Downdrag on a three-pile group of pipe piles

Frottement négatif d'un groupe de trois pieux cylindriques

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SYNOPSIS The development of downdrag load on a group of three piles spaced at four pile diameters was observed for 6½ years. The 32 m long, 324 mm diameter piles were driven through a granular test embankment into a deep deposit of compressible clayey silt. There was no group effect on the development of load. The maximum was 1000 kN in 1½ years, and 900 kN after 6½ years due to changes in embankment loading. The distribution of downdrag loads on the piles for various conditions of vertical effective stress in the soil was adequately estimated with effective stress equations using K_o obtained from a field pullout test and the coefficient of friction between the soil and pile from laboratory tests. The pullout failure load was equivalent to the downdrag and positive skin friction load.

INTRODUCTION

A new bridge over the Saint John River, in Fredericton, was supported on steel end-bearing piles driven through compressible silts into a dense gravel formation. A high approach embankment was required, which could generate large downdrag loads. To obtain design parameters for the piles, the New Brunswick Department of Transportation and the Division of Building Research, National Research Council Canada implemented a field study in 1977 on seven full-scale steel pipe and "H" test piles driven through a granular test fill. Axial compression and pullout tests performed on two end-bearing and two friction piles, were reported by Bozozuk et al. (1979). The three-pile group was used to observe the long-term development of negative skin-friction load.

The axial distribution of downdrag loads calculated from measured pile compressions related to different pore water pressures and vertical effective stresses in the soil is presented. Pile load distributions estimated from effective stress equations using soil parameters obtained from field pullout tests and laboratory soil-pile friction tests are also shown.

TEST EMBANKMENT

The granular test fill was constructed to a height of 11 m in two stages (Figure 1). Stage 1 was raised to an elevation of +3.0 m, providing a good base for installing field instrumentation, and Stage 2 to +9.0 m, providing a surface area of 25 by 40 m. It was constructed with sand fill, with an average measured in-place density of 1825 kg/m³.

From 26 August to 19 October, 1977, dredged sand fill (S) was placed over a large area around the test embankment (Figure 1). The average measured in-place density was 1800 kg/m³. A granular surcharge was stock-piled to +16.5 m just south of the test piles from 8 December, 1977 to 18 April, 1979, when the south approach ramp to the bridge was constructed to +14.5 m. The maximum range of river levels was +8.0 m to +0.8 m.

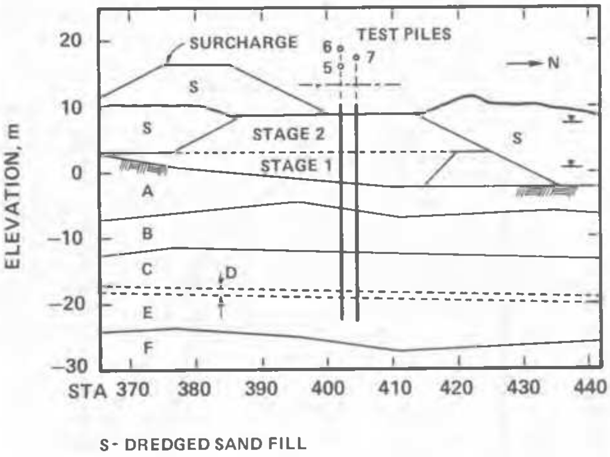

Fig. 1 Section through test embankment showing test piles, dredged sand fill and subsoil formations

SUBSOILS

The soil profile consisted of six major horizontal soil formations (A to F on

Figure 1). Layer A (4 m thick) was heterogeneous compressible soft organic silt and sand with some pebbles and wood. Undisturbed soil samples could not be obtained, but it was assumed to have a density of 1680 kg/m^3. The standard penetration test gave N = 5.

Layer B was a grey layered clayey silt about 6.5 m thick. The natural water content (W) varied from 23 to 38%, the liquid limit (W_L) was 29% and the plasticity index (I_p) was 9%. Grain size analysis indicated 40% clay and 60% silt sizes. The average density (γ_m) was 1840 kg/m^3. In situ vane shear strengths varied from 30 to 50 kPa, and N = 13.

