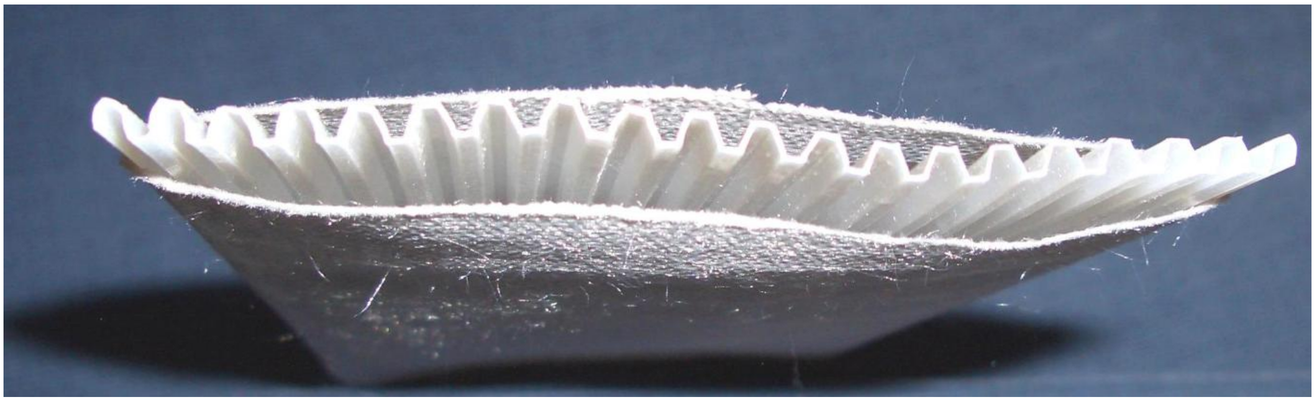

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A case history on use of wick drains and unsuccessful piled foundation

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ABSTRACT

A container terminal, Cai Mep Port in the Mekong delta approximately 80 km southeast of Ho Chi Minh City, Vietnam, was constructed along the Thi Vai River over about 35 m thick soft, deltaic silty clay deposited on dense to compact sand. The port buildings required piled-raft foundations, which comprised 400 mm square, precast concrete piles driven to 18 m through 28 m depth below the original ground level. One building required 760 piles.

The seasonal flooding conditions in the area required raising the ground surface by 2.5 m, which would cause considerable consolidation settlement. To ensure that the settlement would develop before constructing the port facilities and buildings, the consolidation was accelerated by means of wick drains spaced 1.2 m in square configuration and pushed to about 35 m depth. Additional, temporary fill (surcharge) was placed to raise the area by an additional about 7 m. The surcharge was kept on for six to eighteen months at which time the consolidation was considered to be practically completed and, then, as planned, the piles to support the structures were driven.

The settlements amounted to about 3.5 m during the surcharge period. Settlement monitoring, which continued after surcharge removal to final area surface level, showed that the area continued to settle after the removal of the surcharge, indicating that consolidation settlement had not been completed despite the long surcharge period. Indeed, the post-construction settlement over the general port area would exceed the specified limit: 400-mm over a 20-year period. Moreover, which was rather perplexing, the piles were found to settle at the same rate as the ground surface already before the piles received load from the building foundations. Moreover, the continued soil settlement was found to occur mainly below about 20 m depth. The two observations were the key indicators of what had happened at the site: the continued settlement was due to the wick drains not functioning below about 20 m depth.

The problem and its solution were analyzed by means of the Unified Design Method. A remedial procedure was implemented that involved extending the piles to bear in the sand, where no long-term settlement would occur. However, thereafter, it was realized that the continued consolidation below 20 m depth raised the need for additional fill to maintain the ground elevation across the site, resulting in additional and excessive downdrag for the foundations piles and the structures to encounter excessive differential settlement. This caused the project to be abandoned.

INTRODUCTION

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. Construction started in 2008 of the Cai Mep Port container terminal along the Thi Vai River in the Mekong delta (c.f., Figure 1) approximately 80 km southeast of Ho Chi Minh City (Fellenius and Nguyen 2013). The soil profile comprises very soft clay over sand. All structures are placed on piled foundations. The mean water table lies at the ground surface, but it is seasonally above the ground surface—the area then floods. Therefore, the area need to be raised by several metre. This paper describes the design and monitoring of preconstruction fill and surcharge to accelerate settlement with wick drains and reports observations regarding the piled foundations at the site.

SOIL PROFILE

The soil profile at the site consists of 30 to 40 m of compressible 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 %). The figure also shows the grain size distribution and the distribution of cone stress, q_t, 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.

Figure 3 shows a representative CPTU sounding from the site. The cone-stress diagram indicates the soil deposit to be very soft throughout. Vane shear tests, FVT, were also carried out in a few places. The vane shear strength ranged from 10 through 15 kPa at 2 m depth, increasing approximately linearly to about 50 kPa through 80 kPa at 30 m depth. This characterizes the clayey silt as soft to a depth of 20 m and firm below. The correlation coefficient, NKT, between CPTU pore pressure adjusted cone stress and vane shear stress is about 15.

Consolidometer tests showed the soil to be very compressible, as indicated by a Janbu modulus number, *m*, ranging from about 4 through 6 on samples from several depths. The test results showed that the preconsolidation margin was small; the clay is essentially normally consolidated. The reloading modulus number, m_r, was approximately ten times larger than the virgin number, m. Figure 4 shows a void ratio vs. stress diagram on a soil sample from a depth of 9.0 m at the site that is representative for the consolidometer tests at the site. The distribution with depth is shown in Figure 5.

The site of the new container facility extended over an 800 m by 600 m area along the Thi Vai River. The site is subjected to seasonal flooding and the highest water level expected at the site was Elev. +4.0 m, which required raising the ground elevation by about 2 m to Elev. +5.5 m in order 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 (For details of design a wick drain project, see Holtz et al. 1991; 2011 and Fellenius 2025). Moreover, a temporary surcharge was added raising the surface to Elev. +12 m, i.e., by adding an additional 7 m of fill. It was expected that, if the surcharge was removed when about 90 % of the consolidation settlements had developed due to the raised land and surcharge, the thereafter occurring settlement, i.e., the settlement for the finished facility, would be small and acceptable. The specified requirement for the site improvement work was that post-construction settlement of the general port area must not exceed 400 mm over a period of 20 years, when considering potentially continuing consolidation and secondary compression. The 400-mm 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 and for long-term maintenance.

The main approach to using the wick drains and preloading site improvement work, as applied to the project, are illustrated in Figure 6, 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 needed to compensate for the induced settlement.

The wick drain used for the project was a corrugated plastic core, 100 mm wide and 2 mm thick, wrapped with a synthetic filter. Figure 7 shows a photo of the wick drain. The particular wick drain is not robust and could flatten and become squashed at large soil stress, which would impair the flow through the drain. In the extreme, this could cause it to cease to function. In my previous experience with similarly designed types of drain, I found them not suitable for use to deeper embedment than 10 to 15 m including the fill height.

