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Soil disturbance from pile driving in sensitive clay[1]

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Soil disturbance due to the driving of two groups of 116 concrete piles each in sensitive marine clay was studied on a construction project in eastern Canada. Pore-water pressures, heave, and lateral movement of soil and piles, and tests of strength, compressibility, and consistency limits of the soil were observed prior to and up to 3 months after pile driving whereas observations of pore-water pressures were continued for an additional 5 months. Driving of the piles had little effect on the compressibility and consistency limits of the marine clay, but the *in situ* shear strength and cone penetration resistance were reduced by about 15 and 30%, respectively. Soil heave within the group of piles decreased linearly with depth from a maximum of 450 mm (18 in.) at the ground surface to about zero at the pile tips, and in volume amounted to approximately 55% of the soil displaced by the piles. The vertical heave outside the pile group was confined to a horizontal distance of 12 m (39 pile diameters). During pile driving, the lateral movement of previously driven piles was as much as 175 mm (7 in.). Horizontal soil movements measured by inclinometers varied up to 125 mm (5 in.). Pore-water pressures generated during piling exceeded the total overburden pressure by 35–40%. The excess pore pressures dissipated in about 8 months after the piling was completed.

Le remaniement causé par le fonçage dans l'argile marine sensible de deux groupes de 116 pieux de béton a été étudié sur un chantier de construction de l'Est du Canada. Les pressions interstitielles, de soulèvement et le mouvement latéral du sol et des pieux, de même que les valeurs de la résistance, de la compressibilité et des limites de consistance du sol ont été observés avant et jusqu'à 3 mois après le fonçage des pieux; de plus, les observations des pressions interstitielles se sont poursuivies durant 5 mois additionnels. Le fonçage des pieux n'a eu que peu d'effet sur la compressibilité et les limites de consistance de l'argile marine, mais la résistance au cisaillement en place et la résistance au cône ont été réduites d'environ 15 et 30% respectivement. Le soulèvement du sol à l'intérieur du groupe de pieux diminuait linéairement en fonction de la profondeur, soit d'un maximum de 450mm (15 po.) à la surface du sol jusqu'à environ zéro à la pointe des pieux; en volume, ce soulèvement correspondait à approximativement 55% du sol déplacé par les pieux. Le soulèvement vertical en dehors du groupe de pieux se limitait à une distance horizontale de 12 m, soit 39 diamètres de pieux. Durant le fonçage, le mouvement latéral des pieux foncés préalablement atteignait 175 mm (7 po.). Les mouvements horizontaux du sol mesurés par des inclinomètres ont varié jusqu'à 125 mm (5 po.). Les pressions interstitielles générées durant le fonçage ont dépassé de 35 à 40% les valeurs des pressions isostatiques. La dissipation de l'excédent des pressions interstitielles s'est terminée environ 8 mois après la fin du fonçage.

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Introduction

Concrete or other types of piles with high soil displacements have not been used extensively to support heavy loads in sensitive marine clay soils because of the potential disturbance to the subsoils and the possible danger to adjacent structures. Fellenius and Samson (1976) reported on a pilot study conducted in 1974 on a group of thirteen 30-cm (12-in.) precast concrete piles driven through a sensitive marine clay which showed that the subsoils were not seriously disturbed and that concrete piles could be safely used. In 1976, the study was extended to two large groups of 116 precast concrete piles each to be driven 270 m (900 ft) from the original test site.

The objectives of the study were to: (a) evaluate the disturbance of the sensitive marine clay by measuring changes in shear strength, compressibility, and pore-water pressures in the soil, and to measure the extent to which the original conditions are recovered with time after pile driving; (b) monitor the vertical and lateral movements induced on installed piles from driving other piles within the same group; and (c) determine the zone influenced by pile driving by measuring the vertical

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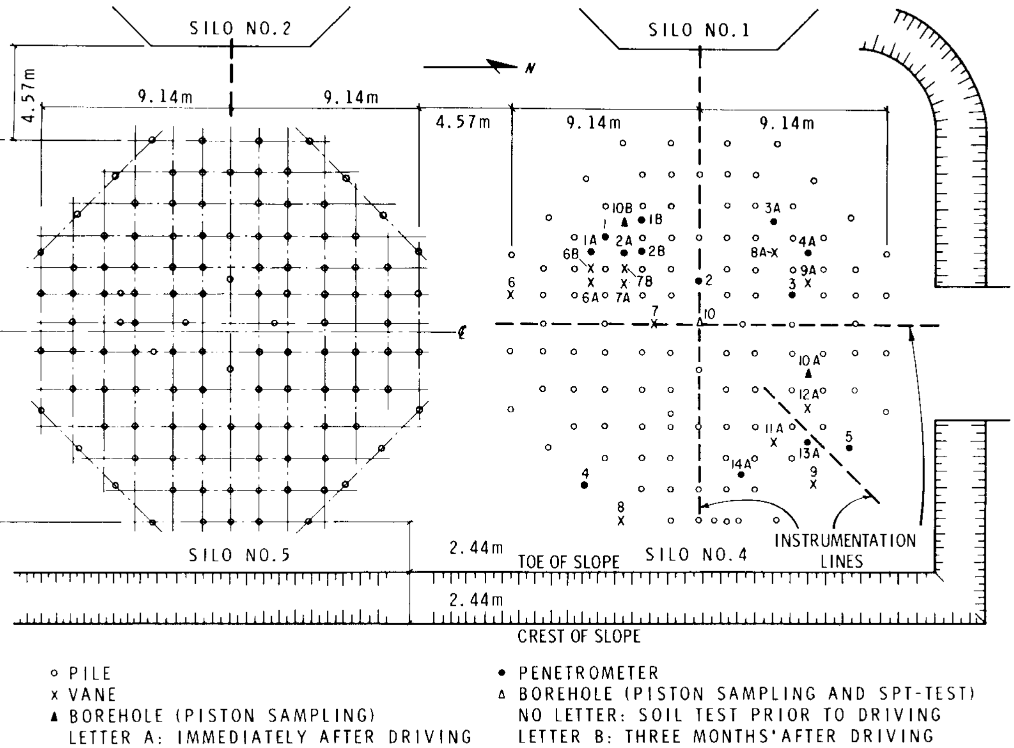

FIG. 1. Location of piles, soil borings, *in situ* tests, and instrumentation lines.

and horizontal soil movements and the changes in pore-water pressures.

The influence of pile driving on shear strength, water content, consistency limits, and compressibility of the soil was investigated by laboratory tests on piston-tube soil samples obtained prior to piling, immediately after, and 3 months after the piles were driven. Shear strength was measured using the Swedish fall cone and compressibility using standard odometer tests. The changes in shear strength were also measured *in situ* using a field vane and static cone penetrometer.

The zone of influence due to piling was established by measuring soil heave at the surface with spiral-foot settlement gauges and at depth with bellow-hose gauges. Horizontal soil movements were measured with inclinometers installed vertically in the ground; excess pore pressures were measured with hydraulic and vibrating wire piezometers.

The effects of pile driving on installed piles were monitored using terrestrial photogrammetry techniques.

Site Conditions

The construction site was located on the SIDBEC-DOSCO property in Contrecoeur, Quebec, on a marine clay plain east of Montreal and south of the St. Lawrence River. The pile project was adjacent to three existing silos supported on end-bearing steel piles. In preparation for piling the site was excavated to a depth of 2.9 m (9.5 ft) from a surface elevation of +18.2 m (+59.7 ft). Figure 1 shows the locations of the piles, the soil borings, and tests carried out at the site. A compilation of the engineering tests conducted on the soils in the laboratory and in the field prior to pile driving is shown on Figs. 2 and 3.

At the original ground surface, there was 1 m (3 ft) of sandy backfill overlying 1.9 m (6 ft) of brown silty clay. When the site was excavated, this material was replaced with 0.6 m (2 ft) of sand and gravel to provide a working surface for the field investigation and pile driving.

Underlying the gravel pad to a depth of 2.3 m (7.5 ft) was a highly plastic grey silty clay stratified with uniform silt. The water content was close to the liquid limit of about 55%, the sensitivity varied from 15 to 20, and the undrained shear strength measured in the laboratory and with the field vane was about 50 kPa (1000 psf).

