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ANALYSIS & MITIGATION OF A LANDSLIDE IN DISCONTINUOUS PERMAFROST

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Abstract: The geotechnical investigation and stabilization design for a landslide located near Fairbanks, Alaska, revealed several issues with testing and modelling landslide shear zones in permafrost materials. The subsurface soils consist of frozen silts with zones of organics and ice inclusions (wedges and lenses), and underlain by zones of unfrozen sand and gravel. A 30 m deep mining pit was excavated downslope of a highway, effectively removing lateral support and causing instability. Borings were drilled and instrumented with inclinometers, standpipe observation wells, vibrating-wire piezometers and thermistors. Measured ground movements indicated that the shear zone passed through both frozen and unfrozen soils. The literature was researched to check for documentation of residual shear strengths on frozen silts; however, there is very little information published on this subject. Attempts to back-calculate the effective residual friction angles were complicated since the actual groundwater pressure distribution along the shear zone could not be reliably estimated. Ultimately, the design of stabilization measures was based on total stress analysis. Back-calculated values of residual strength were expressed in terms of "average cohesion." This analysis method assumes that groundwater conditions after stabilization would be similar to the levels at the time of failure.

INTRODUCTION

The site is located on the Parks Highway approximately 13 km west of Fairbanks (central Alaska, 200 km south of the Arctic Circle), which is an area that is underlain by discontinuous permafrost. Permanently frozen ground, or permafrost, is a thickness of ground that is permanently frozen. Discontinuous permafrost is where zones of permanently frozen exist together with unfrozen ground (Tart, 1996, and Davis, 2000).

In the 1920's, dredging operations related to gold mining required the cutting of a channel through frozen silts. This channel, along with man-made fills and dredge tailings left behind by mining operations, is still evident. In the summer of 1999, a mining company excavated a 30 m deep pit across the dredge channel south of the Parks Highway. The subsurface soils exposed in the mining pit consist of frozen silts with zones of organics and ice inclusions (wedges and lenses). The silt unit ranges in thickness 24 to 30 m and is underlain by a layer of free-draining gold-bearing gravel, and in turn, schist bedrock. The upper 6 m of the silt is thawed and affected by seasonal freezing, whereas the silt below is perennially frozen. The gravel layer is predominantly unfrozen but contains discontinuous frozen zones. At about 30 m deep, the pit excavation encountered artesian groundwater flowing from the gravel. The mining company pumped the water out of the pit to allow excavation and mining operations to continue during September and October, 1999. Dewatering in the pit caused a drawdown of the groundwater table, particularly in the deep gravel layer, which impacted nearby residential wells (Alaska Department of Natural Resources (2000a). By October 1999, cracks were observed in the area between the edge of the open pit and highway, affecting the travel lanes. The mining company subsequently suspended pumping due to the indirect impacts and the pit began to fill with water. Initial cracking developed into fissures, up to 3.6m wide, over a period of 6 months. Figure 1 shows significant failures of portions of the northern pit wall, and slumping and development of scarps typical of a progressive landslide failure in the ground between the pit and the roadway. The eastern pit wall also experienced slump failures as the exposed ice-rich silt thawed.

