Probabilistic pit slope design in the Limpopo metamorphic rocks at Venetia Mine

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Synopsis

The Limpopo Metamorphic Belt is a complex geological terrane with at least three major deformational events. Tectonically juxtaposed lithology, open to isoclinal folding, crosscutting and re-activated shear zones and metamorphic gneisses and schists make pit slope design and maintenance a risky business. Venetia diamond mine's Cut 3 is planned to reach 360 m by 2011. The difficult rockmass created problems in the original pit and a new more comprehensive slope design was necessary. Nearly 9 km of geotechnical drilling were undertaken to produce a new geological model and provide geotechnical information near the final Cut 3 face. All existing data from the pit were brought together into a database. SRK Consulting were brought into the project to audit the process and provide most of the manpower for the data analysis. Existing failures were back-analysed and the geotechnical domains identified. Analyses of only the southern domains are discussed in this paper. Geotechnical engineers can design slopes at any angle, but it is up to the managers at the mine to decide what risk they are willing to accept in a slope design. Once the issue of risk was discussed and settled in respect of the consequences of failure, each of the main geotechnical domains were analysed to produce a slope design at 5% probability of failure with 15% above the first ramp. A risk assessment of the design was then undertaken to determine what risk the design placed on the equipment, personnel and the profitability of the overall mining operation. The risk assessment indicated minimal risk to the operation. One of the main assumptions of the final design is that the pit slopes will be dewatered. A project was immediately put in place to determine a groundwater model for the mine and to thereby determine the best dewatering procedure. However, it should be obvious that it is practically impossible to prevent benches from getting briefly saturated with water during storm conditions. It was therefore necessary to look specifically at supporting the crests of the ramps in the problematic southern domains to guard against the possibility of crest failure during heavy rains. The data available were re-analysed toward this objective. Spiles were recommended as the most cost-effective support system, and a probabilistic approach was used in the design. The final design has a 5% probability of failure, and the analyses indicate a strong sensitivity of the required spile spacing to the strength of the reinforcement material that will eventually be used. The pit slope parameters derived as per the discussion in this paper have just begun to be implemented at Venetia Mine.

Introduction

Venetia Mine is a De Beers' diamond mine situated in the Central Zone of the Limpopo Metamorphic Belt. The mine lies 80 km west of Messina and just 25 km south of the Zimbabwe-Botswana-South Africa border junction at the confluence of the Shashi and Limpopo Rivers. The mine opened for production in 1991, open pit mining two of a cluster of at least 14 distinctly separate kimberlite bodies (Figure 1). The geometry of the kimberlite pipes are unusually irregular, almost certainly reflecting the complex structural nature of the country rock into which the pipes intrude. The kimberlites were emplaced 519 Ma (Phillips, *et al.*, 1998). The two main pipes labelled K001 and K002 are 12.7 and 5 hectares in surface area respectively, and strongly diamondniferous.

Venetia Mine is an open pit mine with a life expectancy of at least 20 years. Underground mining techniques will most likely extend the life of mine in much the same way as the other diamond mines in South Africa. The initial stages of mining comprised two pits, one in the K001 pipe and one in the K002 pipe (Figure 1). Presently the mine is down to bench 10 of 12 in Cut 2 and on bench 3 of 30 in Cut 3, with the K1 and K2 pits now merged at bench 3 level. The benches are 12 m high. The maximum slope angle for Cut 2 is approximately 51° crest-to-crest. The slope angles were derived during the initial feasibility study in 1989 (Terbrugge, 1989) involving a detailed geotechnical analysis of 15 available diamond drillcores from the country rock, surface geological mapping and rock property tests on samples.

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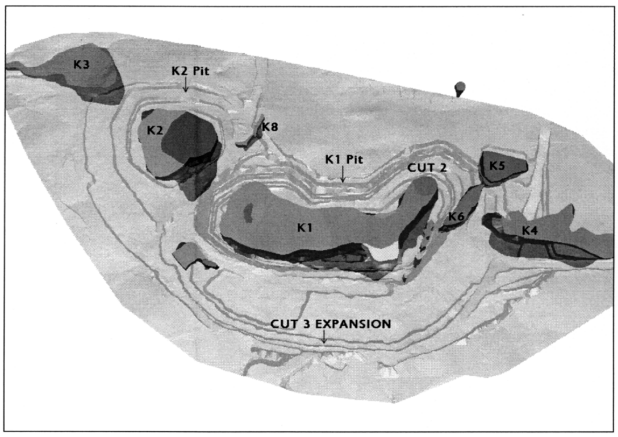


Figure 1—Kimberlite geology model showing all the known pipes. The cut 2 pit with the Cut 3 expansion is indicated

Crucial to the life of mine, particularly of the open pit, is the stripping ratio. The slope angle has the most influence on the stripping ratio. At Venetia Mine the earlier cuts had steeper, more optimistic slope angles because the pit was still shallow and had two access ramps. After 8 years of mining with the onset of Cut 3 it was clear that the design slope angles at Venetia were unfortunately impractical in a number of geotechnical domains. Cut 3 began towards the end of 1999 and is estimated to reach a depth of 360 m. The cut has been planned to be taken in two steps by means of a split-shell design. This means that the waste from the southern shell of the pit will be stripped while the final stages of Cut 2 are mined. Then the mining activity will switch with the waste being stripped from the northern slopes of the pit while ore is mined from the south (Figure 2). The effect of a split-shell schedule of mining allows waste to be deferred to the later stages of the cut thereby reducing the peak tonnage required and improving the overall cash flow for the life of the cut. This paper serves to explain how a complete slope redesign was undertaken for the southern split-shell.

A geological overview

No geotechnical analysis can be done without a reliable geological model. The geological model forms the basis of mining activity and mine planning, and is therefore one of the most crucial steps in any mine development. No detailed geological model for the waste rock had been created before 1999, and therefore no detailed slope design or reliable mine planning could proceed. The requirement to build a model was therefore urgent particularly since the mine was due to begin pushing back waste to new slopes at the end of 1999.

Very few studies have been undertaken in the Central Zone of the Limpopo Metamorphic Belt near the Venetia Farm. The Limpopo Metamorphic Belt is the result of compressive interaction between the Zimbabwe and Kaapvaal Cratons and consists of three distinct zones (Northern, Central and Marginal) separated by major shear zones. Most Central Zone studies have been concentrated in the vicinity of Messina where the Sand River offers good rock exposure. There are strong lithological similarities with tonalitic gneisses, quartzites, meta-pelites, calc-silicates and amphibolites that are found around Messina and Venetia Mine. It has thus far been assumed that the Venetia rocks are part of the Beit Bridge Complex (e.g. Watkeys, 1983), but there is much work that still needs to be undertaken in order to determine this. It is likely that there are major stratigraphic differences.

Work undertaken on Venetia Farm includes Anglovaal mapping in the early 1980s, a pre-mining M.Sc. mapping study (Parrish, 1989) and local in-pit and external mapping by geology and geotechnical staff of Venetia Mine. It was clear that the kimberlite pipes are located in the centre of a large fold structure. The 14 known pipes in the Venetia cluster at the time of the study already existed as a geological model in the Gemcom modelling software. Although unusually irregular in shape they have been modelled with the typical kimberlite volcanic pipe shape reducing in cross-sectional area with depth. Many kilometres of drilling have located the kimberlite contacts to depths beyond that required for the current study.

The first step was a comprehensive drilling programme that targeted the southern slopes initially since the split-shell Cut 3 was due to begin pushing back waste on the south. A total of 5497 m were drilled in the south and another 3372 m in the north in the latter half of 1999. The drilling specifically targeted the rock mass at a position where it was estimated that the final Cut 3 slope would be mined so that the final slope behaviour can be better predicted. The fresh outcrop available in the open pit provided more complete information to build a structural model for the mine. The pit was geologically mapped. Mapping indicated that the pit was situated in the core of a synformal fold. All data from the core logs and pit was inputted into the Gemcom geological modelling package and the software's three-dimensional visualization tools were used to improve the geometric accuracy of the northward verging, shallowly eastward plunging synform model. Gemcom allows the user to interpret the lithology contacts onto pre-defined 2-D cross-sectional planes and then to join the 2-D interpretations together creating 3-D virtual solids (Figure 3). Once the geological model was constructed it could be used to select a few representative geological sections which can be used for slope design.

