CORPS OF ENGINEERS, U. S. ARMY

MISSISSIPPI RIVER COMMISSION

COMBINED MORGANZA FLOODWAY CONTROL STRUCTURE, TEXAS AND PACIFIC RAILROAD AND LOUISIANA STATE HIGHWAY NO. 30

SOILS INVESTIGATION

TECHNICAL MEMORANDUM NO. 3-278

WATERWAYS EXPERIMENT STATION

VICKSBURG, MISSISSIPPI

MAY 1949

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SOILS INVESTIGATION

COMBINED MORGANZA FLOODWAY CONTROL STRUCTURE
AND
TEXAS & PACIFIC RAILROAD AND LOUISIANA STATE HIGHWAY NO. 30

History, Authorization, and Location of Project

1. The originally proposed plan for the Morganza Floodway project included a separate control structure at its head located adjacent to the main-line levee of the Mississippi River, and a high-level crossing for Louisiana State Highway No. 30 and the Port Allen Branch Line of the Texas and Pacific Railroad from Morganza to Lacour. The locations of these features are indicated on plate 1. The crossing was to consist of a combined embankment which would extend into the floodway a short distance and connect with separate reinforced concrete trestles for the highway and railroad. The trestles would be about 18,750 ft in length. Initial construction of the combined embankment was started in 1941 but construction was stopped in 1943 due to cessation of civil work as a result of the war. No work was performed on the trestles except for pile tests. During and following construction of the embankment, a few embankment and foundation failures occurred which indicated the need for further investigation of the stability of the embankment before it was brought to grade.

2. In April 1946 the Waterways Experiment Station was authorized to check the stability of the existing embankment, to analyze the slides which had occurred and to recommend further corrective measures if necessary, to design an extension of the embankment north of Sta 612+00 for 6000 ft, and to investigate borrow materials for this extension. The

results of these investigations appeared in a report entitled "Investigation and Design of Texas and Pacific Railroad (Port Allen Branch Line) and Louisiana State Highway No. 30 Joint Embankment, Morganza Floodway Crossing, Morganza, Louisiana," dated 7 August 1946.

3. The results of the foundation investigations and design for a separate control structure at the head of the floodway are contained in the following reports prepared by this office - Interim Report "Foundation Investigation and Design, Morganza Floodway Control Structure," dated 20 January 1947; Interim Report II "Foundation Investigation and Design - Morganza Floodway Control Structure," dated 7 May 1947; and, Interim Report III "Preliminary Soil Investigation of Foundation - Line D- Morganza Floodway Control Structure," dated 24 July 1947.

Purpose of present investigation

4. Since hydraulic model studies of the separate control structure indicated that the design floodway discharge of 600,000 cfs would not pass through the control structure at the originally proposed location, because of the lack of carrying capacity of the upper part of the floodway, it was decided to combine the control structure with the railroad and highway embankment at Morganza (see plate 1). In November 1947, this office was directed to investigate the foundation for the proposed control structure, the foundation and the borrow materials for the proposed embankment extensions, and to design the sections for these extensions. Also authorized was a review of the report of 7 August 1946, since the proposed plan of employing a combined control structure results in the combined approach embankments located within the floodway becoming part

of the main-line levee system of the Mississippi River and because the grade of the embankments was raised 1-1/2 ft above the original grade.

Field Explorations

Embankment and foundation borings

5. The site of the project lies in the backswamp region west of the Mississippi River near Morganza, Louisiana, as indicated on plate 1. The foundation consists of a deep stratum of highly plastic clay, varying in depth from about 60 to 70 ft at the south end of the embankment to approximately 80 to 90 ft under the north embankment, underlain by clean sand. The water table as indicated by five piezometers was about 6 to 9 ft below the ground surface during the summer months of 1948. Along the site of the existing embankments and proposed extension, 15 undisturbed and 3 auger borings were made in 1946 to obtain information for designing the extension to the south embankment and reviewing the stability of the existing embankments. The locations of these borings are shown on plate 2; logs of the borings may be found in the 1946 "Branch Line" report referred to in paragraph 2. Ten borings were made in February and March, 1948, along the site of the control structure. Five of the borings, three of which (M-19, M-27, and M-31) were of the undisturbed sample type, extended 5 to 10 ft into the underlying sand stratum (a depth ranging from about 70 to 90 ft). Two general sample borings were made to a depth of 40 ft and three to a depth of 15 ft. Seven additional undisturbed borings were made in 1948 to obtain information for design of the embankment not previously investigated. The logs of all borings made in 1948 and all railroad center line borings made in 1946 except M-7, M-14,

and M-15 are shown on plates 3, 4, and 5. Borings made in 1938 and 1939 by the New Orleans District to investigate the foundation for the railroad and highway crossings were used to supplement the more recent borings. In 1940 ten 30-in. hollow octagonal concrete piles were driven to sand to investigate the foundation conditions for the then proposed trestle. Three standard split spoon drive sample borings (M-36, M-27, M-38) were made in 1948 at the site of the control structure as part of the pile testing program for the control structure, in order to locate more definitely the top of the sand, and to determine the relative density of the underlying sand. Undisturbed samples were taken from the first 25 ft of boring M-37. The logs of these borings may be found on plate 6.

Borrow area borings

6. In order to avoid having to haul borrow material across the existing highway and railroad, it was originally planned to take all borrow material from areas west of the railroad. In 1946, twenty auger borings in Borrow Area "E" (MB-1 through MB-20) 15 ft deep were made between Sta 616+00 and 673+00 on a line 893 ft west of the existing railroad center line to determine the suitability of the soil for use in constructing the proposed 6000-ft extension of the south embankment. For the same purpose three lines of auger borings in Borrow Area "D" (MB-21 through MB-80) were made in March 1948 along the north embankment between Sta 723+00 and 799+00 at distances of 750, 1150, and 1550 ft west of the center line of the existing railroad. Sack samples adjacent to eight selected borings in Borrow Area "D" were taken in July and August 1948.

Logs of these borings appear on plate 7, of this report, and plate 13 of the 1946 report. Investigation of the material from these borrow areas revealed that the soil was not particularly suitable for the construction of an embankment. In the hope that better material could be found east of the railroad, the New Orleans District made investigations of Borrow Areas "A", "B", "C" (borings 1 through 51) in October and November 1948. The logs of these borings are shown on plate 8. The location of all above-mentioned borings appear on plate 2.

Soil profile

7. The foundation of the south embankment consists of backswamp deposits of highly plastic clays with some clay silts underlain at a depth of about 60 to 70 ft by sand. This clay has a tendency to be slightly fissured and contains numerous silt seams and strata that accelerate its rate of consolidation when loaded. The foundation of the north embankment is similar to that under the south embankment except that the clay stratum is thicker, ranging from about 80 to 90 ft, and contains several silt strata.

8. Under the control structure the soil profile is fairly uniform; a generalized soil profile based on the borings shown on plate 4, is described below.

Ground surface (elev 32) to elev 9. Between elev 32 and elev 9, the foundation consists of somewhat jointed, brownish-gray clays and silty clays. The average natural water content is about 47 per cent and the average Atterberg limits are: Liquid Limit (LL) = 87, Plastic Limit (PL) = 27, and Plasticity Index (PI) = 60.

Elev 9 to elev -1. Between elev 9 and elev -1 there is a stratum of brownish-gray sandy silts, silty sands, and clay silts, with an average water content of about 33 per cent.

Elev -1 to elev -33. Between elev -1 and elev -33 there is a stratum of gray clay with some silty clay which has an average water content of 43 per cent except for a 5 ft stratum from about elev -9 to -16 which has an average water content of 61 per cent. This stratum also extends beneath the north and south embankment extensions. The average Atterberg limits of this thin stratum are: Liquid Limit = 107, Plastic Limit = 35, Plasticity Index = 72; the over-all average values for the major stratum are: Liquid Limit = 63, Plastic Limit = 23, Plasticity Index = 40. Some roots and other organic matter were encountered in the stratum between elev -1 and -33.

Elev -33 to elev -45. A zone of clay silts, sandy silts, silty sands, and silty clays exists between elev -33 and -45.

Below Elev -45. A clean, uniform, fine to medium sand exists below elev -45. In general, the depth to this stratum increases toward the north end of the structure. The thickness of this stratum is not known, but from geological evidence it is great. Photographs of slices of typical samples from foundation borings M-22, M-27, and M-31 appear on plates 9 and 10.

Laboratory Tests

Foundation

9. General. Extensive laboratory testing was done in the investigation of the foundation beneath the proposed control structure and

embankments. Results of the tests performed on the samples obtained from the 1946 borings may be found in table 1 in the 1946 "Branch Line" report referred to in paragraph 2. Logs of the borings made along the south embankment and the accompanying laboratory data shown on plate 3 of this report have been taken from the aforementioned report.

10. Visual classification and natural water content determinations (except on sands) were made on all samples obtained from the foundation borings made in 1948. Classification tests (mechanical analyses and Atterberg limit tests) were made on selected samples. Seventeen consolidation tests were made on undisturbed samples of clay, and four were made on samples remolded at their natural water content. The shear test program consisted of 108 unconfined compression tests on undisturbed samples, six unconfined compression tests on remolded samples at natural water content, and eight triaxial tests. Five permeability tests were conducted on undisturbed specimens from the upper sandy-silt and clay-silt strata to determine the horizontal permeability of these strata. Specific gravity of the soil was determined in conjunction with each consolidation test. All data mentioned in this paragraph may be found tabulated in table 1.

11. Classification tests. Numerous Atterberg limit tests and mechanical analyses indicate that most of the soil underlying the proposed embankments and control structure is a fat, highly plastic, inorganic, clay (see plate 11). Other tests show the material to be also compressible and to have a high water content. Typical grain-size curves for the foundation soils may be found on plates 12 and 13. Since the 1946 borings were only made to limited depths, a comparison of the soils

between the guide levees can be made only to about elev -10. Averages of test results for clays and silty clays to this limited depth show the following:

	South Embankment	Control Structure	North Embankment
Per cent clay	71	61	60
Liquid limit (LL)	88	80	78
Plastic limit (PL)	27	26	27
Plasticity index (PI)	61	54	51

These data show a somewhat greater value for the average plasticity and clay content of the foundation soils under the south embankment than exists under the control structure and north embankment, which are about the same. The foundation under the north embankment is somewhat drier ($w = 35\% \pm 5\%$) than the south embankment ($w = 40\% \pm 5\%$).

12. For the foundation soils along the structure at this site there is a fairly good correlation between clay content and the liquid limit. This relation may be seen on plate 14.

13. Shear strength tests. Forty unconfined compression tests on undisturbed samples from the clay and silty clay strata beneath the structure gave an over-all average cohesive shear strength of 0.33 ton per sq ft. No consistent relation between strengths of samples from the upper and lower clay strata was found. Six tests on the upper clay stratum at boring M-31 indicated an average shear strength of 0.23 ton per sq ft, which is the only major deviation from the over-all average. As a check on these latter shear strengths, six unconfined compression tests were performed on undisturbed clay samples taken from the first 25 ft of the adjacent boring M-37. These six tests showed an average cohesive shearing strength of 0.32 ton per sq ft, although four had strengths

between 0.22 and 0.29 ton per sq ft. An average strength of 0.31 ton per sq ft was obtained from six unconfined compression tests on remolded samples taken at varying depths beneath the structure. These unconfined compression tests on remolded samples were run to determine the possible loss in shear strength due to remolding as a result of pile driving. In addition, these tests indicate the minimum shear strength that would be obtained in the embankment if the compaction used is sufficient to force the lumps of material together firmly. Four quick triaxial compression tests on samples from within the structure limits indicated an average strength of $\phi = 2°$ and c = 0.32 ton per sq ft. A shear strength of $\phi = 0°$ and c = 0.33 ton per sq ft was assumed for the clays beneath the structure. This value agrees closely with the frictional values computed from the 1940 pile tests to be discussed later.

14. On undisturbed specimens from borings beneath the north embankment, 68 unconfined compression tests gave an average cohesive shear strength of 0.35 ton per sq ft, and two quick triaxial tests from these borings gave an average strength of $\phi = 1°$ and c = 0.40 ton per sq ft. The triaxial tests mentioned in this and the above paragraphs are shown on plates 15 and 16. Under the north embankment, the control structure, and from Sta 672 to 680 under the south embankment, the average shear strength of the foundation borings ranged from about 0.30 to 0.40 ton per sq ft, except for the one boring under the control structure as mentioned above. This same range of shear strengths was indicated for all depths from the ground surface to about elev -20. For the embankments in this sector, a foundation shear strength of $\phi = 0°$ and c = 0.30 ton per sq ft was selected for design purposes.

15. On the basis of the data and results of the 1948 borings, a re-analysis of the south embankment foundation up to Sta 672+00 gave a shear strength of $\phi = 0^{o}$ and c = 0.25 ton per sq ft as being practicable south of Sta 672+00. This latter value, which is somewhat greater than that used in the 1946 analysis, was based on a consideration of the limits, mechanical analyses, water contents, and shear strength of tests run on the 1946 and 1948 borings (see table below). A few very low shear strengths in the 1946 data (see table 1 of the 1946 report) resulted in the selection of an average or design strength which now, on the basis of additional borings and analyses, appears to have been too low.

Range		Average Test Values									
Clay Content	Liquid Limit	Per Cent Clay		Liquid Limit		Plastic Limit		Natural w in Percent		Shear Strength T/Ft^2	
		1946	1948	1946	1948	1946	1948	1946	1948	1946	1948
> 75	> 100	79	83	104	107	27	32	54	66	0.25	0.25
65-75	85-100	74	69	92	92	26	29	46	46	0.25	0.37
55-65	70-85	61	62	79	79	20	26	47	40	0.15*	0.41
45-55	55-70	46	47	61	64	17	25	42	38	0.29	0.34

* Based on 6 specimens.

A stability analysis of existing embankments, assuming a factor of safety of unity, indicated also that the assumed shearing strength of 0.25 T/sq ft to be reasonable.

16. Consolidation tests. Consolidation tests on foundation samples under both embankment extensions and under the control structure indicate that the foundation is very compressible. This is verified by the 2- to 4-1/2-ft settlements under the existing embankments which have been observed since their construction.

17. Twelve consolidation tests on undisturbed clays and silty clays

from the foundation, excluding the thin high water content stratum, along the control structure and north embankment extension gave an average compression index (C_c) of 0.40. One of these samples had a C_c of 0.51 in the undisturbed state and had a C_c of 0.53 when remolded. Three tests on samples from the high water content clay stratum (elev -9 to -16) gave an average C_c of 0.71. Two specimens from this stratum, which were remolded at their natural water content before testing, had an average C_c of 0.65. The pressure-void ratio curves for these tests appear on plates 17 and 18.

18. Eight undisturbed samples along the south embankment extension showed an average C_c of 0.40. Water contents on samples obtained from the 1938 and 1939 borings in this vicinity show that the thin, highly compressible stratum under the north embankment (elev -9 to elev -16) mentioned earlier also exists under this portion embankment.

19. The preconsolidation pressures (P_c) as determined from the test data indicate that the soils above elev 0 are overconsolidated, as the preconsolidation pressures exceed the computed existing overburden pressures by about 0.5 to 1.0 tons per sq ft assuming a water table at 6 ft below ground surface. The brownish, oxidized color of these materials indicates probable overconsolidation as a result shrinkage upon drying either during deposition or subsequently.

20. Permeability tests. Four permeability tests on the sandy silt and clay silt stratum from elev 9 to elev -1 indicate horizontal permeabilities of 6 to 58 x 10^{-7} cm per sec. As some portions of the stratum are more granular than the specimens tested, a horizontal permeability of 100 x 10^{-7} cm per sec was used in the underseepage analyses. The vertical

permeability of the upper clay stratum was computed from consolidation tests on samples from this stratum. The permeability of four test specimens ranged from 5 to 25 x 10^{-9} cm per sec. The maximum value 25 x 10^{-9} cm per sec was used in the underseepage computations. A permeability ratio of $k_h/k_v = 16$ was assumed for the upper clay stratum.

21. Specific gravity. The specific gravity for the samples tested varied from 2.62 to 2.73 with an average of about 2.70.

22. Effect of remolding. A comparison of the shear and consolidation tests that were run on undisturbed and remolded specimens indicates that the foundation material has a sensitivity ratio* of about 1.0. The average shear strength indicated by six remolded specimens was 0.31 ton per sq ft, while the average strength from all the tests on undisturbed samples from beneath the structure was 0.33 ton per sq ft. The consolidation tests run on remolded samples indicated compression indices (C_c) almost identical with those from undisturbed samples.

Borrow materials

23. General. Most of the material to be used in the construction of the embankments consists of high water content, fat, inorganic plastic clays, and silty clays. The natural water content of the soil during the summer is 10 to 15 per cent above a 15-blow optimum for compaction. This material is slow to dry out and is subject to shrinkage and cracking. This may be seen in the results of classification tests and shrinkage limits in table 2. Grain-size curves of this material are shown on plate 19.

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* Ratio of unconfined compressive strength of undisturbed sample to that of remolded sample at same water content.

24. An investigation of the borrow areas east of the embankment site reveals that the materials found in area "A" are leaner than those in either "D" or "E". However, an analysis of the materials in areas "B" and "C" does not indicate an appreciable difference in materials as to warrant their usage in preference to that found in area "D". This conclusion is based on a comparison of the boring logs and on analysis of water contents, as demonstrated in the following paragraph.

25. The borings for area "D" were made in March 1948, at a time when the water table was high, which would tend to cause the water contents of the samples to be correspondingly high. Borings made in area "D" were actually under water in some instances. On the other hand, borings made during the first part of August 1948, for the purpose of taking sack samples, indicated appreciably lower water contents. The water table during the summer as indicated by piezometers in the area was approximately at a depth of 6 to 9 ft. The water contents between March and August (1948) showed an average drop of 15 per cent ($w = 50$ to $w = 35$) in the first 4 ft of depth and 10 per cent ($w = 41$ to $w = 31$) in the second 4 ft. By 11 October the average water table had dropped to a depth of approximately 9 to 10 ft. The results of averaging water contents for the first 8 ft in depth in borrow areas B, C, and D are tabulated below, the averagesbeing restricted to clays, silty clays, and clay silts:

	Water Content-Borrow Areas			
	D		C	B
Depth in ft	March	August	28 October	1 November
0-4	50	35	28	31
4-8	41	31	35	29

26. It is reasonable to assume that after an extended wet period, say from fall through spring, in which time the soil could saturate thoroughly by a rising water table and surface wetting, the water content in areas "B" and "C" would be comparable to those found in the spring in area "D". Since borrow area "E" is probably in a wet state for most of the year material from this area should be used in berm construction only. Borrow area "E" is considered comparable to the area east of the highway investigated by borings 44 through 50 (see plate 2).

27. Compaction tests. Five standard Proctor compaction tests modified by employing a 15-blow compactive effort were run on sack samples taken from Borrow Area "D". These tests were divided between clay and silty clay materials, which dominate the borrow areas as indicated by the logs of borings shown on plates 7 and 8 of this report and plate 13 of the 1946 report. Results of these compaction tests are shown on plate 20. The tests indicate an optimum water content of about 23 per cent and a dry density of 98 lb per cu ft for the silty clay material, and an optimum water content of about 30 per cent with a dry density of approximately 89 lb per cu ft for the clays.

28. Shear tests. Shear strength tests were run on soil specimens compacted at water contents ranging between natural and optimum; the strengths obtained are plotted against water content on plate 20 and are summarized in table 2. Also plotted on plate 20 are the results of unconfined compression tests on remolded foundation samples from the north embankment taken from the first 15 ft of depth. Results of triaxial tests on compacted material are shown on plates 21 and 22. The results of these tests show greater shear strengths at lower water contents, and

greater strengths for the clay material than for the silty clays at the same water content.

29. An analysis was made of the water contents of the two predominate materials found in the borrow areas; namely, the clays and the silty clays. An interpolation of water contents of samples from borings made in March and August 1948 revealed than an average natural water content of 38 per cent for the clay material and 32 per cent for the silty clay material could normally be expected in the borrow pits during the construction season. On the basis of the shear data shown on plate 20 and assuming that the material would dry 3 per cent during placement, a shear strength of 450 lb per sq ft was selected as the design strength of the embankment south of Sta 672; 500 lb per sq ft was used north of Sta 672. The selection of the smaller shearing strength for the design of the south embankment was made before the borings in areas "B" and "C" were made, and before the decision was made to use materials from these areas for construction of the south embankment. These strengths are considered reasonable, due to the fact that shearing strengths obtained from undisturbed samples remolded at their natural water content were of this magnitude, as is shown on plate 20. This would indicate that as long as the lumps of borrow material are pressed together by the compaction process so that the void space is negligible, a shearing strength between 400 and 500 lb per sq ft can be obtained. No frictional resistance was assumed, although quick triaxial tests gave a friction angle of about 2°.

Design of the Foundation for the Control Structure

Piling

30. Pile loadings. As previous investigations indicated that excessive settlements would take place under a slab foundation at this site, a pile foundation is considered necessary to prevent detrimental settlement of the structure. For computing the pile loadings when the gates are closed and the maximum head is acting on the structure, uplift was assumed to decrease from 100 per cent of the net head plus tailwater head at the upstream edge of the base to 5 per cent of the net head, plus tailwater head, at a distance of 20 ft from the upstream edge of the base, and to remain uniform over the remainder of the base. The original design of the piling by the Design Branch of the Mississippi River Commission was based on a maximum compression load of 50 tons per pile and a maximum tension load of 14 tons per pile. The pile layout consisted of vertical and batter compression piles driven to sand and short friction batter piles to carry the tension load. However, subsequent cost studies indicated that higher compression loads with correspondingly fewer piles might be more economical. In addition, a more efficient design of the pile layout was obtained by eliminating the vertical piles and having both sets of batter piles driven to sand. A layout based on 100 tons compression load and 25 tons tension load with 18-in. piles is shown on plate 23.

31. Previous pile tests. In 1940, ten 30-in. hollow octagonal concrete piles were tested along the site preliminary to design of trestles for the railroad and highway high-level crossings. Eight of

these piles, which were driven into the deep sand, when loaded separately carried test loads of 210 tons per pile without failure, with total settlements of less than 0.05 ft. The one pile which failed was driven just to the top of the sand. This pile indicated an average friction along the pile of about 650 lb per sq ft at failure. One pile, which according to the penetration record did not reach sand, had an average friction of 600 lb per sq ft developed under the test load of 210 tons without failure.

32. Compression piles. To determine the feasibility of using friction piles, the pile lengths that would be required were computed. Using the average cohesion from the test data ($c = 0.33$ ton/sq ft) as the frictional strength, a factor of safety of 2.0 and a pile load of 50 tons, a curve of required penetration versus average pile perimeter was obtained (plate 24). This curve indicates than an 18-in. round pile would have to be driven to a penetration of 65 ft to obtain the required load capacity. As the average depth below the base of the structure to the sand stratum is 70 ft, the pile tips would be about 5 ft above the sand. However, as the top of the sand is higher under the south end of the structure, some piles of 65-ft penetration would penetrate into the sand and would not settle, whereas piles under the north end of the structure would be subject to some settlement. To compute the settlement that might take place with 65-ft friction piles, it was assumed that a uniform skin friction would be developed along the length of the piling. An estimate of the settlement was made at the north end of the structure where the underlying clay is thickest, employing consolidation data of samples from boring M-19. Using the laboratory recompression curve above elev -5 and the

virgin curve below elev -5, the settlement that might occur was computed to be about 10 in. for a clay stratum 80 ft thick beneath the base of the structure. With this amount of total settlement, excessive differential settlements might take place along the structure. Therefore, it was decided that the compression piles should be driven to firm bearing in the sand stratum, inasmuch as little additional length would be required. Pile tests made at this site in 1940 indicated that the driving resistance increased very sharply when the sand stratum was reached.

33. Tension piles. As no results of pulling tests are available, the frictional strength (in tension) of the foundation clays was assumed to be half of the average cohesion, or 0.16 ton per sq ft. Using a factor of safety of 2.0, a curve of required pile penetration versus average pile perimeter was developed for a pile load of 25 tons (see plate 24). From this curve it may be seen that 18-in. piles driven to sand will have greater than the required penetration. It is believed that the frictional resistance assumed for the tension piles is conservative and that a higher strength might be used if results of pulling tests were available.

34. Proposed pile tests. A pile testing program is proposed for two purposes. One is to determine the most economical size of compression pile, as cost estimate studies indicate that a pile load of 100 tons might be more economical than a 50-ton loading. The other purpose is to determine the allowable tension load which may be placed on the piles driven to sand. The recommended pile testing program is given in Appendix A; the plan of the pile tests is shown on plate 38.

Underseepage and drainage system

35. Underseepage. As the top clay stratum has a very low

permeability, the quantity of underseepage will be small. However, uplift pressures might develop in the sandy silt stratum (elev 9 to -1) and immediately beneath the base slab. Since uplift was assumed to decrease from 100 per cent of the net head plus tailwater head at the upstream edge of the structure to 5 per cent of the net head plus tailwater head at a distance of 20 ft from the upstream edge of the base, and to remain uniform over the remainder of the base for the design of the piling, studies were made of various drainage systems which will reduce the uplift on the base of the structure to less than that assumed. Three of these systems are discussed in the following paragraphs.

36. In order of increasing effectiveness, the first system studied consisted of a drainage blanket and no, or an ineffective, cutoff wall; another consisted of the same drainage blanket but with the cutoff wall assumed fully effective; and a third utilizes the drainage blanket and cutoff, plus a line of wells placed 50 ft downstream from the heel of the weir to relieve any excess hydrostatic pressure in the pervious silty stratum.

37. Drainage blanket. To relieve any excess hydrostatic pressures that might develop immediately beneath the base slab, a 10-in. blanket of filter sand F has been provided as shown on plate 25. This blanket will be drained by three 4-in. perforated clay collector pipes (1/4-in. holes) surrounded by 6 in. of gravel B. The original gradation of the sand for the blanket, designated as filter sand E, was not met by locally available sands as stated in the report "Survey of Concrete Aggregate and Filter Sands and Gravels, Morganza Control Structure and Bayou Cocodrie Drainage Structure" (WES) dated 25 October 1948. Filter sand F has limiting

gradations which will allow locally available material to be used. The material used should be more or less uniformly graded. The final recommended gradations of the filter materials are shown on plate 26. The maximum length of flow in the blanket to a collector is 15 ft. Each collector will have an outlet in each span of the structure as indicated on plate 25. Each outlet should be protected by a cover plate with 1/2-in. holes bolted to the concrete to prevent entry of debris and small animals. The upstream edge of the blanket is 18 ft from the upstream edge of the concrete cutoff wall.

38. Uplift with drainage blanket alone. To show the effects that a cracked or otherwise ineffective cutoff key would have on the uplift distribution along the base of the structure, an investigation was made assuming no cutoff key. The top clay stratum was assumed to have a vertical permeability of 25×10^{-9} cm per sec and to be 20 ft thick extending to an infinite length and the pervious silty stratum to be 10 ft thick with a horizontal permeability of 100×10^{-7} cm per sec. Using Bennett's equation* for slightly pervious blankets to determine the pressure distribution in the pervious stratum, a flow net was constructed for the upper clay stratum, assuming a permeability ratio of $k_h/k_v = 16$ in the clay. The uplift pressure caused on the base of the structure by a water pressure acting at some depth beneath the structure is the difference between the net water pressure and the submerged weight of soil above that depth. To obtain the maximum uplift at a given point on the base of

* P. T. Bennett, "The Effect of Blankets on Seepage Through Pervious Foundations." Trans. ASCE, Vol. III, 1946, pp 215-258.

the structure, computations at several depths are necessary to determine the maximum difference between water pressure and submerged weight of soil beneath that point. Such computations were made at several points along the structure to obtain the distribution of the maximum uplift pressures. This pressure distribution expressed as a per cent of net head is shown with the flow net on plate 27. The water pressure at the base of the structure as obtained directly from the flow net has also been plotted on plate 27. These pressure distributions indicate that the uplift assumed for the pile loadings of 100 per cent at the heel, 5 per cent 20 ft downstream from the heel, and 5 per cent at the toe is reasonable assuming the sand drain to be operating.

39. Uplift with drainage blanket and cutoff wall. Assuming the top clay stratum to have a vertical permeability of 25×10^{-9} cm per sec and to be 20 ft thick, and the pervious silty stratum to be 10 ft thick with a horizontal permeability of 100×10^{-7} cm per sec, the pressure distribution on the base of the clay stratum was computed by equations developed by Bennett* for slightly pervious blankets. Using this pressure distribution in the pervious stratum and the assumed permeability ratio of $k_h/k_v = 16$ in the clay, a flow net was constructed for the upper clay stratum. The maximum net uplift distribution on the base of the structure for these conditions is shown on plate 27. Also shown on plate 27 is the hydrostatic uplift on the base as taken directly from the flow net. Considering that the sand drain and cutoff wall operate and that these distributions of pressure are accurate, the assumed uplift for pile loading

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* Ibid

is conservative.

40. Uplift with relief wells. To relieve to the maximum possible extent any excess pressures in the pervious silty stratum, a line of relief wells has been provided 50 ft downstream from the heel of the structure. The wells, as shown on plate 25, will consist of an 8-ft length of 1-1/2-in. all-brass well screen with No. 35 slots connected to 1-1/2-in. brass riser pipe placed in a 10-in. diameter hole which extends down through the pervious stratum. The hole will be filled with filter sand A (see plate 25) to 1 ft above the top of the screen. Standard concrete sand may be used for sand A. The remainder of the hole will be filled with gravel B. Once the riser is within the concrete, galvanized pipe may be substituted for the brass pipe if a plastic coupling is used to avoid bimetal contact. The riser pipe will be brought to the top of the baffle pier and capped for clean-out and inspection. A tee connection will be placed at elev 32.25 for the horizontal outlet pipe. The outlets should be enlarged from the 1-1/2 in. pipe to 2-1/2 in. pipe in order to reduce the head loss at the outlet. Then 2-1/2 in. caps drilled with seven 1/2-in. holes, to prevent entry of debris and small animals, should be placed over the ends of the outlets.

41. The design of the well system was based on the assumption of a slightly pervious upstream top stratum (vertical permeability of 25×10^{-9} cm per sec) 20 ft thick extending to an infinite length, an impervious downstream top stratum and a 10-ft pervious stratum with a horizontal permeability of 100×10^{-7} cm per sec which gave an effective length of upstream blanket of 282 ft. With a well spacing of 31 ft 3 in., the computed well discharge is 0.04 gpm and the head midway between wells is

1.3 ft. With this low flow, very little head loss will occur in the well. Thus, the maximum pressure midway between wells would be 1.5 ft, allowing for the elevation of the well outlets above the minimum tailwater. For the purpose of drawing a flow net in the top clay stratum, the pressure distribution in the pervious stratum near and under the structure was assumed to vary as if the upstream top stratum were 282 ft in length and a head of 1.5 ft existed 50 ft downstream from the heel of the structure. At distances more than about 70 ft upstream from the structure, the above distribution does not hold, but is conservative. From the flow net drawn with the assumed permeability ratio of $k_h/k_v = 16$ in the clay, the hydrostatic uplift immediately beneath the base slab was obtained and is shown on plate 27 with the flow net. In this case, the hydrostatic uplift at the base of the structure is the maximum net uplift on the base of the structure. This distribution is considerably less than that assumed for the piling design, as the maximum uplift just inside the key is 7 per cent of the net head. It is considered that the drainage system consisting of a blanket and wells as described above provides a very substantial increase in the safety of the structure for a small additional cost and it is therefore recommended.

42. Riprap filter. The purpose of the gravel blanket beneath the riprap is to retain the foundation material, and the gravel must be of such grading that it will not pass through the voids of the riprap. Because of the clay foundation and the well system beneath the structure, there will be little or no flow from the foundation through the riprap; thus the gravel blanket beneath the riprap need not satisfy the requirements for filters. Based on these considerations, it is recommended that

a 10-in. layer of gravel C be used beneath the riprap. The gradation of gravel C is shown on plate 26. A mixture of 40% of 3/4-in. to 1-1/2-in. and 60% of No. 4 to 3/4-in. concrete aggregate will satisfy the requirements of gravel C. As it may not be possible to obtain locally gravel sizes larger than 3/4 in., an alternate riprap filter consisting of a 4-in. layer of rock spalls immediately beneath the riprap underlain by a 6-in. layer of gravel D is recommended. The gradation of gravel D as shown on plate 26 is satisfied by No. 4 to 3/4-in. concrete aggregate.

Design of Embankments

Strength assumptions

43. Foundation. As explained in paragraph 14, a foundation shear strength of $\phi = 0^\circ$ and c = 0.30 ton per sq ft was selected for the design of the north embankment extension (Sta 720 to 800), the enlargement of the existing embankment north of Sta 800, and the 850-ft embankment section immediately south of the control structure (Sta 672 to 680). For all sections south of Sta 672, a foundation shear strength of $\phi = 0^\circ$ and c = 0.25 ton per sq ft was employed in the design of the embankments. The embankment section from Sta 573 to 581 crosses an old borrow area. By assuming a factor of safety of unity for the existing embankment at this crossing, a stability analysis for a possible slide through the bottom of the pit showed that a shearing strength of 300 lb per sq ft would have to exist along the bottom of the pit. Therefore, along the pit bottom, this shearing strength was used in the design of the embankment enlargement across the old borrow area.

44. Embankment. South of Sta 672 an embankment strength of 450 lb

per sq ft was chosen and north of Sta 672 an embankment strength of 500 lb per sq ft was used. The bases for these selections have been discussed previously in paragraph 29.

45. Although the soils available in this immediate area are not particularly suitable for a high embankment, the flat slopes and berms necessary because of the poor foundation lessen the importance of the requirement for good material, considering the over-all stability of the embankment. Because of the fact that the berms serve merely as a counterbalancing weight, the type of material used in their construction is relatively unimportant, so long as a reasonably good density is obtained.

