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Bearing Capacity of Pile Preloaded by Downdrag

Capacité Portante d'un Pieu Précontraint par Frottement Latéral Négatif

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SYNOPSIS

An instrumented steel-pipe pile, 49 m long, driven through a highway embankment into a deep deposit of compressible marine clay accumulated a peak downdrag load P_N of 1.52 MN after 10 years. After the pile was filled with concrete, a comprehensive load-testing program was carried out over a period of 20 days to investigate the load-carrying capability of the pile. The locked-in downdrag load behaved as stored energy in the pile. Transient and short-term live loads up to P_N and cyclic loads to stress levels of $P_N \pm 33 \frac{1}{3}\%$ were easily supported. Plunging failure occurred at an applied load of $2 P_N$.

INTRODUCTION

A long pile preloaded axially by negative skin friction from downdrag in a consolidating clay formation can be compared to a prestressed beam or column. The locked-in negative skin friction load can be considered as stored energy that can support transient or short-term live loads (Fellenius, 1972).

Consider a long pile with a locked-in load distribution as shown in Fig. 1. The negative skin friction load generated between O and N is resisted by positive skin friction and end bearing. The shaded area under O-N is the stored energy due to prestress from downdrag. Initially, any vertical load equal to or less than P_N , applied for a short period of time, would be carried by the prestress associated with the energy stored above the neutral point N. The axial compression from this load would move the pile down and reduce or eliminate the vertical soil strains that generated the downdrag load. If, however, the load is maintained for too long a time, the continuing consolidation settlements will re-establish the downdrag load which will then be added to the applied load.

The relative movements between the pile and the soil above the neutral point will be reversed if loads greater than P_N are applied, and all loads in excess of P_N will be resisted by positive skin friction. Theoretically, the maximum load P_p that could be applied would be $2 P_N$, because the maximum positive skin friction that could be mobilized above the neutral point would be equal to P_N . For a load $P_p = 2 P_N$, however, the settlement would be too large for all practical purposes, and the pile would be close to plunging failure. If this load could be applied, it would only be for a very short period of time. One must ask, therefore, how much transient load (relative to P_N) can be supported by a long pile that is preloaded by downdrag, and for how long? These questions were studied in a comprehensive load-testing program carried out on a steel-pipe test pile located on the Autoroute

du Québec at Berthierville, east of Montreal, Canada.

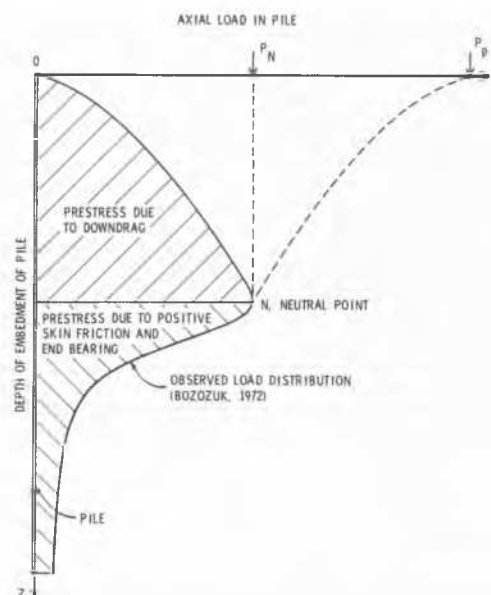


Fig. 1 Theoretical load distribution in prestressed pile under applied load

TEST INSTALLATION

The pipe pile is 49 m long, and has an outside diameter of 324 mm and a wall thickness of 6.35 mm. It was driven open ended in four sections through a 9 m high, 27.4 m wide fine sand embankment with berms into a deep deposit of highly compressible layered silty clay (Fig. 2). The pile was instrumented with eight deformation

