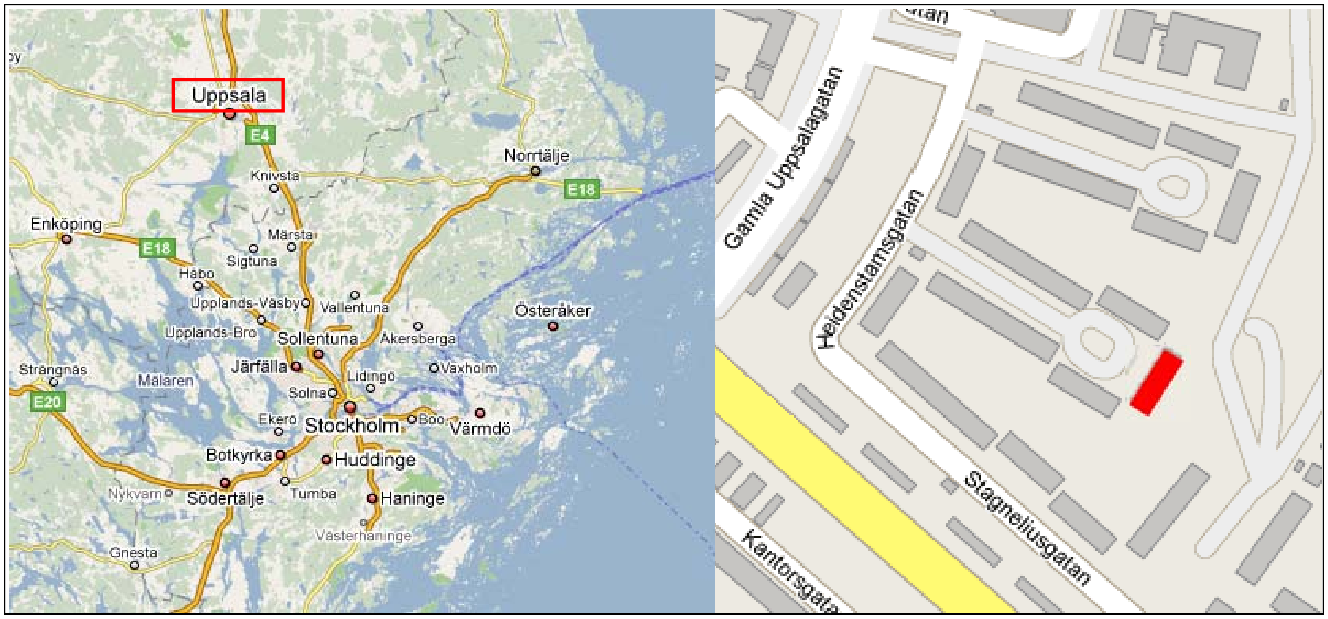

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Settlement of a Three-storey Apartment Building on Piles

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ABSTRACT. Area subsidence, and settlement and deformation in precast concrete piles supporting a three-story building were monitored over a three-year period. Soil profile was 3 m clay crust followed by 13 m of normally consolidated soft clay above sand. Anticipated groundwater table lowering was monitored in three piezometers (open pipes) along with measurements of settlement and shortening of four piles under the building, and settlement of ground surface and building. The results showed that incorporating water infiltration was successful in preventing lowering of the groundwater table. The settlement of the ground surface was consequently negligible, about 20 mm, and the pile deformation was small. The load applied to the piles from the building caused the piles and building to settle about 2 mm. During the following three years, a further 1 mm settlement was observed of which the settlement of the pile toe was about 0.2 mm. The pile shortening corresponded to 500 microstrain deformation of the piles and was one fifth due to creep of the pile material and to four fifths due to negative skin friction. Despite the small soil settlement, drag load due to accumulated negative skin friction along the piles was estimated to be about 200 KN.

1. INTRODUCTION

The following is a summary of a 45-year old case history of a housing development, which contains some observations still of interest. The case was never published, only compiled in a report in Swedish to the funding authority (Fellenius 1970).

The housing development lies in the Uppsala Plain, a fertile farming area around the City of Uppsala about 100 Km north of Stockholm, Sweden. In the early 1960s, a series of 16 three-storey apartment buildings was to be built a virgin area north of the city of Uppsala, Sweden. The soil profile at the site consisted of about 15 m to 20 m of soft postglacial clay overlying sand and gravel, the latter being distal esker sediment. The buildings, including basement floor, were to be supported on square 250-mm diameter, precast concrete piles driven through the clay to bearing in the sand and gravel deposit. Pumping (mining) of water in the sand and gravel a few kilometre away from the site was to commence coinciding with the construction and during a long time afterward. From experience with water mining in the general area,

it was realized that the pumping would lower the groundwater table. The lowering would create a downward gradient in the clay below and around the buildings and subsequent consolidation of the clay, in turn causing the soil to hang on the piles, i.e., negative skin friction accumulating to drag load.
Prior to this project, the potential for the piles becoming adversely affected by negative skin friction and associated drag loads was rarely considered in foundation design in Sweden. However, recently published studies by the Norwegian Geotechnical Institute (Bjerrum and Johannessen 1965), led the designers to decide it necessary to counter the expected drag loads due to the consolidation of the clay by subtracting estimated drag loads from the allowable loads (300 KN) assigned to the piles. Considerable alarm arose when it was realized that the drag load could be about equal to the allowable load assigned to the piles. Moreover, the lowering of the groundwater table was estimated to cause the ground surface at the site to settle about 0.8 m to 1.0 m, which would cause problems with roads and services connected to the buildings. After consultation with the Swedish Geotechnical Institute, it was decided to eliminate or reduce the lowering of the groundwater table at the site by infiltrating water from the nearby river Fyrisån into the esker material (sand and gravel) at a point located about 1 Km west of the site. The infiltration was intended and expected to alleviate both problems. It was however considered prudent to monitor the effect of the infiltration with regard to the depth to the groundwater table and development of deformation of the piles and ground subsidence near the buildings. The author was assigned to plan, execute, and report the study.

The field work started in May 1966 by installing piezometers and drilling a borehole. The pile driving was carried out in August 1966 and followed by excavation of the basement. Basement wall and floor were completed on January 10, 1967, the date of first readings of gages. The fourth floor and roof structure were completed on April 12, 1967, 90 days later, which date is taken as the "zero" reference to the data. The building roof cover and cladding was completed on June 15, 1967. This paper reports the results of the monitoring of pile deformations, settlements, and pore pressures during the construction period and until April 16, 1970, about 1,100 days after completed construction.

