Remediation of the Lube Land Slope Instability at Highland Valley Copper Mine

Sebastien Fortin & Carl Diederichs
Teck Highland Valley Copper Partnership, Logan Lake, B.C., Canada
Rod Kostaschuk
AMEC Earth & Environmental, Calgary, A.B., Canada
Ganan Gananathan
AMEC Earth & Environmental, Burnaby, B.C., Canada

ABSTRACT
The presence of lacustrine sediments within the overburden sequence of the East Wall of the Valley pit creates challenging ground conditions for pit slope design and mine operations at Highland Valley Copper. The Lube Land area of the East Wall experienced a relatively large slope failure in June 2005, which day-lighted pre-sheared lacustrine clays and silts with low residual strengths. Movement rates showed a gradually decreasing trend over the following months. However, another large portion of Lube Land slope failed in 2008 in response to localized mining activities and precipitation, resulting in a disruption of short-term mine plans. This paper presents an overview of remedial measures completed to successfully stabilize the Lube Land slope.

RÉSUMÉ
La présence de sédiments lacustres au sein des dépôts glaciers du mur Est de la fosse ouverte Valley produit des conditions de pentes difficiles pour les opérations minières dans cette portion de la fosse à la mine Highland Valley Copper. Une rupture de pente considérable est survenue en Juin 2005 dans la zone ``Lube Land ´du mur Est, alors que des silts et argiles lacustres à faible force résiduelle furent exposes. Le taux de déplacement de la pente montra un déclin graduel au cours des mois suivants. Une autre rupture de large envergure, survenu en Juin 2008, est attribuée aux activités minières de même qu'à de fortes pluies, ce qui perturba les plans miniers à court terme. Cet article présente un aperçu des mesures correctives complétées pour stabiliser la pente Lube Land.

1 INTRODUCTION

The Highland Valley Copper (HVC) mine site, currently owned and operated by Teck, is located in South Central British Columbia approximately 70km southwest of the city of Kamloops (Figure 1). It is a large surface mine that presently exploits a porphyry copper and molybdenum deposit from three active open pits, namely the Valley, Lornex and Highmont pits. The Valley pit is the largest and deepest of the three pits with maximum slope heights of approximately 700m. The mine life of this pit has recently been extended beyond 2019 and consequently the Valley pit is undergoing a pushback stage.

Lube Land is an in-pit fueling and lubing station used by heavy haul trucks due to its proximity to a critical haul road intersection. Approximately 75m vertical below Lube Land, an area referred to as the Lube Land slope, is the junction between the Jurassic ramp leading to the Jurassic and I-9 waste dumps and the East wall main ramp going to in-pit crushers #4-5 (Figure 2). It is thus a critical area for heavy hauler traffic.

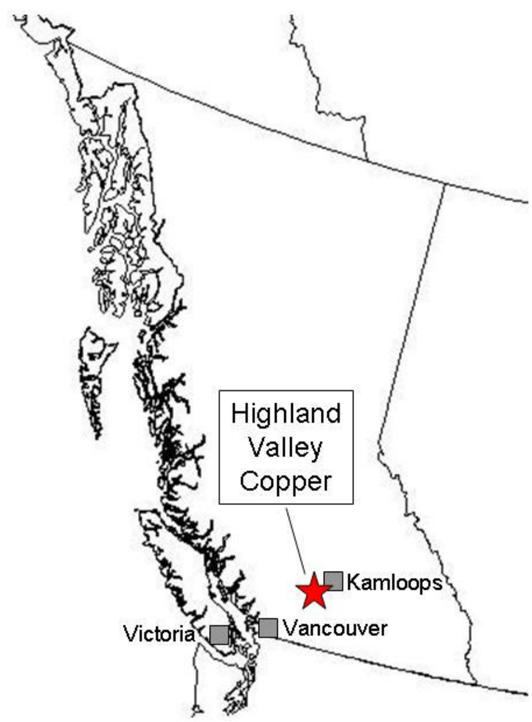

Figure 1. Location of Highland Valley Copper Mine site

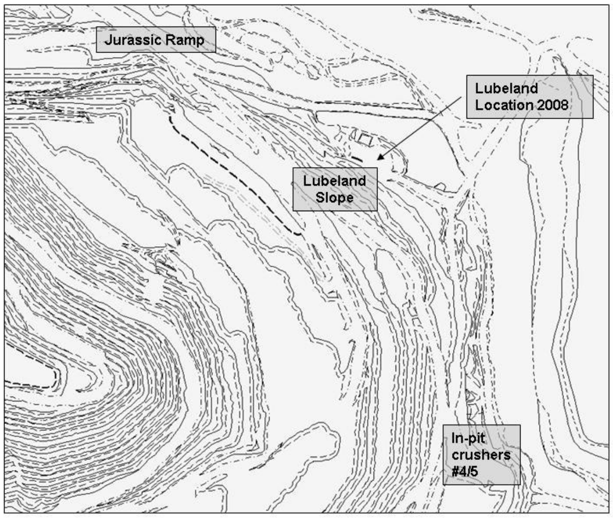

Figure 2. Map of Valley Pit East Wall

2 LUBE LAND SLOPE INSTABILITY

2.1 Geologic Conditions and Related Background

As the name implies, the Valley pit intersects the North-South trending valley of the Thompson-Nicola plateau of Central British Columbia. The eastern half of the Valley has been in-filled by sediments during the "period" (Golder, 2004). These depositional events resulted in a complex sedimentary sequence over 300m thick near Lube Land, comprising glacial tills, glaciofluvial sands and gravels, as well as interbedded and/or laminated lacustrine silts and clays.