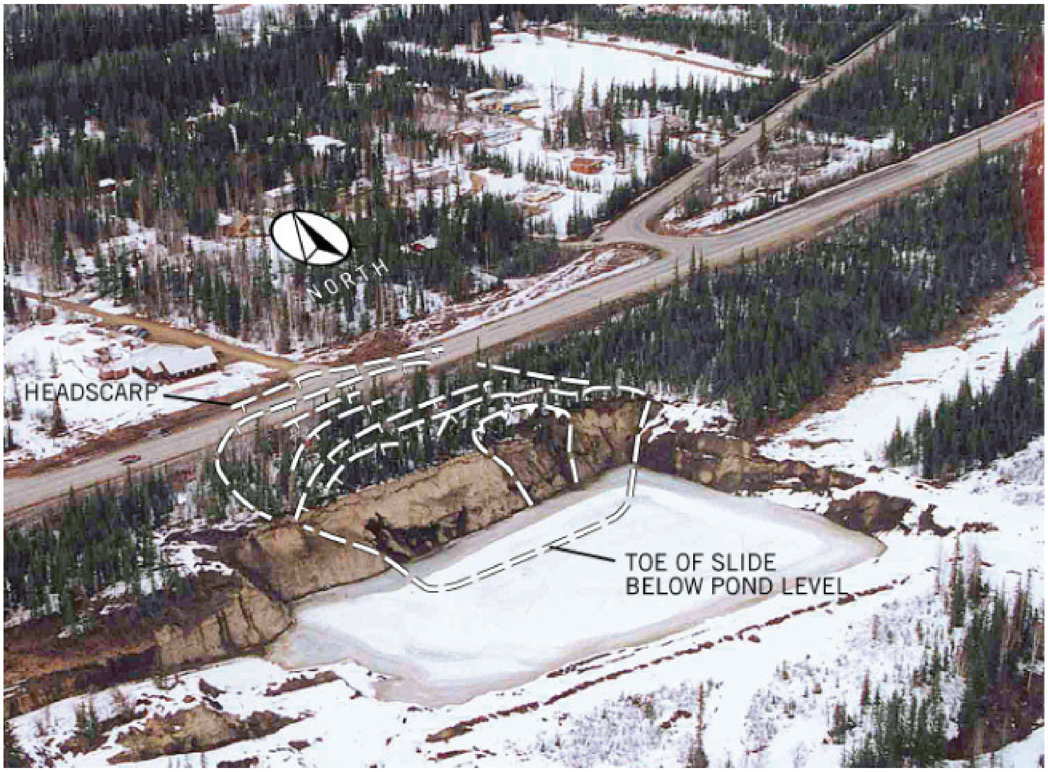

Figure 1. Annotated photograph showing landslide conditions

The landslide footprint is about 150 m wide by 80 m long with the toe exiting into the bottom of the mining pit to the south of the highway. In May 2000, the tension cracks and the distress to the pavement had progressed to the point that a full-width asphalt-concrete overlay patch was required to repair the travel lane. A graben appears to have formed in the south embankment slope. Figure 2 shows representative scarps and slumps.

Figure 2. Landslide Scarps and Slumps

Observations of the upper northern and eastern pit walls revealed localized sloughing and block rotation likely resulting from thawing of the permafrost and ice wedges. In addition, the permafrost between the highway and the mine pit started to thaw, which accelerated ground subsidence. The northern pit wall was affected by ablation, spring activity and translational slide movement. Figure 3 is a photo showing the exposed geologic stratigraphy before the pit filled with water. A landslide toe can be interpreted from the photograph, jutting out from the cut slope close to the contact between the ice-rich permafrost silt and the underlying gravel. By August 2000, the water level had risen to a depth of over 18 m due to inflowing groundwater and silt slough collapsing into the water from the north pit bank, refer to Alaska Department of Natural Resources (2000a and 2000b). Most of the original northern and eastern pit faces had failed into the pit.

The highway pavement subsidence and cracking presented a hazard to motorists, which required daily patrols to maintain. Due to accelerated pavement distress, a mitigation measure was required. To determine appropriate mitigation measures, geotechnical investigations were performed, as summarized by Landslide Technology (2001).