Layer C was a 5.5-metre thick grey-brown layered clayey silt consisting of 45% clay and 55% silt. Water content was 29 to 37%, W_L about 34% and I_p was 12%. In situ vane shear strengths varied from 60 to 90 kPa, N = 14 and γ_m was 1840 kg/m^3.

Measured preconsolidation pressures (σ_p') (Figure 2a) show that the formations are highly overconsolidated. Pressures of 375 kPa will not be exceeded by embankment loading.

Layer D was varved brown clay and silt, 1 m thick.

Layer E was a 7-metre thick layered brown clayey silt with W of 35 to 45%, W_L about 40% and I_p of 20%. It was more clayey, with 64% clay size and 36% silt size. The average γ_m was 1840 kg/m^3. In situ vane shear strengths varied from 90 to 120 kPa, and N = 10. The four test piles terminated in this formation.

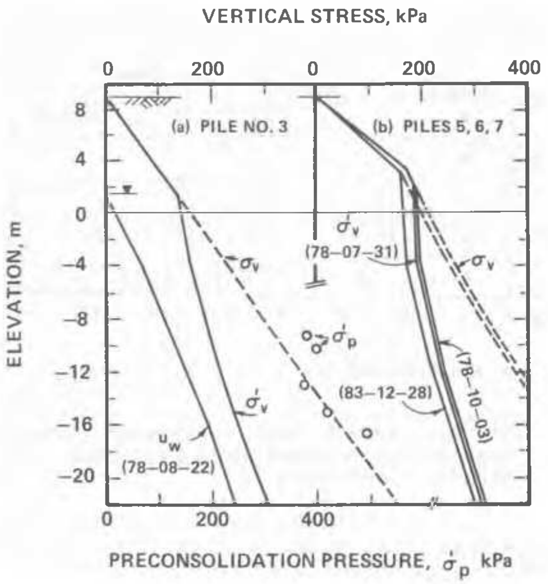

Fig. 2 Distribution of vertical stresses in soil around test piles

Layer F was dense gravel with sand and stones, with N = 64. The end-bearing bridge foundation piles were driven into this formation.

TEST PILES

All pipe piles were 32 m long, 324 mm outside diameter with 7.92 mm wall thickness and weighed 61.74 kg/m. The bottoms were closed with flat steel plates of area 0.0864 m^2. The layout of the three-pile group is shown on Figure 1. The piles formed a triangle with a centre-to-centre spacing of 1295 mm (four pile diameters). They were located 7.5 m east of the control pile, No. 3 which was identical to Nos. 5, 6 and 7.

Piles 5, 6 and 7 were installed on 18 July and No. 3 on 19 July, 1977 to their design elevation in layer E (Figure 1).

Eight telltales were equally spaced around the circumference of each pile to measure axial deformations. Because they were welded continuously to the piles, they added steel and increased the contact area with the soil. The first telltale extended to the base of the test embankment and the remainder were distributed about equally along the remaining length of the pile. (Summary given in Table I.)

TABLE I. Engineering details of instrumented steel pipe piles

Elevation (m)	Contact perimeter (m)	Total steel area (mm^2)
+9.6 to -4.6	1373	9583
-4.6 to -10.6	1284	9154
-10.6 to -16.7	1195	8724
-16.7 to -22.4	1106	8294

[1]Elevation of embankment near test piles +8.8 m.
[2]Length of telltales: 11.3, 14.2, 17.4, 20.2, 23.3, 26.3, 29.3, 32.0 m.

SETTLEMENT

Test Embankment

Most of the test embankment was constructed in the river. Settlement platforms (Series M) were installed on the original ground surface and (Series SA) to greater depths before Stage 1 was completed. The observed settlements at elevations -1.5 and -6.6 m are given for various times in Table II.

After 18 months the settlement at elevation -1.5 m was 187 mm, and 92 mm at -6.6 m. This showed that half of the settlement occurred in layer A, and half in the underlying formations. Unfortunately further observations were not possible because the gauges were damaged.

