Real-Time Monitoring for Embankment Construction overtop of Regina Clay

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ABSTRACT
This paper presents the result of real-time monitoring for embankment construction overtop of Regina Clay. Two embankments were built in fall 2009 as a part of the Lewvan Interchange Overpass development on the SW side of the Regina, SK. The settlement profile and pore-water pressures were monitored in the foundation soils prior to construction, during construction and post construction. This paper documents the geotechnical site characterization, predicted settlement based on initial design, embankment construction detail and the results of monitoring program.

RÉSUMÉ
Ce document présente les résultats de la surveillance en temps réel de la construction d'un remblai avec une base d'argile de Regina. Deux remblais furent construits à l'automne 2009 dans le cadre du développement de l'échangeur Lewvan sur le côté sud-ouest de Regina, SK. Le profil de stabilisation et les pressions d'eau interstitielle ont été surveillés dans les sols de fondation en début de projet, durant la construction et après la construction. Ce rapport documente la caractérisation géotechnique du site, prédit la stabilisation du site basé sur la conception initiale, les détails de construction des remblais et les résultats du programme de surveillance.

1 INTRODUCTION

The Lewvan Interchange construction project in the southwest Regina, SK is being funded in partnership between the provincial and federal government. The existing junction of Highway No. 1 and Lewvan Drive was a very busy intersection with a high collision rate. The approved Lewvan Interchange layout is shown in Figure 1. The new interchange will improve safety and handle the growth in traffic that will arise from new residential and commercial development in the southwest area of Regina and as a result of the Global Transportation Hub.

Two embankments were built in fall 2009 as a part of the Lewvan Interchange Overpass development on the SW side of the Regina, SK. This paper documents the geotechnical site characterization, predicted settlement based on initial design, embankment construction details and the results of the monitoring program. The loading response of the Regina Clay was monitored using vibrating wire piezometers, settlement plates, shape-array accelerometer and ARGUS (web-based remote monitoring system). During the initial construction planning phase, the embankments were proposed to be built around the clock due to tight completion deadline. Therefore, a real-time monitoring system was utilized to closely monitor the foundation response due to loading during and post construction. During construction slope stability models were updated regularly to calculate the factor of safety at different stages of embankment construction. The numerical analysis results were utilized to characterize and manage the risks associated with fast embankment construction.

Figure 1. Proposed Lewvan Interchange layout southwest of Regina, SK.

To the author's knowledge, there is no documented case history providing the loading response of Regina Clay under an embankment. Similar case histories have been documented for other marine clay deposits in different parts of the world (Burn K.N. and Hamilton J.J. 1968, Devata and Darch 1973, Leroueil et al. 1978a and 1978b, Tavenas, F., and Leroueil, S. 1980, Indraratna et al. 1992, Rowe et al. 2001, Kelln et al. 2007 etc).

1.1 Regional Setting

Regina is situated in a former proglacial lake basin (Christiansen 1961). The overlying sediments are highly plastic clays which are fissured and slickensided below 2 m below ground surface (mbgs). Regionally, the clay is referred to as Regina Clay which is very expansive clay and is typically overconsolidated due to desiccation.

Underlying the surficial clays are various tills of the Saskatoon and Sutherland Group with varying thicknesses. Scattered erosional remnants of Tertiary sediments exist between the till and bedrock. Bearpaw Formation and Pierre Shale of the Montana Group forms the bedrock in the Regina area. The stratigraphic chart for the Regina area is outlined in Figure 2 (reproduced from Christiansen and Sauer 2002).

TIME			STRATIGRAPHIC UNITS			
			GROUP	FORMATION	UNIT AND MEMBER	UNIT No. AND DEPOSIT
QUATERNARY	LATE PLEISTOCENE	Late Wisconsin	Saskatoon Group	Regina Clay		10b Clay
						10a Silt
				Battleford Formation	Upper till	9e Till
					Armour Member	9d Sand and silt
						9c Pebbly sand
						9b Gravel
					Lower till	9a Till
		Early Wisconsin		Floral Formation	Upper unit	8f Till
						8e Sand and gravel
		Sangamon			Pasqua Member	8d Upper unit silt and sand
						8c Lower unit silt, sand, and gravel
		Illinoian			Lower unit	8b Silt, sand, and gravel
						8a Till
	EARLY & MID. PLEISTOCENE	Pre-Illinoian	Sutherland Group	Warman Formation		7 Till and stratified deposits
				Dundurn Formation	Upper unit	6c Till and stratified deposits
					Lower unit	6b Till and stratified deposits
						6a Sand and gravel
				Mennon Formation		5 Till
			Empress Group			4 Sand and gravel
TERTIARY					Tertiary sediments	3 Silt, sand, and gravel
LATE CRETACEOUS			Montana Group	Bearpaw Formation	Snakebite Member	2b Silt and clay
					Ardkenneth Member	2a Sand and silt
				Pierre Shale		1 Silt and clay

Figure 2. Stratigraphic chart (reproduced from Christiansen and Sauer 2002).

