

Construction of Road Embankment over Priddis Slough Using Wick Drains and Preloading

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ABSTRACT

To accommodate the continuing growth in the West Macleod area in South Calgary, a new roadway consisting of an approximately 10 m high embankment, known as 194th Avenue S.E. Priddis Slough Crossing, was constructed over an environmentally sensitive low-lying area within a large body of water. The subsurface conditions consist of soft and organic pond deposits over a highly variable sand and gravel layer, over clay deposits ranging from 10 m to 20 m in thickness, underlain by sedimentary bedrock. Area-wide preloading was required to construct the roadway which included constructing a single-span wildlife bridge founded on steel piles. The effects imposed by the on-going settlement was a consideration in the design of the bridge crossing and associated retaining structures. Calculated maximum settlement was in the order of 500 mm. Further, in order to meet the target schedule for road opening, construction of the roadway required installation of area-wide wick drains and preloading technique to accelerate consolidation settlement.

This paper describes the geotechnical and instrumentation program completed during construction, reviews selection of the wick drain material, compares the estimated and measured embankment settlements, and discusses constructability issues during installation of gravel shear keys and wick drains through the surficial sand and gravel layer.

RÉSUMÉ

Afin de répondre aux besoins grandissants de la région Macleod Ouest dans le sud de Calgary, une nouvelle chaussée maintenant connue sous le nom de "194th Avenue S.E. Priddis Slough Crossing" a été construite dans un écosystème sensible comprenant une grande étendue d'eau avec un remblai routier d'environ 10 m de hauteur. Les conditions souterraines consistent en un dépôt mou et organique d'étang sur une couche très variable de sable et de gravier, reposant sur des dépôts d'argile d'une épaisseur variant entre 10 et 20 m, reposant sur un roc sédimentaire. L'aire devait être entièrement préchargée afin de construire la chaussée signifiant qu'un pont à travée unique reposant sur des pieux d'acier permettant à la faune de traverser a dû être mis en place. Les effets du tassement continu ont été considérés dans la conception du pont et des structures de soutènement connexes. Le tassement maximal calculé était de l'ordre de 500 mm. De plus, afin de permettre l'ouverture de la chaussée à la date cible, l'installation de drains verticaux et le chargement de l'aire de construction ont été requises afin d'accélérer le tassement de consolidation.

Cet article décrit le programme géotechnique et d'instrumentation complété durant la construction, examine la sélection des matériaux pour les drains verticaux, compare les tassements estimés aux tassements mesurés dans le remblai et aborde les problèmes liés à la construction des clés de cisaillement en gravier et des drains verticaux à travers la couche de sable et gravier.

1 INTRODUCTION

Construction of a road embankment over organic and thick highly compressible clay deposits is not very common in Calgary, Alberta, given that stiff or better glacial clay till deposits typically overlie much of the city's area. Application of large-scale preloading technique is therefore not commonly implemented in local projects, although it has been used country-wide where thick highly compressible soil deposits are present and when there is a need to expedite the construction schedule.

In 2015, the City of Calgary decided to connect the 194th Avenue between Macleod Trail S.E. and Sheriff King Street S.W. to accommodate the Southwest Calgary Ring Road Project that will eventually remove an existing intersection at Hwy 22X and terminate the only access from the Silverado community to Macleod Trail. The proposed

development consisted of an approximate 3.5 km long new roadway with a road embankment over a low-lying area with standing water (known as Priddis Slough), a bridge crossing (known as wildlife bridge) within the slough, a bridge structure over the future LRT and existing CP rail lines, and other associated retaining walls and culverts. The overall project is known as 194th Avenue Priddis Slough Crossing. Figure 1 shows the project location.

Due to no access across the Priddis Slough with approximate 1.5 m deep of water and soft pond deposits, available geotechnical information was very limited during the design phase. The preliminary designs were completed based on a few shallow test holes drilled on frozen pond surface in winter using small equipment and other nearby test holes located adjacent to the slough. The subsurface conditions were further delineated with a detailed

geotechnical drilling program after the start of construction when a working platform was built across the slough.

