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Publisher's version / Version de l'éditeur:

Canadian Geotechnical Journal, 9, 2, pp. 127-136, 1972-10-01

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BY

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Reprinted from
CANADIAN GEOTECHNICAL JOURNAL
Vol. 9, No. 2, May 1972
p. 127–136

RESEARCH PAPER NO. 512
OF THE
DIVISION OF BUILDING RESEARCH

OTTAWA
October 1972

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Downdrag Measurements on a 160-Ft Floating Pipe Test Pile in Marine Clay[1]

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Received September 10, 1971

Accepted December 23, 1971

Large negative skin friction loads were observed on a 160 ft (49 m) steel pipe test pile floating in marine clay. The test pile was driven, open-ended, on the centerline of a 30 ft (9 m) high granular approach fill on the Quebec Autoroute near Berthierville. Since the installation was made in 1966 the fill has settled 21 in. (53 cm), dragging the pile down with it. Negative skin friction acting along the upper surface of the pile was resisted by positive skin friction acting along the lower end as it penetrated the underlying clay. Under these conditions the pile compressed about ¾ in. (2 cm). Analysis of the axial strains indicated that a peak compressive load of 140 t developed at the inflection point between negative and positive skin friction 73 ft (22 m) below the top of the pile. Negative and positive skin friction acting on the upper surface of the pile exceeded the *in situ* shear strength and approached the drained strength of the soil where excess pore water pressures had dissipated. At the lower end where the positive excess pore pressures were high and relative movement between the pile and the soil was large, the positive skin friction approached the remoulded strength as measured with the field vane. Skin friction was increasing, however, as positive escess pore pressures dissipated.

This paper shows that skin friction loads are related to the combination of (a) *in situ* horizontal effective stresses, (b) horizontal stresses due to embankment loads, and (c) horizontal stresses due to differential settlement of the fill.

On a observé des charges négatives considérables de frottement superficielle sur un pieu tubulaire d'essai de 160 pi (49 m) flottant dans l'argile marine. On a enfoncé le pieu d'essai, avec section ouverte, dans la ligne centrale d'un remblai d'approche de 30 pi (9 m) de hauteur sur l'autoroute de Québec près de Berthierville. Depuis qu'on a effectué cette mise en place en 1966 le remblai s'est tassé de 21 po (53 cm), entraînant le pieu dans sa descente. La friction négative superficielle agissant à la partie supérieure du pieu s'est opposée à la friction positive sur le bout inférieur durant son enfoncement. Sous ces conditions le pieu s'est compressé d'à peu près ¾ po (2 cm). L'analyse de la contrainte axiale a indiqué qu'une charge ultime de compression de 140 t s'est développée au point d'inflection entre le frottement superficiel négatif et positif à 73 pi (22 m) plus bas que la partie supérieure du pieu. Le frottement superficiel positif et négatif agissant sur la surface supérieure du pieu excéda la résistance au cisaillement en place et approcha la résistance drainée du sol où les excès sur la pression de l'eau interstitielle s'étaient dissipés. A la partie inférieure où l'excès sur la pression de l'eau interstitielle positif était élevées et le mouvement relatif entre le pieu et le sol était important, le frottement superficiel positif s'approchait à la résistance remaniée telle que mesurée avec le scissomètre sur place. Le frottement superficiel augmentait, cependant, à mesure que les excès sur la pression de l'eau interstitielle positif se dissipaient.

La présente communication montre le rapport des charges de frottement superficiel avec la combinaison (a) des efforts horizontaux effectifs en place, (b) des efforts horizontaux dus aux charges du remblai et enfin (c) des efforts horizontaux dus au tassement différentiel du remblai.

[1]Paper presented at the Canadian Soils Conference, Halifax, Nova Scotia, September, 1971.

The limited access, high speed Quebec Autoroute traverses deep deposits of compressible marine clay along the north shore of the St. Lawrence River between Montreal and Quebec City. At Berthierville, where bedrock is at a depth of 270 ft (82 m), a railway overhead and a highway overpass are separated by a 30-ft (9 m) high granular embankment with stabilizing berms. The estimated settlement under this fill is about 10 ft (3 m). The highway overpass is supported on end-bearing piles which are subjected to a substantial load from negative skin friction. Field measurements on the magnitude and distribution of negative skin friction load on two of the centerline piles have been reported elsewhere (Bozozuk and Labrecque 1968; Bozozuk 1970).

