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Wick Drains and Piling for Cai Mep Container Port, Vietnam

Bengt H. Fellenius, Dr.Tech, P.Eng., M.ASCE[1)]
Nguyen Minh Hai[2)], M.A.Sc.

[1)] 2475 Rothesay Avenue, Sidney, British Columbia, Canada, V8L 2B9. <Bengt@Fellenius.net>
[2)] Faculty of Civil Engineering, Thu Dau Mot Univ. Binh Duong, Vietnam. <haitdmu@gmail.com>

ABSTRACT Cai Mep Port is a new container terminal along the Thi Vai River in the Mekong delta approximately 80 km southeast of Ho Chi Minh City, Vietnam. The soil profile consists of about 35 m thick soft, deltaic silty clay deposited on dense to compact sand. The conditions required up to 10 m of preconstruction fill to raise the ground by 2.5 m and included a 7.5 m of surcharge kept on for about six months. The settlement was accelerated by means of wick drains at a spacing of about 1.2 m. The settlements were monitored and amounted to about 3.5 m. After removal of the surcharge, 0.4 m square precast concrete piles were driven to depths ranging from 18 through 30 m to serve as piled foundation of the Port buildings. The monitoring showed that the area and the piles continued to settle after the removal of the surcharge, which was due to the wick drains not functioning below about 20 m depth. The measured settlements and the adopted remedial solution are discussed.

INTRODUCTION

Vietnam is integrating into the world economy at an increasing rate, causing a rapid growth in the urban population due to significant rural to urban migration. The development has brought enormous challenges to the society not least when creating the infrastructure to meet the increased transportation and trade demands. The Vietnam geology is characterized by vast areas with thick deposits of soft, deltaic silty clay, numerous rivers and streams, and frequent floods, where new highways, bridges, and ports are now being constructed. The Cai Mep Port is a new container terminal along the Thi Vai River in the Mekong delta approximately 80 km southeast of Ho Chi Minh City (Figure 1). The site comprises about 35 m of very soft clay over sand. The mean water table lies at the ground surface, but is seasonally above the ground surface. Construction requires raising the area by several metre and placing all structures on piled foundations. This paper describes the design and monitoring of preconstruction fill and surcharge to accelerate settlement with wick drains and also reports pile driving observations.

Fig. 1 Artist's view of completed container port (JICA 2006)

SOIL PROFILE

The soil profile at the site consists of about 30 to 40 m of clay and silt deposited on sand with trace clay and silt. Figure 2 shows the distribution of water content and consistency limits. Total saturated density is about 1,600 kg/m^3 throughout the clay (from w_n = 66 %). The saturated density of the sand below the clay is 2,100 kg/m^3 (from w_n = 19 %). Figure 2 also shows the grain size distribution and the distribution of cone stress in the clay from a CPTU sounding made before the construction start.

Except during occasional flooding of the area due to seasonal and tidal variations, the groundwater table is at the ground surface, Elev. +3.5 m. Pore pressure measurements at depths of 5 m, 10 m, 20 m, and 28 m indicate an upward gradient with a hydrostatic distribution from Elev. +5.0 m, 1.5 m above the ground surface, i.e., artesian condition.

The CPTU sounding cone-stress diagram indicates the soil deposit to be soft throughout. The vane shear strength (not shown in Figure 2) ranges from about 10 through 15 KPa at 2 m depth and increases approximately linearly to about 50 KPa through 80 KPa at 30 m depth, characterizing the clayey silt as soft to a depth of about 20 m and firm below. The correlation coefficient, N_{KT}, between CPTU pore pressure adjusted cone stress and vane shear stress is about 15.

Oedometer tests show the soil to be very compressible with a Janbu modulus number ranging from about 4 through 6. The tests indicate that the preconsolidation margin is small; the clay is essentially normally consolidated. The reloading modulus number, m_r, is approximately ten times larger than the virgin number, m. Figure 3 shows a void ratio vs. stress diagram on a soil sample from a depth of 9.0 m at the site and the distribution of modulus number determined in consolidometer tests and evaluated from the CPTU cone stress. The latter is determined by the method described by Massarsch (1994) and Fellenius (2011). Because of the soft, compressible consistency of the clay, the matching of the CPTU cone stress to the consolidometer values required selecting a Modulus Modifier, *a*, equal to 2, instead of the value of 3 normally applicable to clay.

