Consolidation Behaviour of Colluvium Derived from Overconsolidated Glaciolacustrine Deposits Under Highway Embankment Fills

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ABSTRACT
Upgrading the Alaska Highway 97 at South Taylor Hill, BC, across a slow-moving landslide complex, required placing fills, as well as constructing stabilizing toe berms. One of the challenges faced in the project was determining the rate of pore-pressure dissipation in colluvium, derived from overconsolidated interbedded glaciolacustrine silts, sands and clays, underlying the fills. The colluvium was expected to retain some of the overconsolidated behavior, but still have a greater rate of pore pressure dissipation due to the disturbance and presence of higher permeability sands. Oedometer consolidation and cone penetration dissipation tests provided a wide range of consolidation coefficients. Pore pressures in the colluvium were monitored throughout, allowing comparison of observed pore pressure dissipation with laboratory and CPT based estimates. Understanding the variability will give greater confidence in determining the consolidation behavior of colluvium derived from overconsolidated materials and allow for this behavior to be accounted for in the geotechnical design.

RÉSUMÉ

L'élargissement de la route 97 (Alaska Highway) le long du versant sud de la Vallée de la rivière de la Paix au niveau de Taylor, Colombie Britannique, a été compliqué par la traversée d'un complexe de glissement de terrain à taux de fluage réduit, ce qui impliqua le placement de remblai et la construction de risbermes de pied. L'un des défis du projet fut de déterminer le taux de dissipation des pressions interstitielles au sein des dépôts colluviaux dérivés de couches glacio-lacustres surconsolidées de sable, limon, et argile inter-stratifiés, situés sous le remblais. Il était anticipé que les dépôts colluviaux aient préservé leurs caractéristiques de surconsolidation tout en présentant des taux de dissipation de pression interstitielle accrus, reflétant la présence de sable et les perturbations de surfaces associées à la construction. Les résultats d'essais oedométriques de consolidation et de dissipation par CPT ont indiqué une fourchette de coefficients de consolidation très large. La mesure en continu de la pression interstitielle dans les dépôts colluviaux pendant les travaux a permis de comparer les pressions observées à celles dérivées des essais de laboratoire et de CPT. Une meilleure compréhension de la variabilité devrait permettre de déterminer les facteurs régissant la consolidation des dépôts colluviaux dérivés de matériaux surconsolidés avec plus de certitude et d'en intégrer les éléments clés à la conception géotechnique.

1 INTRODUCTION

The British Columbia Ministry of Transportation and Infrastructure (MOTI) has implemented a staged approach to improving a segment of Alaska Highway 97 approximately 4 km south of Taylor, BC, locally referenced as South Taylor Hill (Figure 1). This segment of the Alaska Highway traverses a large active landslide complex located along the south wall of the Peace River Valley. The highway improvements include widening from the existing two-lane geometry to a four-lane geometry and reducing the highway grade. Slide movements have affected this segment of the highway since the road was built along the current alignment by the US Military in 1942. Construction of the Alaska Highway connecting Dawson Creek to Delta Junction in Alaska was completed by the US Military in nine months.

2 GEOTECHNICAL INVESTIGATION

2.1 Site Setting

South Taylor Hill is located on the south wall (right bank) of the Peace River Valley in the Great Plains region to the east of the Rocky Mountain foothills. The total relief between river level and the valley plateau at the top of South Taylor Hill is approximately 300 m. South Taylor Hill is characterized by hummocky topography caused by landslides. Sag ponds (ponded surface water) are common within local topographic lows on the landslide disturbed terrain. Through this section, Highway 97 climbs the hill on a diagonal path at an average gradient of approximately 6 percent. The overall slope of the hill is approximately 18 percent.