In addition, a 12 in. (30 cm) diameter hollow steel pipe pile 160 ft (49 m) long was instrumented and driven vertically on the centerline of the fill into the underlying clay. The upper part was subjected to negative skin friction as the soil settled and the lower part to positive skin friction as the pile penetrated the underlying clay.

The magnitude and distribution of skin friction loads generated in the floating pile over a period of 5 years are reported and compared with predicted values based on *in situ* horizontal effective stresses and the influence of embankment loading. The unit skin friction exerted along the surface of the pile was determined from the load distribution curve and compared with the strength of the surrounding soil.

Description of Test Site

Soils

The subsoils are marine deposits of the ancient Champlain Sea (Karrow 1961). The profile shown in Fig. 1 is made up of a shallow organic topsoil (A), followed by layers of silty sand (B), silty clay (C), fine sand and stratified silt (D), and silt and stratified clayey silt (E) to a depth of 59 ft (18 m). These layers rest upon a thick silty clay layer (F), which extends to 240 ft (73 m) and is underlain by a further 30 ft (9 m) of fine sand overlying bedrock at 270 ft (82 m).

In the upper 59 ft (18 m) the liquid limits of the cohesive soils are equal to or less than the *in situ* water contents, which vary from 28 to 60%. The soils are overconsolidated by about 0.3 tsf (kg/cm^2) and have a shear strength between 300 and 1200 psf (0.15 to 0.59 kg/cm^2) as measured by a field vane. In layer F the water contents vary from 35 to 60%; the soil is overconsolidated by 0.8 tsf (kg/cm^2); and shear strength varies from 1200 to more than 1800 psf (0.59 to 0.88 kg/cm^2) at a depth of 110 ft (34 m). The engineering properties of these soils have been described in detail (Bozozuk and Labrecque 1968).

Embankment

The interconnecting fill between the railway overhead and the highway overpass is 30 ft (9 m) high, 90 ft (27 m) wide, about 400 ft (122 m) long with side slopes of 2:1, and is stabilized with 7ft (2 m) high berms 300 ft (92 m) wide from toe to toe. The fill was constructed in two stages. The berms and 15 ft (5.6 m) of embankment were first left in place for about 6 months, during which time they settled about 3 ft (1 m) (Bozozuk and Labrecque 1968, see Fig. 5). In the second stage the embankment was constructed to a height about 4 ft above finished grade. From October 1964 to May 1966, when the steel pipe pile was installed, the total settlement amounted to about $5\frac{1}{2}$ ft (1.7 m).

Vertical and horizontal stresses exerted on the foundation clay under the centerline of the embankment were determined, assuming plane strain conditions and using the linear elastic theory proposed by Baladi (1967) and Perloff *et al.* (1967). *In situ* stresses, load increments due to the fill and final vertical and horizontal effective stresses used in the analysis are shown in Fig. 1.

Instrumentation

Test Pile

The test pile was a 12 in. (30 cm) diameter ($\frac{1}{4}$ in. (6.4 mm) thick wall) circular steel pipe driven vertically on the centerline of the granular embankment to a depth of 160 ft (49 m) below the top of the fill. It was assembled in five sections, with individual sections butt welded together. Two transits set 90° apart were used to align the pile vertically prior to welding and driving. After

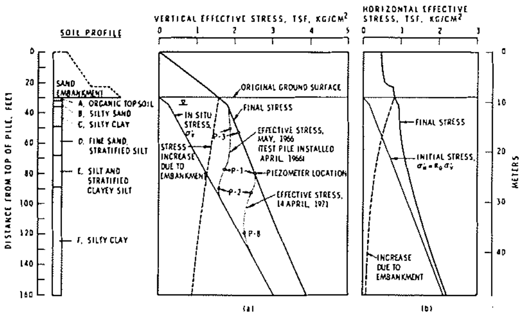

FIG. 1. Distribution of vertical and horizontal effective stresses under centerline of embankment.

