GEOTECHNICAL STRATEGY AND TACTICS AT ANGLO PLATINUM'S PPRUST OPEN PIT OPERATION, LIMPOPO PROVINCE, SOUTH AFRICA

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A research report submitted to the Faculty of Engineering and the Built Environment, University of Witwatersrand, in partial fulfillment of the requirements for the degree of Master of Science in Engineering

Johannesburg, 2006

DECLARATION

I declare that this research report is my own, unaided work. The content covers work done at Potgietersrust Platinums, an Anglo Platinum open pit mining operation, by the author as well as fellow staff members. It is being submitted for the Degree of Master of Science in Engineering in the University of Witwatersrand, Johannesburg. It has not been submitted before by the author for any degree or examination in any other university.

Megan Jane Little

8th day of December 2006

ABSTRACT

Over the last four years Potgietersrust Platinums (PPRust) has successfully implemented new geotechnical strategy and tactics to reduce risk, improving safety but also maximising profitability. A large database of core logging, face mapping and rock testing has been assembled and used in the slope design process. The data has also been used for optimising blast designs on a daily basis through the use of a geotechnical block model. This greatly improves blast fragmentation and therefore loading and milling efficiencies. Slope management includes a limit blasting programme, daily visual inspections, and state-of-the-art slope monitoring equipment, namely GroundProbe radar, Riegl lasers and GeoMoS automated prism monitoring. Slope optimisation incorporates all the field data, operational controls, cost of failure, full economic analysis of various slope angles and fault tree analysis. Savings on waste stripping of hundreds of millions of Rands were gained from the optimisation as slope angles could be increased due to improved geotechnical knowledge and management. PPRust's geotechnical work is considered the benchmark for Anglo American open pit operations.

ACKNOWLEDGEMENTS

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1 INTRODUCTION

'In politics, the path from A to B is never straight. It almost always goes through C, D or F."

- L. Paul Bremer

Geotechnical engineering has an integral role to play throughout the life of an open pit mining operation, from the feasibility stage to mine closure. Overall slope angles play a large role in mine economics and a change of a few degrees can add or remove tens or hundreds of millions of dollars. The economic benefits must be balanced with safety, however, and there is a threshold at which an increased slope angle results in an unacceptably high risk. This can result in slope failure and large cost to the operation. To avoid this, slope optimisation using risk versus reward methodology should be performed both at the outset as well as during the running of the operation. To optimise the slopes, sufficient high quality data is required. A mine's ore reserve model is regularly updated as new core logging and assay data is received and can change the mine design, lifespan and scheduling. In the same way geotechnical information can be regularly collected and analysed to reassess the slope angles and geotechnical risk. Improved computer technology also allows for storage and processing of large amounts of raw data and the creation of 3-dimensional geotechnical models. Advances have also been made with digital photogrammetry which enables rapid mapping of large areas of a pit slope. Slope management, which includes limit blasting, visual inspections, slope support, dewatering and slope monitoring, plays a large role in reducing risk and improving safety in the pit. In the past, slope monitoring has focused on prism monitoring. In the last five years there have been huge steps forward in monitoring technology and rapid, automated monitoring with radar, laser and seismic instrumentation is now available.

While technology has advanced in the last 15 years, many new challenges have also arisen in the South African mining context. These include cost increases, difficulty in retaining technical staff and an increased emphasis on safety (Stacey *et al.*, 1999). For the geotechnical engineer this translates into increased pressure from management to steepen slope angles to reduce waste stripping costs, while maintaining safe highwalls in the pit. The improved monitoring technology makes this possible but produces very large amounts of data. The high staff turnover and general lack of skilled staff in the discipline, hinders the geotechnical engineer's ability to fully utilise the new technology and implement cost saving measures and risk management. It is essential therefore to develop workable geotechnical strategy and tactics at an open pit mine to address these issues. At the start of 2003, Anglo Platinum's only open pit operation, Potgietersrust Platinums (PPRust), did not have this in place. The challenge was to successfully implement new geotechnical tactics to reduce risk by improving safety but also minimising costs. This research report seeks to document this process and provide a methodology that can be implemented at other open pit operations. Background information and general geotechnical practices are given for each section to put the PPRust work in perspective.

The title of the research report was inspired by the book 'Games Foxes Play' by Ilbury and Sunter (2006) in which the authors say that 'Strategy is about where you are going. Tactics are about how you get there. Once you've set off on a voyage, there are only two strategic decisions you can make: change the ultimate destination or cancel the journey altogether and go home. Everything else is tactics.' This research report will discuss both the geotechnical strategy, or philosophy, at PPRust, and the more detailed tactics, or methodology, on how that strategy is achieved. The strategy is threefold: 1) to design fully optimised slopes at all stages of the mining operation; 2) to manage those slopes effectively so that maximum profit is achieved while meeting Anglo American's safety and risk requirements and 3) to utilise geotechnical information to optimise every blast design to achieve the target fragmentation for the processing plant. There are numerous tactics employed to achieve this which fall into the broad categories mentioned below and are detailed in subsequent chapters.

- **Field Data -** The success of a slope design is based on the quality and quantity of the field data used in the analysis. This includes core logging, face mapping, rock strength testing and groundwater studies.
- **Block Modelling –** To make the most of the geotechnical data a block models are created for each pit which include interpolated geotechnical parameters and calculated slope angles, blastability indices and blasting parameters.
- **Slope Stability –** Analysis of the failure mechanisms and stability of the pit slopes is essential in the open pits. Geotechnical zones are defined based on rock mass quality, geological structures and failure mechanisms and used for slope design.
- **Slope Design –** From the block models, failure analysis and numerical modelling, slope design options can be created and compared. The chosen design must meet safety and economic requirements.
- **Slope Management –** For a design to be successful, the slopes must be well-managed throughout mining operations. Limit blasting, visual inspections and slope support are all slope management tools.
- **Slope Monitoring –** To ensure that a slope can be safely mined, state-of-the-art slope monitoring equipment is installed to measure slope deformation that cannot be seen with the naked eye.
- **Slope Optimisation –** All the data and analyses are combined with risk-reward work to determine the optimal slope designs for each slope in each open pit. This includes fault and event tree analysis, cost of failure calculations and NPV optimisation.

Each of the tactics summarised above plays an important role in PPRust achieving the geotechnical strategy, and has enabled PPRust to improve its geotechnical risk profile and become the benchmark for open pit geotechnics in Anglo American. Chapter 2 and 3 provide background information on the mine and the geology. The tactical measures outlined above are dealt with in greater detail in Chapters 4 to 10. Conclusions and recommendations for future work are given in Chapter 11.

2 BACKGROUND INFORMATION

"It is a very sad thing that nowadays there is so little useless information."
- Oscar Wilde

This Chapter provides the reader with a brief background on the location, mining history and methods and exploration at PPRust. This gives context to the work that is detailed in the subsequent chapters.

2.1 Location

Potgietersrust Platinums Ltd (PPRust) is Anglo Platinum's only open pit operation. It is located 35 km north of Mokopane (previously Potgietersrus), in the Limpopo Province of South Africa (Figure 2.1). It is situated in the centre of the northern limb of the Bushveld Complex, a saucer-shaped layered igneous intrusion. The northern limb hosts the Platreef orebody, which is a ~100 m thick tabular body that strikes north-south, dips 45° to the west and reaches a depth of at least 2000 m. The Platreef is a PGM deposit and contains economic quantities of platinum, palladium, rhodium, gold, copper and nickel, which are extracted and processed by Anglo Platinum. PPRust is the only operational mine on the Platreef.

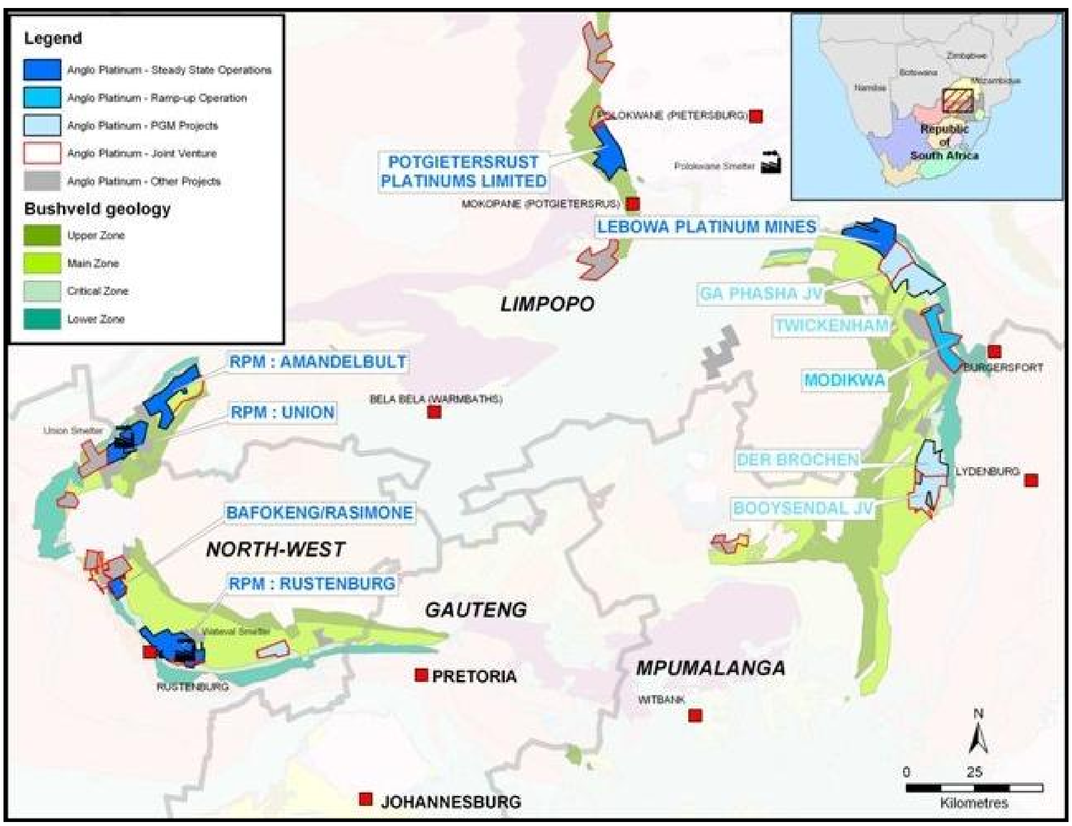


Figure 2.1 Location map of the outcrop of the Bushveld complex and the Anglo Platinum operations (AP, 2006)

2.2 History

The Platreef was discovered in 1924 by Dr Hans Merensky (Allen, 1996) and after a "platinum rush", a shaft was sunk in 1925 by the company Potgietersrust Platinums Limited on the farm Sandsloot. Shafts were also sunk on the farms Vaalkop and Zwartfontein. The company established a treatment plant in September 1926 and produced 110,000 tonnes of concentrate until the financial depression of the 1930's brought mining operations to a halt. Exploration commenced again in the area in 1966 by Johannesburg Consolidated Investments (JCI) for feasibility studies. In 1990, the decision was made to go ahead with a new platinum open pit mine on the Platreef. Sandsloot open pit was situated in the centre of the company's lease area (Figure 2.2) and contained the highest grades. On the 10th January 1992, waste stripping commenced and, on the 12th February 1992, the first blast in the Sandsloot open pit took place. The official mine opening was on the 3rd September 1993 and as of 2005, over 400 million tonnes of rock have been excavated. Operations began on a second pit in August 2002 on the southern section of the Zwartfontein farm, where the underground mining had previously taken place. In August 2006, the first blast was taken for a third open pit, PPRust North, situated on the Zwartfontein and Overysel farms. A fourth open pit, Zwartfontein North, is planned to begin operations in 2007. A concentrator with a capacity to process 200,000 tonnes per month was built at PPRust and was later upgraded to 400,000 tonnes per month. A second concentrator is under construction adjacent to the PPRust North open pit and it is designed to process 600,000 tonnes per month.

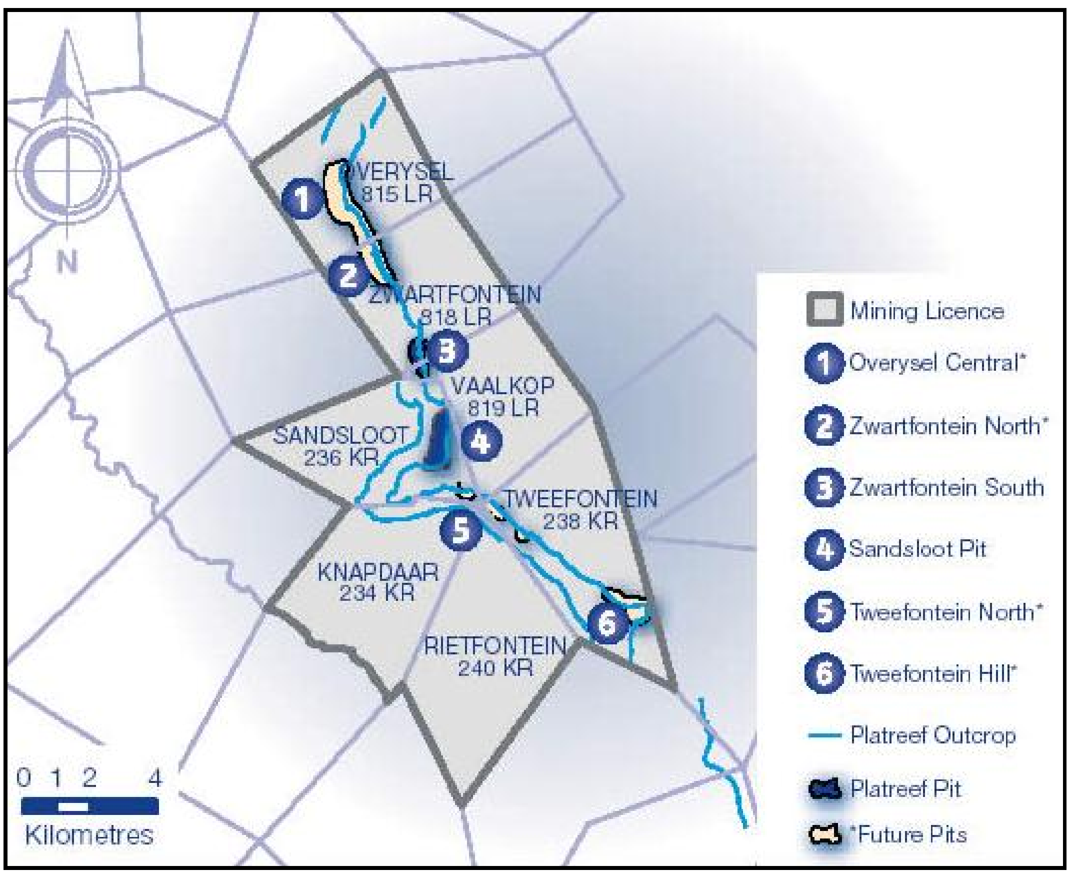


Figure 2.2 Farms and open pits on the PPRust lease area (AP, 2006)

2.3 Mining and Processing

Sandsloot open pit, the world's largest platinum open pit, is currently 1.6 km long, 600 m wide, 260 m deep and has a crest elevation of 1100 m AMSL. It is expected to operate until 2009 when it will reach a final depth of 300 m. Zwartfontein South open pit, situated 1 km north-west of Sandsloot, is 1.5 km long, 500 m wide and 120 m deep and is expected to operate until 2015. Together the 2 pits produced 57 million tonnes of rock and 4.8 million tonnes of ore in 2005. The operation is expanding and the production targets for 2006, 2007 and 2008 are 67 Mt, 100 Mt and 120 Mt respectively. A relatively small footprint is available in each pit resulting in mining congestion, and mine scheduling is a challenge.

PPRust is highly mechanized and the Mining Section only employs 450 permanent staff in the Drill and Blast, Load and Haul and Engineering departments. A fleet of 23 CAT 785B haul trucks (136 t), six Terex MT3700B haul trucks (187 t), one Terex MT4000B (200 t) truck and one CAT 793 haul truck (218 t) are used along with four O&K RH200 face shovels (20 m^3) and one RH340 face shovel (28 m^3). Drilling is done mostly by contractors who use 165 mm waterwells. The mine owns two Ingersoll Rand Pit Viper 260 (271 mm) drill rigs, one Drilltech 1190E (250 mm) drill rig and one Drilltech D40 KSH (165 mm) drill rig. Front-end loaders, backhoes, dozers, graders and water trucks are also owned and used by PPRust Mining. The ramps are 35 m wide and have a gradient of 10%. Sandsloot and PPRust North have a bench height of 15 m while Zwartfontein South has a more conservative 10 m bench to reduce ore dilution. The pits are scheduled to expand in a series of phased cutbacks. Figures 2.3 illustrates the six cutbacks for Sandsloot pit.

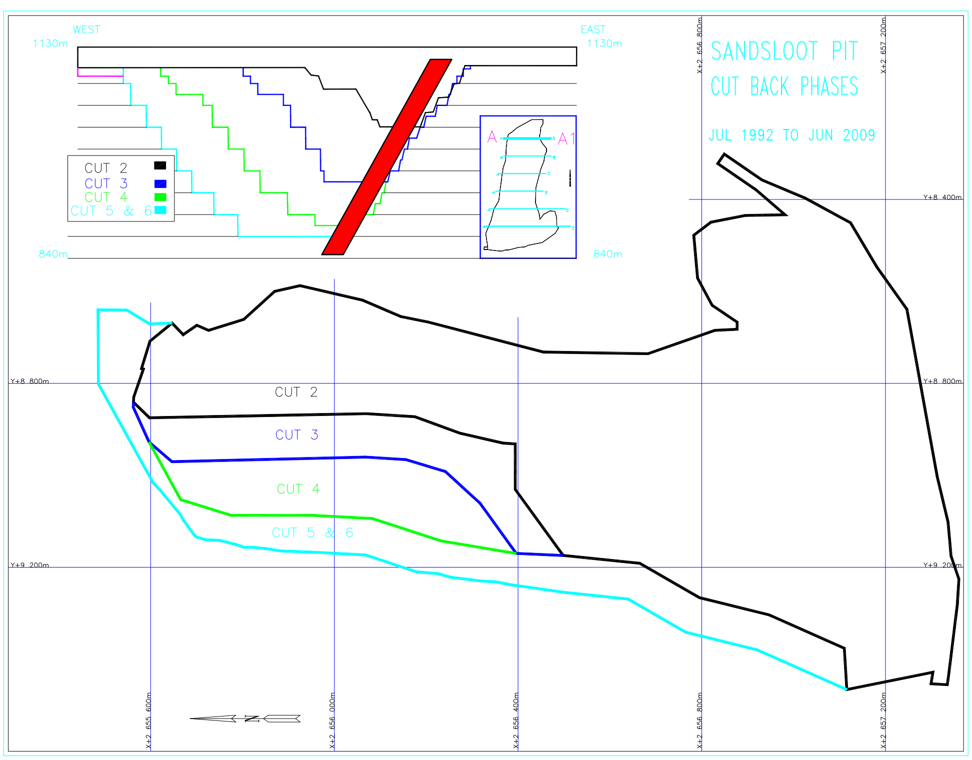


Figure 2.3 Plan and section views of Sandsloot cutback phases (after Bye, 2003)

Blasting is scheduled once a week in each pit with between 0.5 and 1.5 Mt of rock blasted and loaded out. Two contractors, African Explosives Limited (AEL) and Bulk Mining Explosives (BME), supply explosives and technical skills for blasting in the open pits. Grade control sampling is done on all ore blastholes and the blasted muckpile is delineated with colour coded tapes. The ore is separated into five categories, namely low low grade (LLG), high low grade (HLG), G1, G2 and G3 (Figure 2.4). The LLG and HLG (1.7 – 3 g/t) are permanently stockpiled and will be processed near the end of life of mine. Ore with a grade greater than 3 g/t is temporarily stockpiled adjacent to the primary crusher. Smaller trucks operated by contractors move the ore from the stockpiles to the primary crusher.

Below the crusher, the ore goes through a 150 mm grizzly and is split into coarse material (+150 mm), which goes to the conical stockpile and fines (-150 mm), which go to the A-frame(Figure 2.5). The concentrator is then fed by a mix of coarse and fines in a 40% - 60% ratio at an average grade of 4 g/t. The ore passes through two autogenous mills and two ball mills before it enters the flotation cells. The milling rates are dependant on rock hardness and particle size (or fragmentation) thus good blasting can go a long way to optimizing plant performance. The final concentrate produced is ~100g/t and is sent to Anglo Platinum's smelter in Polokwane. From the smelter, the furnace matte is transported to Anglo Platinum's Base Metals refinery in Rustenburg where copper, cobalt and nickel are extracted. The final stage in the refining process is done at the Precious Metals Refinery in Rustenburg where platinum, palladium, rhodium and gold are recovered.

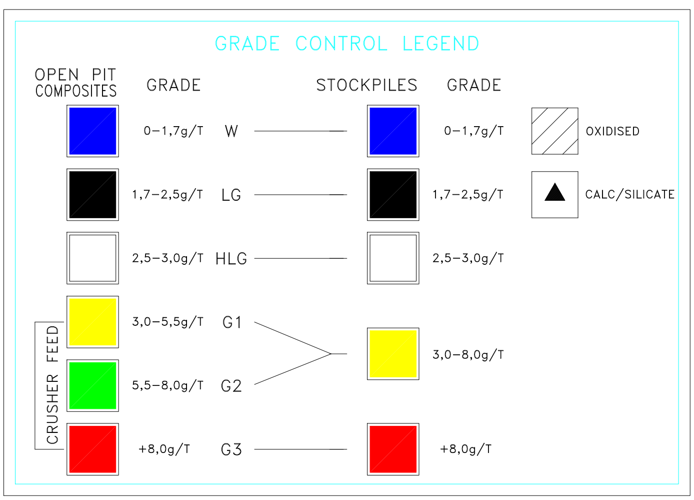


Figure 2.4 Grade categories at PPRust (PPRust, 2005)

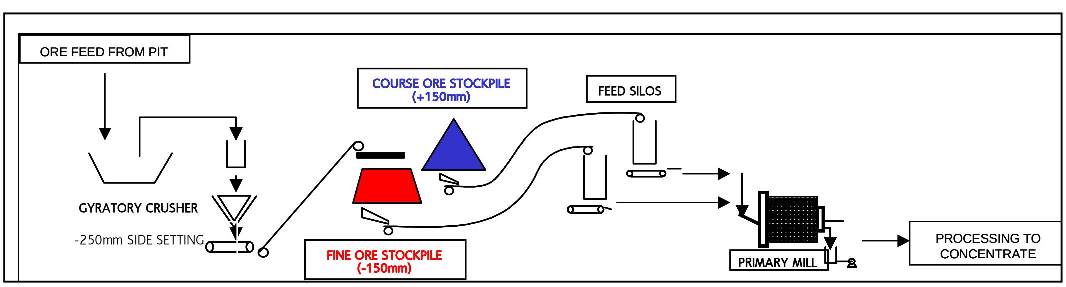


Figure 2.5 Simple diagram of the PPRust plant feed process (Bye, 2003)

2.4 Exploration

The economic strike length of the Platreef is 42 km and Anglo Platinum owns the mineral rights for half of this strike length. In the last few years, Anglo Platinum's exploration drilling on the Platreef has been one of the biggest exploration programs in the world with over 400 km of core drilled since 2001. In 2003, 168 km of core were drilled by 28 drill rigs at a rate of about 1 km a day. Drilling on a 70 m by 70 m grid has been done across the lease area (Figure 2.6) with a 50 m by 50 m grid drilled in places. Borehole depths have reached 2 km and the Platreef is continuous over this depth and is believed to continue to much greater depths.

The core is geologically logged and the entire reef is sampled in 50 cm sections. In-pit drilling on a 35 m grid has also been done in the current open pits on every second or third bench to improve the confidence in the geological block models. The logging data and assay results are stored in a SABLE Warehouse database and imported into Datamine Studio where resource block models are created per pit. This exploration drilling extended over the Overysel, Zwartfontein, Vaalkop, and Tweefontein farms. The new open pits, PPRust North and Zwartfontein North, developed out of this drilling program and the results also indicate that further mining can be done south-east of Sandsloot on the Tweefontein farm. Two operations, Tweefontein North and Tweefontein Hill, are planned as shown in Figure 2.2.

2.5 Conclusion

Potgietersrust Platinums Ltd (PPRust) is Anglo Platinum's only open pit operation. The Platreef orebody is a PGM deposit and contains economic quantities of platinum, palladium, rhodium, gold, copper and nickel, which are all extracted and processed by Anglo Platinum. It was discovered in 1924 by Dr Hans Merensky and underground mining was done in the 1920s. In 1992, the first open pit mine had its first blast and two more open pits have begun operations since then. It is a mechanised load and haul operation and ore is processed on site in a concentrator. A large amount of exploration has been done in the last few years and future open pits and underground operations are feasible. The following chapter will explain the geological setting of the PPRust operation both on a regional and local scale.

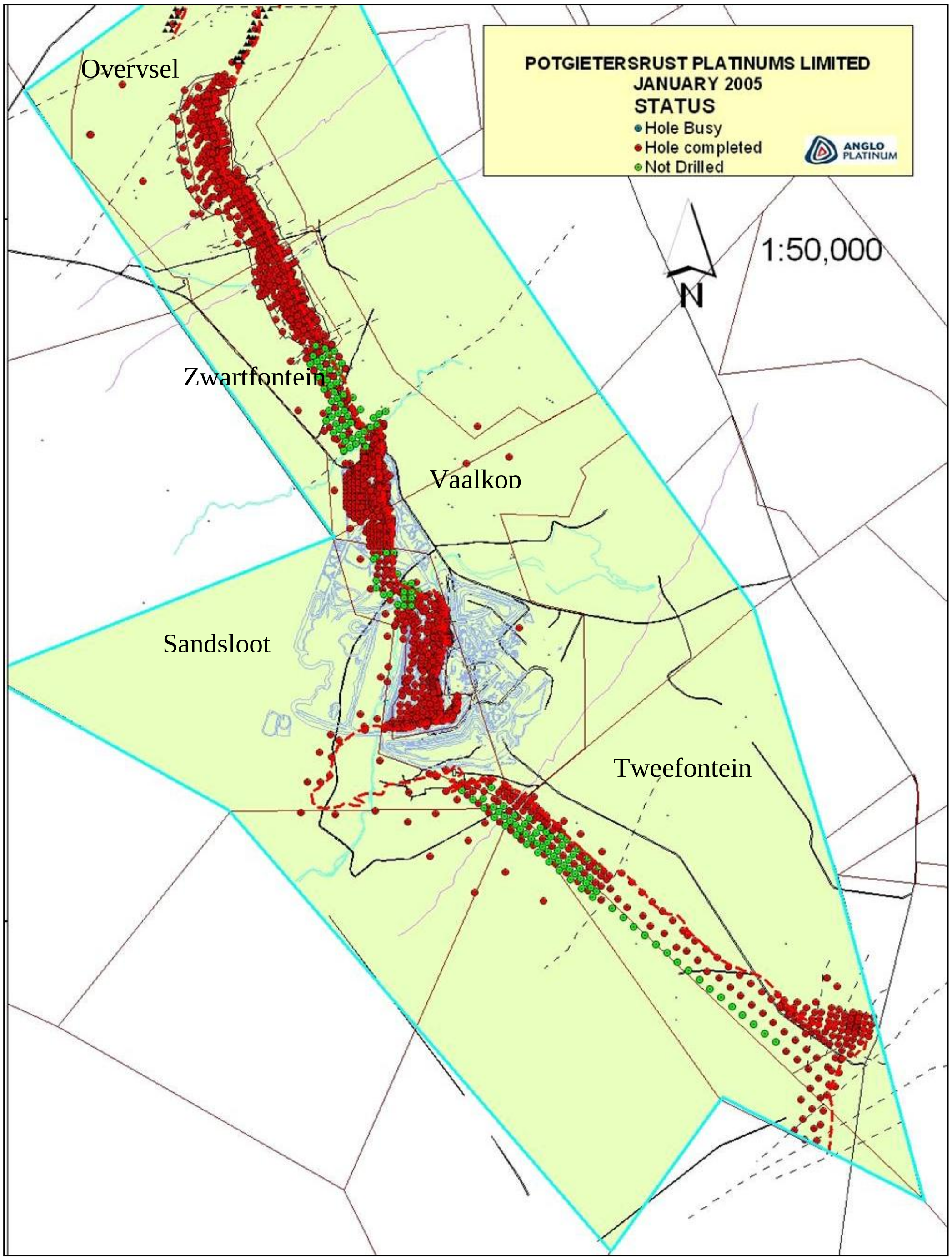


Figure 2.6 Exploration drilling at PPRust as at January 2005 (PPRust, 2005)

3 GEOLOGY

"Discovery consists of seeing what everybody has seen and thinking what nobody has thought"

- *Albert von Szent-Gyorgyi*

The basis of any rock engineering analysis begins with understanding the rock mass. This chapter aims to give a detailed description of the genesis, history and current state of the rock mass that PPRust operates within. The regional geology is explained, followed by a description of the local geology and structure. Details of the joint sets are given as they play a large role in slope stability.

3.1 Bushveld Igneous Complex

The Bushveld Igneous Complex (BIC) is the largest layered intrusion in the world and was emplaced 2060 – 2054 Ma ago (Walvaren *et al.*, 1990). The first phase of the BIC emplacement was the intrusion of a 6-9 km thick succession of mafic to ultramafic rocks called the Rustenburg Layered Suite (RLS) which was soon followed by an acidic phase of granites. The RLS intruded into the Transvaal Supergroup in northern South Africa and has an areal extent of 65 000 km^2. It consists of five compartments or limbs (Figure 3.1) namely, the Far Western, the Western, the Eastern, the (covered) Bethal and the Villa Nora-Potgietersrus Limb in the north.

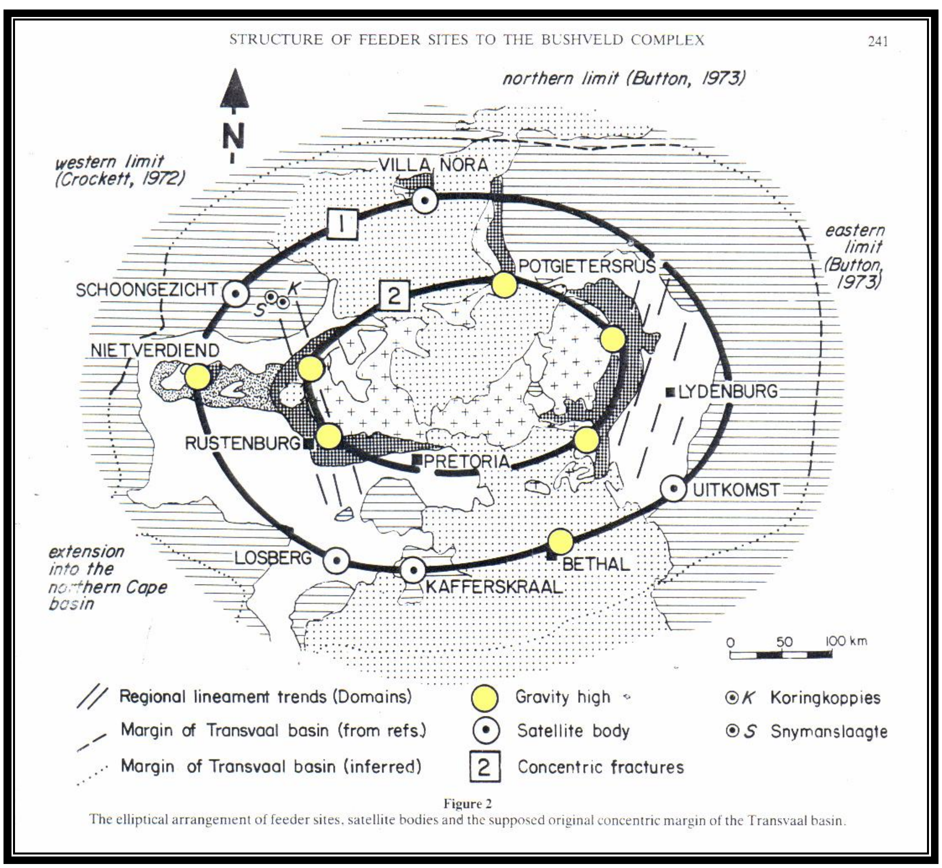


Figure 2
The elliptical arrangement of feeder sites, satellite bodies and the supposed original concentric margin of the Transvaal basin.

Figure 3.1 Location of the 5 limbs and the gravity highs indicating feeder sites (Sharpe *et al.*, 1981)

A number of models have been proposed for the emplacement of the RLS and the complex was originally thought to be one large lopolith that intruded from a central feeder. The stratigraphical variations identified in the five different compartments in the RLS indicated that the layered suite is not that simple. In 1981 Sharpe *et al.* proposed, on the basis of gravity data, that a number of funnel-shaped feeders lying on two ellipses, both concentric with the Transvaal Basin margin, were the conduits for the massive volumes of magma. They theorised that mantle diapirism acting along the Bushveld-Great Dyke lineament produced the BIC and created conical fractures, which formed magma chambers that were subsequently filled. They proposed that the location of the seven feeders is controlled by the regional grain of the Kaapvaal Craton basement. Cawthorn and Webb (2001), however, denied that the gravity data proved the existence of multiple feeder sites claiming that the isostatic response of the crust to emplacement of this huge mass of mafic magma could account for the Bouguer gravity map. They brought evidence that the western and eastern limbs are indeed connected though they did admit that connectivity of the other limbs is still in question. Friese (2002) has proposed a non-plume model (Figure 3.2), where deep-seated suture zones are responsible for the emplacement of the vast volumes of the BIC.

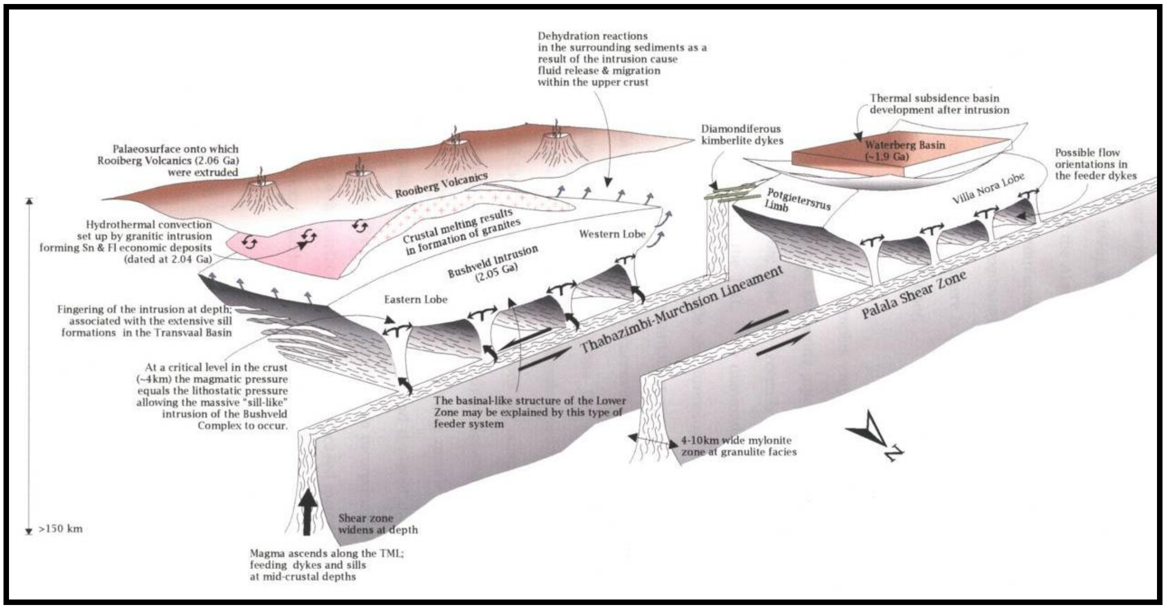


Figure 3.2 Non-plume model for the emplacement of the Bushveld Igneous Complex (Good, 1999)

Friese (2002) proposed that the continent-continent collision during the Ubendian Orogeny ~2.05-1.8 Ga ago induced NW-SE compressive stress fields and reactivated the Neoarchaean sutures in the Kaapvaal Craton basement. This allowed for decompression melting in the lithospheric mantle and mafic to ultramafic magmas subsequently formed. As shown in Figure 3.2, the magmas rose along the palaeosutures (for example the Thabazimbi-Murchison lineament) and were emplaced ~2,05 Ga ago.

3.2 The Northern Limb

The feeder for the northern limb lies just west of Mokopane (previously Potgietersrus), 250 km north-east of Johannesburg, and can be identified by the Potgietersrus gravity high (Figure 3.1). It lies on the intersection between the Bushveld-Great Dyke lineament and the Pietersburg lineament (Von Grunewaldt, 1979). This, in conjunction with the differences in stratigraphy and thickness to the rest of the RLS, indicates that at least some structural control on the emplacement of the northern limb can be assumed. This northern limb covers 2 000 km^2, strikes north-south for 120 km and has a maximum width of 15 km.

The RLS can be divided into five zones, namely the marginal, lower, critical, main and upper zones, in order of decreasing age and depth, which relate to separate magma injections (Ainsworth, 1994). In the northern limb, the older Marginal Zone is absent and the poorly developed Lower and Critical Zones only appear south of Mokopane. The Platreef lies at the base of the Main Zone and is overlain by gabbronorites which are in turn overlain by Upper Zone ferrogabbros (Figure 3.3). The Platreef shows a transgressive relationship with the floor rocks so that its base lies at progressively higher levels in the stratigraphical succession. The floor rocks south of Zwartfontein South belong to the Transvaal Supergroup while the floor rocks north of Zwartfontein South are Achaean granites (Figure 3.3). The Transvaal Supergroup comprises a basal clastic unit, the Black Reef Formation, overlain by dolomites and ironstones of the Chuniespoort Group which is covered by the clastic Pretoria Group which also contains volcanic layers. The Platreef transgresses the underlying Magaliesberg quartzites south of Mokopane, across the Malmani dolomites and Penge banded ironstones (Pretoria and Chuniespoort Groups) and northwards over Archaean Utrecht granites (Harris and Chaumba, 2001).

The basal contact is highly irregular and metamorphism, metasomatism and assimilation have resulted in a complex group of hybrid rock types, namely calc-silicates, parapyroxenites, serpentinites and Granofels, forming in the PPRust area. According to Buchanan *et al.* (1981), sulphide mineralisation is associated with the intrusive contact between the pyroxenite and dolomite, both of which contained sulphur, and the combination resulted in the platiniferous reef that exists today. This is supported by Barton *et al.* (1986) as well as Harris and Chaumba (2001), who used oxygen isotopes to confirm that the intruding magma was contaminated by the footwall dolomites as well as magmatic fluids, which played a large role in the mineralisation. The Platreef plays host to a number of metals, most notably the platinum group metals, as well as gold, silver, nickel, copper and cobalt. The PGE mineralisation, previously thought to be orthomagmatic in origin, has been shown to be affected by post-magmatic hydrothermal fluids indicating a complex deposit that has been subjected to a number of processes during its development (Armitage *et al.,* 2002).

The Platreef orebody strikes roughly north-south in a sinuous outcrop pattern and dips roughly 45° to the west. It steepens to 75° at Tweefontein Hill where the footwall rocks display a synformal structure. At Sandsloot the reef swings east-west around a 'dolomite tongue' which represents a diapir of footwall dolomite (now metamorphosed to calc-silicate) that rose into the magma chamber. Diapirs are common in the north-eastern BIC and are preserved as large domal structures exposed on the margin and within the layered suite. They developed during the emplacement of the lower zone of the BIC as intrusion fingers and are the result of gravitational loading and heating of floor rocks (Uken and Watkeys, 1997).

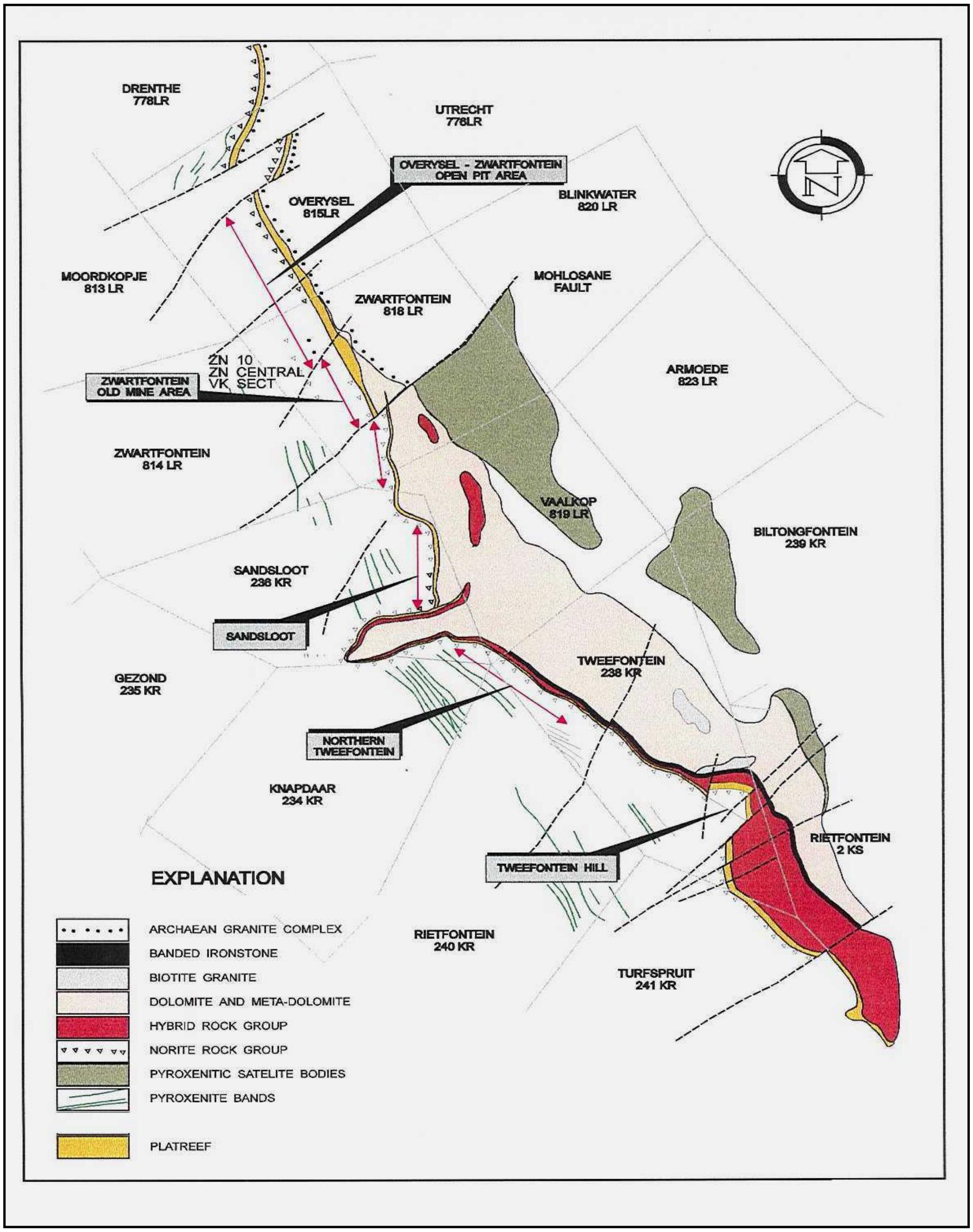


Figure 3.3 Geology outcrop plan of the Northern limb in the PPRust lease area (PPRust, 2005)

3.3 PPRust Lease Area

At PPRust, the Platreef has historically been divided into a C-Reef (medium grained feldspathic pyroxenite), a B-Reef (coarse grained pyroxenite) and an A-Reef (pegmatoidal feldspathic pyroxenite) which forms the base of the reef (Figure 3.4). The C-Reef is often unmineralised and the A-Reef varies greatly in width and grade. The B-Reef has historically had the most consistent and highest grades. At Sandsloot and Zwartfontein South open pits the interaction between the Platreef and the Malmani dolomites has resulted in the formation of calc-silicates and parapyroxenites in the footwall though it is mineralised in places. The original bedding planes are still evident in the calc-silicate and varying degrees of alteration are thought to reflect primary compositional variations. Serpentinisation is pronounced at Zwartfontein South pit and three geological zones have been identified by production geologists. A large calc-silicate xenolith is present in the south-east. At PPRust North, the Platreef intruded above Archaean granite and this has resulted in a granofels footwall. At Tweefontein, bnded iron formation and shales form the country rock and the interaction with the Platreef formed a hornfels footwall.

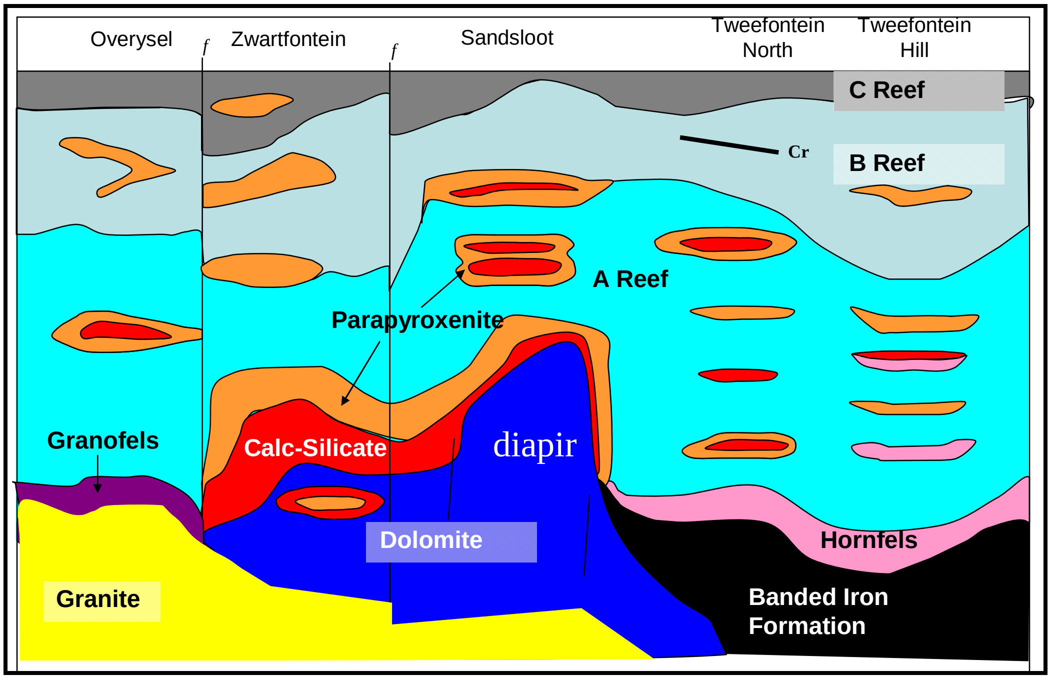


Figure 3.4 Sketch of the change in geology over the PPRust lease area (PPRust, 2005)

3.4 Structure

The PPRust lease area has a complex structural history and is intersected by the following major structures (SRK, 2003):

- Shallow NW-dipping, SE-directed thrusts/thrust zones and associated ENE-trending, sub-horizontal, low-amplitude regional folds within all lithologies. They are the earliest structures dated ~2.1- 2.58 Ga.
- NW- to WNW-trending, moderate- to steeply dipping extensional faults/fault zones that formed ~2.058- 1.86 Ga ago within the Transvaal Supergroup and BIC by reactivation of the Pongola rift fault system developed in the underlying Archaean basement during the Murchison Orogeny.
- ENE- to NNE-trending, steep to sub-vertical predominantly SE-dipping dextral strike-slip shear zones with associated NE-directed, layer/bedding-parallel thrusts/thrust zones developed in shear zone-bounded domains. They formed within the Transvaal Supergroup and BIC by reactivation and above a Neoarchaean sinistral strike-slip system, which developed within the underlying Archaean basement in response to tectonism during the Limpopo Orogeny.

- N-S striking, moderate W-dipping extensional fault zones, with typical undulating gross geometry and an imbricate fan of combined normal dip-slip and sinistral strike-slip duplexes in their immediate hangingwall. This juvenile rift fault system developed within Archaean basement, Transvaal Supergroup and RLS at ~1.35- 1.2 Ga.
- WNW- to WSW-trending extensional fracture/joint zones cross-cut all other structural discontinuities without significant displacements and represent the youngest group of structures at ~1.15- 1.0 Ga.

During and since the break-up of the supercontinent, Gondwana, ~210 Ma ago minor extensional reactivation of all pre-existing structures and the formation of new NE-SW, E-W and ESE-WNW striking extensional fracture zones has occurred. Over the past ~35 Ma, neotectonic NW-SE extensional horizontal far field stress conditions have caused minor tectonic rejuvenations. Structural discontinuities striking approximately parallel to the NE-SW sub-vertical strike-slip shear zones, are kept open and act as natural drainage paths for groundwater. The NW-SE striking structural discontinuities are closed and not water-bearing. The amount and direction of displacement evident on the faults are the product of a long kinematic history spanning over more than ~2 Ga.

3.4.1 Sandsloot pit

Two major faults cross the Sandsloot area, namely the Satellite pit fault in the south-east and the Sandsloot fault in the north-west (Figure 3.5). These are thought to form a major duplex enclosing the Sandsloot pit and numerous smaller duplexes (SRK, 2003). The satellite pit fault has downthrown the orebody by approximately 400 m to the south-east. A satellite pit was excavated to extract this ore and was backfilled once all the ore was removed. Five joint sets, named JS1 to JS5, as well as relict bedding in the calc-silicate footwall, have been identified in the pit and are evident in the stereonet in Figure 3.6 and described in Table 3.1. JS1 and JS3 are the most prominent and coincide with the regional fault zones. JS2 and JS4 are fairly discontinuous while JS5 is found only on the northwest wall. JS3 is the main carrier of groundwater which flows from NE to SW in the region.

JS1 is associated with a major, 100 m wide fault zone which cross-cuts the entire length and depth of the west wall. This strikes parallel to the Sandsloot River (west of the pit) where the joint spacing is 0.1 - 0.5 m. Duplexing occurs within this fault zone resulting in an imbricate fan of joints with a change in dip and dip direction of up to 20°. This range in dip and dip direction can be seen in the stereonet of mapping and logging in Sandsloot in Figure 3.5. The joints within the zones are smooth undulating or smooth stepped and usually have calcite infilling. Roughly every 5 m laterally, a minor fault occurs within the zone usually displaying slickensides, serpentinite, iron-oxide staining and calcite infilling. The fault surfaces are generally smooth undulating or smooth planar. The joints within the fault zones have an average dip/ dip direction of 56/092 and therefore run parallel or sub-parallel to the final pit wall and daylight in the benches and stacks, resulting in small-scale planar or stepped-path failure. This will be discussed in more detail in Chapter 4. A highly sheared zone of pyroxenite separates the hangingwall norite from the Platreef. It is roughly 5 m thick, dips 45° to the east and acts as a natural drainage path for groundwater. Hydrothermal quartzo-feldspathic veins strike N-S and cross-cut the pit at regular intervals of about 30 m. The veins have a high intact rock strength of 320 MPa and are associated with JS1.

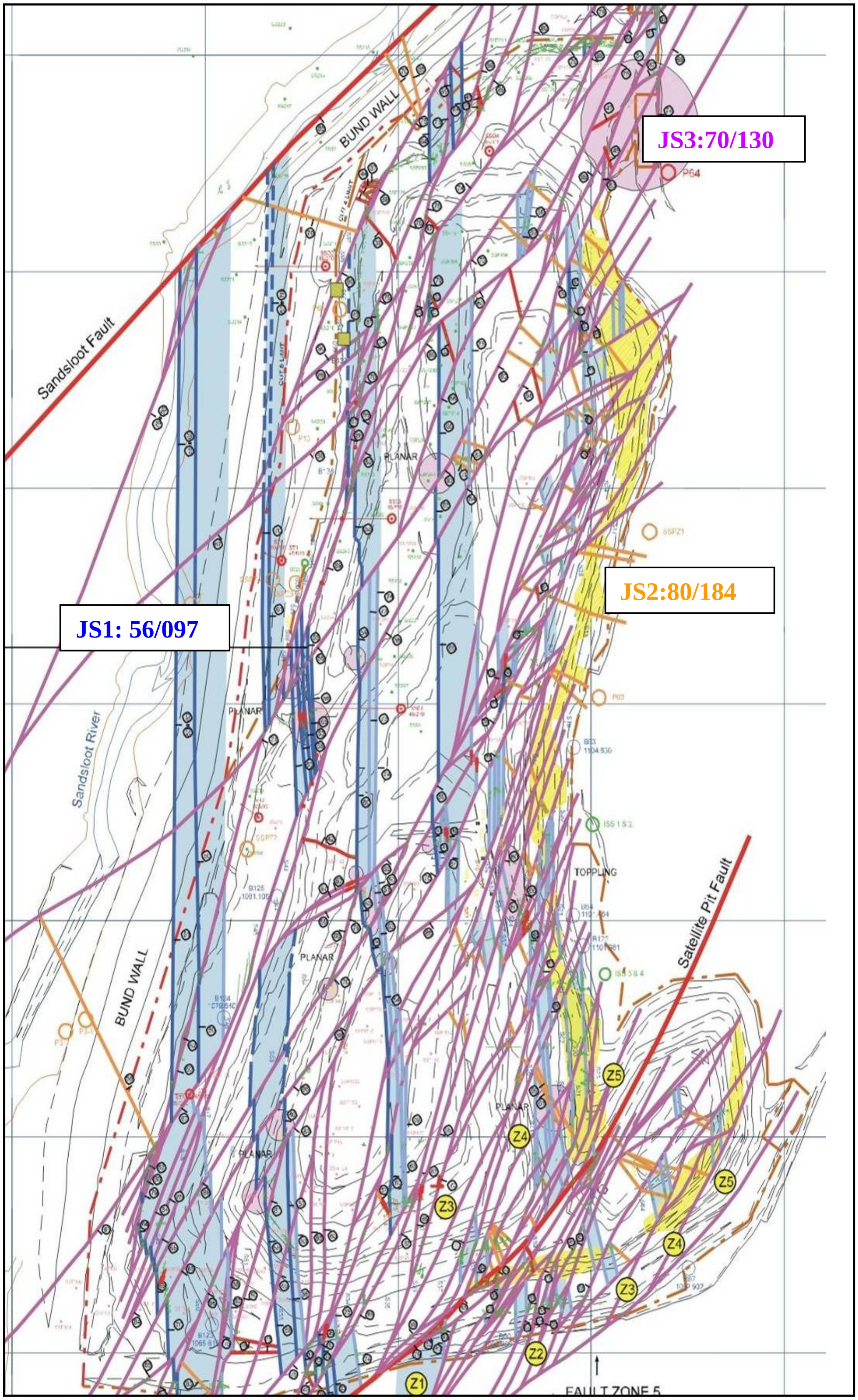


Figure 3.5 Structural Interpretation Plan of Sandsloot (SRK, 2003)

Table 3.1 Summary of the joint set data at Sandsloot

Joint Set	Strike	Dip	Dip Direction	Joint Roughness	Joint Filling	Joint Spacing
JS1	N-S	72° 73°	088° 263°	Rough irregular Undulating	Calcite Serpentinite	0.5m (0.2-3m)
JS2	E-W	78° 82°	357° 183°	Smooth stepped	Calcite	0.4m (0.2-2m)
JS3	NW-SE	70° 62°	310° 125°	Smooth undulating	Serpentinite	0.3m (0.1-5m)
JS4	NNE-SSW	72° 63°	237° 065°	Rough/Irregular Planar	Calcite	0.15m (0.05-0.4m)
JS5	WNW-ESE	12°	305°	Rough/Irregular Planar	Calcite	3m (0.8-5m)
Bedding	N-S	32°	275°	Rough irregular Undulating	Serpentinite	0.5m (0.1-1m)

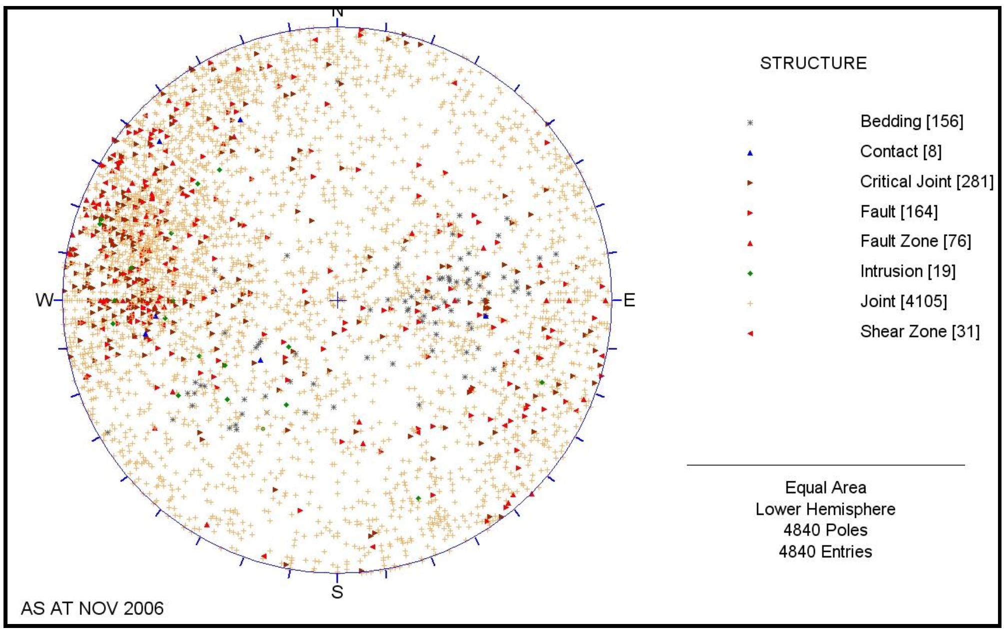


Figure 3.6 Stereonet of all mapping and orientated logging data collected in Sandsloot pit

3.3.2 Zwartfontein South

Zwartfontein South is intersected by 11 major faults which all strike roughly NE-SW (Figure 3.7). The throw on the faults varies from 20 m to 100 m. The five joints sets evident in Sandsloot are present in Zwartfontein South but on a smaller joint spacing, making the highwalls more blocky. There is a sixth joint set, JS6, and the joints are also generally steeper as shown in Table 3.2 and Figure 3.8. A 50 m wide, steeply dipping shear zone strikes NE-SW through the centre of the current open pit and has a joint spacing of 10 cm. The Mohlasane River, north of Zwartfontein South follows the Mohlasane Shear Zone which has offset the Platreef by 400 m. JS1 is not nearly as prominent in Zwartfontein South as in Sandsloot and therefore the stability of the western wall is better.

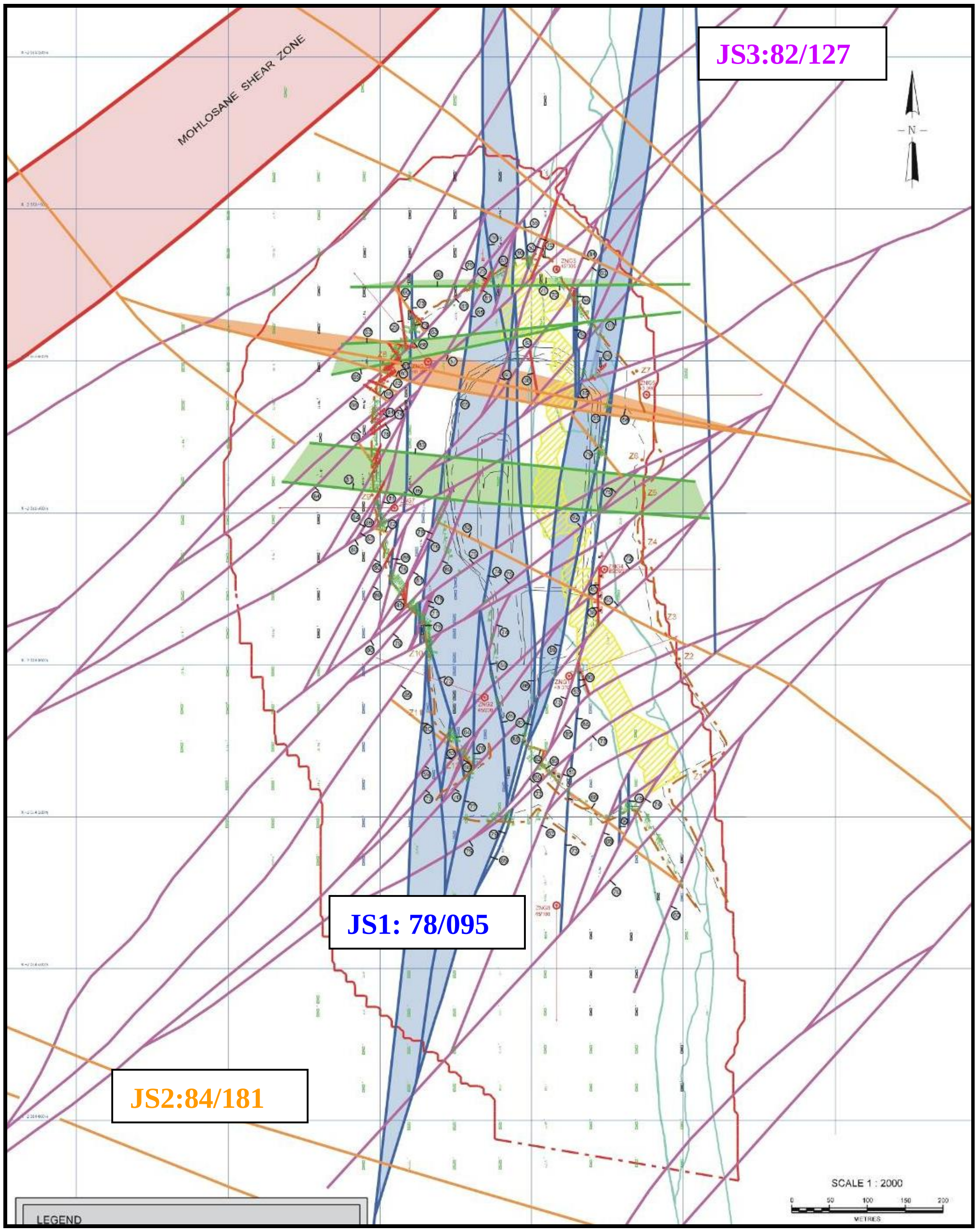


Figure 3.7 Structural interpretation plan of Zwartfontein South (SRK, 2003)

Table 3.2 Joint sets evident in Zwartfontein South open pit

Joint Set	**Strike**	**Dip**	**Dip Direction**	**Dip Range**	**Dip Direction Range**	**Dominant location**
JS1	N-S	78° 84°	95° 270°	58-86 75-84	84-113 258-277	Central pit, E & W
JS2	E-W	84°	181°	73-90	170-198	N wall
JS3	NE-SW	82°	127°	64-90	120-150	SW corner
JS5	NNW-SSE	2° 19°	68° 260°	0-20 0-35	162-317 180-293	Whole pit
JS6	NW-SE	83°	39°	74-90	22-56	SW corner

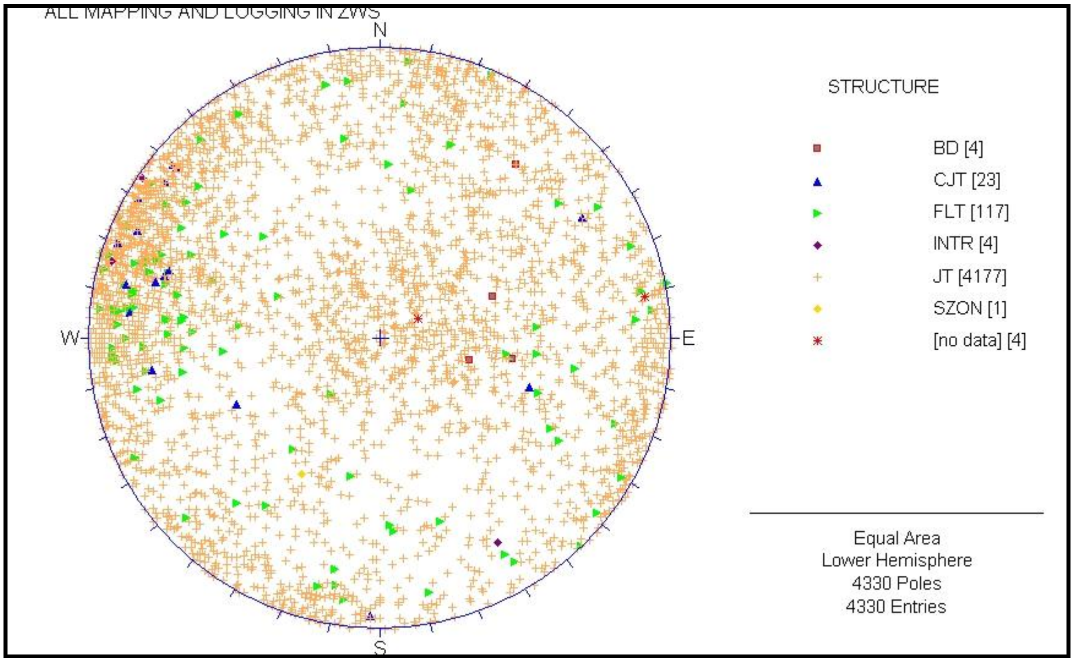


Figure 3.8 Stereonet of all mapping and orientated logging collected in Zwartfontein South pit

3.4.3. PPRust North

PPRust North is currently on its first bench so there is limited structural data. Ten orientated boreholes have been drilled however and stereonet analysis has indicated five joint sets detailed in Table 3.3. Figure 3.9 is a stereonet of the 2996 structures recorded in the orientated borehole core drilled in 2001. JS1 and JS5 appear to be the prominent joint sets as well as a seventh joint set, JS7, which strikes NE-SW and dips 38° to the north-west.