The 10A Unit lacustrine sediments constitute the portion of the East wall found between approximately El. 1070m and 1140m. Subsequent to the 2008 Lube Land failure described below, some thin (300mm) but continuous, very stiff to hard, varved clayey silt layers were observed within the 10A unit, more specifically within the range of El. 1120m to 1128m, but locally found up to 1135m and down to 1108m. These clayey silt layers appear to have experienced slope movements as slickensides were clearly visible both in the field and in core samples. It is also possible these sediments were pre-sheared as a result of glacial trusting in the valley. Unit 10A is underlain by lacustrine clay Unit 10B sediments, which exhibit very low hydraulic conductivities.

The pushback work includes the installation of vacuum-assist groundwater depressurization wells and the eventual construction of a buttress to address weak clay layers in the 10B horizon located below elevation 1070m. At the time of the Lube Land failure, well construction was in progress. Fortunately, the wells were collared near the back of the 1190m bench and were not impacted by the Lube Land failure.

2.2 2005 Lube Land Slope Failure

The first Lube Land slope failure occurred on June 30, 2005 after mining operations reached a critical stage on the East wall and removed toe support near the El. 1115m bench. This slope failure resulted in the loss of the haul road to # 4-5 crushers as approximately 1.6MT of overburden material were mobilized. No personal injuries or equipment damages were incurred by this event as precursor cracks were detected near the crest of the 1190m bench a few hours before the failure and the area had consequently been evacuated. A new haul road was constructed by re-grading the disturbed material of the Lube Land slope. According to verbal accounts from shovel operators, the slope was "advancing towards the shovel" for a few days following the event. A clearly defined sliding plane was identified by a geotechnical engineer in the 10A silts at a few locations and at an approximate elevation of 1125m. A steep back scarp (~65^{o} below horizontal) developed below the crest of the 1190m bench during the slope failure, which was determined to be of the translational block sliding type. This incident resulted in disruption of mining activities for a period of approximately 3 weeks as well as in the loss of some surface dewatering infrastructure. However, the overall financial impact of this instability was minimal for HVC. Following the 2005 event, the Lube Land slope was monitored with slope monitoring prisms (SMP's). The area remained relatively quiet, movement-wise, until June 30, 2008.

2.3 2005 Lube Land Slope Failure – Back-Analysis

A back-analysis of the 2005 Lube Land slope failure was carried out with limited geotechnical information back in 2005. The data available at the time consisted of:

- Presence of tension cracks at the 1190m elevation;
- Groundwater elevation from one vibrating-wire piezometer installed post-failure (DP-78);
- Field observations indicating a translational slide along a "weak" clayey layer at ~El. 1125m;
- Topography of the Lube Land slope from the pit map, which is updated weekly.

Using the information above, an angle of shearing resistance of 9.5^{o} was back-calculated for the "weak" layer found within the 10A silts.

2.4 2008 Lube Land Slope Failure

June 30, 2008: A thin crack, approximately 5m in length and parallel to the crest of the 1190m bench was reported to HVC's geotechnical group by a pit foreman at about 7:30AM (Figure 3). The crack was located between two SMP's which did not record any movement. The area of the crack was inspected regularly during that morning by a geotechnical engineer. By 1PM, the crack had dilated to approximately 30mm wide and opened over a distance of about 15m; it had also propagated to multiple sub-parallel cracks retreating approximately 5m away from the 1190m bench crest. A few hairline cracks were also observed on the El. 1130m bench below the Lube Land slope that were nearly perpendicular to the cracks on the 1190m bench. Single-lane traffic was instigated immediately and a safety berm was constructed to divert the traffic away from the crest.

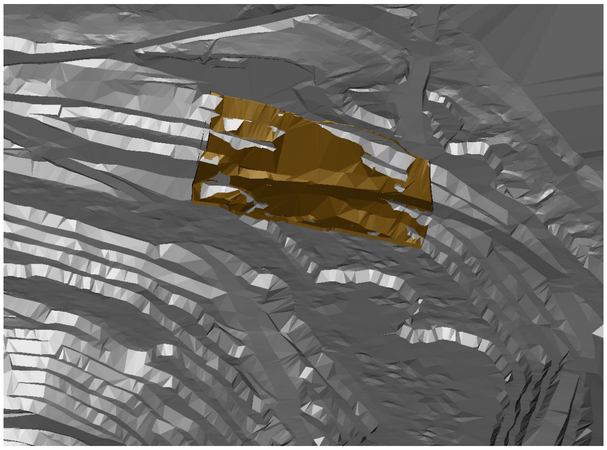

Figure 3. Area exhibiting cracking and heaving between the 1190m and 1115m benches (June 30, 2008).

By about 3PM the slope showed increased signs of strain and cracks were multiplying and widening out in the crest area. A family of sub-parallel cracks had also developed on the 1130m bench, along with the creation of a slight bulge quasi perpendicular to the crest alignment. Access to Lube Land as well as traffic on the haul road to #4-5 crusher was blocked by 4PM and light plants were mobilized in the crest area for night shift monitoring. The only SMP's available at the time were located in the northern portion of Lube Land and did not show any signs of movement acceleration at the time.

