Soils Exploration and Design Considerations for the Greater New Orleans Expressway

II. Soil Design Considerations

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● LAKE Pontchartrain is an oval-shaped lake 43 miles long and 25 miles wide lying north of the city of New Orleans. The bridge now being built across it, at a cost of slightly more than $1,000,000 per mile and at a rate of ½ mile per week, has behind it an exciting history of planning and design. We are here primarily concerned with how this design was influenced by the subsurface materials. First, though, it may be of interest to touch briefly on some of the more general design considerations. Since the bridge itself is the major structure in the expressway, most of this paper relates to it rather than to the approach roads.

General Design Considerations

A traffic survey, including origin and destination surveys at strategic points, was made as one step in determining the economic feasibility of the structure. This survey was conducted by the firm of DeLeuw, Cather & Co., Inc., in collaboration with traffic engineers of our own staff. It indicated that toll revenues, supplemented by income derived from vehicle registration fees and allocated by the state to the parishes, would be adequate to finance the structure and that a 2-lane bridge, affording 24 miles of practically unobstructed driving, would be adequate to provide for the estimated traffic in the foreseeable future.

The terminal points of the bridge were established and tied in by a triangulation network extending across the north shore of the lake and then across the lake. The bridge location was determined primarily by considerations of traffic revenue, right of way, and the best interests of the two parishes, and secondarily by site conditions on the north shore. Site conditions on the lake and on the south shore did not vary sufficiently to affect the location.

A hydrographic survey was made across the lake by fathometer to develop a continuous profile of the bottom and disclose any unsuspected channels or pits. We found depths of water ranging from 13 to 17 feet and some relatively shallow irregularities probably resulting from dredging operations, but no significant deeps.

The structural design of the bridge posed a special challenge. Alternative designs in steel, reinforced concrete, and prestressed concrete were carefully analyzed for cost and adaption to mass production techniques during a short construction schedule. The construction of over 2200 identical spans justified the considerable investment of $6 million in plant and required utmost attention to details of design and methods of fabrication to minimize labor costs and permit performance of as much work ashore as possible. When bids were taken, the prestressed design was low. The 56-foot-long spans are being precast in three casting beds each of which accommodates eight spans in series providing a combined output varying in recent weeks from 48 to 56 spans per week. Pile caps are precast ashore, transported, positioned, and concreted in place on 2-pile bents, and the deck slabs then set on to form the completed structure. Figure 1 shows a deck slab being lowered into position on a barge ready to be moved to the structure.

Soils Design Considerations

A study of the soils was made to determine their influence on the type, length, bearing capacity, and settlement of the piles needed to support the bridge and on the magnitude and duration of the settlement of fill areas. Consideration was given to the behavior of cohesive soils varying from recent peats (having a natural water content of 399 percent, a liquid limit of 348, a plastic limit of 124, and a wet weight of 67 pounds per cubic foot) to very

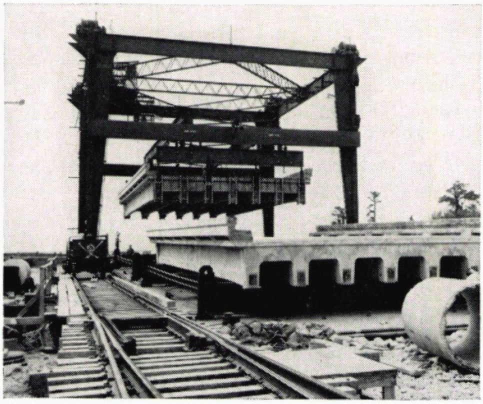

Figure 1. A 56-foot long precast prestressed deck slab being placed on barge.

stiff Pleistocene clays (having a natural water content of 25 percent, a liquid limit of 42, a plastic limit of 17, and a wet weight of 128 pounds per cubic foot). Details of the soils exploration program and the laboratory investigations to establish the properties of the soils have been covered in detail in Part I of this presentation. Let us now see how these data were incorporated into the expressway design.

DETAILED INVESTIGATIONS

Piles

From the information assembled, a clear picture of the foundation problem emerged. Quite consistently we have a depth of water of about 15 feet on the average, then about 25 feet of soft clays and silts, underlaid by sands and clays becoming denser and stiffer with depth down to sound strata at depths of from 65 to 90 feet below water level. A trestle type structure on pile foundations was clearly indicated.

