Ground Improvement by Optimized Preload Program

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ABSTRACT

Preloading of sites underlain by compressible soils is a well-established site development procedure to reduce post-construction settlements of structures supported on shallow foundations. Considerable reduction in the preload duration can often be achieved by installation of vertical drains. It is desirable to extend vertical drains down through the compressible soils impacted by the preload and terminate the drains within relatively incompressible soils. If this cannot be achieved, then the potential exists for unacceptable post-construction differential settlements due to greater settlement below the center of the building than along its edges. If no relatively incompressible soils exist below the compressible soils impacted by the preload, the length of the vertical drains may be increased along the perimeter to achieve similar preload settlements throughout the site due to lesser increases in soil stresses along the preload edges. The site development loads at the subject site, comprising fill required to raise site grades by about 4.0 m and building loads, would induce consolidation below the maximum depth reachable with conventional wick drain mandrels. These loads would induce the greatest post-construction building settlements near the center of the site due to a smaller increase in soil stresses along the perimeter of the site. Hence, increasing the post-construction settlements along the site perimeter, relative to those below the center of the development, would reduce the post-construction differential building settlements. In an attempt to reduce the post-construction differential building settlements, the wick drains at the subject site were installed to three depths ranging between 25 m and 35 m with the depth reducing towards the building perimeter.

This paper will briefly present the results of geotechnical site investigations and the inferred subsurface conditions, which will be followed by a discussion on the preload design and performance. Detailed monitoring was carried out of surface settlement gauges and of deep settlement gauges installed to 43 m depth, in addition to monitoring of pneumatic and standpipe piezometers. The results of the instrumentation monitoring will be presented to assess the conformance of actual preload settlements with those predicted.

INTRODUCTION

Rapid site development was required to meet the Owner's stringent deadlines for construction of a 21,400 m^2 warehouse building. The subject site is located in Pitt Meadows, a suburban area approximately 30 km east of Vancouver in the province of British Columbia, Canada, as shown on Figure 1.

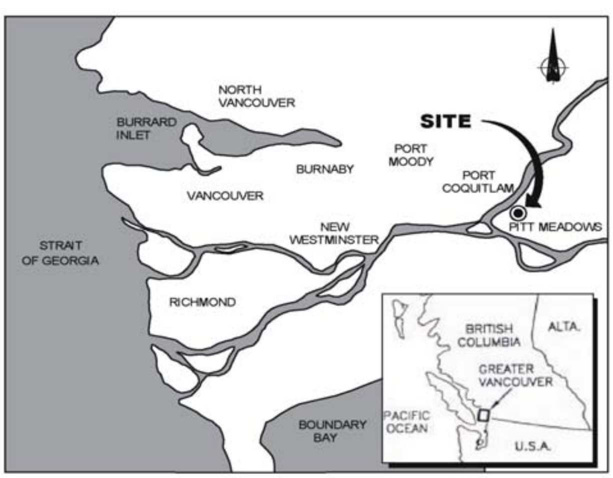


Fig. 1. Approximate site location.

The proposed development on the vacant site included construction of the warehouse building and surrounding paved areas to provide truck access. A relatively heavy uniform permanent slab load of about 25 kPa was anticipated for the 2-storey high building. Flood protection requirements included fill placement from existing site grades at about El. 1.0 m to El. 5.3 m for the building slab, which required permanent fill placement to approximately El. 4.0 m within the surrounding paved areas. The development plan included future building expansions of about 9,500 m^2 and 4,700 m^2 immediately to the west and east of the subject warehouse building, respectively.

