Settlement Analysis on Glacio-Lacustrine Clays using Observational Method for a New Interchange in Winnipeg, Manitoba

David Kurz, Ph.D., P.Eng., Rob Kenyon, Ph.D., P.Eng., FEIC.
KGS Group, Winnipeg, Manitoba, Canada
Prokopi Papadimitropoulos, M.Eng., P.Eng., Jeff Tallin, P.Eng.
Manitoba Infrastructure, Winnipeg, Manitoba, Canada
Jeffrey Barrett, M.Sc.E, P.Eng.
Measurand Inc., Fredericton, New Brunswick, Canada

ABSTRACT
A new interchange located at the north end of Winnipeg, MB, was substantially completed in the fall of 2018 after 3 years of construction. This interchange included seven bridge structures and embankments heights up to 9 m above original elevation. The embankments were constructed on 8 to 15 m of compressible clay. Wick drains were installed in the clay to accelerate settlement and meet the project requirements that 90% and 95% consolidation occurs before placing granular base course and paving, respectively. Instrumentation installed throughout the site included 60 settlement plates, 9 ShapeArrays (SAAs), over 40 piezometers, and over 15 Inclinometers. At times during construction, differing magnitudes of settlement between the settlement plates versus the SAAs lead to difficulty calibrating numerical models. Thus commonly practiced methods to estimate degree of consolidation were employed. These methods included Root Time and Asaoka (1978) methods. An exponential function using regression analysis was fitted to the available settlement data to assist with predicting degrees of consolidation across the site. Porewater pressure dissipation was also reviewed. This paper presents a summary of the data at key locations and a comparison to settlements predicted prior to construction.

RÉSUMÉ
Un nouvel échangeur autoroutier situé à l'extrémité nord de Winnipeg, au Manitoba, a été achevé substantiellement en automne 2018, après trois ans de construction. Cet échange comprenait sept ponts et des remblais avec hauteurs variant jusqu'à 9 m au-dessus de l'élévation originale. Les remblais ont été construits sur une couche de 8 à 15 m d'épaisseur d'argile compressible. Des drains de mèche ont été installés dans l'argile afin d'accélérer le tassement et de satisfaire les exigences du projet indiquant qu'une consolidation à 90% et 95% aient lieu avant la pose de la couche de base granulaire et le pavage, respectivement. Les instruments installés sur le site comprenaient 60 plaques de tassement, 9 ShapeArrays (SAAs), plus de 40 piézomètres, ainsi que plus de 15 inclinomètres. Au cours de la construction, des tassements variables entre les plaques de tassement et les SAA peuvent entraîner des difficultés à calibrer les modèles numériques. Par conséquent, des méthodes couramment utilisées pour estimer le degré de consolidation ont été utilisées. Ces pratiques incluent les méthodes Root Time et Asaoka (1978). Une fonction exponentielle utilisant une analyse de régression a été ajustée aux données de règlement disponibles pour aider à prévoir les degrés de consolidation sur le site. La dissipation de la pression interstitielle a également été examinée. Ce document présente un résumé des données aux emplacements clés et une comparaison avec les tassements prévus avant le début de la construction.

1 INTRODUCTION

The interchange is located in northeast Winnipeg, Manitoba, Canada, at the intersection of Provincial Trunk Highway (PTH) 59N (also known as Lagimodiere Boulevard) and PTH 101 (also known as the north perimeter highway), approximately halfway between the Red River to the West and the Winnipeg Floodway to the East. Figure 1 shows its location. Due to increasing traffic flow and an aging infrastructure, Manitoba Infrastructure (MI) selected the existing interchange for an upgrade to a full free-flow interchange in the mid-2010s with construction underway in 2015. The delivery process was Design-Build with the KGS authors being responsible for geotechnical design.

Figure 2 illustrates the overall site layout for the new interchange. The new interchange consists of nine (9) bridge structures at four locations (labeled as 1A/B/C, 2A/B/C, 3, and 4A/B) with final embankment heights up to

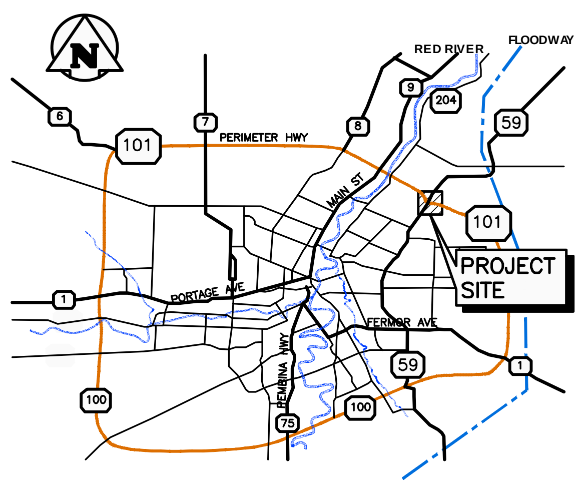


Figure 1. Site Location.