The use of a 54-inch diameter hollow cylindrical prestressed concrete pile was proposed. Piles of this type forming 2-pile bents were considered desirable for several reasons including stiffness, comparatively small number needed, properties of the concrete used, and high bearing capacity.

In July 1953, a 96-foot-long pile of this type was especially manufactured, driven about a mile and a half off the south shore, and loaded as a test pile. The pile penetrated through 15 feet of water and approximately 65 feet of soil, predominantly clays and silty clays varying in consistency from soft to very stiff, before terminating in a stratum of dense fine silty sand. A special driving hammer having a striking energy of 40,600 ft.-lb. was used. The driving resistance during the last 25 feet was about 100 to 150 blows per foot and during the last 6 inches progressed at a rate of 300 blows per foot until reaching refusal at the rate of 720 blows per foot. A week after being driven, the pile was loaded by dead weights, in 30-ton increments, to a total load of 420 tons; a loading equal to three times the design load. Its total settlement under load was 0.41 inch. The load was removed and the pile rebounded 0.28 inch, leaving a net residual settlement of 0.13 inch. No attempt was made to load the pile to failure. Figure 2 shows this test pile being positioned for driving, and Figure 3 the pile carrying the 420-ton load.

An estimate of the ultimate bearing capacity of the pile was made, based on the results of the laboratory soil tests. The pile was treated as a small pier, which is consistent with its dimensions, and analyzed along the general lines Terzaghi presents for such cases. Skin friction was considered equal to the shearing

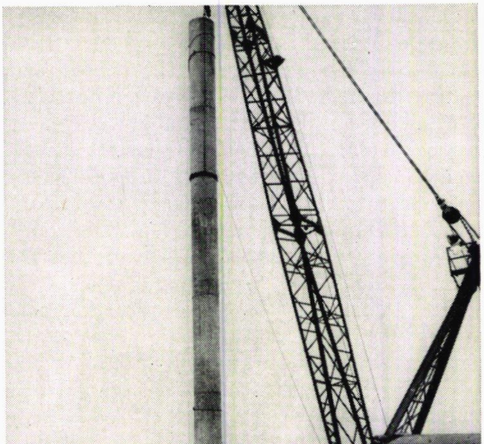

Figure 2. The first 96-foot long, 54-inch diameter test pile being positioned.

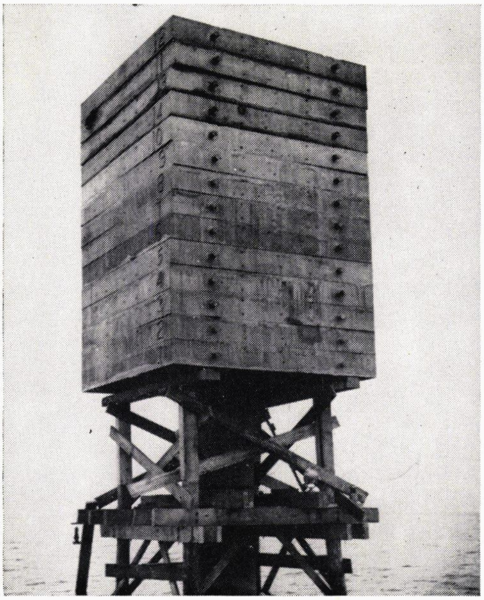

Figure 3. Test pile carrying 420 tons dead weight.

strength of the soil as established by its cohesion developed during unconfined compressive strength tests. An average value of 850 pounds per square foot was determined for the 65-foot embedded length. The problem is complicated by the behavior of the pier in end bearing as affected by the formation of a plug of compacted soil inside the hollow base. At this particular site the total pile bearing capacity was computed to be 870 tons considering the pile to have a solid base; 670 tons considering the internal plug to be not wedged but held only by internal skin friction; and 500 tons considering no internal plug bearing at all.

After the start of bridge construction, five pairs of test piles were driven as part of the contract operations to provide additional pile test data. Each pair consisted of a 54-inch round and a 24-inch square pile driven at locations numbered 1 through 5 from north to south corresponding to Boring Nos. 20, 16, 11, 8, and 5 on the final bridge alignment. A special boring from which continuous undisturbed soil samples were taken was made at each of these five sites. Results of the load tests are shown on Figures 4 and 5. The 54-inch pile at No. 4 location was driven to essential refusal in dense sand and behaved very similarly to the original test pile. It carried 420 tons for one week with a total settlement of 0.35 inch and a net settlement of 0.14 inch. The corresponding 24-inch square pile settled 0.37 inch under a 300-ton load and recovered 0.35 inch upon removal of the load.