TABLE 1. Physical dimensions of steel pipe test pile* with the deformation gauges attached by continuous welding on the oustide

Location along pile, feet	Total cross-sectional area of steel square inches	Perimeter of section in contact with soil feet
0– 40	12.64	4.66
40– 80	11.98	4.33
80–120	11.32	4.00
120–160	10.66	3.66

*Test pile, 12-in. (30 cm) diameter and 160 ft (49 m) long, installed to depth of 130 ft (40 m) below the original ground surface.

each section had been driven, the pile was cleaned for its full length, resulting in a vertical, clean open-ended pile with its lower end 130 ft (40 m) below the original surface of the ground.

Eight deformation gauges (Bozozuk and Jarrett 1967) were spaced at 45° intervals around the outside of the pile, their lengths selected to provide measurements of axial deformation over 20 ft (6 m) intervals. Pairs of gauges were mounted in casings of equal length positioned diagonally opposite each other to maintain axial symmetry. In one casing the reference inner rod extended for the full length of the casing; in the other, the rod terminated at an intermediate plug located 20 ft (6 m) from the bottom end.

The gauge casings, of $\frac{3}{4}$ in. (1.9 cm) diameter iron pipe, were mounted on each section of the pile before it was driven. They were carefully aligned along the long axis, then welded to the pile for the full length of the gauge. During driving the casings were joined with machined sleeves and welded in place. After the pile had been driven the casings were filled with light oil and inner rods of $\frac{1}{4}$ in. (0.6 cm) diameter iron pipe were installed. Special brass fittings mounted on the top of each deformation gauge permitted measurement of relative movements to $\pm$ 0.0001 in. ($\pm$ 0.0025 mm), using a micrometer and a dial gauge.

As the casings were welded continuously to the outside of the test pile, they became an integral part of it. Their presence increased the contact area with the soil and increased the cross-sectional area of steel, which varied with depth as shown in Table 1.

Instrumentation of Soil

Observations on the behavior of the foundation soil under the embankment were made with settlement gauges and Geonor piezometers installed at the locations described by Bozozuk and Jarrett (1967). Four piezometers provided measurements of excess pore water pressures from 20 to 87 ft (6 to 27 m) below the original ground surface. Six settlement gauges and a deep benchmark provided measurements of vertical settlements from the ground surface to a depth of 145 ft (44 m) adjacent to the test installation.

Estimate of Skin Friction

Studies by Johannessen and Bjerrum (1965) suggest that negative skin friction is related to vertical effective stress through the following equation: $\tau_a = \sigma'_v K \tan \Phi'_a$ where σ'_v is the vertical effective stress and $K \tan \Phi'_a$ is the adhesion factor relating σ'_v to skin friction. This relation holds if the surcharge causing consolidation is uniform and of infinite areal extent, in which case the final vertical effective stress varies linearly with depth. For a highway embankment or earth dam supported on compressible clay, the vertical stress increases linearly with depth to the base of the fill, then decreases nonlinearly with depth from the base of the fill. For the latter case the observed negative skin friction loads in piles do not correlate well with the vertical effective stress.

Skin friction can be related to the horizontal effective stress acting on the pile. For a surcharge that is uniform and has an infinite area, the horizontal stress would vary linearly with depth and would be related directly to the vertical effective stress. For the highway embankment the total horizontal stresses would consist of (a) the *in situ* horizontal stress, (b) the horizontal stress due to the embankment loading, (c) the horizontal stresses generated at the top of the fill owing to differential settlement as shown schematically in Figure 2, (d) the horizontal forces due to displacement of soil caused by driving displacement type piles. The latter forces can be neglected for non-displacement type piles.

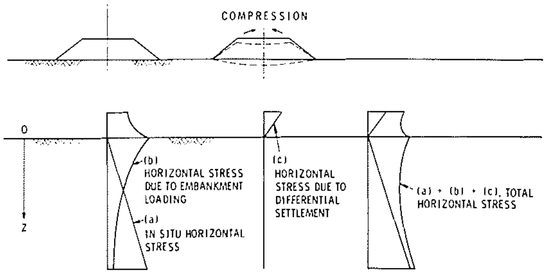

FIG. 2. Generation of horizontal stresses under centerline of embankment on compressible clay.