2 SITE CHARACTERIZATION

Two boreholes were drilled at this site in October 2005 using Failing 1250 mud rotary drill rig. The locations of boreholes BH1 and BH2 are shown in Figure 3. Disturbed (cuttings and split spoon) samples and relatively undisturbed (thin walled tubes) samples were obtained during the site investigation. Natural gamma, spontaneous potential and single-point resistance logs were also recorded for the stratigraphic interpretation.

Stratigraphy encountered during drilling for boreholes BH-1 and BH-2 is detailed in Figure 4 and Figure 5, respectively. BH-1 was drilled to an approximate depth of 61.0 mbgs and BH-2 was drilled to 42.7 mbgs. Regina Clay extended to a depth of about 11.5 mbgs in BH-1 and 9.0 mbgs in BH-2.

2.1 Laboratory Test Results

Samples obtained during the drilling investigation were analysed with a suite of laboratory tests (such as natural moisture content, Atterberg limits, grain size, bulk density, standard Proctor, unconfined compression strength, direct shear strength and consolidation) to evaluate the soil classification, shear strength and compression characteristics of the foundation soils.

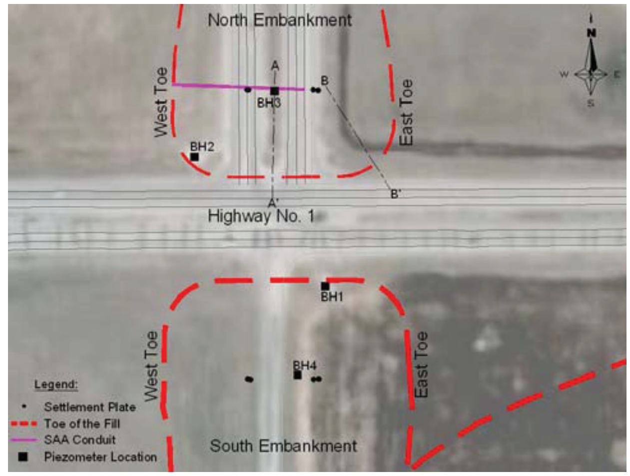

Figure 3. Lewvan Interchange as-built plan view (2009).

2.2 General Properties

Stratigraphic borehole charts (BH-1 and BH-2) including some of the laboratory test and in-situ results are presented in Figure 4 and Figure 5.

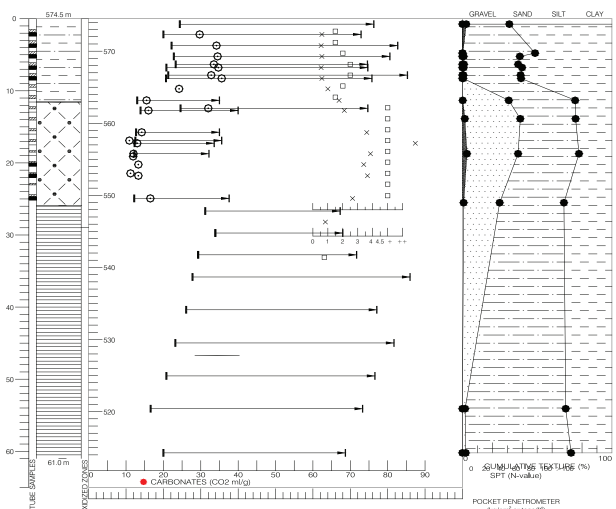

Figure 4. BH-1 – Stratigraphy and laboratory results.

