Case Study of about 13m High Preload in Southwest Calgary, Alberta

Muhammed Al-Kustaban, E.I.T.
Junior Geotechnical Engineer, EXP Services Inc., Vancouver, BC, Canada.

Kai-Sing Hui, P.Eng.
Geotechnical Discipline Manager, EXP Services Inc., Vancouver, BC, Canada.

Steven Saye, P.E.
Senior Geotechnical Engineer, Kiewit Engineering Group Inc., Omaha, NE, U.S.A.

ABSTRACT As part of the Southwest Calgary Ring Road Project in Alberta, road embankments of up to 14m will be constructed. Due to the presence of compressible soils at depth, an extensive field exploration and laboratory program had been conducted. At a section of a new storm trunk sewer, embankment height of up to about 10m will be placed. Based on settlement analysis results, it is deemed that the proposed storm trunk will not be able to tolerate the anticipated total and differential settlements. As a result, a preload was constructed and instrumented with vibrating wire piezometers, deep settlement gauges and shallow settlement gauges. The preload also served as a test preload to calibrate the settlement models for settlement estimation of nearby bridge approaches. This paper summarizes subsurface soil conditions, methods of estimating settlement parameters, settlement and pore pressure field readings and settlement model calibration.

Introduction

Project Background

The Southwest Calgary Ring Road (SWCRR) is located in the immediate vicinity of Calgary, Alberta, and consists of a proposed new six and eight-lane highway (four in each direction) with upgrading of the existing highway for a total mainline project length of approximately 31 km of new construction.

The project scope consists of three major components:

(1) New highway infrastructure construction, which includes 14 interchanges and 3 major water crossings;
(2) Upgrading and improvements to existing highway infrastructure; and,
(3) Operation, maintenance and rehabilitation of new and existing highway infrastructure.

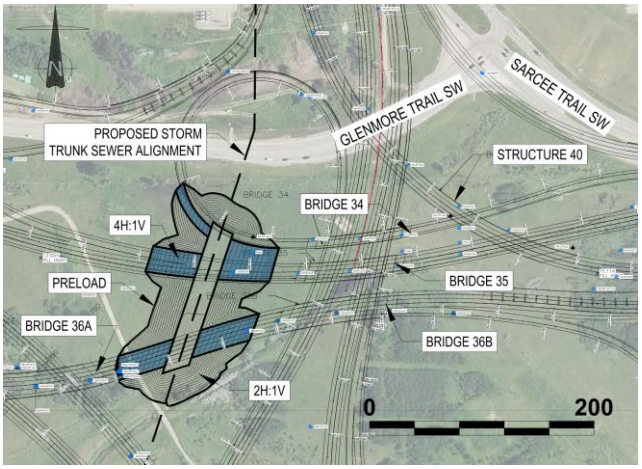
Mountain View Partners (MVP) has a concession agreement with Alberta Transportation (AT) to Design, Build, Finance, Operate & Maintain (DBFOM) the SWCRR over a 35-year period. As concessionaire, MVP has contracted KGL Constructors, a Kiewit, Graham, and Ledcor partnership (KGL) as the Design-Build Contractor. KGL has contracted Parsons, Inc. (Parsons) as the Design Team Lead, to which EXP Services Inc. (EXP) has been subcontracted to provide geotechnical design services for the North Portion of the project.

Sarcee Storm Trunk Sewer and Preload

The Sarcee Interchange is located in the northeast portion of the SWCRR project, a relatively short distance southwest of the Glenmore Trail SW and Sarcee Trail SW intersection; refer to Site Plan in Fig. 1. The interchange consists of five (5) overpasses, three (3) of which carry eastbound and westbound traffic from Glenmore Trail SW crossing over the northbound and southbound lanes of Sarcee Trail SW.

As part of the project design requirements, the Sarcee Storm Trunk would be relocated and would traverse through the west portion of the Sarcee

Fig. 1. Site Plan



interchange heading southwest toward Elbow River where it connects with an existing storm trunk line.

At the northwest quadrant of the interchange, a section of this storm trunk, with approximate depth of about 6m below original ground surface, would be covered by new road embankments of up to about 10m above original ground surface.

The primary objective of the Sarcee Preload was to improve the compressible ground along this section of the Sarcee Storm Trunk.

The secondary objective of the preload was to develop a systematic approach that could be used for estimating settlement parameters for the proposed nearby structures.

