Measurement of Forces Produced in Piles By Settlement of Adjacent Soil

EDWARD V. GANT and JACK E. STEPHENS, University of Connecticut; and
LYLE K. MOULTON, Connecticut State Highway Department

● SETTLEMENT of a soil layer through which a pile penetrates in passing to a firm stratum will tend to transfer load to the pile by negative skin friction. That is, the action of the pile is to support the settling soil, and the direction of the stresses acting on the surface of the pile is downward. The drag of the soil on the pile can produce forces of large magnitude; therefore, this type of loading may be an important consideration in the design of a pile foundation.

The most common loading condition which can lead to drag forces of considerable magnitude is the case in which a fill is placed over a compressible layer, and piles are driven through this layer to a firm stratum prior to complete consolidation of the soil. A similar condition will exist if the fill is placed after the piles are driven. Chellis (2) lists several examples of pile failure attributed to negative friction and Moore (7) describes a failure of a pile under an estimated drag load of nearly 200 tons. Florentin and L'Heriteau (5) also report a case of pile failure produced by negative friction and indicate that the magnitude of the drag forces may be large.

Design of pile foundations for drag forces is sometimes based on a single pile analysis, such as the method suggested by Moore (7), or the design may be based on a cluster or group pile analysis as outlined by Terzaghi and Peck (13). In either event there may be considerable uncertainty concerning some of the design factors such as lateral soil pressure, friction between the piles and soil, bond between cohesive soils and piles, influence of pile driving on shear strength of soil, and the extent to which shear strength of the soil is mobilized.

At the site of a proposed abutment for a bridge on the Connecticut Turnpike a fill approximately 50 ft high was placed on top of a layer of marine mud. It was anticipated that consolidation of the soft mud layer would not be complete at the time the supporting piles for the abutment were driven to a firmer stratum underlying the mud. Therefore, a particularly severe loading condition for drag forces might be imposed on the piles. In view of the many uncertainties involved in the design of the pile foundation for drag forces, an experimental investigation of the nature and magnitude of these forces was made. This paper describes the investigation which was carried out as a joint research project of the Connecticut State Highway Department and the Civil Engineering Department of the University of Connecticut. In July 1955 three 12-in. Monotube fluted piles equipped with electric strain gages on the inside surfaces were driven at the site of the bridge abutment. Readings of the strain gages were taken over a period of several months, and the corresponding drag loads in the piles were computed. Several borings were made at the site of the tests, and soil tests were made to determine certain changes in the soil characteristics resulting from consolidation of the mud layer. In addition, several control measurements, including pore water readings, were taken.

THE TEST SITE AND SOIL CONDITIONS

The pile tests were conducted at the site of the east abutment of the Connecticut Turnpike crossing of the main line of the New Haven Railroad in West Haven, Connecticut. The crossing consisted of a 6-span steel beam and slab bridge with the five piers and two abutments supported on piles. The particular site selected for the pile drag tests was at the north end of the east abutment (Fig. 1) where three piles equipped with electric strain gages (shown circled by the dotted line) and eight additional piles were driven in July 1955. The original ground elevation at the test site was approximately 6-ft above mean sea level, with a deposit of marine mud underlying the area

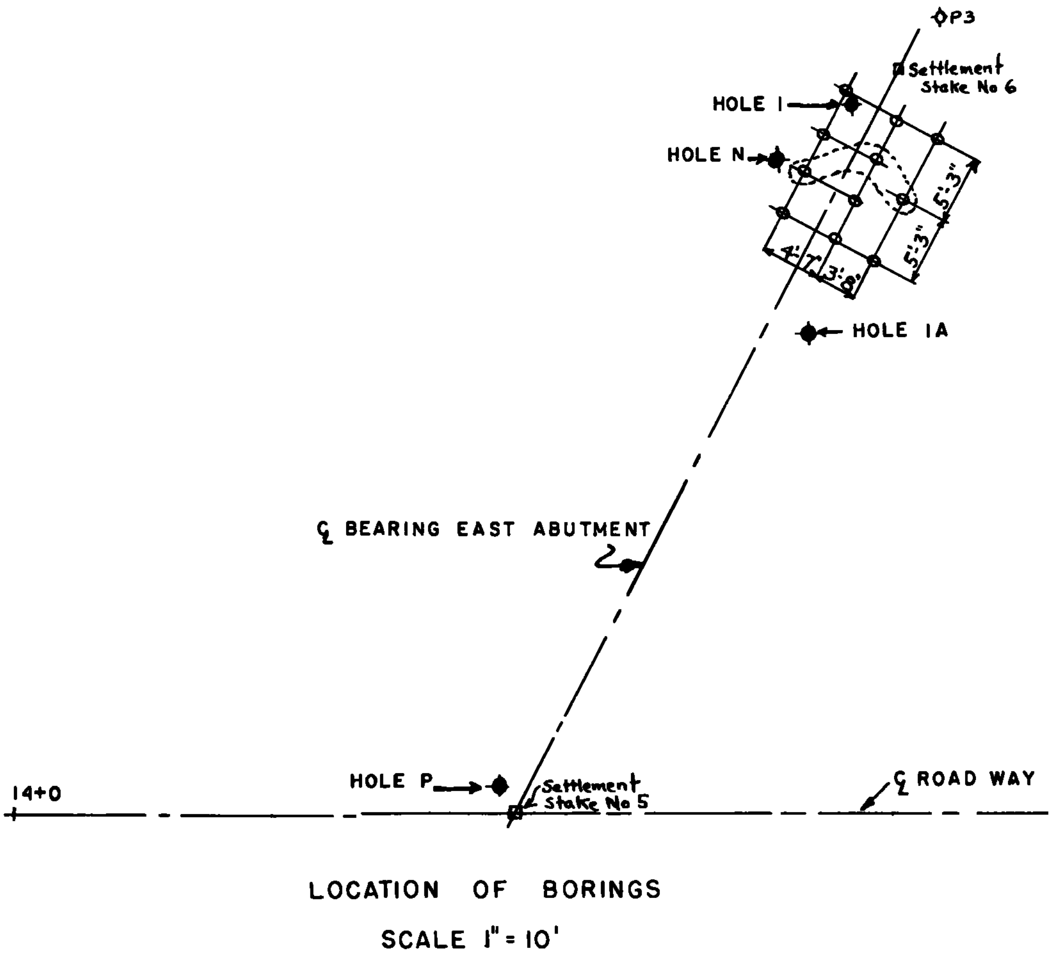

Figure 1. Location plan at east abutment for: test piles; borings for holes N, P, 1, 1A; settlement stakes 5, 6; piezometer P3.

to a depth of about 25 ft below mean sea level. From 1951 to late 1954 fill was placed for the approach to the bridge, finally reaching an elevation of +50 ft including a 5-ft overload. The fill up to elevation +25 ft was hydraulic fill, consisting of medium to fine sand. From elevation +25 ft to final grade the fill was a trucked-in bony glacial till. To expedite settlement of the mud layer, sand drains were installed late in 1951.

The effect of the placement of the embankment on the underlying mud can be seen from Figure 2. By late 1955 over 11 ft of settlement had taken place. The settlements (Fig. 3) at the east abutment (as shown for settlement stakes No. 5 and No. 6, Fig. 1) are larger than at pier No. 5 where the thickness of the mud layer was less and the fill was brought up only to elevation +15 ft. Another important effect of the high fill at the east abutment is that an appreciable settlement took place after the test piles were driven in July 1955.

Starting in 1953 several borings were made near the location of the test pile cluster (Holes N, 1, 1A, Fig. 1). No undisturbed samples were taken in Hole N in 1953; however, undisturbed samples were taken in 1953 in Hole P, near the centerline of the roadway at the east abutment (Fig. 1). Undisturbed samples were taken in Holes 1 and 1A in 1954 and 1955, respectively.

The boring logs (Fig. 4) for the holes at the east abutment show that beneath the fill there is a thin layer of peat and then a layer of clayey silt extending to an elevation of approximately -25 ft. From elevation -25 ft to -38 ft there is a medium sand and then below elevation -38 ft there is a very thick deposit of reddish brown fine sand and silt.

SOIL TESTS

Laboratory tests were performed on samples taken during the borings made in 1953, 1954, and 1955. The tests on the 1953 samples were performed by the Connecticut State Highway Department Laboratory. Tests on samples taken in 1954 and 1955 were performed in the Soil Mechanics Laboratory at the University of Connecticut (12). When the results of the tests on the 1953 and 1954 samples are compared with the results of the 1955 soil tests (Fig. 5) the moisture content at the middle of the clayey silt changed from about 88 percent in 1953 to 70 percent in 1954 and 60 percent in 1955. There was also a considerable change in shear strength as measured by unconfined compression tests. Thus, the cohesion for undisturbed samples changed from about 325 lb per sq ft in 1953 to about 1,200 lb per sq ft in 1955 (the solid line curve, Fig. 5). Cohesion values for remolded samples for 1955 are shown by the dotted line curve. Comparison of the results for undisturbed samples with results for remolded samples shows that the average sensitivity of the clayey silt is 5.33 in 1955. The consolidation tests carried out on the 1955 samples indicate that the consolidation of the clayey silt was still not complete although most of the total settlement had occurred.

In connection with the tests on the samples taken in December 1955 it should be pointed out that the embankment of the east abutment underwent certain changes during 1955. The embankment was benched out in two stages to elevation +35 ft (Fig. 6). Then, in order to drive the piles, the till was excavated to elevation +13 ft, and the area was backfilled with sand in June and July 1955. In November 1955 the fill was excavated to elevation +13 along the entire length of the abutment, except around the

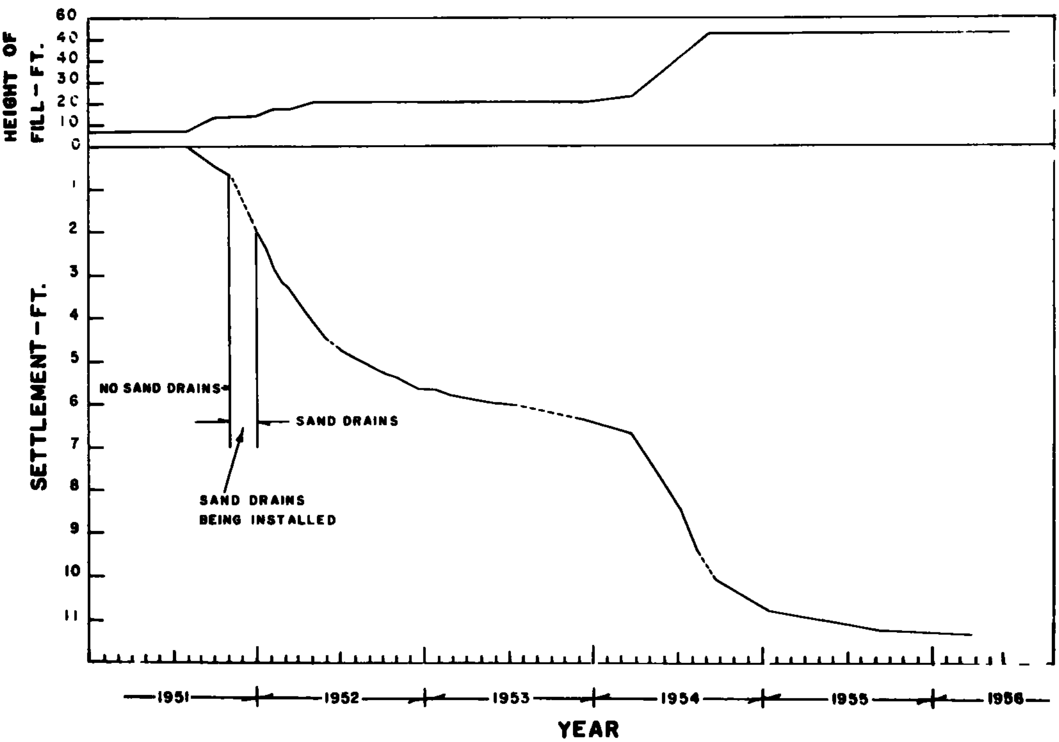

Figure 2. Settlements at the site of the east abutment.

