Back analysis of the Goat Hill North slope failure

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ABSTRACT: This paper presents the study of a crack that formed on the Goat Hill North Waste Rock Pile of the Molycorp Questa Site. The crack formed during the re-grading of the slope and represented an excellent way to benchmark a numerical model to a failure condition at the site. With the back-analysis it was possible to use a geotechnical software package to back-analyze material parameters based upon possible modes of failure. The back-analysis of the slope failure indicated there are two possible modes of failure: i) a deep-seated failure through a relatively weak layer (rubble or colluvium layers) or ii) failure through the rock pile material. It appears unlikely that the crack observed on October 2004 is due to a failure plane through rock pile material alone. If this were the case, the resulting model-determined soil parameters (ϕ = 18°, cohesion = 150 kPa) differ significantly from the *in situ* testing program. It appears likely that the observed slope failure was the result of sliding along a deep-seated weak layer beneath the rock pile material and above the bedrock layers. The resulting material parameters needed to produce failure conditions are consistent with existing shear strength laboratory measurements when this hypothesis is considered.

1 INTRODUCTION

On October 4, 2004, during field measurements collected in trench LFG-008, a crack developed in the middle of the trench. This occurred during the destruction of the Goat Hill North (GHN) rock pile at the Questa site. Soil samples were currently being collected by New Mexico Tech as part of field and laboratory testing program for the Molycorp (Chevron) Waste Rock Pile Weathering Study. The purpose of the study was to determine how the stability of waste rock piles might change over time due to physical, chemical or biological processes. The crack varied in width up to a maximum of 100 mm and extended in an arc pattern around the pushed out section of the slope.

The location of the crack, the lip of the slope, and the intersection point with the 2:1 slope was surveyed by Molycorp personnel. The crack was also widening at a rate that was measured by Molycorp personnel. The data was later transferred to the New Mexico Tech (NMT) database. A plan view of the survey locations may be seen in Figure 1.

The slope had been pushed in a convex arc out from the original slope of the mountain in order to ultimately flatten the slope of the GHN pile and reduce a slope failure risk. Trenches were excavated through undisturbed zones of the original rock pile during the slope destruction process in order to provide insitu field testing / sampling opportunities for the Questa Waste Rock Pile Weathering Study. The crack formed an arc through the excavated trenches in a manner opposite to the lip of the slope. A view of the crack as highlighted from a position directly above the slope may be seen in Figure 2. The crack was marked with stakes for visualization purposes.

The progression of the crack across the trench may be seen in Figure 3. The slope angle at the time of failure may be seen in Figure 4. A bulge at the base of the slope was observed but the precise location of the bulge was undetermined. It was hypothesized that the bulge might be related to the toe kick-out of the slope failure. A photograph of the trench and some of the staked crack locations may be seen in Figure 1 through Figure 4. Figure 1 represents a plan view of the crack and shows the intersection with the original ground surface (orange squares), the location of the crack (red triangles), and the crest of the re-formed slope (triangles). The location of the 2D slice utilized for this numerical modeling study followed through the trench (and resulting field data collected) at the center of the slope.

From survey points and field photographs it was possible to accurately identify the geometry of the upper level of the rock pile on which the trench was situated. The average slope was determined from photographs to be 40.5 degrees.