<u>Stability analyses</u>

46. The method of analysis employed in the design of the embankments was the circular arc method, and the factors of safety were computed from the formula:

$$\text{F.S.} = \frac{\text{total resisting moment (shear strength)}}{\text{net driving moment (weight)}} ,$$

The maximum depth of sliding was taken to be at elev -20, the bottom of the high-water-content clay stratum below which there is an increase in the strength of the clay. On the basis of compaction test results, the unit wet weight of the embankment material was taken to be 110 lb per cu ft.

47. Since the embankment material is subject to considerable shrinkage, and since cracks may develop that will reduce the effective length of the failure arc, a minimum design factor of safety allowed for the analysis of a shallow slide confined principally to within the embankment was 1.5. Such cracking does not significantly affect the analysis

of a foundation failure. It is recommended that, in order to reduce surface cracking to a minimum, a 3- to 5-ft layer of sandy or silty material be used to top the embankment.

48. For deep slides, which for this type of embankment are the most critical considering the thick underlying strata of clay, a minimum factor of safety of 1.2 was selected. Although this factor of safety is somewhat below that normally required for embankments, the stage construction to be employed makes it possible for the foundation material to consolidate a considerable amount during the construction period (estimated approximately 30 per cent consolidation during the first year) which should result in a gain in strength of the foundation. Also, no credit was taken for any frictional resistance. It should also be noted that to require a larger factor of safety would result in excessively large sections and berms and would increase materially the yardage required for the embankment. Existing embankments in the near vicinity of similar cross-sections to those proposed, notably the guide levees and existing embankment sections, are standing and show no signs of foundation failure.

49. Ballast and train loads were not explicitly taken into account in the analysis of the embankments because their effects are of minor importance, amounting to only 2 to 5 per cent of the total driving forces. These loads also would not be added until after the second construction period, at which time on the basis of settlements observed under the existing embankments, the foundation will have consolidated approximately 50 per cent or more; the increase in strength of the embankment and foundation will probably more than compensate for the additional loads. As the foundation is principally of a cohesive nature, the effects of vibratory

loads that may be induced are not considered to be of importance.

50. A summary of the proposed net embankment cross-sections appears on a profile of the crossing on plate 28, and the proposed cross-sections with their respective factors of safety appear on plates 29, 30, and 31. These cross-sections satisfy the requirements for a main-line levee as set forth in the Levee Code. However, it has been decided by the New Orleans District to build the embankment with 1 on 20 berm slopes from Sta 672 to the structure and from the structure to Sta 727 instead of the recommended 1 on 10 berms shown on plate 28 and 30.

Settlement

51. From observations of settlement plates placed during construction at the base of the existing embankments, an empirical basis is available for the prediction of the magnitude and rate of settlement to be expected under the proposed embankment extensions. From extrapolation of time-settlement curves of the existing embankment (see plates 32 and 33), approximately 90 per cent of the total settlement should occur within the first five years after construction begins.

52. Consolidation tests were run on selected samples from the foundation borings in order that an estimate of the amount and rate of settlement to be expected for the north and south embankment extensions could be made. The results of these consolidation tests appear in table 1 and on plates 17 and 18 of this report and in table 1 of the 1946 report.

53. Total foundation settlements for the proposed embankment extensions will probably range from 3-1/2 to 5 ft. This prediction is

based on the results of settlement analyses computed at the sites of various stations along the center line, tabulated below:

Station	Expected total settlement, ft
634+50	4.7
672+00	3.9
728+50	4.0
758+50	4.4
778+50	3.7
798+50	4.6

It is pointed out that these determinations are in line with observed settlements of the existing embankments which are recorded in the 1946 report. From test results, overconsolidation of the foundation material was indicated to exist to a depth of approximately 30 to 35 ft, assuming the ground-water table to be 6 ft below the ground surface. Although the thickness of the clay strata beneath the north embankment is greater than below the south embankment, the increased height and the larger sections required for stability of the south embankment will result in settlements of the same magnitude on both sides of the floodway. The original grade for the existing embankments is 1.5 ft below the proposed grade and in some cases these embankments have never been brought to their original grade. Therefore, further settlement may be expected at these sectors of embankment that are below the proposed grade.

54. Very thin layers of silty material and some thicker layers interspersed within the predominantly clay foundation afford drainage planes for consolidation of the clay and materially reduce the effective thickness of the entire clay stratum as regards the rate of consolidation of the foundation. Thus, estimation of the rate of settlement is considerably more difficult than predetermining the total settlement. From

the time-settlement curves of the existing embankments and time-consolidation data from laboratory tests, an effective thickness of about 11 to 12 ft was determined. Therefore, on the basis of the following assumptions, the time-settlement curves that appear in plates 32 and 33 were determined.

	North Embankment	South Embankment
Total settlement	4.6 ft (At Sta 798+50)	4.7 ft (At Sta 634+50)
Laboratory time for 50% consolidation of a 1.2-in. sample drained top and bottom	29 minutes	47 minutes
Effective thickness of clay stratum	12 ft	11 ft

Terzaghi's theoretical time rate of consolidation for a uniformly loaded stratum

Two construction seasons: 60% of embankment finished during first season, 40% of embankment finished during second season.

Method of construction

55. The berms of the proposed cross-sections for the existing embankment and the extensions will generally cover the existing railroad crossing, and in some cases the present highway crossing. These existing crossings must remain open to traffic until the embankment crossings are completed.

56. It has been proposed by the New Orleans District that the railroad side of the embankment be brought up to grade before the highway side. This procedure would eliminate the necessity of constructing the east berm its full length which would cover the existing railroad. After the railroad side has been brought to grade and ballasted, the existing track would be removed and transferred; the east berm would be finished,

and the highway side of the embankment would then be brought to grade. If this procedure is agreeable to the Louisiana State Highway Department, it is recommended that this method of construction be used. The desired net cross-sections from the first season of construction appear on plates 32 and 33.

57. Since the clay foundation is relatively weak, the embankment should not be constructed so as to overload the foundation at any point by excessive lifts; therefore, all lifts should be brought up evenly across the entire width of the embankment. Because the embankment extensions are relatively long, it may be advisable to construct the embankment in several portions, in which case the slope at the end of the embankment should be no steeper than the adjacent side slopes. Before bringing the center line of the existing embankments to grade, the berm enlargements should be placed first. It is suggested that the fill material be placed so that effective drainage of rainfall is secured, as the water content of the borrow material is already considerably above optimum.

58. As has been discussed in paragraph 53, the total foundation settlement for the proposed embankment extension will probably range from 3-1/2 to 5 ft. An examination of the settlement data on the existing embankment indicates, in accordance with theoretical considerations, that the foundation settlement is a maximum near the center of the embankment and is less at the edges of the crown of the embankment. In order to obtain the net section desired from the construction of the first season, this interim section should be overbuilt by 3 ft at the top of the embankment, tapered to 1.5 ft at 77 ft from the railroad center line, and 0.5 ft at the crown of the berm. Since at the end of construction of the

second construction season the foundation will probably have settled approximately 1-1/2 to 2 ft, the final net section should be overbuilt about 2 ft across the top of the embankment to compensate for future settlement of the foundation. The actual rate and amount of settlement as observed by use of the settlement plates as given in paragraph 77 should be used to make the final determination of the amount of overbuilding. The overbuilding of the final section should not exceed 2-1/2 ft so as to not overload the foundation.

59. Because the high water content and highly cohesive nature of the borrow material makes compaction by standard sheepsfoot rollers difficult, semicompaction by tractors is considered satisfactory. However, the material should be allowed to dry back as close to optimum as it is practicable. It is believed that adequate densities can be obtained by tractor compaction, provided the number of passes is adjusted to insure that the lumps of the soil are firmly pressed together to eliminate any appreciable voids between the lumps. It is difficult to predict just what compactive effort will accomplish this purpose. Since this is an important structure, it is recommended that at least three coverages of the track tread be made with a provision for payment for additional coverages over three, in case it becomes apparent that three coverages are not accomplishing proper consolidation. Each lift should not have a thickness greater than 12 in. of loose material. Careful observations of the compaction operation should be made at the beginning of construction to determine the number of coverages required.

60. Tractors of 10-ton minimum weight and a unit contact pressure of at least 6 psi should be used for compaction. This weight and contact

pressure should be determined for the tractor without bulldozer blade or other attachments. It is the intent of this requirement to restrict the contractor to use of the D-7 or D-8 Caterpillar types of tractor in order to obtain adequate densities of the soil.

61. The use of semicompacted fill will add to the total settlement and may affect the cost of maintenance required to maintain grade.

Design of Abutments

General

62. As the foundation is weak and compressible, the embankment must be connected to the structure in such a manner that an adequate factor of safety of the embankment against failure is maintained and that detrimental differential settlements are avoided.

Preloading

63. Preliminary studies indicated that the large settlement expected beneath the embankments would create a number of problems if the fill at the abutments were placed after completion of the structure. The three principal difficulties are as follows:

a. Control of underseepage at abutments.

b. Deflection of batter piles beneath abutment piers due to subsidence of the soil under the embankment load.

c. Maintenance difficulties on the finished embankment crown.

It is therefore recommended that the abutment areas be preloaded so that a major portion of the troublesome settlement beneath the embankment will take place before the structure is constructed.

64. This preloading may be accomplished by constructing the

embankments to the cross-sections shown on plate 34 with the crown extending to the end of the structure. The end slopes of the embankments will extend into the structure area. The fill should be placed as soon as possible so that the foundation will be loaded for a maximum period of time before the abutments are constructed. If the preloading period is two years, more than 50 per cent of the maximum settlement will have taken place. As some of the preload fill will be higher than the final grade of the embankment, the overloaded areas may reach settlements equal to those expected under the final embankment grade. At the end of the preloading period, the fill in the structure area and the material above the final grades of the embankment will be removed.

65. With this preloading the tendency for the fill to pull away from the curtain wall will be reduced. In addition, the possibility of other cracks and voids opening up is practically eliminated, thus materially reducing the danger of piping. If most of the settlement of the foundation has taken place before the piles for the abutment piers are driven, there will be little deflection of the batter piles due to settlement of the soil under the embankment load. Also the problem of maintaining the transition from the nonsettling structure to the settling embankment will be reduced considerably.

Description of abutments

66. The crown of the embankment stops 130 ft from the control structure as shown on plate 35. Four approach spans carry the highway and railroad from the main embankment to the control structure. The piers for the approach span will be supported on piles driven to sand.

The end of the embankment slopes down from the abutment pier to the end pier of the control structure as shown on plate 35 with the same slope as recommended for the side slopes of the embankment. Since the New Orleans District decided to use a berm slope of 1 on 20 instead of 1 on 10, the actual slopes to be built around the end of the embankment will be warped from a 1 on 20 berm on the side slopes to a 1 on 10 berm on the end slope.

Control of underseepage

67. The water barrier for the section between the main embankment and the end pier of the control structure is a vertical concrete curtain wall on the upstream side of the piers as shown on plate 35. The wall penetrates into the embankment and foundation to the depth indicated on plate 35. To provide an adequate factor of safety against seepage around and under the wall, it is recommended that sheet piling be driven under the last two approach spans and the first span under the structure as shown on plate 35. It is necessary to drive the sheet piling on a batter as shown so that it will not interfere with the bearing piles under the piers.

68. As an additional means of control of underseepage, it is recommended that the backfill on the riverside of the curtain wall be selected lean impervious material as shown on plate 35. There is sandy silt with some clay silt in Borrow Area A which will probably be suitable for this purpose. Some clay may be added to give a more impervious mixture. To intercept and collect any underseepage, the backfill on the landside of the wall should consist of a zone of sand F overlain by 12-in. of gravel C as shown on plate 35. All backfill against the wall should

be compacted with hand-operated power tampers so as to obtain a high degree of compaction.

Wing walls

69. The wing walls act as retaining walls for about 8 ft of fill and as hydraulic training walls for the control structure. As it is desired to eliminate to the fullest extent any differential settlements along the wall or between the training wall and the structure, the training walls should be supported on bearing piles driven to sand. Other types of wall which were given consideration were a T-type wall with a spread footing and a heavy I-type sheet pile wall. The I-type wall would have had to be faced with concrete after all settlement had taken place.

Engineering Measurement Devices

Control structure

70. Settlement. Settlement observations should be made by using marked metal hubs embedded in the concrete superstructure. A hub should be embedded in the beam supporting the upstream crane rail and in the sidewalk next to the railroad at approximately Sta 684, 692, 700, 708, and 716. These observations should be referenced to some well-established bench mark. To measure the settlement of the foundation as a result of the load caused by high water against the structure, it is recommended that settlement plates be installed beneath the gravel under the riprap 50 ft upstream of the structure at about Sta 684, 700, and 716. These plates should be 3 ft square of 1/4-in. plate with a 2-1/2-ft piece of 1-in. pipe welded in the center of the plate. Readings should be made on

top of the riser before and after the riprap is placed and after each high water against the structure or at least once a year.

71. Piezometers. Observations of the performance of the filter blanket and relief wells should be made by the use of piezometers. Five "A" piezometers to measure the hydrostatic pressure midway between wells in the silty stratum should be placed along the line of wells on about 800-ft centers at approximately Sta 684, 692, 700, 708, and 716. With the same spacing along the structure, five "B" piezometers should be placed in the sand blanket midway between collector pipes No. 1 and 2 at the same stationing as above. A typical location and detail sketch of these piezometers appears on plate 36.

72. All piezometer screens should be 4- to 12-in. lengths of 1-in. brass well strainer, such as Claytor-Mark or equal, with No. 35 slots. The "A" piezometers should be placed in a 10-in.-diameter hole that extends to the bottom of the silty stratum (approximately elev -1.0) with the top of the screen at a depth of approximately 20 ft, or at approximately elev 4.0. The hole should be backfilled with filter sand A to the top of the silty stratum (approximately elev +9.0), there a 1-ft sand-bentonite seal should be placed around the vertical pipe. The remainder of the backfill should be tamped clay. The "B" piezometers should be placed horizontally in the 10-in. filter sand F blanket, the center of the piezometer screens being 15-ft downstream from Collector No. 1. The pipes from the "A" piezometers to the riser pipes should be placed on a slope of about 1 on 10 so as to allow any air in the piezometers to escape. Once the pipes from the "A" and "B" piezometers are in the base slab, galvanized pipe may be substituted for the brass pipe if desired, using a

plastic coupling to avoid bimetal contact. All riser pipes should be brought up in a pier at the downstream curb of the highway, and should have a 1-in. vented plug with a countersunk head flush with the top of the curb. The top of each piezometer should be permanently stencilled with the piezometer number.

Embankments

73. Piezometers. It is recommended that piezometers be installed in the foundation and fill of the embankment extensions so that the magnitude of the pore water pressures caused by construction of the fill may be determined. Five piezometer tips should be placed at Sta 630, 664, 758, and 798 at the locations shown on plate 37. These locations provide for one tip in the embankment 5 ft above the ground surface 18 ft riverside of the railroad center line, and 4 tips in the foundation at elev 20 and -10, 18 ft riverside and 60 ft landside of the railroad center line.

74. The tips to be placed in the foundation should be as shown on plate 37. The tip, described in detail in Appendix B and shown on plate 37, is a 4- to 6-in. length of well strainer with a 3/4-in. riser. Two water filled copper tubes connected into a brass tee at the top of the riser transmit the pressure from the tips to a terminal well at the landside toe of the embankment where the respective values are observed on pressure gages. The recommended type of tip to be placed in the embankment as shown on plate 37 is a modification of the Bureau of Reclamation Type A tip. This tip, also a nonflow type, has a brass shell and porous disk designed so that final assembly may be made in the field. Copper tubes connect the tips to the terminal well. The technique of installation

and reading of the piezometers is outlined in Appendix B.

75. To utilize fully the data obtained from the above piezometers, it is necessary to know the location of the water table. For this purpose a piezometer should be installed 30 ft from the landside toe of the embankment at Sta 630, 664, 758, and 798. A detail of this piezometer is shown on plate 37. The tips are 4- to 6-in. lengths of 3/4-in. Clayton-Mark, or equal, all brass well screen with #35 slots placed 12 ft below the ground surface in a 6- to 10-in.-diameter hole. The hole should extent to a depth of 14 ft and be backfilled with filter sand A to a foot above the tip where a 1-ft sand-bentonite seal should be placed around the riser pipe. The remainder of the backfill may be tamped clay. At a short distance above the tip the brass riser pipe should be reduced in size from 3/4 in. to 3/8 in. The riser should be capped about 3 ft above the ground surface. Each piezometer should be permanently marked and protected with guard posts.

76. Settlement plates. To furnish reference points permanently and readily accessible to record the settlement of the embankment, special settlement plates as shown on plate 37 should be placed at Sta 630, 664, 758, and 798 in addition to the plates to be used to determine payment quantities. At each of the above stations a plate should be located 18 and 95 ft riverside of the railroad center line and 60 ft landside of the railroad center line and be firmly embedded at the ground surface. A settlement plate should also be placed under each preloading fill at the abutments. These two plates should be 18 ft riverside of the railroad center line at Sta 679+40 and 721+20.

77. The plates should be 1/4-in. steel plate, 3 ft square with a

4-in. length of 1-1/4-in. pipe welded in the center of the plate. The 1-in. riser pipe should be welded inside the short length of 1-1/4-in. pipe. The riser should be protected by a sleeve of 2-in. pipe as shown on plate 37 to allow for settlement within the embankment. The openings should be filled with waste and heavy grease. The riser and the sleeve should be brought up in 5- or 10-ft sections. If the tops of the risers and sleeves are kept above the top of the embankment during construction, guard posts should be set and particular care taken to prevent damage to the risers by construction equipment. At all times during construction the sleeves should be capped to prevent dirt from clogging the space between the riser and sleeve. When the embankment is brought to grade, the sleeve should be capped 12 in. above grade and the riser 6 in. below the cap of the sleeve. Each sleeve should be permanently marked and protected with guard posts. Readings should be taken immediately before and after each section of riser pipe is added and at monthly intervals until the embankment is completed. For the first year after completion of the embankment, readings may be made every three months, after which time readings every six months will be sufficient. A permanent bench mark should be established at each station where settlement plates are to be installed. These bench marks should be properly referenced and should be placed at least 150 ft from the landside toe of the embankment.

Recommendations and Conclusions

Control structure

78. To prevent excessive differential settlements that would take place under a slab foundation, a pile foundation is recommended for this

structure. Since friction piles will not carry the expected load without undue settlements, bearing piles driven to sand should be used.

79. A series of pile tests to determine the size required for the bearing piles and the allowable tension load is believed necessary for a more economical design of the piling and should be initiated before the design is completed.

80. In order to insure that the uplift pressures used in design of the structure will not be exceeded, the drainage system which incorporates relief wells as shown on plate 25 is recommended.

81. To observe the settlement of the control structure, permanent bench marks should be established in the superstructure. In order to verify the design assumptions of uplift beneath the structure, it is recommended that the performance of the filter blanket and relief wells be observed by the installation of five sets of piezometers in the drainage blanket and in the silty stratum at about 800-ft intervals along the control structure.

82. It is recommended that construction of the control structure start in the middle so that a maximum period of preloading will be allowed at the abutments.

Embankments

83. Because there is to be a raising of grade of the highway and railroad of 1-1/2 ft, and since the existing embankment sections in some places have not been brought to grade, it is recommended that the existing embankments be reinforced using the sections proposed on plates 29 and 31.

84. The foundation beneath the existing embankment will probably

consolidate further, due to raising the original grade line of the highway and railroad, and also since some portions of existing embankment have never been brought to their original grade. Therefore, additional settlement may be expected along these portions.

85. Recommended net sections for the embankment extensions are shown on plate 30. It is estimated that the total settlement to be expected beneath the center of the embankment extensions will range from 3-1/2 to 5 ft, about 90 per cent of this settlement probably occurring during the first five years after construction. As this estimate does not include settlement within the embankment itself, the embankments may have as much as a 4 to 5-1/2 ft total settlement over a five- to ten-year period, assuming that there may be shrinkage and settlement within the embankment of 0.5 ft.

86. As it is necessary to keep the existing highway and railroad open to traffic during construction of the embankment, it is recommended that the railroad side of the embankment be brought up to grade before the highway side, by a stage construction procedure. The berm enlargement should be placed before the existing embankments are raised to their new grade. Overbuilding of the interim net section at the new embankment extensions during the first year of construction should be 3 ft at the top of the embankment, tapered to 1.5 ft at 77 ft from the railroad center line and 0.5 ft at the crown of the berm. The final net section should be overbuilt about 2 ft across the top of the embankment. The exact amount of overbuilding should be determined from observations on the settlement plates beneath the embankment.

87. Although the borrow material to be used in the embankments is

not particularly suitable for embankment construction as it is very plastic and has a high natural water content, it would not be economical to haul a better material in the quantity needed from distant sources. The flat slopes required for the embankment because of the weak foundation material lessen the requirement for high quality of the embankment material so long as an adequate density is obtained. The embankment should be constructed so as not to overload the foundation by excessive overbuilding. The thickness of each lift should not exceed 12 in. of loose material, and each lift should be brought up uniformly across the entire width of the embankment. It is considered satisfactory to use 10-ton minimum weight tractors that have a contact pressure of 6 psi for compacting the embankment material, providing the number of coverages of the track is adjusted to insure that the lumps of soil are firmly pressed together to eliminate any appreciable voids between lumps. At least three passes of the tractors should be made, with a provision for payment for additional passes if proper compaction is not obtained with three passes. Careful observations of the compaction operations should be made at the beginning of construction to determine the number of passes required. The edges of all borrow areas should be at least 100 ft from the toe of the embankments.

88. In order to minimize surface cracking within the embankment, it should be topped with 3 to 5 ft of silty or sandy material.

89. Before placing any ballast on the newly completed railroad embankment, it is suggested that the embankment be allowed to settle for as long a period as possible and then be brought to final grade. Approximately 90 per cent of the total settlement is expected within five years after beginning of construction of the embankment. In order to keep the tracks

of the railroad to grade, a continuous maintenance program of ballasting appears to be the most feasible method. It is suggested that only temporary surfacing be placed on the highway after it is brought to grade until approximately 80 per cent of the total settlement (as determined from the settlement plates) has occurred. At this time the highway side of the embankment could be brought to grade to compensate for expected future settlement and then resurfaced.

90. It is recommended that settlement plates be placed beneath the base of the embankment at Sta 630, 664, 758, and 798.

91. In order to measure the pore water pressures that will be developed in the embankment and foundation due to the construction of the embankment, it is recommended that five piezometers be installed in the foundation and fill at Sta 630, 664, 758, and 798. A record of the water-table fluctuations is necessary to evaluate fully the data from the piezometers and therefore it is recommended that water-table piezometers be installed 30 ft from the toe of the landside slope of the embankment at Sta 630, 664, 758, and 798.

Abutments

92. It is recommended that the abutment areas be preloaded and the foundation allowed to settle for as long a period as possible before the structure is constructed. An embankment settlement plate should be placed under each preloading fill. These two plates should be 18 ft riverside of the railroad center line and should be at Sta 679+40 and 721+20.

93. Four approach spans should be used at each abutment to carry the highway and railroad from the main embankment to the control structure.

The embankment should slope down to the end pier of the control structure with the same slopes as recommended for the side slopes of the embankment.

94. The backfill against the concrete curtain wall should be placed as shown on plate 35 with hand-operated power tampers.

Table 1

SUMMARY OF TEST DATA -- FOUNDATION

Combined Morganza Floodway Control Structure

Boring		Sample				Mechanical Analysis			Atterberg Limits			Nat.	Shear Tests					Consolidation Tests						K_h
Number	Station	No.	Elev MSL	Depth Ft	Soil Classification	% Sand	% Silt	% Clay	LL	PL	PI	w %	Type	w %	γ_d Lb/Cu Ft	ϕ°	C T/Sq Ft	Cc	Pc T/Sq Ft	Po T/Sq Ft	Sp Gr	w %	T_{50} min.	10^{-7} Cm/Sec
M-19	721+00	9	19.7	12.8	Clay, grey, firm	10	25	65	97	29	68	51	UC	47	73		0.34	0.41	1.3	0.5	2.70	51	39	
Ground Elevation 32.5 msl		11	16.5	16.0	Clay, grey, firm							44	UC	46	75		0.38							
		14	14.3	18.2	Clay, grey, firm							50	UC	44	75		0.44							
		19	8.5	24.0	Clay, grey, firm	17	28	55	82	27	55	40	UC	43	77		0.38	0.36	1.6	0.8	2.71	46	20	
		22	5.6	26.9	Clay, grey & brown, firm							48	UC	43	77		0.27							
		27	1.1	31.4	Sandy silt, brown	30	59	11				29	Q_cT	28	96	21°	0.30							
		35	-13.6	46.1	Clay, grey, soft	8	17	75	112	34	78	64	UC	70	57		0.16	0.73	1.2	1.3	2.65	69	38	
		40	-22.4	54.9	Clay, grey, firm	12	12	76	97	32	65	45	UC	48	72		0.36	0.40	2.5	1.5	2.72	41	16	
		41	-28.0	60.5	Clay, grey, firm							45	UC	51	67		0.47*							
		42	-33.4	65.9	Silty clay, grey, firm	16	37	47	65	23	42	26	UC	34	86		0.44	0.42	1.3	1.8	2.67	40	8	
		45	-43.7	76.2	Silty clay, grey, firm	8	42	50	54	24	30	49	UC	33	86		0.38	0.35	2.9	2.0	2.70	36	8	
M-20	728+50	5	27.1	5.5	Clay, grey, firm							40	UC	45	73		0.30							
Ground Elevation 32.6 msl													UC	44	76		0.38*							
		9	22.6	10.0	Silty clay, grey, firm	11	44	45				41	UC	32	89		0.48							
		11	18.9	13.7	Clay, grey, firm							47	UC	43	75		0.44							
		16	13.5	19.1	Clay, grey & brown, stiff	5	20	75				47	UC	44	76		0.55							
		21	7.6	25.0	Clay silt, grey & brown, firm							32	UC	29	91		0.37							
		36	-7.6	40.2	Clay, grey, firm							62	UC	62	63		0.29							
		39	-10.4	43.0	Clay, grey, firm	6	6	88	117	29	88	67	UC	66	59		0.32							
		43	-15.4	48.0	Clay, grey, firm							62	UC	66	59		0.37							
		52	-47.9	80.5	Silty clay, grey, stiff							32	UC	37	82		0.59							
M-21	738+50	5	27.5	5.2	Clay, grey & brown, firm	7	23	70	81	27	54	47	UC	50	70		0.29							
Ground Elevation 32.7 msl													UC	45	73		0.33*							
		12	18.2	14.5	Clay, grey, firm							47	UC	49	72		0.26							
		17	7.4	25.3	Clay, grey, firm	8	26	66	76	23	53	43	UC	41	79		0.42							
		25	-3.2	35.9	Clay, grey, stiff							42	UC	38	82		0.53							
		31	-10.2	42.9	Clay, grey, soft	8	5	87	102	29	73	59	UC	71	57		0.24							
		38	-21.7	54.4	Clay, grey, soft	10	30	60				46	UC	37	83		0.24							
		40	-31.8	64.5	Clay, grey							44	UC	52	69		0.26							
M-22	758+50	4	29.5	4.7	Silty clay, grey, firm							34	UC	32	85		0.29							
Ground Elevation 34.2 msl													UC	33	86		0.33*							
		5	28.3	5.9	Clay silt, grey, soft	14	57	29				32	UC	32	86		0.14							
		6	27.1	7.1	Clay silt, grey, very soft							36	UC	31	89		0.12**							
		7	25.5	8.7	Clay silt, grey, firm							34	UC	31	89		0.32							
		10	21.6	12.6	Clay, grey							47	UC	45	75		0.29							
		11	20.3	13.9	Clay, grey, soft							45	UC	44	76		0.19**							
		13	18.0	15.6	Clay, grey, firm	7	28	65	79	26	53	41	UC	42	78		0.29	0.36	1.3	0.5	2.68	41	12	
		16	14.3	19.9	Clay, grey, firm							48	UC	45	75		0.41							
		18	11.8	22.4	Clay, grey, soft							48	UC	46	75		0.22**							
		19	10.5	23.7	Clay, grey, firm	7	21	72				53	UC	51	71		0.29							
		22	7.2	27.0	Clay, grey, firm							40	UC	44	76		0.33							
		25	3.3	30.9	Clay, grey & brown, firm	7	23	70	84	30	54	42	UC	37	82		0.40	0.38	2.6	0.9	2.71	42	26	
		27	1.1	33.1	Clay, grey, firm							46	UC	40	81		0.34**							
		28	0.0	34.2	Clay, grey, stiff							34	UC	39	82		0.52							
		30	-2.6	36.8	Clay, grey, firm							45	UC	36	82		0.25**							
		31	-3.7	37.9	Clay, grey & brown, firm							49	UC	44	76		0.46							
		37	-10.8	45.0	Clay, grey, firm				92	30	62	59	UC	64	61		0.35	0.78	1.4	1.3	2.69	63	21	
		38	-12.1	46.3	Clay, grey, firm	11	14	75				56	QT	52	69	2°	0.30							
													UC	71	56		0.35							
		42	-19.3	53.5	Clay, grey, firm							42	UC	50	71		0.29							
		43	-25.2	59.4	Clay, grey, firm							48	UC	40	80		0.44							
		46	-32.9	67.1	Clay, grey, stiff	6	34	60	84	26	58	41	QT	37	83	1°	0.50							
													UC	40	80		0.55							
		50	-43.7	77.9	Clay, grey, stiff	6	27	67	76	29	47	44	UC	39	80		0.53							
M-23	778+50	8	24.2	9.9	Clay, grey, firm							43	UC	30	92		0.37*							
Ground Elevation 34.1 msl		9	23.6	10.5	Clay, grey, firm	8	40	52	66	31	35	45	UC	39	79		0.28							
		12	19.6	14.5	Clay, grey, firm							36	UC	36	84		0.36							
		15	15.4	18.7	Clay, grey, firm							46	UC	43	76		0.27							
		22	5.6	28.5	Clay silt, grey, firm							34	UC	31	89		0.42							
		26	-0.2	34.3	Silty clay, grey, firm	10	43	47				40	UC	34	86		0.45							
		34	-10.4	44.5	Clay, grey, firm							55	UC	53	67		0.34							
		42	-35.4	69.5	Silty clay, grey, stiff							38	UC	36	83		0.54							
M-24	786+00	3	23.6	10.5	Silty clay, grey, firm	8	59	33	49	23	26	47	UC	39	80		0.25							
Ground Elevation 34.1 msl													UC	40	78		0.24*							
		6	8.6	25.5	Clay, grey, firm	11	34	55	90	23	67	35	UC	41	80		0.38							
		8	-1.5	35.6	Clay, grey, firm	4	16	80				49	UC	36	85		0.43							
		10	-11.4	45.5	Clay, grey, firm				104	32	72	51	UC	55	67		0.35							
		12	-21.0	55.1	Clay, grey, firm	20	20	60				35	UC	40	81		0.28							
		16	-41.3	75.4	Clay, grey, firm							49	UC	51	78		0.34							
M-25	798+50	4	30.9	3.8	Clay silt, grey, firm							36	UC	31	89		0.29							
Ground Elevation 34.7 msl													UC	28	93		0.25*							
		10	24.2	10.5	Clay, grey, firm	6	26	58	69	26	43	39	UC	38	83		0.36	0.26	1.0	0.4	2.70	34	24	
		13	19.6	15.1	Clay, grey, firm							34	UC	35	85		0.28							
		18	14.7	20.0	Clay, grey, firm	14	18	68	90	30	60	46	UC	46	75		0.32	0.33	0.7	0.6	2.69	44	72	
		24	7.5	27.2	Silty clay, grey, firm	11	41	48				37	UC	37	84		0.32							
		26	5.0	29.7	Clay silt, brown & grey	25	54	21				31	Q_cT	31	93	15°	0.35							
		38	-7.8	42.5	Clay, grey, firm							52	UC	44	76		0.25							
		43	-13.3	48.0	Clay, grey, firm	6	11	83	95	25	70	48	UC	44	76		0.29	0.36	1.4	1.3	2.70	44	24	
		47	-25.8	60.5	Clay, grey, firm	6	27	67	79	30	49	44	UC	38	82		0.41							
		49	-35.8	70.5	Clay, grey, firm	4	31	65				47	UC	42	76		0.25	0.38	1.7	1.9	2.70	40	20	
		52	-50.8	85.5	Clay, grey, firm							36	UC	34	85		0.44							
M-26	672+00	6	25.8	5.4	Clay, grey & brown, soft							55	UC	57	66		0.24							
Ground Elevation 31.2 msl		16	15.6	15.6	Clay, grey & brown, firm	8	18	74	90	29	61	48	UC	51	72		0.35							
		25	6.4	24.8	Clay silt, grey & brown, firm	13	64	23				31	UC	29	93		0.41							
		27	4.8	26.4	Sandy silt, grey	23	61	16				31												11.2
		31	0.5	30.7	Clay, grey & brown, firm	7	24	69				41	UC	41	78		0.39							
		41	-9.3	40.5	Clay, grey, firm	11	22	67	90	27	63	54	UC	49	70		0.44							
		45	-16.0	47.2	Clay, grey, firm	11	14	75				50	UC	47	73		0.49							
		49	-19.2	50.4	Silty clay, grey, firm							45	UC	37	83		0.44							
M-27	679+00	7	24.0	7.7	Clay, grey & brown, firm	8	19	73	100	28	72	55	UC	46	73		0.33	0.41	1.4	0.4	2.66	48	53	
Ground Elevation 31.7 msl		10	21.0	10.7	Clay, grey & brown, stiff							38	UC	34	86		0.54							
		11	20.2	11.5	Clay, grey, firm							48	UC	43	72		0.34*							
		13	18.1	13.6	Clay, grey, firm	6	15	79				54	QT	47	71	3°	0.25							
													UC	46	73		0.25							
		16	15.5	16.2	Clay, grey & brown, firm							58	UC	39	80		0.46							
		20	10.7	21.0	Clay, grey & brown, firm	5	17	78				50						0.51	1.2	0.7	2.72	56	25	
																		0.53*	0.9		2.72	52	90	
		21	10.1	21.6	Clay, grey & brown, firm	8	27	65	91	30	61	47	UC	45	75		0.37	0.38*	1.4	0.7	2.65	45	47	
		24	7.2	24.5	Clay, grey & brown, firm							35	QT	32	89	2°	0.30							
		25	5.8	25.0	Clay, grey & brown, stiff							35	UC	28	94		0.64							
		26	5.2	26.5	Clay silt, grey, soft	26	54	20				12												5.7
		31	-0.1	31.8	Silty clay, grey & brown, firm							43	UC	40	80		0.39							
		34	-3.2	34.9	Clay, grey & brown, stiff	8	30	62				42	UC	37	82		0.52							
		38	-7.1	38.8	Clay, grey, firm	8	24	68				51	UC	49	72		0.38							
		40	-8.6	40.3	Clay, grey, firm							54	QT	49	72	0°	0140							
		41	-10.1	41.8	Clay, grey, firm	10	33	57				68						0.63	1.5	1.2	2.70	53	12	
																		0.53*	0.6	1.2	2.70	55	58	
		42	-10.7	42.9	Clay, grey, firm	6	12	82	114	43	71	72	UC	73	56		0.27	0.77*	1.5	1.2	2.62	65	17	
		44	-12.8	44.5	Clay, grey, firm							73	UC	70	57		0.39							
		47	-15.8	47.5	Clay, grey, firm							45	UC	43	76		0.39							
		50	-23.2	54.9	Clay silt, grey, soft	12	59	29	38	23	15	40												
		52	-30.3	62.0	Sandy-clay	46	21	33	39	20	19	31												
M-29	689+00	3	22.7	9.0	Clay, grey, firm	12	38	50	74	25	49	37												
Ground Elevation 31.7 msl		12	-11.8	43.5	Clay, grey, soft	8	7	85	113	33	80	67												
M-31	699+00	8	21.5	10.5	Silty clay, grey & brown, soft				64	23	41	42	UC	32	85		0.23							
Ground Elevation 32.0 msl		10	19.5	12.5	Clay, grey, soft							51	UC	43	76		0.22							
													UC	48	72		0.33*							
		12	17.7	14.2	Clay, grey & brown, soft							44	UC	44	74		0.20							
													UC	41	77		0.28							
		14	15.7	16.3	Clay, grey & brown, firm							44	QT	43	76	1°	0.35							
		16	13.6	18.4	Clay, grey & brown, soft							52	UC	51	70		0.20							
		18	11.5	20.5	Silty clay, grey & brown, firm							43	UC	48	71		0.27							
		20	9.5	22.5	Silty clay, grey, soft	12	56	32	48	21	27	33												35.0
		22	7.5	24.5	Clay silt, grey & brown, firm							32	UC	32	89		0.42							
		23	6.5	25.5	Clay silt, grey & brown, soft	11	61	28				32												0.6
		26	1.6	30.4	Sandy silt, grey	13	70	17				34												58.0
		31	-4.0	36.0	Silty clay, grey, firm	3	52	45	70	23	47	41	UC	41	82		0.35							
		35	-9.0	41.0	Clay, grey, firm							64	UC	55	65		0.28*							
		36	-10.5	42.5	Clay, grey, firm							69	UC	20	81		0.25							
		38	-12.6	44.6	Clay, grey, soft	5	8	87	102	31	71	67	UC	70	58		0.24							
		39	-14.5	46.5	Clay, grey, soft							69	UC	69	57		0.21*							
		40	-15.5	47.5	Clay, grey, firm							57	UC	67	58		0.23*							
		41	-17.2	49.2	Clay, grey, firm							54	UC	55	67		0.42							
		43	-22.5	54.5	Clay, grey, firm	14	20	66	89	30	59	41	UC	49	72		0.36	0.36	1.5	1.5	2.72	43	8	
M-33	709+00	2	25.0	7.5	Clay, brown, firm	9	35	56	91	29	62	40												
Ground Elevation 32.5 msl																								
M-35	717+50	2	24.4	8.0	Clay, brown, firm				88	29	59	42												
Ground Elevation 32.4 msl																								
M-37	700+00	3	28.3	3.7	Clay, grey & brown, soft							49	UC	42	75		0.22							
		4	27.3	4.7	Clay, grey & brown, soft	5	21	74	95	25	70	45	UC	44	72		0.24							
Ground	115 ft	6	23.0	9.0	Clay, grey & brown, firm	8	25	67				45	UC	37	81		0.48							
Elevation	west of	7	20.8	11.2	Clay, grey & brown, firm	8	39	53	70	23	47	43	UC	34	85		0.46							
32.0 msl	center	9	16.0	16.0	Clay, grey & brown, firm							34	UC	47	72		0.29							
	line of	10	13.8	18.2	Clay, grey & brown, soft	10	16	74				31	UC	50	70		0.24							
	rail-road	13	10.3	21.7	Clay silt, grey, firm	16	55	29				49	UC	29	92		0.38							

