Deloitte & Touche

Engineering Analysis of Vangorda Pit Wall Stability

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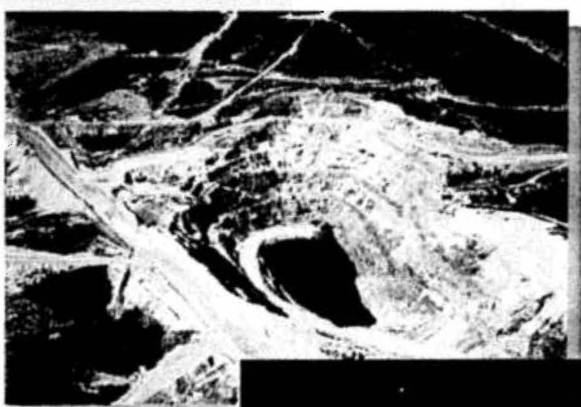

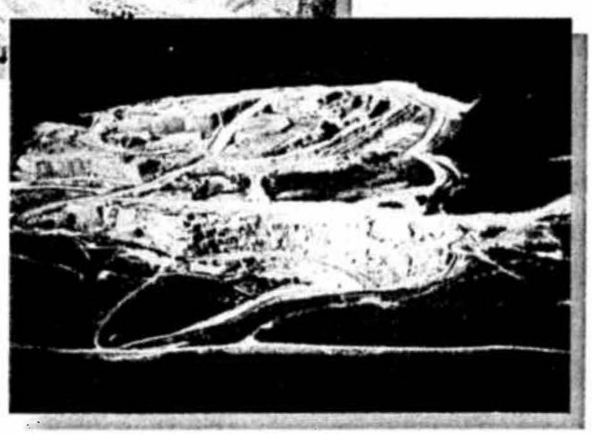

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ENGINEERING ANALYSIS OF VANGORDA PIT WALL STABILITY

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF APPENDICES

ENGINEERING ANALYSIS OF VANGORDA PIT WALL STABILITY

1. Introduction

The Anvil Range Mining Complex, located in Faro, Yukon, ceased operations in January 1998 when Anvil Range Mining Corporation filed for creditor protection under the Companies' Creditor Arrangement Act. Deloitte & Touche Inc. was appointed Interim Receiver of Anvil Range Mining Corporation ("Interim Receiver") on April 21, 1998. The Interim Receiver has overseen the management of the property under the terms of two water licences since that time.

The objective of this engineering analysis of the Vangorda pit walls is to provide the necessary data for the evaluation of alternatives for the Vangorda Creek diversion system and to develop short-term management options. The analysis has, therefore, focused mainly on the stability of the northwest wall and the pit bench that overlooks the flume.

The engineering analysis consisted of a review of documentation, which includes reports prepared by BGC Engineering (BGC), SRK Consulting (SRK), Piteau & Associates and Robertson Geoconsultants. A field inspection was carried out during August 2002 with the objective of gathering information on:

- The performance of the slope walls;
- The performance of the pit bench overlooking the diversion flume;
- The location and condition of fault structures;
- The characteristics of jointing and foliation in the pit wall;
- The characteristics of groundwater seepage, and
- The strength of the intact rock and the rock mass.

The data was evaluated using various rock classification schemes and field evaluation techniques to provide data for stability analyses. Analyses were carried out to assess the potential for circular failure of the entire slope, wedge failure along major structures and planar failure along the foliation planes. Assessments were also made of the potential for long-term instability related to seismicity, as well as the sensitivity of the pit walls to various levels of groundwater and pit flooding. Probabilistic analyses were carried out where appropriate, to assess the likely range of outcomes.

2. Data Review

A review of the existing documentation related to the Vangorda pit revealed useful data on pit instability during mine operations. There was very little data directly related to rock and rock mass strength parameters. Some useful data on foliation strength was obtained from the SRK (1989) report.

The relevant history of Vangorda pit wall movements during operations were documented in site visit reports by Piteau and Associates. These reports were summarized in the report by BGC (2002). The main points are presented below:

- The Vangorda open pit is 1,000m by 300m in area by 100m deep. The preliminary interramp slope angle was designed at 40°.
- Mining at the Vangorda pit started in 1990 but was discontinued between 1993 and late 1994. Mining restarted in 1994 and all operations stopped in 1998.
- Penetrative foliation was critical to maintaining stable slopes in the phyllite schist. Foliation dips 20° to the west with locally flatter and steeper areas due to different phases of folding.
- Faulting within the pit is characterized by primary west-dipping, low angle faults, with a few higher angle faults dipping to the west. A major steeply dipping, east-west trending fault, the Cross Fault, intersects the pit walls some 400m south of the northwest wall of the pit.
- Considerable sloughing and raveling were associated with the disturbed Cross Fault rock mass.
- Five slope movement prisms were installed between the 1130 and 1152 m elevations on the north and east walls of the pit in July 1992.

- In October 1992, some leakage was observed from the flume and considerable seepage was observed along the northern and western sides of the pit bottom. Some bench scale instability was experienced within the Cross Fault on the west wall corner of the pit wall. Seepage was observed from the crest of the wall above the flume down to the pit bottom in the area where the Northwest Fault intersects the pit wall. The locations of the Northwest Fault and Creek Fault are shown in Figure 1.
- A bench scale failure occurred on the west wall of Vangorda pit in October 1992, which was controlled by unfavorably intersecting rock joints and freeze-thaw action.
- No pit monitoring has been undertaken since the cessation of mining in 1998.
- During June 1999, a large rock fall occurred from the bench face overlooking the Vangorda Creek drainage flume. The rock fall damaged the flume. Seven sections of the flume were replaced. Remedial works were undertaken subsequently to remove the remaining unstable rock in the bench face.

The historical data on pit wall movements indicate that bench-scale instability had occurred in several instances in the past, but large-scale pit wall instability had not occurred. Instabilities had been associated with movement along rock joints and raveling or sloughing of the rock material.

3. Rock Mass Characterization

Part of the Vangorda Creek diversion runs along a bench on the north and northwest sides of the Vangorda pit. This bench, referred to as the upper bench is in the order of 20m wide.

A field inspection of the overall pit wall conditions and data collection for stability analyses was carried out during a site visit from August 10 to 16, 2002 by Dr. Gabriel (Essie) Esterhuizen, Rock Mechanics Engineer, and Mr. Tim Coote, Engineering Geologist. During this period, Mr. Jim Cassie of BCG came to the site to provide further input, since he had visited the site in 1999 and was familiar with the conditions at that time. A summary of the general observations of pit conditions and collected data is presented below.

The collected data was evaluated and strength parameters for the rock mass and the foliation planes were determined using various strength and failure criteria. The results of this evaluation are also presented below.

3.1 Upper Bench Conditions

The condition of the upper bench of the Vangorda pit that overlooks the diversion was inspected to assess the current stability, potential for further instability and the potential effect of instability on the flume. Particular attention was paid to the area where the flume crosses the Northwest Fault, since earlier reports indicated that movement may have occurred at this location.

Our inspection did not reveal any signs of recent movement at the location of the Northwest Fault. The flume was leaking at the location of the fault owing to a dislocation of some of the corrugated iron sections. The cause of the dislocation was not readily apparent, but may have been caused by frost heave or rolling rock damage.

The stability of the bench face was assessed by dividing it into a number of geotechnical zones, as shown in Figure 1. Each zone has similar geotechnical properties and is described in Table 1. Photographs showing the flume and the bench conditions in each zone are presented in Appendix A, Figures A1 to A6.

Table 1
Description of Upper Bench Stability Conditions

Zone	Geology	Stability	Potential Effect on Flume
1	Graphitic schist, slightly weathered to medium weathered, black with iron staining along foliation and fractures.	Slope dips at 40°, minor raveling, no rock falls visible.	Very low probability of rock falls impacting the flume owing to shallow dip of slope.
2	Zone bounded by Northwest and Creek Faults. Schist (phyllite) with massive texture. Medium to weak strength, some strong rock.	No unstable wedges or blocks. Rock is competent and unlikely to fail.	Very low probability of rock falls impacting flume.
3	Pellitic schist, slightly weathered to medium weathered, medium to weak rock. Active seeps. Foliation dips at 35° towards pit.	Potential planar failure along foliation. This area was blasted previously to achieve stability. Further failures are likely.	Moderate probability of block sliding and affecting flume.
4	Pellitic schist, slightly weathered to medium weathered, medium to weak rock. Active seeps. Foliation dips flatter than in Zone 3, at 20° towards pit.	Stable wall formed, no signs of prior instability. Large offset from flume forms catch area so that rock falls will not reach the flume.	Very low probability of rock falls affecting flume
5	Saturated soil, gravel silty material with minor saturated debris flow. Contains rock boulders up to 2m. Seepage observed.	Differential erosion of soil likely to loosen large boulders that may roll down the slope and impact the flume	High probability of rock falls impacting flume
6	Variable strength pellitic schist, some extremely weak, black with sulphides.	Toe of bench very close to flume, raveling may affect flume	Moderate probability of affecting flume
7	Pellitic schist, slightly weathered to medium weathered, medium to weak rock.	Low bench height and shallow slope angle.	Low probability of rolling rocks affecting flume

3.2 Pit Wall Conditions

The northwest wall of the Vangorda pit is approximately 80m high and has an overall slope angle of 38° to 42°. The conditions of the pit walls were inspected and it was found that the pit walls are generally stable in the northwest part of the pit. The pit walls consist of variable strength schists, varying from low strength graphitic schist to strong, relatively massive rock units. The dip of the foliation was highly variable in dip and strike orientation. The Northwest Fault and the Creek Fault could be identified in the pit wall, as well as isolated continuous joint structures. These structures were favorably oriented relative to the northwest pit wall and had not contributed to instability.

There were some indications of seepage from the pit walls, but the general impression was not of a saturated pit. Sloughing and raveling of the weaker graphitic schist appears to have occurred over the past number of years and has affected the individual pit benches. Sloughing along the Northwest Fault line was also observed. Figure 2 shows the northwest wall of the pit and points out these features. In the longer term (more than 50 years), sloughing is expected to continue and may ultimately work its way back to the location of the flume.

3.3 Rock Mass Characterization

Descriptions of the rock mass conditions were made using field data sheets, which allowed the Rock Mass Rating (RMR) system of Bieniawski (1989) to be applied. The RMR system, together with the Hoek-Brown failure criterion allows the large-scale strength of the rock mass to be estimated. The rock mass strength estimates from the field assessment are presented in Table 2.

The uniaxial compressive strength (UCS) of the rock types was estimated using the suggested field estimating technique of the International Society for Rock Mechanics (Brown, 1981). The results show that the graphitic and pellitic schist may be described as "Fair Rock" and the stronger schist rocks are "Good Rock."

Table 2
Summary of Rock Mass Characteristics at Vangorda Pit

Location	UCS (MPa)	RMR	Class	Description
Zone 1	5- 25	59	III – Fair Rock	Graphitic schist, dark, fissile
Zone 4	25	49	III – Fair Rock	Pellitic schist, fissile, wet
Zone 6	25	43	III – Fair Rock	Pellitic schist, blocky and rubbly
Zone 7	25 – 50	58	III – Fair Rock	Greyish-green pellitic schist
North wall of pit	50 – 100	79	II – Good Rock	Competent schist

3.4 Geological Structures

3.4.1 Faults

The Northwest Fault was easily discernible in the northwest face of the Vangorda pit. The fault zone is weaker than the surrounding rock and sloughing had resulted along the fault in the slope face. It was possible to identify the fault where it intersected the upper bench, and the flume. The fault represented a zone of weak rock, but the width was not readily discernible due to surface weathering. The orientation of the fault could not be measured directly in the field, but has been assumed to be accurately represented in earlier work as dipping between 55 and 60° in the direction of 103°.