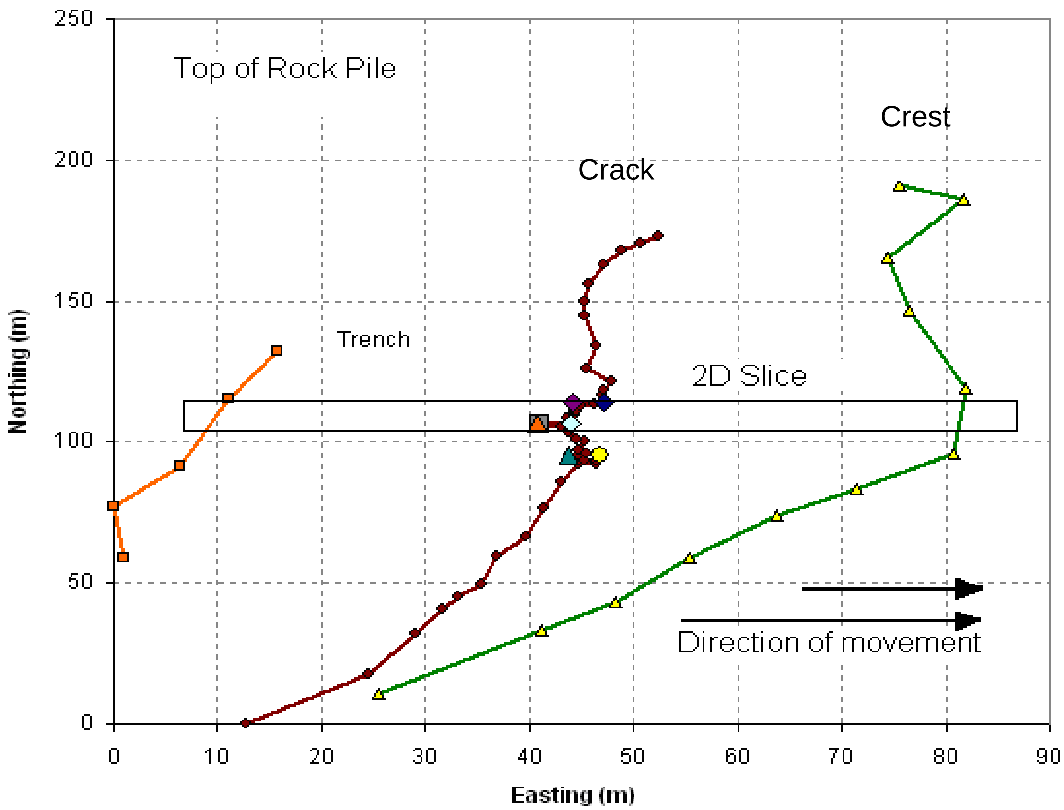

Figure 1. Plan view of the crest along with the location of the cross-section for 2D slope stability back calculations (UTM-Northing of 4062128).

The crack was noted to extend downward in a vertical direction from the surface. The location of the exit point of the crack could not be accurately determined; however, from field observations of bulges on the slope it is estimated the crack exited at a distance greater than 150 ft. from the crest of the slope. The exact location of the exit point of the potential slip surface could not be measured accurately due to safety concerns at the mine site as a result of the slope movement.

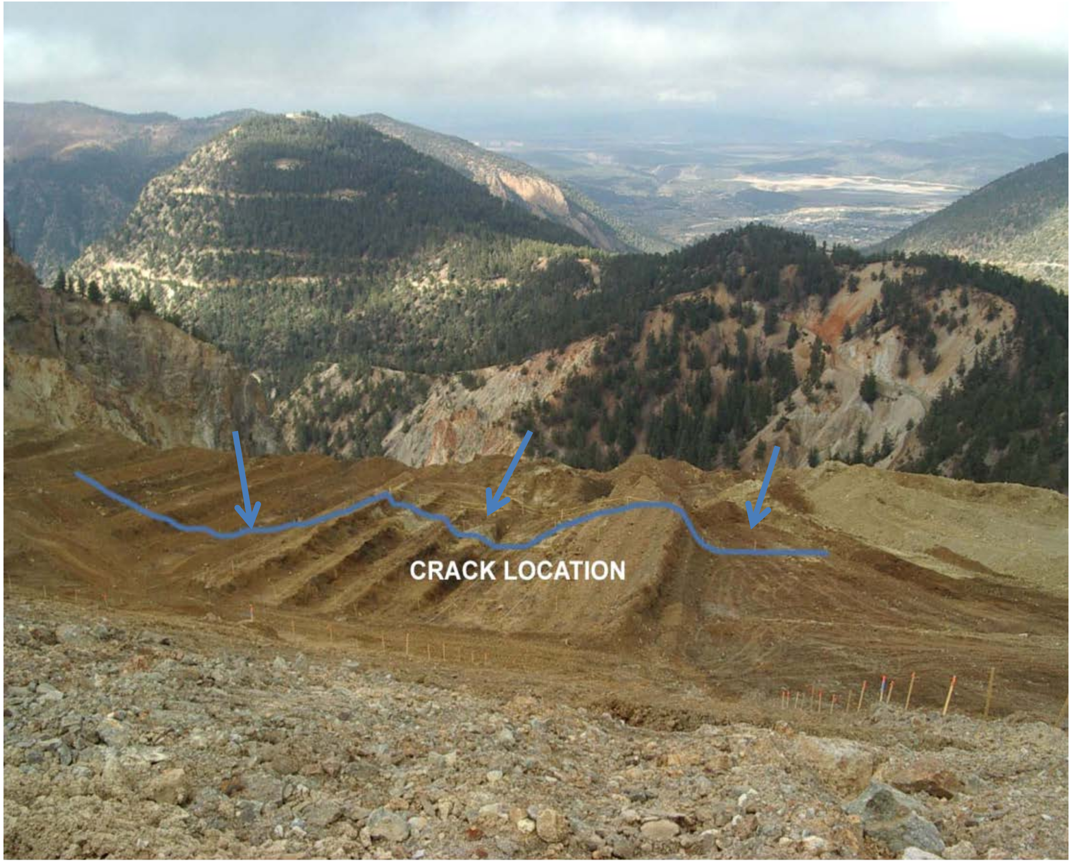

Figure 2 Location of the crack as viewed from up the 2:1 slope (crack highlighted by thick blue line)

Figure 3 Approximate crack locations from the side of the slope (highlighted in blue)

The crack separated a horizontal distance of 0.15-0.24 ft. from October 1 to October 4, 2004 and a vertical distance of 0.6-0.8 ft. The measurements are graphically displayed in Figure 5.