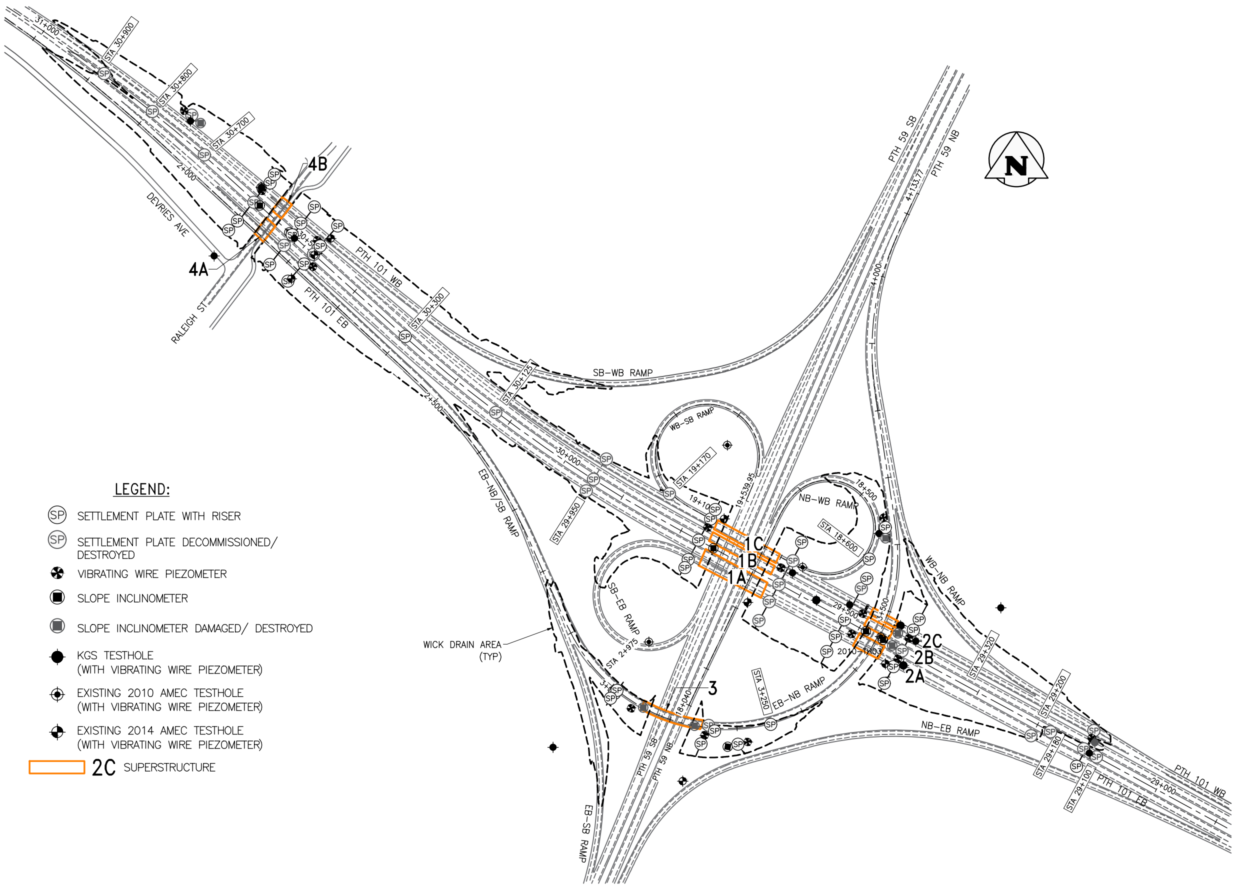


Figure 2. Site Detail.

approximately 10 m above native prairie elevation. From the West, PTH 101 passes over Raleigh St (Structure 4), over PTH 59N (Structure 1) and over the fly-over loop handling EB-NB traffic. The fly-over loop passes over PTH 59N (Structure 3 before passing under Structure 2.

Typical naming conventions at this site used abbreviated locations based on structure number and East or West side (e.g. Structure 1 East as S1E, Structure 2 West as S2W, etc.) and is continued herein during discussion of instruments and data.

1.1 Site Characterization

In general, the site is comprised of approximately 7.7 to 15 m of glaciolacustrine clay overlying approximately 1.7 to 5.8 m glacial till and a dolomitic limestone bedrock. Site investigations were completed in 1965, 1972, and 2004 by MI, in 2014 by AMEC (now Wood.) and 2015 by KGS Group as part of the Design-Build team. Material properties discussed are from Papadimitropoulos (2009), AMEC (2014a) and the 2015 KGS Group site investigation. Material properties for the design phase were prescribed under the contract to be according to AMEC (2014a).

The Lake Agassiz clay dominates the embankment foundation conditions at the site and is generally described as high plastic (CH), silty, moist, stiff, brown to brownish grey in the upper overconsolidated portion and becoming firm to soft and grey with depth in the normally consolidated portion. Trace sands, gravels, silt pockets, pockets of low plastic (CL) clay, and sulphate inclusions are found throughout the mass. The clay had a moisture content range of approximately 23-72%, averaging nearly 50%, and plasticity indices between 30-85%. On average, the hydraulic conductivity was 4.2×10^{-9} cm/s. Pre-consolidation pressures were interpreted to be approximately 134 kPa and 182 kPa for the brown and grey clay, respectively. The overconsolidation ratio for Agassiz clay is typically ranging from 1 to 5, decreasing with depth (Baracos et al. 1980, Baracos et al. 1983, Graham and Shields, 1985).