Surficial Geology

According to published surficial geology map by Moran (1986), the west portion of the Sarcee interchange is located within the Calgary Formation, a Clayey Lacustrine deposit consisting of silt, clay and minor fine sand deposited in a glacier ice-marginal lake environment. Thickness of this deposit could be in excess of 50m in places.

Geotechnical Exploration

As part of the site-specific geotechnical exploration for Bridges 34, 35, 36A and 36B, seven (7) hollow stem auger holes and two (2) Cone Penetrometer Tests (CPT) were advanced in the vicinity of the subject site to depths ranging from about 18.9m to 39.6m below original ground surface.

Results of the geotechnical exploration indicated that the thickness of this Lacustrine Clay (LC) deposit extended up to about 27m at Bridge 36A, and ranged from about 21.5m to 20m at Bridges 34, 35, and 36B. Sand interbeds/layers of variable thicknesses were encountered at variable depths within the LC deposit. The LC deposit was underlain by a glacial hard Clay Till-Like layer (Clay Till) at deeper depths which was in turn underlain by a gravel layer at some locations. Bedrock was encountered below the Clay Till and/or gravel layers, and extended to the full depth of investigation.

Groundwater level is estimated to fluctuate from 1 to 3m below existing grade based on water level measurements upon completion of drilling, readings of vibrating wire piezometers and visual site assessments of nearby ditches/ponds. For design proposes, average groundwater level of 1.5m below original grade was used.

The laboratory work comprised of water content determination, Atterberg limits, vane shear tests, and 1-D Consolidation. In the generally Sarcee area, seven (7) 1-D Consolidation tests were completed following ASTM D2435. Quality of the consolidation test samples were evaluated. Only five (5) samples were deemed acceptable and had a quality rating of 1 (very good to excellent) to 2 (fair to good) as per Lunne et al. (2006). Samples one to three were generally in the preload area, and samples four and five were generally in the Calgary Clay formation. Soil settlement parameters for acceptable quality samples were interpreted and are summarized in Table 1.

Interpretation of Soil Settlement Parameters

Compression Index (C_c) and Recompression Index (C_r) were correlated with soil indices such as Moisture Content (MC) and Liquid Limit (LL) as shown in Figures 2 through 5 to develop design site-specific correlations that could be utilized at nearby structures and similar geological units.

Preconsolidation pressures (σ'_p) from laboratory consolidation test results were estimated using Casagrande (1936), Boone (2010) and Becker et al. (1987) and were used to calibrate CPT correlations following Demers (2002). Based on this site-specific study, σ'_p estimates using CPT data with $N_{\sigma t}$ of 6 [1]

Table 1. Soil Settlement Parameters for Acceptable Quality Samples

ID	Sample Depth (m)	Moisture Content (%)	Liquid Limit (%)	Plastic Limit (%)	Effective Stress (σ') (kPa)	e_0	Preconsolidation Pressures (σ'_p)		C_c	C_r	C_v (cm^2/s)	C_α
							Casagrande (1936)	Becker (1987)				
1	1.8	22	48	26	35	0.63	410	680	0.2	0.06	0.001	0.001
2	7.9	17	31	15	95	0.44	370	375	0.13	0.04	0.01	0.001
3	13.4	23	37	16	150	0.69	400	475	0.17	0.07	0.0004	0.005
4	18.3	25	38	19	200	0.74	410	350	0.13	0.02	0.004	NA
5	21.3	23	25	18	235	0.61	590	570	0.1	0.02	0.05	0.001

Coefficient of Consolidation (C_v), Secondary Compression Index (C_α), Initial Void Ratio (e_0)

Fig. 2. Proposed Site-Specific C_c Correlations

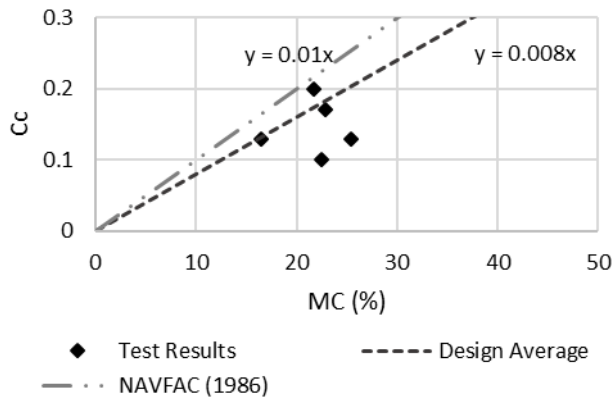
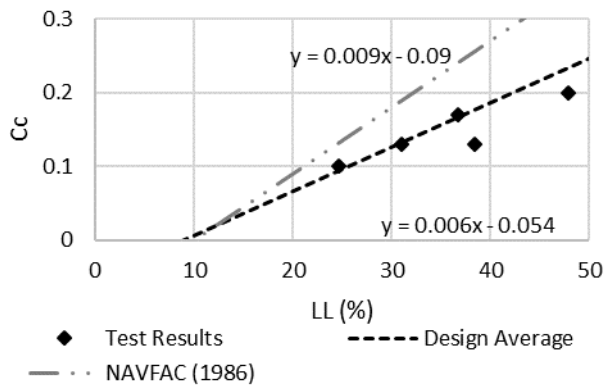