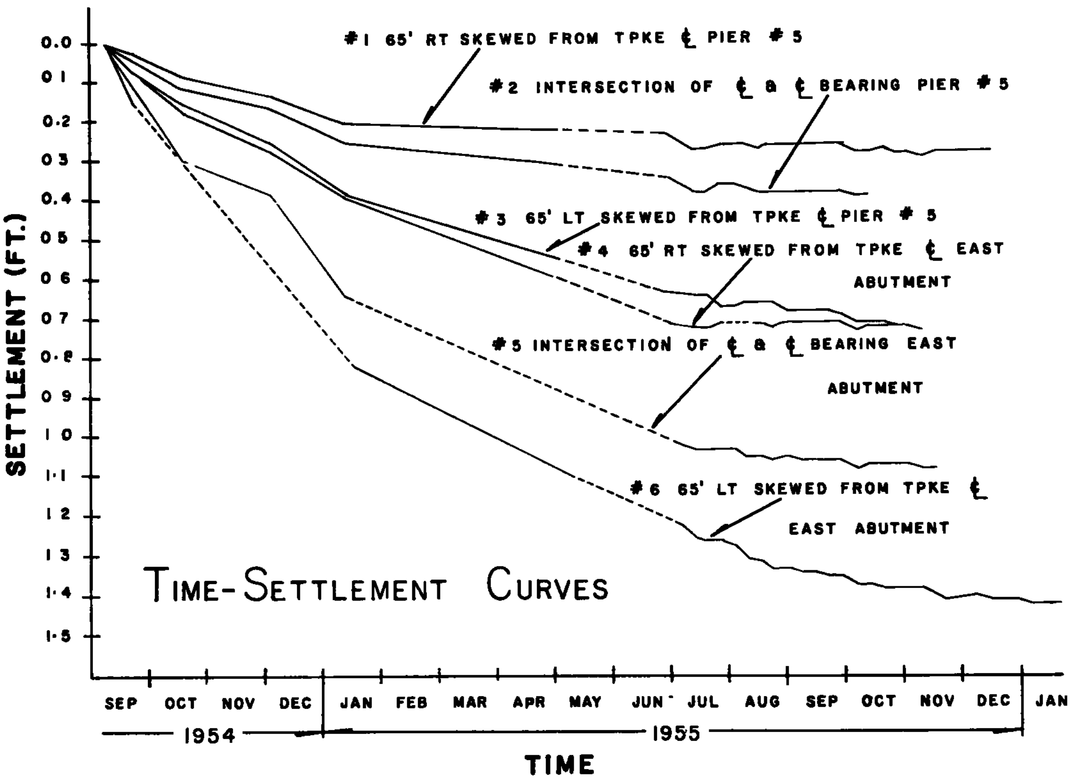

Figure 3. Settlement data for the sites of the east abutment and Pier 5.

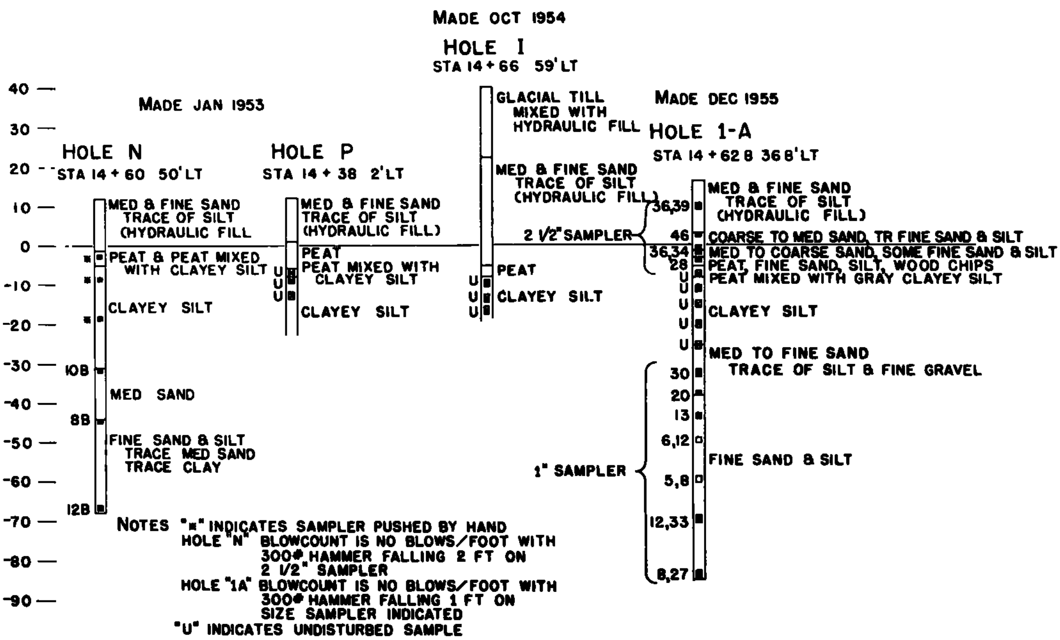

Figure 4. Boring logs for holes N, P, 1 and 1A.

11 test piles, to drive the piles for the abutment.

EXPERIMENTAL PROCEDURE

The initial foundation design for the east abutment was based on three lines of piles, one line of batter and two lines of vertical, to be driven from elevation +35 ft through the fill and mud layers and into the underlying sand. Preliminary estimates indicated that the combined load from drag and the structure might require a penetration below the mud of about 100 ft or a total length of nearly 170 ft. Over half of the estimated penetration was accounted for by anticipated drag forces. In view of the importance

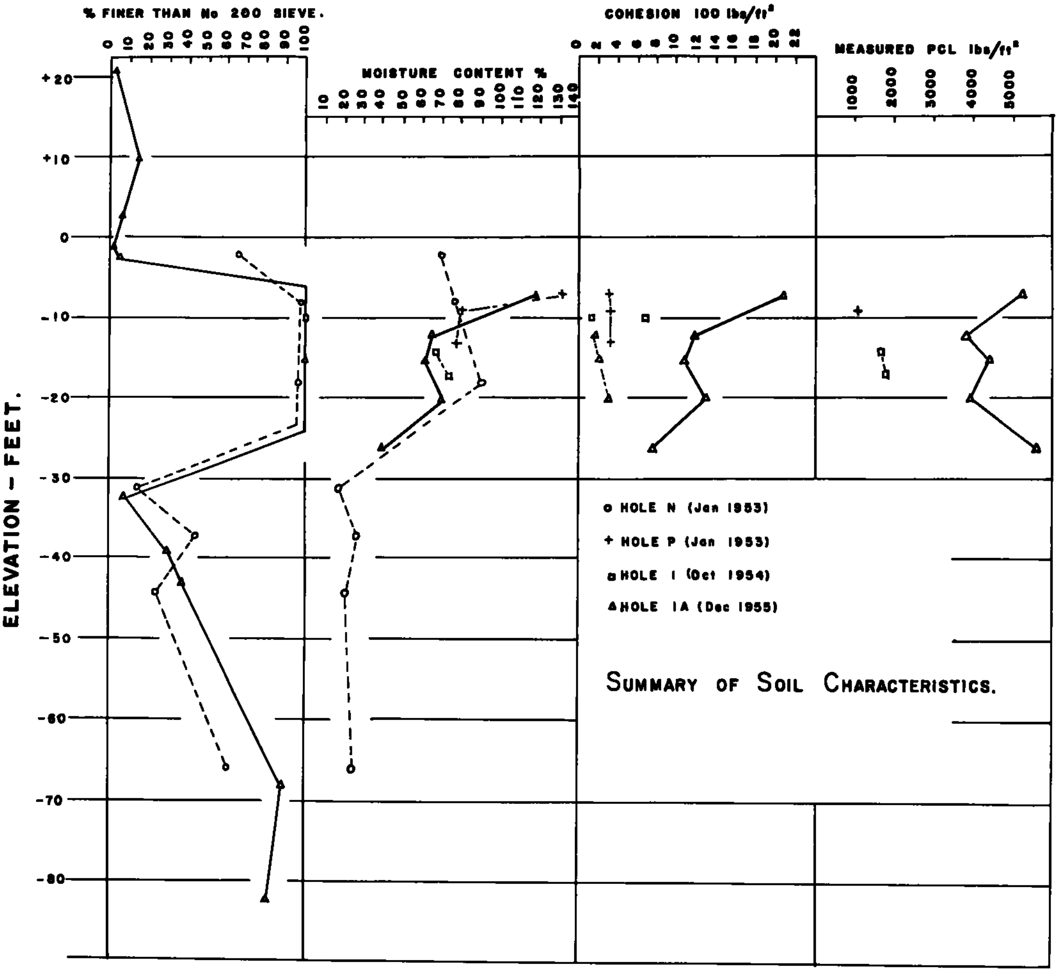

Figure 5.

of drag forces at the West Haven site and the uncertainties in design, it was decided to investigate the nature and magnitude of these forces.

Three experimental piles, two vertical piles and one batter pile, were to be equipped with SR-4 electric strain gages on the inside surfaces and would be part of a cluster of eleven piles driven for group effect. The piles selected were Monotube fluted piles No. 3 gage Type F (tapered) for the nose sections and No. 3 gage Type N 12, 12-in. diameter, for the extensions. The tip diameter was 8 in. Initially it was planned to use for each pile a 30-ft nose section, two 40-ft extensions, and two 30-ft extensions making a total length of 170 ft. However, later one of the 40-ft extensions was omitted.

GAGE INSTALLATION

Several preliminary studies were made to determine the most efficient method of gage installation (10). One method was the use of hand holes cut in the pile at 5-ft spacings. This method had proved satisfactory in the studies by Schlitt (11) and others (9). However, it was abandoned in favor of full length splitting of the pile shells. Although this method required more cutting and welding, it resulted in greater ease of installation and provided for closer inspection of the work. Reese and Seed (8) also used this method in their studies on 6-in. diameter steel pipe piles. The procedure finally adopted was as follows:

1. Each pile section was ripped full length along the center of diametrically opposite flutes so chosen as to avoid the factory weld (Fig. 7).
2. A steel messenger cable was placed under tension in each half of the pile section

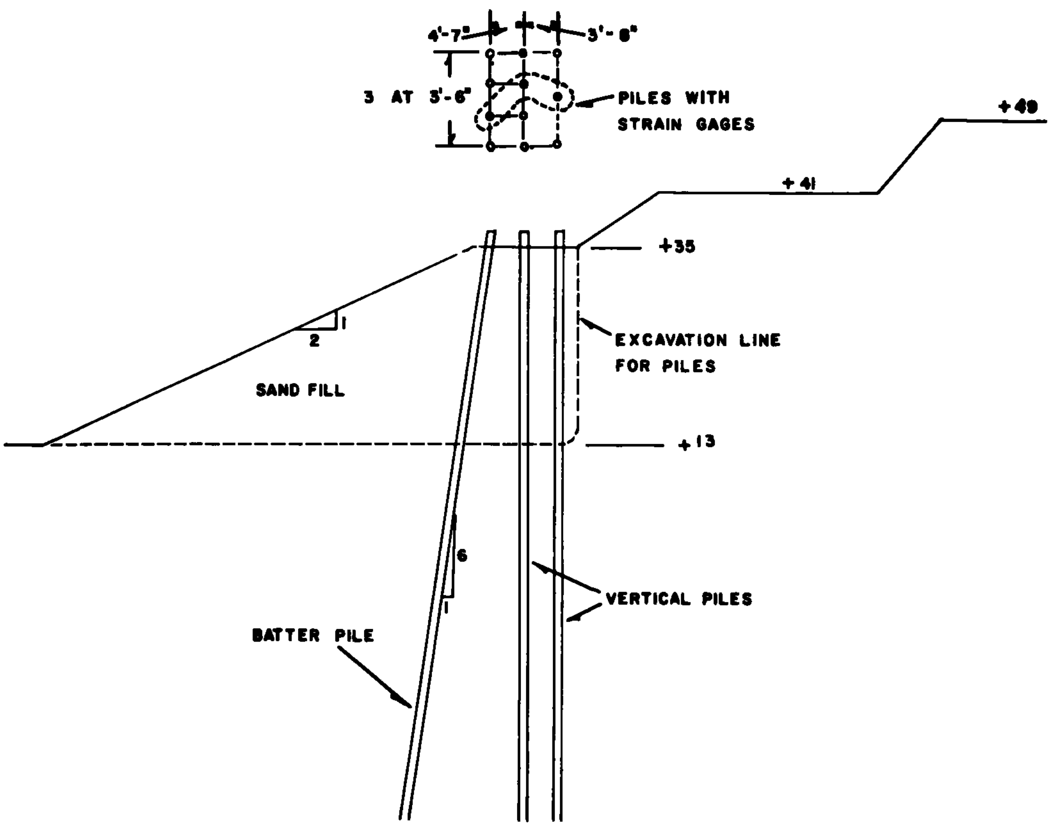

Figure 6. Elevation of the embankment at the east abutment.

to support the gage lead cables.

