# MEASUREMENT OF PILE DOWNDRAG BENEATH A BRIDGE ABUTMENT

John E. Garlanger, Ardaman and Associates, Inc., Orlando, Florida*

Steel H piles for a bridge abutment were driven through 25 ft (7.6 m) of fill, 13 ft (4.0 m) of sand, and 50 ft (15.2 m) of soft clay into a dense glacial till. Seventeen years after the bridge was completed, the differential settlement between the bridge deck and the approach fill had exceeded 18 in. To determine the amount of downdrag loading carried by the piles, which were designed to carry a total loading of 60 tons (530 kN) per pile, measurements of strain and movement were made at the top of one of the piles as the pile was cut free from the abutment and as the skin friction forces were removed through a combination of excavation and electro-osmosis. The measured vertical load and moment at the top of the pile were 114 tons (1015 kN) and 1,140 in.-kips (130 kN-m) respectively. The total vertical load at the top of the glacial till, back-figured from the movement at the top of the pile, was 230 tons (2050 kN), indicating over 100 tons (890 kN) of pile downdrag.

•AS part of a study conducted at the Massachusetts Institute of Technology to develop better methods for predicting downdrag forces on piles, direct and indirect measurements were made of the loads acting on one of the piles supporting an existing bridge abutment. The magnitude of settlement between the approach fill and the abutments indicated that full negative skin friction should have developed along at least 70 ft (21 m) of the piles supporting the abutments. Because none of the piles had been instrumented prior to driving, it was not possible to measure the downdrag loads directly. These loads had to be back-figured from measurements of movement at the top of one of the piles after a section of the pile was cut out and the negative skin friction forces removed.

## THE CUTLER CIRCLE BRIDGE

The measurements of pile downdrag were made beneath the west abutment of the Cutler Circle Bridge, a simply supported, 3-span overpass structure carrying the proposed extension of Interstate 95 over State Route 60 near Revere, Massachusetts. A 25-ft-high (7.6 m) approach fill, constructed over 13 ft (4.0 m) of sand and 50 ft (15.2 m) of soft clay, was placed before construction of the abutments began. A cross section through the embankment and soil profile is shown in Figure 1.

The designers of the bridge predicted that the settlement of the approach fill after 50 years would exceed 3 ft (91 cm). However, they did not "... anticipate any damage to the abutment of the bridge since [they had] coped with the tendency of a drag load on the supporting piles by providing extra piles." Some consideration had been given to surcharging the embankment and foundation, but time-settlement calculations indicated that insufficient time was available for the surcharge to be effective. In addition, stability calculations indicated that the factor of safety, even without the surcharge, was only 1.2 to 1.3. Lightweight fill had been ruled out as being too expensive.

*This paper is based on work performed while the author was Assistant Professor of Civil Engineering at Massachusetts Institute of Technology.

The construction proceeded as follows:

| | |
|---|---|
| August 1956 to November 1957 | Fill placed and compacted |
| February 1957 | Piles for abutments driven |
| March 1957 to June 1957 | Abutments formed and poured |
| June 1957 to July 1957 | Beams for bridge superstructure placed |
| August 1957 | Concrete bridge deck poured |
| January 1958 | Level survey along centerline of completed bridge taken |

The available evidence indicates that the piles (14BP73 H-sections) were driven to at least 7 blows/in. (3 blows/cm) for the last inch with a Vulcan No. 1 pile hammer. The specified design load was 60 tons (530 kN). Because of the extra piles provided to carry the downdrag load, the average load per pile computed from the weight of the bridge deck and the abutments is 40 tons (350 kN).

Settlement data obtained from the Massachusetts Department of Public Works for a station on the bridge deck and a nearby station on the approach fill are plotted in Figure 2. At the abutment, which has settled less than ½ in. (1.2 cm), the embankment has settled approximately 18 in. (46 cm).

The large differential settlement is apparent both behind and in front of the abutment (Figures 3 and 4). There is also visual evidence that the top of the abutment has rotated about the rear piles toward the fill.