2. OBJECTIVES

The objective of the study was to monitor shortening of a couple of the piles in order to estimate build-up of drag load in the piles and to correlate the loads to the simultaneously monitored total pile movement, soil settlement, and pore pressure. The recorded data were to be used in assessing the response of the piled foundations and results of the water infiltration.

3. PROGRAMME

The development encompassed sixteen apartment houses founded on driven ordinary precast concrete piles with a basement floor supported on the piles. One of the houses was chosen for the subject study. The footprint of the house was 12 m by 39 m as indicated in Figure 1. The figure also shows the locations of monitoring stations.

The study included four test piles (P1, P2, P52, and P88), each instrumented with a pair of telltales. The telltales were placed inside a center tube cast within the piles. One telltale measured between the pile head and the pile toe and the other one measured between the pile head and a point 5 m above the pile toe, using a system described by Broms and Hellman (1970). Two of the test piles (P1 and P2) were located under an outside basement wall and two (P52 and P88) were under an inside basement wall.

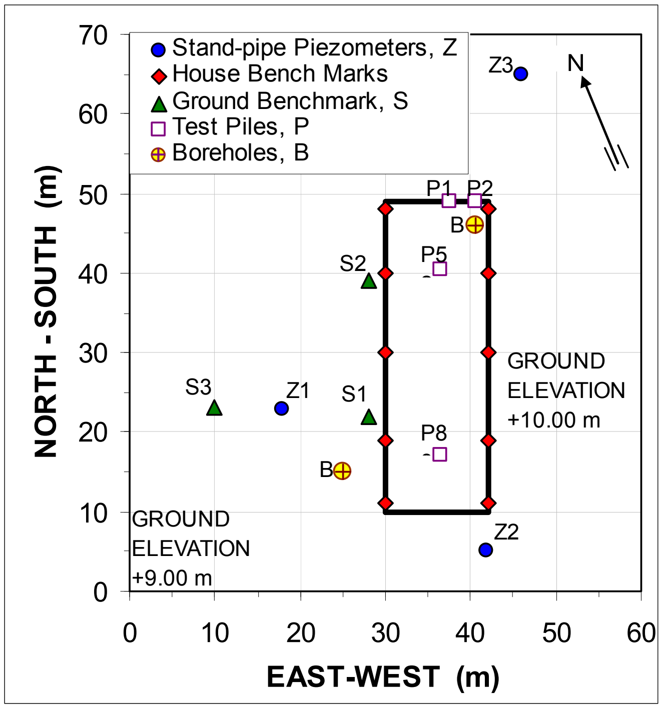

Fig. 1 Plan with location of test piles, monitoring points, borehole, and piezometers

Settlement of the basement wall next to each test pile station (P1, P2, P52, and P88) was measured in relation to a rod driven inside a protective casing about 2 m into the dense sand and gravel esker deposits below the clay layer.

The settlement on the house was additionally monitored by means of surveyor's leveling of ten benchmarks installed at ground level on the outside of the basement wall.

The ground settlement was monitored at each test pile station at a bench mark placed under the basement floor about 0.5 m away from the inside basement wall and referenced to the mentioned rod. Ground settlement was also monitored by surveyor's leveling of three benchmarks placed about 2 m (S1 and S2) and 20 m (S3) away from the house.

The elevation of the groundwater table was surveyed in three stand-pipe piezometers (Z1 - Z3) installed about 1 m into the sand and gravel layer at 5 m, 12 , and 15 m distance away from the building.

After driving the piles, the piles were cut off to design pile elevation. Three about 1 m long pieces were taken to the laboratory for study of deformation parameters. The pieces were placed in water-filled tanks and loaded axially at the same rate and duration as the piles were loaded from the construction of the house, and the shortening of the pieces was monitored. The results of the laboratory test have been reported separately (Fellenius and Eriksson 1970).

4. GENERAL

4.1 Soil Profile

The area of the development was former farmland, rising gently toward the northeast. The original elevation of the ground surface at the piezometers (Z1 and Z2) located south and southwest of the building, the ground benchmarks (S1, S2, and S3), the house corners, and the borehole (B1) at the southwest end was +10.0 m. At the piezometer to the north (Z3), the original ground surface elevation was 12.0 m. The elevation of the final ground surface level was raised and further leveled across the site by adding fill to Elev. 12.5 m. The elevation of the basement floor was designed to Elev. +10.30 m, necessitating an about 1.0 m excavation over the building footprint.

The soil profile at a borehole placed west of the building (Elev.+10.00 m) consisted of about 3 m thick clay crust followed by about 13 m of compressible, soft silty clay and about 4 m of silt with fine sand. Below about 19 m depth, the soil consisted of sand and gravel. The soils below 16 m depth (Elev.-5 m) are esker distal sediments. The undrained shear strength of the clay was determined by means of the Swedish laboratory fall cone on recovered piston samples to range from about 12 KPa immediately below the clay crust and increasing to about 25 KPa at the lower boundary of the clay layer. The water content of the clay was approximately constant with an average of 47 % (void ratio = 1.25). Figure 2 shows diagrams over the distributions of undrained shear strength and water content. The pore pressure measured in the sand and gravel in three piezometers shortly before start of infiltration (late Winter of 1966) corresponded to a phreatic height at about 2.5 m below the ground surface at all three piezometers.

The clay was normally consolidated to slightly preconsolidated. The compressibility was not determined. However, typical range of values of Janbu modulus number (Janbu 1963, 1998; Fellenius 2009) for the clay in the area range from about 5 to 10, and typical preconsolidation margin is about 10 KPa with a recompression modulus number of about 60. Consolidation settlement in the area after a load increase is expected to continue for at least 30 years with about a third occurring within the first about 3 years.