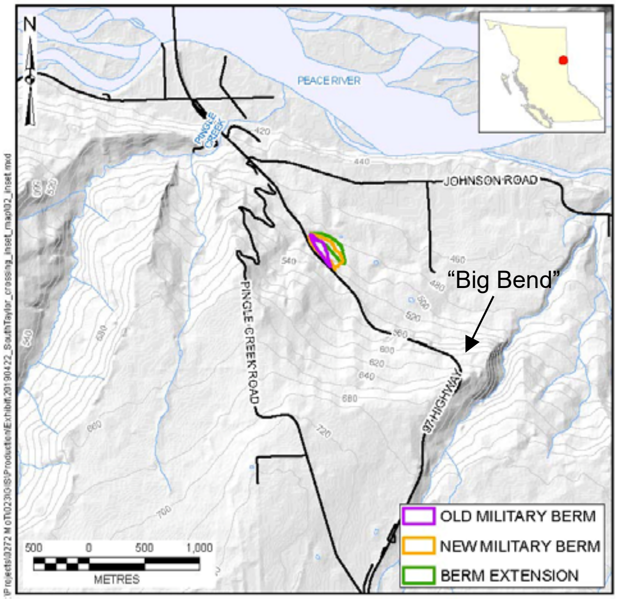

Figure 1. Project Location Plan

2.2 Geology

The overburden geology of the area (Figure 2) reflects the influence of periodic Laurentide glacial advances from the east. The glaciations were separated by interglacial periods during which relict fluvial drainage systems formed, each of which graded to a different base level. At South Taylor Hill, the modern Peace River roughly follows the path of the youngest paleovalley (referred to as the Lower Paleovalley), exposing the Pleistocene sediments (Glacial Lake Mathews glaciolacustrine silts, sands and clays and Laurentide till) that infilled the relict valley during the last (late Wisconsinan) Laurentide ice advance and retreat (Stott (1982), Hartman and Clague (2008) and BGC (2012)). Bedrock geology units found at South Taylor Hill comprise Cretaceous marine shales of the Shaftesbury Formation that conformably underlie Cretaceous non-marine sandstone, siltstone and shale of the Dunvegan Formation (Stott 1982).

2.3 Geotechnical Design Overview

The geotechnical design involved adopting a highway alignment that minimized cuts into the toes of deep-seated landslides in the Glacial Lake Mathews unit above the highway by moving the widened highway out onto granular embankments supported by earthen berms built to buttress the landslides crossed by the highway. Settlement of the highway embankments caused by the consolidation of the underlying relatively low permeability materials was therefore expected. In some specific areas where short-term slope stability was considered to be a potential issue during construction, site-specific recommendations were made for monitoring pore pressure response of the colluvium under the earthen berms and/or slope deformation during construction, with the need to consider staging of construction to allow adequate time for excess pore pressures that may develop, to dissipate. A key task prior to, during, and post construction, was to read piezometers and slope inclinometers installed upslope and within the berm footprint on a regular basis to understand the impact of the placed fills on the groundwater pressures and slide movement.

2.3.1 Surface Drainage

Surface drainage improvements to reduce infiltration of water into the ground and help direct surface runoff away from areas where slope movements could affect the highway were an essential component of the project. Thus, identifying opportunities to improve the surface drainage in areas of known landslide activity was a key component in the geotechnical design assumptions, with emphasis made on drainage of the network of sag ponds on the landslide affected terrain above the highway.

2.4 Ground Investigation

2.4.1 Geotechnical Drilling

As part of the geotechnical design a series of boreholes were drilled in the vicinity of the project to facilitate detailed

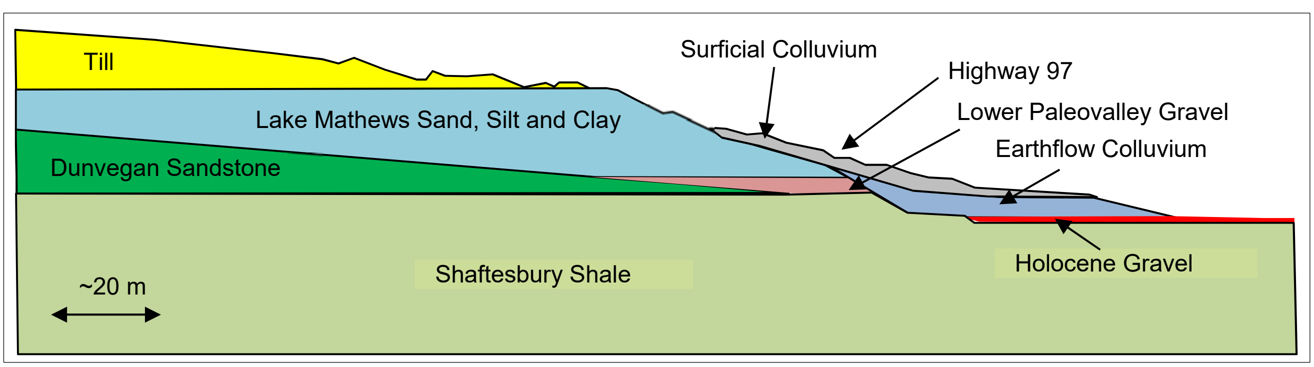

Figure 2. Simplified West-East geological section through the project area

geological logging and sampling for laboratory testing. The majority of boreholes were installed with inclinometers and vibrating wire piezometers (VWP) to facilitate monitoring of slide movements and groundwater levels.