TABLE II. Settlement below embankment near test piles (mm)

Date	M1 (Elevation -1.5 m)	SA1 (Elevation -6.6 m)	Remarks
77-02-28	0	0	No fill
77-04-30	76	33	Fill at +9.0 m
77-07-18	89	40	Piles 5, 6, 7 driven
77-10-26	103	46	Pullout, Pile 3
77-12-31	143	58	Surcharge at +16.5 m
78-03-31	159	74	Surcharge at +16.5 m
78-08-15	187	92	Last survey

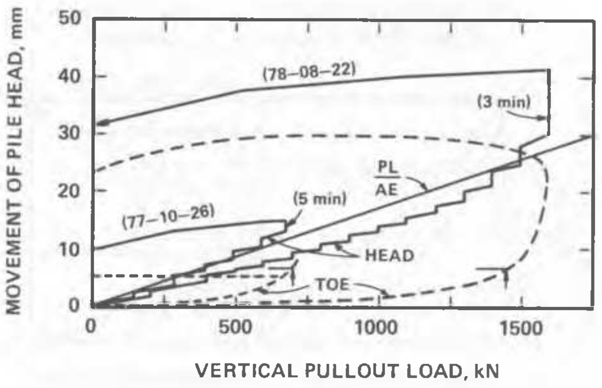

Fig. 3 Pullout tests, pile No. 3

Test Piles

Piles 5, 6 and 7 settled about 80 mm over a period of 6½ years. Settlements at various intermediate times are shown with those for pile 3 on Table III.

TABLE III. Settlement of test piles measured on pile head

Date	Elapsed time (days)	Settlement (mm)			
		No. 3	No. 5	No. 6	No. 7
77-10-14	88	12	12	12	12.5
77-10-26	100	-	13	14	13.5
77-11-28	133	*13	18	20	19
78-07-31	378	*25.5	31	32.5	31.5
78-08-15	393	-	32	33	32
78-10-03	441	*27	34.5	36	35
79-10-04	807	-	52	59	59.5
81-12-18	1614	-	-	75.5	77
82-10-18	1918	-	77	76	-
83-12-28	2354	-	79.5	76	-

*Cumulative settlement adjusted for measured displacements caused by pullout tests.

From 18 July to 26 October, 1977, the piles settled 13 mm and the test embankment 14 mm. On 15 August, 1978, piles 5, 6 and 7 had settled 32 to 33 mm, compared to the 98 mm measured at -1.5m. At -6.6 m, however, the relative movements between the pile and the soil were much less. Pile 3 settled about the same as the piles in the group.

PULLOUT TESTS

The pile was pulled with a 500 tonne hydraulic jack positioned on a calibrated load cell on top of a reaction beam (Bozozuk et al. 1979). Loads were applied in increments of 89 kPa every ten minutes until the pile failed, and then unloaded in three increments. Each load was maintained for nine minutes, allowing one minute for changing loads.

Pullout tests were performed on 26 October, 1977, and 22 August, 1978. The in situ pore water pressures during the first test were about 45 kPa more than during the second. The load deformation curves shown on Figure 3 reflected the effective stress conditions in the soil at these times. The maximum applied load during the first test was 675 kN. It was maintained for only five minutes because the pullout had reached 15 mm and the pile was literally moving out of the ground. In the second test, the maximum was 1595 kN. It was maintained for only three minutes.

Movements of the toe determined from telltale measurements are also shown on Figure 3. They plotted considerably below the elastic compression line defined by $\frac{PL}{AE}$. Upon unloading, the toe of the pile was displaced 5 mm after the first test and 23 mm after the second.

Failure Load

Davisson's (1972) failure criteria for standard pile tests consists of two parts: the elastic compression and the allowable movement of the toe (s). The movement at the toe is determined from

$$s = 3.81 + \frac{B}{120} \qquad (1)$$

where B = pile diameter, mm, giving s = 6.5 mm for the test piles.

Since the toe movements were measured, the above criterion was used to define the failure loads in Figure 3. The toe movement in the first test was only 5.5 mm, so the curve was extrapolated to 6.5 mm using the measured deformation from the previous increment as a guide. The resultant failure loads for the two tests were 700 and 1440 kN, respectively, which includes the weight of the pile (20 kN).