SUBSURFACE CONDITIONS

Information on the soil conditions existed from previous geotechnical investigations completed within or adjacent to the building site. This included 24 solid stem auger drill holes advanced to maximum 30 m depth and 15 electronic Cone Penetration Tests (CPTs) hydraulically pushed to maximum 50 m depth. Previous laboratory testing included moisture content determinations, Atterberg Limits, grain size analysis and one-dimensional consolidation tests on undisturbed soil samples. In addition, a previous test preload and a construction preload had been completed immediately adjacent to the site, which included monitoring of instruments facilitating back-analysis to assess in-situ consolidation properties. In general, the inferred soil conditions consisted of an upper zone containing compressible normally consolidated soft to firm silt deposits with some clay to clayey and loose to compact sand deposits, underlain by a lower zone (below approximately El. -20 m) of normally consolidated compressible soft to firm clayey silt. The thickness of the sand deposits in the upper zone increased significantly towards east and south, which resulted in negligible silt zones above El. –20 m at the southeast corner of the proposed building. CPTs advanced within and immediately adjacent to the building site indicated that the lower clayey silt zone extended to minimum El. –50 m (i.e. firm bearing stratum at depth was not encountered in the test holes). Organic silt and/or peat zones with moisture contents between 100% and 200% were occasionally encountered in the test holes. The total thickness of these discontinuous organic zones were typically 0.3 m to 0.5 m and located between El. –1 m and El. –4 m. A simplified cross-section of the inferred soil conditions within the building footprint and future building expansion areas is shown on Figure 2. Measurements in standpipe piezometers and CPT pore pressure dissipation tests indicated hydrostatic groundwater conditions with the groundwater level at about El. –1.0 m to El. 1.0 m.

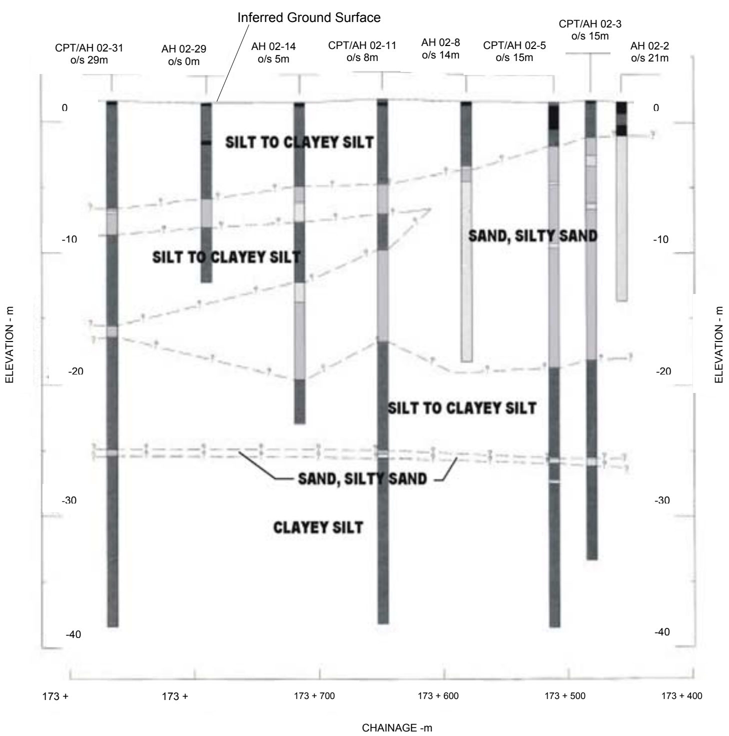


Fig. 2 Inferred soil conditions (from west to east).

In-situ data collected from CPTs advanced into the lower clayey silt zone and data recorded from deep settlement gauges installed within the construction preload indicated the lower clayey silt zone was relatively homogenous, unlike the upper zone. Based on the laboratory results, the preload data recorded at the deep settlement gauges, and empirical correlations using soil index parameters, it was inferred that the compression index, C_c, and in-situ void ratio, e_0, of the clayey silt in the lower zone were about 0.4 and 1.1, respectively. Laboratory consolidation data and back-analysis of monitoring data from the construction preload indicated a vertical consolidation coefficient, C_v, of about 0.3 to 0.5 m^2/month for stress conditions similar to the lower silt at depths of about 20 to 35 m. CPT pore pressure dissipation tests conducted at about El. –30 m to El. -35 m indicated a horizontal consolidation coefficient, C_h, in the range of 1.8 to 2.2 m^2/month.

GEOTECHNICAL EVALUATION

Geotechnical concerns associated with the proposed development included post-construction consolidation settlements (total and differential) of compressible soils in the upper and lower zones due to the required fill placement and the permanent design slab load. Also, liquefaction susceptibility due to seismic loading conditions of the saturated granular soils in the upper zone needed to be assessed.