The southern portion of the Lube Land slope failed at around 3AM on July 1, 2008 (Lube Land South, Figure 4). This slope failure mobilized approximately 2.6MT of overburden soils and cut access to the Lube Land fuel station, and the height of the head scarp below the 1190m bench was approximately 10m (Figure 4) The haul road to #4-5 crusher sheared off during the event, showing vertical displacement of up to 2.5m (Figure 4). The 1130m bench below the haul road showed two sub-parallel pressure ridges bulging up to 4m above bench grade (Figure 4).

Figure 4. Looking down from the headscarp of the Lube Land failure (June 30, 2008). Note the high head scarp, smaller scarps across the haul road, and the wide 1115m bench occupied by the mining equipment.

The 2008 Lube Land South instability had a somewhat more serious impact on the short-term mine plan than the 2005 event. The impact was more serious because the 2008 mine plan was sequenced to release the benches required for the installation of a depressurization system related to the East Wall expansion. The earthworks associated with remediation of this instability initially disrupted the drilling sequence of the wells until a new plan for mining and installation of the depressurization system was formulated. The other indirect impact (discussed below) was the need to switch mining production to generate waste rock for buttress construction, as the Phase 5 South wall mining in progress at the time of the failure was intended to yield ore rock for the latter part of 2008. Therefore, an alternate source of waste rock had to be developed.

The following sections present a summary of the investigation, analysis, design and construction of the remedial measures for the 2008 South Lube Land slope failure.

2.5 Groundwater Conditions

At the time of the 2008 failure, limited groundwater pore pressure information was available. Piezometer DP78 (installed after the 2005 event) is located on the 1190m bench to the northwest of the Lube Land slide. Water level data from DP78 showed the groundwater elevation at ~1133.5m at the time of the failure. Field mapping identified seepage faces at elevations ranging from 1122m to 1125m. These two points were used to develop the groundwater surface used for the back-analysis model of the failure. Vibrating wire piezometers were installed after the failure during the geotechnical investigation phase described below. These piezometers allowed for "fine-tuning" of the piezometric surface used in the back-analysis modeling exercise and also allowed monitoring of groundwater pressures in response to the dewatering efforts described below.

A few piping holes up to 2m in diameter developed around El. 1133m suggesting locally high groundwater gradients across the Lube Land slope. Once developed, these holes would show relatively high initial flow rates (up to 6 L/s) and typically tapered out to low residual flow rates. Groundwater generally flows in a south-westerly direction with flows converging towards a topographic a low point near the base of Unit 10A. Perched water tables were frequently encountered along the Lube Land slope during mining, which effectively reduced pore water pressures in the slope. A surge of ground flow was typically associated with purging perched water pockets through material consolidation and slope relaxation as a result of mining.

2.6 Geotechnical Investigation

Following the Lube Land failure, geotechnical boreholes were drilled throughout the slide area to further evaluate the location of the failure surface, assess pore water pressure conditions, and to monitor on-going subsurface displacements. A map that indicates borehole locations is shown in Figure 5. Five boreholes were advanced by Mud Bay Drilling Ltd. using a truck-mounted SR-074B sonic drill rig. Detailed geotechnical logs were developed for each borehole, and samples were collected for material index testing.

Vibrating wire piezometers were installed in the boreholes to monitor pore water pressures in the slide area. Based on core logging, piezometer tips were located at inferred slip surface elevations. A total of twelve piezometers were installed. These piezometers allowed for fine-tuning of the piezometric surface used in the modeling exercise. The new piezometers also allowed monitoring of ground water levels to ensure that adequate depressurization was completed as per AMEC's remediation plan.

To confirm the location of the failure plane, inclinometers were installed in the five boreholes. Unfortunately, due to the rapid rate at which shearing was occurring, the inclinometers had short life spans, but did allow for confirmation of the elevation at which shearing was occurring in the Lube Land slide mass.

In addition to drilling, four test pits were excavated near the toe of the Lube Land failure on the 1115 meter bench, and exposed a thin medium to high plastic silty clay layer present within the Unit 10A glaciolacustrine sediments. These silty clay layers were found approximately between elevations 1010 and 1114 meters (hereinafter referred to as the lower weak layer). Displacement and slickensides were also observed on thin layers of similar material higher on the slope, at approximate elevations 1126 and 1128 meters (hereinafter referred to as the upper weak layer). Based on these observations, the base of the failure mass was interpreted to be along one or both of these zones within thin but continuous sub-horizontal layers of medium to high plastic silty clay material.

Samples of these weak silty clay layers from the lower layer (near elevation 1114m) were tested for index properties. Several direct shear tests were also carried out to evaluate the residual friction angle of these layers. Based on the direct shear test results and previous studies of the 2005 instability, a residual friction angle of 12 degrees was assigned with no cohesion for all assumed weak layers in the stability models.