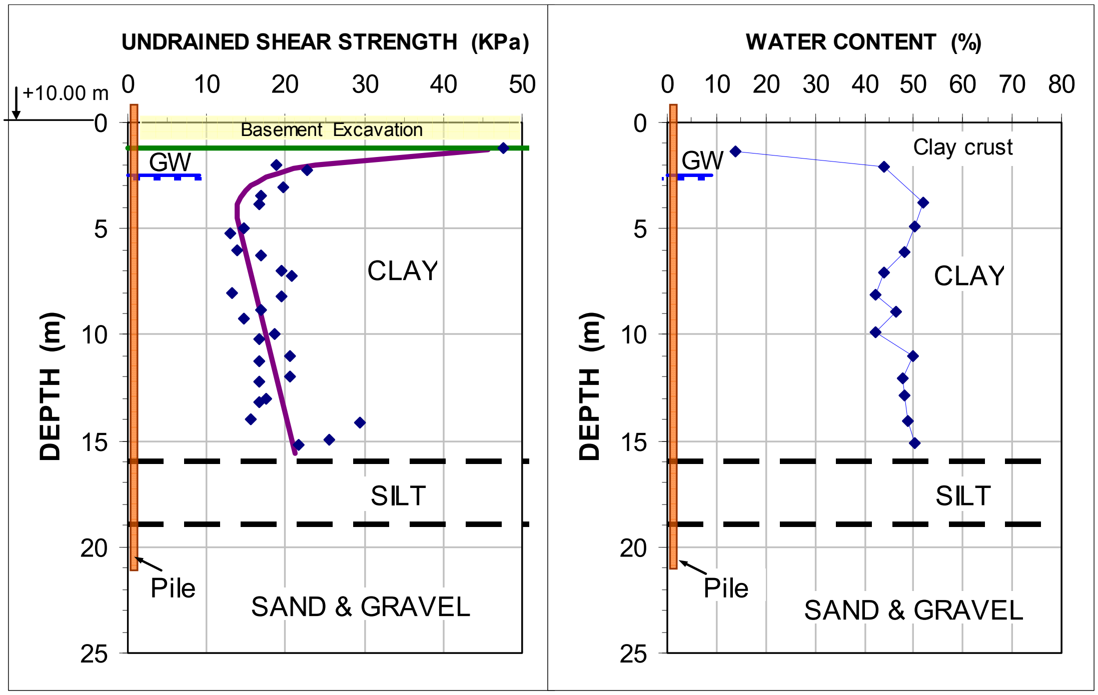

Fig. 2 Profiles of undrained shear strength and natural water content

The clay was normally consolidated to slightly preconsolidated. The compressibility was not determined. However, typical range of values of Janbu modulus number (Janbu 1963, 1998; Fellenius 2009) for the clay in the area range from about 5 to 10, and typical preconsolidation margin is about 10 KPa with a recompression modulus number of about 60. Consolidation settlement in the area after a load increase is expected to continue for at least 30 years with about a third occurring within the first about 3 years.

4.2 Piles and Pile Driving

The piles were vertical, square, ordinary precast concrete piles of 40 MPa nominal concrete cube strength with four 16 mm diameter, ordinary reinforcement bars. The pile toe was closed with a flat steel shoe. The piles were made up from two 12 m long segments spliced in the field with a bayonet type mechanical splice, Type Herkules. The four test piles were equipped with a 41 mm inside diameter steel center tube, and the splice was designed to accommodate the passing of the center tube.

The at the time usually assigned allowable load for square 250-mm precast piles was 400 KN, be the piles shaft-bearing or toe-bearing. The actual working load calculated to be carried by the four test piles ranged from 220 KN to 270 KN. Average working load was 250 KN.

The piles for the subject building were driven in early August 1967. The pile driving was by means of a 30 KN gravity hammer. The height-of-fall was 600 mm during initial driving. To about 18 m depth, the driving required very few blows. Once the pile toe reached the underlying sand below 20 m depth, resistance increased. At termination, the height-of-fall was lowered to 400 mm and the termination criterion was a penetration smaller than 10 mm for each of three series of ten blows. The criterion was satisfied at a toe elevation of about -10.5 m. For each of the test piles, the final penetration was 8 mm for the last series of ten blows. Driving the piles required a total of about 400 blows per pile. The lengths below cut-off elevation, Elev. +10.2 m, of the test piles, Piles 1, 2, 52, and 88, were 20.69 m, 20.45 m, 19.77 m, and 20.84 m, respectively.

Inclinometer measurements of the test piles after the driving showed them to be straight with a deviation from the plumb line ranging from 250 mm through 350 mm.

4.3 Instrumentation

4.3.1 Piezometers. The three piezometers (Z1, Z2, and Z3) consisted of 19 mm diameter pipes equipped with a filter tip and driven into the sand. They were filled with water immediately after driving and the depth below the pipe top was measured during stabilization of the water level inside the pipe, which was rapid, and frequently during the construction of the building and years following. The elevation of the top of the pipe was regularly surveyed and the groundwater table elevation was determined as the depth to the water table inside the pipe subtracted from the pipe top elevation.

4.3.2 Piles. After driving the test piles, a 19.5-mm steel tube was inserted through the pile center tube to 5.0 m above the pile toe, where it was firmly connected to the center tube by means of a special arrangement described by Broms and Hellman (1970) to serve as a tell tale rod. A 0.5 mm diameter, rod was then inserted through the steel tube (i.e., the first telltale) down onto the pile toe (the steel shoe plate) inside the center tube to serve as a second telltale. Pile compression (shortening) was measured between the pile head and the two telltales giving the shortening of the pile from pile head to each lower telltale end and by subtraction of the two values also the compression over the lower about 5 m length of the test pile. The arrangement is detailed in Figure 3 and the dial gages measuring the pile shortening are denoted "1" and "2". Dial Gages "1" and "2" were installed five months later, on January 10, 1967, when the basement floor and walls had been cast. The gages were permanent and reading precision was 0.01 mm.

4.3.3 Settlement. Immediately after the pile driving, inside the basement and at about 0.5 m distance from each test pile, a 2 m long, 50 mm diameter pipe with a plugged end was pushed through the clay so that its top would be a few centimetre below the future basement. The pipe was free to move vertically with the soil. Thereafter, a 25 mm diameter pipe also with a plugged end was inserted through the pipe, expelling the plug in the 50-mm pipe. The 25-mm pipe was continued through the clay to stop at the top of the sand layer. As a third step, a 19.5 mm diameter rod (weight sounding rod) was inserted through the pipe, expelling the plug in the 25-mm pipe. The rod was then continued down into the sand and gravel and hammered to an about 1 m embedment into the sand and gravel (until it could go no deeper).