To determine the existing soil and groundwater conditions beneath the abutment, a subsurface investigation was made in May 1972. A summary of the boring logs, the vane shear test results, and the piezometer data is shown in Figure 5 along with the results of unconsolidated-undrained (UU) triaxial tests performed on 5-in. fixed-piston samples. The piezometer data indicate that consolidation of the clay is essentially complete.

## INSTRUMENTATION

In late November 1972, a sheeted and braced excavation 5 × 7 ft (1.5 × 2.1 m) in plan was made behind the west abutment of the bridge to expose one of the supporting piles. The excavation also exposed an 18-in. (46-cm) gap between the pile cap and underlying fill into which a person could crawl to examine the remaining piles. During the examination of the piles it was observed that two of the batter piles in the front row had been pulled out of the pile cap (Figure 6), further confirming the suspicion that the piles were subjected to large downdrag loads.

Because none of the piles had been instrumented before driving, it was not possible to measure the downdrag loads directly. The only measurements possible were gross movement and strain at the top of the piles after releasing forces from the pile. The forces acting on the pile are the structural load transmitted by the pile cap, the downdrag forces created by the fill, sand, and clay, and the friction and end bearing resistance in the bearing layer. To isolate each component for separate evaluation, the following measurements were made:

1. Horizontal and vertical movement and strain of a point at the top of the pile upon cutting it free from the pile cap;
2. Horizontal and vertical movement of the point after release of the friction in the sand and gravel layers;
3. Horizontal and vertical movement of the point after release of the friction in the clay layer; and
4. The load required to jack the pile back to its original position.

Four Geonor vibrating wire strain gauges and four BLH weldable electrical resistance strain gauges were installed on the pile before it was cut, one each on both flanges and both sides of the web.

Vertical and horizontal deformations were monitored using the dial indicator system shown in Figure 7. The measuring platform and plates suspended from the abutment

Figure 1. Soil profile at Cutler Circle Bridge.

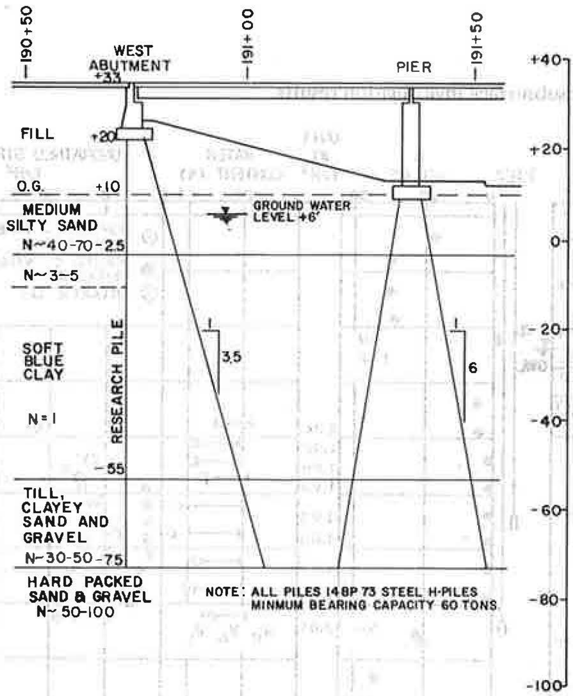



Figure 2. Settlement curves for bridge abutment and approach fill.

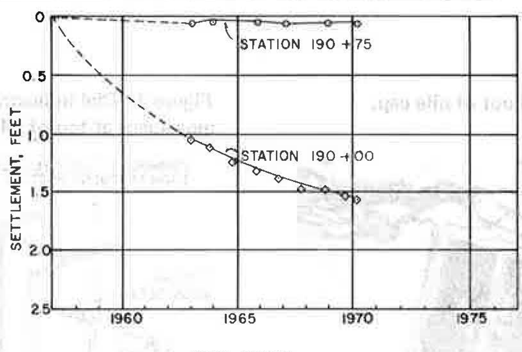



Figure 3. Differential settlement at wing wall.

Figure 4. Breakup of slope protection beneath bridge deck.