Skin Friction Loads Due to In Situ Horizontal Effective Stress

Consider a non-displacement type circular pile driven a depth, L, into a uniform isotropic clay deposit with the groundwater table at the surface. The soil is subjected to a small, uniform vertical pressure, q, of infinite areal extent that causes consolidation of the underlying clay. If the pile is floating, the consolidating clay imposes negative skin friction loads on the upper part of the pile that are resisted by positive skin friction at the lower end. Assuming that the pile is incompressible in relation to the soil and that the skin friction is related to the horizontal effective earth pressure acting on the pile, then the unit skin friction is given by

[1a] $$F = M.\ K_0\ \sigma'_v\ \tan\Phi'$$

where

Φ' = effective friction angle of the soil
σ'_v = vertical effective stress
K_0 = coefficient relating horizontal to vertical effective stress
M = friction factor for the soil acting on the pile surface, where $0 < M \leq 1$

Except for the friction factor, M, this equation, is similar to that proposed by Johannessen and Bjerrum (1965).

At the end of consolidation the unit skin friction can be obtained from

[1b] $$F = M.\ K_0\ \gamma'_s\ Z\ \tan\Phi'$$

where

Z = depth below the ground surface
γ'_s = submerged unit weight of the soil

For a pile of circumference, C, the increment of load due to skin friction will be given by

[2] $$dp = MK_0\ \gamma'_s\ Z\ \tan\Phi'\ Cdz$$

The total negative skin friction load to depth, D, will therefore be

[3a] $$P_{neg} = \int_0^D M.K_0\ \gamma'_s\ \tan\Phi'\ C\ Zdz$$

which can be simplified to

[3b] $$P_{neg} = \beta_1 C \int_0^D Zdz$$

Integrating

[3c] $$P_{neg} = \beta_1 C\ D^2/2$$

where

[3d] $$\beta_1 = M\ K_0\ \gamma'_s \tan\Phi'$$

The above load will be resisted by positive friction generated in the pile from D to L. Hence

[4a] $$P_{pos} = \beta_2 C \int_D^L Zdz$$

or

[4b] $$P_{pos} = \beta_2 C\ (L^2 - D^2)/2$$

Assuming that the pile carries no load in end bearing, then for equilibrium conditions

$$P_{neg} = P_{pos}$$

Hence

[5a] $$\beta_1 C\ D^2/2 = \beta_2 C\ (L^2 - D^2)/2$$

If $\beta_1 = \beta_2$

then

[5b] $$D = L/\sqrt{2}$$

which is the location of the neutral point between negative and positive skin friction loads for the above soil conditions.

The effect of varying the soil constants on the location of the neutral point can be investigated by solving equation [5a] for various ratios of β_{neg} to β_{pos}. For $\beta_{neg} = 3\beta_{pos}$, the neutral point occurs at $L/2$, for $\beta_{neg} = (\frac{1}{3})\beta_{pos}$ at $\sqrt{3}L/2$. Fig. 3 gives load distribution curves for these examples and the location of the neutral point for various ratios of β_{neg} to β_{pos}. If, for example, a dense sand layer exists near the surface of the ground so that β_{neg} is large, the neutral point will develop close to the ground surface. If, however, the lower end of the pile penetrates a dense sand layer in which β_{pos} is large, then the neutral point will shift to the bottom of the pile.

To estimate the skin friction loads generated in the test pile, values for Φ', K_0, and M must be known. Only a limited number of soil tests were available giving the effective friction angle Φ' of the soil. These are summarized in

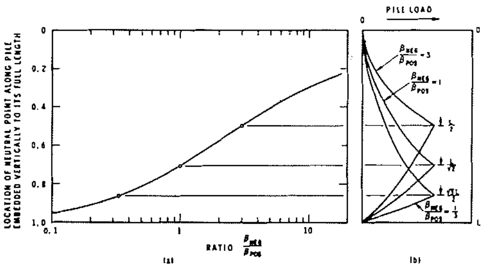

FIG. 3. Ratio: β_{neg}/β_{pos} versus the location of neutral point along floating pile subject to negative and positive skin friction loads due only to *in situ* stresses.