From 2.3 m (7.5 ft) to 15.1 m (49.5 ft) the soil consisted of grey silty clay with dark and pale bands, stratified with thin seams of uniform silt and sand. The thickness of the bands varied from 2 mm (0.1 in.) to 70 mm (3 in.). The soil was

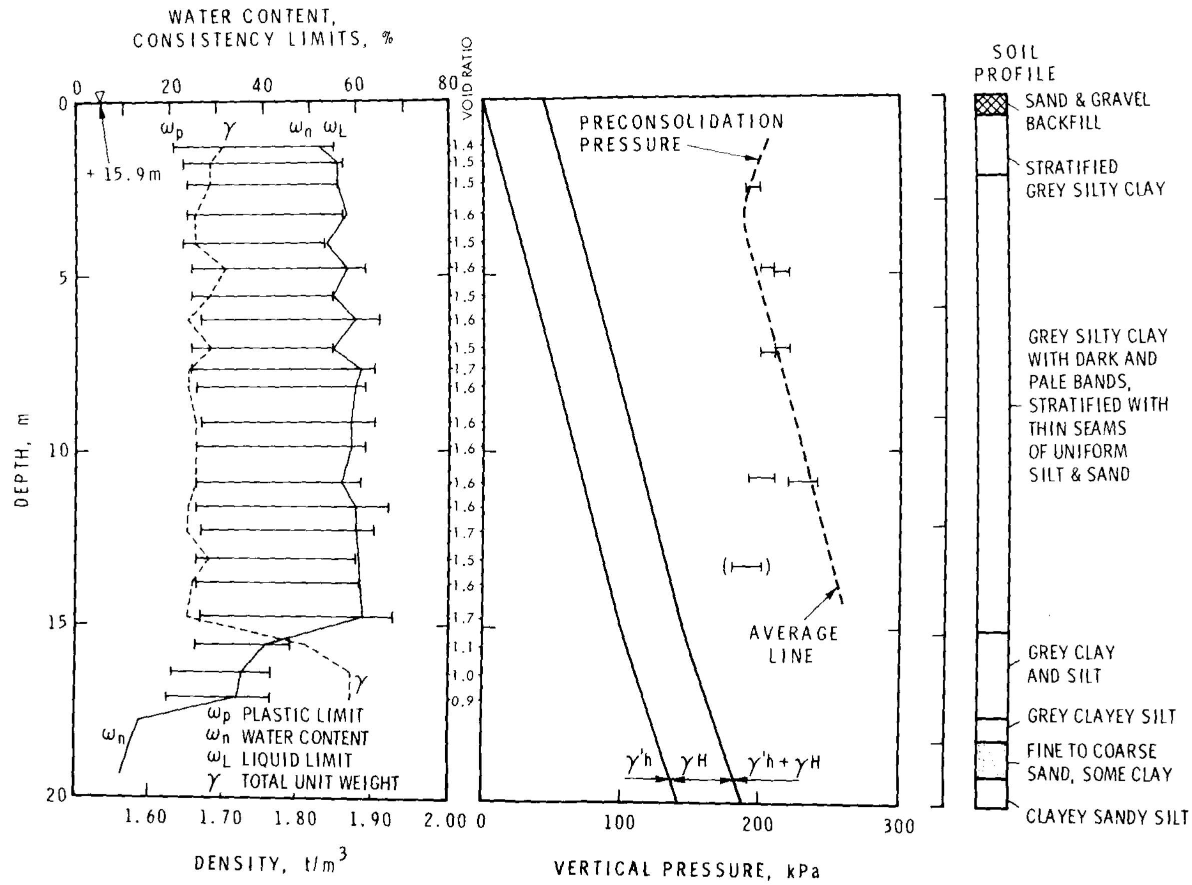

FIG. 2. Variation of consistency limits and preconsolidation pressure with depth.

highly plastic with a natural water content of 60%, which was close to the liquid limit. The sensitivity varied from 15 to 20. The *in situ* undrained shear strength measured with the vane increased linearly with depth through the soil formation from 50 kPa (1000 psf) to about 75 kPa (1600 psf). Consolidation tests showed that it had a void ratio of about 1.6, and was overconsolidated by about 120 kPa (1.3 tsf).

From 15.1 m (49.5 ft) to 17.5 m (57.4 ft) the soil consisted of a grey clay and silt with pale and dark bands in which the natural water content decreased gradually from 40 to 35%. The liquid limits were about 6% greater than the water contents. The measured shear strengths varied considerably. Underlying this formation was 0.6 m (2 ft) of grey clay and silt with a water content of 14%, followed by 1.0 m (3 ft) of fine to coarse sand with some clay.

From 19.1 m (62.6 ft) to 24.4 m (80.0 ft), the soil changed to grey sandy silt with traces of gravel. *N*-values of 15, measured by the standard penetration test (SPT), indicated a medium relative density.

Between 24.4 m (80.0 ft) and 30.5 m (100 ft), which was the maximum depth of sampling, a very dense glacial till with boulders was encountered. This formation was selected to support the end-bearing concrete piles.

The Foundations

Each group of 116 (consisting of the designed 113 plus 3 replacements) standard Herkules H800 precast concrete piles (Fellenius and Samson 1976) supported a massive reinforced concrete octagon-shaped foundation, as shown in Fig. 1, upon which was placed the iron ore storage silos 4 and 5. The hexagon-shaped piles had a diameter of 30 cm (12 in.), a cross-sectional area of 800 cm^2 (124 in.^{2}) and an average length of about 26 m (85 ft). The pile spacing on centers was generally 5–6 diameters with a minimum spacing between any two piles of 4.3 diameters. They were driven to refusal in the glacial till to depths of 25–27 m (82–89 ft). The diameter, circumference, and area of the pile groups were 19 m (62 ft), 62 m (200 ft), and 290 m^2 (3100 ft^2), respectively.

Pile driving started on July 30, 1975 and was

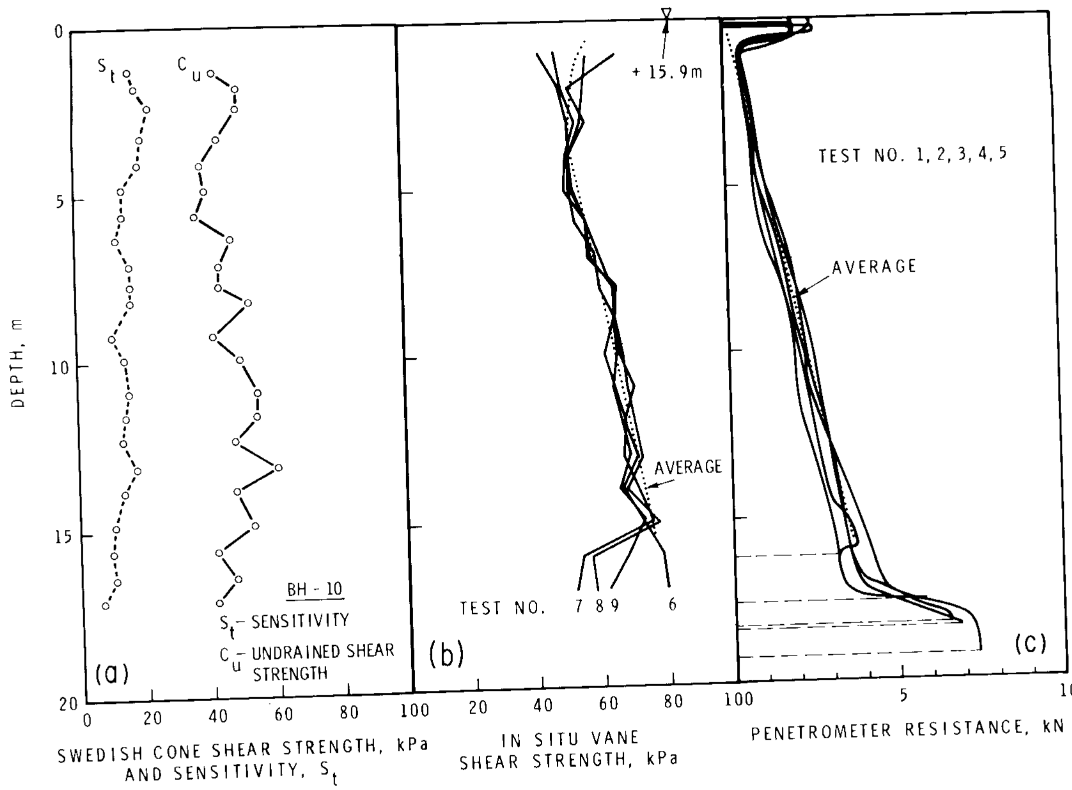

FIG. 3. Variation in sensitivity, Swedish cone undrained strength, *in situ* vane shear strength, and cone penetrometer resistance with depth before pile driving.

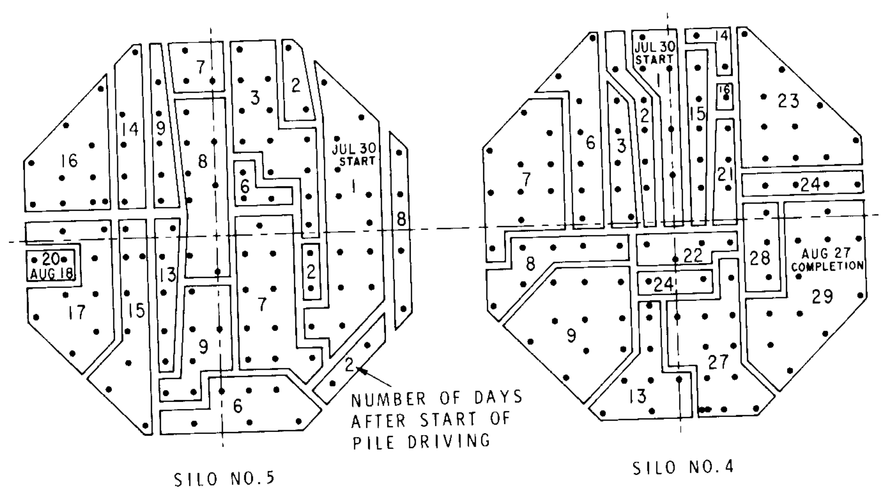

FIG. 4. Sequence of pile driving.

completed 29 days later, on August 27. The sequence of pile driving and the number of piles driven each day after the start of pile driving is shown on Fig. 4.

