# Slope Stabilization at km 201.4 of the Alaska Highway: Site Challenges and Lessons Learned

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ABSTRACT
The Alaska Highway (BC No. 97) traverses numerous areas of difficult, landslide-prone terrain across northern British Columbia and the Yukon. Public Works and Government Services Canada retained Tetra Tech Canada Inc. to evaluate an unstable 100 m long section of highway embankment located at km 201.4 near the community of Pink Mountain, BC. Geotechnical explorations completed in late 2015 revealed that the embankment movements were occurring along a weak sliding plane located at about 9 m depth in glaciolacustrine clay, with movement velocities of about 1 to 2 mm per month. However, following several days of rainy weather in late August 2016, the movements accelerated to a peak velocity of up to 75 mm per day. In response to these events, a portion of the embankment was removed and other emergency measures were implemented to reduce the risks to road users. Permanent slope stabilization works completed in early 2017 included the construction of a rockfill toe buttress and drainage improvements, followed later in the year by restoration of the highway embankment. This paper discusses the site conditions, inferred failure mechanism and contributing factors, efficacy of the emergency response measures, design of the rockfill buttress, construction challenges and other lessons learned.

RÉSUMÉ
La route de l'Alaska (C.-B. 97) traverse de nombreuses zones de terrains difficiles et sujettes aux glissements de terrain dans le nord de la Colombie-Britannique et au Yukon. Travaux publics et Services gouvernementaux Canada a retenu les services de Tetra Tech Canada Inc. pour évaluer une section de remblai routier instable de 100 m située au km 201,4 près de la communauté de Pink Mountain, en Colombie-Britannique. Les explorations géotechniques achevées fin 2015 ont révélé que les mouvements de remblai se déroulaient le long d'un plan de glissement faible situé à environ 9 m de profondeur dans l'argile glaciolacustre, avec des vitesses de déplacement d'environ 1 à 2 mm par mois. Cependant, après plusieurs jours de pluie à la fin d'août 2016, les mouvements ont accéléré jusqu'à atteindre une vitesse maximale de 75 mm par jour. En réponse à ces événements, une partie du remblai a été enlevée et d'autres mesures d'urgence ont été mises en place pour réduire les risques pour les usagers de la route. Les travaux de stabilisation permanente des talus achevés au début de 2017 comprenaient la construction d'une butte d'enrochement en enrochement et des améliorations au drainage, suivies plus tard dans l'année par la restauration du remblai de l'autoroute. Cet article traite des conditions du site, du mécanisme de défaillance déduit et des facteurs contributifs, de l'efficacité des mesures d'intervention d'urgence, de la conception du renforcement en enrochement, des défis de construction et d'autres leçons apprises.

## 1 INTRODUCTION

This paper presents a case history of recent slope stabilization works completed at km 201.4 of the Alaska Highway (BC No. 97) in northern British Columbia (BC).

The Alaska Highway was originally constructed during World War II to connect Alaska with the continental United States through Canada. The highway, completed in 1943 at a total length of approximately 2,450 km, traverses numerous areas of difficult, landslide-prone terrain across northern BC and the Yukon. In BC the highway is maintained by Public Works and Government Services Canada (PWGSC) between km 133 near Fort St. John and km 968 at the BC / Yukon Border. South of km 133 the highway is maintained by the BC Ministry of Transportation and Infrastructure.

The project site at km 201.4 is located near the community of Pink Mountain, about 130 km north of Fort St John, BC (Figure 1). The highway alignment through the km 201.4 area follows the contour of a gently rolling upland which gives way to a broad forested lowland downslope of the highway (Figure 2). This lowland is incised by several small creeks and includes areas of boggy/swampy terrain.

