Evaluation and Stabilization of a Slope Failure at Liard Highway No. 7, km 5.9 (Case Study)

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ABSTRACT
The Liard Highway (NWT No. 7) was constructed between 1969 and 1984. In summer 2007, the road was reconstructed at km 5.9, raising the vertical alignment. A fillslope failure occurred later that summer. The Government of the Northwest Territories, Department of Transportation, reconstructed the fillslope, and installed a French drain in the ditch to intercept groundwater flow. The repairs performed adequately until 2010, when the slope failed again. Tetra Tech EBA Inc. evaluated the site in 2010, finding an underlying swale and high groundwater inflow from upslope. Stabilization works included a longer, deeper French drain along the ditch, and two counterfort drains across the road. The gradient of the upper fillslope was reduced to mitigate surface ravelling of the fill. Follow-up monitoring suggests that slope stability has improved, but surface water erosion remains an issue.

1 INTRODUCTION
The original alignment of the Liard Highway (NWT No. 7) was constructed in between 1969 and 1984 by the Federal Government using local borrow materials, generally consisting of silt. In the summer of 2007, the Government of the Northwest Territories, Department of Transportation (DOT), reconstructed the area by raising the vertical alignment using crushed granular materials consisting of a layer of 50 mm minus-sized sub-base, and a surfacing layer of 20 mm minus-sized base course.

A slope failure occurred at the subject site, km 5.9, in the late summer of 2007 (Figure 1). DOT repaired the slope in 2007 by removing the upper silty soil (fill and/or colluvium) and peat, and reconstructing the slope in consecutive blast-rock benches, and track-compacting the material. A French drain was installed upgradient in the north-bound lane ditch to intercept groundwater flow and direct it towards a culvert to the north. The repairs were understood to have performed fairly well until 2010, when renewed slope movements were observed at approximately the same location as in 2007 (Figure 2).

In late October 2010, Tetra Tech EBA Inc. (Tetra Tech EBA, formerly EBA Engineering Consultants Ltd.), carried out a geotechnical site investigation, including the installation of standpipe piezometers and slope inclinometers in selected boreholes to assess the groundwater and slope stability conditions at the problem area.

Figure 1. Slope failure in 2007 (photo courtesy of DOT)

Based on Tetra Tech EBA’s analyses, some recommendations for slope stabilization were made. Of primary importance was groundwater management, with drawdown to be achieved by installing a longer, deeper French drain along the ditch, and two counterfort drains across the road. After the groundwater control measures were installed, the upper fillslope of the road was restored to a slightly flatter configuration, by building up the fill from the first bench below the road.
The following sections summarize the site description, measures taken to remediate the slope, and the results of the post-construction monitoring.

2 SITE DESCRIPTION

2.1 Site Location

The slope failure is located on the west side of the highway embankment along the Liard Highway at km 5.9, as shown in Figure 3. The highway is located about 1.8 km east and upslope of the Petitot River. The site is about 32 km south of the junction to Fort Liard, NT.

2.2 Climate

Environment Canada maintains and operates a meteorological station at the Fort Liard Airport, with monthly summaries available from 1996 through 2007. No precipitation data was available from 2008 through 2011, unfortunately, so comparisons could not be made to precipitation in 2010. Summertime precipitation records were sufficiently complete between 1997 and 2007, so as to acquire a sense of the relative magnitudes of rainfall events in 2007 as compared to the averages during that period of record.

The month of July 2007 had a record amount of precipitation as compared to the previous 10 years as well as to the months of record following, with 241 mm of rainfall recorded. When comparing to the averages over the 10 years of record, July 2007 had 6.2 times the average monthly precipitation during months with rain during the year (39 mm), and 3.4 times the average monthly precipitation in June/July/August (72 mm). It is not known whether or not the summer of 2010 was unusually rainy compared to the average at this location. Anecdotally, increased precipitation did seem to be linked with increased incidence of slope instabilities elsewhere in the Northwest Territories in both 2007 and 2010.