Static loads induced by placement of about 4.3 m of fill and permanent building loads would result in consolidation of the compressible silt deposits in the upper and lower zones. Monitoring data from previous preloads in the vicinity of the subject site indicated that primary consolidation of the silt within the upper and lower zones could occur (in the absence of wick drains) over a period of several years and several decades, respectively. Considerable differential settlements would occur due to the variability of the thickness of the compressible soils in the upper zone and due to a smaller increase in soil stresses along the building perimeter than below the building center. If the warehouse building was supported on relatively shallow spread footings, then preloading of the building footprint could reduce, but not eliminate, post-construction building settlements. Additionally, the preload duration could be significantly reduced by installation of vertical drains such as wick drains.

Seismic loading conditions for the site are specified by the provincial building code as an earthquake with a 1 in 475 year probability, which is a M-7 (Richter scale) earthquake inducing a peak horizontal bedrock acceleration of 0.21g. Soil amplification studies previously completed in the general area of the site indicated that peak horizontal ground surface acceleration could be of the order of about 0.28 g. Liquefaction analyses were carried out using the CPT data and the empirical method based on work by Seed (Youd et al., 2001). The results of these analyses generally indicated low to very low risk of liquefaction from El. –1 m to El. –6 m and medium to high risk in discontinuous zones between about El. -6 m and El. –18 m. Post-earthquake vertical total settlements due to soil liquefaction were estimated to be of the order of 125 mm to 175 mm based on published volumetric strain relationships (Tokimatsu et al., 1987; Ishihara, 1992). Observations from past earthquakes have indicated that surface manifestations of liquefaction will not be significant for relatively light structures supported on shallow foundations provided these are underlain by a sufficient thickness of non-liquefiable soil over liquefaction susceptible soils (Ishihara, 1985). Relatively shallow footings supporting the subject warehouse building would be underlain by about 8 m or more of non-liquefiable soil, which was judged to be sufficient to limit the impact of a 1 in 475 year earthquake to structural damage and no structural collapse as required by the provincial building code.

PRELOAD DESIGN

The objective of the preload was to reduce post-construction building settlements to acceptable levels over a preload period of no more than 6 months. Post-construction building settlements due to primary consolidation of soils within the upper zone could be eliminated by a sufficiently high preload and installation of vertical drains at a minimum spacing. However, post-construction settlements due to primary consolidation of the lower clayey silt zone would occur due to a limitation depth of 35 m by conventional wick drain mandrels. Thus, post-construction building settlements would occur due to on-going primary consolidation of soils below the wick drains and secondary consolidation of all soils below the footings.

The assessment of wick drain spacing was based on achieving about 75% to 85% primary consolidation (under the preload's imposed stresses) within the soils surrounding the wick drains during the preload period. The wick drain spacing was governed by the slower consolidation rate of the lower soil zone surrounding the wick drains, since the consolidation rates of the upper cohesive soils and granular soils were considerably higher. The design of the wick drain spacing was based on a horizontal consolidation coefficient, C_h, of 0.5 m^2/month. This resulted in a wick drain triangular spacing of 2.0 m c/c, which was expected to result in the required preload consolidation in maximum 6 months even if the actual consolidation rate would be slightly lower than assumed.

Analyses were carried out for different preload heights to assess post-construction building settlements using Terzaghi's one-dimensional consolidation theory. For a preload constructed to 4 m above slab elevation, the post-construction settlements due to primary consolidation below 35 m deep wick drains were estimated to be of the order of about 180 mm near the building center. Including the impact of secondary consolidation of soils below the footings, it was estimated that post-construction total settlements could be of the order of 200 to 300 mm over 30 years.