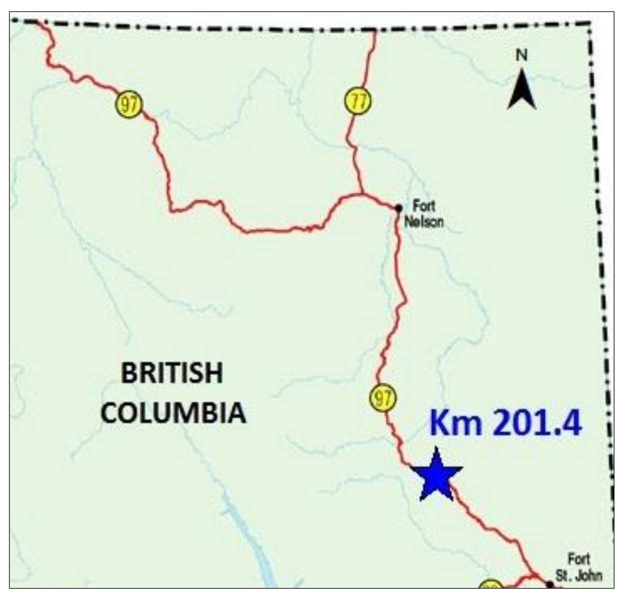


Figure 1. Site Location Map

### 1.1 Background

This section of highway was reconstructed in the early 1980s, which involved realigning the highway onto a 12 to 15 m high embankment fill located upslope (south) of the original 1943 alignment. Further east, at about km 201.1, the embankment fill transitions to an approximately 1 km long through-cut section in sandstone bedrock (Figure 3).

At km 201.4, an approximately 100 m long section of the up-chainage (northbound) lane and adjacent shoulder had experienced cracking and slumping dating back to the early 2000s or possibly earlier. The extent of cracking appeared to be getting worse in recent years, requiring frequent repairs (patching and crack sealing) to maintain an acceptable driving surface. Investigation of the cause of the slope movements, and stabilization of the slope, were identified as a priority by PWGSC due to concerns about corridor reliability and future maintenance costs.

Figure 2. View of km 201.4 slide area in mid-September 2016, looking up-chainage (west) towards Pink Mountain.

Figure 3. View of km 201.4 slide area in early September 2016, looking down-chainage (east) towards Fort St John. The parked vehicle is located at about the transition point to the bedrock through-cut section further to the east.

## 2 SITE DESCRIPTION

### 2.1 Topography and Climate

The km 201.4 site is located at approximately 950 m above sea level and is contained within the Boreal White and Black Spruce biogeoclimatic ecosystem classification zone. Environment Canada weather records from the nearest station at Sikanni Chief (1991 to 2006) indicate that mean annual precipitation is about 565 mm, of which about 180 mm is snowfall. Average daily temperatures range from -15°C in the winter to 15°C in the summer.

### 2.2 Surficial Geology

Based on the nearest available surficial geology mapping (Bednarski 2000), the soils at km 201.4 are inferred to consist of glaciolacustrine deposits (described as "*fine sand, silt and clay deposited in glacier-dammed lakes; > 1m thick; often overlain by organic deposits in lowlands*"), overlying glacial till blankets (described as "*non-sorted debris deposited directly by glacial ice; matrix is sandy to clayey and contains striated clasts of various lithologies; > 1 m thick*"), overlying sedimentary bedrock.

## 3 GEOTECHNICAL SITE EXPLORATION

### 3.1 Drilling and Laboratory Testing

Drilling was carried out in March 2015 to define the soil stratigraphy, groundwater conditions and the depth of the failure plane. A total of four air-rotary (ODEX) testholes were completed using a track-mounted drill rig at the locations shown on Figure 4. Selected soil samples obtained from the drill cuttings, split-spoons and Shelby tubes were submitted for laboratory index testing.

