LONG-TERM LOAD TRANSFER IN END-BEARING PIPE PILES

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Load transfer in small groups of concrete-filled steel pipe piles is determined by means of electric resistance stress and strain meters located in the concrete core along the length of the piles. These load-transfer measurements are compared with those obtained on individual instrumented piles that were load-tested. Measurements of dragdown loads are presented and compared with design values. The effects of live load superimposed on piles subjected to dragdown loads are examined.

•CURRENT knowledge of load transfer in piling is derived from short-term load tests on instrumented piles (e.g., 7, 8, 9, 17, 19, 20). Presumably these results are applied to the design of foundation piling, although it is generally recognized that load transfer in short-term tests may differ from long-term behavior.

Investigations of the long-term behavior of load transfer have been limited to measurements of dragdown loading due to negative skin friction (2, 3, 4, 10, 11, 12, 13, 15). Understandably, these investigations have concentrated on situations where the drag forces were large in comparison with the applied loads. Ground settlements accompanying these dragdown loads have generally been quite substantial.

This paper reports the results of long-term measurements of load transfer in small groups of end-bearing pipe piles. The piles, which are approximately 50 ft (15 m) long, were driven to shale bedrock through successive layers of hydraulic sand fill, a tidal marsh deposit of organic silts and peats, and a glacial outwash and lake deposit. The tidal marsh deposit was stabilized by preloading, but the upper 20 to 30 ft (6 to 9 m) of the piles are being subjected to dragdown loads due to secondary compression of these organic soils. All piles are supporting mainly structural loads, and dragdown constitutes a small fraction of the design loading. The present study deals specifically with the following points:

1. Development of dragdown loading resulting from remolding of the cohesive soils because of pile driving and the soils' subsequent reconsolidation;
2. Development of dragdown loading due to secondary compression of the tidal marsh deposit;
3. Comparison of short-term load transfer as measured in an instrumented test pile with long-term load transfer in foundation piles; and
4. The effect of superimposing transient loading on a pile group subjected to dragdown loading.

PROJECT DESCRIPTION

The expansion of Newark International Airport in New Jersey includes many important structures that are supported on piling. Among these structures are three terminal buildings and an adjoining elevated roadway, a central heating and refrigeration plant, and several bridges for the roadway system. The entire project required more than 500,000 linear feet of piling.

Foundation support is provided by steel pipe piles driven to bedrock and filled with concrete. A 12.75-in. (32.4-cm) outside diameter steel pipe of 0.25 in. (0.64 cm) wall

*This paper is based on work performed while the authors were with the Port Authority of New York and New Jersey.

thickness provides an allowable design capacity of 80 tons (712 kN). Allowable pile loads were determined by deducting 75 percent of the computed dragdown load from the design capacity. Piles are closed at the tip with either a flat plate or an angle-fin driving point.

Subsurface Conditions

The airport site has been developed by filling over a tidal marsh deposit consisting of from 2 to 20 ft (0.61 to 6.1 m) of extremely soft and compressible organic silts and peaty soils. The organic soils are generally underlain by fine-grained gray sands. Below the sands are glacial outwash and lake deposits, glacial till, and bedrock.

The glacial lake deposits consist of reddish-brown silts and clays, frequently varved and preconsolidated to pressures of 3 to 4 tsf (287 to 383 kPa), about $1\frac{1}{2}$ tsf (144 kPa) in excess of the existing overburden pressures. The lake deposits are interposed with outwash material eroded from the more highly elevated outwash deposits to the west of the airport.

Bedrock is a red shale and occurs at depths ranging from 40 to 100 ft (12 to 30 m) below sea level. The upper surface of the bedrock is frequently badly weathered to depths ranging from a few inches to several feet. Overlying the bedrock is usually a stratum of glacial till, a very dense clayey silt with gravel, cobbles, and boulders.

The tidal marsh deposit has been surcharged with sand fill to remove primary settlements in paved areas. Sand drains were used under roadway embankments that interconnect the terminal buildings and are about 10 ft (3 m) above the general grade. However, post-construction settlements have occurred due to secondary compression of the tidal marsh and have caused dragdown loading on those structures that are pile-supported.

Pile Installation

Piles were driven to a resistance of 20 blows/in. (8 blows/cm) with air- or steam-operated hammers, producing 24,000 to 26,000 ft-lb (32.5 to 35.3 kJ) of rated energy per blow. Piles drove easily through the overburden and penetrated several inches into the shale bedrock. During driving, measurements of pile heave were taken and individual piles that heaved more than $\frac{3}{16}$ in. (0.48 cm) were redriven. In addition, for pile groups where more than $\frac{3}{8}$ in. (0.95 cm) heave was observed, all vertical piles were redriven.