The glacial till underlying the glaciolacustrine clay varied across the site, but was generally low-medium plastic, moist, loose to compact, becoming dense to very dense with depth, light brown to grey with a variable percentage of sand, gravel, cobbles and boulders throughout the strata. Standard Penetration Tests yielded N-values ranging from 2 to over 100 (without achieving 0.3 m penetration), averaging N-values between 30-40 blows. The till was generally considered incompressible. Hydraulic conductivity of the glacial till was approximately 2×10^{-6} cm/s.

The dolomitic limestone bedrock underlying the glacial till was generally white to yellow, moderately to highly fractured and slightly weathered. The RQD values were *generally* under 50% in the upper 3 m of bedrock and over 50% below 3 m of bedrock.

Groundwater elevations varied spatially across the site and exhibited historical and seasonal fluctuations of a few metres in the brown and grey clay, till, and bedrock. Generally, however, there is a downward flow gradient from the clay to the bedrock.

1.2 Design Requirements in the Design-Build

The design specifications stipulated that 90% of the combined elastic and primary consolidation settlements be achieved prior to placement of any base course materials. Similarly, 95% of the combined consolidation settlement was to be achieved prior to constructing pavements on the base course. An approved settlement model was to be used for the prediction and assessment of settlements.

To achieve these degrees of consolidation within the timeframe of the construction period, vertical drainage systems (i.e. wick drains) were necessary to accelerate consolidation of the underlying compressible soils, as well as to maintain acceptable pore pressures during construction, and to facilitate construction of the pavement surface. Vertical wick drains were prescribed to be installed under any fill heights greater than 3 m with appropriate transitions to areas without wick drains. The wick drains were also required to not penetrate into the underlying till, leaving an *adequate thickness* of clay (i.e. undisturbed) between the bottom of the drain and top of the till to prevent groundwater interconnection. Table 1 shows values prescribed by MI for various consolidation parameters.

Table 1. Prescribed Consolidation Design Parameters

Design Parameter	Soil Type	Design Value
Coefficient of Consolidation C_v (m^2/s)	Brown/Grey Clay	≤2 x 10^{-8}
C_h/C_v Ratio	Brown/Grey Clay	≤1.5
Compression Index, C_c	Brown Clay	≥0.4
	Grey Clay	≥0.6
Recompression Index, C_r	Brown Clay	≥0.14
	Grey Clay	≥0.21

Numerical modeling was required to be reviewed and revised regularly with construction monitoring data such that the settlement predicted by the model remained within 10% of the measured data.

1.3 Wick Drain Design

To satisfy the design requirements and achieve consolidation within the Contractor's construction timeframe, the wick drains were designed with a triangular spacing of:

- 1 m within the core (pavement) footprint of each embankment to 2 m outside the edge of embankment,
- 1.25 m where embankment height was between 6 - 9 m, or in transition zones to 1.5 m spacing,
- 1.5 m where embankment height was between 3 - 6 m.

This spacing was designed such that 95% consolidation would be achieved in as little as 1 year with 90% consolidation in as little as 0.8 year or sooner if field properties were more favourable than anticipated.

The original clay ground surface was first excavated and graded to provide a crown of approximately 2% grade between the centreline and toe of the embankment. In order to meet initial grades and facilitate construction of the crown, some locations had ±1 m of clay fill added. The height of the crown was based on conservative estimates of anticipated settlements to reduce the possibility of creating a 'bath tub' effect after settlement occurs.

The wick drains were installed through a 0.3 m granular drainage layer into the underlying compressible glacio-lacustrine clay. The drains were terminated approximately 1 to 1.5 m above the till layer to satisfy the requirement of no groundwater interconnection. Figure 2 illustrates the full extent of the zone in which wick drains are installed.

1.4 Initial predictions for settlement values

Using the prescribed design parameters, the wick drain spacing, and assuming an average stratigraphy of 5 m of brown clay and 4.6 m of grey clay, with preconsolidation pressures of 134 kPa and 182 kPa, respectively, estimated settlements were between approximately 410 mm to 690 mm for embankment heights between 5 m to 9 m, using 1D consolidation. Results indicated 95% consolidation could be achieved after approximately 1 year using a 1 m triangular spacing of the drains. This was ultimately reduced by a one to two months with a 1 m overbuild.

During final design, KGS Group (2015) conducted cone penetration tests with pore pressure response and measured the in-situ C_h values to be exceeding the MI prescribed value of $3x10^{-8}$ m^2/s (Table 1). A higher C_h value suggested consolidation might be achieved in less time.

Papadimitropoulos (2009) interpreted slightly different values for void ratio and preconsolidation pressure (105-140 kPa and 215-275 kPa, respectively) and compression indexes in his work but they still fell within anticipated ranges for this clay. He interpreted a compression index C_c in the brown clay ranging from 0.27 to 0.46, averaging 0.355, lower than the minimum prescribed design parameter. Similarly, his interpreted recompression indices for the brown and grey clay were lower than the minimum design parameter listed on Table 1. His estimated settlements were 136 mm to 317 mm for embankment heights between 5 m and 9 m, respectively using one-dimensional consolidation.

In comparison, the estimated settlements were 369 mm and 587 mm using the design prescribed AMEC (2014b) consolidation parameters.