2.3 Surface Conditions

A review of the air photos indicated that the site area is located in an east-to-west draining swale situated on a gentle to moderate slope. Another road or trail was visible on the air photos, further upslope of the highway. To the north and south of the site, small streams were noted, flowing from east to west along either side of a knoll above the road. Some possible slope instabilities were noted on the steeper south-facing slope of the southern stream gully. It is not known whether debris from this area may at times impact the road.

The slope above and below the road is almost entirely covered in deciduous vegetation, often a disturbed-site indicator. Leaning and jackstrawed trees were noted at the toe of the slide area during Tetra Tech EBA’s site visits, indicating soil movement at the toe of slope (Figure 4).
On-site observations indicated that the road slopes down to the north at the site. The road appeared to have crossed a swale at the site location, although the open slope immediately above the site did not make this obvious. According to DOT, bedrock was encountered on the upslope side of the road.

The failed slope below the highway was configured in four benches, not including the crest of the slope at road surface elevation (Figures 4 and 5). The upper three of these benches were created as a slope stabilization measure after the 2007 slide. The fourth bench extended a few metres into the trees at the bottom end of the slide, apparently due to the toe of the slope failure bulging out into the trees, although the crest of the fourth bench appeared also to have been later built up during the 2007 remediations. Due to previous slope remediation work, gravel, cobbles and boulders were prevalent on the slope surface. The uppermost slope section, below BH-06, was overly steep, and therefore subject to raveling significant enough to preclude drilling on the first bench below. Very little snow cover was present at the time of the site investigation (Figure 4).

A 20 cm wide tension crack was observed approximately 1.0 to 1.2 m inside the road shoulder (Figure 2). There was about 10 cm to 15 cm of differential settlement across the crack. The depth of the tension crack was difficult to determine since significant fill had been previously placed into the crack by DOT to prevent water from flowing into it; however, at least 1.0 m to 1.2 m depth could be detected by probing with a wooden stick.

Cracks of about 10 cm in width were observed on the rock benches with settlements about 30 to 40 cm (Figure 6). The width of the headscarp of the slide, as delineated by cracks along the highway, was about 9 to 10 m, becoming 30 m wide on the second bench, and 50 m wide on the third bench, as defined by the cracks on either side of the slope. The depths of the cracks in the sidescarps could not be determined.

Clear water was observed to be flowing from the previously-placed granular bench, near the north end of the slide area. Evidence of soil erosion was identified near the toe of the central slope, but was not active at the time of the site drilling. The seepage channel may be intermittent or it may have changed as a result of the remediation work done in 2007.

Clear water flow was also noted in the north-bound lane ditch. This water flow was conducted to a corrugated steel pipe (CSP) culvert, which was located approximately 50 m north of the centerline of the slide area.

2.4 Subsurface Conditions

Due to the history of the site, including probable ancient slide debris deposits, road construction and upgrading, and slope failures and repairs, the stratigraphy at the site is highly complex. An idealized stratigraphy is summarized as follows:

A layer of gravel fill was noted to be about 3.5 m thick, composing the upper part of the highway embankment. The gravel fill was compact, containing little to some clay, and gravel sizes up to 50 mm, with raveling and surface erosion noted on the steep embankment.

A highly-variable layer of fill consisted of the layers, lenses and benches of soil types produced from the original construction of the highway, and the 2007 slope failure and subsequent slope repairs in 2010 (Figure 7). Constituent fill types included clay, sand, and gravel, with varying amounts of other constituents contributing to each fill type. Blast rock was noted at ground surface but not encountered in the boreholes. The fill varied from about 1.5 to 6 m thick.