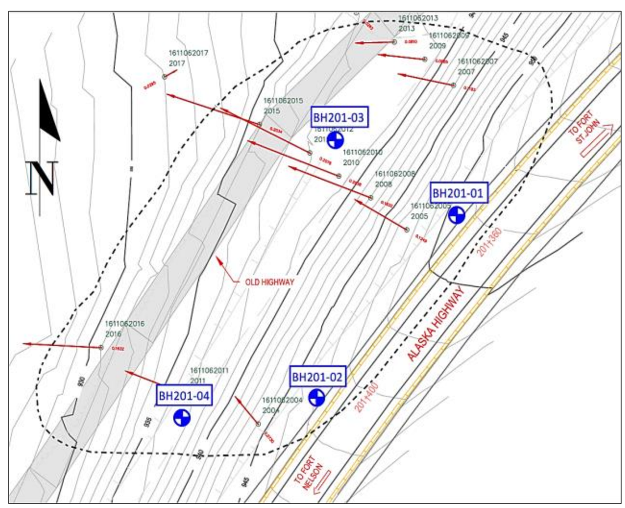


Figure 4. Survey plan of the km 201.4 area showing the testhole locations, topographic contours (1 m interval), and vector arrows of slide movement (100x magnification) established from repeated surveys of rebar pins installed on the slope. The black dashed line denotes the approximate extent of the slide area.

3.2 Soil Stratigraphy

The results of the site exploration were consistent with the soil conditions anticipated from the surficial geology mapping. The interpreted soil stratigraphy below the highway surface was:

- Unit 1: 2 to 6 m of embankment fill, comprised of compact to dense sand and gravel with some silt.
- Unit 2: 6 to 10 m of firm to stiff, medium plastic silty clay / clayey silt with trace to some sand and gravel (inferred glaciolacustrine clay). However, based on observations during drilling, and the elevation range of this unit relative to the surrounding topography, at least a portion of this unit was likely embankment fill.
- Unit 3: 3 to 5 m of stiff to very stiff, medium to high plastic silty clay / clayey silt with trace wood and organic fragments (inferred glaciolacustrine clay).
- Unit 4: 6+ m of very stiff to hard, medium plastic silty clay / clayey silt with trace to some sand and gravel (inferred glacial till-like deposits).

3.3 Instrumentation

70 mm diameter slope inclinometer (SI) casing was installed in the two testholes drilled along the top of the slope (BH201-01 and BH201-02). Both SIs were extended into stable ground at a depth of 20 m below surface.

The results obtained from BH201-01 are presented on Figure 5. Slope movements were occurring along a well-defined failure plane located within Unit 2 between about 7 and 9 m below ground surface. A total of approximately 15 mm of movement was recorded between March and November 2015 (movement rate of about 2 mm/month).

Vibrating-wire piezometers were installed in BH201-03 and BH201-04 within Units 2 and 3, respectively. A pore pressure ratio (Ru) of 0.04 was observed within Unit 2. No pore pressure was observed within Unit 3.

3.4 Back-Analysis of Slope Stability

2-D limit equilibrium slope stability analyses were performed on the slope using commercial software to back-calculate soil strengths in Unit 2 based on the depth and extent of the sliding mass indicated from the inclinometer data and survey data. Soil strengths for the remaining units were estimated based on the testhole data and previous experience in similar materials. The following shear strengths were derived from the back-analyses, based on an assigned Factor of Safety (FoS) of unity:

Table 1. Soil Properties used in Back-Analyses

| Soil Unit [1] | Drained Shear Strength | | Unit Weight ($kN/m^3$) | Ru [3] |
|---|---|---|---|---|
| | c' (kPa) | Ø' (deg.) | | |
| 1 (Fill) | - | 38 | 20 | - |
| 2 (GL Clay) | - | 25 (16) [2] | 19 | 0.04 |
| 3 (GL Clay) | - | 27 | 20 | - |
| 4 (Till-Like) | 5 | 35 | 21 | - |

[1] GL = glaciolacustrine

[2] value in parentheses is the friction angle on the failure plane

[3] Ru = Ratio of the pore water pressure to the overburden pressure.

