01 May 2013, 2:00 pm - 4:00 pm

Braced Sheet Pile Shoring Wall in Sensitive Clay

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ABSTRACT

This case history describes the design and performance of a temporary braced sheet pile shoring wall constructed within the median between heavily-trafficked lanes of the Trans Canada Highway in Langley near Vancouver, British Columbia, Canada. The excavation extended to 9.7 m depth below the existing road grade into soft, high plasticity, sensitive glaciomarine clay. Glaciomarine clay is locally notorious for excavation and embankment stability and foundation settlement problems. The shored excavation was required to provide an access pit to allow the installation of a 3 m diameter steel pipe culvert by Horizontal Pile Driving (HPD).

The braced sheet pile wall was designed using the Terzaghi Apparent Earth Pressure distribution and conventional limit equilibrium analysis methods. The excavation was undertaken in stages as the bracing was installed and ground deformation was monitored using slope inclinometers and by survey of surface targets. The case history describes the performance of the excavation and compares predicted to monitored displacements. A particular issue related to face stability due to clay squeezing and running sand during bulkhead sheet pile removal required to commence HPD for culvert installation. The bulkhead face was stabilized by grouting with a water reactive polyurethane grout prior to sheet pile removal.

INTRODUCTION

The design and performance of braced excavations through soft to medium stiff clay has been extensively studied and documented in the geotechnical literature (e.g., Bjerrum and Eide 1956; Peck 1969; Clough and Reed 1984; Finno *et al.* 1989; Wong and Brooms 1989; Hashash and Whittle 1996; Finno *et al.* 2002; Ukritchon *et al.* 2003; Athanasopoulas *et al.* 2011). A recently constructed temporary braced sheet pile shoring wall constructed in the median between highly-trafficked lanes of the Trans Canada Highway (Highway 1) in Langley near Vancouver, British Columbia, Canada, adds to the literature and provides information on the performance of excavation in local glaciomarine clay. The site location is shown in Figure 1. Highway 1 is the main traffic artery connecting Vancouver with the rest of Canada.

The excavation was required as part of the Design/Build Port Mann/Highway 1 Improvement Project (PMH1 Project) being undertaken by the Kiewit/Flatiron General Partnership. This part of the project entailed installing a new 3 m diameter by 120 m long steel pipe culvert to convey Latimer Creek below both east- and westbound lanes of Highway 1. The culvert was required to replace an existing corrugated steel pipe (CSP) culvert of 1.5 m diameter. Significant challenges were posed because the existing culvert had to remain in service until to the new culvert was completed but was located only 2.3 m from the proposed new culvert.

The culvert invert is about 9 m below highway grade in soft, high plasticity sensitive glaciomarine clay. Few significant excavations had previously been completed in this sensitive glaciomarine clay, and some are reported to be unsuccessful (based on authors' knowledge). A trenchless installation technique called Horizontal Pile Driving (HPD) was selected to install new culvert. In this technique, a 9.7 m deep box excavation (hereafter called a shoring box) with dimensions 23.8 m long by 6.3 m wide was required as a jacking pit in the median between the east-and westbound lanes of Highway 1.

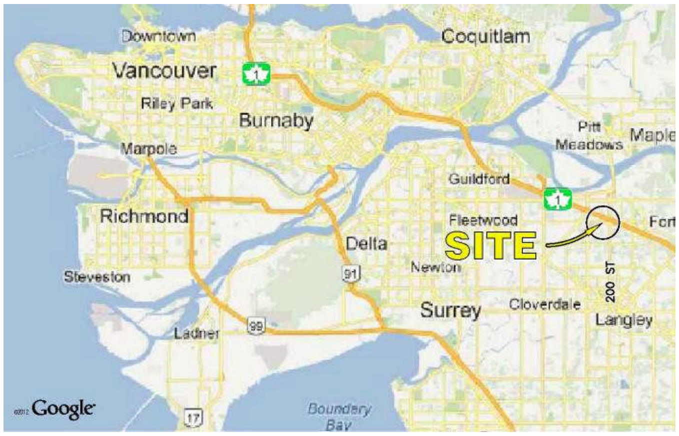

Fig. 1. Locality Plan

The design and construction team for the temporary shoring consisted of:

- Kiewit/Flatiron General Partnership (K/F)
- EBA, A Tetra Tech Company (EBA) geotechnical engineer
- Kamloops Augering Ltd., HPD Subcontractor

This paper describes the subsurface conditions, discusses design of the shoring, summarizes the construction work, and presents the geotechnical monitoring data. Monitored responses are compared with the predicted performance. The paper also discusses the bulkhead face stability issues caused by soft clay squeezing and running sand.

SUBSURFACE CONDITIONS

The underlying soils consist of glaciomarine and marine sediments termed Capilano Sediments, which are deposited following retreat of the Vashon ice sheet (GSC Map 1484A). The Capilano Sediments were deposited in marine environment when the sea was 15 m or more above present sea level. The clays are mostly soft and highly compressible but tend to have a desiccated surface crust of 1 to 3 m thick.

Soil conditions at site were determined from the results of standard soil borings (solid stem auger and mud-rotary drill holes), electric Piezo-cone Penetration tests (CPT), Standard Penetration Tests and Field Vane shear tests. Figure 2 shows that the general subsurface conditions consist of pavement structure, over sand, some silt to silty (fill), over firm low plastic clayey silt to silty clay, over soft to firm high plastic sensitive clay, over low to medium plastic silty clay, over sand.

The static groundwater level was estimated to be at about El. 15 m and likely corresponds to the invert of the existing culvert.

Index properties obtained from the laboratory tests are shown in Fig. 2. The soft sensitive clay is of high plasticity with a liquid limit ranging from 50 to 90% and plasticity index ranging from 30 to 50%. The natural water content ranges from 30 to 80% resulting in Liquidity Index ranges of 0.7 to 1 indicating soft and compressible clay.