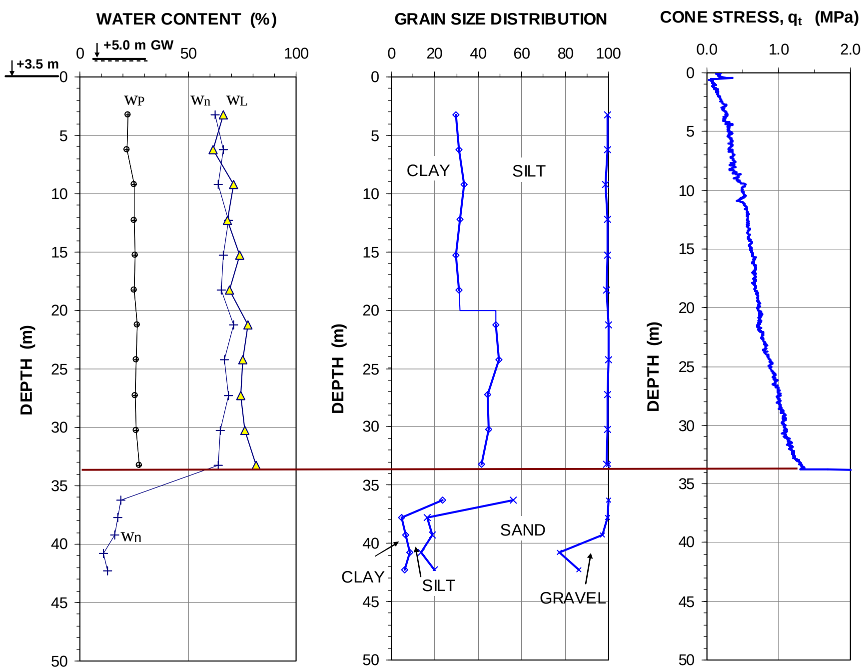

Fig. 2 Consistency limits, grain size distribution, and CPTU cone stress

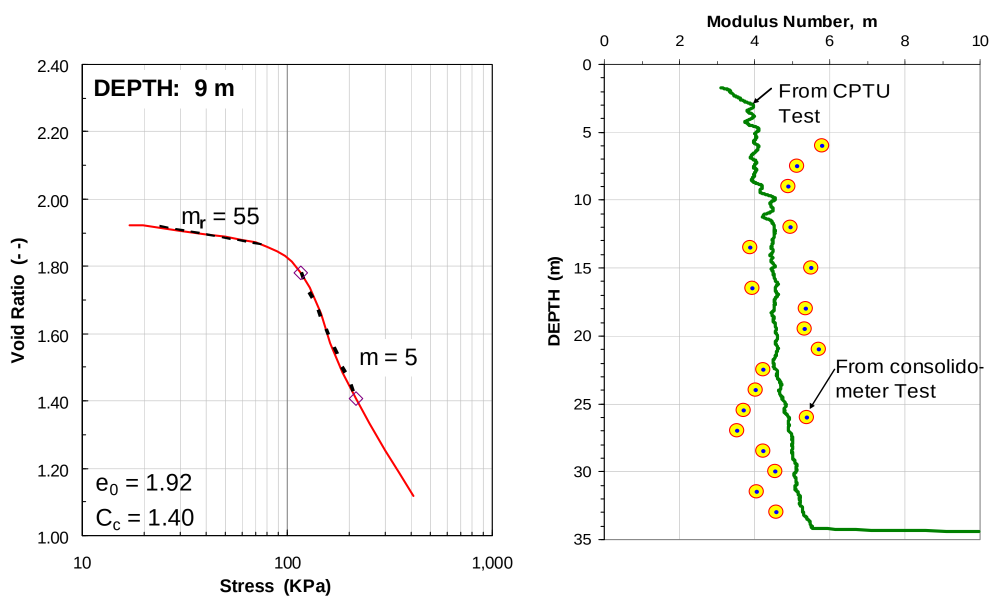

Fig. 3 Typical results of a consolidometer test and distribution of modulus number determined from consolidometer tests and from CPTU cone stress. The depth scale is from original ground surface; Elev. +3.5 m.

DESIGN

The site of the new container facility extends over an 800 m by 600 m area along the Thi Vai River. The highest water level expected at the site is Elev. +4.0 m, which requires raising the ground elevation by about 2 m to Elev. +5.5 m in order to avoid flooding and to create a suitable foundation surface. Because of the thick very compressible clay and silt layer, the fill placed to raise the land will cause significant settlement, which would continue for a very long time. To shorten that time, vertical drains (wick drains) were installed to 37 m depth across the site. Moreover, a temporary surcharge was added raising the surface to Elev. +8 m through Elev. +10 m, i.e., an additional 2.5 m to 4.5 m of fill height. It was expected that if the surcharge was removed when 80 % to 90 % of the consolidation settlements had developed, the thereafter occurring settlement, i.e., the settlement for the finished facility, would be small and acceptable. The required results of the site improvement work was that future ground settlement should not exceed 400 mm over a period of 20 years when considering secondary compression. This limit included the additional consolidation of the clay due to pavement and fill for roadways and loading areas placed in the final stages of the construction.

The main approach to use of the wick drains and preloading site improvement work as applied to the project are illustrated in Figure 4, showing the development of immediate compression and consolidation settlement. The effect of secondary compression is not shown. The figure shows that the amount of fill actually placed will be larger than that indicated by the fill surface elevation, as some of the fill is placed in order to maintain the elevation, i.e., compensate for the settlements.

The wick drain used for the project is a 100 mm wide and 3 mm thick and consists of a corrugated, 0.15 mm thick, plastic core wrapped with a synthetic filter. A photo of the wick drain is shown in Figure 5. The wick drain shown is not robust and could flatten and become squashed at large soil stress, which would impair the flow through the drain, which, in the extreme, could cause it to cease to function. In previous experience of the senior author with similarly designed types of drain, they were not recommended for use to depths larger than 10 m to 15 m including the fill height.

The design assumed that the equivalent cylinder diameter of the drain is equal to that of a circle with the same circumference as the drain (206 mm), i.e., an equivalent diameter of 66 mm. The drain was installed to a depth of 37 m, i.e., into or close to the sand layer below the clay, in a square pattern with a center-to-center distance of about 1.2 m.

To prepare for construction, between in April 2009 and July 2009, an about 1.5 m to 2.0 m thick coarse-grained fill was placed over the original ground level raising the ground level to Elev. +5.3 m. At about the end of September through mid-November, 2009, fill for the temporary surcharge was placed bringing the surface to Elev. +8 m through Elev. +10 m across the site. The surcharge was removed after 8 months, May 20 through June 20, 2010, to a final fill surface at Elev. +5.0 m.