The Creek Fault was also identified in the northwest slope face and in the upper bench. The fault zone is about 1m wide and contains weak gouge materials. The dip of the fault was measured as 65° and the dip direction taken from a map of the Vangorda pit geology as 080° (after D.L. Brown, PhD. thesis reproduced in BCG, 2000). The locations of the faults relative to the pit wall are shown in Figure 1.

3.4.2 Jointing

The dominant joint structure in the rock mass is the foliation, which has variable dip and dip direction in the northwest wall of the pit. Other joint sets are also present, but there were no obvious signs of failure along joint planes. To characterize the joint sets measurements were made of the average joint set orientations at each location that rock classification was carried out. A lower hemisphere equal angle plot of the poles to the measured jointing is presented in Figure 3. The jointing was grouped into three sets, as shown in the figure. Two steeply dipping joint sets are shown, indicated as Set 1 and Set 2. The mean orientation of set 1 is a dip of 75° in a direction of 218° and Set 2 dips at 86° in the direction 291°. The foliation set is indicated as Set 3, shown to be dipping shallowly at 16° in a direction of 168°.

3.4.3 Foliation

The foliation may be seen as a persistent weakness that may cause large scale failure of the pit walls. In the northern part of the Vangorda pit, the foliation dip is highly variable with an average dip of 16° dipping towards 170°, as shown in Figure 3. This orientation is unfavorable for stability of the northwest face of the pit, since plane failure could occur along the foliation. However, inspection of the pit walls did not reveal any signs of past failures along these planes. The historical records also did not indicate that failure had occurred by sliding along foliation.

Estimates of foliation strength (SRK, 1989) showed cohesion values of 0.04 to 0.14 MPa and friction angles of 21 to 25°. To supplement these results, field estimates of foliation strength were determined using the Barton & Choubey (1974) equations for joint strength. The range of parameters and resulting friction angle and cohesion values are presented in Table 3. It was assumed that the ranges of values represented 90% of the possible values. The confining stress was assumed to vary between 0 and 2 MPa across a failure surface. The resulting cohesion falls within the range previously estimated by SRK, but the friction angle is higher since it represents the peak friction angle, and not the residual angle presented in the SRK (1989) report.

Table 3
Strength Properties of Foliation

Property	Low	High	Average	Standard Deviation
Joint roughness (JRC)	7	12	9	-
Wall strength (MPa)	15	25	20	-
Residual friction (°)	20	30	25	-
Cohesion (MPa)	0.040	0.118	0.079	0.023
Friction angle (°)	25.6	42.0	33.8	5.0

3.5 Strength of the Rock Mass

The strength parameters of the rock mass were derived from the field data and limited data from studies carried out while the mine was in operation. The Hoek-Brown failure criterion (Hoek & Brown, 1988) was used to determine estimates of the rock mass cohesion and friction angle, based on the observed RMR values and estimates of intact rock strength. The Hoek-Brown failure criterion makes use of the Geologic Strength Index (GSI) (Hoek, 1994) which is equal to the RMR under dry conditions minus 5 points. GSI values were calculated for the rock mass at every location listed in Table 2. The average GSI value is shown in Table 4.

The Hoek-Brown criterion may be calculated for disturbed and undisturbed rock masses. For open pits, where the rock mass has been subject to blast vibrations and has much greater freedom of movement than in underground excavations, a "disturbed" rock mass is assumed. The strength of a "disturbed" rock mass, σ_m, may be calculated using the Hoek-Brown failure criterion, as follows:

$$m = m_i e^{\frac{100-GSI}{14}}$$

$$s = e^{\frac{100-GSI}{6}}$$

$$\sigma_m = \sigma_3 + \sigma_c \sqrt{m\sigma_3 / \sigma_c + s}$$

where m_i is a characteristic parameter of the rock type, σ_3 is the confining stress and σ_c is the strength of the intact rock. For foliated schist, the value of m_i was taken as 12.0 from published charts (Hoek & Karzulovic, 2000).

To calculate the cohesion and friction angle of the rock mass, the rock mass confining stress was assumed to vary between 0 – 2 MPa along the failure surface. To obtain representative cohesion and friction values, a straight line was fitted through the rock mass strength curve predicted by the Hoek Brown criterion. The cohesion, *c,* and friction angle, ϕ, is calculated as follows from the slope, *m,* and the intercept, *I,* of the straight line.

$$\phi = \arcsin\left(\frac{m-1}{m+1}\right)$$

$$c = i\left(\frac{1-\sin\phi}{2\cos\phi}\right)$$

The resulting rock mass strength curve and the fitted straight line are shown in Figure 4.

For the purpose of the slope analyses, it was assumed that the strength parameters followed a normal distribution. The standard deviations of the cohesion and friction angle are based on published values (Harr, 1987). The final parameters used in the analysis are summarized in Table 4.

Table 4

Rock Mass Strength Properties for Vangorda Pit Assessment

Property	Average	Standard Deviation
GSI	56	-
UCS (MPa)	50	-
Cohesion (MPa)	0.48	0.19
Friction angle (°)	39	3.9

4. Stability Analyses

The stability of the northwest wall of the Vangorda pit was evaluated by considering the potential for large-scale failure of the entire pit wall. Circular failure through the rock mass, large-scale plane failure along the foliation and wedge type failure along the fault structures were considered. Smaller scale bench failure that may be caused by jointing was not evaluated.

Stability analyses were carried out for the following scenarios:

- Current conditions, summer conditions in which the slope face is free to drain
- Maximum Credible Earthquake (MCE)
- Weak rock mass conditions (20% weaker than assumed for current conditions)
- Plane failure along foliation
- Plane failure for unfavorable (steeper) foliation dip
- High groundwater levels in the rock slope

In addition, an analysis was carried out in which the long-term stability of the pit walls under seismic loading was considered.

4.1 Circular Failure Analysis

Method of Analysis

The potential for large-scale circular failure of the pit slopes was evaluated using the program SLOPE/W Version 4.23, a commercial code produced by GEO-SLOPE International Inc. of Calgary, Alberta (1998). This program uses limit equilibrium theory to compute the factor of safety of earth and rock slopes. It incorporates search routines to determine the critical, lowest factor of safety failure surface. Circular failure analyses were performed to assess the potential for circular failure through the rock and soil materials.

Failure probabilities were calculated using the Slope/W ability to perform a Monte Carlo probabilistic analysis. The average and standard deviation of the cohesion and friction angle of the rock mass, as presented in Table 4, were used in the analyses. The standard deviation for these parameters quantifies the degree of uncertainty associated with them. The probability of failure is defined as the probability that the factor of safety is less than 1.0.

Geometry

The section geometry that was analyzed for the Vangorda pit was taken from the topographic map of the area. Since the map did not contain sufficient detail of the toe and crest of the slope, a stylized section was created for analysis. The section that was analyzed cut through the northwest pit wall and through the Vangorda Creek diversion, shown as Profile 1 in Figure 5.

Groundwater

Field observations showed that limited amounts of groundwater were seeping from the upper bench of the Vangorda pit slope. Otherwise, the pit walls did not appear to be saturated. For the purpose of analysis the phreatic surface in the rock mass was assumed to be determined by the water level in the pit. Since no data is available on the likely gradients of the phreatic surface behind the pit walls, a profile was assumed based on judgment and experience.

In winter conditions, the face of the slopes will be frozen and the groundwater is expected to rise since it will not be able to drain at the slope face. A "worst case" analysis was carried out in which it was assumed that the pit walls would become fully saturated under these conditions.

Seismic Loading

The seismic risk analysis showed that the peak acceleration associated with a seismic event having a 475-year return period is 0.05g and the maximum credible event (MCE) would produce a peak acceleration of 0.15g. The effect of seismicity was modeled as a horizontal outward acceleration in the slope stability analyses. The magnitude of the acceleration used in the analyses was 50% of the peak acceleration.

Circular Failure Analysis Results

The Slope/W results for circular failure of the Vangorda northwest wall are summarized in Table 5. Figure 6 shows the profile as analyzed using the Slope/W program.

Table 5
Vangorda Northwest Wall Stability Results for Circular Failure

Condition	Safety Factor	Failure Probability
Current (static load, low water table)	3.06	0.23%
High water table	2.48	1.60%
Reduced strength	2.45	0.73%
Pit flooded	3.40	0.65%
Seismic load (MCE)	2.76	0.29%

4.2 Plane Failure Analysis

Owing to the presence of well-developed foliation in the rock mass, plane sliding analyses were carried out using the algorithm published by Hoek & Bray (1981). The algorithm was modified to include the effects of horizontal seismic loading and was analyzed in a spreadsheet that allowed probabilistic evaluations to be carried out. For the analyses, it was assumed that the foliation planes were sufficiently continuous to result in large-scale failure of the entire rock mass.

The foliation strength parameters shown in Table 3 were used in the analyses. The dip of the foliation was assumed to be 25° with a standard deviation of 3°. This range of dip values is steeper than the dips measured during the site investigation, but reflects the range of dips observed in core drilling during earlier investigations, and may be considered a "worst case" analysis.

Results of the planar failure analyses are summarized in Table 6. These results show that, should continuous foliation planes exist, the failure probabilities would be higher than for circular failure. However, the rock mass is highly contorted and foliation dips were observed to be variable. The probability that planes would exist that were sufficiently continuous to result in large-scale pit wall failure was assessed to be very low. The risk of plane failure was therefore assessed to be very low.

Table 6
Vangorda Northwest Wall Stability Results for Planar Failure
(assuming the existence of continuous foliation planes)

Condition	Safety Factor	Failure Probability
Current (static load, low water table)	1.61	2.7%
High water table	1.37	8.3%
Reduced strength	1.23	6.27%
Steep foliation angle	1.48	0.2%
Seismic load (MCE)	1.29	5.6%

4.3 Wedge Failure Analysis

The potential for large-scale wedge failure along the intersection of the Northwest Fault and the Creek Fault was assessed using kinematic considerations. The orientations of the fault planes and the pit slope were evaluated using stereographic projections. Wedge failure is possible if the intersection line of the two fault planes is shallower than the slope face angle, dips in the same direction as the slope face and is steeper than the friction angle of the two planes. A lower hemisphere stereoplot is presented in Figure 7, shows that the line of intersection of the two faults dips steeper than the slope face angle. Therefore, although the two faults intersect in the slope face, this wedge is kinematically unable to fail.

The potential for the two faults to form a multi-plane block by combining with the foliation was also considered. The result of a block analysis is shown in Figure 8, which shows different views of a potential block. However, such a block would also be kinematically stable, since the two faults and the foliation would form a tapered block that is unable to slide in the direction of the free face. Figure 9 illustrates the block as it would appear in the slope face.

4.4 Fault Tree Analysis

The results of the above stability analyses have been incorporated into a fault tree analysis as part of an SRK study currently in progress, entitled "Risk-Based Design Criteria for Final Closure and Reclamation Plan, Faro Mine, Yukon Territory". The fault tree analysis considered all possible modes of pit wall failure, including circular failure, planar failure along foliation, wedge failure, toppling failure and surface erosion. For each failure mode, the failure probability as well as the probability of occurrence was considered. The results showed that the overall probability that the Vangorda Creek diversion will be breached by failure of the pit walls beneath the flume is 0.0025 or 1:400. The dominant failure mode is failure through the rock mass. A summary of the results is presented in Table 7.

Table 7
Estimated Combined Failure Probabilities of Vangorda Northwest Pit Wall

Failure Mode	Failure Probability
Failure through the rock mass	2.5E-3
Planar failure along foliation	2.8E-5
Wedge failure on major structure	5.9E-6
Toppling failure on major structure	1.0E-6
Surface erosion	1.0E-5
Total Probability	**2.5E-3**

4.5 Effect of Long-term Seismicity

The potential effect of seismicity on the stability of the pit walls depends on the elapsed time and the frequency-magnitude relationship of seismic events. The results presented for circular, wedge and plane failure all considered seismic loading by the maximum credible earthquake. However, over extended periods of time, other earthquakes will occur that may result in pit wall failure. The objective of this part of the study was to determine an estimate of the combined failure probability of all seismic events over various time periods.

