Landslide Stabilization Using Wick Drain Dewatering of Artesian Groundwater Pressures - Highway 40:30 North of Hinton, Alberta

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ABSTRACT
A slow-moving landslide affecting Highway 40:30, about 50 km northwest of Hinton, Alberta, was characterized by an arc-shaped cracking pattern crossing both lanes over a length of 150 m. Movement was measured at the base of a clay unit just above the contact with the underlying bedrock. Artesian pressures are believed to be the cause of the movement. The site was remediated using 3,511 wick drains installed to an average depth of 8.8 m through a gravel working platform to relieve the pressures and stabilize the site prior to grade-widening fill placed to accommodate detour traffic during a culvert replacement. According to the instrumentation results, there has been a noticeable decrease in the groundwater table with a commensurate reduction in movement rates. Although the crack pattern persists in the highway, the frequency of maintenance has decreased and it is expected to decrease further with time.

RÉSUMÉ
Un glissement de terrain lent, situé à une distance d'environ 50 km au nord du village de Hinton, Alberta, se caractérise par un patron de fissures curvilignes sillonnant les deux voies de l'autoroute 40:30 sur un tronçon d'environ 150 m de longueur. Le mouvement, soupçonné d'être dû à des pressions artésiennes, fut établi à la base d'une couche d'argile située au-dessus de l'interface avec le socle rocheux. Le site fut remédié par le relâchement de contraintes lors la construction d'une voie de contournement requise pour le remplacent d'un ponceau au moyen de 3,511 mèches drainantes installées à travers une plateforme de gravier à des profondeurs d'environ 8.8 m. Les données recueillies par l'instrumentation indique une baisse appréciable de la nappe phréatique et une réduction correspondante des taux de mouvement. Malgré la présence du patron de fissures dans la chaussée, la fréquence des travaux d'entretien fut réduite et devra continuer de se réduire avec le temps.

1 INTRODUCTION

This paper presents a case history about the use of wick drains and a granular drainage blanket to lower a high water table believed to be the trigger for slope movement at a highway embankment failure site. The site history, geological setting, and subsurface conditions are presented. Using the stratigraphy and instrumentation readings, analyses were undertaken to assess existing conditions and potential remediation measures. The selected remedial measure is presented along with a summary of the construction aspects and effectiveness of the solution one year after construction.

The site is located about 50 km north of Highway 16 north-west of the town of Hinton, Alberta, on Highway 40 Control Section 30 at km 48.8. The relatively low highway embankment was constructed on a gentle sidehill using a combination of cut and fill in an area with organic deposits. Highway 40 provides an important tourism and freight linkage from Highway 16 to Highway 43 and carries about 1220 vehicles per day.

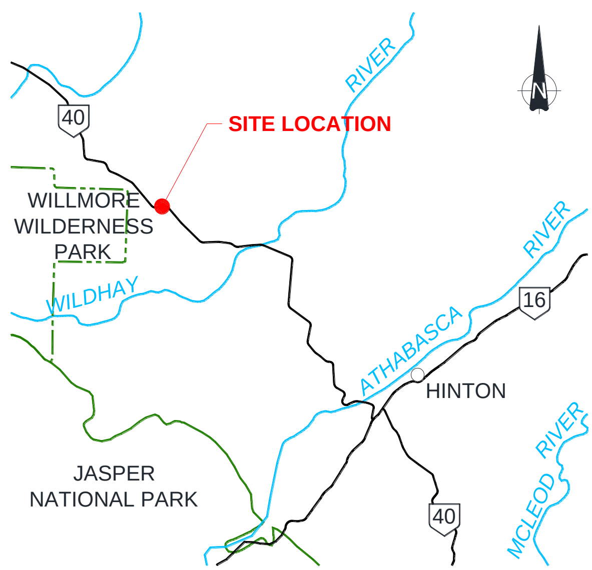

Figure 1. Key Plan

2 PROJECT SETTING

2.1 Site History

Highway 40 was constructed through the mid-1970s to late-1980s. The highway is a two-lane paved roadway, and at the site, is built up with about 3 m of fill and a 5H:1V sideslope that extends southward onto a broad plain covered with a thick organic deposit through which Fred Creek flows. Details about any issues during the original construction are not available. The site began to experience distress in the late 70's shortly after completion. Ongoing pavement distress has been managed with periodic asphalt patching since that time. The site was added to Alberta Transportation's (AT) GeoHazard Risk Management Program (GRMP) in 2007 with the designation NC48 (located with the North Central Region).