Each drain strip 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. From about the end of September through mid-November, 2009, additional fill was placed bringing the surface to Elev. +8 m through Elev. +10 m across the site. The surcharge fill was removed after 8 months, May 20 through June 20, 2010, to leave a final fill surface at Elev. +5.0 m, 1.5 m above the original ground surface.

Figure 8 shows the layout of the two buildings addressed in this paper and the layout of the benchmarks etc. within and outside the building footprints. 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. Close to the planned location of Building CFS, a piezometer (Pz-09) and a settlement gage (Ext-09) were installed with Pz-tips at depths of 5 m, 15 m, and 25 m and Ext anchors at 10 m, 20 m, 26 m, and 30 m depths.

Piles intended to support the buildings were also installed during mid-November 2010 through February 2011, after the temporary surcharge had been removed (May 20 through June 20, 2010). The piles were driven precast concrete piles with a square 400 mm cross section made up by 10-m segments spliced in the field by welding. This paper focuses on two buildings (labeled CFS and CG or C-Gate) for which the piles were driven to 28 m and 18 m depth below ground level (fill surface), respectively, starting on December 3, 2010 and finishing on January 24, 2011. The intended pile sustained loads (resulting from the later on building construction) were 347 kN/pile for the CFS building and 265 kN/pile for the CG building. Table 1 lists pertinent pile particulars.

Table 1 Pile data

	CFS	CG
Building Area (m^2)	6,960	1,072
Number of piles (#)	747	36
Average Pile c/c (m)	3.1	5.5
Pile Length (m)	28	18
Buoyant Pile Weight	63	40
Sustained Pile Load	383	265

MEASUREMENT RESULTS

Figure 9 shows the settlements measured by the SS-plates at or near the two buildings. The project start—placing the fill—varied across the site. All dates are referred to a Day 0 set to December 1, 2009, the day of the start of the pile driving for the two buildings, labeled CFS and C-Gate.

Several SS-plates were damaged or had to be removed during the construction. However, two SS plates within each building footprint (Plates SS-28 and SS-29, and SS-108 and SS-30, respectively) were functioning and contiguously monitored, as were settlement plates outside and near the buildings. The figure also shows the average settlement of the pile heads in the two buildings as superimposed on the ground settlements measured for SS 28 and SS-108 (these results are discussed in regard to Figure 14).

The total settlement during the consolidation period differed by about 1.0 m between the various SS-plates. The difference was mostly due to the mentioned 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, Day 0, are quite similar, however.

The average settlement at Day 0 was 2.9 m. After removal of the surcharge at Days -160 through -80, within the building footprint, the settlement during the next about 400 days, to about Day 300, amounted to about 250 mm, about four to five times more than anticipated in the design.

Figure 10, lower diagram, shows the settlement development after the removal of the surcharge. The settlements are normalized to the 2.9-m average settlement at Day 0. The upper diagram in Figure 10 shows the fill surface elevation measured at the two buildings (at SS-28 and SS-29). The line labeled as "as placed fill" is the measured surface elevation after settlement. Note, the volume of soil and placed fill settled to below the water table, causing a reduction of the imposed stress, which has been considered in the settlement analysis.

Figure 11 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, 2009, through January 14, 2011, i.e., Day -127 through Day 409, when the Ext-station was removed because it was in the way of the pile driving. The four settlement anchors were referenced to a 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 0.6-m ground-surface settlement 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 (depths) 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 then ongoing relative settlement was twice to several times larger than that above that depth. Evidently, consolidation continued below about 20 m depth after the removal of the surcharge.

Figure 12 shows the pore pressures measured at the CFS building at Elevs. -0.5 m, -10.0 m, -20.0 m, and -33.0 m from June 24, 2009, through September 17, 2010; Day -160 through Day 290 (until about three months after removing the surcharge). The figure also includes the fill height measured next to the piezometer station and 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 a moderating effect of a small preconsolidation margin.

The 5.8 m thick 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 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, i.e., the values disregard the settlement of the piezometer tip. 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 13 shows the pore pressure distribution versus depth. The pore pressures are adjusted to the settlement of the piezometer based on the Ext measurements. From September 17, 2009, 75 days before start of pile driving, the measurements appear to indicate that the pore pressures had returned to the original level and that primary consolidation had been completed. This differs from the indication from the settlement records (Figure 11) and it is possible that the actual settlement of the piezometer tips is smaller than evaluated from the Ext records.

Starting on December 1, 2009 (Day 0), the foundation piles for Building CFS and CG were driven to predetermined pile embedment length for the CFS and CG building, 18 m and 28 m, respectively. 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 made the test data unsuitable for back-analysis. However, a more suitably scheduled static loading 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 0.30 to 0.35, and the plunging mode of response indicated that the clay would only provide insignificant toe resistance.

Both piled foundations are wide. Figure 14 shows a view toward the CG building of the piles driven for the CFS building.

All piles had been driven (Day 41). From then on, the elevations of the pile heads were intermittently monitored. Figure 15 combines the settlement measured by SS-29 and SS-108 located within the footprint of the CFS and CG buildings with the pile head settlement values. The pile head settlements are plotted from setting the first reading equal to the settlement measured at the settlement plates on Day 41. The pile head monitoring had to be terminated when the building construction started. Moreover, 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, outside the building footprints.

First to take note of is that the ground surface continued to settle at a rate much larger than anticipated in the design; second, also the piles settled and settled at the same rate of settlement as that of the ground surface (the SS plates). Particularly the latter was a surprising observation. It indicates that, for Building C-Gate, the settlement must have occurred below the pile toe level, 18 m, and that, for Building C-Gate, the piles (pile-toe at 28 m depth) must have been subjected to downdrag with an equilibrium depth at about 15 to 20 m depth. This is commensurate with the mentioned observation that the consolidation was not completed in the lower portion of the profile (c.f., Figure 11) and supports the conclusion that the wick drain did not function below about 15 depth.

BACKGROUND AND 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 (1947, 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.

$$t = \frac{D^2}{8c_h}\left[\ln\frac{D}{d} - 0.75\right]\ln\frac{1}{1-U_h} \quad (1)$$

where
- t = time from start of consolidation
- D = zone of influence of a drain; = 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

For conventional vertical consolidation, the degree of consolidation and, therefore, the compression is laterally equal. This is not the case for radial flow, however, where the degree of consolidation is largest nearest the drains and smallest at the mid-point between the drains. The vertical compression is therefore not laterally equal and the theory does not fully represent the actual mechanism. However, this is of little practical consequence. Moreover, presence of pervious coarser soil in thin layers, seams, or lenses, improve the function of vertical drains, and such layers often control the pore pressure dissipation rate.