3. Type A-5 SR-4 electric strain gages were installed with Duco cement at 5-ft intervals along the pile sections for zones expected to produce drag and at 10-ft intervals in other zones. Two active gages were provided at each station with two additional active gages at alternate stations (Fig. 8). A temperature gage was provided at each gage station.
4. After the gages were glued to the shell, they were waterproofed and given mechanical protection. An asphalt paving cement was spread over a gage and fused to the steel by curing with heat lamps, then covered with Armstrong A-2 adhesive and given another coating of asphalt cement.
5. The two halves of the shell were then clamped and adjusted to the original dimensions. Gages were protected by cooling and the shell was welded.

Studies showed that the residual stresses produced by the splitting and rewelding were of small magnitude and would have only a negligible effect on the resistance of the pile to further load. It was also found by test in the laboratory that gages installed in this manner would correctly indicate loads at the gage stations.

Figure 7. Pile section cut in halves lengthwise for gage installation.

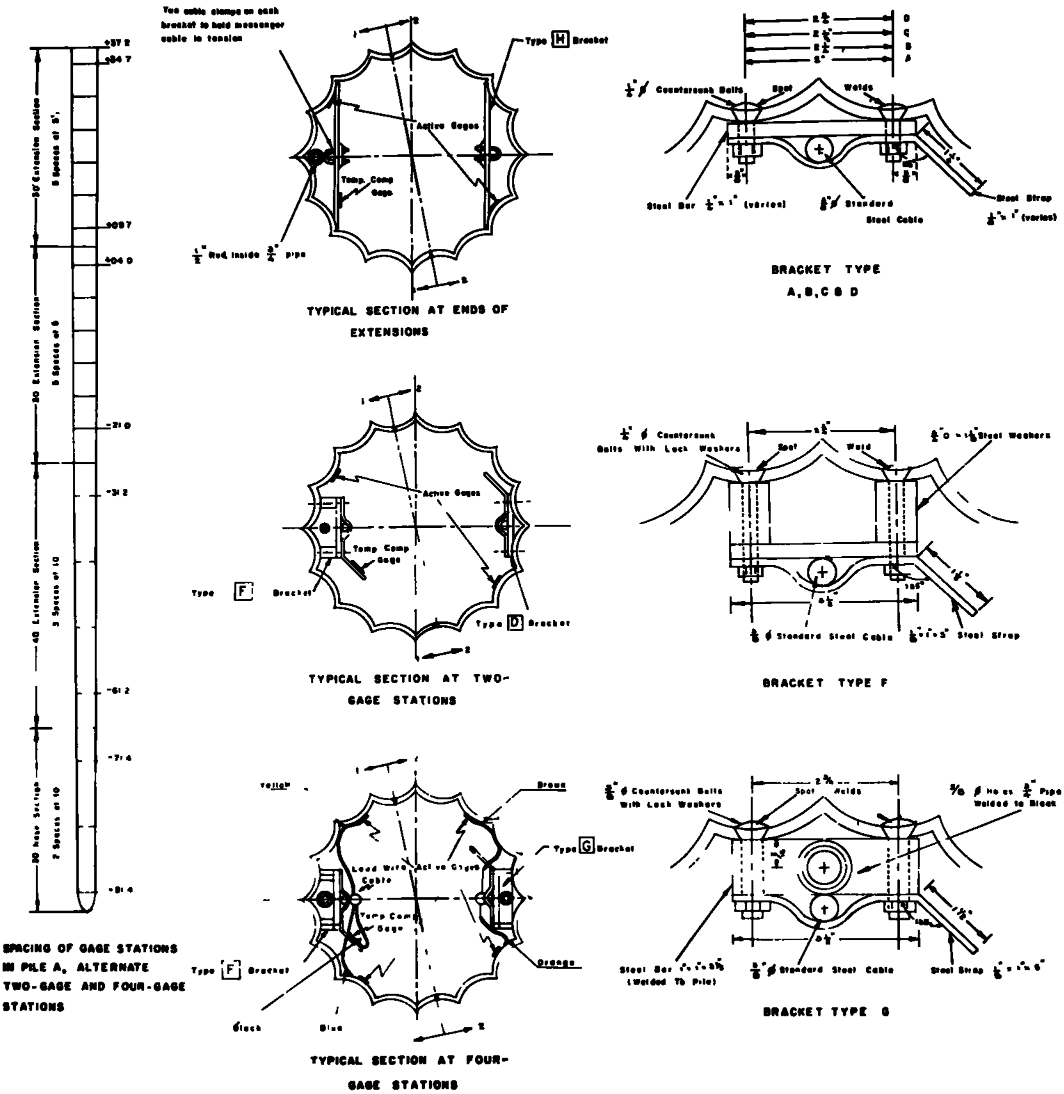

Figure 8. Location of gages along piles as driven and typical sections.

MECHANICAL SYSTEM

As a check on the SR-4 gages, a mechanical system of measuring the strain over 15-ft gage lengths was also installed. This system consisted of rods and pipes starting at different cable brackets and extending to the top of the pile. As each member was fixed to the pile shell at its lower end only, any relative motion between the upper ends of the various rods and pipes would be caused by strain occurring in the pile shell between the levels at which the members were anchored. The upper portion of each rod was located inside a pipe fastened at a higher level and restrained laterally by the higher cable brackets. For ease in field splicing, the rods and pipes installed in any extension section were threaded to connect with the rods in the next section.

Unfortunately, the vibrations produced in the pile sections by hard driving tore the mechanical system loose, and damaged a large number of the SR-4 strain gages.

CONSTRUCTION PHASE

The installation of the gages was carried out at the University of Connecticut, and

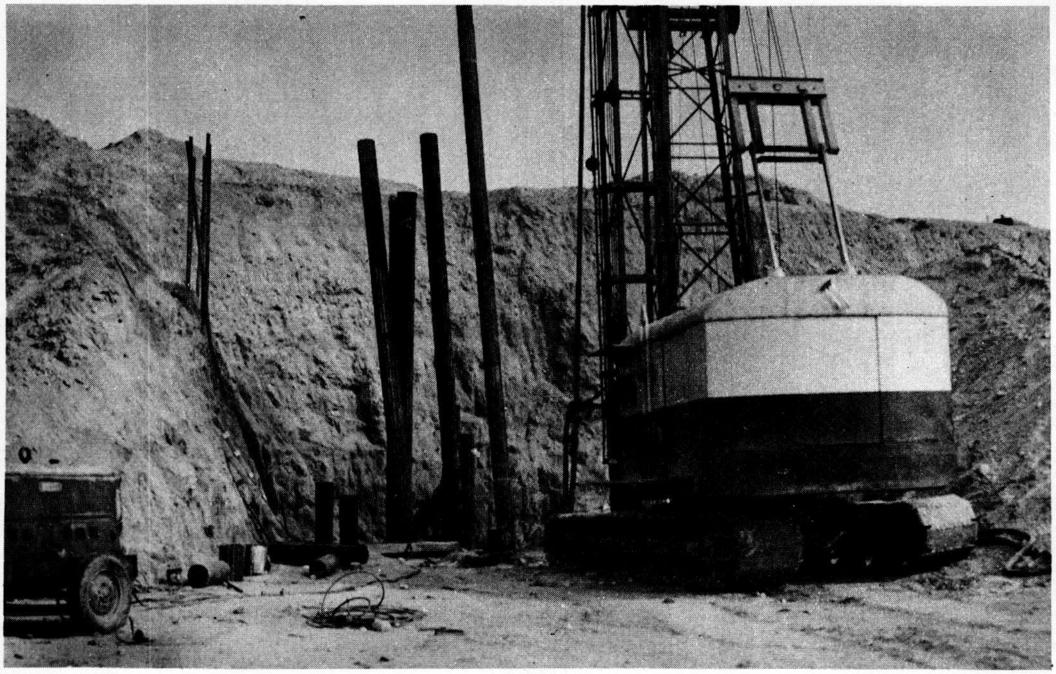

Figure 9. Pile test site during pile driving operations after excavation of embankment to elevation +13 ft.

the sections were trucked to West Haven.

At the job site partial assembly of the pile sections was completed. The cable connectors for the nose and the adjacent extension section for each pile were joined and

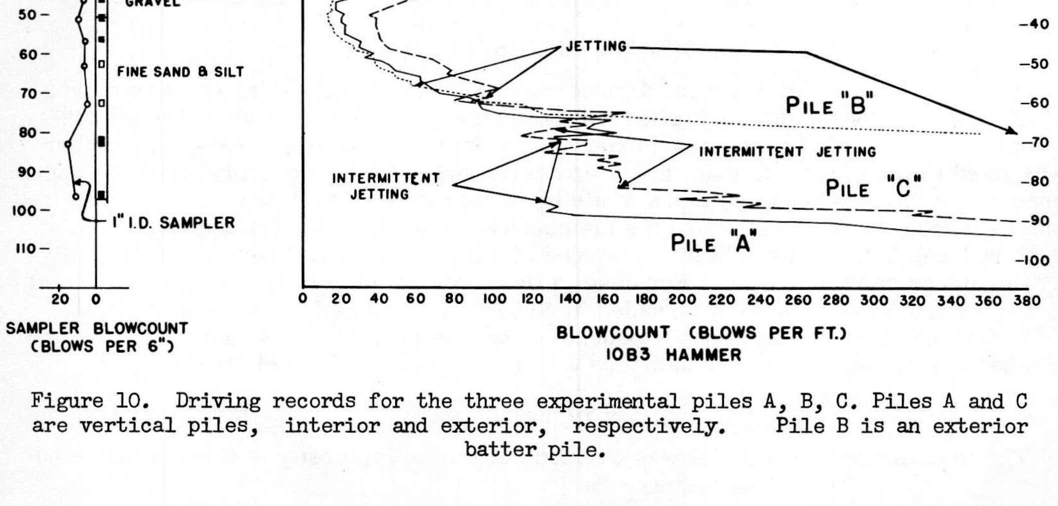

Figure 10. Driving records for the three experimental piles A, B, C. Piles A and C are vertical piles, interior and exterior, respectively. Pile B is an exterior batter pile.

waterproofed, and the welded pile splice was made. The connections between the other two sections of the piles were made in a similar manner. Thus, for driving purposes, each pile consisted of a 70-ft section, which included the nose section, and a 60-ft extension which carried the mechanical strain system. It was necessary to make only one splice in each pile after the driving had begun.

One of the piles in the test pile group which had no strain gages was to be driven initially to determine the best method of driving and any special precautions necessary to keep the damage to the strain gage system to a minimum.

However, the first attempts to drive this pile met with failure, because the upper 13 ft of the fill consisted of very dense trucked-in coarse glacial till. After the contractor had exhausted all his resources in attempting to penetrate the fill, even resorting to jetting and the use of a spud, it was decided to excavate a portion of the fill down to

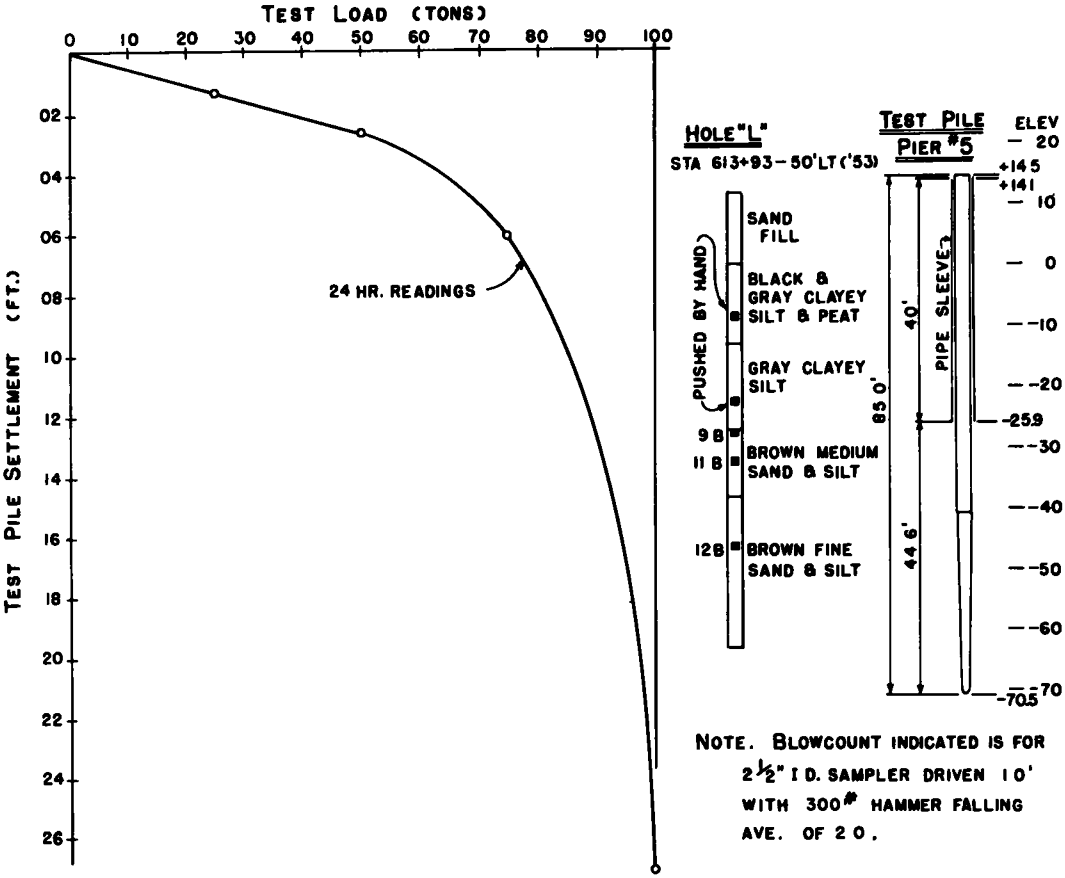

Figure 11. Test pile results at Pier 5.

elevation +13 ft (Fig. 6 and 9). The compactness and density of the upper portion of the fill is illustrated by the fact that it took two days and a total pull of 140 tons to extract the spud driven to 38 ft below the proposed bottom of the footing. After the excavation was completed, the remainder of the pile was driven. Jetting was necessary to start this pile moving after the delay between the initial attempts to drive it and the time when the driving was renewed. The remainder of the pile group including the experimental piles were then driven with a 10B-3 hammer (Fig. 9).