There are two culverts which were installed in 1975 at the location where Fred Creek flows under the highway about 130 m east of the NC48 site. These corrugated steel pipe (CSP) culverts consisted of a main 1,800 mm diameter pipe and an overflow 1,500 mm diameter pipe. The importance of these two larger diameter culverts near the subject site is described later.

AT's earliest reference to landslide movement at this site was in 1979 when large quantities of fill were routinely placed to maintain the shape of the roadway due to continued deformation. At about the same time, three 229 mm diameter perforated pipes were placed in the failing area and a 91 m interceptor drain was installed below the upslope ditch. A gravel toe berm was designed and may have been installed on the downslope side to buttress the slide area; however, field inspections in the late 2000's could find no evidence of this feature.

In 1980, records indicated that an additional 150 mm of vertical movement had occurred with at least two rounds of patching required to maintain the roadway surface. A geotechnical investigation was completed by AT and consisted of six test holes and in-situ vane shear testing and block sampling of the muskeg. Further assessment was done the following year which concluded the road distress was a result of a weak subgrade and fill foundation, sloping and variable bedrock depth, and high groundwater table. Several remedial options were developed at this time with the selected method to consist of stone columns at 1.5 m spacing installed through the roadway. It is not known if the recommended option was constructed. Records further indicate that cracking re-appeared in 'new' pavement in 1982.

In 2005, Thurber undertook a site reconnaissance at the request of Alberta Transportation due to continued deterioration at the site. Approximately 42 tonnes (2,000 m^3) of asphalt patch had been placed shortly before Thurber's site reconnaissance. A geotechnical investigation with instrumentation (inclinometers, SI, and pneumatic piezometers, PZ) was done in 2006. The report included development of potential remediation measures (Jacques Whitford 2007). In 2006, another thick asphalt overlay was required, and the site was formally added to the Alberta Transportation's GRMP. This initiated annual site inspections by geotechnical staff, and biannual instrumentation readings.

In response to the significant flooding in Alberta in June 2013, AT retained Thurber to evaluate several sites in the North Central Region with the potential for high-water-related impacts (High Water Mitigation Program) which included this site. Due to continued degradation and poor performance of the two culverts adjacent to the site, AT retained ARA Engineering Ltd. to manage their replacement. A detour would be required for that work and there was concern that the additional fill required for the detour that would be placed over the NC48 slide would further destabilize the landslide area. Thus, the NC48 site remediation work was included in the contract for the culvert replacement. Thurber utilized the preliminary engineering study done previously as the basis for detailed design of the jointly-selected remediation option. The slide remediation construction work was undertaken in 2016 and included the installation of additional instrumentation at the site.

2.2 Physiological and Geographic Setting

The site is located in the foothills of the Rocky Mountains. The bedrock underlying the site is of the Brazeau Formation consisting of non-marine, thickly-bedded sandstone and mudstone containing some tuffs and thin coal beds (Hamilton 1999). In general, the terrain in the area consists of clay till with an immediately adjacent area of organic terrain through which the Fred Creek flows (Bayrock 1980). Hydrogeological mapping in this area is incomplete due to the complexity of flow in the folded bedrock of the foothills; however, there are numerous flowing wells noted in the region (Barnes 1976). Although the regional flow is typically downwards toward bedrock aquifers, local flow at the site was noted to be in near-surface sediments flowing from high to low ground at Fred Creek.

2.3 Site Investigations

As discussed above, a site investigation was carried out in 2005 which consisted of four test holes in a north-south section approximately perpendicular to the highway (one upslope of the highway and the rest downslope). This investigation installed a pneumatic piezometer in each hole and slope inclinometers in the downslope locations. The slope inclinometers were anchored into the bedrock.

Thurber also installed instrumentation at three locations in 2016 for construction monitoring. One additional pneumatic piezometer was installed in the middle of the slide area (PZ5) and two were installed along the sideslopes of the highway embankment (two nested at PZ6 and a single at PZ7) where the grade-widening fill was to be placed. Locations of all of the instruments are shown on Figure 2.

2.4 Site Conditions and Observations

Thurber documented the site conditions in 2005 and annually from 2007-2011 within the GRPM, and again in 2014 under the High Water Mitigation assessment. Routine inspections after 2011 were undertaken by various regional consultants retained under the GRMP.

