MAP
SCALE IN MILES
PROJECT LOCATION
SCALE
LOCATION OF WORK
MISSISSIPPI
LOUISIANA
Existing Borrow Pit
Approach channel
Slope - 0.167 %
El. 15.0
El. 13.0
Intake approach
5-10'x15' barrels
Piez 6
Piez 1-3, 2-3
Piez 2, 3, 4, 5
Center line of structure
Crown of levee El 56.0
Sta. 626+29 Levee - Sta. 100+00 Structure
Boring No. 4
Boring No. 3
Boring No. 2
Boring No. 6
Boring No. 7
Boring No. 8
Boring No. 9
Operating tower
Stilling basin
Piez 7
Centerline of cofferdam with crown elev. 52
Outlet channel
El. 5.0
Sta. 105.00
Sta. 103+60
Waste area 20.0' min. crown, El. 38±
Transition
Sta. 110+50
Levee under construction. Not included in this contract
Bayou closure. Included in this contract
200.0' transition
P.C. Sta. 94+98.42
P.T. Sta. 97+25.00
P.T. Sta. 97+00
Warp
1.26
1.51
1.37
1.24
1.09

PLATE I



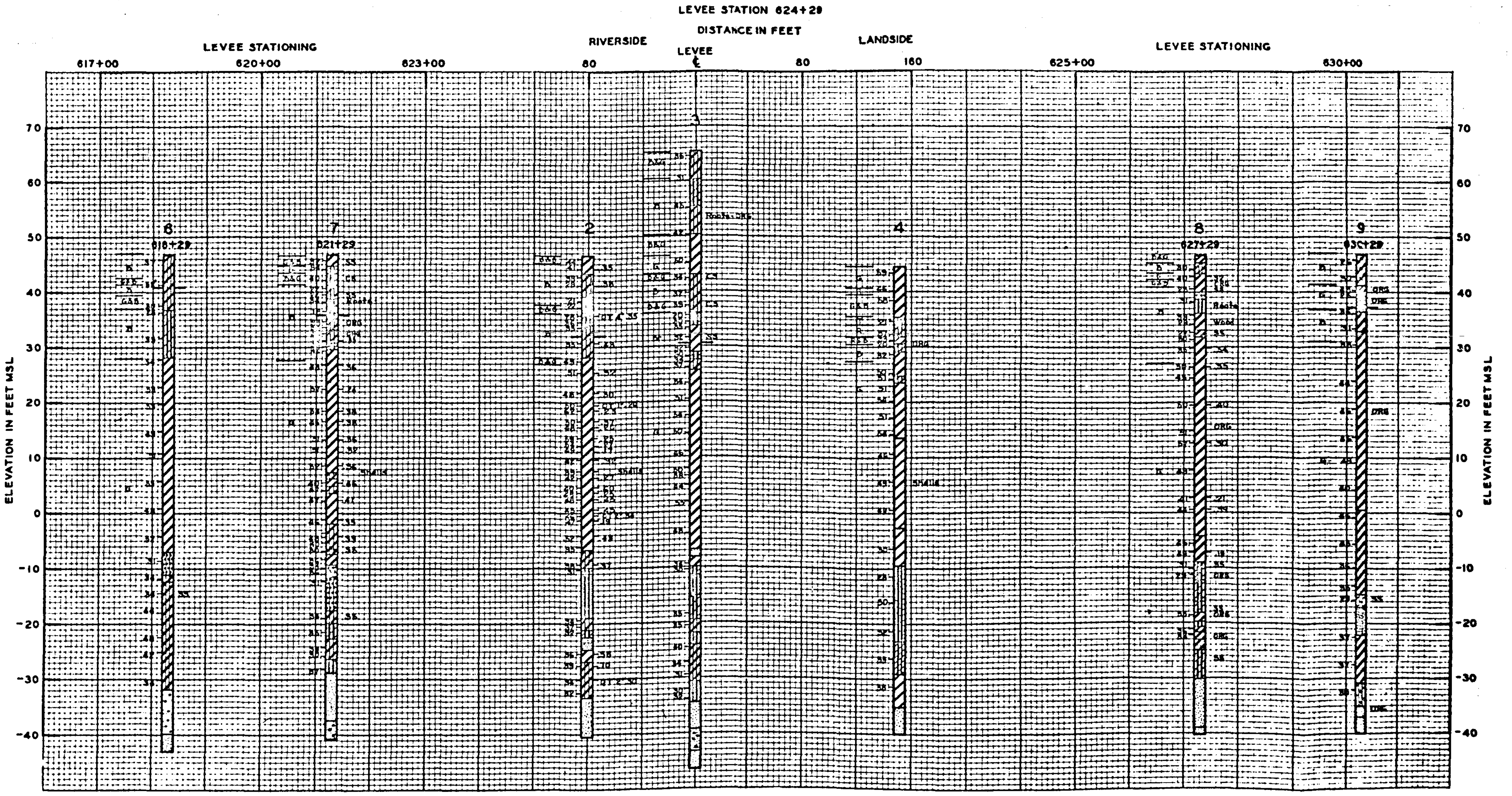
BAYOU COCODRIE DRAINAGE STRUCTURE

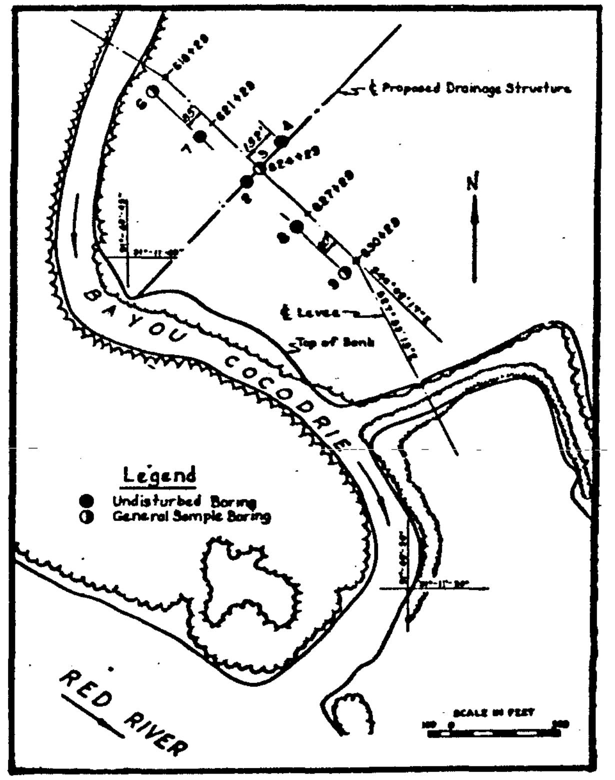
PLAN AND PROFILE

MAY 1949

FILE 3133

PLATE 2





PROFILE 85 FT RIVERSIDE OF LEVEE ℄

PROFILE ALONG ℄ OF STRUCTURE

PROFILE 80 FT RIVERSIDE OF LEVEE ℄

LEGEND

CLAY
SILTY CLAY
SANDY CLAY
CLAY SILT
SANDY SILT
SILTY SAND
SAND
GRAVELLY SAND
WATER TABLE
SS THIN SAND LAYERS OR LENSES
CS THIN CLAY LAYERS OR LENSES
ORG ORGANIC MATTER

COLORS:
B = BROWN
G = GRAY
R = RED
T = TAN

NOTES

BORINGS MADE IN JANUARY, FEBRUARY AND MARCH 1948.

UNDISTURBED SAMPLE BORINGS: 2, 4, 7, AND 8 MADE WITH 5-IN. DIAMETER SHELBY TUBE.

GENERAL SAMPLE BORINGS: 3, 6 AND 9.

FIGURES TO LEFT OF BORINGS ARE NATURAL WATER CONTENTS IN PERCENT OF DRY WEIGHT.