It was estimated that at 100% completion of primary consolidation, the soil stresses would increase by about 100 kPa near the building center at a depth of about 30 m due to the required raising of site grades and the 25 kPa design slab load. This increase in soil stresses at about 30 m depth would be less towards the building perimeter and it was estimated that the minimum soil stress increase would be about 60 kPa at the building corners. Hence, post-construction settlements due to primary consolidation of soils below a uniform wick drain length would be less along the building perimeter than the center. However, the differential post-construction settlements could be reduced by increasing the total post-construction settlements along the building perimeter relative to those below the building's center. This could be achieved by reducing the wick drain length along the building perimeter, which would increase post-construction settlements by an amount equal to the reduction in settlements occurring during the preload period. Settlement analyses indicated that the post-construction differential settlements could be reduced to acceptable levels by decreasing the wick drain length from 35 m at the building center to 25 m along the building perimeter. An outline of the building footprint and the variable wick drain lengths are shown on Figure 3 with the wick drains extending 10 m beyond the building. The design of the variable wick drain lengths included the variability in soil stress increases due to general raising of site grades around the building to El. 4.0 m, future preloading on the east and west sides of the building for expansions, and a lower preload height at El. 7.4 m south of the building for paved loading dock area.

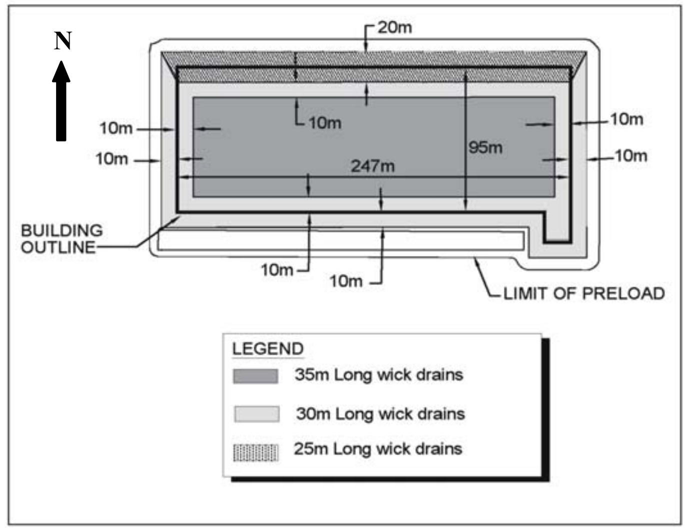


Fig. 3 Building outline with variable wick drain lengths spaced triangular at 2.0 m c/c.

It was estimated that differential post-construction settlements over 30 years could result in general building rotations up to about 0.15% with potential rotations of about 0.3 % to 0.4% occurring in localized areas due to the presence of the discontinuous organic soils. Rotation of about 0.15% is generally acceptable for most types of warehouse buildings, whereas rotations of 0.3% to 0.4% may require incorporation of structural measures to accommodate the settlements without damaging the building.

The above long-term post-construction settlement predictions were based on the soil properties previously presented in this paper. Based on these assumed soil properties and the soil stratigraphy, the settlement analyses indicated that settlements during a 6 month preload period would be about 80 to 100 mm for soils below 35 m deep wick drains near the building center. Similar preload settlements were predicted for soils below the wick drains installed within the remaining building portions as indicated in Figure 4.

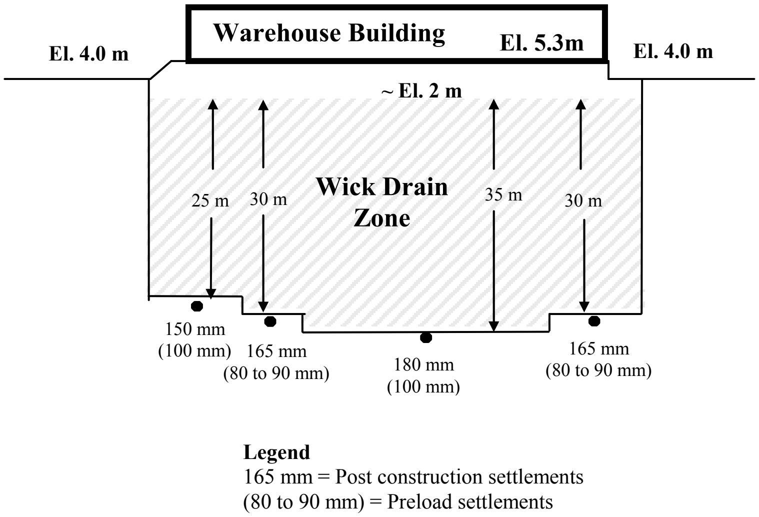


Fig. 4 Predicted primary settlements below wick drains with base of model assumed at El. –70 m (view from north to south).