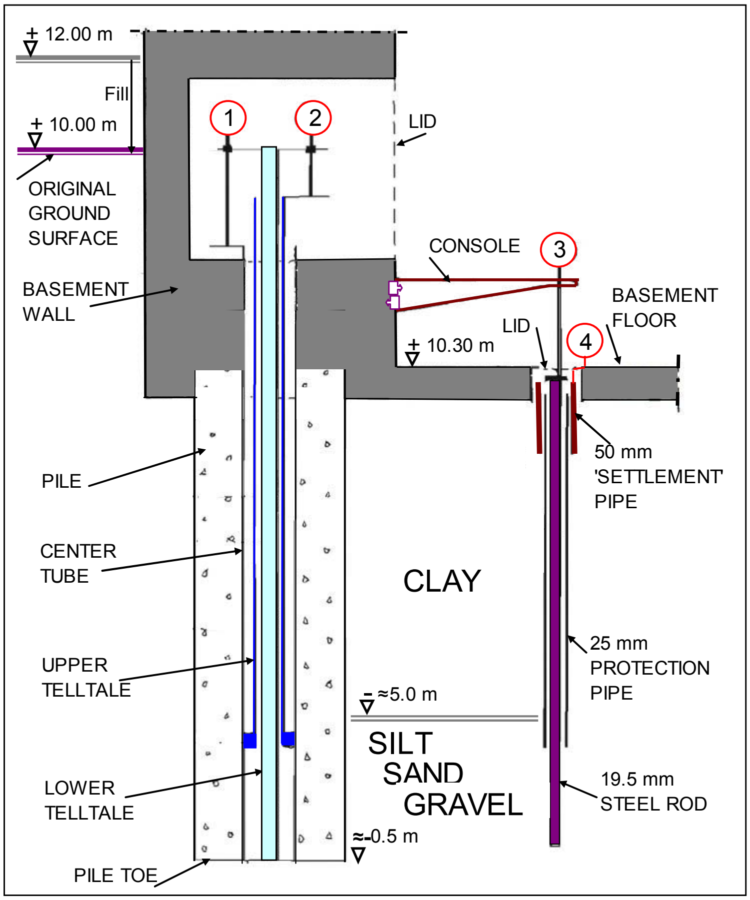

Fig. 3 Measurement arrangement at the test piles (not to scale)

The 19.5-mm rod was to serve as reference to the settlement of the building wall at the particular test pile location as measured via the console and Dial Gage at " 3". The 25 mm outer pipe was to prevent the settling clay from hanging on to the rod and introducing strain and compression in the rod. It was assumed that the so-protected rod would not move or change length. The 50-mm, 2 m long pipe was to serve as reference to settlement of the clay with respect to the floor and, thus indirectly to the rod. Gage at "4" denotes ground settlement measurements of the 50 -mm pipe at the test piles, which were made by means of a steel ruler and not by a dial gage. A series of repeated readings indicated that the ruler measurement precision was about 1 mm.

The body marked "console" (Figure 3) is a removable arm that was attached to the wall in an assured and unchangeable (repeatable) position for each occasion of measurement. The Gage 3, a dial gage, provided the distance to the top of the rod. A reduced distance would indicate a settlement of the basement wall and pile heads. Gage 3 had a reading precision of 0.01 mm. However, a series of trial readings involving a large number of placing and removing the console indicated that the accuracy of the movement values was at best about 0.05 mm.

Referencing a change of the telltale-measured length (Dial Gage "1") of the pile at the same measurement occasion to the change of distance to the rod (Dial Gage "3") would indicate the amount of movement of the pile toe. Thus, a shortening of the full length equal in value to the settlement of the basement wall, would indicate that the pile toe had not moved, i.e., all settlement would be due to compression of the pile. Were instead the measurements to indicate a shortening larger than the settlement, the difference between the values would be due to the pile toe moving down. Conversely, a shortening smaller than the settlement, would indicate that the pile toe would have moved up.

Ten steel survey benchmarks were drilled and grouted into the side of the building basement wall (Figure 1) to provide additional and independent points for monitoring building settlement.

To serve as ground surface settlement benchmark, two 24 mm diameter, 2 m long pipes (S1 and S2) were pushed into the clay crust in the edge of a lawn area 2 m outside the building. A third benchmark was obtained by survey leveling of a day water well located in the paved driveway about 20 m away from the building.

Surveyor's leveling provided elevations of all house and ground benchmarks, and top of piezometer pipes. The surveyor's reference was a benchmark drilled and secured into a bedrock outcrop about 300 m away from the test site.

5. Results

5.1 Piezometers

The results of monitoring of the water level in the piezometers is shown in Figure 4. Note that the ground elevation at Piezometer Z3 is 1.0 m higher than at Piezometers Z1 and Z2. The three curves indicate that the water level fluctuated seasonally by about 1.5 m and that, on average, the average groundwater table remained stable at 3.0 m depth over the three years of observation.

5.2 Settlement of Ground Surface

Figure 5 shows the settlement of the ground surface measured from the basement floor (Gage 4 in Figure 3) near the four test piles (the values are not adjusted for survey measurements of basement wall benchmarks). Surveyed settlement at Settlement Pipes S1 and S2 is also shown. Settlement Pipe S3 was severely compromised by frost movements and had to be abandoned.