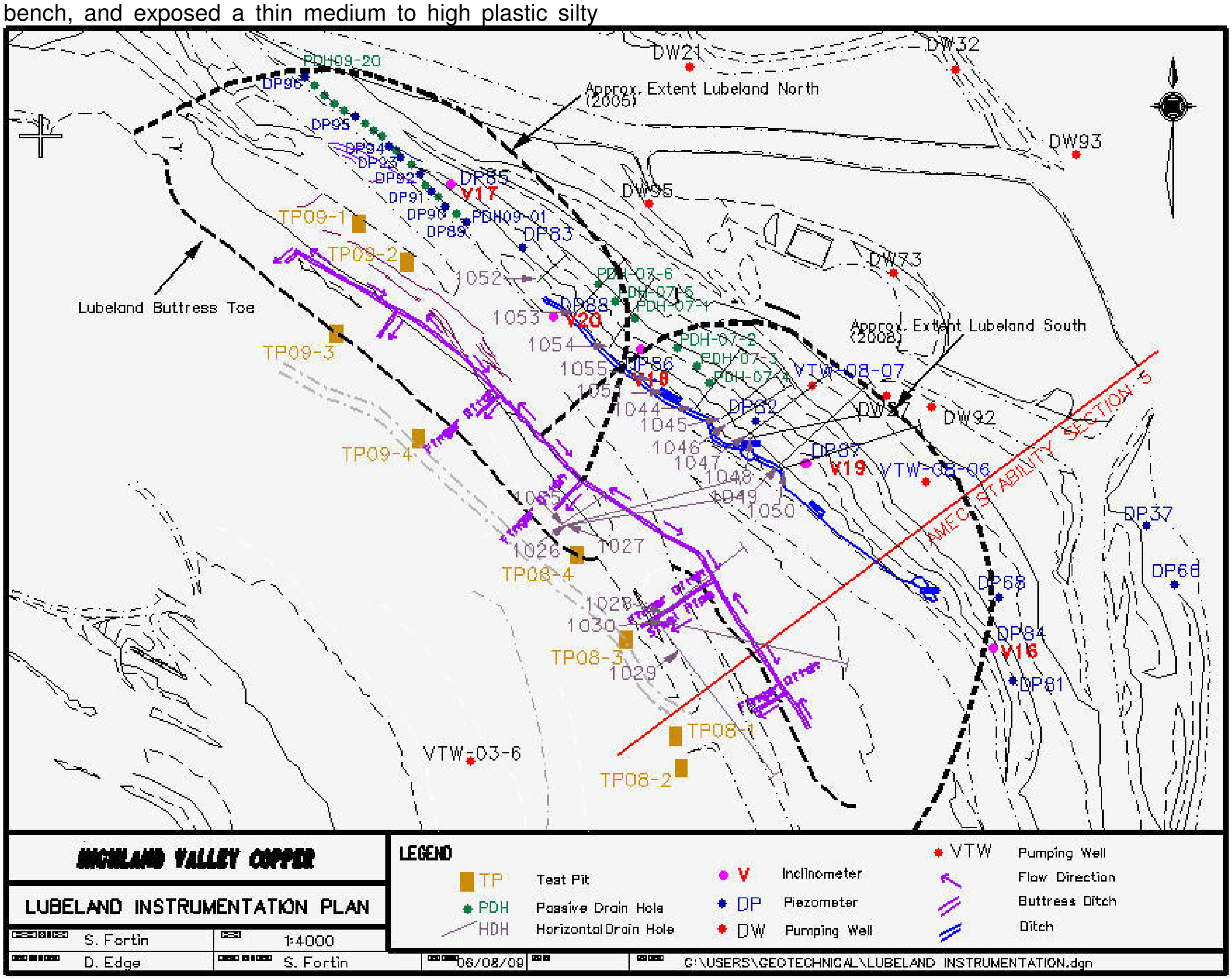

Figure 5: Plan view of the Lube Land area showing geotechnical and dewatering efforts.

2.7 2008 Lube Land Slope Failure – Back-Analysis

Bench heights at HVC are 15m. The pit slope failure at the south Lube Land area occurred on June 30, 2008, and extended across 4 to 5 benches from the 1190 m bench down to the 1130m and 1115m benches. The information gathered from site observations and survey data showing the limits of the failure, main headscarps and horizontal displacement vectors is shown in Figure 5. The information suggests that a small circular failure occurred between the 1130m and 1115m benches centered near Section line 2.

This small failure occurred where the mining shovel was actively excavating, and appears to have removed the confinement and triggered movement of a much larger slide mass. A 2-meter high toe heave was noted on the 1130meter bench near Section 5, and the head scarp was up to approximately 10 meters in height. Strong seepage was also observed from approximate elevation 1122m in this area. Horizontal displacement vectors (derived from pre and post-failure survey data) for the large slide mass are notably longer at the northwest end of the slide mass near the small failure than they are at the southeast end of the large slide, indicating there was a significant rotational component to the movement of the large slide mass. It is interpreted that this rotational component of movement resulted in the toe heave and cracking on the 1130 m bench that is quasi perpendicular to the main headscarp on the 1190m bench.

The back analyses of the Lube Land slope failure were carried out at two cross sections (Sections 2 and 5) that were judged to be most critical because of steep pre-failure topography. It was observed in the field that the head scarp of the failed surface was near vertical and approximately 10m high, which indicates possible tension cracks at the crest of the pre-failure slope. Therefore, in the back analyses, 10m deep tension cracks were assigned at the crest of the slope to replicate the pre-failure conditions. The water table was assigned based on the observations of seepage from the slope and monitoring well data. The water table location and the stability analyses were updated in the models as data from the post-failure investigations and piezometers became available. The material properties used in the analyses are summarized in Table 1 and were largely based on previous laboratory testing.