Fig. 3. Proposed Site-Specific C_c Correlations



provided a good correlation with oedometer interpretations for "silt mix" soils with Robertson's (2009) Soil Behavior Type (SBT) ≥ 4 (i.e., $I_c \geq 2.95$), see Fig. 6 for output sample. For "clay mix" soils with SBT≤3 (i.e., Ic<2.95), acceptable correlation with oedometer results was obtained using $N_{\sigma t}$ of 3.

[1] $N_{\sigma t}=(q_t - \sigma_{vo})/ \sigma'_p$ where

σ'_p= Preconsolidation pressure (kPa)

q_t = Corrected total tip resistance (kPa)

σ_{vo}= Total vertical stress (kPa)

Based on local experience, it was anticipated that coefficients of consolidation (C_v) interpreted from the oedometer tests would differ significantly from those estimated from a field preload test which is also supported by Leroueil (1988) observations when comparing C_v determined in the laboratory to those estimated from embankment settlement analysis. Therefore, preliminary field coefficients were estimated based on NAVFAC (1986) correlation which were about 10 to 100 times larger than interpreted oedometer values.

Fig. 4. Proposed Site-Specific C_r Correlations

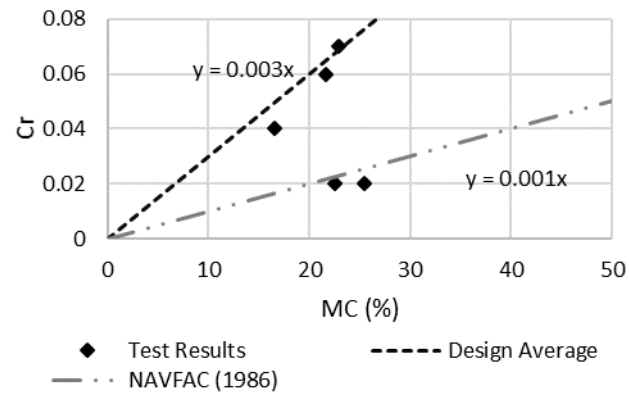


Fig. 5. Proposed Site-Specific C_r Correlations

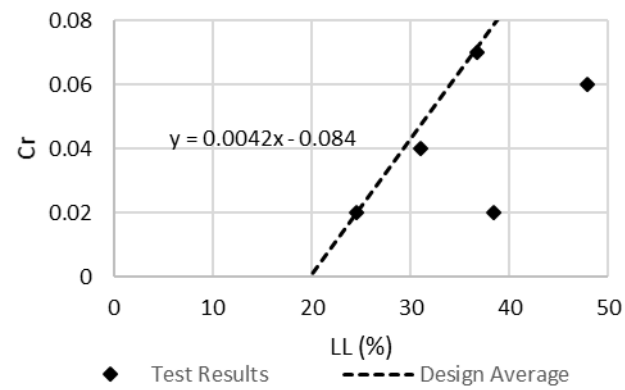
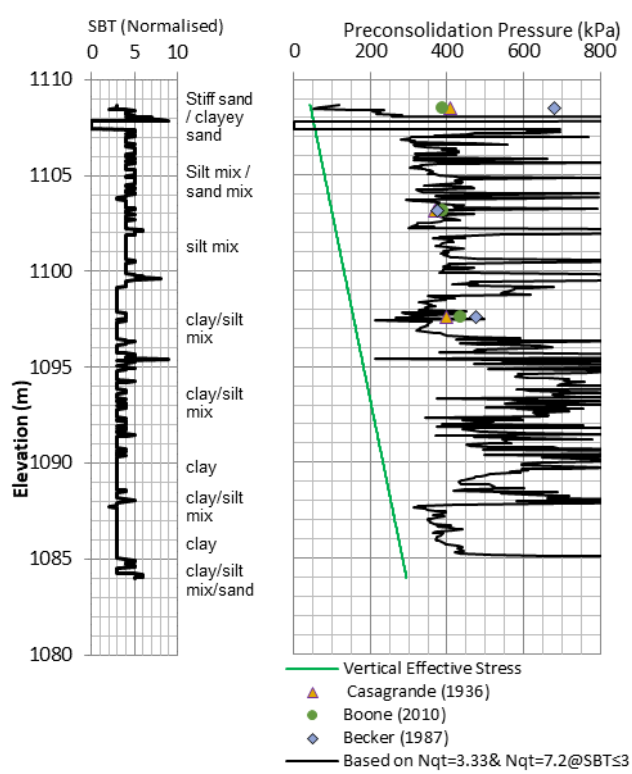