The measured peak undrained shear strength of the clay ranges from 24 kPa to 40 kPa between El. 10 m and El. 15 m. The field vane test results indicated a high sensitivity of the clay, defined as the ratio of peak to remolded undrained shear strength, ranging from 3.5 to 15. CPT interpretation using a cone bearing factor (N_{kt}) equal to 14 gives undrained shear strengths of about 25 kPa at El. 15 m increasing to about 45 kPa at El. -3 m.

Table 1 summarizes the approximate engineering properties of the soils present on the site.

Table 1. Engineering Properties

Soil Description	Elevation Range (Thickness) (m)	Undrained Shear Strength (kPa)	Friction Angle (°)	Unit Weight (kN/m^3)
Sand & Gravel (Fill) *	+21.8 to +22.5 (0.7)	-	38°	19
Sand (Possible Fill)	+19.0 to +21.8 (2.8)	-	36°	19
Clayey Silt to silty Clay	+17.0 to +19.0 (2)	40	-	18
Clay – Sensitive	+10.0 to +17.0 (7)	25 to 30	-	17.5
Silty Clay	-3.0 to +10.0 (13)	40 to 45	-	18
Sand	< - 3.0	-	-	-

* includes Asphalt/Concrete layer

DESIGN OF EXCAVATION SUPPORT SYSTEM

Figure 3 shows a plan view and Figs. 4 and 5 show elevations of the excavation and support system. The final excavation depth varied from 9.7 m (El. 12.8 m) below existing road grade at the eastbound (South Headwall) and westbound (North Headwall) highway locations to 5.7 m in the median sidewalls (East and West Wall) prior to construction of the concrete foundation slab. The ground surface was lowered at the centre of the median to El. 18.5 m to provide construction access.