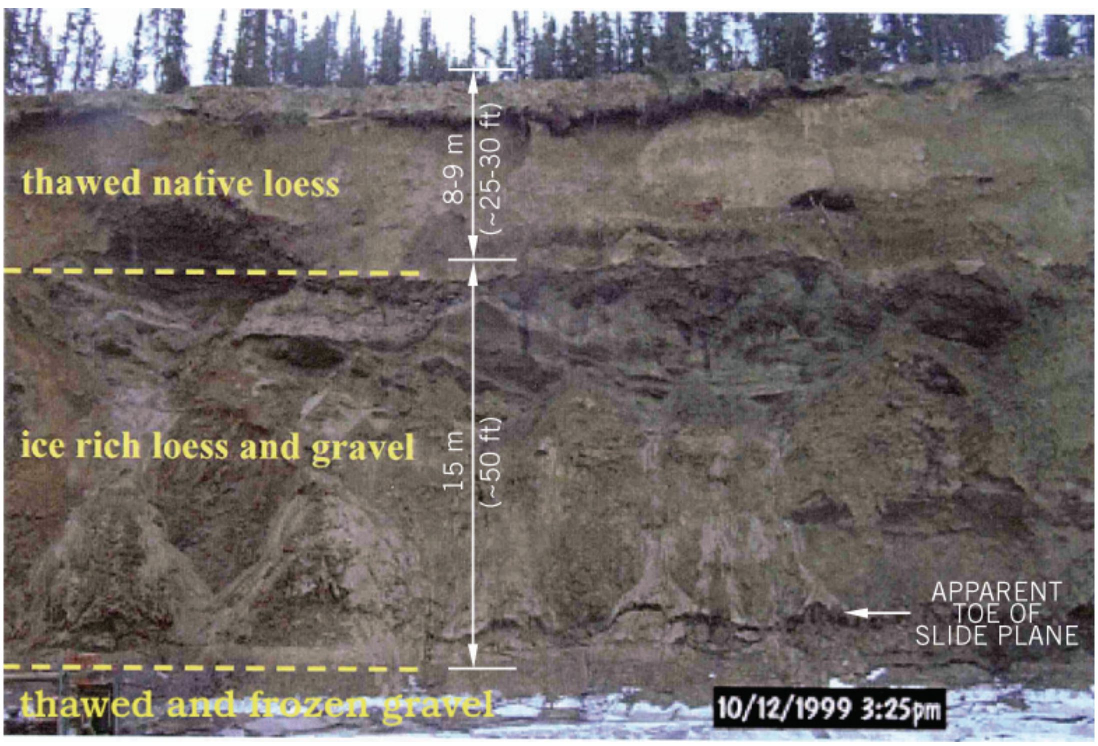

Figure 3. Stratigraphy in mining pit wall (Alaska Department of Natural Resources)

GEOLOGY AND CLIMATE

The project is located in the physiographic province of the Yukon-Tanana Upland. Typically, the geology comprises schist bedrock (Birch Creek formation) that is overlain by glacial outwash gravel and sand, which in turn is covered by wind-blown silt (loess) and/or redeposited loess (water-borne or colluvial silt). Lenses of organic material occur throughout the redeposited silt. Although the Fairbanks area was never glaciated, the thickness of these glacial-related outwash gravels and loessal silts can be several hundred feet. Refer to Péwé (1982) and USDA (1963).

Permafrost is common within the flood plain deposits and most north facing slopes. The permafrost contains both fine ice lenses and interstitial ice. On colluvial slopes, it can contain large ice masses under the redeposited loess (Tart, 1996). The permafrost table is commonly lowered where the natural vegetation including the insulating mat of moss have been removed. Locally, due to the natural decomposition of the organic materials, the reworked silts may contain pockets of methane gas (USDA, 1963).

The climate consists of large diurnal and annual air temperature variations with small amounts of precipitation, cloudiness, and humidity. Temperatures have varied from a high of 37° to a low of –54° C. On average, there are approximately 100 freeze-free days a year. The mean annual air temperature is -3° C (26° F), while the mean annual precipitation is 30 cm (12 inches), based on USDA (1963).

EXPLORATIONS AND INSTRUMENTATION

Seven borings were completed in two phases of explorations to investigate the conditions related to the landslide. The drilling encountered moist to wet loessal silts to a depth of about 6 to 7.6 m, underlain by frozen silt to at least 21.3 to 24.4 m, as shown on the cross section in Figure 4. The permafrost silt contained ice lenses and interstitial ice. It was not possible to detect shear zones in the samples recovered. There are confirmed water-bearing zones. Methane gas was encountered in organic silts at about 16 m deep in two borings. Below the permafrost silt, the material graded to a combination of quartz and schist gravel. Occasional layers of silt were encountered in the gravel unit. The gravel is underlain by weathered schist bedrock.