The piles were not driven to any specified formula value, but were driven to a predetermined length. Since the experience with the first pile indicated that some difficulty would be encountered in achieving this desired penetration, it was decided to jet a hole

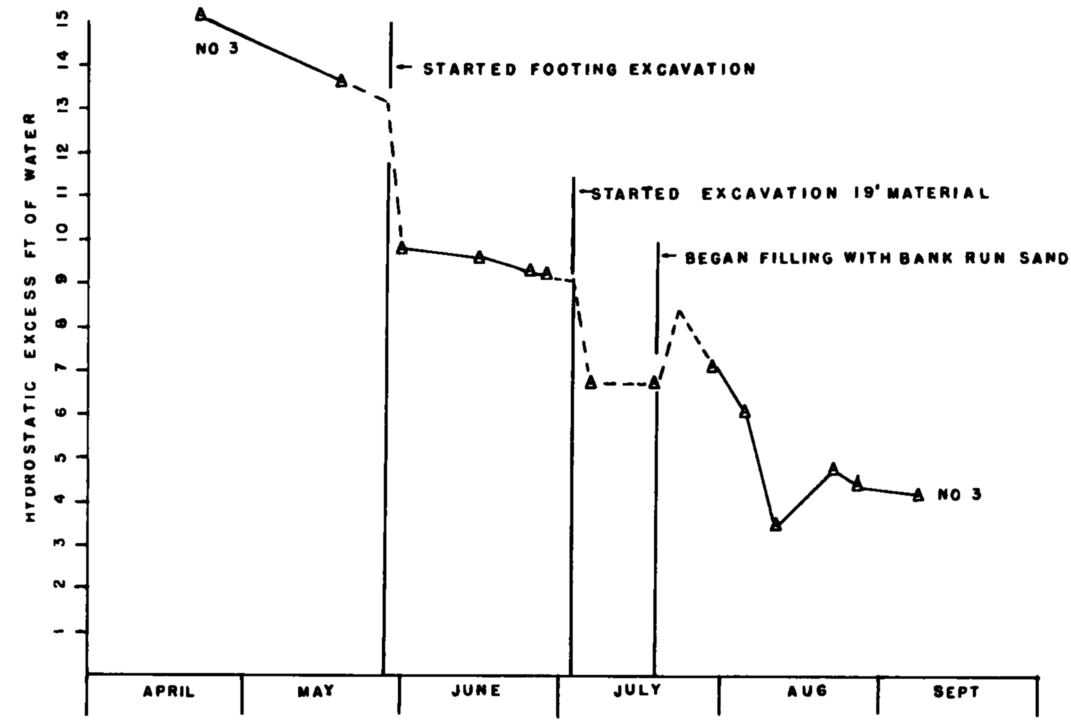

Figure 12. Pore water pressure measurements at the north end of the east abutment, 1955.

90 ft deep for each pile before it was driven. It was later found necessary to resort to jetting adjacent to each pile to minimize the energy absorption and possible damage to the strain gage system. The driving records for the three experimental piles with strain gages are shown in Figure 10.

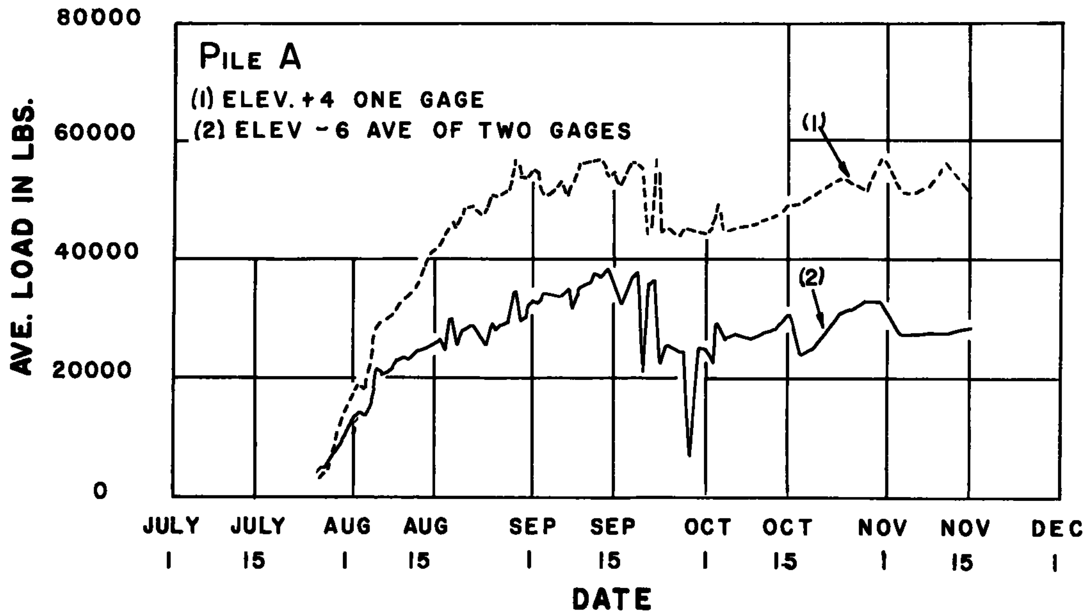

Figure 13. Indicated soil drag loads in Pile A at elevations +4 ft and -6 ft.

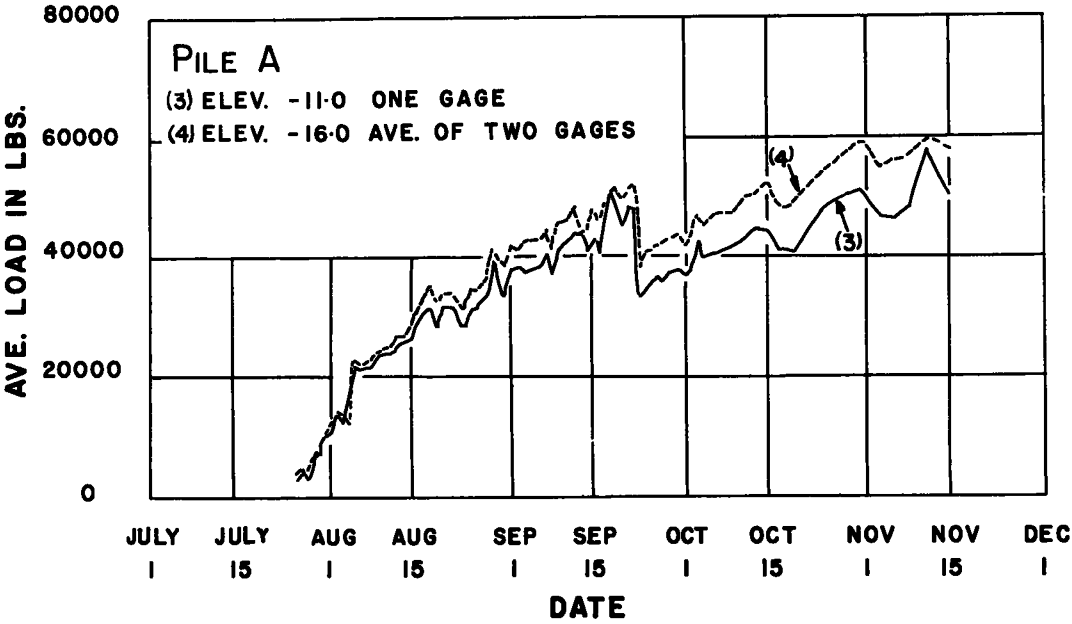

Figure 14. Indicated soil drag loads in Pile A at elevations -11 ft and -16 ft.

GAGE READINGS

The three experimental piles were driven between July 12 and July 14, 1955. The area around the piles from elevation +13 ft to +35 ft was then backfilled with bank run sand and compacted (Fig. 6). The resistances of the electric strain gages to ground were checked and connections were made to an instrument shed which housed the balancing and switching units. On July 25 the gages were zeroed in and readings were begun on July 26, 1955. In order to minimize drift a DC bridge was used (Anderson Model 301 Strainmeter).

Readings of all active gages were made daily for a period of several weeks and then two or three times per week. Around the middle of November 1955 excavation was started at the east abutment in preparation for the driving of the supporting piles. The material around the upper part of the experimental piles slumped as a result of the construction operations. Since the loading conditions had been changed by the partial removal of the fill, regular reading of the gages was discontinued at this point.

TEST PILES

Two special pile load tests were performed at piers near the east abutment to obtain definite values of the frictional resistance offered by the soil underlying the mud. One was near the south end of Pier No. 4, and the other was near the north end of Pier No. 5. These piles were load tested in four load increments to 82 tons and 100 tons, respectively. To eliminate possible effects of the fill and clayey silt on the test results, the piles were driven through 18-in. pipe sleeves which had previously been driven to the bottom of the clayey silt and cleaned out. The test pile at Pier No. 4 was driven to a final blow count of 64 blows per foot for the last foot and the one at Pier No. 5 was driven to a final blow count of 95 blows per foot, both with a 10B3 hammer. The load test results for the pile driven at Pier No. 5 are shown in Figure 11.

CONTROLS

At the same time that the planning stages of the project were initiated, controls were installed at the project site. These included settlement stakes at each end and the center of Pier No. 5 and the east abutment (Fig. 2 and 3) and a cluster of six piezometers adjacent to the north end of the east abutment. Three of these piezometers were installed at a radius of 1.5 ft from the theoretical location of a 20-in. diameter sand drainage well and at the approximate quarter points in the clayey silt layer. Three more were installed at an approximate radius of 3 ft from the sand well and were spaced vertically in the same manner as the other three. Both the settlement records shown in Figures 2 and 3 and the pore water pressure measurements illustrated in Figure 12 indicate that considerable consolidation of the mud layer took place after the test piles were driven.

Readings were made on the settlement stakes and piezometers until they were either destroyed or made inaccessible by construction operations.

TEST RESULTS

Readings of individual strain gages were converted to stress and then the average stress at each elevation was multiplied by the corresponding steel area to obtain the indicated drag load. Initially, gages were distributed throughout the length of the three experimental piles (Fig. 8). However, a large number of gages were damaged when the mechanical system failed during driving operations. In addition, some gages ceased to operate satisfactorily during the course of the investigation although, in general, the methods of gage installation and waterproofing proved to be satisfactory for long term tests.