Piles at the other four locations were terminated in materials as follows: No. 1—stiff clay; No. 2—loose to medium dense silty sand; No. 3—medium stiff silty clay; No. 5—medium dense clayey or sandy silt. At these locations the 54-inch diameter piles failed in bearing at loads of 300 to 360 tons. At the No. 2 location the pile was left standing under a load of 210 tons for 32 days to note its time-settlement behavior. All of the measurable settlement took place during the first two days after loading, as follows: 0 days— 0.105 inch; 1 day— 0.122 inch; 2 days —0.125 inch, with no further change to 32 days. The pile was then unloaded, rebounding 0.087 inch leaving a net settlement of 0.038 inch, and reloaded to failure at 300 tons. Settlement readings were

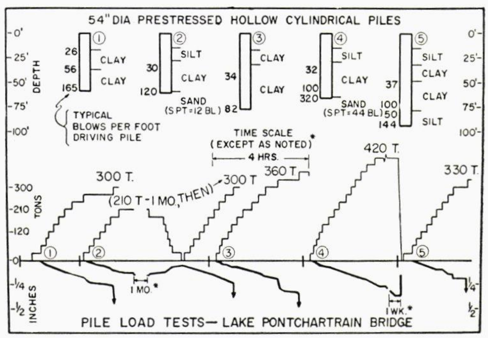

Figure 4. Summary of behavior of five 54-inch diameter test piles in Lake Pontchartrain.

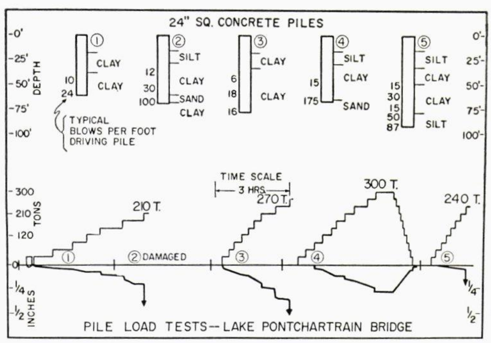

Figure 5. Summary of behavior of five 24-inch square test piles in Lake Pontchartrain.

made by several methods; values shown are from 0.001-inch graduated dial gages. A comparison was made between the actual failure load and the computed bearing capacity of the four large piles that could be loaded to failure. Computed values were higher than observed values up to a maximum of twice as high. The greatest differences occurred where the pile penetrated through many feet of stiff clays. In such cases the contribution of the computed external skin friction was unrealistically high, at least when based on undisturbed rather than remolded strength values.

At the time of writing (December 1955), pile driving operations have advanced about half way across the lake. This distance represents nearly 2200 piles, roughly 200,000 linear feet. Figure 6 shows one of these piles being driven by the 48,750 ft.-lb. energy hammer used. There has been surprisingly little trouble in obtaining piles having the necessary bearing capacity and placed accurately in position. The greatest single problem, on the contractor's part, has been the selection of the correct length of pile to use to avoid excessive cut-off or extension. Additional borings by the contractor and behavior of piles immediately driven afford only limited help in selecting the length of the next pile to be driven. In spite of the contractor's feeling that no two piles in the lake drove the same, certain area trends are evident in long runs (10–100 bents) with no change in pile length. Figure 7 shows the cap-placing operations while in the background the pile-cutting crews may be seen. In some cases piles reach essential refusal, say 200 blows per foot, sooner than expected. In many cases driving can be stopped at exact cut-off elevation with the pile developing an adequate penetration resistance of 96 blows per foot or greater. In a few cases driving must continue through a portion of the 12 feet above water surface that piles normally extend to reach their cut-off elevation. Piles are customarily manufactured in lengths of 96, 88, 80, and 72 feet. Criteria for acceptability of piles include a minimum driven tip elevation of −55 feet, and a minimum penetration resistance of eight blows per inch or an acceptable load test under a minimum dead weight of 210 tons. Scores of load tests on the structure piles, many more rigorous than required, confirmed the adequacy of these criteria. Figure 8 shows load testing of one of the structural piles. The use of pre-jetting to prevent piles from "taking up" before reaching the minimum acceptable penetration has been quite successful. The four-jet unit used by the contractor is exceptionally well rigged for close control and may be used before or during driving. Figure 9 shows two piles being driven with the jets in operation. The laminated oak cush-