The purpose of the back analysis was to identify possible soil strata/soil property scenarios that would produce a slip surface and failure condition at the location measured in the field. The following analysis identifies possible soil parameters that could produce failure. For this analysis it was assumed that the crack as observed in the field represents failure. It is noted the crack could possibly represent displacement that might not have ultimately represented failure conditions.

Figure 4 Photo showing the angle of the slope during crack formation

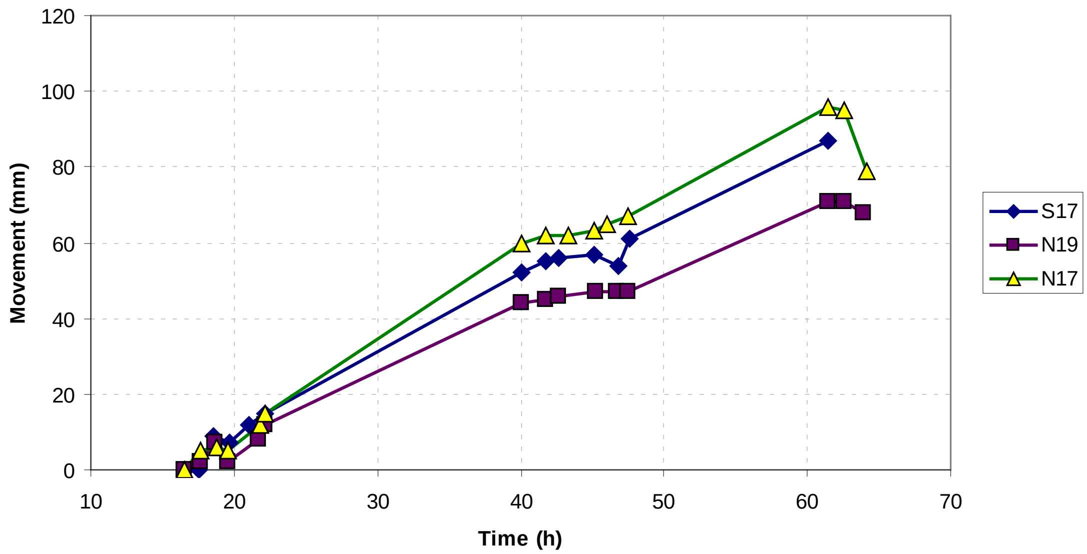

Figure 5 Plot of crack separation versus time on October 2-4, 2004 for identified sampling locations S17, N19, and N17 close to the trench

2 SOIL PARAMETERS AND GEOMETRY

An existing field and laboratory program had previously been performed as part of the weathering study to evaluate the effects of weathering on waste rock pile stability. Shear strength data was provided by testing programs at Norwest, the University of British Columbia, and New Mexico Tech. The result of this testing was effective peak friction angles between 36 to 47°. The spread in friction angle values was estimated due to sampling location / composition and is not a reflection of the degree of weathering spatially throughout the pile. It was of interest i the back-analysis to discover if the back-calculated material properties would match with the laboratory shear-box determined values.

For the purposes of the back analysis, it was assumed the shear strength of the rock pile material is homogenous throughout the entire slope. It is also assumed that the slip surface does not go through bedrock and that the shear strength of the bedrock is significantly higher than that of the rock pile material. There is a weak soil layer between the bedrock and the rock pile material as reported in the Norwest (2004) report. It is assumed the shear strength of the rubble zone is less than that of the rock pile material. For the present analysis it is assumed that the colluvium and the rubble zone are combined into a single layer. The layers are combined because of the lack of data to justify a significant distinction between the layers. Combining the layers is also considered conservative in the present analysis. The finalized geometry utilized in the analysis may be seen in Figure 6.

Tensiometer readings and gravimetric water contents were also taken along the crack on October 3, 2004. These readings illustrate moist conditions along the top of the crack at the time of failure (Figure 7).