Table 2. K_0 was not measured. In most normally consolidated clays it can be estimated from K_0 = 1-sin Φ' (Jaky 1948). For overconsolidated clays, however, this relation gives values that are too low. A value of $K_0 = 0.7$ was assumed for the complete soil profile. This value agrees with field measurements made in overconsolidated marine clay by Eden and Bozozuk (1969).

Potyondy (1961) found a relation for skin friction between various soils and construction materials, here defined as M. His coefficients varied from 0.6 for clay on steel to 0.7 for silt on steel to over 0.8 for saturated sand on steel. Because of the nature of the soils at the site a value of $M = 0.7$ was selected for the soil profile.

Using the dimensions in Table 1 and the above coefficients the load distribution was determined and plotted in Fig. 4(a). The maximum load was 70 t, occurring at a depth of about 86 ft (26 m) below the original ground surface.

Skin Friction Loads Due to Horizontal Stresses Generated by the Embankment

Following the outlined procedure the increment of skin friction load under the centerline of the fill resulting from horizontal stresses will be given by:

[6a] $$\Delta P = M \tan \Phi'. \ C. \ \Delta L. \ \Delta\sigma_X$$

where Φ', M, C are as defined previously

ΔL = increment of length of pile under consideration

$\Delta\sigma_X$ = average horizontal effective stress due to the embankment load acting on ΔL.

TABLE 2. Friction angles applied to the various soil formations around the steel pipe test pile

	CIU test results			
Depth, feet	Φ', degrees	Source of information	Soil formation	Φ' used in study degrees
			Sand fill	45 (assumed)
8–11	27	NRC, 154–24–1 to 8		
10–11	31	Consultant's report*		
11–12	27.5	Consultant's report*	B, C, D, E	28.25
50–53	28	NRC, 154–28–3 to 9		
70–73	22	NRC, 154–30–2 to 8		
			F	23.5
90–93	25	NRC, 154–32–3 to 8		

*Terratech, private communication.

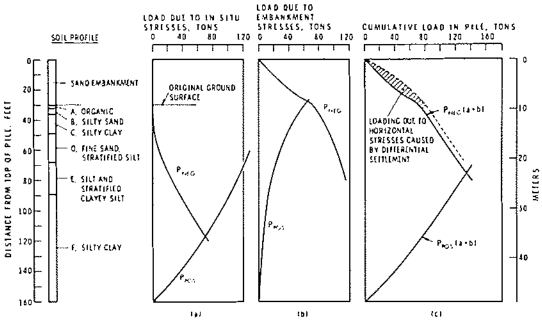

FIG. 4. Determination of skin friction load in 12-in. (30 cm) diameter steel pipe pile floating in marine clay.

The cumulative negative skin friction load to the neutral point, a distance D from the top of the pile, will be:

$$[7] \qquad P_{neg} = \Gamma C \sum_{i=1}^{n_D} \Delta L_i \, \Delta \sigma_{X_i}$$

where

$$\Gamma = M \tan \Phi'$$

$$\sum_{i=1}^{n_D} \Delta L_i = D$$

Similarly, the cumulative positive skin friction load generated from L to D:

$$[8] \qquad P_{pos} = \Gamma C \sum_{i=n_L}^{n_{D+1}} \Delta L_i \, \Delta \sigma_{X_i}$$

where

$$L - D = \sum_{i=n_{D+1}}^{n_L} \Delta L_i$$

Assuming $\Phi' = 45°$ and $M = 0.85$ for the sand fill and using the horizontal stresses due to the embankment shown in Fig. 1(b), the negative and positive skin friction loads were determined and plotted in Fig. 4(b). By projecting the positive friction load curve from the bottom of the pile to intersect the negative friction load curve from the top, a peak load of 65 t is obtained about 28 ft (8 m) from the top of the pile.

To obtain the total load generated in the floating test pile, the load distributions in Fig. 4(a) and 4(b) were added to give the cumulative load distribution shown in Fig. 4(c). This gives a peak load of 135 t, occurring 75 ft (23 m) from the top of the pile. The load distribution curve in Fig. 4(c) can also be determined directly from the total horizontal effective stresses acting on the pile shown in Fig. 1(b) and applying equations [7] and [8].

