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Slope stability study of an open pit gold mine project in interior Alaska

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Abstract: The study tasked by Ryan Lode Mines, Inc. involved analysis of pit slope stability for two future pits, Ryan and Curlew. A geological discontinuity survey and groundwater information were obtained through a field program. Twenty core logs containing geotechnical information were used for rock mass classification. The kinematic analysis was performed based on a friction angle (ϕ=35°), the distribution of geological structures, and a dry slope condition. Factors of safety of pit slopes in two future mines were determined using the limit equilibrium method. The mine slopes and benches designed by Mine Development Associates (MDA) were analyzed. The analysis indicated that both pits should have an overall safety factor above 1.0, provided the slopes are kept dry. However, slopes in both pits exceeding 91.4 m (300 ft) high will become critical, when water fills the cracks and discontinuities.

1. Introduction

A typical open pit mine may have as few as two or three slope failures during its operation life. Delineation of those isolated unstable slopes in a mine requires adequate planning to obtain the necessary critical information before mine operation is commenced. The Department of Mining and Geological Engineering at the University of Alaska Fairbanks was tasked by Ryan Lode Mines, Inc. to assess the potential slope instability problems on the Ryan Lode property.

The approach to this slope stability study involved three stages of tasks:

- Step 1 involved an evaluation of the geological, geomechanical, and hydrologic data provided. The information included lithology, discontinuity conditions, fracture systems, core recovery, and RQD. The strength properties of rock materials were obtained by performing the point load tests, Brazilian tests, and uniaxial and triaxial compression tests on rock cores.
- Step 2 included two tasks: establishment of a geo-model and classification of rock mass quality. The geo-model was constructed based on information of geological structures to delineate the potentially unstable structural domains from the stable structural domains. The geomechanics classification was performed using the data collected in step 1.
- Step 3 consisted of detailed analysis of failure modes in the mine area. Ranges of the factor of safety for benches and overall pit slopes were evaluated using numerical techniques for plane and wedge sliding. Sarma's method (a closed form solution) and the finite element method were also carried out for several slope profiles that were considered critical.

2. Field Investigation

Mine geology

The Ryan Lode mine is located on Ester Dome, approximately 6 miles northwest of Fairbanks, Alaska (Fig. 1). The mine was an open pit operation with a plan to enlarge the existing three pits (i.e. North, Hilltop, and South pits) and to excavate another pit, south of the existing pits, in the Curlew section of the property.

Ester Dome is underlain by quartz-mica schists and micaceous quartzite of the Yukon-Tanana metamorphic complex that has experienced extensive hydrothermal mineralization. The Yukon-Tanana metamorphic terrain is made up of two sequences of formations ranging in age from late Precambrian through late Paleozoic. The mineralization occurs as fault-controlled metalliferous quartz veins in the schists and micaceous quartzite (Hawkins et al., 1982).

The stratigraphy on Ester Dome is similar to that in the Fairbanks area; the schists and quartzite have been subjected to several deformational events. Measurements of fold axes, foliation and mineral lineations suggest that the rocks in the dome area have been uplifted into an asymmetric dome (Hall, 1985). The orientations of the major lode veins and faults on Ester Dome vary between N50°E and N65°W (Hill, 1933). The joints and fractures, based on measurements by Hall in 1985, indicate a preferential north-south orientation. These fracture systems define the local groundwater flow regime. However, the complex stratigraphy makes it impossible to predict either the depth-to-water or well yields. The fracture systems in the more competent quartzite are much better developed than in the schists. Most of the fractures are steeply dipping.

Geotechnical core logs

Twenty core drill logs were made available to the investigators. The logs gave relatively detailed descriptions regarding the core lithology, types of discontinuities, surface conditions of discontinuities, filling materials, core recovery, and RQD. The purpose of the geotechnical log was to facilitate interpretation of the geological and engineering conditions of the mine property.