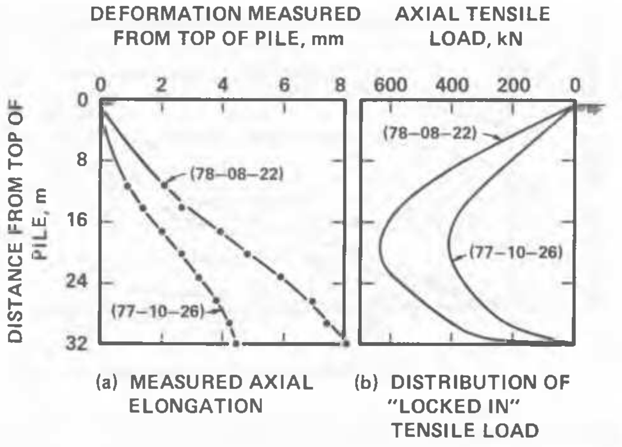

Fig. 4 Distribution of pile deformation and locked-in tensile load, pile No. 3, upon unloading after pullout to failure

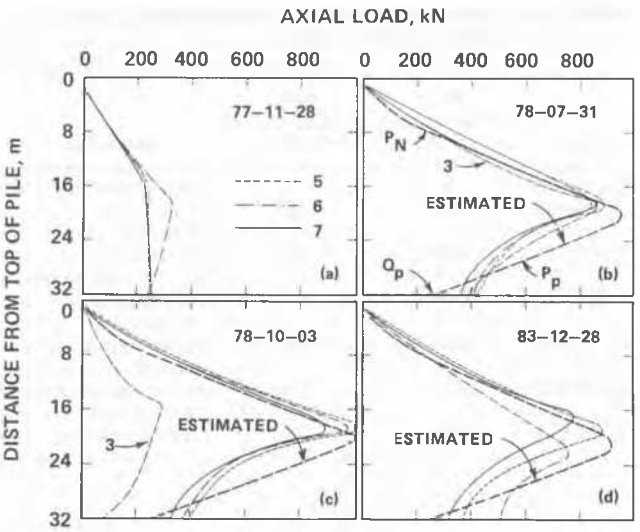

Fig. 5 Observed and estimated downdrag loads for group of three steel pipe piles

Locked-in Tension Load

Upon unloading, the test pile was stretched 4.5 mm after the first test and 8 mm after the second. The measured elongations after complete unloading, shown on Figure 4(a), were caused by soil:pile friction, which prevented the pile from regaining its original length.

The axial tensile loads determined from the deformations are shown on Figure 4(b). The maximum loads were 410 kPa for the first test and 640 kPa for the second.

PILE LOAD DISTRIBUTION

Pile compressions for the three-pile group were observed for 6½ years. Several significant changes in embankment geometry resulted from placing and moving granular fill as the south approaches to the bridge were formed, and in pore water pressures, which affected the downdrag loads. Four particular times were selected to illustrate development of negative skin friction load and response to changes in vertical effective stress in the soil.

Observed Changes in Loads

The pile compressions were converted into axial pile loads, (Figure 5) using the data in Table I.

On 28 November, 1977, 133 days after installation, the excess pore pressures were still very high, and the test piles had settled 18 to 20 mm (Table III). The distribution of axial pile load due to downdrag is shown on Figure 5(a). The loads increased linearly and rapidly to a depth of 14 m for piles 5 and 7 and 18 m for pile 6. Below these depths the load increased very slowly for piles 5 and 7 and decreased for pile 6. The shape of the curves indicates that negative skin friction loads were under development and were easily resisted by positive skin friction and end bearing.

On 31 July, 1978, pile settlements had increased to 31 mm compared to about 95 mm for the base of the fill in the same time period. Downdrag loads in piles 5, 6 and 7 had increased to 890, 840 and 860 kN, respectively, 18-19 m from the top of the piles (Figure 5b). The downdrag load of 860 kN in pile 3 was comparable, and it developed in 278 days since the first pullout test. The increases were due to the higher effective vertical soil stress around the piles (Figure 2b), due to the addition of the surcharge load to elevation +16.5 m and to lower pore water pressures in the soil. The load distribution curve for pile 3 is close to that for the three piles in the group.