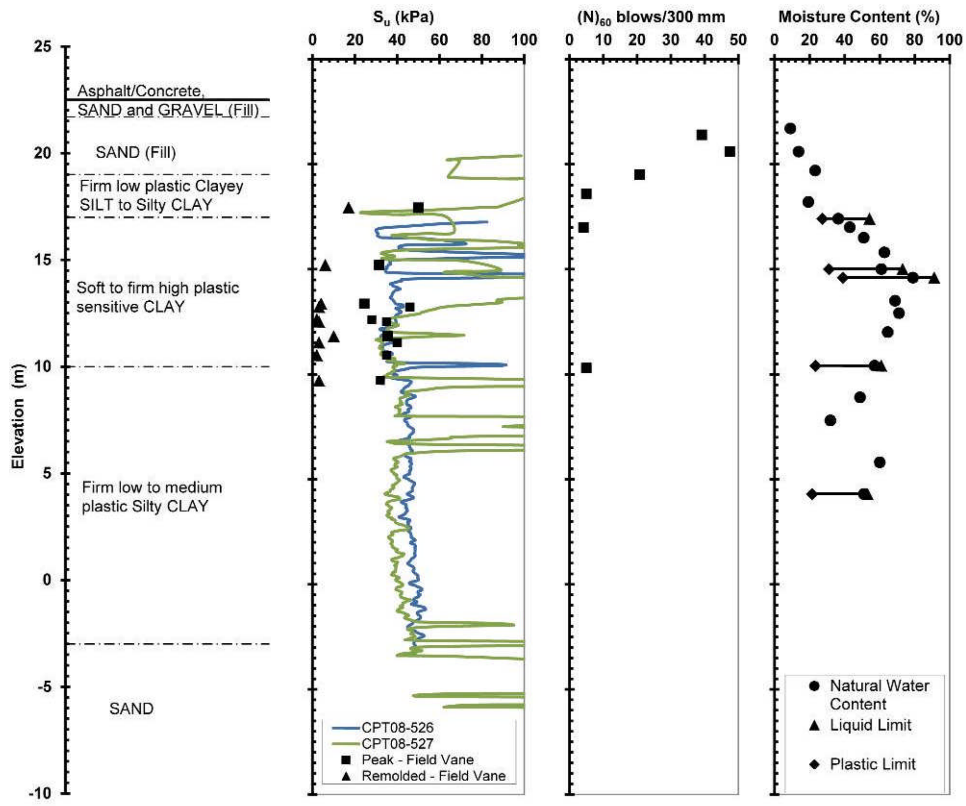

Fig. 2. General subsurface conditions

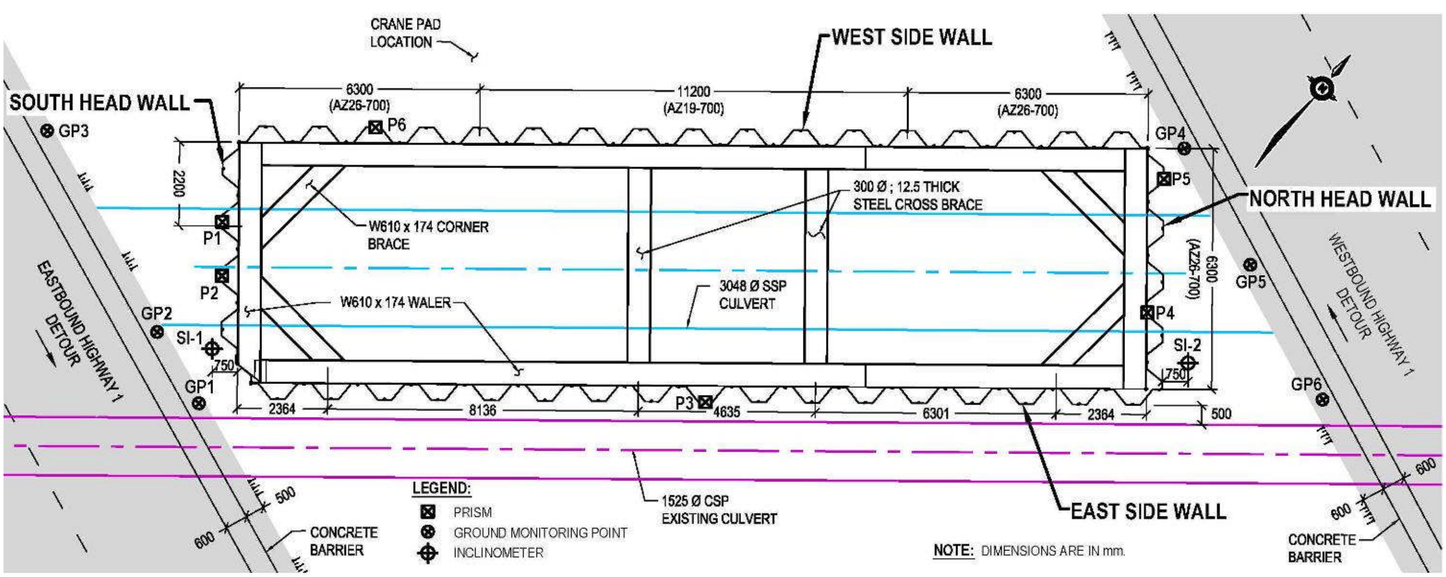

Fig. 3. Plan view of shoring wall and monitoring locations

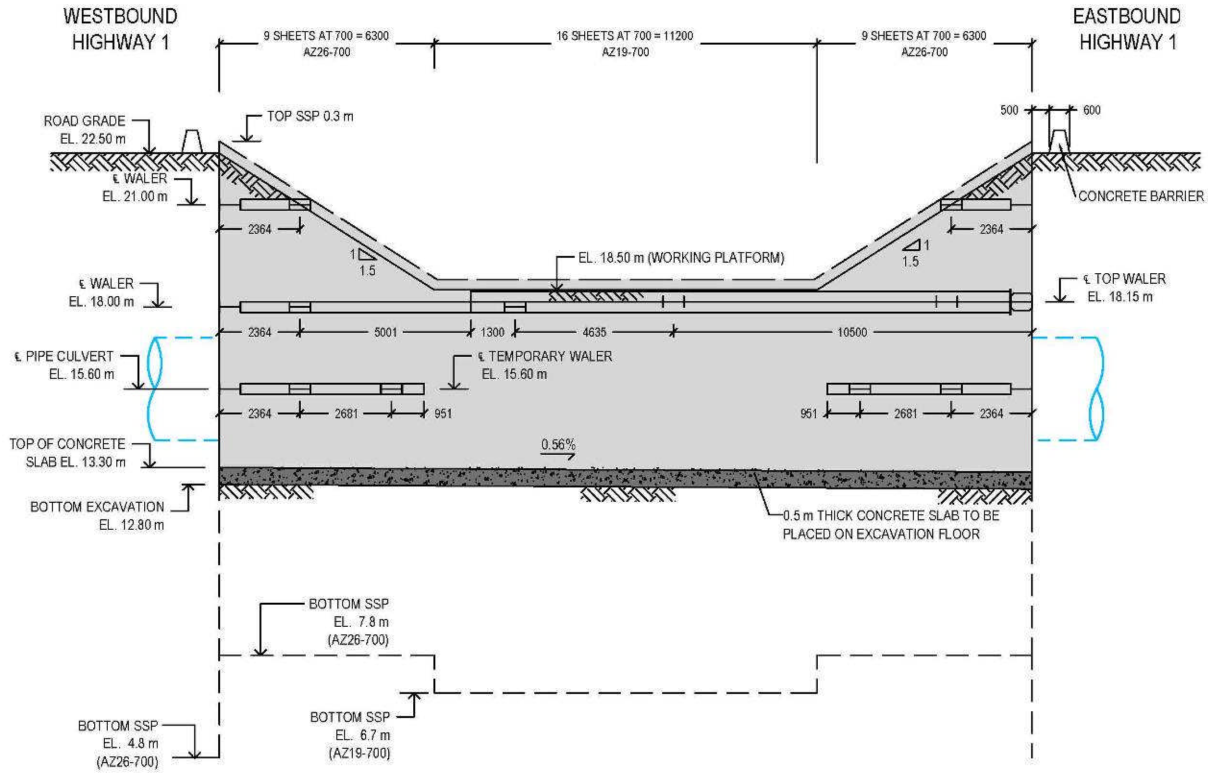

Fig. 4. East and west sidewall elevation view

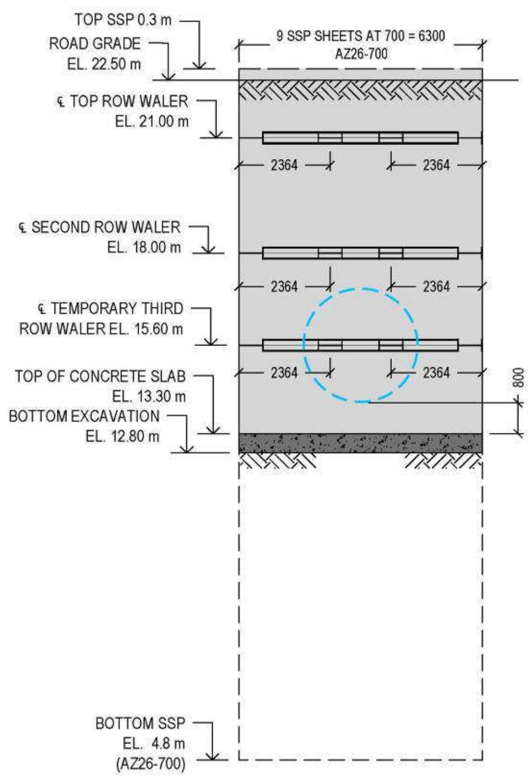

Fig. 5. North and south headwall elevation view

The braced sheet pile walls were designed for both undrained and drained conditions due to uncertainty in the excavation and construction schedule. However, undrained conditions governed the design and are presented here. The design analyses included a nominal surcharge pressure of 16 kPa to account for highway traffic behind the headwall sheet piles.