The site is located within a shallow valley through which Fred Creek runs. The soils in the lower portions of the valley are covered with organic muskeg-like terrain. The highway crosses this valley obliquely and the creek runs through the two culverts just east of the NC48 site. The slope on which the highway is situated is inclined at approximately 4 degrees with higher ground (about 40 m higher) located about 400 m upslope of the highway.

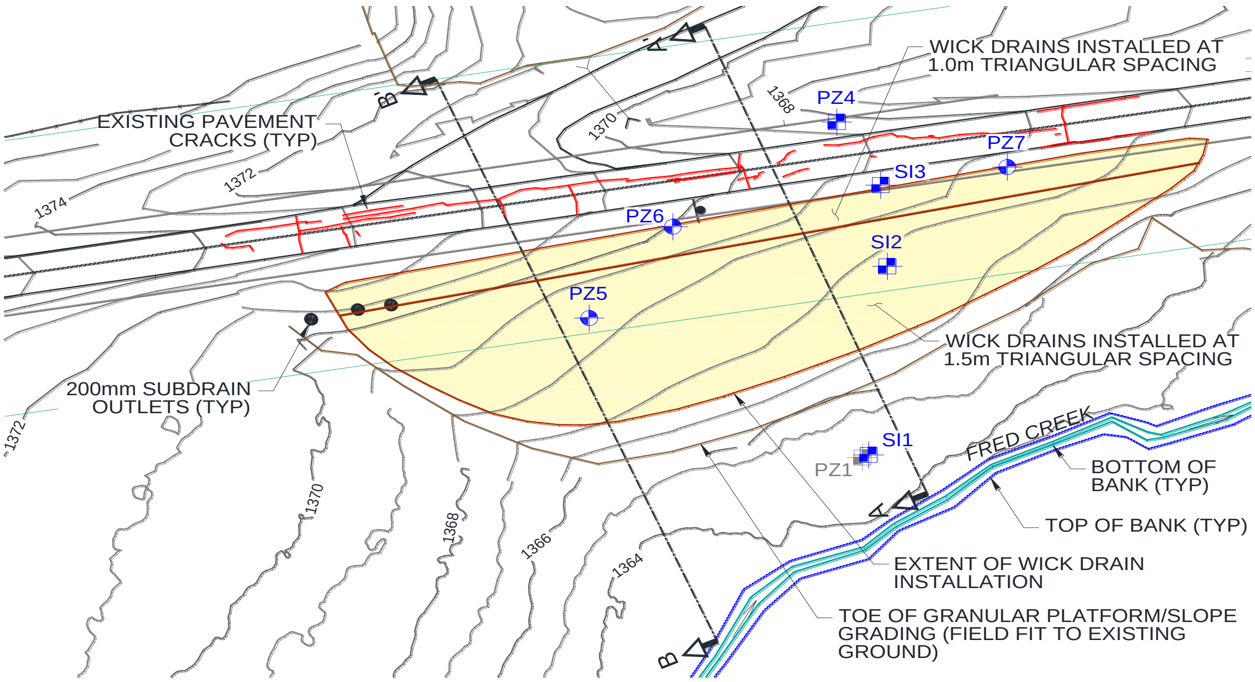

Figure 2. Site plan showing crack pattern prior to construction, cross-section location, and wick drain footprint.

In 2005, the distress was documented as a 60 m long arc-shaped crack through the paved highway with crack widths up to 40 mm. No vertical differential across the crack was observed at that time; however, the highway had recently been patched. The crack pattern appeared to be the backscarp of the re-activated 1979/1980 landslide, possibly triggered by toe erosion from the Fred Creek and/or a high water table. Although toe erosion was not mentioned in the historical documentation as a factor, it is may have been in the re-activation of movement. The terrain downslope of the highway was uneven, and a toe bulge could not be identified. By the time of the 2008 annual routine inspection, the crack length had extended to about 130 m. During the 2014 inspection for the HWMP, the crack had extended to 150 m in length.

2.5 Stratigraphy

The geotechnical investigation done in 2005 identified that the general stratigraphy consisted of clay embankment fill overlying peat. Below the 1 m to 2 m of peat, there was 4 m to 5 m of medium plastic clay over medium plastic clay shale. At one test hole (SI2), there was a 1 m layer of clay till between the clay and bedrock. The 2016 investigation for construction instrumentation identified similar stratigraphy; however, there were some differences: the depth to clay shale bedrock was less than expected, the depth and extent of fill in some areas was greater than expected, and a layer of coal was identified within the bedrock at one test hole. Laboratory testing done in 2016 determined that the clay fill and clay till were medium plastic (liquid limits of 47 and 34 percent, respectively), while the clay and clay shale were high plastic (liquid limits of 69 and 140 percent, respectively). The clay shale and coal contained bentonitic lenses.