During preliminary design, the preliminary estimates of the magnitude and rate of consolidation settlement were reviewed with ISL Engineering and Land Services, who was the designer and construction manager of the overall project. It was decided that preloading of the road embankment was required to accelerate settlement and meet the target road opening schedule in late 2018.

This paper presents the embankment and wildlife bridge designs, details of the preload program, instrumentation used to monitor settlement, comparison of the predicted and actual settlement data and discussion of the challenges during construction of the MSE walls, selection and installation of wick drains.

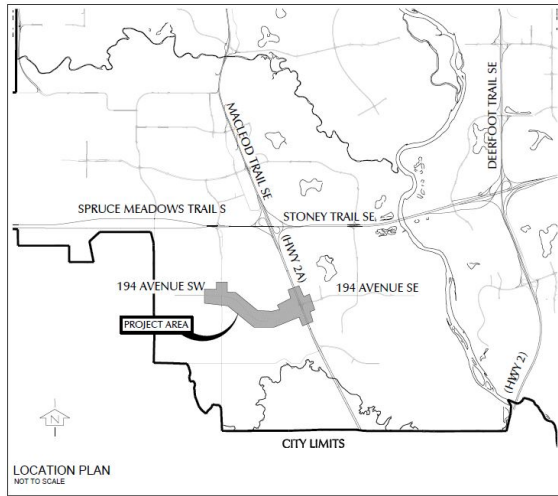


Figure 1. Project Site Location

2 SITE DESCRIPTION

2.1 Surficial and Bedrock Geology

The soil deposits of interest to this study are thought to have been deposited during late stages of the Wisconsin glacialiation, when Lake Calgary drained southward along the ice margin towards a lower level lake in the Highwood River valley. Coarse-grained fluvial channel sediments were deposited along the ice margin during this episode, overriding previously deposited sands, silts and clays of glaciolacustrine and till origin (Moran, 1986).

Bedrock underlying the study area is mapped as non-marine sandstones, siltstones and mudstones belonging to the Paskapoo Formation (Prior et al., 2013).

The surficial geological setting of the project area is shown on Figure 2.

2.2 Subsurface Conditions

As discussed above, limited geotechnical information was available before construction due to no access across the Priddis Slough. A total of thirty Cone Penetration Test (CPT) soundings and three test holes were completed during construction after a working platform was constructed using imported clay fill materials placed over

the standing water and organic pond deposits. The thickness of the working platform ranged from approximately 2 m to 2.5 m, which kept the top of the working platform dry with approximately 0.5 m to 1 m above the pond water level.

The subsurface conditions underlying the Priddis Slough are considerably different than the western uplands and consist of a thicker sequence of glaciolacustrine deposits interbedded with glacial tills, underlain by relatively deep bedrock. Generally, a 1.5 m water and organic clay deposits are present within Priddis Slough over a highly variable sand/gravel layer ranging from less than 1 m to 20 m thick, over a soft to firm, medium to high plastic clay deposit, underlain by bedrock at depths ranging from about 20 m to 30 m. Occasional sand and/or gravel seams were noted within the clay deposits at depths. The CPT tip resistance in sand and gravel layers generally ranged from 15 MPa to 30 MPa and was generally described as compact to dense in the test holes. Typical CPT soundings completed at the west slough, east slough and east shore are shown on Figure 3a, 3b and 3c, respectively.