The indicated drag loads for pile A, the interior vertical in Figure 6, are plotted in Figures 13, 14, 15, 16 as a function of time. Figure 17 shows the distribution of the drag loads for three different dates approximately six weeks apart. In general, the distribution of load for the exterior vertical and batter piles followed a pattern similar to that shown for pile A. During the period November 15-29, 1955, fill was removed at the east abutment to permit driving of the entire pile foundation of about 100 piles. The fill was excavated to elevation +13 ft up to the face of the cluster of 11 test piles and as a result the sand fill between the test piles slumped to an average elevation of

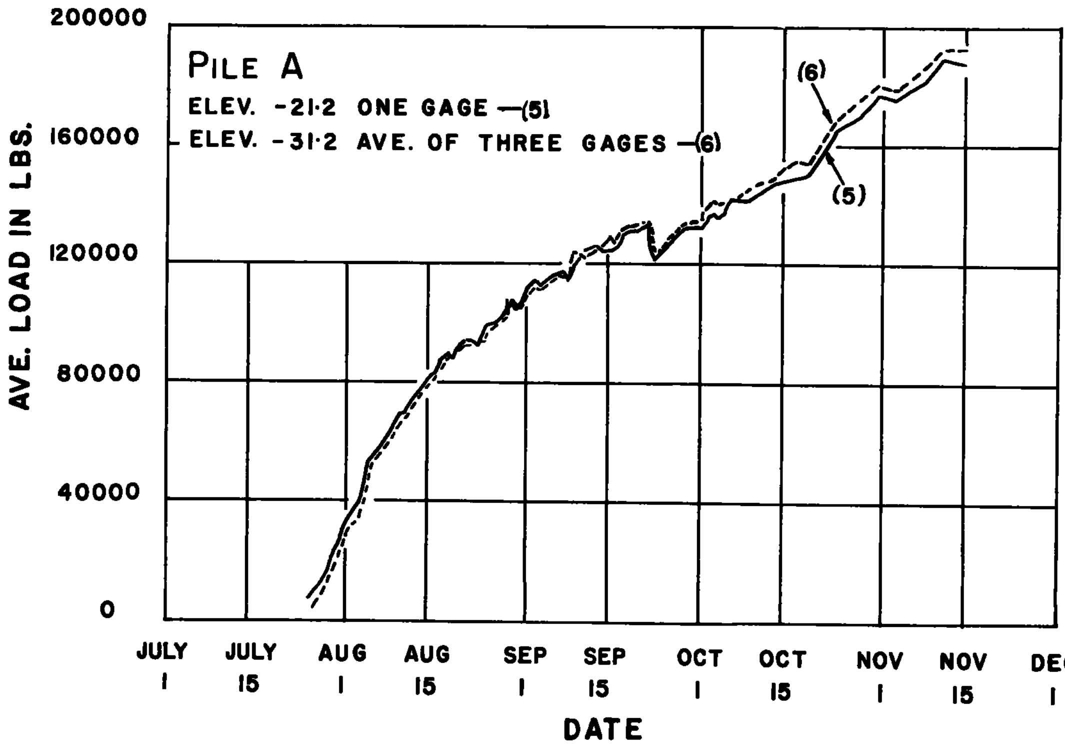

Figure 15. Indicated soil drag loads in Pile A at elevations -21.2 ft and -31.2 ft.

TABLE 1

RESPONSE OF STRAIN GAGES TO PARTIAL UNLOADING - PILE A

Elevation (ft)	Number of Gages	Indicated Load (kips)		Change in Load Nov. 15-29 (kips)
		Nov. 15	Nov. 29	
+4	2	50.5	35.6	14.9
-1	1	44.6	24.9	19.7
-6	2	28.1	8.4	19.7
-11	1	50.1	22.7	27.4
-16	2	57.8	31.7	26.1
-21	1	187.2	169.7	17.5
-31.2	3	192.3	175.9	16.4
-51.2	2	141.4	128.5	12.9
-61.2	1	141.6	121.8	19.8

about +20 ft. The response of the strain gages in pile A to the unloading is illustrated in Table 1. The start of large scale construction operations at the east abutment, with altered loading conditions for the piles, marked the end of the formal test period. However, periodic strain readings indicated that, at least for several weeks after the removal of load, the drag load remained substantially constant.

The progressive consolidation of the mud layer resulted in increased drag loads. However, the effect of the continuing consolidation was different for different layers. As shown in Table 2 and Figure 17, most of the drag caused by the soil above the clayey silt was accumulated in the first three weeks. On the other hand, the transfer of load by the clayey silt took place at a much slower rate and the total load in the pile on August 15 amounted to only about one-half of the November 15 load. Further, comparison of the load gains shows (Table 2) that the increases above elevation -31.2 ft in

TABLE 2

CHANGE IN LOAD - PILE A

Elevation (ft)	Indicated Load Change (kips)		Percent of Nov. 15 Loads		
	Aug. 15 - Oct. 1	Oct. 1 - Nov. 15	on Aug. 15	Change Aug. 15 - Oct. 1	Change Oct. 1 - Nov. 15
+4	3.2	6.0	82	6	12
-1	0.6	0.2	98	0.1	0.1
-6	-1.1	3.6	91	-4	13
-11	9.7	13.6	54	19	27
-16	12.0	16.7	50	21	29
-21	51.4	55.2	43	27	30
-31.2	55.5	58.1	41	29	30
-51.2	43.0	73.0	18	30	52
-61.2	36.0	83.5	16	25	59

the two arbitrary six-week periods (August 15 to October 1, and October 1 to November 15) were roughly equal although somewhat greater gains were measured in the second period. The increased loads measured below the clayey silt in the positive friction zone point to a redistribution of the positive frictional stresses with a probable growth in resisting stresses at lower elevations with time.

The maximum indicated drag loads do not appear unusually large when account is

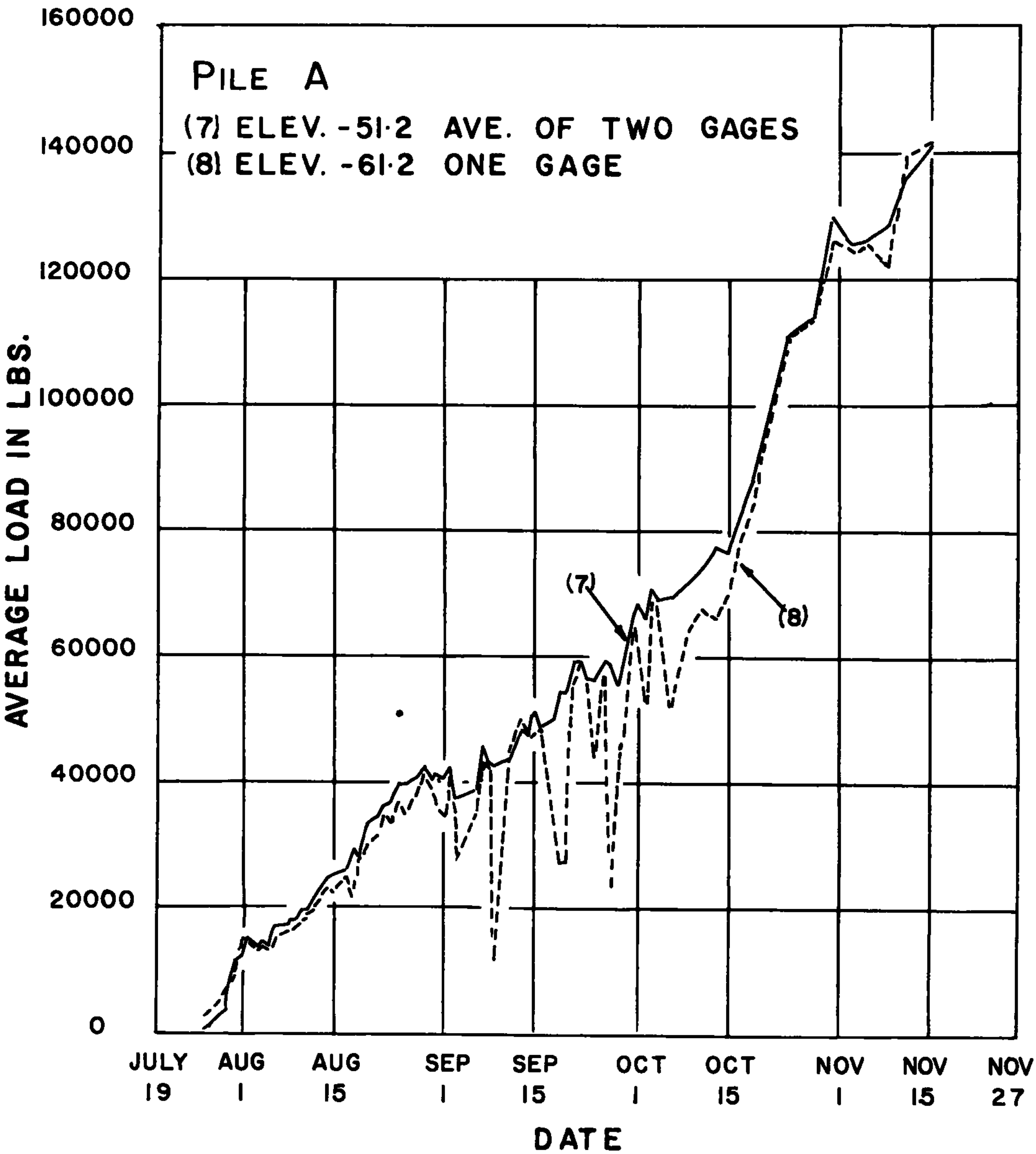

Figure 16. Indicated soil drag loads in Pile A at elevations -51.2 ft and -61.2 ft.

taken of the combination of a high fill on a soft stratum and the fact that the mud layer was incompletely consolidated when piles were driven. However, explanation of the distribution of load obtained in the test (Fig. 17) requires further study of the mechanism of stress transfer from soil to a pile. The maximum indicated bond stresses for the load distribution exceed to a considerable extent the cohesion of the silt as measured by unconfined compression tests on undisturbed samples. This could mean that the bond between the silt and pile is not approximately equal to the shear strength of the soil, but instead is much greater. Or, the shear strength of the soil near the pile may increase with time to a value much higher than that for soil unaffected by the pile driving operation. Gray (6) has suggested an increase in shear strength near the pile and other investigators have also pointed to the fact that often soil will adhere to a pile when it is pulled from the ground. From Figure 17, the maximum indicated bond stress for a load transfer of 130,000 lb in 5 ft is about 8,000 lb per sq ft. However,

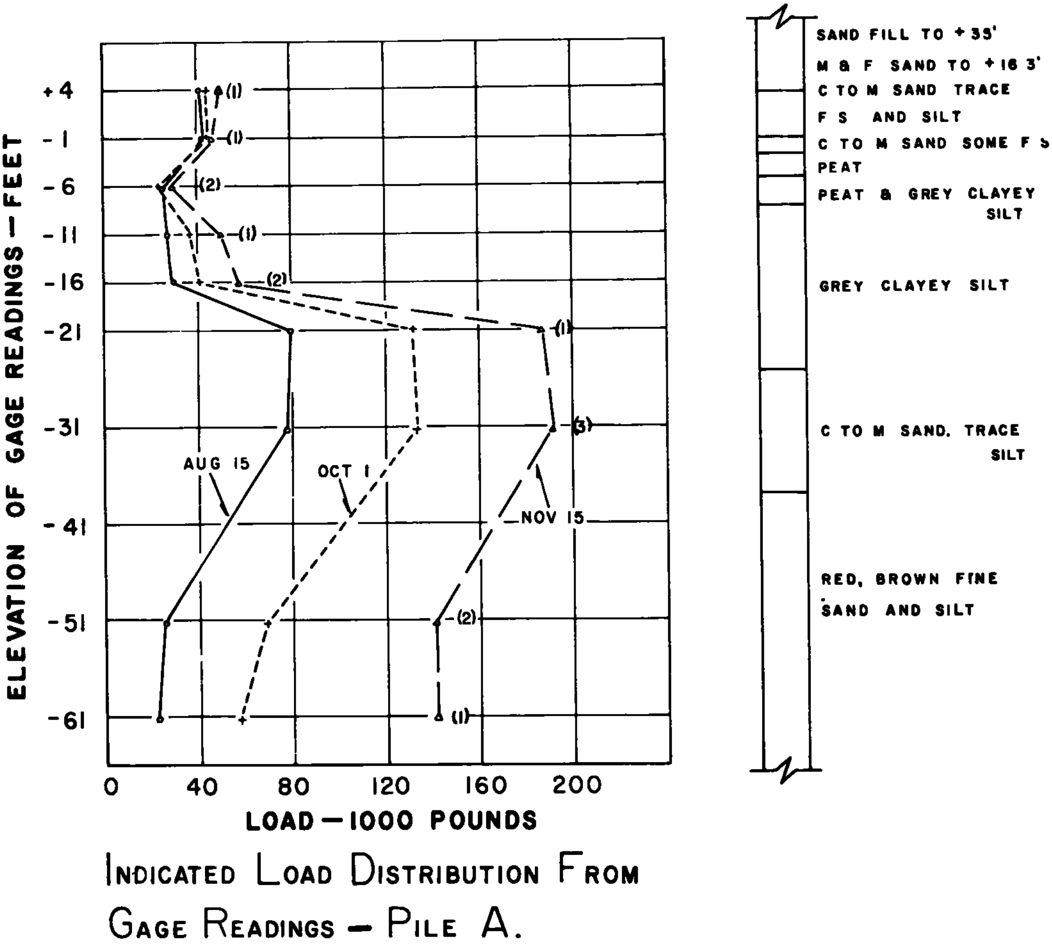

Figure 17. Indicated distribution of soil drag loads in Pile A for three different dates.

at a distance of one and a half diameters from the face of the pile the required shear stress is reduced to about 2,000 lb per sq ft. From Figure 5, a shear strength of the order of 2,000 lb per sq ft may conceivably be attained in the silt when sufficiently consolidated.