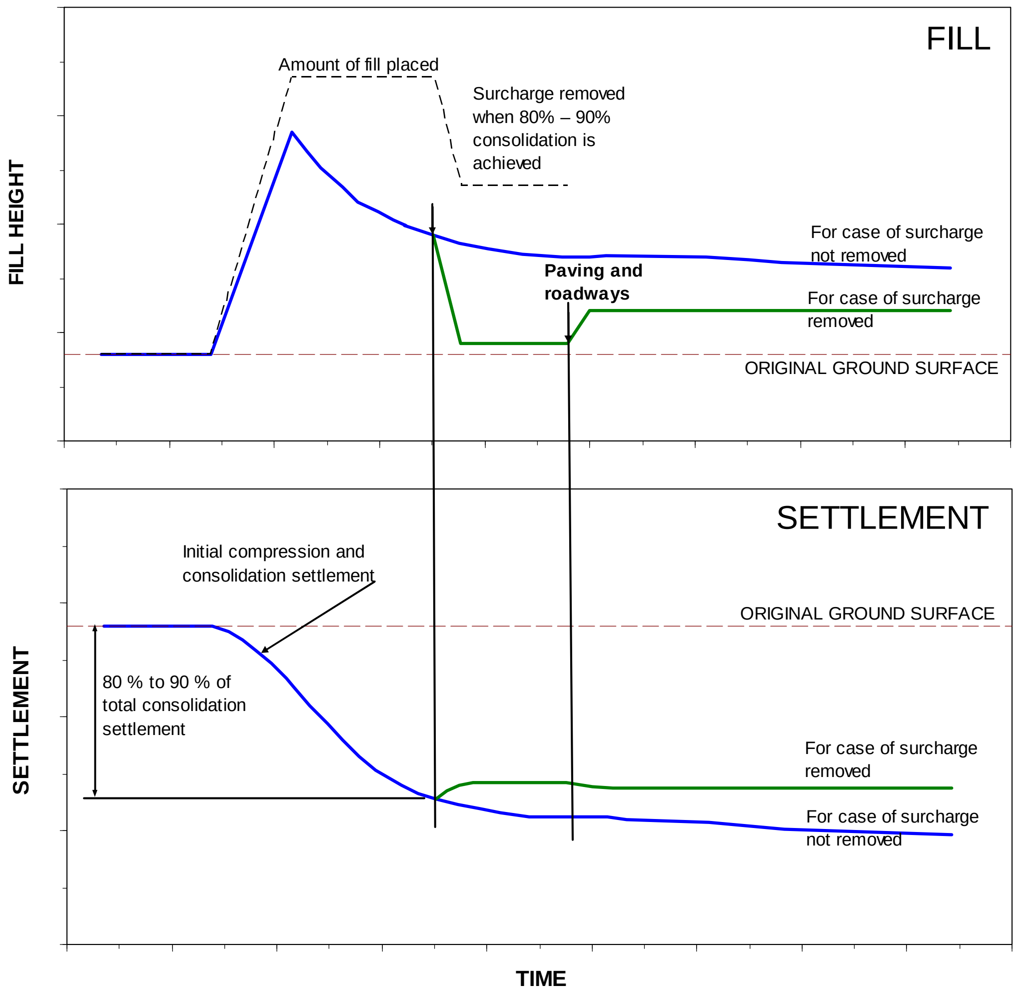

Fig. 4 Principles of wick drain and preloading site improvement work

Fig. 5 Photo of the wick drain used for the project

To monitor the settlement, in February 2009, before the placing of the fill, a large number of settlement benchmarks (SS-plates) were installed on the original ground surface. Pore pressure was measured in three locations (Pz) near the buildings with piezometer tips installed at depths of 5 m, 15 m, and 25 m. Settlement distribution with depth was measured at the same three locations by means of extensometer gages (Ext) placed at 10 m, 20 m, 26 m, and 30 m depths in the clay (Ext-09). Figure 6 shows the layout of the two buildings included in this paper and the layout of the benchmarks etc. within and outside the building footprints.

Piles intended to support the buildings were also installed during mid-November 2010 through end of February 2011, after the temporary surcharge had been removed. The piles were driven precast concrete piles with a square 400 mm cross section made up by 10-m segment spliced in the field by welding. This paper focuses on two buildings (labeled CFS and CG) for which the piles were driven to 28 m and 18 m depth, respectively, starting on December 3, 2010 and finishing on January 24, 2011. The intended working loads to be placed later on were 347 KN for the CFS building and 265 KN for the CG building. Table 1 lists pertinent pile data. The calculated average imposed stress includes the buoyant weight of the piles.

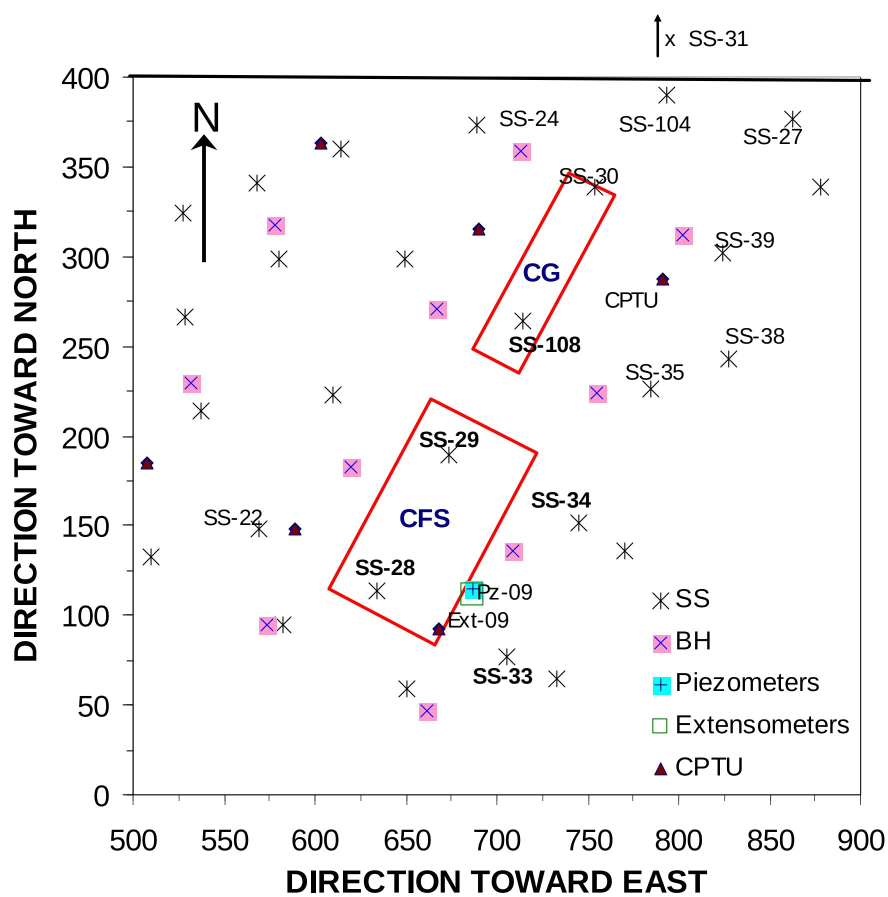

Fig. 6 Locations of CFS and CG buildings and layout of field instrumentation

TABLE 1 Pile data

Building	Area (m^2)	Number of Piles (#)	Average Pile Spacing (m)	Pile Length (m)	Buoyant Pile Weight (KN)	Pile Working Load (KN)	Average Imposed Stress (KPa)
CFS	6,960	747	3.1	28	63	383	48*)
CG	1,072	36	5.5	18	40	265	10**)

*) Includes 5 KPa live load **) Not including stress on ground floor (7 KPa)

MEASUREMENT RESULTS

Figure 7 shows the settlements measured by the SS-plates at or near the buildings. Day 0 is December 1, 2009, the day of the start of the pile driving for the two buildings, labeled CFS and CG. Several SS-plates were damaged or had to be removed during the construction. However, two SS-plates within each building footprint (SS-28 and SS-29, and SS-108 and SS-30) were functioning and several settlement plates outside and near the buildings were also available for continued monitoring. The records show that after removal of the surcharge, the settlement during the next 300 days amounted to about 250 mm, about four to five times more than anticipated. The average settlement at Day 0 (December 1, 2010) was 2.90 m. The total settlement during the consolidation period differs by about 1.0 m between the various SS-plates. The difference is mostly due to the fact that the monitoring started at different times after the fill had been placed near the plate. The trends immediately before and after the pile driving are quite similar. Figure 7 also shows the average settlement of the pile heads in the two buildings superimposed on the settlements measured for SS-28 and SS-108 (discussed in regard to Figure 12).