2.4.2 Cone Penetration Testing

Cone penetration testing (CPT) was carried out in the area proposed for a large fill berm known as the Military Berm (named after an existing berm at that location considered to have been placed by the US military during the road's initial construction in 1942). CPT holes were advanced using a truck mounted rig with a ballasted weight of 30,600 kg. Piezocones with tip areas of 15 cm^2 and load capacities of 9 and 18 tonnes were used. Cone tip stress, sleeve stress and pore pressure were continuously recorded as the cone was advanced. The pore pressure readings were correlated with equilibrium pore pressures recorded in nearby VWPs. The CPT results were used to estimate undrained shear strength values and overconsolidation ratios, and to cross-validate the classification of soil in nearby drill holes. Pore pressure dissipation tests were carried out at regular intervals and at targeted locations in several of the CPT holes. The results were used to estimate hydraulic conductivities and consolidation coefficients using methods presented in Kulhway and Mayne (1990), Robertson et al. (1992) and Lunne et al. (1997). Consolidation testing (1D oedometer) was also carried out on core soil samples collected during the investigation with the results used to estimate consolidation coefficients and compression indices using methods presented in Casagrande (1936) and Taylor (1948).

2.5 Consolidation-settlement and Pore Pressure Dissipation

Consolidation parameters derived from 1D oedometer tests indicated that the colluvium retained some of the overconsolidated characteristics of the parent Glacial Lake Mathews deposits. Overconsolidation ratios (OCR) measured in the colluvium unit were typically greater than 2. Pore pressure dissipation tests carried out during CPT in the colluvium unit also displayed an overconsolidation signature, in which dissipation was typically preceded by an initial rise in pore pressure (Campanella et al., 1986).
Based on the oedometer test results on colluvium samples taken from a borehole near the highway, consolidation settlements were estimated to range up to approximately 0.2 m along the crest of the new highway embankment where the thickness of the underlying colluvium unit is approximately 25 m. This estimate was based on an average existing overconsolidation ratio of 2, a minimum effective preconsolidation pressure of 150 kPa, a compression index of 0.18, and a recompression index of 0.06 for the colluvium deposits.

Consolidation coefficients based on oedometer tests on clayey samples from the colluvium unit ranged between approximately 1 and 10 m^2/yr. However, consolidation coefficients based on the CPT dissipation tests, which were carried out at regular depth intervals and were therefore not biased to a certain sample composition, ranged up to three orders of magnitude higher than that from 1D oedometer tests, reflecting the influence of higher permeability sandy zones within the colluvium deposits. It was expected that higher permeability zones would influence the overall rate of consolidation in the colluvium and non-engineered fill units in the Military Berm area and that consolidation times would be relatively limited. However, the spatial variability of these units meant the expected overall rate of consolidation was difficult to predict from the available data. Consequently, it became necessary to determine the actual pore pressure response and settlement rates of these materials through careful monitoring during construction.

Based on the CPT dissipation results, the time required to achieve 90 percent consolidation of the colluvium deposits below the new highway embankment was estimated to range between approximately 1 and 4 years, while the time required to achieve 50 percent consolidation was estimated to range between approximately 3 months and 1 year. These estimates were based on estimated representative average consolidation coefficients ranging between 20 and 70 m^2/yr (based on median values from the CPT dissipation results) and an estimated representative drainage path length of 10 m.

Given the high degree of variability in the available dissipation test data, estimates were associated with a high degree of uncertainty. However, comparable pore pressure dissipation rates had been observed by MOTI under similar embankment construction and geological conditions during a culvert replacement project near Dawson Creek, BC and during waste embankment construction completed as part of previous improvement works within the South Taylor Hill project area near the 90-degree bend known as the 'Big Bend' (Figure 1) in 2003.