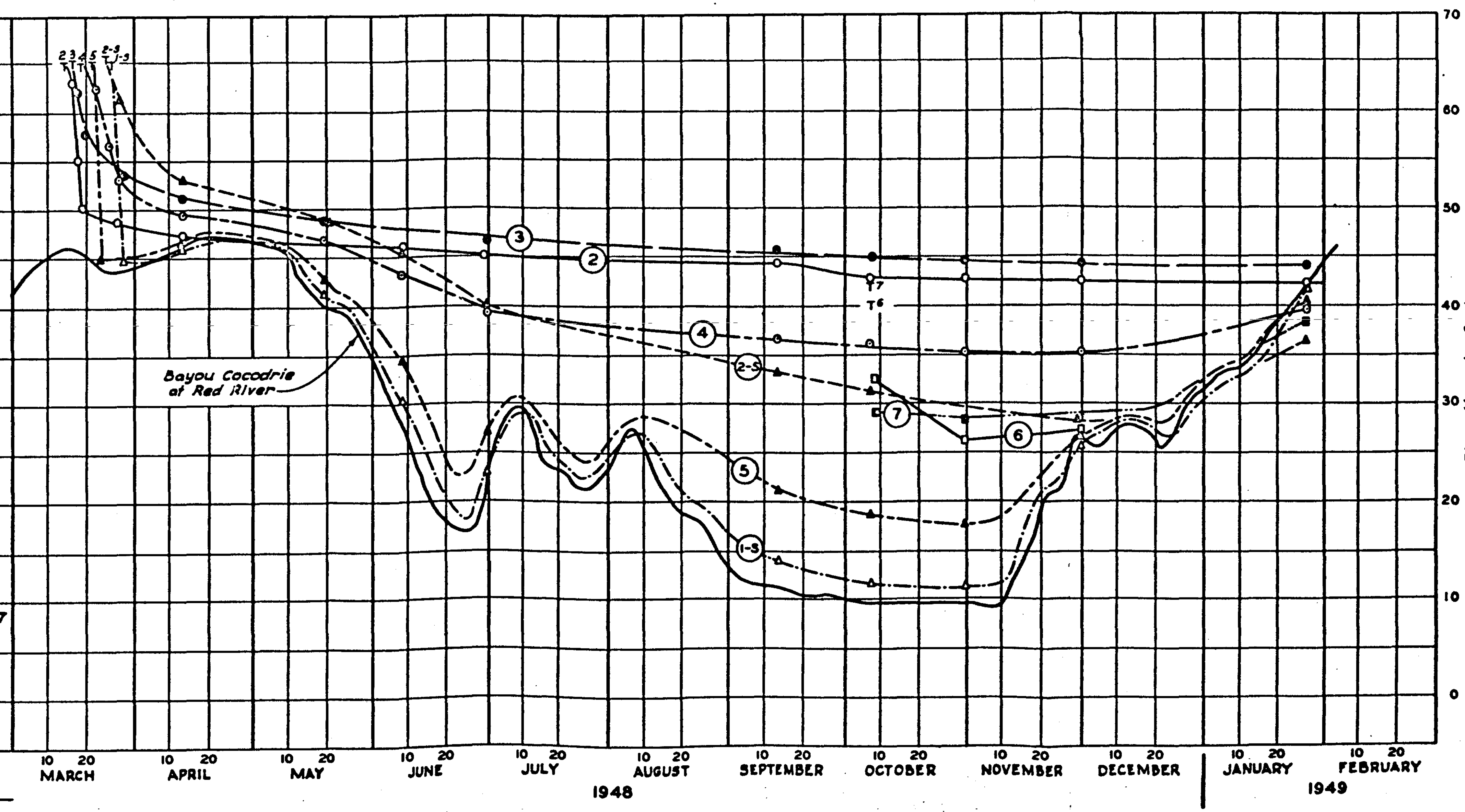
FIGURES TO RIGHT OF BORINGS ARE MAXIMUM SHEAR STRENGTHS OF UNDISTURBED SAMPLES (φ IN DEGREES, COHESION IN TONS/SQ FT). "QT" DESIGNATES QUICK TRIAXIAL COMPRESSION TESTS. OTHER VALUES SHOWN ARE FROM UNCONFINED COMPRESSION TESTS.

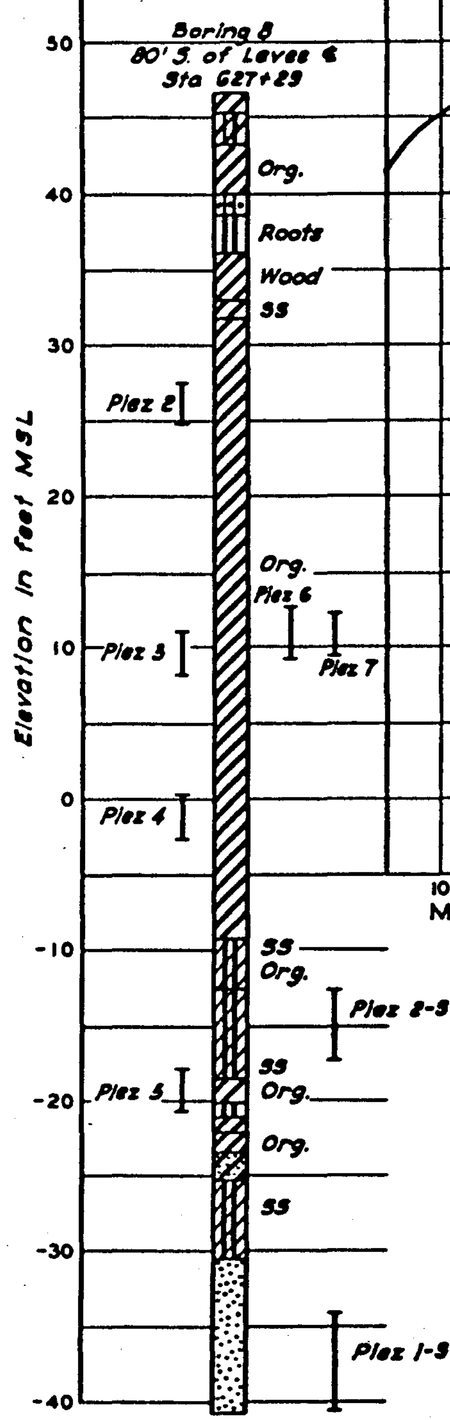
BAYOU COCODRIE DRAINAGE STRUCTURE

PLAN AND BORING LOGS

SCALES AS SHOWN

MAY 1949 FILE 3133





LEGEND

- ○ Piezometer 2
- ● Piezometer 3
- ⊙ Piezometer 4
- △ Piezometer 1-S
- ▲ Piezometer 5
- ▲ Piezometer 2-S
- □ Piezometer 6
- ■ Piezometer 7

T Indicates date installed and elevation of top of pipe.

I Limits of sand backfill around or beneath piezometer tip.

See plate 3 for symbols used in graphical log.

PIEZOMETER LOCATION

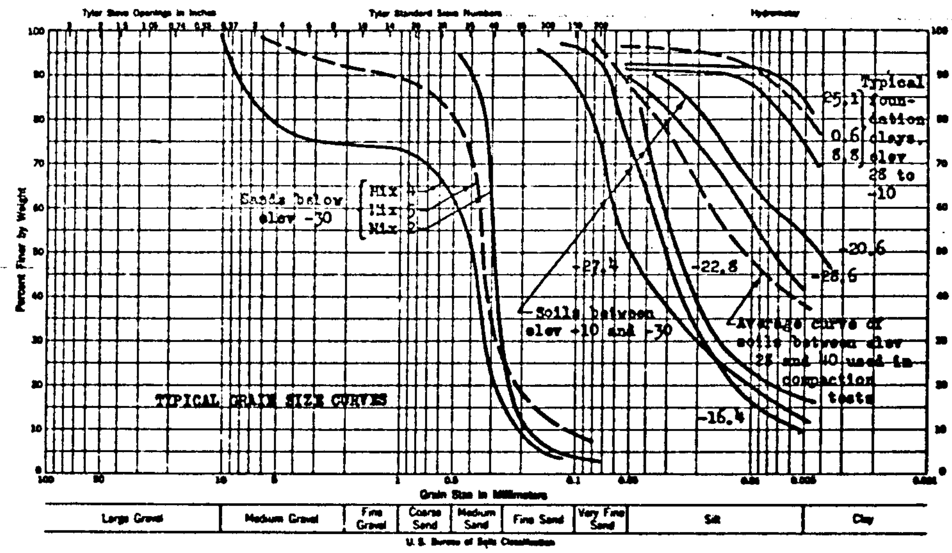
No	LEVEE STATION	DISTANCE FROM LEVEE ℄
2	628+39	15 ft Landside
3	628+34	15 ft Landside
4	628+29	15 ft Landside
5	628+24	15 ft Landside
6	628+29	180 ft Landside
7	628+29	300 ft Riverside
1-S	627+28	15 ft Landside
2-S	627+29	15 ft Landside