A 0.05 to 0.3 m thick peat layer was encountered beneath the fill in all the test holes except boreholes BH-03 and BH-07. DOT encountered peat up to 1.0 m thick in 2007 beneath 0.5 to 1.0 m of silt fill and/or possible colluvium in two testpits along the ditch. The nearest of the testpits would have been located roughly between the road shoulder and BH-07 (TP-04), and the
other about 25 m to the south (TP-06). A centerline borehole from Public Works Canada’s (PWC) 1978 drilling program, designated km 246, Hole No. 7, and located at Station 246+880, encountered peat to a thickness of 1.8 m at ground surface.

Precise locations for the DOT and PWC test holes were not available. The locations of the DOT test pits were estimated based on the understanding that the 2007 and 2010 headscarsps were in about the same location. The PWC borehole location was approximated based on recorded PWC chainage compared to overall Liard Highway chainage from the junction with Highway 1 near Fort Simpson, to the British Columbia – Northwest Territories border. The entire slope may have had thick peat before construction or it may have been only a local anomaly. The boreholes within 1.74 km south and 1.33 km north of PWC’s Hole No. 7 encountered no more than 0.3 m of peat and, most often, no peat at all, providing some confidence that the peat encountered by PWC and DOT was at the same location, at the subject site. We surmise that either some of the peat at this location was removed during construction (since very little peat thickness was observed by Tetra Tech EBA beneath the embankment), or the fill/colluvium encountered by DOT at ground surface served to consolidate the peat.

2.5 Groundwater Conditions

Groundwater was recorded during the drilling investigation and later site visits. Groundwater measurements to date are presented in Table 1. Apparent artesian pressure was noted in BH-04 and/or BH-05 during the drilling investigation and follow-up site visits, resulting in water levels above ground surface. Considerable icing was observed in the culvert on the north side of the site. The seepage at the toe of the slope below the road appeared to be active year-round.

Table 1. Summary of groundwater levels

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2.6 Permafrost Conditions

The site is in the region of sporadic discontinuous permafrost. No permafrost or frozen soil was detected during the site investigation.

3 SLOPE EVALUATION

3.1 Sliding Mechanism and Contributing Factors

The initial slope monitoring results indicated a sliding layer located within the loose and/or soft-to-firm till-derived colluvium, just above the hard/dense till. The movements in BH-01 and BH-02 are summarized in Figures 8 and 9.

Figure 7. Benched slope, BH-03 (white), BH-02 (red)

Beneath the peat, a 1.0 to 4.7 m thick layer of till-derived colluvium was encountered in all of the test holes except borehole BH-07 on the upslope edge of the road/detour. The colluvium was found to be soft to stiff or loose, and was often difficult to distinguish from similar-textured fill soils, though the peat layer helped to distinguish the two.

Beneath the colluvium, a spatially-variable layer of clay and/or sand till was encountered. This layer was usually readily distinguishable from the overlying colluvium due to a much higher density or consistency, indicated by much higher Standard Penetration Test N-blow counts. The till tended to be very stiff to hard, or dense to very dense.

The soil at the site was highly variable spatially, and test holes only a few metres apart sometimes revealed radically different stratigraphies, both due to (assumed) natural slope events and human intervention on the slope.

Although apparent movements were also noted near ground surface, these upper soil movements were also
considered attributable to seasonal changes in the soil. Conversely, the movements at depth, although small, indicated very precisely where the primary sliding zone was located.

Figure 9. Deflections in BH-02 recorded in 2011.

Contributing factors to the slope failure included:

- Groundwater originating from upslope of the road. The local drainage area is concentrated in a gently to moderately sloping swale under the road. Water serves to increase the soil porewater pressure and soften the soils;
- The groundwater monitoring results suggest that the water in the slope is acting as a spring, emerging under artesian pressure on the lower two benches of the slope. The spring(s) could be natural or triggered by road construction.
- Reduced effective stress in the slope due to high porewater (artesian) pressures appear to have contributed to the slope failures.
- Record precipitation that can trigger or accelerate slope movements. The fill slope and/or underlying weak soils may have been gradually creeping, with extreme rain or spring thaw events making the movement more noticeable.
- Loading of the soft / loose till-derived colluvial soils as a result of road construction/maintenance triggering or accelerating movements in soft underlying soils.
- A weak peat layer that also readily transports water, producing a preferential sliding surface.