Table 3.3 Joint sets evident in PPRust North orientated borehole core

Joint Set	Strike	Dip	Dip Direction	Dip Range	Dip Direction Range
JS1	N-S	57°	92°	41 – 75°	82 – 114°
JS2	E-W	84° 84°	185° 004°	80 – 90° 83 – 90°	171 -192° 343- 022°
JS3	NW-SE	80° 85°	125° 291°	72 – 90° 79 – 90°	115 – 138° 282 – 304°
JS4	NNW-SSE	68°	346°	65 – 78°	338 – 351°
JS5	WNW-ESE	5° 2°	303° 110°	0 – 16° 0 – 8°	300 – 322° 100 – 120°
JS7	NE-SW	28°	334°	10 – 40°	300 – 341°

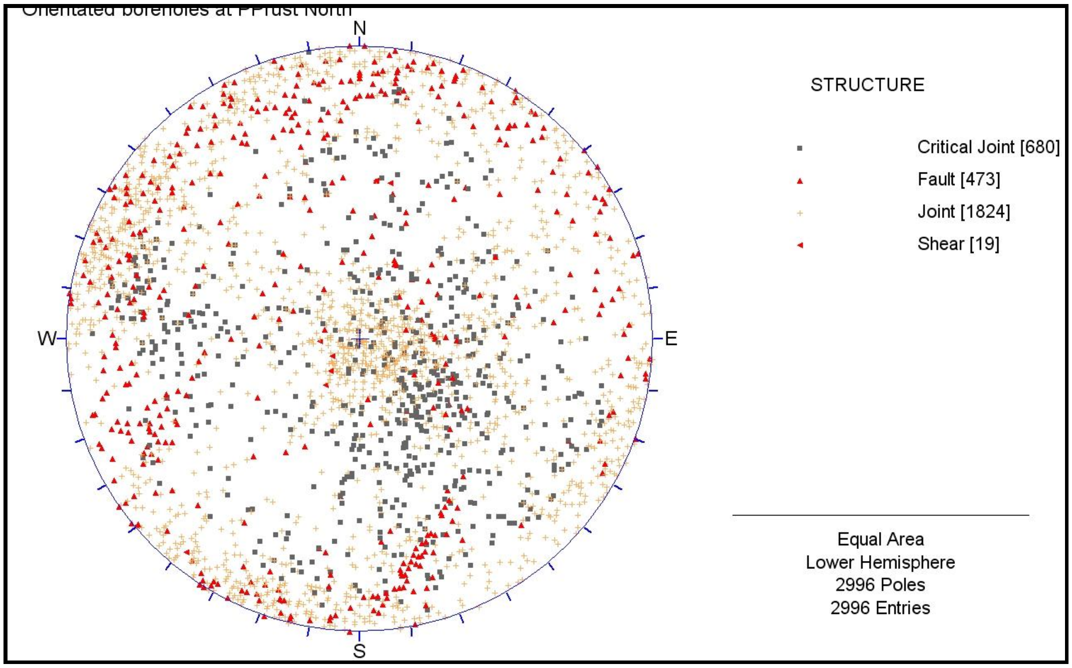


Figure 3.9 Stereonet of orientated logging for PPRust North

3.5 Geohydrology

PPRust is situated in the Mogalakwena River catchment (Figure 3.10) adjacent to the Groot Sandsloot River, which has a mean annual runoff of 1.731×10^6 m^3 (Bye, 2003). The annual groundwater recharge rate is 1.86×10^6 m^3 /a. Annual rainfall varies between 350 mm and 700 mm and falls in the summer from November to March. The area surrounding the mine is a fractured rock mass that has few primary aquifers but instead water is stored in discontinuities. This type of aquifer has low storage potential and is unpredictable. The main aquifer occurs at the base of the weathering profile which sits anywhere from 2 m deep to 43 m deep. The mine has three wellfields, which form a series of elongated troughs sub-parallel to the strike of the Platreef (Bye, 1999). The highest yielding wells are located within weathered pyroxenite, as the permeability is higher and varies less than in other weathered rock types. Campbell and Heidstra (1994) concluded that the aquifer had a transmissivity value of 60 m^2/day, storativity of 4×10^{-3} and a hydraulic conductivity 10^{-3} cm/sec.

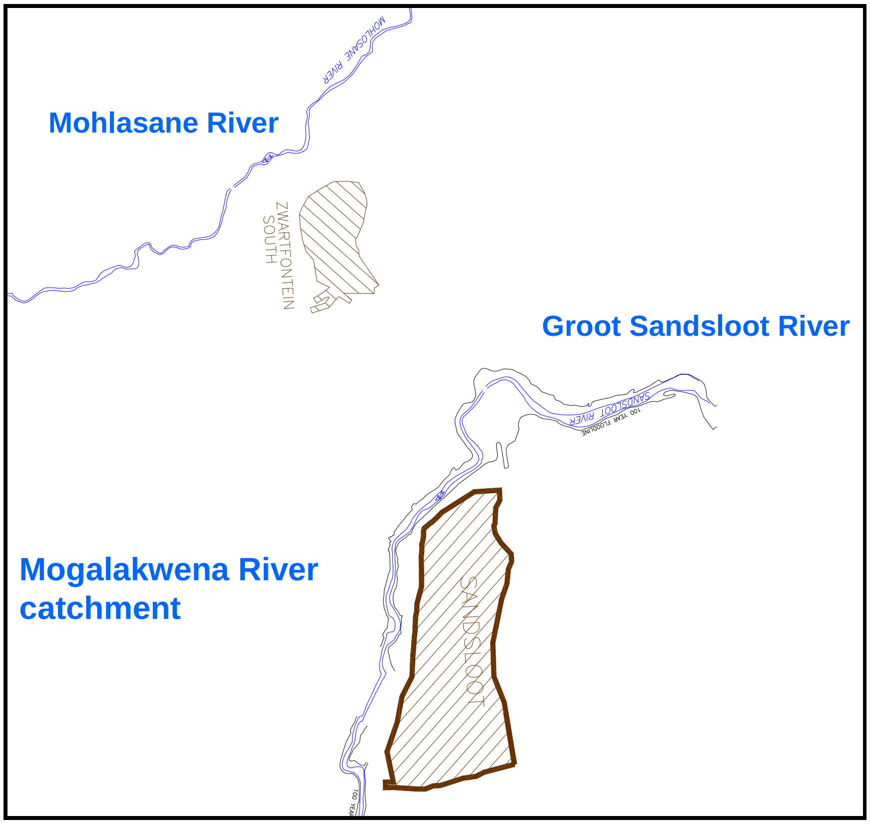


Figure 3.10 Plan of major rivers in the PPRust lease area (PPRust, 2003)

Groundwater flows from north-east to south-west predominantly in the faults and joints of JS3. This is evident in the open pits where the northern and eastern walls are usually wet while the western and southern walls remain dry during winter and only a few joints allow water flow in summer. The shear zone at the contact between the norite hangingwall and the pyroxenite reef in Sandsloot is a large carrier of groundwater and is usually flowing. The open pits have limited influence on the groundwater in the surrounding area, affecting a radius of 400 m (SRK, 2003).

3.6 Conclusion

PPRust is situated on the Northern Limb of the Bushveld Igneous Complex and targets the Platreef, a pyroxenitic orebody dipping 45° to the west. The reef is overlain by a thick norite package and transgresses a number of footwall rocks including calc-silicate, granofels and hornfels which were all formed as a result of the interaction between the Platreef and the country rocks during emplacement. Regional structures are dominated by NE-SW faults while seven joint sets have been identified in the three pits. Rainfall is low in the area and groundwater flow is structurally controlled. This data forms the basis for geotechnical work at PPRust which is described in more detail the following chapters.

4 FIELD DATA

"The origin of the science of classification goes back to the writing of the ancient Greeks but the process of classification, the recognition of similarities and the groupings of objects based thereon, backs to primitive man."

- Prof. Robert R. Sokal

In order to obtain and build on an initial understanding of the geology and structure at a mine, detailed geotechnical data collection is done. This Chapter describes the geotechnical data collection methodology at PPRust. It includes logging, mapping and rock testing and the databases that are used to store the relevant data as well as the integration and visualisation of the various data types.

4.1 Introduction

Primary geotechnical data is collected by borehole core logging, by surface and face mapping and by field and laboratory rock testing. This data is used to understand the rock mass that is being mined and to determine parameters that are used for analysis. If the quality and quantity of data is not sufficient than the analysis and resultant designs will be inaccurate. It is important that the correct data is collected according to international standards, that it is stored in stable databases and is optimally and appropriately used. At PPRust, all diamond drilling core is logged geotechnically and point load tested by contractors. Orientated drilling and lab testing is done as required. Face mapping is performed when time allows and point load tests are done for all face maps. The logging and window face mapping are used to determine rock mass ratings and the point load test and UCS lab test results are also used in these calculations. All of this data is stored in two databases, SABLE and MineMapper3D, which were customised for Anglo Platinum at PPRust. The rock mass ratings are used for geotechnical zoning and slope and blast design. Orientated drillhole logging and line survey mapping are used for the identification of faults, shears and joint sets, which in turn are used for kinematic failure analysis and structural modelling. Other laboratory tests are used for slope analysis and plant design.

4.2 Rock Mass Classification Systems

The first rock mass classification systems were developed in the 1940's and 50's but were quite simple. In the 1970's major advances were made in the field resulting in today's internationally accepted Barton *et al.*'s Rock Quality Index, Q (1974) and Bieniawski's Geomechanics Classification (or Rock Mass Rating, RMR) (1973) systems. Since then these two systems have been refined, modified and correlated by the authors as well as other geotechnical engineers but they still remain the most popular.

The purpose and requirements of a rock mass classification system according to Bieniawski (1973) are that it should:

- Divide a rock mass into groups of similar behaviour
- Provide a good basis for understanding the characteristics of a rock mass
- Facilitate the planning and design of structures in rock by yielding quantitative data

- Provide a common basis for effective communication among all persons concerned
- Be simple and meaningful in terms
- Be based on measurable parameters which can be determined quickly and cheaply in the field.

Until 2003, PPRust used Laubscher's Mining Rock Mass Rating (MRMR) system (1990), which is a modification of Bieniawski's (RMR). Anglo Platinum's underground operations used only Bieniawski's RMR and Barton *et al.*'s Q systems. A logging standard was required for Anglo Platinum thus all three rock mass rating systems were combined into one log. Hoek *et al.*'s Geological Strength Index (1995) was also included for its simplicity and the hope that site-specific correlations would be found that could save time later on. Using a single log instead of three or four separate logs was chosen for the time and cost savings that it would provide. Using four rating systems enables quality control and auditability and ensures that the standard can be used at all Anglo Platinum operations. Hoek (2001) recommends that at least two methods be used at any site during the early stages of a project as different classification systems place different emphases on the various parameters.

Figure 4.1 illustrates the similarities and differences between the four rock mass rating systems used at PPRust. The three main systems have modified versions of their ratings and Laubscher produced a new version in 2000. The figure shows the rating as a combination of the individual parameters and the dotted lines group parameters together. It is evident that there is much in common between the systems and this enabled a single log to be developed. It also indicates that there are important differences, which means that the results from each classification system must be used thoughtfully, with the correct application. In the following sections each rating system will be briefly described.

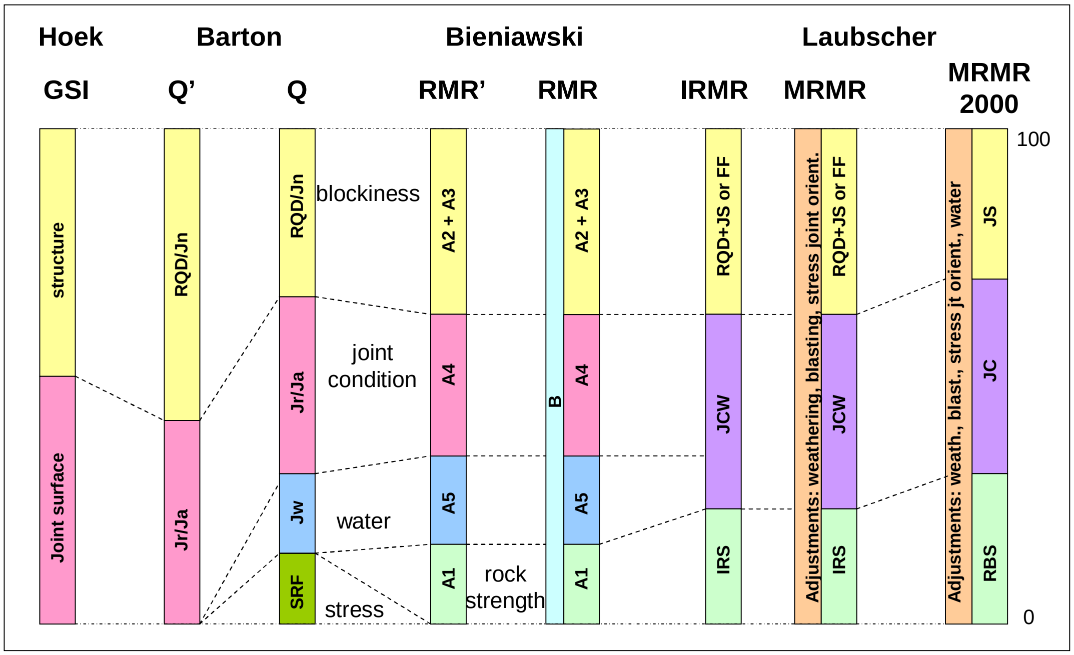


Figure 4.1 Comparison of the four rock mass rating systems used at PPRust

4.2.1. Rock Tunnelling Quality Index, Q

The Q system was developed by Barton, Lien and Lunde in 1973/4 and was based on over 200 case records of underground excavations using face mapping as the primary data. It aimed to provide a rating on which to base tunnel support design for the whole spectrum of rock qualities. This system classifies rock masses with respect to six in situ parameters as shown in Table 4.1.

Table 4.1 Barton *et al.*'s Q system parameters and ratings

Parameter	Abbreviation	Rating Range
Rock Quality Designation	RQD	10 to 100
Joint Set Number	Jn	0.5 to 20
Joint Roughness Number	Jr	0.5 to 4
Joint Alteration Number	Ja	0.75 to 20
Joint Water Reduction Factor	Jw	1.0 to 0.05
Stress Reduction Factor	SRF	0.5 to 15

The rock quality index, Q is determined using the following formula:

Q = (RQD/Jn) * (Jr/Ja) * (Jw/SRF)

The formula produces a result that ranges from 0.001 to 1000 which has been categorised into nine rock mass classifications shown in Table 4.2.

Table 4.2 Q rock mass classifications

Q Rating	Rock Mass Description
0.001 to 0.01	Exceptionally Poor
0.01 to 0.1	Extremely Poor
0.1 to 1	Very Poor
1 to 4	Poor
4 to 10	Fair
10 to 40	Good
40 to 100	Very good
100 to 400	Extremely good
400 to 1000	Exceptionally good

The advantage of the system is that it not only provides a descriptive scale such as poor, fair and good, but also gives values for principal properties controlling tunnel stability in rock masses. The equation is made up of three terms and each one represents a certain characteristic of the rock mass.

- **RQD/Jn** represents the overall structure of the rock mass and is a crude measure of the average rock block size.
- **Jr/Ja** represents the roughness and alteration of the joint walls and their filling. The value of Jr/Ja should relate to the surface most likely to allow failure to initiate. Jr/Ja can be said to represent the inter-block shear strength.
- **Jw/SRF** is a complicated empirical factor describing the "active stresses".

If Q is going to be used for the analytical or numerical modelling then the influence of stress and water are taken into account within the model. The SRF and Jw parameters become redundant with reference to the classification and can be set to one. This reduces the rating to a modified Q, or Q'.

Q' = (RQD/Jn) * (**Jr/Ja**)

Barton *et al.* (1974) admit that if the Q system were to be used for other applications, such as slope stability, then it would need to be re-evaluated. As PPRust is an open pit operation the Q value is not suitable. The two terms in the Q' formula could be used on their own, however, for indicating in-situ rock block size and inter-block shear strength. In this way the Q system could be applied at PPRust and further work needs to be done on this.

4.2.2 Bieniawski's Geomechanics Classification

Bieniawski's Rock Mass Rating (RMR) system was proposed in 1973 but was refined in 1977. It was based upon case histories drawn largely from civil engineering and the mining industry tended to regard the classification as conservative therefore modifications have been proposed over time (Bieniawski, 1989). This scheme uses six parameters to classify a rock mass as shown in Table 4.3. Bieniawski recognised that not all parameters have equal importance and hence assigned a rating to each parameter by a weighted numerical value. The sum of the ratings can range from 8 to 100, which results in the final rock mass rating, RMR, which is a value out of 100.

Table 4.3 Bieniawski's RMR parameters and their ratings

Parameter	**Abbreviation**	**Rating**
Uniaxial compressive strength (UCS)	A1	0 to 15
Rock quality designation (RQD)	A2	0 to 20
Spacing of discontinuities	A3	0 to 20
Condition of discontinuities	A4	0 to 30
Groundwater condition	A5	0 to 15
Orientation of discontinuities	B	0 to -12

The fourth parameter, joint condition (A4), is sub-divided into 5 categories, which are: persistence (E1), separation (E2), roughness (E3), infilling thickness and hardness (E4) and joint wall alteration (E5). Parameters A1 to A5 are used to classify the rock mass independent of the proposed excavation. When the RMR is to be used to determine support requirements and assess general stability, the relative orientation of the dominant discontinuity set with respect to the proposed excavation must be taken into account. Parameter B provides for adjustments to the RMR for tunnelling, mining, foundations and slopes, according to Table 4.4.

Table 4.4 Bieniawski's joint orientation adjustment for RMR

Orientation of critical joint set with respect to tunnel or mine excavation	**Rating**
Very Favourable	0
Favourable	-2
Fair	-5
Unfavourable	-10
Very Unfavourable	-12

Each geotechnical unit has a rock mass rating (RMR) calculated using the formula:

RMR = A1 + A2 + A3 + A4 + A5 + B

Based on this relationship Bieniawski (1973) proposed 5 rock mass classes, from 'very good rock' to 'very poor rock' which he modified in 1977 to those shown in Table 4.5.

Table 4.5 Bieniawski's RMR classes

Rock mass class	**Description**	**RMR**
I	Very good rock	81 - 100
II	Good rock	61 - 80
III	Fair rock	41 - 60
IV	Poor rock	21 - 40
V	Very poor rock	0 - 21

Bieniawski's parameters are similar to Barton *et al.*'s parameters but have the following differences:

1. Intact rock strength is used as a separate parameter in RMR but is implicit in SRF in Q – 'the characteristics of a rock mass cannot be separated from the characteristics of the rock material' (Bieniawski, 1973)
2. No stress taken into account for RMR – 'it is considered when selecting support measures' (Bieniawski, 1976)
3. Joint orientation is used for RMR but not Q – 'Q would be less general and the simplicity would be lost' (Barton *et al.*, 1974).

In 1976, Bieniawski determined a correlation between the RMR and Q values based on analysis of 117 case histories. This correlation is:

RMR = 9 *l*n Q + 44

Numerous other correlation equations have been proposed by Rutledge and Preston (1978), Cameron-Clarke and Budavari (1981) and Abad *et al*. (1984). In 1996, Goel *et al*. proposed an alternative correlation between simplified Q and RMR, recognising the fact that the 2 systems collect slightly different data and thus should not be compared in their normal state. They stated that the above equations produce unsatisfactory correlation coefficients of 0.81 or lower. They propose two new indices, the rock mass number, N, which is Q without SRF, and the rock condition rating, RCR, which is RMR without the joint orientation and intact rock strength. These two indices were determined for 63 case histories to develop the following correlation equation which has a satisfactory correlation coefficient of 0.92.

RCR = 8 *l*n N + 30

Rawlings *et al.* (1995) reviewed the previous correlations and proposed four new equations. They looked at both Q and an 'unfactored' Q where SRF=1 and determined that the relationship between RMR and Q is bilinear. They propose that this produces a much better correlation and is the reason why the correlation coefficients of previous work are not as good as could be hoped.

Bieniawski's RMR has been successfully applied to rock slopes as well as tunnels, giving it an advantage over the Q system at PPRust.

4.2.3 Laubscher's Mining Rock Mass Rating

This scheme was introduced in 1976 by Dennis Laubscher as a modification of Bieniawski's RMR to cater for diverse mining situations. It recognises that the in-situ rock mass ratings (IRMR) have to be adjusted according to the mining environment, so that the final rating (MRMR) can be used for mine design. It also allows for two methods of calculating IRMR – using fracture frequency or RQD and joint spacing.. MRMR accounts for effects of blasting, weathering, stress changes and joint orientation. It was updated in 2000 (Laubscher and Jakubec, 2000) to make provision for rock block strength, cemented joints and using water as an adjustment. The term RMR was used until 2000 when it was changed to IRMR to avoid confusion with Bieniawski's RMR. The Anglo Platinum standard is based on the 1990 paper, however, as all the historical logging has been done with it and it is easier to use.

As with Bieniawski's RMR, the MRMR approach is that the rock mass is divided into geotechnical zones and each zone is assigned an in-situ rating based on measurable geological parameters. Each parameter is weighted according to its importance and assigned a maximum rating such that the sum of the maximum ratings is 100. Laubscher chose five parameters, as shown in Table 4.6, to rate a rock mass.

Table 4.6 Laubscher's IRMR parameters

Parameter	**Abbreviation**	**Rating Range**
Intact Rock Strength	IRS	0 – 20
Rock Quality Designation	RQD	0 – 15
Joint Spacing	JS	0 – 25
Fracture frequency	FF	0 – 40
Joint Condition and Water	JCW	0 – 40

The joint condition and water (JCW) parameter includes joint roughness, joint wall alteration and joint filling for various groundwater conditions. The in-situ rock mass rating (IRMR) can be calculated in two ways:

IRMR = IRS + RQD + JS + JCW
or
IRMR = IRS + FF + JCW

IRMR is used to classify the rock mass independent of the proposed excavation. When the IRMR is to be used to determine support requirements and assess general stability, the rock mass rating adjustments should be applied to determine the MRMR. These adjustments should be applied per geotechnical zone in the borehole or in the mine. There are four adjustments, namely for weathering, blasting, induced stresses and joint orientation. Not all four adjustments should be applied but rather the two dominant ones (Laubscher and Jakubec, 2000). For example in an open pit the induced stresses are low and the joint orientation and blasting are far more important. In underground operations the induced stress plays much larger role while the effect of weathering is limited. Thus in the Anglo Platinum standard, the open pit operations use only the blasting and joint orientation adjustments while the underground operations use the induced stress and joint orientation adjustments. The adjustments that are applied to this IRMR are shown in Table 4.7 below.

Table 4.7 Laubscher's MRMR adjustments

Adjustment	Abbreviation	Range
Weathering	Aw	30 – 100 %
Joint orientation	Ajo	63 – 100 %
Stresses	As	60 – 100 %
Blasting	Ab	80 – 100 %

The final mining rock mass rating is calculated by multiplying the IRMR by the adjustments.

MRMR = IRMR * Aw * Ajo * As * Ab

The IRMR classification is divided into five classes (Table 4.8), with A and B subdivisions, with ratings of 20 per class.

Table 4.8 Laubscher's rock mass classes based on IRMR

Rock mass class	Description	IRMR
1A , 1B	Very good rock	81 - 100
2A , 2B	Good rock	61 - 80
3A , 3B	Fair rock	41 - 60
4A , 4B	Poor rock	21 - 40
5A , 5B	Very poor rock	0 - 21

Laubscher differs from Bieniawski in three important ways:

1. He offers two alternative equations for determining an IRMR – using FF or RQD and JS combined.
2. Joint roughness is subdivided into large and small scale roughness, while joint separation and joint persistence are not included
3. Adjustments for blasting, stress and weathering as well as joint orientation are provided

Analysis of the PPRust data has shown that RQD is an inadequate method of classification as the rock is very competent and most RQD values fall above 95%. The core drilled is usually NQ (38 mm) or BQ (45 mm) which is deemed too small according to Deere's original 1963 definition of RQD. Therefore the value for IRMR using FF is used.

4.2.4. Hoek *et al.*'s Geological Strength Index (GSI)

The Geological Strength Index (GSI) is a quick and easy means of estimating the strength of a jointed rock mass. The GSI system has evolved over many years based on practical experience and field observations. In 1980, Hoek and Brown recognized that the characteristics which control rock mass deformability and strength were similar to the characteristics adopted in Q and RMR rock mass classification systems. A table was proposed and widely accepted by the geotechnical community. In a later update, Hoek and Brown (1988) suggested that the material parameters for a jointed rock mass could be estimated from Bieniawski's 1976 RMR, assuming dry conditions and a favourable joint orientation. As this does not work for very weak rock (RMR<25), a new index called GSI was introduced (Hoek *et al.*, 1995). The GSI system consolidates various versions of the Hoek–Brown criterion into a single simplified and generalized criterion that covers all of the rock types normally encountered in underground engineering (Cai *et al*., 2004).

The GSI is based on only two parameters that describe the rock mass – overall structure and overall joint surface conditions. They are displayed in a simple chart (Figure 4.2) which is used to determine the GSI, a rating from zero to 100. It should be used by experienced loggers and mappers, however, often a quick analysis is required and experienced staff are lacking. Here a technical assistant could estimate a GSI and the value could be used to give an idea of rock mass conditions where gaps in the data exist.

Hoek *et al.* (1995) derived two equations to relate GSI to Bieniawski's RMR and Barton's Q. For RMR, it is assumed that A5 = 15 and B = 0 which makes the minimum RMR = 23. Thus it only applies to RMR>23. In Q, it is assumed that SRF=1 and Jw=1 so the modified Q is used.

$$\mathbf{GSI = RMR - 5}$$

and

$$\mathbf{GSI = 9Log_eQ' + 44}$$

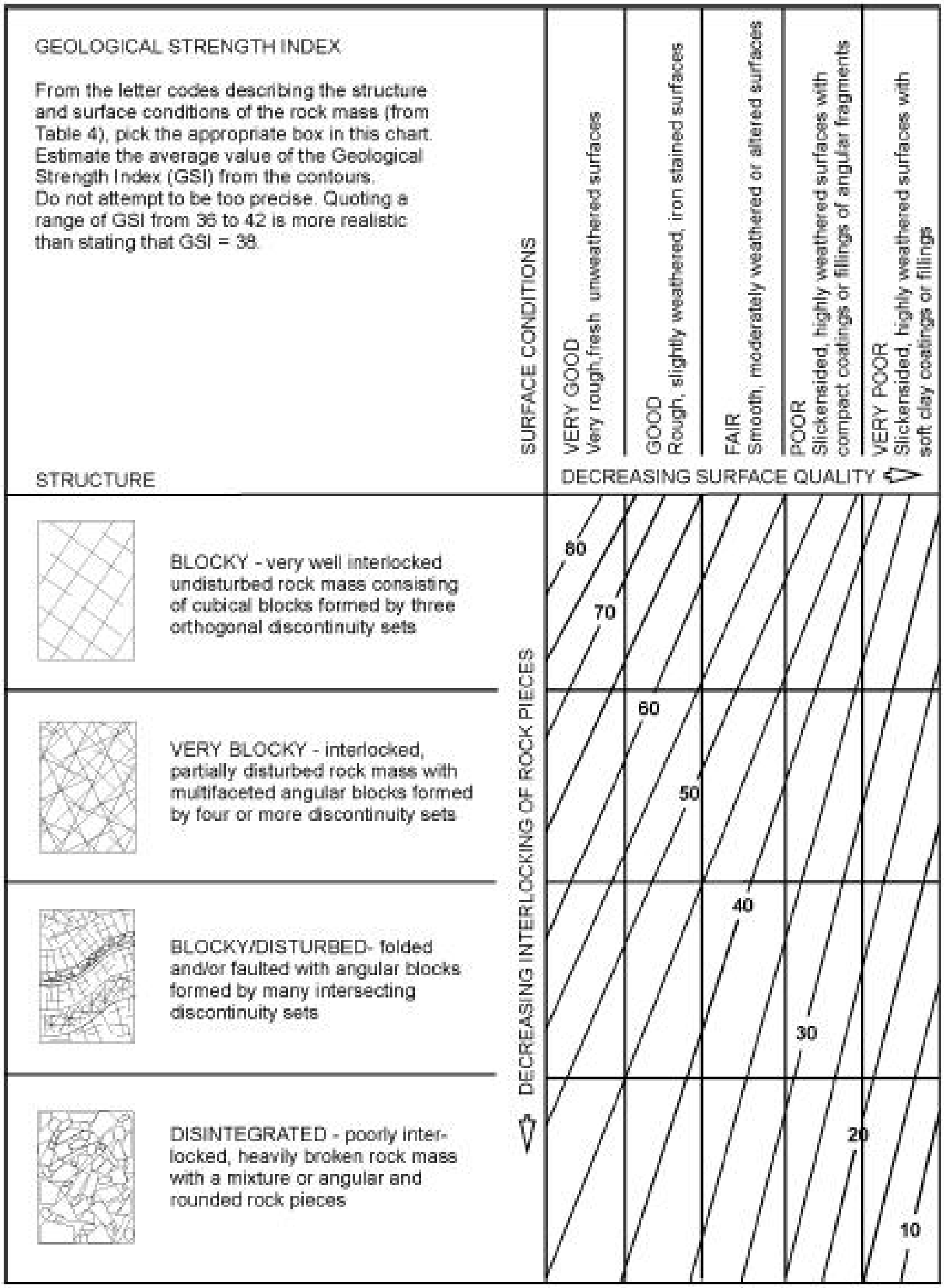


Figure 4.2 Hoek *et al.*'s GSI chart (1995)

4.3 Core Logging

At PPRust, all exploration and in-pit diamond drilling core is logged geotechnically in order to calculate rock mass ratings. Orientated drilling is done as required for the identification of faults, shears and joint sets. As mentioned in Chapter 2, a huge amount of exploration has been underway in the past five years at PPRust. Therefore from 1997 to 2005 over 320 km of core have been geotechnically logged at PPRust. The majority of this logging was done by contractors employed to log exploration core. Previously, logging was done on paper and then manually inputted into Excel where rock mass rating calculations were performed before the data was imported into Datamine. This was very time consuming, difficult to manage and prone to error. With the huge amount of data being collected a geotechnical logging database was required. The Anglo Platinum Geology departments were already using SABLE Data Works, so a geotechnical database in SABLE was developed at PPRust for Anglo Platinum. Validation checks, standardized lookups and site specific limits can be applied to every field in each table to ensure the data is accurate and usable.

Bieniawski's rating system requires the logger to divide the rock mass into geotechnical units based on a change in lithology and/or structure i.e. based on a change in rock mass properties. This methodology is used at PPRust for logging. Figure 4.3 illustrates this methodology with a simplified definition of these zones for a vertical exploration borehole that intersects the hangingwall, reef and footwall in an open pit. The contacts between the hangingwall norite, reef pyroxenite and footwall calc-silicate would all be separate geotechnical units. A shear zone on the hangingwall contact would also be a distinct unit. A lens of parapyroxenite within the Platreef would have to be separated out as a separate unit, splitting the Platreef into three units. A fault zone cross-cutting all the lithologies would further divide the borehole – in this case subdividing the norite into three units. The result is eight geotechnical units and each unit will have a different rock mass rating. When logging, more than eight units will be identified due to changes in mineralogy and structure but it is important to keep the scale of the operation and what the rock mass ratings are used for in mind.

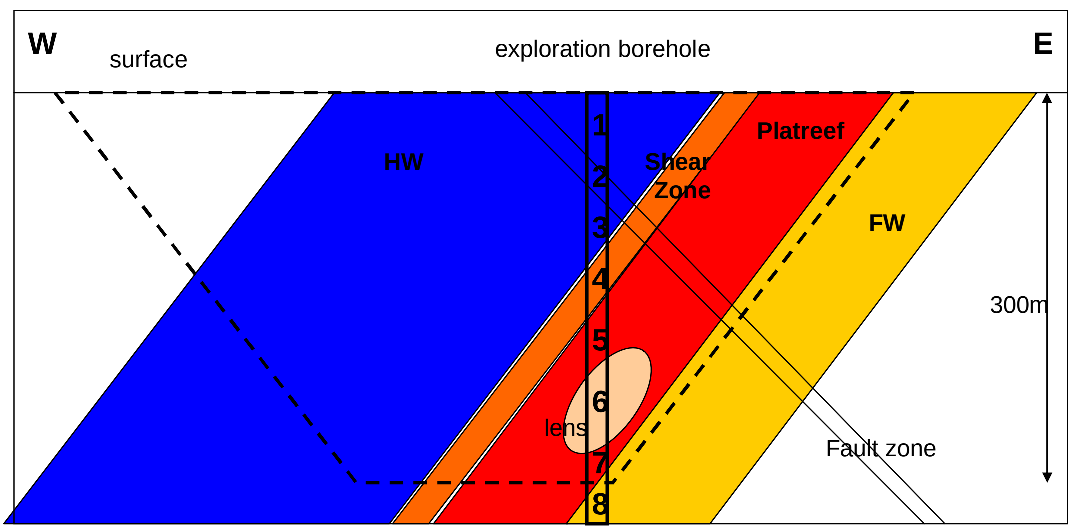


Figure 4.3 Simplified cross-section through Sandsloot open pit and an exploration borehole showing eight different geotechnical units and the scale of the operation

4.3.1 SABLE core logging database

A database can be defined in various ways:

- "An organised store of data for computer processing" *(Oxford English Dictionary)*

- “A collection of data arranged for ease and speed of search and retrieval.” (*American Heritage Dictionary)*
- “An organised body of related information” *(WordNet)*

With the large amount of logging data that is collected at PPRust, and within the Anglo Platinum group, it is obvious that a database is essential for a number of reasons:

- Large amounts of data can be stored on the network
- Data can be located quickly and easily
- Data can be manipulated and analysed efficiently
- Different data is stored in different tables but can be linked by key fields for data analysis purposes

SABLE Data Works is a Johannesburg based software company that supplies and supports standardized borehole logging database software. Anglo Platinum has worked together with SABLE for many years to develop a geology core logging and sample assay database. In 2003, the need for a similar geotechnical logging database was identified and in August a project to develop this began. The advantages of using a SABLE database are as follows:

- Standardised geotechnical logging for the whole Anglo Platinum group
- Data safely stored in a database on a sequel server on the network which is regularly backed up
- Drop down menus ensure easy and accurate data input
- Security and a range of user rights ensures data integrity
- Validation tools simplifies data validation and improves auditability
- Sequel tool allows the manipulation of large amounts of data
- Data export and import functions speeds up data transfer
- SABLE head office in Johannesburg provides support during working hours

All geotechnical logging data at PPRust is now collected in the coreyard on a laptop with a wireless connection so that the data goes directly into the SABLE Warehouse (SDWh) database on a dedicated server on the network. The PPRust database forms a part of the Anglo Platinum database which is located at Head Office in Johannesburg. This ensures all relevant parties have continual access to the latest data which is daily backed up by the IT department. Only authorised users can access the database and each user has defined rights for every single table in the database. For example, geologists have ‘read only’ rights to the geotechnical tables but ‘write’ access to the geological tables.

SDWh is structured in a table tree format as shown in Figure 4.4. Each business area has a number of farms and shafts or pits on which diamond drilling is done. Each farm has a number of boreholes each of which has one or more deflections (D0, D1, etc). For each deflection, a number of logs can be used for inputting data. For geotechnical purposes, there are seven possible geotechnical input logs (in black) and various geology and survey logs that may be used for analysis (in green). The data is stored and viewed in tables which can be formatted to be user friendly.

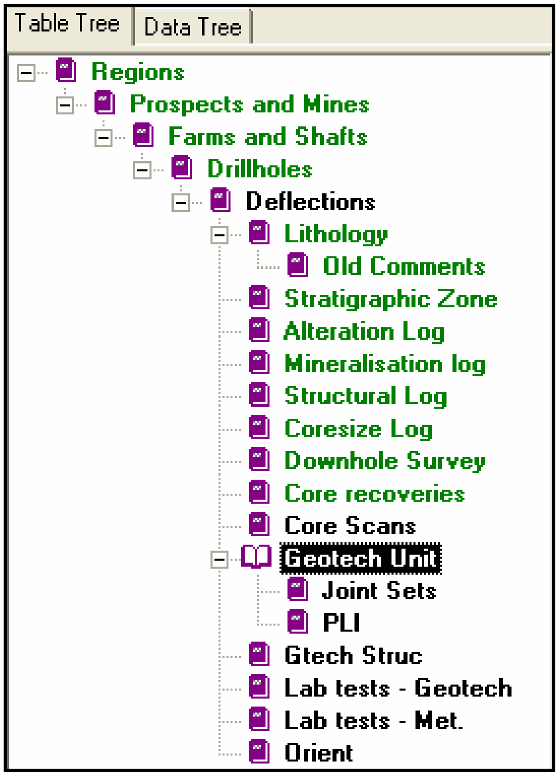
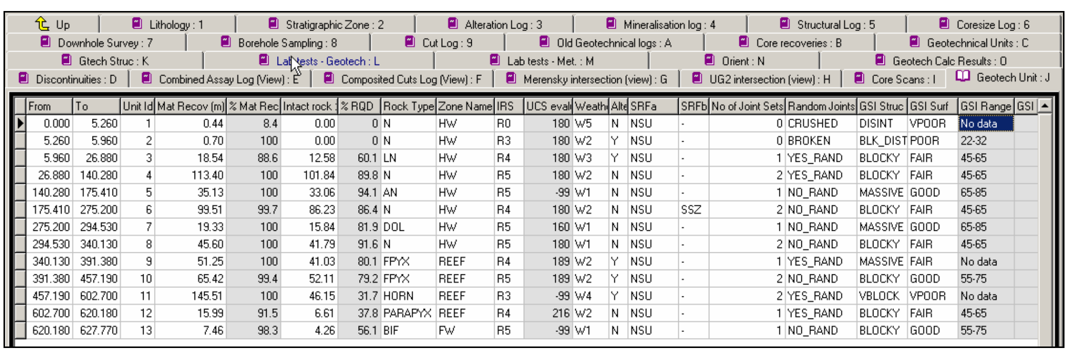


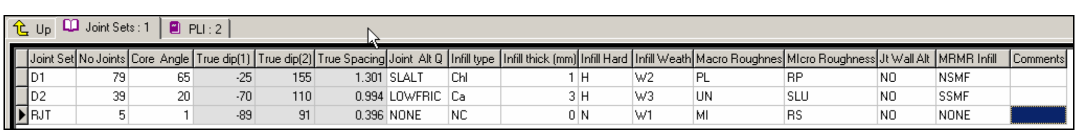
Figure 4.4 SABLE Data Warehouse table tree structure at PPRust

The 'Geotech Unit' table (Figure 4.5) contains all the overall geotechnical data for each geotechnical unit in the borehole while the 'Joint Sets' has all the specific information on each joint set in each geotechnical unit. The 'Geotech Structures' log stores all the information on single structures in the borehole. It is combined with the 'Joint Sets' table in Sable Data 1 (Figure 4.6).



From	To	Unit Id	Mat Recov (m)	% Mat Rec	Intact rock	% RQD	Rock Type	Zone Name	IRS	UCS eval	Weath	Alte	SRFa	SRFb	No of Joint Sets	Random Joints	GSI Struc	GSI Surf	GSI Range	GSI
0.000	5.260	1	0.44	8.4	0.00	0	N	HW	R0	180	W5	N	NSU	-	0	CRUSHED	DISINT	VPOOR	No data	
5.260	5.960	2	0.70	100	0.00	0	N	HW	R3	180	W2	Y	NSU	-	0	BROKEN	BLK_DIST	POOR	22-32	
5.960	26.880	3	18.54	88.6	12.58	60.1	LN	HW	R4	180	W3	Y	NSU	-	1	YES_RAND	BLOCKY	FAIR	45-65	
26.880	140.280	4	113.40	100	101.84	89.8	N	HW	R5	180	W2	N	NSU	-	2	YES_RAND	BLOCKY	FAIR	45-65	
140.280	175.410	5	35.13	100	33.06	94.1	AN	HW	R5	-99	W1	N	NSU	-	1	NO_RAND	MASSIVE	GOOD	65-85	
175.410	275.200	6	99.51	99.7	86.23	86.4	N	HW	R4	180	W2	N	NSU	SSZ	2	NO_RAND	BLOCKY	FAIR	45-65	
275.200	294.530	7	19.33	100	15.84	81.9	DOL	HW	R5	160	W1	N	NSU	-	1	NO_RAND	MASSIVE	GOOD	65-85	
294.530	340.130	8	45.60	100	41.79	91.6	N	HW	R5	180	W1	N	NSU	-	2	NO_RAND	BLOCKY	FAIR	45-65	
340.130	391.380	9	51.25	100	41.03	80.1	FPYX	REEF	R4	189	W2	Y	NSU	-	1	YES_RAND	MASSIVE	FAIR	No data	
391.380	457.190	10	65.42	99.4	52.11	79.2	FPYX	REEF	R5	189	W2	Y	NSU	-	2	NO_RAND	BLOCKY	GOOD	55-75	
457.190	602.700	11	145.51	100	46.15	31.7	HORN	REEF	R3	-99	W4	Y	NSU	-	2	YES_RAND	VBLOCK	VPOOR	No data	
602.700	620.180	12	15.99	91.5	6.61	37.8	PARAPYX	REEF	R4	216	W2	N	NSU	-	1	YES_RAND	BLOCKY	FAIR	45-65	
620.180	627.770	13	7.46	98.3	4.26	56.1	BIF	FW	R5	-99	W1	N	NSU	-	1	NO_RAND	BLOCKY	GOOD	55-75	

Figure 4.5 'Geotech Unit' log in SABLE Data Warehouse



Joint Set	No Joints	Core Angle	True dip(1)	True dip(2)	True Spacing	Joint Alt Q	Infill type	Infill thick (mm)	Infill Hard	Infill Weath	Macro Roughnes	MIcro Roughness	Jt Wall Alt	MRMR Infill	Comments
D1	79	65	-25	155	1.301	SLALT	Chl	1	H	W2	PL	RP	NO	NSMF	
D2	39	20	-70	110	0.994	LOWFRIC	Ca	3	H	W3	UN	SLU	NO	SSMF	
RJT	5	1	-89	91	0.396	NONE	NC	0	N	W1	MI	RS	NO	NONE	

Figure 4.6 'Joint Sets' log in SABLE Data Warehouse

Point load index test results are stored in the PLI log while all laboratory rock strength testing results are stored in the 'Lab – Geotech' and Lab – Met.' logs. The 'Orient' log is used when orientated drilling is done. The 'Core Scan' logs used are for storing scanned images of the core while a 'Core Photo' log for storing photographs taken of each core box is currently in development.

The geotechnical logs were initially designed in SABLE Data 1 (SD1) – the precursor to SABLE Data Warehouse (SDWh). The calculations required to produce the rock mass ratings have not yet been developed in SDWh. Therefore SDWh is used for data capture and storage but it is then viewed in the 'SD1 Wrap-around' where the calculations are performed. Figure 4.7 illustrates the different tables used in the two software packages and how they relate to one another. The data from SD1 is used for further analysis in Datamine, which will be discussed in Chapter 7.

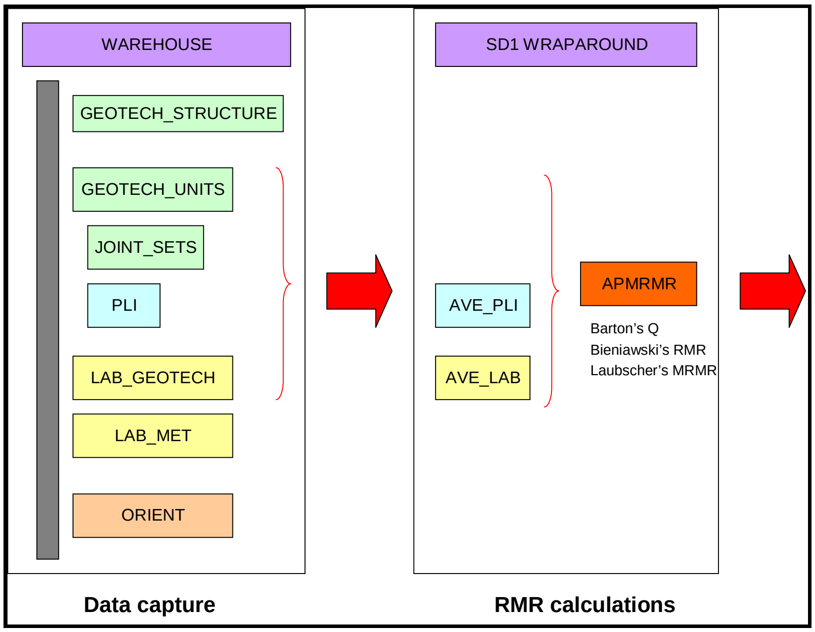


Figure 4.7 Illustration of the tables in SABLE Data Warehouse and SABLE Data 1 wraparound

The logs are first validated in SDWh which highlights standard errors, such as overlaps, and allows the user to rapidly rectify the problems. Rock mass rating calculations for all 4 systems are then done in SD1 at the touch of a button and the results are stored in a separate log called APMRMR. Average PLI and UCS values for each geotechnical unit are also calculated and stored in AVE_PLI and AVE_LAB logs as they are used in calculations for Bieniawski and Laubscher's rating systems. The project settings for the calculations can only be edited by the system administrator for PPRust. This person can choose whether to calculate Bieniawski's RMR, Barton's Q and/or Laubscher's RMR and whether the modified versions of each system are required. Correlations between the four different systems can also be calculated.

A graphical log (Figure 4.8) was designed in SABLE View which provides a quick and easy way of analysing and auditing the log. It shows all the raw data as well as the Barton's Q, Bieniawski's RMR and Laubscher's MRMR, using both the 'FF' and the 'JS and RQD' methods. This graphical log can be created for any number of boreholes in the database at the push of a button. The layout, colours and patterns are standardized for Anglo Platinum so that anyone can easily compare boreholes from different areas in the Group.

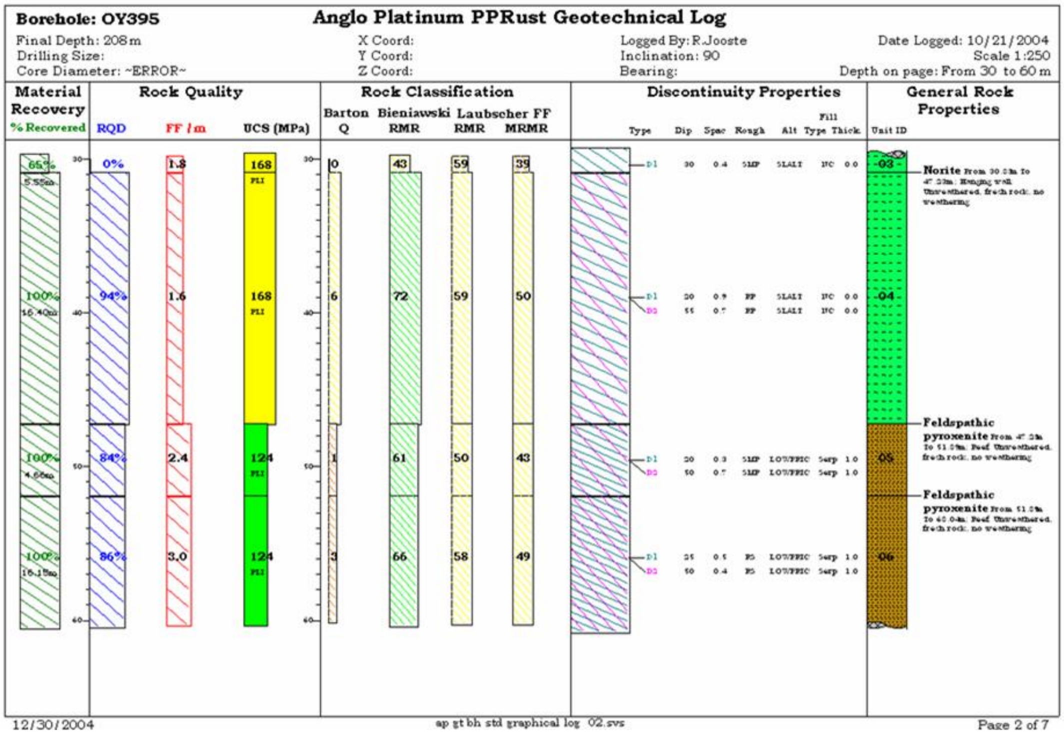


Figure 4.8 Geotechnical graphical log automatically generated by SABLE

In order to reduce the number of input fields in SABLE whilst still combining all 4 rating systems, a few adaptations had to be made when calculating the results. These are minor and do not reduce the accuracy of the results. They are:

- The Laubscher roughness profile was used instead of Bieniawski's and Barton's roughness definitions. This was chosen because Laubscher's profile is an adaptation of Bieniawski's and is more detailed, and thus can be simplified to obtain the other two definitions.
- In core logging, it is impossible to know the persistence and separation of a discontinuity. Thus constants are used in Bieniawski's E1 and E2 parameters. The average value is used in each case.
- Barton, Bieniawski and Laubscher all take water into account. This is impossible to measure in core, which lies in a coreyard for days before being logged, when it is drenched in water to aid the logging process. Thus a constant of dry conditions is used in each case. The ratings must be changed to reflect the true water situation when the results are used for design.

The PPRust borehole logging database currently has 1063 boreholes covering 337 km. These have been divided into geotechnical units, all of which have 4 classifications (Q, RMR, IRMR and MRMR). It has been validated and is exported to Datamine for 3D modelling. Table 4.9 summarises the logging data according to farm.

Table 4.9 Summary of all boreholes logged geotechnically for rock mass classification purposes

Farm	No. of boreholes	Total metres drilled	Years drilled	In-pit holes
Sandsloot	223	71 405.39	1996-2006	74
Zwartfontein	412	130 638.55	1999-2006	101
Overysel	264	79 513.8	1999-2006	0
Knapdaar	5	25 76.41	2004-2005	0
Vaalkop	60	20 681.03	1999-2005	0
Rietfontein	1	728.16	2005	0
Tweefontein	62	31 607.70	2004-2005	0
TOTAL	1063	337 171.04	1996-2006	175

All the geotechnical logging data collected at PPRust prior to October 2003 was collected according to Laubscher (1990). In order to transfer all the historical data into the new standard with Bieniawski and Q, some conversions had to be made. Currently only the Laubscher MRMR values are used for slope design but there is potential for further usage and development of the alternative rock mass classification systems at PPRust especially as the operations go underground. The SABLE database has been implemented at all other operations in Anglo Platinum creating the first geotechnical logging standard in the company.

4.3.2. Orientated drilling

Since the PPRust operations began in 1992, three orientated drilling programmes have been performed. In 2001, five orientated holes were drilled in Sandsloot open pit on the western highwall to determine whether the overall slope could be steepened for economic gains. Also in 2001, ten boreholes were drilled on the Zwartfontein and Overysel farms (five per farm) for a pre-feasibility study for future open pits. In 2003, four orientated boreholes were drilled in Sandsloot open pit on the western highwall to determine whether the shear zone daylighting in the penultimate cutback extended into the final cutback. Eight boreholes were also drilled in the year-old Zwartfontein South open pit for joint set identification and kinematic failure analysis. Every joint in every borehole was logged and rock mass ratings were also calculated. All the data has been imported into the SABLE Warehouse database where true dip and dip directions of each joint are calculated. Table 4.10 summarises the orientated drilling at PPRust and Figure 4.9 is a plan of the drillhole locations.

Table 4.10 Summary of all orientated logging at PPRust

Pit/Farm	Boreholes	Total length	Date drilled	Purpose
Sandsloot pit	ST1-5	830m	March 2001	Slope optimisation
Overysel	OYGT1-5	1301m	April 2001	Pre-feasibility
Zwartfontein	ZNGT1-5	1451.46m	May 2001	Pre-feasibility
Sandsloot pit	SSO1-4	715.75m	September 2003	Final slope design
Zwartfontein South pit	ZNG1-8	1409.36m	October 2003	Structural analysis

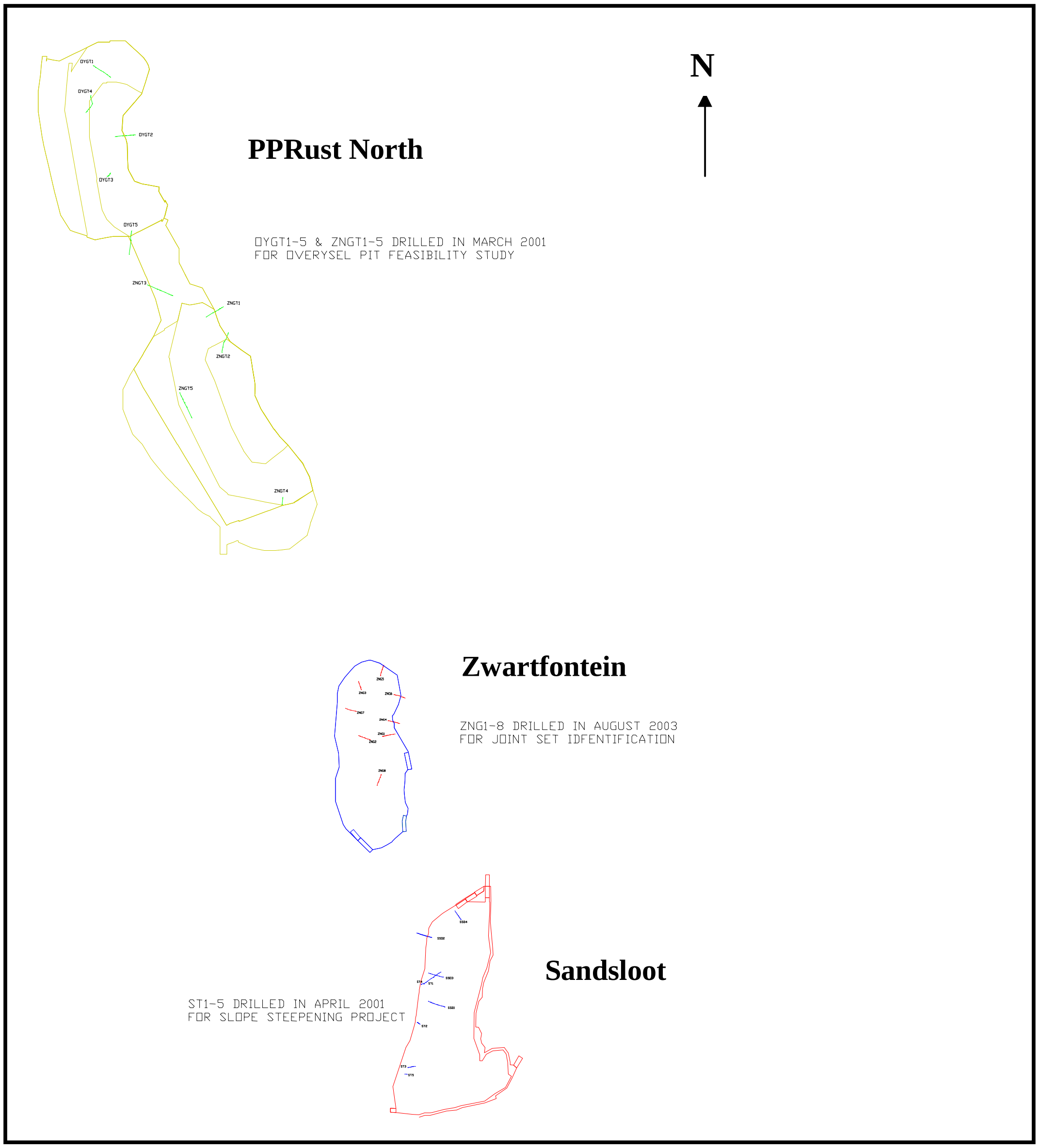


Figure 4.9 Location of all orientated boreholes drilled at PPRust

4.4 Face Mapping

The ideal situation in any open pit is that all faces are mapped geologically and geotechnically as soon as they are exposed. With over 400 km of new face exposure in 2005 at PPRust and production ramping up, this is impossible with the current staff quota. The mapping done at PPRust includes both rock mass ratings (Figure 4.10) and line surveys. Previously the data was collected on paper, inputted into MS Excel© where calculations were performed before it was converted to a comma separated values (csv) format and exported to Datamine. This was time consuming and prone to error. In 2003 MineMapper3D software was introduced at PPRust for geological mapping (Hansmann *et al*., 2005). As a database for geotechnical mapping data was also required, geotechnical mapping input tables were developed in MineMapper3D in 2004. They were designed to match the SABLE logs so that the same data is collected for both logging and mapping and can be used in conjunction for design work. Line surveys are the equivalent of orientated logging as they record every joint that cross-cuts a face at a certain elevation.

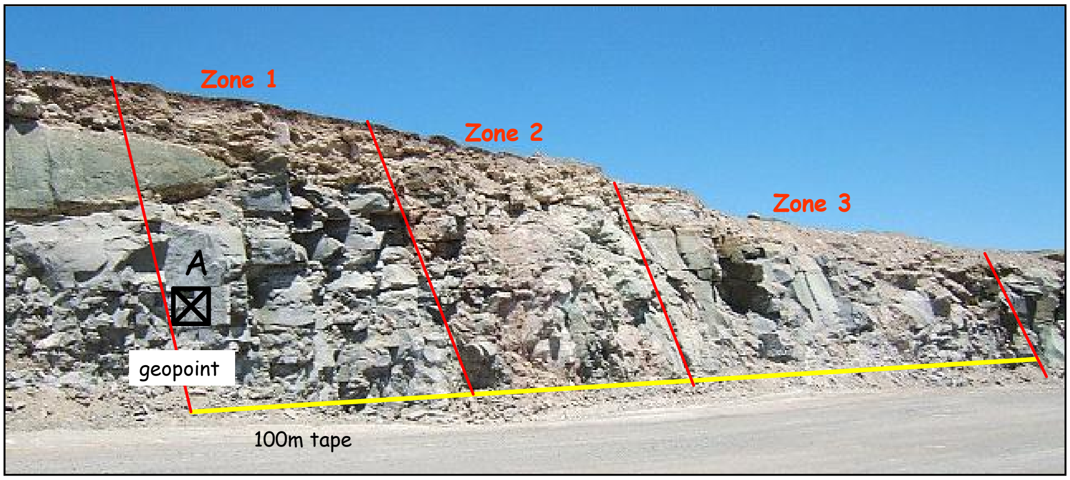


Figure 4.10 Illustration of geotechnical zones for a face map in Zwartfontein South open pit, used for rock mass ratings

4.4.1 MineMapper3D

MineMapper3D is a database program developed by the Canadian software company Century Systems to store and visualise 3D mapping data. In South Africa it is distributed and supported by DatamineSA in Johannesburg. MineMapper3D has a central database on a dedicated network server at PPRust as well as local databases on each user's computer. Datamine also can provide logging database software which can be linked to the MineMapper3D database. Figure 4.11 illustrates the flow of logging, mapping and testing data from the pit, coreyard and lab into a central database. This is the ideal case as it is easier to manage one database. Anglo Platinum has however chosen to use SABLE as many years of money and effort have been put into it. The central database stores all the mapping data while each local database stores only what is chosen by the user. The data can only be edited in the local database. Each bench has a map that contains all the 3D mapping data for that bench in a specific pit. Thus data can only be added, edited or viewed per bench. The data is stored in tables but is viewed in true 3-dimensional space (Figure 4.12). A complementary software program, Fusion, is used to access the maps in the central database. In Fusion all the available maps are visible and the user can select maps to check out or copy to the local database. Once finished with a map, it is checked back into the central database using Fusion. The database can be queried in another complementary software program, Query Manager. After mapping in the pit, the user checks out the relevant bench map into their local database, adds their data and then checks the updated map back into the central database. In this way no data can be overwritten or lost. If the bench map is already checked out, the user can see in Fusion who has the map and can go to them and ask them to check it back in if they are finished. Copies of the maps can also be made so individuals can manipulate the data as they choose without affecting the central database. The user can also choose to email all other relevant users when checking in or checking out a bench map. When adding data to a checked out map, the user imports any relevant survey data and then digitises each facemap's perimeter, contacts and structures and inputs the data into defined tables. Each object is stored with unique identities in separate tables so that it can be referenced.

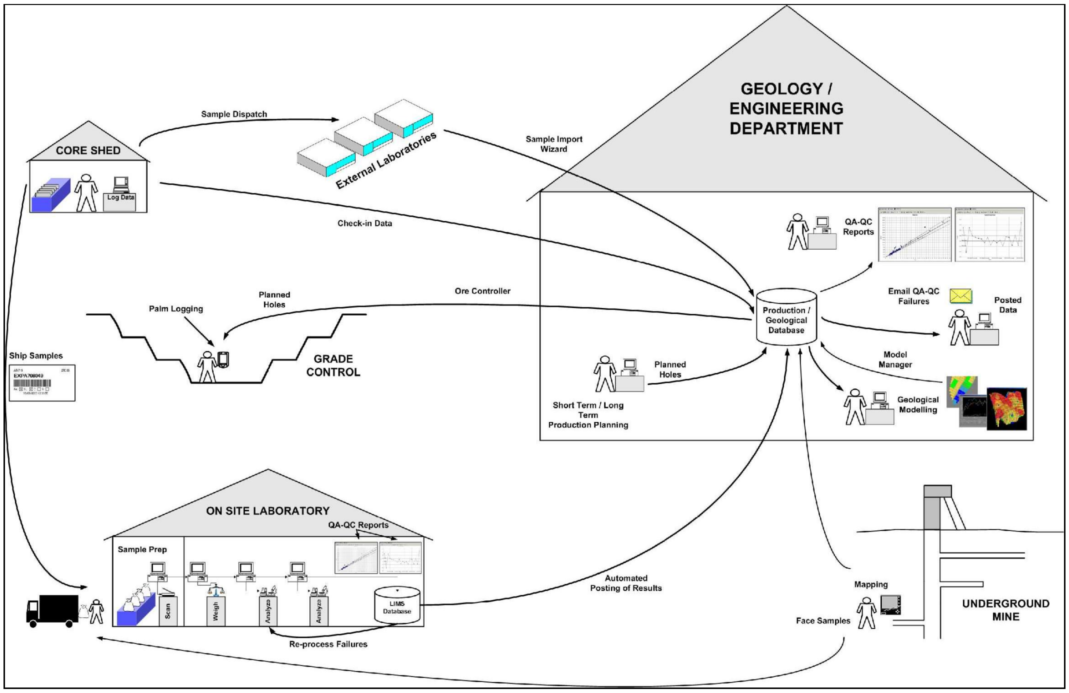


Figure 4.11 Flow of data from the pit, coreyard and lab into a central database

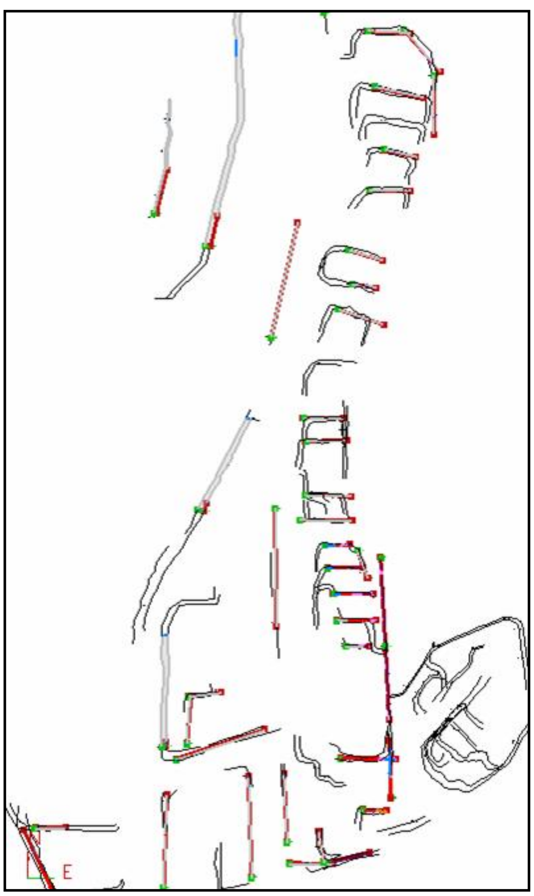


Figure 4.12 Bench map in MineMapper3D for Sandsloot open pit

The geotechnical setup in MineMapper3D developed at PPRust has 8 tables that mirror logs developed in SABLE Data 1 in 2003. This includes 5 input tables, namely two rock mass rating tables, a line survey table, a point load table and a lab testing table. The user digitises the geotechnical units defined at the face and then draws an interval line across the face (Figure 4.13) and inputs the relevant data into the table (Figure 4.14). Point load test and laboratory test data can be added in the same way. Three calculation tables for rock mass ratings, average point load results and average lab test results, are generated at the push of a button. Data interpretation, such as delineating hangingwall and footwall contacts (Figure 4.15), geotechnical zones and structures can be done very easily in MineMapper3D and these interpreted lines can be exported to Datamine for further analysis and modelling.