Notes: UC = unconfined compression test; QT = quick triaxial test; Q_cT = consolidated quick triaxial test. w% from strength and consolidation tests is water content before testing. Each UC test value is the average of two 1-in.-diameter test specimens. T_{50} is average time for 50% consolidation under loadings of 0.4, 0.8, 1.6, and 3.2 tons per sq ft. Po based on water table at depth of 6 ft

* Remolded specimens.

** 2.8-in.-diameter test specimen. Strength value for one test only.

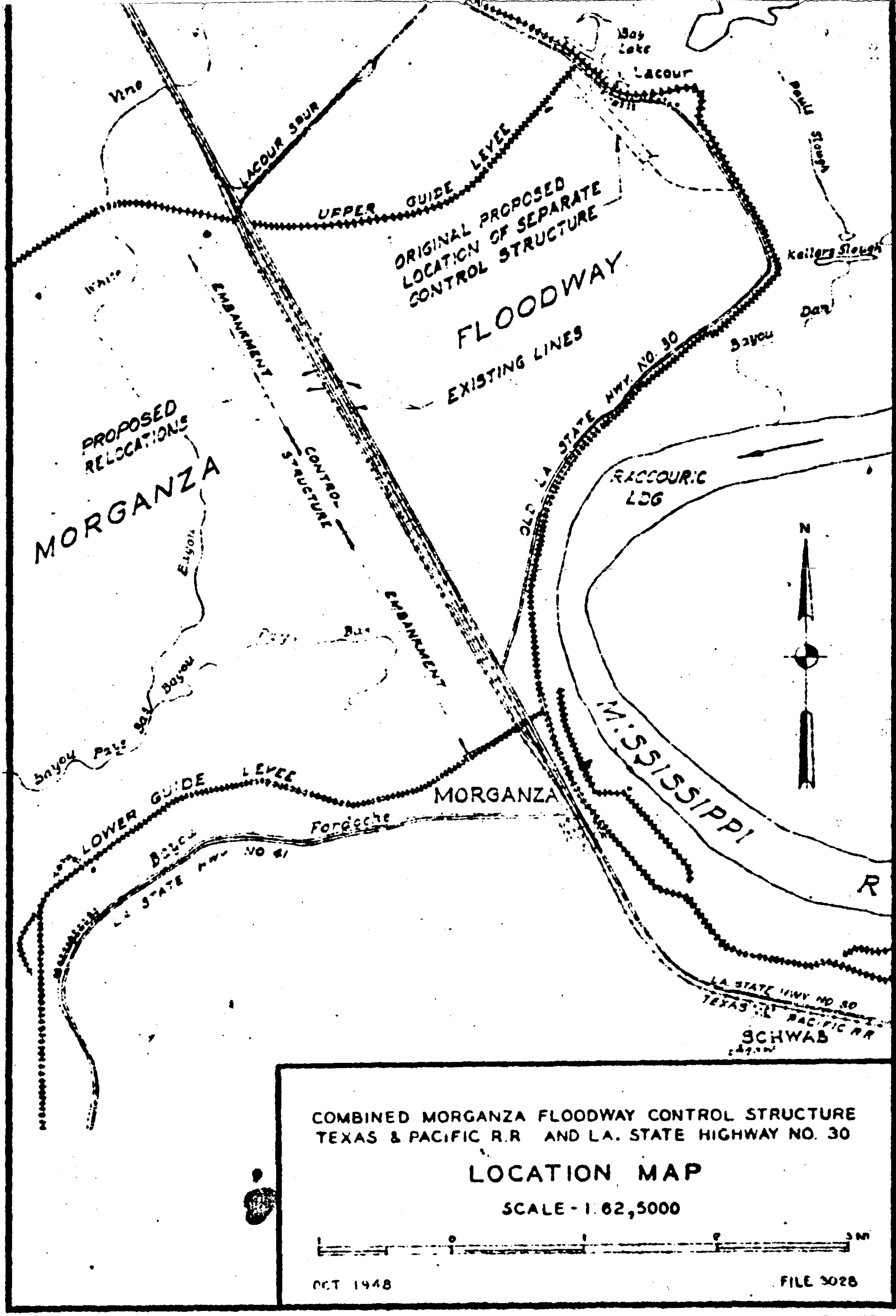
Table 2

SUMMARY OF TEST DATA -- BORROW AREAS

Combined Morganza Floodway Control Structure

			Mechanical Analysis			Atterberg Limits			15-Blow Proctor		Shear Tests							
Sack Sample Number	Boring Number	Soil Classification	% Sand	% Silt	% Clay	LL	PL	PI	Opt. w %	γ_d Lb/Cu Ft	Type	Nat. w %	Test w %	γ_d Lb/Cu Ft	ϕ^o	c T/Sq Ft	Sp Gr	Shrinkage Limit
1	MB-24	Clay	8	39	53	62	28	34	29	91	UC	38	28	91	-	1.03	2.69	13
											UC	"	29	91	-	0.63		
											UC	"	35	82	-	0.29		
											QT	"	36	82	2°	0.15		
2	MB-31	Silty clay	11	55	34	44	23	21				34						
3	MB-36	Clay	9	38	53	61	25	36	28	90	UC	35	28	91	-	1.35		12
											Q_cT	"	32	87	14°	0.20		
											QT	"	35	82	2°	0.27		
											UC	"	36	82	-	0.34		
4	MB-48	Silty clay	16	49	35	42	21	21	23	98	UC	34	23	99	-	0.62		14
											UC	"	23	98	-	1.14		
											UC	"	26	94	-	0.52		
											QT	"	27	96	2°	0.33		
											QT	"	31	89	0°	0.16		
											UC	"	32	86	-	0.21		
5	MB-55	Silty clay	8	48	44	54	23	31				35						
6	MB-66	Silty clay	14	43	43	57	23	34	24	97	UC	30	24	97	-	1.19	2.70	15
											UC	"	24	96	-	1.25		
											QT	"	29	91	1°	0.22		
											Q_cT	"	29	91	14°	0.13		
7	MB-69	Silty clay	12	49	39	47	23	24	21	100	UC	35	20	97	-	1.32		17
											UC	"	25	97	-	0.96		
											UC	"	30	90	-	0.28		
											QT	"	31	89	2°	0.16		
											UC	"	34	84	-	0.16		
8	MB-76	Clay	8	29	63	75	26	49	31	87	UC	35	27	87	-	1.17	2.69	14
											UC	"	31	88	-	0.62		
											UC	"	32	86	-	0.65		

Notes: UC = unconfined compression test; QT = quick triaxial compression test; Q_cT = consolidated quick triaxial compression test. w% from strength tests is water content before testing. Each UC test is the average of two 1-in.-diameter test specimens taken from the top and bottom of the compaction mold.



COMBINED MORGANZA FLOODWAY CONTROL STRUCTURE
TEXAS & PACIFIC R.R AND LA. STATE HIGHWAY NO. 30

LOCATION MAP

SCALE - 1:62,5000

OCT 1948

FILE 3028

PLATE 1

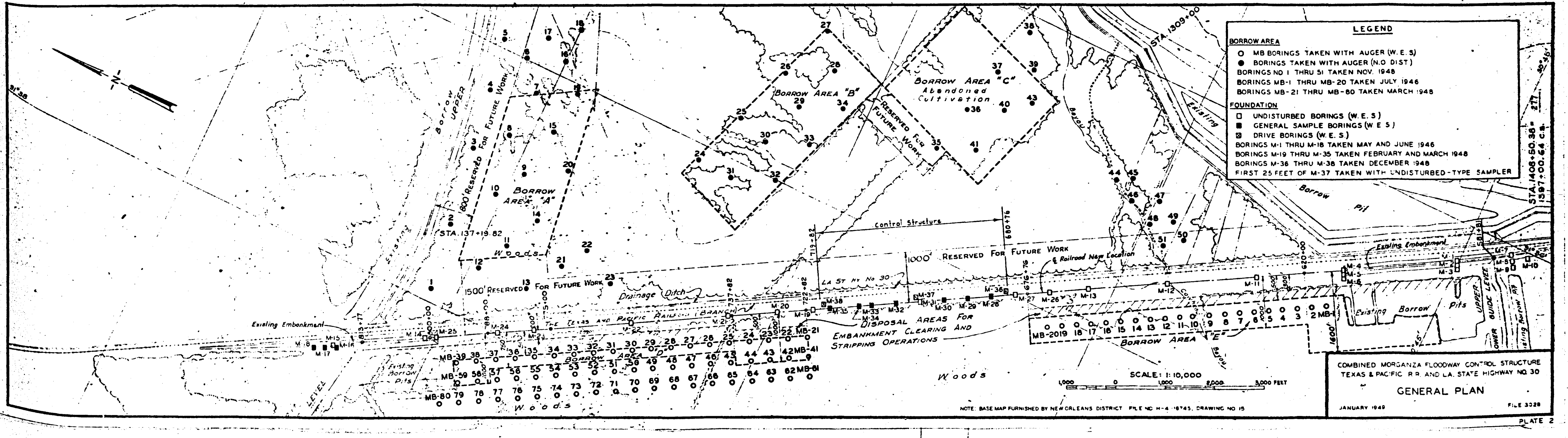

LEGEND
BORROW AREA
MB BORINGS TAKEN WITH AUGER (W. E. S)
BORINGS TAKEN WITH AUGER (N.O DIST)
BORINGS NO 1 THRU 51 TAKEN NOV. 1948
BORINGS MB-1 THRU MB-20 TAKEN JULY 1946
BORINGS MB-21 THRU MB-80 TAKEN MARCH 1948
FOUNDATION
UNDISTURBED BORINGS (W. E. S)
GENERAL SAMPLE BORINGS (W E S)
DRIVE BORINGS (W. E. S.)
BORINGS M-1 THRU M-18 TAKEN MAY AND JUNE 1946
BORINGS M-19 THRU M-35 TAKEN FEBRUARY AND MARCH 1948
BORINGS M-36 THRU M-38 TAKEN DECEMBER 1948
FIRST 25 FEET OF M-37 TAKEN WITH UNDISTURBED-TYPE SAMPLER
Borrow Area "A"
Borrow Area "B"
Borrow Area "C"
Abandoned Cultivation
Borrow Area "D"
Borrow Area "E"
800' Reserved For Future Work
1500' Reserved For Future Work
1000' Reserved For Future Work
Reserved For Future Work
Upper Borrow
STA. 137+19.82
STA. 1309+00
STA. 1408+50.38 = 1567+00.64 C.B.
Control Structure
680+76
719+82
Woods
Drainage Ditch
The Texas and Pacific Railroad Branch
LA ST HY NO 30
Railroad New Location
Existing Embankment
Existing Borrow Pits
Borrow Pit
Upper Guide Levee
Lower
Disposal Areas For Embankment Clearing And Stripping Operations
SCALE: 1:10,000
1,000 0 1,000 2,000 3,000 FEET
NOTE: BASE MAP FURNISHED BY NEW ORLEANS DISTRICT FILE NO H-4-18745, DRAWING NO 15
COMBINED MORGANZA FLOODWAY CONTROL STRUCTURE
TEXAS & PACIFIC R.R. AND LA. STATE HIGHWAY NO. 30
GENERAL PLAN
JANUARY 1949
FILE 3328

PLATE 2

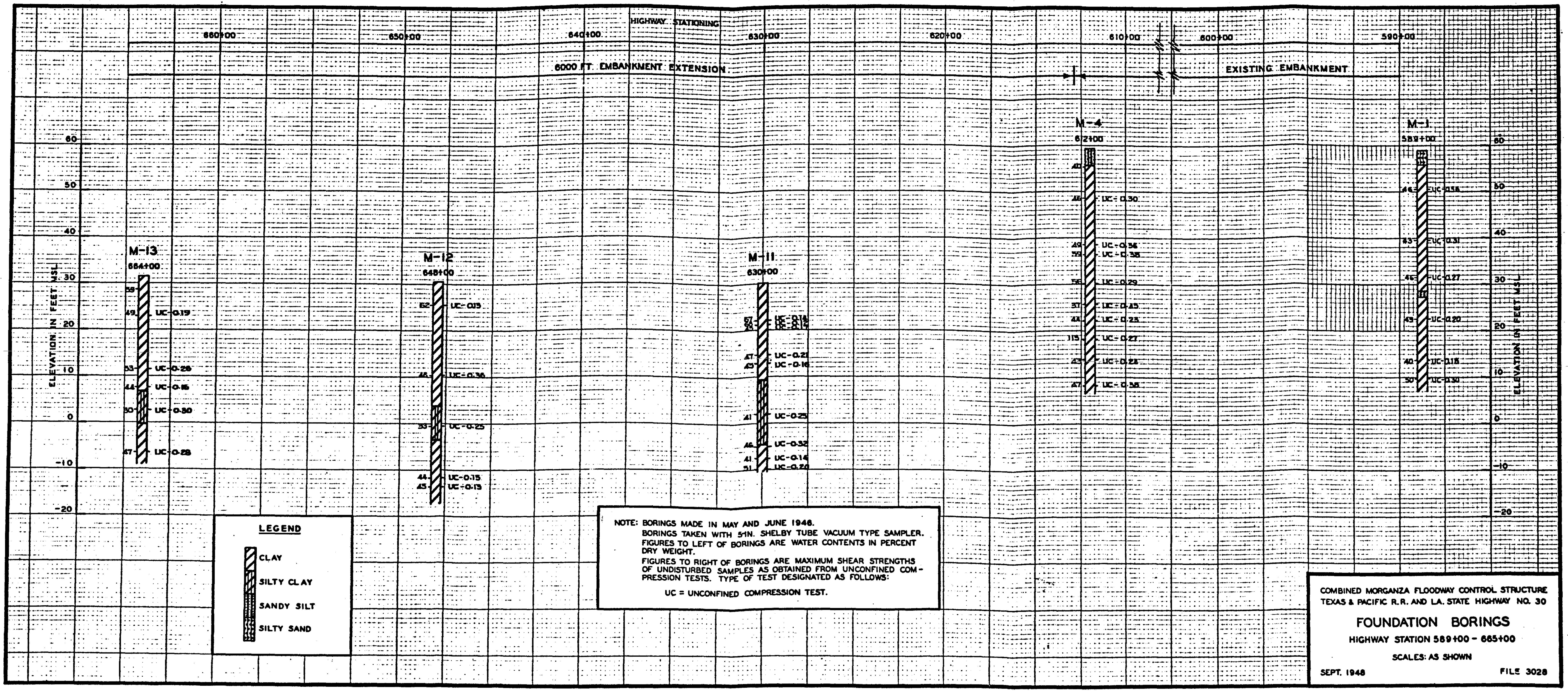
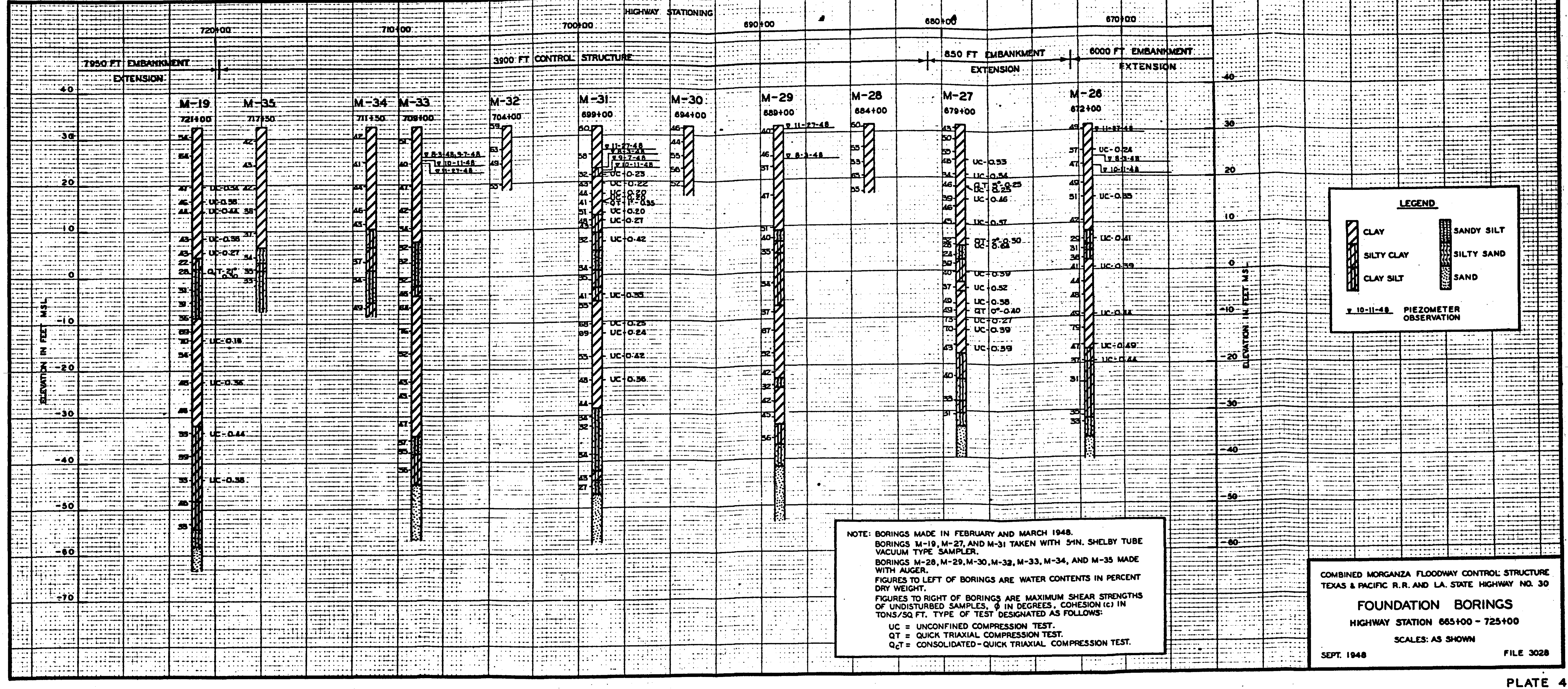

HIGHWAY STATIONING
660+00
650+00
640+00
630+00
620+00
610+00
600+00
590+00
6000 FT. EMBANKMENT EXTENSION
EXISTING EMBANKMENT
ELEVATION IN FEET MSL
M-13
664+00
M-12
648+00
M-11
630+00
M-4
612+00
M-1
589+00
LEGEND
CLAY
SILTY CLAY
SANDY SILT
SILTY SAND
NOTE: BORINGS MADE IN MAY AND JUNE 1946.
BORINGS TAKEN WITH 5 IN. SHELBY TUBE VACUUM TYPE SAMPLER.
FIGURES TO LEFT OF BORINGS ARE WATER CONTENTS IN PERCENT DRY WEIGHT.
FIGURES TO RIGHT OF BORINGS ARE MAXIMUM SHEAR STRENGTHS OF UNDISTURBED SAMPLES AS OBTAINED FROM UNCONFINED COMPRESSION TESTS. TYPE OF TEST DESIGNATED AS FOLLOWS:
UC = UNCONFINED COMPRESSION TEST.
COMBINED MORGANZA FLOODWAY CONTROL STRUCTURE
TEXAS & PACIFIC R.R. AND LA. STATE HIGHWAY NO. 30
FOUNDATION BORINGS
HIGHWAY STATION 589+00 - 665+00
SCALES: AS SHOWN
SEPT. 1948
FILE 3028

PLATE 3



NOTE: BORINGS MADE IN FEBRUARY AND MARCH 1948.

BORINGS M-19, M-27, AND M-31 TAKEN WITH 5-IN. SHELBY TUBE VACUUM TYPE SAMPLER.

BORINGS M-28, M-29, M-30, M-32, M-33, M-34, AND M-35 MADE WITH AUGER.

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

FIGURES TO RIGHT OF BORINGS ARE MAXIMUM SHEAR STRENGTHS OF UNDISTURBED SAMPLES, ϕ IN DEGREES, COHESION (c) IN TONS/SQ FT. TYPE OF TEST DESIGNATED AS FOLLOWS:

UC = UNCONFINED COMPRESSION TEST.
QT = QUICK TRIAXIAL COMPRESSION TEST.
Q_CT = CONSOLIDATED-QUICK TRIAXIAL COMPRESSION TEST.

COMBINED MORGANZA FLOODWAY CONTROL STRUCTURE
TEXAS & PACIFIC R.R. AND LA. STATE HIGHWAY NO. 30

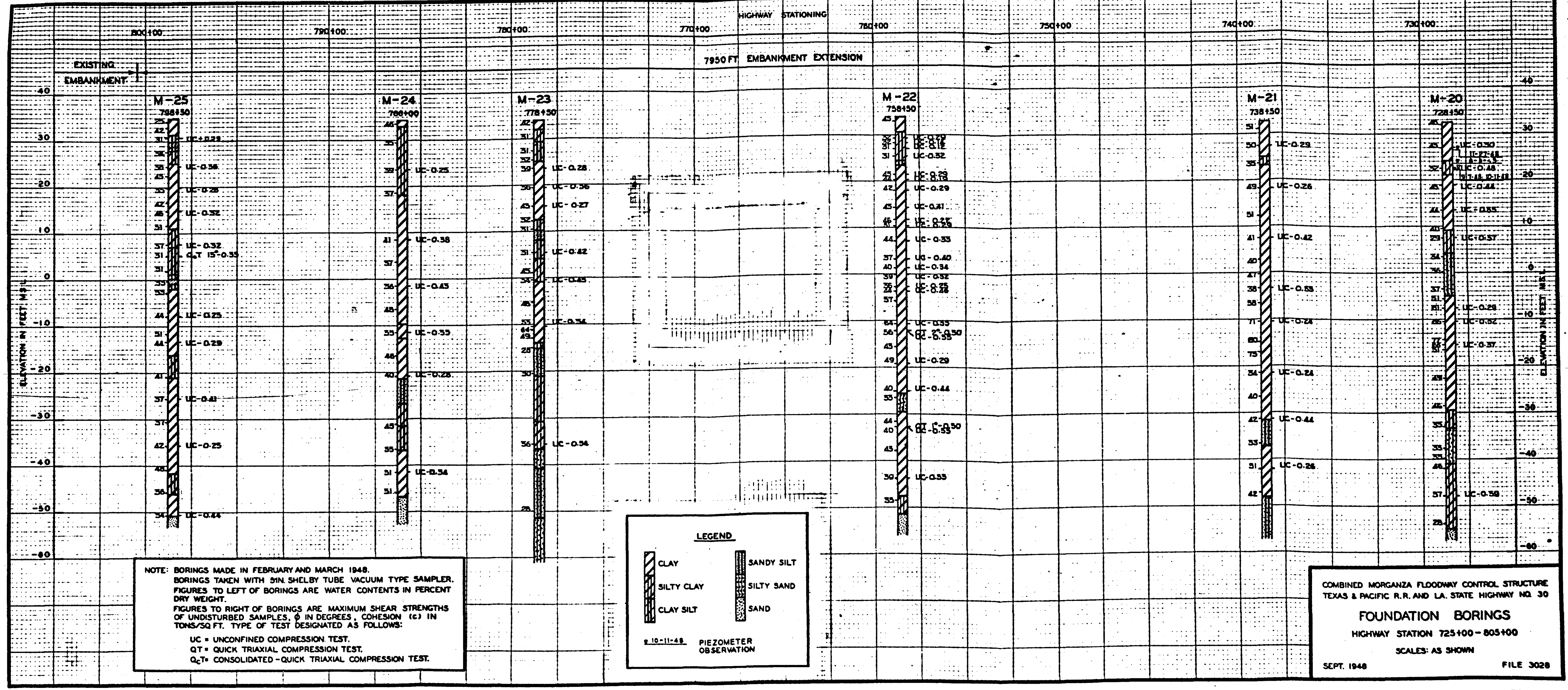
FOUNDATION BORINGS

HIGHWAY STATION 665+00 - 725+00

SCALES: AS SHOWN

SEPT. 1948 FILE 3028

PLATE 4



NOTE: BORINGS MADE IN FEBRUARY AND MARCH 1948.
BORINGS TAKEN WITH 5IN. SHELBY TUBE VACUUM TYPE SAMPLER.
FIGURES TO LEFT OF BORINGS ARE WATER CONTENTS IN PERCENT DRY WEIGHT.
FIGURES TO RIGHT OF BORINGS ARE MAXIMUM SHEAR STRENGTHS OF UNDISTURBED SAMPLES, ϕ IN DEGREES, COHESION (c) IN TONS/SQ FT. TYPE OF TEST DESIGNATED AS FOLLOWS:

UC = UNCONFINED COMPRESSION TEST.
QT = QUICK TRIAXIAL COMPRESSION TEST.
Q_cT = CONSOLIDATED-QUICK TRIAXIAL COMPRESSION TEST.

COMBINED MORGANZA FLOODWAY CONTROL STRUCTURE
TEXAS & PACIFIC R.R. AND LA. STATE HIGHWAY NO. 30

FOUNDATION BORINGS

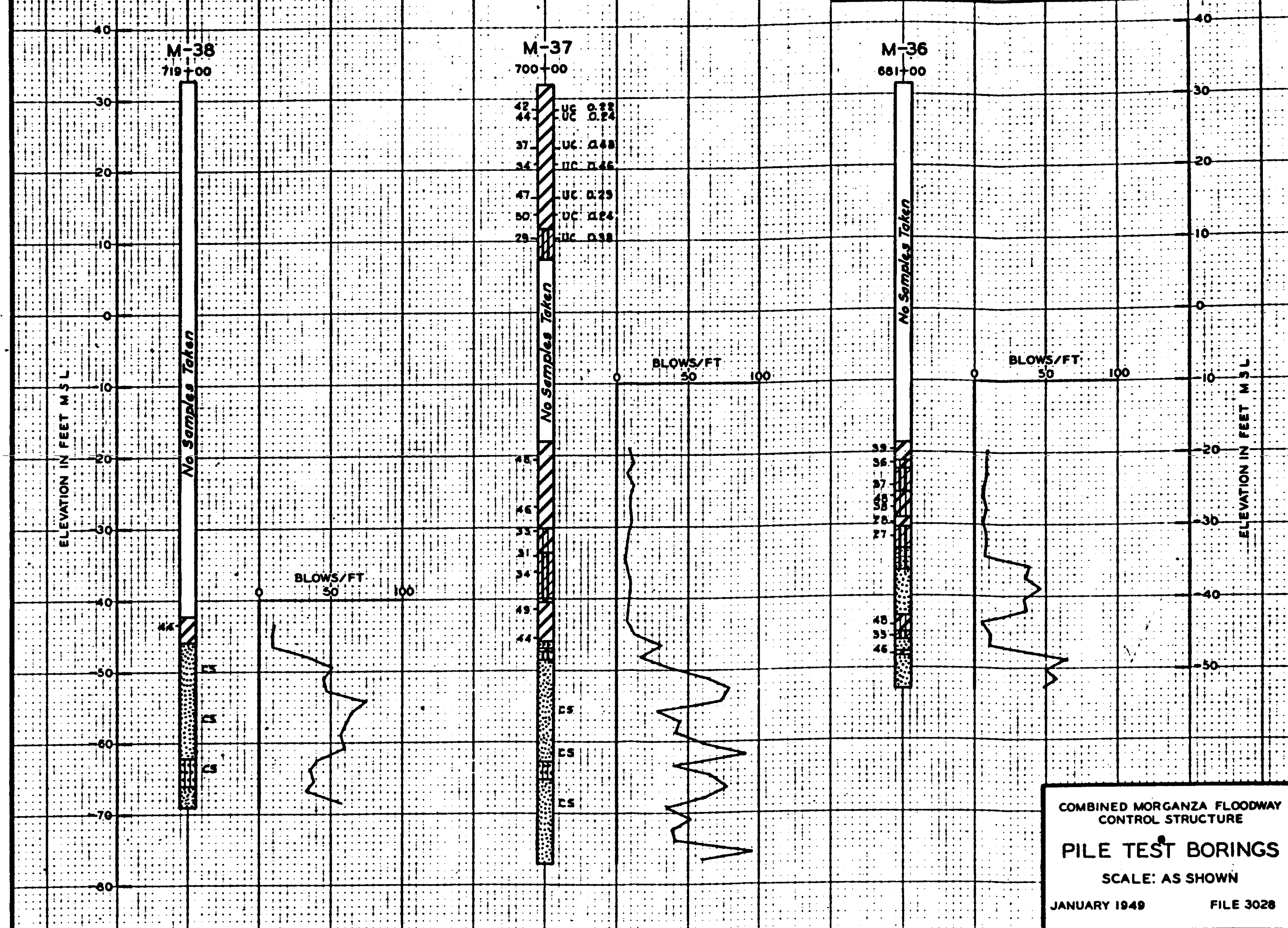
HIGHWAY STATION 725+00 - 805+00

SCALES: AS SHOWN

SEPT. 1948 FILE 3028

PLATE 5

NOTES:
1. BORINGS MADE IN DECEMBER 1948.
2. BORINGS MADE WITH 5-IN. SHELBY TUBE AND 1-½ IN. SPLIT SPOON SAMPLERS.
3. DRIVING RESISTANCE OBTAINED USING 140 LB HAMMER WITH 30 IN. DROP.
4. SEE PLATE 5 FOR LEGEND.

