Temporary Slope Stabilization at the Airport Trail Tunnel Excavation, City of Calgary

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ABSTRACT
The City of Calgary identified Airport Trail as a key East-West Corridor providing connection from Deerfoot Trail to Stoney Trail. The Calgary International Airport expansion project resulted in a need to develop a tunnel to accommodate east-west traffic under the new runway and taxiways. This tunnel was constructed as a cut and cover tunnel requiring excavation up to 19 metres deep in the local tills and Tertiary soft rocks. This paper describes the rock slope issues and mitigating measures to allow safe construction to proceed.

RÉSUMÉ
La ville de Calgary a identifié l'Airport Trail comme étant un corridor est-ouest clé assurant la connexion entre la Deerfoot Trail et la Stoney Trail. Le projet d'expansion de l'aéroport international de Calgary a donné lieu à un besoin de développer un tunnel pour répondre au trafic est-ouest sous la nouvelle piste et les nouvelles voies de circulation. Ce tunnel a été construit comme un tunnel couvert nécessitant une excavation jusqu'à 19 mètres de profondeur dans les tills locaux et les roches tendres du Tertiaire. Ce document décrit les problèmes de talus rocheux et les mesures d'atténuation afin de permettre une construction sécuritaire.

1 INTRODUCTION

The Calgary International Airport was undergoing the single largest expansion in its history. The expansion included a $640 million runway development program which added a 4267 m (14,000 ft) runway along with associated taxiways and other infrastructure. In conjunction with the runway addition is The City of Calgary's 36 m wide by 620 m long tunnel underneath the runway to accommodate the Airport Trail route which provides an additional access from the east side of Calgary to the airport while relieving some of the congestion on Deerfoot Trail. The tunnel was opened to traffic in the spring of 2014.

Construction of the Airport Trail Tunnel (ATT) required excavating over 600,000 m^3 of soil and rock using the cut-cover method of tunnel construction. The 15 m deep by 50 m wide excavation exposed about 3-5 m of soils and 10-12 m of bedrock. The ATT structure was built with cast-in-place reinforced concrete which required various tradesmen to work at the base of the excavation and near the toe of the 13-17 m high slopes. The side slopes of the temporary excavation were exposed for several months between August 2011 to October 2012 during which the concrete structure and backfill activities were initiated.

PCL Parsons Dufferin ATU Joint Venture (PPD-JV) was appointed construction manager by The City to manage the onsite construction activities for the project.

Geotechnical site investigation work (core drilling and seismic surveys) completed by others was conducted from 2008 through to 2010. The ATT project design started in April 2011 with excavation commencing in July 2011. Various scenarios for cut slopes were investigated including:

- Conservative 1:1 excavation slopes that would pose less risk but would take longer to excavate and would require a higher construction cost due to increased excavation and backfill quantities and larger cranes required to support the structural concrete work.
- Near vertical excavation slopes with engineered shoring to maintain stability which was both cost and schedule prohibitive.
- Globally stable slopes at 0.5H:1V that would require some local stability improvements that would be identified as excavation progressed.

The third option was chosen and excavation progressed while the slopes were visually monitored. Once the cut slopes were scaled of loose rock, PVC sheets were placed to mitigate the slopes from rapid weathering and erosion.

2 PROJECT CONTEXT

Figure 1: Cut Slope Scenarios

Figure 1A - 1:1 slopes increase the volume of excavation, soil and rock slope face area, however, these slopes are still subject to hazards caused by raveling and spalling of loosened material and therefore still require protection

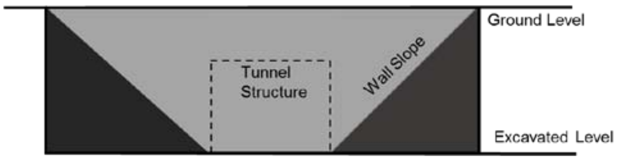


Figure 1B - Near vertical slopes require engineered shoring

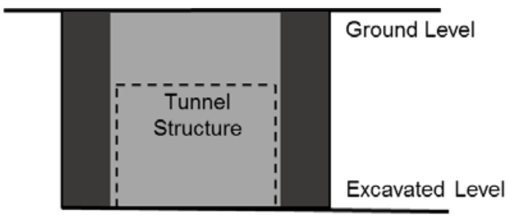
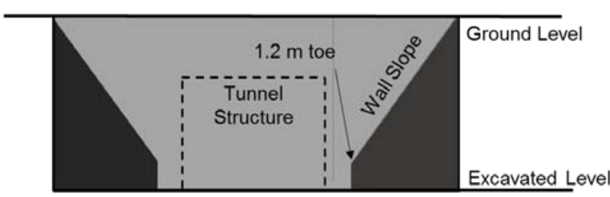