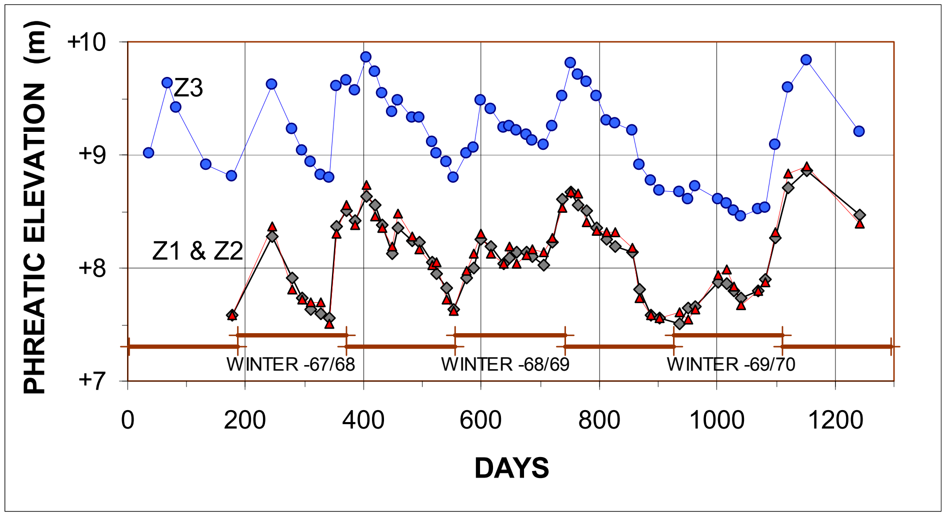

Fig. 4 Phreatic elevation measured in Piezometers Z1 - Z3

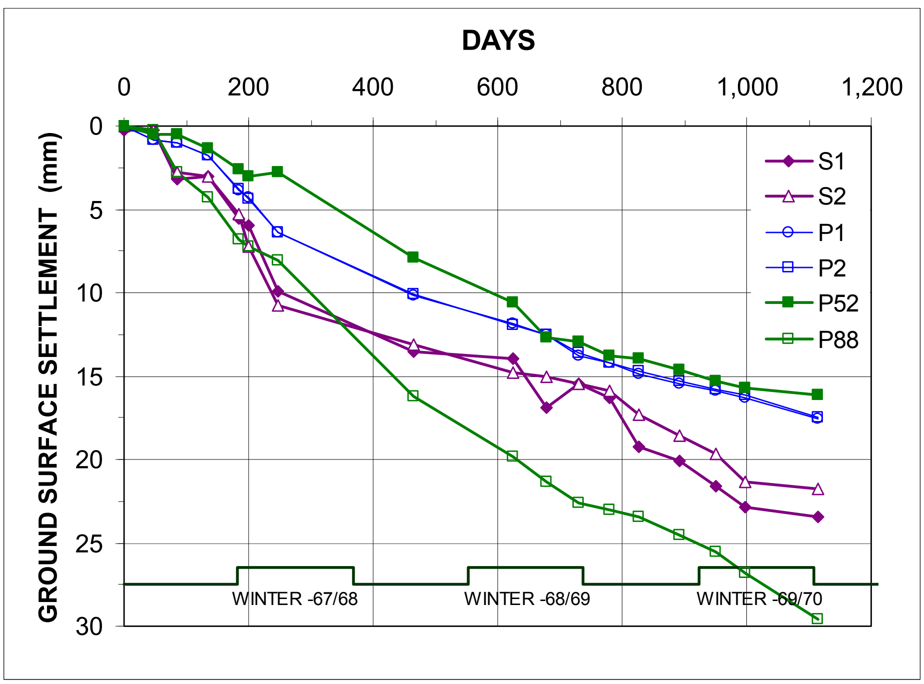

Fig. 5 Settlement of ground surface at benchmarks close to building and at test piles

The S1 and S2 surveyed settlements are slightly larger than the values measured inside the building near three of the four piles. The settlements measured at Piles P1, P2, and P52 piles are very similar to each other; values at Piles P1 and P2 are almost identical. In contrast to the other three pile locations, the ground settlement close to Pile 88, which is located inside the building, is larger than that measured at the other three test pile stations and at S1 and S2. It could have been expected that because of the soil hanging on the piles and/or the fact of having basement excavation as opposed to fill would have caused smaller settlement for the soil close to the piles, as indicated for Piles P1, P2, and P55. However, the settlement at the Pile 88 test station is larger than that at the other three stations. Considering the distance to the outside fill and the location inside the building where the soil had been excavated, a smaller settlement similar to that measured at Pile 52 would instead be expected. The author has no direct observation that could explain the anomaly. However, it is probably due to that at the 2 m long pipe used to indicate the settlement had been placed in soil that had been disturbed during the excavation of the basement ('digging too deep and backfilling with loose soil').

The settlement during the three years of study ranged from about 15 mm to about 30 mm. As the groundwater table did not show a change, the settlement is considered due to consolidation of the clay after placing the about one metre thick fill over the site. The data do not show any trend to slowing down and the 10-mm per year rate will probably have continued for several years. No measurements were taken beyond the first three years, however. Calculation using the software UniSettle (Goudreault and Fellenius 2011) with the input described in Section 4 show a consolidation settlement of 90 mm with 35 mm occurring within the first three years.

5.3 Settlement of Building

Figure 6 shows the measured settlement of the building as determined from console measurements (Dial Gage "3") and the survey values of the ten bench marks fixed to the basement wall. The survey values are average values of the measurements and difference between maximum and minimum values is about one mm. The console measurements of the basement wall settlement and the mean survey settlement agree well and show that the building (i.e., pile heads) settled about 1.5 mm during the three years after end of construction.

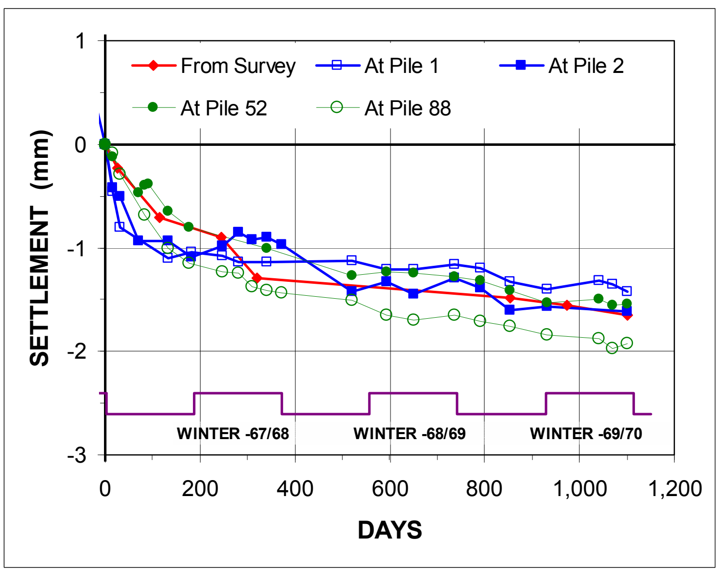

Fig. 6 Settlement of basement wall measured near the test piles and according to survey results

5.4 Observations in the Piles

5.4.1 During Construction

Figures 7 and 8 show the measurements in Piles P1, P2, P55, and P88 during construction and about 60 days after construction was completed**:** change of pile length over the upper and full length telltales (as determined from Dial Gages "1" and "2", respectively**)**, and the subtraction of the two values, i.e., the change over the 5 m lower length of the pile. The figures also show the wall movements determined from Dial Gage "3" and the difference between that value and the lower length, i.e., the pile toe movement. Negative values indicate pile shortening, settlement, and toe penetration, respectively.