The Kjellman-Barron formula is often supplemented with consideration of smear effect, soil remolding, and non-Darcy flow. However, in view of the uncertainty of the coefficient of consolidation, c_h, which at best can only be determined within a factor of ±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, the disturbance from the installation, and the unequal lateral distribution of the consolidation, applying such refinements are not meaningful in the design of a wick drain installation.

The mathematically equivalent cylinder diameter of the 200-mm drain circumference is smaller than about 20 mm. However, this does not take into account the fact that the soil flowing back after mandrel withdrawal opens up flow-channels around the drain (the insertion mandrel has a much larger cross section than the drain). The actual equivalent cylinder diameter must be determined from back-calculation of actual observations establishing the theoretical width of the equivalent cylinder diameter without modification for soil remolding, etc. Most such back-calculations have shown the diameter to use in wick drain analysis to be 200 to 300 mm. For the subject project, the design assumed that the equivalent cylinder diameter of the drain was equal to 200 mm and I have applied this value to the back-analysis.

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 (200 mm), the zone of influence (1.35 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, then, by the removal of the surcharge. A purpose of the surcharge was to provide a margin for adding new load (stress) to the site. Provided that the full consolidation had occurred prior to the surcharge removal, the removed surcharge stress is that margin.

The loading input is illustrated in Figure 16, 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. The lower diagram shows the settlement measured at settlement plate SS-29 in the CFS building.

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

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 extrapolating the 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.

BACK-CALCULATIONS

The fit between calculated and measured settlements was focused on the settlements measured at 6 months, 9 months, and 18 months after 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 $c_h = 4.5\text{x}10^{-8}$ m^2/s (= 1.4 m^2/year). Vertical flow, c_v, was disregarded.

The calculations and analyses were carried out using the UniSettle software (www.UnisoftGS.com) with input of stress events (c.f. Figure 16). Figure 17 shows the results of the measured settlement versus time and the UniSettle calculated curve fitted to the measured by adjusting input parameters. The fit to the settlement measured up to the end of the consolidation period (24 months) was obtained for input of an immediate compression modulus number of 150 (j = 1) and a consolidation virgin modulus number, m, of 6 (j = 0.5) and reloading (unloading) modulus number, m_r, of 60 (j = 0.5). However, while the input resulted in a fit between calculated and measured settlement development for the first 24 months including six months of monitoring after the removal of the surcharge, the fit was not good beyond this time. The calculations indicated that only little settlement should occur beyond the 24 months, but the plot of the measured settlement shows that significant settlement did indeed occur beyond 24 months.

To achieve a fit also to the later development, required input of the condition that the drains did not work below about 20 m depth and that the consolidation between 20 and 35 m depths followed vertical drainage. The UniSettle calculations for above 20 m depth combined vertical and horizontal consolidation, while, for below 20 m depth, included only c_v, the vertical coefficient of consolidation. Integrating the supposition that some minor amount of drain function 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 $30\text{x}10^{-8}$ m^2/s (about 9 m^2/year).

Fitting to 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 to a large extent be per normal consolidation conditions and the presence of the drains would cause renewed settlement to develop over short time within the upper 20 m. Therefore, placing new fill, which will be necessary in order to maintain the minimum surface height, will result in significant additional settlement, which would adversely affect the project.

The fit shown in Figure 17 can be further improved by input 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.

It is also possible that the drain function also in the upper layers had become compromised due to the large relative compression of the clay and subsequent micro-folding, a development that this particular corrugated drain is susceptible to.

REMEDIATION

At this time in the construction of the port facilities, it became clear that the port grounds would continue to settle and that the piled foundations would settle along with the ground. The calculations indicated that the long-term settlement of the piled foundations of the CFS and CG buildings could exceed 500 mm; well in excess of acceptable values. To alleviate the situation, starting on about October 1, 2011, the piles for both buildings were extended and driven well into the sand layer below the clay to depths of about 40 to 44 m in order to ensure that the equilibrium plane would lie below the clay layer and in non-settling soil. The 13 to 23 m lengthening was obtained by adding a pile segment to each initially installed pile, welding the end plates together, and then driving the pile into the sand below the clay.

Extending the piles and driving them deeper raised concern for the structural integrity of the piles. However, the driving records did not indicate excessive variation of penetration resistance and the lengthening was declared successful. A few piles were subjected to dynamic testing with the Pile Driving Analyzer. The results were considered to show acceptable pile response. However, post-project (2012) CAPWAP analysis of the test records showed low-integrity at different depths in the PDA-tested piles. Indeed, the PDA-tests showed that piles were severely damaged below the original pile toe depth although not all to the extent of a total break.

Figure 18 shows the results of a CAPWAP analysis of a hammer impact on a lengthened pile when at 36 m depth. It is likely that the results are typical of the lengthened piles. The analysis indicates severe damage.

From about Day 500 onward, about 50 days after the end of construction (March 2012), the settlement of the CFS and C Gate buildings was monitored for about nine months. The measured building settlements are shown in Figure 19 as added to the settlement records presented in Figure 15. The average settlement over the nine months amounted to 6 to 11 mm, indicating a successful outcome of lengthening the piles. In contrast, the ground surface outside the building footprints continued to settle, indeed, the rate increased as the ground level of the area around the buildings was prepared and paved.

Moreover, it was obvious that significant settlement will continue to develop over the general container storage area. To maintain the ground elevation and protect the site from the seasonal flooding will require placing additional fill, which will cause additional settlement. Because of the presence of the wick drains, functioning as they are in the upper about 20 m depth, each maintenance adding of fill would add restart consolidation and fairly rapidly, too. Therefore, costly and function disturbing new maintenance work at the site of the port area would be frequently needed.

The indication that the redriving of the lengthened piles severely damaged the piles appears to contradict the observations that the foundation piles ceased to settle with the ground. However, settlement of a wide piled foundation is a function of the compressibility of the soil below the pile toe level (Fellenius 2018). I suspect the reason for the short-term ceasing of building settlement is less the effect of the piles obtaining bearing in the sand below the compressible clay, and more due to the large volume of concrete introduced to the clay, reducing its average compressibility, thus, resulting in reduced compression of the soil and, therefore, reduced foundation settlement.

The Cai Mep container port was never opened.

CONCLUSIONS

1. The settlement monitoring was thought to have 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 equilibrium plane, that is, below about 20 m depth.