Skin Friction Loads Due to Horizontal Stresses Caused by Differential Settlement of the Embankment

An embankment constructed upon a compressible clay subsoil will settle differentially, with the minimum occurring at the toe and increasing to a maximum under the centerline. "Bending" will induce compressive stresses near the top of the fill and this will be superimposed on the existing horizontal stresses, as illustrated in Fig. 2. The additional horizontal stresses would increase the negative skin friction loads at the top of the pile, as suggested by the shaded area in Fig. 4(c).

A field study is currently under way to measure horizontal stresses. A 14 ft (4 m) high granular highway embankment constructed on marine clay has been instrumented with earth pressure cells distributed vertically down the centerline and along the base of the fill. Preliminary observations indicate that the horizontal stresses are increasing at the top and decreasing at the bottom as the fill settles. Maximum horizontal pressures developed, however, at one third of the distance from the top of the fill.

Field Measurements

Settlements and Pore Water Pressures

Vertical settlements under the centerline of the fill measured over the five years since the test pile was installed in April 1966 are summarized in Fig. 5. During this time the ground surface settled 1.77 ft (54 cm) or about 25% of the total since the fill was constructed. Most of this was due to consolidation of the upper 60 ft (18 m) of the soil profile, although the amount was not measured because the settlement gauge had been damaged shortly after it was installed. The settlement was 0.15 ft (4.6 cm) at 100 ft (30 m), reducing to 0.06 ft (1.8 cm) at 145 ft

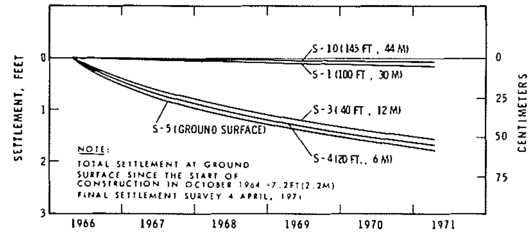

FIG. 5. Measured settlement under centerline of embankment since installation of steel pipe test pile.

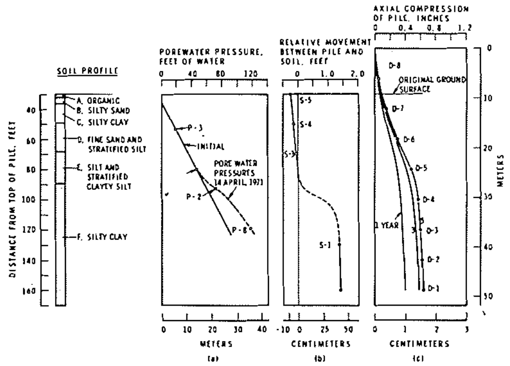

FIG. 6. Pile compressions, relative soil movements and pore water pressures at steel pipe test pile after 5 years.

(44 m), which was relatively small compared with that of the ground surface.

As the piezometers were installed about a year after the fill was completed, a considerable amount of the excess pore pressure had dissipated. The effective stresses at that time are shown in Fig. 1(a). After 5 years (April 1971) the excess pore pressures had completely dissipated to a depth of 50 ft (15 m), but they were still quite large at 90 ft (27 m), as shown in Fig. 6(a). The change in the vertical effective stress over this period is shown in Fig. 1(a).

Test Pile

Measurements on the top of the test pile show that it had settled 1.54 ft (47 cm) compared with the 1.77 ft (54 cm) the original ground surface had settled over the same period. The relative movements of the pile compared with those of the surrounding soil are plotted in Fig. 6(b). In the upper part of the profile where the excess pore pressures had dissipated relative movements were quite small, reducing to zero at a depth of about 50 ft (15 m) below the ground surface. Below 70 ft (21 m), where the excess pore pressures were high, relative movements were very large, increasing to 1.4 ft (43 cm) at the bottom of the pile.

Compressions measured with the deformation gauges showed that the pile had compressed axially 0.64 in. (1.6 cm) in 5 years, distributed as indicated in Fig. 6(c). For the most part it occurred within the embankment and the top 50 ft (15 m) of the soil profile, indicating a rapid build-up of load in this part of the pile. Compressions were small below this depth, especially in the silty clay (F) layer where the excess pore water pressures and the relative movements were large.

