FOUNDATIONS OF MISSISSIPPI RIVER BRIDGE AT BATON ROUGE, LOUISIANA

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SYNOPSIS

This report discusses principally the foundation explorations and the use of the principles of soil mechanics in the design of the piers for the Baton Rouge Bridge

Two series of borings were made. In 1934, 2-in cores were taken to depths of 200 ft and in 1936, 3½-in. cores were taken to depths of 300 ft. From these borings a profile of the material underlying the bridge site was made and samples secured for laboratory study.

From the consolidation tests on samples and the estimated loads to be imposed by the piers estimates of expected settlements were made which were later found to be in substantial agreement with the observed settlements thus far noted

The information secured from the undisturbed samples indicated that the soil underlying the pier bases is capable of supporting the loads without excessive settlement, and that uniform settlement might be expected

It is concluded that application of the principles of soil mechanics to deep foundation problems makes it possible to determine in advance the magnitude of the settlement to be expected and to provide for this settlement, to anticipate with some degree of accuracy the amount of settlement to be expected during construction stages and to further anticipate progressive settlement over a period of years. Further, it is possible, after anticipated settlements have been determined to adjust the sizes of the piers, or base areas, so as to get piers of approximately equal settlement.

Loading of test piles also indicated the possibility of the application of the principles of soil mechanics to the design of pile foundations, although in this case the need for continued research is pointed out

A bridge crossing the Mississippi River at Baton Rouge, Louisiana had been under discussion for many years prior to the construction of the structure completed in August of this year. Preliminary plans had been prepared as far back as 1914, but financing was impossible until 1936 when the Project was taken over for construction by the Louisiana Highway Commission. Final plans for the bridge were prepared in 1936 by the Bridge Department of the Louisiana Highway Commission, and in so far as foundations were concerned this organization had available a great deal of data on deep pier construction in the general vicinity of the proposed construction. In 1923 and 1924 the L. & N. Railroad Company, in constructing their bridges across the Rigolets and Chef Menteur Passes east of the City of New Orleans, used open caissons of circular design going to depths of approximately 130 ft. below low water. These piers were founded on hard clays and packed sands, being sunk through 70 to 100 ft. of sands and clays of various densities. These bridge piers were the first deep open dredge type caissons to be built in this region of the lower Mississippi, and the results obtained were such as to prove conclusively that the deep underlying sand and clay formations are capable of supporting bridge pier loadings. In 1927 the Louisiana Highway Commission constructed its bridges over these same Passes, using the same type caissons landed at depths from 100 to 137 ft. below mean Gulf level, and these structures have stood without appreciable settlements. Since that time other bridges have been built, using open dredge caissons, over the Atchafalaya River where the formation is much like that found in the Mississippi. Piers in this stream were started in 1930 and were sunk as deep as 178 ft. mean Gulf level. These piers have also stood without appreciable settlements. Later, from 1933 to 1935 deep bridge piers were constructed for the Mississippi River Bridge at New Orleans, caissons being sunk to elevation −170.

It was with the information available from all of this construction that the design of piers for the Baton Rouge Bridge proceeded.

It is the purpose of this paper to describe the method of attack used in arriving at the final foundation plans, and the results up to the present time.

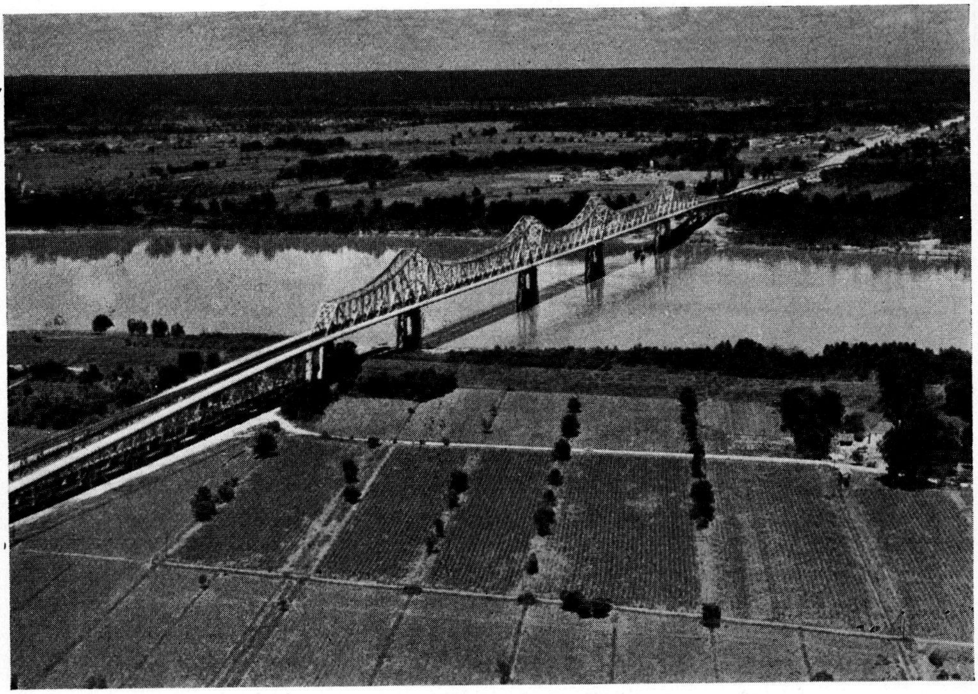

Figure 1. The Baton Rouge Bridge

DESCRIPTION OF THE STRUCTURE

The length of the steel structure is 2.31 miles divided as follows:

East Approach Viaduct
- Railroad—3587 ft.
- Highway—935 ft.

Main River Crossing—3326 ft.

West Approach Viaduct
- Railroad—5298 ft.
- Highway—1719 ft.

The approach structures are steel viaducts supported on precast concrete pile foundations.

The main river crossing consists of a cantilever type structure with the following arrangement of spans:

Span No. 1—490 ft.—Shore Anchor Span.

Span No. 2—848 ft.—Cantilever and Suspended Span.

Span No. 3—650 ft.—Anchor Span.

Span No. 4—848 ft.—Cantilever and Suspended Span.

Span No. 5—490 ft.—Shore Anchor Span.

The structure carries a single railway track and two driveways, 20 ft. in width, with one three foot sidewalk on each road-way, these walkways being outside the trusses.

Clearance above low water of 112.4 ft. is provided, the variation between high and low water being 47.4 ft.

The main river structure is supported on six piers, two of which are in the deep water of the river, two at the low water edge, and two (the shore anchor piers) in the high banks. Five of these piers are of the open dredge caisson type, and one, the shore pier on the West river bank, is founded on a timber pile foundation. The caissons for the four main piers are rectangular, having twelve dredging wells, and vary in size from 50 ft. by 80 ft. at Pier No. 5 to 63 ft. by 82 ft. at Piers 2 and 3 These piers are founded at depths of 113 ft. at Pier No 5 and 184 ft. at Pier No. 2 below low water. The caisson for Pier No. 1 is circular in shape, having a diameter of 47.5 ft., and is founded 130 ft below ground.

The total cost of the project including railway and highway approaches and connections was approximately $10,000,000.00.

FOUNDATION STUDIES

Loads considered on the foundations consisted of the following:

1. Original weight of earth and water.
2. Total direct load on base (live load and dead load).
3. Braking.
4. Wind.

Various possible combinations of these loads were studied and results compared with loadings of other structures constructed in this region having similar foundation conditions. (See Table 1)

These combinations produced possible foundation loads as high as 17 tons per sq. ft. However, considering the effect of friction on the sides of the embedded portions of the caissons, and the passive pressures exerted against horizontal faces, it developed that this high figure could be materially reduced. In fact it developed in the case of Pier No. 3 which is the one located in the deepest water and thus having the least embedment, that sufficient passive earth pressure against the sides of the caisson would be developed to care for approximately 50 percent of the applied horizontal forces.

BORINGS

Two series of borings were made prior to construction. The first series made in 1934, being cores two inches in diameter taken to depths as great as 200 ft., was used in the preliminary studies of foundation types and landing elevations. The second series, of $3\frac{1}{2}$-in. diameter cores taken to depths of approximately 300 ft., was made in 1936, and from these borings undisturbed samples were taken for laboratory testing. These cores were taken with a 3-ft. sampling tube having a cutting edge slightly smaller in diameter than the barrel so as to cause minimum disturbance of the sample during driving. Cores were taken from the core barrel by first removing the cutting edge and then applying hydraulic pressure. Samples were immediately sealed in glass containers and delivered to the Louisiana Highway Commission laboratory for damp room storage and testing. Satisfactory undisturbed samples of the clay and sand formations were obtained. A soil profile of the river section was prepared from the boring data (See Figure 2) and from this profile it will be noted that although the soil formations underlying the bridge site are quite irregular in shape, the borings show the various strata to be fairly homogeneous throughout their depths and the stratification is such that it lends itself to consolidation analysis.

LABORATORY STUDIES AND HOW USED

As a check on foundation determinations made from observation of material underlying the foundations of the structure and comparison with loadings on similar foundations, it was decided to make consolidation tests and compute for estimated settlements. This was done to

TABLE 1

Computed Pressures on Foundations under Piers 1 to 5 for Various Conditions of Loading