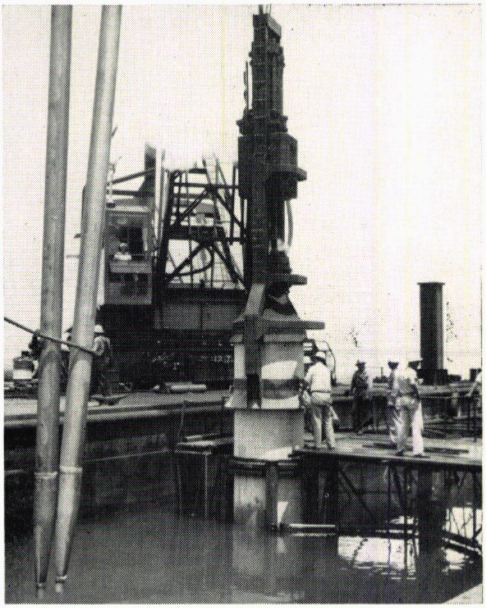

Figure 6. Driving a 54-inch diameter pile to grade with Raymond 4/0 hammer.

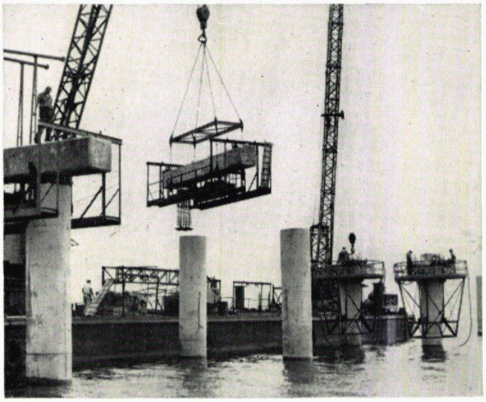

Figure 7. Cutting of piles to grade and placing precast bent caps.

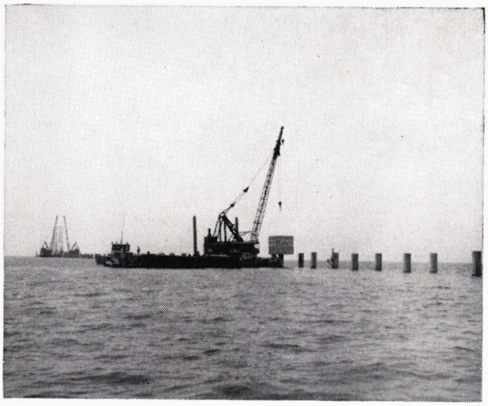

Figure 8. Load testing a structural pile to 270 tons.

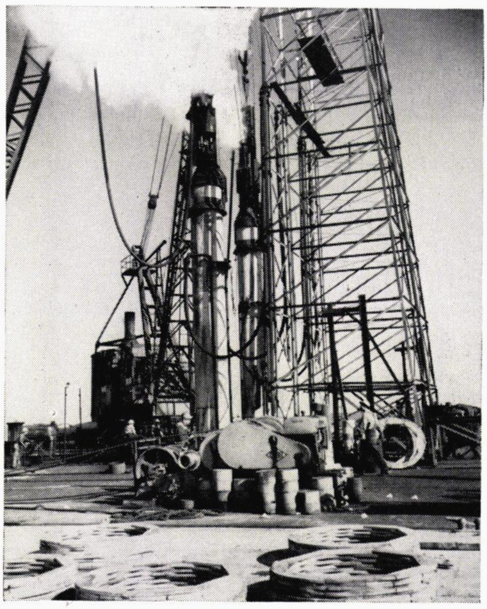

Figure 9. Driving two piles with assistance from water jets.

ion blocks used to protect the tops of the piles may also be seen.

Island and Turnaround

A fill in the lake forming a large artificial island over which the bridge would pass was proposed about nine miles from the south shore. The expected settlement of the island was investigated. Borings disclosed soft highly compressible clay down to 40 feet below water surface. It was planned that this clay would be removed and replaced with fill material extending 12 feet above the water. Clays below −40 are generally medium stiff or stiff and frequently contain sand layers and pockets. Typical stratification, soil properties, and a summary of the time settlement behavior expected of these underlying clays are shown in Figure 10. A settlement of approximately one-half foot occurring over a period of 30 years was indicated. Consolidation of the soil below −60, the bottom of the borings, would increase this value somewhat. Total settlement was expected to increase only slightly, however, since adjacent deeper borings indicated sands and stiff clays in lower strata.

This island fill was later replaced in the final design with pile supported turnaround ramps.