4. Measured settlement distribution with depth showed that, at the time of removal of surcharge, the on-going relative settlement (mm/m) below about 20 m depth was much larger than that above that depth.
5. The observed settlements fitted well an analysis using horizontal drainage above 20 m depth (with wick drains functioning) and vertical drainage (wick drains not functioning) 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, to lower the equilibrium plane into non-settling soil layers.
7. Dynamic tests and CAPWAP analyses showed that the piles were severely damaged and broken to the extent that they would provide limited support.
8. Nevertheless, short-time monitoring of the building settlement indicated that the lengthening of the piles had the desired effect of preventing further pile settlement. However, it is likely that the main cause of the reduced settlement of the wide piled foundations was due to the stiffening effect of the pile volume introduced in the soil below the original pile toe level.
9. The settlement continuing outside the building footprints would need to be compensated for by adding fill to avoid flooding. The new fill would cause new settlement which would occur relatively rapidly within the upper 20 m depth, where the wick drains were shown to be working. Need for maintaining the ground level would therefore occur frequently, if not continuously.
10. The monitoring system was designed under the assumption that the wick drain site improvement scheme would be successful and only needed to indicate 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 Cai Mep case history demonstrates that a 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, an up-front special study with closely spaced drains and a surcharge is needed. Moreover, the design needs to include several stations for monitoring settlement and pore pressures at several depths through the profile, which effort needs to start well before all other activities commence and continue throughout the project.

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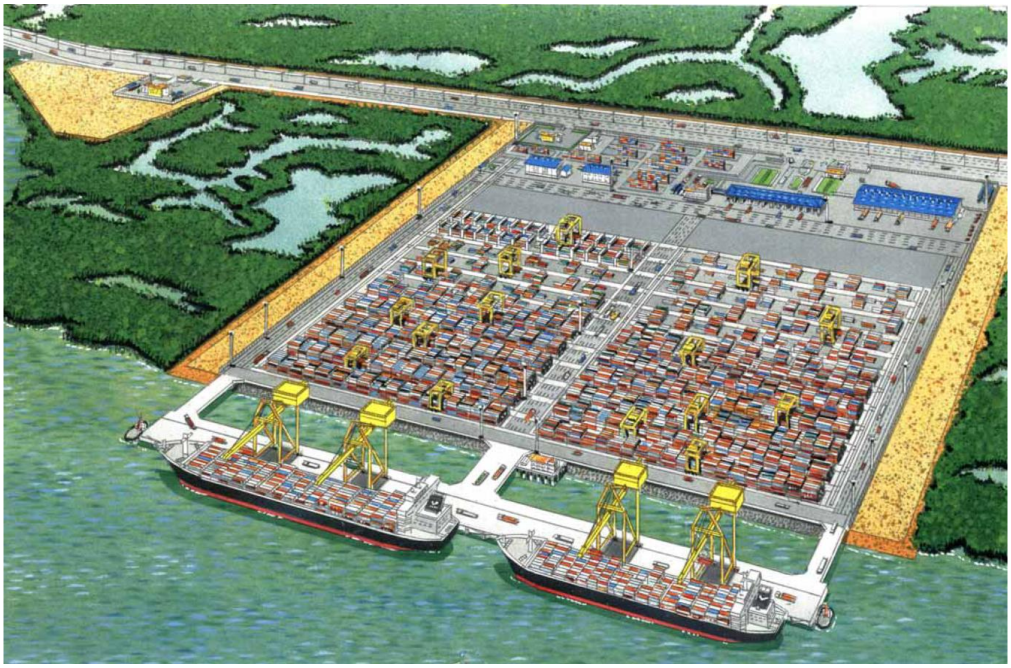

Fig. 1 Artist's view of completed container port

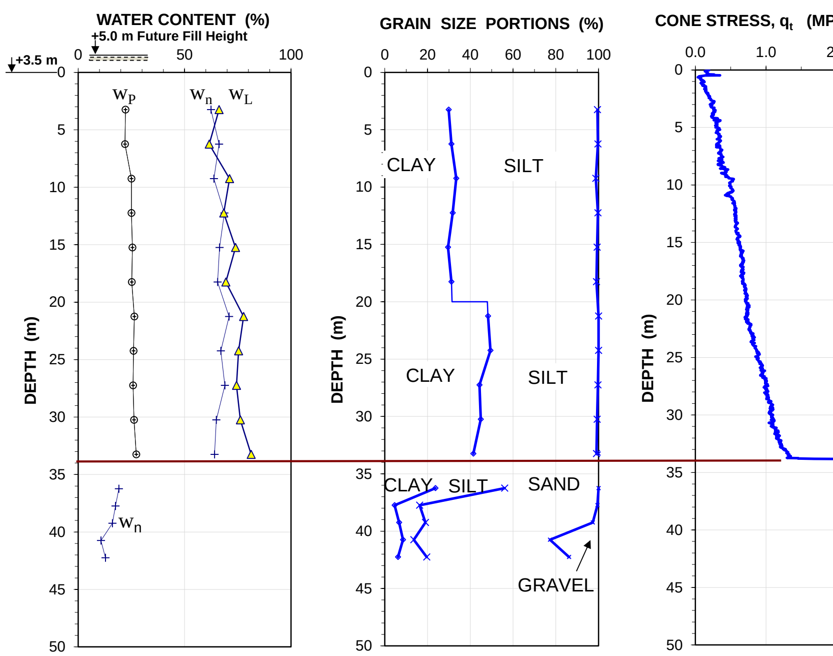

Fig. 2 Consistency limits, grain size distribution, and CPTU cone stress

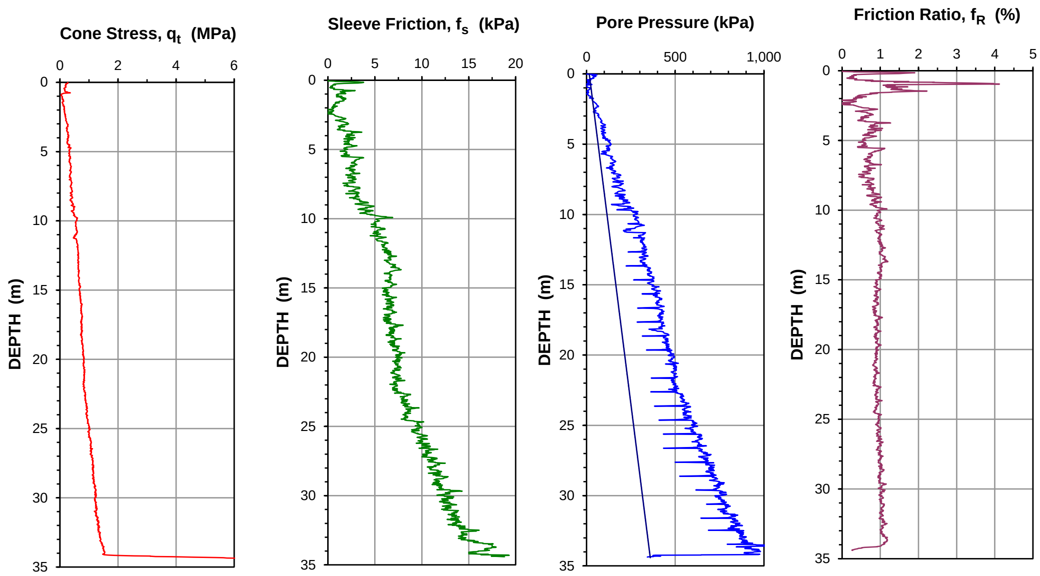

Fig. 3 Results of a CPTU sounding at the site. The spikes occurring when adding the next rod have not been removed (visible in the pore pressure diagram).

