GEOQuébec 2015
Challenges from North to South
Des défis du Nord au Sud

Slope Stabilization at km 229 and km 701-703 of the Alaska Highway: Site Challenges and Lessons Learned

Jason Pellett, P.Eng./P.Geo. & Andrew Horwood, AScT
Tetra Tech EBA, Vancouver, British Columbia, Canada

ABSTRACT
The Alaska Highway (BC No. 97) traverses numerous areas of difficult, landslide-prone terrain across northern BC and the Yukon. Public Works and Government Services Canada retained Tetra Tech EBA Inc. to evaluate unstable slopes at km 229 and km 701-703 and to complete the design of stabilization works. At km 229, slope movements were occurring along a very weak sliding plane in glaciolacustrine clay. Stabilization works included a cut-off drain along the ditch and a series of counterfort drains below the road. Follow-up monitoring suggests the slope movements have slowed but not stopped. At km 701-703, three sections of the highway along the eastern shore of Muncho Lake had a dangerously narrow shoulder due to erosion and slumping of the shoulder material into the lake. Stabilization of these areas was successfully completed using wire-basket MSE walls. This paper discusses the site conditions, design of the stabilization works, construction challenges and other lessons learned.

RÉSUMÉ
L'autoroute "Alaska Highway" no.97 traverse plusieurs zones à risque de glissement de terrain à travers le nord de la Colombie-Britannique et le Yukon. Tetra Tech EBA Inc. a été retenu par Travaux Publics et Services Gouvernementaux Canada pour évaluer les talus potentiellement instables au km 229 et entre les km 701 à 703 le long de l'«Alaska Highway» et pour réaliser la conception des travaux de stabilisation requis. Au km 229, l'origine des mouvements fut identifiée le long d'un plan de glissement dans une argile glaciolacustre. Les travaux de stabilisation réalisés incluent la construction d'un drain le long du fossé et une série de drains à contrefort sous la route existante. Le suivi du comportement suggère que les mouvements du talus ont ralenti, mais ne sont pas arrêtés. Entre les km 701 et 703, l'autoroute longe la rive Est du lac Muncho, trois (3) segments entre ces km ont un accotement très étroit due à une érosion et à des affaissements du matériel de remblais dans le lac. Ces segments ont été stabilisés à l'aide de murs de terre armée jumelée avec une protection contre l'érosion de type gabions. Cet article discute des conditions des sites, de la conception des travaux de stabilisation, des défis rencontrés et des autres apprentissages.

1 INTRODUCTION

This paper presents a case history of recent slope stabilization works completed at km 229 and km 701-703 of the Alaska Highway (BC No. 97) in northern British Columbia in order to improve public safety and reduce highway maintenance costs.

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.

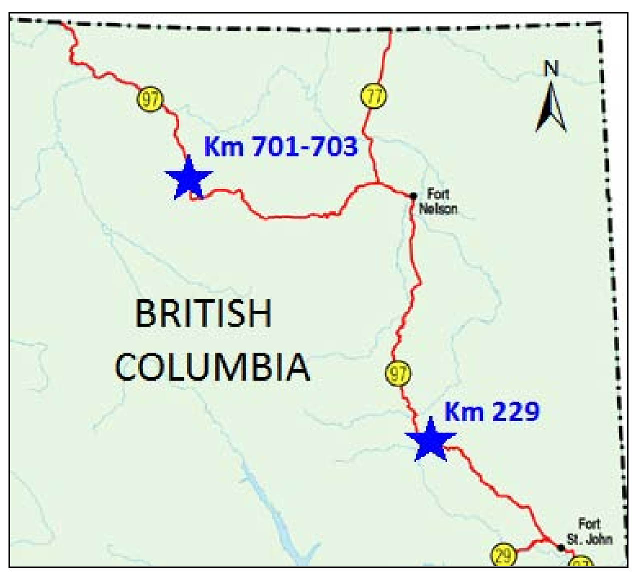

Figure 1. Site Location Map

At km 229, an approximately 300 m long section of the southbound lane and adjacent shoulder had experienced ongoing slope movements since the highway was reconstructed in the late 1980s, and annual repairs were required to maintain the highway grade. Tetra Tech EBA Inc. evaluated the site for PWGSC in 2009, finding that movement of the slope was occurring along a very weak sliding plane in glaciolacustrine clay. Groundwater levels were monitored continuously using portable data-loggers, revealing significant seasonal variations and sharp increases during periods of heavy rainfall.