Pressures are given in tons per square foot

	A	B	C	D	E	F	G	H	I	J	K	L	M
Pier No	Original Weight of earth and water on foundation in tons per sq ft earth at 120 lb per cu. ft	Total direct load on base L L. + D L.	B-A	Braking	45 deg wind transverse	45 deg. wind longitudinal	30 per-cent Wind + wind on train	C + D	C + D E + F	C + D + G	Equivalent pressure on base from friction on caisson at 600 lb. per sq. ft.	I − K 1 25	J − K 1 25
Case 1—Bouyancy allowance for earth at 120 lb. per cu. ft. and for water													
1	7 5	9 8	2 3	0	0	0	3 6	2 3	2 3	5 9	−2 3		2 9
2	10 9	13 7	2 8	2 8	1 2	2 0	2 5	5 6	8 8	8 1	−2 5	5 0	4 5
3	7 5	11 9	4 4	2 8	1 1	2 0	2 7	7 2	10 3	9 9	−1 5	7 0	6 7
4	9 7	13 7	4 0	0	1 4	0	3 4	4 0	5 4	7 4	−2 5	2 3	3 9
5	7 8	11 7	3 9	3 3	1 2	2 1	2 6	7 2	10 5	9 8	−1 7	7 0	6 5
Case 2—Same as Case 1 except bouyancy allowance for earth at 100 lb per cu. ft.													
1	6 3	9 8	3 5	0	0	0	3 6	3 5		7 1	−2 3		3 9
2	9 1	13 7	4 6	2 8	1 2	2 0	2 5	7 4	10 6	9 9	−2 5	6 5	5 9
3	6 6	11 9	5 3	2 8	1 1	2 0	2 7	8 1	11 2	10 8	−1 5	7 7	7 4
4	8 3	13 7	5 4	0	1 4	0	3 4	5 4	6 8	8 8	2 5	3 5	5 0
5	6 5	11 7	5 2	3 3	1 2	2 1	2 6	8 5	11 8	11 1	−1 7	8 1	7 5
Case 3—Same as for Case 1 except bouyancy allowance for water only													
1	2 5	9 8	7 3	0	0	0	3 6	7 3	7 3	10 9	−2 3		6 9
2	5 6	13 7	8 1	2 8	1 2	2 0	2 5	10 9	14 1	13 4	−2 5	9 3	8 7
3	5 0	11 9	6 9	2 8	1 1	2 0	2 7	9 7	12 8	12 4	−1 5	9 0	8 7
4	5 6	13 7	8 1	0	1 4	0	3 4	8 1	9 5	11 5	−2 5	5 6	7 2
5	3 4	11 7	8 3	3 3	1 2	2 1	2 6	11 6	14 9	14 2	−1 7	10 5	10 0
Case 4—Same as Case 1 except river bed eroded to −90 at Pier 3 and to −70 at Pier 4													
1	7 5	9 8	2 3	0	0	0	3 6	2 3		5 9	−2 3		2 9
2	10 9	13 7	2 8	2 8	1 2	2 0	2 5	5 6	8 8	8 1	−2 5	5 0	4 5
3	7 0	11 9	4 9	2 8	1 1	2 0	2 7	7 7	10 8	10 4	−1 2	7 6	7 3
4	8 8	13 7	4 9	0	1 4	0	3 4	4 9	6 3	8 3	−2 0	3 5	5 0
5	7 8	11 7	3 9	3 3	1 2	2 1	2 6	7 2	10 5	9 8	−1 7	7 0	6 5
Case 5—Superimposed loads only No reduction for bouyancy													
1		9 8		0	0	0	3 6		9 8	13 4	−2 3		8 9
2		11 5		2 8	1 2	2 0	2 5	14 3	17 4	16 8	−2 5	11 9	11 4
3		9 3		2 8	1 1	2 0	2 7	12 1	15 2	14 8	−1 5	10 9	10 6
4		10 9		0	1 4	0	3 4	10 9	12 3	14 3	−2 5	7 8	9 4
5		10 6		3 3	1 2	2 1	2 6	13 9	17 2	16 5	−1 7	12 4	11 8

satisfy the engineers of all of the agencies involved in the construction of this project that satisfactory foundations would be obtained at the predetermined depths

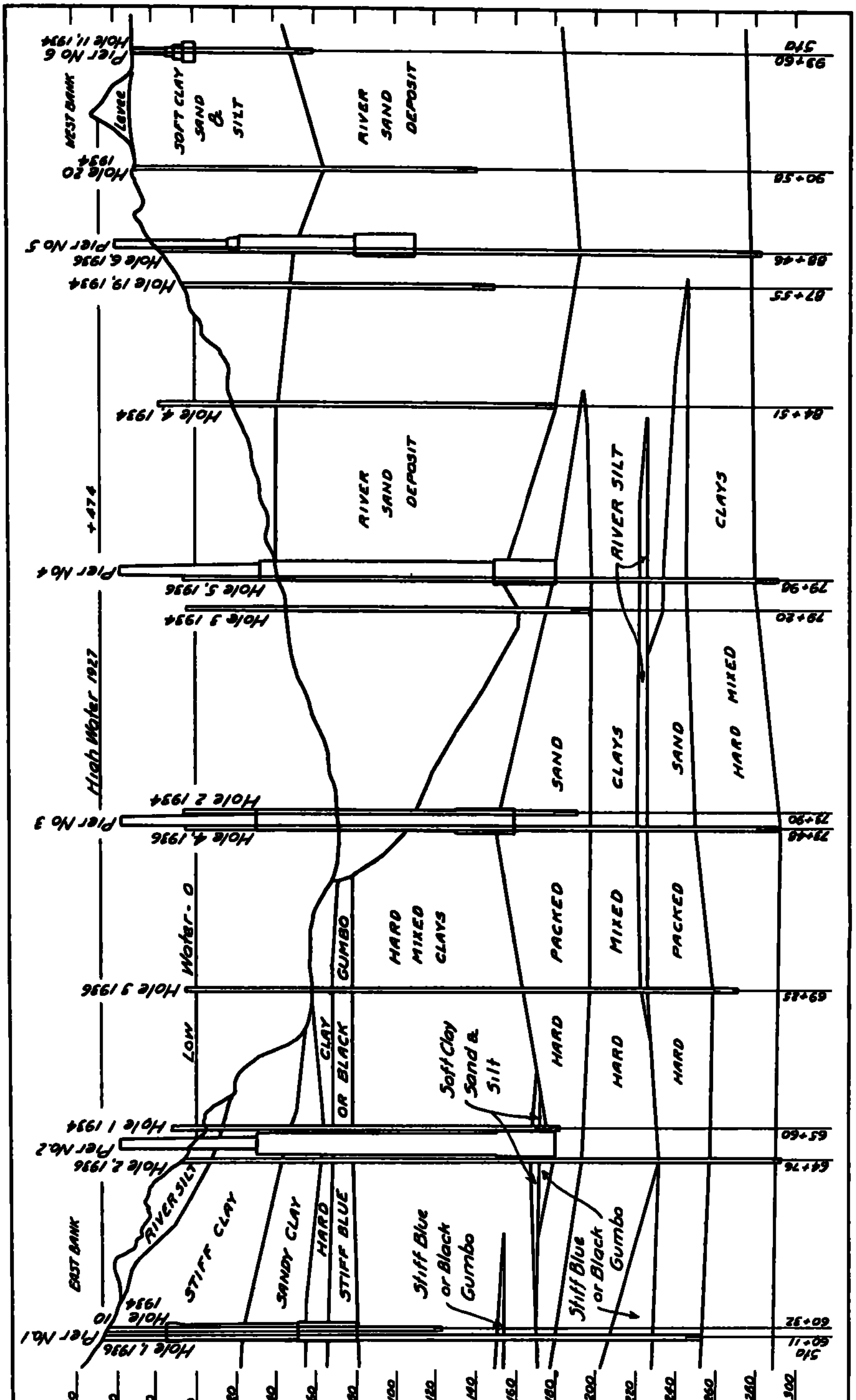

Figure 2. Foundation Borings Baton Rouge Bridge

and that the construction would be safe and feasible.

The Highway Commission laboratory obtained two compression machines of the Harvard type with which consolidation tests were made on 36 samples. The procedure used was that outlined in the paper prepared by Mr. F. A. Robeson of the Bureau of Public Roads, contained in *Public Roads*, Vol. 16, Nov. 1936. Samples were checked for moisture content in the laboratory by specific weight and specific gravity. Representative results are shown in Table 2. Attempts were made to check moisture contents of samples in the field by air drying one cubic inch samples cut from the undisturbed cores

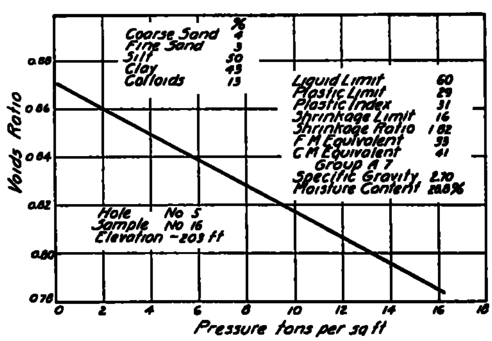

Figure 3. Consolidation Curve

immediately upon removal from the ground, but results of this determination were not consistent nor comparable with laboratory determinations.

The methods used in analyzing the various strata which support the piers were as follows.

Since the borings show the various strata to be fairly homogeneous throughout their depths and that drainage courses (sand strata) are provided for the consolidation of the clays, the formation as previously stated lended itself to consolidation analysis. In computing the overburden loads the weight of the soil was taken at an average of 100 lb per cu. ft. No deduction was made for buoyancy in either the soil overburden or the volume displaced by the pier structure. The weight of the water considered on the foundation was taken to the elevation of low water, mean Gulf level. The pressure was considered as distributed uniformly over the entire area of the footing using trapezoidal pressure distribution; that is, the pressure increasing with increasing depth in the compressible stratum. It was considered necessary to subdivide the footings of the rectangular caissons into units of such size that the load on each would be sufficiently small to be considered a point concentration and for this purpose the foundations were divided into squares of approximately six feet. Total ultimate settlements were computed by the formula:

$$\text{Settlement} = \frac{e1 - e2}{1 + e2} \times D1.$$

Where $e1$ = Average voids ratio prior to loading.

$e2$ = Average voids ratio after loading.

$D1$ = The depth of the strata considered.

Using the stresses and corresponding voids ratio from the consolidation curve (Figure 3) the typical results shown for Pier No. 1 in Table 3 were obtained.

Time consolidation tests were made for each pier location from samples from those strata below the pier bases. Figure 4 contains the results for Pier No. 4, between elevation −179 and −198 and −198 and −224. The time consolidation tests made on samples of the sand stratum on which Piers 2, 3, and 4 are founded show that between 90 and 100 percent consolidation occurs almost immediately and the tests plotted, result in two straight lines, one vertical and one horizontal. The settlements, therefore, due to consolidation of the sand strata, were considered to occur with the application of load. The rate of consolidation of the clay strata was computed in accordance with the method outlined in the

TABLE 2

Physical Characteristics of Samples

Description	Elevation	Moisture Content	Liquid Limit	Plastic Limit	Shrinkage Limit
BORING 1—1936					
		%	%	%	%
Hard Blue Clay	−83	18 7	42	19	12
Hard Blue Clay	−123[a]	21 0	52	26	13
BORING 2—1936					
Grey Sand & Gravel	−183[a]	20 2	22	0	0
Hard Blue Sandy Clay	−203[a]	31 1	81	29	10
Hard Blue Sandy Clay	−213[a]	27 7	43	20	18
BORING 4—1936					
Fine Grey Packed Sand	−163[a]	20 8	0	0	0
Light Blue Clay	−203[a]	20 8	61	29	30
BORING 5—1936					
Very Hard Blue Clay ..	−173	24 6	56	23	13
Grey Packed Sand Some Gravel	−183[a]	19 9	0	0	0
Very Hard Blue Clay	−203[a]	28 8	60	29	16
BORING 6—1936					
Coarse Greyish Brown	−113[a]	16 0	0	0	0
Very Hard Blue Clay	−193[a]	26 8	67	27	14

[a] Indicates samples used for determining consolidation and time settlement.