Figure 5. Summary of subsurface investigation results.

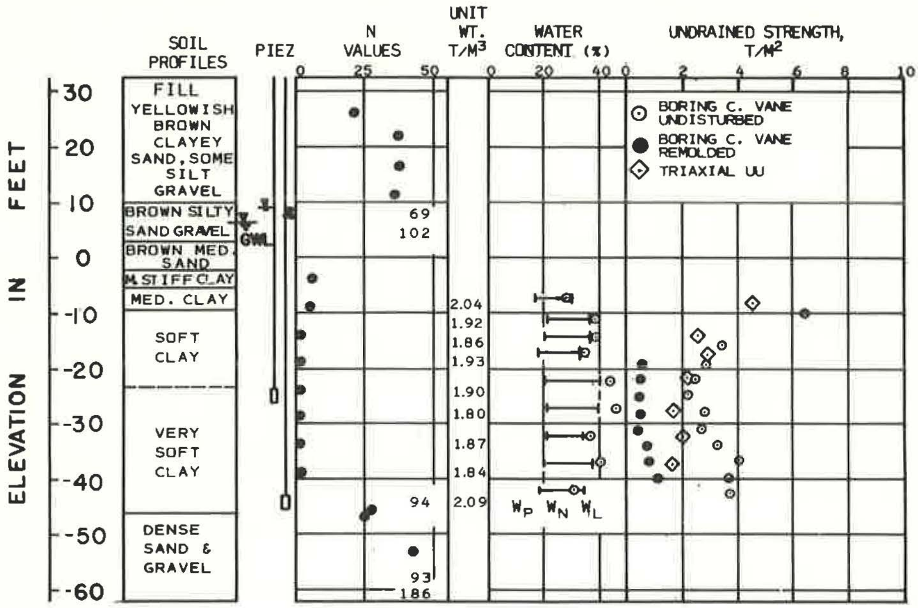



Figure 6. Batter piles pulled out of pile cap.

Figure 7. Dial indicator setup to measure movement at top of pile.

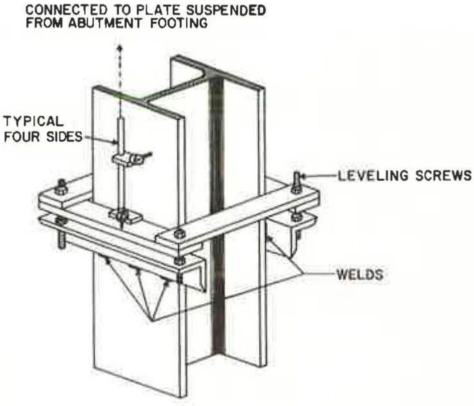
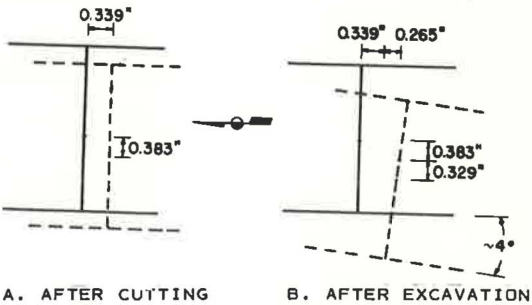



Figure 8. Movement of pile in horizontal plane after cutting and after excavating sand and gravel.



could be leveled so that initially the platform would be horizontal and the rods plumb. With this system, it was possible to correct the measured vertical movement for any false movements that might be introduced should the pile tilt, twist, or translate upon cutting. Thus the corrected vertical movement, provided that the pile was essentially vertical throughout its length, represents only the axial extension of the pile plus any rebound of the pile tip. If the pile was not vertical throughout its length, the corrected vertical movement would be less than the axial extension of the pile, and any pile loading deduced from that movement would be less than the actual loading.

As a precaution, the vertical movement was also obtained by measuring the distance between the two reference points on the pile, one above and one below the section that was cut out. In addition, the movement of the abutment during cutting was monitored.