The lateral load from the excavation was analyzed using the apparent earth pressure distribution for soft to firm clay condition (Peck 1969 and CFEM 4th edition). The program SPW911 v2.39 by Pile Buck International, Inc. was used for internal stability analyses of the wall. The methods in SPW911 are based on design recommendations and criteria described in the British Steel Piling Handbook 7th edition.

The limit equilibrium method was used to check the global stability. The project specification required a minimum factor of safety (FS) of 1.5 against base heave and 1.3 for global stability. Base heave was analyzed using the Terzaghi (1943) and Bjerrum and Eide (1956) methods. The FS with respect to base heave was estimated to be 1.6 and 1.4 with the Terzaghi (1943) method and the Bjerrum and Eide (1956) method, respectively. Base heave was calculated to be 150 mm to 200 mm due to excavation of the soil within the shoring. A 0.5 m thick reinforced concrete foundation slab was required at the bottom of the excavation to support the HPD equipment and winch load, and provide basal stability and lateral support to the shoring during HPD installation.

Global stability of the wall was analyzed using the GeoStudio 2007 program Slope/W version 7.17 using the Morgenstern-Price method. The analysis was performed for undrained conditions before the construction of the concrete slab. The sheet pile global stability FS was 1.25 at the headwall (Fig. 6). However, the global stability assessment was based on 2-dimensional analysis whereas the excavation headwalls were of limited length and would be subjected to 3-dimensional effects. Based on consideration of the beneficial contribution of the 3-dimensional effects, the global stability FS was considered adequate.

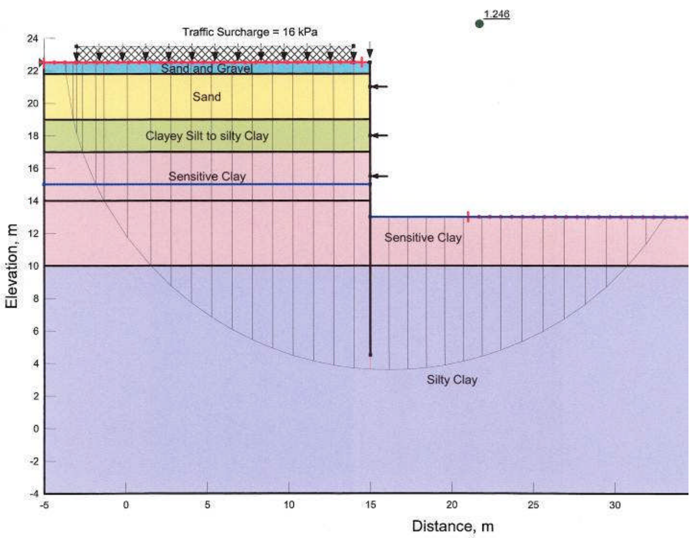

Fig. 6. Global stability of shoring headwall

The final excavation support design consisted of AZ26-700 and AZ19-700 structural steel sheet pile (SSP) walls with three levels of internal bracing. The sheet pile lengths below the existing ground surface varied from 17.7 m at the headwall locations to 11.8 m at the median sidewalls. The internal bracing consisted of walers, corner braces and cross braces (see Figs. 3, 4 and 5 for details). The sheet piles, walers, corner braces and cross braces were specified to meet the requirement of ASTM A 572 Grade 50 or CSA G40.21 Grade 350 W and welding was done as per CSA W59-03 requirement. The third row of bracing was temporary and was removed once the concrete foundation slab gained sufficient strength.

Preliminary estimates of lateral wall deformation were based on the empirical method developed by Clough *et al.* (1989) and an observational approach was used to monitor the soil displacement and adjust the bracing design as required. Clough *et al.* (1989) presented a design chart for soft to medium clays supported by flexible walls that allows estimates of lateral movement in terms of effective system stiffness, S, defined as,

$$S = EI / (\gamma_w h^4_{avg}) \qquad (1)$$

Where:
EI = wall flexural stiffness per horizontal unit of length, where, E = elastic modulus of the wall, kPa; and I = moment of inertia per length of the wall, m^4
h_{avg} = average vertical spacing between supports, m
γ_w = unit weight of water, kN/m^3.

The system stiffness for the Latimer Creek shoring wall was about 385. The normalized lateral deformation predicted by the Clough *et al.* (1989) method was 0.45% and the lateral deformation estimated to be about 43 mm for the wall height (H) of 9.7 m. Wall deflection was also calculated using the SPW911 program and maximum deflection was 52 mm at El. 10.8 m.

INSTRUMENTATION

The locations of two slope inclinometers (SI-1 and SI-2) are shown in Fig. 3. The slope inclinometers were installed after sheet pile installation (SI-1 on December 3, 2011 and SI-2 on December 5, 2011). The tips of SI-1 and SI-2 went to 9.8 m and 9.3 m, respectively, below the bottom of the SSP.

Six survey prisms (P1 to P6) were installed on top of the sheet pile walls, and six ground monitoring points (GP1 to GP6) were installed on the highway pavement. Due to traffic, ground monitoring points could not extend further into the highway lanes.

Monitoring of inclinometers and surveying of prisms and ground monitoring points were performed on a regular basis: every day during excavation and bi-weekly after removal of third row of bracing.