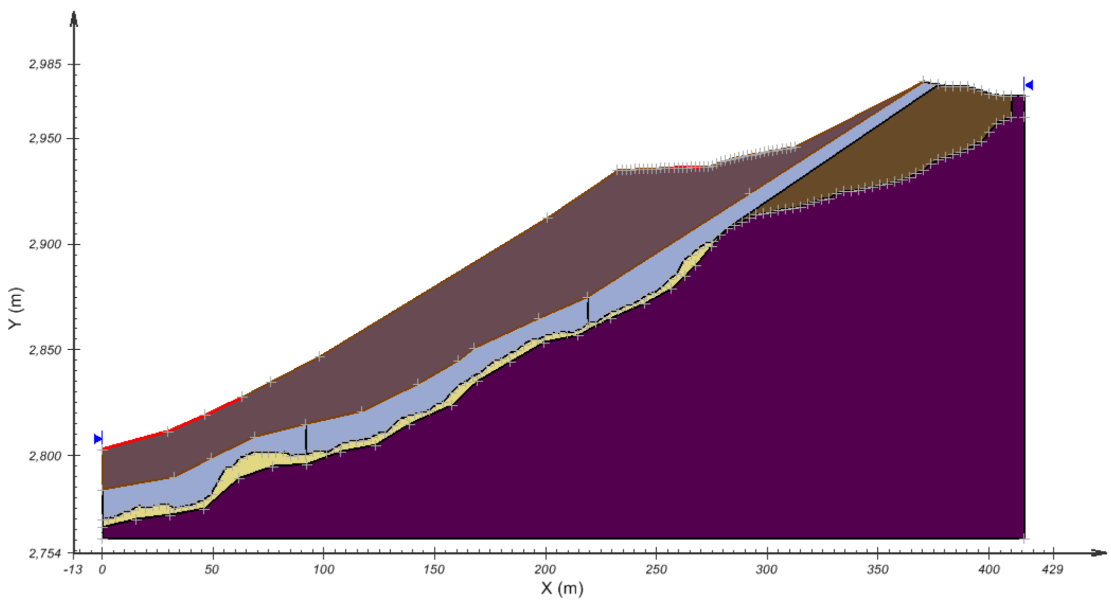

Figure 6 Established geometry of the 2D cross-section through the center of the slope failure

3 ANALYSIS

Several scenarios were developed based on differing assumptions regarding the slope geometry of the rock pile. The various scenarios were analyzed using both the SVSLOPE® and the SLIDE™ slope stability software packages. The models analyzed in each slope stability program were identical. The factor of safety results between the two software packages varied only in the third decimal place (Table 1) and is considered insignificant in this analysis. Only circular slip surfaces were considered in this analysis.

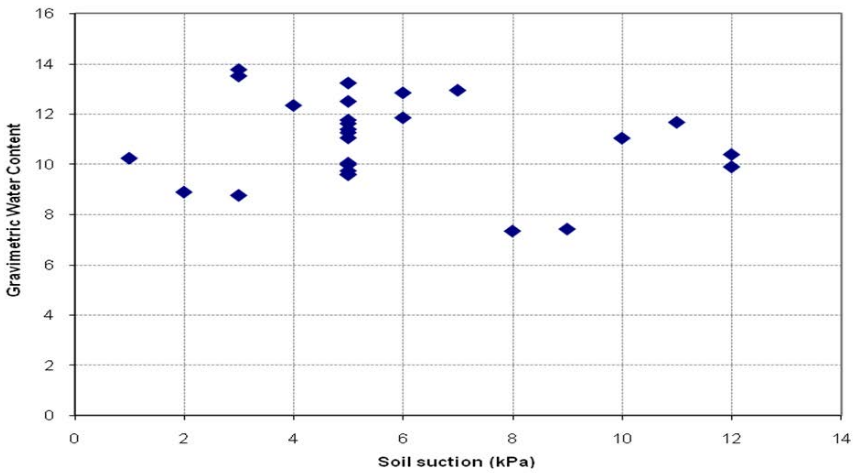

Figure 7 Tensiometer readings taken along crack on October 3, 2004

The four scenarios are described below.

- Case 1: Fixed Entry of Failure Surface - Strong rock pile material and weak rubble zone
- Case 2: Fixed Entry of Failure Surface - Homogeneous model
- Case 3: Variable Critical Slip Surface (CSS) location – Strong rock pile material and weak rubble zone
- Case 4: Variable Critical Slip Surface (CSS) location – Homogeneous model

3.1 *Case 1: Fixed Entry of Failure Surface - Strong rock pile material and weak rubble zone*

Measured peak strength friction angles for the rock pile material are between 36 to 47°. In the Case 1 scenario it is assumed that the rock pile material has strength values of cohesion = 15 kPa and a friction angle = 36°. The strength of the rubble zone is then reduced until the factor of safety of the critical slip surface falls below 1.0. The General Limit Equilibrium (GLE) method of slices was used in this analysis. An entry and exit trial-and-error methodology was used to identify the location of the critical slip surface. The entry point of the slip surface is known from the survey data. The exit point is approximated based on a combination of field observations of a bulge at the toe as well as a sensitivity slope stability analysis regarding the likely location of the slip surface. The results of a typical analysis may be seen in Figure 8.