2.6 Pore Pressure Monitoring during Construction

During design, slope stability under short-term undrained loading conditions (i.e. during embankment or berm construction) in the relatively low permeability glaciolacustrine and related silts, sands and clay materials, was modelled using an effective stress analysis approach. This was achieved by applying pore pressure coefficient $\overline{B}$ (the ratio of the resulting change in pore water pressure at depth to the change in vertical total stress due to loading; Skempton, 1954) to the colluvium in the model. A $\overline{B}$ value of 0.7 was initially selected based on work by Bishop (1954) who reported $\overline{B}$ values ranging between 0.5 and 0.7 for sandy clay and clay-gravel materials used in dam construction. This was supported from monitoring data obtained during a culvert replacement project by MOTI near Dawson Creek, BC, where $\overline{B}$ values up to 0.6 were observed as fill was placed on glaciolacustrine deposits with similar characteristics to those found at South Taylor Hill. Slope stability back analysis indicated the Factor of Safety was near 1 with $\overline{B}$ values of 0.7. Therefore, during embankment construction (between 2014 and 2016), a threshold for observed average $\overline{B}$ values was set at 0.7. Exceeding this threshold was to trigger suspension of fill placement and allow adjustment of the design and/or construction sequence/method if needed before proceeding.

During construction average $\overline{B}$ values greater than 1 were observed in the colluvium and theoretically values should not exceed 1. The $\overline{B}$ values were calculated from pore pressure response within the colluvium, recorded at VWP locations where there was no fill or a reduced fill thickness placed directly over the piezometer. At some locations the pore pressure response observed was significantly higher compared to the actual rise in total vertical stress as a result of fill placement. This suggested that the pore pressures were responding to fill placed elsewhere within the berm footprint and that use of an average $\overline{B}$ threshold to monitor for excess pore pressure during construction was somewhat unreliable. However, it was noted that when average calculated $\overline{B}$ values exceeded 1, in some cases much greater than 1, slope movements that had not been recorded prior to filling were noted at depth in the colluvium, based on recorded slope inclinometer readings suggesting an influence of shear-induced pore pressures. Ultimately an extension to the initial berm was required due to the observed deeper ground movements.

2.7 Long Term Pore Pressure Response

The pore pressure response following construction was monitored with 20 vibrating wire piezometers installed in 8 boreholes (Figure 3).

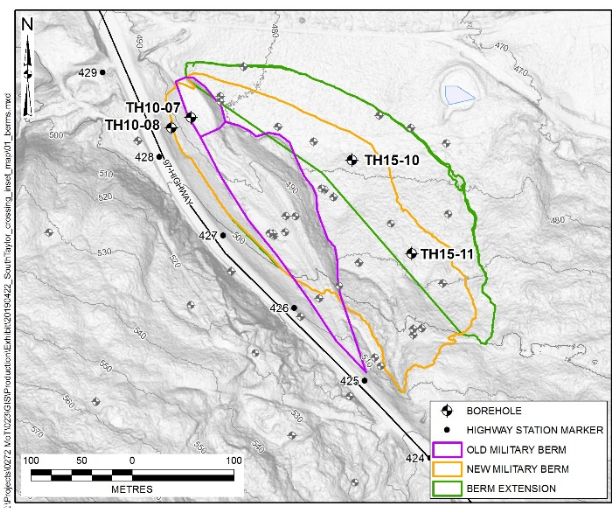

Figure 3. Borehole layout plan in Military Berm Area. Vibrating wire piezometer data and calculated pore-pressure dissipation rates for labelled boreholes summarized in Table 1.

Data were collected on either one-hour or six-hour intervals using automated dataloggers. The results from nine VWPs installed in four boreholes where there are pre-construction baseline readings and post-construction records up to March 2019 are discussed in more detail.
Pre-construction baseline pore pressures were established for the VWPs installed in boreholes 10-07 and 10-08 using an average of the pore pressure readings from April 01, 2011 through March 31, 2014. Pre-construction baseline values were established for the piezometers installed in 15-10 and 15-11 using an average of the piezometer readings from May 1, 2015 to June 30, 2015. The pre-construction baselines excluded the post-installation pore pressure stabilization period. The period considered for the baseline for 10-07 and 10-08 was prior to the start of construction. Existing instrumentation was extended carefully through the embankment fill during construction (Figure 4). However, for 15-10 and 15-11 some fill had been placed in the vicinity of the boreholes prior to their installation.


Figure 4. Extending instrumentation (inclinometer and VWP) through the embankment fill using ribbed large diameter plastic conduit and careful hand packing of sand inside the conduit and careful packing of berm fill around the casing exterior.

