TABLE III

Actual Settlement Observations on Exhibits Buildings at the Texas Centennial Central Exposition

Building	Type of Column	Maximum	Minimum	Average
4 and 5	Interior	0.624"	0.528"	0.585"
"	Wall	0.828"	0.432"	0.613"
"	Corner*	0.600"	0.600"	0.600"
43	Interior	0.624"	0.588"	0.606"
43	Wall	0.792"	0.276"	0.475"
43	Corner	0.960"+	0.372"	0.588"

* Only one column.

\+ This column is at a corner where an "L" projects from the structure. Therefore, the structure covers 270° around the column instead of 90° as in the case of the usual corner.

Since part of the voids in this clay are filled with air we can expect a much greater rate of settlement during the earlier loading periods because the air can move through the soil at a greater speed than water. Curves in Fig. 7 and 8 show this to be true. It is also interesting to note that after the consolidation test was complete there were still some voids in the soil filled with air although water was available through the lower porous disc, during the entire test and under a head of approximately 3.3cm.

The following conclusions may be drawn from these tests:

1. The field loading test on this clay gave estimated settlements lower than the actual observed settlements.
2. Pressure distributions under the footings should be calculated before estimating the amount of settlement.
3. The estimated settlement calculated from the laboratory consolidation curve and distributed pressures, checks very closely to the observed settlements. However, one may well question whether or not future observations will check as well.
4. We cannot consider that the voids in clay are entirely filled with water in all cases.
5. Although water was available during the entire consolidation and rebound test, the clay did not take up sufficient moisture to fill the voids.

No. F-4

ABSTRACT

SETTLEMENT RECORDS OF THE MISSISSIPPI RIVER BRIDGE AT NEW ORLEANS

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Prior to the erection of the Mississippi River Bridge above New Orleans two series of borings were made. The first borings were made in 1926 and used for the tentative selection of foundation types, construction methods and landing elevations. The second series was conducted in 1933 just before construction was begun. The soil stratification as determined by the borings, the location of the borings, and the profile of the bridge and the main piers are shown in Fig. 1. Undisturbed samples were obtained by means of a sampling tube illustrated in Fig. 2. Other borings than those shown were made for a distance of several thousand feet along the approaches to the bridge, but this paper confines itself to a study of the main piers.

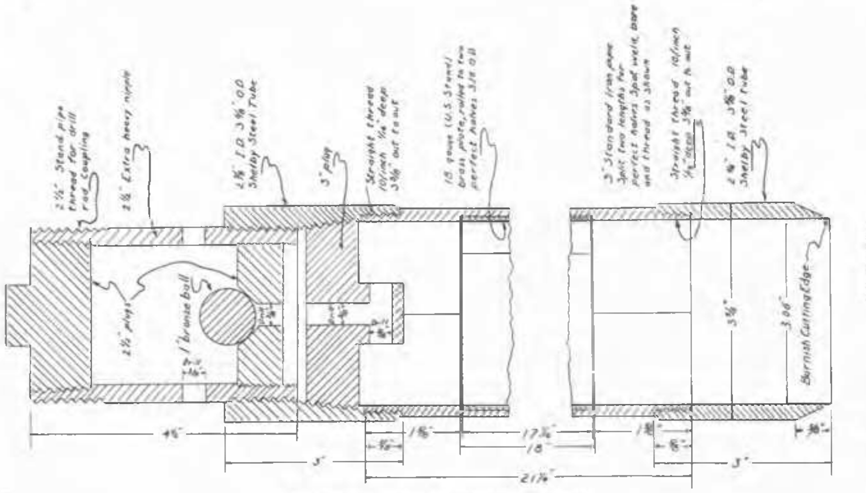
Using the undisturbed samples obtained in the 1933 boring program estimates were made of the settlements of the nine main piers. The results of the estimates were forwarded to the engineers in charge of the work three months before the construction of the piers was begun.

Observations of settlement were taken on all the piers from the time of sealing the caissons. These observations are being continued and the records for the first six months of 1936 will be appended to this paper as they become available. A comparison between the estimated and observed settlements is shown in Table II.

