CORPS OF ENGINEERS, U. S. ARMY

REVIEW OF SOILS AND FOUNDATION DESIGN AND FIELD OBSERVATIONS MORGANZA FLOODWAY CONTROL STRUCTURE, LA.

TECHNICAL MEMORANDUM NO. 3-384

PREPARED FOR

THE PRESIDENT, MISSISSIPPI RIVER COMMISSION

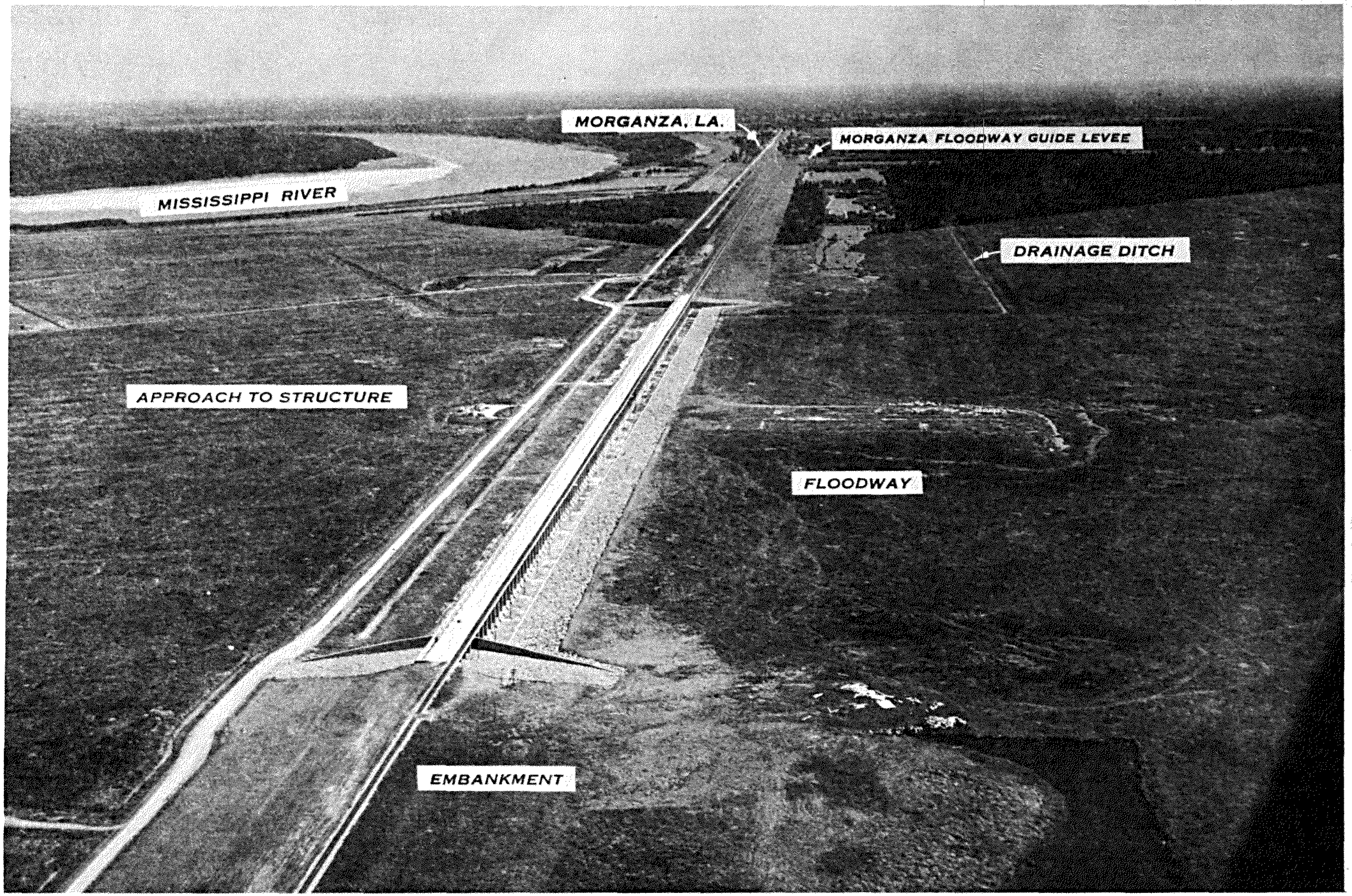
BY

WATERWAYS EXPERIMENT STATION

VICKSBURG, MISSISSIPPI

ARMY-MRC VICKSBURG, MISS.

AUGUST 1954



FRONTISPIECE. Aerial view of the Morganza Floodway Control Structure

PREFACE

This report is one of a number of similar reports on studies of the construction and behavior, from a foundation and soil mechanics standpoint, of recently completed structures in the Lower Mississippi Valley Division. The purpose of the studies is to compare field experience and performance with design predictions, and from such comparisons and observations gain information and experience that will be valuable in the design and construction of future projects in the Lower Mississippi Valley. These studies are being made for the President, Mississippi River Commission, by the Waterways Experiment Station. Structures being studied are those for which foundation and soil mechanics investigations were made and design was prepared by the Waterways Experiment Station.

The studies discussed in this report concern the construction and behavior of the Morganza Floodway Control Structure, and its adjoining embankments which serve as a main-line levee along the west side of the Mississippi River and also carry the Port Allen Branch Line of the Texas and Pacific Railroad, and Louisiana State Highway No. 30. The structure is located adjacent to the Mississippi River approximately 35 miles northwest of Baton Rouge, Louisiana, and 3 miles northwest of Morganza, Louisiana.

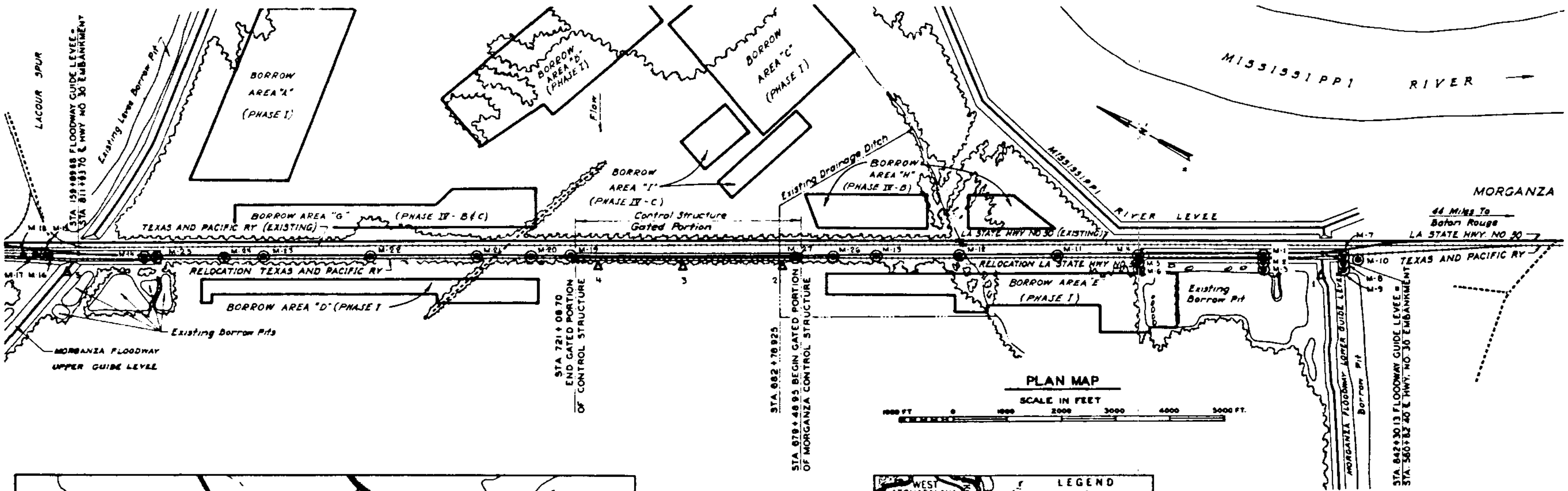
The Morganza Floodway Control Structure was designed by the Mississippi River Commission and was built under the supervision of the New Orleans District, CE. Initial construction of the combined embankment was started in 1941 but was stopped in 1943 because of cessation of civil works during World War II. Work on construction of the adjoining embankments was resumed in December 1949; these embankments were essentially completed in January 1954. Pile tests to determine the size and allowable loading for the pile foundation beneath the structure were performed during the summer of 1949. Preload fills to reduce differential settlements at the abutments for the structure were placed at the south abutment in November 1949 and at the north abutment in July 1950. Driving of the pile foundation for the control structure proper was started in October 1950 and completed in August 1951. Work on the superstructure was also begun in October 1950 and was completed in February 1953. The first train crossed the structure on 18 November 1952 and highway traffic will probably begin in the latter part of 1954.

Observations of settlement plates and hubs, piezometers, and movement markers placed in and beneath both the embankment and control structure were made during construction and have been continued up to the present time. It is planned to continue these observations and to analyze and report any future pertinent data in the form of appendices to this report.

The studies and analyses of data presented in this report were made by Mr. C. I. Mansur, and Messrs. John A. Focht, Jr. and William Emrich (formerly of the Waterways Experiment Station). The field explorations were under the supervision of Mr. T. B. Goode. The studies were performed under the general direction of Messrs. W. J. Turnbull, W. G. Shockley, and S. J. Johnson (formerly of the Experiment Station), Soils Division, Waterways Experiment Station. This report was prepared by Mr. Mansur.

Data on construction procedures and engineering measurement devices were furnished by Messrs. M. G. Chitty, H. A. Huesmann, and John W. Harris of the New Orleans District, CE. This report was reviewed prior to publication by Messrs. Huesmann and Harris of the New Orleans District, and Messrs. Howard B. Gray and E. H. Eckler, Jr., of the Mississippi River Commission.

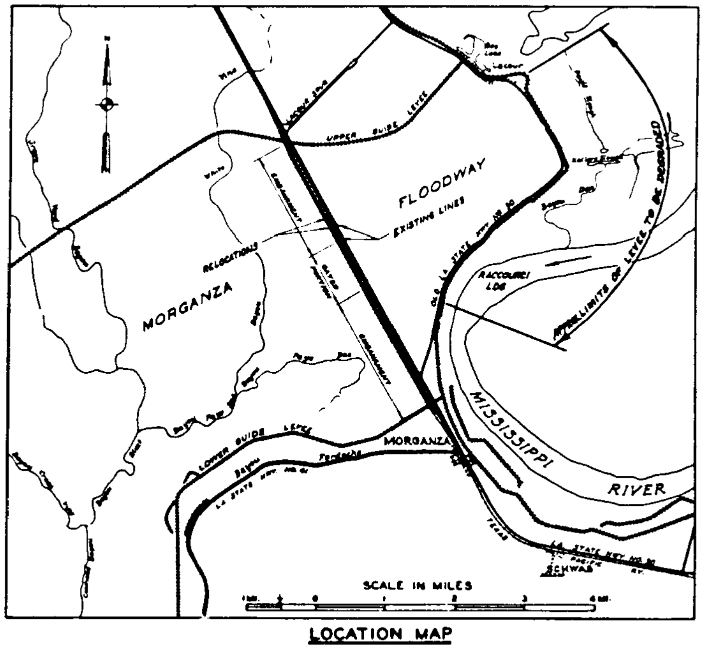
CONTENTS



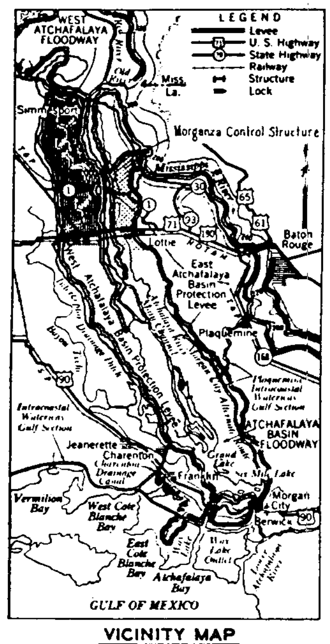
LEGEND

- ● GENERAL SAMPLE BORINGS
- ◉ UNDISTURBED SAMPLE BORINGS
- △ PERMANENT BENCH MARKS

NOTE: LETTER SUFFIX TO PHASE NUMBERS INDICATES WHETHER MATERIAL WAS TO BE USED FOR BERM OR CORE.



LOCATION MAP



VICINITY MAP

Fig. 1. General maps, and plan of foundation borings and borrow areas for embankments

REVIEW OF SOILS AND FOUNDATION DESIGN AND FIELD OBSERVATIONS MORGANZA FLOODWAY CONTROL STRUCTURE, LOUISIANA

PART I: INTRODUCTION

Description of the Structure

1. The Morganza Floodway Control Structure is located adjacent to the west bank of the Mississippi River approximately 35 miles northwest of Baton Rouge, Louisiana, and 3 miles northwest of Morganza, Louisiana. It consists primarily of a gated concrete weir, approximately 4000 ft long, for passing excess floodwaters from the Mississippi River into the Atchafalaya Basin. The structure also provides a crossing over the floodway for Louisiana State Highway No. 30, the Port Allen Branch Line of the Texas and Pacific Railway Company, and the main line of the Kansas City Southern Railway Company. The weir is connected to the guide levees of the floodway by earth embankments which also serve as a portion of the main-line levee system of the Mississippi River. A vicinity and plan map of the structure and embankments is shown in fig. 1.

2. The gated portion of the control structure has 125 bays, each having a span of 31 ft 3 in. center-to-center of piers. A partial plan and elevation of the structure is shown in fig. 2. Each pier is supported by 27, 20-in. precast concrete piles driven to sand on a 2-on-1 batter. Eleven of the piles are battered upstream and 16 downstream. A plan of the pile foundation and section of the structure are shown in fig. 3. The design pile loads are 100 tons in compression and 25 tons in tension. The ability of the piles to carry these loads was based on a series of pile-loading tests conducted at the site prior to construction.

3. The gated portion of the control structure is designed to discharge a maximum flow of 600,000 cfs. The weir crest is at elevation 37.5 msl, which is approximately 5 ft above the average approach channel elevation. The structural design of the gated portion was based on a maximum headwater elevation of 60.0 msl with the gates closed and an

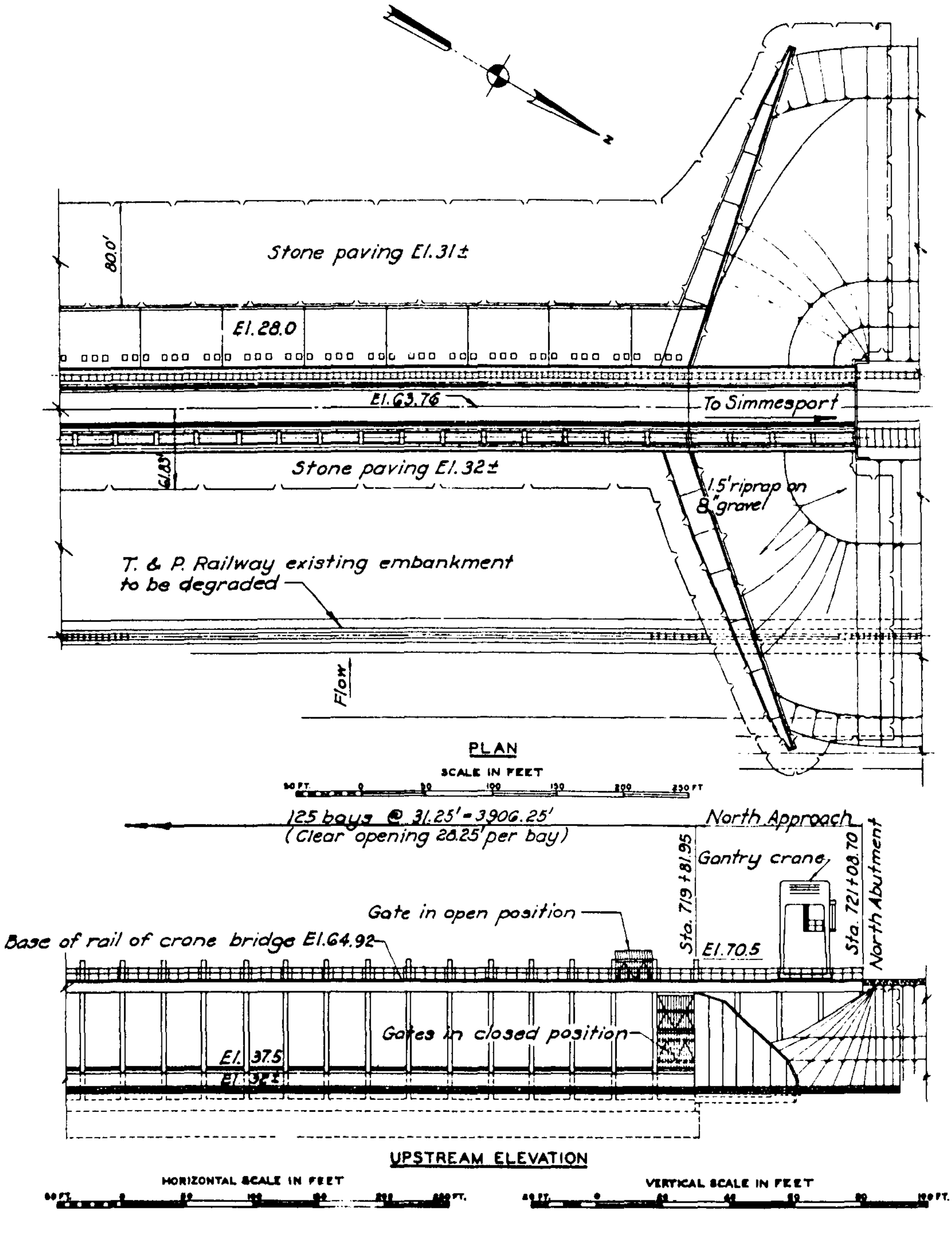


Fig. 2. Partial plan and elevation of structure

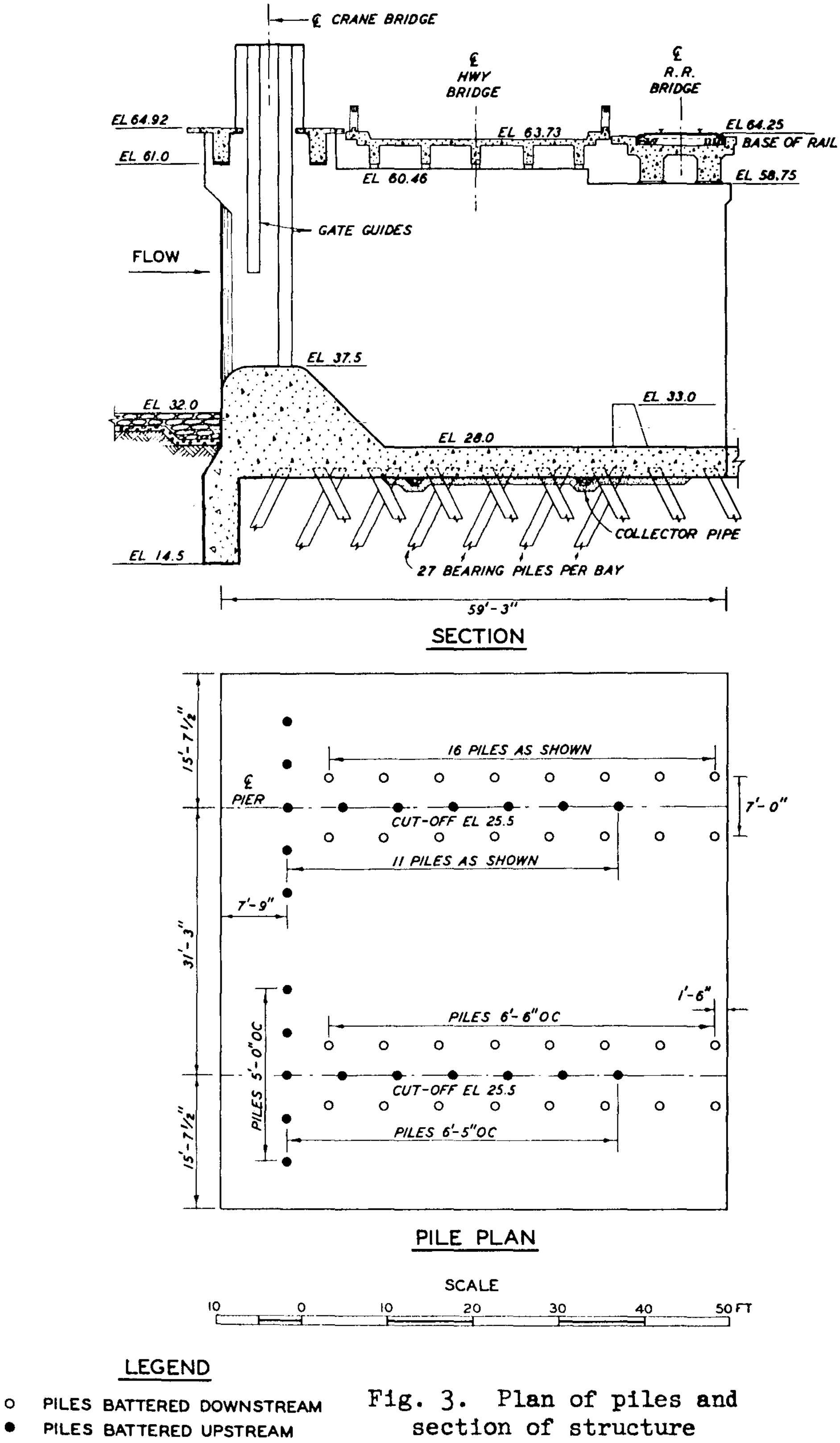


Fig. 3. Plan of piles and section of structure

elevation of 57.0 with the gates open. The maximum net head assumed for the structure was 28 ft. The stilling basin below the gated portion consists of a concrete apron containing a row of baffle piers, 5 ft high, and an end sill 4 ft high with top of sill at elevation 32.0. The area downstream of the stilling basin is paved with riprap and derrick stone for a distance of 80 ft and the area upstream of the structure is paved with riprap for a distance of 30 ft. An aerial view of the completed structure is shown in fig. 4.

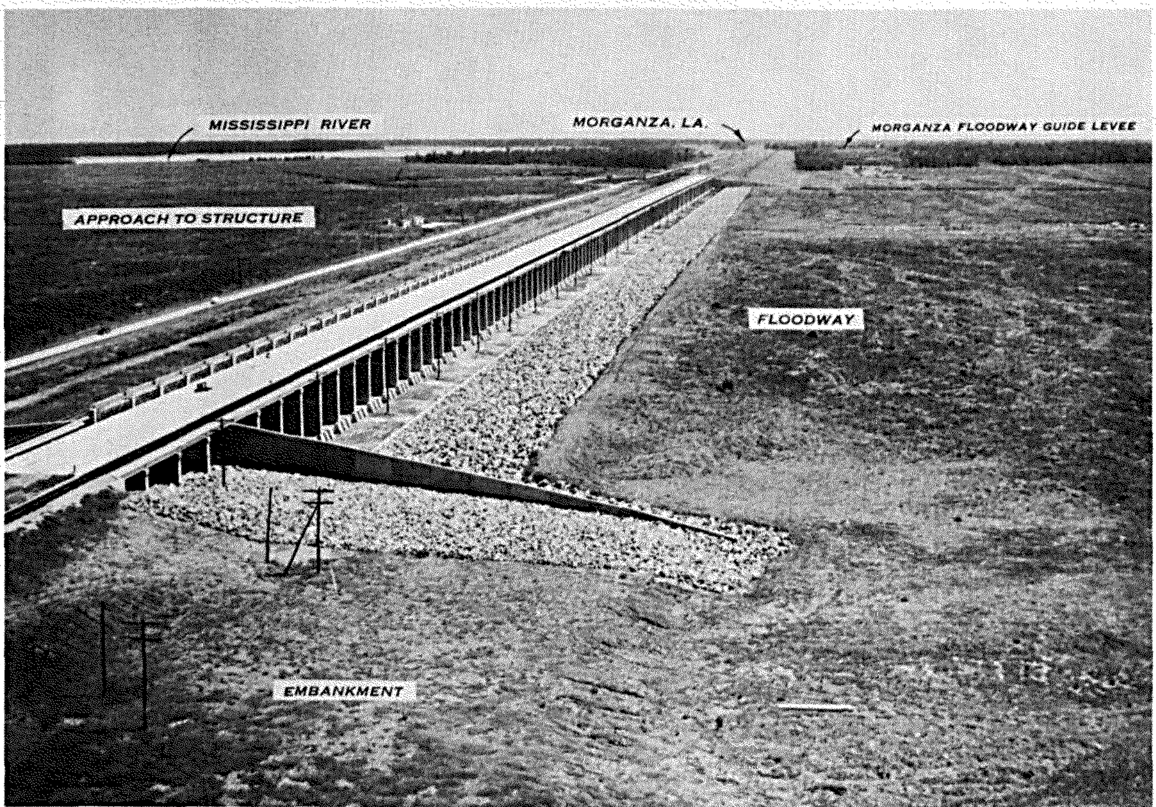
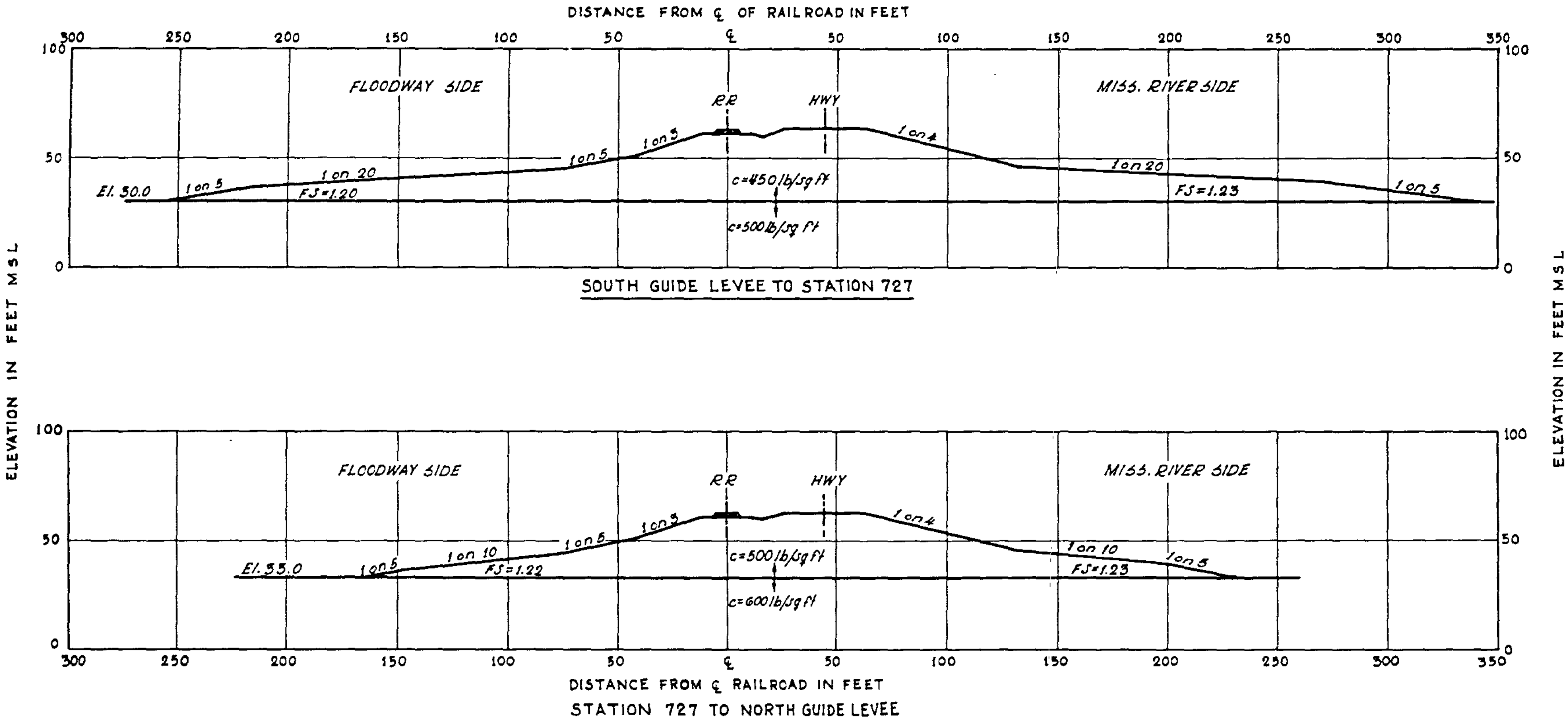


Fig. 4. Aerial view of control structure

4. The control structure is connected to adjacent guide levees by embankments each approximately 9500 ft long. The embankments are about 30 ft high and have a base width of 400 to 500 ft. Typical sections of the embankments are shown in fig. 5.

5. In order to eliminate rerouting of the railroad and because of the magnitude of the project, the construction was accomplished in several phases, as described subsequently.



Note:
Factors of safety obtained by circular arc method with depth of sliding to elev. -20

Fig. 5. Typical cross sections of embankments

Scope of This Report

6. This report presents a summary of the foundation conditions and the design studies performed for the Morganza Floodway Control Structure and adjacent embankments, together with certain observations made before, during, and after construction. Comparisons of design predictions with those observed are also included. More specifically, the report deals with field explorations which were made in connection with the design and construction of the structures; foundation conditions; characteristics of the foundation soils and borrow materials for the structures; stability and settlement of embankments adjacent to the control structure; pile loading tests made to determine the size and carrying capacity of piles used beneath the structure; the pile foundation for the control structure; abutments; underseepage and the drainage system beneath the control structure; and stone protection. Information regarding the original design and pile loading tests which has been presented in previously published reports* is included in highly condensed form for the purpose of making this report a complete discussion of the investigation to date, and to permit comparisons between design predictions and field behavior. Observations made during construction of the structure and embankments are described in more detail. All pertinent data obtained from settlement plates and hubs, piezometers, and other engineering measuring devices during and since construction of the control structure and embankments are also included. Details of field investigations, laboratory tests, design analyses, and specifications not included may be found in the references given at the end of this report.

---

* See items 1-6 of list of references following the text of this report.

PART II: FIELD EXPLORATIONS AND FOUNDATION CONDITIONS

7. The site of the project is in the backswamp region west of the Mississippi River near Morganza, La. The foundation beneath both the embankment and the structure consists of strata of predominantly highly plastic clays about 80 ft thick underlain by a very thick stratum of sand. The average water content of the clay strata ranges from about 42 to 60 per cent, and liquid limits range from 60 to 100 per cent. The contact between the sand and the clays is somewhat irregular and a few clay lenses exist in the upper portion of the sand along certain reaches.

Foundation for Embankments

8. A plan of the borings made to investigate the foundation for both the south and north embankments is shown on fig. 1. Logs of the borings made along the center line of the embankments are shown in figs. 6 and 7. The borings were drilled as cased borings with truck-mounted, rotary core drills. Undisturbed samples were obtained from below the casing with 5-in. ID vacuum-type samplers. The casing was advanced by means of a power auger and bailer.

9. The foundation of the south embankment consists of backswamp deposits of highly plastic clays with some clay silts underlain by sand at a depth of about 60 to 70 ft. The clays are slightly fissured, and contain numerous silt seams and strata that accelerate their 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 slightly stronger, less compressible, and thicker, ranging from about 80 to 90 ft; it also contains several silt strata.

Borrow Materials

10. The locations of borrow areas, from which material used in construction of the embankments was obtained, are shown on fig. 1. Auger-type borings were drilled to a depth of 15 ft to investigate the borrow areas.

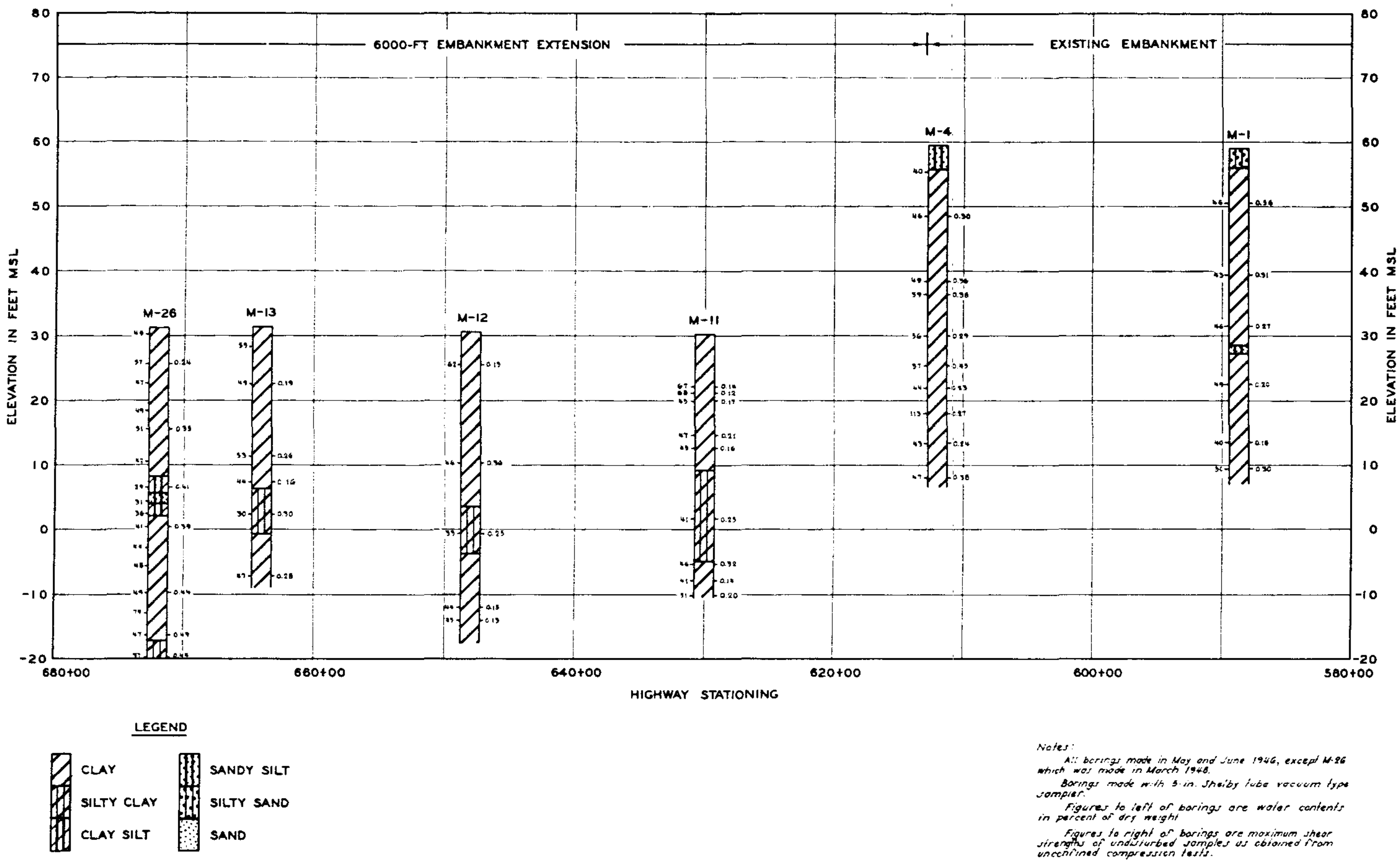


Fig. 6. Logs of borings, south embankment, highway sta 589+00-675+00

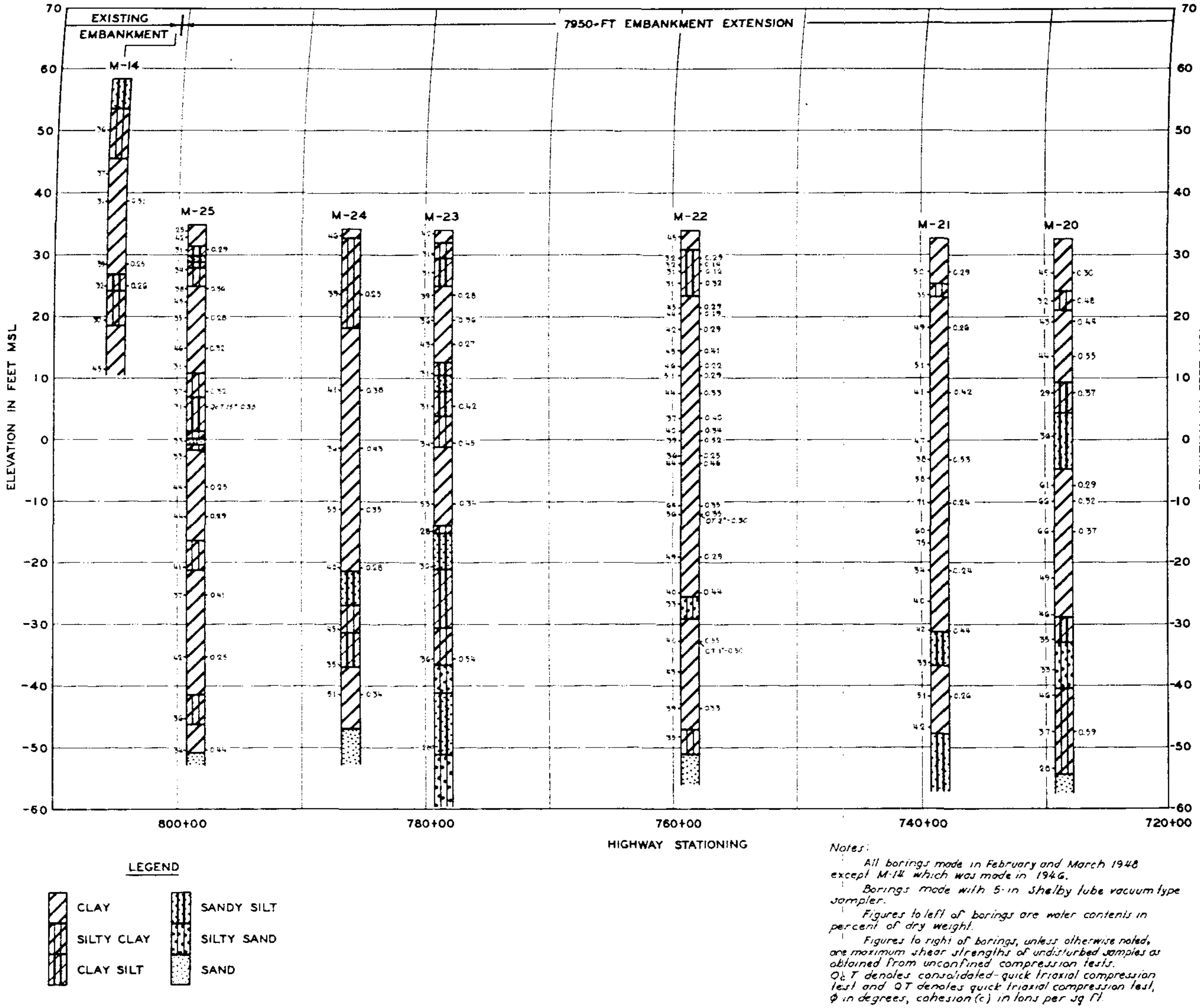


Fig. 7. Logs of borings, north embankment, highway sta 725+00-805+00

11. Most of the material used in construction of the embankments consisted of high water content, fat, inorganic, plastic clays and silty clays. Investigations made prior to construction of the embankments indicated that the natural water content of the borrow soils during the summer months would be 10 to 15 per cent above optimum water contents (22 to 32%) for compaction, as determined by a compaction test using 15 blows rather than the standard 25 blows per layer. The material is slow to dry out and is subject to shrinkage and cracking.

12. Borings made in borrow area "D" in August 1948 showed that the average water content of the upper 8 ft of soil had dropped considerably from that noted from borings made in March 1948. The average drop was 15 per cent in the upper 4 ft and 10 per cent in the next 4 ft. During the summer the water table in area "D" was at a depth of approximately 6 to 9 ft. By mid-October, the average water table had dropped to a depth of about 9 to 10 ft. The average water contents for the upper 8 ft of soil in borrow areas "B," "C," and "D" are tabulated below:

Water Content of Borrow Areas, 1948

	Area "D"		Area "C"	Area "B"
Depth in ft	March	August	28 October	1 November
0-4	50	35	28	31
4-8	41	31	35	29

Foundation for Control Structure

13. The foundation beneath the control structure is similar to that previously described for the embankments and consists predominantly of highly plastic clays approximately 80 ft thick, underlain by a bed of sand approximately 50 ft thick. In the clay stratum, there is a stratum of sandy silt approximately 10 ft thick at approximately elev 8.

14. A plan of the borings made to investigate the foundation along the control structure and logs of selected borings are shown in fig. 8.

15. Explorations for determination of the required length of piles included an additional 75 deep borings made on about 100-ft centers along the structure. Most of these were of the split-spoon type. Fig. 9 presents logs of and split-spoon resistances obtained from these borings.

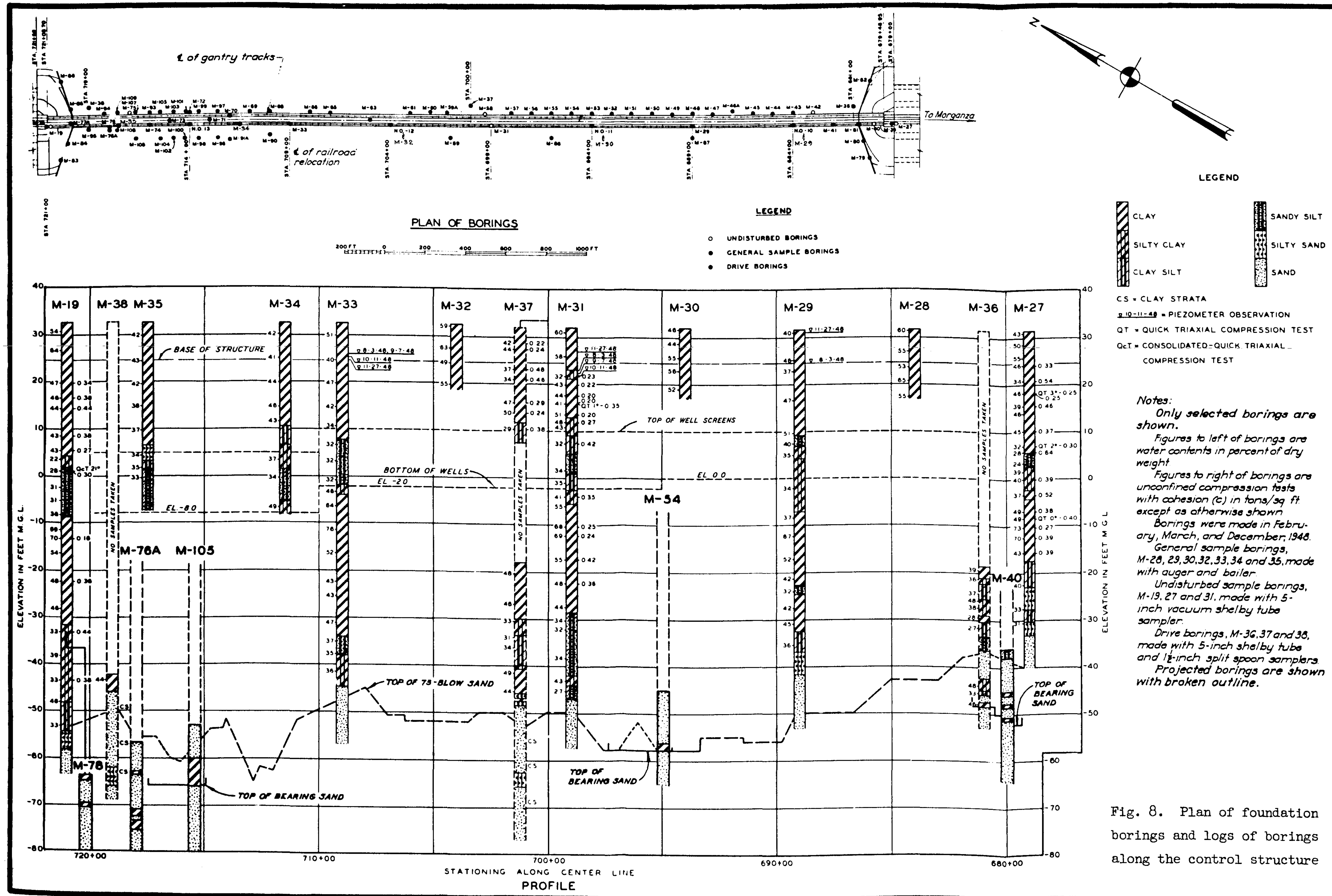


Fig. 8. Plan of foundation borings and logs of borings along the control structure

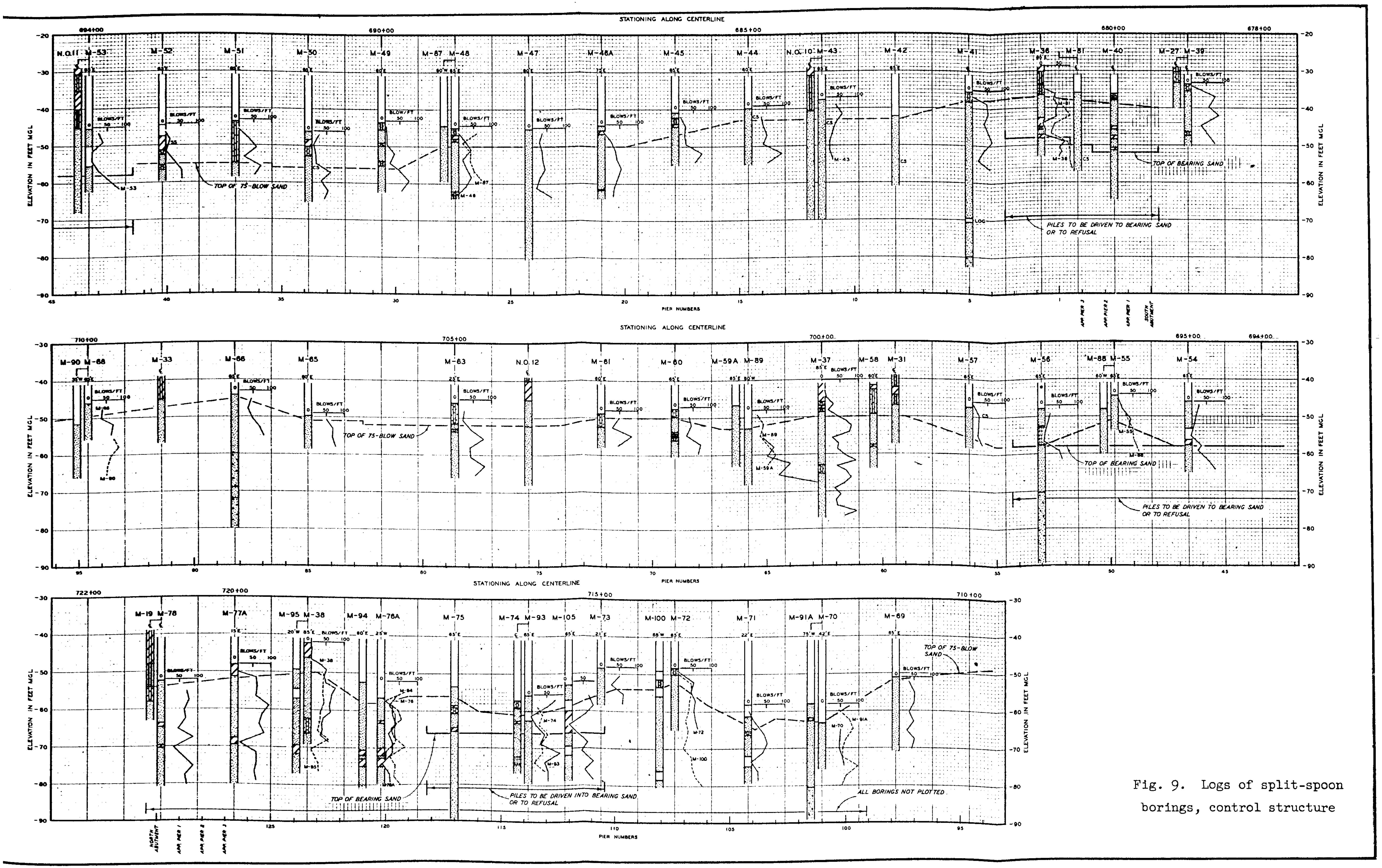


Fig. 9. Logs of split-spoon borings, control structure

The borings were located along both sides of the center line of the structure near the center of the tips of the pile groups battered upstream and downstream.

16. The undisturbed sample borings were drilled by the same method used for the embankment borings. The split-spoon type borings were made with a 1-3/8-in. ID, 2-in. OD Raymond type split-spoon sampler. The sampler was driven with a 140-lb hammer falling 30 in. The split-spoon type borings were advanced between drives by the fishtail method, using drilling mud with an average weight of about 80 lb per cu ft.

17. Consolidation tests on samples of the clay from beneath the structure showed that the foundation is very compressible. Existing embankments 30 ft high in the vicinity of the structure had settled 2-1/2 to 5 ft. Thus, it was necessary that the piles be driven to sand to support the structure without excessive settlements.

18. Records of borings made with a standard split-spoon sampler showed the sands to have an average driving resistance of about 55 blows per ft. The sand underlying the clay is of fine to medium gradation. Some thin lenses of clay and lignitic sands were found along some reaches in the upper part of the main sand stratum.

PART III: SOIL CHARACTERSITICS

Foundation for Embankments and Control Structure

19. Extensive laboratory testing was carried out as a part of the investigation of the foundation beneath the control structure and the embankments. The laboratory tests included visual classification and natural water content determination, mechanical analyses and Atterberg limits tests, consolidation tests, shear tests (including many unconfined compression tests on both undisturbed and remolded samples and numerous triaxial tests), and a few permeability tests on undisturbed specimens of the upper sandy silt stratum beneath the control structure. The results of the laboratory tests on the foundation soils beneath the embankments are summarized in tables 1 and 2, and beneath the control structure are given in table 3. Natural water contents and shear strengths obtained from tests on undisturbed samples from the foundation beneath the embankments and control structure are also shown on the logs of borings in figs. 6, 7, and 8.