The VWP records for the post-construction period were examined, and a post-construction baseline was defined using an average for the stable readings extending from the last record back one year. The post-construction pore pressure dissipation rate was calculated and compared with the pre-construction estimates. To estimate the times to 50% and 90% pore pressure dissipation, the seasonal fluctuation in the data was smoothed using an exponential decay function fit to the data. These features are shown in Figure 5.

Precipitation records from the Fort St. John airport weather station for the monitoring period were compared to the records for the last climate normal period, 1981 to 2010. As shown in Figure 6, six out of the past seven years have had precipitation greater than the climate normal average, which could be contributing to overall higher pore pressures.

3 DISCUSSION

Results of the pore pressure dissipation analyses are summarized in Table 1.

One finding from the pore pressure dissipation monitoring was that in all but one instance, the post-construction pore pressures have apparently stabilized at a pore pressure different than the pre-construction baseline. There is higher uncertainty in the pre-construction baseline for the VWPs installed in boreholes 15-10 and 15-11 as the averaged readings span a much shorter time period and the instruments were installed after construction had already commenced, however, significant differences are also noted in the results from VWPs installed in 2010 where there was a three-year pre-construction average used as the baseline. Additionally, there is some uncertainty with respect to the post-construction baseline, as a longer monitoring period would be needed to more confidently assess a stable post-construction pore pressure. There are many factors which could influence the post-construction pore-pressures. The drainage blanket installed beneath the base of the berm and the upslope sag pond drainage system could cause a reduction in the pore pressure compared to pre-construction conditions. Increased precipitation, closing fractures and compressing infilled material between more intact colluvial blocks could result in the observed increase in pore pressures.

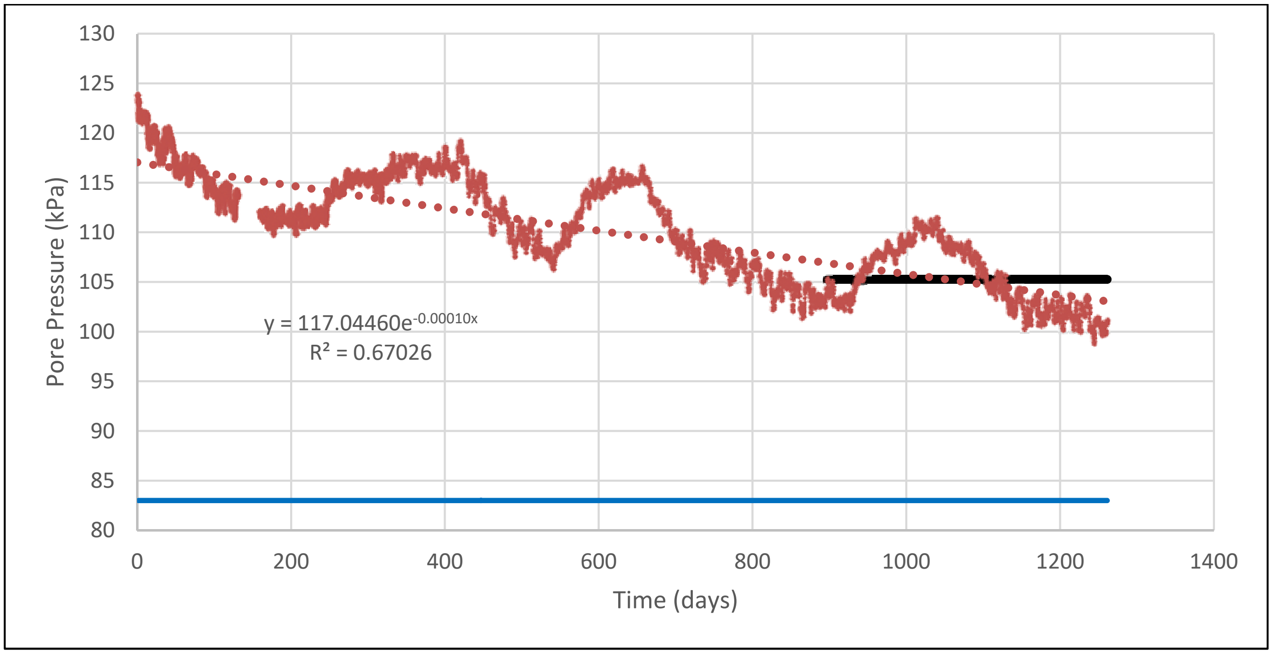

Figure 5. Field pore pressure dissipation at 10-07 after the construction period peak, showing the pre-construction baseline (blue) and the annual average (black).