Fig. 6. Preconsolidation estimates using CPT data



Initial Settlement Estimate

Interpreted settlement parameters following the site-specific correlations discussed above were used to estimate total and differential settlements of the proposed storm trunk due to new embankments loads.

Total settlement was estimated to be approximately 380mm, with 90% consolidation was expected to occur within 18months. These estimates were calculated assuming the drainage path is one-way and encountered sand layers and seams were ignored since they were variable in thickness and did not appear to be connected to provide an effective drainage path. The hard Clay Till soils were assumed to be incompressible. Secondary settlement was also assumed to be negligible for all soils.

Preload Ground Improvement

Preload Configuration

A 13m preload, which accounts for the proposed 10m embankments plus to 30% surcharge was proposed to address estimated total and differential settlements. The proposed preload has crest area of about 20m by 130m and side slopes of about 4H:1V along proposed road alignments and 2H:1V outside of proposed road areas. The variance in side slopes was designed to optimize and reduce required soil volume and limit abrupt change in differential settlement along proposed road alignments.

Preload Monitoring

To monitor the ground response to the preload, an instrumentation program was implemented and consisted of:

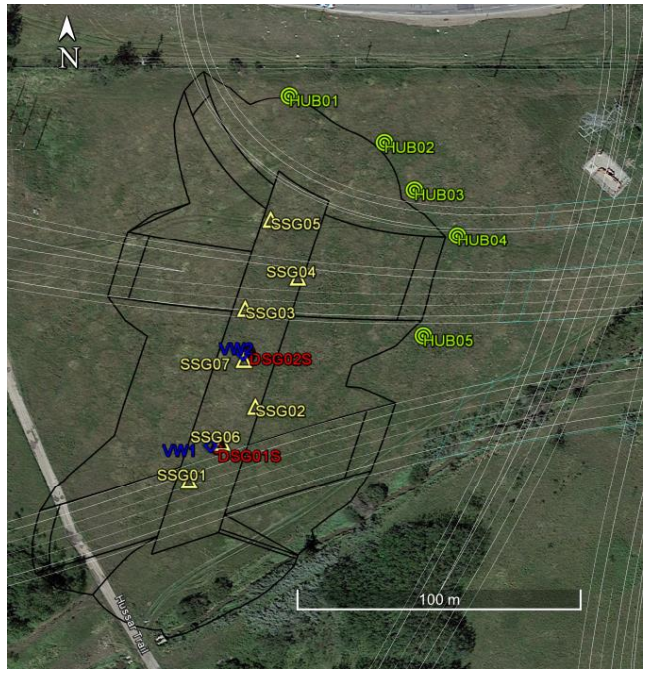
(1) Two (2) piezometer groups (VW); each group has three (3) nested vibrating wire piezometers;
(2) Seven (7) shallow settlement gauges (SSG);
(3) Two (2) deep settlement groups (DSG); each group has two (2) gauges inside a tube casing; and,
(4) Five (5) settlement hubs (HUB).

Approximate locations of the installed instruments are shown in Fig. 7, Instrumentation Plan.

Preload Construction

The preload construction commenced on December 21, 2016 and finished on January 30, 2017. Preload construction was suspended during the holiday season and only about 1m of fill was placed between December 21, 2016 and January 3, 2017. On January 19, 2017, about 50% of the preload construction was completed.

Fig. 7. Instrumentation Plan



Generally, top of preload elevation ranged from 1125m to 1123m, which correspond to approximately 13m to 13.2m above original ground surface. The preload was built out of silty clay soils with wet density that ranged from 2,225 kg/m^3 to 1,720 kg/m^3 and an average wet density of 1,965 kg/m^3.