The results of a seismic hazard analysis (SRK, 2002) were used to determine the probability that an event of a particular magnitude will be exceeded in a given time period. For the purpose of this calculation, time periods of 1, 10, 25, 50, 100 and 500 years were considered. As the time period increases, the probability of a large seismic event occurring also increases. This means that as one considers the stability of the pit over increasing lengths of time, the probability of it failing due to seismic loading will also increase.

A calculation was made for each time period, to assess the potential that the pit slope will fail as a result of seismicity. The calculation method and interim results are summarized in Appendix B.

The results are summarized in Figure 10, which indicates how the probability of failure of the slope increases with time owing to the increasing risk of seismicity. For comparison, the static failure probability of the slope is 0.23% (or 1:435) and if a time period of 500 years is considered, the probability increases to 0.245% (or 1:408). This increase is considered to be negligible and the seismic hazard is, therefore, insignificant to the stability of the pit walls over the longer term.

4.6 Upper Bench Stability

Since the Vangorda Creek diversion flume is sensitive to rock falls from the upper pit bench face, a rock fall hazard rating system developed for highways (Wyllie, 1987 in Kliche, 1999) was applied. The system takes into account the rock conditions, slope geometry and location of the roadway (flume in this case) relative to the slope face. The system was slightly modified, to exclude factors that are not applicable, such as sight distance and roadway width. The modified rating system has a maximum rating of 729 on a logarithmic scale. Details of the rating system and how the system was applied for the different zones is presented in Appendix C. The resulting ratings for the different zones of the upper bench are presented in Table 8 and graphically in Figure 11. The rockfall hazard was categorized as follows:

- 27 – 150 : Low hazard of rockfalls affecting flume;
- 150 – 300: Moderate hazard of rockfalls affecting flume;
- 300 – 600: High hazard of rockfalls affecting flume; and
- 600 –972: Very high hazard of rockfalls affecting flume.

It can be seen from the results that only Zone 5 has a "high hazard" rating and Zones 3 and 6 have "moderate hazard" ratings. The hazard ratings confirm our assessment of the likelihood of rockfalls impacting the flume, presented in Table 1. Zones with high and moderate ratings will require remedial works to ensure the flume functions in the long term. Areas with a "low hazard" rating are judged to be stable and will require no further action. Remedial actions that may be considered are flattening of the slope angles, removal of unstable rocks and creation of a catchment area adjacent to the flume. It should be noted that the rockfall that occurred in June 1999 is located in Zone 3 (Figure 1). This area has since been partially remediated (see Appendix A, Figure A3).

Table 8
Results of Rockfall Hazard Rating Assessment

Zone	Likely failure mode	Rating	Rockfall Hazard
Zone 1	Raveling, sloughing	123	Low Hazard
Zone 2	Rock falls	75	Low Hazard
Zone 3	Plane failure on foliation	222	Moderate Hazard
Zone 4	Rock falls	63	Low Hazard
Zone 5	Rolling rocks and erosion	315	High Hazard
Zone 6	Raveling and sloughing	168	Moderate Hazard
Zone 7	Raveling	96	Low Hazard

The above results show that the bench face overlooking the flume is likely to result in further disruption of flow and remedial actions will be required.

5. Discussion and Conclusions

Our observations of the conditions of the Vangorda open pit indicated that the northwest pit wall is currently stable. There were no signs of large-scale movements that may have occurred in the recent past. The pit benches below the Vangorda Creek diversion flume had been affected by raveling and sloughing, which is an on-going process. Sloughing was particularly noticed along the location of the Northwest Fault and is expected to encroach on the diversion flume over the medium term (i.e. next 50 years).

Analyses of the pit stability under current conditions indicated that the factor of safety against large-scale circular failure was relatively high at 3.06 with an expected failure probability of 0.23%, or 1:435.

An analysis of the potential for planar failure along the foliation resulted in failure probabilities of 2.7% under current conditions. However, the foliation is undulating and is unlikely to be continuous over the lengths required to result in massive failure. Therefore, the likelihood of failure along the foliation is negligible.

The Northwest Fault and the Creek Fault were assessed for the potential to form a large-scale wedge, but it was concluded that the resultant wedge cannot fail due to its geometry relative to the pit wall.

Fault tree analyses showed that the overall probability that pit wall failure will breach the Vangorda Creek diversion flume is about 0.25%or 1:400.

An assessment of the long-term seismic effects on pit stability was carried out and it was found that seismicity will have a negligible effect on the overall probability of a pit wall failure, even if a 500 year period is considered. The results showed that the probability of failure of the pit owing to seismicity over the next 500 years is 0.245% compared to 0.23% for the current static conditions.

The upper pit bench that overlooks the Vangorda Creek diversion flume represents a hazard to the operation of the flume since rock falls or sloughing may impact the flume or result in blockages. Two sections were identified which will require remedial work to ensure that the flume will not be affected by instability of this bench.

Remedial work would include slope flattening, removal of loose boulders and creation of catchment space adjacent to the flume.

This report, **1CD003.15 - Engineering Analysis of Vangorda Pit Wall Stability**, was prepared by:

STEFFEN ROBERTSON AND KIRSTEN (CANADA) INC.

per Dr. Gabriel Esterhuizen
Principal Mining Engineer

6. References

Reports

1. BGC Engineering Inc., 2002. Draft report on Evaluation of Short Term Management Options Vangorda Diversion Flume, Faro Mine, Yukon.
2. McLenehan R., 1987. Site Investigation of the Vangorda Pit Area, Memo.
3. Piteau Associates, 1990. Geotechnical Review of Faro and Vangorda Pits, Letter Report to Curragh Resources.
4. Piteau Associates, 1992. Geotechnical Review of Faro and Vangorda Pits. Letter Report to Curragh Resources.
5. Robertson Geoconsultants Inc., 1996. Anvil Range Mining Complex – Integrated Comprehensive Abandonment Plan. Report to Anvil Range Mining Corp.
6. Steffen Robertson & Kirsten, (B.C.) Inc., 1989. Vangorda Plateau Development, Initial Environmental Evaluation, Report to Curragh Resources.
7. SRK Consulting, 2002. Report No. 1CD003.10 Risk-Based Design Criteria for Final Closure and Reclamation Plan, Faro Mine, Yukon Territory, October.

Other Publications

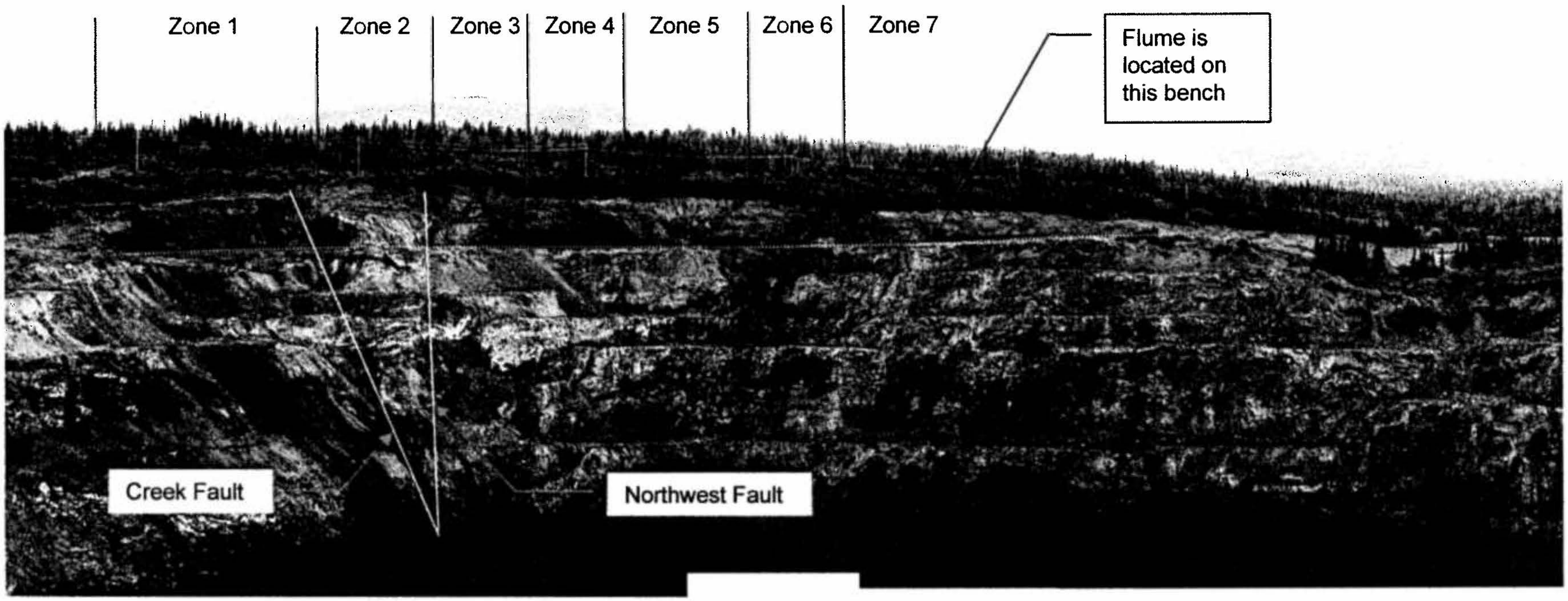
1. Barton, N. & Choubey, V., 1977. The Shear Strength of Rock Joints in Theory and Practice, Rock Mech. V. 10 pp. 1-54.
2. Bieniawski, Z.T., 1989. Engineering Rock Mass Classifications. John Wiley, New York.
3. Brown, E.T. (ed.), 1981. Rock Characterization, Testing and Monitoring – ISRM Suggested Methods. Oxford, Pergamon, pp. 171 – 183.
4. Geo-Slope International Inc., 1998. SLOPE/W Version 4.23, Calgary, Alberta.
5. Harr, M.E., 1987. Reliability Based Design in Civil Engineering, McGraw Hill.
6. Hoek, E., 1994. Strength of Rock and Rock Masses. ISRM News Journal. V2 No. 2, pp. 4-16.
7. Hoek, E. & Bray, J.W., 1981. Rock Slope Engineering, Revised Third Edition, IMM, London.

8. Hoek, E. & Brown, E.T., 1988. The Hoek-Brown Failure Criterion – a 1988 Update. 15th Canadian Rock Mech. Symp. (ed. J.C, Curran), University of Toronto, pp. 31-38.

9. Hoek, E.& Karzulovic, A,. 2000. Rock-Mass Properties for Surface Mines. Slope Stability in Surface Mining, (eds. Hustrulid, McCarter & van Zyl), SME, Denver, pp. 59-69.