COMPUTED DRAG LOAD

For the test pile site, the results of extensive laboratory testing and field observations indicate that the mud layer was incompletely consolidated under the loads applied, especially toward the middle of the layer, at the time piles were driven. Tests also show that the shear strength of the clayey silt, as an indicator of possible bond stresses, was greatly increased by consolidation (Fig. 5). The manner in which the fill was replaced around the upper portion of the piles suggests that the lateral pressure of this soil on the pile corresponded to the active Rankine state. The jetting operation through fill material down to elevation -5 ft also points toward the active Rankine state. The following computations illustrate two procedures for estimating the maximum drag loads produced by the severe loading conditions at the test pile site.

Single Pile Analysis

α = 110 lb per cu ft Elev +35 ft to +6 ft

α = 48 lb per cu ft Elev +6 ft to -5 ft (submerged)

α = 38 lb per cu ft Elev -5 ft to -25 ft (submerged)

Coefficient of lateral soil pressure = $\frac{1}{3}$ (Elev +35 ft to -5 ft). Angle of friction between pile and soil of 30 deg (Elev +35 ft to -5 ft).

Based on these values, the drag load in the pile is 29,500 lb at Elev +6 ft and 53,500 lb at Elev -5 ft. From Elev -5 ft to -25 ft the drag of the silt can be estimated on the basis that the cohesion of the soil is fully mobilized at a distance from the pile surface equal to the radius of the pile. For an average cohesion of 1,200 lb per sq ft (Fig. 5) the drag of the silt is 150,000 lb. This gives a total drag load of 203,500 lb at elevation -25 ft compared with approximately 200,000 lb as obtained from the test. In this procedure it is assumed that the deformation in the silt is sufficient to develop the shearing strength of the soil and that the bond stresses at the surface of the pile are large in comparison to the shear strength.

Cluster Analysis

The influence of the batter piles (Fig. 6) decreases with depth. Therefore, the pile cluster analysis can be made on the basis of the group of seven vertical piles. The perimeter of the cluster at the face of the piles is 32.3 ft and the total area within the cluster is 4.67 ft by 11.5 ft or 53.7 sq ft. Full arching of the soil between the piles is assumed.

The cluster analysis, based on the same soil properties as used in the single pile analysis, gives a total drag load of 63,400 lb in the pile at elevation +6 ft and 100,300 lb at elevation -5 ft. From elevation -5 ft to -25 ft the drag produced by arching of the soil within the cluster and a cohesion of 1,200 lb per sq ft on the perimeter amounts to 116,300 lb. Therefore, the total drag load at elevation -25 ft is 216,000 lb as compared with approximately 200,000 lb from the test results. In this procedure, the stress transfer to the piles above elevation -5 ft requires larger bond stresses than were indicated for the single pile analysis.

The two methods of analyses indicate about the same total drag load at the bottom of the silt. However, it is clear that they are very different in terms of distribution of load and it cannot be assumed that the methods will always give results as close as obtained here. It must also be emphasized that for any given loading condition involving negative skin friction the magnitude of the maximum drag load in the piles is a function of the consolidation that takes place after the piles are driven.

ACKNOWLEDGMENT

The investigation described in this paper was carried out as a joint research project of the Connecticut State Highway Department and the Civil Engineering Department of the University of Connecticut. Particular acknowledgment is made of the cooperation and assistance of Philip Keene, Engineer of Foundations, who initially suggested the project and made available considerable data obtained by the Connecticut State Highway Department.

A. J. Silva and R. E. Machol, graduate assistants, and B. K. Ramiah, research assistant, all of the University of Connecticut, assisted in the experimental program.

The assistance of many members of the Connecticut State Highway Department is also greatly appreciated.

REFERENCES

1. Casagrande, A., "The Structure of Clay and Its Importance in Foundation Engineering," Journal, Boston Society of Civil Engineers, April 1932.
2. Chellis, R.D., "Pile Foundations," New York, McGraw-Hill Book Co., Inc., 1951, Chapter 16.
3. Crandall, L.L., "Electrical Resistance Strain Gauges for Determining the Transfer of Load from Driven Piling to Soil," Proceedings, Second Int. Conf. on Soil Mech. and Foundation Engr., Vol. 4, p. 122 (1948).
4. Cummings, A.E., Kerkhoff, G.O., and Peck, R.B., "Effect of Driving Piles Into Soft Clay," Transactions, American Society of Civil Engineers, Vol. 115, pp. 275-350 (1950).
5. Florentin, J. and L'Heriteau, G., "About an Observed Case of Negative Friction on Piles," Proceedings, Second Int. Conf. on Soil Mech. and Foundation Engr., Vol. 5, p. 155 (1948).
6. Gray, Hamilton, "Discussion of Experiences with Predetermining Pile Lengths by W.W. Moore," Transactions, American Society of Civil Engineers, Vol. 114, p. 371 (1949).
7. Moore, W.W., "Experiences with Predetermining Pile Lengths," Transactions, American Society of Civil Engineers, Vol. 114, p. 357 (1949).
8. Reese, L.C. and Seed, H.B., "Pressure Distribution Along Friction Piles," Proceedings, American Society for Testing Materials, Vol. 55, p. 1156 (1955).
9. Report of Committee, "Steel and Timber Pile Tests — West Atchafalaya Floodway," Proceedings, American Railway Engineering Association, Vol. 52, pp. 149-202 (1951).
10. Report on "A Study of Methods of Installing SR-4 Electric Strain Gages in Monotube Pile Sections," Progress Report, University of Connecticut, March 1955.
11. Schlitt, H.G., "Group Pile Loads in Plastic Soils," Proceedings, Highway Research Board, Vol. 31 (1952).
12. Silva, Armand, J., "A Study of Soil Characteristics in Relation to Forces Produced in Friction Piles," Master of Science Thesis, University of Connecticut, June 1956.
13. Terzaghi, Karl and Peck, Ralph B., "Soil Mechanics in Engineering Practice," New York, John Wiley and Sons, Inc., 1948, pp. 473-474.

Discussion

GREGORY P. TSCHEBOTARIOFF, Professor of Civil Engineering, Princeton University; Associate, King and Gavaris, Consulting Engineers — The paper deals with a topic of considerable practical importance. The detrimental effects of drag on piles produced by the settlement of adjacent soil have been widely recognized in a qualitative manner for some time. Precise quantitative information has, however, been lacking and any attempts to obtain such information are most welcome.

It is unfortunate that circumstances beyond the control of the authors, namely the presence of fill of such density that 22 ft of it had to be excavated to permit driving of experimental piles — after which the fill had to be replaced again, created unusual and most unfavorable conditions for this study. The special nature of these conditions has not been given full consideration by the analysis in this paper of some rather peculiar results of measurements which it reports.

Specifically, the drag value remained within what will be shown to be reasonable limits down to elevation -16 ft, that is, over a total distance of 51 feet: 29 ft through granular fill above water level, plus 11 ft through granular fill below water level, plus 11 ft through peat and clayey silt. Then, over a distance of only 5 ft through the same clayey silt, as reported on Figure 17, the drag value suddenly was more than trebled, being increased by some 325 percent to a total value of 186 kips or 93 tons. This is

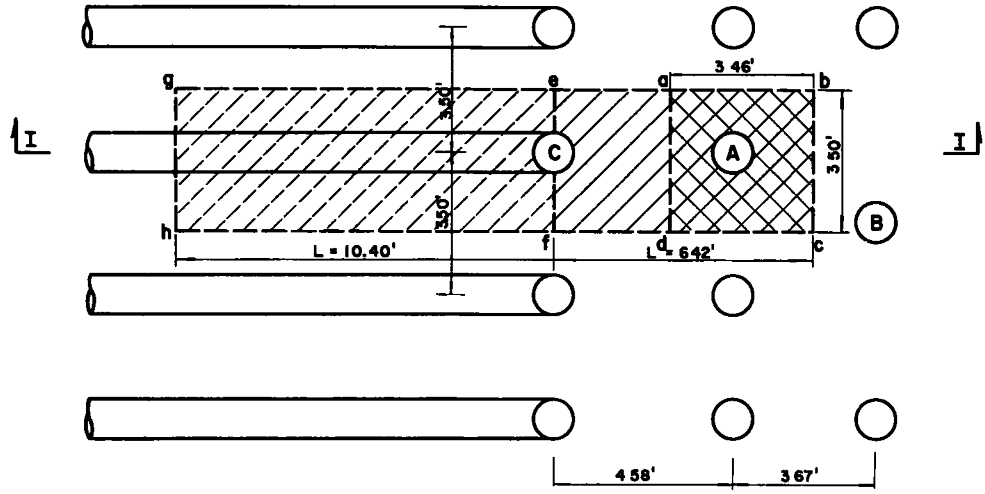

Figure 18. Plan showing column abcd of soil carried by Pile A (see Fig. 19 for Section I-I).

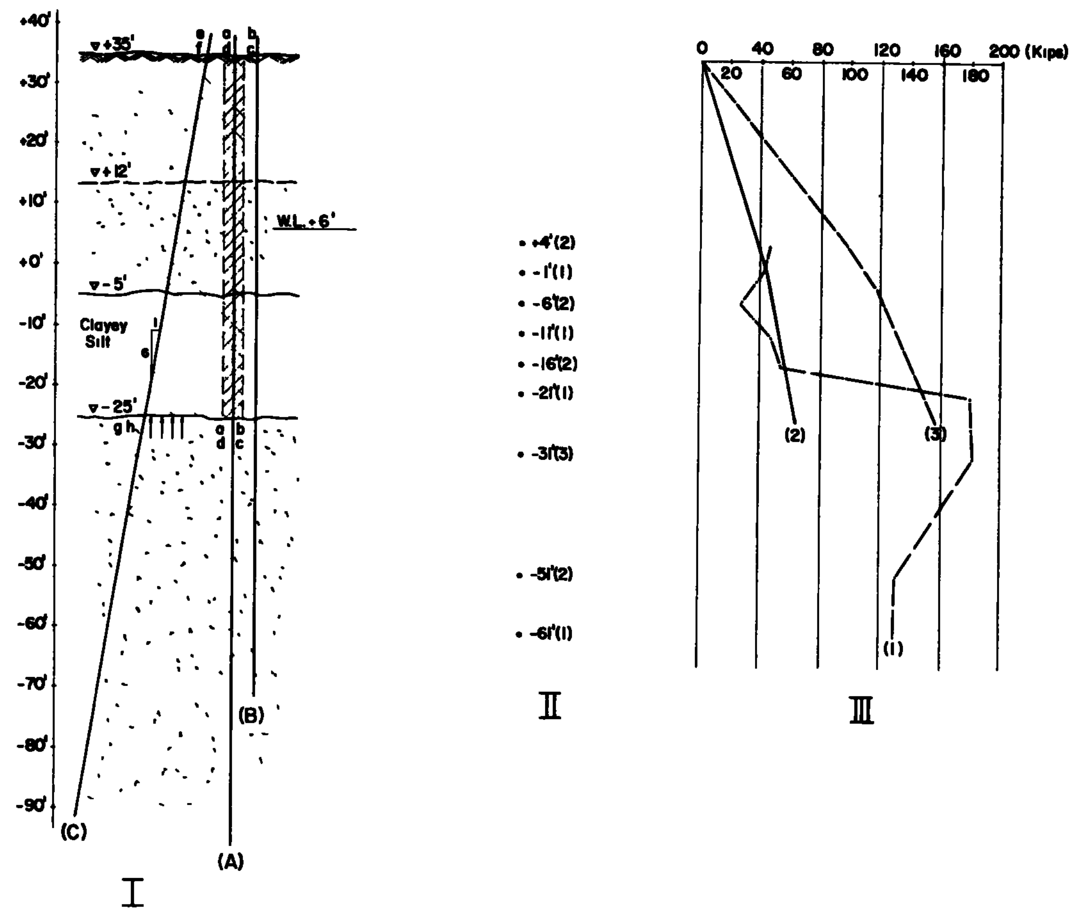

Figure 19. (I) Section I-I on Figure 18; (II) Elevations of SR4 strain gages (number of gages shown in brackets); (III) Computed pile loads: (1) from SR4 readings (by the authors); (2) weight of soil column abcd; (3) weight of soil prism bcefgh.

more than the weight of the soil around that pile.