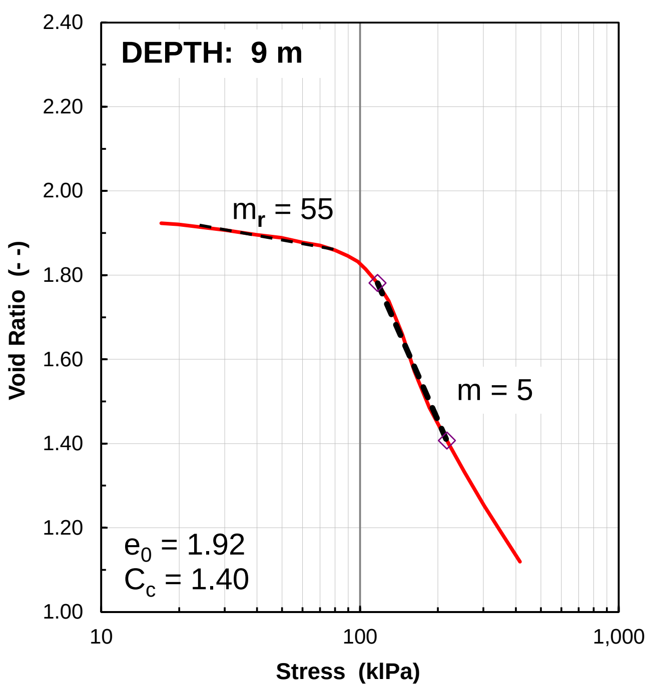

Fig. 4 Results of a consolidometer test from 9 m depth

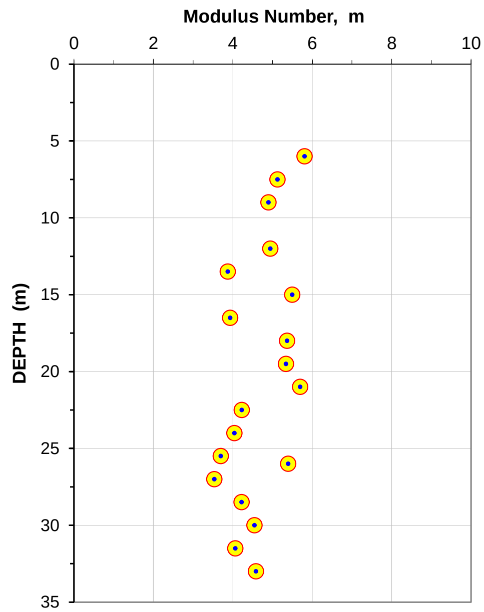

Fig. 5 Distribution of modulus number determined from consolidometer tests. The depth reference is from original ground, Elev. +3.5 m.