Sampling and Field Testing

Soil investigations at the test site began on June 24, 1975, about 5 weeks before the start of piling. Continuous soil sampling for laboratory testing was carried out to a depth of 18.1 m (59.5 ft) at BH-10 (Fig. 1) with an NGI fixed piston sampler (Bjerrum 1954), modified to take 71 mm (2.8 in.) diameter cores. The sample tubes had a wall-to-area ratio of 11%. Split spoon sampling and standard penetration tests were carried out to depths from 18.3 to 30.5 m (60 to 100 ft). Soil sampling was repeated immediately after pile driving at location BH-10A to a depth of 14.8 m

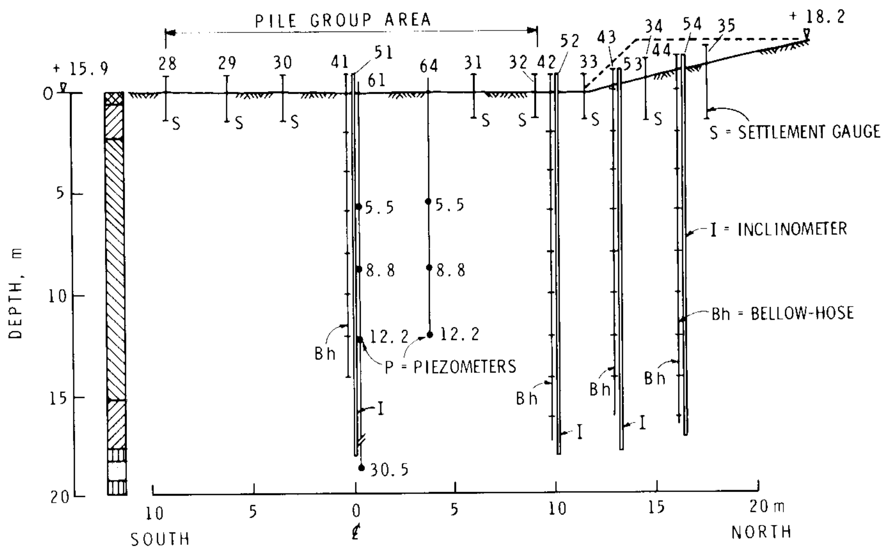

FIG. 5. Location of instrumentation along the north–south line, silo 4.

(48.6 ft) and 3 months later at BH-10B to a depth of 14.0 m (45.9 ft). Samples from the latter borings were taken at 1-m (3.3-ft) intervals of depth midway between the piles. Soil samples not used for engineering tests were photographed in natural, semi-dry, and dry conditions to obtain visual evidence of soil disturbance due to pile driving.

Continuous static cone penetration tests were performed at locations 1 to 5 (Fig. 1). A Nilcon cone, with a diameter of 35 mm (1.38 in.), an area of 10 cm^2 (1.56 $in.^2$), and an apex angle of 45° (Sanglerat 1972) was used for the tests. This equipment permitted the separation of the end cone resistance from the total shaft resistance. The tests were repeated immediately after pile driving at locations 1A, 2A, 3A, 4A, 13A, and 14A, and again 3 months later at locations 1B and 2B. These tests were conducted to depths varying from 14.2 m (46.6 ft) to 20.2 m (66.3 ft) midway between the piles.

The undrained shear strength was measured *in situ* using a Nilcon vane with dimensions of 65 × 130 mm (2.56 × 5.12 in.). The initial tests were performed at locations 6 to 9 (Fig. 1), with tests performed every 1 m (3.3 ft) to a depth of 17 m (56 ft). The vane shear tests were repeated at locations 6A, 7A, 8A, 9A, 11A, and 12A immediately after pile driving, and 3 months later at 6B and 7B. These latter tests were also performed at intervals of 1 m (3.3. ft) to depths varying from 9 m (29.5 ft) to 17 m (55.8 ft) midway between the piles.

Field Instrumentation

Field instrumentation consisting of surface heave and bellow-hose gauges, inclinometers, and piezometers was installed along the instrumentation lines running north to south, east to west, and in the northeast quadrant of silo 4 (Fig. 1) to measure the influence of piling originally scheduled to be driven in progression from south to north and west to east in the excavation. Some surface heave gauges were installed along a line west of silo 5. The offset distances and depths of most of the instrumentation are shown in Figs. 5 and 6.

Sixteen surface heave gauges were installed at the locations numbered 21 to 35 and 37 (Figs. 5 and 6). Gauge 36 was located 46 m (150 ft) north of the piles. They consisted of bronze spiral-foot gauges (Bozozuk 1968), installed at elevation +14.5 m (+47.6 ft), which was about 1.5 m (5 ft) below the floor of the excavation. These instruments were protected by a 7.6-cm (3-in.) diameter steel casing installed to a depth of 0.5 m (2 ft) and grouted to the soil for lateral support. Vertical movement of the spiral-foot gauges was measured by level surveys to the projecting inner rods.

In addition to the surface heave gauges, seven bellow-hose gauges specially constructed for the project were installed at the locations numbered 41 to 47 (Figs. 5 and 6), to a depth of about 18 m (60 ft). They consisted of spirally-reinforced axially-flexible plastic hoses fitted internally with copper rings spaced at about 2 m (6 ft). They were installed in a predrilled hole and grouted in place in such a way that the hose could move

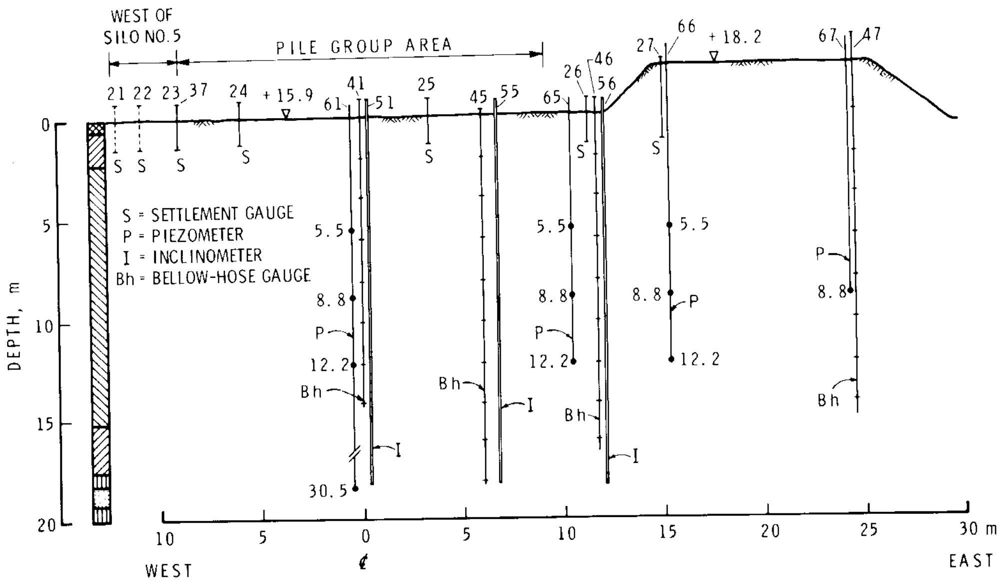

FIG. 6. Location of instrumentation along the east–west line, silo 4.

freely in a vertical direction along with the surrounding soil. Soil movements were measured by an electric probe that gave the location of each ring with respect to the top of the casing, which in turn was surveyed relative to a bench mark. A detailed description of this gauge is given by Bozozuk and Fellenius (to be published).

Six inclinometer tubes were installed to depths of 19 m (62 ft) to measure horizontal soil movements at the locations numbered 51 to 56 (Figs. 5 and 6). The tubes were semi-rigid polyvinyl chloride (PVC) with an inside diameter of 38 mm (1.5 in.). A Swedish Geotechnical Institute inclinometer probe (Fellenius 1972) was used to measure the change in inclination of the tubes in the east to west and north to south directions at every 0.33 m (1.08 ft) of depth.