## MEASUREMENTS

### Pile Cutting

To maintain a concentric load in the pile, cutting began with both flanges and then proceeded in toward the center of the web. The elastic deflection resulting from the release of bending moments in the pile after cutting is shown in Figure 8a. The vertical movement of the pile after cutting was 0.40 in. (10.2 mm) up.

The axial load and bending moment acting on the pile, computed from the strain gauge data (agreement between the two strain gauge systems was with 10 percent), were 114 tons (1015 kN) and 1,140 in.-kips (129 kN-m) respectively. The measured axial load is almost twice the design load, and the maximum fiber stress of 22 ksi (152 MPa) as a result of this bending moment is almost twice that allowed by the Boston building code.

### Excavation of the Sand and Gravel

Above the water table, the sand and gravel were removed by hand in a 4 × 4-ft (1.2 × 1.2-m) caisson-type excavation. Below the water table, the sand was not removed but was placed in suspension by jetting down around the pile using a thick bentonite slurry. The deflected position of the pile after excavation of the sand and gravel is shown in Figure 8b. As shown in the figure, the horizontal movement of the pile was accompanied by a rotation of the pile about its own axis. The vertical movement of the pile plotted against depth of excavation is shown in Figure 9. Also shown are the corresponding N values at various depths. The total vertical movement was 0.097 in. (2.46 mm) up.

### Electro-Osmosis

To remove the negative skin friction in the clay layer, a direct electric current was applied between the test pile (cathode) and a neighboring pile (anode) to induce large pore pressures at the face of the test pile. The results of the electro-osmosis are shown in Figure 10. The pile moved up a total of 0.157 in. (3.98 mm) during electro-osmosis.

### Pile Reloading

With the electro-osmosis still applied, the pile was loaded with a hydraulic jack up to 220 tons (1958 kN) in 25-ton (222-kN) increments. The load at the top of the pile was measured with a calibrated electrical resistance load cell. The results of the reloading are shown in Figure 11. It was not possible to test load the pile to failure because of the danger of buckling the 20 ft (6 m) of unsupported pile left freestanding above the clay strata by the excavation of the sand and gravel.

### Additional Measurements

During the research period the temperature in the work area remained around 30 F ±5 F, despite a fluctuation outside from 10 to 50 F. The temperature of the pile, as measured by a thermistor attached to the pile, varied even less.

Pore pressures at mid-depth of the clay were monitored throughout the research period. No significant pore pressure changes were noted, even during the electro-osmosis.

Figure 9. Vertical movement of pile during excavation of sand and gravel.

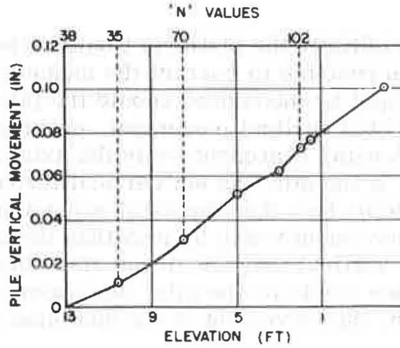



Figure 10. Vertical movement of pile during electro-osmosis.

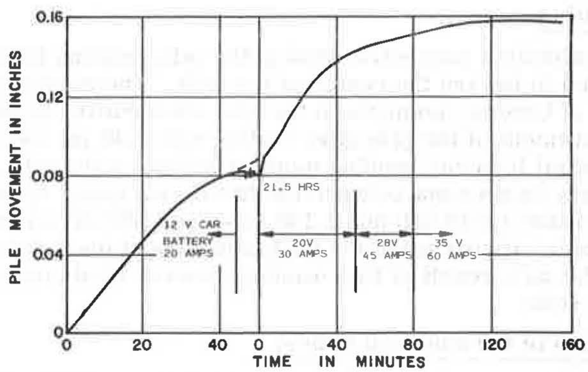



Figure 11. Results of pile load test.

