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## Testing of drivability of concrete piles and disturbance to sensitive clay<sup>1</sup>

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The results reported are of an investigation of a group of thirteen 12 in. (30 cm) diameter precast concrete piles driven through 60 ft (18 m) of sensitive marine clay followed by 10 ft (3 m) of silt and sand and 13 ft (4 m) of very dense silt to end bearing in glacial till. The purpose of the test is to study the drivability of the piles through very dense soil and to measure the disturbance caused to the sensitive clay by the driving of displacement piles. Following a literature review, the paper presents the soil conditions at the site and the testing program. The test results are discussed and experience gained from the follow-up of the driving of 520 piles at the site is presented.

Visual observations during pile driving, and analysis of driving records, show that a high pile quality is necessary at the site. The large driving resistance encountered in the very dense silt (150 to 300 blows/ft) would prevent low quality piles from reaching the competent glacial till and developing the needed bearing capacity.

Pile loading tests showed the piles to have an ultimate bearing capacity exceeding 450 tons (4.0 MN). It was established that the shaft resistance in the clay during test loading was between 100 and 125% of the undrained shear strength of the clay as measured by field vane testing. In comparison, an uplift test to failure showed that the uplift shaft resistance along the pile in the clay was only 60% of the undrained shear strength of the clay.

The pile driving developed large pore pressures in the clay which exceeded the effective overburden stresses. The excess pore pressures dissipated over a period of slightly more than 3 months. Vane testing within the pile group immediately after driving showed that a shear strength reduction of about 15% was caused by the piles. At a distance of 2 ft (0.6 m) outside the pile group, no strength reduction was found. The reduction within the group was gradually regained during the dissipation of the induced pore pressures. Laboratory testing on clay samples obtained within the pile group 75 days after pile driving showed a smaller value of the preconsolidation pressure of the clay, but no change in the compression indices.

On présente les résultats d'une étude sur un groupe de treize pieux de béton préfabriqué d'un diamètre de 12 po (30 cm) battus à travers 60 pieds (18 m) d'argile marine sensible, suivis de 10 pieds (3 m) de silt et sable et 13 pieds (4 m) de silt très dense, et s'appuyant en pointe dans le till glaciaire sous-jacent. L'étude a pour but l'examen du comportement au battage des pieux à travers un sol très dense et la mesure du remaniement de l'argile sensible causé par le battage de pieux à déplacement. Suite à une revue de la littérature, on décrit le site des expériences, le programme d'essais, les résultats obtenus, et on relate l'expérience acquise de la surveillance du battage de 520 pieux sur le même chantier.

L'inspection du battage des pieux et l'analyse des données de battage démontrent que des pieux de haute qualité sont nécessaires sur ce chantier. La résistance élevée au battage dans le silt très dense (150 à 300 coups/pied) ne permettrait pas à des pieux de moindre qualité d'atteindre le till glaciaire dans lequel on peut mobiliser une résistance en pointe adéquate.

Des essais de charge ont démontré que la capacité portante limite des pieux est supérieure à 450 tonnes (4.0 MN). Au cours de ces essais, on a établi que le frottement latéral mobilisé le long du pieu était entre 100 et 125% de la résistance au cisaillement non-drainée de l'argile mesurée au scissomètre. D'autre part, le frottement latéral mesuré dans des essais à l'arrachement correspond à 60% seulement de la résistance non-drainée de l'argile.

D'importantes pressions interstitielles de battage se sont développées dans l'argile, atteignant des valeurs supérieures à la pression effective du mort-terrain. Cette surpression interstitielle s'est dissipée au cours d'une période d'un peu plus de trois mois. Des essais au scissomètre effectués à l'intérieur du groupe de pieux immédiatement après battage ont démontré que la

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résistance non-drainée de l'argile avait subi une réduction de 15%. A une distance de 2 pieds (0.6 m) du groupe de pieux, aucune diminution de la résistance de l'argile n'est apparente. Cette perte de résistance de l'argile a été recouvrée durant la dissipation des pressions interstitielles de battage. Des essais de laboratoire sur échantillons d'argile prélevés 75 jours après le battage des pieux démontrent que l'argile possède une valeur plus faible de la pression de préconsolidation mais aucun changement des indices de compressibilité.

[Traduit par la revue]

### Introduction

Many of the rapidly developing areas on the south shore of the St. Lawrence River near Montreal are located over deposits of sensitive marine clay. Such soil conditions present intricate geotechnical problems, particularly for the foundations of heavy buildings.

At Contrecoeur, the extension of the Sidbec steel plant will be founded on several thousand feet of bearing piles, driven through 60 ft (18 m) of sensitive clay, followed by 10 ft (3 m) of silt and sand, and about 13 ft (4 m) of very dense silt to end bearing in glacial till at depths of 80 to 100 ft (25 to 30 m). Cost analysis has shown precast concrete piles to be considerably more attractive than the low-displacement steel piles currently used in the area. Before selecting the concrete piles, it was necessary to investigate whether the hard driving through the very dense silt and the disturbance problems normally associated with the use of displacement piles in sensitive clay would allow the use of these piles.

The technical literature suggests that the problems with displacement piles in soft sensitive clay might be of less importance than previously anticipated. Also, it is believed that the particularly difficult driving conditions could be overcome by using high quality concrete piles. To verify this, an extensive pile test was undertaken at the site to study all factors related to the use of precast concrete piles for the proposed plant extension.

When driving piles in clay, the clay is remoulded and displaced vertically and horizontally, and sudden increase of pore pressures and lateral total pressures is generated. The soil displacements could result in movement of nearby existing foundations, in a reduction in the shear strength of the clay and an increase in its compressibility.

After the pile driving, the clay reconsolidates gradually and regains some of its lost strength during the dissipation of the pore pressures. Simultaneously, consolidation settlements occur

which, in certain cases, can cause concern. When piling in or near a slope, the problem of slope stability needs to be considered.

### Literature

In a classical paper, Casagrande (1932) discusses theoretically the effect on the soil compressibility from the driving of friction piles in clay and mentions that this could increase settlements rather than decrease them. Although Casagrande cautions that it will be necessary to conduct field investigations to study the actual effect of pile driving, the impact of his discussion has been tremendous. "Some engineers even believe that pile driving in soft clays is always detrimental and should be avoided" (Cummings *et al.* 1948).

Results of field investigations were first published in 1948. Cummings *et al.* (1948) report a field test made to determine the extent of the disturbance produced by driving displacement piles into a soft clay deposit. The natural structure of the clay was appreciably disturbed close to the piles. At a distance of about 2 pile diameters, the clay within a pile group was disturbed only by a negligible amount. A lengthy and interesting discussion followed the publishing of the paper.

The same year, Housel and Burkey (1948) and Tschebotarioff and Schuyler (1948) published papers supporting the findings of Cummings *et al.* (1948).

Bjerrum and Johannessen (1960) report measurement of pore water pressure induced by driving square 12 in. (30 cm) concrete piles and open-end hollow 8 in. (20 cm) square steel-box piles. The piles were driven for two bridge abutments in a soft sensitive plastic clay. Measurements taken at the concrete pile group showed that 3 to 4 months after the driving, considerable excess pore pressures still remained in the clay. Therefore, it was decided to drive low-displacement steel-box piles at the other side of the river, where a slope stability problem existed. The steel piles were driven at

a spacing of 7.5 diameters and pore pressure measurements were taken every day. The pore pressure increase was large, despite the use of the widely spaced low-displacement piles. After pile driving, the pore pressures were "so great within the central part of the pile group that the total overburden was carried by the water pressure". From a distance of about 5 diameters outside the pile group, the excess pore pressure then tapered off to very small values at a distance of 15 diameters from the pile group. The induced pore pressures dissipated rapidly at first, but more slowly with time. One year after driving, 40% of the excess pore pressure measured 2 weeks after driving still remained.

Klohn (1961) reports measurements of large ground surface heave and heave of previously driven piles, when driving end bearing timber piles in clay. The author discusses the engineering difficulties caused by the heave and suggests remedies, such as using low-displacement piles and careful retapping sequences. Additional examples and suggestions are given in the discussions to the paper.

Holtz and Lowitz (1965) studied the effect of driving 70 ft (21 m) long displacement piles at 3.5 diameter spacing in a lean low sensitivity clay on the undrained shear strength and compressibility of the clay and found that the pile driving did not cause significant loss of shear strength or change of compression indices.

Lambe and Horn (1965) studied the influence on an adjacent existing building from groups of partially preaugered and partially driven 70 ft (21 m) long concrete piles. Excess pore pressures developed due to the pile installation, measurable at a distance of 100 ft (30 m) from the piles. During the pile installation, the ground surface within the pile groups heaved about 0.2 ft (60 mm), then settled back about 0.02 ft (6 mm) during the dissipation of the induced pore pressures. The adjacent building foundations, which were less than 10 ft (3 m) away, heaved 0.02 ft (6 mm), then settled 0.03 ft (9 mm).

Lo and Stermac (1965) investigated theoretically, and with reference to field studies, the magnitude and distribution of pore pressures induced from pile driving and showed that the maximum induced pore pressure can exceed the effective overburden pressure by

up to 30% in normally or very slightly over-consolidated clays or loose silts. The authors suggest that the induced pore pressures would taper off over a distance of 16 pile diameters from the piles.

Orrje and Broms (1967) measured influence on undrained shear strength and pore pressures from driving groups of concrete and timber piles in soft sensitive plastic clay. The shear strength was slightly reduced within a zone of 1.5 pile diameters around closely spaced piles. The average shear strength was not appreciably affected for a pile spacing of 4 pile diameters. The induced pore pressures dissipated rapidly after the driving.

Broms and Bennermark (1967) report a failure of a slope in sensitive clay. The failure was caused by driving timber piles in the upper part of the slope. The soil displacement caused by the piles resulted in a shear strength reduction in the clay by 20 to 30%. Once the shear strength was reduced, the weight of the sliding mass was sufficient to cause additional displacements and additional reduction of the shear strength and thus the slope failure. No pore pressure measurements are reported.