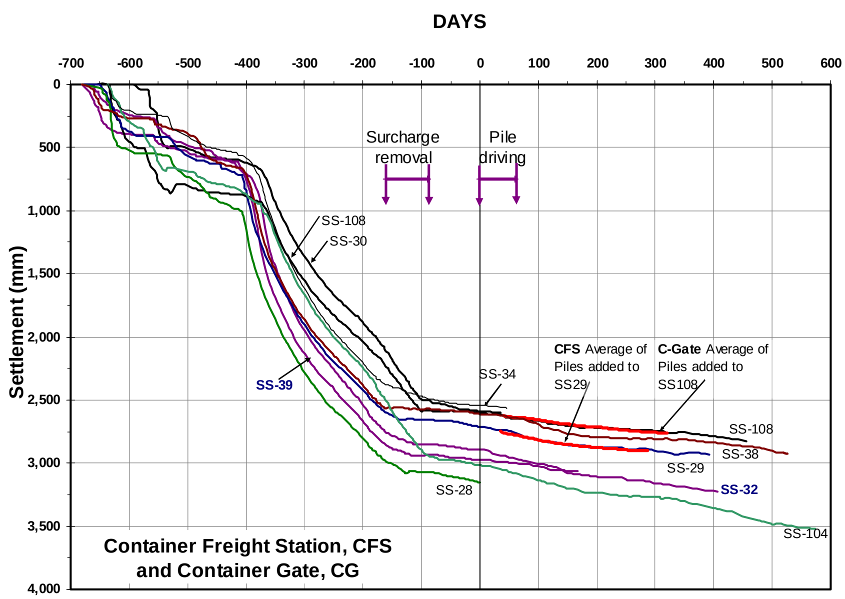

Fig. 7 Ground surface and pile settlements

Figure 8 shows the settlements measured at the plates when normalized to the average settlement of 2,900 mm at Day 0 in order to illustrate the settlement development after the removal of the surcharge. The figure also shows the fill height measured at the two buildings (at SS-28 and SS-29). The dashed line indicted as "as-placed fill height" is the measured fill elevation plus the measured settlement. The volume of soil and fill settling below the water table causes a reduction of the imposed stress, which has to be and has been considered in the settlement analysis.

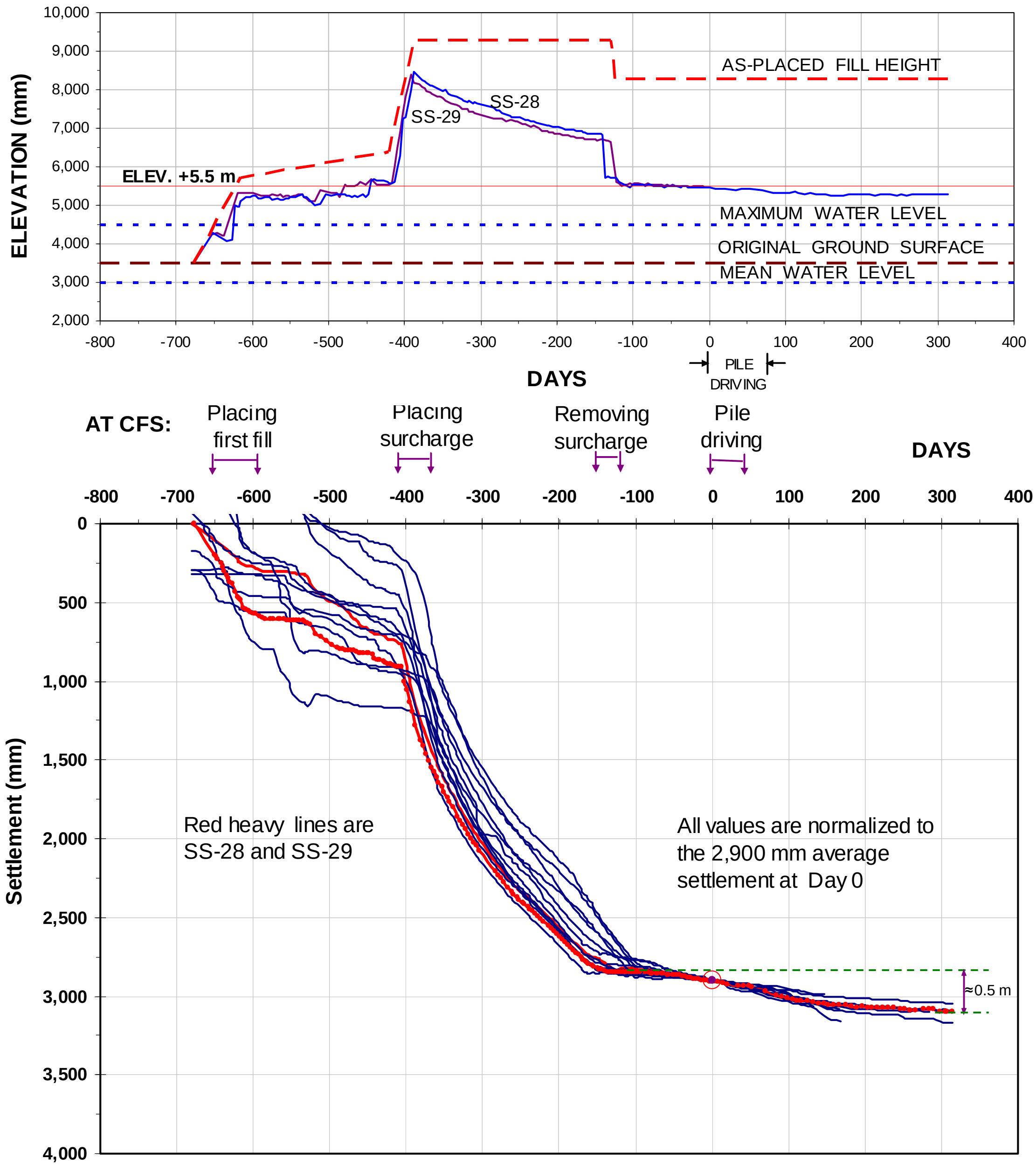

Fig. 8 Fill and surcharge elevation and ground surface settlement

Figure 9 shows the settlement distribution with depth as measured at extensometer station, EXT-09 next to the CFS building at 10, 20, 26, and 30 m depths (original depths) from July 17 through January 14, 2011, i.e., Days -504 through 42, when the Ext-station had to be removed as it was in the way of the pile driving. The four settlements anchors were referenced to the presumed zero for the fifth anchor point placed at 30 m depth. The ground surface settlement (uppermost anchor) was found to agree approximately to the average of SS-28, SS-29, SS-33, and SS-34 for the same time period. At the installation of EXT-09, a ground surface settlement of 0.6 m had already developed. The settlement distribution is almost linear from the fill surface to zero at 30 m depth. The sketch to the right in Figure 9 shows the relative settlement within the anchor points as measured during the last five months, i.e., after the removal of the surcharge. The sketch shows that below about 20 m depth, the ongoing relative settlement is twice to several times larger than above that depth. Evidently, consolidation continued below about 20 m depth after the removal of the surcharge.