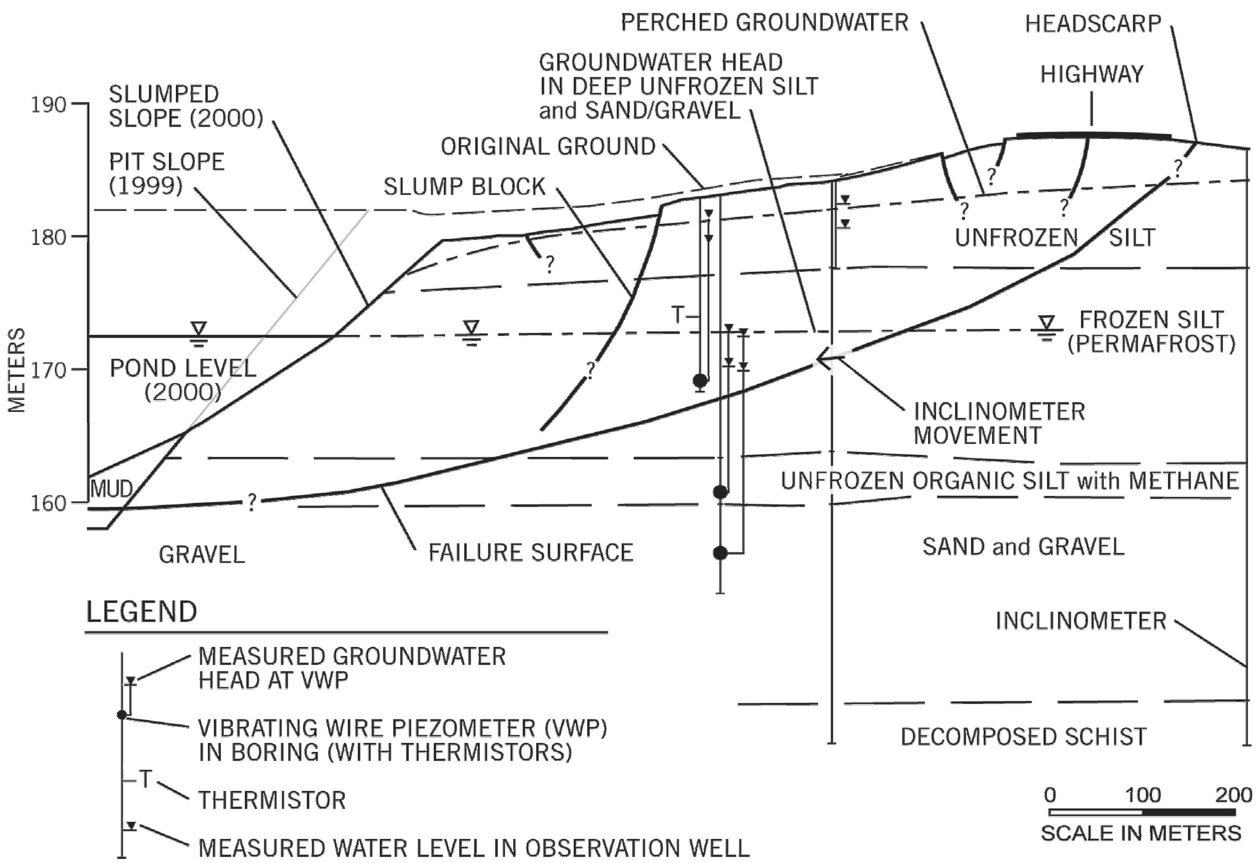

Figure 4. Interpreted landslide cross section

A sample of the silt near the depth of movement (14 m) was tested for residual shear strength. This sample has a liquid limit of 33% and a plasticity index of 6%. The ring shear test was performed on a remolded specimen at room temperature using an equivalent strain rate of 9x10-4 mm/minute. The resulting residual friction angle is $\phi'_r = 30.5^o$. Laboratory testing to determine residual shear strength on frozen soils is complex, requiring temperature controlled conditions during testing. A wide variance of shear strength results could be expected due to the composition of soil, organics and ice. The literature was researched to check for an inventory of residual shear strengths on frozen silts; however, there is very little information published on this subject (based on personal communications with Nixon, 2000). Triaxial consolidated-undrained (CU) tests were performed on two undisturbed samples of silt at depths of 6 to 7.3 m. Wet unit weights were about 1750 kg/m^3 (110 pcf). The moisture contents ranged from 37 to 45%. The testing produced peak shear strength angles of internal friction (φ'_{cu}) of 35^o to 40^o.