It was observed that the upper portion of the clay shale bedrock was highly weathered and resembled lacustrine clay leading to the conclusion that the clay shale contact previously identified may be higher than thought.

2.6 Instrumentation Readings and Interpretation

The main failure surfaces were identified at about 7 m below ground surface in SI2 and SI3 and about 2 m below ground surface in SI1 indicating that the slip surface became shallower with distance from the highway and confirming that the toe was in the muskeg terrain south of the highway. As of 2013 (a seven year monitoring period), the cumulative movements in these slope inclinometers were 12 mm at SI1, 65 mm at SI2, and 71 mm at SI3. The movement rate measured at SI1 was between 0.3 m and 2.7 m depth and was higher between 2007 and 2009 and slowed after that time. This shallow movement appeared to exhibit seasonal variability (downslope movement in winter, upslope in summer) which may be due to frost movement of the upper, near-surface portion of the casing.

At SI2, the movement rate was negligible until the spring of 2011 when displacement began and continued relatively steady rate thereafter. This instrument also exhibited a seasonal pattern with minimal or negative displacement measured between the fall and spring readings (that is, over winter) while 15 mm to 20 mm was measured between spring and fall (through the summer period). At SI3, movement has been ongoing since installation in 2007 at a relatively slow rate until spring of 2011 when the movement rate increased significantly at the same time as it increased at SI2.

All of the porewater pressures measured in the pneumatic piezometers (PZ) were artesian. The tips of the piezometers are located in soft clay layers less than 0.5 m above the underling clay shale (or clay till at PZ2).

Table 1: Piezometric Readings for Design

PZ	Location	Depth (mBGS)	Max. Level (mBGS)	Fall 2013 (mBGS)
PZ1	65m S of hwy	5.5	-2.2	N/A
PZ2	20m S of hwy	7.2	-5.0	-5.0
PZ3	S side of hwy	7.2	-0.4	N/A
PZ4	N side of hwy	4.3	-0.7	-0.1

Note: mBGS is metres below ground surface where a negative values reflects piezometric readings above ground surface.

3 ASSESSMENT

3.1 Mechanism of Failure

Based on the measured movement depths, the failure plane is likely situated at the base of the medium plastic clay unit at, or just above, the contact with the underlying materials (clay shale or clay till). Although not visible in the irregular terrain, the toe of the movement is considered to be located within about 30 m of the creek. It was anticipated that the embankment, at the time of construction, was only marginally stable due to the artesian pressures. Slope movement and surface subsidence associated with periods of heavier precipitation could have caused pore pressure levels to increase further. Higher pore pressures resulted in a reduction in the effective stress within the clay layer providing less resistance to slope movement. The pneumatic piezometer readings indicated pore pressures up to 5 m above the existing ground surface, which may have been a result of artesian pressure in the area combined with localized excess pore water pressures induced by slope movements. The additional load applied at the roadway surface caused by asphalt patching and overlays over the years required to re-establish the highway grade through this area likely also had contributed to the ongoing slope movements.

Ongoing erosion of the toe of the slope by Fred Creek may also have contributed to the instability. This erosion would be exacerbated during high water events. Without a reduction in the destabilizing mechanisms affecting the site, the pore water pressure or embankment loading, it was anticipated that ongoing movement would continue indefinitely.

3.2 Repair Options

The preliminary report undertaken in 2007 (Jacques Whitford) considered several methods to stabilize this slide. These options were assessed in relation to the potential detour fill placement scheme and planned grade widening:

- Installation of sub-drains (trench drains) particularly upslope of the highway. Additional analysis determined that drains of this nature would have a limited effect on the deep pore water pressures.
- Replacement of the upper embankment with light-weight fill. While reducing the driving force, this option would not address the pore water pressure, require a detour itself, and was expected to uneconomical;
- Construction of a pile wall. This would also not address the pore water pressures and would be uneconomical;
- Installation of wick drains in the zone of sliding to reduce the deep artesian pore water pressures and that they be installed as far in advance of construction as possible to allow for dissipation of the pressures.