Pile Instrumentation

Three pile groups were selected for long-term measurements of load transfer. The groups consisted of 5 to 8 piles, and they were located in areas where dragdown loading was expected to vary because of differences in the anticipated magnitude and rate of post-construction settlement. Since a principal objective of the testing was to measure the effect of superimposing a live load on piling subject to dragdown loading, pile groups supporting elevated roadways were selected for load measurement because these groups could be conveniently subjected to live loading by positioning heavy vehicles on the roadway. The locations of the instrumented groups are shown in Figure 1, and a simplified geologic profile is shown in Figure 2.

Electric resistance strain and stress meters (Carlson Elastic Wire Strain and Stress Meter) were used for pile load measurements (18). A string of from 3 to 6 strain meters was placed in each pile during placement of the concrete core. The meters were mounted in positioning brackets that served to align them with the axis of the pile and protect them during the concrete filling operation (Figure 3). This type of instrumentation was selected because previous experiences with these strain meters had shown them to be stable and reliable. In addition to reading strains to an accuracy of a few microinches, the meter is also an accurate thermometer. As a minimum, each pile of the group had strain meters at the pile top, the bottom of the tidal marsh deposit, and the pile tip so that pile loads could be measured at these locations.

To permit determination of the creep strain behavior of the concrete core, a pair of strain and stress meters was positioned at the tops of selected piles. Dummy strain

Figure 1. Location plan.

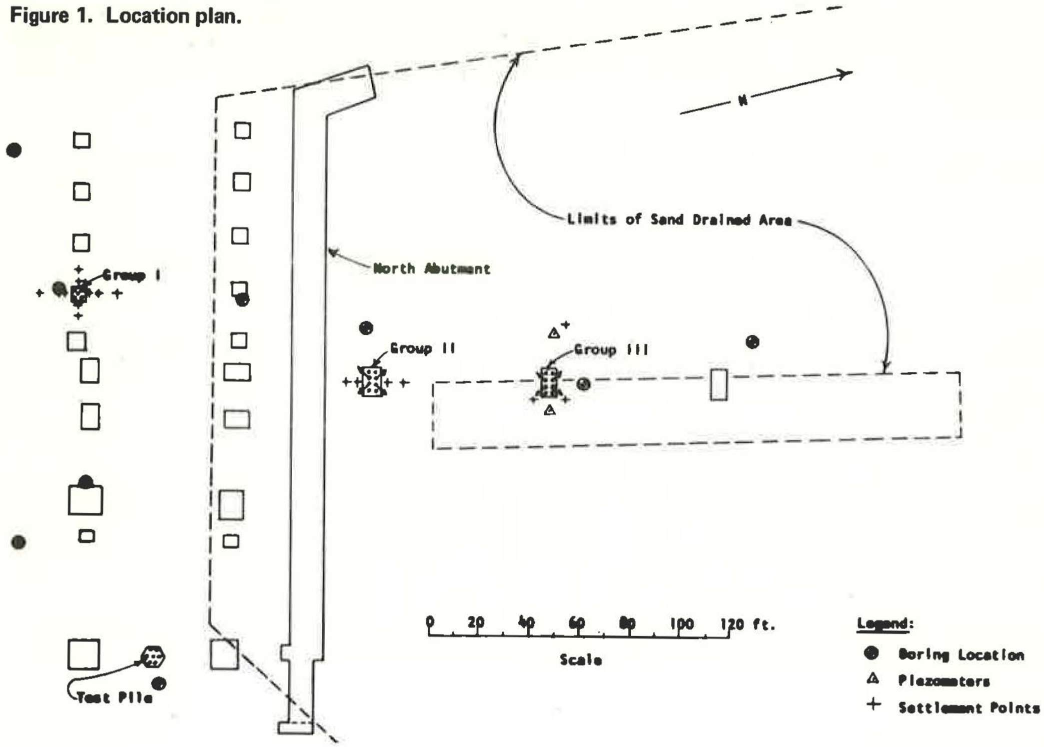


Figure 2. Section through foundations.

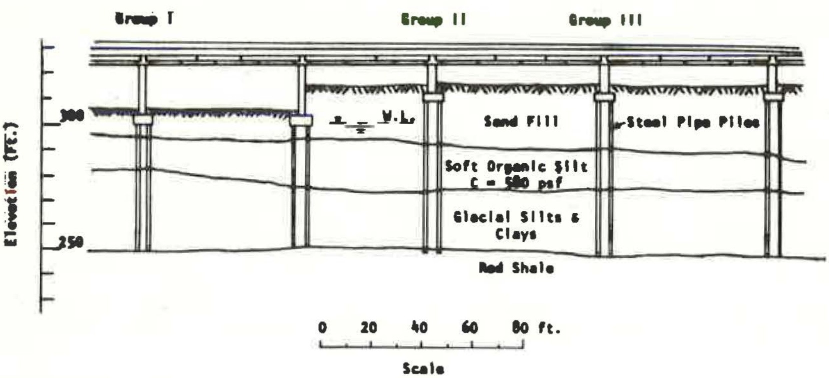


Figure 3. Installing strain meter and positioning bracket.