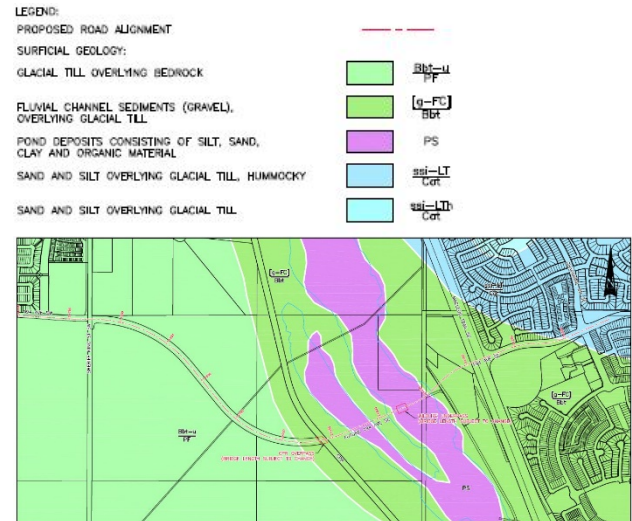


Figure 2. Surficial Geology

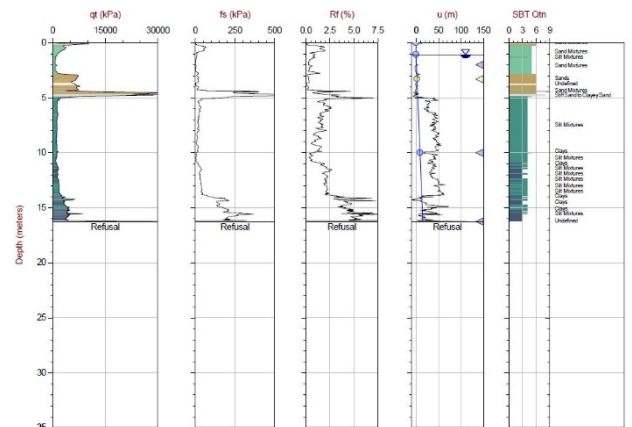


Figure 3a. Typical CPT Sounding Completed at West Slough (Station 2+402)

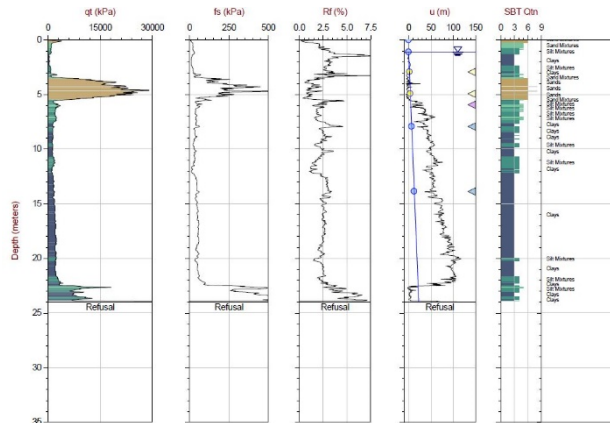


Figure 3b. Typical CPT Sounding Completed at East Sloth (Station 2+642, Wildlife Bridge)

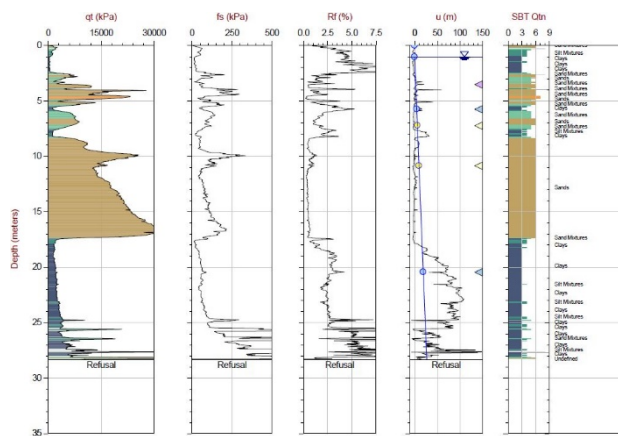


Figure 3c. Typical CPT Sounding Completed at East Shore (Station 2+702)

3 EMBANKMENT, BRIDGE AND MSE WALL DESIGNS

3.1 Embankment

The embankment configuration was designed to satisfy the global stability with the objective to minimize the amount of fill by optimizing the side slope angles. Given the presence of the soft, organic clay pond deposits, a typical embankment side slope of 3H:1V would not be possible to satisfy the global stability. The solution utilized and included in the design of gravel shear keys embedded into the native sand/gravel along the toes of the side slopes. The gravel shear key has a width of two times the fill height (approximate 20 m wide for a 10 m high embankment height) and a depth of approximately 3 m to 3.5 m depending on the depth to the native sand and gravel. These shear keys enabled the embankment side slopes to be designed at 3H:1V to 3.5H:1V to satisfy global stability.