3.2 Semi-Quantitative Slope Analysis

3.2.1 Back-Analyses

Back-analyses were performed on the slope at its approximate configuration in 2010, based on the topographic data provided by Sub-Arctic Surveys Ltd. and cross-section-specific topographic information collected by Tetra Tech EBA. Using index properties of soils determined by laboratory testing of representative soil samples from the site, and comparing to results from other sites, shear strength parameters were estimated for the site soils. A factor of safety of one was confirmed for a failure of the overall slope. Piezometric surfaces were modelled for the granular fill, peat, and colluvial layers, accounting for the apparent artesian conditions in the vicinity of BH-05 and BH-01 on the lower slope.

A typical back-analysis is shown in Figure 10, with various colours representing the idealized soil stratigraphy described previously, dashed blue lines for groundwater, and cross-hatching for the failed soil.

Figure 10. Typical back-analysis for overall slope

The resulting parameters were checked for a failure of the upper granular fill immediately below the road. The findings indicated that the upper slope was not the critical element in the slope stability problem. Rather, if the overall slope had a factor of safety greater than one, the upper slope would most likely also have a factor of safety greater than one with respect to potential deep-seated movements, though not for the afore-mentioned ravelling.

3.2.2 Analyses for Slope Stabilization

The parameters obtained from the back-analyses were used to evaluate various potential solutions for slope stabilization. A solution to draw down the groundwater table was considered to be of primary importance and this was evaluated first. The concept incorporated a French drain in the ditch above the road, deeper than the existing drain, and long enough to encompass the entire slide area, with two counterfort drains across the road to lower the groundwater table within the embankment and move water rapidly off-site, as shown in Figure 11.

Figure 11. Slope drainage concept

Tetra Tech EBA evaluated three possible scenarios:

1. The groundwater table in all soil layers except the colluvial soil layers was drawn down to the bottom of the drains, while the colluvial soil layers did not change. In this case, the factor of safety was 1.1, a very slight improvement.
2. The groundwater table in all soil layers was lowered, but the colluvial layers would see only a partial decrease in the groundwater table. This resulted in a more significant increase in the factor of safety, to 1.3.

3. The groundwater table in all the soil layers including the colluvium was drawn down to at/near the bottom of the drains, with an increase in the factor of safety to about 1.4.

Since the counterfort drains did not penetrate through the bottom of the colluvial soils in the slope model, the water table was modelled only as low as the bottom of the drains. If the French drain along the ditch were installed so that it penetrated into the till, it should capture most of the water from upslope, and the lower colluvial layer at the bottom end of the slide should in fact have a groundwater table lower than modelled. Because the influence of soils and groundwater conditions along the sides of the slope area was unknown, Scenario 3 seemed reasonable without being potentially too unconservative (Figure 12).

For restoration of the fill slope, Tetra Tech EBA considered three possible scenarios:

1. The 2010 fill slope configuration could be maintained with the same steep slope at the top immediately below the road. That steep initial slope was at about 1.3H:1V (74% or 37°).

2. The 2010 fill slope configuration could be modified slightly, keeping the lower slope as is, but flattening the steep upper slope to about 1.8H:1V (55% or 29°).

3. The entire 2010 fill slope could be flattened to about 3H:1V (33% or 18°) by infilling the slope between the crest of slope at the road edge and the crest of the lowermost bench at the treeline.

The first scenario would not require any remedial fill to be placed, other than restoring the road grade to its original configuration. The factor of safety in this scenario was 1.5 for the upper fill slope and 1.4 for the overall slope. However, improving the long-term performance of the slope should also include mitigating erosion and surficial ravelling. Repeated events of this nature could eventually affect the integrity of the road.