At km 701-703, three sections of the highway along the eastern shore of Muncho Lake had a dangerously narrow shoulder due to erosion and slumping of the shoulder material into the lake. Tetra Tech EBA Inc. evaluated the site for PWGSC in 2013, finding that the road was supported by deteriorating gabion baskets founded on timber cribbing and blast rock.

The following sections discuss the conditions at each site in more detail, the stabilization measures undertaken and the post-construction performance of each site. The challenges of working in remote northern areas and other lessons learned during construction are also discussed.

2 KM 229 SLOPE STABILIZATION

2.1 Site Location

Km 229 of the Alaska Highway is located near the village of Pink Mountain, approximately 155 km north of the city of Fort St. John. The site is located at the margin of a broad plain which extends northward to the Beatton River valley. South of km 229 the highway grade climbs towards Pink Mountain through gently rolling mountainous terrain.

Figure 2. View of unstable slope at km 229, looking south towards Pink Mountain (Photo taken in September 2009).

2.2 Background

This section of the Alaska Highway was reconstructed by PWGSC in the late 1980s. During construction, very wet and soft ground conditions were reportedly encountered in the km 229 vicinity, to the point that traffic had to be rerouted a significant distance around the area. Slope movements affecting an approximately 300 m long section of the southbound lane and adjacent shoulder were experienced shortly after construction and continued to occur periodically thereafter, to the point that annual re-grading and patching were required in order to maintain the highway grade. Attempts were made by PWGSC to stabilize the area using interceptor ditches and perforated drains to reduce groundwater inflows from upslope but were unsuccessful. A series of rebar survey pins were also installed by PWGSC to monitor the slope, from which it was found that the slope was continuing to move at a rate of approximately 30 to 40 mm per year.

Investigation of the slope movements and stabilization of the slope were identified as a priority by PWGSC due to concerns about future maintenance requirements and to facilitate future repaving of the highway.

2.3 Topography and Climate

The site ranges from about 995 m to 1010 m above sea level within the foothills of the northern Rocky Mountains. Environment Canada weather records from the nearest station at Sikanni Chief (period 1991 to 2006) indicate that mean annual precipitation is about 565 mm, of which about 180 mm is snowfall. The average daily temperature ranges from about -15°C in the winter to 15°C in the summer.

2.4 Geological Setting

The surficial geology of this area is discussed in Bednarski (2000, 2001). The retreat of glaciers from northeast BC at the end of the last ice age resulted in the blocking of major drainage systems and the formation of large ice-dammed lakes in mountain valleys and the adjacent continental plain. Thick sequences of glaciolacustrine sand, silt and clay were deposited on the bottom of these lakes from the sediment-laden meltwater. These sediments are underlain at depth by glacial till and sedimentary bedrock.

2.5 Field Explorations

2.5.1 Site Reconnaissance

Tetra Tech EBA completed a ground reconnaissance of the km 229 area in June 2009. The key observations from this reconnaissance were as follows:

- The slopes below the highway are approximately 6 m high with relatively gentle grades (8 to 12 degrees), although locally steeper 30 degree slopes are present near the treeline.
- The slopes were generally wet including pockets of standing water observed at the base of the slope.
- The forested slopes above and below the highway right-of-way were also locally wet and soft with observed seepages and “wet-site” vegetation.
- Tension cracks up to 50 mm wide and 3 m long were observed along the highway shoulder. The southbound lane was also cracked and rutted.

2.5.2 Drilling and Laboratory Testing

Drilling was carried out in September 2009 to define soil stratigraphy and groundwater conditions and the nature of the observed slope movements. A total of nine solid stem auger testholes and three cone penetration test soundings were completed using a track-mounted drill rig. Soil samples obtained from the auger testholes were submitted for geotechnical laboratory index testing at Tetra Tech EBA's laboratory.