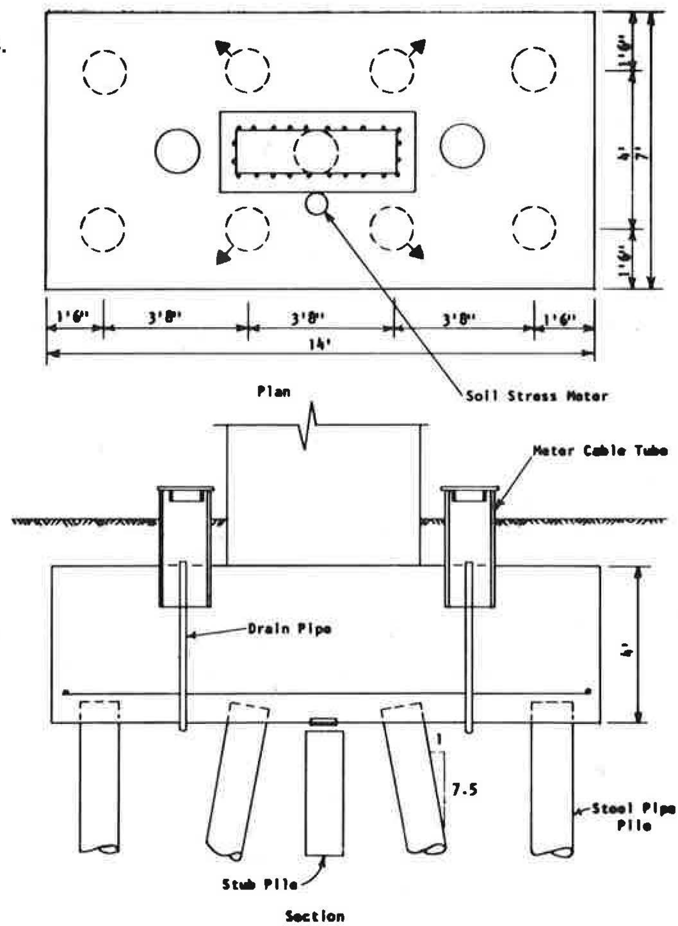


Figure 4. Plan and section, instrumented groups 2 and 3.

meters were installed in short, unloaded stub piles to record strains due to shrinkage and temperature changes. Soil stress meters were used to measure the contact pressure at the base of pile caps. The arrangement of instruments for pile groups 2 and 3 is shown in Figure 4.

Prior to construction of the foundations, several instrumented piles were load-tested to prove out the proposed design loading and to measure the loads carried by skin friction and point bearing. To provide specific information in the vicinity of the instrumented pile groups, an additional test pile was instrumented and load-tested during foundation construction. This information provides a direct comparison of load transfer in test piles with that measured in the foundation piles. As shown in Figure 1, the test pile was located 150 ft (46 m) east of group 1. The soil profile and driving record are shown in Figure 5, and the load-settlement record is shown in Figure 6. Pile settlements were very small. Under the total test load of 200 tons (1779 kN), the gross settlement was only 0.045 ft (1.37 cm) and net settlement 0.018 ft (0.55 cm).

For the test pile, the fill and organic silt surrounding the pile were removed after pile installation and replaced by a mud slurry. The load at the pile top was measured with an electric resistance load cell. With this arrangement, it was possible to measure the pile stiffness with the upper strain meter when the test load was cycled at the end of each load increment. These data are used to calculate the pile loads at the other meter locations.

Computation of Pile Loads

Axial loads in foundation and test piles were determined from strain measurements by multiplying measured strain by the pile stiffness after correcting for strains due to temperature variations, shrinkage, and creep. The general accuracy of the instrumentation and methods of computation was verified by a number of checks:

1. A comparison of concrete stresses obtained from direct measurements by Carlson stress meters placed in selected piles with the indirect measurements provided by strain meters placed at the same location;
2. A comparison of computed loads from instrumentation measurements with calculated structural loads determined from construction records (Figure 7); and
3. A comparison of computed loads from instrumentation measurements with calculated live loads imposed during the transient load test.

Correction for concrete creep at a sustained high stress level upon construction completion was achieved by adopting a viscoelastic model consisting of a chain of Kelvin models and a Maxwell model (1, 16). Test results with computed pile loads are shown in Figures 5, 7, 8, 10, and 11.

LOAD TRANSFER

Load transfer is defined as the transfer of load between a pile and the surrounding soil. It is considered to be positive when load is being transferred from pile to soil. When the surrounding soil settles relative to the pile and load is transferred from soil to pile, negative load transfer, or dragdown, occurs.

To summarize the results of this study and to compare these results with the findings of other investigators, it is useful to express load transfer in fundamental terms. Load transfer in piles has been expressed in terms of effective stress (6, 14). According to this concept, shaft friction due either to positive skin friction or negative skin friction (dragdown) is related to the effective overburden pressure ($\bar{p}$) by the simple relationship

$$F_s = K \tan \phi' \bar{p} \tag{1}$$

or

$$F_s = \beta \bar{p} \tag{2}$$

where K is the coefficient of lateral earth pressure and ϕ' is the effective angle of friction between the soil and the pile shaft.