Bridge Terminals

North Plaza. The transition between the pile-supported bridge and the normal roadway takes place on a plaza fill area at each bridgehead. Steps were taken to develop as much of the expected fill settlement as possible during the construction period in order to minimize later roadway subsidence. At the north plaza area an additional 10 to 15 feet of surcharge fill, above finished grade, was provided. This surcharge remained in place for 90 days by which time a maximum of almost 1.5 feet settlement had occurred and further settlement had essentially ceased. Figure 11 shows a simplified diagram of the plaza area and the settlement plate records. The greater settlement on the east side resulted from a thicker layer of more recently deposited sediments filling an old dredged slip in that area. The computed ultimate settlement, based on the typical stratification as shown, which corresponds fairly well with the materials on the west side, was slightly more than 0.6 foot; a fair agreement with the observed value. Observed time of settlement was much more rapid than the computed time. This probably results from the pronounced horizontal stratification and the presence of thin sand and silt layers not disclosed by the boring. Figure 12 shows a side view of the fill, about 30 feet high at the "bump" and 25 feet elsewhere. The aerial view in Figure 13 was taken after the surcharge had been removed and the end bridge span placed.

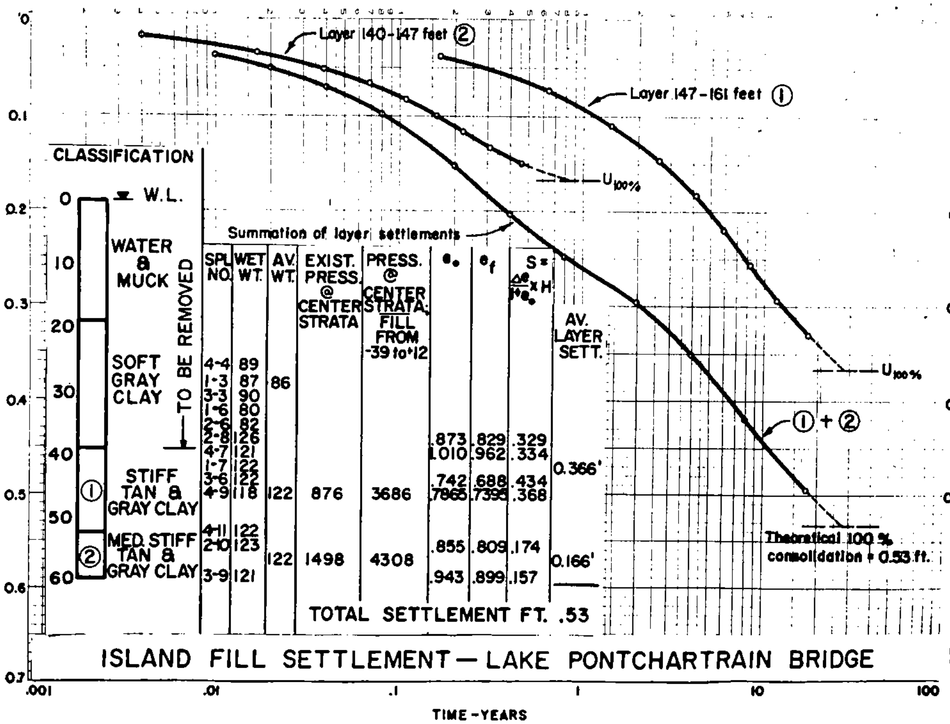

Figure 10. Predicted settlement behavior of proposed island fill.

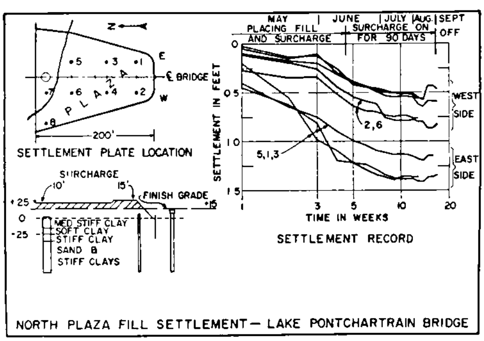

Figure 11. Settlement behavior of plaza fill at north bridgehead.

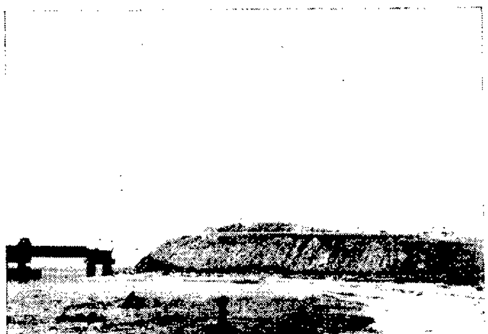
Figure 12. North plaza fill including 10-foot high surcharge and short length of 15-foot high surcharge.