Table 3. Summary of results of pre- and post-construction VWP readings and pore pressure dissipation rates based on post-construction pore-pressures.

	Elevation (m)	Pore pressure (kPa)		Dissipation time (days)	
Borehole	VWP el.	Pre-construction	Post-construction	T50	T90
10-07	480.6	12	24	365	824
	469.3	84	105	218	878
	462.3	58	45	327	863
10-08	479.0	101	144	263	504
15-10	467.5	87	74	170	800
	462.0	75	73	280	813
15-11	472.0	70	56	173	623
	467.5	26	16	299	771
	462.0	123	109	340	809

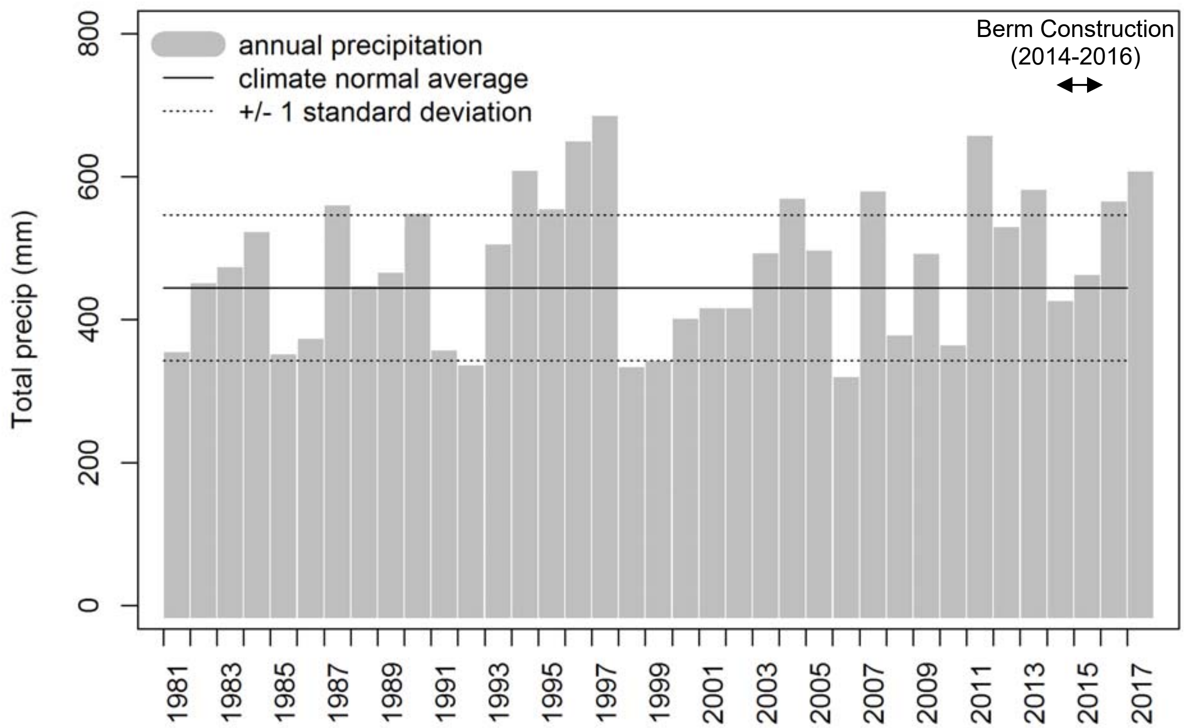

Figure 6. Annual total precipitation from 1981 through 2017 at Fort St. John Airport, with the average annual precipitation and +/- 1 standard deviation from the 1981 to 2010 climate normal period.

4 CONCLUSIONS

The results of the pore pressure dissipation readings indicate a complex response to the changes in loading and surface and sub-surface drainage, as well as potential effects of precipitation. The T_{90} measured from the data available to date for the VWPs generally agrees with the 1- to 4-year estimate obtained from the CPT dissipation tests (note there are some VWPs that may still be dissipating). However, the variability in the post-construction pore pressure baseline cannot be definitively explained and could not be predicted using the methods of analysis employed for this design. High variability in pore pressure dissipation should be expected when placing fills on colluvial slopes, especially when the parent material is overconsolidated. A view of the completed embankment is presented in Figure 7.

Figure 7. New "Military Berm" fill embankment in 2016; one-year post construction.

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