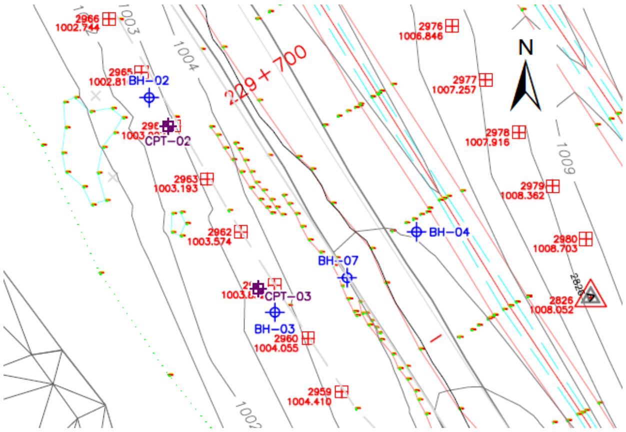

Figure 3. Survey plan of km 229 area showing testhole and rebar survey pin locations, topographic contours and location of tension cracks.

2.5.3 Soil Conditions

The soil stratigraphy at km 229 inferred from the testhole data was as follows:

- Unit 1: 0.5 to 2.5 m of variable fill and topsoil.
- Unit 2: 1 to 2 m of compact sand to silty sand, in places interbedded with silty clay.
- Unit 3: 1 to 5 m of highly plastic, soft to stiff, glaciolacustrine silty clay with lenses of sand and silt; generally increasing in thickness to the north.
- Unit 4: 7 m or more of low plastic, very stiff silty clay glacial till with some sand and gravel. Testholes were terminated in this layer.

2.5.4 Instrumentation

Slope Inclinometers (SIs) were installed in three of the testholes. The SIs were extended into stable ground at a depths of approximately 12 m below surface.

The results obtained from BH-03, located approximately 15 m downslope of the highway within the middle of the slide area are shown on Figure 4. From this figure it can be seen that the slope movements were occurring along a well-defined, discrete failure plane at a depth of about 5 m below ground surface, near the base of Unit 3.

Three vibrating-wire piezometers were also installed in Units 2 and 3, at depths of up to 5.3 m below ground surface. The piezometers were connected to battery-powered data loggers to provide automatic data acquisition on 24-hour intervals. The data obtained from the piezometer installed in BH-02 is shown on Figure 5. The key findings from the piezometer monitoring were as follows:

- The measured water levels were relatively shallow, which was consistent with the general site conditions and observations during drilling.
- Water levels fell by approximately 0.5 m between October 2009 and February 2010. This trend coincided with the onset of freezing temperatures and accumulation of snow.
- Water levels rose by about the same amount between March and April 2010, in response to rising temperatures and melting of the snowpack.
- Several sharp "spikes" in the data were recorded between April and October 2010, generally coinciding with rainfall events recorded at the Fort St. John airport which was the nearest source of daily weather records.
- Similar seasonal trends in the data were observed between October 2010 and May 2011.