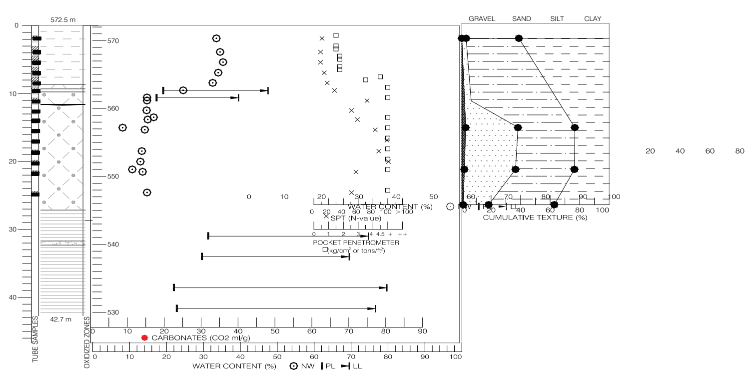

Figure 5. BH-2 – Stratigraphy and laboratory results.

The plasticity index of Regina Clay was found to vary from 41 to 64. The overconsolidated state of the Regina

Clay is reflected by the liquidity index of samples less than 1 and close to zero. A summary of the basic laboratory test results is provided in Table 1.

Table 1. Summary of laboratory test results.

Soil/ Property	Min.	Max.	Average	Median
Clay				
Plasticity Index	40.7	64.0	52.3	52.9
Liquidity Index	0.18	0.27	0.22	0.20
Bulk Density (kg/m^3)	1810	1948	1898	1902
q_u (kPa)	140	412	222	181
MDD (kg/m^3)	1341	1383	1361	1362
OMC (%)	30	32	31	31
Till				
Plasticity Index	13.8	28.4	22.2	22.7
Liquidity Index	-0.46	0.19	0.0	0.05
Bulk Density (kg/m^3)	2136	2317	2231	2235
q_u (kPa)	1535	531	986	983

q_u = Unconfined Compression Strength (kPa)
MDD = Maximum Dry Density (kg/m^3)
OMC = Optimum Moisture Content (%)

2.3 Consolidation Test Results

The consolidation parameters obtained from the laboratory testing are summarized in Table 2. A literature review suggests that a compressive index (C_c) =0.20 and a recompression index (C_r) =0.065 are representative consolidation parameters for high plasticity clay (Fredlund et al, 1980). These values are lower than those tabulated in Table 2. Higher values may be the result of sample disturbance. "*Disturbance is caused by inserting a sampling tube into the soil, transportation, relaxation of stresses, drying, and temperature changes during storage, and further disturbance during trimming and installation into the test cells.*" (Graham, 2006)

The preconsolidation pressure (σ'_p) measured for till was 200 kPa and 100 kPa, which was lower than expected considering geology and its high undrained shear strength.

Table 2. Summary of consolidation parameters.

Sample Id	Depth (m)	e_o	C_c	C_r	σ'_p (kPa)	c_v (cm^2/s)
Clay						
3001	1.4–1.95	0.85	0.25	0.11	300	4.6×10^{-4}
3003	3.05-3.5	0.91	0.25	0.15	200	1.4×10^{-4}
3005	4.55-5.0	0.80	0.27	0.10	600	6.5×10^{-4}
3007	6.10-6.55	0.86	0.29	0.13	300	3.2×10^{-4}
3009	7.6-8.05	1.08	0.31	0.16	200	2.1×10^{-4}
3055	1.5-1.95	0.84	0.23	0.11	170	2.4×10^{-4}
3060	4.55-5.0	0.96	0.35	0.17	150	2.2×10^{-4}
Till						
3015	12.2-12.5	0.42	0.12	0.06	200	5.5×10^{-4}
3015	13.7-14.1	0.47	0.10	0.04	100	1.5×10^{-3}

e_o = Initial void ratio
C_c = Compressive index
C_r = Recompression index
σ'_p = Preconsolidation pressure
c_v = Coefficient of consolidation

2.4 Direct Shear Test Result

One clay sample was subjected to direct shear strength testing during the design phase to obtain the peak and residual shear strength parameters. The results of direct shear strength testing are presented in Figure 6.

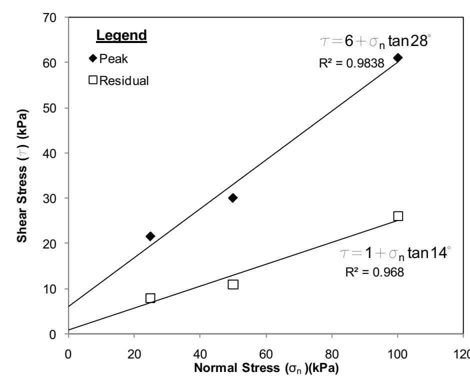

Figure 6. Direct shear test result for clay sample.