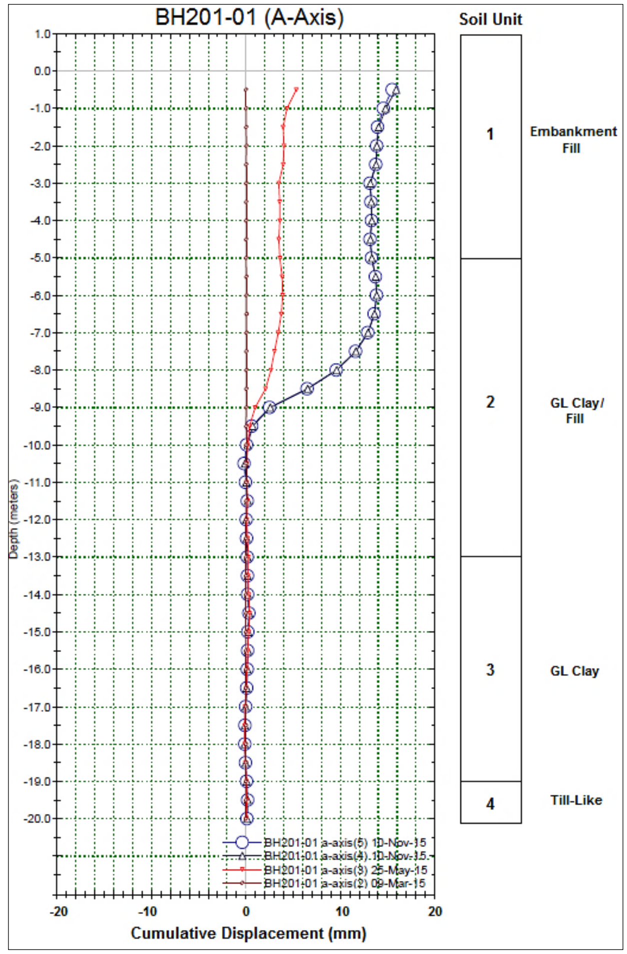


Figure 5. Recorded movements (A-Axis) from the SI casing installed in BH201-01, located at the top of the slope.

A typical slope stability back-analysis is shown in Figure 6, with various colours representing the inferred soil stratigraphy and vertical hatching of the critical failure surface.

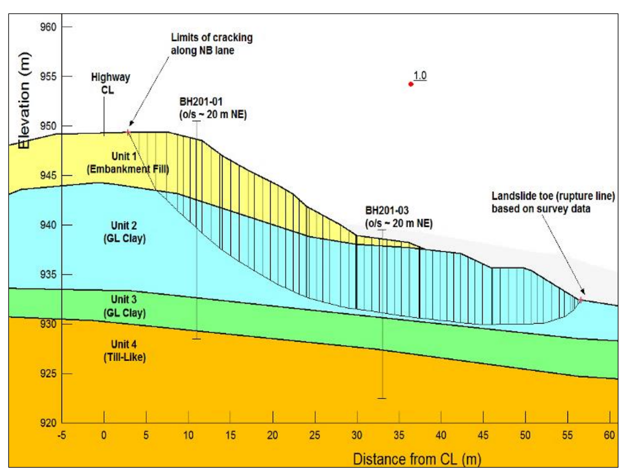


Figure 6. Typical back-analysis of the km 201.4 slide area.

3.5 Inferred Failure Mechanism

Based on the site history, and the results of the exploration, the original cause of the embankment failure at km 201.4 was attributed to the increased loads imparted on Unit 2 by the weight of the embankment fill, coupled with progressive softening of this material over time from repeated drying and wetting of the embankment. Movement of the slope in recent years was also likely exacerbated by above-average annual precipitation in this area of the province between 2010 and 2015, as well as by increasing volumes of heavy truck traffic on the highway.

# 4 SLIDE BEHAVIOUR UNDER EXTREME LOADING

## 4.1 M4.6 Earthquake – August 2015

In recent years, this section of the Alaska Highway has experienced a significant number of small to moderate earthquakes induced by natural gas extraction in the region (BCOGC 2014; Atkinson et al. 2015). On the afternoon of August 17, 2015, a M4.6 earthquake occurred at a gas well site located only 3 km north of the km 201.4 slide area, which to date is one of the largest induced earthquakes ever recorded in Canada. Shaking from this event was felt as far away as Charlie Lake, located approximately 100 km to the southeast.