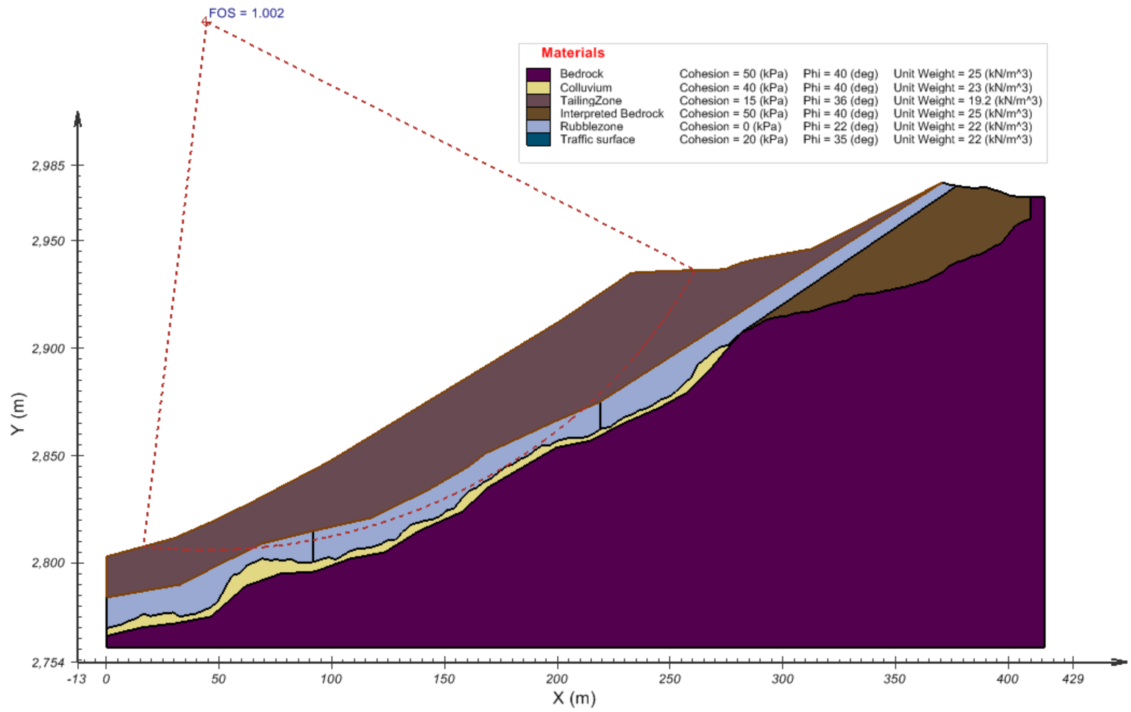

Figure 8 Typical results of analysis of a potential failure through the rubble zone (SVSLOPE)

Table 1 Results of Morgenstern-Price calculations of failure through the rubble zone or colluvium

Run #	c	Phi	FoS(M-P)		
			Slide	SVSlope	
	kPa	Deg.		Moment	Force
4	0	24.0	0.997	1.000	1.000
7	5	23.5	0.990	0.993	0.993
8	10	23.5	0.998	1.001	1.001
9	15	23.0	0.992	0.994	0.994
10	20	22.5	0.984	0.986	0.986
11	30	22.0	0.985	0.987	0.987
12	40	21.5	0.986	0.988	0.988

The combinations of cohesion and friction angle which produce a factor of safety of 1.0 are presented in Table 1. All possible combinations of cohesion and friction angle are presented in Table 1 regardless of whether or not the combinations appear to be reasonable. It is worth noting that not all combinations are reasonable based on the observed location of the slip surface. Given the deep location of the slip surface it appears unlikely that a cohesion value of less than 10 kPa is possible since a reasonable amount of cohesion is needed to force the CSS to have reasonable depth. The friction angle of approximately 22° is similar to the shear strength values of the colluvium as measured by Norwest.