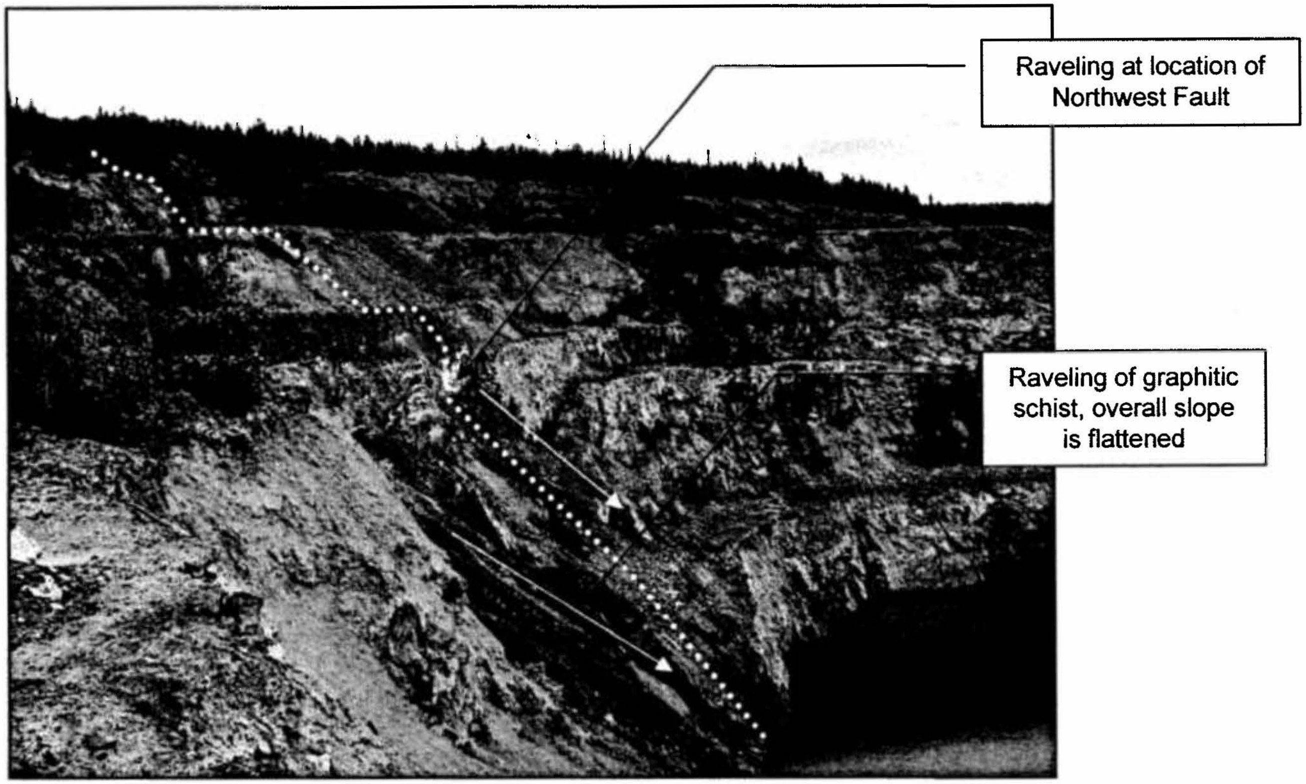
10. Kliche, C.A., 1999. Rock Slope Stability. Society for Mining Metallurgy and Exploration Inc.

Figures



Northwest Face of Vangorda Pit, August 10, 2002

SRK Consulting Engineers and Scientists			FIGURE 1 View of Vangorda pit northwest face showing zoning of upper bench and location of faults
PROJECT NO. 1CD003.15	DATE April 2003	REVISION B	



Raveling of benches has resulted in flattening of overall slope. In the long term (50 years +), progressive raveling is likely to encroach on the flume.

SRK Consulting Engineers and Scientists			FIGURE 2 Raveling of northwest wall of Vangorda pit
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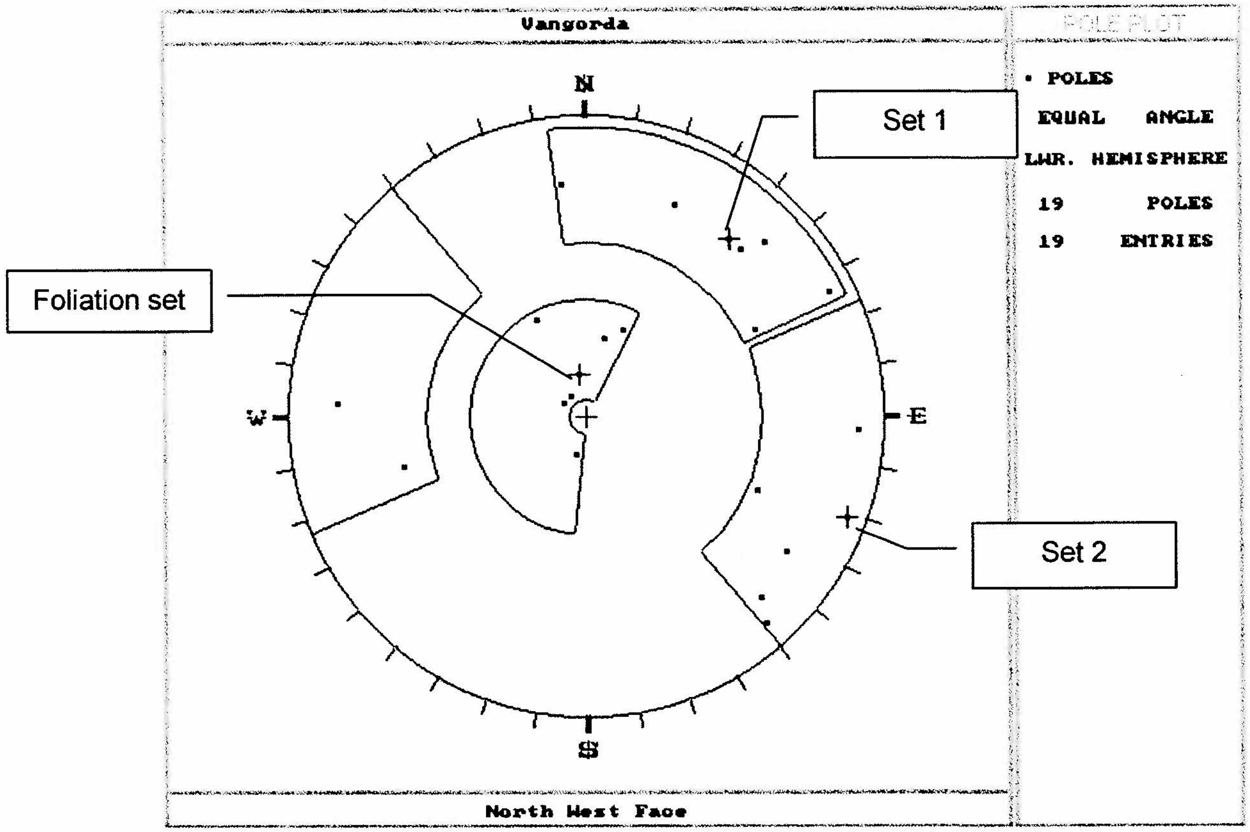

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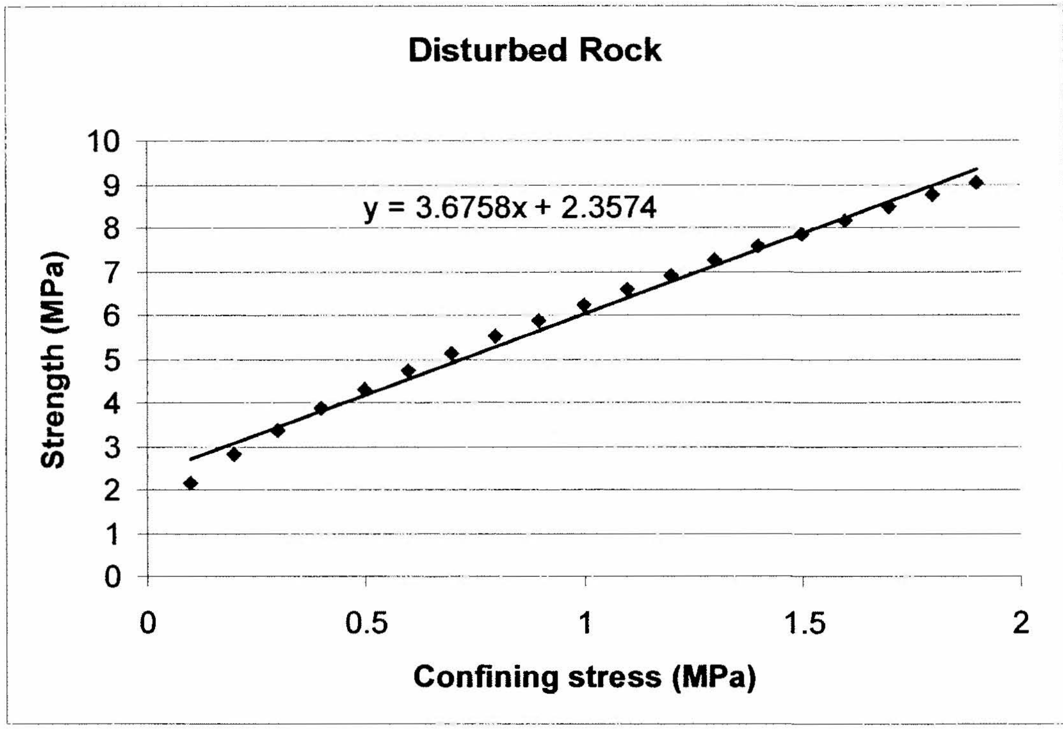
PROJECT NO.	DATE	REVISION
1CD003.15	April 2003	B

FIGURE 3
Vangorda pit lower hemisphere plot of poles to joint sets (each point represents the average orientation of a group of joints)



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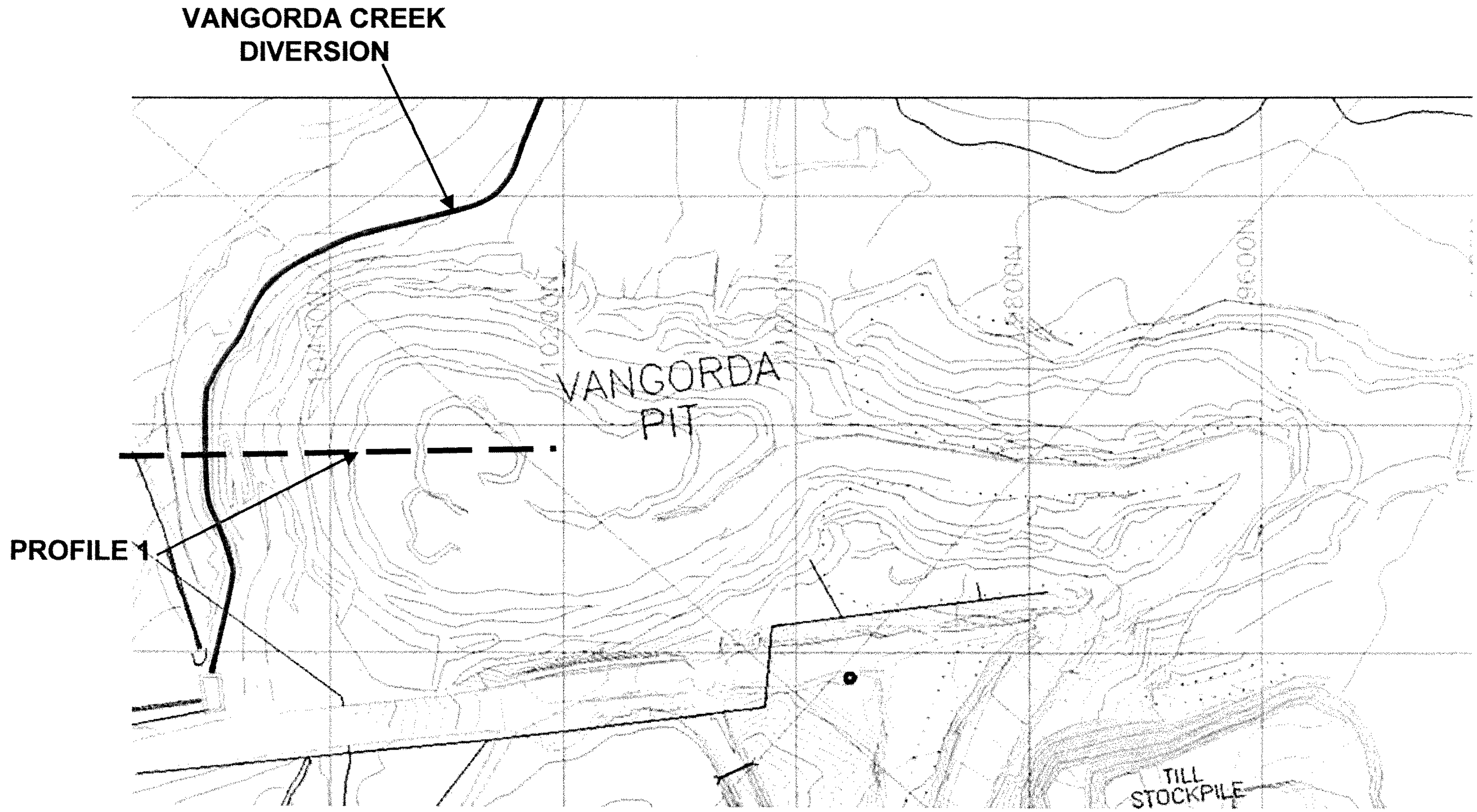
PROJECT NO.	DATE	REVISION
1CD003.15	April 2003	B