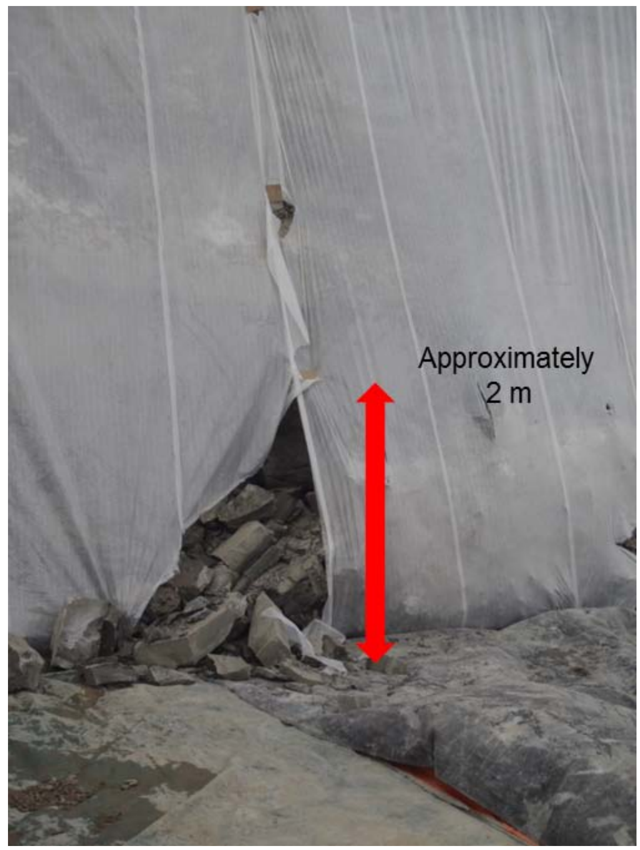
Figure 1C – Engineered Shoring (example)

Figure 1D - 0.5H:1V globally stable slopes offer reduced excavated volumes relative to Figure 1A with modest slope protection



By late November of 2011, the excavation of the initial portion of the tunnel had been completed and some concrete work had started when an uncontrolled rockfall incident near the toe of the open cut rock slope occurred. Given its safety implication, this rockfall incident shut down all construction activity below the excavation slopes. The delay and the slope protection scope had the potential to have significant impacts to the overall project plan (cost and schedule). The rock fall occurred as a wedge failure where the bottom 1.2 m was excavated vertical to allow more working room to complete form work and concrete work. The PVC tarps that were draped on the slopes partially guided the rockfall. It was also observed that the PVC tarps were holding ravelled rock pieces in various locations potentially leading to additional safety concerns if the tarps failed. They were not designed to retain broken rock.

Figure 2: Rockfall Incident Photograph



In mid-December of 2011, Terracon Geotechnique Ltd. was retained to investigate this incident, the construction safety of the slopes and to recommend mitigating measures that would allow the construction activities at the site to resume and continue in a safe manner. Terracon conducted a review of the initial rockfall site, original design considerations, geotechnical site investigation findings, construction plan, local geology, construction issues and analyses to develop recommendations for mitigation strategies. In addition, Terracon conducted immediate and regular slope monitoring to proactively observe the slopes to ensure the site was safe for work to proceed while further mitigation measures were evaluated.

3 DESIGN CONSIDERATIONS AND INITIAL SITE OBSERVATIONS

3.1 Development

The bedrock was studied in a geotechnical investigation completed by another engineering consultant during the planning phases of the project, in which the unconfined compressive strength of the rocks was tested on selected bedrock core samples. The results indicated that the bedrock strengths range from hundreds of kPa to 39.5 MPa and during the excavation localized rock strengths were tested to 150 MPa. Of all the stratified rocks, the claystone is associated with lower strength, whereas the sandstone represents the most competent rock.

Site investigations included a seismic examination to estimate rock hardness and advice on rock excavation methods. This investigation work did not characterize spatially or physically, the fracture families in the non-tectonically disturbed soft rocks.