The geology indicated three primary structural domains for consideration, the intermediately northward dipping southern limb of the fold, the axis of the fold and the steeply northward dipping northern limb of the fold. Crosscutting the folding are north-east striking faults that have dextral strike-slip shear senses. One large fault (Lezel Fault) and splay

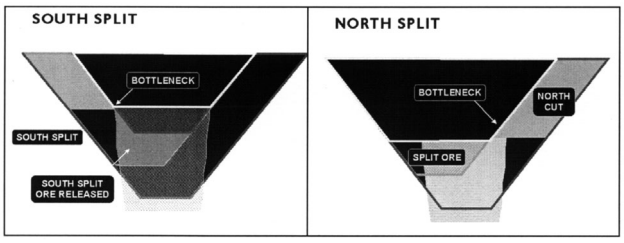


Figure 2—Diagram illustrating the split-shell concept as applied to Venetia Mine (Kear and Gallagher, 2001). First the southern waste is mined with northern ore, then the northern waste is mined with southern ore

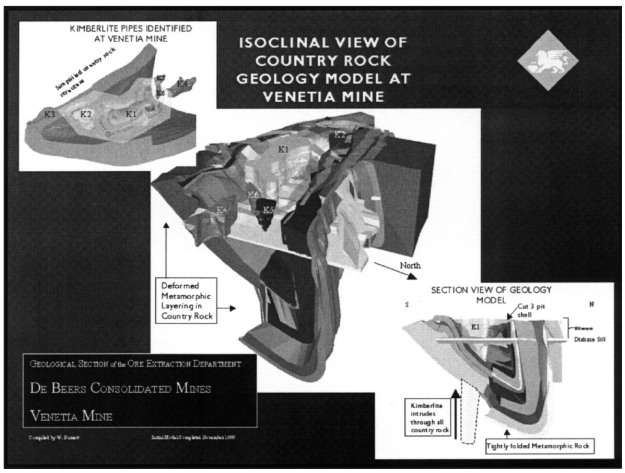


Figure 3—Current solids geology model for Venetia Mine

(Tina Fault) cut through the pit influencing the geometry of the kimberlites. Lezel fault has a displacement varying along its length from 150 m to 200 m. An older fault (Gloudina Fault) following near to the axis of the fold and striking east-west has been displaced by the Lezel fault.

A number of lithological domains were also identified. Most importantly the kimberlite ore and the country rock waste behave very differently. Within the country rock there are also two packages of rocks that have been tectonically juxtaposed together prior to the main folding event. The first group of rocks is the most documented group and is exposed in the current Cut 2 pit slopes. This group is referred to as the metamorphic package (MM1) and comprises predominantly Biotite Schist, Biotite Gneiss, Quartzo-feldspathic Gneiss and Amphibolitic Gneiss. MM1 rocks are strongly sheared and deformed, with clear mineral lineations and definite schistose and gneissic fabrics. MM1 is located in the centre of the core of the fold through which most of the kimberlites have intruded.

The second package is known as the metasedimentary package (MS1) and comprises limestones and marbles interbedded with what has been described on the mine as phyllites and calcareous argillites. Fuchsitic quartzite is also common particularly adjacent the tectonic contact between the MM1 and MS1. Near the open pit the rocks of the MS1 group are typically less sheared (except on definite shear zones) with less strongly developed mineral lineations. The MS1 group of rock types contain far fewer garnets. Both the MM1 and MS1 have a regional hydrothermal overprint most strongly evident near shear zones. The most significant effect of this is the retrograde epidotization of plagioclase that reduces the strength of the intact rock material.

Crosscutting all the rock types, except the kimberlites, are pegmatite dykes and a 20–60 m thick sill of dolerite. The sill lies at around 250 m below surface. A dyke branches from the sill and rises to the surface a few hundred metres north of the pit. The age relationship between the sill and the tectonics has not been completely established yet, but it must post-date the formation of the major synform structure.

Geotechnical input data

Simultaneous with the geological modelling process is the ongoing geotechnical data collection. The Gemcom software has a Microsoft Access database storage area. It was used to store face mapping and core logging data. The most important engineering data sampled from core are the joint properties, including joint frequencies, infill, geometry, alteration and orientation. Rock Quality Designation (RQD), Total Core Recovery, Solid Core Recovery, rock strength and weathering were also measured. Special attention was placed on identifying structural features or zones of weakness. Similar information from face mapping is stored in the Gemcom database. Joint information such as spacing and trace lengths are continually being collected by means of line sampling. Queries set up in Microsoft Access allow all the face mapping and core logging data to be used to calculate Rock Mass Ratings (Laubscher, 1990) automatically for use in the slope design process. See Table I for example average RMR data.

All available rock property data were used during the project, including Uniaxial Compressive Strength, Tensile Strength, Young's Modulus and density. Testing programmes included an investigation carried out at the CSIR-Miningtek laboratory in 1989 and 1998. Further tests were done in 1999 on the newly modelled metasedimentary group of rocks. See Table II for the summarized rock property data. Some direct shear test results were available from tests on the metamorphic fabric-parallel jointing (labelled J3). Using the Mohr-Coulomb equation the laboratory tests indicate the average peak shear parameters are 300 kPa for cohesion (but at high confining stresses) and 31° for the friction angle. An average residual friction angle of 26° was obtained.

All documented and undocumented slope and bench failures in the pit were revisited to make sure that the failure plane geometry and surface property data (e.g. joint orientations, dimensions, Barton-Bandis (1982) joint roughness coefficients) were properly collected. On-site tilt tests of the failure rock material help constrain the friction angle giving an average of 31°±4°. A total of 69 documented failures of various types were available for analysis by the end of the project. All measured data and analysis results are stored in the geotechnical database to be easily queried during slope design. The metamorphic layering-parallel joint set (J3) is the dominant set involved in every failure, except the circular failures in the kimberlite.

As mentioned above the J3 joint set is the 'parting' set in the metamorphic fabric. It is dominant in all the non-igneous rock types except some massive amphibolites and marbles. Therefore like the fabric it dips steeply on the north of the pit

Table I

Summarized RMR data for some of the main groups of rock types. Table taken from SRK, March 2000

Rock Type	RMR (core)	RMR (in pit)
Biotite Gneiss Group (including Quartzo Feldspathic Gneiss)	59	56
Biotite Schist Group	52	46
Amphibolite Group	565	8
Fuchsitic Quartzite	48	-
Phyllite	46	-
Kimberlite	49	53
Marble	51	51

and at intermediate angles on the south. The surface minor-scale geometry typically ranges between 8 and 12 on the Joint Roughness Coefficient scale (Barton and Choubey, 1977) and can be described as smooth-undulating in the MM1 and smooth-undulating to smooth-planar in the MS1 as per the RMR system (Laubscher, 1990). There is usually no infill or alteration, but in the MM1 group of rock types, the joint surface almost always contains a fine layer of biotite suggesting the surface properties of the Biotite Schist (friction angle of 32°) are the best parameters to use in the modelling. The large-scale geometry of the J3 planes are straight to undulating, particularly in the southern slopes where a late stage folding event with east-west compression has caused superimposed northward plunging open folds creating downward plunging 'fold troughs' that fail easily. It is likely that the J1 joint set is near axial-planar to this folding. The J2 joint set is parallel to the NE striking faulting and the J4 joint set is conjugate to J2. The J6 joint set is parallel to intermediately dipping splays of the Gloudina fault. J5, J7 and J8 are rare intermediately dipping sets that might be related to the diabase or pegmatite intrusive events.

Risk assessment

Deterministic failure analysis involves assigning a definite (average) value to each variable used in calculating a Factor of Safety for a specific slope configuration. In reality each variable has a distribution of likely values with a mean value and a standard deviation. Probabilistic analysis involves using the variables and their distributions to calculate a distribution of Factor of Safety values. The Probability of Failure is then the area under the distribution curve less in value than 1, divided by the total area under the distribution.