Load Distribution in Test Pile

Distribution of axial load at various times over the 5 year period is plotted in Fig. 7, using measured compressions and assuming that the modulus of elasticity of the steel was 30×10^6 p.s.i. (2.11×10^2 kg/cm^2). The pile started to accumulate skin friction loads almost immediately. After about 1 month, the peak load was 38 t, 68 t after $\frac{1}{2}$ year, 86 t after 1 year, and 140 t after 5 years. A line drawn through the maximum load points shows that the neutral point shifted downward. If skin friction is related to effective stress, this can be expected to occur because the excess pore water pressures dissipate to greater depths with time. After 5 years the peak load of 140 t occurs 73 ft (22 m) below the top of the pile where excess pore water pressures have completely dissipated. It also occurs at a point where there is little or no relative movement between the pile and the surrounding soil.

At the top of the pile, but within the embankment, the early development of the knee in the curves indicates a rapid build-up of load due to negative skin friction. This is possible if skin friction is related to the horizontal stresses within the fill and to the horizontal stresses generated by differential settlement, but not if related only to the vertical effective stress.

Negative skin friction loads generated in the upper part of the pile are dissipated through positive skin friction below the neutral points delineated by the line drawn through the maximum load points. The shape of the curves below the neutral points indicates a rapid transfer of load to the soil to a depth of about 70 ft (21 m) below the ground surface. In this zone relative movements between the pile and the soil are small and most of the excess pore water pressures have dissipated. Below this depth, where the rate of load transfer is small, the excess pore water pressures are high and the relative movement between the pile and the soil exceeds 1 ft (30 cm).

At the bottom of the pile, an end-bearing load of about 11 t is indicated. It compares well with the theoretical value of 10.7 t predicted by Meyerhof (1953):

$$Q = (S_u N_c + \gamma D) A$$

where

N_c = 9, bearing capacity factor for deep foundations

S_u is the undrained strength of the soil

γD is the overburden pressure at depth D

A is the end area of the pile.

This is unexpected since the bottom end is open and should allow the underlying soil to enter the pile.

Fig. 7 compares the predicted with the observed distribution of load in the test pile. The good agreement between the predicted load of 135 t occurring 75 ft (23 m) from the top of the pile with the measured 140 t at 73 ft (22 m) is most encouraging, considering the simplifying assumptions for the coefficients M, K_0, Φ' used in the analysis. Within the sand embankment the predicted loading curve neglects the effects of skin friction due to differential settlement of the fill. Consequently, the shape of this part of the curve should differ from that of the observed curve. Below the fill, predicted loading agrees well with that observed at 5 years; the curves are parallel for the most part, up to the neutral point. Below the neutral point there is a marked difference in shape of the curves. Because this is the region of high excess pore water pressures and of high relative movements, the soil may be in a partially remoulded condition, especially for the bottom 50 ft (15 m) of the pile. If this is the case, the values assumed for the coefficients M and K_0 are too great. From 75 to 100 ft (23 to 30 m) from the top of the pile the loads were dissipated in positive skin friction at a greater rate than was predicted, indicating that the values assumed for M and Φ' were too small. It must be remembered, however, that the predictions are based on the condition that consolidation has been completed. As this condition is approached, the family of positive skin friction curves will fan out and approach the predicted curve.

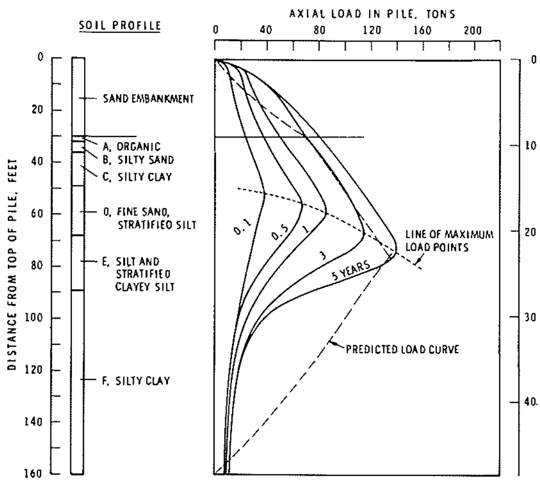

FIG. 7. Variation of skin friction load in 12-in. (30-cm) diameter steel pipe pile with time.