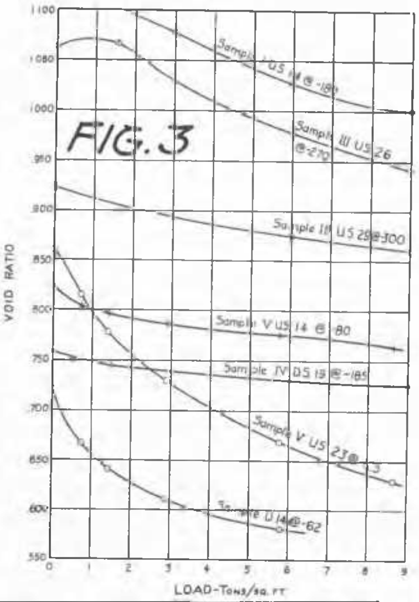
The settlement estimates led to two practical results:

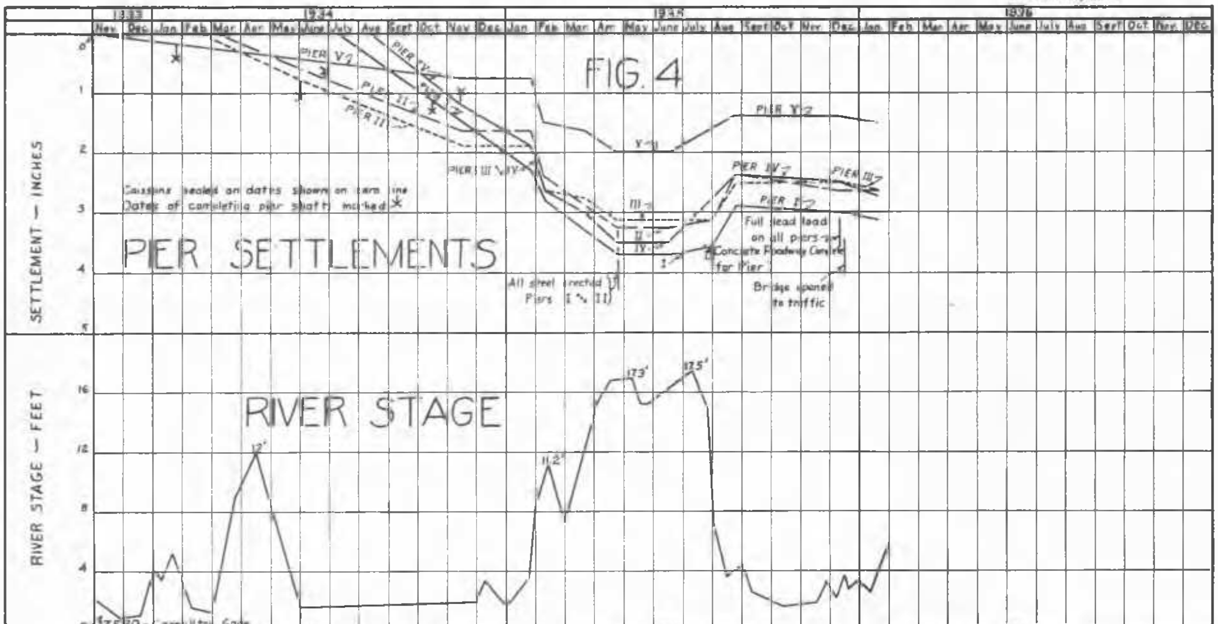
1. The truss bearing plates were redesigned to allow jacking the trusses back to position and the bridge seats were finished at higher elevations than called for on the plans;
2. It was agreed to land the caisson for Pier A 15 feet above the plan elevation. It was thought that this would effect a saving of over $10,000, several times the cost of the actual settlement analysis. This caisson over-ran the new proposed landing elevation during construction, however, and was finally brought to rest at the plan elevation.