3 DESIGN PHASE

3.1 Settlement Analysis

Settlement of the foundation under the approach fill was estimated using the software package UniSettle published by UniSoft Ltd. The vertical stresses in the foundation model were determined based on the Boussinesq distribution. The model utilized results of consolidation tests, including the preconsolidation pressure and recompression index. The model assumed that there would be insignificant settlement in the shale stratum below a depth of 26 m.

Settlement was estimated under the fill centreline, and assumed a 3 percent grade on the roadway. Results of analyses are illustrated in Figure 7. The maximum estimated foundation settlement for a 9 m fill height was about 610 mm, which occurs at a point about 20 m from the face of the abutment. Approximately 150 mm of the settlement occurs within the till stratum, the remaining 460 mm in the clay. The estimated foundation settlement at the abutment was found to be about 390 mm, with differential settlement near the abutment equal to 220 mm approximately. The differential settlement of 220 mm came from the difference between the abutment settlement of 390 mm and the maximum settlement of 610 mm.

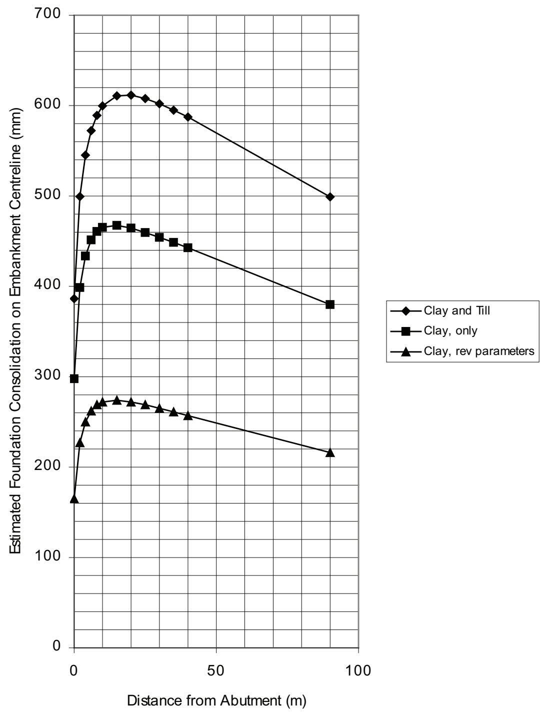

Figure 7. Estimated settlement under fill centreline for a 9 m fill height.

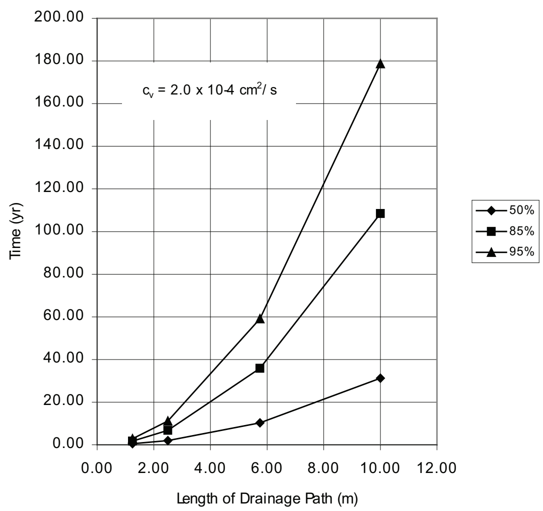

Figure 8. Estimated time for consolidation.

The approach fill was estimated to settle less than two percent (180 mm) of total embankment fill height (9 m) with well compacted clay or till.

The rate of consolidation is strongly dependent upon the length of the drainage path for water during consolidation. The relationship between the length of the drainage path and the time for consolidation is illustrated in Figure 8. The maximum length for the drainage path will be about 10 m, which is the approximate thickness of the clay stratum, and assumes drainage towards the surface, only. However, the actual length of drainage path will be significantly shorter due to the presence of fractures and fissures within the clay and underlying till strata. The fracture spacing is most likely less than 2 m, meaning that 50% consolidation will likely have occurred in less than 1 to 2 years, and 85% in less than about 3 years.

3.2 Slope Stability Analysis

Slope stability analysis was conducted using the software SLOPE/W from GeoSlope International Inc. The limit equilibrium Morgenstern-Price method with a half-sine force function was used to determine the embankment slope angles to meet a minimum factor of safety (*Fs*) of 1.30. Table 3 shows the soil properties used for slope stability analysis.