Eleven hydraulic piezometers were installed at locations 61 to 63. These were Geonor porous bronze tips connected with the ground surface with plastic tubing encased in steel E-drill rods. Four piezometers were installed within the pile group to depths 5.5 m (18 ft), 8.8 m (29 ft), 12.2 m (40 ft), and 30.5 m (100 ft) at location 61 (Figs. 5 and 6); four reference piezometers (location 63) were installed 46 m (150 ft) north of the pile group at the same elevations; and three (location 62) were installed in the northeast quadrant 0.6 m) (2 ft) from the pile group at depths of 5.5 m (18 ft), 8.8 m (29 ft), and 12.2 m (40 ft). Measurements were made by sounding with an electric probe until the water columns rose above the ground surface. Bourdon pressure gauges were then attached to the piezometers to record the artesian pressures.

Eleven Geonor vibrating wire piezometers were installed at five locations (64–68). At locations 64–66 (Figs. 5 and 6), they were installed at depths 5.5 m (18 ft), 8.8 m (29 ft), and 12.2 m (40 ft). One was installed at location 67 (Fig. 6) at a depth of 8.8 m (29 ft) and another (at location 68) at the same depth in the northeast quadrant, 3.3 m (10 ft) from the edge of the pile group.

Two bench marks consisting of steel bolts placed in the concrete foundations of silos 1 and 2 served as reference elevations for all level surveys.

Terrestrial photogrammetry, a technique which permits dimensional measurements from stereo pairs of photographs, was used to monitor the vertical and lateral movements induced on installed piles by driving additional piles within the same group. Two steel scaffolds were erected 10 m (32.8 ft) apart, 14 m (46 ft) east of the first row of piles at silo 4, to support the cameras (Wild, P-31) 3.7 m (12 ft) above the original ground surface. This height was required in order to photograph as many piles as possible because they tended to obscure one another as they projected over 5 m (16 ft) above the floor of the excavation. Control points for the photographs consisted of bolts, well defined joints, and white crosses painted on silo 1 (Fig. 7). Targets for the measurements consisted of white crosses painted on each pile

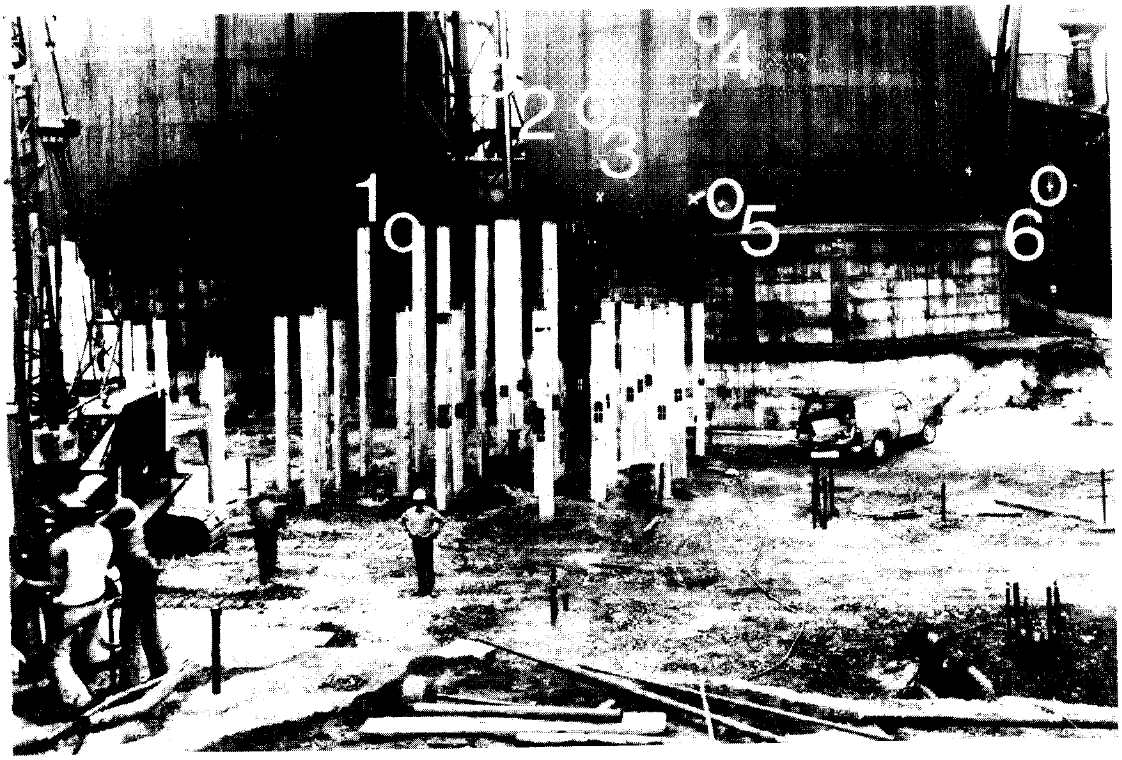

FIG. 7. Control points (numbers 1 to 6) and targets (crosses) for terrestrial photogrammetry measurements. (Photo by M. C. van Wijk, Division of Physics, NRCC.)

after it was driven. Details of this technique are given by Massarsch (1975) and Bozozuk *et al.* (to be published).

Effect of Piling on Soil Properties

Consistency Limits

Engineering tests for consistency limits on soil samples obtained immediately after pile driving and 3 months later were compared with the initial test results. The comparisons showed that the pile driving had no effect on water content, consistency limits, and density of the soil. Furthermore, an examination of photographs of thin vertical sections of the soil (at natural water content, semi-dry, and dry conditions) sampled after the piles were driven showed no evidence of physical distortion. All the soil layers were distinct and horizontal, as they were before any piles had been driven. It would appear, therefore, that the only effect of the pile driving was to physically displace the soil.

Preconsolidation Pressures

The consolidation test is a sensitive indicator of soil disturbance. Figure 8*a* shows a comparison of preconsolidation pressures measured on soil samples obtained after piling with the initial test values. The same loading schedules were carefully maintained. As can be seen in the figure there was no difference in the results and no significant differences in the compression index C_c. There was, however, a slight difference in recompression index C_r; it changed from 0.03 to 0.04 after pile driving.

It would appear that driving displacement-type piles at a center-to-center spacing of 5 to 6 pile diameters causes little or no disturbance of this sensitive marine clay midway between the piles. It is possible that the disturbance created by the release of confining pressures when soil samples are removed from the ground by tube sampling, no matter how carefully done, is far more severe than that caused by displacing the soil from driving piles. Consequently minor disturbances from the pile driving would be masked by that resulting from soil sampling.

Swedish Fall Cone Shear Strength

The results of the shear strength tests performed on piston-tube samples obtained before pile driving (BH-10), at the conclusion of pile driving (BH-10A), and 3 months after the piles were driven (BH-10B) are plotted on Fig. 8*b*. Although there was a fair amount of scatter in the plotted points, a small reduction in strength due

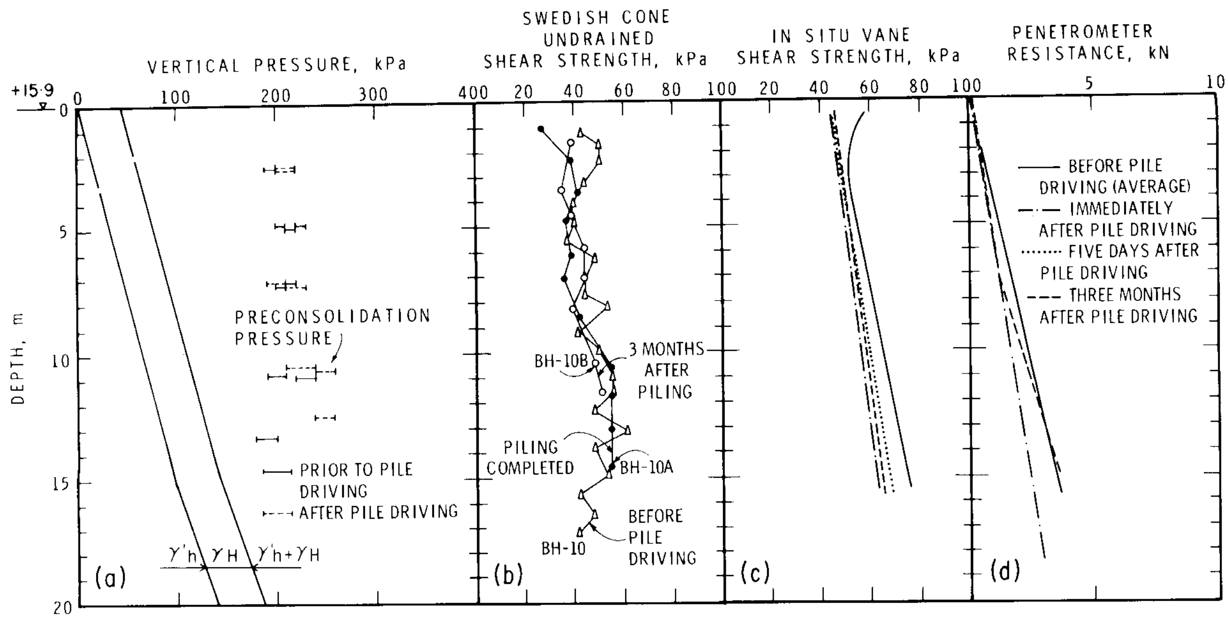

FIG. 8. Effect of piling on preconsolidation pressure, Swedish cone and *in situ* vane shear strength, and cone penetrometer resistance.

to pile driving can be seen. This reduction was more pronounced within the upper 3 m (10 ft), but was not very significant below that depth.