The telltale measurements presented in Figure 7 and 8 show that the test piles shortened by about 1 to 2 mm due to build-up of load on the piles as the construction progressed. Shortening of the pile lower length was negligible. However, the values were obtained during almost 90 days of construction during a Winter season. The basement walls and floor were affected by cold temperature that at times was as low as -20 °C. After zero time, April 12, 1967, heating the basement started (to about +20 °C). The wall settlement values measured before Time Zero are therefore not reliable. The full length shortening of the pile is more representative for the building settlement until that time. For the same reason, the movement (rise) of the pile toe shown in each figure is not reliable.

The lengthening ('swelling') of Pile P52 during the first about 60 days of measurements is illogical considering that the loads applied to the piles were increasing. Any length increase is expected to have occurred between driving (August) and the start of the monitoring (January). Lengthening of the piles due to heave induced by excavation is not a cause as the basement excavation was made well before the piles were driven. One cause of lengthening of concrete piles after driving is swelling due to absorption of water. The laboratory study of the recovered pile specimens showed that they absorbed water and swelled by 10 $\mu\varepsilon$, that is about 0.2 mm for a 20+ m length of pile, but it occurs within the first days or weeks after submersion. Most probable cause of the recorded "lengthening" of the piles is the temperature variation during the first about 60 days of monitoring. Additional information from the laboratory tests is that the concrete pile temperature lengthening coefficient was 8 $\mu\varepsilon$ per degree celcius. Temperature change would be expected to have had less influence on the telltale measurements, however, as the major length of the pile is placed in a temperature-stable environment. The lengthening of Pile 52 during the first months of monitoring is indicated in both the telltale measurements and the console records. The two set of measurement are independent, which makes it probable that that the same influence acted on both sets. The conclusion is that although pile compression and settlement will have occurred also during construction period, the records are best studied when referred to a zero defined as the time of completion of the construction.

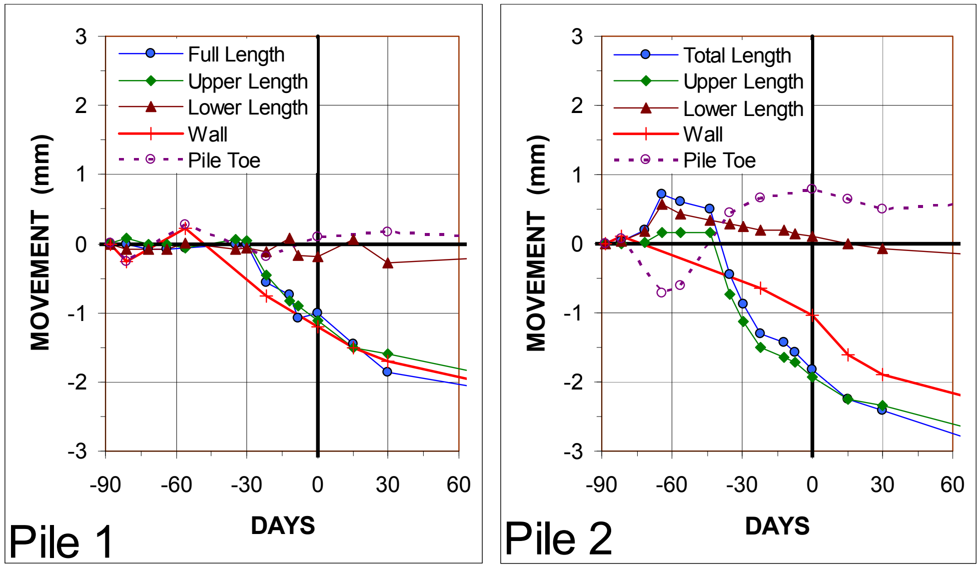

Fig. 7 Piles 1 and 2 movements during two months after start of construction

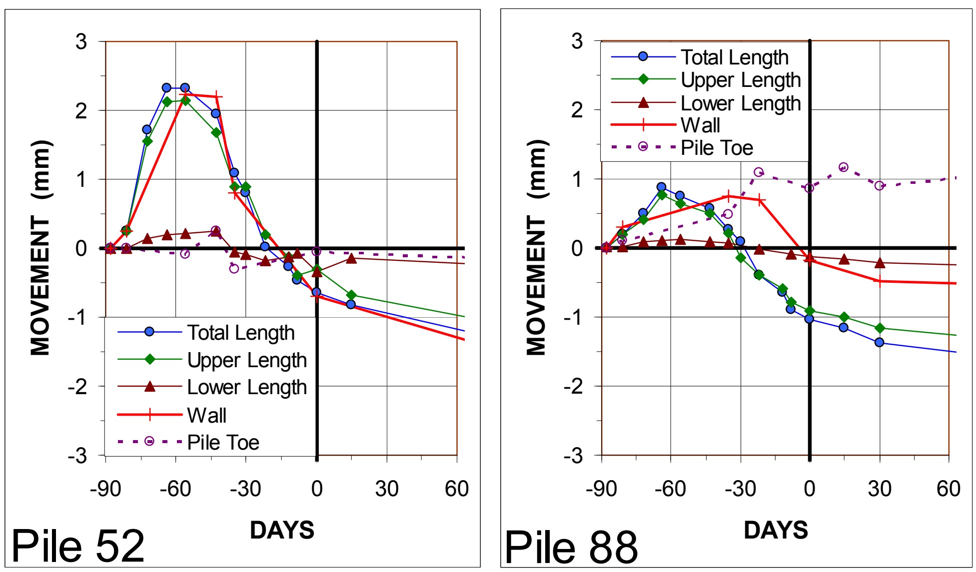

Fig. 8 Piles 52 and 88 movements during two months after start of construction

5.4.2 After Construction

Figures 9 through 12 show the measurements in Piles P1, P2, P55, and P88 from end of construction to the end of monitoring three years (1,100 days later). The records are very similar for the four test piles. The piles experienced a total length shortening of about 2 mm. The settlement of the building wall next to the piles is about marginally smaller than the shortening of the pile indicating a minimal pile toe penetration, about 0.2 mm to 0.3 mm. The shortening of the upper and lower lengths of pile were about 1.5 mm and 0.5 mm, respectively, which corresponds to an average strain of 100 $\mu\varepsilon$ for both lengths.