The soil overburden was characterized as till (boulder clay), stiff to very hard with a typical thickness of 2.5-4 m overlying, unconformably, the soft rocks of the Tertiary Paskapoo Formation. The soft rock lithologies ranged from soft, weather sensitive clays, to hard siltstone and very hard sandstones. All lithologies were fractured by vertical to subvertical joint families together with bedding thickness usually less than 50-75 cm. These joints disrupt the excavated rock faces with resulting local shallow unstable zones. The joint patterns, bedding, and variable lithologies resulted in slopes that were irregular once scaled and susceptible to ravelling. The harder lithologies resulted in larger blocks of rock that posed a significant safety threat as the softer rock layers deteriorated allowing the larger blocks to fall from the slopes.

Local perched water tables were predicted in the geotechnical site investigation and found as evidenced by water that would seep from the freshly excavated slopes for various periods of time before disappearing.

3.2 Slope Designs

Because of the planned cut-and-cover methodology, the cut side slope was considered to be temporary in nature. Therefore, the unshored till and rock faces were designed at 1:1 and 0.5 to 1 (H:V) respectively with the lowest 1.5 m cut vertical leaving a compound bench face on each side of the excavation.

The 0.5 to 1 slopes in rock were determined to be globally stable. The need for slope protection measures was expected locally and would be determined on a case by case basis as the excavation proceeded.

3.3 Excavation Method

The original construction plan called for an open cut excavation approximately 700 m long with a maximum relief of 19 m from ground surface to the floor of the excavation. A cut and cover construction technique including an open excavation without shoring of the side slopes was visualized.

The method of excavation included terrain levelers, large backhoe excavators, large ripper dozers and mechanical (rotary) rock breakers. Blasting was not required nor permitted. The excavation method did not cause dynamic stress loading of the wall but did result in irregular cut faces with slightly variable face angles and local overhangs with 0.5 to 1 m relief and in one case 1.5 m of overhang. These irregular rock faces combined with the joint system caused local rock failures. Initially the rock faces for the softer strata were cut to a reasonably smooth face but the action of weathering at joint and bedding surfaces and scaling of the slopes resulted in the irregular slope faces where softer materials would undercut and expose the more resistant layers.

3.4 Temporary Groundwater Control

The perched groundwater systems were local, sporadic, and caused minor face weeping. One spring location provided heavy flows from the rock at a slope toe location on the south side of the excavation. It was also observed that storage of runoff water for sediment control on the nearby runway construction project also caused infiltration and flow through the ground to the tunnel which reduced once the storage ponds were relocated.

Groundwater inflows were controlled by ditches, sumps and pumps.

3.5 Weathering Control

At the time when the rockfall was reported, the cut face was covered by reinforced PVC sheets, or tarps, to prevent the cut face from weathering and eroding, as recommended in the original geotechnical report. Although the tarps protected the slopes from direct impact by rain and snow the rock continued ravelling beneath the tarps. The tarps reduced the ability to visually inspect the slopes for instability concerns and had to be abandoned.

3.6 Earth Pressures

The shallow nature of the excavation, (nominally excavated in height that would be equivalent to one bench height in a typical rock quarry) did not develop significant lateral earth pressures. Additionally, Calgary is a seismically stable area and no dynamic loadings were included in the design for the temporary slopes.

4 GEOLOGY OF THE CALGARY AREA

The bedrock of the Calgary urban area consists primarily of flat lying to gently dipping sandstone, siltstone, and claystone of the Paskapoo Formation (Figure 3). Sediments in the area of the ATT excavation are from the Lacombe Member of the Paskapoo Formation which contains variable beds of fluvial channel sand and muddy floodplain sediments laid down during the Paleocene Epoch. The unit has been characterized as having marked inhomogeneity both vertically and horizontally within the succession (Hamblin, 2004). The siltstone and claystone lithologies are weather susceptible consisting of medium to dark grey or greenish-grey rocks with thin fine-medium grain sand interbeds. The siltstone and claystone beds are soft relative to the sandstone and display blocky fracturing with little to no fissility.

Consistent with the regional bedrock geology, the Tertiary sediments at the tunnel construction site are essentially flat lying and the beds are interpreted to have negligible deformation by tectonic forces. The observed fracture network overprinting the bedded sediments at the ATT site showed no sign of displacement. Bedding orientation relative to the fracture array can be seen in Figure 4.