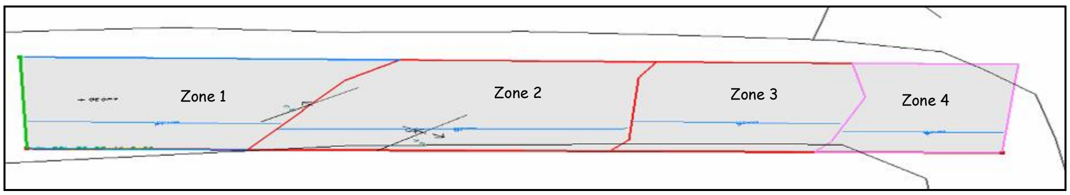


Figure 4.13 Example of a 3D facemap in MineMapper3D with digitised, coloured geological contacts, structures and four geotechnical zones

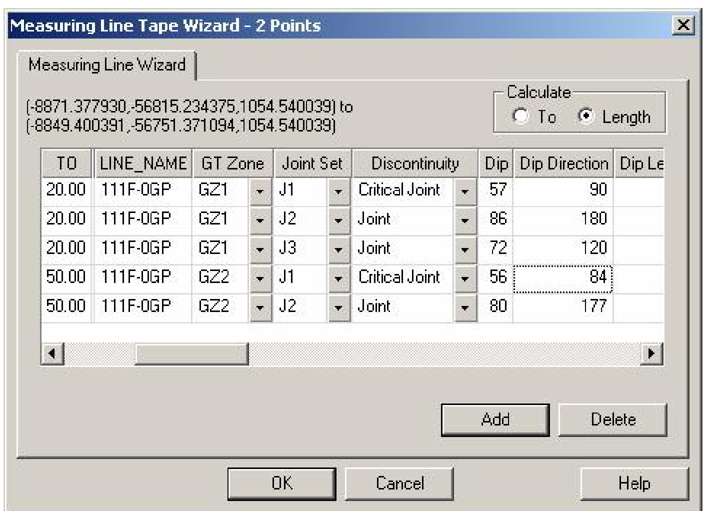


Figure 4.14 Example of the input table for geotechnical window mapping data used to calculate rock mass ratings

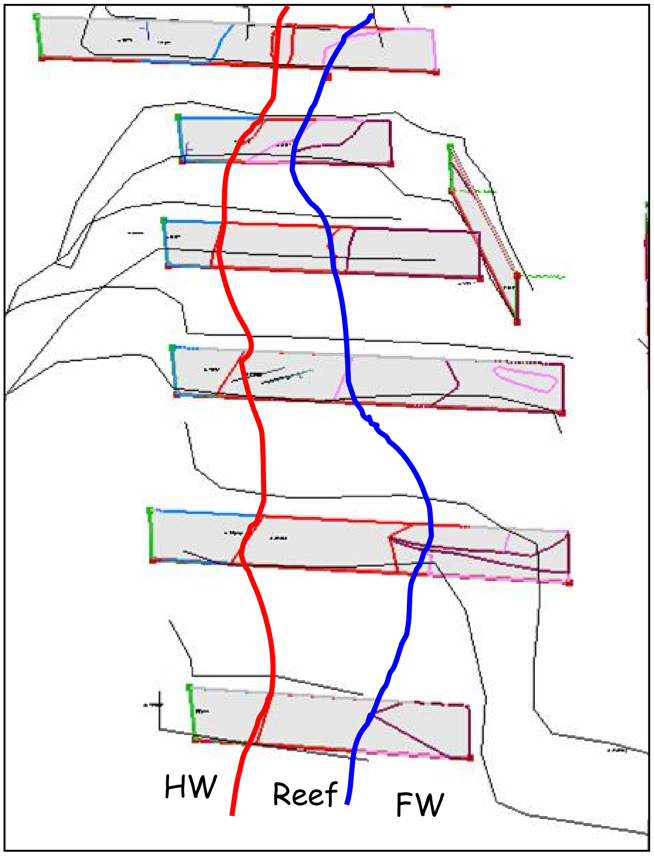


Figure 4.15 Example of data interpretation in MineMapper3D

As with the SABLE database, there are many advantages in using MineMapper3D:

- Standardised geotechnical logging for the whole Anglo Platinum group
- Data safely stored in a database on a sequel server on the network which is regularly backed up
- Drop down menus ensure easy and accurate data input
- Security and a range of user rights ensures data integrity
- Validation tools simplify data validation and improve auditability
- Data export and import functions enable and speed up data transfer
- Datamine office in Johannesburg provide support during working hours
- 3D visualisation and interpretation of mapping data.

4.4.2 SiroVision digital photogrammetry

SiroVision is a digital photogrammetry software program developed by CSIRO in Australia that enables safe and comprehensive mapping of pit highwalls (Poropat, 2001). Large areas can be quickly mapped and it provides an excellent record of the changes in pit faces over time. A normal high resolution digital camera is set up on a tripod and its position is surveyed. A photograph is taken of the face in question, which must include a surveyed reference point of some sort. The tripod is moved a certain distance (e.g. 50 m) to the left, its position is surveyed and a second photograph is taken of the same face (Figure 4.16). The distance depends on the distance from the camera to the face and the 2 photographs must overlap by 90%. This can be repeated for all the areas on the slope. The photographs are downloaded from the camera and are brought into the software along with the survey readings.

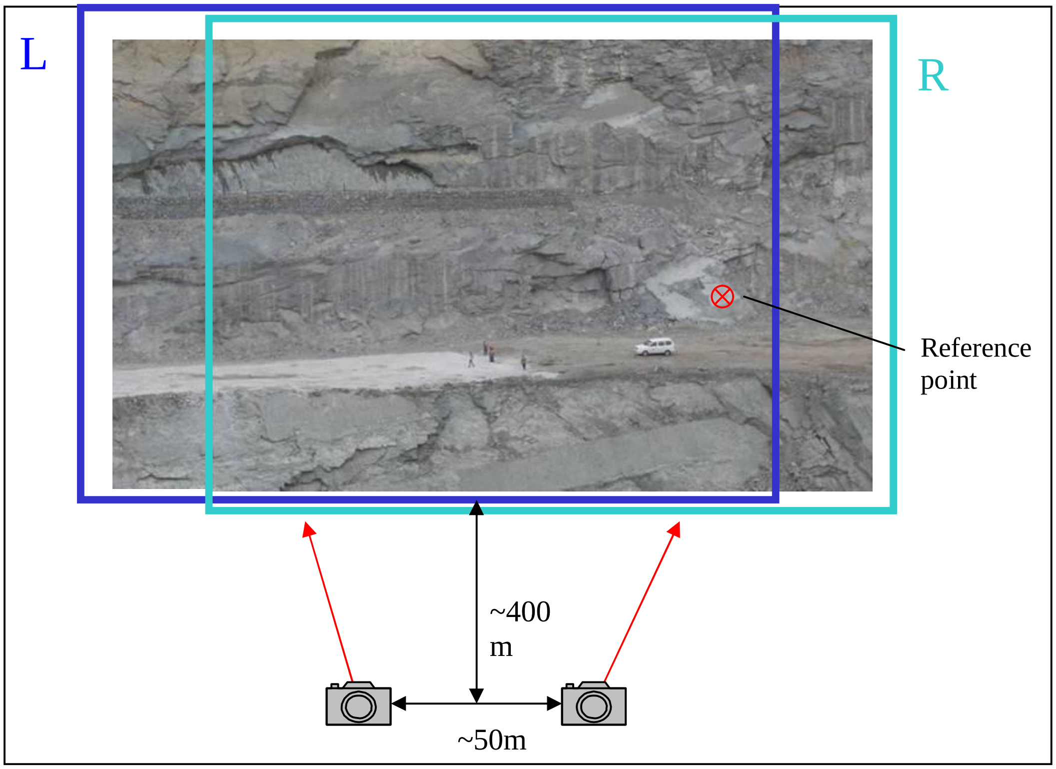


Figure 4.16 Method of obtaining photographs for SiroVision

SiroVision is made up of 2 parts, namely Siro3D and SiroJoint. Siro3D is used to import the images and surveyed coordinates, which are then combined and converted into a 3D image which is accurate to one degree orientation. The 3D image is a combination of a 3D point cloud and a 3D photograph. The images are saved and then opened in SiroJoint which is used to interpret the images. Joints, faults, dykes and geological contacts can be easily digitised in SiroJoint and orientations (dip and dip direction) can then be measured off the image (Figure 4.17). These readings will often be more accurate than ones taken in the field as they average hundreds points on the whole surface. When a geologist takes a reading he measures only a few points on a large surface. Also many of the flat lying joints in a mapping face cannot be reached by a geologist thus the mapping data is biased towards the vertical structures. With SiroVision however, all structures in a face can be measured thus the bias is removed.

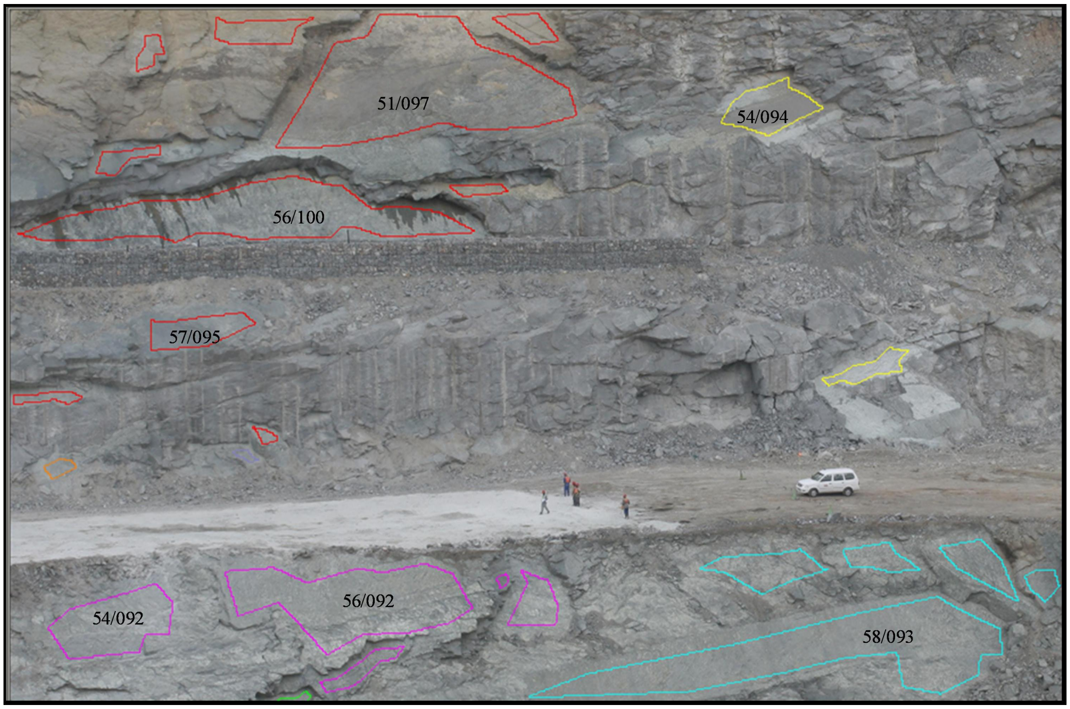


Figure 4.17 Dip and dip directions of a number of joints read off the SiroVision 3D image of a small section of the west wall in Sandsloot open pit

The orientation readings can be plotted on stereonets in SiroJoint and grouped into joint sets. At PPRust, this is used for joint set identification, kinematic failure analysis and geotechnical zoning. The structures can be extrapolated and overlaid over pit designs in SiroJoint or exported into design and draughting packages. It greatly improves the rock engineer's ability to identify potential failure planes in 3D space and to predict where they will cause failure on the benches below. This is particularly effective on the west wall at Sandsloot open pit where there is no access to faces and mapping the accessible faces is dangerous. Also the size of the exposed failure planes ensures reliable readings and allows for accurate extrapolation. Although SiroVision does not have a database structure, it allows vast numbers of joints and faults to be measured. This greatly improves joint set identification and kinematic failure analysis. By careful storage of the data and a logical, systematic approach used in the data collection process, SiroVision can be optimally utilised.

The aim at PPRust is to regularly map all the highwalls with SiroVision and for each failure plane and potential failure plane to be exported to Datamine. The planes would then be extrapolated laterally and down slope to provide an idea of where failures may occur on lower benches (Figure 4.18). This enables better identification of where slope monitoring must be focussed. It also provides a qualitative method of monitoring failures. Comparisons of photographs could highlight small failures that were not noticed with other monitoring techniques. SiroVision analysis also aids the planning department in designing around the failures and incorporating the cost of cleanup and secondary blasting into their economic analysis. It is therefore important for the Rock Engineering department to work closely with the Survey, Geology and Planning departments to get the maximum benefit out of the system. SiroVision is supported by DatamineSA in Johannesburg and work is underway to integrate MineMapper3D and

SiroVision. This will allow the SiroVision data to be stored in a database ensuring that the data is not lost.

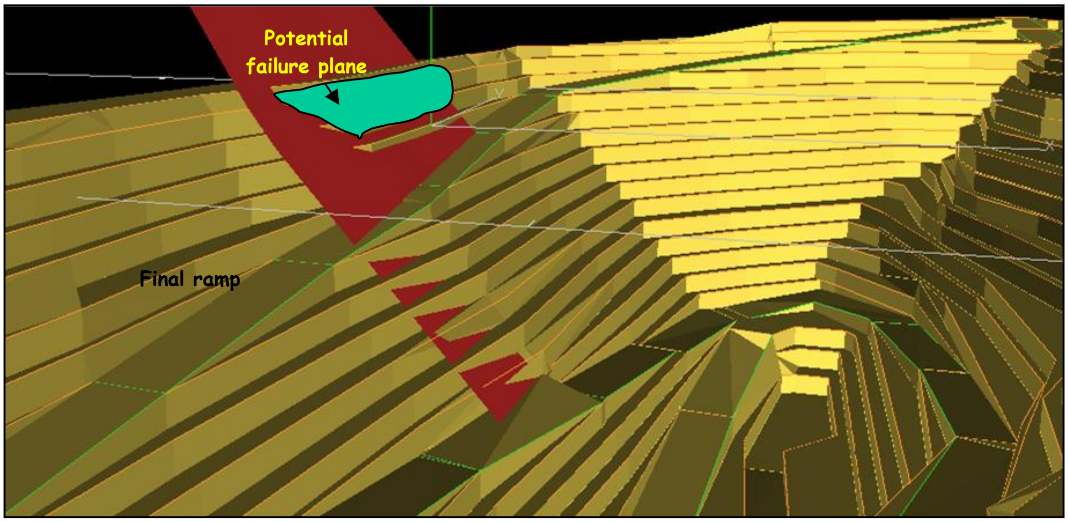


Figure 4.18 Failure plane measured in SiroJoint imported in Datamine and extrapolated on lower benches and ramps for stability analysis

4.5 Rock Testing

Since 1996 more than 15 600 lab and field tests have been commissioned by the PPRust geotechnical department. This included both core samples and block samples which are later cored in the lab. The data has been held in various MS Excel spreadsheets and MS Word and .pdf documents. The data was used for the purpose at the time and then left untouched for many years. This rock testing data has now been included in the SABLE and MineMapper3D databases to preserve it and to make use of it.

Intact rock strength plays a large role in rock mass ratings thus a good rock testing database will improve the accuracy of the rock mass ratings. Over the past 6 years 329 uniaxial compressive strength (UCS) tests have been done on PPRust's major rock types. As of January 2005, point load testing is done on all rock types in every logged borehole at PPRust which has resulted in a rapidly growing database of over 9000 point load tests. The point load values of the major rock types have been compared to the corresponding lab UCS test results and site specific point load factors have been developed. In the SABLE and MineMapper3D rock mass rating calculations, the best available rock testing data is used. If UCS tests were done on the borehole then those results are used. If not then it is likely that point load tests were done and those results are used. If these are also not available then an average UCS for the rock type taken from historical lab testing data is used. For the few rock types that have not been tested, an estimated UCS is used based on Bieniawski (1973). Other strength tests include 125 elastic properties tests, 34 Brazilian tensile tests, 16 triaxial tests and 47 shear tests and the results have been used for slope stability analysis. Drillability tests were done in order to assist the drilling department in their drill bit selection and budgeting. Metallurgical testing, namely 78 Bond work index (BWI) tests and 82 drop weight tests (DWT), was also done for plant design. This was particularly important in 2005/6 as the design for the new concentrator at PPRust North project was underway. All these tests are now in the SABLE database. Table 4.11 summarises the full rock testing database at PPRust.

Table 4.11 Summary of rock testing at PPRust

Test type	Sandsloot	Zwartfontein	Overysel	Tweefontein	Total
UCS	97	105	137	0	329
Point load	2422	5772	1898	2360	14,499
Indirect Tensile	34	0	0	0	34
Triaxial	16	0	0	0	16
Elastic properties	63	35	32	0	125
Shear tests	47	0	0	0	47
BWI	36	23	19	0	78
DWT	11	21	16	0	48

4.5.1 Laboratory strength tests

Various laboratory test programmes have been carried out during the life of the mine by CSIR, UKZN, ARC, RockLab and GroundWorks. Most notable are the programs of March 2003 and November 2002. These test programs comprised UCS tests, triaxial tests, Brazilian tensile tests and shear box tests on natural joints and artificial saw cut surfaces. The laboratory tests on samples from the Sandsloot pit were performed on norite, calc-silicate, feldspathic pyroxenite, pegmatoidal feldspathic pyroxenite, parapyroxenite and serpentinised pyroxenite. Uniaxial compressive strength testing is the most common method for lab testing of rock strength (ISRM, 1979). The UCS and elastic properties test results for PPRust rocks are summarised in Table 4.12.

Table 4.12 UCS and elastic properties test results for dominant rock types at PPRust

Rock type	Farm	UCS (MPa)			Poisson's ratio	Young's modulus (GPa)
		Tests	Ave	Std Dev		
Norite	Overysel	17	191	50.1	0.2	85.4
	Sandsloot	6	176	11	0.3	82.6
	Zwartfontein	20	183	25.3	0.3	75.8
Feldspathic pyroxenite	Overysel	31	177	44.8	0.3	128.1
	Sandsloot	12	209	84.4	0.3	124.8
	Zwartfontein	24	188	62.1	0.3	88.4
Pyroxenite	Overysel	4	78	35.6	0.4	79
	Sandsloot	6	159	15.6	0.2	72.5
Parapyroxenite	Overysel	6	177	91.3	0.2	56.6
	Sandsloot	62	185	55.1	0.2	119.2
	Zwartfontein	13	223	36.2	0.3	89.4
Serpentinite	Overysel	13	121	40.8	-	-
	Sandsloot	2	192	9.2	0.3	48.3
	Zwartfontein	4	139	23.3	-	-
Granofels	Overysel	31	215	68.5	0.4	101.5
	Zwartfontein	9	203	46	-	-
Calc-silicate	Overysel	2	161	58	-	-
	Sandsloot	13	157	66.3	0.3	51.3
	Zwartfontein	6	129	50.5	-	-

4.5.2 Point load tests

Point load testing at PPRust is done according to the ISRM standard (1985). Diametral testing is done on ten samples per rock type per borehole. This data is inputted into SABLE where the Is50 value is calculated. Due to the large number of tests done in the last two years, site-specific conversion factors have been calculated for all the major rock types, which have lab UCS test results. Most of these factors fall within range of 20 to 25 recommended by ISRM (1985). Table 4.13 lists the conversion factors used at PPRust.

Table 4.13 PLI to UCS conversion factors used at PPRust

Rock type	**Farm**	**PLI – UCS (MPa)**			**UCS Conversion factor**
		Tests	**Ave**	**Std Dev**	
Norite	Overysel	345	178	62.8	23
	Sandsloot	128	199	72.3	
	Tweefontein	50	205	61.8	
	Zwartfontein	97	205	47.4	
Feldspathic pyroxenite	Overysel	930	178	49.6	22
	Sandsloot	34	197	81	
	Tweefontein	50	188	59.4	
	Zwartfontein	521	198	48.3	
Pyroxenite	Overysel	103	127	63	16
	Sandsloot	47	132	65.3	
	Tweefontein	20	139	47.7	
	Zwartfontein	33	111	36.8	
Parapyroxenite	Overysel	57	172	86.1	28
	Sandsloot	93	184	105	
	Tweefontein	10	344	194.5	
	Zwartfontein	142	211	60.2	
Serpentinite	Overysel	150	129	45.2	28
	Sandsloot	14	109	32	
	Zwartfontein	72	146	58.3	
Granofels	Overysel	470	212	68.1	26
	Sandsloot	20	230	80.1	
	Zwartfontein	273	208	49	
Calc-silicate	Overysel	22	121	59.9	25
	Sandsloot	64	116	63.8	
	Tweefontein	10	224	147.7	
	Zwartfontein	68	148	59.5	

4.5.3 Bond Work Index tests

The Bond Ball Mill Work Index (BMWI) test is an industry standard for determining the Bond Work Index of an ore sample under ball milling conditions (JKTech, 2006). The BMWI is a measure of the resistance of the material to crushing and grinding. The Bond Work Index is used to determine the energy requirements of a ball milling process for: design of new equipment and grinding circuits, optimisation of existing ball mill circuits to maximise throughput and/or minimise power usage and characterisation of an ore body for feasibility studies. It is a 'locked cycle' test conducted in closed circuit with a laboratory screen. The closing screen size is selected so that the product P80 (percent passing = 80%) from the test is as close as possible to the product P80 expected from the circuit under design. Bond Work Index testing has been done at PPRust for the current plant at Sandsloot and the future plant at PPRust North. The results are used for design purposes but could also be used on a blast by blast basis. Each major rock type has been tested (Table 4.14) and this could be used to predict the mill throughput per blast as the geologists record the rock type of every blast. This would aid the costing and planning in the plants.

Table 4.14 Test results for Bond Work Index for dominant rock types at PPRust

Rock type	**Farm**	**BWI**		
		Tests	**Ave**	**Std Dev**
Feldspathic pyroxenite	Overysel	9	23.4	2.23
	Sandsloot	2	24.8	.57
	Zwartfontein	9	24.9	1.67
Pyroxenite	Overysel	3	23	.53
	Sandsloot	5	22	2.52
	Zwartfontein	1	22.1	-
Parapyroxenite	Overysel	5	22.1	1.73
	Sandsloot	10	23.9	1.85
	Zwartfontein	1	24.1	-
Serpentinite	Sandsloot	1	29.5	-
	Zwartfontein	3	29.2	1.72
Granofels	Overysel	2	18.8	1.2
	Zwartfontein	1	23.8	-

4.5.4 Drop weight test

In an autogenous mill, there are two main mechanisms of breakage: impact (high energy) and abrasion (low energy). The drop weight test measures the impact parameters, A and b, by simply dropping a weight from a predetermined height onto a particle so that it breaks (JKTech, 2006). At least 60 kg of -100 mm+12 mm rock chips are required for the test, with 100 kg being the ideal sample mass. The rock chips are divided into 5 size fractions and for each fraction, between 10 and 30 particles are broken at each of 3 energy levels, giving 15 size/energy combinations. The breakage products are collected and sized and this distribution is normalised with respect to the original particle size to determine the amount of breakage for each energy/size combination. The convention is to use the % broken product passing one tenth of the original particle size. This value, $t_{10,}$ along with the specific energy of comminution, E_{cs} measured in the test, are substituted into the formula below to determine the parameters 'A' and 'b' for each test. A best fit is determined and 'A' and 'b' are multiplied to determine the impact breakage resistance, 'A*b', which is reported.

$$t_{10} = A*(1- E^{-b*Ecs})$$

Abrasion breakage testing is performed with a tumbling test of selected single size fractions. The standard test tumbles 3kg of -55+38 mm particles for 10 minutes at 70% critical speed in a 305 mm by 305 mm laboratory mill. The product is sized and the t_{10} value determined. The abrasion parameter, t_a, is then defined as:

$$t_a = t_{10}/10$$

This is more applicable to semi-autogenous grind (SAG) mills where most of the breakage occurs at low energy levels. For both A*b and t_a, the lower the value, the greater the resistance of the rock to that type of breakage. All the major rock types at PPRust have been tested by this method and the results are summarised in Table 4.15. The results are used for design purposes but could also be used on a blast by blast basis. Each major rock type has been tested and this could be used to predict the mill throughput per blast as the geologists record the rock type of every blast. This would aid the costing and planning in the plants.

Table 4.15 Results of drop weight testing for dominant rock types at PPRust

Rock type	**Farm**	**DWT**		
		Tests	**Ave**	**Std Dev**
Feldspathic pyroxenite	Overysel	11	28.74	3.3
	Sandsloot	6	32.47	7.41
	Zwartfontein	14	31.34	3.98
Pyroxenite	Overysel	2	44.5	0.57
	Sandsloot	4	40.5	9.94
	Zwartfontein	4	34.9	12.41
Parapyroxenite	Overysel	4	29.2	5.85
	Sandsloot	7	28.8	3.69
	Zwartfontein	5	27.1	1.72
Serpentinite	Overysel	3	31.7	3.39
	Sandsloot	2	29.7	.49
	Zwartfontein	2	33.2	6.41
Granofels	Overysel	2	33.7	0.21
	Zwartfontein	2	29.8	4.77
Calc-silicate	Sandsloot	1	28.6	-
	Zwartfontein	1	46.7	-

4.6 Field Data Viewer

Often one needs a quick summary of the data stored in all the databases to check when the latest data was added, who added it and whether it has been used yet. To facilitate this, the Field Data Viewer (FDV) was developed by the author. FDV is a set of MS Access forms that query the SABLE and MineMapper3D databases and show the important information on rock testing, logging and mapping. Each section has a number of more detailed forms, including forms for geologists, orientated holes, relogged holes and in-pit drilling. It also has links to the relevant AutoCAD, Datamine and DIPS files. It is particularly useful for managers who do not have time to look at data in detail as well as new staff who are not familiar with the data. Figures 4.19 and 4.20 are examples of two of these forms that query the databases.

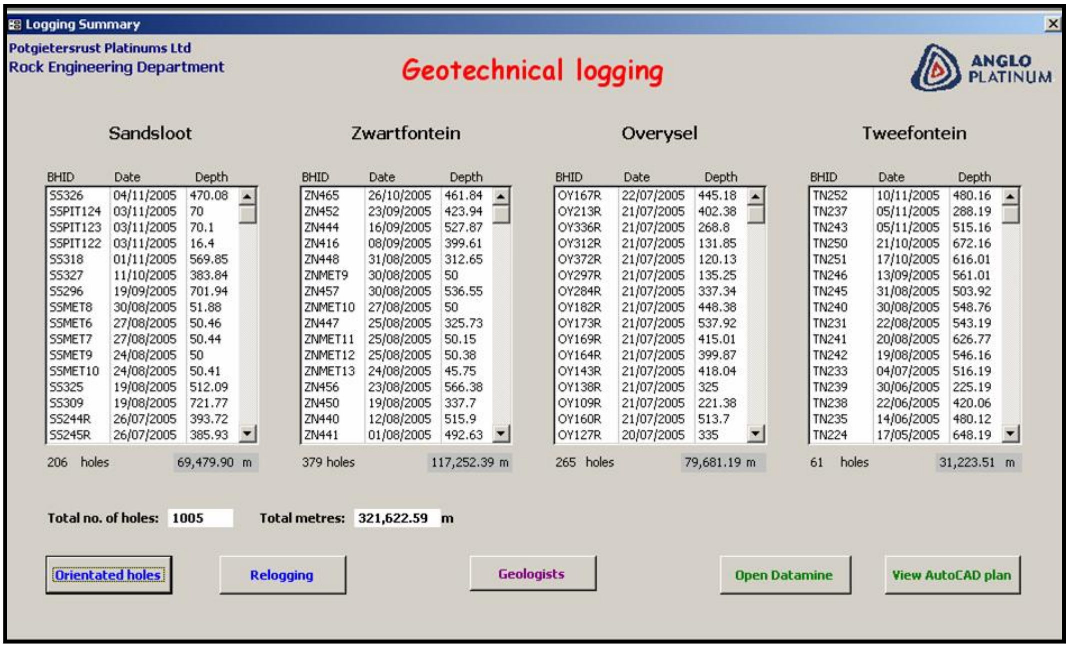


Figure 4.19 FDV form for geotechnical logging at PPRust

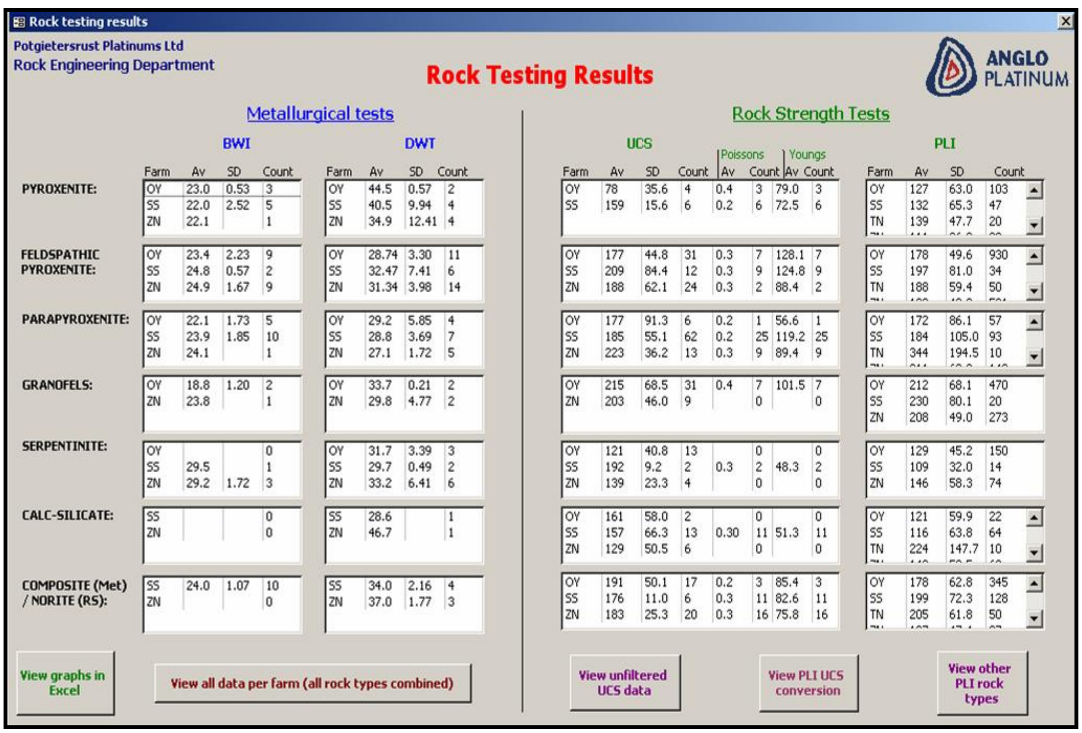


Figure 4.20 FDV form for rock testing results at PPRust

4.7 Data Integration and Visualization

In order to integrate and interpret the testing, mapping and logging geotechnical data, it is imported into Datamine, PPRust's modelling software program. The implementation of SABLE and MineMapper3D databases has made this process easier and quicker as well as more auditable. In Datamine the boreholes can be viewed in 3D, which is not possible in SABLE, and this improves the ability of the geotechnical engineer to analyse the logging data as well as apply adjustments for Bieniawski and Laubscher's systems more accurately. The unadjusted ratings (RMR and IRMR) for boreholes are imported into Datamine and the final ratings are calculated based on the boreholes' location in the open pit. Each open pit has a Datamine project where all the files are stored – pit designs, geological structures, boreholes etc. In order to create 3D boreholes, three files are needed.

- A collar file with the borehole ID and collars (X, Y, Z)
- A survey file with the borehole ID, depth, bearing and dip
- A data file with the borehole ID, from, to and any other data fields. At PPRust they are rock type, UCS, RQD, FF, IRMR(FF), IRMR(JS&RQD), RMR, Q, PLI

Three empty files with this format are created in each Datamine project and links are setup to import the data from FDV, which queries, filters and sorts the data from SABLE and MineMapper3D. In this way the three files are populated for each open pit project. They are then combined using Datamine's HOLES3D process which produces 3D boreholes. The facemaps are treated as horizontal boreholes for viewing and modelling purposes. The boreholes can then be viewed and colour-coded for the analytical work. Various parameters for slope and blast design can also be calculated using Datamine's EXTRA command.

A Datamine script (Figure 4.21) was written by the author to simplify the importing, viewing and calculation processes as well as to standardise them so any geotechnical user can manipulate the data. The script also allows the user to view the relevant pit designs and blast patterns so that the field data can be compared to the actual designs and conditions in the open pit.

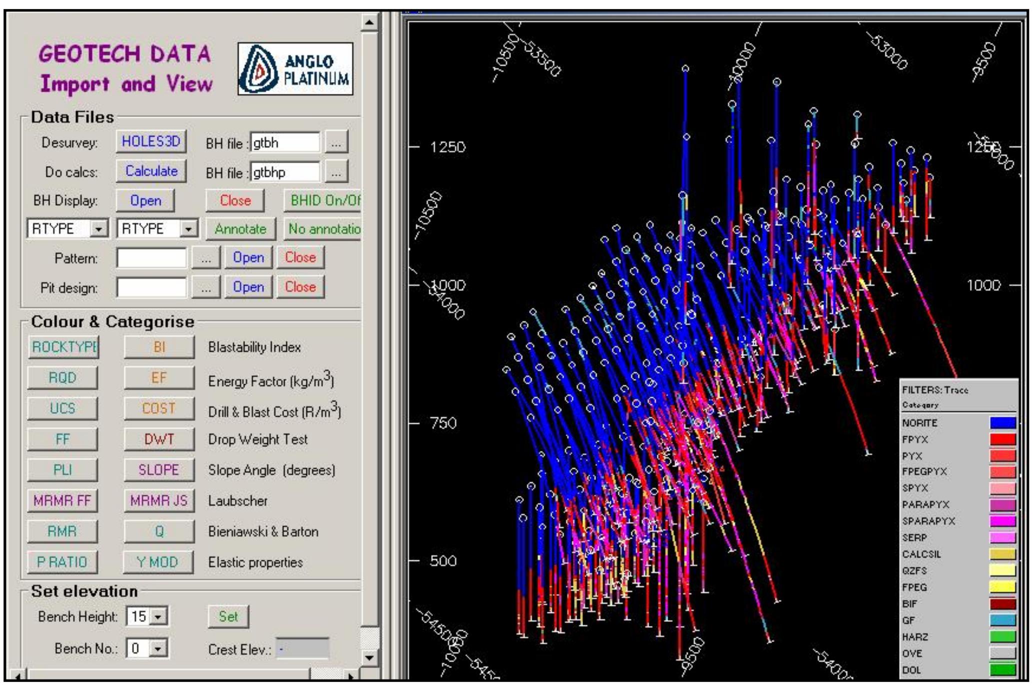


Figure 4.21 Datamine script developed at PPRust and visual display of boreholes for the proposed PPRust North open pit.

The next step is obviously to make use of all this data and this is done firstly by modelling it in Datamine. This will be discussed in Chapter 7.

4.8 Geotechnical Information Location System

An open pit mine is essentially a long-term project where geotechnical data must be collected an analysed on an ongoing basis. Over the years people come and go and much time can be wasted by newer staff members in merely searching for old data and analyses. If they do not find the relevant data then they are forced to collect more and the original money is wasted. If they do not find the analytical work then much time is spent re-analysing the same data, usually in the same way. To avoid this problem GILS was developed. GILS is a MS Access application designed by Peter Nathan of CGSS Consulting, in conjunction with the author, with the purpose of navigating through the entire geotechnical folder on the PPRust computer network. Each document on the network that relates to the Rock Engineering department is added to GILS. It is assigned a date, a name, one or two categories, a project (if relevant), a cutback design number (if relevant) and any number of keywords. A user can search for a document based on any of these identifiers, which is very helpful when looking for old documents.

The main form (Figure 4.22) shows the latest documents that have been added to GILS and allows the user to select documents for one or all the pits. The user can also add a new document, edit the system setups or do an audit on files in GILS.

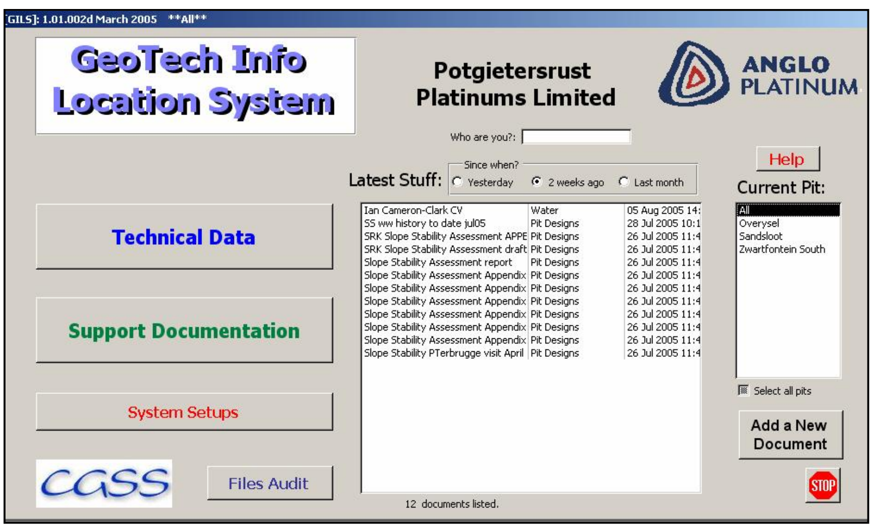


Figure 4.22 GILS main form

The documents are divided into two sections, 'Technical Data' and 'Support Documentation', each of which is accessed via the main form. Technical Data is sub-divided into 8 categories: Field data, Monitoring, Projects, Inspections, Modelling, Pit Design, Slope Analyses and Slope Support. There are further sub-divisions in each of these categories. Support Documentation is sub-divided into seven categories: Risk Management, Safety, Documents, Special Areas and DME (Department of Minerals and Energy). The documents in the chosen category are listed and can be filtered on name, date and category (Figure 4.23). On the right hand side of the form, the generic applications are listed. When the user clicks on a document name, the application that is used to view the file remains while the others disappear. The user can then simply click on the button and open the file. Other, more specialised applications such as SSR, are listed at the bottom of the form when the specific category is chosen. For example when monitoring is chosen SSR, GeoMoS, Laser and ISSI software applications are shown. GILS is a very simple application that can go a long way to saving time and therefore money for any department at PPRust.

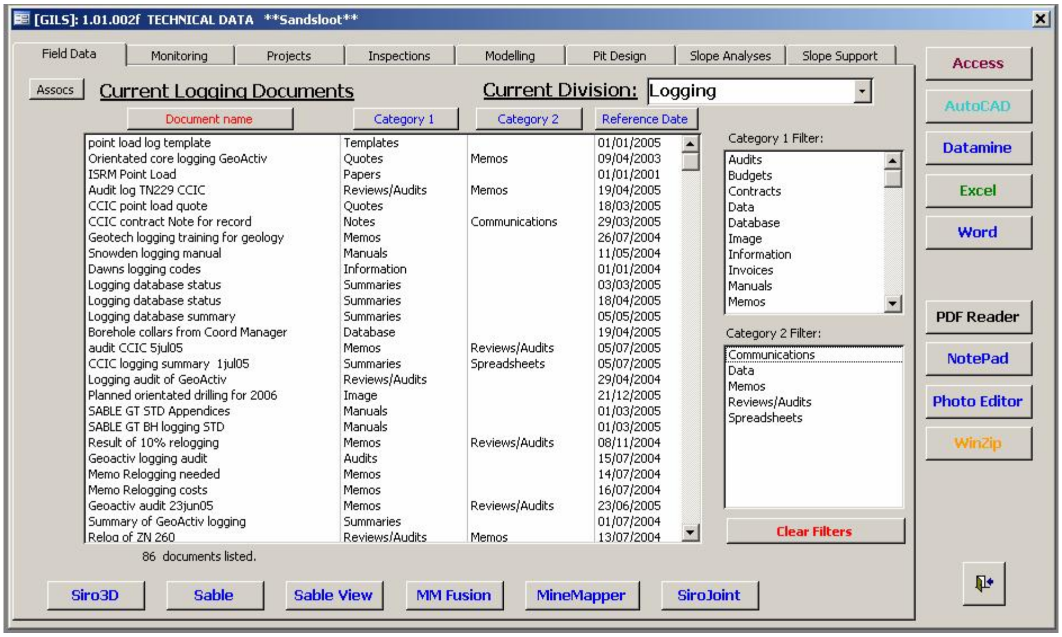


Figure 4.23 Technical data tab in GILS showing logging related files and applications

4.9 Conclusion

This Chapter has shown in detail how geotechnical data is collected and stored at PPRust. Due to a decision made in 1998 to log every exploration borehole geotechnically, PPRust now has a huge logging database. The decision in 2003 to log not only Laubscher MRMR but also Bieniawski's RMR and Barton's Q enabled the creation of an Anglo Platinum logging standard. The development of the geotechnical SABLE database makes this information usable and more reliable. The MineMapper3D database allows the same to be true for the mapping data that is collected. The rock testing database is also very comprehensive and includes not only typical geotechnical strength tests such as UCS, but also strength tests that are used by the metallurgists. The data collection at PPRust is done with the entire mining and processing operation in mind, both on a short term and long term scale. By using international standards and sound databases provide the mine with reliable and relevant information. The following chapter will explain how the data is used for design purposes.

5 BLOCK MODELLING

"… when you can measure what you are speaking about, and can express it in numbers, you know something about it, but when you cannot express it in numbers your knowledge is of a meagre and unsatisfactory kind…"

- Lord Kelvin (1824-1907)

This Chapter describes the usage of the geotechnical data collected in block modelling. It gives the step-by-step method of creating a geotechnical block model and its uses in slope design, blast design, plant design and other applications.

5.1 Introduction

A 3D geotechnical block model was developed and implemented by Dr Alan Bye at PPRust over a period of six years, from 1998 to 2003. He designed a model for Sandsloot north pit as that was the only area in the operation with sufficient geotechnical information and it was the focus of the operation at the time. The concept used was the same as that of an ore reserve model where borehole data is used to interpolate a value for hundreds of blocks throughout the open pit. In this case, instead of interpolating grade, the geotechnical parameters, UCS, RQD, FF and MRMR were interpolated into 15 x 15 x 15 m blocks. The interpolated data was then used to calculate ideal slope angles as well as blastability indices for each block. This enabled slope and blast optimisation in Sandsloot open pit. The implementation of the geotechnical model dramatically improved loading efficiencies and milling rates. Since 2003, geotechnical block models for Zwartfontein South open pit, PPRust North open pit and the Tweefontein North and Tweefontein Hill projects have been created. Additional functionality has also been added to the initial script so that slope and blast designs can be optimised further.

5.2 Creating a Geotechnical Block Model

The development of a geotechnical block model is only possible if a fairly large geotechnical database is available for the area of interest. As shown in Chapter 4, this is the case at PPRust. The amount of data will never match that of an ore reserve model, where sampling is done every 50 cm along the reef while geotechnical 'samples' consist of zones that range from 2 m to 80 m in length. These zones can however be split into smaller samples of the same value which improves the interpolation. Most of the parameters have a normal distribution and a borehole spacing of 150 m is sufficient for the confidence required. Once the data is collected and stored in a usable format – ideally a database as described in Chapter 4 – then a number of steps must be followed to develop the geotechnical model. Figure 5.1 is a simple flow diagram of the process that is followed. These steps are detailed below and refer to the method used for Zwartfontein South open pit. The method used for Sandsloot is similar but simpler and was described in Bye (2003). The entire process has been written into three user-friendly Datamine scripts by the author for use at PPRust. This ensures transparency, auditability, efficiency and accuracy.

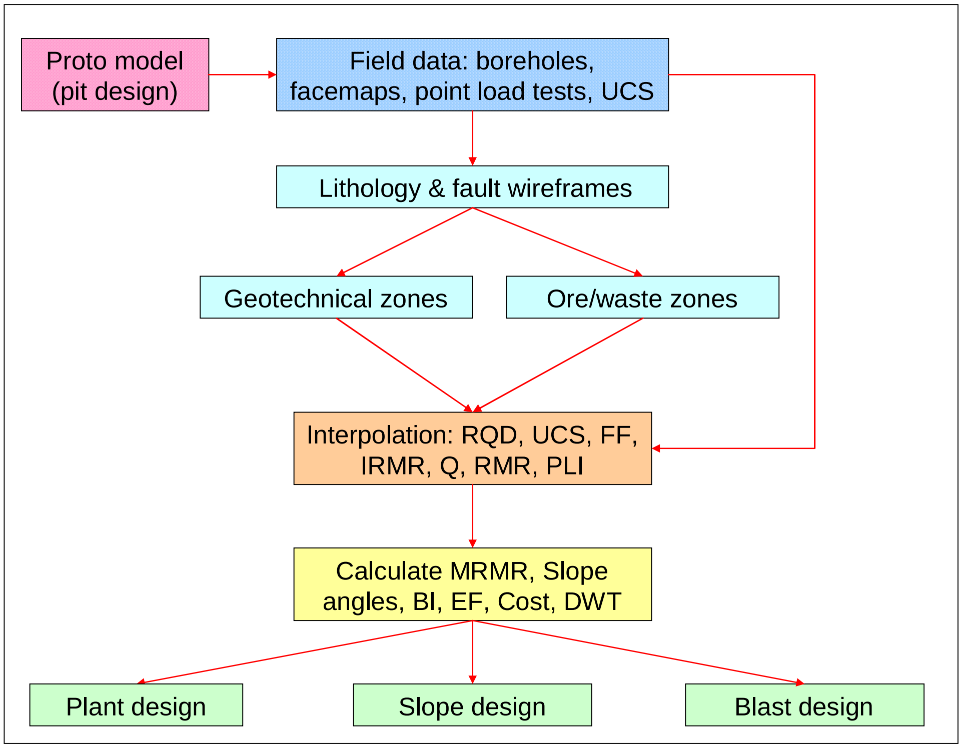


Figure 5.1 Flow diagram of the development of the geotechnical block model

Step 1 Create proto model

The first script, shown in Figure 5.2, is used for creating the populated block models. The first step in this script is to create a proto model – a definition of the extent of the model. This is based on the pit design that is being mining or is going to be mined. It is advisable to put the limits at least 100 m outside the pit limits to model the rock mass at depths in the highwalls and to allow for changes in design. The proto model is created using the PROTOM command in Datamine. Firstly, the origin (X, Y, Z), the block dimensions and the number of blocks along each axis are defined. For Sandsloot and PPRust North, the blocks are cubes with 15 m sides as the benches are 15 m high. In Zwartfontein South the blocks are cubes with 10 m sides as the benches are 10m high. In Tweefontein the blocks are 20 m high as the final design is undecided as yet. It is best to create a separate model for each cutback in each pit as this allows smaller models to be created, which are faster to update. It also ensures more accurate application, as slope geometry often changes in each cutback. The current model for Zwartfontein South applies to Cut 4, which is underway at the moment, while a model for Cut 5 is being developed.

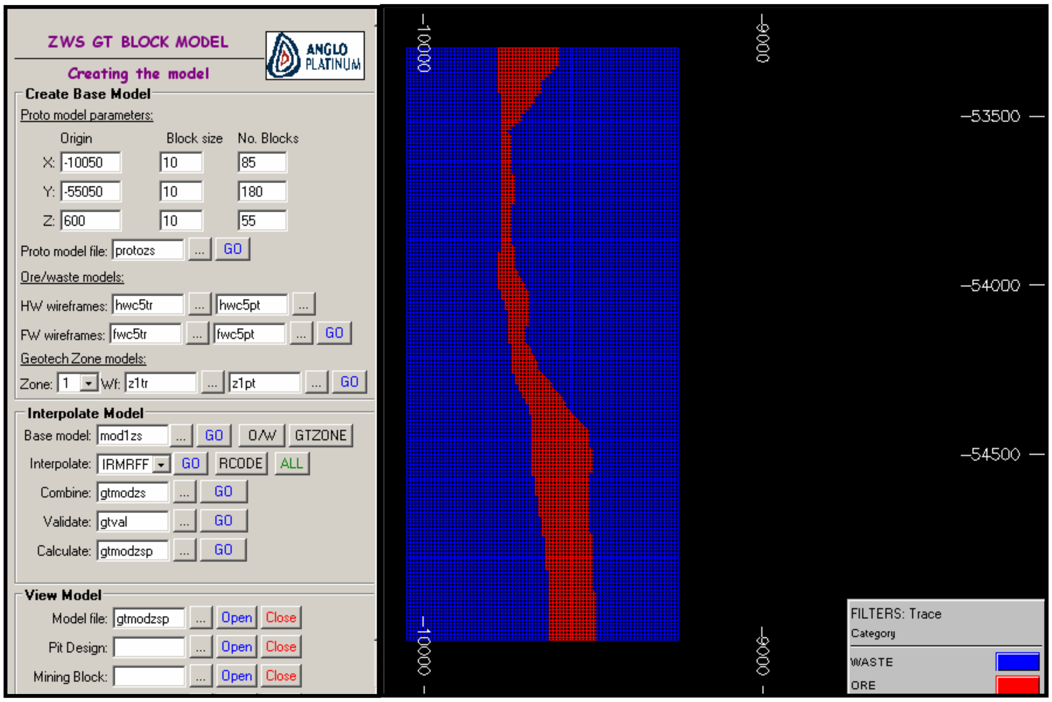


Figure 5.2 First script used to create the block model showing the ore and waste models

Step 2 Import field data and create wireframes

The geotechnical boreholes and facemaps are imported into Datamine as explained in Chapter 4. This link should be refreshed to ensure the latest data is being used. The data files should be validated before the data is desurveyed to form the 3D boreholes. By definition, the value in each geotechnical zone applies to every centimetre along that zone. This means that each zone can be sub-divided into small sections e.g. 50 cm which makes the number of samples comparable to ore reserve models. The boreholes and facemaps are sub-divided or 'composited' in Datamine terms, to 3 m samples. This greatly increases the confidence in the model interpolation later on. It is useful to import the geological logging and mapping data to verify the geotechnical logging and for structural and lithological wireframing. Wireframes are 3D surfaces in Datamine that are generated from strings. At PPRust the Geology department creates fault and ore contact wireframes as part of the ore reserve modelling process and these can be used to identify geotechnical domains.

Step 3 Define ore/waste models

As the model will be used for blast design it is important to define the location of the ore and waste as their fragmentation targets differ. The ore reserve model data was used to create wireframes of the top of reef and base of reef contacts. These contacts do not necessarily correspond to lithological contacts as mineralisation can occur in the hangingwall and footwall and the various pyroxenite facies. Three separate models were created from the proto model with an 'Ore/Waste' field, which determines whether they are ore or waste. The three models were then combined into one model. The script enables the user to locate and use wireframes and/or models that have been previously defined. It also allows the user to view the models as they are built. Figure 5.2 shows a section through the Zwartfontein South Cut 4 ore/waste model.

Step 4 Define geotechnical zone models

Each pit is divided into geotechnical zones according to failure mechanisms and rock mass properties. As these affect the slope design and blastability index, separate block models need to be created for each geotechnical zone. This is done by creating wireframes around each zone and filling them with blocks from the proto model. A new field 'Geotech Zone' is created in each model and its respective number filled in. The zone models are then combined and the entire model is combined with the ore/waste model created in Step 3. The script enables the user to locate and use wireframes and/or models that have already been previously defined. It also allows the user to view the models as they are built. This simple model provides the foundation on which the complete model is built. Figure 5.3 shows a plan view of the Zwartfontein South Cut 4 geotechnical zone model with the Cut 4 perimeter and ramps overlaid.

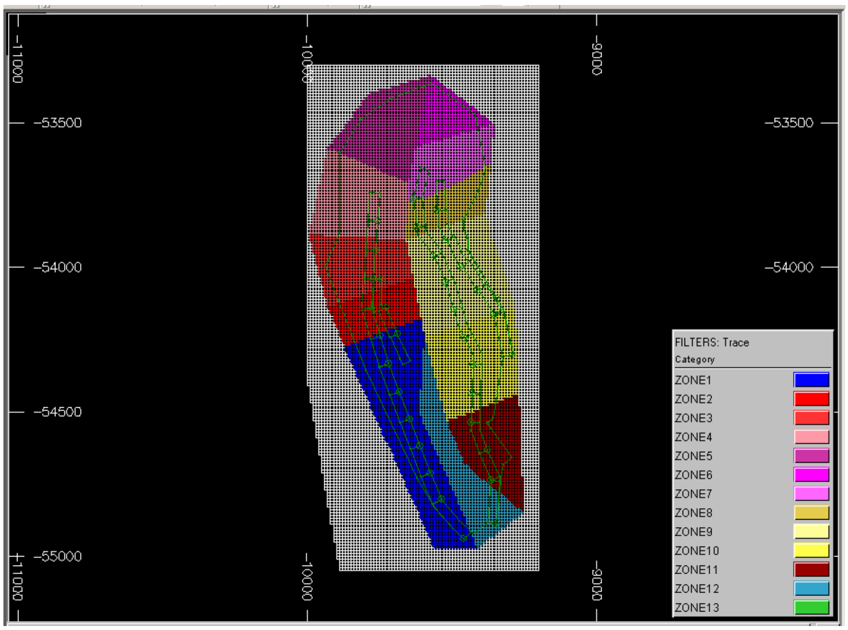


Figure 5.3 Zwartfontein South Cut 4 geotechnical zone model slice

Step 5 Interpolate geotechnical data

The geotechnical data is interpolated into the model blocks using the inverse distance squared statistical method. In this method every sample within the search ellipse contributes to the value of the block but each sample is weighted according to the square of its distance from the block (Figure 5.4). The distance is taken as the midpoint of the sample to the midpoint of the block. This geostatistical technique is chosen over kriging due to the normal distribution of most of the data. A search ellipse with 200 m along 45° dip, 100 m along the normal to 45° dip and 200 m along strike is used at PPRust as the stratigraphical layers dip at 45° to the west. This interpolation is performed for UCS, RQD, FF, IRMR (using FF), RMR, Q and PLI for each geotechnical zone. The interpolation can be done one parameter at a time or all at once with the script. A separate interpolation is done for rock type using the nearest neighbour method. Each rock type is assigned a numeric code so that this can be done. Rock type should not, theoretically speaking, be interpolated, however, Zwartfontein South has highly complex geology due to faulting and thrusting and it is difficult to wireframe the geological contacts. The nearest neighbour method is used to give an idea of the rock type of the block but is not used for any calculations.

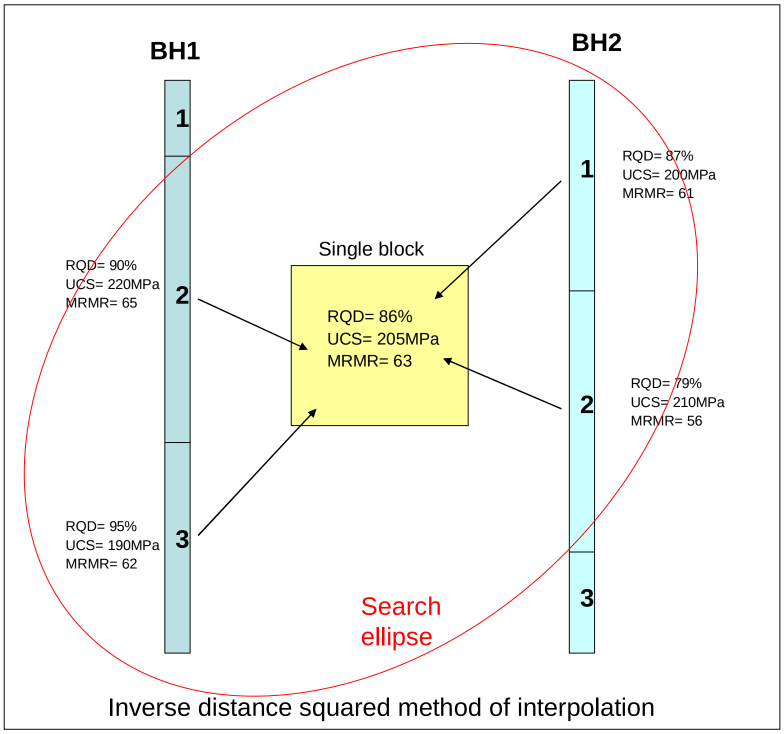


Figure 5.4 Simple illustration (not to scale) of interpolation of geotechnical borehole data into a single block in Datamine. Geotechnical zones 1 in BH1 and 3 in BH2 are not used in the interpolation as they fall outside the search ellipse.

This interpolation produces a geotechnical model of hundreds of blocks that each contain the following parameters: X, Y, Z, Ore/Waste, Geotech Zone, Rock Code, UCS, RQD, FF, IRMR, RMR, Q and PLI. The model can be viewed using the script which allows the user to colour-code the model according to any of the interpolated parameters, as shown in Figure 5.5.

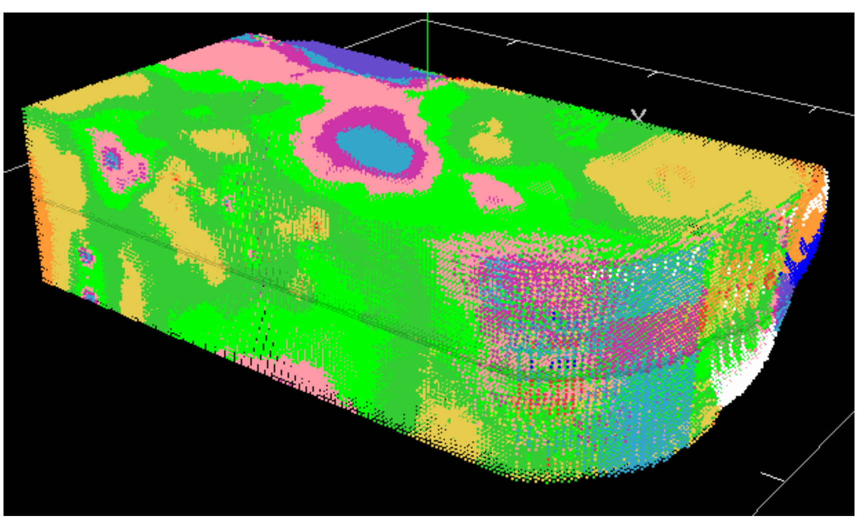


Figure 5.5 Zwartfontein South Cut 4 block model coloured on IRMR(FF)

This model can now be used for slope design and blast design. A different script is used for each one so different users can perform different tasks.

5.3 Slope Design Application

Laubscher's MRMR can be used to calculate an ideal slope angle using Haines and Terbrugge's (1991) slope design chart (Figure 5.6). This chart allows the user to calculate a slope angle based on MRMR, slope height and factor of safety (FOS). In mining, a FOS of 1.2 is usually used for slopes while a FOS of 1.5 can used for ramps. A formula for a 100 m stack at FOS=1.2 was derived from the chart:

Slope angle = 0.4456*MRMR+35.226

The PPRust script is used to apply Laubscher's adjustments to IRMR based on the geotechnical zone in the model. This produces a MRMR value per block. A slope angle (100 m stack, FOS=1.2) for each block in the model is calculated using the script.

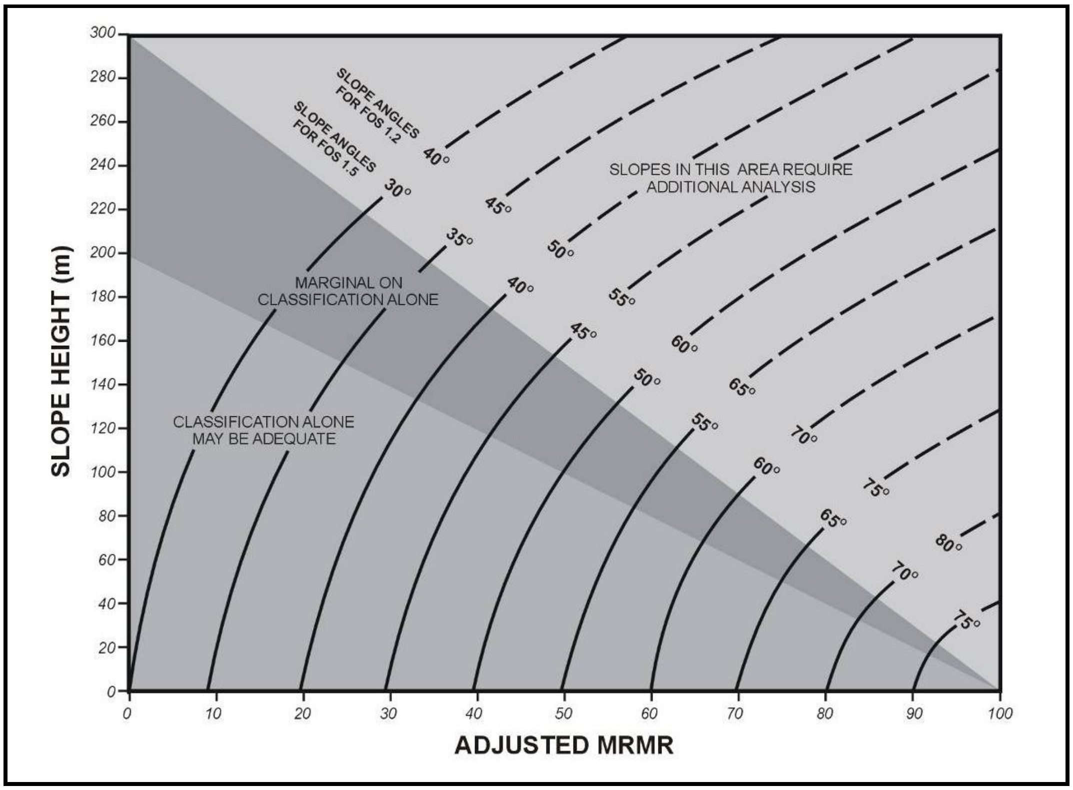


Figure 5.6 Haines and Terbrugge slope design chart

A second script was developed (Figure 5.7) to use the model for slope design. Equations for other stack heights at FOS=1.2 were also derived so that slope angles for different stack heights can be calculated. The required slope height is selected and the slope angle is calculated. The varying slope angles can be colour-coded with the 'Slope' button and viewed on plan or in section with the pit design (Figure 5.8). The 'Visualise' button places the model data on a chosen cutback design, highlighting areas of high risk (Figure 5.7). Standard cross-sections can be used or any user-defined sections can be displayed. This aids in the initial design process but obviously geological structure, groundwater, monitoring and mining method must be taken into account for the final design. The ‘SRisk’ or Slope Risk button highlights where the slopes are over or under designed. This helps to identify where slope monitoring and/or support needs to be installed or increased and where slope design may need to be changed. The slope design functionality has highlighted the fact that the Zwartfontein South pit is under-designed and the slope angles could be increased by at least 5 degrees. This translates to a cost benefit of over R1 billion for the life of the mine.

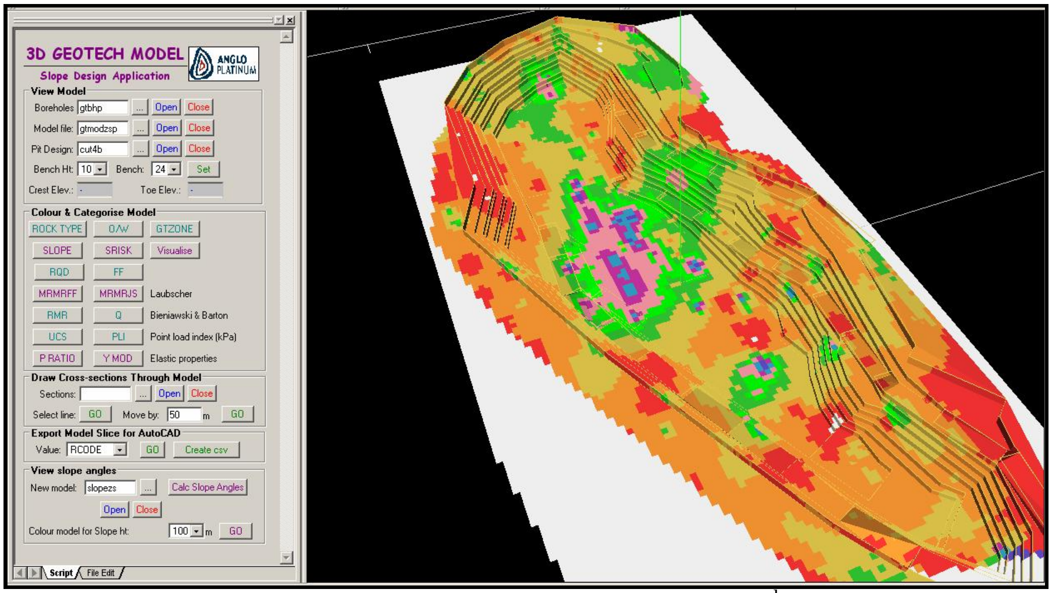


Figure 5.7 Visualisation of MRMR on the pit slopes with the 2nd script

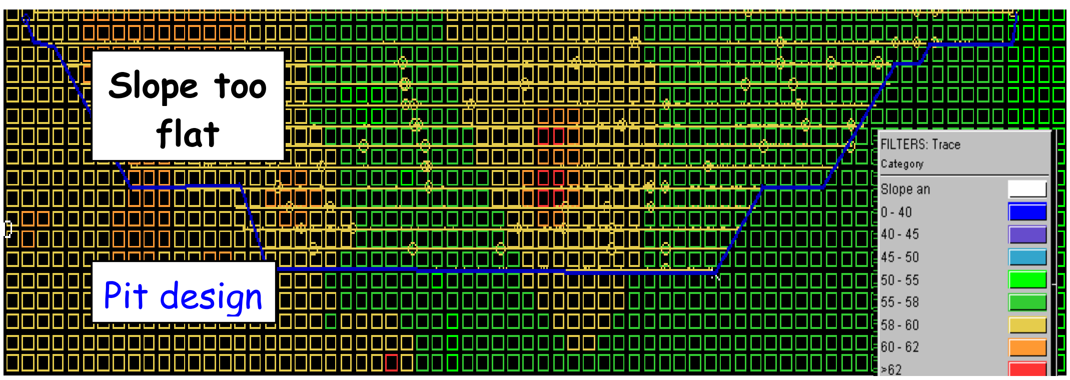


Figure 5.8 Vertical slice through the Zwartfontein South block model coloured on Slope angle for a 100m stack at FOS=1.2 indicating the western wall is under-designed.

Apart from calculating slope angles, the model is used to refine the geotechnical zones discussed in Chapter 6. Other rock properties, such as cohesion and friction angle, can be included in the model (based on rock type) and used in failure analysis. Even with this functionality, the geotechnical block model currently only provides a guide to slope design. The full design process includes numerical modelling which is discussed in Chapter 7. The block model does improve the confidence in the data used for design though, and it provides a tool for much more regular review and incorporation of new data. It is a cheap and effective way of increasing the reliability of design.

Steffen (1997) suggested classifying slope designs into three classes in the same way that geological models are classed, as specified by the SAMREC code. The three classes in order of increasing confidence are:

1. *Inferred slope angle*

Based on experience in similar rock types and rock mass classification. Reasonable inference of geology and groundwater.

2. *Probable slope angle*

Based on information that allows assumption of continuity of lithologies. All major structures and joint sets must be identified and a small sample of testing for properties of in-situ rock and joint surfaces performed. Groundwater data based on a few piezometers. Data allows simplified design models and sensitivity analyses.

3. *Proven slope angle*

Confirmed continuity of lithologies. Detailed structural mapping and rock testing that can be extrapolated for the rock mass. Reliable piezometer installations give high confidence in the groundwater model. Data confidence is 85%.

The geotechnical block model provides much of the information needed to qualify the resource as ‘proven’ which is the goal. The plan for the future is to incorporate all the geotechnical data available – groundwater, monitoring etc – into one model so that the slope can be designed with the highest confidence.

5.4 Blast Design Application

Another major strength of the geotechnical block model is that it crosses the departmental barriers between rock engineering, blasting and process. Geotechnical data can be used to determine a blastability index which is used for blast design. This is used to optimise blasting and produce ideal fragmentation profiles required by the processing plant.