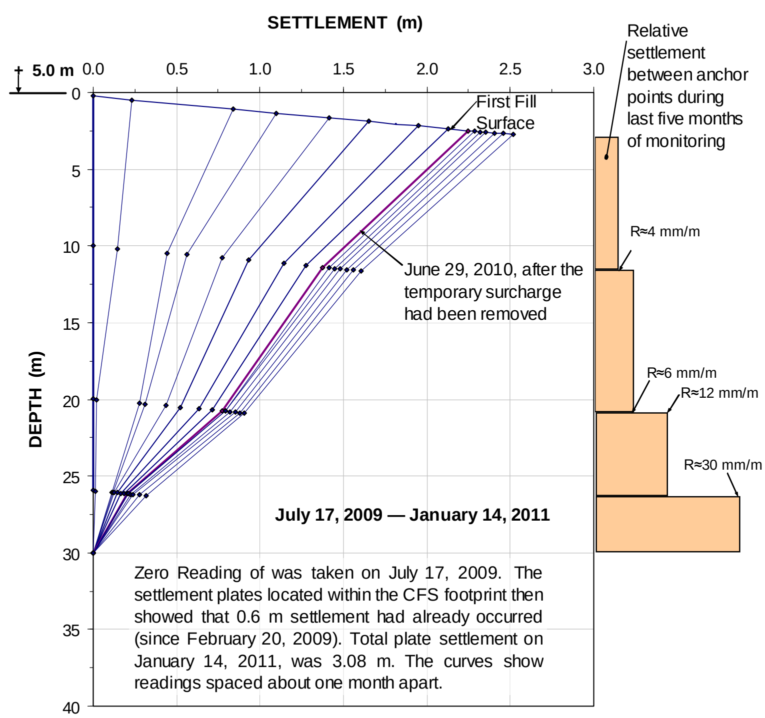

Fig. 9 Distribution of settlement with depth

Figure 10 shows the pore pressures measured at the CFS building at Elev. -0.5 m, -10.0 m, -20.0 m, and -33.0 m from June 24, 2009, through September 17, 2010; Day -527 through Day -77 (until about three months after removing the surcharge). The figure also includes the fill height measured next to the piezometer station. The figure shows that the first fill placement across the site (to Elev. +5.5) only resulted in a modest increase of pore pressure, whereas the subsequent placement of fill to full height gave a distinct pore pressure response. The modest initial response could be interpreted as an effect of a small preconsolidation margin.

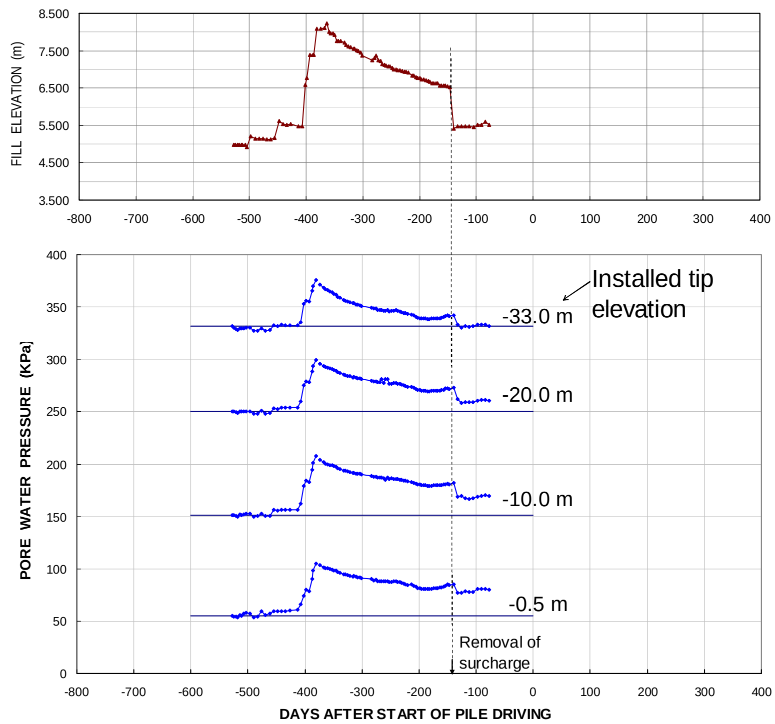

Fig. 10 Fill height and pore pressures measured between Days -527 and -77 at the CFS building

The 5.8 m as-placed fill, which imposed a maximum stress of 100 KPa, resulted in a maximum pore pressure increase of 47 KPa, 50 KPa, 43 KPa, and 37 KPa measured at the four piezometers, respectively. The pore pressures appeared to have stabilized at about Day -200. When the surcharge was removed (about Day -150), the pore pressures reduced somewhat, and after removing the surcharge, the values of remaining pore pressure were 25 KPa, 20 KPa, 10 KPa, and 0 KPa, respectively, corresponding to about 50 %, 40 %, 20 %, and 0 % of the maximum values. The appearance of pore pressures remaining after the removal of the surcharge is due to that the measured pore pressures are referenced to the original piezometer depths. The pore pressures values from 33 m depth were probably affected by the proximity of the sand layer 2 m below and, also, by that the settlement at that depth must have been small. Figure 11 shows the pore pressure distribution versus depth. As seen, when the measured pore pressures are adjusted to the settlement of the piezometer, the measurements indicate that the pore pressures had returned to the original level.