On 3 October, 1978, the water level in the river was at its lowest elevation, the excess pore water pressures had almost dissipated and pile settlements had reached 35 mm. Under the increased vertical effective stresses in the soil (Figure 2b), downdrag loads reached peak values of 1000, 970 and 890 kN, respectively, in piles 5, 6 and 7 (Figure 5c). The maximum load in pile 3 was 290 kN, which was generated in the 41 days since the second pullout test.

On 28 December, 1983, 6½ years after installation, the bridge had been in operation for over two years. Pile settlements were about 80 mm and had stabilized in the last year. Vertical effective stresses were also lower (Figure 2b), due to removal of the surcharge and construction of the approach road. Long-term downdrag loads on piles 5, 6 and 7 were reduced to 890, 760 and 790 kN, respectively.

Relation Between Pullout and Downdrag Loads

A long pile preloaded axially by downdrag in a consolidating soil formation can be compared to a prestressed column. Downdrag (P_N) is in equilibrium with the negative skin friction forces dragging the pile down and resisted by positive skin friction plus end bearing for the effective stress conditions at the time. A 49-metre-long steel pipe pile in a consolidating marine clay easily supported an applied axial load equal to the maximum downdrag for a week without movement (Bozozuk, 1981). During the same experiment it was demonstrated that the axial load could be increased to about 2 P_N (corrected for end bearing) before the pile failed. The applied load could be increased to 2 P_N (assuming no end bearing) if positive skin friction was mobilized for the full length of the pile. This would occur if the relative movements between the pile and soil above the neutral point were reversed.

Pulling a pile out of the ground would mobilize skin friction resistance along its embedded depth. The pullout should therefore be related to mobilized downdrag load. Figure 5(b) shows a single pile prestressed by downdrag in equilibrium with the effective stress conditions in the soil. The total skin friction (P_s) in the pile is the sum of negative and positive friction loads:

$$P_s = P_N + P_P \tag{2}$$

Since the positive skin friction load $P_P = P_N - Q_P$ (end bearing),

$$P_s = 2\,P_N - Q_P \tag{3}$$

On 22 August, 1978, pile 3 was pulled a second time. The failure load, minus the weight of the pile, was 1420 kN. Although the effective stresses in the ground at the time were not the same as for piles 5, 6 and 7 on 31 July and 3 October 1978, they were close enough (Figure 2b) to check the hypothesis.

On 31 July, 1978 (Figure 5b), P_s for test piles 3, 5, 6 and 7 using equation (3) was, respectively, 1355, 1360, 1270 and 1360 kN (average of 1340 kN). On 3 October, 1978 (Figure 5c), for piles 5, 6 and 7, P_s was 1600, 1570 and 1470 kN, respectively (average of 1550 kN). The averaged loads compared very well with the measured 1420 kN, the difference being entirely due to different vertical effective stresses in the soil.

The relation could be improved if the end-bearing loads were better known. The measured compression at the bottom 2.7-metre length of each pile in Figures 5(b) and 5(c) is due to a combination of end bearing and positive skin friction loads.

Estimated Distribution of Skin Friction Load

If the vertical effective stresses around a pile driven into a consolidating saturated soil are known, the cumulative negative skin friction load generated in the pile down to the neutral point (D) can be determined from Bozozuk (1972):

$$P_N = \sum_{i=1}^{n_D} C_i\ \Delta L_i\ K_o\ \sigma'_{vi}\ \tan\delta'_i \tag{4}$$

where C_i = perimeter between pile and soil,
ΔL_i = incremental length of pile,
K_o = relation between horizontal and vertical effective stress in the soil,
σ'_{vi} = average vertical effective stress around ΔL_i,
δ'_i = effective friction angle between the soil and the surface of the pile ($\tan\delta'_i = M \tan\phi'$ where ϕ' = effective friction angle of the soil, and M depends upon type of soil and pile surface).

Similarly the cumulative positive skin friction load mobilized in the pile from the toe (L) up to the neutral point (D) is given by:

$$P_P = \sum_{i=n_L}^{n_{D+1}} C_i\ \Delta L_i\ K_o\ \sigma'_{vi}\ \tan\delta'_i \tag{5}$$

The exact location of the neutral point need not be known beforehand. Extending both calculated curves for P_N and P_P until they intersect automatically establishes D.