Fellenius and Broms (1969) report measurements of soil heave with depth and induced pore pressures from driving two single 12 in. precast concrete piles at a spacing of 30 ft (10 m) in a normally consolidated sensitive clay. The driving caused the ground surface to heave 0.8 in. (20 mm) close to the piles. The heave decreased with increasing depth and small settlements were measured below a depth of 15 ft (5 m). At a point located 16 and 36 ft (5 and 11 m) away from the two piles, respectively, the ground surface heaved 0.2 in. (6 mm), which heave decreased with depth. The measured pore pressures close to the piles exceeded locally the total overburden pressure by 20%. The excess pore pressures dissipated in 120 days. Only negligible excess pore pressures (one gauge only) were measured at a distance of 16 to 36 ft (5 to 11 m) away from the piles.

D'Appolonia and Lambe (1971) report measurements of pore pressures and heave during driving of concrete filled pipe piles in preaugered holes in Boston blue clay. The measurements showed that the preaugering reduced, but did not eliminate, the development

of excess pore pressures. At a distance of 33 ft (10 m) away from the piles, however, the induced pore pressures were small. Due to the pile driving, adjacent structures heaved 0.01 to 0.03 ft (3 to 10 mm). Measurable heave occurred at distances of 100 ft (30 m) and greater. No damage to the adjacent structures was observed.

Hagerty and Peck (1971) measured heave caused by driving a group of piles in a non-sensitive clay and report that 50% of the pile volume was obtained as a heave of the ground surface within the pile group.

Torstensson (1973) measured heave, pore pressures, and influence on undrained shear strength and compressibility, when driving a large shaft bearing timber pile group in a soft sensitive clay. The induced heave of 5 in. (12 cm) within the group corresponded to about 40% of the pile volume. The heave was shown to be effectively reduced by preboring. The excess pore pressures exceeded the total overburden pressure and dissipated in 2.5 months. The shear strength of the clay within a distance from the piles of 1.5 pile diameters was reduced by 10%. No tendency of time dependent regain of strength was observed. Oedometer testing on clay samples taken before and 2.5 months after the pile driving indicated a small disturbance of the soil structure and, also that the clay had still not reconsolidated fully. During 3.2 years following the driving, the foundations settled 0.7 in. (17 mm). A similar building on a raft foundation settled twice as much.

Bergdahl and Nilsson (1974) report inclinometer measurements of four 12 in. (30 cm) precast concrete test piles which were driven prior to the driving of a large pile group around the test piles. The piles were driven through about 30 ft (10 m) of clay. The measurements show that the test piles moved away from the advancing group piles by an amount ranging from 0.3 to 1.1 ft (8 to 35 cm) with an average of 0.4 ft (11 cm). The inclinometer measurements showed also that bending was induced in the piles. The test piles were later test loaded successfully to a maximum load of 330 tons (2.9 MN).

Holtz and Boman (1974) report a failure of a river bank in silty clay, which was caused by excess pore pressures induced from driving

a group of 61 timber piles. The excess pore pressures were reduced by applying drain strips on the piles before driving, allowing the piling work to be completed. Fellenius (1975a) shows in a discussion on the paper that the pile driving affected the pore pressures within a distance of 66 ft (20 m) or 80 pile diameters away from the pile group.

Other valuable papers pertaining to the disturbance from pile driving in clay have been published by Bjerrum (1973), Brzezinski *et al.* (1973), Casagrande and Poulos (1969), D'Appolonia (1969), De Mello (1969), Flaate (1972), Hagerty and Garlanger (1972), Hansbo *et al.* (1973), Massarch (1974), Milligan *et al.* (1962), and Zeevaert (1972).

A study of negative skin friction on piles developed as a result of the reconsolidation of the remolded sensitive clay after driving, has been published by Fellenius (1972), who measured the resulting drag load to correspond to a negative skin friction of 15 to 20% of the initial undrained shear strength of the clay or about 5% of the vertical effective stress.

In a design manual by the Norwegian Geotechnical Institute (1973), it is mentioned that a drag load corresponding to a negative skin friction equal to 10% of the vertical effective stress can be expected due to reconsolidation of the sensitive clay alone.

### Site Conditions

The test site is located on the Sidbec property at Contrecoeur, Quebec, where the soil conditions are characterized by a thick deposit of marine clay from the ancient Champlain sea, a soil formation frequently found in the St. Lawrence valley.

The soil conditions at the test site consist of a 2 ft (0.6 m) surface fill of crushed stone underlain by stratified clay and silt to 6 ft (2 m) depth (Fig. 1). From 6 ft (2 m) to about 60 ft (18 m), is encountered a slightly overconsolidated banded light and dark grey clay, the thickness of the bands varying from 0.25 in. to 3 in. (6 to 60 mm). The marine clay has a sensitivity of about 20, is highly plastic, and has a natural water content of about 60%, which slightly exceeds the liquid limit. The clay contains occasional silty fine sand layers, varying in thickness from 0.05 to 0.5 in. (1 to 12 mm). The percentage of par-

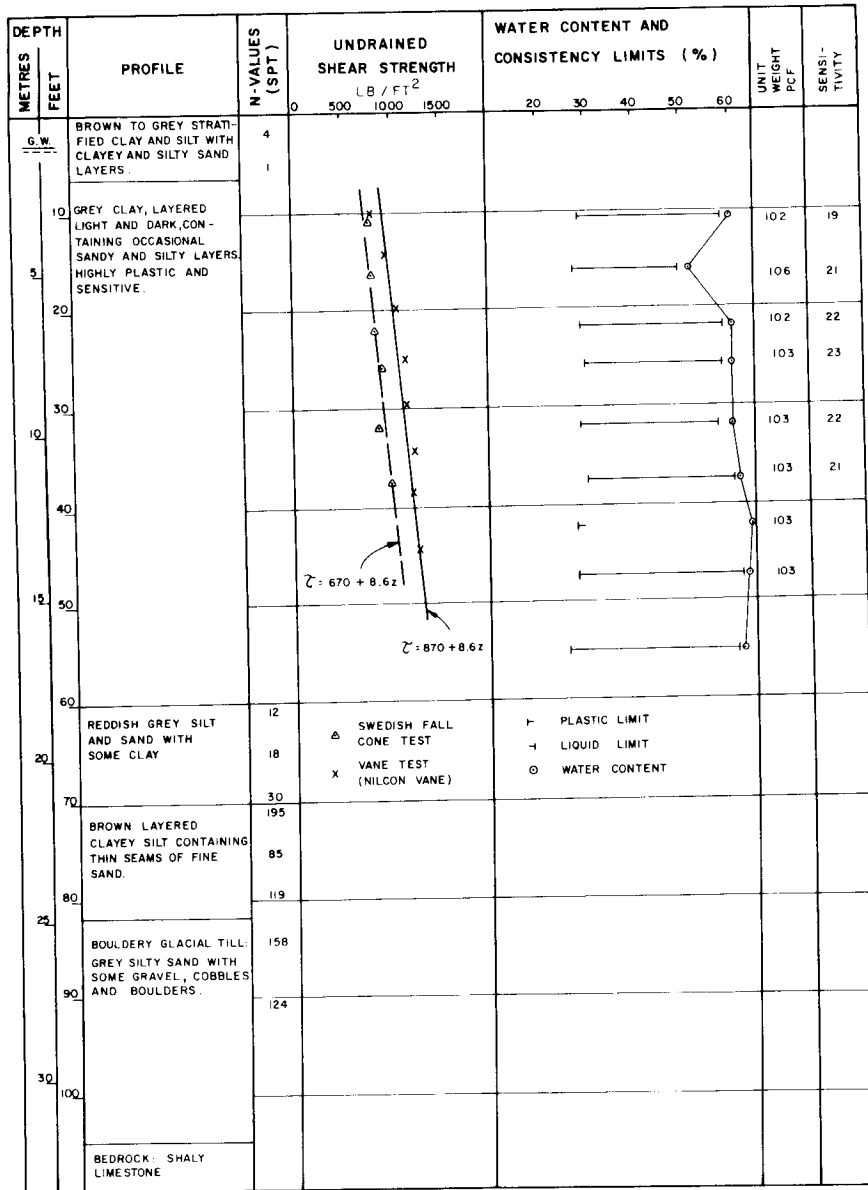


FIG. 1. Typical profile and soil properties at the test site.

ticles finer than 0.002 mm in the clay is about 60%. The preconsolidation pressure in the clay is 1.2 to 1.4 tsf (120 to 140 kPa) over the effective vertical stress existing in the soil. The undrained shear strength of the clay as measured with the field vane tester is about 900 psf (45 kPa) near the upper boundary and increases linearly with depth to a value of 1400 psf (70 kPa) at 60 ft (18 m) depth.

The clay is underlain by a layer of loose to

compact clayey silt and sand to about 70 ft (21 m) depth, where a very dense clayey silt is encountered. At about 82 ft (25 m) depth, the soil consists of very dense bouldery sandy silty glacial till. Bedrock, consisting of shaly limestone, is found at about 105 ft (32 m) depth.

At the time of the field investigation, the ground water table was about 4 ft (1.2 m) below the ground surface. For approximately

the first 30 days, mid-February to mid-March, the upper 2 ft (0.6 m) of the soil were frozen.

### Testing Program

To investigate the suitability of precast concrete piles at the given site, the following program was undertaken with two specific objectives:

(A) Determination of the disturbance effect on the sensitive clay, caused by driving a group of 13 displacement piles, in terms of pore pressure increase, soil heave, reduction in shear strength and change in compressibility of the clay.

(B) Determination of the behaviour of the precast concrete piles during hard driving and their capacity in bearing and uplift.

#### Test layout and soil testing

The test included the driving of thirteen 12 in. (30 cm) Herkules H800 concrete piles in a group with a center to center spacing of 4 pile diameters according to the layout shown in Fig. 2. Ten of the piles were driven to refusal in the glacial till (about 84 ft or 26 m) to study the driving behaviour and the ultimate bearing capacity of the piles. The three remain-

ing piles (pile No. 8, 9, and 10) were driven to a predetermined depth of 59.5 ft (18 m) to study the uplift capacity of the piles in the clay layer and to fill out the group.

The instrumentation to monitor pore pressure in the clay consisted of eight Gloetzl pneumatic piezometers placed two by two in four holes. Four piezometers were placed within the pile group and four 6 ft (1.8 m) outside at depths of 15, 20, 28, and 40 ft (4.5, 6.1, 8.5, and 12.2 m). All piezometers were put down at the latest 1 week prior to pile driving and readings were taken to verify that the piezometers had stabilized before the pile driving started. Readings of the piezometers were taken during the pile driving and several times after driving.