5.4.1 Blastability index

In 1986 Lilly used geotechnical parameters to develop a Blastability Index (BI) to provide estimates of the difficulty of blasting a rock mass. He identified five rock mass parameters that he believed contributed significantly to the performance of blast. They are:

- the structural nature of the rock mass
- the spacing of planes of weakness
- the orientation of planes of weakness
- the density of the rock
- the strength of the rock

Lilly’s BI has been widely used and has proved to be a simple yet effective tool for the blasting engineer. The equation for calculating BI is:

BI = 0.5*(JPO + RMD + JPS + RDI + S)

where JPO = Joint plane orientation
RMD = Rock mass description
JPS = Joint plane spacing

RDI = Rock density index
S = Strength

The assessment of each parameter was designed to be very simple so that a quick and easy assessment of a blast block could be made immediately prior to blasting. At PPRust, the detailed geotechnical data allows for slightly more defined assessment as explained below.

JPO

JPO, or joint plane orientation, refers to the orientation of the joints in the blast block in relation to the free face. It is impossible to know months in advance where the free face will be therefore a JPO is assigned per geotechnical zone in the block model. This is based on whether the joint sets are horizontal (which aids blasting) or inclined (which hinders blasting) according to Lilly's simple table (Table 5.1).

Table 5.1 JPO ratings

JPO	Rating
Horizontal	10
Dip out of face	20
Strike normal to face	30
Dip into face	40

RMD

RMD, or rock mass description, represents the overall rock mass conditions for the blast block, which is obviously very important in blast design. Lilly proposed a simple table (Table 5.2) for determining the RMD, however, recent work done at PPRust has shown that RMD can be estimated by halving Laubscher's IRMR (using FF) to obtain RMD for the PPRust data.

Table 5.2 RMD ratings

RMD	Rating
Powdery, friable	10
Blocky	20
Massive	50

RMD = IRMR (FF) /2

JPS

JPS, or joint plane spacing, describes the spacing of joints in the blast block. This tends to control the size and shape of the blasted muckpile fragments. Again Lilly developed a very simple table to rate the joint spacing (JS), which is also a rating for fracture frequency (FF), as FF is merely the inverse of JS (Table 5.3).

Table 5.3 JPS ratings and the equivalent FF

JPS (m)	Rating	FF (/m)
Close (<0.1m)	10	>10
Intermediate (0.1-1m)	20	1-10
Wide(>1m)	50	<1

In order to give more definition to the rating, a more detailed table was developed by Bye (2003) which he used to plot a graph of FF versus JPS. In this way he developed a bilinear relationship between FF and JPS and used 2 linear equations to calculate JPS based on FF. These equations, however, can produce values outside the range 0 to 50 that Lilly determined for JPS. The concept of using graphs instead of three single values is good and therefore the author proposed another method: take four points of (FF,JPS) to represent the three ranges for FF (Table 5.4) and use these four points to create a tri-linear graph (Figure 5.9) to determine three equations for JPS. The points (0,50) and (100,0) will ensure that the JPS never exceeds the range zero to 50. The point (1,30) was chosen as it is halfway between 10 and 50, which was Lilly's original range for 1<FF<10. The point (100,1) was chosen as the maximum practical FF that can be measured – it equates to a fracture every centimetre. This graph is shown in Figure 5.9.

Table 5.4 Points used for determination of FF conversion to JPS rating

FF (/m)	JPS
0	50
1	30
10	11
100	0

The three equations, which all have a correlation coefficient of 1, are:

If FF<1 then **JPS = -20FF+50** (30<JPS<50)
If 1<FF<10 then **JPS = -(19/9)FF+32.11** (10<JPS<30)
If FF>10 then **JPS = -(11/90)FF+12.22** (JPS<10)

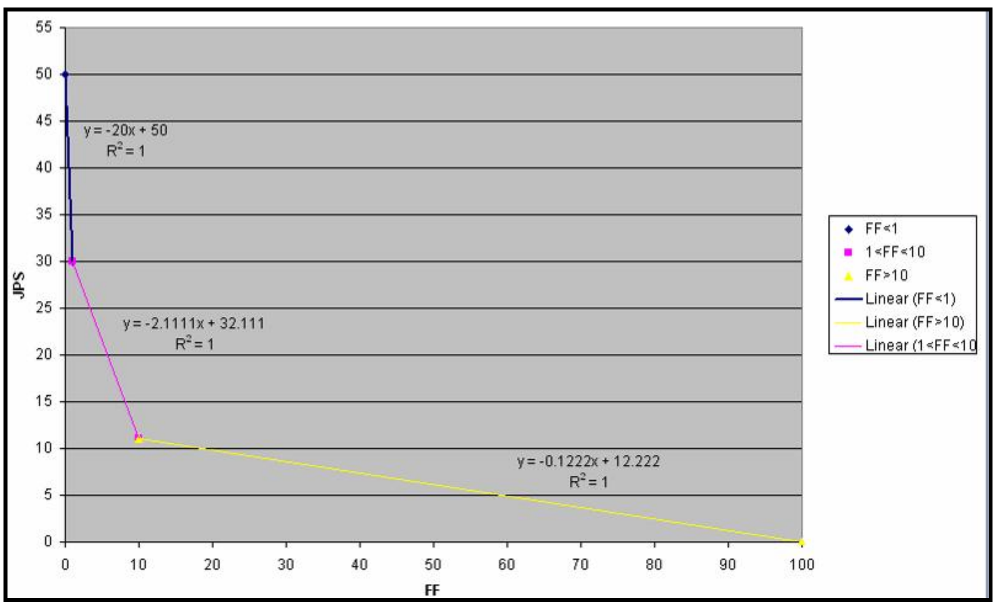


Figure 5.9 Graph showing tri-linear relation between FF and JPS

RDI

RDI, or rock density index, is simply the density of the rock mass in the blast block. Higher density rocks require more blast energy to move. The densities at PPRust are fairly homogenous and a standard 3.1 g/cm^3 is applied to the ore while a standard 3.0 g/cm^3 is applied to the waste.

RDI = (density * 25) – 50

S

S, or strength, represents the rock strength, which obviously affects the blastability of the rock. It is related to the UCS measured in megapascals (MPa).

S = UCS * 0.05

The usefulness of the BI is that it can be used in Cunningham's Kuz-Ram equation (1986) to determine the fragmentation that a given powder factor (or energy factor) would produce or alternatively, what energy factor should be used to produce a required fragmentation. Cunningham (1986) related Lilly's BI to a 'rock factor', A, with the simple equation:

A = BI * 0.12

This rock factor is used in the Kuz-Ram equation which he adapted from Kuznetsov (1972):

$$X = A*K^{0.8}*Q^{0.167}*(RWS/115)^{-0.633})$$

Where X = Predicted mean fragmentation diameter (cm)
A = Rock Factor
Q = Mass of explosive per blast hole (kg)
K = Powder factor (explosives per m^3 of rock) (kg/m^3)
RWS = Relative weight strength of explosive

The mass of explosives per blasthole, Q, is calculated with the following formula:

$$Q = L_{ca}*R_c$$

Where: L_{ca} = Actual charge length (m) = H_b-H_s+H_{sd}
H_b = Bench height (m)
H_s = Stemming height (m)
H_{sd} = Subdrill height (m)
R_c = Linear charge density (kg/m) = $(\pi/4)* \rho *(D/1000)^2$
D = Blasthole diameter in (mm)
ρ = Relative density of explosives (kg/m^3)

The explosives HEF206 and P700 are used at PPRust. They both have a density of 1250 kg/m^3 and have a RWS of 98 and 96 respectively. Sandsloot and PPRust North mine 15 m benches with a 2.5 m subdrill and 6 m stemming. Zwartfontein South mines a 10 m bench with a 2 m subdrill and 4 m of stemming. The blasthole diameters used are 165 mm and 270 mm. So the value of Q will vary according to pit, explosive and drill diameter.

The equation below is useful for determining the mean fragment size of a blast block. The required fragment size has already been determined at PPRust and instead the energy factor (EF) to produce that mean fragment size is required. This can be calculated using the same equation but rewritten as:

$$\mathbf{EF = X/(A*Q^{0.167} * (RWS/115)^{-0.633})^{-1.25}}$$

Figure 5.10 illustrates the flow of information from geotechnical field data to the blastability index to the Kuz-Ram equation.

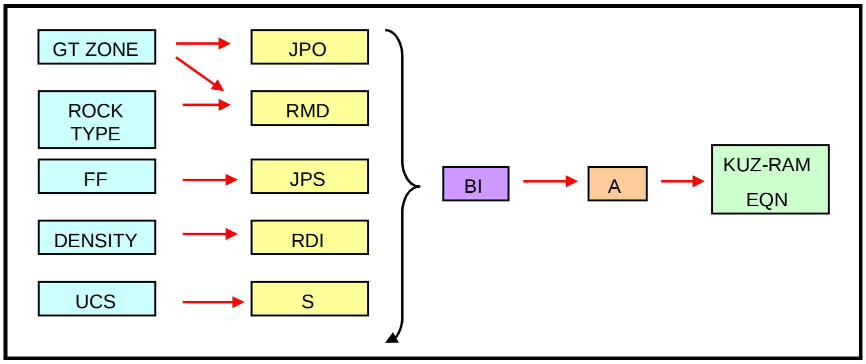


Figure 5.10 The flow of information from geotechnical data to Kuz-Ram

In order to use this information for blast design, the burden and spacing must be calculated. This is done with the following formulae:

$$\mathbf{B=L_{ct}*R_c/(1.15*EF*H_b)^{0.5}}$$

Where: B = Burden
L_{ct} = Linear charge length = $H_b - H_s$
R_c = Linear charge density in kg/m $=(\pi/4)*\rho*(D/1000)^2$
H_b = Bench height (m)
H_s = Stemming height (m)
D = Blasthole diameter (mm)
ρ = Relative density of explosives (kg/m^3)
EF = Energy factor (kg/m^3)

S =1.15*B

Where: S = Spacing
B = Burden

A correlation was developed between the required energy factor and cost per cubic metre based on the known drill and blast costs at Sandsloot (Bye, 2003), which included drilling, explosives, labour and maintenance.

$$\mathbf{Cost/m^3 = EF * 2.946}$$

All the geotechnical data necessary to calculate a BI is modelled in Datamine as described earlier. It is a simple step to calculate a BI per block which is then used to calculate an energy factor, drill and blast cost, burden and spacing that will produce the required fragmentation. A third script was written to facilitate this (Figure 5.11). This calculation is performed by clicking the 'Run Calcs' button in the script. Text boxes and drop-down menus were included in the script so that the value for Q (explosive charge) can be recalculated for various bench heights, subdrill depths, stemming heights, explosives, and drillhole diameters.

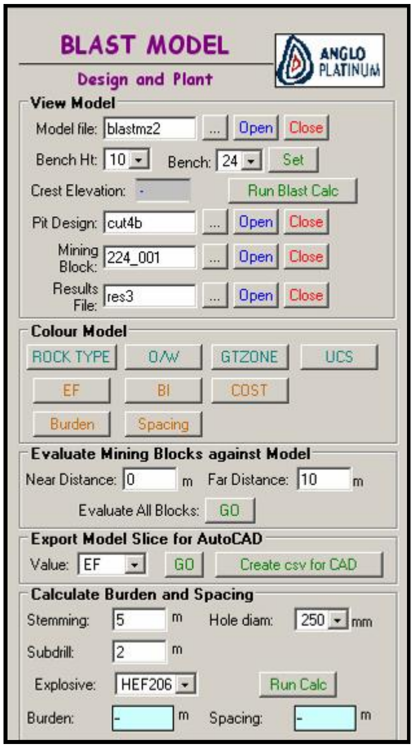


Figure 5.11 Third script for blast design

The script also has a series of buttons to filter the data on BI, EF, burden, spacing and Cost. The user can import any blast design boundary into the model and query the model for all the information within that boundary. A results file is created with the average model information for each rock type.

5.4.2 Fragmentation

The definition of the fragmentation targets was a lengthy process based on processing plant and Load and Haul requirements, and what was practical in the pit. The measure for the Plant requirements is mean fragment size (P_{50}) which is the sieve size that 50% of the fragmented material will pass through. The fragmentation profile is also of high importance as the plant uses autogenous mills which rely on the rocks to break each other into smaller fragments. They require a coarse feed (+125 mm) and a fine feed (-25 mm) for optimum performance. The particles between 25 mm and 125 mm, defined as the critical size, are discharged as pebbles and have to be mechanically crushed. Digital fragmentation analysis was done from 2001 to 2002 using Split-Desktop® which is an image-processing program designed to calculate the size distribution of rock fragments by analyzing digital greyscale images taken by a digital camera in the field (Split®, 1999). Split-Desktop® software ‘provides an economical alternative to manual sampling and screening and an objective quantitative measure rather than subjective qualitative estimates’ (BoBo, 2005). Figure 5.12 illustrates the Split analysis process. After blasting, the muckpile is photographed with a digital camera. Two identical balls of ~20 cm diameter are placed on the muckpile in various areas to obtain a good sample of the range in fragment size. The photographs are imported into the Split-Desktop software. The Split-Desktop software has five progressive steps for analyzing each image.

1. The scale is determined for each photograph
2. Automatic delineation of the fragments in each of the images that are processed
3. Editing of the delineated fragments to ensure accurate results
4. Calculation of the size distribution based on the delineated fragments
5. Graphing and various outputs to display the size distribution results

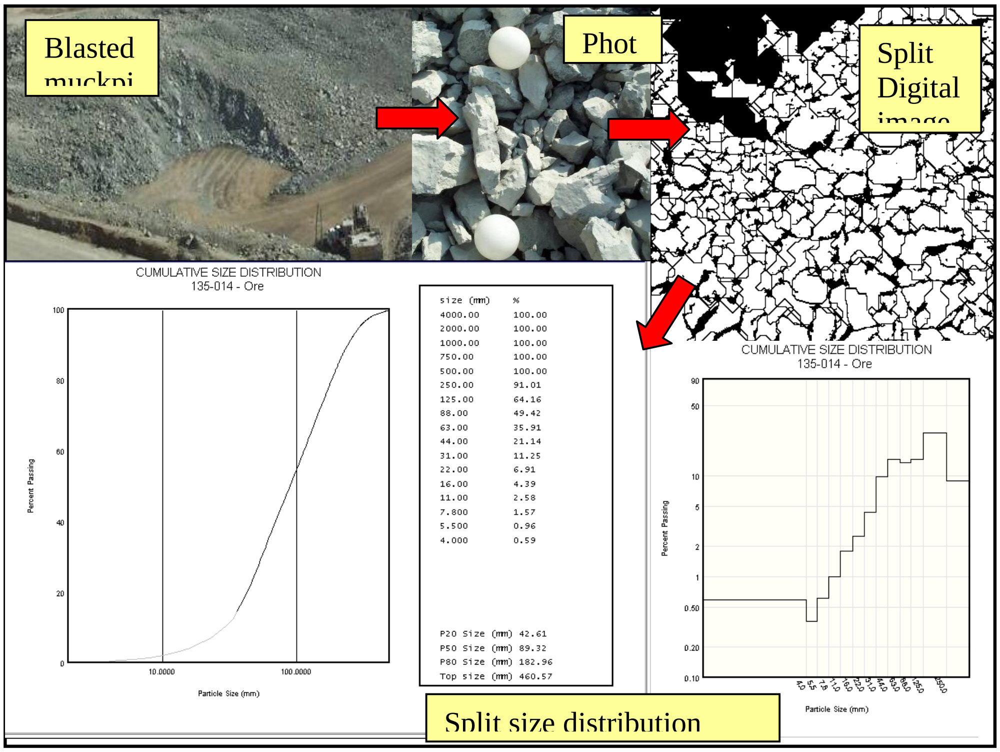


Figure 5.12 Split digital fragmentation analysis (after Bye, 2003)

Eighteen blasted muckpiles from a wide range of powder factors (0.94 – 2.28 kg/m^3) and hole diameters (171 mm – 311 mm) were analysed to determine the fragmentation profiles from both ore and waste. The percentage of coarse and fine fragment sizes is important for the autogenous mill as breakage is a result of particles colliding. The results showed that a mean fragment size of 150 mm for ore should be delivered to the primary crusher (-250 mm) on the way to the Plant. This target was met by increasing the energy factors and the ore fragmentation profile changed over the trial period to meet the mill requirements (Figure 5.13). The blasting costs in 2003 increased from R1.31 per tonne to R1.89 per tonne of rock which totalled R231 538 per month. The cost of rock fragmentation by blasting equates to only 10 -15 % of the overall comminution costs of the operation at PPRust (Bye and Bell, 2001). This extra cost is far outweighed by the savings gained in the plant. The crushing costs in 2003 were reduced from R0.72 to R0.31 per tonne of ore which equates to a cost saving of R168 877 per month for the primary crusher alone. The plant milling rates are largely dependant on feed size and hardness. They were therefore increased by 8.8% due to the finer fragmentation. This accounted for R2.1 million per month in additional revenue (Bye, 2003).

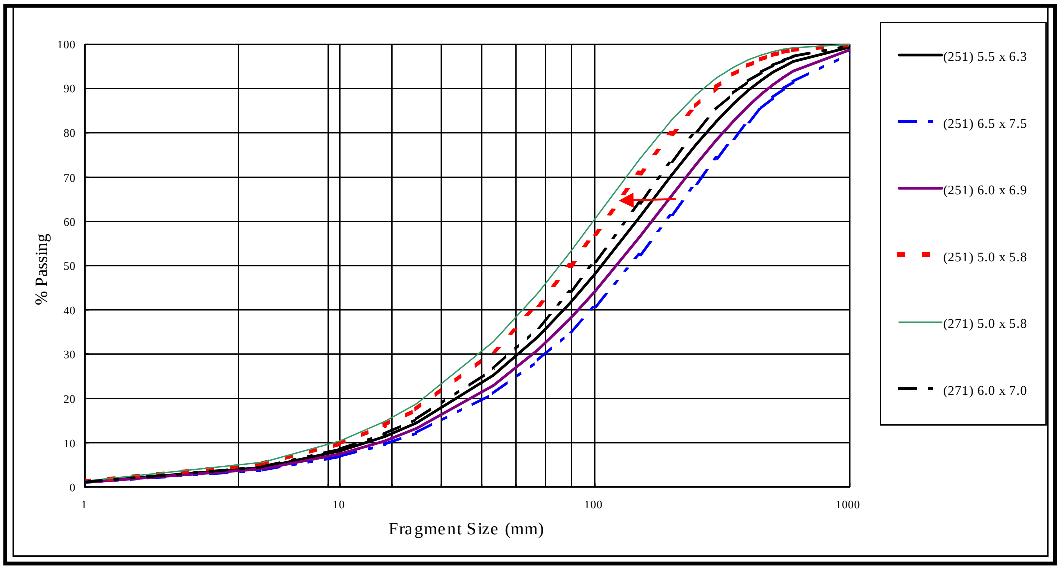


Figure 5.13 History of ore blast fragmentation curves at Sandsloot pit (Bye, 2003)

A primary measure for the Load and Haul efficiencies is instantaneous loading rate (ILR) which is the tonnes per hour loaded by a face shovel (Figure 5.14). It is recorded at PPRust by the Modular Mining Truck/Shovel Dispatch® system. The ILR of 238 blasts were assessed to determine the ideal fragment size for loading (Bye, 2003). A target ILR of 3200 t/hr for waste and 3300 t/hr for ore was set based on the analysis. This was to coincide with the 150 mm mean fragmentation target of the plant and equated to a 230 mm fragmentation target for waste. With the implementation of the geotechnical block model, the instantaneous loading rates improved by 8.5%.

Figure 5.14 RH200 face shovel loading a blasted muckpile into a haul truck

5.4.3 Application in AutoCAD

The blast design results produced in the block model are only useful to those people who have Datamine and therefore access to the model. In order to make the data available to the survey, draughting, planning, drilling and blasting personnel, it is exported in .csv format from Datamine and saved to a specified network location. From there it can be accessed from AutoCAD, which is used for the draughting of all the blast requests and designs. Customised menus have been created by Peter Nathan of CGSS for importing the block model data and colour-coding it in the same way as in Datamine. The blasting department can then overlay the information on the official blast request plans (Figure 5.15) and adjust their design accordingly for every blast. A window appears (Figure 5.16) that shows the blaster the parameters used in the model to calculate the energy factor, burden and spacing and allows him to adjust these numbers. A blast design template (Figure 5.17) also appears in the pattern boundary which he can rotate or translate. As he changes the blast design parameters, so the template changes on the screen. This window also shows the total number of holes that need to be drilled, the pattern area and the average UCS for the pattern. This is used by the drillers for drill bit selection.

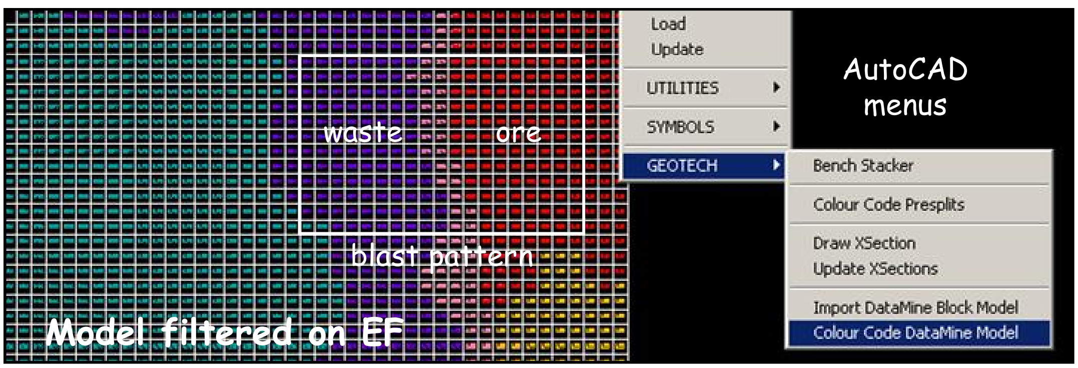


Figure 5.15 Model bench slice filtered on energy factor (EF) with imported blast boundary overlaid and AutoCAD menus for importing and colour coding the model.

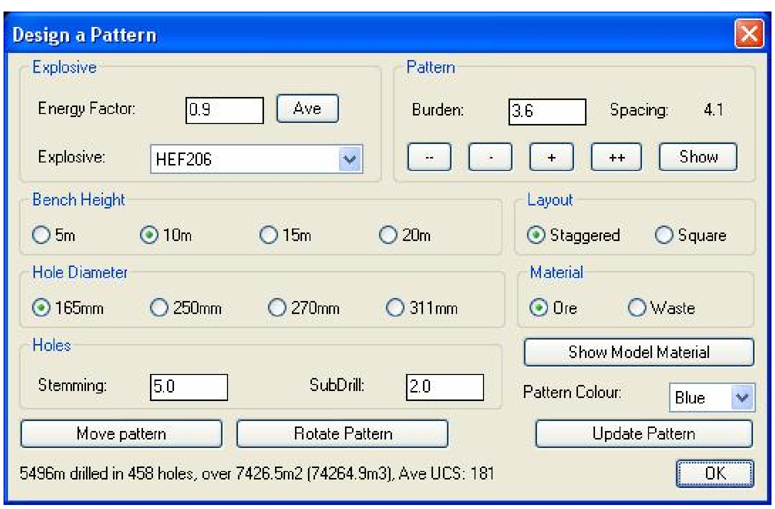


Figure 5.16 Customised blast design window in AutoCAD

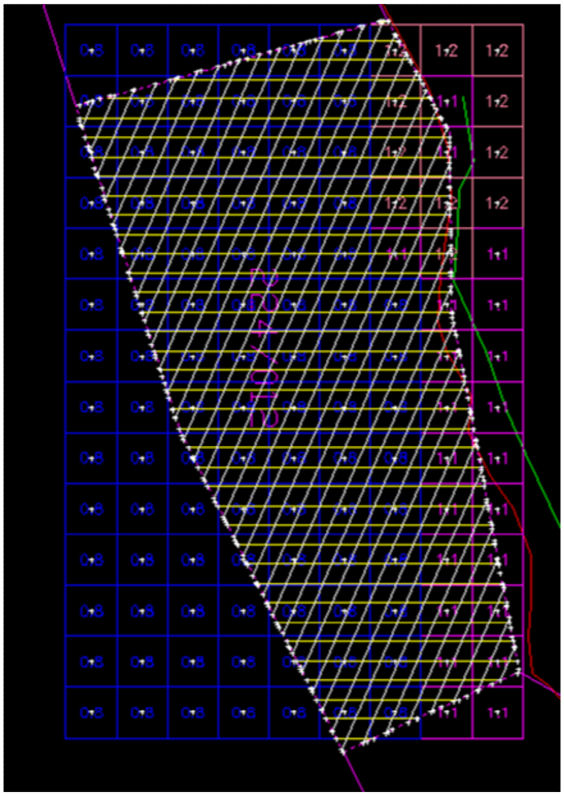


Figure 5.17 Configurable blast design pattern overlaid on the block model in AutoCAD

Geotechnical data has been used in the selection of drill bits and in determining pull-down pressure, rotation speed and air feed on the drill rigs (Bye and Bell, 2001). By improving the drilling performance, the quality of the blasting improved and the drilling costs were reduced by 30 %. Table 5.5 details the varied drilling parameters for the different rock types and their varying rock mass properties. The geotechnical block model is now used to aid the Drilling department in drill bit selection and prediction of penetration rates which is based on the UCS of the rock.

Table 5.5 Empirical drilling parameters per rock type (after Bye and Bell, 2001)

Rock type	**MRMR**	**UCS (MPa)**	**RQD (%)**	**Penetration rate (m/hr)**	**Pull down (kPa)**	**Drill bit**
Norite	53	190	80	15 (11-18)	13790	RM52
Pyroxenite	48	160	65	15(12-20)	12410	RM51
Parapyroxenite	56	200	75	14(11-18)	13790	RM51
Serpentinised Parapyroxenite	61	270	70	14(10-16)	13790	RM51
Calc-silicate	42	140	55	17(13-25)	12065	RM52

5.5 Plant Design Application

The use of the block model for blast design obviously has a big impact on plant efficiencies. There is also potential to use the block model for daily predictions on mill throughput. From the lab test results for UCS and DWT, a correlation with an 80% reliability was made between the two values as per the equation:

DWT = (UCS/2889.5)$^{-1.2566}$

As UCS is interpolated into every block in the model, a DWT estimate is calculated for each block based on this formula. Also, the elastic properties of each rock type are known from lab testing and these are calculated into each block. This information can be used by the metallurgists to predict mill throughput rates and to prepare accordingly. Improved mill throughputs increases the plant recovery and ounces produced. This has huge financial implications and the utilisation of the geotechnical data in this way is the next step that must be taken at PPRust.

5.8 Conclusion

Geotechnical block models have been developed over the last eight years at PPRust for all current open pits. The model uses geotechnical data collected in the field to interpolate a UCS, RQD, fracture frequency and rock mass rating for hundreds of blocks. The model calculates ideal slope angles, blastability indexes, energy factors, costs, drop weight test values and elastic properties for every one of the blocks. Datamine scripts were designed to enable any user to query and export relevant data on a regular basis. The model is used to optimise slope designs. Current and future pit designs are compared with what the latest geotechnical data predicts is the optimal design. This highlights where slope angles are too steep or too shallow and allows for ongoing slope optimisation. This can result in massive cost savings of the order of hundreds of millions of Rands. The model is also used by the drill and blast department to customise every blast design in order to account for rock mass conditions and thus attain the fragmentation targets. The results show that the model adds value to the Mine to Mill process by dramatically improving loading efficiencies and milling rates. Additional revenue of over R2 million a month is realised in the plant alone. The drillers also use the block model for drill bit selection. The data can also be used by the processing plant for predicting mill throughput and improving efficiencies. The model therefore has a wide range of applications which are summarised in Table 5.6. It is likely that a 3D geotechnical block model will become the norm in the mining industry in a few years time as a primary mineral resource management tool.

Table 5.6 Uses of the geotechnical block model

PARAMETER FILTER	**USE**
Rock type, FF, RQD	Geotechnical zoning
Rock properties	Failure analysis
MRMR, Slope angle	Slope optimisation
BI/EF	Blast design
Cost	Drill and Blast budget
UCS	Drill bit selection
DWT & elastic properties	Plant design

6 SLOPE STABILITY

“I skate to where the puck is going to be, not where it has been.”

- Wayne Gretsky

This Chapter explains the basic slope stability concepts and describes the slope stability conditions at PPRust. Geotechnical zones at PPRust are briefly discussed and a case study of a bench failure is given.

6.1 Introduction

Jointed rock slopes are generally stable as there is no freedom of movement for blocks of rock (Hoek, 2001). In a mining environment, however, regular blasting constantly opens up space into which blocks of rock can immediately fail and thus slope failure is a common occurrence. A number of factors determine the slope stability but the most important considerations are the orientation of the discontinuity plane with respect to the orientation of the slope, and the dip of the discontinuity. These two parameters will determine the type of failure mechanism as well as the size of the failure. The geometry of the slope is therefore important. The second most important factor in slope stability is the resistance to shear of the rock which is described by cohesion and the angle of friction of the rock mass. It is also affected by water pressure, weathering and roughness of the discontinuity planes. The amount and rate of deformation are dependant on the geology, mining method and slope design. Slope movement does not need to hinder mining operations if the failure mechanisms are understood and the slopes are properly monitored.

As slope stability is dependant on slope geometry and rock strength, the open pits are divided into geotechnical zones, defined according to location and rock type. In order to determine the dominant failure mechanisms in each of the delineated zones, the DIPS computer programme is used at PPRust. DIPS is a stereographic programme developed by RocScience©, which plots structural data on a stereonet and then enables kinematic analysis to be performed. A stereonet for each type of failure, namely wedge, toppling and planar failure, is plotted for each zone and the dominant failure mechanism for each zone is determined. Individual structures which fall in the region of failure on the stereonet are identified as critical and likely to fail. One cannot automatically assume that they will fail, however, as the resistance to shear of the rock mass plays a role as well.

6.2 Failure Mechanisms

Four basic failure mechanisms have been identified in slopes, namely wedge, toppling, planar and circular failure. The first three occur in hard rocks while circular failure only occurs in very weak material such as soils, weathered overburden or intensely jointed rocks. Circular failure is not defined by discontinuities but is free to find a structure of least resistance and follows a circular path (Hoek and Bray, 1981). Circular failure has occurred at Sandsloot in the Satellite pit in the south-east but as the area has been backfilled, circular failures are no longer a problem at PPRust.

6.2.1 Planar failure

Planar failure is a simple mechanism whereby a prism of rock slides downslope along a planar surface (Figure 6.1a). It will only occur when a discontinuity dips out of the rock face at an angle less than the slope angle but greater than the friction angle. The strike of the plane must be parallel or sub-parallel (± 20˚) with the slope face. Release surfaces must be present, striking perpendicular to the failure plane and they must have negligible resistance. These strict criteria consequently mean that large scale planar failure is a rare failure mechanism. Planar failure can be identified on a stereonet (Figure 6.1b) by drawing in the slope angle plane and a corresponding daylight envelope. A pole friction cone is drawn to show the friction angle. Discontinuities whose poles plot in the region within the daylight envelope but outside the pole friction cone are likely to experience planar failure.

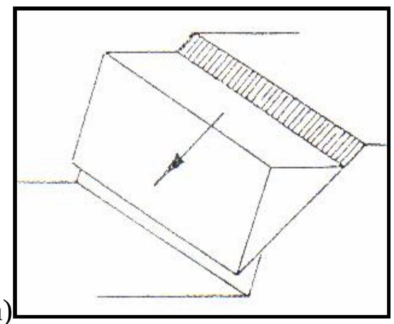
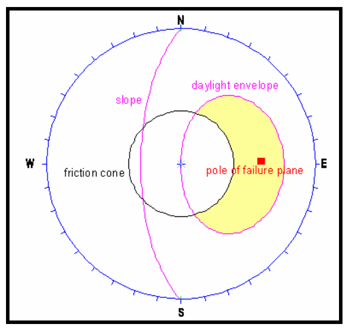
a)  b) 

Figure 6.1 a) Sketch of a simple planar failure (Hoek and Bray, 1981) and
b) its accompanying stereonet (region of failure in yellow)

6.2.2 Wedge failure

Wedge failure occurs on cross-cutting structures that create tetrahedral blocks which are susceptible to sliding (Figure 6.2a). Sliding takes place along the intersection of two discontinuity planes that strike obliquely across the slope face. Sliding only occurs if the inclination of the line of intersection daylights in the slope face and is greater than the friction angle. No release surfaces are required and thus it is a common mechanism in hard rocks. Wedge failure is identified on a stereonet (Figure 6.2b) by plotting the slope angle plane and the plane friction cone. Any intersections of discontinuity planes that plot in the region between the cone and the slope plane are likely to fail.

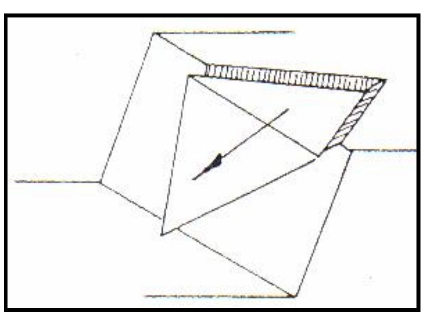
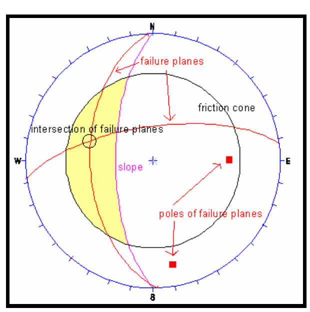
a)  b) 

Figure 6.2 a) Sketch of a simple wedge failure (Hoek and Bray 1981) and
b) its accompanying stereonet (region of failure in yellow)

For calculation of the FOS for wedge failure the RocScience analysis programme SWEDGE is used at PPRust. Input data includes two discontinuities, cohesion, friction angle, slope height, slope angle and density as well as the option of including a tension crack, the bench width, an overhanging crest, water pressure and depth, a seismic force and any other external force. The weight of the trucks on the ramps is included as an external force of 200 tonnes.

6.2.3 Toppling failure

Toppling failure involves the rotation of columns or blocks of rock about some fixed base (Figure 6.3a) and occurs where the rock contains numerous closely-spaced discontinuities (Hoek and Bray, 1981). Blocks or layers bend downslope under their own weight and interlayer slip allows failure to occur. Goodman and Bray (1976) have described a number of types of toppling mechanisms including flexural, block and block-flexure toppling as well as a number of secondary toppling modes. The rocks at PPRust display predominantly flexural toppling where well developed, steeply dipping discontinuities divide the rock into blocks which bend under gravity and then break. Toppling failure will occur when a discontinuity dips into the face and strikes parallel or within 30° of the strike of the slope face. For failure to occur, the normal to the discontinuity plane must be shallower than the friction angle added to the slope angle (Goodman, 1989). This is called the slip limit.

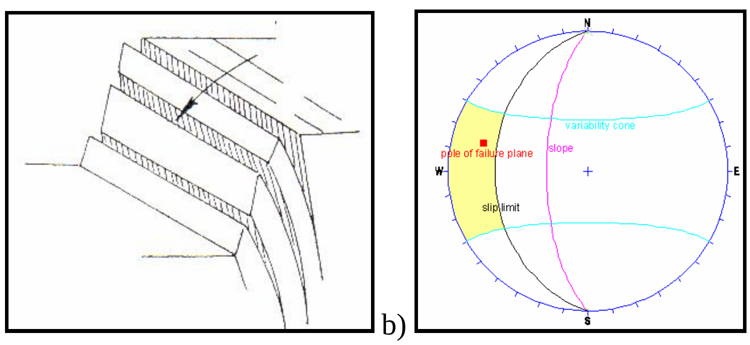
a) b)


Figure 6.3 a) Sketch of a simple toppling failure (Hoek and Bray, 1981) and
b) its accompanying stereonet (region of failure in yellow)

Toppling failure can be identified on a stereonet by drawing the slope angle plane and a slip limit plane as well as a variability cone (30˚) which limits the direction of movement of the block by restricting the strike of the discontinuity plane. Discontinuities whose poles plot in the region beyond the slip limit and between the variability cone lines are likely to topple.

As toppling failure does not involve simple sliding it cannot be analysed by basic calculation of a factor of safety. Analyses in various open pits around the world (e.g. Cyprus Baghdad Copper Mine, Arizona) have been conducted using limit equilibrium analysis of individual blocks (Goodman and Bray, 1976 quoted in Martin, 1990). In this method each block is assessed as a separate free body in sequence from the top block down. The unbalanced forces of each block are resolved and transferred to the next block until the net unbalanced force on the toe block can be calculated and used to determine the stability of slope. Variation in groundwater conditions and shear strength can be used to assess variation in net unbalanced force. This is a complicated method requiring detailed knowledge of numerous parameters. Currently, numerical modelling provides the best method of determining the probability of toppling failure (e.g. Ishida 1990, Singh and Dhar, 1994).

Martin (1990) warns that deep-seated toppling should be considered for any high mine slope and a number of features and controls characterize and provide warning for such failures. They include

- slopes as shallow 30˚ with widely spaced through going discontinuities striking sub-parallel to the slope and with a dip of 60˚ - 90˚
- discontinuities evenly spaced at 15 - 30 m
- squeezing in the toe of the slope causes cracking
- development of obsequent scarps and goat trails as the slope sags

Martin (1990) claims that all toppling failures are extremely sensitive to groundwater pressure and mining activity therefore careful monitoring of critical slopes is vital though major toppling failures are rare.

6.3 Factors Influencing Stability

Water Pressure

Water content in the rock can play a major role in stability of the highwall. Plots of saturation level versus FOS show that the saturation levels only have a large effect on the FOS when they reach 70% or more. The PPRust area experiences heavy rainfall from November to February with an average of 89 mm per month while the rest of the year is dry. Thus water pressures are a concern only during the summer months when pit dewatering needs to be increased. Heavy rains are often the cause of major failures and it is important to be aware of the critical areas that are susceptible to this. There has been much debate over the influence of water on slope stability but it appears to have limited influence in both pits.

Tension Cracks

Tension cracks develop as a result of small shear movements within a rock mass therefore when they are seen in a slope crest they indicate that shear failure has been initiated within the rock mass (Hoek and Bray, 1981). A tension crack will develop where a discontinuity daylights and thus it acts as a warning of failure and not the reason for the failure. Thus daily monitoring of slope crests will give a good indication of critical areas of potential failure and is highly recommended. Undercutting of a slope during mine operations must be avoided as it can cause a tension crack to form. If it does occur then close monitoring of the slope crest must be undertaken.

Blasting

Blasting of over a million tonnes of rock occurs on a weekly basis at PPRust and plays the largest role in destabilising the highwalls. Critical joints are particularly susceptible as they allow explosion gases to vent and thus fracturing follows the joints. It is important therefore to know where the critical areas of potential failure are so that an accurate warning can be given before each blast.

6.4 Geotechnical Zones at PPRust

It is very important in an open pit to delineate geotechnical zones. The zones will indicate a change in rock type, slope design, failure mechanism and rock mass properties. Slope management can then be tailored to each zone instead of utilising a blanket approach which may compromise safety. Geotechnical zoning can be based on a simple visual inspection of the slopes or it can involve detailed drilling, mapping, testing and analysis. The geotechnical zones at PPRust are chosen according to rock type, which influences frequency and spacing of structures, locality, which dictates the orientation of the highwall, rock mass ratings, jointing and failure mechanisms. Geotechnical zones for Sandsloot were first defined by Bye (1996) and later redefined by Little (2002) while zones for Zwartfontein South were determined for Cut 4 in 2006 by the Rock Engineering department. These will be refined for Cut 5 and any other future cutbacks.

6.4.1 Sandsloot geotechnical zones

There are 8 geotechnical zones in Sandsloot open pit as shown in Figure 6.4. Structural data from line surveys and mapping of major structures were plotted on stereonets in DIPS. Joint set identification and kinematic failure analysis was then performed.

Zones 1 and **8** consist of calc-silicates and parapyroxenites in the eastern highwall. A permanent footwall ramp cuts through the slope. It has experienced minor wedge and toppling failure in the past and a stack-scale planar failure in 1998, after heavy rainfall. The dominant failure mechanism is toppling failure caused by the critical joints of JS1 that strike sub-parallel to the slope face. The probability of toppling failure is low however due to the lack of release surfaces. A gabion wall, a reinforced soil structure, was constructed in 1999 to create the footwall ramp and recover extra ore. The highwall below had to be steepened to accommodate this and this area is the highest risk. Monitoring of the gabion wall has shown that it is stable. In the event of failure, access into the pit will continue as there are two hangingwall ramps.

Zone 2 represents the southern highwall in the south pit which hosts the calc-silicate footwall, blebs of parapyroxenite and the pyroxenite orebody. It is faulted, and small-scale failure is evident on the ramps. Wedge failure, on JS1 and JS3 joints, is the dominant failure mechanism though minor circular failure has occurred in the past at shallow depths in the weathered highwall.

Zones 3 and **4** cover the western highwall in the south and central pit and consist of fairly homogeneous competent gabbronorites cross-cut by the fault zone described in Chapter 3. Seven stack failures have occurred there in the past 3 years as the final cutback was excavated. A permanent ramp running north-south is being developed in this zone which will link the north and south pits and provide access to the pit from the west. Planar and stepped path failure in this zone will always be a problem and the highwalls are closely monitored.

Zone 5 delineates the northernmost section of the western highwall which has slightly different geotechnical properties to the rest of the western highwall. It displays fracture cleavage related to a number of shallow faults that cross-cut the zone and the area to the north thus the highwall is more fragmented and susceptible to minor failures. The critical joints associated with JS1 dip slightly more steeply (75°) and more to the south

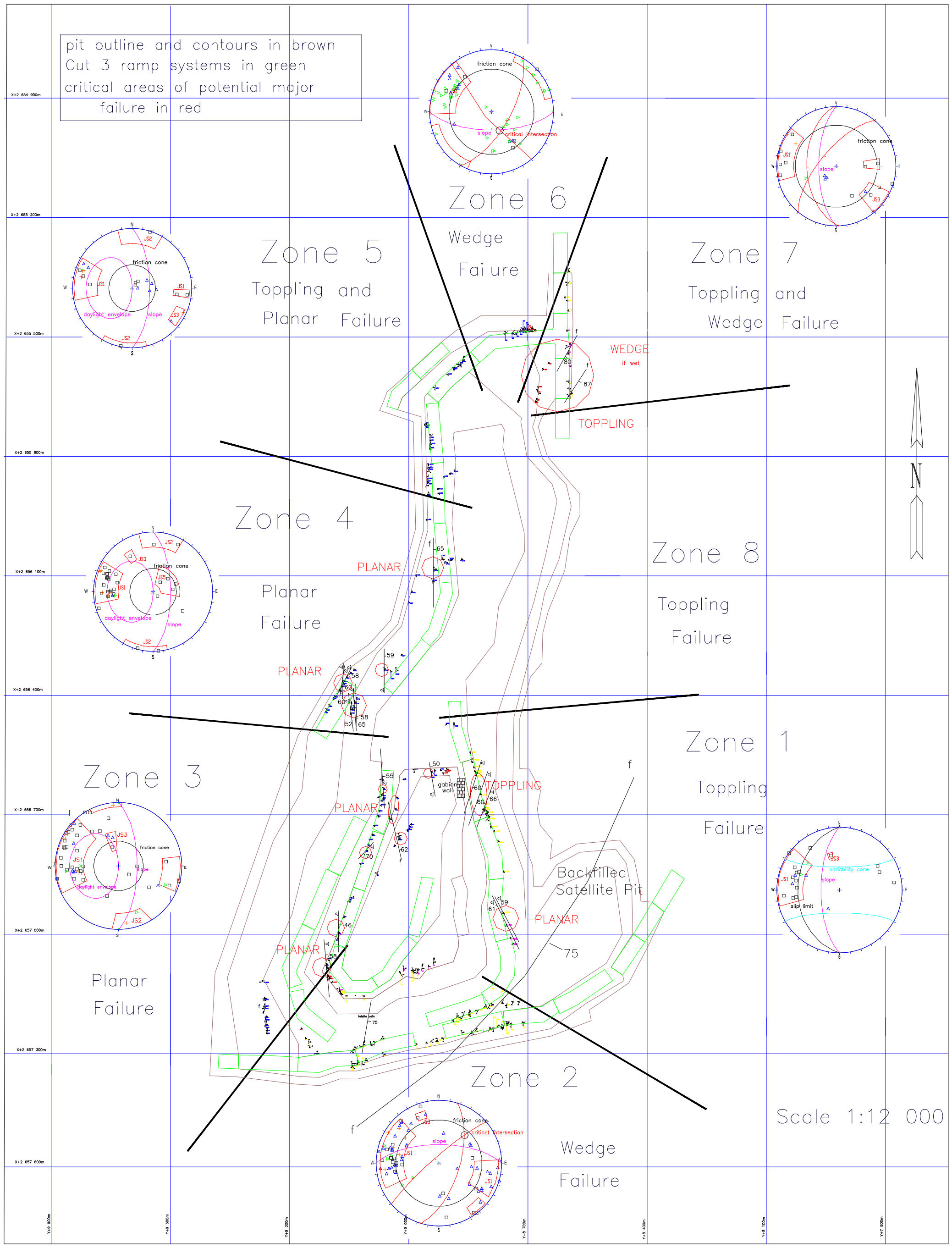


Figure 6.4 Geotechnical zones for Sandsloot open pit

(108°) than elsewhere on the western highwall. The permanent ramp along the highwall is at a shallow depth and poses no major slope failure problem however rockfall hazard is high due to large boulders in the weathered crest. In 2005, over 600 m^2 of wire meshing was installed to significantly reduce this danger. This is discussed further in Chapter 6.

Zone 6 delineates the northern highwall which exposes Platreef pyroxenites that are highly faulted and jointed. Eight faults can be subdivided into two sets though both generally strike NE-SW. Both fault sets display fracture cleavage and have many closely spaced joints, which increase in density near to the faults. This has resulted in the creation and failure of many small fragments, with wedge failure being the only mechanism present. The fault zones provide a path for water flow and thus this zone is a wet area. The permanent ramp along the highwall is at a shallow depth and poses no major slope failure problem however rockfall hazard is high due to large boulders in the weathered zone. As with Zone 5, wire meshing has been installed to significantly reduce this danger.

Zone 7 contains interfingered calc-silicate and parapyroxenite which were recently found to contain low grade. The pit design was altered to flatten the slope angle to gain extra ore in the footwall. Toppling, planar and wedge failure could occur, however, only wedge failure is predicted on lower benches.

6.4.2 Zwartfontein South geotechnical zones

There are 12 geotechnical zones for Cut 4 in Zwartfontein South (Figure 6.6) which is structurally and lithologically more complex than Sandsloot. As described in Chapter 3, line surveys, window mapping, orientated drilling and SiroVision digital photogrammetry have been done in Zwartfontein South since it started in 2002. Based on mapping, visual inspections of the pit slopes and the slope orientation, the pit was divided into 12 zones. The structural data was collated for each zone and plotted on stereonets in DIPS. Joint sets and failure mechanisms for each zone were identified in DIPS and RocPlane was used to calculate FOS for the joint sets in each zone. Rock mass ratings were also calculated and modelled in Datamine to confirm the geotechnical zone boundaries.

Zones 1, 3 and 4 on the western wall, are fairly competent norite (Figure 6.9) with planar failure identified as the dominant potential failure mechanism. Only one double bench planar failure has occurred on a low angle joint in Zone 6. The remaining failures are less than 5 m in height and fail out during blasting thus do not pose a safety risk. There is potential for wedge failure but it should be on a small scale. JS1 is the dominant joint set.

Zone 2 delineates the west wall norites intersected by a shear zone that strikes NE-SW across the pit. It has a joint spacing of 10 cm, a MRMR of 42 and is a carrier of groundwater. The norites experience minor wedge failures.

Zone 5 is on the north-west wall of the pit and consists of competent norites. There is a change in the dominant joint set from the rest of the west wall from the JS1 joints which strike N-S, to the JS3 joints which strike NE-SW. The pit slope also changes strike to NE-SW so the joints are sub-parallel and planar failure is the expected failure mechanism. There is still the possibility of wedge and toppling failure.

Zone 6 covers the orebody pyroxenites on the north wall. Wedge failure is the dominant failure mechanism but this only occurs on a very small scale due to the high fracture frequency in the rock mass.

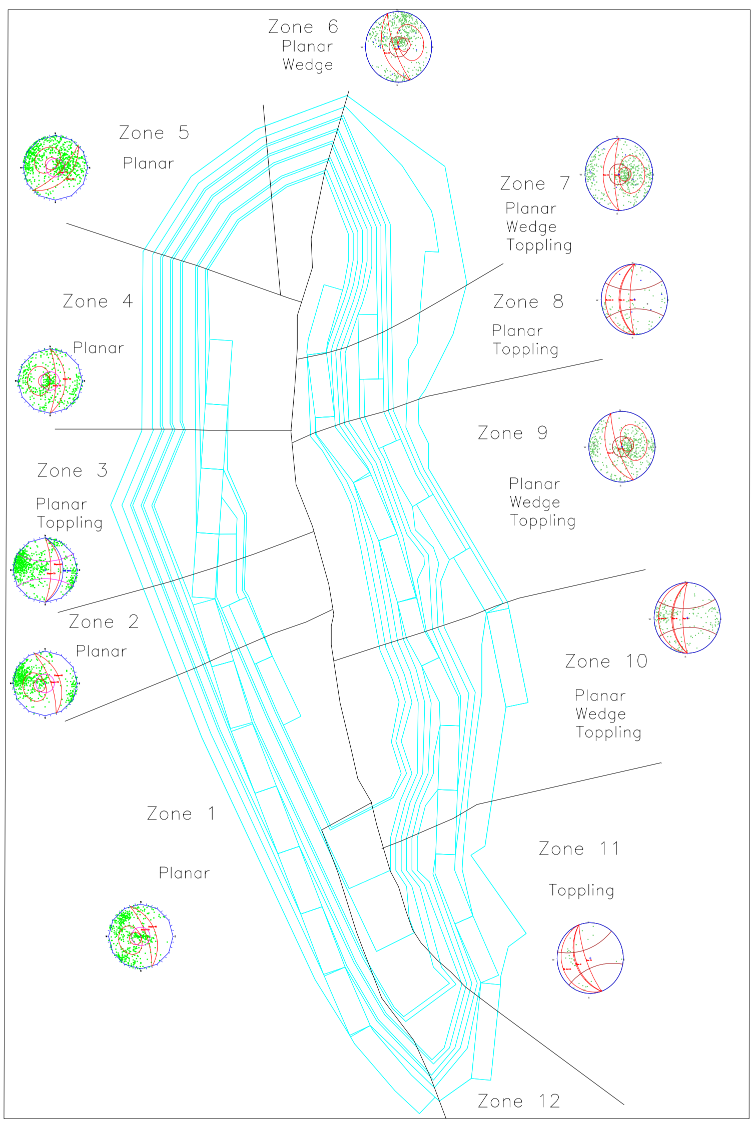


Figure 6.5 Geotechnical zones for Zwartfontein South open pit

Zones 7, 9 and 10 are footwall parapyroxenites on the east wall. Planar, wedge and toppling failure, are possible. Only one double bench planar failure has occurred since the pit started.

Zone 8 delineates the east wall parapyroxenites intersected by a shear zone that strikes NE-SW across the pit. It has a joint spacing of 10 cm, a mining rock mass rating of 42 (according to Laubscher) and is a carrier of groundwater. The parapyroxenites experience minor wedge failures.

Zone 11 delineates the calc-silicate xenolith that is found in the south-east of the pit. The rock mass is a lot weaker in this zone as calc-silicates have lower UCS and relict bedding planes cross-cut the entire wall with a spacing of 20 cm.

Zone 12 covers the orebody pyroxenites on the south wall. Wedge failure is the dominant failure mechanism but this only occurs on a very small scale due to the high fracture frequency in the rock mass.

6.5 Slope Instability at PPRust

The main slope stability concern at PPRust is stack failure on the western wall in Sandsloot open pit. The final cutback is being mined therefore long-term stability is at stake. The failures cannot be designed out as doing so is not economically viable. These failures are the result of the intersection of a large-scale fault zone with the entire west wall, shown in Figure 6.6 (Little, 2005). The 100 m wide fault zone hosts many faults spaced 5 m apart and joints spaced 20-50 cm apart which dip out of the face at roughly 55°. There is a large range in dip angles within the fault zone however which manifest as an imbricate fan of joints in many places. This makes the location of a particular fault or joint difficult to determine. The stereonet in Figure 6.7 and the rosette plot in Figure 6.8 show the dominance of the fault zone on the west wall as well as the range in dip and dip direction.

The fault zone runs parallel to the final west wall causing planar or stepped path failures where structures daylight. Bench scale planar failures occur during blasting on most benches on Cut 6 as back break of about 2 - 5 m follows the JS1 planes. The material is loaded out by the face shovels and the crests are cleaned using a scaling rig and/or backhoe. This generally reduces risk of rockfall though catchment berms are reduced from the designed 10 m to less than 8 m for 15 m benches. The risk of slope failure occurs where solid blocks of norite are frozen on the face, held up by rock bridge above an undercut toe. The failure mechanism is a combination of loss of cohesion on these planes and shearing through the rock bridge as a result of blasting. The failures are rapid brittle failures that occur in less than two hours and are of the order of tens to a few thousand tonnes.

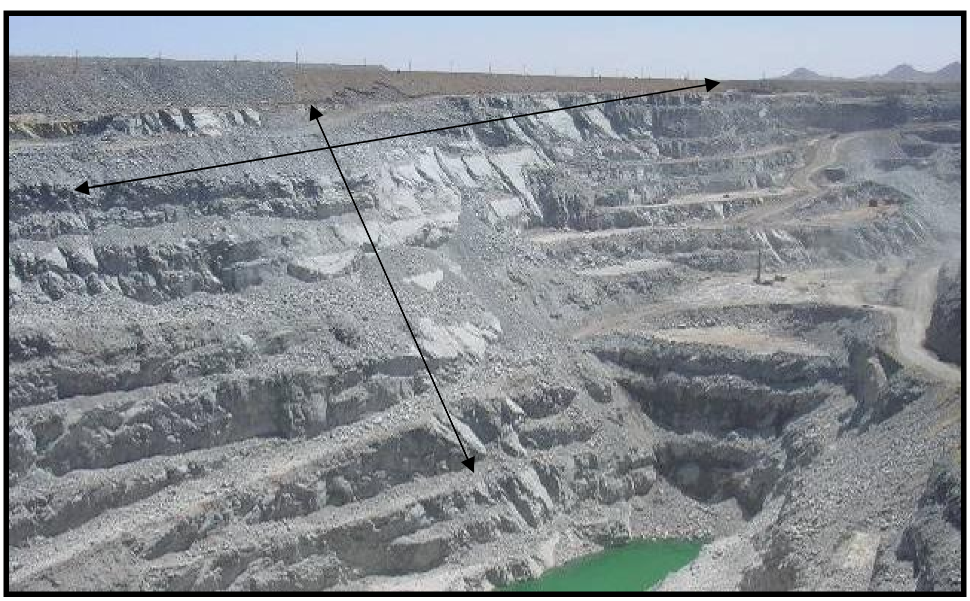


Figure 6.6 Photograph (looking NW) of fault zones cross-cutting the west wall of Sandsloot open pit.

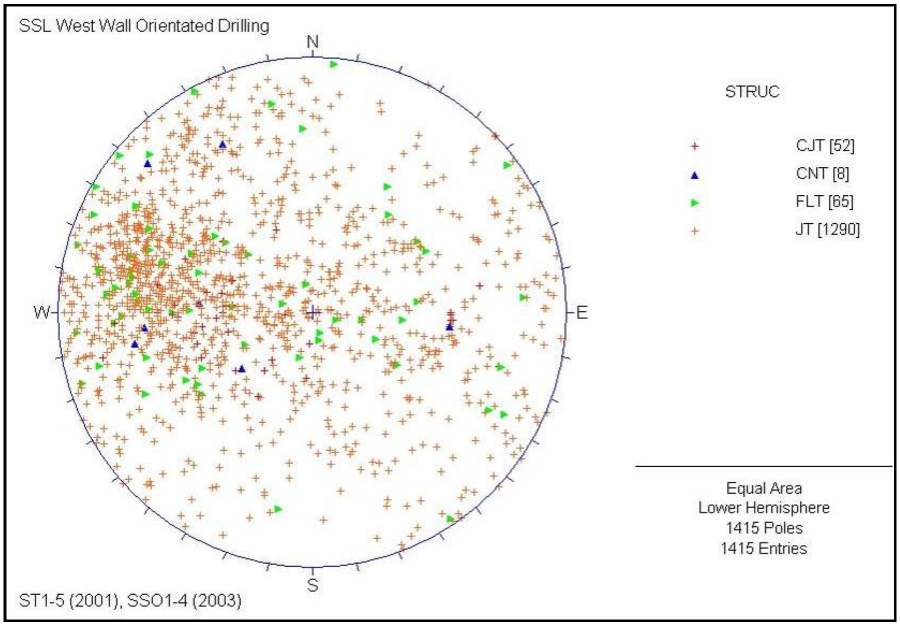


Figure 6.7 Stereonet of orientated core data from the Sandsloot west wall

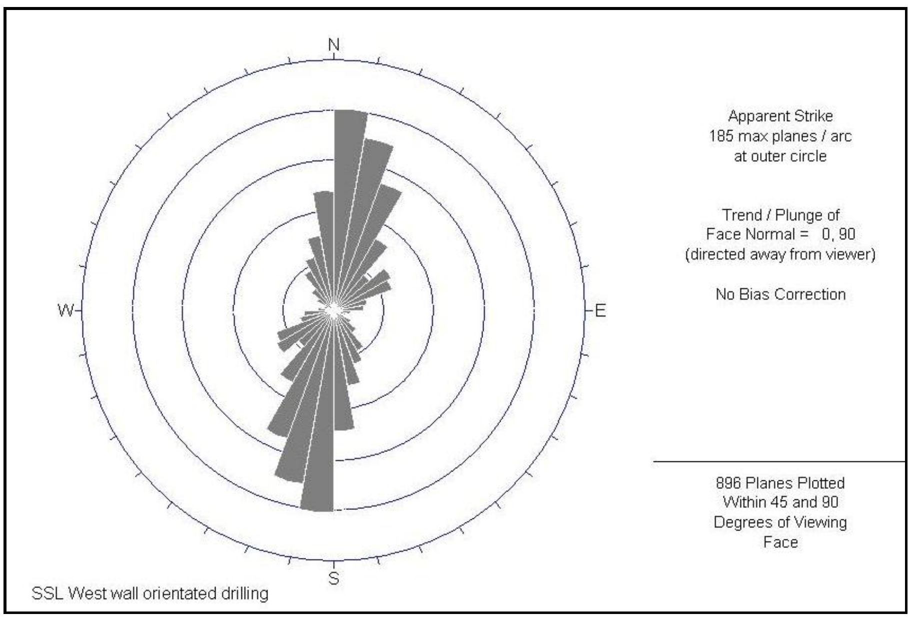


Figure 6.8 Rosette plot of orientated core data from the Sandsloot west wall

It is imperative that the failure mechanism is understood to deal with these failures properly. The joints and faults in the fault zones clearly provide the failure plane but jointing perpendicular to this plane (JS2: dip/dip direction roughly 85/183) provides release surfaces for failure. The JS2 joints are discontinuous however and for failure to occur the norite rock bridge between the release joints must shear through (Figure 6.9). This shearing can be induced by blasting, undercutting of the toe and groundwater pressure. This will be discussed further in the case study.

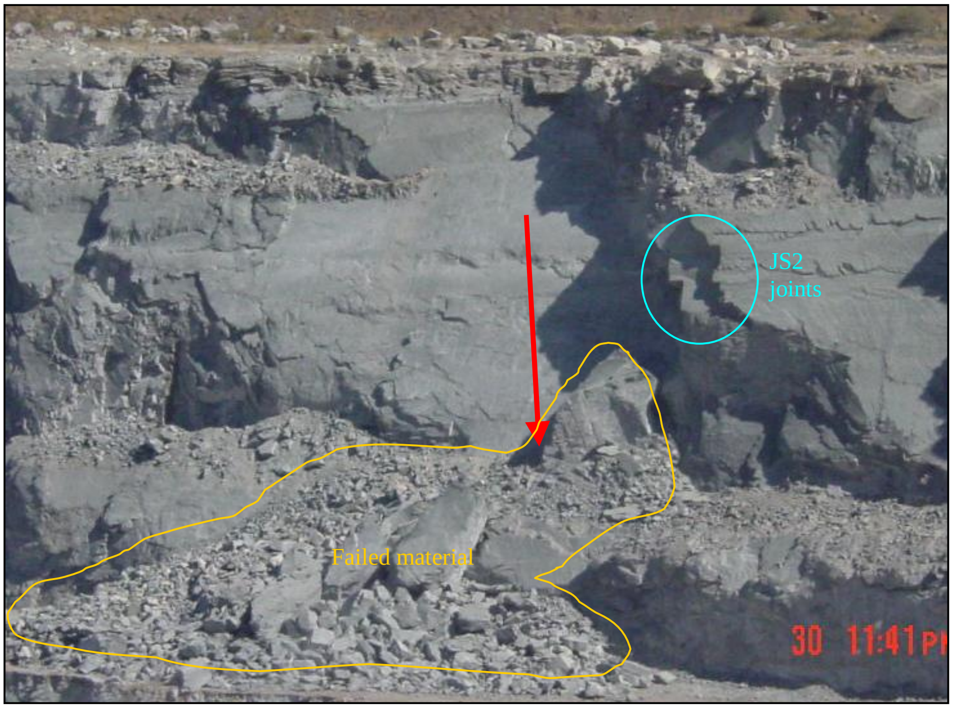


Figure 6.9 Photograph of a 30,000 t stack failure, showing minor discontinuous release surfaces of JS2

6.6 Bench Failure Case Study

In March 2004 a small planar failure occurred on Bench 11 (20 m below the weathered zone) in the northern region of the west wall in Sandsloot (Figure 6.10). Roughly 2 600 t of rock failed onto the bench below (Bench 14) leaving an overhang of ~14 500 t held up by rock bridge, which posed a serious safety hazard. Rainfall had occurred a few days prior to failure and a back-hoe had been at work cleaning the toe of the face. Water was draining out the failure plane at the time of failure and continues to do so, even during the dry winter season. The failure plane therefore must intersect the groundwater table. The joint along which the failure occurred was one of many intersecting the Bench 11 face though it cut the toe of the presplit. Failure had occurred on other joints above the failure plane during blasting thus removing the entire catchment berm. As evident in Figure 6.16, only a few presplit barrels are visible with most of the face sitting at 60° along the failure planes. The minor joint spacing was measured at 0.1-0.5 m and the spacing of major joints (or faults) was measured at 1-5 m. The failure plane itself was smooth undulating with a range in dip of 43° – 66° and range in dip direction of 084° – 127° and calcite, serpentinite and felsite infill and iron oxide staining.

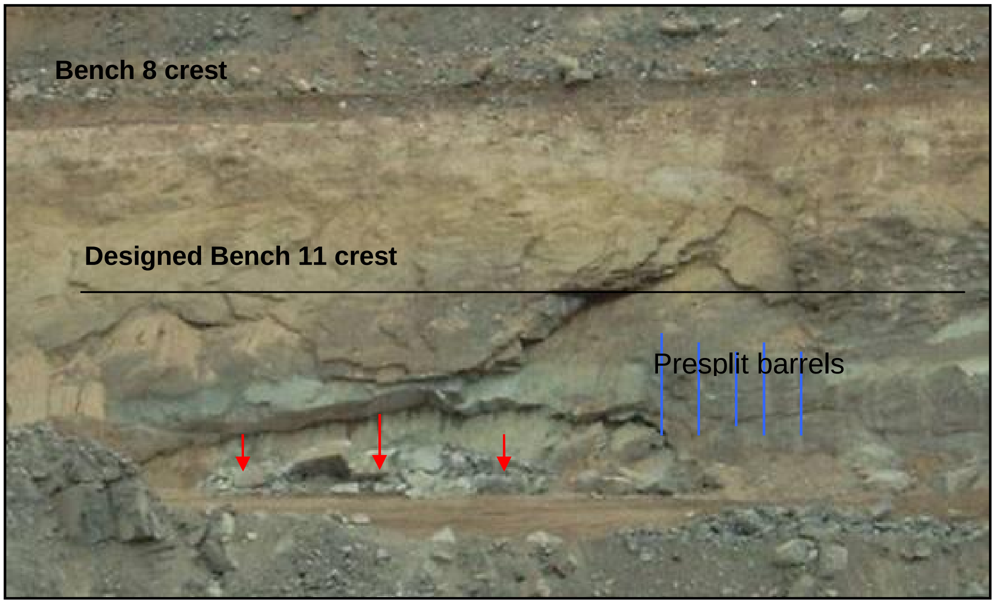


Figure 6.10 Photograph taken a few hours after the failure on the Bench 11 face.

Back analysis using Hoek and Bray's (1981) planar failure equation gave a cohesion of 116 kPa when using a wet friction angle of 25°. As the porosity of norite is 2% it was assumed that the rock was not saturated but that water seeped down along the joint. Thus instead of using the water pressure in the equation the wet friction angle of 25° was used instead of the dry value of 30°. The failure was not true planar failure however, rather stepped path failure, as there were no continuous release surfaces. Instead, the shear strength of the norite rock bridge (13 MPa) was overcome.

In August 2003, an orientated drilling programme had been undertaken to delineate the fault zones in the norite and identify where they would intersect the west wall. Diamond drill holes were studied as well to aid the analysis. One of the holes, SSO2, happened to pass 45 m metres north of the failure on Bench 11 and so the core was studied again with this in mind. The core is very competent, with RQD's above 95%, and the average Mining Rock Mass Rating (MRMR), using fracture frequency, (Laubscher, 1990) for the hole is 56. This gives a corresponding slope angle (Haines and Terbrugge, 1991) of 62° (Table 6.1 and Figure 6.11). These values correlate well with other MRMR data collected for Sandsloot west wall. At first glance the fault zone (found in Zone 3) is no different from the rest of the core (Figure 6.12) though by plotting the joint orientations on a stereonet together with the face mapping readings, the fault zone is clearly seen (Figure 6.13). When the SSO2 Zone 3 log is compared with the face mapping readings, the correlation becomes even clearer as evident in Table 6.2.