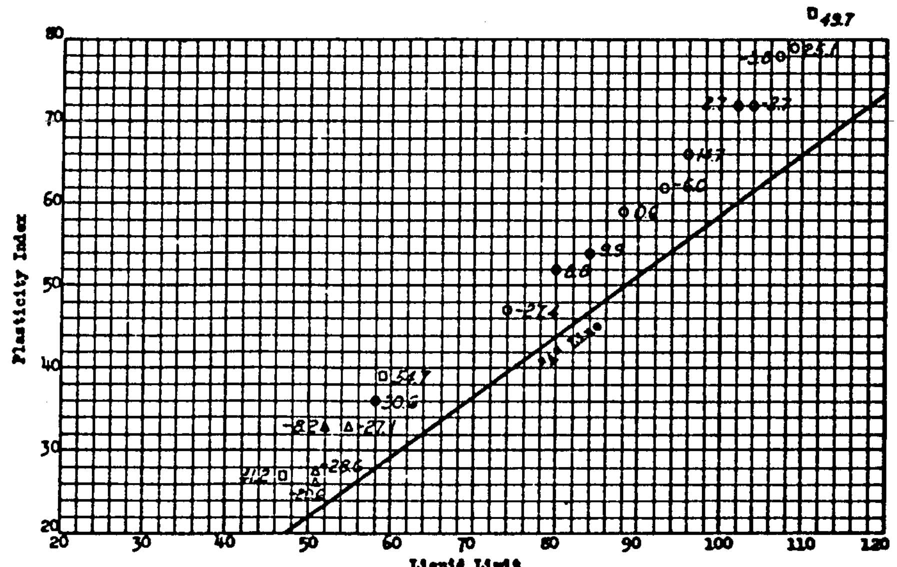
BAYOU COCODRIE DRAINAGE STRUCTURE

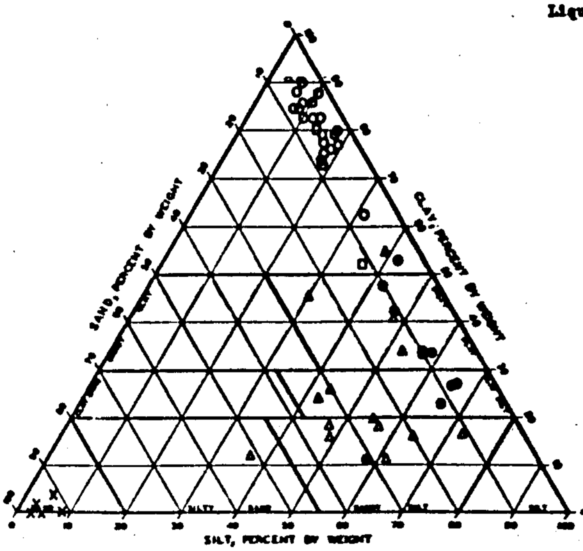
PIEZOMETER OBSERVATIONS

MAY 1949 FILE 3133

PLATE 4







SOILS CLASSIFICATION TRIANGLE
LOWER MISSISSIPPI VALLEY DIVISION

Legend

□ = Levee material (above elev 40)
● = Red River deposits elev 28 to 40
O = Foundation clays elev -10 to 28
▲ = Heterogeneous soils elev -30 to -10
X = Sands below elev -30

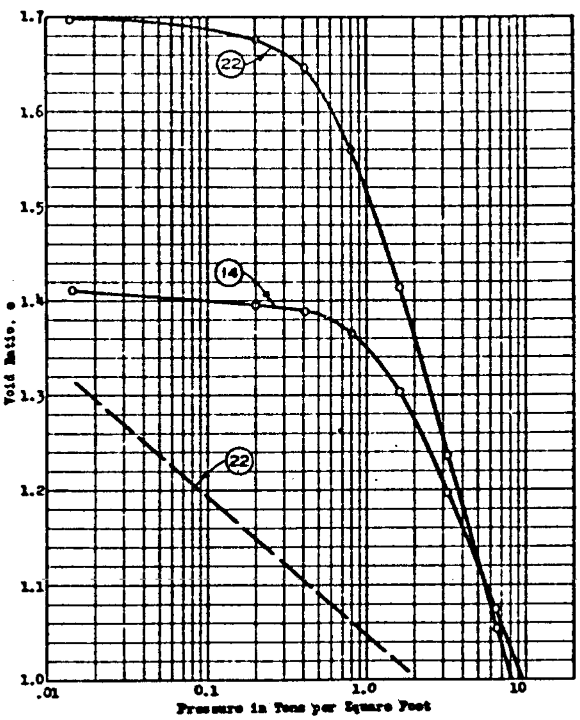
Note: Figures shown on plots are elevations in feet msl of samples tested.

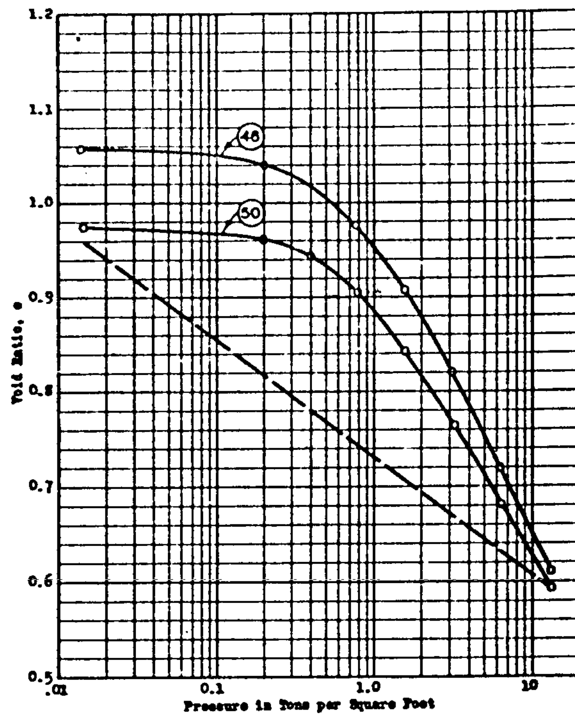
BAYOU COCODRIE DRAINAGE STRUCTURE

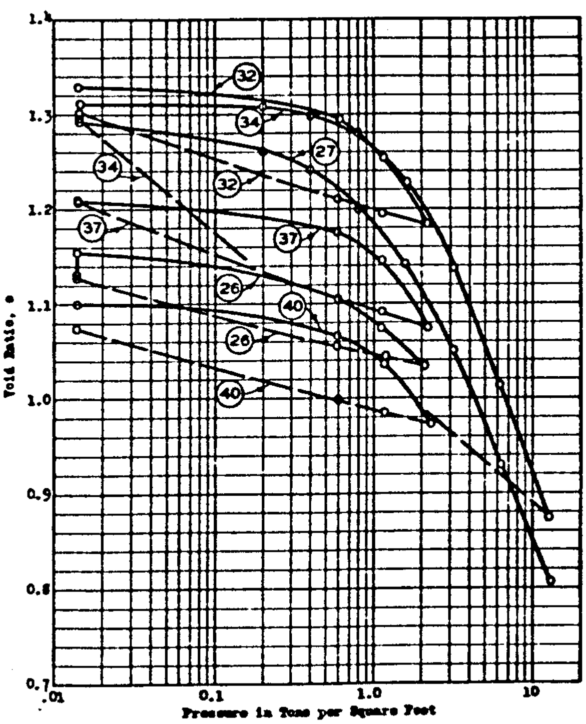
SOIL CLASSIFICATION DATA

MAY 1949 FILE 3133

PLATE 5







Sample	Elev Ft, msl	Classification	w	C_c	C_s	P_c T/Ft^2	P_o* T/Ft^2
14	25.1	Clay	55	0.46	—	1.7	1.1
22	14.7	Clay	65	0.58	—	0.8	1.3
26	9.9	Clay	41	—	0.045	-	1.5
27	8.8	Clay	49	0.49	—	1.2	1.5
32	2.7	Clay	48	—	0.055	-	1.6
34	0.6	Clay	50	0.45	—	1.8	1.7
37	-2.7	Clay	45	—	0.060	-	1.7
40	-6.0	Clay	40	—	0.045	-	1.8
46	-20.7	Silty clay	41	0.34	—	0.9	2.2
50	-28.6	Silty clay	36	0.27	—	0.8	2.4