The third scenario, in which the entire slope would be graded to a 3H:1V configuration, could make the road safer for the travelling public in the event of a breach of the guardrail. However, the additional loading from the fill on the slope reduced the factor of safety in the slope model, due to the loose and/or soft to firm colluvium:

The second scenario, with the addition of fill only to the uppermost slope section, also added load to the slope and resulted in a very small decrease in the factor of safety for the overall slope, to 1.35. However, the improved protection against erosion and surficial sloughing on the upper fill slope was thought to compensate for the small reduction in the factor of safety.

4 POST-CONSTRUCTION CONSIDERATIONS

Based on the concept shown in Figure 11, Tetra Tech EBA provided the following guidance for post-construction expectations:

- We anticipated that groundwater levels would be significantly lowered within the slide mass.
- We noted that this drawdown could occur gradually over several months or more, due to the relatively low permeability in portions of the fill and/or colluvial soils. The progress of the drawdown might also vary across the site, since these materials also vary significantly in drainage characteristics. In addition, seasonal fluctuations would likely continue to occur in response to rainfall and snowmelt events.
- As such, it was considered possible that tension cracks would re-appear on the highway and/or road shoulder following construction, until such time as the drainage measures became effective.

5 CONSTRUCTION SUMMARY

5.1 General

Rowe’s Construction Ltd. built the slope stabilization measures on behalf of DOT. Tetra Tech EBA was on site from September 30 to October 13, 2011, to observe the work. Figure 13 shows the stabilization measures installed, including the flattened fill slope area. The following sections summarize the observations.
blast rock up to a depth of about 0.6 to 1.0 m below the ditch invert, wrapped with geotextile, then capped to limit the infiltration of surface water (Figure 14).

Figure 14. Blast rock placed on geotextile in French drain.

5.3 Counterfort Drains

The counterfort drains were constructed in the same way as the French drain, with variations in cover thickness and/or capping materials according to location. The north counterfort drain varied from about 4.5 m deep at the east end, to 8 m at the west road shoulder, terminating directly south of the existing culvert outlet. The south drain varied from 6.5 m at the east end, to 8.5 m at the west road shoulder, terminating about 8 m east of the treeline. The trenches were typically about 1.3 m wide at the base, with top-widths of 3 to 8 m.

Observations of the counterfort drain construction are presented in Figure 15. A yellow marker identifies the culvert outlet.

Figure 15. Outlet of north counterfort drain near culvert

5.4 Upper Slope Flattening

Once the drains were complete, blast rock was placed along and above the first bench below the road to flatten the gradient in the slope failure area. The blast rock layer thickness varied significantly, depending on topography.

5.5 Construction Challenges

Sloughing of the trench side walls occurred at various locations in the French and counterfort drains. Sloughing was sometimes severe, especially in the deep trench sections beneath the road. Efforts to limit sloughing included opening only short sections of trench at one time (10 to 15 m), and cutting back the upper slopes of the trench. Collapsed sections of trench had to be overexcavated and the installation redone.

Constraints imposed by traffic also created significant challenges during construction. Trenches for counterfort drains beneath the road were not permitted to be left open overnight, and could only be open over one lane of the highway at a given time, so traffic could pass safely. This meant that the counterfort drains had to be completely or partially backfilled before the end of each day or before opening a trench in the opposite lane.

6 FOLLOW-UP SLOPE MONITORING

Follow-up slope monitoring did not commence immediately upon completion of the stabilization works; however, two sets of post-construction readings have now been acquired from the remaining instrumentation on site. As might be expected, the first set of slope inclinometer readings in 2013 showed significant changes compared to the pre-construction readings, due to the large disturbance resulting from the major excavation works on the site. The water level readings in 2013 and 2014, however, indicated that the groundwater table had been significantly drawn down, by 0.82 m in BH-04 and 0.98 m in BH-06-SP1 in 2013 and 2014, compared to prior levels at the same time of year (Table 1).