The ground temperatures were measured with vibrating wire piezometers and thermistors. The resulting ground temperature profile on the few dates measurements were made showed temperatures at or slightly below freezing from 5.8 to 18.9m deep. The minimum temperature measured was -0.19° C (31.65° F).

Groundwater measurements indicate the presence of two separate aquifers. An upper aquifer appears to be perched above the permafrost. The lower aquifer is an artesian condition in the gravel confined beneath the permafrost zone. The permafrost appears to have a few unfrozen zones that could channel groundwater from either the perched or artesian aquifers, causing a wide range of possible pore pressures along the actual landslide shear zone.

The two inclinometers in the active slide detected landslide displacement along the failure plane. Figure 4 shows the depth of measured shear displacement at a depth of 14.3 m (47 ft), recording an average rate of 1.7 mm/day over 2 months. The inclinometers installed on the north side of the highway (upslope of the headscarp) did not detect landslide-related movements.

Figure 4. Landslide toe after cut slope ablated and pit filled with water

Since the pit walls were ablating and sloughing (see Figure 5), mud was forming in the bottom of the pond. A probing program was conducted to measure the thickness of the mud, which was performed during the winter while the pond surface was frozen. Holes were made in the ice to insert the probe. The probe consisted of a steel pipe with two different attachments (one was a steel plate to probe the top of the soft mud and the other a point to penetrate through the mud to the surface of the underlying gravel). The underwater surface of the mud was relatively flat indicating good dispersal of the silt as it sloughed into the pond. Although some gentle mounding occurred primarily in front of the active landslide, the greatest accumulations (approximately 5m) are where the pit was dug deepest. For reference, the pit bottom had been constructed deeper in this area to accommodate collection and subsequent pumping of groundwater that was entering and filling the pit and disrupting mining activities.

INTERPRETATION OF LANDSLIDE MECHANISM

The depth of the slide varies from head to toe (refer Figure 4). The landslide was caused by the mining excavation, which effectively removed toe support. At the mine pit wall, the basal landslide shear zone appears to be below the water level in the pit, near the contact of the permafrost silt over the gravel unit. The upper thawed silt also appeared to be sliding locally along the contact with the underlying permafrost. Ground movements determined by survey monitoring were greatest towards the mining pit. The ground in the lower half of the slide moved about 1m in a 6 week period (21 mm/day average), while the edge of the highway experienced smaller rates of movement (about 2 mm/day). To the east and west of the landslide, the ground surface appeared relatively undisturbed (except for thawing) and no evidence of landsliding was observed.

STABILITY ANALYSES

Preliminary stability analyses were performed to evaluate probable failure mechanisms. Slope stability computations were performed for a range of possible slide shear zone orientations (labelled shear zone cases 1-3 on Figure 6). The three potential shear zone cases that were selected for analysis included: 1) a shallow zone that would move along the contact of the thawed silt and the underlying permafrost, 2) a shear zone that passes through the measured movement depth in the inclinometer and exits midway in the pit wall, and 3) a deeper shear zone that passes through the measured movement depth in the inclinometer and exits near the bottom of the permafrost silt in the pit wall, near the contact with the underlying gravel unit. All three possible shear zone interpretations were analyzed to make sure that the mitigation measure to be designed would be effective for all plausible cases.