Table 1. Overburden Material Properties

Material (from top to bottom)	Unit Weight (kN/m^3)	Friction Angle (°)	Cohesion (kPa)
Till & Upper Aquifer	22	36	0
Main Aquifer	22	36	0
Unit 10A	20	33	30
Unit 10A – Weak Layer	20	12	0
Unit 10B	20	12	0
Unit 10C	22	33	0
Basal Aquifer	22	42	0
Buttress Material (Sand)	22	33	0

One of the direct shear tests indicated that the residual friction angle of the lower weak layer may be as high as 26 degrees whereas earlier tests on the upper weak layer yielded 12 degrees. Therefore, an additional back analysis was completed to evaluate an alternative geologic interpretation, wherein the lower weak layer is stronger than the upper. To drive the back-analysis FoS to unity for failure though the upper 12 degree material required somewhat different assumptions on pre-failure conditions, including having the weak layers dip down slightly towards the pit wall, and assuming higher water table prior to failure. These analyses indicated that a combination of slightly dipping weak layer and high piezometric surface may also have caused the slope failure through the upper weak layer present within Unit 10A. The monitoring well data and site observations of seepage emerging from the pit slope suggest that the higher piezometric surfaces were unlikely to be present at the time of slope failure. However, the proposed remedial measures were also checked against the scenario of having high piezometric surfaces with the dipping weak layers

The results of the back analyses of Sections 2 and 5 are shown in Figures 6 and 7 respectively. The estimated Factor of Safety at both sections was about 0.98 for failures extending down to the hypothesized weak layer between elevations 1110m and 1114m, which concurred with the slope failure that occurred on June 30, 2008. The FoS for slip surfaces passing through the upper weak layer between elevations 1026m and 1028m is higher, at approximately 1.08. These results indicate that if the weak layers are of equal strength (12 degrees), then the lower-most one is the critical surface for stability. Therefore, it was recommended that remedial measures be designed to provide protection against instability controlled by either the shallow or deep weak layers.

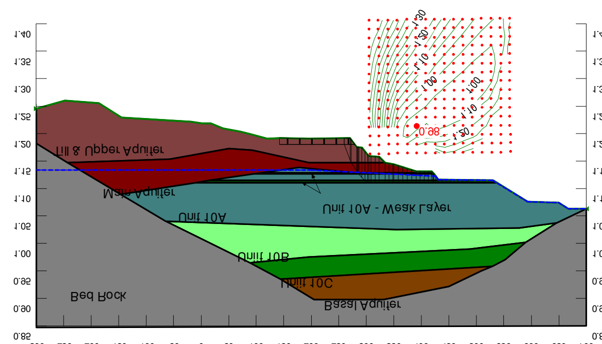

Figure 6. Section 2: Stability Back Analyses (Failure through lower Unit 10A weak layer).

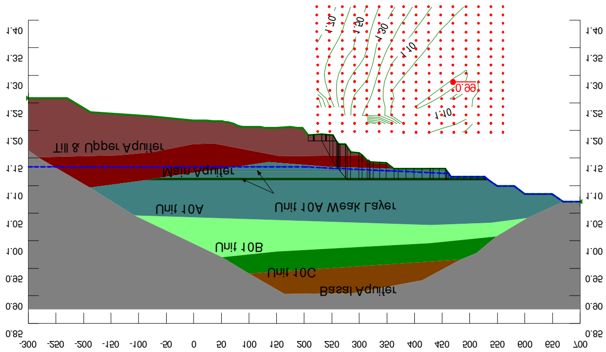

Figure 7. Section 5: Stability Back Analyses (Failure through lower Unit 10A weak layer).

2.8 Predictive Analyses for Design of Remedial Measures

In order to stabilize the slope, three types of remedial measures were investigated as follows:

1. Flatten the slope above the 1115m bench by pushing the 1190m crest back by 45m, 60m or 75m, and also pushing back lower benches by lesser amounts to attain a flatter, uniform overall slope angle.
2. Lower the water table through dewatering methods.
3. Add a buttress on the 1115m bench at the toe of the failure.

At the time of the failure, the 1115m bench had not been pushed back to the final pit slope associated with the mine expansion and was therefore very wide as shown on Figures 3 through 7. The benches above 1130m had already been pushed back in this area. Pushback of the 1130m bench was in progress at the time of the failure, which is in fact what triggered the failure. A critical design criterion for the remedial measures was to ensure that the target factor of safety would be achieved for the long term slope configuration after completion of the push back. Modification of the configuration of the final pit slope below elevation 1115m was not an option.

The target long term Factor of Safety was approximately 1.2 using residual strength in the weak clay layers; this value is HVC's design criterion for all pit slopes excavated through overburden materials. This value was selected based on perceived acceptable risk pertaining to mining equipment/infrastructure and is consistent with previous design work completed for the Unit 10B depressurization and buttressing (Golder Associates, 2004 and 2005) for the East wall expansion. Because all remedial options required some push-back of the top of the slope (at elevation 1190m) into undisturbed soil, it was assumed there were no tension cracks for potential slip surfaces with head scarps at this elevation.