Figure 5. Load transfer and soil properties, test pile.

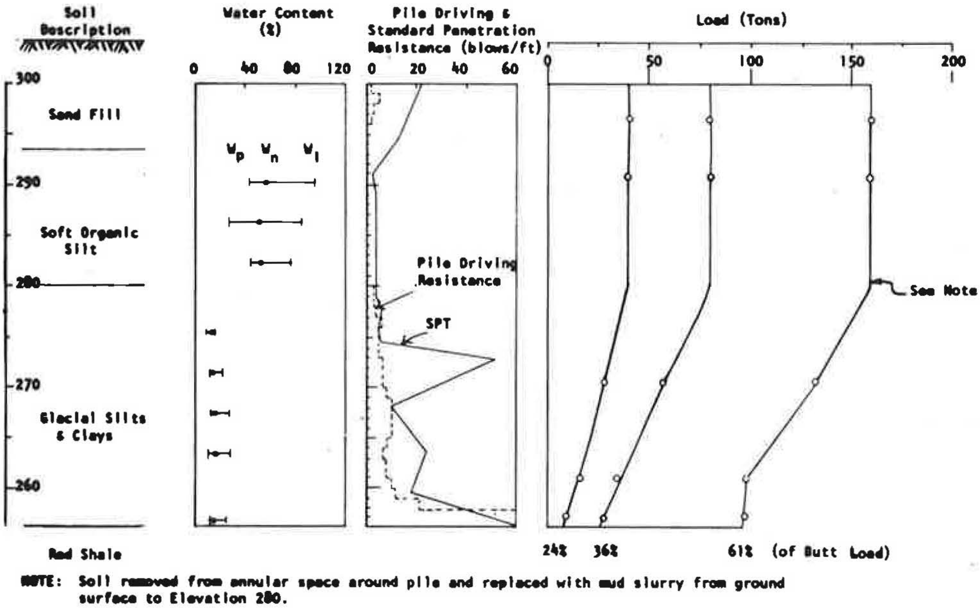


Figure 6. Load settlement diagram, test pile.

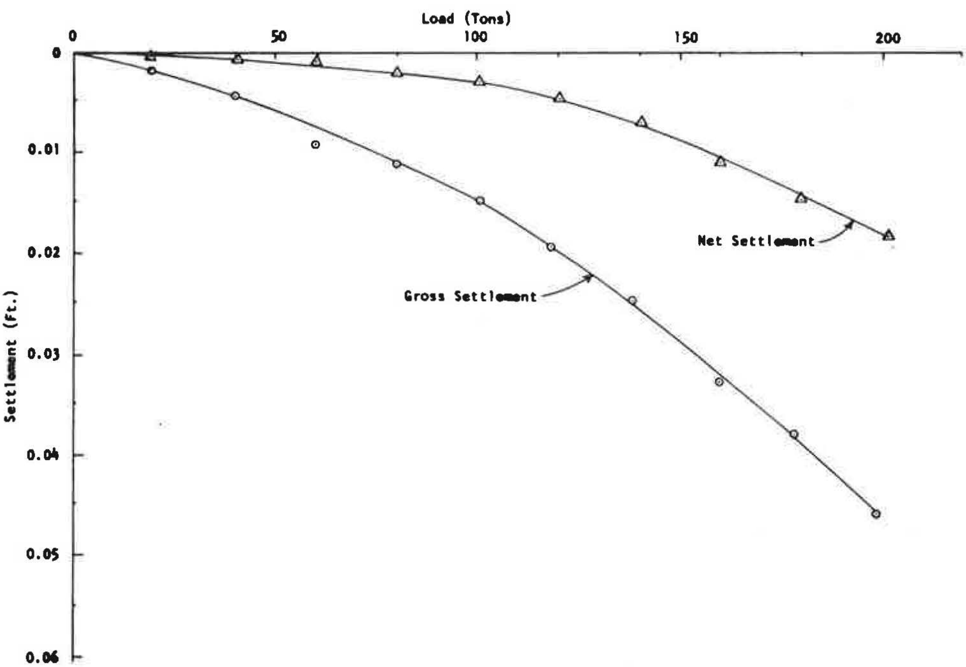


Figure 7. Comparison of calculated loads versus instrumentation.