Table 1

Summary of Laboratory Test Data - Embankment Foundation, 1946 Borings

Station	Range	Boring No.	Boring Elev mGl	Sample No.	Sample Elev mGl	Depth ft	Soil Classification	Mechanical Analysis % Sand	% Silt	% Clay	Atterberg Limits LL	PL	PI	Nat. WC %[1]	Unconfined Compr Tests γ_d lb/cu ft	c tons/sq ft	Quick Triaxial Tests γ_d lb/cu ft	ϕ^o	c tons/sq ft	Consolidation Tests[2] C_c	P_c tons/sq ft	Over-burden tons/sq ft	Sp Gr
574+30	127.5' west CL RR	M-10	35.3	8	24.1	8.4- 9.6	Clay, org, dk brown	9	33	58	82	22	60	49	70	0.13							
				16	16.7	18.0-19.2	Clay, org, dk gray	10	23	67	84	25	59	46									
				17	15.6	19.2-20.3	Clay, org, dk brown							39		0.29[3]							
				20	12.0	22.8-24.0	Clay, org, blue	11	24	65	77	23	54	41									
				26	5.7	30.0-31.2	Clay, org, brown	11	34	55	79	20	59	47	69	0.20							
577+50	CL RR	M-7	48.1	5	43.1	4.5- 5.5	Clay, org, brown	8	12	80	96	24	72	45									
				16[4]	31.8	15.8-16.8	Clay, org, brown	9	26	65	71	22	49	44	76	0.48							
				23	24.6	23.0-24.0	Clay, blue	13	24	63	93	26	67	46									
				26	21.5	26.1-27.2	Clay, blue							48	73	0.26							
				29	18.0	29.5-30.7	Clay, blue	5	21	74				52	71	(Existing failure plane in sample)							
				34	12.1	35.4-36.6	Clay, blue	7	21	72	89	26	63	49	73	0.36							
				38	7.5	40.0-41.2	Clay, blue	7	10	83	107	25	82	52	68	0.19							
	127.5' west CL RR	M-8	35.4	8	25.7	8.9-10.5	Clay, brown	10	28	62	86	28	58	47									
				13	18.8	15.8-17.4	Sandy silt, brown	30	55	15	28	22	6	31	93	0.23							
				17	13.0	21.8-23.0	Clay, blue	11	19	70	85	27	58	44									
				19	9.9	24.8-26.2	Clay, blue							54		0.17[3]	(Existing failure plane in sample)						
				21	7.2	27.5-28.8	Clay, blue	7	13	80	87	29	58	47	66	0.12[3]							
				24	3.0	31.8-33.0	Clay, blue							45		0.20[3]							
				28	-3.2	37.5-39.7	Clay, blue	15	27	58	73	20	53	49	67	0.08							
	255' west CL RR	M-9	27.3	4	22.7	4.0- 5.3	Clay silt, org, brown	21	53	26	40	25	15	33									
				12	11.1	15.4-16.7	Clay, org blue	14	14	72	95	33	61	55									
				15	6.9	19.5-21.3	Clay, blue	7	16	77	100	26	74	60	61	0.14							
589+00	CL RR	M-1	59.0	7	51.3	7.2- 8.3	Clay, dk gray	17	17	66	91	28	63	46	73	0.56							
				9	49.0	9.4-10.6	Clay, org, dark gray	12	9	79				42									
				17	40.2	18.2-19.4	Clay, dark gray	12	13	75	77	21	56	43	77	0.31							
				25[4]	31.4	27.1-28.2	Silty clay, org dk gray	17	32	51	68	17	51	46	74	0.27							
				32	23.6	34.8-36.0	Clay silt, gray	7	16	77	91	22	69	49	72	0.20							
				37	17.8	40.4-41.9	Clay, brown							40	83	0.18							
				41	13.1	45.3-46.5	Clay, blue	5	8	87				45									
				44	9.6	48.8-49.9	Clay, blue	7	7	86				50	71	0.30							

Table 1 (Continued)

Station	Range	Boring No.	Boring Elev mGl	Sample No.	Sample Elev mGl	Sample Depth ft	Soil Classification	Mechanical Analysis % Sand	Mechanical Analysis % Silt	Mechanical Analysis % Clay	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI	Nat. WC %[1]	Unconfined Compr Tests γ_d lb/cu ft	Unconfined Compr Tests C tons/sq ft	Quick Triaxial Tests γ_d lb/cu ft	Quick Triaxial Tests ϕ	Quick Triaxial Tests C tons/sq ft	Consolidation Tests[2] C_c	Consolidation Tests[2] P_c tons/sq ft	Consolidation Tests[2] Overburden tons/sq ft	Sp Gr
	75' west CL RR	M-2	45.0	8[4]	36.0	8.4- 9.6	Clay, org, brown	12	28	60	79	19	60	39	78	0.27							
				13	30.0	14.4-15.6	Clay, org, blue-gray							34	86	0.27							
				18	26.5	20.9-22.1	Clay, brown	8	17	75	97	24	73	47	72	0.32							
				28	11.5	32.9-34.1	Clay, blue	8	17	75	95	37	58	47	74	0.21							
				33	5.5	38.9-40.1	Clay, blue							49									
				34	4.3	40.1-41.3	Clay, blue							51		0.20[3]							
				37	0.6	43.7-45.1	Clay, blue	7	25	68	82	22	60	52	68	0.08							
	150' west CL RR	M-3	33.1	9	23.0	9.6-10.8	Clay, br gray	3	14	83	93	30	63	53	64	0.16							
				18	12.1	20.4-21.6	Clay, blue	7	11	83	88	28	60	47	66	0.08							
				20	9.7	22.8-24.0	Clay, blue							46		0.12[3]							
612+00	CL RR	M-4	59.4	4	55.6	3.2- 4.3	Clay, brown	6	33	61	84	35	49	40									
				10	48.4	10.4-11.5	Clay, br gray	8	29	63	89	34	55	46	70	0.30	72	3°	0.50				
				17	40.0	19.0-20.1	Clay, org, br gray	7	24	69	89	47	42	46			72	4°	0.40				
				18	38.8	20.1-21.1	Clay, org, br gray							49	72	0.36							
				20[4]	36.6	22.3-23.4	Clay, org, br gray							59	61	0.38							
				26	30.1	29.0-30.1	Clay, org, gray, brown							56	72	0.29	64	4°	0.25				
				29	26.4	32.6-33.7	Clay, org, gray, brown	6	31	63				44									
				30	25.1	33.7-34.8	Clay, org, gray, brown							37	81	0.45							
				32	23.0	35.9-36.9	Clay, org, gray, brown	5	17	78				42									
				33	22.0	36.9-38.0	Clay, org, gray, brown	8	19	73	83	22	61	46	70	0.23	69	2°	0.25				
				35	19.6	39.2-40.4	Clay, org, gray brown				123	34	89	66			56	2°	0.35	0.68	1.80	1.84	
				36	18.4	40.4-41.6	Clay, org, dark gray	6	17	77				113	38	0.27							
				37	17.1	41.6-42.8	Clay, org, dark gray	5	20	75	116	26	90	53			64	2°	0.40[5]	0.56	1.80	1.88	
				40	13.7	45.2-46.2	Clay, org, dark gray							44	74	0.24							
				42	11.4	47.4-48.6	Clay, org, dark gray	8	12	80	107	24	83	49									
				45	8.0	50.8-52.0	Clay, org, br, gray	6	15	79	112	29	83	47	73	0.38	72	6°	0.36				
				46	7.0	52.0-53.1	Clay, org, dark gray	5	15	80	108	37	71	42						0.42	2.2	2.06	2.69
	75' west CL RR	M-5	44.7	5	39.4	4.7- 5.8	Clay, brown	7	23	70	84	20	64	49									
				13[4]	30.6	13.5-14.7	Clay, brown	5	14	81	102	18	94	48									
				21	22.7	22.5-23.6	Clay, brown	7	16	77	99	19	80	48	70	0.24							
				24	18.8	25.0-26.6	Clay, org, gray	7	18	75	110	45	65	58	63	0.17							
				29	11.5	31.6-33.0	Clay, gray	3	17	83				48	71	0.18							
				32	9.4	35.8-36.8	Clay, blue	8	22	70	90	28	62	50	70	0.22							
				35	4.6	39.4-40.8	Clay, blue	9	31	60	75	22	53	41						0.35	1.90	1.41	2.73
				36	3.4	40.8-41.9	Clay, blue				63	20	43	41									
				45	-7.1	51.2-52.4	Silty clay, blue	4	54	42	58	22	36	49	73	0.20							
				49	-12.1	56.2-57.4	Clay, blue (shells)	12	11	77	103	20	83	62	63	0.55				1.00	1.80	1.81	
	150' west CL RR	M-6	30.8	6	24.4	5.9- 7.0	Clay, brown	10	14	76	90	15	75	52									
				9	20.8	9.4-10.6	Clay, org, brown				144	52	92	107	41	0.25							
				15	13.6	16.6-17.8	Clay, blue	6	18	80	84	15	69	54									
				17	11.2	19.0-20.2	Clay, blue							49		0.13[3]							
				19	8.8	21.4-22.6	Clay, blue	7	23	70	96	29	67	44	76	0.31							
				24	5.6	24.7-28.6	Clay, blue	12	11	77	51	12	39	33									
630+00	CL RR	M-11	30.2	7	22.4	7.2- 8.4	Clay, org, gray brown							67		0.14[3]							
				8	21.2	8.5- 9.6	Clay, org, gray brown							68	58	0.12							
				9	20.0	9.6-10.8	Clay, gray brown	7	29	64	73	20	53	45	73	0.17				0.46	0.57	0.24	2.68
				13	14.8	14.8-16.0	Clay, br gray							47		0.21[3]							
				15	12.4	17.2-18.4	Clay, br gray							46		0.16[3]							
				16	11.4	18.4-19.3	Clay, br gray	8	14	78	84	24	60	44	71	0.06				0.47	1.80	0.44	2.71
				24	1.1	28.5-29.7	Silty clay, br gray	7	48	45	59	17	42	41	79	0.25							
				29	-5.6	35.2-36.4	Clay, br gray							46		0.32[3]							
				31	-7.6	37.6-38.1	Clay, blue gr							41	75	0.14							
				33	-10.4	40.1-41.2	Clay, blue gr							51		0.20[3]							
648+00	CL RR	M-12	30.6	3	25.8	4.1- 5.5	Clay, brown	8	22	70	75	24	51	62	59	0.13				0.42	0.30	0.11	2.63
				14 (top)										25	88	0.38							
				14 (mid)	12.7	17.3-18.5	Clay, org, blue							33	83	0.36	Tests made for comparison						
				14 (bot)										34	80	0.31							
				16	10.5	19.6-20.8	Clay, blue				85	24	61	46	72	0.36				0.38	1.90	0.47	2.66
				24	-0.8	30.3-32.6	Clay silt, brown, gray	10	65	25				33	85	0.25				0.31	2.40	0.74	2.69
				30	-10.9	40.8-42.3	Clay, blue	10	10	80	82	25	57	37						0.41	2.40	1.46	2.70
				31	-12.5	42.3-44.0	Clay, blue							44		0.15[3]							
				32	-14.1	44.0-45.4	Clay, blue							45		0.13[3]							
664+00	CL RR	M-13	31.5	3	28.5	2.4- 3.6	Clay, org, br	15	25	60	83	24	59	59									
				8	22.4	8.5- 9.7	Clay, org, brown, gray	6	18	76	75	22	53	49	73	0.19				0.40	1.00	0.21	2.70
				17	11.0	19.8-21.1	Clay, org, brown, blue	4	9	87	101	23	78	53	68	0.26				0.38	1.40	0.48	2.67
				20	7.2	23.7-24.8	Clay, br, blue							44		0.16[3]							
				24	2.4	28.5-29.8	Silty clay, gr	7	55	38	41	18	23	30	98	0.30				0.25	0.90	0.70	2.70
				31	-6.3	37.2-38.4	Clay, blue	8	23	70	75	25	50	46						0.30	0.59	0.89	2.73
				32	-7.5	38.4-39.6	Clay, blue							46		0.28[3]							
801+00	CL RR	M-14	56.6	8	48.1	8 - 9	Silty clay, org, blue	15	42	43	49	12	37	36									
				15	41.1	15 -16	Clay, brown	7	46	57	69	23	46	37									
				19	37.1	19 -20	Clay, brown, blue	12	34	54	60	26	34	31	93	0.51							
				22[4]	34.1	22 -23	Clay, org, brown, blue	12	32	56	64	14	50	36									
				28	27.1	29 -30	Clay, org, brown, blue	9	30	61	86	18	68	39	78	0.25							
				29	25.1	31 -32	Clay, org, brown, blue							28			91	6°	0.20				
				31	23.1	33 -34	Clay silt, brown, blue	9	64	27	40	13	27	32	86	0.26							
				35	18.1	38 -39	Silty clay, brown, blue	6	51	43	55	21	34	30									
				43	9.1	47 -48	Clay, blue	7	18	75	76	16	60	45									

(Continued)

Table 1 (Continued)

Station	Range	Boring No.	Boring Elev mGl	Sample No.	Sample Elev mGl	Depth ft	Soil Classification	% Sand	% Silt	% Clay	LL	PL	PI	Nat. WC %[1]	Unconfined Compr Tests γ_d lb/cu ft	Unconfined Compr Tests C tons/sq ft	Quick Triaxial Tests γ_d lb/cu ft	Quick Triaxial Tests ϕ^o	Quick Triaxial Tests C tons/sq ft	Consolidation Tests[2] C_c	P_c tons/sq ft	Over-burden tons/sq ft	Sp Gr
819+83	CL RR	M-15	54.4	9	45.3	8.5-9.7	Clay, gray	7	30	63	80	25	55	41	78	0.41							
				15	38.6	15.2-16.3	Clay, org, blue	10	25	65	82	22	60	41	77	0.46	77	4^o	0.45[6]				
				17[4]	36.3	17.4-18.8	Clay, br, gr	5	28	67	76	17	59	39	83	0.40	81	5^o	0.36				
				22	30.9	23.0-24.0	Clay, br, gr							36	78	0.56							
				26	26.1	27.7-28.8	Clay, org, gr	5	19	76	93	25	68	40	76	0.24							
				29	22.7	31.1-32.3	Clay, gr, br	10	30	60				37	79	0.22	75	3^o	0.22				
				32	19.7	34.0-35.3	Clay, br, gr	8	41	51	66	11	55	37	83	0.41	83	4^o	0.38				
				34	17.3	36.5-37.7	Silty clay, brown, gray	10	57	33	44	18	26										
				39	11.0	42.9-43.9	Clay, br, gr	6	28	76	92	24	68	40	74	0.20							
				45	4.0	49.8-50.9	Silty clay, brown, gray	13	53	34	40	16	24	26									
819+47	43' west CL RR	M-16 (Auger)	42.5	2	35.0	7.5	Clay, brown	7	33	60	72	21	51	35									
				4	27.5	15.0	Clay, brown	9	30	61	83	24	59	41									
821+60	43' west CL RR	M-17 (Auger)	41.6	1	39.1	2.5	Clay, brown	8	32	60	72	19	53	39									
				3	30.6	11.0	Clay, org, blue	6	30	64				42									
				4	25.6	16.0	Clay, brown	12	21	67	90	28	62	53									
823+75	43' west CL RR	M-18 (Auger)	43.0	2	37.5	5.5	Clay, brown	4	26	70	79	23	56	50									
				3	33.0	10.0	Silty clay, brown	10	50	40	83	28	55	55									
				4	27.0	16.0	Clay, brown	11	36	53	67	20	47	43									

Notes:
1 Water contents are those of test specimens. Water contents are based on dry weight.
2 Sample thickness = 1.20 in., drained top and bottom.
Average time for 50% consolidation of clay samples tested was approximately 40 minutes for a load of 1.5 ton/sq ft.
3 5-in.-diameter test specimens.
4 Samples listed following this notation are foundation samples.
5 Ultimate strengths by quick direct shear tests:
Boring M-4 sample 20 $\phi = 6^o$, C = 0.60 tons/sq ft.
Boring M-4 sample 37 $\phi = 3.5^o$, C = 0.50 tons/sq ft.
6 Consolidated quick triaxial comp. test:
Boring M-15 sample 15: $\phi = 8^o$, C = 0.55 tons/sq ft.

Table 2

Summary of Laboratory Test Data - Embankment Foundation, 1948 Borings

Boring Number	Boring Station	Sample No.	Sample Elev msl	Sample Depth ft	Soil Classification	% Sand	% Silt	% Clay	LL	PL	PI	Nat. w %	Shear Tests Type	w %	γ_d lb/cu ft	ϕ^o	C T/sq ft	Consolidation Tests C_c	P_c T/sq ft	P_o T/sq ft	Sp Gr	w %	T_{50} min	K_h 10^{-7} cm/sec
M-20	728+50	5	27.1	5.5	Clay, gray, firm							40	UC	45	73		0.30							
													UC	44	76		0.38*							
Ground elev 32.6 msl		9	22.6	10.0	Silty clay, gray, firm	11	44	45				41	UC	32	89		0.48							
		11	18.9	13.7	Clay, gray, firm							47	UC	43	75		0.44							
		16	13.5	19.1	Clay, gray and brown, stiff	5	20	75				47	UC	44	76		0.55							
		21	7.6	25.0	Clay silt, gray and brown, firm							32	UC	29	91		0.37							
		36	-7.6	40.2	Clay, gray, firm							62	UC	62	63		0.29							
		39	-10.4	43.0	Clay, gray, firm	6	6	88	117	29	88	67	UC	66	59		0.32							
		43	-15.4	48.0	Clay, gray, firm							62	UC	66	59		0.37							
		52	-47.9	80.5	Silty clay, gray, stiff							32	UC	37	82		0.59							
M-21	738+50	5	27.5	5.2	Clay, gray and brown, firm	7	23	70	81	27	54	47	UC	50	70		0.29							
													UC	45	73		0.33*							
Ground elev 32.7 msl		12	18.2	14.5	Clay, gray, firm							47	UC	49	72		0.26							
		17	7.4	25.3	Clay, gray, firm	8	26	66	76	23	53	43	UC	41	79		0.42							
		25	-3.2	35.9	Clay, gray, stiff							42	UC	38	82		0.53							
		31	-10.2	42.9	Clay, gray, soft	8	5	87	102	29	73	59	UC	71	57		0.24							
		38	-21.7	54.4	Clay, gray, soft	10	30	60				46	UC	37	83		0.24							
		40	-31.8	64.5	Clay, gray							44	UC	52	69		0.26							
M-22	758+50	4	29.5	4.7	Silty clay, gray, firm							34	UC	32	85		0.29							
													UC	33	86		0.33*							
Ground elev 34.2 msl		5	28.3	5.9	Clay silt, gray, soft	14	57	29				32	UC	32	86		0.14							
		6	27.1	7.1	Clay silt, gray, very soft							36	UC	31	89		0.12**							
		7	25.5	8.7	Clay silt, gray, firm							34	UC	31	89		0.32							
		10	21.6	12.6	Clay, gray							47	UC	45	75		0.29							
		11	20.3	13.9	Clay, gray, soft							45	UC	44	76		0.19**							
		13	18.0	15.6	Clay, gray, 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, gray, firm							48	UC	45	75		0.41							
		18	11.8	22.4	Clay, gray, soft							48	UC	46	75		0.22**							
		19	10.5	23.7	Clay, gray, firm	7	21	72				53	UC	51	71		0.29							
		22	7.2	27.0	Clay, gray, firm							40	UC	44	76		0.33							
		25	3.3	30.9	Clay, gray and 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, gray, firm							46	UC	40	81		0.34**							
		28	0.0	34.2	Clay, gray, stiff							34	UC	39	82		0.52							
		30	-2.6	36.8	Clay, gray, firm							45	UC	36	82		0.25**							
		31	-3.7	37.9	Clay, gray and brown, firm							49	UC	44	76		0.46							
		37	-10.8	45.0	Clay, gray, 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, gray, firm	11	14	75				56	QT	52	69	2^o	0.30							
													UC	71	56		0.35							
		42	-19.3	53.5	Clay, gray, firm							42	UC	50	71		0.29							
		43	-25.2	59.4	Clay, gray, firm							48	UC	40	80		0.44							
		46	-32.9	67.1	Clay, gray, stiff	6	34	60	84	26	58	41	QT	37	83	1^o	0.50							
													UC	40	80		0.55							
		50	-43.7	77.9	Clay, gray, stiff	6	27	67	76	29	47	44	UC	39	80		0.53							
M-23	778+50	8	24.2	9.9	Clay, gray, firm							43	UC	30	92		0.37*							
		9	23.6	10.5	Clay, gray, firm	8	40	52	66	31	35	45	UC	39	79		0.28							
Ground elev 34.1 msl		12	19.6	14.5	Clay, gray, firm							36	UC	36	84		0.36							
		15	15.4	18.7	Clay, gray, firm							46	UC	43	76		0.27							
		22	5.6	28.5	Clay silt, gray, firm							34	UC	31	89		0.42							
		26	-0.2	34.3	Silty clay, gray, firm	10	43	47				40	UC	34	86		0.45							
		34	-10.4	44.5	Clay, gray, firm							55	UC	53	67		0.34							
		42	-35.4	69.5	Silty clay, gray, stiff							38	UC	36	83		0.54							
M-24	786+00	3	23.6	10.5	Silty clay, gray, firm	8	59	33	49	23	26	47	UC	39	80		0.25							
													UC	40	78		0.24*							
Ground elev 34.1 msl		6	8.6	25.5	Clay, gray, firm	11	34	55	90	23	67	35	UC	41	80		0.38							
		8	-1.5	35.6	Clay, gray, firm	4	16	80				49	UC	36	85		0.43							
		10	-11.4	45.5	Clay, gray, firm				104	32	72	51	UC	55	67		0.35							
		12	-21.0	55.1	Clay, gray, firm	20	20	60				35	UC	40	81		0.28							
		16	-41.3	75.4	Clay, gray, firm							49	UC	51	78		0.34							
M-25	798+50	4	30.9	3.8	Clay silt, gray, firm							36	UC	31	89		0.29							
													UC	28	93		0.25*							
Ground elev 34.7 msl		10	24.2	10.5	Clay, gray, 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, gray, firm							34	UC	35	85		0.28							
		18	14.7	20.0	Clay, gray, 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, gray, firm	11	41	48				37	UC	37	84		0.32							
		26	5.0	29.7	Clay silt, brown and gray	25	54	21				31	QT	31	93	15^o	0.35							

Table 2 (Continued)

Boring Number	Boring Station	Sample No.	Sample Elev msl	Sample Depth ft	Soil Classification	Mechanical Analysis % Sand	% Silt	% Clay	Atterberg Limits LL	PL	PI	Nat. w %	Shear Tests Type	w %	γ_d lb/cu ft	ϕ^o	c T/sq ft	Consolidation Tests C_c	P_c T/sq ft	P_o T/sq ft	Sp Gr	w %	T_{50} min	K_h 10^{-7} cm/sec
M-25 (Contd)		38	-7.8	42.5	Clay, gray, firm							52	UC	44	76		0.25							
Ground		43	-13.3	48.0	Clay, gray, firm	6	11	83	95	25	70	48	UC	44	76		0.29	0.36	1.4	1.3	2.70	44	24	
elev		47	-25.8	60.5	Clay, gray, firm	6	27	67	79	30	49	44	UC	38	82		0.41							
34.7 msl		49	-35.8	70.5	Clay, gray, 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, gray, firm							36	UC	34	85		0.44							
M-26	672+00	6	25.8	5.4	Clay, gray and brown, soft							55	UC	57	66		0.24							
Ground elev 31.2 msl		16	15.6	15.6	Clay, gray and brown, firm	8	18	74	90	29	61	48	UC	51	72		0.35							
		25	6.4	24.8	Clay silt, gray and brown, firm	13	64	23				31	UC	29	93		0.41							
		27	4.8	26.4	Sandy silt, gray	23	61	16				31												11.2
		31	0.5	30.7	Clay, gray and brown, firm	7	24	69				41	UC	41	78		0.39							
		41	-9.3	40.5	Clay, gray, firm	11	22	67	90	27	63	54	UC	49	70		0.44							
		45	-16.0	47.2	Clay, gray, firm	11	14	75				50	UC	47	78		0.49							
		49	-19.2	50.4	Silty clay, gray, firm							45	UC	37	83		0.44							

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. P_o based on water table at depth of 6 ft.
* Remolded specimens. ** 2.8-in.-diameter test specimen. Strength value for one test only.

Table 3

Summary of Laboratory Test Data - Control Structure Foundation, 1948 Borings

Boring Number	Boring Station	Sample No.	Sample Elev msl	Sample Depth ft	Soil Classification	Mechanical Analysis % Sand	% Silt	% Clay	Atterberg Limits LL	PL	PI	Nat. w %	Shear Tests Type	w %	γ_d lb/cu ft	ϕ^o	c T/sq ft	Consolidation Tests C_c	P_c T/sq ft	P_o T/sq ft	Sp Gr	w %	T_{50} min	K_h 10^{-8} cm/sec
M-19	721+00	9	19.7	12.8	Clay, gray, firm	10	25	65	97	29	68	51	UC	47	73		0.34	0.41	1.3	0.5	2.70	51	39	
Ground		11	16.5	16.0	Clay, gray, firm							44	UC	46	75		0.38							
elev		14	14.3	18.2	Clay, gray, firm							50	UC	44	75		0.44							
32.5 msl		19	8.5	24.0	Clay, gray, 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, gray and 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, gray, 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, gray, 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, gray, firm							45	UC	51	67		0.47*							
		42	-33.4	65.9	Silty clay, gray, 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, gray, firm	8	42	50	54	24	30	49	UC	33	86		0.38	0.35	2.9	2.0	2.70	36	8	
M-27	679+00	7	24.0	7.7	Clay, gray and brown, firm	8	19	73	100	28	72	55	UC	46	73		0.33	0.41	1.4	0.4	2.66	48	53	
		10	21.0	10.7	Clay, gray and brown, stiff							38	UC	34	86		0.54							
Ground		11	20.2	11.5	Clay, gray, firm							48	UC	43	72		0.34*							
elev		13	18.1	13.6	Clay, gray, firm	6	15	79				54	QT	47	71	3°	0.25							
31.7 msl													UC	46	73		0.25							
		16	15.5	16.2	Clay, gray and brown, firm							58	UC	39	80		0.46							
		20	10.7	21.0	Clay, gray and 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, gray and 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, gray and brown, firm							35	QT	32	89	2°	0.30							
		25	5.8	25.9	Clay, gray and brown, stiff							35	UC	28	94		0.64							
		26	5.2	26.5	Clay silt, gray, soft	26	54	20				12												57
		31	-0.1	31.8	Silty clay, gray and brown, firm							43	UC	40	80		0.39							
		34	-3.2	34.9	Clay, gray and brown, stiff	8	30	62				42	UC	37	82		0.52							
		38	-7.1	38.8	Clay, gray, firm	8	24	68				51	UC	49	72		0.38							
		40	-8.6	40.3	Clay, gray, firm							54	QT	49	72	0°	0.40							
		41	-10.1	41.8	Clay, gray, 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, gray, 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, gray, firm							73	UC	70	57		0.39							
		47	-15.8	47.5	Clay, gray, firm							45	UC	43	76		0.39							
		50	-23.2	54.9	Clay silt, gray, 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, gray, firm	12	38	50	74	25	49	37												
Ground elev 31.7 msl		12	-11.8	43.5	Clay, gray, soft	8	7	85	113	33	80	67												
M-31	699+00	8	21.5	10.5	Silty clay, gray and brown, soft				64	23	41	42	UC	32	85		0.23							
Ground		10	19.5	12.5	Clay, gray, soft							51	UC	43	76		0.22							
elev													UC	48	72		0.33*							
32.0 msl		12	17.7	14.2	Clay, gray and brown, soft							44	UC	44	74		0.20							
													UC	41	77		0.28							
		14*	15.7	16.3	Clay, gray and brown, firm							44	QT	43	76	1°	0.35							
		16	13.6	18.4	Clay, gray and brown, soft							52	UC	51	70		0.20							
		18	11.5	20.5	Silty clay, gray and brown, firm							43	UC	48	71		0.27							
		20	9.5	22.5	Silty clay, gray, soft	12	56	32	48	21	27	33												350
		22	7.5	24.5	Clay silt, gray and brown, firm							32	UC	32	89		0.42							
		23	6.5	25.5	Clay silt, gray and brown, soft	11	61	28				32												6
		26	1.6	30.4	Sandy silt, gray	13	70	17				34												580
		31	-4.0	36.0	Silty clay, gray, firm	3	52	45	70	23	47	41	UC	41	82		0.35							
		35	-9.0	41.0	Clay, gray, firm							64	UC	55	65		0.28*							
		36	-10.5	42.5	Clay, gray, firm							69	UC	20	81		0.25							
		38	-12.6	44.6	Clay, gray, soft	5	8	87	102	31	71	67	UC	70	58		0.24							
		39	-14.5	46.5	Clay, gray, soft							69	UC	69	57		0.21*							
		40	-15.5	47.5	Clay, gray, firm							57	UC	67	58		0.23*							
		41	-17.2	49.2	Clay, gray, firm							54	UC	55	67		0.42							
		43	-22.5	54.5	Clay, gray, 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 elev 32.5 msl																								
M-35	717+50	2	24.4	8.0	Clay, brown, firm				88	29	59	42												
Ground elev 32.4 msl																								
M-37	700+00	3	28.3	3.7	Clay, gray and brown, soft							49	UC	42	75		0.22							
		4	27.3	4.7	Clay, gray and brown, soft	5	21	74	95	25	70	45	UC	44	72		0.24							
Ground elev 32.0 msl	115 ft west of center line of rail-road	6	23.0	9.0	Clay, gray and brown, firm	8	25	67				45	UC	37	81		0.48							
		7	20.8	11.2	Clay, gray and brown, firm	8	39	53	70	23	47	43	UC	34	85		0.46							
		9	16.0	16.0	Clay, gray and brown, firm							34	UC	47	72		0.29							
		10	13.8	18.2	Clay, gray and brown, soft	10	16	74				31	UC	50	70		0.24							
		13	10.3	21.7	Clay silt, gray, 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. P_o based on water table at depth of 6 ft.
* Remolded specimens.

Classification

20. Atterberg limits tests, mechanical analyses, and water content tests indicated that most of the soils underlying the embankments and the control structure are fat, highly plastic, inorganic clays, with high water contents. The range in liquid limits and clay content (particles smaller than 0.005 mm) obtained from samples taken from the foundation soils beneath the embankments and control structure is shown in fig. 10. The correlation obtained between clay content and liquid limit is also illustrated by this figure. Typical grain-size curves for foundation soils are shown in fig. 11.

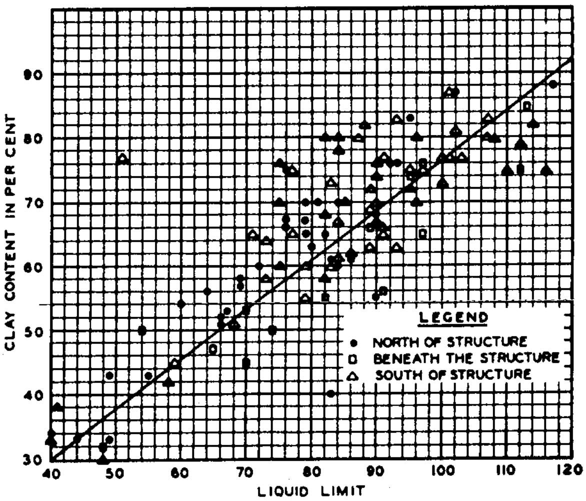


Fig. 10. Clay content versus liquid limit

Shear strength

21. Forty unconfined compression tests on undisturbed samples from the clay and silty clay strata beneath the control structure gave an over-all average cohesive shear strength of 0.33 tons per sq ft. An average strength of 0.31 tons 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 through remolding as the result of pile driving. Four unconsolidated, undrained triaxial compression tests on samples from the foundation along the structure indicated an average strength of $\phi = 2^{\circ}$ and c = 0.32 tons per sq ft.

22. Sixty-eight unconfined compression tests made on samples taken beneath the north embankment gave an average cohesive shear strength of 0.35 tons per sq ft; two unconsolidated, undrained triaxial tests from these borings gave an average strength of $\phi = 1^{\circ}$ and c = 0.40 tons per sq ft. The average shear strength of the foundation borings under the

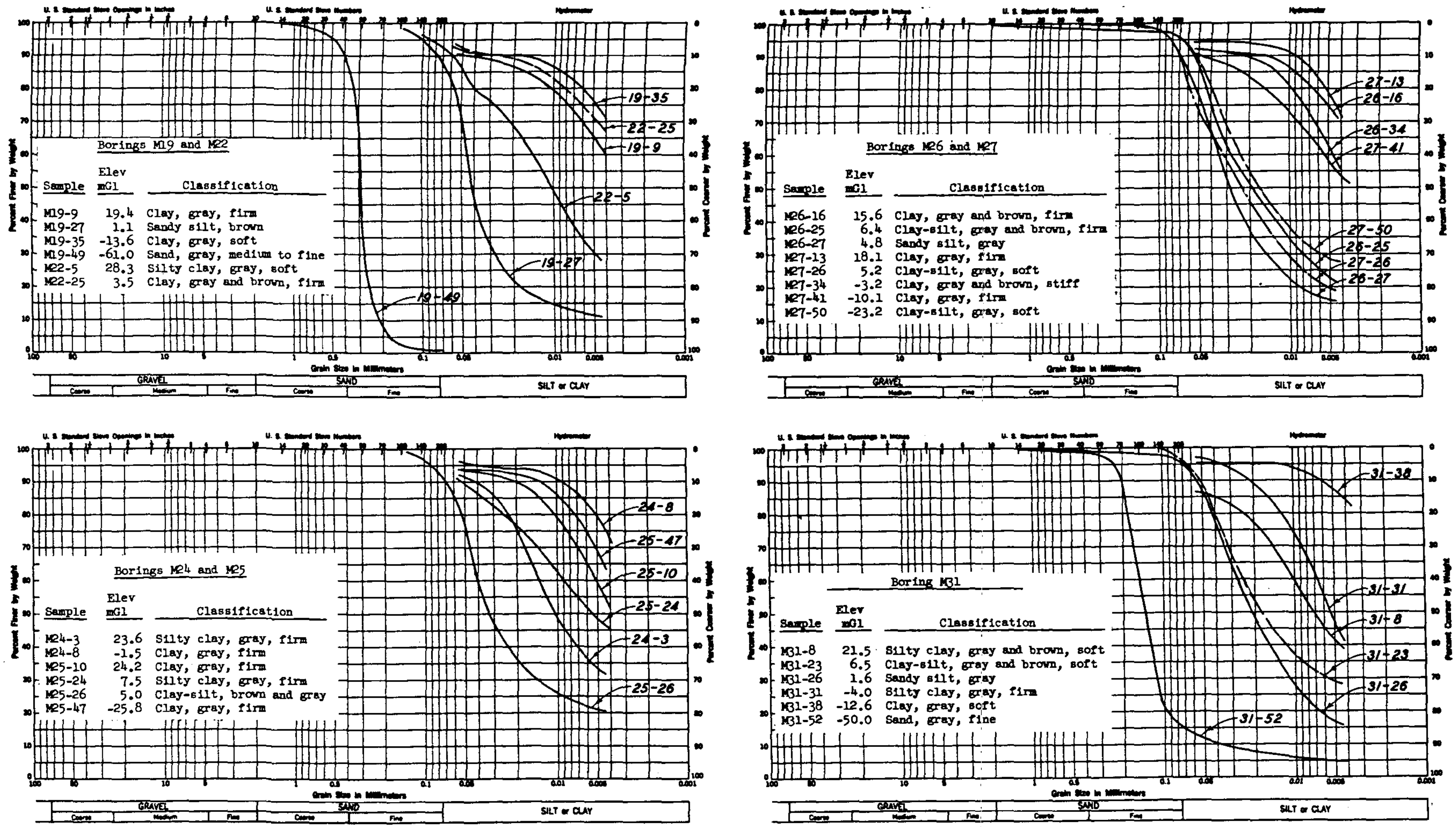


Fig. 11. Grain-size curves of foundation soils

north embankment, the control structure, and from station 672 to 680 under the south embankment ranged from about 0.30 to 0.40 tons per sq ft. This same range of shear strength was indicated from the ground surface to about elev -20. A shear strength of $\phi = 0^{\circ}$ and c = 0.30 tons per sq ft was selected for design purposes for the foundation from the north end of the north embankment to station 672 on the south embankment. From this station to the south end of the south embankment, a shear strength of c = 0.25 tons per sq ft was selected for design purposes.

Consolidation tests

23. Consolidation tests on foundation samples under the control structure and the embankment indicate that the foundation is very compressible. The compressibility of the foundation was verified by 2- to 4.7-ft settlements under the portions of the embankments previously constructed by the Louisiana Department of Highways in 1941-43. Twelve consolidation tests on undisturbed clays and silty clays from the foundation, excluding a thin high-water-content stratum, beneath the control structure and embankments gave an average compression index (C_c) of 0.40. Tests on samples from the thin high-water-content clay stratum (elev -9 to -16) gave an average C_c of 0.68. The preconsolidation pressures (P_c) as determined from the test data indicate that the soils above elev 0 are overconsolidated, as the preconsolidation pressures exceeded the computed existing overburden pressures by about 0.5 to 1.0 ton per sq ft, assuming a water table at 6 ft below the ground surface. The brownish, oxidized color of these materials indicates overconsolidation by alternate wetting and drying. Pressure-void ratio curves obtained from the consolidation tests made on samples from the foundation beneath the control structure and embankments are shown in figs. 12 and 13, respectively.

Permeability

24. Four permeability tests on the sandy silt and clay silt stratum from elev +9 to -1 along the control structure indicate horizontal permeabilities of (60 to 580) x 10^{-8} cm per sec. As some portions of the stratum are more granular than the specimens tested, a horizontal permeability of 1000 x 10^{-8} cm per sec was used in the underseepage analysis.

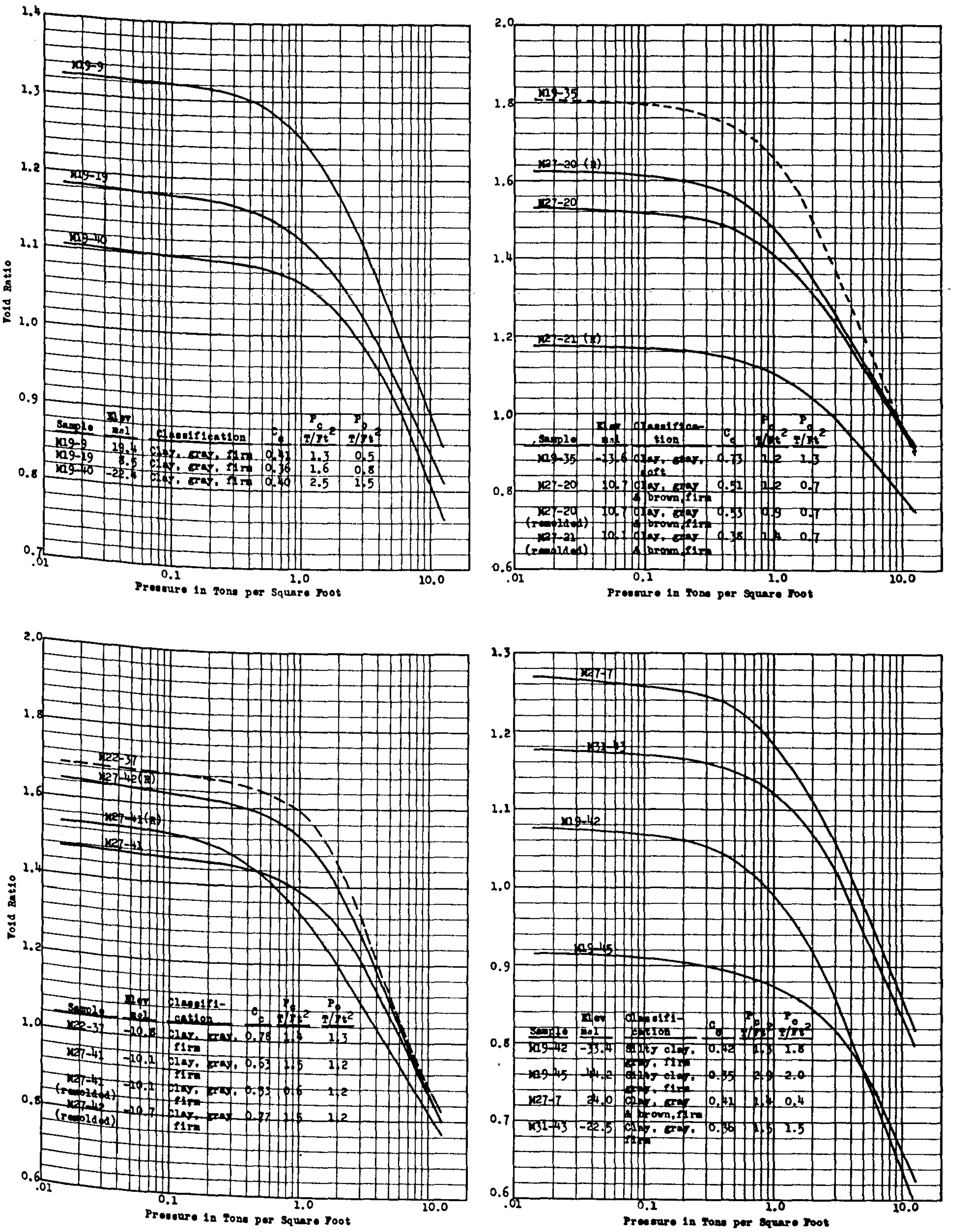


Fig. 12. Consolidation curves, control structure foundation

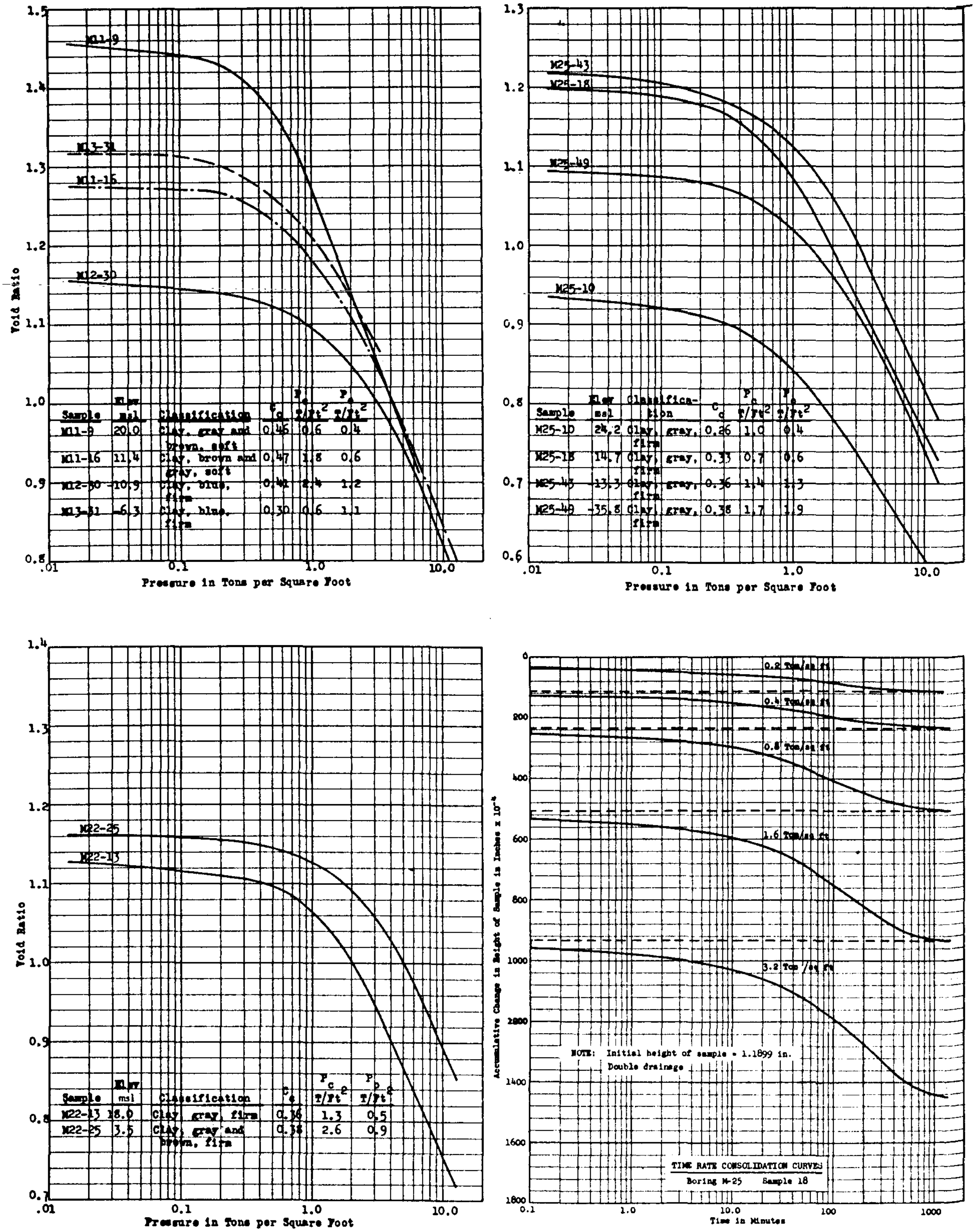


Fig. 13. Consolidation curves, embankment foundation

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 (0.5 to 2.5) x 10^{-8} cm per sec. A maximum value of 2.5 x 10^{-8} cm per sec was used in the underseepage computation. A permeability ratio of $\frac{k_h}{k_v}$ = 16 was assumed for the upper clay stratum.

Specific gravity

25. The specific gravity of the samples tested varied from 2.62 to 2.72 with an average of about 2.70.

Effect of remolding

26. A comparison of the shear and consolidation tests that were run on undisturbed and remolded specimens indicated that the foundation clay has a sensitivity ratio of about 1.0. Consolidation tests run on remolded samples indicated compression indices almost identical with those of undisturbed samples. Therefore, there should be little or no tendency for the clay, remolded by driving of the foundation piles, to consolidate and settle away from the base of the structure or cause downward drag on the piles.

Density of foundation sands

27. The sand underlying the clay strata along the control structure is a uniformly graded fine sand composed of subrounded to subangular grains. Driving of a standard 1-3/8-in. ID split-spoon sampler with a 30-in. drop of a 140-lb hammer indicated that the sand has an average driving resistance of about 55 blows per ft. The relative density of the sand as determined by laboratory tests is about 70 to 90 per cent.

Borrow Materials for Embankments

28. Most of the material used in the construction of the embankments consisted of fat, inorganic plastic clays and silty clays having a high water content. The only exception was the materials in borrow area "A" where the soils were leaner than those found in the other areas. In general, the average water content of the borrow materials ranged from about 28 to 35 per cent. The liquid limits of the borrow materials

for the embankment proper ranged from about 40 to 60 per cent. Some materials used in construction of the berm had liquid limits as high as 85 per cent. A summary of the laboratory tests on the borrow materials is given in table 4.

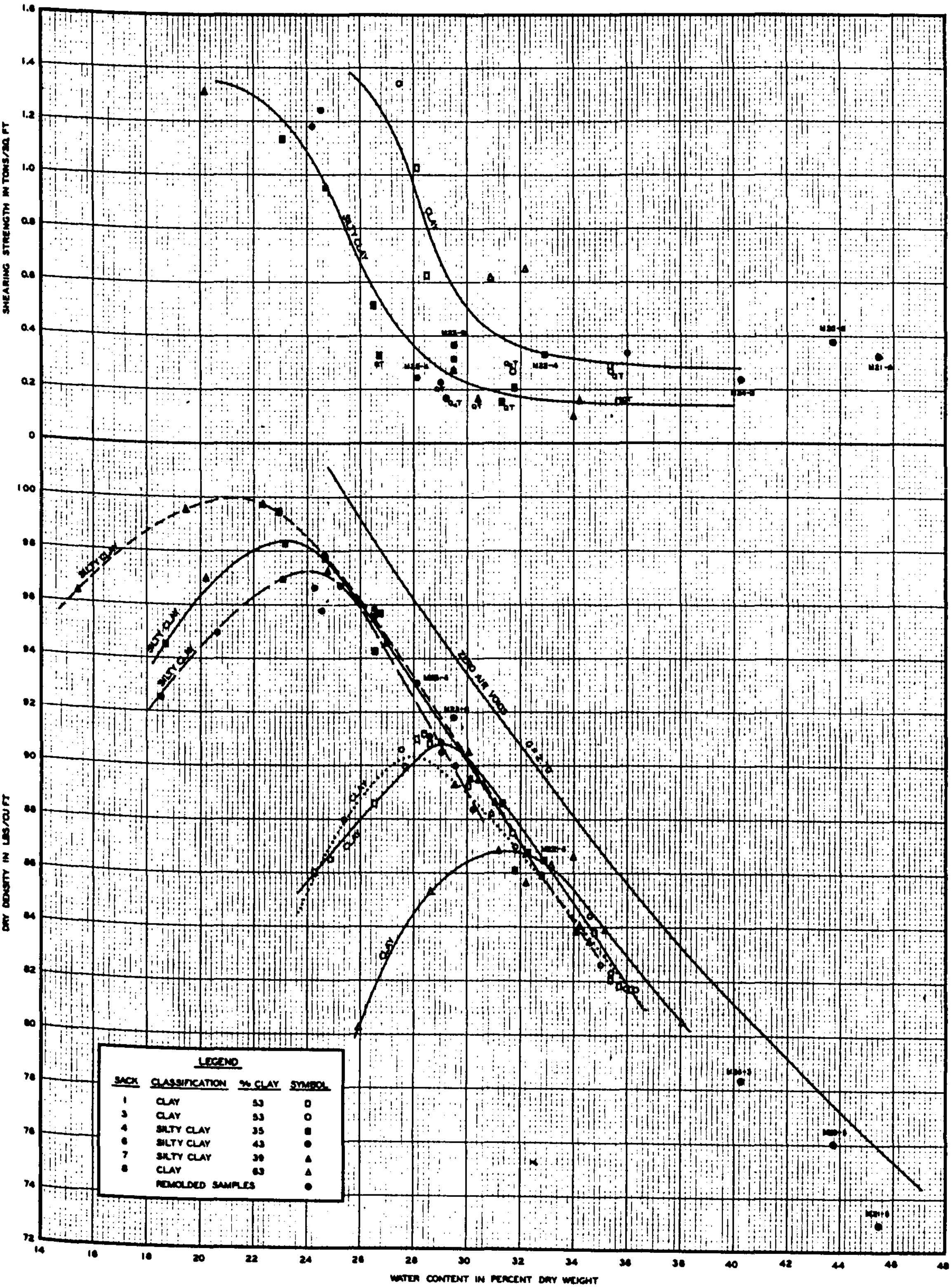
Compaction

29. Five standard, Proctor compaction tests, modified by employing a 15-blow compactive effort, were run on sack samples taken from borrow area "D." Results of these compaction tests are shown in fig. 14. The tests indicate optimum water contents of about 23 to 30 per cent, and maximum dry densities of 98 and 89 lb per cu ft for the silty clay and clay materials, respectively.

Shear strength

30. Shear strength tests were run on soil specimens compacted at water contents ranging between natural and optimum; the strengths obtained are plotted against water contents on fig. 14 and are summarized in table 4.

31. An analysis of water contents of the predominant materials in the borrow areas indicated that the clay materials would probably be placed at an average water content of approximately 35 per cent and that the silty clay materials would probably be placed at an average water content of approximately 32 per cent. On the basis of the data shown in fig. 14 and moisture contents of the borrow soils, a shear strength of 450 lb per sq ft was selected as the design strength of the embankment south of station 672 and 500 lb per sq ft for the embankment north of station 672. These strengths were considered reasonable inasmuch as the shearing strengths obtained from undisturbed samples remolded at natural water content were of this magnitude, as is shown on fig. 14. This would indicate that as long as the lumps of borrow material are pressed together by the compactive process so that the void space is negligible, a shearing strength of between 400 and 500 lb per sq ft should be obtained. No frictional resistance was assumed, although quick triaxial tests gave a frictional angle of about 2 degrees for the borrow soils.



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 SYMBOL.

Fig. 14. Compaction curves and maximum shearing strengths of borrow materials

Table 4

Summary of Laboratory Test Data, Embankment and Borrow Materials

Sack Sample Number	Boring Number	Soil Classification	Mechanical Analysis % Sand	% Silt	% Clay	Atterberg Limits LL	PL	PI	15-Blow Proctor Opt. w %	γ_d Lb/Cu Ft	Shear Tests 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^o	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^o	0.20		
											QT	"	35	82	2^o	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^o	0.33		
											QT	"	31	89	0^o	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^o	0.22		
											Q_cT	"	29	91	14^o	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^o	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.

PART IV: EMBANKMENT FOR LEVEE, HIGHWAY, AND RAILROAD

Design of Embankments

Shear strength of foundation and embankments

32. A foundation shear strength of $\phi = 0^{\circ}$ and $c = 0.30$ tons per sq ft was selected for the design of all embankments north of station 672; a foundation shear strength of $\phi = 0^{\circ}$ and $c = 0.25$ tons per sq ft was employed in the design of the embankment south of station 672.

33. An embankment shear strength of 500 lb per sq ft was used for the embankments north of station 672 and 450 lb per sq ft for the embankments south of station 672.

34. Although the soils available in the immediate vicinity of the structure were not particularly suitable for a high embankment, the flat slopes and berms necessary because of the poor foundation lessened the importance of the requirements of good material, considering the over-all stability of the embankments. The type of material used in the construction of the berms was relatively unimportant because they serve only as counterbalancing weight.

Stability analyses

35. The circular arc method of analysis was employed in the design of the embankments. 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 and expected water contents, the unit wet weight of the embankment material was taken to be 110 lb per cu ft.

36. Since the embankment material is subject to considerable shrinkage, and since cracks may develop that will reduce the effective length of the failure arc, the minimum factor of safety used in the analysis of shallow slides confined principally within the embankment was 1.5. Such cracking does not significantly affect the analysis of a foundation failure. In order to reduce surface cracking to a minimum, a 1-1/2- to 3-ft layer of sandy or silty material was used to top the embankment.

37. For deep slides, a minimum factor of safety of 1.2 was selected. Although this factor of safety is slightly below that normally required for embankments, the stage construction employed permitted the foundation clays to consolidate during the construction period which should result in some gain in strength of the foundation. A larger factor of safety would have resulted in excessively large sections and berms, and would have increased materially the yardage required for the embankments.

38. Ballast and train loads were not explicitly taken into account in the design of the embankments because their effects are of minor importance, insofar as deep failures are concerned, amounting to only 2 to 5 per cent of the total driving forces. These loads also were not added until after the second construction period by which time considerable consolidation was expected to have occurred.

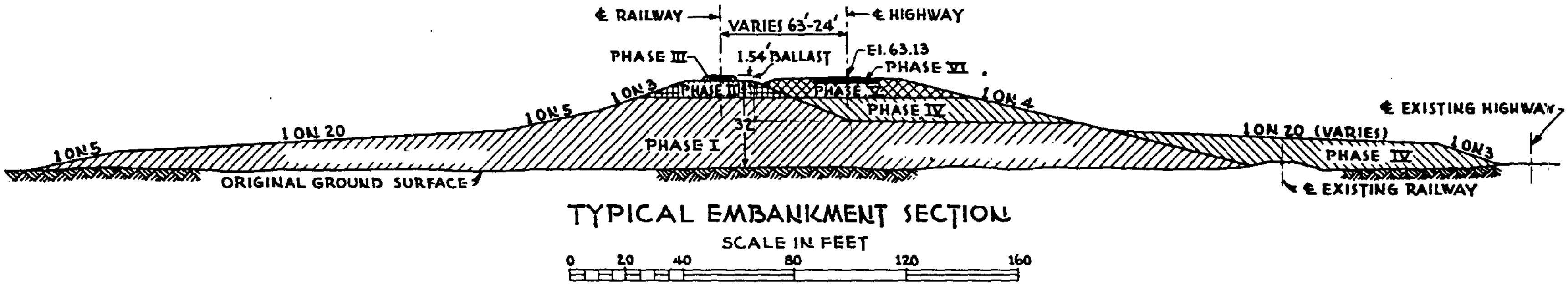
39. A typical section of the south embankment where the foundation shear strength was taken to be 500 lb per sq ft is shown at the top of fig. 5; shown at the bottom of this figure is a typical section of the north embankment where the foundation shear strength was taken to be 600 lb per sq ft. Factors of safety for these sections are also shown on fig. 5, page 5.