CONSTRUCTION SEQUENCES OF SHORING WALL

Table 2 summarizes the sequence of construction activities of the shoring wall. Sheet piles were installed using a vibratory hammer APE 200 (rpm 2050 recorded during the pile installation). Because of the soft sensitive clay, some of the sheet piles sunk or were dragged down while driving adjacent piles. The sheet pile at the southeast corner sunk about 1.5 m more than design depth and a short piece of sheet pile had to be welded to avoid soil falling into the excavation. Sheet piles at the south headwall sunk more than in other walls. The contractor had to weld sheet piles with grade beam to stop them sinking while driving adjacent ones. Sheet piles were not driven to final set until all the sheet piles were driven to a certain depth to prevent sinking of adjacent sheet piles.

Table 2. Sequence of Construction Activities

Date	Activities
Nov.1 to 18, 2011	Site clearance, crane pad construction and slope cut
Nov. 22 to Dec. 02, 2011	Sheet pile installation
Dec. 3 to 4, 2011	First row of bracing installation at El. 21 m
Dec. 5, 2011	First stage excavation down to El. 17 m
Dec. 6 to 10, 2011	Second row of bracing installation at El. 18 m
Dec. 11, 2011	Second stage excavation down to El. 14.6 m
Dec. 12 to 14, 2011	Third row of bracing installation at El. 15.6 m
Dec. 15, 2011	Final Stage excavation down to El. 12.8 m in south headwall side and south cross brace installation
Dec. 16, 2011	Final stage excavation down to El. 12.8 m in north headwall side and north cross brace installation; final stage excavation at the centre area of shoring
Dec. 19, 2011	Concrete slab pour
Dec 28, 2011	Third row of bracing removal

Excavation within the shoring box was undertaken using a combination of mini excavator and large excavator due to space limitations.

The final stage of excavation was performed in sequence to keep the wall stable. The excavation was performed down to El. 12.8 m at the south headwall location first on December 15, 2011. The excavation was started fully across the width of the shoring box at the south end and then backed towards the cross brace. The south cross brace at the third row was installed the same day as excavation, which was critical to reduce the risk of shoring instability. Subsequent excavation was completed down to final elevation (El. 12.8 m) at the north headwall location on December 16, 2011, and the north cross brace at the third row was installed the same day. The final excavation at the centre area of the shoring box was completed during the night shift of December 16, 2011.

Figure 7 shows the excavation at the south and north headwall location below the third row of bracing. The excavation was successfully completed to the final elevation and the reinforced concrete slab was poured on December 19, 2011.

Fig. 7. Excavation Photographs below third row of bracing (a) South headwall (b) North headwall

FIELD OBSERVATIONS

Soil Conditions

The soils encountered in the excavation were significantly different in the north and south halves of the shoring box (see Fig. 7).

- **South Side:** 1.5 m of pavement materials, over about 3 m of damp brown grey silty sand fill, over about 1 m of soft to firm moist brown organic silt, with occasional wood logs. The organic silt layer was underlain by soft to firm wet mottled grey organic clay of about 1 m thick, over about 1.1 m of soft to firm wet mottled grey clayey silt to silt, over 0.3 m thick loose wet mottled grey sand, some gravel to gravelly, over 0.3 m of soft to firm moist to wet light yellow grey clay, over soft to firm wet dark grey clay. Laboratory tests performed on samples of dark grey clay indicated water content ranged from 62 to 79%, liquid limit ranged from 74 to 91% and plasticity index ranged from 41 to 52%. Liquidity index varied from 0.7 to 0.77.

- **North Side:** 0.3 m thick asphalt/concrete pavement, over 0.3 m of sand and gravel, over 0.3 m of asphalt/concrete, over about 4.6 m of damp brown grey sand, some silt to silty fill, over about 2.1 m of damp to moist grey to brown grey silt, some sand, some clay, over about 0.3 m of loose wet mottled grey sand, some gravel to gravelly, over 1.1 m of firm moist to wet light yellow grey clay, over firm moist to wet dark grey clay.

Perched water was observed at the centre area of the shoring box at El. 15 m in the sand and gravel layer.

The north side of excavation had a thicker layer of sand and silt, compared to the clay at the south side. Clay at the north side was firm and had less moisture than soft and wet clay at the south side. Sy and Gillespie (2012) also observed more and thicker inter-layers or seams of sand in north approach of 200 Street at Highway 1 (about 150 m east of the excavation site) compared to the clay at the south approach.

<u>Lateral Soil Movement</u>

Figures 8 and 9 show lateral soil displacement towards the excavation measured at inclinometer SI-1 and SI-2 locations along with soil conditions observed during excavation. Because of the construction schedule, baseline readings of the inclinometer measurement could not be established prior to the start of the excavation and therefore, lateral soil displacement due to excavation down to El. 17 m (December 5, 2011) are not included in Figs. 8 and 9. However, the measured lateral displacements in the inclinometers were likely less than about 2 mm during the period December 3 to 10, 2011.

As the excavation progressed in the soft clay, soil moved incrementally towards the excavation. Figures 8 and 9 both show similar patterns of movements. Lateral movements extend below the bottom of the excavation. The maximum lateral movements occur at about 0.5 m below the final excavated grade, and are equal to about 20 mm (i.e., 0.21% of *H*) at SI-1 and 8 mm (0.08% of *H*) at SI-2. The differing values of lateral soil movement reflect the differing soil conditions encountered in the excavation. The observed lateral displacements are about half those predicted using the Clough *et al.* (1989) method and about 40% of the SPW911 calculation. The small lateral displacements are likely due to the 3-dimensinal nature of the shoring box and good workmanship during construction of shoring wall.

Toe movement of the sheet pile occurred after the excavation depth reached at El. 14.6 and increased with excavation depth. Maximum toe movements of the sheet pile are about 5 mm and 3 mm in the south and north headwall locations, respectively.