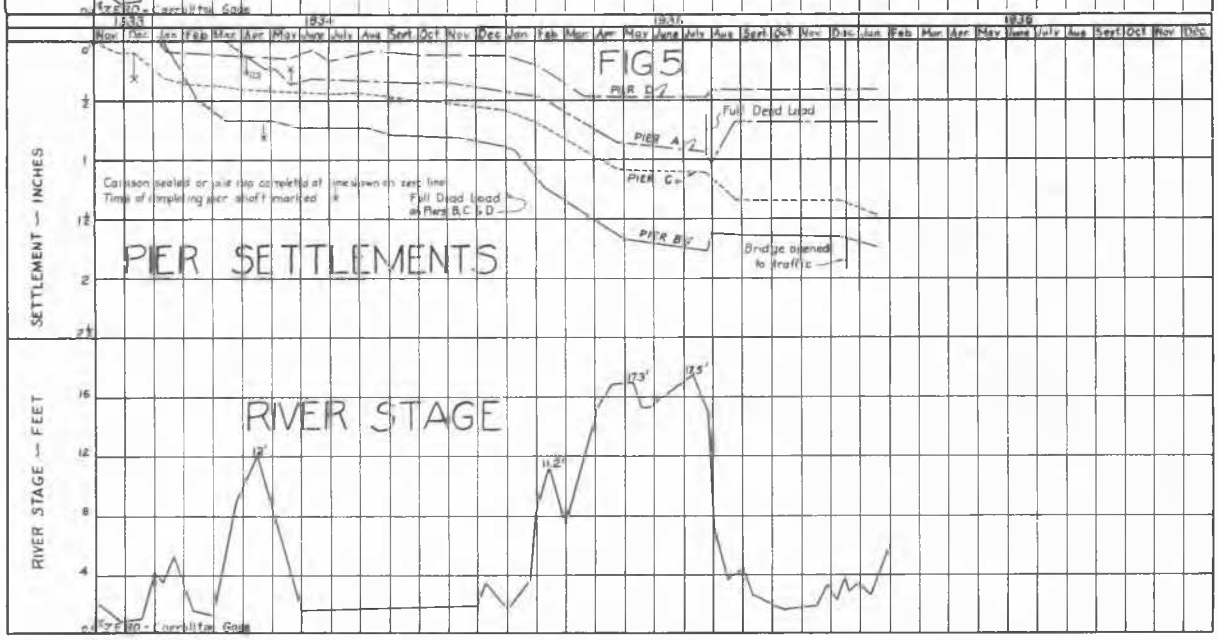
Adaptability of Conditions to Soil Studies. As shown in Fig. 1 the soil stratification lends itself rather readily to consolidation analysis. The various strata are reasonably homogeneous throughout their respective depths, and obvious drainage course are provided for the consolidation of the clay