Philosophies as to what risk is acceptable in slope design vary from one situation to the next. A common philosophy suggests that a probability of failure of between 10 to 15% may be acceptable in situations where the cost of clean up (or mining to flatter angles) is less than the cost of stabilization. However, slopes that contain ramp systems should not exceed 5% probability of failure. General guidelines for risk assessment in pit design of slopes are shown in Table III. Most of the Venetia's Cut 3 slopes fall into the category 1 in Table III, except for those slopes above the ramp system that full into category 2.

A full risk assessment was undertaken during this study to determine the risk that slope failure would have on mining equipment, mining personnel, public relations and mine finances. The probability 'A' of the slope design being incorrect can be estimated as a starting point. Alternatively the required design probability of slope failure 'A' (e.g. 5% from Table III) can be used as the starting point. The probability 'B' of the failure being in certain parts of the slope (such as ramp) can be assessed. The probability 'C' of a shovel being in that part of the pit can be assessed. The probability 'D' that the failure could totally destroy the shovel can be assessed. From these estimations the probability that the mine would suffer a financial setback of losing a shovel in a failure could be calculated from A*B*C*D.

The above illustrates one branch of a risk tree of possibilities that was set out for all mine equipment and personnel. The probability of any end-branch result actually happening could be thus calculated and a financial impact for the mine was estimated. Slope management techniques such as slope monitoring and dewatering are included. The results were presented to the mine management team, and it was decided that the consequences of a 5% probability of failure used as a design criterion for the slopes was an acceptable risk for the mine. Mine management now fully understand

Table III

General risk categories used for slope design (Kirsten, 1983)

Categories	Risk profile	Acceptable Probabilities of Failure
1	Critical slopes where failure may impact on continued operation and safety of the pit	<5%
2	Slopes where failure may have a significant impact on costs and safety	<15%
3	Slopes where failure may have a minor impact on costs and where minimal safety hazard exists	<30%

Table II

Average rock property data for the dominant rock types at Venetia Mine

Rock type	Uniaxial Compressive Strength (MPa)	Tensile Strength (MPa)	Poissons's Ratio	Young's Modulus (GPa)	Relative Density (kg/M_3)
Amphibolite	143	17.3	0.267	89.0	2996
Biotite Gneiss	182	14.6	0.239	71.6	2739
Biotite schist	86	10.5	0.211	53.1	2843
Quartzo-feldspathic gneiss	147	13.8	0.205	73.6	2666
Diabase	263	27.7	0.260	100.0	2984
Country rock breccia	80	11.9	0.205	59.0	2638
Hybabyssal kimberlite	124	12.3	0.234	45.4	2743
Tuffaceous kimberlite breccia	48	7.3	0.206	19.4	2556
Fuchsitic quartzite	159	12.0	0.138	83.2	2686
Phyllite	132	20.9	0.253	59.2	2774
Argillite	119	16.0	0.160	51.9	–
Limestone	145	16.0	0.299	81.6	2845
Marble	154	11.2	0.241	76.8	2760

the potential impact of a failure and has been part of the decision-making process.

Back-analysis

The geotechnical domains as defined previously on the mine were accepted during this study (Figure 4). The definition of the domains is based on the geological domains with the rock properties, the history of failure-types experienced and the orientation of the slope face. A description of the domains can be found in Table VI. Planar failure is the most dominant failure type in the southern pit, with most instability on the southern slopes occurring during blasting and loading. This indicates low natural cohesion or that cohesion is destroyed during blasting.

As mentioned above, the recorded failure data was back analysed to determine a possible range of cohesion values that describe the failure planes. The back-analysis assumes friction angles based on the tilt tests and assumes standard deviations and distribution curves for the input data. The software used for the analyses was a combination of Swedge (Rocscience Inc., Geomechanics Software & Research) for wedges and standard equations (for example, Hoek and Bray, 1981) on custom design spreadsheets for planar failure situations. The methodology used was to vary the cohesion for the analysis until a Factor of Safety of 1 is achieved for the analysis. Deterministically, the cohesion thus calculated would therefore represent a best-case value (highest possible value). However, this last statement is made cautiously because when a probabilistic approach to failure analysis is used it is clear that failure blocks with a Factor of Safety over 1 can still have a good probability of failing.

In some cases where the failure block failed some months after mining the bench, the back-analysis was used to determine a cohesion representative of a Factor of Safety of 1.2 (more likely to give a more accurate estimate of the cohesion). In cases where the presence of groundwater or rainwater had contributed to the failure, water pressure was also estimated and included. Usually maximum water pressure is assumed as a worst-case scenario. The more data analysed in this way the better informed the geotechnical engineer's judgement in choosing a representative cohesion for the slope design. This sort of analysis also makes sure that the different types of possible failure are recorded and evaluated for each of the geotechnical domains.

Various friction angles (e.g. 32°, 35°, and the tilt test peak and residual angles) were chosen in order to do a range of analyses so that sensitivity between the variables could be established. Average cohesion results from the planar failure calculations remained close to 10 kPa with coefficients of variation up to 80%. The calculations (for planar and wedge failures) using the tilt test residual angles are considered the most accurate and indicate that the average for the cohesion, both overall and for just Biotite Schist rich rock types, are 10 kPa and 8 kPa respectively (see Figure 5).

The probability of failure of the rockmass in a particular geotechnical domain could be reflected fairly accurately by the percentage area in that domain that has actually failed,

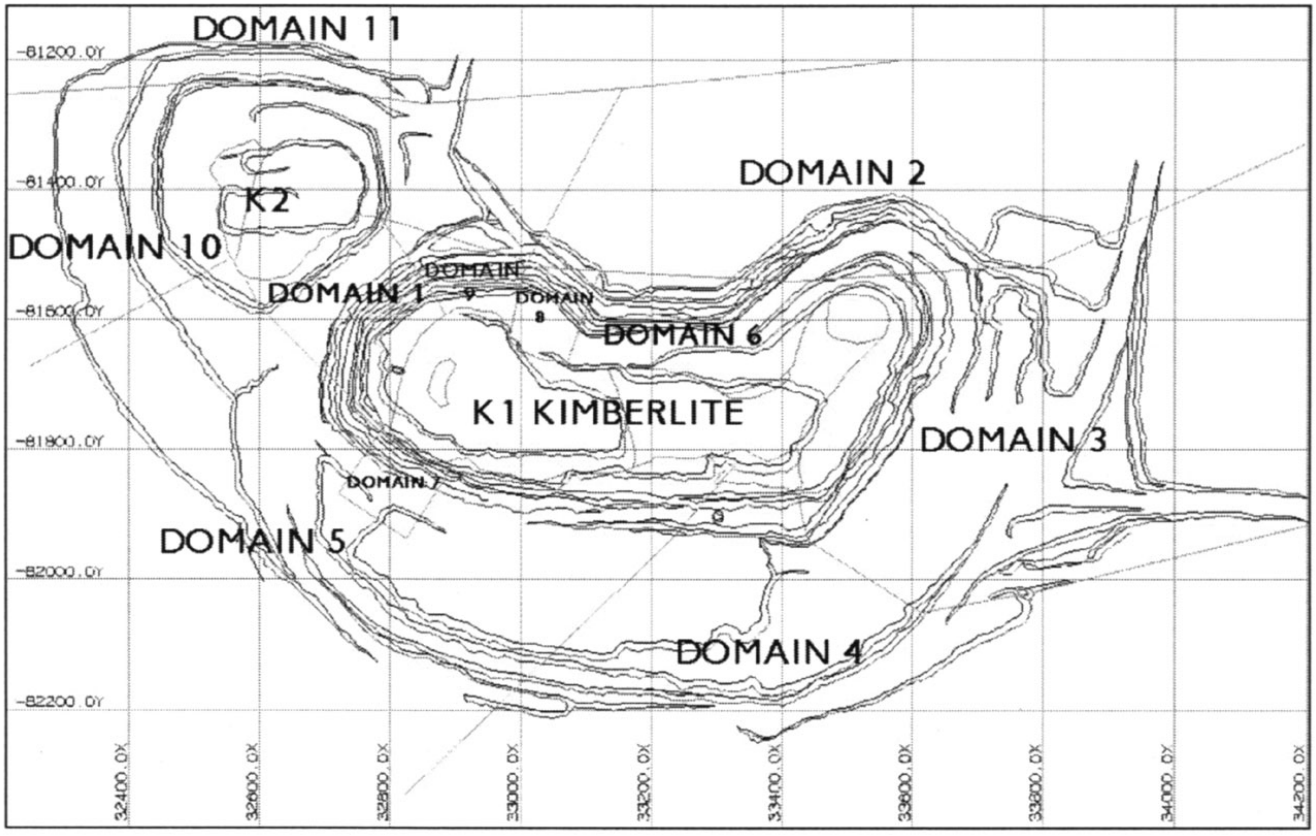


Figure 4—Geotechnical domain boundaries as applied to the current open pit. Domains are based on geology and understood slope behaviour