The authors state that this load of 93 tons does not appear unusually large. This statement and the explanations presented by the authors in its support cannot be accepted by the writer for reasons which are as follows:

(1) The authors leave out of consideration one of the essential criteria which limits maximum possible drag loads, the maximum possible weight of the soil which surrounds the pile. A study of this additional criterion refutes the contention of the authors which attributes the high strain gage readings below a 54 ft depth to vertical drag loads only. The following points are of importance:

(a) The response of the strain gages to partial unloading as reported by Table 1 was somewhat erratic, but the average recorded by the 15 gages at all elevations was 18.7 kips. This value corresponds to the weight of a column abcd of soil 15 ft high shown double hatched in plan in Figure 18 (the excavation is stated to have resulted in a drop of the soil surface between the piles from +35 ft to + 20 ft).

(b) It will be noted from Figure 18 that the area abcd approximately corresponds to (i.e., it equals 83 percent of) the maximum area of soil which could be assigned to the pile A had all the piles around it been vertical.

(c) It will be further noted from curve 2 (Fig. 19) that the weight of the column of soil abcd closely agrees with the drag values computed from readings on all 8 strain gages at the 5 elevations down to and inclusive of elevation -16 ft, i.e. through all of the fill and through two-thirds of the depth of the compressible clayey silt layer.

(d) An extreme but most improbable assumption is that due to the presence of batter piles on one side of it, the pile A would carry a larger prism bcefgh which would include all soil under the batter piles as shown on Figure 18 and 19. The weight of this prism would exceed by some 250 percent values given by the 8 strain gages at the 5 elevations down to and inclusive of elevation -16 ft, as shown by curve 3 on Figure 19, but even then the weight of that prism would fall short of the drag computed from readings on the 4 strain gages at the next two elevations, that is, 4 ft above and 6 ft below elev -25 ft of the surface of the deep non-cohesive granular stratum in which the piles are embedded.

(2) References by the authors to allegedly similar observations made elsewhere are not relevant to a numerical evaluation of this case:

(a) The drag load of nearly 200 tons estimated by Moore (7) caused a pile to fail under conditions which were substantially different from the present ones. It can be seen from Figures 17 and 18 (pp. 366-367) that Moore's piles were spaced much more widely apart than the piles of the present case (Figures 1 and 18) so that a much greater weight of soil could come to hang on them.

(b) It is true that soil will often adhere to a pile when it is pulled from the ground. For instance the writer has observed a shell of stiffer clay one or two inches thick around the face of excavated concrete piles. This is natural since such piles facilitate consolidation of the clay along their faces by permitting drainage through them. In the case of extracted H-piles clay will frequently remain stuck between their flanges since the surface of adhesion of clay to metal is then almost twice as great as the clay shearing surface related to this phenomenon. It is also conceivable that thin patches of clay might adhere even to cylindrical surfaces of steel due to rusting or other chemical action on the clay. But the writer knows of no evidence (and the authors have not produced any) which would justify their suggestion that the zone of increased strength of the clay might in the present case extend up to a distance of one and a half diameters from the face of pile A.

(3) Some of the assumptions on which the authors base their "single pile" and their "cluster" analysis are in contradiction with the observations which they report. Thus, they allegedly obtain agreement at the bottom of the silt layer between the drag loads as computed by them from the strain gage readings and as estimated by them on the basis of soil strength data. This alleged agreement is obtained by assuming a load transfer due to an average potential cohesion of 1.2 ksf (Fig. 5), 6 in. from the face of the pile along the entire 20 ft depth of the silt layer, from -5 ft to -25 ft. This assumption is in direct contradiction to Figure 17 of the authors which shows variations of the total drag load with depth. Figure 20 (II) shows the variation of the shearing strength

needed at face of pile if the curve in Figure 17 for November 15 is to be explained by drag loads as is attempted by the authors. The shearing strength needed to explain the computed drag loads at the face of the pile of 12-in. diameter equaled 1.2 ksf for the first 5 ft only (from -6 ft to -11 ft); then it dropped to 0.51 ksf over the next 5 ft, but increased to 8.05 ksf over the subsequent 5 ft. At a distance of 6 in. from the face of the pile these values would be reduced to one-half of the figures given.

(4) The authors offer no explanation which could account for the enormous differences shown on Figure 20 (II) in the effective shearing strength for different parts of the silty clay layer needed to explain their hypothesis that only drag forces act on the piles. The following shows that such differences cannot be assumed actually to exist:

(a) The trend of the actual variation of the potential shearing strength with

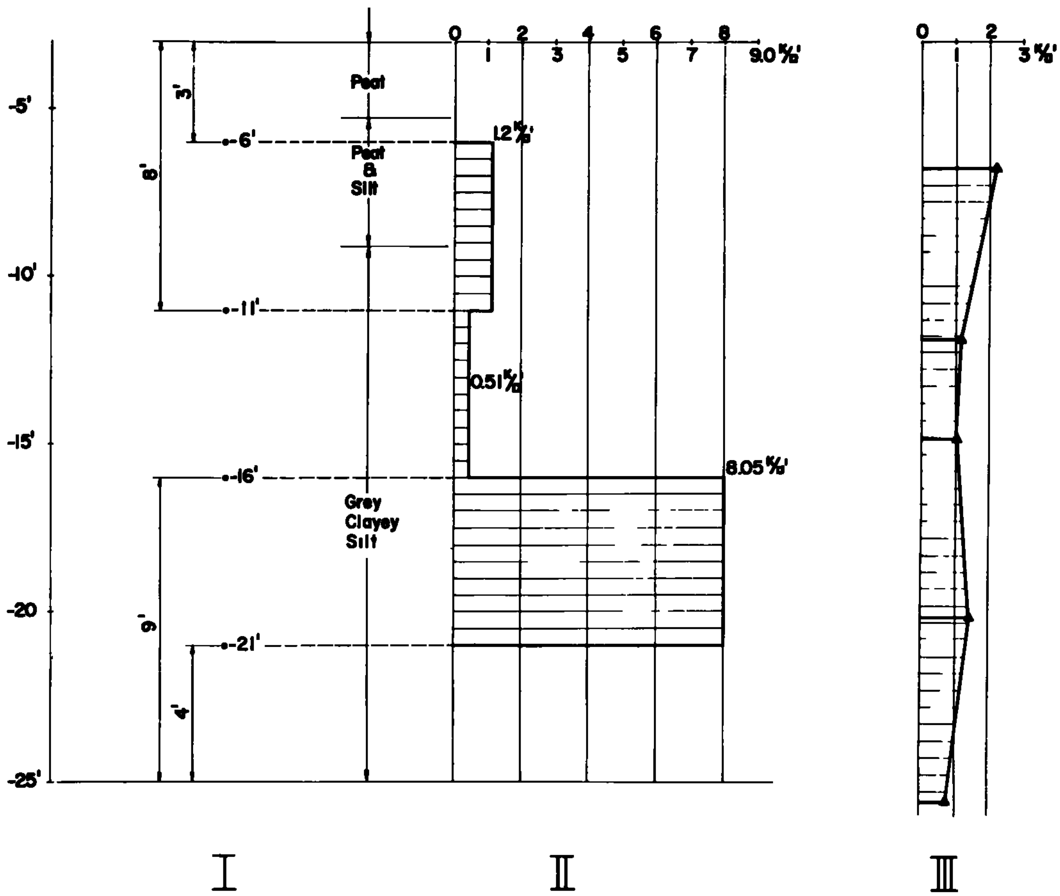

Figure 20. (I) Distance of points of measurement from pervious boundaries; (II) Shearing strength needed at face of pile if curve (I) of Figure 19 (also Figure 17) is to be explained by axial drag loads only; (III) Actual shearing strength of cohesive layer on December 1955 as per Figure 5 .

depth (Figures 5 and 20, III) for the December 1955 boring indicates a greater shearing strength of some 2.1 ksf in the upper part of the clayey silt, even though it is stated to be mixed there with some peat. In the lower part the actual potential shearing strength is only approximately half as large, 1.2 ksf. This actual trend is the reverse of the one shown on Figure 20 (II) as corresponding to the hypothesis of purely axial drag loads.

(b) Although the low rate of transfer of load (0.51 ksf) at the center of the clayey silt layer may be explained by a smaller degree of consolidation, this explanation cannot hold for the differences between the top (1.2 ksf) and the bottom (8.05 ksf)

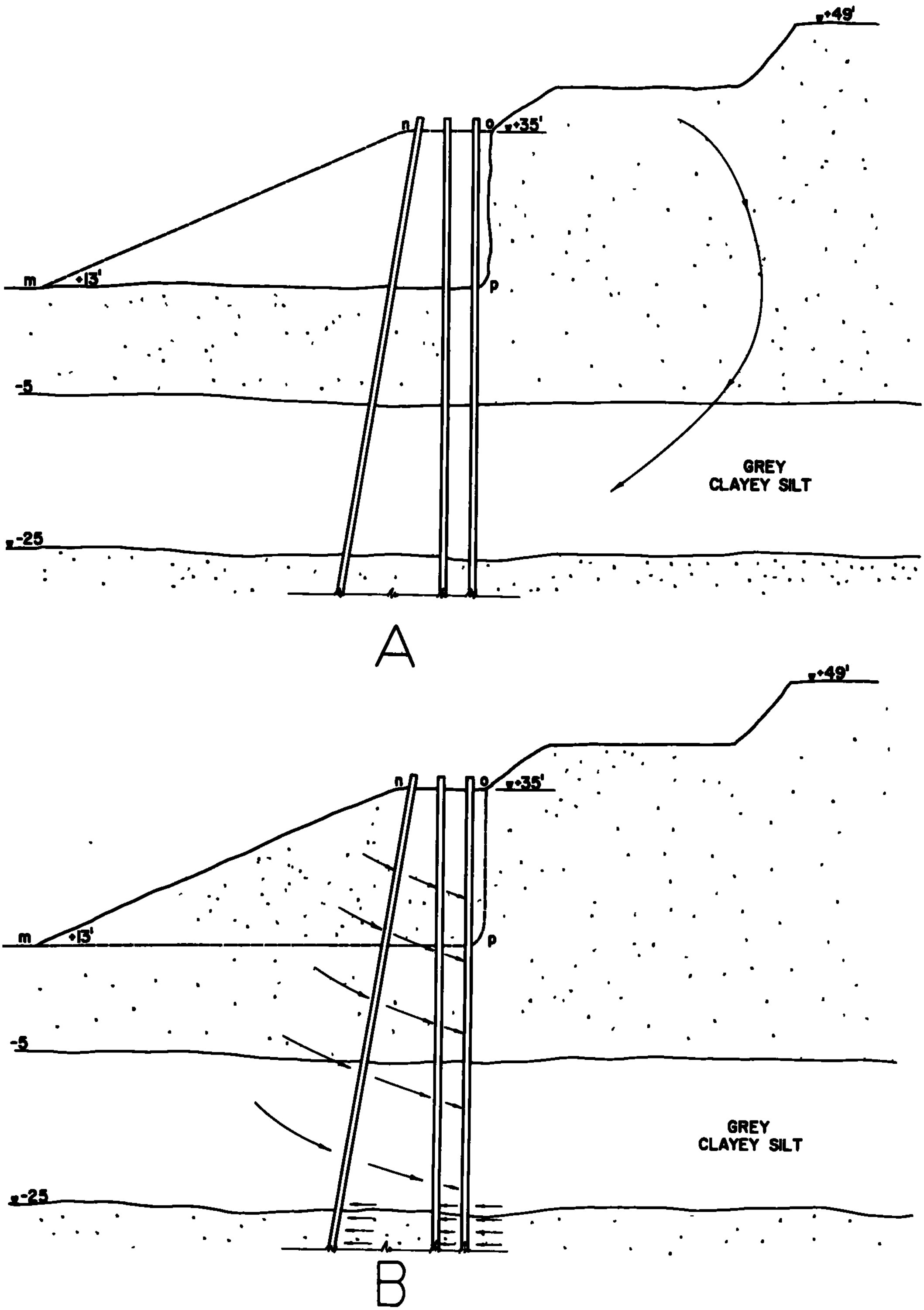

Figure 21. Soil deformations induced: (A) First by excavation of embankment wedge mnop and then (B) By its replacement after pile driving.

parts of the layer since the distances to pervious boundaries (Fig. 20, I) are approximately the same in both cases.