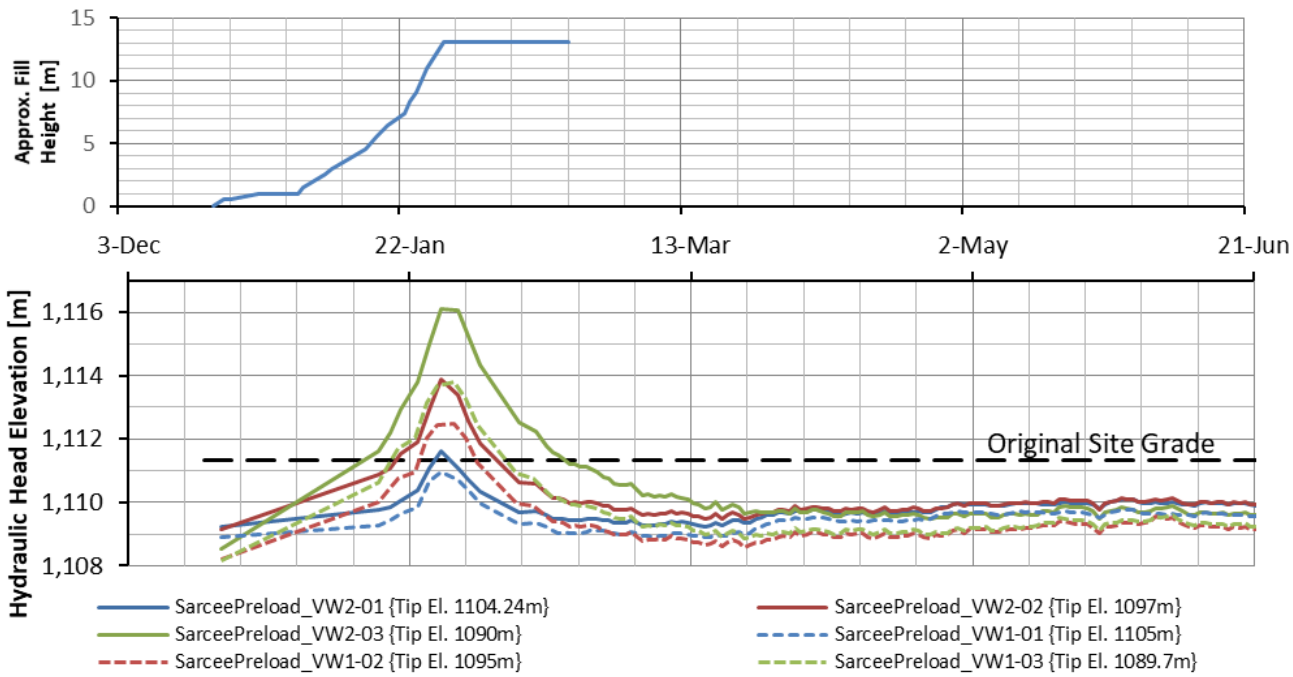
Preload Monitoring Results

Due to construction scheduling issues, no piezometers readings were recorded between December 21, 2016 and January 16, 2017. From January 16 to February 13, 2017, the vibrating wire piezometers were read three times a week. On February 13, 2017, VW piezometers were equipped with automatic data loggers with reading frequency set at one (1) reading per day.

During the monitoring period, Vibrating Wire piezometers recorded an excess porewater pressure that ranged from 19kPa to 68kPa in response to preload construction and increased with depth as shown in Fig. 8. It was suspected that high excess porewater pressure readings at shallow depths were missed and not recorded as a result of fast dissipation rates caused by the presence of fissures and sand pockets/seams. Excess porewater pressure peaked between January 28 and 30, 2017 that corresponded to end of preload construction.

Shallow gauges recorded settlements ranging from about 310mm to 420mm, as shown in Fig. 9. Deep gauges recorded 60mm and 95mm of settlement at 19.5m and 16.3m depths, respectively. Due to construction activities, lateral movements of the settlement gauges were recorded and the highest

Fig. 8. Excess Porewater Pressure Response to Preload Construction.



reading was about 1m at the top of some deep settlement gauges. Therefore, correction of settlement readings and elimination of suspected and unrepresentative settlement gauges was warranted.

Settlement hubs at the slope toes were monitored for vertical and lateral movement. No significant movements were recorded.