Construction of Embankments

40. As the berms of the new embankment generally covered the existing railroad, it was necessary to construct the embankment and relocate the railroad and highway in six phases in order to prevent interruption of traffic. The different phases of construction of the embankments are illustrated in fig. 15. The work carried out under each phase, together with the starting and completion dates of each, is summarized in table 5.

Preparation of embankment foundation

41. Areas to be covered by the embankment were stripped or excavated so as to remove all loose organic material, brush, or other trash subject to decay. All rotten stumps and roots were grubbed and removed; sound stumps were not grubbed. The foundation for the



ORDER OF CONSTRUCTION

PHASE I	INITIAL EMBANKMENT	PHASE IV	COMPLETION OF EMBANKMENT ON EAST SIDE
PHASE II	RAILWAY SUBGRADE	PHASE V	HIGHWAY RELOCATION
PHASE III	RAILWAY RELOCATION	PHASE VI	COMPLETION OF HIGHWAY

Fig. 15. Phases for construction of embankment and relocation of highway and railroad

Table 5

Construction Phases and Costs for Construction of Embankments and Relocation of Highway and Railroad

Phase	Description	Starting Date	Completion Date	Quantities cu yd	Unit Price	Costs
I	Initial embankment	27 Oct 1949	3 Oct 1951	3,180,000	$ 0.34	$1,295,400
II	Railway subgrade	27 June 1951	8 Nov 1951	92,000	1.09	103,225*
III	Railway location	Sept 1952	April 1953	Job	----	432,395**
IV	Completion of embankment on east side	16 June 1953	Dec 1953	1,140,000	0.37	421,800
V	Highway relocation	20 Oct 1953	2 March 1954	117,000	0.85	99,450
VI	Completion of highway***	April 1954	------------	-------	----	350,500
				Approximate total cost		$2,702,770

* Includes cost of seeding and sodding.

** Through Dec 1953. Small outstanding costs and maintenance costs to April 1954 not included.

*** Highway under construction at time report was prepared.

embankment was thoroughly disked and scarified before any fill was placed.

Borrow areas

42. Borrow areas used in the various phases of embankment construction are shown in fig. 1. A minimum distance of 100 ft was maintained between the toes of the embankment and the nearest edge of the borrow pits. Borrow pits were kept drained so that all excavation operations were carried out above ground-water level; no underwater excavation for borrow was permitted.

43. Borrow for the embankments was excavated from the top to the bottom of the pit in order that the varying strata of earth would be mixed when the material was placed in the embankment.

Placement of fill

44. Fill for the embankments was removed from the borrow areas by means of draglines and was transported to the embankment in 13-cu-yd Euclids and tractor trailers. It was spread on the embankment by bulldozers to an approximate depth of 12 in. of loose material. The layers were built the full width of the embankment. Material for the western berms was obtained from borrow pits located west of the embankment, whereas materials for other portions of the embankment were obtained from borrow pits located east of the embankment.

45. Because of the plastic nature of the borrow materials, it was contemplated in the design of the embankment that these materials would be placed at essentially the natural water content of the material in the borrow pit. Generally, the borrow materials were considered dry enough for compaction when the ground and embankment were dry enough to support the equipment used for hauling and spreading the material without unreasonable cutting up of the embankment already in place. After the material had been spread in loose lifts approximately 12 in. thick it was compacted by at least three passes of a D-8 tractor weighing 21 tons and exerting a unit pressure of 10 psi. A certain amount of additional compaction was obtained by traffic of the hauling equipment. This procedure resulted in about as good compaction as could be obtained considering the high water content and plastic nature of the clay fill material.

46. Where the embankment crossed bayous, the bayous were dammed off and any boggy or unsuitable material was removed before any fill was deposited. The work was done during a dry season and there was no water in the bayous.

47. No water content or density determinations were made of the fill material as it was placed.

Overbuilding

48. As will be discussed subsequently, the settlement of the foundation for the embankment was estimated at 3.5 to 5 ft. In order to obtain the net section desired from the construction of phase I, the inner section was overbuilt by 3 ft at the top of the embankment, tapered to 1-1/2 ft at a distance of 77 ft from the railroad center line, and 0.5 ft at the crown of the berm. By the end of construction of phase II of the embankment, it was estimated that the foundation would probably have settled approximately 1.5 to 2 ft; therefore, the final net section of the railroad portion of the main embankment was overbuilt 2.0 to 2.5 ft to compensate for future settlement of the foundation.

Table 6

Estimated Settlement and Overbuild of Railroad Portion of Embankment

January 1951

Sta	Computed Foundation Settlement for Net Embankment Section	Assumed Embankment Consolidation and Foundation Settlement Due to Overbuilding	Estimated Total Settlement of Railroad Crown	Estimated Settlement by 1 January 1952	Overbuild Estimate January 1951		As Constructed		Gross Grade October 1953
					Computed	Recommended	Gross Grade	Overbuilt ft	
630	4.2 ft	0.5 ft	4.7 ft	1.5 ft	3.2 ft	2.5 ft	64.3	2.7	62.4
664	3.5 ft	0.5 ft	4.0 ft	2.1 ft	1.9 ft	2.0 ft	63.6	2.0	62.2
758	4.0 ft	0.5 ft	4.5 ft	1.7 ft	2.8 ft	2.5 ft	64.4	2.8	63.2
798	5.0 ft	0.5 ft	5.5 ft	2.5 ft	3.0 ft	2.5 ft	64.2	2.6	63.1

Note: Net grade of embankment (top phase II) before ballast = 61.6 msl.

Overbuilding of the final section was restricted to a maximum of 2.5 ft so as not to overload the foundation. Although no detailed computations were made, overbuilding of the embankment during the final phase of construction was estimated to increase the total foundation settlement to 4.5 to 5.5 ft.

Pore Pressure in Embankment and Foundation

49. In order to determine pore water pressures caused by construction of the fill, five piezometers were placed in the embankment and foundation at four different stations (630, 664, 758, and 798) as shown in table 7 and fig. 16. In addition, a sixth piezometer was installed at each of these stations 30 ft from the landside toe of the embankment

Table 7

Location of Engineering Measurement Devices

Engineering Measuring Device	Number	Station	Distance from Center Line of Railroad Ft	Initial Elevation Settlement Plate or Piezometer Tip Ft - MSL	Initial Elevation Top of Riser or Gage Ft - MSL
Embankment and foundation piezometers	1	630+00	16 US	-10.00	31.79 (gage)
	2		20 US	+20.00	31.79 "
	3		18 US	+35.60	31.79 "
	4		62 DS	-10.01	31.79 "
	5		58 DS	+20.00	31.79 "
	6		290 DS	+20.00 ±	31.43 (riser)
	1	664+00	16 US	-10.01	32.16 (gage)
	2		20 US	+20.00	32.16 "
	3		18 US	+36.20	32.16 "
	4		62 DS	-10.03	32.16 "
	5		58 DS	+20.00	32.16 "
	6		290 DS	+19.30	32.38 (riser)
	1	758+00	16 US	-10.01	34.61 (gage)
	2		20 US	+20.03	34.61 "
	3		18 US	+38.77	34.61 "
	4		62 DS	-10.00	34.61 "
	5		58 DS	+20.00	34.61 "
	6		290 DS	+21.44	34.55 (riser)
	1	798+00	16 US	-10.00	35.14 (gage)
	2		20 US	+20.01	35.14 "
	3		18 US	+39.50	35.14 "
	4		62 DS	-9.99	35.14 "
	5		58 DS	+20.01	35.14 "
	6		290 DS	+22.05	35.24 (riser)
Settlement plates beneath embankment	1	630+03	95 US	29.92	60.10
	2	630+03	18 US	29.96	65.00
	3	630+03	60 DS	29.75	50.84
	4	664+04	95 US	30.33	60.20
	5	664+04	18 US	30.76	65.81
	6	664+04	60 DS	30.52	54.54
	15	758+03	95 US	33.05	61.66
	16	758+03	18 US	33.34	67.04
	17	758+03	60 DS	34.56	58.66
	18	798+03	95 US	33.87	61.26
	19	798+03	18 US	33.96	66.33
	20	798+03	60 DS	34.17	57.54
Settlement hubs on structure	1, 2, 3, & 4	See fig. 39	See fig. 39	See fig. 39	---
Piezometers beneath structure	A, B, & C	See fig. 39	See fig. 39	See fig. 39	See fig. 39
Settlement plates beneath weir	A & B	See fig. 39	See fig. 39	See fig. 39	See fig. 39
Settlement plates beneath riprap	A	690+00	Plates A, B, and C are	29.69	32.18
	B	700+00	20 ft upstream of up-	28.85	31.35
	C	710+00	stream edge of weir	29.18	31.69

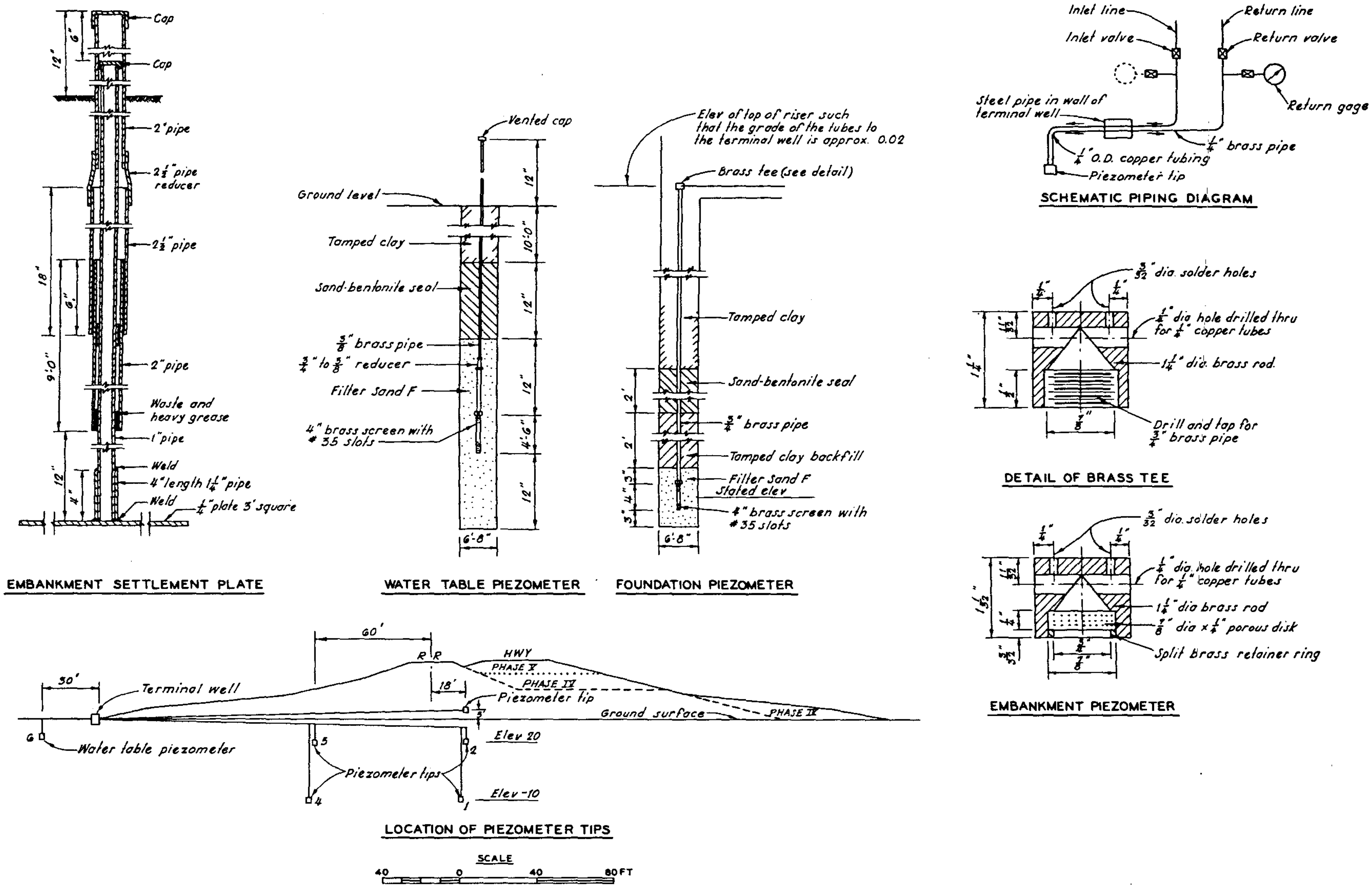


Fig. 16. Piezometers and settlement plates

in order to determine the natural water table. All the piezometers except those used to determine the natural ground water table were of the closed system type. The piezometer tips installed in the foundation beneath the embankment were placed in 8-in. holes. These tips consisted of a 4-in. length of all-brass well strainer with No. 35 slots. The hole for the piezometer was filled with sand for 2 to 3 in. above and below the tip. The hole around the 3/4-in. brass riser pipe was backfilled above the sand with 2 ft of tamped clay, then with 2 ft of sand-bentonite seal, above which the hole was filled with tamped clay. All backfill around the foundation piezometers was placed under water so that air would not be entrapped in the hole. The top of the riser was set so that the grade of the tubes to the terminal well would be approximately 2 per cent. All joints in the riser and tubing were heavily leaded and the joints at the tees were soldered.

50. The piezometer tip in the embankment was not placed until the embankment had reached a grade of 7 ft above the ground surface. The backfill around the tip was the same material that had been removed from the hole in which the tip was placed. No sand or porous material was used around these tips. All trenches carrying tubes from the piezometers were placed on either ascending or descending grades from the tips to the terminal wells, so that there would be no humps in the tubing which might permit the collection of air. Material used for backfill around the tubing both in the holes and in the trenches was typical of the adjacent material. No pockets of relatively porous or uncompacted material were left in the trench. Care was taken in placing and compacting the backfill in the trenches cut for the tubes to the piezometers tips.

51. Tubes from the piezometer tips were brought into the terminal well through a steel pipe in the wall of the well. After the installation was complete, the steel pipe was filled with asphaltic material. Just inside the terminal well, the brass tubes were changed to 1/4-in. brass pipe. A cutoff valve was installed on the ends of both brass pipe leads. A tee connection was installed on both leads of the tip side of the line. A short line in the valve was attached to the tees so that

gages could be connected onto the end of each line. The gages used were 4-1/2-in. Ashcroft, Type 1014, which were capable of measuring pressures up to 30 psi in 1/2-psi increments, and vacuums up to 30 in. of mercury in 1-in. intervals. The gages were calibrated prior to installation.

52. At the time of installation, freshly boiled water was circulated through the tubes and the tips until no air bubbles appeared in the return line. After installation, the lines were de-aired whenever it was considered likely that air was present in the tubes or riser pipe.

53. Generally, readings were taken approximately every two weeks during the construction period and every month in the interim between construction periods. Check readings were obtained occasionally by connecting a master gage into the line at the extra tee in the inlet line.

54. Readings of the above-described piezometers obtained both during and after construction of the embankment are plotted on figs. 17-18. The height of fill above the piezometer tip at the time of reading is also plotted on each of these figures, together with the natural water table as indicated by the piezometer adjacent to the embankment.

55. From the data shown on figs. 17 and 18, it appears that most of the piezometers functioned fairly satisfactorily with the exception of piezometers 1, 2, 4, and 5 at station 758 between November 1950 and 20 March 1951, piezometers 1, 2, and 5 at station 798 from January 1951 to 25 January 1952, and piezometer 4 from January to June 1951. It is doubtful that piezometer 5 at station 798 functioned properly until after January 1953. All of the No. 6 piezometers, installed to measure the elevation of the natural water table adjacent to the embankment, appeared to function very satisfactorily.

56. It was originally intended to obtain the <u>net</u> pore pressure in the foundation by subtracting the elevation of the natural water table (as measured by the No. 6 piezometers located 30 ft downstream from the toe of the embankment) from the gage pressure converted to ft msl. The elevation of the natural water table, as shown on figs. 17 and 18, varied seasonally from the ground surface to a depth of approximately 6 ft. Subtraction of the elevation of the natural water table from the elevation of the pore pressure in ft msl in the foundation resulted in <u>net</u>

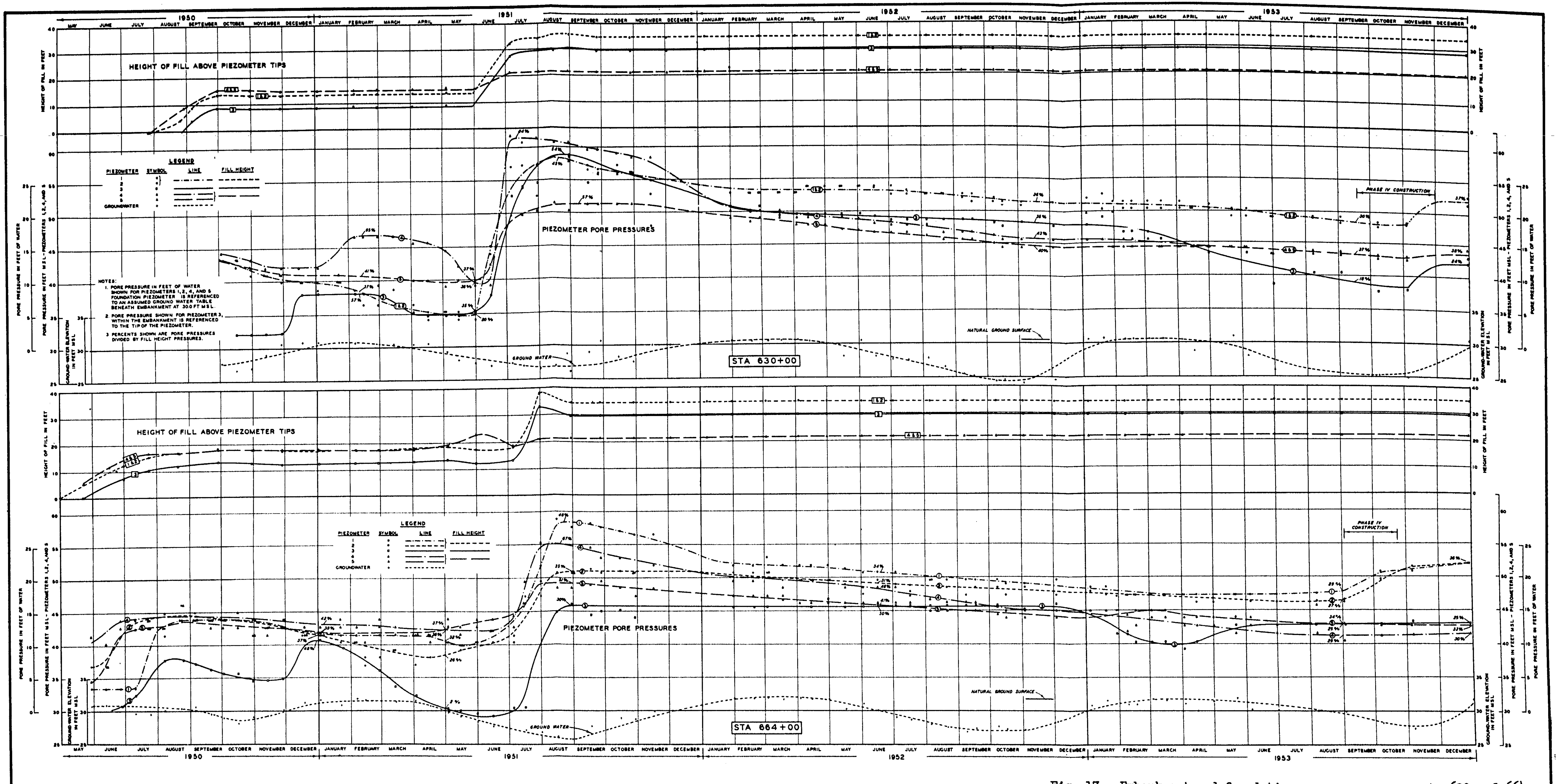


Fig. 17. Embankment and foundation pore pressures - sta 630 and 664

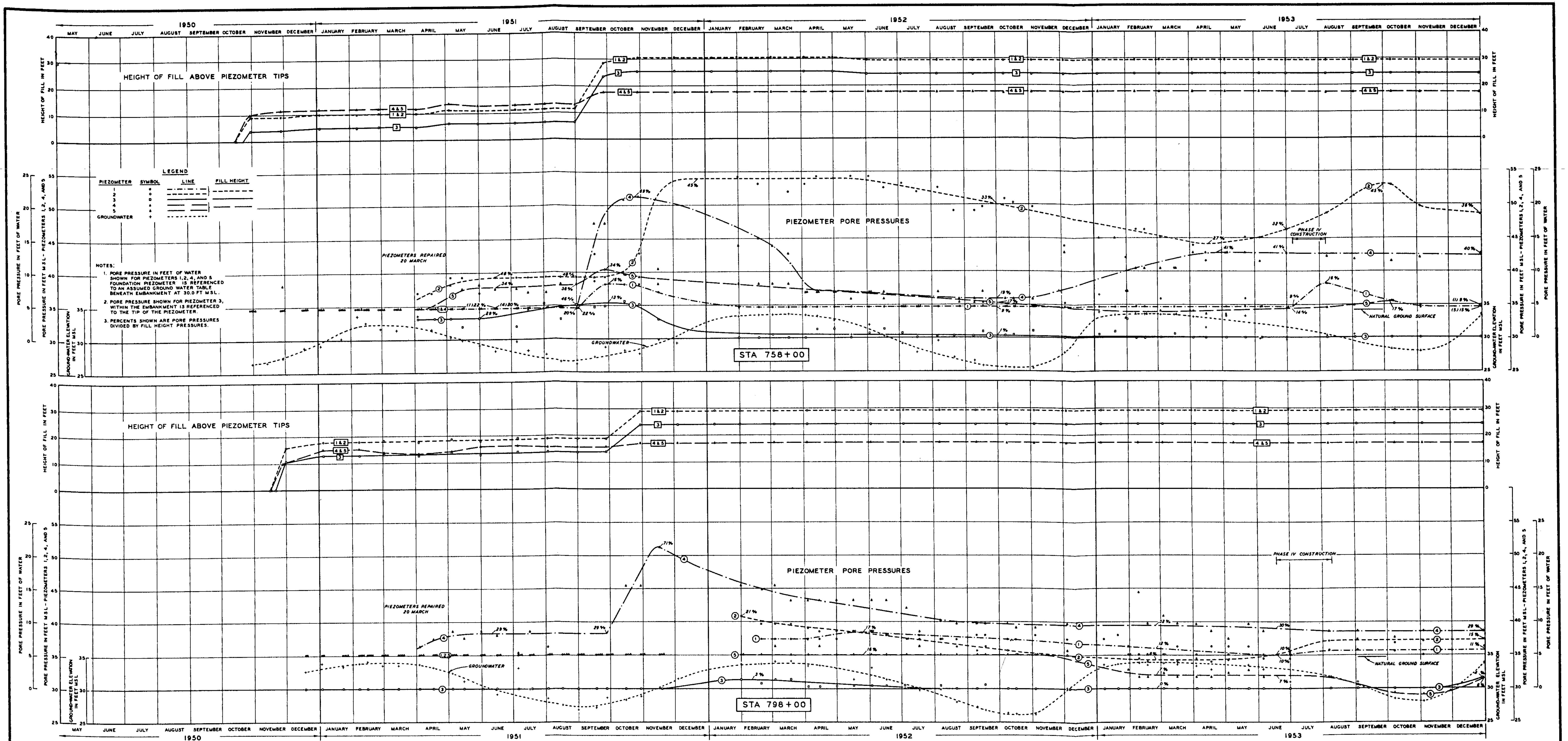


Fig. 18. Embankment and foundation pore pressures - sta 758 and 798

pore pressures that tended to decrease with time but also exhibited cyclical tendencies even with constant embankment loadings. As there is no reason for net pore pressures to rise and fall with a constant load, it was concluded that the water table beneath the embankment is probably more or less constant and equal to approximately the average, or slightly higher, elevation of the water table on each side of the embankment. Seasonal variation in the water table beyond the embankment is attributed to desiccation rather than to a draining action. Therefore, with no drying possible beneath the wide-base embankments, the water table beneath them probably varies little with the seasons and probably remains only 2 or 3 ft below the natural ground surface. On the basis of the above reasoning, the net pore pressures as measured by the foundation piezometers were taken as the height of pore pressure in feet of water above an assumed ground-water elevation beneath the embankments of 30 msl (see left scale of figs. 17 and 18).

57. The foundation piezometers indicated a rapid increase in pore pressure as the height of fill was being increased (summer and fall of 1951) and a gradual decrease in pore pressure with respect to time under a constant height of fill. Phase IV construction during the fall of 1953 caused another increase in pore pressure and, as was expected, piezometers 1 and 2 indicated higher pore pressures than piezometers 4 and 5 because they were located nearer the point of added load. (See fig. 16 for relation of phase IV load to location of piezometer tips.)

58. The foundation pore pressures in per cent of the pressure due to the height of fill were computed for different dates during construction and are shown on figs. 17 and 18. (The percentages shown are not quite exact, as they were based on the ratio of net pore pressures to weight of fill above the piezometer tip instead of the more correct ratio of net pore pressure to vertical foundation stress as computed from elastic theory.) The average of the maximum foundation pore pressures obtained at stations 630, 664, and 758 were 58, 50, and 41 per cent, respectively; the maximum pore pressure obtained was 85 per cent of the fill height pressure. These high pore pressures occurred just after the height of fill was increased during the summer of 1951. Most

of the piezometers at station 798 were not functioning properly when the maximum pore pressures could be expected to occur (see fig. 18).

59. From the pore pressure measurements, the foundation appeared to be about 60 to 70 per cent consolidated under the fill just before the height of fill was increased during the summer of 1951. Similarly, the foundation was about 70 to 80 per cent consolidated under the increased fill height just prior to starting construction phase IV. These latter data closely check the percentages of consolidation as determined from the observed settlements.

60. The tips of the No. 3 piezometers were installed in the embankment at elevations a few feet above the assumed average ground-water level of 30.0 ft msl. Therefore, the pore pressures in feet of water indicated by these piezometers are referenced to the tip elevations, as are the heights of fill shown on figs. 17 and 18. It can be seen on fig. 17 that comparatively high pore pressures occurred in the south embankment at station 630 and station 664, as respective maximum pressures of 54 and 30 per cent of the fill height pressures were obtained, whereas in the north embankment the maximum pore pressures were only 13 and 3 per cent of the fill height pressures, respectively, at station 758 and station 798.

61. The high pore pressures in the south embankment occurred just after the fill height was increased during the summer of 1951. Equally high pore pressures (in per cent of fill height) also occurred in the south embankment in the fall of 1950 after the initial fill was placed. The pore pressure in the base of the south embankment decreased to about 15 to 25 per cent by the time the phase IV addition was made.

62. Very small pore pressures developed in the north embankment and they decreased rapidly shortly after the fill had been placed. It is believed the higher pore pressures and slower rate of dissipation of pore pressure in the south embankment were due to presence of wetter and more plastic clays in the south embankment as compared to the north embankment.

Stability of Embankment

63. Although some rather high pore pressures were observed during

construction, all sections of the embankment proved stable both during and after construction. Stability analyses of the embankment at station 630 utilizing the cross section and pore pressures as observed on 16 July 1951, and an estimated slow shear strength of the foundation of $\phi = 16^{\circ}$, c = 0.15 ton per sq ft indicated a factor of safety of 1.7.

Settlement of Embankment

64. Observations of settlement plates placed during construction at the base of existing embankments adjacent to the project provided an empirical basis for predicting the magnitude and rate of settlement to be expected for the new embankment (see figs. 19 and 20). Extrapolation of the time-settlement curves of the existing embankments shown on figs. 19 and 20 indicated that approximately 90 per cent of the total settlement of the new embankment should occur within the first five years after construction began.

Settlement analyses

65. Settlement analyses based on consolidation tests previously discussed indicated that total settlements in the foundation beneath the (net) embankment would probably range from 3.5 to 5 ft. Computed settlements at specific stations along the embankments are tabulated below:

Table 8

Estimated Total Settlement of Embankment

Station	Estimated Total Settlements, ft Net Emb Sect (May 1949)	Net Emb Sect (Jan 1951)	Overbuilt Sect + Emb Consol (Jan 51)
634+50	4.7	4.2	4.7
672+00	3.9	3.5	4.0
728+50	4.0	3.1	4.1
758+50	4.4	4.0	4.5
778+50	3.7	3.9	4.4
798+50	4.6	5.0	5.5

Overbuilding of the embankment plus internal consolidation was estimated to increase the total settlement of the top of the embankment by approximately 0.5 ft. The estimated settlements in table 8 were computed on

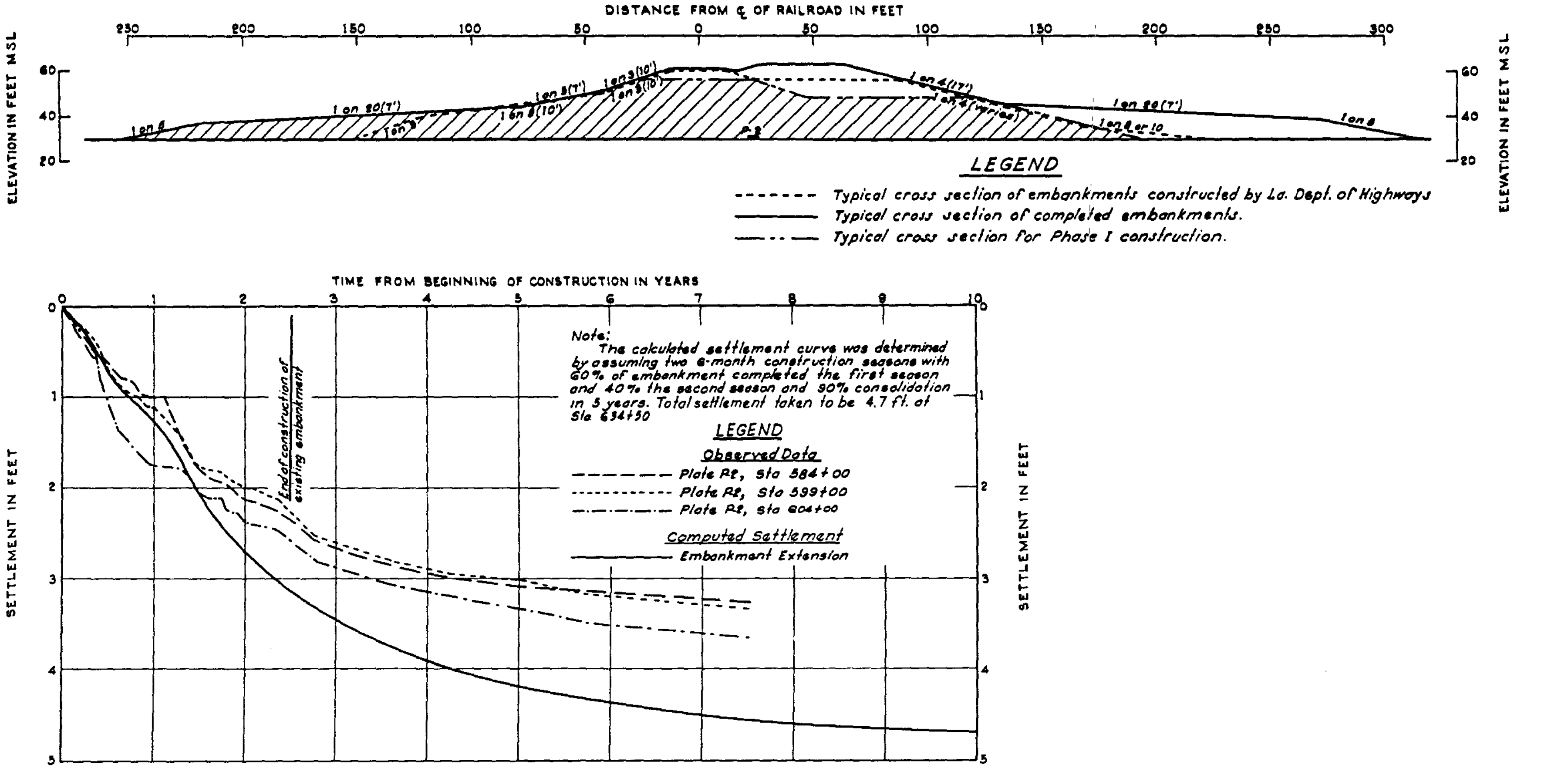
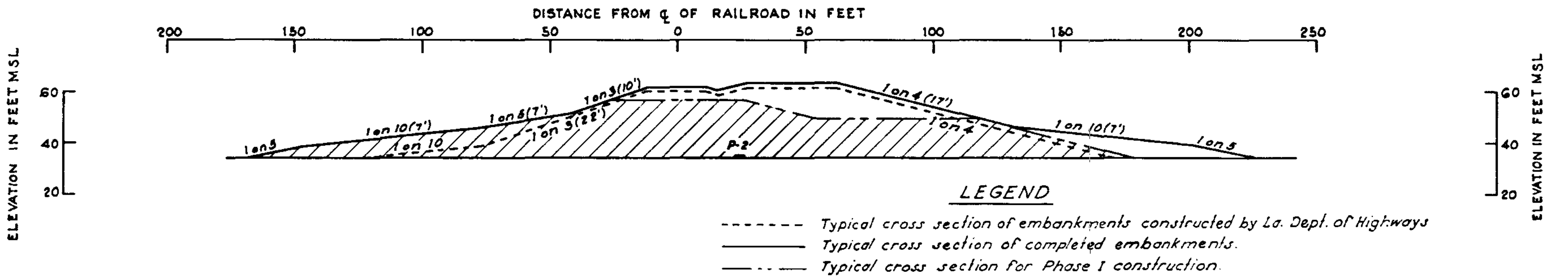


Fig. 19. Rate of embankment settlement, sta 581+00 to 679+00



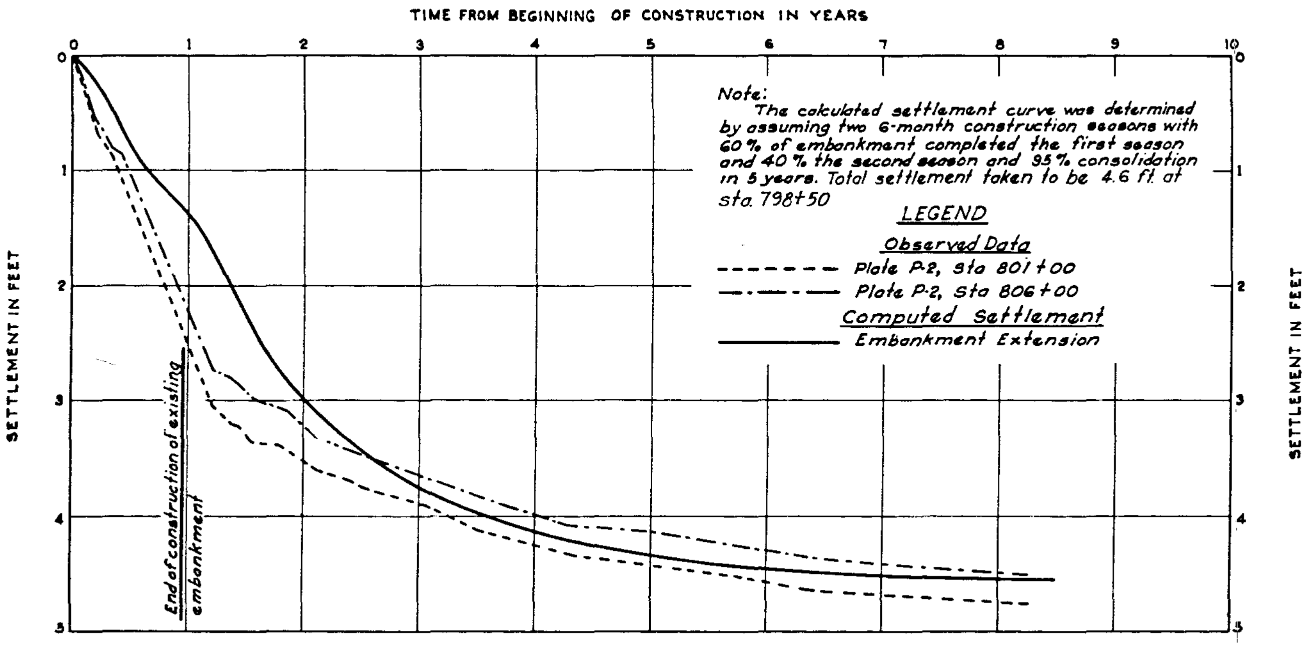


Fig. 20. Rate of embankment settlement, sta 721+00 to 812+00

the basis of laboratory pressure void ratio curves for soil samples taken in the first 30 to 35 ft, whereas the straight line portion of the laboratory consolidation curves was used for samples below this depth. The reason for this is that the upper 30 to 35 ft of the foundation material is considered to be overconsolidated, whereas the soils below this depth appear to be normally consolidated. The ground-water table was assumed to be at a depth of approximately 6 ft below the natural ground surface in the settlement computations. Although the clay stratum beneath the north embankment is thicker than that beneath the south embankment, the more compressible foundation clays beneath the south embankment and the larger sections required for stability of the south embankment resulted in settlements of the same magnitude for both the south and north embankments.

66. The presence of layers of silty soils of various thicknesses interspersed within the predominantly clay foundation affords lateral drainage for consolidation of the clay, and materially reduces the effective thickness of the entire clay stratum as regards the rate of consolidation of the foundation. Thus, estimation of the rate of settlement was considerably more difficult than predetermination of the total settlement. Before computation of the time rate of consolidation of the new embankments was attempted, the effective thickness of the foundation strata was computed on the basis of time-settlement curves obtained from the embankments previously constructed by the Louisiana Department of Highways, and time-consolidation data from laboratory tests. From these computations, the effective thickness of the foundation clay strata was computed to be about 11 or 12 ft. The time-settlement curves shown on figs. 19 and 20 were computed on the basis of the following:

	North Embankment	South Embankment
a) Total settlement	4.6 ft (sta 798+50)	4.7 ft (sta 634+50)
b) Laboratory time for 50% consolidation of a 1.2-in. sample	29 min	47 min
c) Effective thickness of clay stratum	12 ft	11 ft

d) Terzaghi's theoretical time rate for uniformly loaded stratum

e) Construction rate: first season, 60%; second season, 40%

Overbuilding of railroad embankment

67. In January 1951 an estimate was made of the amount that the railroad portion of the embankment should be overbuilt in order to compensate for consolidation of the embankment and settlement of the foundation after completion of that portion of the embankment. The estimate was based on the theoretical analyses and an analysis of the actual settlement that had occurred beneath the preload fills at the abutments of the control structure and at four stations along the embankment. The following assumptions and estimates were made in determining the required overbuild:

a. The railroad portion of the embankment would be completed between June and December 1951.

b. The settlements for the net embankment sections given in table 8 were reduced approximately 10 per cent on the basis of settlement data obtained to January 1951 from the preload fills at the abutments for the control structure, except between stations 790 and 800 where the computed settlement was increased approximately 10 per cent on the basis of settlement data obtained from settlement plates beneath the existing embankment at station 801.

c. An allowance of 0.5 ft was made for consolidation within the embankment and settlement due to overbuilding which will occur after completion of the railroad portion of the embankment.

d. The maximum amount of overbuilding considered safe without impairing the stability of the embankment and foundation was taken as 2.5 ft.

e. Total settlement of the railroad portion of the embankment with load on December 1951 taken as 95 per cent of that computed for the entire embankment.

68. A summary of the results obtained in computing the overbuild of the embankment is given in table 6, page 34. The amount of overbuild required could not be computed closer than ± 0.5 ft from the data available. It was estimated that the amount of overbuild recommended would insure that the grade of the railroad will stay above the net grade

required for at least five years.

69. A comparison of the estimate of required overbuild made in January 1951, on the basis of the embankment staying above grade for five years (1957), with a revised estimate made in January 1954 is shown in table 9. The revised (January 1954) estimate was made by extrapolating

Table 9

Comparison between Estimated and Observed Settlements and Overbuild

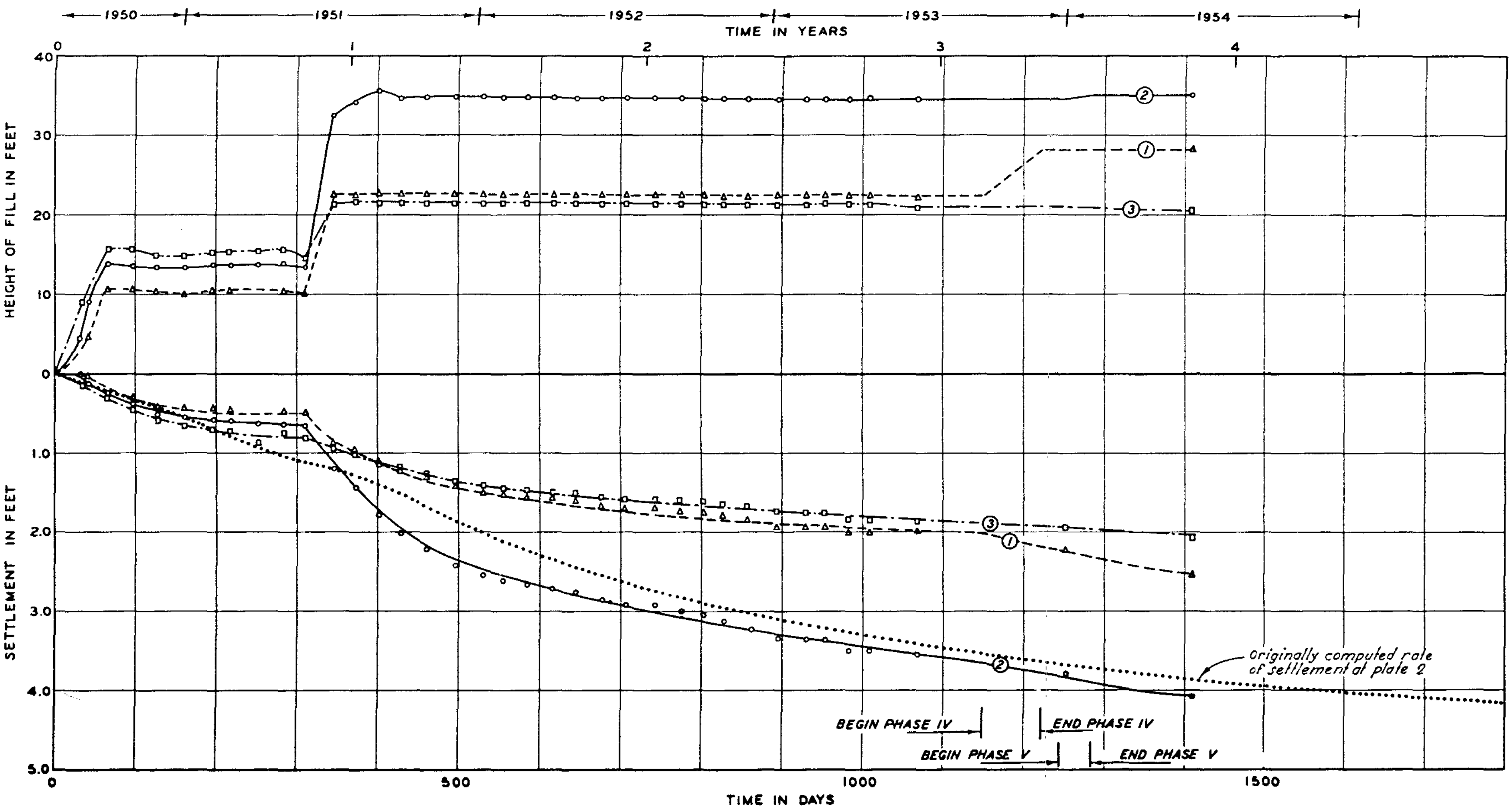
	Foundation Settlement 1 Jan 1952, ft		Overbuild*, ft			
					Overbuild Required to Compensate for Settlement by Dates Shown (Estimated in January 1954)	
Sta	Estimated (Jan 1951)	Observed	Estimated (Jan 1951)	As Constructed	By 1957	By 1965
630	1.5	2.5	3.2	2.7	2.8	3.1
664	2.1	2.3	1.9	2.0	2.4	2.8
758	1.7	1.9	2.8	2.8	2.7	3.6
798	2.5	2.1	3.0	2.6	3.1	3.6

* All estimated overbuilds are based on an assumed consolidation within the embankment after construction and foundation settlement due to an overbuild of 0.5 ft.

arithmetic and semilogarithmic settlement curves (figs. 21-25) to obtain the total foundation settlements by January 1957 and January 1965, subtracting the observed foundation settlement on the date the embankment was completed, and adding 0.5 ft for embankment consolidation and foundation settlement resulting from overbuilding. An examination of the data in table 9 shows that the overbuild estimated in January 1951 (for 1957) agrees closely (within 0.5 ft) with estimates made in January 1954 from settlement data available at that time. It appears that by 1965 the embankment may be 0.5 to 1.0 ft below the design net grade.

Observed settlements

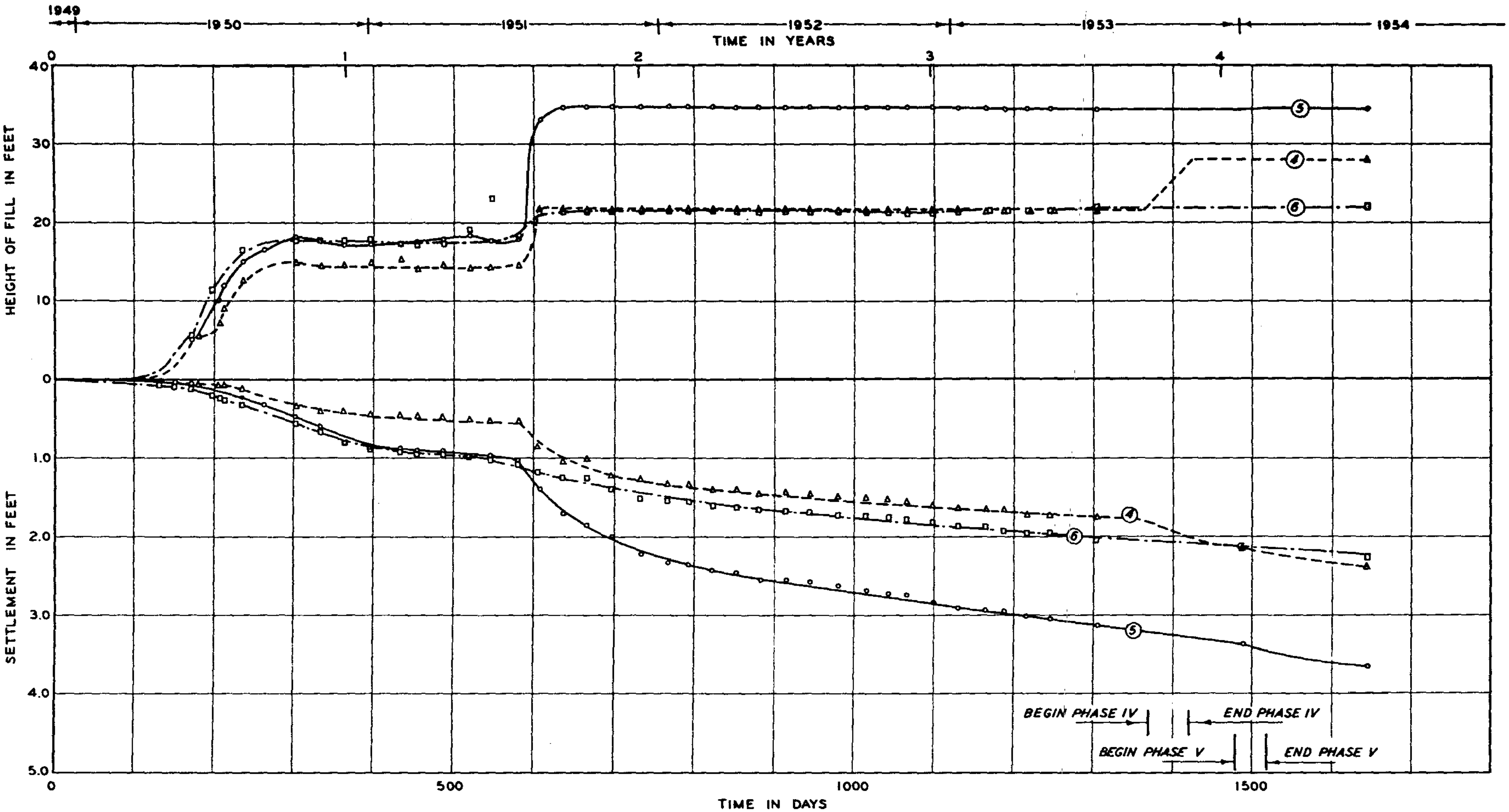
70. Settlements of the foundation as observed at several points along the embankment during and after construction are plotted on figs. 21-25. Figs. 21-24 are arithmetic plots of foundation settlement vs time and of height of fill over settlement plates vs time. The rate and total amount of settlement, as originally predicted (May 1949) on the



Notes.

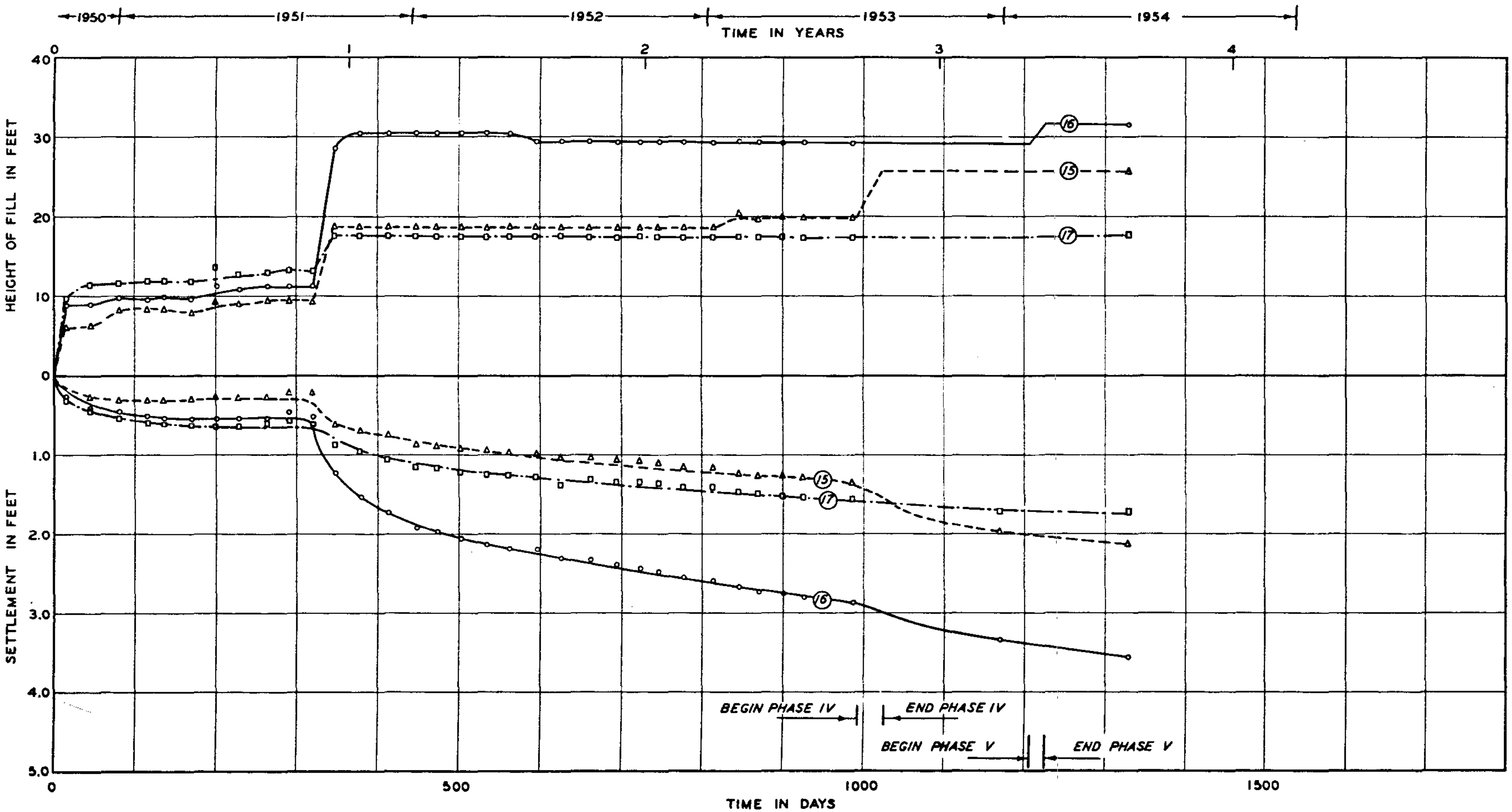
1. *Plate 1, 95 ft upstream from R.R. center line. Plate 2, 18 ft upstream from R.R. center line. Plate 3, 60 ft downstream from R.R. center line.*
2. *Half loading time = 55 days.*
3. *Zero time = 25 July 1950.*
4. *Construction rate assumed for computing rate of settlement was uniform application of 60% of load the first construction season and 40% the second season.*
5. *Numbers in circles refer to settlement plate numbers.*

Fig. 21. Settlement of foundation beneath embankment, sta 630+00



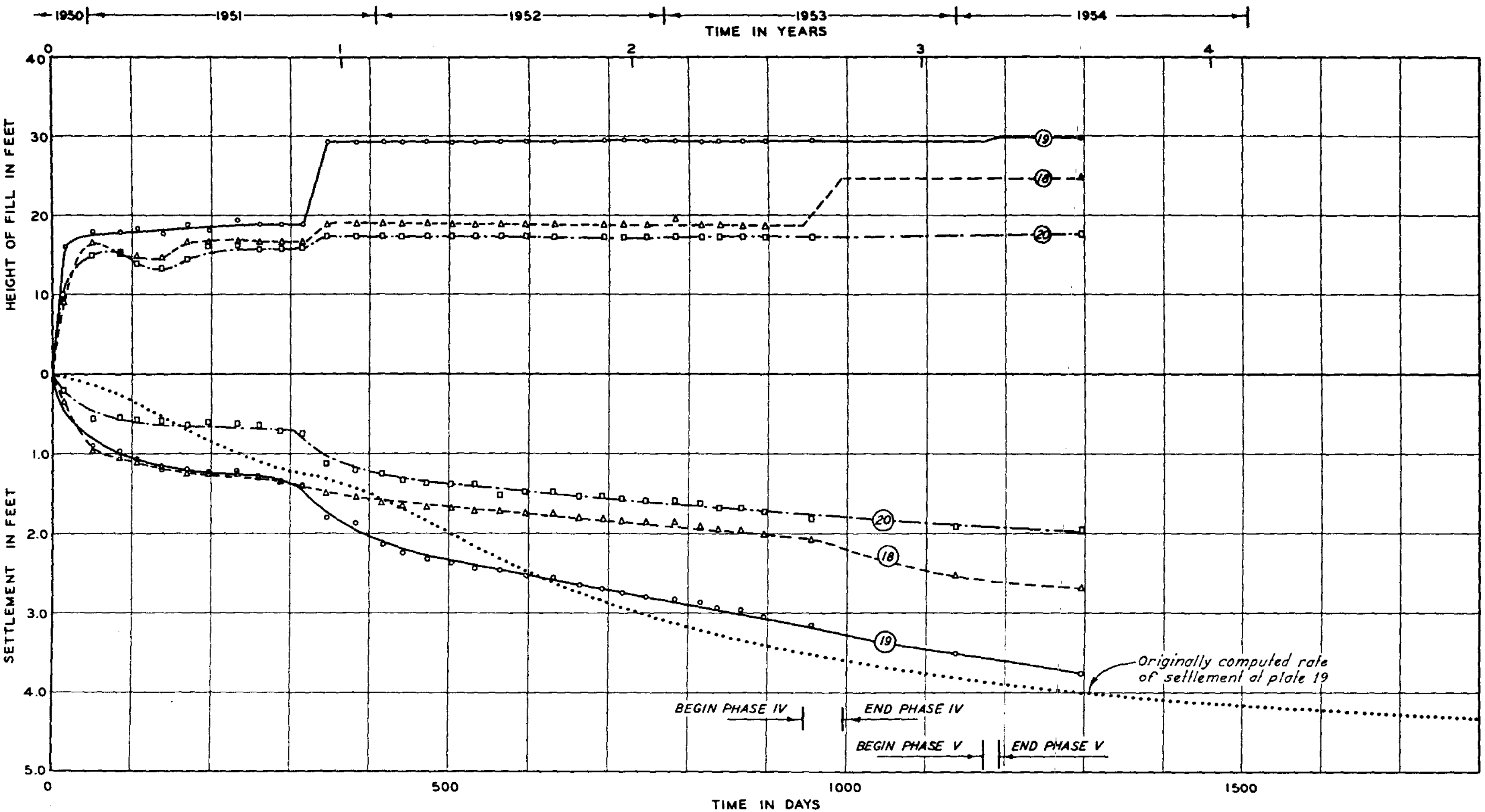
Notes:
1. Plate 4, 95 ft upstream from R.R. center line.
Plate 5, 18 ft upstream from R.R. center line.
Plate 6, 60 ft downstream from R.R. center line.
2. Half loading time = 150 days.
3. Zero time = 1 Dec 1949.
4. Numbers in circles refer to settlement plate numbers.