Measurements of ground surface heave were taken at one point inside and two points outside the pile group. The heave gauges consisted of spiral augers, placed at about 7 ft (2 m) depth. The auger stems were protected by a casing from the influence of frost and upper granular soil. The measurements were made by accurate leveling of the auger stem top. The maximum error in the heave measurements is approximately 0.02 ft (6 mm). The pile head elevations were also monitored by levelling.

Seven vane borings were carried out at various distances from the piles: one prior to driving, four 1 to 3 days, and three 79 to 83 days after driving. The vane tests were performed at every 5 ft (1.5 m) to about 40 ft (12 m) depth using a Nilcon vane tester, size 5.1 in. by 2.6 in. (130 mm by 65 mm).

Soil was sampled in two boreholes at 5 ft (1.5 m) intervals to about 40 ft (12 m) depth using a 3 in. (76 mm) stationary piston sampler. Borehole I was put down prior to pile driving and Borehole III, 75 days after driving. The laboratory testing included water content, Atterberg limits, Swedish fall cone tests, and oedometer consolidation tests.

The installation of the instruments and soil testing and sampling prior to pile driving were performed between February 15 and February 21, 1974.

#### Pile driving tests

Between February 27 and March 6, 1974, all 13 test piles were driven in the locations and sequence shown in Fig. 2. The driving was

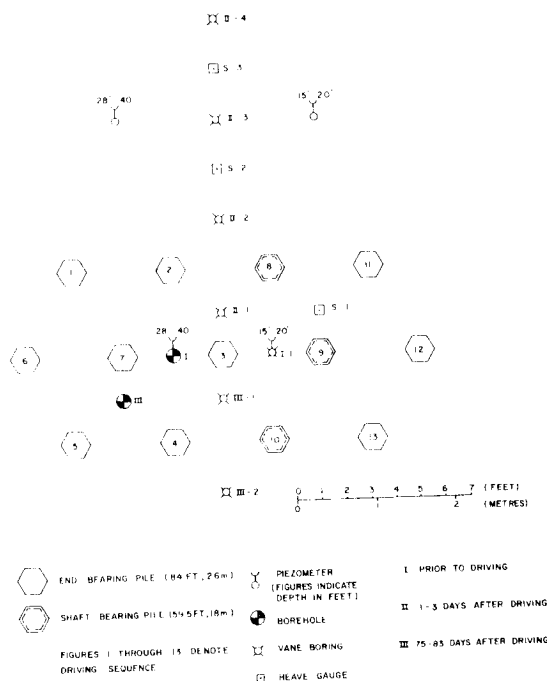


Fig. 2. Layout of piles, instrumentation and soil tests.

performed with a drop hammer weighing 4.32 tons (38 kN). The hammer height of fall while the pile tip was in clay was maximum 1 ft (30 cm). When the pile tip reached the very dense silt, the height of fall was increased to 2 ft (0.6 m). A diesel hammer Link-Belt 520, was alternatively used. As the ground was frozen to a depth of about 2 ft (0.6 m), a 3 ft (1.0 m) hole was prebored at each pile location. A more detailed description of the tested piles and the pile driving equipment is given below.

#### *Pile loading tests*

Three out of the 10 endbearing piles were load tested, pile No. 6, 7, and 13. The reaction load was obtained from a kentledge arrangement, and maximum test load varied from 360 to 445 tons (3.2 to 4.0 MN). The testing procedure was a quick Maintained-Load method with load increments of 20 tons (0.18 MN) applied every 15 min as proposed by Fellenius (1975*b*). For pile No. 6 which was provided with center pipe, a tell-tale was placed in the pile by which the tip movement and the compression of the pile were measured.

Two uplift tests were carried out, one on a full length end bearing pile (No. 5), and the other on a short shaft-bearing pile (No. 9). The shaft bearing pile was driven to 59.5 ft (18 m) depth and protected by a casing from the influence of the upper 7.5 ft. (2.2 m) of the soil layers, leaving 52 ft (16 m) in contact with the soil. Also the uplift test procedure was performed according to the quick Maintained-Load method.

### **Piles and Driving Equipment**

#### *Pile description*

All test piles were standard 12 in. (30 cm) Herkules H800 precast concrete piles. They were picked at random from the contractor's stock and delivered to the site in 20 and 40 ft (6 and 12 m) long segments, which were spliced in the field with Herkules mechanical steel joints cast at the ends of the pile segments.

The concrete was specified to have a minimum 7500 psi (52 MPa) 28 day strength, and the reinforcement steel, a yield point of 86 000 psi (593 MPa). Tests on concrete cylinder specimens from the concrete pours varying in dates from October 23, 1973 to February

13, 1974 showed the 7 day strength to be 5940 to 8210 psi (41.0 to 56.6 MPa) with a weighted average of 7170 psi (49.5 MPa), and the 28 day strength to vary from 7590 to 8895 psi (52.4 to 61.4 MPa) with a weighted average of 8380 psi (57.8 MPa). The mill test on the steel showed a yield point of 86 380 psi (596 MPa).

A difference in pour dates made a difference in the age of the piles at the time of driving, which ranges between 3 weeks (pile No. 7) and 19 weeks (pile No. 6).

The piles are of hexagonal cross section with a circumference of 42 in. (106 cm), an area of 124 in.<sup>2</sup> (800 cm<sup>2</sup>) and are longitudinally reinforced with six deformed No. 5  $\frac{5}{8}$  in. (16 mm) bars. To eliminate the risk of spalling and cracks in the pile, and to optimize the bending strength of the pile, the concrete cover over the longitudinal reinforcement is 1.0 in. (25 mm). The 0.2 in. (5 mm) diameter spiral reinforcement is distributed over the entire length of the pile segment by 150 turns spaced evenly over a 40 ft (12 m) length, *i.e.* a pitch of 3.2 in. (8 cm).

The Herkules splices consist of two extremely plane heavy steel plates, each connected to the pile segment by six 20 in. (50 cm) long bars of the same type as the longitudinal reinforcement and threaded and bolted to the steel couplings. The pile tip (bottom end of first segment) is provided with a rock shoe consisting of a 1 in. (25 mm) thick steel plate with a separate centrally located point of hardened steel alloy—hardness 590 Brinell—8 in. (20 cm) in length and 2 in. (5 cm) in diameter, placed in a 6 in. (15 cm) tube with heavy lateral triangular supports. Pile No. 6 was provided with a 1.7 in. (42 mm) center pipe giving access to the pile end.

#### *Driving equipment*

The driving was alternatively performed with a drop hammer and a diesel hammer.

The drop hammer weighed 4.32 ton (38 kN) and was used with a steel helmet (saddle type) weighting 600 lb (2.7 kN). The cap block consisted of 8 in. (20 cm) of hardwood and the cushion pad of 2 in. (5 cm) (8 layers) of hard masonite. During driving, the head of the pile was protected by the female part of the steel splice and a loose 1.8 in. (46 mm) thick



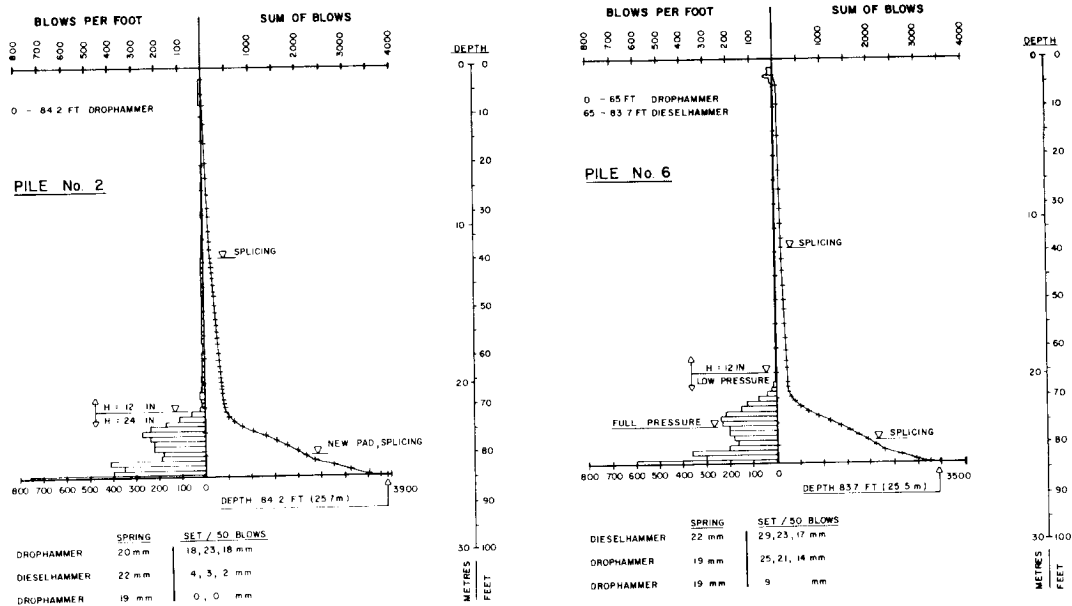


FIG. 3. Driving diagrams of pile No. 2 and 6.

steel cap weighing 40 lb (180 N). The drop hammer has a capacity of about 35 blows/min at a height of fall of 2 ft (60 cm). Driving energy at 2 ft (0.6 m) height of fall is 17 300 ft-lb (23.5 kNm).

The diesel hammer was a Link-Belt 520 with a rated maximum energy of 26 300 ft-lb (35.7 kNm). The impact was softened by the same cushion pad and steel cap, as was used with the drop hammer. The total weight of the diesel hammer is about 8 tons (71 kN) and the ram weight is 2.5 tons (22 kN). The diesel hammer has a capacity of about 80 blows/min at full pressure.

## Results

### Pile driving

Figure 3 shows the results from the driving of pile No. 2 and 6. The diagrams are typical of the driving of the 10 end bearing piles. The driving resistance is given in terms of blows per foot of penetration and the accumulated sum of blows. The change of height of fall of the drop hammer or change of pressure in the diesel hammer have been marked in the diagrams at the location of the pile tip when the change occurred. On termination of the driving, and for the retapping series, the spring of the pile for each blow and the set (penetration) for

50 blows were recorded. All piles were re-tapped at least once with both hammers, when the neighbouring piles had been driven.