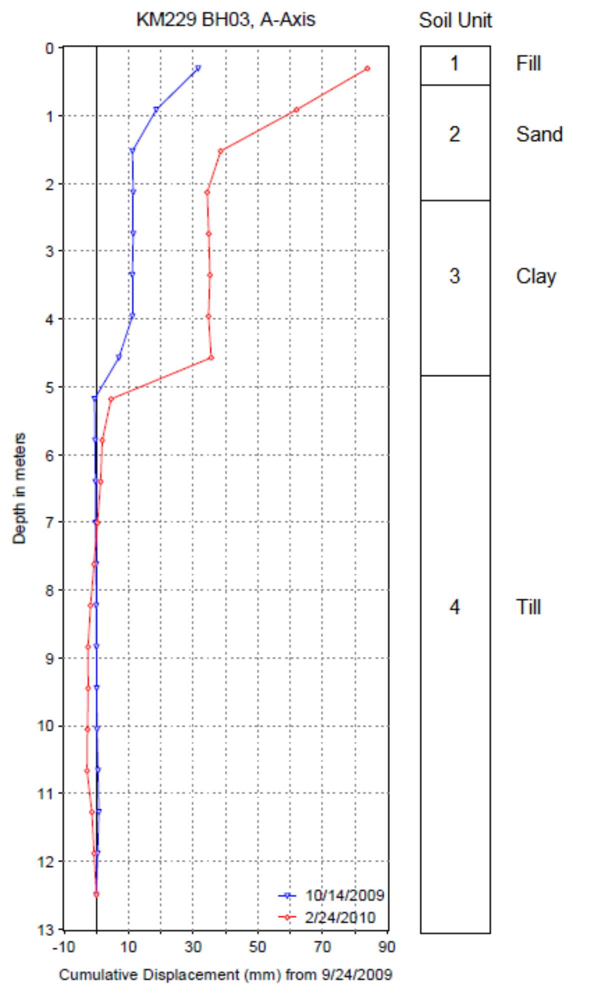

Figure 4. Movement in Slope Inclinometer installed in BH-03 in September 2009.

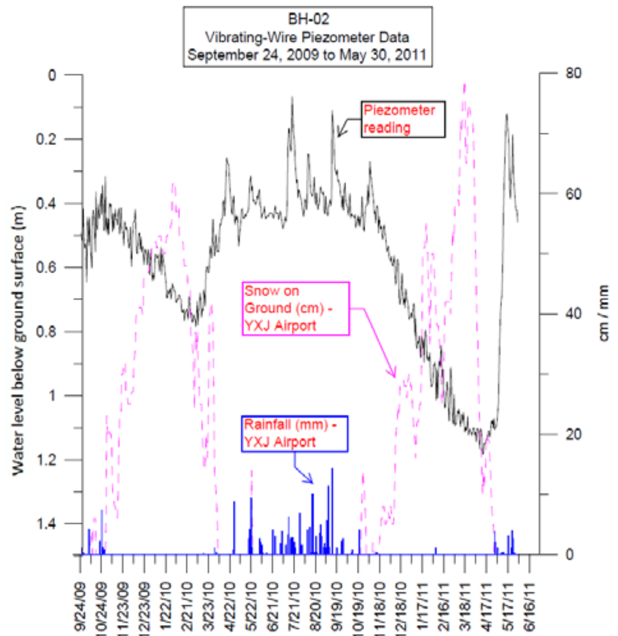

Figure 5. Vibrating-wire piezometer data from testhole BH-02 recorded between September 2009 and May 2011.

2.6 Back-Analyses of Slope

Two-dimensional limit equilibrium slope stability analyses were performed on the slope using commercial software in order to back-calculate soil strengths (based on a discrete failure plane in Unit 3), and to assess the sensitivity of water levels to slope stability. The following shear strengths were derived from the analyses, based on an assigned Factor of Safety (FoS) of unity:

Table 1. Soil Properties used in Back-Analyses

Soil Unit	Effective (Drained) Shear Strength Parameters		Unit Weight (kN/m^3)
	c' (kPa)	ø' (degrees)	
1 (Fill)	-	33	19.5
2 (Sand)	-	30	19
3 (Clay)	-	27 (12)[1]	17
4 (Till)	5	40	20.5

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

A typical back-analysis is shown in Figure 6, with various colours representing the inferred soil stratigraphy, dashed blue lines for groundwater, and vertical hatching of the critical failure surface.

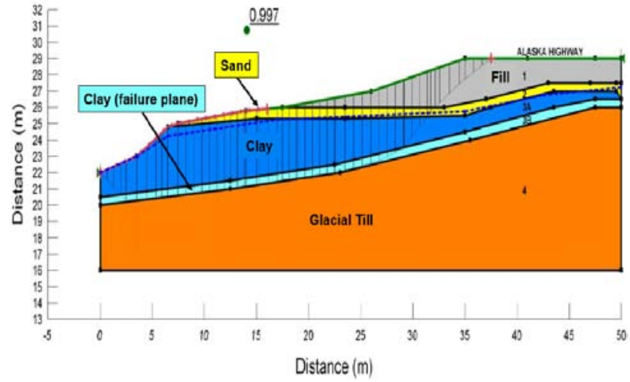

Figure 6. Typical back-analysis of the km 229 slide area.