Fig. 22. Settlement of foundation beneath embankment, sta 664+00



Notes:
1. *Plate 15, 95 ft upstream from R.R. center line.*
   Plate 16, 18 ft upstream from R.R. center line.
   Plate 17, 60 ft downstream from R.R. center line.
2. *Half loading time = 40 days.*
3. *Zero time = 15 Oct. 1950.*
4. *Numbers in circles refer to settlement plate numbers.*

Fig. 23. Settlement of foundation beneath embankment, sta 758+00



Notes:

1. Plate 18, 95 ft upstream from R.R. center line.
   Plate 19, 18 ft upstream from R.R. center line.
   Plate 20, 60 ft downstream from R.R. center line.
2. Half loading time = 25 days.
3. Zero time = 15 Nov 1950.
4. Construction rate assumed for computing rate of settlement was uniform application of 60% of load the first construction season and 40% the second season.
5. Numbers in circles refer to settlement plate numbers.

Fig. 24. Settlement of foundation beneath embankment, sta 798+00

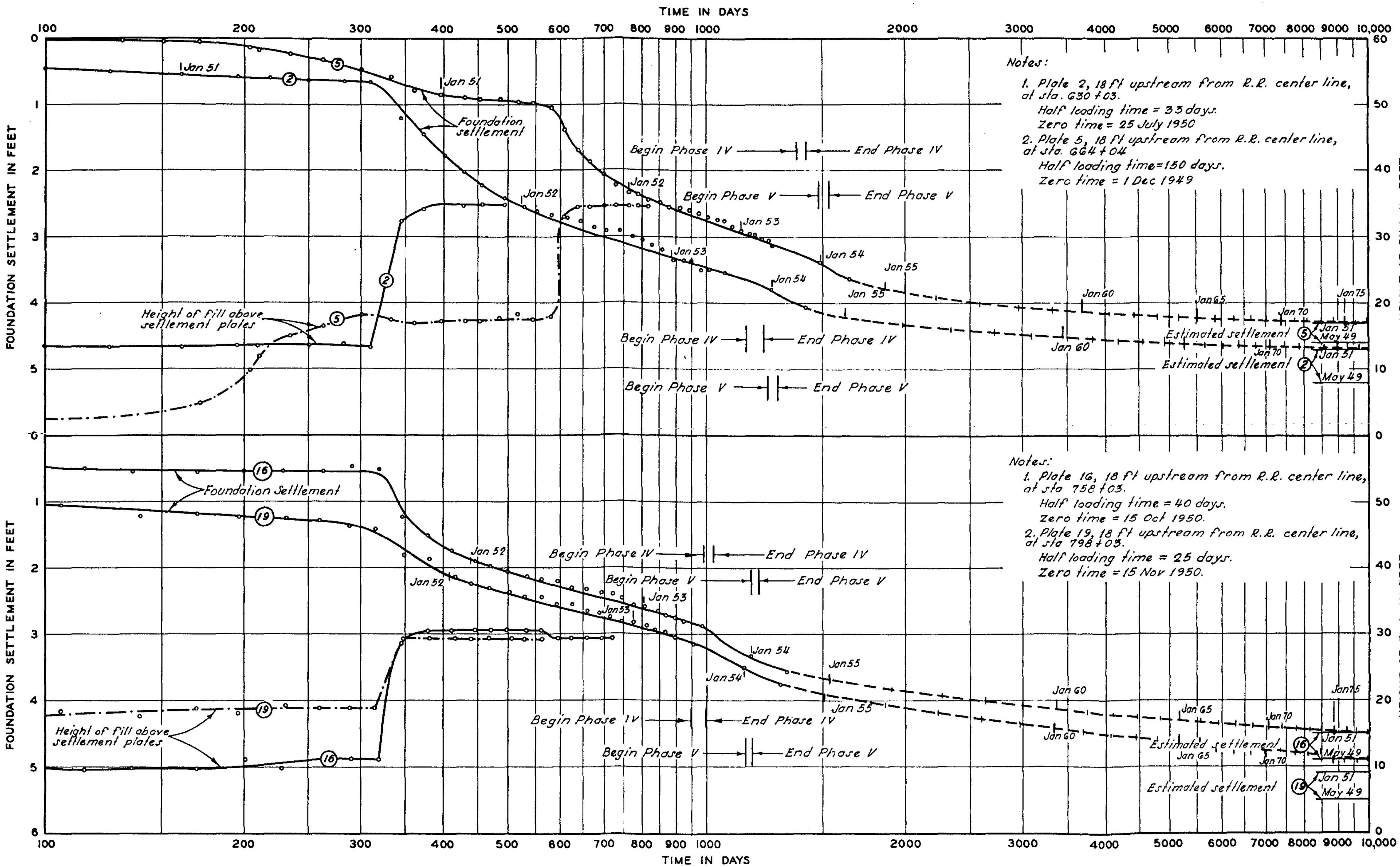


Fig. 25. Summary of embankment settlement

basis of a net embankment section as indicated by settlement plates 2 and 19, are also plotted on figs. 21 and 24, respectively. Foundation settlement data obtained from the plates beneath the center of the embankment are plotted for stations 630, 664, 758, and 798 as a semilog plot on fig. 25.

71. A comparison of predicted foundation settlements as estimated on the dates indicated is shown below:

	Total Foundation Settlement in Feet		
Station	Est May '49*	Est Jan '51*	Est Jan '54**
630	5.2	4.7	4.6
664	4.4	4.0	4.2
758	4.9	4.5	4.2
798	5.1	5.5	4.6

* Above estimated settlements include 0.5 ft for settlements resulting from overbuilding.

** Foundation settlement estimated to occur by 1965 from fig. 25.

The estimates made in January 1951 were revised on the basis of settlement records obtained from the preload fills at the abutments of the control structure, which are subsequently discussed. The settlement estimates made in January 1954 were based on visual extrapolation of the curves shown in fig. 25. Considering the uncertain construction schedule and the magnitude of the foundation settlement, it appears that very good agreement has been obtained between estimated and observed amounts and rates of settlement. However, the accuracy of the estimated rate of settlement can largely be attributed to advance settlement observations made on adjacent, previously constructed embankments.

PART V: PILE LOADING TESTS

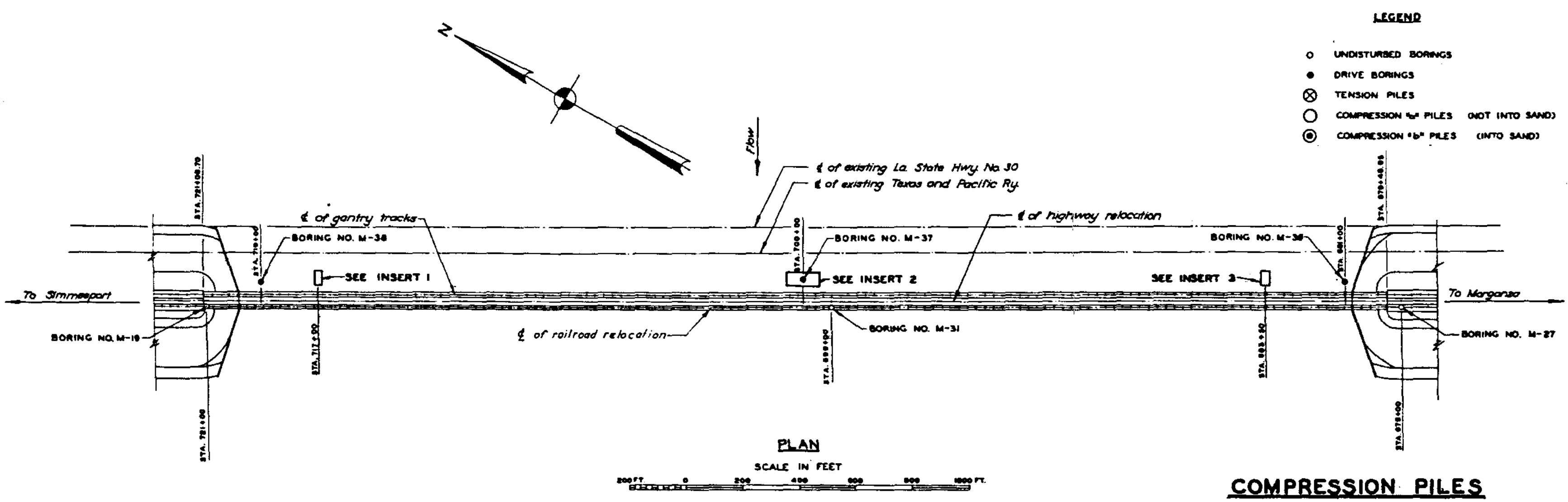
72. A comprehensive pile testing program was undertaken at the site of the control structure to determine the size of piles required to carry a 100-ton compression load without any significant movement, and to determine the tension load this size pile could carry. The soil conditions at the site necessitated the use of long piles driven through soft clay into sand at a depth of approximately 80 ft. All test piles were driven vertically for ease of testing. (Although tip bearing capacity of vertical piles is somewhat greater than that of battered piles, the difference probably does not amount to more than 5 to 15 per cent.)

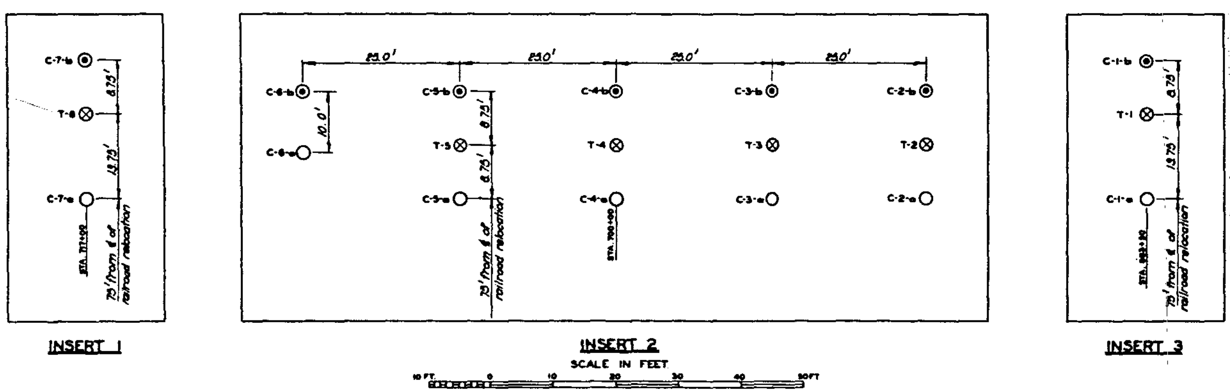
General Test Procedures

73. As the clay overlying the sand was very compressible, compression tests were made in such a manner that the bearing capacity of the portion of the piles driven into the sand could be evaluated. For the purpose of simplicity, that portion of the pile driven into sand is hereafter called the "tip"; the point and the sides of the pile in sand are both included in this definition and both carry part of the load. To determine the bearing capacity of the tip it was necessary either to eliminate completely or to measure the frictional resistance developed along that portion of the pile in the clay stratum. In the tests the resistance developed in the clay was determined by testing the piles in pairs. The frictional resistance in the clay stratum was determined from piles stopped about 5 ft above the top of the sand; the combined frictional resistance and tip capacity were determined from piles driven into the sand. Piles stopped in clay are called "a" piles in this report, whereas piles driven into sand have been termed "b" piles. Tension piles also were driven into sand. Holes were excavated at each test pile location to eliminate any skin friction in material above the proposed base of the structure.

Types of Piles and Driving Equipment

74. Fig. 26 shows the types and sizes of piles tested in





COMPRESSION PILES

PILE PAIR	TYPE OF PILE	DIA. OF PILE IN INCHES
C-1	Pipe	24
C-2	Monotube tapered	8(tip)
C-3	Monotube constant section	12
C-4	Pipe	18
C-5	Pipe	24
C-6	Pipe	30
C-7	Precast concrete	22(square)

TENSION PILES

PILE	TYPE OF PILE	DIA. OF PILE IN INCHES
T-1	Pipe	24
T-2	Monotube tapered	8(tip)
T-3	Monotube constant section	12
T-4	Pipe	18
T-5	Pipe	24
T-6	Pipe	24

Fig. 26. Test pile layout

compression at locations along the length of the structure; the types and sizes are also listed in the following tabulation.

Type of Pile	Size and Description
Cylindrical pipe	3/8 in. wall thickness 18-in., 24-in., 30-in. diameters 1.5-ft-, 1.9-ft-, 2.4-ft-long conical points
Monotube, uniform tapered	No. 7 gage metal 18-in. butt diameter, tapered 1 in. in 7 ft to 8-in.-diameter hemispherical tip
Monotube, constant section	No. 3 and No. 7 gage metal 12-in. diameter, 9-in.-long conical point
Precast concrete	22-in. square, point tapered to 8 in. in 2 ft

A tension test was conducted on each pile with the exception of the 30-in.-diameter pipe pile and the precast concrete pile.

75. The piles were driven by a skid-mounted, whirler-type driver with 82-ft fixed leads. A Harrison Vulcan No. OR hammer, delivering 30,000 ft-lb, was used to drive the 24- and 30-in. pipe piles and the 22-in.-square concrete piles. A Harrison Vulcan No. 1 hammer, delivering 15,000 ft-lb, was used for driving all anchor piles, the Monotube piles, and the 18-in. pipe piles. All cast-in-place piles were filled with concrete a few days after driving. A period of 4 to 6 weeks elapsed after driving before any loads were applied to the piles.

Compression Test Piles

Load arrangement

76. The test piles were loaded by jacking against concrete weights stacked on platforms over the test piles as shown on fig. 27. The platforms were supported on 12 timber piles arranged six in a row on each side of the test pile. Timber cribbing was used around the anchor piles as a safety measure. The weights were precast concrete slabs and rectangular blocks weighing approximately 3.5 and 5 tons, respectively. Hydraulic jacks seated on fabricated steel pedestals placed on top of the test piles were used to apply the loads to the piles. Two 200-ton-, six 100-ton-, and four 60-ton-capacity jacks were used.

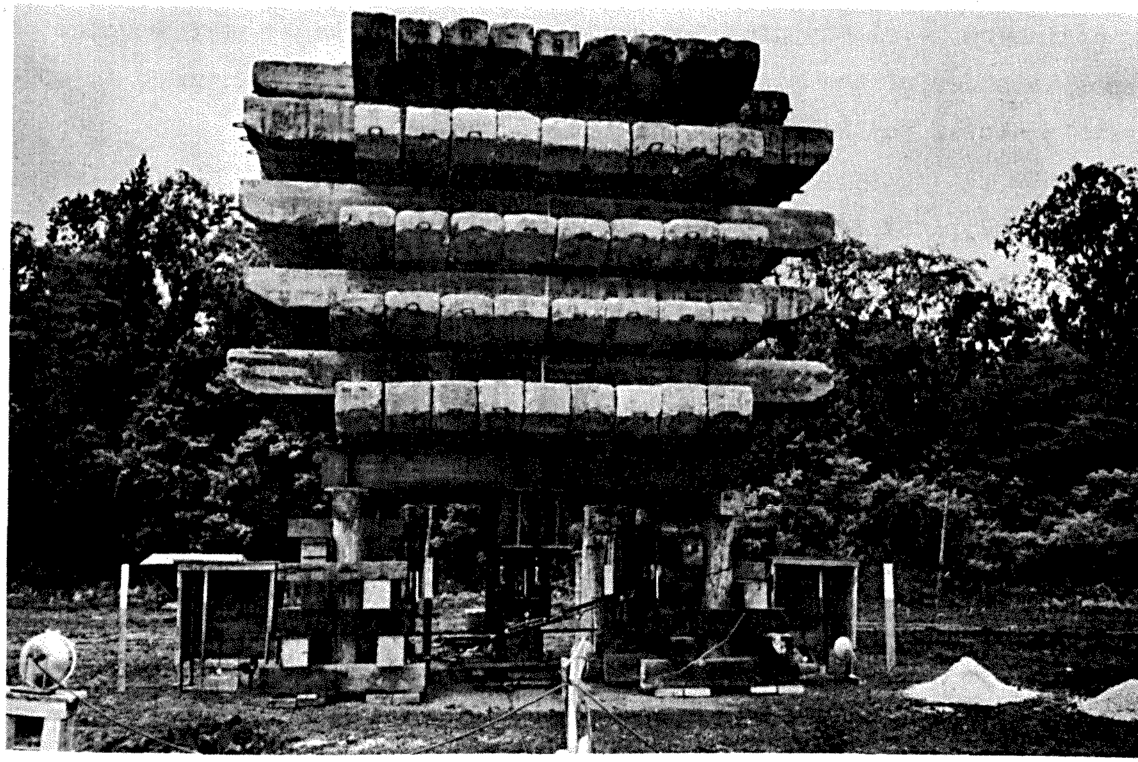


Fig. 27. Test setup for 24-in. pipe pile driven into sand

Loading procedure

77. "a" piles. The piles seated in clay were loaded and tested to failure prior to loading the piles that were driven into sand. In general, loads were applied to the piles in 20-ton increments at the rate of 1-1/2 tons per minute. Time-settlement curves were plotted during the tests to estimate the ultimate settlement under each increment of load. After completion of the 40-, 80-, and 160-ton loads and at the end of the test, the piles were loaded and allowed to rebound. Unloading and reloading were done at the rate of 1-1/2 tons per minute. All piles seated in clay were loaded to failure.

78. "b" piles. The "b" piles were loaded in the same manner as the corresponding "a" piles. All "b" piles were loaded to failure except the 30-in. pipe pile which successfully carried 400 tons without failure. Rebound of all piles was allowed at the end of the test.

Measurement of movement

79. The movement of the pile under load was measured by means of

an engineers level and dial deflection gages. The gages were mounted on opposite sides of the pile, the movement of which was referenced to concrete slabs 12 to 14 ft away from the test pile.

Test data

80. Typical test data for compression piles C-4-a and C-4-b are shown graphically on figs. 28 and 29.

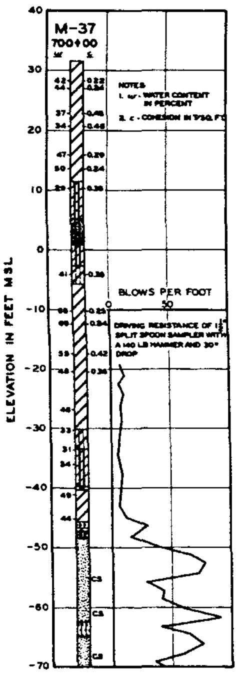
81. Load settlement curves. A curve of gross movement of the pile head vs pile load was developed from the test data on each pile. The points representing loads that were held for 24 hours were given primary consideration in drawing the curves shown on figs. 30 and 31. This procedure resulted in a curve that represents (up to the maximum load) the movement of the pile head under sustained load, neglecting consolidation of the clay foundation.

82. Net and elastic settlement curves. As each test pile was allowed to rebound under no load a number of times during the test, it was possible to determine the net and elastic settlement curves for each pile. The points representing the net settlement remaining after rebound and the amount of elastic rebound are plotted for each rebound for each pile on figs. 30 and 31.

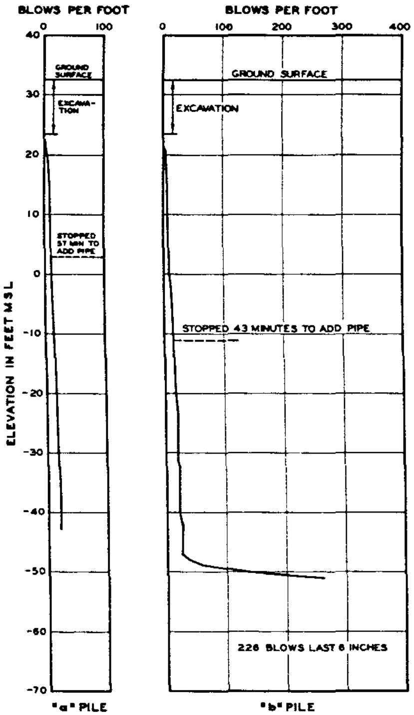
83. Tip-load curves. A curve approximating the load carried by the tip of the "b" pile (the portion of the pile in the sand) was computed for each "b" pile by subtracting at any gross settlement the "a" pile load from the "b" pile load. This method is not exactly correct, as the load carried by a friction pile is not distributed to the foundation in the same way as the load on a point bearing pile because, at similar movements of the top of the pile, the distribution of strain is different along the two piles. However, no other method of developing a tip-load deflection curve from this type of test was considered to be any more accurate.

Pile failure loads

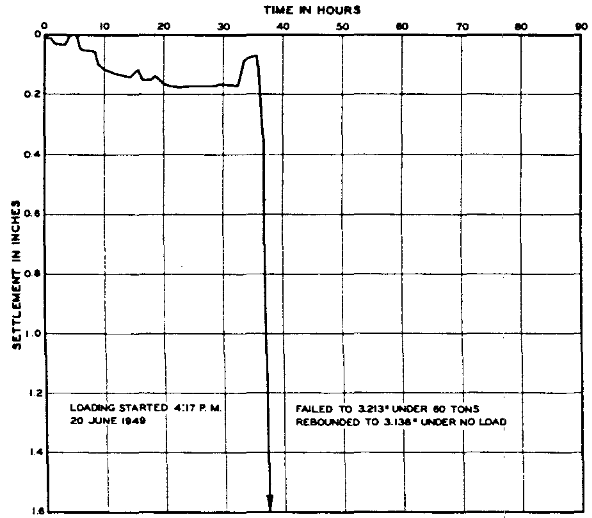
84. The determination of the "failure load" of a test pile is usually arbitrary. Therefore, various procedures were utilized to obtain a value for each pile. The procedures used for selecting a failure load for each compression pile were as follows:



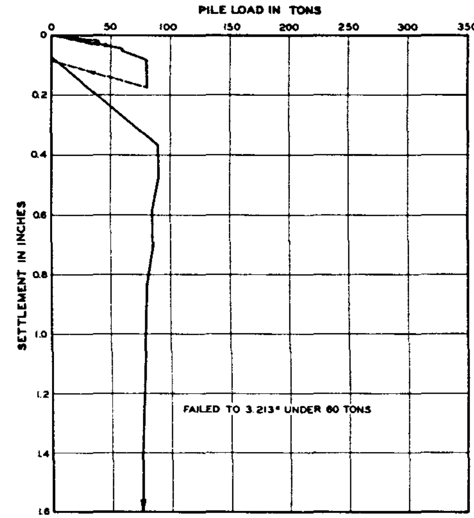
SOIL DATA



DRIVING RECORDS



TIME-DEFLECTION

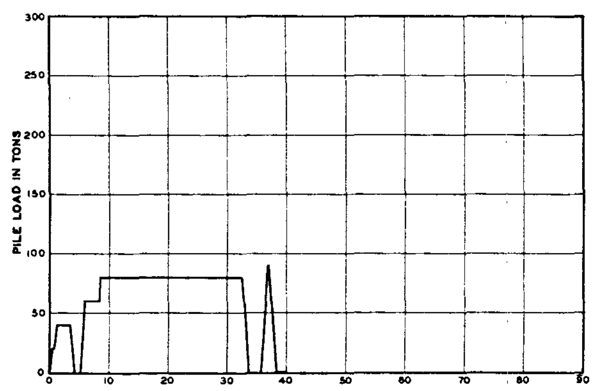


LOAD-DEFLECTION

DRIVING DATA

Pile driven with Vulcan No 1 hammer

Weight of hammer	*9,700 lbs.*
Weight of striking parts	*5,000 lbs.*
Weight of driving head	*2,355 lbs.*
Steam pressure	*110 psi*
60 blows per minute	



TIME-LOAD

PILE DATA

18-inch O.D. steel pipe pile, ⅜-inch wall thickness, conical point 1.5 feet long, located 75 feet east of Sta. 700+00

Driven length	*79.6 feet*
Loaded length	*78.7 feet*
Imbedded length	*67.1 feet*
Elevation of tip	*- 43.6 feet MSL*

Pile driven 23 May 1949 and filled with concrete 3 June 1949.

Note:

Boring data from El. 7.5 to -18.0 from Boring M-31 at Sta. 699+00. Boring M-37 located 85 feet east of Sta. 700+00.

Fig. 28. Compression pile test data; pile C-4-a, 18-in. pipe pile

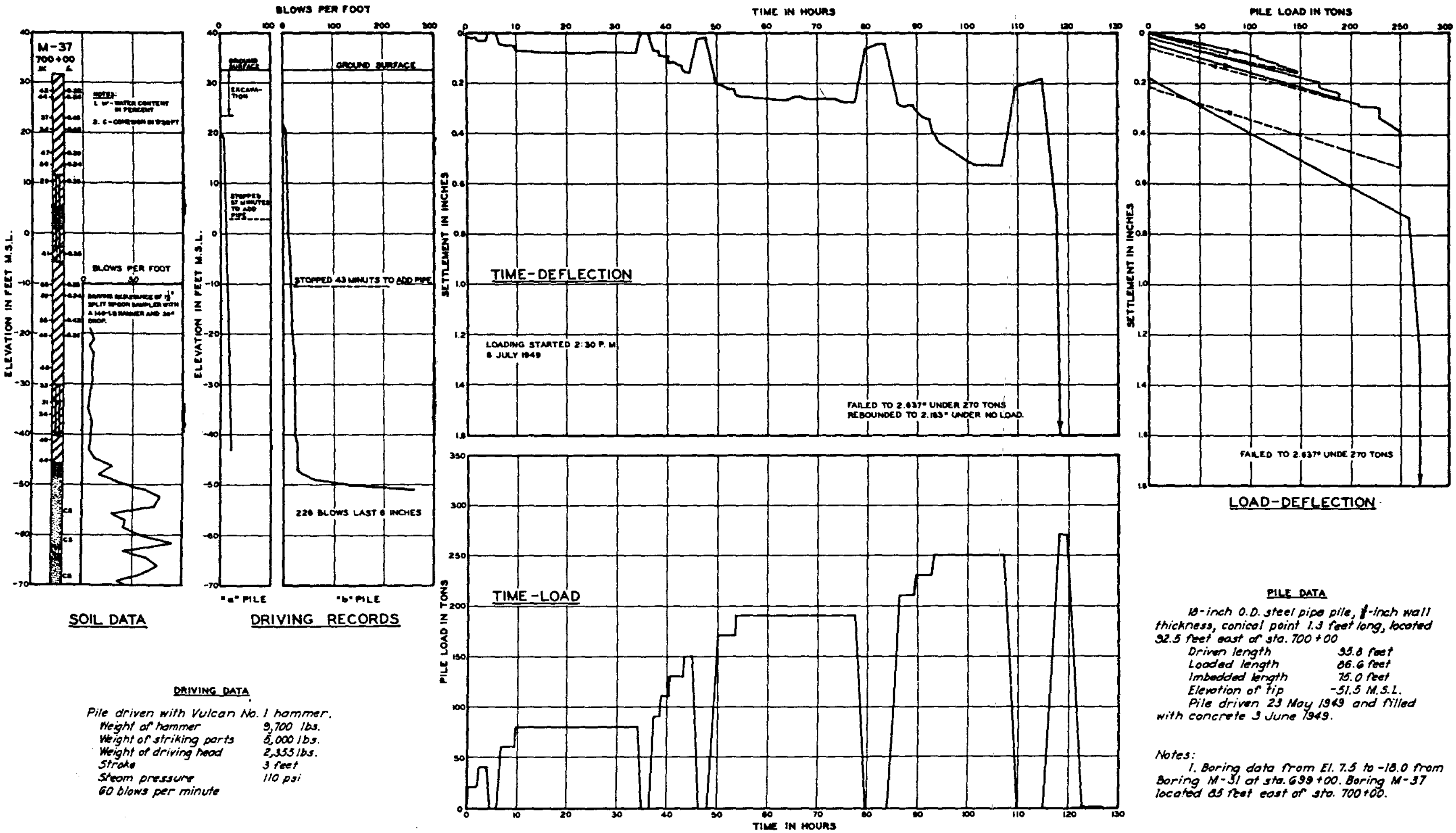


Fig. 29. Compression pile test data; pile C-4-b; 18-in. pipe pile

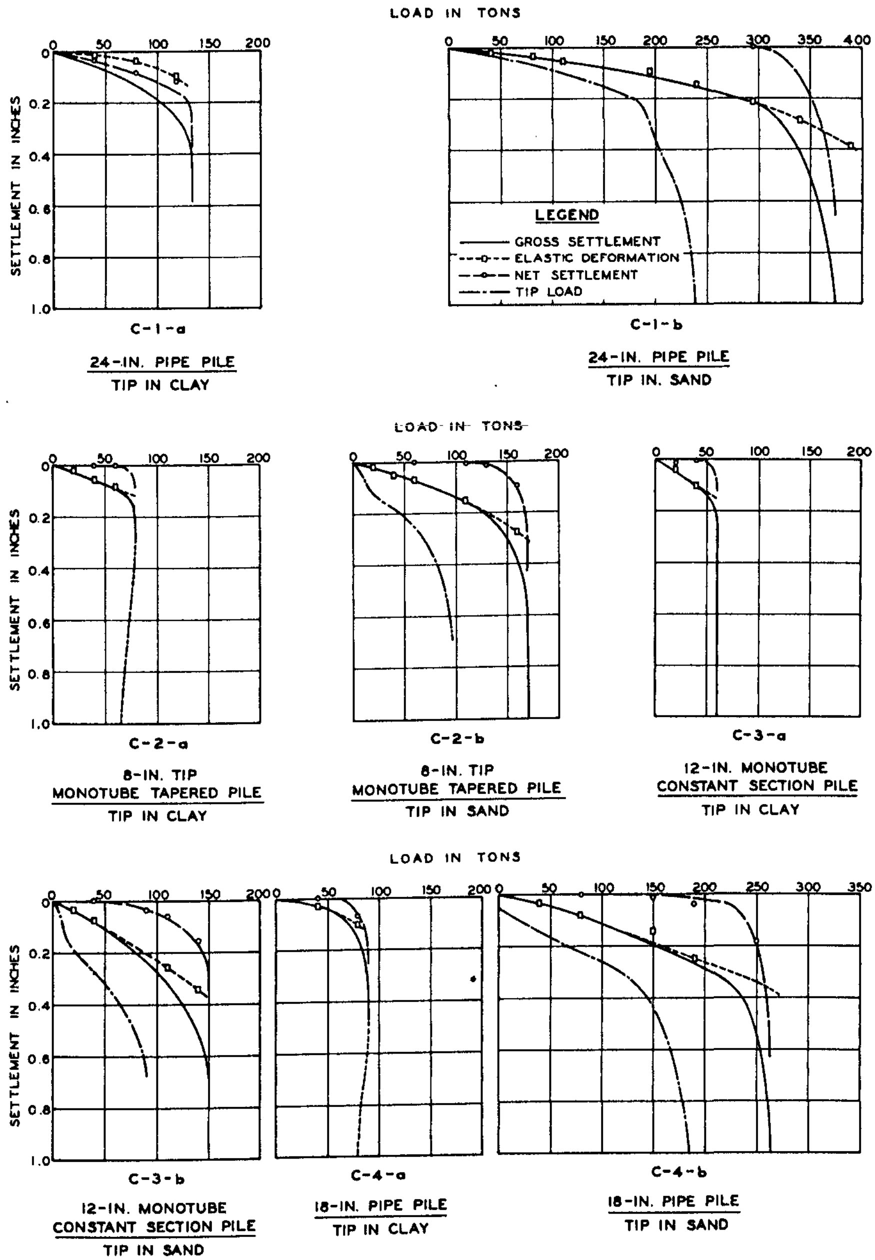


Fig. 30. Load settlement curves, piles C-1 through C-4

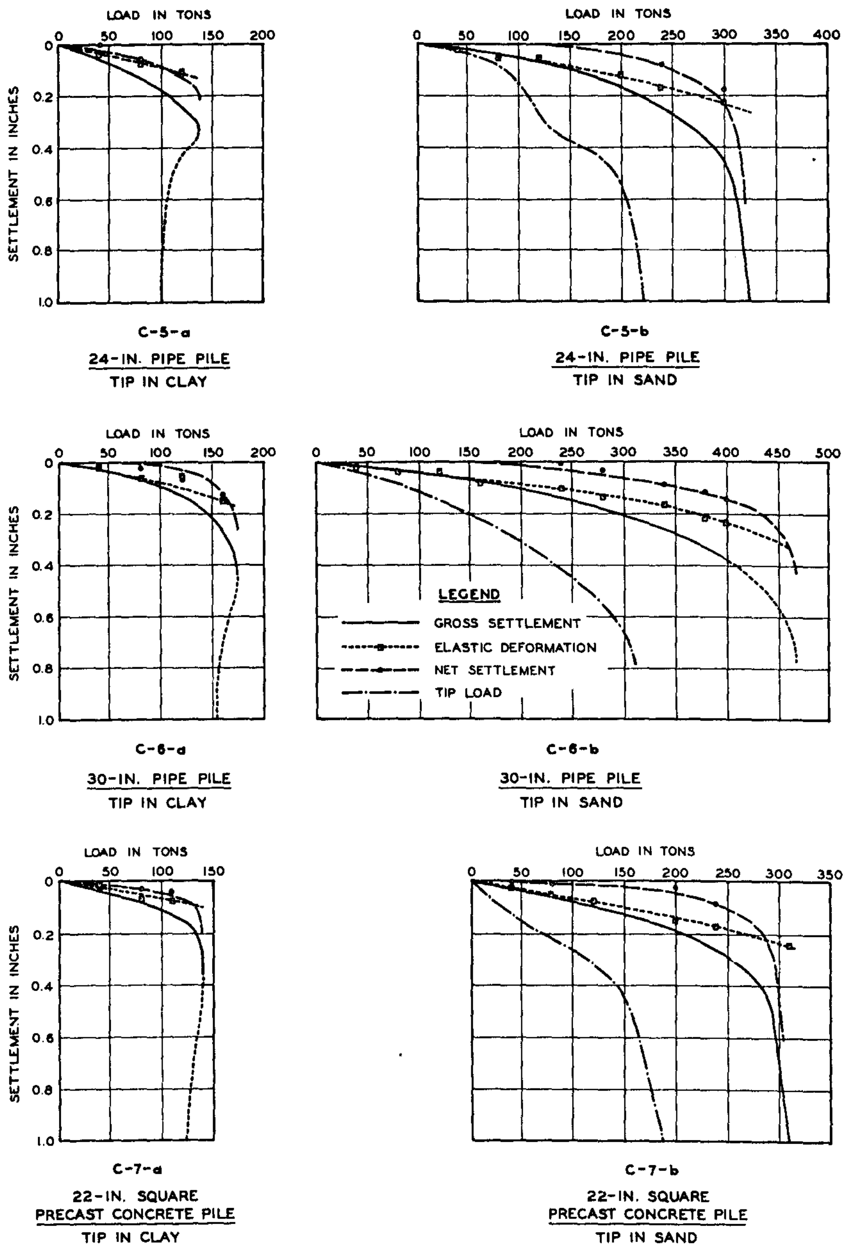


Fig. 31. Load settlement curves, piles C-5 through C-7

a. Determine the load which produced a net settlement of 0.25 in.

b. Determine the load indicated by the intersection of tangent lines to the flatter portion of the gross settlement curve and to the steeper portion of the same curve.

c. By inspection, determine the load beyond which there was an increase in gross settlement disproportionate to the increase in load.

d. Determine the load at which the slope of the net settlement curve is four times the slope of the elastic deformation curve.

e. By inspection, determine the load beyond which there was an increase in net settlement disproportionate to the increase in load.

85. The results of the analysis of each test are given in table 10 with the average of the five determinations. This average is used in all further analyses. The 30-in. pipe pile driven into sand, C-6-b, did not fail under a maximum load of 400 tons. However, a failure load was estimated for this pipe by extrapolating the settlement-load curve. No significant differences in the values derived by the five determinations are indicated for any pile.

Average bearing capacity of pile tip

86. As the piling was to be designed so that the portion of the pile in the sand would carry the design load, the average relation of the bearing capacity of the pile tip to the diameter of the pile tip was determined. Two methods were utilized to compute the relation. In the first the difference between the average failure loads of the "a" and "b" compression piles, as listed in table 10, was taken as the bearing capacity of the tip of the "b" piles. The second method utilized the tip failure load as determined by inspection of the tip-load curve. The capacity of the tip of each "b" pile as determined by each of the two methods is tabulated in table 10 and is plotted versus the tip diameter on fig. 32. Each of these methods indicates an approximate straight-line relation between the tip load and tip diameter. Since the penetration into sand of all piles was not the same, a direct comparison was not possible.

Table 10

Compression Test Pile Failure Loads

		Method of Failure Load Determination						
Pile	Size and Type	0.25-in. Net Settlement Tons	Tangent Intersection Gross Curve Tons	Inspection of Gross Curve Tons	Slope of Net 4 times Slope of Elastic Tons	Inspection of Net Curve Tons	Average of Five Methods of Interpretation	Inspection of Tip Load Curve for Failure Load Tons
C-1-a	24-in. pipe	134	131	125	132	130	130	---
C-1-b		357	360	320	347	330	343	---
Diff.		223	229	195	215	200	213	185
C-2-a	8-in. tip Monotube	80	78	78	76	73	77	---
C-2-b	tapered	169	170	145	164	155	162	---
Diff.		89	92	67	88	82	85	80
C-3-a	12-in. Monotube	60	57	56	57	55	57	---
C-3-b	Cons. Sect.	149	143	145	148	140	145	---
Diff.		89	86	89	91	85	88	90
C-4-a	18-in. pipe	89	78	80	85	75	81	---
C-4-b		254	258	230	240	240	244	---
Diff.		165	180	150	155	165	163	165
C-5-a	24-in. pipe	139	135	137	135	136	136	---
C-5-b		304	307	305	302	300	304	---
Diff.		165	172	168	167	164	168	185
C-6-a	30-in. pipe	174	173	165	160	160	166	---
C-6-b*		450	462	450	455	445	452	---
Diff.*		266	289	285	295	285	286	280
C-7-a	22-in. square pre-	139	138	135	130	135	135	---
C-7-b	cast conc.	293	290	285	283	285	287	---
Diff.		154	152	150	153	150	152	140

* C-6-b was not loaded to failure; values indicated are based on estimated curves.

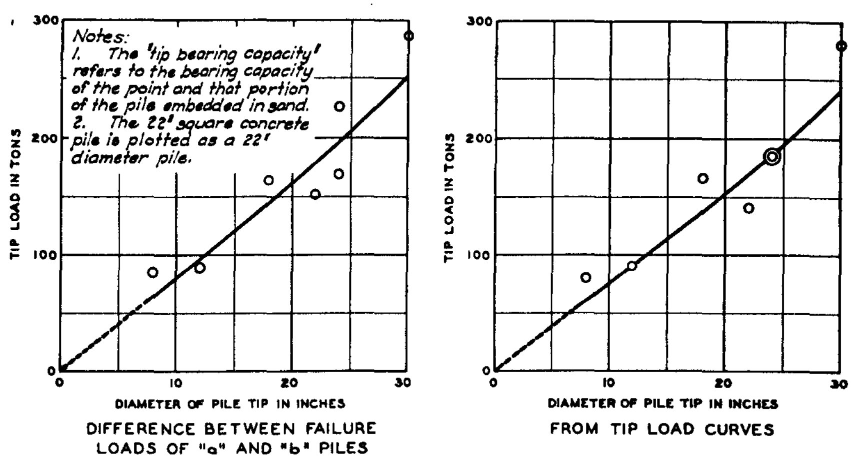


Fig. 32. Average tip bearing capacity

Average skin friction in clay

87. The skin friction developed by the "a" compression piles stopped in clay above the sand at average failure loads was computed using the nominal diameter and the embedded length of the pile. The resulting values were as follow:

Pile	Type of Pile	Diameter of Pile in.	Skin Friction lb/sq ft
C-1-a	Pipe	24	730
C-2-a	Monotube, tapered	8 (tip)	694
C-3-a	Monotube, constant section	12	543
C-4-a	Pipe	18	524
C-5-a	Pipe	24	660
C-6-a	Pipe	30	650
C-7-a	Precast concrete	22 (square)	589
	Average	--	630

The average value of 630 lb per sq ft checks closely the average shear strength of 660 lb per sq ft as determined by unconfined compression tests on undisturbed samples from the foundation borings.

Required size of pile

88. The total capacity of a pile at this site as regards resistance to plunging into the ground is the sum of the capacities developed in the clay by skin friction and in sand by skin friction and point resistance. This capacity was measured by the piles driven into sand, type "b." The load on a pile driven into sand initially will be carried in large part

by skin friction in the clay. However, as the clay at the site is quite compressible, the clay eventually will be relieved of almost all of its supporting stress by consolidation within the clay strata. If the total load applied to the pile is greater than the capacity of the portion of the pile in sand, the pile will not fail by plunging but will settle appreciably because of consolidation. Thus, to guard against detrimental settlements, the long-time capacity of the piles was taken as only the capacity of the tip of the piles in sand. On the basis of data shown on fig. 32 a pile diameter of 20 in. was selected as being adequate to carry the design compression load of 100 tons with a factor of safety of approximately 1.5 against detrimental settlement, and a factor of safety of approximately 3.0 against failure by plunging. Octagonal piles with a minimum dimension of 20 in. were used for lengths up to 100 ft; 20-in. square piles were used where lengths greater than 100 ft were required.

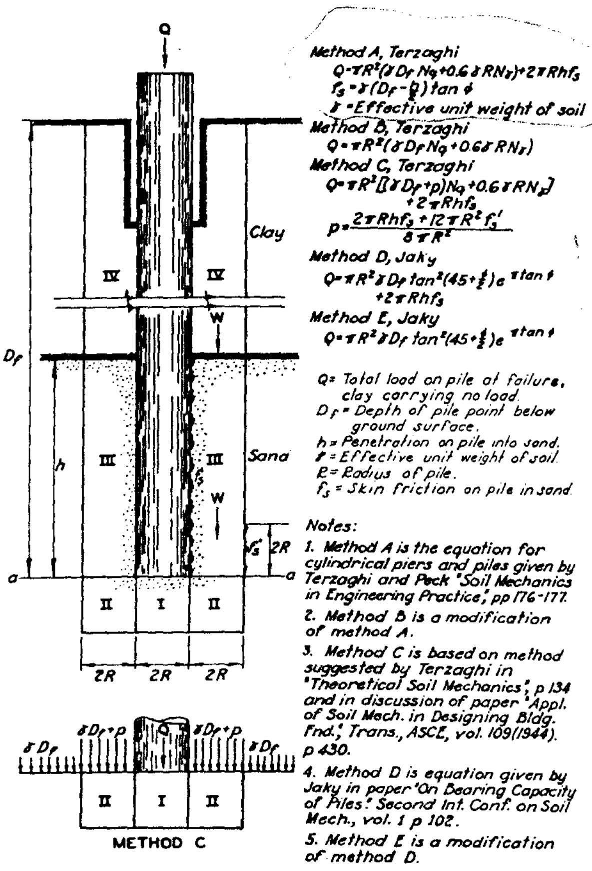


Fig. 33. Theoretical bearing capacity of pile driven into sand

Bearing capacity formulas

89. For comparison purposes, the bearing capacity of the different piles was computed using the following basic methods: (a) the bearing capacity equation for cylindrical piers and piles, by Terzaghi and Peck; (b) an approach suggested by Terzaghi for deep piers; and (c) an equation by Jaky for piles (see fig. 33).

90. The bearing capacities of various types of piles driven through 85 ft of overburden clay and 5 ft into sand were computed using the methods given in fig. 33. A shear strength of c = 0 and $\phi = 30^\circ$ was used for the sand. The results obtained from the computations are represented by the curves

on fig. 34. The measured tip bearing capacities of the piles, adjusted to the same conditions assumed for computation of the curves (table 11), are also shown on this figure. These data indicate that method "A" (Terzaghi and Peck) and method "D" (Jaky) fit the test data reasonably well.

Notes:

(•)
1. Failure loads from tip load curves corrected for additional length of "b" piles in clay and adjusted to a 5-ft penetration into sand and an 85-ft depth of the point on basis of Terzaghi A equation.
2. Curves computed by equations on figure 35 for $\phi = 30°$, a 5-ft penetration into sand, and an 85-ft depth of the point.

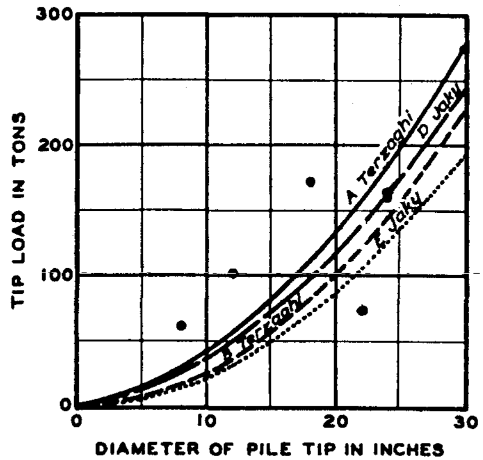


Fig. 34. Theoretical and observed tip bearing capacity of piles driven into sand

Tension Test Piles

Load arrangement

91. The tension piles were loaded by jacking against 12 timber anchor piles, 6 in a row on each side of the pile. An 8-in. I-beam was embedded about 9 ft into the concrete filling the pile shell to make the tension connection

Table 11

Adjusted Tip Failure Loads

Pile	Size and Type	Tip Failure Load,[1] ton	Corrected Tip Failure Load,[2] ton	Adjusted Tip Failure Load,[3] ton
C-1-b	24-in. pipe	185	176	160
C-2-b	8-in. tip Monotube tapered	80	74	62
C-3-b	12-in. Monotube constant section	90	85	101
C-4-b	18-in. pipe	165	159	173
C-5-b	24-in. pipe	185	171	163
C-6-b	30-in. pipe	280	276	275
C-7-b	22-in.-square precast concrete	140	120	115

1 Tip failure load determined by inspection of tip load curve.
2 Correction made to loads indicated by tip load curves on basis of greater length in clay of "b" piles than "a" piles.
3 Adjustment of corrected loads made to 5-ft penetration into sand and 85-ft depth of point using Terzaghi A equation.

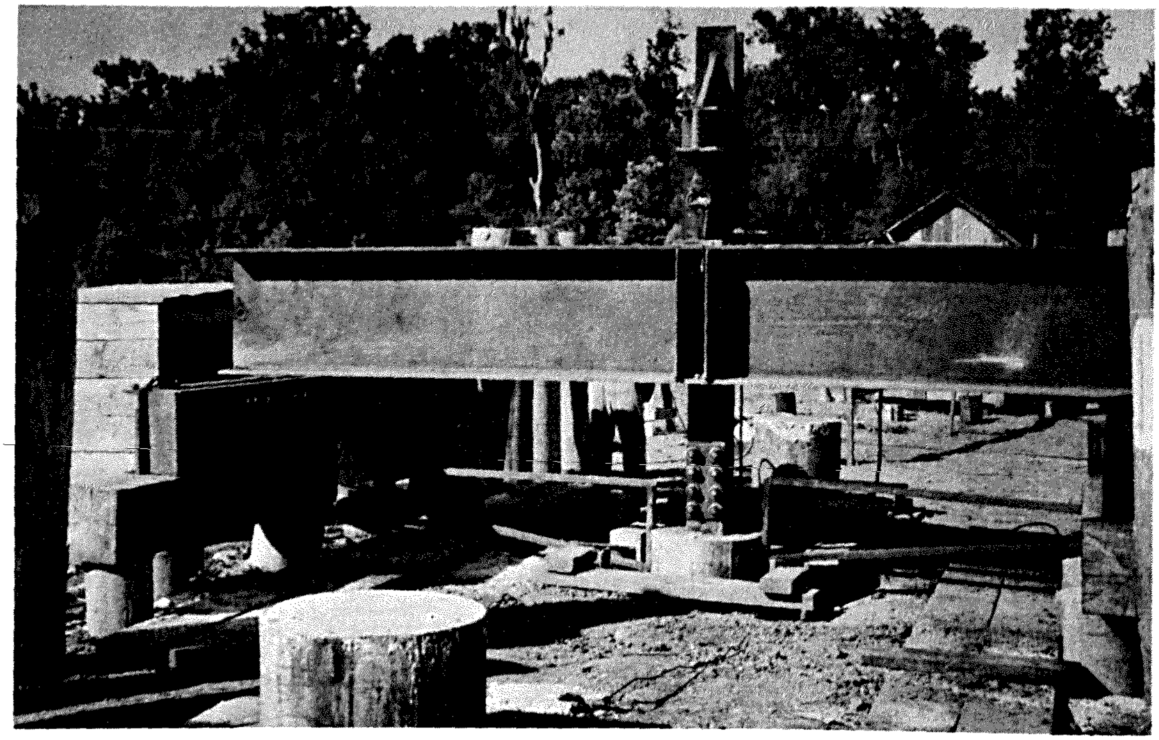


Fig. 35. Tension test on 24-in. pipe pile (T-1)

as shown in fig. 35. Two 100-ton hydraulic jacks were used to apply load to the test piles.

Loading procedure

92. Compression loading. As the piles beneath the structure will be subjected to a compression load prior to any tension load, piles T-1, T-2, T-4, T-5, and T-6 were subjected to a compression load of 85 tons, and pile T-3 was subjected to a load of 50 tons prior to testing in tension. The loads were applied at the rate of 1-1/2 tons per minute and were maintained for 12 hours, after which the loads were removed at the rate of 1.0 ton per minute.