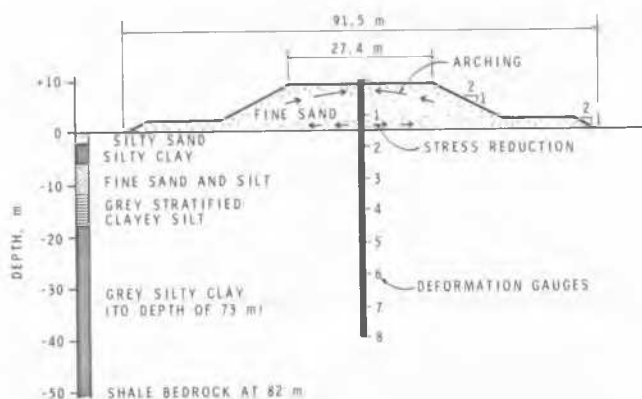


Fig. 2 Installation of steel-pipe pile, 49 m long, 32 cm diameter, through sand embankment

gauges in order to measure the axial compressions caused by downdrag loads with time, and the soils below the fill were instrumented with settlement gauges and piezometers (Bozozuk and Jarrett, 1968).

SOIL

The subsoils are of marine origin (Karrow, 1961). The soil profile (Fig. 2) consists of layers of silty clay, fine sand and silt, and stratified clayey silt, to a depth of 18 m. These layers rest upon a very thick deposit of grey silty clay, which extends to a depth of 73 m. In the upper 18 m, the cohesive soils are over-consolidated by about 30 kPa and have a shear strength of between 14 and 57 kPa as measured by a field vane. The liquid limits are equal to or less than the in situ water contents which vary from 28 to 60%. The silty clay formation below 18 m is over-consolidated by 80 kPa, and has a shear strength varying from 57 to 86 kPa and water contents varying from 35 to 60%. The groundwater table is normally less than 1.5 m below the ground surface. The engineering properties of these soils have been described in detail (Bozozuk and Labrecque, 1968; Samson and Garneau, 1973).

EMBANKMENT

Construction of the sand embankment began in October 1964. It was raised to a surcharge height of 10.7 to 11.3 m in November 1965, and finally paved in June 1967 (Samson, 1968; Samson and Garneau, 1973). When the test pile was installed in May 1966, the embankment had settled about 1.8 m. The fill settled an additional 761 mm by the end of May 1976, and continued to settle at a rate of 36.7 mm/a or 0.101 mm/d. Most of the settlement was due to consolidation of the upper 18 m of soil because the excess pore water pressures had essentially dissipated to this depth. At 27.5 m, however, the excess pore water pressure was still 75% of the applied vertical stress.

TEST PILE

The test pile settled 694 mm during this time; the rate of settlement at the end of May 1976 was 35.5 mm/a or 0.097 mm/d. Downdrag loads generated in the pile reached a peak 1.27 MN five years after installation (Bozozuk, 1972) and 1.52 MN at the end of May 1976 (Fig. 3). They occurred at the neutral point, which coincided with the base of the fine sand and silt formation.

The distribution of skin friction load is related directly to the horizontal effective stress acting on the pile. The magnitude and distribution of these stresses could be reasonably estimated at the centre line of the embankment at the end of construction, but after several years the large differential settlements developed arching stresses at the top and stress reductions at the base of the highway fill (Fig. 2). The changed stress conditions affected the distribution of downdrag load in the pile within the sand embankment (Bozozuk, 1972).

The horizontal effective stress on the pile below a depth of 27.5 m (36 m from top of pile, Fig. 3) is also difficult to determine. In addition to the known high excess pore water pressures generated from the weight of the embankment, unknown excess pore pressures are generated as the pile is pushed continuously (at a rate of 0.097 mm/d) into the very thick silty clay formation. Consequently, little or no positive skin friction was mobilized on the pile shaft below this depth. Since the horizontal effective stresses are therefore generally unknown 10 years after installation and cannot be determined, the behaviour of the pile in terms of effective stress and the coefficient of friction between the pile and soil could not be analyzed.

LOAD TEST PREPARATION AND PROGRAM

The magnitude and distribution of downdrag load in the pile determined from measured compressions 10 years after installation is shown in Fig. 3. For the maximum load $P_N = 1.52$ MN, the unit stress in the steel pile exceeded 190 MN/m^2 , which was close to the maximum allowable for the steel. Consequently the pile had to be strengthened if any load tests were to be carried out.