2 INSTRUMENTATION

This interchange was fully instrumented as construction proceeded but not all instrumentation was installed at the same time. Figure 2 displays a snapshot of the instrumentation layout. A typical instrumented cross section at an abutment is shown in Figure 3.

Over 40 vibrating wire piezometers were installed at key locations including each approach embankment, key slopes (i.e. 1:1 slopes), a buried pipeline crossing, and at three locations outside of the wick drain zone and beyond the area of construction influence. The vibrating wire piezometers were installed at various depths, depending on location (i.e height of embankment or near abutments) and local stratigraphy. Typically, each location contained a nested installation of two (2) to four (4) piezometers with one (1) each in brown clay and grey clay at a minimum with a potential one (1) to two (2) in either till and/or bedrock or a piezometer placed in the embankment fill.

Over 15 slope inclinometers (SI) were installed at various locations with typically one (1) or two (2) at each abutment plus other key locations, for example, to monitor mechanically stabilized slopes as steep as 1H:1V. SI data is not presented in this paper but only plastic strains were measured with most strains at or under 25 mm. Measured horizontal displacements did however reach 80 mm at two (2) locations. Some locations showed indications of compressive strains due to loading and settlement.

Nine (9) ShapeArrays (SAAs) were installed horizontally with one (1) at each of the eight (8) abutments and one (1) at a pipeline crossing, totaling roughly 770 linear metres of SAA. The length of the SAAs installed at each structure varied based on the width of the embankments. The Structure 1 and Structure 2 embankments were between 160 m to 200 m total length and were comprised of two (2) separate SAAs that overlapped 1.5 m in the centre of the embankment. Single SAA strings were used for Structure 3 (50 m and 55 m), Structure 4 (115 m and 130 m), and the pipeline crossing (43 m). The SAA nodes were spaced 0.5 m for Structures 1 and 2 and 1 m for the remaining locations. The SAA strings (25 mm diameter) were installed in 50 mm PVC casing in the granular drainage layer. The endpoints for each SAA, located at the toe of the embankments, were outfitted with vertical stick-ups to facilitate regular surveys.

Approximately 60 settlement plates (SP) were installed at key locations across the site either in clusters (i.e. at the abutments) or as single plates is less crucial areas. These cluster locations typically included five (5) plates: two (2) at the toe or mid-slope of the embankments, two (2) near each crest or shoulders of the embankments or outside shoulders of the road, and one (1) at the centerline of the embankments. The settlement plates were approximately 0.6 m by 0.6 m plywood plates with a 12.5 mm (½") threaded rod extended to the surface and protected with 25 mm (1") PVC pipe risers. The plates were typically installed at elevations that coincided with SAAs (see Figure 3). Many settlement plates were impacted during construction that lead to questionable readings and either subsequent repair work (i.e. straightening a bent rod) or abandoning of the plate. Settlement plates were surveyed by the contractor approximately every two (2) weeks.

Six (6) data logger nodes were installed at the site with nodes located north of S1W, north of S2E, between S1E-S2W-S3E, south of S3W, south of S4W and south of S4E. Each node used a Campbell Scientific CR6 data logger complete with solar panels and back-up batteries. Piezometers too far to connect to these nodes were connected to separate multichannel data loggers.

3 MEASURED SETTLEMENTS

Monitoring of settlement and degree of consolidation included evaluation of the piezometric and SAA data as well as the surveyed data from the settlement plates and surveyed embankment fill heights. All of the collected instrumentation data was compared to the survey of embankment fill heights and reviewed for trends, outlier data, and other construction activities (e.g. impacted instrument, or spike/drop in readings following localized loading/unloading).

Piezometric data was collected to analyze dissipation trends versus fill heights and time and were compared to the background piezometers located away from construction, and the elevations of the granular drainage