3.2 *Case 2: Fixed Entry of Failure Surface - Homogeneous model*

In Case 2 scenario, it was assumed that the soil parameters of the rock pile material and the rubble zone are the same. The entry and exit points of the slip surface were fixed and a circular slip surface was assumed. The radius and center of the assumed slip surface was allowed to vary based on a series of increments related to the assumed entry and exit points. For these scenarios, the Morgenstern-Price limit equilibrium method of slices was used in the analysis.

A series of model runs were performed in which the cohesion was assumed to be zero and the effective friction angle allowed to vary until a factor of safety just below 1.0 was achieved. Cohesion was subsequently added in the additional model runs and the effective friction angle varied until failure conditions were achieved. An example of the critical slip surface may be seen in Figure 9.

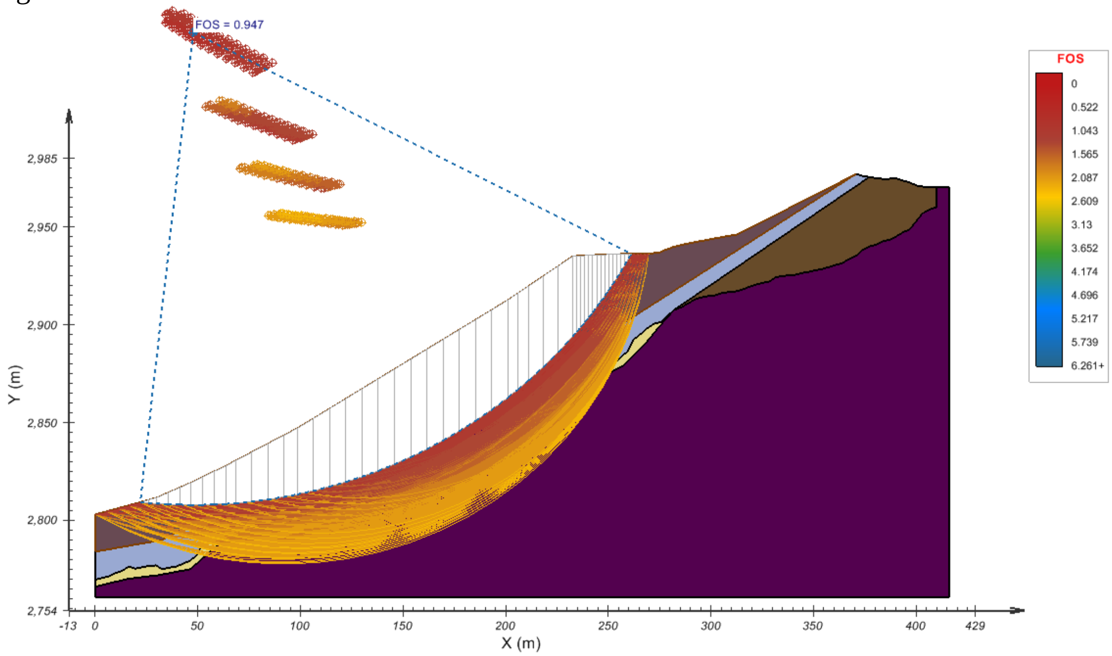

Figure 9 Example location of the critical slip surface for Case 2 (SVSLOPE)

The series of model runs resulted in a relationship between cohesion and effective friction angle as presented in Figure 2. Since cohesion is needed for the slip surface to remain deep in the slope it is assumed that a minimum amount of cohesion is needed in this case. It is reasonable to assume that one of the combinations of the friction angle and the cohesion developed during the LFG-008 trench.

Table 2 Results of homogeneous failure conditions for Case 3 (Morgenstern-Price method)

Run #	c	Phi	FoS(M-P)		
			Slide	SVSlope	
	kPa	Deg.		Moment	Force
1	0	33.0	0.982	0.981	0.981
2	5	32.5	0.992	0.992	0.992
3	10	31.5	0.984	0.984	0.984
4	15	31.0	0.994	0.995	0.995
5	20	30.0	0.986	0.988	0.988
6	25	39.5	0.993	0.994	0.994
7	30	29.0	0.988	1.000	1.000
8	40	27.5	0.993	0.995	0.995

What is problematic in this scenario is the fact the friction angle of the material is required to be between 27-33 degrees in order to achieve failure conditions. This range of friction angles is significantly different than the 36-47 degrees friction angle measured by Norwest, the University of British Columbia and New Mexico Tech. It is therefore considered unlikely this scenario is realistic.

3.3 *Case 3: Variable CSS location – Strong rock pile material and weak rubble zone*

The previous two analyses assumed the location of the slip surface was fixed based on field observations. The attempt of the previous slope stability analysis was to determine reasonable shear strength parameters while fixing the location of the critical slip surface. It is worth noting that a slope stability analysis should be able to correctly identify the location of the slip surface if the model is correctly replicating field soil parameters and geometry. In the Case 3 and 4 analyses the location of the potential slip surface is unrestrained. Material parameters for the angle of internal friction and cohesion are then varied manually in order to cause the location of the critical slip surface (CSS) to replicate field observations.