As part of the back-analyses, it was also determined that the FoS was decreased by approximately 5% for each 0.25 m rise in water levels. It was therefore evident from the piezometer data that seasonal fluctuations in water levels had a significant effect on slope stability.

2.7 Stabilization Options

A variety of conceptual options were presented to PWGSC to stabilize the slope, including lowering the highway grade, excavation and replacement of the slide mass with granular fill, buttressing the toe of the slope, reinforcing the failure surface using shear piles or soil-cement columns, or installing drainage works to lower the water levels in the slope. 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 using a series of counterfort (trench) drains extending downslope of the highway to the treeline, consisting of a series of parallel trenches excavated below the failure surface and backfilled with coarse, free-draining granular fill.

2.8 Stabilization Design

The counterfort drains were designed following the approaches described in Cornforth (2005). From the design calculations, the drains were spaced every 12 m, with a minimum trench width of 1 m and depth of at least 0.5 m into till (Unit 4) to provide a minimum FoS of 1.3 under the anticipated seasonal range of post-construction water levels.

To reduce groundwater inflows into the slide area, the stabilization design also included the installation of a cut-off drain along the existing drainage ditch next to the opposite (northbound) side of the highway. This involved deepening the ditch by up to 2.5 m, lining the base of the excavation with an impermeable membrane and backfilling the excavation with a 150 mm diameter perforated steel drainage pipe surrounded by coarse granular fill as used for the counterfort drains.

2.9 Construction

Construction of the slope stabilization works was awarded to White Bear Industries Ltd. who completed the work over an approximately two-week period in October 2011. 16 counterfort drains were constructed below the highway in addition to the installation of the cut-off drain along the opposite side of the highway, with construction costs totalling $590,000. Repairs were also made to existing culverts in the general vicinity of the slide area to remove accumulated sediment and vegetation growth. The granular fill for the counterfort drains consisted of a well-graded 75 mm minus crushed gravel material sourced from an existing borrow pit located at km 262 of the Alaska Highway.

Tetra Tech EBA were not on site during construction; however, based on information provided by the contractor and PWGSC field staff, the conditions encountered during construction of the counterfort drains were as follows:

- Water seepage was generally not encountered in the trench excavations, which extended to a depth of up to 7 m below ground surface. This was the maximum practical depth that could be achieved based on trench stability conditions and the reach of the excavator.
- The interface between the clay (Unit 3) and the till (Unit 4) was identified by relatively abrupt changes in soil colour, texture and stiffness. The contractor described the clay as “peeling off” of the till, which suggests there has been significant weakening along this interface as a result of slope movement.
- All of the drains were excavated into till (Unit 4), with the exception of the three northernmost drains where till was not encountered.
- Minor new cracking was observed along the highway shoulder and southbound lane during construction of the counterfort drains, indicative of the marginal stability of the general area.

2.10 Post-Construction Performance

Based on anecdotal information from PWGSC staff, the repairs performed adequately for about one year, at which point cracking and rutting re-appeared along the southbound lane to a similar extent as before. Overall, the post-construction performance suggests that the slope movements have slowed but not stopped. Work is currently in progress to re-assess the site conditions and to complete the design of supplemental slope stabilization measures.

3 KM 701-703 SHOULDER STABILIZATION

3.1 Site Location

Km 701-703 of the Alaska Highway is located along the eastern shore of Muncho Lake, approximately 250 km north of the city of Fort Nelson, within Muncho Lake Provincial Park. The highway follows the shoreline of Muncho Lake from km 699 to km 711, and within this section is relatively narrow with several sharp curves and high rock cuts. The highway grade is relatively flat at approximately 2 to 3 m above lake level.

3.2 Background

Three short (30 to 50 m long) sections of the highway between km 701 and 703 had developed a dangerously narrow shoulder over the last several years as a result of erosion and slumping of the shoulder material into Muncho Lake, and were continuing to deteriorate. Shoulder widths in these sections were generally between 0.5 and 1 m compared to 1.5 m or greater in adjacent areas. The stabilization and widening of these areas was identified as a priority by PWGSC out of concern for public safety and to reduce future highway maintenance requirements. Figure 7 shows the location of the three sites (herein referred to as Areas 1 to 3) along the shore of Muncho Lake.