Starting at Day 0, piles were driven to predetermined depths for the buildings. The pile embedment length for the CFS and CG building were 18 m and 28 m, respectively. When all piles had been driven, the elevations of the pile heads were from then onward intermittently monitored. Figure 12 combines the settlement measured by SS-29 and SS-108 located within the footprint of the CFS and CG buildings and the pile head settlement values. First to take note of is that the area continued to settle at a rate much larger than anticipated in the design; second, also

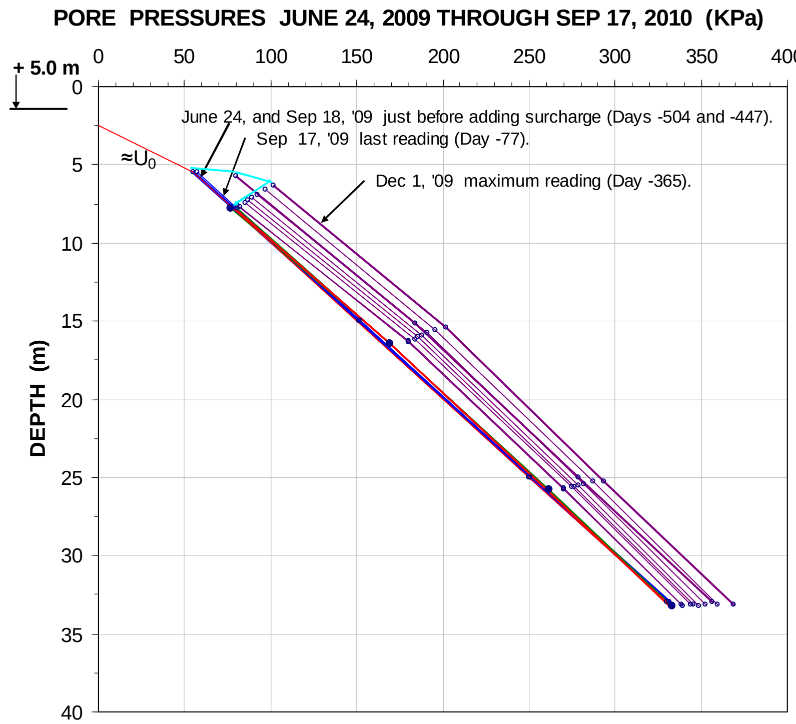

Fig. 11 Pore pressures versus depth

the piles settled and settled at the same rate of settlement as that of the ground surface. This indicates that the settlement occurred in the deep soil layer below the pile neutral plane, which can be estimated to be at somewhere between the depths of 13 m and 20 m for the two pile lengths. The definite conclusion from this indication is that the consolidation was not completed in the lower portion of the profile, as also indicated by the observation about the ongoing consolidation below about 20 m depth (Figure 9). It would seem that the wick drain used was not suitable for the deep installation.

Around end of September, 2010, one pile at each building location was subjected to a static loading test to twice the working load. The piles were not instrumented and the tests included an unloading/reloading event at the intended working load, which makes the test data unsuitable for analysis of load distribution and capacity. A test was recently performed on a same size pile driven to 22 m depth about 6,000 m up the river where the soil is very similar. This pile was tested applying load until plunging failure occurred. A back analysis indicated that the shaft resistance at the maximum test load correlated to beta-coefficients of about 0.3 to 0.35, and the plunging mode of response indicated that the clay provides insignificant toe resistance.

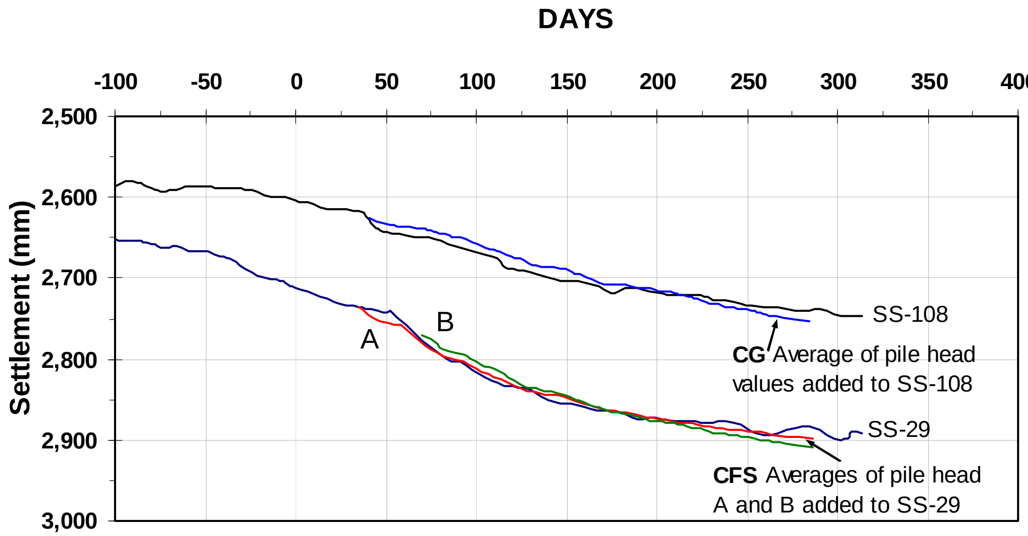

Fig. 12 Settlements of SS-29 and SS-108 located within the footprint of the buildings and the average settlement of the pile heads from end of driving.

BACKGROUND AND INPUT TO ANALYSIS

The analysis of pore pressure dissipation in fine-grained soils (consolidation) and subsequent settlement in the presence of vertical drains applies the theory of Barron (1948) and Kjellman (1948a; 1948b), which is based on radial flow toward a circular drain in the center of a cylinder of homogeneous soil with an impervious outer boundary surface (Hansbo 1960; 1979; 1981; 1994). The theory is summarized in the Kjellman-Barron formula, Eq. 1. The Kjellman-Barron formula is based on the assumption of presence of horizontal (radial) flow only and a homogeneous soil.

$$t = \frac{D^2}{8c_h}[\ln\frac{D}{d} - 0.75]\ln\frac{1}{1-U_h} \tag{1}$$

where t = time from start of consolidation
D = zone of influence of a drain;
D = 1.05 c/c for triangular spacing; 1.13 c/c for square spacing
d = equivalent diameter of a drain
U_h = average degree of consolidation for radial (horizontal) flow
c_h = coefficient of horizontal consolidation

The Kjellman-Barron formula is often supplemented with consideration of smear effect, simultaneous vertical drainage, and non-Darcy flow. However, in view of the uncertainty of the coefficient of consolidation, which at best can only be determined within a factor of about 5, the diminutive vertical direction drainage in a thick clay layer in comparison to the distance between the drains, the uncertainty of the equivalent cylinder diameter, and the disturbance from the installation, applying such refinements are not meaningful for actual cases.

The fixed input to the back-analysis of the settlement measurements to fit the calculated values to the measured values consisted of the mentioned equivalent cylinder diameter (66 mm), the zone of influence (1.3 m), the soil profile, which for the subject case is quite homogeneous, and the loading in the form of the as-placed fill progressively reduced by first the buoyancy effect (as the fill settles below the water table), and finally by the removal of the surcharge. A purpose of the surcharge was to provide a margin for adding new load (stress) to the site. If the full consolidation had occurred prior to the removal, then, the removed surcharge stress is that margin.