FIGURE 4
Strength curve for the rock mass in the northwest wall of the Vangorda pit



North

200 m

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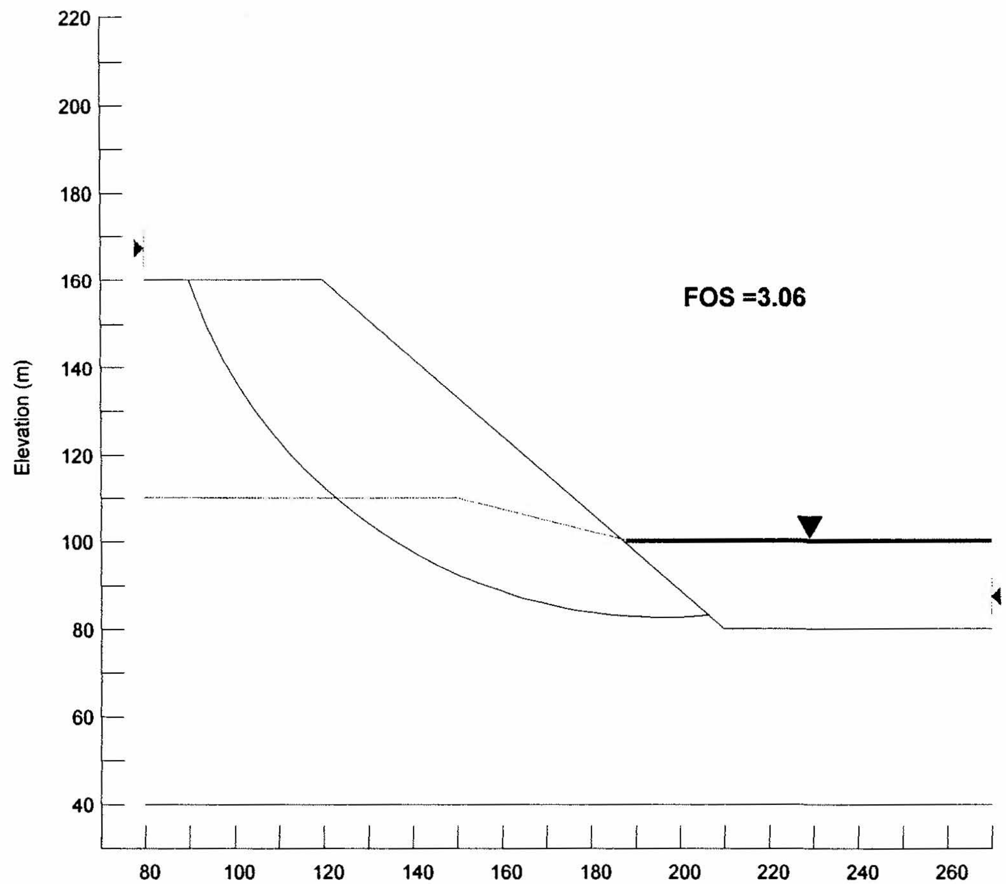
PROJECT NO.	DATE	REVISION
1CD003.15	April 2003	B

FIGURE 5
Vangorda pit showing location of the Vangorda Creek diversion and Profile 1 selected for analysis



	Description	Soil Model	Unit Weight	Cohesion	Phi	Piez. Line #
Soil 1	Rockmass	Mohr-Coulomb	24 (SD=0)	480 (SD=192)	39 (SD=3.9)	1 (SD=0)

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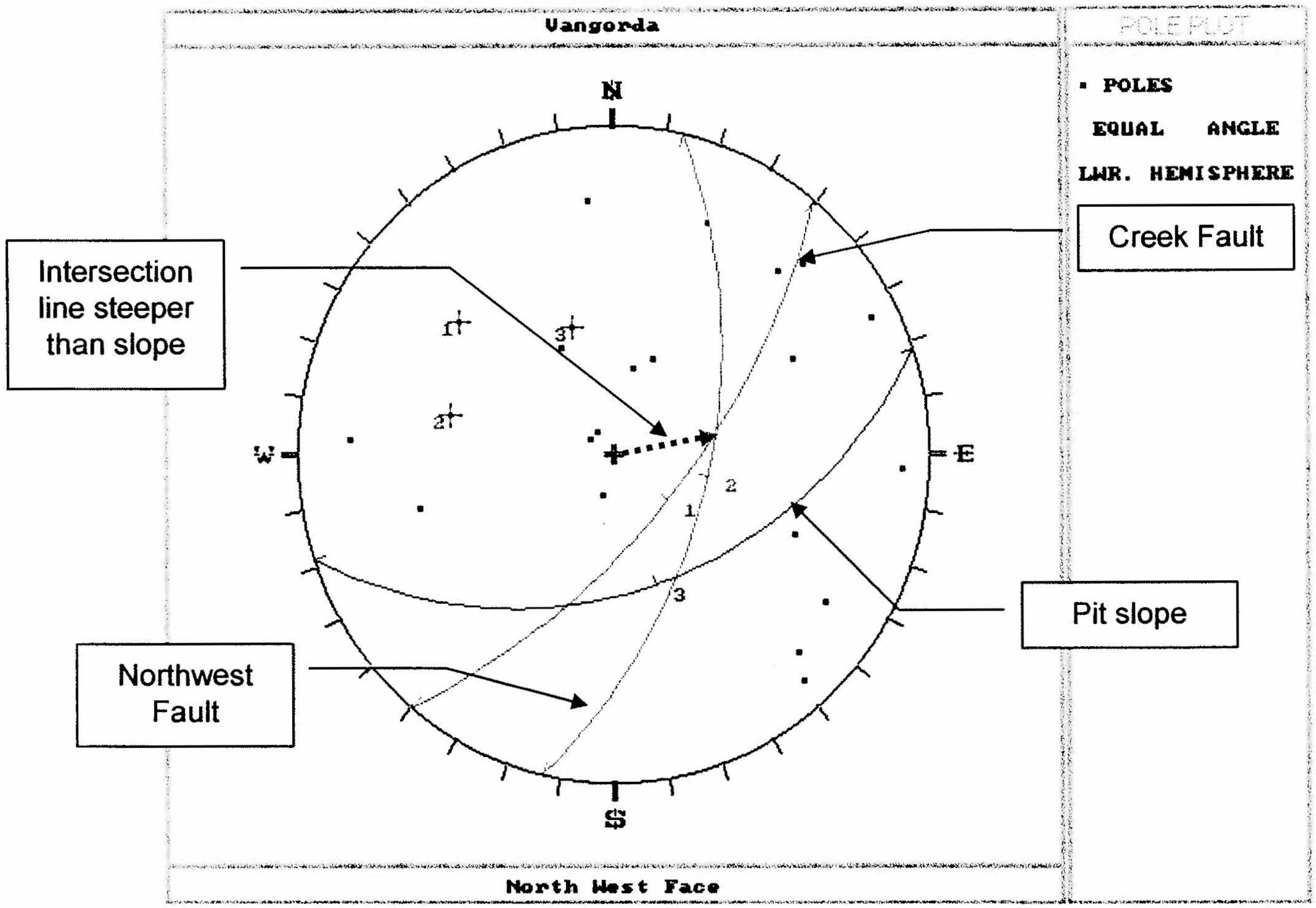
PROJECT NO.	DATE	REVISION
1CD003.15	April 2003	B

FIGURE 6

Cross section for analysis of Vangorda pit northwest wall



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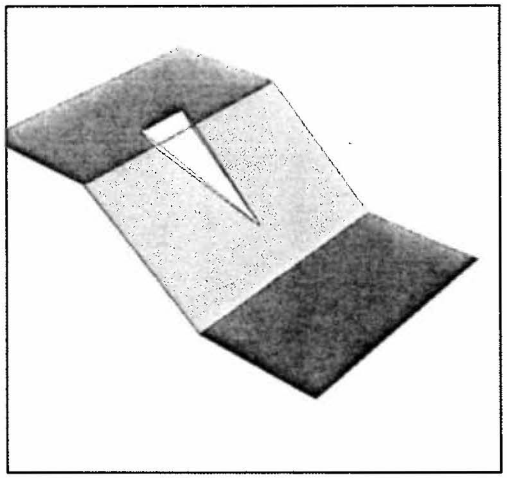
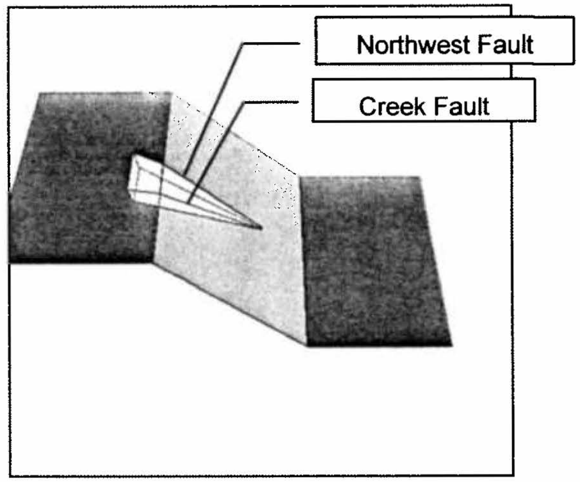
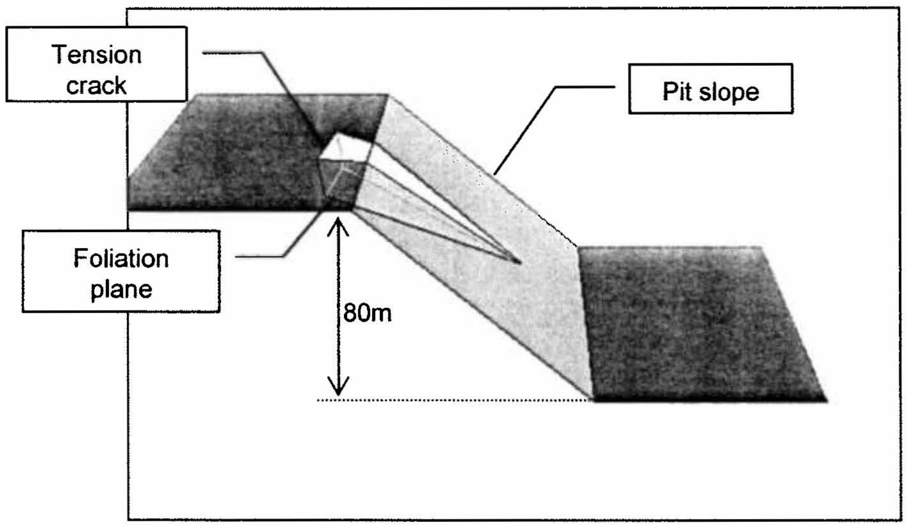
PROJECT NO.	DATE	REVISION
1CD003.15	April 2003	B

FIGURE 7
Lower hemisphere stereoplot showing Northwest Fault, Creek Fault and the northwest face of the Vangorda pit