Figure 7. Site plan of km 701-703 vicinity showing location of Areas 1 to 3.

3.3 Topography and Climate

Muncho Lake is situated within the northern Rocky Mountains and lies at an elevation of approximately 820 m above sea level. The lake has an overall length of 12 km and an average width of 1 km. Muncho Lake is a relatively deep lake with a maximum depth of over 200 m.

Environment Canada weather records from the nearest station at Muncho Lake (period 1981 to 2010) indicate that mean annual precipitation is about 500 mm, of which about 180 mm is snowfall. The average daily temperature ranges from about -15°C in the winter to 15°C in the summer.

3.4 Geological Setting

According to Massey et al. (2005) the exposed bedrock along the eastern shore of Muncho Lake comprises grey dolomite belonging to the Upper Silurian to Middle Devonian Muncho-McConnell Formation. Upslope of the highway cut the bedrock surface is mantled by veneers to blankets of glacial drift and colluvium.

3.5 Field Explorations

3.5.1 Drilling and Laboratory Testing

Drilling was carried out along the highway at Areas 1 to 3 in April 2013 to define soil stratigraphy and the depth to bedrock. A total of eleven testholes were completed using a truck-mounted sonic drill rig, which were all terminated in bedrock at depths of up to 9 m below road surface. Soil samples obtained from the testholes were submitted for geotechnical laboratory index testing at Tetra Tech EBA's laboratory.

3.5.2 Site Reconnaissance

Tetra Tech EBA completed a ground reconnaissance of Areas 1 to 3 as part of a follow-up site visit in May 2013 when Muncho Lake was substantially ice-free. The key observations from this reconnaissance were as follows:

- The highway shoulder in these areas was fairly steep (up to 50 degrees) and supported by deteriorating gabion baskets as well as unstable and eroding riprap. According to PWGSC, the gabion baskets were thought to have been constructed sometime in the 1960s.
- The gabion baskets at Area 2 had toppled (slumped) outwards and had experienced significant differential settlement.
- Surface erosion from highway run-off and wave erosion from Muncho Lake both appeared to be contributing to the loss of the shoulder fill.
- The bottom row of the gabion baskets at each site was generally situated below lake level, and as such could not be observed in detail. At Area 2, log cribbing was also visible underneath the gabions at several meters below lake level.
- Tension cracks were observed along the highway shoulder and southbound lane at Areas 2 and 3.

Figure 8. View of Area 2 showing the existing riprap and gabion baskets. (Photo taken in May 2013).

3.5.3 Soil and Bedrock Conditions

The soil stratigraphy revealed by the drilling generally consisted of 0.5 to 2 m of asphalt (chip seal) and granular road fill, overlying 1 to 3 m of riprap and weathered/ fractured bedrock, overlying sound bedrock. The contact between these units was difficult to distinguish in the recovered cores due to drilling disturbance and the similar nature of the material.

3.6 Stabilization Options

3.6.1 Site Challenges

A number of challenges had to be considered in the evaluation of potential remedial options to stabilize these areas, including:

- The stabilization work would have to be completed while keeping the highway open to one-way traffic.
- Downslope of the highway, Muncho Lake drops off steeply with water depths in excess of 90 m, which posed a potential safety hazard to motorists and to construction crews.
- The variable nature of the road fills and the irregular depth to the underlying bedrock limited the viability of some options while increasing the potential for construction challenges. There was also the possibility that the shoulder repairs could experience differential settlement over time.
- The site is very remote (over three hour's drive north of Fort Nelson), with no cellular coverage, which meant that logistical and communication challenges could arise during construction.
- Muncho Lake is located within a Class "A" Provincial Park, and previous environmental studies showed these sites to have moderate to high quality fish habitat. As such, potential solutions needed to be aesthetically pleasing with minimal or no in-water work which could disrupt the fish habitat and riparian vegetation. Any option requiring in-water work would likely require extensive environmental permitting which could take several months or more to obtain.
- PWGSC had limited funds to complete these repairs and the work had to be completed within a short construction window due to environmental, weather, and safety constraints.