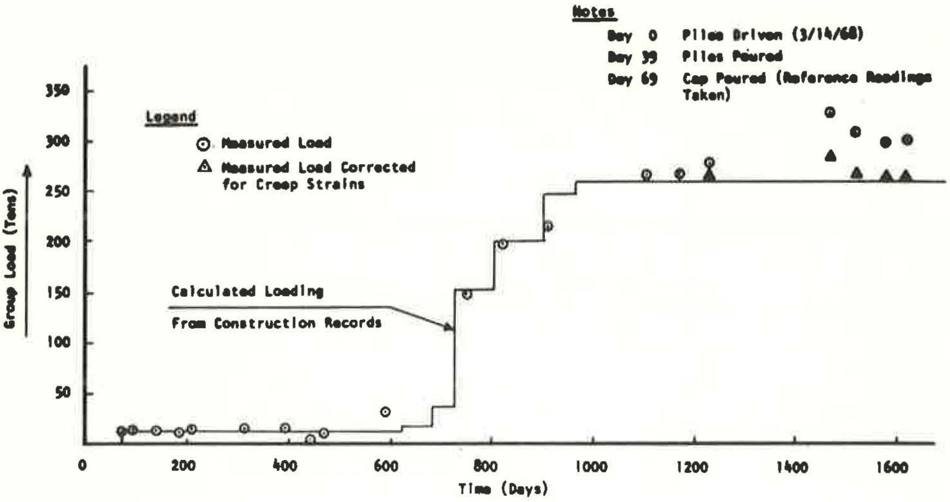
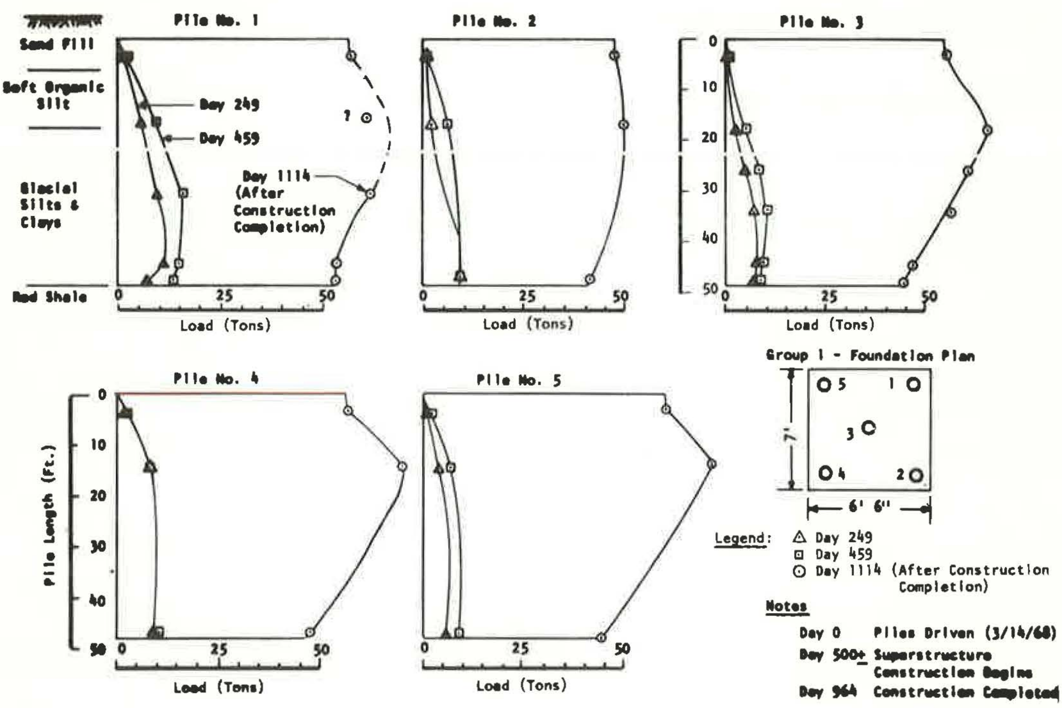


Figure 8. Load transfer, group 1.



The magnitude of the lateral earth pressure coefficient K depends on the soil type, the stress history of the soil, and the method of pile installation. The value of ϕ' depends on the soil type and the properties of the pile surface. Average values of β can be obtained empirically from measurements of load transfer in instrumented test piles by restating Eq. 2 as

$$\beta = \frac{F_s}{\bar{p}} \tag{3}$$

and taking F_s as the slope of the load transfer curve.

For normally consolidated clays, the results of a large number of pile load tests show that the value of β ranges between the limits of 0.25 to 0.4 (6). Long-term measurements of negative skin friction yield values of β that range from 0.20 to 0.25 for tests where the dragdown loads were fully developed (14).

For overconsolidated clays, the effective stress approach is more complex, mainly because of the effect of remolding resulting from pile driving and because of wide variation in the value of the coefficient of lateral earth pressure (K). For these reasons, there is considerable scatter in the measured values of β; however, β values are larger than for normally consolidated clays.

TEST RESULTS

Settlement and Piezometer Observations

Settlement observations on pile caps show that the application of structural and dragdown loads caused a maximum cap settlement of only $^1/_8$ in. (0.32 cm). Post-construction observations of ground settlements and piezometric levels indicate that the preload was effective in stabilizing the tidal marsh deposits. Ground settlements over a $2^1/_2$-year period show settlement rates that are generally equal to or less than the rates of settlement due to secondary compression, as predicted on the basis of laboratory tests. For groups 1 and 2 and the sand-drained portion of group 3 (Figure 1), the maximum rate of ground settlement is 0.1 in./year (0.25 cm/year). Piezometer data in the non-sand-drained area adjacent to group 3 show excess pressures of 2.4 psi (16.5 kPa) at the center of the compressible layer, indicating that the tidal marsh deposit is slightly underconsolidated at this location. The maximum rate of ground settlement in this area is 0.3 in./year (0.76 cm/year).