The open-ended hollow-pipe pile was pumped free of water on 25 August 1976 and filled with concrete having a 28-day strength of 41.4 MN/m^2 . The top of the pile was fitted with an enlarged cap to provide adequate support for a 500 t load cell, required to measure the applied test loads. In addition, the exposed shaft was instrumented with four SR-4 strain gauges in order to measure the composite deformation modulus D from the proposed load tests. The modulus was required to determine the distribution of applied loads in the pile from the compressions measured with deformation gauges.

A steel loading frame, capable of supporting 400 t of concrete blocks was erected over the pile. Each of the two frame supports consisted of three 1 m diameter vertical steel pipes resting on a 2.1 m wide, 6.1 m long foundation pad, placed directly on the paved roadway on both

sides of the pile. The centre of each pad was located 3.05 m or 9.5 pile diameters from the test pile. This distance was considered necessary to reduce or eliminate any transfer of stress from the foundation pads to the test pile. A 500 t capacity hydraulic jack, powered with nitrogen gas was positioned over the load cell on the test pile.

The test program was divided into three phases:

Schedule A: Investigation of the performance of the pre-stressed friction pile loaded to $P_N/3$, $2 P_N/3$ and P_N ($P_N = 1.52$ MN), over a period of 11 days.

Schedule B: Investigation of the performance of the pile loaded to $2 P_N/3$, $4 P_N/3$ and $2 P_N$, over a period of 7 days.

Schedule C: If the pile did not fail during loading in Schedule B -

- 1) Load pile to failure;
- 2) Subject pile to a series of cyclic loads to investigate the effect on shaft friction.

These tests were to be completed in two days.

Loading began 28 October 1976, 64 days after the concrete was placed in the pile. The deformation modulus of the concrete-filled pipe pile was obtained from the load tests and is reported along with other relevant data in Table I.

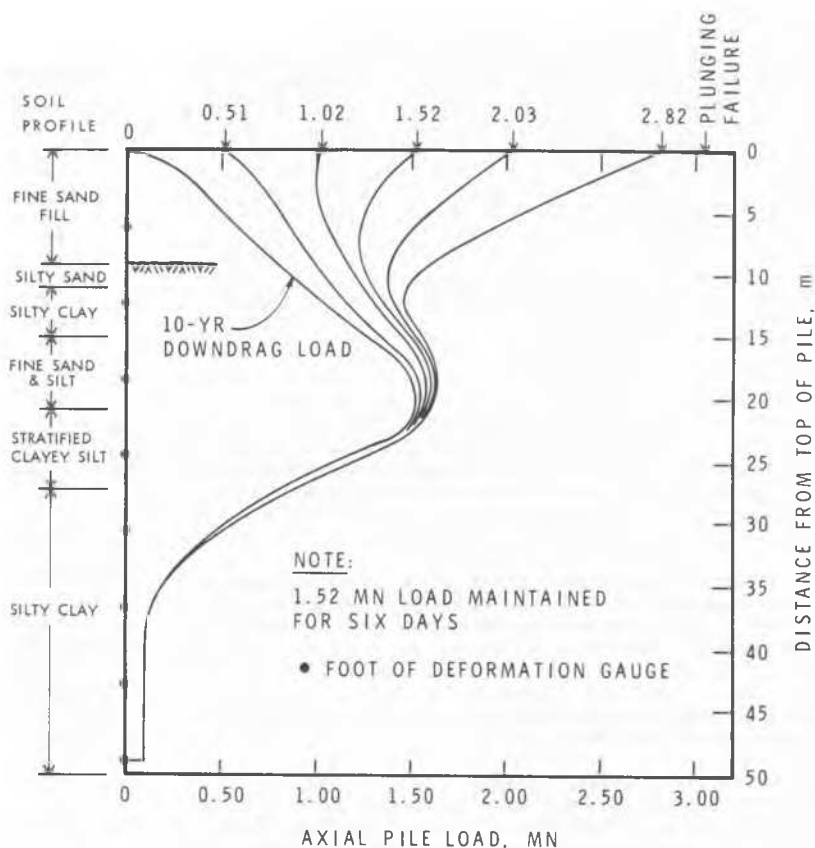


Fig. 3 Load distribution in test pile

TEST RESULTS

The loading procedure consisted of applying an initial load of 0.20 MN (first load that could be read on the load cell), followed by incre-