3.6.2 Options Considered

A variety of conceptual options were presented to PWGSC to repair these areas, including placing rock fill into Muncho Lake, re-aligning the highway upslope, installing sheet pile or concrete retaining walls anchored into bedrock, or reconstructing the shoulder slopes with new gabion basket walls or mechanically stabilized earth (MSE) walls. As with the km 229 project, each of these options was assessed on a variety of metrics, from which PWGSC ultimately decided to proceed with MSE wall option, but with a "scaled-down" design leaving the bottom rows of existing gabions in place to reduce construction costs and the potential need for in-water work.

3.7 Stabilization Design

Design of the MSE walls proceeded using a proprietary wall system consisting of galvanized wire-form basket facing elements infilled with 100 mm minus stone separated from the structural fill by a layer of geotextile. The design also included the installation of new gabion mattresses, where needed, to form a level working surface for MSE wall construction and to enhance drainage along the base of the wall. The advantages of the MSE wall system included the relative ease of basket installation and geogrid connections, the flexible, free-draining and settlement tolerant nature of the system, and durability under cold weather and exposure to water.

Commercial software was used to analyze the internal and external stability of the MSE walls based on the soil properties listed in Table 2. The MSE walls were inclined at a batter angle of 1H:6V and were up to 2.7 m in height in order to provide a minimum shoulder width of 1.5 m. From the design calculations, the reinforcing layers (geogrid) were spaced every 0.45 m vertically and extended to lengths of up to 4.2 m behind the face of the wall in order to provide a minimum global FoS of 1.3 under static loading.

Table 2. Soil Properties used for MSE wall design

Soil Unit	Effective (Drained) Shear Strength Parameters		Unit Weight (kN/m^3)
	c' (kPa)	Ø' (degrees)	
Existing Fill	-	35	18
Structural Fill	-	34	20
Riprap/Weak Bedrock	-	42	20

A typical global stability analysis is shown in Figure 9, with various colours representing the inferred soil stratigraphy, dashed blue lines for groundwater, and vertical hatching of the critical failure surface.

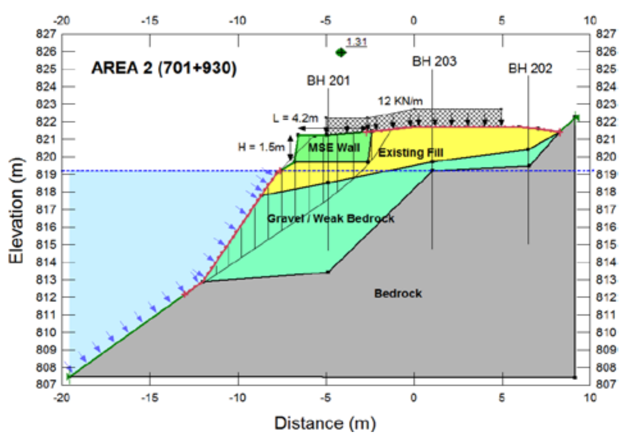

Figure 9. Typical global slope stability analysis of the proposed MSE wall at Area 2.

3.8 Construction

Construction of the slope stabilization works was awarded to White Bear Industries Ltd. who completed the work over an approximately three-week period in late September and early October 2014. A total of 125 m^2 of MSE wall and 95 m^2 of new gabion mattress (as measured along the face of the wall) were successfully installed, with construction costs totalling $410,000 at the completion of the project. Additional riprap was also placed at the ends of the MSE walls for scour protection. Previously stockpiled aggregates and blasted rock from nearby borrow sources were used to produce the fill materials required for construction.

The conditions in the excavations were broadly consistent with the testhole data, except for some areas where shallow bedrock, buried rows of gabion baskets and old tie-back cables were encountered. The excavation of these materials was completed without incident.

Figure 10. MSE wall construction at Area 3. (Photo taken in October 2014).