It was determined at that time that installation of wick drains provided the best solution to remediate this site and had the further advantage of not requiring work to occur on the highway surface. Although toe erosion by the creek is a contributing factor, armouring of the creek bank was assessed to provide only modest benefits as erosion would become less of a trigger once the extreme pore water pressures were reduced.

3.3 Stability Analyses

In order to determine soil properties, a back-analysis of the existing conditions was undertaken. Section A-A' was used for this analysis and was aligned such that it was oriented in the resolved direction of the slide movement and in the vicinity of the 2007 test holes. Initial soil and groundwater parameters were selected based on the available subsurface information and groundwater readings and previous experience on similar sites and the back-analyses used to refine the parameters to obtain a factor of safety near unity. The cracks on the highway and likely toe bulge above the creek were used to control the slip surface location. The soil parameters determined by the back-analysis are shown on Figure 3.

Two potential slip surfaces were identified as representative and used as the basis for evaluating the remedial option. The first surface (A in Figure 3) had a shorter overall failure plane which resulted in a back-calculated factor of safety (FS) of 0.97. The second surface (B in Figure 3) assumed a longer failure plane (toe closer to the creek) which encompassed the shallow movement observed at the southernmost inclinometer (SI1). The resulting FS under the same presumptive soil strengths and measured groundwater conditions was 1.03. These analyses were expected to represent the range of likely failure surfaces operative at this location.

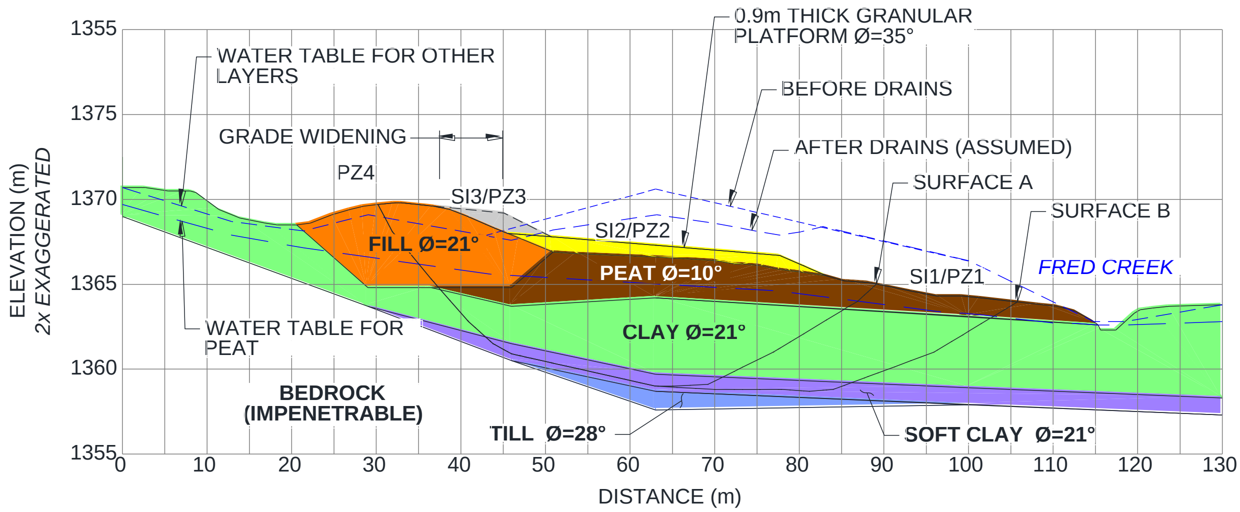

Figure 3. Generalized slope stability scenario

The remedial option evaluated was a 25 m extent of wick drains (measured perpendicular away from the highway) installed over the zone immediately downslope from the existing roadway embankment to reduce the artesian pore pressures in the clay layer where the failure surface is presumed. In addition, a 0.9 m thick working platform (required for wick installation, protection, and long term drainage) was also included in the model. The effect of the remedial measure was assessed on both of the failure surfaces identified in the back-analysis. The FS on the slip surfaces increased from 0.97 to 1.77 (82 percent) and from 1.03 to 1.51 (47 percent), respectively, for surfaces A and B. For these analyses, it was assumed that the wick drains would lower the artesian groundwater level to at least halfway between the original condition and the top of the sand blanket platform (lower water table shown in Figure 3).