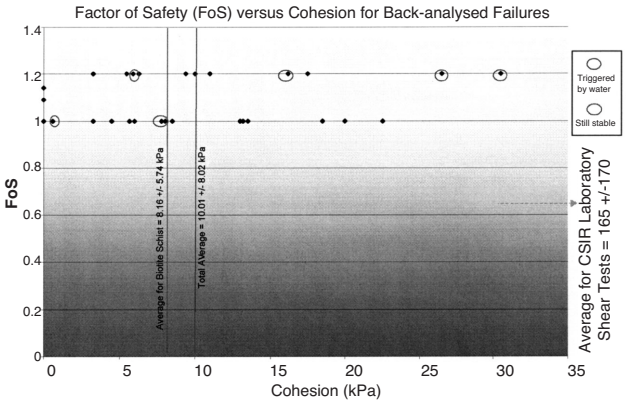


Figure 5—A graph showing the results of back-analyses in the Venetia pit where cohesion has been calculated from an estimated Factor of Safety

assuming that enough time has passed and the area is statistically large enough. In this case the probability of failure can be measured from the state of the slope face. Photographs were therefore taken of the different domains around the Venetia pit and where possible the percentage area of the face that had failed was calculated to give a probability of failure for that domain. The slope's parameters (e.g. joint orientation, face angle and bench height) for that domain were then used to determine the cohesion required giving this measured probability of failure. This method can be a more accurate indication of the slope's cohesion and is compared to the values calculated above. See Table IV for the results of this approach. The southern domains 4 and 5 stand out clearly in this exercise as requiring serious design attention.

Slope design

Once the geology and the geotechnical parameters were collated the slope design was carried out. Mining Rock Mass Ratings (MRMR of Laubscher, 1990) were calculated from the RMR data. The following adjustments were considered. Weathering adjustments were applied to the schists and phyllite of 95% and to the kimberlite of 70% based previous experience. On the southern slopes an orientation adjustment of 70% was applied because of unfavourably orientated layering. A stress adjustment of 95% was universally applied because stress relief does not significantly influence shallow to moderate depth open pit mines. A 90% blasting adjustment was also applied all around the pit. A preliminary estimation of the slope angles could then be taken from the empirical chart of Haines and Terbrugge (1991). The results of the major southern Cut 3 domains are indicated in Table V.

The range in slope angles for all the country rock types is from 50° to 56°. Based on the above some of the final slope angles in Table VI are recommended. Note that kimberlite weathers rapidly to a weak rock mass and it has been recommended to further adjust the above stack angle from 53° to 50° for kimberlite slopes. The presence of abundant unpredictable kimberlite bodies in eastern domain 3 reduces the angle to 50°. The empirical approach accounts best for rockmass behaviour situations. It cannot be applied to the southern domains 4 and 5 where stability is controlled strongly by the orientation of the J3 joints and the cohesion on those joints.

Probabilistic slope design during this study focused on domains 4 and 5 that have been recognised as the most critical. Bench heights and face angles determined for these domains could then be applied to the less critical domains. Values used during the probabilistic design are the averages determined during the geotechnical data collection and back-analysis discussed above. Assumptions made were that the coefficients of variation for the cohesion was 40% (recommended by Harr, 1977) and 12% for the friction angle, and that all variables are normally distributed.

To determine the optimum combination of bench height and bench angle Swedge was used to calculate probability of failure versus bench height and probability of failure versus bench angle. For the southern domains (Figures 6a and 6b) 75° face angles give a 5% probability of failure for a 12 m high bench.

Plane failure analyses were carried out on domains 4 and 5 to determine the slope angles that gave a Factor of Safety of 1.2 for various slope heights. Probability of failure was also calculated for stack heights of 48 m, 96 m, 120 m, and 180 m (Figure 7). In this way the relationship between optimum slope angle and stack height could be determined. Table 6 summarizes the results of the entire Cut 3 design process. The details of the analyses used for the northern domains are not discussed in this paper, but were undertaken by SRK staff using software packages such as Swedge, XSTABL and FLAC.

Failure analyses provide an indication of the weight of the wedge failing. The minimum spill berm could therefore be calculated from (Piteau, *et al.*, 1982):

Minimum Spill Berm (equilize) = (Average wedge volume * 1.5)$^{1/3}$

Table IV

Cohesion as determined from back-analysis of slope photographs assuming 12 m (1 bench) slope heights. Some of the northern domains have been included in this Table for comparison purposes.

Domain	Percentage Failure Area	Back-Analysed Cohesion
Domain 2	2–4%	40–60 kPa
Domain 3	6%	-
Domain 4	79%	14.5 kPa
Domain 5	47%	15 kPa
Domain 6	5%	32 kPa
Domain 8	7%	28 kPa
Domain 9	7%	28 kPa

Table V

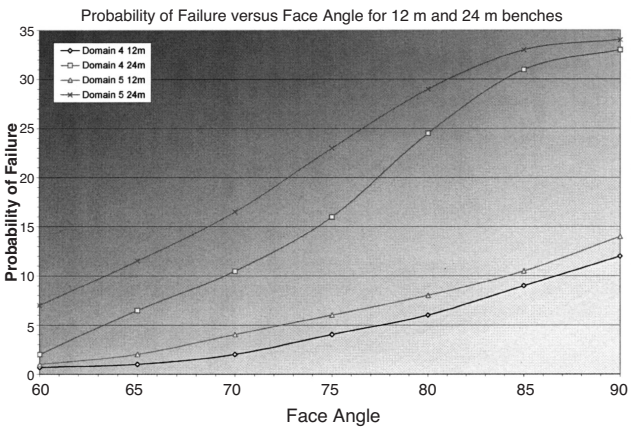
Empirical slope angles from the Haines and Terbrugge (1991) chart for stack heights of 96 m and a Factor of Safety of 1.2

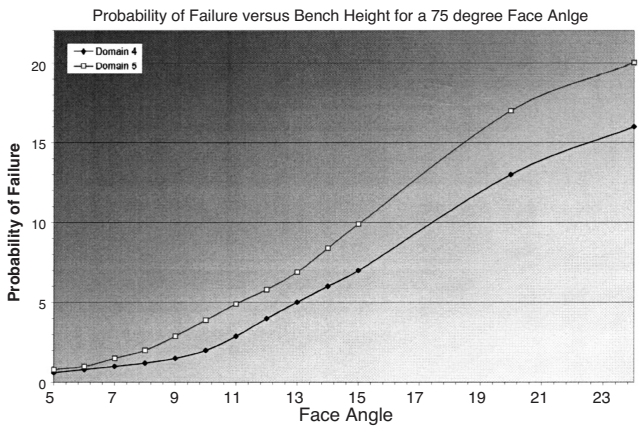
Domains	Average MRMR	Empirical stack angle
Southern Domains 4 and 5	32–33	52–53
Eastern Domain 3	38	55
Western Domain 1	42	56
Diabase (south)	44	57
Kimberlite (unweathered)	34	53

Table VI

Summary Table of the Cut 3 north and south domains with the recommended slope angles determined at the end of the design process