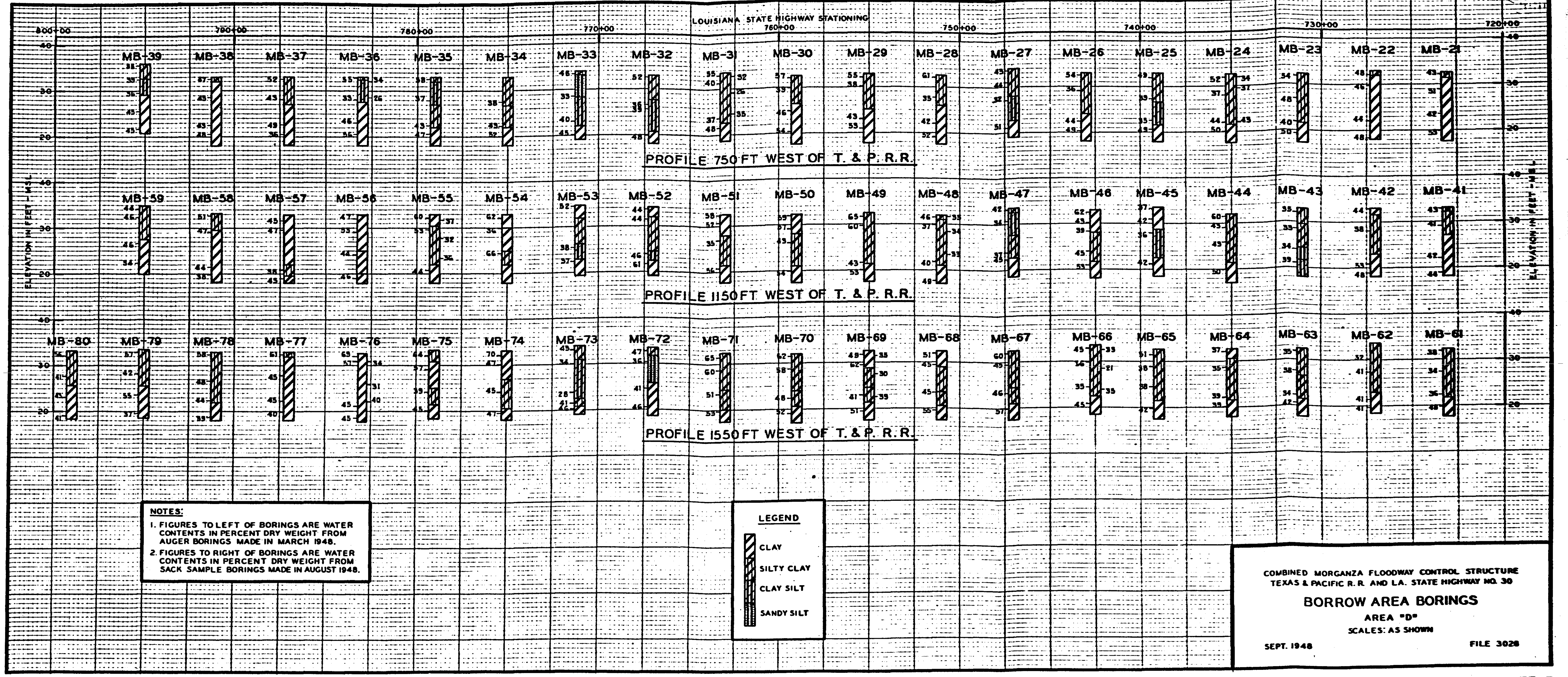


COMBINED MORGANZA FLOODWAY CONTROL STRUCTURE

PILE TEST BORINGS

SCALE: AS SHOWN

JANUARY 1949 FILE 3028



NOTES:

1. FIGURES TO LEFT OF BORINGS ARE WATER CONTENTS IN PERCENT DRY WEIGHT FROM AUGER BORINGS MADE IN MARCH 1948.
2. FIGURES TO RIGHT OF BORINGS ARE WATER CONTENTS IN PERCENT DRY WEIGHT FROM SACK SAMPLE BORINGS MADE IN AUGUST 1948.

LEGEND

- CLAY
- SILTY CLAY
- CLAY SILT
- SANDY SILT

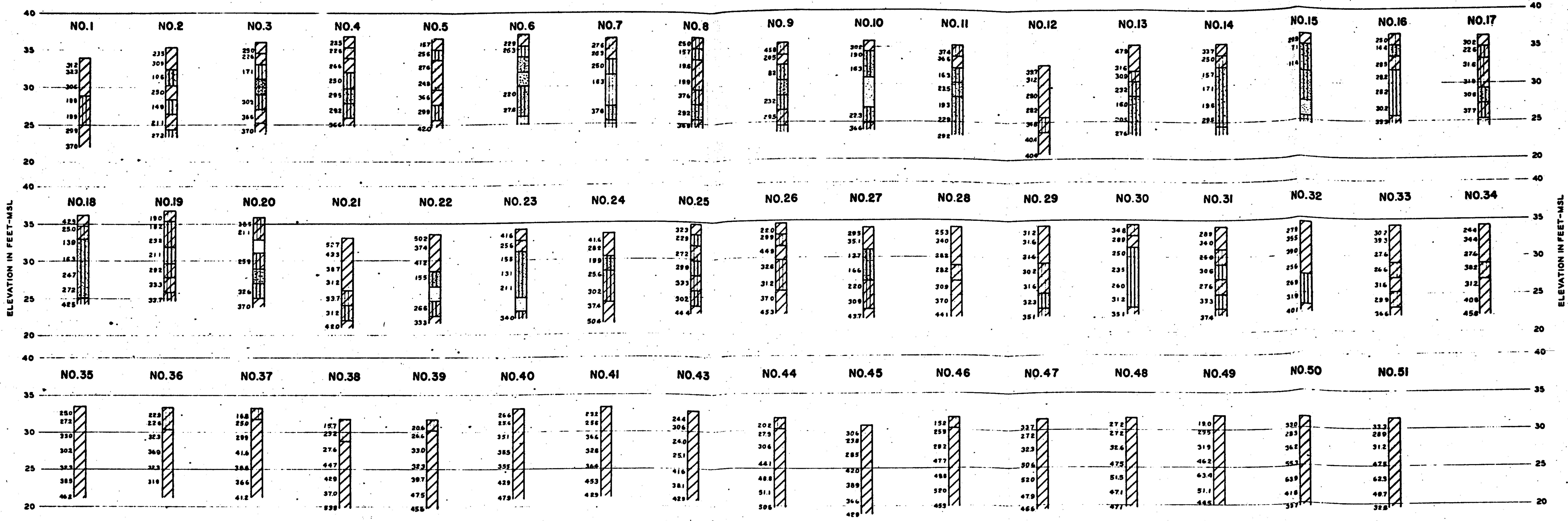
COMBINED MORGANZA FLOODWAY CONTROL STRUCTURE
TEXAS & PACIFIC R.R. AND LA. STATE HIGHWAY NO. 30

BORROW AREA BORINGS

AREA "D"

SCALES: AS SHOWN

SEPT. 1948 FILE 3028



NOTE:

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

2. BORINGS MADE WITH AUGER BY THE NEW ORLEANS DISTRICT IN OCTOBER AND NOVEMBER 1948

LEGEND

- CLAY
- SILTY CLAY
- CLAY SILT
- SANDY SILT
- SILTY SAND
- SAND
- SILT

COMBINED MORGANZA FLOODWAY CONTROL STRUCTURE
TEXAS & PACIFIC R. R. AND LA. STATE HIGHWAY NO. 30

BORROW AREA BORINGS

AREAS A, B, AND C

SCALES: AS SHOWN

FEBRUARY 1949 FILE 3028

PLATE 8

Boring M-22 - Sample 7
Elev +25.5
Clay silt, with some sand, grey & brown, firm

Boring M-22 - Sample 22
Elev +7.2
Clay, somewhat mottled, grey, firm

Boring M-22 - Sample 37
Elev -10.8
Clay, fat & very compressible, grey, firm.

Boring M-22 - Sample 50
Elev -43.7
Clay, with sandy silt lenses, grey, stiff (sample disturbed)

Combined Morganza Floodway
Control Structure

PHOTOGRAPHS OF SLICED SAMPLES
(one-third size)

Nov 1948 File 3028

Boring M-27 - Sample 16
Elev +15.5
Clay, mottled grey & brown with some concretions and silt lenses, firm

Boring M-31 - Sample 31
Elev -4.0
Silty clay with silt lenses, grey, firm

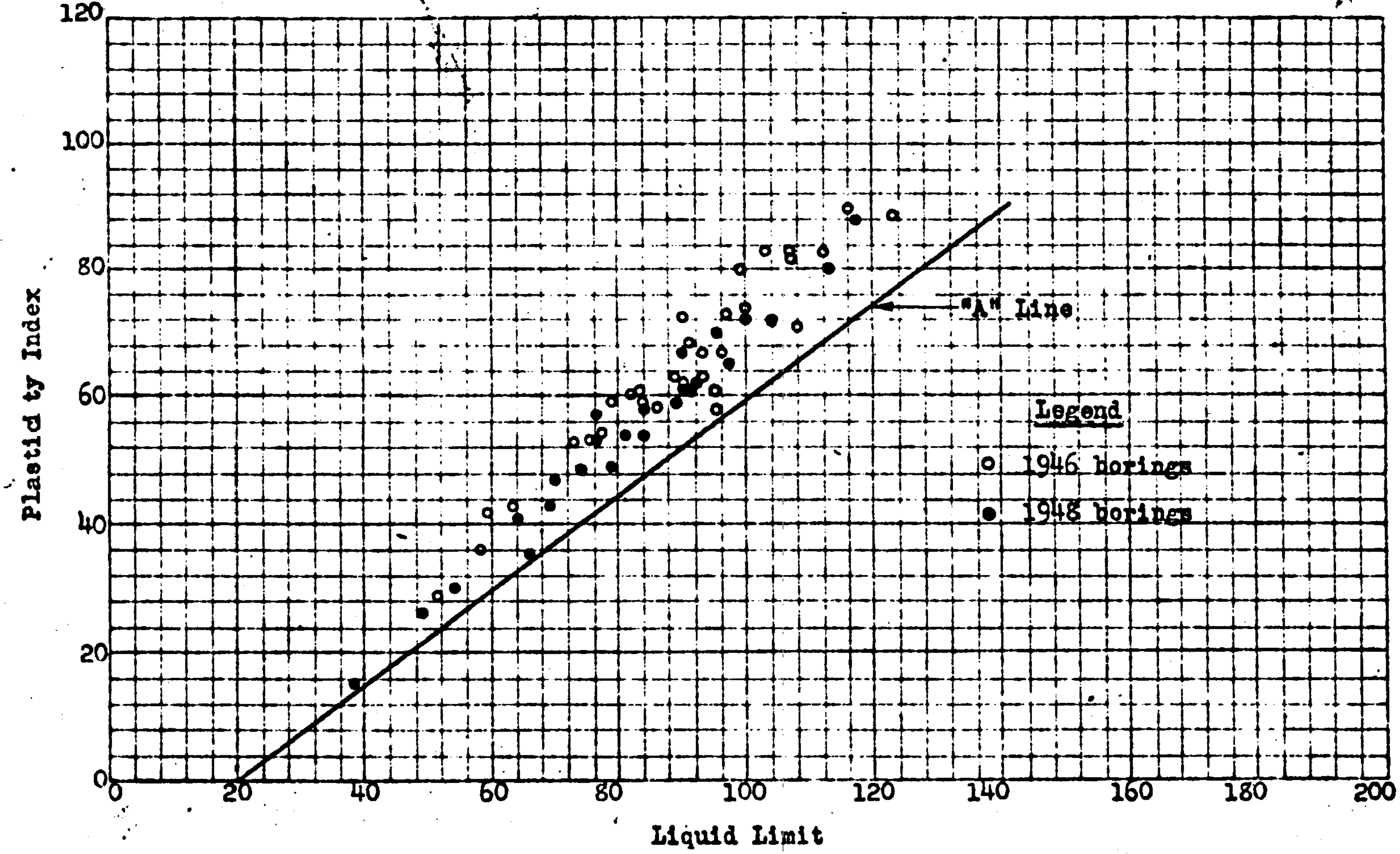
Boring M-31 - Sample 9
Elev +19.4
Clay, mottled grey, firm

Boring M-31 - Sample 43
Elev -22.5
Clay, with some silt pockets, grey, firm

Combined Morganza Floodway
Control Structure

PHOTOGRAPHS OF SLICED SAMPLES
(one-third size)

Nov 1948 File 3028

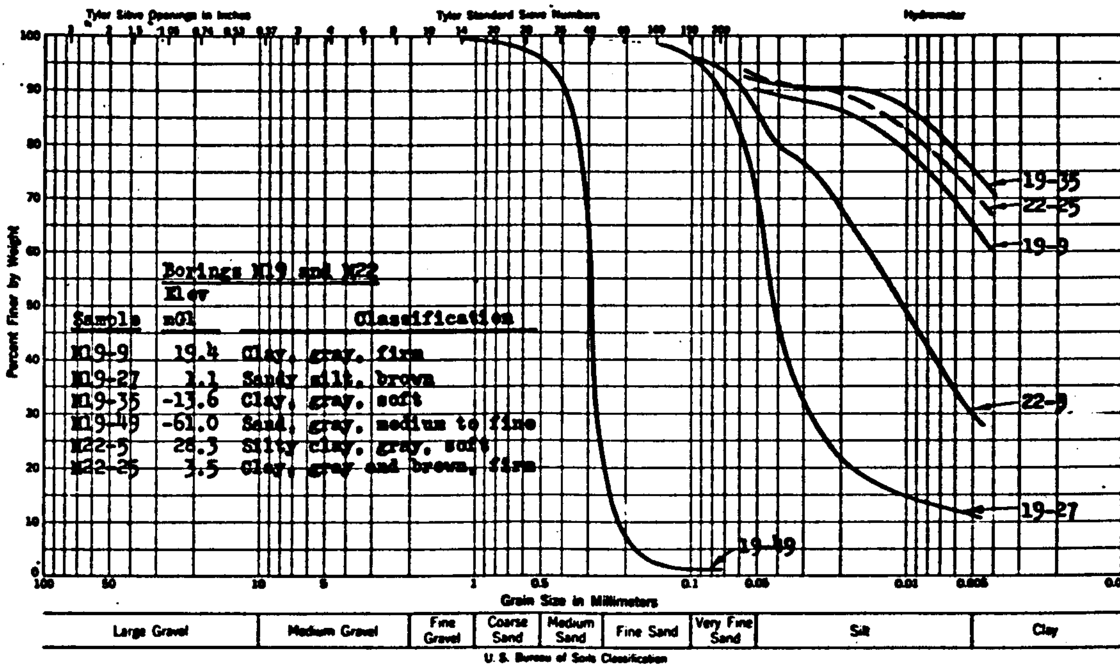


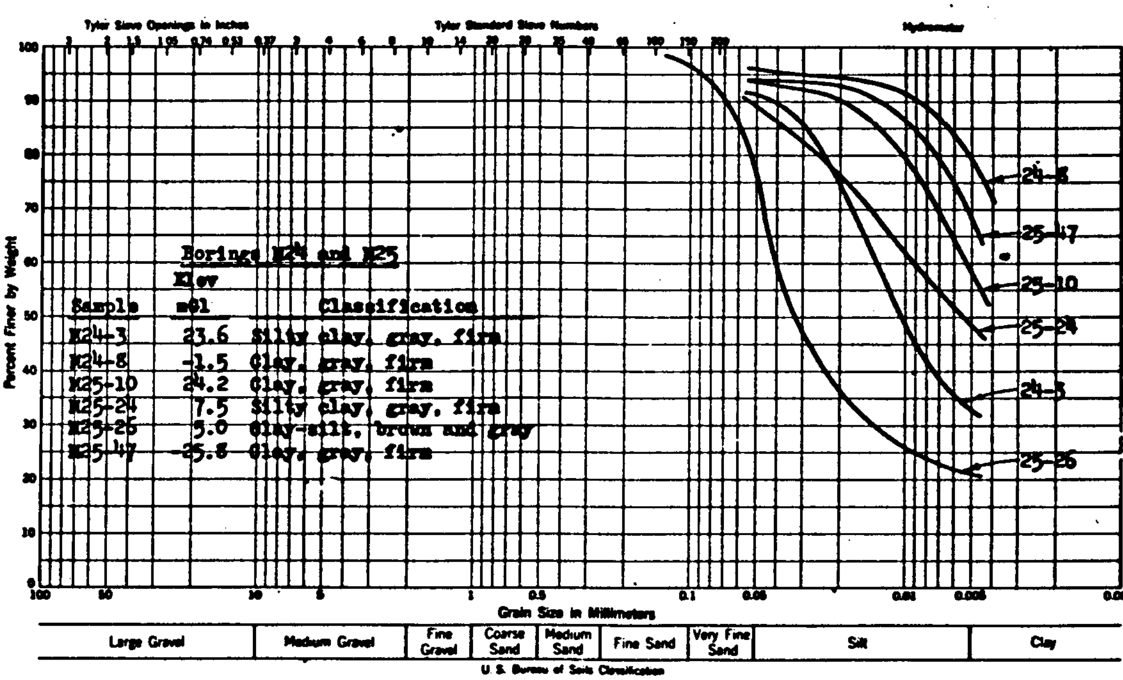
Combined Morganza Floodway Control Structure

PLASTICITY CHART

November 1948

File 3028

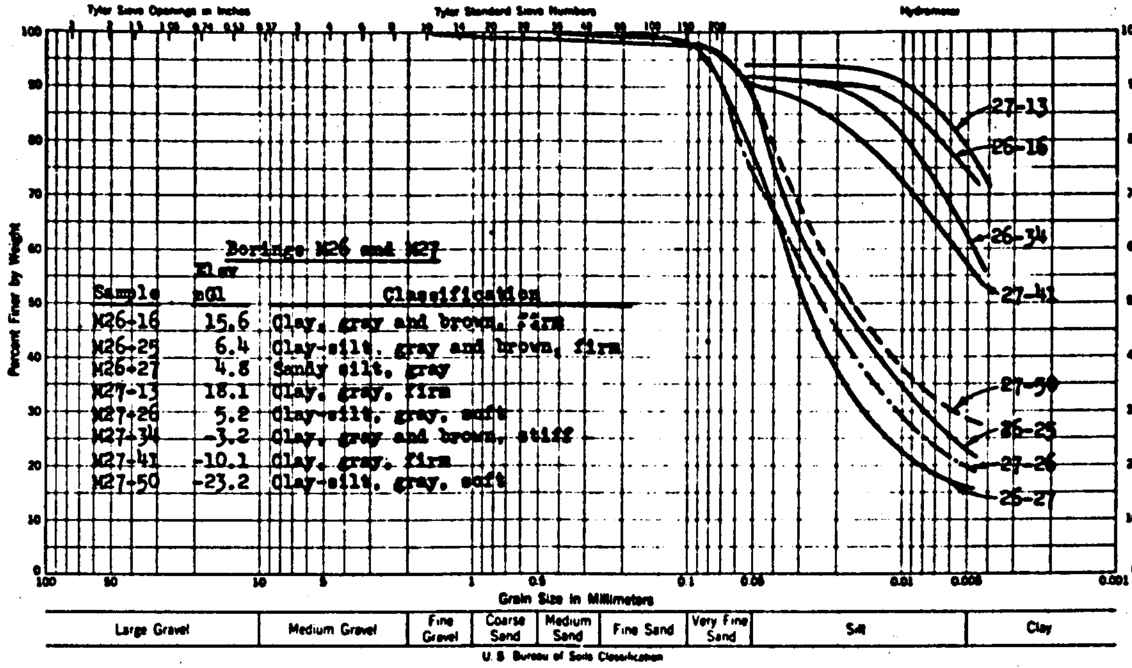


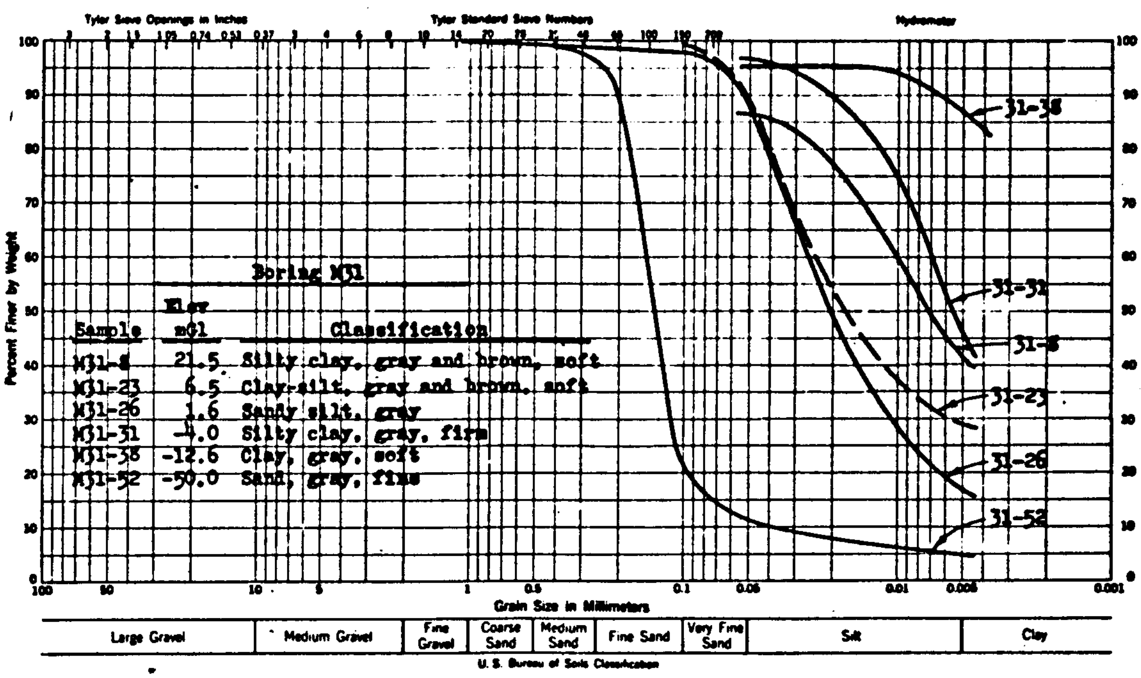


COMBINED MORGANZA FLOODWAY CONTROL STRUCTURE
TEXAS & PACIFIC R.R. AND LA. STATE HIGHWAY NO. 30

TYPICAL GRAIN SIZE CURVES OF FOUNDATION SOILS

NOV 1948 FILE 3028

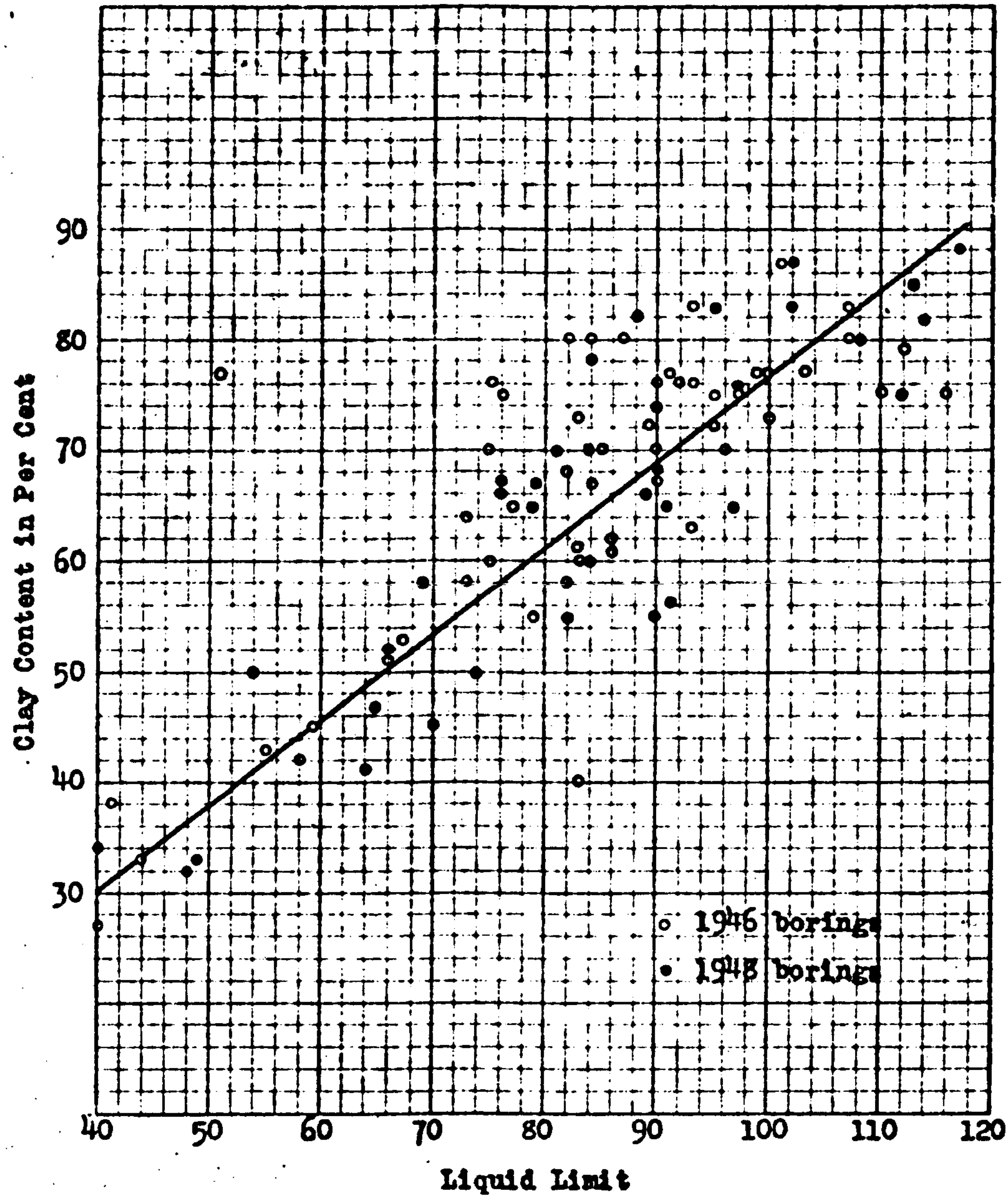




COMBINED MORGANZA FLOODWAY CONTROL STRUCTURE
TEXAS & PACIFIC R.R. AND LA. STATE HIGHWAY NO. 30

TYPICAL GRAIN SIZE CURVES OF FOUNDATION SOILS

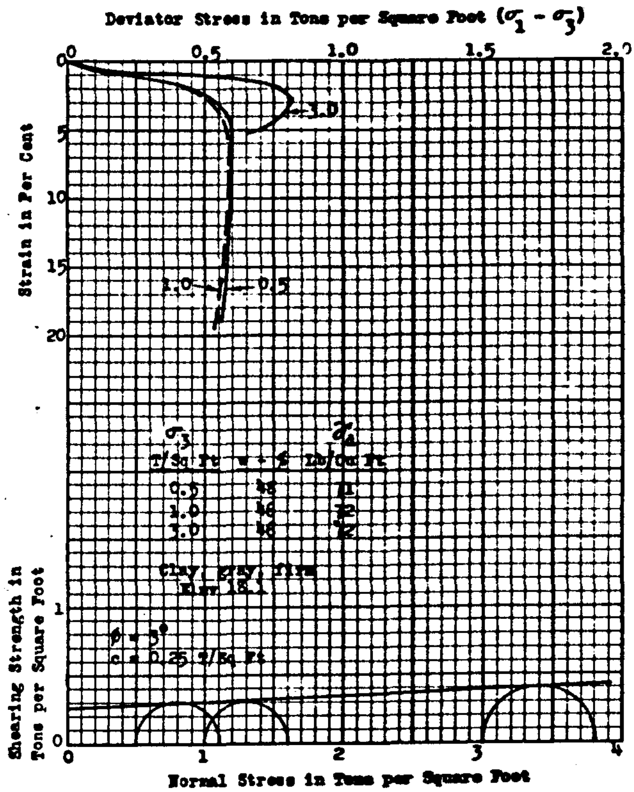
OCT 1948 FILE 3028



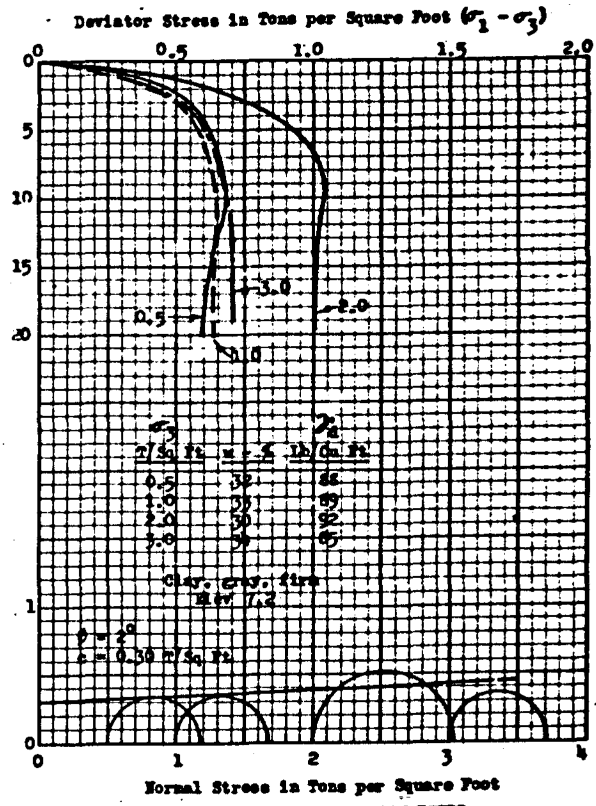
Combined Morganza Floodway Control Structure

CLAY CONTENT VERSUS LIQUID LIMIT

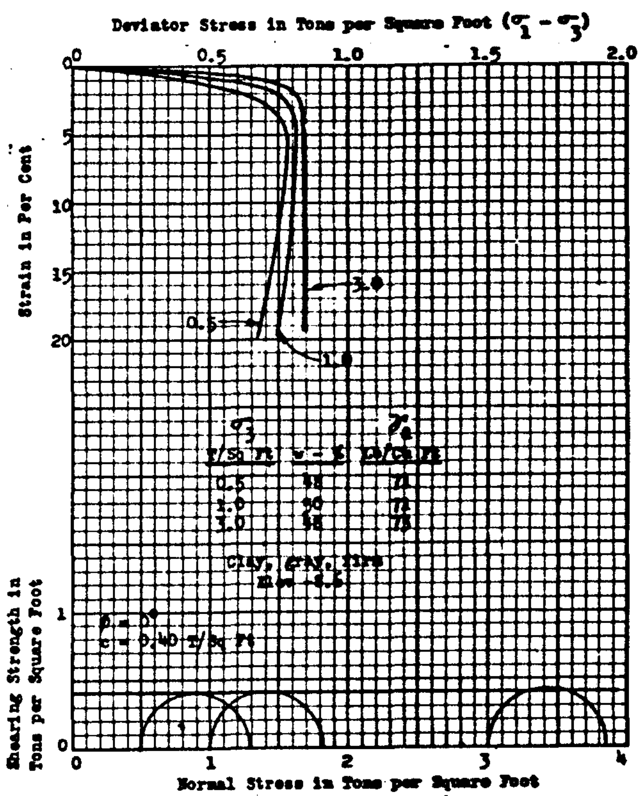
November 1948 | File 3028



QUICK TRIAXIAL COMPRESSION TESTS
Boring M27 Sample 13



QUICK TRIAXIAL COMPRESSION TESTS
Boring M27 Sample 24

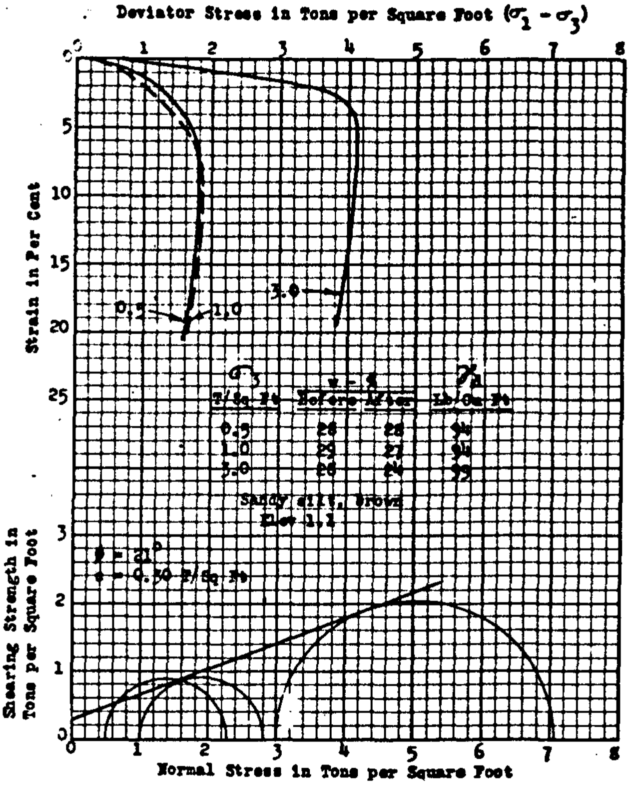


QUICK TRIAXIAL COMPRESSION TESTS
Boring M27 Sample 40

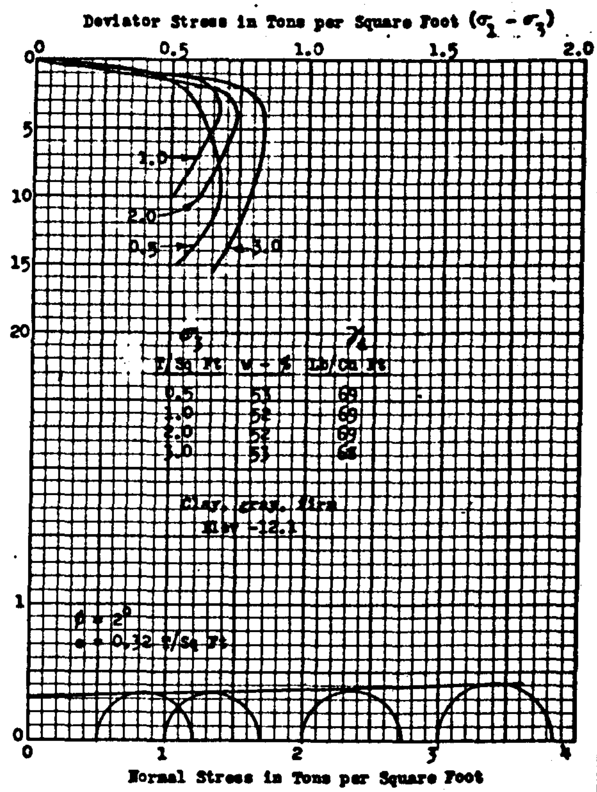
COMBINED MORGANZA FLOODWAY CONTROL STRUCTURE
TEXAS & PACIFIC R.R. AND LA. STATE HIGHWAY NO. 30