SECTIONAL VIEW OF SAMPLING TUBE
FIG. 2







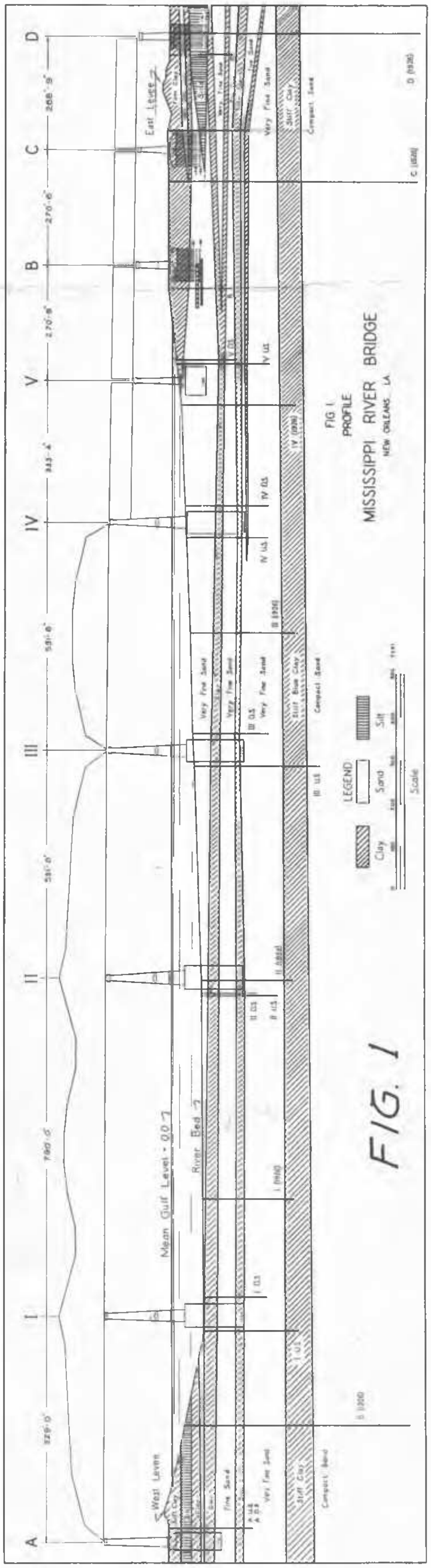


FIG. 1

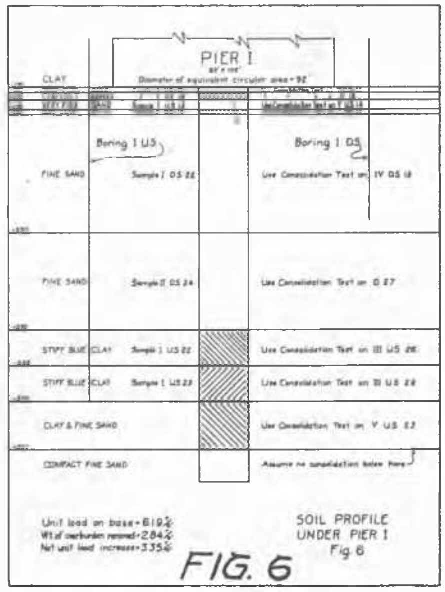


FIG. 6

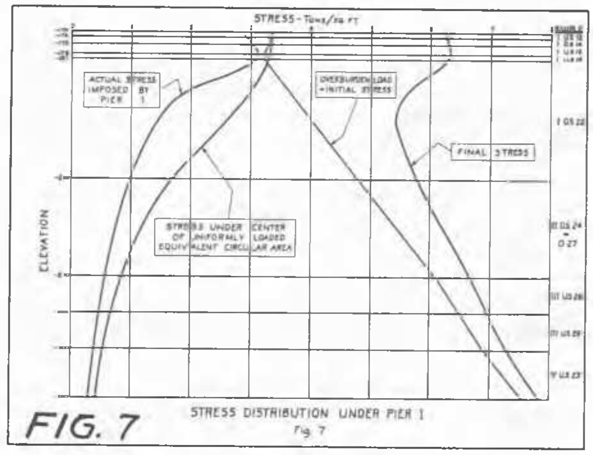


FIG. 7

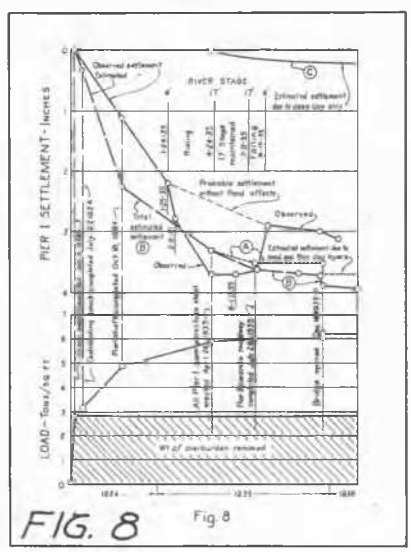


FIG. 8

strata. Reliable undisturbed samples were obtained.

Difficulties were introduced into the analysis by the hit-or-miss occurrence of compressible clay lenses near the landing elevations of the main river piers and by the choppy nature of the deposit underlying the east shore piers B, C and D. Further uncertainties were introduced by the fact that the piles under these piers were not driven to the depths shown on the plans, by the indeterminate amount of load which may be taken by the skin friction of the deep caissons, and by the sand island method of construction which would tend to cause preconsolidation of the soil.

Sampling and Testing. Of the four hundred core samples shipped to the laboratory ninety were of a size suitable for undisturbed tests. All samples were classified as "sand", "silt" or "clay" according to the "proposed" classification scale shown in Fig. 1 of a paper "Soil Mechanics as a Practical Science" presented by the author to this conference. Samples containing appreciable admixtures of sand, silt or clay were classified as sandy, silty or clayey. Undisturbed samples were tested for moisture content, Atterberg limits, specific weight, specific gravity of grains and some for grain distribution by sieves and hydrometer. Representative results are shown in Table I. Twenty consolidation tests were made, and the characteristics thus found were applied to all soils shown by the routine tests to be similar. This procedure is also illustrated in Table I. Consolidation curves of samples used in the settlement analysis of Pier I are shown in Fig. 3.

Three facts regarding the samples obtained are worthy of note.