Figure 10 shows the maximum lateral soil movement and demonstrates that significant creep movements developed in the clay after excavation to the final grade until the concrete slab gained strength, and the creep movements reduced thereafter.

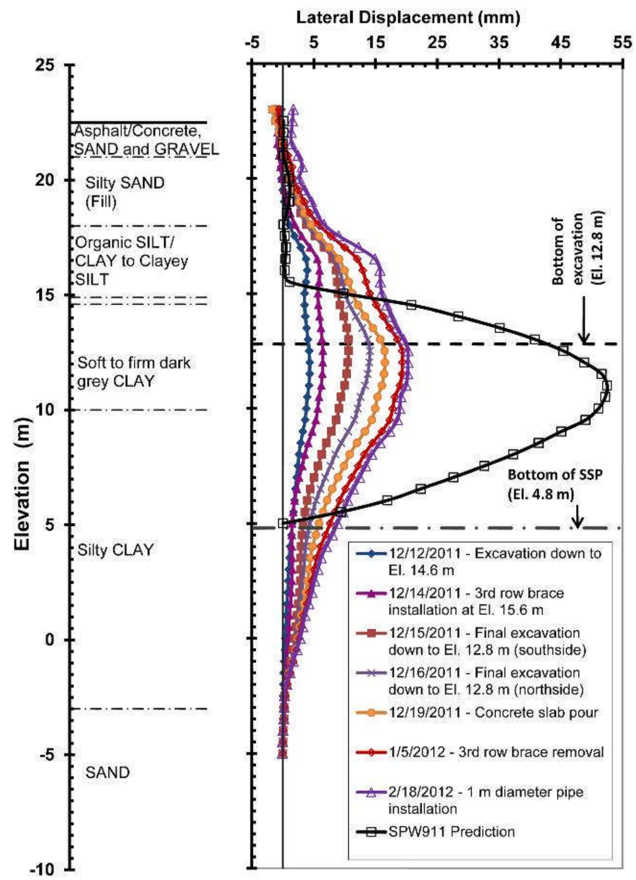

Fig. 8. Lateral soil movement observed at the south headwall (SI-1)

<u>Ground Surface Settlement</u>

Figure 11 shows the ground settlement during excavation and construction. Observed maximum ground settlements were about 12 mm (0.12% of *H*) and 8 mm (0.08% of *H*) in the south and north headwall locations, respectively. As expected, settlements decreased away from the shoring box. Minor settlement of less than 10 mm likely occurred below Highway 1, but measurement was not possible due to the heavy traffic.

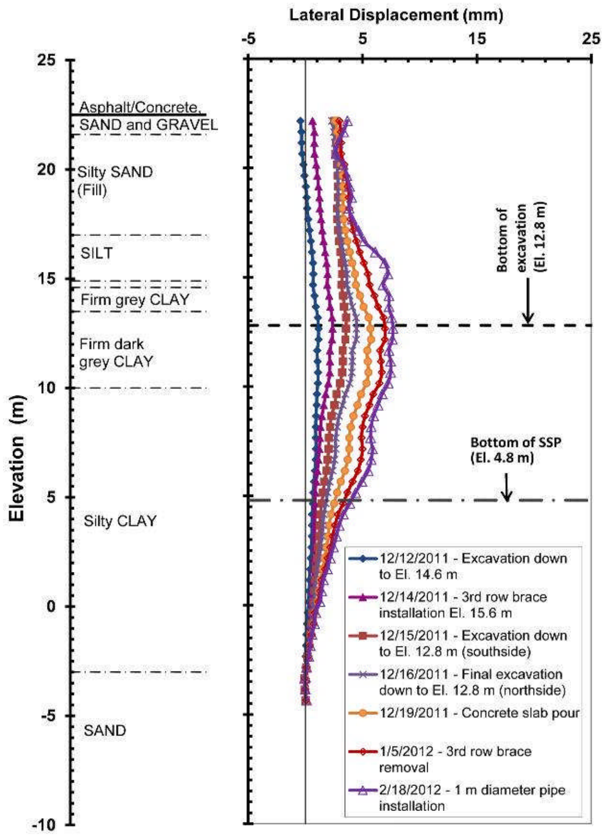

Fig. 9. Lateral soil movement observed at the north headwall (SI-2)

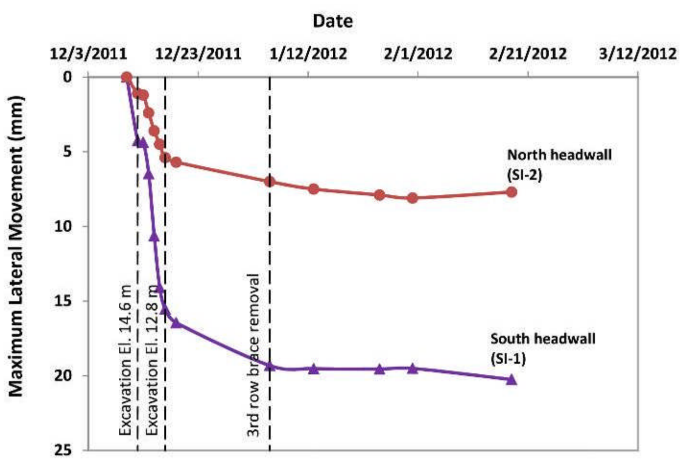

Fig. 10. Maximum lateral soil movement at the south and north headwalls (SI-1 & SI-2)