TABLE 3

Amount of Consolidation

Pier No 1 Base 47 ft 6 in Diameter Gross Vert Load 8 7 Tons per sq ft Elevation Base −80 0

Stratum		Average Initial Stress	Average Final Stress	Average Initial Void Ratio	Average Final Void Ratio	Ultimate Settlement	
From Elev	to					Immediate	Progressive
							ft
−80	−90	6 60	8 88	0 5052	0 4912	—	0 0931
−90	−100	7 10	9 06	0 5022	0 4900	—	0 0812
−100	−110	7 60	9 07	0 4991	0 4901	—	0 0603
−110	−120	8 10	9 13	0 4960	0 4895	—	0 0435
−120	−130	8 60	9 33	0 4928	0 4883	—	0 0301
−130	−140	9 10	9 64	0 4897	0 4865	—	0 0215
−140	−150	9 60	10 00	0 4867	0 4843	—	0 0162
−150	−160	10 10	10 415	0 4838	0 4820	—	0 0121
−160	−165[a]	10 475	10 725	0 4816	0 4802	—	0 0043
Below	−165	—	—	—	—	—	0 0000[b]
Total Ultimate Settlement							0 3627 4⅜ in

[a] Lower Limit of Clay and beginning of Sand Stratum
[b] Assumed

report of the Special Committee on Earths and Foundations of the American Society of Civil Engineers, *Proceedings* A. S. C. E., Vol. 59, No. 5, May, 1933. The results of the information thus obtained are shown in Table 4.

From the information obtained by the analysis of undisturbed samples, it was apparent that the soil underlying the pier bases is competent to support the pier loads without excessive settlements. Further it was shown that such settle-

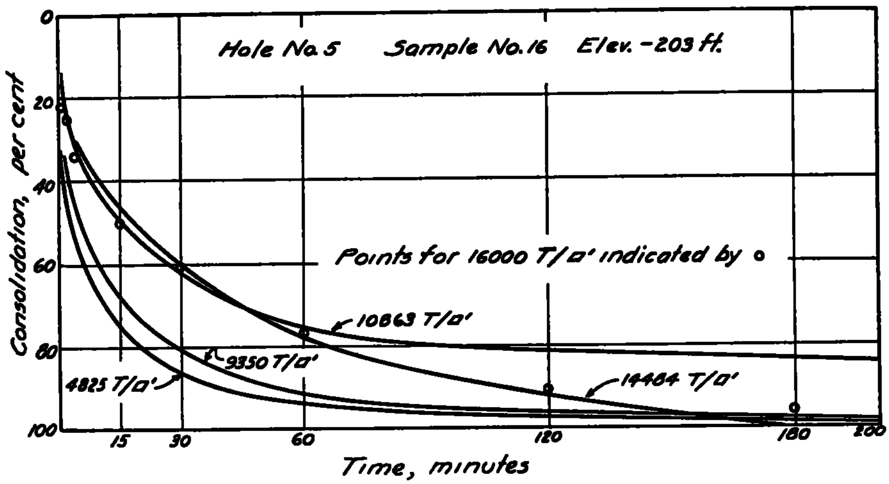

Figure 4. Time-Consolidation Tests for Pier No. 4

TABLE 4

Time	Estimated Pier Settlements in Inches				
	I	II	III	IV	V
Seal or footing completed	0	0	0	0	0
Distributing Block Completed	0	¼	¾	½	½
Pier Shaft Completed	0	⅜	3	¾	¾
Superstructure Completed	¼	1½	5¼[a]	2⅜	1½
Observed Settlement	¼	⅞	1¼	2¼	¾
Jan. 1942	¼	1¾	5¾	2¾	1½
Jan 1943	⅜	2⅜	6⅛	3⅛	1¾
Jan 1944	½	2⅝	6½	3½	2
Jan. 1947	⅝	3⅜	6¾	3⅞	2¼
Jan 1950	¾	4	7	4¼	2⅜
Jan 1955	⅞	4¾	7	4½	2½
Jan 1960	1	5¼	7	4¾	2⅝
Estimated Ultimate (Taken as 80 per cent consolidation of Clay Strata)	3½	6⅛	7	4¾	2¾
Estimated for First 20 Years (1940–1960) after opening to traffic	¾	3¾	2	2⅜	1¼

[a] Sample No 5—Elevation −163, Boring 4–1936 was a disturbed sample and compression test does not run parallel to tests of similar material borings 2 and 5.

ments as might be expected to occur after completion of the structure will be quite uniform.

In comparing the foundations at Baton Rouge with those at New Orleans, test and analysis tended to show the underlying clays at Baton Rouge to be substantially more consolidated than the deep clays at New Orleans, and since no difficulty has been experienced with the New Orleans Bridge due to excessive settlements, the piers at Baton Rouge, having similar loadings were considered to be satisfactory.

CONSTRUCTION METHODS

Pier construction was started in the Summer of 1937. The contract required that work be started on Piers 3 and 4 which are located in the river channel, and it was at the sites of these two piers that operations were begun. The first work consisted of the weaving and sinking of board mattresses 250 ft. by 450 ft., which was necessary for the landing of the steel cylinders containing the sand islands, (these piers were constructed by the so called, "Sand Island Method" which has been described in *Civil Engineering*, July, 1936, Vol. 6, No. 7, "Pier Foundations for the New Orleans Bridge" by N. F. Helmers) and to prevent scour around them.

At the location of this structure ordinary low water stands between 2 and 5 ft. above mean Gulf level, and flood heights have reached an elevation of 47 4 ft. At low stages the water flows at a velocity of 2 to 3 m p.h. which increases at flood stages to 6 to 8 m.p h. The velocities at flood stages are sufficient to cause scour and with a sizeable obstruction in the river channel, such as a sand island with its docks, etc., velocities did increase to an excess of 6 m.p.h. adjacent to the sand islands. This of course, caused considerable scour in the vicinity of the piers, and the mattresses used proved to afford little if any protection during high water. Scour in excess of 40 ft. was found along the side and below Pier No. 4.

The "Sand Islands" used in the river channel were 111 ft. in diameter at Pier No 4 and 121 ft. in diameter at Pier No. 3.

Considerable construction difficulty was experienced by the Contractors, they being faced with the problem of trying to erect sand islands, construct and sink caissons through these islands and well into the river bed in one low water season, say from July to January or February in order that their work might be made safe from flood effects. It was found that a considerably longer time than anticipated was required to make ready the sand islands for the pier construction, and from February, 1938 to July, 1938, Pier No. 4 was rendered inactive on account of the top of the caisson being under water and much of the dock structure having scoured out and washed away.

These two piers though finally completed in very good shape required more than one and one-half years in their building

The design of the caissons provided for jetting facilities to aid in the pier sinking, it being realized by the designers that undoubtedly at some time during the sinking operations difficult sinking would be encountered. Jets were in two classes, the first inside the caisson for use in removing material from the cutting edges, the second flowing outside the caisson to relieve side wall pressure and friction. These jets were employed to good advantage in a number of instances to straighten out a lean or aid in sinking. It was found, however, that care had to be exercised in the use of the outside jets to avoid the occurrence of dangerous "blow-ins" whereby material would flow from outside the caisson under the cutting edge and up into the dredging wells. One such occurrence at Pier No. 4 caused the contractor to practically lose the cofferdam, water rising above the river level and bursting joints designed to take compression but not tension, there having been no provisions taken to equalize the pressure.

After caissons had been sunk to their final elevations great care was taken to clean out material lodged under the caisson walls and then to clean out all loose sands and clays from the foundation.

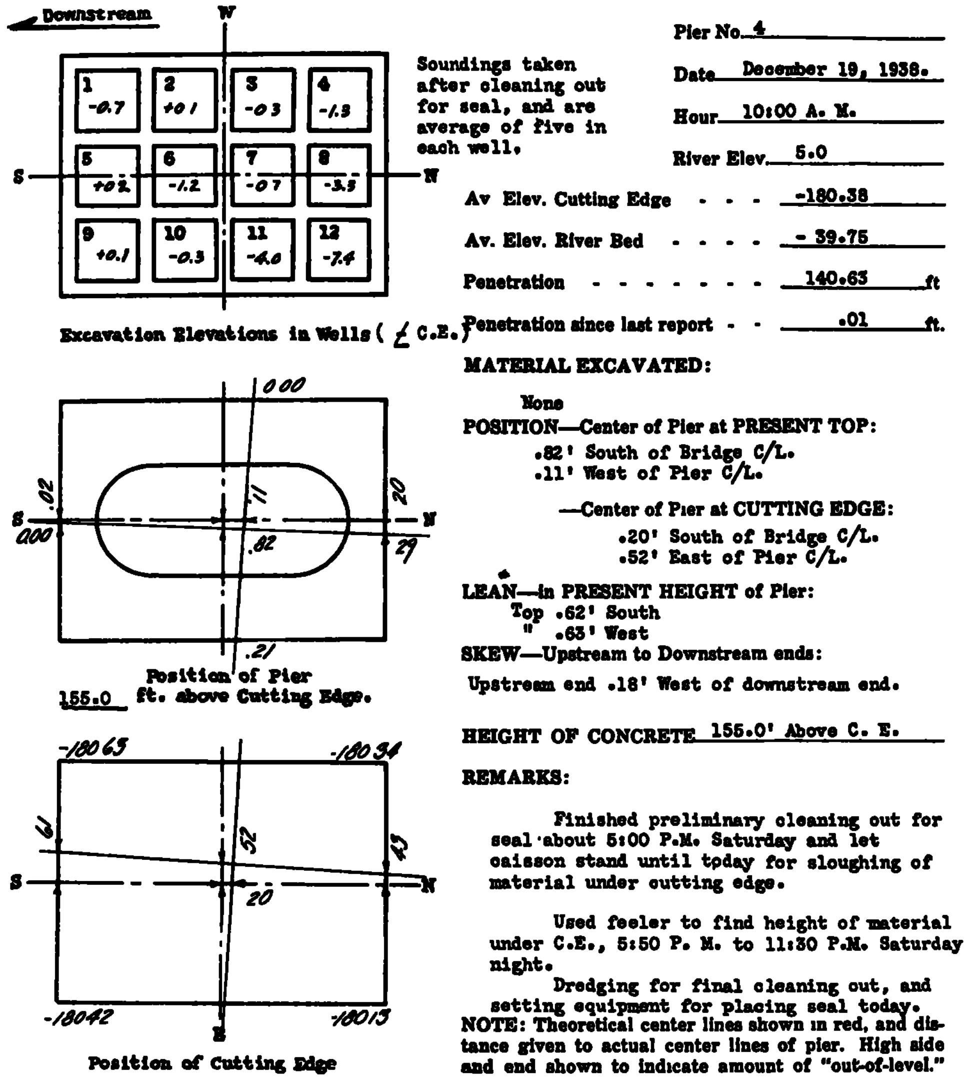

Figure 5. Pier Sinking Report

During the later or final stages of sinking some run-ins of material under the outside cutting edges was noticeable, dredging having been carried below the cutting edges

The washing out was accomplished with a rotary washer having a 4-in. feed pipe and a 1-in nozzle operated at a pressure of 200 lb. per sq. in. Samples of foundation material were taken up from at least five points in each dredging well and inspected to make sure that the foundation was in satisfactory condition. In the case of Pier No. 3, dredging had been carried several feet below the cutting edge in an attempt to obtain greater penetration and rather than place additional concrete in the seal course, back fill was made with concrete sand which when leveled by agitating and other operations preparatory to sealing did provide a good base. The final operation prior to placement of concrete seal was to thoroughly agitate the water in the caisson with jets and dredge buckets, this being done to place into suspension any soft material on the foundation. Locations of piers were closely watched as sinking progressed and final positions were in all cases well within the requirements of the specifications.