INSTRUMENTATION MONITORING

A relatively thin working platform of hydraulically placed sand fill was completed mid 2002 to result in general site grades between approximately El. 1.5 m and El. 2.0 m. Installation of wick drains conforming to the design shown on Figure 3 was completed in August 2002. Hydraulic placement of sand fill continued in September 2002 to raise site grades and to construct the preload. The fill was placed in several stages in different areas and was essentially completed by the end of October 2002 with minor fill placement in early November 2002.

Most of the instruments to monitor the preload progress were installed immediately after completion of wick drain installation. The instruments within the building preload comprised 75 surface settlement gauges, 8 deep settlement gauges, 11 standpipe piezometers and 15 pneumatic piezometers. The deep settlement gauges (Reed Switch gauges) were installed to about El. –42 m with monitoring points at approximately every 1.5 m depth interval. The piezometers were installed to different depths allowing measurement of pore pressures to a maximum depth of El. –40 m. The following summarizes the recorded preload monitoring data with the last survey completed at the end of April 2003.

Surface Settlement Gauges (SGs)

The surface settlement gauges were generally located in a 25 m by 25 m grid. The total settlements recorded during the preload period varied from approximately 600 mm at the east building edge to about 1800 mm at the west building edge. Based on the rectangular hyperbola method to assess ultimate primary consolidation settlements (Sridharan et al, 1987), these total preload settlements indicated about 80% to 90% completion of primary consolidation under the imposed preload stresses. The increase in effective soil stresses due to the preload was greater at the locations with less than 1800 mm of preload settlements, since the height of preload to be removed increased as the preload settlements decreased. Typical data recorded at a surface settlement gauge located near the building center is shown on Figure 5.

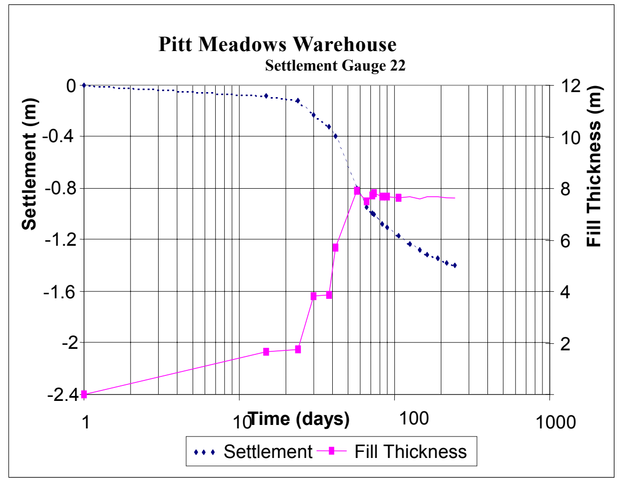


Fig. 5 Settlement and fill height at surface settlement gauge.

Deep Settlement Gauges (DSGs)

The deep settlement gauges were located at strategic locations including near the center, corners and perimeter of the proposed building footprint. Each deep settlement gauge was located in plan immediately adjacent to a surface settlement gauge to allow for comparison of total settlements. Minor corrections to the DSG data were occasionally carried out, when the incremental cumulative total settlement increase between two survey dates was significantly different than the total settlement increase recorded at the adjacent surface gauge over the same period. These offsets to the DSG data were obvious and could have been caused by any of the many possible errors associated with collecting deep settlement data. The differences between total settlements recorded at the surface settlement gauge and the cumulative total settlement recorded at the adjacent deep settlement gauge were generally less than 50 mm over the same time period, which generally extended from beginning of September 2002 to end of April 2003. These differences were likely associated with:

- survey accuracy;
- settlement of soil between the elevation of the base plate for the surface settlement gauge and the elevation of the upper survey point of the adjacent deep settlement gauge;
- inaccurate adjustments during data processing required due to bent surface settlement gauge pipe and/or deep settlement gauge casing.

A summary of the DSG data and a comparison to total settlement recorded at the adjacent SG are given in Table 1, which includes preload settlements recorded at DSGs below wick drains.