Settlement of the embankment in relation to the anticipated construction schedule was also considered during the design phase. Based on the preliminary settlement analyses using limited geotechnical information, vertical wick drains with a 1 m by 1 m triangular spacing installed to depths between 20 m and 25 m were included

in the preliminary design to accelerate the rate of settlement. Details of the settlement analyses and comparison of the predicted and actual settlement are further discussed in Section 5.

3.2 Wildlife Bridge

The wildlife bridge and the associated MSE wall designs had to accommodate the on-going consolidation settlement as well as maintaining the global stability for short-term during construction and long-term after the road embankment was opened to traffic. The bridge abutments are supported by steel H-piles driven to refusal into bedrock to minimize settlement due to downdrag forces imposed by the on-going ground settlement around the piles. Such pile settlement was estimated by defining the neutral plane (i.e. where the skin friction along the piles changes from negative to positive) and was considered tolerable. The negative skin friction values were provided to the structural designers to check for the pile structural capacity.

3.3 Mechanically Stabilized Earth (MSE) Walls

The MSE head and wing walls under the abutments were originally designed to be founded on a deeper shear key (below the shear keys along the toes of the embankment) to satisfy the global stability requirement. However, after review of the site conditions with the contractor, the deeper shear key was considered very difficult to construct and would require substantial dewatering to facilitate construction of the wall pads. Consequently, the wall pads were raised about 3.5 m higher and the MSE walls had to be redesigned with longer reinforcement straps to satisfy the global stability requirement.

4 CONSTRUCTION OF SHEAR KEYS AND WICK DRAINS

4.1 Shear Keys

The shear keys along the toe of the embankment were installed using imported 80 mm minus crushed sandy gravel placed with approximate 1 m to 1.5 m thick lift over wet condition. Compaction of the gravel materials was achieved as much as possible using the bucket of an excavator. This was considered acceptable based on the reduced strength parameters used in the stability analyses.

Installation of the shear keys under the MSE walls at the wildlife bridge was more complicated since imported 40 mm open-graded washed gravel materials were used to minimize the potential of self-settlement of the gravels and reduced bearing capacity. This required sub-excavation under the pond level to remove the organic pond deposits to expose the native sand and gravel. A non-woven geotextile was then installed over approximately 1.5 m of standing water at the bottom of the excavation. Although installation of non-woven geotextile under water was difficult and physically demanding, it was completed to the satisfaction of the designer and QA/QC inspectors.

Temporary stability of the preload embankment fill during the shear key sub-excavation was analyzed. A

specific construction sequence was required with a short section of about 10 m to 20 m wide excavation at a time to minimize sloughing and instability of the preload embankment fill. Signs of instability, such as tension cracks, were checked frequently throughout during the excavation. The shear key installation was completed without notable instability issues.

4.2 Wick Drains

A project specification with specific details for selection of wick drain materials, quality control and testing, installation equipment and installation procedure was made available to the contractor during the tender phase and project execution. This is further described in the following sub-sections.

4.2.1 Wick Drain Materials

The specification for the wick drain materials is summarized in Table 1.

Table 1. Specification for Wick Drains

Properties	Minimum Average Roll Value
Width	100 mm
Thickness	3.5 mm
Grab Tensile Strength	1.8 kN
Grab Elongation	60%
Grab Strength Filter	970 N
Bursting Strength	1,000 N
Tear Strength Filter	270 N
Permeability Filter	1.3×10^{-4} m/s
Pore Size	75 μ m
Discharge Capacity (200 kPa and unit gradient)	8.20×10^{-5} m/s

The contractor proposed a few wick drain products manufactured by CeTeau based in Netherland with branch offices in Malaysia, Mexico, Thailand and United Kingdom. It required a number of clarifications through the wick drain selection process since the product specifications did not align with the project specification which was based on typical projects completed in North America. Ultimately, the CeTeau Drain product CT-D823D was selected. This was the closest match to the project specification while meeting all the design assumptions and requirements.