Based on settlement data, it was interpreted that most of the primary consolidation was completed by May 12, 2017 which corresponds to about 113 days, using day one equal to 50% of preload placement.

Total settlement was estimated to be in the range of 350mm to 450mm based on interpretation of semi-log settlement graph, as shown in Fig. 9.

Total settlement could also be estimated using the simple hyperbolic method. In this method, total settlement is estimated as the slope invert of time vs settlement/time charts (Kodandaramaswamy et al.1980). Using the method, total settlement is estimated to be in the range of 350mm to 480mm as shown in Fig. 10. With the existing data set, it was observed that the hyperbolic method was only applicable after 50 days from the commence of the preload construction.

Secondary settlement (i.e., creep/long-term settlement) was estimated to be in the range of 10mm to 30mm over a log-cycle based on interpretation of semi-log settlement graph, Fig. 9, and hyperbolic settlement chart, Fig. 10.

Fig. 9. Semi-log Settlement Charts for Recorded Settlement

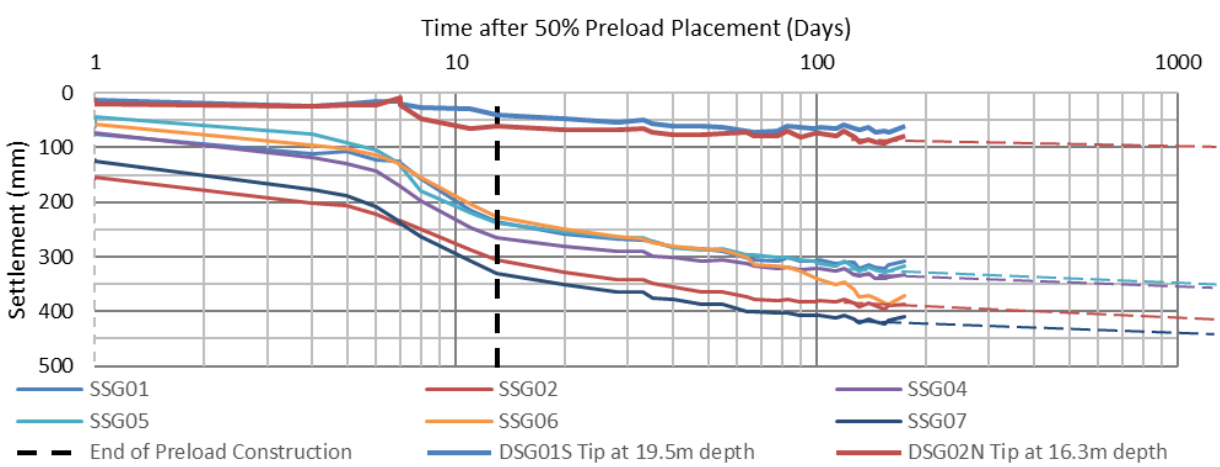
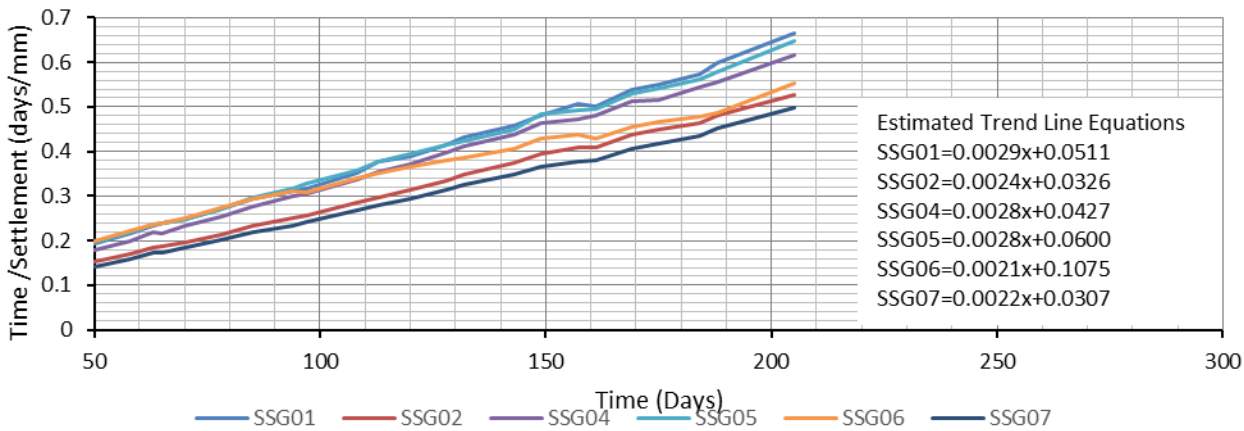


Fig. 10. Hyperbolic Settlement Charts for Recorded Settlement



Settlement Back Analysis

As discussed earlier, the developed systematic approach for estimating settlement parameters will be calibrated using recorded preload data. The calibrated developed method would then be used for estimating settlement parameters for the proposed nearby structures.