Through the clay, the driving energy was kept low. This was required to minimize tensile forces in the pile, which otherwise would develop due to lack of end resistance. When the pile tip had reached the silt layer, the drop or pressure was increased. The maximum drop height was specified to 24 in. (60 cm) which according to relationships presented by Broms and Hellman (1970) gives a maximum dynamic stress in the pile during driving in the approximate range of 2000 to 4000 psi (13.8 to 27.6 MPa). At termination of driving, the stress near the pile tip could be larger due to reflection of the shock wave in the very dense glacial till. The range of stresses in the pile, induced by the diesel hammer, is about the same as for the drop hammer.

The driving diagrams show that the driving resistance was small or nonexistent through the clay down to about 60 ft (18 m) depth. In the loose to compact clayey silt to about 70 ft (21 m) depth, the resistance was quite small, about 10 to 50 blows/ft. The variation of resistance is more due to small variations of the height of fall of the drop hammer or of the pressure in the diesel hammer, than to the

actual difference in driving resistance of the soil. At about 70 ft (21 m) depth, where the very dense silt was encountered, the driving resistance increased to about 150 to 300 blows/ft or 12 to 25 blows/in. As judged from the driving records and the data of nearby boreholes, the glacial till was reached by all test piles at approximately 81 to 83 ft (24.5 to 25 m) depth and penetrated it by about 1 to 2 ft (0.3 to 0.6 m) with continuously increasing resistance. At termination, the sets per 50 blows were generally between 1.0 and 0.5 in. (25 and 12 mm), corresponding to 600 to 1200 blows/ft or 50 to 100 blows/in. The increase of resistance in the glacial till is very evident and of a character normally encountered when driving piles to hard bedrock.

The total number of blows required to drive the piles ranged from 2100 to 3900 with the smallest number of blows required for the diesel driven piles. No deterioration of any of the test piles was observed following this hard driving.

Pile No. 1 to 7 were driven in a group with pile No. 7 driven last and in the center of the group in order to study if any densification effect in the very dense silt was obtained from previously driven neighbouring piles. No such influence was observed, nor did the retapping of each pile after the neighbouring piles were driven show any noticeable loosening or densification in the soil around already driven piles.

The driving records have been analysed by means of the wave equation. The computer program used in the one dimensional analysis is presented by Edwards (1967), and the analysis confirms the stated approximate range of driving stresses given an average maximum stress over the pile area of 2600 psi (18 MPa). The wave equation analysis was also used to predict the ultimate bearing capacity of the piles. The predicted bearing capacity vs. the driving resistance is shown in Fig. 4.

The computed ultimate bearing capacity shown in Fig. 4 has an asymptotic maximum value of 155 tons (1.38 MN), when the driving resistance approaches infinity. At a resistance of 15 to 20 blows/in., which is representative of the resistance obtained in the very dense silt, the predicted bearing capacity is about 110 tons (1.0 MN). Previous testing at the site of piles terminating in the very dense silt

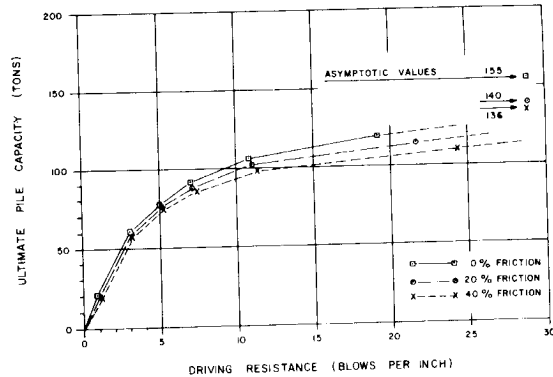


FIG. 4. Results of wave equation analysis assuming distributions of total resistance from 0% shaft friction and 100% end bearing to 40% shaft friction and 60% end bearing.

have shown ultimate failures approximately in agreement with this value, when the shaft resistance in the reconsolidated clay is deducted. However, as shown below, the ultimate capacity of the piles driven into the glacial till greatly exceeded the predicted capacity. The discrepancy is due to the fact that the driving resistance in the till is more representative for the three-dimensional problem of seating the pile tip into the till and compacting the soil around the pile tip, than to the effects of friction forces and dampening, which is assumed in the one-dimensional wave equation analysis.

#### Pile loading tests

**Testing method and arrangement.** As part of the testing program, three piles (pile No. 6, 7, and 13) were tested to high loads and considerably beyond twice the design load. A quick Maintained-Load test method (M.L.-test) was adopted consisting of small load increments applied in short constant time intervals, and measuring the pile movements induced by each applied load. The time interval can be as short as only a few minutes. However, this can be difficult to achieve with manual pumps. For this reason, 15 min intervals were used. The number of load increments before reaching the ultimate pile capacity in the quick M.L.-test should be between 15 and 20. Therefore, as ultimate pile capacity was anticipated to be at least 400 tons (3.6 MN), increments of 20 tons (0.18 MN) were chosen.

The test loads were applied using two 285 ton (2.54 MN) capacity 14 in. (35.6 cm)

diameter hydraulic jacks reacting against a kentledge consisting of steel ingots to a total weight of about 450 tons (4.0 MN). Pile movements were recorded by two dial extensometers with a graduation of 0.001 in. (0.025 mm) and placed diametrically opposed on two measuring beams supported on stakes driven into the ground outside the kentledge limits and at 8 ft (2.4 m) distance from the nearest kentledge support.

As mentioned, pile No. 6 was provided with a center pipe through the pile to measure the pile compression between the pile head and the pile tip. This was achieved by inserting a rod through the pipe and placing a dial extensometer on the rod at the pile head and measuring movements between the rod and the pile head at the edge of the center pipe. The difference between the pile head movement and the pile compression gives the movement of the pile tip.

**Results.** The results of the test on pile No. 6 are presented in Fig. 5. As shown, the movement of the pile head follows a slightly curved line up to a load of 420 tons (3.74 MN), when the kentledge platform lifted and the pile had to be unloaded. The net movement immediately after unloading was 0.345 in. (8.76 mm), and when the extensometers were removed 13 h later, the pile head had recovered to 0.241 in. (6.12 mm). The pile tip started to move at 160 ton (1.43 MN) load and at the maximum load of 420 tons (3.74 MN), the tip movement was 0.209 in. (5.31 mm). The pile tip returned to 0.011 in. (0.28 mm) net movement following unloading. Had greater reaction capacity been available, the pile test would have been carried further, as the 420 ton (3.74 MN) load is less than the ultimate capacity of the pile.

The maximum tip penetration of 0.209 in. (5.31 mm) is less than the penetration necessary to reach soil failure at the pile tip. This statement is verified by the almost complete rebound of the tip to a net settlement of 0.011 in. (0.28 mm) following unloading.

Beyond about 300 ton (2.7 MN) load, all the additionally applied load goes unrestrained by shaft friction to the pile tip, as judged from the linear behaviour of the measured pile compression. Thus, the measured pile compression beyond the 300 ton (2.7 MN) load can be

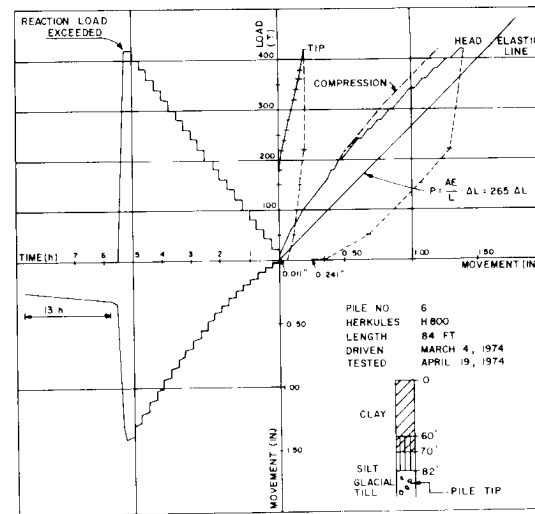


FIG. 5. Results of test loading pile No. 6.

used to calculate the elastic modulus of the pile. This calculation gives  $E = 4.3 \times 10^6$  psi ( $3.0 \times 10^{10}$  Pa), which is in agreement with the range of values reported in the literature for precast concrete piles (Fellenius and Eriksson 1969). The load–movement diagram of Fig. 5 has been completed with the computed elastic line of the pile, *i.e.* the compression of the total pile length, had it been free from the shaft friction and resting on an unyielding bottom.

From the above observations, a minimum value of the pile tip load can be estimated. The pile tip started to move at a load of 160 tons (1.43 MN), and at about 220 tons (2.0 MN) the tip movement became appreciable, about 0.02 in. (0.5 mm). At this latter load, the slope of the pile head curve becomes parallel to the elastic line, indicating that the larger portion of the hereafter applied load reaches the pile tip. As derived previously, beyond the 300 ton (2.7 MN) load, all the applied load is transmitted to the pile tip. Thus, the minimum pile tip load at the maximum test load of 420 tons (3.7 MN) can be estimated to be the larger portion of the applied load between 220 and 300 tons (2.0 and 2.7 MN), and a minimum of 50 tons (0.45 MN), plus all of the load from 300 to 420 tons (2.7 and 3.7 MN). Conservatively, the tip load is then estimated to be at least about 170 tons (1.5 MN).

The calculated elastic modulus and the mea-

sured pile compression for each load increment have been used to calculate an average load in the pile. The result of this calculation has been used in Fig. 6 to obtain an approximate load distribution during the test by plotting the calculated average load at midpoint of the pile and connecting the plotted pile head loads and midpoint average loads with straight lines to the pile tip (in reality, the load distribution lines are slightly curved, as the unit shaft friction increases with depth). Thus, an approximate estimate of the pile tip loads is obtained. Also, in Fig. 6, the measured tip penetration has been plotted against the pile tip load so determined.

The estimated tip loads on Fig. 6 are the maximum tip loads. Consequently, the actual tip load at the test load of 420 tons (3.7 MN) can be bracketed between the above conservative minimum value of 170 tons (1.5 MN) and the maximum value of 220 tons (2.0 MN) obtained from Fig. 6. Thus, a total shaft friction force of minimum 200 tons (1.8 MN) and maximum 250 tons (2.3 MN) was acting on the pile during the test, corresponding to an average skin friction of about 1300 to 1600 psf (60 to 80 kPa) acting along the full length of the pile.