For the buttress option, preliminary analyses indicated that a buttress placed on the 1115m bench would not result in a significant increase in the factor of safety, because the critical slip surface would follow the weak layers below 1115m elevation and pass beneath the buttress fill. The addition of a shear key below the buttress that truncated the lowest weak layers was clearly necessary. Therefore, excavation of a shear key trench from the 1115m bench located beneath the buttress fill and down to an elevation no higher than 1110m was recommended.

The results of the slope stability analyses indicated that none of the 3 types of remedial measures alone would increase the factor of safety to the target value. All three types of remedial measures were required to achieve the target, and the sequence of implementation of these measures was also critical, to ensure that short-term factors of safety during implementation of remedial measures were acceptable. For short term stability, it was considered acceptable for factors of safety for single bench failures at the excavation face to drop below 1.0, however factors of safety for multi-bench failures had to remain above 1.0. Single bench failures could be managed by the mining equipment, but multi-bench failures could not.

Based on a series of slope stability analyses, the following summarizes the steps, in chronological order, that were recommended to remediate the pit slope area affected by the June 30, 2008 Lube Land slope failure.

1. Flatten the overall slope to approximately 2.6H:1V by pushing the 1190m crest back by 45m, and also pushing back lower benches by lesser amounts to attain this uniform overall slope angle. Excavate from the top down (i.e. 1190 m bench, followed by 1175 bench, followed by 1160 bench, etc.)
2. Prior to or concurrent with step 1, construct a small buttress of clean sand and gravel material on the 1115 m bench to support the 1130m bench. This buttress should be 7m high and 20m wide, and would be placed directly against the existing slope without any excavation. Steps 1 and 2 result in a temporary FoS slightly above 1.1 without having to implement dewatering.
3. Implement dewatering measures (such as passive or active dewatering wells) that lower the piezometric elevation to 1115m or lower for a distance of 100 meters into the slope from the back of the 1115m bench on the ultimate pit slope. Install piezometers to confirm that the piezometric surface has been lowered to 1115m or lower before proceeding to the next step.
4. Excavate the 1115m bench back to the ultimate pit wall location. The Factor of Safety for this condition is approx. 1.17.
5. Using slot-mining techniques with maximum slot length of 30 meters, excavate the shear key to truncate the weak layers encountered in the 1110m to 1114m elevation range, and immediately backfill the shear key and construct the buttress using clean sand and gravel, placed in 600mm lifts and compacted with truck and dozer traffic. Inspect the excavation to confirm that the shear key excavation has indeed truncated all the weak layers. Only move on to mining the next slot when the buttress for the first slot is complete. The result of this final step is to increase the long term FoS to approximately 1.25 (See Figures 8 and 9).

Additional stability analyses were also completed wherein the above proposed remedial measures were applied to the alternative geological section described earlier, with weak layers in the 1125m to 1128m range. The Factor of Safety for the remediated slope was similar for failures through the upper weak layer, and much higher for failures through the lower weak layer. These check runs indicated that the proposed remedial measures would address long term stability equally well for the two geological interpretations modeled.

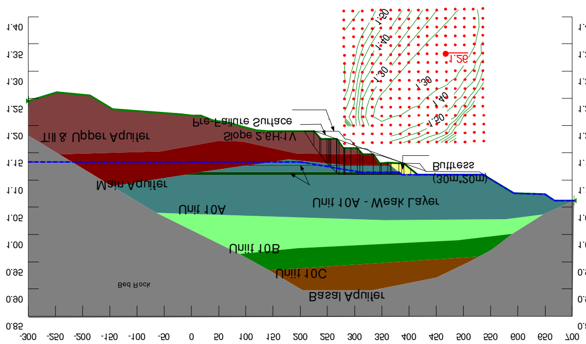

Figure 8. Remedial Option: Section 2 of the Lube Land slope, showing crest unload, rock buttress, and target groundwater depressurization. (AMEC, 2008)

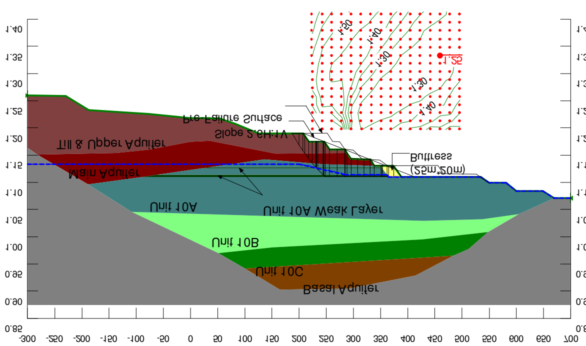

Figure 9. Remedial Option: Section 5 of the Lube Land slope, showing crest unload, rock buttress, and target groundwater depressurization. (AMEC, 2008)

2.9 Lube Land Slope Remediation Work

A remediation plan was rapidly developed following the June 30, 2008 Lube Land failure by AMEC Earth and Environmental (AMEC). The plan had to minimize production downtime to Phase 5 mining activities, improve the long-term stability of the Lube Land area to the target factor of safety, and avoid any impact to the 10B buttress design and associated GAIA depressurization wells. As described above, the optimized remediation plan achieved the 1.2 target FoS through three separate components; unloading of the crest and flattening the overall slope angle above the 1115m bench, installation of ground depressurization measures and finally reinforcement of the Lube Land slide mass with a rock buttress and key trench at the toe.