Dragdown Loads

Test results show that dragdown loading resulted from two distinct phenomena:

1. Remolding of the cohesive soils because of pile driving and the subsequent reconsolidation of the soils, and
2. Secondary compression of the tidal marsh deposit.

The piles for group 1 were driven 19 months prior to the start of construction of the superstructure. This delay in the construction provided an opportunity to study dragdown loading resulting from remolding of the cohesive soils because of pile driving and the soils' subsequent reconsolidation. Dragdown loading due to this phenomenon developed slowly, reaching a maximum about 250 days after pile installation. As shown in Figure 8, a neutral point defining the change from negative to positive skin friction was reached deep in the glacial lake deposit at a depth approximately equal to 90 percent of the pile length. Thereafter, the development of drag load due to secondary compression of the tidal marsh deposit caused a progressive increase in compressive pile strains that gradually reduced the negative skin friction in the underlying glacial lake deposit. With the application of the structural loads, positive load transfer developed in the glacial silts and clays and dragdown occurred entirely within the tidal marsh deposit and the overlying sand fill.

As shown in Figure 9, dragdown loading due to secondary compression of the tidal marsh developed slowly, reaching a magnitude of about 9 tons/pile (80 kN/pile) after 400 days. With the application of the structural loads, there was an abrupt decrease

Figure 9. Development of dragdown with time, group 1.

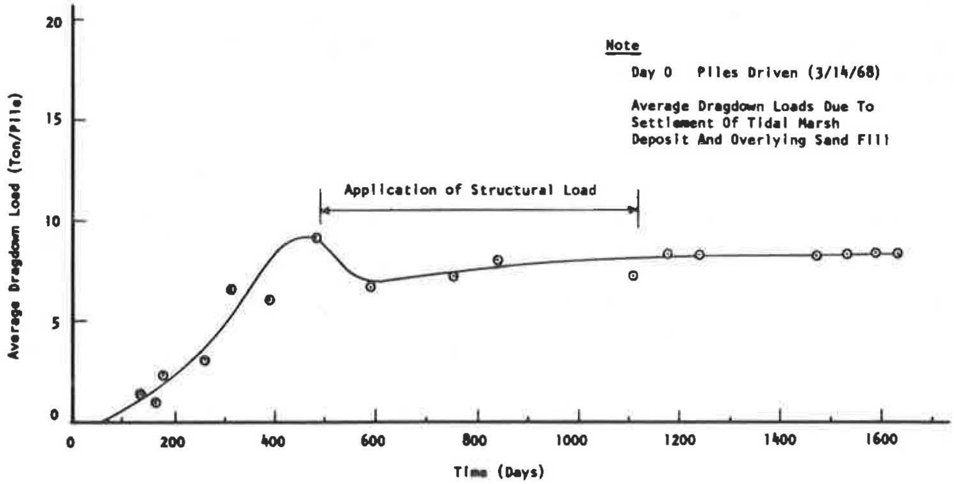


Figure 10(a). Load transfer in vertical and batter piles, group 2.

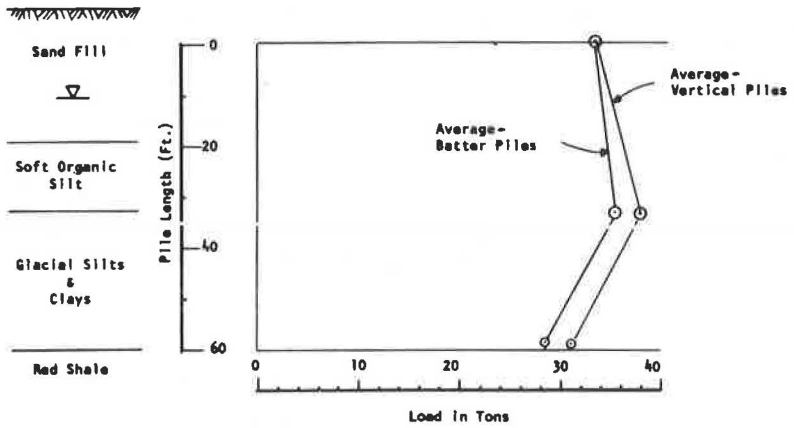
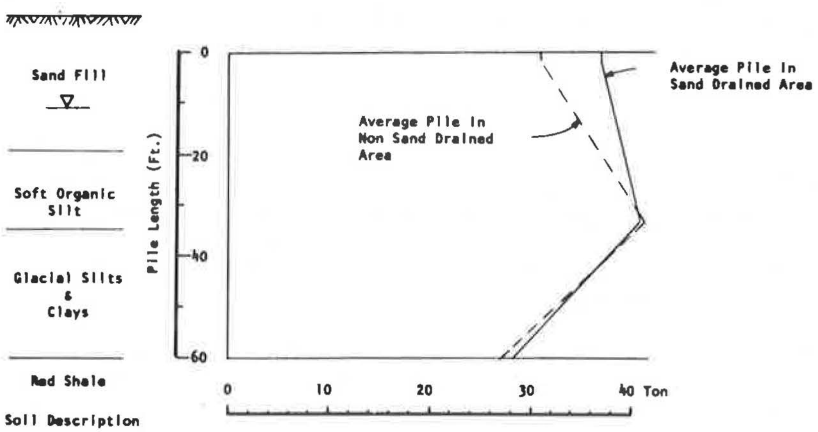