TRIAXIAL COMPRESSION TESTS

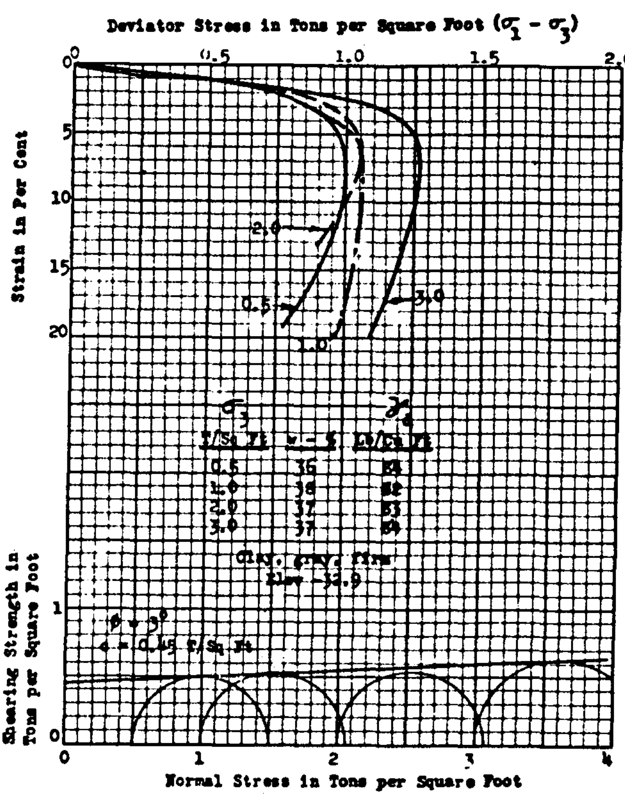
OCT 1948 FILE 3028



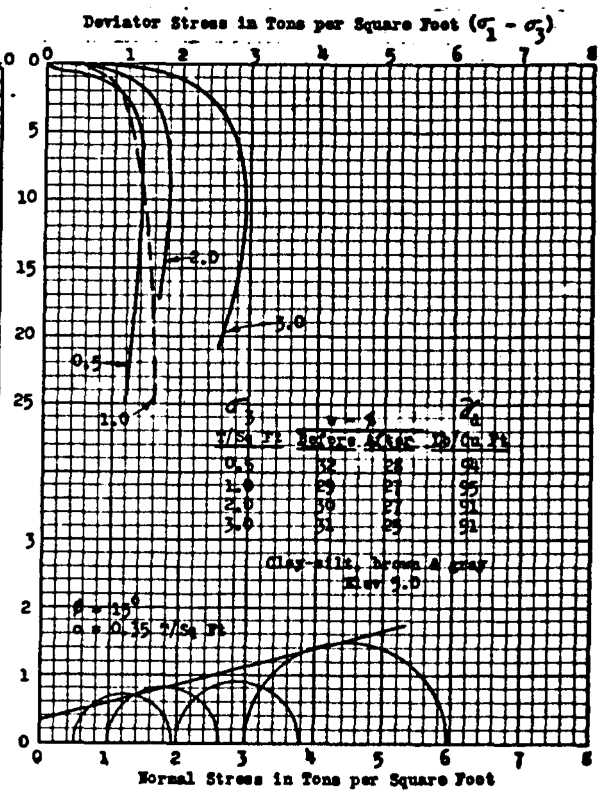
CONSOLIDATED QUICK TRIAXIAL COMPRESSION TEST
Boring M19 Sample 27



QUICK TRIAXIAL COMPRESSION TESTS
Boring M22 Sample 38



QUICK TRIAXIAL COMPRESSION TESTS
Boring M22 Sample 46

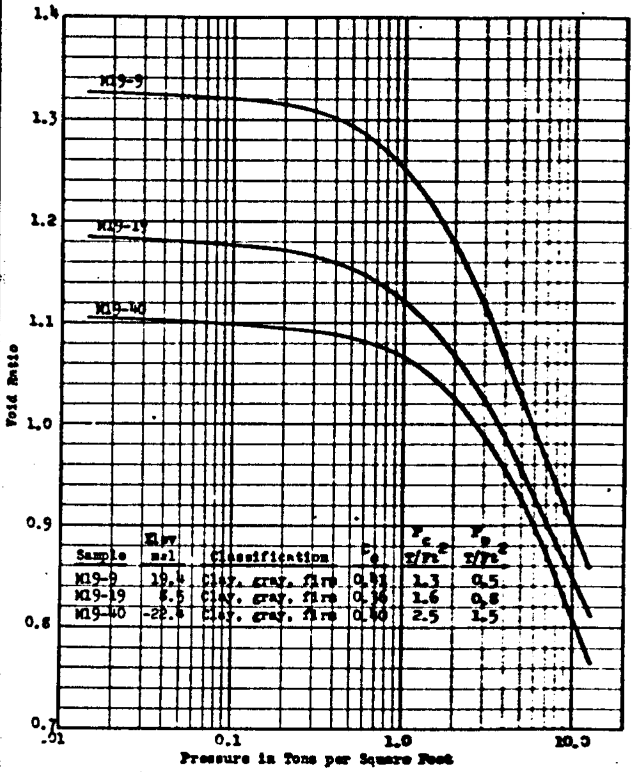


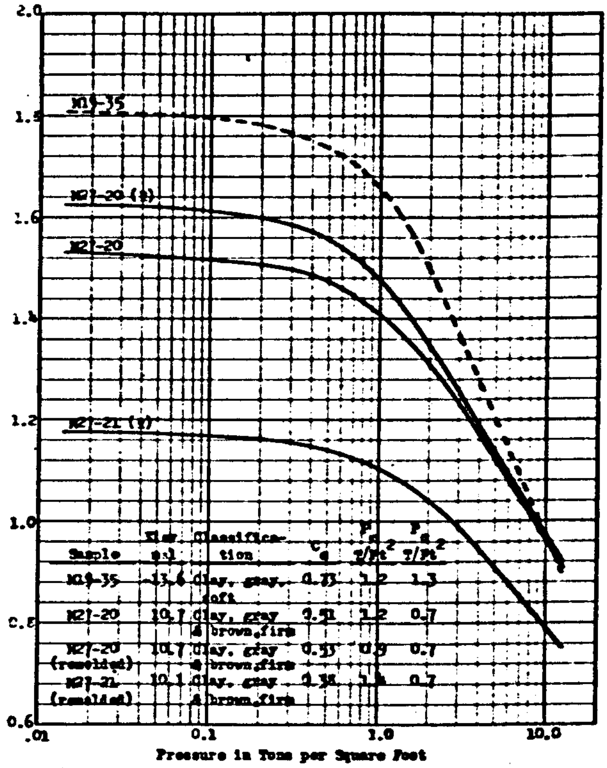
CONSOLIDATED QUICK TRIAXIAL COMPRESSION TEST
Boring M25 Sample 26

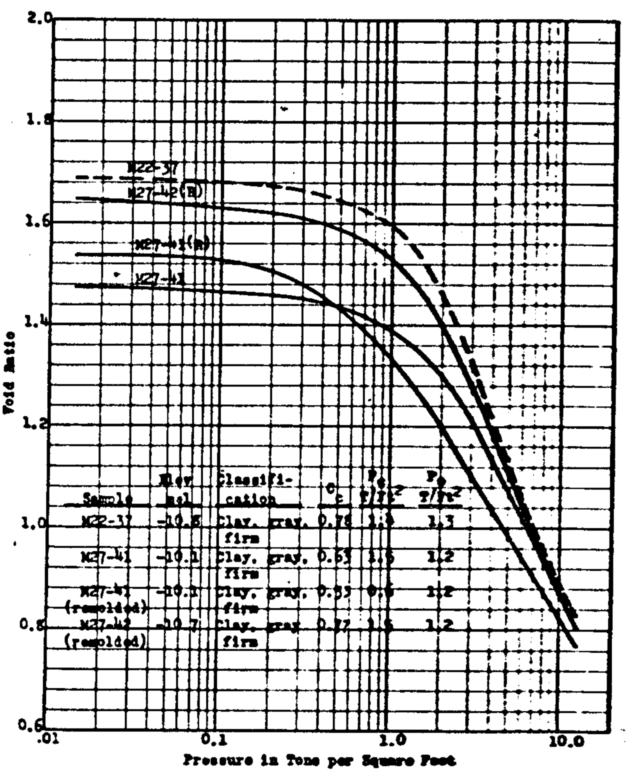
COMBINED MORGANZA FLOODWAY CONTROL STRUCTURE
TEXAS & PACIFIC R.R. AND LA. STATE HIGHWAY NO. 30

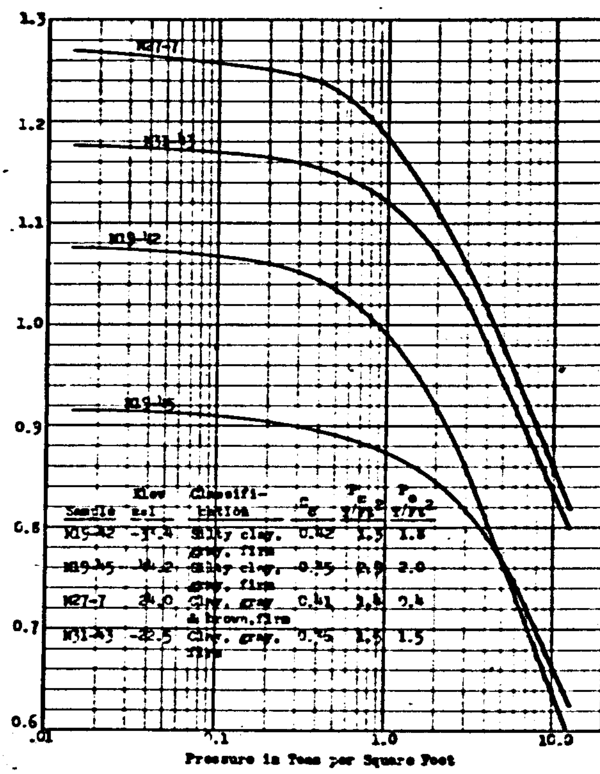
TRIAXIAL COMPRESSION TESTS

OCT 1948 FILE 3028





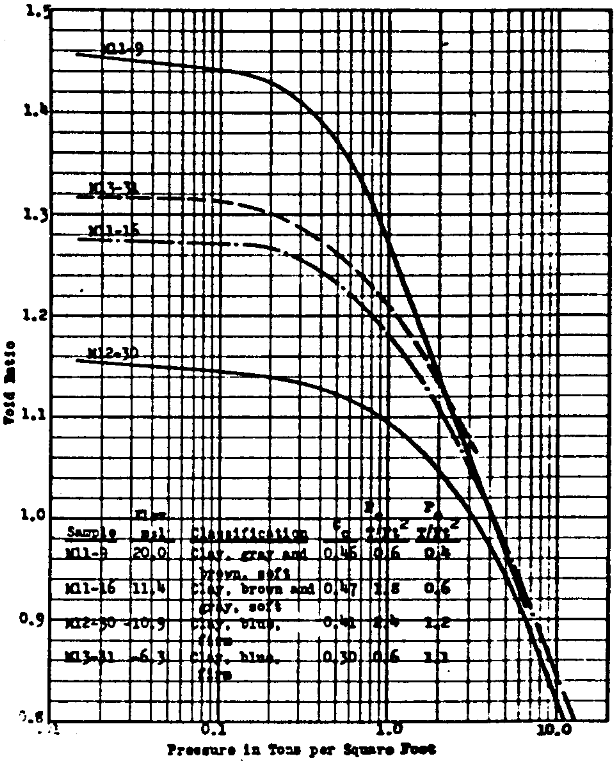


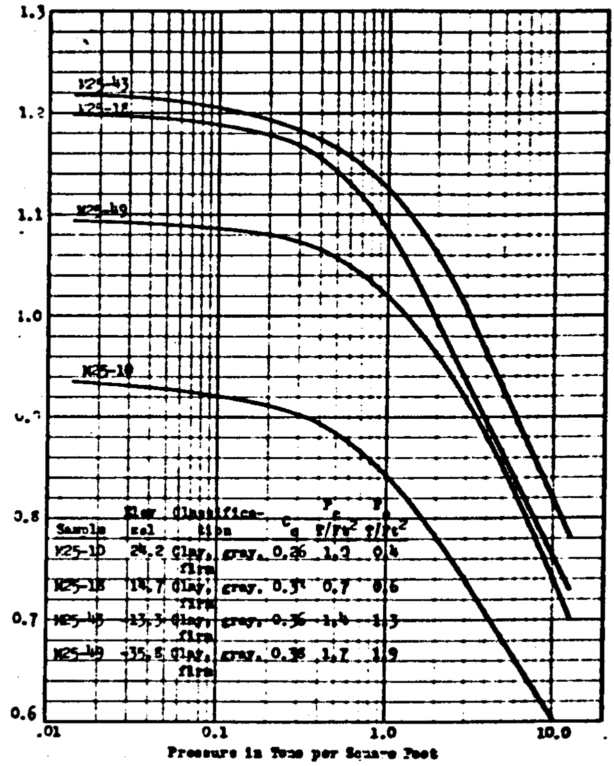


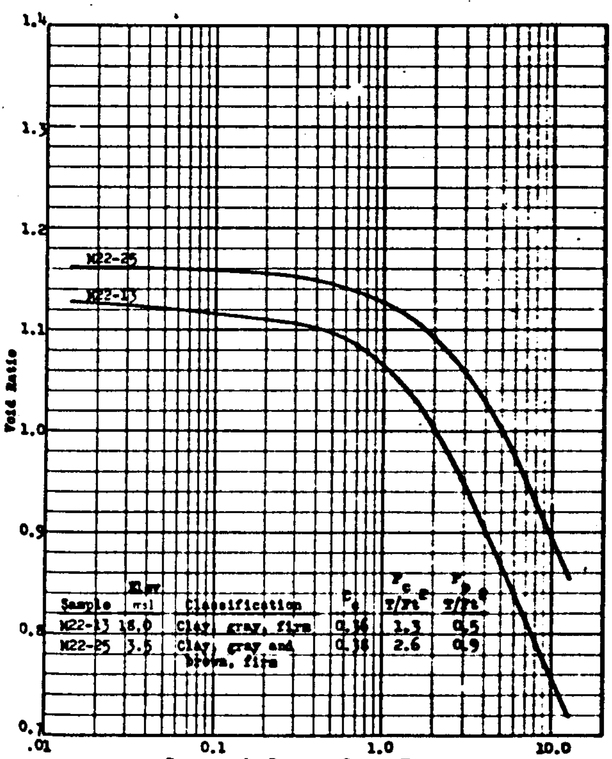
COMBINED MORGANZA FLOODWAY CONTROL STRUCTURE
TEXAS & PACIFIC R.R. AND LA. STATE HIGHWAY NO. 30

CONSOLIDATION CURVES FOR
CONTROL STRUCTURE FOUNDATION

NOV 1948 FILE 3028



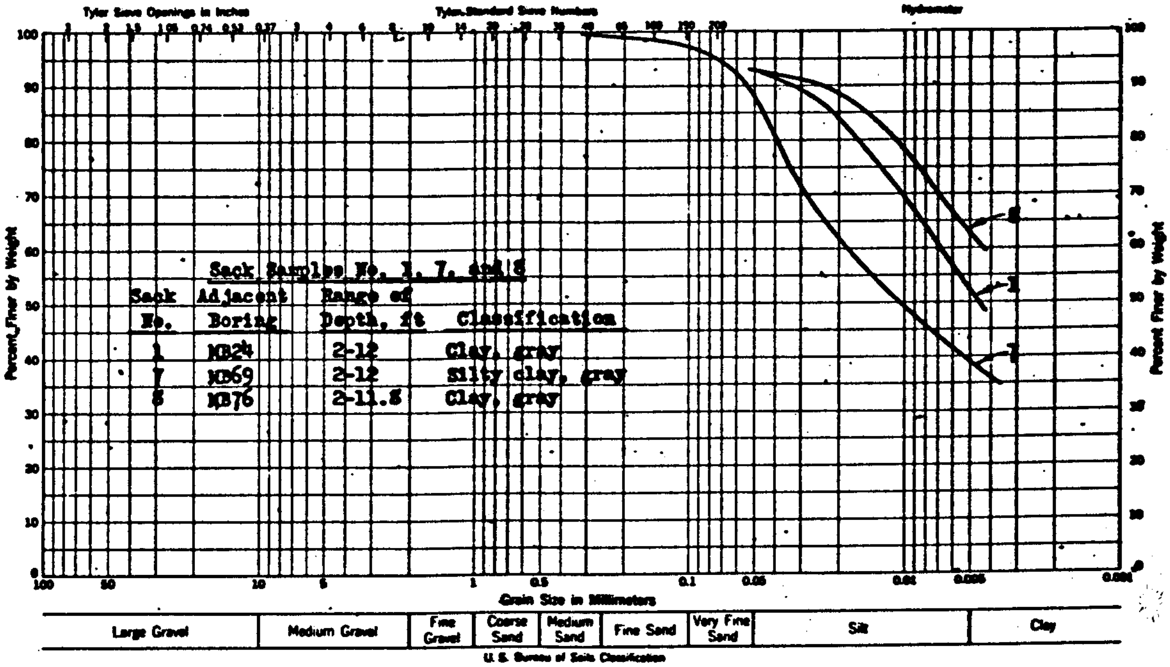


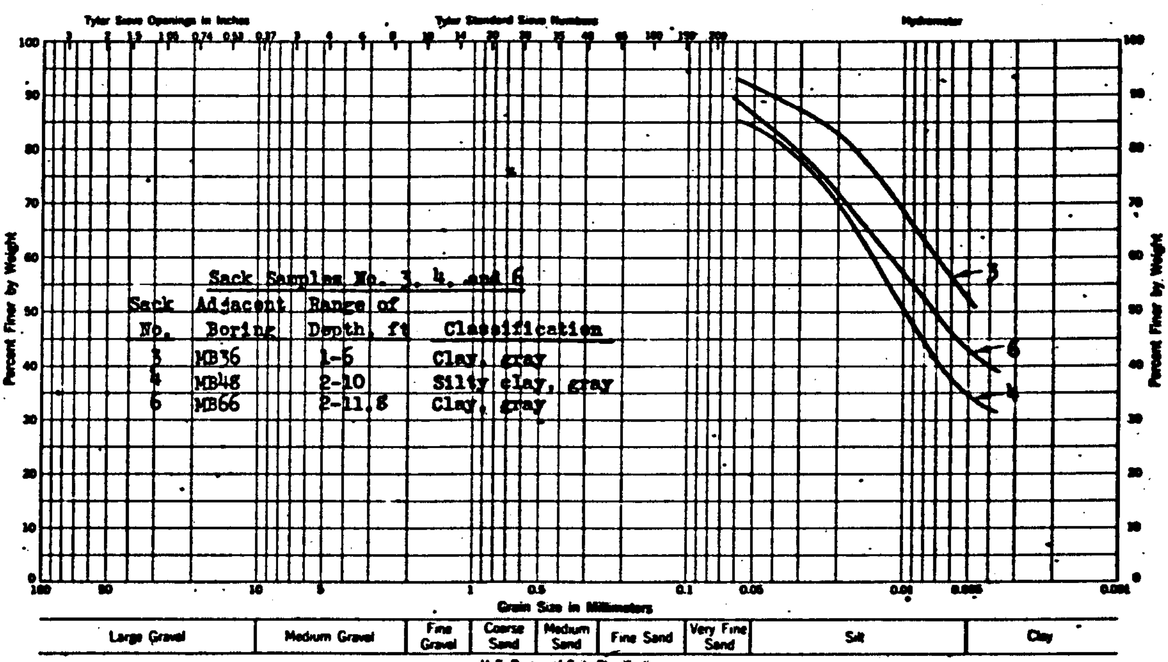


COMBINED MORGANZA FLOODWAY CONTROL STRUCTURE
TEXAS & PACIFIC R.R. AND LA. STATE HIGHWAY NO. 30

CONSOLIDATION CURVES FOR EMBANKMENT FOUNDATION

NOV 1948 FILE 3028

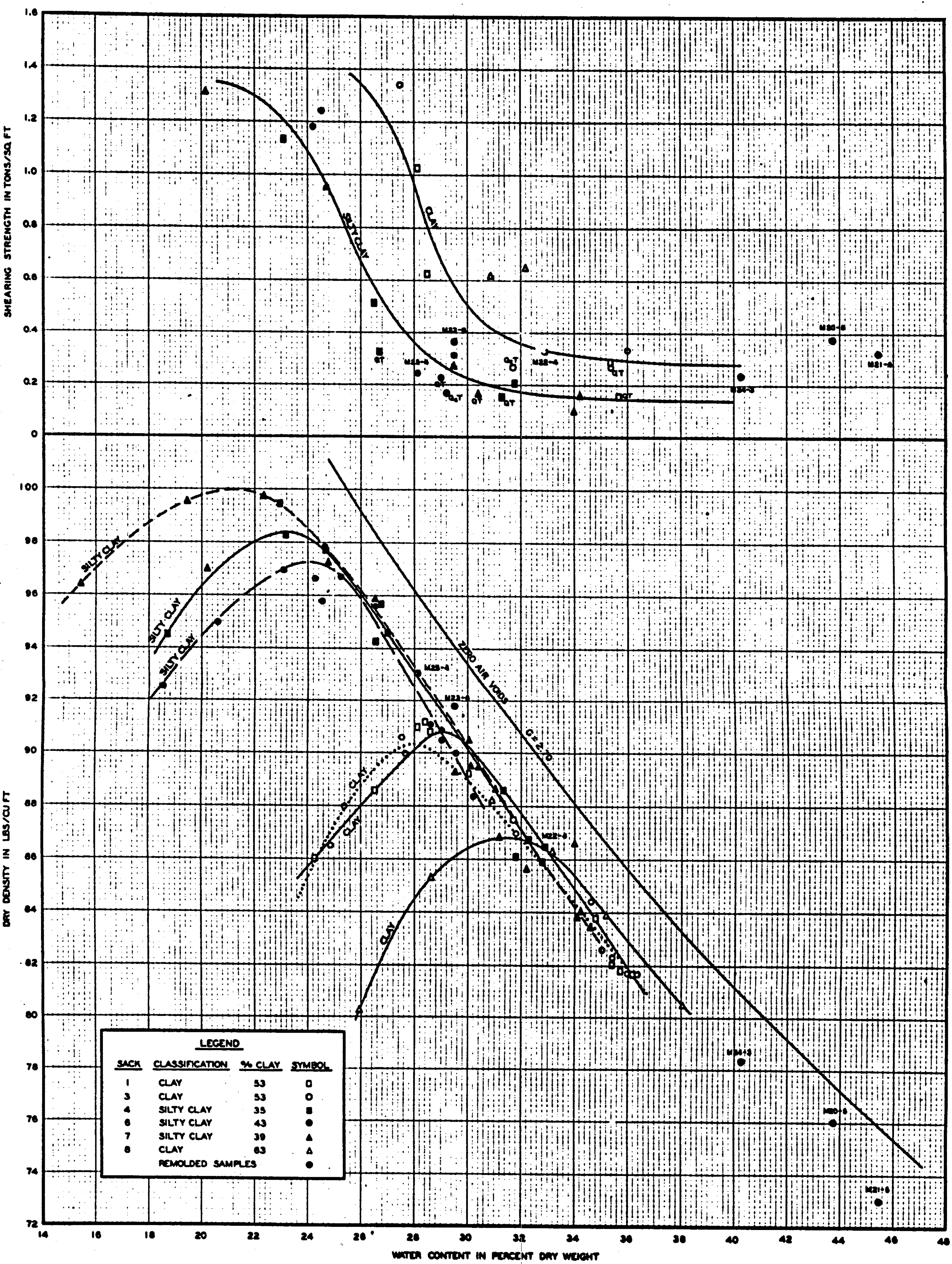




COMBINED MORGANZA FLOODWAY CONTROL STRUCTURE
TEXAS & PACIFIC R.R. AND LA. STATE HIGHWAY NO. 30

GRAIN SIZE CURVES OF BORROW MATERIAL

OCT 1948 FILE 3028

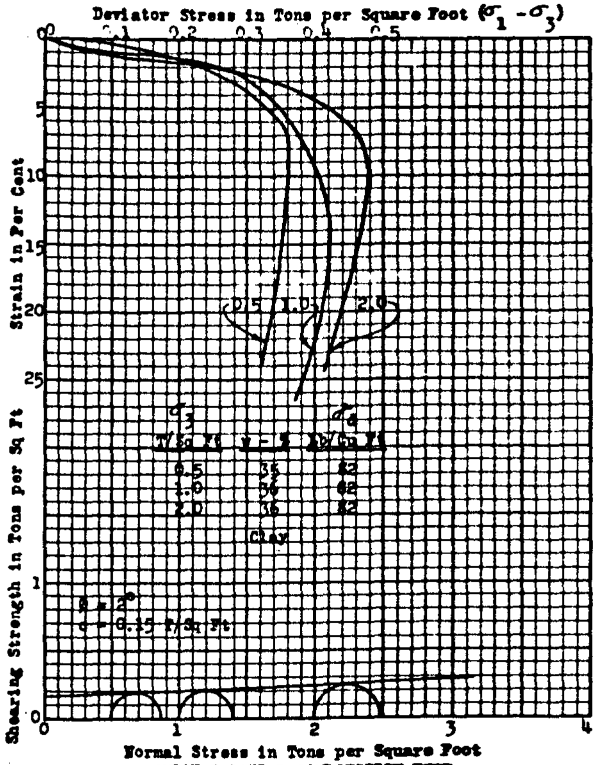


NOTE: ONLY 15 BLOWS PER LAYER USED IN THE PROCTOR COMPACTION TESTS.
MAXIMUM SHEARING STRENGTHS WERE OBTAINED FROM UNCONFINED AND TRIAXIAL COMPRESSION TESTS.
SHEARING STRENGTHS FOR REMOLDED SAMPLES WERE OBTAINED FROM UNCONFINED COMPRESSION TESTS.
SHEARING STRENGTHS OBTAINED FROM TRIAXIAL COMPRESSION TESTS DENOTED BY QT(QUICK) AND QcT(CONSOLIDATED-QUICK) ADJACENT TO ADJACENT SYMBOL.

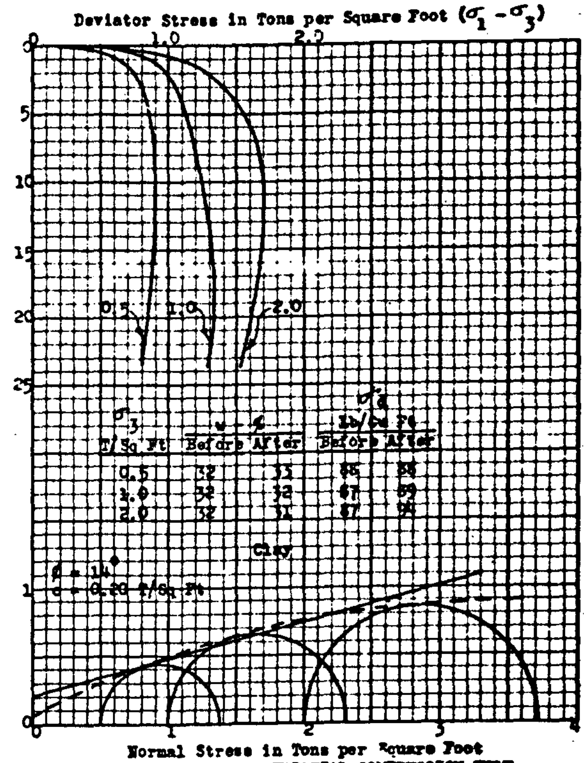
COMBINED MORGANZA FLOODWAY CONTROL STRUCTURE
TEXAS & PACIFIC R.R. AND LA. STATE HIGHWAY NO. 30

COMPACTION CURVES
AND
MAXIMUM SHEARING STRENGTHS

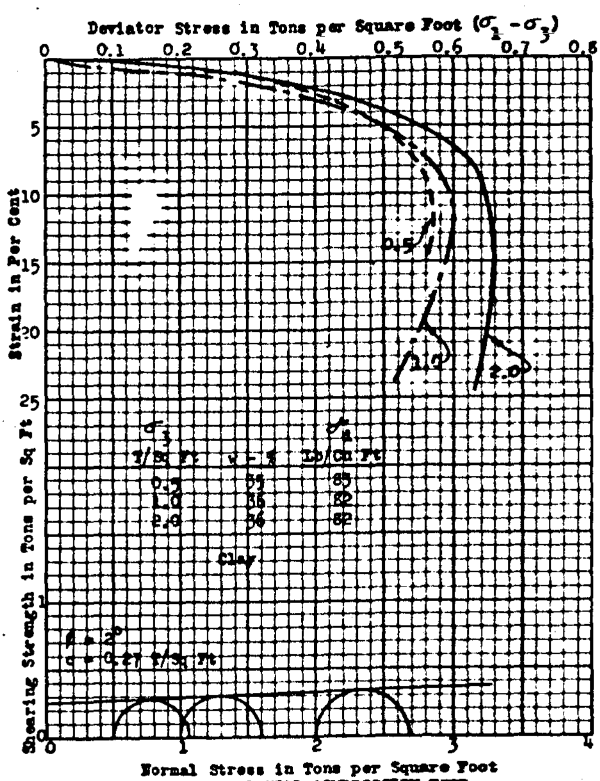
SEPTEMBER 1948 FILE 3028



QUICK TRIAXIAL COMPRESSION TEST
Sack No. 1



CONSOLIDATED QUICK TRIAXIAL COMPRESSION TEST
Sack No. 3



QUICK TRIAXIAL COMPRESSION TEST
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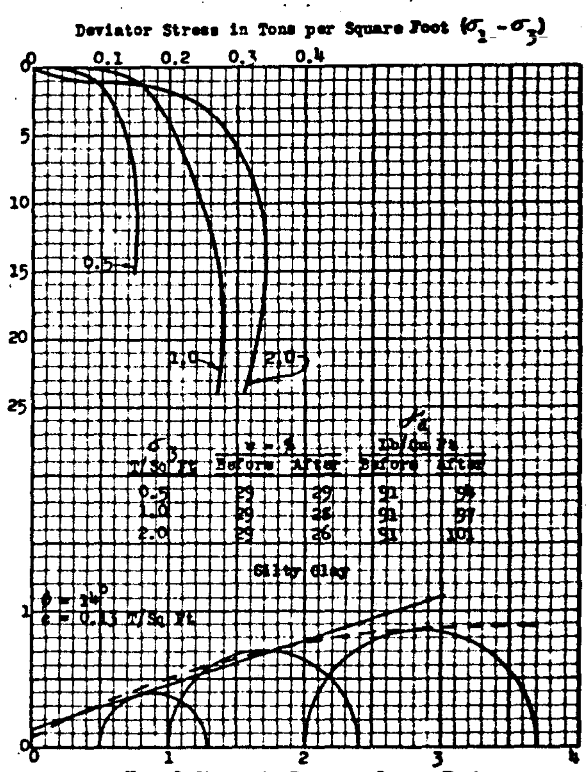
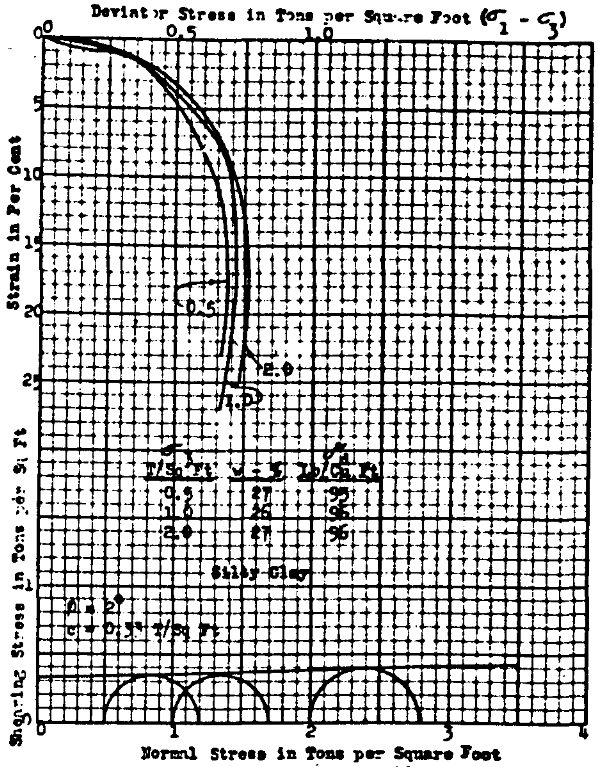