The final position of Pier 4 is shown in Figure 5.

ESTIMATED SETTLEMENTS AND OBSERVED SETTLEMENTS

Estimates of settlement were made for Piers 1, 2, 3, 4, and 5, and as construction proceeded observations were made at regular intervals for actual settlements It must be recognized that an attempt to record small variations is most difficult in the lower Mississippi Valley where the ground, due to varying water table, is unstable to such extent as to make benchmarks difficult, if not impossible, to hold. However, observations have shown that the estimates of settlements made prior to construction both as to magnitude and rate were in all cases very close to those actually observed. (See Table 4.)

Whether or not the close agreement between the predictions of settlement and the observations recorded to date is a coincidence or the result of sound methods of sampling, testing, and settlement analysis is, of course, difficult to say. It is certainly conclusive, however, that reasonable estimates can be made by applying the theories and methods known today to be applicable to problems of foundation settlement and this fact should be of interest to all foundation engineers interested in the application of soil mechanics. It is quite true that in the analysis made of these foundations some factors were not considered such as friction on the walls of the caisson, buoyancy, weight of water above low water elevation, and the effect of possible scour and lateral flow. The reason for omitting friction or reducing the foundation pressures by some assumed value was that in the opinion of the author such a reduction would not be consistent with the method of analysis used; for to apply such a factor to reduce foundation pressure or stress would be like to the man pulling himself up by his bootstraps. With this same reasoning reduction of pressure on the base of the foundations by consideration of buoyancy was also neglected. Weight of water above low water elevation, and effect of possible scour were neglected as these were so variable and indeterminate as to make any figures set up of little or no value, for use in the problem presented by this construction. No consideration was given to lateral flow since there has been up to this time no reliable method developed for estimating effects from this source; also, it was the author's further opinion that lateral flow of the material under the foundation is not likely, due to the existing load, earth and water, to which it is subjected.

OBSERVED EFFECTS OF HIGH WATER

Observations on many structures show variations in elevation with changes in ground water or surrounding water elevations and much has been published on this subject. Two actions have been observed, the first, which is the most common and easily explainable, being the case where, by the action of hydrostatic uplift the structure rises with in-

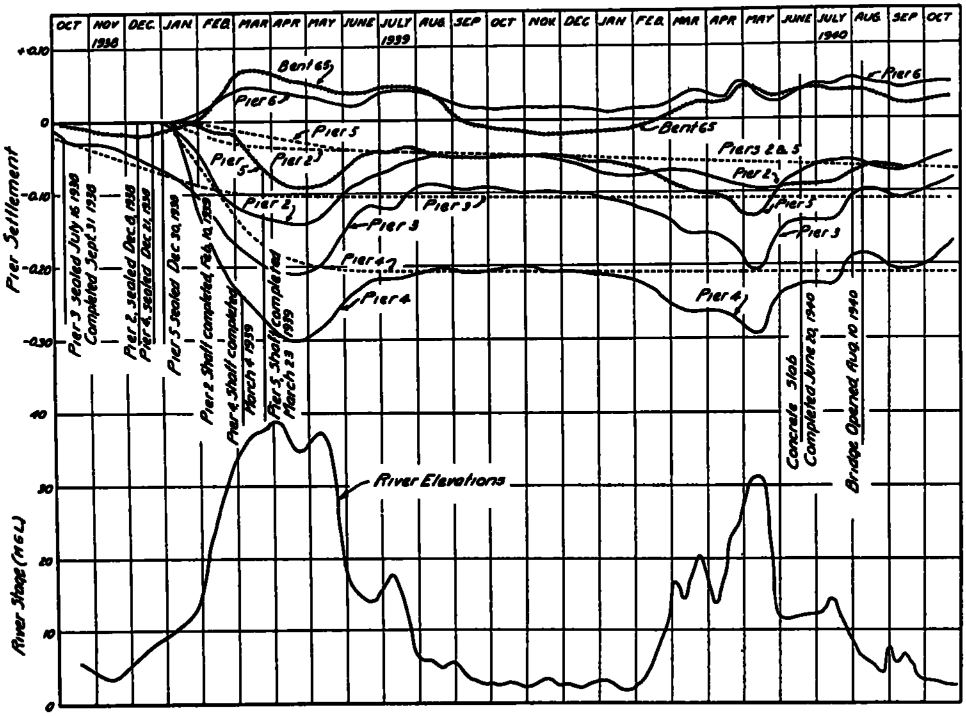

Figure 6. Pier Settlement Readings Using Pier No. 1 as Benchmark
Dotted lines are probable settlement

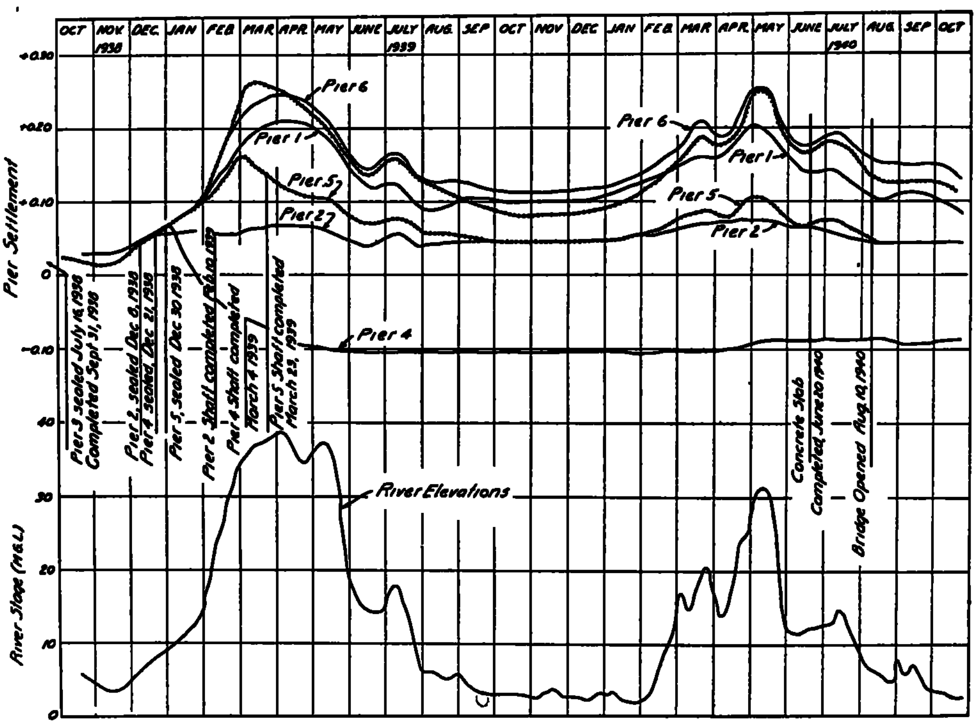

Figure 7. Pier Settlement Readings Using Pier No. 3 as Benchmark

creasing water height and settles as the water level falls. The second case, is that where, with increased water height the structure settles, rising again as the water falls. Observations on the Baton Rouge piers have shown both effects depending upon what was used as a bench-mark. The construction work was controlled from a bench-mark, located on the east bank of the river. This is a heavy clay bank underlayed by pervious material from elevation −20 to −50 and below elevation −80, and which extends out into the river. Observations taken from the bench-mark on the east bank on to the structure located on the west bank, which is a soft clay and silt formation down to elevation −50 and underlayed by a fine sand, showed that during a rise in the river, pedestal foundations for the approach structure, which were founded on driven precast concrete piles having a penetration of 50 ft. would rise. Levels taken on river Piers 2, 3, 4, and 5 from the same bench-mark showed the opposite effect in that these piers appeared to settle with the rising water. However, it is the authors' opinion that the bench-mark on the east bank was and is effected by hydrostatic pressure from the water in the river thus causing the apparent raising and lowering of the river piers. By using one of the constructed piers as a bench-mark, it develops that the deep piers which are Nos. 2, 3, and 4 are effected little if any by changes in water elevation, but that the relatively shallow piers, No. 1 on the east bank and Nos. 5 and 6 on the west bank are effected directly according to their respective depths. Figure 6 shows results of observations from bench-mark located on the east bank and Figure 7 shows results of observations using Pier No. 3 founded at elevation −160 as bench-mark. The variations recorded during two high water periods from Pier No. 3 show a variation per foot of water level of approximately 0.004 ft. at Pier No. 6, and the west approach pedestal foundation adjacent to this pier. A variation slightly less was recorded at Pier No. 1 on the east bank which is founded at a deeper elevation.

PILE FOUNDATIONS

Pier No. 6 was designed with pile foundation, the base of footing being 48 ft. wide and 75 ft. long. In this area were driven 400, 50-ft. untreated timber piles. The two outside rows were battered; first row 1½ in. in 12 in. and second row ¾ in. in 12 in. The elevation of the base of footing is at −0.71 mean Gulf level which places the pile points at elevation −50 or into the sand underlying the clay and silt deposit which forms the upper strata of the west river bank. Pile loading, including weight of overburden, dead load, live load, and wind is 27.2 tons per pile. To determine the pile lengths to be driven in the foundation, two loaded test piles were provided for in the contract. The piling was specified to have a minimum tip diameter of 8 in. and butt diameter not less than 14 in., nor greater than 20 in.

The two test piles were driven at opposite ends of the foundation with penetrations of 48 and 50 ft. respectively below bottom of footing. It was found impossible to penetrate the underlying sand strata without jetting, and, in this instance, jetting each and every pile would, in all probability, have done more harm than good. The test piles showed a driving resistance with a No. 2 Vulcan single acting steam hammer of about a hundred blows per foot. The method of loading these tests was as follows:

Timber weight boxes loaded with sand, and having a total load in excess of the load applied to the test pile (about 80 tons) were used. Each load box was supported on four timber piles; one at each corner of the box; and load was applied to the test pile by means of a 500 ton hydraulic jack. This jack was placed on top of the test pile and jacked against the bottom of the load box. The applied load was measured by the pressure shown on a tested gauge at the jack pump. Loads were applied to test piles No. 1 and No. 2 at the same time.