Domain	Position	Description	History of failure	Slope height	Slope angle
1	Saddle between K1 and K2	J3 is dipping north, but generally perpendicular to the face. There is local folding deformation	Generally stable. Local planar failure on folds.	< 180 m	55°
2	North-east corner of K1	J3 is dipping steeply northwards (80° overturned to vertical) away from the pit. J3 is parallel to the slope in places	Locally unstable with some cases of toppling failure.	Top weathered 0–24 m Un-weathered 96 m	50° 56°
3	Eastern side of K1	The hinge of the synform is found and J3 is typically near perpendicular to the slope but varies greatly in orientation across the hinge deformation.	Generally stable, but with rare wedge failure.	< 180 m	50°
4	South-east slope of K1	J3 dips northwards at 45–65° directly into the pit. The southern limb of the synform, east of the Lezel Fault.	Unstable with 79% of benches experiencing planar failure.	48 m, 96 m 120m,180m	48°, 45° 44°, 42°
5	South-central and south-west slopes	J3 dips northwards at 32–45° directly into the pit. The southern limb of the synform, west of the Lezel Fault.	Unstable with 47% of benches experiencing planar failure.	48 m, 96 m 120 m, 180 m	40°, 37° 36.5°, 36°
6	North-central slope	The hinge of the synform. Competent biotite gneiss. J3 is sub-parallel to the slope. Fault splays with penetrative J6 jointing in places. The slope face is sub-parallel to the fold axis	Unstable wedges where J3 intercepts unfavourably with faults and J6.	Top weathered 0–24m Un-weathered 96 m	50° 56°
7	South-west of K1	Small domain of country rock breccia enclosed within domain 5 and not reaching Cut 3 limits.	Stable	96 m	55°
8	West of north central slope in K1	Small domain in which the Gloudina Fault has been intercepted.	Unstable planar failure on fault surface.	Top weathered 0–24 m	50°
9	North-west corner of K1	Sub-horizontally layered biotite schists and amphibolite.	Stable	Un-weathered 96 m	56°
10	East and west of K2	Face perpendicular to a complex fold hinge zone. Tight to isoclinal folding and associated faulting strike near perpendicular to the face.	Generally stable, but with rare bench-scale wedges.		
11	North K2	Metasedimentary phyllites, quartzites and marbles dip at low angles northwards away from the pit.	Stable, except for rockmass failure in weathered zone.	Top weathered 0–60 m Un-weathered 96 m	50° 56°





Figures 6a and b—Figures show the results of probabilistic analyses that determine the optimum bench face angle for the southern domains, 75° at 12 m heights for a 5% probability of failure

Calculated minimum berm widths for various domains ranged from 4 m to 6.5 m. These minimum berm widths are all within the domain berm widths that can be calculated from the recommended slope angles.

Support design

The Cut 3 slope design assumes that the mine has a dewatering system in place. As a result of this study a comprehensive groundwater project has been started in order to characterize the groundwater behaviour around Venetia Mine so that proper dewatering strategies can be implemented. However, of concern is the situation where heavy downpours of rain saturate the benches in the pit. The Limpopo climate causes strong summer rains making the chances of bench saturation very good. This cannot be

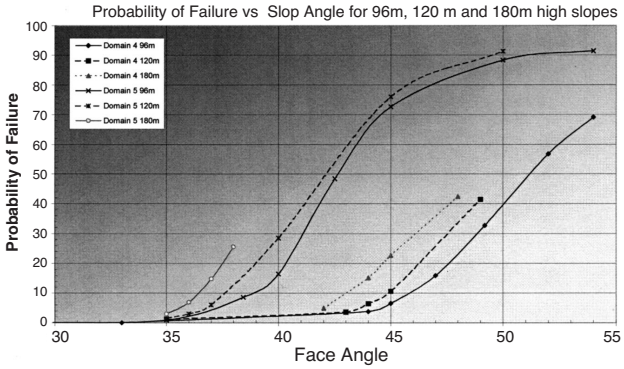


Figure 7—The graph illustrates how the slope angle for the southern domains affects the probability of failure of the slope for various slope/stack heights. Domains 4 and 5 slope angles in Table VI are taken from this graph

prevented. Bench failures caused by the rains can be accommodated as part of the risks of open pit mining, but the risk of crest failure of the ramp systems is not acceptable.

In the same manner as the previous slope design calculations, the probability of failure of bench crests with no support in full storm-water was calculated at 91% in domain 4 and 60% in domain 5. Such failures would be capable of removing from the ramp crest up to 7 m in domain 4 and 12 m in domain 5. The next stage of this project was therefore to design a suitable support system for the ramps passing through the unstable domains 4 and 5. Large sections of the ramp in these areas have been lost in the past.

Requirements for a slope support system are as follows:

- Support the ramp crest to within a 5% probability of failure
- To be the most cost-effective option
- Provide a fairly stiff support prior to loading of the trim blast from the face, which is usually the stage when most of the failures occur
- Not to increase the width of the berm below the ramp beyond that recommended by the slope design.

The most suitable option envisaged that meets the above criteria is the spiling support method. Spiles are holes drilled into the bench (usually vertically) that are filled with a fully grouted steel rod or cable.

Figure 8 shows the extent of domains 4 and 5 in the Cut 3 design. About 2800 metres of ramp needs to be supported. In order to obtain a reliable spiling depth the spiles must pin all the layering (J3) undercut by the bench face with 95% reliability. It was determined by means of Monte Carlo simulation that 5% of the J3 planes have dips (assumed normally distributed) less than 35° in domain 4 and 32° in domain 5. So all planes extending from the base of the bench face being supported that are greater in angle than 35° and 32° respectively must be pinned by the spiles (see Figure 9). However, the spiles are for a ramp crest, and the ramp has a gradient from one bench to the next. Which bench is to be supported? It was decided that it was logical that the design support up to one and half benches at the centre of a ramp segment (ratio distance 0.5 in Figure 9). The required depth of the spiles would then differ depending on the distance down the ramp and the distance from the crest (illustrated in Figure 9). It should be noted that because of the close