In Situ Vane Shear Strength

A series of four vane test borings was made: four before pile driving, four immediately (within 24 h) after driving, two 5 days after, and two 3 months after the piles were driven. The results from each series were averaged and are presented in Fig. 8*c*. Immediately after pile driving, there was an overall reduction in shear strength of about 15%, although it was generally greater in the upper 2 m (6 ft). There was a small regain in shear strength 5 days after the piles were driven, but 3 months later it was intermediate between the measurements made immediately and those made 5 days after pile driving. Considering that the vane borings were scattered around the site (Fig. 1), the variations in the measured strengths are acceptable. It appeared, therefore, that the piling reduced the *in situ* shear strength by about 15%, and that this reduction was not regained within the following 3 months.

Static Cone End Resistance

A series of three penetrometer soundings was carried out: five before pile driving, six immediately (within 24 h) after driving, and two 3 months later. The averaged results from each series (Fig. 8*d*) show that the pile driving caused a reduction in cone resistance of about 30%, which was only partially regained in the following 3 months.

Displacement of Installed Piles Due to Pile Driving

Stereo pairs of photographs were taken once a week during the pile driving operations to record any changes in the position of piles installed during the interval. In many cases the piles in the foreground (Fig. 7) obscured targets on other piles. After all the piles were driven only 30% of the total number and 18% of those driven the previous week could be stereoscopically interpreted on the photographs. The results, plotted on Fig. 9, were considered sufficient to assess the overall pile movements.

The general direction of movement of in-place piles was approximately away from and in a direction perpendicular to the area of pile driving. The magnitude of these movements can be appreciable and it varies. When the piles were driven in zone II, those in zone I moved as much as 175 mm (7 in.) (dashed vector for pile 28, Fig. 9). The solid vectors indicate pile movements due to piling in zone III; the dotted vectors due to piling in zone IV. The movements of some of the piles changed direction as the piling proceeded from one area to another. The magnitude of the movements can be obtained from the pile movement scale shown on the figure. The accuracy of the measurements was within 15 mm in the north–south and

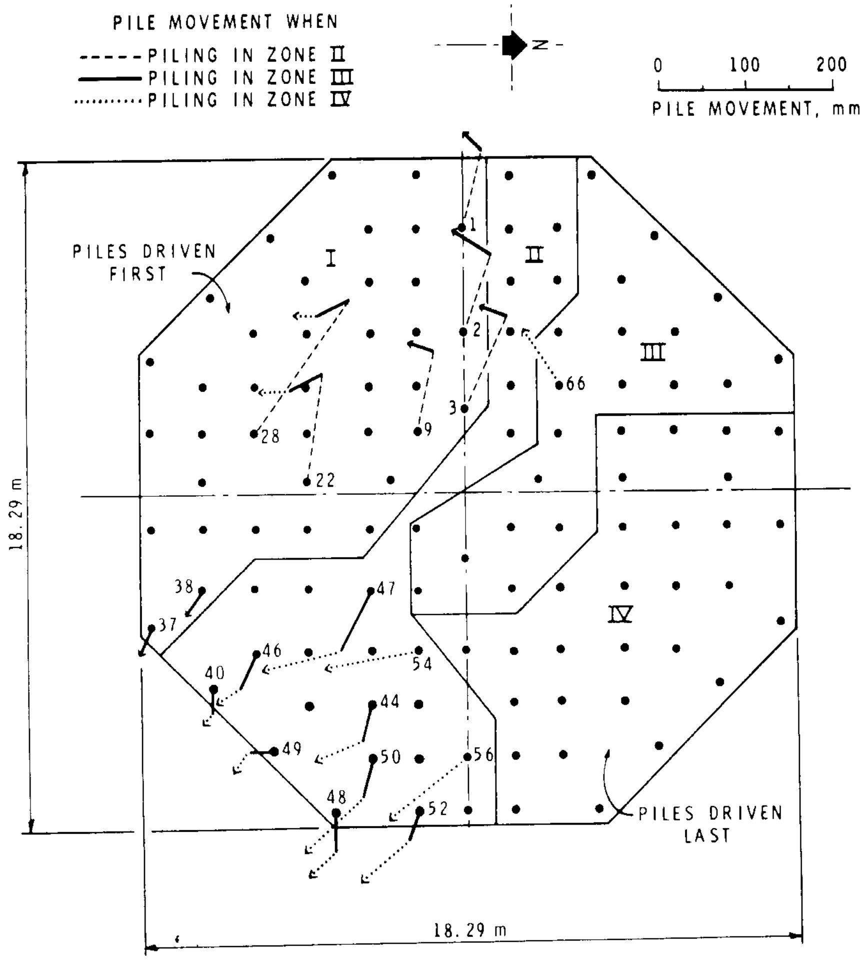

FIG. 9. Photogrammetric measurements of horizontal pile movements due to pile driving in successive zones.

20 mm in the east–west directions (Bozozuk *et al.*, to be published).

The photogrammetric measurements and engineering level surveys of the piles indicated no heave or settlement of the piles.

Horizontal Soil Movements Due to Piling

Although the utmost care was taken to protect the field instrumentation during construction activities, a pile segment was accidentally dropped on inclinometer 55 (Fig. 6), damaging it beyond repair. Measurements were successfully made, however, with the remaining five inclinometers.

When inclinometers are used to measure lateral soil movements, the bottom end of the casing is usually installed deep enough to provide a stable reference point for the calculations. This would have meant installing them to depths greater than the end-bearing piles. As this was not economically possible, the movements of the tops of the inclinometer tubes were monitored using the photogrammetric surveys, thus providing a secondary reference datum for the calculations. The 'forest' of projecting piles obscured some of the inclinometer tubes from the cameras, however, and consequently, absolute horizontal soil movements were measured only at locations 52 and 54, and only relative movements were obtained at 51, 53, and 56. The measurements indicated that the pile driving generally pushed the soil laterally away from the pile group in large blocky masses.

The absolute horizontal soil movements measured with inclinometers 52 and 54 north of the piling area were plotted (Fig. 10). Inclinometer 52 north of the pile group moved a distance varying from 125 mm (5 in.) at the ground surface to 45 mm (1.8 in.) at a depth of 18 m (59 ft). At inclinometer 54, 7 m (23 ft) north of the pile group, outward lateral movements occurred to a depth of 10.7 m (35 ft) below the floor of the

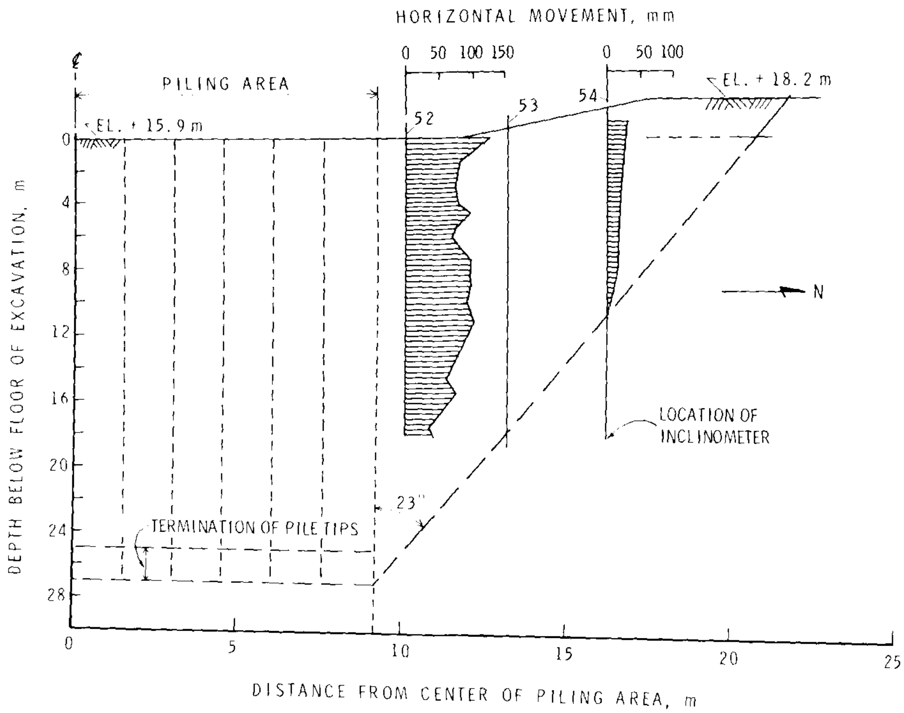

FIG. 10. Horizontal soil movements north of piling area.

excavation. A straight line connecting the tip of the pile group with this point defined an angle of 23° from the vertical. This line also passed through the tip of inclinometer 53, indicating that it could have been affected by lateral soil movements caused by driving the piles.