1. Satisfactory undisturbed samples of clean fine sand were obtained;
2. Silt varves lying between capillary clays lost all their moisture, apparently to the adjacent clay.
3. Disturbance due to wall friction inside the sampling tube was apparent within a distance of about 1/8 inch from the edge.

Methods of Analysis. The detailed method of analysis is described in Appendix A for Pier I. The other piers were analyzed according to the same methods.

Settlement Observations. Settlement observations of the main river piers, I to V inclusive, are shown in Fig. 4; of the main shore piers, A to D inclusive, in Fig. 5. Early observations of the shore pier settlements are not guaranteed. Observations of the river piers are considered to be reliable. The agreement between these observations and the estimates is thought to be reasonable. The order of magnitude of the estimates is correct.

Evaluation of the Settlement Estimates. Skin friction of the deep caisson for Pier A is suggested in explanation of the appreciable discrepancy between estimated and observed settlements for this pier. The discrepancies noted in the settlements of Piers III and IV may possibly be attributed to uncertainties relative to the occurrence of the compressible clay lenses mentioned above, and in the case of Pier IV to a large blow-in occurring during construction. The relative settlements of Piers B, C and D agree with the estimates, but the total settlements observed are considerably less than those estimated.

Effect of Floods. A very interesting and novel occurrence is illustrated in Fig. 4 and 5 showing the behavior of the main piers during the extended flood period of 1935. Rapid settlements of as much as one and one-half inches occurred as the river rose to flood stage. These settlements were offset by corresponding rebounds as the flood stage subsided five months later. This action appears as a sag below the normal consolidation curves for the piers. Check observations indicate that the bench marks were unaffected by the flood. Bed rock lies over 2000 feet below the surface at this location, and the author suggests the possibility of elastic compression of the soil caused by the weight of fifteen feet of water. The author, however, has not found any satisfactory explanation of the occurrence, and calls on the members of the conference for an explanation.

Acknowledgments. The author acknowledges the cooperation of the engineering firms of Moran and Proctor and of Modjeski, Masters and Case in providing the data which form the basis for the paper, and of the Belt Line Railway Company for settlement observations of the piers made subsequent to the opening of the bridge.

Appendix A. The soil profile underlying Pier I was determined by the borings to be as shown in Fig. 6.

In computing the overburden load, shown in Fig. 7, the submerged weight of the soil was taken. The stress distribution at the base of the caisson seals was assumed to be of the form typical of rigid footings, increasing from a minimum of 50% of the average at the center to a practical maximum of 200% of the average at the edges. The stress was assumed to be distributed through the underlying soil regardless of its nature according to Boussinesq's equation using a factor, n, of 3. By a laborious series of computations not shown in the paper relationships were established between the readily determined "center stresses under uniformly loaded equivalent circular areas" and the "average stresses under the actual piers". The curves for both these sets of values, appropriately labeled, and for the final stresses existing under the completed structure are shown in Fig. 7. The total ultimate settlements are computed by the usual manner, using these stresses and corresponding void ratios from the consolidation curves in Fig. 3, as shown in Table III. Time studies of the strata between -172 and -175 and between -179 and -181 indicated that 95% consolidation would occur within one-half and two days, respectively. The settlements due to these strata were therefore combined with the settlements due to the sand strata and were considered to occur immediately upon application of load. The rate of consolidation of the deposit between -270 and -320 was calculated in accordance with the method outlined in Appendix A of the author's paper, "Soil Mechanics as a Practical Science". It was found that less than 3/8 inch settlement would occur due to this deposit within two years after application of load, and it was therefore thought unnecessary to consider the construction schedule. Consolidation was assumed to begin at the completion of the superstructure steel erection. Curve C, Fig. 8 shows the estimated settlement due to this deep-lying deposit; Curve A shows the estimated settlement due to the clay lenses and the sand strata; and Curve B shows the total estimated settlement of the structure obtained by adding Curves A and C. Fig. 8 also shows the observed settlements of the pier, the occurrence of the 1935 flood stage and the load-time schedule.