The piles were loaded in increments, the first being 25 tons. This was increased by 5 tons at 6-hr. intervals until a load of 45 tons was reached, after which the time was increased to 12 hr. Load was then applied at the 12-hr. interval with the same amount of increase (5 tons) until a final load of 75 tons was

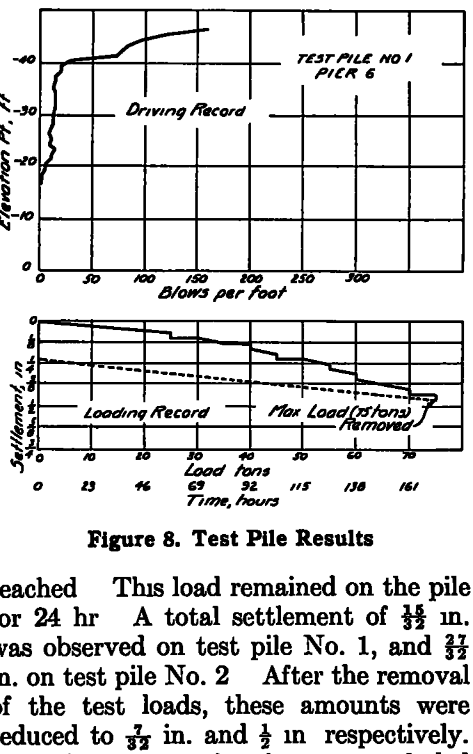

Figure 8. Test Pile Results

reached This load remained on the pile for 24 hr A total settlement of $\frac{15}{32}$ in. was observed on test pile No. 1, and $\frac{27}{32}$ in. on test pile No. 2 After the removal of the test loads, these amounts were reduced to $\frac{7}{32}$ in. and $\frac{1}{2}$ in respectively. From his information it was concluded that piling driven to elevation -48 would safely support in excess of the design load, and the contractor was authorized to procure and drive 50 ft. piling in the foundation. In driving the permanent piles it was found that the elevation of the underlying sand strata varied to some extent over the area occupied by the footing, and therefore some of the piles did not obtain full anticipated penetration. The piles making up this foundation are considered by the author to be largely point bearing as the tests show that the pile having been jetted, thereby disturbing the action of skin friction, acted much the same under load as the non-jetted pile, further very little driving resistance was encountered until the point of the pile was at or in the sand. (See Figure 8 for test pile results.)

To determine pile lengths for the approach foundations 29 precast concrete piles were driven and loaded. For the east approach where piles were driven into hard clay, a penetration of 20 to 30 ft. was considered satisfactory. These east approach tests were made with 14-in. square piles for all of the pedestal foundations, driving being done with a No. 1 Vulcan single acting steam hammer. Driving in the east approach showed in general equal driving resistance for equal penetration and loading up to 75 tons showed little variation. Load tests were made with loading tanks placed on platforms set on the pile. Loads were applied in increments beginning at 40 tons, increased by 10 tons after 12 hr., then increasing by 5 tons at 12-hr. intervals until a load of 80 tons was reached.

The test piles driven in the west approach where the formation is a wet clay, sand and silt of varying density were 16-in. square precast concrete piles. These were driven to a penetration of 40 to 50 ft. Load tests were placed in much the same manner as for the tests made on the east approach except that time interval was increased to 24 hr., and the final load remained in place for 48 hr. Three of the 16-in. piles were loaded to complete failure; and here it is interesting to note that a pile would hold a load of 80 to 90 tons for several hours then suddenly turn loose, settling until the load would become supported on its blocking. This behavior brings out the importance of the time factor in the making of test loads on piling; for by neglecting the time factor, most misleading results might be obtained. These tests showed a recovery in settlement on the piles not loaded to failure between $\frac{1}{16}$ and $\frac{1}{8}$ in. in general and a total settlement on the average of $\frac{1}{4}$ in. The maximum settlement recorded without complete failure was $\frac{5}{8}$ in. under 100 tons.

From the information obtained from these tests lengths of piling to be driven in the permanent structure were determined at 50 and 55 ft.; the 55-ft piles being placed in those foundations over which embankments were later constructed.

Driving resistance was closely recorded on all permanent piles, and it was found that the resistances obtained were equal to and greater than those obtained on the corresponding test piles.

Pile loadings on the approach structure were computed to be 15 to 22 tons including dead load, live load, traction and sway.

In the case of the west railway abutment, 18-in piles were driven to a penetration of 50 ft. below natural ground and through an embankment 25 ft. high which had been built for several months, and here a progressive settlement has been and is being observed. Total settlement in 18 months has been recorded in the amount of approximately seven inches; adjacent bents, however, have been effected very little.

No attempt was made to check bearing capacity by any predetermined formula, the test pile driving record being used to determine whether or not satisfactory resistance was obtained.

Observations for settlement of the approach foundations have been made at regular intervals and in this regard it is interesting to note that only where embankments have been constructed around or adjacent to the pile foundations, thus causing consolidation and lateral flow of the clays into which the piles have been driven, have settlements been recorded. In one instance it was found that the footings for bents 83, 84, 85, and 86 showed a settlement of approximately 2 in after the placement of an embankment having an average height of approximately 10 ft., although, that part of the embankment surrounding bents 83 and 84 had been in place several months prior to piling driving in these two bents; further no settlement was recorded in the highway approach abutments located at the outer edges of the embankment though these pilings had been driven at the same time as pedestal piles in bents Nos. 83 and 84.

CONCLUSION

From the author's experience on this project, it is concluded that application of the principles of soil mechanics to deep foundation problems make it possible for the engineer to determine in advance of construction the magnitude of the settlement to be expected and to provide for this settlement; to anticipate with some degree of accuracy the amount of settlement to be expected during construction stages and to further anticipate progressive settlement over a period of years. With these determined it is not difficult to decide whether or not a given foundation is satisfactory for the structure contemplated. Further, it is possible, after anticipated settlements have been determined to adjust the sizes of the piers, or base areas, so as to get piers of approximately equal settlement.

It is also believed possible to apply the principles of this science to the design of pile foundations especially in cohesive material where the pile is dependent for support upon the frictional or shearing resistance of the soil, provided continued research along this line is carried on

In the construction of embankments much progress has been made in obtaining compaction by control of moisture, and the principles for obtaining best results were put into practice on the embankment construction of the project. However, difficulties have come up from consolidation and lateral flow of the underlying clay formations.

From the experience in the pier sinking operations it is not believed that skin friction on the side of a caisson is a foundation problem of first magnitude for this friction must be broken in order to sink the caisson. Further disturbance due to run-ins of material from under the cutting edge of a caisson where the foundation is of sand is not unduly great. It is and has been possible to remove loose and soft material from the foundation without undue difficulty and place the foundation in a desirable condition before sealing. The piers for the Baton Rouge bridge and others observed in this region were sunk with the material along side moving slowly downward carrying the pier down as material was removed from the dredging wells and the only noticeable effect was that, where friction was completely broken by this action, sinking was easier and initial settlement took place in a shorter time.

DISCUSSION ON BRIDGE FOUNDATIONS AND SOIL MECHANICS

PROF. W. S. HOUSEL, *University of Michigan:* The paper by Mr. Erickson describes the second notable structure of this kind on the lower Mississippi River which has been brought prominently to the attention of foundation engineers in recent years. The other is the bridge at New Orleans known as the Huey P. Long Bridge. The conditions under which these two bridges were built are almost identical, as are the types of structures and the methods of construction. In both cases the character of the foundation offered a major problem both in design and construction.

The difficulty and uncertainty connected with an accurate evaluation of the foundation discouraged the construction of the bridges for many years in spite of their evident need. The investigations when undertaken were probably as elaborate and painstaking as any that have been recorded. Due to the progressive attitude of those directly concerned with the work, the history of the projects is being quite completely written in engineering literature for the benefit of all who are sufficiently interested to analyze the results and correlate them with their own experience.

The author of the present paper is to be complimented as a practicing engineer on bringing clearly to the attention of the profession the problems which were encountered, the methods used in solving these problems, and finally the extent to which the predictions based on the application of soil mechanics are being realized. The same thing may be said of those men who were primarily concerned with the identical problems in the case of the Huey P. Long Bridge. Inasmuch as this bridge has now been in service for slightly more than five years, more information on the actual behavior of the foundations is available than in the case of the Baton Rouge Bridge, and the present discussion will deal largely with this experience record which does, however, have direct application to the paper which Mr. Erickson has presented.

The last available results of observation of the Huey P. Long Bridge were presented by Professor Kimball at the annual meeting of the American Society of Civil Engineers in January, 1940, and have been published for some months.[1] Numerous other articles have been written on preliminary investigations and construction experience and are listed by Kimball.[1]

It is this information which makes it possible for any student of the subject to sift the evidence and come to some logical conclusion rather than to become lost in a maze of conflicting data. In effect the authors of both these papers have said, here is what we did, here is what we didn't do, and here are the results

In this discussion the writer has attempted after reviewing the available information to piece together inferences which may be drawn from the data to arrive at conclusions which in his own mind represent the most logical answer, pending future developments. If in this connection more consideration be given to the things which were not done, it is not in a spirit of criticism but in an attempt to get at the truth under the urge of the intellectual curiosity which the engineers for these two bridges have successfully stimulated.

REVIEW OF PREVIOUS ARTICLES

In reviewing previous articles on the New Orleans bridge the information dealing with soil mechanics procedure is largely contained in Professor Kimball's 1936 paper and subsequent discussions in the Proceedings of the International

[1] William P. Kimball, "Settlement Studies of Huey P Long Bridge," *Civil Engineering,* Vol 10, p 145, March, 1940.

Conference on Soil Mechanics and Foundation Engineering The other articles to which he refers cover the construction features and some pertinent details. In these articles the writer found a number of recorded facts and opinions which have bearing on possible conclusions and which are included as a basis of discussion.

Two sets of borings were made, the first in 1926 and the second and final set in 1933. The 1926 borings and subsequent tests served as the basis for the decision to build the bridge and for the selection of the type of substructure, the construction methods, and the landing elevation. Samples were tested by Doctor Terzaghi who prepared a preliminary report on the load bearing characteristics of the clay strata and the probable settlement of piers founded above them. Carleton S. Proctor, Member A.S.C E., states that these "investigations . . . developed . . . the fact that pier settlements would be less and allowable intensities greater at this level than at the lower levels previously assumed." The writer has not had an opportunity to review Doctor Terzaghi's preliminary report or Mr. Proctor's paper on "Bridge Foundations," but inasmuch as an additional set of borings was made in 1933 for the purpose of estimating pier settlement, presumes that these later data provide an adequate basis for the present discussion.