Table 3. Soil properties used for stability analysis.

Soil Type	γ (kN/m^3)	ϕ' (^{o})	c' (kPa)
Clay	18.3	17.5	5
Till	18.3	30.0	5
Brecciated Clay	18.3	14.0	1

γ = Soil Density
ϕ' = Effective friction angle
c' = Effective cohesion

The primary fill material to be utilized for embankment construction was Regina Clay. "*A common occurrence in cuts or fills of swelling soils is their reduction in strength with time.*" (Widger and Fredlund, 1976). Based on engineering judgement to avoid shallow failures and the results of numerical analysis it was recommended that the side slopes of the embankment should be 5:1 (H:V). Piezometric conditions involving a perched piezometric surface at ground surface for the upper 1.5 m of the embankment and a deep site piezometric surface within 1 m of the original ground surface were analyzed.

4 CONSTRUCTION MONITORING

The settlement profile and pore-water pressures were monitored in the foundation soils prior to construction, during construction and post construction. Vibrating wire piezometers, settlement plates and a shape array accelerometer (SAA) were installed prior to construction. The locations of these instruments are shown on the site plan (see Figure 3). The embankment height and settlement plate monitoring was undertaken using RTK GPS and level instruments.

4.1 Instrumentation Installation

4.1.1 Vibrating Wire Piezometers

A grouted-in installation method (McKenna, 1995) was utilised to install two vibrating wire piezometers underneath each embankment prior to construction. The vibrating wire piezometers were installed in borehole BH-3 and BH-4 and the location of these boreholes is shown in the site plan (see Figure 3).

Piezometers 3A (Tip elevation 567.83 masl) and 3B (Tip elevation 571.23 masl) were installed in the centreline ditch of the existing Lewvan Drive Road with a ground elevation equal to 572.33 masl. Piezometers 4A (Tip elevation 569.11masl) and 4B (Tip elevation 572.01 masl) were installed underneath the existing south grid road pavement with a ground elevation equal to 572.61masl.

4.1.2 Settlement Plates

A total of four settlement plates were installed at the base of the fill at the location of the proposed east and west shoulder for both the north and south embankments to monitor foundation settlement. At the end of construction four additional settlement plates were placed on top of the embankment to monitor total settlement of the foundation and fill. The locations of the settlement plate installation are shown in the site plan (see Figure 3).

4.1.3 Shape Array Accelerometer

A 60 m long shape array accelerometer (SAA) obtained from Measurand Inc. was installed underneath the North embankment (see Figure 3). The SAA was installed starting from west toe of north embankment and ending approximately 10 m past the centreline. During installation, the SAA was housed in a 2” PVC pipe and covered with fill to protect the instrument from damage due to construction equipment and to allow recovery of the SAA at the end of monitoring.

4.2 Remote Monitoring Setup

During the initial construction planning phase, the embankments were proposed to be built around the clock due to a tight completion deadline. Therefore, a real-time monitoring system (ARGUS) was utilized to closely monitor the foundation response due to loading during and post construction. One sub-station was setup, south of existing Highway No. 1 and the main communication station was setup north of Highway No. 1. Photograph 1 shows the remote monitoring setup used in the Lewvan Interchange project. The cables from the vibrating wire piezometers and SAA were routed to one of the monitoring stations.

Photograph 1. Remote monitoring station setup.

5 CONSTRUCTION MONITORING RESULTS

5.1 Embankment Fill

The embankment fill utilized for construction was primarily Regina Clay. The fill was placed in lifts of 150 mm and each lift was compacted to 97% standard Proctor maximum dry density or more, with moisture content equal to ± 2% optimum moisture content. Prior to construction, excess water was observed to be ponding near surface. Therefore, horizontal sand drains (1 m deep and 1 m wide) were installed every 15 m on centre, to allow near surface pore-water pressures to dissipate during abutment loading.

Fill height vs. time plot for the as-built south and north embankments is shown Figure 9 and Figure 10, respectively. The south embankment was constructed from sub-grade to final grade elevations from 13 Oct 2009 to 08 Nov 2009 (i.e., 26 days). Frequent rainfall and difficult access road conditions were responsible for the delay in construction of the north embankment. The north embankment was constructed in 12 days i.e., from 08 Nov 2009 to 19 Nov 2009. The majority of the north embankment fill was placed in 4 days i.e., from 08 Nov 2009 to 12 Nov 2009. At the end of 12 Nov 2009 the fill height was approximately 1 m below the final grade.