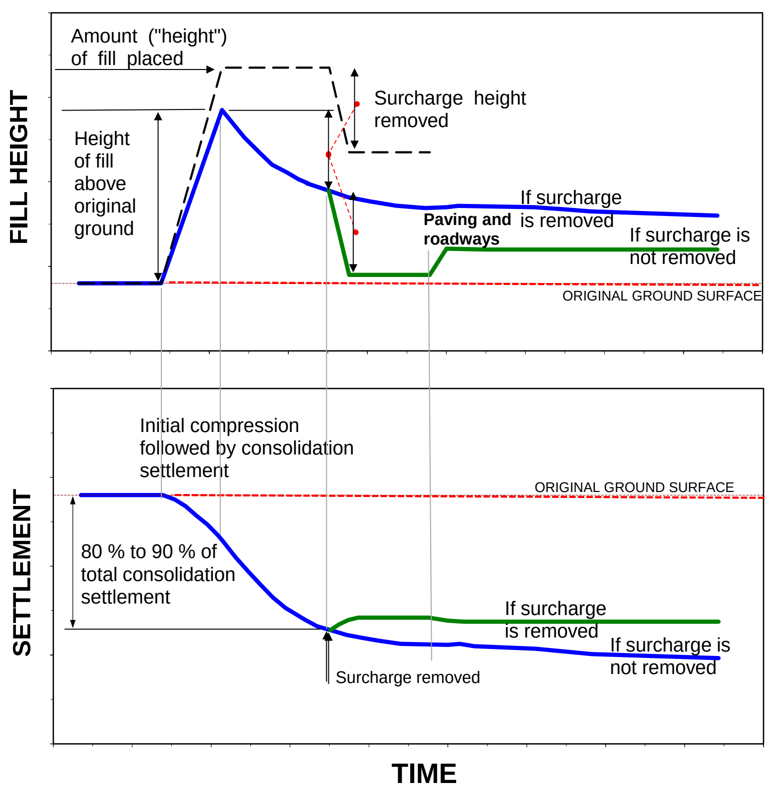

Fig. 6 Principles of wick drain and preloading in site improvement work

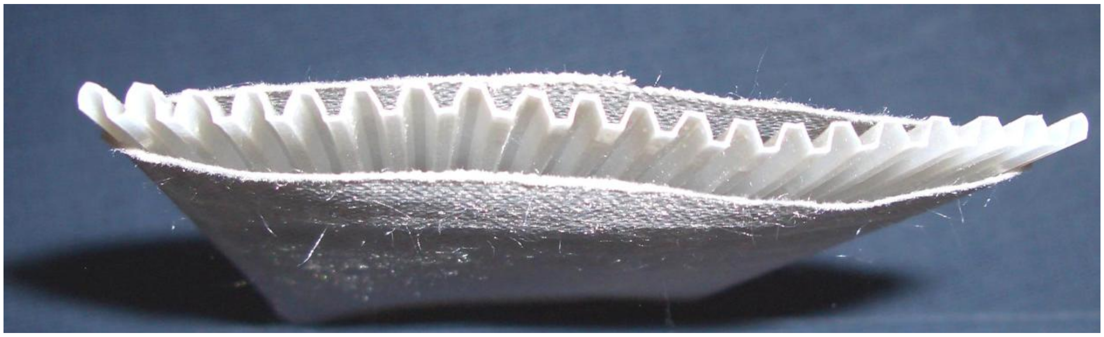

Fig. 7 Photo of the wick drain used for the project

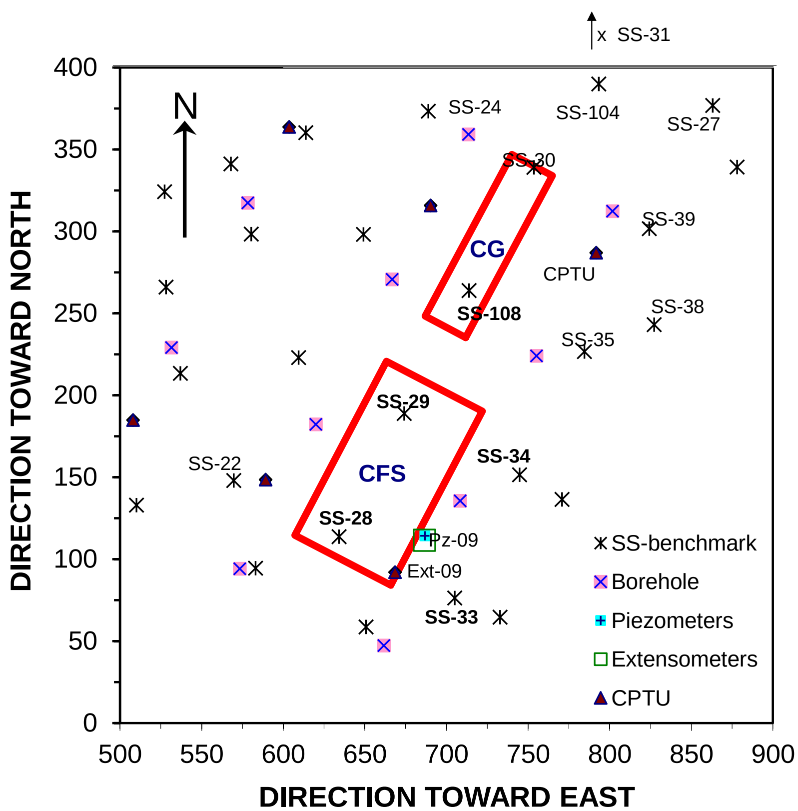

Fig. 8 Locations of CFS and CG buildings and layout of field instrumentation

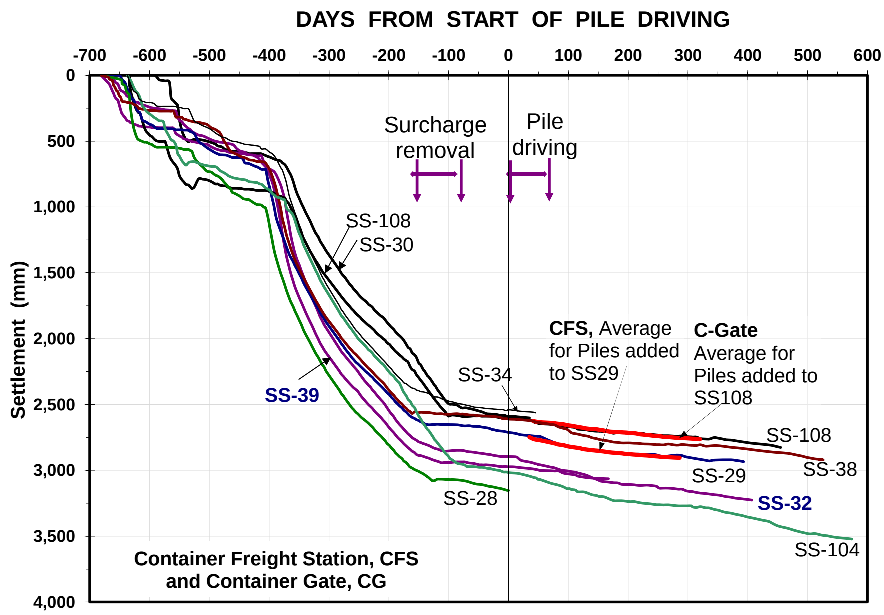

Fig. 9 Settlement of ground surface and piles

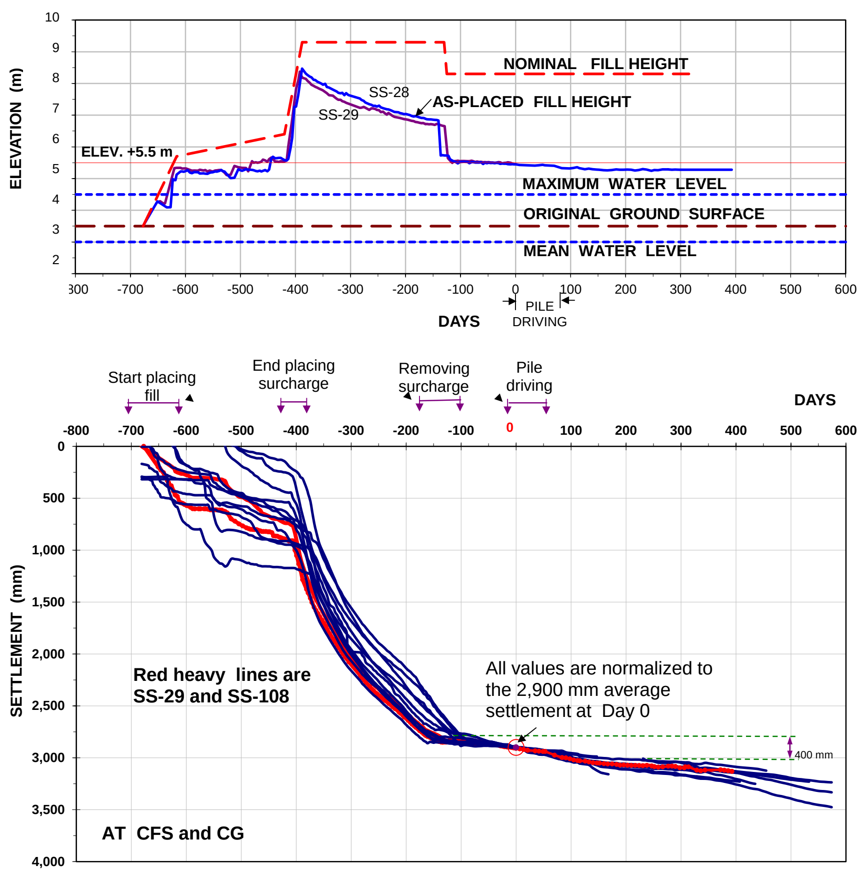

Fig. 10 Normalized ground surface settlement

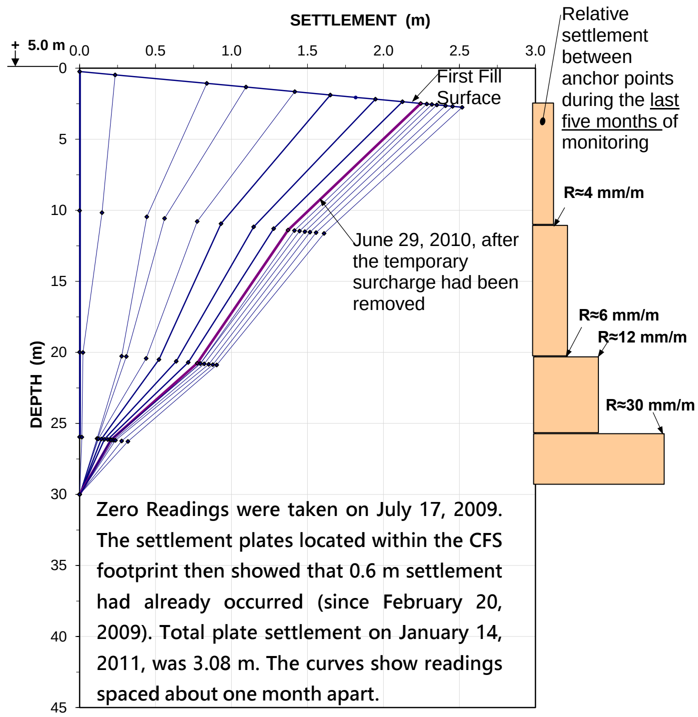