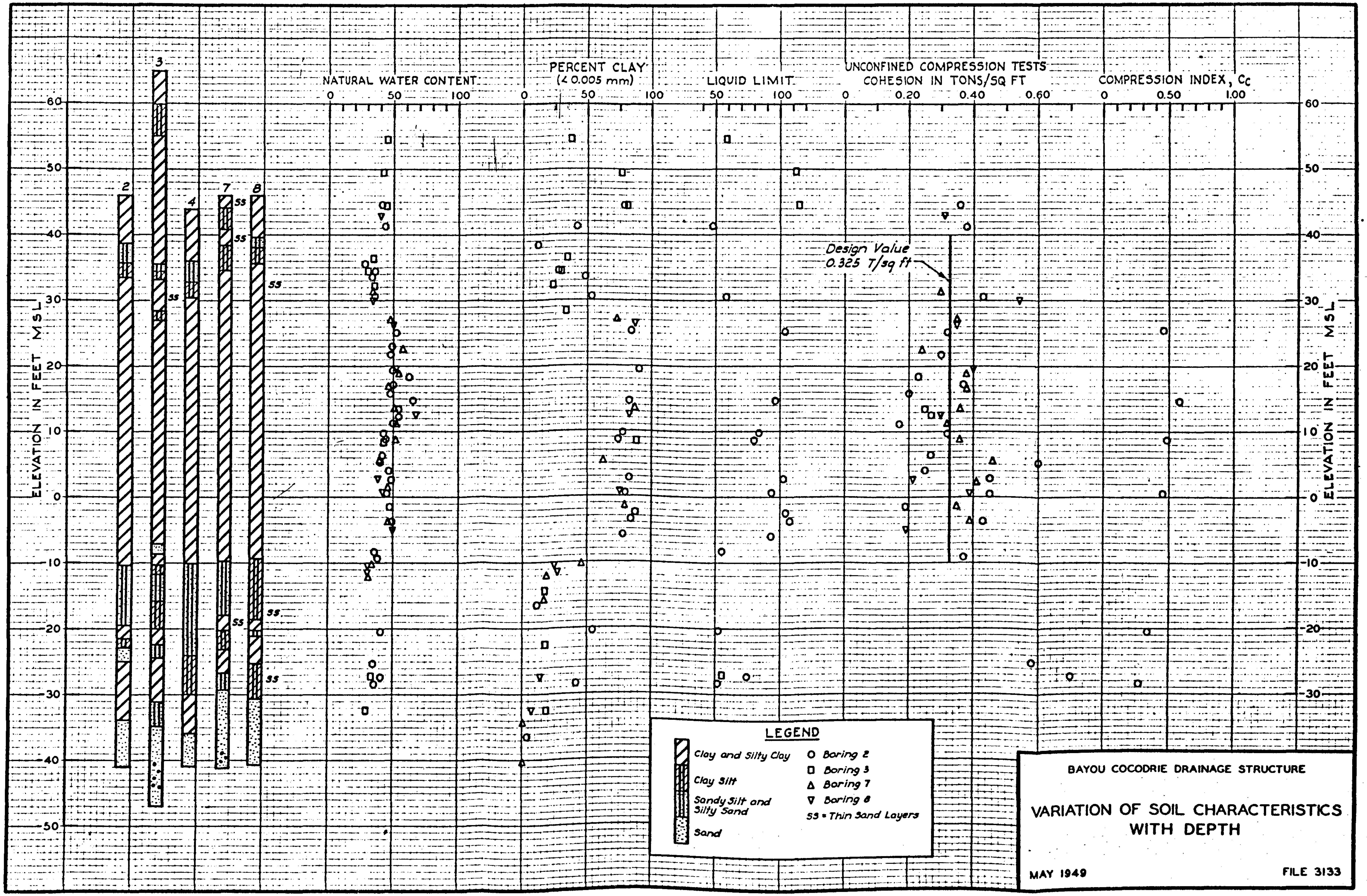
* Assuming 50 per cent consolidation under existing levee.

w = Natural water content
C_c = Compression index
C_s = Swelling index
P_c = Preconsolidation pressure
P_o = In situ pressure (assuming 50 per cent consolidation under existing levee).

BAYOU COCODRIE DRAINAGE STRUCTURE

PRESSURE-VOID RATIO CURVES UNDISTURBED SAMPLES BORING 2

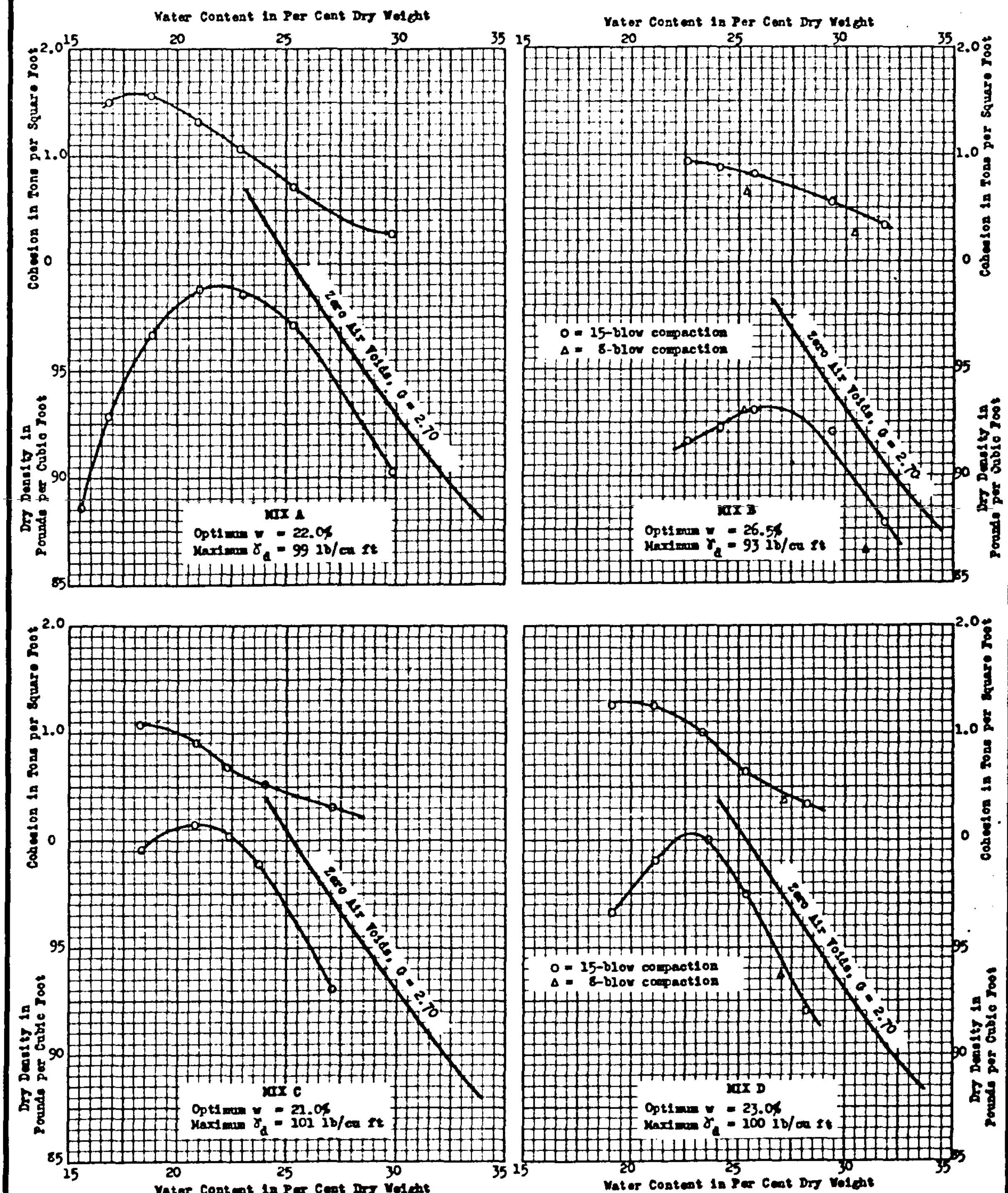
MAY 1949 FILE 3133



BAYOU COCODRIE DRAINAGE STRUCTURE

VARIATION OF SOIL CHARACTERISTICS WITH DEPTH

MAY 1949 FILE 3133



Mix	Boring	Elev Ft, msl	% Sand	% Silt	% Clay	LL	PL	PI
A	7	30-34	6	52	42	47	19	28
B	7	34-40	16	46	38	55	18	37
C	8	32-41	20	45	35	48	18	30
D	3	32-35	10	52	38	50	17	33

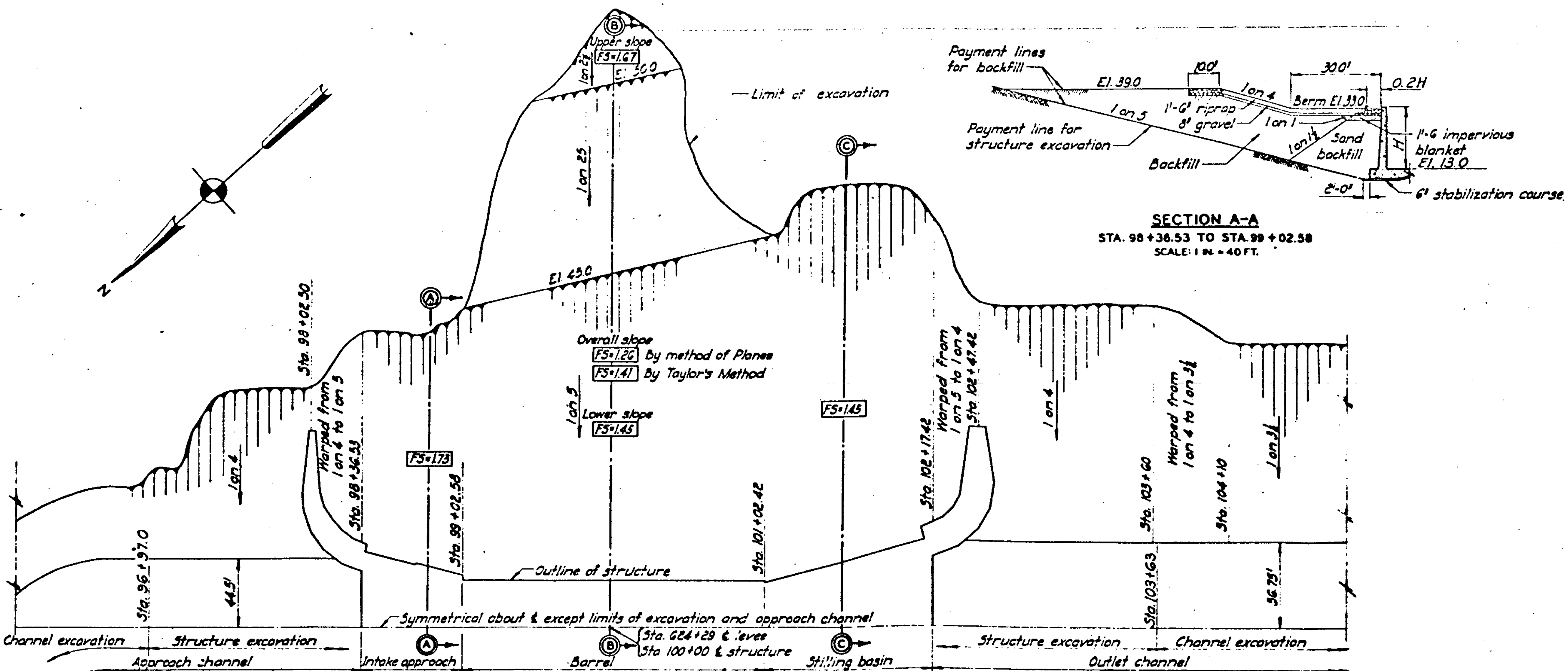
Note: All mixes classified as silty clays. Cohesion = one-half unconfined compressive strength.