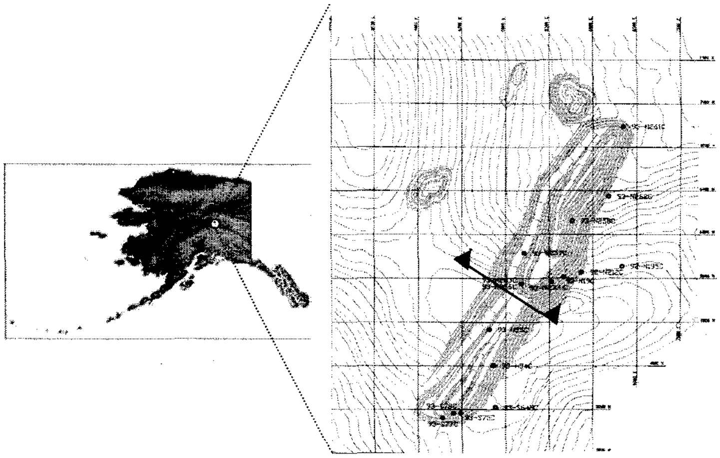

Fig. 1. Map of Alaska showing location of the mine near Fairbanks, Alaska and the drill hole locations.

Discontinuity survey

A detailed discontinuity survey of the North, Hilltop and South pits was performed. The survey data was classified into three discontinuity types: faulting, foliation, and jointing. The contour plots were analyzed for families of discontinuities at the site, which appeared as clusters in the stereo plots. Three faults, three foliation sets and six joint sets were identified from the stereo plots (Fig. 2). Table 1 lists the orientations of cluster centers for the major discontinuity sets.

Groundwater condition

The hydrologic cycle on Ester Dome is characterized by a semiarid climate, perennial and ephemeral streams of relatively low discharge, and a fractured rock aquifer of very low hydraulic conductivity with greater vertical hydraulic conductivities than horizontal conductivities.

The depth-to-water table information from 14 monitoring stations located throughout the mine area was analyzed In view of the discontinuity systems in the dome area and the information from the monitoring stations, the aquifer is probably unconfined (Fig. 3).

The Ryan shear in the mine site may have a directional effect on the flow pattern. It might act as a barrier for flow normal to it, but might have a control for flow parallel to it. For this study, the shear zone and the rock masses below the groundwater table in the mine were considered saturated. However, the shear zone was not considered in the analysis of the final pit slope stability, because the shear zone would be removed in the course of mining.

Table 1. Orientation of the major discontinuity sets.

Location	Cluster Center	
	Pole Bearing, °	Pole Plunge, °
Faults	110	37
	300	22
	132	5
Foliations	281	78
	314	68
	298	52
Joints	263	4
	109	8
	40	16
	44	10
	260	2
	50	12

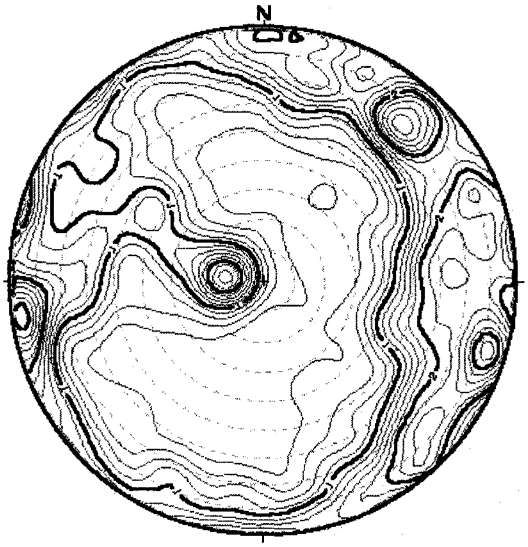

Fig. 2 Stereoplot of the poles of Hilltop discontinuities.

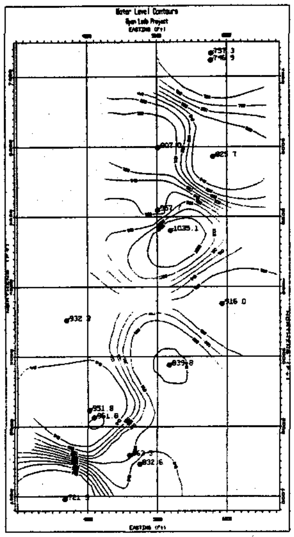

Fig. 3. Groundwater elevation map.