The loading input is illustrated in Figure 13, which shows the stress from the measured height of fill (SS-29) and the reduction of the fill height due to the soil and fill settling below the water table vs. months from the project start. The figure shows the correction of imposed stress for the loss of applied stress due to buoyancy. The stress adjusted for this effect is indicated in the figure and approximated to three loading and one unloading occasions.

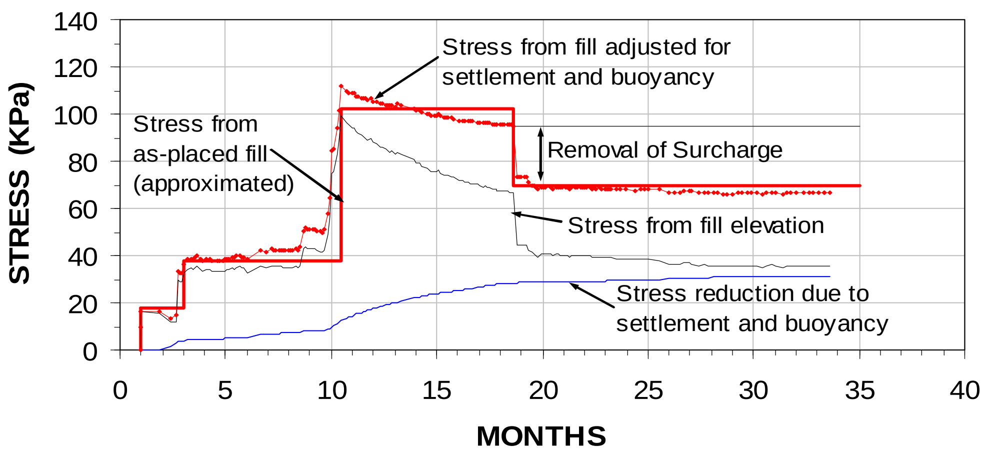

Fig. 13 Actual loading and unloading stresses

The clay is homogeneous and the same set of soil parameters can be applied to represent the full 35 m depth of the clay. For example, the same virgin modulus number (m = 5) was assigned to the entire clay profile. The reloading modulus number (m_r) was assumed to be ten times the virgin modulus number.

The flexible input (in addition to the predetermined input to the calculations in fitting the measured to the calculated settlements) consists of the preconsolidation margins, modulus numbers of immediate compression, and the coefficients of consolidation. Adjustments of these values were made to obtain a reasonable fit both between the settlement values and their development with time. This process "calibrated" the site and wick drain input to the consolidation analysis and allowed an extrapolation calculation to determine the long-term development of settlement at the site for the piled foundations and the effect of the additional fill placed for roadbeds, container stacking yards, and storage areas.

RESULTS OF BACK-CALCULATIONS

The fit between calculated and measured settlements was focused on the settlements measured at a 6 months, 9 months, and 18 months from placing the first fill. The coefficient of consolidation was determined by the condition that 80 % to 90 % consolidation be 9 months for the wick drain installation, which resulted in $c_h = 4.5\ 10^{-8}\ m^2/s$ (= 1.4 m^2/year). Vertical flow of water, c_v, was disregarded.

The calculations were carried out using the software UniSettle4 (Goudreault and Fellenius 2011) with input of stresses as given by Figure 13 and the values presented in Section 5 and Figure 13. Figure 14 shows the results of the calculations. The fit to the settlement measured up to the end of the consolidation period (24 months) is relatively simple. The input values are a modulus number for immediate compression of 150 (j = 1) and a virgin modulus number of 6 (j = 0.5). However, the calculated settlement development only agreed for the first 24 months of consolidation, six months after the removal of the surcharge. Although the calculations indicate that only little settlement should occur beyond the 24 months, significant settlement was indeed occurring.

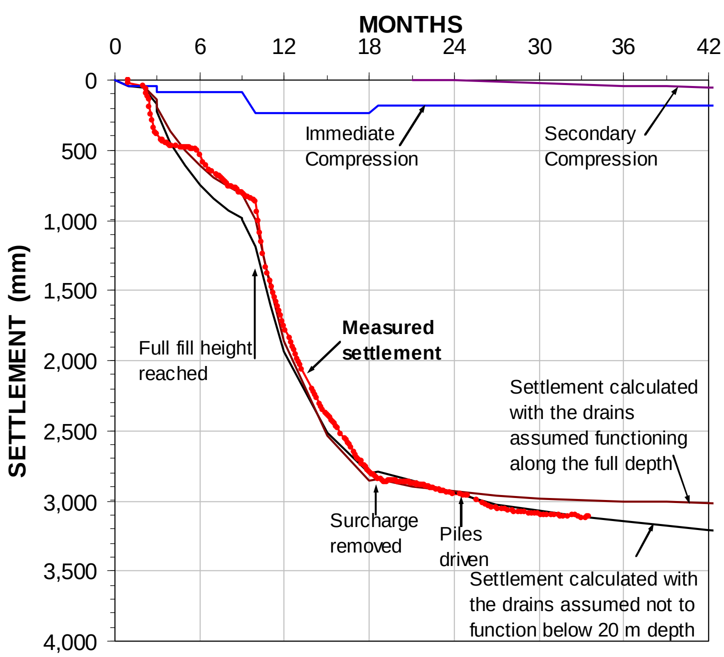

Fig. 14 Results of back-analyses to fit measured settlements

To achieve a fit also to the later development required the assumption that the drains would not work below about 20 m depth, that the consolidation between 20 m and 35 m depths followed vertical drainage, and that the calculation was, therefore, governed by a vertical coefficient of consolidation. Reflecting the supposition that some function of the drains did remain below 20 m depth, the calculation assumed that the time for 80 % to 90 % consolidation would be 20 years for single drainage with a c_v of $30\ 10^{-8}\ m^2/s$ (about 9 m^2/year).

Fitting the delayed settlement in the lower 15 m of the soil profile showed that only about 30 % (300 mm) of the consolidation below 20 m depth would have occurred at 24 months after start as opposed to almost 100 %. That is, most of the consolidation below 20 m depth was still to develop after the surcharge removal. Even more important, the calculations showed that the response to stress from new loads placed on the site would be per virgin conditions and the presence of the drains would cause the renewed settlement to develop over short time. Placing new fill, which will be necessary in order to maintain the minimum surface height, will result in significant additional settlement.