When the end-bearing load at the toe (Q_P) is substantial enough to carry a fair proportion of the downdrag load, it must be evaluated as it identifies the start of the P_P curve (Figure 5b).

Vesić (1977) suggested that Q_P be estimated from the Standard Penetration Test from $Q_P = A \cdot q_P$, where A is the end area of the pile and

$$q_P = \beta\ \bar{N}\ \text{tsf (USA)} \tag{6}$$

$\bar{N}$ = N when N < 15
and β = 2 for saturated clays
= 4 for saturated sands.

Because the test piles were in the ground for long periods of time, a value of 3 as opposed to 2 (Bozozuk et al. 1979) was chosen for the calculations. Using N = 10 for soil layer E, and A = 8.64 × 10^{-2} m^2 (0.93 ft^2) gave Q_P = 248 kN, which was 50% greater than the 165 kN (β = 2) used in the 1979 analysis.

All terms in equations (4) and (5) must be evaluated to determine the distribution of skin friction load along the pile. The increment of pile length (ΔL_i) was selected according to length of telltale, pile geometry (Table I) and the soil profile. C_i was also obtained from the table. The average vertical effective stress (σ'_{vi}) opposite ΔL_i was obtained from the appropriate effective stress curves on Figure 2. Two very important unknowns were δ'_i and K_o.

The coefficient of friction between the soil and the pile (δ') was obtained from laboratory tests in which a steel cylinder was rotated inside a soil mass under various effective contact pressures (Bozozuk et al. 1979). The test results for the fine sand fill gave $\delta' = 29.6°$, $\phi' = 33°$, M = 0.88. For the clayey silt, three tests gave δ' of 24, 25 and 27.5°, averaging 25.5°, $\phi' = 31°$ and M = 0.78.

K_o was evaluated from the pullout test performed 22 August, 1978 . Since the contact area changed with depth because of the attached telltales, a weighted average C = 1273 mm was used for the pile. Similarly, because of the different soil formations, a weighted average $\delta' = 26.8°$ was obtained for the soil. Using the effective vertical stresses plotted on Figure 2(a), and the above values of C and δ' in equation (4) gave an operational K_o of 0.478. This value was assumed to apply to both the sand fill and the underlying clayey silts.

Knowing C, δ' and K_o, the distribution of skin friction loads for the various effective stress conditions given on Figure 2(b) was calculated using equations (4) and (5). The estimated load distributions for 31 July, 1978, 3 October, 1978, and 28 December, 1983, are shown on Figures 5(b, c and d) respectively. The estimated downdrag loads (P_N) compared very well with the observed curves for most of the piles in the group. Below the neutral point, the comparison of the positive friction loads was not as good. This was attributed to poorer measurements of the end-bearing load acting on the foot of the pile. Furthermore, the estimated end-bearing load (Q_p) from $\beta = 3$ rather than from $\beta = 2$ was too high. Using $\beta = 2$ reduces Q_p to 165 kPa, which shifts the estimated positive skin friction curve about 85 kPa to the origin. The correlation between the estimated and observed distribution of P_p is considerably improved. Nevertheless the estimated load distributions were considered quite good. It appeared that there was no group effect on the development of downdrag load.

CONCLUSIONS

1. Observed downdrag loads acting on a group of three full-scale pipe piles spaced at four pile diameters showed that they performed as single piles.

2. The magnitude and distribution of downdrag loads on pipe piles in compressible clayey silts are directly related to the vertical effective stresses in the surrounding soil and to soil settlement relative to the piles.

3. The tensile load locked in a pile by soil friction following pullout to failure is directly related to the vertical effective stress and can be appreciable.

4. The pullout failure load can be defined by applying Davisson's criteria to pile toe movement measured during a load test.

5. The pullout failure load is equal to the combined negative and positive skin friction loads generated in the pile for the effective stress conditions at the time of test.

6. The pullout load test can be used to obtain an operational K_o for a soil profile under an embankment load, provided the coefficient of friction between the pile and soil is known.

7. The distribution of skin friction loads on the test piles was reasonably estimated for various conditions of vertical effective stress using effective stress equations.

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