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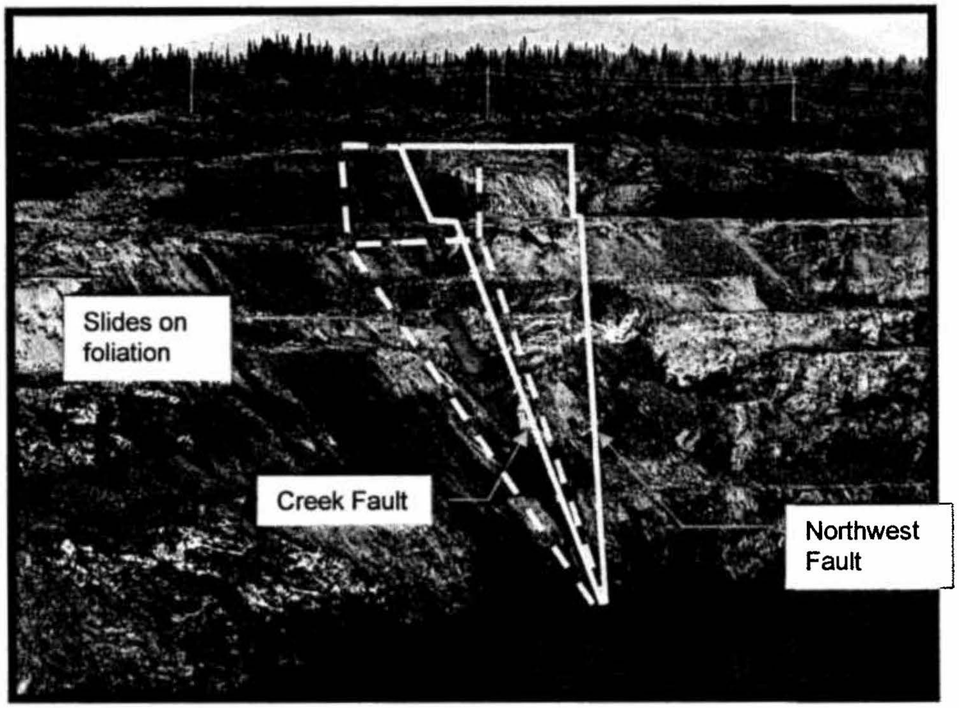
PROJECT NO.	DATE	REVISION
1CD003.15	April 2003	B

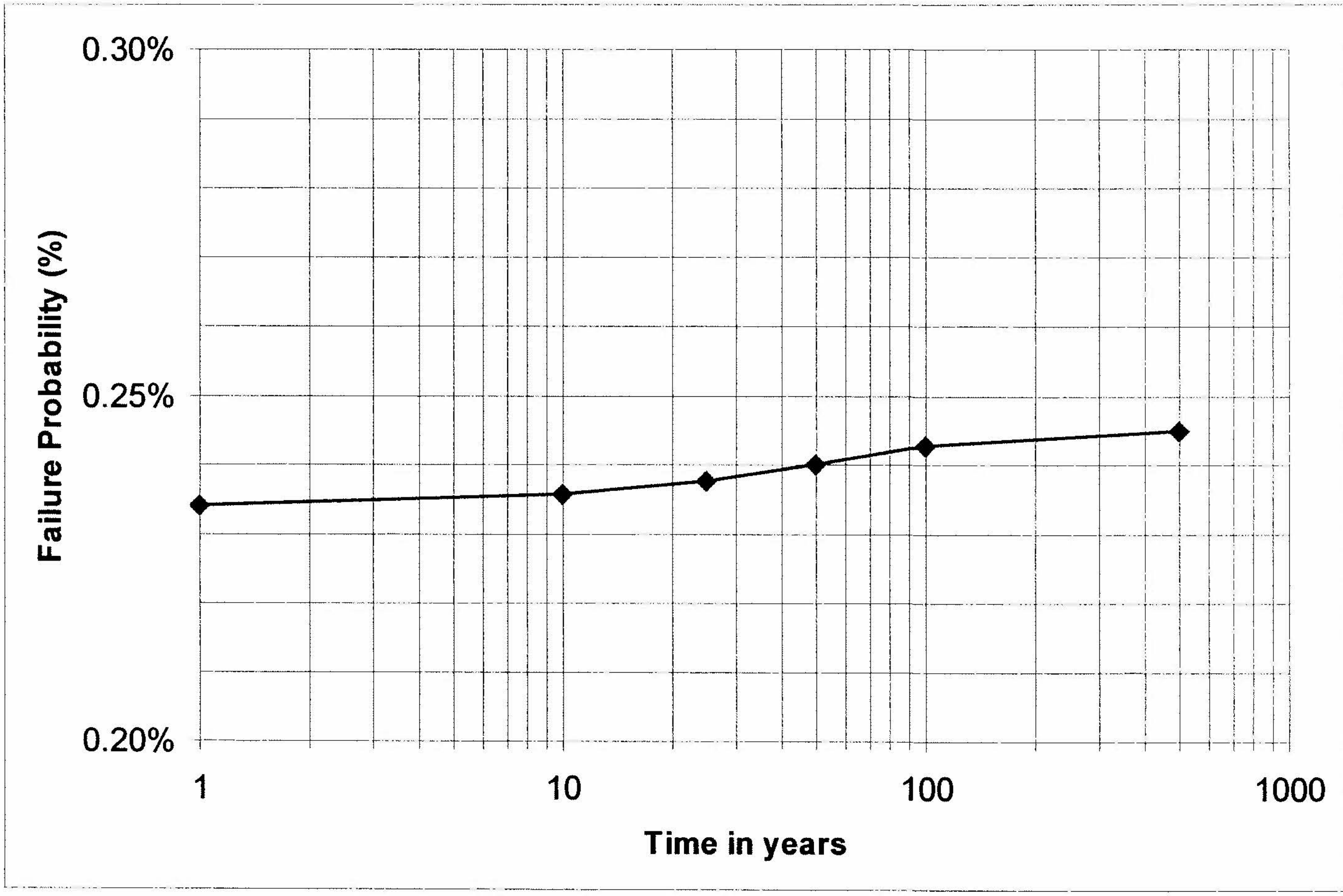
FIGURE 8

Views of potential block formed by Creek Fault, Northwest Fault and foliation planes

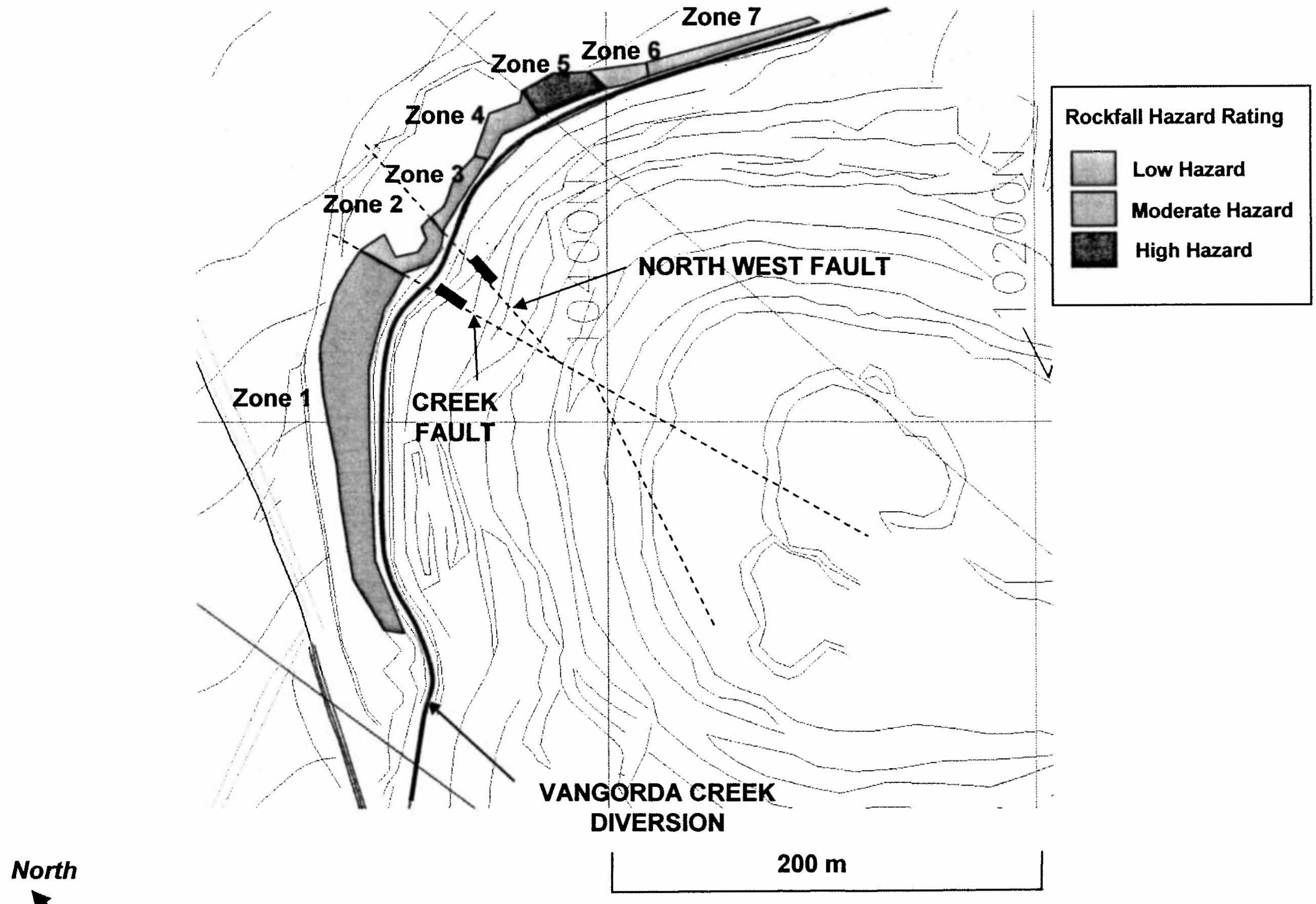
Stability of potential wedge formed by Northwest Fault and Creek Fault – with foliation as sliding plane.
Analysis shows that failure is not kinematically possible – the wedge is tapered towards the free face of the slope and cannot move outwards.