BAYOU COCODRIE DRAINAGE STRUCTURE

COMPACTION AND REMOLDED SHEAR STRENGTH DATA

MAY 1949 FILE 3133

PLATE 8



SECTION A-A

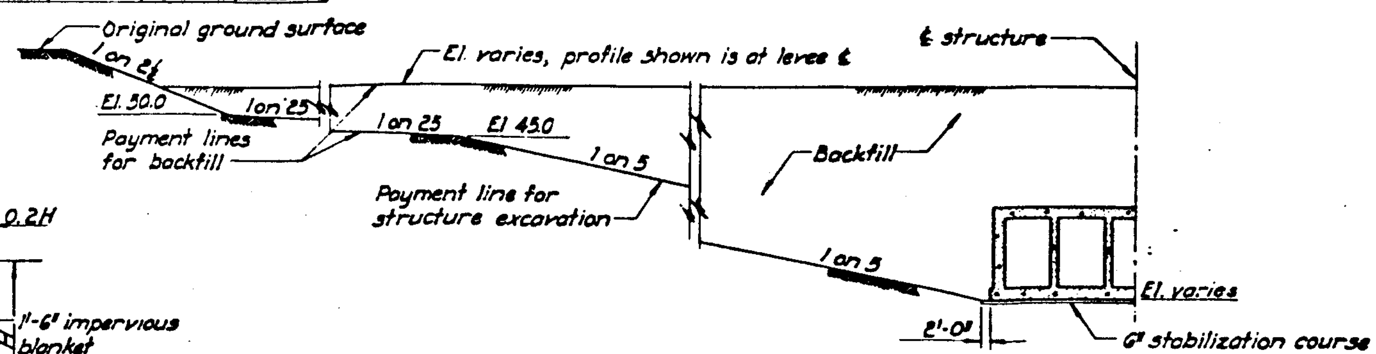
STA. 98+36.53 TO STA. 99+02.58

SCALE: 1 IN. = 40 FT.

HALF PLAN

SCALE: 1 IN. = 80 FT.

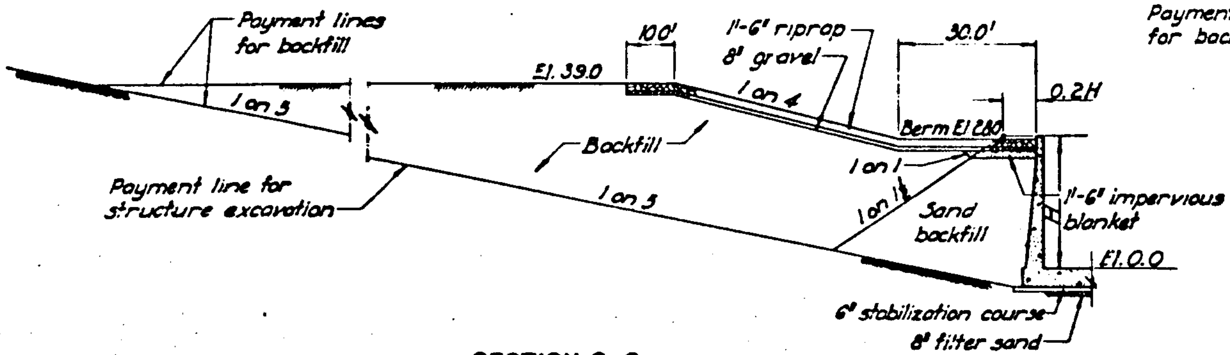
40 FT. 0 40 80 120 160 200 FT.



SECTION B-B

STA. 99+02.58 TO STA. 101+02.42

SCALE: 1 IN. = 40 FT.



SECTION C-C

STA. 101+02.42 TO STA. 102+17.42

SCALE: 1 IN. = 40 FT.

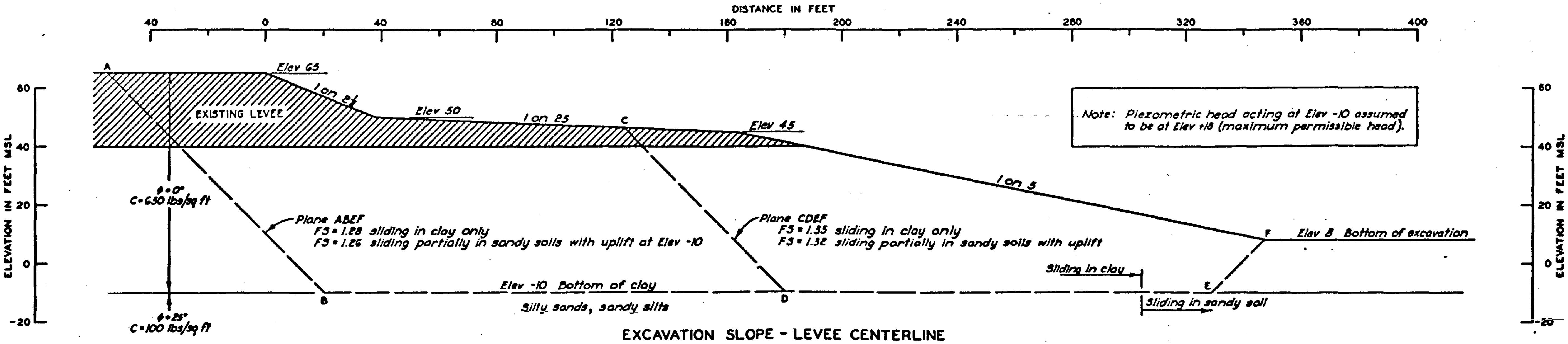
20 FT. 0 20 40 60 80 100 FT.