South Plaza. The south plaza fill was located on much more unfavorable material than the north plaza in both the original and final alignment of the bridge. The fill itself is a peninsula extending northward about 700 feet into the lake and tapering from approximately 300 feet to 150 feet in width. It also extends southward 400 feet from the shoreline, across a 300-foot-wide levee and dropping to grade in another 100 feet. Vertical sand drains were provided for this entire plaza area as well as a surcharge of 10 feet additional fill.

At the plaza location on the original alignment the computed ultimate settlement was 9.1 feet under the combined weight of fill and

Figure 13. North plaza with surcharge removed and sea wall in place.

surcharge. The construction contemplated the complete prior removal of a peat deposit down to approximately elevation −15. Soft clays contributing to the settlement extended from elevation −15 to −70.

Soil conditions at the present plaza location are somewhat better. Soft clays extend only to elevation −54 on the typical log. Settlement computations were based on a construction procedure which contemplated the removal of soft material down to elevation −15, backfilling to elevation 22 incorporating sand drains (16 inches in diameter on 10-foot centers) in the fill and a 10-foot high surcharge on top of the fill, a waiting period for settlement to take place, and removal of the surcharge to the grade elevation of about elevation 12. Figure 14 shows a plan view and section of the plaza area. Computations indicated a total expected settlement of 4.2 feet resulting from the weight of the fill and of 5.9 feet resulting from the

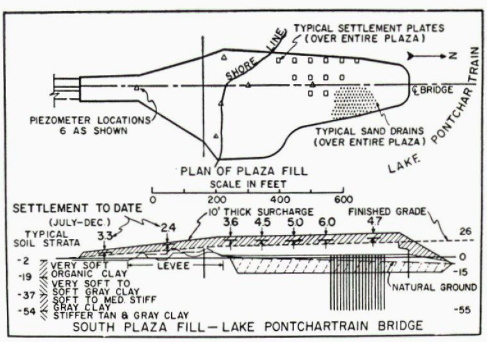

Figure 14. South plaza fill showing plan and section.

weight of fill plus surcharge. A record of the actual settlement of the fill taken from field data is shown on Figures 14 and 15. At the time of writing the surcharge fill is still in place. Settlement plates in the lake portion of the plaza were not placed until the fill, of clam

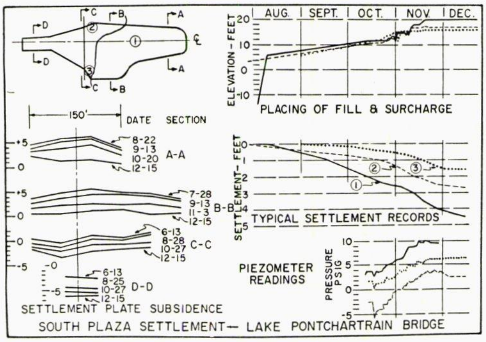

Figure 15. Typical settlement plate and piezometer reading records at south plaza.

shell to a few feet above water, had been built up from elevation −15 to about elevation +4. Settlement during this period was estimated from borings made to the bottom of the shell fill. This fill had been placed on a sand blanket carefully leveled off at elevation −14.

Generalizing broadly, the settlement has been 4 feet south of the levee, 3 feet under the levee, and 5 to 6 feet in the lake fill. This is in fair agreement with the computed values if the rate of settlement is considerably advanced over the computed rate. Indications are that this is the case but further observation of the time-settlement curve will be most informative. Piezometer readings showing pore-water pressure variations have been somewhat erratic. The average increase in pressure has been compatible with the increase in overburden height. Individual readings have no doubt been influenced by the distance and performance of the closest sand drain. The use of portions of the plaza areas as a storage yard for concrete sheet piling for the surrounding sea wall has also caused local variations in readings. In some cases a definite downward trend has been leveled out or even reversed by this loading. Figure 16 is an aerial view of the south plaza taken while the placing of the fill and of the sand drains was in progress.