Extending the straight line upwards, it intersected the level of the excavation at a distance of 11.5 m (38 ft) from the edge of the pile group. If this line defined the actual limits of horizontal soil movements due to pile driving, it should correlate with the outer limits of vertical soil heave.

Soil Heave Due to Pile Driving

Vertical soil heave was measured along two orthogonal lines through silo 4 using spiral-foot and bellow-hose gauges. The spiral-foot gauges provided measurements at a depth of 1.5 m (5 ft) below the floor of the excavation. Surveys on the protective casing located within the pile group indicated that on the average the soil heaved an additional 50 mm (2 in.) above this depth. The results of the measurements within and outside the pile group including those from the bellow-hose gauges were plotted for the south to north and west to east sections on Fig. 11. The average maximum heave at a depth of 1.5 m (5 ft) within the group was about 400 mm (16 in.). Outside the group the heave diminished rapidly reducing to zero at a distance of about 12 m (39 ft) from the piles.

The variation of soil heave with depth was measured with bellow-hose gauges. During their installation, gauge 41 was grouted with a pure bentonite grout which proved to be too soft. Subsequently 20% cement was added to the bentonite grout for the remaining gauges (Bozozuk and Fellenius, to be published).

During the pile driving, lateral squeezing of the soil within the pile group tilted some of the bellow-hose measuring rings which prevented observations below the location of the tilt. It also increased the error of the measurements to about ±8 mm.

The measured distributions of heave with depth at the end of pile driving were plotted on Fig. 12. Gauge 41, at the center of the pile group, provided questionable data below a depth of 8 m. Gauge 45, however, showed that the heave decreased with depth inside the pile group from 350 mm (13 in.) near the surface to about 190 mm (7.5 in.) at a depth of 18 m (60 ft). An average straight line through the curve would show about zero heave at the depth of the pile tips. Similarly, gauges 42, 43, and 46, located outside the group of piles, showed decreasing heave with depth. Gauge 47, located 15 m (50 ft) from the group, showed negligible vertical soil movements, indicating that

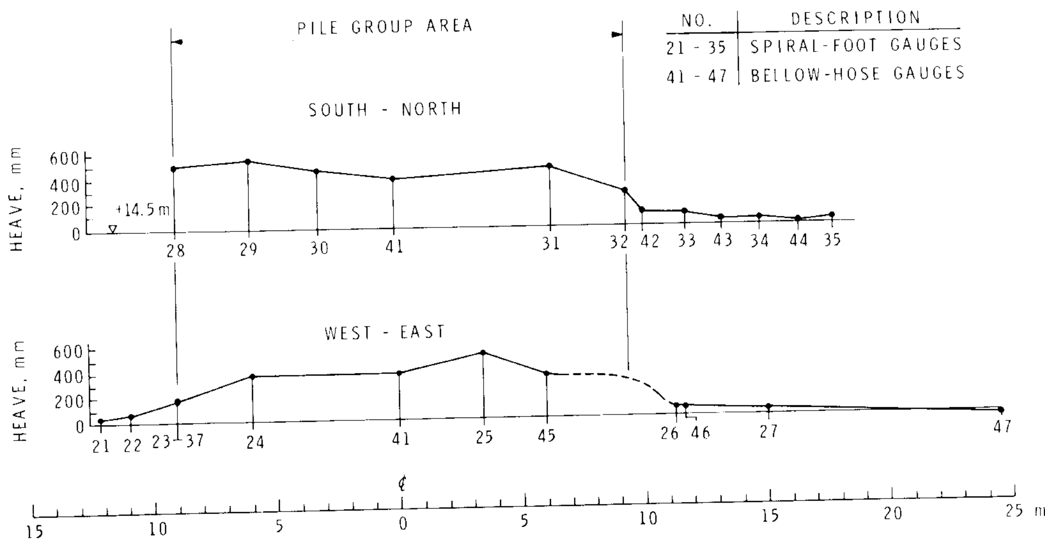

FIG. 11. Vertical soil heave at a depth of 1.5 m within and adjacent to the pile group.

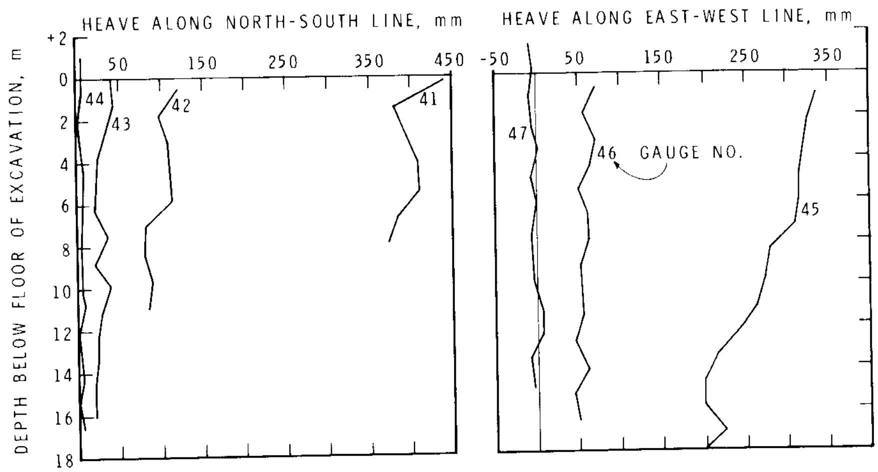

FIG. 12. Heave measured with bellow-hose gauges.

the soil was unaffected by the pile driving at this distance.

Assuming that vertical soil heave with depth was symmetrical about the center of the pile group, the bellow-hose measurements were used to prepare contours of vertical heave (equiheave lines) (Fig. 13). The maximum gradient occurred near the edge of the group of piles. The contours were essentially parallel and approximately vertical just outside the group. Zero vertical heave occurred near the ground surface 12 m (39 ft) from the edge of the piles, and 7 m (23 ft) from the edge of the piles at a depth of 12 m (39 ft). Considering the accuracy of these measurements, this contour compared extremely well with the limits of horizontal soil movements shown earlier.

This agreement was further checked by plotting the measured vertical heave at the ground surface with distance from the group of piles (Fig. 14), and calculating the ratio of the volume of soil heaved to the volume of installed piles. Within the pile group, the average soil heave was 450 mm (18 in.), giving a heaving volume of 130 m^3. The volume of 116 piles, installed to an average depth of 26 m, was 234 m^3. Therefore the ratio of the soil displaced within the group was 55% of the pile volume. The average soil heave outside the pile group to a distance of 3 m was 110 mm. Including this volume, the volume of soil now displaced to the volume of piles increased to 65%. Including the soil heaved to a distance of 12 m from the piles, the volume displaced increased to over 80%. For the assumption made, this was an excellent check on the field observations and implied that the volume change due to consolidation was minimal.

The measurements of the spiral-foot and bellow-

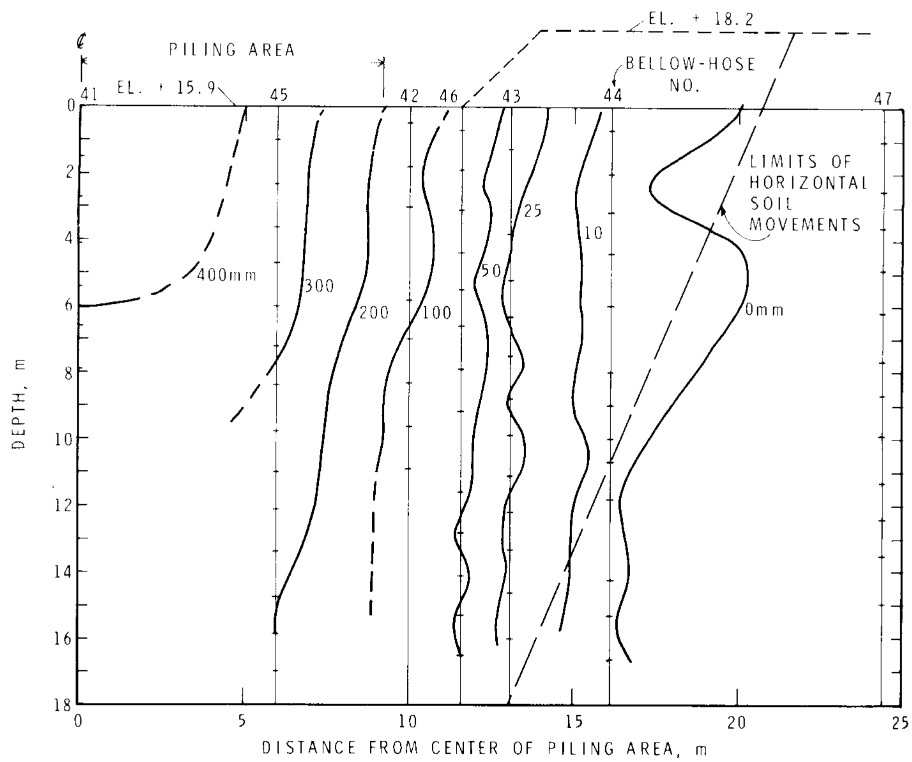

FIG. 13. Contours of vertical soil heave due to piling.

hose gauges were continued for 3–4 weeks after the pile driving, showing that the ground within the pile group settled an average of 5 mm (0.25 in.). Because of the construction activities that followed, further settlement surveys were not possible and no correlations with pore pressure dissipation were made.