Figure 10(b). Load transfer in sand-drained and non-sand-drained areas, group 3.



in dragdown loading followed by a very gradual increase; 1,600 days after pile installation the drag load averaged 8.2 tons/pile (73 kN/pile).

For pile groups 2 and 3, the construction of the roadway structure started shortly after installation of the piles, so there was insufficient time to develop drag loading resulting from remolding and subsequent reconsolidation of the cohesive soils. Load transfer for pile groups 2 and 3 is shown in Figure 10.

Very little difference was observed between dragdown loading on vertical piles when compared to batter piles. The design of the foundations had anticipated that batter piles would be subjected to larger dragdown loads due to the weight of overlying soil acting on that portion of the batter pile that is within the zone of soil settlement. However, ground settlements to date have been small and apparently not sufficient to cause additional dragdown loading.

A comparison of the measured post-construction dragdown load with calculated values is given in Table 1. There are two observations of particular significance:

1. There is good agreement between measured and computed values for piles in the non-sand-drained areas, and
2. In areas treated by sand drains, the drag forces were reduced and ranged from only 22 percent to 39 percent of calculated values. This indicates that ground settlements in sand-drained areas were less than the threshold values needed for the full development of dragdown forces.

Positive Load Transfer

It is particularly interesting to note the differences in positive load transfer between foundation piles and the test pile. For foundation piles, the positive load transfer within the glacial soils was far less than that measured in the test pile.

Measurements of load transfer for the test pile are shown in Figure 5. A substantial amount of the applied load is carried by friction. As load was applied, the magnitude of skin friction increased gradually at a decreasing rate. The average value of mobilized skin friction was 810 psf (38.8 kPa) for the 40-ton (356-kN) loading and 1,520 psf (72.8 kPa) for the 160-ton (1423-kN) loading.

As the magnitude of mobilized skin friction approached a constant value, an increasing proportion of each additional increment of load was transferred to the pile tip. In the high load range most of the load applied is supported by point resistance.

Positive skin friction in the foundation piles was relatively constant with depth and much smaller than that observed in the test pile. A comparison of these data is shown in Table 1 and Figure 11. For foundation piles, positive skin friction mobilized within the glacial soils was only 16 to 42 percent of that measured in the test pile at comparable applied loads, and in terms of peak skin friction the range was only 10 to 26 percent of that mobilized by the test pile. Due to reduced positive skin friction and the superimposed dragdown loads, a very high proportion of the applied load reached the tip of the foundation piles. For group 2, which is subjected to only minor dragdown loading, 84 percent of the applied load reached the pile tip.

Transient Load Test

Pile group 1 supporting the arrival roadway was selected to measure changes in load transfer resulting from the superposition of transient loading on piles subjected to dragdown loading. Two trucks were positioned to produce column loads of 20, 35, and 50 tons (178, 311, and 445 kN). All meters were read when the trucks were in a parked position.

As shown in Figure 11, most of the applied load was carried in friction. The distribution of load transfer was similar to that observed in the test pile. The application of the 50-ton (445-kN) transient load reduced the dragdown loading acting on the cap from 38 tons to 15 tons (338 kN to 133 kN), a 60 percent reduction. Figure 12 shows the reduction in dragdown loading as a function of the applied transient load. The total test load of 50 tons (445 kN) was cycled 15 times with no apparent change in the distribution of load transfer.

Table 1. Summary of load transfer characteristictics, test pile and pile groups.

Pile Group	Point Load Percent Butt Load	Average Positive Skin Friction, psf (β-Values)	Measured Post-Constr. Drag/Pile, tons	Calculated Dragdown Load, tons[a]
1	84	265 (β = 0.13)	8.2[b]	6.5
2	84	170 (β = 0.05)	3[c]	13.5
3	87	300 (β = 0.08)	5.5[c] 15.5[b]	13.5
Test pile[d]	25	940 (β = 0.31)		
Test pile[e]	61	1,520 (β = 0.50)		

[a]Dragdown loads calculated with β = 0.4 for sand fill, β = 0.3 for organic silt.
[b]Piles located in non-sand-drained area.
[c]Piles located in sand-drained area.
[d]50-ton load.
[e]160-ton load.

Figure 11. Comparison of load transfer, test piles versus foundation piles.