The selected slip surface location shown in Figure 10 is based on an approximate match of entry and exit points and results in the following material parameters:

Rock Pile Material:
- Cohesion = 15 kPa
- Friction angle = 36 degrees

Colluvium/Rubble:
- Cohesion = 7 kPa
- Friction angle = 24 degrees

The identified material parameters are reasonable and provide a level of continuity with measured laboratory and field results. The measured friction angles for *in situ* tests showed a lower limit value of 36 degrees. The friction angle for the colluvium / rubble zone is also consistent with the properties of the colluvium determined in the Norwest (2004) study.

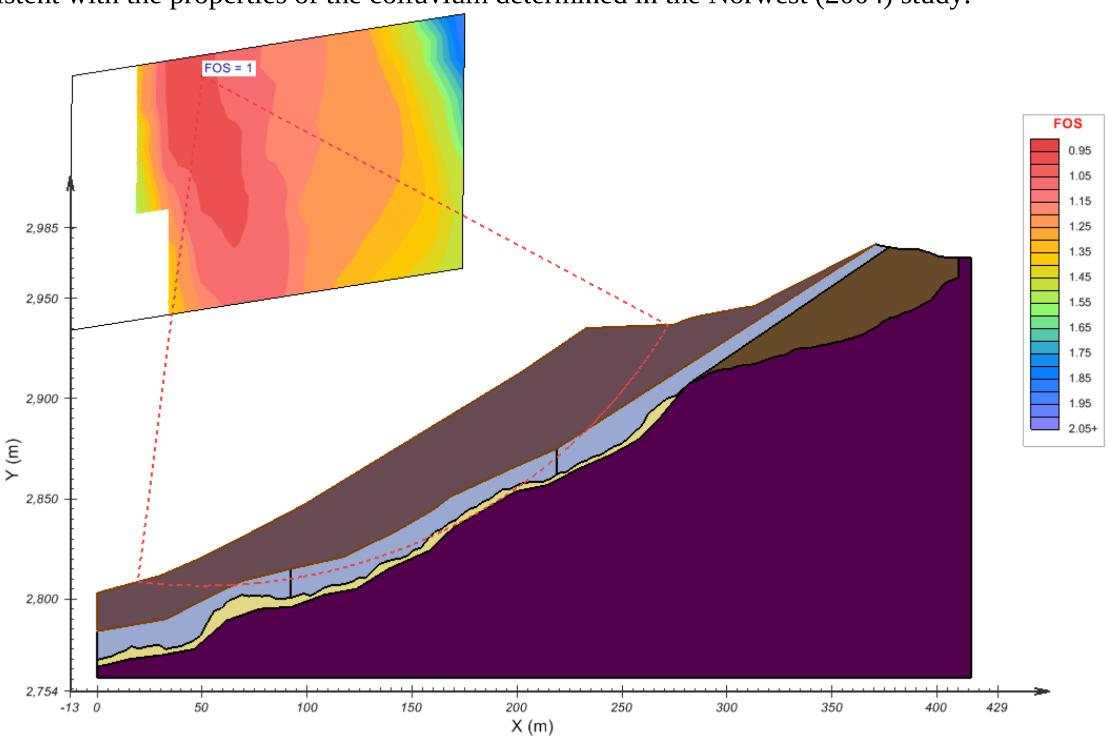

Figure 10 Location of slip surface for weak rubble / colluviums (SVSLOPE)

3.4 *Case 4: Variable CSS location – Homogeneous model*

Case 4 represents the scenario when the rock pile and rubble regions are given the same material parameters. The location of the critical slip surface is allowed to freely vary within the confines of the grid and radius search technique. The material parameters were adjusted until the upper entry point of the critical slip surface location matched field observations and the calculated factor of safety was approximately equal to 1.0.

The resulting critical slip surface is very similar in location to the critical slip surface determined in Case 3 and may be seen in Figure 11. The soil parameters used to achieve this critical slip surface are quite different than obtained for Case 3 and are given below:

Rock Pile Material:
Cohesion = 150 kPa
Friction angle = 18 degrees

These parameters are unrealistic in comparison to site-measured parameters. The high cohesion values are needed in order to cause the CSS location to be deeper in the pile and match the observed exit point of the slip surface. It is reasonable to conclude the rock pile and rubble zones do not have the same material properties.