An anchor plate with a dimension of 160 mm by 80 mm and a thickness of 1.2 mm was proposed by the contractor. The projected dimension was 100 mm by 80 mm which met the project specification of maximum 8,000 mm².

4.2.2 Wick Drain Installation

The installation equipment was selected based on the following criteria:

- Adequate power to be demonstrated by several trial drains

- Constant rate of advancement methods - Vibratory installation methods, impact hammers, and/or hydraulic jetting not allowed
- Mandrel with uniform and smooth exterior shape that minimized disturbance to the surrounding soils during installation

The APE Model 200 Bottom-Drive wick drain installation equipment was selected by the contractor which had a maximum static push force of 247 kN, a dynamic force of 498 kN and a combined static/dynamic force of 746 kN. This equipment was considered adequate by the contractor at the start of the project based on the preliminary geotechnical information without completion of CPT soundings to assess the density of the native sand/gravel layers underlying the pond deposits. Several trial wick drains were attempted using static push which failed to penetrate through the native sand and gravel. A few of the trial wick drains were attempted using vibratory method (i.e. a combined static and dynamic force) which were successful in penetrating through the native sand and gravel.

After subsequent discussion and evaluation of different installation method, it was decided to allow the contractor to use the vibratory method in light of the very tight project schedule. The wick drain installation continued for about two months when the equipment mast was severely cracked possibly due to the continuing vibration that caused material failure. A second wick drain installation equipment was mobilized to site shortly after; however, it was not equipped with a vibratory driver and therefore, was not capable to penetrate through the native sand and gravel. Consequently, pre-augering of open holes through the native sand and gravel were completed to facilitate installation of the remaining wick drains.

Installation in pre-augered holes allowed a good portion of wick drains to advance to the required depth of 20 m to 25 m; however, a considerable portion of wick drains still encountered pre-mature refusal, possibly due to a second native sand and gravel layer at a greater depth. Furthermore, the anchor plates were frequently broken and detached from the wick drain materials during installation. A double anchor plate setup was required to reduce the frequency of anchor plate material failure.

Upon completion of the wick drain installation program, a total of 10,770 number and 202,790 m linear length of wick drains were installed.

5 EMBANKMENT SETTLEMENT

Estimate of the embankment settlement was initially completed using the preliminary geotechnical information which was limited to data collected adjacent to the slough. The consolidation properties of the underlying clay soils were assessed based on the results of one-dimensional consolidation tests and correlations from the soils' index properties. Although the preliminary settlement estimate was not sufficient to finalize the required wick drain configurations, it concluded that a pre-loading construction technique was required to accelerate consolidation settlement in order to meet the road opening schedule.

Further embankment settlement estimate was completed after additional geotechnical information, which included thirty CPT soundings and three test holes completed within the slough, became available after the working platform was built during construction.

5.1 Settlement Analysis

To estimate consolidation settlement, settlement analyses were completed using the stress-deformation package Sigma/W coupled with Seep/W. A set of representative sections was selected along the alignment, including sections at Station 2+402 (west slough), 2+642 (east slough, wildlife bridge) and 2+785 (east shore where bedrock depth is the greatest). The embankment height at Section 2+402 was about 11.2 m while the other two sections had an embankment height of about 10 m. All three sections had a top width of 40 m and side slopes of 3.5H:1V.

The consolidation properties of the clay soils were estimated from three sources: (1) Correlations from CPT soundings by Mayne (2001, 2005 and 2007) and Kulhawy & Mayne (1990); (2) Results of four one-dimensional consolidation tests; and (3) Correlations from soils' index properties. A comparison of the consolidation properties estimated from Sources (1) and (2) is illustrated on Figures 4 and 5 for the west and east slough, respectively.