It may be of interest to note these few facts concerning the placing of the sand drains. A layer of one to two feet of a more easily compactible silty sand was placed on top of the uniform sand constituting the sand blanket. This provided the contractor a better working surface, at about elevation 8 or 9, over which his equipment worked. The hollow closed mandrel almost dropped under its own weight through the soft clay strata—say a foot or two to the blow. Penetration resistance increased rapidly when the stiff clays near elevation −52 were reached. Driving was stopped at around 10 blows per foot to keep the mandrel from hanging up. Hardest driving, around 40 blows per foot, was encountered while driving through consolidated organic soils under the existing levee. The shell fill was penetrated at about 25 to 30 blows per foot. The sand column was held in place in the ground by 125 pounds per square inch steam pressure as the mandrel was withdrawn. Figures 17 and 18 show the final stages in driving the mandrel just before loading with sand from the tip-up hopper, and a view of a completed sand drain. The 2000 sand drains were placed at a cost of less than $0.50 per foot.

Approaches

Jefferson Parish, between the south shore of Lake Pontchartrain and the Mississippi River, is surrounded by levees and drained by a system of canals discharging through pumping stations into the lake. A general area subsidence has taken place over the years caused by the lowering of the water table. Often only a foot or two thick surface crust covers the compressible upper deposits of peats and soft clays. As may be imagined, the construction of stable highways over such ground is a problem.

Figure 16. Aerial view of south plaza construction.

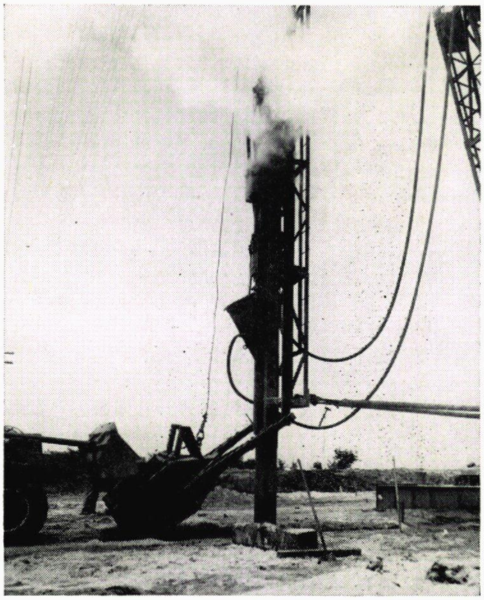

Figure 17. Driving of 16-inch diameter sand drain mandrel to 55 feet below lake level.

Figure 18. View of completed sand drain at fill surface.

The basic method of construction adopted for the south approaches was as follows. Five feet, or in some cases, 8 feet of peat were removed to form a trench the width of the roadway which was backfilled with a silty sand obtained from the Bonnet Carre Spillway. The forward 200 feet or so of this advancing fill was kept built up higher than the rest to provide a 5-foot thick surcharge. The weight of the nose of the surcharge displaced soft pockets or layers of peat ahead into the open trench where they were removed by clam shell or dragline. On any section of roadway the length of time the surcharge was present was sufficient to develop essentially all of the consolidation that any remaining peat layer would undergo. No significant consolidation was developed in the relatively impermeable clays, of course, from this short term surcharge loading.

Various modifications of this general approach were used. In some cases the surcharge was deliberately held back a few hundred feet from the end of the advancing fill to prevent developing deep rotational type failures. Such failures tended to occur if the 2 or 3 feet of material below the bottom of the 5-foot-deep excavation were of comparatively greater shear strength than the soil on which it rested. Examples would be a system of fibrous peat overlying soft clay or a system of sandy silt overlying soft clay. In other cases an additional foot or two excavation was performed to remove compressible clays or peats completely. A fixed surcharge of a few feet was provided for several months at the ends of the small bridges crossing the drainage canals on Causeway Boulevard (formerly Harlem Avenue). Settlements of 1½ to 2 feet in a 3-month period were observed here.

Major pile-supported traffic interchange structures located at the intersection of Causeway Boulevard with the Airline Highway and with Jefferson Highway were required. Piles for the approach structures have generally

Figure 19. Aerial view of construction of "3-prong" access roads converging on north bridge approach.

been designed to penetrate a few feet into the Pleistocene clays or sands usually located 60 to 90 feet below ground surface. Such piles, depending upon their type, are assigned design loads of from 30 to 40 tons. The first test pile at the Airline Highway site (8-inch tip, 14-inch butt, 90-foot penetration) carried 70 tons with $\frac{1}{16}$-inch gross settlement over 48 hours and $\frac{1}{64}$-inch net settlement. It carried 100 tons with $\frac{11}{64}$-inch gross settlement and $\frac{2}{64}$-inch net settlement after unloading. Piles driven through the upper sedimentary clays around the New Orleans area seem to develop more than the usual increase in penetration resistance when given a chance to set-up for a short period of time. At a depth of 80 feet this particular pile increased from 9 to 60 blows per foot resistance after an 18-hour delay in driving. Many other similar examples could be cited where shorter delays in driving were involved. Bearing capacities based on dynamic formulas such as the ENR, using these higher values, are still conservative when compared with piles load-tested to failure, although not by a factor of six.