Table 6.1 Summary of Geotechnical log of SSO2

From	**To**	**Zone**	**Rock**	**RQD**	**FF**	**No. Joints**	**MRMR (FF)**	**Slope angle**
0	10.22	1	N	39	2.0	24	50	59
10.22	53.41	2	N	98	1.0	40	54	61
53.41	71.41	3	N	99	1.5	26	54	61
71.41	119.41	4	N	100	0.25	13	61	64
119.41	143.41	5	N	98	0.8	19	53	60
143.4	166.41	6	N	99	0.25	13	61	64
116.5	186.41	7	N	97	2.0	62	60	64

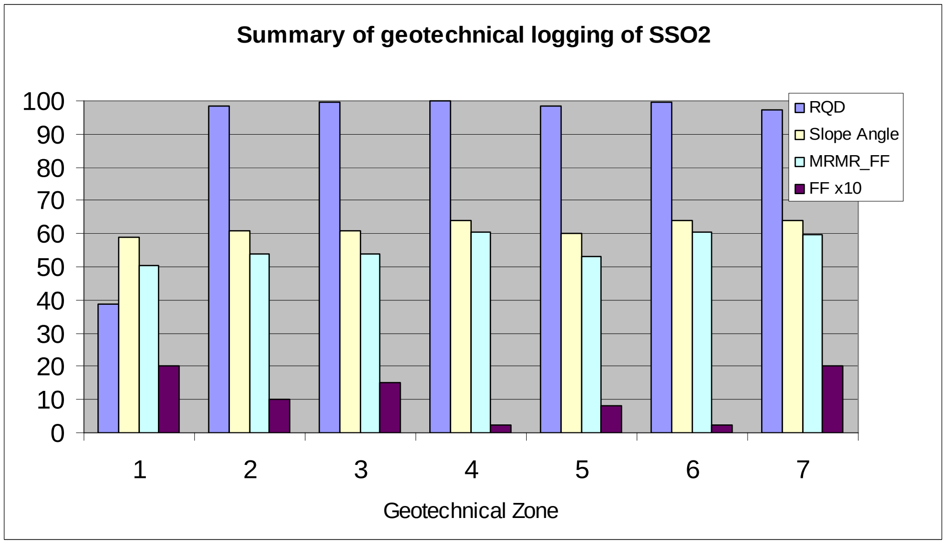


Figure 6.11 Summary bar chart of geotechnical log of SSO2

Figure 6.12 Photograph of the fault zone identified in the core (54 m – 64 m) which looks exactly the same as the rock outside the fault zone

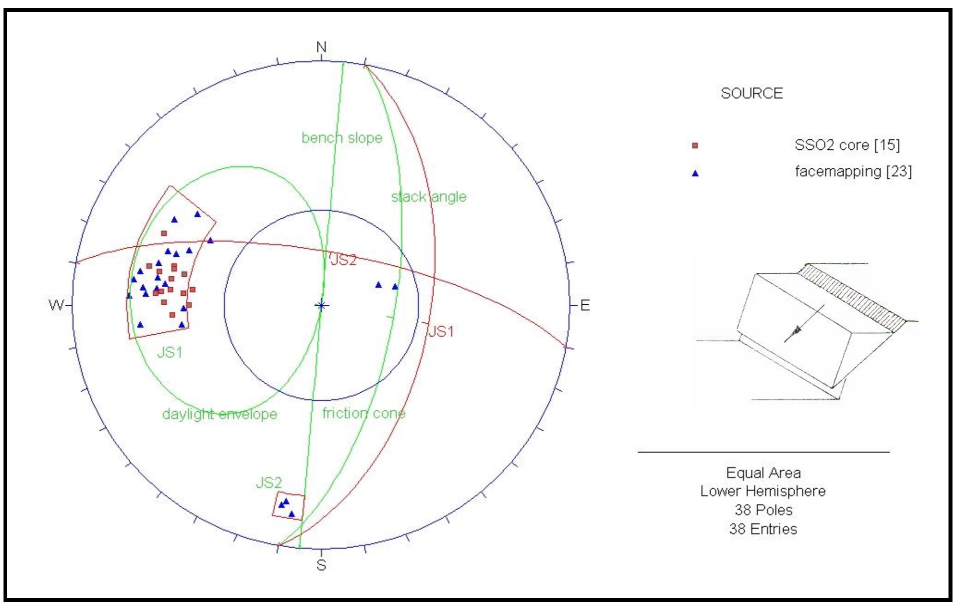


Figure 6.13 Stereonet plotted in DIPS of the face mapping on the failure and the fault zone identified in the SSO2 core

Table 6.2 Comparison of face mapping and core logging for failure

Parameters	**Face mapping of failure**	**SSO2 Zone 3 Core log**
RQD	100 %	99 %
Width of fault zone	5 m	10 m
Average joint spacing	0.5m	0.56m
Average dip/dip direction	56/100	52/098
Range in dip	43-66	43-60
Range in dip direction	084-127	086-115
Joint roughness	Smooth Undulating	Smooth undulating
Joint filling	Serpentinite, calcite, felsite, FeO staining	Serpentinite, calcite, felsite, FeO staining, mylonite

What can be concluded from this case study is that failure occurred within a fault zone that continues behind the face and into the face below. Also it is clear that only orientated core, not normal exploration core, can be used to identify and delineate these fault zones. The benches below Bench 11 are likely to experience similar failures therefore the failure plane was modelled and extrapolated in Datamine to the permanent hangingwall ramp on Bench 17 below the failure. Figure 6.14 shows how the failure plane will intersect the adjacent trim and ramp and enables the operations team to prepare for potential failures. One must remember that there are potential failure planes on either side of the failure plane. This fact, together with the undulating nature of the discontinuities in the fault zone, means that the exact failure plane on lower benches cannot be predicted.

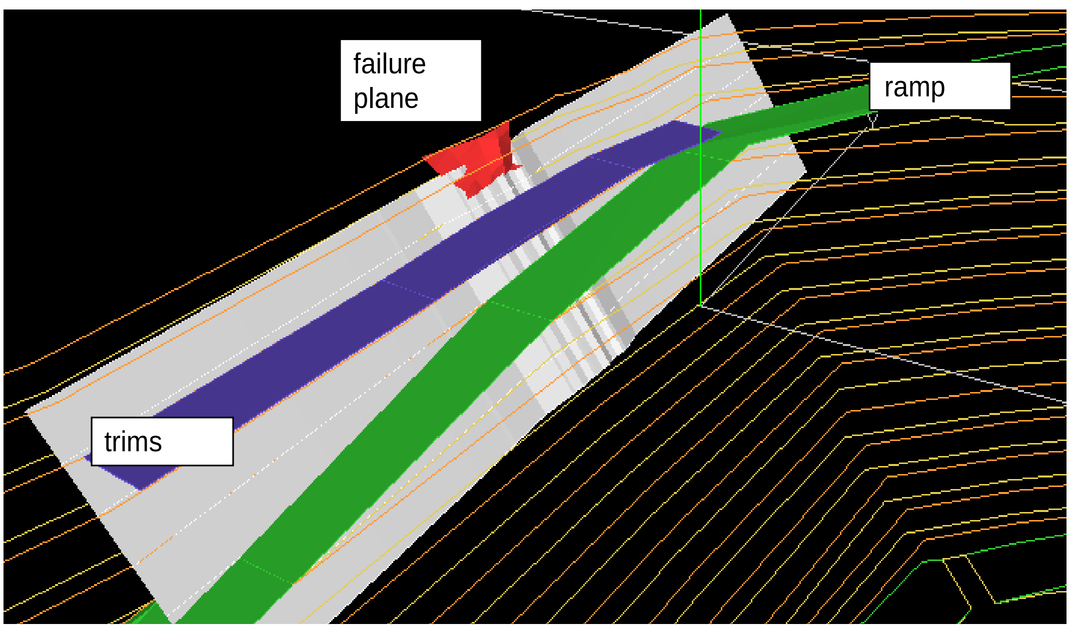


Figure 6.14 Failure plane on B11 extrapolated laterally and to the trims and ramp below

Another observation from this case study is that the many sealed joints or calcite veins seen in the core are open joints in the face, which is obviously a result of blasting. Even though the blasting techniques at PPRust are excellent, the design on the final west wall is often not achieved and failures still occur due to the nature of the joints and faults. This is because the energy from the blasts extends existing joints, progressively opens up healed joints and reduces rock bridge. Thus when the rock adjacent to a fault zone is removed, it fails out and the result is a 60° dipping bench along the failure planes as was evident in Figure 6.21. The energy from the blasts follows the joints as they are paths of least resistance. If you consider that one to two million tonnes of rock are blasted in the pit every week one can assume that the healed joints are being progressively opened up and the rock bridge is partially broken at each blast in the vicinity. Thus when the rock adjacent to a fault zone is removed, it fails out. The key, therefore, to understanding the progressive cohesion loss concept on the west wall in Sandsloot is to understand and quantify the effect blasting has on the highwalls.

6.7 Blast Damage

It is impossible to prevent blast damage to the highwalls but by understanding the impact of blasting on the walls though, this can be managed. Figure 6.15 diagrammatically shows the effect of blasting on adjacent rock on the western highwall at Sandsloot (Bye *et al.,* 2005). Three zones have been identified and are graphed in Figure 6.16.

Zone of rock cracking – blasthole pressures are higher than the UCS of the rock and therefore crack the rock (0 - 2 m from the presplit)

Zone of joint cracking – blasthole pressures are greater than the joint fill strength and thus open up the sealed joints (2 - 3 m from presplit but dependant on joint strength)

Zone of rock bridge – blasthole pressures crack rock bridge creating a partially fractured rock mass (3 - 20 m from presplit). It can result in overhangs and potential stack failures

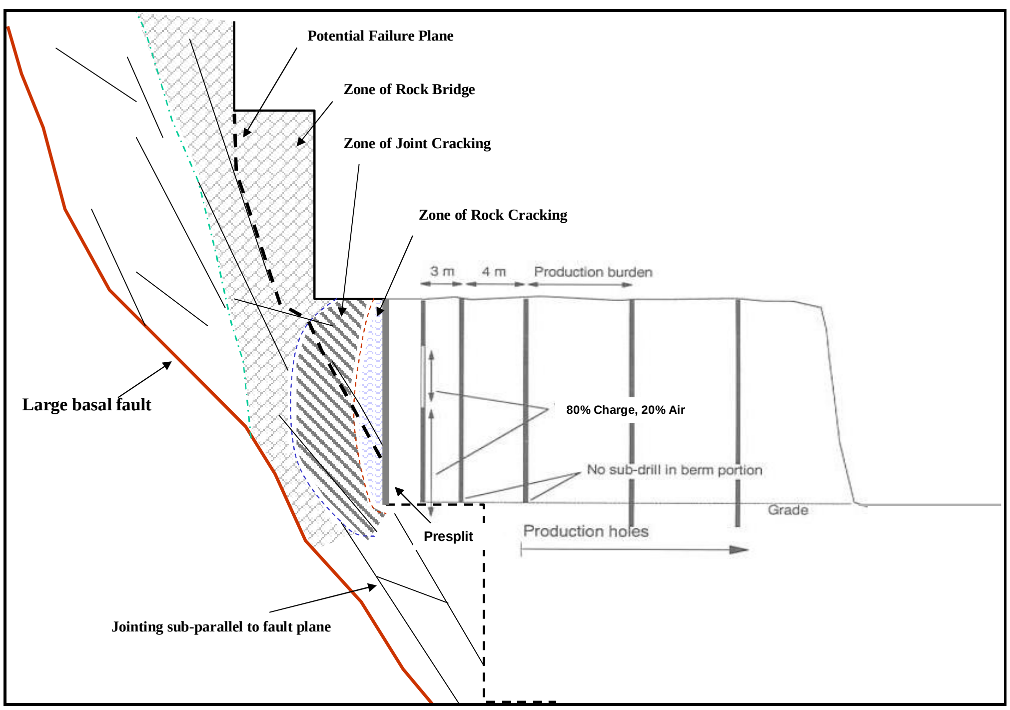


Figure 6.15 Sketch of the effect of blasting on the west wall in Sandsloot (Bye *et al.*, 2005)

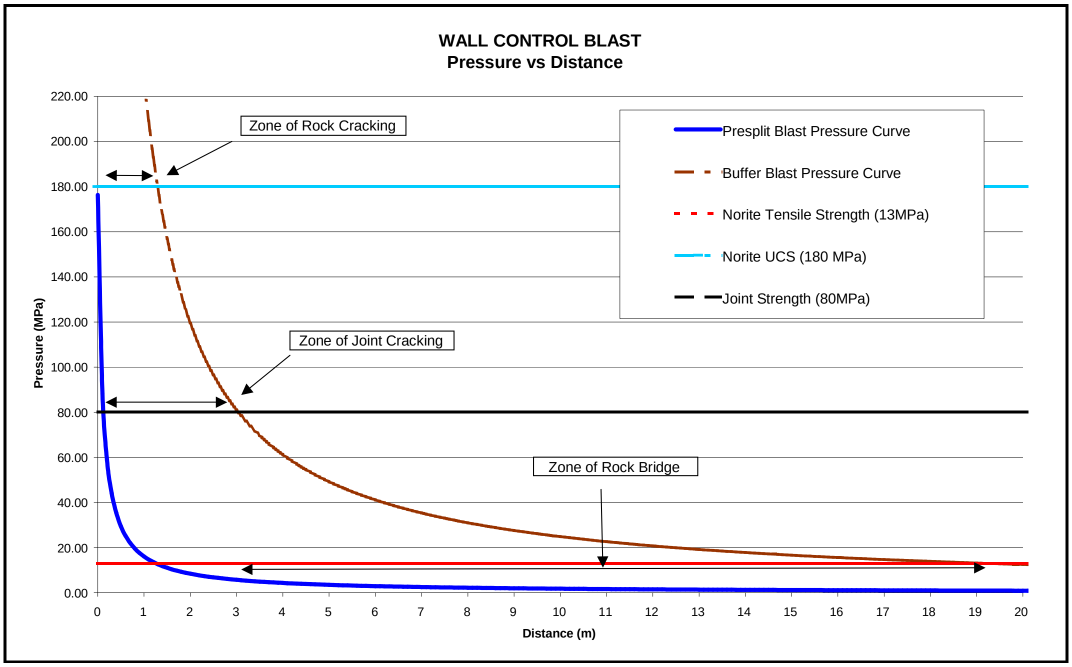


Figure 6.16 Assessment of blast damage zones which depend on distance from a blast (Bye *et al.*, 2005)

Figure 6.16 shows the drop in blasthole pressure with distance and the extent to which the rock is fractured. The graph plots the UCS of norite (180 MPa), the joint strength (80 MPa) and the tensile strength of norite (13 MPa) as thresholds which the blasthole pressure passes at certain distances from the blasthole. It indicates that a blast can badly damage the rock up to 20 m away. The changing peak particle velocities (PPV) of blast vibrations as they travel away from the blast (and into the highwalls) have also been calculated for various blasts. A blast on Bench 23 in Sandsloot pit caused stack failure 100 m away on the Bench 14 face and a PPV of only 46 mm/s was calculated. Another bench failure occurred when a PPV of only 30 mm/s travelled through the rock. To put that number in perspective, the PPV at the hole on detonation is 7000 m/s or $7x10^9$ mm/s. This analysis shows that not only the immediately adjacent blasts are affecting the joint cohesion and rock bridge. Of course even if it were possible to measure the blast vibrations that had travelled through a certain piece of rock over the life of the mine, one still could not say what the joint cohesion and rock bridge were like within the rock and whether it would fail. It does go a long way however to understanding the failure mechanism and predicting areas of instability.

6.8 Conclusions

Slope stability is well understood at PPRust and detailed geotechnical zoning of both Sandsloot and Zwartfontein pits has been done. The main slope stability concern at PPRust is the stepped path failures that occur on the west wall of Sandsloot pit. The failures cannot be designed out for economic reasons. Instead detailed analysis of these failures has been done to understand the failure mechanism and to predict future failures. This analysis includes investigating the blast damage and results showed that the highwalls are destabilised at least 20 m behind the face adjacent to a blast. Slope stability is a key factor when designing pit slopes which is the content of the next Chapter.

7 SLOPE DESIGN

"Scientists discover what is; engineers create what has never been."
- Theodore von Karman (1911)

This Chapter discusses design methodologies proposed by Bieniawski, Stacey and Steffen *et al.* and relates them to open pit mines and PPRust. It describes slope analysis techniques, such as numerical modelling, and details the slope design history at PPRust.

7.1 Introduction

The purpose of collecting, modelling and analysing geotechnical data, as described in detail in Chapters 4, 5 and 6, is to design an open pit slope that fulfils both economic and safety requirements determined by mine management. The amount of work that goes into a slope design depends on the stage of the operation – a pre-feasibility study will require a simple slope design using a few orientated drillholes and some lab testing. The design will be conservative, with relatively flat slope angles which are used in the economic calculations. An intermediate cutback design will have large amounts of core logging, face mapping and rock testing information and the slopes will be relatively steep to maximise the net present value (NPV) of the operation. The final pit design, however, will use the most geotechnical information and analysis yet have slightly flatter slopes than the intermediate cutbacks as these slopes will become permanent structures and the long-term stability is very important. So a pit design is not only based on the geotechnical properties of the rock but the mining method, lifespan, human exposure and NPV. As Hoek (2001) says 'A good engineering design is a balanced design in which all the factors which interact, even those which cannot be quantified are taken into account. The duty of the design engineer is not to compute accurately but to judge soundly.'

The architecture of an open pit slope design is described by bench height and angle, stack (or inter-ramp) height and angle, overall slope height and angle, catchment berm width, ramp width and ramp location. A bench is a single mining cut on a certain elevation. Two benches are separated by a catchment berm or step-off. These two terms are used interchangeably and can also be used to describe a single wider section within the design. A stack is a number of benches, often referred to as the section between two ramps. The overall slope is the entire height of the excavation for a cutback. The design can also be said to incorporate drainage and depressurisation measures, rockfall protection fences, reinforcement or support. Reinforcement and support include rock bolts and mesh, drape fences, shotcrete, pre- and post-tensioned cables, shear pins and buttresses.

Geotechnical engineering is a fascinating field as it integrates science and engineering. Knowledge of the science of geology is important for understanding a rock mass – how it formed and reacts. The ability to turn a rock mass into an open pit mine, however, requires the practical skills of an engineer. As Bieniawski (1991) puts it "It is design which makes engineers out of applied scientists." Slope design is a particularly difficult form of engineering design as the rock mass is a complex, heterogeneous material which can fail in a number of ways that are often hard to predict. In other engineering design, the specific material properties are prescribed to meet the design needs. A rock mass cannot be changed, except with support, to suit the ideal design scenario. Instead the design must suit the rock mass. Also, failure will never be completely absent from design as there will always be an element of uncertainty (or risk) in the strength of a rock mass and the location of geological structures. Bieniawski (1993) comments that this risk is often disguised as a 'safety factor' which more appropriately should be termed 'a factor of ignorance' but risking and accepting failure as part of the design and construction process is not only natural in engineering but is also good.

Other variables that affect open pit slope design are the metal prices and exchange rates which have massive effects on the economic success of a slope design. Political circumstances and government policy can affect the strategy of a mining company. New technology can improve risk management and can change the mining method. Staff turnover can have both negative and positive impacts on a mine. Each of these factors carries a degree of risk and need to be taken into account when designing pit slopes.

7.2 Design Methodology

In 1987 the Accreditation Board for Engineering and Technology (ABET) described engineering design as "the process of devising a system, component or process to met desired needs. It is a decision-making process (often iterative), in which the basic sciences, mathematics, and engineering sciences are applied to convert resources optimally to meet a stated objective. Among the fundamental elements of the design process are the establishment of objectives and criteria, synthesis, analysis, construction, testing and evaluation. In addition, sociological, economic, aesthetic, legal and ethical considerations need to be included in the design process."

7.2.1 Bieniawski's System Design Methodology

In 1991 Bieniawski proposed the System Design Methodology (SDM) to deal with the complexities of rock engineering design. SDM is a systematic decision-making process which can be seen as a check-list for the designer. It was developed using work by Suh (1990) as a basis. Suh had two axioms: the Independence Axiom and Information axiom, stating that best design is a number of independent components satisfied by the simplest solution. These two axioms gave the design process scientific basis and paved the way for further design principles such as Bieniawski's SDM. Suh's two axioms are necessary but not sufficient for rock engineering due to the complex nature of rock masses.

Bieniawski's SDM is composed of six design principles:

1. <u>Independence Principle</u>: There exists a minimum set of independent functional requirements that completely characterize the design objectives
2. <u>Minimum Uncertainty Principle</u>: The best design is one which poses the least uncertainty concerning geological conditions
3. <u>Simplicity Principle</u>: The complexity of any design solution can be minimized by creating the fewest number of design components forming a part of the design solution and corresponding to the appropriate functional requirement. In this way, the design objectives are uniquely satisfied in terms of the problem definition.
4. <u>State-of-the-Art Principle</u>: The best design maximises technology transfer of the findings derived from state-of-the-art research and best practice.

5. Optimisation Principle: The best design is optimal design which is evolved from quantitative evaluation of alternative designs based on the optimisation theory, including cost effectiveness considerations.
6. Constructability Principle: The best design facilitated the most efficient construction of the rock engineering solution with the components of the design solution being implemented by the most efficient construction procedures.

In 1993, Bieniawski used the six design principles to formulate a comprehensive ten-step design process, shown in Figure 7.1. These ten steps enable the engineer to complete the entire design process from the initial statement of the problem through to the implementation of the design. Functional requirements and constraints give the design independence. Data collection must be done to meet the minimum uncertainty principle. A design concept is determined to ensure simplicity of design. Analysis is done and alternatives are proposed, taking the state-of-the-art principle into account. Evaluation and optimisation lead to a final recommendation which is then implemented. This is a linear process and works well in civil engineering applications which usually result in a once-off design and construction. For an open pit mine that operates for a number of years in a volatile economic environment, this linear approach has its limitations. Many of the steps would need to be revisited to maintain an optimal slope design. The requirements and constraints to meet economic and safety requirements will change over time. Information should be collected on a regular basis which should prompt further analysis and more alternatives. Ongoing evaluation and optimisation promotes an efficient and cost-effective mine. Lastly, there is much scope for implementing new designs for different cutbacks but there is the flexibility of changing a slope design as it is being mined.

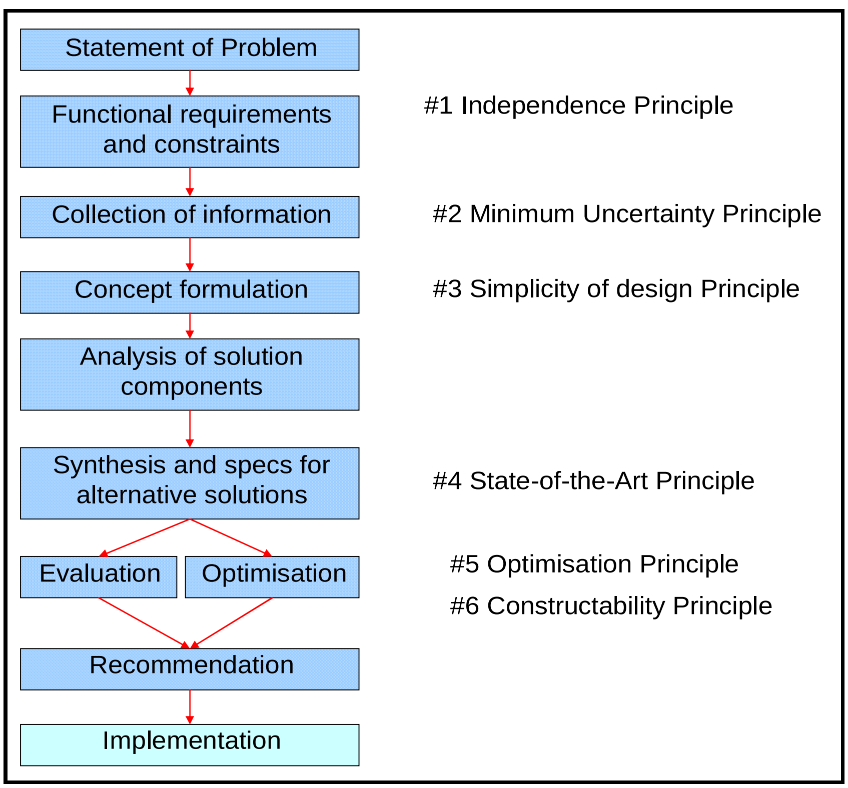


Figure 7.1 Bieniawski's (1993) design methodology and SDM

7.2.2 Stacey's engineering circle of design

Stacey (2006) brings the geotechnical engineer's attention to a business model that applies remarkably well to the design process. In their book 'Games foxes play – planning for extraordinary times' the authors Ilbury and Sunter (2005) put forward a 'strategic conversation' methodology. They present a ten step circular process that they utilise to aid businesses in being more effective and successful in the current working environment. As illustrated in Figure 7.2, this process is divided into two phases, 'defining the game' and playing the game' which can be called the strategic and tactical phases. The management team of the business would run through all ten steps starting with the scope of their business through to the meaning of winning, or being successful. The players, rules, uncertainties and scenarios all define the specific working environment. A SWOT analysis leads to a choice of options from which a decision must be made that will realise the required measurable outcomes.

Figure 7.2 Ilbury and Sunter's (2005) strategic conversation methodology

Stacey compared this 'strategic conversation' model to Bieniawski's SDM and proposed an engineering circle of design that combines the two, as shown in Figure 7.3. Stacey believes that this circular method is more effective since the linear approach can result in individuals leading the process in the wrong direction. The key to the circular process is that 'a conclusion reached later on in the conversation can lead to a review of earlier material'. In open pit geotechnics, the game is the slope design and the phases are defining and executing the design. The first phase is crucial as it will determine the success of the second phase. The execution phase is obviously also very important as it ensures that the defined design is still relevant.

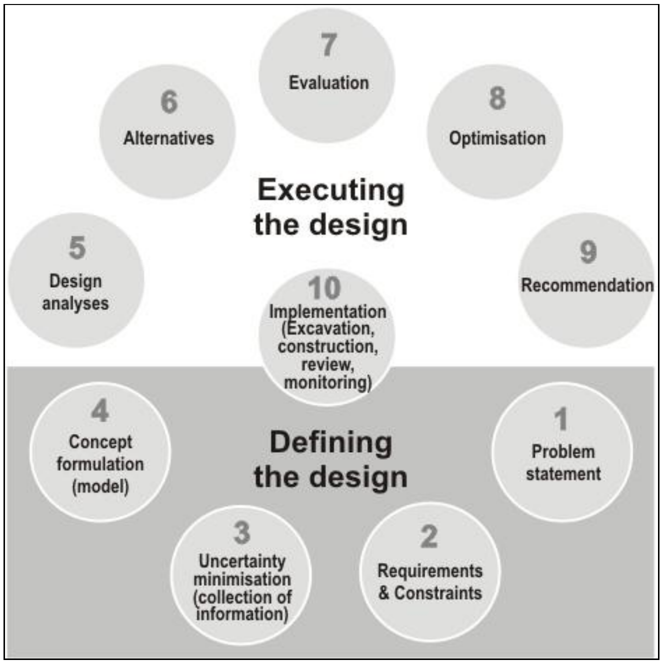


Figure 7.3 Stacey's (2006) engineering circle of design

7.2.3 Steffen *et al.*'s risk consequence approach

In the last few years a new mine design approach has been introduced by Steffen *et al.* (2006). In the past, the slope design process has been limited to the geotechnical engineers – highly technical and usually risk averse people. Mining is a high risk business, however, and usually the slope angle – determined by the geotechnical engineers – plays a major role in the economic calculations. Therefore the slope design often is more conservative than the mine management would choose. Management are not included in the design process, however, and do not even know what their geotechnical risk profile is. Steffen *et al.*'s (2006) risk consequence approach to open pit slope design addresses these issues. The geotechnical risk is one aspect of total mine planning that has risk and it must be seen as a part of the whole process. Geology, metal prices and exchange rates all have an element of unpredictability and thus all have associated risk. Steffen *et al.* (2006) say that in their experience the geological, metallurgical and mining systems input is at a much higher level of knowledge and confidence than the geotechnical input. To quantify the geotechnical risks involved in a mine Steffen *et al.* (2006) employ the fault and event tree methodology.

An event tree is a graphical framework that is used to estimate the probability of occurrence of an undesirable event such as a fatality, damage to equipment or economic loss (Anon., 1999). The framework, or fault tree, illustrates the sequence of events that lead to the undesirable event, and probabilities of occurrence are assigned to each event by a group of experts. This group would include the mining operation managers, geotechnical department staff and geotechnical consultants, and others. It is a powerful tool as it highlights where the problems are and how one can reduce the risk of an event as serious as a fatality. Its greatest asset is that it quantifies the geotechnical risk and enables management to make informed decisions on the level of risk the company is willing to operate at. Thus this method reverses the usual slope design process which begins with data collection and ends with a determination of risk. Instead, the acceptable risk is chosen by mine management and the slope design is tailored to meet that requirement. It is by nature an iterative approach and agrees with Stacey's design circle.

7.2.4 PPRust slope design approach

The risk consequence approach was adopted at PPRust in 2003 and the Anglo American acceptable risk level has been adopted. Figure 7.4 illustrates the process of slope design that is now used at PPRust. The definition of acceptable risk applies to all current and future cutbacks and open pits. Field data collection, as described in Chapter 4,forms the basis for design. The geotechnical data is modelled in Datamine, as discussed in Chapter 5, and is used to given a preliminary estimate of slope angles. The models highlight areas where the slopes are over- or under-designed and areas lacking sufficient data. This may lead to further data collection in those areas. Failure analysis is performed and is used to geotechnically zone the pits, as described in Chapter 6. A change in geotechnical zoning will affect the block models so they may be revisited. A number of options are reviewed from an economic and safety point of view and an initial slope design is chosen. Once mining of the designed slopes commences, slope management and slope monitoring are performed to evaluate the design and maintain safe working conditions. These will be discussed in detail in Chapters 8 and 9. Further data is collected, modelled and analysed on a regular basis. The combination of all of this work is used for slope optimisation (explained in Chapter 10) which flows back to slope design. This iterative process continues for the life of the mine and is reviewed regularly by Anglo Platinum, Anglo American and third party consultants.

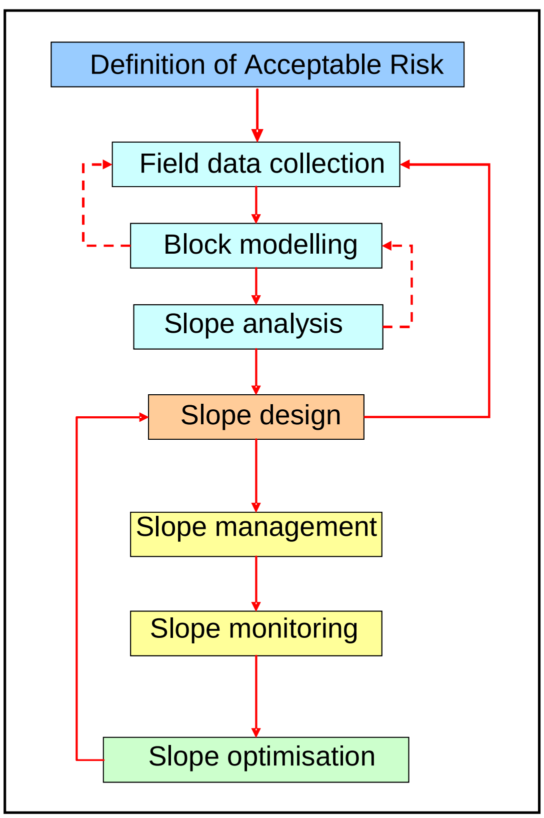


Figure 7.4 Flow diagram of the PPRust slope design process

7.3 PPRust Slope Design History

The thickness and tabular nature of the Platreef orebody are suited to open pit mining methods, which is considerably cheaper than conventional underground mining. The relatively shallow 45° dip of the orebody, however, results in a very high stripping ratio of between 6:1 and 10:1 at different stages of the operation. The waste stripping is kept to a minimum in the early stages of the mine to maximise the NPV so that capital expenses can be recovered as soon as possible. The open pits are mined in a series of cutbacks to facilitate this. Sandsloot open pit was chosen as the first pit of the six potential pits to be mined at PPRust as it contains the highest grade over a consistent strike length.

7.3.1 Sandsloot pit

Feasibility studies for the slope designs were performed during the 1970's and 1980's by Steffen, Robertson and Kirsten Consulting Engineers (SRK). The first reports were issued in 1980 and 1991 and they formed the basis of geotechnical design work for Sandsloot open. When mining commenced in 1992, six cutbacks were planned for Sandsloot with the east wall deepening over time and following the 45° dip of the orebody while the west wall would deepen and extend westwards. It is the west wall therefore which has been through a number of design changes over the years as shown in Figure 7.5 and Table 7.1. The initial slope was designed with a 45° west wall slope with 74° stacks of two 10 m 80° benches (Design 1). A 15 m catchment berm separated the stacks and a 2 m step-off separated the benches. In 1994, the 2 m step-off was taken out of the design as it increased rockfall risk and instead 20 m double benches (80° inclined) were mined (Design 2). As a result the stack angle steepened to 80° and the overall slope angle was increased to 48°. This was acceptable as the wall proved to be very competent. It also reduced waste stripping and thus increased the profit margin. As the scale of mining increased in 1996, the benches were increased to 15 m (Bye, Bell and Jermy, 1999) as this is more cost effective. The catchment berms were widened to 20 m and the 2 m step-off was re-introduced resulting in 76° stacks while maintaining the same overall slope angle (Design 3). Up to this point the designs had been done solely by SRK. In 1996 and 1997 a large amount of geotechnical data was collected by the mine geologists and it indicated that the west wall slope could be steepened to 51°. The stacks were increased in height to 45 m at 73.5° with a 23 m catchment berm between them (Design 4).

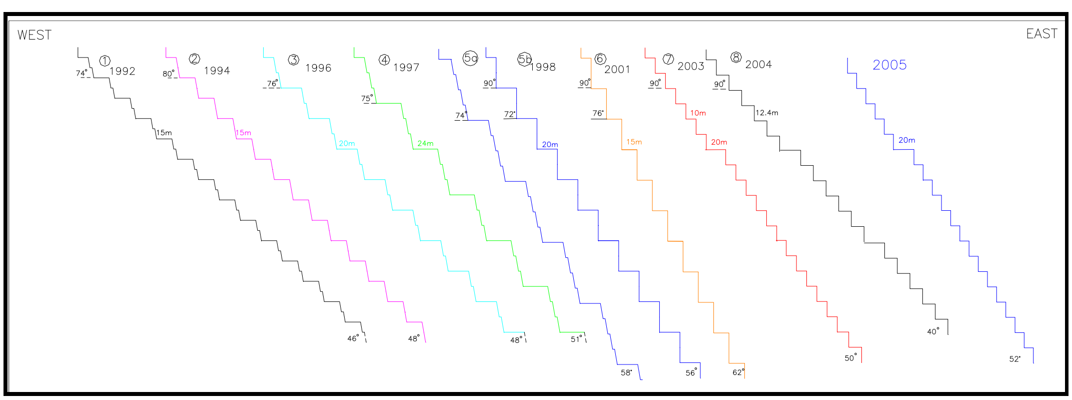


Figure 7.5 Slope design history for Sandsloot's west wall

Table 7.1 Slope design history for Sandsloot pit

Design	**1**	**2**	**3**	**4**	**5a**	**5b**	**6**	**7**	**8**	**9**
Year designed	1992	1994	1996	1997	1998	1998	2001	2003	2004	2005
Stack Height	20m	20m	30m	45m	60m	60m	60m	60m	60m	60m
Bench Height	10m	10m	15m	15m	15m	15m	15m	15m	15m	15m
Double Bench	-	20m	30m	-	-	30m	30m	-	-	-
Slope height	290m	290m	290m	290m	320m	320m	320m	320m	280m	300m
Slope angle	45°	48°	48°	51°	58°	58°	65°	50°	40°	52°
Stack angle	74°	80°	76°	73°	74.5°	71°	76°	61°	55°	65°
Bench angle	80°	80°	80°	80°	80°	90°	90°	90°	90°	90°
Catchment berm	15m	15m	20m	23m	20m	20.5m	15m	20m	30m	20m
Step-off	2m	0m	2m	2m	2m	0m	15m	10m	12.4m	8.7m
Ramp width	20m	20m	30m	30m	30m	30m	30m	30m	45m	30m

In 1998, SRK performed 2D FLAC analyses to optimise the bench, stack and slope angles (SRK, 1998). Ubiquitous joint model runs, taking the critical joints on the west wall in Sandsloot, were done with variations in cohesion (0 – 300 kPa) and critical joint angle (50° - 65°). From the analysis a stability envelope for Sandsloot was developed and an empirical design chart (Figure 7.6). The FLAC models also proved that the overall slope angle for the 300 m final slope could be steepened from 51° to 58° which translated into a saving in waste stripping costs of $28 million. It also extended the life of the mine by deepening the pit by 30 m (two benches) which would result in $100 million in additional profit (Bye and Bell, 2001). The stack design was first changed from three to four 15 m inclined benches with a 20 m catchment berm separating the 60 m stacks (Design 5a). This 60 m stack was then changed to two vertical 30 m benches with a 20.5 m catchment berm (Design 5b) to reduce the risk of bench scale planar failure on the west wall. Also, angled presplitting was proving too difficult with the planar joints. The wide berms allowed for easy access and drilling space and were incorporated into the ramp design so that the slope angle did not have to be reduced. The 30 m double benches could be presplit in one blast which saved time and money and improved the quality of the drilling.

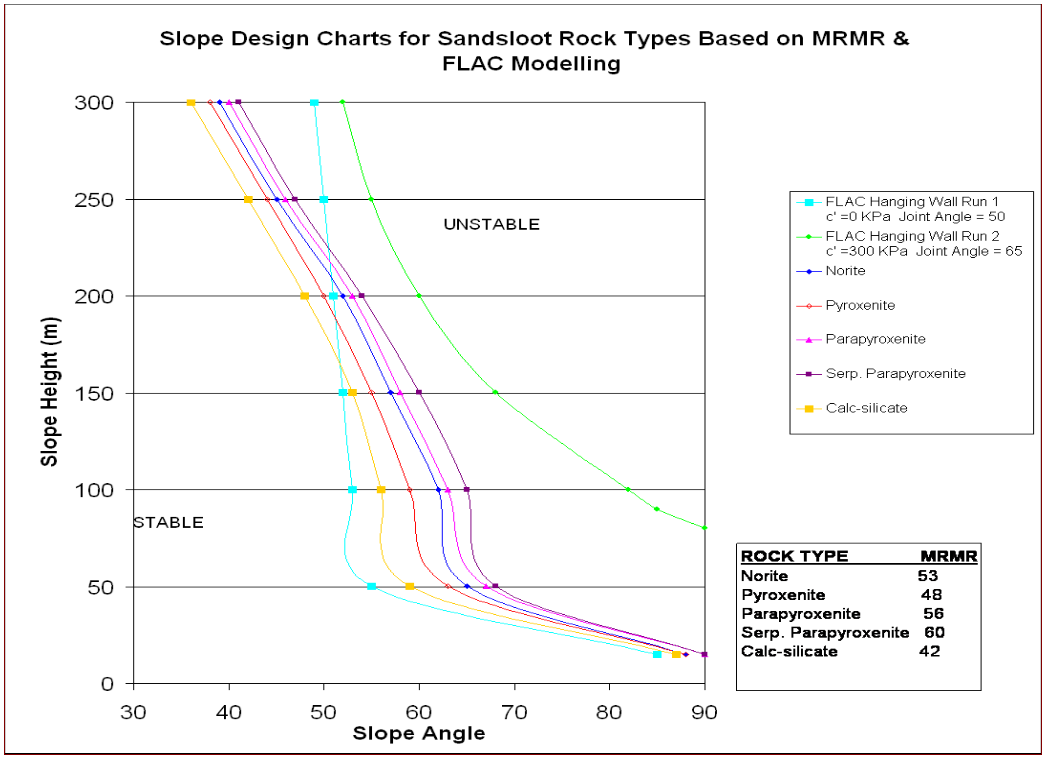


Figure 7.6 FLAC analysis for Sandsloot west wall slope optimisation

Sandsloot's first cutbacks, Cut 1 and Cut 2, were successfully mined with nothing more than minor bench scale failures and the full extent of the fault zone on the west wall was not realised. The west wall was therefore steepened for Cut 3, an interim cutback reaching a depth of 260 m, to maximise the NPV. This design, with a 60 m stack at 76° (Design 6) was also successfully mined with only minor bench failure along JS1 joints. In 2003 mining on Cut 4 commenced, following the same design as Cut 3 as it was also an interim cutback. It was on this cutback that the major fault zone was intersected and a 30 000 t failure occurred in June 2003. This led to a flattening of the slope angle for the cutback and a review of the final slope which would be mined in two cutbacks, Cut 5 and Cut 6. The slope design was changed in September 2003 and included a second ramp on the west wall. This extra ramp flattened the slope and provided an alternative access should a major failure occur in the future. The resultant overall slope angle was 50° with four single 15 m benches in a 61° stack (Design 7). In 2004, a number of bench and stack failures occurred on the west wall and the slope was flattened again to 42° (Design 8) until further slope analysis could be done. Slope optimisation was performed in 2005 and resulted in the steepening of the final slope angle to 52°. This is discussed in more detail in Chapter 10.

7.3.2 Zwartfontein South pit

As with Sandsloot, the east wall of Zwartfontein South follows the orebody at roughly 45° and generally deepens over the life of mine (Figure 7.7). The west wall extends westwards and to greater depths with each cutback. Zwartfontein South is designed conservatively, as evident in Table 8.2, due to the highly faulted and jointed nature of the rock mass. This negatively affects slope stability as well as grade control. The benches are 10 m high and initially had 10 m catchment berms. This was changed to alternating 3 m and 12 m step-offs and a 20 m catchment berm halfway down the slope.

Zwartfontein South is currently being mined on the Cut 4 design which has a final depth of 120 m and a slope angle of 42°. Slope optimisation is currently underway to steepen the slopes and initial work indicates that they can be mined at 52°. This work is discussed in Chapter 10.

Table 7.2 Zwartfontein South slope designs

Cutback	**1**	**2**	**3**	**4**	**5**
Year mined	2002	2003	2004	2005	2007-
Stack Height	80m	80m	80m	80m	60m
Bench Height	10m	10m	10m	10m	10m
Slope height	90m	120m	120m	120m	300m
Slope angle	45°	45°	45°	42°	45°
Stack angle	56°	56°	56°	5°	50°
Bench angle	90°	90°	90°	90°	90°
Catchment berm	20m	20m	20m	20m	20m
Step-off	10m	10m	3m/12m	8m	8m
Ramp width	30m	30m	30m	30m	30m
Slope length	400m	650m	1050m	1575m	1700m

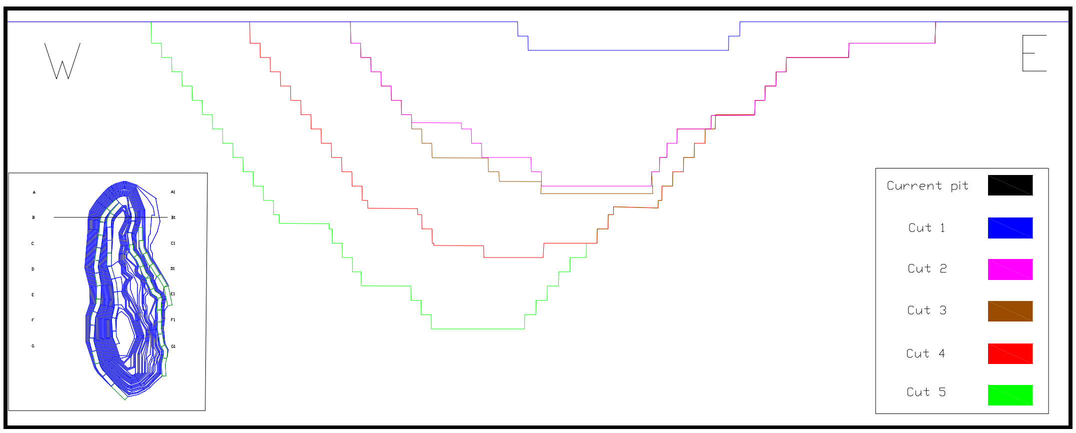


Figure 7.7 Cross section through Zwartfontein South slope designs for all five cutbacks

7.3.3 PPRust North pit

PPRust North is a massive open pit with a 5.5 km strike length and a planned final depth of 500 m and 13 cutbacks have been designed. It differs from Sandsloot and Zwartfontein South in that the cuts initially move along the strike and not the dip of the orebody (Figure 7.8). The earlier cutback west walls therefore have longer standing times (in the order of 5-10 years) but can be aggressively designed as the latter cutbacks extend these walls to the west and to greater depths. The east walls follow the orebody at roughly 45° and generally deepen over the life of mine. Cut 1 has a depth of 90 m and a slope angle of 45°. Larger equipment is being used at PPRust North for cost efficiency and the bench heights are therefore 15 m.

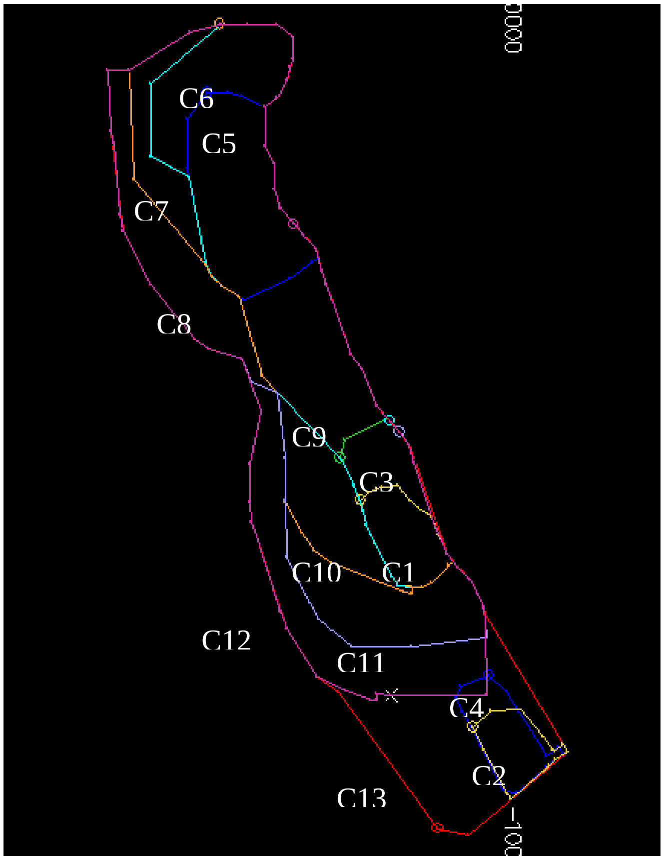


Figure 7.8 Plan view of PPRust North cutbacks C1 to C13 in Datamine

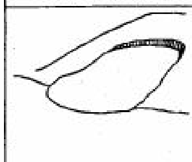
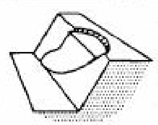
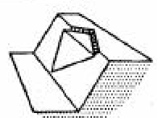
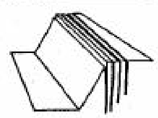
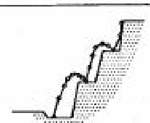
7.4 Slope Analysis Methods

Traditionally, open pit slope design has been based upon a factor of safety (FOS) against sliding along well defined failure surfaces. The FOS can be defined as the factor by which the shear strength parameters may be reduced in order to bring the slope into a state of limiting equilibrium (Morgenstern, 1991). The numerical value of the FOS chosen for a particular design depends upon the level of confidence which the designer has in the shear strength parameters, the groundwater pressures, the location of the critical failure surface and the magnitude of the external driving forces acting upon the structure (Hoek, 2001). Different factors of safety may be appropriate for different stages in the design of a rock structure. The uncertainty associated with the properties of geotechnical materials and the great care which has to be taken in selecting appropriate values for analyses has prompted several authors to suggest that the traditional deterministic methods of slope stability analyses should be replaced by probabilistic methods. Probabilistic analyses have been accepted for many years in the design of open pit mine slopes because open pit planners are familiar with the concepts of risk analysis applied to ore grade and metal price fluctuations. They have become far more widespread in geotechnical design in recent years, however, the current perception is that the factor of safety is more meaningful than the probability of failure (Steffen *et al.*, 2006). The FOS is limited when one does economic analysis of a slope and for this reason a probability of failure is more useful. In general terms, a FOS of 1.0 is approximately equal to a Probability of Failure (Pf) of 50%. Typical FOS and Pf values for mining projects are as follows (SRK, 2004):

- Individual bench: 1.05 to 1.10 i.e. 35% (recommended limit)
- Bench stack: 1.20 to 1.25 i.e. 10% - 15% (recommended limit)
- Overall slope: 1.35 to 1.50 i.e. <5% (recommended limit)

Slope failures can either be rock mass failures or structurally controlled failures. In the former, the rock mass is so highly fractured or weathered or under such great stress, that it fails as a single mass, ignoring the structures within it. This is the case in soft rock mines and the deeper mines. In structurally controlled failure, simple wedge, planar, toppling failures or more complex stepped path failures occur. The two scenarios must be treated quite differently when it comes to slope design. The differing rock fall hazards produced by both must also be considered. Table 7.3 (Hoek, 2001) summarises some of the typical stability problems, critical parameters, analysis methods and acceptability criteria which apply to an open pit mine. The analysis obviously depends largely upon correct structural interpretation of the rock mass and kinematic failure analysis as described in Chapter 6. What follows is a description of the slope design analytical tools that have been used at PPRust.

Table 7.3 Typical problems, critical parameters, methods of analysis and acceptability criteria for slopes (Hoek, 2001)

STRUCTURE	TYPICAL PROBLEMS	CRITICAL PARAMETERS	ANALYSIS METHODS	ACCEPTABILITY CRITERIA
 Landslides.	Complex failure along a circular or near circular failure surface involving sliding on faults and other structural features as well as failure of intact materials.	• Presence of regional faults. • Shear strength of materials along failure surface. • Groundwater distribution in slope, particularly in response to rainfall or to submergence of slope toe. • Potential earthquake loading.	Limit equilibrium methods which allow for non-circular failure surfaces can be used to estimate changes in factor of safety as a result of drainage or slope profile changes. Numerical methods such as finite element or discrete element analysis can be used to investigate failure mechanisms and history of slope displacement.	Absolute value of factor of safety has little meaning but rate of change of factor of safety can be used to judge effectiveness of remedial measures. Long term monitoring of surface and subsurface displacements in slope is the only practical means of evaluating slope behaviour and effectiveness of remedial action.
 Soil or heavily jointed rock slopes.	Circular failure along a spoon-shaped surface through soil or heavily jointed rock masses.	• Height and angle of slope face. • Shear strength of materials along failure surface. • Groundwater distribution in slope. • Potential surcharge or earthquake loading.	Two-dimensional limit equilibrium methods which include automatic searching for the critical failure surface are used for parametric studies of factor of safety. Probability analyses, three-dimensional limit equilibrium analyses or numerical stress analyses are occasionally used to investigate unusual slope problems.	Factor of safety > 1.3 for "temporary" slopes with minimal risk of damage. Factor of safety > 1.5 for "permanent" slopes with significant risk of damage. Where displacements are critical, numerical analyses of slope deformation may be required and higher factors of safety will generally apply in these cases.
 Jointed rock slopes.	Planar or wedge sliding on one structural feature or along the line of intersection of two structural features.	• Slope height, angle and orientation. • Dip and strike of structural features. • Groundwater distribution in slope. • Potential earthquake loading. • Sequence of excavation and support installation.	Limit equilibrium analyses which determine three-dimensional sliding modes are used for parametric studies on factor of safety. Failure probability analyses, based upon distribution of structural orientations and shear strengths, are useful for some applications.	Factor of safety > 1.3 for "temporary" slopes with minimal risk of damage. Factor of safety > 1.5 for "permanent" slopes with significant risk of damage. Probability of failure of 10 to 15% may be acceptable for open pit mine slopes where cost of clean up is less than cost of stabilization.
 Vertically jointed rock slopes .	Toppling of columns separated from the rock mass by steeply dipping structural features which are parallel or nearly parallel to the slope face.	• Slope height, angle and orientation. • Dip and strike of structural features. • Groundwater distribution in slope. • Potential earthquake loading.	Crude limit equilibrium analyses of simplified block models are useful for estimating potential for toppling and sliding. Discrete element models of simplified slope geometry can be used for exploring toppling failure mechanisms.	No generally acceptable criterion for toppling failure is available although potential for toppling is usually obvious. Monitoring of slope displacements is the only practical means of determining slope behaviour and effectiveness of remedial measures.
 Loose boulders on rock slopes.	Sliding, rolling, falling and bouncing of loose rocks and boulders on the slope.	• Geometry of slope. • Presence of loose boulders. • Coefficients of restitution of materials forming slope. • Presence of structures to arrest falling and bouncing rocks.	Calculation of trajectories of falling or bouncing rocks based upon velocity changes at each impact is generally adequate. Monte Carlo analyses of many trajectories based upon variation of slope geometry and surface properties give useful information on distribution of fallen rocks.	Location of fallen rock or distribution of a large number of fallen rocks will give an indication of the magnitude of the potential rockfall problem and of the effectiveness of remedial measures such as draped mesh, catch fences and ditches at the toe of the slope.

7.4.1 Empirical slope design

Laubscher (1990) determined an Indicative Overall Slope Angle (IOSA) design table (Table 7.4) for initial slope stability assessments. This was extended by Haines and Terbrugge (1991) into a design chart, which was discussed briefly in Chapter 5 and shown in Figure 5.6. Using this table or chart empirical slope evaluations have been performed at PPRust since 1992. The Mining Rock Mass Rating (MRMR) values are used to determine slope angles for various slope heights. With the use of structurally derived parameters for individual benches, it is possible to create a bench stack angle versus bench stack height relationship for the determination of berm widths. These bench stack geometries can be compared with those obtained from the empirical evaluation. It is then feasible to proceed to develop overall slope geometries for the various design materials, within each of the design sectors of the pit. The final slope architecture is then created. These angles represent a starting point for more rigorous numerical analyses.

Table 7.4 Indicative Overall Slope Angle (IOSA) Design Chart (Laubscher, 1990)

MRMR	100	90	80	70	60	50	40	30	20	10	0
Slope Angle	>75	75	70	65	60	55	50	45	40	35	<35

7.4.2 Limit equilibrium analysis

There are numerous software packages on the market today that simplify the analytical techniques used for slope design. It is important for the engineer to understand the strengths and weaknesses and choose the appropriate technique and software for the application. This is done by understanding the failure mechanism, as discussed in Chapter 6. Limit equilibrium methods are most commonly used in rock slope engineering even though they generally require 2D rigid block assumptions which do not properly represent the rock mass (Stead *et al.*, 2001). This static method is ideal for simple block failure along discontinuities but does not take progressive deformation and internal disruption of the rock mass and therefore it is inadequate for failure by complex mechanisms. All limit equilibrium techniques compare resisting forces to the disturbing forces in a rock slope. The critical input parameters are (Stead *et al.*, 2001):

- Representative geometry and material characteristics
- Rock mass shear strength parameters (cohesion and friction)
- Discontinuity shear strength characteristics
- Reinforcement characteristics
- External support data

The various software packages are mostly deterministic (FOS only) but there is an increased use of probabilistic methods. The assumptions used in the different packages do vary (Stead *et al.*, 2001). RocScience© have a number of excellent user friendly tools for limit equilibrium analysis including Slide and SWedge. SWedge is used on site at PPRust for simple wedge analysis. The software allows the user to define two joints, a tension crack, bench geometry, cohesion and friction angles, density, reinforcement, water pressure and seismic load. The user can also choose to calculate a FOS or a Pf. A wedge weight is calculated and the mode of sliding is determined. The limitations are that only a single bench is considered and only two joints can be defined. For large scale analysis i.e. stacks and overall slopes, Slide is utilised. Slide is a limit equilibrium analysis programme capable of evaluating slope stability using “simplified method of slices” techniques, including Bishop’s simplified method, to calculate a Factor of Safety (FOS) for slopes (RocScience, 2000). The program is capable of Monte Carlo simulation, using global or overall methods, to calculate a probability of failure (Pf). The model is reliable and simple to interpret and the input parameters are well established and understood.

PPRust outsources much of the analysis to consultants who used Slide to determine factors of safety and probability of failure. The pit stratigraphy is simplified to facilitate numerical modelling of the open pits (Figure 7.9). The zones selected for modelling included the hangingwall norite, the footwall parapyroxenite (which includes serpentinised parapyroxenite and calc-silicates) and the shear zones (which occur in the hangingwall and footwall). Although these simplifications do not fully reflect the complex pit geology, they serve to group the pit into geotechnical domains, i.e. zones of similar geotechnical properties, which are acceptable for numerical modelling purposes. Due to the varying geometry of the slopes and structural complexities within Sandsloot and Zwartfontein, numerous sections were required for the Slide analysis in order to representatively analyse the two pits.

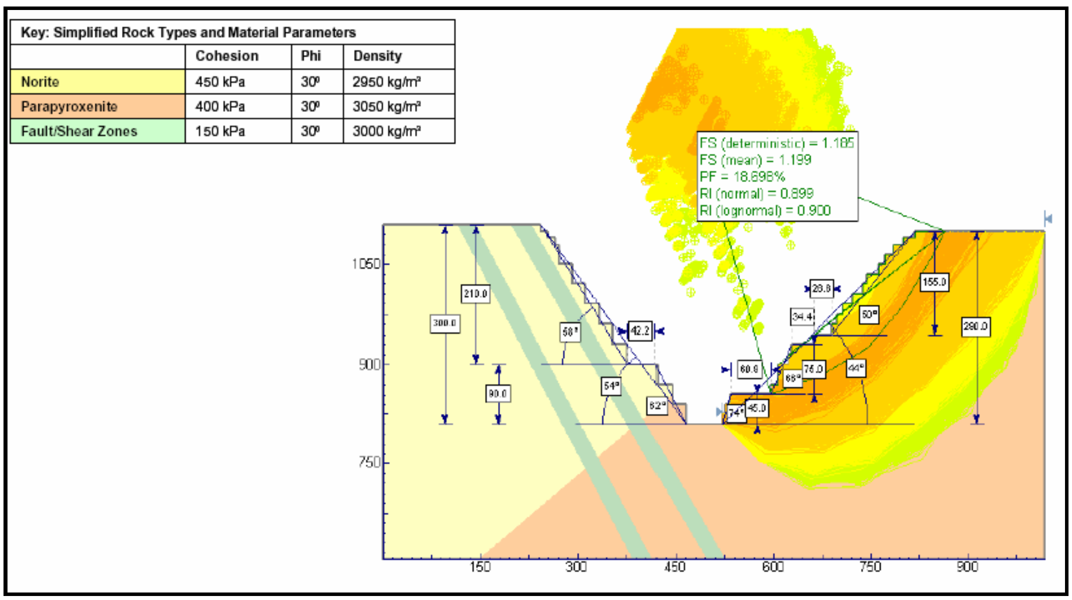


Figure 7.9 Example of Slide analysis done at PPRust by SRK (Section A, East, Sandsloot)

Based on the laboratory testing data, and incorporating both the mine’s existing rock strength data and SRK’s experience from similar projects, Mohr-Coulomb material parameters were determined for the primary lithological units. The geotechnical parameters used in the slope stability analysis are presented in Table 7.5. The FOS results reported for the Sandsloot and Zwartfontein South pits were obtained using Bishop’s Simplified method slices, which was selected due to the tested reliability of this approach.

Table 7.5 Geotechnical parameters used for PPRust slope stability analysis (SRK, 2003)

Lithology	Material Parameters				
	Density (kg/m^3)	MRMR	UCS (MPa)	c (kPa)	phi (°)
Sandsloot Norite HW	2950	50	180	450	30
Sandsloot P-Px FW	3050	40	157	400	30
Sandsloot Shear Zones	3000	44	-	150	30
Zwartfontein Norite HW	2950	52	187	500	30
Zwartfontein P-Px FW	3050	30	199	450	30
Zwartfontein Shear Zones	3000	51	-	150	30

7.4.3 Numerical modelling

Numerical modelling techniques allow for more complex slope analyses where geometry, material anisotropy, non-linear behaviour, in situ stresses and coupled processes can be taken into account. Analysis methods can be divided into three approaches: continuum, discontinuum and hybrid modelling (Stead *et al*., 2001) and are summarised in Table 7.6.

Continuum analyses include finite-difference and finite element approaches. The majority of published continuum analyses in recent years have used 2D finite difference code FLAC (Stead *et al*., 2001). It includes time dependant behaviour, coupled hydro-mechanical and dynamic modelling and allows the user a wide choice of constitutive models to characterise the rock mass. It assumes plane strain conditions which are unlikely to be accurate for inhomogeneous slopes. As computer processing has improved, the 3D continuum code FLAC3D has been developed. Discontinuum methods include discrete-element and distinct-element modelling. The rock mass is seen as an assemblage of blocks that can be rigid or deformable. Movement on discontinuities controlled predominantly by the joint shear stiffness is included. It is the most commonly used numerical method with distinct-element codes such as UDEC being the most popular. The geotechnical engineer must determine which method is suitable, though using both will probably give the best results. Time and financial constraints will dictate to a certain extent.

Table 7.6 Numerical methods of analysis (Coggan *et al*., 1998)

Analysis method	Critical input parameters	Advantages	Limitations
Continuum Modelling (e.g. finite-element, finite-difference)	Representative slope geometry; constitutive criteria (e.g. elastic, elasto-plastic, creep etc.); groundwater characteristics; shear strength of surfaces; *in situ* stress state.	Allows for material deformation and failure. Can model complex behaviour and mechanisms. Capability of 3-D modelling. Can model effects of groundwater and pore pressures. Able to assess effects of parameter variations on instability. Recent advances in computing hardware allow complex models to be solved on PC's with reasonable run times. Can incorporate creep deformation. Can incorporate dynamic analysis.	Users must be well trained, experienced and observe good modelling practice. Need to be aware of model/software limitations (e.g. boundary effects, mesh aspect ratios, symmetry, hardware memory restrictions). Availability of input data generally poor. Required input parameters not routinely measured. Inability to model effects of highly jointed rock. Can be difficult to perform sensitivity analysis due to run time constraints.
Discontinuum Modelling (e.g. distinct-element, discrete-element)	Representative slope and discontinuity geometry; intact constitutive criteria; discontinuity stiffness and shear strength; groundwater characteristics; *in situ* stress state.	Allows for block deformation and movement of blocks relative to each other. Can model complex behaviour and mechanisms (combined material and discontinuity behaviour coupled with hydro-mechanical and dynamic analysis). Able to assess effects of parameter variations on instability.	As above, experienced user required to observe good modelling practice. General limitations similar to those listed above. Need to be aware of scale effects. Need to simulate representative discontinuity geometry (spacing, persistence, etc.). Limited data on joint properties available (e.g. jk_n, jk_s).
Hybrid/Coupled Modelling	Combination of input parameters listed above for stand-alone models.	Coupled finite-element/distinct-element models able to simulate intact fracture propagation and fragmentation of jointed and bedded media.	Complex problems require high memory capacity. Comparatively little practical experience in use. Requires ongoing calibration and constraints.

SRK utilised FlacSlope for recent PPRust numerical modelling work to determine and confirm factors of safety and modes of failure determine by Slide. FlacSlope is a finite-difference (continuum numerical modelling) software package that allows the user to build models with specified zones and to select plane strain, plane stress or axis-symmetric conditions (SRK, 2003). The user can change settings to manipulate the zones and is able to specify a number of materials with various densities and strength parameters. The user cannot specify different elastic properties for the different materials, since it is assumed that the FOS calculations are not affected by varying the stiffness. This attribute limits the ability of FlacSlope to correctly determine the mode of failure, which must be accounted for when interpreting results. The shear zones at PPRust were modelled using isotropic weak zones, the location being inferred using the available geological data. The analyses were carried out using the same representative cross-sections used in the Slide analysis, using an elastic, perfectly plastic material model (Figure 7.10). The input parameters for the FlacSlope model are the same as used for the Slide analysis. A comparison of the factor of safety calculated using Slide and using FlacSlope produced a correlation coefficient of 0,93. There are, however, two notable exceptions where the FOS was underestimated using FlacSlope. This may be attributed to FlacSlope being unable to accurately model the complex material zones for particular sections, resulting in the models having to be simplified. This resulted in the factor of safety value being underestimated. In such instances, the Slide value is deemed to be the more reliable value.

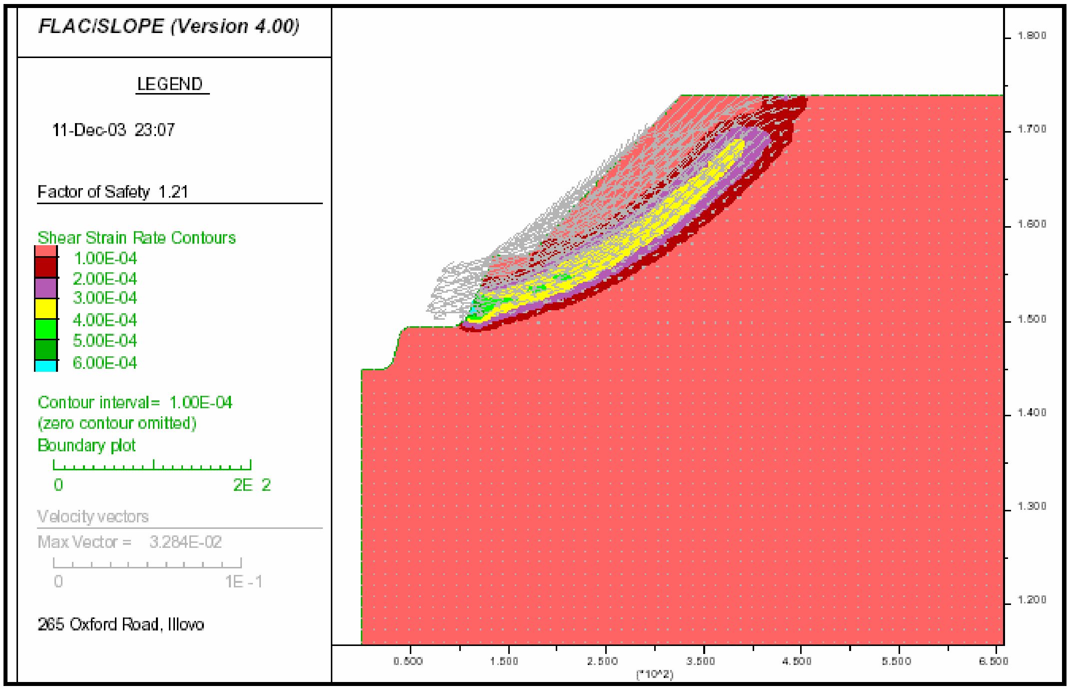


Figure 7.10 Example of FlacSlope analysis done at PPRust by SRK (Section A, East, Sandsloot) showing shear strain rates and velocity vectors

The results from the Slide and FlacSlope analyses were used for the most recent designs at PPRust. The FOS values indicate the stable angle at which the slopes can be mined. Ideally the slope angle that produces a FOS of 1.2 is used. Theoretically this achieves safety and economic requirements. The design work does not stop there as the probabilities of can be used in event and fault tree analysis. This is explained further in Chapter 10 in the slope optimisation context.