The settlement estimates for nine piers were made from consolidation tests on undisturbed samples from the 1933 borings and were made available to the engineers three months before actual construction began. They led to two practical results, a change in the landing elevation of Pier A with a substantial saving and a change in the elevation of the bridge seats with provision for jacking the truss bearing plates back to position after settlement had taken place. Difficulties were introduced into the analysis by the hit-or-miss occurrence of clay lenses near the landing elevations of the main river piers. In a comparison of the observed and predicted settlement of Pier A, skin friction is suggested as a possible reason for the discrepancy noted.[2]

In several short discussions of the New Orleans bridge a number of interesting opinions regarding the foundation behavior are to be found.[3] Doctor Terzaghi expresses surprise at the good agreement between computed and observed settlement of the river piers whose source of settlement is located within a thick bed of fine sand subject to rapid consolidation. He observes that on the basis of the assumption of lateral confinement, which is one of the conditions of the confined compression test, the ratio of settlement to load should be a decreasing function as load is increased. However, he cites a loading test which he performed by loading a concrete platform with an area of 10 sq ft. on a bed of sand under comparable conditions in which the ratio of settlement to load increased rather than decreased, an indication that lateral yield rather than vertical compaction was an important factor in loaded sand. He concludes that the satisfactory agreement was due to a chance factor not present in many cases.

A good deal of attention in these discussions is devoted to the intermittent fall and rise of the main river piers as the river stage rises and falls. The elastic character of these changes is well recognized, the only point at issue being whether the dominating factor is lateral yield or vertical compression. Doctor Terzaghi favors the lateral yield as many experiments indicate that if the

[2] William P Kimball, "Settlement Records of the Mississippi River Bridge at New Orleans," *Proceedings*, International Conference on Soil Mechanics and Foundation Engineering, Vol 1, Paper F-4, p. 85 (1936).

[3] Discussion by Terzaghi, ibid, Vol. 3, Paper F-22, p 96, 1936

depth of the sand is greater than one-half the width of the loaded area, the greater part of the settlement is due to lateral yield. In this case the river is approximately 2600 ft wide and the sand bed approximately 2000 ft. deep. Professor Kimball favors vertical compression as lateral yield would cause heaving, but Doctor Terzaghi points out that the heave is always imperceptible unless the material is loaded close to its ultimate bearing capacity.[4]

Pertinent information contained in the other articles having to do with the construction features will not be assembled at this point but will be included in the writer's attempt to recapitulate the situation.

FACTORS NEGLECTED IN THE ANALYSIS

In this recapitulation the first subject that will be considered are those factors which the published information indicates were not considered in evaluating the settlement of the main river piers. These are five in number and include the following:

1. Lateral yield.
2. Skin friction or side shear on the caisson piers
3. Preliminary consolidation due to the sand islands.
4. Disturbance of soil strata during construction
5. Dimensional effects due to variation in the size of the piers

While each of these factors will be considered in turn it appears as a preliminary consideration that on the basis of nothing more than engineering judgment any one of the first four factors might very possibly produce effects under existing conditions of the same magnitude as the total settlement recorded and certainly the discrepancy between computed and observed values. The fifth and last factor, variation in size of the piers, may be readily disposed of and does not appear to be significant.

This discussion will be limited to the four main river piers which are all landed at Elev. 170 on the top of a stratum classified as sand having a depth of approximately 100 ft.

Size Effects

Piers 1 and 2 are 65 by 102 ft. while Piers 3 and 4 are 53 by 92 ft. These represent rectangular bearing areas transmitting to the top of the sand layer net soil pressures varying from 3300 to 6700 lb. per sq. ft. Assuming equal settlements for all piers, which incidentally is very close to true according to the observations, the possible boundary effects may be estimated by the linear equation for bearing capacity proposed by the writer. From many pressure plate experiments performed on unconfined sand it is well known that the edge of a bearing area resting on the free surface of granular material is a source of weakness, so that smaller areas will carry less load than larger areas at comparable settlements. This variation can be expressed as a perimeter factor but in this case all the bearing areas are so large that the boundary effects are insignificant. Further than this there is little reason to expect the highly confined sand to follow the behavior of the unconfined loading surface, and as a matter of fact the contrary behavior could be anticipated. It is concluded, therefore, that variation in size of the bearing areas can be dismissed as having negligible effect.

Lateral Yield

It must be apparent from the opinions cited in previous discussions that lateral yield may be the most important factor that has not been considered in the analysis based on a confined consolidation test. While recognizing consolidation as an important factor in

[4] Discussion by Kimball, ibid, Vol. 3, Paper F-23, p 97, 1936

evaluating total settlement it has long been the writer's contention that lateral yield or displacement is a most important practical consideration in foundation design not only for granular materials but for cohesive ones as well.

In the case of granular materials now under consideration it is reassuring to find that others have that view. Doctor Terzaghi has said that when the depth of sand stratum was greater than about one-half the width of bearing area the greater part of the settlement is due to lateral yield. In this case width of bearing areas are 53 and 65 ft. and the depth of sand to the 50-ft. clay layer is 100 ft., with depth-width ratios of 1.89 and 1.54 respectively.

While there may be a settlement contribution from the 50-ft. layer of stiff blue clay even though it is at a depth 1½ to 2 times the width of the areas, the writer cannot help but feel that this contribution is of a lower order of magnitude than that which occurs in the region of pressure concentration not much in excess of a depth equal to the width of the bearing area.

Further than this if the sand is properly classified it would appear that inundated and subjected to considerable static pressures, it should be quite well compacted and its consolidation would be much less important than the lateral yield. The aspect of whether or not the material so classified may have cohesive characteristics will be discussed later.

Skin Friction or Side Shear on the Caisson Piers

The action of what might be called either skin friction or side shear on the caisson piers while possibly not so vital a consideration as lateral yield appears to the writer to be a factor which could not be neglected with any reasonable expectation of getting close to a right answer. In Table 1 are presented the computations made to determine the ratio of side shear areas to bearing areas for each pier. In determining the depth of caisson buried below river bottom the only information available was taken off small scale drawings in several of the published articles so is only approximate. However, any changes which might be made from a more adequate source of data should not substantially affect the final result.

After computing the friction length and area for each pier, the last column of the table shows the ratio of side shear to bearing area. It is found that the shear areas are approximately 5, 6, 7, and 8 times the bearing areas for Piers 1, 2, 3, and 4 respectively. If any basis can be established by which the frictional resistance or shearing resistance may be related to total bearing capacity in terms of these relative areas it is evident at once that side shear or skin friction may be an even more important factor than end bearing.

TABLE 1

Pier No.	Dimensions: Width	Dimensions: Length	Bearing Area, A_b	Friction Length	Friction Area, A_f	Ratio $A_f \div A_b$
	ft.	*ft*	*sq ft.*	*ft*	*sq ft*	
1	65	102	6630	95	31,730	4 8 (5)
2	65	102	6630	115	38,410	5 8 (6)
3	53	92	4876	115	33,350	6 8 (7)
4	53	92	4876	135	39,230	8 1 (8)

Some observations made during construction lend support to the idea that side friction cannot be taken lightly. Helmers reports that after the caissons sunk approximately 70 ft. it was necessary to excavate the sand island for each 10-ft. drop to reduce skin friction.[5] He also noted that it was once necessary to excavate 6 feet below the interior cross-walls and 2 feet below the outside walls

[5] N F Helmers, "Pier Foundations for the New Orleans Bridge," *Civil Engineering*, July, 1936, Vol. 6, No 7, p 442

when sinking the caisson through a 25-foot clay stratum. Engel also notes occasional difficulty due to the caisson hanging up particularly in clay layers but also in the sand island.[6] He also notes difficulties which were encountered a number of times due to blow-ins which did not seriously affect the progress of construction or ultimate stability of the piers.

Preliminary Consolidation due to Sand Islands

Very little mention is made in any of the published articles reviewed by the writer of the possible effect of the sand islands which were built to accommodate the sinking of the caisson. These bodies of sand were held in circular steel shells 122 ft. in diameter for Piers 1 and 2 and 111 ft. in diameter for Piers 3 and 4. As near as the writer could determine from the descriptions, the sand islands were built up to approximately El. +18. Assuming the sand in place as made up of 65 per cent solids with a specific gravity of 2.65 the computed net pressures on the river bottom vary from approximately 5600 to 7000 lb. per sq. ft., in one case slightly less and in three cases somewhat in excess of the net pressures applied on the pier bearing areas.

The net pressures due to the sand islands are applied over areas larger than the pier bearing areas but at river bottom which is 95 to 135 ft. above pier landing elevations, depths which are comparable to the diameters of the islands themselves From this viewpoint consolidation effects at greater depths might be assumed to be of minor importance. It is doubtful if they could be neglected in the light of the rapid consolidation characteristics of the sands which might easily furnish substantial effects even in the shorter periods of time over which the sand pressures were applied. In connection with the time period involved the only information noted by the writer was given by Helmers who stated that from starting erection of the ring to sealing the caisson for Pier 2 the time elapsed was 135 days.

Such effect as this preliminary consolidation may have had, should have decreased observed settlement of the piers inasmuch as removal of the sand island was performed during and immediately after the sinking of the caissons. This would bring settlements for Piers 3 and 4 into better agreement with predictions but the converse would be true for Piers 1 and 2.

Disturbance of Soil Strata during Construction

The last of the neglected factors to be considered is the possible disturbance of the soil strata during construction, and in the writer's opinion this is by no means the least important. Reference has already been made to disturbance noted during construction in the form of blow-ins, most of which were minor and none of which apparently interfered seriously with construction They are, however, indicative of disturbance of the surrounding mass and it is not difficult to see how this could scarcely be avoided considering the magnitude of the operation. Its effect on settlement of the piers after they had been landed is, of course, impossible to predict but the behavior of the piers themselves may be taken as evidence bearing on this point.

The effect of disturbing the soil structure is well known both in the field and laboratory and doubtless many instances can be cited which appear to be comparable and in which disturbance is quite evidently the source of settlement. In one case, in the writer's experience, tests on a steel cylinder pile were being

[6] H J Engel, "Construction of the New Orleans Bridge," *Civil Engineering*, Dec, 1935, Vol 5, No 12, p 775

made after excavating the core of soil to the bottom of the cylinder. A minor blow-in occurred and it was several weeks before the skin friction on the test pile regained its former value. Similar experience is quite common in driving piles and numerous instances of excessive settlement due to such disturbance are on record. Settlement observations on a Detroit grade separation where several piers were placed on friction piles resulted in settlements of approximately 3 in. before the piers came to equilibrium. On this same structure more lightly loaded piers designed as spread footings came to equilibrium at design settlements of less than ½ in., the only apparent answer for this difference being in disturbance due to driving piles

Further discussion of disturbance as a source of settlement will be taken up after consideration of the pier settlement and in connection with an attempt to analyze their behavior.

SETTLEMENT OBSERVATIONS

The observed settlements of the main river piers provide the most reliable basis for any analysis of behavior and these data tell their own story with little regard for predictions. It seems quite reasonable to regard them as large size load tests, and if their behavior may appear to be at variance with most carefully conceived predictions theoretical considerations must conform or admit defeat. In Table 2 are listed those settlement observations which may be taken as one basis for further discussion. These results have been scaled from figures in Professor Kimball's article[1] and again there may be minor inaccuracies, but there should be no substantial discrepancies with respect to original data.