3. Kinematic Analysis

Combined effects caused by all three types of geological discontinuities gave an overview of the maximum stable angles for the slopes in the future Ryan mine. Fig. 4 shows four generalized design regions with each having one recommended slope angle and modes of potential failures if the slope exceeds the suggested angle. This kinematic analysis was based on a friction angle (ϕ=35°), the distribution of geological structures, and a dry slope condition.

- Region I: maximum stable slope angle <80°, potential wedge failures.
- Region II: maximum stable slope angle <60°, potential plane and wedge failures.
- Region III: maximum stable slope angle <50°, potential plane and wedge failures
- Region IV: maximum stable slope angle <40°, potential wedge failures

Because there were significant variations in concentration and orientation of those discontinuity sets in the studied area and because the strength of the rock masses was based solely on frictional angle of that material, the final design of the pit slopes would likely deviate from this analysis. Identification of the critical blocks and wedges and determination of the factors of safety of pit slopes using limit equilibrium method and finite element method were then performed.

4. Stability Analysis

Rock mass quality of the mine strata in the area was rated based on the Geomechanics Classification System (i.e. RMR). Information contained in those 20 core drill logs provided the basis for a general rock mass rating. An analysis of the basic RMR values indicated that the quality of the rock masses at the mine site varies from very poor (i.e. RMR < 20) to good (i.e. 60 < RMR =< 80) with a mean of 32.2 (i.e. poor rock quality) and a standard deviation of 9.2. Vertical variations of the rock mass quality on the mine property were further contoured on an interval of 7.62 m (25 feet).

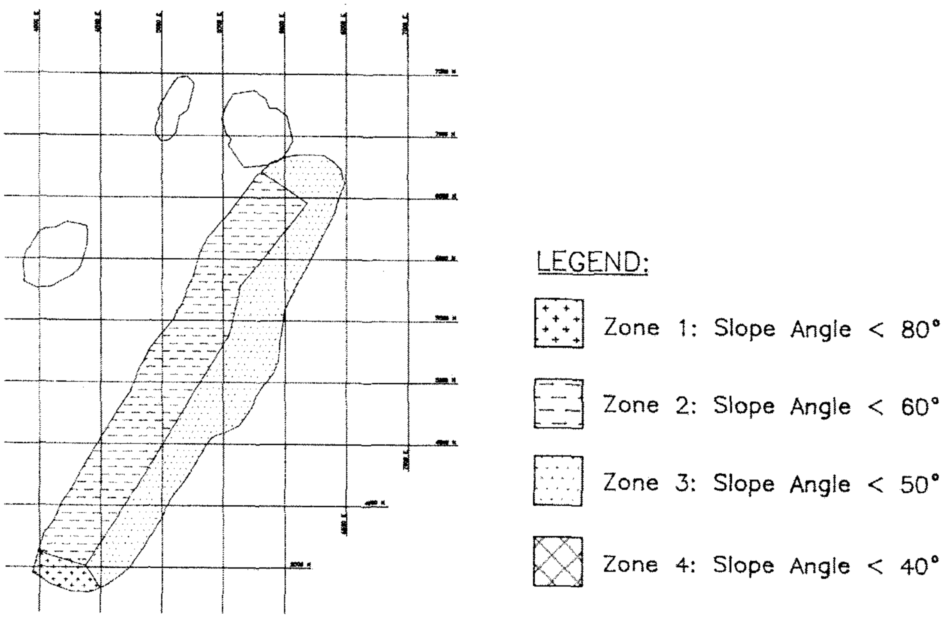

Fig. 4. Kinematic analysis for the future pits.

Mechanical strengths of the rock materials were determined using several laboratory testing procedures including point load tests, Brazilian tests, uniaxial compressive and triaxial compressive tests. The uniaxial compressive strength of quartz-mica schist was noted to vary between 1.0 MPa (140 psi) for heavily weathered specimens to 36.2 MPa (5250 psi) for unweathered rock specimen. Granodiorite had a uniaxial compressive strength ranging from 28.6 MPa (4146 psi) to 112.5 MPa (16320 psi) with the degree of weathering from moderately weathered to unweathered. Quartz monzonite had a uniaxial compressive strength of 80.2 MPa (11630 psi) with no decomposition to 64.3 MPa (9320 psi) with moderate decomposition. Among the three types of rock materials tested, quartz-mica schist was the weakest. Triaxial compression tests showed that schist failed at 66.3 MPa (9610 psi) axial stress with a confining pressure of 3.4 MPa (500 psi). Quartz monzonite had an axial strength of 113.8 MPa (16510 psi) and granodiorite had an axial strength of 265.5 MPa (38500 psi) at a confining pressure of 3.4 MPa (500 psi).