CONSOLIDATED QUICK TRIAXIAL COMPRESSION TEST
Sack No. 6

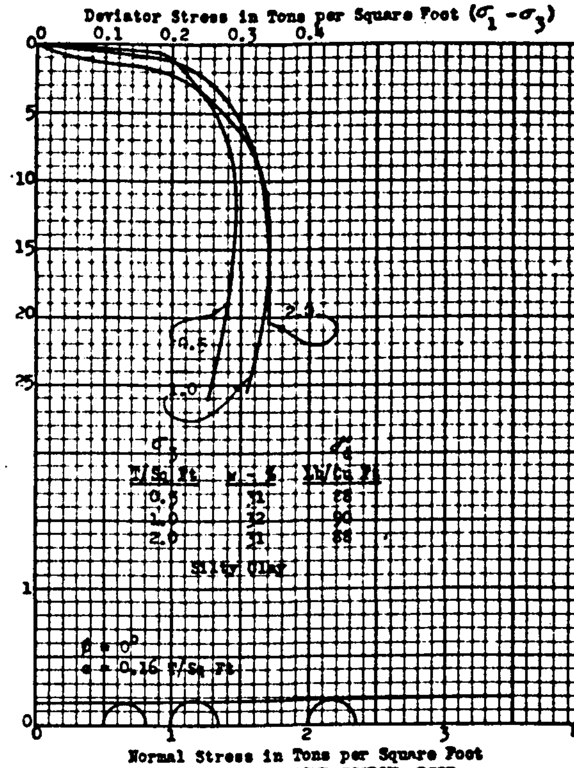
COMBINED MORGANZA FLOODWAY CONTROL STRUCTURE
TEXAS & PACIFIC R.R. AND LA. STATE HIGHWAY NO. 30

TRIAXIAL COMPRESSION TESTS ON BORROW MATERIAL

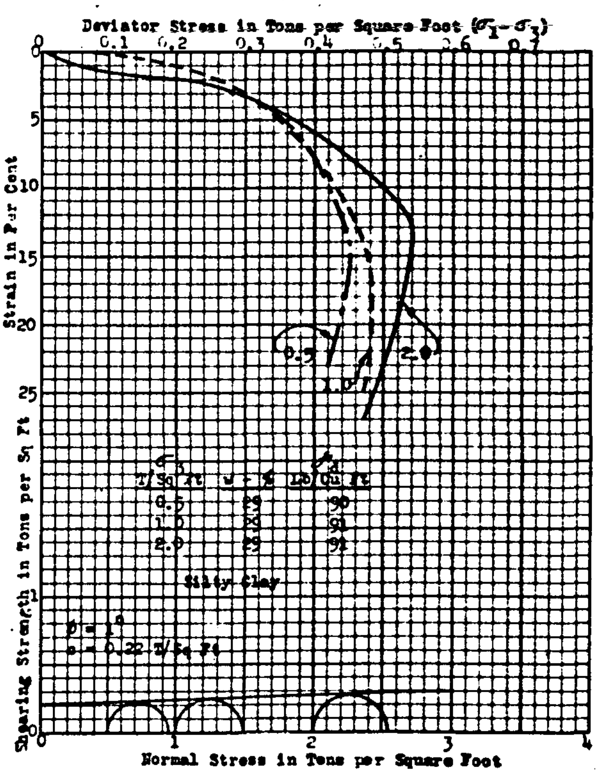
NOV 1948 FILE 3028



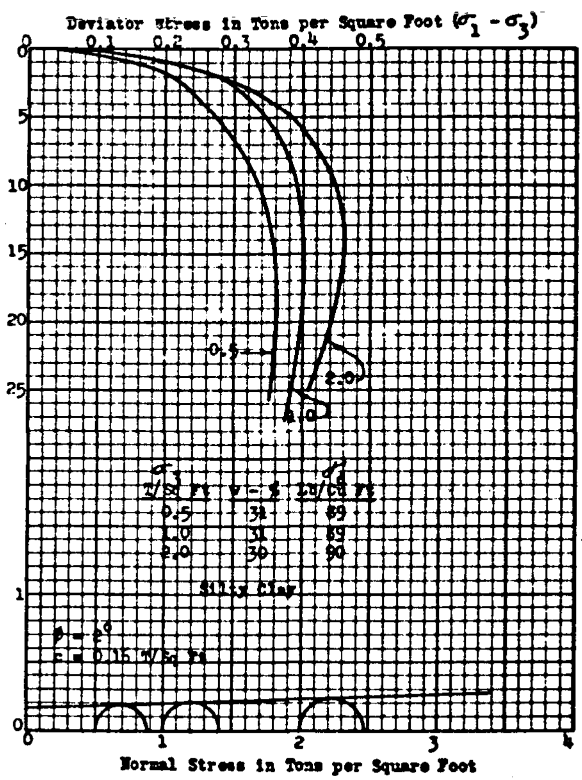
QUICK TRIAXIAL COMPRESSION TEST
Sack No. 4



QUICK TRIAXIAL COMPRESSION TEST
Sack No. 4



QUICK TRIAXIAL COMPRESSION TEST
Sack No. 6

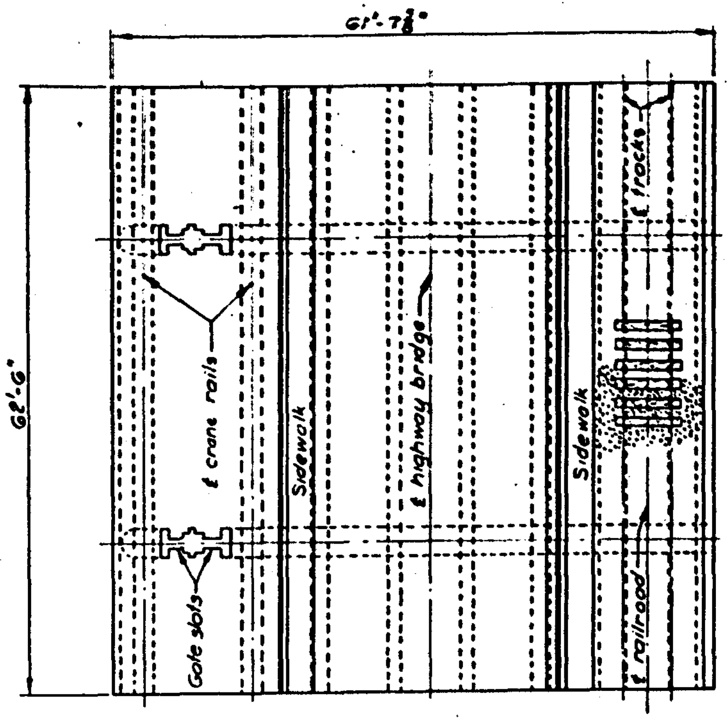


QUICK TRIAXIAL COMPRESSION TEST
Sack No. 7

COMBINED MORGANZA FLOODWAY CONTROL STRUCTURE
TEXAS & PACIFIC R.R. AND LA. STATE HIGHWAY NO. 30

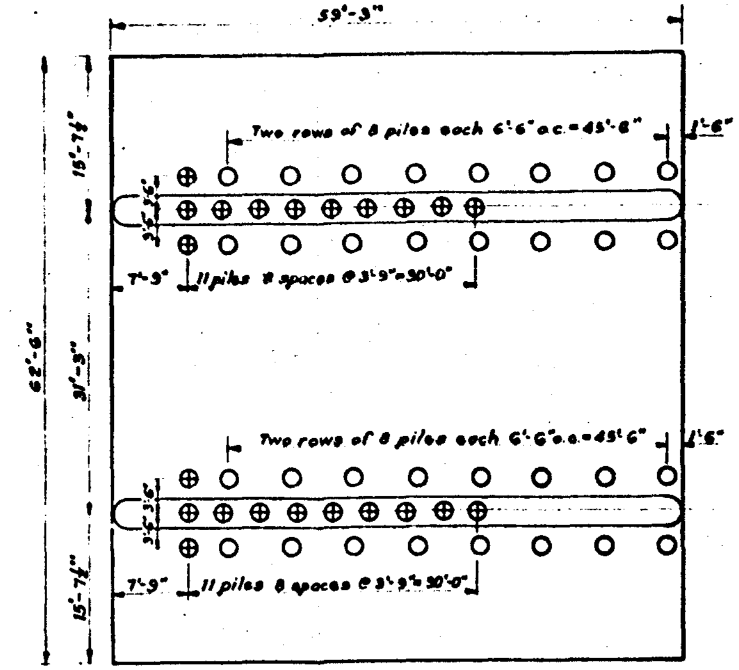
TRIAXIAL COMPRESSION TESTS ON BORROW MATERIAL

NOV 1948 FILE 3028



PLAN OF ONE MONOLITH

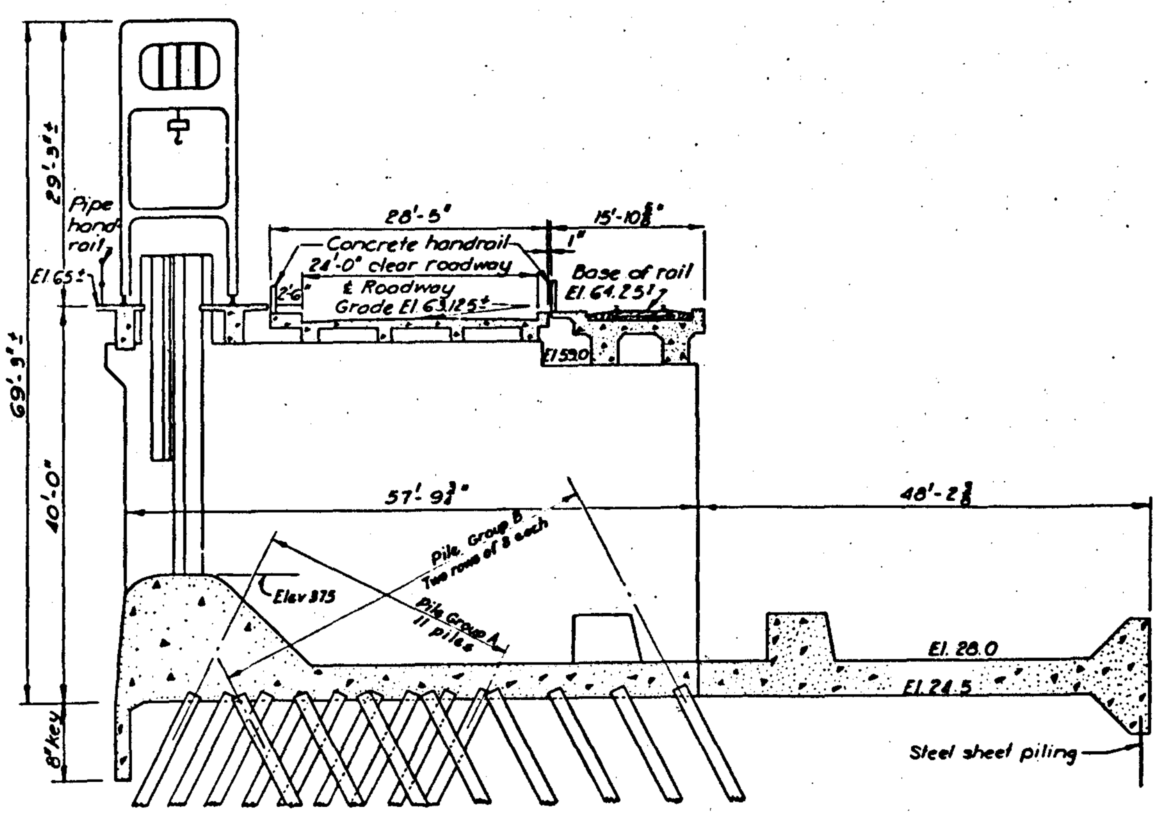
Stilling basin not shown



PILING LAYOUT PLAN

⊕ GROUP A

O GROUP B



TYPICAL SECTION

Note:

This piling layout is based on 18-in. piles loaded to 100 tons compression and 25 tons tension.

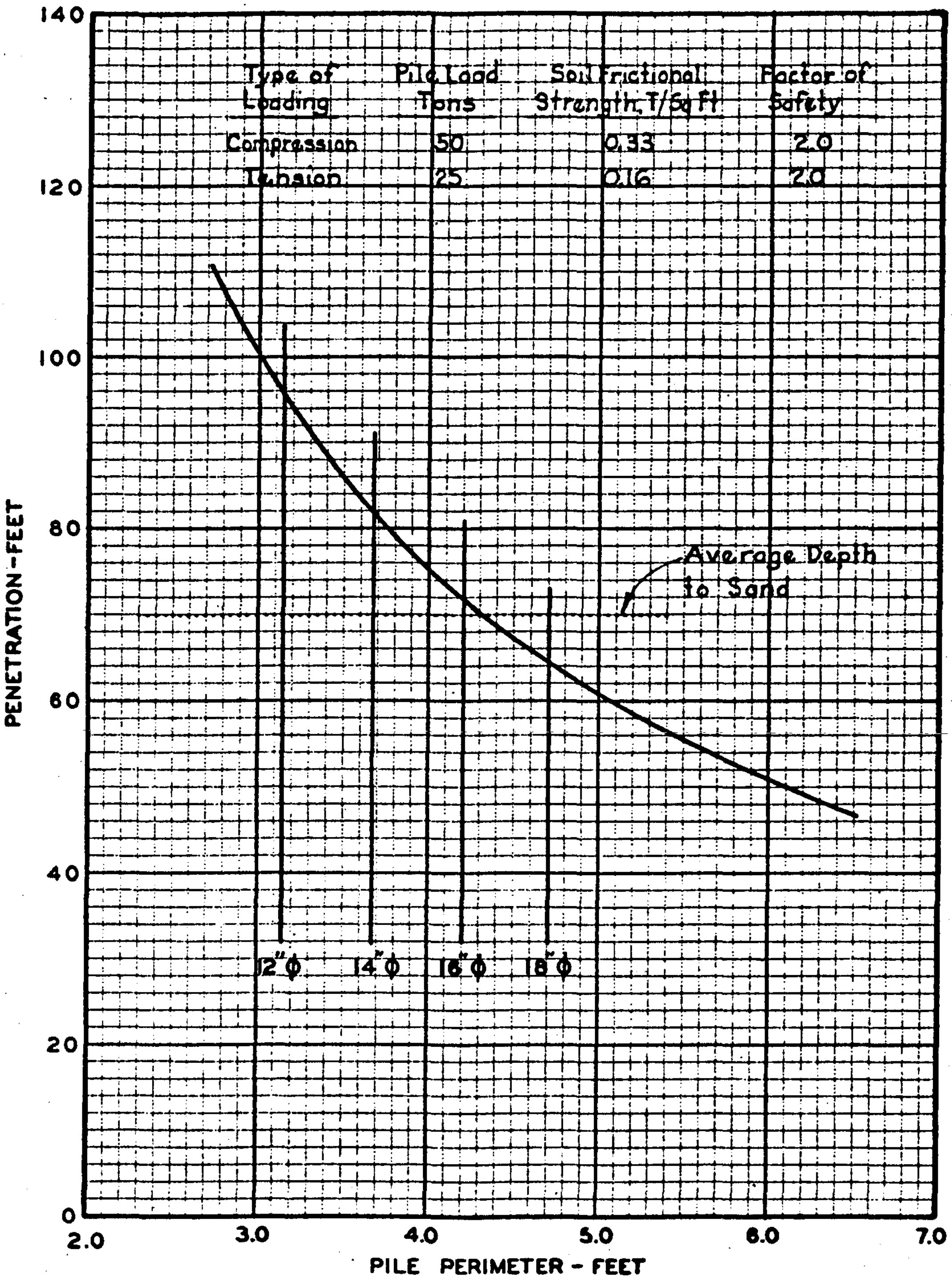
Final size and spacing of piles to be based on results of pile tests.

COMBINED MORGANZA FLOODWAY CONTROL STRUCTURE
TEXAS & PACIFIC R.R. AND LA. STATE HIGHWAY NO. 30

DETAILS OF STRUCTURE

SCALE: AS SHOWN

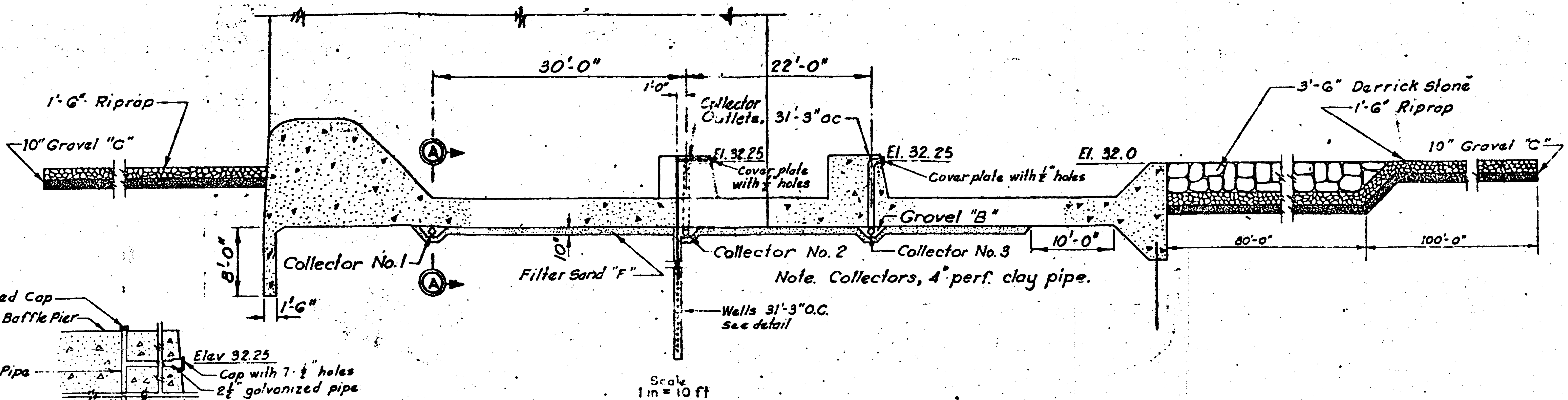
NOV 1948 FILE 3028

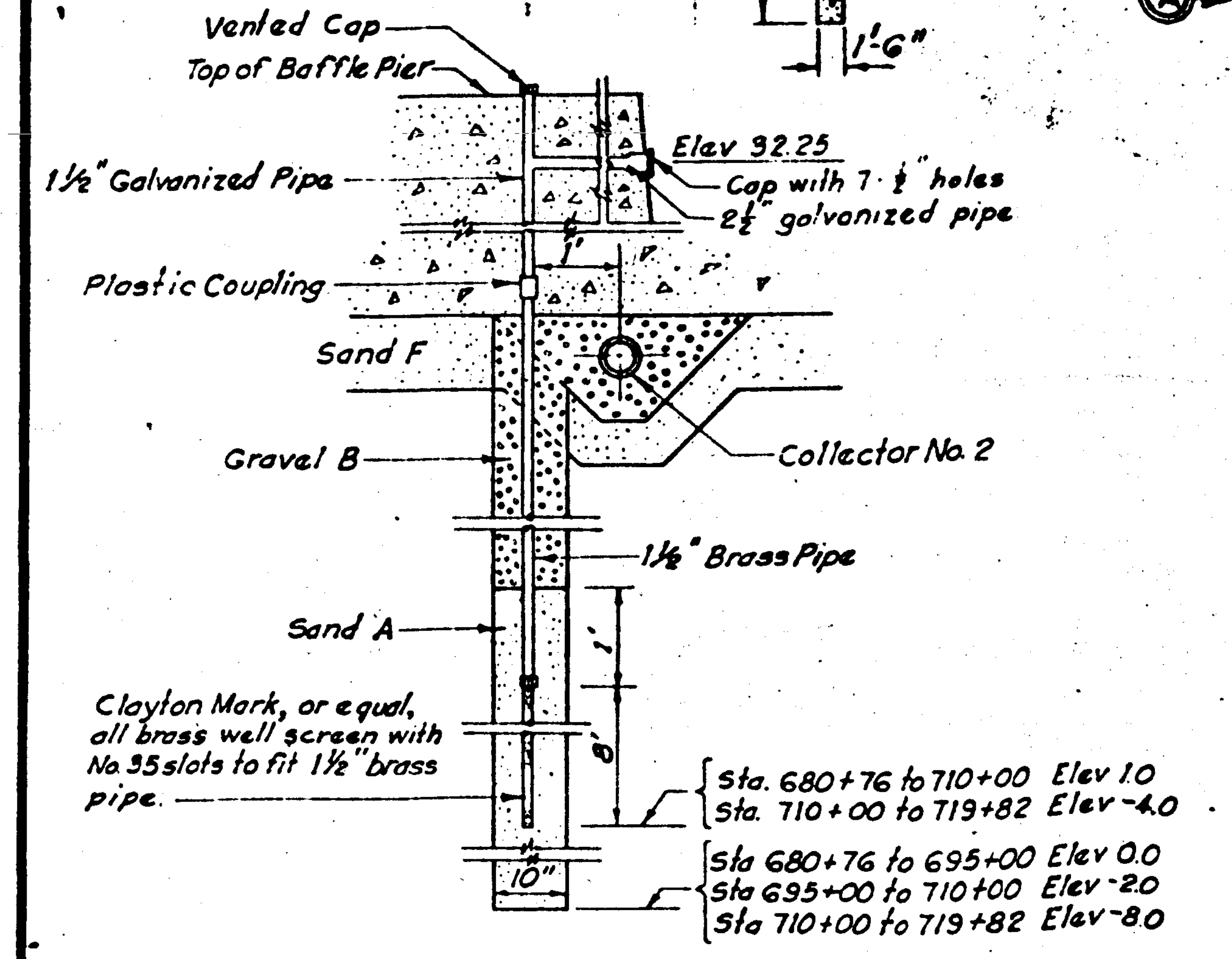


COMBINED MORGANZA FLOODWAY CONTROL STRUCTURE

DESIGN OF FRICTION PILES

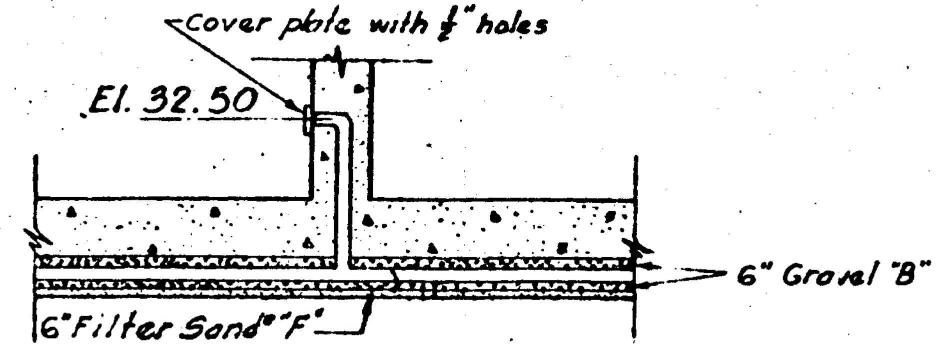
NOV 1948 FILE 3028





WELL DETAIL

1/2 in. = 1 ft



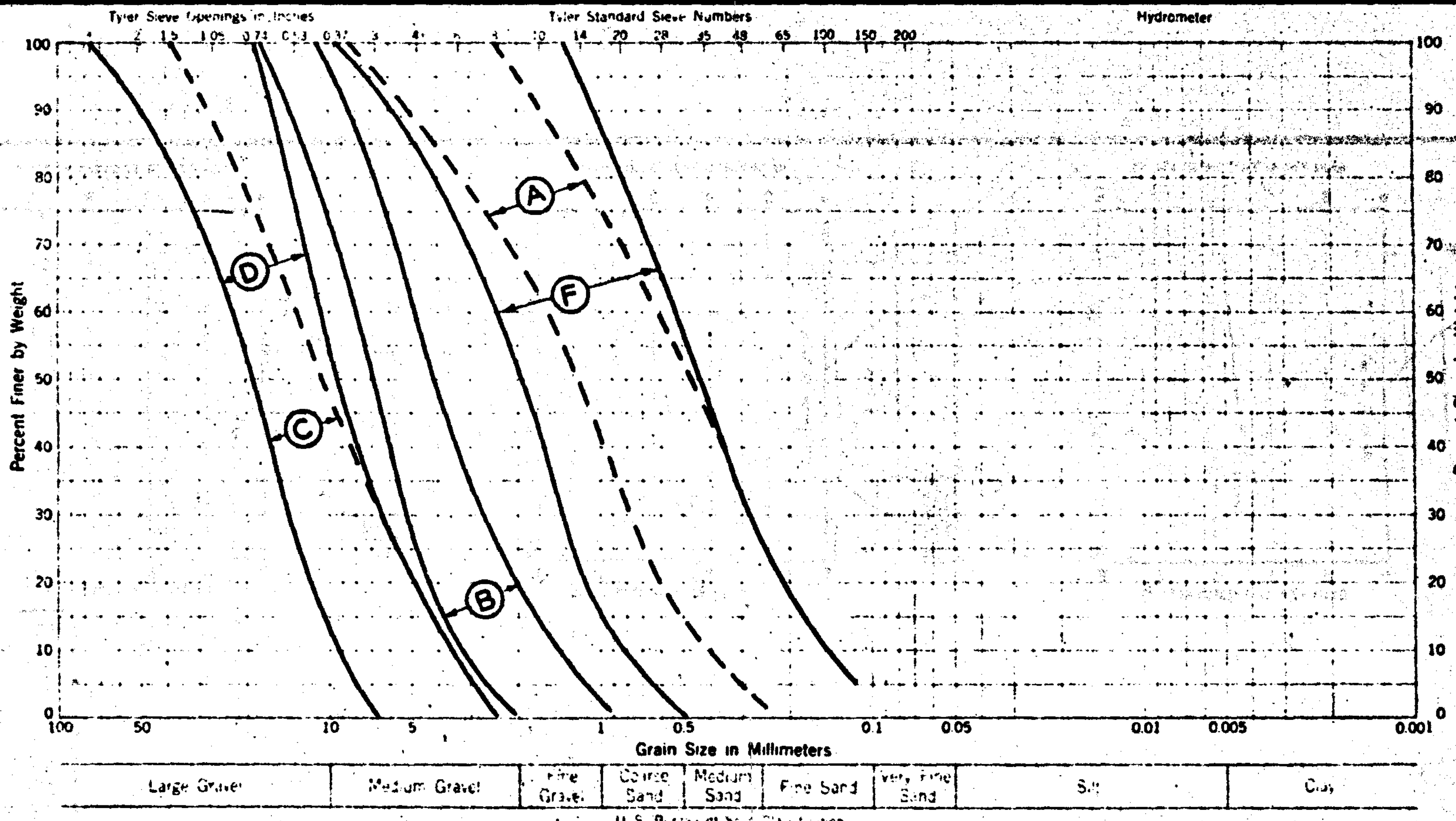
SECTION A-A

COMBINED MORGANZA FLOODWAY CONTROL STRUCTURE
TEXAS AND PACIFIC R.R. AND LA. STATE HIGHWAY NO. 30

DRAINAGE DETAILS

SCALES: AS SHOWN

NOV 1948 FILE 3028



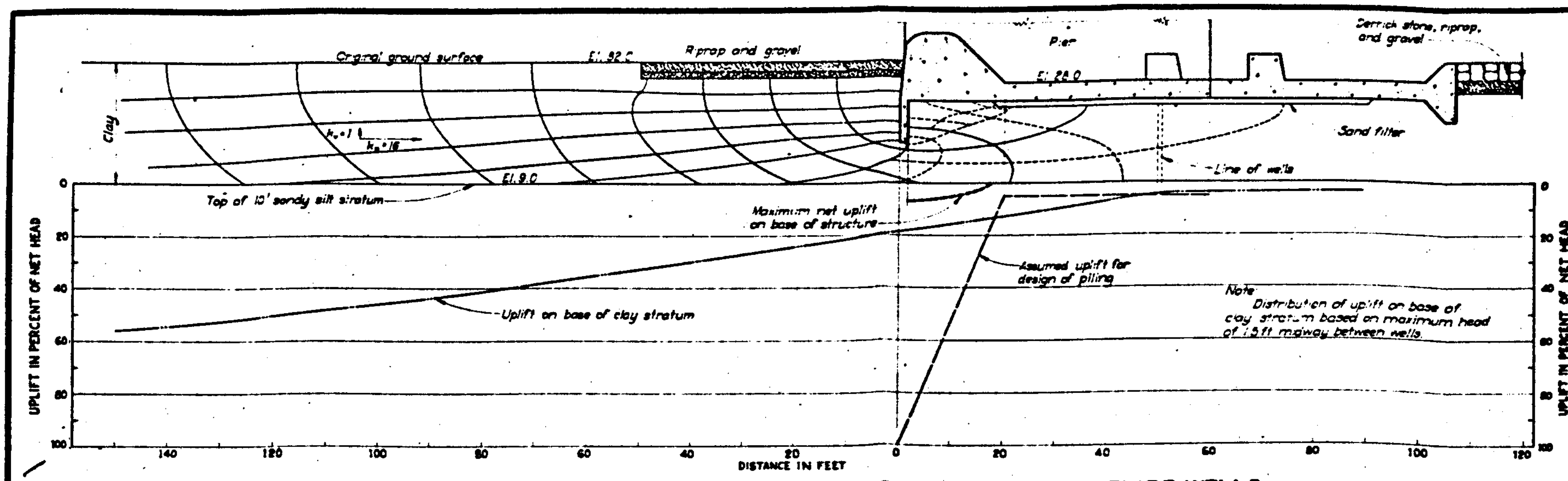
Combined Morganza Floodway Control Structure

FILTER MATERIALS

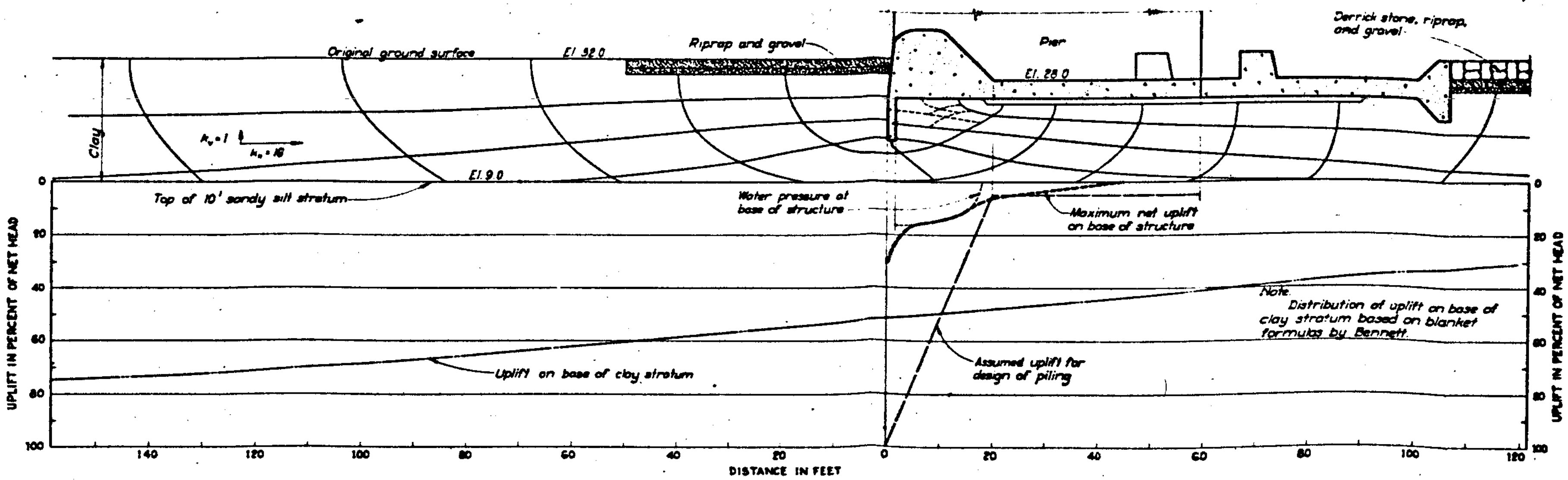
May 1949

File 3028

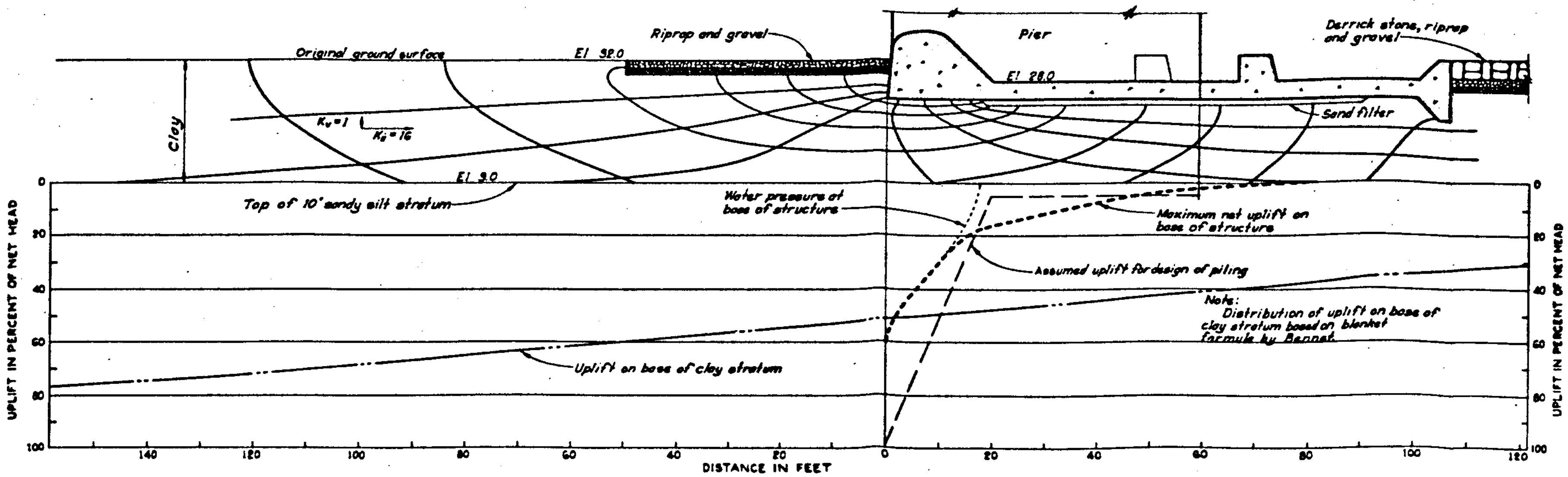
PLATE 26



FLOW NET AND UPLIFT PRESSURE WITH SAND BLANKET AND RELIEF WELLS



FLOW NET AND UPLIFT PRESSURE WITH SAND BLANKET ONLY



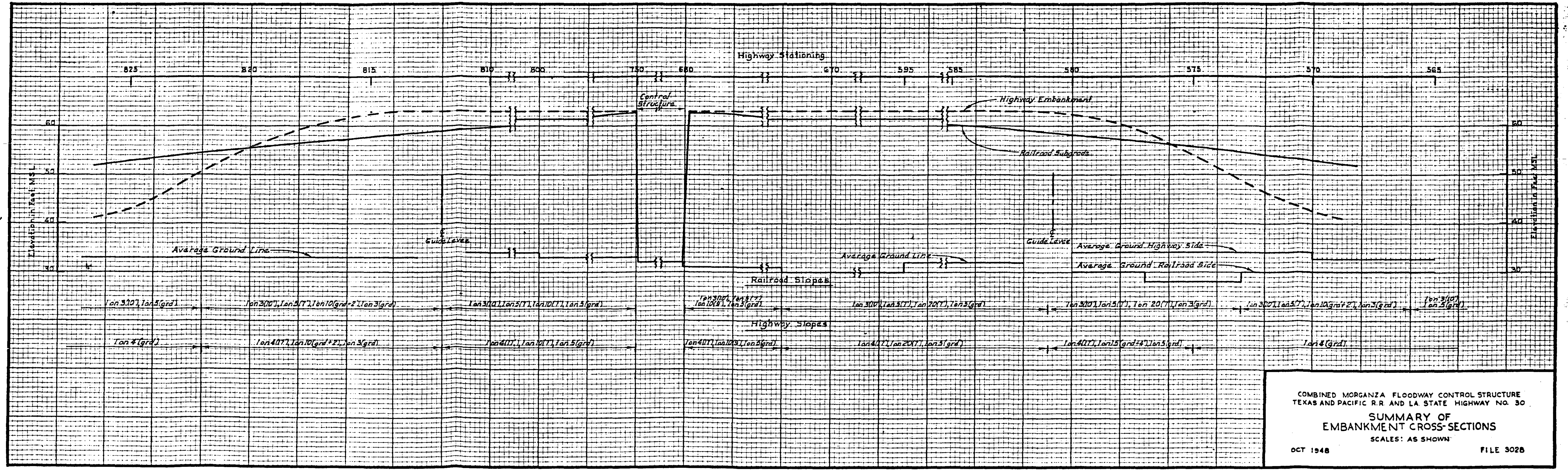
FLOW NET AND UPLIFT PRESSURE WITH SAND BLANKET AND NO CUTOFF