Figure 3: Upper Cretaceous - Tertiary Depositional Assemblages, Western Canada Sedimentary Basin

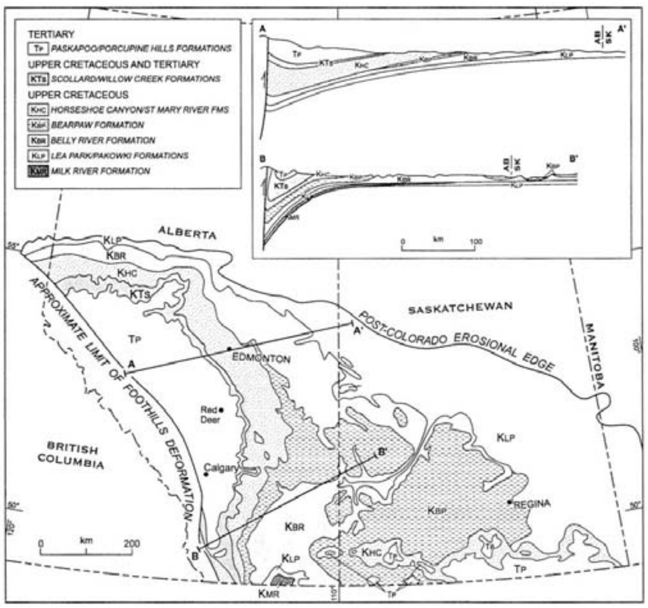
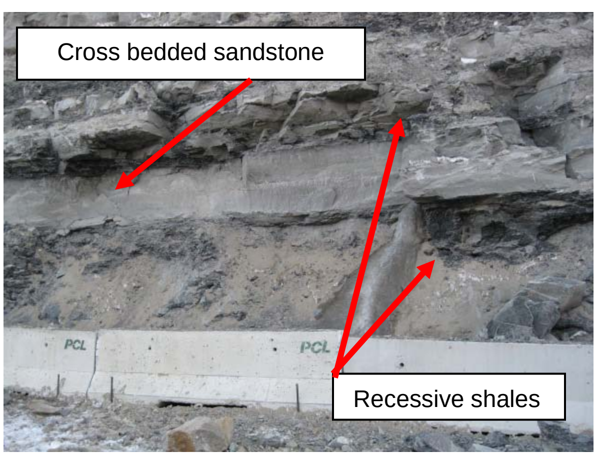


Figure 4: Typical Bedrock Vertical Composition at ATT Excavation (December 16, 2011)



*Note fracture orientation and spacing

The Calgary urban area bedrock sedimentary sequence of poorly consolidated interbedded mudstone, siltstone and sandstone layers has compressive strengths in the 5-40 MPa range.[1]

In general, the soft rock lithologies (poorly competent siltstones and shales/claystones) of the Paskapoo Formation exhibit lower strength parameters than the sandstones of the succession and therefore are considered a geological setting with a higher geotechnical hazard. Typical shear strength parameters of these soft rocks are summarized in Table 1.

Table 1: Typical Shear Strength Parameters of Fine Grained, Soft Sedimentary Rocks of Alberta (Locker, 1973)

Material	Peak		Residual	
	φ [deg]	C [kN/m^2]	φ_r [deg]	C_r [kN/m^2]
Siltstone	40	420	25	70
Clayey Siltstone	35	140	20	39
Bentonite	14	42	8	0

The low shear strength parameters of the ATT excavation soft rock stratum, and the wall failures recorded, warranted stabilization of the excavation wall and wall face stability monitoring during the backfilling operations.

5 CONSTRUCTION ISSUE ANALYSIS

5.1 Open Tunnel Excavation

The ATT excavation exposed unconsolidated till, siltstone/sandstone, and shale bedrock. Several small failures were recorded before and after excavation was completed. Failures were limited to shallow slides in the till, topples and rock falls, and sporadic slips within the bedrock. Due to the steep excavation slope, depth of the excavation, excavation wall rock properties and structures, and the narrow working areas between the rock faces and the tunnel concrete wall, the risk of injuries, equipment damages and/or construction shut downs was relatively high.