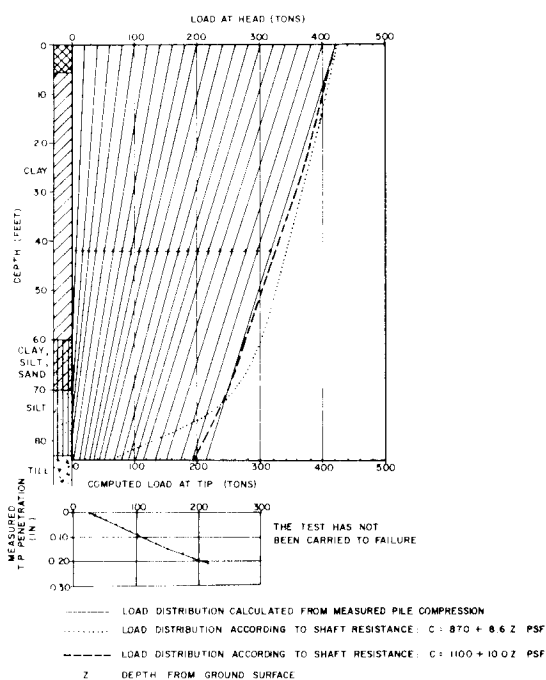


FIG. 6. Approximate load distribution during test loading on pile No. 6.

The shaft resistance at 420 ton (3.7 MN) load at the pile head can be more closely estimated than by the straight load distribution lines shown in Fig. 6. A new pile load distribution line corresponding to a skin friction on the upper 70 ft (21 m) of the shaft, equal to the vane shear strength of the clay ( $c = 870 + 8.6z$  psf), is drawn as a dotted line in the diagram of Fig. 6. The dotted line distribution warrants a very high shaft resistance in the soil below 70 ft (21 m) depth (about 30 000 psf or 1.5 MPa) and an unrealistically low tip resistance (60 tons or 0.5 MN). Such high shaft resistance and low tip resistance are mutually incompatible. The dotted line distribution is derived from the condition (evaluation principle after Fellenius 1969) that, in order to give the same pile deformation as measured during the test, the area to the right of the straight load-distribution line from 420 tons (3.7 MN) at the pile head must be equal to the area to the left of the same straight load-distribution line, as measured to the dotted distribution line.

Based on the same principle, a new shaft resistance distribution giving a load distribution line that better answers to the actual conditions was found by trial and error and is shown as a dashed line in the diagram. The dashed load distribution gives a more realistic value of the shaft resistance in the silt (8300 psf or 400 kPa). The new line corresponds to a shaft resistance in the clay with the equation  $c = 1100 + 10.0z$  psf. This equation corresponds to a unit shaft resistance in the clay that is greater than the initial undrained shear strength of the clay by about 25%.

The dashed load distribution gives a pile tip load of 190 tons (1.7 MN), which lies within the range mentioned earlier. The above discussion of the shaft resistance has aimed to establish this load value. In the process and as a first approximation, the authors used the undrained shear strength of the clay as given by the vane tester. In doing so, the authors do not claim that the shaft resistance of the pile is governed by the undrained shear strength. However, a detailed effective stress analysis of the actual shaft resistance is beyond the scope of this paper.

The above discussion assumes that there is no locked-in compression in the pile. However,

during the 43 days, which elapsed between the pile driving and the test loading of pile No. 6, some reconsolidation of the clay and subsequent negative skin friction must have resulted. As mentioned in the literature review, Fellenius (1972) measured negative skin friction after full dissipation of the pore pressures (150 days after driving) on the order of 15 to 20% of the initial undrained shear strength in the clay, or 5% of the effective overburden pressures. Applying the same values for pile No. 6, which is considered conservative, an estimated total drag load of about 20 tons (0.18 MN) is derived. An initial drag load of this magnitude acting on the pile prior to test loading would move all the average loads in Fig. 6 a corresponding distance to the right. The dotted load distribution would then become more realistic and the resulting estimated tip load become about 230 tons (2 MN).

Thus, in conclusion, the shaft resistance in the clay is between 100 and 125% of the initial undrained shear strength of the clay, and the tip load at the maximum test load is between 190 and 230 tons (1.7 and 2.0 MN).

The remaining compression of the pile of 0.23 in. (5.8 mm) after unloading corresponds to an average load of 60 tons (0.54 MN) in the pile. This is the effect of the shaft friction from the soil preventing the full rebound of the pile.

The maximum test load of 420 tons (3.7 MN) is not the failure load of the pile. Maximum shaft resistance of the pile was reached, but the glacial till at the pile tip was still being loaded within the pseudo-elastic range and the maximum tip resistance was not reached.

Results of test loading of pile No. 7 and 13 to 445 and 360 tons (4.0 and 3.2 MN), respectively, do not show any significant difference of load movement behaviour from the results of the test on pile No. 6, nor was the ultimate soil resistance reached. Pile No. 6 and 7 had a driving resistance at refusal of 9 mm per 50 blows (1700 blows/ft). The driving of pile No. 13 was purposely stopped at the three times smaller driving resistance of 27 mm per 50 blows (560 blows/ft). However, the test loading results show no difference between the piles which can be related to the difference in refusal resistance.

#### *Pile uplift tests*

*Testing method.* End bearing pile No. 3 and shaft-bearing pile No. 9 were tested for uplift. The reaction was obtained from two adjacent piles through a beam resting on these piles. A quick M.L.-test method was used also for the uplift test. As the load increments had to be small (6 ton or 53 kN increments were chosen) and ordinary readings of jack pressure manometers often are inaccurate in the low pressure range, a load cell was used to measure the load. The load cell was vibrating wire dynamometer type DL of Telemac International Inc. As expected, the load cell measurements show that the load increments of the 285 ton (2.54 MN) jack deviated from the intended value, due not only to the manometer inadequacy in the low load range, but to the difficulty of reading the exact pressure. However, the load cell was slow in reading and thus the load increments were applied by referring to the jack pressure manometer, whereas the actual load was measured with the load cell. The mentioned inaccuracy of the jack pressure gave variations in the magnitude of the individual load increments, which, however, did not impair the test.

Previous uplift proof-testing of the Herkules H800 pile has been carried out to a maximum load of 90 tons (0.8 MN) and the ultimate test load has been reached in failure of the pile head by snapping of the reinforcement due to unavoidable eccentric pull. To minimize the effect of eccentricities and to ensure a greater pull than otherwise anticipated, the pile head of pile No. 3 was reinforced by casting a reinforced concrete pile cap around the pile head. The pull was then applied through a horizontal bar inserted through the cap. Unfortunately, the pile cap proved to be the weakest point, and at a pull of 67 tons (0.60 MN) a sudden failure of the cap occurred.

*Results from pile No. 9.* The results of the uplift test on shaft bearing pile No. 9 are presented in Fig. 7a. This pile was loaded in two cycles, because in a first test, the clamps, which held the dial extensometers measuring the pile head movement, loosened at a load of 48 tons (0.43 MN) and the pile had to be unloaded before the extensometers could be fastened again. In the second cycle, performed 4 days later, soil failure occurred at 51.4 tons (0.458

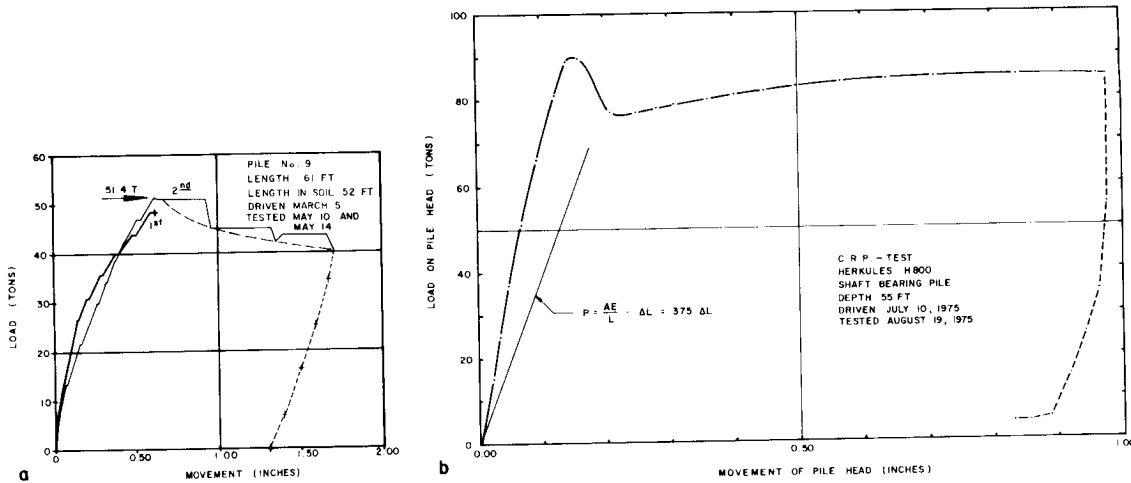


FIG. 7. a. Results of uplift testing of pile No. 9. b. Results of test loading of a shaft bearing pile (the test is a part of a later investigation about 600 ft or 180 m outside the site).

MN). Beyond this peak value, three equilibrium load points were obtained on the curve by keeping a constant load for a while then letting the load on the pile release to a stable value. Each time this was repeated, a smaller equilibrium load was obtained. The dashed curve combining the peak value and the three equilibrium values correspond approximately to the test loading curve, which would have been obtained from a Constant-Rate-of-Penetration test (C.R.P.-test) (Fellenius 1975b).

The two load-movement curves show that the deformations were greater in the low load range in the second cycle as compared to the first cycle. This is due to the fact that in the first cycle, the shear resistance of the sensitive clay was fully mobilized in the upper portion of the pile. In reloading in the second cycle, only a smaller value could be mobilized and thus the applied load was brought further down the pile. However, the failure load of the second cycle is not considered to have been significantly affected by the previous loading cycle.

The failure load of 51.4 tons (0.458 MN) corresponds to an utilization of about 60% of the shear strength of the clay as measured with the vane tester before piling, and gives an average unit shaft resistance of 700 psf (35 kPa). In comparison, the previous interpretation of the compression test on pile No. 6 shows that in compression the unit shaft resis-

tance in the clay was 100 to 125% of the initial shear strength of the clay.

Another comparison is obtained from the compression test loading of a 55 ft (17 m) long identical shaft bearing pile as a part of a later investigation in 1975. This test was carried out about 600 ft (180 m) from pile No. 9. The test results are shown in Fig. 7b. The pile failed at a load of 90 tons (0.80 MN). Subtracting a tip load of  $9 \times C_{11} = 5$  tons (0.05 MN), the remaining load corresponds to a shaft resistance of about 90% of the initial undrained shear strength of the clay.