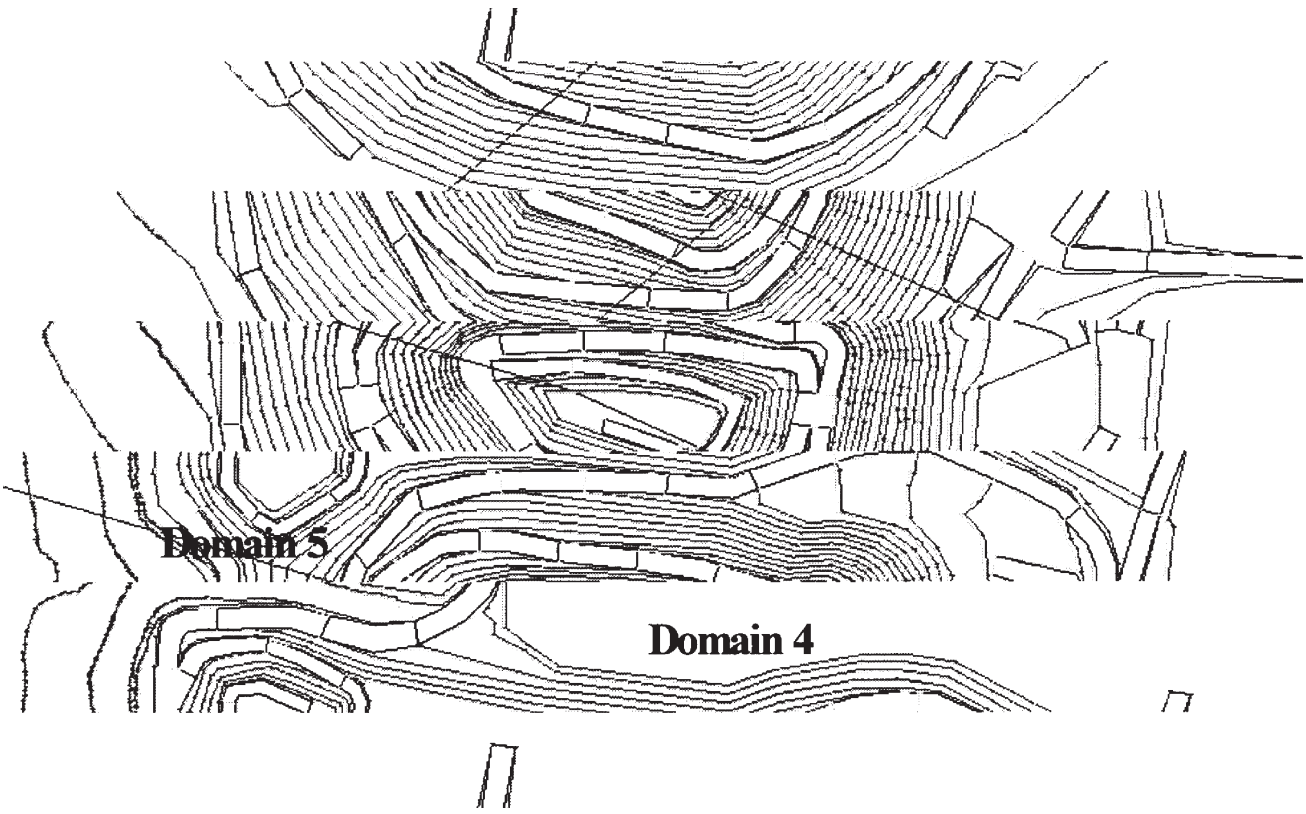


Figure 8—The domains 4 and 5 superimposed on the current Cut 3 pit design. The ramps within these two domains need to be supported

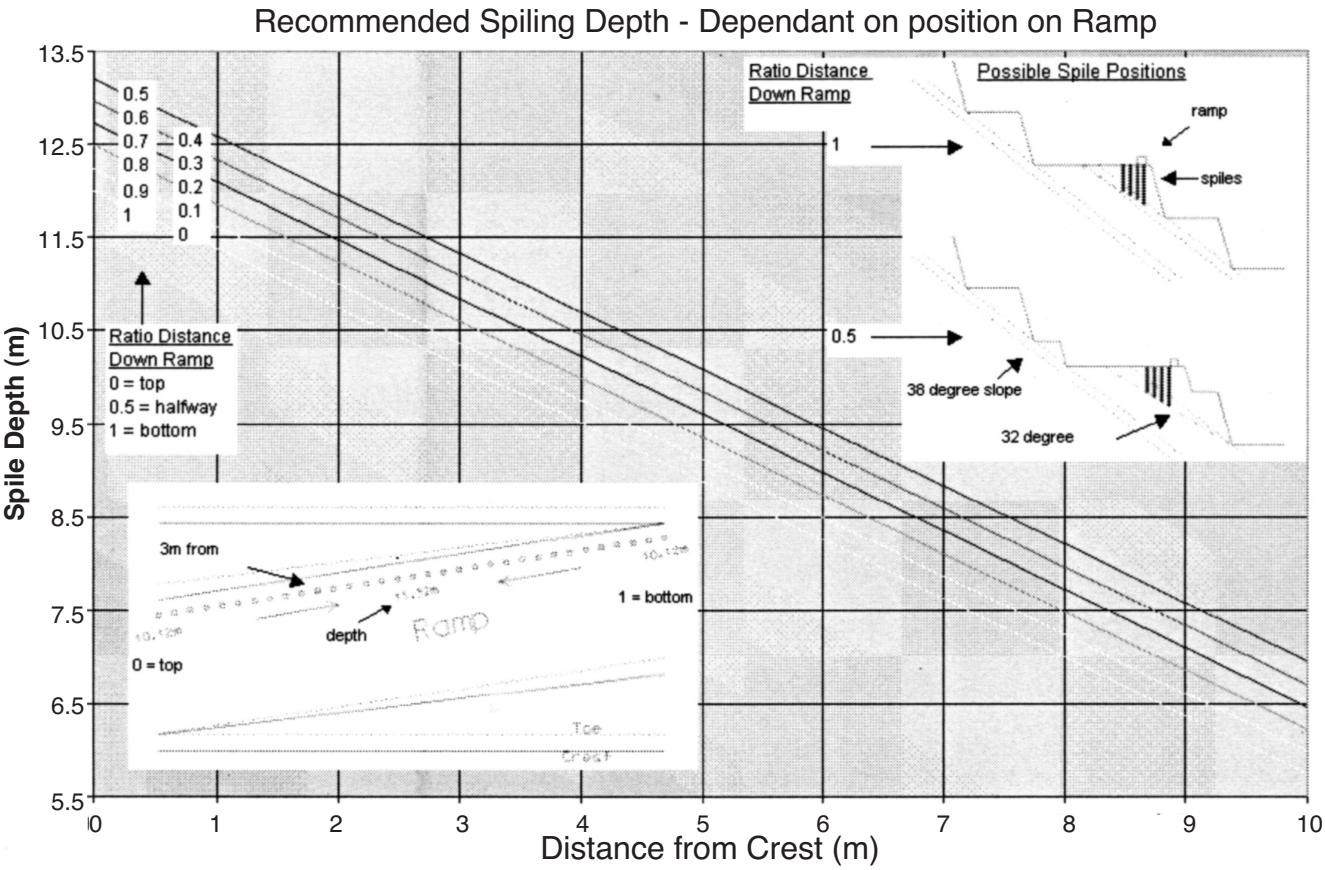


Figure 9—The graph indicates how the recommended depth of spiling must vary for a 5% PoF depending on the distance from the crest, for various positions down the ramp. For example, if the ramp gradient is such that the ramp takes 100 m to go from bench 1 to bench 2 the recommended minimum depth of spiling at a point 60 m down the ramp is derived as follows. The ratio distance is 60/100 = 0.6. The recommended distance from the crest is 3 m. From the graph the spile depth is 11.1 m. Also shown on the graph are the dashed boundary lines for domain 4 indicating very similar depth requirements

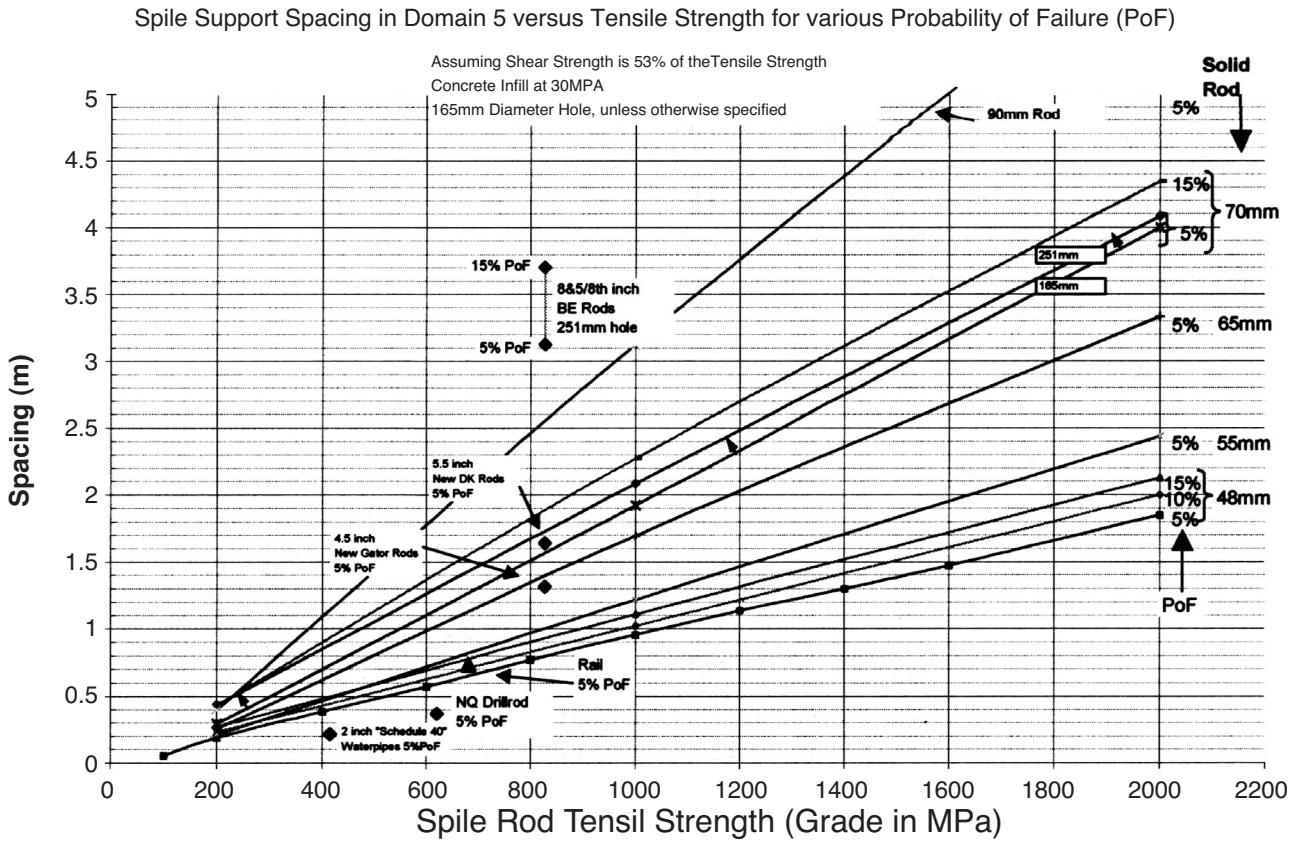
rockmass jointing spiles that are too close to the face could experience the rock unravelling around them. Spiles beyond 5 m from the crest would be ineffectually supporting the loose blast-damaged part of the sliding wedge that lies within 1.5 m from the surface, and could cause vehicle tyre damage. 3 m is recommended as the ideal. Based on the above the recommended spile depth for both domains is a simplified but effective 11.5 m.