COMBINED MORGANZA FLOODWAY CONTROL STRUCTURE
TEXAS & PACIFIC R.R. AND LA. STATE HIGHWAY NO. 30

FLOW NETS AND UPLIFT PRESSURE

SCALE: AS SHOWN

OCT 1948 FILE 3028



COMBINED MORGANZA FLOODWAY CONTROL STRUCTURE
TEXAS AND PACIFIC R.R AND LA. STATE HIGHWAY NO. 30

SUMMARY OF
EMBANKMENT CROSS-SECTIONS

SCALES: AS SHOWN

OCT 1948 FILE 3028

PLATE 28

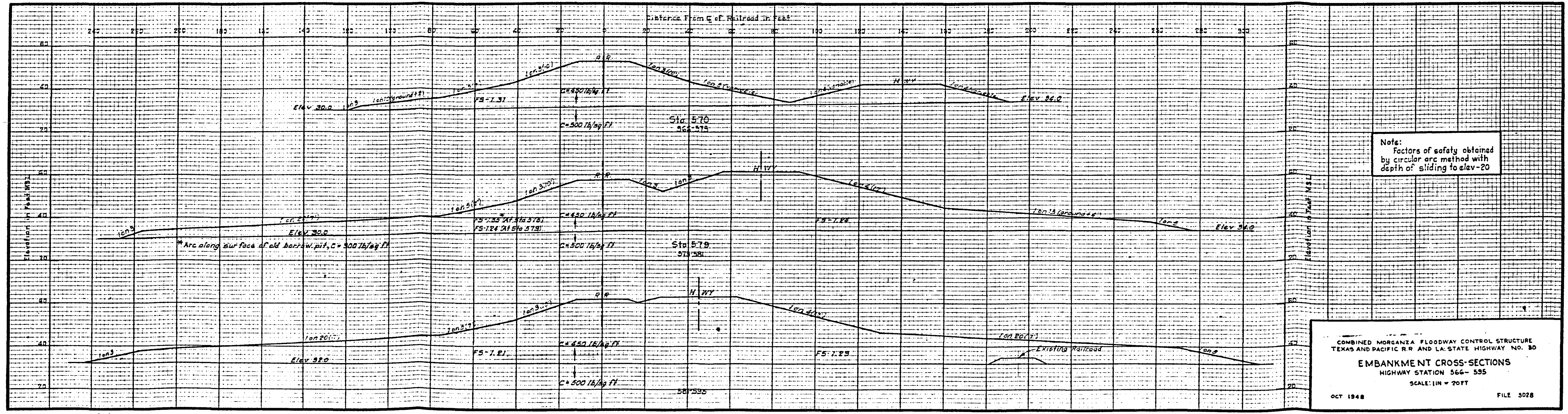
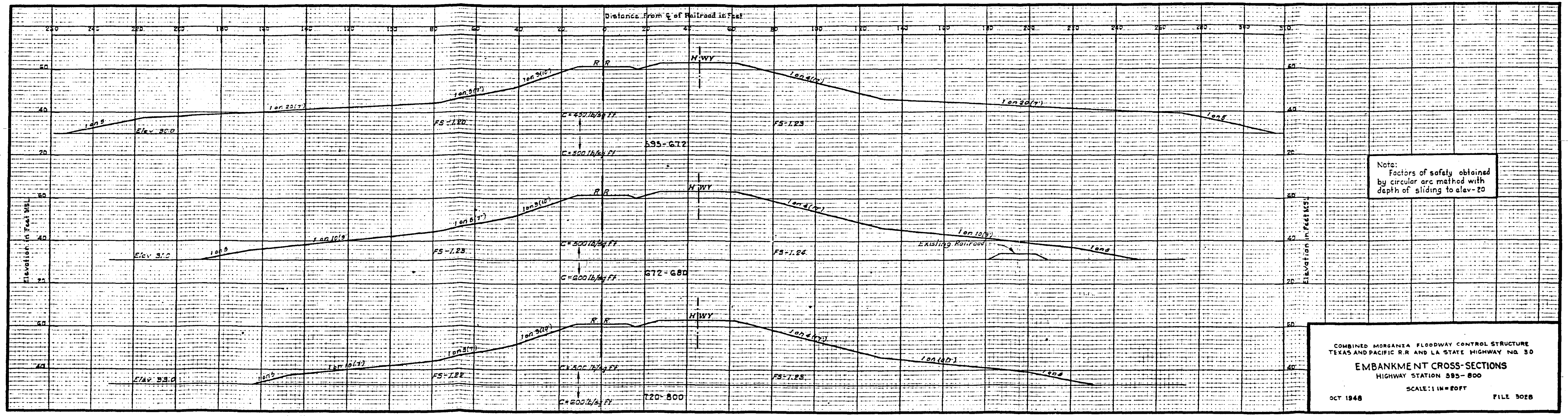

Distance From ℄ of Railroad in Feet
Elevation in Feet MSL
R.R.
HWY
Elev 30.0
Elev 34.0
FS-1.31
C=450 lb/sq ft
C=500 lb/sq ft
Sta 570
FS-1.35* (At Sta 576)
FS-1.24 (At Sta 579)
FS-1.24
Elev 30.0
Elev 34.0
*Arc along surface of old borrow pit, C=300 lb/sq ft
Sta 579
FS-1.21
FS-1.23
Elev 32.0
Existing Railroad
Note:
Factors of safety obtained by circular arc method with depth of sliding to elev-20
COMBINED MORGANZA FLOODWAY CONTROL STRUCTURE
TEXAS AND PACIFIC R.R. AND LA. STATE HIGHWAY NO. 30
EMBANKMENT CROSS-SECTIONS
HIGHWAY STATION 566-595
SCALE: 1 IN = 20 FT
OCT 1948
FILE 3028

PLATE 29


Distance from ℄ of Railroad in Feet
Elevation in Feet M.S.L.
R R
H WY
1 on 3 (10')
1 on 3 (7')
1 on 20 (7')
1 on 5
1 on 4 (7')
1 on 30 (7')
Elev 30.0
FS-1.20
FS-1.23
C=450 lb/sq ft
C=500 lb/sq ft
595-672
1 on 10 (9')
Elev 31.0
FS-1.23
FS-1.24
C=500 lb/sq ft
C=600 lb/sq ft
672-680
Existing Railroad
1 on 10 (7')
Elev 33.0
FS-1.22
FS-1.25
C=500 lb/sq ft
C=600 lb/sq ft
720-800
Note:
Factors of safety obtained by circular arc method with depth of sliding to elev-20
COMBINED MORGANZA FLOODWAY CONTROL STRUCTURE
TEXAS AND PACIFIC R.R AND LA STATE HIGHWAY NO. 30
EMBANKMENT CROSS-SECTIONS
HIGHWAY STATION 595-800
SCALE: 1 IN=20FT
OCT 1948
FILE 3028

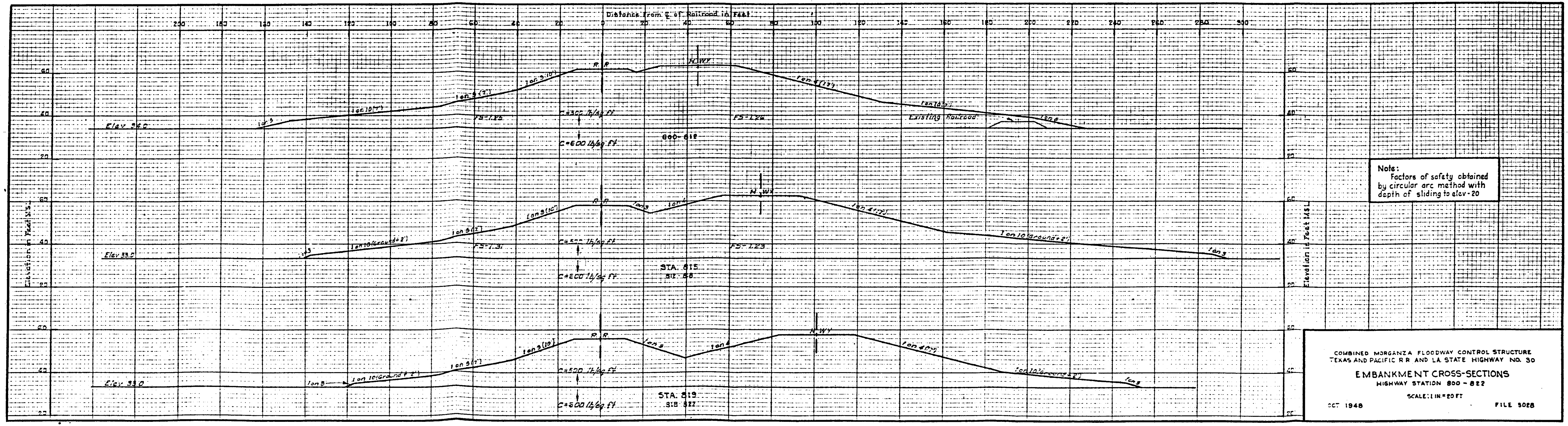
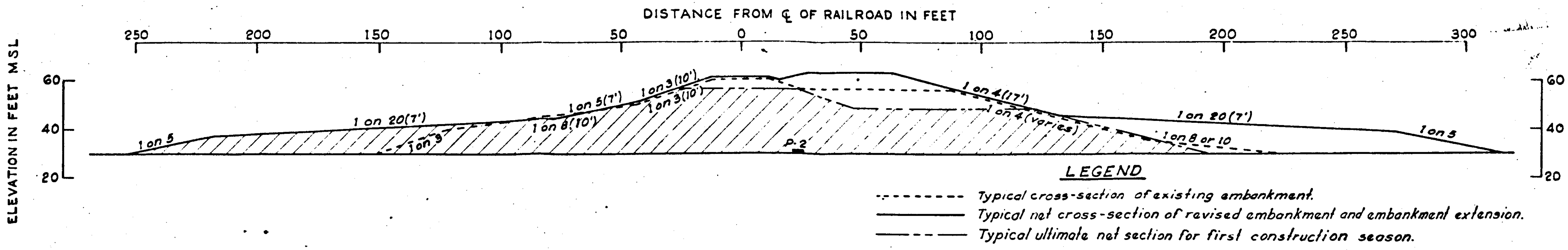
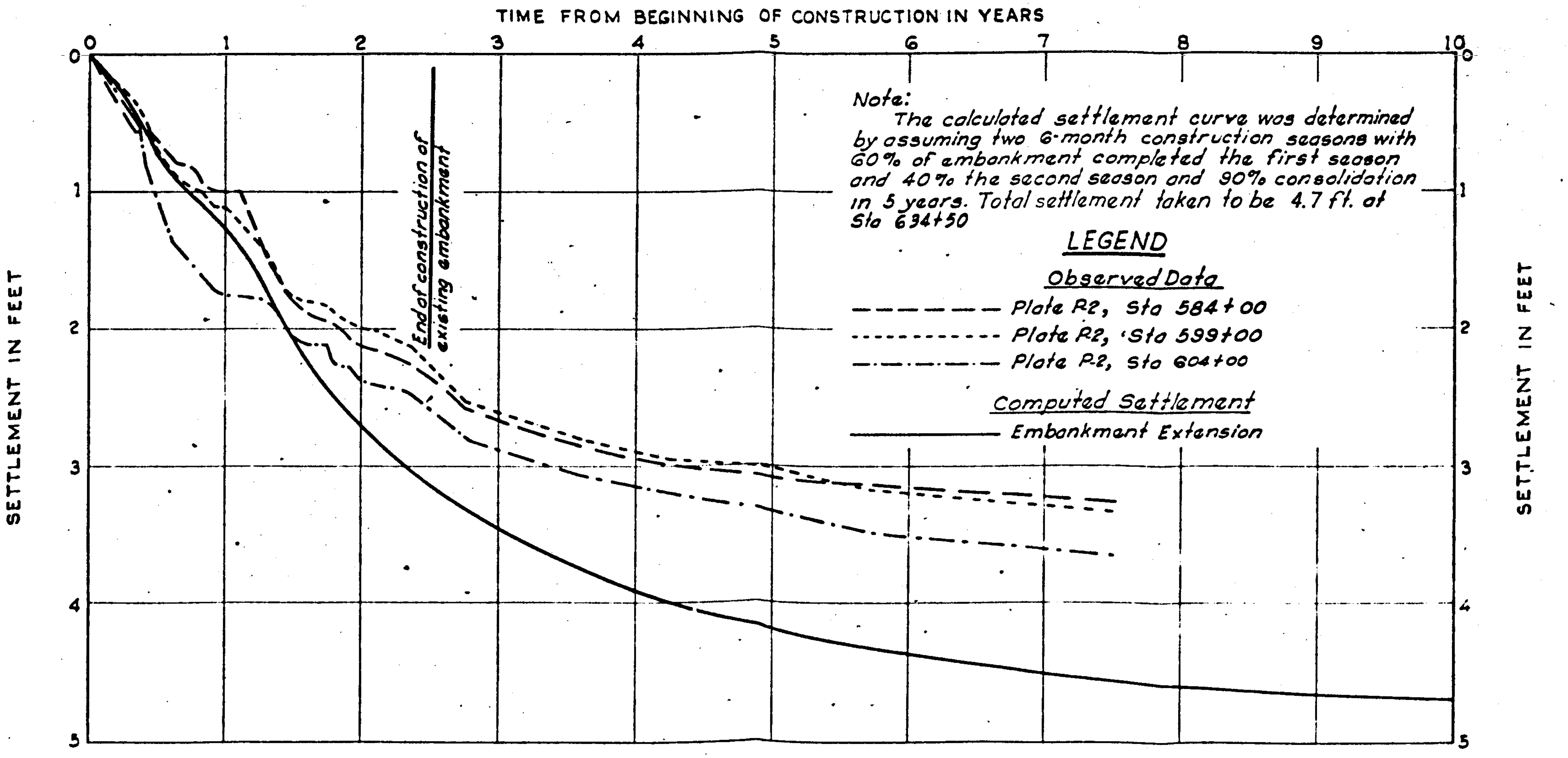

Distance from ℄ of Railroad in Feet
Elevation in Feet MSL
R.R.
H.WY.
Elev 34.0
FS-1.25
FS-1.26
C=500 lb/sq ft
C=600 lb/sq ft
800-812
Existing Railroad
Elev 33.0
FS-1.31
FS-1.29
STA. 815
812-818
STA. 819
818-822
Note:
Factors of safety obtained by circular arc method with depth of sliding to elev-20
COMBINED MORGANZA FLOODWAY CONTROL STRUCTURE
TEXAS AND PACIFIC R R AND LA STATE HIGHWAY NO. 30
EMBANKMENT CROSS-SECTIONS
HIGHWAY STATION 800 - 822
SCALE: 1 IN.=20 FT
OCT 1948
FILE 3028

PLATE 31



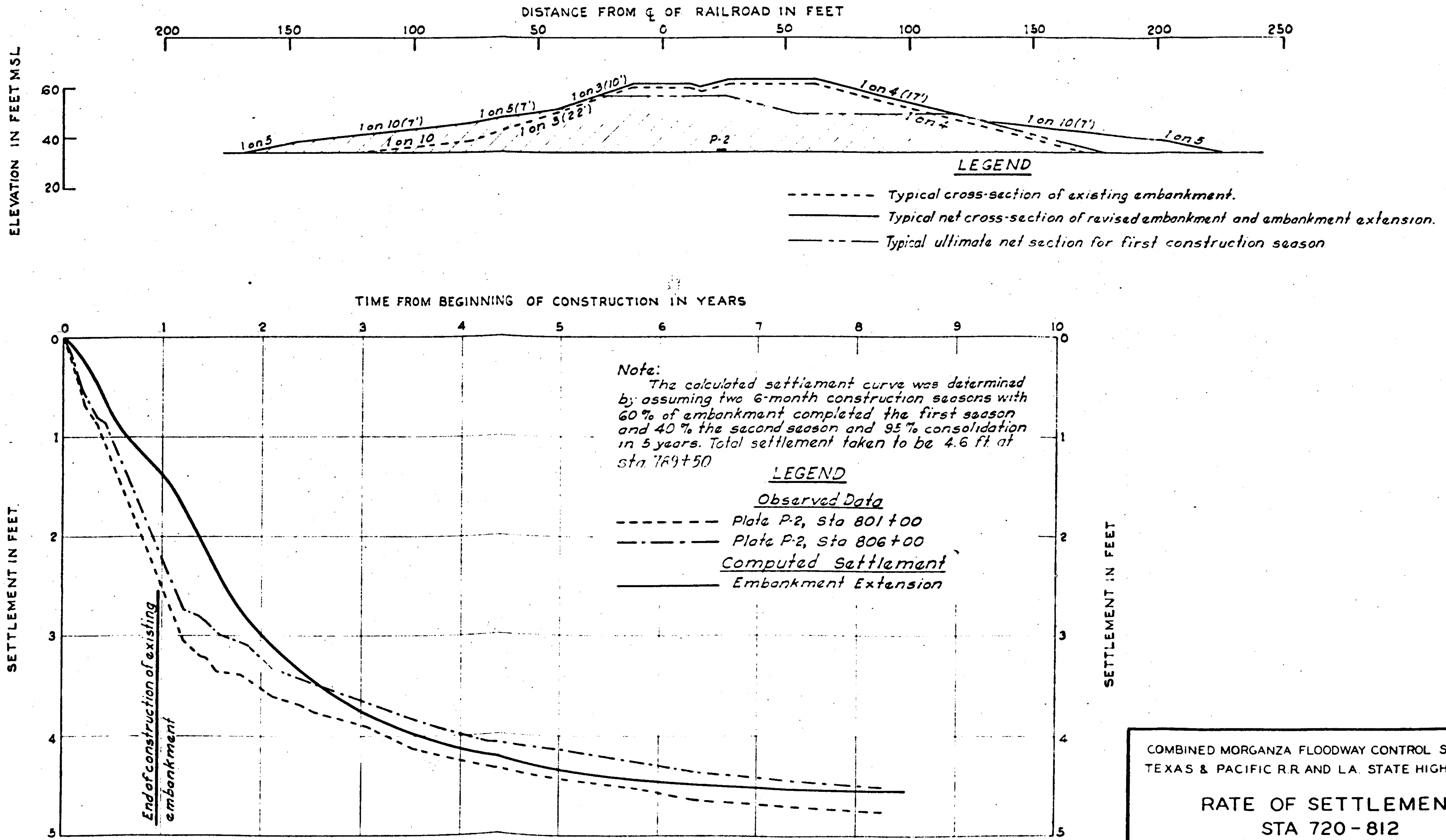


COMBINED MORGANZA FLOODWAY CONTROL STRUCTURE
TEXAS & PACIFIC R.R. AND LA. STATE HIGHWAY NO. 30

RATE OF SETTLEMENT
STA 581 - 680

SCALE: AS SHOWN

OCT 1948 FILE 3028

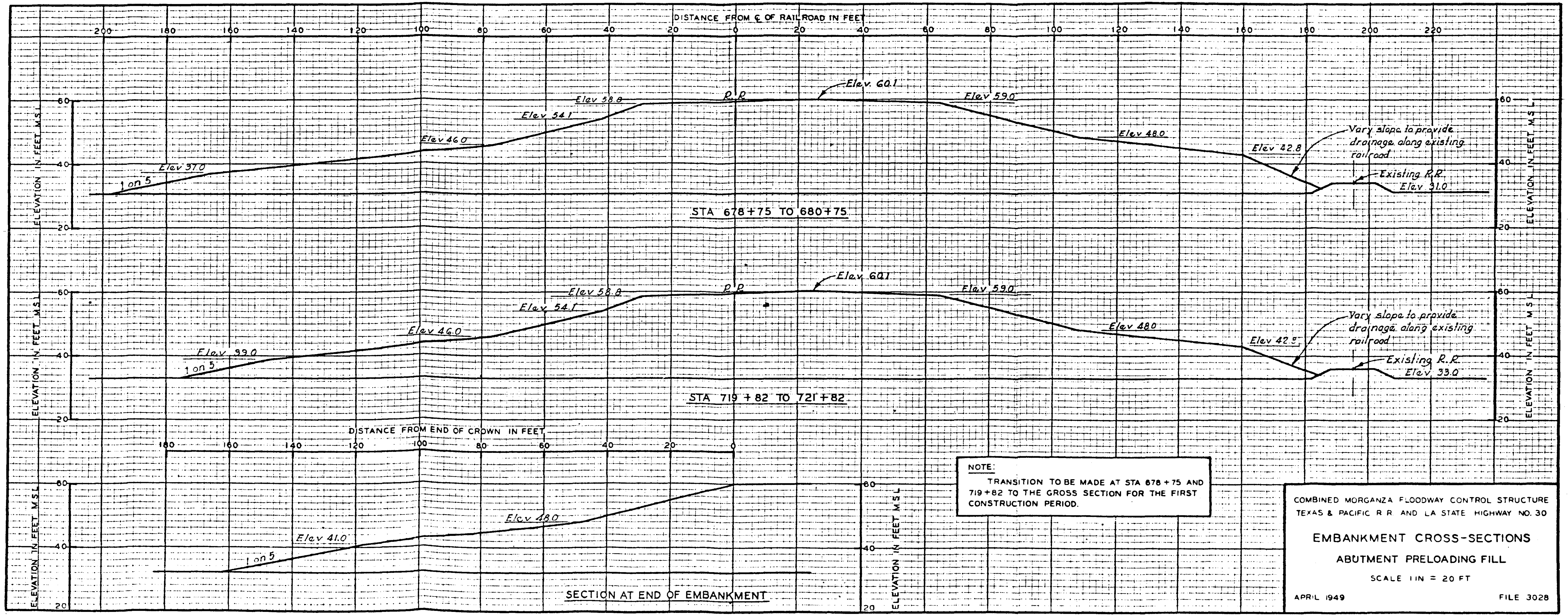


COMBINED MORGANZA FLOODWAY CONTROL STRUCTURE
TEXAS & PACIFIC R.R AND LA. STATE HIGHWAY NO. 30

RATE OF SETTLEMENT
STA 720 - 812

SCALE: AS SHOWN

OCT 1948 FILE 3028



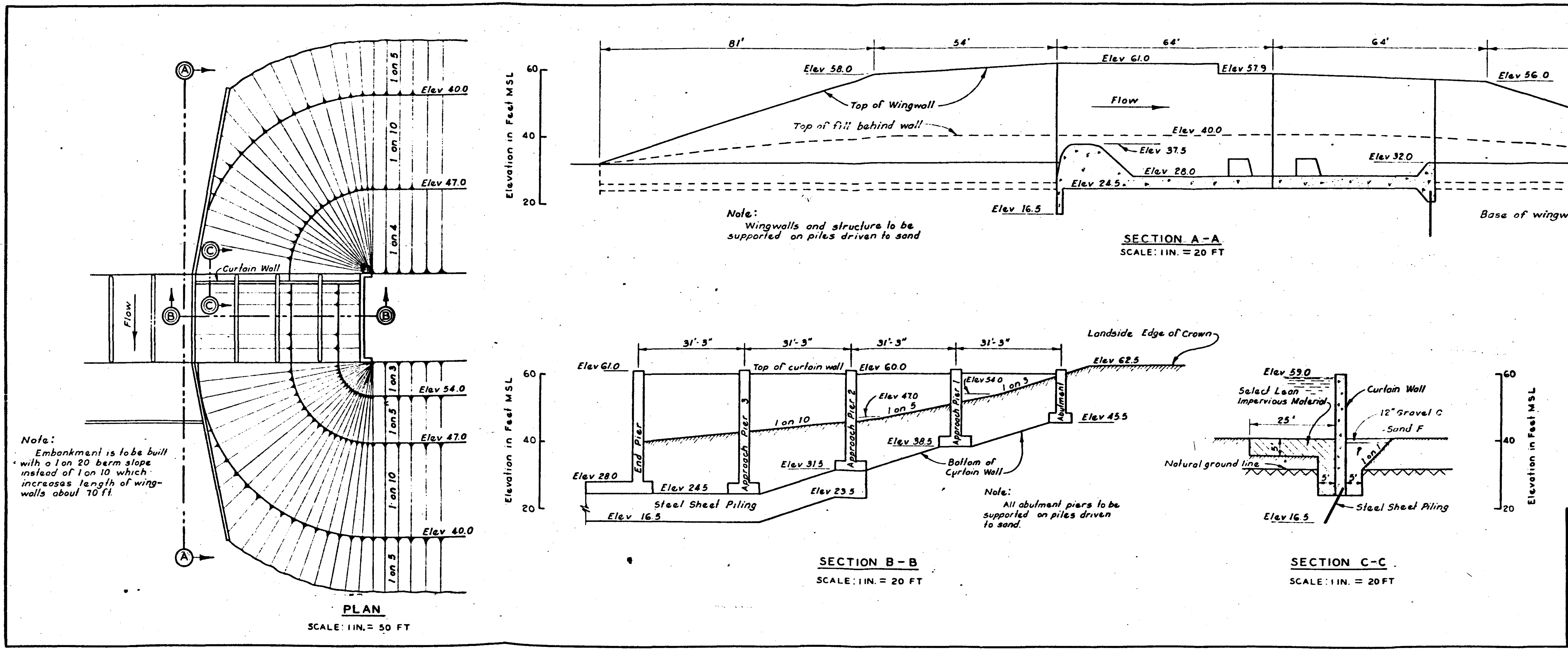
NOTE:
TRANSITION TO BE MADE AT STA 678+75 AND 719+82 TO THE CROSS SECTION FOR THE FIRST CONSTRUCTION PERIOD.

COMBINED MORGANZA FLOODWAY CONTROL STRUCTURE
TEXAS & PACIFIC R.R. AND LA STATE HIGHWAY NO. 30

EMBANKMENT CROSS-SECTIONS
ABUTMENT PRELOADING FILL

SCALE 1 IN = 20 FT

APRIL 1949 FILE 3028



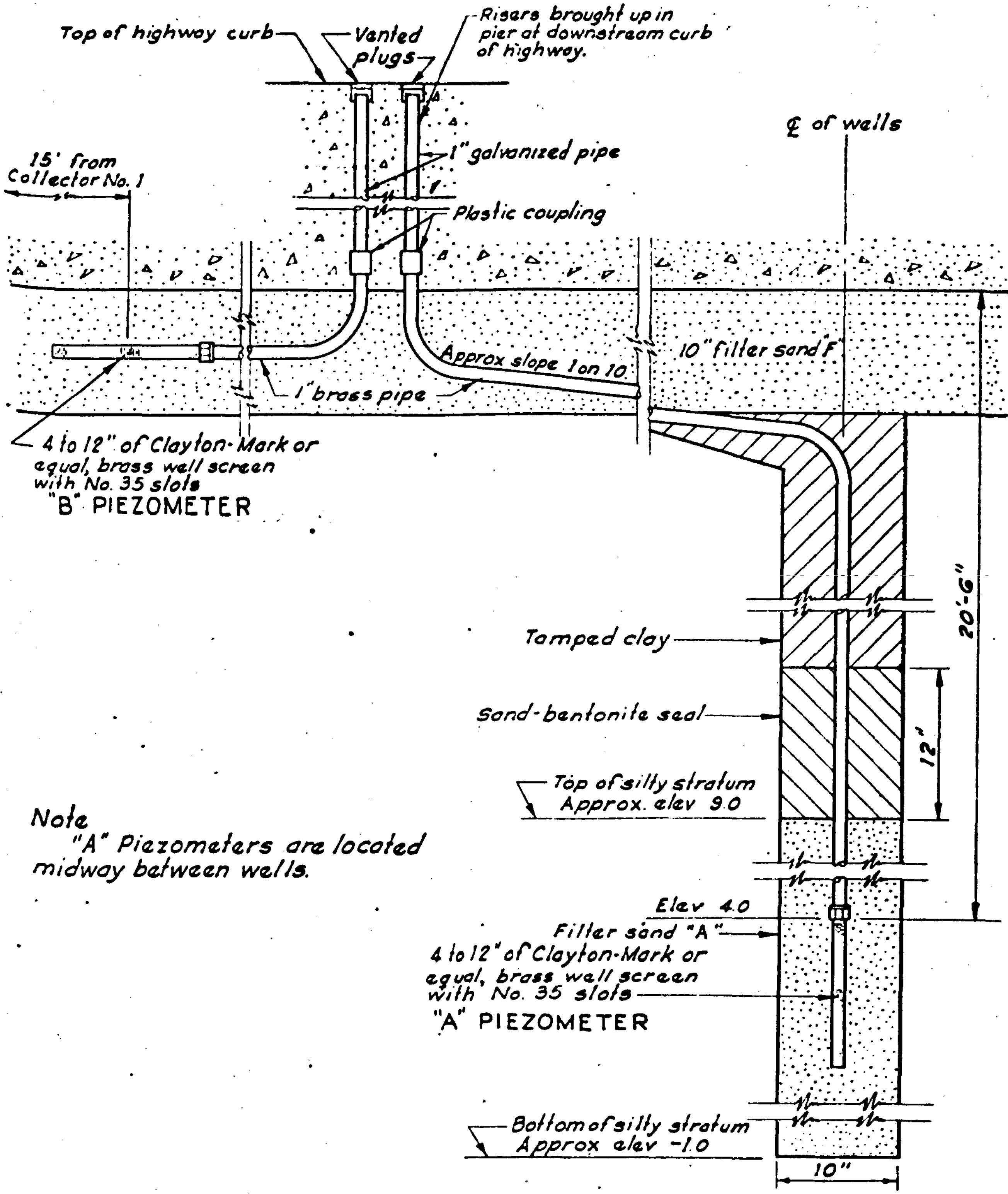
COMBINED MORGANZA FLOODWAY CONTROL STRUCTURE