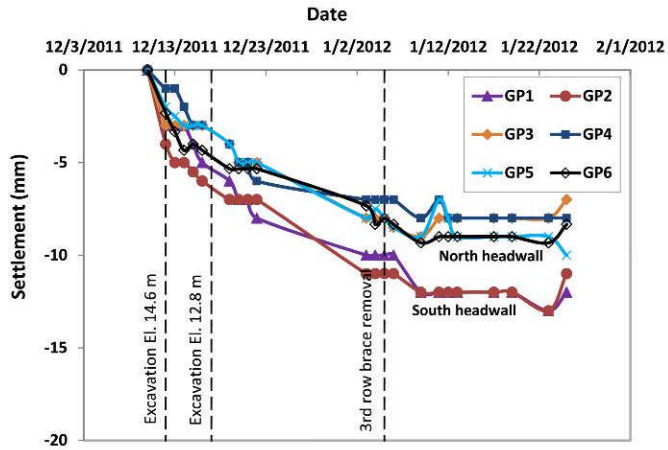

Fig. 11. Ground settlement observed at GP1 to GP6

Sheet Pile Top Movement

Figure 12 shows the vertical and horizontal movements of the top of the sheet pile at P1 and P4 locations. Prism P1 moved about 12 mm away from the excavation in a southwest direction. Prism P4 also moved about 5 mm away from the excavation in a northeast direction. The lesser movement of P4 compared to P1 was compatible with the lower inclinometer displacement and ground settlement in the north headwall than in the south headwall. Settlement was generally negligible and within the survey measurement accuracy. Prism observation indicates the sheet pile top moved out of the excavation. Note that inclinometer monitoring shows the toe of sheet pile moved towards the excavation. The bracing has pushed the sheet pile out of the excavation at the top.

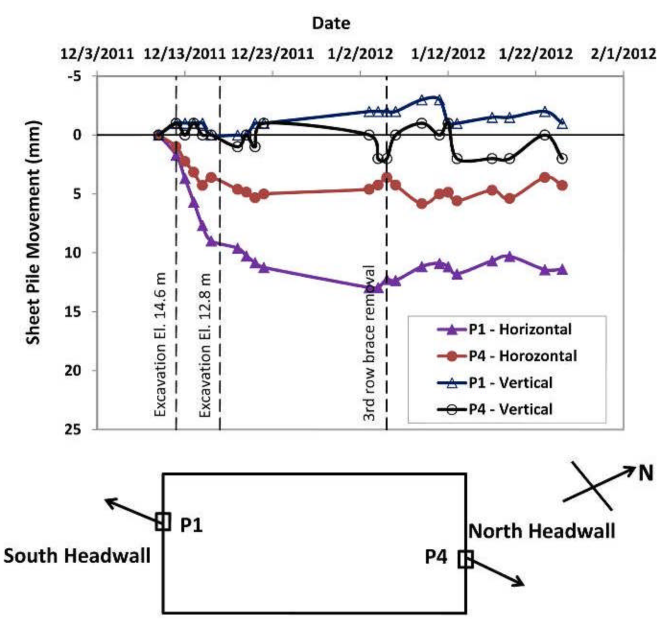

Fig. 12. Sheet Pile top movement observed at Prisms P1 & P4

FACE STABILITY

Prior to installation of the 3 m diameter pipe by HPD, a 100 mm diameter rod was hydraulically pushed using laser guidance along the pipe alignment to provide the design grade and alignment. Thereafter, a 1067 mm diameter steel pipe (pilot hole) was installed by pipe ramming along the direction of the previously installed rod. The installation of rods and pilot holes was completed from January 14 to February 10, 2012. Following completion of the pilot holes, clay was observed squeezing around the pipe at both headwall locations. In addition, running sand was encountered at the north headwall.

The stability of the bulkhead cut was made using the stability number (*N*) approach developed by Brom and Bennermark (1967):

$$N = \gamma \text{ x } d / s_u \tag{2}$$

Where:
γ = total unit weight of soil, kN/m^3
d = depth from ground surface to tunnel axis, m
s_u = undrained strength of clay, kPa.

If $N > 5$, the face is anticipated to be stable in plastic clay (Peck 1969). Therefore, to provide a stable face at the 7 m depth to the tunnel axis, the undrained shear strength of the clay needed to be greater than 26 kPa.

North Headwall Bulkhead

In order to stabilize the face, the HPD sub-contractor grouted around the bulkhead face in the north headwall by using water reactive polyurethane grout (De Neef "Hydro Active® Cut"). A catalyst called "Cut Cat" was added 5% in the Hydro Active® Cut and additional 5% catalyst called "Fast Cat" was added where running sand was encountered in the face. Figure 13a shows the location of the grout injection hole patterns. Grout was injected to about 0.9 m to 1.2 m behind the bulkhead.

Water reactive polyurethane grout has previously been used successfully for water control in similar applications (Kriekemans 1984 and Town 2003). The grout when injected into wet soil reacts with the in-situ soil water resulting in high volume expansion causing compression of the surrounding soil near the injection point.

The performance of high expansion grout in highly sensitive clay was of concern. Therefore, the strength of the retained soil behind the bulkhead following grouting was tested by conducting vane shear tests in horizontally drilled hand auger holes cut through the sheet pile bulkhead. The holes extended between 0.5 m and 1.7 m behind the bulkhead.

The undrained shear strength was measured to mainly greater than 25 to 40 kPa to about 1 m behind the bulkhead. Based on these results which an average were greater than the minimum required undrained shear strength of 26 kPa, it was considered safe to proceed with removing the sheets, provided the adjacent traffic lanes were closed and emergency backup plans were established.

The sheet pile was precut around the pilot hole to give 8 wedges ("pizza wedges"), leaving 35 to 50 mm steel uncut to hold the wedges in place. The wedges were then removed in an alternating sequence, and the HPD machine was quickly advanced up to the unsupported face to provide support, and a controlled density fill plug was poured into the advance segment of the pipe.