BAYOU COCODRIE DRAINAGE STRUCTURE

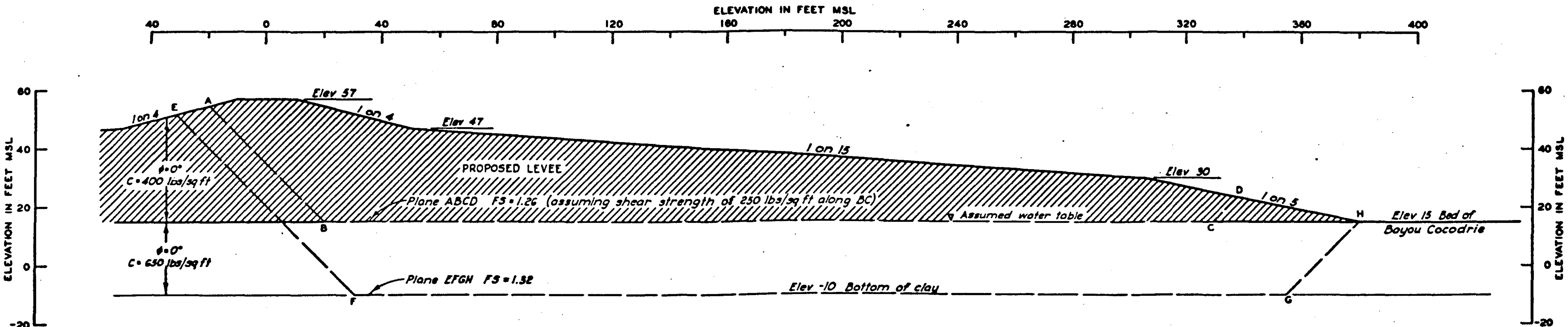
EXCAVATION PLAN AND SECTIONS

MAY 1949

FILE 3133



EXCAVATION SLOPE – LEVEE CENTERLINE



LEVEE SECTION – BAYOU CROSSING

BAYOU COCODRIE DRAINAGE STRUCTURE

STABILITY ANALYSIS OF EXCAVATION SLOPES AND LEVEE SECTION

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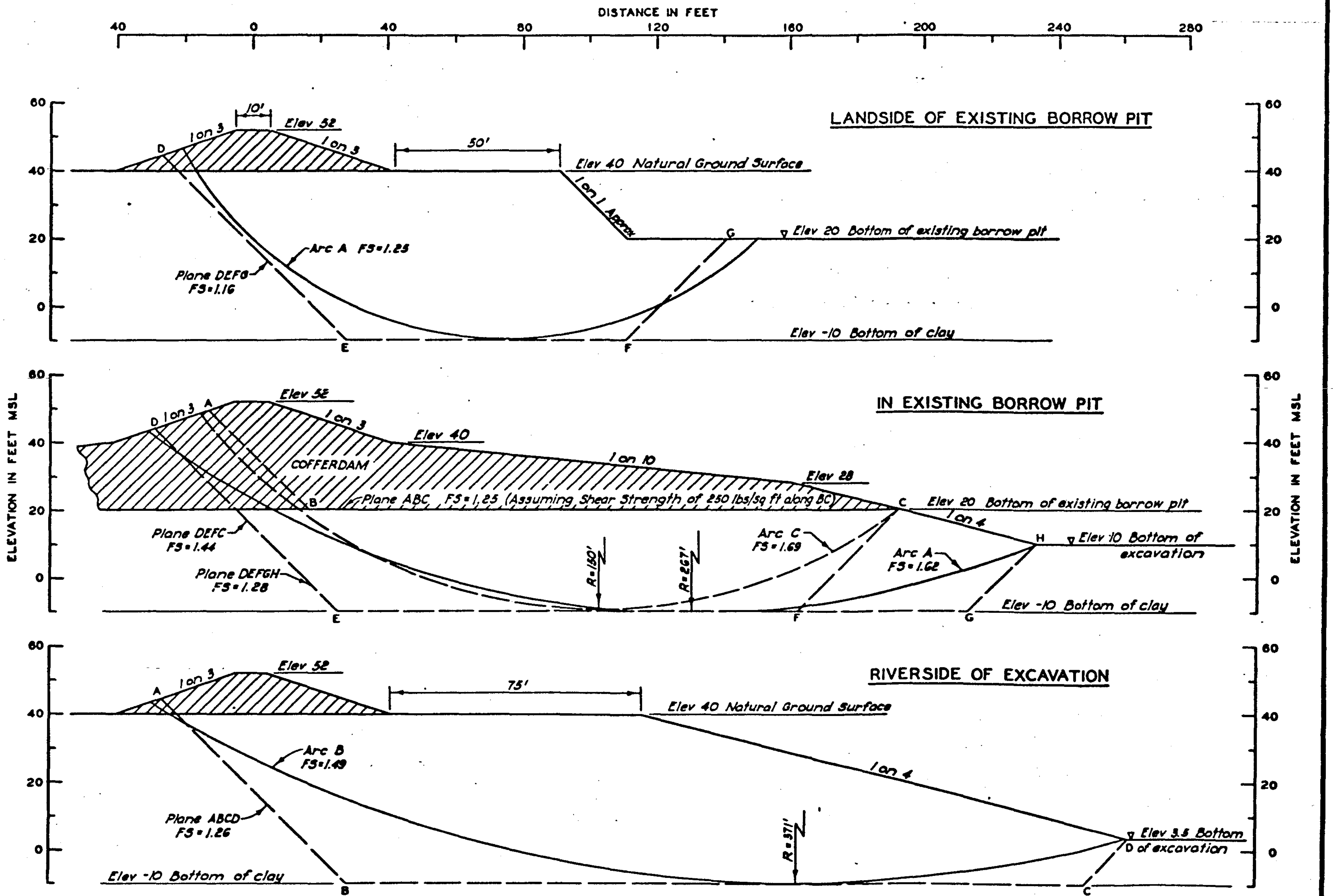
PLATE 10



Data: Shear Strengths: Cofferdam $\phi = 0$ C = 400 lbs/sq ft
Foundation $\phi = 0$ C = 650 lbs/sq ft
Unit Weights: Saturated or wet 110 lbs/cu ft
Submerged 47.5 lbs/cu ft
▽ = Assumed water table elevation

BAYOU COCODRIE DRAINAGE STRUCTURE

STABILITY ANALYSES OF COFFERDAM

MAY 1949

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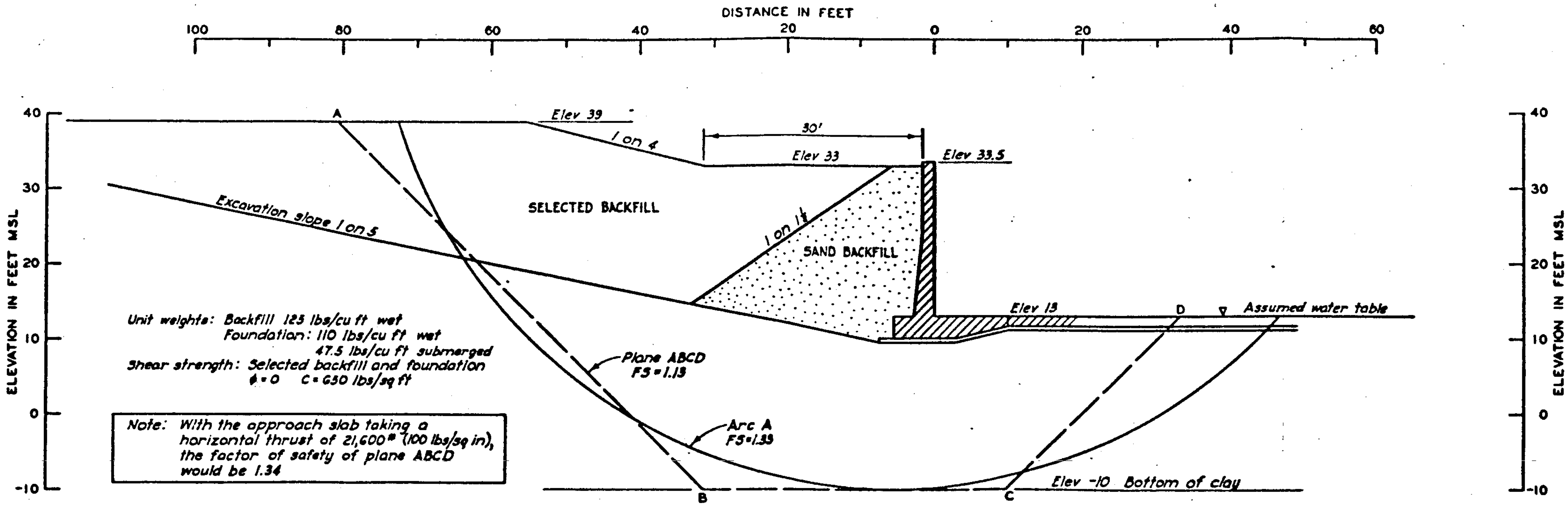
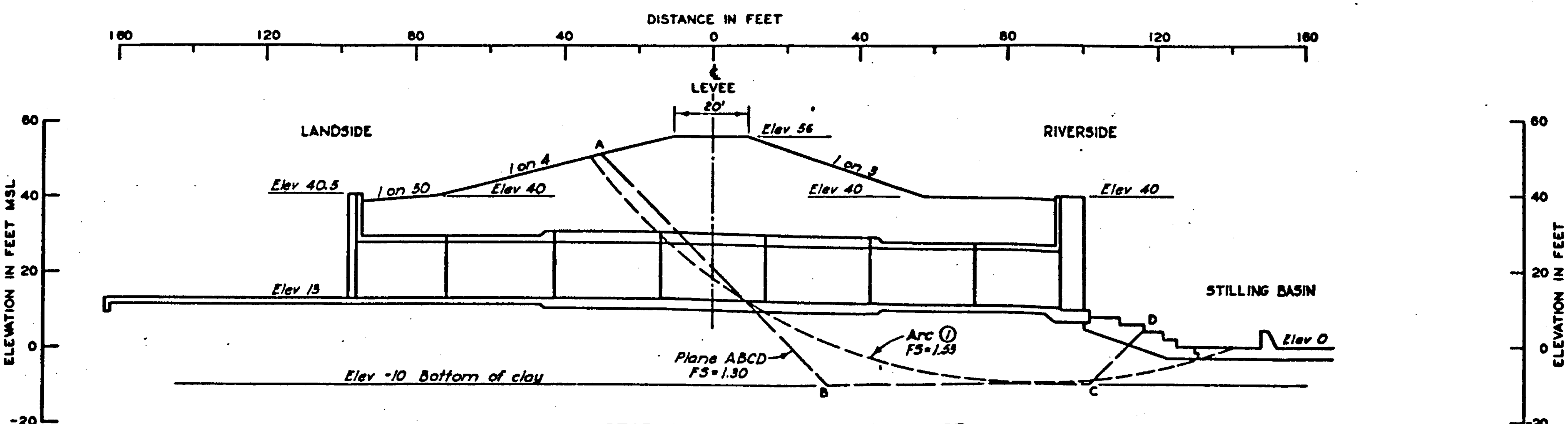
PLATE 11