7.5 Rock Fall Analysis

The bench-berm configuration that is ultimately designed needs to take into account the rockfall hazard present on the slope. Berms act as rockfall catchments and therefore should be as wide as possible to ensure that rockfall does not continue down the slope. Where inclined benches are designed the berms need to be narrower to fit a desired slope angle while vertical benches allow for wider berms. Double benching is often used to create wider catchments further apart, which may be a safer alternative. The rockfall hazard will depend on the rock properties, the shape and size of the loose rocks, the amount rainfall and water draining down the slope and the exposure of the pit personnel. To assess rockfall hazard at PPRust, the RocScience© software program RocFall is used.

RocFall is a very simple program that uses an algorithm to calculate the trajectory of a falling rock based on changes in velocity as it rolls and bounces over a specified slope geometry (RocScience, 2001). The user designs a slope and assigns material properties to each section with standard deviations assigned to each parameter. The user 'seeds' the slope with one or many rocks and defines initial conditions for the rock/s: horizontal velocity, vertical velocity, mass and angular velocity. The user defines the project settings: number of rocks, minimum velocity cut-off, which friction angle to use, sampling intervals, coefficient of normal restitution and random-number generation method. The number of rocks to throw is essentially the number of iterations that the software performs. The average final impact velocity is calculated. By running through a number of bench-berm configurations one can determine how to minimise the rockfall hazard. This analysis has been done in both Sandsloot and Zwartfontein South open pits. For the latter, the study showed that double benching was a safer option (Figures 7.11 and 7.12). This is due to the crest backbreak to failure planes on the west wall. These planes provide a skid surface that projects the rockfall further downslope and over the small single bench catchment. By double benching, the number of crests, and therefore backbreak, is reduced and the berms are twice as wide.

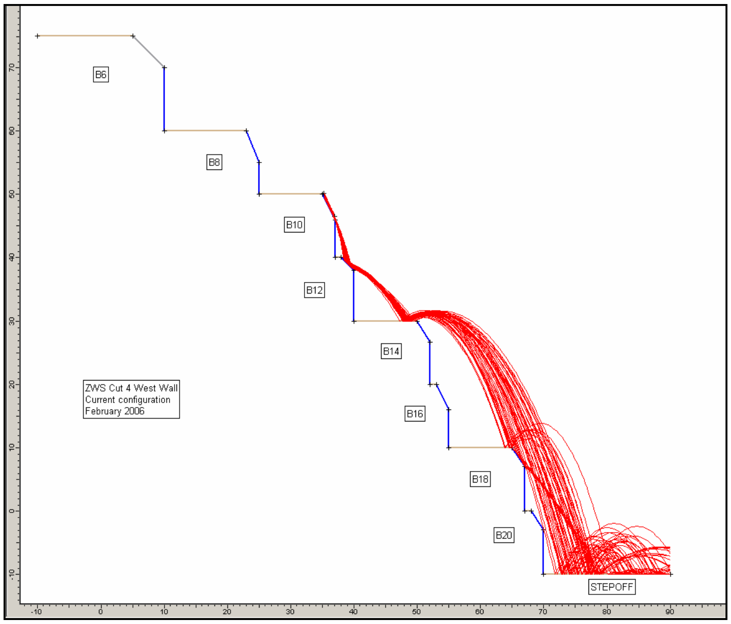


Figure 7.11 RocFall analysis for Zwartfontein South west wall single benched

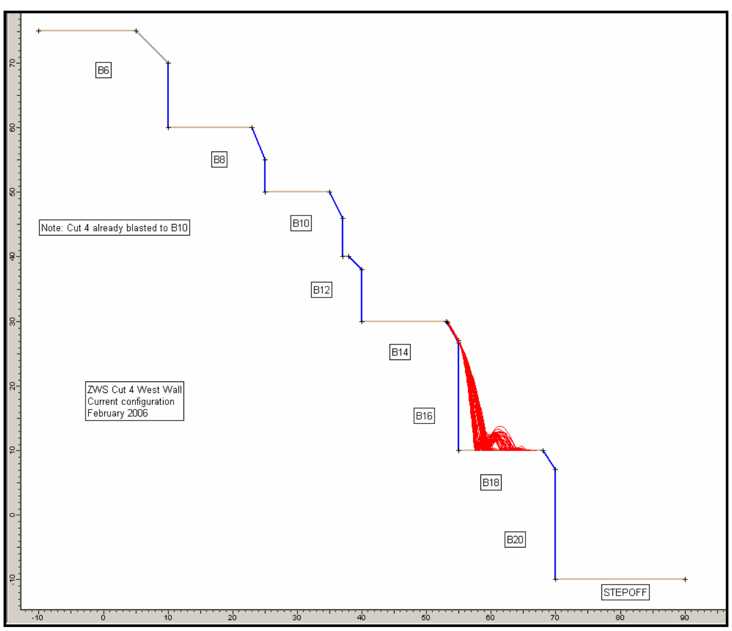


Figure 7.12 RocFall analysis for Zwartfontein South west wall double benched

It is evident from all these analytical techniques that the field data that is collected must be of a high quality and sufficient quantity to ensure that the results are as close to the truth as possible. With the large logging, mapping and testing databases at PPRust the uncertainty in the models is reduced.

7.6 Authorisation Tracking System

All pit designs at PPRust go through a rigorous review process by Anglo Platinum, Anglo American and outside consultants. An annual Geotechnical Review Board (GRB) was created in 2003 and ensures the correct slope design methodology and management is in place. During a review it became clear that a better audit trail was needed for slope design approvals. It was too easy for the incorrect design to be taken off the LAN and utilised. For this reason the Authorisation Tracking System, ATS, was designed by CGSS consultant Peter Nathan, in conjunction with the author. ATS is an electronic approval system for any document that must be circulated to a number of people in the Mine Technical Services at PPRust. Initially it was aimed at pit designs only but it rapidly extended to accommodate blast requests, blast patterns, limit changes, bench signoffs, grade composites and cut designs and field inspections are stored in ATS. It has been designed to allow for the authorisation of any type of document and there is much scope for future development. ATS provides a complete audit trail of any authorisation and improves transparency and efficiency.

ATS has a Visual Basic front-end which updates and queries a MS Access database located on the LAN. Users are defined by the administrator and access is passworded. The main window is called the Scoreboard (Figure 7.13) and it lists the authorisations that are currently in circulation and indicates their key fields. Each authorisation has a defined routing list (Type) with an owner and creation date. A deadline is assigned to the authorisation and any number of files can be associated to an authorisation at any time. A users tag (Ore/Waste) can be assigned for additional information. The Scoreboard shows who an authorisation is pending with (Current Owner), how long they have held it for (For) and who is next in the routing list (Next Owner). The authorisations can be sorted by any of these columns by clicking on the column heading. They can also be filtered, using the dropdown lists at the bottom. A help manual is available from the Scoreboard.

When a user logs in, a red box appears at the top of the Scoreboard if there are any authorisations pending their approval. By clicking on it the Pending window will open and show which authorisations need to be approved. In each routing list, each person has a user rating:

A1 = Approval required before authorisation continues down routing list
A2 = Approval required before authorisation can be closed
A3 = Read only

The A1, A2 and A3 Approvals are listed separately in the Pending window. By clicking on an authorisation in any of the three lists, it will be highlighted in the Scoreboard and can then be opened. Double clicking on the relevant authorisation allows the user to view the details in the ‘Authorisation details’ window (Figure 7.14).

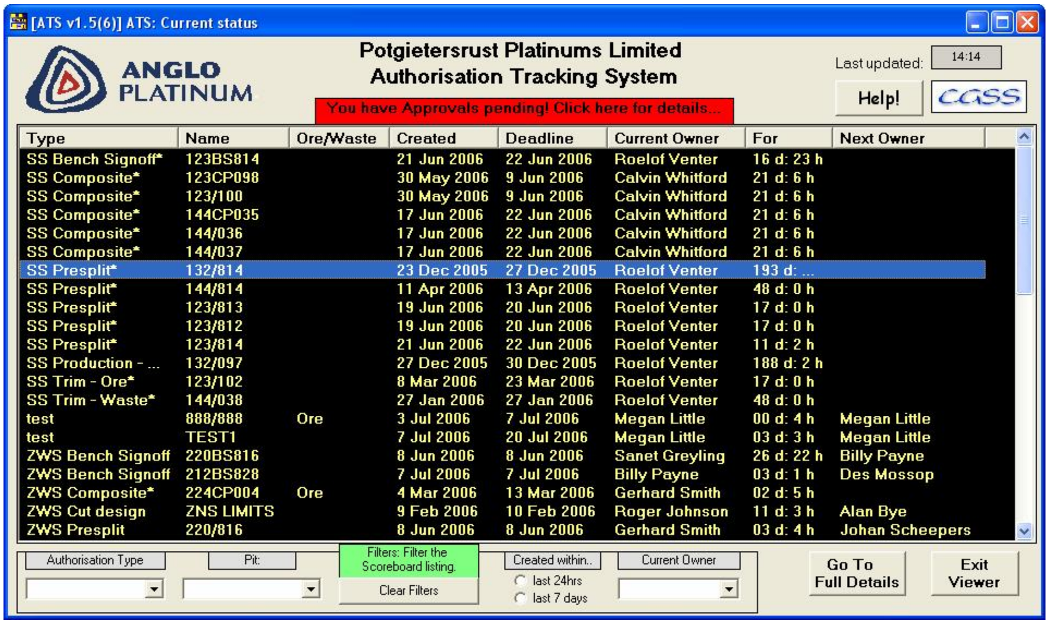


Figure 7.13 ATS Scoreboard for viewing current authorisations

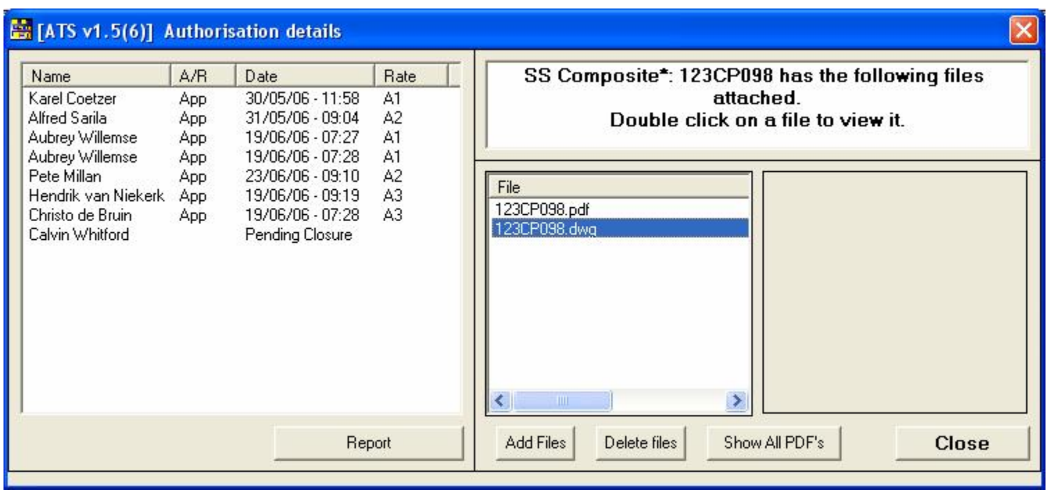


Figure 7.14 ATS Authorisation Detail window

Here the route that the authorisation has taken is shown. Each person in the list has either approved or rejected the authorisation and may have added a comment or a file. Approvals are done from this window by double clicking on the relevant name. This will open up a window where the person may type in a comment and Accept the authorisation or Reject it to any A1 on the routing list that has already given approval. ATS automatically sends out emails to the relevant parties when a document is approved or rejected. The email will contain the name of the document, who sent it and whether the recipient is an A1, A2 or A3. For blast patterns, if an authorisation is not approved then staking data cannot be taken off the LAN and drilling cannot continue. This ensures that mining is not done off plan and without the correct approval.

There is also a ‘Database Maintenance’ window (Figure 7.15) for adding, editing and deleting authorisations, users and routing lists. It is accessed from the Scoreboard and has four tabs – Authorisations, Users, Routing List Templates and Setups. The Authorisations tab lists all the authorisations currently in circulation as well as those that have been fully approved and archived. The Users tab lists all the ATS users and each person can edit their details here. New routing lists are added in the Routing list templates tab and old lists can be edited or deleted. The setups tab is where the administrator can change system settings such as the location of the help manual.

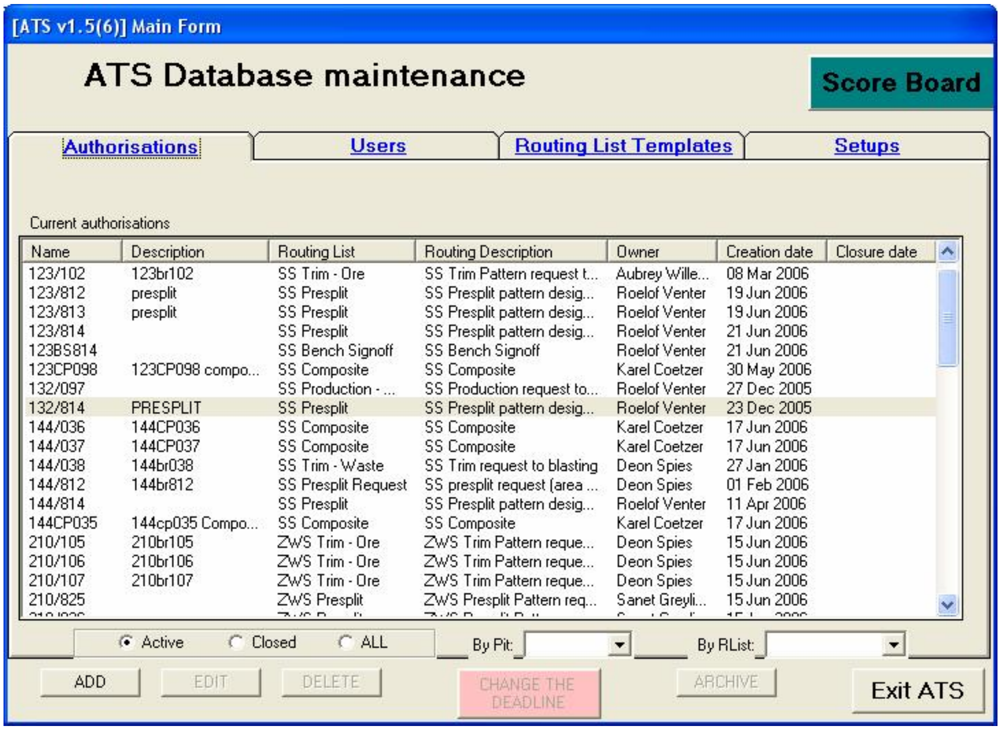


Figure 7.15 ATS database maintenance window

ATS has been in use at PPRust Mining since October 2005 and has successfully replaced the paper signoffs. It has improved the auditability of designs and placed responsibility on the appropriate people's shoulders. It is currently being reviewed for implementation throughout Anglo Platinum.

7.7 Conclusion

Slope design is the main geotechnical function on an open pit mine. Slope design plays a major role in mine economics and in the case of PPRust a single degree can add R200 million to the overall profitability. The design methodologies provided by Bieniawski (1991), Stacey (2006) and Steffen *et al.* (2006) allow the geotechnical engineers to work towards the appropriate designs. The risk consequence approach has been employed at PPRust and incorporates geotechnical data, block modelling, failure analysis into initial slope designs. Empirical techniques, limit equilibrium analysis and numerical modelling have been used for calculating factors of safety and probabilities of failure. Slope management and slope monitoring are added to the geotechnical knowledge base and utilised in slope optimisation to ensure that the slope designs are maximising profit while meeting the stringent safety and risk requirements of Anglo Platinum. This is further discussed in the next chapter.

8 SLOPE MANAGEMENT

"Design is not just what it looks like and feels like. Design is how it works."
- Steve Jobs

This Chapter describes slope management measures at PPRust and explains how they improve slope stability in the open pits. This includes limit blasting practices, slope support, visual inspections and dewatering.

8.1 Introduction

A large amount of work goes into a designing a slope as described in the previous chapters. Mining of the slope though is never without problems and it is important to manage the slope during its operating life. This can be done is a number of ways. Limit blasting reduces blast damage so that the bench limits are maintained and the slopes are more stable. Visual inspections by both geotechnical and operational personnel are done on a daily basis to check whether the design is performing as planned. Slope support is installed if it is not. It is usually a cheaper option than redesigning the slope. Slope monitoring is a major form of slope management at PPRust and therefore it will be discussed in greater detail in Chapter 9. To achieve good slope management measures, commitment from the geotechnical engineers, surveyors and mining operations personnel is necessary. This is the case at PPRust and has been a major contributing factor in reducing geotechnical risk levels to acceptable standards.

8.2 Limit Blasting

Open pit mining in hard rock utilises the controlled destruction of a rock mass to extract ore. The challenge that the blaster faces is to achieve the fragmentation targets while minimising the damage to the rock slopes. In order to do this, the blaster needs a good understanding of the factors that control rock fragmentation and damage. Before discussing this further a brief explanation of basic blasting terminology is explained and illustrated in Figure 8.1 and Figure 8.2.

8.2.1 Blasting terminology

A blast pattern usually has one or more free faces which are open sides of the blast block. A box cut is a blast with no free faces. The two descriptors of the blast pattern layout are burden and spacing. The burden is the distance perpendicular to the free face between two rows in a pattern while the spacing is the distance parallel to the free face between two rows in a pattern. The spacing is usually 1.15 times the burden. The blastholes can be laid out in a square or staggered pattern. In a square pattern the holes line up in rows in both directions. In a staggered pattern the rows are offset from one another. A staggered pattern usually gives better fragmentation as the blastholes areas of influence do not overlap or leave gaps which is the case in a square pattern.

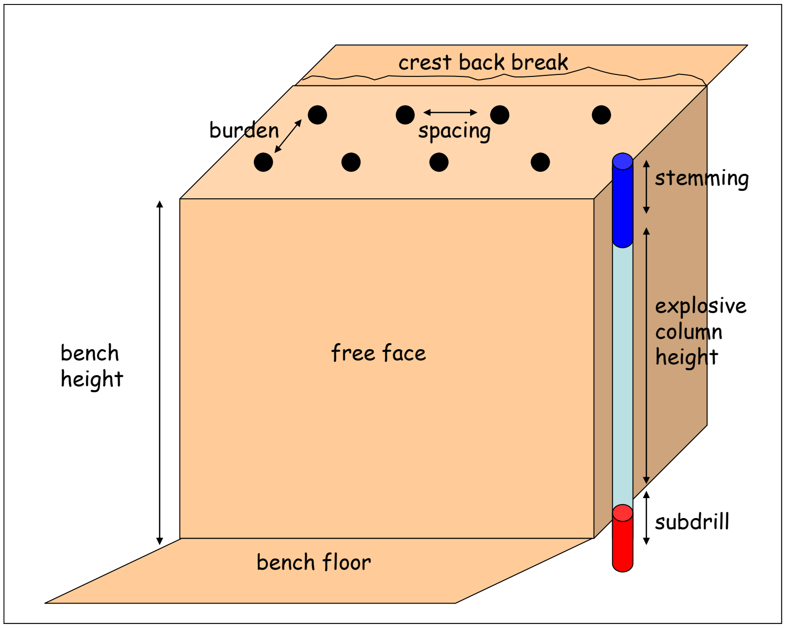


Figure 8.1 Blast design sketch and terminology

The blasthole is never completely filled with explosives. The explosives column height is the length of the blasthole that is filled with explosives. Stemming refers to the top section of the blasthole where a plug of material, such as rock chips, is placed to keep the blast energy within the rock mass and prevent flyrock. The subdrill is the extra blasthole length drilled to ensure the entire block is fragmented and to achieve a smooth bench floor. The subdrill will damage the bench below. This is not a problem for the part of the bench that will be blasted and loaded out but it is a concern for the section that will form the next catchment berm. This damage weakens the slope and reduces its stability. To avoid this, the subdrill can be shortened or not drilled at all. The blastholes with shortened subdrill are called crest protection holes. Back break describes the amount of damage done to the crest of the remaining sidewall. The powder factor is the mass of ANFO explosives (in kg) required to break a cubic metre of rock. The strength of an explosive is a measure of the work done by a certain weight of volume of explosive. It is expressed as a ratio relative to the standard explosive, ANFO, and called its relative weight strength (RWS). The energy factor is the mass of any explosive (in kg) required to break a cubic metre of rock. It is simply the multiplication of the powder factor and the RWS (in %).

8.2.2 Limit blasting practices

When good limit blasting practice is in place, three different kinds of blast patterns are designed – presplits, trim blasts and production blasts. A production blast is a large blast pattern drilled with large diameter holes at least 15 m from the bench limit. It produces the most energy and does the most damage. Trim blasts are blasts adjacent to the bench limit that have a reduced charge to minimise the amount of energy that impacts the sidewalls. They are drilled with smaller diameter holes and are usually only 10 - 25 m wide. The one or two rows closest to the highwall may have reduced charges, and are then called buffer rows. A presplit is a single row of closely spaced holes which are blasted simultaneously with the aim of creating a fracture along the bench limit. Some of the blast energy from the subsequent production and trim blasts will escape out of this fracture and reduce the damage done to the sidewall. The spacing of the presplit holes is very important as if they are too far apart a fracture will not form. The explosive charge also has to be high enough to overcome the tensile strength of the rock mass.

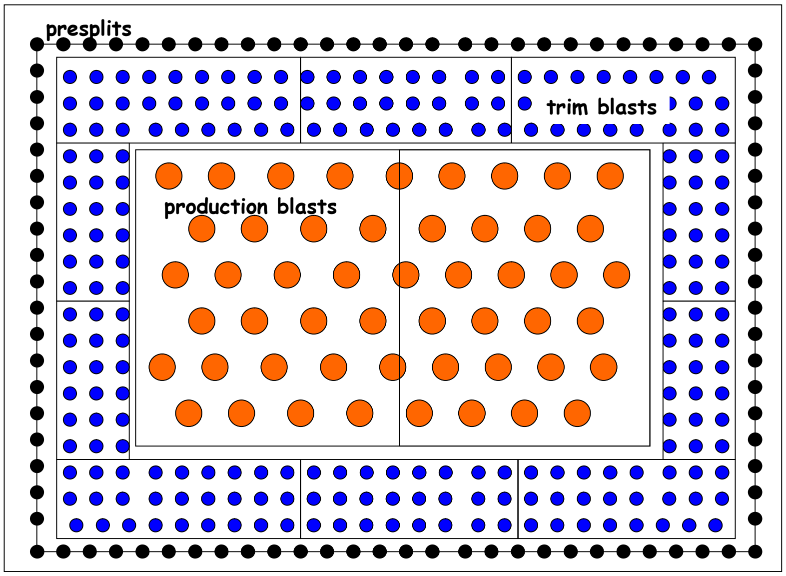


Figure 8.2 Sketch of a bench with 3 different types of blast pattern designs – presplits along the limit, trims (square) and production blasts (staggered)

During blasting, two mechanisms, with very different time frames, operate to damage to a highwall (Cunningham, 2003):

1. strain waves and expanding gases of detonation, leading to crushing and crack development within a few milliseconds
2. inertial mechanisms leading to ground shift which lasts for tens or hundreds of milliseconds

There are various methods of reducing the blast damage caused by these two mechanisms. They are:

- Presplits
- Buffer rows
- Trim blasts
- Crest protection holes
- Suitable explosive type
- Direction of blast movement parallel to sidewall
- Correct timing between rows
- Electronic detonators

The first four points refer to the blast design geometry and have already been discussed above. There is a huge variety of explosives in today's global market, and it is important that the correct type is chosen not purely on cost and mining method, but also with blast damage in mind. Different explosives will emit different amounts of shock energy and gas energy. Shock energy cracks the rock while gas energy opens up fractures in the rock. Explosives that emit high gas energy have a more detrimental effect on the sidewalls. The timing of each blast plays a major role in the fragmentation and the blast damage. The direction of blast movement required will determine the timing design. By ensuring that the blasted material moves parallel to the highwall and not perpendicular to it, the blast energy is directed away from the highwall. Similarly, by timing the detonation of each blast row to allow enough time for the outer sections of the block to move far enough to create an open space for the next section to move into, the energy going into the highwall is reduced. The introduction of electronic delay detonators (EDDs) can significantly reduce the scatter in the timing of a blast. This reduces the chance of out-of-sequence firing which would negate the positive effects of timing designed to reduce highwall damage (Cunningham, 2003).

8.2.3 PPRust limit blasting

A comprehensive wall control blasting programme is used at PPRust. This was developed primarily by Dr Alan Bye over a number of years – from 1998 to 2003. Using his geotechnical knowledge, he fine-tuned the blast designs for ore versus waste as well as production versus trim blasts. Specific presplit configurations were designed for different geotechnical zones which reduced costs by 25 to 80% (Bye and Bell, 2001). Vertical presplits are currently drilled on every bench limit at a spacing of between 1 m and 2.5 m. Inclined presplits were initially designed as they result in less crest damage and safer walls, if drilled accurately. Due to the small footprint of the open pit they became logistically unmanageable and resulted in operational delays and therefore gave way to vertical presplits. Trim blasts are drilled adjacent to the final wall of every cut in both pits. They are 15 - 25 m in width and consist of three to six rows of smaller diameter (165 mm) holes (Figures 8.3). The average pattern design is 4 m burden and 4.5 m spacing which produces a powder factor of ~1,1 kg/m^3. The two rows adjacent to the presplit in the trims are buffer rows and have an 80% reduced charge. Crest protection holes are drilled adjacent to the presplit and they reduce backbreak and the rockfall hazard.

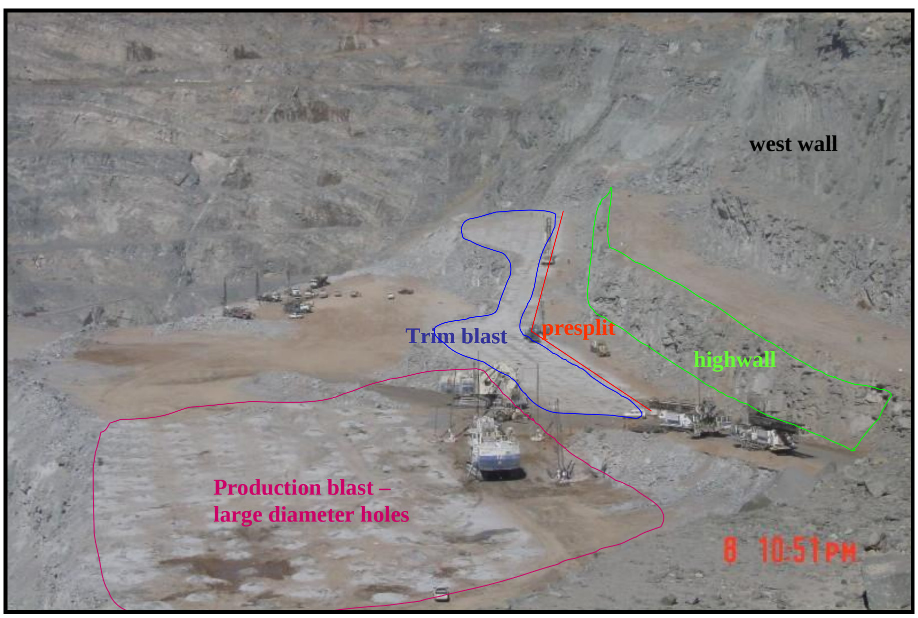


Figure 8.3 Photograph of a production shot, trim, presplit and the adjacent highwall in Sandsloot pit

Quality control in blasting is critical as the timing of detonation is based on the planned pattern geometry and blasthole depth. If the holes are not in position or are drilled to the incorrect depth, then the timing will be out and the effectiveness of the wall control programme will be reduced. A drillhole accuracy plan has been developed in AutoCAD with Peter Nathan for drilling quality control. The plan is generated for each blast pattern by the surveyors when they import the blasthole data after drilling. AutoCAD calculates the maximum, minimum and average distance variation between the planned, staked and actual blasthole locations and plots this on a new drawing. It automatically draws a template with all the relevant information around the pattern. This can be issued as a report to the Drill and Blast Operations Manager. Quality control is also performed by a wall control foreman who is responsible for the successful blasting of all presplits and trims. He is also responsible for highwall cleanup. The face shovels clean the face as much as possible when loading out a trim blast. Scaling rigs and backhoes are then used to clean whatever loose material remains. This has proven very effective and has significantly reduced the rockfall hazard by more than 100%.

Extensive trials of electronic delay detonators (EDDs) began in 2002 at PPRust, using both BME and AEL products. The aim was to improve limit blasting and fragmentation and to control blast vibrations and flyrock (Bye, 2005). Previously shock tube detonation was used which is limited in its timing accuracy and can produce a 10 % deviation from the timing design. EDDs give actual detonation times within 0.1 ms of the designed times allowing for timing repeatability and flexibility. This gives them a much greater ability to manipulate the movement of the blast and to reduce the destabilising effect on the highwalls (Cunningham, 2003). Rigid quality control was implemented for the trials and clear targets of loading and milling rates were set. Digital fragmentation analysis using Split-Desktop was conducted to accurately measure the results and define baseline targets. Instantaneous loading rates (ILR) of 238 blasts were assessed over a two year period to determine the ideal loading rate for ore and waste. The rock response times were measured for the major rock types which showed that longer delays were needed to allow the rock mass to move out enough to allow the blast energy to escape. The trials were very successful and today all blasts at PPRust are done using EDDs. The benefits of the EDDs are:

- blast damage is reduced and thus slope stability is improved
- expanded waste patterns reduce costs
- shovel productivity improved and ILR reaching >4800 t/hr at times
- consistent ore fragmentation targets achieved
- destructive interference created to reduce blast vibrations

A number of other mines have trialled EDDs and found similar results. For example, De Beers Consolidated Mines Ltd tested EDDs for all their mines over an 18 month period and are systematically replacing shock tube detonation due to the positive results. The increased cost of the EDDs was justified by the improved quality control and reliability. Other benefits were improved fragmentation control, back break reductions, increased development face advance underground and the reduction of oversize during production blasting (Grobler, 2003).

8.3 Visual Inspections

Slope design is based on modelling large amounts of geotechnical data in specialised software packages. Although huge benefit can be derived from computer modelling, it cannot replace time spent in the pit inspecting the face. Visual inspections are essential for observing whether the slope design is appropriate, for identifying rockfall hazards and for assessing the stability after blasting. The Rock Engineering department at PPRust is committed to spending the time inspecting the face and four specific inspections are performed on a regular basis. These are:

- Daily inspections
- Detailed inspections
- Monthly hazard plan inspections
- Presplit inspections

The flow diagram in Figure 8.4 illustrates how these four inspections performed by the Rock Engineering department are reported and what the response to each inspection is. They will be discussed individually in subsequent sections.

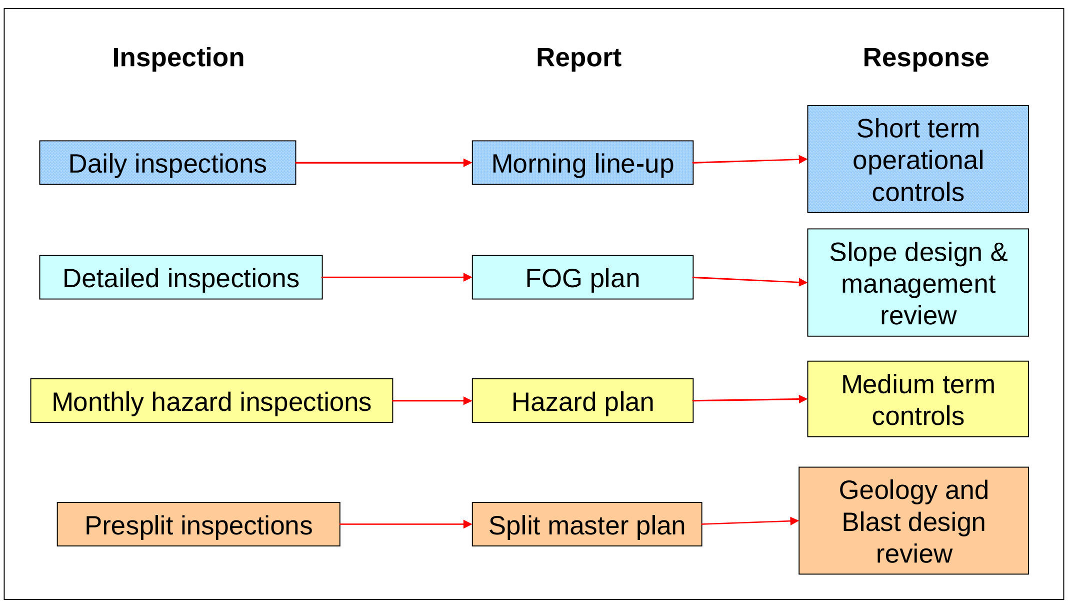


Figure 8.4 Flow diagram showing the four inspections performed by the Rock Engineering department, where they are reported and what the response to each inspection is

The data is collected on paper in the field and initially was inputted into MS Excel spreadsheets. In order to manage the data and to make it of optimal use, a database was set up in MS Access by the author. The main form (Figure 8.5) enables the user to quickly see what inspections have been done where, by whom and on what date. The user can then view or edit any inspection in the database or add a new inspection to the database simply by clicking a button. Each inspection sheet has been converted from MS Excel format to MS Access format reducing the chance of error and speeding up the data capture process. In the future the data will be inputted straight into MS Access in the field using a pocket computer to save time. Emails are sent out to the operations managers when a new inspection is added to the database. They can then open up the database on the network, add their comments and subsequently communicate the observations to their subordinates who implement the risk mitigation recommendations. By storing the inspections in a database the data is secure, auditable, standardised and readily available. This ensures that it is acted upon immediately and the relevant people are held accountable.

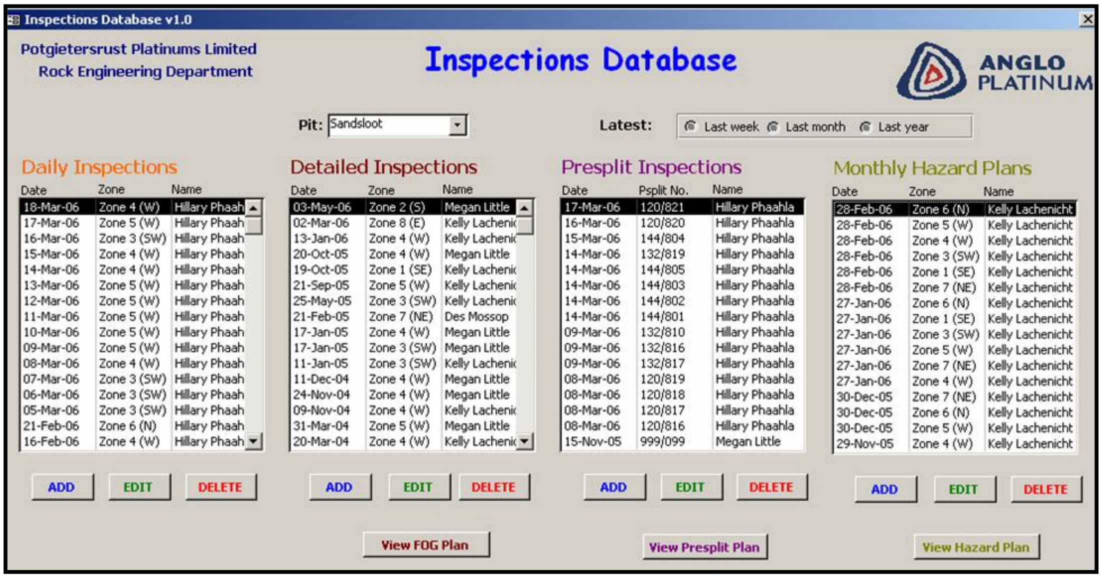


Figure 8.5 PPRust Rock Engineering inspections database main form

8.3.1 Daily inspections

Every morning the Rock Engineering (RE) technical assistant inspects the highwalls of the current working areas. He uses a simple report sheet (Figure 8.6) to record comments on the crest, face, water, current operations, areas of concern and remedial action required. He takes digital photographs of the area and saves them in a specific location on the network. He inputs all the data into the Inspections database and adds links to the photographs so that they can be viewed in the database. He then emails the Chief Rock Engineer who distributes the report via ATS and raises any issues at the following morning line-up at 7am the next day. The operations managers respond with comments on the form, in ATS and at the line-up meeting. They are responsible for implementing any operational remedial measures while the Survey and Rock Engineering departments are responsible for ensuring the required slope monitoring is in place.

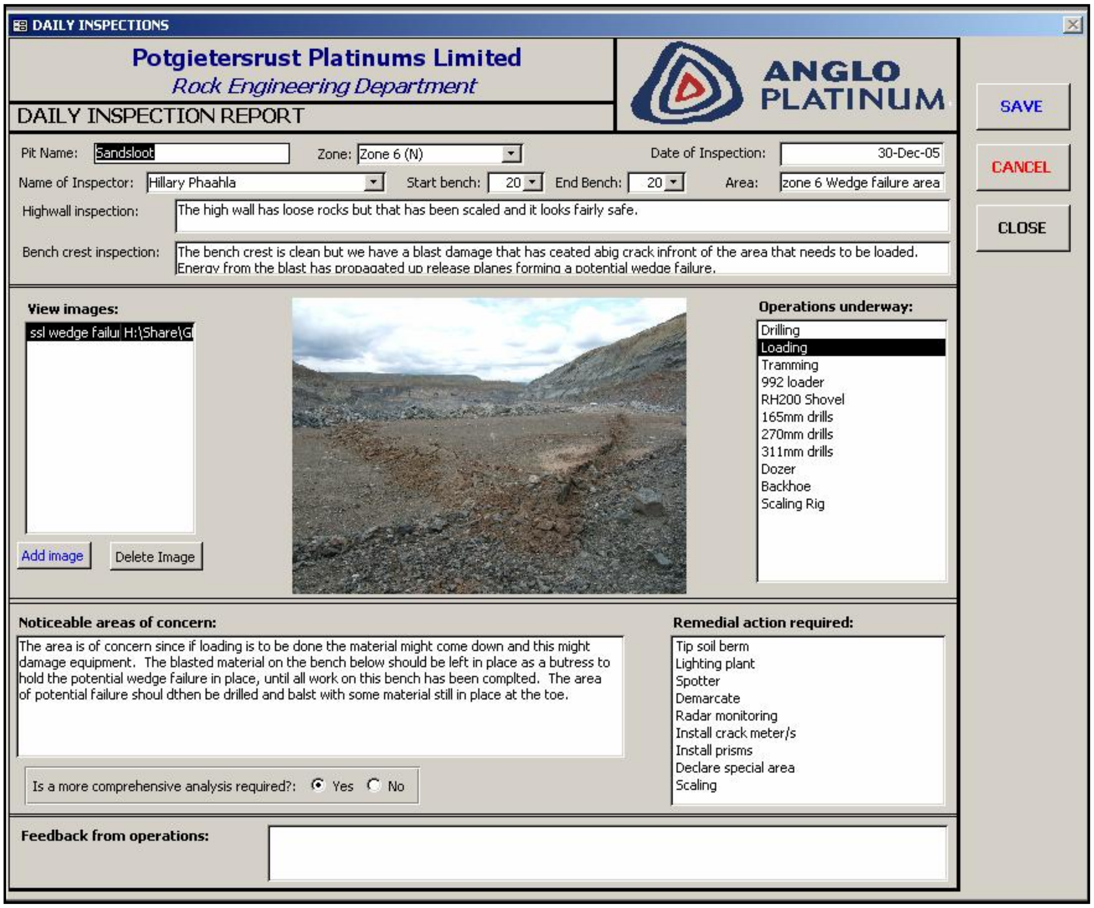


Figure 8.6 Example of a completed daily inspection form as it appears in the Inspections database

8.3.2 Detailed inspections

In the event of a fall of ground (FOG) a detailed inspection is performed by one of the Rock Engineers at PPRust. A four-page report sheet is used to record data and comments on the following:

- *General*: location of FOG, operations underway, brief description
- *Safety*: effect on operations, recommendations
- *Failure surface*: joint/fault orientation and condition
- *Groundwater*: evidence of groundwater, recent rainfall data
- *Support*: support measures in place or required
- *Blasting*: blast parameters for recent blasting activity in the area
- *Monitoring*: slope monitoring measures in place or required

The inspector takes digital photographs of the area and saves them in a specific location on the network. He/she inputs all the data into the Inspections database and adds links to the photographs so that they can be viewed in the database. He/she also adds the failure to the FOG plan in AutoCAD and creates an image file of the location of the failure and links it to the database (Figure 8.7). The inspector then emails the Chief Rock Engineer who distributes the report to all relevant managers via ATS and may declare the area a Special Area. The operations managers respond with comments on the form, in ATS and at the monthly Special Areas meeting. They are responsible for implementing any operational remedial measures while the Survey and Rock Engineering departments are responsible for ensuring the required slope monitoring and/or support is in place.

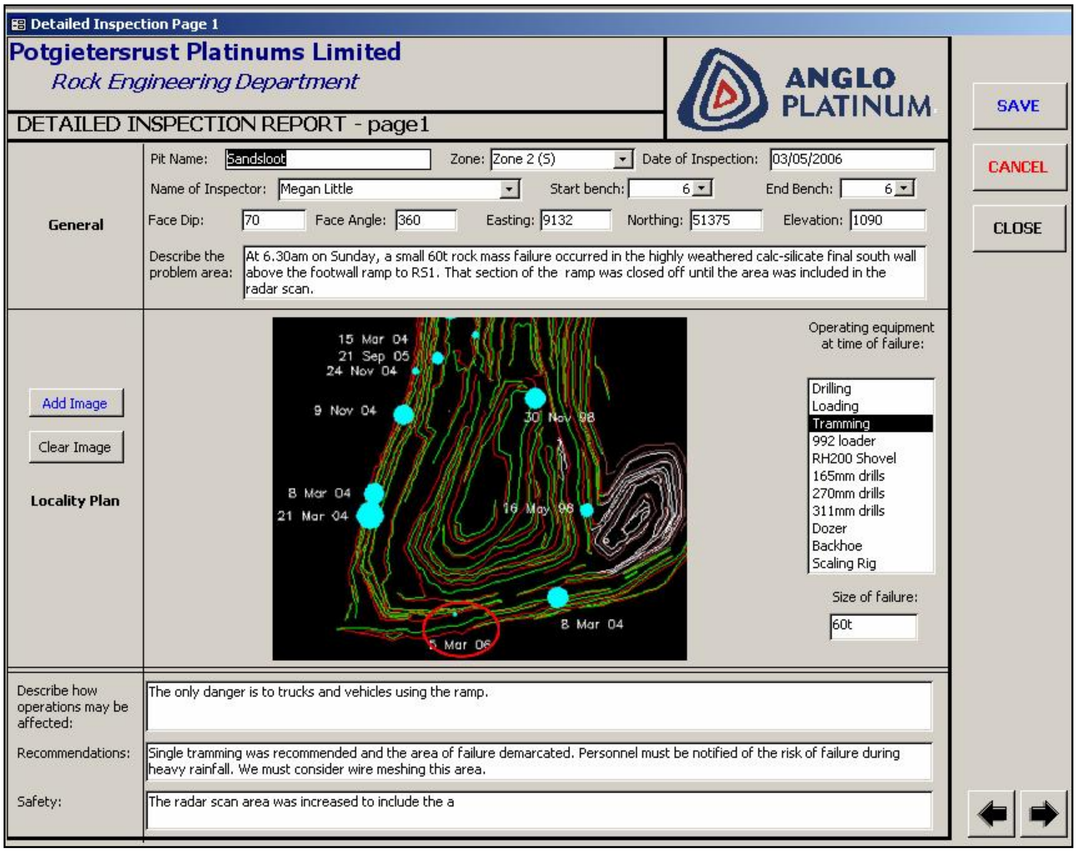


Figure 8.7 Example of the first page of a completed detailed inspection as it appears in the Inspections database

Special Areas are areas in the pit that pose a slope stability concern. During the course of normal mining operations some areas have an increased risk of slope instability and/or rockfall. Such an area may be declared a Special Area if it can be identified as such and will require additional attention. Special Areas are designated whenever they appear or when it is anticipated that there is an increased risk of rockfall or slope failure occurring in an existing or proposed working place. This designation allows management to make rapid modifications where such action is urgently required. All such areas are motivated and documented. The Chief Rock Engineer must pay particular attention to special areas and records the location of all special areas, the classification and declaration procedure and the monitoring of such areas. These records are available for scrutiny. The controlling body for management of all slope instability at PPRust is the Special Areas Committee. The committee is a multi-disciplinary team consisting of specialists from all areas of surface mining. Special Areas will be classified as follows :

- *Precautionary Area:* Area in which precautionary measures are applied at the discretion of the mine management in consultation with the Chief Rock Engineer and the Special Areas Committee.

- *Restricted Area:* Area in which the procedures decided upon by the Special Areas Committee and recorded in writing for the specific area become obligatory for the duration of the declaration.
- *Prohibited Area:* Area in which entry is prohibited for all personnel except the Chief Rock Engineering and/or his designate. Any other rock engineering staff shall require written permission from the responsible Manager (2.8.1 appointee) to enter such areas.

8.3.3 Monthly hazard plan inspections

The RE monthly hazard inspection was designed at PPRust to provide an indication of the geotechnical risk to the pit personnel in the coming month's working areas. It was based on the Anglo American risk matrix for open pit mines. The inspections are performed at the end of every month and are presented at the Special Areas meeting, which is on the first Wednesday of each month. The rock engineers complete a comprehensive spreadsheet for each working area for the coming month. As evident in Figure 8.8, the input sheet can be divided into two main sections – geotechnical and operational – and each section is further subdivided as follows:

Geotechnical:
- Slope design
- Water management
- Rock mass description
- Geotechnical data and design
- Failure potential

Operational:
- FOG potential
- Blasting performance
- Monitoring
- Evacuation effectiveness

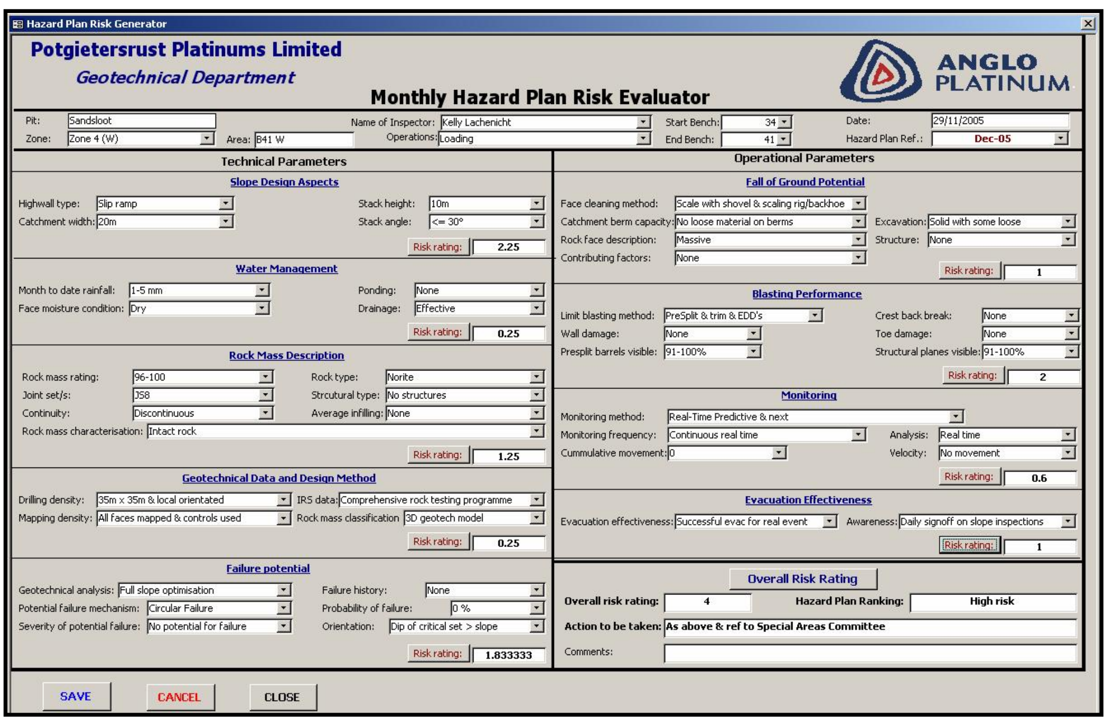


Figure 8.8 Example of a completed hazard plan inspection as it appears in the Inspections database

A rating is calculated for each sub-section and they are combined to determine an overall risk rating that ranges from 1 to 10. This rating is converted to one of 6 risk classes from ‘Very low risk’ to ‘Extremely high risk’ (Table 8.1). The risk ratings and classes are calculated in the MS Access database form by the push of a button thus saving time and avoiding calculation errors. Each risk class has a colour, which is then plotted on the current pit plans to produce a monthly hazard plan for each pit (Figure 8.9). These hazard plans are linked to the inspections database thus when emailed, the operations managers can quickly view them and print and distribute them to their staff. They are displayed in all shift change areas and green areas so that all pit personnel are aware of the dangers in their working place at all times. Each risk rating has an associated action which the pit superintendent is responsible for implementing. By utilising the database the hazard plans are available on the PPRust network so any of the mine personnel can check up on the hazards in their area.

Table 8.1 Risk classes with their ratings, colours and required actions

Rating	**Class**	**Colour**	**Action**
0-1-2	Very low risk	Cyan	Notify people in working area and treat asap.
1-3	Low risk	Blue	Notify people in working area and treat asap. Adhere to stand-offs.
3-4	Medium risk	Green	Demarcate highwall stand-off, monitor, treat asap.
4-5-6	High risk	Brown	Demarcate, monitor, treat asap. Refer to Special Areas committee.
6-7-8	Very high risk	Orange	Demarcate with SSR, treat asap. Operate in area by permission only.
8-9-10	Extremely high risk	Red	No entry. Cease all operations until rectified.

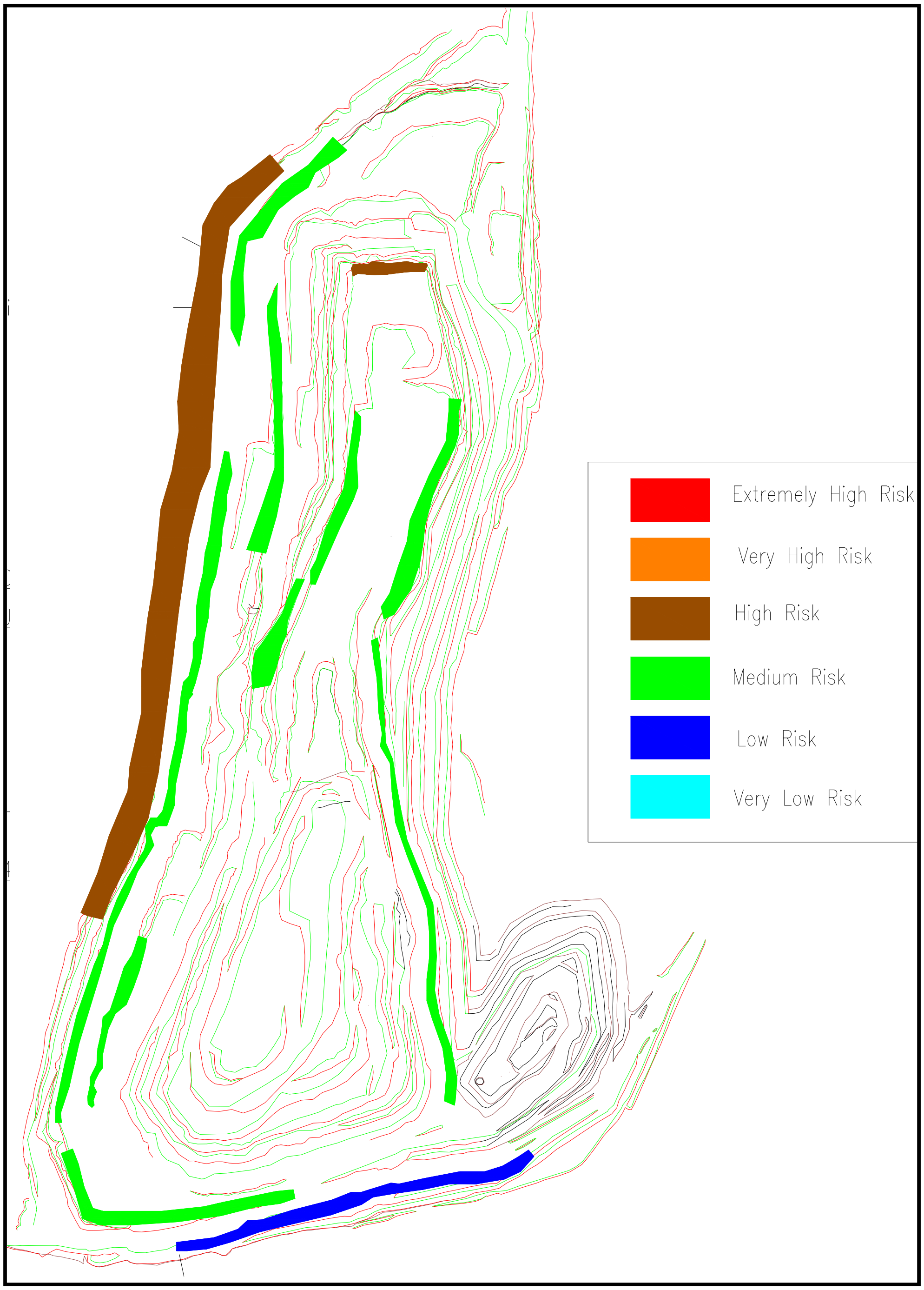


Figure 8.9 Example of a monthly hazard plan for Sandsloot pit

8.3.4 Presplit inspections

Presplit inspections were designed to measure the success of the presplit and trim blast designs. They enable the rock engineers and blast engineers to determine the cause of damaged final walls and take remedial action ahead of further blasts. After each presplitted wall is cleaned, the RE technical assistant will inspect it using a detailed sheet, shown in Figure 8.10. The sheet has three sections: general information; highwall condition and blast design. Each section collects the following information:

- *General*: Presplit name and date blasted; adjacent trims and their date blasted; geology, structure and GSI
- *Highwall condition*: drilling accuracy, scaling, damage, frozen faces, hard toes, face length
- *Blast design*: presplit angle, number of rows, hole diameter, powder factor, initiation and timing

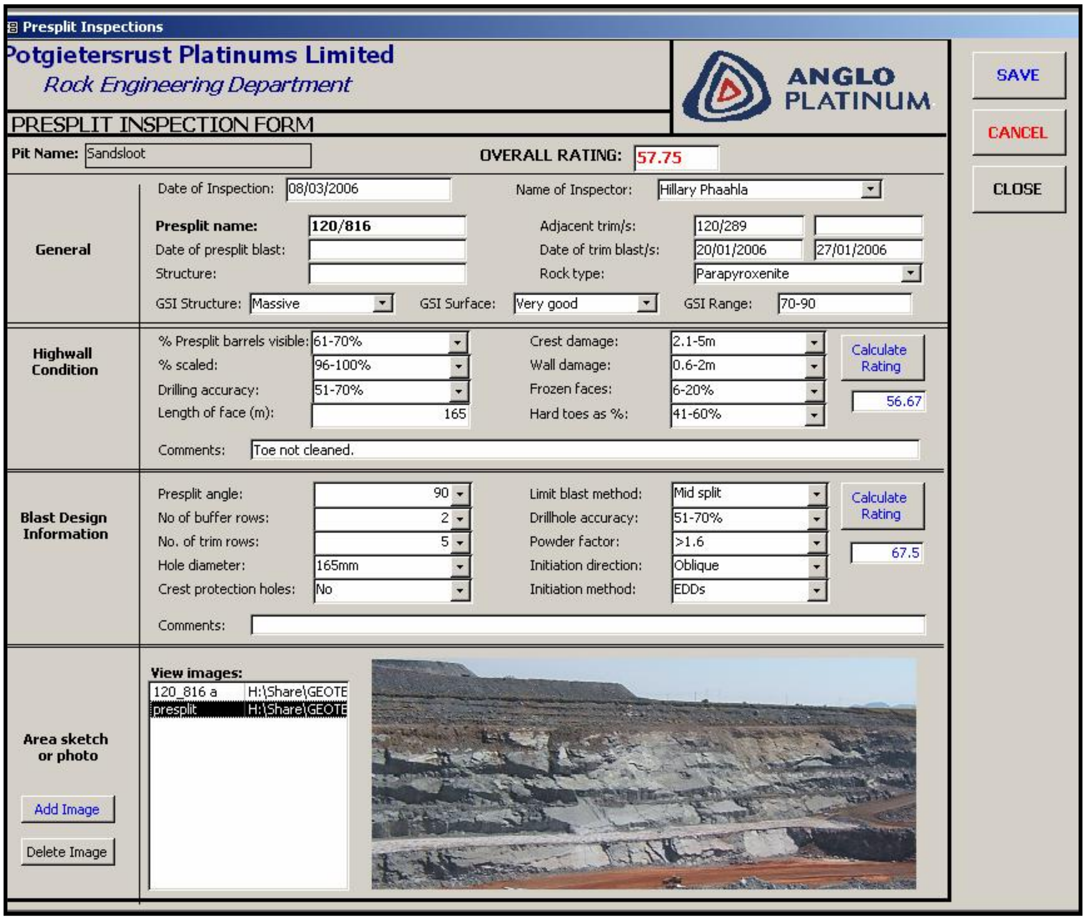


Figure 8.10 Example of a completed presplit inspection as it appears in the Inspections database

A rating system was developed to convert all the data that is captured on the form into a single value between zero and 100 that quickly indicates how badly the highwall was damaged. This value is converted to a classification from ‘Very poor’ to Very good’ and is assigned a colour as shown in Table 8.2. Each parameter has its own rating table and contributes to the overall rating out of 100. Ratings for highwall condition (out of 90) and blast design (out of 10) are calculated separately to aid in the identification of problem areas. Some of the parameters are weighted more heavily than others and have a rating out of 15 instead of 10. The calculations performed are as follows:

Table 8.2 Presplit Inspection classes

Rating	Classification	Colour
0-40	Very Poor	Red
40-50	Poor	Orange
50-60	Fair	Yellow
60-80	Good	Green
80-100	Very Good	Blue

Overall Rating = Blast Design Rating + Highwall Rating

Highwall Rating = Presplit Barrels Visible /10 + Scaling /10 + Drill Accuracy /10 + Crest Damage /15 + Wall Damage /15 + Hard Toes /15 + Frozen Faces /15

Blast Design Rating = (Presplit /10 + Initiation Method /10 + Initiation Direction /10 + Drilling Accuracy /10 + No. of trims rows /10 + No. of buffer rows /10 + Powder factor /10 + Crest protection holes/10) / 8

All blast patterns are designed in AutoCAD and stored on the PPRust network by the Draughting Department. Anyone with access to AutoCAD can open up any of the presplit designs. They also have the option to view all the presplits for a specific bench on what is called the 'Split Master' plan. The presplit ratings are automatically overlaid on these split master plans in AutoCAD (Figure 8.11) which are linked to the database. This enables all mining and technical personnel to view the presplit ratings and get an idea of where there are problem areas. Those areas with low ratings are investigated.

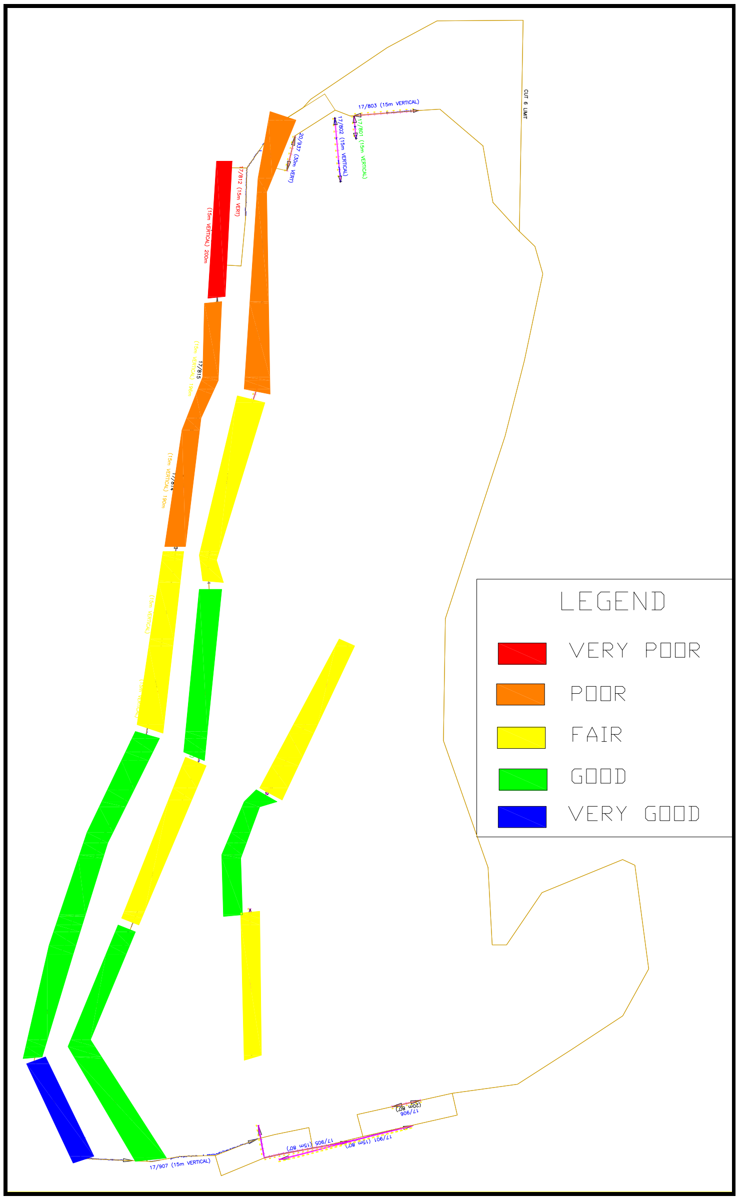


Figure 8.11 Example of a Split master plan coloured by presplit ratings

A poor presplit can be the results of a number of factors:

- geological structures (influence how blast energy is used and dissipated)
- inaccurate drilling (poor quality control)
- incorrect presplit design (spacing is too big or too small)
- incorrect charging (too little energy)
- buffer cut off by presplit (incorrect timing)
- incorrect trim design (powder factor too high or too low, initiation direction can be wrong, timing can be wrong)

It is also important to look at the presplit ratings adjacent to the presplit being analysed as that will help determine where the problem lies. If most of the presplits in the area have low ratings then it needs to be dealt with immediately. If it is the exception then there is a good chance it will not be a problem on the next presplit. Once the cause of damage is determined, steps can be taken to solve the problem. If the drilling is inaccurate, the drilling sub-contractors can be penalised financially. If the design is incorrect, trials can be performed to improve it. Management needs to be informed of these problems and their solutions, and the colour-coded Split Master plans make that task much simpler.

The Presplit Inspection Form has been in use since early 2004 and has produced good results. The following examples give an indication of the ratings assigned to different presplits at PPRust. The first example (Figure 8.12) shows a good wall with accurate drilling and no hard toes or frozen faces. There is crest damage, however, which reduced the rating to 87.

Figure 8.12 Presplit rating Example 1: Overall rating = 87

Example 2 (Figure 8.13) is a typical face on the west wall of Sandsloot which is a special case due to the fault zones that cross cut it and cause bench failure. As the fault zones are so extensive the presplit ratings were designed to reduce the effect of structure on the overall rating. Thus the rating is still ‘Good’ at 66 which is correct as, though a large portion of the wall has failed out, it is not unsafe because it can be scaled and there are no frozen faces. As discussed in Chapter 6, blast energy interacts with the geological structures and causes failure. The wall is managed by slope monitoring. The ‘good’ rating means the blasters are not penalized by something that is out of their control.

Figure 8.13 Presplit rating Example 2: Overall rating = 66

Example 3 (Figure 8.14) is one of the worst walls inspected with the Presplit Inspection Form and produced a rating of only 24. The damage was a result of poor blast design and lack of quality control.

Figure 8.14 Presplit rating Example 3: Overall rating = 24

8.2.5 Foremen inspections

By law, the foremen in the pit are legally responsible for making a workplace safe. They must perform visual inspections at the start of each shift. To aid the foremen at PPRust in identifying unstable highwalls, a daily geotechnical inspection sheet was designed by the Rock Engineering department (Figure 8.15). The sheet forces the foremen to look for specific indicators and to accept responsibility for taking any necessary action. They are also required to state in the shift change logbook whether they believe the working areas are safe. The indicators of instability that they look for are:

- Water coming out of the face and/or ponding at the toe.
- Tension cracks on the bench crest
- Loose rocks on the highwall
- Ravelling of small rocks down the face
- Cracking noises in the face

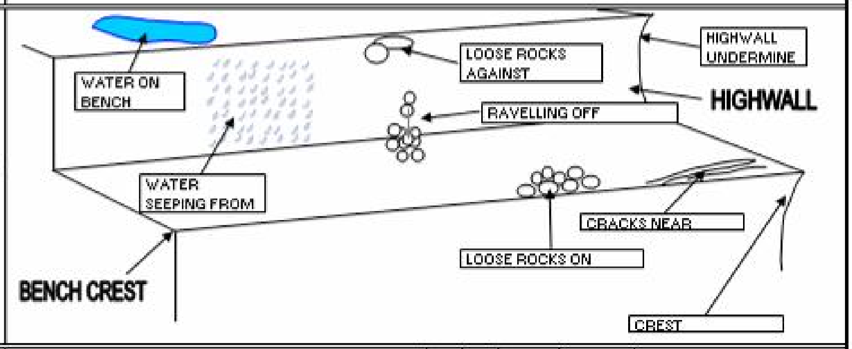
They also check whether the necessary signboards and monitoring tools are in place. The foremen are held accountable for their inspections and therefore take greater care in their workplace. This improves the safety of the workers and helps the geotechnical engineers identify and keep record of stability problems in the open pits.