The ground motions experienced at km 201.4 during this event are unknown, but were likely equivalent to a Modified Mercalli Intensity (MMI) class V to VII event with a Peak Ground Acceleration (PGA) between 0.1 and 0.6 g (G. Atkinson, pers.comm.). Curiously, however, no obvious evidence of cracking or slope movement was found at the site following this event. As can be seen from Figure 5, the rate of slope movement (~ 2 mm/month) between March and May 2015 (red line) is nearly identical to the rate of movement between May and November 2015 (blue line), the latter of which spans the time frame in which the earthquake occurred.

## 4.2 Heavy Rainfall Event – August 2016

The km 201.4 area experienced a heavy rainfall event in late August 2016. According to Environment Canada's climate records from the nearest weather station at Fort St John airport, a total of 101 mm of rain fell between August 27 and September 1, corresponding to a 20-year storm event. The abnormal weather patterns continued into mid-November 2016, with snowfall accumulations and daily temperatures both well above the long-term seasonal averages. A brief chronology of the slide behavior over this time is summarized below.

- September 1: Existing tension cracks along the highway shoulder appeared dilated with possible settlement of the road surface within the slide limits.
- September 15: Cracks continuing to dilate; road surface settled by about 300 mm within the slide limits; pavement shows visible buckling and distress (Figure 2). The slope inclinometer casings and piezometers installed in 2015 were sheared off.
- September 28: Maintenance crews were on site to re-level the concrete road barriers and patch the affected section of the northbound lane.
- October 10: Cracks had re-emerged within the recently patched area, with the extent of the slide area appearing to migrate (expand) to the west. The affected area of roadway was settling by about 10 to 15 mm per day based on visual estimation.
- October 31: Extensive cracking and distress within the recently patched area; affected section of roadway has settled by a total of about 400 mm over the month of October, bringing the total settlement to date (since the August rainfall event) to about 600 mm. Maintenance and traffic control crews were on site to direct traffic around the slide area.
- November 1: An array of 29 survey monitoring pins was installed on the slope and into the treeline north of the highway to monitor conditions in greater detail.
- November 5: The survey data revealed that the slide area extended into the treeline and was moving at a rate of up to 75 mm per day, with the largest movements located at about km 201.36 (Figure 4).

Figure 7. View of km 201.4 slide area in late October 2016, looking west towards Pink Mountain.

# 5 SLIDE STABILIZATION

## 5.1 Emergency Unloading

Based on the observed increase in the rate of slope movement indicated by the survey data, and to facilitate safe access to the site for construction of the permanent stabilization works, the upper portion of the slide mass was unloaded (excavated) to reduce the driving forces on the slope. A total of approximately 4,500 $m^3$ of embankment fill (equivalent to an 80 m long by 7 m wide by 8 m high cut) was removed from the slope starting on November 5; see Figure 8. The depth and extent of embankment removal was designed to achieve a minimum FoS of 1.1 as determined from limit equilibrium slope stability analysis. As can be seen from Figure 9, the unloading work proved to be very effective, with movement of the slope essentially stopped by November 6.

Figure 8. View of emergency unloading work (embankment removal) in early November 2016, looking west towards Pink Mountain.

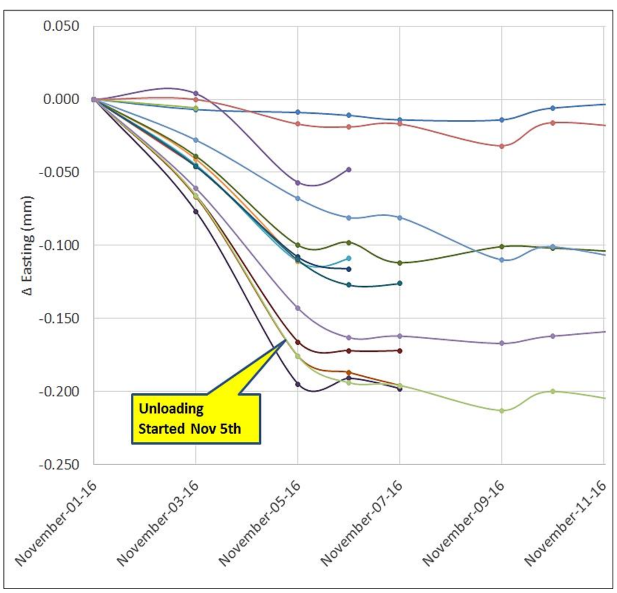


Figure 9. Slope movement survey data (Δ Easting), November 1 to 11, 2016. Y-axis scale = +50 to -250 mm.