TABLE I

Description	Elevation	Moisture Content	Liquid Limit	Plastic Limit	Shrinkage Limit
		Boring A U.S.			
Medium Sand	-125.0	18.3	0	0	0
Stiff Clay	-155.0	27.6	61.0	22.3	20.4
Very Fine Sand	-190.0	26.9	0	0	0
		Boring A D.S.			
Fine Grey Sand[a]	-115	22.2	0	0	0
Silty Clay	-155	26.1	44	22.2	19.5
Fine Sand[a]	-189.9	20.8	0	0	0
		Boring I U.S.			
Very Fine Sand	-170.0	30.3	0	0	0
Clay--Silt Varves	-170.0	32.5	48.5	22.5	30.1
Stiff Clay	-175.0	41.6	67.9	31.6	25.6
Blue Clay[c]	-180.0	41.5	68.5	31.7	23.5
Med.Sand,Clay & Stones[a]	-271.0	22.4	0	0	0
Blue Clay[b]	-283.0	37.4	75.0	28.2	19.5
Blue Clay[b]	-299.0	39.0	77.7	30.4	16.7
		Boring I D.S.			
Clay--Silt Varves	-170.0	31.0	39.0	21.7	23.3
Blue Clay	-175.0	35.0	60.0	28.5	17.7
Fine Sand[a]	-224.5	20.3	0	0	0
		Boring II U.S.			
Clayey Silt	-160.0	27.7	36.5	23.6	22.9
Very Fine Sand	-175.0	27.5	0	0	0
Silty Clay[d]	-175.5	38.0	44.7	22.0	19.2
Very Fine Sand[a]	-185.0	23.6	0	0	0
Fine Sand	-199.0	24.2	0	0	0
		Boring II D.S.			
Fine Sand	-175	25.0	0	0	0
Blue Clay[c]	-185	43.3	73.5	35.0	13.4
Blue Clay	-188	38	70.5	31.0	14.7
Fine Sand	-249	17.2	0	0	0
		Boring III U.S.			
Very Fine Sand	-170.0	27.7	0	0	0
Clay & Veg. Mat.	-170.0	38.8	67.3	36.1	
Very Fine Sand[a]	-185.0	24.5	0	0	0
Stiff Blue Clay	-270.0	40.7	74.7	33.7	14.8
Stiff Blue Clay	-300.0	38.4	79.2	32.8	15.3
Clayey Silt	-320.0+	22.0	31.3	18.4	18.9
Very Silty Sand	-320.0-	17.3+	0	0	17.4
Fine Sand	-339.5	21.0	0	0	0
Fine Sand	-349.5	19.0	0	0	0

TABLE I (cont.)

Description	Elevation	Moisture Content	Liquid Limit	Plastic Limit	Shrinkage Limit
		Boring III D.S.			
Very Fine Sand[a]	-169.5	24.3	0	0	0
Stiff Clay	-170	48.2	79.2	36.7	12.5
Clay--Silt Varves[d]	-175	33.8	49.0	21.5	19.3
Very Fine Sand[a]	-220	23.0	0	0	0
Fine Sand[a]	-229	21.2	0	0	0
		Boring IV U.S.			
Very Fine Sand[a]	-170	24.4	0	0	0
Stiff Clay	-175	39.6	56.0	31.1	18.0
Fine Sand	-205.5	20.7	0	0	0
Blue Clay[c]	-206	41.7	69.0	27.7	17.0
Very Fine Sand[a]	-229	24.8	0	0	0
		Boring IV D.S.			
Very Fine Sand	-170	30.7	0	0	0
Silty Clay	-175	27.3	65.5	30.6	19.4
Very Fine Sand[a]	-175	23.0	0	0	0
Very Fine Sand	-185	23.2	0	0	0
		Boring V U.S.			
Very Fine Sand	-80	28.5	0	0	0
Clay & Fine Sand	-125	27.5	26.4	15.7	17.0
Gray Sandy Silt	-155	26.0	30.6	25.4	23.2
Blue Clay	-195	40.5	70.0	29.6	17.0
		Boring V D.S.			
Very Fine Sand	-79	27.6	0	0	0
Med. Sand Shells	-145	18.1	0	0	0
		Boring C			
Silty Clay	+3	35.4	44.4	23.2	20.8
V.F. Sand with trace of Clay	-25	27.5	0	0	0
Silty Clay[d]	-75	37.4	42.8	23.6	22.3
Sandy Clay	-90	27.8	38.1	23.4	21.6
Blue Clay	-104	35.4	70.0	27.0	15.7
Slightly Sandy Silt	-145	19.4	22.4	17.8	15.0
Very Fine Sand with trace of Clay[a]	-170	23.7	24.0	0	0
Stiff Clay	-170.4	48.0	80.0	35.0	
Blue Clay Sand Pockets[d]	-180	30.0	42.8	21.9	20.9
		Boring D			
Silty Clay	-5	37.2	40.4	23.5	25.2
Clayey Silt	-30	34.7	39.2	24.4	21.3
Sandy Silt	-62	30.6	32.8	22.2	24.1
Very Fine Sand	-80	30.1	0	0	0
Fine Sand	-138	19.0	0	0	0