5.2 Foundation Settlement

The settlement data obtained from the settlement plates for the south and north embankments are shown in Figure 9 and Figure 10, respectively. At the end of construction maximum settlement was equal to 81 mm and 31 mm for the south and north embankments, respectively. Since the end of construction an additional 36 mm of consolidation settlement has been observed for both the embankments.

Temperature readings along the length of the SAA for various times are shown in Figure 11. A large temperature differential was noted along the first 4.5 m length of the SAA and was responsible for excessive noise observed in the vertical displacement readings for the first 4.5 m. Therefore, the vertical displacement readings for the first 4.5 m of the SAA were ignored.

obtained from SAA at 35 m (see Figure 13) and the north embankment settlement data (see Figure 10).

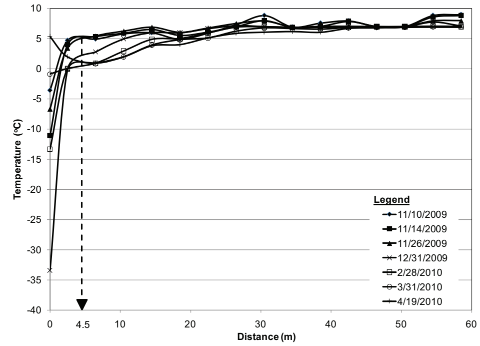

Figure 11. Temperature readings along the length of SAA instrument at different times.

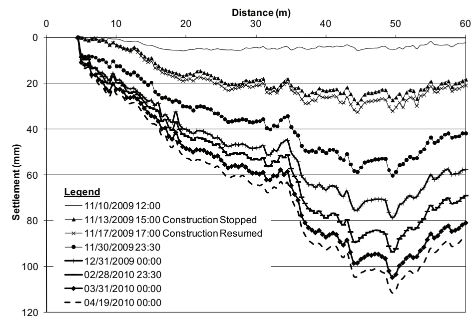

Figure 12. Settlement profile obtained from SAA.

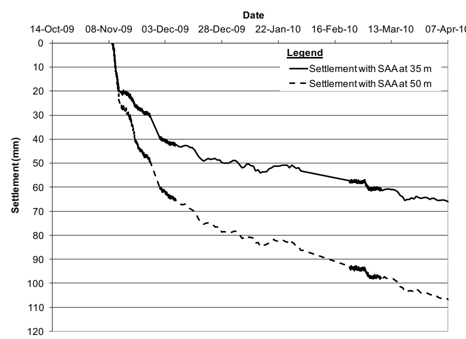

Figure 13. Settlement vs. time plot from SAA.

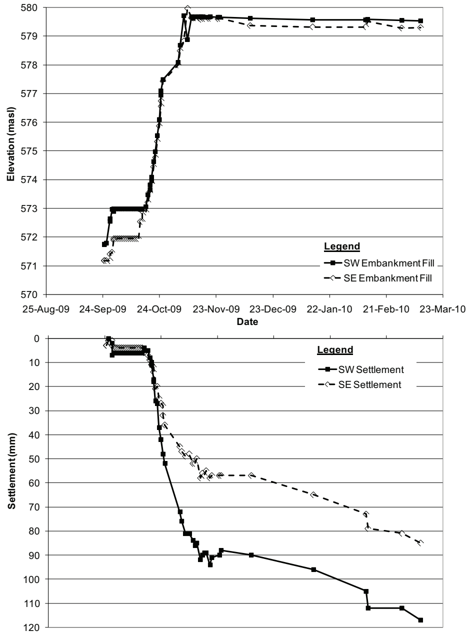

Figure 9. As-built south embankment fill and foundation settlement.

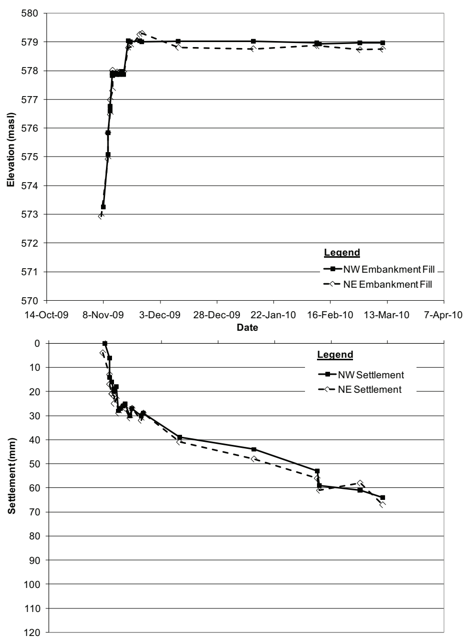

Figure 10. As-built north embankment fill and foundation settlement.