During the unloading work, perched water was encountered within the embankment fill at the contact with the road base gravels, with visible seepage discharge out of the excavation face. This confirmed the obvious conclusion that the accelerated slope movements over the fall of 2016 were the result of significant water inflow into the slide mass through the tension cracks on the highway, thus reducing the effective stress on the failure plane. However, on further review of the site topography, highway grading and testhole data, it was observed that a portion of this water may have infiltrated into the embankment fill via the roadside ditches along the through-cut section to the east, then flowed downhill, along the highway alignment, towards the slide area.

## 5.2 Permanent Stabilization

### 5.2.1 Options Assessment

A variety of conceptual options were evaluated to stabilize the slope, including highway realignment, drainage improvements, toe buttressing, pile walls, soil nailing, deep soil mixing and lightweight fill. Each option was assessed in terms of the approximate construction cost, improvement in slope stability, long-term maintenance requirements and potential construction risks. Based on a careful weighing of these factors, the preferred solution was to stabilize the slope by constructing a rockfill buttress at the toe of the slope combined with various drainage improvements. Construction of the buttress was to be followed by replacement of the excavated embankment fill to re-establish the northbound lane.

### 5.2.2 Stabilization Design

The rockfill toe buttress was designed to achieve a minimum FoS of 1.5 for global stability under long-term (drained) loading. Limit-equilibrium analyses were performed using commercial software to analyze both the global stability of the slope and the “local” stability of the rockfill buttress under both short-term (undrained) and long-term (drained) loading, using an estimated friction angle of 42 degrees for the rockfill material. However, as weak sandstone was found to be the only potential source of rockfill along this area of the Alaska Highway, additional analyses were completed using a friction angle of 36 degrees to account for potential construction disturbance (crushing) and long-term degradation (spalling/weathering) of the rock particles. It was found that reducing the friction angle to 36 degrees only had a minor effect on the results.

The location and typical extent of the proposed buttress can be seen on the example slope stability analysis output in Figure 10. The buttress measures approximately 90 m in length, 30 to 40 m in width, and 2 to 4 m in thickness. From the analysis results, it was found that the buttress needed to be relatively wide to force the failure plane (and other potential slip-circles) to pass through the buttress rockfill, rather than daylighting upslope of the buttress or passing underneath the buttress.

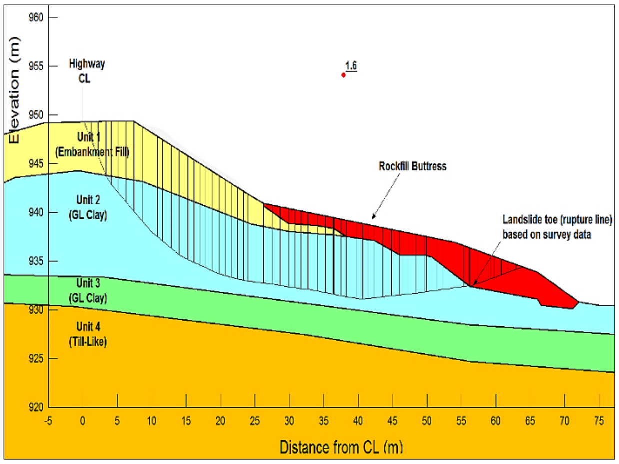


Figure 10. Typical global stability analysis of the proposed rockfill buttress at km 201.4.

To manage groundwater inflows, improve filter compatibility, and enhance the frictional resistance at the interface between the native clay soils and rockfill material, the design incorporated a 300 mm thick drainage blanket of crushed base gravel at the base of the buttress. A series of “finger” drains, consisting of 1 to 2 m deep trenches backfilled with perforated steel drainage pipe surrounded by crushed base gravel, were also included in the design to convey the accumulated water away from the slope.