The fill for future grade widening was anticipated to add approximately 3.3 m of road top width measured from the edge of the existing embankment over the slide area. The side slope was to be 4H:1V. The short-term effect of the grade widening fill was modeled using presumptive B-bar parameters of 0.6 for the existing clay fill of the embankment and 0.7 for the native clay to account for the initial increase in pore water pressures that could result with placement of the fill. The analysis assumed that the wick drains were installed far enough in advance of the detour construction to allow for dissipation of the existing artesian pressures (that is, the B-bar increase applied to the lowered water table rather than existing conditions). The resulting short-term FS were 1.39 and 1.25, respectively for A and B, which is still an improvement of 43 percent and 21 percent as compared to original conditions. After construction and the reduction of excess pore water pressures, the long-term stability was estimated to improve to FS of 1.73 and 1.50, respectively, for the two surfaces A and B considered.

4 DESIGN AND CONSTRUCTION

4.1 Wick Drain Layout and Methodology

The construction generally adhered to the design specifications. The surface area within the wick drain footprint was cleared of vegetation by cutting it flush to ground surface to avoid disturbing the peat soil structure. A woven geotextile (polypropylene with grab tensile strength of 1.4 kN) was laid across the area before placing Alberta Transportation Designation 2, Classification 25 (Des 2-25 – maximum size of 25 mm) gravel across the wick drain footprint. The initial lift of gravel was placed with a minimum thickness of 500 mm and formed the platform through which the wick drains were installed.

The wick drains were installed using a hydraulically-operated stitcher mounted on a Hyundai 350 LC-9 Excavator operated by Hayward-Baker. The wick drain material was Mebra Drain MD-7407. The wick drains were installed at a 1.5 m triangular spacing except for an approximately 7.2 m-wide zone adjacent to the highway embankment where the spacing was decreased to 1.0 m in anticipation of higher pore pressures generated by additional embankment fill. The wick drains were installed through the granular drainage blanket layer into the underlying muskeg and clay and were anchored using steel plates that were threaded onto the end of the wick drain material before pushing the mandrel into the ground. The wick drains had a typical stickup of 0.5 m above the granular drainage layer which were connected with staples to horizontally-laid strip drains (same product as the wick drains). The gravel pad followed the general terrain surface and sloped downward to the south.

The gravel platform was installed in late April 2016 followed by installation and initialization of four pneumatic piezometers. After a short shutdown due to snow, a total of 3,511 wick drains were installed over five days in May

2016. Up to three attempts were specified in the case of premature shallow refusal and a total of 53 wick drains required more than one attempt. The wicks were intended to penetrate approximately 11 m into the clay shale bedrock; however, the average depth of penetration was 8.8 m. Only 178 wick drains (five percent) met the design penetration depth which indicated an irregular and higher elevation of the clay shale contact than assumed. Adjacent to the highway, there were several wick drains that failed to achieve penetration due to the presence of cobbles and boulders placed sometime in the past for erosion protection that had become partly covered with later fills. The total length of wick drains installed was 32,549 m. The total strip drain length was 3,920 m.

Once the strip drains were installed, the entire pad surface was covered with 200 mm of Des. 2-20 gravel (maximum size of 20 mm). This was thinner than the specified cover of 300 mm to reduce the total quantity of granular material required as the initial 500 mm lift required more material than estimated. The horizontal strip drains were extended approximately 2 m on to the 4H:1V face of the initial lift and covered with Des. 6-125 gravel (maximum size of 125 mm) to provide erosion protection from overland flow. The granular platform was essentially complete on May 19, 2016, at which time the site was shut down due to wet weather.

4.2 Fill Placement

Work resumed in July 2016 and a set of instrumentation readings was taken prior to start-up and indicated that pore water pressures had decreased enough to proceed with additional fill placement. The contractor was authorized to start placing grade-widening fill to accommodate the culvert detour road but was limited to 1 m vertical per week pending the ongoing piezometric response. Thurber undertook intermittent readings during fill placement to assess the B-bar response to the increasing levels of fill as the detour was constructed. The fill was completed in mid-October 2016 and a compost blanket was then placed over the gravel pad to assist in the revegetation of the site.