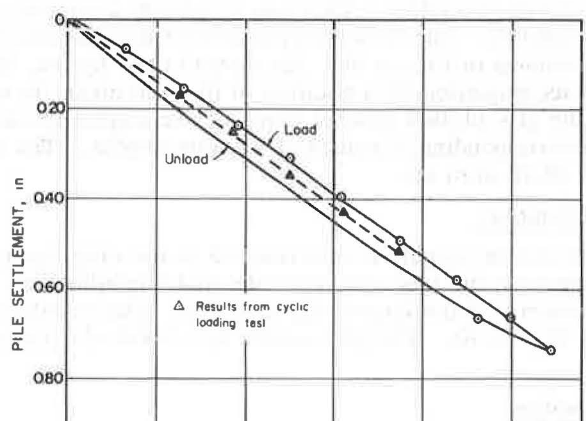



Table 1. Summary of vertical movement at various stages of load test.

| Stage of Test | Movement of Top of Pile, in. Predicted Range (average) | Measured |
|---|---|---|
| After cutting | 0.060-0.755 (0.234) | 0.401 |
| After excavating sand | 0.048-0.240 (0.112) | 0.097 |
| After electro-osmosis | 0.080-0.180 (0.124) | 0.157 |

## ANALYSIS

Before the pile was exposed and the measurements taken, predictions were made of the movement of the pile during the various stages of the test by six foundation engineers practicing both in the United States and abroad (3). The range in predicted movement and the measured movements are given in Table 1. Although there was a fairly large range in the predicted movements, there was general agreement among the predictors on how the downdrag forces and the related movements could be computed. For example, all but one of the predictors calculated the negative skin friction acting along the soil-pile interface using some form of the equation $f_s = \beta \bar{p}_o$, where $f_s$ is the negative skin friction, $\beta$ is a constant depending on the soil conditions, and $\bar{p}_o$ is the effective overburden pressure.

Using the same procedures set forth by the six predictors, it was possible (5), by varying the parameter $\beta$ and by selecting reasonable values for the positive skin friction, $f_s$, and the coefficient of subgrade reaction in the bearing layer, $k_s$, to back-figure a number of load-depth curves that fit the observed data. For example, the load-depth curves in Figure 12 yield movements that are within 3 percent of those measured during the field test.

These curves and the corresponding movements were obtained as follows: First, the effective overburden pressure-depth relationship ($\bar{p}_o$ - z) was obtained by subtracting the measured pore water pressures from the sum of the original geostatic stresses and the distributed stresses induced by the embankment loading (4, 7); second, using the assumed values of $\beta$ given in the figure, the negative skin friction values were computed and plotted as a function of depth; third, the load-depth curves ($P_z$ - z) for depths above the bearing stratum at various stages of the test were determined by integrating the skin friction-depth curves with a planimeter and multiplying the integral by the outer box perimeter of the pile, i.e.,

$$P_z = \text{perimeter} \int_0^z p_o dz$$

fourth, the load-depth curves for depths within the bearing stratum were obtained using the positive skin friction values given in the figure; fifth, the elastic extension of the pile for the various stages of the test was determined by dividing the planimetered area between the load-depth curves corresponding to those stages by the cross-sectional area and the modulus of elasticity of the pile, i.e.,

$$\delta_E = 1/AE \int_0^L P_z dz$$

and sixth, the rebound of the pile tip, computed by dividing the change in bearing pressure by the selected value of the coefficient of subgrade reaction, was added to the elastic extension of the pile to obtain the total movement, $\delta_T$.

In all, a total of three solutions among the more than 20 trials analyzed gave movements close to those measured in the field test. In these three solutions $\beta$ for the sand varied from 0.30 to 0.40, $\beta$ for the clay varied from 0.20 to 0.25, the positive skin friction in the bearing stratum varied from 1.25 to 1.40 tsf (120 to 135 kPa), and the coefficient of subgrade reaction varied between 4,300 and 5,500 t/ft$^3$ (1370 and 1750 mN/m$^3$). The total load in the pile at the top of the bearing stratum varied from 215 to 245 tons (1900 to 2170 kN).