The downward pile shaft resistance at the site is thus about 50 to 100% greater than the upward shaft resistance. It is well known that uplift shaft resistance is lower than downward shaft resistance. However, it is normally not considered that the difference is as large as found in these tests.

The shaft bearing capacity of a pile depends on the lateral effective pressure on the pile shaft. Small diameter changes would affect this pressure. The difference of shaft bearing capacity in compression as opposed to uplift can be attributed to the fact that a compressed member increases its diameter, while in tension, the diameter is decreased. The movements are extremely small. Assuming a Poisson's ratio of 0.3 and using an elastic modulus of  $4 \times 10^6$  psi ( $2.97 \times 10^{10}$  Pa), the increase of the pile

diameter for an average compression load of 350 tons (3.1 MN) can be calculated to be 0.1 mm. The decrease of the pile diameter for an average tension load of 35 tons (0.3 MN) is 0.01 mm.

The effective lateral pressure depends also on the vertical pressure in the soil. In a compression test, the vertical pressure increases and thus, theoretically, the shaft resistance increases correspondingly, while in an uplift test the reverse occurs. The authors believe that a combination of the two effects could have caused the difference observed in the present test.

#### Soil disturbance

**Heave measurements.** As shown in Fig. 2, the ground surface heave caused by the pile driving was measured at three points. The heave measurements are given in Table 1, and show that at gauge S-1 the soil within the pile group heaved totally 0.23 ft (70 mm). Gauge S-2 shows a heave of 0.14 ft (40 mm), and gauge S-3, 0.11 ft (30 mm). It can be assumed that the heave is caused by the soil being displaced by the piles to a volume equal to the volume of piles for the full depth of the clay of about 60 ft (18 m), *i.e.* 670 cu ft (19 m<sup>3</sup>). Taking that the average heave within the pile group is 0.23 ft (70 mm) and that it linearly tapers off away from the pile group, one calculates a maximum zone of influence of 30 ft (9 m) distance from the piles. The literature references state that about 50% of the displaced soil could be obtained as a heave. When using this value, one calculates the zone of influence to be maximum 20 ft (6 m) distance from the piles. The actual observations indicate that the zone is greater than 8 ft (2.4 m).

The levelling of the pile heads showed that the piles did not heave with the clay.

The heave gauge readings taken until April 11, 36 days after end of pile driving, show that no additional heave nor settlement took place after the pile driving. In mid-April, work started at the site for the pile test loading, causing unavoidable disturbance to the heave gauges, and preventing further study of possible ground movements.

**Pore pressure.** Figure 8 shows the excess pore pressures measured from the start of the pile driving on February 27, 1974 through May

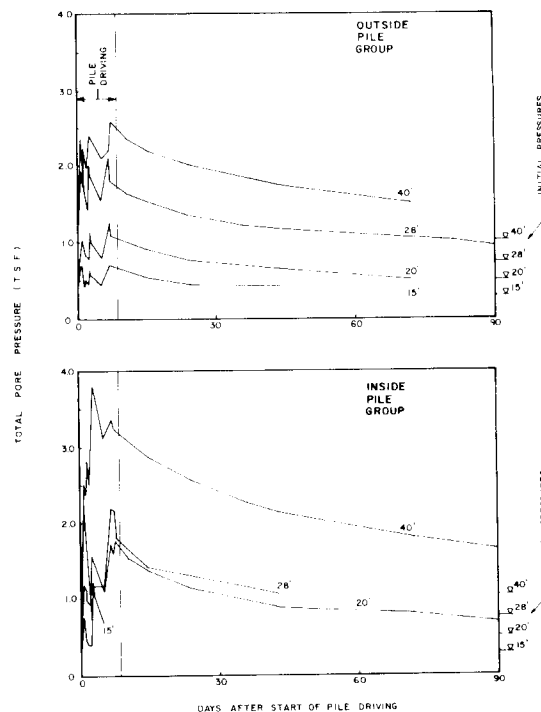


Fig. 8. Total pore pressures in the clay inside and outside the pile group during 90 days.

28, 1974, 90 days later, *i.e.* 82 days after the end of the pile driving. The piezometer at 15 ft (5 m) depth inside the pile group was damaged by the piles. Four of the other piezometers ceased to function for unknown reasons at various times after the pile driving. Due to pile testing activities and construction work at the site, further measurements were prevented until the end of July, 5 months after the pile driving. These latest measurements taken at the remaining three piezometers show that no excess pore pressure remained.

The pore pressure measurements taken during pile driving are shown in the diagrams in Figs. 9a and b. The pore pressures are plotted against time after the start of pile driving. The diagrams also show the distance of each test pile to the particular pair of piezometers, plotted at the date of driving. The open dots indicate the driving of about the last 15 ft (4.5 m) of pile No. 1 and 2.

As shown in Figs. 9a and b, most of the pore pressure increase occurred after the driving of the first piles. The occasional rapid dissipation of the induced pore pressures is an

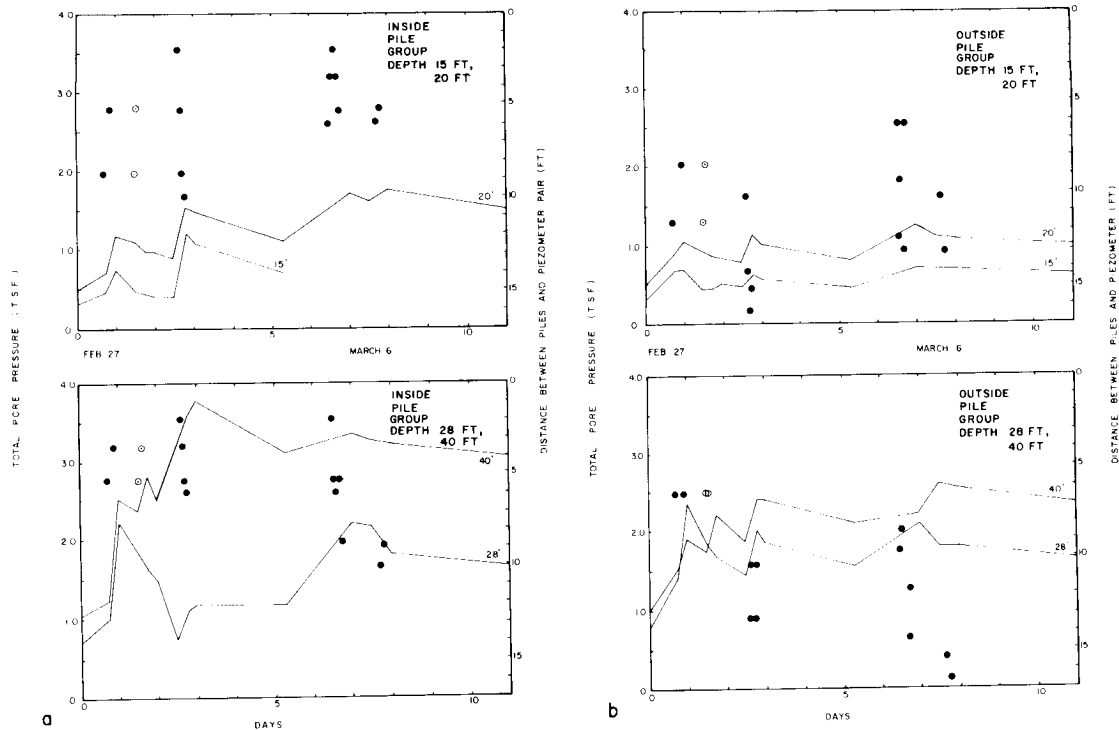


FIG. 9. Pore pressures during pile driving and the distance of each test pile to the particular pair of piezometers. The two open dots indicate the driving of about the last 15 ft (5 m) of pile No. 1 and 2.

indication of occurring fracturing of the clay. Figure 9b shows that already, the first two piles (No. 1 and 2), driven at a distance of 6 to 8 ft (1.8 to 2.4 m) from the piezometer pairs, caused the maximum pore pressure increase. The continued pile driving did not induce additional pore pressures, but seem to have slowed the rate of dissipation.

As would be expected, the pore pressures measured 6 ft (1.8 m) outside the pile group are smaller than those measured inside the group. However, the difference is small and all gauges show that very large excess pore pressures were caused by the pile driving. In fact, all gauges but one (15 ft or 5 m depth outside) measured pore pressures, which exceed considerably the initial total overburden pressures, as shown in Figs. 10a and b.

The dissipation of pore pressure was rapid at first, but became slow with time. The measurements show that about 3 months after pile driving, complete dissipation had not yet been reached, but 2 months later no excess pore pressure remained.

*Vane shear strength.* The results of the vane shear tests are plotted in Figs. 11 and 12. Fig. 11 shows that the pile driving caused a small reduction of about 15% of the shear strength of the clay inside the pile group, but no shear strength reduction could be observed outside the pile group even at a distance as close as 2.3 ft (0.7 m) from the nearest piles.

Vane testing at 79 to 83 days after the end of the pile driving (test No. III-1 and III-2) shows that a regain of shear strength has occurred within the pile group (Fig. 12).

*Laboratory tests.* Laboratory testing has been carried out on undisturbed clay samples obtained at the site prior to the pile driving and compared with the results on samples obtained from a borehole put down inside the pile group 75 days after the end of pile driving.