GENERAL

Date of inspection : ____________ Name : ____________

Loading area : ____________ Shift : ____________

EXPLANATION SKETCH



INSPECTION REPORT

BENCH CREST :	YES	NO	SAFE	UNSAFE	ACTION TAKEN
CREST UNDERMINED?					
CRACKS NEAR CREST?					
CRACKS VISIBLY WORST THAN PREVIOUS INSPECTION?					
DUST COMING FROM CRACKS?					
LOOSE ROCKS ON CREST?					
HIGHWALL :	**YES**	**NO**	**SAFE**	**UNSAFE**	**ACTION TAKEN**
LOOSE ROCKS AGAINST HIGHWALL?					
HIGHWALL UNDERMINED?					
SMALL STONES AND BOULDERS RAVELLING OFF WALL?					
WATER SEEPING FROM HIGHWALL?					
WATER ON BENCH ABOVE?					
HAZARD SIGNBOARDS IN WORKING AREA?					
WORKPLACE CLEARLY DEMARCATED?					
HAVE ALL SUBORDINATES BEEN INFORMED?					
DISTANCE FROM HIGHWALL TO WORKPLACE.				metres	

IMMEDIATE SAFETY MEASURES REQUIRED? (Y ____________

YES, DESCRIBI ____________

SIGNED

INSPECTED BY : ____________

SUPT. : ____________

L&H MANAGER. : ____________

R. E. MANAGER : ____________

DATE : ____________

Figure 8.15 Daily geotechnical inspection sheet for foremen at PPRust (PPRust, 2005)

8.4 Slope Support

In some cases, slopes are designed with slope support included in the design to strengthen the rock mass. This has not been the case at PPRust and instead slope support has been added in localized unstable areas as a slope management tool. For the first 11 years of mining there were very few slope instability problems. The permanent east wall in Sandsloot is very stable while the west wall only experienced problematic instability in 2003. The bench and stack failures on this west wall are a hazard but the cost and logistics that would be involved in supporting the entire wall is prohibitive. Instead slope monitoring tools are used to manage that problem. These are discussed in Chapter 9. There have been three uses of slope support in the last six years in Sandsloot and they are briefly discussed below. The other two pits have not shown any need for slope support.

8.4.1 Sandsloot footwall ramp gabion wall

Gabions are steel wire baskets filled with rock used as an energy absorbing device to dissipate wave energy in a variety of civil engineering applications. Gabions were used for the first time in the mining industry at PPRust in 2000 (Bye and Rorke, 2002). The planned permanent footwall ramp was found to contain ore and to avoid losing the ore or the ramp, the ore was excavated and the ramp was backfilled. This was the best option both from a geotechnical and economic perspective. A near vertical, stone-packed gabion wall was constructed to contain the backfilled material (Figure 8.16). This wall has been stable since construction and has sustained tramming on the ramp for six years. A minor slope failure occurred below the gabion wall and this was shotcreted and bolted to maintain the stability of the gabion wall and therefore the permanent ramp.

Figure 8.16 Gabion wall supporting the backfilled footwall ramp in Sandsloot (Bye and Rorke, 2002)

8.4.2 Sandsloot Bench 11 gabion wall

The case study discussed in Chapter 6 describes a small scale failure and the rock overhang that it left behind. The overhang represented a potential failure of a few hundred tonnes of norite and a number of methods of managing this failure were discussed. Blasting it down was considered too dangerous as was high density rock bolting. These two methods could also have resulted in less stable conditions. The chosen solution was to build a gabion wall below the failure in the hope that if it did fail it would be held back by the wall. The gabions were filled with waste material and were held in place with 20 m long steel dowels. The final wall was ~100 m long, 2 m high and 5 m from the toe of the failure (Figure 8.17). Since construction, no movement has occurred on the overhang so the wall has not been put to the test. It has contained minor rock falls however and therefore has improved the safety on the ramp.

Figure 8.17 Gabion wall built below the Bench 11 failure in Sandsloot pit

8.4.3 Sandsloot boulders wire meshing

The crest of the west wall in Sandsloot contains many large boulders which form in the weathered norite. They pose a safety risk as they are fairly loose and if they fall they roll for quite a distance. The area of most concern was the north-western corner of the pit above the permanent hangingwall ramp. To eliminate this risk, wire meshing was used to cover the entire 15 – 45 m highwall above the ramp. Steel dowels were installed on the pit crest to secure the mesh which is held on the face with steel pipes (Figure 8.18). Rock falls that have occurred since construction have been contained by the wire mesh and are easily cleaned out by a dozer. This has significantly improved the safety on the hangingwall ramp.

Figure 8.18 Wire meshing to reduce the rockfall hazard

8.5 Dewatering

As discussed in Chapter 3, groundwater flow at PPRust is fracture controlled. Water is known to destabilise slopes and cause failure. At PPRust groundwater and rainfall flows on open joints and faults and have played a role in bench and stack failures. The best method of dewatering is usually by drilling boreholes outside the pit and pumping out the groundwater. A groundwater study was done by SRK in 2004 on Sandsloot open pit. Ten boreholes, GW1 to GW10, were drilled to 250 m depths and pumps and piezometers were installed. As shown in Table 8.3, pumping produce highly variable results with yields ranging from zero to 30.5 m^3 per hour. This study showed that dewatering by pumping could not work at PPRust. An alternative dewatering option is to drill toe drains into water bearing fractures. Unlike pumping, this allows water onto the fractures in the slope and is more a reactive than a proactive method. Inclined boreholes are drilled into the face on structures that either carry groundwater or are expected to carry groundwater. This was trialled on the west wall in Sandsloot on Bench 32 but was unsuccessful. It was difficult to target the correct water-bearing structure as there are so many potential failure planes. Also the boreholes were limited to 12 m by the available drilling equipment. Further trials should be done to investigate the application of toe drains at PPRust. Though these dewatering techniques were unsuccessful, conventional dewatering in the pits is performed by the operational staff by digging sumps on every bench. The water in the sumps is pumped out and this dewaters the blast blocks and improves blasting and also reduces the wear on the truck tyres. The final technique employed at PPRust to reduce the negative impact of water on the slopes is the creation of gutters on the pit perimeter and along the ramps. This is aimed at reducing the destabilising effects of heavy rainfall.

Table 8.3 Summary of piezometer information at Sandsloot pit

Name	Location	Final depth	Water strikes	Yield	Final yield	Piezo	Depths
GW1	Bundwall, centre of pit	250m	Dry	Dry	Dry	1	Open to 250m
GW2	Bundwall, southern portion of pit	250m	105-106m	2.82m^3/hr	2.82m^3/hr	1	106m
GW3	W of bundwall, centre of pit	250m	19-20m	0.3m^3/hr	0.8m^3/hr	1	22m
			105-106m	0.78m^3/hr		2	107m
GW4	Slightly N of SSPZ1 (E wall)	250m	20-21m	0.3m^3/hr	0.9m^3/hr	1	22m
			37-38m	0.5m^3/hr		2	64m
			62-63m	0.7m^3/hr		3	205m
			203-204m	0.9m^3/hr			
GW5	N boundary of pit on SSL river	250m	58-59m	Seepage (0.0m^3/hr)	0.4m^3/hr	1	61m
			135-136m	0.4m^3/hr		2	110m
			98-99m	0.1m^3/hr		3	138m
GW6	Between the 2 seismic houses	250m	41-42m	Seepage (0.0m^3/hr)	2.82m^3/hr	1	50m
			49-50m	1.6m^3/hr		2	100m
			99-100m	2.82m^3/hr			
GW7	On top of northern bundwall	167m	165-166m	30.48m^3/hr	30.48m^3/hr	Test pump	
GW8	Bundwall, northern tip of pit	250m	Dry	Dry	Dry	1	40m
GW9	Outside mine boundary, near small crusher	250m	19-21m	1m^3/hr	2.0m^3/hr	1	20m
			87-90	1.47m^3/hr		2	91m
			117-120m	1.5m^3/hr		3	118m
GW10	SW of Dispatch on haul road	250m	57-58m	1m^3/hr	2.8m^3/hr	1	66m
			65-66m				90m
			84-90m				99m
			98-99m				

8.6 Conclusions

Slope management is crucial to ensure the success of a slope design. Comprehensive limit blasting is practised at PPRust to reduce blast damage and attain pit limits. Visual inspections are performed on a daily basis by geotechnical and operational staff in all working areas. The geotechnical data is stored in a MS Access database developed on site and is available to the operations personnel. A minor amount of slope support has been implemented including gabion walls, shotcrete, bolting and wire meshing. Dewatering techniques have been investigated but the fracture controlled groundwater flow at PPRust hinders pumping. Sumps and gutters are used instead to minimise the effect water has on destabilising the slopes. These have all reduced the risk of slope failure and rockfall in Sandsloot pit. For more comprehensive slope management, state-of the-art monitoring technology is used. This is described in the following chapter.

9 SLOPE MONITORING

“If the only tool you have is a hammer, you tend to see every problem as a nail.”
-- *Abraham Maslow*

This Chapter details the slope monitoring tools and methods employed at PPRust. This includes state-of-the-art technology in automated prism monitoring with GeoMoS, laser monitoring with Riegl, radar monitoring with GroundProbe and micro-seismic monitoring with ISSI. Crackmeters, piezometers and the PPRust monitoring database are also described.

9.1 Introduction

Experience shows that all natural and man-made rock slopes deform with time in response to excavation. The amount of deformation and the rate at which they deform are dependant on the geology, mining method and slope design. With increasing economic pressures in mining, steep slopes are designed more often and it is generally accepted that if no failures occur then the design is too conservative. Slope movement does not need to hinder mining operations if failure mechanisms are understood and slopes are properly monitored. It therefore makes economic sense for every open pit mine to install suitable monitoring systems. This allows for more aggressive slope designs while maintaining safe working conditions for mine personnel. The cost of the monitoring equipment will usually be far outweighed by the extra revenue generated by the steeper slopes and the savings gained from fewer damages and injuries.

The following considerations must be taken into account when selecting the correct monitoring instrumentation and setup:

- Failure mechanism
- Purpose of instrumentation
- Parameters to be measured
- Magnitude of slope movement
- Rate of slope movement
- Size of slope movement
- Location of instrumentation on site
- Budget constraints
- Support provided by instrument supplier

Ideally a monitoring system should be put in place at the beginning of operations, in which case these criteria will have to be predicted by the geotechnical engineers. Once they have been defined and prioritised, the type, number, accuracy and frequency of measurements can be determined. This then determines what instrumentation is best suited to the operation as well as the labour and cost requirements. A compromise may need to be made, for example sacrificing accuracy for range of the instrument. Usually more than one type of instrumentation is used which may compensate for any compromises made.

A slope monitoring programme can also be divided into time frames where different instruments would be applicable. Long term, medium term and short term deformation trends need to be determined. Different parts of a surface mine usually have different slope designs and geology and therefore require different monitoring methods. The area of highest risk is the first priority and instrumentation must suit those areas.

In order to improve safety and mine more economically, a comprehensive slope monitoring strategy has been implemented at PPRust (Figure 9.1). In the last four years four new state-of-the-art monitoring systems have been installed, namely a GeoMoS automated prism monitoring system, prismless Riegl laser scanners, a GroundProbe slope stability radar (SSR) and an ISSI microseismic monitoring system. Groundwater monitoring and visual monitoring have also been improved over the same time period. The slope monitoring strategy is shown in Figure 9.1. The primary monitoring tools are used to identify high risk areas. The SSR is then set up in that area to provide early warning of failure so evacuation can be successfully done. Fault tree analysis has proved that with this comprehensive slope monitoring strategy the geotechnical risk at PPRust is greatly reduced, allowing mining to continue safely and economically in challenging conditions.

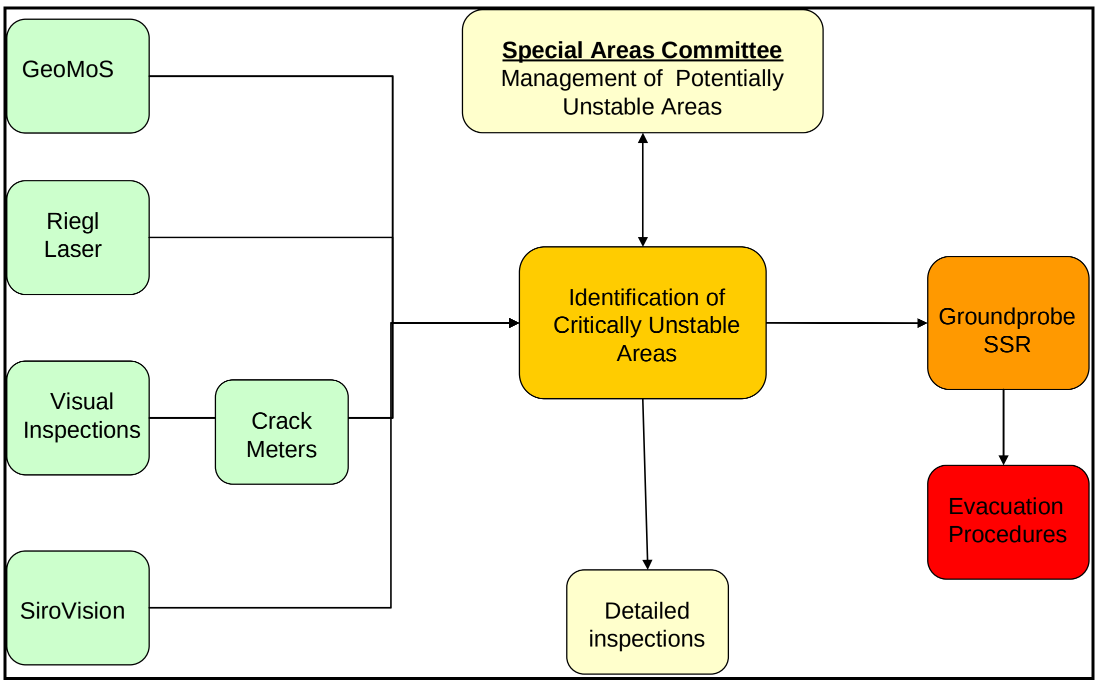


Figure 9.1 Slope monitoring strategy at PPRust (PPRust, 2005)

9.2 Prism Monitoring

The traditional method of slope monitoring which has been used on most open pit mines is prism monitoring. Prisms are installed on the highwalls at a regular spacing, both horizontally and vertically, and on critical areas in the open pit. Surveyors measure and report the prisms position in 3D space on a daily, weekly or monthly basis, depending on the geotechnical requirements. The measurements must be performed at the same time of day so that accurate comparisons can be made because pressure and temperature affect the measurements. The geotechnical engineers then analyse the data, looking for significant movement, and report any potential areas of slope failure to the mining personnel. Manual survey monitoring is time consuming and prone to human error thus automated total stations have been developed in recent years. These are theodolites which are programmed to take readings of selected prisms at selected times of the day and night. They send their data at regular intervals via radio link to an office computer.

There are a number of reasons why it is better to have an automated system:

- Movements occur continuously over time
- Sudden changes in movement need to be detected when they occur
- Online analysis enables real time calculations
- Measurement stations are not always accessible
- Analysis is more reliable with regular scheduled measurements
- Manual labour is expensive and time consuming
- Automatic systems reduce human error
- Efficient, fast and reliable
- Manual measurement is boring

9.2.1 GeoMoS automated monitoring at PPRust

Manual prism monitoring was performed by surveyors from 1997 to 2003 at PPRust. In October 2003 three automated theodolites were installed to save time, increase the number of measurements and improve the accuracy and precision. Prisms are installed on the highwalls at a regular spacing, 50 m horizontally and 45 m vertically, and on critical areas throughout the open pits (Figure 9.2). The prism positions are measured every four hours by the automated theodolites, two of which are permanently set up on beacons on the western and eastern crests of Sandsloot open pit. The third theodolite is set up on the crest of eastern wall of Zwartfontein South but is moved to the western wall at night. The theodolites are housed in steel structures for protection from flyrock, harsh weather conditions and theft (Figure 9.3). The data is sent, as it is captured, via radio link to an office computer in the Survey office where it is stored in the GeoMoS software program (Leica Geosystems, 2005). The Survey Department is responsible for maintaining the theodolites and prisms and for collecting and storing the data. The rock engineers then analyse the data, looking for significant movement, and report any potential areas of slope failure to the mining personnel.

The theodolites are controlled and monitored by Leica's GeoMoS software (Leica Geosystems, 2005) which stores the data in a SQL database on the network. It allows the user to view and filter the data and to plot graphs of displacement, velocity and vector movement of one or many selected prisms. There are three displacement plots – longitudinal (Figure 9.4), transverse and height - for the movement along the x, y and z axes. The vector plot displays and combines these three movements into an absolute movement (Figure 9.5). The velocity plot uses the longitudinal displacement to calculate a rate of movement – or velocity of movement. All the graphs can display raw or smoothed data. Systems errors can be seen by the sudden displacement on all prisms at the same time as seen in Figure 9.4. Atmospheric changes result in an oscillating plot with the effect of the order or 5 mm. To avoid false alarms that this can cause, readings must be compared to the readings taken at the same time e.g. 6am every morning. The data can be smoothed to average out the atmospheric fluctuations. Smoothing is not always the best way to view the data. A survey error can cause an outlier in the data which would ruin the smoothed curve.

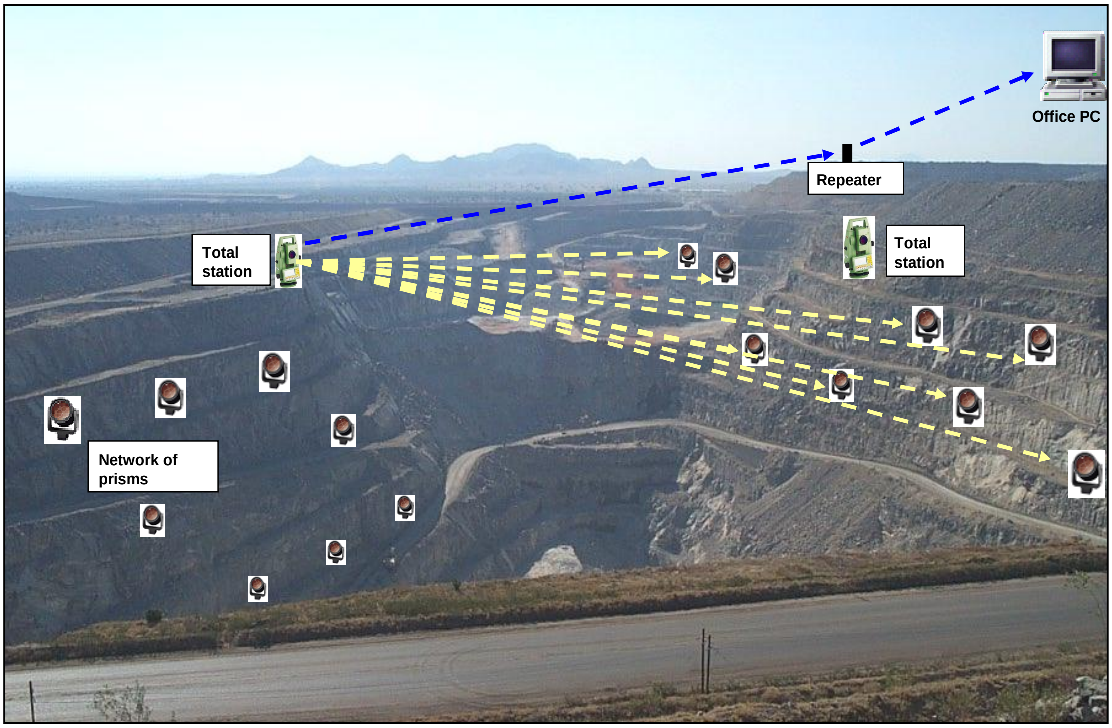


Figure 9.2 Prism network and monitoring setup at Sandsloot open pit


Figure 9.3 Theodolite in a protective steel house

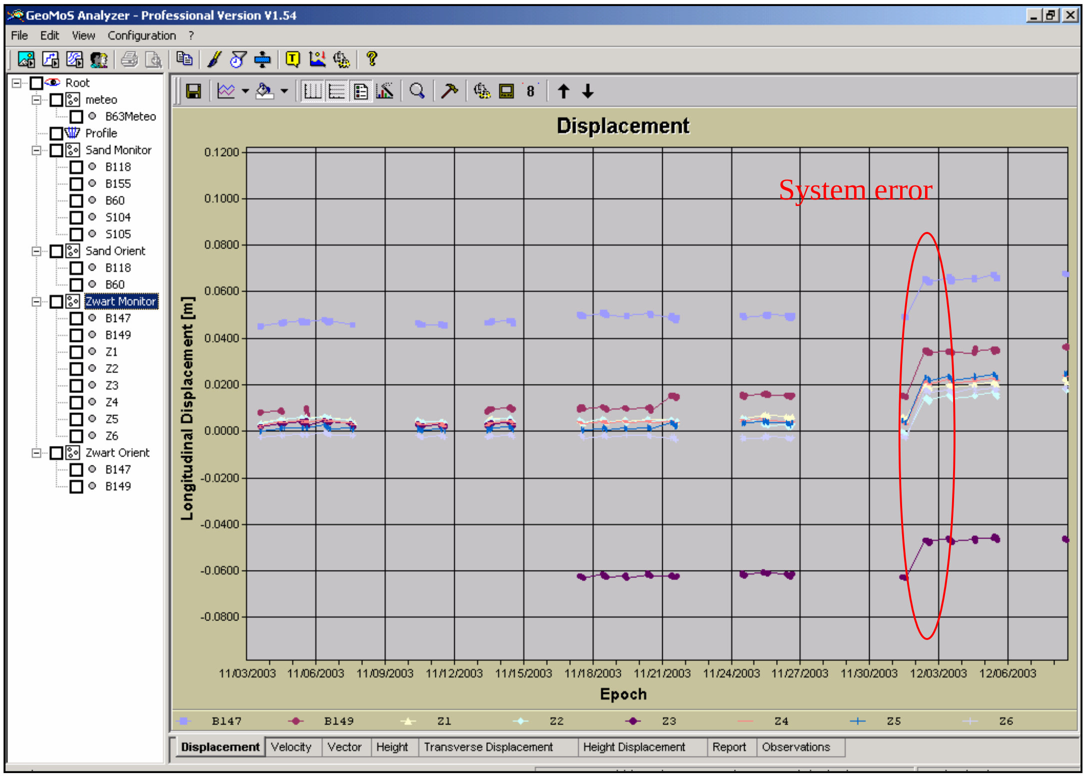


Figure 9.4 Displacement plot in GeoMoS Analyser showing a system error

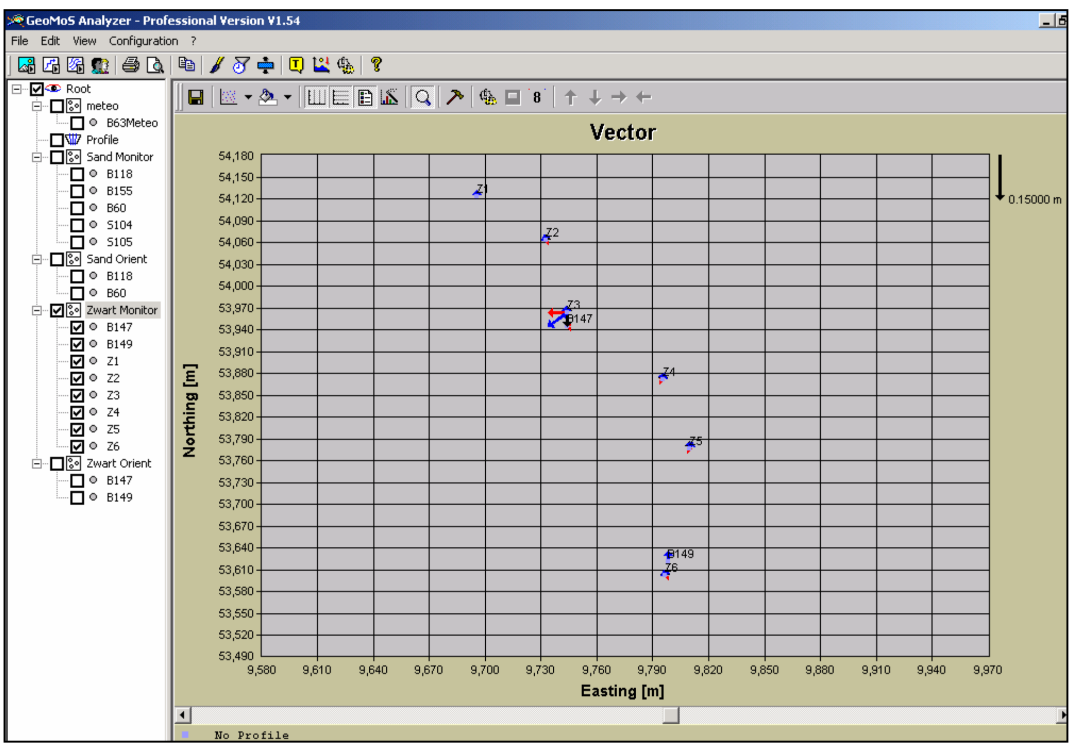


Figure 9.5 Vector plot in GeoMoS

The graphs are plotted automatically whenever new data is received, which enables quick and easy identification of slope movements. Alarms can be set by the rock engineers at site-specific trigger levels and sms's can be sent out to the relevant parties when those levels are exceeded. If an alarm is sounded, the Survey department will check that it is not a result of survey error and then report it to the Rock Engineering department who investigate the area of concern. If the movement occurs in a working area the area will be evacuated until it is declared safe. Extra prisms may be installed or other monitoring devices may be set in place until the slope fails. Alternatively failure could be induced to remove the risk completely so that operations can continue.

It is important that prisms are properly installed otherwise a false alarm might be given due to movement of the prism within the wall. A hole is drilled into the wall with a Hilti drill and a specially made steel bar is grouted in (Figure 9.6). The prism is then firmly attached onto the bar with a steel casing around the prism to protect if from rockfalls and flyrock. The prism is pointed in the direction of the theodolite that will survey the prism. The theodolite must also be firmly secured onto a stable beacon to ensure there is no false movement recorded.

a) 

b) 

Figure 9.6 a) Installation of prisms b) prism with a protective casing

The failures recorded so far on the west wall have occurred very rapidly (under two hours) and the prism monitoring has not been able to provide warning of failure for evacuation purposes. Thus the GeoMoS system is used at PPRust for identifying long term slope movement trends and indicating where slope failure is more likely to occur. The response to slope movement is therefore to declare a Special Area, perform detailed inspections and install other monitoring devices.

Even though prisms are securely installed and a steel protective casing is fitted around the prism, many prisms are damaged or lost due to rockfall, slope failure and flyrock. The west wall in Sandsloot is especially difficult to maintain prism coverage due to the regular bench failures which result in loss of catchment berms and therefore loss of access. The prism installation can also be dangerous and it is time consuming and expensive. Prism monitoring is also limited by the fact that it measures movements of single widely-spaced points on a slope. There is the possibility for a failure to occur between the points without it being recorded. As a result of these limitations, the Riegl laser and GroundProbe radar monitoring systems have been implemented at PPRust.

9.3 Laser Monitoring

In an open pit environment lasers have been used largely for volume calculations and digital imaging. Slope monitoring is a recent development and can be used instead of, or in conjunction with, prism monitoring. As it requires no prisms on the face it solves many the problems mentioned earlier.

The laser scanner operates by sending out narrow pulses or beams of light which reflect off the highwall back to the scanner. A receiver system times, counts and processes the returning light. The time measurement is converted to a distance by using the formula:

Distance = (Speed of Light x Time of Flight) / 2

Laser measurements are affected by (Riegl, 2006):

- Reflectivity of the object - highly reflective objects may saturate some laser detectors, while the return signal from low-reflectivity objects may occasionally be too weak to register as valid.
- Sunlight and Reflections/Angle of Measurement - a strong sunlight reflection off a highly reflective target may "saturate" a receiver, producing an invalid or less accurate reading.
- o Dust and Vapour – scattering of the laser beam and the signal returning from the target. Last-pulse measurements can reduce or eliminate this interference.

Laser measurements are not affected by:

- Day or Night
- Target's Angle of Repose
- Background Noise and Radiation
- Temperature and Temperature Variations

9.3.1 Riegl laser monitoring at PPRust

Prismless laser monitoring was introduced at PPRust in February 2005 to fill in the gaps where prisms have been lost and there is no access. Two Riegl LPM-2K laser scanners (Riegl, 2005) for slope monitoring are permanently installed in steel protective houses on the crest of the eastern highwalls in both Sandsloot and Zwartfontein South open pits (Figure 9.7). The laser scanners have a range of 2 km, are battery operated, require no levelling and are eyesafe under all operating conditions. A camera is attached to the side of the laser and takes photographs at the start of scanning.

Figure 9.7 a) Protective steel house on the east wall crest of Sandsloot pit
b) LPM-2K laser with camera installed in steel house

The lasers are controlled by 3DLM Site Monitor computer software which allows the user to specify monitoring points and frequency as well as group certain points. The exact x, y and z coordinates of specified points are programmed into Site Monitor and these points act as 'virtual prisms'. In Sandsloot these points are spaced 5 m apart, both horizontally and vertically across the entire 2 km long and 100 m deep slope. The wall is divided into 20 zones and the laser scans the points one by one and returns to the first point at 6am every morning (Figure 9.8). It takes nine hours to scan the entire west wall which is 500 m to 1 km from the scanner (depending on the angle). The accuracy is 20-50 mm which is comparable with GeoMoS.

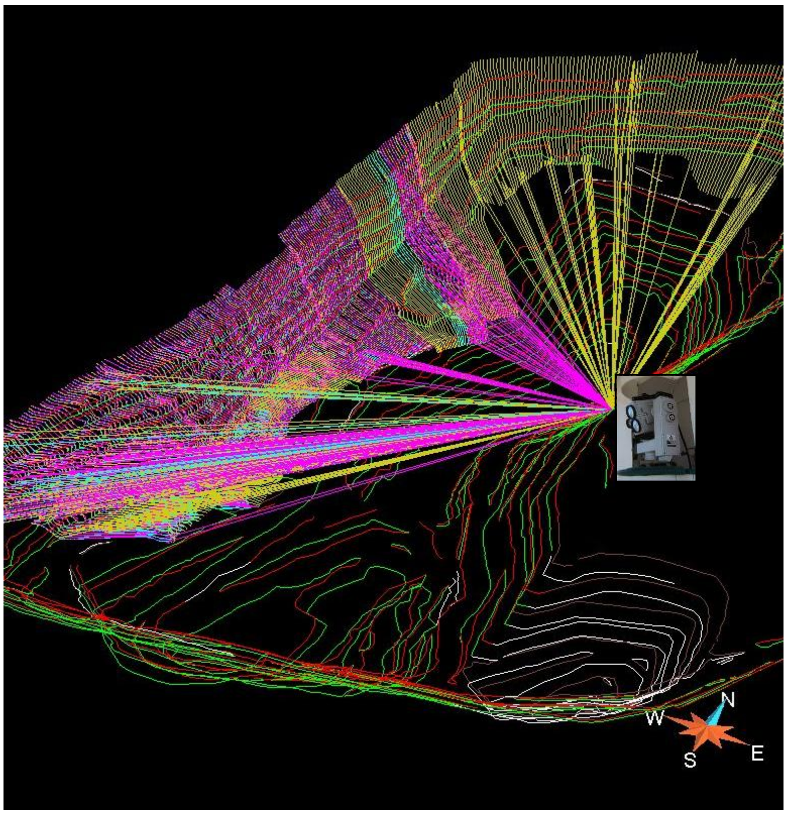


Figure 9.8 Illustration of the laser scanning the west wall of Sandsloot open pit

The laser transmits the data by radio to a computer in the Survey office where the data is downloaded into Site Monitor Analyser and PolyWorks software for analysis. The data appears as a point cloud which can be rotated, filtered and coloured as required. The data can be exported in ASCII format thus can be brought into AutoCAD and Datamine. Figure 9.9 shows a series of nine adjacent scans (blue and green) which are brought in as point clouds into Datamine. A digital terrain model (DTM) is then created in Datamine to better view the data.

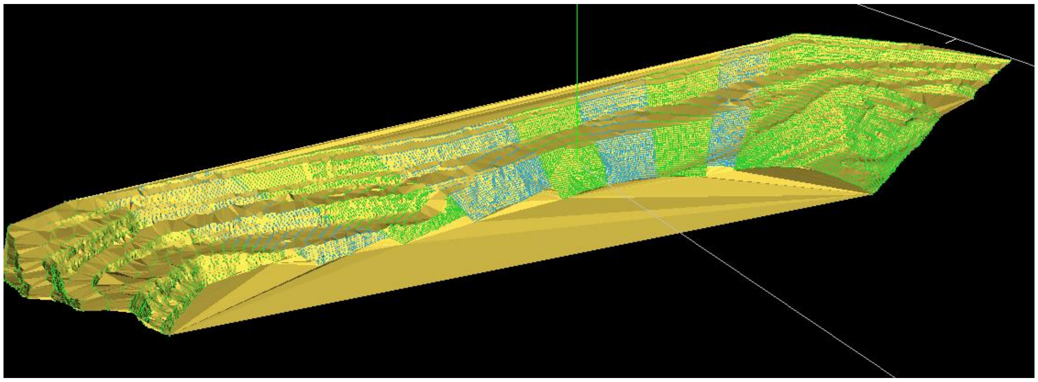


Figure 9.9 Riegl laser point cloud and DTM in Datamine

The point clouds are very useful for volume calculations and visually analysing the state of the pit slope. More importantly, the point clouds are compared, point by point, and progressive slope movement is calculated and plotted (Figure 9.10). A photograph of the scanned region is displayed and the movement is overlaid in various colours. Contour plots can be made of the movement data and alarms can also be set up as with the GeoMoS system.

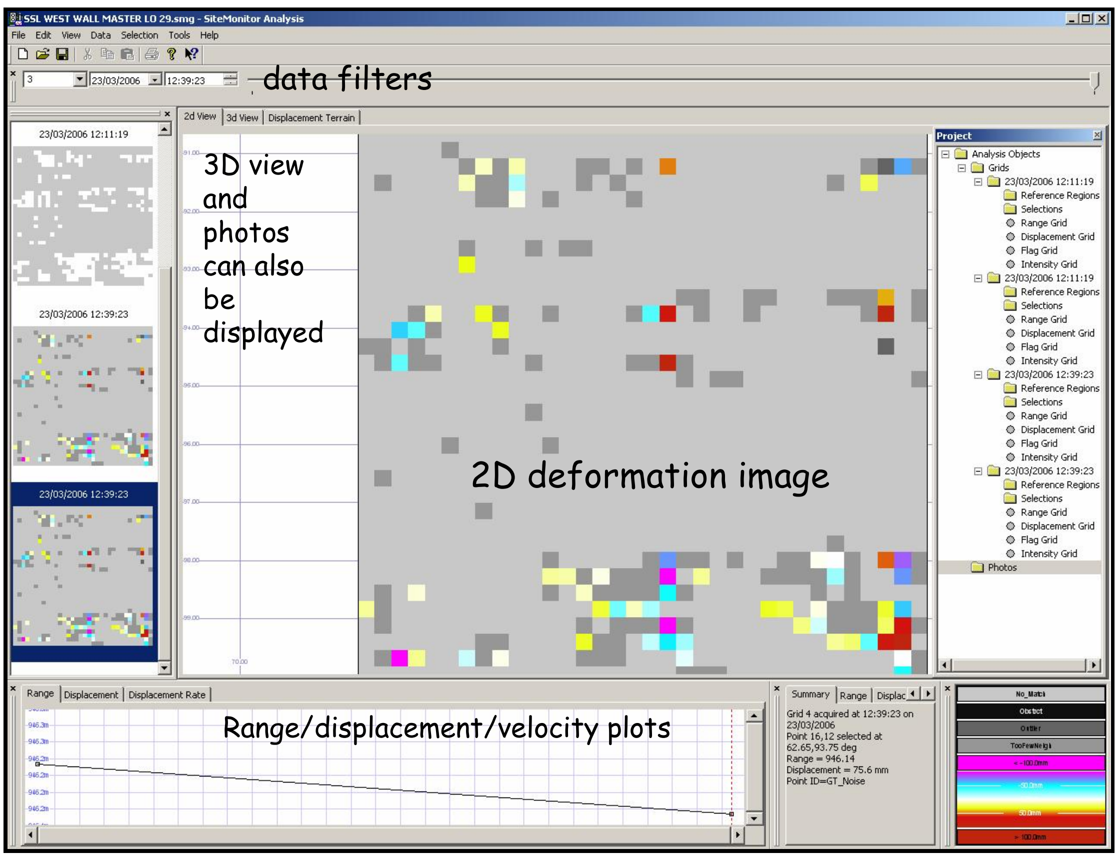


Figure 9.10 SiteMonitor display showing slope deformation over time

The main advantages of using laser monitoring are that it processes large amounts of data and it does not require prisms on the face. The Site Monitor Analyser software was developed by Riegl in conjunction with PPRust which is the first open pit in the world to use lasers for slope stability monitoring. Laser monitoring, however, has the same disadvantage as the prism monitoring in that it cannot provide early warning of failures for evacuation purposes at PPRust as brittle failure occurs in under two hours. It is therefore also used for long term monitoring trends and for identifying high risk areas where the GroundProbe radar must be put in place.

9.4 Radar Monitoring

As discussed, prism and laser monitoring are limited by the fact that the collection and processing of data is slow and measurements can only really be compared once a day. The GroundProbe Slope Stability Radar (SSR) has been developed to provide real-time early warning of slope failure in an open pit.

9.4.1 GroundProbe Slope Stability Radar (SSR)

GroundProbe (GroundProbe, 2005) is a Brisbane-based company that developed the SSR over a number of years and made it commercially available in 2003. The SSR (Figure 9.11) has a 0.92 m parabolic dish mounted on a sturdy tripod and controlled by a radar electronics box (REB). The beam width is ~2 degrees and the dish can be positioned between -15°and 165° in elevation from the horizontal and between -170° and 170° in azimuth. The scan speed is ~12 minutes for 2000 pixels on the wall. The pixel size is determined by the range extent of a 1° angle increment. For a rock slope 100 m away the pixels will be 2 m by 2 m. The radar source produces a frequency of 9.4 – 9.5 GHz thus in South Africa a license from ICASA is required to operate the SSR.

The SSR is a mobile system that can be relocated in roughly an hour. It is also self sufficient as it has a diesel generator on board, at the back of the trailer, which powers the mechanical movements of the dish as well as the electronic equipment. It can therefore be set up anywhere in an open pit as required. The dish is attached to a tripod which is lowered to the ground and jacked up so that the scanning antenna is not connected to the generator thus cannot be affected by its movements. A camera is fixed to the dish and photographs can be taken whenever required. Generally one photograph every two hours during the day is sufficient. This is set on the computer which is situated behind the dish and is controlled by the computer electronics box (CEB). Once setup and turned on, the SSR takes 14 photographs which it converts to a mosaic of the entire area that it can see and scan. The operator then selects a 2D scan area on the slope and scanning begins (Figure 9.12). The scan time is dependant on the size of the slope area selected and the distance from the slope. The range on the SSR is 850 m, however that is doubled if the 1.8 m dish (SSRX) is installed. An atmospheric region is selected on the scan region which is used to compensate for atmospheric disturbances caused by local changes in pressure, temperature and humidity.

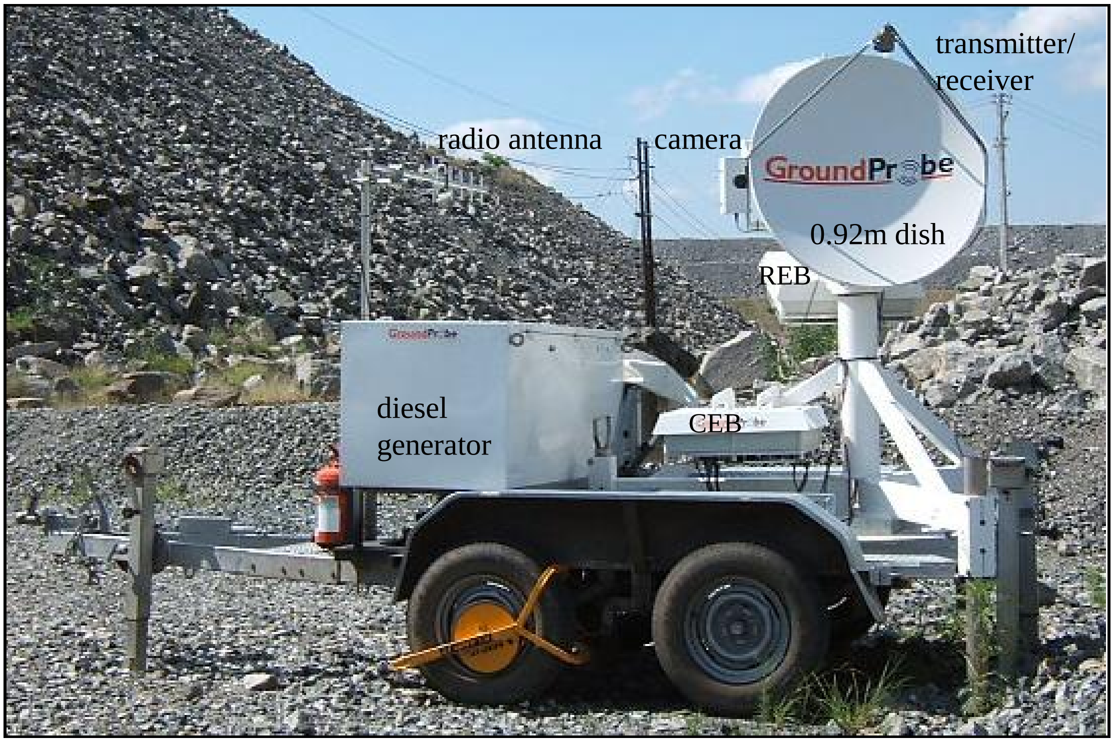


Figure 9.11 GroundProbe Slope Stability Radar (SSR) at PPRust

Figure 9.12 Illustration of SSR scanning technique in Sandsloot open pit

The SSR uses differential interferometry to measure sub-millimetre movements on a rough rock face (Noon, 2005). It does this by comparing the phases of the radar signals it receives from one scan to the next. Any phase difference that it records is converted to a measurement in millimetres. It displays this information on a computer screen as a pixelated 2D image using hot and cold colours to indicate movement (Figure 9.13). The hot colours indicate movement out of the face and cold colours indicate movement away from the face – i.e. rocks have fallen out of the face. The most recent photograph of the scan area is shown next to the 2D image so that the operator can see where the movement is occurring on the slope. The operator can choose what time period (since the scanning started) to view as well as what level of movement to colour the plot on. Any number of deformation versus time graphs can be plotted on any area in the scan to make the interpretation of the data simpler. This will show the deformation history of a particular area. The operator can zoom in on the graphs as well as on the photographs and the deformation plots.

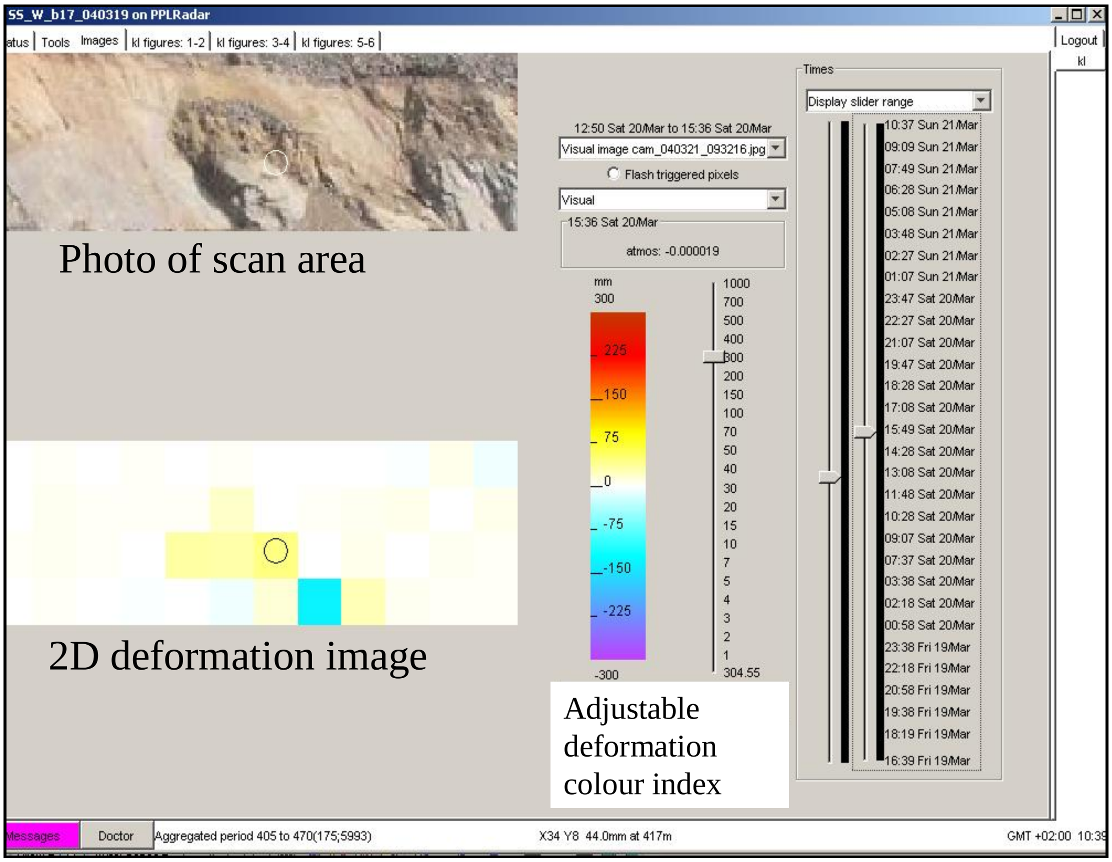


Figure 9.13 SSR viewing screen which shows a photograph of the scanned area, 2D deformation image, colour scale and time scale.

Customisable alarm settings and masking options are also available (Figure 9.14). On the alarm settings, the operator can select the amount and speed of movement that is considered indicative of failure. This must also be determined by experienced geotechnical engineers. The operator can also mask out areas that could cause false alarms for example where trucks and shovels are operating or where loose material is situated. The operator can set red and orange alarms. The orange alarms are there to alert the geotechnical engineers to a potential problem while the red alarm is considered urgent and evacuation of the risky area in the pit usually follows.

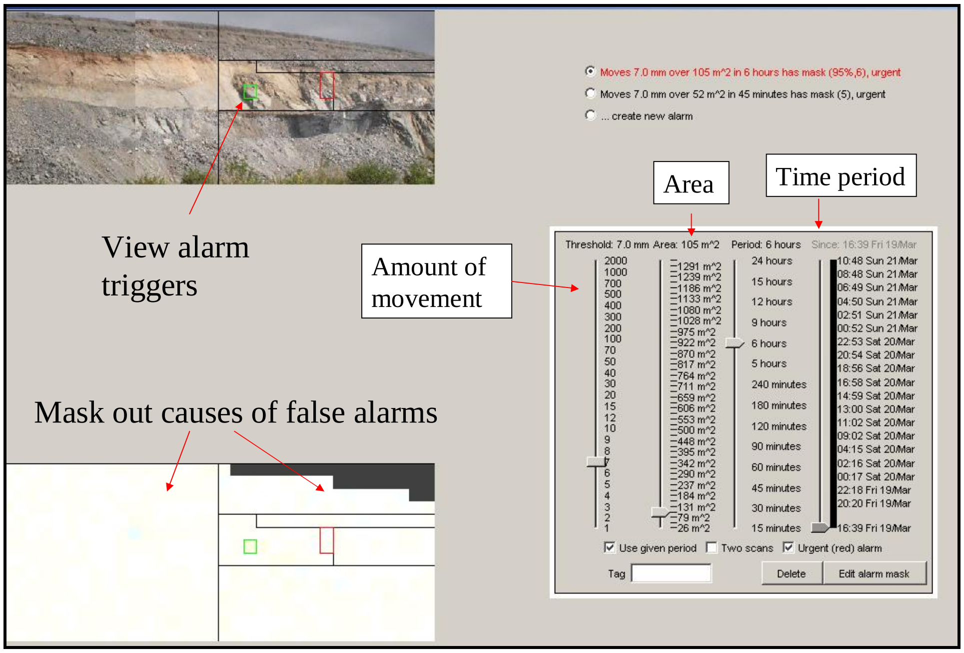


Figure 9.14 SSR alarm settings screen on the computer

When movement does exceed the set limits then an alarm screen appears (Figure 9.15) with instructions on what the operator must do. At the same time, sms's can be sent out to all relevant parties – usually the members of the geotechnical department. Thus the SSR is a real-time early warning device ideal for rapid small brittle failures which would not be picked up with conventional monitoring techniques.

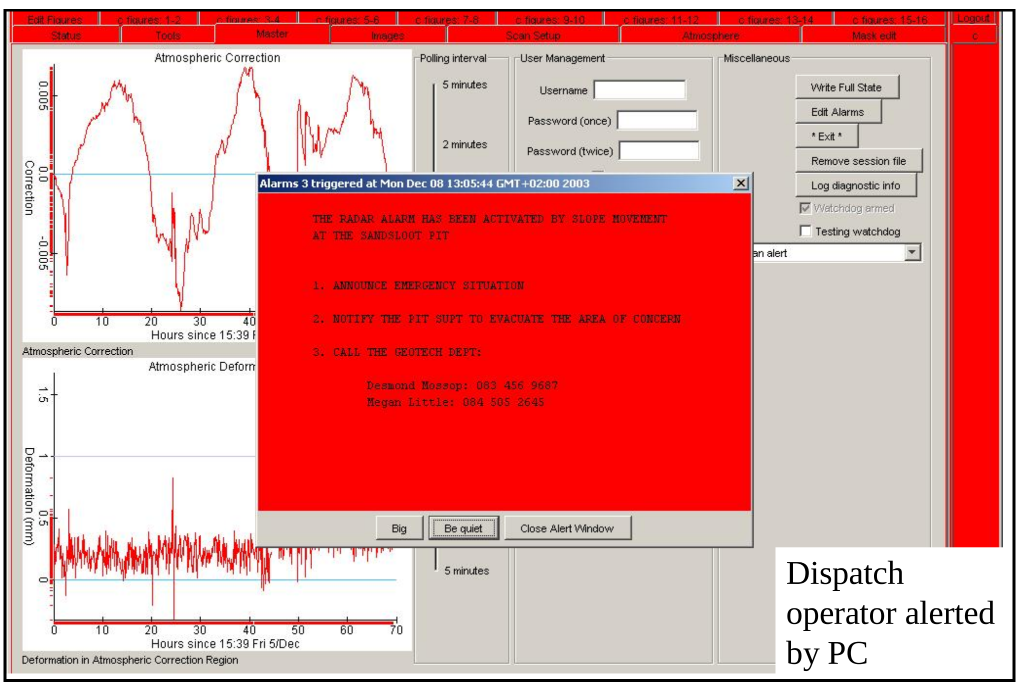


Figure 9.15 SSR red alarm screen displayed when the set limit is exceeded

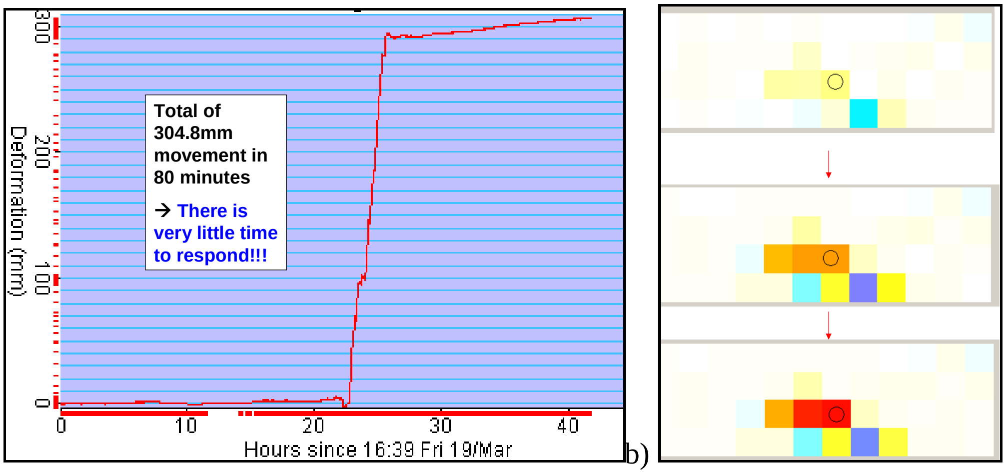
There are four other alarms that can sound. A yellow alarm indicates that no data has been received from the SSR. This could be due to a fault with the SSR or with the radio link to the office. A green alarm indicates a minor fault with the control room computer. A blue alarm merely informs the user that the radar has been relocated and a new data file is in use. A grey alarm indicates there is insufficient data received from SSR.

9.4.2 SSR at PPRust

The GroundProbe slope stability radar (SSR) was implemented at PPRust in November 2003 to monitor the Sandsloot west wall. It was the first SSR implementation outside Australia. Previously recorded failures were of the order of tens to a few thousand tonnes and appeared to occur instantaneously. The SSR scans 70% of the Sandsloot west wall 24 hours a day from a position on the crest of the east wall. Red and orange alarms are set and checked daily. At PPRust rapid brittle movements are monitored thus the maximum movement (red) is set at 15 mm. When movement does exceed the set limits the red flashing alarm screen shown in Figure 9.17 appears with instructions for an evacuation. At the same time, sms's are sent out to all members of the geotechnical department. A siren with red lights is set up in the working area in the pit that is being monitored by the SSR. It is linked to the computer in the Control room and goes off when a red alarm sounds. This ensures that the workers are notified of imminent failure and can evacuate without the communication from the control room.

Eight brittle failures have been recorded in Sandsloot with the SSR and they all show that the slope movement occurs over less than 2 hours. This gives the operations staff very little time to respond – in some cases only 20 minutes. The SSR does provide early warning and people and equipment have been successfully evacuated at PPRust as a result. It is evident that at PPRust the SSR is the only monitoring tool that allows mining operations to continue safely under the high risk west wall in Sandsloot. An example of such a failure is shown in Figure 9.16 and 9.17 below. The SSR measured 300 mm maximum movement, however, the actual failure was a 5 m slip downslope. This difference is a result of the radar's position 150 m away and a few benches below. The measurement of 300 mm is in the direction of that radar position. This is always kept in mind when setting alarms and analysing data. The graph in Figure 9.17a shows a rapid acceleration from a stable wall to failure in just 80 minutes. The deformation plots in Figure 9.17b show the change in deformation that the radar screen portrays. The change in colour makes it easy to see the failure occurring.

Figure 9.16 Photograph taken by the radar of a stack failure on the west wall



a) b)

Figure 9.17 a) SSR deformation versus time plot indicating rapid brittle failure
b) successive deformation images with change in colour showing the progressive movement

Figure 9.18 shows another brittle failure that was monitored by the SSR in Sandsloot. Only a few tonnes failed out of the west wall on Bench 23 but the SSR recorded it and again showed a rapid acceleration where failure occurred in under two hours. In both cases, personnel were evacuated from the pit to ensure their safety. This proved that the SSR is the best tool to monitor the west wall in Sandsloot. The prisms showed no sign of deformation and if a laser scanner had been in place, it would have only indicated movement the next day.

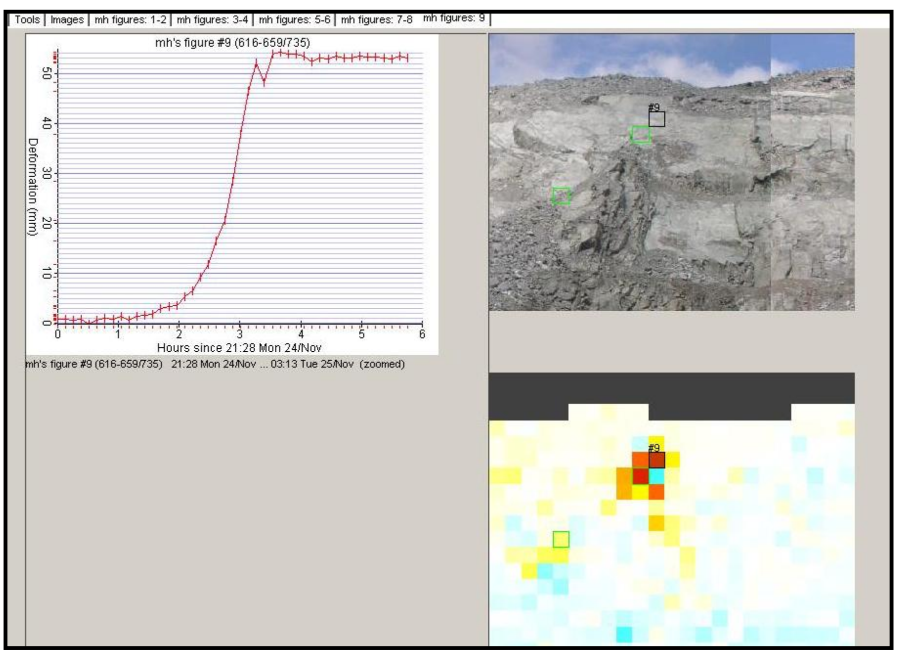


Figure 9.18 Recording of a small brittle failure by the SSR

It is imperative therefore that the SSR is placed in the correct position at all times. When drilling is done adjacent to the west wall the SSR may be moved closer or to a better position. It is also important that alarms are responded to properly. Every time any alarm is triggered it is recorded in a log book in the Control room by the trained Dispatch operators. They are also required to sign acknowledgement of the alarm settings every day. Pit superintendents are trained on the use of the SSR and the functionality of the software. They play an important role in evacuating the pit and ensuring the SSR is given priority in the operation. GroundProbe have set up daily, weekly and monthly maintenance sheets as well as fault reporting sheets which the site staff can email to their main office for investigation and correction. They have 24 hour support and the data from every site is emailed to Johannesburg and/or Brisbane as it is received from the SSRs. This ensures that the availability of the SSR is kept high and response times are kept low.

Over 30 GroundProbe radars are in use at various mines around the world, including Sishen (McGavigan, 2006) and ThabaZimbi (De Beer, 2006) in South Africa. A SSR was successfully used by Nchanga in Zambia (Naismith and Wessels, 2005) to manage a large scale slope failure. GroundProbe is not the only company that has developed a radar system for slope monitoring however they are the leaders in the mining industry. An Italian company, LiSALab, (LiSALab, 2003) has a similar system which they use predominantly for landslide monitoring. They have done some work in a quarry but are focussed on the environmental problems faced in Europe. In South Africa, Reutech (Reutech, 2006) has recently developed a system very similar to GroundProbe's SSR. It is currently in use at a South African coal mine. It is georeferenced and has good survey functionality.

9.5 Seismic Monitoring

The previous sections have looked at tools that monitor surface deformation only whether it be points on the surface or large scan areas. Seismic monitoring aims to predict slope deformation by measuring microseismic events caused by brittle movements within the slope itself (ISSI, 2006). Analysis of microseismic events using multiple geophones enables the location of source and therefore the discontinuity on which movement is occurring. This provides a true 3-D picture of the rock mass unlike the 2-D picture obtained with surface monitoring. Seismic monitoring systems are the norm in underground gold and platinum mines but have only recently been implemented at open pits by ISSI. Boreholes are drilled into the pit slopes and two geophones are installed in each hole (Figure 9.19). The geophones must form a 3-dimensional spread of seismic sensors around the area in the slope that is being monitored. The geophones record all microseismic movements down to 0.004 mm. These sensors are connected to a StandAlone QS data logger at the top of the hole where the data is stored in a hard disk, powered by solar panel. It is collected or sent to an office computer via radio link. The seismograms (Figure 9.20) of common events are post-associated for off-line processing and interpreted by seismologists at ISSI. Standard reports and 3D processed data are produced on a monthly basis. Increased seismic activity can provide early warning of slope failure and trends in the data can potentially identify weak failure planes. The ISSI system has been implemented with varying degrees of success. Navachab open pit found that the system gave them six weeks warning of a large slope failure prior to prism movement (Lynch *et al*., 2005). It is therefore a long-term monitoring system which aids in the understanding of weaknesses in a rock mass.

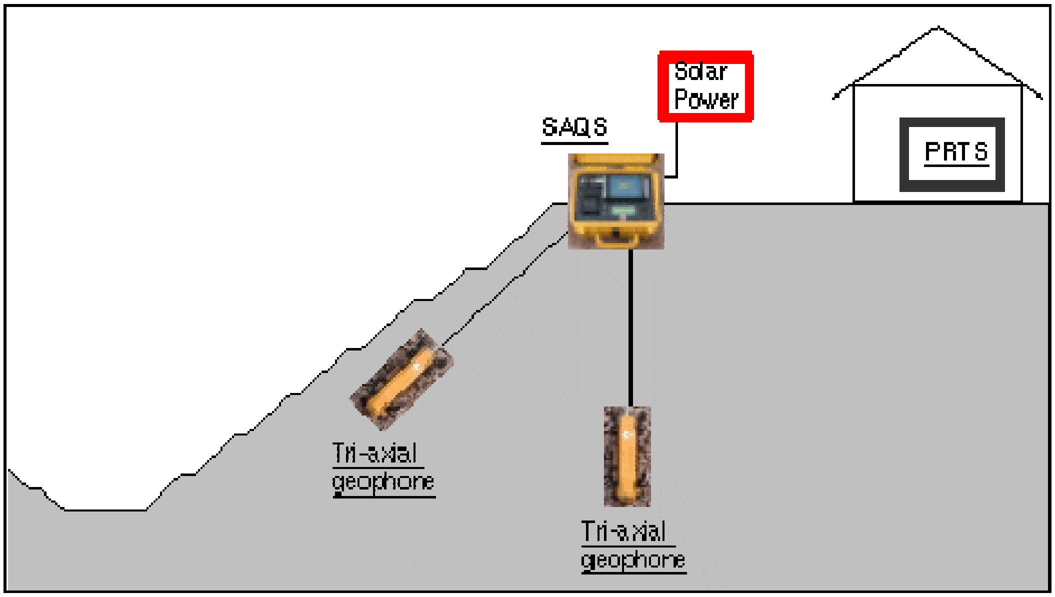


Figure 9.19 Sketch showing the setup of a seismic network in an open pit (ISSI, 2006)

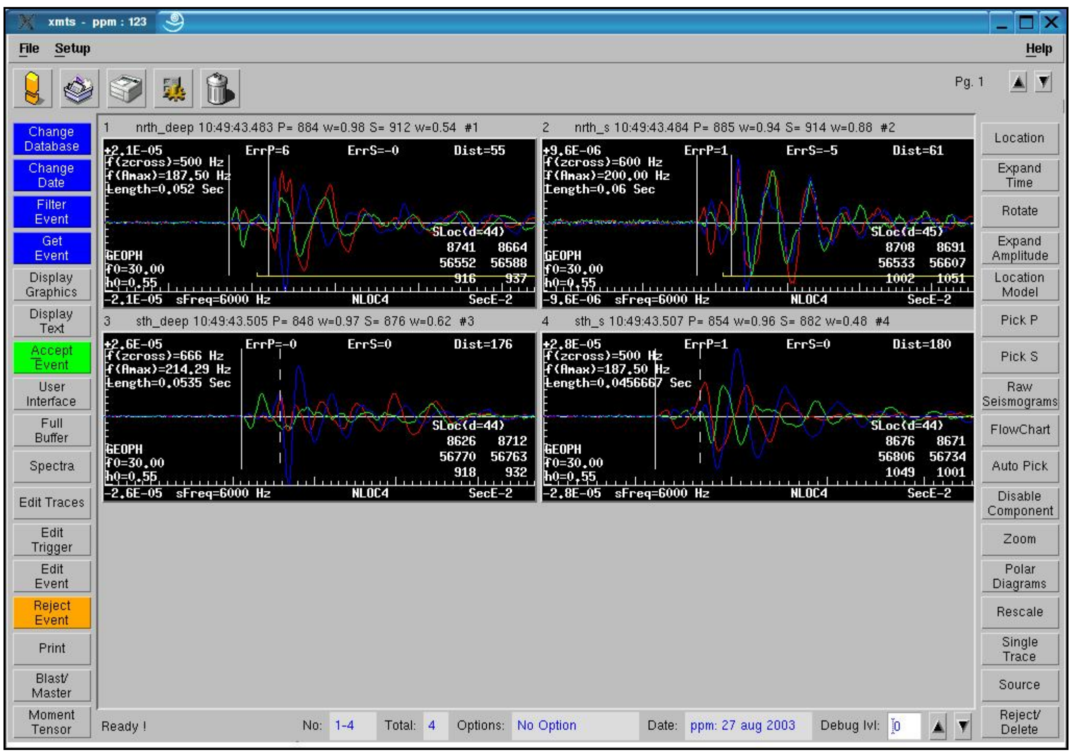


Figure 9.20 Example of the seismic traces of one seismic event, measured with four ISSI geophones.

9.5.1 ISSI microseismic monitoring at PPRust

A 4-geophone ISSI system was installed in the eastern slope of Sandsloot in May 2003. Monitoring began on a trial basis with the view to provide long-term monitoring of the slope above the permanent footwall access ramp. Seismic monitoring continued until June 2005 and monthly reports were issued by ISSI. Monitoring ceased as less than 10 microseismic events were being recorded a month and the rock engineers agreed that the wall was very stable and did not warrant the expensive ISSI system.