Table 1 Summary of DSG data and comparison of total preload settlements at adjacent surface settlement gauges.

Location	Wick drain length	Total Settlements DSG	SG	Settlements below drains
DSG 02-7	30 m	1360 mm	1419 mm	100 mm
DSG 02-8	35 m	1412 mm	1463 mm	100 mm
DSG 02-11	35 m	883 mm	875 mm	75 mm
DSG 02-12	30 m	1346 mm	1296 mm	110 mm
DSG 02-13	30 m	858 mm	804 mm	100 mm
DSG 02-14	25 m	965 mm	935 mm	100 mm
DSG 02-15	25 m	563 mm	629 mm	90 mm
DSG 02-16	30 m	484 mm	481 mm	90 mm

The DSG data indicated that soils up to about 2 m below the wick drains were influenced by the wicks in terms of an accelerated consolidation rate. The preload settlements below the wick drains presented in Table 1 exclude the settlements that occurred in this transition zone.

An example of DSG data is shown on Figure 6, which is for data recorded between August 12, 2002 and April 30, 2003.

As the DSG data in Figure 6 shows, the incremental settlement rates were negligible between monitoring points for sequential surveys completed towards May 2003. The incremental settlement rates between monitoring points generally spaced approximately 1.5 m apart were maximum 2 to 3 mm from April to May 2003 for all the DSG's. Considering survey accuracy and settlements due to secondary consolidation, it was judged that primary consolidation for soils surrounding the wick drains were practically 100% complete at the end of April 2003.

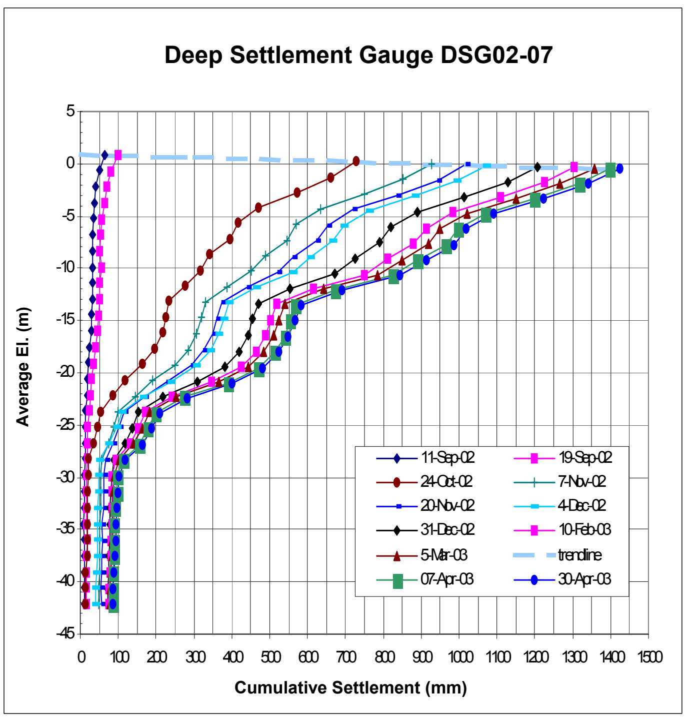


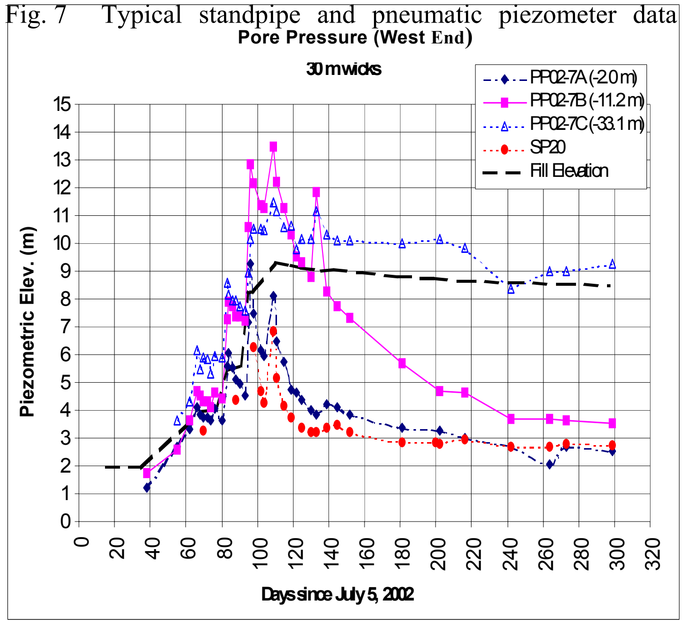
Fig. 6 Recorded DSG data in area with 30 m deep wick drains.