The settlement profile obtained from the SAA underneath the north embankment is shown in Figure 12. A maximum settlement of 112 mm was noted at the centreline of the embankment on 19 April 2010. A plot showing settlement vs. time from the SAA at 35 m (node closest to the NW settlement plate) and 50 m (centreline of north embankment) is shown Figure 13. Close agreement was found when comparing settlement data obtained from SAA at 35 m (see Figure 13) and the north embankment settlement data (see Figure 10).

5.3 Fill Settlement

The fill settlement vs. time plots for the south and north embankment are shown in Figure 14 and Figure 15, respectively. The settlement plates on top of the fill have been monitored since 11 Dec 2009. Fill settlement was calculated by subtracting the foundation settlement from the data obtained from the settlement plates on top of the fill.

The maximum fill settlement observed to date for the south and north embankment is equal to 73 mm and 69 mm, respectively.

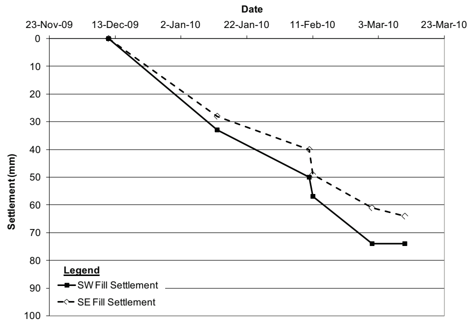

Figure 14. South embankment fill settlement.

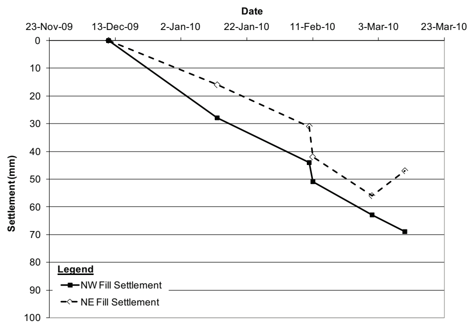

Figure 15. North embankment fill settlement.

5.4 Foundation Pore-water Pressures

Foundation pore-water pressures beneath the south and north embankment fills are shown in Figure 16 and Figure 17, respectively. The contractor on site accidentally cut the vibrating wire piezometer (4A and 4B) cables while installing horizontal sand drains at the south embankment on 24 Sep 2009. The piezometers were replaced by installing two additional piezometers at the same elevation.

At the end of the south embankment construction the pore-water pressure response due to loading was measured to an equivalent total head of 574.9 masl (piezometer 4A). Piezometer 4B did not show much response to loading because it was installed near surface in proximity to a surficial sand drain. This shows that the sand drains were effective in dissipating surficial pore-water pressures close to the drains. At the end of construction the pore-water pressure response due to the north embankment loading was measured to an equivalent total head of 576.25 masl (piezometer 3A) and 574.9 masl (piezometer 4A). The pore-water pressures have been observed to dissipate slowly since the end of construction.

On 13 Nov 2009 the measured pore-water pressures in piezometers 3A and 3B were 575.71 masl and 575.04 masl, respectively. At the same time the fill height at the settlement plate location was equal to 577.9 masl. Approximately 1.1 m of fill remained to be placed to reach the final grade elevation of 579.0 masl.

Since the pore-water pressures were higher than originally anticipated, the contractor was stopped from placing fill near the settlement plates and the end slope of the north embankment. Numerical models were updated to calculate the factor of safety due to the increased pore-water pressures.

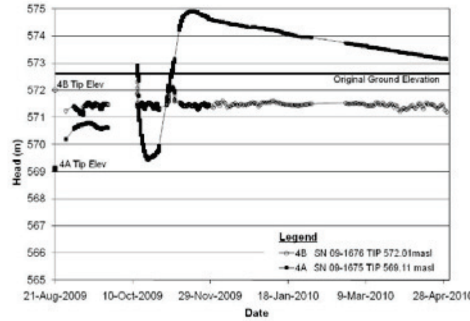

Figure 16. Foundation pore-water pressures underneath S embankment.