Visual observations at the time of the bulkhead removal showed the sand layer in the upper half of the face was moist. The sand was no longer running and was stable likely because of the beneficial effect of the grout. In addition, the clay in the lower half of the face was moist and firm. However, during the HPD cutting edge advancement, a small amount of material sloughed from the upper face due to HPD machine vibration. No cracks, deformation or settlement was observed in the adjacent Highway 1 lane.

South Headwall Bulkhead

In the south headwall bulkhead, the undrained shear strength of clay behind the bulkhead was measured before and after grout injection (Fig. 13). Shear strength testing before grout injection, using a hand vane shear indicated undrained shear strengths of about 30 to 32 kPa in the lower half of the bulkhead and about 33 to 45 kPa in the upper half. The clay was wet.

The south headwall bulkhead grouting and removal of bulkhead followed a similar procedure as used on the north headwall bulkhead. The south headwall bulkhead was removed on March 26, 2012. The soil exposed at the face at the time of bulkhead removal was moist and firm, and had a lower water content and likely higher undrained shear strength than before grouting. No sloughing occurred in the open bulkhead face, and no cracks, deformation or settlement on the adjacent Highway 1 lane were observed.

Undrained shear strength tests were performed using the hand vane shear test in the grouted soil during the soil removal from the 3 m diameter pipe on March 31, 2012. The measured strengths were greater than before grouting (see Fig. 13). Soil was moist and firm. Visual observation showed layers of grout in the soil. The spreading of the grout was non-homogeneous, which is similar to that described by Hellmeier *et al.* (2011) in their laboratory tests on cohesive soil. It is likely that grout followed weak layers in the clay.

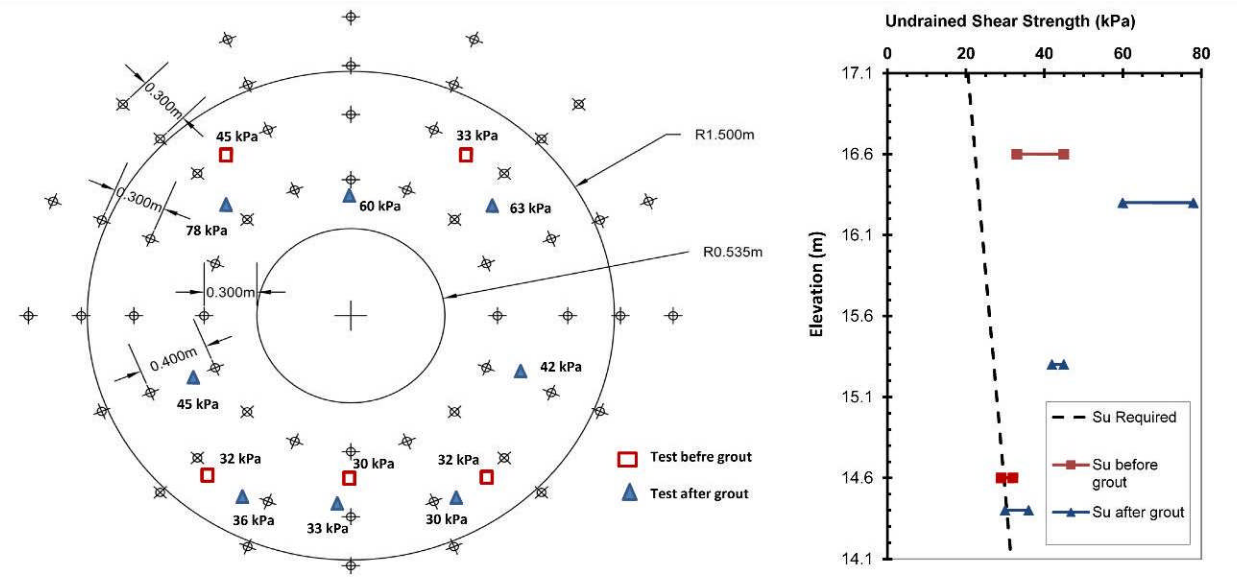

Fig. 13. (a) Location of grout injection hole pattern in bulkhead face including location of undrained shear strength testing, (b) Undrained shear strength of clay before and after grout injection in the south bulkhead face

CONCLUSION

A case history on the design, construction procedure and observed performance of a temporary braced sheet pile shoring wall was presented. Face stability of the bulkhead face was also discussed during the 3 m diameter pipe installation. The following conclusions can be drawn from this case history:

1. The braced sheet piling system used proved successful in supporting a 9 m deep excavation in soft, sensitive clay. In particular, the system was able to readily accommodate unexpected variations in the soil conditions, including the presence of running sand.
2. The Capilano Sediment glaciomarine clay varies over short distances and has sand seams.
3. The sheet piles were installed within 0.5 m of an existing CSP culvert, and no adverse effects were noted.
4. The shoring performed somewhat better than predicted and the measured maximum soil lateral displacements were 20 mm in the soft clay (south headwall) and 8 mm in the sand over firmer clay of the north headwall. The predicted soil lateral movements using the Clough *et al.* (1989) and SPW911 program were greater than the measured values. This is likely due to the 3-dimensional nature of the shoring box and bracing systems.
5. Water reactive polyurethane grout improved the mechanical behavior of the clay sufficiently to stabilize the exposed bulkhead cut face. We encourage further research to better understand the applicability of water reactive polyurethane grout in the geotechnical engineering.

ACKNOWLEDGEMENTS

Diligent work by the Kiewit/Flatiron General Partnership contributed to the success of this project. Special thanks goes to Mr. Joel Butcher, Field Engineer of Kiewit/Flatiron General Partnership, who made outstanding efforts to ensure that all the instrumentations were protected from damage; and instrumentation monitoring data was provided in a timely manner. Approval granted by Kiewit/Flatiron General Partnership for publication of this paper is acknowledged.

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