The spile spacing is the next crucial specification. A probabilistic analysis was done with various combinations of spiling support. The results are graphically displayed in Figures 10a and 10b. The analysis is based on the following assumptions:

- The failure is planar failure
- The spile is pinned vertically through the failure plane being analysed
- In order for failure to occur shear must occur along the failure plane overcoming the cohesion on the plane
- Shear must also occur through the concrete's apparent cross-sectional area on the plane of the failure, where the shear strength is about 15% the tensile strength
- Shear must also occur through the steel reinforcement bar's apparent cross-sectional area on the plane of the failure, where the shear strength is 53% the tensile strength.

In other words, the spile is a passive support system and contributes to the Factor of Safety by increasing the shearing force required along the failure plane to initiate failure (increasing effective cohesion). Important specifications are the tensile strength of the steel rod, the rod outer and inner diameter, concrete tensile strength, overall spile hole diameter, and the frequency of spiles. The face angle, failure plane angle, bench height, cohesion, friction angle, rock density, additional truck load and water pressure are all included in the analysis.

Important for the support design is the rod's tensile strength and diameter, together with the frequency of spiles required. These variables are plotted in Figures 10a and 10b for a probability of failure of 5% unless otherwise specified. The sensitivity of these variables is clearly illustrated. A number of common sources of steel rod have been analysed and shown in the graphs. From the spile spacings the estimated total cost of using a particular spile material can be calculated (e.g. Figure 11). These graphs represent the spile design recommendations. The actual spile spacing and cost will depend on the properties of the steel rods sourced as reinforcement. The graphs provide an excellent guideline and suggest that more expensive, thicker, high tensile steel will actually reduce the total cost of spiling.



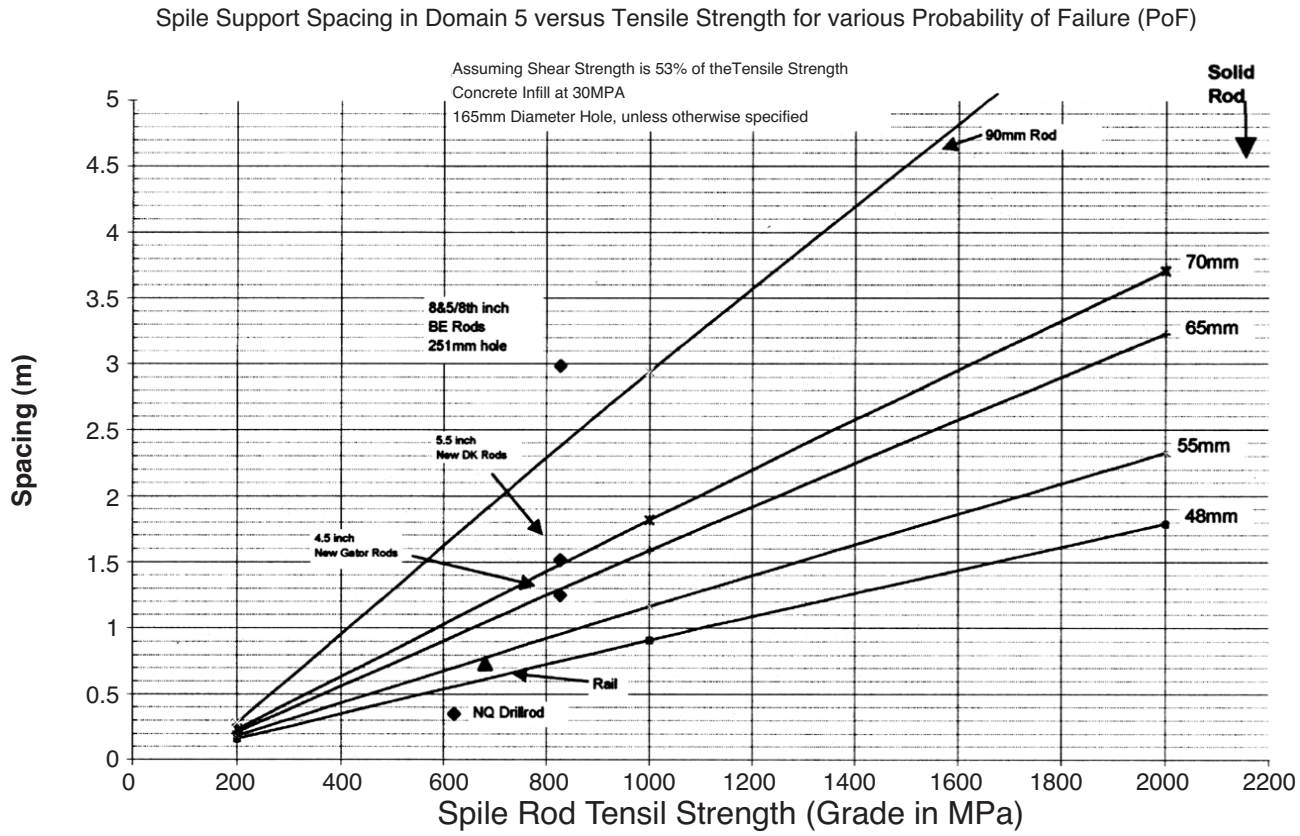


Figure 10—(a) This Figure shows how the spile spacing varies with tensile strength of the support material in domain 5. Included are drillrods and pipes that have a specified strength and therefore a specified spacing. Except where indicated the results are for a 5% probability of failure. The influence on the spacing by increasing the probability of failure to 15% (red lines) can be seen by comparing the 5% line with the 15% line for the same rod diameter (see 48 mm, 70 mm and BE Rods). The influence of increasing the diameter of the hole (not the support rod) from 165 mm to 251 mm is indicated (see arrows with the 70 mm 5% rod). Assumptions for the graph are that the hole (inside and outside the rods) is filled with 30 MPa concrete, and that the shear strength is about 53% that of the tensile strength. (b) Spile spacings for domain 4 for 5% PoF