SRK Consulting Engineers and Scientists			FIGURE 9 Vangorda pit showing potential block formed by the Creek Fault, Northwest Fault and foliation planes
PROJECT NO. 1CD003.15	DATE April 2003	REVISION B	



SRK Consulting Engineers and Scientists			FIGURE 10 Effect of seismicity on failure probability over time
PROJECT NO. 1CD003.15	DATE April 2003	REVISION B	



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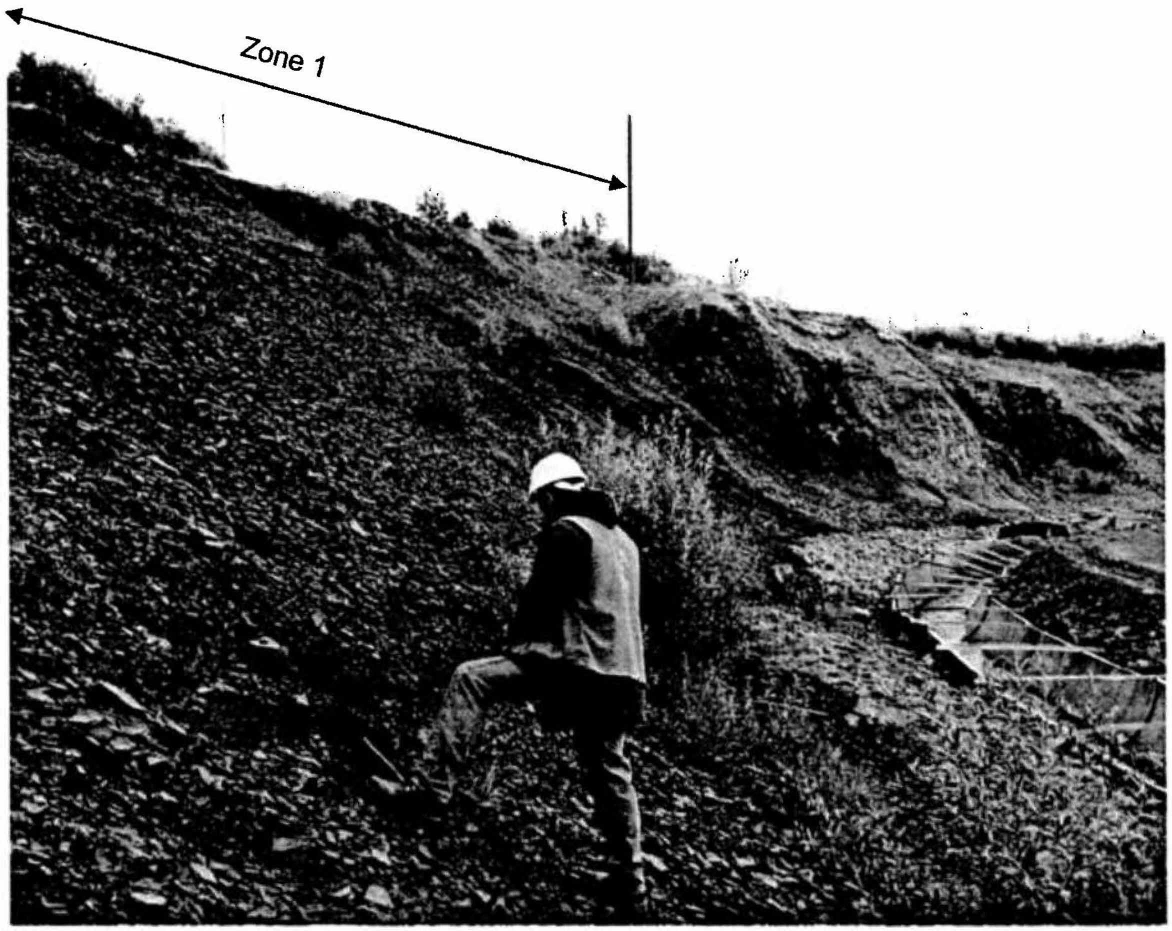
PROJECT NO.	DATE	REVISION
1CD003.15	April 2003	B

FIGURE 11
Rockfall Hazard Rating for the different zones of the upper bench

Appendix A

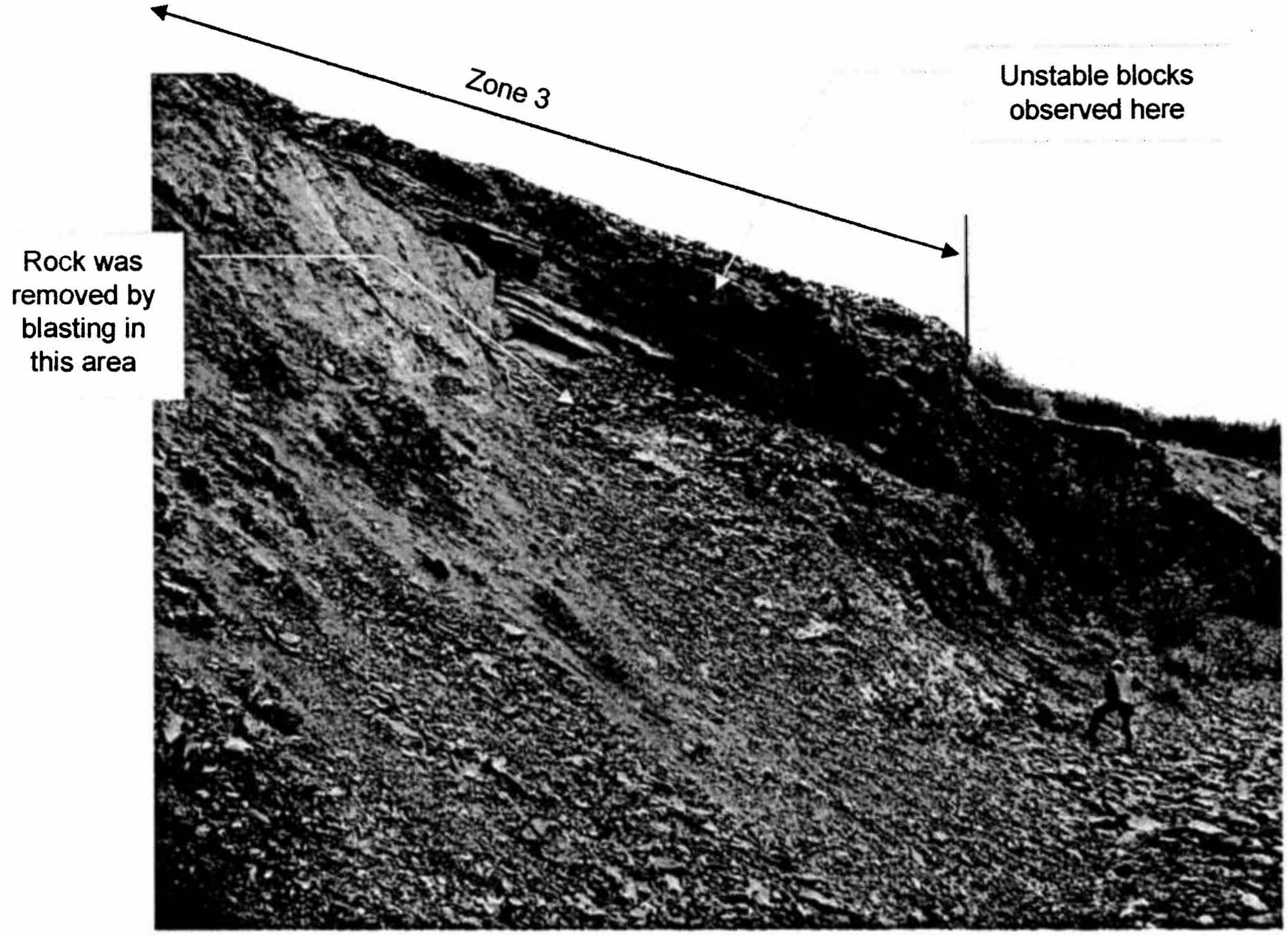
Photographs of Upper Pit Bench

Figures A1 to A6 present photographs taken during the site visit which took place on 10 & 11 August 2002. The photographs illustrate the different zones that are referred to in the report and contain comments on observed conditions.



Zone 1: Schist – slopes at 40 degrees. Surface ravelling but no signs of recent instability. Low risk of failure affecting the flume.

SRK Consulting Engineers and Scientists			FIGURE A1 Zone 1
PROJECT NO. 1CD003	DATE April 2003	REVISION B	



Zone 3: Pellitic schist with strongly developed foliation planes dipping at 35 degrees, forming failure surfaces. Active seepage. Moderate risk of failure affecting the flume.

SRK Consulting Engineers and Scientists			FIGURE A3 Vangorda pit
PROJECT NO. 1CD003	DATE April 2003	REVISION B	

Zone 5: Talus-soil slope, saturated, containing boulders up to 2m. Erosion and saturated debris flows. Considerable erosion over last two years noted by BGC. Slope at 40 degrees. High risk of further soil slumps and rolling boulders affecting the flume.

SRK Consulting Engineers and Scientists			FIGURE A5 Zone 5
PROJECT NO. 1CD003	DATE April 2003	REVISION B	

Appendix B

Long-term Seismic Hazard Analysis

APPENDIX B

Long Term Seismic Hazard Analysis

The potential effect of seismicity on the stability of the pit walls depends on the elapsed time and the frequency-magnitude relationship of seismic events. The seismic acceleration associated with an event that has a 10% probability of exceedence in 50 years (1 in 475-year return period) is widely used for design purposes. The acceleration associated with the maximum credible earthquake (MCE) may also be determined from an event with a return period of 1:10,000 years. The results of a seismic hazard analysis of the Anvil Range area are summarized in Table B1 below, from SRK Consulting report 1CD003.10, October 2002. Figure B1 shows the distribution of the probability of exceedence for different time periods from this report. These data were used to assess the long-term effects of seismicity on pit wall stability.

Table B1
Probabilistic Assessment of the Peak Ground Acceleration

Return Period [years]	Prob. Exceedence in 50 yrs.	Acceleration [g]
1:475	10%	0.05
1:10,000	0.5%	0.15

For long term stability considerations, such as mine closure, the cumulative failure probability over long time periods may be calculated from the distribution of seismic accelerations. The calculation was carried out by first determining the probability that a seismic acceleration would occur in a given intervals for the different time periods shown in Figure B1. For this calculation, intervals of 0.01 g were selected. The resulting probabilities are shown in Figure B2. This figure shows, for example, that the probability of an acceleration of between 0.014 and 0.015 occurring in a period of 1 year is 3.6×10^{-5}, while the probability in a period of 100 years is 3.5×10^{-3}, which is roughly one hundred times greater. The next stage involved determining the failure probability of the pit for each of the acceleration intervals. This was obtained by modeling the Vangorda pit with the Slope/W software and applying different seismic accelerations in the range of 0.02g to 0.30g.

The resulting failure probabilities are shown in Figure B3. A curve was fitted through the points to allow failure probabilities to be determined for any value of the seismic acceleration. The fitted curve also smoothes the results.

To calculate the combined failure probability of events in all intervals, it was assumed that the seismic events are independent events. The joint failure probability of the independent events was calculated as follows:

$$P = 1-(1-P_1)\cdot(1-P_2)\cdot(1-P_3)\cdot(1-P_4)\ldots(1-P_n)$$

Where P_i is the probability of the individual event occurring. In this case the P_i values are given by the probability of failure of the pit wall if subject to the seismic acceleration in the interval. The P_i values were calculated as:

P_i = Probability that seismic event will occur in interval x Probability of failure of pit slope when subject to seismic event

For example, the probability that the pit will fail in a period of 50 years owing to a seismic event with acceleration of between 0.014 and 0.015 is:

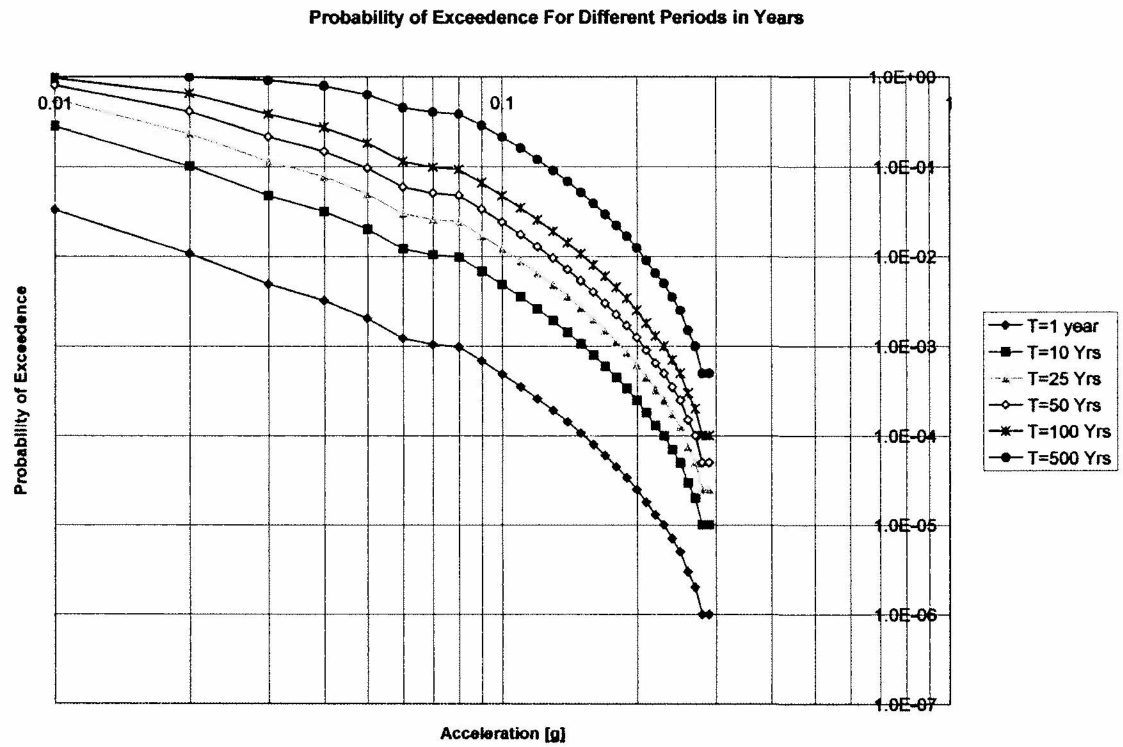
$$P_{0.014\text{-}0.015} = (1.79 \times 10^{-3}) \cdot (0.0029)$$
$$= 0.0005 \text{ or } 0.05\%$$

By accumulating all the probabilities for the different seismic intervals, the joint failure probability is obtained. The results are shown in Table B2 and illustrated in Figure B4. The results show that owing to the increasing potential for seismicity with time, the failure probability increases from 0.23% for current static conditions to 0.245% after 500 years. Should the time period be extended to infinity, the failure probability will be 0.29%, which is the probability of failure associated with the MCE.