Standpipe and Pneumatic Piezometers (SPs and PPs)

The 11 standpipe piezometers were spaced relatively evenly within the proposed building footprint and slotted within the upper granular fill to provide information about the elevation of the groundwater table throughout the site. This information was used in conjunction with pneumatic piezometer data to assess the amount of excess pore pressure dissipation occurring during the preload period at nearby pneumatic piezometers. The pneumatic piezometers were located at depths between El. –2.0 m to El. -39.1 m.

Typical piezometric data recorded at PPs and at an adjacent SP located near the middle of the west building edge is shown in Figure 7, which assumes hydrostatic groundwater conditions as encountered during the site investigations. Surveys of the PP instruments stick-up were carried out to correct the PP filter elevation occurring due to settlement of the instruments. The installation elevations of the PPs are shown in the legend on Figure 7.

Fig. 7 Typical standpipe and pneumatic piezometer data



recorded from August 12, 2002 to April 30, 2003.

One PP of the 15 installed stop functioning relatively early in the preload period. Nine of the remaining 14 PPs were located at elevations above the bottom of the wick drains. Data recorded at eight of these PPs indicated 90% or more dissipation of the excess pore pressure induced by the preload and about 80% dissipation at one location. Measurement errors and/or instrumentation problems may have caused the lower indicated dissipation at this one location.

As expected, the data recorded at the 5 PPs located below the bottom of the wick drains all indicated a much lower dissipation rate as shown in Table 2.

Table 2 Dissipation rate for PPs below the wick drains at end of April 2003.

Piezometer	Depth below wick drains	Dissipation Rate
PP02-7C	~ 5 m	26 %
PP02-8C	~ 6 m	20 %
PP02-13	~ 5 m	28 %
PP02-14	~ 8 m	16 %
PP02-15	~ 10 m	36 %

Except for data collected at PP02-15, the data recorded at the relatively limited PPs located below the wick drains indicates a dissipation rate of about 20 to 25% approximately 5 m below the wick drains with a possible decrease in dissipation rate with depth as would be expected.

DISCUSSION

The geotechnical evaluation for the proposed development on the subject site identified post-construction settlements to be the primary concern, particularly those of the differential variety. Prediction of settlements comes inevitably with a certain amount of uncertainty, which is a function of the suitability of the settlement model used in the settlement analyses and of the variability and complexity of subsoil conditions relative to the idealizations that are required for modeling purposes. Input parameters to the model (i.e. soil stratigraphy and soil properties) were well defined for the subject site. In addition, the model was based on Terzaghi's consolidation theory, which has for several decades proven useful in assessment of settlements. However, as a well known quote by Terzaghi states, "mother nature has no contract with mathematics…she has even less of an obligation to laboratory test procedures and results", it is critical to confirm predictions of such an important issue as post-construction settlements, especially when post-construction settlements are predicted to be considerable as at the subject site.

Locally, preloading is routinely used as a ground improvement method to reduce post-construction settlements to tolerable levels. Experience has shown that limiting preload instrumentation to surface settlement gauges is typically sufficient to assess the preload duration. However, additional preload instrumentation was required at the subject site, given the optimized nature of the preload and its objectives, to confirm a satisfactory preload program and to verify the suitability of the settlement model as a means of better predicting the post-construction total and differential settlements.