2.10 Crest Unload & Slope Flattening

The parking area located just South of the Lube Land was mined out, which effectively removed approximately 350,000 T off the crest of the Lube Land slope. In addition, a 45m push-back of the 1190m crest, and the associated reshaping of the 1160m and 1175m benches in order to achieve a uniform overall slope angle of 2.6H:1V was completed in October 2008.

2.11 Dewatering & Depressurization

Six active pumping wells were installed within the 1100m to 1190m elevation range to reduce pore pressures in the thin silty-clay beds within the Lube Land slope. Three conventional dewatering wells and three vacuum-assisted wells were installed with flow rates ranging between 0.3 and 1.3 L/s.

In addition to active pumping wells, passive dewatering methods were used. Seven horizontal drain holes (HDH) were drilled from ~1130m elevation between 50 and 100 meters into the slope. One HDH was dry, while all other holes produced initial flows up to approximately 5 L/s.

Twenty vertical drains (rock columns) were also drilled at 10m spacing along the Jurassic haul ramp near the base of the Lube Land slide. These drains were completed by drilling 600mm diameter boreholes to between 24m and 40m depth using a Soilmec SR-30 hydraulic auger rig. The boreholes were then backfilled with a clean crush product ranging from 75mm to 225mm in diameter. Every second rock column was instrumented with a vibrating-wire piezometer. Since completion of the relief wells on March 14, 2009, up to 4.3m drop in piezometric elevation has been recorded to date.

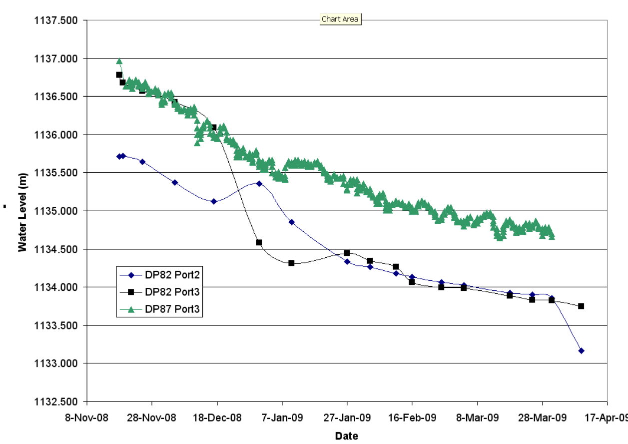

Figure 10: Time trends of selected piezometers showing positive response to horizontal drain holes.

The dewatering effort generally achieved the piezometric elevation targets used for buttress design, with the exception of a localized 30m wide area in the north, where pressures remained 3m to 4m above target values (Figure 10). Horizontal and vertical passive drain holes proved to be the most cost-effective method at lowering pore pressures from the silty 10A soils.

2.12 Buttress Construction

Construction of the Lube Land buttress began October 1, 2008. The buttress was constructed in thirteen separate cuts using slot mining to minimize the strike length of the weak zones exposed. The procedure for each slot was as follows:

1. Material from the 1115m bench to the 1130m bench was excavated flat at 1115m elevation using a Bucyrus 495HR shovel (Figures 11 and 12).
2. Once the flat pass at 1115m elevation was complete, either the shovel or a loader then completed a second pass to sink to the 1110m elevation to form the shear key under the buttress footprint (Figure 13).
3. Once the shear key excavation was complete, a deeper drainage ditch was excavated and backfilled with crushed rock (Figure 13). The drainage ditch was daylighted to the pit slope to allow gravity drainage.
4. The waste rock buttress was then dumped in from the 1130m bench to fill the shear key and construct the buttress fill. In total, approximately 1.62 million tones of blasted run-of-mine waste rock was placed in the buttress.

The above procedure followed the recommended steps resulting from the slope stability analyses, except that the small temporary buttress (Step 2 above) was omitted based on risk assessment: an increased temporary risk of single bench failures was accepted by HVC. Also, uncompacted, angular waste rock was used instead of compacted sand for the buttress fill to achieve similar shear strength to the sand without the need for compaction, and the deeper drain trench below the shear key was added by HVC. These relatively minor changes proved successful and reduced the effort and cost for the remedial work significantly.

Figure 11. Bucyrus 495R Shovel excavating block 6.

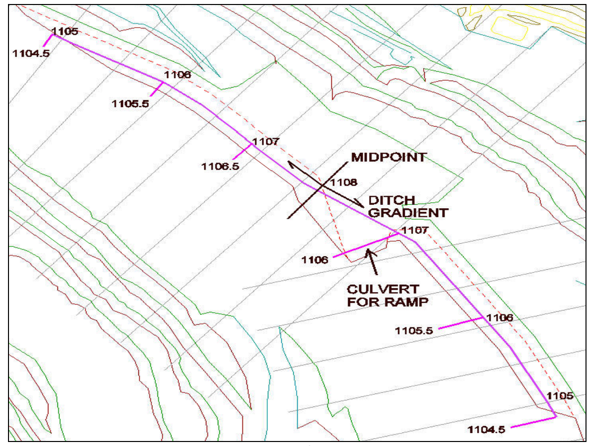

Figure 12. Layout plan of buttress mining blocks and underlying drainage ditch system.