Observation of the uncontrolled rockfall incident sites suggests that the root cause is the alternating layers of the weak, rapidly weathered claystone/shale and hard, more competent siltstone and sandstone. Based on the observation and analysis, Terracon engineers interpreted the rockfall to be attributed primarily to the following processes:

A. As the side slope is cut and subject to various elements, the poorly indurated shale/claystone becomes fissured under active weathering and gravity transport, and eventually constitutes a progressive mass wasting (ravelling) process and contributes to small talus pile formation at the toe when the highly fissured shale/claystone, usually in beds of 1 cm or so, disintegrates;

B. When the shale/claystone ravels, the overlying rock is undercut or develops an overhang condition. Eventually, as the ravelling reaches a critical point or joint surface, the overlying rock cannot support its own weight as well as those above it and breaks away, causing a rockfall event.

At the time when the first rockfall was reported, the cut face was covered by plastic sheets, or tarps, to prevent the cut face from additional weathering, as recommended in the original geotechnical report.

Rock weathering, or strength degradation, generally involves physical processes such as wet-dry cycling, freeze-thaw cycling and possibly swelling pressures that can be created by the clay fraction in the shale/claystone. The use of tarps to prevent the shale/claystone from significant weathering was not successful although they did reduce erosion on the slopes during rain. Chemical weathering was also possible, albeit no tests were conducted in the geotechnical investigation to definitively support the possibility. Isolated groundwater seepages were observed on the slope cut faces and they also contributed to the accelerated strength degradation of the claystone strata.

During the site investigations, core drilling combined with seismic surveys were conducted exclusively. The core was logged for Rock Quality Designation (RQD) values. The RQD, which is an indicator parameter of the intact or fractured nature of the core runs ranged from 0-93%, with an average value of about 44%. Rock with this RQD generally is classified as being of poor quality. Rock strength was perceived to be important in the design of the side slope. The project plan was to monitor the excavation slope and institute stability measures as required based on observations. It was fully expected that mitigations would be needed once the rock was exposed.

The excavation method did not cause dynamic stress loading of the wall but did result in irregular cut faces with slightly variable face angles and local overhangs with 0.5-1 m relief and in one case 1.5 m of overhang. These structures combined with the joint system caused local rock failures that were eventually successfully controlled with a mesh and anchor system.

5.2 ATT Storm Water Lift Station Excavation

This excavation is situated at the west portal of the ATT structure. This excavation is approximately 15 m in depth below the tunnel floor. The stratigraphy/lithology is similar to the tunnel geology.

However a tension zone defined by a crack 15 m long set back 1.5- 2 m from the wall crest developed shortly after the excavation was completed. A stability analysis was completed and it was decided to bolt this tension zone.

6 MECHANICAL ROCK SLOPE STABILIZATION MEASURES

Terracon provided geotechnical construction monitoring expertise which concentrated on immediate rock wall mechanical stability mitigation measures and ongoing inspection of temporary slope stability of the ATT excavation and the lift station excavation.

Once the mechanical slope stability measures were installed as described in Section 6.1, it was expected that the construction activities (i.e. workers) would be able to continue working in this area. Weekly inspections and reports as described in Section 6.2 were submitted to PPD-JV that verified acceptable slope risks for the workers or, alternatively, recommended remedial measures to reduce the risk to allow workers into the excavations.

6.1 Mechanical Slope Stability Measures

In light of the tension zone which developed in the sandstones at the top of the West face of the excavation. Terracon completed a rock bolt design to stabilize this incipient failure zone. This mitigation measure was completed with the installation of approximately 25 bolts of 19 mm #6 Dywidag threadbase (CSA G30.18) cut to 5 m lengths, complete with flat face plates 250 mm square. This tension zone was stabilized and monitored as part of the monitoring effort of the storm water tank excavation. The bolts were placed in two horizontal rows on a 1 m by 1 m spacing, with the top row being 1 m below the crest.

Terracon provided recommendations for stabilizing the slopes for the ATT storm water lift station excavation slopes using a mesh-anchor system similar to that used for the ATT excavation.

The design parameters for the mesh-anchor system, including materials specifications, pre-installation rock cut surface stabilization procedures, mesh-anchor system installation requirements, and required routine post-installation inspections are described below. Setting the design specifications for the mechanical stabilization system recognized that large diameter cast in place concrete piles would be drilled and placed through the lower portion of the mechanically stabilized rock slopes.