4.3 Instrumentation Results

Baseline readings were taken at the existing piezometers prior to start of the granular fill placement. Additional readings were taken after PZ5 through PZ7 were installed in late April 2016. Routine piezometer and inclinometer readings were taken during wick drain installation and completion of the upper lift of the granular pad and resumed again during placement of the detour fill. Notable readings during this period are summarized in Table 2 for the piezometers and Table 3 for the slope inclinometers. A rise in the piezometric surface was observed at PZ2 after the gravel pad was placed which corresponded with a significant increase in the rate of movement at all of the slope inclinometers, particularly as compared with the long-term movement rates (as reported under the April 22, 2016, column in Table 3). The increased rate of movement at SI1, located outside of the gravel pad, confirms that the toe of the slide likely extends further south beyond SI1. There was only a modest rise in piezometric surface at PZ3 (0.2 m) so the increase in the rate of movement is assumed to be measuring the overall slide mass movement triggered by the fill placed between SI1 and SI3. The peak incremental rate of movement (as measured relative to the prior reading) was 1897 mm/year at SI2 which was a cause for concern (determined from 42 mm of cumulative movement in only eight days).

The piezometers south of the highway (PZ2 and PZ5) had an expected limited response to the fill placement. As discussed earlier, the response measured in the piezometers below the highway embankment where the fill was being placed (PZ6A, PZ6B, and PZ7), did show up to 0.5 m of increase in the piezometric surface but, as discussed earlier, the B-Bar response was less than 0.2, which was deemed acceptable.

Table 2: Piezometric Readings During and Post-Construction

PZ	Location	Tip Depth (mBGS)	Before Gravel Pad (mBGS)[1]	Peak During Wick Drain Installation (mBGS)	After Wick Drains (mBGS)[2]	Before Detour Fill (July 5, 2016) (mBGS)[2]	Peak During Detour Fill (Fall 2016) (mBGS)[2]	Fall 2018 (mBGS)[2]
PZ1	65m S of highway	5.5	-2.2	N/A	N/A	N/A	N/A	-2.3
PZ2	20m S of highway	7.2	-4.1	-5.3	-3.3	-1.9	-0.9	-0.5
PZ4	N side of highway	4.3	0.1	-0.1	0.3	0.7	0.0	0.5
PZ5	30m S of highway	5.2	N/A	-2.1	-2.0	-1.6	-1.8	-0.5
PZ6A	S side of highway	5.2	N/A	0.3	0.7	0.9	0.5	0.5
PZ6B	S side of highway	7.6	N/A	-2.9	0.8	0.7	0.6	0.8
PZ7	S side of highway	6.1	N/A	-0.3	-0.2	0.4	-0.2	0.3

mBGS is metres below ground surface where a negative value reflects piezometric readings above ground surface

1 – Readings taken May 5, 2016 except for the damaged PZ1 for which the most-recent reading from May 2013 is reported. This pneumatic piezometer was repaired in Fall 2016.

2 – Readings for PZ2 after installation of the wick drains are not reliable due to the displacement that occurred at that location which may be impeding nitrogen flow in the pneumatic tubing.

Table 3: Slope Inclinometer Readings During Construction

SI	Movement Zone (mBGS)	Pre-Construction (April 22, 2016)[1]		Shortly After Gravel Pad (May 10, 2016)		End of Construction (Sept/Oct 2016)	
		Cumul. (mm)	Long-Term Rate (mm/yr)	Cumul. (mm)	Inc. Rate (mm/yr)	Cumul. (mm)	Inc. Rate (mm/yr)
SI1	4.6 m – 7m	1.6	0.5	9.2	162.0	18.6	0.3
SI2	6.4 m – 10.1 m	64	6.7	108.7	112.5	188.4	24.0
SI3	6.4 m -9.4 m	68	7.1	68.0	24.7	66.0	0

mBGS is metres below ground surface; “Inc. Rate” refers to the incremental movement rate determined since the previous reading.
1 – Reading taken from May 1, 2016, for SI1, which was repaired just after the gravel pad was installed.

Four sets of readings were taken during the detour fill placement and the maximum calculated B-Bar did not exceed 0.14. There was almost no response in the piezometers that were located within the footprint of this detour fill.

Toward the end of the construction period, SI2 became difficult to read due to the continuing deflection of the casing and was abandoned. The final reading was taken on September 8, 2016, and the readings at PZ2 since that time are suspect as there were signs that the pneumatic tubing may be pinched.