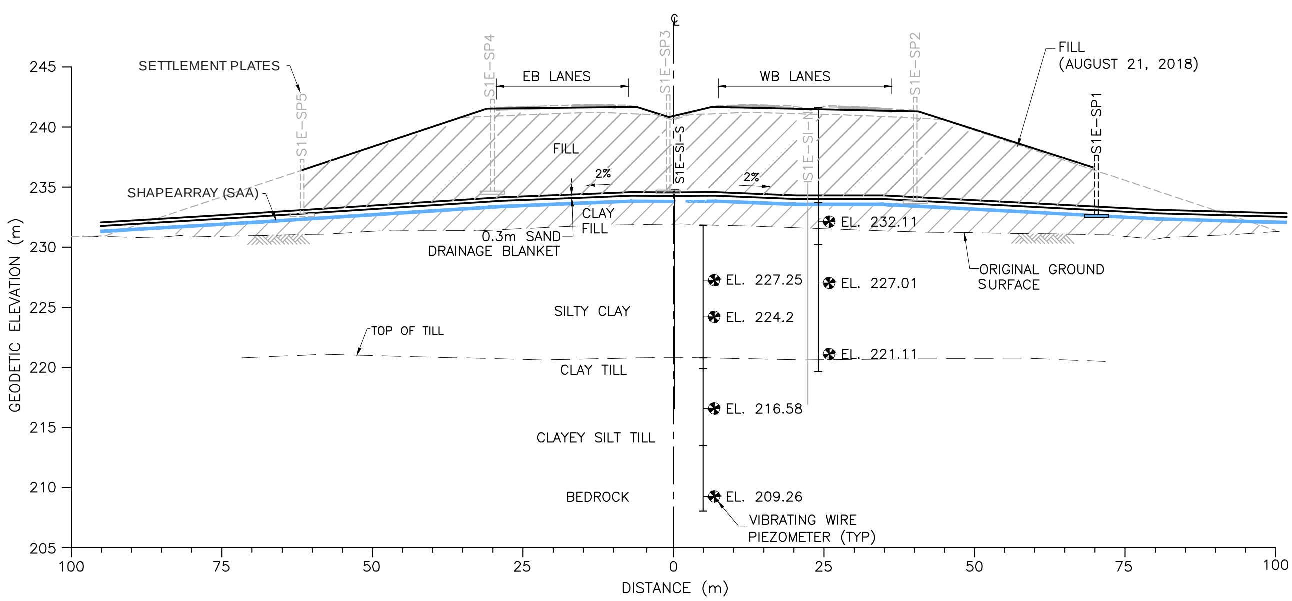


Figure 3. Example Cross Section Detail at S1E.

layer installed at the top of the vertical wick drains. Piezometric data in the embankment fill were used to calculate the respective B-Bar response and overall stability of the embankment. Piezometer data is discussed further in Section 5.

The purpose of the figures and discussion herein are to illustrate the various challenges with interpreting field data from different instruments and estimating the degree of consolidation of embankments of varying heights.

At some locations there were differences between measured settlements by the settlement plates versus the SAAs as high as 400 mm in some locations. This difference was primarily attributed to construction activities in and around the settlement plates. Settlement plates that were impacted and repaired during construction lead to a vertical shift in some data due to the repair. Some settlement plates within the same cluster (i.e. the same abutment) showed readings consistent with SAA readings at a couple of plates but not at others and no known impact or damage to the plate. Figure 4 illustrates the settlement of a centreline settlement plate, SP3, versus the closest corresponding SAA node at S1W, each with different installation times. Figure 5 illustrates the difference in settlement of a centreline settlement plate, SP2 and closest corresponding SAA node at S3E, now with more similar installation times. Overall, the accuracy of the settlement plate data is subject to the level of protection provided during construction and the accuracy of the survey.

Throughout construction, several SAAs displayed erratic rapid downward and upward movements despite no or marginal changes in embankment fill heights while other SAAs displayed no noticeable movements. These movement were observed even under several metres of fill at nodes that did not experience sub-zero (0°C) temperatures. However, despite nodes interior to the embankment not experiencing sub-zero temperatures, movement of the endpoints become translated throughout the entire SAA string as cumulative displacement is calculated in reference to an assumed fixed end. With the help of Measurand, these trends were determined to be a result of freeze/thaw and frost heaving of the endpoints. Applying a manual correction to the SAA data using the survey of the endpoints removes the effect of heave/settlement of the endpoints due to winter.

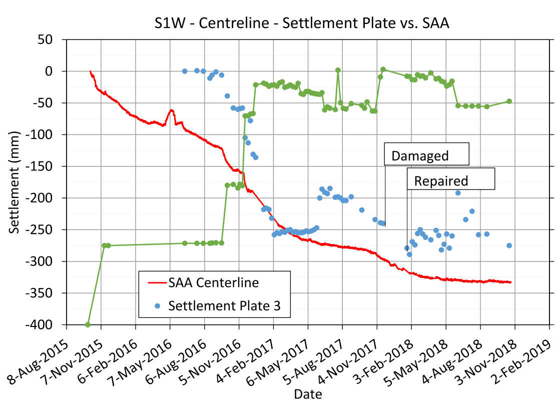


Figure 4. S1W SP3 vs. SAA data, embankment centerline.

Figure 6 displays the raw and corrected SAA data at the centreline of S1W along with the line of best fit through the bi-weekly survey data of the end of the SAA string. Here the apparent impact that winter, frost heave, and partially completed embankments can be seen to have on apparent settlements. There is reasonable correlation (shaded areas in Figure 6) with the temperature data (not shown) with temperatures less than 0°C noted in the SAA in December 2016 and greater than 0°C in April/May 2017. Similarly, temperatures less than 0°C are noted in January 2018 and greater than 0°C in May 2018. The last survey data at the site was obtained in late October 2018. While no survey was completed over the winter 2018/19, the SAA nodes dropped marginally below 0°C but the SAA did not indicate any impact due to winter. The attenuated effect of winter is attributed to increasing soil cover from incomplete embankment toes to partially completed embankment slopes to fully buried SAA endpoints with only vertical survey stick-ups exposed. In April 2017, it was puzzling to see over 50 mm of heave under 4.5 m of fill. Measurand helped improve our understanding of the SAAs and indicated that this 'heave' is related to movement of the endpoints (i.e. due to frost heave). Accounting for the effect of frost heave on the raw SAA data compared to the survey corrected data is vital to understanding the SAA data. Further analysis of 2019 data is ongoing.