Note: a: Consolidation data on Sample IV D.S. 19 at -185 assumed for this material;
b: Consolidation data on Sample III U.S. 29 at -300 assumed for this material;
c: Consolidation data on Sample V U.S. 37 at -195 assumed for this material;
d: Consolidation data on Sample C 2 at +3 assumed for this material.

TABLE II

TIME		PIER SETTLEMENTS, INCHES								
		A	I	II	III	IV	V	B	C	D
Seal or footing completed	Estimated	0	0	0	0	0	0	0	0	0
	Observed	0	0	0	0	0	0	0	-	-
Distributing block completed	Estimated	1/2	1/4	1/4	0	0	0	-	-	-
	Observed	0	1/4	1/4	1/4	3/8*	-	-	-	-
Pier shaft completed	Estimated	1 1/4	2 1/4	2	3/4	1/2	0	3/4	1/4	1/4
	Observed	1/2	1 1/8	3/4	3/4	1 1/8	3/16	3/4	1/2	1/8
Full dead load applied	Estimated	2 1/4	3 1/4	3 7/8	1 1/4	1 1/2	3/4	4 1/2	3	1
	Observed	1	3 1/2	2 1/2	2 1/2	2 1/2	3/8	1 3/4	1 3/8	3/8
Bridge opened	Estimated	2 1/2	3 7/8	4	1 3/8	1 5/8	1 1/8	4 5/8	3	2
	Observed	3/4	3	2 5/8	2 1/2	2 1/2	1 3/8	1 3/4	1 1/2	3/8
June 1,1936	Estimated	2 1/2	3 7/8	4 1/8	1 1/2	1 5/8	1 1/8	4 5/8	3 1/2	2 1/4
	Observed	1	3 3/8	[illegible]		3	[illegible]		[illegible]	
1940	Estimated	3 1/8	4 1/4	4 5/8	1 3/4	1 3/4	1 1/4	4 3/4	4 1/2	4
	Observed									
1945	Estimated	3 1/4	4 1/2	4 7/8	1 7/8	1 3/4	1 1/4	4 3/4	4 1/2	4 1/2
	Observed									
1950	Estimated	3 1/4	4 3/4	5	1 7/8	1 7/8	1 1/4	4 3/4	4 1/2	4 1/2
	Observed									
Ultimate	Estimated	3 1/4	5 3/8	5 5/8	2 1/4	2 1/8	1 1/4	4 7/8	4 3/4	4 3/4
	Observed									

* Large blow in during construction.

TABLE III

STRATUM FROM ELEVATION	TO	AVERAGE INITIAL STRESS	AVERAGE FINAL STRESS	AVERAGE INITIAL VOID RATIO e_i	AVERAGE FINAL VOID RATIO e_f	ULTIMATE SETTLEMENT* N.T.# (Inches)	T.# (Inches)
-170	-172	2.87	6.20	0.785	0.773	0.16	---
-172	-175	2.96	6.30	0.610	0.578	0.70	---
-175	-179	3.07	6.36	0.784	0.772	0.32	---
-179	-181	3.16	6.31	1.076	1.032	0.51	---
-181	-230	4.00	5.77	0.735	0.730	1.69	---
-230	-270	5.45	6.18	0.5115	0.5105	0.32	---
-270	-285	6.30	6.83	0.973	0.967	---	0.55
-285	-300	6.70	7.13	0.870	0.866	---	0.39
-300	-320	7.20	7.56	0.646	0.641	---	0.73
Below	-320	---	---	---	---	0+	0+

* Computed from the formula: $S = 12\,h \frac{e_i - e_f}{1 + e_i}$, where h is the thickness of the stratum in feet.

Settlements which will occur immediately are shown in the column marked N.T. Settlements which will occur progressively are shown in the column marked T.

+ Assumed.