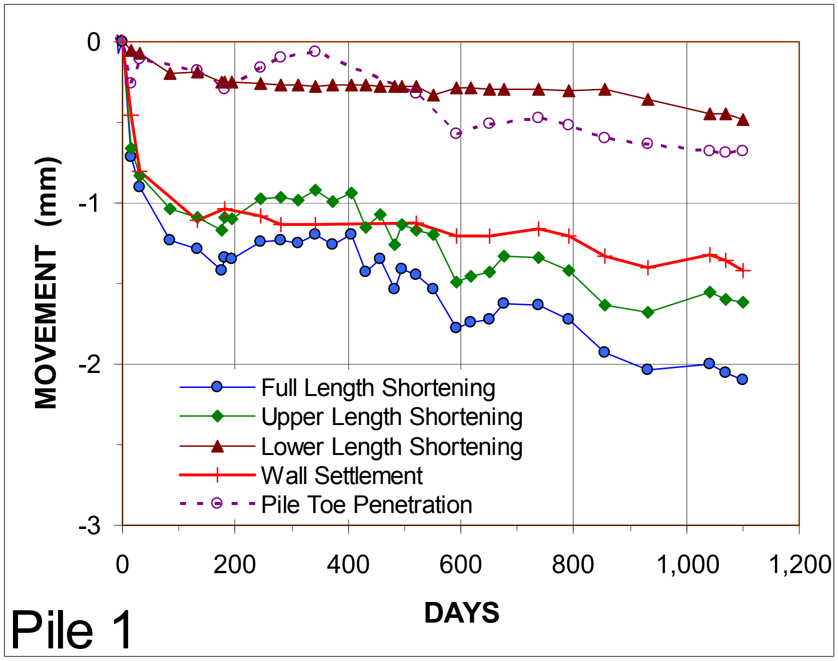

Fig. 9 Pile P1 shortening and building settlement after completion of building

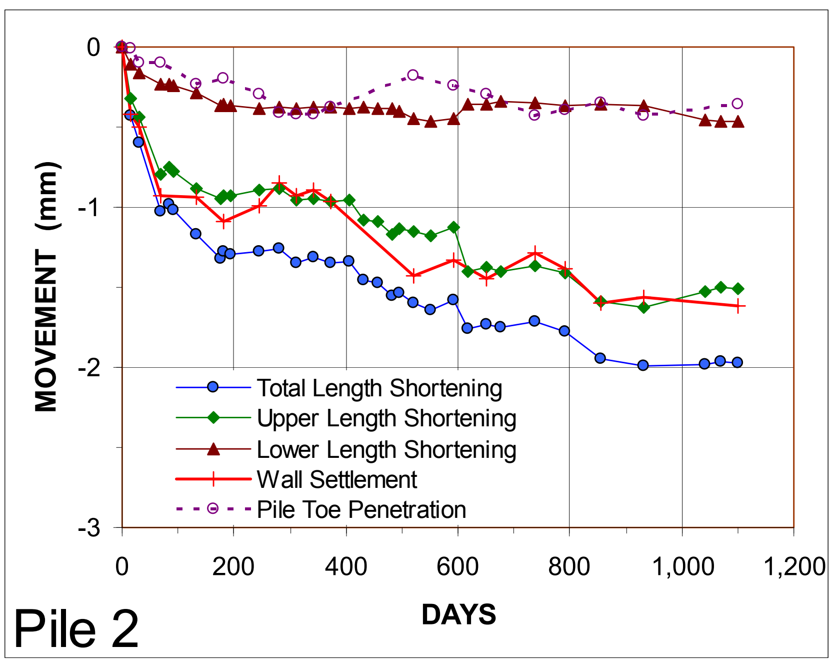

Fig. 10 Pile P2 shortening and building settlement after completion of building

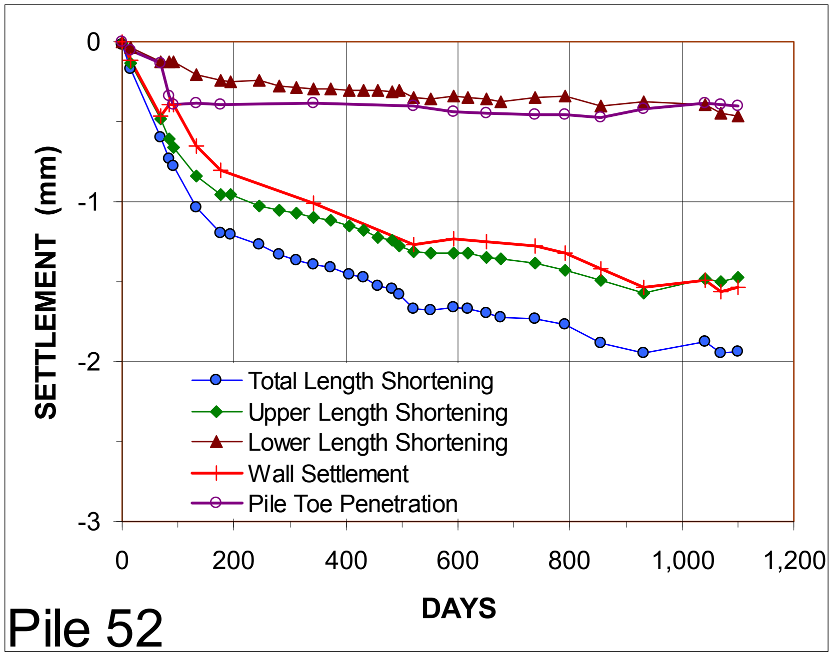

Fig. 11 Pile P52 shortening and building settlement after completion of building

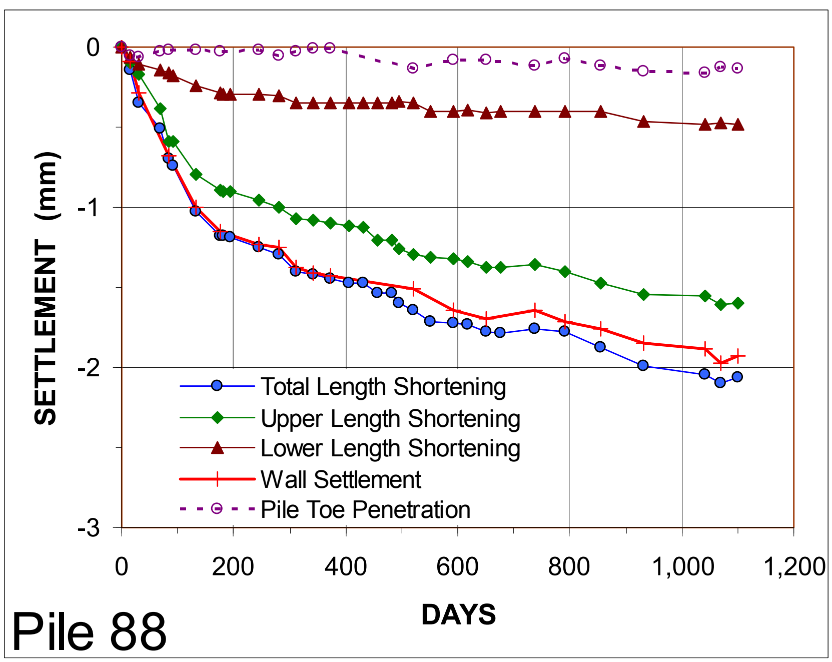

Fig. 12 Pile P88 shortening and building settlement after completion of building

6. Discussion

The laboratory study indicated that the Young's modulus for the short pieces was 32 GPa including consideration of creep. It also indicated that the pile would exhibit creep amounting to 10με to 15 με under constant load for the three years. These values allow an approximate estimation that the average strain corresponds to an average additional load in the pile of 200 KN. That is, load was imposed on the pile after completed construction in addition to the load imposed from the building, and it is of about the same magnitude as the working load from the building.

The added load is due to drag load from accumulated negative skin friction and developing shaft and toe resistance in response to that load. The observation that the added average load would be the same in the upper and lower lengths of the pile, therefore, in the entire length of the pile could lead to the false belief that the drag load would be a constant value all through the pile. The true mechanism is illustrated in Figure 13 by means of a typical graph prepared to fit the approximate soil profile, the length of pile, the load applied to the pile at end of construction, and the load imposed during the three years that gave the observed strain increase in the pile.

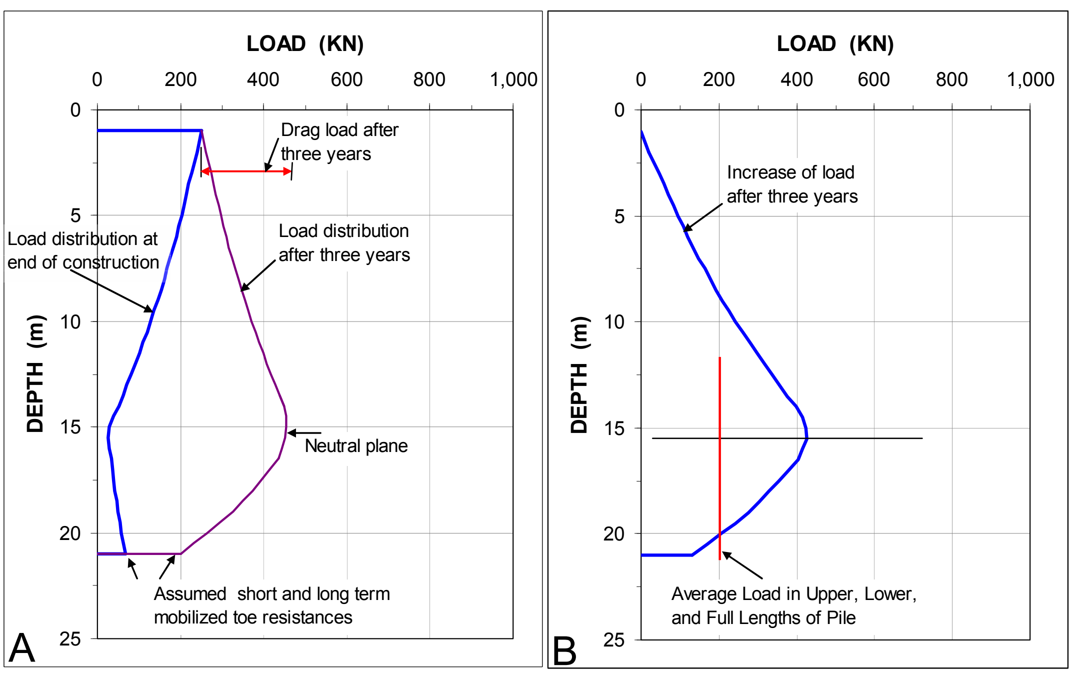

Fig. 13 A. Typical distribution of load in a pile at end of construction and three years later
B. Increase of load in the pile during three years of monitoring

At end of construction, the load applied to the pile head is 250 KN. It is transferred to the soil though shaft resistance, and the load distribution curve is added to an assumed typical load distribution in the pile at end-of-driving. As the negative skin friction builds up the load in the pile increases. The figure shows the typical long-term load distribution after the three years. The distributions follow the effective stress distribution in the