The total settlements are for 1939 and were taken from the low water curves and predicted settlements are from the original curves of 1933. The total settlements might have been taken for an earlier date, but inspection of the curves indicates that the relative settlements of the several piers would be practically the same.

The first point which strikes the observer is the fact that total settlements for all four main river piers are the same within quite narrow limits The maximum departure from the average is only 5.6 per cent in the case of Pier 2, while it is 2.4 per cent for Pier 1 and less than 1 per cent for both Piers 3 and 4. It appears that this behavior would be a most singular coincidence if no logical reason for it could be uncovered. The disturbing feature of this equality of settlements is that it has been observed in spite of a variation in applied pressure from 6700 to 3300 lb. per sq. ft. Further than this the more lightly loaded piers are those which have the greatest side shear areas which should further decrease their settlement if friction or shear is a factor.

To conclude that there is no relationship between load and total settlement under any conditions might at first glance appear to violate the most firmly established principles of resistance of materials. On the other hand, it is equally difficult to imagine that a caisson which had been sunk through a considerable depth of a soil mass by methods calculated to break down the resistance to sinking would immediately come to equilibrium from disturbance and start behaving in accordance with an undisturbed material as soon as the caisson was sealed and particularly if side shear was a substantial factor of resistance. There appears to be sufficient evidence from excessive settlements under the disturbance of driving piles to justify the viewpoint that a large part of the total settlements recorded may be due to disturbance during construction.

The period of time necessary for the effects of disturbance to disappear and the normal resistances to be mobilized might well be as long as the time required to complete erection of the structure when full dead load would be applied It may be noted that such a deduction would be in substantial agreement with the settlement charts. Considering that the construction methods for each of the four piers was quite closely restricted to identical procedures by existing conditions it does not appear unreasonable to expect the effects of disturbance to be comparable in each case There is, of course, little reason to anticipate as close agreement as shown in this case and little or no possibility of predicting the actual amounts of settlement. Likewise, it must be equally true that no normal load-settlement relationships can be applied during this period and even agreement in the order of magnitude of such predictions appears to be largely a matter of chance

It is after the effects of disturbance have disappeared that the settlement of the piers may show some conformity with normal laws of behavior In Table 2 are shown the rates of settlement scaled from the charts and representing present behavior Insofar as present observations may be used Piers 1 and 2 show uniform progressive settlement as a function only of time which is in accordance with the conventional relationship of lateral yielding or plastic flow. Piers 3 and 4 show a decreasing rate of settlement with time in accordance with conventional relationships of consolidation. While there is still indicated a very small rate at the last observation, on the order of $\frac{1}{4}$ inch in 10 years, it appears that these latter piers may be very close to static equilibrium.

No attempt is made to compare predicted and measured rates of settlement where in some cases the predicted curves appear to provide for a uniform rate and in other cases a decreasing rate. The writer is not familiar with the method of introducing a uniform rate factor into the consolidation theory, if that has been done, and is under the impression that such behavior is characteristic rather of lateral yield.

TABLE 2

OBSERVED SETTLEMENTS

Pier No.	Net Pressure	Total Settlement (1939)				Error		Rates of Settlement	
		Observed Δ_o		Predicted Δ_p		$\frac{\Delta_p - \Delta_o}{\Delta_p}$	$\frac{\Delta_a - \Delta_o}{\Delta_a}$	1 Year Period	10 Year Period
	lb per ft²	*ft*	*in*	*ft*	*in*	%	%	*in.*	*in*
1	6700	265	3 45	370	4 45	+23	−2 4	180	1 80
2	6200	287	3 18	347	4 27	+26	+5 6	216	2 16
3	5000	283	3 40	144	1 73	−97	−0 0	036	0 36
4	3300	282	3 38	143	1 72	−97	−0 3	024	0 24
Average						60	2		

Avg Settlement Δ_a = 3 37

At any rate on the basis of rates of settlement there appears to be some relation between load and settlement although there is no direct proportionality evident. Nor could such a simple relation be expected if side friction is a factor as the more lightly loaded piers have greater friction areas available and this factor has been neglected in computing the net pressures given.

ANALYSIS OF POSSIBLE BEHAVIOR

After having considered the various factors which may have some bearing on the pier settlement and arrived at some conclusion as to their relative importance it is impossible to avoid the temptation of attempting to bring them into some semblance of agreement with the actual observation. While this attempt may be regarded as speculation it still may have value and being hind-sight it may have an unfair advantage over foresight but a better chance of success.

Treatment as a Cohesive Material

As the first assumption the behavior of the piers will be analyzed as if the relationships for a purely cohesive material were applicable, reducing simply to shear which is assumed to be mobilized only after the period of disturbance has disappeared.

TABLE 3

COMPUTED BEARING CAPACITY

Pier No	Shear Resistance Factors			Total Bearing Capacities for Various Shear Values				Net Applied Pressure
	Side Shear	Developed Pressure	Total	200	400	500	600	
								lb. per sq ft
1	5	4	9	1800	3600	4500	5400	6700
2	6	4	10	2000	4000	5000	6000	6200
3	7	4	11	2200	4400	5500	6600	5000
4	8	4	12	2400	4800	6000	7200	3300

Under this assumption the shearing resistance is a function of cohesion and independent of normal pressure.

The computations for this analysis are indicated in Table 3.

In column 2 are shown the shear resistance factors previously computed for each pier as the ratio of side shearing area to bearing area. This factor represents the relative contribution of side shear in terms of applied pressure on the bearing area. The developed pressure factor represents the applied pressure which may be developed as the difference between the principal pressures on two elements of mass in a two-dimensional stress system. This factor is taken as four times the shearing resistance It may be noted in passing that this difference varies from 3.14 to 5 14 as developed by other investigators, depending upon the basic assumptions used. Derivations of this factor are presented in other publications and will not be discussed here as the present comparison is not materially affected. The sum of these two factors represents a figure which may be multiplied by the assumed shearing resistance to obtain the total bearing capacity including side shear and bearing.

By comparing these total bearing capacities with the net applied pressure a shear value may be selected which would produce the settlement behavior of the piers which has been previously discussed. If the observed settlement rates are interpreted to mean that Piers 3 and 4 are coming to equilibrium a shear value between 400 and 500 lb. per sq ft. would be indicated. In this case it may be noted that the analysis assuming relationships for a purely cohesive material leads to the conclusion that Piers 1 and 2 may undergo a continued progressive settlement. It should be noted that this rate, on the order of 2 in. in 10 years, is probably not serious from the standpoint of the bridge and its continued service. If it is found in the future that a very slow settlement rate persists for Piers 3 and 4 it could be taken as an indication that the shear finally mobilized is something less than 400 lb. per sq. ft

The above analysis was based on the assumption that the relationships of a purely cohesive material were applicable. It may be pertinent to consider whether or not this might be the case and there does appear to be some justification for the assumption. In the first place the borings for both bridges indicate that the river deposits to some depth were made up of many superimposed layers of sand, silt, and clay and in some cases they were predominantly silt and clay.

These observations are indications which make it appear not too unreasonable to consider that the soil may act as a cohesive material The character of the settlement as far as present rates are concerned bears out this assumption at least for Piers 1 and 2 while Piers 3 and 4 are not yet clearly indicative. It may also be noted that the general theory of consolidation which has been used to predict settlement is ordinarily considered as applicable only to saturated clays.

If the soil stratum on which the piers are landed was essentially granular any analysis of the pier behavior leading to a deduction of available bearing capacity and settlement would contain essentially the same factors. However, the side shear would vary with the depth and static pressure and it appears that the allowable bearing pressure which might be imposed would in all probability be much higher than for a cohesive soil. The passive resistance available under the relatively high overburden pressure would be more than sufficient to bring the pier bearing areas to equilibrium.

With respect to settlement in a granular mass it would still be impossible to predict the effect of disturbance but after the initial period of adjustment it would appear that the sand strata should be relatively well compacted and settlement consequently of a low order of magnitude.

In conclusion it appears to the writer that the final answer as to accuracy with which the settlement behavior of these two bridges has been predicted by the application of fundamental principles of soil mechanics will not be given until after a considerably larger period of observation. It is to be hoped that the engineers who are making the observations will continue to keep the profession advised from time to time of the results, which will greatly increase their value to progress in foundation practice. The contribution which these two projects represent is outstanding and the authors of the various articles are to be highly commended for making their experience available as general knowledge to all who are interested in the correlation of theory and practice in soil mechanics.

PROF. ROBERT F. LEGGET, *University of Toronto*· The paper under review includes not only an admirable account of the conception, design and construction of the foundations of this important Mississippi River bridge but also the data necessary for a clear appreciation of the way the piers have "performed" since the bridge went into service. The fact that this performance involves movements to be measured only as inches over the years is no reason why the usual term describing the action of an engineering achievement when in service should not be applied even to bridge piers.

The settlements of the piers so far observed do not agree accurately with the settlements predicted on the basis of preliminary soil tests and design calculations. They are, however, of the same order of magnitude This fact, in itself, is an encouragement to all interested in the application of soil mechanics studies to such practical problems as bridge pier design and performance. In view of the many uncertain factors involved, this relative agreement is worthy of note; it provides a good answer to the criticisms of those who still scoff at the practical utility of careful undisturbed sampling, laboratory soil tests, and settlement calculations.

To the student of soil mechanics, however, the differences between calculated and observed settlements is too great to escape notice; it provides an irresistable temptation to further enquiry. There are a number of factors that were apparently neglected in settlement calculations, of varying degrees of importance. Some of these are mentioned by Mr. Erickson in his paper and of one of them he says:

"It is quite true that in the analysis made of these foundations some factors were not considered such as friction on the walls of the caisson The reason for omitting friction or reducing the foundation pressures by some assumed value was that in the opinion of the author such a reduction would not be consistant with the method of analysis used, for to apply such a factor to reduce foundation pressure or stress would be like to the man pulling himself up by his bootstraps"

It is significant, and of more than usual interest, to note that in the published description of the comparable settlements of the piers of another large bridge across the Mississippi River, a similar disclaimer with regard to "skin friction" (to use the common term) is included. Professor W. P. Kimball, in his paper on the settlement of the piers of the Huey P. Long Bridge at New Orleans,[1] states that

"It has been further assumed that all the load is transmitted to the soil at the bottom of the caisson seal The load that might be permanently transmitted through skin friction along the sides of the pier has never been determined"

The ambiguous if not surprising statement in the second sentence of this quotation was the subject of discussion in letters published after the appearance of Professor Kimball's paper Mr. G. L Freeman, a member of the firm which designed the Huey P. Long Bridge foundations, wrote[2].