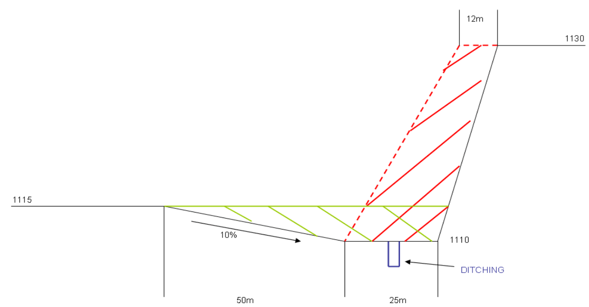

Figure 13: Schematic cross-section of conceptual Lube Land buttress design. French drain acts as shear key, de-coupling lower slickensided horizon.

2.13 Surface Water Control

Surface water was diverted away from the crest zone of the Lube Land slope immediately upon first observations of tension cracks. This practice was subsequently implemented as standard operating procedure. A water arch (loading station) located approximately 100m behind the back scarp was decommissioned within days of the failure event.

2.14 Slope and Buttress Performance Monitoring During Construction

On-going surface displacements were measured by establishing a network of slope monitoring prisms (SMP's) that were monitored using automated survey equipment (Newcomen et al., 2003). Prisms were surveyed on a four-hour rotation, providing movement data six times during a 24-hour period. Displacements measured from SMP's were used to monitor slope stability during slot mining. Prism data clearly showed an acceleration of movement as each slot was excavated, with a corresponding deceleration as the rock buttress was dumped in (Figure 14).

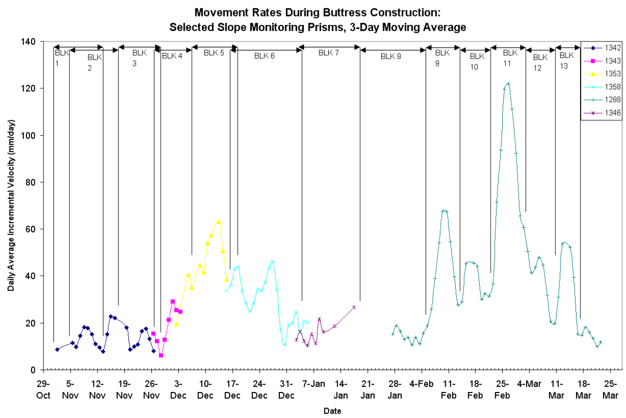

Figure 14: Slope monitoring prism velocity vs. slot mining showing acceleration and deceleration of the slope as material was excavated and the rock buttress placed.

2.15 Conclusion

The Lube Land stabilization buttress was completed in early April 2009: over 5 months after construction start and 10 months after the slope failure event. Movement rates have significantly decreased and observable deformation is minimal, especially in South Lube Land. The active and passive well system has generally lowered pore pressures in the slope below or close to design target values, although a pressurized zone remains in North Lube Land. At the time of writing this paper, another weak layer of pre-sheared silts from the 10A unit with a slight pit-ward dip has been day-lighted at El. 1108m in North Lube Land, which is below the base of the current buttress. This occurrence required an extension to the North Lube Land buttress, now extending down to El. 1100m in this area.

The Lube Land slope experience demonstrates that thin, locally continuous horizons of weak lacustrine sediments can have a significant influence on global slope stability as well as on mining operations. Prior to the remediation work, the Lube Land slope appeared to be in a meta-stable state and highly susceptible to mining activity near its base. Both instability events also proved the weak layer is very sensitive to increases in pore pressure induced by rainfall. Both events occurred at the end of June, which is the wettest month of the year at the mine site.

The East Wall of the Valley pit roughly coincides with the bottom of the North-South trending Highland valley and thus presents a complex geological arrangement of fluvial, glacial and lacustrine sediments and perched water tables. To support the East wall push back, further geotechnical investigation is required to identify the presence of other potentially weak zones that may occur at lower elevations within the 10A silt unit below the shear key.

ACKNOWLEDGEMENTS

The authors gratefully acknowledge Frank Amon and Peter Witt for allowing publication of this paper and to Dave Assmus and Mario Costa for their full support in implementing the remediation plan at the Lube Land slope. Also special thanks to staff members from HVC's Mine Engineering and Mine Operations departments for their constant collaboration in completing this project. Comments and input from Al Chance of Golder Associates Ltd. are also acknowledged.

REFERENCES

AMEC 2008. Open Pit Slope Stability Assessment – South Lube Land Overburden Slope. Memo submitted to HVC, dated July 31, 2008.

Golder Associates Ltd. 2005. Valley Pit – Proposed Expansion Pit East Wall Overburden Slope Stability Assessment, Highland Valley Copper. Report submitted on April 19, 2005. 36p.

Golder Associates Ltd. 2004. Valley Pit East Wall Overburden Slope Stability Assessment and Design, Highland Valley Copper. Report submitted on May 26, 2004. 42p.

Golder Associates Ltd. 2009. Results of Lube Land Clay Samples Testing. Memo submitted to HVC, dated March 4, 2009.

Newcomen, H.W., Murray, C., and Shwydiuk, L. 2003. Monitoring Pit Wall Deformation in Real Time at Highland Valley Copper, Proceedings of the Fourth International Conference on Computer Applications for the Mineral Industry, Calgary, Alberta, Canada, September 8-10, 2003.