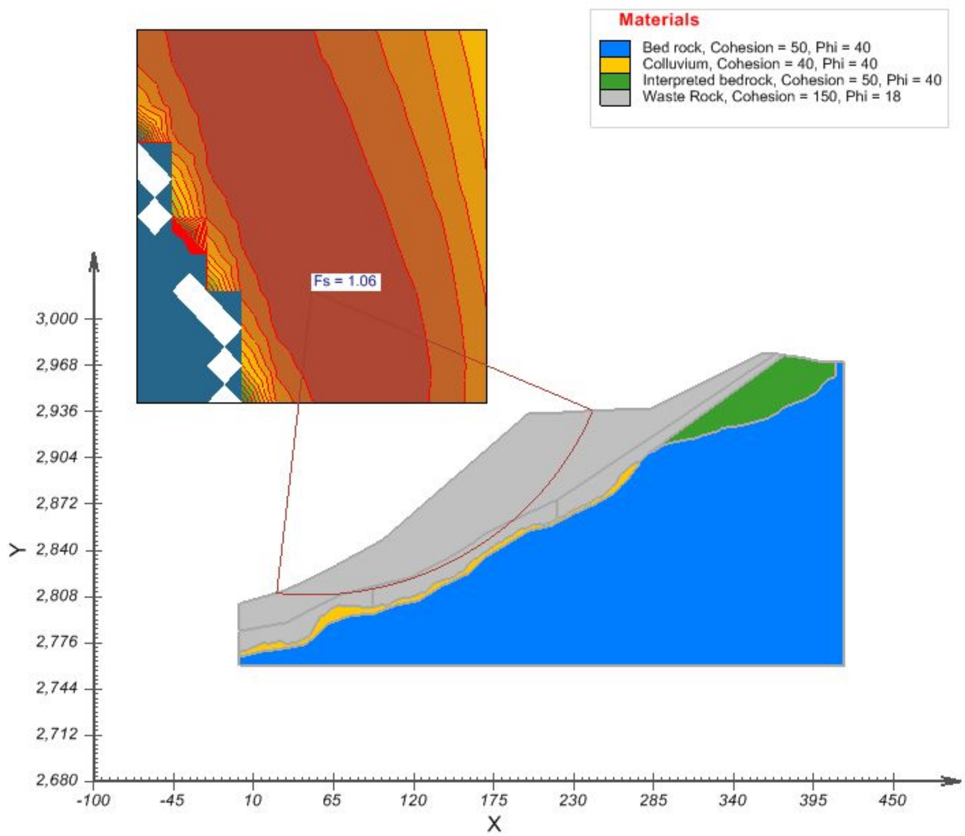

Figure 11 Resulting slip surface from a homogeneous model (SVSLOPE)

4 CONCLUSIONS AND RECOMMENDATIONS

The back-analysis of the slope failure at Goat Hill North in October, 2004 indicates that there are two possible modes of failure. These modes of failure are i) a deep-seated failure through a relatively weak layer of rubble or colluvium layers and ii) failure through the rock pile material. The results of the various failure cases analyzed is as follows.

Case 1, 3: Strong rock pile material and weak rubble zone: If the observed failure surface was initiated by a deep weak layer then it is the indication of this analysis that reasonable material parameters of rubble/colluviums are as follows:

CSS Location Fixed:
Friction angle: 22.3 to 23.5°
Cohesion: 25 to 10 kPa.

CSS Location Variable: In this case the rock pile material parameters are fixed with a friction angle of 36° and a cohesion value of 15 kPa. It is worthy of note that the colluvium/rubble properties are consistent with the properties obtained by Norwest (2004). The rubble properties obtained for the assumed failure conditions are as follows:

Friction angle: 24°
Cohesion: 7 kPa

Case 2, 4: Homogeneous model: If the observed failure surface went entirely through the rock pile material then the reasonable properties of the rock pile material replicating assumed failure conditions are unrealistic. Therefore, this scenario is considered physically unrealistic.

It is important to note that that the slip surface ultimately passes through both original rock pile material and material dozed over the crest of the rock pile. The center of the crack passed through original rock pile material. It is unclear as to whether the values for the shear strength properties calculated from these analyses are representative of peak or residual strength values.

The following points summarize the findings:

- It appears unlikely that the crack observed on October, 2004, is due to a failure plane through rock pile material alone. If this were the case, the resulting model-determined soil parameters (ϕ = 18°, cohesion = 150kPa) differ significantly from the *in situ* testing program.
- It appears likely that the observed slope failure was the result of sliding along a deep-seated weak layer beneath the rock pile material and above the bedrock layers. When this hypothesis is considered, the resulting material parameters needed to produce failure conditions are consistent with existing field observations.

5 REFERENCES

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