STABILITY ANALYSIS
INTAKE APPROACH WALL - DEEP SLIDING



STABILITY ANALYSIS OF STRUCTURE
CENTERLINE PROFILE

Note:
1. Shear strengths: Levee, backfill and foundation φ = 0 C = 650 lbs/sq ft.
Zero shear strength assumed for portion of sliding planes passing through barrels.
Shear strength of chute and stilling basin slabs assumed to be same as earth.
2. Horizontal restraint of stilling basin walls and slab not considered in analysis.
3. Tensile resistance of reinforcing steel in base of barrels: 135,000 lbs/ft width of structure.
4. Unit weights: Levee and backfill: 125 lbs/cu ft wet
Foundation 47.5 lbs/cu ft submerged
Maximum conduit section: 53 lbs/cu ft
Minimum conduit section: 47 lbs/cu ft
5. Weight of tower monolith plus backfill: 66,650 lbs.
6. Water table assumed to be at base of barrels.

BAYOU COCODRIE DRAINAGE STRUCTURE

STABILITY ANALYSIS
STRUCTURE AND APPROACH WALL

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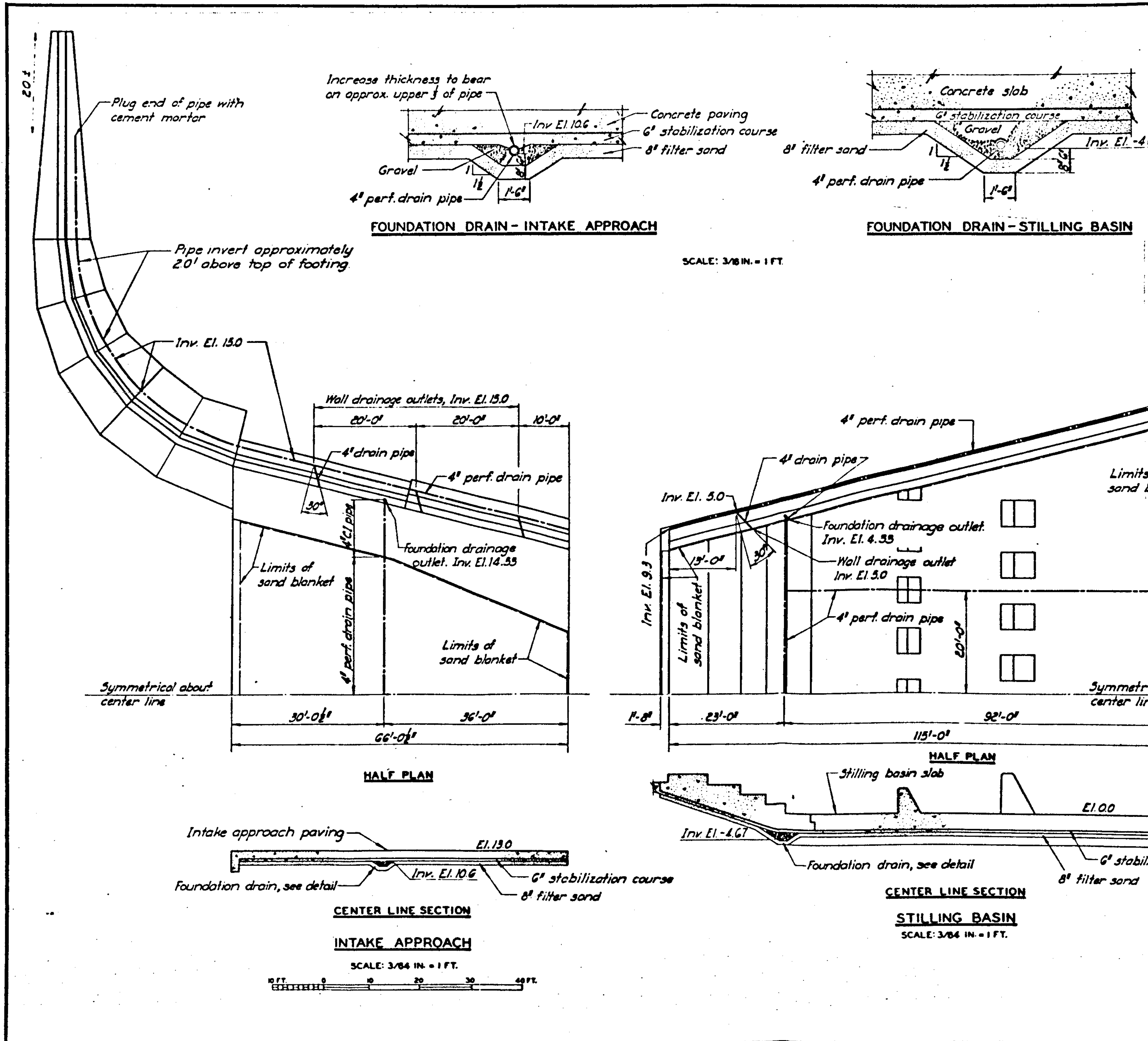

BAYOU COCODRIE DRAINAGE STRUCTURE

FOUNDATION AND BACKFILL DRAINAGE SYSTEMS

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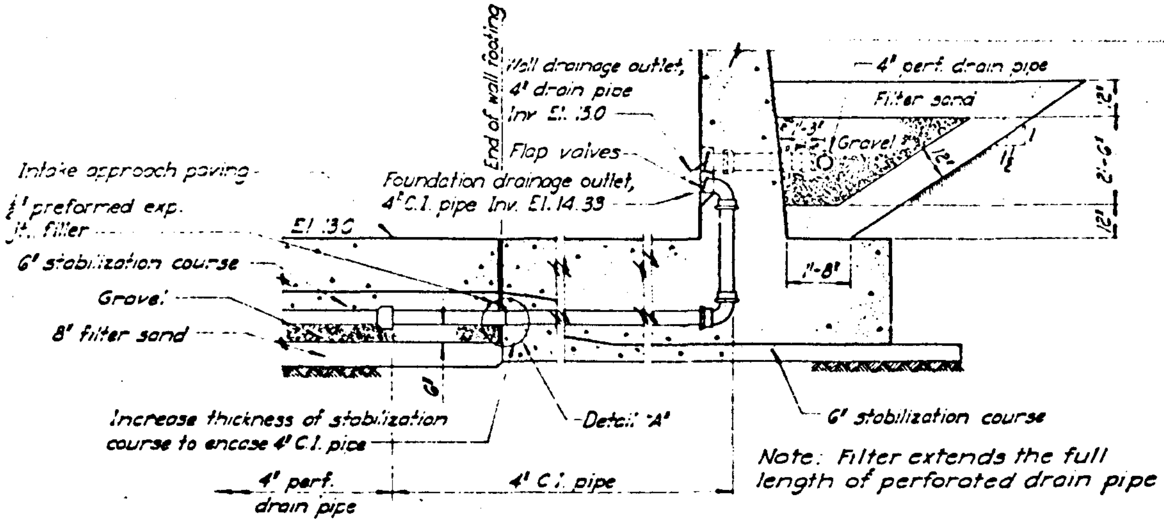
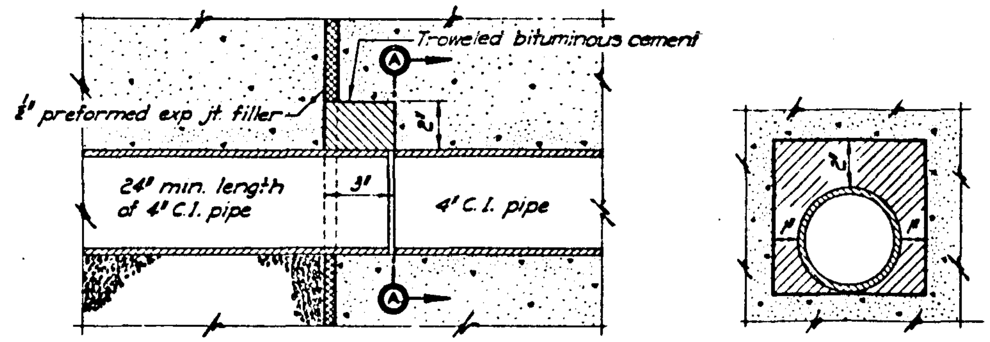