Visual examination revealed that the pale and dark bands and occasional silt seams, which were horizontal prior to the pile driving, were occasionally distorted and inclined in the samples obtained after driving (photo in Fig. 13). The obvious reason for the distortion is



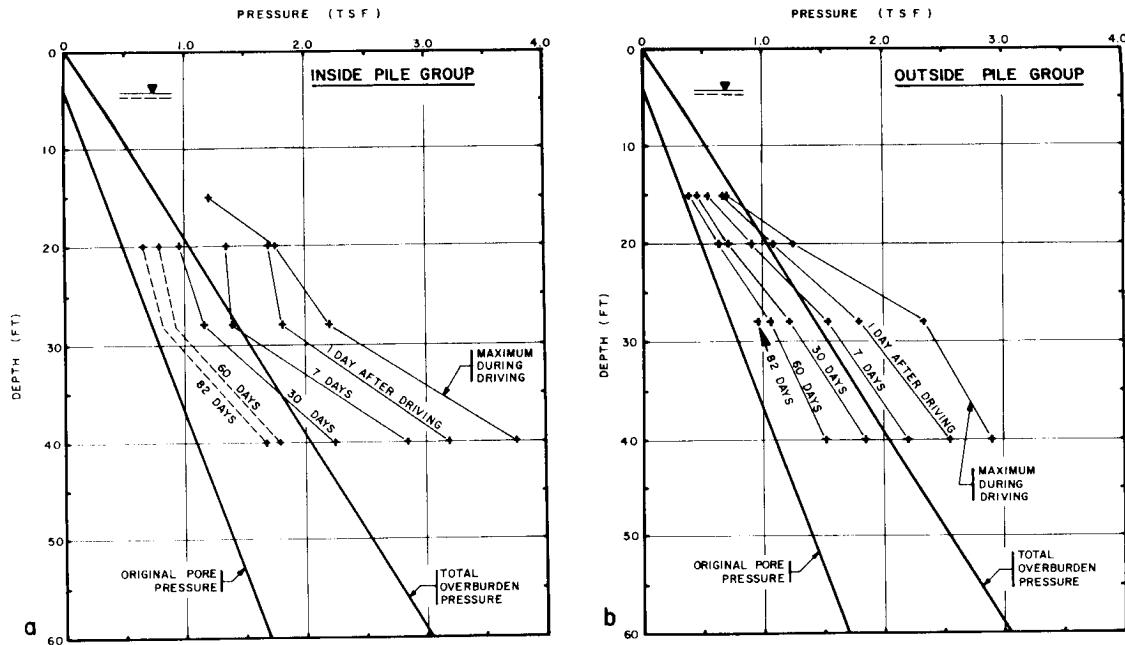


FIG. 10. Vertical distribution of original pore pressure, total overburden pressure and measured pore pressures (a) inside the pile group, and (b) outside the pile group.

TABLE 1. Heave measurements in ft (mm)

Date	Heave gauge			Note
	S-1	S-2	S-3	
Feb. 27	0.00	0.00	0.00	Before start of pile driving (Pile No. 1 and 2 to 70 ft or 20 m).
Feb. 28	0.07 (21)	0.01 (3)	0.02 (6)	Pile No. 1 and 2 completed.
March 1	0.16 (49)	0.07 (21)	0.07 (21)	Piles No. 3, 4, 5, and 6 driven.
March 4	0.15 (46)	0.07 (21)	0.06 (18)	Morning: no driving March 2nd and 3rd (weekend).
March 4	0.15 (46)	0.08 (24)	0.07 (20)	Evening: retapping of pile No. 1 to 6.
March 5	0.21 (64)	0.12 (37)	0.11 (34)	Pile No. 7, 8, 9, 10, and 11 driven and retapped.
March 6	0.23 (70)	0.14 (42)	0.11 (34)	Pile No. 12 and 13 driven and retapped.
March 28	0.21 (64)	0.13 (40)	0.12 (37)	22 days after pile driving.
April 11	0.22 (67)	0.13 (40)	0.12 (37)	36 days after pile driving.

the disturbance and lateral squeezing of the clay caused by the pile driving.

The fall cone strength tests and consistency limits on clay samples obtained within the pile group show no measurable effect originating from the pile driving, despite the high induced pore pressures and distortion of the clay.

The comparison between eight oedometer tests performed on samples obtained prior to and 75 days after pile driving, shows that the clay has reconsolidated to a preconsolidation value exceeding the effective overburden stress by 0.7 to 1.1 tsf (70 to 110 kPa) with an average value of 0.9 tsf (90 kPa) at the tested

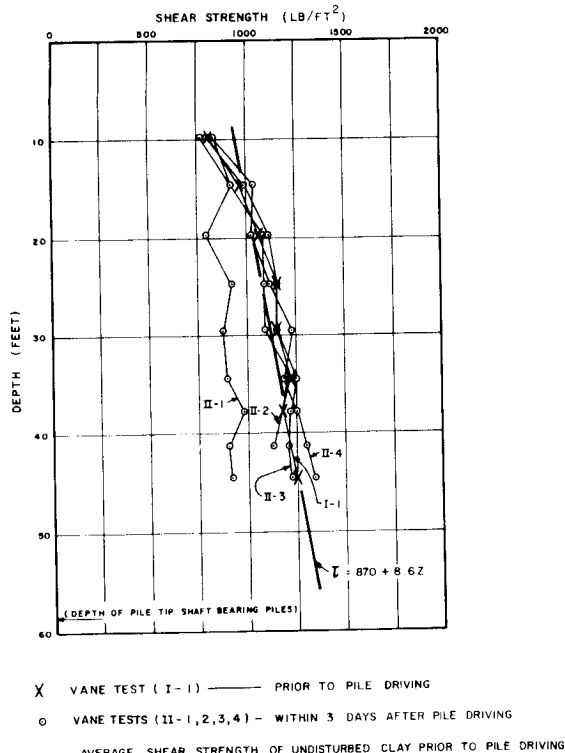


FIG. 11. Results of vane tests. Original shear strength compared with results obtained within 3 days after the end of the pile driving.

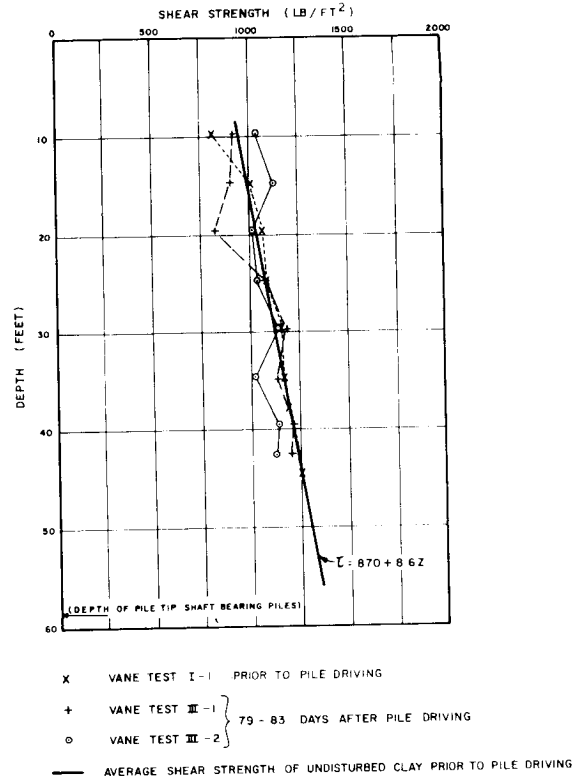


FIG. 12. Results of vane tests. Original shear strength compared with results obtained 79 to 83 days after the end of pile driving.

depths. The new overconsolidation value is smaller than the original value of the clay of about 1.3 tsf (130 kPa). As indicated by the pore pressure measurements, the reconsolidation of the soil was not quite finished at the time of sampling. Thus, it is plausible that with time the preconsolidation value will increase and may reach its original value. The compressibility of the clay is practically unaffected both on the recompression branch and the virgin branch of the consolidation curve. A typical example of oedometer test results is shown in Fig. 14.

**Driving of the Foundation Piles**

The results of the investigation showed that the displacement type Herkules concrete piles were suitable for the subsurface conditions prevailing at the Sidbec site and technically comparable with the more expensive steel piles. However, to minimize the risk of disturbing adjacent existing foundations, it was specified that piles installed within 20 ft (6 m) distance

of existing foundations be prebored through the upper 30 ft (9 m) of the soil. The minimum center to center spacing of the piles was taken at 4 pile diameters.

To acquire sufficient bearing capacity, it was specified that the piles be driven to the competent glacial till layer. The allowable bearing capacity of the 12 in. (30 cm) Herkules H800 piles was taken at 120 tons (1.1 MN) with 135 tons (1.2 MN) allowed in combination with exceptional transient loads. The allowable load for the smaller standard pile, the 8.5 in. (22 cm) Herkules H420, was taken at 75 tons (0.7 MN). The allowable uplift capacity was 25 tons (0.22 MN) and 20 tons (0.18 MN), respectively, and the allowable lateral load, 4 tons (35 kN) and 2 tons (18 kN), respectively.

To penetrate the very dense silt and seat the piles into the glacial till, a very hard driving was necessary. The specifications called for a refusal requirement of two consecutive series of

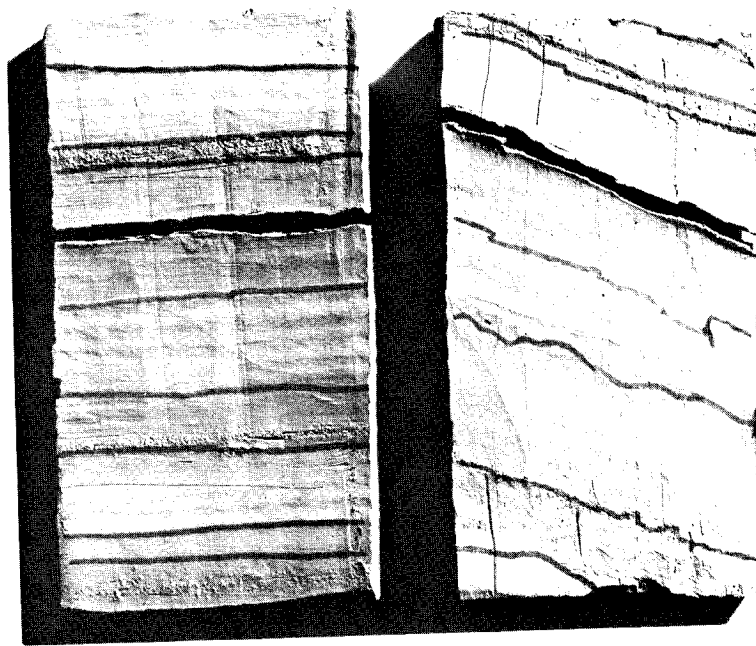


FIG. 13. Photo of samples obtained from a depth of 36 ft (11 m) prior to pile driving (left) and after pile driving (right). The clay samples are photographed when air dried and the limits between the different bands and layers have been accentuated by a marker.