A failure criterion suggested by Hoek and Bray was used to assess the strength properties of the rock masses. Laboratory measurements from the uniaxial and triaxial compression tests were used to derive the material and rock quality dependent parameters, m_i. The m_i parameter was 13.8 for intact schist, 31.9 for quartz monzonite, and 54.5 for granodiorite. The m and s parameters for the above three types of rock masses were further determined based on the m_i values and RMR ratings. Shear strengths of schist, quartz monzonite, and granodiorite rock masses at different normal stresses were then predicted based on the m and s values (Fig. 5).

Factors of safety of pit slopes in two future mine pits were determined using the limit equilibrium method, Sarma's closed form solution, and the finite element method. The mine slopes and benches designed by Mine

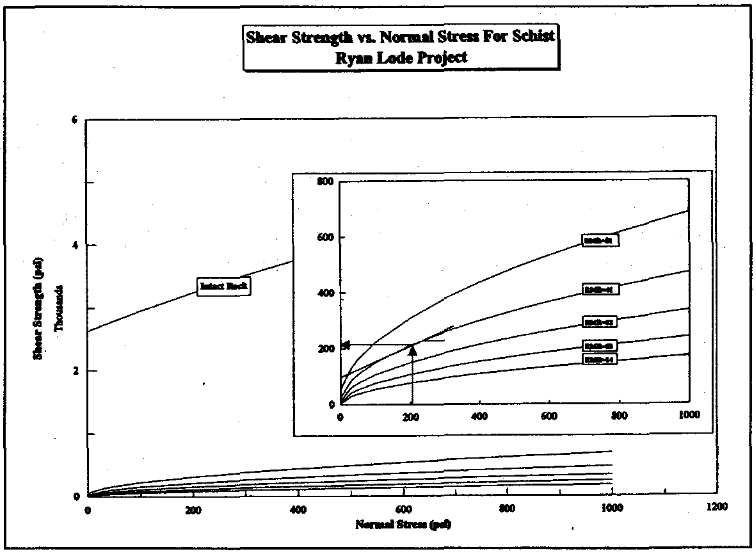

Fig. 5. Shear strength of Fairbanks schist.

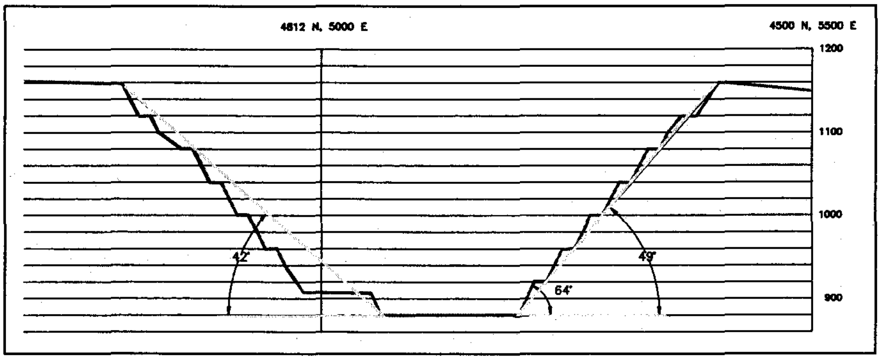

Fig. 6. Cross-section of the future Ryan pit wall.

Development Associates (MDA) were analyzed. Three slope profiles were studied in detail to assess the effects of modes of failure, existence of a vertical tension crack, changes of water saturation in the tension crack, and variations of rock mass quality in the mine area on the overall pit wall and bench stability. One of the example slopes with a height of 77.7 m (255 ft), an overall slope angle of 49° (right wall) and 42° (left wall) and a -2° and 0° upper slopes respectively was shown in Fig. 6. The cohesion (c), friction angle (ϕ), and shear strength along a plane of 24° was selected from the laboratory strength measurements of schist with different RMR ratings and at a normal stress of 1.1 MPa (155 psi). The discontinuity plane dipping at an angle of 24° was the average dip angle of foliation from the three existing pits. A sensitivity analysis carried out indicated the critical angle for the discontinuity was between 20 and 30 degrees. At the critical dip angle, the sensitivity analysis was also carried out with respect to changing RMR values.