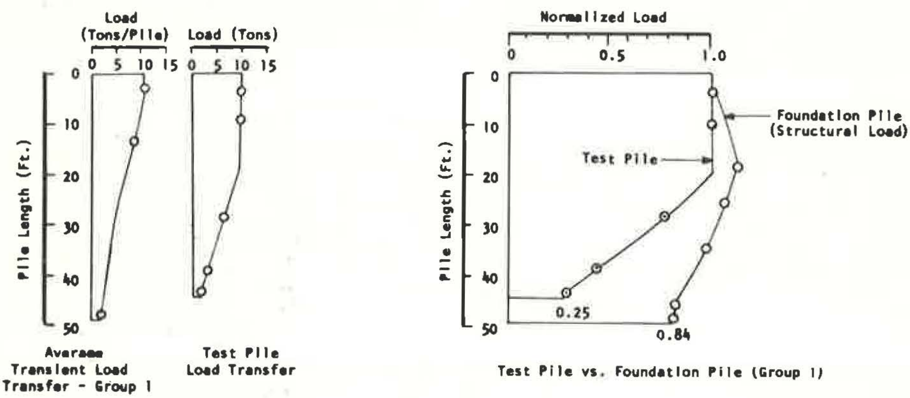
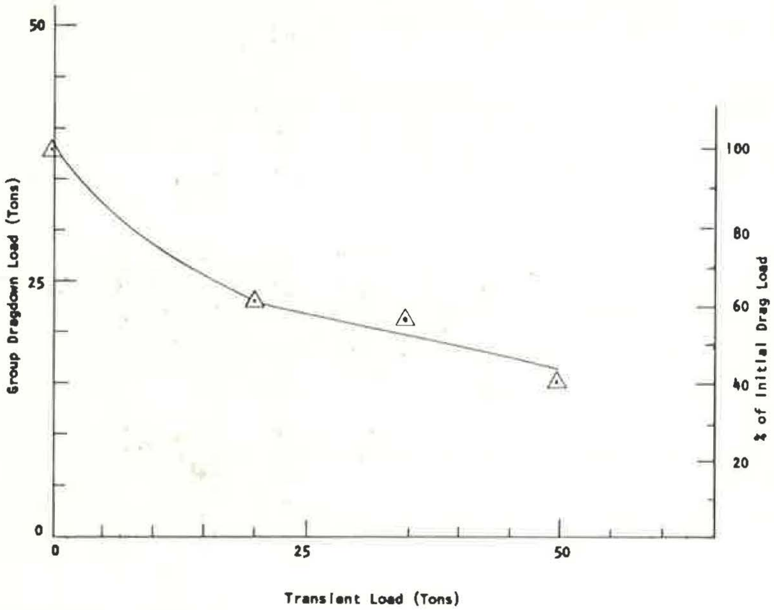


Figure 12. Dragdown load during transient load test, group 1.



Miscellaneous

Soil stress meters located in the soil beneath pile caps recorded small compressive stresses equivalent only to the weight of the pile cap, verifying that column loads were fully supported by the piling. For pile group 1, soil stresses averaged 3.5 psi (24.2 kPa) in the period of construction (2 years) as compared to 3.6 psi (24.8 kPa) representing the weight of the cap only.

CONCLUSIONS

The conclusions of this testing program are necessarily limited by the particular geologic conditions of the site. Comparisons are difficult because of the lack of previous data concerning long-term behavior of pile groups, and there are only a few examples in the literature of measurements of dragdown loading in areas of small ground settlements. This study is in essence a case history; however, some of the following conclusions may have general application to end-bearing piles driven through cohesive soils:

1. Load transfer in foundation piles differed significantly from that measured in test piles. Values of mobilized positive skin friction in the foundation piles ranged from 16 percent to 42 percent of those measured in the test pile at comparable applied butt loads and were only 10 percent to 26 percent of measured test pile values for peak skin friction. Due to dragdown loading and reduced positive skin friction, most of the load applied to the foundation piles reached the pile tips.
2. Dragdown loading resulted from two causes: (a) remolding of the cohesive soils because of pile driving and the subsequent reconsolidation of the soils, and (b) secondary compression of the tidal marsh deposit.
3. The application of structural loads obliterated the effects of dragdown loading due to remolding, but it appears that this effect contributed to the reduction in positive skin friction developed in the glacial lake deposit.
4. For pile groups located in non-sand-drained areas, the measured dragdown loading agreed closely with computed values for ultimate dragdown. However, for pile groups in sand-drained areas, the measured dragdown loads were considerably less than computed values. Dragdown loads on piles battered 1 on 7.5 were no greater than those on vertical piles; however, ground settlements to date have been very small.
5. The application of transient loading of 50 tons (445 kN) on pile group 1 reduced the dragdown loading due to compression of the tidal marsh deposit from 38 tons to 15 tons (338 kN to 133 kN), a 60 percent reduction. Fifteen applications of load cycling did not alter the load transfer behavior.

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