93. Tension loading. The pull was applied to the test piles in 8-ton increments at the rate of 1.0 ton per minute. After completion of the 24-, 48-, 72-, and 96-ton loads on pile T-5 and at the end of the test, rebound was allowed. Unloading and reloading were at the rate of 1.0 ton per minute. All piles, except T-5 and T-6, were loaded to failure; the limit of the loading equipment was reached before these two piles failed.

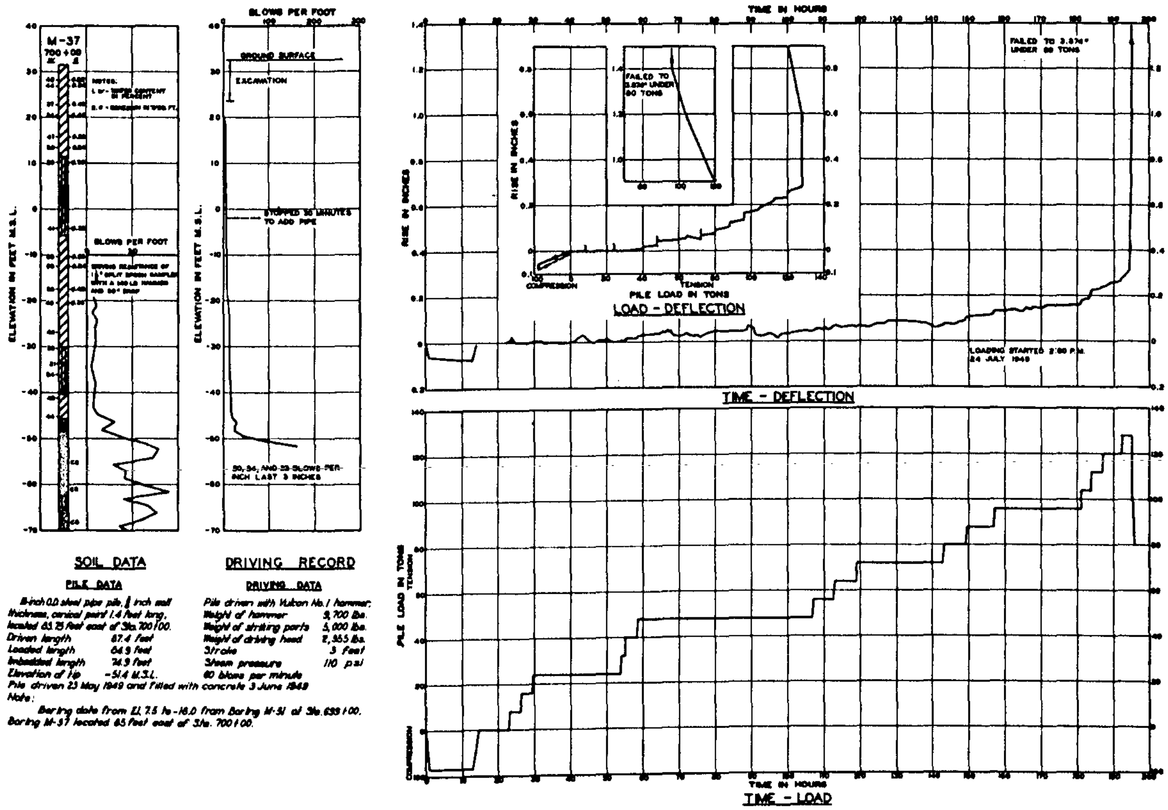


Fig. 36. Tension pile test data, pile T-4, 18-in. pipe pile

Test data

94. Typical test data for tension pile T-4 are shown graphically on fig. 36. Curves of gross movement of the pile head were developed from the test data on each pile and are shown in fig. 37. In the preparation of these curves, the points representing loads held for 24 hours were given primary consideration. Thus the resulting curve should represent (up to the maximum load) the movement of the pile head under sustained load (neglecting expansion of the clay foundation).

Pile failure loads

95. In analysis of the load-rise curves on fig. 37, two different conditions of failure were considered. The first was that the rise of the piles during loading would be sufficient to produce a detrimental effect on the structure. A gross rise of 0.25 in. was taken to be the limiting criterion for this condition which was used to determine the

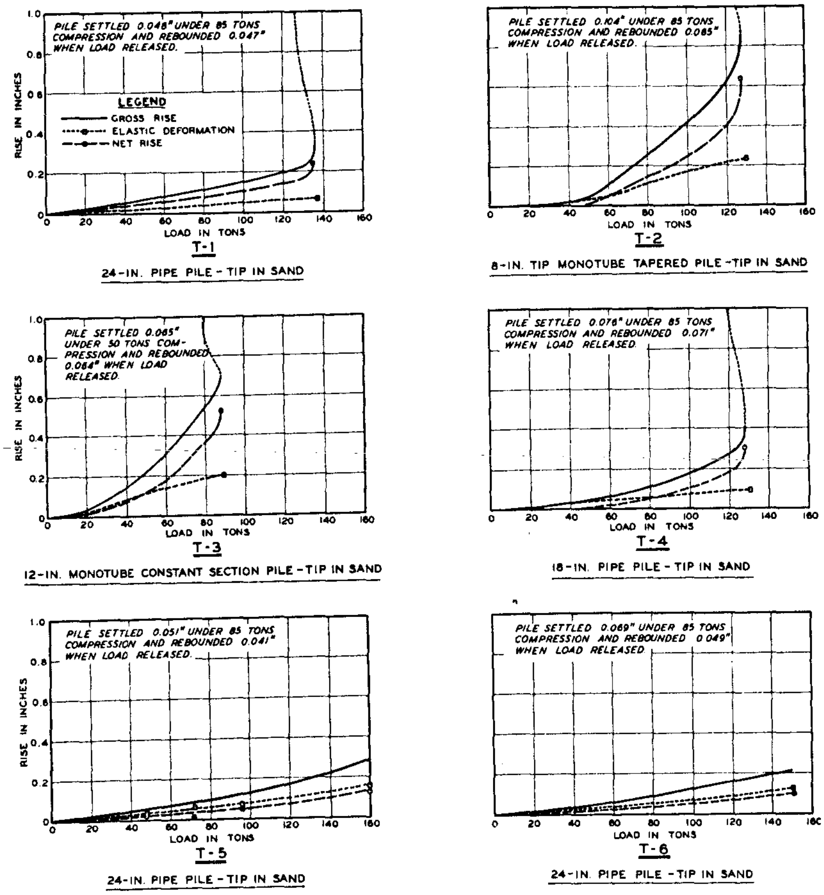


Fig. 37. Load rise curves, piles T-1 through T-6

allowable tension load. The second condition was the complete failure of the pile under load, causing a reduction in the ability of the pile to carry the load. The failure load for the second condition was determined by an inspection of the gross and net rise curves for the load beyond which deflection increased disproportionately to the increase in load. The second condition was used to determine the average tension skin friction in the clay. The results of the analysis of data from each test are presented in table 12.

Table 12

Tension Test Pile Failure Loads

Pile	Size and Type	Determination of Failure Load, ton			
		0.25-in. Gross Rise	Gross Rise*	Inspection of Curve Gross Rise	Inspection of Curve Net Rise
T-1	24-in. pipe	133	139	134	134
T-2	8-in. tip Monotube tapered	80	75	125	122
T-3	12-in. Monotube constant section	53	52	88	87
T-4	18-in. pipe	117	117	125	125
T-5	24-in. pipe	150	141	200	200
T-6	24-in. pipe	167	134	160	160

* Load for 0.25-in. gross rise corrected to an embedded length of 75 ft.

Average load capacity

96. As the embedded lengths varied somewhat, the loads which produced a gross rise of 0.25 in. were corrected to an embedded length of 75 ft by assuming that the tension capacity was directly proportional to the length. These corrected values, listed in table 12, are total loads applied to the head of the pile. These loads are also plotted against the average pile diameter on fig. 38. The tapered Monotube pile was plotted at a diameter of 13 in., the average of the tip and butt diameters, which were 8 and 18 in., respectively. The average line drawn through the points indicates that a 20-in.-diameter round pile 75 ft long can carry the design tension load of 25 tons with a factor of safety of approximately 4.5.

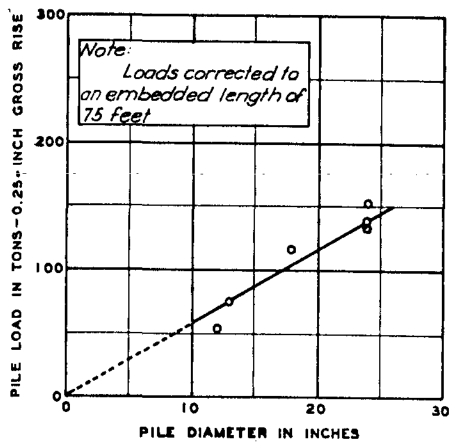


Fig. 38. Average tension capacity

Average skin friction in clay

97. It is not possible to compute directly from the test data on the tension piles the skin friction developed in the clay. Tension piles were driven into the sand, and the sand normally should have a materially different skin friction on the pile than the clay. The total load carried by the clay was computed by subtracting from the pile failure load, determined by inspection of the net curve, the buoyant weight of the pile and the total force developed by friction in the sand, assuming a coefficient of earth pressure of 1.0 and an angle of internal friction of the sand of 30°. Unit skin friction in the clay was computed from this corrected failure load, using the embedded length in the clay and the nominal diameter of the pile. Resulting values are tabulated in table 13. As piles T-5 and T-6 were not loaded to failure, the skin friction values listed for them were the values developed under the maximum load. Average skin friction shown in table 13 was based on the results from piles T-1 through T-4. From the data in table 13 it is concluded that the skin friction developed by either a tension or compression pile in the clay at failure is fairly close to the average strength of the foundation clays as indicated

Table 13

Corrected Skin Friction in Clay of Tension Test Piles

Pile	Size and Type	Tension Skin Friction lb/sq ft	Compression Skin Friction, lb/sq ft
T-1	24-in. pipe	608	730
T-2	8-in. tip Monotube tapered	829	694
T-3	12-in. Monotube	706	543
T-4	18-in. pipe	639	523
T-5	24-in. pipe	651	659
T-6	24-in. pipe	350	588
Average		695	623
Average of tension and compression tests		660	

Note: Average based on piles T-1 through T-4.
Average skin friction of piles C-1-a through C-7-a, 630 lb/sq ft.
Average cohesive strength of foundation clays as determined by unconfined and quick triaxial tests, 660 lb/sq ft.

by unconfined and unconsolidated undrained triaxial compression tests.

98. The fact that the tapered Monotube pile T-2 indicated the highest skin friction is not believed to be highly significant, because the variations in skin friction indicated by the various types of piles could be due to variations in soil strengths and not necessarily to characteristics of the piles. From the one test made it appears that at the site tested a pile with a small taper, such as 1 in. in 7 ft for piles C-2 and T-2, will respond to loading either in tension or compression in a manner similar to a pile of constant section with the same average diameter.

Dynamic Pile Driving Formulas

Formulas investigated

99. Six dynamic pile driving formulas were used to compute the capacity of the compression piles driven into sand. These formulas as given on the following page include the intended factors of safety as indicated. The symbols used are grouped after the formulas.

Engineering-News	$R = \dfrac{2\ W\ h}{s + 0.1}$	(F.S. = 6)
Engineering-News (modification 1)	$R = \dfrac{2\ W\ h}{s + 0.1\ \dfrac{P}{W}}$	(F.S. = 6)
Engineering-News (modification 2)	$R = \dfrac{2\ W\ h}{s + 0.3}$	(F.S. = 6)
Eytelwein (modified)	$R = \dfrac{2\ W\ h}{s + 0.3\ \dfrac{P}{W}}$	(F.S. = 6)
Canadian National Building Code	$R = \dfrac{4\ W\ h\ n\ 0.9}{s + 1/2\ C}$	(F.S. = 3)
	$n = \dfrac{W + 0.5\ e^2\ P}{W + P}$	
	$C = \dfrac{3\ R}{A}\left(\dfrac{12\ L}{E} + 0.0001\right)$	
Pacific Coast Uniform Building Code	$R_u = \dfrac{12\ Wh\ \dfrac{W + KP}{W + P}}{s + \dfrac{12\ R_u\ L}{A\ E}}$	(F.S. = 1)

K = coefficient; 0.25 for steel piles; 0.10 for all other piles

F.S. = 4 recommended by Pacific Coast code

A = average cross-sectional area of driven parts in square inches

C = coefficient of restitution; 0.5 for ram striking steel anvil on steel or concrete piles; 0.4 for ram striking steel anvil containing a wood cushion

E = modulus of elasticity of pile material

h = height of free fall of ram in feet

L = length of pile in feet

P = weight of driven pile in pounds

R = design carrying capacity of pile in pounds

R_u = ultimate carrying capacity of pile in pounds

s = set per blow in inches

W = weight of falling mass in pounds

The Canadian National Building Code and the Pacific Coast Uniform Building Code formulas are modifications of the Hiley formula. It was not possible to use the Hiley formula, as the rebound of the pile during driving was not measured. The driving data of the piles stopped above the sand ("a" piles) were not used to compute the indicated capacity by the driving formula. It is a known fact that pile-driving formulas are meaningless for friction piles driven in clay.

Relation of formulas to test results

100. The driving resistance for the last 3 in. of penetration was used in each formula to compute the design capacity of each of the compression piles driven into sand. These values are given in table 14, together with computed ratios of formula capacities based on load tests. It appears in general from the data presented that the pile-driving formulas did not give capacities which checked either the total or tip capacity of the test piles at the site of the Morganza Floodway Control Structure. The only good agreement obtained was between the "ultimate capacity" as computed from the Pacific Coast formula and the "ultimate tip test capacity." In connection with the average ratios shown in table 14, it is pointed out that formulas which give ratios greater than 1 are on the radical side, whereas formulas which give ratios less than 1 are on the conservative side. The ultimate capacity indicated by the Engineering-News formula and its modifications and the Eytelwein formula was considerably more than either the total capacity of the pile or the capacity of the tip alone (ratios of 1.5 to 4.0). The formulas of the Canadian National and Pacific Coast Uniform Building Codes indicated conservative ultimate capacities, as the ratio of code capacity to test capacity ranged from 0.4 to 1.0.

Table 14

Comparison between Test Results and Pile Driving Formulas

Pile	Size and Type	(1) Total Ultimate Test Capacity Tons	(2) Design Short Time (Test) Load F.S. = 2.0	(3) Ultimate Tip Test Capacity Tons	(4) Design Long Time (Test) Load F.S. = 1.5	(5) Engr. News R Tons	(5) Engr. News (Mod. 1) R Tons	(5) Engr. News (Mod. 2) R Tons	(5) Eytelwein R Tons	(5) Canadian National R Tons	(5) Pacific Coast R Tons
C-1-b	24-in. pipe	343	172	185	123	230	243	92	97	45	51
C-2-b	8-in. tip Monotube tapered	162	81	80	53	109	123	45	52	19	21
C-3-b	12-in. Monotube constant section	145	73	90	60	118	183	46	78	18	19
C-4-b	18-in. pipe	244	122	165	110	118	92	46	37	29	29
C-5-b	24-in. pipe	304	152	185	123	230	244	100	101	43	54
C-6-b	30-in. pipe	452	226	280	187	238	204	92	79	48	55
C-7-b	22-in. sq precast concrete	287	144	140	93	230	56	100	19	59	39
(6) Av. Ratio:	$\frac{\text{Design Formula Capacity}}{\text{Design Short Time (Test) Load}}$					1.35	1.30	0.55	0.53	0.27	0.28
Av. Ratio:	$\frac{\text{Design Formula Capacity}}{\text{Design Long Time (Test) Load}}$					1.80	1.69	0.74	0.69	0.36	0.36
Av. Ratio:	$\frac{\text{Ultimate Formula Capacity}^{(7)}}{\text{Total Ultimate Test Capacity}}$					4.05	3.90	1.65	1.59	0.41	0.55
Av. Ratio:	$\frac{\text{Ultimate Formula Capacity}^{(7)}}{\text{Ultimate Tip Test Capacity}}$					7.20	6.76	2.96	2.76	0.72	0.96

(1) The total test capacity was taken from "Average Failure Load" - Table 1.
(2) Total ultimate test capacity ÷ 2.0. In other words, the factor of safety against plunging under a "short time" load is 2.0.
(3) The ultimate tip test capacity was determined by inspection of tip load curves.
(4) Ultimate tip test capacity ÷ 1.5. In other words, the factor of safety against detrimental settlement under a "long time" load is 1.5.
(5) Formula capacities indicated by the pile driving formulas are design loads based on the driving resistance for the last 3-in. of driving.
(6) Average ratios were computed by averaging data for individual piles.
(7) Design formula capacity times recommended factors of safety.

PART VI: FOUNDATION FOR CONTROL STRUCTURE

Description of Pile Foundation

101. Studies of the settlement of the control structure founded on a system of friction piles driven into clay indicated that settlements of as much as 10 in. might be expected. This amount of settlement was considered intolerable and therefore it was decided to drive compression piles to an underlying stratum of bearing sand. On the bases of the pile tests previously discussed, it was concluded that 20-in.-diameter piles driven 5 ft into the underlying sand would support the compression loads of 100 tons* and would also take tension loads of 25 tons, which can exist. In the final design, the piers for the gated portion of the control structure are supported by twenty-seven, 20-in., precast concrete piles driven to sand on a 2-on-1 batter. Eleven of the piles are battered upstream and sixteen downstream. A plan of the pile foundation and section of the gated portion of the structure are shown in fig. 3, page 3. Piles under 100 ft in length are of octagonal shape, whereas piles over 100 ft in length are square.

102. Piles beneath the abutments, approach piers, and wing walls are of the same type and dimensions as those beneath the gated portion of the structure; however, most of the piles under the wing walls were driven vertically rather than on a batter. The design load on the piles beneath the abutments was reduced to 50 tons, and that for the piles beneath the approach piers adjacent to the abutments to 92 tons to allow for drag on these piles resulting from settlement of the embankment fill adjacent to the abutments. The batter of the piles beneath the abutments and first approach piers was steepened to minimize the effect of foundation settlement on these piles.

---

* Maximum and average pile loads due to dead load of structure = 86 tons and 65 tons, respectively.
Maximum pile load with maximum water load against structure and design uplift acting = 94 tons.
Maximum pile load with maximum water load against structure and no uplift against base of structure = 118 tons.
The above indicated pile loads are in compression.

103. The average length of piles beneath the weir or gated portion was 86 ft with some piles as long as 110 ft. The average length of the piles beneath the different piers is listed in table 15, pages 94 and 95.

104. The piles were made of concrete which had an average 14-day compressive strength of 4300 psi and an average 28-day strength of 5000 psi. They are reinforced with 8 longitudinal 1-1/8-in.-diameter bars with 3/8-in. spiral reinforcement on a pitch of 6 in. The spiral reinforcement of the octagonal piles has a pitch of 2 in. at the tip and head of the pile. The tips of the piles are tapered in 10 in. to a point with an approximate diameter of 6 in.

105. At the time this report was written, there have been only 2.5 ft of water against the structure; therefore, the maximum loading to which the piles beneath the weir have been subjected is that created by the weight of the structure.

Settlement of Structure

106. The greater portion of the deep sand stratum beneath the structure contains no compressible clay strata; therefore no significant settlement is expected of the piles driven along such reaches. However, some relatively thin strata of clay were found within the upper part of the sand stratum at several locations. In addition, a few lenses of compressible lignitic sand were found scattered through the sand at some locations. The elimination of all differential settlements would have necessitated driving the piles to such a penetration that the pile tips would be below all compressible strata. However, cost studies indicated the most economical design to be one based on an allowable differential settlement of 1 in. Accordingly, the gates of the superstructure were designed to absorb a differential settlement of 1 in. between piers in adjoining monoliths. Therefore, penetration through compressible strata was not attempted where settlement analyses indicated differential settlements of less than 1 in.

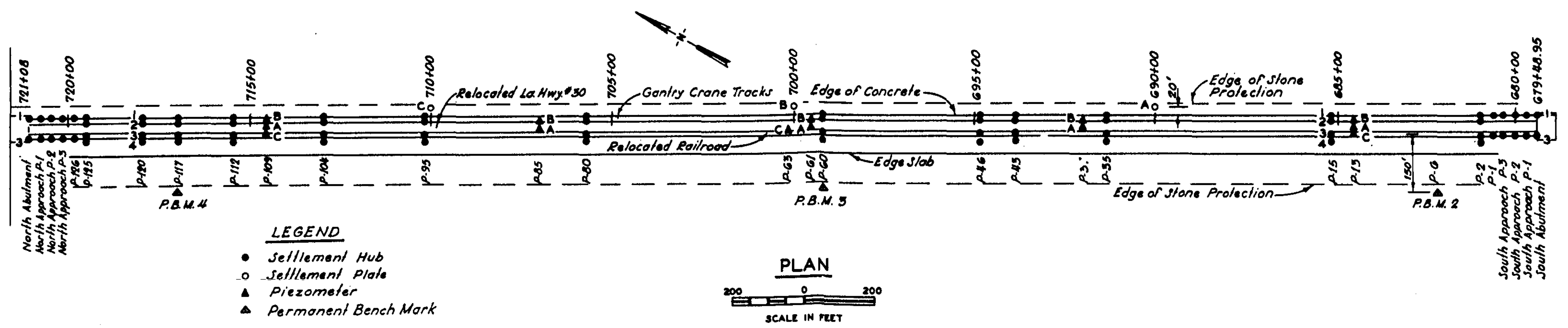
107. At three locations, station 677+50 (south abutment) to station 681+50, station 693+40 to station 697+50, and station 715+00 to station

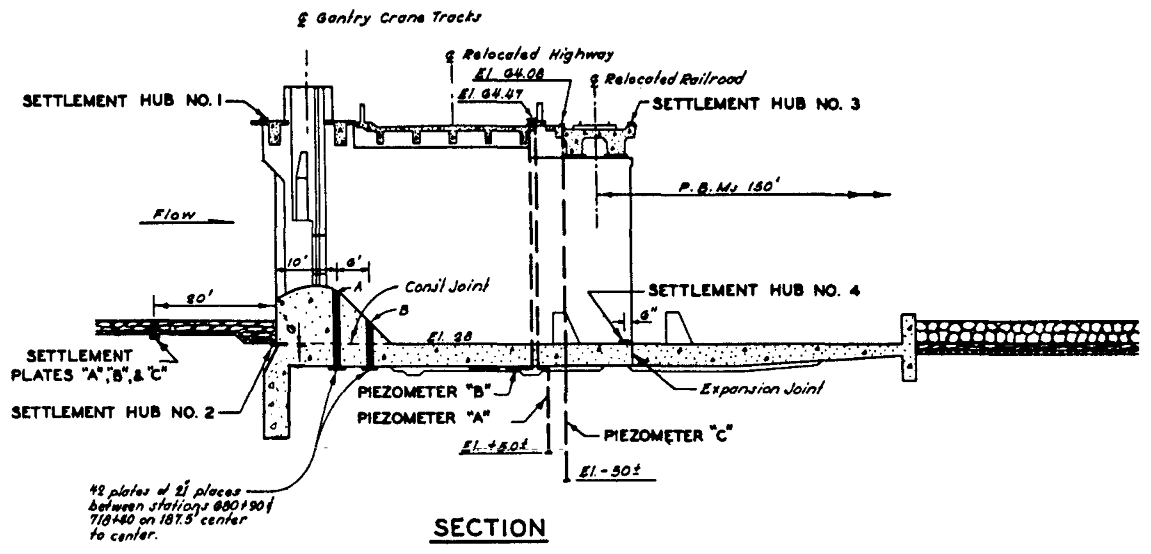
717+20, strata of clay exist that it was thought might cause differential settlements of more than 1 in. (see fig. 43, page 97) unless the strata were penetrated by the piles. It was believed from split-spoon driving data that by overdriving, the piles might be driven through the sand overlying these clay strata and into the bearing sand beneath the clay. Therefore, along these three reaches an attempt was made to drive the piles through the compressible strata in the upper part of the sand by requiring additional driving to that normally specified; the piles were either driven through the clay strata or were driven to refusal, as subsequently defined. It was believed that overdriving the piles would limit the differential settlements between monoliths to probably less than 1/2 in. and not more than 1 in.

108. At a few locations where the compressible strata in the upper part of the sand foundation could not be penetrated, it was estimated that differential settlements greater than 1 in. might eventually occur. It was realized that at these locations the gates between adjacent monoliths might be affected adversely and, as a result, some future maintenance and repairs might be necessary. However, because of the monetary savings involved, it was concluded that the possibility of a few gates requiring some future maintenance was proper engineering design.

109. Before driving of the pile foundation was begun, an estimate was made of the settlement that might occur along those reaches where clay strata exist if the piles were not driven through them. The results of these settlement computations, together with settlements observed since construction, are summarized in the following paragraphs. The success of the overdriving in these reaches is also discussed.

110. In order that settlements might be measured during and after construction of the structure, metal hubs were embedded in the concrete superstructure, abutments, and approach piers as they were poured. A plan and diagram of the different settlement hubs installed and the dates of installation are shown in fig. 39. The first settlement hubs, Nos. 2 and 4, were installed in the base of the structure when it was poured. However, the initial elevations of these hubs were not determined until after the concrete had set around the pile heads. Another set of hubs





ELEVATIONS OF SETTLEMENT HUBS ON CONTROL STRUCTURE

LOCATION	NORTH ABUTMENT		NORTH APPROACH PIER NO. 1		NORTH APPROACH PIER NO. 2		NORTH APPROACH PIER NO. 3		PIER NO. 126		PIER NO. 125			
STATION	721+08.70		720+75.70		720+44.45		720+13.20		719+81.95		719+50.70			
HUB NO.	1	3	1	3	1	3	1	3	1	3	1	2	3	4
12 July 1951	—	—	—	—	—	—	—	—	—	—	—	—	—	—
26 October 1951	—	—	—	—	—	—	—	—	—	—	—	—	—	—
19 March 1952	—	—	—	—	—	—	—	—	—	—	—	27.761	—	27.938
3 December 1952	64.877	63.987	64.831	63.998	64.846	64.038	64.865	63.979	64.859	64.003	64.861	—	64.014	27.900
25 March 1953	64.861	63.974	64.809	63.984	64.822	64.030	64.842	63.969	64.844	63.994	64.854	—	64.015	—
23 July 1953	64.851	63.965	64.797	63.973	64.809	64.019	64.831	63.959	64.834	63.987	64.849	—	64.009	27.892
3 November 1953	64.842	63.958	64.787	63.963	64.797	64.007	64.818	63.948	64.822	63.983	64.839	—	64.003	27.887
11 February 1954	64.836	63.949	64.779	63.953	64.789	63.998	64.812	63.941	64.814	63.976	64.831	27.646*	63.992	27.880

LOCATION	PIER NO. 120				PIER NO. 117				PIER NO. 112				PIER NO. 104			
STATION	717+94.45				717+00.70				715+44.45				712+94.45			
HUB NO.	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4
12 July 1951	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
26 October 1951	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
19 March 1952	—	27.878	—	27.971	—	27.853	—	27.918	—	27.711	—	27.955	—	27.818	—	27.975
3 December 1952	64.838	—	64.079	27.928	64.884	—	64.000	27.882	64.856	—	64.038	27.926	64.839	—	64.048	27.948
25 March 1953	64.841	—	64.085	—	64.895	—	64.007	—	64.869	—	64.047	—	64.852	—	64.058	—
23 July 1953	64.840	—	64.081	27.937	64.893	—	64.000	27.887	64.865	—	64.041	27.927	64.851	—	64.053	27.952
3 November 1953	64.832	—	64.074	27.930	64.890	—	63.995	27.882	64.859	—	64.036	27.923	64.846	—	64.047	27.947
11 February 1954	64.823	27.809	64.067	27.923	64.882	27.777	63.987	27.879	64.852	27.648	64.028	27.918	64.843	27.766	64.041	27.945

LOCATION	PIER NO. 95				PIER NO. 80				PIER NO. 60				PIER NO. 46				PIER NO. 43			
STATION	710+13.20				705+44.45				699+19.45				694+81.95				693+88.20			
HUB NO.	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4
12 July 1951	—	—	—	—	—	27.673	—	27.996	—	27.536	—	27.946	—	27.805	—	28.007	—	—	—	—
26 October 1951	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	27.557	—	27.920
19 March 1952	—	27.577	—	27.943	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
3 December 1952	64.941	—	64.068	27.915	64.936	—	64.051	27.959	64.898	—	64.047	27.912	64.914	—	64.070	27.966	64.940	—	64.080	27.892
25 March 1953	64.954	—	64.079	—	64.950	—	64.061	—	64.916	—	64.061	—	64.926	—	64.086	—	64.952	—	64.092	—
23 July 1953	64.952	—	64.075	27.919	64.948	—	64.061	27.960	64.917	—	64.057	27.917	64.926	—	64.078	27.967	64.950	—	64.087	27.896
3 November 1953	64.944	—	64.070	27.917	64.943	—	64.058	27.959	64.908	—	64.056	27.919	64.918	—	64.082	27.973	64.942	—	64.090	27.900
11 February 1954	64.944	27.514	64.066	27.918	64.946	27.612	64.049	27.959	64.908	27.481	64.048	27.912	64.919	27.759	64.075	27.967	64.933	27.505	64.089	27.897

ELEVATIONS OF SETTLEMENT HUBS ON CONTROL STRUCTURE

LOCATION	PIER NO. 35				PIER NO. 15				PIER NO. 2			
STATION	691+39.20				684+13.20				681+06.95			
HUB NO.	1	2	3	4	1	2	3	4	1	2	3	4
26 October 1951	—	27.765	—	27.892	—	27.580	—	27.896	—	—	—	—
19 March 1952	—	—	—	—	—	—	—	—	—	27.492	—	27.977
3 December 1952	64.926	—	64.059	27.865	64.893	27.544	64.037	27.854	64.869	27.462	64.028	27.932
25 March 1953	64.934	—	64.068	—	64.898	—	64.049	—	64.870	—	64.033	—
23 July 1953	64.932	—	64.064	27.866	64.894	—	64.045	27.853	64.862	—	64.033	27.932
3 November 1953	64.929	—	64.066	27.868	64.894	—	64.034	27.857	64.857	—	64.024	27.928
11 February 1954	64.929	27.712	64.066	27.868	64.895	27.511	64.031	27.856	64.857	27.399*	64.017	27.925

LOCATION	PIER NO. 1		SOUTH APPROACH PIER NO. 3		SOUTH APPROACH PIER NO. 2		SOUTH APPROACH PIER NO. 1		SOUTH ABUTMENT	
STATION	680+75.70		680+44.45		680+13.20		679+81.95		679+48.95	
HUB NO.	1	3	1	3	1	3	1	3	1	3
26 October 1951	—	—	—	—	—	—	—	—	—	—
19 March 1952	—	—	—	—	—	—	—	—	—	—
3 December 1952	64.922	64.043	64.836	64.000	64.819	63.997	64.851	64.031	64.912	64.013
25 March 1953	64.920	64.044	64.828	63.999	64.806	63.997	—	64.028	64.900	64.005
23 July 1953	64.911	64.042	64.819	63.992	64.797	63.989	***	64.020	64.899	63.996
3 November 1953	64.901	64.037	64.808	63.986	64.783	63.980	—	64.013	64.882	63.987
11 February 1954	64.905	64.035	64.809	63.981	64.784	63.973	—	64.006	64.874	63.983

ELEVATIONS OF SETTLEMENT PLATES BENEATH CONTROL STRUCTURE

STATION	680+90		682+77		684+65		686+52		688+40		690+27		692+14		694+02		695+90		697+77		699+65	
PLATE	A	B	A	B	A	B	A	B	A	B	A	B	A	B	A	B	A	B	A	B	A	B
Orig. Elev. Top of Rod	31.31	37.15	31.34	37.21	31.20	37.12	31.21	37.05	31.48	37.10	31.35	37.01	31.25	37.08	31.21	37.08	31.52	37.03	31.34	37.11	31.39	36.99
Length of Rod	7.03	12.82	8.58	14.47	7.01	12.90	7.03	12.93	7.29	12.97	7.30	12.94	7.04	12.91	7.08	12.93	7.24	12.93	7.20	13.00	7.28	12.92
Orig. Plate Elev.	24.28	24.33	22.76	22.74	24.19	24.22	24.18	24.12	24.19	24.13	24.05	24.07	24.21	24.17	24.13	24.15	24.28	24.10	24.14	24.11	24.11	24.07
28 January 1954	31.23	37.07	31.24	37.11	31.10	36.99	31.14	36.91	31.42	36.98	31.26	36.91	31.15	37.02	31.14	37.01	31.46	36.95	31.33	36.96	31.34	36.97

STATION	701+52		703+40		705+27		707+15		709+02		710+90		712+77		714+65		716+52		718+40	
PLATE	A	B	A	B	A	B	A	B	A	B	A	B	A	B	A	B	A	B	A	B
Orig. Elev. Top of Rod	31.30	37.05	31.31	37.00	31.32	37.09	31.17	36.98	31.79	37.34	31.29	37.14	31.35	37.29	31.70	37.12	31.51	37.09	31.49	37.52
Length of Rod	7.15	12.90	7.18	12.92	7.10	12.88	7.11	12.92	7.29	12.94	7.09	12.94	7.13	12.95	7.30	12.94	7.28	12.94	7.10	12.89
Orig. Plate Elev.	24.15	24.15	24.13	24.08	24.22	24.21	24.06	24.06	24.50	24.40	24.20	24.20	24.22	24.34	24.40	24.18	24.23	24.15	24.39	24.63
28 January 1954	31.22	36.99	31.24	37.03	31.29	37.05	31.12	36.94	31.66	37.24	31.22	37.04	31.29	37.23	31.66	37.05	31.49	37.08	31.45	37.48

NOTE: "A" settlement plates 37.9 ft upstream of center line of railroad. "B" plates 43.9 ft upstream of center line of railroad.

ELEVATIONS OF SETTLEMENT PLATES BENEATH UPSTREAM RIPRAP

STATION	710+00	700+00	690+00
PLATE	C	B	A
11 February 1953	29.18	28.85	29.68
6 March 1953	29.14	28.81	29.55
10 April 1953	29.18	28.85	29.62
30 April 1953	29.19	28.87	29.63
11 February 1954	29.138	28.820	29.529

MISS. RIVER ELEVATION AT MORGANZA, LA.

Date	Elev ††	Date	Elev ††
12 Jul 51	35.7	5 Feb 54	25.6
19 Mar 52	34.9	11 Feb 54	19.0
3 Dec 52	9.7	12 Feb 54	17.5
11 Feb 53	21.4	25 Mar 54	13.4
6 Mar 53	28.9	1 Apr 54	13.3
26 Mar 53	34.3	7 Apr 54	14.9
10 Apr 53	33.7		
30 Apr 53	32.0		
23 Jul 53	15.3		
3 Nov 53	5.1		
29 Jan 54	21.9		

ELEVATIONS OF WATER SURFACE IN PIEZOMETERS

LOCATION	PIER NO. 109			PIER NO. 85		PIER NO. 63	PIER NO. 61		PIER NO. 37		PIER NO. 13		
STATION	714+50.70			707+00.70		700+13.70	699+50.70		692+00.70		684+50.70		
PIEZOMETER	A-5	B-5	C-3	A-4	B-4	C-2	A-3	B-3	A-2	B-2	A-1	B-1	C-1
Elev top of pipe	64.35	64.35	64.06	64.38	64.38	64.07	64.40	64.40	64.45	64.45	64.40	64.40	64.03
Water elev in pipe 29 Jan 1954	29.60	28.15	**	**	**	-	28.30	28.25	-	28.20	-	28.22	17.11
Water elev in pipe 5 Feb 1954	29.58	28.03	**	**	**	17.68	28.33	28.12	-	28.20	27.08	28.11	20.43
Water elev in pipe 12 Feb 1954	29.58	27.85	**	**	**	19.37	28.33	28.12	-	28.20	27.08	28.10	20.43
Water elev in pipe 25 Mar 1954	-	-	18.60	-	27.50	-	27.50	-	-	27.50	-	-	-
Water elev in pipe 7 Apr 1954	-	27.95	18.46	28.03	-	18.07	27.95	28.10	-	28.20	29.60	27.90	18.23

PERMANENT BENCH MARKS

NO.	LOCATION	ELEVATION TOP	ELEVATION BOTTOM
1	350 ft downstream from center line T&P Railroad, Sta 581+00	59.400	-66.4
2	150 ft downstream from center line T&P Railroad, Pier 6, Sta 682+13	32.243†	-64.7
3	150 ft downstream from center line T&P Railroad, Pier 60, Sta 699+19	32.403	-52.6
4	150 ft downstream from center line T&P Railroad, Pier 117, Sta 717+00	32.710	-68.0
5	350 ft downstream from center line T&P Railroad, Sta 812+00	58.007	-84.0

Initial elevations established 10 and 11 July 1951.

NOTE: Elevations refer to mean sea level.

* Hub apparently damaged during construction.

** Sand in riser pipe.

*** This point under rail.

† Original elev of P.B. M2 was 32.261. After being hit by tractor, about April 1952, the elev became 32.243.

†† Interpolated from Red River Landing, La. and Bayou Sara, La. gages. Morganza Control Structure is located 60% the distance from Red River Landing gage to Bayou Sara gage.

Zero of Red River Landing gage = elev 3.49

Zero of Bayou Sara gage = elev 3.76

Fig. 39. Plan and diagram of settlement hubs, and settlement observations of control structure

was embedded in the beam supporting the upstream crane rail and in the downstream retaining wall for the railroad ballast. Readings of these settlement hubs are referenced to five deep permanent bench marks installed in the underlying sand foundation at locations shown on figs. 1 and 39. These permanent bench marks consist of 1-1/4-in. black iron pipe equipped with driving point driven into the deep underlying sand and encased with a 2-1/2-in. black iron pipe casing. The casing is protected for about 8 ft into the ground with a concrete shell. Both pipes are filled with a slurry of Baroid up to within 18 in. of the top, and the remaining 18 in. is filled with oil. Each pipe is capped with a brass plug. Concrete blocks about 15-in. cube were cast around the pipes located in the floodway; the blocks extend about 6 in. into the ground. Each installation is protected by pipe guard rails supported by 4 concrete posts 3 ft on centers.

111. Settlement observations made during construction and since completion of the structure are shown in tabular form on fig. 39. Plots of data obtained from selected settlement hubs are shown on figs. 40-41. Settlement of the entire structure as observed on the last date that precise levels were run is plotted on fig. 42. Some of the movement (approximately 0.10 in.) of the top of the structure (indicated by hubs 1 and 3 in fig. 39) is attributed to expansion and contraction of the concrete superstructure resulting from seasonal temperature changes since installation of the reference hubs.

112. Settlement of the top of the control structure, abutments, and approach piers from the time they were essentially complete to February 1954 is shown by the solid line on fig. 42. Settlements shown for the top of the structure do not include any settlement occurring during construction. Settlement of the base of piers 2 to 125 of the control structure is shown by the dashed line on fig. 42. The settlements shown for the bases of piers 2 to 125 include settlements occurring during construction of the piers and bridges and since their completion. Settlements occurring during construction include the elastic deformation of the piles caused by the dead load of the structure and possibly some vertical movement due to bowing in the piles as a result of their

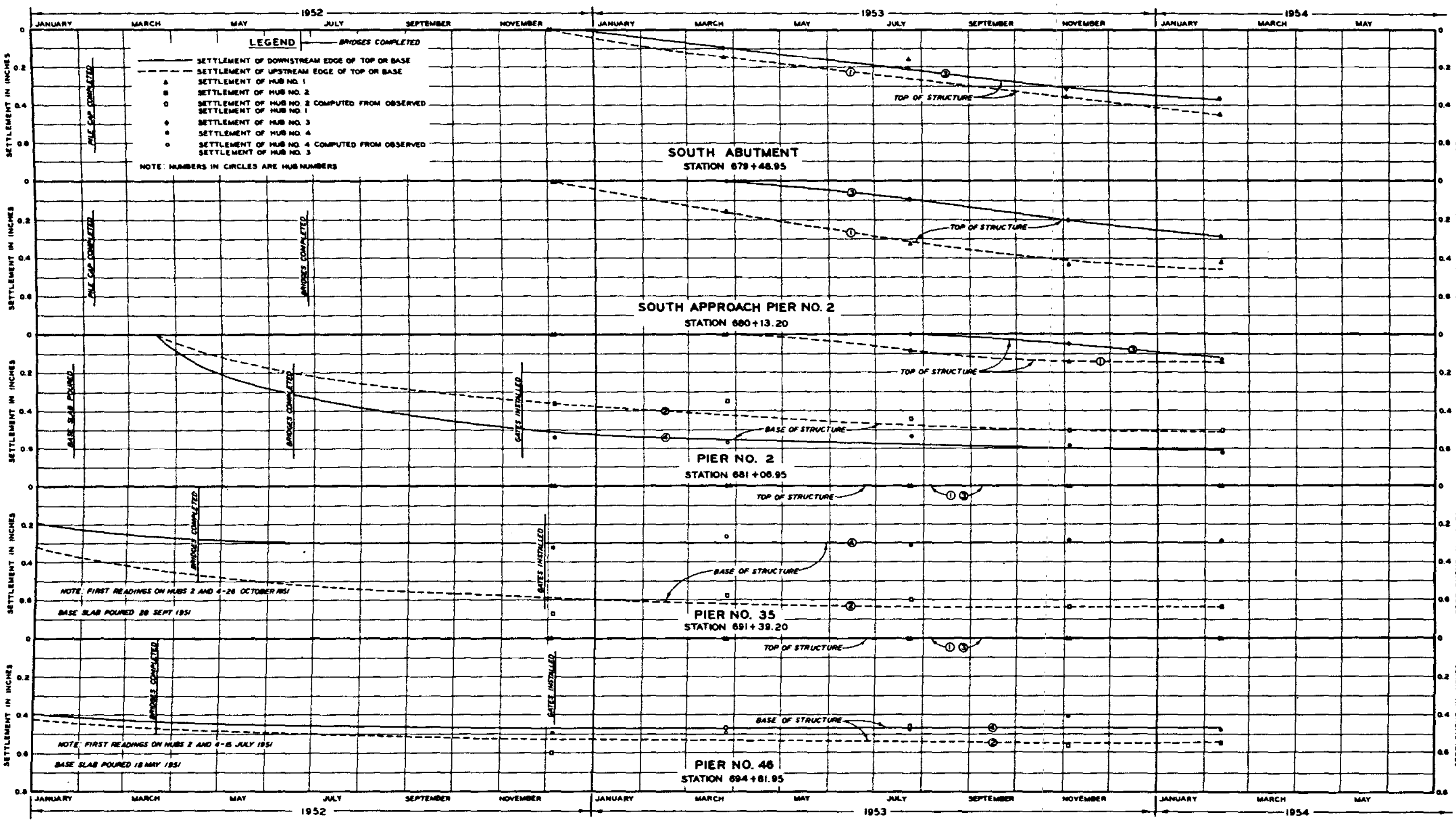


Fig. 40. Settlement of piers, south abutment to pier 46

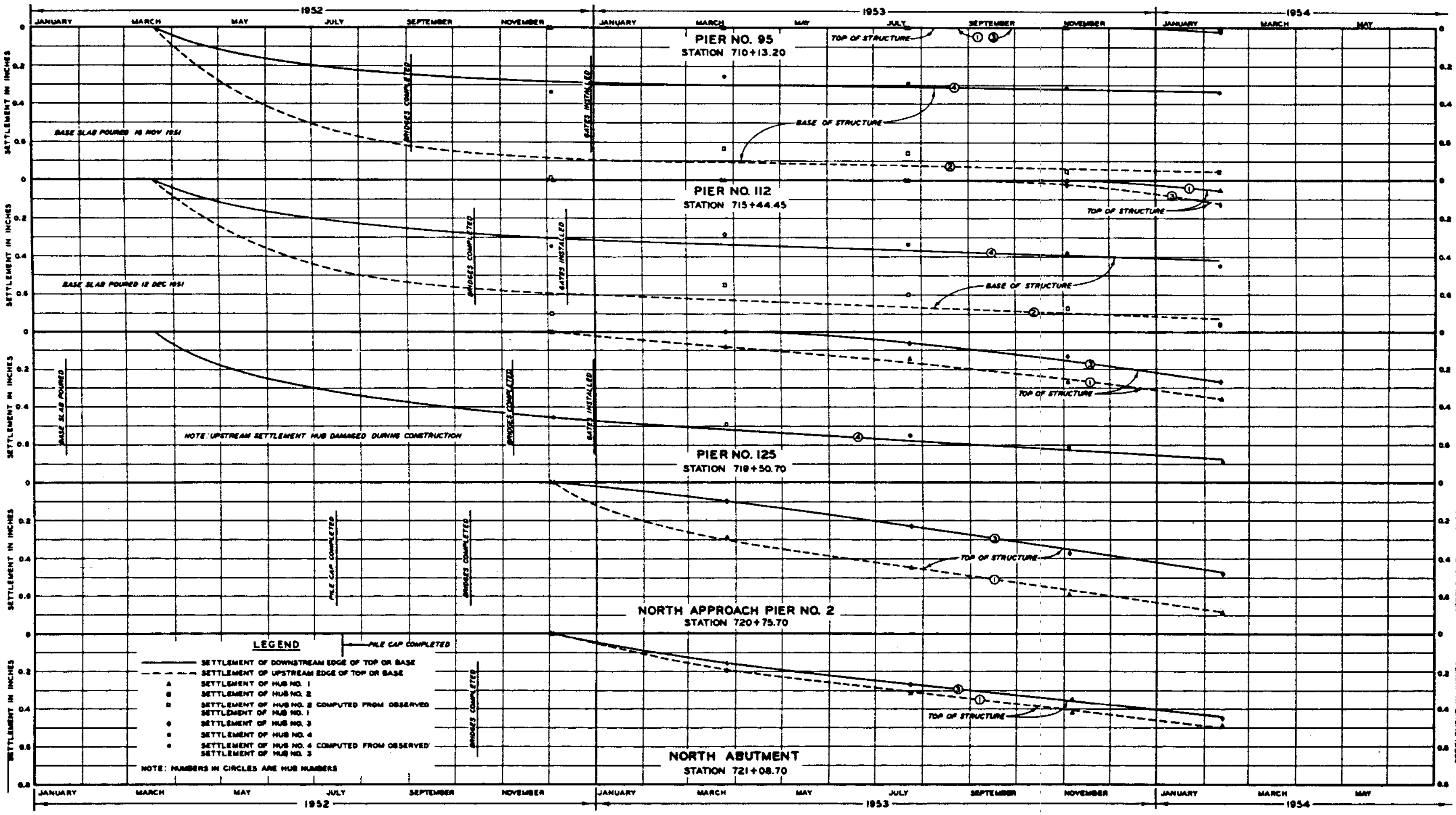
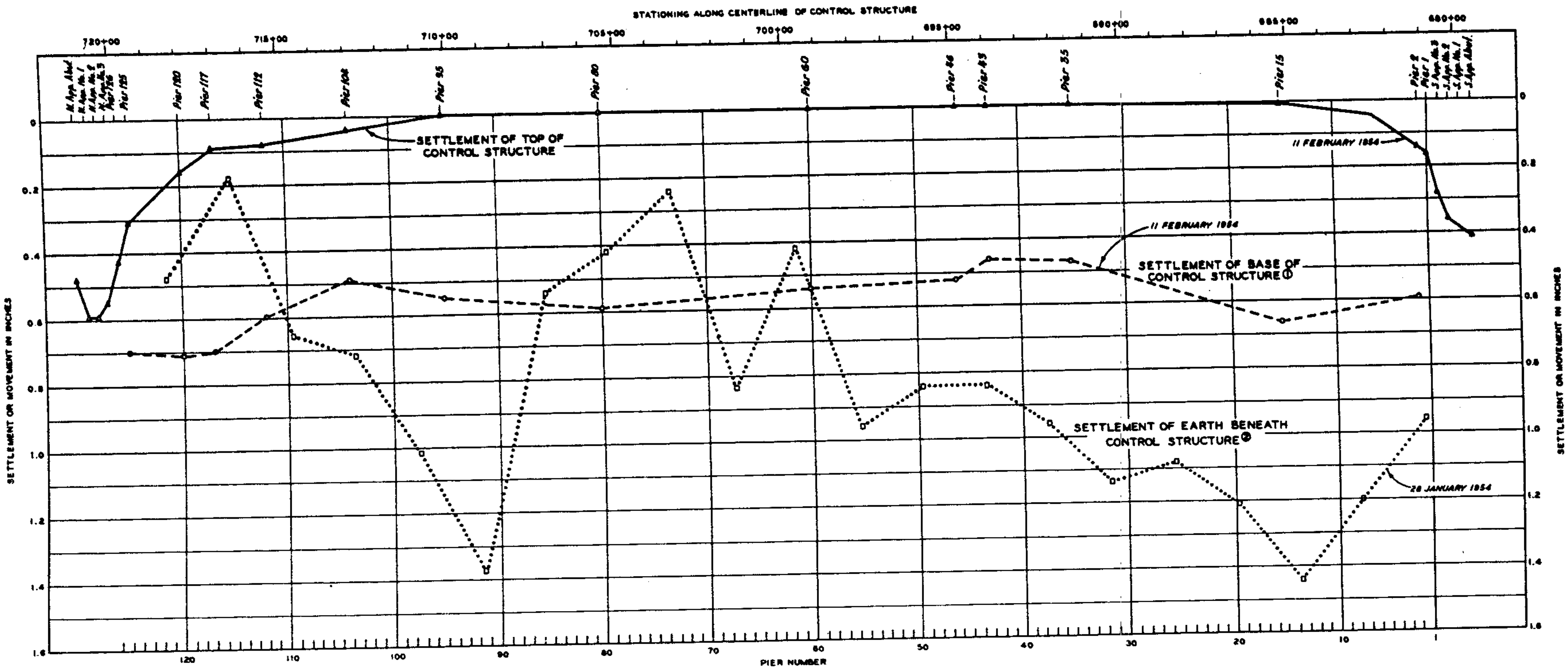


Fig. 41. Settlement of piers, pier 95 to north abutment



NOTES:

① SETTLEMENTS SHOWN FOR BASE OF STRUCTURE ARE AVERAGES OF HUBS 2 AND 4 IN BASE OF STRUCTURE. SETTLEMENTS SHOWN FOR TOP OF STRUCTURE ARE AVERAGES OF HUBS 1 AND 3 ON TOP OF STRUCTURE.

LOAD ON PILES BENEATH PIERS 1 AND 126 APPROXIMATE HALF THAT ON PILES BENEATH OTHER STRUCTURE PIERS.

② SETTLEMENT OF EARTH BENEATH CONTROL STRUCTURE IS AVERAGE OF TWO SETTLEMENT PLATES 6 FT APART, 4 INCHES BELOW THE BOTTOM OF THE BASE SLAB. THIS SETTLEMENT INCLUDES FOUNDATION SETTLEMENT CAUSED BY WEIGHT OF 4-FT BASE SLAB BEFORE CONCRETE IN MONOLITH SET, SETTLEMENT CAUSED BY WEIGHT OF BASE SLABS OF ADJACENT MONOLITHS SUBSEQUENTLY POURED, AND A CERTAIN PORTION OF STRUCTURE SETTLEMENT AFTER THE CONCRETE IN THE BASE SLAB HAS SET UP AROUND THE PILE HEADS.

Fig. 42. Settlement of control structure and earth beneath structure

batter. For an average dead load of 65 tons per pile and a pile length of 90 ft, the elastic and plastic deformation within the piles themselves has been estimated at 0.15 to 0.25 in.

113. Except at the ends of the control structure, settlement of the base of the structure has been very uniform, less than 0.1-in. difference between monoliths, and has amounted to approximately 0.5 in. It is believed that for the following reasons little movement of the pile tips has occurred as a result of their sinking into the sand or of consolidation of clay seams or strata underlying the pile tips: (1) the movement of most piers has stopped; (2) there has been practically no measurable movement of the top of the structure; (3) the anticipated elastic and plastic deformation of the piles could account for a considerable portion of the base settlement; and (4) the settlement of the base of the piers has been very uniform. Exceptions to this statement are the piers at the end of the control structure, the abutment piers, and approach piers.

114. From the data shown on figs. 39-42 it appears that practically no settlement of the top of the control structure has occurred, or is likely to occur, except for that portion of the structure south of pier 2 and north of pier 117. So far, settlements of the piers at the ends of the structure have been well within the originally estimated amount of movement and are discussed in more detail in the following paragraphs.

Estimated and observed settlements of <u>various sections of control structure</u>

115. <u>South abutment to pier 3.</u> Borings M-36 and M-40, in the vicinity of station 680, showed the existence within the sand of clay strata having a total thickness up to 3-1/2 ft overlain by 5 to 8 ft of sand (see fig. 43, page 97). Settlement computations indicated that a maximum settlement of 2.6 in. might occur at pier 1 or 2 if the pile tips should stop above the clay strata at elev -41; it was believed that the piles along this reach could be driven through the clay strata by overdriving. Consequently, all piles along this reach were overdriven except for those under the downstream wing wall more than 53 ft from the center line of the structure.

116. As may be seen from figs. 43 and 44, pages 97 and 104, it

was not possible to drive all of the piles to the estimated "top of bearing sand." However, some of the piles did penetrate through the existing clay strata.