TABLE I
Engineering Data for Concrete-Filled Steel-Pipe Pile

Distance from top of pile, m	Perimeter in contact with soil, m	Cross-Section Area, m ²			Deformation modulus, D** MN/m ²
		Steel, A _s *	Concrete, A _c	Total, A	
0 - 12.2	1.42	8.15 x 10 ⁻³	75.67 x 10 ⁻³	83.82 x 10 ⁻³	41 200†
12.2 - 24.4	1.32	7.73 x 10 ⁻³	75.67 x 10 ⁻³	83.40 x 10 ⁻³	40 400
24.4 - 36.6	1.22	7.30 x 10 ⁻³	75.67 x 10 ⁻³	82.97 x 10 ⁻³	39 500
36.6 - 48.8	1.12	6.88 x 10 ⁻³	75.67 x 10 ⁻³	82.55 x 10 ⁻³	38 700

* Steel area adjusted for deformation gauge casings which are welded continuously to outside of pipe pile.

** Deformation modulus $D = \frac{1}{A} [A_c E_c + A_s E_s]$
 $E_s = 200\,000$ MN/m²; $E_c = 24\,100$ MN/m²

† Measured with strain gauges.

ments of 0.10 MN every 10 min until the required load was reached. At each load, the settlement of the pile head was measured with two dial gauges reacting against two steel reference beams supported on steel posts driven into the fill beyond the influence of the loading frame. An independent measure of the settlement was obtained with engineering level surveys referenced to a deep bench mark. Pile compressions registered by the deformation gauges were recorded manually and automatically with an electric recording system. A similar procedure was followed during unloading of the pile. The loading record for the complete study is shown in Fig. 4.

Schedule A

The first load equal to $P_N/3 = 0.51$ MN was applied in 30 min and maintained for 17 h. Apart from the initial elastic compression of 1.2 mm (Fig. 5), no additional settlement occurred over this period of time. The distribution curve for the applied load (Fig. 3) shows that it was carried by the prestress in the pile.

The second load equal to $2 P_N/3 = 1.02$ MN was maintained for 19 h. The initial settlement of 2.7 mm (Fig. 5) also did not change over this time period, but this may have been due to a small reduction in applied load after 19 h (Fig. 4). The load distribution curve (Fig. 3) shows that this load was also carried by the prestress in the pile.

The third load equal to $P_N = 1.52$ MN was also essentially carried by the prestress in the pile, but the slope in the upper portion of the curve indicates that some of the load may have been carried by positive skin friction (Fig. 3). A very small portion of the load was also transferred to the pile below the neutral point.

The load was maintained for about six days (Fig. 6). After the initial settlement of 5.2 mm (Fig. 5), the pile continued to settle at an average rate of 0.127 mm/d (with respect to fill) or 3.8 mm/month. Before it was loaded, however, it was settling 0.12 mm/month slower than the fill, indicating that if this load was applied for a long period of time, positive skin friction could eventually be mobilized along the whole length of the pile. This is a real possibility because the pile can move very slowly into the soil. This could not happen if the pile was bearing on rock.