The fit shown in Figure 14 can be further improved by accepting input of values with decimal precision and playing a bit with c_v-value as well as adjusting the lower, non-functioning length of the drain to shorter or longer than 20 m. However, this effort would be just a cosmetic effort and not change the conclusion of the back-analysis that the lower length, seemingly about 15 m, of the wick drains had not functioned as intended, resulting in a smaller than intended level of consolidation when the surcharge was removed. The long-term consequence of this is that when additional fill is placed over the site for constructing road beds and storage areas and restoring the ground to design elevation, additional consolidation settlement will develop, which could adversely affect the project.

After construction of the CFS and CG buildings on the piles as initially driven, the assessment of the long-term conditions of the piled foundations showed that the continued building settlement could exceed 500 mm; well in excess of acceptable values. This has been recognized and the piles for both buildings were lengthened and driven well into the sand layer below the clay to depths of about 40 m to 44 m in order to ensure that the neutral plane would lie below the clay layer and in non-settling soil. Figure 15 shows a view of the CFS building taken toward the CG building before the piles were lengthened. Average pile spacing is 3.1 m.

Fig. 15 View on October 4, 2011, from south end of CFS building showing some of the about 750 piles driven for the CFS building (Authors' photo)

The redriving of the lengthened piles raised concern for the structural integrity of the piles and many piles did indeed break and had to be replaced. However, the driving records do not indicate excessive variation of penetration resistance and the lengthening was considered successful. Moreover, measurements of settlements of the CFS and CG buildings after the completion of the foundation slabs—from approximately Day 450 (March 1, 2012) until completing this manuscript (July, 2012)—showed no larger settlement than commensurate with the increase of axial load in the piles, a few millimetre, only. In contrast, as shown in Figure 16, the ground surface outside the building footprints continued to settle, indeed, the rate of settlement increased as the ground level of the area around the buildings was restored. The SS-29 and SS-108 plates were in the way of the construction and had to be removed. The curves have been extended by the settlement trend measured in the still functioning plates, SS-33 and SS-104.

The pile head elevations were monitored until the pile were to be lengthened and the measurements showed the piles to settle at the same rate as the settlement plates, as indicated in Figure 16. The piles will be subjected to drag loads that are larger than the working loads placed on the piles. However, this is of no concern as the pile is structurally able to resist the resulting axial loads—the dead load from the building plus drag load.

The detailed information on settlement distribution and pile response was not adequate for an analysis that could have shown a shorter and more economical lengthening to be feasible.

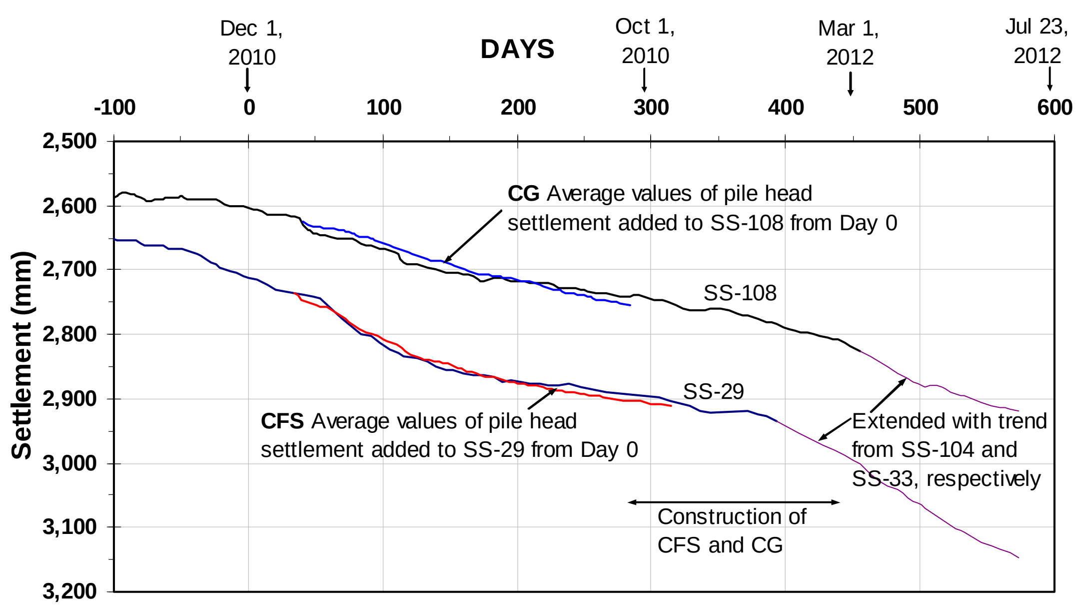

Fig. 16. Settlements measured at the SS-plates near the respective building and settlement of the piles before lengthening

Moreover, neither is the information detailed enough to enable a reliable prediction to be made for the magnitude of settlements to accommodate when fill for pavements and such is placed. High water level would flood some portion of the site if the settlement becomes too large, unless the area elevation is vigorously maintained.

CONCLUSIONS

1. The settlement monitoring indicated that the consolidation of the wick-drain treated site proceeded as designed as to time development and magnitude, and that 80 to 90 % of the consolidation was completed when the temporary surcharge was removed.

2. The monitoring of the settlement continued after the temporary surcharge had been removed and the settlements showed to be significantly larger than predicted in the design.

3. The piles which had been installed six months after the removal of the surcharge, settled, and the settlement monitoring for 10 months showed them to settle practically equal with the ground surface around them. This is a sign that the settlement of the ground occurred below the pile neutral plane, that is, below about 20 m depth.

4. Measured settlement distribution with depth showed that the relative settlement (mm/m) below about 20 m depth was much larger than that above that depth.

5. The observed settlements was fitted to an analysis using horizontal drainage above 20 m depth and vertical drainage below 20 m depth, that is, assuming that the wick drains did not function fully below 20 m depth.

6. It became obvious that piled foundations would experience excessive long-term settlement. The piles were therefore lengthened to penetrate well into the sand layer, i.e., to depths of 40 to 41 m, which ensured that the neutral plane will be in non-settling soil layers.

7. Short time monitoring of the building settlement indicated that the lengthening of the piles had the desired effect of preventing further piles settlement.

8. The monitoring system was designed under the assumption that the wick drain site improvement scheme would be successful and only needed to show when the 80 to 90 % consolidation level had been reached. Had the scheme been successful, no monitoring beyond checking the settlement of the ground surface would have been necessary. However, the design of a monitoring system must address the possibility that the system would not be successful or that some aspect not foreseen could come into the picture and interfere with the scheme. Therefore, a monitoring system needs to include several stations for measuring

settlement and pore pressures with depth and at several depths through the profile and this effort needs to start well before all other activities start and continue throughout the project.

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