As shown on Figures 4 and 5, the consolidation properties correlated from the two sources reasonably aligned with each other. Of particular interest is the over-consolidation ratio (OCR) ranging from 1.43 near the bottom of the clay deposits at about 20 m depth to 3.5 near the top. During placement of the 10 m to 11.2 m high embankment, the OCR used in the analyses would result in an initial portion of the consolidation settlement following the recompression curve (C_r) as over-consolidated clay soils, and a later portion as normal consolidated clay soils following the compression curve (C_c). This was considered reasonable from a settlement estimate perspective, based on our experience of similar lacustrine clay soils in Calgary and understanding of the geological setting in the Priddis Slough area.

The hydraulic conductivity of the clay soils was estimated based on the pore-water dissipation tests completed in the CPT soundings and the results of the one-dimensional consolidation tests. A comparison of the hydraulic conductivity values derived from the two sources is illustrated on Figure 6 for the west and east sloughs. The comparison shows that the hydraulic conductivity values estimated from the one-dimensional consolidation tests were about one to two orders of magnitude lower. Average values were used in the analyses to estimate rate of settlement.

During detailed design, the settlement analyses were completed with different wick drain spacings of 1 m, 1.5 m and 2 m to evaluate the rate of consolidation settlement to meet the construction schedule. At the wildlife bridge where the construction schedule was more critical, a 1 m wick drain spacing was selected. Away from the wildlife bridge, a 20 m to 30 m transition zone (along the length of the embankment) with 1.5 m wick drain spacing was selected and the rest of the embankment was installed with

wick drains at 2 m spacing. The selection of the wick drain spacings was based on the criteria of maximum 100 mm and 150 mm post-construction settlement at the wildlife bridge and the rest of the embankment, respectively. The post-construction settlement was defined as the total long-term settlement after the embankment fill was in place for a 6-month period (i.e. after a 6-month preload period).