It was a positive indication that the rates of movement measured in the slope inclinometers had slowed significantly with SI1 and SI3 incremental rates below the long-term pre-construction average. The incremental rate at SI2 had decreased from the peak rate during construction but remained above the long-term average when it was discontinued. Considering the Fall 2018 piezometer readings, all are below the pre-construction levels with the exception of PZ1 (beyond the toe of the treatment zone) which had remained similar to pre-construction conditions. This may be due to ongoing seepage observed at and beyond the pad that is outside the influence of the wick drains (that is, the piezometric surface “rebounds” to above ground surface outside of the pad). Better than anticipated, the water level at PZ2 (in the middle of the treatment zone) had dropped 3.6 m which is a significantly better result than the drop of 2 m conservatively assumed during the design phase. A similar reduction was observed at PZ5 which decreased from 2.1 m to 0.5 m above ground surface (difference of 1.6 m). The shallower PZ6A appears to have been relatively indifferent to the treatment though the deeper PZ6B water level decreased 3.7 m.

5 DISCUSSION AND CONCLUSIONS

5.1 Costs

The approximate costs of this mitigation work at the NC48 site were $465,000 for the clearing, granular pad, and compost cover and $139,000 for the wick and strip drains for a total of $604,000.

5.2 Performance

In general, the implementation of the repair option followed the design and appears to have functioned as intended. The most-recent visual inspection of the site was in May 2018 and documented under the GeoHazard assessment program (Stantec 2018a). At that time, it appeared that the highway had been overlaid in the past year; however, cracks were visible in the westbound lane but no worse than in 2017. Nonetheless, there has been sufficient movement for the cracks to reflect through the recent patch. The Authors’ experience with this type of repair (lowering artesian pressures through various dewatering schemes) is that the full effect of the repair does require some time, perhaps several years, before the landslide movement is arrested (or brought to an asymptotic low movement rate). Minor erosion was noted in some locations and several areas of seepage were observed within the footprint of the granular pad or at the toe. No signs of instability were noted in the granular berm.

Readings of the instrumentation since Fall 2016 have been done by the current GRMP consultant (Stantec 2018b). Note that due to a change in the type of inclinometer probe (metric probe instead of imperial), the measurements use Fall 2016 as the baseline. In the fall of 2018, only SI1 and SI3 were read as SI2 sheared off during construction. At that time, two movement zones were identified in SI1 from 0.3 m to 2.8 m and from 4.3 m to 6.3 m but with rates less than 1 mm/year in both zones which is less than the historical maximum rates of 19 mm/year and 2 mm/year, respectively. The lower zone corresponds to the one monitored by Thurber and the cumulative movement since 2016 is only 2 mm. There were also two movement zones identified in SI3 from 0.8 m to 2.8 m and from 5.8 mm to 8.2 m with negative movement rates (this would be upslope so physically unlikely and reported as no movement). The negative movement means that no conclusion can be drawn as there is likely incompatibility between the readings taken with the metric probe (current) and the imperial probe used by Thurber prior to and during construction. This will be assessed in the summer of 2019.

The change in the piezometer surfaces was assessed by comparing the Fall 2016 piezometer readings (shortly after construction was completed) with those from Fall 2018 (shown in Table 2). Starting furthest away from the highway, PZ1, which is located beyond the wick drain treatment area, has increased about 0.4 m and is above ground surface. In the central portion of the treatment area, PZ2 is stable and remains slightly above ground surface while PZ5 has dropped 1.2 m and is still slightly above ground surface. Along the south embankment shoulder, PZ6A dropped 0.3 m while PZ6B increased 0.2 m and PZ7 also increased 0.3 m (and remains just below ground surface). In the upslope ditch, PZ4 was unchanged.

5.3 Conclusions

The wick drain treatment has been effective in lowering the groundwater table at this site. The movement rates as measured in the instrumentation and observed visually, have slowed which confirms the original assessment that the high groundwater table was at likely mostly responsible for the instability observed at this site. It is anticipated that the movement rates will continue to subside over the long term as the wick drains continue to depress the groundwater table.

Most components of the construction went as anticipated. Minor changes were required to the design as construction progressed. If given the opportunity to undertake this work again, more care and attention would have been given to placement of the initial lift of the granular platform. This material was placed too quickly and thickly and likely resulted in the premature shearing of the instrumentation at SI2 (inclinometer and piezometer). As shown in Table 3, there was significant deflection measured after the placement of the granular platform which continued, albeit at a slower rate, afterward. Better data during detour fill placement (obtaining corresponding fill elevations with the piezometer readings) would have allowed better understanding of the pore water pressure response; however, at this particular site, the response was muted so there were no concerns with instability.

6 ACKNOWLEDGEMENTS

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