Back-Analysis Results and Discussion

Preload back-analysis was completed using Settle3D, Version 4.01 developed by Rocscience Inc. Settle3D is a 3-dimensional program for the analysis of vertical consolidation and settlement under foundation, embankment, and surface loads. The program calculates the effective stresses in the soil mass induced by area loads using elastic theory and solves for the 1-D consolidation and settlement in a series of time increments using a finite-difference numerical solver.

The preload shape was imported into the software with an average unit weight of 19.2kN/m³. The model's loads were staged to simulate the fill construction stages.

Estimated soil settlement parameters based on uncalibrated site-specific correlations for C_c, C_r and σ'_p resulted in a consolidation settlement estimate of about 455mm which was about 14% greater than the maximum recorded settlement and was about 30% larger than the observed average settlement.

Through back-analysis, site-specific correlations were calibrated and better consolidation settlement estimates were obtained when using C_r=0.0028*MC instead of the original correlation of C_r=0.003*MC, refer to Fig. 11 and Fig. 12.

Coefficient of consolidation, C_v as per NAVFAC (1986) correlation resulted in a slower dissipation rate of excess porewater pressure when compared to

Fig. 11. Comparison between Recorded Settlement and Calibrated Settlement Model Estimates

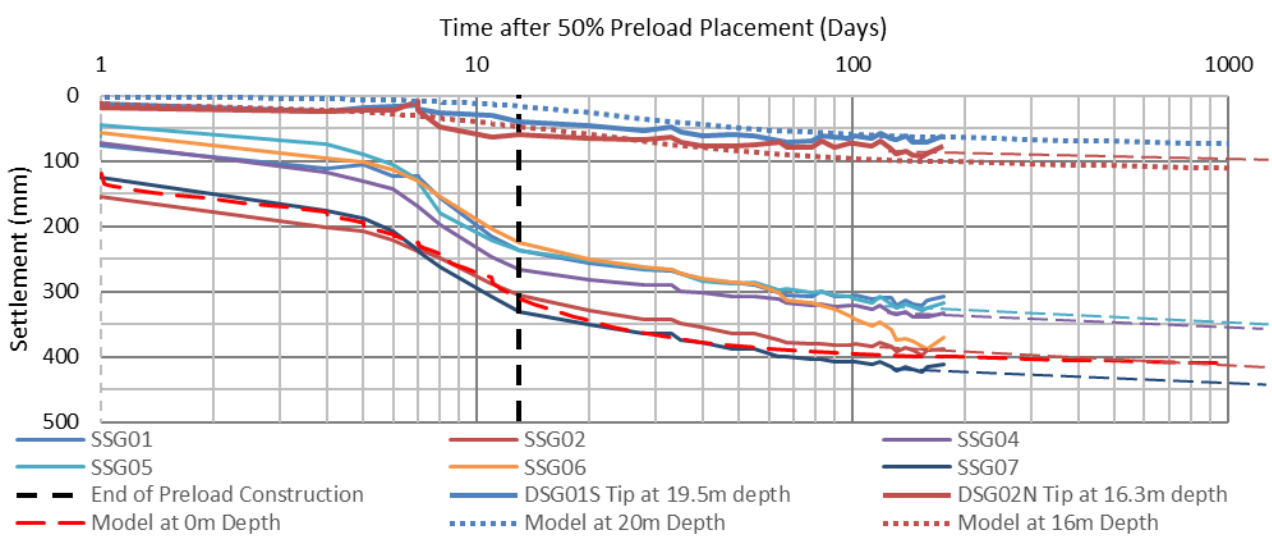
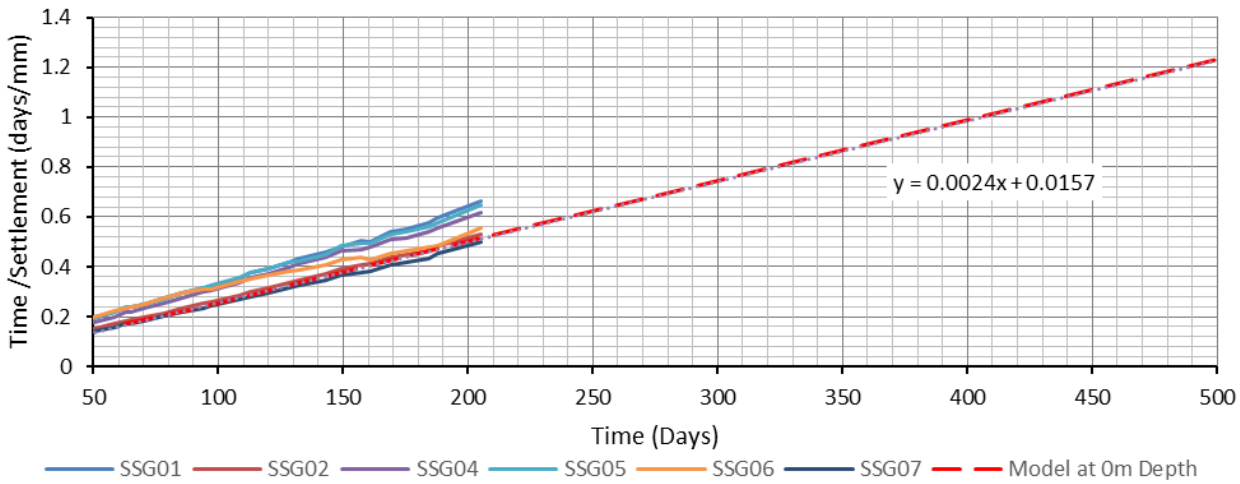