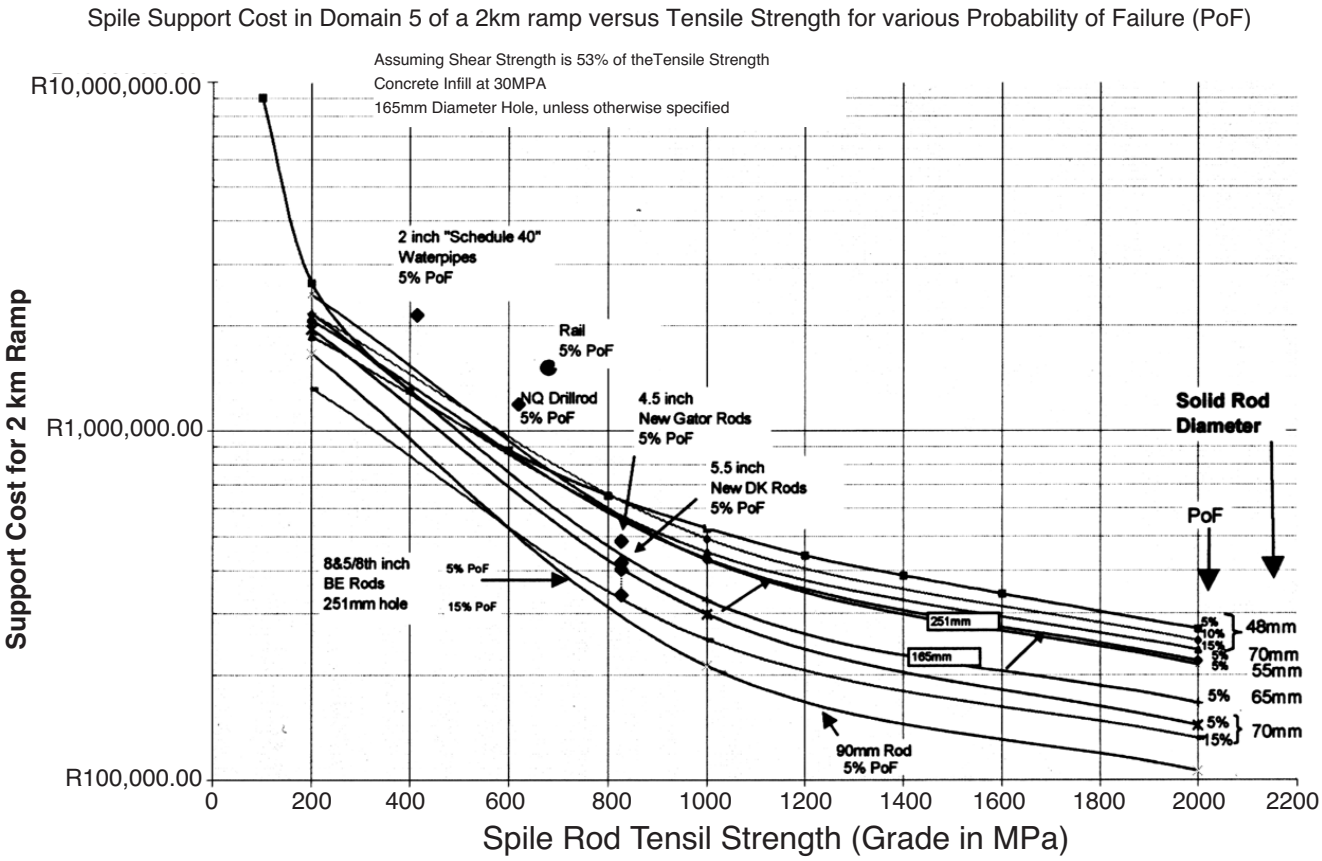


Figure 11—This graph shows how the spiling costs vary with tensile strength of the support material in domain 5. Included are drillrods and pipes that have a specified strength and therefore a specified cost. Except where indicated the results are for a 5% probability of failure. The influence on the spacing by increasing the probability of failure to 15% (red lines) can be seen by comparing the 5% line with the 15% lines for the same rod diameter (see 48 mm, 70 mm and BE Rods). The influence of increasing the diameter of the hole (not the support rod) from 165 mm to 251 mm is indicated (see arrows with the 70 mm 5% rod). These costs are subject to cost of the steel reinforcement

Conclusions

Plane failure, wedge failure, circular failure, toppling failure and stepped-path failures occur in the Venetia Mine pit. Geotechnical domains (design sectors) were defined based on the slope geology and behaviour in the open pit environment. Exploration drilling, face mapping, failure back-analysis and sample tests are the main sources of geological and geotechnical information for the slope design. A geological model was created using the Gemcom modelling package. The model is a synclinorium dominantly comprising gneisses, schists and metasediments, and crosscut by a number of deformational events.

The southern domains 4 and 5 required a detailed probabilistic approach to the design of the slopes. A risk assessment was undertaken with the mine management and it was agreed that the slope design should have a 5% probability of failure if the slope contains a ramp. The existing slope angle for Cut 2 of 51° (crest-to-crest) was too steep for the southern pit slopes. New slope angles were determined such that the probability of failure of 5% would not be exceeded. As a result the southern slope angles for Cut 3 range from 45° down to 37°. This represents an increase in the stripping ratio for the mine, but a very necessary step if unacceptable risks are to be avoided. A bench height of 12 m at a face angle of 75° was determined to be optimum.

This study also determined that during stormwater conditions the probability of failure of the ramp crests in the southern domains 4 and 5 are unacceptably high. A probabilistic analysis of the crests containing a spiling reinforcement system allowed optimum spile depth and spacing for a variety of reinforcement material types to be calculated. The reinforcement system was also designed to have a 5% probability of failure. For this design it is assumed that the spiles simply increase the effective cohesion of the failure planes. For failure to occur the spile would have to undergo shear failure through the cross-sectional area of the spile. Analyses indicate a sensitive relationship between spile spacing and reinforcement shear strength.

The design of the Cut 3 northern domains are not discussed in this paper, but the results are tabulated (Table VI). Future work to be undertaken by the Venetia geotechnical staff includes the validation of the geological model. In particular the exact position of the fault systems needs to be determined. The groundwater potential of these structures is currently being quantified in order for a dewatering system to be designed. Future support design may be required for structures intersecting Cut 3 slopes.

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Technikon honours exceptional mining alumnus*

Eddie Scholtz, a diplomate of the Technikon Witwatersrand's School of Mines has been awarded the prestigious title, Alumnus of the Year 2001. The announcement was made at the Technikon's annual Alumnus and Young Achiever of the Year awards, which took place at the Johannesburg Country Club on 17 October 2001.

The Alumnus of the Year is awarded to an individual who is an outstanding example of what diplomates and graduates from the TWR can achieve. Speaking at the awards ceremony, Cyril Gebhardt, Deputy Chairperson of the TWR's Alumni Convocation said, 'Just to be nominated should be an honour for the candidates. These awards also bring added value to the TWR by highlighting the prominence of our alumni. This is a project that the Technikon can be proud of!'.

Scholtz is Managing Director of the largest coal mining company in South Africa, Ingwe Coal Corporation—a position that he has held since 1999. He is in direct control of eight collieries and assets worth R5.6 billion. Ingwe Coal Corporation is not only the largest coal exporter in the world but 50 per cent of the country's electricity supply is dependent on the output of coal from this company's operations.

In 1978 Scholtz obtained his National Diploma for Technicians in Coal Mining from the TWR. Since then he has made a major contribution to the development of the country's wealth and economy in positions of management on various coal mines.

From 1977 to 1982 he worked at Ermelo Mines where he introduced the first continuous miner into Ingwe and started pillar extraction in Ermelo. As a Project Engineer, he was responsible for the development of three shafts.

In 1986 Scholtz was promoted General Manager of Kilbarchan, where he successfully converted the mine into a trade and export mine after the closure of N'gagane Power Station.

On 1 December 1988 Scholtz became General Manager of Optimum Colliery where he spent six years before being promoted, in 1995 to Ingwe's Head Office as a Senior Manager, in charge of a number of mines. In 1999 he was appointed as Managing Director of the Ingwe Coal Corporation.

Scholtz has maintained his links with the TWR by promoting the value of diplomates within the Ingwe Group of companies—an estimated 60% of the production management of the Collieries of the Group are from the TWR mining programmes.

Scholtz says that his motto is that there is no such thing as a problem, only a challenge. 'My company has served me well, the coal mining industry has served me well and my TWR diploma has served me well. If I had to do it all over again I wouldn't change a thing,' he says. ◆

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