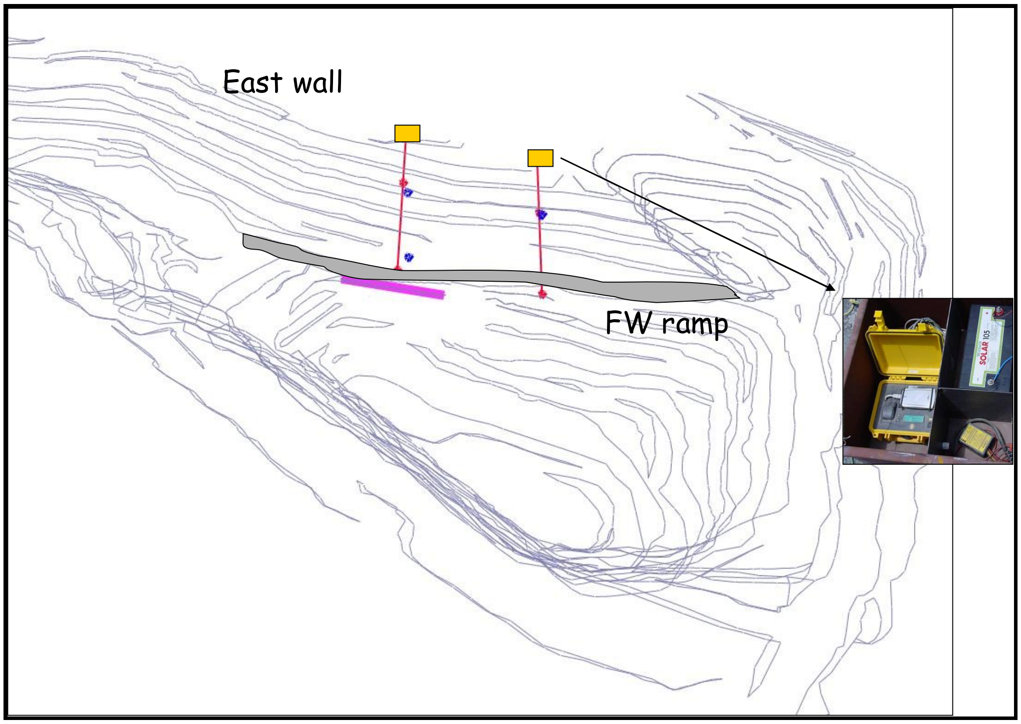


Figure 9.21 ISSI setup in Sandsloot open pit east wall above the gabion ramp.

9.5.2 Navachab case study

Navachab gold mine is situated 200 km east of Swakopmund, Namibia. They were the first open pit to successfully install a seismic monitoring system in 2000. The installation was a result of concern that popping noises were heard in the slope immediately after blasting. Their system initially consisted of four triaxial sensors in a tetrahedral array in their east wall. They have increased coverage by installing eight more sensors. In October 2002 to January 2003 a rapid increase in seismic events occurred (Figure 9.22) which corresponded to mining activity at the bottom of the east slope. Conventional prisms monitoring of the same area also showed early warning but it only began in December 2002. This represents a time lag of 45 days thus the seismic system was shown to provide extremely early warning of slope deformation and failure, which occurred at the end of December 2002.

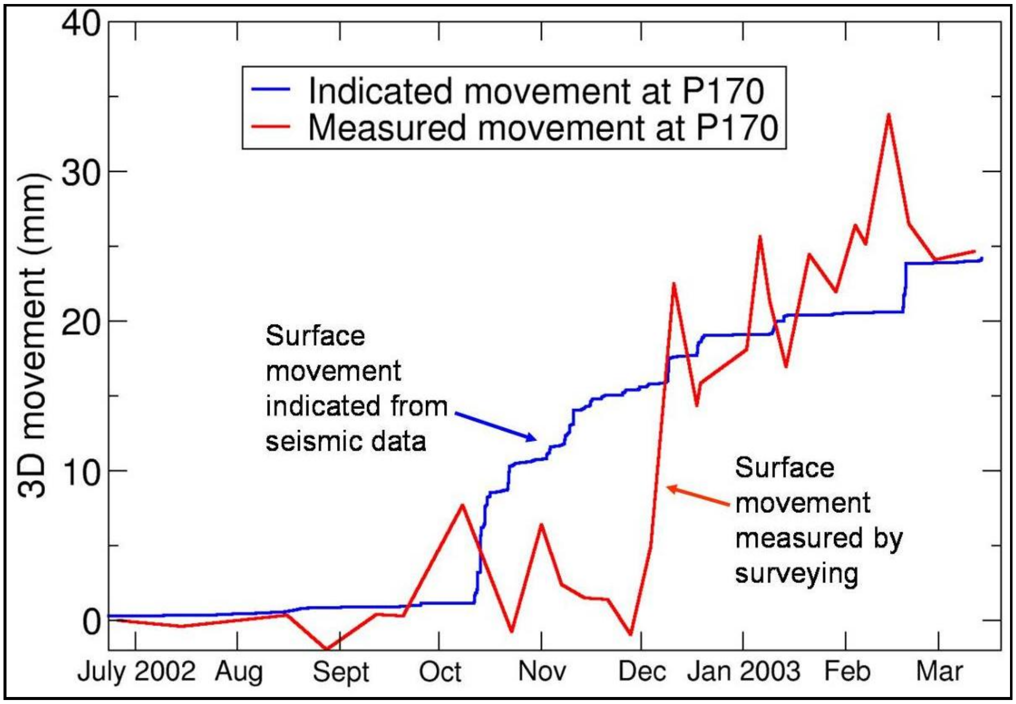


Figure 9.22 Graph showing early warning given by the seismic system at Navachab (Lynch *et al*., 2005)

The potential benefits of using the ISSI system in an open pit mine are:

1. Indicates scale and timing that mining activities are affecting slope stability
2. Indicates known and unknown seismically active geological structures
3. Infers relative surface movements 30-45 days before movements are seen on the surface. This delay depends on rock properties and the locations of the seismic events.

9.6 Groundwater Monitoring

PPRust receives about 450 mm of rainfall a year. The groundwater flow is from NE to SW and is structurally controlled. It therefore does not pose a major slope stability problem but is very difficult to manage. Piezometers were installed around Sandsloot in 2005 to monitor the groundwater levels. Rainfall gauges are checked daily and water level readings are taken once a week.

9.7 Crackmeters

The formation of tension cracks at the crest of a slope may be the first visible sign of instability (Hoek and Bray, 1981). Numerous types of crack meters and surface extensometers are available to measure this movement. They range from crude and simple devices that require daily visits and manual measurements to sophisticated electrical devices that sound a warning in the office or via sms whenever critical movement occurs. Borehole extensometers are used to measure progressive opening of cracks within a slope. They are excellent tools for seeing what is happening within a rock slope but they are expensive and difficult to install and maintain.

Very simple extensometers were built at PPRust to provide automatic warning to workers below an unstable tension crack. A battery-powered alarm light and siren are attached to a trip-wire that extends over the tension crack and is secured on the other side. If the tension crack widens, the wire is pulled and the alarm goes off. Crackmeters built by GroundWork are more commonly installed over tension cracks at PPRust as they are more reliable than the extensometers. They simply consist of a metal ruler and wire held tautly between two metal pegs which are inserted into the rock on the other side of a tension crack (Figure 9.23). Each day the Rock Engineering technical assistant records the measurement on the ruler and informs his superiors of any movement.

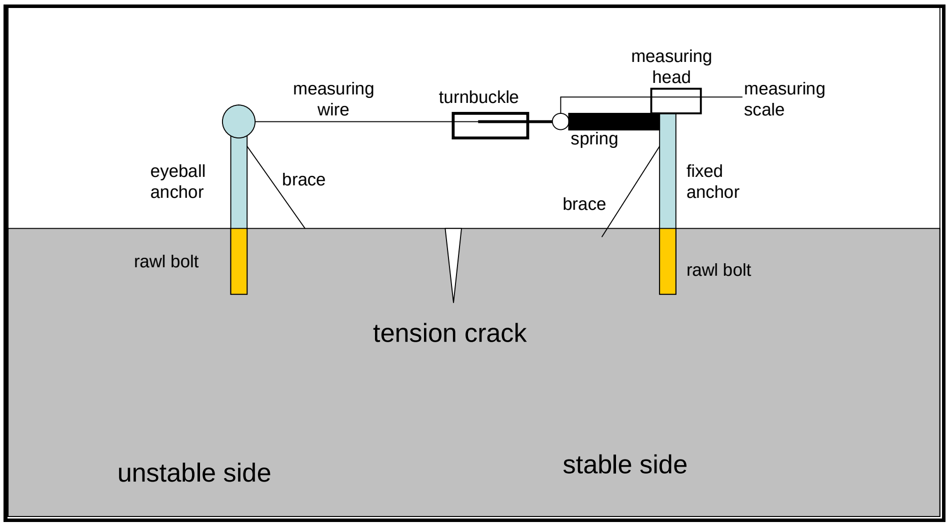


Figure 9.23 Simple crack meter installation

9.8 Slope Stability Monitoring Database

In 1999, a MS Access database and user front-end was designed and implemented by Peter Nathan of CGSS, at the request of the geotechnical and survey departments. It was called SSMON (Slope Stability Monitoring) and stored the manual prism monitoring data, piezometer data and extensometer data (Figure 9.24). In 2004 it was upgraded to include the GeoMoS prism monitoring data, rainfall gauge measurements and crackmeter data. The data is stored in a number of linked MS Access databases on the network with data for each year for each pit stored separately. The user can select which pit to query and which year/s to view. Profiles, or cross-sections through the pit, can be defined in SSMON and data viewed per profile. SSMON is linked to the mine's draughting package, AutoCAD, which displays the movement vectors on cross-sections or 2D plans. In this way the data can be overlaid on slope designs and compared with other monitoring data. This has improved understanding of the nature and deformation of the rock mass thus enabling better predictions of future slope failure.

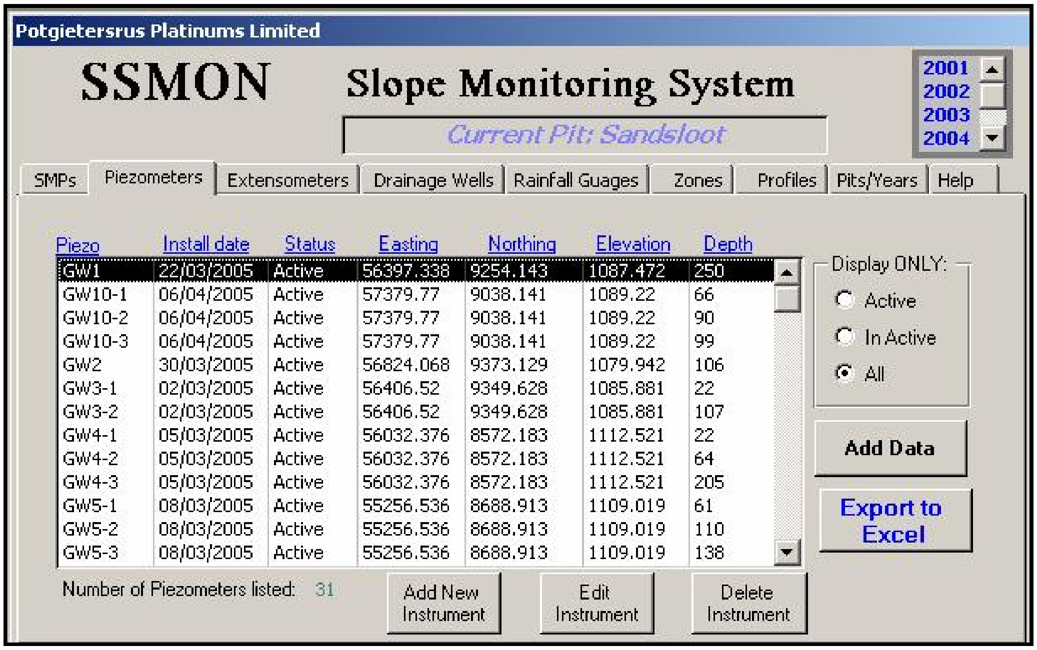


Figure 9.24 SSMON slope monitoring database front-end

9.9 Conclusion

From 2003 to 2005 a comprehensive slope stability monitoring programme was been implemented at PPRust. The prism monitoring was automated in 2003 using GeoMoS to analyse the data and produce alarms. Due to the brittle failures on the west wall in Sandsloot many prisms were lost and many areas could not be accessed thus the Riegl laser system was installed in 2005 to measure ‘virtual prisms’ on that area. Due to the rapidity of the failures, it was determined that the GroundProbe slope stability radar (SRR) was the only monitoring device that could provide early warning of failure on the west wall. Eight failures have been recorded with the SSR since November 2003. They show that the pit personnel can have as little as 20 minutes warning before failure occurs. The ISSI microseismic monitoring trial was not as successful and was terminated in June 2005 after two years in which no significant movement was seen. Table 9.1 summarises the main features of each monitoring system. Piezometers were installed in January 2005 to better monitor the groundwater and rainfall is measured in gauges. Crackmeters are installed over tension cracks to visually monitor the slope crest stability. Much of this data is stored in a MS Access database, SSMON, which was developed for the mine. PPRust aims to use all available, relevant monitoring technology at its disposal to ensure that the risk posed by deforming rock slopes is kept to a minimum. With this slope monitoring strategy PPRust can continue to operate safely in challenging geotechnical conditions.

Table 9.1 Comparison of slope monitoring techniques

Monitoring tool	Analysis Frequency	Operational time	Measurement type
GeoMoS	Once a day	4 hours a day	True 3D vectors
Riegl Laser	Once a day/week	9 hours a day	In direction of laser
GroundProbe Radar	Every few minutes	24 hours a day	In direction of radar
ISSI seismics	When event occurs	24 hours a day	3D location in space

10 SLOPE OPTIMISATION

"Human beings are perhaps never more frightening than when they are convinced beyond doubt that they are right."

- Laurens van der Post

This Chapter describes the risk in an open pit environment, the rewards that steeper slope designs can produce and the method of finding the balance between the two. Slope optimisation that has been done for the Sandsloot and Zwartfontein South open pits at PPRust in the last two years. This incorporates all the work previously discussed and shows how geotechnical information ultimately benefits the mining operation.

10.1 Introduction

In Chapter 7 the slope design methodology for PPRust was explained and illustrated in Figure 7.5. At the start of an open pit an initial design is chosen based on the available geotechnical data and analysis, economic constraints and safety requirements. The geotechnical data for the pit area is restricted to core logging and rock testing though inferences can be made from nearby open pit information. Once mining is underway in the new open pit, mapping, inspections and slope monitoring data is collected. Further core logging and rock testing will also be done and added to the geotechnical databases. With new information at hand, the geotechnical engineers must perform further slope design analyses which will be more accurate. The geotechnical risks in the pit will also become more evident and can be better incorporated into the slope design. This is best done with the fault tree analysis method. The risks are compared with the rewards so that management can make an informed decision. The cost of a slope failure includes clean-up, haul road repairs and re-access, lost production, unrecoverable ore and damage to equipment and infrastructure (Lilly, 2002). This process of improving the slope design based on new information and changing conditions is called slope optimisation. Ultimately, the slope design must maximise NPV whilst meeting Anglo American's stringent safety requirements.

10.2 Risk

Risk is defined as the probability of occurrence of a certain event multiplied by the consequence of that event. In open pit geotechnics, the event is slope failure and the consequences can be loss of life, injury, damage to equipment, loss of production or, in the worst case, mine closure. The probability of failure is calculated using various slope analysis methods, as described in Chapter 7. There are four approaches to dealing with risk: tolerate, transfer, eliminate or reduce probability and/or impact. Often it is not possible to remove the risk completely but it will be possible to minimise and manage it. That is the approach taken at PPRust where failure on geological structures in the open pits cannot be designed out but must instead be well managed.

Probability is usually expressed as a number from zero (no risk) to one (failure guaranteed) or as a ratio e.g. '1 in 1000'. Most people do not understand what the term 'probability' means, especially small probabilities. Cole (1992) defined categories for risk to life, property and money to make them easier to understand. These are shown in Table 10.1.

Table 10.1 Risk categories (after Cole, 1992)

	Annual likelihood of total loss		
Degree of risk	**To life (fatality)**	**To property (destruction)**	**To money (bankruptcy)**
Very risky	1 in 100	1 in 10	1 in 1
Risky	1 in 1,000	1 in 100	1 in 10
Some risk	1 in 10,000	1 in 1,000	1 in 100
A slight chance	1 in 100,000	1 in 10,000	1 in 1,000
Unlikely	1 in 1,000,000	1 in 100,000	1 in 10,000
Very unlikely	1 in 10,000,000	1 in 1,000,000	1 in 100,000
Practically impossible	1 in 100,000,000	1 in 10,000,000	1 in 1,000,000
Overall rate	1 in 95	1 in 2000	1 in 130

Risk of loss of life is used as it is ‘stark and unambiguous’, however, for each fatality there are at least ten serious injuries and large financial losses (Hambly and Hambly, 1994). This must always be kept in mind when dealing with risk to loss of life. Quantifying risk is not more useful if it is compared to a standard risk level.

10.2.1 F-N curves

One method of comparison of risks is f-N and F-N curves, the latter being the cumulative form of the former (Christian, 2004). These curves are graphical representations of the relationship between the annual probability of an event and the number of fatalities it causes. The economic loss can also be included on the curves but this is not the norm. Figure 10.1 is an example of a f-N curve given by Baecher (1982), plotting mine pit slopes, foundations and dams amongst other work environments. The vertical axis shows the annual probability of fatality while the horizontal axes have both the economic loss and loss of human life. In this example the risk on mine pit slopes is very high. Today the true value is thought to be 1 in 600.

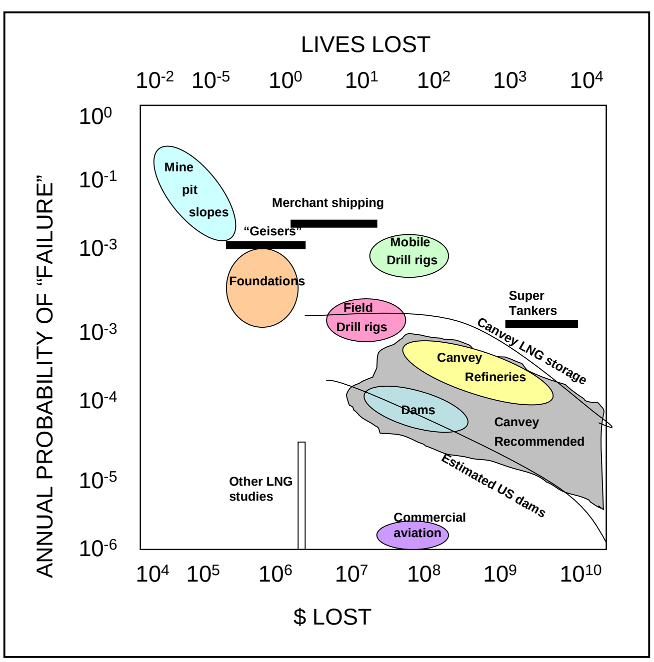


Figure 10.1 f-N plot of annual risk cost or number
of lives (after Baecher, 1982)

Figure 10.2 is an F-N curve showing the acceptable risk levels for fatalities for large dams. These risk levels are used in the civil engineering industry as standards and have been employed for comparison at PPRust. The number of fatalities is compared to the annual probability of occurrence of loss of life.

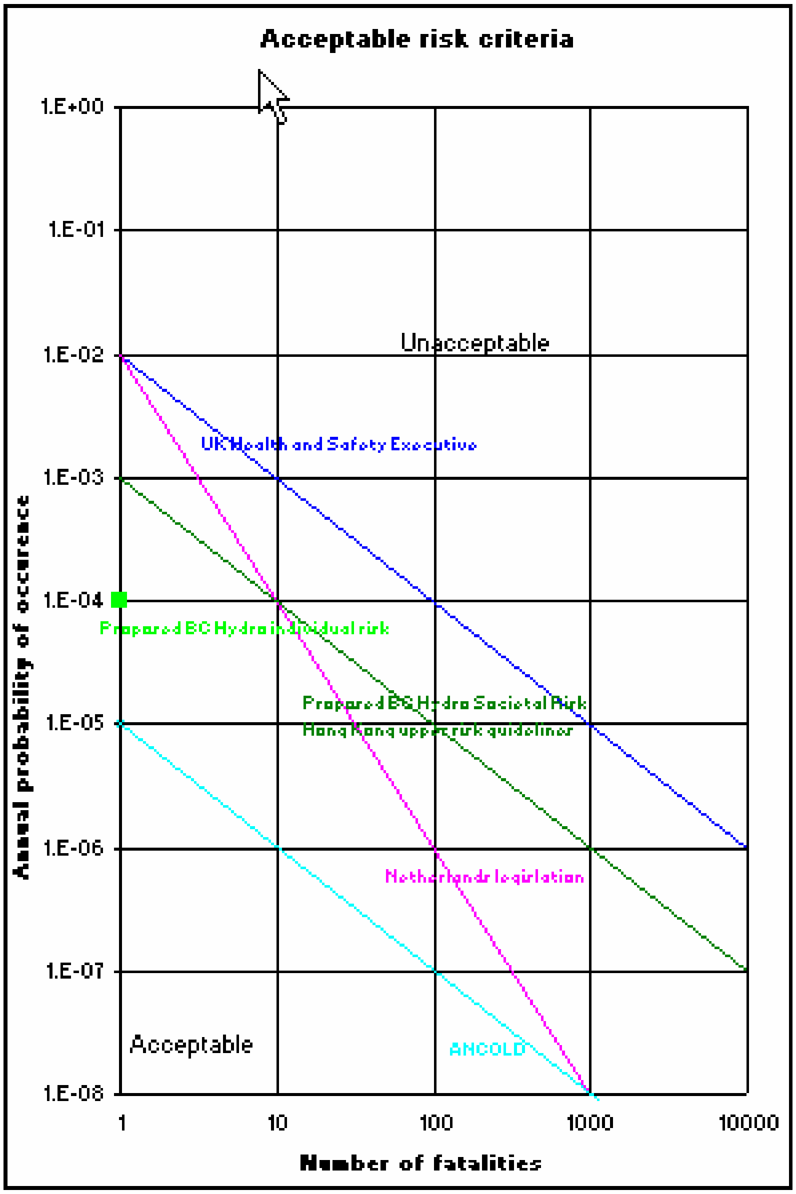


Figure 10.2 Published acceptable risk levels for large dams

10.2.2 Anglo American standard

Anglo American, and therefore Anglo Platinum, policy is to not expose employees to greater risk than they experience at home. The accepted risk level of a normal 12 year old child in the USA has a risk of loss of life of 1 in 10 000 (or 0.01%). This has been adopted by Anglo American as its acceptable level of risk for all operations. Whether this standard is realistic is up for debate. Mines are generally seen as high risk work environments and every employee knows and accepts that. Cole (1992) refers to work by Starr, Rudman and Whipple (1976) where they state that "... the individual exposed to an involuntary risk is fearful of the consequences, makes aversion his goal, and therefore demands a level for such involuntary risk exposure as much as 1000 times less than would be acceptable on a voluntary basis". If this is true one could argue that the risk level required by Anglo American is stricter than is necessary. PPRust has adopted this acceptable risk level however and all slope designs are now based upon it.

Regular audits are done by Anglo American on geotechnical risk at all the operations to ensure compliance to the acceptable risk level. The areas that are considered are:

- Pit layout
- Failure history
- Water management
- Blasting performance
- Rock mass strength versus design
- Rockfall potential
- Geotechnical data levels
- Geotechnical design method
- Monitoring systems
- Evacuation effectiveness

Each of these is assigned a rating out of ten for both impact and likelihood and plotted on a matrix (Figure 10.3). They are combined to give an overall rating which is also plotted on the matrix. This immediately highlights where the problem areas are and what needs to be prioritised. The improvements made at PPRust in all of these areas in the last four years have reduced the risk to the extent that PPRust now has the lowest geotechnical risk in the Anglo American group. This risk status is reviewed on an annual basis by Anglo technical Division.

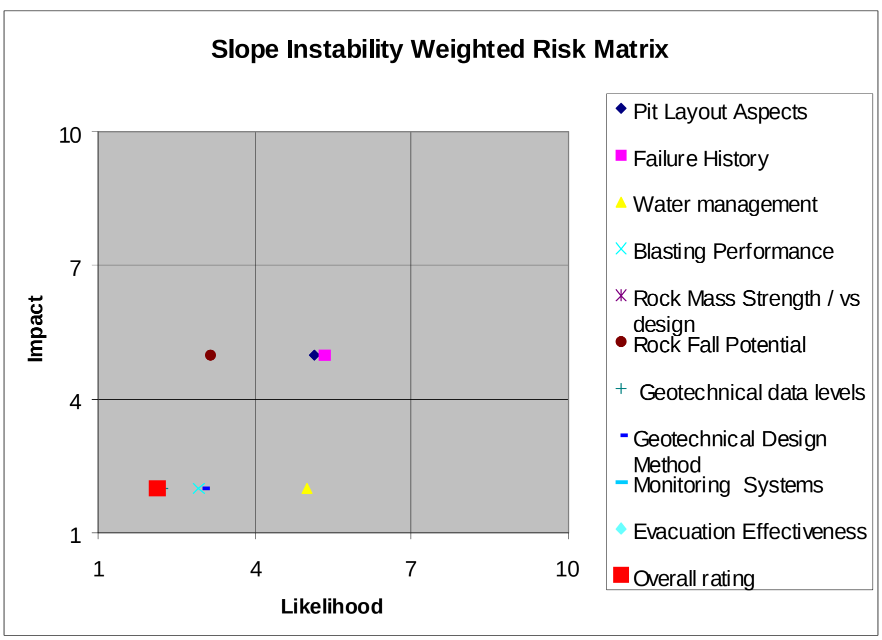


Figure 10.3 Anglo American risk matrix for PPRust

10.3 Fault and Event Tree Analysis

10.3.1 Theory of fault and event tree analysis

Geotechnical risk analysis follows five basic steps (after Anon, 1999):

1. Identify unsatisfactory/undesirable events (e.g. fatality)
2. Estimate their probability of occurrence over some time frame
3. Estimate their consequences
4. Estimate change in probability and consequence associated with alternative plans of improvement (e.g. use radar)
5. Make decisions based on risk versus cost and benefits

Event trees and fault trees can be used to do this analysis and this technique is a crucial element of the slope optimisation done at PPRust. As explained in Chapter 7, an event tree is a graphical framework that is used to account for and estimate the probability of occurrence of an undesirable event such as a fatality, damage to equipment or economic loss. The framework illustrates the sequence of events, starting with an initiating event, and attempts to consider all subsequent possibilities leading to the undesirable event (Figure 11.4). Each event branches into two or more mutually exclusive events. Probabilities of occurrence are assigned to each event by a group of experts. This group would include the mining operation managers, geotechnical department staff and geotechnical consultants. The Total Probability Theorem (Papoulis, 1984) is used to obtain a probability of the undesirable event or outcome. The theorem states:

Given *n* mutually exclusive events A1, ..., An whose probabilities sum to unity, then

P(B) = P(Ai).P(B | Ai)

where *B* is an arbitrary event, and P(*B* | Ai) is the conditional probability of *B* assuming Ai.

The theorem uses Bayes' rule that states:
For >1 possible branches in an event tree
P(outcome) = P(branch1) + P(branch2) +…+ P(branch*n*)

So in other words:

P(outcome) = sum of P(branches) = sum of (product of P(events))

Therefore the individual event values are multiplied to get the path outcome and the paths are added to get the overall outcome.

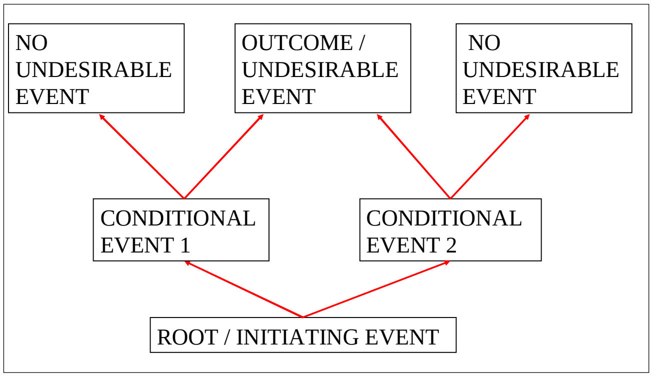


Figure 10.4 Simple event tree framework

A fault tree is an alternative to an event tree – an outcome of interest is first identified (e.g. fatality) and one then works backwards to identify necessary antecedent events. This method saves time and is easier to do but is not as good for economic analysis. Fault trees and event trees are powerful tools as they highlight where the problems are and how one can reduce the risk of an event as serious as a fatality. Their greatest asset is that they quantify the geotechnical risk and enable management to make informed decisions on the level of risk they are willing to operate at.

10.3.2 PPRust fault tree analyses

At PPRust, detailed fault tree analysis was performed for Sandsloot and Zwartfontein South open pits. The fault trees were designed to calculate the probability of a fatality as a result of bench, stack or overall slope failure. The design of the fault tree and the probabilities assigned, were agreed upon by a team of experts - the rock engineers and operations managers at PPRust, the head office consulting geotechnical engineer and SRK consultants. The probabilities of failure determined in the FlacSlope and Slide analyses, described in Chapter 7, were used as design probabilities, or failure under normal operating conditions. In order to take abnormal operational conditions into account probabilities were assigned to over-mining, poor blasting, heavy rainfall, change in rock geological conditions and seismic events (Figure 10.5). A probability of occurrence (Po) of the abnormal conditions and a probability of it causing failure (Pcf) were estimated for each condition. A probability of occurrence was also assigned to failure under normal operating conditions. The final operating probability of failure is calculated by adding the products (Po * Pcf) for each condition.

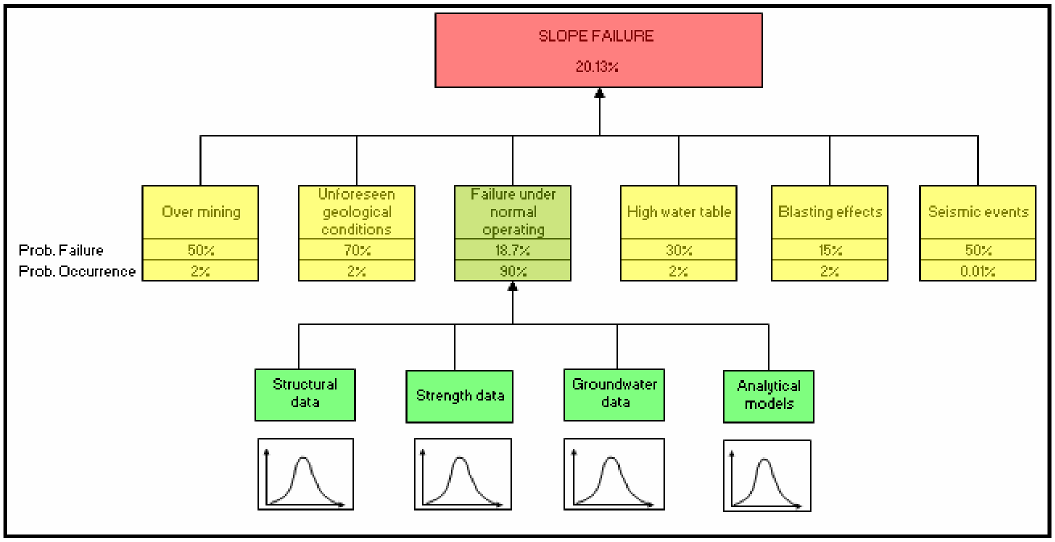


Figure 10.5 determination of the operating probability of failure

This probability of failure was then transferred to the fault tree shown in Figure 10.6. This fault tree shows the sequence of events that occur once a slope failure begins. The primary monitoring includes GeoMoS, laser and visual inspections. If an area of instability is identified then either the area is evacuated or the radar is positioned to monitor that area. If the radar sounds an alarm for imminent failure then evacuation begins. If not, the awareness of the staff may allow for evacuation without alarm. For each event, the probability of the effectiveness and ineffectiveness (their sum being one) was assigned. If any of the steps in the process are ineffective then fatality will occur if people were working in the area. The calculations show that each event is important to the overall risk and steps had to be taken to improve the less obvious aspects such as evacuation drills and awareness. Evacuation drills are now done on a six-monthly basis at PPRust and there has been such an improvement that it now takes under 15 minutes to evacuate the pit. Awareness has been improved by including the slope stability concerns and slope management techniques in the induction video. It has also been presented to all mining personnel on all levels.

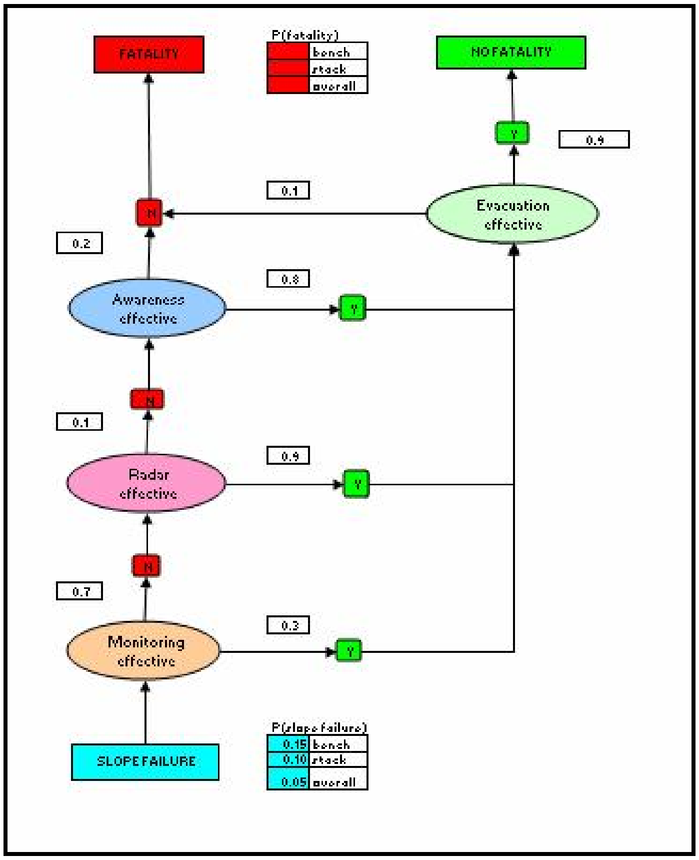


Figure 10.6 Fault tree design for PPRust open pit

Loss of life is dependant on people actually working in the risky area at the time of failure. The annual human exposure is therefore incorporated into the final probability of fatality calculation in the fault tree. It is calculated according to the following equation:

$$\text{Annual human exposure} = \frac{\text{(No. of people * No. of hours per day * No. of days)}}{\text{(Av. lifetime exposure in hours * No. of people)}}$$

Table 10.2 shows the method of calculation of manhours per day at PPRust. The mine schedule and availability of drill rigs was taken into account.

Table 10.2 Exposure calculation for PPRust mining conditions

Occupation	**Staff**	**Hours/Day**	**Total Man Hours**
Driller	18	17	306
Foreman	4	17	68
Mine Supt.	1	0.5	0.5
Surveyor	3	1	3
Drill Supt.	1	3	3
Maintenance	1	2	2
Geologist/Geotech	1	0.5	0.5
Total	29	41	383

The exposure is then multiplied by the probability of fatality to obtain the true probability of fatality in the open pit. This value is then compared to the acceptable risk level of 0.01%.

Various fault tree scenarios were run to determine the effect that improved monitoring techniques as well as improved evacuation, dewatering and wall control would have on mitigating risk to personnel. In particular, the effect of the GroundProbe slope stability radar was studied. Results showed that the radar significantly reduced the safety risk thereby making it an essential part of the slope monitoring and risk management programme at PPRust. It also highlighted the importance of primary monitoring and efficient evacuations. Without any high-tech equipment an open pit mine can significantly reduce risk merely by improving the evacuation technique and visual inspections, both of which are inexpensive. Human exposure, and thus probability of fatality, can also be reduced by intelligent mine planning and the use of safer mining equipment.

An economic event tree was also designed for damage to equipment and production delays which cause loss of profit, and force majeure and is shown in Figure 10.7.

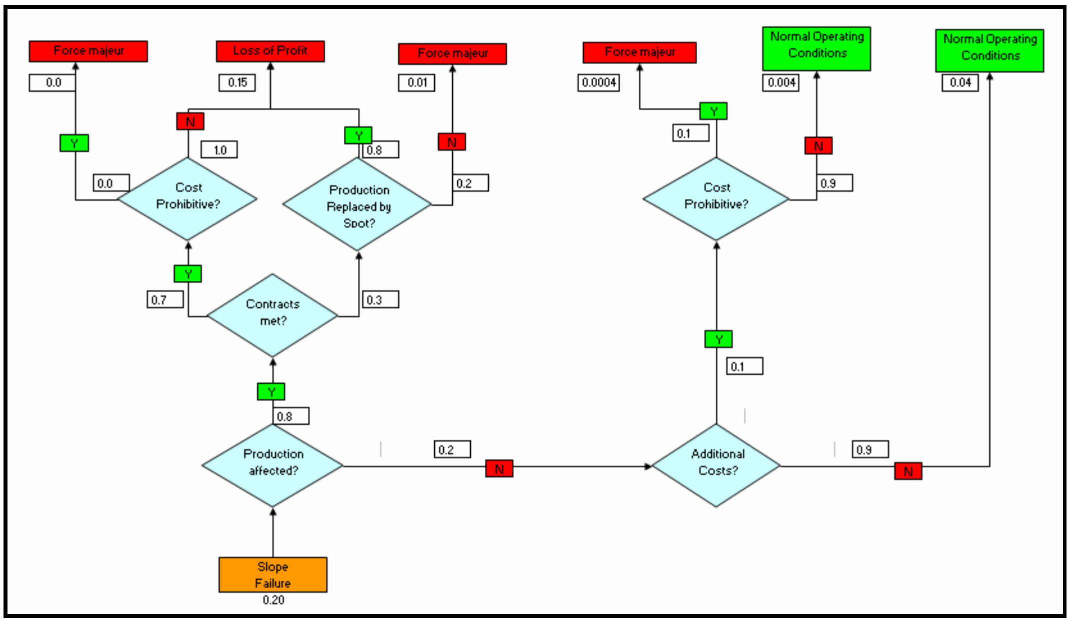


Figure 10.7 Economic event tree for PPRust

10.3 PPRust Slope Optimisation

Most of the steps in the slope optimisation process have already been described in detail in previous chapters. By using the SABLE and MineMapper3D databases, geotechnical field and lab data can be validated and modelled within a week of being collected. This allows for a very up to date block model, and slope designs can be re-assessed on a regular basis. The database improves the quality of the data as a single standard is used, reducing human error and the impact of the high turnover of staff members that is common in the industry. Slope design is based on a number of parameters which all have a degree of variability. The more data there is available and the higher the confidence in the accuracy of the data, the lower the risk. By being able to view the logs in Datamine in their correct spatial location, errors and anomalies can easily be spotted. Corrections can then be made to the logs while the borehole core is still accessible. Slope analysis can also be done regularly and with a greater degree of confidence. Understanding the impact of blasting on highwalls aids in understanding the failure mechanisms and stability of the slope. Visual inspections of highwalls and presplits and detailed inspections of failures contribute to this understanding as does slope monitoring. The GroundProbe radar has shown how quickly failures can occur and Riegl laser and GeoMoS prisms monitoring are expected to measure stress relief over time. All of this work serves to reduce the geotechnical risk inherent in slope design.

10.3.1 Sandsloot Cut 6 optimisation

In 2005 a full slope optimisation was run for Sandsloot's final cutback, Cut 6, on the western wall. In 2004, the overall slope had been flattened from 50° to 44° due to a number of bench and double bench failures. The ramps were widened from 30 m to 45 m and the bench berm configuration changed from 15 m bench/ 9 m berm to 15 m bench/ 12.6 m berm (Figure 10.8). The 60 m stacks were designed at 55°. This design reduced the overall slope to 230 m, losing three benches of ore. As the operations team improved their highwall cleanup and limit blasting, it was evident that the slope could be mined steeper and the ore regained.

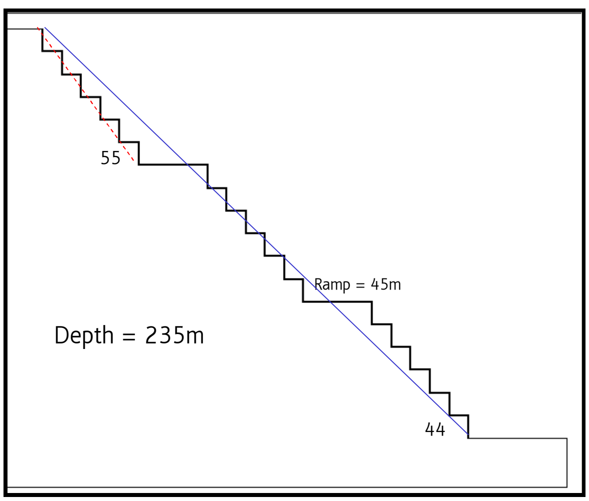


Figure 10.8 Conservative 2004 slope design for Sandsloot west wall

From the previous analytical work it was clear that bench failures would occur during blasting but could be easily cleaned up and did not pose a big risk. Also, the overall slope was deemed to be stable and the chance of failure relatively low. Stack failure proved to be the highest risk and therefore it became the focus of the study, with stacks of 60 m chosen. Six scenarios, C1 to C6, were considered with the stack angles varied from 45° to 70° at 5° intervals. These limits were chosen as the flattest failure planes are 45° and the steepest practical slope is 70°. With ramps set at 30 m, the corresponding overall slope angle for each option was calculated. Inter-ramp angles and angles for the remaining 120 m of Cut 4 (Bench 29 to 50), which would also adopt the design, were also determined per option. These angles were plotted at their respective slope heights and stability curves for each option were drawn as shown in Figure 10.9. The stability limit curve based on slope analysis in FlacSlope and Slide was also plotted on the same chart. This closely matched the 65° stack indicating that it was the ideal stack design. This option had a corresponding slope angle of 50°. The 70° stack stability curve is way above the limit and therefore too unsafe, while the other stacks are all far below it and therefore too conservative. The risk of failure must also be converted into a risk of fatality to determine whether they fall within the risk limits. The fault trees were run for two main scenarios – with and without Special Areas. The Special Areas imply that the slope stability radar is monitoring the area all the time. Each slope design option was used in the fault tree and the results were plotted in Figure 10.10. This shows that with the radar in place, mining can operate safely at the 65° stack.

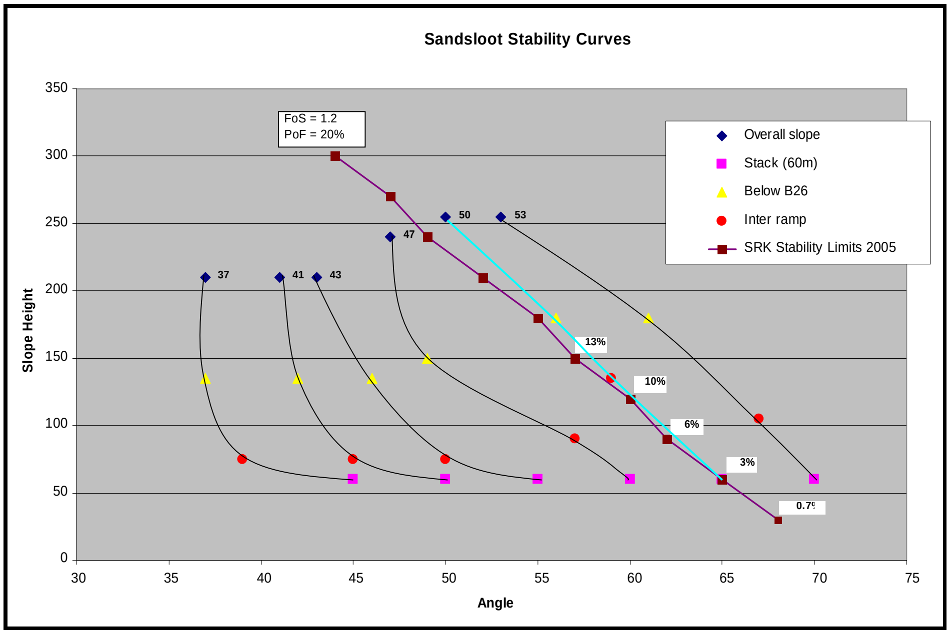
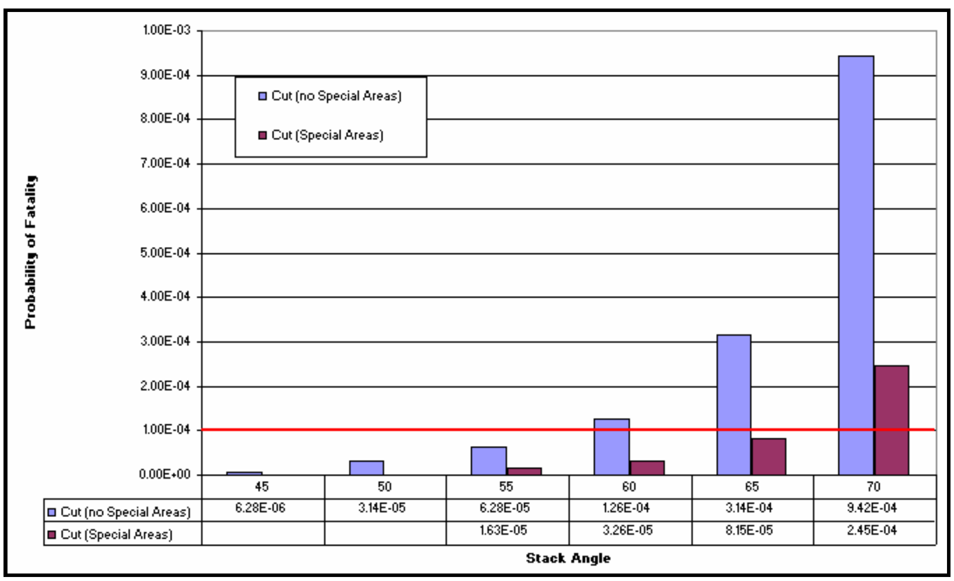


Figure 10.9 Stability curves for Sandsloot's Cut 6 overall slope, inter-ramp slope and stack and Cut 4 slope below Bench 26



	45	50	55	60	65	70
Cut (no Special Areas)	6.28E-06	3.14E-05	6.28E-05	1.26E-04	3.14E-04	9.42E-04
Cut (Special Areas)			1.63E-05	3.26E-05	8.15E-05	2.45E-04

Figure 10.10 Risk evaluation for Sandsloot Cut 6 west wall

It is not enough to know the stability and risk of the slope design scenarios. The economic viability of each option must also be analysed. To do this, the NPV for each one of the six scenarios was calculated by the long term planners using Whittle4D software which utilises the overall slope angle. Other inputs are grade, tonnage, basket metal price, R/$ exchange rate, plant recovery, plant costs and mining costs. Before the NPV runs were done, the mining cost was recalculated incorporating a 'cost of failure'. This 'cost of failure' includes clean-up, slope remediation, haul road repair, equipment redeployment, unrecoverable ore, damage to equipment, rock engineering staff and pit dewatering. The extra costs for each option are shown in Table 10.3 which indicates that the cost of failure increases as the slope angle increases. This is because the probability of failure increases. Support costs were considered and were found to increase rapidly at stack angles greater than 60°. The high costs as well as the logistical difficulty, production delays and unsafe working conditions that installing the support would cause resulted in the decision to not support the west wall

Table 10.3 Break-down of remedial costs of slope failure for each design scenario

	Cut 6 C1	Cut 6 C2	Cut 6 C3	Cut 6 C4	Cut 6 C5	Cut 6 C6
LoM	3.95	4.5	4.8	5.35	6.02	6.31
Stack Angle (60m)	45	50	55	60	65	70
Failure Volume	7,900	22,500	240,000	535,000	752,500	946,500
Clean-up Cost	R 237,000	R 675,000	R 7,200,000	R 16,050,000	R 22,575,000	R 28,395,000
Slope Remediation	R 790,000	R 2,250,000	R 4,800,000	R 8,025,000	R 12,040,000	R 13,882,000
Haul Road Repair	R 395,000	R 4,500,000	R 14,400,000	R 21,400,000	R 30,100,000	R 37,860,000
Equipment re-deploy	R -	R 100,000	R 300,000	R 400,000	R 500,000	R 600,000
Unrecoverable ore	R -	R 82,567,000	R 165,135,000	R 198,162,000	R 247,703,000	R 330,271,000
Damage to Equipment	R -	R 5,000,000	R 10,000,000	R 20,000,000	R 30,000,000	R 35,000,000
Additional Rock Eng Costs	R -	R -	R -	R 5,703,100	R 6,417,320	R 6,726,460
Pit Dewatering			R 2,000,000	R 2,000,000	R 2,000,000	R 2,000,000
Total Costs	R 1,422,000	R 95,092,000	R 203,835,000	R 271,740,100	R 351,335,320	R 454,734,460
Additional Mining/Clean-up costs	R 58,272,000	R 58,272,000	R 65,556,000	R 72,840,000	R 87,408,000	R 87,408,000
Expected Costs (PoF)	**R 58,273,422**	**R 58,462,184**	**R 67,594,350**	**R 78,274,802**	**R 104,974,766**	**R 155,618,169**
Ancillary Anchor Costs	R -	R -	R -	R 5,703,100	R 6,417,320	R 6,726,460
Anchor Costs	R -	R 2,250,000	R 18,000,000	R 25,000,000	R 90,000,000	R 240,000,000
Total Anchor Costs	R -	R 2,250,000	R 18,000,000	R 30,703,100	R 96,417,320	R 246,726,460

Even though the cost of failure increases with slope angle, the NPV still increases because waste stripping costs far outweigh them. Figure 10.11 plots the tonnes of ore mined against the stripping ratio for each option. The 65° stack is shown to have the lowest stripping ratio. The total NPV, as well as the contributing factors, was plotted for each scenario (Figure 10.12). It is evident that for a stack angle greater than 65°, the profit margin does not increase at the same rate as for flatter angles. This indicates that the 65° stack is the optimum design with an estimated profit of R752 million for the remaining life of Sandsloot. This is R139 million more than the profit obtained for the previously designed 55° stack. This is assuming that recoveries are mill constrained i.e. limited by the current mill throughput. The new plant at PPRust North may eliminate the constraint and increase the profit dramatically to R1.3 billion.

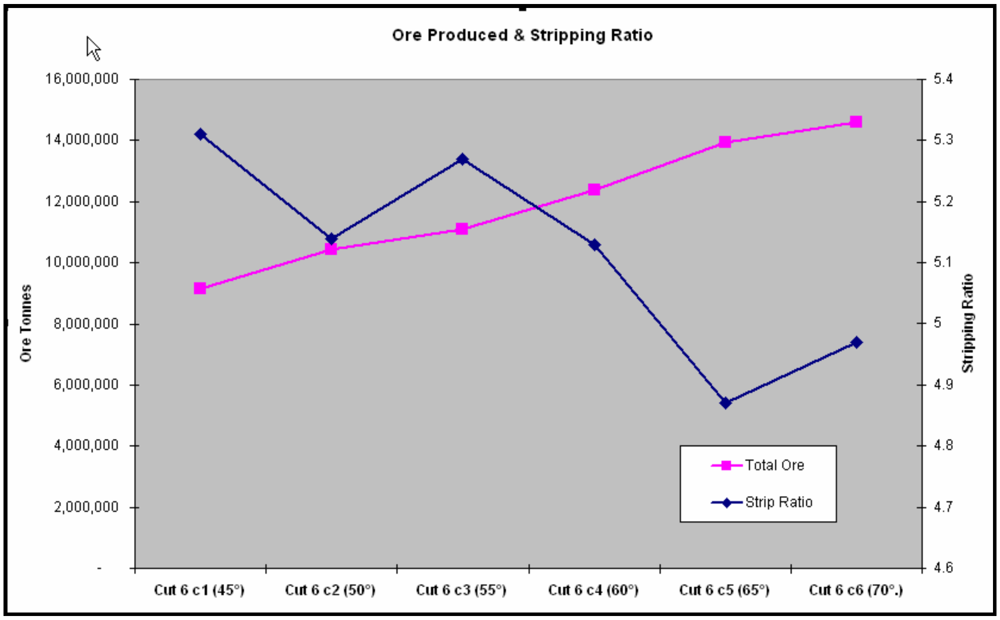


Figure 10.11 Tonnes of ore versus stripping ratio for the slope angle options

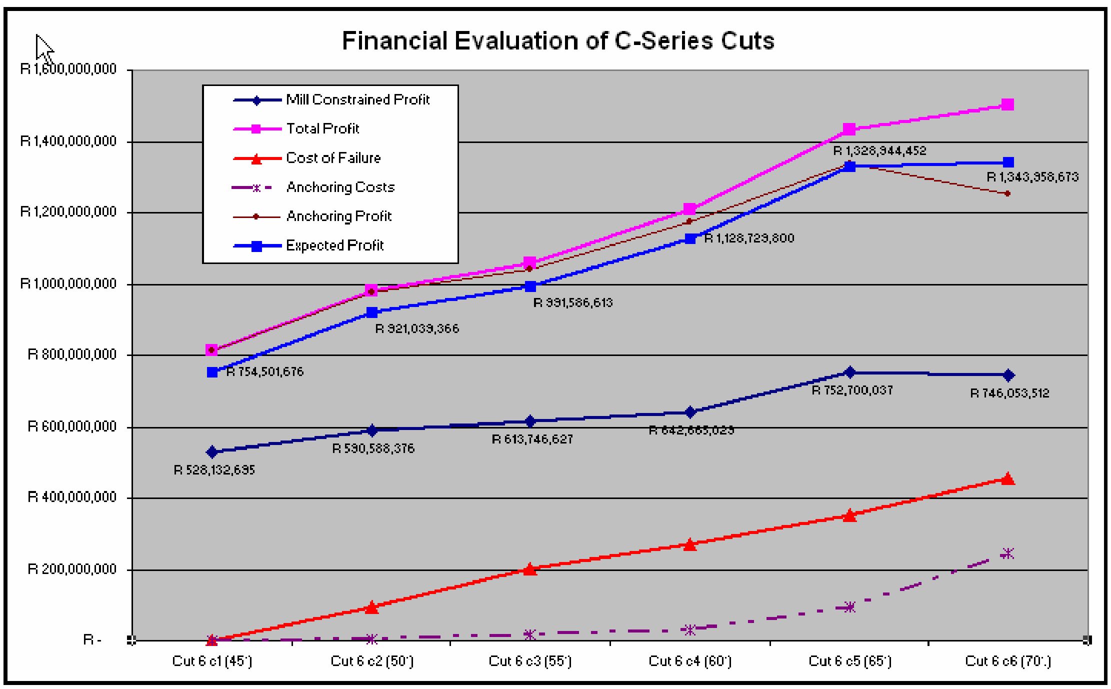


Figure 10.12 Financial evaluation of Sandsloot west wall Cut 6 options

The last step was to compare the economic analysis against the fault tree results. The fault tree analysis allows us to calculate a probability of fatality from a probability of failure by incorporating operational management. This is more meaningful to mine management and there is an acceptable risk level. The NPV results were plotted against probability of fatality in a risk versus reward chart (Figure 10.13). Combining all the different analyses, it became evident that the 65° stack would be the optimised design but that the radar would have to be in place while it was being excavated to prevent the risk to loss of life from exceeding the acceptable limits.

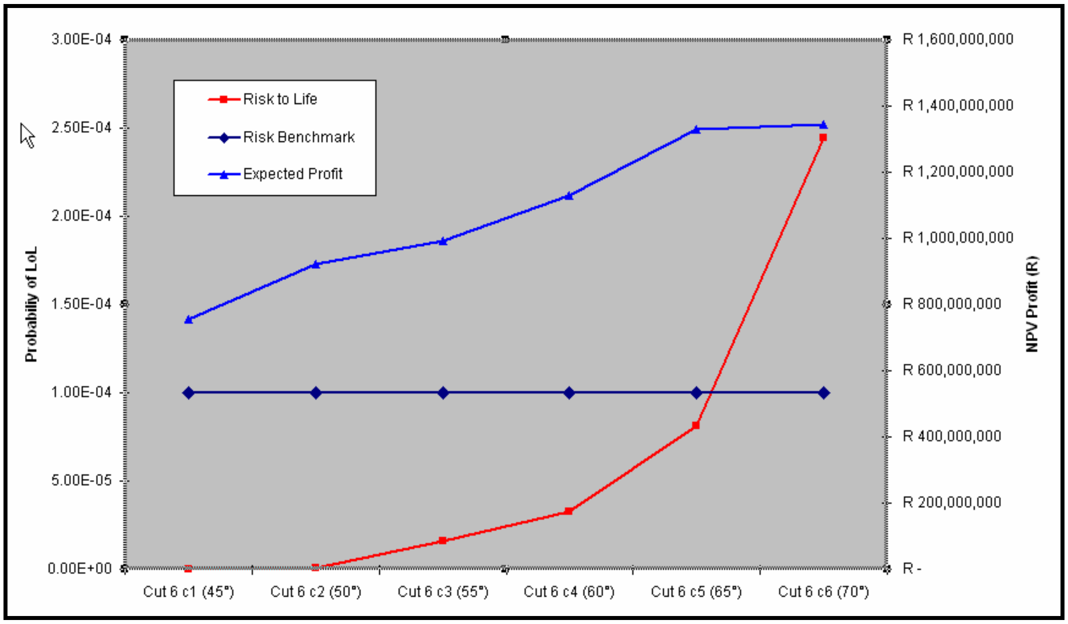


Figure 10.13 Risk versus economic reward for Sandsloot Cut 6 west wall design

The final slope design for Cut 4 and Cut 6 was a 65° stack over 60 m resulting in an overall slope of 52° (Figure 10.14). This slope is currently being mined under 24-hour radar surveillance. It is performing well and proving that steep slopes can be mined safely in difficult geotechnical conditions. The slope management plays a crucial role in achieving this steep slope and the total cost is roughly R7 million. This is far outweighed by the profits gained by the reduction in waste stripping and additional ore recovered. Slope optimisation at Sandsloot has added huge value to PPRust and has laid the foundation for future optimisation on other open pits.

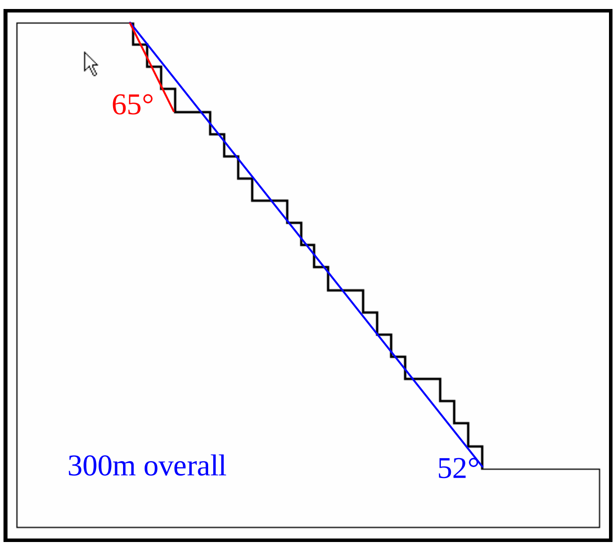


Figure 10.14 Optimised slope design for Sandsloot west wall Cut 6

10.3.2 Zwartfontein South Cut 5 optimisation

Slope optimisation for the Zwartfontein South pit began in 2006 for Cut 4, the penultimate cutback. Twelve geotechnical zones were defined in the pit with their respective failure mechanisms. Slope monitoring currently consists of GeoMoS prism monitoring on all slopes and Riegl laser monitoring on the west wall. Fault tree were setup and different probabilities were assigned based on the whether the radar was used or not. Cost-benefit studies were done and limit equilibrium slope analysis was done with Slide. This is still underway and should be implemented in 2007. Initial work estimates that over R300 million profit can be added to the bottom line.

10.4 Conclusions

Every open pit has some degree of geotechnical risk associated with it. It is important to be able to quantify that risk and take steps to manage it. There are a number of ways of measuring risk but the event and fault tree methodology has been employed at PPRust. This allows a probability of loss of life to be calculated from a probability of failure and operational management tools. Good slope monitoring, evacuation and awareness all play a role in reducing the risk. Slopes can be steepened if the safety risk is reduced resulting in increased profits. Slope optimisation has been performed at PPRust a number of times with the 2005 analysis producing additional revenue of at least R139 million for the remaining three years that Sandsloot open pit will operate. This far outweighs the cost of slope management and redesign that totals ~R7 million. In the case of Sandsloot, the rewards are worth the risk.

11 CONCLUSIONS AND RECOMMENDATIONS

“I haven't a clue as to how my story will end. But that's all right. When you set out on a journey and night covers the road, you don't conclude that the road has vanished. And how else could we discover the stars?”

- *Anonymous*

Geotechnical engineering has an integral role to play throughout the life of an open pit mining operation. Overall slope angles play a large role in mine economics at PPRust where a change of a few degrees affects profitability by hundreds of millions of Rands. The economic benefits must be balanced with safety, however, and there is a threshold at which an increased slope angle results in an unacceptably high risk. This can result in slope failure and large cost to the operation. Slope optimisation using risk versus reward methodology has become an integral part of the design at PPRust. Geotechnical data can also be used for blast design optismisation to improve fragmentation, loading rates and mill throughput. This is done on a daily basis at PPRust and the results have produced large financial gains.

The geotechnical strategy at PPRust is threefold: 1) to design fully optimised slopes at all stages of the mining operation; 2) to manage those slopes effectively so that maximum profit is achieved while meeting Anglo American’s safety and risk requirements and 3) to utilise geotechnical information to optimise every blast design to achieve the target fragmentation for the processing plant. There are numerous tactics employed to achieve this which have been developed and implemented over the last few years at PPRust.

Geotechnical field data is collected by logging all exploration core, point load testing the same core, in-pit mapping and laboratory strength testing. Two databases have been set up in the last three years to handle the large amount of data collected. A logging database was created in SABLE, a mapping database was created in MineMapper3D, and rock testing data was incorporated in both of these databases. This has improved the quality of the data and increases the confidence of the subsequent analysis. Rock mass ratings are calculated from the data in the software programs and this data is used for geotechnical zoning, slope design and blast design. Orientated logging was done to identify fault zones and joint sets in the pits. SiroVision digital photogrammetry has been implemented to improve mapping quality and quantity and to predict where stack failures could occur.

Geotechnical block models have been created for all pits and future pits in Datamine a similar way to ore reserve models. They interpolate a UCS, RQD, FF, IRMR and PLI and calculate MRMR, slope angle, blastability index , energy factor and cost. The energy factor is used by the drill and blast department to customise every blast design in order to account for rock mass conditions and thus attain the fragmentation targets set by the customers, Load and Haul and the Process Plant. The results show that the model adds value to the mine to mill process by dramatically improving loading efficiencies and milling rates. Additional revenue of over R2 million a month is realised in the plant alone. It is likely that a 3D geotechnical block model will become the norm in the mining industry in a few years time as a primary mineral resource management tool.

The main slope stability concern is stepped path stack failure on the west wall of Sandsloot pit. A number of brittle failures varying in size from 50 t to 30 000 t have occurred in the last four years. Progressive blasting plays a large role in the failure mechanism and analysis has shown that damage occurs 20 m behind the face. As the failures cannot be designed out due to economic reasons, a comprehensive risk management strategy is in place at PPRust.

Slope designs have been modified for Sandsloot over the years to maximise the NPV or to improve the safety in the pit. Empirical methods, limit equilibrium and numerical modelling techniques have all been used for overall slope analyses. The latest designs are based on Slide and FlacSlope models. Rockfall analysis is also done to determine bench heights and berm widths. The designs go through a rigorous approval process which is communicated and recorded via a custom-made software tool, ATS, or Authorisation Tracking System.

Slope management at PPRust includes a comprehensive limit blasting programme that has improved the stability of the highwalls. The introduction of EDDs has enabled better control of blast and therefore reduced blast damage. Standard visual inspections for the geotechnical engineers have been implemented and the data is stored in a database and communicated to pit personnel. Monthly inspections are used to create hazard plans while presplit inspections are used to identify and remediate poorly blasted pit slopes. Pit foremen are also required to complete slope stability inspection sheets and to take responsibility for keeping their working area safe. Slope support is minimal at PPRust though gabion walls, shotcrete, bolts and wire meshing have all been utilised to stabilise the pit slopes in Sandsloot. Dewatering has been ineffective due to the structural controlled flow of groundwater. Sumps and gutters are dug to reduce the effect of water on the operations and slope stability.

A state-of-the-art slope stability monitoring programme has been implemented at PPRust. The prism monitoring was automated in 2003 using GeoMoS to analyse the data and produce alarms. Due to the brittle failures on the west wall in Sandsloot many prisms were lost and many areas could not be accessed therefore the Riegl laser system was installed in 2005 to measure ‘virtual prisms’ on that area. Due to the rapidity of the failures, it was determined that the GroundProbe slope stability radar (SRR) was the only monitoring device that could provide early warning of failure on the west wall. Eight failures have been recorded with the SSR since November 2003 and showed that the pit personnel have less than two hours warning before failure occurs. The ISSI microseismic monitoring trial was not successful and was terminated in June 2005 as no significant movement was seen. Piezometers were installed in January 2005 to better monitor the groundwater and its influence on slope stability. Crackmeters are installed over tension cracks when they appear. This data is stored in the SSMON database and visualised in AutoCAD.

Full risk-reward slope optimisation studies have be done at PPRust that took into account all the field data collected, the operational controls in place, the cost of failure, full economic analysis of various slope angles and fault tree analysis. This allowed the geotechnical department to describe geotechnical hazards as geotechnical risks to management as well as add ~R130 million to the profitability of Sandsloot pit for the remaining three years of life of the pit. The steeper designs are conditional on radar monitoring the west wall at all times. It is estimated that savings on waste stripping at Zwartfontein South in the order of hundreds of millions of Rands can be realised.

The PPRust geotechnical department is now in a stage where the tools and systems are in place to ensure data collection, analysis and optimal usage. The challenge is to maintain the high level of work as well as to take it further. With the huge amount of geotechnical data having been and still being collected at PPRust, much geostatistical work can be done. Comparisons between rock mass rating systems and investigation of wider use of the systems, especially GSI and Q, can be done. Comparison of the different monitoring tools, radar, prisms, laser and seismic would aid in the application of the tools as well as the understanding of the change in the rock mass over time. There is much data at PPRust that has not been analyzed and this would be of value not only to the mine but to the geotechnical community at large. The block models are not being used to their full potential and can be further developed for the daily running of the process plant as well as for NPV calculations by the long term planners. Integration of the various software packages is being investigated by SiroVision, GroundProbe, Riegl and Datamine. This would add value by saving time and enabling better analysis of the data. As technology advances one can expect improved computer software for modeling and data analysis as well as monitoring equipment and processing tools. The work done in the last four years has resulted in PPRust becoming the benchmark for open pit geotechnics in Anglo American. The challenge is now to stay there.

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