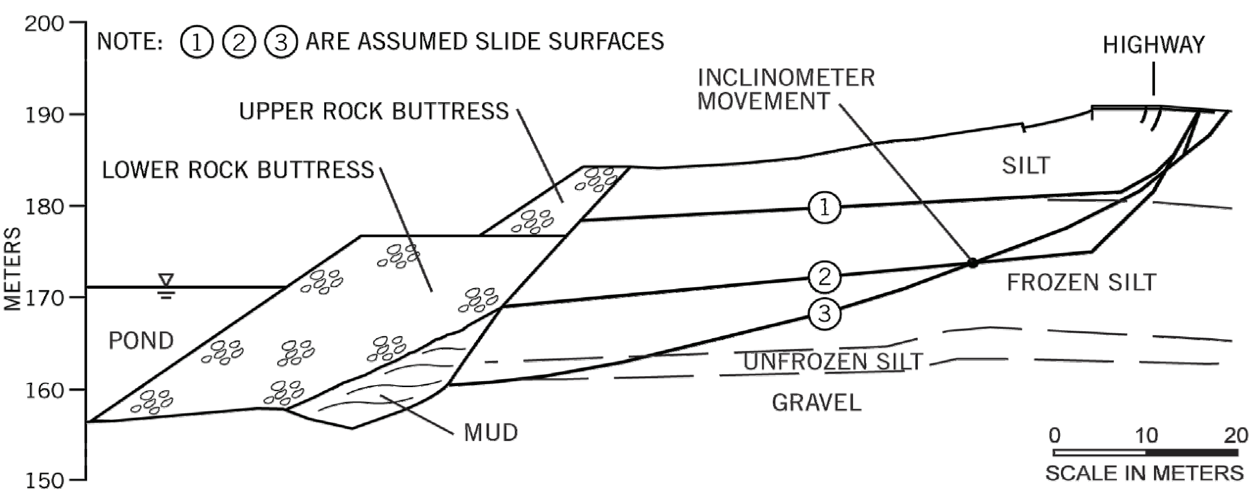

Figure 6. Stability analysis cross section

A series of stability analyses were performed to check the reasonableness of shear strength assumptions along the three potential shear zone cases. The active wedge in the upper unfrozen silt layer was assigned a shear strength of $\phi' = 30°$ and $c' = 0$ in all the analyses, based on the laboratory shear test. The residual shear strength along the translational shear zone was subsequently back-calculated, assuming a Factor of Safety, FS, of 0.99 to represent landslide stability during creep movement.

Initially, effective stress back-analyses were performed. However, the parametric analyses for shear surfaces 2 and 3 had to account for shearing through permafrost (frozen silt that included ice wedges). Several interpretations of groundwater pressures within the permafrost indicated a wide range of possible residual shear friction angles, ϕ_r'. The discontinuous permafrost could be influenced by both the underlying artesian pressures as well as water seeping down from the overlying perched groundwater. It is also possible that the discontinuous permafrost could contain zones that do not directly experience groundwater pressure, particularly prior to failure.

In back-calculating the average residual shear strength there was concern to not over-predict the value of the apparent friction angle in order to avoid overestimating the increased resistance in subsequent calculations for mitigation design. For example, increased normal forces due to placement of rockfill on the slide mass in connection with buttress construction would result in theoretical calculations of higher net shear resistance that might not be fully realized.

The foregoing concern was solved by using the total stress analysis approach since it is not sensitive to groundwater level assumptions. The equivalent cohesion calculated by using total stress methods acknowledges that there is groundwater but eliminates it from potential errors in the shear strength calculation. Back-calculated values of residual shear strength can be expressed in terms of average cohesion for use in total stress analyses. This form of analysis is applicable when the groundwater conditions will not be substantially different in the future. For additional guidance on the use of total stress analysis and cohesive shear strengths, refer to Cornforth (2005) and Duncan (1996).

The first back-calculation for the residual shear strength of the ice-rich permafrost silt was for the slide initiation case, where an average undrained shear strength of 6300 to 8300 kg/m^2 was back-calculated (shear zone case 3). The results of the post-failure analyses (assuming landslide geometry during creep movement) resulted in back-calculated average undrained residual shear strengths for the three shear zone cases. The results are shown in the following Table 1, based on an assumed Factor of Safety, FS, of 0.99.