## 6 CONSTRUCTION

Construction of the permanent stabilization works was awarded to Tangle Ridge Custom Crushing Ltd. who completed the work over the month of February 2017. The constructed works included approximately 9,100 m$^3$ of rockfill buttress, 1,400 m$^3$ of drainage blanket and 145 m of finger drains, with construction costs totaling approximately $740,000. The specified rockfill material was a 600 mm (24”) minus, well-graded angular aggregate with a maximum of 5% fines, which was placed in lifts and compacted by several passes of heavy tracked equipment.

The work was completed without any major issues, although there were occasional challenges with cold weather, equipment breakdowns, and difficulties achieving the specified rockfill gradation. Daily surveying of the highway embankment above the work area showed cumulative movements of the slide of less than 15 mm over the duration of construction (one month), which was within tolerable limits.

A second phase of work was successfully completed over the summer of 2017 to replace the embankment fill removed for emergency unloading of the slope in November 2016, and other minor drainage improvements.

### 6.1 Post-Construction Performance

The northbound lane of the highway was re-paved through the km 201.4 area in late September 2017 following completion of the embankment repair work. The rockfill buttress appears to be performing satisfactorily to date, with no obvious movement or distress to the highway.


Figure 11. View of rockfill buttress construction in late February 2017, looking west towards Pink Mountain. Note tracked equipment in the middle of the photo for scale.

Figure 12. View of the completed rockfill buttress in early August 2017, looking east towards Fort St John.

## 7 CONCLUSIONS AND LESSONS LEARNED

The key conclusions and lessons learned from the case history presented in this paper are:

1. Existing landslides, particularly those seated in weaker soils with low residual shear strengths, are sensitive to slight changes in overburden stress and pore water pressure. As the events from this case history show, a single rainstorm event may be sufficient to cause a sudden acceleration of an existing slide. Similarly, the removal of a portion of the embankment fill was found to be a rapid and effective means to arrest further slope movement.
2. Slide events such as that experienced at km 201.4 may become more frequent and severe in the future in response to climate change. According to the latest climate change models published by the Pacific Climate Impacts Consortium, northern BC is projected to experience an approximate 5% increase in annual precipitation by the 2020s (https://www.pacificclimate.org/analysis-

tools/plan2adapt). Climate change effects are also anticipated to result in an increased frequency and intensity of heavy rainfall and storm events (https://www2.gov.bc.ca/gov/content/environment/climate-change/adaptation/impacts).

3. Induced seismicity hazards have received a lot of attention in recent years, both in Canada and the USA, with the development of unconventional oil & gas extraction technologies such as hydraulic fracturing. Previous studies by others (e.g. Atkinson et al. 2015) have found that moderate induced earthquakes may be damaging to infrastructure located near the epicenter; however, as the observations from km 201.4 show, such events may not have a significant effect on slope stability.
4. For highways projects, and other linear infrastructure, it is important to consider potential 3-dimensional drainage paths where slide areas have developed near transitions from cut sections to fill sections.
5. Repeated ground surveys of an array of rebar pins installed across a slide mass are a simple, effective, and economical way to monitor the rate of slope movement, delineate the slide extents, and understand the movement mechanism and mode of failure. The authors have found that this approach tends to be more successful than sophisticated remote sensing techniques, particularly if the slide area is heavily vegetated.
6. Constructability is sometimes overlooked during the design process (Burland et al. 2012), but this is a key consideration for remote northern projects where high-quality aggregates and other typical construction materials such as concrete may not be readily available. In addition, the construction work is often undertaken by local contractors, who may be unfamiliar with more sophisticated construction methods and equipment. Designs that can be easily "field fitted" to suit the conditions encountered during the work are generally preferable.
7. Material durability during both initial placement and over long-term conditions is an important design consideration for projects where aggregates are sourced from sandstone or other weak rock types.

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