3.8.1 Key Success Factors

There were several benefits to using wire-faced MSE walls for this project, including:

- The additional available shoulder width at the completion of construction exceeded PWGSC's expectations given the challenging site conditions and the modest construction costs expended.
- Completion of the project construction without the need for any night time road closures or load width restrictions. A minimum of single lane alternating traffic with negligible traffic delays was maintained throughout the entire project.
- Completion of project construction within budget and ahead of schedule, thus returning the highway to two way traffic sooner than initially expected.
- Completion of the construction work entirely in the dry without needing to complete any work within Muncho Lake. This significantly reduced the environmental permitting requirements and corresponding schedule risks to the project.
- The materials and equipment used to construct the walls resulted in a more sustainable solution compared to other remedial options, based on the following:
    - o The use of existing stockpiles of aggregates and blasted rock from local borrow sources resulted in significantly lower material costs and reduced environmental impacts.

- o Reduced CO_2 emissions compared to other options as no concrete, grout or other cement-based products were required to complete the work.
- o Due to the compact and light weight nature of the MSE wall components, only one transport truck was required to deliver all of the wall materials to site.

Figure 11. View of completed wire-basket MSE wall at Area 2. (Photo taken in October 2014).

3.9 Post-Construction Performance

Tetra Tech EBA visited the km 701-703 area in February 2015 while en route to another project site along the Alaska Highway. The MSE walls appear to have experienced minor settlements in some areas, but overall are performing as expected.

4 CONCLUSIONS AND LESSONS LEARNED

The case histories presented in this paper provide an overview of the challenges associated with working in remote northern areas. The key conclusions and lessons learned from km 229 and km 701-703 are as follows:

- For the back-analysis of unstable slopes under drained loading, it is important to have thorough understanding of the groundwater regime in order to determine the correct friction angle for design purposes. As the case history from km 229 demonstrates, water levels may have significant seasonal variation.
- It is vital to manage client expectations in terms of post-construction performance where there is a possibility that the slope stabilization works may not fully remedy the problem, particularly in the case of drainage measures which may take time to become fully effective. The designer should also ensure there is a clear understanding of any assumptions or limitations inherent to the design.
- For remote sites, it may not be possible or practical for the designer to be on site during construction. With this in mind, communication and logistical challenges need to be well thought out at the start of the project, and detailed procedures put in place during construction. Depending on the nature of the project, it may also be prudent to incorporate additional conservatism into the design to account for unforeseen conditions, changes in construction techniques or other issues that could affect the performance of the stabilization works.
- Constructability is sometimes overlooked during the design process (Burland et al. 2012), but this is a key consideration for remote northern projects where materials such as concrete are not 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 construction are generally preferable.
- Sustainability is one of the key challenges facing the engineering community in the 21st century (Burland et al. 2012). The key principles of sustainability in terms of social, environmental and economic impacts should be incorporated into geotechnical practice where possible. Sustainability was a key consideration in the design of the slope stabilization works at km 701-703.

ACKNOWLEDGEMENTS

The writer would like to thank Public Works and Government Services Canada for permission to release the results of these case histories. Thanks is also given to Brian Hall for reviewing an earlier version of this manuscript.

REFERENCES

Bednarski J.M. 2000. Surficial geology, Trutch, British Columbia (94G); Geological Survey of Canada, Open File 3885, Scale 1:250,000.

Bednarski J.M. 2001. Drift composition and surficial geology of the Trutch map area (94G), northeastern British Columbia; Geological Survey of Canada Open File D3815.

Burland, J., Chapman, T., Skinner, H. and Brown, M. 2012. *ICE Manual of Geotechnical Engineering, Volume 1 – Geotechnical Engineering Principles, Problematic Soils and Site Investigation*. ICE Publishing. 697 pp.

Cornforth, D. 2005. *Landslides in Practice: Investigation, Analysis, and Remedial/Preventative Options in Soils*. John Wiley and Sons, Hoboken, NJ. 596 pp.

Massey, N.W.D., MacIntyre, D.G., Desjardins, P.J. and Cooney, R.T., 2005: Digital Map of British Columbia: Tile NO8 Northwest B.C., B.C. Ministry of Energy and Mines, GeoFile 2005-8.