Comparison of Skin Friction with Soil Strength

Using the 5 year load distribution curve of Fig. 7, the average skin friction on 10 ft (3 m) increments of length of pile was calculated and compared with the shear strength of the surrounding soil. The undisturbed and remoulded strengths of the soil were measured with a field vane and the drained strengths were calculated from the relation.

$$S'_u = \sigma'_v \tan \Phi'.$$

Results are given in Table 3.

Within the sand embankment very large unit friction is acting on the pile. It varies from over 1700 p.s.f. (0.83 kg/cm²) at the top of the fill to over 700 p.s.f. (0.34 kg/cm²) at the base. From the ground surface to 20 ft (6 m) below the fill the negative skin friction is almost twice the undisturbed strength but only $\frac{1}{2}$ to $\frac{1}{3}$ the drained strength. From 20 to 40 ft (6 to 12 m) it is approximately $\frac{1}{2}$ the undisturbed strength.

From 44 to 70 ft (13 to 21 m) below the fill the unit positive skin friction exceeds the

TABLE 3. Average skin friction determined from pile loads after 5 years compared with the strength of the surrounding soil.

Location along pile feet	Depth below ground surface feet	Average skin friction pounds per square foot		*In situ* field vane strength pounds per square foot		Drained strength $\sigma'_v \tan\Phi'$ pounds per square foot
		Negative	Positive	Undisturbed	Remoulded	
0–10		1720				
10–20		990				
20–30		770				
30–40	0–10	730		330	30	1870
40–50	10–20	690		400	30	2130
50–60	20–30	650		900	30	
60–70	30–40	550		1100	40	
70–74	40–44	120		1100	30	
74–80	44–50		1150	1100	30	
80–90	50–60		2650	1200	40	2700
90–100	60–70		1700	1300	50	
100–110	70–80		650	1300	50	2310
110–120	80–90		300	1600	70	
120–140	90–110		140	1700	130	3030
140–160	110–130		80	1840	60	

undisturbed strength of the soil. The maximum skin friction of 2650 p.s.f. (1.29 kg/cm^2) from 50 to 60 ft (15 to 18 m) is double the undrained strength and almost equals the drained strength of 2700 p.s.f. (1.32 kg/cm^2). Below 70 ft (21 m) the skin friction decreases rapidly to the remoulded strength of the soil.

This analysis indicates that there is little or no relation between the skin friction exerted on the pile and the *in situ* shear strength of the soil. In the upper part of the soil profile, where the relative movements between pile and soil are small and all excess pore pressures have dissipated, the unit skin friction approaches but does not exceed the drained strength. Towards the bottom end of the pile, where the excess pore pressures and relative movements between pile and soil are large, the unit skin friction decreases to but does not fall below the remoulded strength of the surrounding soil.

Conclusions

1. Skin friction loads exerted on a floating pipe test pile in marine clay are related to the effective horizontal pressures acting on its surface.
2. Predicted skin friction loads compare favorably with observed loads, with respect to both location of the neutral point and magnitude of load.
3. Horizontal stresses generated in an embankment as a result of differential settlement require further study.
4. Unit skin friction exerted on the test pile is not related to the *in situ* shear strength of the soil. Where relative movements between pile and soil are small and excess pore pressures have dissipated, skin friction approaches but does not exceed the drained strength. Where relative movements and excess pore pressures are large, skin friction decreases to the remoulded strength.

Acknowledgments

Thanks are due to the Quebec Autoroute Authority, Terratech Ltd., and Labrecque, Vezina, and Associates for their help and cooperation during this study. Special thanks are also due to colleagues in the Geotechnical Section, and to Mr. C. B. Crawford, Assistant Director of the Division, under whose direction the early part of this work was carried out.

This paper is a contribution from the Division of Building Research, National Research Council of Canada, and is published with the approval of the Director of the Division.

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