It appears that while groundwater has not been drawn down to the bottom of the drains over the entire site, this was achieved over at least part of the site (a combination of Scenarios 2 and 3). We note that the invert of the south counterfort drain is 0.90 m lower than the water level measured in BH-06-SP1, so plainly the groundwater had not been drawn down as far as the bottom of the south counterfort drain at the road shoulder. It does appear as though it may have been drawn down to the bottom of the north drain, however, based on the estimated elevations of the drain invert at the road shoulder and at the outlet.

The standpipe at BH-06 was not found in 2014, so it is not known whether or not the drain would have achieved further drawdown in the groundwater table at BH-06. The water level in BH-04 was nearly the same in 2014 as in 2013, so it is possible that the maximum drawdown effect has already been achieved at this location. BH-04 is lower on the slope than BH-06 and, therefore, the west (lower) end of the drain has less effect on the groundwater elevation at BH-04 than it does at BH-06. However, even though the counterfort drains daylight at the approximate elevation of ground surface on either side of BH-04, the artesian conditions at BH-04 appear to have been eliminated, at least at the times of measurement.

No obvious seepage was noted at the drain outlets, although seepage at the north drain was obscured by flow and pooling water that appeared to originate from the culvert (2013 and 2014). No water was present in the shallow swale between BH-01 and BH-04, below the toe of the fill/colluvium (marked intermittent seepage on Figure 5). Nor was other water seepage noted along the toe, although the slope toe was difficult to see due to the
thick underbrush, with seepage points further obscured by episodes of heavy rain during the 2013 site visit.

The drawdown from the drains is not enough to draw the groundwater table below the apparent sliding layers at BH-01, but it appears to be sufficient to draw it down below the sliding layer within the upper clay fill in BH-02, and below the suspected sliding layer in the peat, in BH-06, where the greatest proportion of the slope fill load is located. Artesian conditions no longer prevail, but the groundwater table is still within the soil debris.

![Image](image.png)

**Figure 16. Deflections in SI-01 (BH-01) near toe of slope**

Two distinct sliding planes were noted in BH-01: at about Elevation 483.3 m just above the hard/dense till (clay or sand, depending on location); and at the base of the peat layer at about Elevation 487.9 m (Figure 16). The overlying fill or colluvium also appears to be moving, with likely influences from seasonal freeze-thaw effects and creep due to seasonally-high soil moisture contents, as noted in the pre-construction monitoring, as well as consolidation. Possible movement in the colluvium just below the peat could be related to continued consolidation due to loading from the overlying soil debris. Rates of slope movements are in general greatly reduced compared to the pre-construction readings.

Aside from monitoring of the instrumentation, visual observations indicated that while global stability seems to have greatly improved since implementation of the stabilization measures, surface erosion is still a significant issue in the relatively fine gravels of the upper layers of the road cross-section. Surface runoff from the road cuts into the road grade, with one 10-minute downpour observed during the 2013 site visit resulting in 15 to 30 cm of downcutting near the crest of the fill, and more rain in the following 12 hours or so resulting in another 30 cm of downcutting (Figure 17).

**7 CONCLUSIONS**

While challenges were encountered during construction of the slope stabilization measures, adaptations to the proposed designs and construction methods were made as needed during the course of the work to accommodate the highly-variable ground conditions. As expected, some post-construction deformation has been experienced. Some successes have been realized with the drawdown of the groundwater table and a reduction in the rate of slope movements.

![Image](image.png)

**Figure 17. Erosion at fill crest from surface water off road**

Continued monitoring will show whether the groundwater table will continue to draw down, and whether the rate of slope movements will continue to decline. Continued visual observations at the subject site will also help to track possible ongoing erosion problems due to water runoff from the road surface, and flag necessary repairs before the erosion becomes critical.

**ACKNOWLEDGEMENTS**

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**REFERENCES**