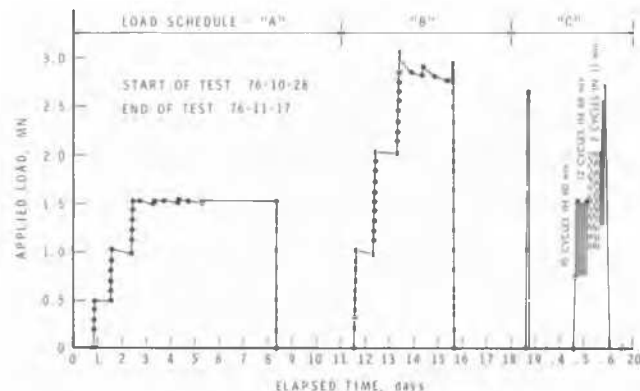


Fig. 4 Loading record for test pile

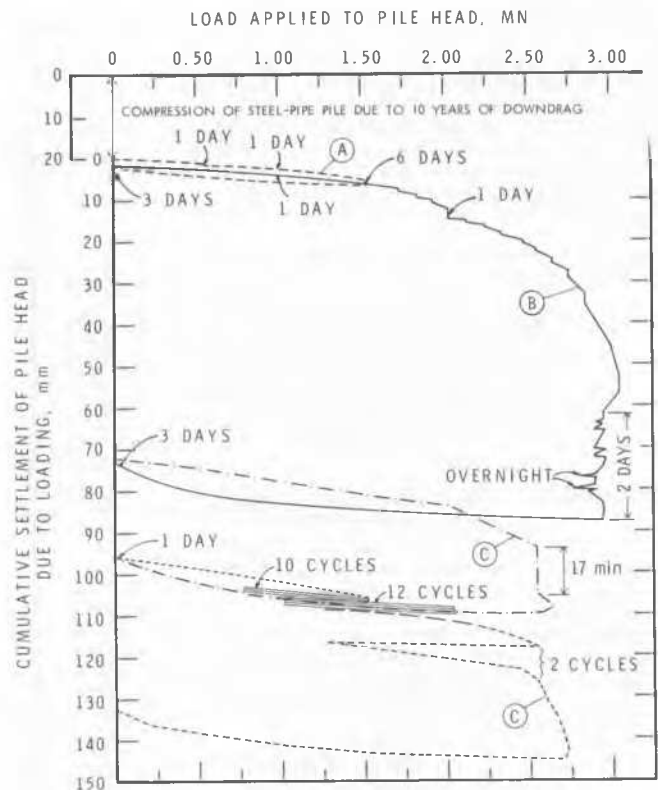


Fig. 5 Load:deformation behaviour of test pile

The pile was unloaded in 12 increments and allowed to rest for three days (Fig. 4). It rebounded 5.4 mm during this time, indicating a net settlement of 1.3 mm due to the loading program.

Schedule B

The first load equal to $2 P_N/3 = 1.02$ MN caused an initial settlement of 4.1 mm (Fig. 5). After 17 h, the load dropped off to 0.97 MN (Fig. 4) and the settlement reduced to 3.9 mm, again showing that the preloaded pile can support this load.

For the second load, $4 P_N/3 = 2.03$ MN, the initial settlement of 12.3 mm (Fig. 5) increased by 2.4 mm to 14.7 mm after 22 h. Figure 3 shows that a large portion of the load was carried by positive skin friction mobilized within the sand embankment. Some of the applied load was also transmitted to the pile below the neutral point.

The third load, $2 P_N = 3.06$ MN, was reached momentarily at a large settlement of 53 mm (Fig. 5). As this load could not be maintained, it defined the peak load at plunging failure. The load was readjusted to between 2.9 and 2.95 MN, but even this load reduced to about 2.8 MN overnight on two occasions. Attempts to maintain the 2.95 MN load resulted in an increase in settlement of 34 mm in 52 h.

Figure 3 shows the load distribution in the pile after the applied load was allowed to decrease from 3.06 MN to 2.82 MN. Positive skin friction was now mobilized down to the neutral point, and

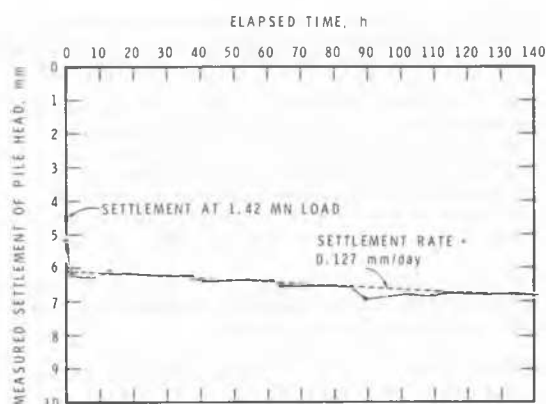


Fig. 6 Measured rate of settlement for the 1.52 MN load

although some load was transmitted to the pile below, very little additional positive skin friction and no additional end bearing were mobilized. As the pile had moved down about 80 mm, it follows that the maximum skin friction and end bearing were fully mobilized for the effective stress conditions that existed in the soil at the time. The pile was unloaded gradually in about 20 increments and allowed to rest for three days. It rebounded 15.3 mm, leaving a net total settlement of 72.3 mm.