Fig. 12. Comparison between Recorded Settlement and Calibrated Settlement Model Estimates



measured piezometer readings. To simulate field conditions, sand layers/pockets were modelled as drainage paths and trial and error analysis was completed which resulted in a C_v value of 0.1cm^2/s, which was up to 2.5 times faster than the NAVFAC method. Based on the back-analysis, it was concluded that the drainage path through the sand layers/pockets was the factor governing the rate of consolidation. It should also be noted that the rate of consolidation is also controlled by lateral drainage which is typically about 10 times faster than vertical drainage. However, due to the large variability in the sand layers/pockets and the limitation of Settle3D, lateral drainage was ignored.

Readings from deep settlement gauges suggested that secondary settlement is present at depth. The layer experiencing the secondary settlement could not be accurately identified; however, it is suspected that the high plastic clay layer located on top of the hard Clay Till is susceptible to secondary consolidation. A C_α/C_c value (Terzaghi et al. 1996) of 0.04 assigned to this clay layer yielded a reasonable secondary settlement estimates.

Conclusion

The Calgary Lacustrine Clayey Formation is a compressible soil unit and could undergoes significant soil settlement when applied loads exceed the soil preconsolidation pressures. Back-analysis of the Sarcee Preload resulted in the following:

- The sand layers, pockets and seams in this formation appear to provide a fast vertical and lateral drainage and hence faster settlement rates when compared to typical clay soils. A coefficient of consolidation of 0.1cm^2/s which is about 10 to 350 times faster than the average C_v obtained from the oedometer tests closely simulated observed settlement consolidation rate and dissipation rate of excess porewater pressure.
- The site-specific correlations (i.e., C_c=0.008*MC and C_r=0.003*MC) provide settlement estimates with 15% to 30% accuracy. However, settlement estimates which closely matched maximum recorded settlements were obtained when C_r was estimated using C_r=0.0028*MC instead of C_r=0.003*MC.
- Preconsolidation pressure (σ'_p) could be reasonably estimated following Demers (2002) using $N_{\sigma t}$ of 6 for silty sandy soils (i.e., $I_c \geq 2.95$) and $N_{\sigma t}$ of 3 for clayey soils(I_c<2.95).
- High plastic Lacustrine clayey soils within this formation may be susceptible to secondary settlement. A C_α/C_c value (Terzaghi et al.1996) of 0.04 is a reasonable estimate for such soils.

The settlement rate of the Calgary Lacustrine Clayey Formation seemed to follow a rectangular hyperbolic relationship after a time period of applying loads. The settlement estimates using the hyperbolic method provided good long-term settlement estimates that reasonably matched recorded settlements and limit the subjective interpretations of the semi-log method.

Acknowledgments

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