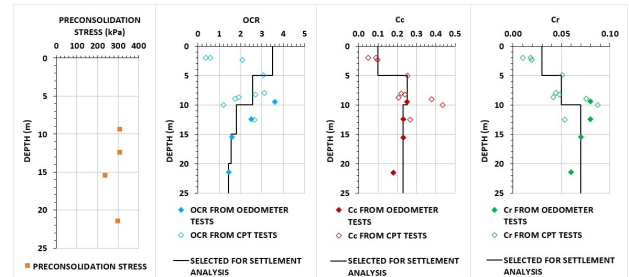


Figure 4. Consolidation Properties for Clay at West Slough

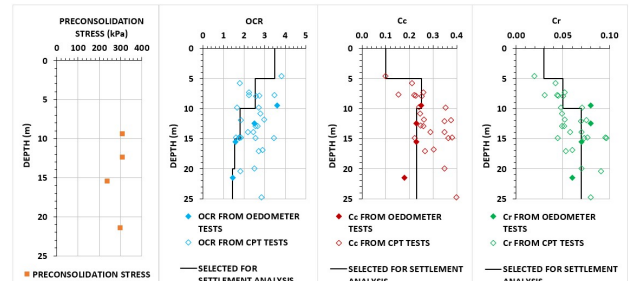


Figure 5. Consolidation Properties for Clay at East Slough

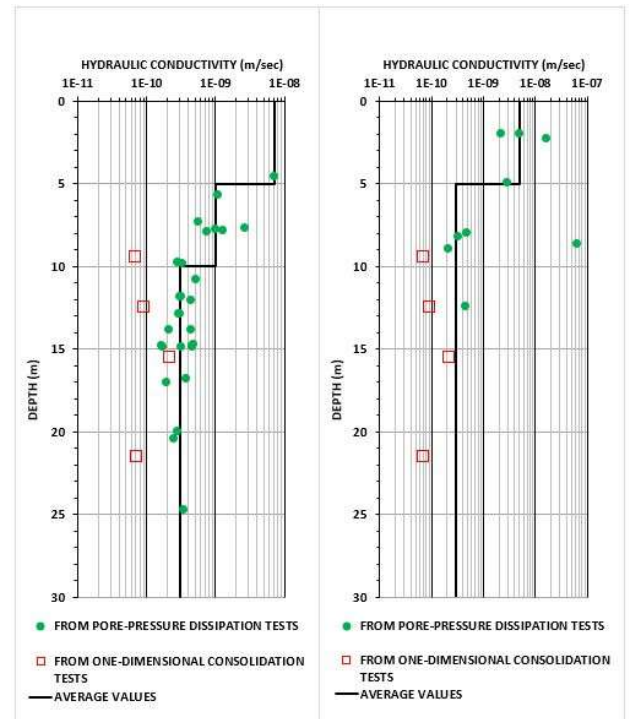


Figure 6. Hydraulic Conductivity Values for Clays in West and East Sloughs

5.2 Monitoring of Embankment Settlement

Several instrumentation options, including Shape Array Accelerometers (SAA), were evaluated in terms of effectiveness, constructability and cost. Settlement plates were selected to monitor the embankment settlement. Settlement plates were installed at the three sections discussed in Section 5.1.

The settlement plates consisted of a 0.6 m by 0.6 m square plywood connected to multiple steel riser pipes encased within PVC casings. The steel riser pipes were extended as the embankment fills were built up. The top of the steel riser pipes and the surrounding embankment elevations were surveyed on a daily basis to record the actual embankment settlement in relation to the placed embankment fill at the three sections.

5.3 Estimated and Actual Embankment Settlements

5.3.1 Section 2+642 (Wildlife Bridge at East Slough)

The overall embankment fill placement started at the wildlife bridge location to accommodate construction of the associated MSE walls after the target 100 mm post-construction settlement was reached. The embankment fill reached the full height of 10 m in December 2017 and an additional 2 m of surcharge fill was placed to accelerate the consolidation settlement. The additional 2 m surcharge fill was removed in February 2018 when 360 mm of settlement had occurred, about 78% of the interpreted total settlement of 462 mm. The 10 m preload fill at the wildlife bridge was removed in late April 2018 to facilitate the bridge foundation and MSE wall construction when 400 mm of settlement had occurred, about 86.5% of the interpreted total settlement. The remaining settlement was estimated at about 60 mm which met the design criterion of maximum 100 mm.

The estimated and actual settlements together with the embankment fill height are shown on Figure 7.

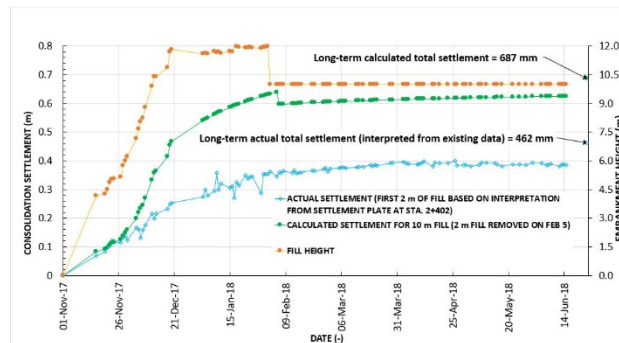


Figure 7. Estimated and Actual Settlements at 2+642 (Wildlife Bridge at East Slough)

The total estimated settlement was 687 mm which was 49% more than the interpreted settlement of 462 mm. As shown on Figure 7, the substantial difference became notable after the embankment fill reached a height of about 2 m when the estimated settlement followed a relatively steeper path (i.e. increased rate and magnitude of settlement) while the actual settlement curve was consistent with that during the first 2 m fill placement. This

can be explained by the OCR values selected in the analyses. As discussed above, the settlement calculation was based on compression index (c_c) for a normal consolidated clay after the embankment fill height exceeded 2 m. The actual settlement curve indicated that the clay soils behaved as over-consolidated soils during the entire embankment fill placement to the full height of 10 m.

One possible explanation to the inconsistency between the over-consolidated and normal consolidated responses of the clay soils at this site is the stress distribution from the embankment fill through the native sand and gravel, to the underlying clay soils. It is postulated that some of the embankment fill load may have been distributed across the dense sand and gravel layer (see Figure 3b) and resulted in a reduction of vertical stress onto the underlying clay soils. Although this was considered in the settlement analyses, it is possible that the analyses may have underestimated the reduction of vertical stress through the dense sand and gravel layer. However, this could not be verified since the vibrating wire piezometers installed within the clay soils did not react to the increasing embankment fill load during placement due to the presence of wick drains surrounding the piezometers, which relieved the stress induced pore-water pressure laterally and up through the wick drains.