Table B2

Failure Probabilities Due to Seismic Loading for Different Time Periods

Earthquake Acceleration (g)		Seismic Coefficient (g)	P(f)	Failure probability over time					
From	To			T=1 year	T=10 Yrs	T=25 Yrs	T=50 Yrs	T=100 Yrs	T=500 Yrs
0.00	0.01	0.005	0.0023	0.2262%	0.1670%	0.1006%	0.0433%	0.0080%	0.0000%
0.01	0.02	0.010	0.0024	0.0054%	0.0439%	0.0795%	0.0950%	0.0730%	0.0011%
0.02	0.03	0.015	0.0024	0.0014%	0.0131%	0.0291%	0.0480%	0.0656%	0.0197%
0.03	0.04	0.020	0.0025	0.0004%	0.0040%	0.0094%	0.0170%	0.0278%	0.0281%
0.04	0.05	0.025	0.0025	0.0003%	0.0029%	0.0069%	0.0130%	0.0228%	0.0404%
0.05	0.06	0.030	0.0025	0.0002%	0.0020%	0.0049%	0.0095%	0.0175%	0.0458%
0.06	0.07	0.035	0.0026	0.0000%	0.0005%	0.0011%	0.0022%	0.0041%	0.0130%
0.07	0.08	0.040	0.0026	0.0000%	0.0002%	0.0004%	0.0007%	0.0014%	0.0047%
0.08	0.09	0.045	0.0027	0.0001%	0.0008%	0.0019%	0.0038%	0.0073%	0.0262%
0.09	0.10	0.050	0.0027	0.0001%	0.0005%	0.0013%	0.0026%	0.0051%	0.0200%
0.10	0.11	0.055	0.0027	0.0000%	0.0004%	0.0009%	0.0018%	0.0035%	0.0149%
0.11	0.12	0.060	0.0028	0.0000%	0.0003%	0.0007%	0.0013%	0.0026%	0.0113%
0.12	0.13	0.065	0.0028	0.0000%	0.0002%	0.0005%	0.0009%	0.0018%	0.0084%
0.13	0.14	0.070	0.0029	0.0000%	0.0001%	0.0003%	0.0007%	0.0014%	0.0064%
0.14	0.15	0.075	0.0029	0.0000%	0.0001%	0.0003%	0.0005%	0.0010%	0.0049%
Cumulative failure probability				0.234%	0.236%	0.238%	0.240%	0.243%	0.245%



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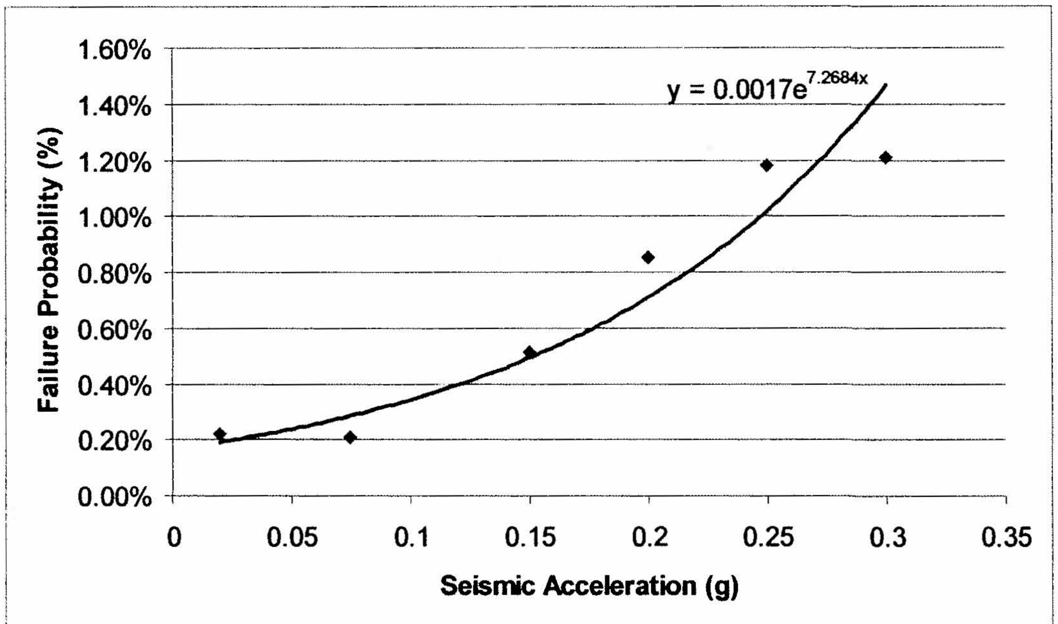
PROJECT NO.	DATE	REVISION
1CD003	April 2003	B

FIGURE B1
Probability of exceedence for different periods



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PROJECT NO.	DATE	REVISION
1CD003	April 2003	B

FIGURE B3
Failure probabilities of Vangorda pit under various seismic accelerations for partly drained (current) conditions

Appendix C

Rock Fall Hazard Assessment

APPENDIX C
Rock Fall Hazard Assessment

The potential for rock falls to affect the Vangorda Creek diversion flume was assessed visually during a site inspection in August 2002. A rock fall hazard rating system was applied to present the observations in a systematic manner. The rating system used was developed for the Oregon Department of Transportation to assess the rock fall hazard for highways (Wyllie, 1987 as presented in Kliche, 2000). The system consists of a preliminary rating to identify areas of potential rock falls and a detailed rating to prioritize the rock fall hazard. The detailed rating system was applied to rate the bench face overlooking the Vangorda Creek diversion. The detailed rating system considers 12 categories of risk, each having a rating score of 3 points to 81 points, resulting in a maximum of 972 points. Some of the categories do not apply to the diversion flume assessment, such as vehicle risk and decision sight distance, which refer to the traffic risks. The modified system used consisted of the following categories:

- Slope height
- Catchment effectiveness
- Geologic structures
- Rock friction
- Differential erosion features
- Differential erosion rates
- Potential rock fall size
- Climate

Table C1 presents a list of the factors used in the assessment and a brief description of the ratings. Table C2 shows how the different zones were rated as well as the total ratings. The location of each zone and a brief description of the zones is presented in the main section of this report and photographs of the zones are presented in Appendix A. Since Zone 5 is not strictly a rock slope, the ratings for geological structures and friction were applied as moderately unfavorable at 27 points for each factor. For the purpose of this report, the ratings were grouped into four categories as listed below:

- 27 – 150 : Low hazard of rock falls affecting flume;
- 150 – 300: Moderate hazard of rock falls affecting flume;
- 300 – 600: High hazard of rock falls affecting flume; and
- 600 – 972: Very high hazard of rock falls affecting flume.

It can be seen from the results that only Zone 5 has a "high hazard" rating and Zones 3 and 6 have "moderate hazard" ratings. Zones with high and moderate ratings are assessed to require remedial works to ensure the long term functioning of the flume. Areas with a "low hazard" rating are judged to be stable and will require no further work. Remedial actions that may be considered are flattening of the slope angles, removal of unstable rocks and creation of a catchment area adjacent to the flume.

Table C1
Hazard Rating Scores (after Kliche, 1999)

	Rating Score			
Category	3 Points	9 Points	27 Points	81 Points
Slope height	25 ft	50 ft	75 ft	100 ft
Ditch effectiveness	Good catchment	Moderate catchment	Limited catchment	No catchment
Average vehicle risk	25% of the time	50% of the time	75% of the time	100% of the time
Percent of decision sight distance	Adequate sight distance; 100% of low design value	Moderate sight distance; 80% of low design value	Limited sight distance; 60% of low design value	Very limited sight distance; 40% of low design value
Roadway width including paved shoulders	44 ft	36 ft	28 ft	20 ft
Geologic character, case 1: structural condition	Discontinuous joints; favorable orientation	Discontinuous joints; random orientation	Discontinuous joints; adverse orientation	Continuous joints; adverse orientation
Geologic character, case 1: rock friction	Rough, irregular	Undulating	Planar	Clay infilling or slickensided
Geologic character, case 2: differential erosion features	Few differential erosion features	Occasional differential erosion features	Many differential erosion features	Major differential erosion features
Geologic character, case 2: difference in erosion rates	Small difference	Moderate difference	Large difference	Extreme difference
Block size or quantity of rockfall per event	1 ft or 3 yd^3	2 ft or 6 yd^3	3 ft or 9 yd^3	4 ft or 12 yd^3
Climate and presence of water on slope	Low to moderate precipitation; no freezing periods; no water on slope	Moderate precipitation or short freezing periods or intermittent water on slope	High precipitation or long freezing periods or continual water on slope	High precipitation and long freezing periods, or continual water on slope and long freezing periods
Rockfall history	Few falls	Occasional falls	Many falls	Constant falls

Table C2

Rock Fall Hazard Ratings

Section	Height (ft)	Catch-ment	Geologic structures	Friction	Erosion Features	Erosion Rate	Fall volume per event	Climate Rating	Fall history	Rating	Class	Risk Level
Zone 1	30	Limited	DR	P	O	M	rubble	L	O			
Points	3	27	9	27	9	9	3	27	9	123	1	Low
Zone 2	45	Moderate	DR	U	F	S	rubble	L	F			
Points	9	9	9	9	3	3	3	27	3	75	1	Low
Zone 3	45	Moderate	CA	P-U	F	S	3ft	L	C			
Points	9	9	81	6	3	3	3	27	81	222	2	Moderate
Zone 4	45	Good	DF/R	P-U	F	S	rubble	L	F			
Points	9	3	6	6	3	3	3	27	3	63	1	Low
Zone 5	45	Limited	-	-	MJ	E	>4ft	L	M			
Points	9	27	27	27	81	81	9	27	27	315	3	High
Zone 6	25	None	DF	PU	O	L	rubble	L	O			
Points	3	81	3	6	9	27	3	27	9	168	2	Moderate
Zone 7	20	Moderate	DF	PU	O	L	1-2ft	L	O			
Points	3	9	3	6	9	27	3	27	9	96	1	Low

Explanation:

Geologic structure:	DR = discontinuous/random joints CA = Continuous joints/adverse orientation DF = Discontinuous joints/favorable orientation R = Random orientation	Erosion Rate:	M = Moderate difference S = Small difference E = Extreme difference L = Large difference
		Climate Rating:	L = Long freezing periods
Friction:	P = Planar U = Undulating	Fall history	F = Few C = Constant falls M = Many falls
Erosion features:	F = Few MJ = Major O = Occasional		