Table 1. Back-Calculated Undrained Residual Shear Strengths

Shear zone 1	Shear zone 2	Shear zone 3
1070 kg/m^2 (220 psf)	3600 kg/m^2 (735 psf)	6500 kg/m^2 (1335 psf)

The landslide repair would consist of a combination of rock buttresses to satisfy all 3 potential shear zone cases. A smaller upper buttress built on top of a larger lower buttress is shown in Figure 6. The upper buttress would stabilize a possible shallow slide represented by shear zone case 1 and halt further sloughing of the exposed pit wall cut slope. The lower buttress would stabilize deeper shear zone cases 2 and 3. Stability analyses were performed on the selected conceptual buttress mitigation options. The width of the buttresses perpendicular to the roadway was sized to satisfy a factor of safety requirement of 1.3. This criterion (FS=1.3) was selected to account for uncertainty regarding the geometry and depth of the actual shear zones and possible changes in shear strength as permafrost conditions continue to thaw.

If an effective stress analysis method was used to estimate a residual shear friction angle, a compound groundwater surface would need to be assumed. However, it was determined that these analyses could overestimate the design case Factor of Safety by 10 percent. Therefore, further trial and error assumptions would need to be made to try to reconcile these variables. Consequently, the effective stress approach was not utilized for design since the total stress approach is considered more applicable with the level of information available.

DESIGN AND CONSTRUCTION

The design evaluated methods to construct the recommended rock buttress over the soft silt that formed underwater in the bottom of the pit. The soft silt (mud) would likely be displaced as the buttress rock was placed upon it. However, if the buttress rock was placed too quickly or constructed on too steep a slope, the buttress could experience localized slides. It was expected that with a careful rate of placement, rapid failure of the buttress could be avoided. Even with careful construction practices, it was possible that some of the mud would be trapped under or squeezed into the rock buttress. Therefore, the stability analyses included modelling of a mud layer beneath the rock buttress.

A filling approach was designed to slowly displace the mud, by placing the rockfill first against the face of the mining pit and then towards the pond, as shown on Figure 7. The advancing front edge of the rockfill was angled to achieve displacement of the mud. Using this approach, the slope of the rock buttress was able to be maintained at 1.5H:1V. Underwater soundings confirmed the angle of the steep rockfill slope that was achieved and also verified that the mudline beyond the rockfill toe had increased in elevation due to the formation of mudwaves from the displaced soft soil.

Figure 7. Staged construction of rock buttress, advancing across the mining pit pond

The placement of the rockfill into the mining pit pond caused an equal volume of water to displace and therefore the pond level rose. The rise in pond level was anticipated, and it was determined that eventually excess water would spill into the adjacent abandoned dredge channel. Utilizing the old dredge channel to convey displaced water helped control water quality because the channel was vegetated and provided a filter. The fill placement was conducted at a relatively slow rate, and monitoring verified it was not causing excessive turbidity. Downstream water quality was also checked. The rock buttress was constructed successfully and landslide movements essentially stopped. Figures 8 and 9 show the completed rock buttresses. The highway pavement was subsequently repaired by removing the cracked surfacing, then reinforcing the subgrade and reconstructing the base course and asphalt-concrete wearing surface. Observations over the past five years confirm that there has been no reoccurrence of scarps or subsidence in the highway pavement.

Figure 8. Raising the height of rockfill for the upper rock buttress

Figure 9. Completed rock buttresses

CONCLUSIONS

Determination of residual shear strength across discontinuous permafrost is a complex situation for conventional laboratory testing and stability analyses. While tests have previously been performed to determine peak shear strengths of frozen soils, research confirmed the probable lack of reliable residual shear strength test data on frozen soils.

In discontinuous permafrost soils, the determination of a groundwater profile can be complex. Normally, effective stress stability analyses are preferred for landslide studies in unfrozen ground. However, those analyses require reliable and reasonable groundwater profiling. This paper demonstrated the use of total stress stability analyses when the groundwater profile is not known with certainty, as is the case for the permafrost conditions at this site. Satisfactory results can be achieved in the total stress analyses as long as groundwater conditions are not significantly different after construction compared to during sliding.

Construction of the rock buttress mitigation measure accounted for mud that had accumulated at the bottom of the water-filled pit. A mud displacement technique was successfully utilized. The rock buttress has stabilized the landslide and restored the highway.

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