Figure 5: Example of Rock Bolted Zone

6.1.1 Materials Associated with Mesh and Anchor Installation

The following wire mesh requirement and anchorage systems were utilized for the planned mesh-anchor system to be installed at the ATT site:

A. Triple twist Maccaferri gabion mesh (Type 8 by 10) with the minimum diameter of the mesh wire 0.12" or 3.05 mm.
B. Suspension cable and respective anchorage system along the crest recommended by BAT Construction Ltd., the installation contractor: 5/8" size cable; #8 Dywidag flush mount anchors spaced a maximum of 3.0 m, installed a minimum of 1.0 m into competent bedrock, grouted with epoxy resin-based glues. The anchors were positioned a minimum of 2.0 m from crest, or as close to the Jersey barriers as practicable.
C. Split Set type stabilizer system (Model SS-33; length of 762 mm or 30") and associated domed plate (6" by 6" by 0.16") used to anchor the mesh onto the rock cut surface. Typical pattern of slope surface anchorage is 3 by 3 m. A certain amount of field fitting to place the anchors was required to pin the mesh tightly to the rock profile.
D. Eight (8)" overlap of two adjacent wire mesh sheets, stitched with Spenax clips. The overlap strips were also anchored using Split Set stabilizers.

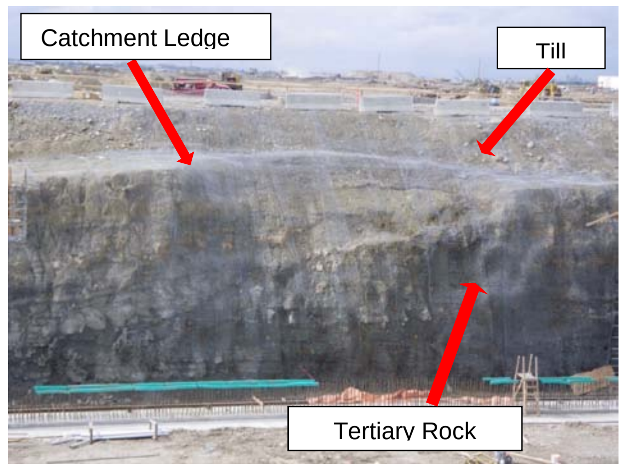
6.1.2 Mesh-anchor System Installation Requirements

A. Prior to mesh-anchor system installation, the slope cut face was scaled to a reasonably uniform face with limited protrusions so that no loose rocks existed. When necessary, the geotechnical engineer was consulted as appropriate to inspect, identify target areas and confirm scaling quality.
B. All prominent overhangs were removed, or otherwise stabilized, before scaling and mesh-anchor system installation.
C. Suspension cable anchors along the crest: The anchors securing the suspension cable were positioned a minimum of 2.0 m away from the crest, or as close to the Jersey barriers as practicable. Minimum embedment length of 1.0 m into competent bedrock was required.
D. Stabilizers anchoring the mesh to rock cut face:
   - The Split Set type stabilizers were used for securing the meshes on the slope cut face.
   - Split Set type stabilizer generally needs proper drill hole diameter to perform its design function. To determine the proper anchor drill hole diameter(s), pull test(s) were conducted to gather relevant information prior to actual anchor installation.
   - For installation quality verification, a minimum of one (1) pull test every 150 m^2 of slope rock cut face area and sufficient tests for all rock strength types was recommended.
   - The stabilizers were installed in a pattern of 3 by 3 m. Depending on rock quality at the anchor location as well as local geological conditions (e.g., overhang, weak rock, etc.), more stabilizers were needed to warrant integrity and proper functioning of the mesh-anchor system, i.e., field fitted as appropriate.
   - During mesh and anchor installation, the mesh was secured to the rock face as tightly and fittingly as practicable, with additional stabilizers used as required.
   - The mesh was anchored securely at the slope toe area with a maximum of three meter spacing and additional anchors as required.

6.1.3 Catchment Ledge Structure

Where there were tills and/or fill materials along and in the close proximity of the slope crest area, it was recommended that a catchment ledge on the bedrock surface approximately 2 m wide be constructed between the excavation crest and the toe of the piled fill materials to catch potential rolling loose rocks/soil chunks. To accomplish this specification, the fill material was pulled back to construct the catchment ledge structure.