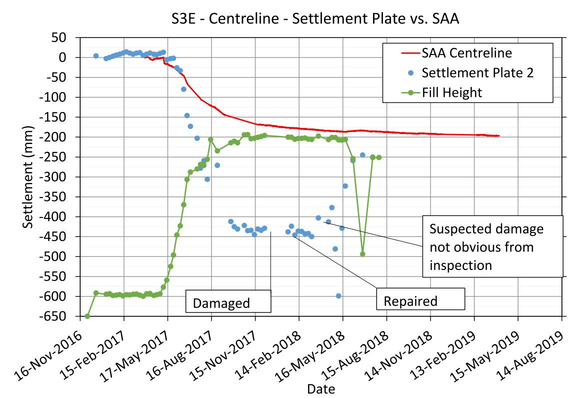


Figure 5. S3E SP2 vs SAA data, embankment centreline.

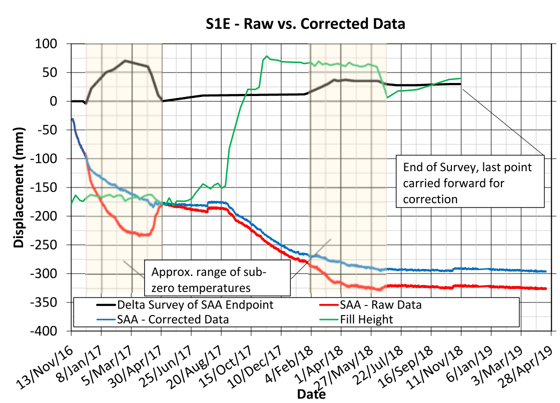


Figure 6. S1E raw and corrected SAA data, embankment centreline.

4 NUMERICAL MODELLING

A design numerical model for this project was initially established to represent the site conditions and for some time was updated regularly with additional loading, changes in geometry of modelled cross sections, and observations from the measured data. The GeoStudio software suite, developed by Geo-Slope International, was used to set up the design models and to provide initial predictions of settlement using the wick drain configurations, slope stability, and pore pressure response. Slope stability modeling is not discussed further in this paper. Papadimitropoulos (2009) used this software to analyze representative embankment cross sections.

It became apparent during construction that separate numerical models would be required at each location if the settlement model was required to be constantly updated to be within a specified 10% agreement with measured data. This was complicated further where there were apparent discrepancies between settlement plate data and SAA data. The prescribed modeling parameters, while generally representative of published parameters for the Agassiz clays (e.g. Baracos et al. 1980, Baracos et al. 1983, Graham and Shields, 1985) also contained restrictions, particularly by limiting values (Table 1) that are potentially not the minimum values found in the Agassiz clays.

Additionally, there was difficulty in determining appropriate parameters for each modeled embankment. The consolidation and hydraulic properties (i.e. OCR, C_c, C_r, C_h, C_v, k_h, k_v, k_{smear}) were found to vary spatially, as one would expect in geotechnical applications, but over short distances within the same model. For example, a set of parameters that closely represented measured data at an embankment centreline was more than 10% out when compared to the shoulders or toe, or vice versa. Also, parameters varied between embankments across the site.

In addition, the parameters selected for a model could correlate well to the observed settlement during initial loading, but could correlate poorly during subsequent loading; either there was a discontinuity in the trend of the settlement plate, or SAA trended to more or less settlement than predicted by the model. This meant that parameters used to adequately predict settlement for an initial fill height became somewhat inadequate (i.e. falling out of the specified 10% tolerance) after the embankment was raised perhaps one (1) or more metres to the next fill height.

5 THE OBSERVATIONAL METHOD & DISCUSSION

Ultimately, it was determined that constant modification of consolidation parameters in the numerical models was not a good approach. It was concluded that estimating the degree of consolidation (U%) should be based on applying commonly practiced analytical approaches. These analytical approaches include Root Time and Asaoka (1978) methods. The Log Time method is not included herein as that method is dependent on knowing full field consolidation parameters in advance whereas the Root Time and Asaoka (1978) methods rely on early and ongoing consolidation data. It is noted that Root Time gives an indication of t_{90} or 90% consolidation and judgment is used for values exceeding 90%.

The Asaoka (1978) method for estimating degree of consolidation (U%) is applied based on measured field data and plotting the data in a S_n vs S_{n-1} plot where S_n is the measured settlement at an equally distributed time step and S_{n-1} is the measured settlement of its previous time step. A linear regression line is derived from the plotted data points and used to determine where the linear regression crosses a 1:1 line to provide an estimate of the degree of consolidation. Where there was a gap or uneven time step in data due to an omission of suspect data, missed survey or damage, an interpolated data point was inserted to respect the required consistency in the time intervals in the Asaoka (1978) method. Figure 7(a) illustrates the Asaoka (1978) method completed for a fill height of 8.9 m on settlement plate #2 (centreline) at S3E. Figure 5 displays the raw survey values for the data presented in Figure 7(a).

Where the quality of data was below average, a “line of best fit” (abbreviated LoBF) approach was used to examine the general trend of the settlement plate (or SAA). This approach used a regression analysis of the data using a fitted exponential curve in the form:

$$S(t) = Ae^{Bt} + C \qquad [1]$$

Where S(t) is the settlement, t is time, and A, B, C are constants. For the final analyses presented in this paper, Equation 1 was defined such that the start time for the analysis of the measured data is generally consistent with the start time for the final embankment loading. The C-value in Equation 1 may be used as the projected final consolidation value.