On the north side of Lake Pontchartrain, in St. Tammany Parish, subsurface conditions are quite different and much better than on the south shore. The whole area lies on the exposed Prairie Terrace, the latest deposits of the Pleistocene Age, and stiff clays and sands are the soils most frequently encountered. Local materials along the right of way provided satisfactory sub-base and base course materials. A view of the general area showing early construction of the approach roads converging on the north bridgehead is shown in Figure 19.

ACKNOWLEDGMENTS

Design and supervision of construction of the Greater New Orleans Expressway has been performed by the firm of Palmer and

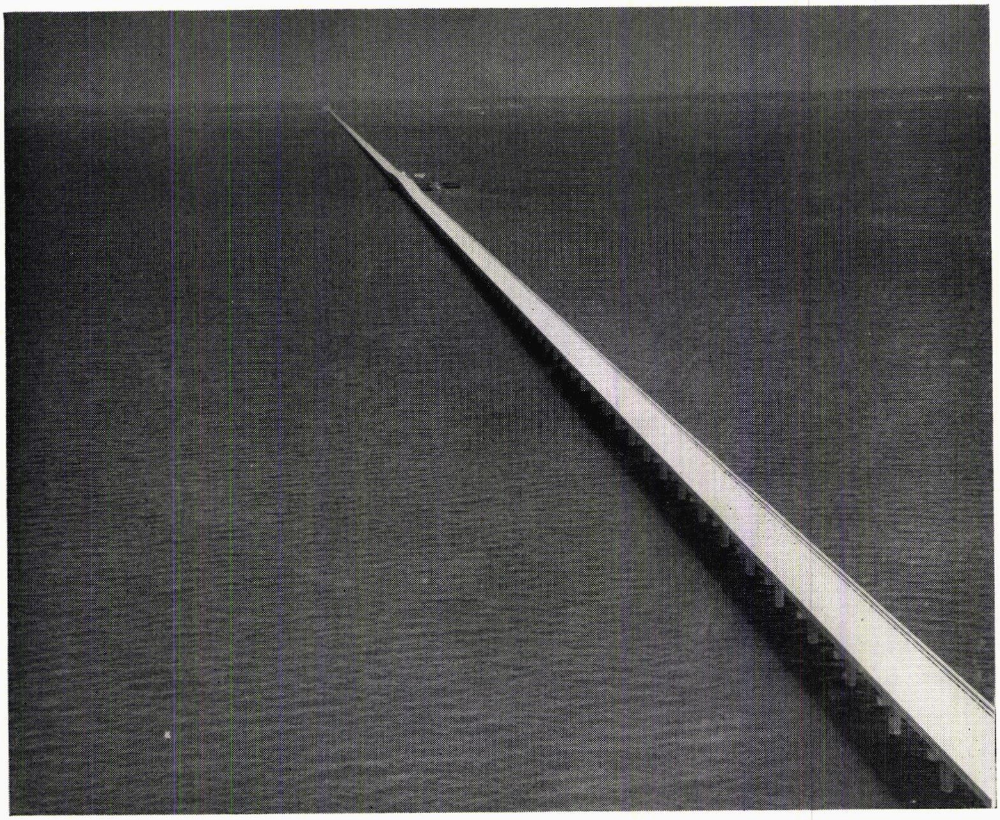

Figure 20. View of Lake Pontchartrain Bridge looking towards north shore.

Baker, Inc., Consulting Engineers, of Mobile, Ala. The contractor for the Lake Pontchartrain Bridge and portions of the approach road system is the Louisiana Bridge Company—a joint venture of Brown and Root, Inc., of Houston, Texas, handling the bridge construction, and T. L. James and Co., Inc., of Ruston, Louisiana, handling the approach road construction. Contractor for the traffic interchange at the Airline Highway is the Louisiana Paving Company.

Figure 20 shows an aerial view of a portion of the structure already completed, reflecting the good efforts of all who have contributed to its success but who are not mentioned here by name.