Fig. 11 Settlement with depth July 17, 2009 through January 14, 2011

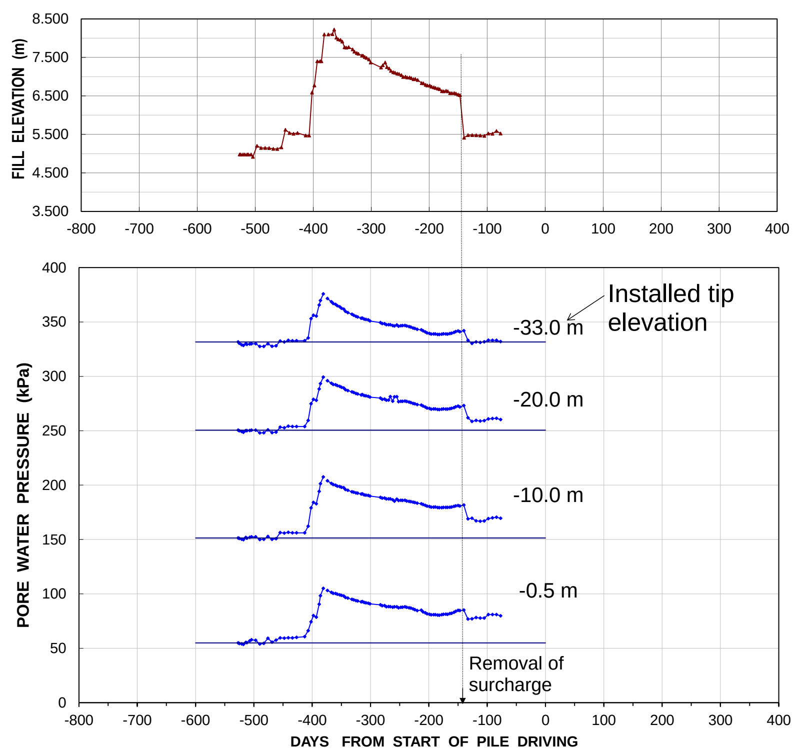

Fig. 12 Fill height and pore pressures measured between Days -527 and -77 at the CFS building

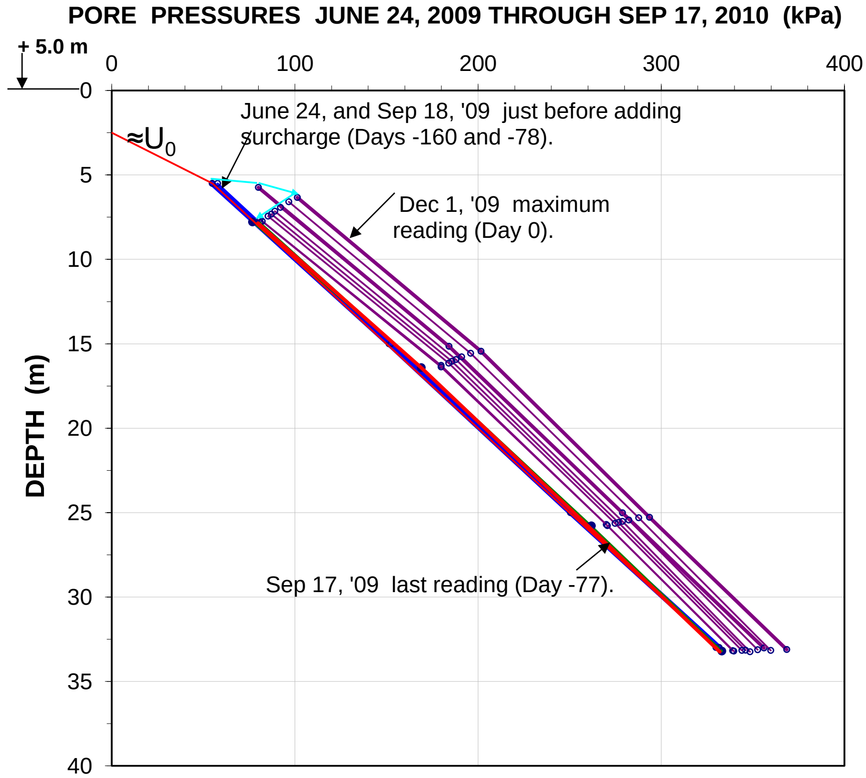

Fig. 13 Pore pressures versus depth with the piezometer tip adjusted to settlement

Fig. 14 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).

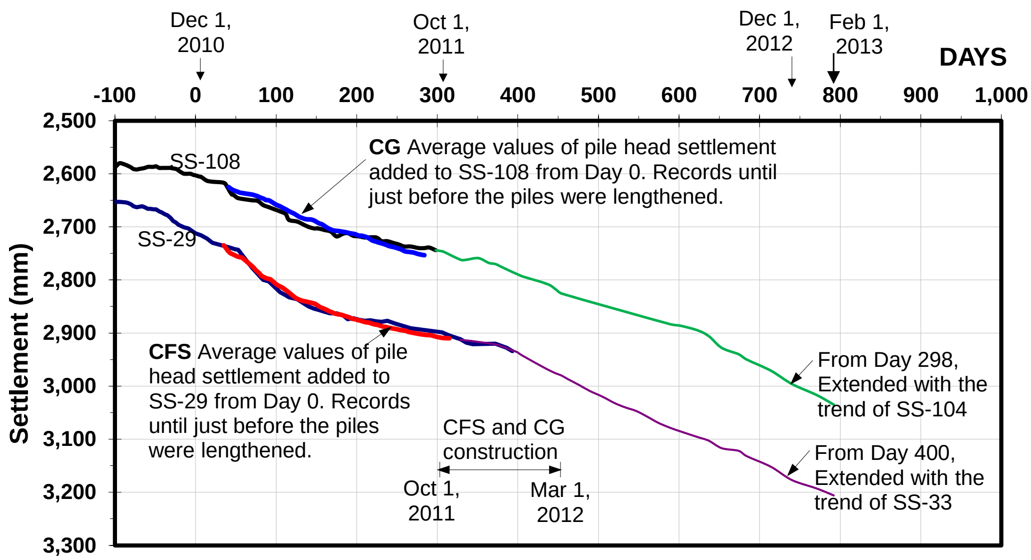

Fig. 15 Settlements of SS-29 and SS-108 located within the footprint of the buildings and the average settlement of the pile heads from end of driving.