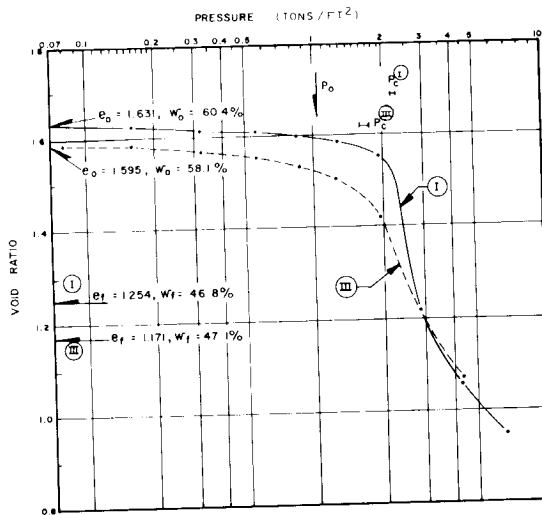


FIG. 14. Results from oedometer tests on clay samples from 36 ft (11 m) depth performed on samples obtained (I) before driving and (III) 75 days after pile driving.  $P_c$ : preconsolidation pressure,  $P_0$ : initial effective vertical pressure.

50 blows with a maximum penetration of 2.0 in. (50 mm) each, followed by a third series of 50 blows with a maximum penetration of 1.5 in. (38 mm). In addition, retapping by one

series of 50 blows with a maximum penetration of 1.5 in. (38 mm) was to be performed when neighbouring piles have been driven. The driving was performed by a diesel hammer type Link-Belt 520 at full driving pressure.

The hard driving necessitated a very high quality of piles and pile components, such as splices and rock shoes. The quality used in the investigation was specified for the foundation.

In the summer of 1974, the piling for the Sidbec plant extension started in two areas. Near the test site, 148 piles were driven and about 1500 ft (450 m) away from the test site an additional 372 piles were driven. The piles met refusal in the glacial till within 3 ft (1 m) length variation as compared to adjacent piles. The penetration during the last series of 50 blows was generally 1.0 in. (25 mm) or smaller. The retapping series showed generally a penetration smaller than obtained for the last driving series and several piles gave zero penetration in retapping. However, for one group of about 50 piles, one-third of the piles showed a surprising difference and moved several inches for a few blows in retapping, before reaching a new refusal. This effect is

believed to be caused by these piles stopping on boulders in the glacial till and the subsequent driving of adjacent piles dislocated the boulders at the pile tip of the first piles. In retapping these piles, they were brought to renewed contact with the boulders. The occurrence proved the retapping schedule to be well worth undertaking.

Only seven of the 520 piles broke during driving. No single direct cause of the pile breakage could be established until a careful review of the inspection reports revealed that 23 pile segments out of about 1500 segments used for the piles had a concrete strength, as determined by cylinder testing, of 1000 to 1500 psi (7 to 11 MPa) lower than the specified value of 7500 psi (52 MPa). All seven piles which broke had one or more segments of the inferior concrete strength. This observation confirms the necessity for the specified high concrete quality for the piles for the soil conditions at the site.

### Summary and Conclusions

#### *Pile driving and pile quality*

The pile driving during the testing program was hard and lengthy. The driving time with the diesel hammer was found to be about half the time needed when using the drop hammer, mainly due to the higher blow rate the diesel hammer provided; 80 blows/min as compared with 35 blows/min for the drop hammer.

Each pile received a total number of blows ranging from 2100 to 3900 blows to reach end bearing in the glacial till, with the lesser number required for the diesel hammer. In driving through the very dense silt, the diesel hammer required an average of about 170 blows/ft and the drop hammer an average of 250 blows/ft. Both hammers produced dynamic peak stresses in the pile in the range of 2000 to 4000 psi (14 to 28 MPa).

Visual observations during driving and calculations performed on the driving records indicate that the high pile quality was necessary to resist the hard driving needed to reach the glacial till stratum in which the full structural bearing capacity of the pile member can reliably be mobilized. The necessity for high concrete quality is confirmed by the driving performance of the first 520 foundation piles at the site.

A lesser pile quality would result in severe

cracking of the piles with loss of driving energy and thus a so-called false refusal. Ultimately, piles would break during driving. To avoid this, a lighter driving would be required. However, the large driving resistance, ranging from 150 to 300 blows/ft to penetrate the very dense silt and develop end bearing in the glacial till, does not permit such reduction in the present case.

The pile tests show the piles to have an ultimate bearing capacity of at least four times the current allowable pile load. A lighter refusal criterion would thus seem possible. However, in the present case, the resistance to penetrate the very dense silt is about 3 in./50 blows (75 mm/50 blows) and the refusal set in the glacial till must be distinguishable from the driving resistance in the silt. Therefore, only a minor slackening of the refusal criterion is possible, as compared to the driving resistance used for the test piles. By specifying for the foundation piles, two consecutive sets of 2.0 in. (50 mm) maximum penetration per 50 blows, followed by a third series of 1.5 in. (38 mm) maximum penetration, complete assurance is achieved that the pile tip is well seated into the competent glacial till.

#### *Effect on soil from pile driving*

The current literature reports that large excess pore pressures develop when driving piles in clay. Most of these pore pressures dissipate within a few months after the pile driving. However, the dissipation could result in settlements of existing shallow foundations. The literature also confirms the generally accepted value of ground heave within a pile group corresponding to about 50% of the soil volume displaced by driven piles. The reported disturbance of the clay between the piles in terms of reduction of undrained shear strength is on the order of 10 to 20%. This small reduction does not significantly affect the design of adjacent shallow foundations or the lateral capacity of the piles. However, in a few papers, conflicting views are expressed as to the possible detrimental effect of driving displacement piles in sensitive clay, particularly with reference to ground heave, lateral displacement, and settlements during dissipation of the induced pore pressures.

In the present investigation, very high pore pressures were generated during pile driving,

which exceeded the effective overburden stress by a factor of 2 within a distance of 6 pile diameters outside the pile group. These pore pressures dissipated in 3 to 5 months time after the pile driving. The ground surface heaved 0.23 ft (70 mm) inside the pile group and 0.12 ft (30 mm) at a distance of 8 pile diameters outside the pile group. However, the heave measurements are limited and the pile group is small, which limits the conclusions that can be drawn from the measurements. In terms of undrained shear strength, the disturbance of the pile driving was limited to the clay within the pile group, where a reduction of 15% was measured immediately after the pile driving. This strength reduction was regained during the dissipation of induced pore pressures. Oedometer tests performed 75 days after the pile driving showed a small reduction of the overconsolidation value in the clay, which reduced from about 1.3 to about 0.9 tsf (130 to 90 kPa).

The present investigation on the effect of the driving of displacement piles shows that the resulting disturbance in terms of changes of the soil properties is limited to the zone within the pile group and is partially restored following the dissipation of induced excess pore pressures. Thus, the driving of displacement piles at the site of the investigation does not prevent footings or other shallow foundations from being placed near the pile foundations, provided that there is a time lag of 3 to 5 months between pile driving and applying the building load on adjacent shallow foundations, to allow for the full dissipation of excess pore pressures.

Piles driven close to existing shallow foundations or floors on grade could cause unacceptable heaving. However, footings at a distance of 20 ft (6 m) or more from a pile group would be little affected. For piles driven closer than 20 ft (6 m) from an existing footing or floor on grade, the displacement of the clay must be prevented by special measures, such as preboring through the upper 30 ft (9 m) of the soil.

The reconsolidation of the clay after the pile driving will cause a drag load on the piles due to negative skin friction. This drag load is gradually applied to the pile during the reconsolidation period of the clay which in the present case is about 3 months. Therefore, the

drag load will be obtained before the full working load is applied on the piles and it will only have the beneficial effect of preloading the pile. It will be eliminated as the working load is applied to the piles and compresses the pile elastically. Thus, measures to eliminate or reduce negative skin friction caused by the reconsolidation of the clay after driving is not necessary, nor even desirable in this case.

#### *Summary*

In summary, the investigation has shown that:

1. It is possible to drive the piles through the very dense silt and into the glacial till, where the full structural bearing capacity of the piles can reliably be mobilized.

2. The wave equation prediction of ultimate pile bearing capacity is approximately correct in the silt layer, where driving resistance is about 15 to 20 blows/in. However, the wave equation analysis grossly underestimates the ultimate pile bearing capacity in the glacial till, where the driving resistance is 50 to 100 blows/in.

3. The pile loading tests show the ultimate bearing capacity of the 12 in. (30 cm) Herkules H800 pile to exceed 450 tons (4 MN). The ultimate shaft capacity of the pile in the clay and underlying silt and sand is about 200 tons (1.5 MN), and the maximum applied pile tip load in the test is within a range of 190 to 230 tons (1.7 to 2.0 MN), which, however, is less than the ultimate value.

4. At the site, the pile shaft resistance in compression in the clay is about 50 to 100% greater than the shaft resistance in uplift.

5. The ground heave within the group of 13 piles is 0.23 ft (70 mm) and influences the area around the pile group to a distance of at least 8 ft (2.5 m), and probably 20 ft (6 m).

6. Large excess pore pressures develop inside and around the pile group. The induced pore pressures exceed by a factor of 2 the initial effective overburden stress in the soil. Most of these pore pressures dissipate over a period of 3 months, and after 5 months no excess pore pressures remain.

7. The pile driving causes a reduction of about 15% of the undrained shear strength of the clay inside the pile group immediately after the pile driving. However, no shear strength

reduction could be observed outside the pile group even at a distance as close as 2.3 ft (0.7 m) from the nearest pile. Vane testing 80 days after the driving, when most of the excess pore pressures had dissipated, shows that a regain of the shear strength had occurred within the pile group.

8. Samples of the clay obtained from within the pile group show distinct visual signs of distortion.

9. No disturbance caused by the pile driving can be found, when comparing laboratory tests on clay samples obtained before pile driving and 75 days after pile driving in terms of water content, Atterberg limits, and shear strength with the Swedish fall cone. Oedometer tests show that a small reduction of the preconsolidation pressure is measurable on clay samples obtained 75 days after the driving.

10. The stringent quality requirements and driving inspection specified for the piling work proved to be needed insofar as the only piles (seven piles out of 520 piles) which broke during driving, happened to be made with a concrete strength less than specified, and that the retapping schedule disclosed that in one part of the site, driving of adjacent piles dislocated the end-support of previously driven piles.

The investigation presented in this paper is limited to a relatively small group of 13 piles, the lateral soil displacements and vertical distribution of ground settlements were not investigated, and the number of measuring instruments was limited. At present, a more extensive research program is under way at the site involving two adjacent groups of 115 piles each, and containing a greater number of monitoring instruments including measurements of lateral soil displacement, settlements, and several other factors not included in the investigation reported in this paper.

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