"When sinking to such great depths, the consideration of skin friction is necessary to the proper determination of the proportions of the caissons, which must be designed with sufficient weight to permit sinking without unreasonable difficulty

"It is not the practice of the writer's firm to consider skin friction as a permanent factor in the support of deep caissons It is realized that a certain amount of support actually may be provided by the soil surrounding the caisson, and such support, if any, leads to discrepancies between theory and practice in soil mechanics"

It is to this suggested concept of discrepancies between theory and practice in soil mechanics that the writer wishes to direct attention, particularly in view of the striking fact that skin friction was admittedly neglected in the settlement calculations for both of these two notable bridge projects.

Clear distinction must first be made between the use of skin friction in the design of bridge piers and in the calculation of their anticipated settlement. Data on skin friction is admittedly uncertain, and its inclusion in design calculations must still be an empirical proceeding. Although some allowance for skin friction has been made in the design calculations for many large bridge piers, particularly in India, it is quite understandable that the designers of both sets of Mississippi River bridge piers now under review should have considered it unwise to make any such allowance in their designs. So far as the writer is aware, no criticism of this course has yet been voiced in public, or even inferred. It would not be fitting for any such comment to be made on the basis of necessarily brief published descriptions. This discussion, therefore, has no reference—direct or indirect—to the inclusion or exclusion of skin friction in design calculations.

Both Mr Erickson and Professor Kimball, however, have put the profession in their debt by publishing not only general details of the basis of the respective bridge pier designs but also the results of calculations as to the anticipated settlements of the piers. They both explain that, even in these calculations, skin friction on the sides of the piers was not taken into consideration. It is now suggested that this procedure is illogical and incorrect; it may go far towards explaining the strange

[1] Kimball, W P, "Settlement Studies of Huey P Long Bridge", *Civil Engineering*, March 1940, Vol 10, p 147

[2] Freeman, G. L, Letter (titled "More on Skin Friction"), *Civil Engineering*, September 1940, Vol 10, p 596

variations in the ratio of anticipated to actual settlements for the different bridge piers.

If the methods and findings of soil mechanics are to be applied in settlement calculations it is clearly useless to go to great refinement in computation if all the factors influencing settlement are not taken into account And, whatever may be thought about its permanence and its reliability for use in design, friction will be developed between the sides of bridge piers and the soil surrounding those parts that are below ground surface level when in final position. If this friction exists, it will affect pier settlement; it cannot therefore be neglected in any attempt at accurate prediction of pier settlement, even though it may be extremely difficult to make the necessary allowance for it in computations.

Any attempt to revise the settlement calculations for the piers of either of the bridges under consideration, taking skin friction into account, would be an undertaking of no small magnitude. Suffice to say therefore, at this time, that in Professor Housell's discussion of Mr. Erickson's paper will be found an estimate of the significance of skin friction in relation to the settlement of the piers of the Huey P. Long Bridge. As an addendum to this treatment, some factual data on skin friction may usefully be put on record in this place.

Bridges in India include many notable structures, across the great winding rivers of that eastern land, comparable in general respects with the Mississippi bridges now under discussion. Engineers associated with these bridges have placed on record many records of actual values of skin friction and the writer has elsewhere[3] reproduced some typical figures used in Indian bridge pier design. Regarding actual measurements of skin friction, the following extracts are of interest.

[3] Legget, R F, Letter (titled "Data on Skin Friction"), *Civil Engineering*, July 1940, Vol. 10, p 452

(a) R Mair in his description of the construction of the Willingdon Bridge across the River Ganges near Calcutta, (Min Proc Inst C. E., Vol 235, 1934, pp 59 and 60) states "It is difficult to put forward any figures or calculations, but on two occasions the cutting edge of caisson No 8 was practically devoid of support, and the resistance then worked out at the high figure of 896 lb per sq ft in one case and 1232 lb in the other The Author calculated it to be 896 lb per sq. ft on caisson No 1 Such figures are certainly high in clay, and after a caisson has been founded and the ground has been given time to settle, the frictional resistance must be of considerable effect in supporting the load on the piers"

(b) In the discussion of this paper (pp 115 and 123), data on measured values of skin friction are given by Mr P L Pratley (for Canada) and Sir Francis Spring (for India) as follows.
Boulder Clay, with rectangular steel caissons having the cutting edge 40 ft down, air leakage kept to a minimum, 750 to 800 lb per sq ft ,
Clay, values varying from 390 to 890 lb per sq ft

(c) In the same discussion (p 95) a brief description is given of the sinking of a caisson through sand for the Nagavalli Bridge of the Bengal-Nagpur Railway A point was reached beyond which no progress in sinking could be made until cans of oil had been introduced by divers beneath and around the cutting edge Sinking resumed when the oil appeared on the surface, but could not be started again on the day following until more oil was introduced for lubrication against the skin friction on the caisson, which was thus sunk to rock level

Records of Indian bridge engineering contain a number of similar instances of measured values for skin friction. It would be of interest to know if Mr. Erickson has available any comparable data for the Baton Rouge bridge pier sinking.

In the face of such positive evidence as to the values that skin friction against bridge piers may attain, it is impossible to accept as a logical procedure the complete neglect of this factor in calculating anticipated pier settlements for the two bridges under review. The discrepancies between calculated and actual settlements may be explained by this neglected factor, at least in part. In view of this, it is unfortunate that there should have been created, by the publication of these settlement records, such an impression in the minds of some engineers that the differences can be referred to by one of so wide experience as Mr G. L. Freeman (*op. cit.*) as "discrepancies between theory and practice in soil mechanics." Until all factors have been included in the calculations of these settlements the discrepancies cannot be charged against soil mechanics, either in theory or in practice. It is greatly to be hoped, therefore, that when further data on the settlement of the Baton Rouge bridge piers can be made available to the profession, some indication of the possible influence of skin friction upon the calculated settlements may be presented at the same time.

MR. C. A. HOGENTOGLER, *Public Roads Administration:* In my opinion the present meeting marks a distinct step forward in the practical application of soil mechanics.

A wealth of valuable material has been presented on the use of direct loading and laboratory test data in the evaluation of subgrade supporting requirements for road surfaces. The use of pavement deflections as a criterion in this respect simplifies very much the research that must be carried on in order to determine the validity of existing theories. For those interested in the evaluation of subgrade support by means of laboratory tests, theoretical approaches presented have opened up the way to classify subgrades on the basis of the supporting strength of soils in either natural state or at any other state at which it is practical to compact subgrades or embankments. Further extensions of the work will permit a classification on the basis of thicknesses of the different types of pavements required.

Most encouraging for those interested in the application of modified theory is the discussion which Mr. Palmer presented of Professor Spangler's excellent paper It demonstrates how correction coefficients can be used in order to have theoretical expressions truly predict the performance of soil en masse.

With respect to the application of soil mechanics in the construction of large bridges, Mr. Erickson's report was most refreshing. We are thankful to the Louisiana Highway Commission for sending him. There has been increasing evidence of a great void between those who produce soil mechanics data and the practicing engineer who spends millions of dollars putting up the structures. Here is one time that we had a connecting link—a man who did the constructing and who was at the same time interested in applying soil mechanics in the work.

PROF. G. P. TSCHEBOTAREFF, *Princeton University:* I wish to join Professor Leggett in complimenting Mr. Erickson and the Louisiana State Highway Department on the very complete and thorough studies carried out on the Baton Rouge bridge.

I have only one critical comment to offer. It would appear that not only clay samples, but also samples of fine packed sand and of sand with gravel were extracted from boreholes at considerable depths, were considered as being "undisturbed" and were used for determining consolidation and time-settlement relationships.

It should be stressed that no satisfactory methods have yet been developed so far as would permit the really undisturbed extraction of sand samples under the above conditions. The loosening of originally compact sand strata or the compaction of originally loose granular deposits is equally possible. In view of this fact, good agreement between advance settlement forecasts and observations made on sand deposits is to be considered as accidental. The only thing that can be definitely established by laboratory tests of this type where sands are concerned is the least possible compression of a sand in a state of maximum density artificially created in the laboratory; also, the rate of its compression, since the rate of compression of a sand is only slightly affected by changes in its density.

These considerations make it appear advisable to obtain supplementary empirical data on the original density of sand deposits in-situ by recording the penetration resistance of standard devices driven in for exploratory purposes. The standardization of such devices is essential and has yet to be performed. Once done, such standardization will increase still further the utility for future designs of studies similar to the ones carried out so thoroughly at Baton Rouge The importance of such studies for control purposes of the structure itself, as at Baton Rouge, is obvious and need not be further emphasized in this discussion.

PROF J. D. WATSON, *Duke University:* I am very grateful for the opportunity to indicate some other practical applications of soil mechanics. I have in mind two simple soil tests which furnish criteria of great practical value; yet these tests are but little used by highway engineers

In previous sessions you have heard a great deal about settlement studies and about soil stabilization. These two topics are of fundamental importance. Also, they are complex problems which require a great deal of study and investigation for a proper solution. But what I wish to suggest are two simple tests which can be performed in any well-equipped soils laboratory. Furthermore, the results of these tests constitute criteria which can be applied directly to the problem at hand. The tests are Casagrande's criterion for the susceptibility of a soil to frost heaving, and the use of Darcy's coefficient of permeability as a measure of the drainability of a soil.

The criterion for the susceptibility of a soil to frost heaving is as follows: any uniform soil which contains as much as 10 per cent of grains, the size of which is 0.02 mm., or less, in diameter, will be subject to frost heaving if an adequate supply of ground water is available, and if the proper temperature and rate of freezing exists. Likewise, any well-graded soil which contains as much as 3 percent of grains, the size of which is 0 02 mm., or less, in diameter, will be subject to frost heaving if an adequate supply of ground water is available, and if the proper temperature and rate of freezing exists. Casagrande evolved this criterion more than 10 years ago when he was making a study of frost heaving for the New Hampshire State Highway Department. The criterion is still successfully used by New Hampshire, and Maine has also used it with complete success for a number of years. I commend its use to all highway departments who have frost heaving troubles.

For many years the validity of Darcy's law for the flow of water through soil has been continuously questioned. Recently, however, it has been incontrovertibly shown ("An Experimental Investigation of Protective Filters," G E. Bertram, published by the Graduate School of Engineering, Harvard University, Cambridge, Mass, 1940) that Darcy's law is not open to question, but that erratic results from permeability tests will inevitably occur unless unusual precautions are taken to eliminate dissolved gases from the water used in the testing. The technique for removing, and for keeping removed, these dissolved gases is comparatively simple. Here, then, is a criterion by which the drainability of a soil can be measured in the laboratory. The application of this criterion to soils in highway construction will save thousands upon thousands of dollars annually, not only to eliminate needless construction (such as, drainage ditches in cuts which won't drain anyway), but also to indicate in advance those points where remedial drainage measures will have to be applied. Also, the problem of drainage for airports is almost always a serious one. Here then is the means for measuring in the laboratory with a falling head permeameter the coefficient of permeability, which is also a measure of the drainability of the soil. In view of the vast program of airport construction which is now getting underway the inclusion of this test in the program of soils investigation is particularly pertinent.