The data recorded at the surface settlement gauges confirmed an anticipated total settlement trend with preload settlements decreasing towards the southeast corner of the proposed building footprint. The surface settlement gauge data indicated that the average primary consolidation of soils below the gauges were about 80% to 90% under the stresses imposed by the preload. Since the average consolidation is about 80% to 90% and the consolidation of soils below the wick drains will be less, this suggests that the soils in the wick drain zone are consolidated at least by 80% to 90%. This is equivalent to effective soil stress increases of approximately 110 to 125 kPa (for 80% to 90% consolidation, respectively) below the building center using a unit weight of 16.5 kN/m^3 for the about 8.5 m combined permanent and preload fill height. This is similar to the maximum increase in effective soil stresses due to the permanent raising of site grades (about 5.5 m of fill required at maximum preload settlement locations) and the design slab load of 25 kPa. Hence, while 80% to 90% consolidation was achieved under the preload stresses, essentially 100% primary consolidation was achieved under the imposed stresses of the completed site development.

The deep settlement gauge data indicated total cumulative settlements very similar to the total settlements recorded at the adjacent surface settlement gauge as shown in Table 1. In addition, the deep settlement gauge data indicated preload settlements below the wick drains in a narrow range from 75 mm to 110 mm. This range agrees well with the predicted range of 80 to 100 mm. Sensitivity analyses were completed in the preload design phase to assess the consolidation properties of the soil below the wick drains. Sensitivity analyses were again completed at the end of the preload period, which indicated a relatively limited range of the consolidation properties of soils below the wick drains was required to achieve conformance between predicted and recorded preload settlements. Thus, the deep settlement gauge data served to validate the settlement model even though the accuracy is rather difficult to assess for the minor preload settlements occurring over a short period. Such validation increases the confidence in the estimates of post-construction settlements. However, it should be noted that the settlement model would predict identical settlements as those recorded at the nearby construction preload, which remained in place for about 2 years.

The standpipe piezometers indicated a general increase in the groundwater table by almost 1 m throughout the site occurring over the preload period.

The pneumatic piezometers located within the wicked zone indicated dissipation of excess pore pressure induced by fill placement required to raise site grades and to construct the preload was generally 90% or more. This dissipation rate agrees very well with the interpretations based on the surface settlement gauge data and the deep settlement gauge data.

Settlement analyses completed in the preload design phase indicated preload settlements and post-construction settlements due to primary consolidation of soils below the wick drains in the order of 80 to 100 mm and about 180 mm, respectively. Hence, the average degree of consolidation of soils below the wick drains would be in the order of 30 to 35% at the end of the preload period. This is slightly above the approximately 20 to 25% degree of consolidation indicated by data recorded at pneumatic piezometers located below the wick drains. Experience by the authors has indicated that the consolidation rate indicated by pneumatic piezometers can sometimes lag the consolidation rate indicated by settlement gauges.

CONCLUSION

Preloading is a well-established ground improvement method to reduce post-construction settlements for structures founded on relatively shallow foundations. Reduction of the preload period can be accomplished by installation of wick drains. For the subject site, the wick drain lengths were varied in an attempt to optimize the impact of the preload program (i.e. reduce post-construction differential building settlements to acceptable levels). This resulted in wick drain lengths of 35 m within the building center, which reduced to minimum 25 m along one of the building perimeters. Monitoring data collected during the preload period confirmed the predicted impact of the preload designed with an optimized wick drain configuration. Reducing the post-construction total settlements to an estimated range of about 200 to 300 mm over 30 years is expected to result in general building rotations of about 0.15% with building rotations of about 0.3% to 0.4% in localized areas. Building rotations of this magnitude caused by post-construction differential settlements may result in negligible to minor structural distress to a relatively flexible building structure.

However, it became apparent following installation of wick drains and placement of preload fill that the predicted post-construction building rotations would likely not be acceptable due to stringent requirements for satisfactory operation of the warehouse. This resulted in the structural designer selecting a raft foundation instead of conventional spread footings that had been anticipated in the preload design phase. Since the preload design was govern by settlements occurring below building footings, the preload design would have been identical for other shallow foundation types such as the selected raft foundation.

It should also be noted that the detailed structural design indicated that the maximum slab load considered to be permanent would be rack loads, which would induce a pressure of about 25 kPa over an area of approximately 2 m^2. Thus, assuming a uniform permanent slab load of 25 kPa within the entire building in the preload design was conservative.

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