The values of $\beta$ back-figured from the observed movements are in good agreement with data obtained by other investigators (1, 2), as are the positive skin friction values for the bearing stratum (6). The values of the coefficient of subgrade reaction are higher than those measured by Thompson and Brierly (8) for a compression pile in another glacial till in the Boston area, but a higher value would be expected for pile rebound because the plastic settlements that occur during compression do not occur during rebound.

Figure 12. Load-depth curves in pile at various stages of test.

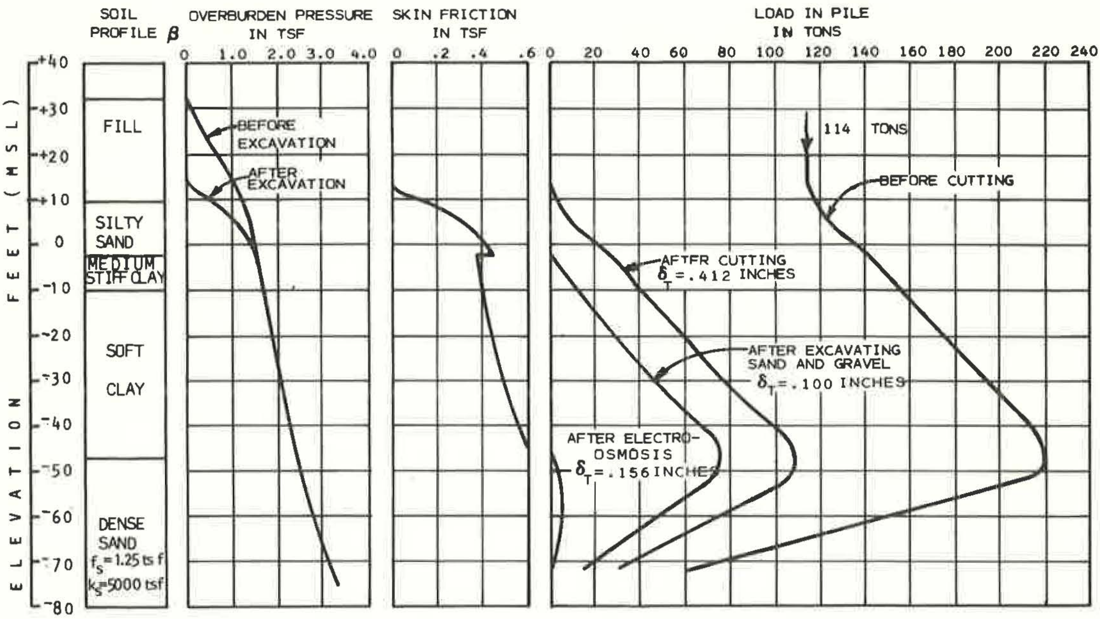
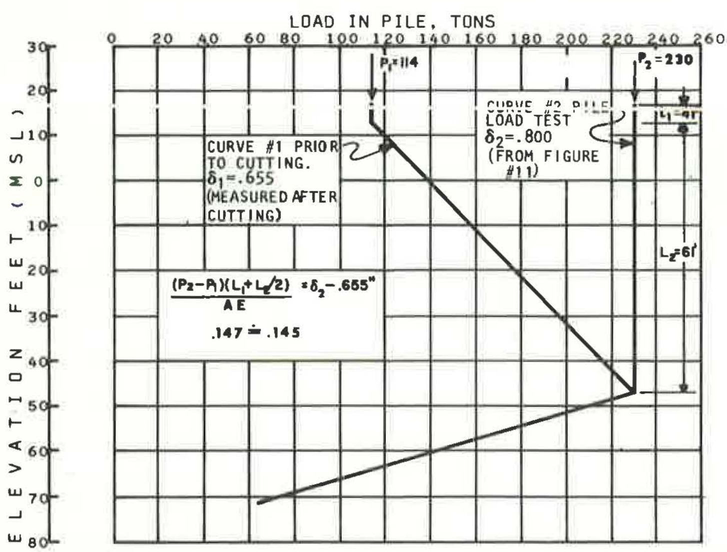



Figure 13. Procedure for determining downdrag load from pile load test.