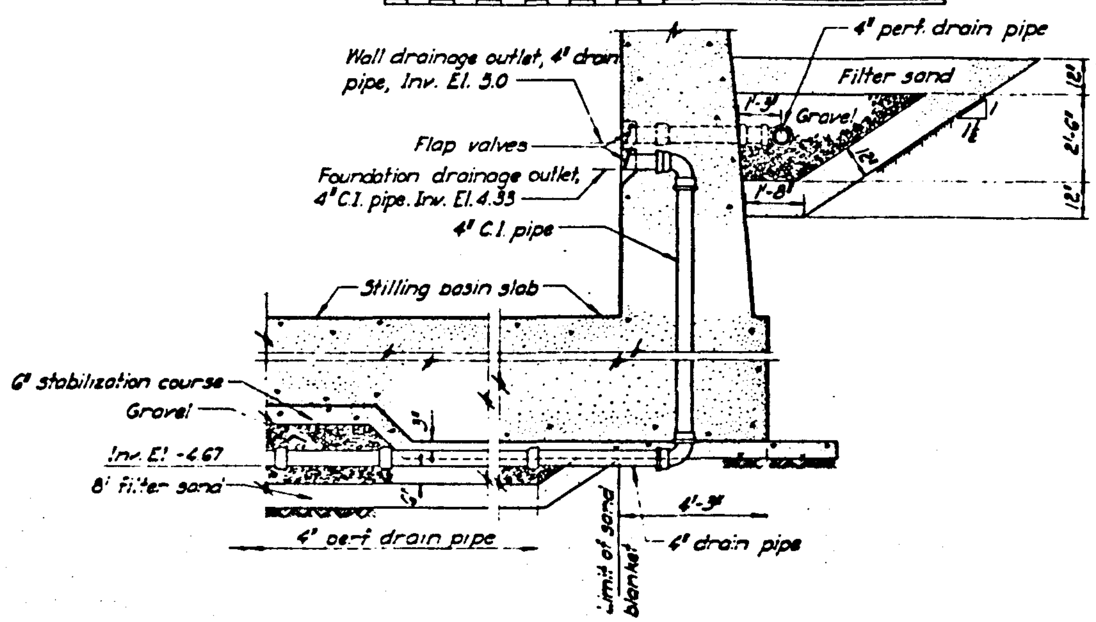
TYPICAL INTAKE APPROACH OUTLETS

SCALE: 3/16 IN. = 1 FT.



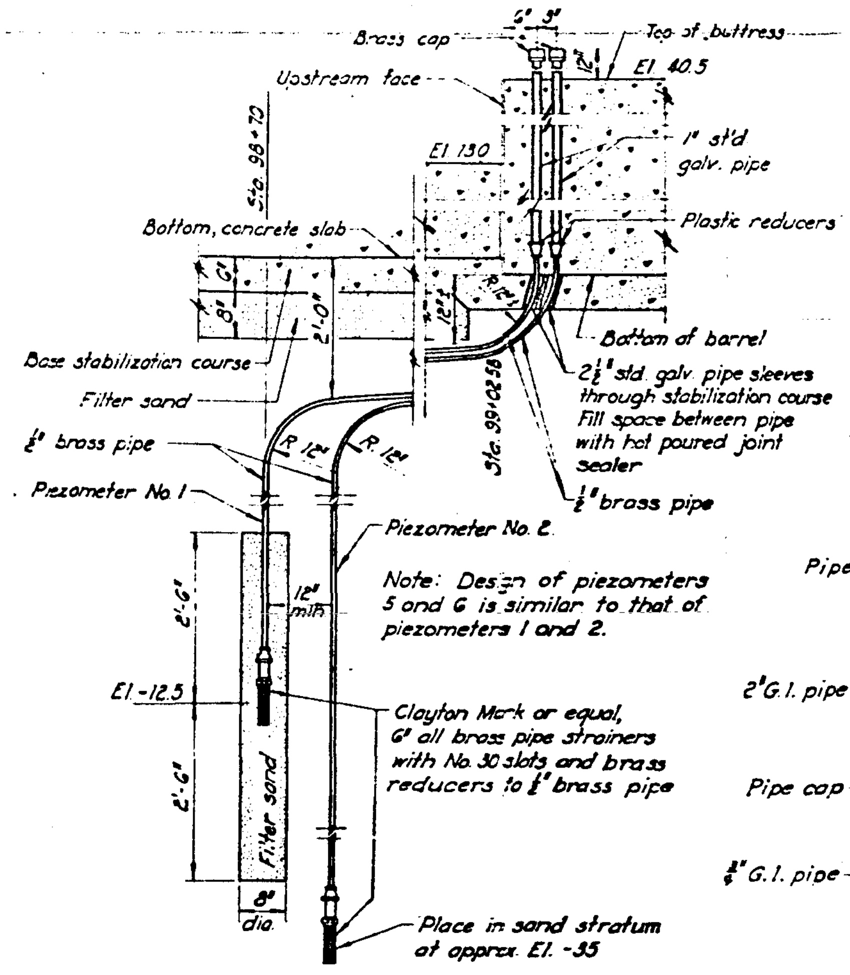
DETAIL "A"

SCALE: 1 1/2 IN. = 1 FT.



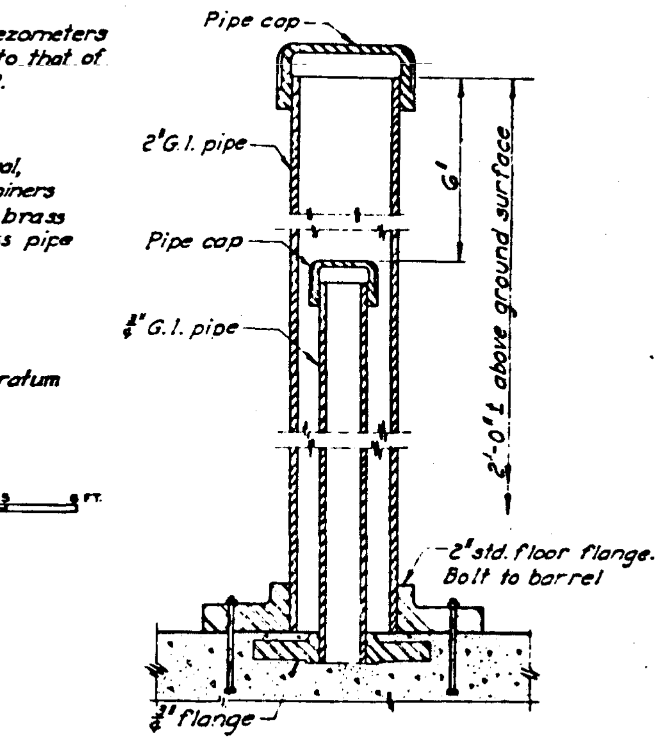
TYPICAL STILLING BASIN OUTLETS

SCALE: 3/16 IN. = 1 FT.



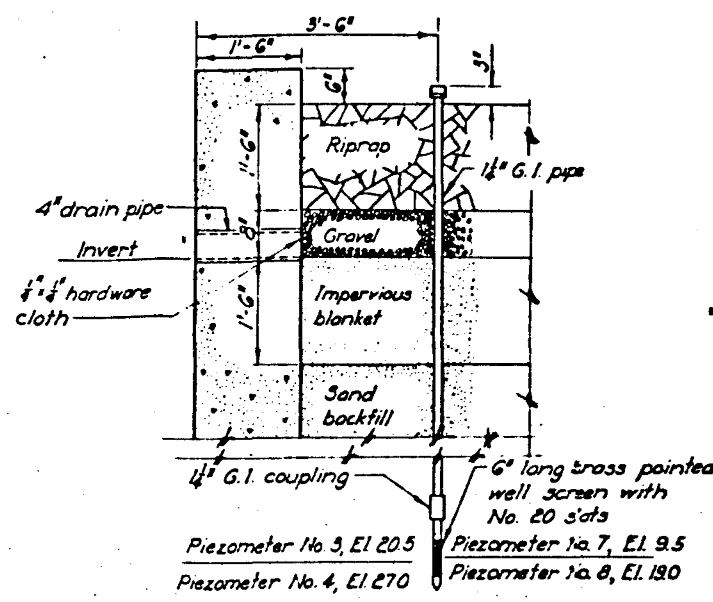
PIEZOMETER NOS. 1 & 2

SCALE: 3/8 IN. = 1 FT.



SETTLEMENT PLATE

SCALE: 3 IN. = 1 FT.



PIEZOMETER NOS. 3, 4, 7, & 8

SCALE: 3/8 IN. = 1 FT.

BAYOU COCODRIE DRAINAGE STRUCTURE

PIEZOMETERS, OUTLET DRAINS AND SETTLEMENT PLATE

MAY 1949

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PLATE 14