soil and, incidentally, also approximately the distribution of undrained shear strength. The figure indicates that the maximum load in the pile at the neutral plane would be about 450 KN and the drag load would be about 200 KN. Figure 13B shows a plot of the distribution of the load increase in the pile and indicates that the average load in the pile along upper, lower, and full lengths is about 200 KN. The load indicated at the pile toe is small. Fully mobilized capacity is expected to be at least 1,500 KN.

At the time, it was expected that — subject to a successful water infiltration — the small settlement developing due to the one metre of fill would not result in any appreciable drag load on the piles. However, the study showed that drag load will develop even if the soil settlement is very small. More important, the observations showed that the pile and building settlement was negligible.

SUMMARY

The study addressed specific objectives of the houses and general area of the site. However, it still resulted in important observations of general validity. The conclusions of the study are summarized as follows.

The water infiltration was successful in preventing the groundwater table from being lowered due to the water mining. Consequently, the expected—"feared"—regional subsidence was prevented.

The soil settlement due to the placing of about one metre of fill over the site resulted in small settlement of the ground surface of the area amounting to about 15 mm to 30 mm over the first three years.

The building basement settlement is negligible, about 1.5 mm and mostly due to 'elastic' shortening of the piles from drag load developing despite the small soil settlement.

The drag load is estimated to be about 200 KN, resulting in a maximum axial load in the pile at the neutral plane of about 400 KN to 500 KN, well below the axial strength of the pile section.

The study showed that drag load has minimal negative effect on a piled foundation and that concerns for piled foundations in compressible soils should be directed toward determining how soil settlement will affect the foundations in terms of settlement rather than of load. The insight gained in the study was one of the important parts of setting the author to the path toward developing the "Unified Pile Design" approach (Fellenius 1984; 1988; 2004).

ACKNOWLEDGEMENT

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