SCHEDULE C

The pile was loaded rapidly in increments of 0.5 MN to a plunging failure load of about 2.55 MN (Fig. 5). This was only 83% of the peak load observed in Schedule B, indicating that the large pile movements had reduced the amount of positive skin friction that could be mobilized.

Three series of cyclic loads were applied to the pile after it was unloaded and allowed to rest for one day (Fig. 5). Cycling the load 10 times between 0.76 and 1.52 MN caused a residual settlement of 0.08 mm/cycle. Cycling the load 12 times between 1.02 and 2.03 MN also caused a residual settlement of 0.08 mm/cycle. The net settlement at a load of 2.03 MN after 22 load cycles was 13.8 mm, which surprisingly compared with the settlement measured on the pile for the same load in Schedule B (Fig. 5).

Two cycles of load between 1.27 and 2.55 MN caused very large pile movements. The loads were then increased in increments of 0.05 MN to a maximum of 2.72 MN, for a total settlement of 143 mm.

It appears that cycling the load about 10 times within a stress range of $\pm 33 \frac{1}{3}\%$ of P_N has very little destructive effect on the load carrying capability of the preloaded friction pile. Cycling the load did however cause a residual settlement of 0.08 mm/cycle.

DISCUSSION

The axial compression on the pile due to the development of downdrag load over a period of 10 years is shown at the top of Fig. 5. This compression represented a locked-in load of 1.52 MN, which was fixed in the steel pipe when it was filled with concrete. If the locked-in stresses or preload had been ignored, the load distribution curves (Fig. 3) would have been completely different.

Fellenius (1972) demonstrated that downdrag loads on a pile add on to existing static dead loads as consolidation of the surrounding soil takes place and if the dead load is increased the downdrag loads redevelop and add on to this new level of load. This implies that all piles must be designed to carry the design dead loads and the maximum downdrag load that can develop during the life of the structure. The test program described in the paper, however, shows that the downdrag load or locked-in preload could be available to carry short-term transient or live loads. This is very important because the trend of present-day construction is toward a decrease in the weight of the structures and a relative increase in transient loads which can amount to 60% of the total loads acting on the foundations (Bakholdin and Sturov, 1979).

The testing program demonstrated that axial loads up to the locked-in downdrag load can be applied to the friction pile for about one month if 4 mm of settlement is acceptable. Larger loads can also be carried but for considerably smaller periods of time. Positive skin friction was mobilized for the full length of the pile when the larger loads were applied. Plunging failure occurred however when the load was increased to two times the maximum downdrag load.

The pile "set up" very quickly after it was loaded to plunging failure. Following a rest period of three days, plunging failure occurred at 83% of the initial peak failure load or at 90% of the residual bearing capacity (defined at settlements exceeding 60 mm). After a further rest period of one day it was possible to apply 22 cycles of load up to a stress level of 2.03 MN ($4P_N/3$). Applying a total of 24 load cycles did not further reduce the bearing capacity of the pile.

CONCLUSIONS

1. The 1.52 MN downdrag load, which developed in the floating friction pile over a period of 10 years, can be considered as a prestress that is capable of responding to transient, cyclic and short-term live loads.
2. Applied axial loads equal to or less than P_N (1.52 MN) can be supported for one month if 4 mm of settlement is acceptable.

3. Applying cyclic loads within a stress level of $P_N \pm 33 \frac{1}{3}\%$ does not significantly affect the shaft friction mobilized on the pile. Each cycle causes a residual settlement of .08 mm/cycle.
4. Pile "set-up" is very rapid in the marine clays at the test site.
5. To support applied loads varying from P_N to $2 P_N$, positive skin friction must be mobilized in the upper consolidating soil.

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