5.3.2 Section 2+402 (West Slough)

The embankment fill at the west slough reached the full height of 10 m in April 2018 when the settlement reached 302 mm, about 57.5% of the total estimated settlement of 525 mm. The settlement reached 375 mm in July 2018, about 71.4% of the total estimated settlement, when it met the design criterion of maximum 150 mm of total post-construction settlement.

The estimated and actual settlements together with the embankment fill height are shown on Figure 8.

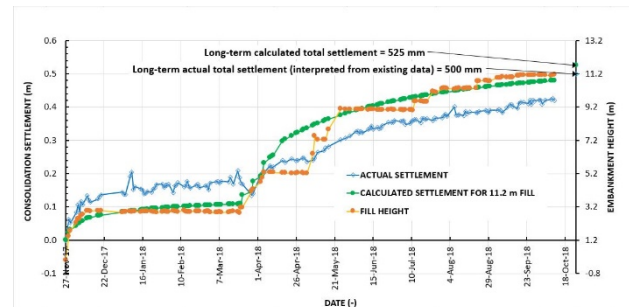


Figure 8. Estimated and Actual Settlements at 2+402 (West Slough)

As shown on Figure 8, the actual interpreted settlement of 500 mm is only 5% less than the estimated settlement of 525 mm. It is difficult to evaluate the over-consolidated and normal consolidated clay soil behavior from the settlement curves since the fill placement was not completed at a consistent pace. However, unlike the embankment settlement at the wildlife bridge, it is believed that some of the clay soils behaved as normal consolidated soils since the dense sand and gravel layer was much thinner (see

Figure 3a) and much of the embankment fill load would transfer onto the underlying clay soils. This is supported by the consistency between the estimated and actual settlement.

5.3.3 Section 2+785 (East Shore)

The actual settlement measured at the east shore was highly erratic. About 30 mm or less settlement was measured when the embankment fill reached a height of 7.5 m, and it dramatically increased to 110 mm when the embankment height increased from 7.5 m to 10 m. The total settlement was measured at 131 mm in March 2019, about 9 months after the embankment reached the full height, which was significantly less than the estimated settlement of 491 mm.

One explanation of the erratic and less settlement is the presence of irregular sand and gravel layers at the east shore (see Figure 3c) which were frequently found to be interbedded in the clay soils. It is believed that much of the embankment load was distributed across the multiple sand and gravel layers which resulted in significantly lower vertical stress onto the clay soils resulting in less settlement.

6 CONCLUSION

Embankment design and construction of the road embankment over Priddis Slough are described in this paper. Specific details with respect to the challenges encountered during shear key installation, wick drain material selection and installation are also discussed.

Settlement estimates including the selected soils' consolidation properties in the analyses are discussed and compared to the actual settlement measured during construction. The magnitude of settlement was found to be variable in response to the presence of the overlying sand and gravel layers, which was believed to have distributed the embankment load laterally and reduced the vertical stress onto the underlying clay soils. The variable sand and gravel layers were encountered in two of the three sections which showed lower settlement than estimated. At one of the three sections where the presence of the sand and gravel layers was insignificant, the estimated settlement closely matched the actual settlement with a difference of less than 5%.

The actual rate of settlement was found to be more favourable, since preload periods of 3 months at the west slough and 4 months at the east slough were required compared to the estimated 6-month preload period. It is believed that average hydraulic conductivity values selected in the settlement analyses were conservative, and the upper values would yield an estimated preload period closer to the actual 3 to 4 months. Nonetheless, the rate of settlement was the critical factor during construction. The more favourable rate of settlement provided greater flexibility and allowed the project to complete in time for the target road opening.

The road was officially opened on November 2, 2018, following a large celebration on-site from all parties involved and the City of Calgary.

7 REFERENCES

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