Effect of Pile Driving on Pore-water Pressures

Of the 22 piezometers installed at eight locations, the piezometers at station 68, and the deep one at station 61 (30.5-m (100-ft) depth) were destroyed during construction. Later, when the formwork for the silo foundations was constructed, the piezometers remaining at stations 61 and 62 had to be abandoned. Stations 62 and 68 were located outside the pile group along the northeast line (Fig. 1); 61 was located at the center (Fig. 5); and 63 at 46 m (150 ft) north of the pile group.

Essentially all the piezometers reacted to the pile driving. Piezometers at stations 61 and 64 (located within the group of piles) measured comparable pore-water pressures, registering a maximum of about 150% over the initial conditions at a depth of 8.8 m (29 ft). For the same elevation, piezometer stations 62 and 65 (located just outside the pile group) indicated an increase in pore-water pressure of 100%. Pore pressures continued to decrease with distance from the piles, reducing to about 50% at 6 m (20 ft) at station 66 and about 25% at 15 m (50 ft) at station 67. The increase of 25% represented a small excess pore-water pressure of 20 kPa or 2 m (6.6 ft) head of water.

The development of excess pore-water pressures as the piles were driven is illustrated on Fig. 15, which shows the pore pressure records for station 65, 1.3 m east of the piling area. The diagram is a composite of measured pore pressures, distance from the piezometer that each pile was driven, and the days that the piles were installed. Forty-four piles were installed in the first 3 days in the areas shown on Fig. 4, 11–29 m (36–95 ft) from the piezometers. They generated the small increases in pore pressure measured at depths of 5.5, 8.8, and 12.2 m (18, 29, and 40 ft). Pore pressures continued to increase as the number of piles increased and the distance to the piezometers decreased. At day 13, a peak pore pressure was recorded at a depth of 12.2 m, just after 9 piles had been driven at a distance of 1–6 m (3–20 ft). Pore pressures subsequently decreased as the piling distance increased and the number of piles driven decreased. On day 27, the piezometers again reacted strongly when 12 piles were installed at a distance of 1–8 m (3–26 ft).

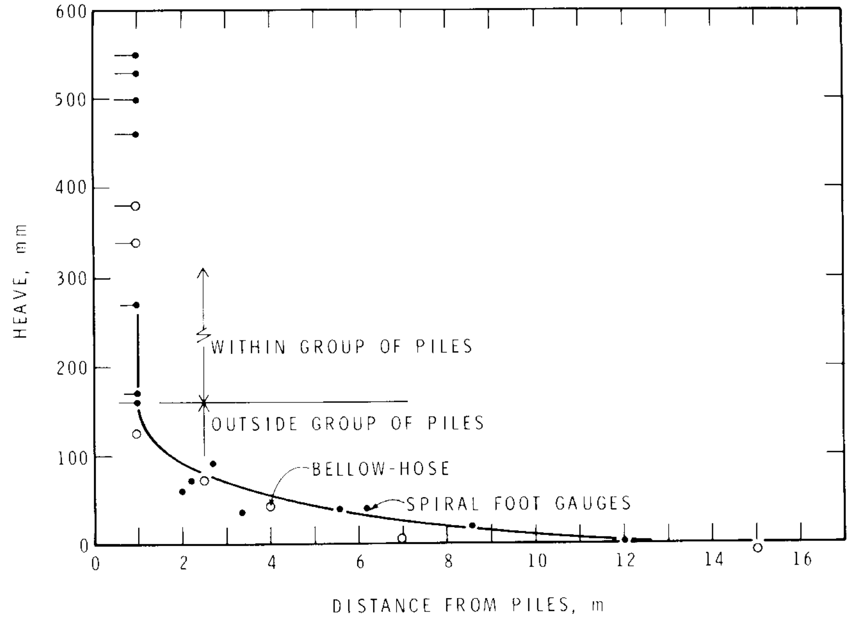

FIG. 14. Relation between heave of ground surface and distance from group of piles.

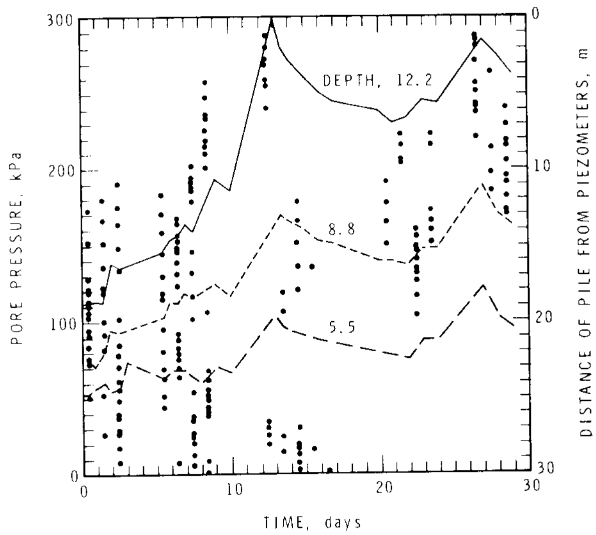

FIG. 15. Pore pressures at piezometer station 65, during pile driving.

The variations of pore water pressures measured at stations 64 (within the pile group), 65 (1.3 m east of pile group), and 66 (6 m east of pile group) over a period of 300 days from the start of pile driving are shown on Fig. 16. The excess pore pressures generated during the pile driving dissipated relatively rapidly during the first month after the piling was completed. The rate then decreased, becoming approximately linear with time. From day 85 to 125, the foundations for the silos

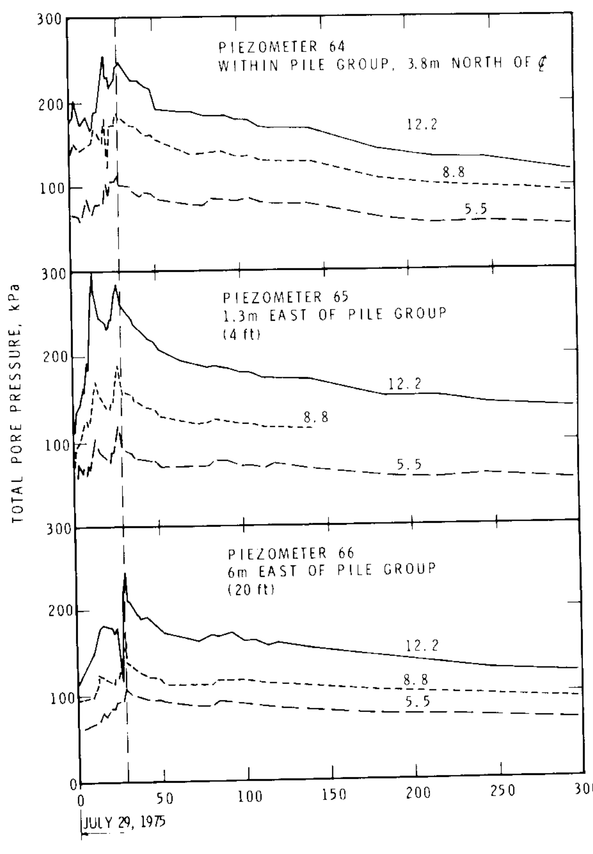

FIG. 16. Variation of pore-water pressures with time.