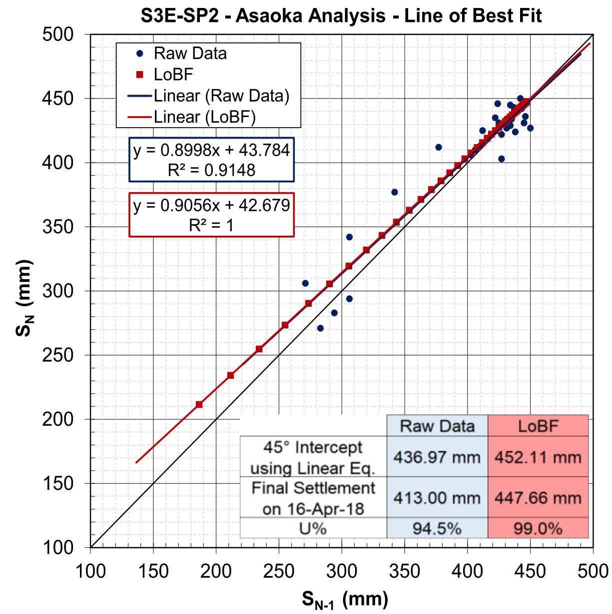
Figure 7(a) also illustrates the raw data and the LoBF of the raw data using Equation 1 which yielded estimated degrees of consolidation of 94.5% and 99.0%, respectively, nearly 11 months after final embankment height was achieved. Figure 7(b) displays the Root Time method using the LoBF and the estimated time of 90% consolidation. The LoBF projects an estimated final settlement of 452 mm and the settlement plate data is near that estimated value. The omitted data in Figure 7(b) is a result of suspect surveyed data or damage to the settlement plate as noted in Figure 5 and also begins to coincide with 1 m offloading.

Figure 8 displays the SAA data with a fitted LoBF and Root Time analysis that also confirms >90% consolidation was achieved. The analyses for the instruments in Figures 7 and 8 began in April/May 2017 and were completed August 2018. Despite the difference in the magnitude of settlement, both the settlement plate (~446 mm) and the SAA (~187 mm) indicated a t_{90} from Root Time would be achieved in October 2017 (~6 months after initial loading), and both analyses indicate that approximately 95% consolidation was achieved by March 2018 (~10 months after initial loading) for the settlement plate and August 2018 (~16 months after initial loading) for the SAA. Comparatively, wick drain design was anticipated to achieve 95% in approximately 1 year.

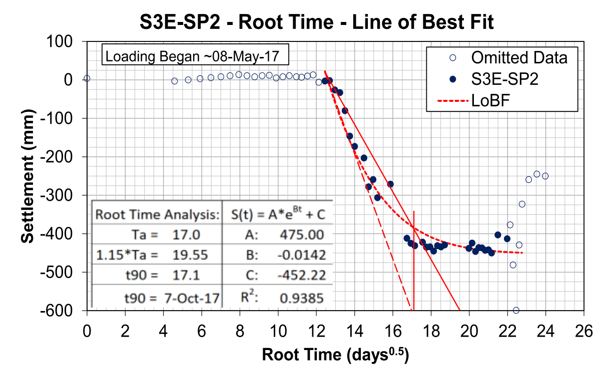
Figure 9 illustrates the behaviour of the SAA at the centreline of S2E and represents a good example of using traditional observational methods to analyze data. The S2E embankment was loaded in four (4) phases. The first phase began immediately after installation of the SAA where fill was raised up to ~1.6 m and remained in place for ~11 months with no appreciable settlement. In the second phase the fill was raised up to ~4.6 m in 2.5 months; however, a malfunction occurred in the data logger that was not resolved until after the 4 m fill height was achieved. The starting time in Figure 9 of 0 root days is 13.5 months since installation with ~73 mm of settlement. The load remained constant at 4 m for 9 months (~16.5 root days) and consolidated to 178 mm. The third phase raised the embankment fill to 9.6 m in 2.5 months and remained constant for ~7 months (~23.5 root days, total ~32 months since installation). The final phase removed the overbuild and lowered the embankment fill height to 8.8 m in ~0.5 months and achieved final design clay fill height.

In the March 2018 (~22 root days), the Contractor was planning summer work and requested a prediction on the degree of consolidation at S2E in order to plan spring/summer construction. Since a LoBF had been established for the first significant loading (Phase 2) with an $R^2 = 0.98$ and predicted settlement of 179 mm, the procedure was to be repeated for Phase 3. Due to the impact of winter (i.e. Figure 6) on the SAA at S2E a prediction was not confirmed until May 2018 (~23 root days). The prediction provided included the Asaoka (1978) method (U% = 97%), Root Time (t_{90} = 21 root days or February 2018), current settlement at 284.5 mm and a maximum predicted settlement, S(t) = 298.5 mm. Although on both a Root Time scale and a linear scale (not shown), the settlements measured by SAA did not appear to be levelling off at 23 root days, the average U% appeared to be over 95% using widely accepted observational methods. The result in combination with removal of the ~0.8 m overbuild proved a high degree of confidence that consolidation was sufficiently complete. As shown in Figure 9, monitoring continued up to the end of October 2018. The analysis in October 2018 indicated U% to be approximately equal to 100% using the Asaoka (1978) method and settlements of 290 mm (current) and 295.8 mm (maximum). Similar results were obtained for the embankment settlements at the other abutments.