117. As of February 1954, settlement of the piers at the south end of the structure ranged from approximately 0.1 in. at pier 2 to 0.4 in. for the south abutment (see fig. 42). The settlement of these piers can probably be attributed to consolidation of thin strata of clay beneath the pile tips as a result of stresses created primarily by the adjoining embankment. Some of the settlement of the abutment and approach piers can possibly be attributed to penetration of the piles as a result of overload created by the drag effect of the still-settling foundation clay. So far, the observed settlements of these piers are considerably less than the originally estimated maximum of approximately 2.5 in. However, the data in figs. 39 and 40 for these piers show that they are still settling at a discernible rate. Whether or not the settlement of these piers will exceed the originally estimated maximum of 2.5 in. cannot be predicted at this time.

118. Piers 4 to 41. Since no compressible strata within the sand were disclosed by the borings along this section, no settlement of the structure was anticipated between piers 4 to 41; therefore no overdriving of the piles for these piers was attempted. No measurable settlement of the top of the structure along this reach has occurred, and settlement of the base of these piers has apparently stopped (see figs. 39 and 40).

119. Piers 42 to 54. Borings M-53, -54, and -56 indicated a compressible stratum of clay and lignitic sand within the sand. A settlement of 1.5 in. was computed at piers 46 and 47 if the tips of the piles were not driven through the clay. Accordingly, the piles beneath these piers were overdriven, and most of the piles were driven into the "bearing sand." However, some of the piles may not have been driven through the soft stratum (see fig. 46, page 106).

120. Figs. 39, 40, and 42 show that there has been no measurable settlement of the top of the structure for piers 42 to 54, and settlement of the base of the structure which occurred during construction apparently stopped before the structure was completed.

121. Piers 55 to 110. The borings did not indicate any compressible strata or strata with low driving resistance within the sand along piers 55 to 110. Accordingly, no settlement of these piers was anticipated and no overdriving of the piles was specified.

122. As shown on fig. 42, no measurable settlement of the top of the control structure has been noted between piers 55 and 110. Settlement of the base of the structure along this reach of piers is typical of that observed for the entire structure. Furthermore, as shown in figs. 39 and 41, the settlement of the base of the structure had ceased by the time the superstructure was completed.

123. Piers 111 to 117. Borings M-74, -75, -105, and -106 indicated the existence of clay strata, and some lignite and sand with low driving resistance, within the upper part of the sand stratum along piers 111 to 117 (see fig. 43). A maximum settlement of 4.3 in. was estimated at pier 112 if the tips of the piles were stopped above the clay. It was thought possible to penetrate these strata by overdriving. Apparently, the extra hard driving did cause some of the piles to penetrate these strata, whereas others did not (see fig. 49, page 109).

124. The base of the structure between piers 111 to 117 has settled slightly more (approximately 0.65 in.) than the base slab for the remainder of the control structure. Also, some slight settlement of the top of the structure, amounting to approximately 0.1 in. has taken place since completion of the superstructure.

125. Piers 118 to north abutment. Borings along piers 118 to north abutment showed the existence of soft clay strata up to 4.5 ft thick in the main sand stratum, overlain by sand 12 to 20 ft thick. Penetration of this clay stratum by overdriving was not attempted because of the thickness of the overlying sand. Settlements of 1.4 in. were estimated with the tips of the piles at elev -64.

126. Observations made on top of piers 118 through the north abutment show that settlement of as much as 0.6 in. has occurred (see figs. 39 and 42).

127. Time plots of the settlement of north approach pier 2 and the north abutment show that settlement of these piers is still occurring at

the rate of approximately 0.03 in. per month. The settlement of these piers can largely be attributed to consolidation of the clay strata beneath the tips of the piles caused primarily by stresses created by the adjacent embankment and to a lesser extent by drag on the piles. Whether or not the settlement of the north approach pier will exceed the originally estimated 1.4 in. cannot be determined at this time.

Settlement of foundation upstream of structure

128. During design of the pile foundation for the structure, the question was raised as to whether or not the water load on the ground surface might cause settlement of the foundation immediately upstream of the weir and a vertical deflection of the batter piles extending upstream of the weir. Computations indicated that as much as 6 in. of settlement might possibly result from a sustained water load equal to the project flood. However, if settlement did occur, it was thought that it would probably take place over a very long period of time, and that the amount of bending in the piles would not be detrimental. To afford a check on this, settlement plates were installed beneath the riprap. These settlement plates were installed after the gravel blanket beneath the riprap had been placed but prior to the placing of the riprap. The riprap was placed after the base slab and weir for the control structure were constructed.

129. Fig. 39 shows the initial elevation of these settlement plates, determined immediately after they were set in February 1953, as well as elevations determined since their installation. The following total settlements of the plates have been observed on the dates shown.

Settlement Plate		Settlement in inches			
Number	Station	6 March '53	10 April '53*	30 April '53*	11 Feb '54
A	690	-1.6	-0.7	-0.6	-1.8
B	700	-0.5	0	+0.2	-0.4
C	710	-0.5	0	+0.2	-0.4
Average	---	-0.9	-0.2	-0.1	-0.9

* Settlement data shown for April 1953 are considered questionable.

130. In view of the very small increase of net load on the

foundation created by the weight of an 8-in. gravel blanket and approximately 18 in. of riprap, little or no settlement of the foundation beneath the riprap would be expected unless there was rebound of the foundation while the excavation for the structure and riprap was open. If full rebound is assumed to have occurred and the recompression curve on loading is parallel to the laboratory recompression curves (no special rebound and recompression curves were obtained for any of the foundation clays along the control structure), then 1.5 to 2.0 in. of total settlement might be anticipated as a result of placing the gravel blanket and riprap. If 50 per cent of the total settlement is assumed to have occurred to date, a settlement of 1.0 in. might be expected. The observed average settlement of the settlement plates to date has been 0.9 in.

131. Although the water load on the foundation upstream of the weir has been only 2.5 ft to date, it may have contributed slightly to the settlement of the plates. No conclusions can yet be drawn regarding the effect of water loads on foundation settlement upstream of the structure.

Top of Sand, Pile Penetration, and Pile Length

Estimated top of sand, pile penetration, and pile length

132. Top of sand. In order that the required length of piles for each of the piers might be determined, the top of "bearing sand" was first estimated from a comparison of split-spoon boring data and pile-driving resistance data obtained where test piles had been driven. From this analysis it was decided that the top of bearing sand was at that elevation corresponding to driving resistances of 35 to 40 blows per ft on the split-spoon samples, depending upon the hydrostatic pressure in the sand foundation, and that this was equivalent to about 75 blows per ft on a 20-in. concrete pile 80 or 90 ft long with a 30,000-ft-lb hammer, or 66 blows per ft for a 32,500-ft-lb hammer, the hammer used for driving the piles beneath the control structure. The average top of sand as originally estimated from the split-spoon borings is shown by the long-dashed line in fig. 43 and is listed for each pier in table 15. A

Table 15

Summary of Pile Driving Data

Pier	Average Top of Sand mGl Est.	Average Top of Sand mGl Obs.	Avg Pile Length ft	Avg Pile Length in Clay ft	Avg Length of Cutoff ft	Penetration into Sand ft Avg Est.	Penetration into Sand ft Avg Obs.	Penetration into Sand ft Max.	Max. Variation of Top of Sand from Average, ft Est.	Max. Variation of Top of Sand from Average, ft Observed -	Max. Variation of Top of Sand from Average, ft Observed +	Allowance for Penetration in Sand and Variability in Top of Sand, ft Est.	Allowance for Penetration in Sand and Variability in Top of Sand, ft Opt. Obs.
1	--	-36.5	73.6	69.2	16.4	---	4.4	15.0	---	2.6	2.5	---	--
2	--	-38.5	75.3	71.5	14.7	---	3.8	17.5	---	7.7	7.0	---	--
3	--	-36.9	75.2	69.7	14.8	---	5.5	21.4	---	3.4	2.5	---	--
4	-38	-38.0	72.5	71.0	4.5	5.6	1.5	3.7	4.0	2.7	2.5	10.0	3.5
5	-38	-38.1	72.2	71.1	4.8	5.6	1.1	2.5	4.0	0.9	2.3	10.0	3.5
6	-40	-39.4	74.2	71.8	4.9	5.6	2.4	6.0	4.0	1.2	2.0	10.0	5.8
7	-41	-37.9	73.6	70.9	5.8	5.6	2.7	5.0	4.0	2.5	3.0	10.0	5.0
8	-42*	-37.9	74.5	70.8	7.3	5.6	3.7	8.1	4.0	2.6	4.9	10.0	5.7
9	-41*	-37.3	74.4	70.2	8.6	5.6	4.2	5.5	4.0	1.3	1.7	10.0	6.9
10	-41*	-38.4	75.3	71.4	7.7	5.6	3.9	5.2	4.0	2.1	2.8	10.0	6.4
11	-41*	-39.2	76.4	72.3	6.6	5.6	4.1	8.8	4.0	3.8	3.0	10.0	8.4
12	-41*	-40.6	77.8	73.9	5.2	5.6	3.9	8.4	4.0	3.8	3.9	10.0	7.9
13	-41*	-42.3	80.0	75.8	3.0	5.6	4.2	7.1	4.0	3.5	4.3	10.0	7.2
14	-41*	-37.3	75.4	70.2	7.6	5.6	5.2	14.6	4.0	6.2	7.7	10.0	11.7
15	-43	-39.6	76.0	72.7	7.0	5.6	3.3	12.5	4.0	4.5	5.3	10.0	10.9
16	-45	-38.3	76.4	71.3	8.6	5.6	5.1	15.3	4.0	6.5	6.4	10.0	10.5
17	-46	-39.6	76.5	73.0	9.5	5.6	3.5	13.2	4.0	5.0	5.4	10.0	7.2
18	-48	-38.6	76.8	71.6	11.2	5.6	5.2	13.2	4.0	5.8	4.7	10.0	8.5
19	-49	-40.1	78.4	73.3	10.6	5.6	5.1	11.9	4.0	5.0	3.8	10.0	9.5
20	-49*	-42.1	79.7	75.5	11.3	5.6	4.2	6.1	4.0	4.9	4.5	10.0	7.9
21	-49*	-43.3	81.3	76.9	9.7	5.6	4.4	8.0	4.0	4.9	4.5	10.0	8.1
22	-49*	-44.0	81.8	77.6	9.2	5.6	4.2	7.5	4.0	2.5	4.2	10.0	6.8
23	-48*	-44.9	82.8	78.7	8.2	5.6	4.1	5.2	4.0	3.8	2.9	10.0	7.3
24	-47*	-46.0	83.5	79.9	7.5	5.6	3.6	5.0	4.0	4.2	4.4	10.0	8.4
25	-48*	-47.3	84.7	81.3	6.3	5.6	3.4	5.0	4.0	4.8	5.0	10.0	8.3
26	-48*	-48.2	85.8	82.3	5.3	5.6	3.5	6.7	4.0	4.5	6.7	10.0	8.7
27	-48*	-48.2	86.1	82.4	4.9	5.6	3.7	5.4	4.0	2.6	4.1	10.0	7.4
28	-50	-49.1	87.6	83.4	3.4	5.6	4.2	6.0	4.0	3.6	2.3	10.0	7.6
29	-53	-49.2	88.1	83.5	5.9	5.6	4.6	7.9	4.0	2.6	3.9	10.0	9.3
30	-55*	-50.2	89.3	84.6	7.7	5.6	4.7	7.6	4.0	2.3	2.1	10.0	7.2
31	-55*	-49.4	88.6	83.7	8.4	5.6	4.9	7.8	4.0	2.3	2.1	10.0	7.3
32	-55*	-49.7	88.8	84.0	8.4	5.6	4.8	7.6	4.0	3.3	2.9	10.0	8.3
33	-55*	-50.8	89.9	85.3	7.1	5.6	4.6	5.8	4.0	1.4	1.3	10.0	6.5
34	-55*	-51.1	89.8	85.6	7.2	5.6	4.2	7.7	4.0	1.3	1.3	10.0	6.8
35	-55*	-50.9	90.0	85.4	7.0	5.6	4.6	7.2	4.0	8.1	1.7	10.0	6.8
36	-50*	-50.2	87.4	84.6	8.6	5.6	2.8	5.3	4.0	2.1	1.3	10.0	6.5
37	-49*	-51.4	90.3	85.9	5.7	5.6	4.4	7.4	4.0	1.3	2.8	10.0	6.3
38	-50*	-50.2	89.4	84.6	6.6	5.6	4.8	7.4	4.0	1.9	1.3	10.0	7.7
39	-52*	-48.7	89.0	83.0	7.0	5.6	6.0	10.5	4.0	4.2	3.0	10.0	8.4
40	-54*	-47.5	87.8	81.5	8.2	5.6	6.3	10.8	4.0	4.2	3.7	10.0	9.8
41	-55	-49.2	90.4	83.5	9.6	5.6	6.9	17.0	4.0	4.0	6.1	10.0	10.5
42	--	-49.4	91.2	83.7	8.8	---	7.5	16.4	---	3.6	2.8	--	--
43	--	-49.4	92.1	83.7	8.1	---	8.4	24.2	---	3.2	4.1	--	--
44	--	-47.2	88.3	81.2	11.7	---	7.1	14.7	---	5.0	3.9	--	--
45	--	-48.1	91.0	82.3	9.2	---	8.7	24.9	---	4.3	8.3	--	--
46	--	-46.6	89.1	80.5	10.9	---	8.6	20.7	---	3.9	11.6	--	--
47	--	-47.2	89.9	81.2	10.1	---	8.7	21.7	---	4.0	5.1	--	--
48	--	-46.3	87.6	80.3	12.4	---	7.3	13.0	---	2.4	4.8	--	--
49	--	-45.9	88.3	79.8	10.6	---	8.5	23.7	---	3.0	5.5	--	--
50	--	-44.0	89.1	77.6	10.0	---	11.5	25.0	---	2.2	1.9	--	--
51	--	-42.5	89.6	76.0	9.6	---	13.6	30.8	---	2.3	2.8	--	--
52	--	-42.1	88.1	75.5	11.9	---	12.6	18.1	---	1.2	1.2	--	--
53	--	-42.1	87.8	75.6	12.2	---	12.2	20.3	---	1.8	8.2	--	--
54	--	-42.6	89.2	76.1	10.8	---	13.1	21.8	---	1.9	3.2	--	--
55	-52*	-46.1	87.4	80.0	14.4	5.6	7.4	16.6	4.0	3.6	2.2	10.0	10.0
56	-51*	-48.9	88.1	83.1	11.9	5.6	5.0	16.8	4.0	3.6	2.2	10.0	7.0
57	-51*	-51.1	92.8	85.6	6.2	5.6	7.2	14.6	4.0	7.0	3.0	10.0	13.4
58	-50*	-54.0	95.1	88.9	1.7	5.6	6.2	12.3	4.0	3.3	2.4	10.0	9.1
59	-50	-54.6	93.8	89.5	1.3	5.6	4.3	8.4	4.0	2.6	7.0	10.0	6.0
60	-50	-52.4	90.9	87.0	4.1	5.6	3.9	6.4	4.0	3.1	4.0	10.0	6.7
61	-50	-52.2	91.3	86.9	3.7	5.6	4.4	5.8	4.0	3.8	3.8	10.0	7.2
62	-50	-49.7	87.9	84.0	7.1	5.6	3.9	5.8	4.0	6.3	4.2	10.0	8.7
63	-50	-49.0	87.4	83.2	7.6	5.6	4.2	10.0	4.0	3.0	1.9	10.0	7.2
64	-50*	-46.9	86.0	80.9	10.0	5.6	5.1	13.3	4.0	1.8	2.6	10.0	7.5
65	-50*	-46.7	85.2	80.6	11.8	5.6	4.6	9.0	4.0	3.1	7.9	10.0	6.7

(Continued)

* Revised estimated top of sand as of 1 February 1951.

Table 15 (Cont'd)

Pier	Average Top of Sand mGl Est.	Average Top of Sand mGl Obs.	Avg Pile Length ft	Avg Pile Length in Clay ft	Avg Length of Cutoff ft	Penetration into Sand ft Avg Est.	Penetration into Sand ft Avg Obs.	Penetration into Sand ft Max.	Max. Variation of Top of Sand from Average, ft Est.	Max. Variation of Top of Sand from Average, ft Observed -	Max. Variation of Top of Sand from Average, ft Observed +	Allowance for Penetration in Sand and Variability in Top of Sand, ft Est.	Allowance for Penetration in Sand and Variability in Top of Sand, ft Opt. Obs.
66	-50*	-50.1	88.0	84.5	10.0	5.6	3.5	5.0	4.0	5.2	4.1	10.0	7.6
67	-50*	-53.9	90.9	88.6	7.1	5.6	2.3	4.2	4.0	5.6	2.6	10.0	3.8
68	-50*	-51.5	89.0	86.1	8.0	5.6	2.9	5.9	4.0	4.9	4.3	10.0	6.2
69	-50	-51.1	89.1	85.7	5.9	5.6	3.4	7.2	4.0	3.3	3.6	10.0	5.2
70	-50	-54.1	90.2	88.9	4.8	5.6	1.3	1.5	4.0	0.5	0.9	10.0	2.5
71	-50	-52.4	88.7	87.1	6.3	5.6	1.6	2.0	4.0	1.6	1.7	10.0	3.8
72	-50	-48.8	85.5	83.0	9.5	5.6	2.5	4.5	4.0	0.9	1.9	10.0	4.6
73	-49*	-49.0	84.9	83.1	11.1	5.6	1.8	3.0	4.0	0.8	1.2	10.0	3.6
74	-48*	-47.9	85.1	82.0	11.9	5.6	3.1	5.9	4.0	3.0	1.9	10.0	4.8
75	-47*	-49.0	85.2	83.2	11.8	5.6	2.0	3.9	4.0	1.7	1.3	10.0	3.9
76	-48*	-46.9	83.0	80.8	14.0	5.6	2.2	4.5	4.0	1.2	1.5	10.0	4.7
77	-49*	-47.0	83.9	81.1	13.1	5.6	2.8	3.0	4.0	1.3	0.9	10.0	4.6
78	-50*	-46.8	85.6	80.8	11.4	5.6	4.8	5.0	4.0	1.1	0.8	10.0	5.9
79	-50*	-49.1	86.6	83.4	10.4	5.6	3.2	5.6	4.0	3.0	2.3	10.0	4.5
80	-50*	-50.0	86.6	84.4	10.4	5.6	2.2	3.2	4.0	1.0	0.6	10.0	3.2
81	-50*	-49.9	86.0	84.3	11.0	5.6	1.7	2.3	4.0	0.9	0.5	10.0	2.9
82	-50*	-49.7	85.6	84.0	11.3	5.6	1.6	2.2	4.0	2.2	1.2	10.0	2.6
83	-50*	-49.5	85.3	83.8	10.5	5.6	1.5	2.2	4.0	1.3	1.4	10.0	3.5
84	-50*	-49.5	85.4	83.8	10.6	5.6	1.6	1.9	4.0	1.7	0.9	10.0	2.8
85	-50*	-48.5	84.1	82.7	11.9	5.6	1.4	2.6	4.0	1.7	1.3	10.0	2.9
86	-49	-46.8	82.8	80.8	10.2	5.6	2.0	3.0	4.0	1.4	2.2	10.0	3.8
87	-47	-45.2	80.9	79.0	10.1	5.6	1.9	3.7	4.0	1.3	2.3	10.0	4.7
88	-45	-44.1	79.3	77.7	9.8	5.6	1.6	5.1	4.0	1.4	1.1	10.0	3.8
89	-46	-44.0	79.4	77.7	10.6	5.6	1.7	2.9	4.0	0.5	0.6	10.0	3.2
90	-46	-43.9	78.7	77.5	7.3	5.6	1.2	2.0	4.0	0.4	0.5	10.0	2.3
91	-47	-43.9	79.0	77.5	8.0	5.6	1.5	2.4	4.0	0.4	0.6	10.0	2.7
92	-48	-44.3	79.6	78.0	8.4	5.6	1.6	4.2	4.0	0.6	2.4	10.0	3.9
93	-48	-44.5	79.9	78.3	8.1	5.6	1.6	3.7	4.0	1.3	0.6	10.0	3.7
94	-49	-45.6	81.5	79.4	7.5	5.6	2.1	3.8	4.0	0.6	0.7	10.0	3.0
95	-50	-48.2	84.4	82.4	6.6	5.6	2.0	3.7	4.0	2.9	1.4	10.0	3.6
96	-51	-48.9	85.1	83.1	6.9	5.6	2.0	5.0	4.0	1.3	2.2	10.0	4.8
97	-51	-49.0	85.0	83.2	7.0	5.6	1.8	2.6	4.0	1.0	1.2	10.0	2.6
98	-52	-49.5	85.8	83.9	9.2	5.6	1.9	5.0	6.0	0.7	0.7	12.3	3.3
99	-56	-54.6	94.2	89.5	5.1	5.6	4.7	25.0	6.0	5.1	13.6	12.3	1.5
100	-60	-59.5	97.5	95.0	6.4	5.6	2.5	12.6	6.0	3.4	6.9	12.3	2.2
101	-63	-60.9	99.6	96.5	7.4	5.6	3.1	6.1	6.0	1.3	1.5	12.3	6.8
102	-63	-61.5	99.2	97.3	7.7	5.6	1.9	4.1	6.0	3.5	4.7	12.3	7.2
103	-62	-56.5	94.7	91.6	11.3	5.6	3.1	12.0	6.0	7.7	3.3	12.3	5.8
104	-65	-56.1	96.1	91.2	13.6	5.6	4.9	15.7	6.0	9.6	9.7	12.3	11.9
105	-60*	-51.6	90.4	86.2	16.2	5.6	4.2	10.9	6.0	6.1	1.0	12.3	12.4
106	-56*	-47.6	85.5	81.7	13.5	5.6	3.8	6.9	6.0	2.3	9.1	12.3	6.5
107	-52*	-46.6	84.3	80.6	12.7	5.6	3.7	10.4	6.0	2.3	2.9	12.3	7.1
108	-52*	-47.5	84.4	81.6	12.6	5.6	2.8	5.3	6.0	1.6	4.2	12.3	5.2
109	-53*	-48.8	86.7	83.0	10.3	5.6	3.7	5.7	6.0	3.1	5.9	12.3	4.0
110	-53*	-52.9	91.4	87.6	8.9	5.6	3.8	5.0	6.0	6.1	7.8	12.3	13.7
111	--	-52.9	94.1	87.6	15.6	---	6.5	15.9	---	6.3	9.9	--	--
112	--	-52.5	91.7	87.1	18.3	---	4.6	11.3	---	6.0	5.4	--	--
113	--	-53.3	92.7	88.1	17.3	---	4.6	10.8	---	7.8	4.2	--	--
114	--	-55.9	96.1	90.9	13.9	---	5.2	13.7	---	8.2	3.9	--	--
115	--	-55.5	98.2	90.5	11.8	---	7.7	17.5	---	6.9	6.4	--	--
116	--	-53.8	93.3	88.6	16.7	---	4.7	17.7	---	5.6	8.9	--	--
117	--	-50.2	90.0	84.6	20.0	---	5.4	16.6	---	10.1	10.2	--	--
118	--	-48.2	86.6	82.3	23.4	---	4.3	9.5	---	4.7	3.4	--	--
119	--	-52.5	90.3	87.2	6.7	5.6	3.1	5.3	4.0	5.1	4.2	10.0	6.6
120	--	-51.4	89.1	85.9	10.3	5.6	3.2	8.7	4.0	2.4	4.7	10.0	6.1
121	-56*	-50.4	87.9	84.8	11.6	5.6	3.1	10.1	4.0	2.2	6.0	10.0	7.9
122	-53*	-49.0	85.5	83.2	9.5	5.6	2.3	5.0	4.0	1.6	1.8	10.0	5.7
123	-50	-48.1	84.5	82.2	6.5	5.6	2.3	5.0	4.0	2.4	1.3	10.0	3.9
124	-51	-49.6	87.8	83.9	4.2	5.6	3.9	7.0	4.0	1.4	2.6	10.0	6.5
125	-51	-51.4	88.2	85.9	3.8	5.6	2.3	5.0	4.0	2.5	1.3	10.0	4.7
126	-52	-51.6	88.0	86.1	5.0	5.6	1.9	3.5	4.0	1.3	1.1	10.0	3.2

* Revised estimated top of sand as of 1 February 1951.

summary of the maximum, minimum, and average top of sand, penetration of piles into sand, variation in top of sand, and optimum allowance for penetration into and variation in top of sand is given in table 16, page 99.

133. Variation in top of sand. A study of the logs of borings shown in fig. 43 and of other borings made adjacent to the center line of the structure showed considerable and inconsistent variation in the top of the bearing sand. Variations of ±4 ft (±4.4 ft on batter) from the average for any pier were noted, except for piers 98-110 where a variation of ±6 ft (±6.7 ft on batter) was found (see table 15). It was decided to cast all the piles at any one pier as long as the greatest estimated required length for that pier since cost studies showed this procedure to be less expensive than that of casting the piles shorter and then splicing some of them.

134. Penetration into sand. Analysis of the driving records of the test piles indicated that the piles penetrated on the average 4.3 ft into bearing sand. A value of 5 ft (5.6 ft on batter) was selected as the average penetration into sand for design purposes. This penetration was required unless the driving resistance exceeded 325 blows per ft (with a 32,500-ft-lb hammer) before a 5-ft penetration was obtained.

135. Pile lengths. The lowest estimated pile tip elevation, except in the few reaches where clay was encountered below the top of sand, was determined by subtracting 5 ft from the estimated average top of sand for penetration of the piles into sand and then subtracting an additional 4 or 6 ft to allow for variation in the top of the sand. The design elevation of the pile tips thus obtained is shown by the short-dashed line in fig. 43. The length of pile to be cast for any one pier was determined by computing the distance on a 2-on-1 batter from the cutoff elevation to the design tip elevation and adding 2.5 ft for reinforcement extension. After some of the first piles were driven an analysis of the driving data indicated that the actual top of sand was somewhat higher than originally predicted; therefore in January 1951 the estimated average top of sand and design elevation of the pile tips were raised some as shown by the dash-dot lines in fig. 43.

136. Where the piles were to be overdriven in an attempt to

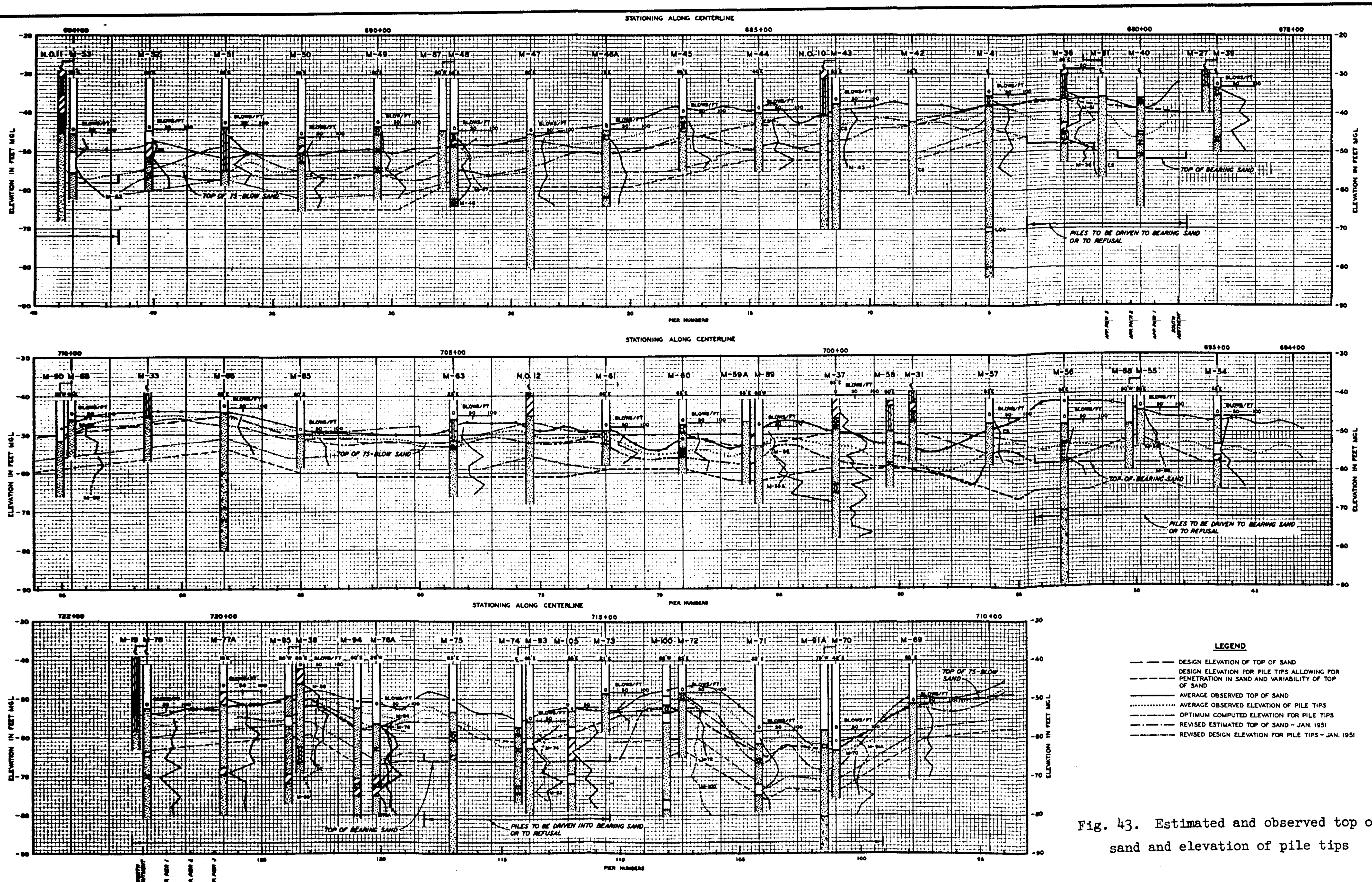


Fig. 43. Estimated and observed top of sand and elevation of pile tips

Table 16

Summary of Pile Lengths, Penetration into Sand, and Variation in Top of Sand

Pile lengths	
Average pile length	86.1 ft
Maximum pile length	110.0 ft
Top of sand	
Average elevation of the average top of sand at all piers	-47.7 ft
Highest elevation of average top of sand at any pier	-36.5 ft
Highest elevation of top of sand at any pile	-31.1 ft
Lowest elevation of average top of sand at any pier	-61.5 ft
Lowest elevation of top of sand at any pile	-68.1 ft
Maximum difference in elevation of top of sand at the same pier (117)	20.3 ft
Maximum difference in elevation of average top of sand between any piers	25.0 ft
Maximum difference in elevation of top of sand between any piles	37.0 ft
Pile penetration into sand	
Average penetration of piles into sand:	
Nonoverdrive piers	3.4 ft
Overdrive piers	7.7 ft
Maximum penetration of any pile into sand:	
Nonoverdrive piers	25.0 ft
Overdrive piers	30.8 ft
Variation in top of sand	
Average "minus" variation in top of sand:	
Nonoverdrive piers	-3.2 ft
Overdrive piers	-5.3 ft
Average "plus" variation in top of sand:	
Nonoverdrive piers	+3.1 ft
Overdrive piers	+5.3 ft
Maximum "minus" variation in top of sand at any pier:	
Nonoverdrive piers	-13.6 ft
Overdrive piers	-11.6 ft
Maximum "plus" variation in top of sand at any pier:	
Nonoverdrive piers	+13.6 ft
Overdrive piers	+11.6 ft
Optimum allowance for penetration into sand and variation in top of sand	
Average computed optimum allowance for penetration into sand and variation in top of sand for nonoverdrive piers	6.3 ft

penetrate through clay strata below the top of the sand (see fig. 43), the length of piles was computed on the assumption that they could be driven through the clay strata within the sand and would penetrate 5 ft vertically into the underlying sand. An addition of 2 ft was also made for variation in the top of the bearing sand.

137. The piles were driven to refusal along these reaches if the tips of the piles were above compressible strata. Driving resistances considered refusal for 20-in. concrete piles were approximately 500 blows per ft for octagonal piles and 550 blows for square piles. A review of available information on hard driving of piles indicated that the piles could sustain this amount of driving without damage.

Comparison of estimated and observed top of sand, pile penetration, and pile length

138. A numerical comparison of estimated and observed values of top of sand, variation in top of sand, penetration into sand, and allowance for variation and penetration into sand for groups of piers where piles were not overdriven is given in table 17. The average lengths of piles in these groups are also shown in this table.

139. Top of sand. The estimated and observed average elevations of the top of sand at each pier are listed in table 15 and are plotted in fig. 43. The observed average top of sand as indicated by a driving rate of 66 blows per ft varies from 4.6 ft below the estimated top of sand to 8.9 ft above, with an average of 2.1 ft above (see table 15 and item 2, table 17). Inspection of fig. 43 and table 18 discloses that the observed average top of sand has a closer correlation to the top of sand indicated by classification of samples from the borings than by the split-spoon driving resistances. It is possible that the small split-spoon sampler was more sensitive to local variations in the silty and fine sand at the top of the over-all sand stratum than were the 20-in. piles. However, in view of the large variations in the top of the sand discussed in the next paragraph, it is felt that the predicted average elevations agreed in a satisfactory manner with the observed average sand elevations.

Table 17

Comparison of Estimated and Observed Values of Top of Sand, Variation in Top of Sand, Penetration into Sand, Allowance for Variation Plus Penetration into Sand, and Length of Piles for Groups of Piers where Piles Were Not Overdriven

	Piers 4-41	Piers 55-97	Piers 98-110	Piers 119-126	All non-overdrive
Top of Sand					
Difference in observed and original estimate in ft (vertical)	4.6	2.1	4.8	2.8	3.4
Difference in observed and revised estimate in ft (vertical)	3.2	0.6	4.0	2.2	2.1
Design allowance for variation in top of sand, ft (vertical)	±4	±4	±6	±4	---
Observed variation, ft (vertical)	+3.6 -3.7	+2.3 -2.2	+4.1 -5.5	+2.4 -2.9	+3.0 -3.2
Penetration into Sand					
Observed, ft (slant)	4.2	2.9	3.4	2.8	3.4
Allowance for Variation Plus Penetration into Sand					
Design allowance, ft (slant)	10.0	10.0	12.3	10.0	---
Optimum allowance, ft (slant)	7.6	5.0	6.7	5.6	6.3
Pile Lengths					
Original design, ft (slant)	93.2	95.2	105.9	98.7	96.1
Length driven, ft (slant)	82.2	85.8	91.5	87.7	85.3
Revised design length, ft (slant)	89.8	94.4	102.1	95.8	93.8
Optimum length, ft (slant)	85.7	87.2	94.9	90.5	87.9

Note: Values shown are based on averages for piers indicated. Design penetration into sand 5 ft on vertical, 5.6 ft on slant.

Table 18

Difference in Top of Sand As Indicated by Sample Classification, Split-spoon Driving Data, and Pile Driving Data

Average for Pier Group	Sample Classification vs Split-spoon Driving Data	Pile Driving Data vs Sample Classification	Pile Driving Data vs Split-spoon Driving Data
South abut. - 3	+1.0	-1.1	-0.2
4 - 41	+4.5	-0.8	+3.3
42 - 54	+9.3	-0.4	+9.9
55 - 97	+2.7	-1.5	+1.9
98 - 110	+3.2	+1.2	+4.7
111 - 118	+4.5	+1.4	+5.9
119 - north abut.	+2.2	+1.3	+3.5
All piers	+3.9	0.0	+4.2

Note: + Top method showed top of sand higher than indicated by bottom method.
- Top method showed top of sand lower than indicated by bottom method.

140. Variation in top of sand. In the original analysis it was estimated that variation in the top of sand might be ± 4 ft with the exception of piers 98 through 110 where a value of ± 6 ft was indicated. The observed maximum variations of the top of sand from the average top of sand at each pier are listed in table 15. The "minus" values indicate elevations lower than the average top of sand elevation at that pier and the "plus" values indicate higher elevations. The average variation for the nonoverdrive reaches except for piers 98 through 110 was -2.9 and +2.8 ft. The average variation for piers 98 through 110 was -5.5 and +4.9 ft. These figures check closely the estimated values of ± 4 ft and ± 6 ft, respectively. The average variation for the overdrive reaches was -5.3 and +5.3 ft.

141. The greatest total variation in the elevation of the top of the sand at any one pier was 20.3 ft at pier 117 between piles 14 and 26 which are only about 8 ft apart. Total variations of about 19 ft were observed also at piers 15, 99, and 194. The maximum difference in elevation of the top of sand along the structure was 37 ft between pile 16 in pier 14, and pile 5 in pier 99. The maximum difference in elevation of the average top of sand was 25 ft and occurred between piers 1 and 102.

142. Penetration into sand. The test piles penetrated an average of 4.3 ft into the sand so an allowance of 5 ft vertically was made in computing the length of pile to be cast. This amounts to 5.6 ft of actual pile length due to the batter. In actuality, the piles in the nonoverdrive reaches had an average penetration into the sand of 3.4 ft. The penetrations ranged from 0.8 to 25.0 ft. The average estimated and observed penetrations into sand, together with the maximum observed penetrations, are listed for each pier in table 15. The top of sand, as indicated by each pile, and the final pile tip elevations are shown for selected piers in figs. 44-49. These figures show graphically the magnitude of variations in driving into sand at individual piers. The minimum, average, and maximum penetrations into sand at each pier are plotted in fig. 50.

143. Pile lengths. The average lengths of the piles driven at each pier under the weir, together with the average length of cutoff, are tabulated in table 15. The average length of pile cutoff for piers in nonoverdrive reaches was 8.5 ft. Cost studies, subsequently discussed, showed that, in general, it was more economical to overestimate the lengths of the piles by 3.7 ft than to underestimate the lengths by 1.0 ft.

144. Inasmuch as the piles in the overdrive reaches in general did not penetrate to the originally estimated depths, the design lengths were on the average 12.9 ft longer than the actually-driven lengths. Even though it was evident early in construction that the estimated lengths were too long, it was decided not to shorten the casting lengths, as the driving in the overdrive reaches varied widely.

Economic considerations regarding pile lengths

145. In the original estimate of required pile lengths, 10 ft of pile was added to the length represented by the depth to the average top of sand, to provide for variation in the top of sand plus penetration into the sand for piers 4 through 41, 55 through 97, and 119 through 126, and 12.3 ft for piers 98 through 110. Cost analyses based on estimated bid prices and the assumption that the pile lengths for any one

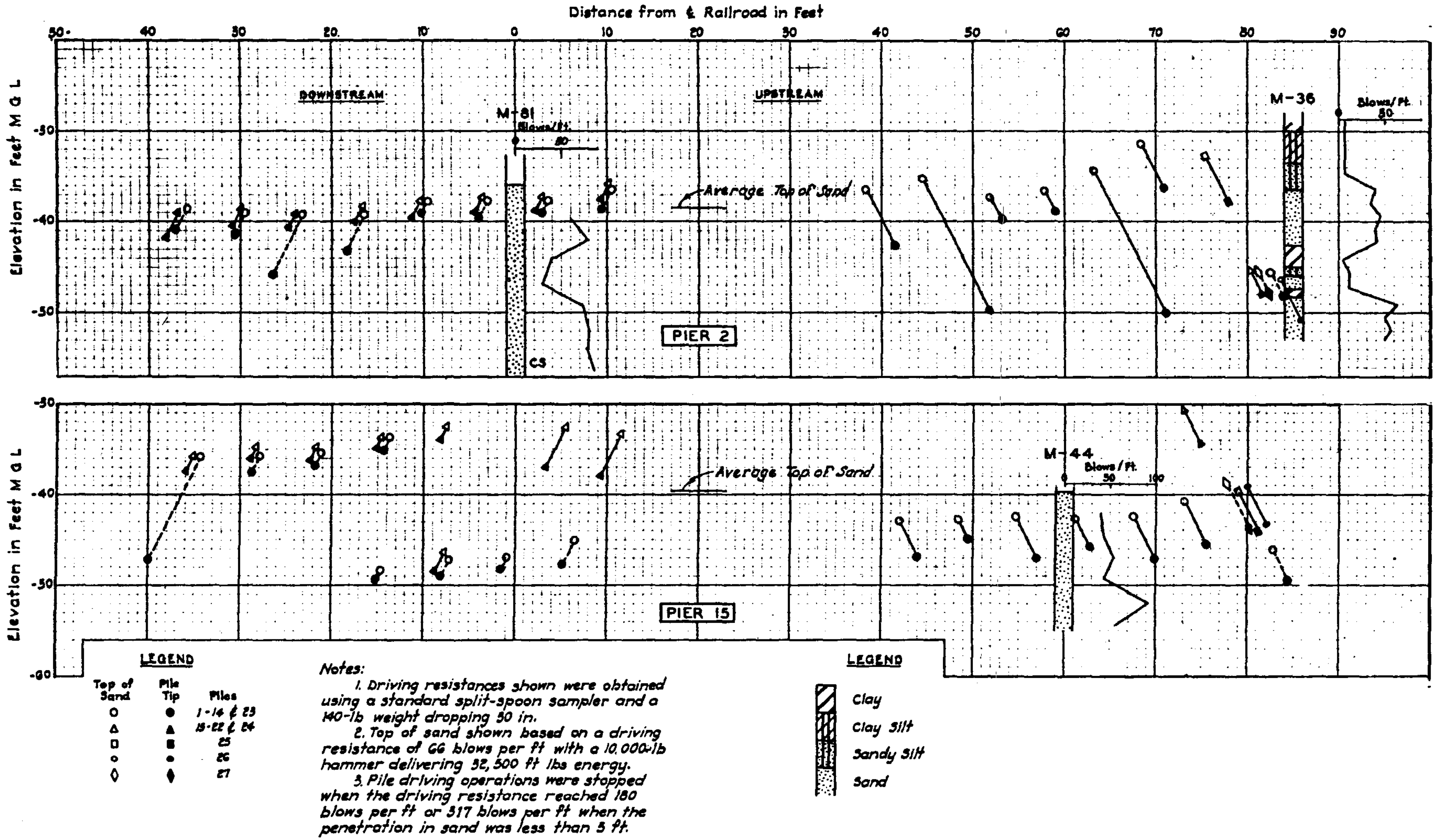


Fig. 44. Top of sand and pile tip elevations, piers 2 and 15

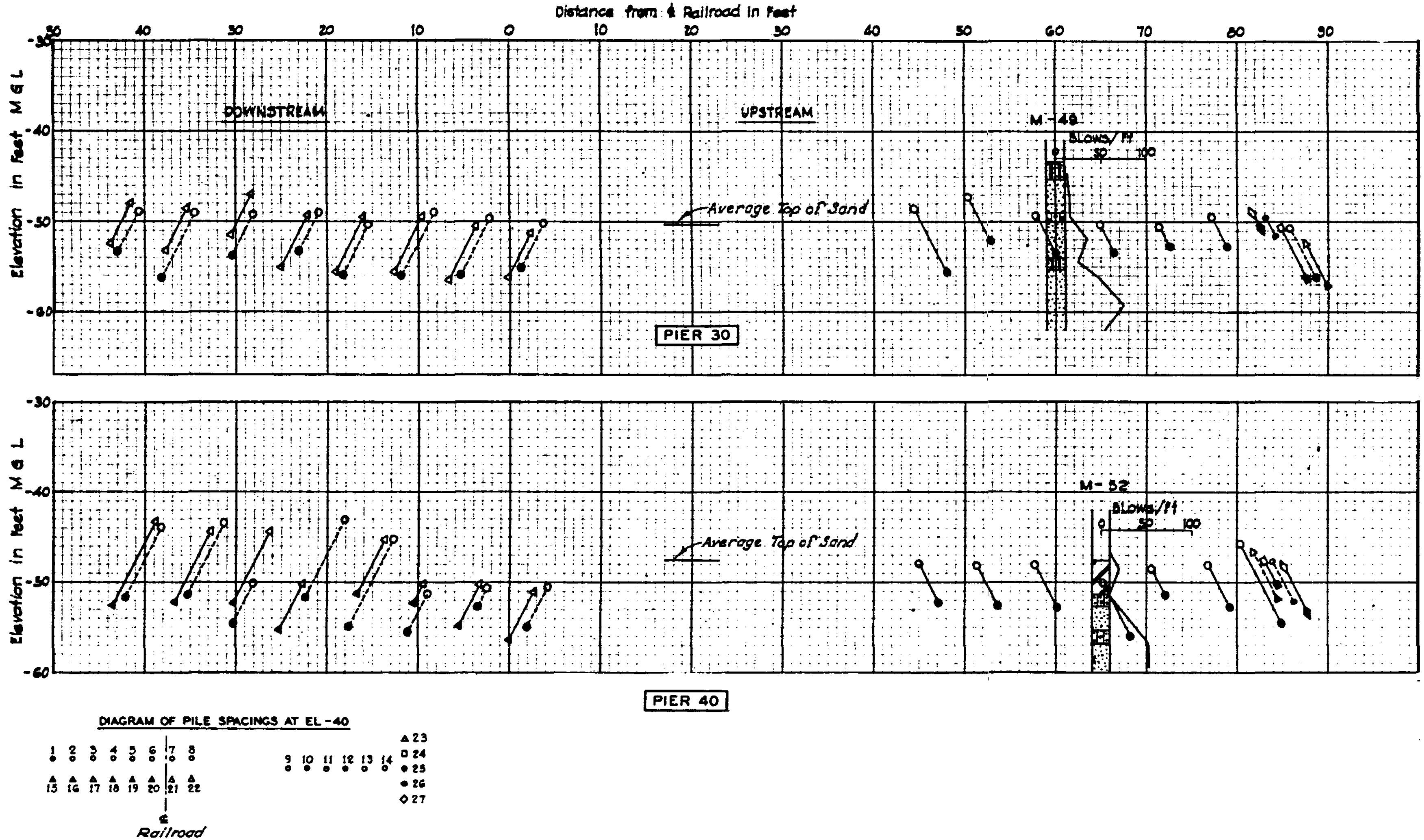


Fig. 45. Top of sand and pile tip elevations, piers 30 and 40

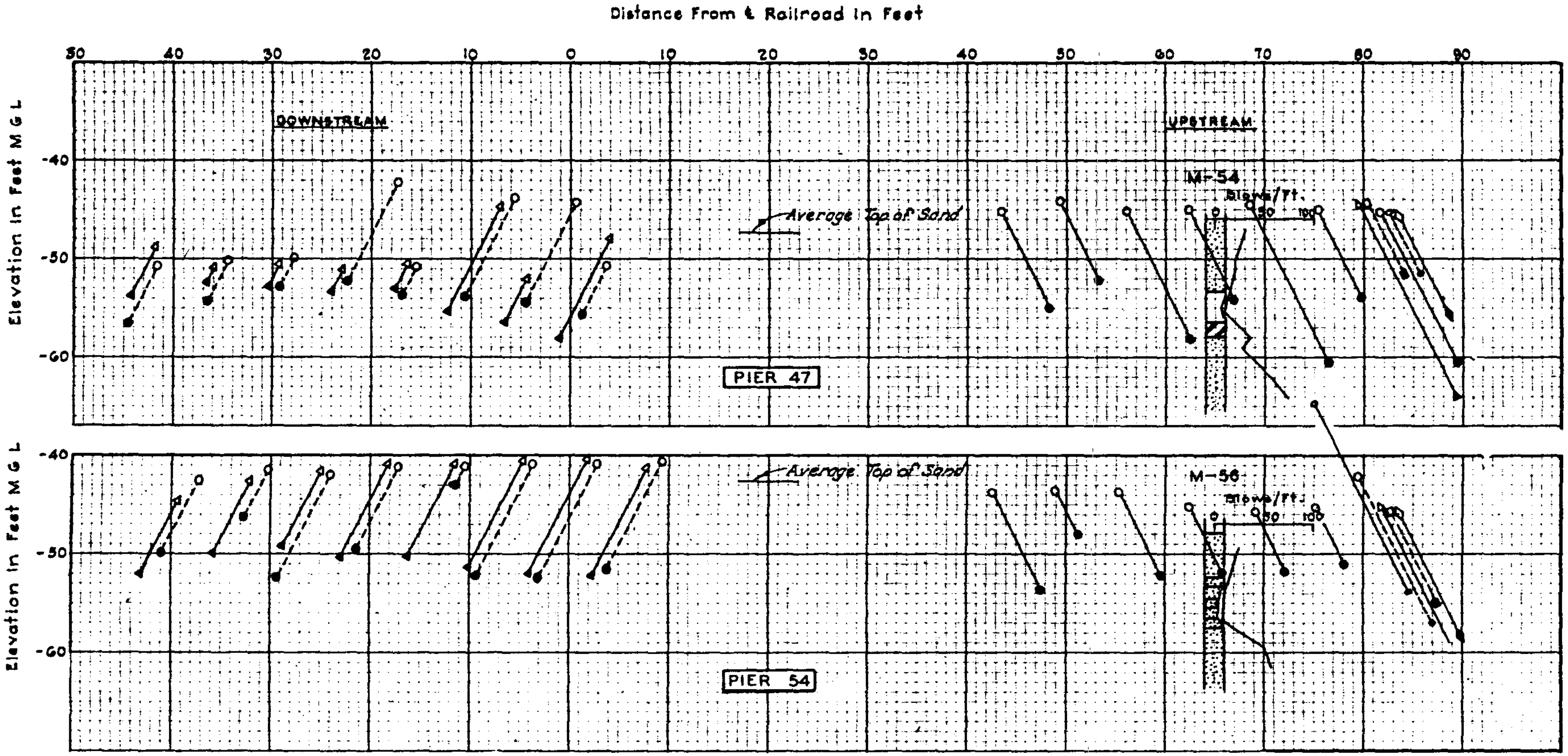


Fig. 46. Top of sand and pile tip elevations, piers 47 and 54

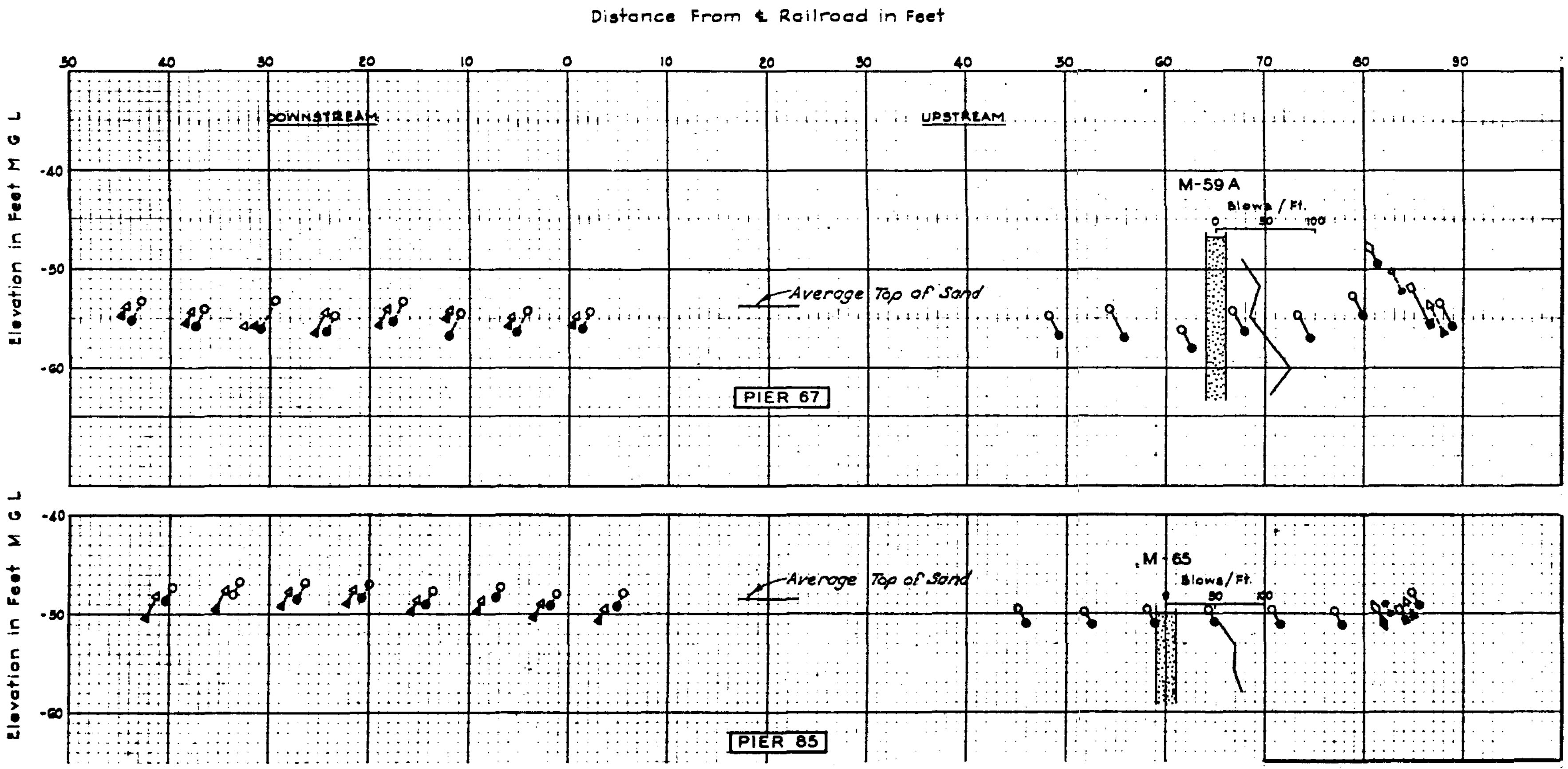


Fig. 47. Top of sand and pile tip elevations, piers 67 and 85

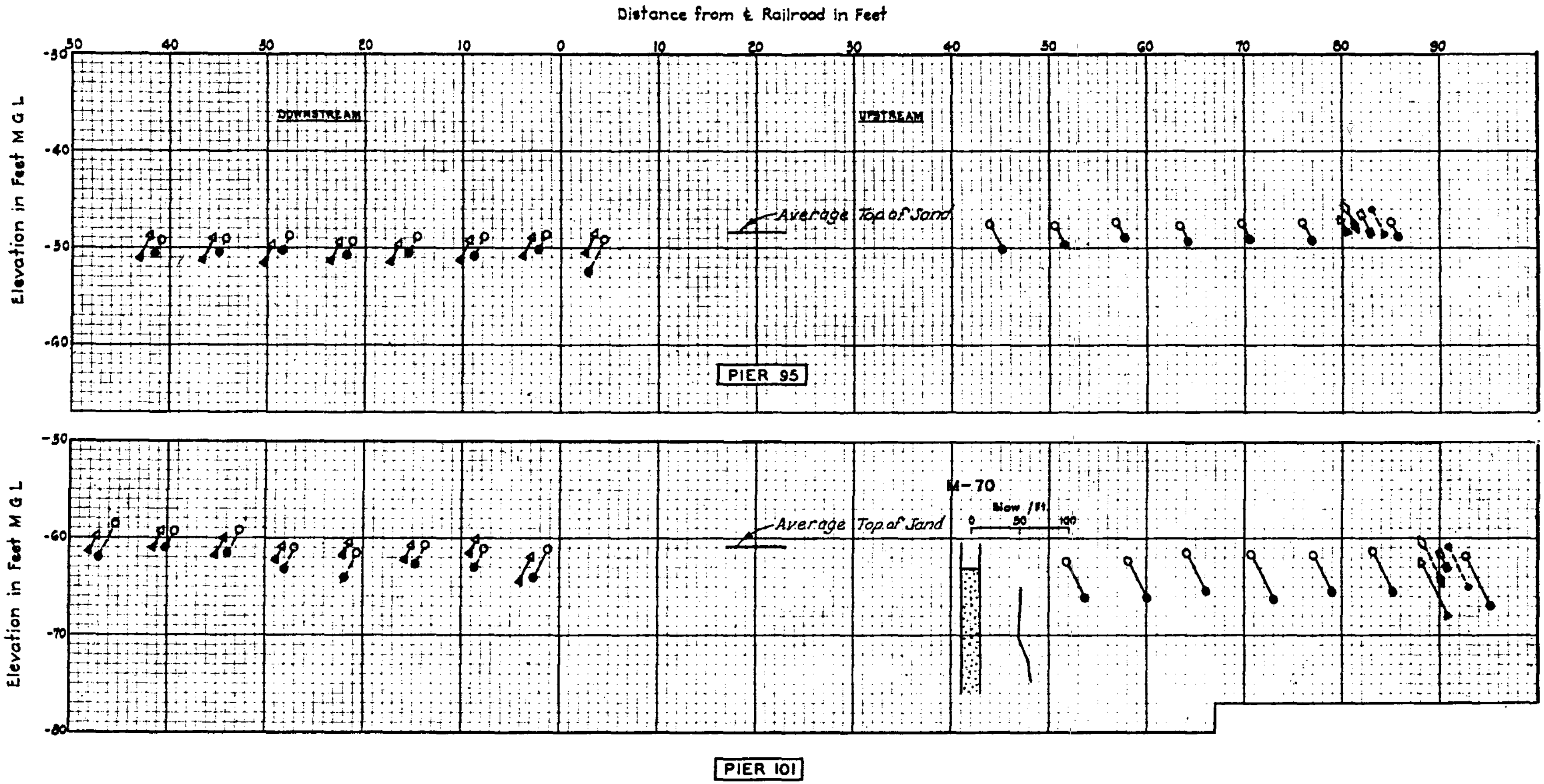


Fig. 48. Top of sand and pile tip elevations, piers 95 and 101

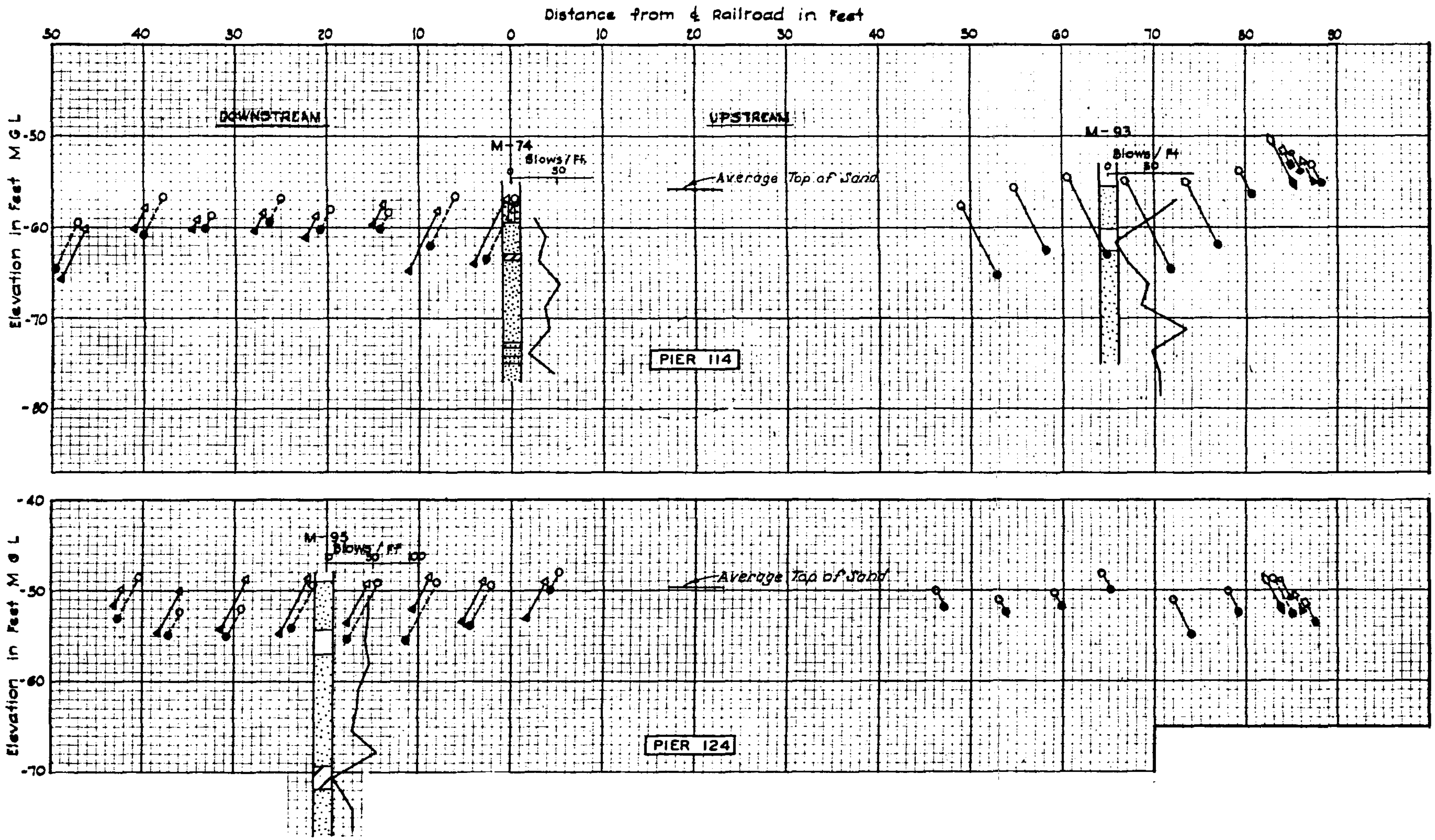


Fig. 49. Top of sand and pile tip elevations, piers 114 and 124

pier would vary uniformly from the shortest to the longest pile indicated the most economical pile length for a given pier to be the longest pile expected for that pier. By using actual contract unit prices and the lengths of piles as driven, it was possible to compute what would have been the most economical length of pile to cast for each pier. The cost of the piles at any pier below the average top of sand at that pier was computed for various allowances for variation in the top of sand plus penetration of pile below the average top of sand by the equation:

$$C = (5.50) \cdot 27 \cdot X + 113\, N_s + (5.50)\, \Sigma S$$

where C = cost of piles below average top of sand in dollars

X = assumed allowance for variation of top of sand plus penetration into sand in feet

N_s = number of piles that would have been spliced if length of piles cast had been based on X

ΣS = total length of splices required if lengths of piles cast were based on X

$5.50 = contract price for casting 1 ft of pile

27 = number of piles per pier

$113.00 = contract price for one splice

Different lengths of the various piles below the average top of sand at each pier were assumed as X and the values of N_s and ΣS were determined from the pile-driving data. Plots of C versus X, such as shown in fig. 51, were made for each pier. The optimum allowance in pile length for variation plus penetration below the average top of sand at any pier is the length X that would give a minimum cost. These values are listed in table 14 and are plotted on fig. 50. Intersection of the plotted data with the straight line shown on fig. 51 represents the allowance for penetration into sand and variation in top of sand which would eliminate need for splicing any piles. The optimum elevation of the pile tips at each pier is indicated on fig. 43.

146. The original estimate that the most economical length of pile at any pier would be that of the longest one required for that pier was largely borne out by computation of the optimum lengths. At almost two-thirds of the nonoverdrive piers the longest pile driven was also indicated to be the optimum length. Over half of the other nonoverdrive piers

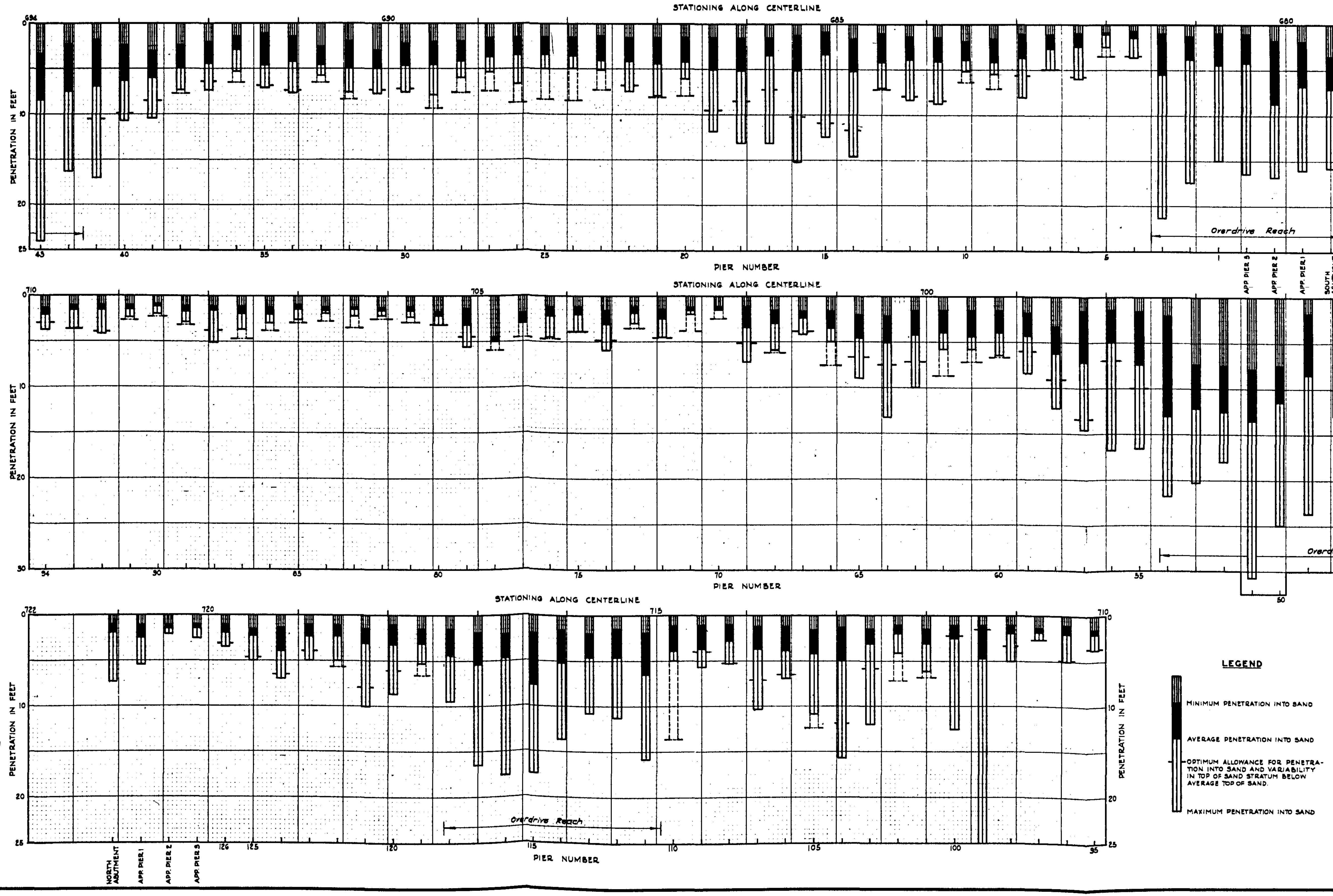


Fig. 50. Penetration of piles into sand stratum

showed an optimum length equal to the second longest pile at a computed cost, C, very little different from that of the longest pile.

147. The apparent discrepancy on fig. 50 of computed optimum allowance being greater than the maximum penetration into sand, as indicated at several piers, can be readily explained. This graph does not take into account the variation in the top of sand at the pier. As a rule, at those piers where optimum allowance is more than maximum penetration, the difference in the two values is the variation of top of sand indicated by the pile with the maximum penetration from the average top of sand at that pier.

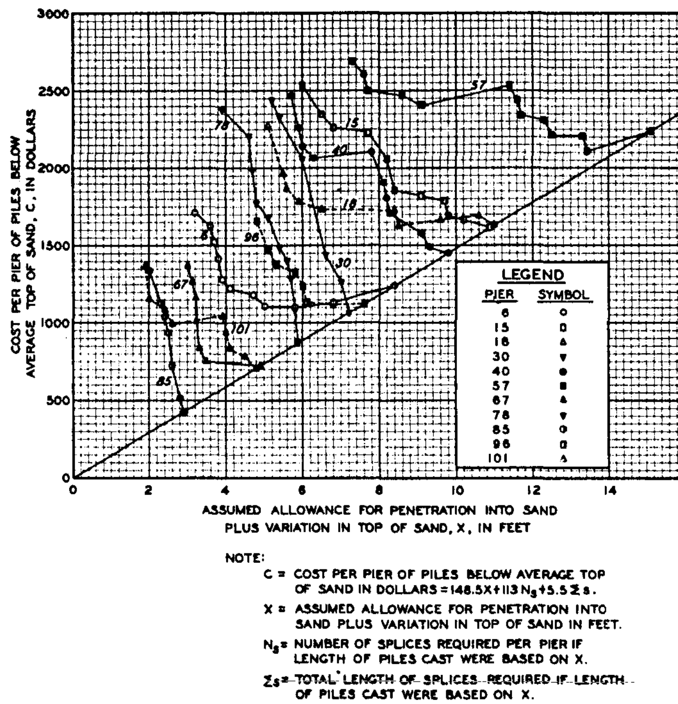


Fig. 51. Computation of optimum allowance for penetration into sand plus variation in top of sand

148. Statistical analyses of individual and cumulative percentages of piers with different averages of pile penetration into sand, and optimum allowance for penetration into sand plus variation in top of sand, are plotted on figs. 52 and 53, page 114.

149. The average optimum allowance for penetration into sand plus variation in top of sand at the various piers as computed from observed data was 6.3 ft. This value is less than the original design allowance of 10 ft. A study of the data presented in fig. 51 and similar data for other piers showed that it was more economical to cast the piles 3 to 5 ft too long than to cast them 1 ft too short. If pier 78 is taken for example, it can be seen from fig. 51 that the optimum allowance for penetration into sand plus variation in the assumed sand elevation is 5.9 ft, the minimum cost point on the cost curve for pier 78. Had the

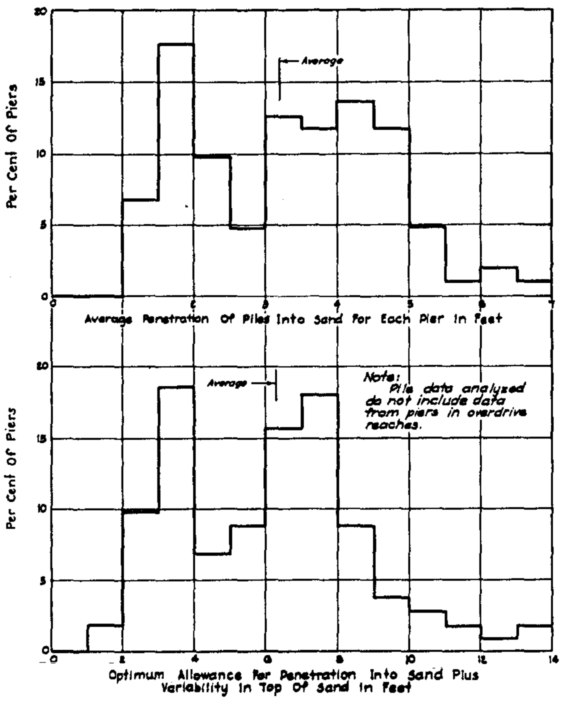


Fig. 52. Individual percentage of piers with various average penetration of piles into sand and optimum allowance for penetration into plus variation in top of sand

allowance for penetration been 4.9 ft, or 1 ft too short, it can be seen that the cost of the piles would have been the same as the cost of having cast the piles 6 ft too long.

150. The data previously presented show that variation in the top of sand and penetration into sand at any one pier makes it practically impossible to determine precisely the correct length of piles required at any individual pier or group of piers in the area of the Morganza Floodway. This is further illustrated by experience gained in driving piles for the trestle of the Texas and Pacific Railroad high-level crossing on the Morganza Floodway from McNeilly to Red Cross, La. Here the contractor attempted to determine the required pile length at each bend of the trestle by driving a test pile before placing his order for piling. Although this procedure is considered the best method for

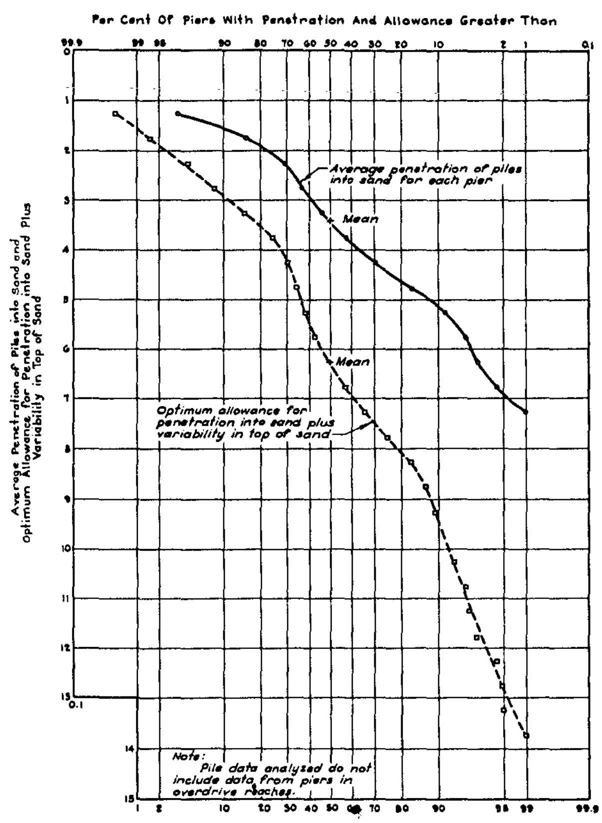


Fig. 53. Cumulative percentage of piers with various average penetration of piles into sand and optimum allowance for penetration into plus variation in top of sand

determining required pile lengths, the contractor missed the required pile lengths by as much as 25 ft at some bents.

151. As the original estimates were intended to be conservative and to keep the number of splices to a minimum, it is believed that the methods of analysis used in forecasting the required lengths of piles for the nonoverdrive reaches were quite accurate and resulted in nearly the most economical pile lengths with a minimum of splicing.

Overdriving of piles

152. In general, the overdriving of the piles was not too successful in securing penetration of the pile tips to depths originally designated "bearing sand." Fig. 43 shows that the average elevation of the pile tips at the various piers is 2 to 10 ft above the originally estimated "top of bearing sand." However, at each pier one or more of the piles did succeed in penetrating the strata with low driving resistance so that the tips were in "bearing sand." Study of the detailed driving records showed that overdriving did cause about one-third of the piles in the overdrive reaches to pass through highly resistant sand into and through material having a resistance of less than 18 blows per 0.1 ft. The driving records shown for piles 13 (pier 2) and 11 (pier 47) on fig. 54 and for pile 11 (pier 114) on fig. 55 are typical. The rate of 18 blows per tenth of a foot was the absolute minimum resistance required by the specifications at completion of driving. It is believed that the low-resistance material in the sand penetrated by about half of the above-mentioned one-third of the piles was clay as the driving resistances dropped to less than 10 blows per tenth of a foot (see driving record of pile 3 in pier 47 on fig. 54, and pile 4 in pier 54 on fig. 55). The driving resistance in the principal upper clay stratum was less than 6 blows per tenth of a foot. Most of the piles penetrating sand continued to build up resistance to driving as indicated by pile 6 in pier 2 as shown in fig. 54. A large number of piles, particularly at piers 42 through 54, had driving resistances similar to those shown for pile 4 under pier 54, and pile 4 under pier 114 in fig. 55. These piles went through a considerable depth of material having a driving resistance of sufficient value to be considered "sand" but lower than the minimum

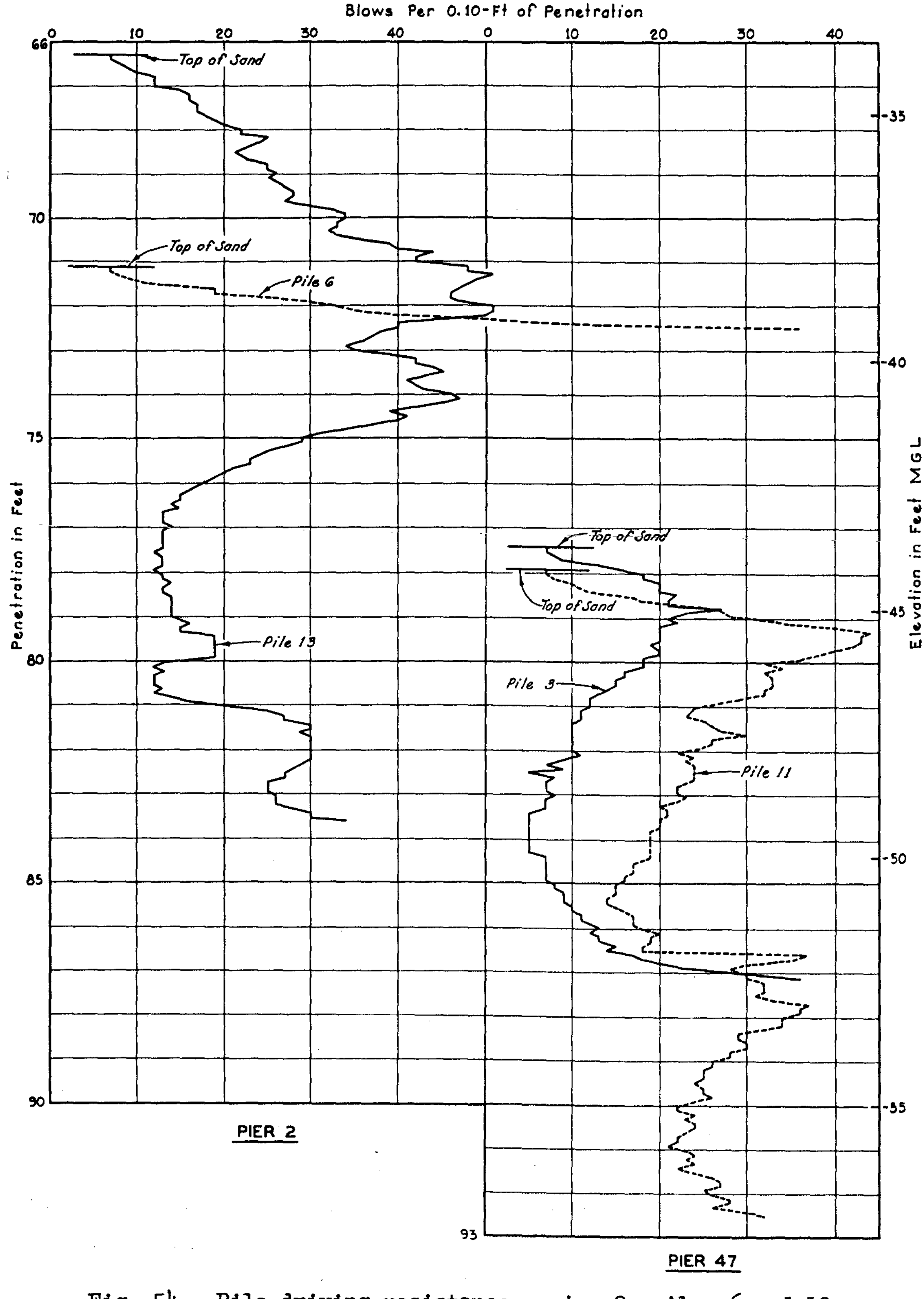


Fig. 54. Pile driving resistance - pier 2, piles 6 and 13; pier 47, piles 3 and 11

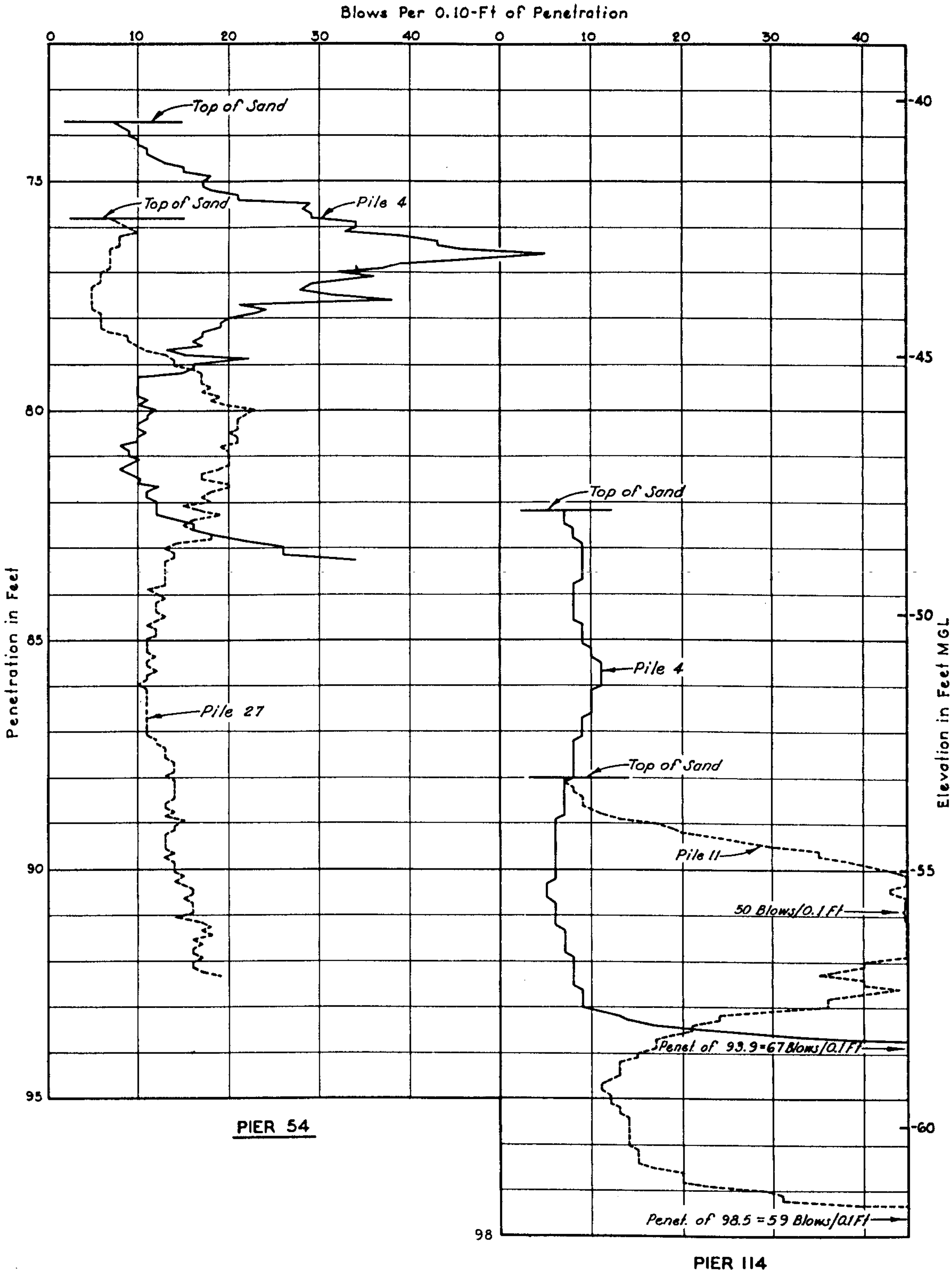


Fig. 55. Pile driving resistance - pier 54, piles 4 and 27; pier 114, piles 4 and 11

required by the specifications before reaching the required resistance. As the compressible strata in the upper part of the sand are probably discontinuous and intermittent, it may be that overdriving caused the majority of the piles to penetrate through most of the lenses and strata of compressible material which otherwise would have been below the pile tips.

Pile Driving

153. The piles were driven with two Universal pile drivers with 136-ft leads which could be set for vertical driving or in-and-out batter driving. The hammers were Vulcan No. 0 modified with a 10,000-lb ram to deliver 32,500 ft-lb of energy per blow. The hammers were of the single-acting type. One of the pile driving rigs and a portion of the pile foundation are shown in fig. 56. Two rigs were used, and both started

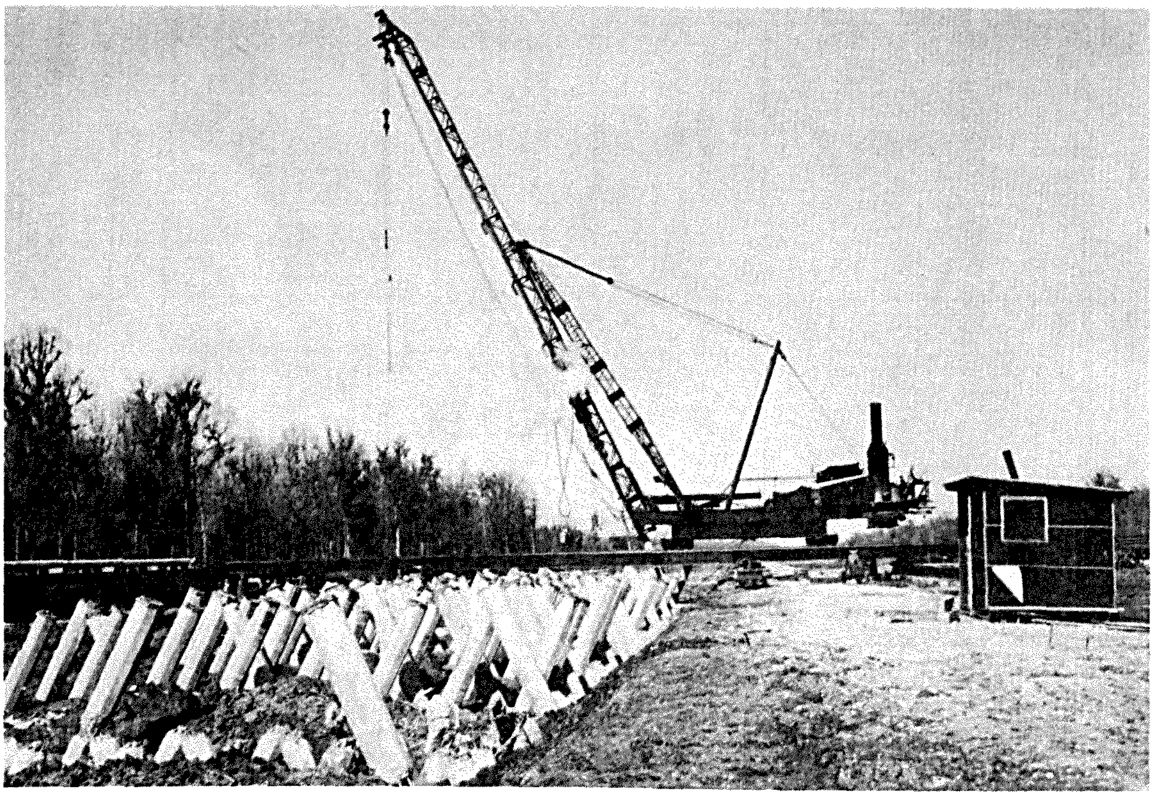


Fig. 56. Excavation and precast concrete piling for control structure. (Note batter of piles)

at approximately the center of the structure and worked toward opposite ends so as to permit the maximum amount of consolidation beneath the preload fills at the ends of the structure where it joins the highway and railroad embankment.

154. With the size of hammer used, the top of the sand was considered to be reached when the driving resistance became 7 blows per 0.1 ft. The resistance at completion of driving was required to be 18 blows per 0.1 ft and this resistance was considered satisfactory only if the pile had penetrated at least 5 ft into sand. The engineers on the project were instructed to consider as satisfactory an average driving rate of 18 blows or more per 0.1 ft for a distance of 0.3 ft provided there was no decrease in the resistance to driving. A minimum driving rate of 32 blows per 0.1 ft for piles not penetrating 5 ft into the sand was established. The driving resistances originally established as "refusal" for piles to be overdriven were 50 blows per 0.1 ft for octagonal piles and 55 blows per 0.1 ft for square piles. This rate was increased later by the resident engineer at the site.

155. The usual driving procedure at each pier was to start driving at the downstream end of the center row and work to the upstream end. The outer rows were driven next from the upstream to the downstream end. This procedure resulted in a minimum of tightening up of the foundation and permitted expansion in the subsoil without undue displacement of the piles previously driven. It was originally estimated that there would be 3 to 4 ft of heave of the ground surface in the vicinity of the piers as a result of driving the piles, actually only about 1.0 ft of heave occurred.

156. The piles sank approximately 16 to 20 ft into the ground under the combined weight of the pile and hammer, approximately 17 tons. The driving resistance through the clay built up gradually such that the driving rate in the 10 ft above the sand was normally about 2 blows per 0.1 ft. The resistances increased sharply from low values in clay to high values in the sand within 1 to 2 ft. After the pile had penetrated into sand the driving rate was measured for each 0.1-ft penetration. Driving resistances for selected piles at four piers in overdrive reaches

are shown in figs. 54 and 55. Most of the piles in the nonoverdrive reaches were driven to final resistances in excess of 32 blows per 0.1 ft as they did not penetrate 5 ft into sand.

157. Piles in the overdrive reaches were driven considerably harder than the originally stipulated "refusal" rate (50 to 55 blows per 0.1 ft) in an effort to drive the pile tips to elevations below the clay strata. This rate of driving selected as refusal was based on a study of records of hard driving of precast concrete piles on other projects. This study showed that reinforced concrete piles would not be damaged if the driving energy was restricted to less than 5000 ft-lb per inch penetration per square inch of pile area, if the driving is properly done. Some of the piles withstood driving energies up to 12,000 to 15,000 ft-lb per inch penetration per square inch of pile area without material damage to the pile head for a penetration of 0.3 ft. The concrete of which the piles were made had a test strength of at least 4000 psi. The heads of the piles were protected with pads consisting of 3 Celotex or cottonwood boards, each 7/8-in. thick. Very little, if any, apparent damage was done to the top of the pile for driving resistances up to 32 blows per 0.1 ft of penetration. Some of the piles were driven as hard as 200 blows per tenth of a foot for 0.3 ft and others as high as 400 blows for the last 0.1 ft without material damage.

158. Driving the piles on the specified batter of 2 on 1 created no particular problem. The flatness of this batter, however, did cause some concern regarding drifting of the piles during driving and deflection of the pile tip when it struck sand. As a check on this, a 5-in. ID steel pipe was cast in the center of three piles, to within 2 ft of the pile tip; these piles were subsequently driven as piles 3, 11, and 13 of pier 44. The deflection of these piles was determined by a photographic drift indicator which was lowered into the piles after completion of driving.

159. Pile 3 was driven to a resistance of over 50 blows per 0.1 ft penetration; however, it penetrated only 2.2 ft into sand. Pile 11 was driven to a resistance of 64 blows per 0.1 ft penetration and penetrated 7.1 ft into the sand. Pile 13 penetrated a total of 12.2 ft into sand

and driving was terminated when the resistance reached 32 blows per 0.1 ft penetration.

160. The drift of these piles and maximum bow as measured from a straight line from the top to the tip are tabulated below:

Pile	Drift in ft	Maximum Bow in ft
3	0.32	0.27
11	1.92	0.52
13	1.10	0.38

A plot of the observed deflection of the above piles is shown in fig. 57.

161. A computation of the stresses created in the concrete and reinforcement steel by the deflection observed in pile 13 showed the following maximum stresses in the steel and concrete:

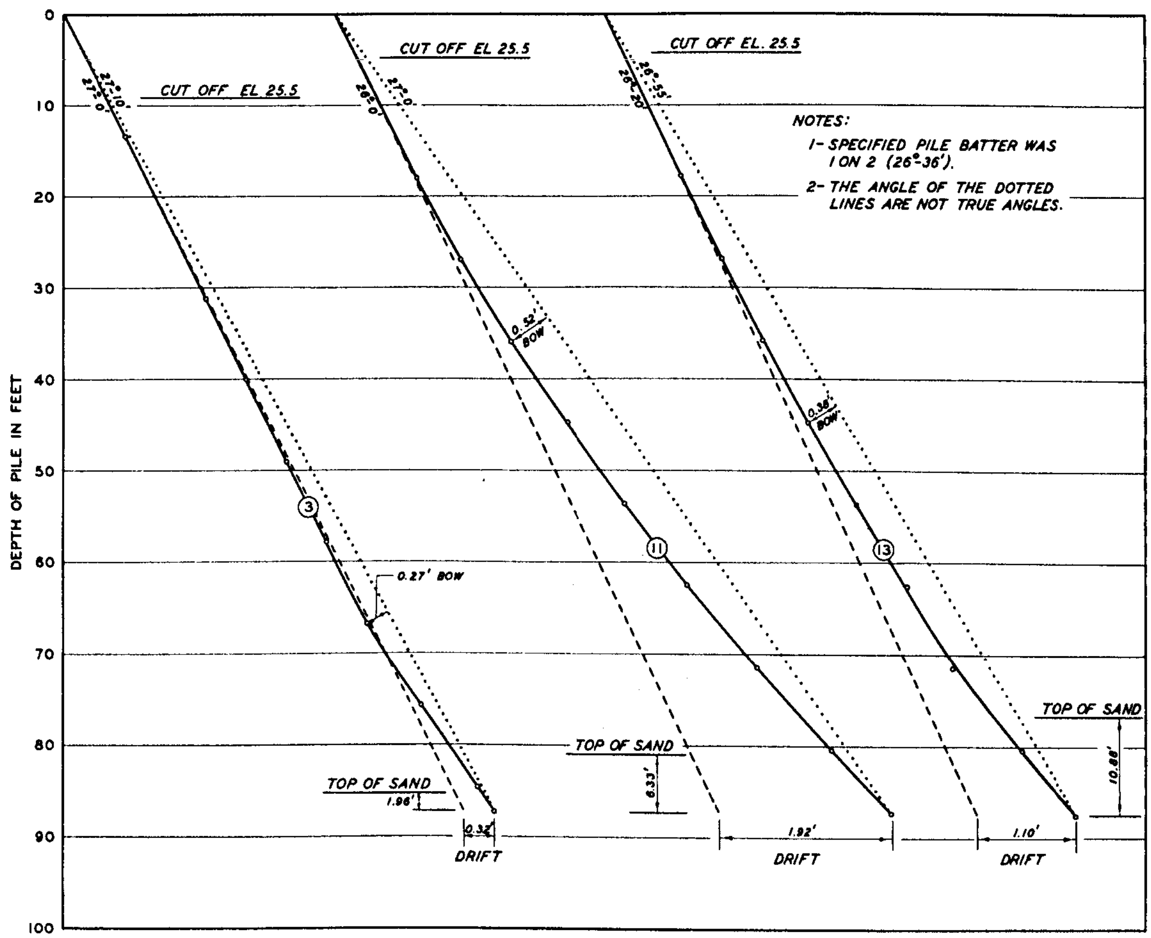


Fig. 57. Deflection of piles 3, 11, and 13, pier 44

Load on Pile	Type of Pile	Stress in psi: Concrete (Compression)	Stress in psi: Steel (Tension)
None	Octagonal	1290	18,800
None	Square	1440	24,000
100 ton	Octagonal	1265	2,560
100 ton	Square	1480	6,950

Stresses considered allowable are 1560 psi for the concrete and 24,000 (temporary) and 20,000 (permanent) psi for the steel reinforcement. It was estimated by design engineers of the Mississippi River Commission that after one year the stresses due to bending would only be one-third of the above computed values. Considering the fact that the deflection and stresses created in the piles were not excessive, the batter and driving procedure were considered satisfactory. However, a batter of 2 on 1 is considered the flattest that should be used for similar foundation conditions and driving.

162. A portion of the complete pile foundation for the structure is shown in fig. 58. A rapid rate of driving the piles was achieved by the contractor, the Raymond Concrete Pile Co. The average driving rate in the nonoverdrive reaches was 7 piles per 8-hr shift. No trouble was experienced in handling either the 20-in., 100-ft-long or shorter octagonal piles, or the 100-ft-long or longer square piles. The piles were supported at four points during handling.

163. The split-spoon driving tests indicated a tendency for the driving resistance to be reduced by increased pore pressures created by high river stages in the adjacent Mississippi River. Accordingly, it was thought that the river stages might also have some effect on penetration of the piles into sand. However, a plot of river stage vs average penetration into sand showed no definite relation, and if the river stage had any effect on pile penetration it was masked by variations in the density of the sand at the different piers.

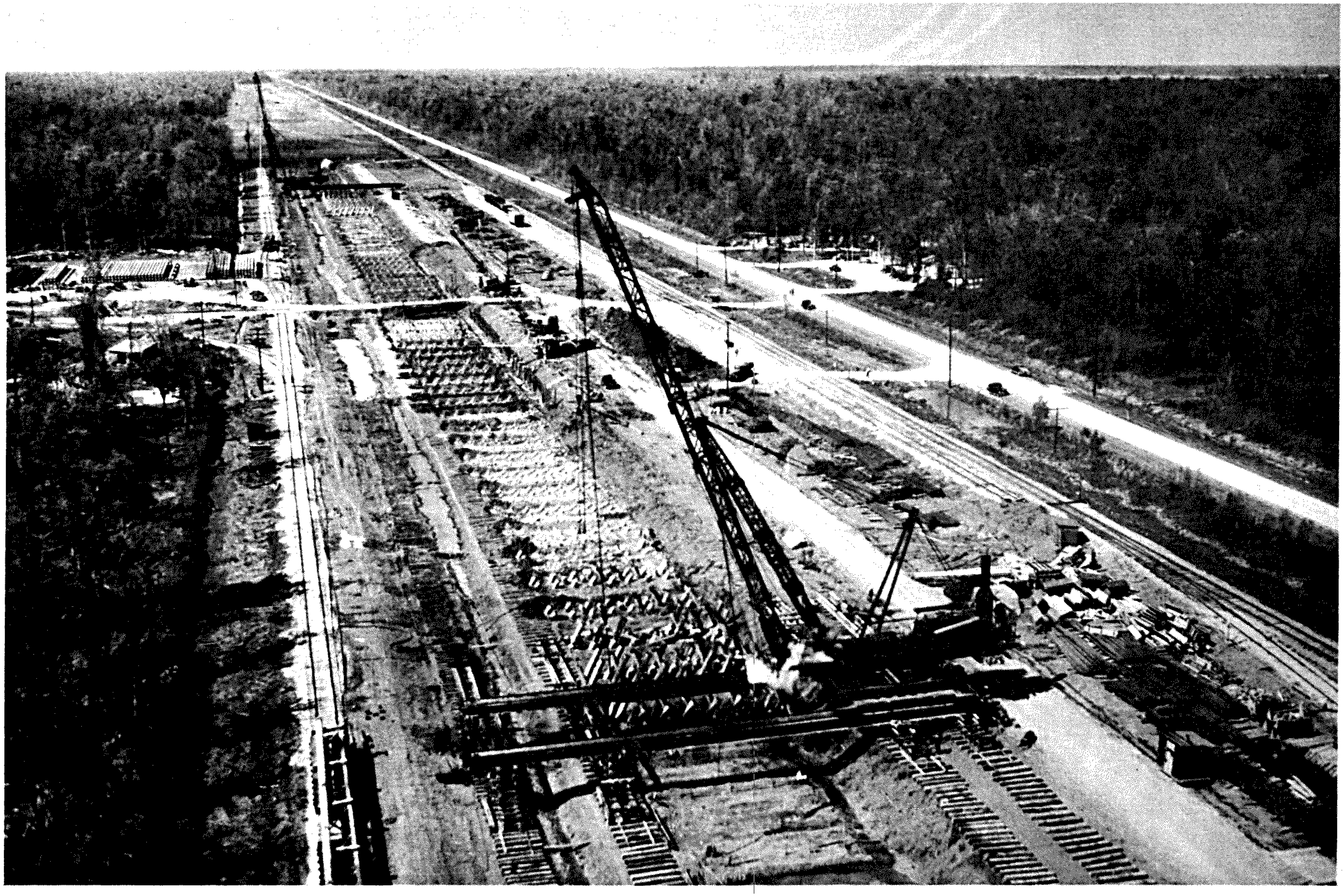


Fig. 58. Pile foundation for control structure

PART VII: ABUTMENTS

164. Four approach spans carry the highway, railway, and crane track from the main embankment to the gated portion of the control structure. The piers for the approach spans are supported on piles driven to sand. Details of the abutment are shown in fig. 59. The crown of the embankment ends at the abutment piers from which the embankment slopes downward to the end pier of the gated portion of the structure with the same slope as the side slopes of the embankment, except that the 1-on-20 berm on the side slopes is warped to a 1-on-10 berm on the end slope. The wing walls shown in fig. 59 act as retaining walls for about 8 ft of fill and as hydraulic training walls for the control structure.

Preloading of the Abutments

165. Because of the weak and compressible foundation it was necessary to connect the embankments to the control structure so that detrimental differential settlements would be avoided.

166. Preliminary studies indicated that the large settlement expected beneath the embankments would create a number of problems if the fill at the abutments was placed after completion of the structure. The three principal difficulties anticipated were: (a) control of underseepage at the abutments; (b) deflection of batter piles beneath the abutment piers due to subsidence of the soils under the embankment load; and (c) maintenance of the finished embankment crown. It was therefore decided that the abutment areas would be preloaded so that a major portion of the troublesome settlement beneath the ends of the embankment would take place before the structure was constructed.

167. This preloading was accomplished by constructing the ends of the embankments to the cross section shown on fig. 59 with the crown extending to the end of the gated portion of the control structure. The end slopes of the embankments thus extended into the structure area. An effort was made to place the preload fills as early as possible during construction so that the foundation would be loaded for a maximum period

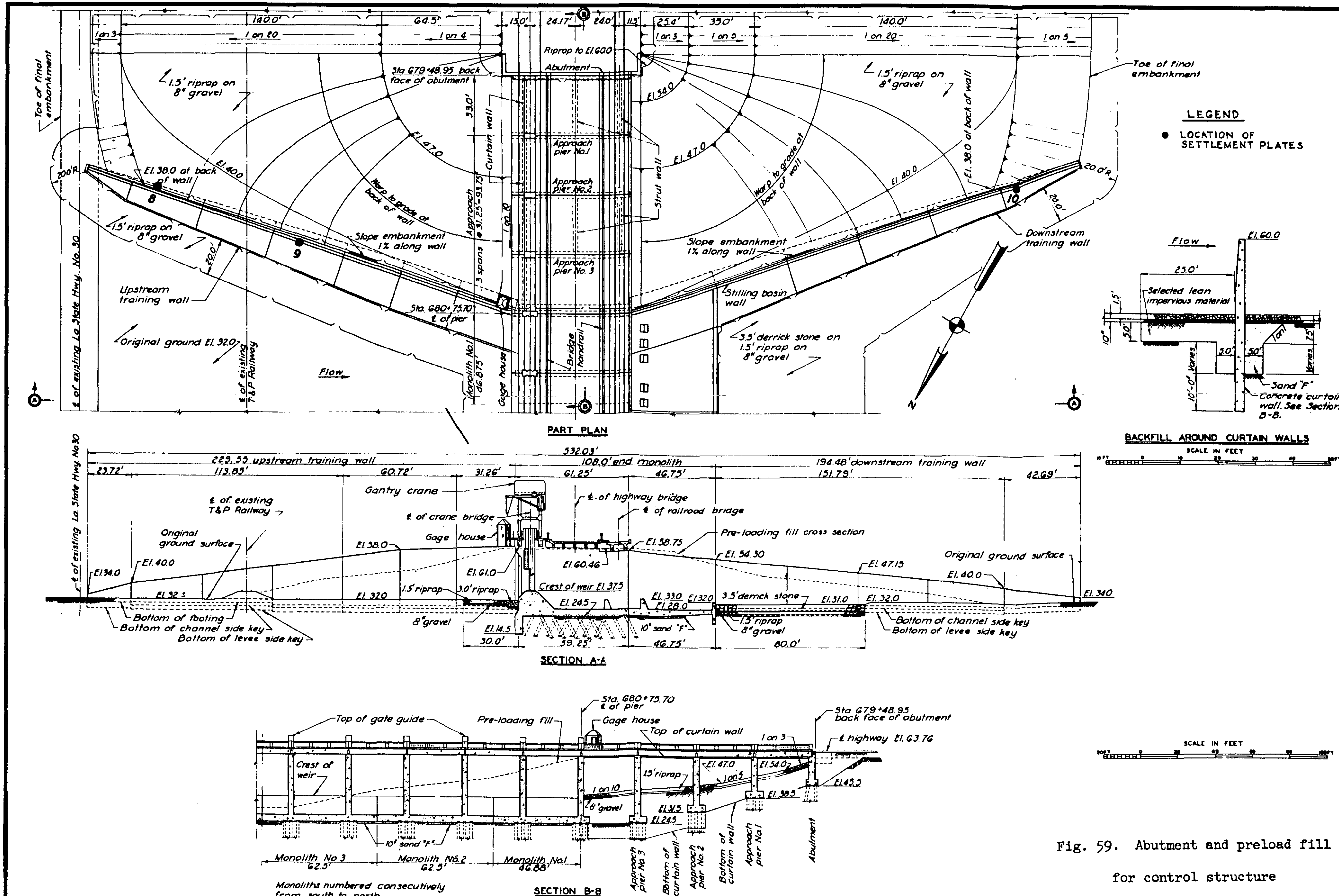


Fig. 59. Abutment and preload fill for control structure

of time before the abutments were constructed. The preload fill for the south abutment was constructed in October 1949; however, it was not possible to construct the preload fill for the north abutment until July 1950 because of weather conditions and the high water content of borrow materials. As some of the preload fill was higher than the final grade of end of the embankment, it was anticipated that some parts of the overloaded areas would reach settlements equal to those expected under the final embankment grade. At the end of the preloading period (just before driving the piles for the abutment piers) the fill in the structure area and above the final grade of the embankment was removed. The preload fills in the abutment areas were removed during the period April-September 1951, resulting in a total period of loading for the south abutment of 1-1/2 years, and for the north abutment of 1 year. Settlement of the preload fills at both abutments is plotted on fig. 60. This figure shows that 2.6 ft of settlement of the foundation at the south abutment was achieved before driving the piles for the abutment and approach piers; 2.4 ft was obtained at the north abutment.

168. Preloading the abutments reduced the tendency for the fill to pull away from the curtain wall. In addition, the possibility of other cracks and voids opening up was largely eliminated, thus materially reducing the danger of piping. Since most of the settlement of the foundation beneath the abutment took place before the piles for the abutment piers were driven, it was thought

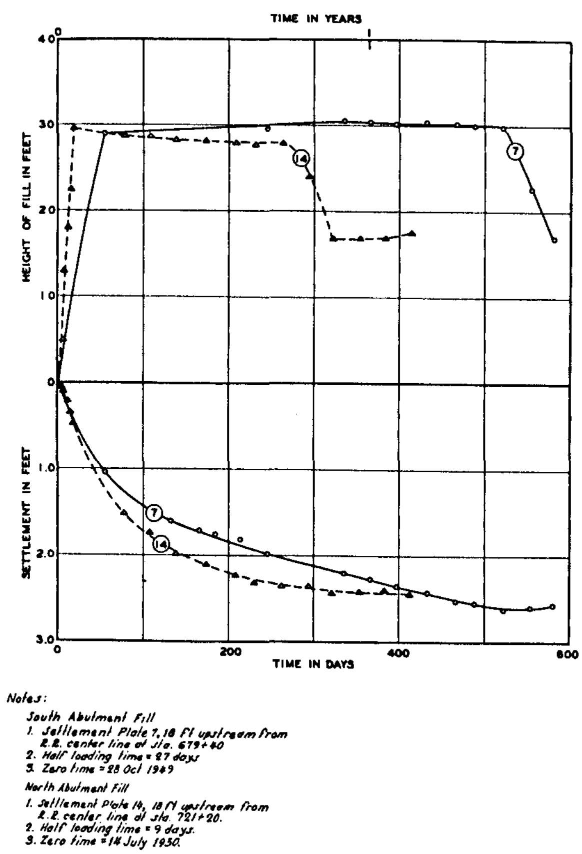


Fig. 60. Settlement of preload fills at control structure

that little deflection of the batter piles due to settlement of the soil under the embankment load would occur. Also the problem of maintaining a transition from the nonsettling structure to the settling embankment was materially reduced.