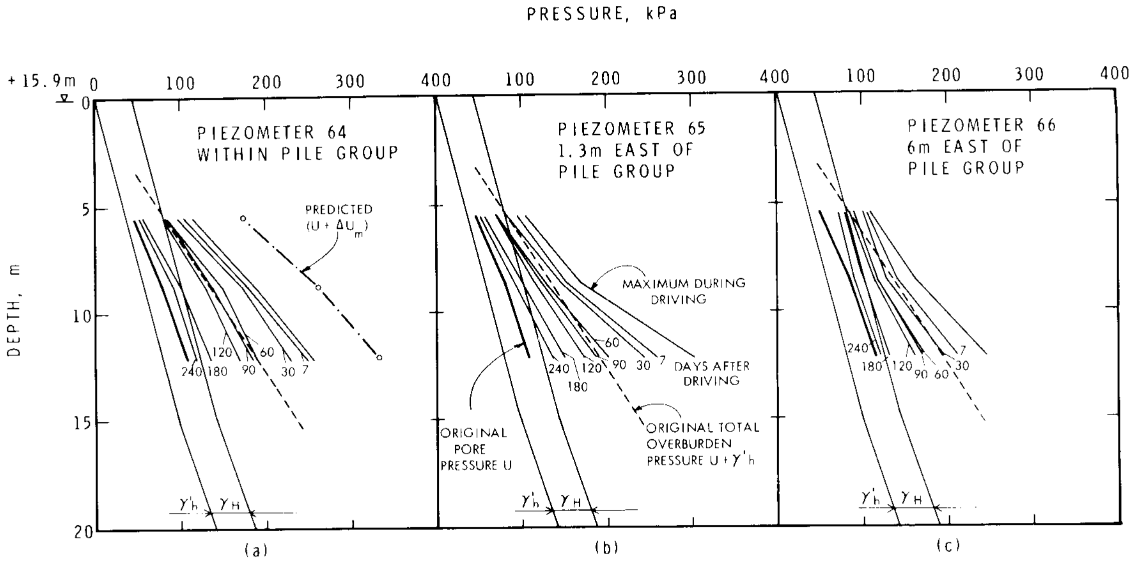

FIG. 17. Comparison of observed pore pressure profiles with the original total overburden and pore-water pressures at three locations.

were constructed, which involved backfilling the site. This increase in load was responsible for the slight rise in pore-water pressures observed over this period.

The maximum pore-water pressures observed at these piezometer locations are compared with the total overburden pressures on Fig. 17. The original pore pressures and overburden pressures are illustrated as solid and dashed lines, respectively. The maximum pore-water pressures and those measured at 7, 30, 60, 90, 120, 180, and 240 days after pile driving are also shown. The figures showed that the generated pore pressures exceeded the original total overburden stress at all three locations by as much as 35–40%. As already mentioned, the piezometer at station 67 did not show an appreciable increase in pore pressure. Thus the pore-water pressures generated by driving the piles exceeded the total overburden pressure for a distance greater than 6 m (20 pile diameters) and less than 15 m (50 pile diameters) from the edge of the pile group.

This figure also shows that the excess pore-water pressures dissipated to the total overburden pressure in 60–90 days after the piles were driven. Complete dissipation to the original pore pressure conditions required about 8 months.

Discussion

Soil Study

Engineering soil tests performed on thin-walled piston-tube samples (obtained after 116 piles had been driven at a pile spacing of generally 5–6 diameters center-to-center) indicated no significant changes in water content, consistency limits, or preconsolidation pressure of the soil. Furthermore, no physical distortion of the soil formations was observed. Results of a study on a group of 13 similar piles driven in the same soil at a spacing of 4.0 diameters center-to-center (Fellenius and Samson 1976) provided similar conclusions with respect to water content and consistency limits. There was, in addition, a small reduction in preconsolidation pressure and some physical distortion of the soil formations. It appears, therefore, that a minimum pile spacing of 5 diameters, center-to-center, is required to avoid significant disturbance of the soil midway between them, due to driving groups of displacement piles in sensitive marine clay.

Shear Strength

The pile driving reduced the *in situ* vane shear strength of the soil within the pile group by about 15% and the cone penetration resistance by 30%. These were partially regained within 3 months after pile driving. The findings were consistent with the observations from the 13 pile group (Fellenius and Samson 1976).

Soil and Pile Movements

The average ground heave within the group of 116 piles was 450 mm (18 in.), which was considerably greater than the 70 mm (3 in.) reported for the smaller group of 13 piles, which were driven when the ground was frozen (Fellenius and Samson 1976). Outside the 116 pile group, the heave decreased to an average of 110 mm (4.3 in.) at a distance of 3 m (10 ft), then gradually diminished to zero at a distance of 11 m (36 ft). The heave within the large group of piles represented 55% of the pile volume; it exceeded 80% for the whole area affected.

Horizontal soil movements accompanied the vertical heave. At the edge of the pile group the horizontal soil movements were 125 mm (5 in.) at the ground surface in a direction away from the piles. The movements decreased with distance becoming zero at 12 m (39 ft), correlating well with the heave measurements. Combining the observations, the limits within which vertical and horizontal soil movements took place were defined by a plane represented by a straight line starting from the tip of the piles inclined at a slope of 23° from the vertical.

The photogrammetric measurements indicated that driving piles adjacent to those already in the ground caused them to move laterally as much as 175 mm (7 in.). Engineering specifications generally call for positioning piles within a maximum horizontal deviation of 75 mm (3 in.) from their designed locations. The measurements clearly show that for a large group of displacement piles in sensitive clay, lateral movements of piles can be expected to be greater than generally specified in present building codes.

In summary, the observations showed that substantial soil movements occurred within and adjacent to the large group of piles. Appreciable soil movements occurred to a distance of 15 pile diameters, and became negligible at about 39 pile diameters from the pile group. These values were about double those observed for a smaller 13 pile group (Fellenius and Samson 1976). For structures that can tolerate or resist some soil movements, however, this information can serve as a useful guide for selecting or specifying a horizontal clearance distance for driving groups of displacement piles in sensitive marine clay.

Pore Pressures

Distribution of the excess pore-water pressures was generally parallel to the total overburden pressure at the site. They exceeded the total overburden pressure by about 30–40% within the pile group and beyond to a distance of more than 20 pile diameters. The excess pore pressures at a distance of 50 pile diameters were negligible. These pore pressures dissipated fully about 8 months after the piles were driven as compared to the 5 months observed for the 13 pile group in the same soil (Fellenius and Samson 1976).

The maximum excess pore pressures generated near the surface of piles driven in normally consolidated clay can be estimated from the equation given by Lo and Stermac (1965):

$$\Delta U_m = [(1 - K_0) + (\Delta U/P)_m]\, P_0'$$

where K_0 = coefficient of earth pressure at rest, P_0' = effective overburden pressure, P = consolidation pressure used in consolidated undrained triaxial strength test with pore pressure measurements, and $(\Delta U/P)_m$ = maximum pore pressure ratio obtained from the test.

For the soils at the test site, Blanchet (1976) found that the pore pressure ratio became constant when the consolidation pressure used in the test was the preconsolidation pressure P_c' of the soil. He obtained a maximum pore pressure ratio of 0.70 for the marine clay.

Blanchet (1976) proposed the following equation:

$$\Delta U_m = (1 - K_0)\, P_0' + (\Delta U/P)_m P_c'$$

where $K_0 = (1 - \sin \phi')R_0^{\frac{1}{2}}$ (Meyerhof 1976), R_0 = overconsolidation ratio of the soil, and ϕ' = angle of internal friction (28–29° for the soil).

Applying these relations to the soil within the large group of piles, the estimated maximum excess pore-water pressures at depths 5.5 m (18 ft), 8.8 m (29 ft), and 12.2 m (40 ft) are 130, 180, and 225 kPa (1.4, 1.9, and 2.3 tsf), respectively. The predicted total pore-water pressures $(U + \Delta U_m)$ of 175, 260, and 330 kPa (1.8, 2.7, and 3.4 tsf), respectively, are plotted on Fig. 17*a*.

The predicted pore-water pressures were considerably over the observed values. Part of this discrepancy may be due to the fact that pore pressure measurements were made between the piles whereas the predictions apply at the pile surface. Good agreement would have been obtained if a maximum pore pressure ratio of 0.5 had been used instead of 0.7. It is evident that further studies on this problem are warranted.

Conclusions

1. Driving groups of displacement piles in sensitive clays at a minimum center-to-center pile spacing of 5 diameters has little effect on the soil mass between the piles, except in the annular zone around each pile where it is known that the clay is highly disturbed.

2. Pile driving reduced the *in situ* vane shear strength by 15%, and the cone penetration resistance by 30%. The lost strength and penetration resistance were partially recovered in the 3-month period following pile driving.

3. Photogrammetric measurements showed that in-place piles were pushed laterally as much as 175 mm (7 in.) after additional piles were driven in the group.

4. Vertical soil heave within the pile group amounted to 450 mm (18 in.) or 55% of the soil volume displaced by the piles. The heave decreased rapidly with distance from the pile group.

5. The vertical and horizontal soil movements outside the pile group occurred within a cone-shaped volume of soil with its apex at the tip of the piles and its surface inclined 23° away from the pile group. The cone intersected the ground surface about 40 pile diameters from the pile group. Over 80% of the total volume of soil displaced by the piles was accounted for within the outer limits of the cone.

6. The maximum pore-water pressures measured after pile driving within the group, and 6 m (20 ft) or 20 pile diameters from the group, exceeded the total overburden pressures by 35–40%. The vertical distributions of the pore-water pressures were essentially parallel to the overburden pressures. The influence of pile driving on the pore pressures at 15 m (50 ft) or 50 pile diameters from the group of piles was very small as the pore pressure increase was small.

7. The generated excess pore-water pressures had dissipated completely 8 months after the pile driving.

8. The predicted maximum pore-water pressures that apply at the surface of the pile greatly exceeded the values measured in the field between the piles. Further studies are required to improve prediction methods.

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