TEXAS & PACIFIC R.R. AND LA. STATE HIGHWAY NO 30

DETAILS OF ABUTMENTS

SCALE: AS SHOWN

APRIL 1949 FILE 3028

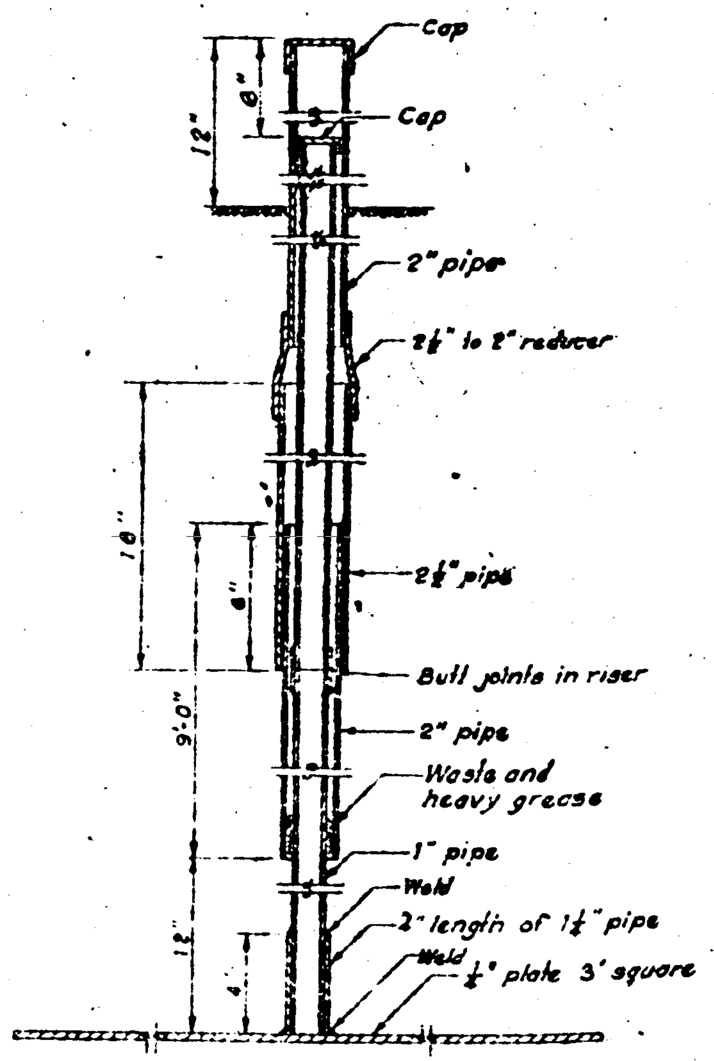


COMBINED MORGANZA FLOODWAY CONTROL STRUCTURE
TEXAS & PACIFIC R.R. AND LA. STATE HIGHWAY NO. 30

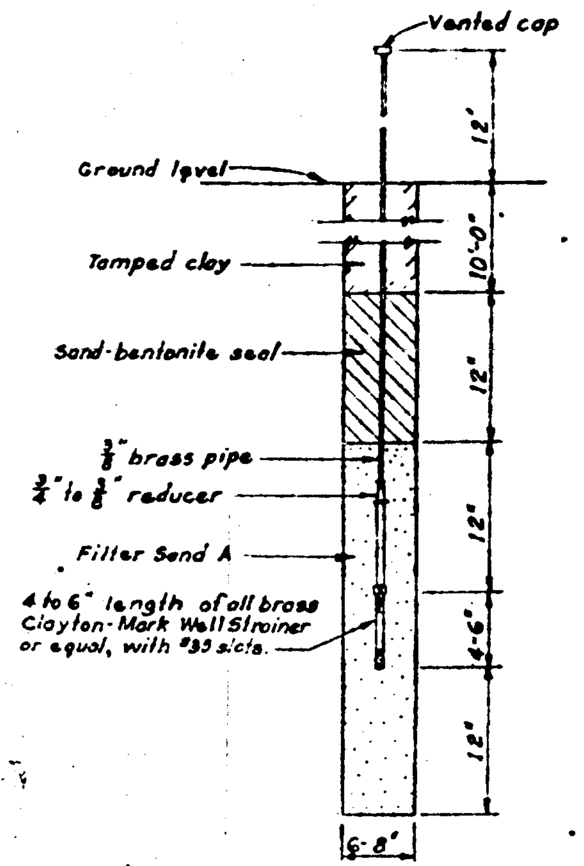
DETAIL OF PIEZOMETERS A AND B

SCALE: 1 IN = 1 FT

NOV 1948 FILE 3028

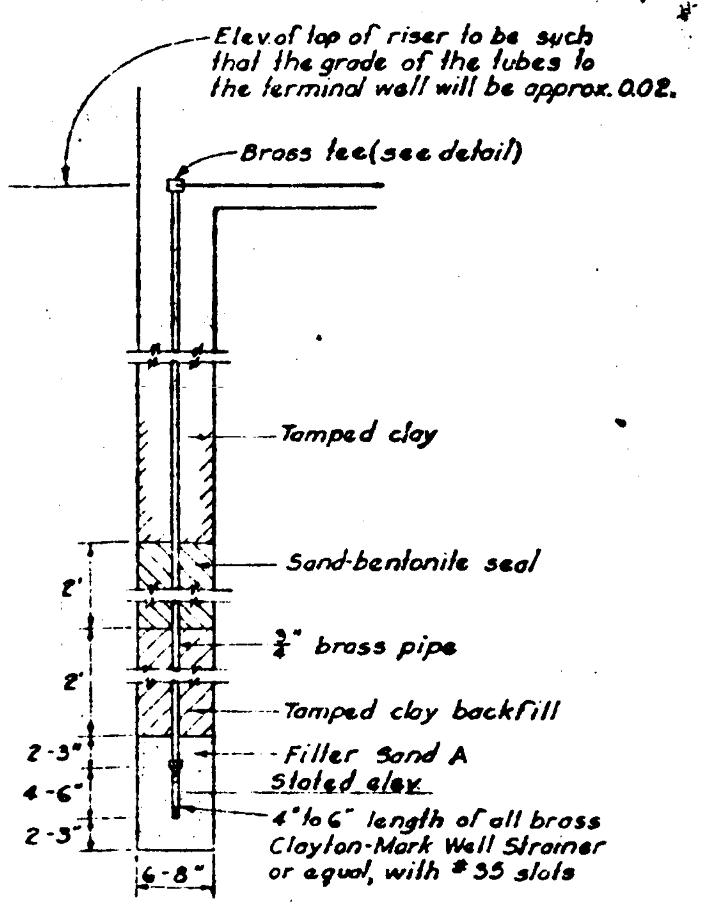


EMBANKMENT SETTLEMENT PLATE



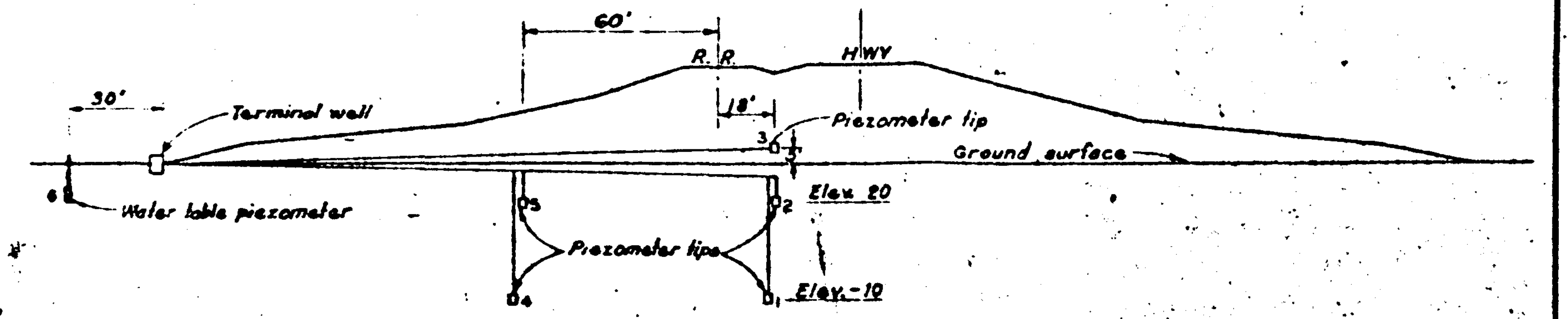
WATER TABLE PIEZOMETER

SCALE: 1 IN. = 1 FT



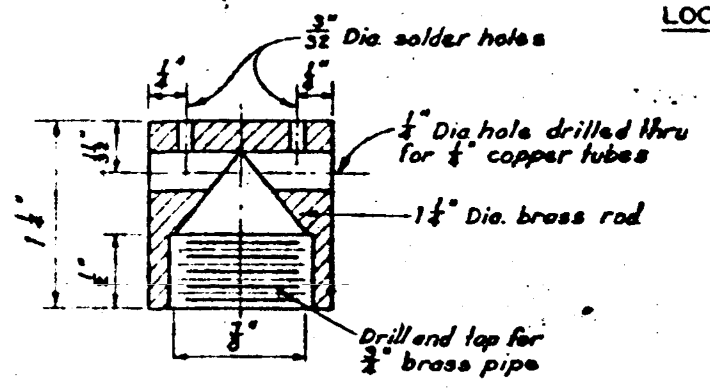
FOUNDATION PIEZOMETER

SCALE: 1 IN. = 1 FT



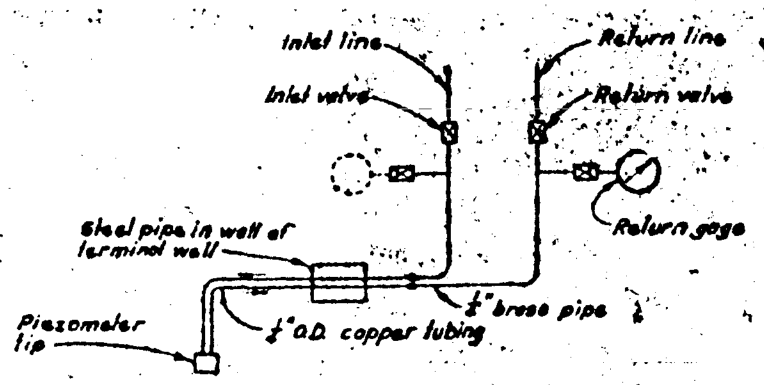
LOCATION OF PIEZOMETER TIPS

SCALE: 1 IN = 40 FT

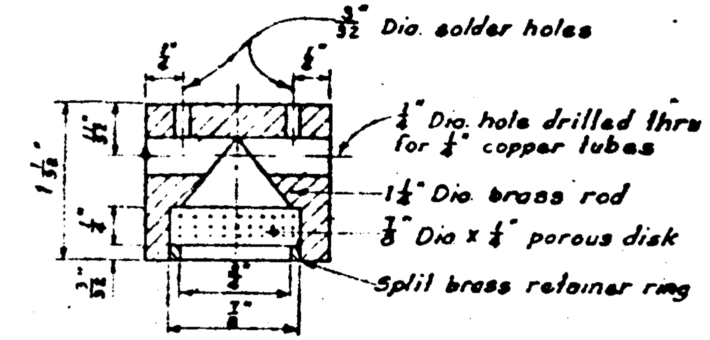


DETAIL OF BRASS TEE

SCALE: 1 IN. = 1 IN.



SCHEMATIC PIPING DIAGRAM



EMBANKMENT PIEZOMETER

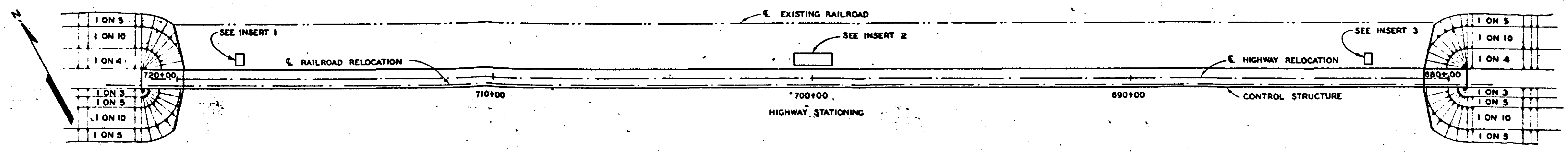
SCALE: 1 IN. = 1 IN.

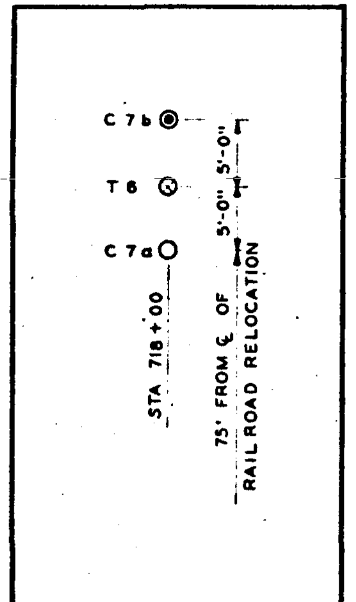
COMBINED MORGANZA FLOODWAY CONTROL STRUCTURE
TEXAS AND PACIFIC R.R. AND LA. STATE HIGHWAY NO. 30

ENGINEERING MEASUREMENT DEVICES

SCALES: AS SHOWN

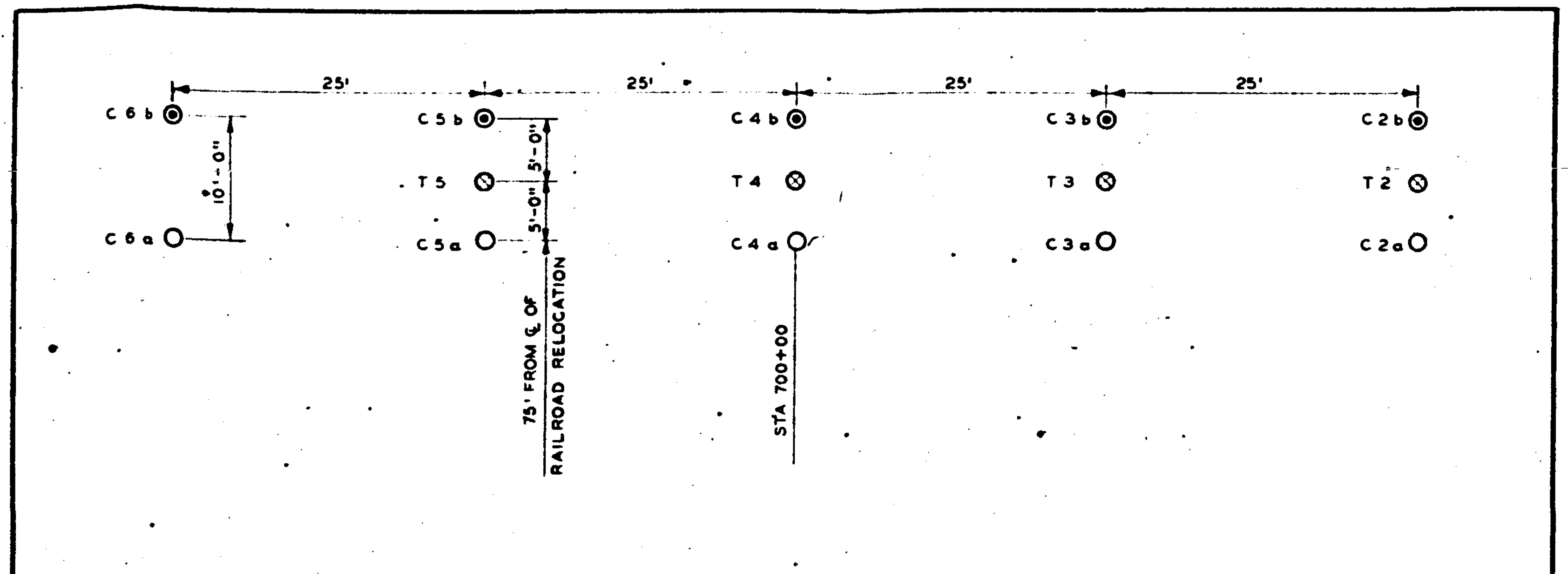
APRIL 1948 FILE 3028





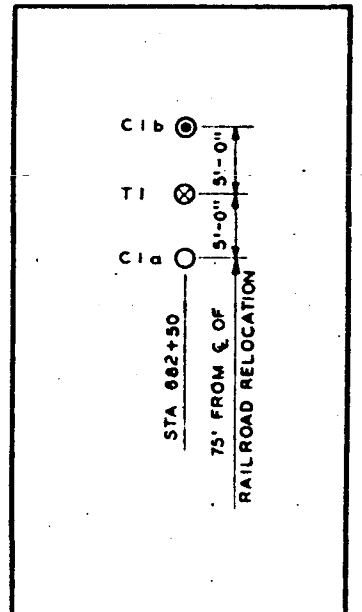
SCALE: 1 IN.= 10 FT

INSERT 1



SCALE: 1 IN.= 10 FT

INSERT 2



SCALE: 1 IN.= 10 FT

INSERT 3

LEGEND

- ⊗ TENSION PILES
- ○ COMPRESSION "a" PILES (NOT TO SAND)
- ◉ COMPRESSION "b" PILES (INTO SAND)

COMPRESSION PILES

PILE PAIR	TYPE OF PILE	DIAM. OF PILE IN.
C 1	PIPE	24
C 2	UNIFORM TAPER	8 (TIP)
C 3	CONSTANT SECTION	12
C 4	PIPE	18
C 5	PIPE	24
C 6	PIPE	30
C 7	PIPE	24

TENSION PILES

PILE	TYPE OF PILE	DIAM. OF PILE IN.
T 1	PIPE	24
T 2	UNIFORM TAPER	8 (TIP)
T 3	CONSTANT SECTION	12
T 4	PIPE	18
T 5	PIPE	24
T 6	PIPE	24

COMBINED MORGANZA FLOODWAY CONTROL STRUCTURE

TEXAS & PACIFIC R. R. AND LA. STATE HIGHWAY NO. 30

PLAN OF PILE TESTS

SCALE: 1 IN.= 200 FT

NOVEMBER 1948 FILE 3028

APPENDIX A

PILE LOADING TEST PROGRAM

Introduction

1. It is recommended that pile tests be made at the site of the control structure as a possible aid to a more economical design of the foundation. The test program should include compression and tension piles to determine the size of the compression piles that would be required and the allowable tension loads. The original design of the piling beneath the structure included compression piles to sand, loaded to 47 tons, and short tension piles loaded to a maximum of 14 tons tension. However, cost studies indicated that a higher compression pile loading might be more economical. In addition, a redesign of the pile group layout made it possible to eliminate the short tension piles. The maximum design loads for the present pile layout as shown on plate 23 are 21 tons in tension and 96 tons in compression on long piles driven to sand.

2. The pile tests were located as shown on plate 38 so that the tests would be near existing undisturbed borings (see plate 2). Considerable laboratory data including shear tests are available on borings near each end and the middle of the structure. Split spoon drive sample borings have been made at the three test locations to locate the top of the deep sand and to determine its relative density (see plate 6). Undisturbed samples were obtained to a depth of 25 ft in boring M-37 near the center of the structure to check the low strength indicated by the upper portion of boring M-31.

General

3. Driving of piles. All test piles should be driven to the tip elevation specified in tables 3 and 4 without interruption insofar as practicable. The hammer used should be of the single-acting, steam or air driven, capable of delivering not less than 15,000 ft lb of energy per blow. The pile driver should not have free hanging leads. A continuous record of the driving resistance (number of blows per foot) should be kept for all piles. All piles should be driven vertically with a maximum deviation of about 4 in. from the stated location as listed in table 3 for the compression piles and table 4 for the tension piles and as shown on plate 38. All test and anchor piles should rest for two weeks after driving before testing. ~~The order of driving the piles at any station~~ should be compression "b" pile, tension pile, compression "a" pile. If the driving record of a "b" pile does not indicate a driving resistance of 30 blows per in. at the specified tip elevation given in table 2, the pile should be driven further until the resistance becomes 30 blows per in. or until the tip of the pile is 15 ft below the top of sand as indicated by the driving record. The tension pile should be driven to the same tip elevation as the "b" compression pile. The "a" compression pile should be driven to the specified tip elevation or to a tip elevation 5 ft above the top of the sand as indicated by the driving record of the "b" pile, whichever is higher.

4. Measurement of movement. A continuous time-movement record should be kept on all test piles. The movement may best be measured by dial gages reading direct to 0.001 in. mounted on a steel beam securely

anchored. The anchors for the beam should be at least 10 ft from the test and anchor piles. Gages should be placed on opposite sides of the test pile so that the average movement can be determined. Observations should be made with a level on the anchor piles, the anchors of the reference beams, and the test piles before each new load increment is applied or removed.

5. Excavation. The area around each test pile should be excavated to elev 23.5, as that is the elevation of the bottom of the drainage blanket beneath the structure. This excavation, which may be done before or after driving, need be only 6 to 12 in. larger than the pile.

6. Deviation from stated procedure. If field conditions indicate that minor changes in procedure are necessary or desirable to obtain better results from the tests, the engineer in charge of the pile tests may require that such changes be made. A record should be kept as to the exact procedure followed if any changes are made.

7. Removal of piling. By driving the test piles about 80 ft east of the center line of the railroad relocation, the piles used in the test program will not interfere with the piles to be driven beneath the structure. However, the test and anchor piles must be cut off below elev 29 or pulled so as not to interfere with the placing of the upstream riprap. The holes remaining after any of the piles are pulled should be backfilled with tamped clay to prevent entry of seepage into the silty layer that exists at a depth of about 25 ft.

Types of Piles

Pipe piles

8. It is considered that steel pipe piles will be necessary to

carry a design compression load of about 100 tons. It is felt further that the size of pile required will not be less than 18 in. diameter and probably will be 24 in. Therefore, it is recommended that tests be conducted on 24-in. pipe piles near each end and the middle of the structure and on 18- and 30-in. pipe piles near the middle of the structure.

Metal shell piles

9. In order to have a complete series of tests, it is recommended that tests be performed on piles which would carry an estimated design load of approximately 50 tons. It is suggested that uniform tapered piles with an 8-in. tip (Monotube F-sections) and 12-in. constant-section piles (Monotube N-section) be tested at the center of the structure. It is believed that the action of these two types of metal shell piles will be similar to that of pipe, precast concrete, or other commercial piles with a taper of 1 in. in 7 ft or less, and with parallel sides, respectively.

Compression Piles

General

10. It is considered necessary to conduct the compression pile tests in such a manner that the bearing capacity of the portion of the pile driven into the sand will be evaluated. In order to do this it is necessary to eliminate completely or to measure the frictional resistance developed along the pile in the clay stratum. The effect of the clay could be completely eliminated by testing a pile driven through a casing. The resistance developed along a pile in the clay may be determined if piles are tested in pairs: one driven into the sand and one driven to

above the sand. The latter method has been selected by the Office, Chief of Engineers. All piles should be tested to failure or to the limit of the loading equipment.

Size and type of piles

11. It is recommended that 24-in. diameter piles be tested in pairs near each end and the middle of the structure to determine variations that may exist due to different foundation conditions. At the center of the structure, pairs of 18- and 30-in. pipe piles and 12-in. constant section and uniform-taper piles should also be tested so that the most economical size of pile may be determined. If the driving record of a "b" pile driven into sand does not indicate a driving resistance of 30 blows per in. at the specified tip elevation given in table 3, the pile should be driven further until such resistance is obtained or until the pile tip is 15 ft below the top of the sand as indicated by the driving record. The "a" pile should be driven to the specified tip or to a point 5 ft above the top of the sand as indicated by the driving record of the "b" pile. The pipe piles may be driven with the end either open or closed and then carefully filled with concrete. If the pipe piles are driven open-ended they should be carefully cleaned out prior to filling with concrete. Care should be taken not to disturb the sand beneath the tip of the pile during the cleaning operation. A tremie seal should be placed in the bottom of the pipe piles before the piles are pumped free of water and the remainder of the concrete is placed.

Means of loading

12. The load should be applied by hydraulic jacks. The reaction

for the jacks may be either anchor piles or dead weight on a platform or a combination of both. If anchor piles are used, the total embedded surface area of the anchor piles should be not less than 1800 sq ft for the "a" piles and 4000 sq ft for the "b" piles. The anchor piles should be not closer than 5 ft to each other and the test pile. If the dead weight method is used, the total weight should be in place before testing starts. The weight for the "a" piles should be 200 tons and for the "b" piles be 400 tons, or the capacity of the corresponding "a" pile plus 210 tons, whichever load is larger. All beams and connections in the loading yoke and the jacking equipment should be designed for the maximum load to be applied to the pile.

Loading procedure -- pipe piles

13. "a" piles. The "a" piles of each pair should be tested first and should be loaded to failure. The loads should be applied in 20-ton increments at a rate of 1-1/2 tons per minute. Each increment should be maintained until 80 per cent of the estimated movement has taken place, or for 24 hours, whichever is shorter, except as otherwise stated. At loads of 80 and 160 tons, the load should be maintained until all movement stops, or for 24 hours, whichever period is longer. After completion of the 40-, 80-, 120-, and 160-ton loads and at the end of the test, the piles should be unloaded and allowed to rebound. Unloading and reloading should be at a rate of 1-1/2 tons per minute. The piles should be allowed to rebound until 80 per cent of the estimated rebound has occurred, or for 12 hours, whichever is shorter, except at the end of the test when complete rebound should be allowed. Time-movement curves should be plotted

as the test progresses so that the duration of the load increments may be determined.

14. "b" piles. The "b" piles should be loaded and tested as given above for the "a" piles until each "b" pile is loaded to the capacity of the corresponding "a" pile. The loading of the "b" pile should then be continued with the same increments and rates as given for the "a" piles. When the additional load is 100 tons and 200 tons, the load should be maintained until all movement stops, or for 24 hours, whichever is longer. After the additional loads of 60, 100, 160, and 200 tons and at the end of the test, the piles should be unloaded and allowed to rebound as given above for the "a" piles.

Loading procedure -- metal shell piles

15. "a" piles. The "a" piles of each pair should be tested first and should be loaded to failure. The loads should be applied in 10-ton increments at a rate of 1-1/2 tons per minute. Each increment should be maintained until 80 per cent of the estimated movement has taken place, or for 24 hours, whichever is shorter, except as otherwise stated. At loads of 40 and 80 tons, the load should be maintained until all movement stops, or for 24 hours, whichever period is longer. After completion of the 20-, 40-, 60-, and 80-ton loads and at the end of the test, the piles should be unloaded and allowed to rebound. Unloading and reloading should be at a rate of 1-1/2 tons per minute. The piles should be allowed to rebound until 80 per cent of the estimated rebound has occurred, or for 12 hours, whichever is shorter, except at the end of the test when complete rebound should be allowed. As for the pipe piles, time-movement curves should be

plotted as the test progresses so that the duration of the load increments may be determined.

16. "b" piles. The "b" piles should be loaded and tested as given above for the "a" piles until each "b" pile is loaded to the capacity of the corresponding "a" pile. The loading of the "b" pile should then be continued with the same increments and rates as given for the "a" piles. When the additional load is 50 tons and 100 tons, the load should be maintained until all movement stops, or for 24 hours, whichever is longer. After the additional loads of 30, 50, 80, and 100 tons and at the end of the test, the piles should be unloaded and allowed to rebound as given above for the "a" piles.

Tension Piles

General

17. Tension tests should be performed on 24-in. pipe piles near each end and the middle of the structure. Near the middle of the structure, tension tests should also be performed on an 18-in. pipe pile, a 12-in. constant section metal shell pile (Monotube N-section), and a uniform tapered pile (Monotube F section). It is believed that a tension test is not necessary on the 30-in. pipe pile. The locations and approximate tip elevations of all tension piles are given in table 4. The tip of tension piles as driven should be at the same elevation as the tip of the adjacent "b" compression pile.

Means of loading

18. Each pile should be loaded by jacking against two anchor piles

or the adjacent compression piles. If the compression piles are used as anchors, the compression tests at any station should be completed before the tension test is initiated. The anchor piles if used may be timber of such circumference and length that the total embedded area of the anchor piles is at least 1.5 times the embedded area of the test pile. The anchor piles should be 5 ft from the test pile. The reaction beam and the connection between the beam and the test pile should be designed to carry 150 tons. The connections between the reaction beam and the anchor piles should be designed to load the test pile to 100 tons compression as well as 150 tons tension. Hydraulic jacks of sufficient capacity to produce a 150-ton loading on the test pile should be provided.

Loading procedure

19. Prior to being tested in tension, all piles should be loaded in compression to a maximum load of 100 tons with the exception of piles T2 and T3 which should be loaded to 50 tons. The load should be applied at a rate of 1.5 tons per minute. This load should be maintained for at least 12 hours before being removed at a rate of 1 ton per minute. The tension load should be applied as soon as possible after the compression load is removed. The tension load increments should be 8 tons. The rate of applying each increment should not be more than 1 ton per minute to avoid any sudden application of load to the pile. Each increment, except at 24, 48, 72, and 96 tons total load, should be maintained until 80 per cent of the estimated movement under that increment has taken place, or for a maximum period of 12 hours. At loads of 24, 72, and 96 tons, the load should be sustained until all movement stops and for at least 24

hours. At the 48-ton load the load should be maintained for 48 hours. On pile T5 the 48-ton load should be held for 7 days. On pile T5 it is recommended that the pile be allowed to rebound under no load after the 24-, 48-, 72-, and 96-ton loads. The rebound should be allowed to continue until 80 per cent of the rebound has occurred, or for a maximum of 12 hours. The maximum rate of unloading and reloading should be 1 ton per minute. All tension tests should be carried to failure if possible. Rebound should be measured at the end of the test on all test piles not loaded to failure. To follow the progress of the test it is recommended that time-movement curves, plotted to either arithmetic or semilogarithmic scales, be plotted as the test progresses. Only on the basis of such curves can it be estimated when 80 per cent of the movement has occurred.

Table 3

COMPRESSION PILE TESTS

Pile	Type of Pile	Diameter of Pile In.	Approximate Elevation of Tip	Station	Distance from Proposed R.R. Center Line-Ft
C1a	Pipe	24	-30	682+50	75
C1b	Pipe	24	-53	682+50	85
C2a	Uniform taper	8 (tip)	-42	699+50	75
C2b	Uniform taper	8 (tip)	-52	699+50	85
C3a	Constant section	12	-42	699+75	75
C3b	Constant section	12	-52	699+75	85
C4a	Pipe	18	-42	700+00	75
C4b	Pipe	18	-52	700+00	85
C5a	Pipe	24	-42	700+25	75
C5b	Pipe	24	-52	700+25	85
C6a	Pipe	30	-42	700+50	75
C6b	Pipe	30	-52	700+50	85
C7a	Pipe	24	-41	718+00	75
C7b	Pipe	24	-51	718+00	85

Table 4

TENSION PILE TESTS

Pile	Type of Pile	Diameter of Pile In.	Approximate Elevation of Tip	Station	Distance from Proposed R.R. Center Line-Ft
T1	Pipe	24	-53	682+50	80
T2	Uniform taper	8 (tip)	-52	699+50	80
T3	Constant section	12	-52	699+75	80
T4	Pipe	18	-52	700+00	80
T5	Pipe	24	-52	700+25	80
T6	Pipe	24	-51	718+00	80

APPENDIX B

INSTALLATION AND READING OF EMBANKMENT PIEZOMETERS

Installation

1. In order to obtain reliable data, it is imperative that the utmost care be observed in the construction of the piezometer tips and the splicing of the brass tubing. It is recommended that the procedure outlined by the Bureau of Reclamation to assemble the tips and splice the tubing be followed.*

2. The tips to be located at elev 20 and -10 in the foundation beneath the embankment are to be placed in individual 6- to 8-in. holes at points approximately 18 ft riverside and 60 ft landside of the center line of the railroad. The tip should be a 4- to 6-in. length of all-brass well strainer, such as Clayton-Mark or equal, with #35 slots. The hole should be filled with filter sand A for 2 to 3 in. above and below the tip. The hole should be backfilled around the 3/4-in. brass riser pipe above the sand with 2 ft of tamped clay, then 2 ft of sand-bentonite seal, and the remainder with tamped clay. All backfill should be placed under water so that air will not be entrapped in the hole. A brass tee, shown in detail on plate 37, should be placed at the top of the riser. The copper tubes should be soldered into the brass tee after it is placed on the riser. The elevation of the top of the riser should be such that the grade of the tube to the terminal well will be approximately 0.02. All

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* Field Manual for Rolled Earth Dams, U. S. Dept. of Interior, Bureau of Reclamation, Appendix B, November 1947, pp 58-59.

joints in the riser and tubing should be heavily leaded and it would be advisable to solder all joints. These piezometers should be installed and provision made for reading them before construction of the embankment is started.

3. The tip in the embankment, detail shown on plate 37, should not be placed until the embankment has reached a grade of 7 ft above ground surface at a point in the embankment about 18 ft riverside of the center line of the railroad with the porous stone down. The backfill around the tip should be material from the hole in which the tip is placed. No sand or porous material should be used around these tips.

4. Trenches approximately 12 in. wide and 1-1/2 ft deep will serve to carry the piezometer tubing to the terminal well located at the toe of the landside slope. All trenching should be on an ascending or descending grade from the tips to the terminal wells so that there will not be any humps in the tubing in which air might collect.

5. The material used for backfill around the tubing both in the holes and in the trenches should be typical of the adjacent material. No pockets of relatively porous or uncompacted material should be left in the trench. After the hand cleanup of the trench, a protective cushion of approximately 2 in. of compacted material should be placed in a systematic manner with the individual tubes separated approximately 2 in. placed side by side. After spacing the tubes in the trench, a minimum of 3 in. of material should be placed over the tubing and compacted by hand-operated power tampers. The remaining portion of the trench should be backfilled and power-tamped in 4-in. layers, using typical embankment material. A minimum of 12 in. of compacted material must cover the tubing before

rolling operations are permitted over the trench. Regular embankment-placing methods should be used over the completed trenches.

6. The tubes should be brought into the terminal well through a steel pipe in the wall of the well. After the installation is complete this steel pipe should be filled with asphaltic material. Just inside the well the brass tubes should be changed to 1/4-in. brass pi e. A cutoff valve should be on the ends of both brass pipe leads. A tee connection should be installed on both leads on the tip side of the line. A short line and a valve should be attached to the tees so that the gages may be connected onto the end of each line. The gages should be capable of measuring pressures up to 30 lb per sq in. and should be calibrated before installation. A schematic piping diagram is shown on plate 37. All pipes should be permanently marked with the piezometer number and should be attached to a panel board or the wall of the terminal well.

7. The terminal well should be a small manhole constructed of concrete located at the landside toe of the embankment. The well should extend 18 to 24 in. above the ground surface. Provision must be made for covering the well when readings are not being taken. It is recommended that the sides and bottom of the well be waterproofed. A small sump should also be provided in the bottom of the well.

Readings

8. The general procedure outlined by the Bureau of Reclamation for reading the piezometers should be followed. Before any readings are taken, it is of the utmost importance that the tubes be free of air. At the time of installation, water should be circulated through the tubes

and the tips until no air bubbles appear in the return line. It is recommended that freshly-boiled water be used for this deairing process. After installation this deairing may be done whenever it is felt that air is present in the tubes or the riser pipe.

9. Readings should be made on all piezometers shortly after their installation. Readings should be taken every two weeks during the construction period and every month in the interim between construction periods. It is recommended that a check reading be obtained on each piezometer at every fourth set of readings by connecting a master gage into the line at the extra tee on the inlet line. Readings should be taken every 6 months after construction has been completed until the readings indicate that the pore pressures have been completely dissipated.