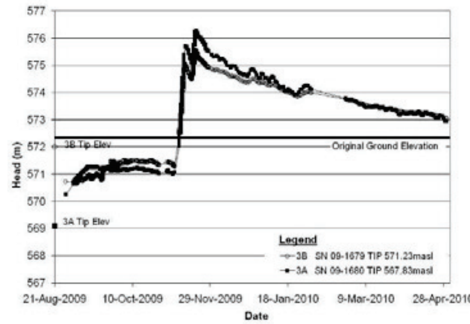

Figure 17. Foundation pore-water pressures underneath N embankment.

Skempton's pore-water pressure parameter $\bar{B}$ was calculated in response to loading as per equation 1 (Skempton, 1954).

$$\bar{B} = \Delta u / \Delta \sigma \quad [1]$$

Where Δu = change in pore-water pressure (kPa)

$\Delta \sigma$ = change in total stress (kPa)

At the end of construction of the south embankment $\bar{B}$ was found to be equal to 0.39 on 09 Nov 2009. For the north embankment, $\bar{B}$ was found to be approximately equal to 0.53 on 13 Nov 2009 and at the end of construction on 20 Nov 2009.

5.5 Slope Stability Models – Update

Numerical models were updated on 13 Nov 2010 for the north embankment to incorporate the measured pore-water pressure response of the foundation and also the as-built height and slope of the embankment. Based on the measured pore-water pressure response it was estimated that the pore-water pressure would be equal to 577 masl when the fill was raised to its final grade.

The calculated *Fs* at the end of the construction for the as-built side slope (5H: 1V) was found to be greater than 1.3. The major concern at the end of construction was the stability of the end slope (3H: 1V) directly north of existing westbound lane of Highway No. 1 and abutment corners where the slope transitions from 3H: 1V to 5H: 1V. Two cross-sections (A-A' and B-B') were analyzed as a part of this study (see Figure 3).

The results of the numerical analysis for cross-section A-A' and B-B' are shown in Figure 18 and Figure 19, respectively. The calculated *Fs* was found to be equal to 1.21 for cross-section A-A' as the existing westbound lane of Highway No. 1 was found to act as a toe berm. The calculated *Fs* was found to be less than 1.1 at the end of construction for cross-section B-B'. A small toe berm was incorporated in cross-section B-B' to increase the calculated *Fs* during construction to 1.1 (see Figure 19).

Based on the results of numerical modelling the following recommendations were made to the construction crew – 1) Incorporate a toe berm (4 m wide with top elevation equal to 573 masl) along the abutment corners, 2) Halt construction if the pore-water pressures approach 577 masl and allow the pore-water pressures to dissipate to maintain a construction *Fs* greater than 1.1.

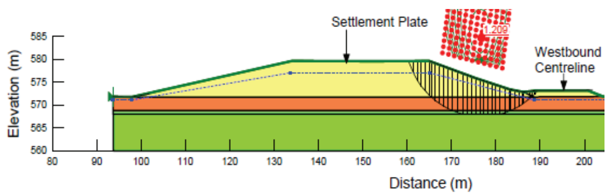

Figure 18. Slope stability result for cross-section A-A'.

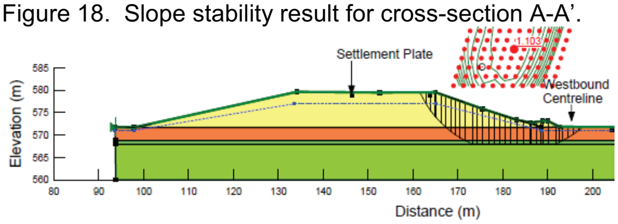

Figure 19. Slope stability result for cross-section B-B'.

6 SUMMARY AND CONCLUSIONS

This paper documents the settlement and pore-water pressure response of Regina Clay due to embankment construction. The close monitoring of the pore-water pressures and foundation settlement were critical to managing the risks associated with fast embankment construction. The numerical models were updated during construction to ensure the calculated factor of safety was greater than 1.1 at the end of construction.

The authors believe that the effectiveness of the monitoring program could have been increased by doing the following 1) Installing additional piezometers at depth to better predict the pore-water pressure bulb, 2) Installing SAA at a distance of 5 m from the toe of the embankment, and 3) Covering the first 5 m of SAA with 1 m fill to prevent the temperature effects on the instrument.

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