(c) The average ratio of the cohesion to the preconsolidation loads (PCL values) reported in Figure 5 is approximately 0.30. This is a reasonable value for this type of material. It therefore follows that a preconsolidation load of 26.8 ksf would have to be applied in order to obtain the cohesion of 8.05 ksf (Fig. 20, II). This value of the preconsolidation load is 26.80/4.34 = 6.2 times greater than the unrelieved weight of the overburden at elev -21 ft. This ratio would have to be much higher if the actual vertical pressures at that elevation are considered, since a large part of the weight of the overburden has already been transferred by drag to the piles. Thus, pressures of very high passive intensity would have to be present between elev -16 ft and -21 ft in order to explain strain gage readings there by drag only. Therefore, since drag essentially is a phenomenon of active or neutral pressures, it is not possible to explain these readings by drag only. Furthermore, it is inconceivable that pressures several times higher than the weight of the overburden could be localized for unknown reasons in the lower part of the clayey silt layer without affecting the overlying parts of that layer.

(5) It is most unfortunate that no reliable settlement readings of the experimental piles themselves appear to have been taken. This data could have shown in conjunction with the load test pile data (Fig. 11) whether 93 tons drag load actually had been applied to pile A.

(6) The statement by the authors that strain gages on the experimental piles B and C recorded in general a distribution of load similar to that of the pile A which is discussed by their paper contains basic contradictions with their hypothesis of drag loads as well as a clue to the solution of the problem:

(a) Vertical soil loads due to gravity forces cannot produce axial loads only in a batter pile; such forces must inevitably produce in a batter pile bending as well.

(b) It therefore follows that if vertical piles reacted to changes in load in a manner identical to that of a batter pile, then these changes in load must have produced bending not only in the batter pile, but in the vertical piles also.

(7) All the inconsistencies in the explanation by "drag" of these peculiar measurement results are resolved if the causes and the nature of possible bending of pile A are studied.

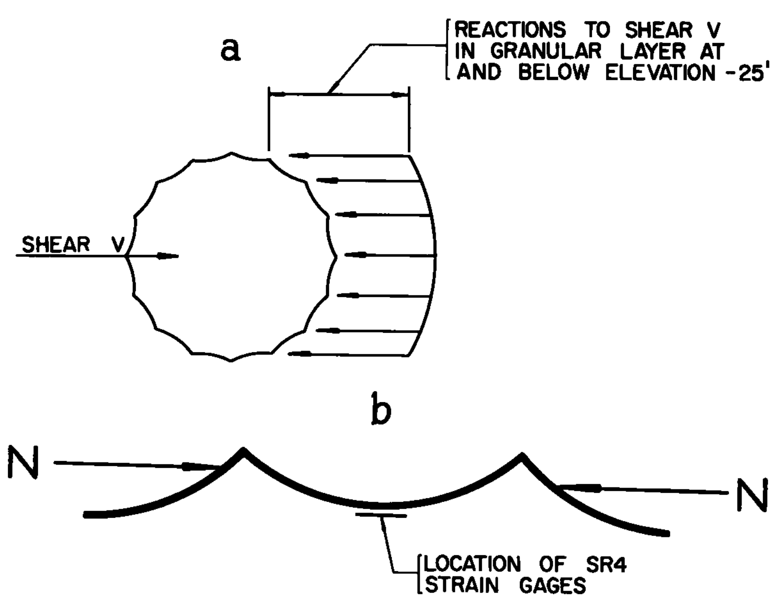

Figure 22. Sketch illustrating probable effect of unsymmetrical loading on SR4 strain gages mounted on empty shells of fluted Monotube piles.

(a) The removal of the prism mnop of fill shown (Fig. 21A) must have induced tendencies for slight soil movements (indicated by arrows on that diagram) prior to and during the pile driving.

(b) The replacement of the prism of fill mnop after the pile driving must have induced a reversed trend indicated by the arrows in Figure 21(B). As a result, compared to their zero readings on the strain gages, the pile shells must have then acted as slender dowels fixed in the non-cohesive granular soil stratum at elev -25 ft. The effects would have been felt just above and below elev -25 ft, exactly where a sudden and abnormal increase in strain gage readings was noted at elev -21 ft and below that level (Fig. 19).

(c) The fixation at elev -25 ft must have induced bending (primary) of the entire shell of the pile, plus the transmission of a shear V and resulting one-sided loading at that elevation.

(d) This type of one-sided loading (Fig. 22a) is bound to produce secondary bending in a horizontal plane through the fluted portions of the shell (Fig. 22b), that is, at right angles to the direction of the SR4 gages. Stresses exceeding the yield point can then easily be induced in a horizontal plane at the location of the SR4 strain gages and, due to the Poisson ratio effect can be recorded by them as high axial strains in a vertical direction. The primary and secondary bending combine to produce a complicated stress picture which it is now unfortunately impossible to analyze further since only 39 percent of the originally small number of strain gages actually operated.

CONCLUSIONS

1. The actual drag load on pile A does not exceed the values recorded down to elev -16 ft, some 60 kips or 30 tons.
2. The sudden increase of strain gage readings at and below elev -21 ft does not mean that the drag was increased there to some 180 kips or 90 tons as suggested by the authors. These readings merely reflect the bending and dowel action of the thin pile shells due to one-sided lateral pressures caused by the replacement after pile driving of some 22 ft of excavated fill.
3. The end slope of a high embankment is not the best place to study axial drag loads on vertical piles.
4. Thin fluted shells are not the best kind of piles for drag measurements by means of strain gage readings if any axially unsymmetrical loading is possible.
5. The number of SR4 gages installed was much too small in the first place. The usual practice of providing under laboratory conditions 50 percent extra gages in excess of the minimum needed should be changed under field conditions to provide 200 to 300 percent extra gages because of the hazards of field installation and operation.
6. A positive result of the whole investigation is the need which it demonstrates for consideration of bending stresses in all abutment piles driven through soft cohesive soil.

EDWARD V. GANT, JACK E. STEPHENS, and LYLE K. MOULTON, Closure—The discussion presented by Professor Tschebotarioff represents an interesting and thoughtful review of certain aspects of the investigation described in the paper. Professor Tschebotarioff's comments on the experimental procedure are especially helpful. However, his main thesis is that the loads measured for elevations below about -16 ft were produced by bending rather than by negative friction. He also comments on the lack of agreement between the values for drag load below elevation -16 ft, as determined in the investigation, and the magnitude and distribution of the loads which he computes on the basis of certain assumed boundary conditions.

Throughout the investigation particular attention was paid to the possibility of bending. Initially strain gages were installed (Fig. 8) in such a way as to eliminate bending effects. Careful analysis of all the gages which remained active failed to show anything more than rather minor bending in the vertical pile A. This was also true for the other vertical pile studied. The batter pile did seem to subject to bending, evidently caused by the eccentricity of the drag loads. In answer to a question raised by Professor Tschebotarioff on the distribution of load in the batter pile, it should be noted that the writers stated that the distribution in the exterior vertical and the batter pile followed a pattern similar to that shown for pile A (Fig. 17). It was not stated that the distributions were identical. It was unfortunate that most of the gages in the exterior vertical and batter piles were damaged. However, for the few gages which continued to operate in those piles it appeared that there were increases in axial loads in the cohesive layer at about the same elevation as for pile A.

There are other reasons also why the bending action described by Professor Tschebotarioff must be questioned. Thus, there is no explanation offered as to how the bending action would increase with time as the measured forces did, Figures 13-16. On the other hand, there is a large amount of experimental evidence available to indicate that for most compressible soils the shear resistance, and probably the bond between the pile and the soil, does increase with a decrease in excess hydrostatic pressure and added consolidation (Fig. 5 and references (4) and (8)). For the compressible soil adjacent to the experimental piles the buildup in shear strength, and therefore in drag forces, was produced by the re-consolidation of the soil disturbed by pile driving, and the continuing consolidation produced by the recent fill. Another factor which has some bearing on the question of bending is the effect of the removal of the large volume of fill during the period November 15-29, 1955. The bending action described by Tschebotarioff is based on the removal and replacement of fill over the relatively short length of abutment occupied by the experimental piles. In the period November 15-29, fill was removed to elevation +13 ft over the whole length of the abutment, except within the pile group where the fill slumped to elevation +20 ft. While the fill was not replaced immediately, it does seem that if bending of the type described actually occurred then the gages should have indicated a large change in the pile loads. Instead, the changes were fairly small (Table 1). In fact, Tschebotarioff draws a comparison between these changes and the weight of soil removed from within the pile group, and no mention is made of bending effects caused by the removal of the soil.

Tschebotarioff states that one of the essential criteria which limit maximum possible drag loads is the maximum possible weight of the soil which surrounds the pile. This leads to considerable ambiguity since the criterion does not define the role of the soil outside the pile group. Actually, according to his calculations, Tschebotarioff assumes (Fig. 18 and 19) that the soil outside the pile group has no effect on the interior pile A so far as drag is concerned. However, this overlooks the fact that the soil outside the group tends to settle relative to the piles and the soil within the pile group and, as a result, shear stresses are mobilized on vertical planes at the boundary of the pile group. If the deformation is great enough then the maximum shear resistance of the soil and the bond stresses on the pile surface are mobilized.

The mechanics of stress transfer from the soil outside the pile group to the soil within the group and individual piles is obscure and ideas on the distribution of the load along the pile cannot be fixed until further test results are available. Some of the contributing factors, however, can be identified. Thus it is known that the re-consolidation of a compressible soil after remolding by action of pile driving may result in values of unconfined compressible strength greater than for the undisturbed soil, (4), (8). Reese and Seed (8) found that the rate of increase in shear strength for soil adjacent to a pile agrees with the rate of decrease in excess hydrostatic pressure. Cummings, Kerkhoff, and Peck (4) have suggested the possibility of horizontal drainage as a factor in the growth of shear strength in the soil near a pile. For the test site in West Haven, there was the possibility of horizontal drainage at all levels of the compressible silt since there were vertical sand drains in the area. In addition the effect of the overlying fill was to produce a continuing consolidation through a combination of vertical and horizontal drainage. The conclusion must be, then, that the sum of the effects was to produce a large shearing resistance in the soil near the piles. The value could conceivably vary along the length of pile, depending on the extent of remolding at different depths and the rate of decay of excess hydrostatic pressure. There is little known concerning the variation in shear strength with distance from a pile. Casagrande (1) suggests that extensive remolding takes place near a pile with some disturbance up to one and a half diameters from the pile. If this is the case then the value of one and a half diameters might be considered the maximum distance within which the shear strength is increased over the value for undisturbed soil. Possibly the value varies over the length of pile and, as the writers pointed out, this may account for the variable rate of transfer of load to the pile A. It is, of course, impossible to reconcile the measured distribution with the calculations presented by Tschebotarioff, since he neglects the deformations that are produced in the soil outside the pile group and the resulting load transfer.

The purpose of the writers in presenting sample calculations of pile drag loads was simply to compare the results obtained from conventional design methods with the results obtained in the investigation. An additional result was the indication of the assumptions used in the comparison. It is certainly not to be expected that computations of the type used will result in a distribution of pile drag loads compatible with measured values. The best that can be hoped for in this type of approximation is that the maximum computed values to be used in design are somewhere near the actual values. For the case considered, and using the assumptions as shown, the design values did approximate the measured values. All cases, however, cannot be considered alike. In each case the consolidation that is likely to take place after piles are driven, and the effect of such consolidation on soil properties, should be carefully studied before a decision is reached on the possible build-up of drag loads.

Tschebotarioff has made an interesting comparison between the average decrease in load along the pile in the period November 15-29, 1955 and the volume of soil within the pile group that slumped when the soil outside the pile group was excavated to elev +13 ft. The principal criticism of the comparison is that the assumption is made that the unloading results in equal decreases in loads at all lower levels. The fact of the matter is that the decreases measured were not all the same (Table 1). The factors which contributed to the decreases were the decrease in the weight of soil between the piles and the rebound of the compressible layer. The decrease in the weight of soil within the pile group took place when the fill outside the pile group was excavated to elev +13 ft and the soil within the pile group slumped to an average elevation of +20 ft. However, pile A was an interior pile and the fill was somewhat higher around the pile than elevation +20 ft. The rebound of the compressible layer was caused by the decrease in total stress within the layer. Further, the decrease in total stress was, of course, caused by the excavation of fill to elev +13 ft, which was about 7 ft below the average height of fill within the pile group. Therefore, it is reasonable to expect that the rebound would result in differences in the decreases in load at the several levels. The use of an average decrease and the comparisons made therefrom are therefore not considered to be meaningful.

Tschebotarioff evidently did not understand the purpose that the writers had in mind in giving some reference to the literature on drag loads. The references to Moore (7), Chellis (21), Florentin and L'Heriteau (5) were made merely to give the reader some knowledge of what had been written on the subject and, at least, the opinion of others.