The pore water pressure, shown as Total Head, was also used to verify the completion of consolidation settlement of the embankments. Figure 10 illustrates dissipation of Total Head in the upper brown clay and the lower grey clay between S1E and S2W, the final section of embankment raised to full height at site. The start of the Total Head is truncated to show only the dissipation from peak Total Head achieved due to placement of fill. The bedrock piezometer is not included in this paper but shows a lowering trend from El. 220.5 m to 217.3 m during this same period. This begs the question as to how much of the change in pore water pressure in the clay was due to dissipation of excess pore pressure and how much was due to changes in the head in the bedrock. Despite some variation in the data, the Total Head response confirms the same consolidation timeframe as the settlement data. Total Head values are at or below the bottom elevation of the granular drainage layer. The upper and lower clay piezometers indicate a downward gradient implying further consolidation would be via single drainage through the 1-1.5 m of undisturbed clay between the bottom of the wick drain and the underlying till.



(a)



(b)

Figure 7. S3E SP2: (a) Asaoka (1978) method using raw data and a LoBF, and (b) Root Time with LoBF.

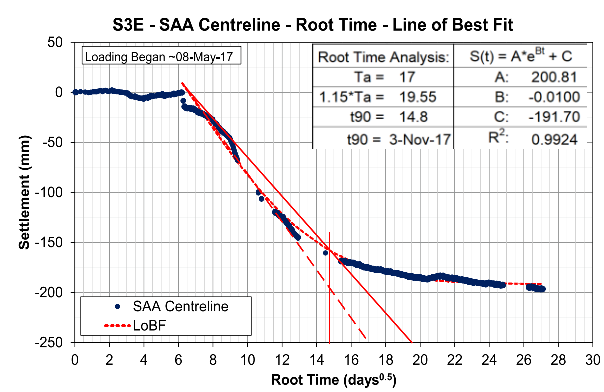


Figure 8. S3E SAA centerline, Root Time with LoBF.

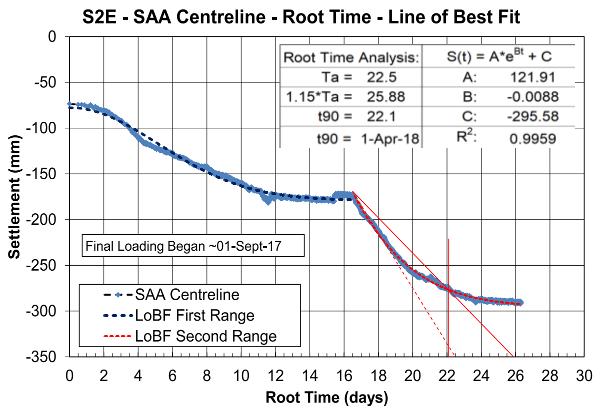


Figure 9. S2E SAA centreline, Root Time with LoBF.

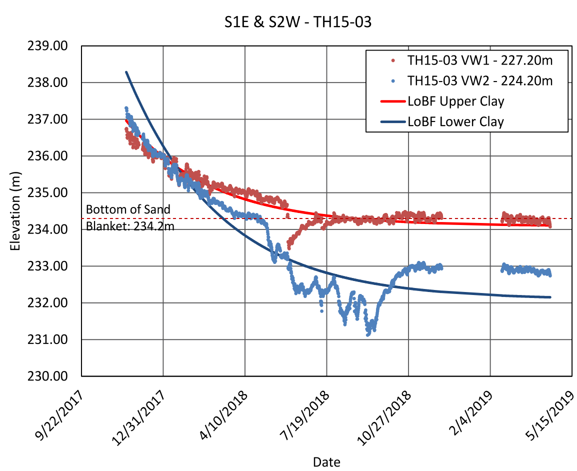


Figure 10. Total Head dissipation - S1E and S2W.

Most piezometers at the site now indicate Total Head to be at or below the elevation of the granular drainage layer; however, at five (5) locations Total Heads appear to have stabilized albeit at elevations as much as 1.4 m above the bottom of the drainage layer. Observations are ongoing; however, there is no indication of issues at the site or in corresponding instrumentation.

6 CLOSING REMARKS

The observational approach using classic analytical methods has been demonstrated to be successful at this newly constructed interchange in Winnipeg, Manitoba. While only a snapshot of the available data is presented herein, the analytical methods were successfully used for estimating the degree of consolidation and for providing conclusions for 90% and 95% consolidation for the site.

Although some variations exist between the various methods, the overall results are reasonably consistent across the site and also consistent with the design intent of wick drain spacing to achieve 95% consolidation in approximately 1 year from attaining full design embankment heights. Additional data is currently being collected for analysis to confirm predictions and performance of the embankments.

While the most consistent and reliable datasets are the SAAs, data from settlement plates are still valuable to observe trends. Current settlements as measured by the SAA range from approximately 130 to 400 mm for final fill heights ranging from approximately 5.3 m to 9.7 m. These settlements are reasonably consistent with initial predictions by Papadimitropoulos (2009) but are on the lower side of the predictions by KGS Group and AMEC (2014b) using properties from AMEC (2014a).

Lessons learned at this site have reiterated the importance of having redundancy in instrumentation to help confirm behaviour and/or data, the value in quality surveys, the protection of instrumentation, and the importance in understanding the raw readings of the instrumentation.

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