Figure 6: Catchment Ledge Structure



6.5 Post-installation Inspection Requirements

After the mesh-anchor system installation was completed and prior to workers being dispatched to work at the toe areas, the following inspections and/or patrols were performed:
A. Shift inspection of the mesh/rock cut face by site personnel before workers start working at and around the slope toe area. Inspections focused on signs of rock cut face movement, signs of newly formed loose rocks, mesh-anchor system integrity, evidence of increased seepage, or other items of significance. Written logs of the shift patrols were maintained.
B. It was required that a geotechnical engineer perform weekly rock cut face integrity inspections while the excavation was opened.
C. More frequent inspection of the mesh/rock cut face, both by site personnel and a geotechnical engineer, were warranted following severe climate events such as rapid snow melt, ice-jacking induced by freeze-thaw, heavy rainfall, rapid water runoff, etc.
D. A weekly report was developed from these inspections and submitted to the PPD-JV that provided additional observations and recommendations for remedial stability mitigative

measures if needed, due to imminent danger or an assessment indicating that the area was opened for construction activities to continue.

7 HEALTH, SAFETY, AND ENVIRONMENTAL CONTROLS

A system of risk reduction protocols was implemented to minimize hazards attributed to the rock slopes of the open cut tunnel excavation and the slopes of the storm water storage and control excavation at the north-west end of the open cut tunnel. These being open cut structures through a thin horizon of glacial till and the underlying Tertiary sandstones and shales, the key ground control hazards were the high relief (15-19 m), steep slopes (70^o) and the uncontrolled rock movement from the till and rock slope faces. The original rock slope designs were based on a geotechnical site investigation. As per the project plan, once the rock was exposed, mitigative solutions were identified, evaluated, and implemented including mechanical ground control and catchment ledges.

In addition, several HS&E controls were put into effect during the early stages of rock excavation after the first rock-fall event that occurred in November 2011, after normal shift hours. Fortunately, this event caused no injury or damage.

The implemented HS&E controls included:

A. To mitigate rainfall and snow melt run-off, the daylighted excavation crest/perimeter was shielded with a continuous modest 0.5 m high drainage berm of the local boulder clay till on both sides of the excavation. This mitigation measure was cost effective in preventing the rock cut slopes from being temporarily saturated and it reduced the freeze-thaw cycle damage to the slopes. These drainage berms also helped mitigate flooding of the excavation.
B. Temporary groundwater was controlled through construction of a temporary ditch/sump collection system which kept the excavation fully drained.
C. A ground control Quality Management System focused on Safe Work Procedures and Job Hazard Analyses;
D. Appropriate signage around the excavations and at ramp access routes into the tunnel;
E. Barriers along the rock excavation crests and barriers set back from the rock slope toes;
F. Visual monitoring of the rock slopes on a continuing basis during erection of the tunnel formwork for footings and walls; see Section 6.2 for details
G. Several forms of mitigative mechanical support of the open cut tunnel rock slopes, (pin and mesh, rock bolts and shotcrete, were employed to stabilize the slopes). See Section 6.1 for details;
H. Periodic inspection of the excavation rock slopes and renewal of the pin and mesh protective slope screens when screen ballooning from movement of rock developed;
I. Daily ground control advisory notes were posted on the shift entry sign-off documents as reminders for each contractor on each shift.

These controls eliminated the hazards associated with uncontrolled ground movements through the remainder of the project. That is, those rock-fall events that did occur were small and all were safely contained by the foregoing controls. The progress and schedule of the project, subsequent to November 2011, were unaffected by ground control conditions.

8 CONCLUSION

The project plan was to monitor the excavation slope and institute stability measures as required based on observations and demonstrated rock slope behaviour. It was fully expected that mitigations would be needed once the rock was exposed. Several recommendations to consider when planning and executing projects of a similar nature can be concluded from this successful project:

Site investigations for soft rock excavations must include oriented fracture analysis and characterization which can be conducted by mapping trenches or from oriented core. Weathering behavior of freshly exposed soft rocks, while difficult to predict, merit consideration through the planning process.

The resistance of the rocks, particularly of the weaker rocks such as claystone, to weathering effects could be investigated and quantified using an established laboratory test procedure (slake durability).

When excavated rock slopes are adjacent to construction activity, worker exposure must be mitigated. In this geological setting, a combination of a mesh anchor system, rock bolting, catchment berms or ledges, frequent slope inspections and daily alert bulletins provided adequate worker protection for the duration of the project.

The use of engineered and monitored slopes provided a solution that yielded the required level of safety and was cost and schedule effective.

In an adjacent much shorter overpass excavation on the airport site composed entirely of soft shale and mudstone, shotcrete was successfully applied to prevent weathering and erosion to stabilize those slopes

ACKNOWLEDGEMENTS

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