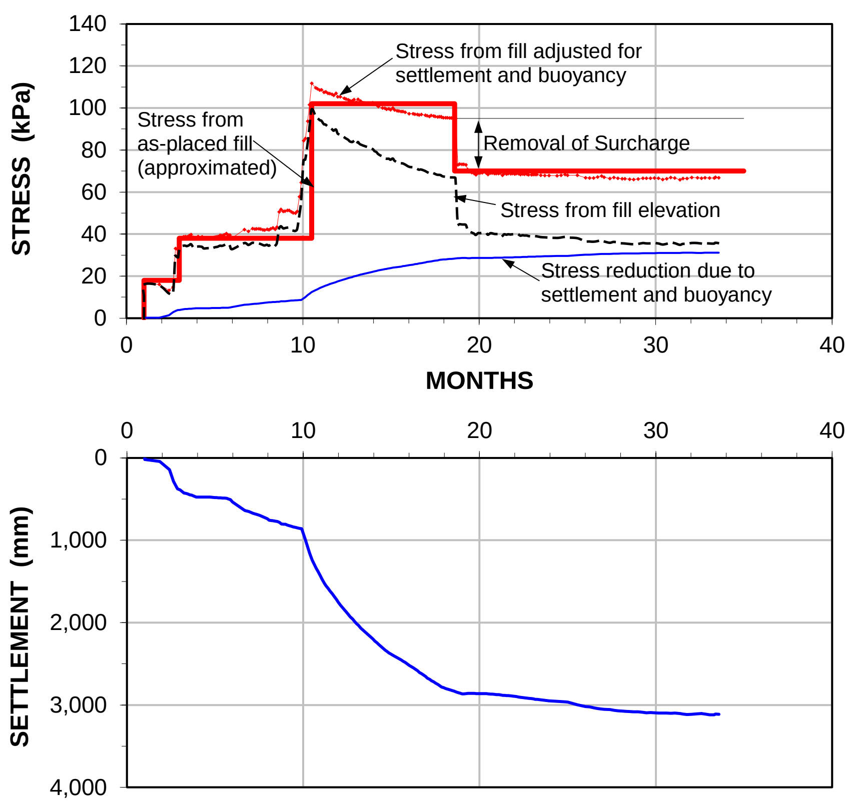

Fig. 16 Actual loading and unloading stress and measured settlement vs. time

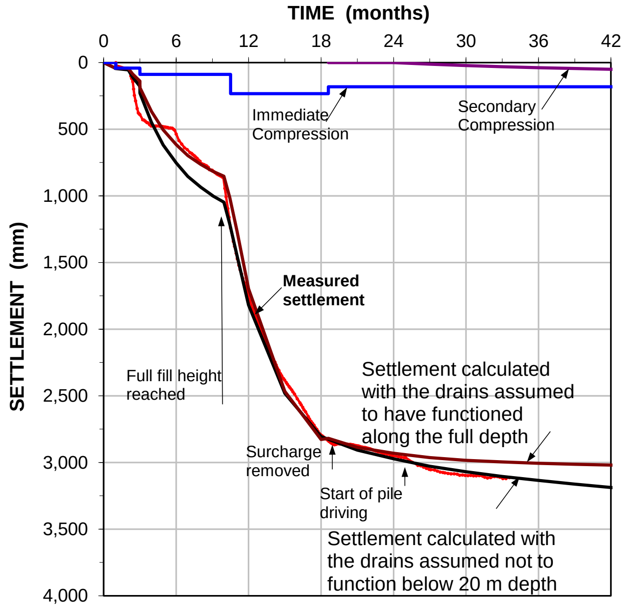

Fig. 17 Results of UniSettle back-analyses fitted to measured settlements

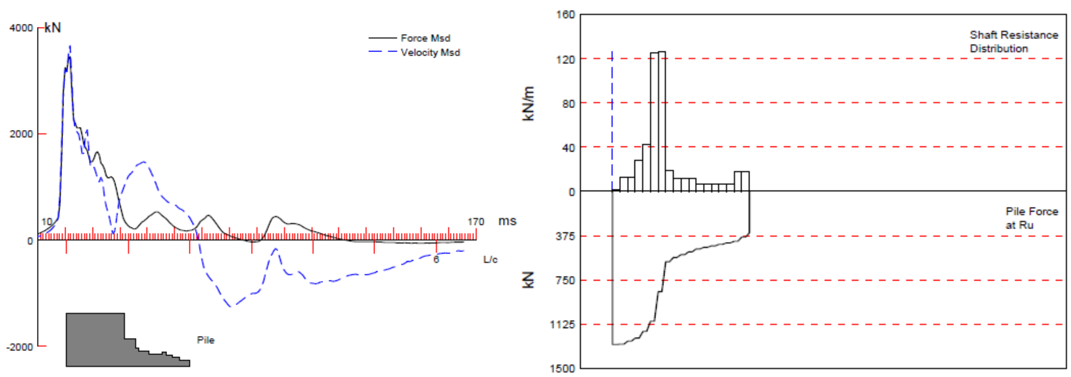

Fig. 18. Results of CAPWAP analysis on a lengthened pile. Courtesy of SACL Ltd., Ottawa)

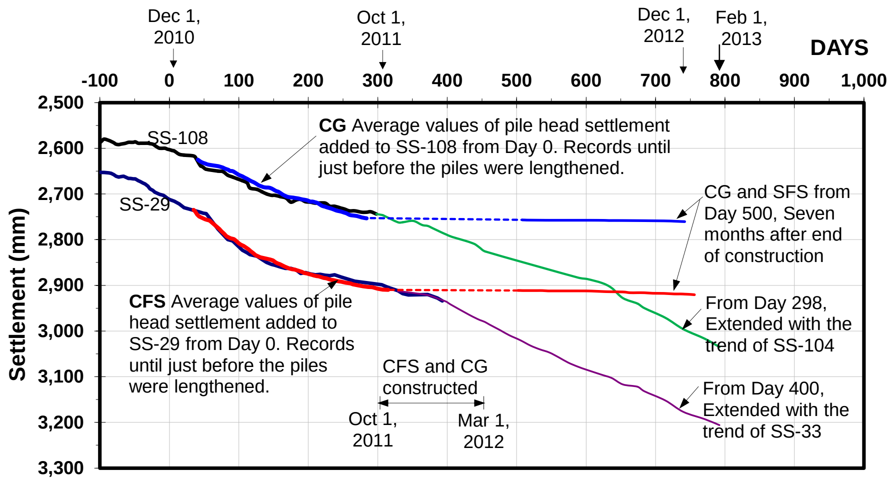

Fig. 19 Figure 15 with the results of monitoring the settlement of the C-Gate and CFS buildings (starting two months after end of construction)