The example slope at a dry condition with an RMR value of 14 and no crack in the slope mass was found to have a safety factor of 1.52 for plane failure. The use of an RMR value of 14 included 98% of the rock masses in the mine area that would have strength properties equal to or higher than the values selected for analysis. The same slope with a vertical crack 41.5 m (136 ft) behind the crest of the slope and extending downward to the discontinuity plane, which represents the worst possible drained slope in the field, was found to have a factor of safety of 1.31. Results of the studies indicated that pit slopes having similar geological condition and mechanical

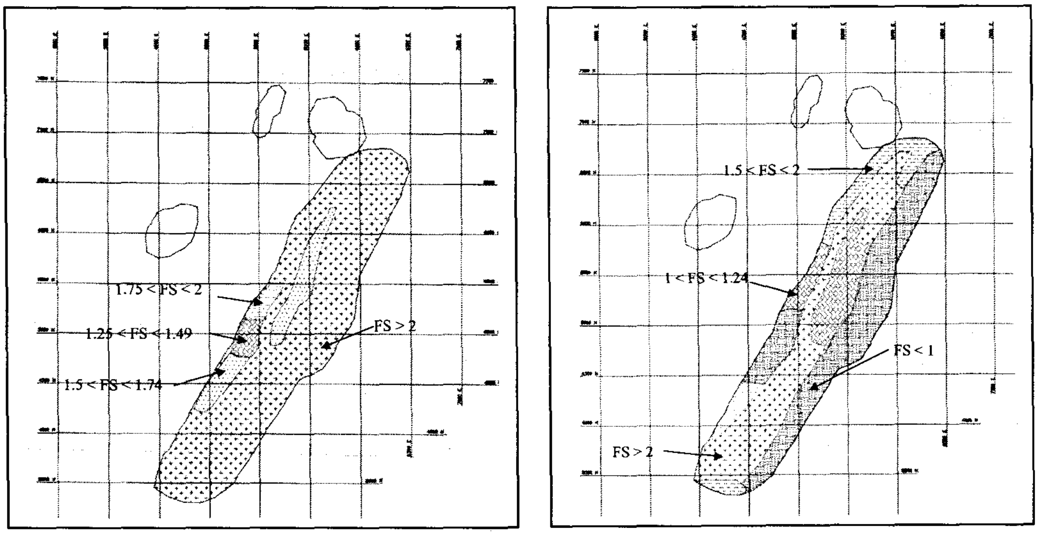

Fig. 7. Pit wall stability analysis (left) dry condition and (right) wet condition.

strength would have adequate stability. The relatively low RMR value (i.e. RMR = 14) used in the calculation yielded a conservative assessment on the overall stability of the pit walls.

Similar analysis was carried out on the same example slope with an RMR value of 23; an inclusion of 85% of the rock masses on the property. The factor of safety was found to be 1.80 for a dry slope with a vertical tension crack. The slope yielded a safety factor of 1.01 at a completely saturated condition. The same slope with higher RMR ratings (i.e. RMR = 32, 41, and 51) had safety factors of 2.47, 3.54, and 5.21, respectively. The saturated slope with poor to fair rock quality (i.e. RMR = 32 to 51) was found to have a safety factor between 1.36 and 2.95 (Fig. 7).

5. Conclusions

The analysis indicated that both pits should have an overall safety factor above 1.0, provided the slopes are kept dry. Under a worst-case scenario, it is concluded that the Ryan Pit will probably be more stable than the Curlew Pit. One of the reasons is that, in general, the slopes in the Ryan Pit will not be as high as that to be constructed in the Curlew Pit. The stability on the east side of the Curlew Pit will be marginal, having factors of safety between 1.00 and 1.24. However, slopes in both pits exceeding 91.4 m (300 ft) high will become critical, when water fills the cracks and discontinuities. Caution should be taken during mine production and the final stage of mining to divert surface and subsurface water away from the mine area.

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