169. In May 1951 the following analyses of the expected settlements of the abutment piers were made. The data were based on settlement analyses made during 1950 and observations of settlement plates which had been placed beneath the preload fills at both abutments. The data given below were considered applicable to both the north and south abutment piers for the structure. The following is a summary of the analyses made:

(a)	Ultimate settlement of clay stratum beneath preload fill at abutment	3.8 ft
(b)	Total settlement of clay stratum beneath final section of embankment at abutment pier	4.0 ft
(c)	Settlement of clay stratum beneath abutment pier beneath preload fill by the time piles were driven	2.8 ft
(d)	Settlement of clay stratum beneath abutment pier after the piles were driven	1.2 ft
(e)	Total settlement of clay stratum beneath first approach pier for final embankment section	3.0 ft
(f)	Settlement of clay stratum beneath first approach pier after piles are driven	0.2 ft
(g)	Vertical settlement at mid-point of clay stratum at abutment of pier after piles are driven	0.2 ft
(h)	Settlement of clay stratum beneath second and third approach piers after piles are driven	0.0 ft
(i)	Total settlement of clay stratum beneath wing walls from about the center of wall to the end of wall	0.2 to 0.5 ft

170. The above data show that additional settlement would probably occur at the abutment, at the approach pier nearest the abutment, and at the intersection of the existing railroad with the upstream wing walls after the foundation piles were driven. This additional settlement would be due in part to the incomplete consolidation obtained under the

preload fills and to the additional fill required to bring the embankment to final grade. As a result of the additional settlement of the fill, it was thought that there would be some drag on the piles, with some resulting settlement, of the abutment and first approach pier and wing wall.

171. On the basis of the above data, it was decided that the piles beneath the abutment piers would be driven vertically and on a 20-on-1 batter. Also, in order to prevent overloading of these piles as a result of drag created in the foundation by settlement of overlying embankment fill, the number of piles was increased so that the maximum normal design load per pile would only be 50 tons for the abutment and 92 tons for the first approach pier. It was also decided that the piles beneath the first approach piers would be driven on a steeper batter than the originally planned 2-on-1 batter, namely 5 on 1, and would be overdriven so as to obtain the maximum possible bearing capacity.

172. Originally it was planned to preload only the foundation for the abutment and approach piers. However, settlement analyses of the upstream wing wall at the south abutment at a point midway between the existing highway and railroad, and at a point on the center line of the existing railroad, indicated an ultimate settlement behind the wing walls at these points of about 0.9 ft due to the earth embankment. Therefore, it was decided to extend the preload fill between the existing highway and railroad over the location of the wing walls. With this additional preload fill, it was estimated that if the fill were placed by 1 June 1950, practically all of the settlement at a point midway between the existing highway and railroad would take place by 1 October 1951, the estimated date for removal of the preload fill and driving the piles for walls. The preload fills were placed in October 1949 and July 1950. A settlement analysis at a point on the center line of the existing railroad indicated that 0.5 to 0.6 ft of settlement might take place at this point after the piles for the wing walls were driven. Therefore, these piles may be overloaded to some extent as a result of drag created by the settling foundation. The preload fills were also placed so as to cover the location of the downstream wing walls. As a result of the preload fill, it was estimated that little or no

settlement would occur under the downstream wing walls. The additional preload fills for the wing walls corresponded approximately to the final sections of the embankment and approximately half of the fill remained in place as part of the final embankment.

173. In order to determine the rate of settlement of the preload fill at the wing walls, 6 settlement plates were installed at the locations shown in table 19; locations of plates 8, 9, and 10 at the south abutment are shown on fig. 59. The settlement of these plates as observed just before the preload fill was removed and the piling driven is given in the following table.

Table 19

Settlement of Foundation beneath Preload Fill at Wing Walls

Plate No.	Station	Distance from Center Line RR in ft	Settlement in ft 4 May 1951
8	680+10	245 east	0.22
9	680+40*	170 east	0.48
10	680+10	210 west	0.14
11	720+45	245 east	0.32
12	720+20*	170 east	0.80
13	720+45	210 west	0.36

* Fill was already in place at these locations, so the reference point was established by anchoring a 1-in. riser pipe in about 1 ft of concrete placed in a 6- or 8-in. hole at the original ground surface.

Reference hubs were also established on top of the preload fill along the wing walls for the purpose of obtaining settlement data. However, these hubs were damaged during construction operations and no satisfactory data were obtained from them.

Control of Underseepage

174. The water barrier for the section between the main embankment and the end pier of the gated portion of the control structure is a vertical concrete curtain wall on the upstream side of the piers. The wall penetrates into the embankment and foundation to the depth shown

on fig. 59. The curtain wall was extended into the foundation under the two abutment spans adjacent to the gated opening to provide an adequate factor of safety against seepage around and under the wall. An additional means of control of underseepage consisted of backfilling the riverside of the curtain wall with selected lean impervious soil. The backfill on the landside of the wall consists of a zone of sand overlain by 12 in. of gravel and 18 in. of riprap to intercept and collect possible underseepage.

PART VIII: UNDERSEEPAGE AND DRAINAGE SYSTEMS

175. As the top stratum of clay has a very low permeability, the quantity of underseepage beneath the control structure should be small. However, without some means of pressure relief uplift pressures might develop in a sandy silt stratum between elev 9 and elev -1 and immediately beneath the base slab. As it was desirable from the standpoint of structural design to keep uplift pressures at a minimum, a drainage system was designed to reduce uplift pressures beneath the structure.

Description of Drainage System

176. The drainage system consists of a 10-in.-thick sand drainage blanket beneath the structure, plus a line of relief wells to relieve excess hydrostatic pressures in the sandy silt stratum. Gradations of the filter materials used fell within the limits shown in fig. 61. Details of the drainage system are shown in fig. 62. The relief wells shown in fig. 62 consist of an 8-ft screen of 1-1/2 in. ID Norton porous

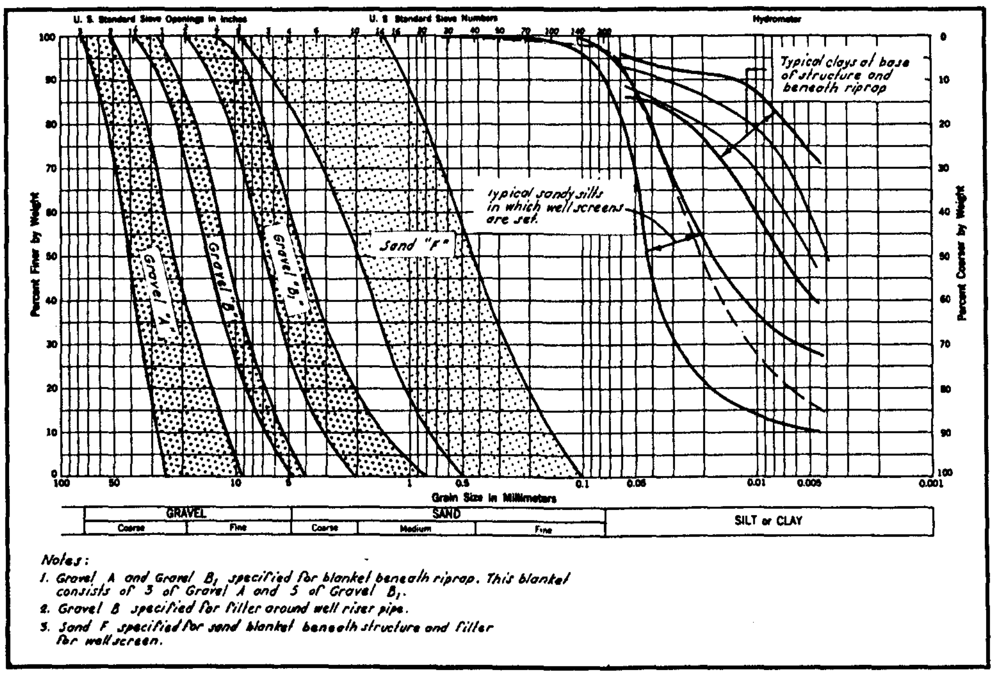


Fig. 61. Filter materials for control structure

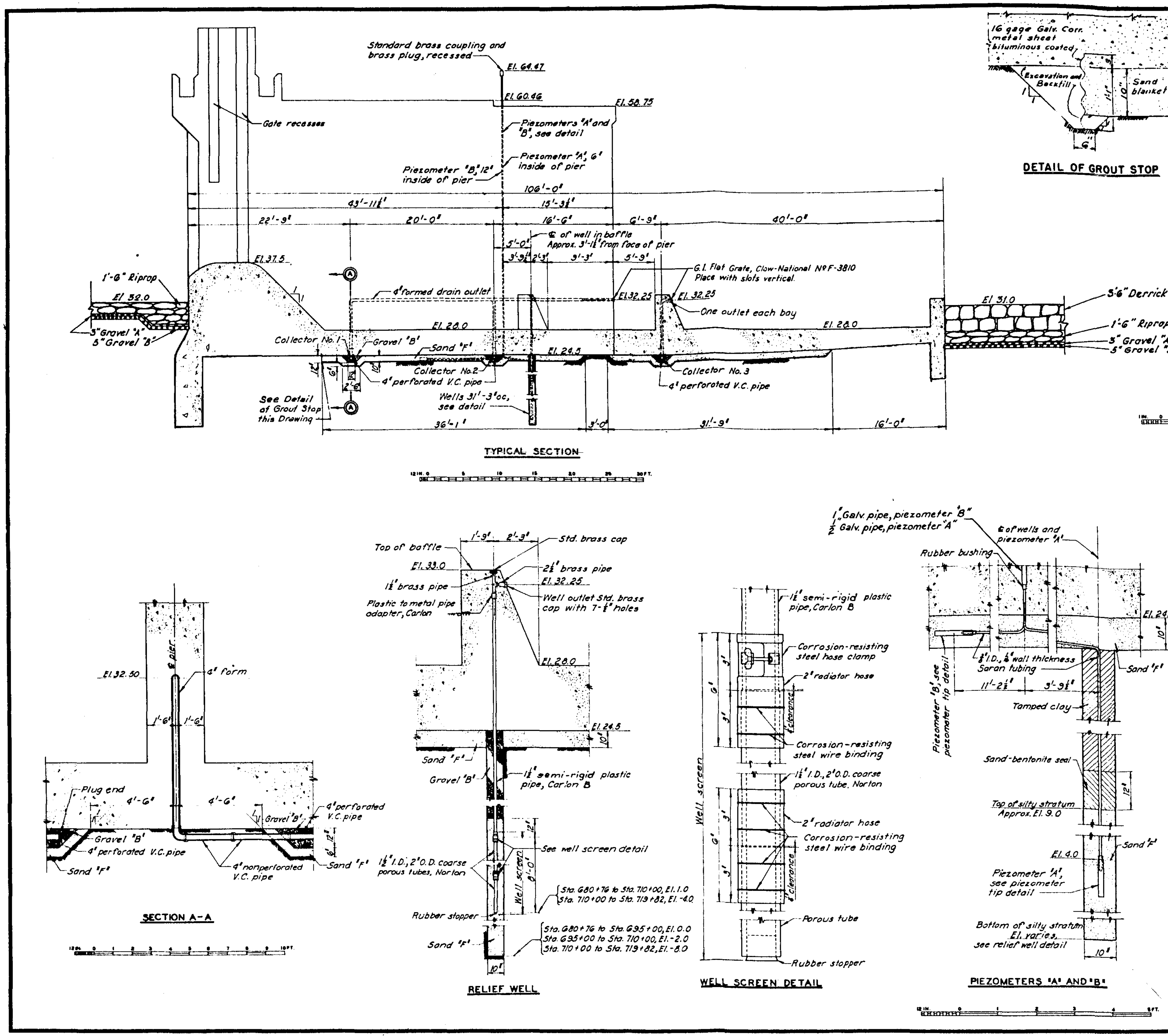


Fig. 62. Drainage system and piezometers beneath control structure

tube surrounded with a filter of sand F, and a riser pipe of 1-1/2-in. ID polyethelene semirigid plastic pipe (Carlon B) surrounded with gravel B. A 10-ft-deep key was constructed along the upstream edge of the base slab for protection against scour and piping.

177. The design of the well system was based on the assumption of a slightly pervious upstream topstratum ($k_v = 2.5 \times 10^{-8}$ cm per sec) 20 ft thick extending to an infinite length, an impervious downstream topstratum, and the 10-ft semipervious sandy silt stratum with $k_h = 1000 \times 10^{-8}$ cm per sec. For these assumptions the effective length of the upstream blanket is 282 ft. With the well spacing used (31.25 ft) the well discharge was computed to be 0.04 gpm and the head midway between the wells 1.3 ft for the project flood. Very little head loss should occur in the well with this low flow. For the purpose of drawing a flow net in the top clay strata, the pressure distribution in the pervious stratum under the structure was assumed to vary as if the upstream topstratum were 282 ft in length and a net head of 1.5 ft existed in the line of wells. The flow net and hydrostatic uplift immediately beneath the base slab are plotted in fig. 63. The plotted hydrostatic uplift at the base of the structure is the maximum net uplift on the base of the structure. No data are yet available on hydrostatic uplift beneath the structure during high water.

178. The relief wells were installed by driving a 10-in. casing, with a plate on the bottom, to the required depth, placing the screen and riser pipe, and then backfilling with sand and gravel as the casing was removed. The plate on the bottom of the temporary casing was removable and stayed in the bottom of the hole when the casing was withdrawn. The filter material around the screen and riser pipe was placed by means of a tremie to prevent segregation of the material. After the filter was placed to the required elevation the well was surged using compressed air and water. The surging was performed by placing an air hose and a water hose in the well between the casing and riser and discharging air and water under pressure so as to cause flow of water through the filter and screen and up through the riser. Because of the low permeability of the sandy silt stratum in which the screens are

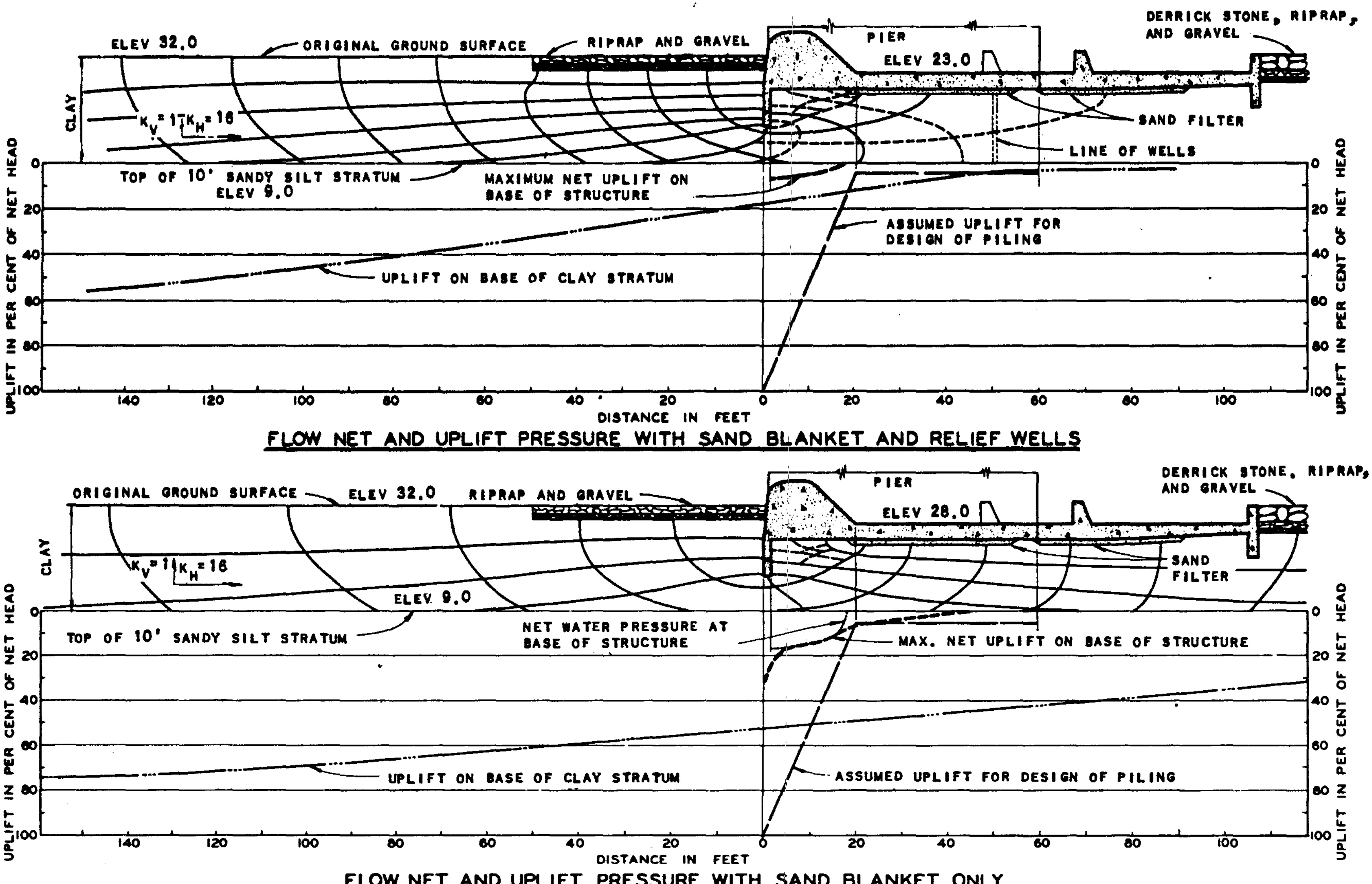


Fig. 63. Flow nets and uplift pressure beneath control structure

installed, it was impossible to obtain any flow of water from the wells. An attempt was made to pump a few of the wells with a pitcher pump, but no water could be produced. The relief wells were installed as construction of the structure progressed between 15 November 1951 and 20 December 1952.

179. The decision to drive the casings for the relief wells with a plate on the bottom was made by field engineers. The hole should have been augered or fishtailed down to prevent smear along the outer periphery of the hole through the sandy silt stratum to be drained. Although very little flow was ever expected from the relief wells, the method used for advancing the hole for the wells may have significantly impaired their efficiency.

180. The only high water yet recorded against the structure occurred on 19 May 1953 when a heavy rain filled the approach to the structure up to elev 34.8, thereby creating a net head of 2.5 ft on the wells. At that time the forebay was not connected with the Mississippi River and water stood against the structure for a shortt ime only. No flow was observed from the wells nor was any expected for such conditions. (The levee between the Mississippi River and the control structure was still intact as of April 1954.)

Piezometers

181. The performance of the drainage blanket and relief wells beneath the structure will be checked by piezometers, the locations of which are shown on figs. 39 (page 83) and 65. Five "A" piezometers, to measure the hydrostatic pressure midway between wells in the sandy silt stratum, were placed along the line of wells on about 800-ft centers at stations 684+51, 692+01, 699+51, 707+01, and 714+51. Three deep "C" piezometers with screens in the deep, underlying sand were installed at piers 13, 63, and 109. The screens for the "C" piezometers were of brass and the riser pipe was of Carlon B plastic pipe. The risers for these piezometers extend up through the piers and are capped and referenced as are the other piezometers. The "A" piezometers were installed by

driving a 10-in. casing with a loose plate on the bottom, installing the piezometer point, filter and riser, and then withdrawing the casing.

182. Five "B" piezometers were placed in the sand blanket midway between collector pipes 1 and 2 at the same stations as listed above for the "A" piezometers. These piezometers are illustrated in fig. 62.

183. The screens for piezometers "A" and "B" were 12-in. lengths of 1-in.-ID coarse Norton porous tube. The tubes from the screen to the riser pipe consisted of 3/8-in.-ID Saran plastic tubing with 1/16-in. wall thickness. The tubes from the "A" piezometers to the riser pipe were placed on a slope of about 1 on 10 to allow any air in the piezometers to escape. Riser pipes for the "A" piezometers consisted of 1/2-in. galvanized pipe, and risers for the "B" piezometers consisted of 1-in. galvanized pipe which was brought up in a pier at the downstream curb of the highway.

184. None of the piezometers were observed while water was against the control structure. Piezometer readings obtained in January-April 1954 are tabulated in fig. 39, and are plotted on fig. 64 together with

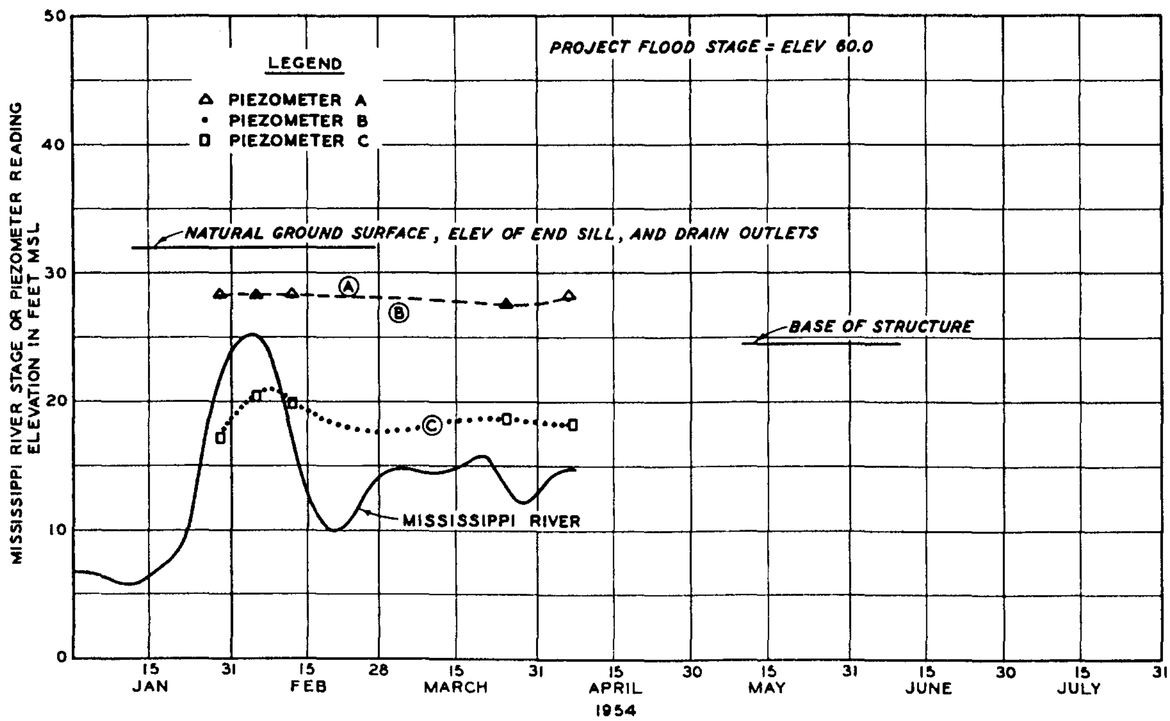


Fig. 64. Water table beneath control structure

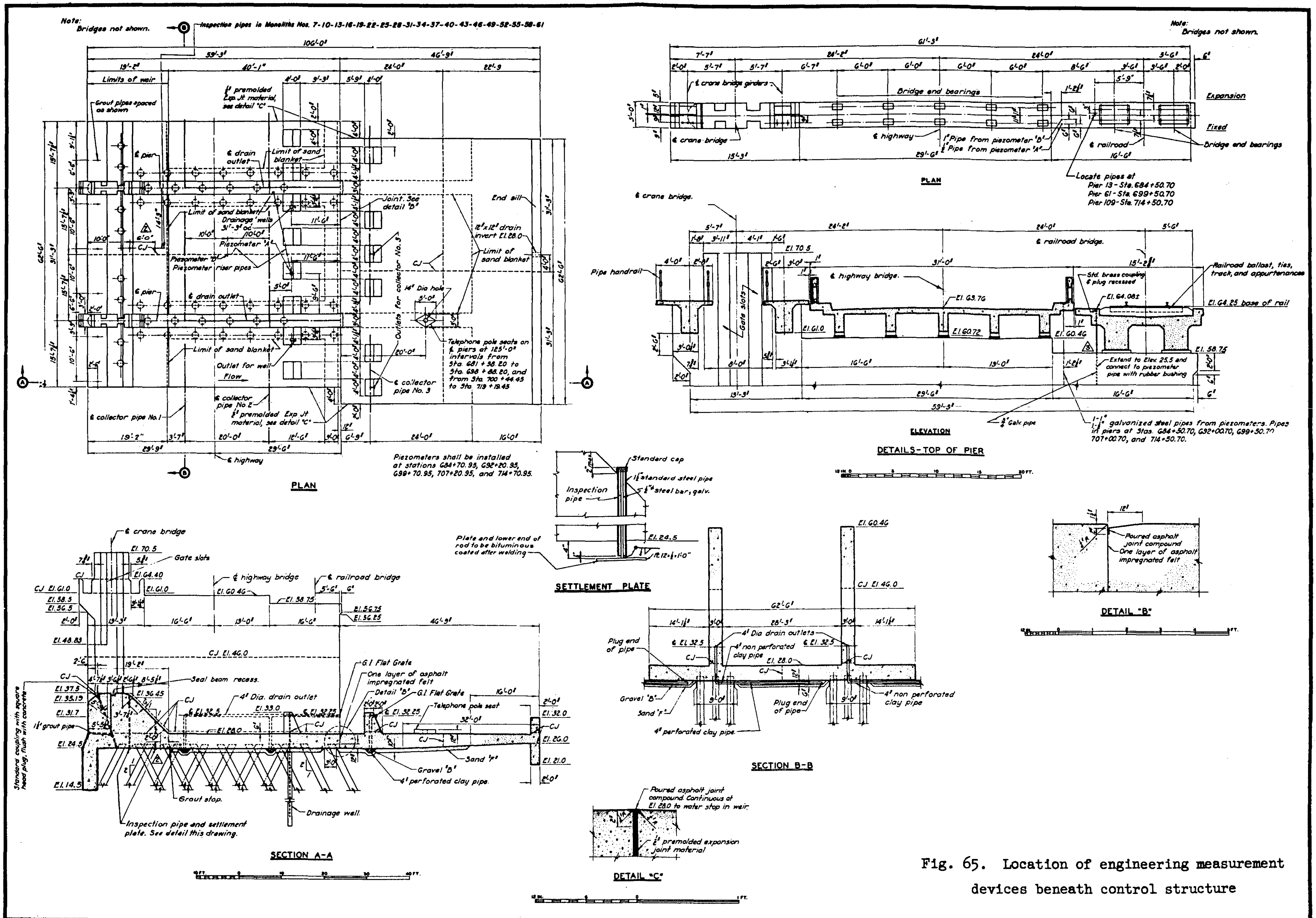


Fig. 65. Location of engineering measurement devices beneath control structure

the stage of the Mississippi River opposite the structure. These readings only reflect the elevation of the water table in the sandy silt stratum and deep sands beneath the control structure on the dates of observation. From the data presently available it appears that the water table in the silt stratum beneath the structure is not affected by river stages below the top bank of the river, whereas the hydrostatic pressure in the deep sands equals approximately the stage in the Mississippi River opposite the structure.

Voids beneath Structure

185. During design of the control structure some concern was expressed regarding the possibility that the soil beneath the base slab of the structure might shrink or settle away from the structure and thus allow an open path for seepage. It appears on the basis of piezometer readings that the bottom of the base slab is at approximately the lowest summer ground-water level observed (see figs. 17, 18, and 62). In view of this and the added protection against drying out afforded by the presence of the structure, there appears to be no basis for believing that the soil could shrink away from the base of the structure through drying of the material beneath it. The possibility of the clay foundation settling, and thereby leaving an open space beneath the base of the structure, also was examined. The pile driving operations in this type material were not expected to cause very much, if any, subsequent consolidation of the clay under its own weight, because the clay at the site is not sensitive and does not become more compressible when disturbed. Also, the clay beneath the structure is overconsolidated to a considerable depth. For these reasons, there seemed to be no need for concern that the clay would consolidate and leave the bottom of the base slab. No construction is planned adjacent to either the upstream or downstream sides of the structure that could cause consolidation of the clay under

the structure. However, as a safety precaution against scour upstream of the structure and the possibility of piping beneath the structure, a concrete cutoff wall extending to a depth of 17.5 ft beneath the top of the riprap was constructed. In addition, grout pipes were installed in the weir extending down to a point immediately beneath the base of the structure (see fig. 65).

186. Two lines of inspection holes 6 ft on centers were provided in the weir downstream from the gates to permit periodic inspection for settlement of the foundation away from the base of the concrete structure. The inspection holes also will be used to check the effectiveness of grouting any voids that might develop beneath the base of the structure. The upstream line of holes is located 10 ft downstream from the face of the weir. Each pair of inspection holes is on 187.5-ft centers between station 680+90 and station 718+40. Details of these inspection holes are shown on fig. 65. The holes were formed in the concrete with 1-1/2-in.-ID black steel pipe. Settlement plates were installed in the foundation material 4 in. below the concrete directly under the inspection holes. These plates are 12 in. square, of 1/2-in. bituminous-coated steel plate, with a 1/2-in.-diameter galvanized iron rod welded in the center of each plate. The iron rods extend up through the 1-1/2-in. pipe and permit the taking of levels for determination of settlement.

187. A check on these settlement rods in January 1954 showed approximately 0.5 to 1.4 in. settlement of the earth immediately beneath the base of the structure (see figs. 39 and 42, pages 83 and 88). The average settlement was 0.8 in. Settlement of the earth beneath the control structure as plotted on fig. 42 is the average settlement of two settlement plates 6 ft apart and 4 in. below the bottom of the base slab. This settlement includes foundation settlement caused by weight of the 4-ft base slab before concrete set in the monolith, settlement caused by weight of base slabs of adjacent monoliths subsequently poured, any reconsolidation of the foundation under its own weight as a result of remolding caused by pile driving, and possibly a certain portion of structure settlement after the concrete in the base slab had set up around the pile heads.

188. As the settlement plates beneath the base slab were placed before readings were started on the hubs installed in the base, the plates beneath the base of necessity would have to move down an amount equal to <u>at least</u> the settlement of the concrete base. Therefore, it is believed that the points on the dotted line, "Settlement of Earth beneath Control Structure," on fig. 42 above the line labeled "Settlement of Base of Control Structure" are probably in error. A possible explanation for such error may lie in the fact that only second-order leveling was used in surveying these plates, whereas precise leveling was used in determining the settlement of the base and top of the control structure. If it is assumed that the points on the dotted line above the dashed line on fig. 42 are at least as low as the dashed line, the average settlement of earth beneath the control structure then would be approximately 0.9 in.

189. No readings were taken on the settlement plates beneath the structure immediately after pouring the base slabs, and therefore the amount of instantaneous settlement of the earth beneath the wet concrete before it set is not known. An attempt was made to estimate the instantaneous settlement of the 4-ft-thick slab of fluid concrete from laboratory test data. As no consolidation tests were made specifically for this purpose, the estimate of instantaneous settlement of the wet concrete was made in the following manner:

<u>a</u>. The clay foundation was divided into various strata; the existing overburden pressure, p_1 , computed at the center of each stratum; the increase of pressure created by the fluid concrete on these strata computed; and samples representative of the strata on which consolidation tests had been made were selected.

<u>b</u>. Plots of $\dfrac{\text{Instantaneous strain}}{\text{Increase in consolidation pressure}}$ vs consolidation pressure were made for each sample tested.

<u>c</u>. Values of $\dfrac{\text{Instantaneous strain}}{\text{Increase in consolidation pressure}}$ were taken from these curves for p_1's corresponding to the different strata.

<u>d</u>. Values obtained in <u>c</u> were then multiplied by the increase in pressure, Δp , created by the wet concrete on the various strata and the thickness of the corresponding strata.

e. The sum of the values obtained in d was taken as the maximum possible instantaneous settlement of the wet concrete.

190. The maximum instantaneous settlement estimated by the above procedure was approximately 0.65 in. Thus, as much as 0.65 in. instantaneous settlement plus 0.55 in. settlement of base after hardening of concrete or 1.2 in. of settlement of the plates beneath the base slab can possibly be attributed to settlement of the wet concrete before hardening and downward movement of the pile heads during and after completion of the structure. The average settlement of the plates beneath the base slab is 0.9 in. with a maximum of about 1.4 in. Thus, it is believed that there has been relatively little if any settlement of the clay away from the base of the structure as a result of consolidation of the underlying clays to a lower void ratio because of remolding. However, as discussed below, there is evidence of some small voids beneath the base slab, which probably can be attributed to settling of the underlying clay.

191. In February 1954, the grout pipes originally installed in the weir of the control structure (see fig. 65) were filled with water, and the rate of fall in the pipes was observed. The purpose of these tests was to determine the presence of any voids beneath the base of the structure. The results of the tests are presented in table 20.

192. With the grout pipes full of water, a head of approximately 9 ft was created beneath the base slab. As any voids beneath the structure are filled with water, since the bottom of the base is below the water table, it is impossible to estimate the size or extent of any voids beneath the structure. Although the rate of fall of water in the grout pipes was relatively slow except in 4 or 5 pipes, the size of voids still could be appreciable because of the impermeable nature of the underlying clay and the fact that there is no outlet for any free water under the structure between the key and the grout stop immediately upstream of the sand blanket (see fig. 65). In view of the fact that 20 to 30 of the grout pipes did take water at a significant rate, it is believed that any voids under the structure, although probably small

Table 20

Measurement of Leakage through Grout Holes in Weir

February 1954

Hole No.	Initial Depth to Water in Grout Pipe in Feet	Readings After Filling Grout Pipe: Elapsed Time in Hours	Fall of Water in Feet	Elapsed Time in Hours	Fall of Water in Feet	Remarks
3-B	1.8	----	---	24.67	1.3	
9-C	-	----	---	24.00	4.8	Plugged at 8.0 ft
13-A	-	1.33	0.0	24.02	0.5	Plugged at 8.9 ft
13-C	-	1.25	0.2	23.95	0.9	Plugged at 11.3 ft
16-B	8.8	1.20	0.8	23.88	5.2	
17-B	8.6	1.13	1.1	23.82	6.7	
18-B	8.8	1.07	0.0	23.75	0.8	
20-A	0.4	1.00	0.0	23.68	0.1	Plugged at 10.7 ft
22-A	9.2	0.83	9.0	23.53	9.3	
23-C	8.5	0.68	0.8	23.38	1.3	
24-C	8.7	0.62	7.4	23.38	8.8	
26-A	7.8	0.47	0.2	23.38	1.3	
28-A	8.2	0.40	8.0	23.38	8.9	
31-B	8.8	0.25	0.4	23.38	5.6	
33-A	9.1	1.75	1.7	2.80	3.4	
34-C	7.7	1.68	9.1	2.80	9.1	
35-B	9.2	1.62	2.0	2.80	3.5	
37-A	8.8	1.55	7.6	2.87	8.3	
38-B	9.2	1.48	9.2	2.87	9.2	
39-C	8.8	1.42	8.2	2.87	8.3	
40-C	3.3	1.38	0.0	2.87	0.0	
41-C	7.9	1.37	7.0	2.85	8.1	Plugged at 11.0 ft
42-C	9.1	1.35	9.2	2.85	9.3	
43-B	9.1	1.32	8.0	2.85	8.7	
44-B	0.9	1.30	0.0	2.85	0.0	Plugged at 9.7 ft
45-B	8.8	1.27	0.2	2.83	0.4	
46-B	9.2	1.25	2.1	2.83	5.3	Plugged at 10.1 ft
47-B	11.5	1.20	~~0.0~~	~~2.85~~	~~0.0~~	~~Plugged at 11.0 ft~~
48-B	9.7	1.18	9.6	2.83	9.6	
49-B	8.5	1.12	0.1	2.85	0.3	Plugged at 12.0 ft
51-B	8.3	1.07	2.3	2.85	5.3	
53-B	8.9	1.03	0.0	2.85	0.0	Plugged at 11.0 ft
55-B	7.7	1.00	0.1	2.85	0.5	
57-B	7.6	0.97	0.4	2.85	0.7	
59-B	8.8	0.93	0.0	2.85	0.1	Plugged at 10.9 ft
61-B	9.4	0.90	0.1	2.85	8.6	
63-A	7.8	0.97	0.4	2.85	1.2	
65-A	8.5	0.85	7.9	2.85	8.9	
67-B	8.7	0.80	0.9	2.85	3.3	
69-A	7.5	0.77	0.0	2.85	0.2	Plugged at 11.1 ft
71-C	7.8	0.73	0.4	2.83	1.2	
73-B	7.6	0.70	1.9	2.83	6.3	
75-B	-	----	---	----	---	Plugged at 0.3 ft. Dry
77-B	8.6	0.67	0.5	2.83	2.3	
79-B	8.5	0.55	6.5	2.83	8.5	
81-C	8.2	0.53	7.4	2.83	8.3	
83-A	7.2	0.50	0.1	2.83	0.4	Plugged at 10.5 ft
85-C	7.3	0.48	4.3	2.83	7.0	
87-C	---	----	---	----	---	Plugged w/sand at 0.7 ft. Dry
89-B	8.0	0.43	0.0	2.95	0.0	Plugged at 11.0 ft
91-A	8.5	0.25	7.9	2.95	8.7	
93-C	10.3	0.23	0.3	2.95	1.1	
95-B	7.6	----	----	----	---	
97-A	9.3	0.37	8.3	2.77	9.5	
99-A	9.1	0.35	6.8	2.75	9.2	
101-B	7.8	1.85	3.1	23.98	6.7	
103-C	7.8	1.83	6.2	23.98	8.2	
105-A	7.2	1.78	0.0	24.03	0.5	
106-C	7.4	1.77	0.4	---	---	Plugged at 9.0 ft
108-B	8.6	1.73	0.0	24.07	0.1	
110-B	9.2	1.70	0.7	24.07	4.3	
112-B	7.8	1.68	0.0	24.08	0.1	Plugged at 11.0 ft
114-C	8.0	1.65	0.6	24.10	6.3	Plugged at 13.0 ft
116-B	8.9	1.63	1.2	24.12	7.6	
118-A	4.4	1.63	0.0	24.12	0.1	Plugged at 6.0 ft
120-A	7.6	1.62	0.0	24.13	1.1	Plugged at 7.6 ft
122-A	8.1	1.58	2.3	24.15	8.4	
124-A	9.4	1.57	0.5	24.15	3.7	

NOTES:

1. Inside diameter of grout holes = 1.61 in.
2. Number of grout holes refer to bay numbers beginning at the south end of the control structure. Designation of grout pipe "A", "B", and "C" refers to the three holes in each bay, the "A" holes being the southernmost pipe.
3. Location of the grout holes in the weir is shown on figure 62.

and relatively few, should be grouted with a thin grout before any major high water occurs. The grouting should be deferred as long as possible in order to allow the remolded clay beneath the structure to consolidate fully.

PART IX: STONE PROTECTION

193. The area for a distance 30 ft upstream of the weir is protected by riprap 18 to 36 in. thick, underlain by a gravel blanket consisting of 5 in. of coarse concrete aggregate covered with 3 in. of large gravel. The specified gradation of the gravel materials is shown on fig. 61, page 132, together with the gradation of the gravel actually placed. The riprap consisted of stones grading from not more than 10 per cent weighing less than 200 lb to 60 per cent or more weighing 500 lb to 2400 lb.

194. The area downstream of the stilling basin sill was protected for a distance of 80 ft with derrick stones 42 in. thick underlain by 18 in. of riprap which in turn rested on 3 in. of very coarse gravel underlain by 5 in. of coarse concrete aggregate.

195. The riprap and derrick stone protection for the approach and floodway were placed between 24 May 1952 and 1 April 1953. As yet there has been no flow through the structure to test the adequacy of the channel protection.

PART X: SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

196. The Morganza Floodway Control Structure and adjoining embankments are relatively new and as of April 1954 they have not been subjected to an appreciable head of water. Therefore, it is not possible to draw any final conclusions regarding the performance of the control structure. However, data obtained so far and analyses presented in this report are indicative, and can be used to draw certain tentative conclusions.

197. Data obtained from piezometers, settlement plates, and hubs installed on and beneath the Morganza Control Structure and adjoining embankments have proved useful not only in verifying original design assumptions and analyses, but also by revealing certain behavior characteristics of the control structure which were not wholly anticipated in the original soils and structural design. Careful observations and the keeping of adequate engineering records during construction are most important in future evaluations of the performance and safety of hydraulic structures and in the interpretation of data obtained from engineering measurement devices.

198. The field exploration methods used for exploring the foundation for the control structure and adjoining embankments apparently proved successful. Penetration resistance data obtained from use of the split-spoon sampler proved satisfactory for determining the top of the deep underlying sand. However, it appears that the elevation of the top of sand was revealed equally as well by frequent sampling in either general sample or undisturbed borings.

199. Data obtained from pile loading tests and from the performance of the earth embankments indicated that the laboratory shear and consolidation tests revealed the true characteristics of the clays beneath the control structure and embankments. It has not been possible to check predicted coefficients of permeability for the clay and silt strata encountered at the site from field observations. As no field water-content and density samples were obtained from the embankment, no conclusions can be drawn regarding densities and shear strengths of the embankment predicted from laboratory tests.

200. The satisfactory performance of the embankments adjoining the control structure, as regards their stability, indicates that the design strengths, methods of analyses, and factors of safety selected were satisfactory. As no failures occurred either during or since construction, it is of course impossible to know whether or not the embankment sections were overdesigned.

201. The phase method of constructing the embankments proved satisfactory from both the traffic and stability standpoints. It was assumed in design that no water content control would be exercised over the fill material, and that it would be placed and compacted only by means of a crawler-type tractor; this method of placement and compaction proved satisfactory for the embankments as designed.

202. The water table and pore pressure piezometers installed beneath the embankments appear to be functioning satisfactorily. However, to insure their proper functioning it was necessary to flush the pore-pressure type of piezometers periodically with de-aired water and to check the gages and the reading of each tube leading to the piezometer tip. Some trouble was also encountered with freezing of the ends of the piezometer tubes and gages in the terminal boxes. Provision should be made to prevent such freezing.

203. The water table adjacent to the embankments varies seasonally from the ground surface to a depth of approximately 6 ft. Seasonal variation of the water table beyond the embankment is attributed to desiccation rather than to draining action. Analysis of the pore-pressure observations and water-table measurements indicates that the water table beneath the embankment probably varies relatively little with the season and also probably remains relatively close to the natural ground surface.

204. Placement of embankment fill created a rapid increase in pore pressure in the underlying fat clay. During the period of most rapid fill placement, average maximum foundation pore pressures amounted to approximately 40 to 60 per cent of the pressure of the overlying fill; a maximum pore pressure of 85 per cent of fill pressure was recorded by one piezometer. Pore-pressure observations showed that the foundation

beneath the embankments was about 70 to 80 per cent consolidated just before starting construction of phase IV. These data agreed closely with the percentages of consolidation as determined from observed settlements.

205. Although some rather high pore pressures were observed during construction, the embankments proved stable both during and after construction. A stability analysis of the embankment at station 630, utilizing the cross section and pore pressures as observed in July 1951 and the slow shear strength of the foundation, showed the embankment at that time to have a factor of safety of 1.7 using this method of analysis.

206. Considering the uncertain construction schedules and the magnitude of foundation settlements, very good agreement was obtained between estimated and observed amounts and rates of settlement. However, the accuracy of the estimated rate of settlement can largely be attributed to advance settlement observations made on adjacent, previously constructed embankments. On the basis of observed settlement observations, it appears that the embankments will reach a total settlement of approximately 4 to 5 ft.

207. Although the control structure has yet to be subjected to a full water load, the pile foundation has been subjected to the full dead load of the structure and appears to be performing satisfactorily. Therefore, it is concluded that the 20-in.-diameter piles selected on the basis of the pile loading tests and driven as described in this report will safely support the structure.

208. The skin friction developed by friction piles in the clay at the control structure site is nearly the same in tension as in compression, and is approximately equal to the average strength of the clay as indicated by unconfined compression and unconsolidated, undrained triaxial compression tests on undisturbed samples of the clay. In this connection, it is pointed out that approximately the same unconfined compression shear strength was obtained for remolded samples as for undisturbed samples of this particular clay.

209. The pile driving formulas referred to as method A, Terzaghi and Peck, and method D, Jaky, are believed to be the most reasonable

theoretical methods of those considered for use in estimating the capacity of a pile tip driven into sand. These methods are rather empirical and should not replace load tests. This is borne out by the scattered results and lack of complete agreement of the pile test data with the capacities indicated by the theoretical methods. In general, capacities of piles computed by the various dynamic pile driving formulas investigated did not correspond to either the ultimate total test load or ultimate tip load, or the design total or tip loads.

210. Except at the ends of the control structure, settlement of the base of the structure has been very uniform—less than 0.1 in. difference between monoliths—and has amounted to approximately 0.5 in. It is believed that little if any movement of the pile tips has occurred as a result of sinking into sand, or of consolidation of any clay seams or strata underlying the pile tips. The overdriving, which was done along certain reaches of the structure, may have prevented any appreciable settlement of the structure except at the ends of the structure. Approximately 1/3 of the 0.5-in. settlement of the base of the structure can be attributed to elastic deformation of the pile foundation. Settlement data obtained to date indicate that practically no settlement of the top of the control structure has or is likely to occur, except for that portion of the structure south of pier 2 and north of pier 117 where these piers are underlain by thin strata of clay and the earth embankment ties into the structure. So far settlements of the piers at the ends of the structure are within the originally estimated amount of movement.

211. As there has been no appreciable water load against the structure, no conclusions can yet be drawn as to whether or not the clay foundation upstream of the structure will settle as a result of a water load. The settlement of the steel plates placed beneath the riprap upstream of the structure has amounted to approximately 1/2 to 1-1/2 in. Some of this settlement can possibly be attributed to consolidation of the clays from the weight of the gravel blanket and riprap; however, movement of these settlement plates has been somewhat erratic.

212. In view of the large variations in the top of the foundation

sand which underlies the control structure, it is believed that the predicted average elevations agreed in a satisfactory manner with the observed average elevations. The average observed top of sand, as indicated by the pile driving records, agreed within 2 ft with the average top of sand which had been predicted on the basis of field explorations. The originally estimated variation in the top of sand at any one pier was ± 4 ft, except at piers 98 through 110 where a variation of ± 6 ft was estimated. The average observed variation in top of sand as revealed by the pile driving records showed variations of approximately ± 3 ft and ± 5 ft, respectively, for these piers. The test piles penetrated an average of 4.3 ft into the sand. An allowance of 5 ft vertically for penetration into sand was used for computing the length of pile to be tested. In actuality, the piles in the nonoverdrive reaches penetrated an average of 3.4 ft into the sand. The penetrations ranged from 0.8 to 25.0 ft.

213. The average optimum allowance for penetration into plus variation in top of sand at the various piers, as computed from observed data, was 6.3 ft. This value is somewhat less than the original design allowance of 10 ft. A cost analysis of the pile driving data showed that it was more economical to cast the piles 3 to 5 ft too long than to cast them 1 ft too short and splice where necessary. The data presented in this report show that the very considerable variation in the top of sand and the penetration into sand at any one pier made it practically impossible to determine precisely the correct length of piles required at any individual pier or groups of piers in the area of the Morganza Floodway. As the original estimates were intended to be conservative and to keep the number of splices to a minimum, the methods of analysis used in forecasting the required length of piles in the nonoverdrive reaches are considered quite accurate and resulted in pile lengths of nearly the most economical length with a minimum of splicing.

214. Approximately a third of the piles in the overdrive reaches are believed to have penetrated through compressible strata in the upper part of the massive sand foundation. Field explorations and the pile driving data show that the compressible strata are usually discontinuous and intermittent and, if this is true, the overdriving may have caused

the majority of the piles to penetrate through most of the lenses and strata of compressible material in the sand that otherwise would have been below the pile tips.

215. Overdriving the piles at Morganza did relatively little damage to the pile heads. Some of the piles were driven as hard as 200 blows per 0.1 ft for 0.3 ft, and others as hard as 400 blows for the last 0.1 ft without material damage.

216. Driving the piles on a batter of 2 on 1 created no particular problem. The flatness of this batter, however, did cause some drifting of the piles during driving and deflection of the pile tip when it struck sand. Measurements on three of the piles revealed drifts amounting to 0.3 to 1.9 ft with maximum bows in the pile of 0.3 to 0.5 ft. This amount of bow in the piles created compression stresses in the concrete pile of approximately 1300 psi, and tension stresses in the steel reinforcing of approximately 19,000 psi. With the load of the structure on the piles, the compression stresses in the concrete remained approximately the same, whereas the tension stresses in the steel were reduced to approximately 2500 psi. It was concluded from the observations and tests made that a batter of 2 on 1 is the flattest that should be used for similar foundation conditions and driving.

217. Preloading the abutments for the control structure resulted in a settlement of approximately 2.5 ft which otherwise would have taken place after completion of the structure and placing of the abutment fills. Preloading the abutments reduced the tendency of the fill to pull away from the curtain wall, largely eliminated the possibility of cracks and voids opening up with the resulting danger of piping, and reduced the amount of deflection that might be caused in the batter piles because of settlement of soil under the embankment load. Preloading the abutment has also reduced the problem of maintaining a transition from the non-settling structure to the settling embankment.

218. As yet no conclusions can be drawn regarding the functioning of the relief well system installed in the silt stratum beneath the structure, nor regarding the functioning of the drainage blanket installed beneath the structure. It appears that in general most of the

piezometers are functioning satisfactorily.

219. Piezometer readings obtained to date only reflect the elevation of the water table in the silt stratum and deep sands beneath the control structure. The data presently available indicate that the water table in the silt stratum beneath the structure is not affected by river stages below the top bank of the river, whereas the hydrostatic pressure in the deep sands equals approximately the stage of the Mississippi River opposite the structure.

220. A check on settlement plates installed immediately beneath the base of the structure in January 1954 showed that the earth immediately beneath the structure had settled approximately 0.5 to 1.4 in. The average settlement was approximately 0.9 in. Most of this settlement can be attributed to instantaneous settlement of the clay foundation while the concrete was fluid, and to settlement of the base after hardening of the concrete. It is believed relatively little if any settlement of the clay away from the base of the structure has occurred as the result of the underlying clays consolidating to a lower void ratio because of remolding. However, there is some evidence of small voids beneath the base slab which can probably be attributed to settling of the underlying clay. Any voids under the structure, although probably small and relatively few, should be grouted before any major high water.

221. As of this date (1954), there has been no flow through the structure to test the adequacy of the channel protection and therefore no conclusions regarding it can be drawn at this time.

222. It is recommended that continued observations be made at appropriate time intervals on all of the engineering measurement devices that are operable. Particularly complete observations should be made during periods of high water which may have a pronounced effect both on settlement of and pore pressures in the embankment, and on well flow and substratum pressures beneath the control structure. It is recommended that such data be analysed and reported in the form of appendices to this report.

References

1. Corps of Engineers, Waterways Experiment Station, "Combined Morganza Floodway Control Structure, Texas & Pacific Railroad, and Louisiana State Highway No. 30 - Soils Investigation." Technical Memorandum 3-278, Vicksburg, Mississippi, May 1949.

2. Corps of Engineers, Mississippi River Commission, "Morganza Floodway Control Structure - Analysis of Design for Gated Portion." Vicksburg, Mississippi, November 1950.

3. Corps of Engineers, New Orleans District, "Morganza Control Structure - Pile Tests." New Orleans, Louisiana, December 1949.

4. Corps of Engineers, Waterways Experiment Station, "Pile Loading Tests - Combined Morganza Floodway Control Structure." Technical Memorandum 3-308, Vicksburg, Mississippi, January 1950.

5. Corps of Engineers, Waterways Experiment Station, "Determination of Required Pile Lengths - Combined Morganza Floodway Control Structure." Technical Memorandum 3-317, Vicksburg, Mississippi, August 1950.

6. Corps of Engineers, Waterways Experiment Station, "Morganza Control Structure - Review of Use of Long and Heavily Loaded Piles." Vicksburg, Mississippi, October 1950.

7. Specifications and Contract Drawings for:

   (a) Pile Loading Tests for Morganza Control Structure by Hired Labor, New Orleans District, dated 25 March 1949.

   (b) Semi-compacted Embankment for Morganza Control Structure (Phase I), New Orleans District, dated 26 February 1949.

   (c) Excavation and Pre-cast Concrete Piling for Gated Portion of Morganza Control Structure, New Orleans District, dated 27 March 1950.

   (d) Construction of Superstructure of the Gated Portion of the Morganza Control Structure, New Orleans District, dated 26 May 1950.

   (e) Riprap and Derrick Stone Protection for the Gated Portion of the Morganza Control Structure, New Orleans District, dated February 1951.

   (f) Embankment (Phase II) Plan, Profile, Borrow Areas, Borings, and Typical Sections, New Orleans District, dated May 1951.

   (g) Embankment (Phase IV and V) Plan, Borings, Borrow Areas and Profiles, New Orleans District, dated August 1952.