BY TRIAL AND ERROR

ELASTIC MOVEMENT CORRESPONDING TO AREA BETWEEN CURVES #1 AND #2 MUST EQUAL MEASURED MOVEMENT CORRESPONDING TO CURVE #2 MINUS MEASURED MOVEMENT CORRESPONDING TO CURVE #1

The total load in the pile at the top of the bearing stratum was also back-figured from the results of the pile load test (Figure 11). The procedure, which assumed only that the downdrag load in the pile increases linearly with depth, is shown in Figure 13. Using this procedure, the total load in the pile at the neutral point prior to cutting the pile was determined to be 230 tons (2050 kN). This value, along with the values determined using the previous analysis, indicate that the downdrag load in the pile was greater than 100 tons (890 kN).

## CONCLUSIONS

1. The total downdrag load carried by the test pile was at least 100 tons (890 kN) and possibly as much as 130 tons (1160 kN).
2. The load and moment transmitted by the abutment to the top of the pile were 114 tons (1015 kN) and 1,140 in.-kips (129 kN-m) respectively.
3. Even though at least one of the piles was carrying almost four times the design load and several of the piles had pulled out of the pile cap, the abutment had settled less than ½ in. and appeared to be performing satisfactorily.
4. Downdrag loading can be predicted reasonably well using the equation $f_s = \beta\, p_o$ to determine the maximum negative skin friction and using empirically determined parameters for the soil constant $\beta$. Values of $\beta$ equal to 0.20 to 0.25 for clay and 0.35 to 0.50 for sand appear to be reasonably conservative.
5. Not enough is known about the interaction between pile and pile cap to predict the load and movement transmitted from the pile cap to the pile. More field measurements in this area are needed.

## ACKNOWLEDGMENTS

The research described in this paper was performed at the Massachusetts Institute of Technology under the general direction of T. W. Lambe. Other major participants in the research were L. A. Wolfskill and Samuel A. Leifer. The research project was sponsored by the Massachusetts Department of Public Works and the Federal Highway Administration. The author wishes to express his appreciation to those mentioned and to all of the other staff in the M.I.T. Department of Civil Engineering who contributed to the research. The opinions, findings, and conclusions expressed are those of the author and not necessarily those of the Massachusetts Department of Public Works or the Federal Highway Administration.

## REFERENCES

1. Bjerrum, L., Johannessen, I. J., and Eide, O. Reduction of Negative Skin Friction on Steel Piles to Rock. Proc. 7th Int. Conf. on Soil Mechanics and Foundation Engineering, Mexico City, 1969, Vol. 2, pp. 27-34.
2. Endo, M., Minou, A., Kawasaki, T., and Shibata, T. Negative Skin Friction Acting on Steel Pipe Piles in Clay. Proc. 7th Int. Conf. on Soil Mechanics and Foundation Engineering, Mexico City, 1969, Vol. 2, pp. 85-92.
3. Garlanger, J. E., D'Appolonia, D. J., Davis, E. H., Esrig, M. I., Focht, J., and Perez-Guerra, G. Prediction of the Downdrag Load at Cutler Circle Bridge. Symposium on Downdrag of Piles, M.I.T. Department of Civil Engineering, 1973.
4. Gray, H., and Hooks, I. J. Charts Facilitate Determination of Stresses Under Loaded Areas. Civil Engineering, ASCE, June 1948.
5. Leifer, S. A. Field Evaluation of Negative Skin Friction Forces at a Bridge Abutment. Masters thesis, M.I.T. Department of Civil Engineering, 1973.
6. Mansur, C. J., and Kaufman, R. I. Pile Load Tests, Low Sill Structure, Old River, Louisiana. Trans. ASCE, Vol. 123, 1958, pp. 715-743.
7. Osterberg, J. O. Influence Values for Vertical Stresses in a Semi-Infinite Mass Due to an Embankment Loading. Proc. 4th Int. Conf. on Soil Mechanics and Foundation Engineering, London, 1957.
8. Thompson, D. E., and Brierly, G. S. Load Transfer in Long Pipe Piles in Clay. To be published.